SECTION 13: GEOTECHNICAL ENGINEERING REPORT

The following report dated May 2008 documents and summarizes the geotechnical exploration, testing, and geotechnical engineering recommendations that have been completed for design of the East End Bridge. Since completion of this report there have been revisions made to the bridge design. Therefore, parts of this report may no longer be applicable or have not been updated for the current design.

Also, attached to the May 2008 report is the results of supplemental geotechnical work that was completed in March 2011 for the Indiana abutment.

Geotechnical engineering recommendations for all substructure foundations will need to be updated and finalized during final design.

THE OHIO RIVER BRIDGES

SECTION 5 - EAST END BRIDGE OVER OHIO RIVER

KYTC ITEM NO. 5-745.00

GEOTECHNICAL ENGINEERING REPORT

MAY 12, 2008

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1.0 INTRODUCTION

This geotechnical evaluation was authorized by the Kentucky Transportation Cabinet (KYTC) and the Indiana Department of Transportation (INDOT) through the Bi-State Management Team (BSMT) as part of the design services for the proposed I-265 East End Bridge over the Ohio River near Louisville, Kentucky.

PB Americas, Inc. (PB) is the prime consultant for the Louisville-Southern Indiana Ohio River Bridges Project, as part of the Phase 3B – Preliminary Bridge Design for the East End Bridge. This report has been jointly prepared by PB and geotechnical subconsultant Fuller, Mossbarger, Scott, and May Engineers Inc. (FMSM).

1.1 Project Description

The proposed East End Bridge is a cable-stayed structure that will link the Gene Snyder Freeway in Kentucky (KY 841) with the Lee Hamilton Highway in Indiana (IN 265). This bridge will carry six lanes of traffic over the Ohio River at Mile Point 596, approximately 11 miles upstream of the McAlpine Lock and Dam. The general location of the site is shown in Figure 1, Site Vicinity Map.

The East End Bridge layout is shown on Figure 2, Boring Location Plan. The overall length of the bridge is 2,510 feet, with a 1,235-foot long center span. The bridge will be a cable-stayed structure, with two tower piers (Piers 3 and 4) in the river and two anchor piers (Piers 2 and 5) near the banks of the river. On the Kentucky side, a transition pier (Pier 1) is included in the scope of work. The bridge abutment on the Indiana side is skew to the bridge centerline, paralleling the slope of the bank.

This report presents the subsurface data and geotechnical design recommendations for the bridge foundations and Indiana abutment. The report also addresses stability of the rock slope in the vicinity of the Indiana abutment.

Elevations in this report are referenced to the project datum, NAVD88.

1.2 Scope of Services

The scope of services for this geotechnical evaluation includes preparation of a boring plan, performance of borings on land and water with soil sampling and rock coring, geologic mapping of rock outcrops in the vicinity of the Indiana abutment, field testing of seismic velocities by P-S logging techniques, laboratory testing of soil and rock, foundation type evaluations, geotechnical analyses of land and river piers and the Indiana abutment for bearing, uplift, and lateral loads, and preparation of this report. This report includes discussions of geology, results of drilling, seismic testing, and laboratory testing, results of engineering analyses, discussion of constructability issues, and recommendations for bridge foundations. The engineering analyses include drilled shaft analyses for the bridge piers, and shallow foundation and stability analyses for the Indiana abutment.

This report has been jointly prepared by PB and geotechnical subconsultant Fuller Mossbarger Scott & May Engineers, Inc. (FMSM). Subsurface explorations, laboratory testing, preparation of records and summaries of data, and evaluation of the geology and subsurface conditions has been performed by FMSM as a subconsultant to PB. Geotechnical evaluation, analyses and recommendations for foundations and slopes, have been performed by PB, with the assistance of FMSM.

2.0 GENERAL PHYSIOGRAPHIC FEATURES

2.1 Physiography of Kentucky

The East End Bridge project is located in the northwestern portion of Central Kentucky within the Outer Bluegrass Physiographic Region. This region is characterized by gently rolling lowland due to the outcrop of Ordovician and Silurian carbonates and shales that are situated on the crest and flanks of the Cincinnati Arch. These erosion resistant rocks, combined with the structural features of the Cincinnati Arch and the Ohio River Floodplain result in a region that exhibits low to moderate topographic relief. However, along the Ohio River Valley, steep ravines and bluffs descend from the bluegrass plains to the river terraces. Overburden soils within this portion of the Outer Bluegrass Physiographic Region generally consist of loess underlain by residual clay soils. An exception to this is the Ohio River Floodplain, which consists of lacustrine and outwash deposits. Surface drainage patterns observed in the area of the bridge are typically dendritic and flow toward the Ohio River.

2.2 Physiography of Indiana

The Indiana portion of the project site is located in the southeastern portion of Indiana, within the Muscatatuck Regional Slope, Indiana's equivalent of the Outer Bluegrass Physiographic Region of Kentucky. The Muscatatuck Regional Slope is characterized by a gently sloping plain that has been dissected by streams flowing to the Ohio River. The Muscatatuck group consists of westward dipping carbonates, along with westward dipping shale that underlie the region. These rocks are of Devonian, Silurian, and Ordovician age and are exposed on the westward side of the Cincinnati Arch. In the immediate vicinity of the bridge site the topography presents a steep slope transitioning from an upland area down to the Ohio River. Soils on this slope are typically shallow with occasional bedrock outcrops visible.

2.3 Ohio River

At the location of the East End Bridge the Ohio River is approximately 1,900 feet in width at normal pool. The pool elevation is controlled by McAlpine Locks and Dam which is located approximately 10.8 miles downstream of this site. The pool is typically maintained at a normal elevation of 420 feet above mean sea level. The 100-year flood elevation of the river is 452.8 feet. During the drilling programs for the East End Bridge, the water surface varied from elevation 418 feet to 420 feet. The maximum depth of the river encountered during the drilling was 44 feet at the location of the Indiana Tower Pier. This corresponds to a river bottom elevation which varied from 420 feet to 374 feet.

3.0 GEOLOGY

Available geologic mapping (Geologic Map of Parts of the Jeffersonville, New Albany and Charlestown quadrangles, Kentucky-Indiana. Kentucky Geologic Survey, 1974 and Geologic Map of the Anchorage Quadrangle, Jefferson and Oldham Counties, Kentucky, 1971) was used to identify and characterize the bedrock at the bridge site. In addition, a geologic map, availability of ground water, and columnar section and water-bearing character of the rocks in Bullitt, Jefferson, and Oldham Counties, Kentucky are presented as Figures 3a through 3c in the report.

3.1 Regional Geology

Regionally, the East End Bridge site is located within the boundaries of the Cincinnati Arch. The Cincinnati Arch is described as a prominent elongated north-trending regional uplift (anticlinal fold) that extends from the Nashville Dome in central Tennessee to northwestern Ohio. Essentially the Cincinnati Arch separates the Appalachian Basin from the Illinois and Michigan basins. Structural features of the arch, related to the East End Bridge, are the Springdale Anticline and the Lyndon Syncline. No geologic faults are shown in the immediate vicinity of the bridge.

3.2 Local Geology

3.2.1 Kentucky Geology

The backstation portion of the bridge (Piers 1 and 2) crosses alluvium, primarily lacustrine and outwash deposits, associated with the Ohio River Floodplain. Consisting of intermixed clay, silt, sand, and gravel, these deposits can be in excess of 100 feet deep and were deposited by glacial activity during the Pleistocene Epoch of geologic time. More specifically, deposition occurred during Illinoisan and Wisconsinan glaciations. Bedrock underlying the Ohio River Floodplain reportedly consists of the Drakes Formation. The Drakes Formation is generally described as limestone that is olive gray to grayish green, very fine grained, grades to dolomitic and becomes interbedded with medium to dark gray shale as depth increases. The limestones of this Ordovician bedrock are susceptible to chemical weathering and development of solution features. Typically these features occur along bedding planes, joints and fractures, and can be evidenced by clay filled seams in the unit.

3.2.2 Ohio River Geology

Overburden soils consisting of sand and gravel, deposited as both glacial outwash and fluvial deposits, range in thickness from roughly 90 feet at the Kentucky shoreline to less than 11 feet at the Indiana shore. Bedrock strata located beneath the Ohio River consist of the Osgood Formation, Brassfield Formation, and the Drakes Formation. The Osgood formation is generally described as limestone interbedded with shale, both of which are dolomitic. The Brassfield Formation is described as a crystalline grained limestone with dolomitic and glauconitic zones. The Drakes Formation is identified as limestone that grades to dolomitic limestone and becomes interbedded with shale as depth increases.

3.2.3 Indiana Geology

Available geologic mapping (Geologic Map of the 1^o X 2^o Louisville Quadrangle, Indiana, Showing Bedrock and Unconsolidated Deposits, Indiana Geologic Survey, 1972) indicates the area in the vicinity of the East End Bridge is underlain by Quaternary sediments and soils, as well as Devonian, Silurian, and Ordovician age bedrock. These sediments and soils consist of outwash deposits from the Pleistocene epoch of geologic time. Referred to as the Wheeling-Sciotovile-Otwood complex these sediments and soils are described as very deep, well drained and moderately well drained, nearly level to moderately steep, eroded, and are occasionally flooded for brief durations.

The ground surface elevation varies from over 490 feet at the Indiana Abutment location to approximately 420 feet at the location of Pier 5, resulting in the involvement of several geologic units. Rock units noted in the literature consist of, in order of descending lithology, the Sellersburg, Jeffersonville, and Louisville Limestones, Laurel Dolomite, and Osgood Formation. Where these formations are exposed in local roadcuts and in the quarry to the north of the bridge site, localized solutioning of the various limestones was noted. The solutioning is primarily noted in joints, fractures, and bedding planes, with clay noted as a common replacement material. This indicates the limestones underlying the bridge site are also susceptible to solutioning.

The Sellersburg Limestone is subdivided into two members. The upper Beechwood Member is described as a limestone that sits unconformably on the Silver Creek Limestone, the lower member of the Sellersburg Limestone. The unconformity is marked by a dark gray shale seam containing phosphatic pebbles and quartz sand. The Silver Creek Member is a limestone that is argillaceous, dolomitic, and fossiliferous. Below the Silver Creek Members is the Jeffersonville Limestone.

The Jeffersonville Limestone is medium to coarse grained, thin to thick bedded, fossiliferous, and rests unconformably on the Louisville Limestone. The unconformity is marked by a sharp transition from the coarse grained Jeffersonville Limestone above to the fine grained Louisville Limestone below. This Louisville Limestone is generally described as a fossiliferous, dolomitic, massive limestone that rests on top of the Waldron Shale. The Waldron Shale is a silty, dolomitic, and pyritic clay shale that, when exposed, often undercuts the above-lying Louisville Limestone. Beneath the Waldron Shale the mapping identifies the Laurel Dolomite. This unit is subdivided into two sections of which both consist of dolomitic limestone and are separated by a clay shale layer that is up to 2.5 feet in thickness. Beneath the Laurel Dolomite is the Osgood Formation. This formation consists of limestone interbedded with shale, both of which are dolomitic.

3.3 Regional Seismicity

Seismicity within the region surrounding the bridge site varies widely depending on location. The western portions of the states of Kentucky and Indiana are dominated by the New Madrid and Wabash Valley seismic source zones. In general, these zones are fairly active with many documented historical seismic events. A series of four earthquakes, part of the New Madrid Earthquakes of 1811 and 1812, in southeast Missouri and northeast Arkansas, reportedly caused the Mississippi River to flow backwards and were of sufficient intensity to topple chimneys in Louisville (Kentucky Transportation Research Report KTC-96-4). A major earthquake centered in Charleston, South Carolina in 1886 was also strongly felt in Kentucky. More recently, an earthquake centered in Sharpsville, Kentucky in 1980 was felt throughout the area along the Ohio River.

The East End Bridge will be located in the north-central region of Kentucky and the south-central region of Indiana. Both the Kentucky and Indiana portions of the bridge will likely experience less frequent earthquakes because the source zones are quite distant from this area. The nearest of these, the Wabash Valley source zone, is on the order of 100 miles west-northwest of the project site and occupies portions of southwest Indiana, southeastern Illinois and northwestern Kentucky.

According to the recent earthquake history and studies in Central United States, the Wabash Valley zone may be able to trigger an earthquake as large as magnitude 7 (http://www.cusec.org/S_zones/Wabash/index.htm). In the Wabash Valley zone, a recent earthquake occurred April 18, 2009, was recorded as a magnitude of 5.2. After the main shock, there were 6 aftershocks on the same date with magnitudes of up to 4.6. The epicenter was 6 miles south of West Salem, Illinois, which is about 200 miles from the bridge site.

4.0 FIELD RECONNAISSANCE

The proposed East End bridge site encompasses both the Indiana and the Kentucky sides of the Ohio River. Pier 1 and Pier 2 fall on the Kentucky bank and at the river's edge respectively, while Piers 3 and 4 are the main tower piers within the river. Pier 5 is located on the Indiana bank at the river's edge, and the Indiana Abutment is located near the top of a steep wooded slope. On the Kentucky side the bridge is located within the floodplain of the Ohio River. Within the bridge limits a maximum topographic relief of approximately 15 feet can be measured from the location of Boring AC-1 to the normal pool elevation of 420 feet. The land use at the location of Piers 1 and 2 is residential, with vegetation consisting of moderate tree cover, shrubs, and grass covered lawns.

The Indiana side of the bridge is located on a relatively steep slope of the Ohio River. A maximum topographic relief of approximately 80 feet is achieved by comparing the elevation of the Ohio River's normal pool with the surface elevation of boring AC-26 at the Indiana Abutment (elevation 498 feet). At an elevation of approximately 436 feet, existing River Road traverses the bridge site. The current land use is residential and land cover consists of heavily forested land. Physical features noted on the site indicate the area of the Indiana Abutment was previously the location of a small limestone quarry. This quarry activity left behind short vertical faces of limestone to the north of the abutment, as well as occasional mounds of spoil rock and soil.

Because of the steep slope of the Indiana bank, the relatively shallow soils inferred by the numerous rock outcrops, and the orientation of Pier 5 and the Indiana Abutment, geologic outcrop mapping of the Indiana slope was performed by FMSM personnel. After reviewing topographic and geologic literature while at the site, a total of eight locations were identified for performance of geologic outcrop mapping.

Approximately 1000 feet north of the Indiana Abutment location, a closed limestone quarry of significant size is currently being developed for residential home sites. Permission was obtained to view and map portions of the quarry entrance and wall for correlation with data obtained at the bridge site.

4.1 Surface Conditions

At the Indiana Abutment location the site was heavily wooded with bedrock and boulders, remaining from quarrying operations, visible on the ground surface. The ground surface falls steeply from the abutment to the location of River Road where bedrock is exposed in a rock cut for the existing roadway. Soils in this area appear to be relatively thin above the elevation of River Road and could be described as a combination of colluvial and residual in origin.

4.2 Geologic Mapping of Rock Exposures

Geologic mapping of the eight outcrops was performed in accordance with "Rock Slopes Reference Manual", Publication No. FHWA HI-99-007. The geologic outcrop mapping depicting the data collected is presented in Appendix C.

Equipment used in the mapping process included a Brunton Compass (used to obtain strike and dip of discontinuities), altimeter (source of elevation), GPS unit (coordinate source), 300 feet tape measure, 12 feet tape measure, digital camera, rock hammer, and pocket penetrometer (estimates soil strength). In order to calibrate the Brunton Compass the magnetic declination of the area must first be determined. The magnetic declination of the project site, as obtained from the USGS Jeffersonville Quadrangle, is 2°NW of True North. Once the magnetic declination was known the Brunton Compass was calibrated accordingly at the Base Station.

A total of eight outcrops were selected and assigned titles and outcrop numbers. Those outcrops were mapped over a two day time period ending on October 4, 2007.

The elevations included in the observation summaries refer to the base of the mapped outcrop. Observations made at each outcrop are summarized below:

<u>Site # 1- Base Station</u>

Site # 1 would serve as the Base Outcrop, a source of known elevation used to calibrate the altimeter. The Base Outcrop was visited at the beginning of each day to calibrate the altimeter and conduct the daily safety meetings. The location of site # 1 is N 38° 20' 42.2" W 85° 38' 46.5" with a base elevation of 495 feet. This is also the location of Boring AC-25.

<u>Outcrop # 1 - West Old Quarry Wall</u>

The location of Outcrop # 1 is N 38° 20' 43.6" W 85° 38' 48.4" with a base elevation of 479 feet. The rock unit exposed at this outcrop is the Jeffersonville Limestone of Silurian age. A description of the Jeffersonville Limestone exposed at this outcrop is a coarsely crystalline grained, brownish gray, medium strong, slightly to moderately weathered limestone that weathers to a light gray and buff (brownish gray) color with a blocky structure. Bedding planes were noted to be horizontal. Three discontinuity sets were observed with strikes ranging from N 12° E to N 80° W with dips ranging from 83° S to 88° S. These discontinuities are further described in the following paragraphs.

Joint Set #1 exhibited a strike of N 12° E with a dip of 88° South. This discontinuity set exhibited apertures ranging from 0.1 – 1.1 feet, with a rough surface, low persistence, and undulating surface shape. Evidence of water flow was noted from the discontinuity set being filled with clay which exhibited a strength of 5 tons per square foot.

Joint Set #2 exhibited a strike of N 80° W with a dip of 83° South. This discontinuity set exhibited apertures ranging from 0.1 – 0.9 foot, with a rough to smooth surface, medium persistence, and undulating surface shape. Evidence of water flow was from clay, exhibiting a strength of 5 tons per square foot, filling the discontinuity set.

Joint Set #3 exhibited a strike of N 60° W with a corresponding dip of 85° South. This discontinuity set exhibited apertures ranging from 0.1 – 1.1 feet, with a rough to smooth surface, low persistence, and stepped to undulating surface shape. Evidence of water flow included clay, exhibiting a strength of 3.1 tons per square foot, filling the discontinuity set.

Outcrop # 2 – Solution Feature Ridge

The location of Outcrop # 2 is N 38° 20’ 44.7” W 85° 38’ 44.2” with a base elevation of 493 feet. The rock unit exposed at this outcrop is the Sellersburg Limestone of Devonian age. A description of the Sellersburg Limestone exposed at this outcrop is a coarsely crystalline grained, brownish gray, medium strong, slightly to moderately weathered dolomitic limestone that weathers to a light gray and buff (brownish gray) color with a blocky structure. The bedding planes were noted to be horizontal. One major discontinuity set was observed with a strike of N 23° E and a dip of 86° S, and presented an aperture range of 1.5 – 6.0 feet widening toward the base. The discontinuity showed evidence of water flow from clay deposits being present which exhibited a strength of 1 ton per square foot.

Outcrop# 3 – Knobs

The location of Outcrop # 3 is N 38° 20’ 39.8” W 85° 38’ 47.7” with a base elevation of 500 feet. Rock units exposed at this outcrop consist of both the Sellersburg Limestone and the Jeffersonville Limestone. The Sellersburg Limestone is described as a coarsely crystalline grained, brownish gray, medium strong, slightly to moderately weathered dolomitic limestone that weathers to a light gray and buff (brownish gray) color with a columnar structure. The Jeffersonville Limestone is described as a coarsely crystalline grained, brownish gray hard limestone (dolomite) that weathers to a light gray and buff (brownish gray) color. One major discontinuity set consisting of two major joints was observed. Joint strikes range from N 2°E to N 13° E with a dip range from 82° S to 87° S. These discontinuities are further described in the following paragraphs.

North Knob Joint Set #1 exhibited a strike of N 2° E with a corresponding dip of 82° S. This discontinuity set exhibited apertures ranging from 0.2 – 6.2 feet, with a rough surface, low to medium persistence, and stepped to undulating surface roughness. Noted evidence of water flow includes clay deposits which exhibited a strength of 0.3 tons per square foot.

South Knob Joint Set #2 exhibited a strike of N 4° E with a corresponding dip of 86° N. This discontinuity set exhibited apertures ranging from 0.2 – 4.7 feet, with a rough surface, low to medium persistence, and undulating surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 4.5 tons per square foot. The joints were also lined with calcite and limestone travertine providing evidence of previous water transport. Bedding planes for both the Sellersburg and Jeffersonville Limestones were noted to be horizontal.

Outcrop # 4 – East Quarry Bluff Cut

The location of Outcrop #4 is N 38°20’ 56.0” W 85° 38’ 39.9” with a base elevation of 453 feet. Rock units exposed at this outcrop are the Louisville Limestone underlain by the Waldron Shale. The Louisville Limestone is described as a coarsely crystalline grained, brownish gray hard limestone that weathers to a light gray and buff (brownish gray) color. The Waldron Shale is described as clay shale that is dark greenish gray in color that weathers to a light gray, silty, and contains dolomitic zones. The Waldron Shale has been noted to undercut the above lying Louisville Limestone during the weathering process. One major discontinuity set was observed consisting of two joints, however one joint was inaccessible for strike and dip measurements. The other joint exhibited a strike of N 38° W and a dip of 78° S. Bedding planes observed were noted to be horizontal.

Outcrop # 5 – Road Cut to Quarry Bluff Estates

The location of Outcrop # 5 is N 38° 20’ 59.5” W 85° 38’ 41.8” with a base elevation of 452 feet. The rock unit exposed at Outcrop # 5 is the Louisville Limestone. The Louisville Limestone is a coarsely crystalline grained, brownish gray hard limestone that weathers to a light gray and buff (brownish gray) color and is horizontally bedded. Two major discontinuity sets were observed with a total of six joints with a strike range of N 0° to N 27° E with a dip range of 73° S to 86° N. Iron staining was also noted at an elevation of 464.8 feet.

Outcrop# 6 – North Old Quarry Wall

The location of Outcrop # 6 is N 38° 20’ 44.9” W 85° 38’ 47.5” with a base elevation of 493 feet. The rock unit exposed at this station is the Sellersburg Limestone. The Sellersburg Limestone is described as a horizontally bedded coarsely crystalline grained, brownish gray, medium strong, slightly to moderately weathered dolomitic limestone that weathers to a light gray and buff (brownish gray) in color with a columnar structure. No discontinuity feature was observed at this site; however some solution features were observed, such as pitting on the rock face as well as circular depressions indicative of solutioning.

Outcrop # 7 – Solution Feature Outflow

The location of Outcrop # 7 is N 38° 20’ 44.5” W 85° 38’ 43.5” with a base elevation of 437 feet. Rock units exposed at this site are the Louisville Limestone underlain by the Waldron Shale. The Louisville Limestone is described as a coarsely crystalline grained, brownish gray hard limestone that weathers to a light gray and buff (brownish gray) color and the Waldron Shale is described as clay shale that is dark greenish gray in color that weathers to a light gray, silty, and contains dolomitic zones. One major discontinuity set was observed containing three joints with a strike range of N 83° W to N 78° W with a dip range of 85° S to 87° S. The bedding planes of both units are horizontal. Colluvial slopes were also noted at the base of the discontinuities with an overburden thickness of up to two feet. Solution feature outflow refers to a series of three joint sets that correspond to the solution features that lie directly above the aforementioned joint sets. These features are described in detail below:

Joint Set #7 exhibited a strike of N 87°W with a corresponding dip of 87° South. This discontinuity set exhibited apertures ranging from 0.3 – 3.4 feet, with a rough surface, low persistence, and a stepped surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 3.1 tons per square foot.

Joint Set #2 exhibited a strike of N 83° W with a corresponding dip of 85° South. This discontinuity set exhibited apertures ranging from 0.3 – 3.4 feet, with a smooth surface, low persistence, and a smooth surface. No evidence of water flow was observed at this joint set.

Joint Set #3 exhibited a strike of N 78° W with a corresponding dip of 86° South. This discontinuity set exhibited apertures ranging from 0.3 – 3.4 feet, with a rough to smooth surface, low persistence, and undulating surface shape. Evidence of water flow includes clay deposits which exhibited a strength of 0.6 ton per square foot.

Outcrop # 8 – Upper River Road Cut

The location of Outcrop # 8 is N 38° 20’ 40.6” W 85° 38’ 43.8” with a base elevation of 436 feet. Rock strata observed at this site is the Louisville Limestone. The Louisville Limestone is described as a horizontally bedded coarsely crystalline grained, brownish gray hard limestone that weathers to a light gray and buff (brownish gray) color. Four major discontinuity sets were observed containing a total of twelve joints. The strike range for these discontinuities is N 21° W to N 88° W with a corresponding dip range of 79° S to 88° N. These discontinuities are further described in the following paragraphs.

Joint Set #1 exhibited a strike of N 49° W with a corresponding dip of 81° South. This discontinuity exhibited an aperture ranging from 1.4 – 2.7 feet with a rough to smooth surface roughness, medium persistence, and a stepped to undulating surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 3.1 tons per square foot.

Joint Set #2 exhibited a strike of N 23° W with a corresponding dip of 88° North. This discontinuity exhibited an aperture ranging from 0.7 – 1.9 feet with a rough surface, medium persistence, and an undulating surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 4.6 tons per square foot.

Joint Set #3 exhibited a strike of N 21° W with a corresponding dip of 88° South. This discontinuity exhibited an aperture ranging from 0.4 – 2.6 feet with a rough surface, medium persistence, and an undulating surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 2.7 tons per square foot.

Joint Set #4 exhibited a strike of N 74° W with a corresponding dip of 85° South. This discontinuity exhibited an aperture ranging from 0.1 – 0.4 foot with a rough surface, medium persistence, and a planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 5.0 tons per square foot.

Joint Set #5 exhibited a strike of N 88° W with a corresponding dip of 86° South. This discontinuity exhibited an aperture ranging from 0.1 – 3.3 feet with a rough surface, medium persistence, and an undulating surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 5.0 tons per square foot.

Joint Set #6 exhibited a strike of N 82° W with a corresponding dip of 87° South. This discontinuity exhibited an aperture ranging from 0.1 – 0.3 feet with a rough surface, medium persistence, and a planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 5.0 tons per square foot.

Joint Set #7 exhibited a strike of N 86° W with a corresponding dip of 76° North. This discontinuity exhibited an aperture ranging from 0.1 – 1.4 feet with a rough surface, medium persistence, and an undulating to planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 5.0 tons per square foot.

Joint Set #8 exhibited a strike of N 79° W with a corresponding dip of 79° North. This discontinuity exhibited an aperture ranging from 0.1 – 1.1 feet with a rough surface, medium persistence, and an undulating to planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 5.0 tons per square foot.

Joint Set #9 exhibited a strike of N 78° W with a corresponding dip of 83° North. This discontinuity exhibited an aperture ranging from 0.7 – 1.8 feet with a smooth surface, medium persistence, and an undulating to planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 3.3 tons per square foot.

Joint Set #10 exhibited a strike of N 27° W with a corresponding dip of 88° North. This discontinuity exhibited an aperture greater than 3.0 feet with a smooth surface, medium persistence, and a planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 1.8 tons per square foot.

Joint Set #11 exhibited a strike of N 79° W with a corresponding dip of 79° South. This discontinuity exhibited an aperture ranging from 0.1 – 3.1 feet with a smooth surface, medium persistence, and a planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 0.1 ton per square foot.

Joint Set #12 exhibited a strike of N 48° W with a corresponding dip of 79° North. This discontinuity exhibited an aperture ranging from 0.1 – 3.3 feet with a smooth surface, medium persistence, and a planar surface shape. Noted evidence of water flow includes clay deposits which exhibited a strength of 3.6 tons per square foot.

5.0 SUBSURFACE INVESTIGATION PROGRAM

In October 2005, FMSM advanced four borings as a preliminary exploration for the bridge. Each boring was drilled from a floating plant at the preliminary locations of Piers 2 through 5 and designated Boring B-1 through B-4. From June 13 to October 10, 2007 a total of 28 additional borings (ten from a floating plant and eighteen from a land based drill) were performed at the locations of the piers, Indiana Abutment and retaining wall for the bridge. These 2007 borings are designated AC-1 through AC-28. Both investigations were performed in general accordance with the Kentucky Transportation Cabinet (KYTC) Geotechnical Manual in terms of drilling, sampling, and laboratory testing The locations of the borings are presented on both Figure 2 and on the geotechnical drawings in Appendix A.

5.1 Boring Program

5.1.1 General

FMSM performed traditional geotechnical drilling and sampling operations for the bridge substructure element locations using truck-mounted drill rigs for land work, and a truck-mounted drill rig positioned on a floating barge for borings advanced beneath the Ohio River. Drilling and sampling operations were performed using hollow-stem augers or casing advancement techniques from the ground surface to the top of bedrock. Drilling personnel collected samples of the soils from specific borings at approximate five-foot intervals. Soil sampling typically consisted of performing standard penetration tests (SPT) in non-cohesive soils and in cohesive soils having significant gravel contents. Cohesive soils were sampled with undisturbed thin-wall (Shelby) tubes.

Upon reaching bedrock, FMSM switched to NQ2 sized rock coring equipment to obtain a minimum of approximately 40 feet of rock core sample from each of the four preliminary borings (B-1 through B-4), and 50 feet of rock core at the planned locations of Piers 1 through 5. These rock cores provide identification of bedrock strata and samples for strength testing in support of foundation design. At the location of the Indiana Abutment, two borings were drilled 40 feet into the bedrock and one boring (AC-23) was advanced approximately 70 feet into bedrock. The purpose of the additional rock coring footage in Boring AC-23 was to provide continuous rock core data from the elevation of the abutment to the elevation of Pier 5. This information would be used to design a rock cut slope if River Road required relocation into the hillside.

In addition to traditional drilling operations, Boring AC-3 was drilled using PQ-sized coring tools in bedrock to allow geophysical testing of the soils and bedrock on the Kentucky side of the river. The PQ-sized rock core boring allowed installation of flush joint casing of sufficient size to pass the geophysical equipment from the ground surface to the bottom of the boring.

5.1.2 Summary of Borings

A total of 32 borings were drilled during the exploration for the East End Bridge over the Ohio River. The locations and graphical logs of these borings are shown on the Subsurface Data Sheets, respectively, in Appendix A. A summary of borings drilled by FMSM for this exploration is presented in Table 1. All borings were performed at the original planned locations of the substructure elements and retaining wall. Three sample and rock core borings were performed at each anchor/transition pier (Piers 1, 2, and 5). Five sample and rock core borings were drilled at each of the main tower locations (Piers 3 and 4) within the river. Three sample and rock core borings were performed at the location of the Indiana Abutment, and two for the Indiana Abutment retaining wall. Additionally, eight rock soundings were advanced to bedrock by a truck mounted drill equipped with solid stem continuous flight augers in front of and behind the planned location of the Indiana Abutment and abutment retaining wall to better identify bedrock surface elevations. The stations and offsets of the boring locations along with latitudes and longitudes are included in Appendix B. The results of the drilling program were used to develop a top of bedrock contour map at the bridge site. This map is presented as Figure 4 in the report.

During the drilling process in soils, attention was given to the description and consistency of the soils encountered. Soils were identified in terms of classification, color, grain size, consistency, and moisture content. The location or absence of the groundwater table was also noted on the logs by the geologist in the field.

Because of the size of this bridge and the loads to which it could be subjected, rock bearing foundations are anticipated for substructure support. Immediately following the drilling process the bedrock was described by a geologist in terms of classification, color, grain size, bedding characteristics, and other descriptions. Fractures, clay seams and other notable features were also recorded on the boring log. As an indication of general competency of the rock cored, the Rock Quality Designation (RQD) of each coring run was recorded. The Standard RQD is defined as the cumulative length of intact pieces longer than four inches divided by the length of the coring run and expressed as a percentage. Generally, the higher the RQD value the more competent the rock mass. In addition to the Standard RQD, a “KY” RQD was also recorded. The KY RQD is defined as cumulative length of pieces longer than four inches which cannot be broken by hand pressure divided by the length of the coring run and expressed as a percentage. Typically the KY RQD is a lower value than the Standard RQD. The RQD values, both Standard and KY, for the rock core borings drilled for the East End Bridge varied from a low of 0 to a high value of 100 with lower values typically recorded in the upper or weathered portions of the bedrock strata. A complete listing of the RQD values recorded for the borings is presented in Table 2.

Table 1. Summary of Boring Locations and Elevations

Sub-structure Element	Boring No.	Station	Offset	River Water Surface Elevation	Water Depth	Ground Surface Elevation	Top of Bedrock		Ground Water Table		Bottom of Hole	
							Depth	Elevation	Depth	Elevation	Depth	Elevation
Pier 1	AC-1	187+18.6	44.6' Lt.	---	---	434.1	100.3	333.8	20.6	413.5	150.5	283.6
	AC-2	187+28.4	13.5' Lt.	---	---	434.0	100.4	333.6	14.4	419.6	139.4	294.6
	AC-3	187+46.6	60.9' Rt.	---	---	433.7	98.8	334.9	12.0	421.7	150.6	283.1
Pier 2	AC-4	189+81.7	62.0' Lt.	419.8	15.7	404.1	91.7	328.1	---	---	143.9	275.9
	AC-5	189+46.1	63.7' Rt.	---	---	428.9	98.4	330.5	Dry	---	148.4	280.5
	B-1	189+60.0	CL	419.6	0.0	419.6	81.0	338.6	0.0	419.6	126.6	293.0
Pier 3	AC-6	193+51.9	CL	419.4	40.9	378.5	87.2	332.2	---	---	138.2	281.2
	AC-7	193+94.5	68.1' Lt.	419.5	40.5	379.0	89.1	330.4	---	---	139.1	280.4
	AC-8	193+95.1	1.2' Rt.	419.4	40.7	378.7	86.5	332.9	---	---	137.9	281.5
	AC-9	193+95.2	70.0' Rt.	419.4	40.6	378.8	86.2	333.2	---	---	140.8	278.6
	B-2	194+50.0	CL	420.2	42.0	378.2	87.3	332.9	---	---	116.0	304.2
Pier 4	AC-10	205+97.9	70.0' Lt.	418.3	44.0	374.3	84.2	334.1	---	---	135.5	282.8
	AC-11	205+93.8	0.7' Rt.	419.4	39.1	380.3	81.7	337.7	---	---	132.9	286.5
	AC-12	205+94.4	71.2' Rt.	418.4	40.5	377.9	81.7	336.7	---	---	131.7	286.7
	AC-13	206+53.0	1.6' Lt.	418.9	38.7	380.2	82.7	336.2	---	---	132.3	286.6
	B-3	205+50.0	CL	419.9	41.0	378.9	79.5	340.4	---	---	122.0	297.9
Pier 5	AC-14	210+56.2	72.4' Lt.	---	---	436.0	12.1	423.9	Dry	---	81.5	354.5
	AC-15	210+35.1	37.3' Rt.	419.4	4.2	415.2	27.0	392.4	---	---	77.3	342.1
	B-4	210+30.0	CL	420.2	1.0	419.2	11.0	409.2	---	---	56.2	364.0
Indiana Abutment	AC-18	212+20.0	25.0' Lt.	---	---	495.2	4.0 *	491.2 *	---	---	4.0	491.2
	AC-20	212+30.0	56.0' Lt.	---	---	494.7	3.1	491.6	Dry	---	44.0	450.7
	AC-21	212+42.0	37.0' Rt.	---	---	490.1	1.4 *	488.7 *	---	---	1.4	488.7
	AC-22	212+46.0	35.0' Lt.	---	---	493.9	2.8 *	491.1 *	---	---	2.8	491.1
	AC-23	212+50.0	CL	---	---	493.3	3.0	490.3	Dry	---	74.2	419.1
	AC-25	212+68.0	27.0' Rt.	---	---	497.3	6.9 *	490.4 *	---	---	6.9	490.4
	AC-26	212+70.0	55.0 Rt.'	---	---	498.5	7.4	491.1	Dry	---	47.5	451.0
Indiana Abutment Wing Wall	AC-16	212+17.0	139.4' Lt.	---	---	496.1	6.3 *	489.8 *	---	---	6.3	489.8
	AC-17	212+17.0	87.0' Lt.	---	---	492.0	1.6	490.4	Dry	---	12.0	480.0
	AC-19	212+26.0	86.0' Lt.	---	---	492.6	1.7 *	490.9 *	---	---	1.7	490.9
	AC-24	212+67.0	95.4' Rt.	---	---	492.9	2.6 *	490.3 *	---	---	2.6	490.3
	AC-27	212+87.0	125.0 ' Rt.	---	---	493.6	1.2	492.4	Dry	---	27.2	466.4
	AC-28	212+91.0	90 .0' Rt.	---	---	497.5	5.5 *	492.0 *	---	---	5.5	492.0

* Depths and elevations of auger refusal.

Table 2. Summary of Rock Core Data

Substructure Element	Boring No.	Depth Interval	Elevation	KY RQD (%)	Std. RQD (%)
Pier 1	AC-1	100.3 – 101.8	333.8 – 332.3	0	0
		101.8 – 103.3	332.3 – 330.8	0	0
		103.3 – 113.3	330.8 – 320.8	60	70
		113.3 – 123.3	320.8 – 310.8	45	57
		123.3 – 133.3	310.8 – 300.8	81	92
		133.3 – 143.3	300.8 – 290.8	70	84
		143.3 – 150.5	290.8 – 283.6	72	82
	AC-2	100.4 – 103.5	333.7 – 330.6	74	74
		103.5 – 105.4	330.6 – 328.7	55	74
		105.4 – 110.4	328.7 – 323.7	41	56
		110.4 – 120.4	323.7 – 313.7	37	82
		120.4 – 129.4	313.7 – 304.7	42	50
		129.4 – 139.4	304.7 – 294.7	51	66
	AC-3	99.5 – 103.6	334.2 – 330.1	93	93
		103.6 – 108.6	330.1 – 325.1	40	40
		108.6 – 113.6	325.1 – 320.1	86	86
		113.6 – 118.6	320.1 – 315.1	68	68
		118.6 – 123.6	315.1 – 310.1	94	94
		123.6 – 128.6	310.1 – 305.1	82	82
		128.6 – 133.6	305.1 – 300.1	100	100
		133.6 – 138.6	300.1 – 295.1	72	72
		138.6 – 143.6	295.0 – 290.1	86	86
		143.6 – 148.6	290.1 – 285.1	100	100
		148.6 – 150.6	285.1 – 283.1	85	85
Pier 2	AC-4	92.3 – 95.8	327.5 – 324.0	14	14
		95.8 – 105.8	324.0 – 314.0	22	38
		105.8 – 115.8	314.0 – 304.0	59	83
		115.8 – 125.8	304.0 – 294.0	48	51
		125.8 – 135.8	294.0 – 284.0	71	90
		135.8 – 143.9	284.0 – 275.9	89	99
	AC-5	98.4 – 103.4	330.5 – 325.5	8	88
		103.4 – 113.4	325.5 – 315.5	38	38
		113.4 – 123.4	315.5 – 305.5	40	56
		123.4 – 133.4	305.5 – 295.5	45	73
		133.4 – 143.4	295.5 – 285.5	80	84
		143.4 – 148.4	285.5 – 280.5	44	88
	B-1	85.0 – 89.0	334.6 – 330.6	10	15
		89.0 – 99.0	330.6 – 320.6	25	69
		99.0 – 109.0	320.6 – 310.6	37	50
		109.0 – 119.0	310.6 – 300.6	56	70
		119.0 – 126.6	300.6 – 293.0	71	76

Table 2. Summary of Rock Core Data

Substructure Element	Boring No.	Depth Interval	Elevation	KY RQD (%)	Std. RQD (%)
Pier 3	AC-6	87.7 – 93.2	331.7 – 326.2	33	35
		93.2 – 98.2	326.2 – 321.2	22	28
		98.2 – 108.2	321.2 – 311.2	26	62
		108.2 – 118.2	311.2 – 301.2	60	79
		118.2 – 128.2	301.2 – 291.2	48	69
		128.2 – 138.2	291.2 – 281.2	55	78
	AC-7	89.1 – 94.1	330.4 – 325.4	28	32
		94.1 – 99.1	325.4 – 320.4	18	40
		99.1 – 109.1	320.4 – 310.4	42	53
		109.1 – 119.1	310.4 – 300.4	45	68
		119.1– 129.1	300.4 – 290.4	66	73
		129.1 – 139.1	290.4 – 280.4	63	86
	AC-8	87.0 – 89.9	332.4 – 329.5	0	0
		89.9 – 93.5	329.5 – 325.9	42	44
		93.5 – 103.5	325.9 – 315.9	36	46
		103.5 – 113.5	315.9 – 305.9	23	33
		113.5 – 123.5	305.9 – 295.9	41	47
		123.5 – 133.5	295.9 – 285.9	49	68
		133.5 – 134.9	285.9 – 284.5	93	93
	AC-9	90.2 – 96.3	329.2 – 323.1	18	18
		96.3 – 106.3	323.1 – 313.1	47	47
		106.3 – 116.3	313.1 – 303.1	50	71
		116.3 – 126.3	303.1 – 293.1	48	66
		126.3 – 136.3	293.1 – 283.1	68	75
		136.3 – 140.8	283.1 – 278.6	76	89
	B-2	88.0 – 93.0	332.2 – 327.2	44	44
		93.0 – 103.0	327.2 – 317.2	56	73
		103.0 – 113.0	317.2 – 307.2	63	95
		113.0 – 116.0	307.2 – 307.2	37	37
Pier 4	AC-10	84.2 – 85.5	334.1 – 332.8	0	0
		85.5 – 95.5	332.8 – 322.8	22	27
		95.5 – 105.5	322.8 – 312.8	63	75
		105.5 – 115.5	312.8 – 302.8	57	75
		115.5 – 125.5	302.8 – 292.8	63	92
		125.5 – 135.5	292.8 – 282.8	68	68
	AC-11	81.7 – 82.9	337.7 – 336.5	0	0
		82.9 – 87.9	336.5 – 331.5	38	52
		87.9 – 97.9	331.5 – 321.5	75	86
		97.9 – 107.9	321.5 – 311.5	72	79
		107.9 – 117.9	311.5 – 301.5	74	93
		117.9 – 127.9	301.5 – 291.5	63	83
		127.9 – 132.9	291.5 – 286.5	74	78

Table 2. Summary of Rock Core Data

Substructure Element	Boring No.	Depth Interval	Elevation	KY RQD (%)	Std. RQD (%)
Pier 4	AC-12	81.7 – 83.5	336.7 – 334.9	0	0
		83.5 – 88.5	334.9 – 329.9	18	20
		88.5 – 98.5	329.9 – 319.9	38	58
		98.5 – 108.5	319.9 – 309.9	46	71
		108.5 – 118.5	309.9 – 299.9	62	80
		118.5 – 128.5	299.9 – 289.9	81	81
		128.5 – 131.7	289.9 – 286.7	100	100
	AC-13	82.3 – 84.3	336.6 – 334.6	0	0
		84.3 – 89.3	334.6 – 329.6	28	28
		89.3 – 99.3	329.6 – 319.6	25	25
		99.3 – 109.3	319.6 – 309.6	65	73
		109.3 – 119.3	309.6 – 299.6	59	72
		119.3 – 129.3	299.6 – 289.6	65	73
		129.3 – 132.3	289.6 – 286.6	57	57
	B-3	82.0 – 92.0	337.9 – 327.9	39	55
		92.0 – 102.0	327.9 – 317.9	59	92
		102.0 – 112.0	317.9 – 307.9	60	91
		112.0 – 122.0	307.9 – 297.9	64	80
Pier 5	AC-14	15.0 – 19.2	421.0 – 416.8	64	64
		19.2 – 29.2	416.8 – 406.8	70	80
		29.2 – 34.0	406.8 – 402.0	52	63
		34.0 – 43.0	402.0 – 393.0	64	64
		43.0 – 53.0	393.0 – 383.0	95	95
		53.0 – 62.9	383.0 – 373.1	97	97
		62.9 – 72.9	373.1 – 363.1	73	90
		72.9 – 81.5	363.1 – 354.5	80	98
	AC-15	27.0 – 29.3	392.4 – 390.1	65	78
		29.3 – 32.3	390.1 – 387.1	33	50
		32.3 – 42.3	387.1 – 377.1	62	82
		42.3 – 47.3	377.1 – 372.1	66	66
		47.3 – 52.3	372.1 – 367.1	92	92
		52.3 – 62.3	367.1 – 357.1	77	77
		62.3 – 72.3	357.1 – 347.1	68	85
		72.3 – 77.3	347.1 – 342.1	64	64
	B-4	15.5 – 21.5	404.7 – 398.7	53	60
		21.5 – 31.5	398.7 – 388.7	85	98
		31.5 – 41.5	388.7 – 378.7	84	97
		41.5 – 51.5	378.7 – 368.7	83	87
		51.5 – 56.2	368.7 – 364.0	80	96

Table 2. Summary of Rock Core Data

Substructure Element	Boring No.	Depth Interval	Elevation	KY RQD (%)	Std. RQD (%)
Indiana Abutment	AC-20	3.7 – 7.5	491.0 – 487.2	61	61
		7.5 – 17.5	487.2 – 477.2	80	84
		17.5 – 27.5	477.2 – 467.2	61	83
		27.5 – 37.5	467.2 – 457.2	70	93
		37.5 – 44.0	457.2 – 450.7	95	95
	AC-23	3.6 – 8.0	489.7 – 485.3	82	82
		8.0 – 18.0	485.3 – 475.3	45	49
		18.0 – 23.0	475.3 – 470.3	83	83
		23.0 – 28.0	470.3 – 465.3	60	78
		28.0 – 38.0	465.3 – 455.3	96	96
		38.0 – 48.0	455.3 – 445.3	100	100
		48.0 – 57.5	445.3 – 435.8	82	82
		57.5 – 67.5	435.8 – 425.8	96	96
		67.5 – 74.2	425.8 – 419.1	80	87
	AC-26	7.4 – 17.5	491.1 – 481.0	80	80
		17.5 – 27.5	481.0 – 471.0	60	71
		27.5 – 37.5	471.0 – 461.0	62	91
		37.5 – 47.5	461.0 – 451.0	86	92
Indiana Abutment Wing Wall	AC-17	2.0 – 7.0	490.0 – 485.0	86	86
		7.0 – 12.0	485.0 – 480.0	74	74
	AC-27	1.8 – 6.5	491.8 – 487.1	87	87
		6.5 – 12.0	487.1 – 481.6	35	35
		12.0 – 17.2	481.6 – 476.4	0	0
		17.2 – 22.2	476.4 – 471.4	0	22
		22.2 – 27.2	471.4 – 466.4	31	66

5.2 Field Wave Velocity Measurements for Seismic Design

In order to provide shear wave and compression wave velocities for soil and bedrock at the bridge site in support of seismic analyses, suspension velocity measurements were obtained in the soils and bedrock of the site. Pier 1 was selected as the location for the test because it presented the deepest soil deposits for the site. On October 17, 2007 OYO suspension velocity measurements were performed in a cased, water filled hole at the location of Boring AC-3 by GEOVision Geophysical Services of Corona, California. The resulting report produced by GEOVision is presented in its entirety in Appendix D.

In general, the method consisted of lowering a single probe down the cased boring. The probe contained both a sending unit and receivers. The sending unit created a horizontal pressure wave in the borehole fluid which was converted to shear and compression waves in the surrounding soil or bedrock. The receivers recorded the resulting waves and the data was filtered to calculate compression (p) and shear (S_H) wave velocities at specific intervals from the ground surface to the bottom of the boring.

5.3 Laboratory Testing Program

5.3.1 General

FMSM personnel conducted laboratory testing of the samples obtained during the field explorations for the bridge at the Lexington, Kentucky laboratory facility in accordance with applicable AASHTO or Kentucky Methods of soil and rock testing. The results of the laboratory testing are shown on the Subsurface Data Sheets presented in Appendix A. Tests performed on the soil samples obtained during drilling consisted of moisture contents, particle size analyses, Atterberg limits, and specific gravity determinations. Groups of SPT samples of like soil types were combined and subjected to composite classification testing. Laboratory testing for undisturbed thin-walled (Shelby) tube samples included unconfined compressive strength (UC) and soil classification tests. The results for all soil testing are presented in Appendix E.

Tests performed on rock core samples recovered from the borings consisted of unconfined compressive strength, direct shear tests, and Slake Durability Index (SDI) testing. The results of the rock testing are presented in Appendix F. Six soil samples and three water samples from the Ohio River were also subjected to resistivity and corrosivity testing. The results of corrosivity and resistivity results are presented in Appendix G.

5.3.2 Soil Classification Testing

FMSM laboratory personnel completed classification tests on 67 samples of the foundation soils collected from the borings at the proposed substructure locations. This testing resulted in the identification of fourteen soil types as defined by the Unified Soil Classification System (USCS) method, and six soil types defined by the American Association of State Highway and Transportation Officials (AASHTO) system. Of the 67 samples tested, 18 samples classified as SP-SM by the USCS system, 12 classified as SW-SM, and 12 classified as SM, SW, SP, or SC. Sixteen of the samples tested were classified as GW, GP-GM, GM, GW-GM, or GP, with nine samples being classified as CL, CL-ML, or ML. A summary of the results is presented in Table 3.

The majority of the soils tested were identified as non-cohesive soils. This correlates well with the referenced geologic mapping, which identifies non-cohesive alluvial soils consisting of poorly to well-graded sands and gravels occurring beneath the Kentucky and Ohio River portions of the bridge.

Table 3a. Summary of USCS Soil Classification Data

USCS Soil Classification	Number of Soils Identified
CL	7
CL-ML	1
SM	5
SW-SM	12
SW	3
SC	1
SP-SM	18
SP	3
GP-GM	5
GM	3
GW	6
GW-GM	1
GP	1
ML	1

Table 3b. Summary of AASHTO Soil Classification Data

AASHTO Soil Classification	Number of Soils Identified
A-7-6	3
A-6	4
A-4	3
A-1-b	38
A-2-4	4
A-1-a	15

5.3.3 Unconfined Compressive Strength Testing on Soil

Laboratory personnel completed unconfined compressive strength tests on six selected thin-walled tube samples to provide information from which total stress shear-strength parameters could be estimated. The results of the unconfined compressive strength tests are presented on the appropriate Subsurface Data Sheets and are summarized in Table 4.

The six samples from Borings AC-1 through AC-3 represent cohesive strata of the location of Pier 1 and returned test values with an average strength of 1533 psf. These values ranged from 280 psf to 3140 psf, with two of the values being less than 1000 psf. The lowest value was returned for a sample obtained immediately above the change in material type from a clay to a sand with clay and immediately above the noted water table. It is possible that a very thin sand lense or zone of saturation was present within the sample and established a plane of weakness which resulted in a low failure strength. A value of 680 psf was returned from a sample interval obtained within 3.0 feet of the existing ground surface. This low value could be explained by the presence of a silt lens or organic remnant within the sample. The range of unconfined compressive strength values may also be attributed to the soil material being from alluvial deposits which by nature can be non-uniform over relatively short vertical or horizontal distances.

Table 4. Summary of Unconfined Compressive Strength Tests on Soil

Boring No.	Station	Offset	Depth Interval (ft)	Unit Weights		Moisture Content (%)	U.C. Strength (psf)	USCS Classification
				Dry (pcf)	Wet (pcf)			
AC-1	187+18.6	44.6 Lt.	2.5 – 4.5	89.8	106.9	19.1	3140	CL
AC-1	187+18.6	44.6 Lt.	10.0 – 12.0	101.5	125.2	23.4	1160	CL
AC-2	187+28.4	13.5 Lt.	5.0 – 7.0	94.8	112.4	18.6	2820	CL
AC-2	187+28.4	13.5 Lt.	20.0 – 22.0	104.0	128.5	23.6	280	CL
AC-3	187+46.6	60.9 Rt.	2.5 – 4.5	82.7	110.7	33.9	680	CL
AC-3	187+46.6	60.9 Rt.	10.0 – 12.0	88.4	118.8	34.3	1120	CL

5.3.4 Unconfined Compressive Strength Testing on Rock

A total of 50 rock core samples obtained from the borings were tested for unconfined compressive strength. Samples were selected at elevations within or below the likely drilled shaft rock socket limits. The consideration of shallow foundations at the Indiana Abutment focused the selection of two samples at shallow elevations. The results of testing varied from a low value of 13 tons per square foot returned by a shale sample from Boring AC-14, to a high value of 1,037 tons per square foot in a limestone shale mixture sample recovered from Boring AC-15. For individual rock test results refer to Table 5.

5.3.5 Direct Shear Testing of Rock Samples

To support deep foundation design, a total of 30 rock core samples from the borings were subjected to direct shear testing. The samples were specifically oriented in the testing mold in order to facilitate shear along a limestone/shale interface. This orientation was considered to present the most representative failure surface within the samples recovered. Because the pier foundations are likely to consist of drilled shafts socketed into bedrock, the normal stress confining the sample was estimated to be equal to the existing overburden pressure at the depth of the sample being tested. Both peak and post peak values of shear stress were recorded during the test. A maximum peak shear stress of 542.5 pounds per square inch was returned at a normal stress of 46.3 pounds per square inch in Boring AC-15. The minimum peak shear stress returned from testing was 20 pounds per square inch under a normal stress of 31.4 pounds per square inch in Boring AC-6. As would be expected, the post peak shear stresses were lower than the peak stresses under the same normal stress. The maximum post peak shear stress recorded was 88.4 pounds per square inch at a normal stress of 64.9 pounds per square inch in Boring AC-3. The minimum post peak shear stress recorded was 10.9 pounds per square inch under a normal stress of 35.3 pounds per square inch in Boring AC-15. These maxima and minima vary greatly because of the tests being performed in multiple geologic units and of the variance in failure surface competency and orientation. Refer to Table 5 for results of individual direct shear tests.

5.3.6 Slake Durability Index Testing

Samples of the bedrock cored were selected for Slake Durability Index (SDI) testing. This test simulates the weathering processes of bedrock exposed to the elements, and is typically performed on shales. The process involves placing a measured weight of rock sample in a closed wire basket which rotates vertically while submerged in water. The sample pieces are subjected to a series of tumble (wet) and dry cycles and then weighed. The remaining sample weight is divided by the original sample weight to determine the percentage of remaining sample (SDI value). Therefore, the more durable the rock the more sample remains, and the higher the SDI value.

The use of this test, relative to the East End Bridge, relates to the installation of drilled shafts in bedrock. If drilled shafts are installed into bedrock, the SDI tests will indicate if there should be concern if water is used as the circulating agent. Shales with low SDI values may degrade upon exposure to drilling fluid and be removed, leaving spaces between limestone/dolomite layers. The layers may become loose and collapse into the shaft during concrete placement, thereby jeopardizing the integrity and structural capacity of the drilled shaft. Based on the guidelines presented in the Kentucky Transportation Cabinet – Geotechnical Manual, shales that have an SDI from 50 to 94 are potentially degradable and those with an SDI of less than 50 should be considered soil-like (degradable). SDI values above 94 indicate the sample is durable and should not degrade. The results of SDI testing preformed for the East End Bridge are presented in Table 6.

Six of the 41 samples tested returned values with an SDI value less than 50 and are therefore considered degradeable. Review of the rock core indicates the shale layers in the tested intervals are typically less than 0.4 feet in vertical thickness. The locations of these seams should be further evaluated during the final design of the bridge foundations relative to the tip elevation of the shaft and anticipated installation practices.

Table 5. Summary of Laboratory Rock Test Data

Structure Element	Hole #	Depth (ft)	Elevation (ft)	Direct Shear Test				Unconfined Compression Strength		Rock Type
				Peak		Post Peak				
				Normal Stress (psi)	Peak Shear Stress (psi)	Normal Stress (psi)	Peak Shear Stress (psi)	Peak Strength (tsf)	Failure Type	
Pier 1	AC-1	114.95 - 115.35	319.16 - 318.76	---	---	---	---	625	Shear	Mix*
	AC-1	114.05	320.06	66.7	246.9	66.7	37.5	---	---	Mix*
	AC-1	120.10	314.01	73.5	178	73.5	47.5	---	---	Mix*
	AC-1	127.30 - 127.65	306.81 - 306.46	---	---	---	---	296	Shear	Mix*
	AC-2	103.10	330.89	51.3	70.1	51.3	24.7	---	---	Mix*
	AC-2	113.55	320.44	63.1	135.9	63.1	50.2	---	---	Mix*
	AC-2	119.10 - 119.50	314.89 - 314.49	---	---	---	---	538	Undetermined	Mix*
	AC-2	124.70 - 125.10	309.29 - 308.89	---	---	---	---	179	Shear	Mix*
	AC-3	102.00	331.72	50.5	136.1	50.5	37	---	---	Mix*
	AC-3	114.90	318.82	64.9	322.5	64.9	88.4	---	---	Mix*
	AC-3	124.10 - 124.7	309.62 - 309.02	---	---	---	---	525	Shear	Mix*
	AC-3	128.60 - 129.20	305.12 - 304.52	---	---	---	---	300	Cone and Split	Mix*
Pier 2	AC-4	102.15	317.65	43.3	443.5	43.3	37.8	---	---	Mix*
	AC-4	116.65	303.15	59.4	112.8	59.4	58.1	---	---	Limestone
	AC-4	117.40 - 117.80	302.40 - 302.00	---	---	---	---	662	Columnar	Mix*
	AC-4	123.60	296.20	67.0	207.8	67.0	26.2	---	---	Mix*
	AC-4	124.35 - 124.75	295.45 - 295.05	---	---	---	---	409	Shear	Mix*
Pier 3	AC-6	97.40	322.00	31.4	20.0	31.4	18.3	---	---	Shale
	AC-6	105.00 - 105.35	314.40 - 314.05	---	---	---	---	594	Cone and Split	Mix*
	AC-6	110.70	308.70	45.8	164.8	45.8	38.6	---	---	Mix*
	AC-6	113.35 - 113.70	306.05 - 305.70	---	---	---	---	309	Shear	Mix*
	AC-6	128.20 - 128.55	291.20 - 290.85	---	---	---	---	586	Cone and Split	Mix*
	AC-7	100.00 - 100.35	319.50 - 319.15	---	---	---	---	691	Cone and Shear	Mix*
	AC-7	103.15	316.35	36.2	39.9	36.2	19.5	---	---	Mix*
	AC-7	109.30 - 109.65	310.20 - 309.85	---	---	---	---	881	Cone and Split	Mix*
	AC-7	111.90 - 112.35	307.60 - 307.15	---	---	---	---	837	Cone and Split	Mix*
	AC-7	115.80	303.70	50.3	196.0	50.3	36.1	---	---	Mix*
	AC-8	98.55	320.85	33.0	371.8	33.0	35.7	---	---	Mix*
	AC-8	99.30 - 99.70	320.10 - 319.70	---	---	---	---	838	Cone and Split	Mix*
	AC-8	102.85 - 103.20	316.55 - 316.20	---	---	---	---	34	Cone	Shale
	AC-8	113.75	305.65	49.8	83.1	49.8	32.2	---	---	Mix*
	AC-8	120.65 - 121.05	298.75 - 298.35	---	---	---	---	577	Shear	Limestone
	AC-9	100.90	318.50	32.9	82.8	32.9	14.0	---	---	Shale

Table 5. Summary of Laboratory Rock Test Data

Structure Element	Hole #	Depth (ft)	Elevation (ft)	Direct Shear Test				Unconfined Compression Strength		Rock Type
				Peak		Post Peak				
				Normal Stress (psi)	Peak Shear Stress (psi)	Normal Stress (psi)	Peak Shear Stress (psi)	Peak Strength (tsf)	Failure Type	
Pier 3 (Continued)	AC-9	105.75 - 106.15	313.65 - 313.25	---	---	---	---	507	Undetermined	Mix*
	AC-9	115.95	303.45	49.6	191.9	49.6	19.3	---	---	Mix*
	AC-9	116.50 - 116.90	302.90 - 302.50	---	---	---	---	535	Undetermined	Mix*
	AC-9	119.80 - 120.15	299.60 - 299.25	---	---	---	---	760	Shear	Limestone
Pier 4	AC-10	100.20 - 100.50	318.10 - 317.80	---	---	---	---	373	Shear	Mix*
	AC-10	101.35	316.95	36.3	109.1	36.3	24.9	---	---	Mix*
	AC-10	104.80 - 105.20	313.50 - 313.10	---	---	---	---	253	Undetermined	Mix*
	AC-10	112.50 - 112.90	305.80 - 305.40	---	---	---	---	961	Cone and Split	Mix*
	AC-10	117.45	300.35	54.2	458.6	54.2	34.1	---	---	Mix*
	AC-11	89.75	329.65	27.0	181.8	27.0	33.4	---	---	Mix*
	AC-11	92.10 - 92.55	327.30 - 326.85	---	---	---	---	412	Undetermined	Mix*
	AC-11	103.80	315.60	42.3	68.7	42.3	44.4	---	---	Mix*
	AC-11	104.60 - 105.00	314.80 - 314.40	---	---	---	---	431	Cone and Split	Mix*
	AC-11	106.95 - 107.30	312.45 - 312.10	---	---	---	---	636	Undetermined	Mix*
	AC-12	94.00	324.40	31.2	328.0	31.2	29.4	---	---	Mix*
	AC-12	102.85 - 103.10	315.55 - 315.30	---	---	---	---	501	Shear	Mix*
	AC-12	103.40 - 103.75	315.00 - 314.65	---	---	---	---	724	Undetermined	Mix*
	AC-12	109.30	309.10	47.9	93.9	47.9	28.7			Mix*
	AC-12	111.40 - 111.80	307.00 - 306.60	---	---	---	---	640	Cone and Split	Mix*
	AC-13	90.65 - 91.00	328.25 - 327.90	---	---	---	---	728	Undetermined	Mix*
	AC-13	95.15	323.75	32.9	237.6	32.9	12.2	---	---	Mix*
	AC-13	97.75 - 98.10	321.15 - 320.80	---	---	---	---	335	Undetermined	Mix*
	AC-13	104.05	314.85	42.9	98.6	42.9	27.8	---	---	Mix*
	AC-13	105.85 - 106.20	313.05 - 312.70	---	---	---	---	465	Undetermined	Mix*
Pier 5	AC-14	29.20 - 29.55	406.78 -406.43	---	---	---	---	13	Shear	Shale
	AC-14	30.50	405.48	30.3	345.7	30.3	12.5	---	---	Mix*
	AC-14	34.10 - 34.45	401.88 - 401.53	---	---	---	---	988	Cone and Shear	Limestone
	AC-15	35.40 - 35.80	384.00 - 383.60	---	---	---	---	1037	Undetermined	Mix*
	AC-15	48.20 - 48.60	371.20 - 370.80	---	---	---	---	616	Shear	Limestone
	AC-15	49.90	369.50	35.3	354.6	35.3	10.9	---	---	Shale
	AC-15	59.70	359.70	46.3	542.5	46.3	20.2	---	---	Mix*
	AC-15	60.40 - 60.80	359.00 - 358.60	---	---	---	---	693	Cone and Shear	Mix*
Indiana Abutment	AC-20	8.45 - 8.85	486.23 - 485.83	---	---	---	---	602	Shear	Limestone
	AC-20	28.2	466.48	30.3	50.6	30.3	19.5	---	---	Mix*
	AC-20	29.80 - 30.20	464.88 - 464.48					80	Shear	Shale

Table 5. Summary of Laboratory Rock Test Data

Structure Element	Hole #	Depth (ft)	Elevation (ft)	Direct Shear Test				Unconfined Compression Strength		Rock Type
				Peak		Post Peak				
				Normal Stress (psi)	Peak Shear Stress (psi)	Normal Stress (psi)	Peak Shear Stress (psi)	Peak Strength (tsf)	Failure Type	
Indiana Abutment (Continued)	AC-23	23.85 - 24.25	469.46 - 469.06	---	---	---	---	291	Cone and Shear	Mix*
	AC-26	10.50 - 10.90	488.04 - 487.64	---	---	---	---	862	Cone and Shear	Limestone
	AC-26	27.9	470.64	29.2	40.3	29.2	20.5	---	---	Mix*
	AC-26	32.40 - 32.80	466.14 - 465.74	---	---	---	---	98	Shear	Shale
Indiana Retaining Wall	AC-17	6.45 - 6.85	485.59 - 485.19	---	---	---	---	564	Shear	Limestone
	AC-27	4.20 - 4.55	489.42 - 489.07	---	---	---	---	518	Shear	Limestone
	AC-27	25.90 - 26.35	467.72 - 467.27	---	---	---	---	302	Shear	Limestone

*-Mix- Mixture of shale and limestone

Table 6. Summary of Slake Durability Index Testing

Substructure Element	Hole No.	Sample Depth (feet)	SDI
	AC-1	106.7 - 107.2	77.7
	AC-1	117.2 - 117.9	76.3
	AC-1	128.1 - 128.7	95.8
	AC-2	104.8 - 105.4	82.2
Pier 1	AC-2	120.7 - 121.1	95.3
	AC-2	129.9 - 130.9	91.1
	AC-3	87.2 – 88.3	48.0
	AC-3	102.5 - 102.9	95.9
	AC-3	106.7 - 107.9	36.6
	AC-4	94.3 - 95.0	92.9
Pier 2	B-1	117.2 - 118.0	94.8
	AC-6	95.7 – 96.1	56.8
	AC-6	102.1 - 102.8	76.3
	AC-7	106.9 - 107.5	72.1
	AC-8	99.4 - 100.2	60.6
	AC-8	108.2 – 108.9	87.7
Pier 3	AC-8	114.8 - 115.3	6.2
	AC-9	94.7 – 95.1	41.5
	AC-9	107.4 - 108.0	86.8
	AC-9	117.5 - 117.9	6.4
	B-2	102.7 - 103.4	83.1
	AC-10	86.8 - 87.4	71.1
	AC-10	99.5 – 100.3	88.4
	AC-10	108.0 - 108.4	81.7
	AC-11	85.6 - 86.2	33.2
	AC-11	95.3 – 95.8	91.1
	AC-11	109.7 - 110.6	94.4
Pier 4	AC-12	92.8 – 93.5	89.9
	AC-12	101.9 - 102.7	91.9
	AC-12	102.9 - 103.5	10.2
	AC-13	22.1 – 23.9	96.1
	AC-13	97.7 - 98.5	89.5
	AC-13	108.7 - 109.3	97.5
	B-3	48.9 - 49.6	98.1
	AC-14	57.5 - 58.8	96.6
Pier 5	AC-15	25.2 - 26.2	56.5
	AC-15	26.6 - 27.5	88.8
	AC-23	35.7 - 36.4	75.2
Indiana Abutment	AC-26	23.3 - 24.0	71.8
	AC-26	106.7 - 107.2	77.7
	AC-27	117.2 - 117.9	76.3

5.3.7 Corrosivity Tests on Soil and Water

Chemical characterization tests were performed on six composite soil samples obtained during drilling and on three water samples obtained from the Ohio River. These tests were performed to identify potentially corrosive environments or subsurface materials which may affect foundation design. The soil samples were comprised of composite samples of similar soil type from multiple boring locations, and were tested by CTL Group of Skokie, Illinois, in terms of water soluble sulfate content (AASHTO T290) and minimum soil resistivity (AASHTO T288). The water samples were obtained approximately 4 feet below the river surface near the Kentucky shore, near mid-river, and near the Indiana shore. The samples were tested by Microbac Laboratories, Inc. of Louisville, Kentucky in terms of pH (SM4500), Chloride content (EPA 300.0) and Sulfate content (EPA 300). The results of the testing are presented in Appendix G, and are summarized in Tables 7 and 8.

Table 7. Summary of Soil Corrosivity Tests

Material Description	Sample Source	Water Soluble Sulfate (as SO4) (mg/kg of sample)	Minimum Resistivity (Ohm-cm)
Lean Clay	Borings AC-1, 2	4	2118
Silty Sand with Gravel	Borings AC-1, 2	33	3135
Well-graded Sand	Borings AC-1, 2, 3	41	2570
Sand with Silt and Gravel	Borings AC-6, 9, 10,11, 12, 13, 15,	82	1864
Poorly graded Sand with Silt and Gravel	Borings AC-10, 11, 12, 13	144	1356
Well-graded Sand with Silt	Borings AC-5, 9, 10, 11, 12, 13	86	2486

Table 8. Summary of Chemical Analysis of Water

Sample Source	pH (SU)	Chloride (mg/l)	Sulfate (mg/l)
Kentucky Shore	7.50	50	130
Mid-River	7.58	50	130
Indiana Shore	7.65	50	130

6.0 SUBSURFACE CONDITIONS

6.1 Overview of Bridge Site Stratigraphy

Available geologic mapping indicates that areas in the vicinity of the I-265 East End Bridge over the Ohio River are underlain by Quaternary sediments and soil, as well as Devonian, Silurian, and Ordovician age bedrock. These sediments and soil consist of, in lithologic order, alluvium, lacustrine deposits, outwash, as well as loess and eolian sand. The alluvium was deposited during the Holocene epoch while the lacustrine, outwash, loess, and eolian sand deposits were deposited during the Pleistocene epoch of geologic time. Typically soils and sediments located within the Ohio River floodplains are of the Huntington-Melvin-Combs complex, which are classified as sandy, loamy, and silty soils that are very deep, well drained to poorly drained, nearly level to moderately steep, and are flooded frequently.

A plan showing the approximate contours of the bedrock surface, based on the data obtained from this subsurface exploration program, is presented as Figure 4.

6.2 Kentucky Transition Pier

The Kentucky Transition Pier (Pier 1) is located at Station 187+40. At this location, Borings AC-1, AC-2 and AC-3 were drilled to provide soil and bedrock data for design. The results of subsequent soil and bedrock testing were used to develop a generalized subsurface profile for the transition pier to provide strength parameters for design. This generalized profile is presented in Figure 5b.

6.2.1 Stratigraphy

Soils encountered during the subsurface exploration, listed by lithologic order, consisted of lean clay, sandy silty clay, silty sand with gravel, well-graded sand with silt, and well-graded sand. Groundwater was encountered at elevations ranging from 421.7 to 413.5 feet with a bedrock surface elevation that ranged from 333.6 to 334.9 feet.

6.2.2 Soil Conditions

As depicted in the generalized soil profile, the uppermost horizon is approximately 25 feet in thickness and is a medium strength low plasticity lean clay with an average unconfined compressive strength of 1,784 pounds per square foot. Beneath the clay a layer generally described as a sand with silt was encountered with a thickness of approximately 30 feet and a bottom elevation of 379 feet. This sand was described as brown to gray, medium to coarse-grained, and loose to medium in consistency. Below elevation 379 feet a well-graded sand with silt and gravel was encountered. This horizon continued to the bedrock surface at elevation 334 feet and was described as medium to coarse-grained and medium dense. Drilling encountered occasional gravels in this horizon.

6.2.3 Rock Conditions

The bedrock encountered at Pier 1 correlates will with the referenced geologic mapping, for the Drakes Formation and is described as limestone (60%) interbedded with shale (40%). The limestone is gray, fine grained, thin bedded and locally argillaceous. The shale is gray, silty, and calcareous. The upper 30 to 40 feet of the bedrock was noted to contain occasional clay seams. Unconfined compressive strength test results varied from 179 to 625 tons per square foot. SDI testing returned values ranging from 37 to 95.

6.2.4 Field Wave Velocity for Seismic Design

The results of suspension logging of Boring AC-3 returned the shear wave velocity ranges presented in Table 9.

Table 9. Summary of Shear Wave Velocity Measurements

Depth	Material	V_S ft/sec
0-25	CL	270-499
25-55	SM, SW	556-1042
55-100	SW-SM	924-1197
100-150.9	Limestone/Shale Mix	2137-7499

Below the water table (elevation 418.3 at Pier 1) and above the bedrock surface compression waves are of little value because the water directly carries the wave signal and returns a typical water velocity on the order of 5000 feet per second. The bedrock returned shear wave velocities between 3,000 and 7,000 feet per second.

6.3 Ohio River – Tower and Anchor Piers

The tower piers for the Ohio River consist of Piers 3 (Station 193+77) and 4 (Station 206+12) and were investigated by Borings AC-6 through AC-13 and Borings B-2 and B-3. The Kentucky Anchor Pier (Pier 2) is located at Station 189+65 and is described by Borings AC-4, AC-5, and B-1. The Indiana Anchor Pier (Pier 5) is located at Station 210+24 and was investigated by Borings AC-14, AC-15, and B-4. A generalized subsurface profile has been prepared for each of these substructure elements and should be utilized during foundation analyses and design. These profiles are presented in Figure 5c, 5d, 5e, and 5f, for Piers 2, 3, 4, and 5 respectively.

6.3.1 Stratigraphy

Soils encountered during the subsurface exploration, listed by lithologic order, consisted of well-graded sand with silt and gravel, poorly graded sand with silt and gravel, and poorly graded sand with silt. Bedrock surface elevations ranged from 423.9 feet at the Ohio River bank on the Indiana side to 334.9 feet at the Ohio River bank on the Kentucky side.

6.3.2 Soil Conditions

The soils at the Kentucky Anchor Pier consisted of approximately five feet of soft, sandy, lean clay above the pool elevation of the river. In order of descending elevation the following horizons were encountered below the water surface: 14 feet of sand with silt which is medium to coarse-grained and medium in consistency, 30 feet of well-graded sand with silt which is medium to coarse-grained and medium in consistency, and 45 feet of well-graded sand with silt and gravel which is medium to coarse-grained, medium to dense in consistency and contains sub-rounded to rounded gravel. The bedrock surface was encountered at elevations ranging from 328.5 to 338.6 feet.

The Kentucky Tower Pier will rest in approximately 40 feet of water on sands and gravel horizons. Beginning at a river bottom elevation of 379 feet and decreasing in elevation, the following four soil horizons were noted: 14 feet of well-graded sand with silt and gravel which is medium to coarse-grained and medium in consistency, 11 feet of well-graded sand with silt and gravel which is medium to dense in consistency, 15 feet of dense to very dense well-graded gravel with silt, and 7 feet of poorly graded sand with silt overlying bedrock which is medium grained and is medium to very dense in consistency.

The Indiana Tower Pier also will be located in approximately 40 feet of water with a river bottom elevation of 379 feet. The three soil horizons encountered below the river bottom in order of decreasing elevation, are: 19 feet of poorly to well-graded gravel with sand which is loose to medium in consistency, 16 feet of poorly graded sand with silt and gravel which is medium to coarse-grained and medium to very dense in consistency, and 7 feet of medium to very dense poorly sorted gravel with silt.

The Indiana Anchor Pier is located at the edge of the Ohio River and will fall on both a steep slope rising out of the river and upon alluvial sands and gravels within the river. Because of utility conflicts, Boring AC-14 was advanced within the limits of River Road. The soil beneath River Road is described as moist sandy lean clay which is soft in consistency, and contains some gravel. Bedrock was encountered at an elevation of 424 feet. Borings B-4 and AC-15 were advanced in the river and encountered river bottom elevations of 419 and 416 feet, respectively. Beneath the river bottom, in Boring AC-15, four feet of loose sandy silt with gravel was encountered overlying 17 feet of dense to very dense poorly graded gravel with silt and sand. Bedrock was encountered at an elevation of 392 feet.

6.3.3 Rock Conditions

At the Kentucky Anchor Pier location the bedrock surface elevation varied from 328.5 to 338.6 feet. The top 52 feet of the bedrock was described as limestone (55%) interbedded with shale (45%). This correlates well with the referenced mapping of the Drakes Formation. The limestone is gray, fine grained, thin bedded, locally argillaceous and locally fossiliferous. At approximate elevation 283 feet the percentage of shale in the unit increased to 70 percent and the limestone decreased to 30 percent.

The bedrock underlying both the Kentucky and Indiana Tower location is also of the Drakes Formation and is described as limestone (which varies from 50% to 80% of the unit) interbedded with shale (30 to 50% of the unit). The limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous. The shale is silty, laminated to thinly bedded, calcareous and fossiliferous. The surface elevation of the bedrock varied from 330.4 to 333.2 feet at the Kentucky Tower and from 334.1 to 340.4 feet at the Indiana Tower location.

The bedrock surface rises significantly at the location of the Indiana Anchor Pier, with an elevation of 423.9 feet encountered in Boring AC-14 and 392.4 feet noted in Boring AC-15. Five distinct rock units were identified in these borings and belong to two geologic formations as identified in the referenced geologic mapping. The Laurel dolomite is represented by four of the rock units and was encountered between the elevations of 421 and 392 feet. Between elevation 421 and 417 feet the bedrock is a gray, medium grained limestone which is thin to medium bedded. From elevation 417 to 407 the unit consists of 60 percent limestone and 40 percent shale which are interbedded. The limestone is gray, fine to medium grained and very thin to medium bedded, and the shale is gray and silty. A layer of gray to red shale was encountered from elevation 407 to 404 feet, and described as very thin bedded and silty. Testing on this shale indicates it is of low strength and highly degradable. Beneath this shale, a light gray, fine grained, thinly bedded limestone unit was noted from elevation 404 to 392 feet. From elevation 394 to 392 feet this unit becomes dolomitic and is greenish gray in color. Below elevation 392 feet, the Osgood Formation was identified as interbedded limestone and shale. The limestone varies from 30 to 60 percent of the unit while the shale varies from 40 to 70 percent of the unit. The limestone is gray, fine to medium grained, very thin to medium nodulary bedded and fossiliferous. The shale of the unit is gray, silty, laminated, calcareous and fossiliferous.

6.4 Indiana Abutment

6.4.1 Stratigraphy

Soils occurring in the area of the abutment consist of sand and gravelly lean clay, and range from two to twelve feet in thickness. Bedrock surface elevations varied from 490 to 492 feet in the borings advanced. A generalized subsurface profile has been prepared for the Indiana Abutment and should be utilized during foundation analyses and design. This profile is presented in Figure 5g.

6.4.2 Soil Conditions

Soils encountered during the drilling program consist primarily of varying percentages of clays, gravels and sands. Previous mining or earth moving operations have mixed the residual site soils with gravel and sand-size particles to attain the current soil matrix. The soil is typically brown, soft, and displays low plasticity because of the granular content. The thickness of these soils overlying bedrock varied from 1 to 7 feet.

6.4.3 Rock Conditions

The bedrock surface elevations encountered at the Indiana Abutment location varied from 490 to 492 feet. Beneath this surface, the first of three rock strata was cored. This rock unit consisted of gray limestone which is fine grained, very thick bedded, locally contains clay seams, and was approximately 16 feet in thickness. At the location of Boring AC-27 a clay seam with a thickness of two feet was encountered from elevation 483.6 to 481.6. The limestone terminated at elevation 474 feet, at the top of a shale unit with a thickness of 15 feet. This shale unit was dark gray and tan, very thick, bedded, and ended at the top of another limestone unit at elevation 459 feet. From elevation 459 to 423 feet a gray and tan limestone was cored. This rock was described as fine grained, medium bedded to very thick bedded with zones fractured, and locally dolomitic. Below the limestone a shale layer with a thickness of two feet was recovered from elevation 423 to 421 feet, and was described as dark gray and medium bedded. Beneath the shale and above the bottom of the boring at elevation 419 feet a light gray, medium bedded, limestone was encountered.

7.0 GEOTECHNICAL EVALUATION

The East End Bridge over the Ohio River is being designed using AASHTO LRFD methods. Drilled shaft foundations are planned at Piers 1 through 5, and a spread footing foundation is planned at the Indiana Abutment. Analyses have been performed for foundation bearing, uplift and lateral load conditions.

7.1 Geotechnical Design Parameters

Geotechnical design parameters have been developed for each of the substructure elements, as shown in Figures 5a through 5f, Generalized Subsurface Profiles. The generalized subsurface profiles were developed based on average conditions as represented by the borings at each substructure element. Evident outliers in the data were not included when developing the average design parameters. For the bedrock formations, the Mohr-Coulomb strength parameters, cohesion c and friction angle phi, were based on the AASHTO procedures for the use of Rock Mass Ratings (RMR) for estimation of strength (AASHTO C10.4.6.4). The calculated strength parameters at the Indiana Abutment were reduced due to the presence of clay seams at boring AC-23.

For laterally loaded drilled shaft evaluations, use of the average top of rock elevation as depicted in Figure 5 would have been potentially underconservative in predicting deflections under lateral load. Where the top of rock is deeper than average conditions, deflections under imposed lateral load may be larger. In contrast, for the case of imposed deflections due to thermal expansion, or for seismic loading, higher stresses will be obtained where rock is shallower than average conditions, due to a shallower depth of fixity. For preliminary engineering analyses, the average rock elevation is used.

Design scour depths for the project have been developed by Wilbur Smith Associates, with the scour analysis based upon the 100-year storm event. The total design scour depth at the river piers, Piers 3 and 4, is approximately 40 to 43 feet below the bottom of channel. At Pier 3, about 5 to 8 feet of soil is anticipated to remain after scour, and at Pier 4, scour is anticipated to extend to top of rock. To a lesser extent, scour is also anticipated at the transition and anchor piers, Piers 1, 2 and 5, with scour depths of about 13 to 20 feet below the ground surface.

7.2 Seismic Design Parameters

The East End Bridge is considered to be a “Critical” bridge. As such, it is expected to remain serviceable (with minor, repairable damage) following a significant earthquake, and to withstand a lesser earthquake with virtually no damage. Therefore, the East End Bridge is being designed using a dual-seismic hazard approach, considering two sets of ground motions:

- higher level Safety Evaluation Earthquake (SEE) with a return period of approximately 2500 years
- lower level Functional Evaluation Earthquake (FEE) with a return period of approximately 500 years

For the preliminary design phase, design response spectra for the 2500-year SEE were computed following the methodology given in MCEER/ATC 49 Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (2003). A review of the boring logs indicated that site class is between Site Classes C and D. Conservatively, Site Class D was used for the horizontal (longitudinal and transverse) ground motions. The vertical ground motions were computed as 70% of the Site Class B spectra. The response spectra curves used for the preliminary design phase are shown in Figure 6.

In the final design phase, the ground motion inputs (response spectra) for the SEE and FEE will be determined based on a site specific study for the East End Bridge. This will include site response analyses based on data from the boring logs and shear wave velocity measurements (P-S logging) taken at the site.

7.3 Recommended Foundation Types

Rock bearing foundations are recommended for support of the Kentucky transition pier, anchor piers, river piers and the Indiana abutment. Due to the depth to bedrock, drilled shaft foundations are recommended for the Kentucky transition pier, anchor piers, and the river piers. Drilled shaft foundations should be socketed into rock to take advantage of the side friction afforded by the rock socket in compression and uplift, and the high lateral resistance of the rock socket to aid in restraining deflection under lateral loads, particularly at the main pier where the overburden may scour to a depth of more than 40 feet.

Driven pile foundations could not be advanced into rock without predrilling, and therefore would not provide the necessary resistance to uplift and lateral loads. Therefore, driven pile foundations are not recommended for foundations.

At the Indiana abutment, where the depth to rock is shallow, a continuous spread footing foundation bearing on rock is recommended.

7.3.1 Pier 1 - Kentucky Transition Pier

As shown in Table 1, the depth to top of rock at the three borings at Pier 1 ranged from 98.8 to 100.4 feet below ground surface. The corresponding top of rock elevation ranged from 333.6 to 334.9 feet. The preliminary design scour depth at this pier was subsequently determined to be 15.2 feet, or elevation 418.8 feet; a scour depth of 16 feet was used in the foundation analyses. Rock-socketed drilled shaft foundations are recommended, with one drilled shaft supporting each pier column.

7.3.2 Pier 2 - Kentucky Anchor Pier

As shown in Table 1, the depth to top of rock at the three borings at Pier 2 ranged from 81.0 to 98.4 feet below ground surface or river water surface, with a corresponding top of rock elevation ranging from 328.1 to 338.6 feet. The design scour was determined to be 13.3 feet, or elevation 390.8 feet; analyses were performed using a scour depth of 14 feet below the mudline. Rock-socketed drilled shaft foundations are recommended, with one drilled shaft supporting each pier column.

7.3.3 Piers 3 and 4 - Tower Piers

As shown in Table 1, the depth to top of rock at the five borings at Pier 3 ranged from 86.2 to 89.1 feet below river water surface at the time of drilling, with a corresponding top of rock elevation ranging from 330.4 to 333.2 feet. The overburden depth above top of rock ranged from 45.3 to 48.6 feet. The design scour depth at Pier 3 is 40.7 feet, leaving less than 8 feet of overburden soils for the design scour condition.

The depth to top of rock at the five borings at Pier 4 ranged from 79.5 to 84.2 feet below river water surface, with a corresponding top of rock elevation ranged from 334.1 to 340.4 feet. The overburden depth above top of rock ranged from 38.5 to 42.6 feet. The design scour depth at Pier 4 is 42.9 feet, so essentially all soil is anticipated to be removed by scour during the 100-year storm event.

Analyses for Piers 3 and 4 were performed with a preliminary design scour depth of 40 feet.

7.3.4 Pier 5 - Indiana Anchor Pier

As shown in Table 1, the depth to top of rock at the three borings at Pier 5 ranged from 11.0 to 27.0 feet below ground surface or river water surface. The corresponding top of rock elevation at the borings ranged from 392.4 to 423.9 feet. The top of rock rises from north to south along the line of the pier, because the south end of the pier is closer to the steep Indiana bank of the Ohio River. The south boring was offset to the river bank due to utility conflicts, and therefore top of rock at the southernmost foundation location is likely lower than top of rock elevation 423.9 feet as encountered at the boring location. Analyses were performed using a preliminary design scour depth of 17 feet below the mudline; subsequently, the design scour was determined to be 19.7 feet, or elevation 395.5 feet. Rock-socketed drilled shaft foundations are recommended, with one drilled shaft supporting each pier column.

7.3.5 Indiana Abutment

As shown in Table 1, the depth to top of rock or auger refusal at the five borings and eight auger probes at the Indiana Abutment and wing wall ranged from 1.2 to 6.9 feet below ground surface. The top of rock elevation or refusal elevation ranged from 488.7 to 492.0 feet. A spread footing foundation bearing on rock is recommended for the abutment, and the MSE wall wing walls should also bear on rock. The design bearing elevation of the abutment foundation is approximately elevation 484.0 feet. The south and north MSE wing walls have design top of leveling pad elevations of about 485.0 and 490.0, respectively.

7.4 Foundation Analyses, Drilled Shafts

For the tower piers, a total of eighteen drilled shafts are planned at each pier, arranged as an elliptical outer ring of twelve drilled shafts and two rows of three drilled shafts within the ellipse. For Piers 1, 2 and 5, three individual drilled shafts are planned per pier. The layouts of the drilled shafts at each pier are shown on the Preliminary Design Plans dated December 2007.

Analyses for axial loads in bearing, axial loads in uplift, and lateral loads for the drilled shafts at Piers 1 through 5 are discussed in the following subsections. The drilled shafts will be constructed with permanent steel casing to top of rock. In the analyses performed for this report, the permanent steel casing has been included in the drilled shaft section above top of rock.

Two alternative shaft diameters are under consideration, including:

- 8'-6" diameter shaft (O.D. of steel casing) with 8'-0" diameter rock socket
- 8'-0" diameter shaft (O.D. of steel casing) with 7'-6" diameter rock socket

The project is being designed in accordance with the AASHTO LRFD Bridge Design Specifications, 2007. The preliminary design loads on the drilled shafts were established by PB's structural engineers using the structural analysis program LARSA 4D v7.0. The maximum factored shaft head demands were determined using LARSA based on the worst case from the LRFD load combinations for Strength I through V limit states and the Extreme Event I (seismic) limit state. For geotechnical analyses, a range was applied to the preliminary design maximum loads, in consideration of the preliminary stage of the design.

The highest compression loads, up to about 17,000 kips per drilled shaft, are anticipated at the two tower piers, with significant compression loads also anticipated on drilled shafts at Pier 1 (Kentucky Transition Pier). Uplift loads of up to about 2,700 kips are anticipated at the tower piers, with minor uplift loads (up to 200 kips) also anticipated at Pier 2 (Kentucky Anchor Pier). The maximum compression on the shafts in the main tower foundations is anticipated under the Strength IV Limit State (Dead Load + Water + Wind + Temperature). The maximum uplift on the shafts in the main tower foundation is anticipated under the Strength III Limit State (Dead Load + Water + Wind + Temperature). The ranges of loads evaluated are summarized in Table 10 below:

Table 10. Range of Drilled Shaft Loads for Geotechnical Evaluation

Pier	Location	Axial Load, Per Drilled Shaft, kips		Lateral Load kips	Bending Moment k-ft
		Uplift	Compression		
Pier 1	Kentucky Pier	N/A	10,000 to 13,000	200 to 250	4,000 to 6,000
Pier 2	Kentucky Anchor Pier	100 to 200	2,500 to 3,500	200 to 250	4,000 to 6,000
Pier 3	Kentucky Tower Pier	2,000 to 2,700	13,000 to 17,000	1,000 to 1,500	40,000 to 60,000
Pier 4	Indiana Tower Pier	2,000 to 2,700	13,000 to 17,000	1,000 to 1,500	40,000 to 60,000
Pier 5	Indiana Anchor Pier	N/A	3,500 to 4,500	300 to 500	11,000 to 13,000

Note 1: All values presented in Table 10 are factored loads and bending moments.
Note 2: For Piers 3 and 4, the head of the drilled shaft is assumed to be fixed against rotation. Moments at these pier heads include the effects of horizontal shear.
Note 3: Pier 1 loads were developed for a prior configuration of the bridge structure which included extension of the concrete box girder across Transylvania Road. The final desing loads are anticipated to be lower, and analyses will be updated during final design.

7.4.1 Axial Bearing

Preliminary design loads have been provided by PB structural designers. The range of axial loads is summarized in Table 10.

Axial compression load is assumed to be carried entirely in the bedrock, by combined rock socket side friction and end bearing at the base of the rock socket. The contribution of the overburden soil to drilled shaft axial capacity is neglected, based on considerations of scour potential, as well as strain incompatibility between soil and rock side friction. For the tower piers, group effects have been neglected since the drilled shafts will achieve their full axial capacity in rock.

Preliminary design charts have been developed showing compressive capacity as a function of rock socket length, as presented in Figures 7a, 7b, 8a, 8b, 9a, 9b, 10a, and 10b. The compressive capacity includes both socket friction and end bearing, and has been evaluated for both 7.5-foot and 8.0-foot diameter sockets. The capacities shown on the charts are factored resistances, and include a resistance factor of 0.7 on socket friction and end bearing, corresponding to the case where static load tests (Osterberg load cell tests) are to be conducted. For extreme limit states (earthquake, ice, or vessel impact, etc.), a resistance factor of 1.0 is used. The compressive capacity of the 7.5-foot diameter shafts as a function of shaft length is shown in Figure 7a for Pier 1 and in Figure 8a for Piers 2 through 5, with the Extreme Limit State case shown in Figures 7b and 8b for Pier 1 and Piers 2 through 5, respectively. The compressive capacity of the 8.0-foot diameter shafts as a function of shaft length is shown in Figure 9a for Pier 1 and in Figure 10a for Piers 2 through 5, with the Extreme Limit State case shown in Figures 9b and 10b for Pier 1 and Piers 2 through 5, repsectively. Refer to Appendix H for calculations.

A minimum center-to-center spacing of 2.5 socket diameters should be provided between shafts.

7.4.1.1 Pier 1 - Kentucky Transition Pier

The maximum anticipated factored load of 13,000 kips in compression can be resisted by a drilled shaft with a rock socket length of 27 feet for a 7.5-foot diameter socket, or 25 feet for an 8.0-foot diameter socket. The socket length at the transition pier is governed by the axial compression load since at this location a rock socket is not needed to develop fixity of the drilled shafts to lateral load. Based on the load demands provided by the structural engineers, there are no uplift loads on the drilled shafts at the Kentucky transition pier. Refer to the compressive capacity charts in Appendix H for results of analyses.

7.4.1.2 Pier 2 - Kentucky Anchor Pier

The maximum anticipated factored load of 3,500 kips in compression can be resisted by a drilled shaft with a rock socket length of 5 feet for a 7.5-foot diameter socket, or 4.5 feet for an 8.0-foot diameter socket. The minimum design socket length is 1.5 times the socket diameter, and therefore the 7.5-foot and 8.0-foot diameter sockets must have minimum socket lengths of 11.25 and 12.0 feet, respectively. Lateral loads may govern the required minimum rock socket length in final design. Uplift load demand at the Kentucky anchor pier is small and will not govern the design socket length. Refer to the compressive capacity chart in Appendix H for results of analyses.

7.4.1.3 Piers 3 and 4 - Tower Piers

The maximum anticipated factored load of 17,000 kips in compression can be resisted by a drilled shaft with a rock socket length of 32 feet for a 7.5-foot diameter socket, or 28 feet for an 8.0-foot diameter socket. These shaft socket lengths will provide the necessary resistance to anticipated uplift loads. Lateral loads may govern the required minimum rock socket length in final design. Refer to the compressive capacity chart in Appendix H for results of analyses.

7.4.1.4 Pier 5 - Indiana Anchor Pier

The maximum anticipated factored load of 4,500 kips in compression can be resisted by a drilled shaft with a rock socket length of 7 feet for a 7.5-foot diameter socket, or 6 feet for an 8.0-foot diameter socket. The minimum design socket length is 1.5 times the socket diameter, and therefore the 7.5-foot and 8.0-foot diameter sockets must have minimum socket lengths of 11.25 and 12.0 feet, respectively. However, lateral loads may govern the actual required rock socket length. Based on the load demands provided by the structural engineers, there are no uplift loads on the drilled shafts at the Indiana anchor pier. Refer to the compressive capacity chart in Appendix H for results of analyses.

7.4.2 Uplift

As shown in Table 10, drilled shafts at Piers 3 and 4, the tower piers, are subject to maximum design uplift loads per shaft of 2,000 to 2,700 kips, applied at the top of the shaft. Uplift loads occur at some of the drilled shafts at Piers 3 and 4 under Strength I through V and Extreme Event load cases; the maximum uplift values are used for evaluation of required rock socket dimensions. In addition, short term uplift is anticipated during the construction condition, based on uplift on the tremie seal; these hydrostatic uplift conditions will be evaluated further in final design. The drilled shafts at Pier 2, Kentucky Anchor Pier are anticipated to be subject to relatively low uplift loads per shaft up to about 200 kips. No uplift loads are anticipated on drilled shafts at Piers 1 and 5. The loads cited above are factored loads based on LRFD analyses.

Preliminary design charts have been developed showing uplift capacity as a function of rock socket length, as presented in Figures 7c, 7d, 8c, 8d, 9c, 9d, 10c, and 10d. In accordance with AASHTO, the resistance factor used for the socket friction for uplift loading was 0.6, corresponding to the case where static load tests (Osterberg load cell tests) are to be conducted. For extreme limit states (earthquake, ice, or vessel impact, etc.), a resistance factor of 0.8 is used for uplift. The uplift capacity of the 7.5-foot diameter shafts as a function of shaft length is shown in Figure 7c for Pier 1 and in Figure 8c for Piers 2 through 5, with the Extreme Limit State case shown in Figures 7d and 8d for Pier 1 and Piers 2 through 5, respectively. The uplift capacity of the 8.0-foot diameter shafts as a function of shaft length is shown in Figure 9c for Pier 1 and in Figure 10c for Piers 2 through 5, with the Extreme Limit State case shown in Figures 9d and 10d for Pier 1 and Piers 2 through 5, respectively. Refer to Appendix H for calculations and preliminary design charts.

7.4.2.1 Pier 2 - Kentucky Anchor Pier

The maximum anticipated factored load of 200 kips in uplift can be resisted by a drilled shaft with a rock socket length of only about 1 ft for either a 7.5-foot diameter socket, or an 8.0-foot diameter socket. However, minimum design socket lengths and/or design for compression and lateral loads will govern the actual required rock socket length. Refer to the uplift capacity chart in Appendix H for results of analyses.

7.4.2.2 Piers 3 and 4 - Tower Piers

The maximum anticipated factored load of 2,700 kips in uplift can be resisted by a drilled shaft with a rock socket length of 11 feet for a 7.5-foot diameter socket, or 10 feet for an 8.0-foot diameter socket. If the load corresponds to an extreme limit state, the required socket lengths will be smaller (9 feet and 8 feet respectively) due to the higher resistance factor allowed by the code. However, compression and lateral loads will govern the actual required rock socket length in final design. Refer to the uplift capacity chart in Appendix H for results of analyses.

7.4.3 Lateral Capacity

Soil-structure interaction modeling of the bridge is being performed by PB structural engineers using LARSA. The LARSA model requires a depth to fixity of the drilled shafts, and the stiffness properties of the drilled shafts. As part of this geotechnical evaluation, LPILE analyses have been performed in order to estimate an equivalent depth of fixity for use in the LARSA model.

LPILE v5, distributed by ENSOFT, is a program for analysis of a single pile or drilled shaft under lateral loading. The program computes deflection, shear, bending moment, and ground response with respect to depth in nonlinear soils or rock. Several drilled shaft lengths may be automatically checked by the program in order to help the user produce a design with an optimum shaft penetration. Soil and rock behavior is modeled with p-y curves internally generated by the computer program following published recommendations for various types of soils, with special procedures programmed for developing p-y curves for rock.

The section properties used in the LPILE drilled shaft analyses are consistent with those used in the LARSA analyses, for both the cased section above top of rock and the uncased rock socket. The properties are summarized below:

- Casing wall thickness 3/4 inch
- Concrete strength 5,000 psi
- Effective cracked section stiffness 65% of uncracked stiffness

A range of load conditions has been considered, with shear and/or moment, as well as compressive load applied at the drilled shaft head. The load range for factored loads is shown in Table 10. The LPILE analyses were performed both with and without scour, since the LARSA model will be used to evaluate loads in both conditions. For use in modeling the bridge for structural analyses of loads and stresses, loads may be factored in accordance with AASHTO LRFD specifications. For prediction of deflection for comparison to deflection service limits, unfactored service loads should be used.

According to AASHTO, the horizontal geotechnical resistance factor for single shaft or shaft group should be 1.0. In addition, the minimum penetration of the drilled shafts below ground should be such that the fixity is obtained.

One purpose of the calculations is to find the minimum required socket length to provide fixity of the shaft. In these calculations, it is assumed that the fixity is achieved with a certain rock socket length, beyond which increasing the rock socket will have no significant effects on the drilled shaft behavior under lateral loads and bending moment.

Analyses were performed for the cases without scour and with the maximum predicted scour. Either case can be critical under different conditions. In addition, some extreme load cases, such as earthquake, are usually applied with one-half of the maximum scour. The half-scour case was not analyzed for this preliminary stage, as it can be approximated by interpolating between the cases of no scour and maximum scour. The extreme earthquake loading case with half-scour should be evaluated during final design.

For the main tower piers (Piers 3 & 4), the shafts are in large groups arranged in an elliptical pattern in plan. The shaft head is therefore assumed fixed against rotation. For the other piers, the shafts are arranged in a single row in the transverse direction (transverse to the centerline of the bridge); therefore, the shaft head is not fixed in the longitudinal direction, and is assumed free to rotate in LPILE analysis.

Per AASHTO, the group effect for horizontal loading should be modeled with a P-multiplier in the p~y curves. If the shafts are spaced at a center-to-center spacing of 3 times diameter, P-multipliers of 0.7, 0.5 and 0.35 should be applied on the leading row, second row, and other rows of shafts, respectively. For the large shaft groups supporting Piers 3 & 4, most of the shafts are in the 3rd row or higher, therefore, a P-multiplier of 0.35 is applied, conservatively. For the shafts supporting other piers, a P-multiplier of 0.7 was applied as all the shafts are in the first row. Note that these p~multipliers were applied to soil only, as they are not applicable to rock.

LPILE output of load-deflection relationships is presented in Appendix G. LPILE analyses have been performed for each pier location, except that based on similarity of ground conditions and loading, a single analysis was performed for the two tower piers at this preliminary stage of design. The LPILE analyses were performed using the average rock elevation at each pier location. During final design, additional analyses should be performed to evaluate the potential effect of variation in top of rock between boring locations.

Based on the results of lateral load analyses, the minimum required rock socket depths to achieve fixity were determined based on the upper limit lateral loading. At Pier 1, since there will be significant overburden remaining after the maximum design scour, the rock socket is not required for the drilled shaft to achieve fixity. Therefore the axial load requirements will govern the rock socket length. At Pier 2, a minimum 5-foot socket is required to achieve fixity. At Piers 3 and 4, a minimum 25-foot rock socket is required for fixity. At Pier 5, a minimum 15-foot socket is required. See Appendix G for a summary and details of the lateral load analyses. Where the design socket required for lateral load resistance is less than 1.5 times the socket diameter, the socket length will be increased to 1.5 times the socket diameter, in accordance with KYTC practice.

One way to model the drilled shaft foundation in the structural analysis is to model the drilled shafts as columns with an equivalent point of fixity (as used in the LARSA model). Calculations were performed based on results of the lateral load analyses to estimate the equivalent point of fixity at each pier location. The calculation was based on a procedure to find a fixity point of an imaginary column, with the same section modulus (EI) as the drilled shaft, at certain distance below the shaft head that would produce similar lateral deflections at the shaft head under the same sets of lateral loading (shear and/or bending moment). The resulting approximate equivalent points of fixity, for cases evaluated with and without scour, are summarized in Table 11 for preliminary design purpose.

Table 11. Approximate Elevations of Fixity

		Shear kips	Moment k-ft	Approx. Elevation of Fixity, ft	
				Without Scour	Max. Scour
Pier 1	KY Transition Pier	200 to 250	4,000 to 6,000	388	374
Pier 2	KY Anchor Pier	200 to 250	4,000 to 6,000	368	349
Piers 3 & 4	KY & IN Tower Piers	1,000 to 1,500	Fixed Against Rotation	330	323
Pier 5	IN Anchor Pier	300 to 500	11,000 to 13,000	386	385

7.5 Foundation Analyses - Indiana Abutment

A cast-in-place concrete retaining wall is planned for the Indiana Abutment, with a total retained height of approximately 39 feet to finished pavement grade. The design of the retaining wall and abutment foundation must be performed in accordance with AASHTO LRFD specifications.

LRFD analyses for shallow foundations and abutments commence with Service Limit State evaluations. Service limit state settlement considerations are generally not anticipated to control design for footings bearing on sound rock. However, due to the presence of a clay seam as encountered at AC-23, and indications of weathering in geologic mapping observations, it is recommended that the factored bearing resistance at the Service I Limit State (settlement) be limited to 20 ksf. This recommended nominal bearing resistance is based on local experience and engineering judgment, with consideration of the influence of clay seams on foundation settlement. A shallow foundation designed in accordance with this recommended nominal bearing resistance, under the anticipated maximum design load of approximately 100 kips per linear foot, is anticipated to experience settlement of about ½ inch.

Global stability was checked using limit equilibrium methods, considering the potential for the loads of the abutment foundation to create a sliding failure of a rock block. The centerline of the abutment foundation is approximately 35 feet in plan from top of slope, and the toe of the abutment foundation will be at least 25 feet from the top of slope.

Due to the offset distance, the risk of abutment loads resulting in rock slope instability was considered low. However, global stability was checked, using unfactored loads for the abutment. Three potential failure modes were evaluated: planar failure along a high angle joint, toppling, and planar failure along a horizontal clay seam. Based on the stereographic projection analysis, 3 major discontinuity sets were identified including a nearly horizontal bedding plane and 2 vertical joint sets. Considering the orientation of the rock slope with regard to the discontinuity orientations, it was not anticipated that planar failure along the high angle joint would occur because the high angle joint does not daylight to the rock slope. However, it was anticipated that toppling of the high angle joint and sliding along the nearly horizontal seams/layers would be feasible at the Indiana Abutment.

The factor of safety was calculated by a simple toppling analytical technique (Kliche, 1999) and indicated that the factor of safety exceeded the minimum required factor of safety of 1.5 based on the geometry as illustrated in Appendix H.

Due to the clay seams in the limestone layers and interfaces between shale and limestone beds, it is possible that the wedge block formed by high-angle joint and horizontal layer such as the clay seam or the interface could slide toward Upper River Road, depending on the strength of the sliding plane.

For the factor of safety calculation, the shear strength parameters along the sliding plane were estimated to be a 1000-psf cohesion together with 5-degree internal friction angle, based on a literature review (Rock Slope Reference Manual, Publication No. FHWA HI-99-007) and previous experience on similar strata.

Factors of safety were calculated for the following 3 different sliding planes:

- Case I: Along clay seam at approximately elevation 478;
- Case II: Along upper interface of limestone and shale beds at elevation 474.2;
- Case III: Along lower interface of limestone and shale beds at elevation of 459.2.

In each case, 3 different stages of slope conditions were considered, including pre-construction, after construction, and after construction with seismic conditions. Also, 3 different failure plane angles (1, 3, and 6 degrees) were applied to each stage in the factor of safety calculation to reflect slight variations of dips of seams and bedding planes.

The failure planes are likely above the groundwater table. However, because surface runoff temporarily collects in vertical and near vertical joints, a lateral hydrostatic pressure and uplift pressure were considered in the factor of safety calculation. It was conservatively assumed that the vertical joint intersecting the nearly horizontal seam/bedding plane was fully filled with water in the analysis.

Results of the factor of safety calculations are shown in a summary table in Appendix H. As a minimum factor of safety criteria for the slope stability analysis, 1.5 for the abutment (“critical structure”) and 1.3 for the abutment wing walls (”non-critical structure”), and 1.1 for seismic loading conditions are applied.

For seismic loading, a horizontal acceleration of 0.1 g was applied as a pseudo-static seismic force acting on the sliding block. Vertical acceleration was not considered in the analysis, as it normally has only a minimal effect on stability calculations. The horizontal acceleration of 0.1 g corresponds to two-thirds of the peak ground acceleration of 0.15 g as shown in the site-specific response spectra in Figure 6. In eight of the nine seismic cases analyzed, the calculated factors of safety under the 0.1 g horizontal acceleration exceeded the minimum required factor of safety of 1.1 for seismic loading. However, in one case the factor of safety was 1.05, which may still be acceptable as long as the deformation during earthquake is within acceptable limits. In addition, the factor of safety can be transiently lower than 1.0 under the peak acceleration of 0.15 g, in which case some lateral movement of the abutment may occur. During final design, a seismic displacement analysis, such as using the Newmark method, should be performed in final design to estimate the magnitude of the displacement. The displacement is likely to be on the order of a few inches.

It should be noted that further investigation and analysis during final design are necessary to better assess bedding plane orientation and the presence and shear strength of clay seams at the location of the abutment and abutment wing walls. Additional seismic analyses will also be needed to estimate lateral slope displacement due to seismic loading.

Loads on the retaining wall and foundation include the bridge structure loads, earth pressure, traffic surcharge, and dynamic earthquake loading. Sliding and overturning must be considered in design of the footing. For stability against overturning, the resultant of forces on the base of the footing must remain within the middle three-quarters of the footing.

Geotechnical analyses to evaluate nominal bearing resistance and nominal sliding resistance are included in Appendix H. The following parameters were used to design the Indiana Abutment:

- Effective stress friction angle of granular backfill = 32°, unit weight γ = 125 pcf
- Factored bearing resistance on bedrock = 20,000 psf; this incorporates a resistance factor of 0.45. Bearing resistance is based on the angle of internal friction ϕ=22° of the rock formation and cohesion C=2900 psf as shown in Figure 5f; bearing resistance has been reduced for settlement considerations. The factored bearing resistance is based on the service limit state.
- Sliding may be resisted by friction between the rock and concrete, with a nominal sliding resistance comprised of adhesion of 1,900 psf and a concrete-rock friction angle of 15 degrees.

Preliminary design of the shallow foundation for the Indiana Abutment provides 2 feet of cover over the top of the foundation concrete. The footing thickness is estimated to be 4 feet, so the foundation will bear about 6 feet below finished grade. This will provide more than the minimum required frost protection of the bearing surface. The bearing elevation shown on the Preliminary Design Plans is approximately Elevation 484.0 feet.

7.6 MSE Retaining Structure, Indiana Abutment Wing Walls

Mechanically Stabilized Earth (MSE) retaining walls are planned for the abutment wing walls. Each MSE abutment wing wall is about 60 feet long, with a maximum wall height of about 37 feet.

The internal stability of MSE walls is typically made the responsibility of the wall vendor. The MSE wall vendor will be required to perform the wall design using LRFD. Based on the AASHTO LRFD specifications, and the subsurface conditions anticipated at the Indiana abutment, the following design parameters may be used for the MSE wall under LRFD:

- Effective stress friction angle of granular backfill = 32°, unit weight γ = 125 pcf
- Internal backfill for MSE must conform to “Reinforced Fill Material” as specified in Section 805 of the KYTC Standard Specifications for Road and Bridge Construction.
- Factored bearing resistance on bedrock = 8,100 psf; this incorporates a resistance factor of 0.45. Bearing resistance is based on only the angle of internal friction ϕ=22° of the rock formation, neglecting cohesion for conservative calculation.
- Minimum strap length for MSE walls = greater of 8 feet or 0.7H where H = wall height
- Sliding must be checked for sliding along the base of the reinforced fill, and sliding along the foundation rock immediately below the reinforced fill. The factored sliding resistance = 46,600 lb/ft; this incorporates a resistance factor of 0.9 and the angle of friction ϕ=22° which is the lower of friction angles of reinforced fill and foundation soil, conservatively neglecting the bedrock cohesion.

An MSE wall external stability analysis, performed in LRFD, is included in Appendix H. Design parameters were as described for the Indiana Abutment. Allowable bearing, overturning, and sliding are checked and found to be adequate in accordance with the above parameters, as summarized in Table 12 below. Global stability is adequate based on the analysis performed for the abutment as discussed in Section 7.5 above.

Table 12. Summary of MSE Wall Analysis

	Factored Resistance (psf)	Factored Load (psf)
Bearing Capacity	8,100	7,800 (Strength 1b)
Sliding	46,600 (Strength 1a)	39,400 (Strength 1a)
Global Stability	OK based on analysis for Indiana Abutment	

* In the stability check against overturning, the maximum eccentricity (e) is 3.8 (Strength 1a) which is smaller than e_{max} = 6.9.

* Note: In the above analysis, 0.75H of strip length of MSE wall was used, which exceeds the required minimum strip length of 0.7H. The slightly longer strip length of 0.75H was required in order to attain a factored load less than the factored resistance in bearing capacity.

7.7 Fills and Embankments, Indiana Abutment

Approach embankments to the bridge are part of the work of Sections 4 and 6 and are not included in this contract for Section 5. However, backfill of the Indiana abutment and wing walls and construction of the reinforced concrete bridge approach slab is part of the work of this contract under Section 5. Embankment fill side slopes in the area of the retaining wall should be not steeper than 3 horizontal: 1 vertical (3H:1V).

Backfill of the retaining wall at the Indiana Abutment shall be performed in accordance with 603.03.04 of the KYTC Standard Specifications, except that those areas which will be beneath or within a proposed roadway embankment must be backfilled according to Subsection 206.03.03 of the Standard Specifications.

Provisions for drainage should be included in the design of the cast-in-place concrete abutment and the MSE wall. Both the abutment and the MSE wall should be backfilled with material meeting KYTC Structure Backfill, a free-draining granular material, within a 45 degree zone behind the wall. The MSE wall is considered free-draining. For the cast-in-place concrete abutment, a properly filtered perforated wall drain should be provided on top of the foundation. The wall drain should discharge through properly filtered drainage weepholes at the face of the cast-in-place retaining wall. If a single weephole becomes clogged, redundancy is provided through the adjacent weepholes.

8.0 CONSTRUCTION CONSIDERATIONS

This section outlines construction considerations for drilled shaft foundations, spread footing foundations, backfill of the Indiana Abutment, and abutment wing walls. These provisions should be incorporated into the construction specifications for the project.

The construction of the bridge foundation will be in accordance with KYTC Standard Specifications. This includes project elements located within the State of Indiana, including the Indiana anchor pier and Indiana Abutment.

8.1 Drilled Shaft Foundations

The drilled shaft foundations will be constructed with permanent steel casings to top of rock, and with rock sockets advanced below the steel casings.

Selection of the method of construction is the responsibility of the contractor. However, given the highly permeable sand and gravel soils, and the difficulty of seating the casing into the limestone bedrock, it is unlikely that the contractor will be able to achieve a watertight seal at the soil-bedrock interface. Therefore, it is anticipated that the wet construction method will be necessary for construction of the drilled shafts.

It is anticipated that the contractor will advance the casing as the shaft is drilled, with drilling conducted under a head of bentonite or polymer slurry to prevent heave of sands into the casing. The contractor could also elect to vibrate or oscillate the casing into place for all or a portion of its depth.

The soil borings note occasional cobbles, based on field observations during drilling, but did not encounter evidence of potential obstructions which would impede excavation of the drilled shafts. However, in glacial outwash formations, ice-rafted boulders are occasionally present, and the contractor should be prepared to remove obstructions if encountered.

When the casing is seated at top of rock, the rock socket will be advanced below the casing. The use of rock augers will not be feasible in the moderately hard rock present at this site. It is anticipated that the contractor will advance the rock socket using drilled shaft rock core barrels, reverse circulation rock drills, or possibly down-the-hole hammers. To achieve the design rock socket capacity, the socket surface must be rough. The contractor should be required to construct a roughened shaft surface, by attaching teeth to the coring device, or by other means acceptable to the engineer.

Before drilling the rock socket, rock coring should be performed at each drilled shaft location where rock coring was not performed during the design phase of the project. Alternatively, the rock cores can be performed prior to initiating shaft excavation. The purpose of the rock coring is to verify the quality of the rock and to identify the presence and thickness of clay seams or voids within or below the design socket, to verify compliance with the acceptance criteria established for the drilled shafts.

The bottom of the drilled shaft excavation must be flat; steps in the bearing surface, or a sloping bearing surface, will not be acceptable. After construction of the rock socket and acceptance of the bearing conditions by the engineer's field representative, the drilled shaft excavation must be thoroughly cleaned out. Cleanout may be performed by a cleanout bucket, or other methods acceptable to the engineer. Final bottom cleaning should be accomplished with the aid of an airlift. Soil or rock cuttings must not be left in place at the bottom of the drilled shaft. At the time of concrete placement, a minimum of 50 percent of the base of the shaft shall have less than ½ inch of sediment, and sediment on the base of the shaft shall not be greater than 1-1/2 inches anywhere on the base of the shaft. The bottom of shaft conditions should be checked with an underwater camera equipped with a sediment measurement gage, such as the Mini-Shaft Inspection Device (Mini-SID) manufactured by GPE, Inc, Gainesville, FL.

Reinforcing steel and concrete must be placed within 36 hours of beginning of the drilled shaft rock socket, to limit the potential for deterioration of the rock socket capacity through slaking of shale.

In the wet method of construction, concrete is placed by tremie methods or pumping. Concrete placement must be in accordance with Section 601of the KYTC Standard Specifications. Concrete slump should be 6.5 to 9.5 inches for tremie placement and not less than 4 inches for the full duration of concrete placement. Proper concrete placement methods must be used to prevent mixing of slurry into the concrete. A plug or valve is required to prevent contamination of the concrete in the tremie pipe or pump discharge pipe. The pump or tremie discharge point must remain at least 10 feet below top of concrete at all times during placement. Concrete placement must be continuous without interruption.

Integrity testing of all drilled shafts will be required by crosshole sonic logging (CSL) methods. Crosshole sonic logging uses water-filled access tubes installed on the reinforcing steel cage. After the concrete has achieved its initial strength, a cable-mounted ultrasonic signal transmitter and multiple cable-mounted receivers are placed in the tubes, and testing is performed. The signal is sent from the transmitter tube and travel time and amplitude are measured in the receiver tubes, with testing performed for the full length of the shaft. Testing is performed between all pairs of adjacent tubes as well as between opposite tubes in the shaft. Anomalies in the presence or strength of the signal may represent potential defects in the drilled shaft concrete.

One CSL tube should be provided for each foot of shaft diameter. CSL testing will be required at all shaft locations. If anomalies are observed in the CSL tests, and in the opinion of the engineer these anomalies constitute potential defects in the shafts, then the engineer may require additional CSL testing and/or coring of the drilled shafts to investigate the anomalies. If defects are determined to exist, the engineer will review the load carrying capacity of the drilled shaft and determine remedial measures, if required, which may include repair of the shaft defects and construction of replacement shafts.

Tolerances for drilled shaft location and plumbness will be required to meet the following criteria:

- Plan location (at top of shaft) +/- 3 inches
- Plumbness +/- ¼ inch per foot of depth
- Top of shaft elevation + 3 inch to -3 inches
- Top of reinforcing steel cage +6 inches to – 3 inches

For the tower piers, the drilled shafts will be constructed by barge-mounted drilled shaft equipment. At anchor piers, barge-mounted equipment and/or a temporary trestle is anticipated. As shown on Figure 2, at the Kentucky Anchor Pier, the northernmost drilled shaft is located immediately adjacent to the bank, where water depth is insufficient for barge-mounted equipment. Therefore, a temporary trestle may be used at this location. The remaining two Kentucky Anchor Pier drilled shafts can be constructed using barge-mounted equipment. At the Indiana Anchor Pier, the southernmost drilled shaft location is on-shore, but is located on the bank of Upper River Road, and a working platform will be needed to facilitate construction. The center drilled shaft location is in a shallow-water area approximately 10 feet from edge of bank, requiring a temporary trestle for construction. The northernmost drilled shaft location can be installed by barge-mounted equipment. Water depth in this area is shallow, and dredging could be needed for the barge access.

Where temporary trestles or working platforms are required, the contractor should be required to submit shop drawings and calculations prepared by a professional engineer registered in the Commonwealth of Kentucky or the State of Indiana, as applicable to the location.

8.2 Drilled Shaft Load Testing

Considering the size and high load bearing capacity of the drilled shafts used for Piers 1 through 5, and the uncertainty regarding rock socket friction and end bearing resistance, it is recommended that a load test program be performed at the start of construction to verify the design rock socket lengths for the required load demand on these shafts. The resistance factors used in the preliminary design analyses were based on the implementation of a load test program.

The recommended test program includes Osterberg load cell tests at dedicated (non-production) drilled shafts. A minimum of three load tests will be required, including one at each of the main piers in the river, and one at the Kentucky transition pier (Pier 1). Testing at the main piers is recommended considering the number of drilled shafts at each of these locations, and the high load demand on these shafts. Testing is recommended at Pier 1 because the load demand at the Pier 1 drilled shafts are considerably greater than at the anchor piers. Also, the rock bearing stratum there contains numerous clay seams, not observed at the other foundation locations, that may influence the available shaft friction and end bearing resistance.

The depths and locations of low SDI zones encountered in borings, as discussed in Section 5.3.6 of this report, will be considered in finalizing the locations of the load tests. At least one load test shaft will be located near a boring that had zones of low SDI values. This will allow evaluation of the potential influence of degradation of the thin (0.4 feet or less) low SDI zones during construction, to assess whether there is a reduction to the shaft friction resistance.

In the Osterberg load test the Osterberg load cells would be positioned near the base of the rock socket. During the test, the load cells are hydraulically activated to apply an upward load to determine the friction resistance along the socket, and a downward load on the socket base to determine the end bearing resistance. The position of the O-cells and the length of the socket will be sized in an attempt to obtain both the nominal friction and end bearing resistance values. However, the test will be limited to a load equivalent to the nominal friction resistance, the nominal end bearing resistance, or the maximum load capacity of the Osterberg load cells, whichever occurs first. During the final design phase of the project, consideration will be given to testing a reduced diameter rock socket to reduce the cost of the load test program and to better balance the friction and end bearing resistance, if necessary. Whether or not reduced diameter test shafts are used, the specifications will require the use of the same type of excavation equipment and same shaft installation procedures that will be used for the production shafts. Also, all test shafts will be instrumented to determine socket and base displacement versus load, and to determine the unit friction resistance along the length of the socket. CSL testing will be required in all of the test shafts to assess the structural integrity of the completed shafts.

The initial test shaft on water and the test shaft on land will also serve as technique shafts, for the contractor to demonstrate the proposed method of drilled shaft construction.

For construction contract budget management, KYTC prefers to have additional drilled shaft quantities in the budget, in the event that shaft lengths need to be increased based on load test results. Therefore, at locations where the axial capacity governs the design tip elevations, the project plans will show the tip elevations 5 feet lower than the elevation determined based on the analyses. If load tests verify the drilled shaft capacities at the design elevation, the load test results can then be used to shorten the shafts relative to the plan quantities.

8.3 Spread Footing Foundations

Excavation and foundation preparation for the Indiana Abutment must be performed in accordance with Section 603 of the KYTC Standard Specifications. Based on the auger probes and rock core borings performed for the abutment, 6 to 8 feet of rock excavation is expected to be necessary for foundation construction. The rock should not be expected to be rippable; controlled blasting or mechanical excavation will be necessary. To limit overbreak and reduce the likelihood of fracturing rock outside the excavation limits, pre-splitting should be required.

The abutment area is lightly developed, so controlled blasting is unlikely to pose a risk of vibration damage to nearby structures or facilities. Vibration monitoring should be performed at the closest structure or existing roadway. Specification limits may be established based on the U.S. Bureau of Mines (USBM) R.I. 8507 criterion. The USBM criterion specifies a maximum peak particle velocity (ppv) of 2 inches per second (ips) at the ground line of the closest structure at frequencies of 40 Hz or greater, with lower limits on ppv at lower frequencies. Blasting should be conducted in accordance with federal, state and local regulations.

The bearing surface of the abutment foundation must be level or stepped with step heights not exceeding 12 inches and an average slope of the stepped surface not greater than 1.5 horizontal : 1 vertical (1.5H:1V). The integrity of the bearing surface shall be checked visually, supplemented by probe holes extending to a depth of 10 feet below the bearing surface, spaced not more than 50 feet on center. The probe holes may be drilled with an airtrak drill, and should be checked for evidence of voids by the use of a hooked rod.

Based on the SDI values for the shale at the Indiana Abutment, the rock here generally is not considered durable. Therefore, after acceptance of the bearing surface by the engineer's field representative, the contractor should be required to place a minimum 3 inch thick lean concrete mud mat to protect the bearing surface.

Groundwater is not expected to be present within the excavation for the Indiana abutment, but the contractor should be prepared to remove surface water and perched water at the soil-bedrock interface.

Excavation must be performed in accordance with all applicable federal, state and local standards, including OSHA 29CFR Part 1926 – Excavations. Excavation safety is the responsibility of the contractor.

8.4 Backfill, Indiana Abutment Retaining Structure

Backfill at the Indiana Abutment and wing wall shall be performed in accordance with 603.03.04 of the KYTC Standard Specifications, except that those areas which will be beneath or within a proposed roadway embankment must be backfilled according to Subsection 206.03.03 of the Standard Specifications.

Where granular fill will be placed against undisturbed or fill materials comprised of clay, a geotextile filter fabric should be provided. The purpose of the geotextile is to reduce migration of fines into the granular medium.

Care should be taken not to overcompact backfill behind retaining walls. Existing surfaces to receive fill should be stripped and benched at an average slope not steeper than 2 horizontal to 1 vertical (2H:1V), with step heights not greater than 1 foot.

REFERENCES AND DATA SOURCES:

AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

Geologic Map of Parts of the Jeffersonville, New Albany, and Charlestown Quadrangles, Kentucky-Indiana. Kentucky Geologic Survey. 1974.

Geologic Map of the 1° X 2° Louisville Quadrangle, Indiana, Showing Bedrock and Unconsolidated Deposits. Indiana Geologic Survey. 1972

Geologic Map of the Anchorage Quadrangle, Jefferson and Oldham Counties, Kentucky. Kentucky Geologic Survey. 1971.

Web Soil Survey 2.0. National Cooperative Soil Survey. 2007

Source Zones, Recurrence Rates, and Time Histories for Earthquakes Affecting Kentucky, Publication No. KTC-96-4. Kentucky Transportation Center. 1996

Rock Slopes Reference Manual, Publication No. FHWA HI-99-007. Federal Highway Administration. 1999

Report of Investigation 8507, U.S. Bureau of Mines 1980

Rock Slope Stability, Society for Mining, Metallurgy, and Exploration, Inc., Charles A. Kliche, 1999.

Figure 10. Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 8.0-foot Diameter Shafts

Figure 10a Compressive Resistance vs. Socket Length
Figure 10b Compressive Resistance vs. Socket Length – Extreme Limit States
Figure 10c Uplift Resistance vs. Socket Length
Figure 10d Uplift Resistance vs. Socket Length – Extreme Limit States

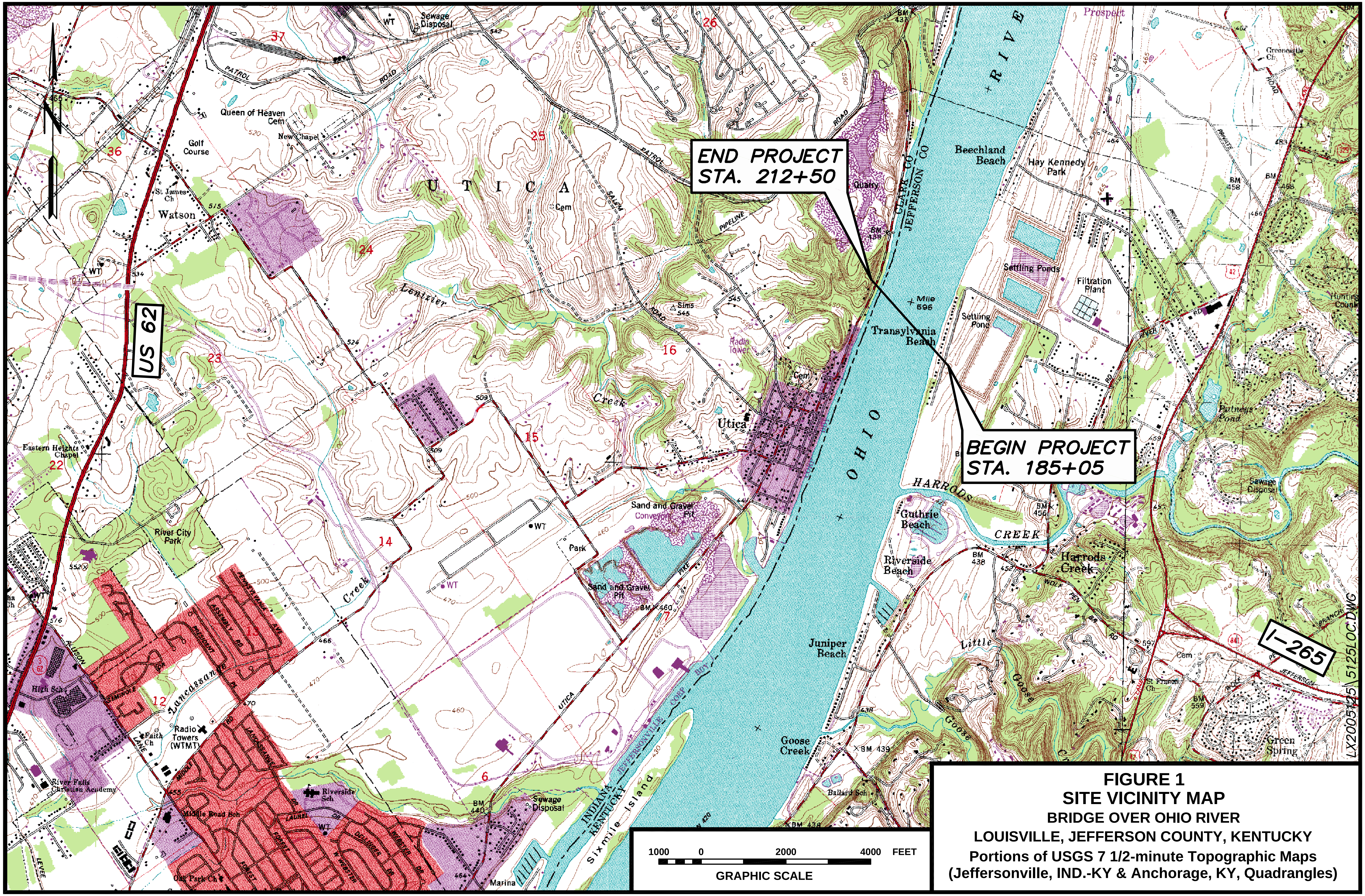
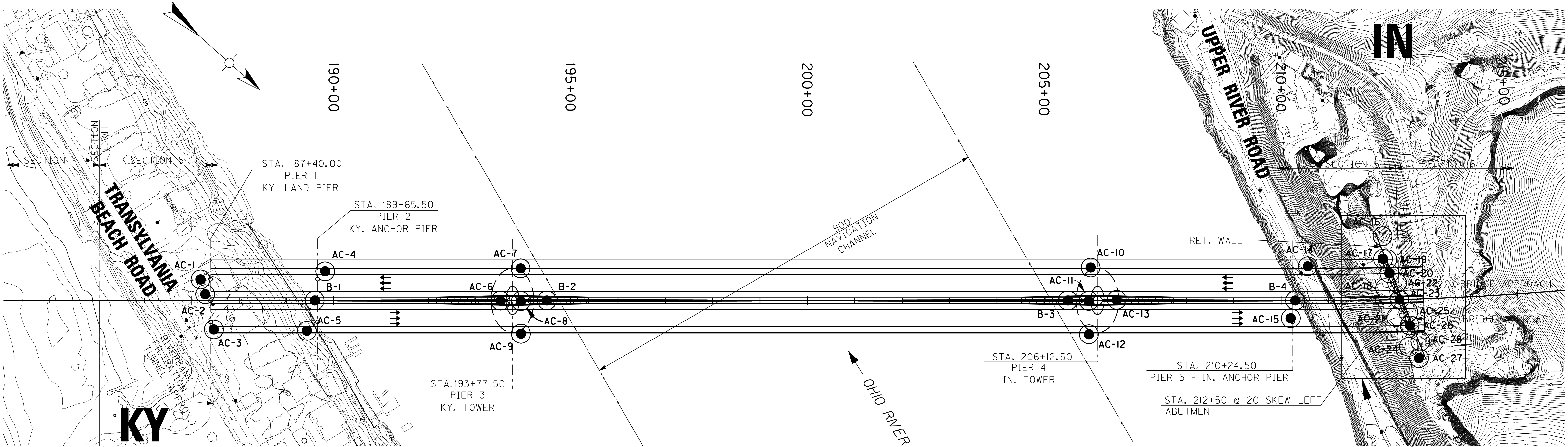
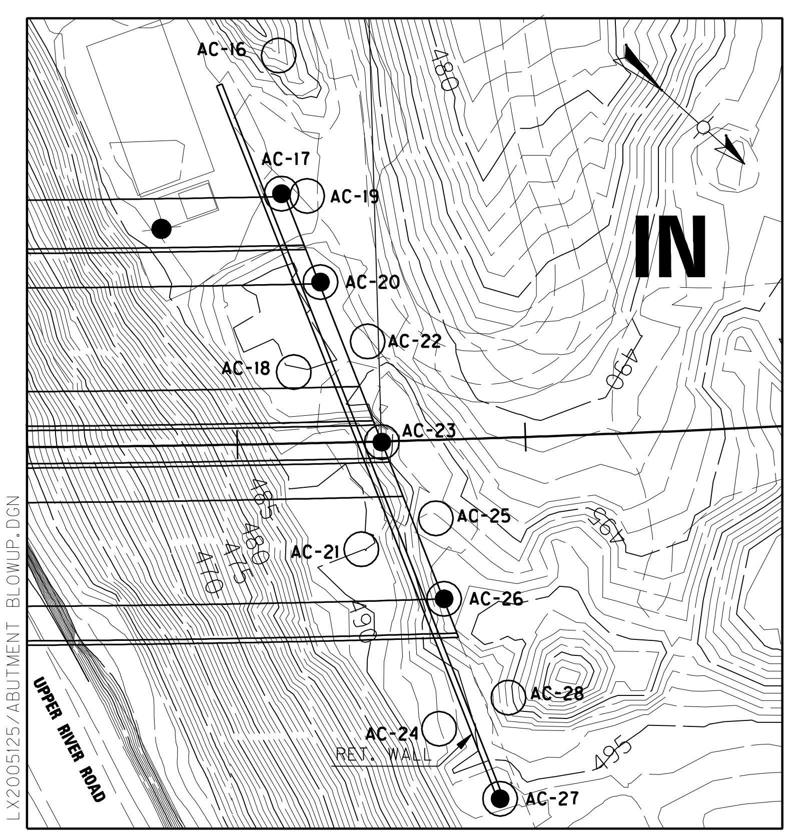


FIGURE 1
SITE VICINITY MAP
BRIDGE OVER OHIO RIVER
LOUISVILLE, JEFFERSON COUNTY, KENTUCKY
Portions of USGS 7 1/2-minute Topographic Maps
(Jeffersonville, IND.-KY & Anchorage, KY, Quadrangles)

SUBSURFACE DATA





INSET A

SCALE: 1"=30'

100 0 100 200 300

GRAPHIC SCALE IN FEET

FIGURE 2

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	KJS/TD/OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I-265** CROSSING **OHIO RIVER**

BORING LOCATION PLANS

SCALE: 1" = 100'

ITEM NUMBER

5-118.00

PREPARED BY
MSM ENGINEERS

SHEET NO.

DRAWING NO.

LX2005125/5125LAYO.DGN

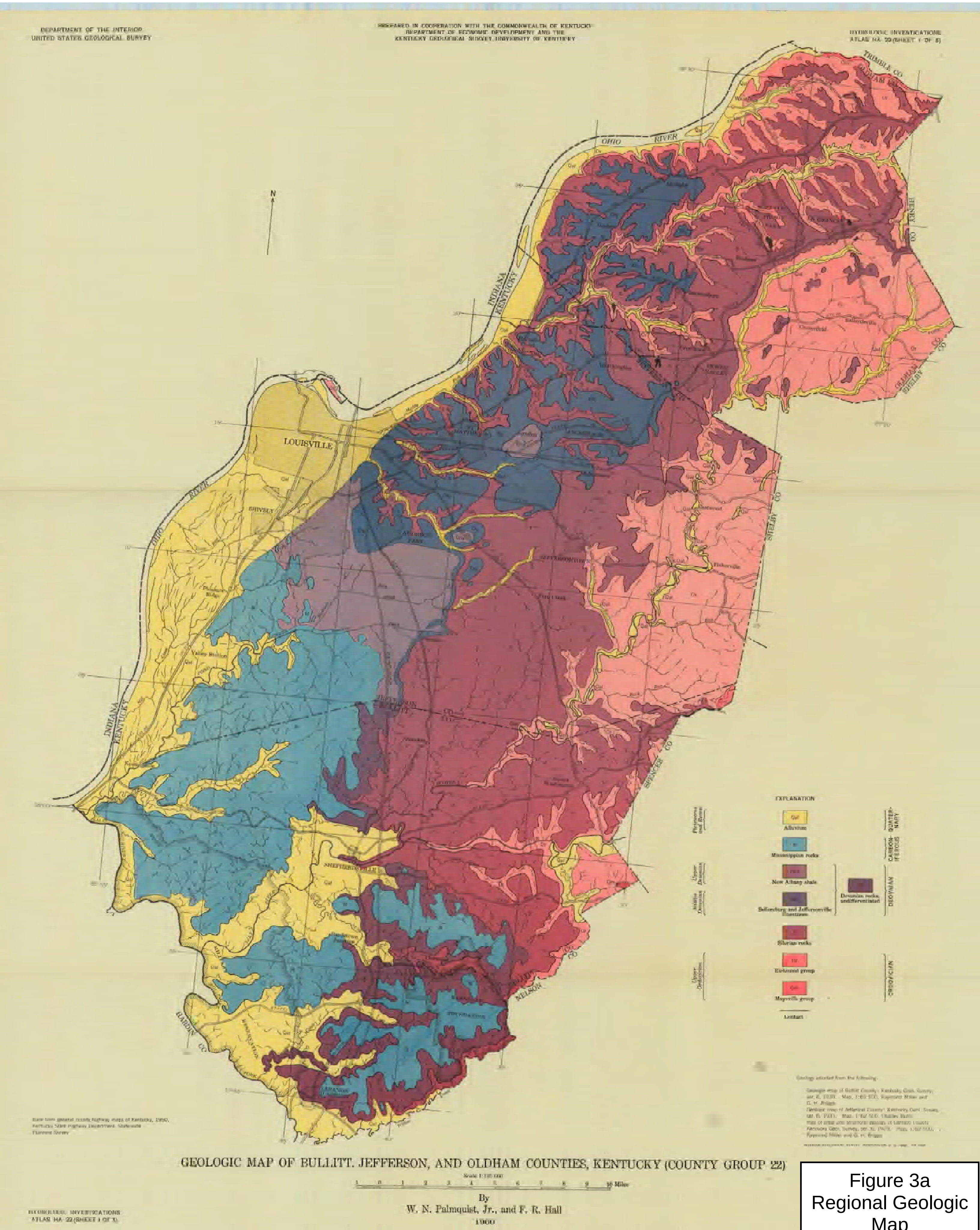
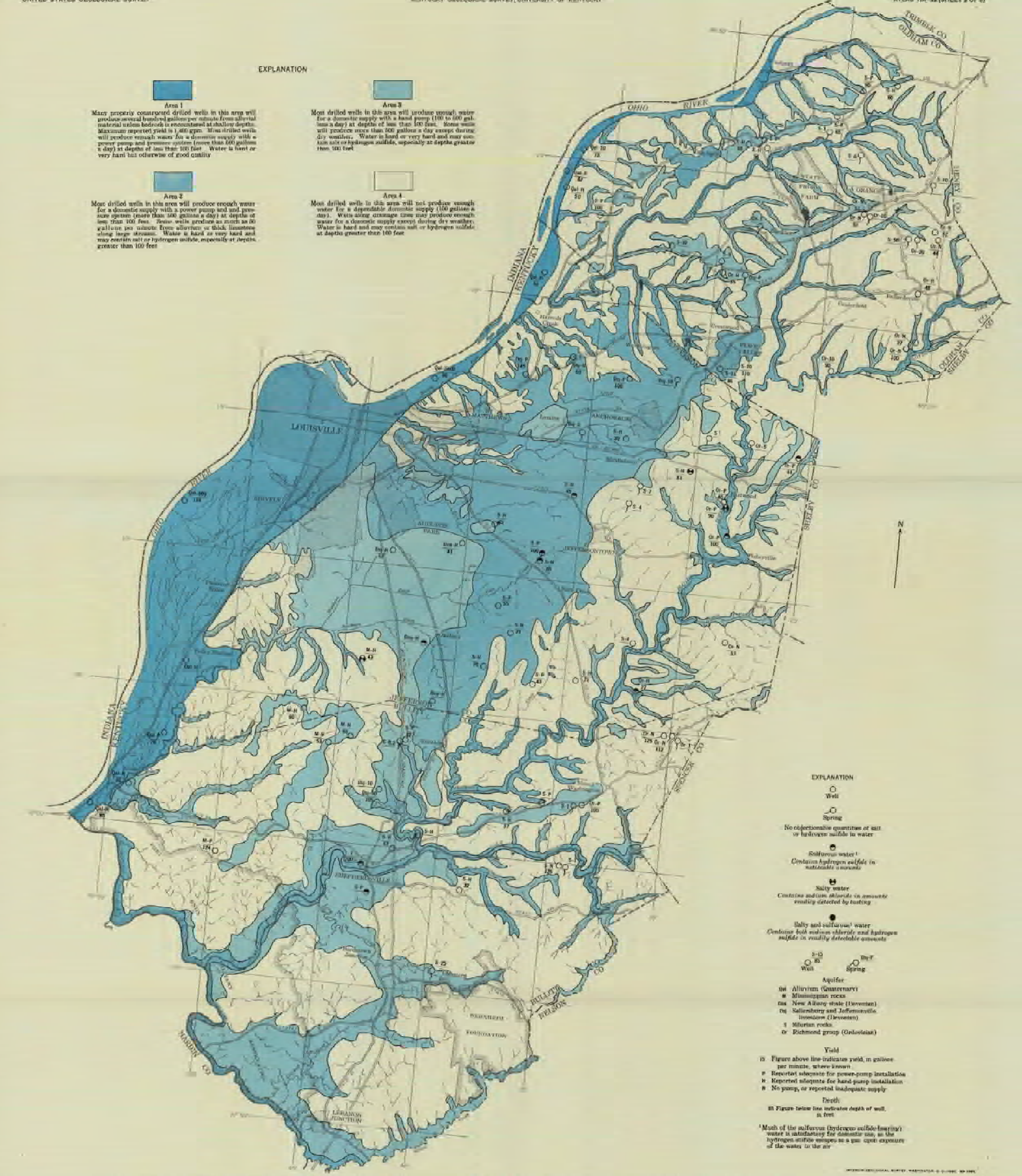


Figure 3a
Regional Geologic Map

DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY

PREPARED IN COOPERATION WITH THE COMMONWEALTH OF KENTUCKY
DEPARTMENT OF ECONOMIC DEVELOPMENT AND THE
KENTUCKY GEOLOGICAL SURVEY, UNIVERSITY OF KENTUCKY

HYDROLOGIC INVESTIGATIONS
ATLAS HA-22 (SHEET 2 OF 3)



AVAILABILITY OF GROUND WATER IN BULLITT, JEFFERSON, AND OLDHAM COUNTIES, KENTUCKY (COUNTY GROUP 22)

Scale 1:125 000

1 0 1 2 3 4 5 6 7 8 9 10 Miles

By

W. N. Palmquist, Jr., and F. R. Hall

1960

HYDROLOGIC INVESTIGATIONS
ATLAS HA-22 (SHEET 2 OF 3)

Figure 3b
Regional Geologic Map

DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY

PREPARED IN COOPERATION WITH THE COMMONWEALTH OF KENTUCKY
DEPARTMENT OF ECONOMIC DEVELOPMENT AND THE
KENTUCKY GEOLOGICAL SURVEY, UNIVERSITY OF KENTUCKY

SYSTEM	SERIES	GROUP	FORMATION	THICKNESS, IN FEET	SECTION	LITHOLOGY	TOPOGRAPHY	HYDROLOGY
QUATERNARY	PLEISTOCENE AND RECENT		ALLUVIUM	0-130		Soil, clay, silt, and fine sand, 5 to 40 feet thick, overlying sand and gravel with clay lenses in the Ohio River valley. Thin deposits of clay, silt, and fine sand with scattered deposits of gravel in tributary-stream valleys.	Flood plains and terraces, as much as 6 miles wide, in the Ohio River valley; broad flat areas in the valleys of the Salt River and large tributaries.	Yields 200 to 500 gpm (gallons per minute) to most wells that penetrate the full thickness of alluvium in the Ohio Valley; yields more than 1,000 gpm to large-diameter wells; yields 100 to 500 gpd to wells in tributary-stream valleys, and may yield more than 500 gpd where gravel is present. Water is hard.
CARBONIFEROUS MISSISSIPPIAN	UPPER MISSISSIPPIAN		SALEM LIMESTONE	50±		Fine-grained siliceous and argillaceous limestone and shale with geodes and chert.	Tops of some of the high ridges and knobs in western Bullitt and southwestern Jefferson Counties, and broad, flat valleys in some places.	Yield 100 to 500 gpd to drilled wells on broad uplands, but almost no water on narrow ridges; yield water to small springs in edges of escarpment. Water is hard but otherwise of good quality.
			WARSAW LIMESTONE	40±				
	LOWER MISSISSIPPIAN	BORDEN[1]	MULDRAUGH FORMATION[2]	75-95		Hard bedded fine-grained siliceous limestone; argillaceous and crinoidal limestone; and calcareous and argillaceous siltstone with drab to black shale, small geodes, and bands and lenses of chert.		
			FLOYDS KNOB FORMATION[1]	1-9		Brown siliceous, oolitic, or crinoidal limestone capped by streak or layer of greenish-black glauconitic silt or clay.		
			BRODHEAD FORMATION[2]	200-220		Argillaceous silty shale with calcium carbonate concretions, grading upward to massive argillaceous shaly siltstone and occasional beds of limestone. Siliceous to crinoidal limestone at top in southern Bullitt County.	Main part of the Mississippian escarpment and many outlying knobs. Resistant rocks of the Muldraugh formation cap the escarpment and larger knobs. The Brodhead formation caps and forms cliffs in the upper part of many knobs in Bullitt County. The New Providence formation underlies the lower dissected slopes of the knobs and escarpment.	Yields 100 to 500 gpd to wells in valley bottoms; may yield more than 300 gpd where thick siltstone beds occur at and below stream level; yields almost no water to wells on hills; yields water to small springs in the limestone and siltstone beds. Water from the shale is soft; from the siltstone, hard; and from the limestone, very hard. At shallow depths below stream level, water may contain salt, sulfate, or iron. The silty shale and siltstone are favorable for dug wells, common in this area. Most dug wells yield less than 500 gpd and many yield very little or go dry in late summer and early fall.
			NEW PROVIDENCE FORMATION[1]	175-205		Argillaceous shale or claystone with ferruginous calcareous concretions and lenses and ferruginous limestone patches and lenses. Fine-grained sandstone layers with interbedded shale at the top.		

Figure 3C
Regional Geologic Section

DEVONIAN	UPPER DEVONIAN		NEW ALBANY SHALE	100±	Black fissile slightly calcareous carbonaceous shale, pyrite scattered throughout and in a layer at the base, and several thin sandstone and shale layers.	Broad, flat areas in southwest-central Jefferson and central Bullitt Counties; gentle lower slopes of much of the Mississippian escarpment and the knobs.	Yields 100 to 500 gpd to shallow drilled wells in broad, flat areas, but almost no water to drilled wells on hillsides; yields water to small springs and dug wells. Water is hard and from depths greater than about 50 feet may contain hydrogen sulfide and iron.
	MIDDLE DEVONIAN		SELLERSBURG LIMESTONE	0-22	Thick-bedded finely to coarsely crystalline argillaceous magnesian limestone, small black phosphatic nodules in upper part.	Rolling upland with sinkholes and underground drainage in northern Jefferson County and broad ridges in western Oldham County. The Jeffersonville thins toward the south and is not present in Bullitt County.	Yield more than 500 gpd to drilled wells in broad, flat valleys or along streams on the upland; yield water to springs. Water is hard.
			JEFFERSON-VILLE LIMESTONE	0-30	Medium- to thick-bedded medium-crystalline to coarsely crystalline limestone, siliceous and cherty in part.		
SILURIAN			LOUISVILLE LIMESTONE	45-100	Thick-bedded fine-grained limestone, magnesian or siliceous in part.	Moderately rolling upland with some sinkholes and underground drainage in south-central Jefferson and north-central Bullitt Counties; broad ridges in south-central Oldham and northeastern Bullitt Counties. Cliffs and ledges in valley sides.	Yields more than 500 gpd to wells drilled in valley bottoms or along streams on broad uplands; yields as much as 50 gpm in places; yields water to springs at contact with underlying Waldron shale. Water is hard and may contain salt or hydrogen sulfide below stream level.
			WALDRON SHALE	10±	Green-gray nonfissile calcareous magnesian siliceous shale.	Slopes between limestone ledges on hillsides; erosion undermines overlying Louisville limestone.	Yields almost no water to wells or springs. Holds up water in the overlying Louisville limestone and prevents recharge to the underlying Laurel dolomite.
			LAUREL DOLOMITE	40±	Thin- to medium-bedded fine- to medium-grained dolomitic limestone.	Ledges and cliffs along streams.	Yields 100 to 500 gpd to wells where it occurs along streams, but almost no water to wells on hillsides; yields water to springs. Water is hard.
			OSGOOD FORMATION	30	Coarse lumpy or fissile calcareous and magnesian shale with prominent fine-grained limestone beds at base and about 3 feet below top.	Slopes between limestone ledges.	Yields water to small springs from limestone beds.
			BRASSFIELD LIMESTONE	4±	Medium-bedded pink to brown coarsely crystalline limestone.	Ledges on slopes and tops of small cliffs of underlying Saluda limestone.	Yields water to springs. Water is hard.
ORDOVICIAN	UPPER ORDOVICIAN	RICHMOND	SALUDA LIMESTONE	30-40	Thick-bedded sandy magnesian limestone in upper part, and coarse lumpy mudstone with thin beds of bluish-gray fine-grained limestone at base.	Cliffs along streams and ledges in hillsides; tops of some low, flat ridges.	Yields 100 to 500 gpd to wells in valley bottoms and on broad ridges, but almost no water to wells on hillsides; yields water to small springs. Water is hard.
			LIBERTY FORMATION	35-50	Coarse bluish-gray shale with thin layers of bluish-gray fine-grained limestone.	Moderately dissected upland areas; moderately steep slopes where shale predominates and less steep slopes where limestone predominates. Steep slopes along large streams and cliffs, in places. Solutional features evident where thick limestone beds underlie streams.	Yield 100 to 500 gpd to wells in large stream valleys, and more where thick limestone is present; yield almost no water to wells on hillsides and ridges; yield water to small springs. Water is hard.
			WAYNES-VILLE LIMESTONE	40±	Thick-bedded green nongranular argillaceous limestone with shale partings, and 10-foot bed of green shale in lower part.		
			ARNHEIM FORMATION	80-100	Thin alternating layers of blue lumpy or rubbly, locally crossbedded, argillaceous limestone and clay shale.		
		MAYS-VILLE	MC MILLAN FORMATION	20 EXP.	Argillaceous limestone and shale.	Stream valleys on east edge of area.	Yields 100 to 500 gpd. Water is hard.

INTERIOR—GEOLOGICAL SURVEY, WASHINGTON, D. C.—1960 MR 3565

[1] As used by Stockdale (1939). [2] Of Stockdale (1939).

GENERALIZED COLUMNAR SECTION AND WATER-BEARING CHARACTER OF THE ROCKS IN BULLITT, JEFFERSON, AND OLDHAM COUNTIES, KENTUCKY (COUNTY GROUP 22)

By
W. N. Palmquist, Jr., and F. R. Hall

1960

HYDROLOGIC INVESTIGATIONS
ATLAS HA-22 (SHEET 3 OF 3)

Figure 3C - Continued
Regional Geologic Section

SUBSURFACE DATA

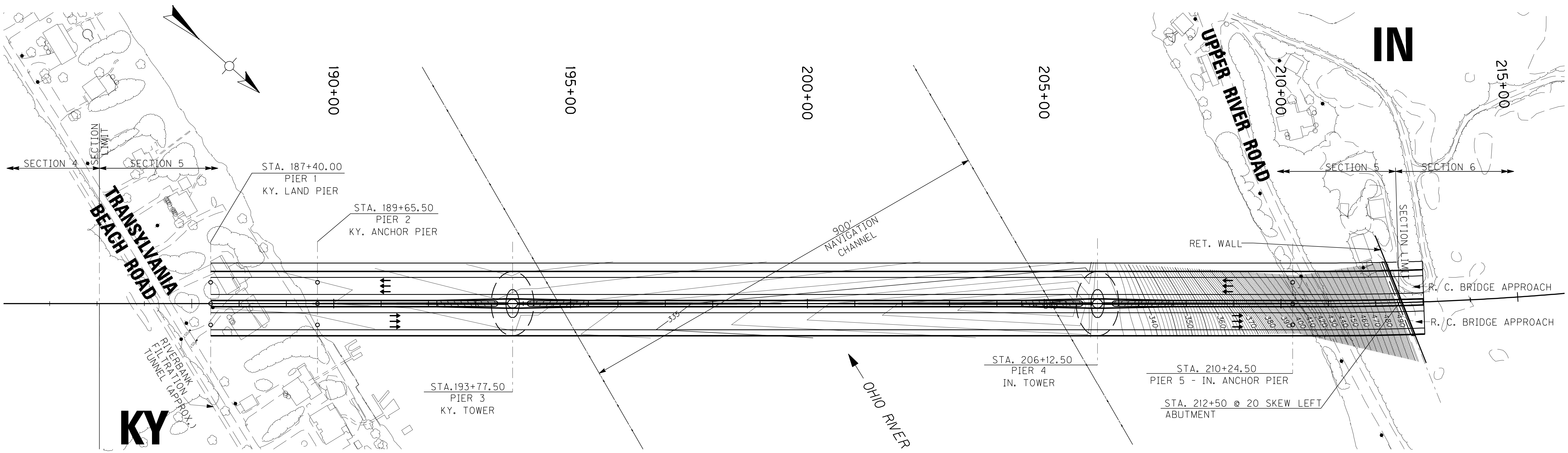


FIGURE 4

SCALE: 1" = 100'

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	KJS/TD

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I–265** CROSSING **OHIO RIVER**

BEDROCK SURFACE CONTOURS

PREPARED BY
MSM ENGINEERS

SHEET NO.
DRAWING NO.

ITEM NUMBER
5–118.00

LX2005125/5125FIG4.DGN

GENERAL SOIL AND BEDROCK PROFILE LEGEND SHEET

I-265 Bridge over the Ohio River

SUMMARY OF PARAMETERS DEVELOPED FOR SOIL PROFILES

Parameter	Units	Description
γ_t	lb/ft^3	Total Unit Weight[1,2]
γ_e	lb/ft^3	Effective Unit Weight[1]
q_U	lb/ft^2	Uniaxial Compressive Strength (soil)
q_U	ton/ft^2	Uniaxial Compressive Strength (rock)
C_U	lb/ft^2	Undrained Shear Strength
SDI	%	Slake Durability Index (Shale only)
ϕ	(°)	Angle of Internal Friction[1,3]
K_S	lb/in^3	Soil Secant Modulus - Static (computer program LPILE^{PLUS}4.0)
K_C	lb/in^3	Soil Secant Modulus - Cyclic (computer program LPILE^{PLUS}4.0)
D_{50}	mm	Particle Diameter Corresponding to 50% Finer
D_{95}	mm	Particle Diameter Corresponding to 95% Finer

1 Values are averages for the horizon where more than one Standard Penetration Test (SPT) was performed.

2 Values of γ_t are capped at a maximum of 133 lb/ft^3 for soil.

3 Values of ϕ are capped at a maximum of 38° for soil.

Figure 5a

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 1 - STA 187+40, CL
Borings AC-1, 2, 3

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Classification)	Parameters	
434.0	0.0			
		Lean Clay (CL)	γ_t (lb/ft^3) = 121	K_S (lb/in^3) = 100
418.3	15.7 ▽		γ_e (lb/ft^3) = 59	D_{50}(mm) = 0.03
			q_U (lb/ft^2) = 1784	D_{95}(mm) = 0.19
			C_U (lb/ft^2) = 892	
409.0	25.0			
		Sand with Silt (SM, SW-SM, and SP-SM)	γ_e (lb/ft^3) = 55	D_{50}(mm) = 0.90
			ϕ' (°) = 32.5	D_{95}(mm) = 20
			K_S (lb/in^3) = 60	
379.0	55.0			
		Sand (SW-SM, SW and SP-SM)	γ_e (lb/ft^3) = 64	D_{50}(mm) = 0.76
			ϕ' (°) = 34.8	D_{95}(mm) = 8.00
			K_S (lb/in^3) = 60	
334.0	100.0	Top of Rock		
		Limestone(60%) interbedded with Shale(40%). Limestone is gray, fine grained, thin bedded and argillaceous. Shale is gray, silty, and calcareous.		
			γ_t (lb/ft^3) = 165	
			SDI (%) = 80	
			q_u(ton/ft^2) = 411	
			c (lb/in^2) = 300	
			ϕ' (°) = 28.0	
283.1	150.9			

Figure 5b

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 2 - STA 189+65.5, CL
Borings AC-4, 5, B-1

Approximate Elevation (ft)	Approximate Depth (ft)	Description STRATA: Description (USCS Description)	Parameters	
428.9	0.0			
		Sandy Lean Clay (CL)	γ_t (lb/ft^3) = 121 K_S (lb/in^3) = 30	D_{50}(mm) = 0.060 D_{95}(mm) = 0.37
423.9	5.0			
420.8	8.1	▽ Sand with Silt (SM)	γ_t (lb/ft^3) = 107 γ_e (lb/ft^3)* = 45 ϕ' (^{o}) = 28.0 K_S (lb/in^3) = 25 K_S (lb/in^3) = 20	D_{50}(mm) = 0.11 D_{95}(mm) = 0.85 (Above Water Table) (Below Water Table)
409.9	19.0			
		Sand (SW, SW-SM)	γ_e (lb/ft^3)* = 68 ϕ' (^{o}) = 35.2 K_S (lb/in^3) = 60	D_{50}(mm) = 1.6 D_{95}(mm) = 19
379.9	49.0			
		Sand with gravel (SW-SM, SP, SP-SM, GP-GM,)	γ_e (lb/ft^3)* = 65 ϕ' (^{o}) = 35.6 K_S (lb/in^3) = 60	D_{50}(mm) = 2.4 D_{95}(mm) = 18
334.9	94.0	Top of Rock		
		Limestone (55%) interbedded with Shale(45%). Limestone is gray, fine grained, thin bedded, argillaceous and fossiliferous. Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous.	γ_t (lb/ft^3) = 165 SDI (%) = 73 q_u(ton/ft^2) = 563 c (lb/in^2) = 300 ϕ' (^{o}) = 28.0	
282.6	146.3			
		Shale (70%) interbedded with Limestone (30%). Shale is gray, fine grained, thin bedded, silty, calcareous, fossiliferous. Limestone is gray, microcrystalline to fine grained, thin bedded, argillaceous, fossiliferous.	γ_t (lb/ft^3) = 160 c (lb/in^2) = 300 ϕ' (^{o}) = 28.0	
275.9	153.0			

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 3 - STA 193+77.5, CL
Borings AC-6, 7, 8, 9, B-2

Approximate Elevation (ft)	Approximate Depth (ft)	Description STRATA: Description (USCS Description)	Parameters	
419.4	0.0			
		Water - Ohio River		
379.4	40.0			
		Sand (SW-SM, SP, SW)	γ_e (lb/ft^3)* = 66 ϕ' (°) = 34.5 K_S (lb/in^3) = 60	D_{50}(mm) = 0.98 D_{95}(mm) = 17
365.4	54.0			
		Sand (SP-SM, SW-SM)	γ_e (lb/ft^3)* = 67 ϕ' (°) = 36.9 K_S (lb/in^3) = 60	D_{50}(mm) = 0.62 D_{95}(mm) = 18
354.4	65.0			
		Gravel (GW, GW-GM)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 11 D_{95}(mm) = 30
339.4	80.0			
		Sand (SP-SM, SM)	γ_e (lb/ft^3)* = 69 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 1.3 D_{95}(mm) = 19
332.0	87.4	Top of Rock		
		Limestone (60%) interbedded with Shale (40%). Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous. Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.	γ_t (lb/ft^3) = 164 SDI (%) = 67 q_u(ton/ft^2) = 647 c (lb/in^2) = 300 ϕ' (°) = 28.0	
278.6	140.8			

Figure 5d

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 4 - STA 206+12.5, CL
Borings AC-10, 11, 12, 13, B-3

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Description)	Parameters	
418.8	0.0	Water - Ohio River		
378.8	40.0	Gravel (GW, GP)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 35.5 K_S (lb/in^3) = 20	D_{50}(mm) = 9.1 D_{95}(mm) = 27
359.8	59.0	Sand (SP-SM, SW-SM, SP)	γ_e (lb/ft^3)* = 68 ϕ' (°) = 37.0 K_S (lb/in^3) = 125	D_{50}(mm) = 2.4 D_{95}(mm) = 18
343.8	75.0	Gravel (GP-GM, GM)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 3.9 D_{95}(mm) = 23
336.2	82.6	Top of Rock Limestone (60%) interbedded with Shale (40%). Limestone is gray, microcrystalline to fine grained, thin, wavy to nodular bedded, fossiliferous, and argillaceous. Shale is gray, silty, laminated to thin bedded, calcareous, and fossiliferous.	γ_t (lb/ft^3) = 165 SDI (%) = 74 q_u(ton/ft^2) = 550 c (lb/in^2) = 300 ϕ' (°) = 28.0	
282.8	136.0			

Figure 5e

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 5 - STA 210+24.5, CL
Borings AC-14, AC-15, B-4

AC-14

Approximate Elevation (ft)	Depth (ft)	Description STRATA: Description (USCS Description)	Parameters
436.0	0.0	(AC-14) Sandy Lean Clay (CL)	γ_t (lb/ft^3) = 120; q_U (lb/ft^2) = 2000; C_U (lb/ft^2) = 1000; K_S (lb/in^3) = 100; D_{50}(mm) = 0.011; D_{85}(mm) = 4.00
423.9	12.1	Top of Rock (AC-14) Limestone, gray, medium grained, thin bedded to medium bedded.	γ_t (lb/ft^3) = 164; q_u(ton/ft^2) = 600; c (lb/in^2) = 10; ϕ' (°) = 20.0
417.0	19.0	(AC-14) Limestone (60%) interbedded with Shale (40%). Limestone is gray, fine to medium grained, very thin to medium wavy bedded. Shale is gray, silty	γ_t (lb/ft^3) = 164; SDI (%) = 96; q_u(ton/ft^2) = 650; c (lb/in^2) = 150; ϕ' (°) = 27.0
407.0	29.0	(AC-14) Shale, gray to red, very thin bedded to thin bedded, silty	γ_t (lb/ft^3) = 146; SDI (%) = 0; q_u(ton/ft^2) = 13; c (lb/in^2) = 5; ϕ' (°) = 20.0
404.0	32.0	(AC-14) Limestone, light gray, fine grained, thin bedded, shale stringers, streaks, and partings	γ_t (lb/ft^3) = 166; q_u(ton/ft^2) = 988; c (lb/in^2) = 150; ϕ' (°) = 32.0
394.2	41.8	(AC-14) Dolomite Limestone, greenish gray, fine, thin bedded	γ_t (lb/ft^3) = 168; q_u(ton/ft^2) = 600; c (lb/in^2) = 150; ϕ' (°) = 32.0
392.0	44.0		

AC-15

Approximate Elevation (ft)	Depth (ft)	Description Strata: Description (USCS Description)	Parameters
419.4	0.0	Water - Ohio River	
415.2	4.2	Sandy Silt with Gravel (ML)	γ_e (lb/ft^3)* = 38; ϕ' (°) = 29.5; K_S (lb/in^3) = 20; D_{50}(mm) = 0.26; D_{95}(mm) = 10
409.4	10	Poorly graded gravel with silt and sand (GP-GM)	γ_e (lb/ft^3)* = 71; ϕ' (°) = 38.0; K_S (lb/in^3) = 125; D_{50}(mm) = 7.1; D_{95}(mm) = 27
392.0	27.4	Top of Rock	

392.0 44.0 / 392.0 27.4 Top of Rock

Limestone (30% - 60%) interbedded with Shale (40% - 70%). Limestone is gray, fine to medium crystalline grained, very thin to medium nodularly bedded, fossiliferous. Shale is gray, silty, laminated, calcareous, fossiliferous.

γ_t (lb/ft^3) = 168
SDI (%) = 97
q_u(ton/ft^2) = 782
c (lb/in^2) = 100
ϕ' (°) = 25.0

342.0 94.0

Figure 5f

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Indiana Abutment
AC- 20, 23, 26

Approximate Elevation (ft)	Approximate Depth (ft)	Description STRATA: Description (USCS Description)	Parameters	
494.4	0.0			
		Gravel (GC)	γ_t (lb/ft^3) = 133 ϕ' (°) = 38.0 K_S (lb/in^3) = 225	D_{50}(mm) = 2.2 D_{95}(mm) = 21
490.8	3.6	Top of Rock		
		Limestone, gray, fine grained, very thick bedded, close fracture spacing, Clay fills some fractures.	γ_t (lb/ft^3) = 165 q_u(ton/ft^2) = 732 c (lb/in^2) = 10 ϕ' (°) = 20.0	
474.2	20.2			
		Shale, dark gray and tan, very thick bedded, close fracture spacing	γ_t (lb/ft^3) = 160 SDI (%) = 74 q_u(ton/ft^2) = 89 c (lb/in^2) = 20 ϕ' (°) = 22.0	
459.2	35.2			
		Limestone, gray and tan, fine grained, medium bedded to very thick bedded, close fracture zones, with some zones dolomitic		
422.8	71.6			
		Shale, dark gray, medium bedded		
420.5	73.9			
		Limestone, light gray, medium bedded, close fracture spacing		
419.1	75.3			

Figure 5g

Figure 6 - East End Bridge
Response Spectra Curves for Preliminary Design
2500-Year Return Period SEE

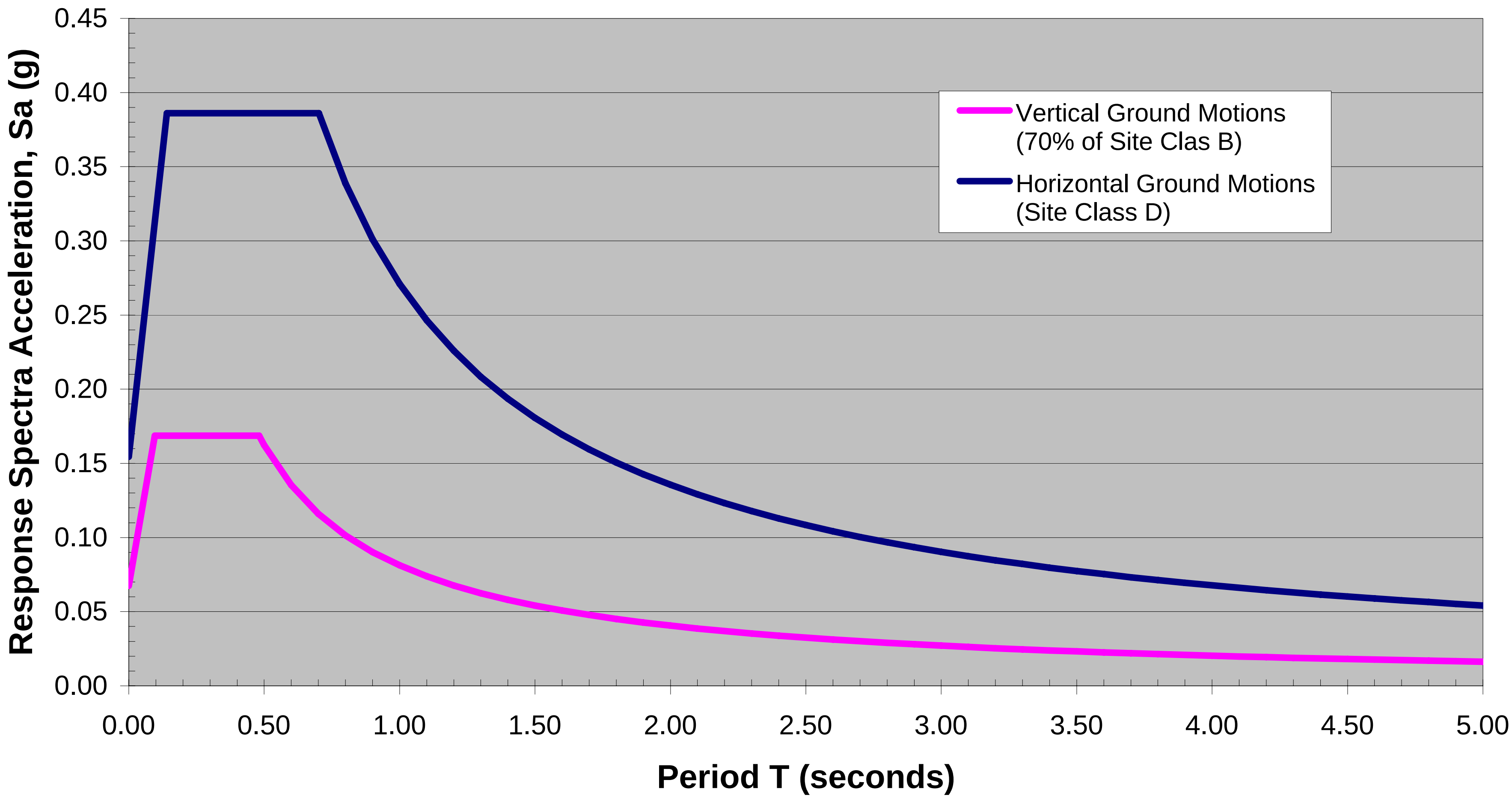
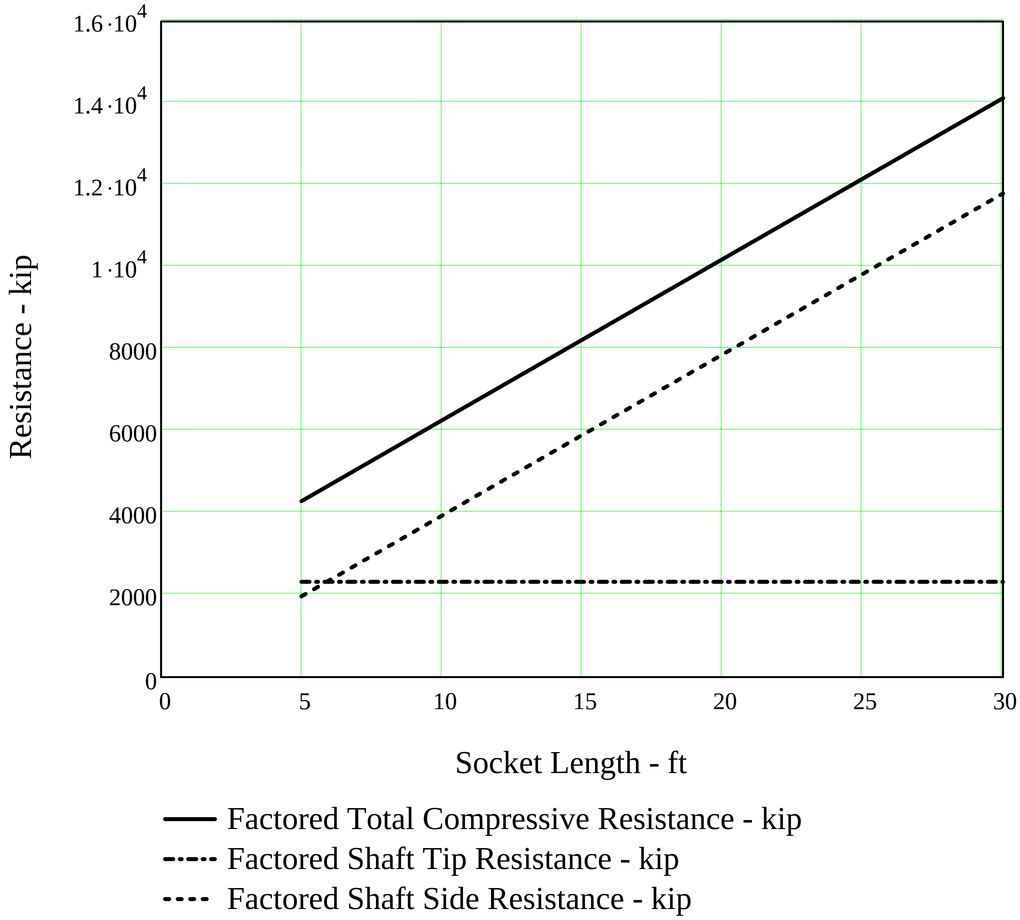


Figure 7a

Drilled Shaft Resistance vs. Socket Length, Pier 1, 7.5-foot Diameter Socket
Compressive Resistance vs. Socket Length



Socket Diameter $D_s = 7.5\,\text{ft}$

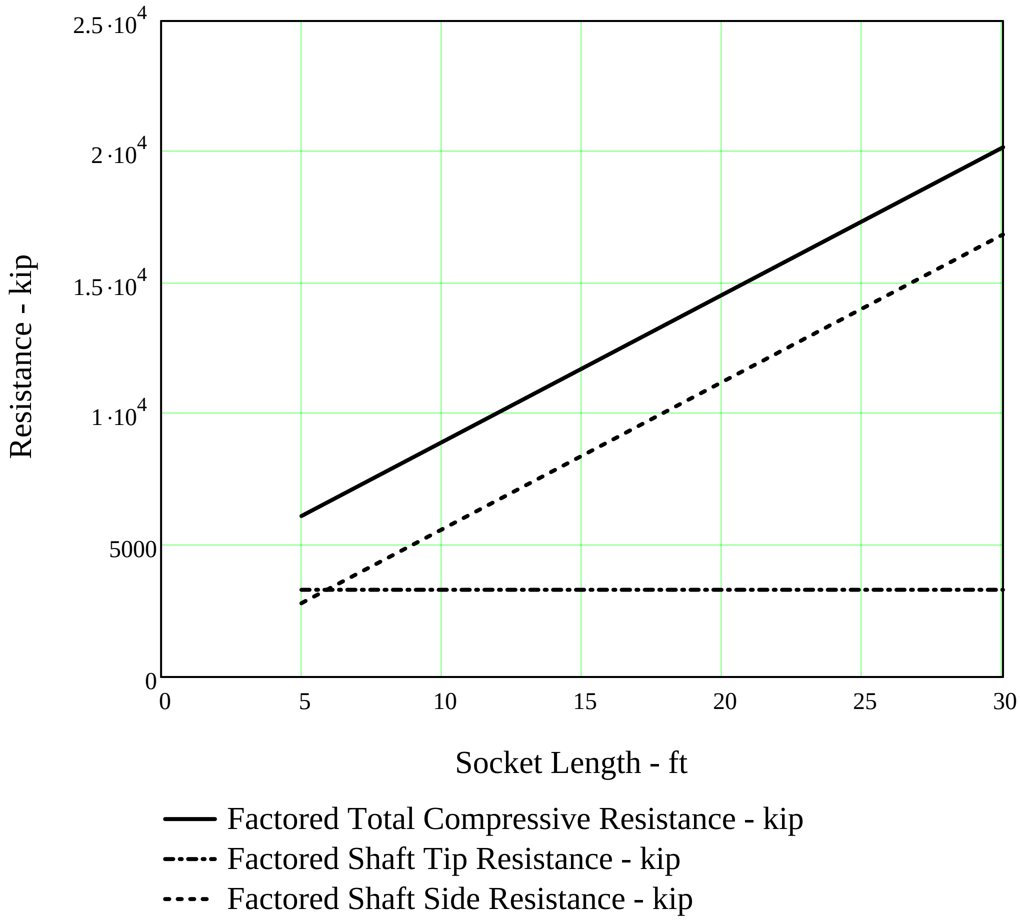
Nominal Skin Friction	$q_s = 165.7\,\text{psi}$	Resistance Factor	$\phi_{qs} = 0.7$
Nominal End Bearing	$q_p = 37.6\,\text{tsf}$	Resistance Factor	$\phi_{qp} = 0.7$

<u>Note</u>:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 7b

Drilled Shaft Resistance vs. Socket Length, Pier 1, 7.5-foot Diameter Socket
Compressive Resistance vs. Socket Length - Extreme Limit States



Socket Diameter $D_s = 7.5\,\text{ft}$

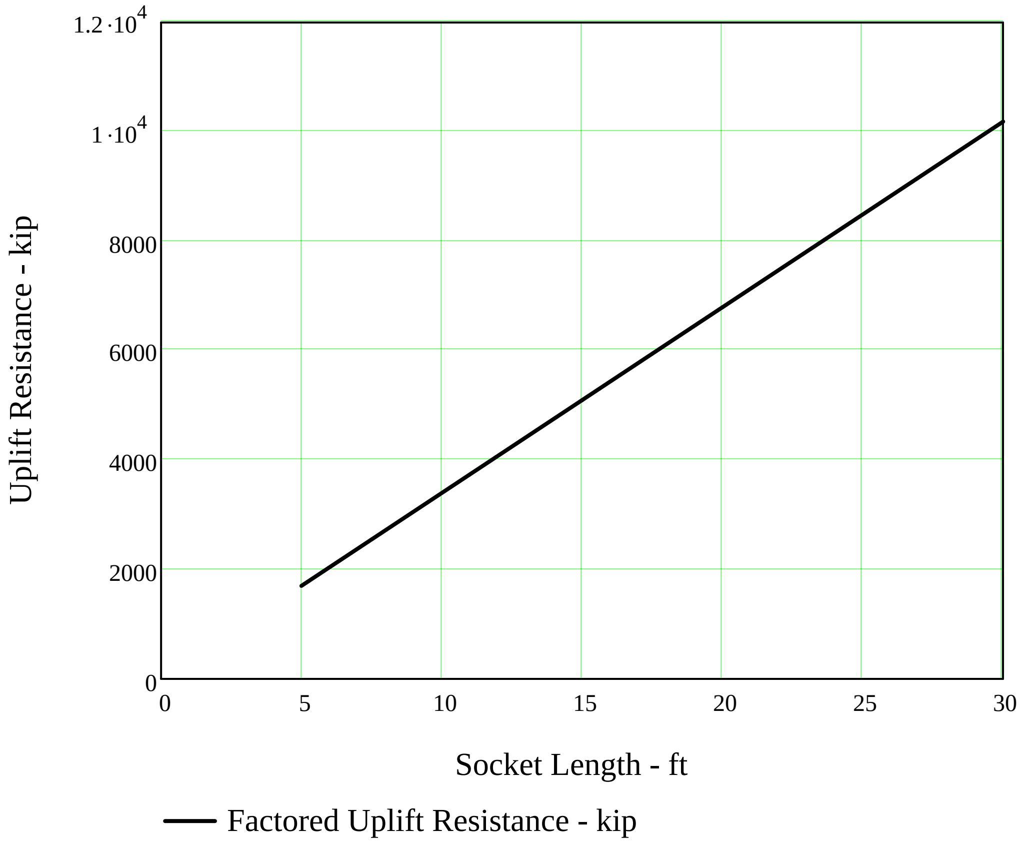
Nominal Skin Friction	$q_s = 165.7\,\text{psi}$	Resistance Factor	$\phi_{qs} = 1.0$
Nominal End Bearing	$q_p = 37.6\,\text{tsf}$	Resistance Factor	$\phi_{qp} = 1.0$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 7c

Drilled Shaft Resistance vs. Socket Length, Pier 1, 7.5-foot Diameter Socket
Uplift Resistance vs. Socket Length



Socket Diameter $D_s = 7.5\,\mathrm{ft}$

Nominal Uplift Resistance $q_s = 165.7\,\mathrm{psi}$

Weight of Shaft $\gamma_{conc} - \gamma_w = 87.6\,\mathrm{pcf}$

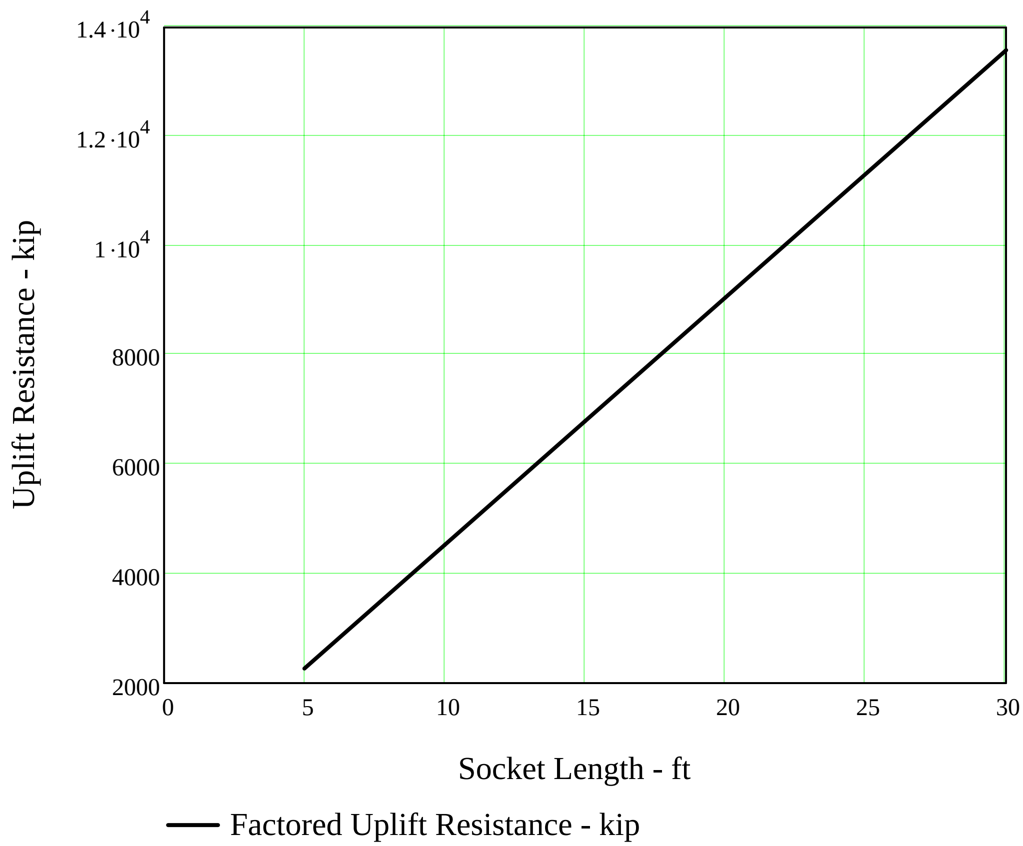
Resistance Factor $\phi_{up} = 0.6$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 7d

Drilled Shaft Resistance vs. Socket Length, Pier 1, 7.5-foot Diameter Socket
Uplift Resistance vs. Socket Length - Extreme Limit States



Socket Diameter $D_s = 7.5\,\text{ft}$

Nominal Uplift Resistance $q_s = 165.7\,\text{psi}$

Weight of Shaft $\gamma_{conc} - \gamma_w = 87.6\,\text{pcf}$

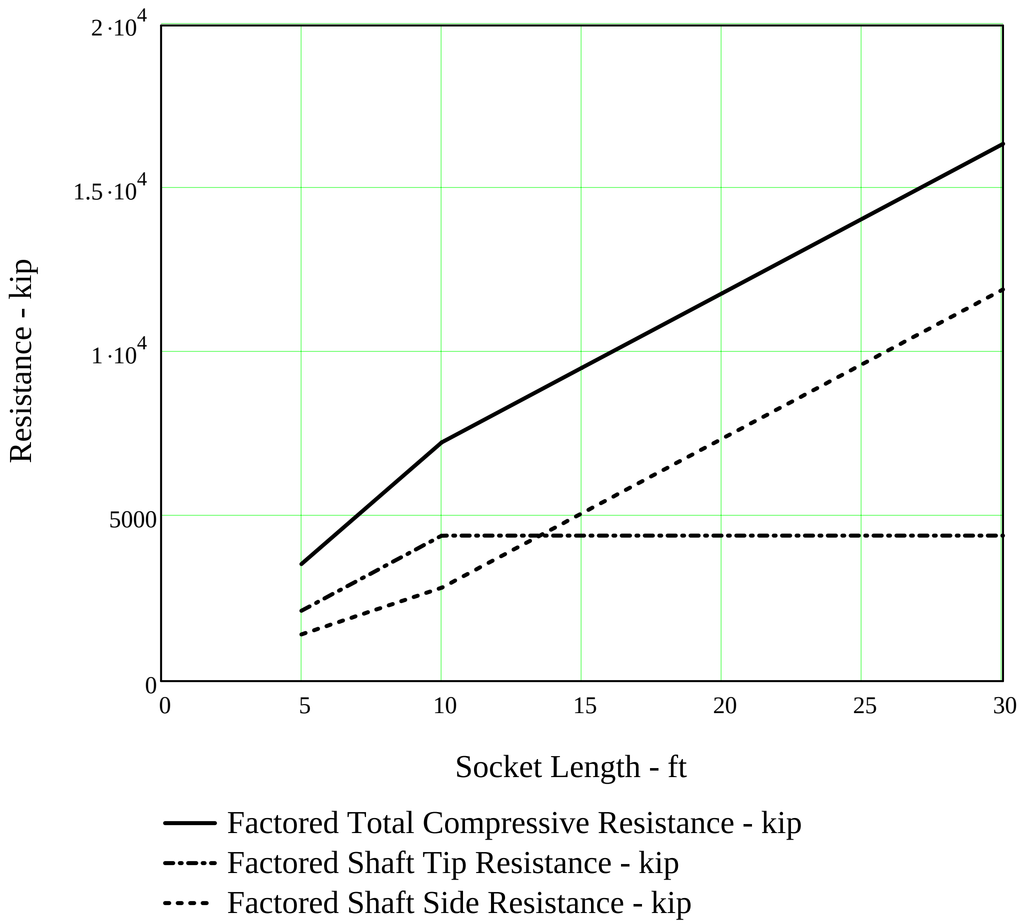
Resistance Factor $\phi_{up} = 0.8$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 8a

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 7.5-foot Diameter Socket Compressive Resistance vs. Socket Length



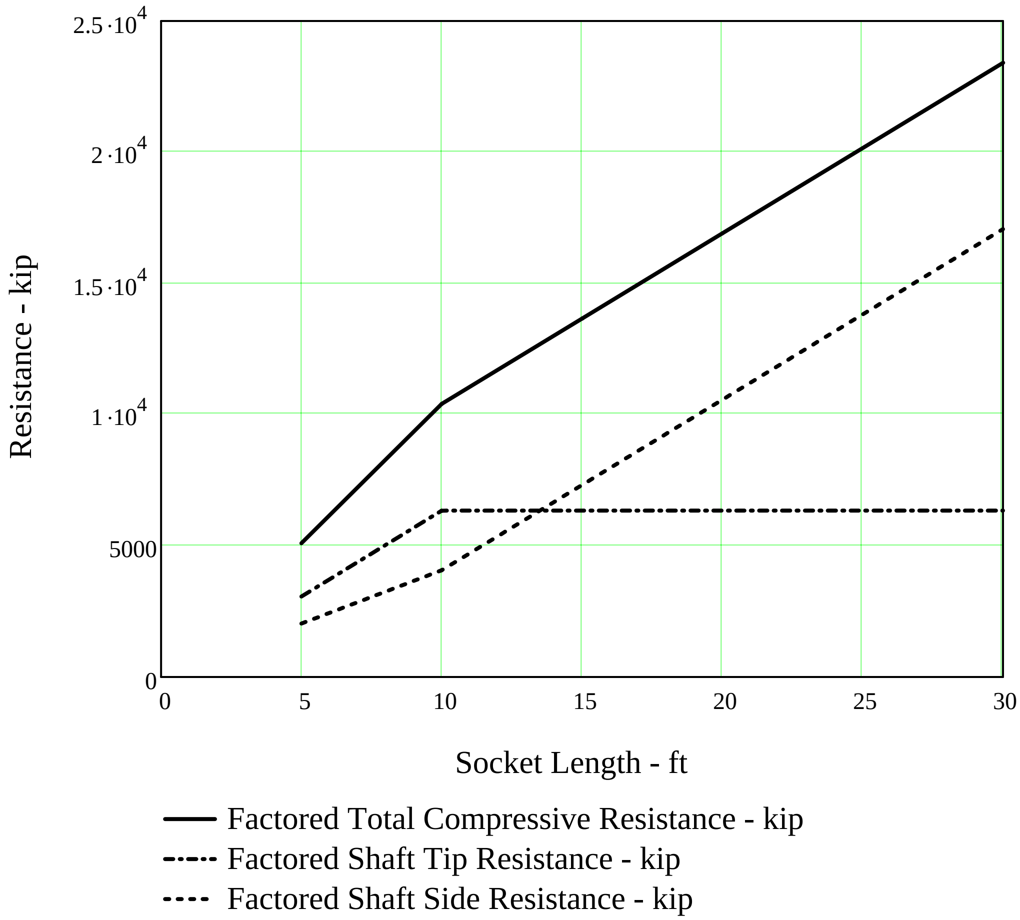
Socket Diameter	$D_s = 7.5\,\text{ft}$	
Nominal Skin Friction	$q_{s1} = 119.8\,\text{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\,\text{psi}$	below 10 ft
	Resistance Factor	$\phi_{qs} = 0.7$
Nominal End Bearing	$q_{p1} = 34.7\,\text{tsf}$	top 10 ft of rock
	$q_{p2} = 71.7\,\text{tsf}$	below 10 ft
	Resistance Factor	$\phi_{qp} = 0.7$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 8b

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 7.5-foot Diameter Socket
Compressive Resistance vs. Socket Length - Extreme Limit States



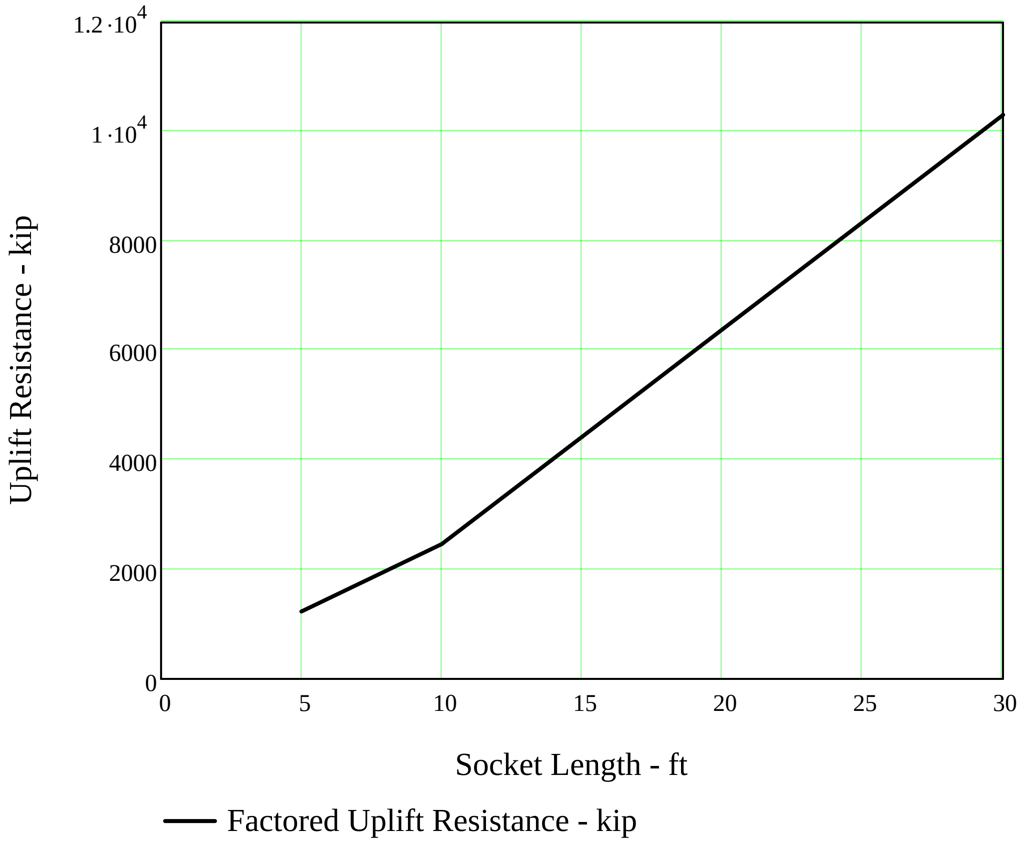
Socket Diameter	$D_s = 7.5\,ft$	
Nominal Skin Friction	$q_{s1} = 119.8\,psi$	top 10 ft of rock
	$q_{s2} = 191.7\,psi$	below 10 ft
	Resistance Factor	$\phi_{qs} = 1$
Nominal End Bearing	$q_{p1} = 34.7\,tsf$	top 10 ft of rock
	$q_{p2} = 71.7\,tsf$	below 10 ft
	Resistance Factor	$\phi_{qp} = 1$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 8c

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 7.5-foot Diameter Socket
Uplift Resistance vs. Socket Length



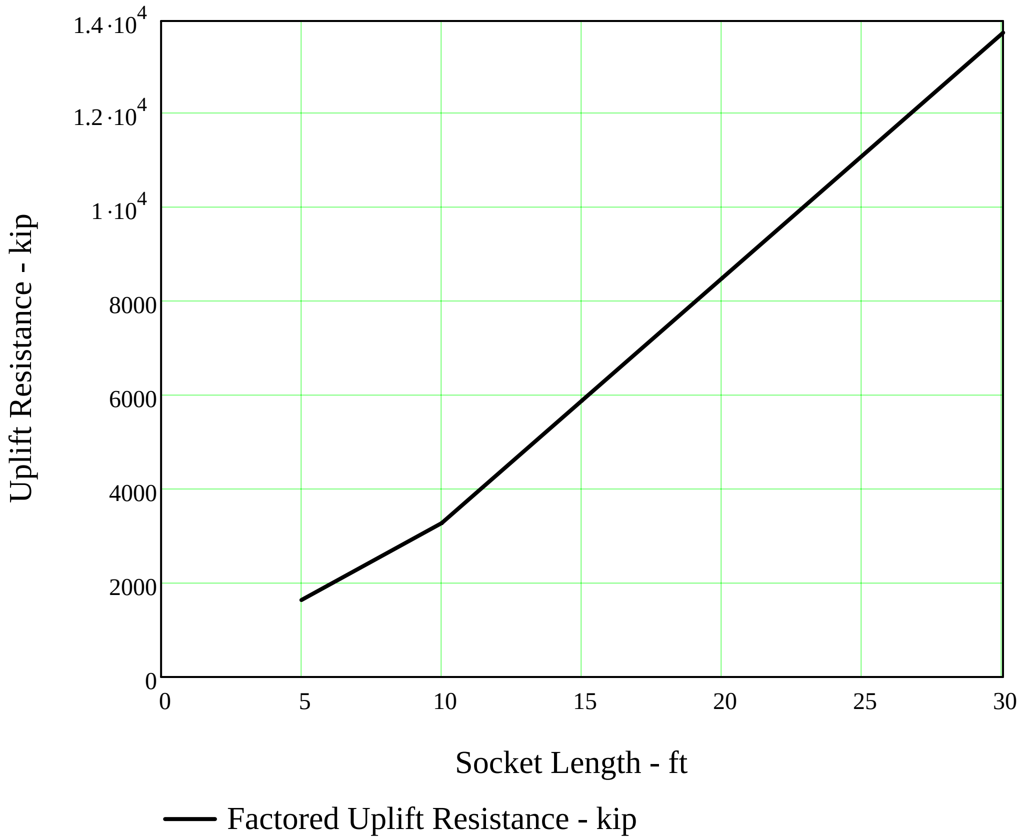
Socket Diameter	$D_s = 7.5\,\mathrm{ft}$	
Nominal Uplift Resistance	$q_{s1} = 119.8\,\mathrm{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\,\mathrm{psi}$	below 10 ft
Weight of Shaft	$\gamma_{conc} - \gamma_w = 87.6\,\mathrm{pcf}$	
Resistance Factor	$\phi_{up} = 0.6$	

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 8d

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 7.5-foot Diameter Socket
Uplift Resistance vs. Socket Length - Extreme Limit States



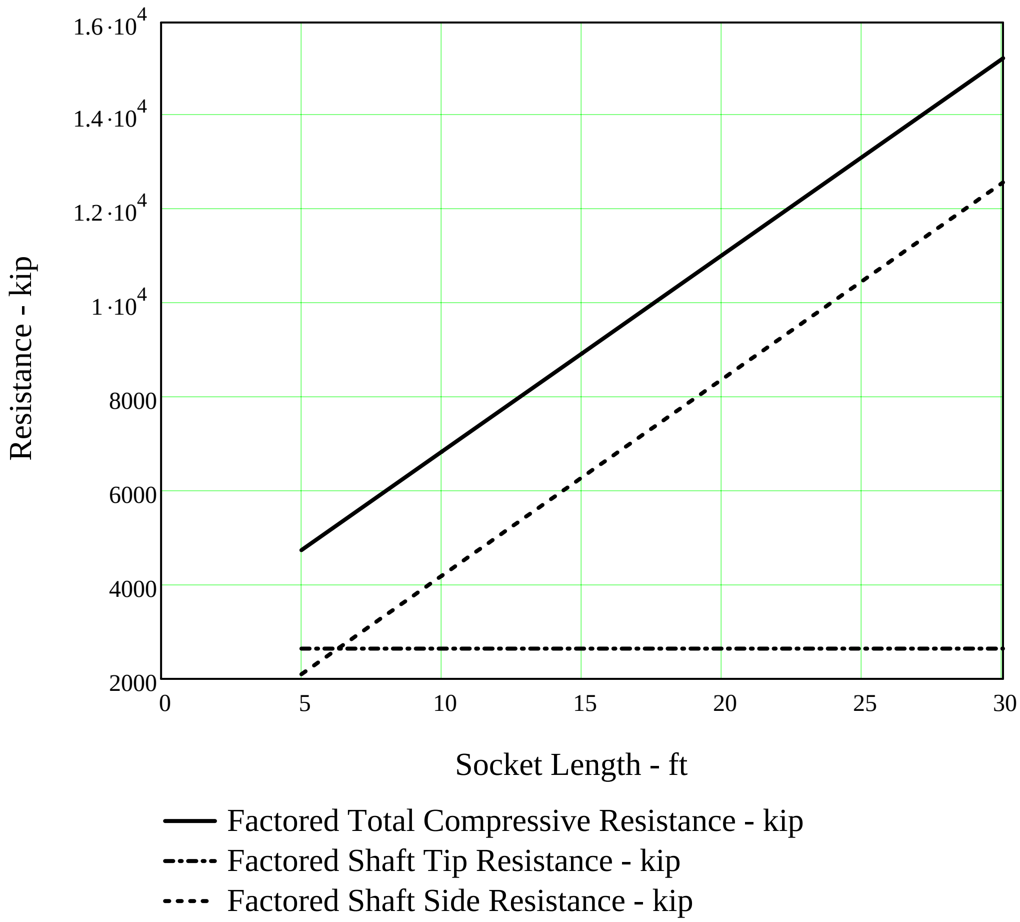
Socket Diameter	$D_s = 7.5\,\mathrm{ft}$	
Nominal Uplift Resistance	$q_{s1} = 119.8\,\mathrm{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\,\mathrm{psi}$	below 10 ft
Weight of Shaft	$\gamma_{conc} - \gamma_w = 87.6\,\mathrm{pcf}$	
Resistance Factor	$\phi_{up} = 0.8$	

<u>Note</u>:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for a 7.5 foot diameter socket, a minimum socket length of 11.25 feet is required.

Figure 9a

Drilled Shaft Resistance vs. Socket Length, Pier 1, 8-foot Diameter Socket Compressive Resistance vs. Socket Length



Socket Diameter $D_s = 8\ \text{ft}$

Nominal Skin Friction $q_s = 165.7\ \text{psi}$ Resistance Factor $\phi_{qs} = 0.7$

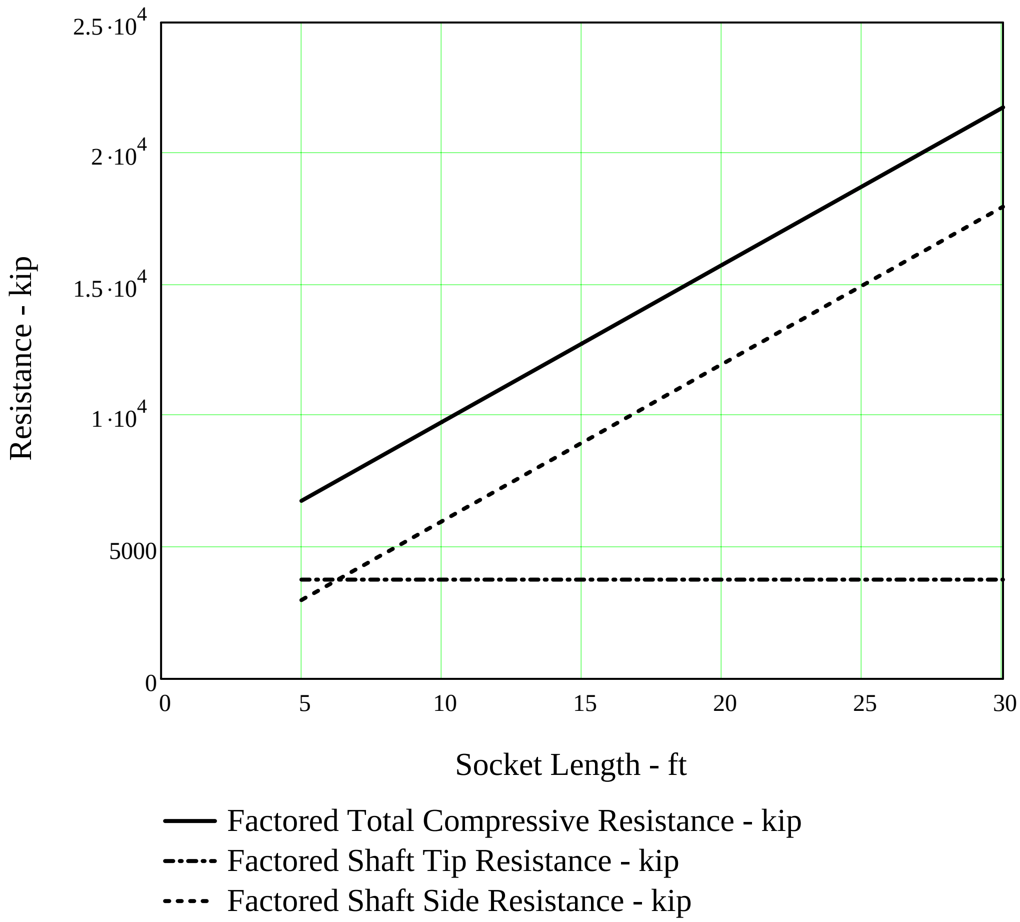
Nominal End Bearing $q_p = 37.6\ \text{tsf}$ Resistance Factor $\phi_{qp} = 0.7$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

Figure 9b

Drilled Shaft Resistance vs. Socket Length, Pier 1, 8-foot Diameter Socket
Compressive Resistance vs. Socket Length - Extreme Limit States



Socket Diameter $D_s = 8\ \text{ft}$

Nominal Skin Friction $q_s = 165.7\ \text{psi}$ Resistance Factor $\phi_{qs} = 1.0$

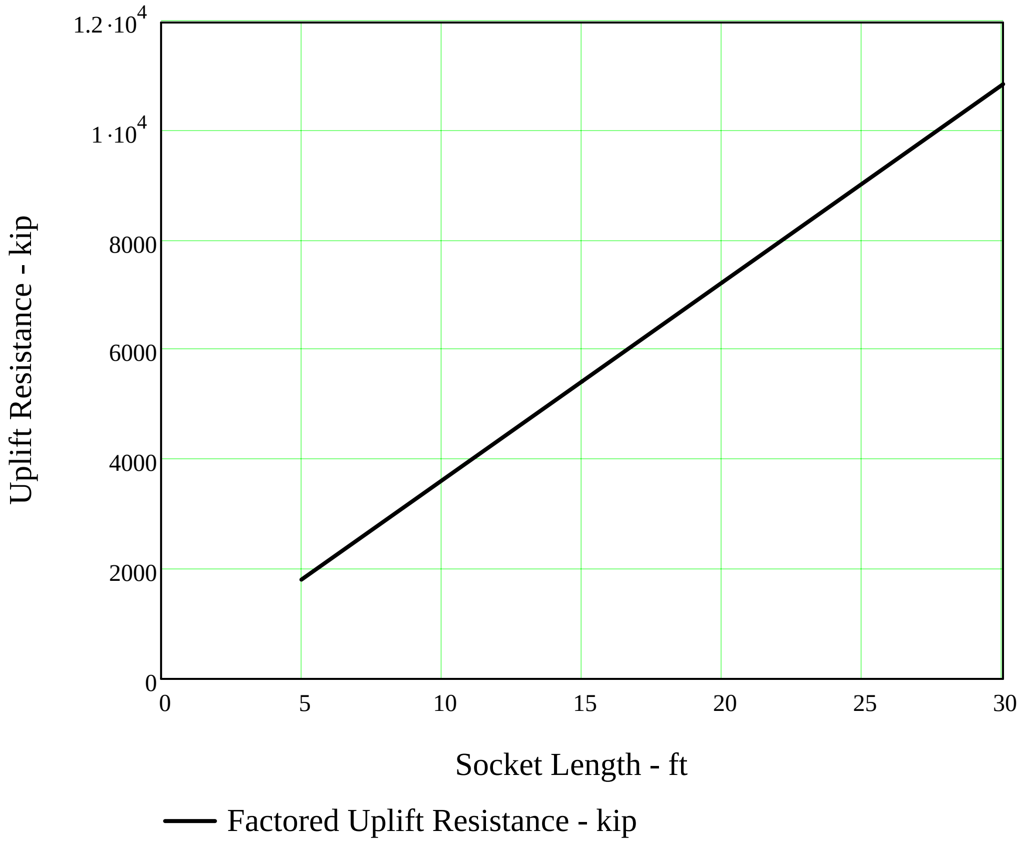
Nominal End Bearing $q_p = 37.6\ \text{tsf}$ Resistance Factor $\phi_{qp} = 1.0$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

Figure 9c

Drilled Shaft Resistance vs. Socket Length, Pier 1, 8-foot Diameter Socket
Uplift Resistance vs. Socket Length



Socket Diameter $D_s = 8\ \text{ft}$

Nominal Uplift Resistance $q_s = 165.7\ \text{psi}$ Weight of Shaft $\gamma_{conc} - \gamma_w = 87.6\ \text{pcf}$

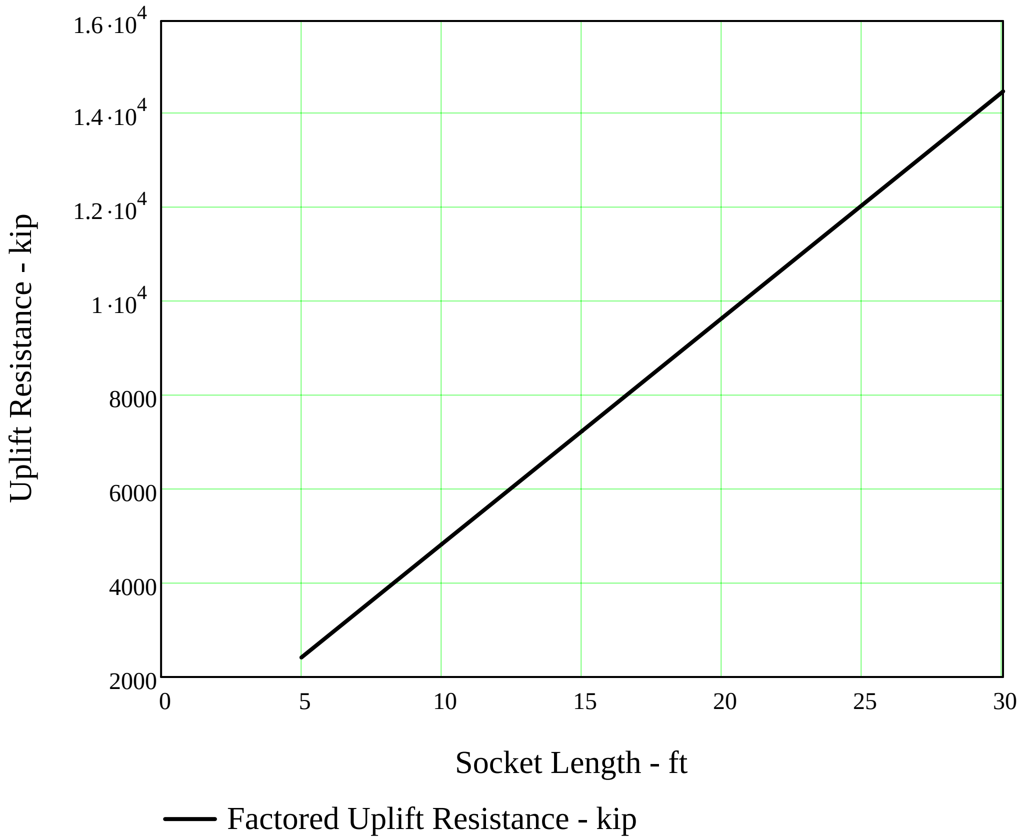
Resistance Factor $\phi_{up} = 0.6$

<u>Note</u>:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

Figure 9d

Drilled Shaft Resistance vs. Socket Length, Pier 1, 8-foot Diameter Socket
Uplift Resistance vs. Socket Length - Extreme Limit States



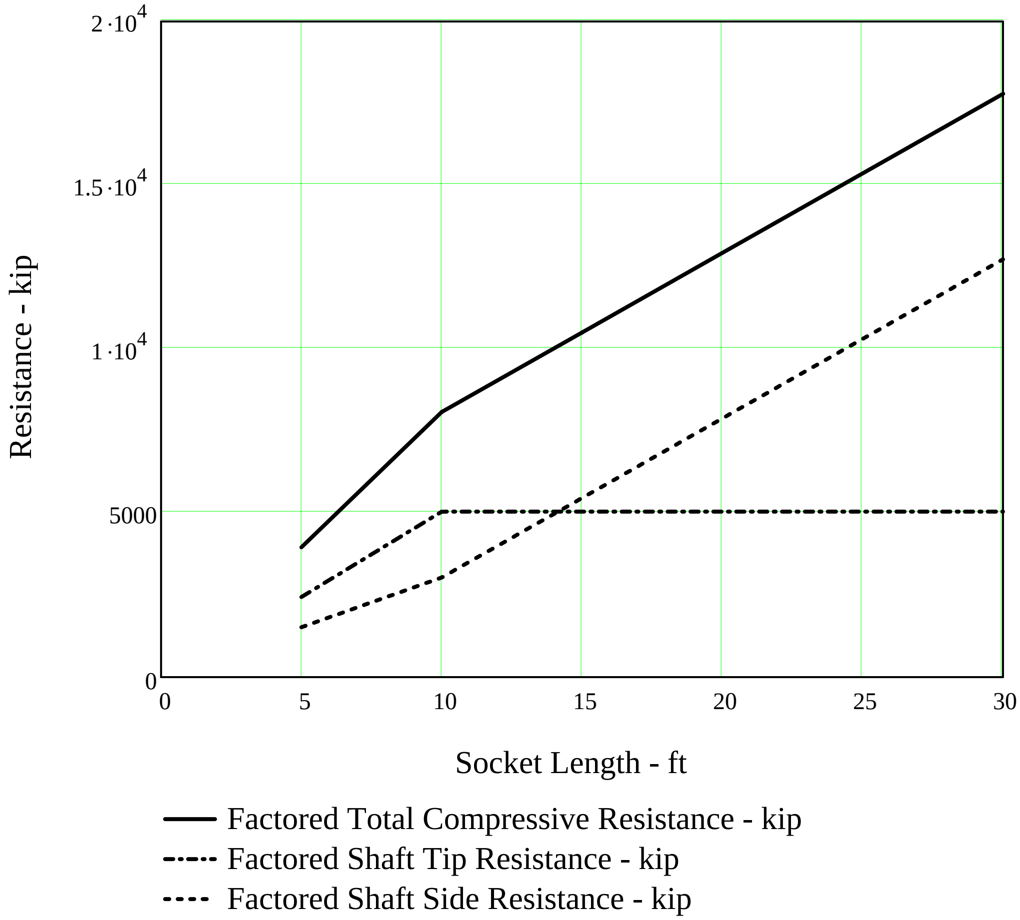
Socket Diameter	$D_s = 8\ \text{ft}$		
Nominal Uplift Resistance	$q_s = 165.7\ \text{psi}$	Weight of Shaft	$\gamma_{conc} - \gamma_w = 87.6\ \text{pcf}$
resistance factor	$\phi_{up} = 0.8$		

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

Figure 10a

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 8-foot Diameter Socket Compressive Resistance vs. Socket Length



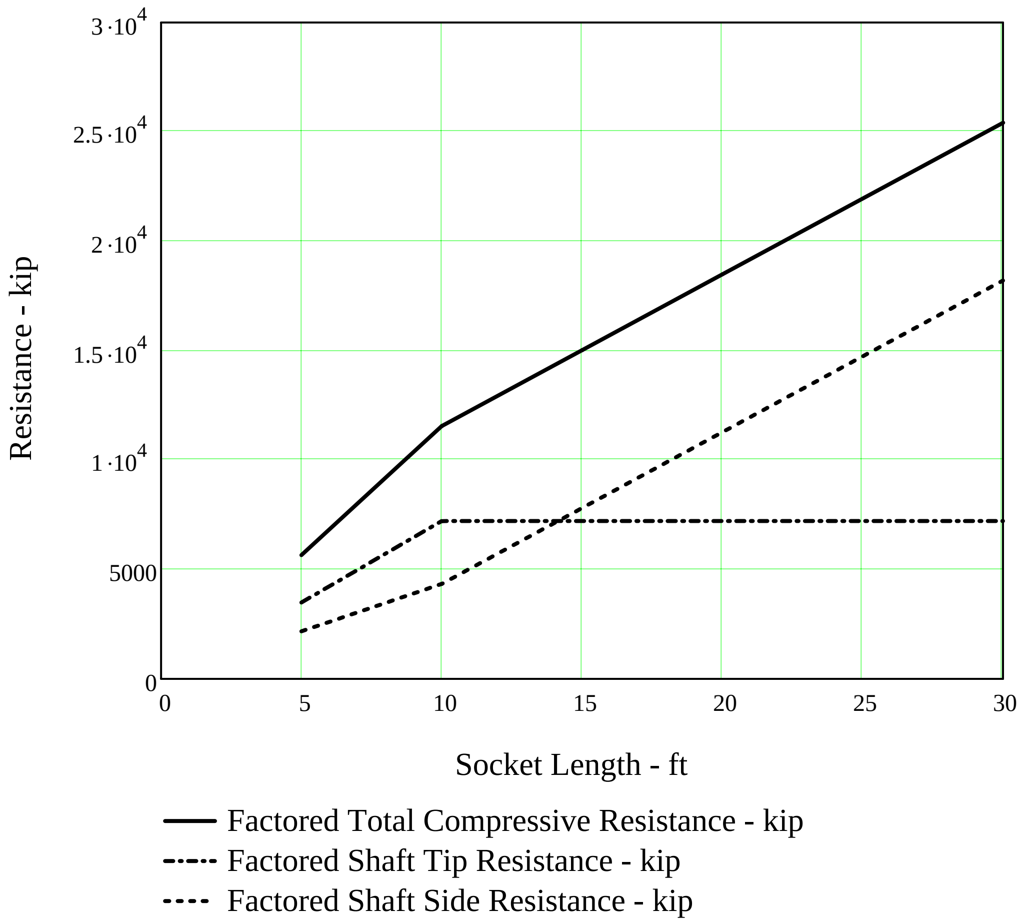
Socket Diameter	$D_s = 8\ \mathrm{ft}$	
Nominal Skin Friction	$q_{s1} = 119.8\ \mathrm{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\ \mathrm{psi}$	below 10 ft
	Resistance Factor	$\phi_{qs} = 0.7$
Nominal End Bearing	$q_{p1} = 34.7\ \mathrm{tsf}$	top 10 ft of rock
	$q_{p2} = 71.7\ \mathrm{tsf}$	below 10 ft
	Resistance Factor	$\phi_{qp} = 0.7$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

Figure 10b

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 8-foot Diameter Socket
Compressive Resistance vs. Socket Length - Extreme Limit States



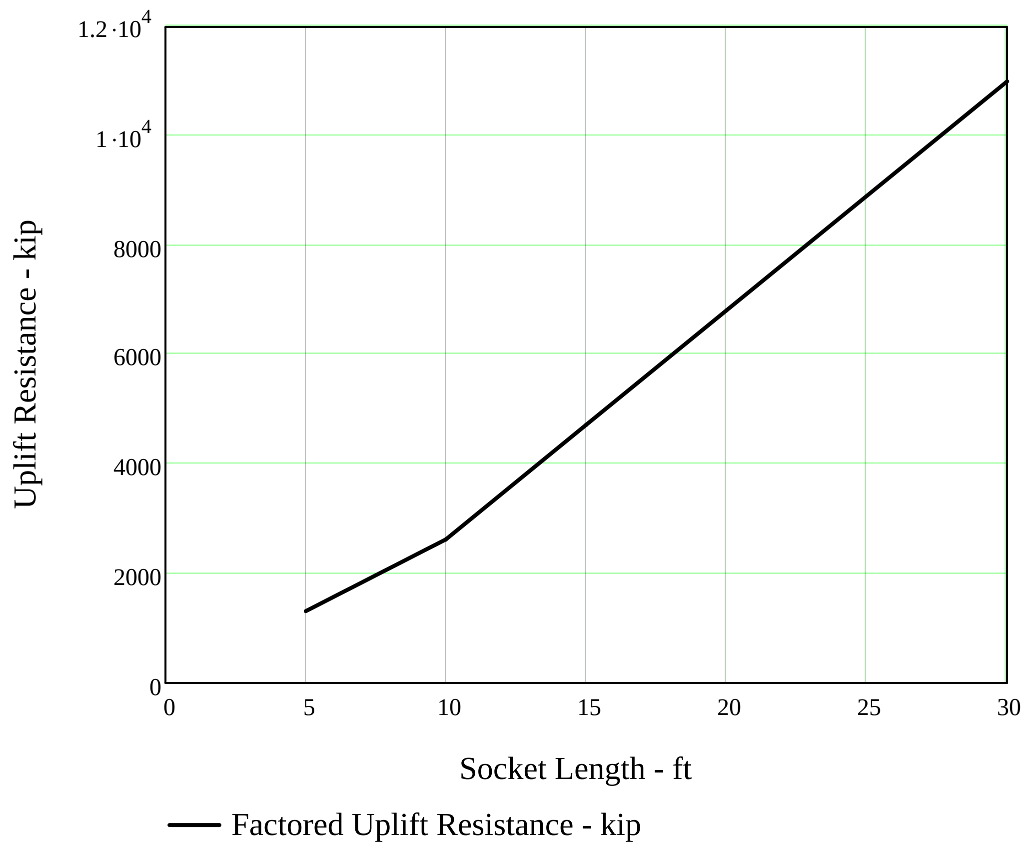
Socket Diameter	$D_s = 8$ ft	
Nominal Skin Friction	$q_{s1} = 119.8$ psi	top 10 ft of rock
	$q_{s2} = 191.7$ psi	below 10 ft
	Resistance Factor	$\phi_{qs} = 1.0$
Nominal End Bearing	$q_{p1} = 34.7$ tsf	top 10 ft of rock
	$q_{p2} = 71.7$ tsf	below 10 ft
	Resistance Factor	$\phi_{qp} = 1.0$

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

Figure 10c

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 8-foot Diameter Socket
Uplift Resistance vs. Socket Length



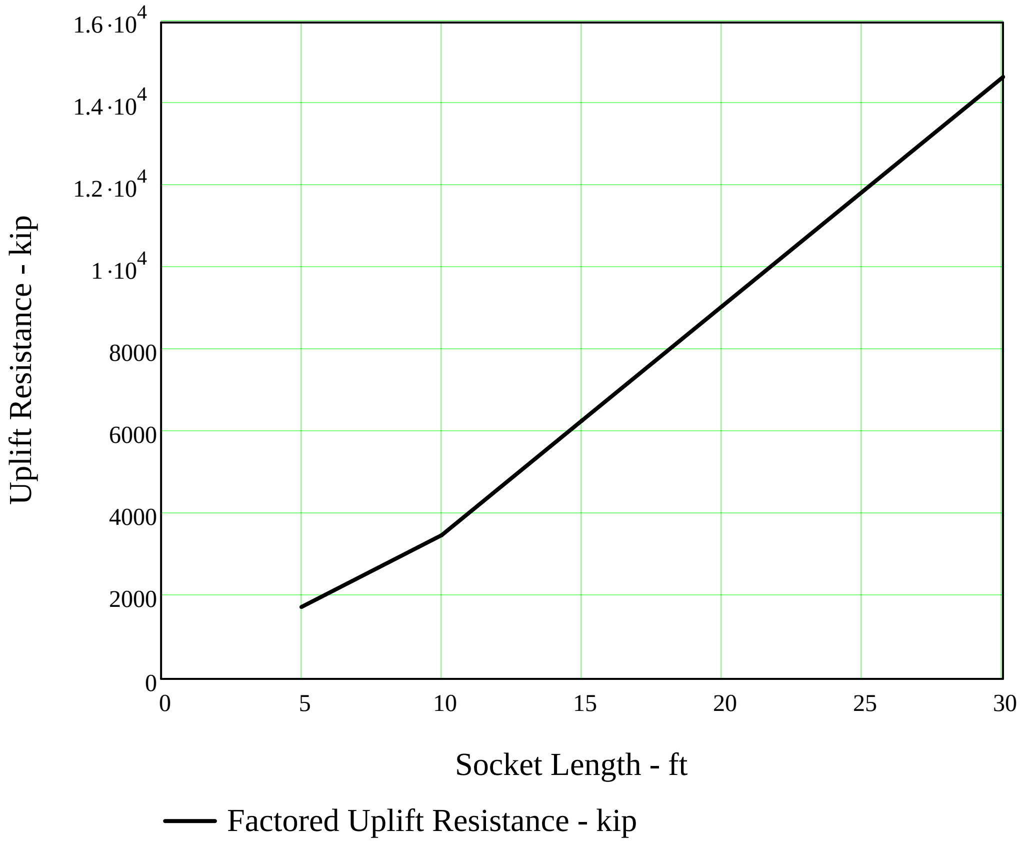
Socket Diameter	$D_s = 8\ \mathrm{ft}$	
Nominal Uplift Resistance	$q_{s1} = 119.8\ \mathrm{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\ \mathrm{psi}$	below 10 ft
Weight of Shaft	$\gamma_{conc} - \gamma_w = 87.6\ \mathrm{pcf}$	
Resistance Factor	$\phi_{up} = 0.6$	

Note:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

Figure 10d

Drilled Shaft Resistance vs. Socket Length, Piers 2 through 5, 8-foot Diameter Socket
Uplift Resistance vs. Socket Length - Extreme Limit States



Socket Diameter	$D_s = 8\,\text{ft}$	
Nominal Uplift Resistance	$q_{s1} = 119.8\,\text{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\,\text{psi}$	below 10 ft
Weight of Shaft	$\gamma_{conc} - \gamma_w = 87.6\,\text{pcf}$	
Resistance Factor	$\phi_{up} = 0.8$	

<u>Note</u>:

Calculated capacities for varying socket lengths are shown. In accordance with KYTC practice for rock-socketed caissons, the minimum socket length shall be 1.5 times the socket diameter, i.e., for an 8 foot diameter socket, a minimum socket length of 12 feet is required.

APPENDICES

APPENDIX A
GEOTECHNICAL SUBSURFACE DATA SHEETS

GEOTECHNICAL SYMBOL SHEET

COUNTY OF	ITEM NO.	SHEET NO.
JEFFERSON	5-118.00	

AASHTO Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35% or less passing 0.075 mm)							Silt-Clay Materials (More than 35% passing 0.075 mm)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analyses, percent Passing											
2.00 mm (No. 10)	50 max	---	---	---	---	---	---	---	---	---	---
0.425 mm (No. 40)	30 max	50 max	51 min	---	---	---	---	---	---	---	---
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of Fraction Passing 0.425 mm (No. 40)											
Liquid Limit	---		---	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min

Unified Soil Classifications

MAJOR DIVISIONS		SYMBOL	NAME
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
	SILTS AND CLAYS LL IS GREATER THAN 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
UNCLASSIFIED MATERIAL		NONE	Non-classified material (i.e. overburden, pavement, slag, etc.) include visual description.

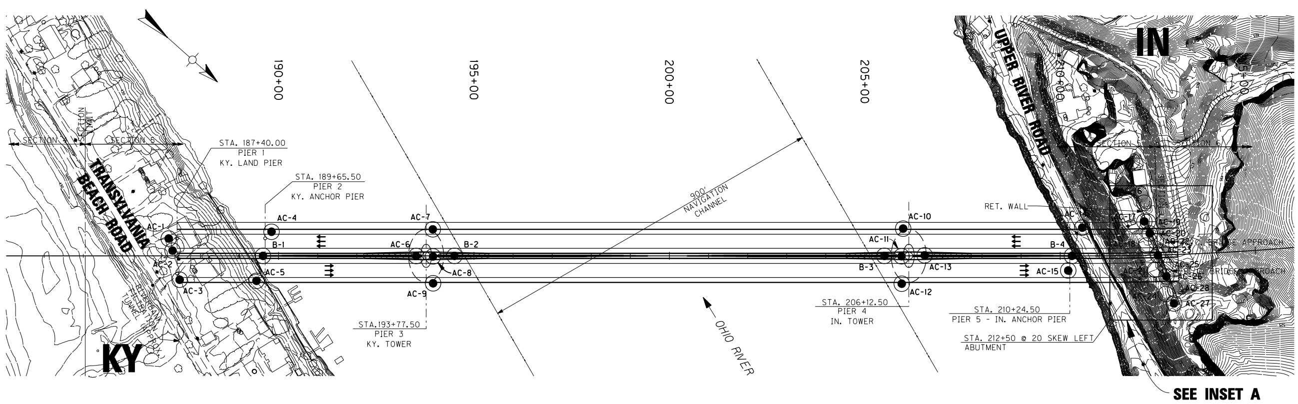
AI Activity Index
LI Liquidity Index
S+C Silt + Clay (% finer than No. 200 Sieve)
Rockline Soundings
Disturbed Sample Boring
Undisturbed Sample Boring
Undisturbed Sample Boring & Rock Core
Rock Core
Open Face Log
Slope Inclinometer Installation
typical applications:
OW Observation Well
Approximate Footing Elevation
(Date) Water Elevation
VS (psf) Field Vane Shear Strength
Thin-walled Tube Sample
< Standard Penetration Test Sample
N Penetration Resistance
Qu (psf) Unconfined Compressive Strength
UU (psf) Unconsolidated Undrained Triaxial Strength
w(%) Moisture Content
KY RQD Rock Quality Designation (Kentucky Method)
STD RQD Rock Quality Designation (Standard Method)
SDI (JS) Slake Durability Index (Jar Slake Test)
REC Core Recovery
ϕ Angle of Internal Friction (Total Stress)
$\bar{\phi}$ Angle of Internal Friction (Effective Stress)
c (psf) Cohesion (Total Stress)
$\bar{c}$ (psf) Cohesion (Effective Stress)
γ (psf) Total Unit Weight
RDZ Rock Disintegration Zone
OB Overburden Bench
IB Intermediate Bench
R Refusal
NR Refusal Not Encountered

LIMESTONE
SANDSTONE
DURABLE SHALE (SDI ≥ 95)
NONDURABLE SHALE (SDI < 95)
COAL
TALUS, MINE WASTE, FILL MATERIAL, BOULDERS, & ETC.
GRANULAR EMBANKMENT
STRUCTURE GRANULAR BACKFILL
SLOPE PROTECTION

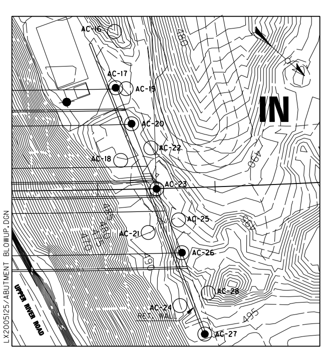
CHECKED BY ____ DATE ____
APPROVED BY ____ DATE ____

19-SEPT-2007 15:00
LX2005125/5125LGD.DGN

SUBSURFACE DATA



SEE INSET A



INSET A
SCALE: 1"=30'

100 0 100 200 300
GRAPHIC SCALE IN FEET

SCALE: 1" = 100'

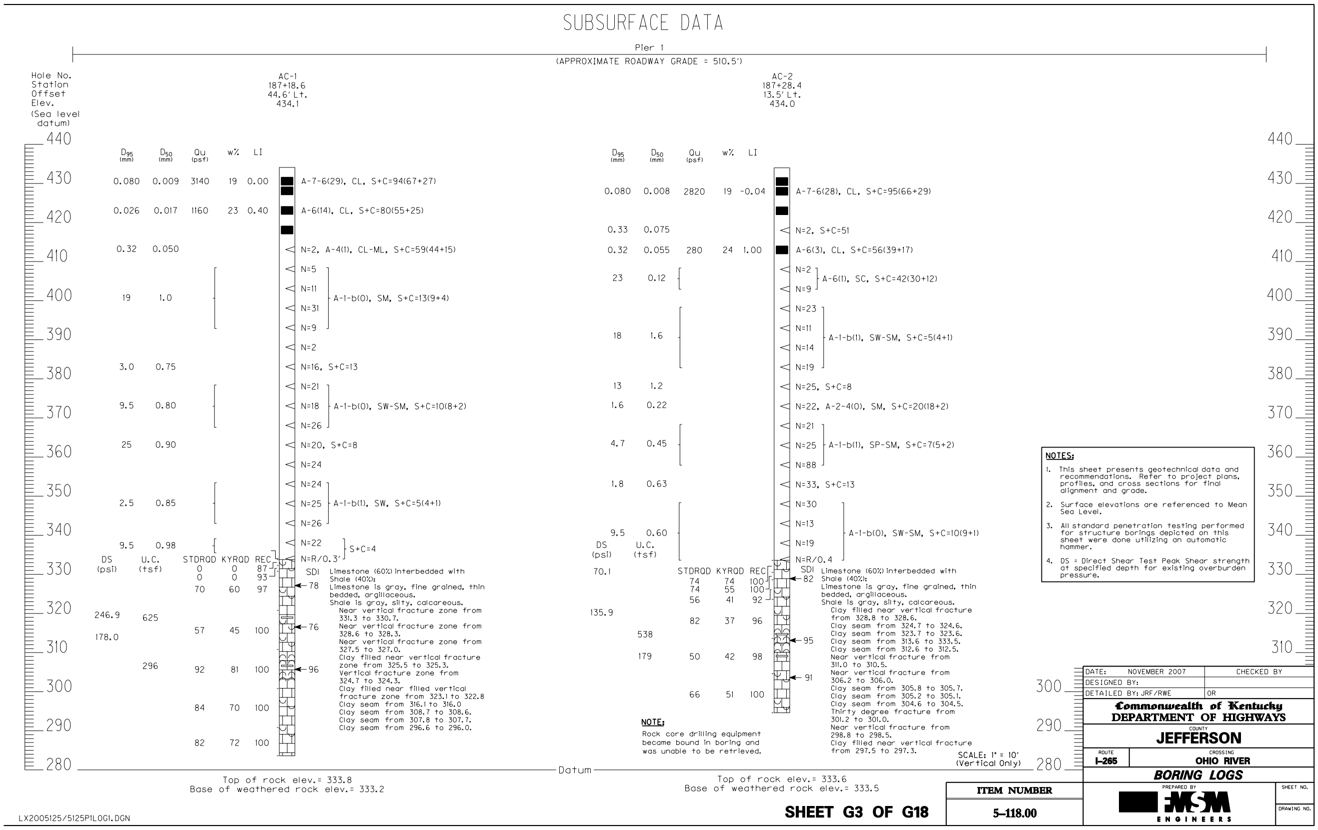
DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	KJS/TD/OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY
JEFFERSON
ROUTE **I-265** CROSSING **OHIO RIVER**
BORING LOCATION PLANS
PREPARED BY MSM ENGINEERS
SHEET NO.
DRAWING NO.

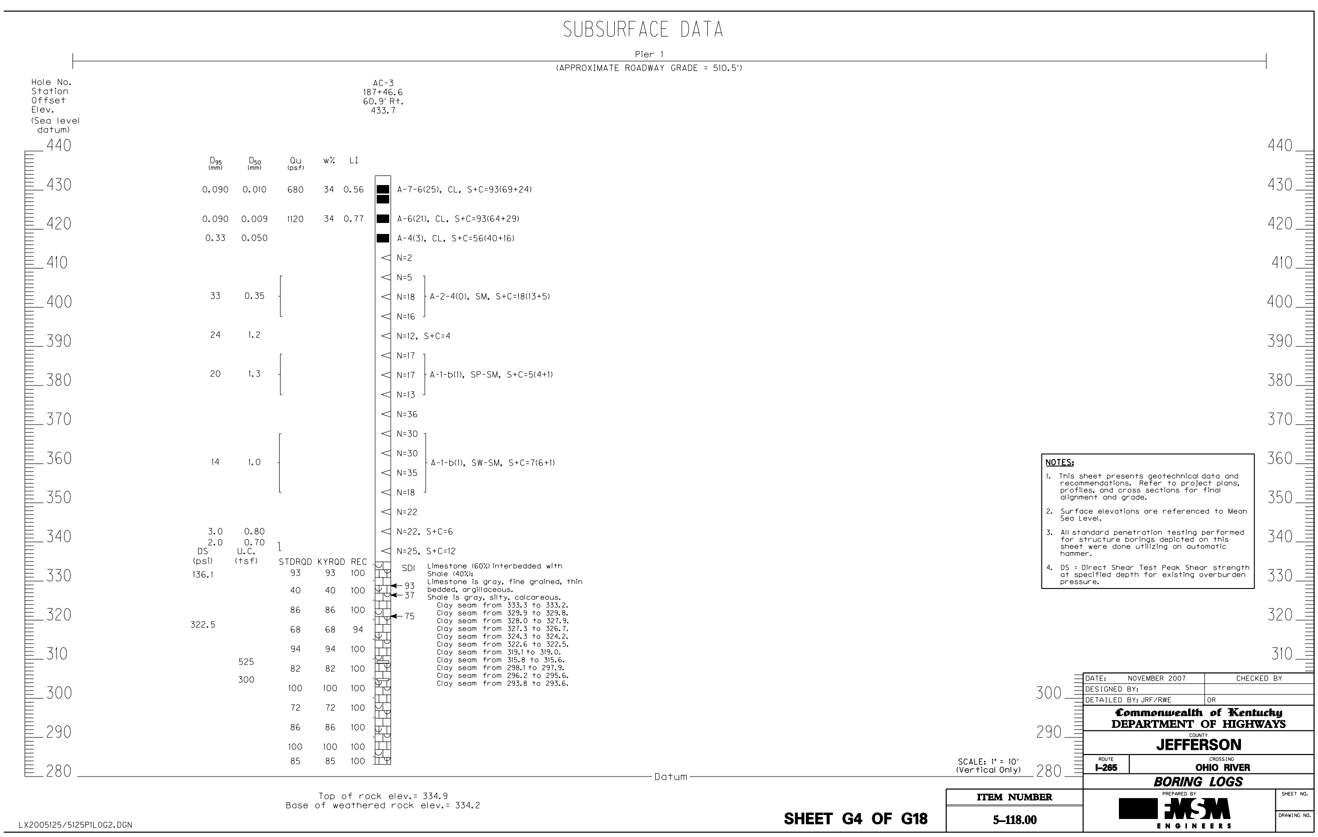
SHEET G2 OF G18

ITEM NUMBER
5-118.00

LX2005125/5125LAYO.DGN



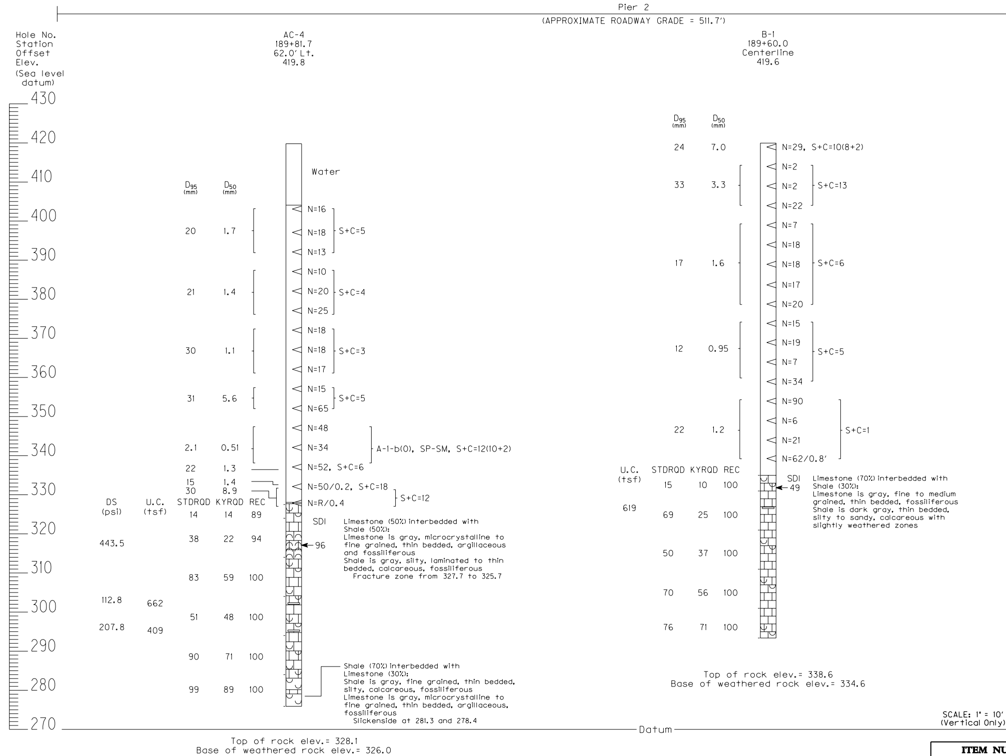
SUBSURFACE DATA
Pier 1
(APPROXIMATE ROADWAY GRADE = 510.5')
Hole No. Station Offset Elev. (Sea level datum)
AC-1 187+18.6 44.6' Lt. 434.1
AC-2 187+28.4 13.5' Lt. 434.0
D95 (mm) D50 (mm) Qu (psf) w% LI
0.080 0.009 3140 19 0.00 A-7-6(29), CL, S+C=94(67+27)
0.026 0.017 1160 23 0.40 A-6(14), CL, S+C=80(55+25)
0.32 0.050 N=2, A-4(1), CL-ML, S+C=59(44+15)
19 1.0 N=5 N=11 N=31 N=9 A-1-b(0), SM, S+C=13(9+4)
N=2
3.0 0.75 N=16, S+C=13
9.5 0.80 N=21 N=18 N=26 A-1-b(0), SW-SM, S+C=10(8+2)
25 0.90 N=20, S+C=8
N=24
2.5 0.85 N=24 N=25 N=26 A-1-b(1), SW, S+C=5(4+1)
9.5 0.98 N=22 N=R/0.3' S+C=4
DS (psi) U.C. (tsf) STDRQD KYRQD REC SDI
Limestone (60%) interbedded with Shale (40%): Limestone is gray, fine grained, thin bedded, argillaceous. Shale is gray, silty, calcareous. Near vertical fracture zone from 331.3 to 330.7. Near vertical fracture zone from 328.6 to 328.3. Near vertical fracture zone from 327.5 to 327.0. Clay filled near vertical fracture zone from 325.5 to 325.3. Vertical fracture zone from 324.7 to 324.3. Clay filled near filled vertical fracture zone from 323.1 to 322.8. Clay seam from 316.1 to 316.0. Clay seam from 308.7 to 308.6. Clay seam from 307.8 to 307.7. Clay seam from 296.6 to 296.0.
Top of rock elev.= 333.8
Base of weathered rock elev.= 333.2
0.080 0.008 2820 19 -0.04 A-7-6(28), CL, S+C=95(66+29)
0.33 0.075 N=2, S+C=51
0.32 0.055 280 24 1.00 A-6(3), CL, S+C=56(39+17)
23 0.12 N=2 N=9 A-6(1), SC, S+C=42(30+12)
18 1.6 N=23 N=11 N=14 N=19 A-1-b(1), SW-SM, S+C=5(4+1)
13 1.2 N=25, S+C=8
1.6 0.22 N=22, A-2-4(0), SM, S+C=20(18+2)
4.7 0.45 N=21 N=25 N=88 A-1-b(1), SP-SM, S+C=7(5+2)
1.8 0.63 N=33, S+C=13
9.5 0.60 N=30 N=13 N=19 A-1-b(0), SW-SM, S+C=10(9+1)
N=R/0.4
Limestone (60%) interbedded with Shale (40%): Limestone is gray, fine grained, thin bedded, argillaceous. Shale is gray, silty, calcareous. Clay filled near vertical fracture from 328.8 to 328.6. Clay seam from 324.7 to 324.6. Clay seam from 323.7 to 323.6. Clay seam from 313.6 to 333.5. Clay seam from 312.6 to 312.5. Near vertical fracture from 311.0 to 310.5. Near vertical fracture from 306.2 to 306.0. Clay seam from 305.8 to 305.7. Clay seam from 305.2 to 305.1. Clay seam from 304.6 to 304.5. Thirty degree fracture from 301.2 to 301.0. Near vertical fracture from 298.8 to 298.5. Clay filled near vertical fracture from 297.5 to 297.3.
NOTE: Rock core drilling equipment became bound in boring and was unable to be retrieved.
Top of rock elev.= 333.6
Base of weathered rock elev.= 333.5
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007 CHECKED BY
DESIGNED BY:
DETAILED BY: JRF/RWE OR
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY JEFFERSON
ROUTE I-265 CROSSING OHIO RIVER
BORING LOGS
PREPARED BY MSM ENGINEERS
ITEM NUMBER 5-118.00
SHEET G3 OF G18
LX2005125/5125P1LOG1.DGN



SUBSURFACE DATA
Pier 1
(APPROXIMATE ROADWAY GRADE = 510.5')
Hole No. Station Offset Elev. (Sea level datum)
AC-3 187+46.6 60.9' Rt. 433.7
D95 (mm) D50 (mm) Qu (psf) w% LI
0.090 0.010 680 34 0.56 A-7-6(25), CL, S+C=93(69+24)
0.090 0.009 1120 34 0.77 A-6(21), CL, S+C=93(64+29)
0.33 0.050 A-4(3), CL, S+C=56(40+16)
N=2
33 0.35 N=5 N=18 N=16 A-2-4(0), SM, S+C=18(13+5)
24 1.2 N=12, S+C=4
20 1.3 N=17 N=17 N=13 A-1-b(1), SP-SM, S+C=5(4+1)
N=36
14 1.0 N=30 N=30 N=35 N=18 A-1-b(1), SW-SM, S+C=7(6+1)
N=22
3.0 0.80 N=22, S+C=6
2.0 0.70 N=25, S+C=12
DS (psi) U.C. (tsf) STDRQD KYRQD REC SDI
Limestone (60%) interbedded with Shale (40%): Limestone is gray, fine grained, thin bedded, argillaceous. Shale is gray, silty, calcareous. Clay seam from 333.3 to 333.2. Clay seam from 329.9 to 329.8. Clay seam from 328.0 to 327.9. Clay seam from 327.3 to 326.7. Clay seam from 324.3 to 324.2. Clay seam from 322.6 to 322.5. Clay seam from 319.1 to 319.0. Clay seam from 315.8 to 315.6. Clay seam from 298.1 to 297.9. Clay seam from 296.2 to 295.6. Clay seam from 293.8 to 293.6.
Top of rock elev.= 334.9
Base of weathered rock elev.= 334.2
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007 CHECKED BY
DESIGNED BY:
DETAILED BY: JRF/RWE OR
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY JEFFERSON
ROUTE I-265 CROSSING OHIO RIVER
BORING LOGS
PREPARED BY MSM ENGINEERS
ITEM NUMBER 5-118.00
SHEET G4 OF G18
LX2005125/5125P1LOG2.DGN

SUBSURFACE DATA

Pier 2
(APPROXIMATE ROADWAY GRADE = 511.7')



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.

SCALE: 1" = 10'
(Vertical Only)

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY **JEFFERSON**

ROUTE **I-265** | CROSSING **OHIO RIVER**

BORING LOGS

PREPARED BY FMSM ENGINEERS

SHEET NO.

DRAWING NO.

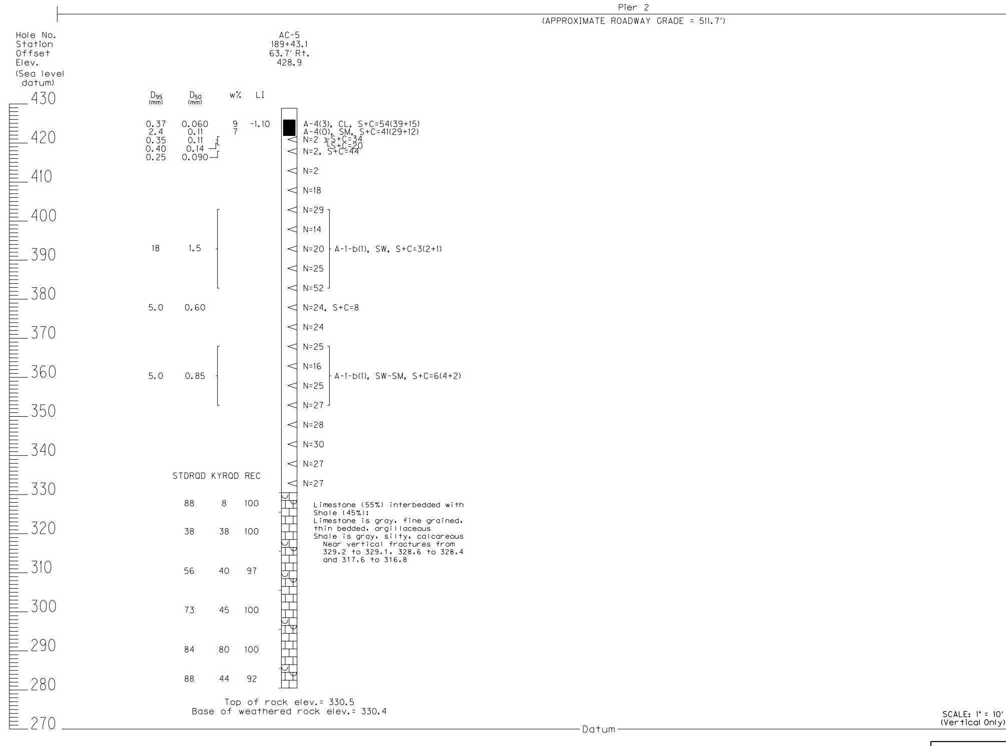
ITEM NUMBER 5-118.00

SHEET G5 OF G18

LX2005125/5125P2LOG1.DGN

SUBSURFACE DATA

Pier 2
(APPROXIMATE ROADWAY GRADE = 511.7')



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.

SCALE: 1" = 10'
(Vertical Only)

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY **JEFFERSON**

ROUTE **I-265** | CROSSING **OHIO RIVER**

BORING LOGS

PREPARED BY FMSM ENGINEERS

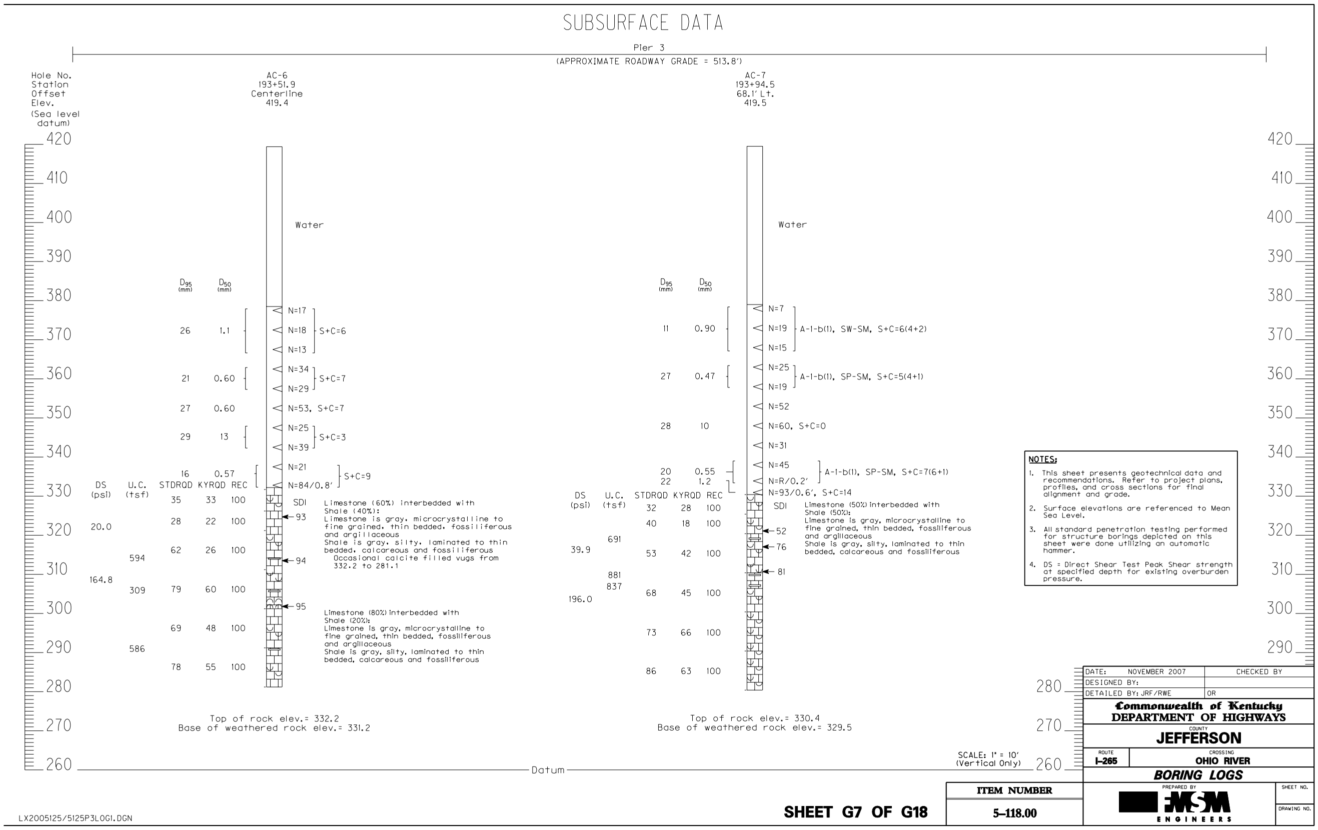
SHEET NO.

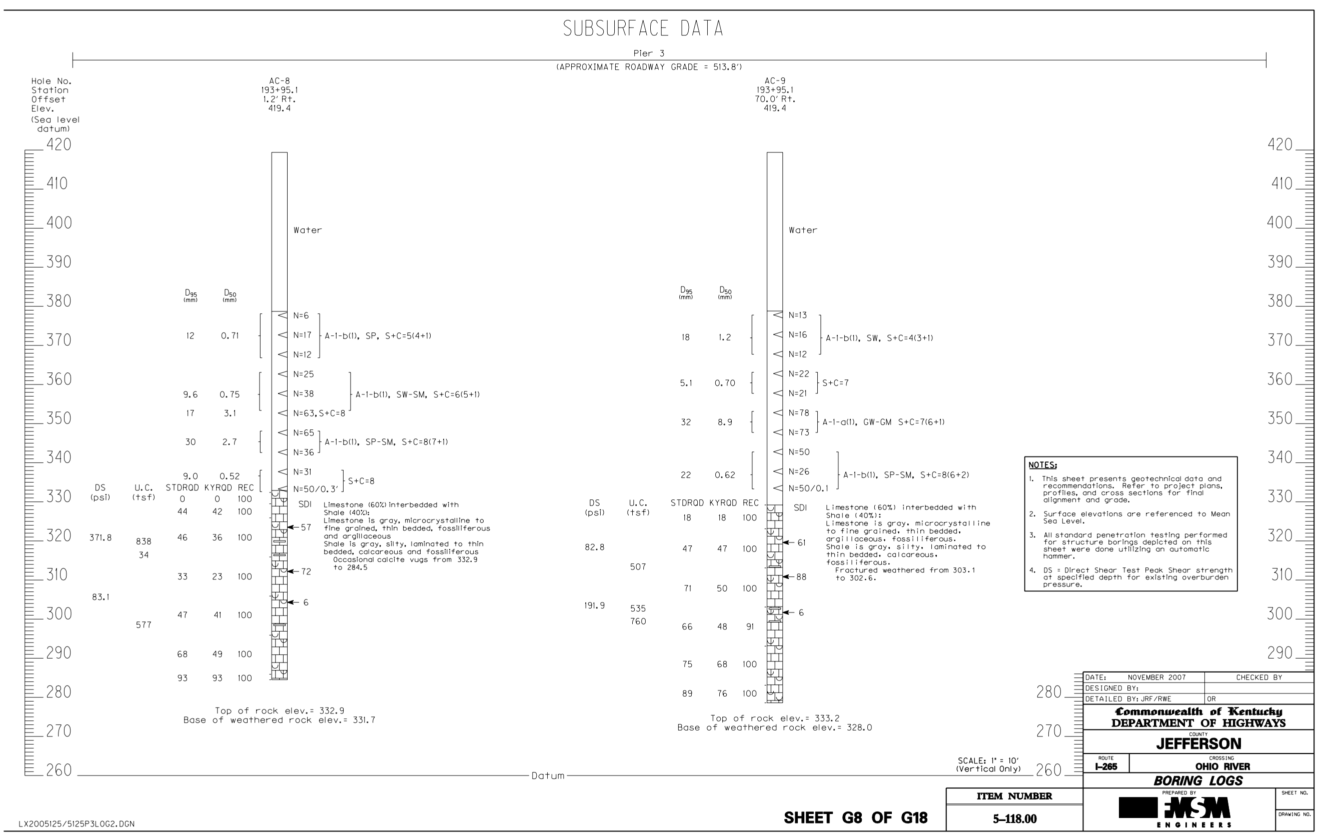
DRAWING NO.

ITEM NUMBER 5-118.00

SHEET G6 OF G18

LX2005125/5125P2LOG2.DGN

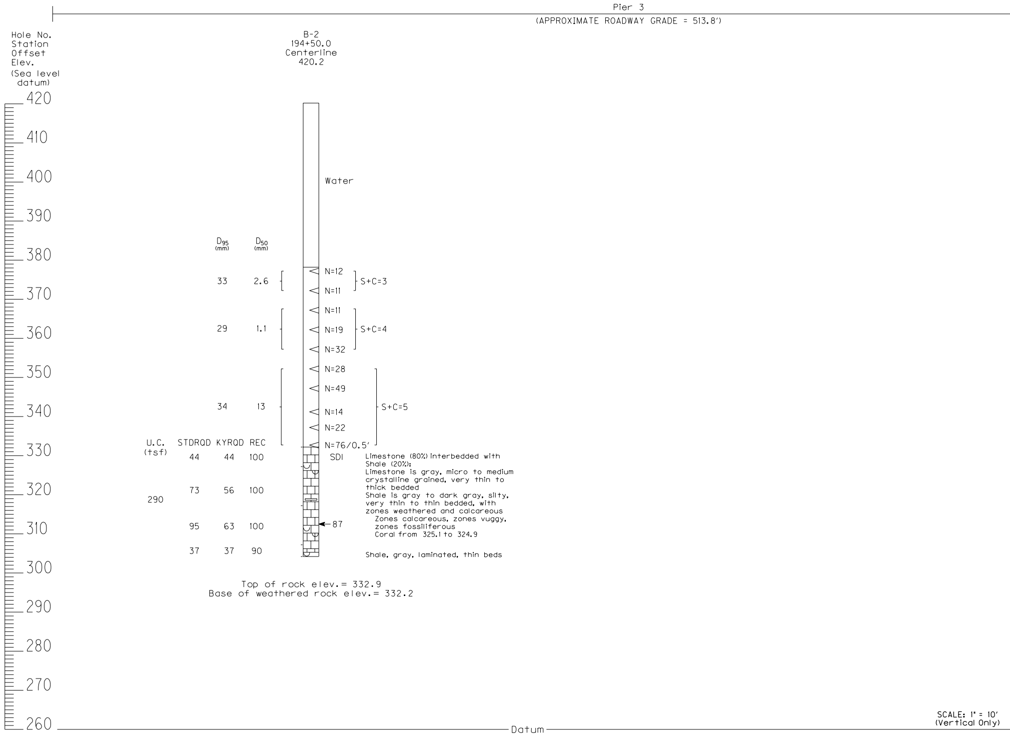

SUBSURFACE DATA
Pier 3
(APPROXIMATE ROADWAY GRADE = 513.8')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-6
193+51.9
Centerline
419.4
AC-7
193+94.5
68.1' Lt.
419.5
Water
D95 (mm)
D50 (mm)
N=17
26
1.1
N=18
S+C=6
N=13
N=34
21
0.60
S+C=7
N=29
27
0.60
N=53, S+C=7
N=25
29
13
S+C=3
N=39
N=21
16
0.57
S+C=9
N=84/0.8'
DS (psi)
U.C. (tsf)
STDRQD KYRQD REC
35 33 100
SDI
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
Occasional calcite filled vugs from 332.2 to 281.1
93
20.0
28 22 100
594
62 26 100
94
164.8
309
79 60 100
95
Limestone (80%) interbedded with Shale (20%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
69 48 100
586
78 55 100
Top of rock elev.= 332.2
Base of weathered rock elev.= 331.2
N=7
11
0.90
N=19
A-1-b(1), SW-SM, S+C=6(4+2)
N=15
N=25
27
0.47
A-1-b(1), SP-SM, S+C=5(4+1)
N=19
N=52
28
10
N=60, S+C=0
N=31
N=45
20
0.55
22
1.2
A-1-b(1), SP-SM, S+C=7(6+1)
N=R/0.2'
N=93/0.6', S+C=14
DS (psi)
U.C. (tsf)
STDRQD KYRQD REC
32 28 100
SDI
Limestone (50%) interbedded with Shale (50%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
40 18 100
52
691
76
39.9
53 42 100
881
81
837
196.0
68 45 100
73 66 100
86 63 100
Top of rock elev.= 330.4
Base of weathered rock elev.= 329.5
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
270
260
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007
CHECKED BY
DESIGNED BY:
DETAILED BY: JRF/RWE
OR
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY
JEFFERSON
ROUTE
I-265
CROSSING
OHIO RIVER
BORING LOGS
PREPARED BY
ENGINEERS
SHEET NO.
DRAWING NO.
ITEM NUMBER
5-118.00
SHEET G7 OF G18
LX2005125/5125P3LOG1.DGN


SUBSURFACE DATA
Pier 3
(APPROXIMATE ROADWAY GRADE = 513.8')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-8
193+95.1
1.2' Rt.
419.4
AC-9
193+95.1
70.0' Rt.
419.4
Water
D95 (mm)
D50 (mm)
N=6
12
0.71
N=17
A-1-b(1), SP, S+C=5(4+1)
N=12
N=25
9.6
0.75
N=38
A-1-b(1), SW-SM, S+C=6(5+1)
17
3.1
N=63,S+C=8
N=65
30
2.7
A-1-b(1), SP-SM, S+C=8(7+1)
N=36
N=31
9.0
0.52
S+C=8
N=50/0.3'
DS (psi)
U.C. (tsf)
STDRQD KYRQD REC
0 0 100
44 42 100
SDI
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
Occasional calcite vugs from 332.9 to 284.5
57
371.8
838
34
46 36 100
72
33 23 100
83.1
6
47 41 100
577
68 49 100
93 93 100
Top of rock elev.= 332.9
Base of weathered rock elev.= 331.7
N=13
18
1.2
N=16
A-1-b(1), SW, S+C=4(3+1)
N=12
N=22
5.1
0.70
S+C=7
N=21
N=78
32
8.9
A-1-a(1), GW-GM S+C=7(6+1)
N=73
N=50
22
0.62
N=26
A-1-b(1), SP-SM, S+C=8(6+2)
N=50/0.1
DS (psi)
U.C. (tsf)
STDRQD KYRQD REC
18 18 100
SDI
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, thin bedded, argillaceous, fossiliferous.
Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.
Fractured weathered from 303.1 to 302.6.
61
82.8
47 47 100
507
88
71 50 100
191.9
535
760
6
66 48 91
75 68 100
89 76 100
Top of rock elev.= 333.2
Base of weathered rock elev.= 328.0
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
270
260
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007
CHECKED BY
DESIGNED BY:
DETAILED BY: JRF/RWE
OR
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY
JEFFERSON
ROUTE
I-265
CROSSING
OHIO RIVER
BORING LOGS
PREPARED BY
ENGINEERS
SHEET NO.
DRAWING NO.
ITEM NUMBER
5-118.00
SHEET G8 OF G18
LX2005125/5125P3LOG2.DGN

SUBSURFACE DATA

Pier 3

(APPROXIMATE ROADWAY GRADE = 513.8')



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY: **JEFFERSON**

ROUTE: **I-265** — CROSSING: **OHIO RIVER**

BORING LOGS

PREPARED BY MSM ENGINEERS

SHEET NO. / DRAWING NO.

ITEM NUMBER: **5-118.00**

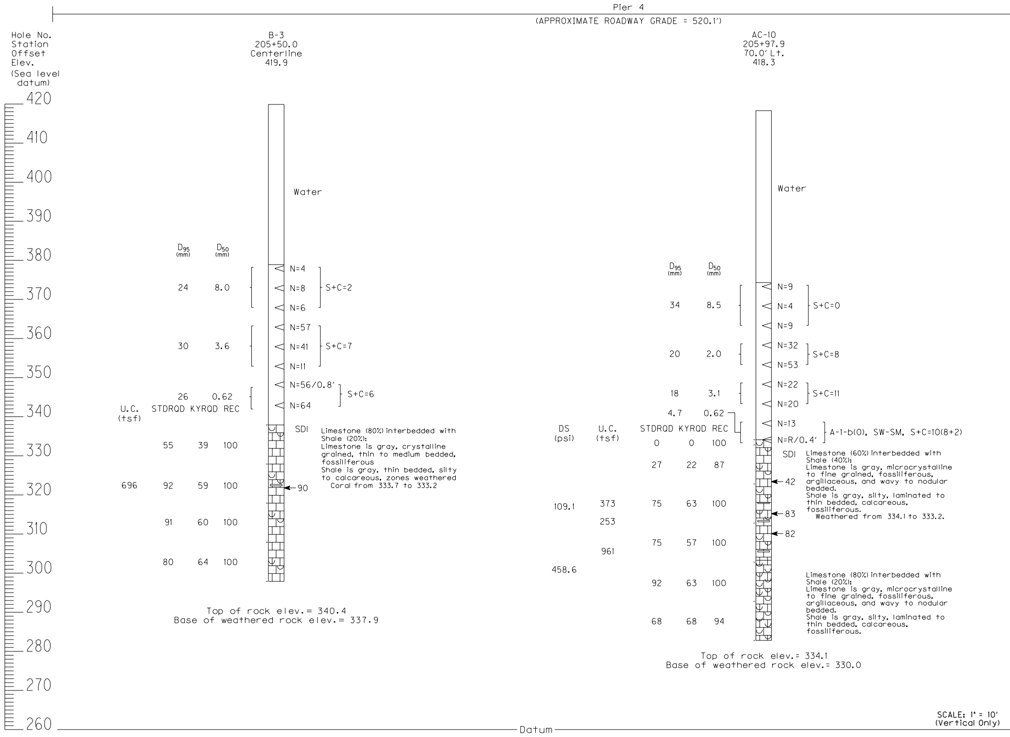
SHEET G9 OF G18

LX2005125/5125P3LOG3.DGN

SUBSURFACE DATA

Pier 4

(APPROXIMATE ROADWAY GRADE = 520.1')



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY: **JEFFERSON**

ROUTE: **I-265** — CROSSING: **OHIO RIVER**

BORING LOGS

PREPARED BY MSM ENGINEERS

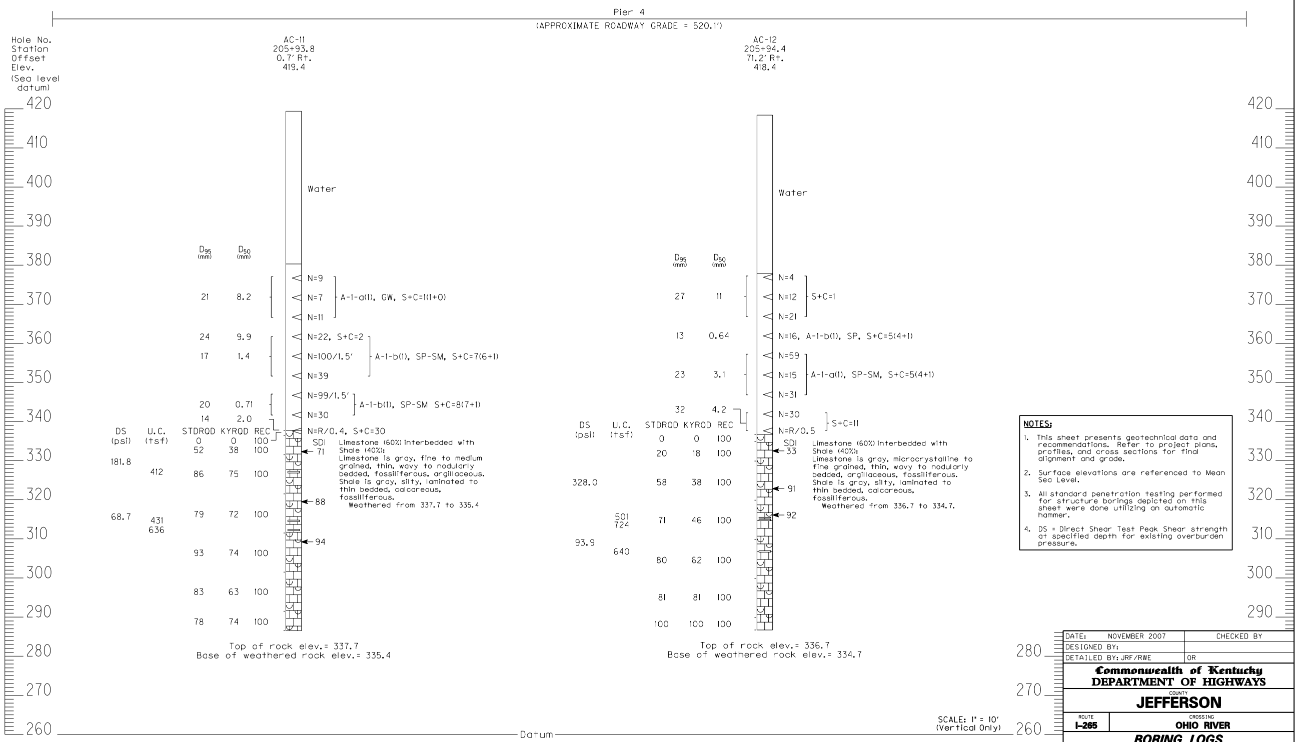
SHEET NO. / DRAWING NO.

ITEM NUMBER: **5-118.00**

SHEET G10 OF G18

LX2005125/5125P4LOG1.DGN

SUBSURFACE DATA



DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I-265** CROSSING **OHIO RIVER**

BORING LOGS

PREPARED BY MSM ENGINEERS

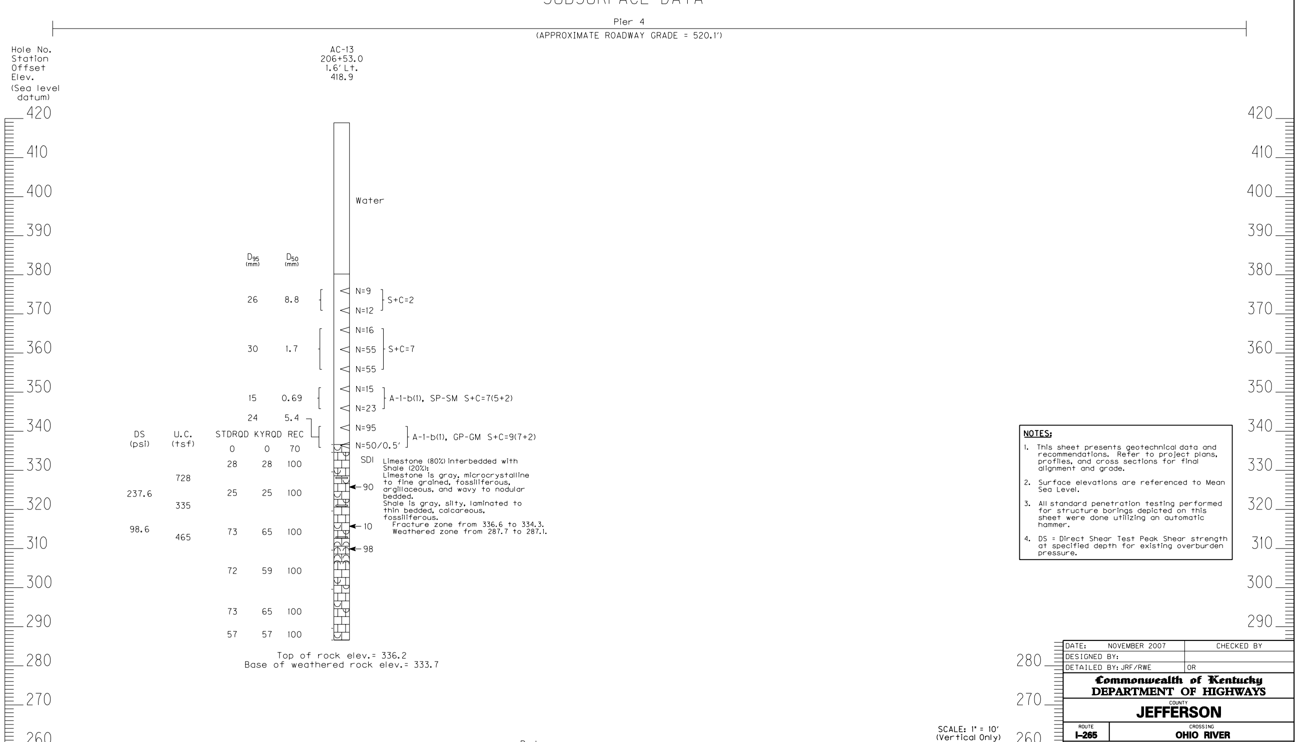
SHEET NO. DRAWING NO.

ITEM NUMBER **5-118.00**

SHEET G11 OF G18

LX2005125/5125P4LOG2.DGN

SUBSURFACE DATA



DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I-265** CROSSING **OHIO RIVER**

BORING LOGS

PREPARED BY MSM ENGINEERS

SHEET NO. DRAWING NO.

ITEM NUMBER **5-118.00**

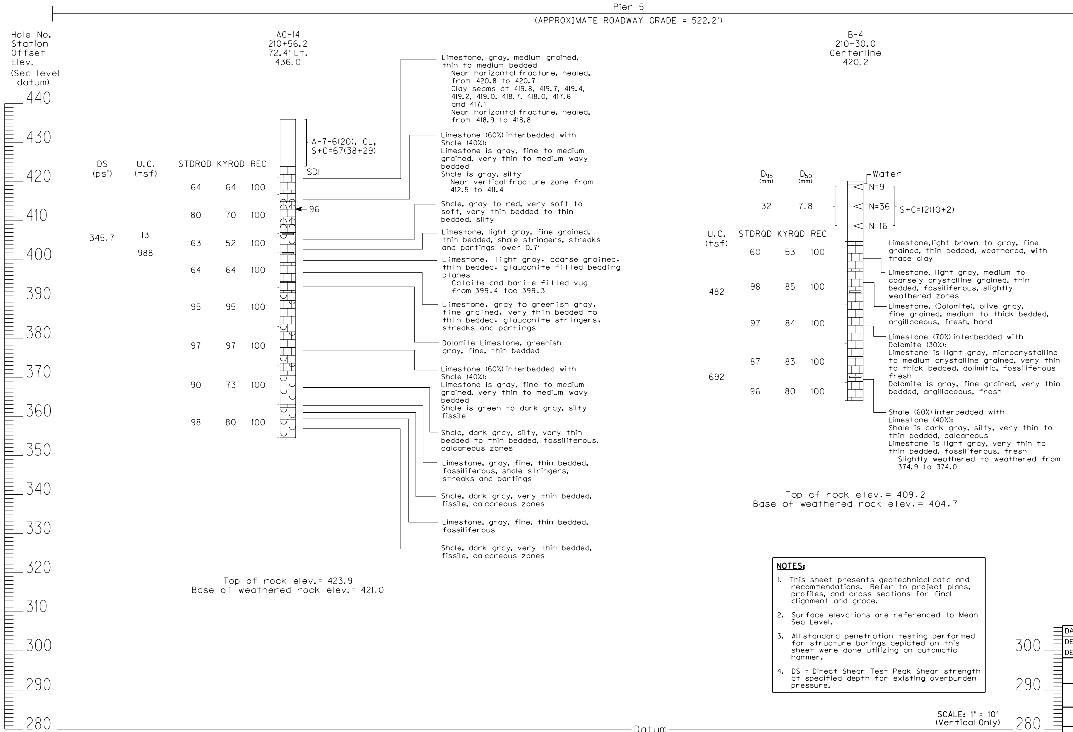
SHEET G12 OF G18

LX2005125/5125P4LOG3.DGN

SUBSURFACE DATA

Pier 5

(APPROXIMATE ROADWAY GRADE = 522.2')



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.

SCALE: 1" = 10' (Vertical Only)

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY **JEFFERSON**

ROUTE **I-265** | CROSSING **OHIO RIVER**

BORING LOGS

PREPARED BY MSM ENGINEERS | SHEET NO. | DRAWING NO.

SHEET G13 OF G18

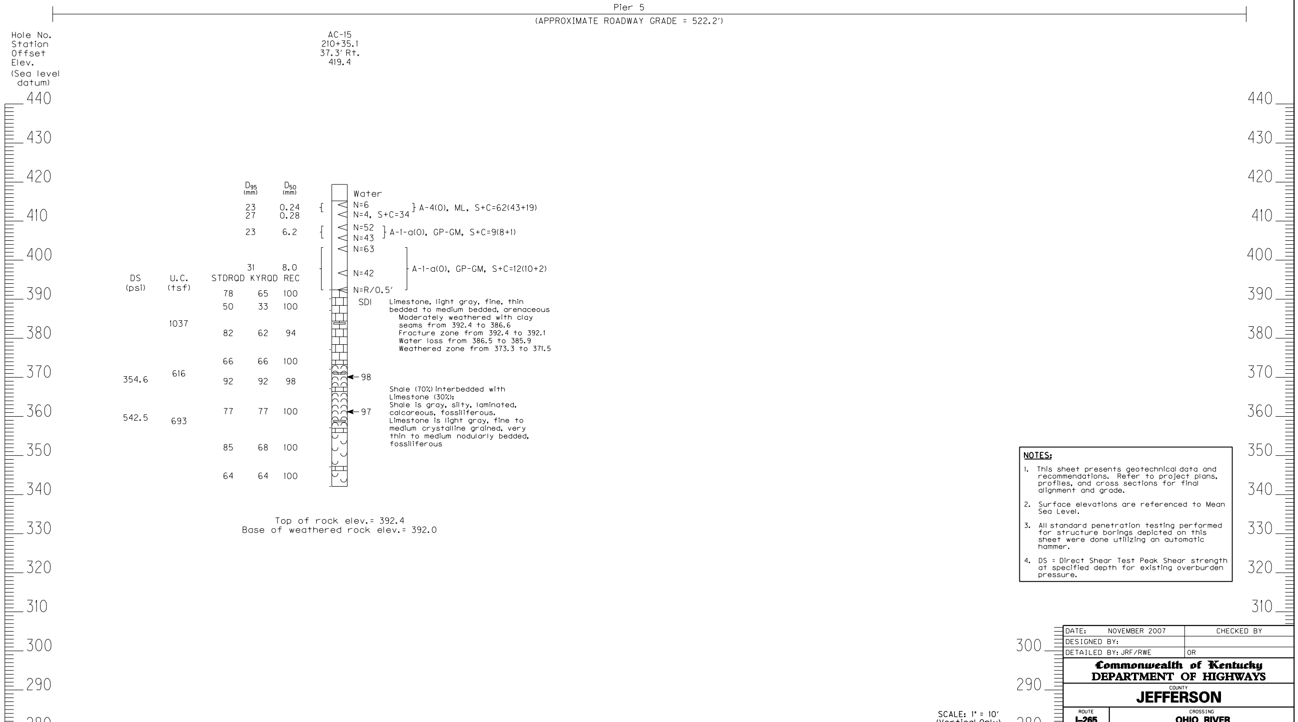
ITEM NUMBER **5-118.00**

LX2005125/5125P5LOG1.DGN

SUBSURFACE DATA

Pier 5

(APPROXIMATE ROADWAY GRADE = 522.2')



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.

SCALE: 1" = 10' (Vertical Only)

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY **JEFFERSON**

ROUTE **I-265** | CROSSING **OHIO RIVER**

BORING LOGS

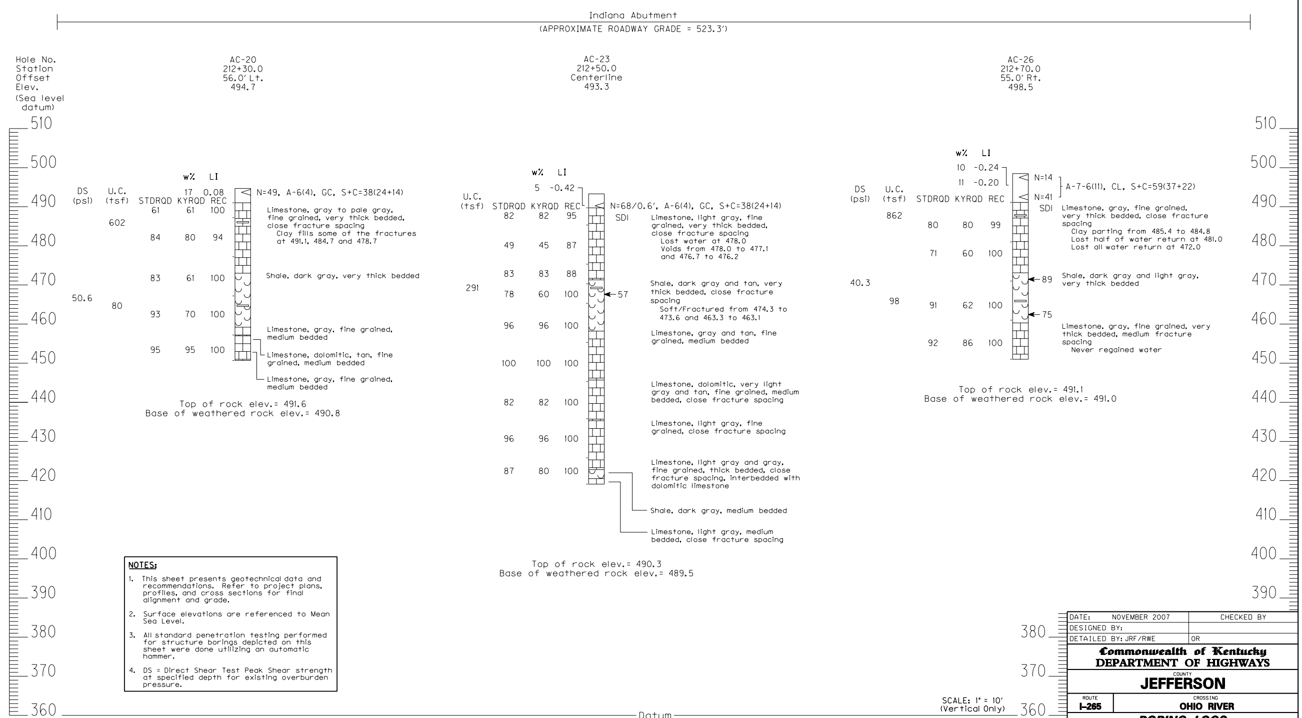
PREPARED BY MSM ENGINEERS | SHEET NO. | DRAWING NO.

SHEET G14 OF G18

ITEM NUMBER **5-118.00**

LX2005125/5125P5LOG2.DGN

SUBSURFACE DATA



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

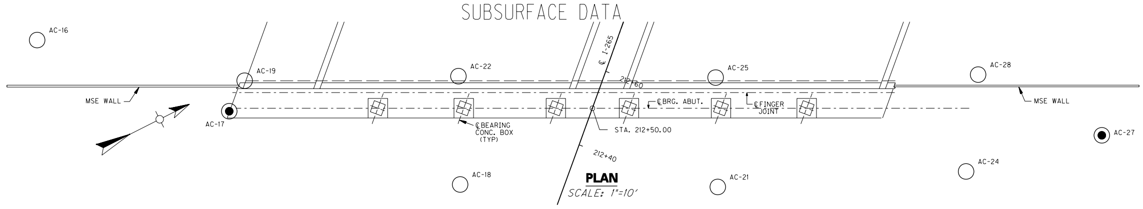
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY **JEFFERSON**
ROUTE **I-265** CROSSING **OHIO RIVER**
BORING LOGS
PREPARED BY MSM ENGINEERS

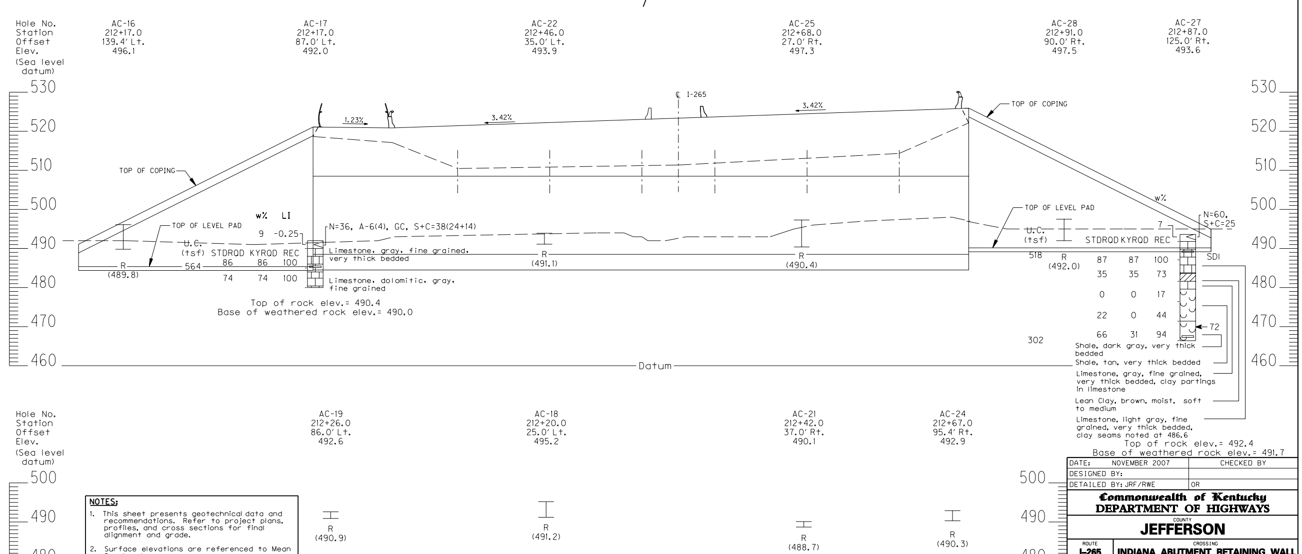
ITEM NUMBER 5-118.00

SHEET G15 OF G18

LX2005125/ABUTMENTLOG1.DGN

SUBSURFACE DATA





NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	OR

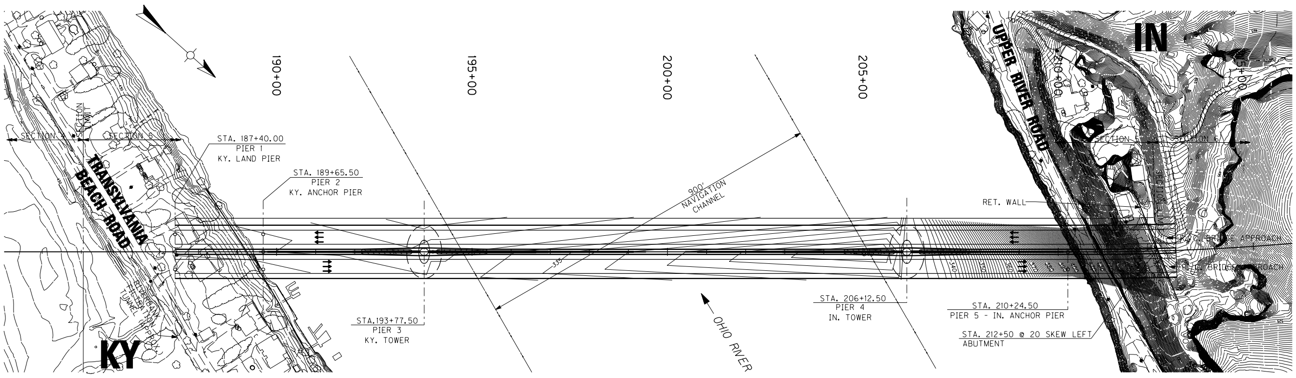
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY **JEFFERSON**
ROUTE **I-265** CROSSING **INDIANA ABUTMENT RETAINING WALL**
PLAN, PROFILE & BORING LOGS
PREPARED BY MSM ENGINEERS

ITEM NUMBER 5-118.00

SHEET G16 OF G18

LX2005125/RWPROF.DGN

SUBSURFACE DATA



100 0 100 200 300
GRAPHIC SCALE IN FEET

SCALE: 1" = 100'

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	KJS/TD

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I-265** CROSSING **OHIO RIVER**

BEDROCK SURFACE CONTOURS

PREPARED BY FMSM ENGINEERS

SHEET NO. DRAWING NO.

ITEM NUMBER
5-118.00

SHEET G17 OF G18

LX2005125/5125BEDROCK.DGN

SUBSURFACE DATA

I-265 BRIDGE PROJECT STRATIGRAPHIC COLUMN

SYSTEM	SERIES	FORMATION		THICKNESS (FT.)	CONTACT ELEVATIONS
DEVONIAN	MIDDLE DEVONIAN	SELLERSBURG LIMESTONE	BEECHWOOD LIMESTONE MEMBER Description: Limestone, light to greenish gray, micritic to fine grained, very thin to thin bedded,unconforimity at base marked by quartz sand and phosphatic pebbles in dark-gray shale seam, weathers moderate yellowish brown, light to medium brown and light olive gray	2'-10'	Unconformity 490'
DEVONIAN	MIDDLE DEVONIAN	SELLERSBURG LIMESTONE	SILVER CREEK LIMESTONE MEMBER Description: Limestone, light-bluish-olive and light greenish gray, fine grained, argillaceous, dolomitic, fossiliferous, basal contact marked by phosphatic pebbles, quartz sand, pyrite and glauconite; Unconformable on underlying Jeffersonville Limestone	0'-15'	Unconformity 480'
DEVONIAN	LOWER & MIDDLE DEVONIAN	JEFFERSONVILLE LIMESTONE	Description: Limestone, brownish to light gray, medium to coarse grained, very thin to thin bedded (upper section), thick bedded (lower section), weathers pale yellowish brown to light yellowish gray, fossiliferous, dolomitic	18'-30'	Unconformity 465'
SILURIAN	MIDDLE SILURIAN	LOUISVILLE LIMESTONE	Description: Limestone (Dolomitic), medium gray to light olive gray, fine to medium crystalline grained, thin to very thin bedded (upper section), thick bedded (lower section), weathers light gray to buff, fossiliferous	45'-75'	420' - 430'
SILURIAN	MIDDLE SILURIAN	WALDRON SHALE	Description: Shale, light brownish gray to dark greenish gray, weathers light gray to yellowish gray, silty, dolomitic, pyritic, typically weathers to undercut above lying rock unit	9'-13'	410'
SILURIAN	MIDDLE SILURIAN	LAUREL DOLOMITE	Description: UPPER SECTION: Limestone (Dolomite), greenish gray to light olive gray, micrograined to very fine crystalline grained, thick bedded, calcite in irregular patches up to 0.8' wide and 0.3' thick, comprises two fifths of unit LOWER SECTION: Limestone (Dolomite), greenish gray to light olive gray, micrograined to very fine crystalline grained, massively bedded, porous, mottled; Two bedding sets are separated by a dark to olive gray dolimitic clay shale that ranges from 0.8' to 2.5' in thickness and is located from 5' to 8' above the basal contact with the Osgood Formation.	~50'	370'
SILURIAN	MIDDLE SILURIAN	OSGOOD	Description: Limestone (Dolomite) (60%) interbedded with Shale (40%) Limestone is dusky yellowish green to grayish green to olive gray, fine to medium grained, very thin to thin bedded, weathers dark yellowish orange to grayish orange. Shale is green to dark gray, fissile, silty and dolomitic.	17'	350'
SILURIAN	LOWER SILURIAN	BRASSFIELD	Description: Limestone, grayish orange to pale yellowish brown, fine to coarse crystalline grained, thin to thick bedded, dolomitic and glauconitic zones present, thin chert lenses also noted	3'-7'	345'
ORDOVICIAN	UPPER ORDOVICIAN	DRAKES	SALUDA DOLOMITE MEMBER Description: Limestone (Dolomite), greenish gray, very finely crystalline grained, glauconitic, weathers yellowish gray, fossiliferous zones	40'-48'	310'
ORDOVICIAN	UPPER ORDOVICIAN	DRAKES	BARDSTOWN MEMEBER Description: Limestone interbedded with Mudstone Limestone is light olive gray with brownish cast, medium to coarse grained, fossiliferous, weathers medium light gray to very light gray. Mudstone is medium dark gray, light olive gray to light greenish gray, mainly carbonate mud with some clay.	35'-44'	280'
ORDOVICIAN	UPPER ORDOVICIAN	DRAKES	ROWLAND MEMBER Description: Limestone (60%) interbedded with Shale (40%) Limestone is olive gray to grayish green, very fine grained to micrograined, dolomitic, glauconitic, weathers light gray to yellowish greenish gray. Shale is medium dark gray to olive gray, dolomitic, weathers light gray, silty, fossiliferous.	20+'	260'

NOTES:

1. Accuracy of elevations at rock contacts were calculated to an accuracy of 20 feet due to the amount of unconformities observed and the accuracy of formation thicknesses from USGS Geologic Quadrangles.

2. All thicknesses were obtained from the KGS Maps 1974 "Geologic Map of parts of the Jeffersonville, New Albany, and Charlestown Quadrangles, Kentucky-Indiana", Author Roy C. Kepferle and 1971 Geologic Map of the Anchorage Quadrangle, Jeferson and Oldham Counties, Kentucky, Authors Roy C. Kepferle, Perry B. Wigley and B. Ray Hawke.

SOURCES:

1. FMSM Observations from Subsurface Explorations (2005 and 2007) and Geologic Mapping (2007)

2. Geologic Quadrangle Map Jeffersonville, New Albany and Charlestown Quadrangles, Kentucky-Indiana", GQ-1211 USGS 1974, Author Roy C. Kepferle.

3. Geologic Map Anchorage Quadrangle, Kentucky GQ-906 USGS 1971, Authors Roy C. Kepferle, Perry B. Wigley and B. Ray Hawke.

SCALE: 1" = 10'
(Vertical Only)

DATE: NOVEMBER 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF/RWE	DR

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I-265** CROSSING **OHIO RIVER**

STRATIGRAPHIC COLUMN

PREPARED BY FMSM ENGINEERS

SHEET NO. DRAWING NO.

ITEM NUMBER
5-118.00

SHEET G18 OF G18

LX2005125/5125STRATLOG.DGN

APPENDIX B
COORDINATE DATA SUBMISSION FORM

COORDINATE DATA SUBMISSION FORM
KYTC DIVISION OF MATERIALS - GEOTECHNICAL BRANCH

County: Jefferson, Kentucky/Clark, Indiana

Road Number: I-265 Over Ohio River

Survey Crew / Consultant: HDR/Quest, Inc.

Contact Person: Kelly Meyer

Item No.: 5-118.00

Mars No.: N/A

Project No.: N/A

Date: November 1, 2007

Notes:

(select one) **Elevation Datum** **Sea Level** **Assumed**

HOLE NUMBER	STATION	OFFSET	ELEVATION (ft)	LATITUDE	LONGITUDE
AC-1	187+18.6	44.6 L	434.1	38.339960690	85.640025200
AC-2	187+28.4	13.5 L	434.0	38.340038260	85.639968700
AC-3	187+46.6	60.9 R	433.7	38.340213490	85.639820600
AC-4	189+81.7	62.0 L	419.8	38.340460380	85.640690600
AC-5	189+43.1	63.7 R	428.0	38.340615920	85.640277100
AC-6	193+51.9	CL	419.4	38.341324310	85.641404200
AC-7	193+94.5	68.1 L	419.5	38.341283310	85.641680300
AC-8	193+95.1	1.2 R	419.4	38.341413520	85.641503800
AC-9	193+95.2	70.0 R	419.4	38.341541430	85.641327400
AC-10	205+97.9	70.0 L	418.3	38.343712750	85.644524500
AC-11	205+93.8	0.7 R	419.4	38.343835740	85.644333300
AC-12	205+94.4	71.2 R	418.4	38.343967810	85.644153800
AC-13	206+53.0	1.6 L	418.9	38.343951290	85.644478800
AC-14	210+56.2	72.4 L	436.0	38.344634740	85.645612100
AC-15	210+35.1	37.3 R	419.4	38.344766150	85.645245900
AC-16	212+17.0	139.4 L	496.1	38.344835410	85.646148600
AC-17	212+17.0	87.0 L	492.0	38.344926570	85.646027800
AC-18	212+20.0	25.0 L	495.2	38.345050280	85.645879000
AC-19	212+26.0	86.0 L	492.6	38.344946000	85.646046800
AC-20	212+30.0	56.0 L	494.7	38.345010970	85.645981000
AC-21	212+42.0	37.0 R	490.1	38.345211960	85.645776500
AC-22	212+46.0	35.0 L	493.9	38.345082320	85.645966700
AC-23	212+50.0	0.0 R	493.3	38.345157190	85.645888700
AC-24	212+67.0	95.4 R	492.9	38.345382280	85.645680300
AC-25	212+68.0	27.0 R	497.3	38.345244270	85.645865100
AC-26	212+70.01	55.0 R	498.5	38.345302000	85.645800100
AC-27	212+87.0	125.0 R	493.6	38.345470240	85.645667800
AC-28	212+91.0	90.0 R	497.5	38.345410840	85.645764700

APPENDIX C
GEOLOGIC MAPPING OF ROCK EXPOSURES

SITE #1 - BASE

N 38° 20' 42.2"
W 85° 38' 46.5"
Elevation: 495'

Site of known elevation used for instrument calibration and daily safety briefings

OUTCROP #2 - WEST OLD QUARRY WALL

N 38° 20' 43.6"
W 85° 38' 48.4"
Elevation: 479'

Discontinuity Data
Strikes Range: N 12° E to N 80° W
Dips Range: 83° S to 88° S
Description: See Data Sheet 1 of 1

Rock Unit Exposed
Jeffersonville Limestone

OUTCROP #3 - SOLUTION FEATURE RIDGE

N 38° 20' 44.7"
W 85° 38' 44.2"
Elevation: 493'

Discontinuity Data
Strike: N 23° E
Dip: 86° S
Description: See Data Sheet 1 of 1

Rock Unit Exposed
Sellersburg Limestone

OUTCROP #4 - KNOBS

N 38° 20' 39.8"
W 85° 38' 47.7"
Elevation: 500'

Discontinuity Data
Strikes Range: N 2° E to N 13° E
Dips Range: 82° S to 86° N
Description: See Data Sheet 1 of 1

Rock Units Exposed
Sellersburg Limestone Underlain by Jeffersonville Limestone

OUTCROP #5 - EAST QUARRY BLUFF CUT

N 38° 20' 56.0"
W 85° 38' 39.9"
Elevation: 453'

Discontinuity Data
Strike: S 38° W
Dip: 78° S
Description: See Data Sheet 1 of 1

Rock Units Exposed
Louisville Limestone Underlain by Waldron Shale

OUTCROP #6 - ROAD CUT TO QUARRY BLUFF ESTATES

N 38° 20' 59.5"
W 85° 38' 41.8"
Elevation: 452'

Discontinuity Data
Strikes Range: N 6° E to S 24° W
Dips Range: 86° N to 90° S
Description: See Data Sheet 1 of 1

Rock Units Exposed
Louisville Limestone Underlain by Waldron Shale

OUTCROP #7 - NORTH OLD QUARRY WALL

N 38° 20' 44.9"
W 85° 38' 47.5"
Elevation: 493'

No Discontinuity Data collected because area corresponds to Outcrop 2
Description: See Data Sheet 1 of 1

Rock Unit Exposed
Sellersburg Limestone

OUTCROP #8 - SOLUTION FEATURE OUTFLOWS

N 38° 20' 44.5"
W 85° 38' 43.5"
Elevation: 437'

Discontinuity Data
Strikes Range: N 78° W to S 83° W
Dips Range: 85° S to 87° S
Description: See Data Sheet 1 of 1

Rock Units Exposed
Louisville Limestone Underlain by Waldron Shale

OUTCROP #9 - UPPER RIVER ROAD CUT

N 38° 20' 40.6"
W 85° 38' 43.8"
Elevation: 436'

Discontinuity Data
Strikes Range: N 21° W to N 88° W
Dips Range: 79° S to 88° N
Description: See Data Sheets 1 thru 4

Rock Units Exposed
Louisville Limestone Underlain by Waldron Shale

100 0 100 200 300
GRAPHIC SCALE IN FEET

DATE: October, 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: JRF	KJS/TD

LX2005125\5125EXHIBIT.DGN

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I-265** | CROSSING **OHIO RIVER**

GEOLOGIC OUTCROP MAPPING

PREPARED BY
MSM ENGINEERS

SHEET NO.

DRAWING NO.

ITEM NUMBER

5-118.00

APPENDIX D
FIELD TEST RESULTS- P-S LOGGING

GEOVision geophysical services

SDC5 – EAST END BRIDGE OVER THE OHIO RIVER, BORING AC-3 SUSPENSION P & S VELOCITIES

November 26, 2007
Report 7472-02

SDC5 – EAST END BRIDGE OVER THE OHIO RIVER, BORING AC-3 SUSPENSION P & S VELOCITIES

Prepared for

Fuller, Mossbarger, Scott and May, Engineers, Inc.
1409 North Forbes Road
Lexington, KY, 40511
(859) 422-3000

Prepared by

GEOVision Geophysical Services
1151 Pomona Road, Unit P
Corona, California 92882
(951) 549-1234
Project 7472

November 26, 2007
Report 7472-02

TABLE OF CONTENTS

FIGURES

TABLES

APPENDICES

APPENDIX A FIGURES

APPENDIX A TABLES

INTRODUCTION

OYO suspension velocity measurements were performed in one land boring at the SDC5 – East End Bridge over the Ohio River, near Louisville, Kentucky, as a component of the evaluation of soil stability and load bearing capacity. Suspension logging data acquisition was performed on October 17, 2007 by Rob Steller of GEOVision. The work was performed under subcontract with Fuller, Mossbarger, Scott and May, Engineers, Inc. (FMSM). Kurt Schaefer served as the point of contact with FMSM.

This report describes the field measurements, data analysis, and results of this work.

SCOPE OF WORK

This report presents the results of suspension velocity measurements collected on October 17, 2007 in the cased boring designated AC-3, as detailed below. The purpose of these studies was to supplement stratigraphic information obtained during FMSM's soil sampling program and to acquire shear wave velocities and compressional wave velocities as a function of depth, which, in turn, can be used to characterize soil condition.

BORING DESIGNATION	DATE LOGGED	STATION	COORDINATES	
			LATITUDE	LONGITUDE
AC-3	10/17/07	187+46.6, 60.9 FT RIGHT	38.34021349	85.6398206

Table 1. Boring location and logging date

The OYO/Robertson Model 3403 Suspension Logging Probe were used to obtain in-situ horizontal shear and compressional wave velocity measurements at 1.64 ft intervals. The acquired data was analyzed and a profile of velocity versus depth was produced for both compressional and horizontally polarized shear waves.

A detailed reference for the velocity measurement techniques used in this study is:

> Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

SUSPENSION INSTRUMENTATION

Suspension soil velocity measurements were performed using the Model 3403 Suspension Logging system, manufactured by OYO Corporation and Robertson Geologging. This system directly determines the average velocity of a 3.28 ft high segment of the soil column surrounding the boring of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the boring producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source (S_H) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.28 ft, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in this survey is 19 ft, with the center point of the receiver pair 12.1 ft above the bottom end of the probe. The probe receives control signals from, and sends the digitized receiver signals to, instrumentation on the surface via an armored 4 conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data.

The entire probe is suspended by the cable, therefore, source motion is not coupled directly to the boring walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the boring and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it impinges upon the boring wall. These waves propagate through the soil and rock surrounding the boring, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. Orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 7.0 ft separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver. In faster soils or rock, the isolation cylinder is extended to allow greater separation of the P- and S_H-wave signals.
4. In saturated soils, the received P-wave signal is typically of much higher frequency than the received S_H-wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in fluid is significantly greater than the dimension of the fluid annulus surrounding the probe (foot versus inch scale), preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.
3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The Model 3403 has six channels (two simultaneous recording channels), each with a 16 bit 1024 sample record. The recorded data is displayed on the control computer for field review before saving the data file for each depth station. Data is stored on disk for further processing. Up to 8 sampling sequences can be summed to improve the signal to noise ratio of the signals.

Review of the displayed data on the computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), sample rate, and summing number to optimize the quality of the data before recording. Verification of the calibration of the Model 3403 digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as outlined in Appendix B.

SUSPENSION MEASUREMENT PROCEDURES

The boring was logged through 3 inch PVC casing, grouted in place and filled with water. The boring probe was positioned with the mid-point of the receiver spacing at grade, and the electronic depth counter was set to zero. The probe was lowered to the bottom of the boring, stopping at 1.64 ft intervals to collect data, as summarized below.

At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed, and the gains were adjusted as required. The data from each depth was reviewed and recorded on disk before moving to the next depth.

Upon completion of the measurements, the probe zero depth indication at grade was verified prior to removal from the boring.

BORING NUMBER	RUN NUMBER	DEPTH RANGE (FEET)	OPEN HOLE DEPTH (FEET)	LOST TO SLOUGH/COLLAPSE (FEET)	SAMPLE INTERVAL (FEET)	DATE LOGGED
AC-3	1	6.6 – 137.2	149.3	NA	1.6	10/17/07

Table 2. Logging date and depth range

SUSPENSION DATA ANALYSIS

The recorded digital waveforms were analyzed to locate the first minima on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 3.28 ft segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data.

The P-wave velocity calculated from the travel time over the 7.0 ft interval from source to receiver 1 (S-R1) was calculated and plotted for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 5.15 ft to correspond to the mid-point of the 7.0 ft S-R1 interval, as illustrated in Figure 1. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting 0.3 milliseconds, the calculated and experimentally verified delay from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

The recorded digital records were studied to establish the presence of clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital FFT - IFFT lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 400 Hz in the slowest zones to 1000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima was picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source or by boring inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuations.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 7.0 ft interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 5.15 ft to correspond to the mid-point of the 7.0 ft S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting 0.3 milliseconds, the calculated and experimentally verified delay from the beginning of the record at the source trigger pulse to source impact.

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. It presents all six seismic records for a given depth on a shared horizontal axis time scale in milliseconds, and a vertical axis scale of arbitrary amplitude, gain ranged to fill the screen. Pick times are demarked with a vertical line across each record, and listed in milliseconds, along the left margin. In Figure 2, the time difference over the 3.28 ft interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 ft/sec, as listed in the top right corner of the display. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering of the S_H-waveform records with an 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by residual P-wave signal.

SUSPENSION RESULTS

Suspension R1-R2 P- and S_H-wave velocities are plotted in Figure 4. The suspension velocity data shown in these figures are presented in Table 3. P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figure A1 to aid in visual comparison. It must be noted that R1-R2 data is an average velocity over a 3.28 ft segment of the soil column; S-R1 data is an average over 7.0 ft, creating a significant smoothing relative to the R1-R2 plots. S-R1 data are presented in Table A1. Good correspondence between the shapes of the P- and S_H-wave velocity curves is observed for this data set. The velocities derived from S-R1 and R1-R2 data are in excellent agreement, providing verification of the higher resolution R1-R2 data.

Calibration procedures and records for the suspension measurement system are presented in Appendix B.

SUMMARY

Discussion of Suspension Results

Both P- and S_H-wave velocities were measured using the OYO Suspension Method in one PVC cased land boring at the alignment of the SDC5 – East End Bridge over the Ohio River, in Louisville, Kentucky. The boring was located in an suburban environment, and no significant signal contamination from ambient vibration was observed.

This boring shows a fairly monotonic increase in S_H-wave velocities from 250 ft/sec near the surface, to about 1200 ft/sec at the bedrock contact at a depth of 99 feet. P-wave velocities step up to water velocity (5000 ft/sec.) at 12 feet, which corresponds closely with the level of the Ohio River nearby. The S_H-wave velocity of the rock increases rapidly between 99 and 112 feet, reaching a S_H-wave velocity of 6000 ft/sec.

Quality Assurance

These velocity measurements were performed using industry-standard or better methods for both measurements and analyses. All work was performed under GEOVision quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.28 ft interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable with estimated precision of +/- 5%. Standardized field procedures and quality assurance checks add to the reliability of these data.

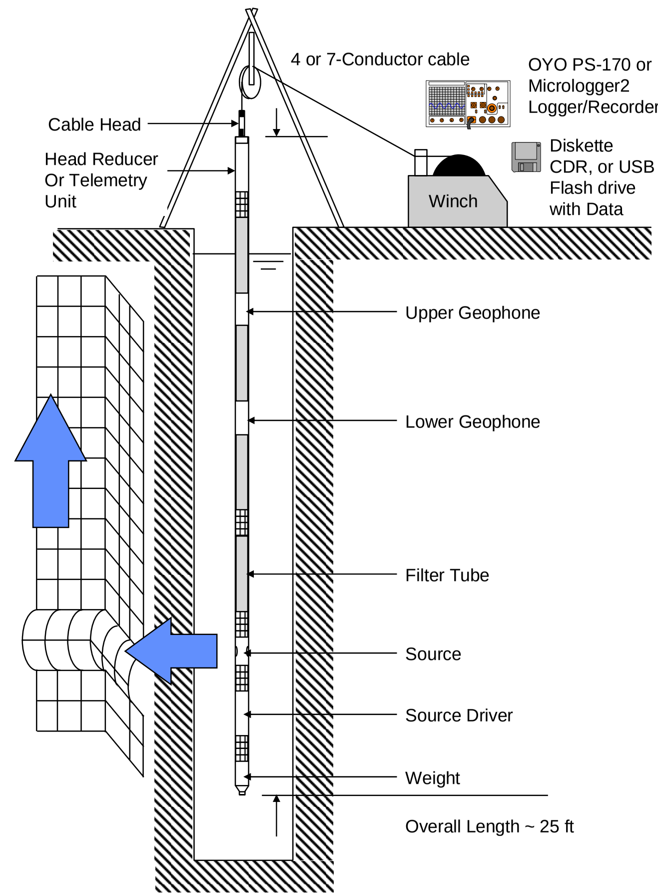
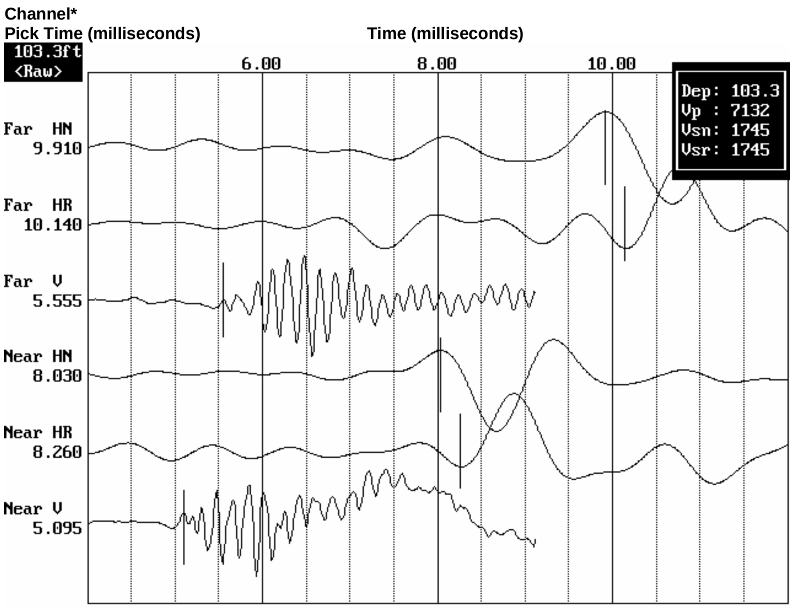
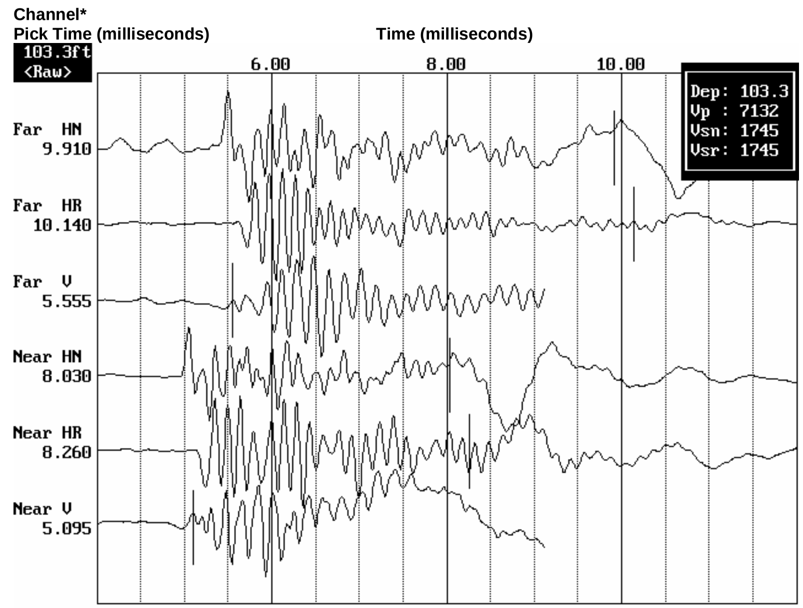


Figure 1. Concept illustration of P-S logging system



***Far HN = Far Horizontal geophone, Normal source direction**
Far HR = Far Horizontal geophone, Reverse source direction
Far V = Far Vertical geophone
Near HN = Near Horizontal geophone, Normal source direction
Near HR = Near Horizontal geophone, Reverse source direction
Near V = Near Vertical geophone

Figure 2. Example of filtered (1400 Hz lowpass) record



***Far HN = Far Horizontal geophone, Normal source direction**
Far HR = Far Horizontal geophone, Reverse source direction
Far V = Far Vertical geophone
Near HN = Near Horizontal geophone, Normal source direction
Near HR = Near Horizontal geophone, Reverse source direction
Near V = Near Vertical geophone

Figure 3. Example of unfiltered record

SDC5 - EAST END BRIDGE OVER OHIO RIVER, BORING AC-3

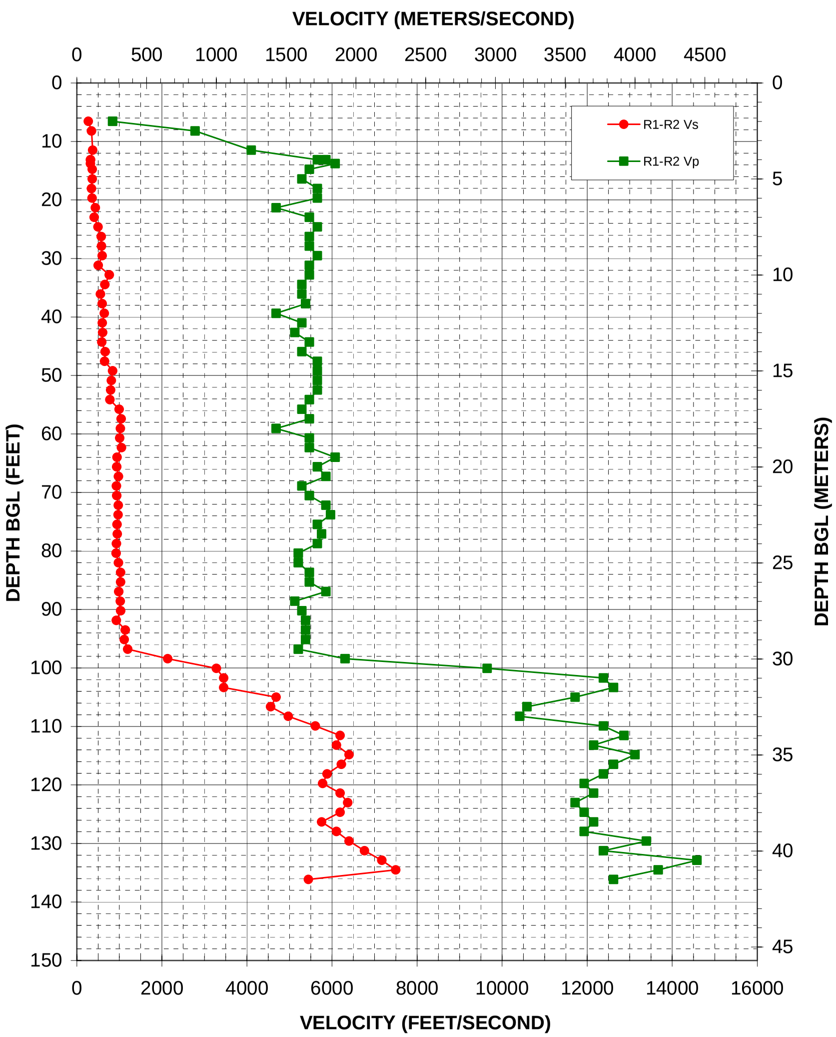


Figure 4. Boring AC-3, Suspension R1-R2 P- and S_H-wave velocities

Depth (meters)	Velocity $V\text{-}S_H$ (m/sec)	Velocity V-p (m/sec)	Depth (feet)	Velocity $V\text{-}S_H$ (ft/sec)	Velocity V-p (ft/sec)
2.0	82	256	6.56	270	841
2.5	105	847	8.20	345	2780
3.5	113	1250	11.48	372	4101
4.0	99	1724	13.12	323	5657
4.0	98	1786	13.12	320	5859
4.2	98	1852	13.78	320	6076
4.5	110	1667	14.76	363	5468
5.0	110	1613	16.40	363	5292
5.5	105	1724	18.04	344	5657
6.0	109	1724	19.69	358	5657
6.5	133	1429	21.33	436	4687
7.0	124	1667	22.97	408	5468
7.5	152	1724	24.61	499	5657
8.0	175	1667	26.25	573	5468
8.5	177	1667	27.89	581	5468
9.0	182	1724	29.53	597	5657
9.5	153	1667	31.17	503	5468
10.0	233	1667	32.81	763	5468
10.5	201	1613	34.45	659	5292
11.0	169	1613	36.09	556	5292
11.5	182	1639	37.73	597	5378
12.0	197	1429	39.37	646	4687
12.5	183	1613	41.01	599	5292
13.0	185	1563	42.65	608	5126
13.5	179	1667	44.29	586	5468
14.0	203	1613	45.93	666	5292
14.5	199	1724	47.57	653	5657
15.0	256	1724	49.21	841	5657
15.5	247	1724	50.85	810	5657
16.0	242	1724	52.49	795	5657
16.5	237	1667	54.13	777	5468
17.0	303	1613	55.77	994	5292
17.5	317	1667	57.41	1042	5468
18.0	313	1429	59.06	1025	4687
18.5	308	1667	60.70	1009	5468
19.0	320	1667	62.34	1050	5468
19.5	288	1852	63.98	944	6076
20.0	286	1724	65.62	937	5657
20.5	299	1786	67.26	979	5859
21.0	284	1613	68.90	931	5292
21.5	286	1667	70.54	937	5468
22.0	297	1786	72.18	974	5859
22.5	296	1818	73.82	971	5965
23.0	288	1724	75.46	945	5657
23.5	290	1754	77.10	951	5756
24.0	284	1724	78.74	932	5657
24.5	282	1587	80.38	924	5208
25.0	299	1587	82.02	979	5208
25.5	313	1667	83.66	1028	5468
26.0	313	1667	85.30	1028	5468
26.5	300	1786	86.94	985	5859
27.0	312	1563	88.58	1022	5126
27.5	314	1613	90.22	1032	5292
28.0	283	1639	91.86	929	5378
28.5	347	1639	93.50	1139	5378
29.0	340	1639	95.14	1116	5378
29.5	365	1587	96.78	1197	5208
30.0	651	1923	98.43	2137	6309
30.5	1000	2941	100.07	3281	9650
31.0	1053	3774	101.71	3454	12381
31.5	1053	3846	103.35	3454	12619
32.0	1429	3571	104.99	4687	11717
32.5	1389	3226	106.63	4557	10583
33.0	1515	3175	108.27	4971	10415
33.5	1709	3774	109.91	5608	12381
34.0	1887	3922	111.55	6190	12866
34.5	1860	3704	113.19	6104	12151
35.0	1951	4000	114.83	6402	13123
35.5	1896	3846	116.47	6220	12619
36.0	1794	3774	118.11	5885	12381
36.5	1762	3636	119.75	5781	11930
37.0	1887	3704	121.39	6190	12151
37.5	1942	3571	123.03	6371	11717
38.0	1887	3636	124.67	6190	11930
38.5	1754	3704	126.31	5756	12151
39.0	1860	3636	127.95	6104	11930
39.5	1951	4082	129.59	6402	13391
40.0	2062	3774	131.23	6765	12381
40.5	2186	4444	132.87	7171	14582
41.0	2286	4167	134.51	7499	13670
41.5	1660	3846	136.15	5445	12619

Table 3. Boring AC-3, Suspension R1-R2 P- and S_H-wave velocity data

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT
QUALITY ASSURANCE SUSPENSION SOURCE
TO RECEIVER ANALYSIS RESULTS

SDC5 - EAST END BRIDGE OVER OHIO RIVER, BORING AC-3

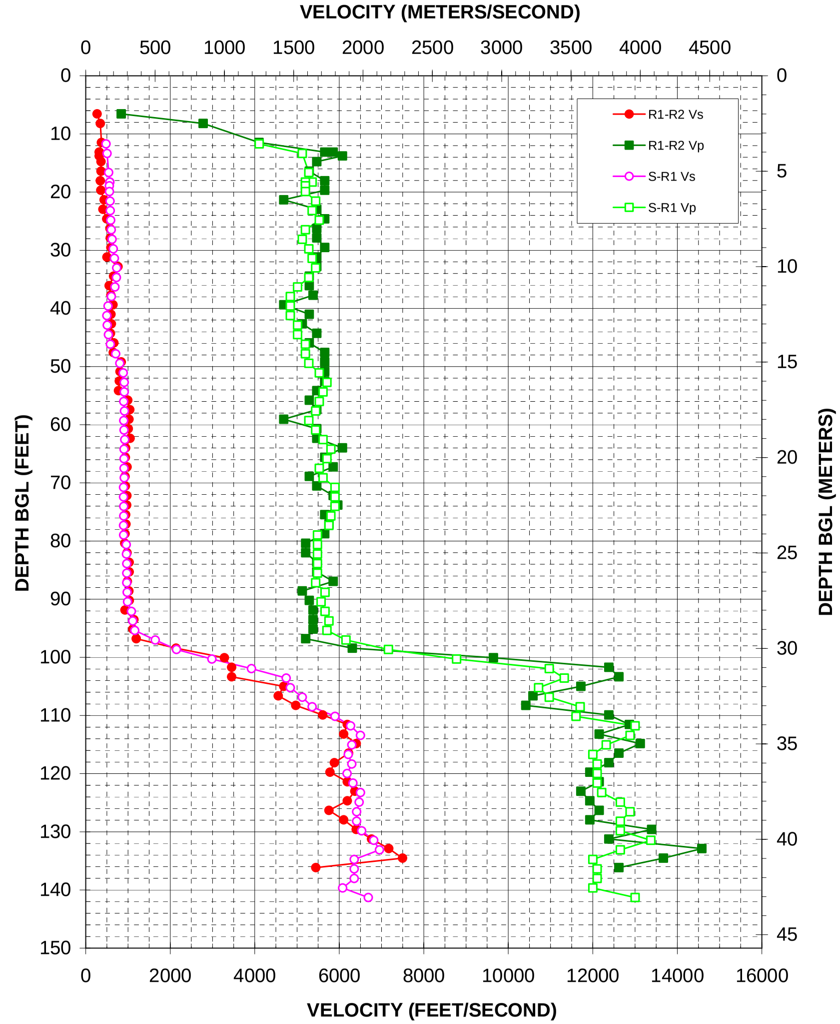


Figure A-1. Boring AC-3, R1 - R2 high resolution analysis and S-R1 quality assurance analysis P- and S_H-wave velocities

	Velocity			Velocity	
Depth (meters)	$V\text{-}S_H$ (m/sec)	V-p (m/sec)	Depth (feet)	$V\text{-}S_H$ (ft/sec)	V-p (ft/sec)
3.6	147	1251	11.71	481	4106
4.1	154	1562	13.35	505	5125
5.1	166	1609	16.63	543	5279
5.6	174	1634	18.27	571	5360
5.6	173	1585	18.27	569	5201
5.8	171	1585	18.93	560	5201
6.1	171	1585	19.91	562	5201
6.6	175	1659	21.56	575	5443
7.1	177	1634	23.20	580	5360
7.6	181	1685	24.84	592	5528
8.1	184	1585	26.48	605	5201
8.6	190	1562	28.12	624	5125
9.1	199	1609	29.76	653	5279
9.6	207	1634	31.40	678	5360
10.1	224	1659	33.04	735	5443
10.6	222	1609	34.68	728	5279
11.1	211	1529	36.32	692	5015
11.6	185	1476	37.96	608	4842
12.1	161	1476	39.60	527	4842
12.6	153	1476	41.24	501	4842
13.1	155	1529	42.88	509	5015
13.6	164	1529	44.52	537	5015
14.1	178	1585	46.16	583	5201
14.6	216	1585	47.80	709	5201
15.1	246	1609	49.44	806	5279
15.6	271	1685	51.08	889	5528
16.1	278	1740	52.72	912	5708
16.6	278	1712	54.36	912	5617
17.1	274	1685	56.00	900	5528
17.6	280	1659	57.64	918	5443
18.1	274	1609	59.28	900	5279
18.6	276	1659	60.93	906	5443
19.1	282	1712	62.57	924	5617
19.6	276	1769	64.21	906	5802
20.1	276	1740	65.85	906	5708
20.6	276	1685	67.49	906	5528
21.1	278	1712	69.13	912	5617
21.6	273	1798	70.77	894	5900
22.1	273	1798	72.41	894	5900
22.6	274	1798	74.05	900	5900
23.1	274	1769	75.69	900	5802
23.6	273	1754	77.33	896	5755
24.1	273	1672	78.97	897	5485
24.6	293	1672	80.61	960	5485
25.1	294	1672	82.25	966	5485
25.6	296	1672	83.89	971	5485
26.1	296	1672	85.53	971	5485
26.6	296	1659	87.17	971	5443
27.1	299	1726	88.81	982	5662
27.6	302	1698	90.45	990	5572

	Velocity			Velocity	
Depth (meters)	$V\text{-}S_H$ (m/sec)	V-p (m/sec)	Depth (feet)	$V\text{-}S_H$ (ft/sec)	V-p (ft/sec)
28.1	330	1726	92.09	1082	5662
28.6	339	1754	93.73	1113	5755
29.1	354	1740	95.37	1160	5708
29.6	501	1877	97.01	1644	6159
30.1	654	2184	98.65	2147	7164
30.6	911	2675	100.30	2988	8776
31.1	1196	3344	101.94	3922	10970
31.6	1446	3452	103.58	4744	11324
32.1	1476	3267	105.22	4842	10719
32.6	1562	3344	106.86	5125	10970
33.1	1634	3567	108.50	5360	11702
33.6	1798	3537	110.14	5900	11605
34.1	1911	3963	111.78	6269	13002
34.6	1981	3927	113.42	6501	12883
35.1	1919	3754	115.06	6297	12318
35.6	1894	3658	116.70	6213	12002
36.1	1919	3690	118.34	6297	12105
36.6	1885	3690	119.98	6186	12105
37.1	1928	3690	121.62	6325	12105
37.6	1981	3722	123.26	6501	12210
38.1	1972	3856	124.90	6471	12650
38.6	1954	3927	126.54	6412	12883
39.1	1954	3856	128.18	6412	12650
39.6	1991	3856	129.82	6531	12650
40.1	2078	4076	131.46	6817	13373
40.6	2119	3856	133.10	6951	12650
41.1	1937	3658	134.74	6354	12002
41.6	1937	3690	136.38	6354	12105
42.1	1937	3690	138.02	6354	12105
42.6	1853	3658	139.67	6079	12002
43.1	2038	3963	141.31	6687	13002

Table A-1. Boring AC-3, S - R1 quality assurance analysis P- and S_H-wave velocity data

APPENDIX B

OYO 170 VELOCITY LOGGING SYSTEM
NIST TRACEABLE
CALIBRATION PROCEDURE

CALIBRATION PROCEDURE FOR
GEOVision SEISMIC RECORDER/LOGGER

Reviewed 4/6/06

Objective

The timing/sampling accuracy of seismic recorders or data loggers is required for several GEOVision field procedures including Seismic Refraction, Downhole Seismic Velocity Logging, and P-S Suspension Logging. This procedure describes the method for measuring the timing accuracy of a seismic data logger, such as the OYO Model 170, OYO/Robertson Model 3403, Geometrics Strataview or Geometrics Geode. The objective of this procedure is to verify that the timing accuracy of the recorder is accurate to within 1%.

Frequency of Calibration

The calibration of each GEOVision seismic data logger is twelve (12) months. In the case of rented seismic data loggers, calibration must be performed prior to use.

Test Equipment Required

The following equipment is required. Item #2 must have current NIST traceable calibration.

1. Function generator, Krohn Hite 5400B or equivalent
2. Frequency counter, HP 5315A or equivalent
3. Test cables, from item 1 to item 2, and from item 1 to subject data logger.

Procedure

This procedure is designed to be performed using the accompanying Seismograph Calibration Data Sheet with the same revision number. All data must be entered and the procedure signed by the technician performing the test.

1. Record all identification data on the form provided.
2. Connect function generator to data logger (such as OYO Model 170) using test cable
3. Connect the function generator to the frequency counter using test cable.

4. Set up generator to produce a 100.0 Hz, 0.25 volt (amplitude is approximate, modify as necessary to yield less than full scale waveforms on logger display) peak square wave or sine wave. Verify frequency using the counter and initial space on the data sheet.

5. Initialize data logger and record a data record of at least 0.1 second using a 100 microsecond or less sample period.

6. Measure the recorded square wave frequency by measuring the duration of 9 cycles of data. This measurement can be made using the data logger display device, or by printing out a paper tape. If a paper tape can be printed, the resulting printout must be attached to this procedure. Record the data in the space provided.

7. Repeat steps 5 and 6 three more times using separate files.

Criteria

The duration for 9 cycles in any file must be 90.0 milliseconds plus or minus 0.9 milliseconds, corresponding to an average frequency for the nine cycles of 100.0 Hz plus or minus 1 Hz (obtained by dividing 9 cycles by the duration in milliseconds).

If the results are outside this range, the data logger must be marked with a GEOVision REJECT tag until it can be repaired and retested.

If results are acceptable affix label indicating the initials of the person performing the calibration, the date of calibration, and the due date for the next calibration (12 months).

Procedure Approval

Approved by:

John G. Diehl	President
Name	Title
[signature]	April 6, 2006
Signature	Date

Client Approval (if required):

Name	Title
Signature	Date

EDISON ESI

A SOUTHERN CALIFORNIA EDISON Company

Metrology
7300 Fenwick Lane
Westminster, CA 92683
Phone: 866-723-2257

Calibration Report

NVLAP Accredited Calibration

GEOVision Geophysical Services
1151 Pomona Road, Unit P
Corona, CA 92882

TEST NUMBER 527438

NVLAP Lab Code:105014-0

Manufacturer:	Oyo
Model Number:	3403
Description:	Unit, Suspension Telemetry,
Asset Number:	160023
Serial Number:	160023
PO Number:	7087-070315-01

Condition As Found:	In Tolerance
Condition As Left:	In Tolerance
Calibration Date:	04/13/2007
Calibration Due Date:	04/13/2008
Calibration Interval:	12 Months

Remarks:

The UUT (unit under test) was calibrated using the customer's procedure. The UUT was operated by the customer's personnel and data collection was observed by SCE personnel. The UUT was found to be in tolerance to customer supplied specifications. Frequency is accredited. Please see attached data.

Standards Utilized

I.D. No.	Mfg.	Model No.	Description	Cal. Date	Due Date
S1-03092	Hewlett Packard	5335A OPT 020, 30,40	Counter, Universal,	12/12/2006	06/12/2007
S1-03355	Hewlett Packard	3325B OPT 001, 002	Generator, Function, Synthesizer	11/08/2006	11/08/2007
S1-03686	Fluke	910	Standard, Frequency, Controlled, Gps	01/18/2007	01/18/2008

Procedure:	Customer
Temperature:	23° C
Humidity:	38% RH
Test No.:	527438

Calibration Performed By:				Quality Reviewer:	
Branson, Craig A	CAB	Metrologist	714-895-0714	[signature]	4/13/07
Name		Title	Phone	Name	Date

GEOVision geophysical services

SEISMOGRAPH CALIBRATION DATA SHEET REV 4/6/06

INSTRUMENT DATA

SYSTEM MFR:	OYO	MODEL NO.:	3403
SERIAL NO.:	160023	CALIBRATION DATE:	04/13/2007
BY:	ROBERT STELLER	DUE DATE:	04/13/2008
COUNTER MFR:	HEWLETT PACKARD	MODEL NO.:	5335A
SERIAL NO.:	2626A10854	CALIBRATION DATE:	12/12/2006
BY:	SCE #S1-03092	DUE DATE:	06/12/2007
FCTN GEN MFR:	HEWLETT PACKARD	MODEL NO.:	3325B
SERIAL NO.:	2847A14447	CALIBRATION DATE:	11/08/2006
BY:	SCE #S1-03355	DUE DATE:	11/08/2007

SYSTEM SETTINGS:

GAIN:	2
FILTER:	10 KHZ
RANGE:	100 MILLISEC
DELAY:	0
STACK: 1 (STD)	1
PULSE:	1.6
DISPLAY:	NA
SYSTEM: DATE = CORRECT DATE & TIME	04/13/2007,09:50AM

PROCEDURE:
SET FREQUENCY TO 100.0HZ SQUAREWAVE WITH AMPLITUDE APPROXIMATELY 0.25 VOLT PEAK. RECORD BOTH ON DISK AND PAPER TAPE, IF AVAILABLE. ANALYZE AND PRINT WAVEFORMS FROM ANALYSIS UTILITY. ATTACH PAPER COPIES OF PRINTOUT AND PAPER TAPES, IF AVAILABLE, TO THIS FORM. AVERAGE FREQUENCY MUST BE BETWEEN 99.0 AND 101.0 HZ.

AS FOUND 100.0 AS LEFT 100.0

WAVEFORM	FILE NO	FREQUENCY	TIME FOR 9 CYCLES Hn	TIME FOR 9 CYCLES Hr	TIME FOR 9 CYCLES V	AVERAGE FREQ.
SQUARE	401	100.0	90.0	90.0	90.0	100.0
SQUARE	402	100.0	90.0	90.0	90.0	100.0
SINE	403	100.0	90.0	90.0	90.0	100.0
SINE	404	100.0	90.0	90.0	90.0	100.0

CALIBRATED BY: ROBERT STELLER (NAME) 04/13/2007 (DATE) Robt Stell (SIGNATURE)

APPENDIX E
LABORATORY TEST RESULTS - SOIL

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Bridge over the Ohio River
Source: AC-1, 2.5'-3.0'
County: Jefferson
Sample Type: UD

Project Number: LX2005125
Lab ID: 146A
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 19.1

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	
No. 10	2	100.0
No. 40	0.425	99.9
No. 200	0.075	94.3
	0.02	64.1
	0.005	38.3
	0.002	27.0
estimated	0.001	20.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.1
Medium Sand	0.1	---
Fine Sand	5.6	5.6
Silt	56.0	67.3
Clay	38.3	27.0

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 48
Plastic Limit: 19
Plasticity Index: 29
Activity Index: 1.07

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.72

Classification
Unified Group Symbol: CL
Group Name: Lean clay
AASHTO Classification: A-7-6 (29)

Comments:

Reviewed by:

File: LX2005125_Sum-146A.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-1, 10.0'-10.5'
County: Jefferson
Sample Type: UD

Project Number: LX2005125
Lab ID: 148A
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 23.4

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	100.0
No. 10	2	100.0
No. 40	0.425	99.6
No. 200	0.075	79.3
	0.02	52.2
	0.005	33.2
	0.002	24.6
estimated	0.001	19.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.4
Medium Sand	0.4	---
Fine Sand	20.3	20.3
Silt	46.1	54.7
Clay	33.2	24.6

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 35
Plastic Limit: 15
Plasticity Index: 20
Activity Index: 0.80

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.68

Classification
Unified Group Symbol: CL
Group Name: Lean clay with sand
AASHTO Classification: A-6 (14)

Comments:

Reviewed by:

File: LX2005125_Sum-148A.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-1, 20.0'-21.5'
County: Jefferson
Sample Type: SPT

Project Number: LX2005125
Lab ID: 198
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	
No. 10	2	100.0
No. 40	0.425	99.7
No. 200	0.075	58.6
	0.02	33.4
	0.005	19.6
	0.002	14.7
estimated	0.001	11.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.3
Medium Sand	0.3	---
Fine Sand	41.1	41.1
Silt	39.0	43.9
Clay	19.6	14.7

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 23
Plastic Limit: 18
Plasticity Index: 5
Activity Index: 0.33

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: CL-ML
Group Name: Sandy silty clay
AASHTO Classification: A-4 (1)

Comments:

Reviewed by:

File: LX2005125_Sum-198.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-1, 25.0'-26.5', 30.0'-31.5', 35.0'-36.5', 40.0'-41.5'
County: Jefferson
Sample Type: SPT Composite

Project Number: LX2005125
Lab ID: 150
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	94.7
3/8"	9.5	80.5
No. 4	4.75	72.3
No. 10	2	65.2
No. 40	0.425	32.3
No. 200	0.075	12.5
	0.02	7.3
	0.005	4.7
	0.002	3.7
estimated	0.001	3.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	27.7	34.8
Coarse Sand	7.1	32.9
Medium Sand	32.9	---
Fine Sand	19.8	19.8
Silt	7.8	8.8
Clay	4.7	3.7

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: SM
Group Name: Silty sand with gravel
AASHTO Classification: A-1-b (0)

Comments:

Reviewed by:

File: LX2005125_Sum-150.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-1, 55.0'-56.5', 60.0'-61.5', 65.0'-66.5'
County: Jefferson
Sample Type: SPT Composite

Project Number: LX2005125
Lab ID: 151
Date Received: 9-28-07
Date Reported: 10-23-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	99.0
3/8"	9.5	95.1
No. 4	4.75	91.3
No. 10	2	85.5
No. 40	0.425	28.7
No. 200	0.075	9.9
	0.02	5.0
	0.005	3.0
	0.002	2.1
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	8.7	14.5
Coarse Sand	5.8	56.8
Medium Sand	56.8	---
Fine Sand	18.8	18.8
Silt	6.9	7.8
Clay	3.0	2.1

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.71

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt
AASHTO Classification: A-1-b (0)

Comments:

Reviewed by:

File: LX2005125_Sum-151 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-1, 80.0'-81.5', 85.0'-86.5', 90.0'-91.5'
County: Jefferson
Sample Type: SPT Composite

Project Number: LX2005125
Lab ID: 152
Date Received: 9-28-07
Date Reported: 10-23-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	98.5
No. 4	4.75	97.3
No. 10	2	90.6
No. 40	0.425	23.3
No. 200	0.075	4.8
	0.02	2.7
	0.005	1.6
	0.002	1.3
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	2.7	9.4
Coarse Sand	6.7	67.3
Medium Sand	67.3	---
Fine Sand	18.5	18.5
Silt	3.2	3.5
Clay	1.6	1.3

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.67

Classification
Unified Group Symbol: SW
Group Name: Well-graded sand
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-152 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM Engineers

Summary of Soil Tests

Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-2, 5.0'-5.5' — Lab ID 159A

County Jefferson — Date Received 9-28-07
Sample Type UD — Date Reported 10-25-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 18.6

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	100.0
No. 10	2	100.0
No. 40	0.425	99.7
No. 200	0.075	94.4
	0.02	69.0
	0.005	40.7
	0.002	28.7
estimated	0.001	22.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.3
Medium Sand	0.3	---
Fine Sand	5.3	5.3
Silt	53.7	65.7
Clay	40.7	28.7

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 48
Plastic Limit: 20
Plasticity Index: 28
Activity Index: 0.97

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.69

Classification
Unified Group Symbol: CL
Group Name: Lean clay
AASHTO Classification: A-7-6 (28)

Comments:

Reviewed by:

File: LX2005125_Sum-159A.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM Engineers

Summary of Soil Tests

Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-2, 20.0'-20.5' — Lab ID 161A

County Jefferson — Date Received 9-28-07
Sample Type UD — Date Reported 10-25-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 23.6

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	
No. 10	2	100.0
No. 40	0.425	99.9
No. 200	0.075	55.7
	0.02	36.7
	0.005	22.3
	0.002	16.6
estimated	0.001	13.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.1
Medium Sand	0.1	---
Fine Sand	44.2	44.2
Silt	33.4	39.1
Clay	22.3	16.6

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 24
Plastic Limit: 13
Plasticity Index: 11
Activity Index: 0.65

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.71

Classification
Unified Group Symbol: CL
Group Name: Sandy lean clay
AASHTO Classification: A-6 (3)

Comments:

Reviewed by:

File: LX2005125_Sum-161A.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-2, 25.0'-26.5', 30.0'-31.5'
Project Number: LX2005125
Lab ID: 163
County: Jefferson
Sample Type: SPT Composite
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	86.9
3/8"	9.5	79.8
No. 4	4.75	75.5
No. 10	2	72.2
No. 40	0.425	69.7
No. 200	0.075	41.8
	0.02	25.3
	0.005	15.9
	0.002	12.2
estimated	0.001	10.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	24.5	27.8
Coarse Sand	3.3	2.5
Medium Sand	2.5	---
Fine Sand	27.9	27.9
Silt	25.9	29.6
Clay	15.9	12.2

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 25
Plastic Limit: 14
Plasticity Index: 11
Activity Index: 0.92

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.72

Classification
Unified Group Symbol: SC
Group Name: Clayey sand with gravel
AASHTO Classification: A-6 (1)

Comments:

Reviewed by:

File: LX2005125_Sum-163.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-2, 35.0'-36.5', 40.0'-41.5', 45.0'-46.5', 50.0'-51.5'
Project Number: LX2005125
Lab ID: 164
County: Jefferson
Sample Type: SPT Composite
Date Received: 9-28-07
Date Reported: 10-23-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	95.9
3/8"	9.5	82.6
No. 4	4.75	72.6
No. 10	2	57.5
No. 40	0.425	16.1
No. 200	0.075	5.2
	0.02	3.0
	0.005	1.7
	0.002	1.2
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	27.4	42.5
Coarse Sand	15.1	41.4
Medium Sand	41.4	---
Fine Sand	10.9	10.9
Silt	3.5	4.0
Clay	1.7	1.2

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-164 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-2, 60.0'-61.5'
County: Jefferson
Sample Type: SPT

Project Number: LX2005125
Lab ID: 166
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	100.0
No. 4	4.75	99.7
No. 10	2	99.2
No. 40	0.425	68.2
No. 200	0.075	20.0
	0.02	5.0
	0.005	3.1
	0.002	2.4
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.3	0.8
Coarse Sand	0.5	31.0
Medium Sand	31.0	---
Fine Sand	48.2	48.2
Silt	16.9	17.6
Clay	3.1	2.4

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: SM
Group Name: Silty sand
AASHTO Classification: A-2-4 (0)

Comments:

Reviewed by:

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-2, 65.0'-66.5', 70.0'-71.5', 75.0'-76.5'
County: Jefferson
Sample Type: SPT Composite

Project Number: LX2005125
Lab ID: 167
Date Received: 9-28-07
Date Reported: 10-24-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	98.7
3/8"	9.5	97.5
No. 4	4.75	95.2
No. 10	2	89.6
No. 40	0.425	48.8
No. 200	0.075	6.5
	0.02	3.3
	0.005	2.2
	0.002	1.7
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	4.8	10.4
Coarse Sand	5.6	40.8
Medium Sand	40.8	---
Fine Sand	42.3	42.3
Silt	4.3	4.8
Clay	2.2	1.7

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.67

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-2, 85.0'-86.5', 90.0'-91.5', 95.0'-96.5', 100.0'-100.4'
County: Jefferson
Sample Type: SPT Composite

Project Number: LX2005125
Lab ID: 169
Date Received: 9-28-07
Date Reported: 10-23-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	94.9
3/8"	9.5	94.9
No. 4	4.75	93.1
No. 10	2	90.4
No. 40	0.425	41.3
No. 200	0.075	10.8
	0.02	4.2
	0.005	2.2
	0.002	1.4
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	6.9	9.6
Coarse Sand	2.7	49.1
Medium Sand	49.1	---
Fine Sand	30.5	30.5
Silt	8.6	9.4
Clay	2.2	1.4

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.66

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt
AASHTO Classification: A-1-b (0)

Comments:

Reviewed by:

File: LX2005125_Sum-169 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Bridge over the Ohio River
Source: AC-3, 2.5'-3.0'
County: Jefferson
Sample Type: UD

Project Number: LX2005125
Lab ID: 177A
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 33.9

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	
No. 10	2	100.0
No. 40	0.425	99.9
No. 200	0.075	93.5
	0.02	65.4
	0.005	35.8
	0.002	24.2
estimated	0.001	18.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.1
Medium Sand	0.1	---
Fine Sand	6.4	6.4
Silt	57.7	69.3
Clay	35.8	24.2

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 45
Plastic Limit: 20
Plasticity Index: 25
Activity Index: 1.04

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.71

Classification
Unified Group Symbol: CL
Group Name: Lean clay
AASHTO Classification: A-7-6 (25)

Comments:

Reviewed by:

File: LX2005125_Sum-177A.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-3, 10.0'-10.5'
County: Jefferson
Sample Type: UD

Project Number: LX2005125
Lab ID: 179
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 34.3

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	
No. 10	2	100.0
No. 40	0.425	99.9
No. 200	0.075	93.4
	0.02	66.7
	0.005	39.9
	0.002	29.4
estimated	0.001	24.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.1
Medium Sand	0.1	---
Fine Sand	6.5	6.5
Silt	53.5	64.0
Clay	39.9	29.4

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 39
Plastic Limit: 17
Plasticity Index: 22
Activity Index: 0.76

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.71

Classification
Unified Group Symbol: CL
Group Name: Lean clay
AASHTO Classification: A-6 (21)

Comments:

Reviewed by:

File: LX2005125_Sum-179.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-3, 16.0'-16.5'
County: Jefferson
Sample Type: UD

Project Number: LX2005125
Lab ID: 180
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	
No. 10	2	100.0
No. 40	0.425	99.9
No. 200	0.075	56.7
	0.02	36.6
	0.005	22.6
	0.002	16.3
estimated	0.001	13.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	0.0
Coarse Sand	0.0	0.1
Medium Sand	0.1	---
Fine Sand	43.2	43.2
Silt	34.1	40.4
Clay	22.6	16.3

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 24
Plastic Limit: 14
Plasticity Index: 10
Activity Index: 0.63

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: CL
Group Name: Sandy lean clay
AASHTO Classification: A-4 (3)

Comments:

Reviewed by:

File: LX2005125_Sum-180.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-3, 25.0'-26.5', 30.0'-31.5', 35.0'-36.5'
Project Number: LX2005125
Lab ID: 182
County: Jefferson
Sample Type: SPT Composite
Date Received: 9-28-07
Date Reported: 10-25-07

Test Results

Natural Moisture Content

Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis

Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	86.6
3/4"	19	77.5
3/8"	9.5	66.8
No. 4	4.75	60.6
No. 10	2	57.5
No. 40	0.425	53.1
No. 200	0.075	17.9
	0.02	10.0
	0.005	6.4
	0.002	4.8
estimated	0.001	4.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	39.4	42.5
Coarse Sand	3.1	4.4
Medium Sand	4.4	---
Fine Sand	35.2	35.2
Silt	11.5	13.1
Clay	6.4	4.8

Atterberg Limits

Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship

Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio

Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity

Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.68

Classification

Unified Group Symbol: SM
Group Name: Silty sand with gravel
AASHTO Classification: A-2-4 (0)

Comments:

Reviewed by:

File: LX2005125_Sum-182.xls Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-3, 45.0'-46.5', 50.0'-51.5', 55.0'-56.5'
Project Number: LX2005125
Lab ID: 184
County: Jefferson
Sample Type: SPT Composite
Date Received: 9-28-07
Date Reported: 10-23-07

Test Results

Natural Moisture Content

Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis

Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	95.6
3/4"	19	94.9
3/8"	9.5	91.1
No. 4	4.75	85.1
No. 10	2	72.6
No. 40	0.425	15.7
No. 200	0.075	5.2
	0.02	2.6
	0.005	1.6
	0.002	0.9
estimated	0.001	0.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	14.9	27.4
Coarse Sand	12.5	56.9
Medium Sand	56.9	---
Fine Sand	10.5	10.5
Silt	3.6	4.3
Clay	1.6	0.9

Atterberg Limits

Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship

Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio

Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity

Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.69

Classification

Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-184 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-3, 65.0'-66.5', 70.0'-71.5', 75.0'-76.5', 80.0'-81.5'
Project Number: LX2005125
Lab ID: 185

County: Jefferson
Sample Type: SPT Composite
Date Received: 9-28-07
Date Reported: 10-23-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	98.2
3/4"	19	98.2
3/8"	9.5	91.4
No. 4	4.75	84.6
No. 10	2	73.6
No. 40	0.425	24.5
No. 200	0.075	6.7
	0.02	2.9
	0.005	1.5
	0.002	1.1
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	15.4	26.4
Coarse Sand	11.0	49.1
Medium Sand	49.1	---
Fine Sand	17.8	17.8
Silt	5.2	5.6
Clay	1.5	1.1

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.68

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-185 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-4/189+81.55, 91.9 Lt., 71.7'-73.2', 76.7'-78.2', 81.7'-82.4'
Project Number: LX2005125
Lab ID: 25

County: Jefferson
Sample Type: SPT Composite
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	100.0
No. 4	4.75	97.4
No. 10	2	94.5
No. 40	0.425	45.3
No. 200	0.075	11.7
	0.02	4.1
	0.005	2.7
	0.002	2.0
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	2.6	5.5
Coarse Sand	2.9	49.2
Medium Sand	49.2	---
Fine Sand	33.6	33.6
Silt	9.0	9.7
Clay	2.7	2.0

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.69

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt
AASHTO Classification: A-1-b (0)

Comments:

Reviewed by:

File: LX2005125_Sum-25 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-5, 3.0'-3.5'
County: Jefferson
Sample Type: ST

Project Number: LX2005125
Lab ID: 201
Date Received: 10-23-07
Date Reported: 11-2-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 9.2

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	99.5
No. 4	4.75	99.4
No. 10	2	99.0
No. 40	0.425	97.3
No. 200	0.075	53.3
	0.02	37.0
	0.005	21.4
	0.002	14.7
estimated	0.001	11.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.6	1.0
Coarse Sand	0.4	1.7
Medium Sand	1.7	---
Fine Sand	44.0	44.0
Silt	31.9	38.6
Clay	21.4	14.7

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 30
Plastic Limit: 20
Plasticity Index: 10
Activity Index: 0.67

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.66

Classification
Unified Group Symbol: CL
Group Name: Sandy lean clay
AASHTO Classification: A-4 (3)

Comments:

Reviewed by:

File: LX2005125_Sum-201 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-5, 5.0'-7.0'
County: Jefferson
Sample Type: ST

Project Number: LX2005125
Lab ID: 202
Date Received: 10-23-07
Date Reported: 11-2-07

Test Results

Natural Moisture Content
Test Method: AASHTO T 265
Moisture Content (%): 6.5

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	
No. 4	4.75	100.0
No. 10	2	94.1
No. 40	0.425	86.0
No. 200	0.075	41.0
	0.02	28.8
	0.005	16.8
	0.002	12.1
estimated	0.001	9.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.0	5.9
Coarse Sand	5.9	8.1
Medium Sand	8.1	---
Fine Sand	45.0	45.0
Silt	24.2	28.9
Clay	16.8	12.1

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.71

Classification
Unified Group Symbol: SM
Group Name: Silty sand
AASHTO Classification: A-4 (0)

Comments:

Reviewed by:

File: LX2005125_Sum-202 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-5, 25.0'-26.5', 30.0'-31.5', 35.0'-36.5', 40.0'-41.5', 45.0'-46.5'
Project Number: LX2005125
Lab ID: 209

County: Jefferson
Sample Type: SPT Composite
Date Received: 10-23-07
Date Reported: 11-2-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	96.4
3/4"	19	96.4
3/8"	9.5	86.5
No. 4	4.75	74.3
No. 10	2	58.2
No. 40	0.425	20.1
No. 200	0.075	3.8
	0.02	2.2
	0.005	1.7
	0.002	1.4
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	25.7	41.8
Coarse Sand	16.1	38.1
Medium Sand	38.1	---
Fine Sand	16.3	16.3
Silt	2.1	2.4
Clay	1.7	1.4

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.72

Classification
Unified Group Symbol: SW
Group Name: Well-graded sand with gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-209 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-5, 60.0'-61.5', 65.0'-66.5', 70.0'-71.5', 75.0'-76.5'
Project Number: LX2005125
Lab ID: 217

County: Jefferson
Sample Type: SPT Composite
Date Received: 10-23-07
Date Reported: 11-2-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	99.5
No. 4	4.75	94.0
No. 10	2	75.1
No. 40	0.425	32.6
No. 200	0.075	6.0
	0.02	2.9
	0.005	1.7
	0.002	1.6
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	6.0	24.9
Coarse Sand	18.9	42.5
Medium Sand	42.5	---
Fine Sand	26.6	26.6
Silt	4.3	4.4
Clay	1.7	1.6

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.72

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-217 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Bridge over the Ohio Rver
Source: AC-7 193+95, 68' Lt., 40.5'-42.0', 45.5'-47.0', 50.5'-52.0'
County: Jefferson
Sample Type: SPT Composite

Project Number: LX2005125
Lab ID: 48
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	94.1
No. 4	4.75	88.9
No. 10	2	80.6
No. 40	0.425	24.4
No. 200	0.075	5.6
	0.02	2.9
	0.005	2.0
	0.002	1.5
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	11.1	19.4
Coarse Sand	8.3	56.2
Medium Sand	56.2	---
Fine Sand	18.8	18.8
Silt	3.6	4.1
Clay	2.0	1.5

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.69

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-48 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-7 193+95, 68' Lt., 55.5'-57.0', 60.5'-62.0'
County: Jefferson
Sample Type: SPT Composite

Project Number: LX2005125
Lab ID: 49
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	93.8
3/4"	19	91.3
3/8"	9.5	86.4
No. 4	4.75	83.1
No. 10	2	78.7
No. 40	0.425	47.8
No. 200	0.075	5.0
	0.02	2.4
	0.005	1.4
	0.002	0.8
estimated	0.001	0.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	16.9	21.3
Coarse Sand	4.4	30.9
Medium Sand	30.9	---
Fine Sand	42.8	42.8
Silt	3.6	4.2
Clay	1.4	0.8

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.67

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-49 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-7 193+95, 68' Lt., 80.5'-82.0', 85.5'-85.7'	Lab ID	51
County	Jefferson	Date Received	8-10-07
Sample Type	SPT Composite	Date Reported	10-17-07

Test Results

Natural Moisture Content

Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis

Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	94.7
3/8"	9.5	85.8
No. 4	4.75	84.1
No. 10	2	81.3
No. 40	0.425	43.6
No. 200	0.075	6.8
	0.02	2.4
	0.005	1.6
	0.002	1.1
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	15.9	18.7
Coarse Sand	2.8	37.7
Medium Sand	37.7	---
Fine Sand	36.8	36.8
Silt	5.2	5.7
Clay	1.6	1.1

Atterberg Limits

Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship

Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio

Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity

Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.68

Classification

Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

FMSM ENGINEERS

Summary of Soil Tests

Project Name	I-265 over the Ohio River	Project Number	LX2005125
Source	AC-8/193+95, 1.22'Lt., 40.7'-42.2', 45.7'-47.2', 50.7'-52.2'	Lab ID	61
County	Jefferson	Date Received	8-10-07
Sample Type	SPT Composite	Date Reported	10-17-07

Test Results

Natural Moisture Content

Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis

Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	93.3
No. 4	4.75	87.6
No. 10	2	79.0
No. 40	0.425	36.6
No. 200	0.075	4.7
	0.02	2.6
	0.005	1.6
	0.002	1.2
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	12.4	21.0
Coarse Sand	8.6	42.4
Medium Sand	42.4	---
Fine Sand	31.9	31.9
Silt	3.1	3.5
Clay	1.6	1.2

Atterberg Limits

Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship

Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio

Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity

Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification

Unified Group Symbol: SP
Group Name: Poorly graded sand
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

Fuller Mossbarger Scott & May FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-8/193+95, 1.22'Lt., 55.7'-57.2', 60.7'-62.2', 65.7'-66.4'
Project Number: LX2005125
Lab ID: 62
County: Jefferson
Sample Type: SPT Composite
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	94.8
No. 4	4.75	87.8
No. 10	2	81.5
No. 40	0.425	34.0
No. 200	0.075	5.8
	0.02	2.6
	0.005	1.5
	0.002	0.8
estimated	0.001	0.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	12.2	18.5
Coarse Sand	6.3	47.5
Medium Sand	47.5	---
Fine Sand	28.2	28.2
Silt	4.3	5.0
Clay	1.5	0.8

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.68

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-62 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-8/193+95, 1.22'Lt., 70.7'-72.2', 75.7'-77.2'
Project Number: LX2005125
Lab ID: 64
County: Jefferson
Sample Type: SPT Composite
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	91.1
3/4"	19	87.2
3/8"	9.5	66.5
No. 4	4.75	57.9
No. 10	2	46.4
No. 40	0.425	31.7
No. 200	0.075	8.1
	0.02	4.0
	0.005	2.0
	0.002	1.4
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	42.1	53.6
Coarse Sand	11.5	14.7
Medium Sand	14.7	---
Fine Sand	23.6	23.6
Silt	6.1	6.7
Clay	2.0	1.4

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.69

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-64 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-9/193+95, 70' Rt., 40.6'-42.1', 45.6'-47.1', 50.6'-52.1'
Project Number: LX2005125
Lab ID: 74
County: Jefferson
Sample Type: SPT Composite
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	95.8
3/8"	9.5	88.6
No. 4	4.75	77.7
No. 10	2	65.4
No. 40	0.425	22.7
No. 200	0.075	4.7
	0.02	2.8
	0.005	1.7
	0.002	1.3
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	22.3	34.6
Coarse Sand	12.3	42.7
Medium Sand	42.7	---
Fine Sand	18.0	18.0
Silt	3.0	3.4
Clay	1.7	1.3

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.56

Classification
Unified Group Symbol: SW
Group Name: Well-graded sand with gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-74 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-9/193+95, 70' Rt., 65.6'-67.1', 70.6'-72.1'
Project Number: LX2005125
Lab ID: 76
County: Jefferson
Sample Type: SPT Composite
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	88.1
3/4"	19	70.4
3/8"	9.5	51.5
No. 4	4.75	40.4
No. 10	2	31.6
No. 40	0.425	18.9
No. 200	0.075	6.4
	0.02	3.1
	0.005	1.4
	0.002	0.8
estimated	0.001	0.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	59.6	68.4
Coarse Sand	8.8	12.7
Medium Sand	12.7	---
Fine Sand	12.5	12.5
Silt	5.0	5.6
Clay	1.4	0.8

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: GW-GM
Group Name: Well-graded gravel with silt and sand
AASHTO Classification: A-1-a (1)

Comments:

Reviewed by:

File: LX2005125_Sum-76 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May

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Summary of Soil Tests

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-9/193+95, 70' Rt., 75.6'-77.1', 80.6'-82.1', 85.6'-87.1'	Lab ID	77
County	Jefferson	Date Received	8-10-07
Sample Type	SPT Composite	Date Reported	10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	93.1
3/8"	9.5	88.2
No. 4	4.75	84.4
No. 10	2	78.9
No. 40	0.425	40.9
No. 200	0.075	7.4
	0.02	3.6
	0.005	2.1
	0.002	1.7
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	15.6	21.1
Coarse Sand	5.5	38.0
Medium Sand	38.0	---
Fine Sand	33.5	33.5
Silt	5.3	5.7
Clay	2.1	1.7

Atterberg Limits
Assumed Non Plastic

Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.68

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel

AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-77 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Fuller Mossbarger Scott & May

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Summary of Soil Tests

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-10/205+98, 70' Lt., 79.0'-80.5', 84.0'-84.2'	Lab ID	89
County	Jefferson	Date Received	8-10-07
Sample Type	SPT Composite	Date Reported	10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	99.2
No. 4	4.75	94.9
No. 10	2	86.9
No. 40	0.425	40.0
No. 200	0.075	10.0
	0.02	5.9
	0.005	3.0
	0.002	2.0
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	5.1	13.1
Coarse Sand	8.0	46.9
Medium Sand	46.9	---
Fine Sand	30.0	30.0
Silt	7.0	8.0
Clay	3.0	2.0

Atterberg Limits
Assumed Non Plastic

Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.69

Classification
Unified Group Symbol: SW-SM
Group Name: Well-graded sand with silt

AASHTO Classification: A-1-b (0)

Comments:

Reviewed by:

File: LX2005125_Sum-89 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River — Project Number: LX2005125
Source: AC-11/205+95,1.3 Lt., 41.7'-43.2', 46.7'-48.2', 51.7'-53.2' — Lab ID: 98
County: Jefferson — Date Received: 8-10-07
Sample Type: SPT Composite — Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	96.1
3/4"	19	92.8
3/8"	9.5	56.1
No. 4	4.75	30.5
No. 10	2	11.8
No. 40	0.425	3.3
No. 200	0.075	1.2
	0.02	0.6
	0.005	0.5
	0.002	0.2
estimated	0.001	0.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	69.5	88.2
Coarse Sand	18.7	8.5
Medium Sand	8.5	---
Fine Sand	2.1	2.1
Silt	0.7	1.0
Clay	0.5	0.2

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.66

Classification
Unified Group Symbol: GW
Group Name: Well-graded gravel with sand
AASHTO Classification: A-1-a (1)

Comments:

Reviewed by:

File: LX2005125_Sum-98 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

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Laboratory Document
Prepared By: MW
Approved BY: TLK

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Summary of Soil Tests

Project Name: I-265 Over Ohio River — Project Number: LX2005125
Source: AC-11/205+95,1.3 Lt., 57.1'-58.2', 61.7'-63.2', 66.7'-68.2' — Lab ID: 100
County: Jefferson — Date Received: 8-10-07
Sample Type: SPT Composite — Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	96.9
3/8"	9.5	87.4
No. 4	4.75	71.9
No. 10	2	56.9
No. 40	0.425	31.5
No. 200	0.075	6.9
	0.02	2.8
	0.005	1.4
	0.002	0.9
estimated	0.001	0.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	28.1	43.1
Coarse Sand	15.0	25.4
Medium Sand	25.4	---
Fine Sand	24.6	24.6
Silt	5.5	6.0
Clay	1.4	0.9

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.65

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-100 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-11/205+95,1.3 Lt., 71.7'-73.2', 76.7'-78.2'	Lab ID	101
County	Jefferson	Date Received	8-10-07
Sample Type	SPT Composite	Date Reported	10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	94.5
3/8"	9.5	89.8
No. 4	4.75	84.6
No. 10	2	75.5
No. 40	0.425	37.8
No. 200	0.075	8.6
	0.02	3.7
	0.005	2.0
	0.002	1.3
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	15.4	24.5
Coarse Sand	9.1	37.7
Medium Sand	37.7	---
Fine Sand	29.2	29.2
Silt	6.6	7.3
Clay	2.0	1.3

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-101 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-12/205+94, 71 Rt., 55.5'-57.0'	Lab ID	112
County	Jefferson	Date Received	8-10-07
Sample Type	SPT	Date Reported	10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	100.0
3/8"	9.5	92.4
No. 4	4.75	87.3
No. 10	2	83.1
No. 40	0.425	39.2
No. 200	0.075	4.5
	0.02	1.8
	0.005	1.0
	0.002	0.8
estimated	0.001	0.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	12.7	16.9
Coarse Sand	4.2	43.9
Medium Sand	43.9	---
Fine Sand	34.7	34.7
Silt	3.5	3.7
Clay	1.0	0.8

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.68

Classification
Unified Group Symbol: SP
Group Name: Poorly graded sand
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-112 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River — Project Number: LX2005125
Source: AC-12/205+94, 71 Rt., 60.5'-62.0', 65.5'-67.0', 70.5'-72.0' — Lab ID: 113
County: Jefferson — Date Received: 8-10-07
Sample Type: SPT Composite — Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	84.7
3/8"	9.5	69.7
No. 4	4.75	56.5
No. 10	2	43.8
No. 40	0.425	25.4
No. 200	0.075	5.3
	0.02	3.0
	0.005	1.8
	0.002	1.3
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	43.5	56.2
Coarse Sand	12.7	18.4
Medium Sand	18.4	---
Fine Sand	20.1	20.1
Silt	3.5	4.0
Clay	1.8	1.3

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.70

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel
AASHTO Classification: A-1-a (1)

Comments:

Reviewed by:

File: LX2005125_Sum-113 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FMSM ENGINEERS

Project Name: I-265 Over Ohio River — Project Number: LX2005125
Source: AC-13/206+50, 0.02 Lt., 67.0'-67.6', 67.6'-68.5', 72.0'-73.5' — Lab ID: 125
County: Jefferson — Date Received: 8-10-07
Sample Type: SPT Composite — Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	98.8
3/8"	9.5	88.3
No. 4	4.75	82.2
No. 10	2	74.4
No. 40	0.425	39.1
No. 200	0.075	7.3
	0.02	4.0
	0.005	2.9
	0.002	2.1
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	17.8	25.6
Coarse Sand	7.8	35.3
Medium Sand	35.3	---
Fine Sand	31.8	31.8
Silt	4.4	5.2
Clay	2.9	2.1

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.71

Classification
Unified Group Symbol: SP-SM
Group Name: Poorly graded sand with silt and gravel
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-125 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

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Summary of Soil Tests

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-13/206+50, 0.02 Lt., 77.4'-78.5', 82.0'-82.7', 82.7'-83.0'
Lab ID: 126
County: Jefferson
Date Received: 8-10-07
Sample Type: SPT Composite
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	78.0
3/8"	9.5	63.4
No. 4	4.75	47.4
No. 10	2	31.5
No. 40	0.425	21.2
No. 200	0.075	9.2
	0.02	5.8
	0.005	3.3
	0.002	2.3
estimated	0.001	2.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	52.6	68.5
Coarse Sand	15.9	10.3
Medium Sand	10.3	---
Fine Sand	12.0	12.0
Silt	5.9	6.9
Clay	3.3	2.3

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.74

Classification
Unified Group Symbol: GP-GM
Group Name: Poorly graded gravel with silt and sand
AASHTO Classification: A-1-b (1)

Comments:

Reviewed by:

File: LX2005125_Sum-126 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

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Summary of Soil Tests

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-14, 0.0'-12.1'
Lab ID: 226
County: Jefferson
Date Received: 10-23-07
Sample Type: Bag
Date Reported: 11-22-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	
3/4"	19	
3/8"	9.5	100.0
No. 4	4.75	99.1
No. 10	2	77.6
No. 40	0.425	69.9
No. 200	0.075	67.3
	0.02	59.5
	0.005	37.7
	0.002	29.4
estimated	0.001	25.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	0.9	22.4
Coarse Sand	21.5	7.7
Medium Sand	7.7	---
Fine Sand	2.6	2.6
Silt	29.6	37.9
Clay	37.7	29.4

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 48
Plastic Limit: 15
Plasticity Index: 33
Activity Index: 1.14

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.75

Classification
Unified Group Symbol: CL
Group Name: Sandy lean clay
AASHTO Classification: A-7-6 (20)

Comments:

Reviewed by:

File: LX2005125_Sum-226 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FULLER MOSSBARGER SCOTT & MAY — FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-15/210+20, 37.3 Rt., 4.2'-5.3', 5.3'-5.7', 6.7'-7.2'
Project Number: LX2005125
Lab ID: 135
County: Jefferson
Sample Type: SPT Composite
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Sieve Size	(mm)	% Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	89.8
3/8"	9.5	86.4
No. 4	4.75	81.7
No. 10	2	68.2
No. 40	0.425	66.6
No. 200	0.075	61.4
	0.02	47.0
	0.005	27.1
	0.002	18.7
estimated	0.001	13.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	18.3	31.8
Coarse Sand	13.5	1.6
Medium Sand	1.6	---
Fine Sand	5.2	5.2
Silt	34.3	42.7
Clay	27.1	18.7

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Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.65

Classification
Unified Group Symbol: ML
Group Name: Sandy silt with gravel
AASHTO Classification: A-4 (0)

Comments:

Reviewed by:

File: LX2005125_Sum-135 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Summary of Soil Tests

FULLER MOSSBARGER SCOTT & MAY — FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-15/210+20, 37.3 Rt., 10.0'-11.5', 12.7'-14.2'
Project Number: LX2005125
Lab ID: 137
County: Jefferson
Sample Type: SPT Composite
Date Received: 8-10-07
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Sieve Size	(mm)	% Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	87.1
3/8"	9.5	63.1
No. 4	4.75	43.7
No. 10	2	30.9
No. 40	0.425	24.6
No. 200	0.075	9.7
	0.02	4.1
	0.005	2.1
	0.002	1.3
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	56.3	69.1
Coarse Sand	12.8	6.3
Medium Sand	6.3	---
Fine Sand	14.9	14.9
Silt	7.6	8.4
Clay	2.1	1.3

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.79

Classification
Unified Group Symbol: GP-GM
Group Name: Poorly graded gravel with silt and sand
AASHTO Classification: A-1-a (0)

Comments:

Reviewed by:

File: LX2005125_Sum-137 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-15/210+20, 37.3 Rt., 15.5'-17.0', 21.7'-23.2', 26.7'-26.9'
Lab ID: 138
County: Jefferson
Date Received: 8-10-07
Sample Type: SPT Composite
Date Reported: 10-17-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	89.6
3/4"	19	79.2
3/8"	9.5	55.5
No. 4	4.75	36.2
No. 10	2	22.3
No. 40	0.425	17.4
No. 200	0.075	11.8
	0.02	5.2
	0.005	2.4
	0.002	1.6
estimated	0.001	1.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	63.8	77.7
Coarse Sand	13.9	4.9
Medium Sand	4.9	---
Fine Sand	5.6	5.6
Silt	9.4	10.2
Clay	2.4	1.6

Atterberg Limits
Assumed Non Plastic
Liquid Limit: ---
Plastic Limit: Non Plastic
Plasticity Index: ---
Activity Index: N/A

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.79

Classification
Unified Group Symbol: GP-GM
Group Name: Poorly graded gravel with silt and sand
AASHTO Classification: A-1-a (0)

Comments:

Reviewed by:

17 20 23

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Bridge over the Ohio River
Project Number: LX2005125
Source: AC-17/212+17.0, 87.0 Lt., 0.0'-1.5' & AC-20/212+30.0, 56.0 Lt., 0.0'-1.5' & AC-23/ 212+50.0 Cl., 2.5'-3.6'
Lab ID: 231
County: Jefferson
Date Received: 10-23-07
Sample Type: SPT Composite
Date Reported: 11-5-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	100.0
1"	25	93.5
3/4"	19	88.6
3/8"	9.5	81.0
No. 4	4.75	64.1
No. 10	2	47.7
No. 40	0.425	42.6
No. 200	0.075	38.5
	0.02	29.7
	0.005	17.5
	0.002	14.2
estimated	0.001	12.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	35.9	52.3
Coarse Sand	16.4	5.1
Medium Sand	5.1	---
Fine Sand	4.1	4.1
Silt	21.0	24.3
Clay	17.5	14.2

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 39
Plastic Limit: 15
Plasticity Index: 24
Activity Index: 1.71

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.76

Classification
Unified Group Symbol: GC
Group Name: Clayey gravel with sand
AASHTO Classification: A-6 (4)

Comments:

Reviewed by:

FMSM ENGINEERS

Summary of Soil Tests

Project Name: I-265 Over Ohio River
Source: AC-26/ 212+70.0, 55.0 Rt., 0.0'-1.5', 5.0'-6.5'
Project Number: LX2005125
Lab ID: 241

County: Jefferson
Sample Type: SPT Composite
Date Received: 10-23-07
Date Reported: 11-5-07

Test Results

Natural Moisture Content
Test Not Performed
Moisture Content (%): N/A

Particle Size Analysis
Preparation Method: AASHTO T 87
Gradation Method: AASHTO T 88
Hydrometer Method: AASHTO T 88

Particle Size		%
Sieve Size	(mm)	Passing
3"	75	
2"	50	
1 1/2"	37.5	
1"	25	100.0
3/4"	19	97.3
3/8"	9.5	94.2
No. 4	4.75	87.9
No. 10	2	65.1
No. 40	0.425	60.6
No. 200	0.075	58.3
	0.02	44.8
	0.005	26.4
	0.002	21.6
estimated	0.001	18.0

Plus 3 in. material, not included: 0 (%)

Range	ASTM (%)	AASHTO (%)
Gravel	12.1	34.9
Coarse Sand	22.8	4.5
Medium Sand	4.5	---
Fine Sand	2.3	2.3
Silt	31.9	36.7
Clay	26.4	21.6

Atterberg Limits
Test Method: AASHTO T 89 & T 90
Prepared: Dry
Liquid Limit: 41
Plastic Limit: 16
Plasticity Index: 25
Activity Index: 1.14

Moisture-Density Relationship
Test Not Performed
Maximum Dry Density (lb/ft^3): N/A
Maximum Dry Density (kg/m^3): N/A
Optimum Moisture Content (%): N/A
Over Size Correction %: N/A

California Bearing Ratio
Test Not Performed
Bearing Ratio (%): N/A
Compacted Dry Density (lb/ft^3): N/A
Compacted Moisture Content (%): N/A

Specific Gravity
Test Method: AASHTO T 100
Prepared: Dry
Particle Size: No. 10
Specific Gravity at 20° Celsius: 2.73

Classification
Unified Group Symbol: CL
Group Name: Sandy lean clay
AASHTO Classification: A-7-6 (11)

Comments:

Reviewed by:

File: LX2005125_Sum-241 Sheet: Summary
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

FMSM ENGINEERS

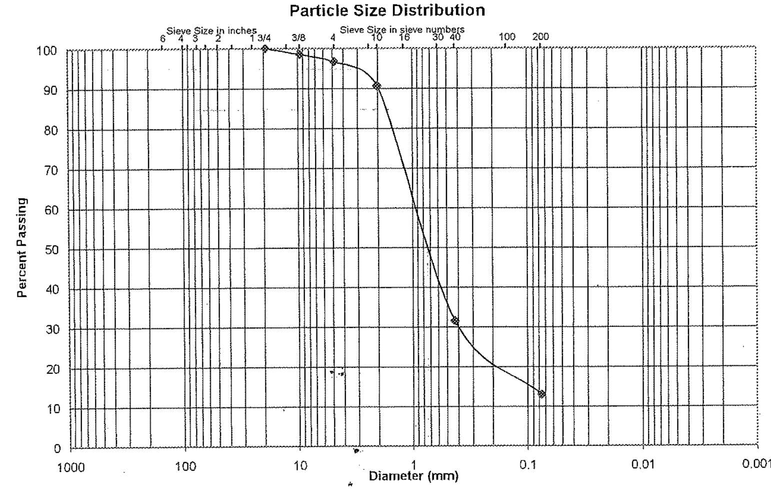
Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Source: AC-1, 50.0'-51.5'
Project Number: LX2005125
Lab ID: 199

Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: DG
Test Date: 10-17-2007
Date Received: 09-28-2007
Sample Dry Mass (g): 359.48
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	5.26	1.5	98.5
No. 4	7.09	1.9	96.6
No. 10	21.79	6.1	90.5
No. 40	212.6	59.1	31.4
No. 200	66.37	18.5	12.9
Pan	45.54	12.7	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-199.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

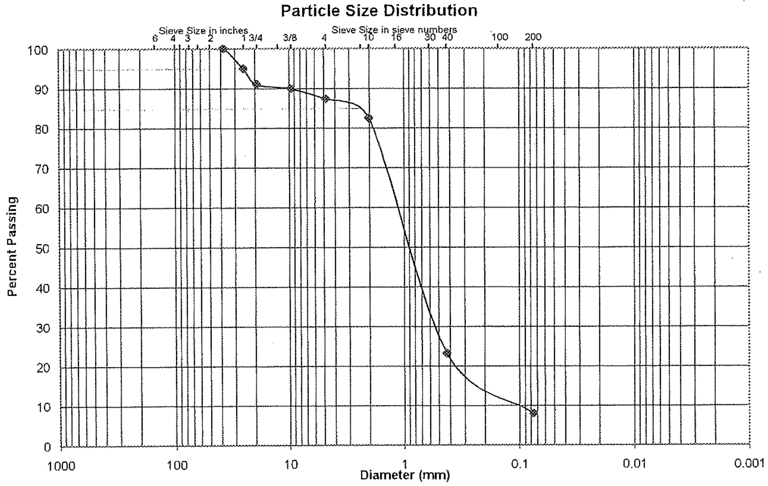
FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-1, 70.0'-71.5'
Project Number: LX2005125
Lab ID: 200

Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: DG
Test Date: 10-17-2007
Date Received: 09-28-2007
Sample Dry Mass (g): 356.87
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	18.13	5.1	94.9
3/4"	14.02	3.9	91.0
3/8"	4.04	1.1	89.9
No. 4	9.15	2.6	87.3
No. 10	17.45	4.9	82.4
No. 40	211.78	59.3	23.1
No. 200	53.99	15.2	7.9
Pan	28.95	8.1	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-200.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

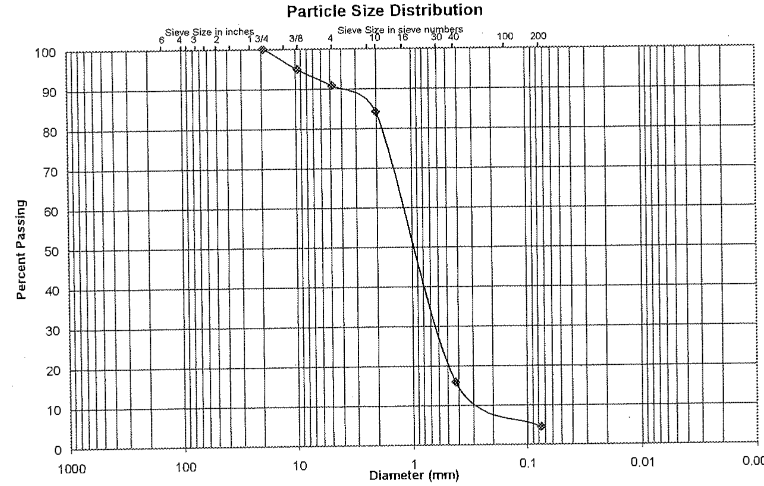
FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-1, 95.0'-96.5', 100.0'-100.3'
Project Number: LX2005125
Lab ID: 153

Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: CSM
Test Date: 10-17-2007
Date Received: 09-28-2007
Sample Dry Mass (g): 365.13
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	18.45	5.1	94.9
No. 4	15.56	4.2	90.7
No. 10	24.19	6.6	84.1
No. 40	249.23	68.3	15.8
No. 200	41.61	11.4	4.4
Pan	16.28	4.5	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-153.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

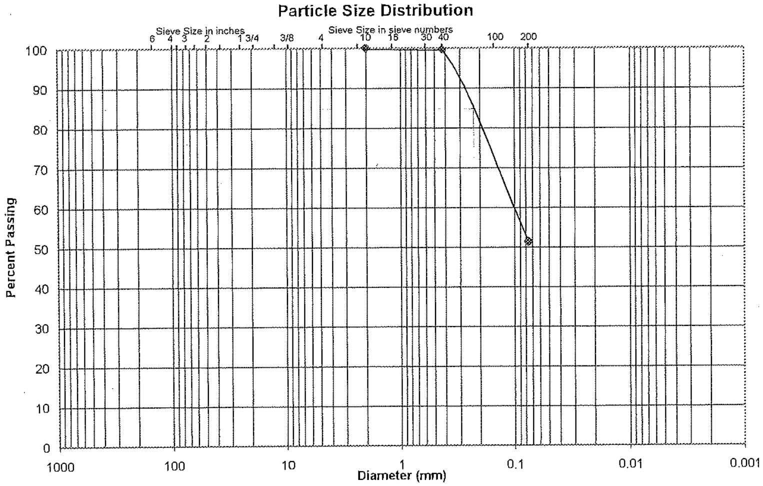
Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River
Source AC-2, 15.0'-16.5'
Project Number LX2005125
Lab ID 162

Particle Shape N/A Particle Hardness N/A Prepared AASHTO T 11 Method A
Tested by CSM Test Date 10-17-2007 Date Received 09-28-2007
Sample Dry Mass (g) 227.91 Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"			
3/8"			
No. 4			
No. 10	0	0.0	100.0
No. 40	0.9	0.4	99.6
No. 200	110.12	48.3	51.3
Pan	118.13	51.8	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-162.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

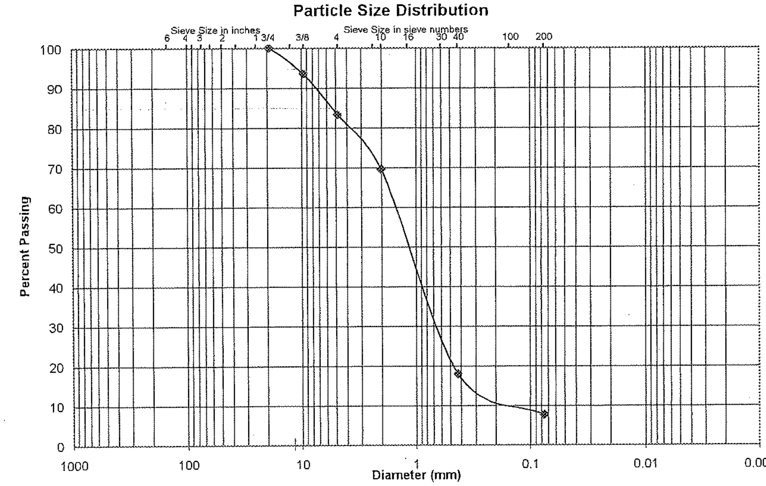
Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River
Source AC-2, 55.0'-56.5'
Project Number LX2005125
Lab ID 165

Particle Shape Angular Particle Hardness Hard and Durable Prepared AASHTO T 11 Method A
Tested by CSM Test Date 10-17-2007 Date Received 09-28-2007
Sample Dry Mass (g) 357.19 Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	23.31	6.5	93.5
No. 4	36.69	10.3	83.2
No. 10	48.77	13.7	69.5
No. 40	184.63	51.6	17.9
No. 200	36.49	10.3	7.6
Pan	26.38	7.3	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-165.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

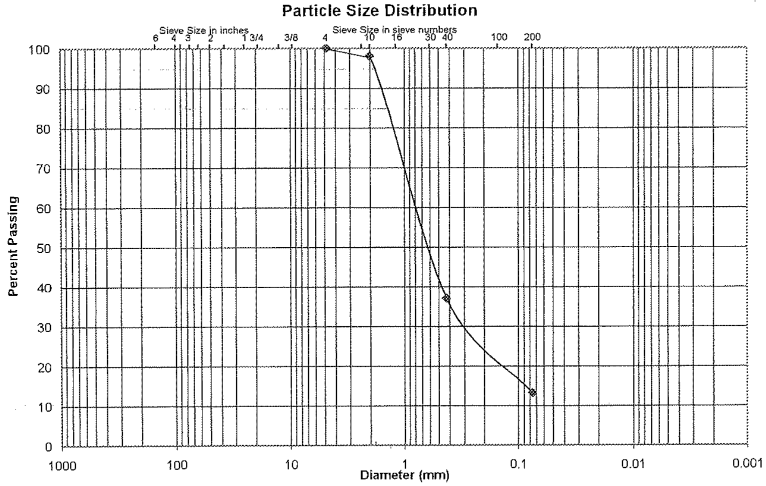
Gradation Analysis
AASHTO T 88

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-2, 80.0'-81.5'
Project Number: LX2005125
Lab ID: 168
Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: CSM
Test Date: 10-17-2007
Date Received: 09-28-2007
Sample Dry Mass (g): 310.41
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"			
3/8"			
No. 4	0	0.0	100.0
No. 10	6.35	2.0	98.0
No. 40	189.35	61.0	37.0
No. 200	73.93	23.9	13.1
Pan	41.54	13.3	---

Particle Size Distribution



Comments ______

Reviewed By ______

File: LX2005125_200-168.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

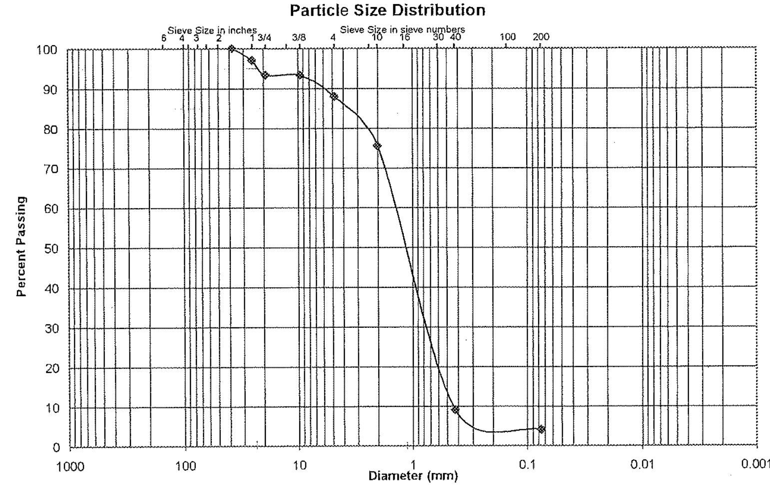
Gradation Analysis
AASHTO T 88

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Source: AC-3, 40.0'-41.5'
Project Number: LX2005125
Lab ID: 183
Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: CSM
Test Date: 10-17-2007
Date Received: 09-28-2007
Sample Dry Mass (g): 382.44
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	11.36	3.0	97.0
3/4"	14.3	3.7	93.3
3/8"	0	0.0	93.3
No. 4	20.44	5.4	87.9
No. 10	47.68	12.4	75.5
No. 40	254.42	66.5	9.0
No. 200	18.8	5.0	4.0
Pan	15.32	4.0	---

Particle Size Distribution



Comments ______

Reviewed By ______

File: LX2005125_200-183.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

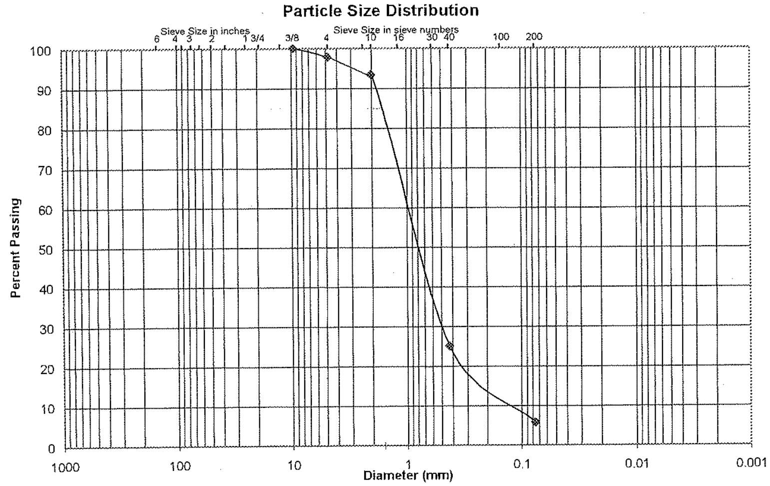
Laboratory Document
Prepared By: MW
Approved By: TLK

FMSM ENGINEERS

Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River
Project Number LX2005125
Source AC-3, 90.0'-91.5'
Lab ID 186
Particle Shape Angular
Particle Hardness Hard and Durable
Prepared AASHTO T 11 Method A
Tested by CSM
Test Date 10-17-2007
Date Received 09-28-2007
Sample Dry Mass (g) 256.85
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"			
3/8"	0	0.0	100.0
No. 4	5.76	2.2	97.8
No. 10	11.48	4.5	93.3
No. 40	175.44	68.3	25.0
No. 200	49.46	19.3	5.7
Pan	15.84	6.1	---



Comments

Reviewed By

File: LX2005125_200-186.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

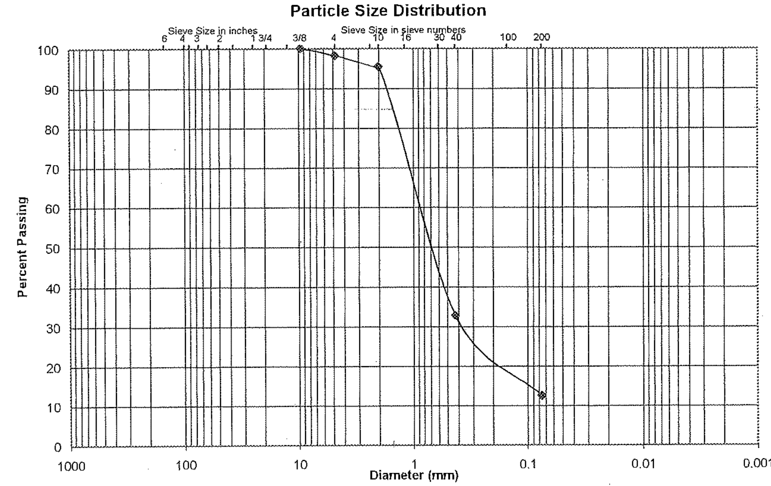
Laboratory Document
Prepared By: MW
Approved By: TLK

FMSM ENGINEERS

Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River
Project Number LX2005125
Source AC-3, 95.0'-96.5'
Lab ID 187
Particle Shape Angular
Particle Hardness Hard and Durable
Prepared AASHTO T 11 Method A
Tested by CSM
Test Date 10-17-2007
Date Received 09-28-2007
Sample Dry Mass (g) 337.18
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"			
3/8"	0	0.0	100.0
No. 4	6.06	1.8	98.2
No. 10	9.31	2.8	95.4
No. 40	211.7	62.7	32.7
No. 200	68.24	20.3	12.4
Pan	41.1	12.2	---



Comments

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File: LX2005125_200-187.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-4/189+81.55, 91.9 Lt., 15.7'-17.2', 21.7'-23.2', 26.7'-28.2' — Lab ID 21
Particle Shape Rounded — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A
Tested by DG — Test Date 08-22-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 395.5 — Analysis based on: Total Sample

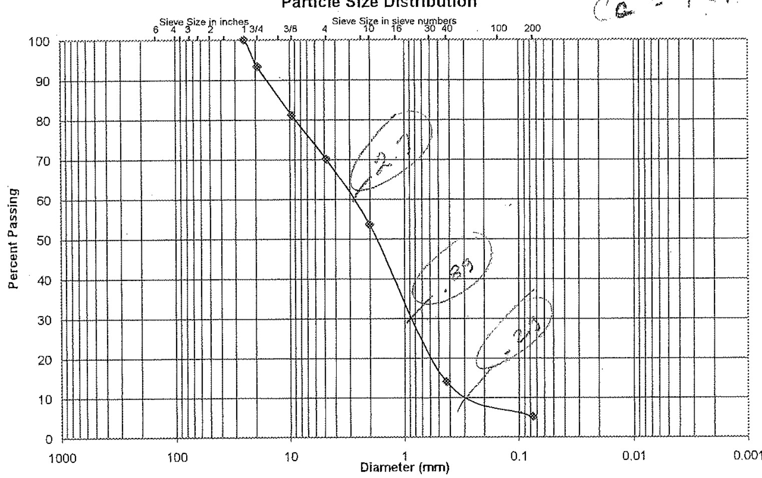
Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	26.57	6.7	93.3
3/8"	48.22	12.2	81.1
No. 4	44.05	11.1	70.0
No. 10	65.05	16.5	53.5
No. 40	156.7	39.6	13.9
No. 200	34.76	8.8	5.1
Pan	21.15	5.4	---

64.9 gravel
5.1 sand 1→30

Well graded sand w/ silt and gravel (SW-SM) A-1-b

Cu = 9.3
CG = 1.01
KG

Particle Size Distribution



Comments ______

Reviewed By ______

File: LX2005125_200-21 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-4/189+81.55, 91.9 Lt., 31.7'-33.2', 36.7'-38.2', 41.7'-43.2' — Lab ID 22
Particle Shape Rounded — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A
Tested by DG — Test Date 08-22-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 499.81 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	35.1	7.0	93.0
3/8"	21.55	4.3	88.7
No. 4	47.74	9.6	79.1
No. 10	87.61	17.5	61.6
No. 40	238.34	47.7	13.9
No. 200	52.09	10.4	3.5
Pan	18.61	3.7	---

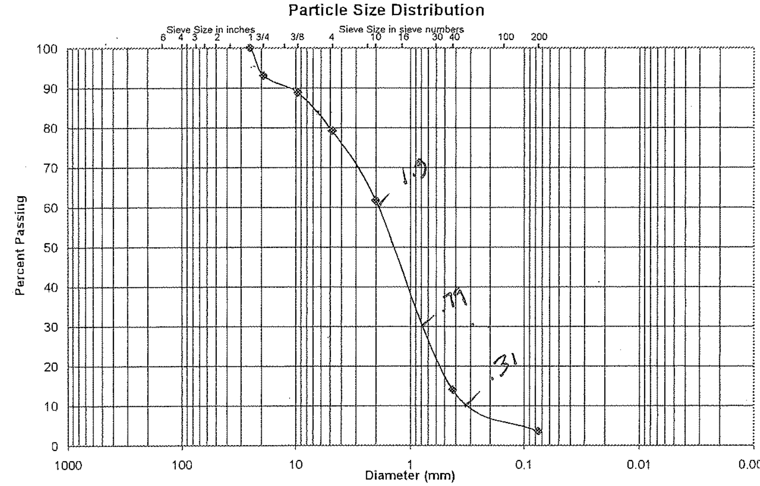
21
75

Well graded sand w/ gravel (SW)

A-1-b KG

Cu = 6.1
CG = 1.06

Particle Size Distribution



Comments ______

Reviewed By ______

File: LX2005125_200-22 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

ENGINEERS

Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-4/189+81.55, 91.9 Lt., 46.7'-48.2', 51.7'-53.2', 56.7'-58.2' — Lab ID 23
Particle Shape Rounded — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A
Tested by DG — Test Date 08-22-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 306.88 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	25.01	8.1	91.9
3/4"	0	0.0	91.9
3/8"	15.35	5.1	86.8
No. 4	13.99	4.5	82.3
No. 10	22.19	7.2	75.1
No. 40	176.19	57.5	17.6
No. 200	44.56	14.5	3.1
Pan	10.23	3.3	---

gravel 18
79
3

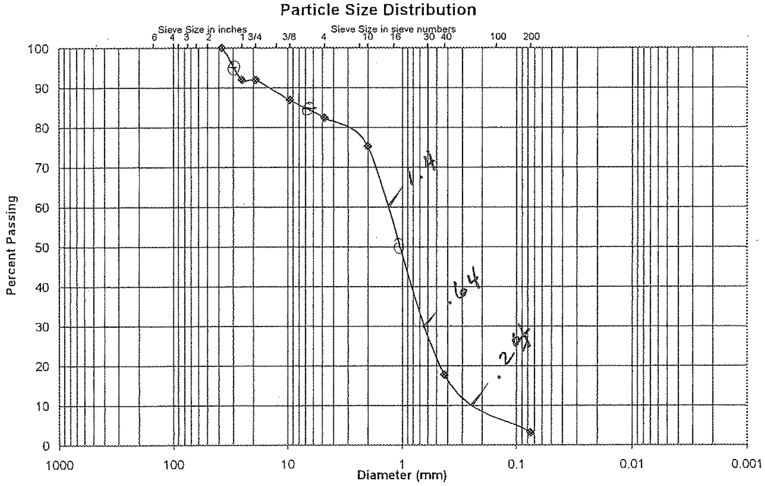
Poorly graded sand w/ gravel
SP
A-1-b

$Cu = \frac{1.4}{.24} = 5.83$

$Cc = \frac{(.64)^2}{.24 \times 1.4} = 1.22$

KG

Particle Size Distribution



Comments

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File: LX2005125_200-23 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

ENGINEERS

Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-4/189+81.55, 91.9 Lt., 61.7'-63.2', 66.7'-68.2' — Lab ID 24
Particle Shape Rounded — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A
Tested by DG — Test Date 08-22-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 521.73 — Analysis based on: Total Sample

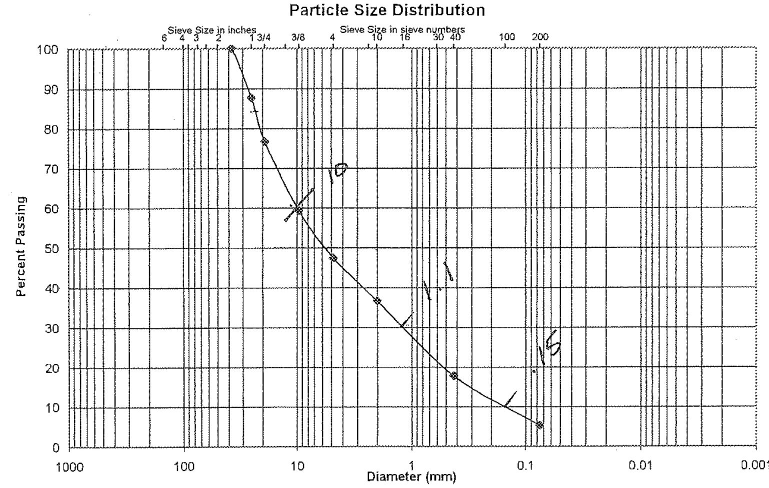
Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	64.68	12.4	87.6
3/4"	57.41	11.0	76.6
3/8"	90.76	17.4	59.2
No. 4	61.95	11.9	47.3
No. 10	56.14	10.7	36.6
No. 40	98.65	18.9	17.7
No. 200	64.92	12.5	5.2
Pan	28.05	5.4	---

53
42
5.

Poorly graded gravel w/ silt and sand
GP-GM
A-1-a
KG

$Cu = 66.7$
$Cc = .81$

Particle Size Distribution



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File: LX2005125_200-24 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

FMSM ENGINEERS

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-4/189+81.55, 91.9 Lt., 82.4'-83.2'	Lab ID	26
Particle Shape	Rounded	Particle Hardness	Hard and Durable
Prepared	AASHTO T 11 Method A		
Tested by	RC	Test Date	08-17-2007
Date Received	08-10-2007		
Sample Dry Mass (g)	255.02	Analysis based on:	Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	21.92	8.6	91.4
3/8"	18.87	7.4	84.0
No. 4	22.34	8.8	75.2
No. 10	28.33	11.1	64.1
No. 40	104.17	40.8	23.3
No. 200	43.05	16.9	6.4
Pan	16.57	6.5	---

25

69

6

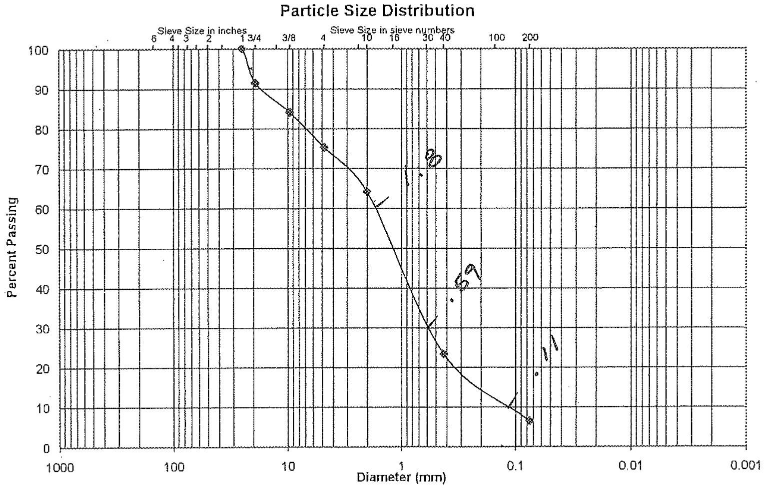
well graded sand w/silt and gravel (SW-SM)

A-1-b

Cu = 16.4 KG

Cc = ~~21.6~~ 1.76

Particle Size Distribution



Comments ______

Reviewed By ______

Gradation Analysis
AASHTO T 88

FMSM ENGINEERS

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-4/189+81.55, 91.9 Lt., 86.8'-87.3'	Lab ID	27
Particle Shape	Angular	Particle Hardness	Hard and Durable
Prepared	AASHTO T 11 Method A		
Tested by	RC	Test Date	08-17-2007
Date Received	08-10-2007		
Sample Dry Mass (g)	122.45	Analysis based on:	Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	16.65	13.6	86.4
No. 4	19.1	15.6	70.8
No. 10	18.39	15.0	55.8
No. 40	27.94	22.8	33.0
No. 200	17.99	14.7	18.3
Pan	22.48	18.4	---

29

53

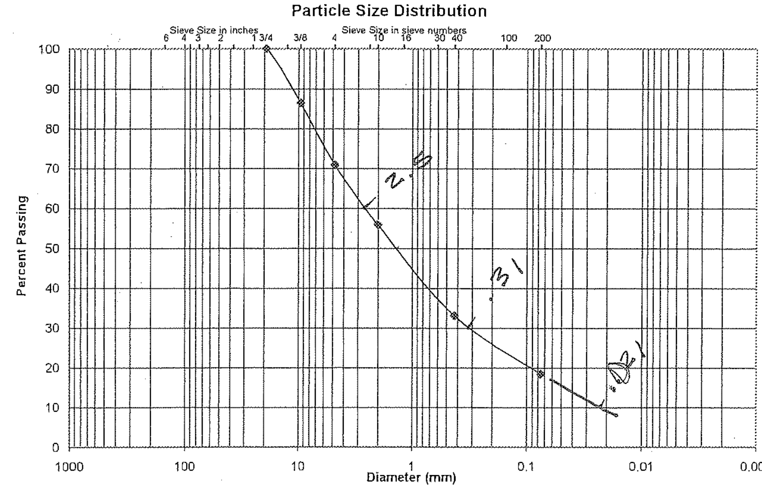
18

Silty Sand with gravel (SM) A-1-b

Cu = ~~11.9~~ 119.0 KG

Cc = ~~0.183~~ 1.83

Particle Size Distribution



Comments ______

Reviewed By ______

Gradation Analysis
AASHTO T 88

Fuller Mossbarger Scott & May ENGINEERS

Project Name I-265 Over Ohio River
Project Number LX2005125
Source AC-4/189+81.55, 91.9 Lt., 87.3'-87.4', 91.7'-92.1'
Lab ID 28
Particle Shape Rounded
Particle Hardness Hard and Durable
Prepared AASHTO T 11 Method A
Tested by RC/RM
Test Date 08-20-2007
Date Received 08-10-2007
Sample Dry Mass (g) 257.66
Analysis based on: Total Sample

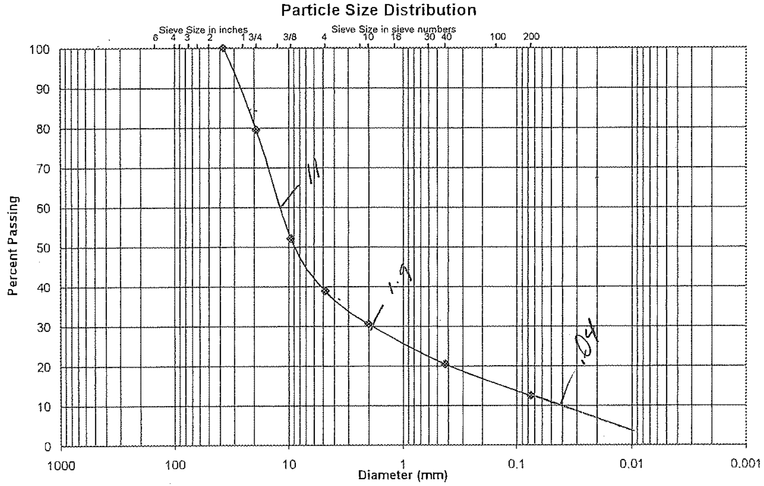
Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	---	---	---
3/4"	53	20.6	79.4
3/8"	70.64	27.4	52.0
No. 4	33.94	13.2	38.8
No. 10	21.6	8.3	30.5
No. 40	25.79	10.1	20.4
No. 200	20.71	8.0	12.4
Pan	32.08	12.4	---

Silty gravel with sand (GM) A-1-a KG

$C_u = \cancel{27.5}$ 275.0

$C_c = \cancel{0.82}$ 8.2

61 / 29 / 18



Comments

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File: LX2005125_200-28 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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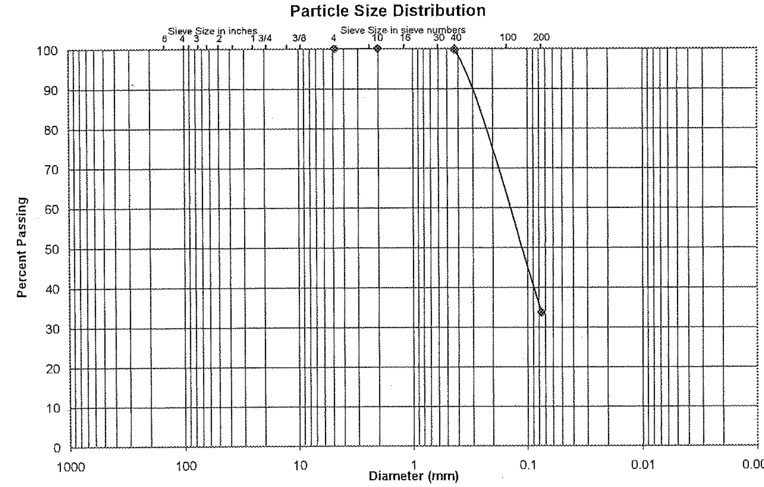
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Fuller Mossbarger Scott & May ENGINEERS

Project Name I-265 Over Ohio River
Project Number LX2005125
Source AC-5, 7.0'-8.2'
Lab ID 203
Particle Shape Angular
Particle Hardness Soft
Prepared AASHTO T 11 Method A
Tested by AW
Test Date 10-23-2007
Date Received 10-23-2007
Sample Dry Mass (g) 276.59
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"			
3/8"			
No. 4	0	0.0	100.0
No. 10	0.07	0.0	100.0
No. 40	0.69	0.3	99.7
No. 200	182.79	66.1	33.6
Pan	92.53	33.4	---



Comments

Reviewed By

File: LX2005125_200-203 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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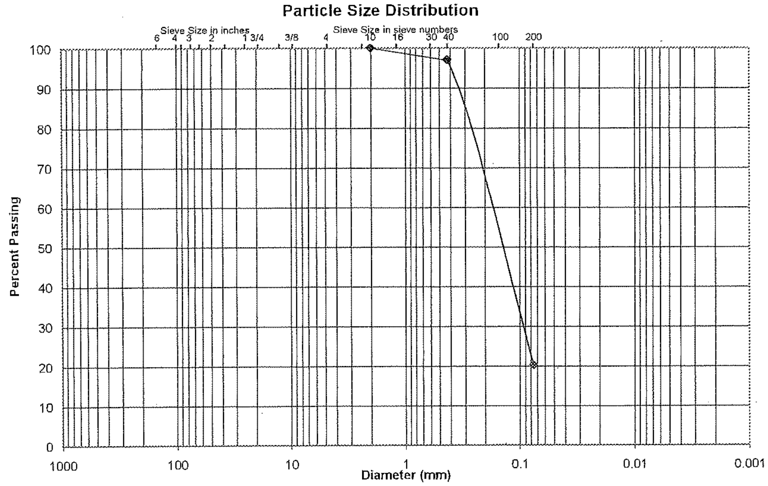
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River | Project Number LX2005125
Source AC-5, 8.2'-8.5' | Lab ID 204

Particle Shape N/A | Particle Hardness N/A | Prepared AASHTO T 11 Method A
Tested by JF | Test Date 10-23-2007 | Date Received 10-23-2007
Sample Dry Mass (g) 145.24 | Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"			
3/8"			
No. 4			
No. 10	0	0.0	100.0
No. 40	4.33	3.0	97.0
No. 200	111.57	76.8	20.2
Pan	29.02	20.0	---

Particle Size Distribution


Comments ______

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File: LX2005125_200-204 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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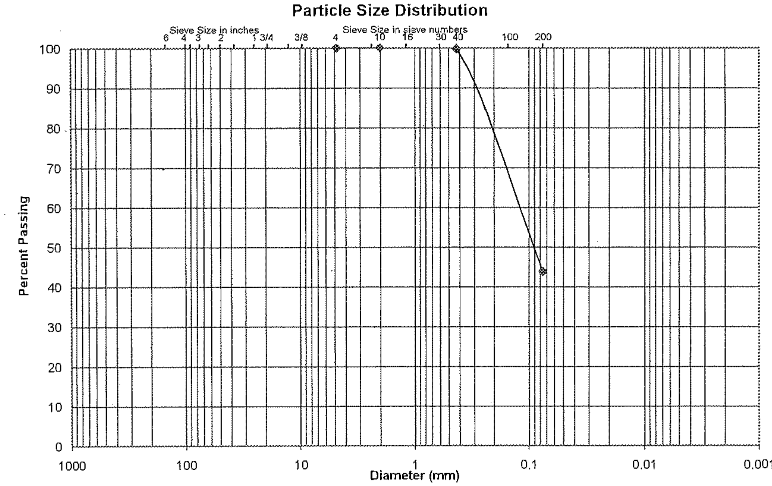
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River | Project Number LX2005125
Source AC-5, 10.0'-10.6' | Lab ID 205

Particle Shape Angular | Particle Hardness Hard and Durable | Prepared AASHTO T 11 Method A
Tested by AW | Test Date 10-23-2007 | Date Received 10-23-2007
Sample Dry Mass (g) 193.67 | Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"			
3/8"			
No. 4	0	0.0	100.0
No. 10	0.03	0.0	100.0
No. 40	0.48	0.3	99.7
No. 200	108.61	56.0	43.7
Pan	84.18	43.5	---

Particle Size Distribution


Comments ______

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File: LX2005125_200-205 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

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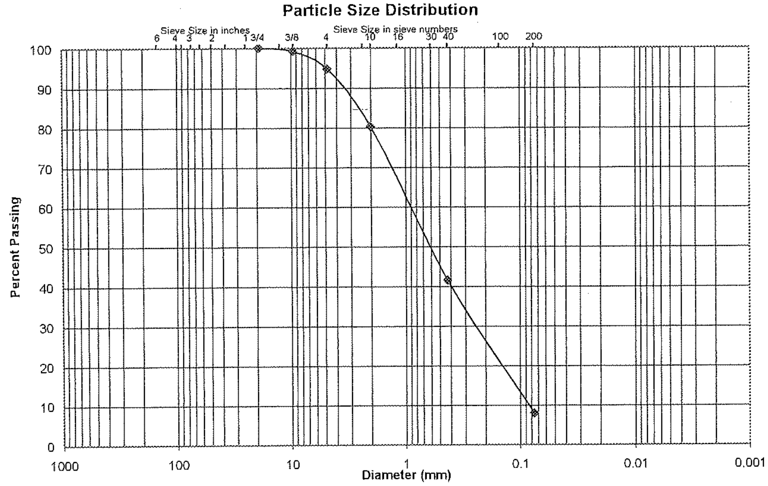
Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-5, 50.0'-51.5'
Lab ID: 215
Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: AW
Test Date: 10-23-2007
Date Received: 10-23-2007
Sample Dry Mass (g): 228.78
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	1.78	0.8	99.2
No. 4	10.45	4.5	94.7
No. 10	33.03	14.5	80.2
No. 40	88.53	38.7	41.5
No. 200	76.88	33.6	7.9
Pan	18.23	8.0	---

Particle Size Distribution



Comments

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Gradation Analysis
AASHTO T 88

FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-6/193+51.7, 0.28' Rt., 40.9'-42.4', 45.9'-47.4', 50.9'-52.4'
Lab ID: 35
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: DG
Test Date: 08-22-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 531.39
Analysis based on: Total Sample

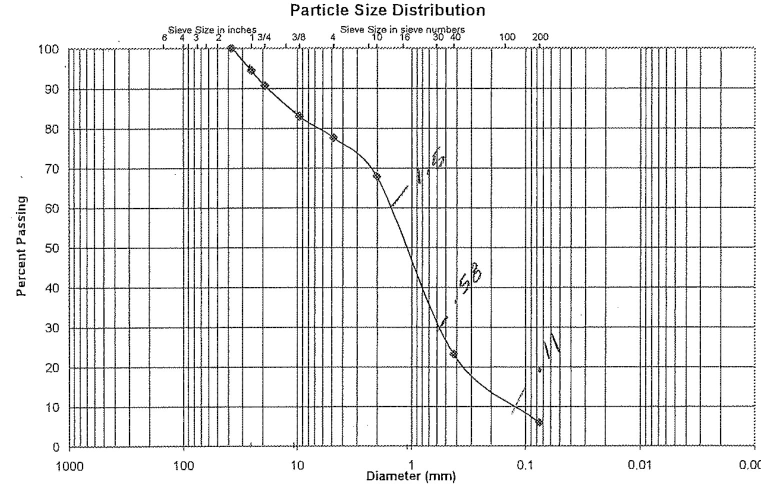
Well graded Sand w/silt and gravel (SW-SM) A-1-b KG

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	29.71	5.6	94.4
3/4"	20.01	3.8	90.6
3/8"	41.04	7.7	82.9
No. 4	28.88	5.4	77.5
No. 10	51.59	9.7	67.8
No. 40	237.3	44.7	23.1
No. 200	91.95	17.3	5.8
Pan	30.9	5.8	---

22
72
6

$C_u = 13.6$
$C_c = 2.04$

Particle Size Distribution



Comments

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Fuller Mossbarger Scott & May

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Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River — Project Number LX2005125

Source AC-6/193+51.7, 0.28' Rt., 55.9'-57.4', 60.9'-62.4' — Lab ID 36

Particle Shape Rounded — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A

Tested by DG — Test Date 08-22-2007 — Date Received 08-10-2007

Sample Dry Mass (g) 444.54 — Analysis based on: Total Sample

	Sieve Size	Grams Retained	% Retained	% Passing
	6"			
	3"			
	1 1/2"			
	1"	0	0.0	100.0
	3/4"	26.85	6.0	94.0
15	3/8"	16.36	3.7	90.3
—	No. 4	24.99	5.6	84.7
	No. 10	27.24	6.2	78.5
78	No. 40	160.44	36.1	42.4
—	No. 200	158.15	35.5	6.9
7	Pan	30.53	6.9	---

Poorly graded sand with silt and gravel (SP-SM)

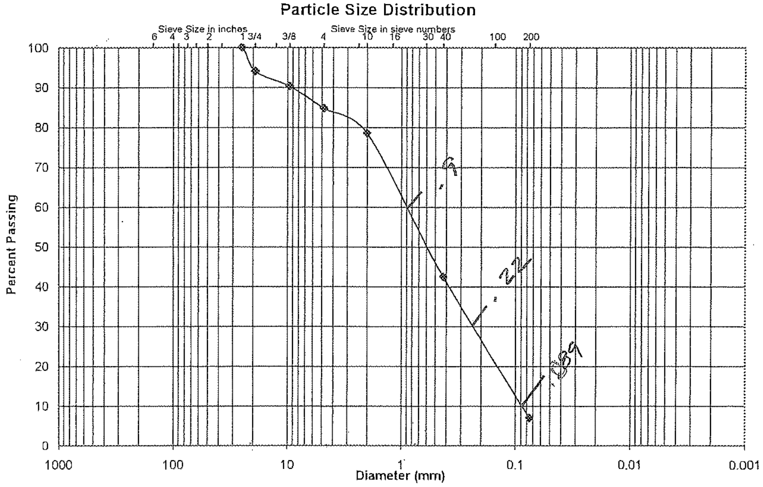
A-1-b

KG

Cu = 10.11

Cc = 0.60

Particle Size Distribution



Comments ____________

Reviewed By ____________

File: LX2005125_200-36 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

Fuller Mossbarger Scott & May

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Gradation Analysis
AASHTO T 88

Project Name I-265 Over Ohio River — Project Number LX2005125

Source AC-6/193+51.7, 0.28' Rt., 65.9'-67.4' — Lab ID 37

Particle Shape Angular — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A

Tested by DG — Test Date 08-22-2007 — Date Received 08-10-2007

Sample Dry Mass (g) 447.77 — Analysis based on: Total Sample

	Sieve Size	Grams Retained	% Retained	% Passing
	6"			
	3"			
	1 1/2"	0	0.0	100.0
	1"	34.04	7.6	92.4
	3/4"	52.88	11.8	80.6
33.5	3/8"	35.29	7.9	72.7
	No. 4	27.92	6.2	66.5
	No. 10	25.77	5.8	60.7
59.2	No. 40	70.43	15.7	45.0
	No. 200	168.59	37.7	7.3
	Pan	31.46	7.0	---

Poorly ~~Well~~ Graded Sand ~~Gravel~~ with silt & gravel ~~sand~~ ~~GW-GM~~

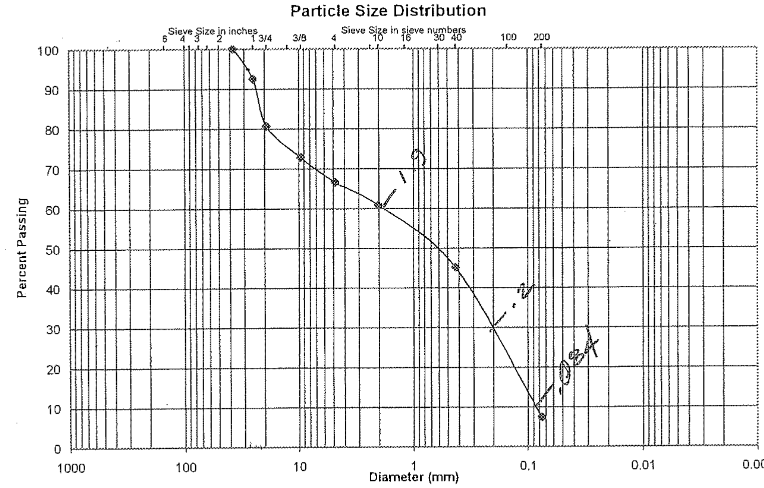
A-1-b SP-SM

Cu = ~~2.36~~ 22.6

Cc = ~~0.025~~ 0.25

KG

Particle Size Distribution



Comments ____________

Reviewed By ____________

File: LX2005125_200-37 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

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Gradation Analysis
AASHTO T 88

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-6/193+51.7, 0.28' Rt., 70.9'-72.4', 75.9'-77.4'	Lab ID	38

Particle Shape: Rounded — Particle Hardness: Hard and Durable — Prepared: AASHTO T 11 Method A

Tested by: DG — Test Date: 08-22-2007 — Date Received: 08-10-2007

Sample Dry Mass (g): 277.15 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	28.97	10.5	89.5
3/4"	63.07	22.7	66.8
3/8"	70.26	25.4	41.4
No. 4	35.65	12.8	28.6
No. 10	28.86	10.4	18.2
No. 40	29	10.5	7.7
No. 200	13.83	5.0	2.7
Pan	7.43	2.7	---

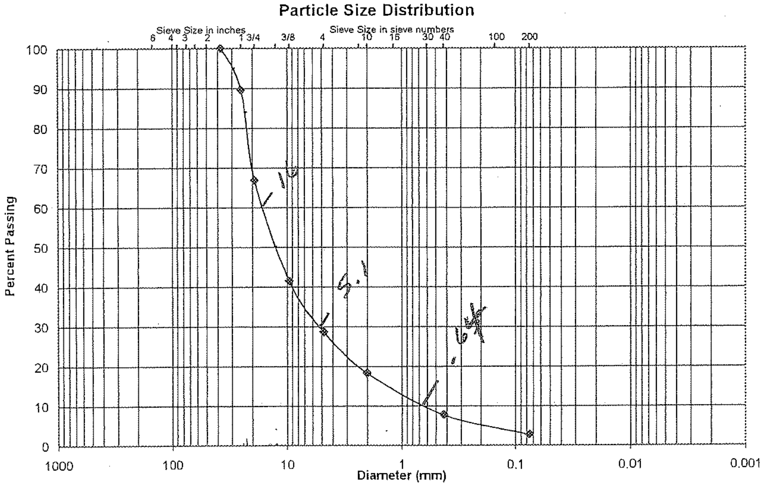
71.4
25.9
2.7

Well graded gravel with sand (GW)
A-1-a

Cu = 25
Cc = 2.54

KG

Particle Size Distribution



Comments ______

Reviewed By ______

File: LX2005125_200-38 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

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Gradation Analysis
AASHTO T 88

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Source	AC-6/193+51.7, 0.28' Rt., 80.9'-82.4', 85.9'-87.2'	Lab ID	39

Particle Shape: Rounded — Particle Hardness: Hard and Durable — Prepared: AASHTO T 11 Method A

Tested by: DG — Test Date: 08-22-2007 — Date Received: 08-10-2007

Sample Dry Mass (g): 472.79 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	8.23	1.7	98.3
3/8"	49.63	10.5	87.8
No. 4	35.99	7.7	80.1
No. 10	27.51	5.8	74.3
No. 40	143.29	30.3	44.0
No. 200	166.58	35.2	8.8
Pan	41.59	8.8	---

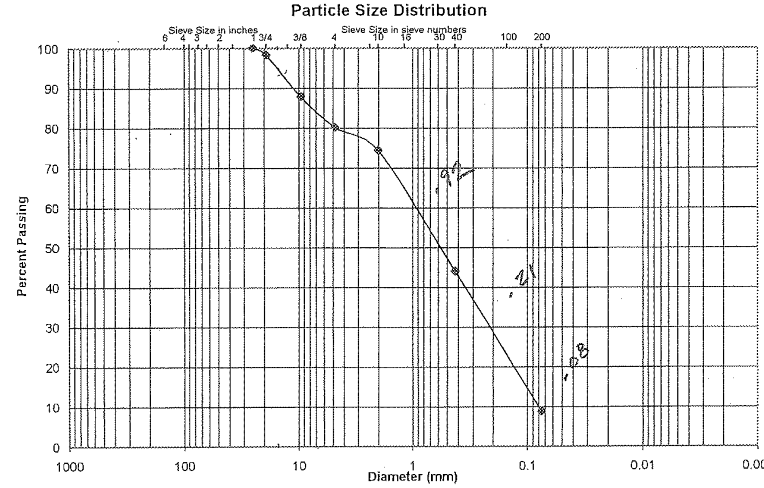
19.9
71.3
8.8

Poorly ~~Well~~-graded sand with silt and gravel (SP-SM)
A-1-b

Cu = 11.5
Cc = 0.60

KG

Particle Size Distribution



Comments ______

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File: LX2005125_200-39 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River — Project Number: LX2005125
Source: AC-7 193+95, 68' Lt., 70.5'-72.0' — Lab ID: 50
Particle Shape: Rounded — Particle Hardness: Hard and Durable — Prepared: AASHTO T 11 Method A
Tested by: RC — Test Date: 08-17-2007 — Date Received: 08-10-2007
Sample Dry Mass (g): 290.72 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	28.27	9.7	90.3
3/4"	64.44	22.2	68.1
3/8"	56.41	19.4	48.7
No. 4	37.9	13.0	35.7
No. 10	32.24	11.1	24.6
No. 40	44.19	15.2	9.4
No. 200	26.85	9.3	0.1
Pan	0.59	0.2	---

64.3

35.6

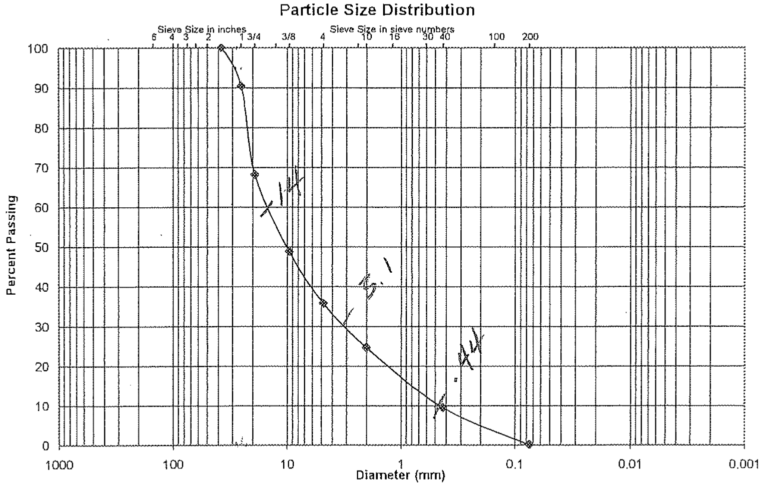
Well graded gravel with sand (GW)
A-1-a

Cu = 31.8

Cc = 1.56

KG

Particle Size Distribution



Comments ______

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File: LX2005125_200-50 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River — Project Number: LX2005125
Source: AC-7 193+95, 68' Lt., 88.0'-89.0' — Lab ID: 52
Particle Shape: Angular — Particle Hardness: Hard and Durable — Prepared: AASHTO T 11 Method A
Tested by: RC — Test Date: 08-17-2007 — Date Received: 08-10-2007
Sample Dry Mass (g): 349.15 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	34.66	9.9	90.1
3/8"	44.97	12.9	77.2
No. 4	28.21	8.1	69.1
No. 10	27.94	8.0	61.1
No. 40	106.89	30.6	30.5
No. 200	57.16	16.4	14.1
Pan	48.91	14.0	---

30.9

55.0

14.1

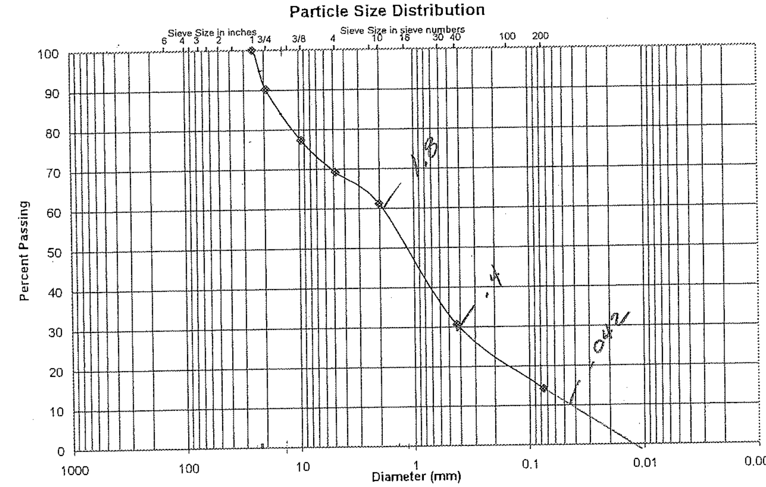
Silty sand with gravel (SM) A-1-b

KG

Cu = 42.86

Cc = ~~.212~~ ~~5.29~~ 2.1

Particle Size Distribution



Comments ______

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File: LX2005125_200-52 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

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Project Name I-265 Over Ohio River
Source AC-8/193+95, 1.22'Lt., 66.4'-67.2'
Project Number LX2005125
Lab ID 63

Particle Shape Angular — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A
Tested by RC — Test Date 08-17-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 255.86 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	71.81	28.1	71.9
No. 4	42.52	16.6	55.3
No. 10	22.98	9.0	46.3
No. 40	31.33	12.2	34.1
No. 200	67.23	26.3	7.8
Pan	19.18	7.5	---

44.7
47.5
7.8

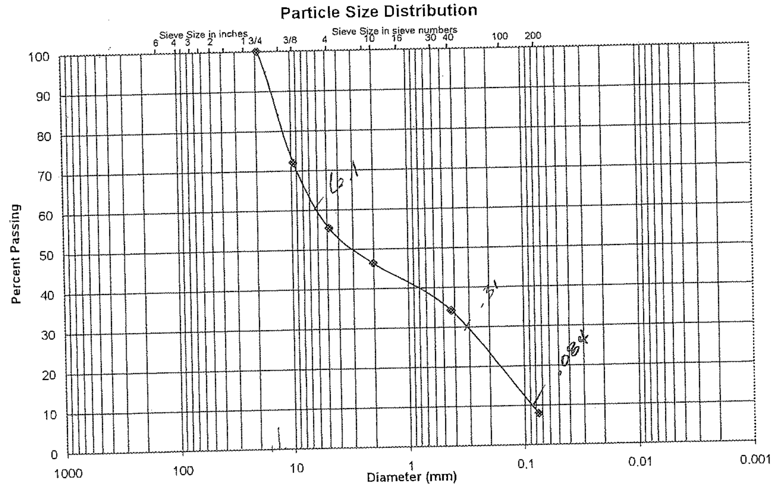
Poorly graded sand with silt and gravel (SP-SM)
A-1-b

KG

$C_u = 72.6$

$C_c = 0.19$

Particle Size Distribution



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File: LX2005125_200-63 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

FMSM ENGINEERS

Project Name I-265 Over Ohio River
Source AC-8/193+95, 1.22'Lt., 80.7'-82.2', 85.7'-86.5'
Project Number LX2005125
Lab ID 65

Particle Shape Need! Input — Particle Hardness Need! Input — Prepared AASHTO T 11 Method A
Tested by RC — Test Date 08-17-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 342.89 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	16.06	4.7	95.3
No. 4	10.52	3.1	92.2
No. 10	8.69	2.5	89.7
No. 40	154.83	45.1	44.6
No. 200	125.05	36.5	8.1
Pan	27.37	8.0	---

7.8
84.1
8.1

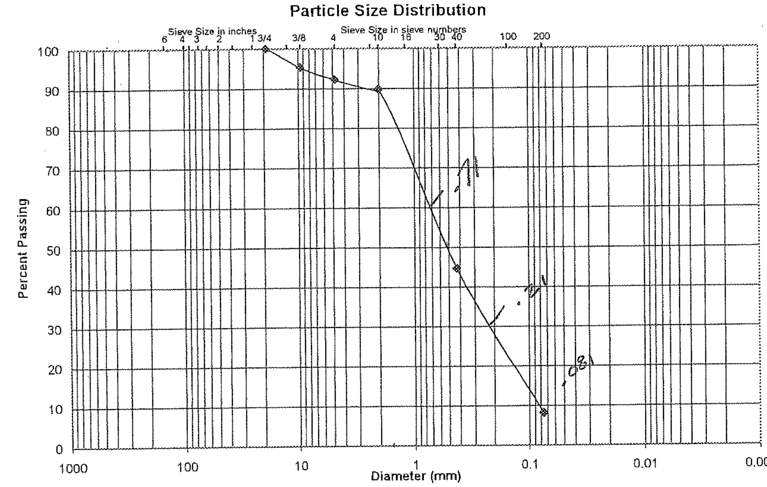
Poorly ~~Well~~ graded sand with silt (~~SW~~ SP-SM)
A-1-b

KG

$C_u = 8.77$

$C_c = 0.77$

Particle Size Distribution



Comments

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File: LX2005125_200-65 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Source: AC-9/193+95, 70' Rt., 55.6'-57.1', 60.6'-62.1'
Project Number: LX2005125
Lab ID: 75
Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RC
Test Date: 08-17-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 434.63
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	8.78	2.0	98.0
No. 4	15.96	3.7	94.3
No. 10	27.39	6.3	88.0
No. 40	237.27	54.6	33.4
No. 200	116.99	26.9	6.5
Pan	28.19	6.5	---

5.7

87.8

6.5

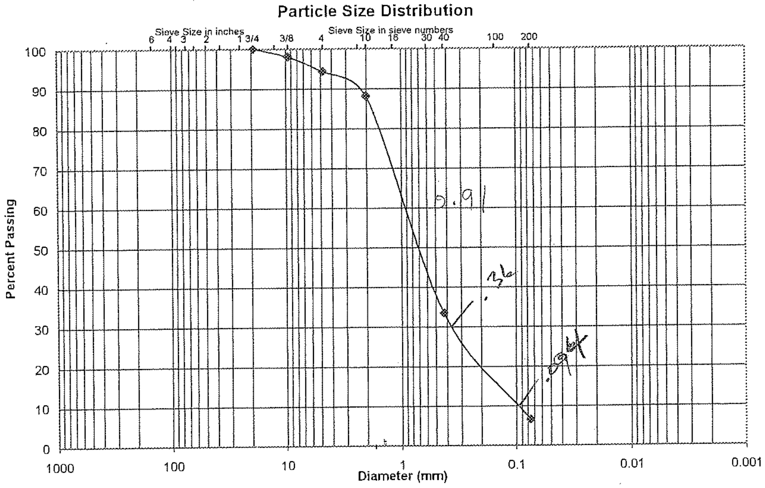
well graded sand with silt (SW-SM) A-1-b

KG

Cu 9.68

Cc 1.52

Particle Size Distribution



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File: LX2005125_200-75 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

Project Name: I-265 over the Ohio River
Source: AC-10/205+98, 70' Lt., 44.0'-45.5', 49.0'-50.5', 54.0'-55.5'
Project Number: LX2005125
Lab ID: 86
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RC
Test Date: 08-20-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 135.74
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	32.55	24.0	76.0
3/4"	0	0.0	76.0
3/8"	28.75	21.2	54.8
No. 4	44.01	32.4	22.4
No. 10	23.85	17.6	4.8
No. 40	5.98	4.4	0.4
No. 200	0.59	0.4	0.0
Pan	0.05	0.0	---

77.6

22.4

-

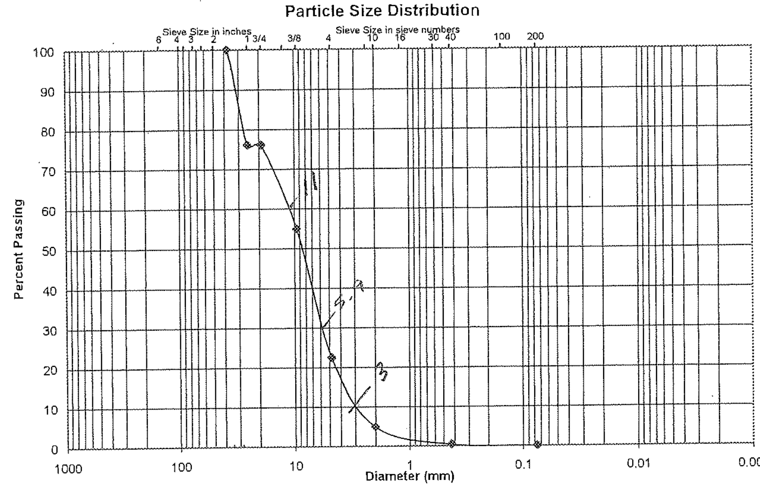
Poorly graded gravel with sand (GP) A-1-a

KG

Cu = 3.7

Cc = ~~10.5~~ 1.05

Particle Size Distribution



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Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

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Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-10/205+98, 70' Lt., 59.0'-60.5', 64.0'-65.5'
Lab ID: 87
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RC
Test Date: 08-07-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 498.35
Analysis based on: Total Sample

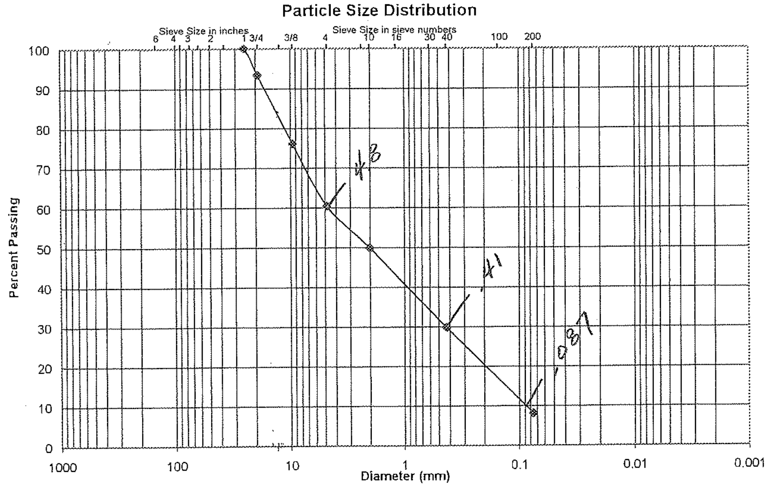
Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	33.27	6.7	93.3
3/8"	86.18	17.3	76.0
No. 4	78.54	15.7	60.3
No. 10	52.11	10.5	49.8
No. 40	100.2	20.1	29.7
No. 200	107.99	21.7	8.0
Pan	39.84	8.0	---

39.7
52.3
8.0

Poorly ~~Well~~ graded sand with silt and gravel (~~SW-SM~~) (SP-SM)
A-1-a
KG

Cu = 55.2
Cc = .40

Particle Size Distribution



Comments

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Gradation Analysis
AASHTO T 88

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Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-10/205+98, 70' Lt., 69.0'-70.5', 74.0'-75.5'
Lab ID: 88
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RC
Test Date: 08-20-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 468.02
Analysis based on: Total Sample

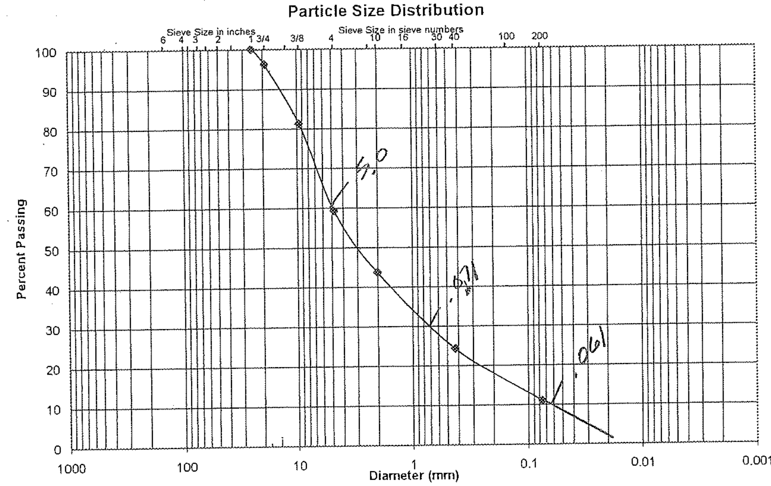
Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	17.97	3.8	96.2
3/8"	70.19	15.0	81.2
No. 4	102.7	22.0	59.2
No. 10	73.01	15.6	43.6
No. 40	91.01	19.4	24.2
No. 200	62.13	13.3	10.9
Pan	50.86	10.9	---

40.8
48.3
10.9

Well graded sand with silt and gravel (~~SW-SM~~) (SW-SM)
A-1-a
KG

Cu = 82.0
Cc = ~~0.02~~ ~~0.006~~ 1.65

Particle Size Distribution



Comments

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Gradation Analysis
AASHTO T 88

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Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-11/205+95,1.3 Lt., 56.7'-57.1' — Lab ID 99
Particle Shape Rounded — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A
Tested by rc — Test Date 08-20-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 196.25 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	55.27	28.2	71.8
3/8"	45.75	23.3	48.5
No. 4	39.31	20.0	28.5
No. 10	31.75	16.2	12.3
No. 40	17.07	8.7	3.6
No. 200	3.71	1.9	1.7
Pan	2.68	1.3	---

71.5

26.8

1.7

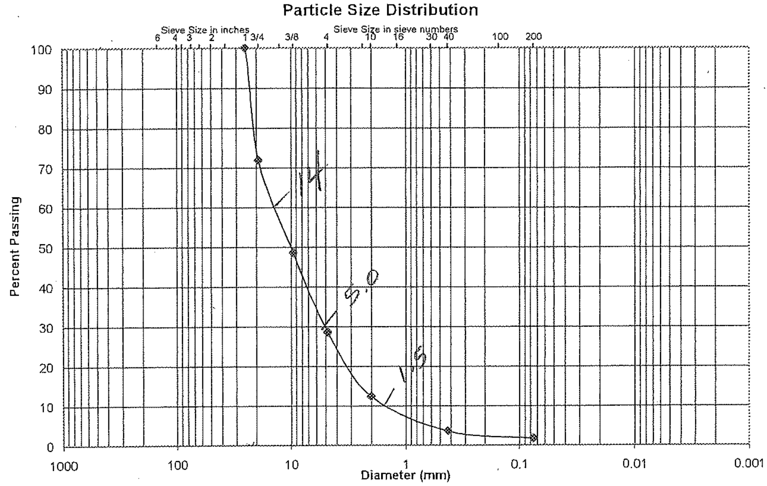
Wellgraded gravel with sand (GW)

A-1-a

$C_u = 9.33$ KG

$C_c = 1.19$

Particle Size Distribution



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File: LX2005125_200-99 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Approved By: TLK

Gradation Analysis
AASHTO T 88

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Project Name I-265 Over Ohio River — Project Number LX2005125
Source AC-11/205+95,1.3 Lt., 81.7'-82.0' — Lab ID 102
Particle Shape Rounded — Particle Hardness Hard and Durable — Prepared AASHTO T 11 Method A
Tested by RC — Test Date 08-20-2007 — Date Received 08-10-2007
Sample Dry Mass (g) 207.12 — Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	33.68	16.3	83.7
3/8"	26.49	12.8	70.9
No. 4	20.57	9.9	61.0
No. 10	22.97	11.1	49.9
No. 40	27.36	13.2	36.7
No. 200	13.19	6.4	30.3
Pan	63.06	30.4	---

39.0

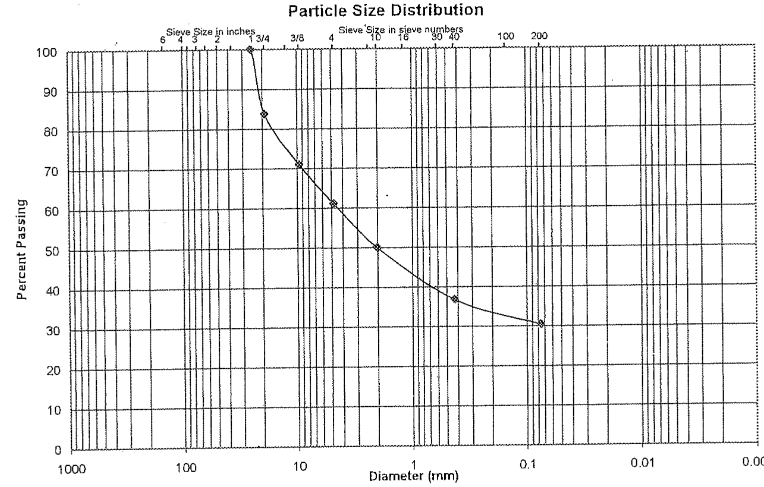
30.7

30.3

Silty gravel w/ sand (GM)

A-2-4

Particle Size Distribution



Comments

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File: LX2005125_200-102 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Gradation Analysis
AASHTO T 88

Project Name: I-265 Bridge over the Ohio River
Project Number: LX2005125
Source: AC-12/205+94, 71 Rt., 40.0'-42.0', 45.5'-47.0', 50.5'-52.0'
Lab ID: 111
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: CM
Test Date: 08-21-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 536.8
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	45.62	8.5	91.5
3/4"	121.8	22.7	68.8
3/8"	116.57	21.7	47.1
No. 4	121.89	22.7	24.4
No. 10	85.37	15.9	8.5
No. 40	35.03	6.5	2.0
No. 200	7.52	1.4	0.6
Pan	2.31	0.5	---

75.6

23.8

0.6

Well graded gravel with sand (GW)

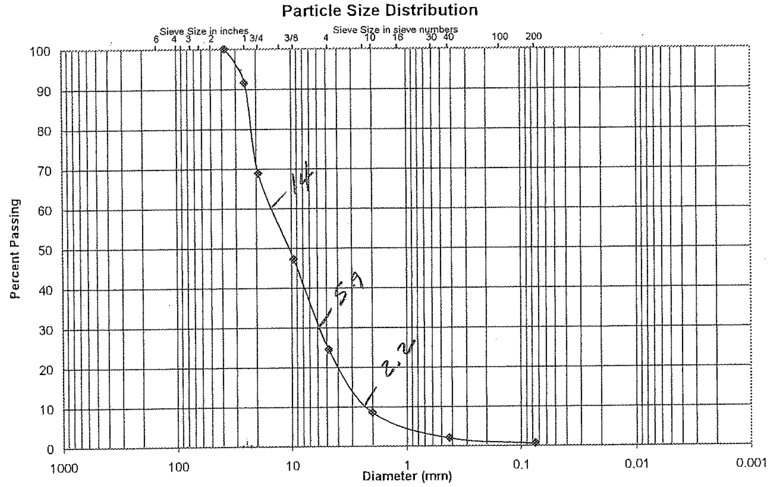
A-1-a

KG

$C_u = 6.4$

$C_c = 1.13$

Particle Size Distribution



Comments

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File: LX2005125_200-111 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Approved By: TLK

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Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-12/205+94, 71 Rt., 75.5'-77.0', 80.5'-81.0'
Lab ID: 114
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RC
Test Date: 08-20-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 401.95
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	80.09	19.9	80.1
3/4"	57.98	14.5	65.6
3/8"	38.12	9.4	56.2
No. 4	21.4	5.4	50.8
No. 10	24.49	6.1	44.7
No. 40	60.75	15.1	29.6
No. 200	75.93	18.9	10.7
Pan	43.12	10.7	---

49.2

40.1

10.8

Poorly ~~Well~~ graded gravel with silt and sand (~~GW~~-GM)

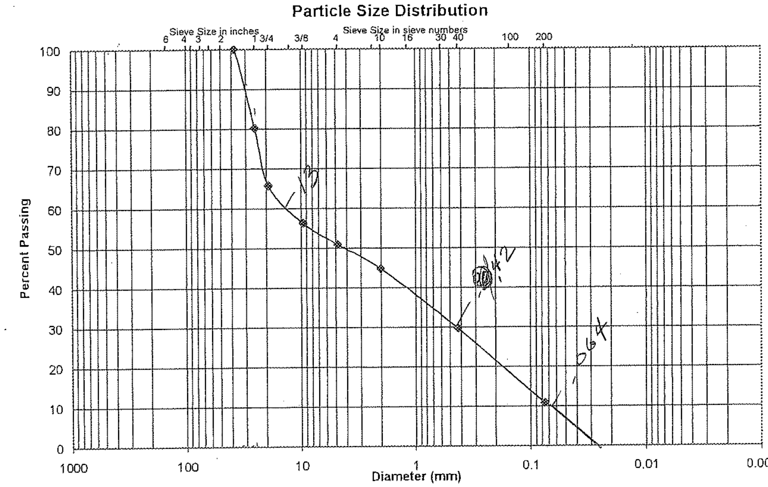
A-1-b GP-GM

KG

$C_u = 203$

$C_c = \cancel{.002}$ $\cancel{0.50}$ .21

Particle Size Distribution



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File: LX2005125_200-114 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Prepared By: MW
Approved By: TLK

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Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-13/206+50, 0.02 Lt., 42.0'-43.5', 47.0'-48.5'
Lab ID: 123
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RM
Test Date: 08-28-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 603.87
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	30.59	5.1	94.9
3/4"	37.51	6.2	88.7
3/8"	212.53	35.2	53.5
No. 4	151.07	25.0	28.5
No. 10	101.46	16.8	11.7
No. 40	51.55	8.5	3.2
No. 200	9.28	1.6	1.6
Pan	10.38	1.7	---

71.5

26.9

1.6

Well graded gravel with sand (GW)

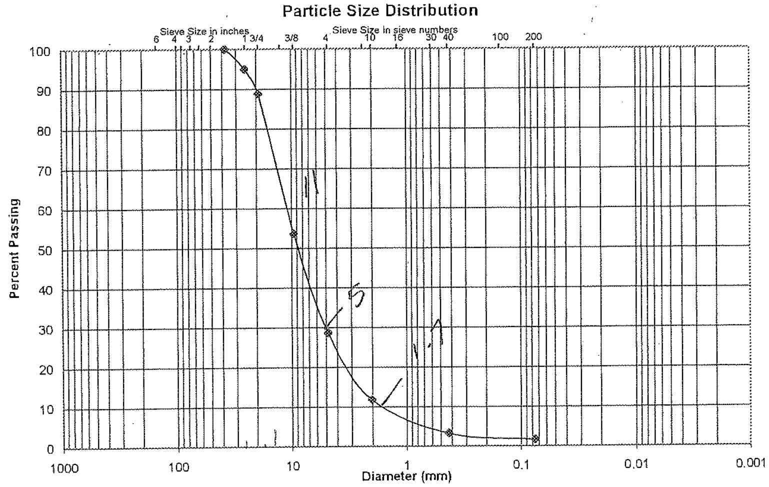
A-1-a

KG

$C_u = 6.5$

$C_c = 1.34$

Particle Size Distribution



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File: LX2005125_200-123 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

FMSM ENGINEERS

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-13/206+50, 0.02 Lt., 52.0'-53.5', 57.0'-58.5', 62.0'-63.5'
Lab ID: 124
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RC
Test Date: 08-20-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 590.14
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	61.98	10.5	89.5
3/4"	41.41	7.0	82.5
3/8"	71.59	12.2	70.3
No. 4	45.12	7.6	62.7
No. 10	53.45	9.1	53.6
No. 40	153.7	26.0	27.6
No. 200	122	20.7	6.9
Pan	41.2	7.0	---

37.3

55.8

6.9

Poorly ~~Well~~ graded sand with silt and gravel (~~SW-SM~~) (SP-SM)

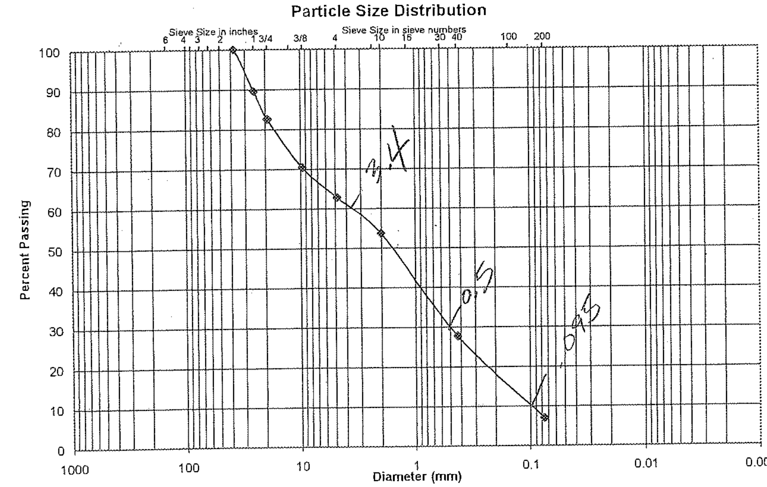
A-1-b

KG

$C_u = 35.8$

$C_c = 0.8$

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-124 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-15/210+20, 37.3 Rt., 7.2'-8.2'
Lab ID: 136
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: RC
Test Date: 08-20-2007
Date Received: 08-10-2007
Sample Dry Mass (g): 284.83
Analysis based on: Total Sample

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	20.81	7.3	92.7
3/4"	9.33	3.3	89.4
3/8"	37.32	13.1	76.3
No. 4	27.75	9.7	66.6
No. 10	17.42	6.1	60.5
No. 40	18.23	6.4	54.1
No. 200	56.43	19.9	34.2
Pan	97.7	34.3	---

33.4

32.4

34.2

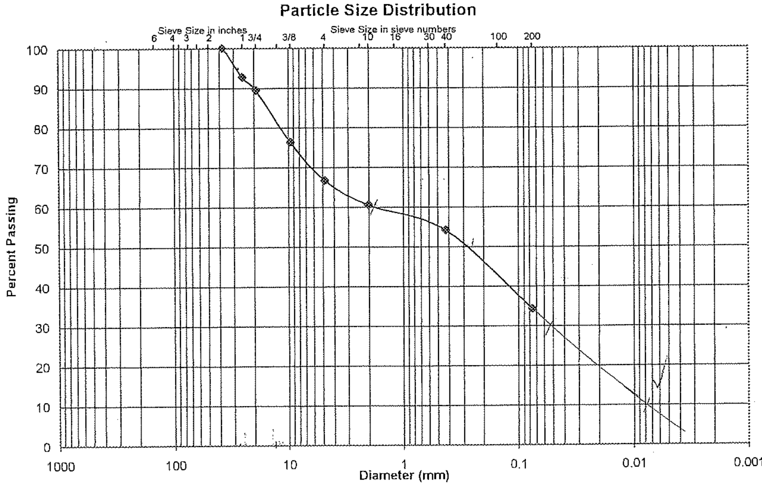
silty gravel with sand (GM) A-2-4

KG

Cu 233.8

Cc 0.19

Particle Size Distribution



1.8

0.052

0.0077

Comments

Reviewed By

File: LX2005125_200-136 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

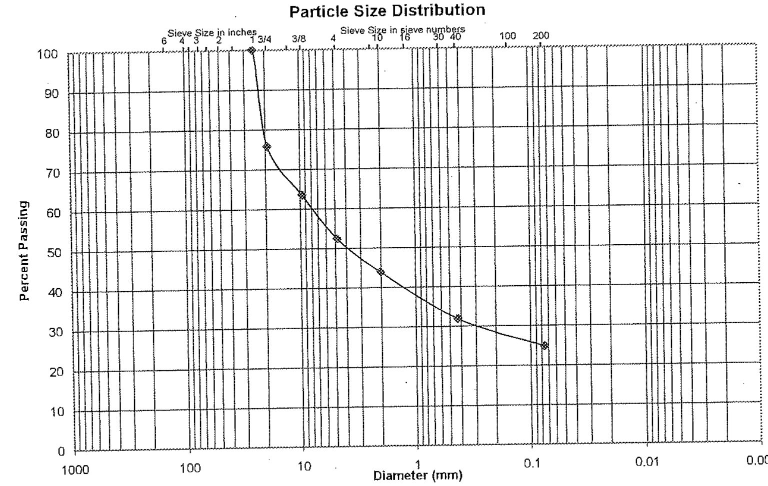
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-27/ 212+87, 125.0 Rt., 0.0'-1.2'
Lab ID: 249
Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: DG
Test Date: 10-23-2007
Date Received: 10-23-2007
Sample Dry Mass (g): 281.67
Analysis based on: Total Sample
Moisture Content (%): 6.5

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	68.31	24.3	75.7
3/8"	34.7	12.3	63.4
No. 4	31.32	11.1	52.3
No. 10	23.89	8.5	43.8
No. 40	33.83	12.0	31.8
No. 200	19.99	7.1	24.7
Pan	70.19	24.9	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-249 Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

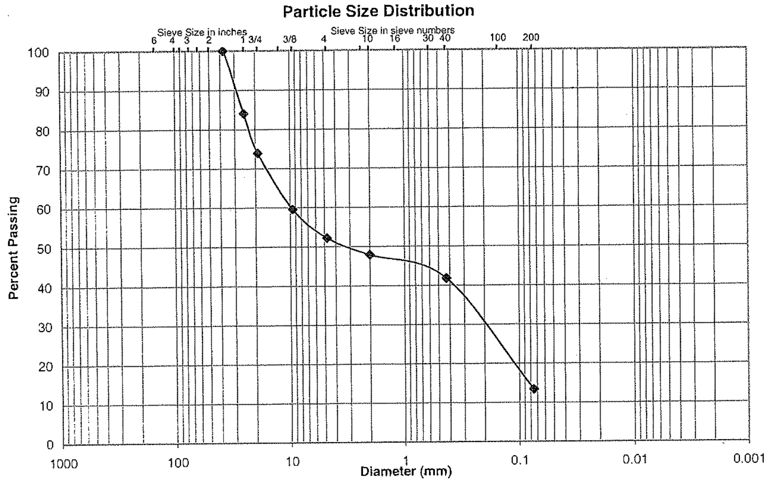
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-1 / 189+60,CL, 5.0'-6.5', 10.0'-11.5', 15.0'-16.5'
Lab ID: 2
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH/DC
Test Date: 11-30-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 542.06
Analysis based on: Total Sample
Moisture Content (%): 16.7

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	86.46	16.0	84.0
3/4"	54.79	10.1	73.9
3/8"	77.87	14.3	59.6
No. 4	39.96	7.4	52.2
No. 10	23.69	4.4	47.8
No. 40	32.66	6.0	41.8
No. 200	153.97	28.4	13.4
Pan	73	13.5	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-2.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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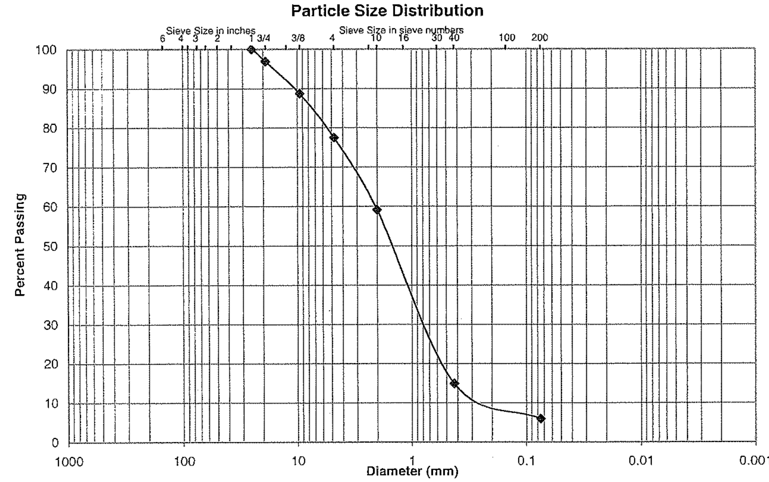
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-1 / 189+60,CL, 20.0'-21.5', 25.0'-26.5', 30.0'-31.5', 35.0'-36.5', 40.0'-41.5'
Lab ID: 3
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH
Test Date: 11-30-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 772.96
Analysis based on: Total Sample
Moisture Content (%): 13.6

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	24.02	3.1	96.9
3/8"	63.3	8.2	88.7
No. 4	86.77	11.2	77.5
No. 10	141.64	18.3	59.2
No. 40	342.36	44.3	14.9
No. 200	69.14	9.0	5.9
Pan	39.64	5.1	---

Particle Size Distribution



Comments

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File: LX2005125_200-3.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

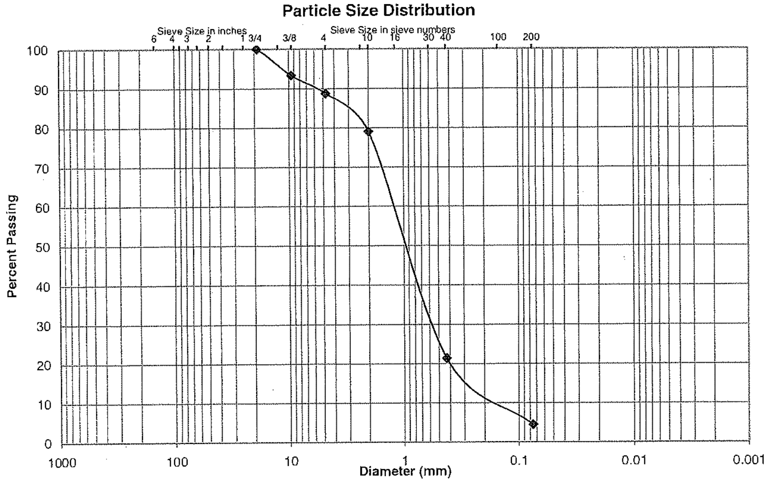
Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-1 / 189+60,CL, 45.0'-46.5', 50.0'-51.5', 55.0'-56.5', 60.0'-61.5'
Lab ID: 4
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH
Test Date: 11-30-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 603.94
Analysis based on: Total Sample
Moisture Content (%): 16.3

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"			
3/4"	0	0.0	100.0
3/8"	40.03	6.6	93.4
No. 4	27.91	4.6	88.8
No. 10	58.49	9.7	79.1
No. 40	348.16	57.7	21.4
No. 200	102.47	16.9	4.5
Pan	26.4	4.4	---



Comments

Reviewed By

File: LX2005125_200-4.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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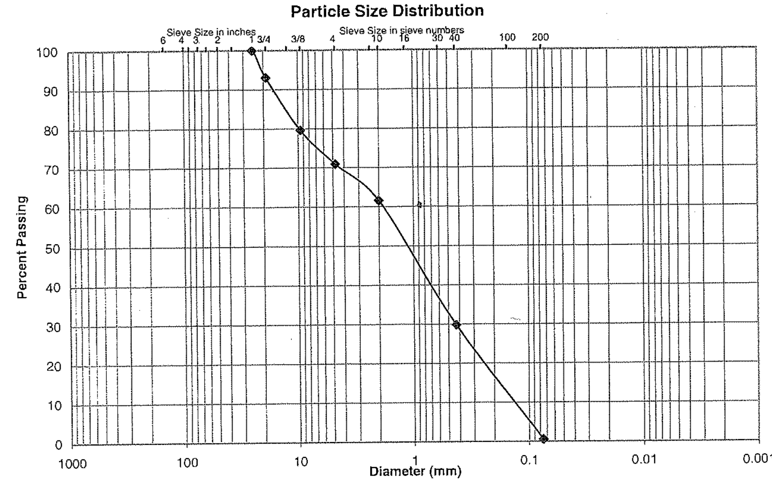
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-1 / 189+60,CL, 65.0'-66.5', 70.0'-71.5', 75.0'-76.5', 80.0'-81.3'
Lab ID: 5
Particle Shape: Angular
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH/DC
Test Date: 12-05-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 529.02
Analysis based on: Total Sample
Moisture Content (%): 7.5

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"			
1"	0	0.0	100.0
3/4"	36.77	7.0	93.0
3/8"	71.03	13.4	79.6
No. 4	46.2	8.7	70.9
No. 10	49.35	9.3	61.6
No. 40	168.04	31.8	29.8
No. 200	154.62	29.2	0.6
Pan	3.6	0.7	---



Comments

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File: LX2005125_200-5.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

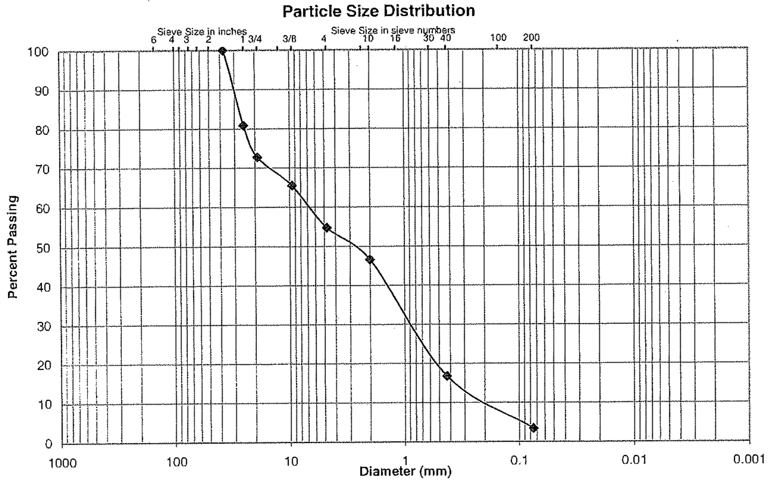
Gradation Analysis
AASHTO T 88

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-2 / 194+50, CL, 42.0'-43.5', 47.0'-48.5'
Lab ID: 8
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: DC/JWH
Test Date: 12-02-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 585.77
Analysis based on: Total Sample
Moisture Content (%): 10.8

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	112.12	19.1	80.9
3/4"	47.7	8.2	72.7
3/8"	42.3	7.2	65.5
No. 4	63.06	10.8	54.7
No. 10	47.82	8.1	46.6
No. 40	174.8	29.9	16.7
No. 200	78.09	13.3	3.4
Pan	17.39	3.0	---

Particle Size Distribution



Comments ____

Reviewed By ____

File: LX2005125_200-8.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

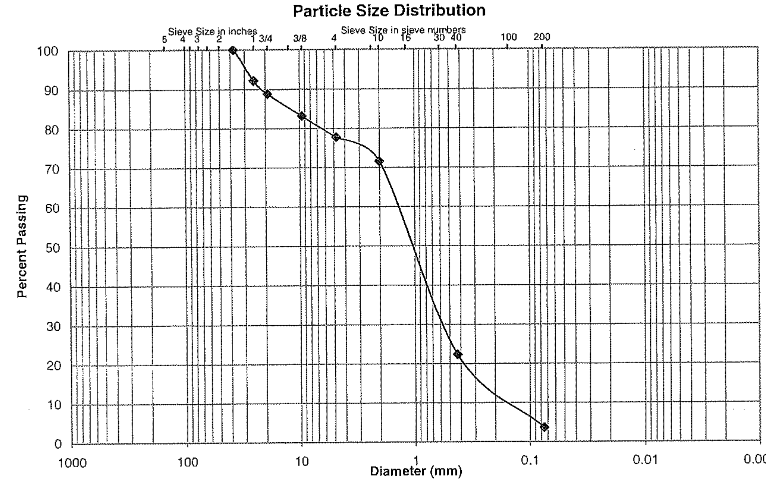
Gradation Analysis
AASHTO T 88

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-2 / 194+50, CL, 52.0'-53.5', 57.0'-58.5', 62.0'-63.5'
Lab ID: 9
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH
Test Date: 11-30-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 558.64
Analysis based on: Total Sample
Moisture Content (%): 15.9

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	43.45	7.8	92.2
3/4"	19.01	3.4	88.8
3/8"	31.58	5.6	83.2
No. 4	30.05	5.4	77.8
No. 10	34.51	6.2	71.6
No. 40	275.41	49.3	22.3
No. 200	104.15	18.6	3.7
Pan	20.77	3.8	---

Particle Size Distribution



Comments ____

Reviewed By ____

File: LX2005125_200-9.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

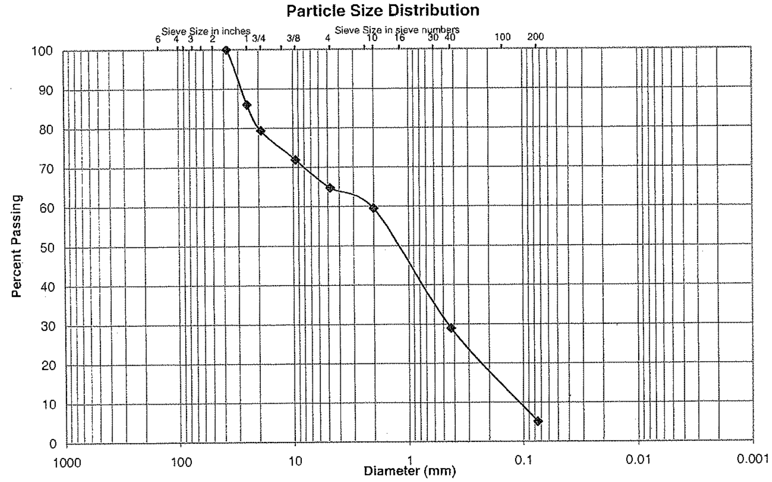
Gradation Analysis
AASHTO T 88

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-2 / 194+50, CL, 67.0'-68.5', 72.0'-73.5', 78.0'-79.5', 82.0'-83.5', 87.0'-88.0'
Lab ID: 10
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH
Test Date: 11-30-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 593.34
Analysis based on: Total Sample
Moisture Content (%): 13.1

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	83.17	14.0	86.0
3/4"	39.5	6.7	79.3
3/8"	44.09	7.4	71.9
No. 4	42.66	7.2	64.7
No. 10	30.92	5.2	59.5
No. 40	181.03	30.5	29.0
No. 200	141.83	23.9	5.1
Pan	30.19	5.1	---

Particle Size Distribution



Comments

Reviewed By

File: LX2005125_200-10.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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Laboratory Document
Prepared By: MW
Approved By: TLK

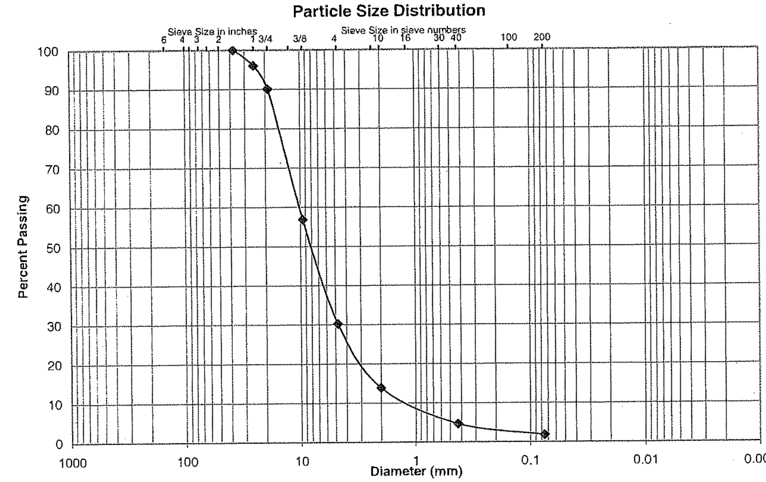
Gradation Analysis
AASHTO T 88

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-3 / 205+50, CL, 41.0'-42.5', 46.0'-47.5', 51.0'-52.5'
Lab ID: 13
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH
Test Date: 12-02-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 614.78
Analysis based on: Total Sample
Moisture Content (%): 8.9

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	24.2	3.9	96.1
3/4"	37.02	6.1	90.0
3/8"	204.09	33.2	56.8
No. 4	163.72	26.6	30.2
No. 10	100.25	16.3	13.9
No. 40	56.96	9.3	4.6
No. 200	17.39	2.8	1.8
Pan	11.47	1.9	---

Particle Size Distribution



Comments

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File: LX2005125_200-13.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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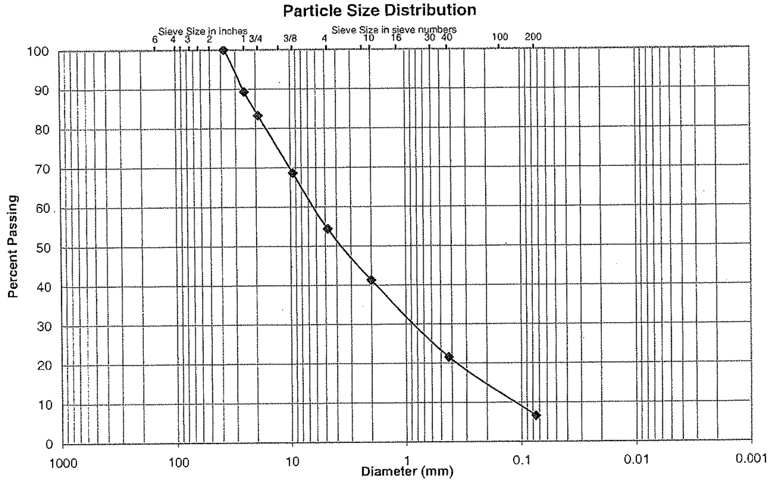
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-3 / 205+50, CL, 56.0'-57.5', 61.0'-62.5', 66.0'-67.5'
Lab ID: 14
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH/DC
Test Date: 12-02-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 523.55
Analysis based on: Total Sample
Moisture Content (%): 10.5

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	56.01	10.7	89.3
3/4"	31.49	6.0	83.3
3/8"	76.77	14.7	68.6
No. 4	74.65	14.2	54.4
No. 10	68.34	13.1	41.3
No. 40	103.38	19.7	21.6
No. 200	78.39	15.0	6.6
Pan	34.55	6.6	---

Particle Size Distribution



Comments ______

Reviewed By ______

File: LX2005125_200-14.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

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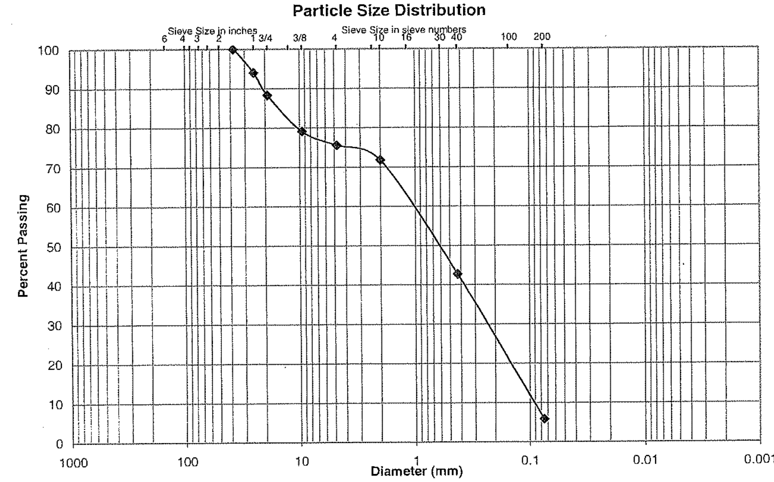
Laboratory Document
Prepared By: MW
Approved By: TLK

Gradation Analysis
AASHTO T 88

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: B-3 / 205+50, CL, 71.0'-72.3', 76.0'-77.5'
Lab ID: 15
Particle Shape: Rounded
Particle Hardness: Hard and Durable
Prepared: AASHTO T 11 Method A
Tested by: JWH
Test Date: 11-30-2005
Date Received: 11-07-2005
Sample Dry Mass (g): 582.63
Analysis based on: Total Sample
Moisture Content (%): 13.9

Sieve Size	Grams Retained	% Retained	% Passing
6"			
3"			
1 1/2"	0	0.0	100.0
1"	34.76	6.0	94.0
3/4"	33.37	5.7	88.3
3/8"	53.56	9.2	79.1
No. 4	20.94	3.6	75.5
No. 10	21.57	3.7	71.8
No. 40	169.72	29.1	42.7
No. 200	214.88	36.9	5.8
Pan	31.45	5.4	---

Particle Size Distribution



Comments ______

Reviewed By ______

File: LX2005125_200-15.xls Sheet: Sieve-Report
Preparation Date: 5-98
Revision Date: 01-2001

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: MW
Approved By: TLK

ENGINEERS

Particle-Size Analysis of Soils
AASHTO T 88

Project Name: I-265 Over Ohio River
Source: B-1 / 189+60,CL, 0.0'-1.5'

Project Number: LX2005125
Lab ID: 1

Sieve analysis for the Portion Coarser than the No. 10 Sieve

Test Method: AASHTO T 88
Prepared using: AASHTO T 87

Particle Shape: Angular
Particle Hardness: Hard and Durable

Tested By: AKS
Test Date: 12-01-2005
Date Received: 11-07-2005

Maximum Particle size: 1" Sieve

Sieve Size	% Passing
3"	
2"	
1 1/2"	
1"	100.0
3/4"	84.0
3/8"	56.2
No. 4	45.1
No. 10	35.4

Analysis for the portion Finer than the No. 10 Sieve

Analysis Based on: Total Sample

Specific Gravity: 2.72

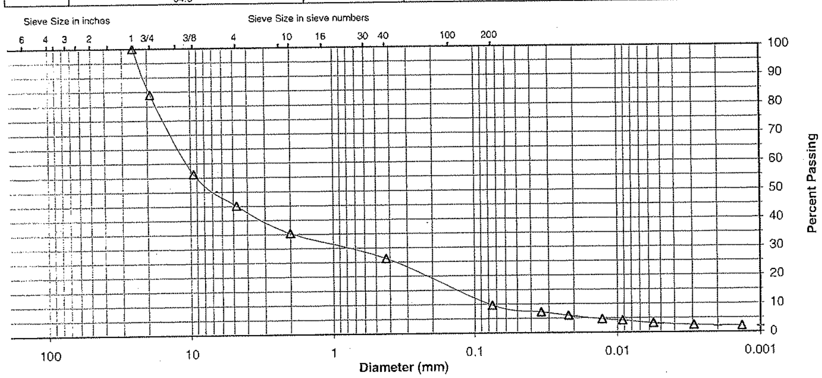
Dispersed using: Apparatus A - Mechanical, for 1 minute

No. 40	26.3
No. 200	9.6
0.02 mm	5.6
0.005 mm	3.0
0.002 mm	2.1
0.001 mm	2.0

Particle Size Distribution

ASTM	Coarse Gravel	Fine Gravel	C. Sand	Medium Sand	Fine Sand	Silt	Clay
	16.0	38.9	9.7	9.1	16.7	6.6	3.0

AASHTO	Gravel	Coarse Sand	Fine Sand	Silt	Clay
	64.6	9.1	16.7	7.5	2.1



Comments ______

Reviewed By ______

File: LX2005125_Sum-1.xls Sheet: Hydro-Report
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

ENGINEERS

Particle-Size Analysis of Soils
AASHTO T 88

Project Name: I-265 Over Ohio River
Source: B-4 / 210+30, CL, 1.0'-2.5', 5.5'-7.0', 10.5'-12.0'

Project Number: LX2005125
Lab ID: 18

Sieve analysis for the Portion Coarser than the No. 10 Sieve

Test Method: AASHTO T 88
Prepared using: AASHTO T 87

Particle Shape: Angular
Particle Hardness: Hard and Durable

Tested By: AKS
Test Date: 12-01-2005
Date Received: 11-07-2005

Maximum Particle size: 1 1/2" Sieve

Sieve Size	% Passing
3"	
2"	
1 1/2"	100.0
1"	86.1
3/4"	71.6
3/8"	54.6
No. 4	39.4
No. 10	27.5

Analysis for the portion Finer than the No. 10 Sieve

Analysis Based on: Total Sample

Specific Gravity: 2.71

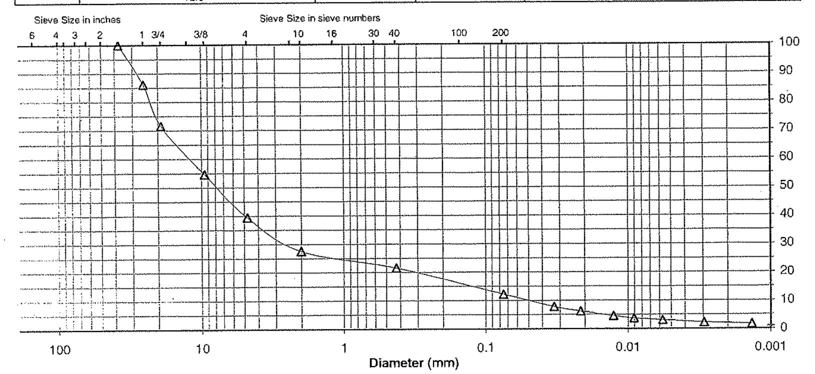
Dispersed using: Apparatus A - Mechanical, for 1 minute

No. 40	21.5
No. 200	12.2
0.02 mm	6.0
0.005 mm	2.9
0.002 mm	2.0
0.001 mm	1.0

Particle Size Distribution

ASTM	Coarse Gravel	Fine Gravel	C. Sand	Medium Sand	Fine Sand	Silt	Clay
	28.4	32.2	11.9	6.0	9.3	9.3	2.9

AASHTO	Gravel	Coarse Sand	Fine Sand	Silt	Clay
	72.5	6.0	9.3	10.2	2.0



Comments ______

Reviewed By ______

File: LX2005125_Sum-18.xls Sheet: Hydro-Report
Preparation Date: 1998
Revision Date: 05-2003

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved BY: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

FMSM ENGINEERS

Project Name I-265 Bridge over the Ohio River
Project Number LX2005125
Source AC-1, 2.5'-4.5'
Lab ID 146A
Visual Description lean Clay (CL), brown, moist, firm
Recovered 1.4'
Test Interval 2.5' - 3.0'

Specimen Type: Undisturbed

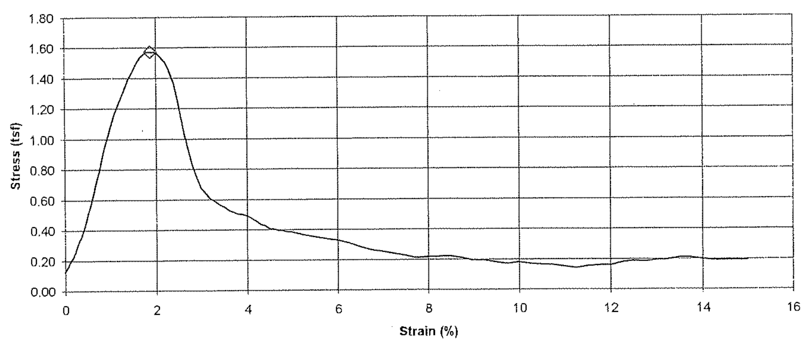
Initial Wet Density (pcf)	106.9	LL	48	Date Extruded	10/15/2007
Initial Dry Density (pcf)	89.8	PL	19	Date Tested	11/16/2007
Initial Moisture Content (%)	19.1	PI	29		
Specific Gravity	2.72				
Degree of Saturation (%)	58.4				
Average Height (in)	6.052				
Average Diameter (in)	2.872				
Height to Diameter Ratio	2.1				

MC Taken Before Test, From Trimmings

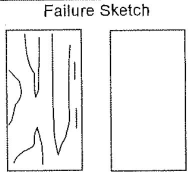
Unconfined Compressive Strength (tsf) 1.57
Undrained Shear Strength (tsf) 0.79

Strain at Maximum Stress (%) 1.9
Strain rate to failure (% / min.) 1.00

Stress vs. Strain



Failure Sketch



Pocket Penetrometer Reading (tsf) 4.0
Torvane Reading (kg/cm^2) N/A

Comments

Reviewed By

LX2005125_UC-146A.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

FMSM ENGINEERS

Project Name I-265 Bridge over the Ohio River
Project Number LX2005125
Source AC-1, 10.0'-12.0'
Lab ID 148A
Visual Description lean Clay with sand (CL), brown, moist, firm
Recovered 1.2'
Test Interval 10.0' - 10.5'

Specimen Type: Undisturbed

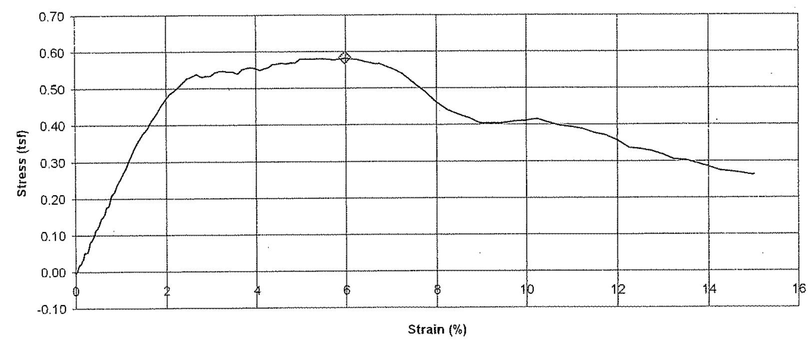
Initial Wet Density (pcf)	125.2	LL	35	Date Extruded	10/16/2007
Initial Dry Density (pcf)	101.5	PL	15	Date Tested	10/16/2007
Initial Moisture Content (%)	23.4	PI	20		
Specific Gravity	2.68				
Degree of Saturation (%)	96.7				
Average Height (in)	6.030				
Average Diameter (in)	2.879				
Height to Diameter Ratio	2.1				

MC Taken Before Test, From Trimmings

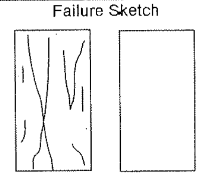
Unconfined Compressive Strength (tsf) 0.58
Undrained Shear Strength (tsf) 0.29

Strain at Maximum Stress (%) 6.0
Strain rate to failure (% / min.) 1.00

Stress vs. Strain



Failure Sketch



Pocket Penetrometer Reading (tsf) 1.0
Torvane Reading (kg/cm^2) N/A

Comments

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LX2005125_UC-148A.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name I-265 Bridge over the Ohio River — Project Number LX2005125
Source AC-1, 10.0'-12.0' — Lab ID 148B
Visual Description lean Clay with sand (CL), brown, moist, firm

Recovered 1.2'
Test Interval 10.5' - 11.0'

Specimen Type: Undisturbed

LL	N/A
PL	N/A
PI	N/A

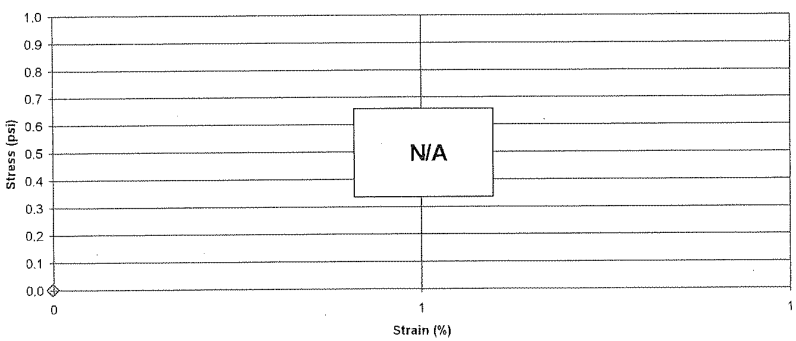
Date Extruded 10/16/2007
Date Tested N/A

Initial Wet Density (pcf)	124.8
Initial Dry Density (pcf)	105.7
Initial Moisture Content (%)	18.0
Specific Gravity	N/A
Degree of Saturation (%)	N/A
Average Height (in)	6.048
Average Diameter (in)	2.879
Height to Diameter Ratio	2.1

MC Taken Before Test, From Trimmings

Unconfined Compressive Strength (tsf)	N/A
Undrained Shear Strength (tsf)	N/A
Strain at Maximum Stress (%)	N/A
Strain rate to failure (% / min.)	N/A

Stress vs. Strain



Failure Sketch

Pocket Penetrometer Reading (tsf) 1.0
Torvane Reading (kg/cm^2) N/A

Comments

Reviewed By

LX2005125_UC-148B.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name I-265 Bridge over the Ohio River — Project Number LX2005125
Source AC-2, 5.0'-7.0' — Lab ID 159A
Visual Description lean Clay (CL), brown, moist, firm

Recovered 1.2'
Test Interval 5.0' - 5.5'

Specimen Type: Undisturbed

LL	48
PL	20
PI	28

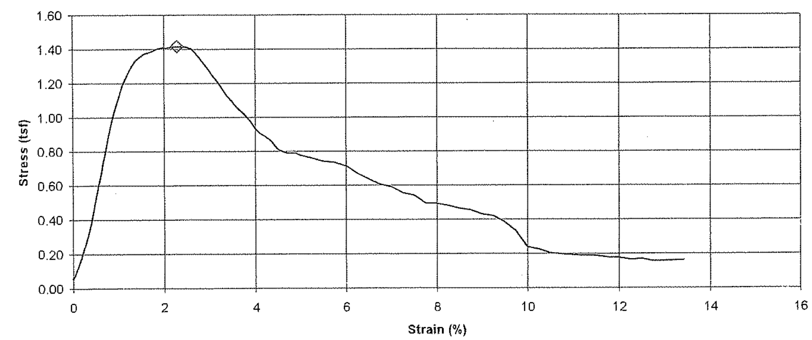
Date Extruded 10/16/2007
Date Tested 10/16/2007

Initial Wet Density (pcf)	112.4
Initial Dry Density (pcf)	94.8
Initial Moisture Content (%)	18.6
Specific Gravity	2.69
Degree of Saturation (%)	64.9
Average Height (in)	6.104
Average Diameter (in)	2.891
Height to Diameter Ratio	2.1

MC Taken Before Test, From Trimmings

Unconfined Compressive Strength (tsf)	1.41
Undrained Shear Strength (tsf)	0.71
Strain at Maximum Stress (%)	2.3
Strain rate to failure (% / min.)	1.00

Stress vs. Strain



Failure Sketch

Pocket Penetrometer Reading (tsf) 5.0
Torvane Reading (kg/cm^2) N/A

Comments

Reviewed By

LX2005125_UC-159A.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

FMSM ENGINEERS

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

Project Name: I-265 Bridge over the Ohio River
Project Number: LX2005125
Source: AC-2, 5.0'-7.0'
Lab ID: 159B
Visual Description: lean Clay (CL), brown, moist, firm

Recovered: 1.2'
Test Interval: 5.5' - 6.0'

Specimen Type: Undisturbed

LL: N/A
PL: N/A
PI: N/A

Date Extruded: 10/16/2007
Date Tested: N/A

Initial Wet Density (pcf): 128.9
Initial Dry Density (pcf): 108.5
Initial Moisture Content (%): 18.8
Specific Gravity: N/A
Degree of Saturation (%): N/A

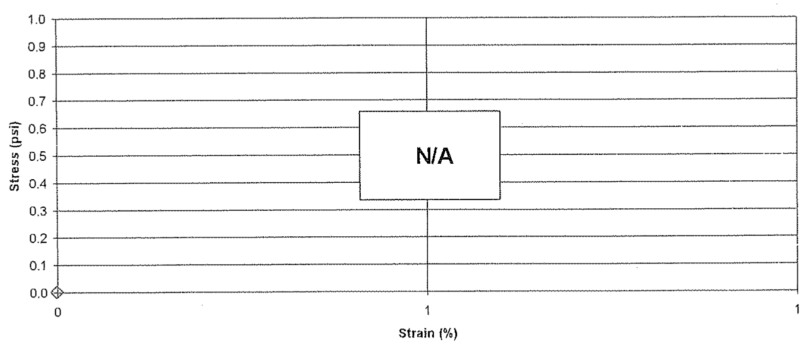
MC Taken: Before Test, From Trimmings

Unconfined Compressive Strength (tsf): N/A
Undrained Shear Strength (tsf): N/A

Average Height (in): 6.011
Average Diameter (in): 2.886
Height to Diameter Ratio: 2.1

Strain at Maximum Stress (%): N/A
Strain rate to failure (% / min.): N/A

Stress vs. Strain



Failure Sketch

Pocket Penetrometer Reading (tsf): 5.0
Torvane Reading (kg/cm^2): N/A

Comments:

Reviewed By:

LX2005125_UC-159B.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

FMSM ENGINEERS

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

Project Name: I-265 Bridge over the Ohio River
Project Number: LX2005125
Source: AC-2, 20.0'-22.0'
Lab ID: 161A
Visual Description: sandy lean Clay (CL), brown, moist, firm

Recovered: 1.5'
Test Interval: 20.0' - 20.5'

Specimen Type: Undisturbed

LL: 24
PL: 13
PI: 11

Date Extruded: 10/16/2007
Date Tested: 06/16/2007

Initial Wet Density (pcf): 128.5
Initial Dry Density (pcf): 104.0
Initial Moisture Content (%): 23.6
Specific Gravity: 2.71
Degree of Saturation (%): 102.0

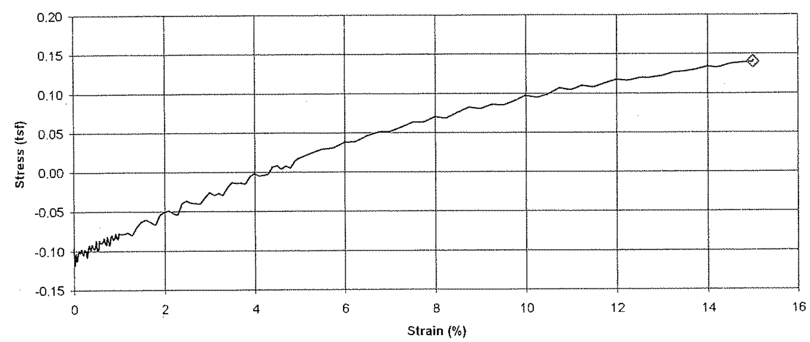
MC Taken: Before Test, From Trimmings

Unconfined Compressive Strength (tsf): 0.14
Undrained Shear Strength (tsf): 0.07

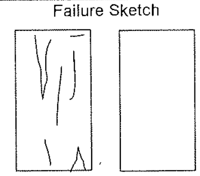
Average Height (in): 5.888
Average Diameter (in): 2.865
Height to Diameter Ratio: 2.1

Strain at Maximum Stress (%): 15.0
Strain rate to failure (% / min.): 1.00

Stress vs. Strain



Failure Sketch



Pocket Penetrometer Reading (tsf): 1.0
Torvane Reading (kg/cm^2): N/A

Comments:

Reviewed By:

LX2005125_UC-161A.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

FMSM ENGINEERS

Project Name: I-265 Bridge over the Ohio River
Project Number: LX2005125
Source: AC-2, 20.0'-22.0'
Lab ID: 161B
Visual Description: sandy lean Clay (CL), brown, moist, firm
Recovered: 1.5'
Test Interval: 20.5' - 21.0'

Specimen Type: Undisturbed

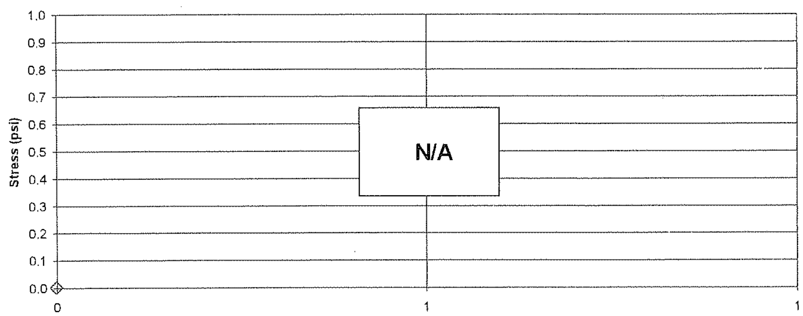
Initial Wet Density (pcf)	122.0	LL	N/A
Initial Dry Density (pcf)	95.3	PL	N/A
Initial Moisture Content (%)	27.9	PI	N/A
Specific Gravity	N/A		
Degree of Saturation (%)	N/A		
Average Height (in)	5.981		
Average Diameter (in)	2.899		
Height to Diameter Ratio	2.1		

Date Extruded: 10/16/2007
Date Tested: N/A

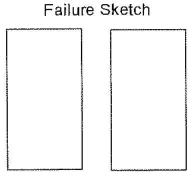
MC Taken: Before Test, From Trimmings

Unconfined Compressive Strength (tsf): N/A
Undrained Shear Strength (tsf): N/A
Strain at Maximum Stress (%): N/A
Strain rate to failure (% / min.): N/A

Stress vs. Strain



Failure Sketch



Pocket Penetrometer Reading (tsf): 1.0
Torvane Reading (kg/cm^2): N/A

Comments

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LX2005125_UC-161B.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

FMSM ENGINEERS

Project Name: I-265 Bridge over the Ohio River
Project Number: LX2005125
Source: AC-3, 2.5'-4.5'
Lab ID: 177A
Visual Description: lean Clay (CL), brown, moist, firm
Recovered: 1.3'
Test Interval: 2.5' - 3.0'

Specimen Type: Undisturbed

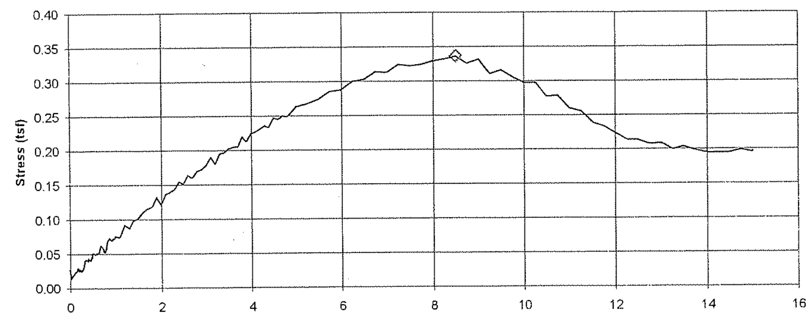
Initial Wet Density (pcf)	110.7	LL	45
Initial Dry Density (pcf)	82.7	PL	20
Initial Moisture Content (%)	33.9	PI	25
Specific Gravity	2.71		
Degree of Saturation (%)	87.8		
Average Height (in)	6.049		
Average Diameter (in)	2.860		
Height to Diameter Ratio	2.1		

Date Extruded: 10/16/2007
Date Tested: 10/16/2007

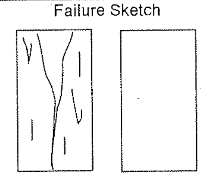
MC Taken: Before Test, From Trimmings

Unconfined Compressive Strength (tsf): 0.34
Undrained Shear Strength (tsf): 0.17
Strain at Maximum Stress (%): 8.5
Strain rate to failure (% / min.): 1.00

Stress vs. Strain



Failure Sketch



Pocket Penetrometer Reading (tsf): 2.0
Torvane Reading (kg/cm^2): N/A

Comments

Reviewed By

LX2005125_UC-177A.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil

ASTM D 2166

Fuller, Mossbarger, Scott & May — MSM ENGINEERS

Project Name I-265 Bridge over the Ohio River
Project Number LX2005125
Source AC-3, 2.5'-4.5'
Lab ID 177B
Visual Description lean Clay (CL), brown, moist, firm

Recovered 1.3'
Test Interval 3.0' - 3.5'

Specimen Type: Undisturbed

LL	N/A
PL	N/A
PI	N/A

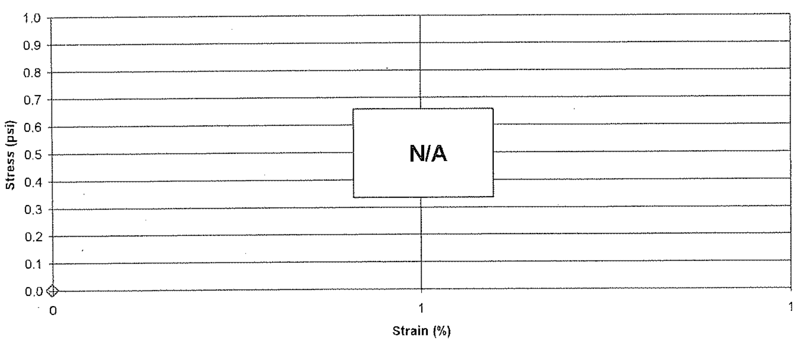
Date Extruded 10/16/2007
Date Tested N/A

Initial Wet Density (pcf)	119.7
Initial Dry Density (pcf)	89.3
Initial Moisture Content (%)	34.1
Specific Gravity	N/A
Degree of Saturation (%)	N/A
Average Height (in)	5.957
Average Diameter (in)	2.894
Height to Diameter Ratio	2.1

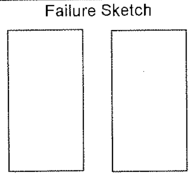
MC Taken Before Test, From Trimmings

Unconfined Compressive Strength (tsf)	N/A
Undrained Shear Strength (tsf)	N/A
Strain at Maximum Stress (%)	N/A
Strain rate to failure (% / min.)	N/A

Stress vs. Strain



Failure Sketch



Pocket Penetrometer Reading (tsf) 2.0
Torvane Reading (kg/cm^2) N/A

Comments

Reviewed By

LX2005125_UC-177B.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil

ASTM D 2166

Fuller, Mossbarger, Scott & May — MSM ENGINEERS

Project Name I-265 Bridge over the Ohio River
Project Number LX2005125
Source AC-3, 10.0'-12.0'
Lab ID 179
Visual Description lean Clay (CL), brown, moist, firm

Recovered 1'
Test Interval 10.0' - 10.5'

Specimen Type: Undisturbed

LL	39
PL	17
PI	22

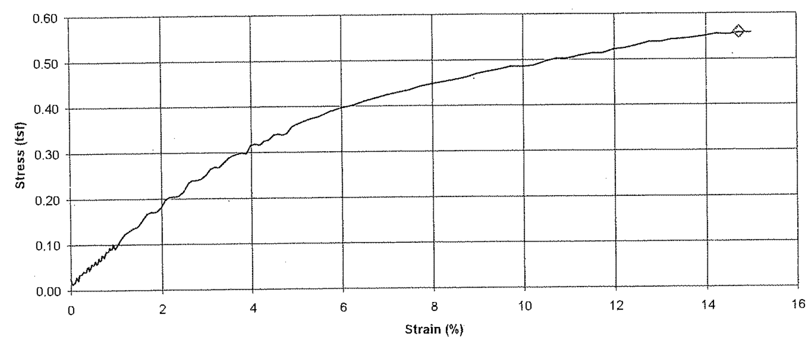
Date Extruded 10/16/2007
Date Tested 10/16/2007

Initial Wet Density (pcf)	118.8
Initial Dry Density (pcf)	88.4
Initial Moisture Content (%)	34.3
Specific Gravity	2.71
Degree of Saturation (%)	101.8
Average Height (in)	6.003
Average Diameter (in)	2.850
Height to Diameter Ratio	2.1

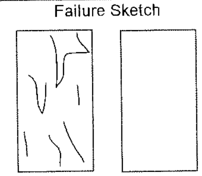
MC Taken Before Test, From Trimmings

Unconfined Compressive Strength (tsf)	0.56
Undrained Shear Strength (tsf)	0.28
Strain at Maximum Stress (%)	14.7
Strain rate to failure (% / min.)	1.00

Stress vs. Strain



Failure Sketch



Pocket Penetrometer Reading (tsf) 1.0
Torvane Reading (kg/cm^2) N/A

Comments

Reviewed By

LX2005125_UC-179.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name I-25 Bridge over the Ohio River — Project Number LX2005125
Source AC-3, 15.0'-17.0' — Lab ID 180A
Visual Description sandy lean Clay (CL), brown, moist, firm

Recovered 1.5'
Test Interval 15.0' - 15.5'

Specimen Type: Undisturbed

Initial Wet Density (pcf)	127.1
Initial Dry Density (pcf)	101.3
Initial Moisture Content (%)	25.4
Specific Gravity	N/A
Degree of Saturation (%)	N/A
Average Height (in)	6.057
Average Diameter (in)	2.826
Height to Diameter Ratio	2.1

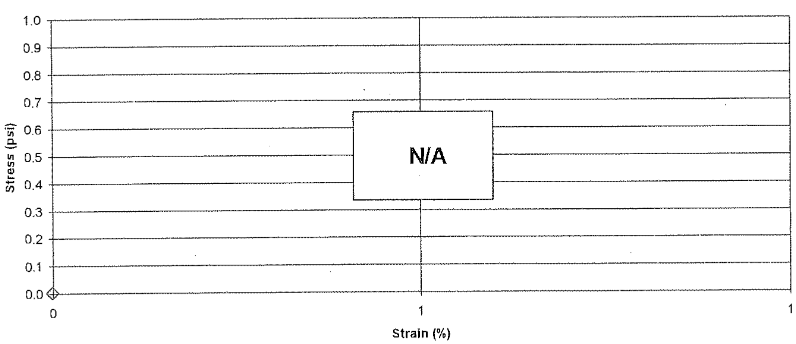
LL N/A
PL N/A
PI N/A

Date Extruded 10/16/2007
Date Tested N/A

MC Taken Before Test, From Trimmings

Unconfined Compressive Strength (tsf)	N/A
Undrained Shear Strength (tsf)	N/A
Strain at Maximum Stress (%)	N/A
Strain rate to failure (% / min.)	N/A

Stress vs. Strain



Failure Sketch

Pocket Penetrometer Reading (tsf) 1.0
Torvane Reading (kg/cm^2) N/A
Comments

Reviewed By

LX2005125_UC-180A.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

Fuller Mossbarger Scott & May — FMSM ENGINEERS

Project Name I-265 Bridge over the Ohio River — Project Number LX2005125
Source AC-3, 15.0'-17.0' — Lab ID 180B
Visual Description sandy lean Clay (CL), brown, moist, firm

Recovered 1.5'
Test Interval 15.5' - 16.0'

Specimen Type: Undisturbed

Initial Wet Density (pcf)	125.9
Initial Dry Density (pcf)	100.5
Initial Moisture Content (%)	25.3
Specific Gravity	N/A
Degree of Saturation (%)	N/A
Average Height (in)	5.863
Average Diameter (in)	2.862
Height to Diameter Ratio	2.0

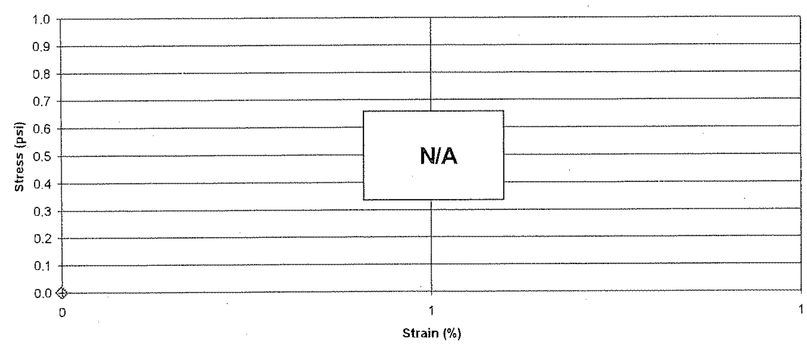
LL N/A
PL N/A
PI N/A

Date Extruded 10/16/2007
Date Tested N/A

MC Taken Before Test, From Trimmings

Unconfined Compressive Strength (tsf)	N/A
Undrained Shear Strength (tsf)	N/A
Strain at Maximum Stress (%)	N/A
Strain rate to failure (% / min.)	N/A

Stress vs. Strain



Failure Sketch

Pocket Penetrometer Reading (tsf) 1.0
Torvane Reading (kg/cm^2) N/A
Comments 16.0' - 16.5' out for classification.

Reviewed By

LX2005125_UC-180B.xls UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Unconfined Compressive Strength of Cohesive Soil

KM 64-522

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-5, 3.0'-5.0'
Lab ID: 201
Visual Description: sandy lean Clay (CL), brown, moist, soft
Recovered: 2'
Test Interval: 3.0' - 5.0'

Specimen Type: Undisturbed

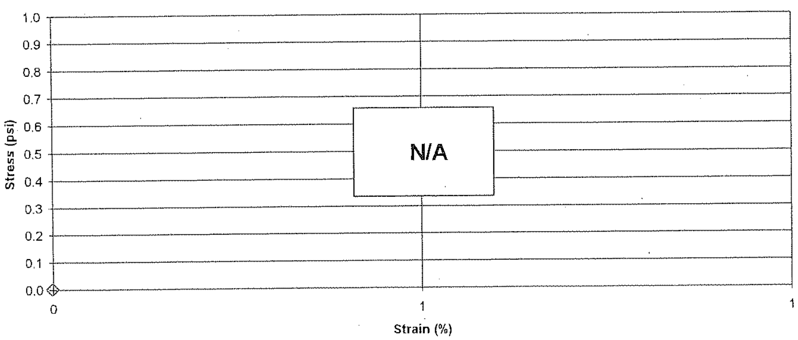
Initial Wet Density (pcf)		LL	N/A	Date Extruded	10/23/2007
Initial Dry Density (pcf)	N/A	PL	N/A	Date Tested	N/A
Initial Moisture Content (%)	9.2	PI	N/A		
Specific Gravity	N/A				
Degree of Saturation (%)	N/A				
Average Height (in)	N/A				
Average Diameter (in)	N/A				
Height to Diameter Ratio	N/A				

MC Taken: Before Test, From Center of Specimen

Unconfined Compressive Strength (tsf): N/A
Undrained Shear Strength (tsf): N/A

Strain at Maximum Stress (%): N/A
Strain rate to failure (% / min.): N/A

Stress vs. Strain



Failure Sketch

Pocket Penetrometer Reading (tsf): 0.5
Torvane Reading (kg/cm^2): N/A
Comments: No 6" test specimen due to multiple sand lenses. 3.0'-3.5' put out for class. The rest was saved in a bag.

Reviewed By: ____

LX2005125_UC-201 UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Unconfined Compressive Strength of Cohesive Soil

KM 64-522

Project Name: I-265 Over Ohio River
Project Number: LX2005125
Source: AC-5, 5.0'-7.0'
Lab ID: 202
Visual Description: silty Sand (SM), brown, moist, soft
Recovered: 2'
Test Interval: 5.0' - 7.0'

Specimen Type: Undisturbed

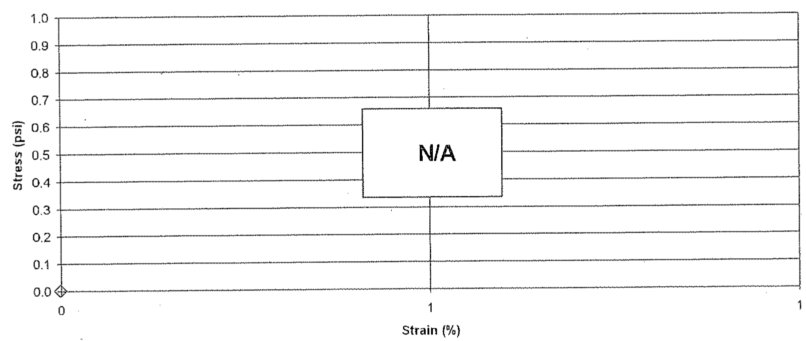
Initial Wet Density (pcf)		LL	N/A	Date Extruded	10/23/2007
Initial Dry Density (pcf)	N/A	PL	N/A	Date Tested	N/A
Initial Moisture Content (%)	6.5	PI	N/A		
Specific Gravity	N/A				
Degree of Saturation (%)	N/A				
Average Height (in)	N/A				
Average Diameter (in)	N/A				
Height to Diameter Ratio	N/A				

MC Taken: Before Test, From Center of Specimen

Unconfined Compressive Strength (tsf): N/A
Undrained Shear Strength (tsf): N/A

Strain at Maximum Stress (%): N/A
Strain rate to failure (% / min.): N/A

Stress vs. Strain



Failure Sketch

Pocket Penetrometer Reading (tsf): 0.5
Torvane Reading (kg/cm^2): N/A
Comments: No 6" test specimen obtained due to multiple sand lenses. 5.0'-7.0' put out for classification testing. Saved in a bag.

Reviewed By: ____

LX2005125_UC-202 UC-report
Preparation Date: 9-17-98
Revision Date: 03-2001

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: MW
Approved By: TLK

FMSM ENGINEERS

Moisture Content of Soil
AASHTO T 265

Project Name ____________

Project Number LX2005125
Tested By RC

Maximum Particle Size in Sample	No. 10	No. 4	3/8"	3/4"	1 1/2"	3"
Recommended Minimum Mass (g)	20	100	500	2,500	10,000	50,000

Material Type: Stratified, Laminated, Lensed, Homogeneous

Source	Lab ID	Date Tested	Material Type	Maximum Particle Size	Material Excluded Amount	Material Excluded Size	Pass Min. Mass? (Y/N)	Can Weight (g)	Wet Soil & Can Weight (g)	Dry Soil & CanWeight (g)	Moisture Content (%)
AC-1, 5.0'-7.0'	147	10/16/07	Hom	No. 10			Yes	19.52	88.80	77.64	19.2
AC-1, 15.0'-17.0'	149	10/16/07	Hom	No. 4			No	20.54	121.52	100.51	26.3
AC-2, 2.5'-4.5'	158	10/16/07	Hom	No. 10			Yes	20.89	86.72	76.60	18.2
AC-2, 10.0'-12.0'	160	10/16/07	Hom	No. 10			Yes	20.91	99.19	86.20	19.9
AC-2, 15.0'-16.5'	162	10/16/07	Hom	No. 10			Yes	21.40	62.16	53.86	25.6
AC-3, 5.0'-7.0'	178	10/16/07	Hom	No. 10			Yes	20.37	106.24	83.45	36.1
AC-3, 20.0'-21.5'	181	10/16/07	Hom	No. 10			Yes	20.90	115.86	100.62	19.1

FMSM ENGINEERS

Moisture Content of Soil
AASHTO T 265

Project Name ____________

Project Number LX2005125
Tested By DG

Maximum Particle Size in Sample	No. 10	No. 4	3/8"	3/4"	1 1/2"	3"
Recommended Minimum Mass (g)	20	100	500	2,500	10,000	50,000

Material Type: Stratified, Laminated, Lensed, Homogeneous

Source	Lab ID	Date Tested	Material Type	Maximum Particle Size	Material Excluded Amount	Material Excluded Size	Pass Min. Mass? (Y/N)	Can Weight (g)	Wet Soil & Can Weight (g)	Dry Soil & CanWeight (g)	Moisture Content (%)
AC-17/212+17.0, 87.0 Lt., 0.0'-1.5'	232	10/23/07	Hom	1 1/2"			No	20.62	114.13	106.16	9.3
AC-20/212+30.0, 56.0 Lt., 0.0'-1.5'	234	10/23/07	Hom	1 1/2"			No	26.26	110.53	98.62	16.5
AC-23/ 212+50.0 Cl., 2.5'-3.6'	238	10/23/07	Hom	3/4"			No	27.17	133.32	128.23	5.0
AC-26/ 212+70.0, 55.0 Rt., 0.0'-1.5'	242	10/23/07	Hom	3/8"			No	28.98	120.06	111.61	10.2
AC-26/ 212+70.0, 55.0 Rt., 5.0'-6.5'	243	10/23/07	Hom	1 1/2"			No	21.20	99.80	92.07	10.9
AC-27/ 212+87, 125.0 Rt., 0.0'-1.2'	249	10/23/07	Hom	1 1/2"			No	311.08	611.10	592.71	6.5

APPENDIX F
LABORATORY TEST RESULTS - ROCK

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

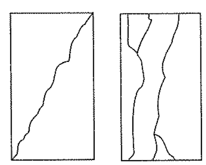
Project Name I-265 Over Ohio River | Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale seams | Lab ID UCR-155
Hole Number AC-1 | Depth (ft/elev) 114.95' - 115.35' | Date Received 09-28-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.780	Weight (lb)	1.401
Perpendicularity	Pass	Diameter (in)	1.981	Wet Unit Weight (pcf)	164.3
End Planeness	Pass	Area (in^2)	3.082	Dry Unit Weight (pcf)	163.2
		Height/Diameter Ratio	2.413	Moisture Content[1] (%)	0.7

Moisture Condition As received, moist
Temperature (°C) 19
Loading Rate (lbf/sec) 110
Peak Load (lbf) 26740

Date Tested 10-16-2007

Failure Sketches



Compressive Strength (psi) 8680

Compressive Strength (tsf) 625

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

File: LX2005125_UCR-155.xls Sheet: Report
Preparation Date: 2-2002
Revision Date: 7-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

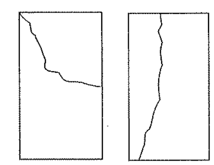
Project Name I-265 Over Ohio River | Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layers | Lab ID UCR-157
Hole Number AC-1 | Depth (ft/elev) 127.3' - 127.65' | Date Received 09-28-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.505	Weight (lb)	1.323
Perpendicularity	Pass	Diameter (in)	1.974	Wet Unit Weight (pcf)	165.8
End Planeness	Pass	Area (in^2)	3.059	Dry Unit Weight (pcf)	161.6
		Height/Diameter Ratio	2.283	Moisture Content[1] (%)	2.6

Moisture Condition As received, moist
Temperature (°C) 19
Loading Rate (lbf/sec) 110
Peak Load (lbf) 12590

Date Tested 10-16-2007

Failure Sketches



Compressive Strength (psi) 4120

Compressive Strength (tsf) 296

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

File: LX2005125_UCR-157.xls Sheet: Report
Preparation Date: 2-2002
Revision Date: 7-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-2
Depth (ft/elev) 119.1' - 119.5'
Project Number LX2005125
Lab ID UCR-173
Date Received 09-28-2007

Dimensional Conformance	
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions			
Height (in)	4.265	Weight (lb)	1.248
Diameter (in)	1.972	Wet Unit Weight (pcf)	165.6
Area (in^2)	3.053	Dry Unit Weight (pcf)	164.4
Height/Diameter Ratio	2.163	Moisture Content[1] (%)	0.7

Moisture Condition As received, moist
Temperature (°C) 19
Loading Rate (lbf/sec) 110
Peak Load (lbf) 22810

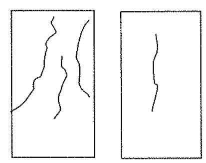
Compressive Strength (psi) 7470

Compressive Strength (tsf) 538

Failure Type Undetermined

Date Tested 10-16-2007

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-2
Depth (ft/elev) 124.7' - 125.1'
Project Number LX2005125
Lab ID UCR-175
Date Received 09-28-2007

Dimensional Conformance	
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions			
Height (in)	4.631	Weight (lb)	1.327
Diameter (in)	1.970	Wet Unit Weight (pcf)	162.5
Area (in^2)	3.048	Dry Unit Weight (pcf)	160.4
Height/Diameter Ratio	2.351	Moisture Content[1] (%)	1.3

Moisture Condition As received, moist
Temperature (°C) 19
Loading Rate (lbf/sec) 110
Peak Load (lbf) 7580

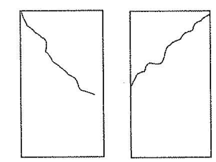
Compressive Strength (psi) 2490

Compressive Strength (tsf) 179

Failure Type Shear

Date Tested 10-16-2007

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

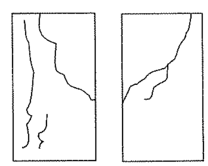
Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone , gray , moderately hard, shale seams 124.4
Lab ID UCR-193
Hole Number AC-3
Depth (ft/elev) 124.1' - 124.7'
Date Received 09-28-2007

Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Specimen Dimensions
Height (in) 7.126
Diameter (in) 3.316
Area (in^2) 8.638
Height/Diameter Ratio 2.149
Weight (lb) 5.963
Wet Unit Weight (pcf) 167.4
Dry Unit Weight (pcf) 166.2
Moisture Content[1] (%) 0.7

Moisture Condition As received, moist
Date Tested 10-16-2007
Temperature (°C) 19
Loading Rate (lbf/sec) 220
Peak Load (lbf) 63030

Failure Sketches



Compressive Strength (psi) 7300
Compressive Strength (tsf) 525
Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A
Corrected Compressive Strength (psi) N/A
Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

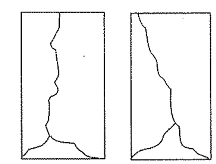
Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Shale , dark gray, moderately hard, limestone layer
Lab ID UCR-194
Hole Number AC-3
Depth (ft/elev) 128.6' - 129.2'
Date Received 09-28-2007

Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Specimen Dimensions
Height (in) 6.616
Diameter (in) 3.316
Area (in^2) 8.636
Height/Diameter Ratio 1.995
Weight (lb) 5.504
Wet Unit Weight (pcf) 166.4
Dry Unit Weight (pcf) 163.5
Moisture Content[1] (%) 1.8

Moisture Condition As received, moist
Date Tested 10-16-2007
Temperature (°C) 19
Loading Rate (lbf/sec) 220
Peak Load (lbf) 35970

Failure Sketches



Compressive Strength (psi) 4170
Compressive Strength (tsf) 300
Failure Type Cone and Split

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient 1.000
Corrected Compressive Strength (psi) 4170
Corrected Compressive Strength (tsf) 300

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-4/189+81.55, 91.9 Depth (ft/elev) 117.4' - 117.8'

Project Number LX2005125
Lab ID UCR-32
Date Received 08-10-2007

Dimensional Conformance	
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

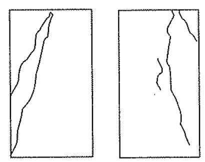
Specimen Dimensions

Height (in)	4.754
Diameter (in)	1.990
Area (in^2)	3.111
Height/Diameter Ratio	2.388
Weight (lb)	1.405
Wet Unit Weight (pcf)	164.2
Dry Unit Weight (pcf)	163.7
Moisture Content[1] (%)	0.3

Moisture Condition As received, dry
Temperature (°C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 28620

Date Tested 09-19-2007

Failure Sketches



Compressive Strength (psi) 9200

Compressive Strength (tsf) 662

Failure Type Columnar

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-4/189+81.55, 91.9 Depth (ft/elev) 124.35' -124.75'

Project Number LX2005125
Lab ID UCR-34
Date Received 08-10-2007

Dimensional Conformance	
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

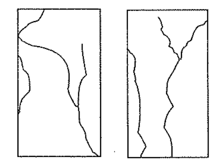
Specimen Dimensions

Height (in)	4.332
Diameter (in)	1.991
Area (in^2)	3.113
Height/Diameter Ratio	2.176
Weight (lb)	1.297
Wet Unit Weight (pcf)	166.2
Dry Unit Weight (pcf)	163.9
Moisture Content[1] (%)	1.4

Moisture Condition As received, moist
Temperature (°C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 17700

Date Tested 09-19-2007

Failure Sketches



Compressive Strength (psi) 5690

Compressive Strength (tsf) 409

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layers
Lab ID UCR-42
Hole Number AC-6/193+51.7, 0.28' Rt.
Depth (ft/elev) 105.0' - 105.35'
Date Received 08-10-2007

Dimensional Conformance

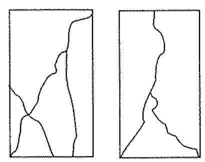
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.700	Weight (lb)	1.362
Diameter (in)	1.981	Wet Unit Weight (pcf)	162.5
Area (in^2)	3.081	Dry Unit Weight (pcf)	161.7
Height/Diameter Ratio	2.373	Moisture Content[1] (%)	0.5

Moisture Condition As received, dry
Date Tested 09-19-2007
Temperature (°C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 25430

Failure Sketches



Compressive Strength (psi) 8250

Compressive Strength (tsf) 594

Failure Type Cone and Split

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale seams
Lab ID UCR-45
Hole Number AC-6/193+51.7, 0.28' Rt.
Depth (ft/elev) 113.35' - 113.7'
Date Received 08-10-2007

Dimensional Conformance

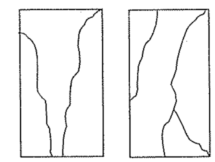
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.212	Weight (lb)	1.242
Diameter (in)	1.978	Wet Unit Weight (pcf)	165.8
Area (in^2)	3.073	Dry Unit Weight (pcf)	165.1
Height/Diameter Ratio	2.129	Moisture Content[1] (%)	0.4

Moisture Condition As Received, dry
Date Tested 09-19-2007
Temperature (°C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 13190

Failure Sketches



Compressive Strength (psi) 4290

Compressive Strength (tsf) 309

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

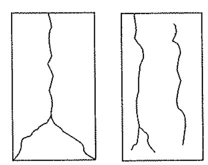
KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layer
Lab ID UCR-47
Hole Number AC-6/193+51.7, 0.28' Rt.
Depth (ft/elev) 128.2' - 128.55'
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	3.919	Weight (lb)	1.108
Perpendicularity	Pass	Diameter (in)	1.976	Wet Unit Weight (pcf)	159.3
End Planeness	Pass	Area (in^2)	3.067	Dry Unit Weight (pcf)	158.6
		Height/Diameter Ratio	1.983	Moisture Content[1] (%)	0.5

Moisture Condition As received, dry
Date Tested 09-19-2007
Temperature (°C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 24960

Failure Sketches



Compressive Strength (psi) 8140

Compressive Strength (tsf) 586

Failure Type Cone and Split

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient 0.999

Corrected Compressive Strength (psi) 8130

Corrected Compressive Strength (tsf) 585

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

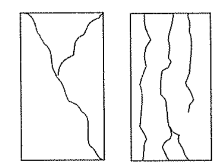
KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale seams
Lab ID UCR-54
Hole Number AC-7 193+95, 68' Lt.
Depth (ft/elev) 100.0' - 100.35'
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	3.998	Weight (lb)	1.181
Perpendicularity	Pass	Diameter (in)	1.986	Wet Unit Weight (pcf)	164.8
End Planeness	Pass	Area (in^2)	3.097	Dry Unit Weight (pcf)	164.2
		Height/Diameter Ratio	2.014	Moisture Content[1] (%)	0.4

Moisture Condition As received, dry
Date Tested 09-19-2007
Temperature (°C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 29720

Failure Sketches



Compressive Strength (psi) 9600

Compressive Strength (tsf) 691

Failure Type Cone and Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Fuller Mossbarger Scott & May

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Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River — Project Number LX2005125

Lithology Limestone, gray, moderately hard, shale seams — Lab ID UCR-58

Hole Number AC-7 193+95, 68' Lt. — Depth (ft/elev) 109.3' - 109.65' — Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.404	Weight (lb)	1.306
Perpendicularity	Pass	Diameter (in)	1.983	Wet Unit Weight (pcf)	166.0
End Planeness	Pass	Area (in^2)	3.087	Dry Unit Weight (pcf)	165.3
		Height/Diameter Ratio	2.221	Moisture Content[1] (%)	0.4

Moisture Condition As received, dry — Date Tested 09-06-2007

Temperature (°C) 21.5

Loading Rate (lbf/sec) 110

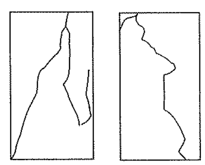
Peak Load (lbf) 37790

Compressive Strength (psi) 12240

Compressive Strength (tsf) 881

Failure Type Cone and Split

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River — Project Number LX2005125

Lithology Limestone, gray, moderately hard, shale layers — Lab ID UCR-59

Hole Number AC-7 193+95, 68' Lt. — Depth (ft/elev) 111.9' - 112.35' — Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	3.600	Weight (lb)	1.063
Perpendicularity	Pass	Diameter (in)	1.982	Wet Unit Weight (pcf)	165.3
End Planeness	Pass	Area (in^2)	3.086	Dry Unit Weight (pcf)	164.6
		Height/Diameter Ratio	1.816	Moisture Content[1] (%)	0.4

Moisture Condition As received, dry — Date Tested 09-19-2007

Temperature (°C) 21.5

Loading Rate (lbf/sec) 110

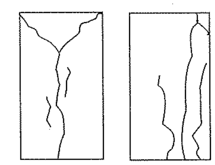
Peak Load (lbf) 35870

Compressive Strength (psi) 11620

Compressive Strength (tsf) 837

Failure Type Cone and Split

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient 0.988

Corrected Compressive Strength (psi) 11480

Corrected Compressive Strength (tsf) 827

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

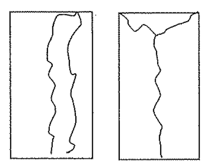
Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layer
Lab ID UCR-68
Hole Number AC-8/193+95, 1.22'Lt. Depth (ft/elev) 99.3' - 99.7'
Date Received 08-10-2007

Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Specimen Dimensions
Height (in) 3.991
Diameter (in) 1.979
Area (in^2) 3.076
Height/Diameter Ratio 2.017
Weight (lb) 1.175
Wet Unit Weight (pcf) 165.4
Dry Unit Weight (pcf) 164.8
Moisture Content[1] (%) 0.4

Moisture Condition As received, dry
Date Tested 09-19-2007
Temperature (^{o}C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 35810

Failure Sketches



Compressive Strength (psi) 11640

Compressive Strength (tsf) 838

Failure Type Cone and Split

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

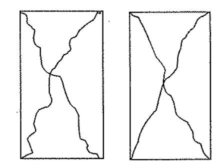
Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Shale, dark gray, soft, limestone layer
Lab ID UCR-69
Hole Number AC-8/193+95, 1.22'Lt. Depth (ft/elev) 102.85' - 103.2'
Date Received 08-10-2007

Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Specimen Dimensions
Height (in) 3.991
Diameter (in) 2.010
Area (in^2) 3.172
Height/Diameter Ratio 1.986
Weight (lb) 1.143
Wet Unit Weight (pcf) 156.0
Dry Unit Weight (pcf) 150.9
Moisture Content[1] (%) 3.4

Moisture Condition As received, moist
Date Tested 09-19-2007
Temperature (^{o}C) 21.5
Loading Rate (lbf/sec) 110
Peak Load (lbf) 1490

Failure Sketches



Compressive Strength (psi) 470

Compressive Strength (tsf) 34

Failure Type Cone

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient 0.999

Corrected Compressive Strength (psi) 470

Corrected Compressive Strength (tsf) 34

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

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Project Name I-265 Over Ohio River

Project Number LX2005125

Lithology Limestone, gray, moderately hard

Lab ID UCR-73

Hole Number AC-8/193+95, 1.22'Lt. Depth (ft/elev) 120.65' - 121.05' Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.445	Weight (lb)	1.312
Perpendicularity	Pass	Diameter (in)	1.980	Wet Unit Weight (pcf)	165.7
End Planeness	Pass	Area (in^2)	3.079	Dry Unit Weight (pcf)	165.2
		Height/Diameter Ratio	2.245	Moisture Content[1] (%)	0.3

Moisture Condition As received, dry

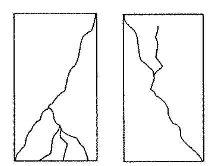
Date Tested 09-20-2007

Temperature ($^\circ$C) 20.2

Loading Rate (lbf/sec) 110

Peak Load (lbf) 24670

Failure Sketches



Compressive Strength (psi) 8010

Compressive Strength (tsf) 577

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

FMSM ENGINEERS

Project Name I-265 Over Ohio River

Project Number LX2005125

Lithology Limestone, gray, moderately hard, shale layers

Lab ID UCR-80

Hole Number AC-9/193+95, 70' Rt. Depth (ft/elev) 105.75' - 106.15' Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.414	Weight (lb)	1.313
Perpendicularity	Pass	Diameter (in)	1.984	Wet Unit Weight (pcf)	166.2
End Planeness	Pass	Area (in^2)	3.093	Dry Unit Weight (pcf)	165.7
		Height/Diameter Ratio	2.225	Moisture Content[1] (%)	0.3

Moisture Condition As received, dry

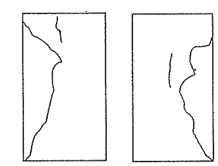
Date Tested 09-20-2007

Temperature ($^\circ$C) 20.2

Loading Rate (lbf/sec) 110

Peak Load (lbf) 21780

Failure Sketches



Compressive Strength (psi) 7040

Compressive Strength (tsf) 507

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

FMSM ENGINEERS

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layers
Lab ID UCR-83
Hole Number AC-9/193+95, 70' Rt.
Depth (ft/elev) 116.5' - 116.9'
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	3.755	Weight (lb)	1.090
Perpendicularity	Pass	Diameter (in)	1.982	Wet Unit Weight (pcf)	162.6
End Planeness	Pass	Area (in^2)	3.085	Dry Unit Weight (pcf)	161.8
		Height/Diameter Ratio	1.894	Moisture Content[1] (%)	0.5

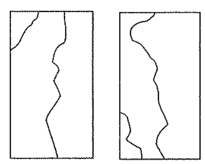
Moisture Condition As received, dry
Date Tested 09-20-2007
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 22920

Compressive Strength (psi) 7430

Compressive Strength (tsf) 535

Failure Type Undetermined

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient 0.993

Corrected Compressive Strength (psi) 7380

Corrected Compressive Strength (tsf) 531

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

FMSM ENGINEERS

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard
Lab ID UCR-85
Hole Number AC-9/193+95, 70' Rt.
Depth (ft/elev) 119.8' - 120.15'
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.015	Weight (lb)	1.184
Perpendicularity	Pass	Diameter (in)	1.983	Wet Unit Weight (pcf)	165.1
End Planeness	Pass	Area (in^2)	3.087	Dry Unit Weight (pcf)	164.4
		Height/Diameter Ratio	2.025	Moisture Content[1] (%)	0.4

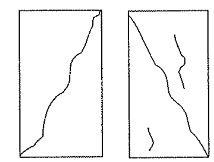
Moisture Condition As received, dry
Date Tested 09-20-2007
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 32570

Compressive Strength (psi) 10550

Compressive Strength (tsf) 760

Failure Type Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

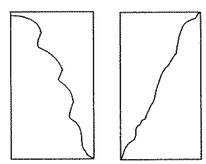
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale layer
Hole Number AC-10/205+98, 70' Lt. Depth (ft/elev) 100.2' - 100.5'
Project Number LX2005125
Lab ID UCR-91
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	3.266	Weight (lb)	0.962
Perpendicularity	Pass	Diameter (in)	1.985	Wet Unit Weight (pcf)	164.4
End Planeness	Pass	Area (in^2)	3.096	Dry Unit Weight (pcf)	162.3
		Height/Diameter Ratio	1.645	Moisture Content[1] (%)	1.3

Moisture Condition As received, moist
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 16020

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 5170

Compressive Strength (tsf) 373

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient 0.975

Corrected Compressive Strength (psi) 5040

Corrected Compressive Strength (tsf) 364

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

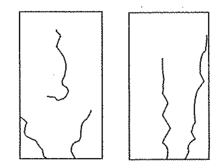
Project Name I-265 Over Ohio River
Lithology Shale, dark gray, soft
Hole Number AC-10/205+98, 70' Lt. Depth (ft/elev) 104.8' - 105.2'
Project Number LX2005125
Lab ID UCR-94
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.521	Weight (lb)	1.330
Perpendicularity	Pass	Diameter (in)	1.982	Wet Unit Weight (pcf)	164.8
End Planeness	Pass	Area (in^2)	3.085	Dry Unit Weight (pcf)	162.2
		Height/Diameter Ratio	2.281	Moisture Content[1] (%)	1.6

Moisture Condition As received, moist
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 10850

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 3520

Compressive Strength (tsf) 253

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray , moderately hard, shale seams
Hole Number AC-10/205+98, 70' Lt.   Depth (ft/elev) 112.5' - 112.9'

Project Number LX2005125
Lab ID UCR-96
Date Received 08-10-2007

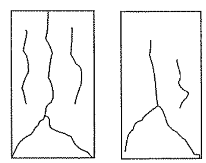
Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Specimen Dimensions
Height (in) 4.483
Diameter (in) 1.982
Area (in^2) 3.084
Height/Diameter Ratio 2.262
Weight (lb) 1.321
Wet Unit Weight (pcf) 165.1
Dry Unit Weight (pcf) 164.5
Moisture Content[1] (%) 0.4

Moisture Condition As received, dry
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 41160

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 13350

Compressive Strength (tsf) 961

Failure Type Cone and Split

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-11/205+95,1.3 Lt.   Depth (ft/elev) 92.1' - 92.55'

Project Number LX2005125
Lab ID UCR-105
Date Received 08-10-2007

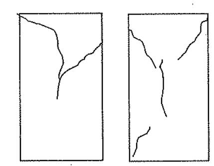
Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Specimen Dimensions
Height (in) 4.444
Diameter (in) 1.987
Area (in^2) 3.102
Height/Diameter Ratio 2.236
Weight (lb) 1.316
Wet Unit Weight (pcf) 165.0
Dry Unit Weight (pcf) 163.7
Moisture Content[1] (%) 0.8

Moisture Condition As received, moist
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 17740

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 5720

Compressive Strength (tsf) 412

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

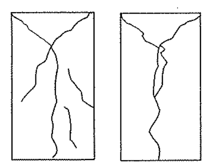
FMSM ENGINEERS

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layer
Lab ID UCR-108
Hole Number AC-11/205+95,1.3 Lt. Depth (ft/elev) 104.6' - 105.0'
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.215	Weight (lb)	1.219
Perpendicularity	Pass	Diameter (in)	1.983	Wet Unit Weight (pcf)	161.8
End Planeness	Pass	Area (in^2)	3.088	Dry Unit Weight (pcf)	160.7
		Height/Diameter Ratio	2.126	Moisture Content[1] (%)	0.7

Moisture Condition As received, dry
Date Tested 09-20-2007
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 18480

Failure Sketches



Compressive Strength (psi) 5980

Compressive Strength (tsf) 431

Failure Type Cone and Split

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

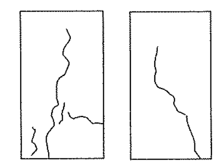
FMSM ENGINEERS

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale seams
Lab ID UCR-109
Hole Number AC-11/205+95,1.3 Lt. Depth (ft/elev) 106.95' - 107.3
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.448	Weight (lb)	1.318
Perpendicularity	Pass	Diameter (in)	1.985	Wet Unit Weight (pcf)	165.5
End Planeness	Pass	Area (in^2)	3.095	Dry Unit Weight (pcf)	164.0
		Height/Diameter Ratio	2.241	Moisture Content[1] (%)	0.9

Moisture Condition As received, moist
Date Tested 09-20-2007
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 27320

Failure Sketches



Compressive Strength (psi) 8830

Compressive Strength (tsf) 636

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

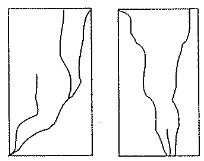
KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layer
Lab ID UCR-119
Hole Number AC-12/205+94, 71 Rt.
Depth (ft/elev) 102.85' - 103.1'
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	3.127	Weight (lb)	0.919
Perpendicularity	Pass	Diameter (in)	1.989	Wet Unit Weight (pcf)	163.4
End Planeness	Pass	Area (in^2)	3.106	Dry Unit Weight (pcf)	162.6
		Height/Diameter Ratio	1.572	Moisture Content[1] (%)	0.5

Moisture Condition As received, dry
Date Tested 09-20-2007
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 21610

Failure Sketches



Compressive Strength (psi) 6960

Compressive Strength (tsf) 501

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient 0.968

Corrected Compressive Strength (psi) 6740

Corrected Compressive Strength (tsf) 485

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

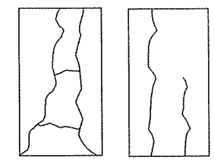
KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale traces
Lab ID UCR-120
Hole Number AC-12/205+94, 71 Rt.
Depth (ft/elev) 103.4' - 103.75'
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.178	Weight (lb)	1.228
Perpendicularity	Pass	Diameter (in)	1.986	Wet Unit Weight (pcf)	163.9
End Planeness	Pass	Area (in^2)	3.099	Dry Unit Weight (pcf)	162.9
		Height/Diameter Ratio	2.103	Moisture Content[1] (%)	0.6

Moisture Condition As received, dry
Date Tested 09-20-2007
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 31170

Failure Sketches



Compressive Strength (psi) 10060

Compressive Strength (tsf) 724

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

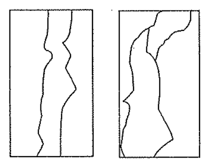
Project Name I-265 Over Ohio River — Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale areas — Lab ID UCR-122
Hole Number AC-12/205+94, 71 Rt. — Depth (ft/elev) 111.4' 111.8' — Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.731	Weight (lb)	1.412
Perpendicularity	Pass	Diameter (in)	1.991	Wet Unit Weight (pcf)	165.7
End Planeness	Pass	Area (in^2)	3.113	Dry Unit Weight (pcf)	165.2
		Height/Diameter Ratio	2.376	Moisture Content[1] (%)	0.3

Moisture Condition As received, dry
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 27660

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 8880

Compressive Strength (tsf) 640

Failure Type Cone and Split

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

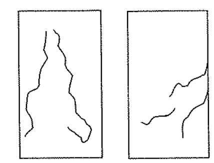
Project Name I-265 Over Ohio River — Project Number LX2005125
Lithology Limestone, gray, moderately hard, shale layers and areas — Lab ID UCR-127
Hole Number AC-13/206+50, 0.02 Lt. — Depth (ft/elev) 90.65' - 91.0' — Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.149	Weight (lb)	1.225
Perpendicularity	Pass	Diameter (in)	1.981	Wet Unit Weight (pcf)	165.5
End Planeness	Pass	Area (in^2)	3.081	Dry Unit Weight (pcf)	165.1
		Height/Diameter Ratio	2.095	Moisture Content[1] (%)	0.3

Moisture Condition As received, dry
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 31160

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 10110

Compressive Strength (tsf) 728

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

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Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale layers
Hole Number AC-13/206+50, 0.02 Lt. Depth (ft/elev) 97.75' - 98.1'

Project Number LX2005125
Lab ID UCR-130
Date Received 08-10-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

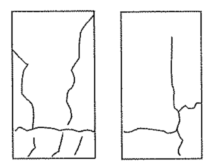
Specimen Dimensions

Height (in)	4.228	Weight (lb)	1.240
Diameter (in)	1.975	Wet Unit Weight (pcf)	165.5
Area (in^2)	3.063	Dry Unit Weight (pcf)	164.0
Height/Diameter Ratio	2.141	Moisture Content[1] (%)	0.9

Moisture Condition As received, moist
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 14230

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 4650

Compressive Strength (tsf) 335

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

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Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-13/206+50, 0.02 Lt. Depth (ft/elev) 105.85' - 106.2

Project Number LX2005125
Lab ID UCR-133
Date Received 08-10-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

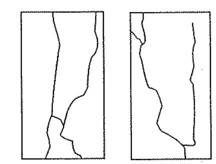
Specimen Dimensions

Height (in)	4.422	Weight (lb)	1.290
Diameter (in)	1.968	Wet Unit Weight (pcf)	165.7
Area (in^2)	3.042	Dry Unit Weight (pcf)	163.1
Height/Diameter Ratio	2.247	Moisture Content[1] (%)	1.6

Moisture Condition As received, moist
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 19640

Date Tested 09-20-2007

Failure Sketches



Compressive Strength (psi) 6460

Compressive Strength (tsf) 465

Failure Type Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Shale, red, very soft
Hole Number AC-14
Depth (ft/elev) 29.2'-29.55'

Project Number LX2005125
Lab ID UCR-228
Date Received 10-23-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.382
Diameter (in)	2.001
Area (in^2)	3.144
Height/Diameter Ratio	2.190
Weight (lb)	1.163
Wet Unit Weight (pcf)	145.9
Dry Unit Weight (pcf)	135.3
Moisture Content[1] (%)	7.8

Moisture Condition As received, moist
Temperature (°C) 23
Loading Rate (lbf/sec) 20
Peak Load (lbf) 580

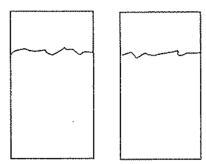
Compressive Strength (psi) 180

Compressive Strength (tsf) 13

Failure Type Shear

Date Tested 10-26-2007

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number AC-14
Depth (ft/elev) 34.1'-34.45'

Project Number LX2005125
Lab ID UCR-230
Date Received 10-23-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.492
Diameter (in)	1.983
Area (in^2)	3.088
Height/Diameter Ratio	2.265
Weight (lb)	1.334
Wet Unit Weight (pcf)	166.1
Dry Unit Weight (pcf)	165.0
Moisture Content[1] (%)	0.7

Moisture Condition As received, moist
Temperature (°C) 23
Loading Rate (lbf/sec) 110
Peak Load (lbf) 42370

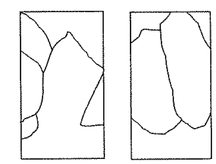
Compressive Strength (psi) 13720

Compressive Strength (tsf) 988

Failure Type Cone and Shear

Date Tested 10-26-2007

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-15/210+20, 37.3 Rt. Depth (ft/elev) 35.4' - 35.8'

Project Number LX2005125
Lab ID UCR-139
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.445	Weight (lb)	1.318
Perpendicularity	Pass	Diameter (in)	1.975	Wet Unit Weight (pcf)	167.2
End Planeness	Pass	Area (in^2)	3.065	Dry Unit Weight (pcf)	166.5
		Height/Diameter Ratio	2.250	Moisture Content[1] (%)	0.4

Moisture Condition As received, dry
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 44120

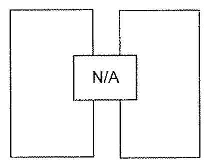
Compressive Strength (psi) 14400

Compressive Strength (tsf) 1037

Failure Type Undetermined

Date Tested 09-20-2007

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Fuller Mossbarger Scott & May

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Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number AC-15/210+20, 37.3 Rt. Depth (ft/elev) 48.2' - 48.6'

Project Number LX2005125
Lab ID UCR-140
Date Received 08-10-2007

Dimensional Conformance		Specimen Dimensions			
Side Planeness	Pass	Height (in)	4.553	Weight (lb)	1.378
Perpendicularity	Pass	Diameter (in)	1.978	Wet Unit Weight (pcf)	170.3
End Planeness	Pass	Area (in^2)	3.073	Dry Unit Weight (pcf)	167.4
		Height/Diameter Ratio	2.302	Moisture Content[1] (%)	1.7

Moisture Condition As received, moist
Temperature (°C) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 26280

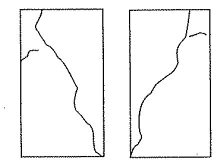
Compressive Strength (psi) 8550

Compressive Strength (tsf) 616

Failure Type Shear

Date Tested 09-20-2007

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

FMSM ENGINEERS

Project Name I-265 Over Ohio River
Lithology Limestone, dark gray, moderately hard, shale seams
Hole Number AC-15/210+20, 37.3 Rt.   Depth (ft/elev) 60.4' - 60.8'

Project Number LX2005125
Lab ID UCR-145
Date Received 08-10-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.656	Weight (lb)	1.379
Diameter (in)	1.972	Wet Unit Weight (pcf)	167.6
Area (in^2)	3.054	Dry Unit Weight (pcf)	164.8
Height/Diameter Ratio	2.361	Moisture Content[1] (%)	1.7

Moisture Condition As received, moist
Temperature (^oC) 20.2
Loading Rate (lbf/sec) 110
Peak Load (lbf) 29410

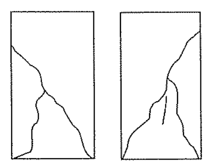
Date Tested 09-20-2007

Compressive Strength (psi) 9630

Compressive Strength (tsf) 693

Failure Type Cone and Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

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Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number AC-17   Depth (ft/elev) 6.45'-6.85'

Project Number LX2005125
Lab ID UCR-233
Date Received 10-23-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.689	Weight (lb)	1.393
Diameter (in)	1.979	Wet Unit Weight (pcf)	166.8
Area (in^2)	3.077	Dry Unit Weight (pcf)	165.8
Height/Diameter Ratio	2.369	Moisture Content[1] (%)	0.6

Moisture Condition As received, moist
Temperature (^oC) 23
Loading Rate (lbf/sec) 110
Peak Load (lbf) 24090

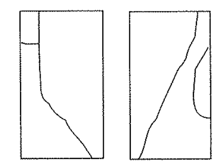
Date Tested 10-26-2007

Compressive Strength (psi) 7830

Compressive Strength (tsf) 564

Failure Type Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number AC-20   Depth (ft/elev) 8.45'-8.85'

Project Number LX2005125
Lab ID UCR-235
Date Received 10-23-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.454	Weight (lb)	1.322
Diameter (in)	1.987	Wet Unit Weight (pcf)	165.4
Area (in^2)	3.101	Dry Unit Weight (pcf)	164.5
Height/Diameter Ratio	2.241	Moisture Content[1] (%)	0.6

Moisture Condition As received, moist
Temperature (°C) 23
Loading Rate (lbf/sec) 110
Peak Load (lbf) 25930

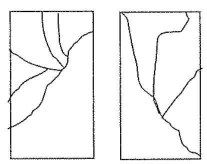
Date Tested 10-26-2007

Compressive Strength (psi) 8360

Compressive Strength (tsf) 602

Failure Type Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

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Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Shale, dark gray , soft
Hole Number AC-20   Depth (ft/elev) 29.8'-30.2'

Project Number LX2005125
Lab ID UCR-237
Date Received 10-23-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.472	Weight (lb)	1.270
Diameter (in)	1.975	Wet Unit Weight (pcf)	160.2
Area (in^2)	3.063	Dry Unit Weight (pcf)	153.2
Height/Diameter Ratio	2.265	Moisture Content[1] (%)	4.6

Moisture Condition As received, moist
Temperature (°C) 23
Loading Rate (lbf/sec) 20
Peak Load (lbf) 3400

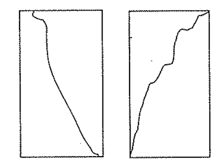
Date Tested 10-26-2007

Compressive Strength (psi) 1110

Compressive Strength (tsf) 80

Failure Type Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Shale, gray, soft
Hole Number AC-23 Depth (ft/elev) 23.85'-24.25'

Project Number LX2005125
Lab ID UCR-239
Date Received 10-23-2007

Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Height (in) 4.527
Diameter (in) 1.981
Area (in^2) 3.083
Height/Diameter Ratio 2.285

Specimen Dimensions
Weight (lb) 1.317
Wet Unit Weight (pcf) 163.0
Dry Unit Weight (pcf) 157.9
Moisture Content[1] (%) 3.2

Moisture Condition As received, moist
Temperature (°C) 23
Loading Rate (lbf/sec) 50
Peak Load (lbf) 12470

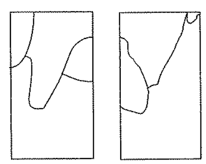
Date Tested 10-26-2007

Compressive Strength (psi) 4040

Compressive Strength (tsf) 291

Failure Type Cone and Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number AC-26 Depth (ft/elev) 10.5'-10.9'

Project Number LX2005125
Lab ID UCR-244
Date Received 10-23-2007

Dimensional Conformance
Side Planeness Pass
Perpendicularity Pass
End Planeness Pass

Height (in) 4.571
Diameter (in) 1.991
Area (in^2) 3.113
Height/Diameter Ratio 2.296

Specimen Dimensions
Weight (lb) 1.370
Wet Unit Weight (pcf) 166.4
Dry Unit Weight (pcf) 165.9
Moisture Content[1] (%) 0.3

Moisture Condition As received, moist
Temperature (°C) 23
Loading Rate (lbf/sec) 110
Peak Load (lbf) 37290

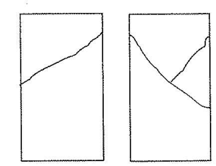
Date Tested 10-26-2007

Compressive Strength (psi) 11980

Compressive Strength (tsf) 862

Failure Type Cone and Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Shale, gray, soft
Lab ID UCR-245
Hole Number AC-26
Depth (ft/elev) 32.4'-32.8'
Date Received 10-23-2007

Dimensional Conformance

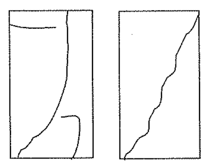
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.483	Weight (lb)	1.268
Diameter (in)	1.976	Wet Unit Weight (pcf)	159.3
Area (in^2)	3.068	Dry Unit Weight (pcf)	152.7
Height/Diameter Ratio	2.268	Moisture Content[1] (%)	4.3

Moisture Condition As received, moist
Date Tested 10-29-2007
Temperature (°C) 21
Loading Rate (lbf/sec) 20
Peak Load (lbf) 4180

Failure Sketches



Compressive Strength (psi) 1360

Compressive Strength (tsf) 98

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name I-265 Over Ohio River
Project Number LX2005125
Lithology Limestone, gray, moderately hard
Lab ID UCR-252
Hole Number AC-27
Depth (ft/elev) 25.9'-26.35'
Date Received 10-23-2007

Dimensional Conformance

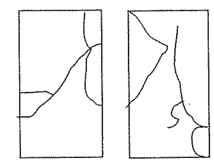
Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.485	Weight (lb)	1.267
Diameter (in)	1.955	Wet Unit Weight (pcf)	162.6
Area (in^2)	3.003	Dry Unit Weight (pcf)	157.4
Height/Diameter Ratio	2.293	Moisture Content[1] (%)	3.3

Moisture Condition As received, moist
Date Tested 10-29-2007
Temperature (°C) 21
Loading Rate (lbf/sec) 110
Peak Load (lbf) 12590

Failure Sketches



Compressive Strength (psi) 4190

Compressive Strength (tsf) 302

Failure Type Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Fuller Mossbarger Scott & May
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Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number AC-27 Depth (ft/elev) 4.20'-4.55'

Project Number LX2005125
Lab ID UCR-250
Date Received 10-23-2007

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.076
Diameter (in)	1.982
Area (in^2)	3.086
Height/Diameter Ratio	2.056
Weight (lb)	1.206
Wet Unit Weight (pcf)	165.7
Dry Unit Weight (pcf)	164.2
Moisture Content[1] (%)	0.9

Moisture Condition As received, moist
Temperature (°C) 21
Loading Rate (lbf/sec) 110
Peak Load (lbf) 22210

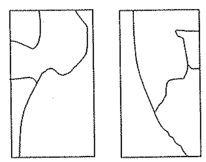
Compressive Strength (psi) 7200

Compressive Strength (tsf) 518

Failure Type Shear

Date Tested 10-26-2007

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Fuller Mossbarger Scott & May
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Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number B-1 / 189+60,CL Depth (ft/elev) 93.1'-93.5'

Project Number LX2005125
Lab ID UCR-7
Date Received 11-30-2005

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.636
Diameter (in)	1.992
Area (in^2)	3.115
Height/Diameter Ratio	2.328
Weight (lb)	1.375
Wet Unit Weight (pcf)	164.5
Dry Unit Weight (pcf)	163.1
Moisture Content[1] (%)	0.9

Moisture Condition As received, moist
Temperature (°C) 21
Loading Rate (lbf/sec) 110
Peak Load (lbf) 26790

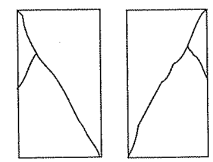
Compressive Strength (psi) 8600

Compressive Strength (tsf) 619

Failure Type Shear

Date Tested 12-07-2005

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient N/A

Corrected Compressive Strength (psi) N/A

Corrected Compressive Strength (tsf) N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

Fuller Mossbarger Scott & May
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Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name: I-265 Over Ohio River
Lithology: Limestone, gray, moderately hard
Hole Number: B-2 / 194+50, CL   Depth (ft/elev): 101.2'-101.6'

Project Number: LX2005125
Lab ID: UCR-11
Date Received: 11-30-2005

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	3.991	Weight (lb)	1.145
Diameter (in)	1.986	Wet Unit Weight (pcf)	160.1
Area (in^2)	3.097	Dry Unit Weight (pcf)	158.6
Height/Diameter Ratio	2.010	Moisture Content[1] (%)	1.0

Date Tested: 12-07-2005

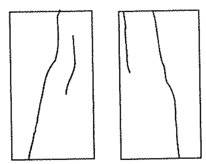
Moisture Condition: As received, moist
Temperature (°C): 21
Loading Rate (lbf/sec): 110
Peak Load (lbf): 12460

Compressive Strength (psi): 4020

Compressive Strength (tsf): 290

Failure Type: Shear

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient: N/A

Corrected Compressive Strength (psi): N/A

Corrected Compressive Strength (tsf): N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

File: LX2005125_UCR-11.xls Sheet: Report
Preparation Date: 2-2002
Revision Date: 7-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
KM 64-523-02

Project Name: I-265 Over Ohio River
Lithology: Limestone, gray, moderately hard
Hole Number: B-3 / 205+50, CL   Depth (ft/elev): 97.2'-97.6'

Project Number: LX2005125
Lab ID: UCR-16
Date Received: 11-30-2005

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

Specimen Dimensions

Height (in)	4.617	Weight (lb)	1.364
Diameter (in)	1.994	Wet Unit Weight (pcf)	163.5
Area (in^2)	3.122	Dry Unit Weight (pcf)	162.7
Height/Diameter Ratio	2.316	Moisture Content[1] (%)	0.5

Date Tested: 12-07-2005

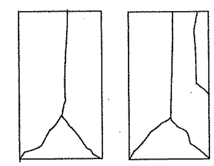
Moisture Condition: As received, moist
Temperature (°C): 21
Loading Rate (lbf/sec): 110
Peak Load (lbf): 30180

Compressive Strength (psi): 9670

Compressive Strength (tsf): 696

Failure Type: Cone and Split

Failure Sketches



Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient: N/A

Corrected Compressive Strength (psi): N/A

Corrected Compressive Strength (tsf): N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

File: LX2005125_UCR-16.xls Sheet: Report
Preparation Date: 2-2002
Revision Date: 7-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name: I-265 Over Ohio River
Lithology: Limestone, gray, moderately hard
Hole Number: B-4 / 210+30, CL   Depth (ft/elev): 28.1'-28.5'

Project Number: LX2005125
Lab ID: UCR-19
Date Received: 11-30-2005

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

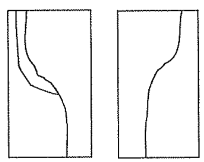
Specimen Dimensions

Height (in)	4.545	Weight (lb)	1.357
Diameter (in)	1.992	Wet Unit Weight (pcf)	165.5
Area (in^2)	3.118	Dry Unit Weight (pcf)	164.7
Height/Diameter Ratio	2.281	Moisture Content[1] (%)	0.5

Moisture Condition: As received, moist
Temperature (°C): 21
Loading Rate (lbf/sec): 110
Peak Load (lbf): 20850

Date Tested: 12-07-2005

Failure Sketches



Compressive Strength (psi): 6690

Compressive Strength (tsf): 482

Failure Type: Shear

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient: N/A

Corrected Compressive Strength (psi): N/A

Corrected Compressive Strength (tsf): N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

File: LX2005125_UCR-19.xls Sheet: Report
Preparation Date: 2-2002
Revision Date: 7-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Unconfined Compressive Strength Of Intact Rock Core

KM 64-523-02

Project Name: I-265 Over Ohio River
Lithology: Limestone, gray, moderately hard
Hole Number: B-4 / 210+30, CL   Depth (ft/elev): 49.9'-50.3'

Project Number: LX2005125
Lab ID: UCR-20
Date Received: 11-30-2005

Dimensional Conformance

Side Planeness	Pass
Perpendicularity	Pass
End Planeness	Pass

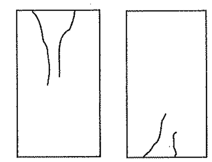
Specimen Dimensions

Height (in)	4.576	Weight (lb)	1.372
Diameter (in)	1.995	Wet Unit Weight (pcf)	165.7
Area (in^2)	3.127	Dry Unit Weight (pcf)	163.6
Height/Diameter Ratio	2.293	Moisture Content[1] (%)	1.3

Moisture Condition: As received, moist
Temperature (°C): 21
Loading Rate (lbf/sec): 110
Peak Load (lbf): 30070

Date Tested: 12-07-2005

Failure Sketches



Compressive Strength (psi): 9620

Compressive Strength (tsf): 692

Failure Type: Undetermined

Alternate Compressive Strength Calculation[2]
(Where Height/Diameter Ratio < 2)

Correction Coefficient: N/A

Corrected Compressive Strength (psi): N/A

Corrected Compressive Strength (tsf): N/A

Comments

[1] Post testing moisture content determination was performed as per ASTM D 2216-98, where as much of the whole specimen as available after compression testing was used in moisture content testing.

[2] The alternate compressive strength calculation is presented when the height to diameter ratio is less than 2, as per KM 64-523-02.

File: LX2005125_UCR-20.xls Sheet: Report
Preparation Date: 2-2002
Revision Date: 7-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

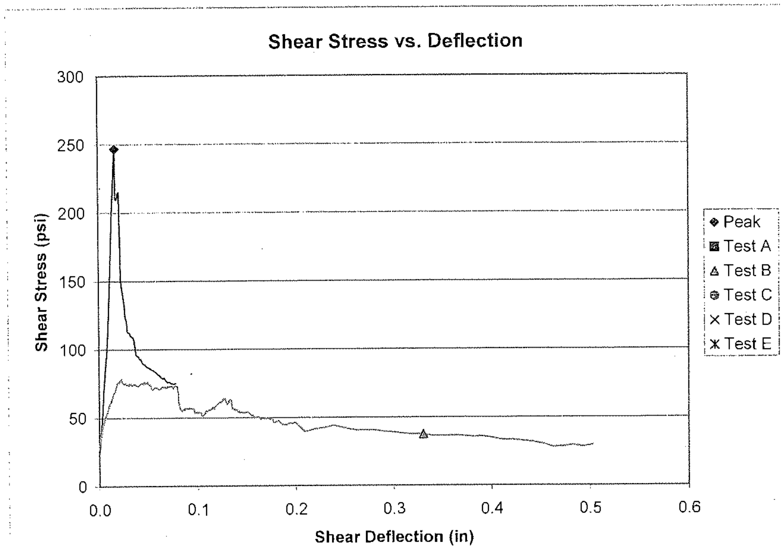
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-1 Depth (ft) 114.05'
Test Type Direct shear of intact specimen

Moisture Condition As received, moist
Roughness (JRC) 12

Project Number LX2005125
Lab ID DS-154
Date Received 09-28-2007

Diameter (in.) 1.977
Angle of Dip (deg.) 0.0
Area(in^2) 3.07

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	66.7	66.7	N/A	N/A	N/A
Peak Shear Stress (psi)	246.9				
Deflection at Peak (in)	0.0175				
Post Peak Stress (psi)	N/A	37.5	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.3307	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-154.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

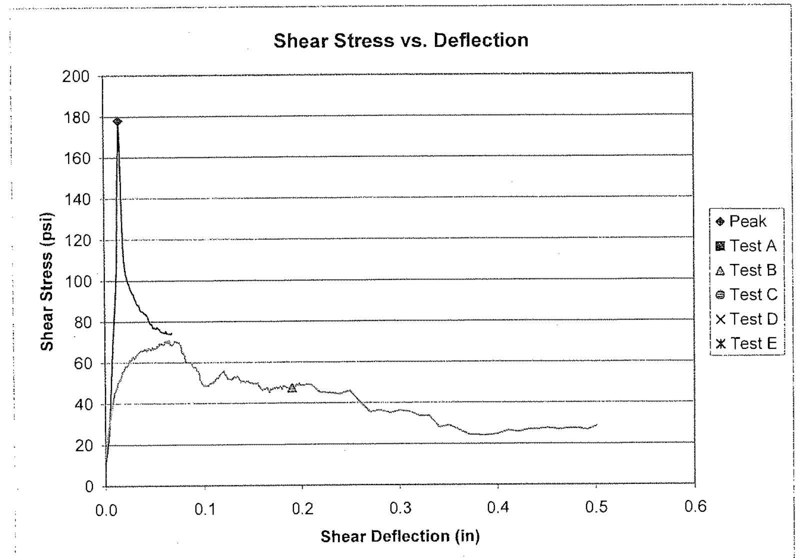
Project Name I-265 Over Ohio River
Lithology Shale, dark gray, moderately hard, limestone layer
Hole Number AC-1 Depth (ft) 120.1'
Test Type Direct shear of intact specimen

Moisture Condition As received, moist
Roughness (JRC) 12

Project Number LX2005125
Lab ID DS-156
Date Received 09-28-2007

Diameter (in.) 1.978
Angle of Dip (deg.) 0.0
Area(in^2) 3.07

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	73.5	73.5	N/A	N/A	N/A
Peak Shear Stress (psi)	178.0				
Deflection at Peak (in)	0.0150				
Post Peak Stress (psi)	N/A	47.5	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.1905	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-156.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Direct Shear Strength of Rock
RTH 203 - 80

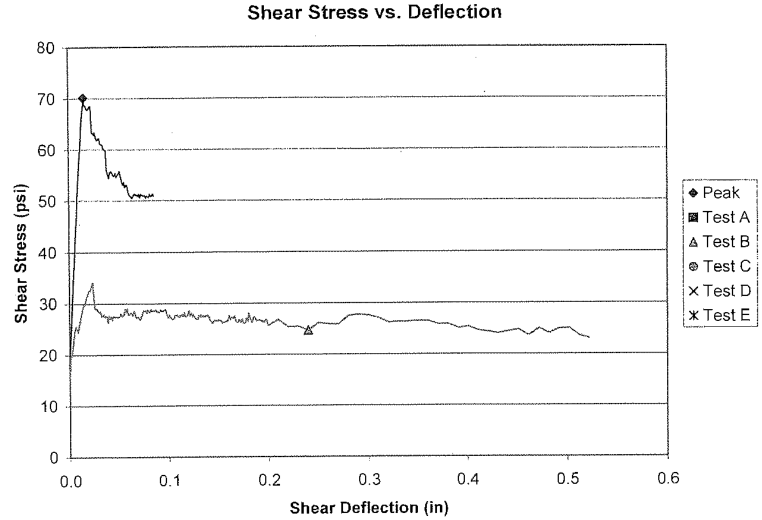
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard , shale layers
Hole Number AC-2 Depth (ft) 103.1'
Test Type Direct shear of intact specimen

Moisture Condition As received, moist
Roughness (JRC) 10

Project Number LX2005125
Lab ID DS-170
Date Received 09-28-2007

Diameter (in.) 1.972
Angle of Dip (deg.) 0.0
Area(in^2) 3.05

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	51.3	51.3	N/A	N/A	N/A
Peak Shear Stress (psi)	70.1				
Deflection at Peak (in)	0.0150				
Post Peak Stress (psi)	N/A	24.7	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2401	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-170.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Direct Shear Strength of Rock
RTH 203 - 80

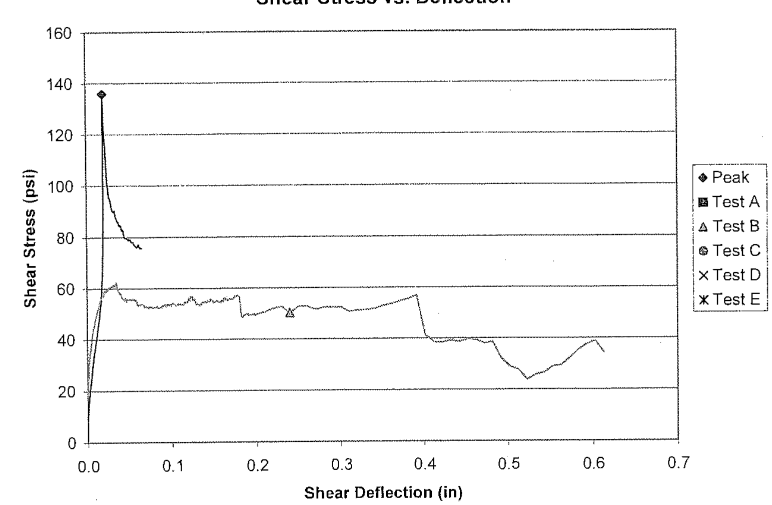
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale layers
Hole Number AC-2 Depth (ft) 113.55'
Test Type Direct shear of intact specimen

Moisture Condition As received, moist
Roughness (JRC) 14

Project Number LX2005125
Lab ID DS-172
Date Received 09-28-2007

Diameter (in.) 1.980
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	63.1	63.1	N/A	N/A	N/A
Peak Shear Stress (psi)	135.9				
Deflection at Peak (in)	0.0200				
Post Peak Stress (psi)	N/A	50.2	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2401	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-172.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

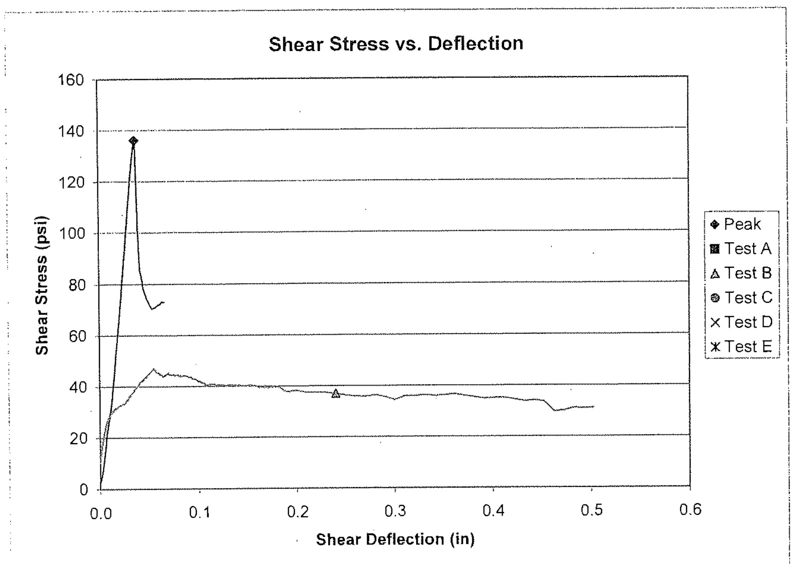
FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale layers
Hole Number AC-3 Depth (ft) 102.0'
Test Type Direct shear of intact specimen
Moisture Condition As received, moist
Roughness (JRC) 8

Project Number LX2005125
Lab ID DS-188
Date Received 09-28-2007
Diameter (in.) 3.325
Angle of Dip (deg.) 0.0
Area(in^2) 8.68

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	50.5	50.5	N/A	N/A	N/A
Peak Shear Stress (psi)	136.1				
Deflection at Peak (in)	0.0364				
Post Peak Stress (psi)	N/A	37.0	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2401	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-188.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

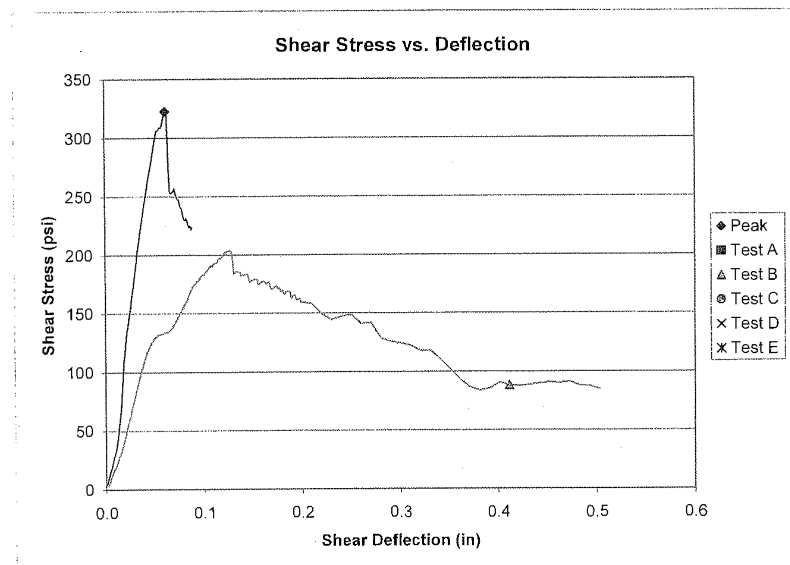
FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale layers
Hole Number AC-3 Depth (ft) 114.9'
Test Type Direct shear of intact specimen
Moisture Condition As received, moist
Roughness (JRC) 14

Project Number LX2005125
Lab ID DS-192
Date Received 09-28-2007
Diameter (in.) 3.327
Angle of Dip (deg.) 0.0
Area(in^2) 8.69

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	64.9	64.9	N/A	N/A	N/A
Peak Shear Stress (psi)	322.5				
Deflection at Peak (in)	0.0613				
Post Peak Stress (psi)	N/A	88.4	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.4118	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-192.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Direct Shear Strength of Rock
RTH 203 - 80

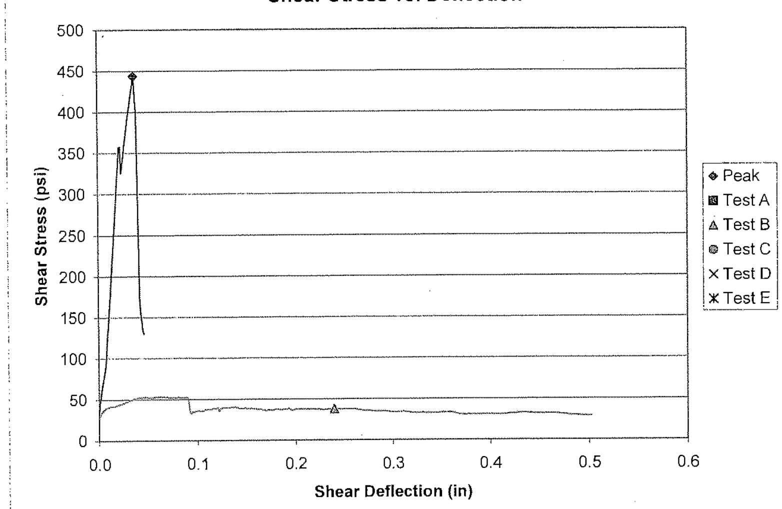
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-4/189+81.55, 91.9 Lt. Depth (ft) 102.15'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-29
Date Received 08-10-2007

Moisture Condition As received, dry
Roughness (JRC) 20

Diameter (in.) 1.982
Angle of Dip (deg.) 16.8
Area(in^2) 3.22

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	43.3	43.3	N/A	N/A	N/A
Peak Shear Stress (psi)	443.5				
Deflection at Peak (in)	0.0364				
Post Peak Stress (psi)	N/A	37.8	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2399	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Direct Shear Strength of Rock
RTH 203 - 80

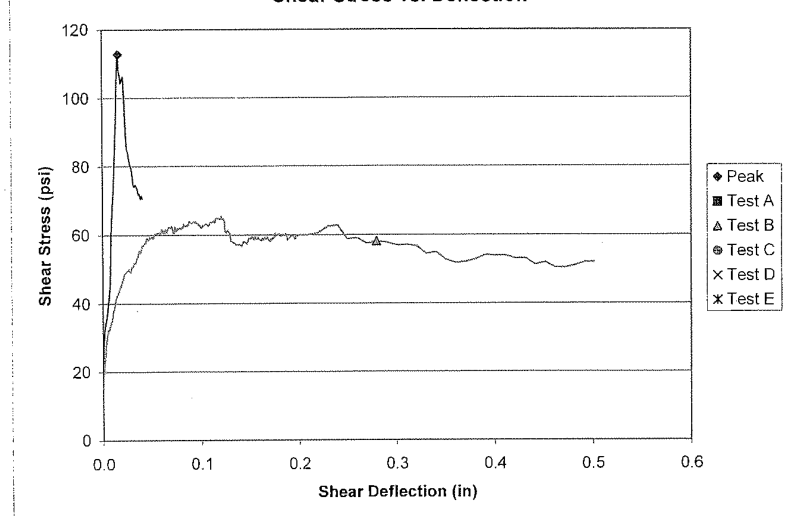
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard
Hole Number AC-4/189+81.55, 91.9 Lt. Depth (ft) 116.65'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-31
Date Received 08-10-2007

Moisture Condition As received, dry
Roughness (JRC) 15

Diameter (in.) 1.979
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	59.4	59.4	N/A	N/A	N/A
Peak Shear Stress (psi)	112.8				
Deflection at Peak (in)	0.0162				
Post Peak Stress (psi)	N/A	58.1	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2800	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

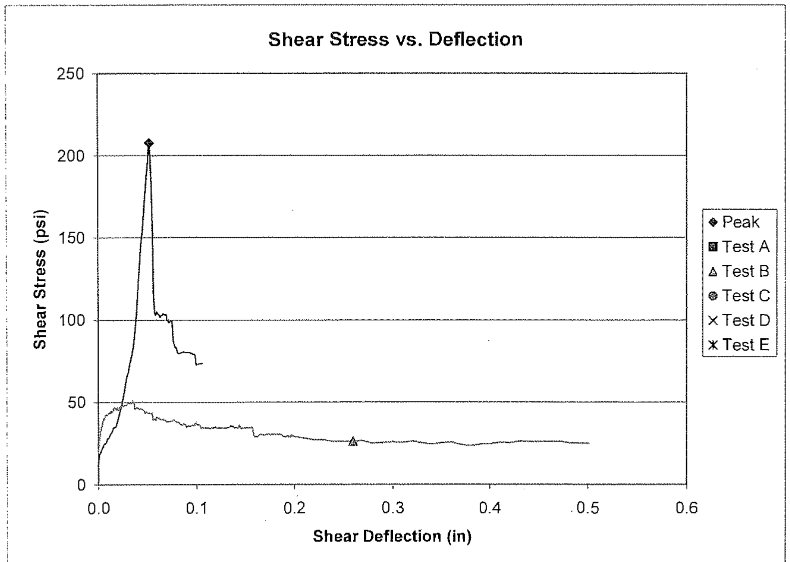
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-4/189+81.55, 91.9 Lt. Depth (ft) 123.6'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-33
Date Received 08-10-2007

Diameter (in.) 1.985
Angle of Dip (deg.) 0.0
Area(in^2) 3.09

Moisture Condition As received, moist
Roughness (JRC) 14

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	67.0	67.0	N/A	N/A	N/A
Peak Shear Stress (psi)	207.8				
Deflection at Peak (in)	0.0523				
Post Peak Stress (psi)	N/A	26.2	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2598	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

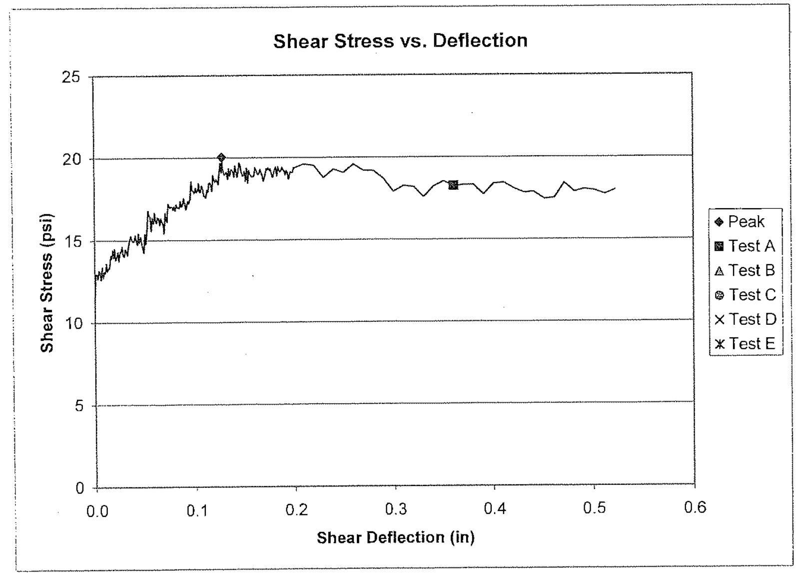
Project Name I-265 Over Ohio River
Lithology Shale, dark gray, very soft
Hole Number AC-6/193+51.7, 0.28' Rt. Depth (ft) 97.4'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-41
Date Received 08-10-2007

Diameter (in.) 1.985
Angle of Dip (deg.) 0.0
Area(in^2) 3.09

Moisture Condition As received, moist
Roughness (JRC) 8

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	31.4	N/A	N/A	N/A	N/A
Peak Shear Stress (psi)	20.0				
Deflection at Peak (in)	0.1277				
Post Peak Stress (psi)	18.3	N/A	N/A	N/A	N/A
Deflection at Residual (in)	0.3608	N/A	N/A	N/A	N/A



Comments

Direct Shear Strength of Rock
RTH 203 - 80

FMSM ENGINEERS

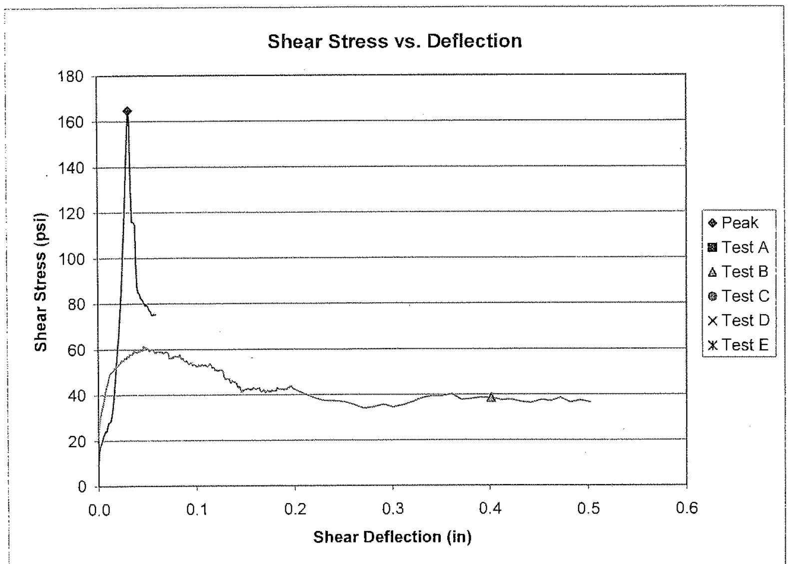
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-6/193+51.7, 0.28' Rt. Depth (ft) 110.7'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 6

Project Number LX2005125
Lab ID DS-44
Date Received 08-10-2007

Diameter (in.) 1.978
Angle of Dip (deg.) 0.0
Area(in^2) 3.07

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	45.8	45.8	N/A	N/A	N/A
Peak Shear Stress (psi)	164.8				
Deflection at Peak (in)	0.0317				
Post Peak Stress (psi)	N/A	38.6	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.4016	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Direct Shear Strength of Rock
RTH 203 - 80

FMSM ENGINEERS

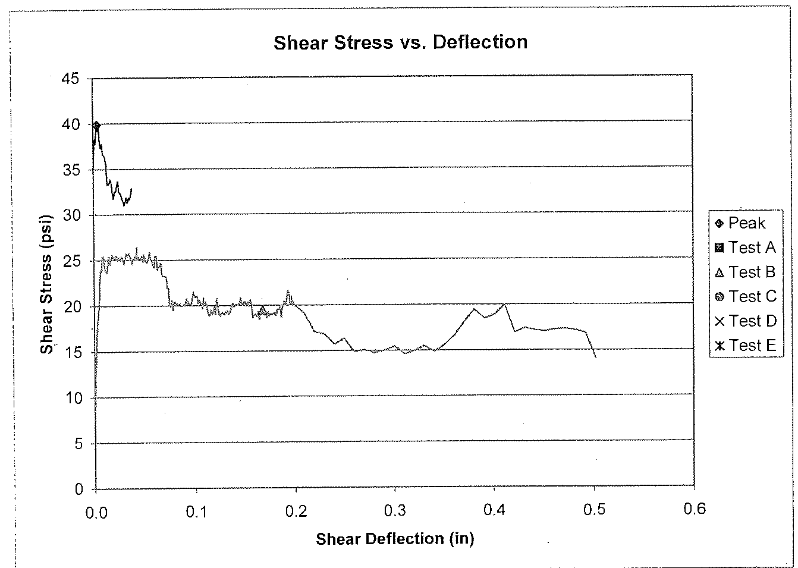
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-7 193+95, 68' Lt. Depth (ft) 103.15'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 8

Project Number LX2005125
Lab ID DS-56
Date Received 08-10-2007

Diameter (in.) 1.983
Angle of Dip (deg.) 0.0
Area(in^2) 3.09

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	36.2	36.2	N/A	N/A	N/A
Peak Shear Stress (psi)	39.9				
Deflection at Peak (in)	0.0039				
Post Peak Stress (psi)	N/A	19.5	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.1677	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

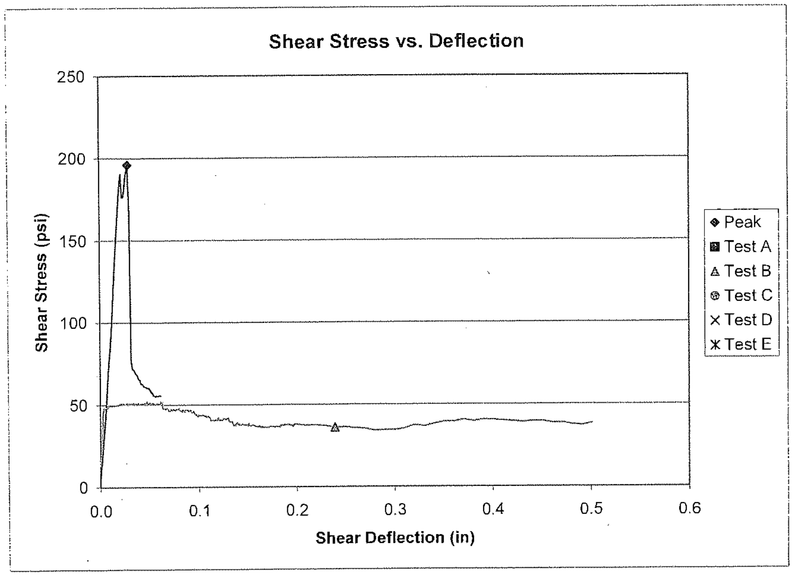
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-7 193+95, 68' Lt. Depth (ft) 115.8'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 2

Project Number LX2005125
Lab ID DS-60
Date Received 08-10-2007

Diameter (in.) 1.979
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	50.3	50.3	N/A	N/A	N/A
Peak Shear Stress (psi)	196.0				
Deflection at Peak (in)	0.0288				
Post Peak Stress (psi)	N/A	36.1	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2397	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

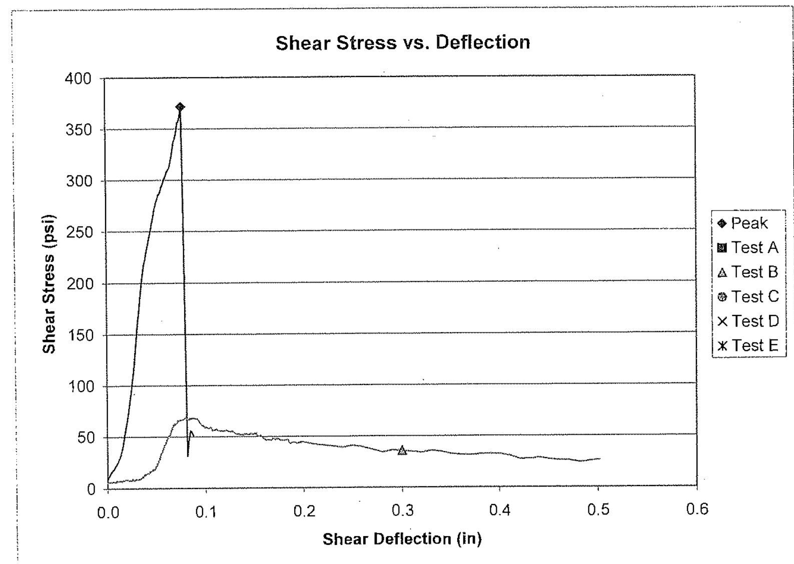
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-8/193+95, 1.22 Lt. Depth (ft) 98.55'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 11

Project Number LX2005125
Lab ID DS-67
Date Received 08-10-2007

Diameter (in.) 1.980
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	33.0	33.0	N/A	N/A	N/A
Peak Shear Stress (psi)	371.8				
Deflection at Peak (in)	0.0767				
Post Peak Stress (psi)	N/A	35.7	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.3002	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

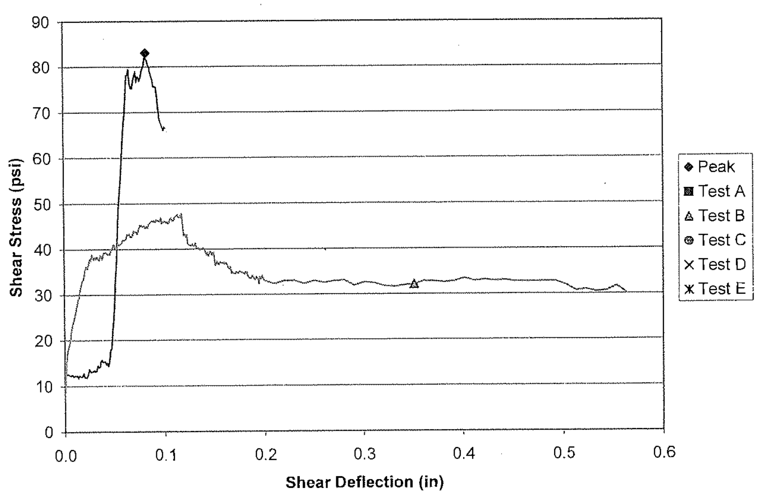
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-8/193+95, 1.22'Lt. Depth (ft) 113.75'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 16

Project Number LX2005125
Lab ID DS-71
Date Received 08-10-2007

Diameter (in.) 1.971
Angle of Dip (deg.) 0.0
Area(in^2) 3.05

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	49.8	49.8	N/A	N/A	N/A
Peak Shear Stress (psi)	83.1				
Deflection at Peak (in)	0.0824				
Post Peak Stress (psi)	N/A	32.2	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.3515	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-71.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

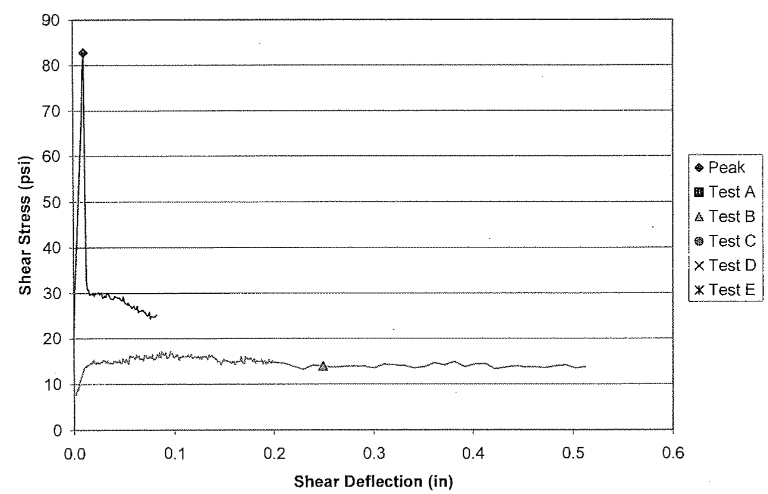
Project Name I-265 Over Ohio River
Lithology Shale, dark gray, soft
Hole Number AC-9/193+95, 70' Rt. Depth (ft) 100.9'
Test Type Direct shear of intact specimen

Moisture Condition As received, moist
Roughness (JRC) 9

Project Number LX2005125
Lab ID DS-79
Date Received 08-10-2007

Diameter (in.) 1.997
Angle of Dip (deg.) 0.0
Area(in^2) 3.13

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	32.9	32.9	N/A	N/A	N/A
Peak Shear Stress (psi)	82.8				
Deflection at Peak (in)	0.0107				
Post Peak Stress (psi)	N/A	14.0	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2499	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-79.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Direct Shear Strength of Rock
RTH 203 - 80

Fuller Mossbarger Scott & May — FMSM ENGINEERS

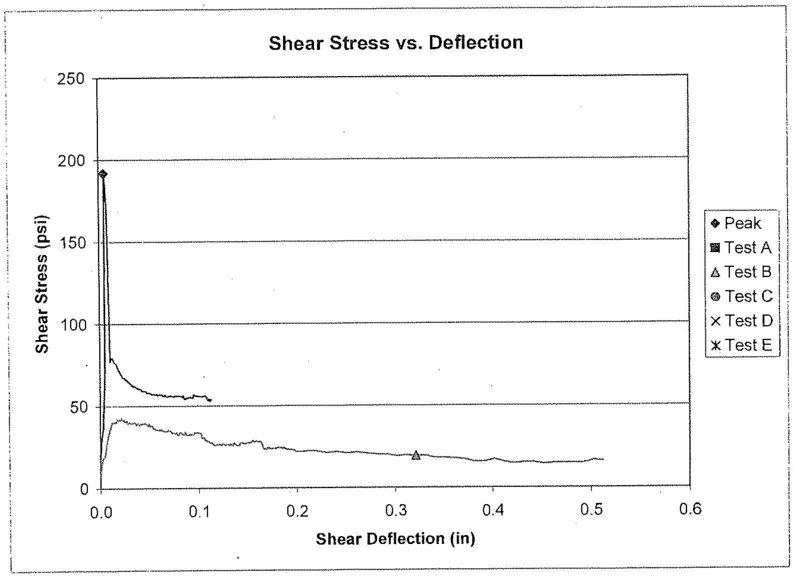
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-9/193+95, 70' Rt. Depth (ft) 115.95'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 12

Project Number LX2005125
Lab ID DS-82
Date Received 08-10-2007

Diameter (in.) 1.976
Angle of Dip (deg.) 0.0
Area(in^2) 3.07

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	49.6	49.6	N/A	N/A	N/A
Peak Shear Stress (psi)	191.9				
Deflection at Peak (in)	0.0051				
Post Peak Stress (psi)	N/A	19.3	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.3213	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-82.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Direct Shear Strength of Rock
RTH 203 - 80

Fuller Mossbarger Scott & May — FMSM ENGINEERS

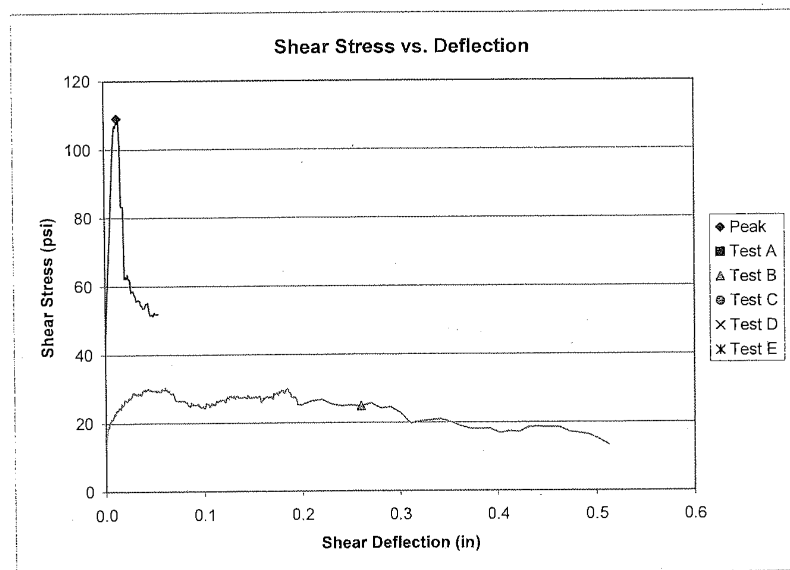
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-10/205+98, 70' Lt. Depth (ft) 101.35'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 14

Project Number LX2005125
Lab ID DS-92
Date Received 08-10-2007

Diameter (in.) 1.975
Angle of Dip (deg.) 0.0
Area(in^2) 3.06

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	36.3	36.3	N/A	N/A	N/A
Peak Shear Stress (psi)	109.1				
Deflection at Peak (in)	0.0131				
Post Peak Stress (psi)	N/A	24.9	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2602	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-92.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Direct Shear Strength of Rock
RTH 203 - 80

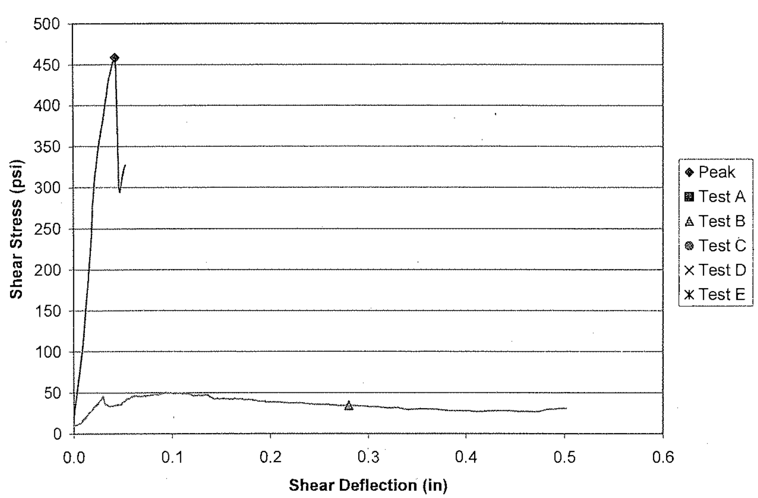
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-10/205+98, 70' Lt. Depth (ft) 117.45'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-97
Date Received 08-10-2007

Diameter (in.) 1.980
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

Moisture Condition As received, dry
Roughness (JRC) 12

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	54.2	54.2	N/A	N/A	N/A
Peak Shear Stress (psi)	458.6				
Deflection at Peak (in)	0.0428				
Post Peak Stress (psi)	N/A	34.1	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2802	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Direct Shear Strength of Rock
RTH 203 - 80

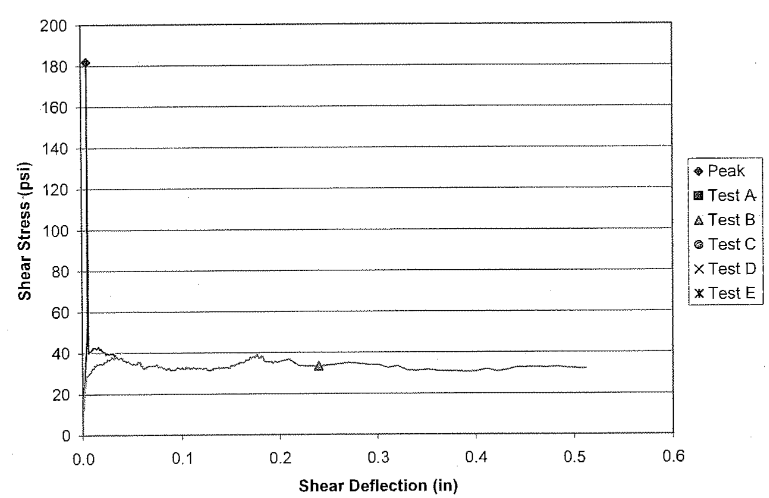
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-11/205+95,1.3 LT. Depth (ft) 89.75'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-104
Date Received 08-10-2007

Diameter (in.) 1.978
Angle of Dip (deg.) 0.0
Area(in^2) 3.07

Moisture Condition As received, dry
Roughness (JRC) 11

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	27.0	27.0	N/A	N/A	N/A
Peak Shear Stress (psi)	181.8				
Deflection at Peak (in)	0.0057				
Post Peak Stress (psi)	N/A	33.4	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2401	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Direct Shear Strength of Rock
RTH 203 - 80

FMSM ENGINEERS

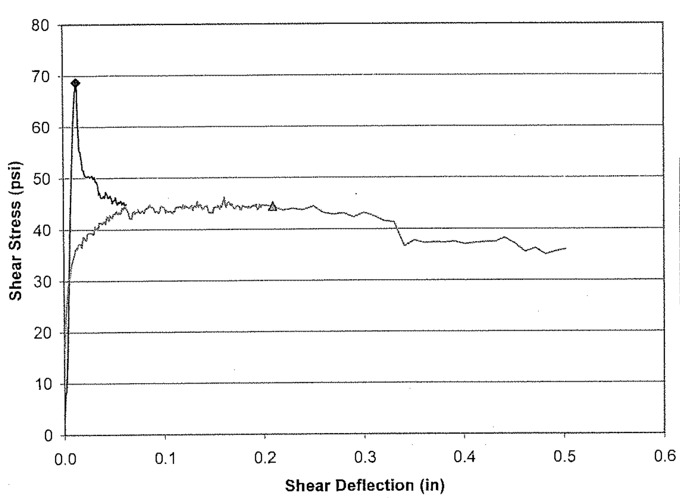
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-11/205+95,1.3 LT. Depth (ft) 103.8'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-107
Date Received 08-10-2007

Diameter (in.) 1.987
Angle of Dip (deg.) 0.0
Area(in^2) 3.10

Moisture Condition As received, dry
Roughness (JRC) 9

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	42.3	42.3	N/A	N/A	N/A
Peak Shear Stress (psi)	68.7				
Deflection at Peak (in)	0.0132				
Post Peak Stress (psi)	N/A	44.4	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2095	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Direct Shear Strength of Rock
RTH 203 - 80

FMSM ENGINEERS

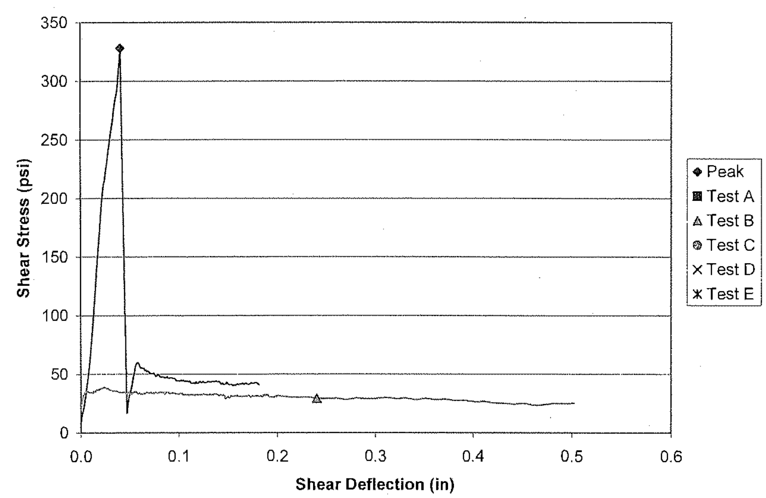
Project Name I-265 Over Ohio River
Lithology Limestone, gray, moderately hard, shale seams
Hole Number AC-12/205+94, 71 RT. Depth (ft) 94.0'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-116
Date Received 08-10-2007

Diameter (in.) 1.982
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

Moisture Condition As received, dry
Roughness (JRC) 11

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	31.2	31.2	N/A	N/A	N/A
Peak Shear Stress (psi)	328.0				
Deflection at Peak (in)	0.0406				
Post Peak Stress (psi)	N/A	29.4	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2404	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

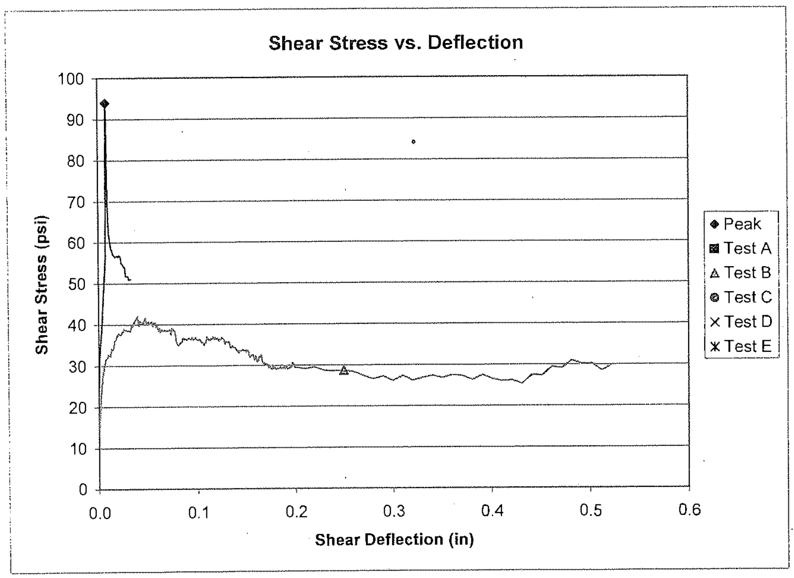
Project Name I-265 Over Ohio River
Lithology Limestone, dark gray, moderately hard, shale seams
Hole Number AC-12/205+94, 71 RT. Depth (ft) 109.3'
Test Type Direct shear of intact specimen

Moisture Condition As received, damp
Roughness (JRC) 8

Project Number LX2005125
Lab ID DS-121
Date Received 08-10-2007

Diameter (in.) 1.980
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	47.9	47.9	N/A	N/A	N/A
Peak Shear Stress (psi)	93.9				
Deflection at Peak (in)	0.0081				
Post Peak Stress (psi)	N/A	28.7	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2496	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

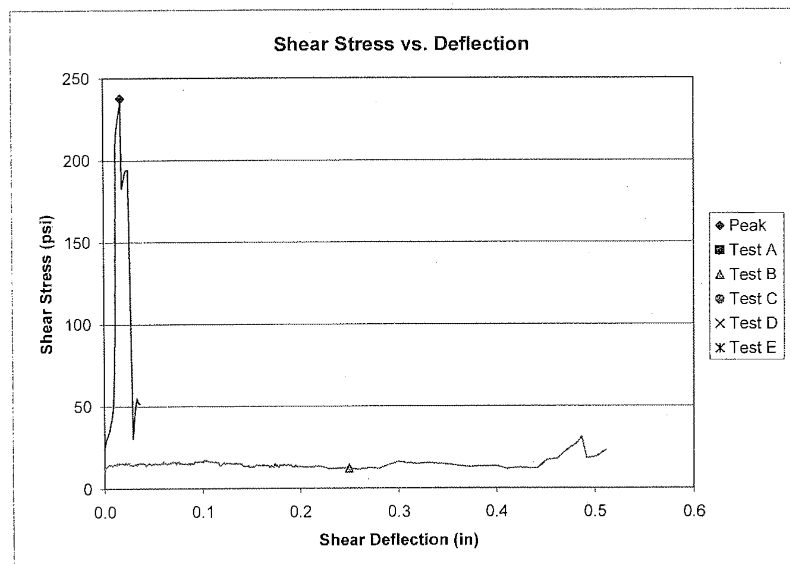
Project Name I-265 Over Ohio River
Lithology Limestone, dark gray, moderately hard, shale seams
Hole Number AC-13/206+50, 0.02 LT. Depth (ft) 95.15'
Test Type Direct shear of intact specimen

Moisture Condition As received, dry
Roughness (JRC) 10

Project Number LX2005125
Lab ID DS-129
Date Received 08-10-2007

Diameter (in.) 1.978
Angle of Dip (deg.) 0.0
Area(in^2) 3.07

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	32.9	32.9	N/A	N/A	N/A
Peak Shear Stress (psi)	237.6				
Deflection at Peak (in)	0.0176				
Post Peak Stress (psi)	N/A	12.2	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2494	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

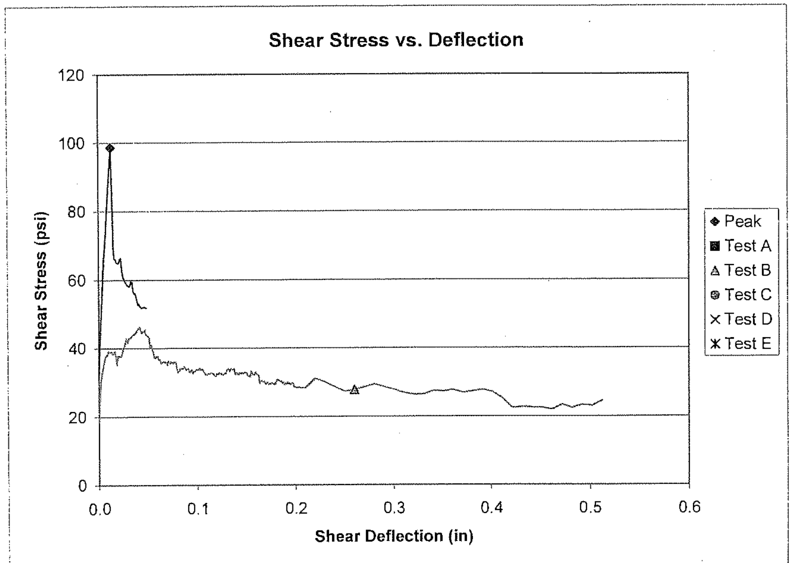
Fuller Mossbarger Scott & May
FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

Project Name I-265 Over Ohio River
Lithology Limestone, dark gray, moderately hard, shale seams
Hole Number AC-13/206+50, 0.02 LT. Depth (ft) 104.05'
Test Type Direct shear of intact specimen
Moisture Condition As received, damp
Roughness (JRC) 8

Project Number LX2005125
Lab ID DS-132
Date Received 08-10-2007
Diameter (in.) 1.975
Angle of Dip (deg.) 0.0
Area(in^2) 3.06

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	42.9	42.9	N/A	N/A	N/A
Peak Shear Stress (psi)	98.6				
Deflection at Peak (in)	0.0125				
Post Peak Stress (psi)	N/A	27.8	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2602	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

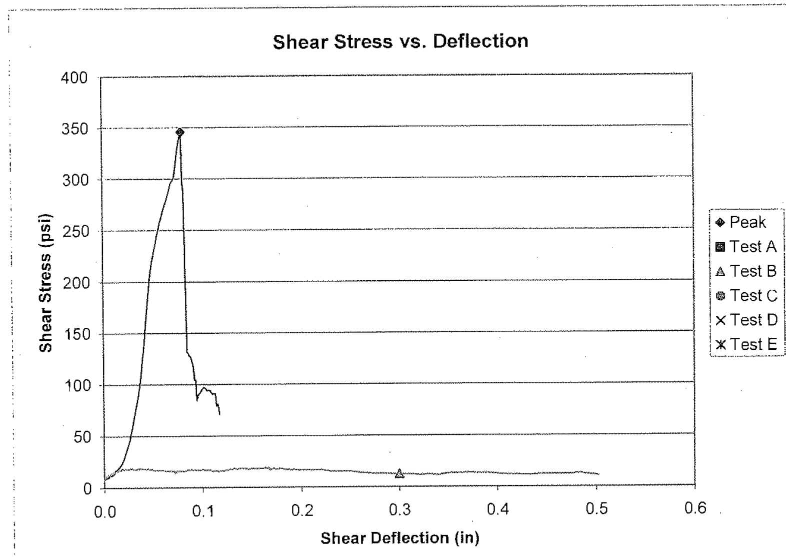
Fuller Mossbarger Scott & May
FMSM ENGINEERS

Direct Shear Strength of Rock
RTH 203 - 80

Project Name I-265 Over Ohio River
Lithology Shale, gray, very soft
Hole Number AC-14 Depth (ft) 30.5'
Test Type Direct shear of intact specimen
Moisture Condition As received, moist
Roughness (JRC) 6

Project Number LX2005125
Lab ID DS-229
Date Received 10-23-2007
Diameter (in.) 1.980
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	30.3	30.3	N/A	N/A	N/A
Peak Shear Stress (psi)	345.7				
Deflection at Peak (in)	0.0798				
Post Peak Stress (psi)	N/A	12.5	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.3004	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower. Multiple failure planes.

Direct Shear Strength of Rock
RTH 203 - 80

FMSM ENGINEERS

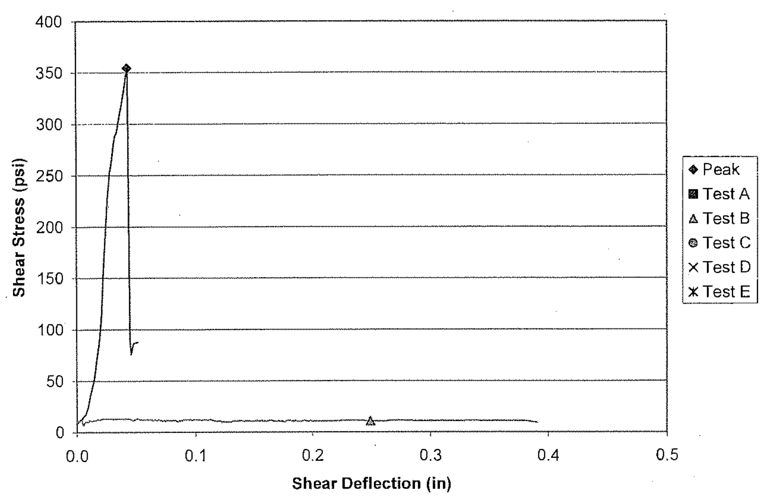
Project Name I-265 Over Ohio River
Lithology Shale, dark gray, moderately hard, limestone seams
Hole Number AC-15/210+20, 37.3 RT. Depth (ft) 49.9'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-142
Date Received 08-10-2007

Diameter (in.) 1.982
Angle of Dip (deg.) 0.0
Area(in^2) 3.08

Moisture Condition As received, dry
Roughness (JRC) 10

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	35.3	35.3	N/A	N/A	N/A
Peak Shear Stress (psi)	354.6				
Deflection at Peak (in)	0.0430				
Post Peak Stress (psi)	N/A	10.9	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2496	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Direct Shear Strength of Rock
RTH 203 - 80

FMSM ENGINEERS

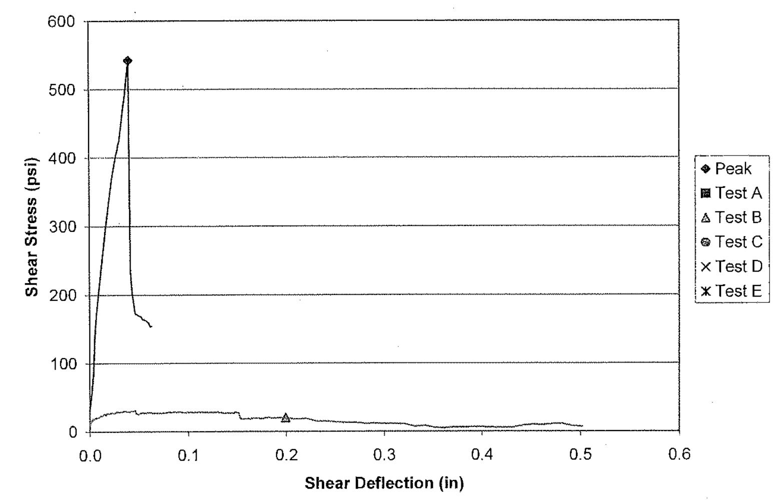
Project Name I-265 Over Ohio River
Lithology Limestone, dark gray, moderately hard, shale seams
Hole Number AC-15/210+20, 37.3 RT. Depth (ft) 59.7'
Test Type Direct shear of intact specimen

Project Number LX2005125
Lab ID DS-144
Date Received 08-10-2007

Diameter (in.) 1.983
Angle of Dip (deg.) 0.0
Area(in^2) 3.09

Moisture Condition As received, damp
Roughness (JRC) 4

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	46.3	46.3	N/A	N/A	N/A
Peak Shear Stress (psi)	542.5				
Deflection at Peak (in)	0.0399				
Post Peak Stress (psi)	N/A	20.2	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.1996	N/A	N/A	N/A



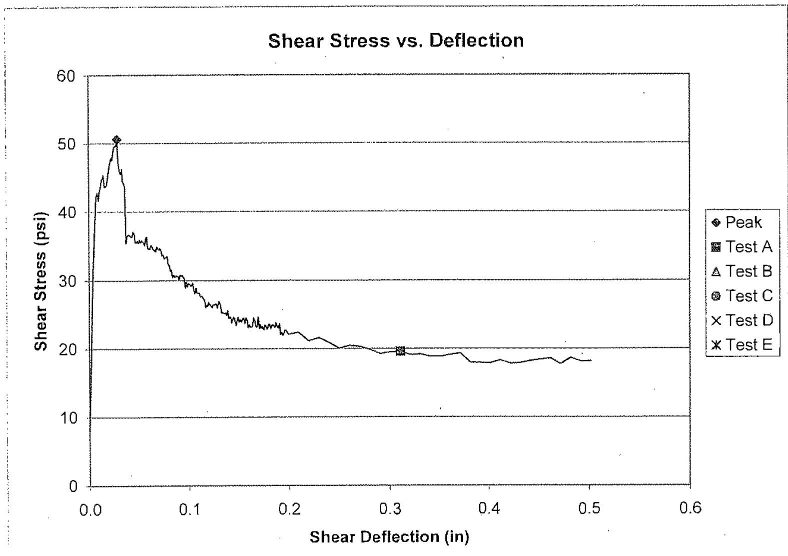
Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

Direct Shear Strength of Rock
RTH 203 - 80

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Lithology	Limestone, gray, moderately hard, shale seams	Lab ID	DS-236
Hole Number	AC-20 Depth (ft) 28.2'	Date Received	10-23-2007
Test Type	Direct shear of intact specimen	Diameter (in.)	1.974
Moisture Condition	As received, moist	Angle of Dip (deg.)	0.0
Roughness (JRC)	3	Area(in^2)	3.06

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	30.3	N/A	N/A	N/A	N/A
Peak Shear Stress (psi)	50.6				
Deflection at Peak (in)	0.0287				
Post Peak Stress (psi)	19.5	N/A	N/A	N/A	N/A
Deflection at Residual (in)	0.3112	N/A	N/A	N/A	N/A



Comments

File: LX2005125_DS-236.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

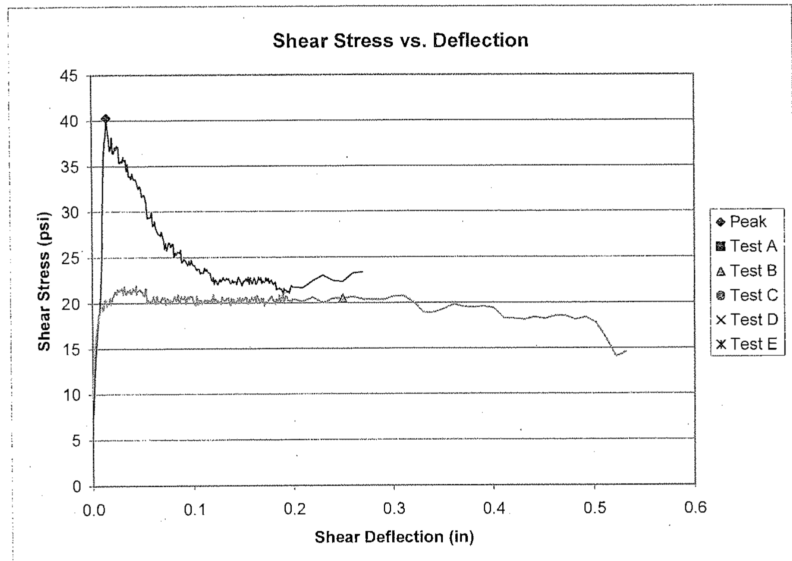
Laboratory Document
Prepared By: JW
Approved By: TLK

Direct Shear Strength of Rock
RTH 203 - 80

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Project Name	I-265 Over Ohio River	Project Number	LX2005125
Lithology	Shale, gray, soft	Lab ID	DS-246
Hole Number	AC-26 Depth (ft) 27.9'	Date Received	10-23-2007
Test Type	Direct shear of intact specimen	Diameter (in.)	1.970
Moisture Condition	As received, moist	Angle of Dip (deg.)	0.0
Roughness (JRC)	7	Area(in^2)	3.05

	Test A	Test B	Test C	Test D	Test E
Normal Load (psi)	29.2	29.2	N/A	N/A	N/A
Peak Shear Stress (psi)	40.3				
Deflection at Peak (in)	0.0145				
Post Peak Stress (psi)	N/A	20.5	N/A	N/A	N/A
Deflection at Residual (in)	N/A	0.2498	N/A	N/A	N/A



Comments The Test A (intact) shear plane extended slightly into the Hydro-stone encasement. Accordingly, Test B (post peak) was performed wherein the Hydro-Stone was chipped away from the shear plane. The resulting post peak stresses differ with Test B lower.

File: LX2005125_DS-246.xls Sheet: Report
Preparation Date: 12-1999
Revision Date: 2-2002

Fuller, Mossbarger, Scott and May Engineers, Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

FMSM ENGINEERS

Slake Durability Index
KM 64 - 513

Project Name I-265 Bridge over the Ohio River

Project Number LX2005125

	Prepared	1st Dry	Initial Weight	1st Run	2nd Dry	2nd Run	3rd Dry	Final Weight
Date	10/8/2007	10/8/2007	10/9/2007	10/9/2007	10/9/2007	10/10/2007	10/10/2007	10/11/2007
By	RM	RM	SB	RC	RC	RC	RC	SB

Lab ID	Sample No./ Hole No.	Sample Depth (ft)	Sample Description	Fragment Description	Tare ID	Tare Weight	Initial Wt. + Tare	Final Wt. + Tare	SDI
195	AC-1	106.7'-107.2'	Limestone w/ shale layers	pile of chips	JH-33	283.53	633.20	555.16	77.7
196	AC-1	117.2'-117.9'	Limestone w/ shale layers	50 % intact	D-22	284.17	760.88	648.03	76.3
197	AC-1	128.1'-128.7'	Limestone w/ shale layers	50 % intact	3949	292.04	789.17	768.25	95.8
171	AC-2	104.8'-105.4'	Limestone w/ shale layers	50 % intact	HC-89	296.08	828.58	733.62	82.2
174	AC-2	120.7'-121.1'	Limestone w/ shale layers	75 % intact	DEL	297.03	777.13	754.44	95.3
176	AC-2	129.9'-130.9'	Limestone w/ shale layers	70 % intact	1921	290.71	833.23	784.80	91.1
189	AC-3	104.7'-105.1'	Limestone w/ shale layers	75 % intact	D-1	299.87	827.21	790.51	93.0
190	AC-3	106.7'-107.9'	Limestone w/ shale layers	50 % intact	ZX-1	288.97	863.75	499.41	36.6
191	AC-3	112.5'-112.9'	Limestone w/ shale layers	50 % intact	Boss	294.76	844.53	708.35	75.2

Checked By G

Reviewed By G

FMSM ENGINEERS

Slake Durability Index
ASTM D 4644

Project Name I-265 OVER Ohio River

Project Number LX2005125

	Prepared	1st Dry	Initial Weight	1st Run	2nd Dry	2nd Run	3rd Dry	Final Weight
Date	8/22/2007	8/22/2007	8/24/2007	8/24/2007	8/24/2007	8/27/2007	8/27/2007	8/29/2007
By	RM	RM	TMK	TMK	TMK	RM	RM	KDG

Lab ID	Sample No./ Hole No.	Sample Depth (ft)	Sample Description	Fragment Description	Tare ID	Tare Weight	Initial Wt. + Tare	Final Wt. + Tare	SDI
30	AC-4/189+81.55, 91.9 Lt.	102.5'-102.9'	gray Limestone w/shale	95% intact	CL17	283.13	933.40	906.86	95.9
40	AC-6/193+51.7, 0.28' Rt.	94.3'-95.0'	gray Limestone w/shale	100% intact	D-41	309.89	870.50	830.89	92.9
43	AC-6/193+51.7, 0.28' Rt.	105.5'-106.3'	gray Limestone w/shale	95% intact	MH1	288.96	776.48	746.13	93.8
46	AC-6/193+51.7, 0.28' Rt.	117.2'-118.0'	gray Limestone w/shale	100% intact	UK19	293.72	768.90	744.03	94.8
53	AC-7 193+95, 68' Lt.	97.9'-98.9'	gray Limestone w/shale	pile of chips	DDW32	294.59	877.58	595.16	51.6
55	AC-7 193+95, 68' Lt.	102.1'-102.8'	gray Limestone w/shale	80% intact	KH20	297.32	842.28	713.24	76.3
57	AC-7 193+95, 68' Lt.	108.5'-109.1'	gray Limestone w/shale	80% intact	SMD8	293.22	771.98	680.76	80.9
66	AC-8/193+95, 1.22'Lt.	95.7'-96.1'	gray Limestone w/shale	large and small chips	56	292.95	787.25	573.65	56.8
70	AC-8/193+95, 1.22'Lt.	106.9'-107.5'	gray Limestone w/shale	60% intact	P30	284.01	814.44	666.62	72.1
72	AC-8/193+95, 1.22'Lt.	114.8'-115.3'	gray Limestone w/shale	pile of chips	D43	309.30	800.77	339.78	6.2
78	AC-9/193+95, 70' Rt.	99.4'-100.2'	gray Limestone w/shale	50% intact	D26	284.18	858.43	632.16	60.6
81	AC-9/193+95, 70' Rt.	108.2'-108.9'	gray Limestone w/shale	95% intact	UK10	297.11	840.07	773.55	87.7
84	AC-9/193+95, 70' Rt.	117.5'-117.9'	gray Limestone w/shale	pile of chips	JW32	282.87	713.62	310.32	6.4
90	AC-10/205+98, 70' Lt.	94.7'-95.1'	gray Limestone w/shale	pile of chips	D-14	293.62	832.82	517.44	41.5
93	AC-10/205+98, 70' Lt.	102.7'-103.4'	gray Limestone w/shale	95% intact	C-2	309.00	855.68	763.26	83.1
95	AC-10/205+98, 70' Lt.	108.0'-108.4'	gray Limestone w/shale	70% intact	KC-16	301.07	831.41	734.46	81.7

Checked By KDG

Reviewed By G

FMSM ENGINEERS

Slake Durability Index
ASTM D 4644

Project Name I-265 OVER Ohio River

Project Number LX2005125

	Prepared	1st Dry	Initial Weight	1st Run	2nd Dry	2nd Run	3rd Dry	Final Weight
Date	8/22/2007	8/22/2007	8/24/2007	8/24/2007	8/24/2007	8/27/2007	8/27/2007	8/29/2007
By	RM	RM	TMK	TMK	TMK	RM	RM	RM

Lab ID	Sample No./ Hole No.	Sample Depth (ft)	Sample Description	Fragment Description	Tare ID	Tare Weight	Initial Wt. + Tare	Final Wt. + Tare	SDI
103	AC-11/205+95,1.3 Lt.	86.8'-87.4'	gray Limestone w/shale	80% intact	JW2	289.51	872.29	703.92	71.1
106	AC-11/205+95,1.3 Lt.	99.5'-100.3'	gray Limestone w/shale	100% intact	DDI	286.05	768.45	712.70	88.4
110	AC-11/205+95,1.3 Lt.	109.7'-110.6'	gray Limestone w/shale	100% intact	KC5	293.67	865.07	832.88	94.4
115	AC-12/205+94, 71 Rt.	85.6'-86.2'	gray Limestone w/shale	50% intact	333	289.44	821.21	465.95	33.2
117	AC-12/205+94, 71 Rt.	95.3'-95.8'	gray Limestone w/shale	100% intact	KG2	296.33	888.43	835.96	91.1
118	AC-12/205+94, 71 Rt.	101.9'-102.7'	gray Limestone w/shale	100% intact	D16	292.88	923.08	871.87	91.9
128	AC-13/206+50, 0.02 Lt.	92.8'-93.5'	gray Limestone w/shale	100% intact	AK32	288.10	839.96	783.96	89.9
131	AC-13/206+50, 0.02 Lt.	102.9'-103.5'	gray Limestone w/shale	pile chips	C1	309.58	762.04	355.79	10.2
134	AC-13/206+50, 0.02 Lt.	108.7'-109.3'	gray Limestone w/shale	100% intact	SW20	298.18	881.65	867.26	97.5
141	AC-15/210+20, 37.3 Rt.	48.9'-49.6'	gray Limestone w/shale	100% intact	P-01	287.40	833.31	822.86	98.1
143	AC-15/210+20, 37.3 Rt.	57.5'-58.8'	gray Limestone w/shale	100% intact	D47	309.66	820.34	803.19	96.6

Checked By KDG Reviewed By

FMSM ENGINEERS

Slake Durability Index
KM 64 - 513

Project Name I-265 Bridge over the Ohio River

Project Number LX2005125

	Prepared	1st Dry	Initial Weight	1st Run	2nd Dry	2nd Run	3rd Dry	Final Weight
Date	10/29/2007	10/29/2007	10/30/2007	10/30/2007	10/30/2007	10/31/2007	10/31/2007	11/1/2007
By	JF	JF	RC	RC	RC	RC	RC	RC

Lab ID	Sample No./ Hole No.	Sample Depth (ft)	Sample Description	Fragment Description	Tare ID	Tare Weight	Initial Wt. + Tare	Final Wt. + Tare	SDI
227	AC-14	22.1'-23.9'	fine gray limestone	100% intact	ZX1	289.06	794.54	774.69	96.1
240	AC-23	25.2'-26.2'	fine gray limestone	50% intact	JH-33	283.61	786.22	567.78	56.5
247	AC-26	26.6'-27.5'	fine gray limestone	90% intact	SW-20	298.09	818.45	759.93	88.8
248	AC-26	35.7'-36.4'	medium gray limestone	85% intact	KH2	296.93	844.47	708.50	75.2
251	AC-27	23.3'-24.0'	fine gray limestone	80% intact	SJ21	283.85	841.08	684.15	71.8

Checked By SB Reviewed By

FMSM ENGINEERS

Slake Durability Index

ASTM D 4644

Project Number LX2005125

Project Name

	Prepared	1st Dry	Initial Weight	1st Run	2nd Dry	2nd Run	3rd Dry	Final Weight
Date	11-30	11/30	12.1.05	12-01-05	12-01-05	12-02	12-02	12-03
By	[illegible]	JWH	KDG	D.C.	D.C.	D.C.	D.C.	MH

Sample No./	Hole No.	Sample Depth (ft)	Sample Description	Fragment Description	Tare ID	Tare Weight	Initial Wt. + Tare	Final Wt. + Tare	SDI
B-1		87.2-88.3	shale gray fine		911	300.6	740.98	516.07	48.9
B-2		107.9-108.0	" " med		1034	286.[illegible]	811.96	742.66	86.8
B 3		97.7-98.5	[illegible] med		D-13	293.[illegible]	812.40	757.88	89.5

Checked By AMC   Reviewed By [signature]

File: ASTM_SDI.xls Sheet: Input
Preparation Date: 1-2001
Revision Date: 02-2002

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: JW
Approved By: TLK

APPENDIX G
CORROSIVITY TEST RESULTS (SOIL AND WATER)

Microbac Laboratories, Inc.
Member ACIL
KENTUCKY TESTING LABORATORY DIVISION
3323 Gilmore Industrial Blvd. Louisville, KY 40213 502.962.6400 Fax: 502.962.6411
Evansville, IN 812.464.9000 | Lexington, KY 859.276.3506 | Paducah, KY 270.898.3637

Chemical, Biological, Physical, Molecular, and Toxicological Services

ELECTRONIC CERTIFICATE OF ANALYSIS

0709-01071

FULLER,MOSSBARGER,SCOTT & MAY
PAUL COOPER
1901 NELSON MILLER PKWY
LOUISVILLE, KY 40223

Date Reported	10/04/2007
Date Due	09/27/2007
Date Received	09/18/2007
Date Sampled	09/18/2007
Invoice No.	11798
Customer #	4100
Customer P.O.	

PROJ: LX2005125

Analysis	Out of Spec	Qualif	Result	Unit	Min	Max	Method	Std Limit	Date	Time	Tech
Sample: 001 LX2005125 1A (Kentucky Shore) Date & Time Sampled: 09/18/2007 @ 11:45											
PH			7.50	SU			SM 4500 H+ B		09/18/07	14:40	JRV
Sample: 002 LX2005125 1B (Kentucky Shore) Date & Time Sampled: 09/18/2007 @ 11:45											
CHLORIDE			50	MG/L			EPA 300.0	0.5	09/26/07	15:00	JPM
Sample: 003 LX2005125 1C (Kentucky Shore) Date & Time Sampled: 09/18/2007 @ 11:45											
SULFATE			130	MG/L			EPA 300.0	0.5	10/01/07	15:00	JPM
Sample: 004 LX2005125 2A (Mid-River) Date & Time Sampled: 09/18/2007 @ 11:45											
PH			7.58	SU			SM 4500 H+ B		09/18/07	14:40	JRV
Sample: 005 LX2005125 2B (Mid-River) Date & Time Sampled: 09/18/2007 @ 11:45											
CHLORIDE			50	MG/L			EPA 300.0	0.5	09/26/07	15:00	JPM
Sample: 006 LX2005125 2C (Mid-River) Date & Time Sampled: 09/18/2007 @ 11:45											
SULFATE			130	MG/L			EPA 300.0	0.5	10/01/07	15:00	JPM
Sample: 007 LX2005125 3A (Indiana Shore) Date & Time Sampled: 09/18/2007 @ 11:45											
PH			7.65	SU			SM 4500 H+ B		09/18/07	14:40	JRV
Sample: 008 LX2005125 3B (Indiana Shore) Date & Time Sampled: 09/18/2007 @ 11:45											
CHLORIDE			50	MG/L			EPA 300.0	0.5	09/26/07	15:00	JPM
Sample: 009 LX2005125 3C (Indiana Shore) Date & Time Sampled: 09/18/2007 @ 11:45											
SULFATE			130	MG/L			EPA 300.0	0.5	10/01/07	15:00	JPM

UNLESS OTHERWISE NOTED, SAMPLES RESULTS ARE REPORTED ON AN AS RECEIVED BASIS

Microbac Laboratories, Inc.
Member ACIL
KENTUCKY TESTING LABORATORY DIVISION
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Chemical, Biological, Physical, Molecular, and Toxicological Services

ELECTRONIC CERTIFICATE OF ANALYSIS

0709-01071

FULLER,MOSSBARGER,SCOTT & MAY
PAUL COOPER
PROJ: LX2005125

Date Reported	10/04/2007
Date Received	09/18/2007
Date Sampled	09/18/2007

Analysis	Out of Spec	Qualif	Result	Unit	Min	Max	Method	Std Limit	Date	Time	Tech

THIS REPORT HAS BEEN REVIEWED AND APPROVED FOR RELEASE: Ken Ford

MICROBAC LABORATORIES, INC.

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Client:	**FMSM Engineers**	CTL Project No:	**404642**
Project:	**P. O. No. LX2005125**	CTL Project Mgr.:	**Richard Stevenson**
		Analyst:	**Manoj Bharucha**
Contact:	**Tom Dicken**	Approved:	
Submitter:	**Tom Dicken**	Date Analyzed:	**November 21, 2007**
Date Received:	**November 19, 2007**	Date Reported:	**November 21, 2007**

REPORT of WATER SOLUBLE SULFATE ANALYSIS

Sample Identification CTL ID	Client ID	Description	Water Soluble Sulfate (as SO4) (mg/Kg of sample)
1996901	I-265 CS-1	Lean Clay	4
1996902	I-265 CS-2	Silty Sand with Gravel	33
1996903	I-265 CS-3	Well Graded Sand	41
1996904	I-265 CS-4	Sand with Silt and Gravel	82
1996905	I-265 CS-5	PG Sand with Silt and Gravel	144
1996906	I-265 CS-6	WG Sand with Silt	86

Notes:
1. This analysis represents specifically the samples submitted as received.
2. The results were determined by gravimetric analysis following AASHTO T290 Sec. 8-16.
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Client:	FMSM Engineers (KY)	CTL Project No:	404642
Project:	I-265 Bridge / GEOTECH	CTL Project Mgr.:	Stevenson Rick
		Analyst:	Igor Kirin I.K.
Contact:	Tom Dicken	Approved:	
Submitter:	Kurt Schaefer	Date Analyzed:	December 03 2007
Date Received:	November 19 2007	Date Reported:	December 05 2007

REPORT OF SOIL RESISTIVITY ANALYSIS

Client's Sample ID: I-265 CS-1
Material type: Lean Clay
CTL Sample ID: 1996901

Water Dosage (ml)	Resistivity (Ohm-cm)
150	40,955
250	13,275
350	2,655
450	2,118
550	2,175
650	2,344
Minimum	2,118

The minimum electrical resistivity of the tested sample was 2,118 ohm-cm.

Notes:
1. This analysis represents specifically the samples submitted on an as recieved basis.
2. The results were determined by titrametric analysis following AASHTO T288.
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Project:	I-265 Bridge / GEOTECH	CTL Project Mgr.:	Stevenson Rick
		Analyst:	Igor Kirin I.K.
Contact:	Tom Dicken	Approved:	
Submitter:	Kurt Schaefer	Date Analyzed:	December 03 2007
Date Received:	November 19 2007	Date Reported:	December 05 2007

REPORT OF SOIL RESISTIVITY ANALYSIS

Client's Sample ID: I-265 CS-2
Material type: Silty sand with gravel
CTL Sample ID: 1996902

Water Dosage (ml)	Resistivity (Ohm-cm)
150	8,474
250	3,870
350	3,135
450	3,446
550	4,039
Minimum	3,135

The minimum electrical resistivity of the tested sample was 3,135 ohm-cm.

Notes:
1. This analysis represents specifically the samples submitted on an as recieved basis.
2. The results were determined by titrametric analysis following AASHTO T288.
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Client:	FMSM Engineers (KY)	CTL Project No:	404642
Project:	I-265 Bridge / GEOTECH	CTL Project Mgr.:	Stevenson Rick
		Analyst:	Igor Kirin I.K.
Contact:	Tom Dicken	Approved:	
Submitter:	Kurt Schaefer	Date Analyzed:	December 03 2007
Date Received:	November 19 2007	Date Reported:	December 05 2007

REPORT OF SOIL RESISTIVITY ANALYSIS

Client's Sample ID: I-265 CS-3
Material type: Well graded sand
CTL Sample ID: 1996903

Water Dosage (ml)	Resistivity (Ohm-cm)
150	8,191
250	3,728
350	2,570
450	2,937
550	3,163
Minimum	2,570

The minimum electrical resistivity of the tested sample was 2,570 ohm-cm.

Notes:
1. This analysis represents specifically the samples submitted on an as recieved basis.
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Client:	FMSM Engineers (KY)	CTL Project No:	404642
Project:	I-265 Bridge / GEOTECH	CTL Project Mgr.:	Stevenson Rick
		Analyst:	Igor Kirin I. K
Contact:	Tom Dicken	Approved:	
Submitter:	Kurt Schaefer	Date Analyzed:	December 03 2007
Date Received:	November 19 2007	Date Reported:	December 05 2007

REPORT OF SOIL RESISTIVITY ANALYSIS

Client's Sample ID: I-265 CS-4
Material type: Sand with silt & gravel
CTL Sample ID: 1996904

Water Dosage (ml)	Resistivity (Ohm-cm)
150	5,028
250	2,655
350	1,921
450	1,864
550	2,288
Minimum	1,864

The minimum electrical resistivity of the tested sample was 1,864 ohm-cm.

Notes:
1. This analysis represents specifically the samples submitted on an as recieved basis.
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Project:	I-265 Bridge / GEOTECH	CTL Project Mgr.:	Stevenson Rick
		Analyst:	Igor Kirin I. K
Contact:	Tom Dicken	Approved:	
Submitter:	Kurt Schaefer	Date Analyzed:	December 03 2007
Date Received:	November 19 2007	Date Reported:	December 05 2007

REPORT OF SOIL RESISTIVITY ANALYSIS

Client's Sample ID: I-265 CS-5
Material type: PG sand with silt & gravel
CTL Sample ID: 1996905

Water Dosage (ml)	Resistivity (Ohm-cm)
150	5,084
250	1,779
350	1,384
450	1,356
550	1,582
Minimum	1,356

The minimum electrical resistivity of the tested sample was 1,356 ohm-cm.

Notes:
1. This analysis represents specifically the samples submitted on an as recieved basis.
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Project:	I-265 Bridge / GEOTECH	CTL Project Mgr.:	Stevenson Rick
		Analyst:	Igor Kirin I.K.
Contact:	Tom Dicken	Approved:	
Submitter:	Kurt Schaefer	Date Analyzed:	December 03 2007
Date Received:	November 19 2007	Date Reported:	December 05 2007

REPORT OF SOIL RESISTIVITY ANALYSIS

Client's Sample ID: I-265 CS-6
Material type: WG sand with silt
CTL Sample ID: 1996906

Water Dosage (ml)	Resistivity (Ohm-cm)
150	10,168
250	5,141
350	2,655
450	2,486
550	3,107
Minimum	2,486

The minimum electrical resistivity of the tested sample was 2,486 ohm-cm.

Notes:
1. This analysis represents specifically the samples submitted on an as recieved basis.
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APPENDIX H
CALCULATIONS

Appendix H-1 Drilled Shaft Vertical Load Calculations
Appendix H-2 Drilled Shaft Lateral Load Calculations
Appendix H-3 Drilled Shaft Point of Fixity Calculations
Appendix H-4 Correlation of SPT Data
Appendix H-5 Rock Stability Analysis
Appendix H-6 Abutment Analysis

APPENDIX H-1
DRILLED SHAFT VERTICAL LOAD CALCULATIONS

SM 12/24/07

PARSONS BRINCKERHOFF
COMPUTATION SHEET

Geotechnical & Tunneling Division

BY: M. Du **DATE:** 12/19/2007 **PROJECT:** East End Bridge
CHECKED BY: S. Mallett **DATE:** 12/20/07 **PAGE** 1 **OF** 28
SUBJECT: Drilled Shaft Vertical Load Calculations for Pier 1
Preliminary Design Socket Diameter = 7.5 ft & 8.0 ft

PURPOSES

To calculate factored resistances for vertical compression and uplift of the drilled shaft using AASHTO Load and Resistance Factor Design procedures, for Pier 1.

References

- Preliminary design drawings
- Boring logs
- Subsurface soil/rock profiles provided by PB Indianapolis Office.
- AASHTO LRFD Bridge Design Specifications, 4th Edision, 2007.
- Ranges of structural loading provided by PB structural engineers.

Index

- Calculations (pp. 1 ~ 25)
- Idealized soil profiles (p. 26)
- Subsurface profiles (pp. 27 ~ 28)

UNITS AND CONSTANTS

(Note: These calculations are performed using Mathcad, which does automatic units conversions. Most commonly used engineering and scientific units and constants are internaly defined in Mathcad. User-defined units and constants can also be assigned.)

Reference:L:\Mathcad\UnitsDefinition.xmcd

atmospheric pressure $p_a \equiv atm$ $p_a = 14.696\,psi$ ✓

$p_a = 2.116\,ksf$ ✓

MATERIAL PROPERTIES

rock quality designation

throughout the depth range for potential rock socket construction, RQD mostly between 50% and 90%, say: $RQD := 70\%$

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uniaxial compressive strength of rock $q_u := 411\,tsf$ $q_u = 5708\,psi$ ✓

concrete compressive strength $fc' := 5000psi$ per structural engineer ✓

PART A -- 7.5 FT DIAMETER SOCKET

DRILLED SHAFT GEOMERY

Vertical loads are designed to be resisted by rock socket alone. Therefore, geometry of rock socket is used in these calculations.

Diameter $D_s := 7.5ft$ ✓

Cross-sectional area $A_p := \frac{\pi}{4} \cdot D_s^2$ $A_p = 44.179\,ft^2$ ✓

Length $L_s := \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft$ Calculations will be performed for a series of length for developing capacity vs length chart.

$i := 0 .. length(L_s) - 1$ $i = \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \end{pmatrix}$

Note: "i" is an index variables defined for vector and matrix operations.

Shaft circumferential area $A_s := \pi \cdot D_s \cdot L_s$ $A_s = \begin{pmatrix} 117.81 \\ 235.619 \\ 353.429 \\ 471.239 \\ 706.858 \end{pmatrix} ft^2$

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SERVICE LIMIT STATE DESIGN

Settlements

All vertical loads are designed to be resisted by rock. Settlements are expected to be negligible.

Horizontal Movements of Shaft and Shaft Groups

Evaluated separately with LPILE.

Settlement Due to Downdrag

No downdrag is anticipated.

Lateral Squeeze

Only applicable for bridge abutment supported on piles installed through soft soils subject to unbalanced embankment fill loading. Not applicable.

STRENGTH LIMIT STATE DESIGN

Effects of water (or groundwater) table, scour, and downdrag are not considered, as the vertical loads are designed to be resisted by rock alone.

Nominal Axial Compression Resistance of Single Drilled Shaft

1. side resistance

from Table 10.4.6.5-1

$RQD = 70\%$ $Em\_Ei_2 := 0.7$ closed joints

from Table 10.8.3.5.4b-1

$\alpha_E := 0.88$ $\left[0.8 + 0.2 \times \frac{0.2}{0.5} = 0.88\right]$

Equation 10.8.3.5.4b-1:

$$q_s := \min\left[0.65 \cdot \alpha_E \cdot p_a \cdot \left(\frac{q_u}{p_a}\right)^{0.5}, 7.8 p_a \cdot \left(\frac{fc'}{p_a}\right)^{0.5}\right]$$

$q_s = 23.9\ \text{ksf}$

$q_s = 165.7\ \text{psi}$

check: $0.65 \cdot \alpha_E \cdot p_a \cdot \left(\frac{q_u}{p_a}\right)^{0.5} = 23.857\ \text{ksf}$

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$$7.8 p_a \cdot \left(\frac{fc'}{p_a}\right)^{0.5} = 304.467\ \text{ksf}$$

total shaft side resistance $R_s := q_s \cdot (\pi \cdot D_s \cdot L_s)$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad R_s = \begin{pmatrix} 2811 \\ 5621 \\ 8432 \\ 11242 \\ 16863 \end{pmatrix} \text{kip}$$

nominal shaft side resistance

2. Tip Resistance

Due to existance of many thin clay seams in the rock formation, it is prudent to use a reduced end bearing resistance for the rock socket.

Note: the Rock Mass Rating (RMR) will be evaluated during final design by FMSM. For preliminary design, the following rock mass qualities are assumed based on general descriptions in the boring logs.

from Table 10.4.6.4-4

fair to good quality rock mass, rock classes A&B (lime stone 60%, shale 40%)

$m := 0.3$ in between the categories $s := 0.0003$

Equation 10.8.3.5.4c-2 (lower bound values)

$q_p := \left(\sqrt{s} + \sqrt{m \cdot \sqrt{s} + s}\right) \cdot q_u$ $q_u = 822\ \text{ksf}$

$q_p = 75.177\ \text{ksf}$

total shaft tip resistance

$R_p := q_p \cdot A_p$ $R_p = 3321\ \text{kip}$

nominal shaft tip resistance

Total Nominal Resistance

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$$R_s + R_p = \begin{pmatrix} 6132 \\ 8942 \\ 11753 \\ 14563 \\ 20185 \end{pmatrix} kip \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft$$

Factored Axial Compression Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

$\phi := 0.85$ however , $\phi <= 0.70$ ← This factor should be applied to Shaft and End resistance obtained from Load Test. OK! for now.

use : $\phi_{qp} := 0.70$ for tip resistance

$\phi_{qs} := 0.70$ for side resistance

Factored total resistance of drilled shaft $R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s$

$$R_R = \begin{pmatrix} 4292.3 \\ 6259.6 \\ 8227 \\ 10194.4 \\ 14129.2 \end{pmatrix} kip$$

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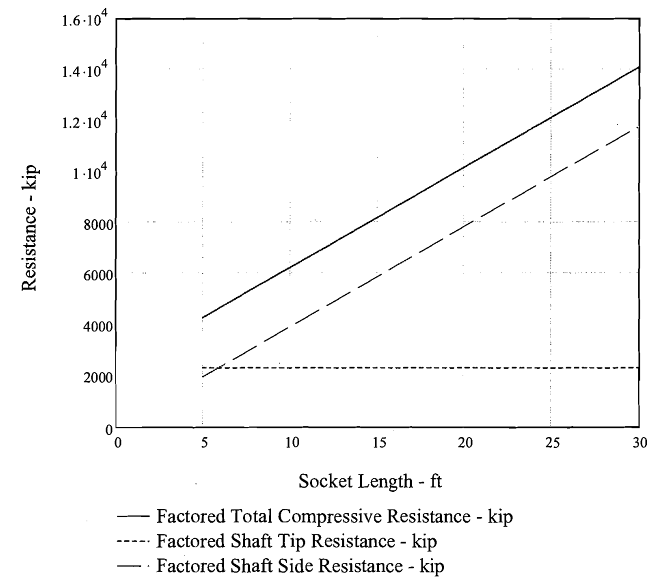
Factored Rock Socket Compression Resistance versus Socket Length

Socket Diameter $D_s = 7.5$ ft



Nominal Skin Friction $q_s = 165.7$ psi

resistance factor $\phi_{qs} = 0.7$

Nominal End Bearing $q_p = 37.6$ tsf

resistance Factor $\phi_{qp} = 0.7$

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Extreme Limit States

Section 10.5.5.3.3

For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used:

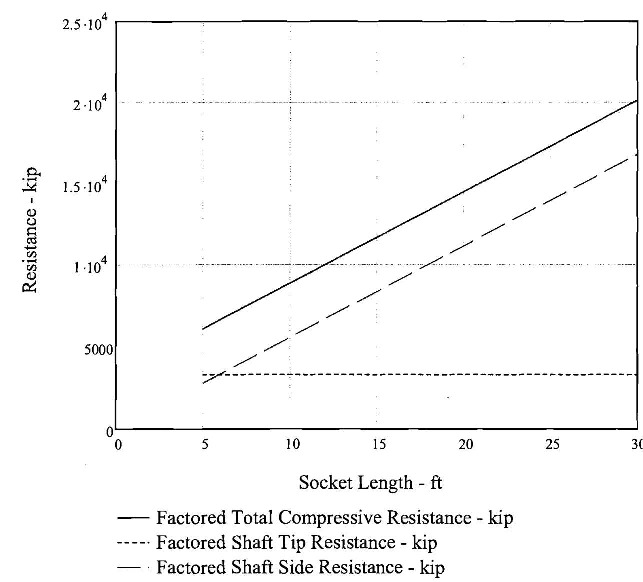
$\phi_{qs} := 1.0$ $\phi_{qp} := 1.0$

Factored total resistance of drilled shaft

$R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s$ $R_R = \begin{pmatrix} 6131.8 \\ 8942.4 \\ 11752.9 \\ 14563.5 \\ 20184.6 \end{pmatrix}$ kip

Factored Rock Socket Compression Resistance versus Socket Length - Extreme Limit States

Socket Diameter $D_s = 7.5$ ft



Nominal Skin Friction $q_s = 165.7$ psi

resistance factor $\phi_{qs} = 1$

Nominal End Bearing $q_p = 37.6$ tsf

resistance Factor $\phi_{qp} = 1$

Chkd: SMS
12/20/07

Nominal Axial Uplift Resistance of Single Drilled Shaft

Acccording to 10.8.3.7.2, the uplift resistance of a straight-sided shaft is similar to that for the side resistance in compression, as calculated above.

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft \qquad R_s = \begin{pmatrix} 2811 \\ 5621 \\ 8432 \\ 11242 \\ 16863 \end{pmatrix} kip$$

nominal shaft side resistance in uplift

weight of shaft

$$W_s := (\gamma_{conc} - \gamma_w) \cdot A_p \cdot L_s \qquad A_p = 44.179\, ft^2 \qquad \gamma_{conc} = 150\, pcf$$

$$\gamma_w = 62.4\, pcf$$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft \qquad W_s = \begin{pmatrix} 19 \\ 39 \\ 58 \\ 77 \\ 116 \end{pmatrix} kip$$

Total Nominal Uplift Resistance

$$R_s + W_s = \begin{pmatrix} 2830 \\ 5660 \\ 8490 \\ 11320 \\ 16979 \end{pmatrix} kip \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft$$

Factored Axial Uplift Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test (serving both compression and uplift purposes) at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

$\phi := 0.85$ however , ϕ <= 0.60

use : $\phi_{up} := 0.60$ for uplift resistance

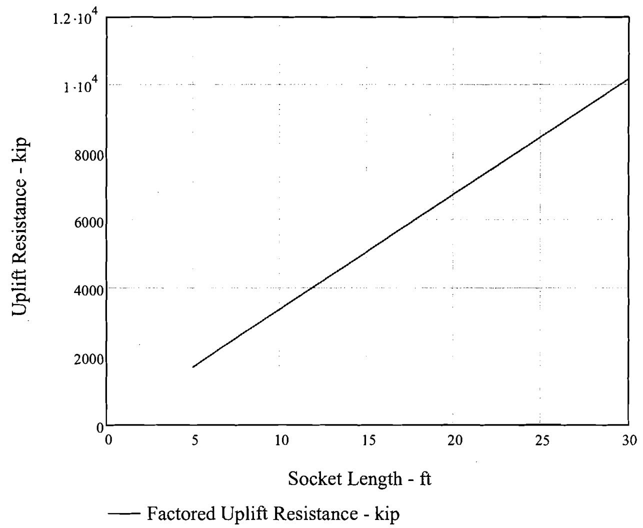
Applicable to shaft resistance obtained from load test.

Factored uplift resistance of drilled shaft $R_R := \phi_{up} \cdot (R_s + W_s)$

$$R_R = \begin{pmatrix} 1698 \\ 3396 \\ 5094 \\ 6792 \\ 10188 \end{pmatrix} kip$$

Factored Rock Socket Uplift Resistance versus Socket Length

Socket Diameter $D_s = 7.5\,\text{ft}$



Nominal Uplift Resistance $q_s = 165.7\,\text{psi}$

weight of shaft $\gamma_{conc} - \gamma_w = 87.6\,\text{pcf}$

resistance factor $\phi_{up} = 0.6$

Extreme Limit States for Uplift

Section 10.5.5.3.3

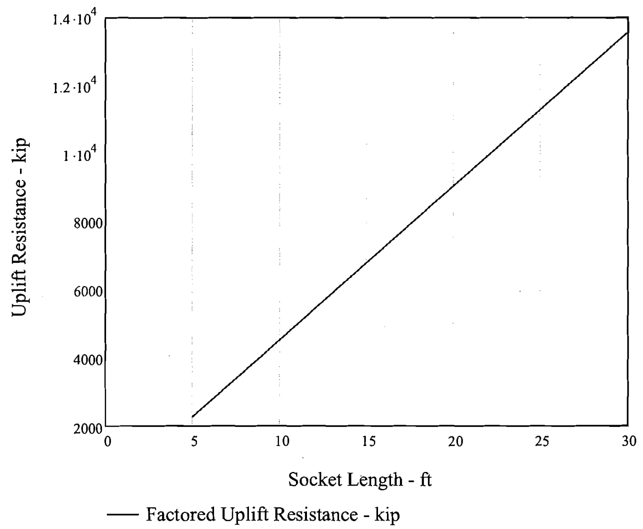
For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used for uplift:

$\phi_{up} := 0.8$ ✓

Factored uplift resistance of drilled shaft $R_R := \phi_{up} \cdot (R_s + W_s)$ $R_R = \begin{pmatrix} 2264 \\ 4528 \\ 6792 \\ 9056 \\ 13584 \end{pmatrix} \text{kip}$ ✓

Factored Rock Socket Uplift Resistance versus Socket Length - Extreme Limit States

Socket Diameter $D_s = 7.5\,ft$



Nominal Uplift Resistance $q_s = 165.7\,psi$

weight of shaft $\gamma_{conc} - \gamma_w = 87.6\,pcf$

resistance factor $\phi_{up} = 0.8$

Chkd SM 12/20/07

PART B -- 8 FT DIAMETER SOCKET

DRILLED SHAFT GEOMERY

Vertical loads are designed to be resisted by rock socket alone. Therefore, geometry of rock socket is used in these calculations.

Diameter $D_s := 8ft$

Cross-sectional area $A_p := \frac{\pi}{4} \cdot D_s^2$ $A_p = 50.265\,ft^2$

Length $L_s := \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft$ Calculations will be performed for a series of length for developing capacity vs length chart.

$i := 0 .. \text{length}(L_s) - 1$ $i = \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \end{pmatrix}$

Note: "i" is an index variables defined for vector and matrix operations.

Shaft circumferential area $A_s := \pi \cdot D_s \cdot L_s$ $A_s = \begin{pmatrix} 125.664 \\ 251.327 \\ 376.991 \\ 502.655 \\ 753.982 \end{pmatrix} ft^2$

Chkd SM 12/20/07

STRENGTH LIMIT STATE DESIGN

Effects of water (or groundwater) table, scour, and downdrag are not considered, as the vertical loads are designed to be resisted by rock alone.

Nominal Axial Compression Resistance of Single Drilled Shaft

1. side resistance

from Table 10.4.6.5-1

$RQD = 70\%$ $\qquad$ $Em\_Ei_2 := 0.7$ $\qquad$ closed joints

from Table 10.8.3.5.4b-1

$\alpha_E := 0.88$

Equation 10.8.3.5.4b-1:

$$q_s := \min\left[0.65\cdot\alpha_E\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5}, 7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5}\right]$$

$q_s = 23.9\ \text{ksf}$

$q_s = 165.7\ \text{psi}$

check: $\qquad 0.65\cdot\alpha_E\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5} = 23.857\ \text{ksf}$

$$7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5} = 304.467\ \text{ksf}$$

total shaft side resistance $\qquad R_s := q_s\cdot(\pi\cdot D_s\cdot L_s)$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad R_s = \begin{pmatrix} 2998 \\ 5996 \\ 8994 \\ 11992 \\ 17988 \end{pmatrix} \text{kip}$$

nominal shaft side resistance

Chkd SM 12/20/07

2. Tip Resistance

Due to existance of many thin clay seams in the rock formation, it is prudent to use a reduced end bearing resistance for the rock socket.

Note: the Rock Mass Rating (RMR) will be evaluated during final design by FMSM. Fur preliminary design, the following rock mass qualities are assumed based on general descriptions in the boring logs.

from Table 10.4.6.4-4

fair to good quality rock mass, rock classes A&B (lime stone 60%, shale 40%)

$m := 0.3$ $\qquad$ in between the categories $\qquad$ $s := 0.0003$

Equation 10.8.3.5.4c-2 (lower bound values)

$q_p := \left(\sqrt{s} + \sqrt{m\cdot\sqrt{s} + s}\right)\cdot q_u$ $\qquad$ $q_u = 822\ \text{ksf}$

$q_p = 75.177\ \text{ksf}$

total shaft tip resistance

$R_p := q_p\cdot A_p$ $\qquad$ $R_p = 3779\ \text{kip}$

nominal shaft tip resistance

Total Nominal Resistance

$$R_s + R_p = \begin{pmatrix} 6777 \\ 9775 \\ 12773 \\ 15771 \\ 21766 \end{pmatrix} \text{kip} \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft}$$

Factored Axial Compression Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

 Chkd SM 12/26/07

$\phi := 0.85$ however , $\phi <= 0.70$

use : $\phi_{qp} := 0.70$ for tip resistance

$\phi_{qs} := 0.70$ for side resistance

See previous comment.

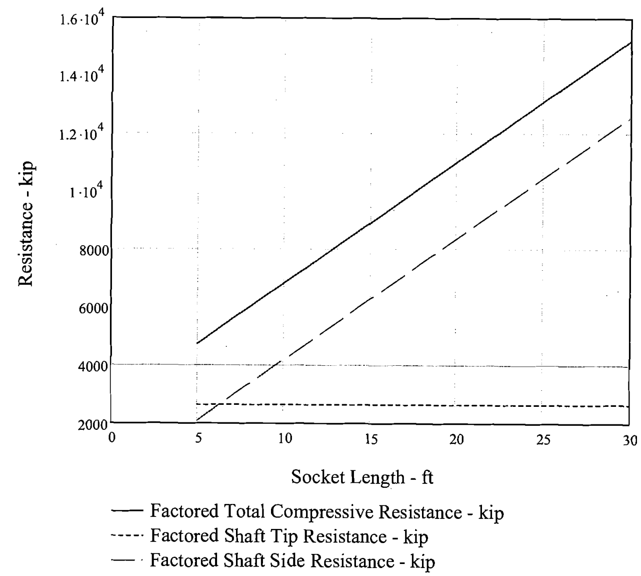
Factored total resistance of drilled shaft $R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s$

$$R_R = \begin{pmatrix} 4743.7 \\ 6842.3 \\ 8940.8 \\ 11039.4 \\ 15236.5 \end{pmatrix} \text{kip}$$

 Chkd SM 12/26/07

Factored Rock Socket Compression Resistance versus Socket Length

Socket Diameter $D_s = 8\ \text{ft}$



Nominal Skin Friction $q_s = 165.7\ \text{psi}$

resistance factor $\phi_{qs} = 0.7$

Nominal End Bearing $q_p = 37.6\ \text{tsf}$

resistance Factor $\phi_{qp} = 0.7$

Chkd : SM
12/20/07

Extreme Limit States

Section 10.5.5.3.3

For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used:

$\phi_{qs} := 1.0$ $\phi_{qp} := 1.0$

Factored total resistance of drilled shaft

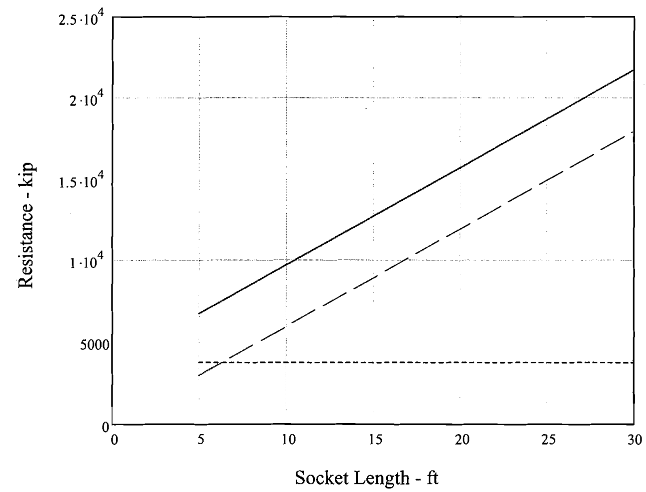
$R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s$

$$R_R = \begin{pmatrix} 6776.8 \\ 9774.7 \\ 12772.6 \\ 15770.6 \\ 21766.4 \end{pmatrix} \text{kip}$$

Chkd SM
12/26/07

Factored Rock Socket Compression Resistance versus Socket Length - Extreme Limit States

Socket Diameter $D_s = 8\,\text{ft}$



Factored Total Compressive Resistance - kip
Factored Shaft Tip Resistance - kip
Factored Shaft Side Resistance - kip

Nominal Skin Friction $q_s = 165.7\,\text{psi}$

resistance factor $\phi_{qs} = 1$

Nominal End Bearing $q_p = 37.6\,\text{tsf}$

resistance Factor $\phi_{qp} = 1$

Nominal Axial Uplift Resistance of Single Drilled Shaft

Acccording to 10.8.3.7.2, the uplift resistance of a straight-sided shaft is similar to that for the side resistance in compression, as calculated above.

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad R_s = \begin{pmatrix} 2998 \\ 5996 \\ 8994 \\ 11992 \\ 17988 \end{pmatrix} \text{kip}$$

nominal shaft side resistance in uplift

weight of shaft

$$W_s := (\gamma_{conc} - \gamma_w) \cdot A_p \cdot L_s \qquad A_p = 50.265 \text{ ft}^2 \qquad \gamma_{conc} = 150 \text{ pcf}$$

$$\gamma_w = 62.4 \text{ pcf}$$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad W_s = \begin{pmatrix} 22 \\ 44 \\ 66 \\ 88 \\ 132 \end{pmatrix} \text{kip}$$

Total Nominal Uplift Resistance

$$R_s + W_s = \begin{pmatrix} 3020 \\ 6040 \\ 9060 \\ 12080 \\ 18120 \end{pmatrix} \text{kip} \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft}$$

Factored Axial Uplift Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test (serving both compression and uplift purposes) at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

SM
12/24/07

$\phi := 0.85$ however , ϕ <= 0.60

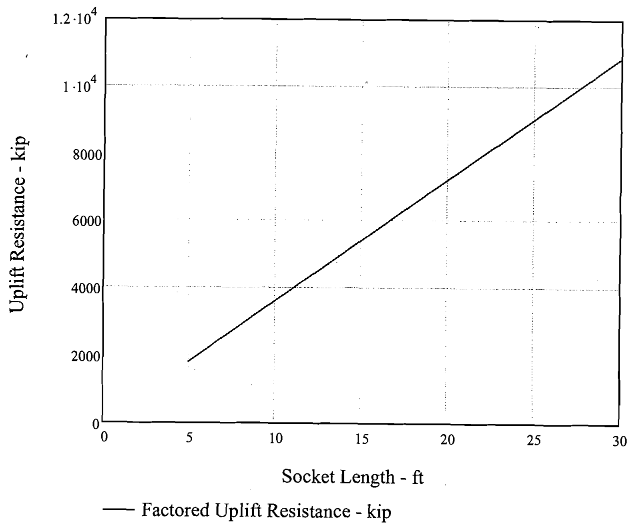
use : $\phi_{up} := 0.60$ for uplift resistance

Factored uplift resistance of drilled shaft $R_R := \phi_{up} \cdot (R_s + W_s)$

$$R_R = \begin{pmatrix} 1812 \\ 3624 \\ 5436 \\ 7248 \\ 10872 \end{pmatrix} \text{kip}$$

Factored Rock Socket Uplift Resistance versus Socket Length

Socket Diameter $D_s = 8\ \text{ft}$



Nominal Uplift Resistance $q_s = 165.7\ \text{psi}$

weight of shaft $\gamma_{conc} - \gamma_w = 87.6\ \text{pcf}$

resistance factor $\phi_{up} = 0.6$

Extreme Limit States for Uplift

Section 10.5.5.3.3

For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used for uplift:

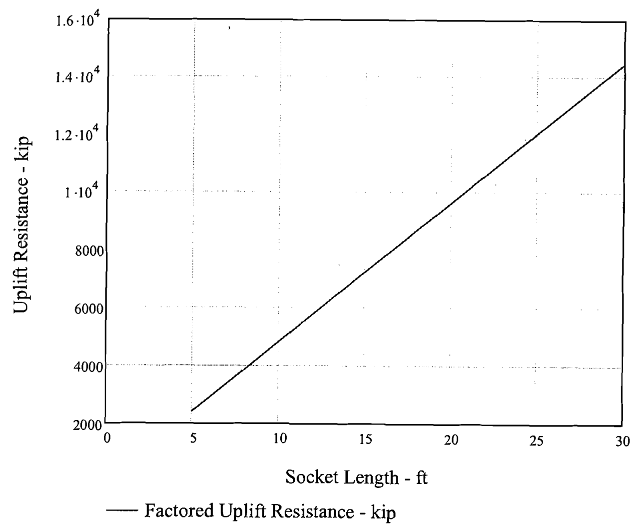
$$\phi_{up} := 0.8$$

Factored uplift resistance of drilled shaft $R_R := \phi_{up} \cdot (R_s + W_s)$

$$R_R = \begin{pmatrix} 2416 \\ 4832 \\ 7248 \\ 9664 \\ 14496 \end{pmatrix} \text{kip}$$

Factored Rock Socket Uplift Resistance versus Socket Length - Extreme Limit States

Socket Diameter $D_s = 8\ \text{ft}$



Nominal Uplift Resistance $q_s = 165.7\ \text{psi}$

weight of shaft $\gamma_{conc} - \gamma_w = 87.6\ \text{pcf}$

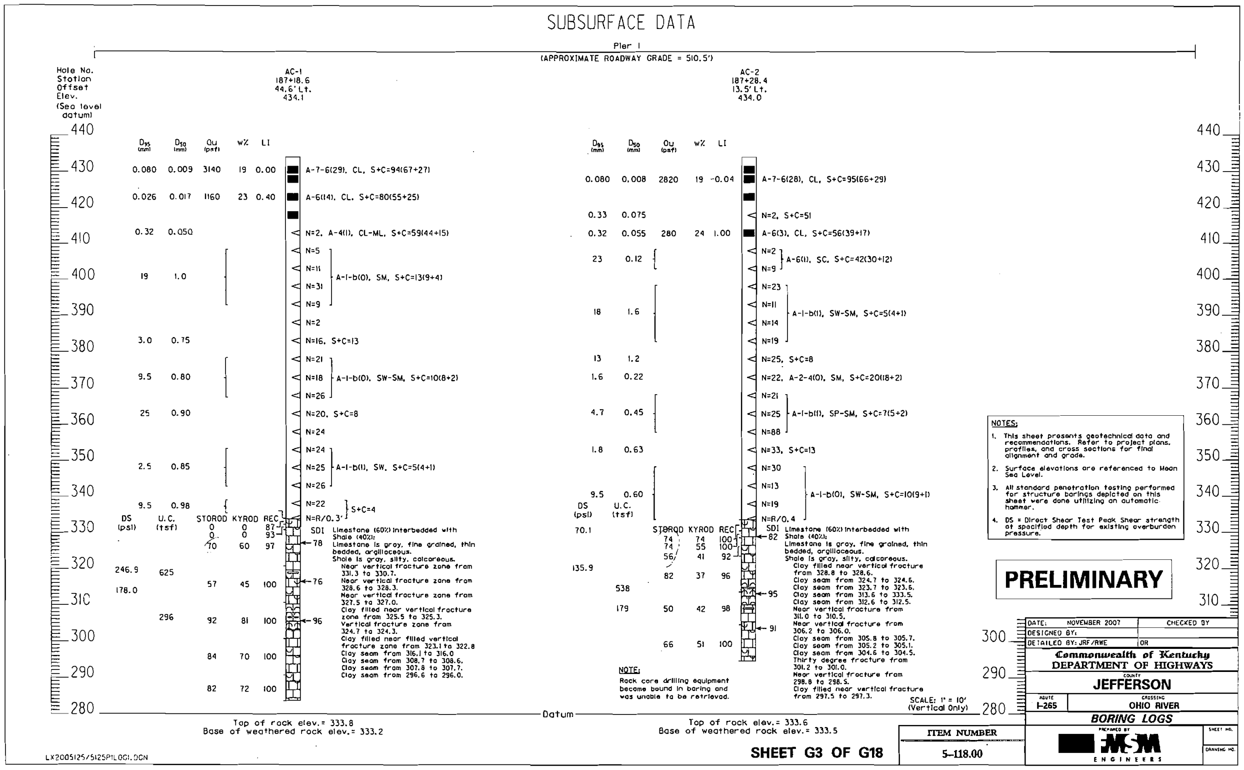
resistance factor $\phi_{up} = 0.8$

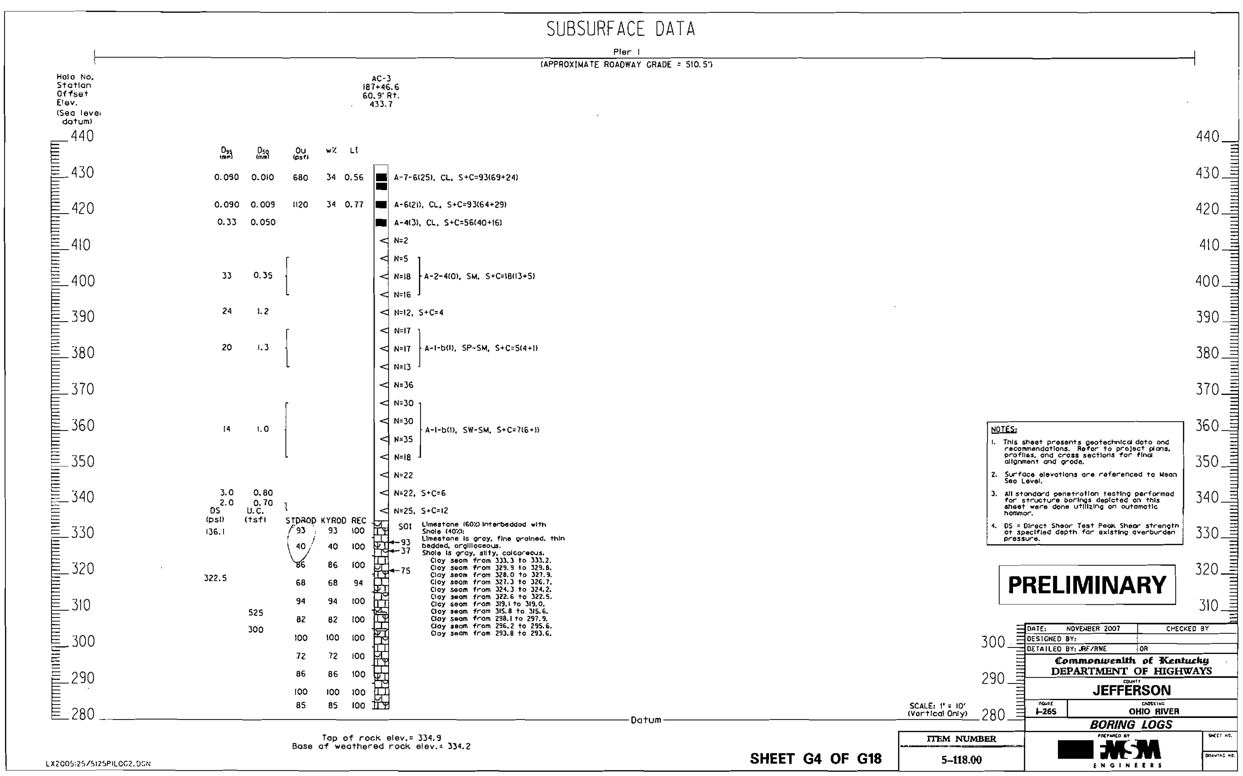
Boring Profiles
12/10/2007

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 1 - STA 187+40, CL
Borings AC-1, 2, 3

Approximate Elevation (ft)	Approximate Depth (ft)	STRATA Description (USCS Classification)	STRATA Parameters	
434.0	0.0			
418.3	15.7	Lean Clay (CL)	$\gamma_t\ (lb/ft^3) = 121$ $\gamma_e\ (lb/ft^3) = 59$ $q_U\ (lb/ft^2) = 1784$ $C_U\ (lb/ft^2) = 892$	$K_S\ (lb/in^3) = 100$ $D_{50}(mm) = 0.03$ $D_{95}(mm) = 0.19$
409.0	25.0	Sand with Silt (SM, SW-SM, and SP-SM)	$\gamma_e\ (lb/ft^3) = 55$ $\phi'\ (^\circ) = 32.5$ $K_S\ (lb/in^3) = 60$	$D_{50}(mm) = 0.90$ $D_{95}(mm) = 20$
379.0	55.0	Sand (SW-SM, SW and SP-SM)	$\gamma_e\ (lb/ft^3) = 64$ $\phi'\ (^\circ) = 34.8$ $K_S\ (lb/in^3) = 60$	$D_{50}(mm) = 0.76$ $D_{95}(mm) = 8.00$
334.0	100.0	Top of Rock Limestone(60%) interbedded with Shale(40%). Limestone is gray, fine grained, thin bedded and argillaceous. Shale is gray, silty, and calcareous.	$\gamma_t\ (lb/ft^3) = 165$ SDI (%) = 80 $q_u(ton/ft^2) = 411$ $c\ (lb/in^2) = 300$ $\phi'\ (^\circ) = 28.0$	
283.1	150.9			


SUBSURFACE DATA
Pier 1
(APPROXIMATE ROADWAY GRADE = 510.5')
Hole No. Station Offset Elev. (Sea level datum)
AC-1 187+18.6 44.6' Lt. 434.1
AC-2 187+28.4 13.5' Lt. 434.0
D85 (mm) D50 (mm) Qu (psf) w% LI
0.080 0.009 3140 19 0.00 A-7-6(29), CL, S+C=94(67+27)
0.026 0.017 1160 23 0.40 A-6(14), CL, S+C=80(55+25)
0.32 0.050 N=2, A-4(1), CL-ML, S+C=59(44+15)
19 1.0 N=5 N=11 N=31 N=9 A-1-b(0), SM, S+C=13(9+4)
N=2
3.0 0.75 N=16, S+C=13
9.5 0.80 N=21 N=18 N=26 A-1-b(0), SW-SM, S+C=10(8+2)
25 0.90 N=20, S+C=8
N=24
2.5 0.85 N=24 N=25 N=26 A-1-b(1), SW, S+C=5(4+1)
9.5 0.98 N=22 N=R/0.3' S+C=4
DS (psi) U.C. (tsf) STDROD KYROD REC
0 0 87
0 0 93
70 60 97
246.9 625
178.0 57 45 100
296 92 81 100
84 70 100
82 72 100
SDI 78 76 96
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, fine grained, thin bedded, argillaceous.
Shale is gray, silty, calcareous.
Near vertical fracture zone from 331.3 to 330.7.
Near vertical fracture zone from 328.6 to 328.3.
Near vertical fracture zone from 327.5 to 327.0.
Clay filled near vertical fracture zone from 325.5 to 325.3.
Vertical fracture zone from 324.7 to 324.3.
Clay filled near filled vertical fracture zone from 323.1 to 322.8
Clay seam from 316.1 to 316.0
Clay seam from 308.7 to 308.6.
Clay seam from 307.8 to 307.7.
Clay seam from 296.6 to 296.0.
Top of rock elev.= 333.8
Base of weathered rock elev.= 333.2
0.080 0.008 2820 19 -0.04 A-7-6(28), CL, S+C=95(66+29)
0.33 0.075 N=2, S+C=51
0.32 0.055 280 24 1.00 A-6(3), CL, S+C=56(39+17)
23 0.12 N=2 N=9 A-6(1), SC, S+C=42(30+12)
N=23
18 1.6 N=11 N=14 N=19 A-1-b(1), SW-SM, S+C=5(4+1)
13 1.2 N=25, S+C=8
1.6 0.22 N=22, A-2-4(0), SM, S+C=20(18+2)
4.7 0.45 N=21 N=25 N=88 A-1-b(1), SP-SM, S+C=7(5+2)
1.8 0.63 N=33, S+C=13
9.5 0.60 N=30 N=13 N=19 N=R/0.4 A-1-b(0), SW-SM, S+C=10(9+1)
DS (psi) U.C. (tsf) STDROD KYROD REC
70.1 74 74 100
74 55 100
56 41 92
135.9 82 37 96
538
179 50 42 98
66 51 100
SDI 82 95 91
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, fine grained, thin bedded, argillaceous.
Shale is gray, silty, calcareous.
Clay filled near vertical fracture from 328.8 to 328.6.
Clay seam from 324.7 to 324.6.
Clay seam from 323.7 to 323.6.
Clay seam from 313.6 to 333.5.
Clay seam from 312.6 to 312.5.
Near vertical fracture from 311.0 to 310.5.
Near vertical fracture from 306.2 to 306.0.
Clay seam from 305.8 to 305.7.
Clay seam from 305.2 to 305.1.
Clay seam from 304.6 to 304.5.
Thirty degree fracture from 301.2 to 301.0.
Near vertical fracture from 298.8 to 298.5.
Clay filled near vertical fracture from 297.5 to 297.3.
NOTE:
Rock core drilling equipment become bound in boring and was unable to be retrieved.
Top of rock elev.= 333.6
Base of weathered rock elev.= 333.5
Datum
440 430 420 410 400 390 380 370 360 350 340 330 320 310 300 290 280
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
DATE: NOVEMBER 2007
CHECKED BY
DESIGNED BY:
DETAILED BY: JRF/RWE
OR
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY
JEFFERSON
ROUTE I-265
CROSSING OHIO RIVER
BORING LOGS
PREPARED BY
MSM ENGINEERS
SHEET NO.
DRAWING NO.
SCALE: 1" = 10' (Vertical Only)
ITEM NUMBER 5-118.00
SHEET G3 OF G18
LX2005125/5125P1LOG1.DGN


SUBSURFACE DATA
Pier 1
(APPROXIMATE ROADWAY GRADE = 510.5')
Hole No. Station Offset Elev. (Sea level datum)
AC-3 187+46.6 60.9' Rt. 433.7
D85 (mm) D50 (mm) Qu (psf) w% LI
0.090 0.010 680 34 0.56 A-7-6(25), CL, S+C=93(69+24)
0.090 0.009 1120 34 0.77 A-6(21), CL, S+C=93(64+29)
0.33 0.050 A-4(3), CL, S+C=56(40+16)
N=2
33 0.35 N=5 N=18 N=16 A-2-4(0), SM, S+C=18(13+5)
24 1.2 N=12, S+C=4
20 1.3 N=17 N=17 N=13 A-1-b(1), SP-SM, S+C=5(4+1)
N=36
14 1.0 N=30 N=30 N=35 N=18 A-1-b(1), SW-SM, S+C=7(6+1)
N=22
3.0 0.80 N=22, S+C=6
2.0 0.70 N=25, S+C=12
DS (psi) U.C. (tsf) STDROD KYROD REC
136.1 93 93 100
40 40 100
86 86 100
322.5 68 68 94
94 94 100
525 82 82 100
300 100 100 100
72 72 100
86 86 100
100 100 100
85 85 100
SDI 93 37 75
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, fine grained, thin bedded, argillaceous.
Shale is gray, silty, calcareous.
Clay seam from 333.3 to 333.2.
Clay seam from 329.9 to 329.8.
Clay seam from 328.0 to 327.9.
Clay seam from 327.3 to 326.7.
Clay seam from 324.3 to 324.2.
Clay seam from 322.6 to 322.5.
Clay seam from 319.1 to 319.0.
Clay seam from 315.8 to 315.6.
Clay seam from 298.1 to 297.9.
Clay seam from 296.2 to 295.6.
Clay seam from 293.8 to 293.6.
Top of rock elev.= 334.9
Base of weathered rock elev.= 334.2
Datum
440 430 420 410 400 390 380 370 360 350 340 330 320 310 300 290 280
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
DATE: NOVEMBER 2007
CHECKED BY
DESIGNED BY:
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OR
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY
JEFFERSON
ROUTE I-265
CROSSING OHIO RIVER
BORING LOGS
PREPARED BY
MSM ENGINEERS
SHEET NO.
DRAWING NO.
SCALE: 1" = 10' (Vertical Only)
ITEM NUMBER 5-118.00
SHEET G4 OF G18
LX2005125/5125P1LOG2.DGN

PARSONS BRINCKERHOFF
COMPUTATION SHEET
Geotechnical & Tunneling Division

BY: M. Du **DATE:** 12/19/2007 **PROJECT:** East End Bridge
CHECKED BY: S. Malhotra **DATE:** 12/24/07 **PAGE** 1 **OF** 28
SUBJECT: Drilled Shaft Vertical Load Calculations for Piers 2 through 5
Preliminary Design Socket Diameter = 7.5 ft

PURPOSES

To calculate factored resistances for vertical compression and uplift of the drilled shaft using AASHTO Load and Resistance Factor Design procedures, for Piers 2 through 5.

References

- Preliminary design drawings
- Boring logs
- Subsurface soil/rock profiles provided by PB Indianapolis Office.
- AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.
- Ranges of structural loading provided by PB structural engineers.

Index

- Calculations (pp. 1 ~ 14)
- Idealized soil profiles (pp. 15 ~ 18)
- Sbsurface profiles (pp. 19 ~ 28)

UNITS AND CONSTANTS

(Note: These calculations are performed using Mathcad, which does automatic units conversions. Most commonly used engineering and scientific units and constants are internaly defined in Mathcad. User-defined units and constants can also be assigned.)

Reference:L:\Mathcad\UnitsDefinition.xmcd

atmospheric pressure $p_a \equiv atm$ $p_a = 14.696\,psi$

$p_a = 2.116\,ksf$

MATERIAL PROPERTIES

rock quality designation

top 10 ft of rock, RQD mostly between 20% and 50%, say: $RQD_1 := 35\%$

below 10 ft, RQD mostly between 50% and 90%, say: $RQD_2 := 70\%$

uniaxial compressive strength of rock $q_u := 550tsf$ $q_u = 7639\,psi$

concrete compressive strength $fc' := 5000psi$ per structural engineer

DRILLED SHAFT GEOMERY

Vertical loads are designed to be resisted by rock socket alone. Therefore, geometry of rock socket is used in these calculations.

Diameter $D_s := 7.5ft$

Cross-sectional area $A_p := \frac{\pi}{4} \cdot D_s^2$ $A_p = 44.179\,ft^2$

Length $L_s := \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft$ Calculations will be performed for a series of length for developing capacity vs length chart.

$i := 0 .. length(L_s) - 1$ $i = \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \end{pmatrix}$

Note: "i" is an index variables defined for vector and matrix operations.

Shaft circumferential area $A_s := \pi \cdot D_s \cdot L_s$ $A_s = \begin{pmatrix} 117.81 \\ 235.619 \\ 353.429 \\ 471.239 \\ 706.858 \end{pmatrix} ft^2$

SERVICE LIMIT STATE DESIGN

Settlements

All vertical loads are designed to be resisted by rock. Settlements are expected to be negligible.

Horizontal Movements of Shaft and Shaft Groups

Evaluated separately with LPILE.

Settlement Due to Downdrag

No downdrag is anticipated.

Lateral Squeeze

Only applicable for bridge abutment supported on piles installed through soft soils subject to unbalanced embankment fill loading. Not applicable.

STRENGTH LIMIT STATE DESIGN

Effects of water (or groundwater) table, scour, and downdrag are not considered, as the vertical loads are designed to be resisted by rock alone.

Nominal Axial Compression Resistance of Single Drilled Shaft

1. side resistance

from Table 10.4.6.5-1

top 10 ft of rock $RQD_1 = 35\%$ $Em\_Ei_1 := 0.1$ for closed joints

below 10 ft $RQD_2 = 70\%$ $Em\_Ei_2 := 0.7$ for closed joints

from Table 10.8.3.5.4b-1

top 10 ft of rock $\alpha_{E1} := 0.55$

below 10 ft $\alpha_{E2} := 0.88$

Equation 10.8.3.5.4b-1:

top 10 ft of rock: $q_{s1} := \min\left[0.65\cdot\alpha_{E1}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5}, 7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5}\right]$

$q_{s1} = 17.2\,\text{ksf}$

$q_{s1} = 119.8\,\text{psi}$

check: $0.65\cdot\alpha_{E1}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5} = 17.249\,\text{ksf}$ ✓

$7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5} = 304.467\,\text{ksf}$ ✓

below 10 ft: $q_{s2} := \min\left[0.65\cdot\alpha_{E2}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5}, 7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5}\right]$

$q_{s2} = 27.6\,\text{ksf}$ ✓

$q_{s2} = 191.7\,\text{psi}$

check: $0.65\cdot\alpha_{E2}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5} = 27.598\,\text{ksf}$ ✓

$7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5} = 304.467\,\text{ksf}$

total shaft side resistance

define function to calculate total shaft side resistance:

$$Rs(L_s) := \begin{cases} q_{s1}\cdot(\pi\cdot D_s\cdot L_s) & \text{if } L_s \le 10\text{ft} \\ q_{s1}\cdot(\pi\cdot D_s\cdot 10\text{ft}) + q_{s2}\cdot[\pi\cdot D_s\cdot(L_s - 10\text{ft})] & \text{otherwise} \end{cases}$$

$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix}\text{ft}$ $R_{s_i} := Rs(L_{s_i})$ $R_s = \begin{pmatrix} 2032 \\ 4064 \\ 7315 \\ 10567 \\ 17069 \end{pmatrix}\text{kip}$ ✓

note: "i" is the index variable defined above

nominal shaft side resistance

2. Tip Resistance

Note: the Rock Mass Rating (RMR) will be evaluated during final design by FMSM. Fur preliminary design, the following rock mass qualities are assumed based on general descriptions in the boring logs.

top 10 ft (upper) poor to fair rock

below 10 ft (lower) fair to good rock

from Table 10.4.6.4-4

top 10 ft poor to fair quality rock mass, rock classes A&B (lime stone 60%, shale 40%)

$m_1 := 0.9$ in between the categories $s_1 := 0.000015$ ✓

below 10 ft fair to good quality rock mass, rock classes A&B (lime stone 60%, shale 40%)

$m_2 := 0.5$ in between the categories $s_2 := 0.0005$

Equation 10.8.3.5.4c-2 (lower bound values)

top 10 ft of rock $q_{p1} := \left(\sqrt{s_1} + \sqrt{m_1 \cdot \sqrt{s_1} + s_1}\right) \cdot q_u$

$q_{p1} = 69.344\ \text{ksf}$

below 10 ft $q_{p2} := \left(\sqrt{s_2} + \sqrt{m_2 \cdot \sqrt{s_2} + s_2}\right) \cdot q_u$

$q_{p2} = 143.48\ \text{ksf}$

total shaft tip resistance

define function to calculate total shaft tip resistance:

$$Rp(L_s) := \begin{array}{|l} q_{p1} \cdot A_p \ \text{ if } L_s < 10\text{ft} \\ q_{p2} \cdot A_p \ \text{ otherwise} \end{array}$$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad R_{p_i} := Rp\left(L_{s_i}\right) \qquad R_p = \begin{pmatrix} 3064 \\ 6339 \\ 6339 \\ 6339 \\ 6339 \end{pmatrix} \text{kip}$$

note: "i" is the index variable defined above

nominal shaft tip resistance

Total Nominal Resistance

$$R_s + R_p = \begin{pmatrix} 5096 \\ 10403 \\ 13654 \\ 16905 \\ 23408 \end{pmatrix} \text{kip} \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft}$$

Factored Axial Compression Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

$\phi := 0.85$ however , $\phi <= 0.70$

use : $\phi_{qp} := 0.70$ for tip resistance

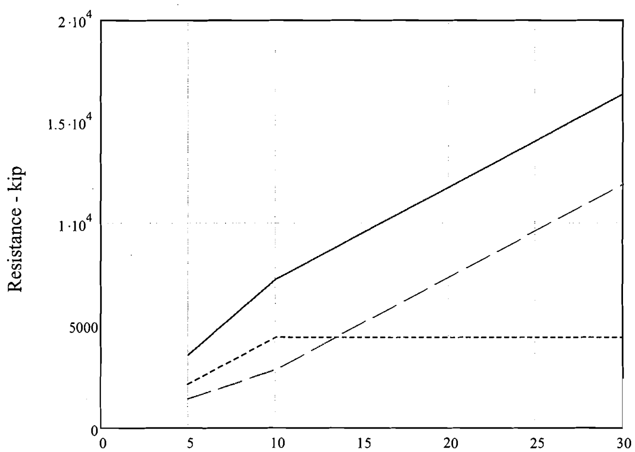
$\phi_{qs} := 0.70$ for side resistance

Factored total resistance of drilled shaft $R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s$

$$R_R = \begin{pmatrix} 3566.9 \\ 7282 \\ 9557.9 \\ 11833.8 \\ 16385.6 \end{pmatrix} \text{kip}$$

Factored Rock Socket Compression Resistance versus Socket Length

Socket Diameter $D_s = 7.5\,ft$



Nominal Skin Friction	$q_{s1} = 119.8\,psi$ ✓	top 10 ft of rock
	$q_{s2} = 191.7\,psi$ ✓	below 10 ft
	resistance factor	$\phi_{qs} = 0.7$
Nominal End Bearing	$q_{p1} = 34.7\,tsf$ ✓	top 10 ft of rock
	$q_{p2} = 71.7\,tsf$ ✓	below 10 ft
	resistance Factor	$\phi_{qp} = 0.7$

Extreme Limit States

Section 10.5.5.3.3

For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used:

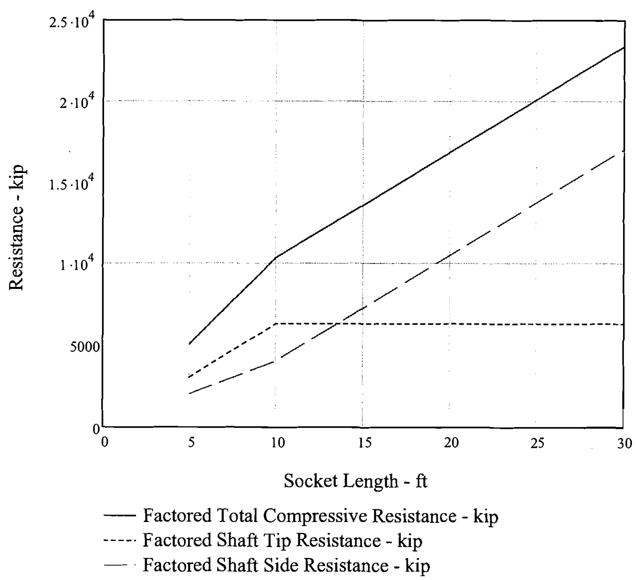
$\phi_{qs} := 1.0$ $\qquad$ $\phi_{qp} := 1.0$

Factored total resistance of drilled shaft

$R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s$ $\qquad$ $R_R = \begin{pmatrix} 5095.5 \\ 10402.8 \\ 13654.1 \\ 16905.4 \\ 23407.9 \end{pmatrix} kip$ ✓

Factored Rock Socket Compression Resistance versus Socket Length - Extreme Limit States

Socket Diameter $D_s = 7.5\,ft$



Nominal Skin Friction

$q_{s1} = 119.8\,psi$ top 10 ft of rock

$q_{s2} = 191.7\,psi$ below 10 ft

resistance factor $\phi_{qs} = 1$

Nominal End Bearing

$q_{p1} = 34.7\,tsf$ top 10 ft of rock

$q_{p2} = 71.7\,tsf$ below 10 ft

resistance Factor $\phi_{qp} = 1$

Nominal Axial Uplift Resistance of Single Drilled Shaft

Acccording to 10.8.3.7.2, the uplift resistance of a straight-sided shaft is similar to that for the side resistance in compression, as calculated above.

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft \qquad R_s = \begin{pmatrix} 2032 \\ 4064 \\ 7315 \\ 10567 \\ 17069 \end{pmatrix} kip$$

nominal shaft side resistance in uplift

weight of shaft

$$W_s := (\gamma_{conc} - \gamma_w) \cdot A_p \cdot L_s \qquad A_p = 44.179\,ft^2 \qquad \gamma_{conc} = 150\,pcf \qquad \gamma_w = 62.4\,pcf$$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft \qquad W_s = \begin{pmatrix} 19 \\ 39 \\ 58 \\ 77 \\ 116 \end{pmatrix} kip$$

Total Nominal Uplift Resistance

$$R_s + W_s = \begin{pmatrix} 2051 \\ 4103 \\ 7373 \\ 10644 \\ 17185 \end{pmatrix} kip \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft$$

Factored Axial Uplift Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test (serving both compression and uplift purposes) at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

$\phi := 0.85$ however , $\phi <= 0.60$

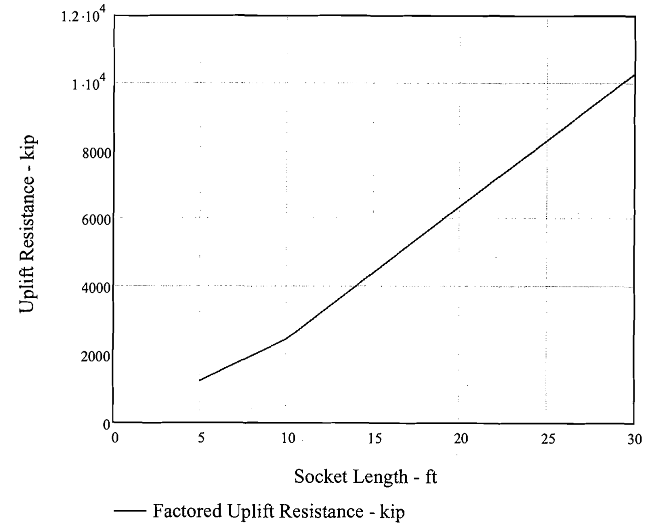
use : $\phi_{up} := 0.60$ for uplift resistance

Factored uplift resistance of drilled shaft $R_R := \phi_{up} \cdot (R_s + W_s)$

$$R_R = \begin{pmatrix} 1231 \\ 2462 \\ 4424 \\ 6386 \\ 10311 \end{pmatrix} kip$$

Factored Rock Socket Uplift Resistance versus Socket Length

Socket Diameter $D_s = 7.5\,ft$



Nominal Uplift Resistance $q_{s1} = 119.8\,psi$ top 10 ft of rock

$q_{s2} = 191.7\,psi$ below 10 ft

weight of shaft $\gamma_{conc} - \gamma_w = 87.6\,pcf$

resistance factor $\phi_{up} = 0.6$

Extreme Limit States for Uplift

Section 10.5.5.3.3

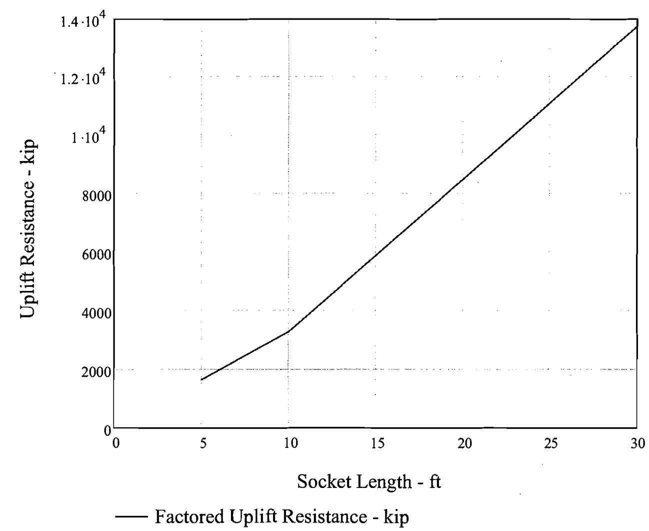
For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used for uplift:

$$\phi_{up} := 0.8$$

Factored uplift resistance of drilled shaft $\quad R_R := \phi_{up} \cdot (R_s + W_s) \qquad R_R = \begin{pmatrix} 1641 \\ 3282 \\ 5899 \\ 8515 \\ 13748 \end{pmatrix} \text{kip}$

Factored Rock Socket Uplift Resistance versus Socket Length - Extreme Limit States

Socket Diameter $\quad D_s = 7.5\,\text{ft}$



Nominal Uplift Resistance	$q_{s1} = 119.8\,\text{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\,\text{psi}$	below 10 ft
	weight of shaft	$\gamma_{conc} - \gamma_w = 87.6\,\text{pcf}$
	resistance factor	$\phi_{up} = 0.8$

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GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 2 - STA 189+65.5, CL
Borings AC-4, 5, B-1

Approximate Elevation (ft)	Approximate Depth (ft)	Description STRATA: Description (USCS Description)	Parameters	
428.9	0.0			
		Sandy Lean Clay (CL)	γ_t (lb/ft^3) = 121 K_S (lb/in^3) = 30	D_{50}(mm) = 0.060 D_{95}(mm) = 0.37
423.9	5.0			
420.8	8.1	▽ Sand with Silt (SM)	γ_t (lb/ft^3) = 107 γ_e (lb/ft^3)* = 45 ϕ' (°) = 28.0 K_S (lb/in^3) = 25 K_S (lb/in^3) = 20	D_{50}(mm) = 0.11 D_{95}(mm) = 0.85 (Above Water Table) (Below Water Table)
409.9	19.0			
		Sand (SW, SW-SM)	γ_e (lb/ft^3)* = 68 ϕ' (°) = 35.2 K_S (lb/in^3) = 60	D_{50}(mm) = 1.6 D_{95}(mm) = 19
379.9	49.0			
		Sand with gravel (SW-SM, SP, SP-SM, GP-GM,)	γ_e (lb/ft^3)* = 65 ϕ' (°) = 35.6 K_S (lb/in^3) = 60	D_{50}(mm) = 2.4 D_{95}(mm) = 18
334.9	94.0	Top of Rock		
		Limestone (55%) interbedded with Shale(45%). Limestone is gray, fine grained, thin bedded, argillaceous and fossiliferous. Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous.	γ_t (lb/ft^3) = 165 SDI (%) = 73 q_u(ton/ft^2) = 563 c (lb/in^2) = 300 ϕ' (°) = 28.0	
282.6	146.3			
		Shale (70%) interbedded with Limestone (30%). Shale is gray, fine grained, thin bedded, silty, calcareous, fossiliferous. Limestone is gray, microcrystalline to fine grained, thin bedded, argillaceous, fossiliferous.	γ_t (lb/ft^3) = 160 c (lb/in^2) = 300 ϕ' (°) = 28.0	
275.9	153.0			

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GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 3 - STA 193+77.5, CL
Borings AC-6, 7, 8, 9, B-2

Approximate Elevation (ft)	Approximate Depth (ft)	Description STRATA: Description (USCS Description)	Parameters	
419.4	0.0	▽ Water - Ohio River		
379.4	40.0			
		Sand (SW-SM, SP, SW)	γ_e (lb/ft^3)* = 66 ϕ' (°) = 34.5 K_S (lb/in^3) = 60	D_{50}(mm) = 0.98 D_{95}(mm) = 17
365.4	54.0			
		Sand (SP-SM, SW-SM)	γ_e (lb/ft^3)* = 67 ϕ' (°) = 36.9 K_S (lb/in^3) = 60	D_{50}(mm) = 0.62 D_{95}(mm) = 18
354.4	65.0			
		Gravel (GW, GW-GM)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 11 D_{95}(mm) = 30
339.4	80.0			
		Sand (SP-SM, SM)	γ_e (lb/ft^3)* = 69 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 1.3 D_{95}(mm) = 19
332.0	87.4	Top of Rock		
		Limestone (60%) interbedded with Shale (40%). Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous. Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.	γ_t (lb/ft^3) = 164 SDI (%) = 67 q_u(ton/ft^2) = 647 c (lb/in^2) = 300 ϕ' (°) = 28.0	
278.6	140.8			

Boring Profiles
12/10/2007

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GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 4 - STA 206+12.5, CL
Borings AC-10, 11, 12, 13, B-3

Approximate Elevation (ft)	Depth (ft)	Description (USCS Description)	Parameters (Description STRATA)	
418.8	0.0	Water - Ohio River		
378.8	40.0	Gravel (GW, GP)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 35.5 K_S (lb/in^3) = 20	D_{50}(mm) = 9.1 D_{95}(mm) = 27
359.8	59.0	Sand (SP-SM, SW-SM, SP)	γ_e (lb/ft^3)* = 68 ϕ' (°) = 37.0 K_S (lb/in^3) = 125	D_{50}(mm) = 2.4 D_{95}(mm) = 18
343.8	75.0	Gravel (GP-GM, GM)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 3.9 D_{95}(mm) = 23
336.2	82.6	Top of Rock Limestone (60%) interbedded with Shale (40%). Limestone is gray, microcrystalline to fine grained, thin, wavy to nodular bedded, fossiliferous, and argillaceous. Shale is gray, silty, laminated to thin bedded, calcareous, and fossiliferous.	γ_t (lb/ft^3) = 165 SDI (%) = 74 q_u(ton/ft^2) = 550 c (lb/in^2) = 300 ϕ' (°) = 28.0	
282.8	136.0			

Boring Profiles

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GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 5 - STA 210+24.5, CL
Borings AC-14, AC-15, B-4

AC-14

Approximate Elevation (ft)	Depth (ft)	Description (USCS Description)	Parameters (Description STRATA)	
436.0	0.0	(AC-14) Sandy Lean Clay (CL)	γ_t (lb/ft^3) = 120 q_u (lb/ft^2) = 2000 C_u (lb/ft^2) = 1000	K_S (lb/in^3) = 100 D_{50}(mm) = #### D_{95}(mm) = 4.00
423.9	12.1	Top of Rock (AC-14) Limestone, gray, medium grained, thin bedded to medium bedded.	γ_t (lb/ft^3) = 164 q_u(ton/ft^2) = 600 c (lb/in^2) = 10 ϕ' (°) = 20.0	
417.0	19.0	(AC-14) Limestone (60%) interbedded with Shale (40%). Limestone is gray, fine to medium grained, very thin to medium wavy bedded. Shale is gray, silty	γ_t (lb/ft^3) = 164 SDI (%) = 96 q_u(ton/ft^2) = 650 c (lb/in^2) = 150 ϕ' (°) = 27.0	
407.0	29.0	(AC-14) Shale, gray to red, very thin bedded to thin bedded, silty	γ_t (lb/ft^3) = 146 SDI (%) = 0 q_u(ton/ft^2) = 13 c (lb/in^2) = 5 ϕ' (°) = 20.0	
404.0	32.0	(AC-14) Limestone, light gray, fine grained, thin bedded, shale stringers, streaks, and partings	γ_t (lb/ft^3) = 160 q_u(ton/ft^2) = 989 c (lb/in^2) = 160 ϕ' (°) = 32.0	
394.2	41.8	(AC-14) Dolomite Limestone, greenish gray, fine, thin bedded	γ_t (lb/ft^3) = 168 q_u(ton/ft^2) = 600 c (lb/in^2) = 150 ϕ' (°) = 32.0	
392.0	44.0	Limestone (30% - 60%) interbedded with Shale (40% - 70%). Limestone is gray, fine to medium crystalline grained, very thin to medium nodularly bedded, fossiliferous. Shale is gray, silty, laminated, calcareous, fossiliferous.	γ_t (lb/ft^3) = 168 SDI (%) = 97 q_u(ton/ft^2) = 782 c (lb/in^2) = 100 ϕ' (°) = 25.0	
342.0	94.0			

AC-15

Approximate Elevation (ft)	Depth (ft)	Description (USCS Description)	Parameters (Description Strata)	
419.4	0.0	Water - Ohio River		
415.2	4.2	Sandy Silt with Gravel (ML)	γ_e (lb/ft^3)* = 38 ϕ' (°) = 29.5 K_S (lb/in^3) = 20	D_{50}(mm) = 0.26 D_{95}(mm) = 10
409.4	10	Poorly graded gravel with silt and sand (GP-GM)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 7.1 D_{95}(mm) = 27
392.0	27.4	Top of Rock		

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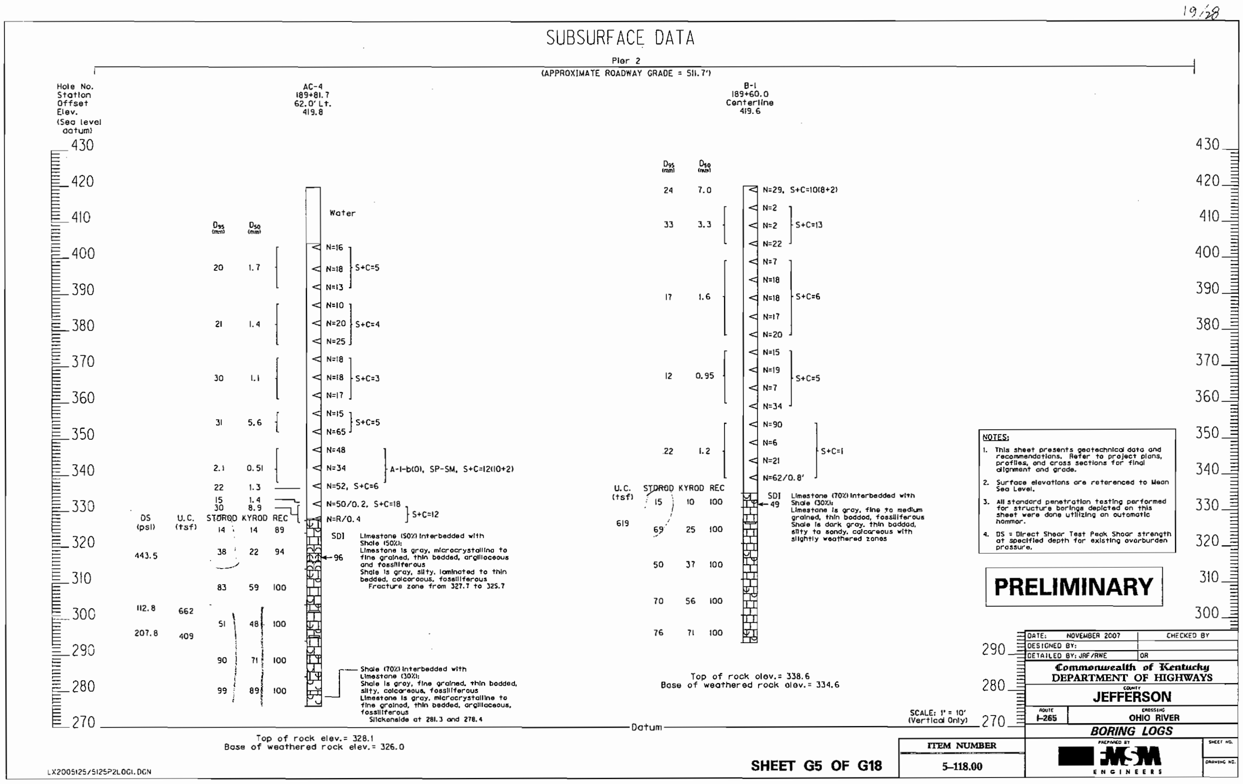
SUBSURFACE DATA
Pier 2
(APPROXIMATE ROADWAY GRADE = 511.7')
Hole No. Station Offset Elev. (Sea level datum)
AC-4 189+81.7 62.0' Lt. 419.8
B-1 189+60.0 Centerline 419.6
Water
N=16 N=18 N=13 S+C=5
N=10 N=20 N=25 S+C=4
N=18 N=18 N=17 S+C=3
N=15 N=65 S+C=5
N=48 N=34 A-1-b(0), SP-SM, S+C=12(10+2)
N=52, S+C=6
N=50/0.2, S+C=18
N=R/0.4 S+C=12
DS (psi) U.C. (tsf) STDROD KYROD REC
SD1 Limestone (50%) interbedded with Shale (50%): Limestone is gray, microcrystalline to fine grained, thin bedded, argillaceous and fossiliferous. Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous. Fracture zone from 327.7 to 325.7
Shale (70%) interbedded with Limestone (30%): Shale is gray, fine grained, thin bedded, silty, calcareous, fossiliferous. Limestone is gray, microcrystalline to fine grained, thin bedded, argillaceous, fossiliferous. Slickenside at 281.3 and 278.4
Top of rock elev.= 328.1
Base of weathered rock elev.= 326.0
N=29, S+C=10(8+2)
N=2 N=2 S+C=13 N=22
N=7 N=18 N=18 S+C=6 N=17 N=20
N=15 N=19 S+C=5 N=7 N=34
N=90 N=6 S+C=1 N=21 N=62/0.8'
SD1 Limestone (70%) interbedded with Shale (30%): Limestone is gray, fine to medium grained, thin bedded, fossiliferous. Shale is dark gray, thin bedded, silty to sandy, calcareous with slightly weathered zones
Top of rock elev.= 338.6
Base of weathered rock elev.= 334.6
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007 CHECKED BY
DESIGNED BY:
DETAILED BY: JRF/RWE
Commonwealth of Kentucky DEPARTMENT OF HIGHWAYS
JEFFERSON
I-265 OHIO RIVER
BORING LOGS
ITEM NUMBER 5-118.00
SHEET G5 OF G18
LX2005125/5125P2LOG1.DGN

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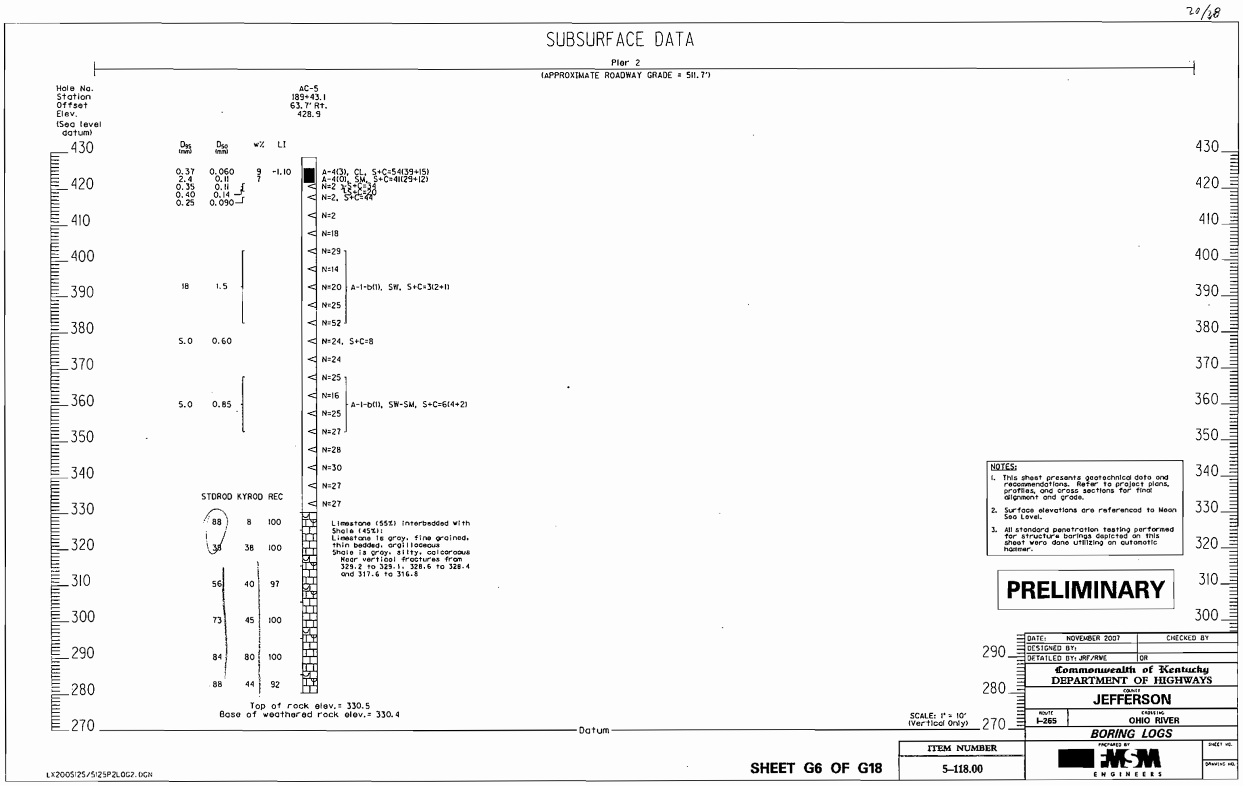
SUBSURFACE DATA
Pier 2
(APPROXIMATE ROADWAY GRADE = 511.7')
Hole No. Station Offset Elev. (Sea level datum)
AC-5 189+43.1 63.7' Rt. 428.9
A-4(3), CL, S+C=54(39+15)
A-4(0), SM, S+C=41(29+12)
N=2 S+C=34
N=2, S+C=44
N=2 N=18 N=29 N=14
N=20 A-1-b(1), SW, S+C=3(2+1)
N=25 N=52
N=24, S+C=8
N=24 N=25 N=16
A-1-b(1), SW-SM, S+C=6(4+2)
N=25 N=27 N=28 N=30 N=27 N=27
STDROD KYROD REC
Limestone (55%) interbedded with Shale (45%): Limestone is gray, fine grained, thin bedded, argillaceous. Shale is gray, silty, calcareous. Near vertical fractures from 329.2 to 329.1, 328.6 to 328.4 and 317.6 to 316.8
Top of rock elev.= 330.5
Base of weathered rock elev.= 330.4
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
PRELIMINARY
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007 CHECKED BY
DESIGNED BY:
DETAILED BY: JRF/RWE
Commonwealth of Kentucky DEPARTMENT OF HIGHWAYS
JEFFERSON
I-265 OHIO RIVER
BORING LOGS
ITEM NUMBER 5-118.00
SHEET G6 OF G18
LX2005125/5125P2LOG2.DGN

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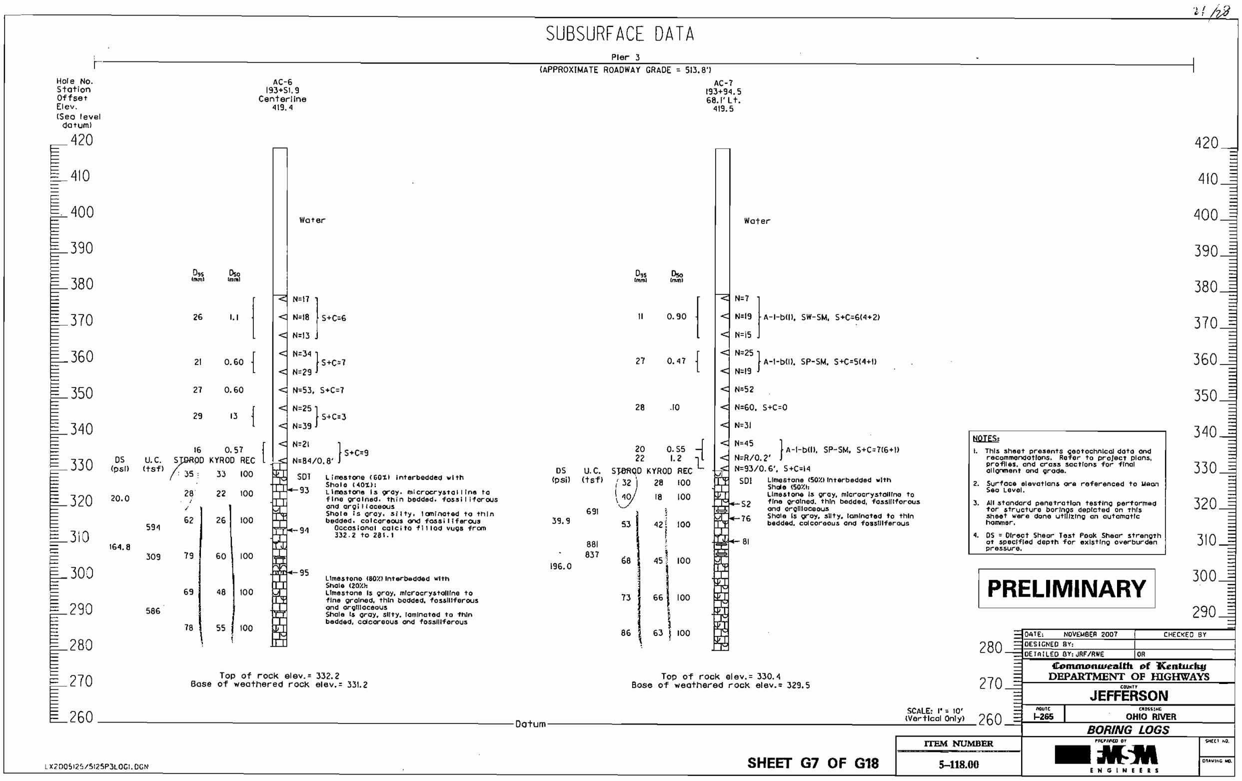
SUBSURFACE DATA
Pier 3
(APPROXIMATE ROADWAY GRADE = 513.8')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-6
193+51.9
Centerline
419.4
AC-7
193+94.5
68.1' Lt.
419.5
Water
N=17
N=18 S+C=6
N=13
N=34
N=29 S+C=7
N=53, S+C=7
N=25
N=39 S+C=3
N=21
N=84/0.8' S+C=9
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
Occasional calcite filled vugs from 332.2 to 281.1
Limestone (80%) interbedded with Shale (20%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
Top of rock elev.= 332.2
Base of weathered rock elev.= 331.2
N=7
N=19 A-1-b(1), SW-SM, S+C=6(4+2)
N=15
N=25
N=19 A-1-b(1), SP-SM, S+C=5(4+1)
N=52
N=60, S+C=0
N=31
N=45
N=R/0.2' A-1-b(1), SP-SM, S+C=7(6+1)
N=93/0.6', S+C=14
Limestone (50%) interbedded with Shale (50%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
Top of rock elev.= 330.4
Base of weathered rock elev.= 329.5
DS (psi) U.C. (tsf) STDROD KYROD REC
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
DATE: NOVEMBER 2007
DESIGNED BY:
DETAILED BY: JRF/RNE
CHECKED BY
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY JEFFERSON
ROUTE I-265
CROSSING OHIO RIVER
BORING LOGS
SCALE: 1" = 10' (Vertical Only)
ITEM NUMBER 5-118.00
SHEET G7 OF G18
MSM ENGINEERS
LX2005125/5125P3LOG1.DGN

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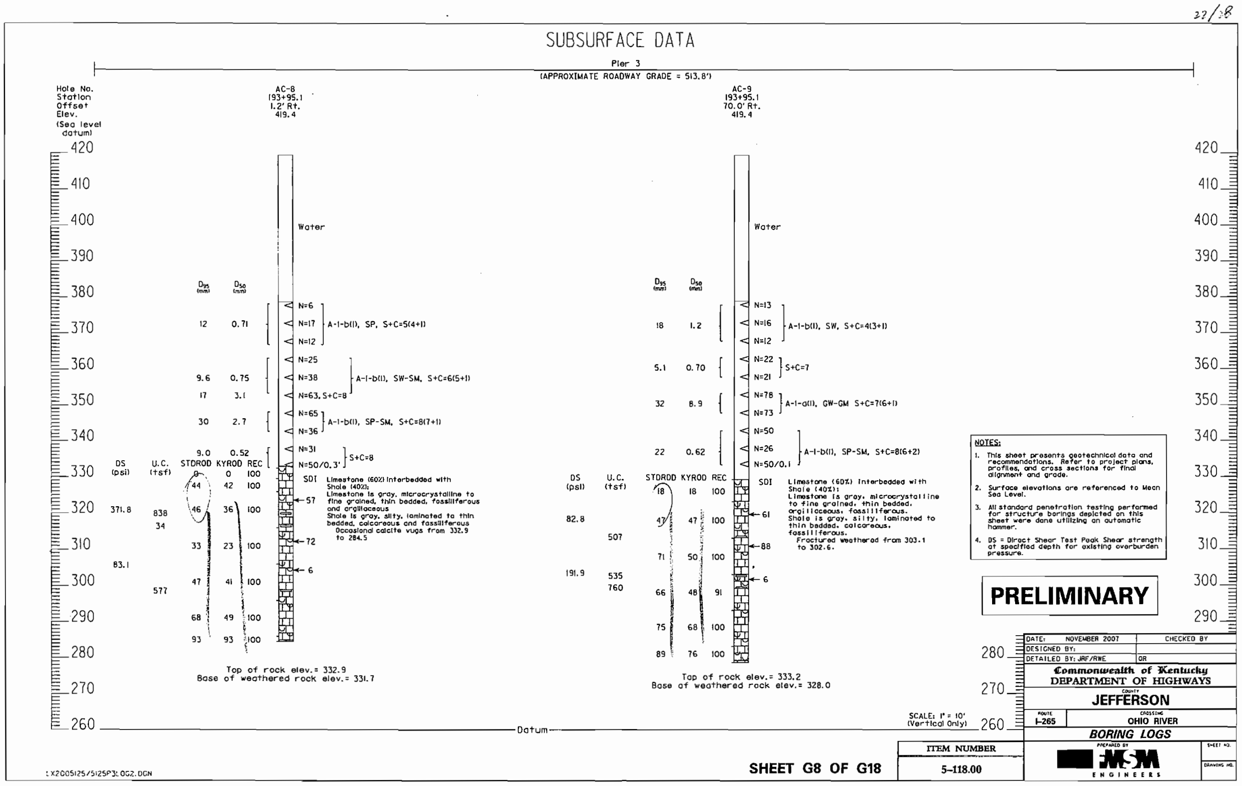
SUBSURFACE DATA
Pier 3
(APPROXIMATE ROADWAY GRADE = 513.8')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-8
193+95.1
1.2' Rt.
419.4
AC-9
193+95.1
70.0' Rt.
419.4
Water
N=6
N=17 A-1-b(1), SP, S+C=5(4+1)
N=12
N=25
N=38 A-1-b(1), SW-SM, S+C=6(5+1)
N=63, S+C=8
N=65
N=36 A-1-b(1), SP-SM, S+C=8(7+1)
N=31
N=50/0.3' S+C=8
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous
Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous
Occasional calcite vugs from 332.9 to 284.5
Top of rock elev.= 332.9
Base of weathered rock elev.= 331.7
N=13
N=16 A-1-b(1), SW, S+C=4(3+1)
N=12
N=22
N=21 S+C=7
N=78
N=73 A-1-a(1), GW-GM S+C=7(6+1)
N=50
N=26
N=50/0.1 A-1-b(1), SP-SM, S+C=8(6+2)
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, thin bedded, argillaceous, fossiliferous.
Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.
Fractured weathered from 303.1 to 302.6.
Top of rock elev.= 333.2
Base of weathered rock elev.= 328.0
DS (psi) U.C. (tsf) STDROD KYROD REC
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
DATE: NOVEMBER 2007
DESIGNED BY:
DETAILED BY: JRF/RNE
CHECKED BY
Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY JEFFERSON
ROUTE I-265
CROSSING OHIO RIVER
BORING LOGS
SCALE: 1" = 10' (Vertical Only)
ITEM NUMBER 5-118.00
SHEET G8 OF G18
MSM ENGINEERS
LX2005125/5125P3LOG2.DGN

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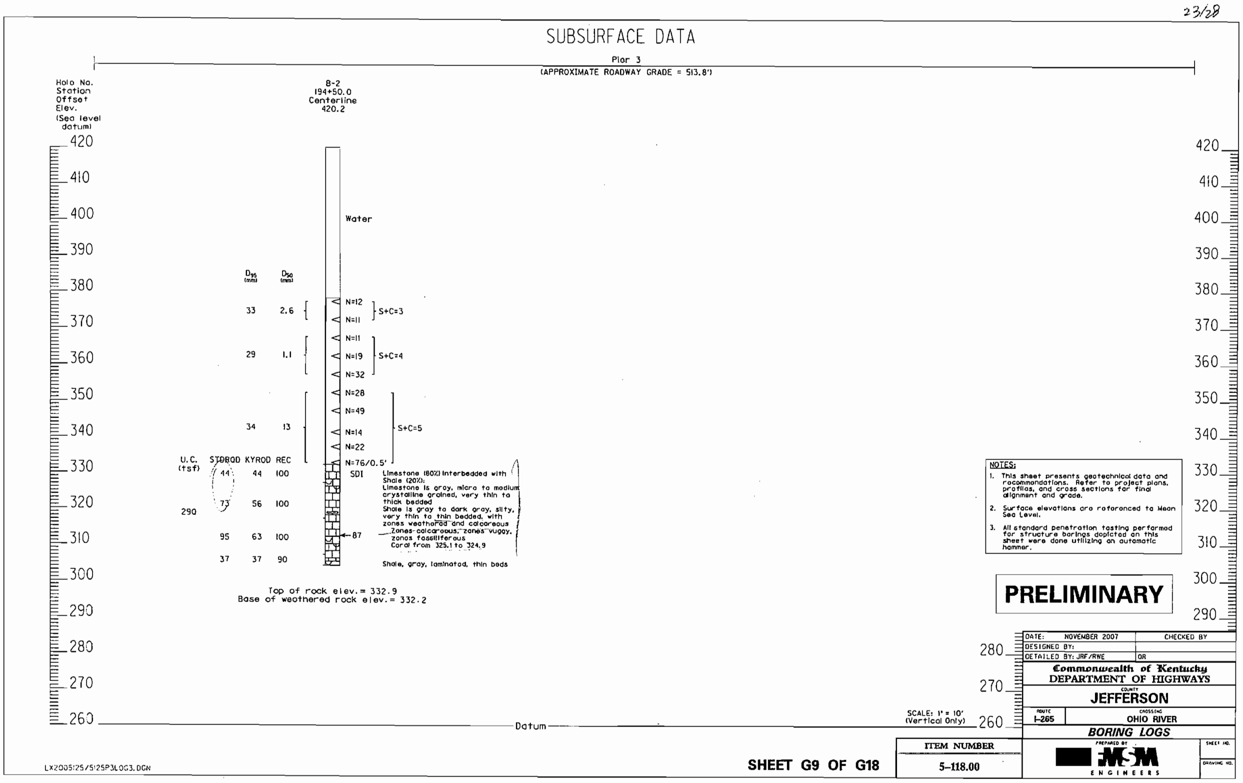
SUBSURFACE DATA
Pier 3
(APPROXIMATE ROADWAY GRADE = 513.8')
Hole No. Station Offset Elev. (Sea level datum)
B-2
194+50.0
Centerline
420.2
Water
N=12
N=11
S+C=3
N=11
N=19
N=32
S+C=4
N=28
N=49
N=14
N=22
N=76/0.5'
S+C=5
SDI
Limestone (80%) interbedded with Shale (20%):
Limestone is gray, micro to medium crystalline grained, very thin to thick bedded
Shale is gray to dark gray, silty, very thin to thin bedded, with zones weathered and calcareous
Zones calcareous; zones vuggy, zones fossiliferous
Coral from 325.1 to 324.9
Shale, gray, laminated, thin beds
Top of rock elev.= 332.9
Base of weathered rock elev.= 332.2
NOTES:
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2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
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I-265
OHIO RIVER
BORING LOGS
SCALE: 1" = 10' (Vertical Only)
Datum
ITEM NUMBER
5-118.00
SHEET G9 OF G18
MSM ENGINEERS
LX2005125/5125P3LOG3.DGN

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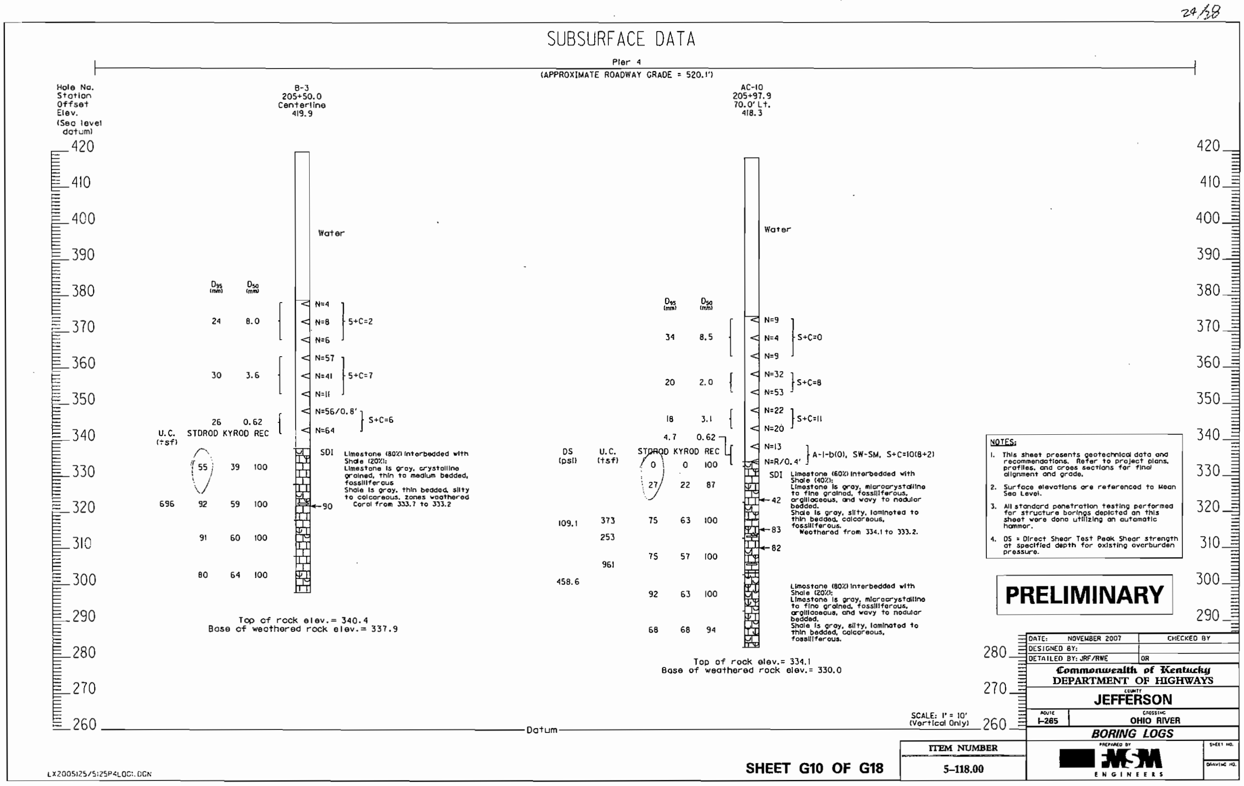
SUBSURFACE DATA
Pier 4
(APPROXIMATE ROADWAY GRADE = 520.1')
Hole No. Station Offset Elev. (Sea level datum)
B-3
205+50.0
Centerline
419.9
AC-10
205+97.9
70.0' Lt.
418.3
Water
N=4
N=8
N=6
S+C=2
N=57
N=41
N=11
S+C=7
N=56/0.8'
N=64
S+C=6
SDI
Limestone (80%) interbedded with Shale (20%):
Limestone is gray, crystalline grained, thin to medium bedded, fossiliferous
Shale is gray, thin bedded, silty to calcareous, zones weathered
Coral from 333.7 to 333.2
Top of rock elev.= 340.4
Base of weathered rock elev.= 337.9
N=9
N=4
N=9
S+C=0
N=32
N=53
S+C=8
N=22
N=20
S+C=11
N=13
N=R/0.4'
A-1-b(0), SW-SM, S+C=10(8+2)
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, fossiliferous, argillaceous, and wavy to nodular bedded.
Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.
Weathered from 334.1 to 333.2.
Limestone (80%) interbedded with Shale (20%):
Limestone is gray, microcrystalline to fine grained, fossiliferous, argillaceous, and wavy to nodular bedded.
Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.
Top of rock elev.= 334.1
Base of weathered rock elev.= 330.0
NOTES:
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4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
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I-265
OHIO RIVER
BORING LOGS
SCALE: 1" = 10' (Vertical Only)
Datum
ITEM NUMBER
5-118.00
SHEET G10 OF G18
MSM ENGINEERS
LX2005125/5125P4LOG1.DGN

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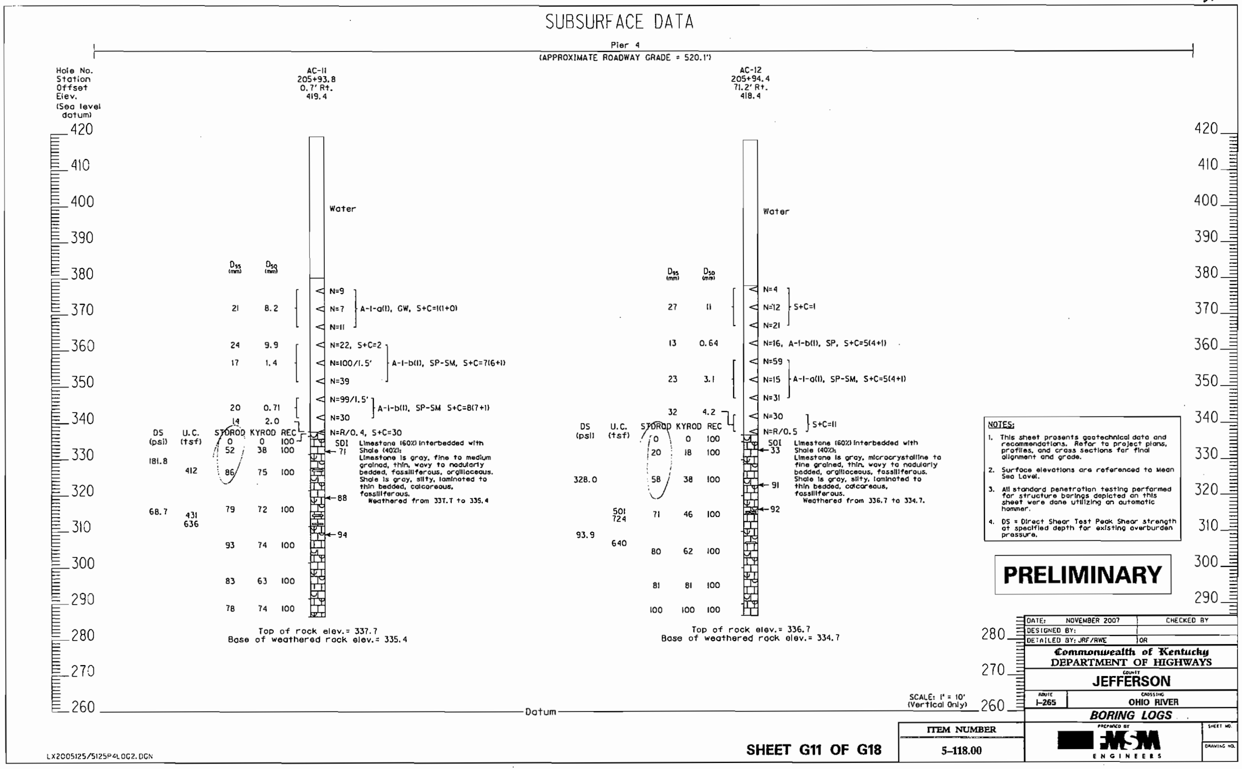
SUBSURFACE DATA
Pier 4
(APPROXIMATE ROADWAY GRADE = 520.1')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-11
205+93.8
0.7' Rt.
419.4
AC-12
205+94.4
71.2' Rt.
418.4
Water
D95 (mm)
D50 (mm)
21 8.2
24 9.9
17 1.4
20 0.71
14 2.0
N=9
N=7
N=11
A-1-a(1), GW, S+C=1(1+0)
N=22, S+C=2
N=100/1.5'
N=39
A-1-b(1), SP-SM, S+C=7(6+1)
N=99/1.5'
N=30
A-1-b(1), SP-SM S+C=8(7+1)
N=R/0.4, S+C=30
DS (psi)
U.C. (tsf)
STDROD KYROD REC
0 0 100
52 38 100
181.8
412
86 75 100
68.7
431
636
79 72 100
93 74 100
83 63 100
78 74 100
SDI
71
88
94
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, fine to medium grained, thin, wavy to nodularly bedded, fossiliferous, argillaceous.
Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.
Weathered from 337.7 to 335.4
Top of rock elev.= 337.7
Base of weathered rock elev.= 335.4
27 11
13 0.64
23 3.1
32 4.2
N=4
N=12
N=21
S+C=1
N=16, A-1-b(1), SP, S+C=5(4+1)
N=59
N=15
N=31
A-1-a(1), SP-SM, S+C=5(4+1)
N=30
N=R/0.5
S+C=11
DS (psi)
U.C. (tsf)
STDROD KYROD REC
0 0 100
20 18 100
328.0
58 38 100
501
724
71 46 100
93.9
640
80 62 100
81 81 100
100 100 100
SDI
33
91
92
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, microcrystalline to fine grained, thin, wavy to nodularly bedded, argillaceous, fossiliferous.
Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.
Weathered from 336.7 to 334.7.
Top of rock elev.= 336.7
Base of weathered rock elev.= 334.7
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
270
260
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
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4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007
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DEPARTMENT OF HIGHWAYS
COUNTY
JEFFERSON
ROUTE
I-265
CROSSING
OHIO RIVER
BORING LOGS
PREPARED BY
MSM
ENGINEERS
SHEET NO.
DRAWING NO.
ITEM NUMBER
5-118.00
SHEET G11 OF G18
LX2005125/5125P4LOG2.DGN

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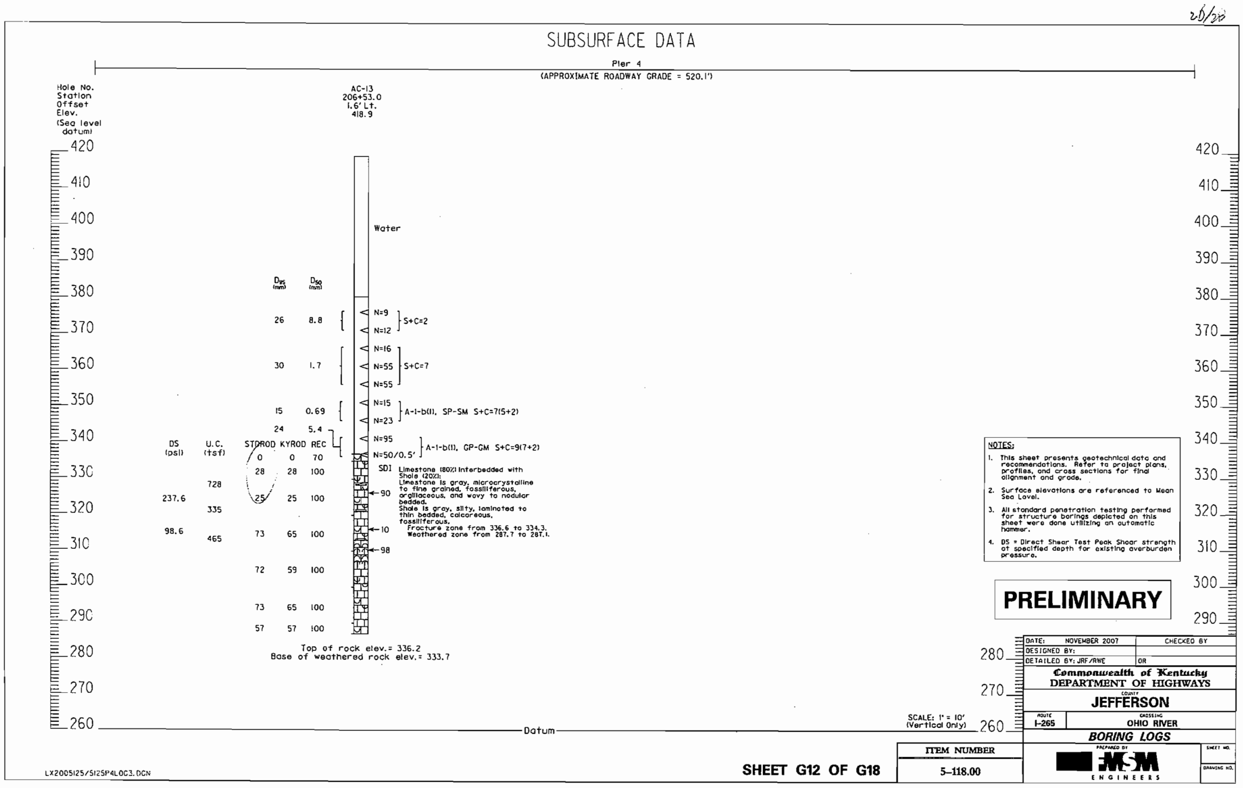
SUBSURFACE DATA
Pier 4
(APPROXIMATE ROADWAY GRADE = 520.1')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-13
206+53.0
1.6' Lt.
418.9
Water
D95 (mm)
D50 (mm)
26 8.8
30 1.7
15 0.69
24 5.4
N=9
N=12
S+C=2
N=16
N=55
N=55
S+C=7
N=15
N=23
A-1-b(1), SP-SM S+C=7(5+2)
N=95
N=50/0.5'
A-1-b(1), GP-GM S+C=9(7+2)
DS (psi)
U.C. (tsf)
STDROD KYROD REC
0 0 70
28 28 100
728
237.6
25 25 100
335
98.6
465
73 65 100
72 59 100
73 65 100
57 57 100
SDI
90
10
98
Limestone (80%) interbedded with Shale (20%):
Limestone is gray, microcrystalline to fine grained, fossiliferous, argillaceous, and wavy to nodular bedded.
Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.
Fracture zone from 336.6 to 334.3.
Weathered zone from 287.7 to 287.1.
Top of rock elev.= 336.2
Base of weathered rock elev.= 333.7
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
270
260
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
SCALE: 1" = 10' (Vertical Only)
DATE: NOVEMBER 2007
CHECKED BY
DESIGNED BY:
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Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY
JEFFERSON
ROUTE
I-265
CROSSING
OHIO RIVER
BORING LOGS
PREPARED BY
MSM
ENGINEERS
SHEET NO.
DRAWING NO.
ITEM NUMBER
5-118.00
SHEET G12 OF G18
LX2005125/5125P4LOG3.DGN

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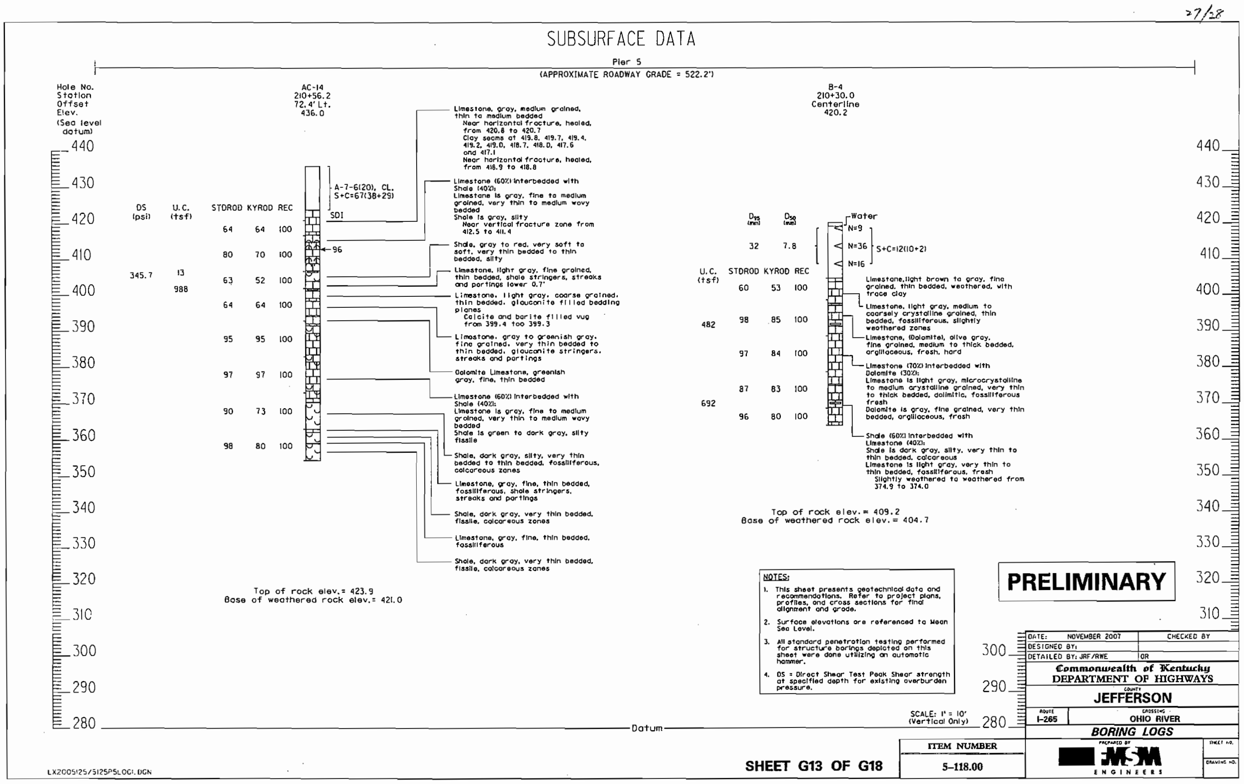
SUBSURFACE DATA
Pier 5
(APPROXIMATE ROADWAY GRADE = 522.2')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-14
210+56.2
72.4' Lt.
436.0
DS (psi)
U.C. (tsf)
STDROD KYROD REC
64 64 100
80 70 100
345.7
13
63 52 100
988
64 64 100
95 95 100
97 97 100
90 73 100
98 80 100
A-7-6(20), CL, S+C=67(38+29)
SDI
96
Limestone, gray, medium grained, thin to medium bedded
Near horizontal fracture, healed, from 420.8 to 420.7
Clay seams at 419.8, 419.7, 419.4, 419.2, 419.0, 418.7, 418.0, 417.6 and 417.1
Near horizontal fracture, healed, from 418.9 to 418.8
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, fine to medium grained, very thin to medium wavy bedded
Shale is gray, silty
Near vertical fracture zone from 412.5 to 411.4
Shale, gray to red, very soft to soft, very thin bedded to thin bedded, silty
Limestone, light gray, fine grained, thin bedded, shale stringers, streaks and partings lower 0.7'
Limestone, light gray, coarse grained, thin bedded, glauconite filled bedding planes
Calcite and barite filled vug from 399.4 too 399.3
Limestone, gray to greenish gray, fine grained, very thin bedded to thin bedded, glauconite stringers, streaks and partings
Dolomite Limestone, greenish gray, fine, thin bedded
Limestone (60%) interbedded with Shale (40%):
Limestone is gray, fine to medium grained, very thin to medium wavy bedded
Shale is green to dark gray, silty fissile
Shale, dark gray, silty, very thin bedded to thin bedded, fossiliferous, calcareous zones
Limestone, gray, fine, thin bedded, fossiliferous, shale stringers, streaks and partings
Shale, dark gray, very thin bedded, fissile, calcareous zones
Limestone, gray, fine, thin bedded, fossiliferous
Shale, dark gray, very thin bedded, fissile, calcareous zones
Top of rock elev.= 423.9
Base of weathered rock elev.= 421.0
B-4
210+30.0
Centerline
420.2
D95 (mm)
D50 (mm)
Water
N=9
32 7.8
N=36
S+C=12(10+2)
N=16
U.C. (tsf)
STDROD KYROD REC
60 53 100
482
98 85 100
97 84 100
87 83 100
692
96 80 100
Limestone, light brown to gray, fine grained, thin bedded, weathered, with trace clay
Limestone, light gray, medium to coarsely crystalline grained, thin bedded, fossiliferous, slightly weathered zones
Limestone, (Dolomite), olive gray, fine grained, medium to thick bedded, argillaceous, fresh, hard
Limestone (70%) interbedded with Dolomite (30%):
Limestone is light gray, microcrystalline to medium crystalline grained, very thin to thick bedded, dolimitic, fossiliferous fresh
Dolomite is gray, fine grained, very thin bedded, argillaceous, fresh
Shale (60%) interbedded with Limestone (40%):
Shale is dark gray, silty, very thin to thin bedded, calcareous
Limestone is light gray, very thin to thin bedded, fossiliferous, fresh
Slightly weathered to weathered from 374.9 to 374.0
Top of rock elev.= 409.2
Base of weathered rock elev.= 404.7
440
430
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
SCALE: 1" = 10'
(Vertical Only)
DATE: NOVEMBER 2007
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BORING LOGS
ITEM NUMBER
5-118.00
SHEET G13 OF G18
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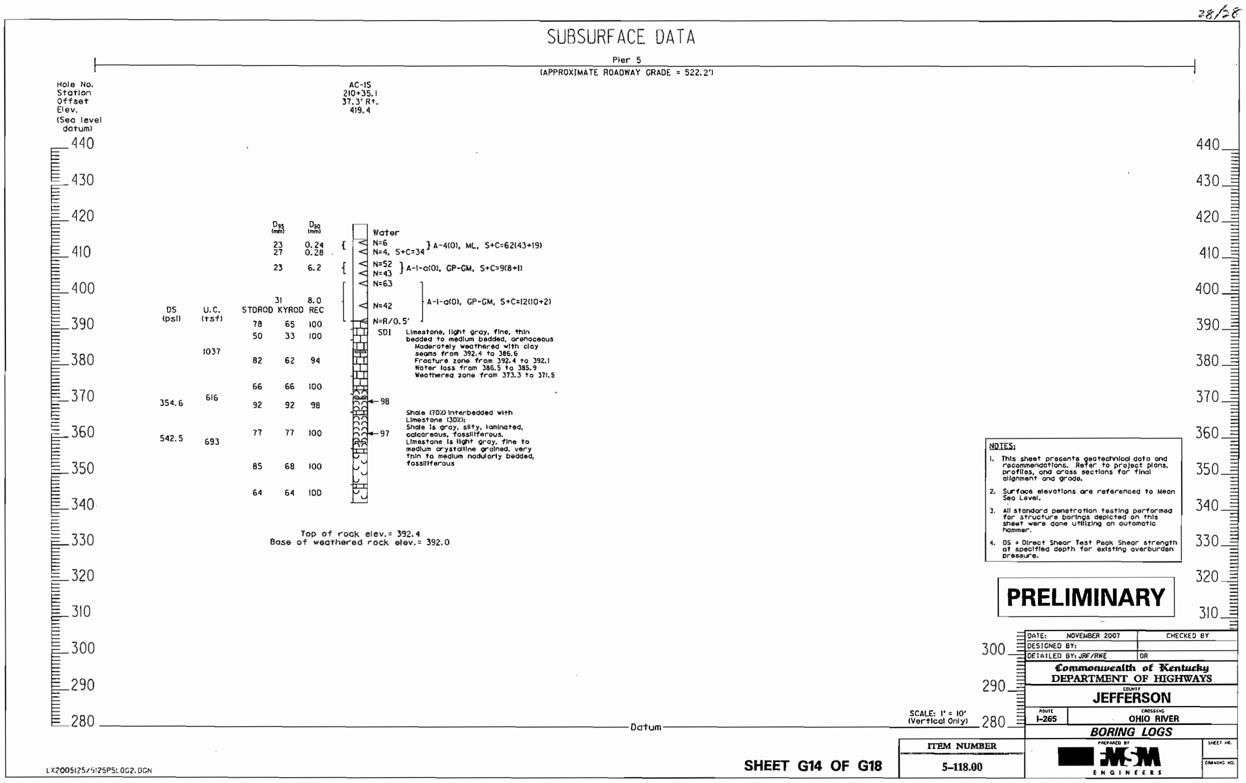
SUBSURFACE DATA
Pier 5
(APPROXIMATE ROADWAY GRADE = 522.2')
Hole No.
Station
Offset
Elev.
(Sea level datum)
AC-15
210+35.1
37.3' Rt.
419.4
D95 (mm)
D50 (mm)
Water
23 0.24
27 0.28
N=6
N=4, S+C=34
A-4(0), ML, S+C=62(43+19)
23 6.2
N=52
N=43
A-1-a(0), GP-GM, S+C=9(8+1)
N=63
31 8.0
N=42
A-1-a(0), GP-GM, S+C=12(10+2)
N=R/0.5'
SDI
DS (psi)
U.C. (tsf)
STDROD KYROD REC
78 65 100
50 33 100
1037
82 62 94
66 66 100
354.6
616
92 92 98
542.5
693
77 77 100
85 68 100
64 64 100
98
97
Limestone, light gray, fine, thin bedded to medium bedded, arenaceous
Moderately weathered with clay seams from 392.4 to 386.6
Fracture zone from 392.4 to 392.1
Water loss from 386.5 to 385.9
Weathered zone from 373.3 to 371.5
Shale (70%) interbedded with Limestone (30%):
Shale is gray, silty, laminated, calcareous, fossiliferous.
Limestone is light gray, fine to medium crystalline grained, very thin to medium nodularly bedded, fossiliferous
Top of rock elev.= 392.4
Base of weathered rock elev.= 392.0
440
430
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
Datum
NOTES:
1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings depicted on this sheet were done utilizing an automatic hammer.
4. DS = Direct Shear Test Peak Shear strength at specified depth for existing overburden pressure.
PRELIMINARY
SCALE: 1" = 10'
(Vertical Only)
DATE: NOVEMBER 2007
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BORING LOGS
ITEM NUMBER
5-118.00
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PARSONS BRINCKERHOFF
COMPUTATION SHEET
Geotechnical & Tunneling Division

BY: M. Du **DATE:** 12/19/2007 **PROJECT:** East End Bridge
CHECKED BY: S. Malhotra **DATE:** 12/24/07 **PAGE** 1 **OF** 14
SUBJECT: Drilled Shaft Vertical Load Calculations for Piers 2 through 5
Preliminary Design Socket Diameter = 8 ft

PURPOSES

To calculate factored resistances for vertical compression and uplift of the drilled shaft using AASHTO Load and Resistance Factor Design procedures, for Piers 2 through 5.

References

- Preliminary design drawings
- Boring logs
- Subsurface soil/rock profiles provided by PB Indianapolis Office.
- AASHTO LRFD Bridge Design Specifications, 4th Edision, 2007.
- Ranges of structural loading provided by PB structural engineers.

Index

- Calculations (pp. 1 ~ 14)
- Idealized soil profiles - not included, see calculation package for 7.5 ft socket diameter
- Subsurface profiles - not included, see calculation package for 7.5 ft socket diameter

UNITS AND CONSTANTS

(Note: These calculations are performed using Mathcad, which does automatic units conversions. Most commonly used engineering and scientific units and constants are internaly defined in Mathcad. User-defined units and constants can also be assigned.)

Reference:L:\Mathcad\UnitsDefinition.xmcd

atmospheric pressure $p_a \equiv atm$ $p_a = 14.696\,psi$

$p_a = 2.116\,ksf$

MATERIAL PROPERTIES

rock quality designation

top 10 ft of rock, RQD mostly between 20% and 50%, say: $RQD_1 := 35\%$

below 10 ft, RQD mostly between 50% and 90%, say: $RQD_2 := 70\%$

uniaxial compressive strength of rock $q_u := 550tsf$ $q_u = 7639\,psi$

concrete compressive strength $fc' := 5000psi$ per structural engineer

DRILLED SHAFT GEOMERY

Vertical loads are designed to be resisted by rock socket alone. Therefore, geometry of rock socket is used in these calculations.

Diameter $D_s := 8ft$

Cross-sectional area $A_p := \frac{\pi}{4} \cdot D_s^2$ $A_p = 50.265\,ft^2$

Length $L_s := \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} ft$ Calculations will be performed for a series of length for developing capacity vs length chart.

$i := 0 .. length(L_s) - 1$ $i = \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \end{pmatrix}$

Note: "i" is an index variables defined for vector and matrix operations.

Shaft circumferential area $A_s := \pi \cdot D_s \cdot L_s$ $A_s = \begin{pmatrix} 125.664 \\ 251.327 \\ 376.991 \\ 502.655 \\ 753.982 \end{pmatrix} ft^2$

SERVICE LIMIT STATE DESIGN

Settlements

All vertical loads are designed to be resisted by rock. Settlements are expected to be negligible.

Horizontal Movements of Shaft and Shaft Groups

Evaluated separately with LPILE.

Settlement Due to Downdrag

No downdrag is anticipated.

Lateral Squeeze

Only applicable for bridge abutment supported on piles installed through soft soils subject to unbalanced embankment fill loading. Not applicable.

STRENGTH LIMIT STATE DESIGN

Effects of water (or groundwater) table, scour, and downdrag are not considered, as the vertical loads are designed to be resisted by rock alone.

Nominal Axial Compression Resistance of Single Drilled Shaft

1. side resistance

from Table 10.4.6.5-1

top 10 ft of rock $RQD_1 = 35\%$ $Em\_Ei_1 := 0.1$ for closed joints

below 10 ft $RQD_2 = 70\%$ $Em\_Ei_2 := 0.7$ for closed joints

from Table 10.8.3.5.4b-1

top 10 ft of rock $\alpha_{E1} := 0.55$

below 10 ft $\alpha_{E2} := 0.88$

Equation 10.8.3.5.4b-1:

top 10 ft of rock: $q_{s1} := \min\left[0.65\cdot\alpha_{E1}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5}, 7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5}\right]$

$q_{s1} = 17.2\text{ ksf}$

$q_{s1} = 119.8\text{ psi}$

check: $0.65\cdot\alpha_{E1}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5} = 17.249\text{ ksf}$

$7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5} = 304.467\text{ ksf}$

below 10 ft: $q_{s2} := \min\left[0.65\cdot\alpha_{E2}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5}, 7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5}\right]$

$q_{s2} = 27.6\text{ ksf}$

$q_{s2} = 191.7\text{ psi}$

check: $0.65\cdot\alpha_{E2}\cdot p_a\cdot\left(\frac{q_u}{p_a}\right)^{0.5} = 27.598\text{ ksf}$

$7.8p_a\cdot\left(\frac{fc'}{p_a}\right)^{0.5} = 304.467\text{ ksf}$

total shaft side resistance

define function to calculate total shaft side resistance:

$$Rs(L_s) := \begin{cases} q_{s1}\cdot(\pi\cdot D_s\cdot L_s) & \text{if } L_s \le 10\text{ft} \\ q_{s1}\cdot(\pi\cdot D_s\cdot 10\text{ft}) + q_{s2}\cdot\left[\pi\cdot D_s\cdot(L_s - 10\text{ft})\right] & \text{otherwise} \end{cases}$$

$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix}\text{ft}$ $R_{s_i} := Rs(L_{s_i})$ $R_s = \begin{pmatrix} 2168 \\ 4335 \\ 7803 \\ 11271 \\ 18207 \end{pmatrix}\text{kip}$

note: "i" is the index variable defined above

nominal shaft side resistance

2. Tip Resistance

Note: the Rock Mass Rating (RMR) will be evaluated during final design by FMSM. Fur preliminary design, the following rock mass qualities are assumed based on general descriptions in the boring logs.

top 10 ft (upper) poor to fair rock

below 10 ft (lower) fair to good rock

from Table 10.4.6.4-4

top 10 ft poor to fair quality rock mass, rock classes A&B (lime stone 60%, shale 40%)

$m_1 := 0.9$ in between the categories $s_1 := 0.000015$

below 10 ft fair to good quality rock mass, rock classes A&B (lime stone 60%, shale 40%)

$m_2 := 0.5$ in between the categories $s_2 := 0.0005$

Equation 10.8.3.5.4c-2 (lower bound values)

top 10 ft of rock $q_{p1} := \left(\sqrt{s_1} + \sqrt{m_1 \cdot \sqrt{s_1} + s_1}\right) \cdot q_u$

$q_{p1} = 69.344\,\text{ksf}$

below 10 ft $q_{p2} := \left(\sqrt{s_2} + \sqrt{m_2 \cdot \sqrt{s_2} + s_2}\right) \cdot q_u$

$q_{p2} = 143.48\,\text{ksf}$

total shaft tip resistance

define function to calculate total shaft tip resistance:

$$Rp(L_s) := \begin{cases} q_{p1} \cdot A_p & \text{if } L_s < 10\text{ft} \\ q_{p2} \cdot A_p & \text{otherwise} \end{cases}$$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad R_{p_i} := Rp\left(L_{s_i}\right) \qquad R_p = \begin{pmatrix} 3486 \\ 7212 \\ 7212 \\ 7212 \\ 7212 \end{pmatrix} \text{kip}$$

note: "i" is the index variable defined above

nominal shaft tip resistance

Total Nominal Resistance

$$R_s + R_p = \begin{pmatrix} 5653 \\ 11547 \\ 15015 \\ 18483 \\ 25419 \end{pmatrix} \text{kip} \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft}$$

Factored Axial Compression Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

$\phi := 0.85$ however , $\phi <= 0.70$

use : $\phi_{qp} := 0.70$ for tip resistance

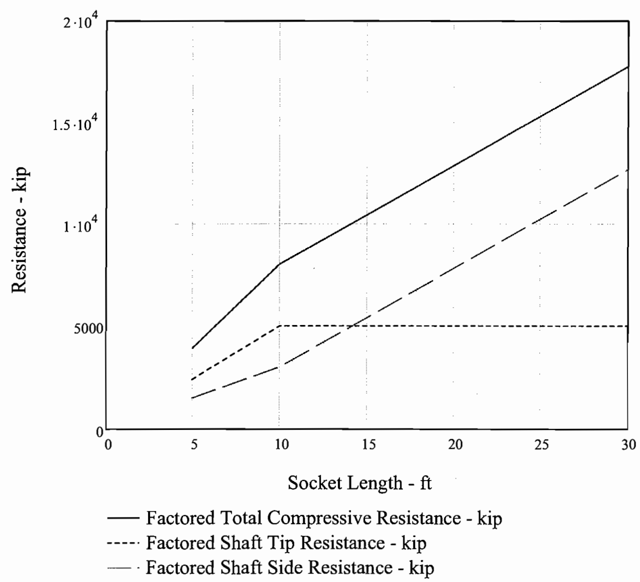
$\phi_{qs} := 0.70$ for side resistance

Factored total resistance of drilled shaft $R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s$

$$R_R = \begin{pmatrix} 3957.2 \\ 8083 \\ 10510.6 \\ 12938.2 \\ 17793.5 \end{pmatrix} \text{kip}$$

Factored Rock Socket Compression Resistance versus Socket Length

Socket Diameter $D_s = 8\ \text{ft}$



Nominal Skin Friction	$q_{s1} = 119.8\ \text{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\ \text{psi}$	below 10 ft
	resistance factor	$\phi_{qs} = 0.7$
Nominal End Bearing	$q_{p1} = 34.7\ \text{tsf}$	top 10 ft of rock
	$q_{p2} = 71.7\ \text{tsf}$	below 10 ft
	resistance Factor	$\phi_{qp} = 0.7$

Extreme Limit States

Section 10.5.5.3.3

For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used:

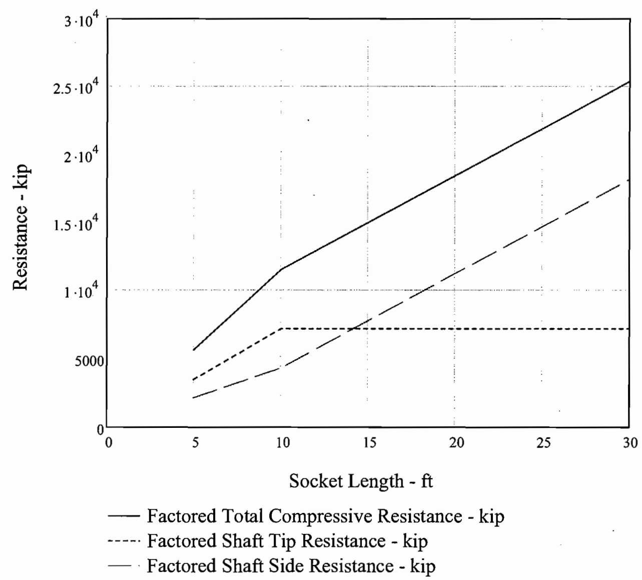
$\phi_{qs} := 1.0$ $\phi_{qp} := 1.0$

Factored total resistance of drilled shaft

$$R_R := \phi_{qp} \cdot R_p + \phi_{qs} \cdot R_s \qquad R_R = \begin{pmatrix} 5653.1 \\ 11547.1 \\ 15015.1 \\ 18483.2 \\ 25419.2 \end{pmatrix} \text{kip}$$

Factored Rock Socket Compression Resistance versus Socket Length - Extreme Limit States

Socket Diameter $D_s = 8\ \text{ft}$



Nominal Skin Friction	$q_{s1} = 119.8\ \text{psi}$	top 10 ft of rock
	$q_{s2} = 191.7\ \text{psi}$	below 10 ft
	resistance factor	$\phi_{qs} = 1$
Nominal End Bearing	$q_{p1} = 34.7\ \text{tsf}$	top 10 ft of rock
	$q_{p2} = 71.7\ \text{tsf}$	below 10 ft
	resistance Factor	$\phi_{qp} = 1$

Nominal Axial Uplift Resistance of Single Drilled Shaft

According to 10.8.3.7.2, the uplift resistance of a straight-sided shaft is similar to that for the side resistance in compression, as calculated above.

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad R_s = \begin{pmatrix} 2168 \\ 4335 \\ 7803 \\ 11271 \\ 18207 \end{pmatrix} \text{kip}$$

nominal shaft side resistance in uplift

weight of shaft

$$W_s := (\gamma_{conc} - \gamma_w) \cdot A_p \cdot L_s \qquad A_p = 50.265\ \text{ft}^2 \qquad \gamma_{conc} = 150\ \text{pcf}$$

$$\gamma_w = 62.4\ \text{pcf}$$

$$L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft} \qquad W_s = \begin{pmatrix} 22 \\ 44 \\ 66 \\ 88 \\ 132 \end{pmatrix} \text{kip}$$

Total Nominal Uplift Resistance

$$R_s + W_s = \begin{pmatrix} 2190 \\ 4379 \\ 7869 \\ 11359 \\ 18339 \end{pmatrix} \text{kip} \qquad L_s = \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 30 \end{pmatrix} \text{ft}$$

Factored Axial Uplift Resistance of Single Drilled Shaft

Resistance Factors, based on tables 10.5.5.2.4-1 and 10.5.5.2.3-2

There will be an Osterberg Cell load test (serving both compression and uplift purposes) at each pier locations, therefore a total of 3 load tests for this "site" (piers 2 through 4). For a medium site variability:

$\phi := 0.85$ however , $\phi <= 0.60$

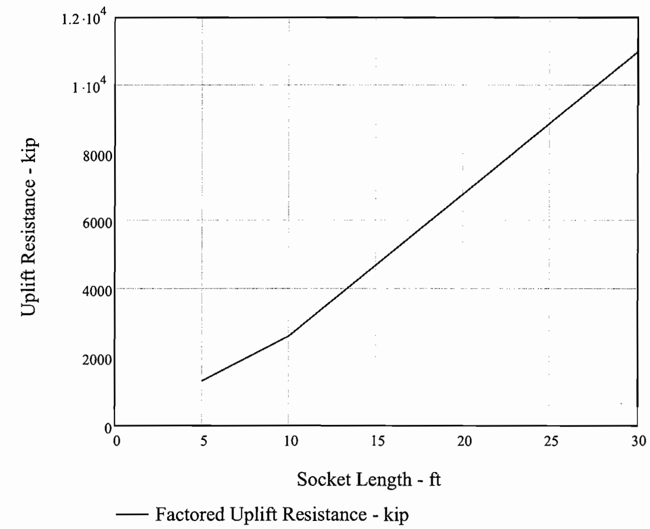
use : $\phi_{up} := 0.60$ for uplift resistance

Factored uplift resistance of drilled shaft $R_R := \phi_{up} \cdot (R_s + W_s)$

$$R_R = \begin{pmatrix} 1314 \\ 2627 \\ 4721 \\ 6815 \\ 11004 \end{pmatrix} kip$$

Factored Rock Socket Uplift Resistance versus Socket Length

Socket Diameter $D_s = 8\,ft$



Nominal Uplift Resistance $q_{s1} = 119.8\,psi$ top 10 ft of rock

$q_{s2} = 191.7\,psi$ below 10 ft

weight of shaft $\gamma_{conc} - \gamma_w = 87.6\,pcf$

resistance factor $\phi_{up} = 0.6$

Extreme Limit States for Uplift

Section 10.5.5.3.3

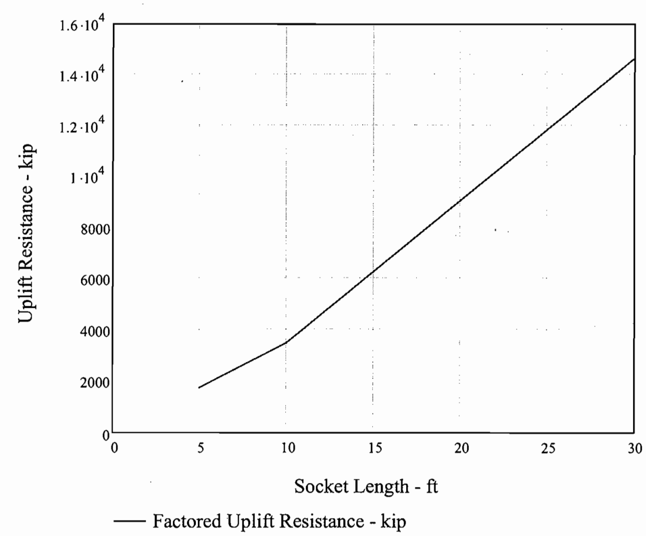
For extreme limit states (earthquake, ice, vehicle impact, or vessel impact, etc.), the following resistance factor is used for uplift:

$\phi_{up} := 0.8$

Factored uplift resistance of drilled shaft $\quad R_R := \phi_{up} \cdot (R_s + W_s) \quad\quad R_R = \begin{pmatrix} 1752 \\ 3503 \\ 6295 \\ 9087 \\ 14671 \end{pmatrix} \text{kip}$

Factored Rock Socket Uplift Resistance versus Socket Length - Extreme Limit States

Socket Diameter $\quad D_s = 8 \text{ ft}$



Nominal Uplift Resistance	$q_{s1} = 119.8 \text{ psi}$	top 10 ft of rock
	$q_{s2} = 191.7 \text{ psi}$	below 10 ft
	weight of shaft	$\gamma_{conc} - \gamma_w = 87.6 \text{ pcf}$
	resistance factor	$\phi_{up} = 0.8$

APPENDIX H-2
DRILLED SHAFT LATERAL LOAD CALCULATIONS

Purpose

To evaluate lateral load deformation behavior of drilled shafts, and to determine minimum rock socket length to provide fixity.

References

1. Preliminary Design Plans, December 2007.
2. Boring logs
3. Idealized soil profiles
4. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 4th Edition, 2007.
5. LPILE Plus 5.0, Ensoft, Inc. 2007.

Index

Design Methodology

Analytical Procedures

Lateral load analysis procedures specified in AASHTO LRFD Bridge Design Specifications (4th Edition, 2007) was used. Soil-structural interaction analyses for the drilled shafts subject to vertical and lateral loads and bending moments are carried out using software program LPILE, which models the shaft as a bending member and the surrounding soil and rock material s non-linear springs (the p~y method). Deflection, bending moment, shear force, and soil reaction pressure are calculated along the shaft length.

LRFD Resistance Factor

Per Table 10.5.5.2.4-1, the horizontal geotechnical resistance factor for single shaft or shaft group should be 1.0. In addition, per section 10.7.3.12 (for piles, also applicable to drilled shaft per 10.8.3.8), the minimum penetration of the piles (shaft) below ground should be such that the fixity is obtained.

Minimum Rock Socket Length Required for Fixity

One purpose of the calculations is to find the minimum required socket length to provide fixity of the shaft. In these calculations, it is assumed that the fixity is achieved with a certain rock socket length, beyond which increasing the rock socket will have no significant effects on the drilled shaft behavior under lateral loads and bending moment.

Scour

Analyses were performed for the cases without scour and with the maximum predicted scour. Either case can be critical under different conditions. In addition, some extreme load cases, such as earthquake, are usually applied with one-half of the maximum scour. The half-scour case was not analyzed for this preliminary stage, as it can be approximated by interpolating between the cases of no scour and maximum scour.

Shaft Diameter

Two shaft sizes are analyzed: (1) 8.5 ft shaft with a 8 ft rock socket, and (2) 8 ft shaft with a 7.5 ft rock socket.

Shaft Head Fixity

For the main tower piers (Piers 3 & 4), the shafts are in large groups arranged in an elliptical pattern in plan. The shaft head is assumed fixed against rotation. For the other piers, the shafts are arranged in a single row in the transverse direction; therefore, the shaft head is not fixed in the longitudinal direction, and is assumed free to rotate in LPILE analysis.

Group Effect

Per 10.7.2.4 (for piles, also applicable to drilled shafts), the group effect for horizontal loading should be modeled with a P-multiplier in the p~y curves. If the shafts are spaced at a center-to-center spacing of 3 times diameter, P-multipliers of 0.7, 0.5 and 0.35 should be applied on the leading row, second row, and other rows of shafts, respectively. For the large shaft groups supporting Piers 3 & 4, most of the shafts are in the 3rd row or higher, therefore, a P-multiplier of 0.35 is applied, conservatively. For the shafts supporting other piers, a P-multiplier of 0.7 was applied as all the shafts are in the first row.

Du, Mangtao

From: Dwyre, Elizabeth
Sent: Tuesday, December 18, 2007 7:22 PM
To: Du, Mangtao
Cc: Castelli, Raymond J.; Hsu, Ruchu; Bryson, John
Subject: RE: EEB: Analyses - Shear and Moment- Drilled Shaft Diameter Discussion / Pile Head Demands - Shear and Moment

Monty,
These are the load cases I am suggesting you run in LPILE, based on John's spreadsheet summary of shear and moment. I don't think it's necessary to run a large suite of loads for these analyses, since we will need to do further work in final design. Let me know if these load cases seem reasonable to you.

Simplified analysis cases, Piers 3& 4 (Tower piers, just run one location) P=12,000 k

V (k)	M (k-ft)
1,000	40,000
1,250	50,000
1,500	60,000

Simplified analysis cases, Piers 1& 2 (need to run both piers since Pier 1 is on land and Pier 2 is in water P= 12,000 k

V	M
200	4,000
225	5,000
250	6,000

LOAD CASES USED IN LPILE ANALYSES

Simplified analysis cases, Pier 5
P=4,000 k

V	M
300	11,000
400	12,000
500	13,000

Liza
Elizabeth M. Dwyre, P.E.
PB
(317) 287-3406 direct
(317) 752-0917 cell
(317) 972-1706 x 3406 office
www.pbworld.com

-----Original Message-----
From: Bryson, John
Sent: Tuesday, December 18, 2007 10:46 AM
To: Dwyre, Elizabeth
Cc: Castelli, Raymond J.; Du, Mangtao; Hsu, Ruchu
Subject: RE: EEB: Analyses - Drilled Shaft Diameter Discussion / Pile Head Demands

Liza,

Attached are the pile head loads including the shear forces from our current global analysis.

By "pile head loads", we mean the forces and moments in the drilled shaft directly beneath the tremie seal for the main tower foundations and at the column/drilled shaft transition for the transition and anchor piers (Piers 1, 2 and 5).

Please note that the attached forces and moments are factored loads, per AASHTO LRFD, for the Strength I through V limit states and the Extreme Event I limit state (seismic load

Du, Mangtao

From: Bryson, John
Sent: Tuesday, December 18, 2007 12:50 PM
To: Dwyre, Elizabeth
Cc: Castelli, Raymond J.; Du, Mangtao; Hsu, Ruchu
Subject: RE: EEB: Analyses - Drilled Shaft Diameter Discussion / Drilled Shaft Section Properties

Drilled Shaft Section Properti...

Liza,

Attached are our calculations for the drilled shaft section properties for the two shaft/socket sizes that you are considering for your report. The properties for the larger shaft option (8'-6" drilled shaft/8'-0" rock socket) are consistent with those used in our latest global (LARSA) analysis. You may use these section properties for your L-Pile analyses.

These section properties are based on 5000 psi concrete with a 3/4-inch casing thickness and an effective (cracked) section of 65% Igross for the concrete in the cased section. The concrete in the rock socket portion is assumed to be uncracked.

The section properties are summarized below.

```
Option 1: 8'-6" drilled shaft with 8'-0" rock socket
      8'-6" OD drilled shaft (cased portion):
            E = 4,074,281 psi
            G = 1,697,617 psi
            Ax =  9,630.78 in^2
            Iz =  5,431,065 in^4
            Density = 0.078525 lb./in^3  (dry density)
      8'-0" OD rock socket (uncased portion):
            E = 4,074,281 psi
            G = 1,697,617 psi
            Ax =  7,238.23 in^2
            Iz =  4,169,220 in^4
            Density = 0.086806 lb./in^3  (dry density)

Option 2: 8'-0" drilled shaft with 7'-6" rock socket
      8'-0" OD drilled shaft (cased portion):
            E = 4,074,281 psi
            G = 1,697,617 psi
            Ax =  8,611.24 in^2
            Iz =  4,356,263 in^4
            Density = 0.078093 lb./in^3  (dry density)
      7'-6" OD rock socket (uncased portion):
            E = 4,074,281 psi
            G = 1,697,617 psi
            Ax =  6,361.73 in^2
            Iz =  3,220,623 in^4
            Density = 0.086806 lb./in^3  (dry density)
```

DRILLED SHAFT SECTION PROPERTIES USED IN LPILE ANALYSES

Thanks,

John A. Bryson
PB Americas, Inc.
One Penn Plaza
New York, NY 10119
tel: (212) 465-5336
fax: (212) 465-5575
cell: (347) 326-4030

Summary of Lateral Load Analyses for Drilled Shafts

	Shaft Head Elevation	Existing Ground/ Mud Line Elevation	Bed Rock Elevation	Max. Scour	Head Fixity	Max. Applied Shear	Max. Applied Moment	Applied Vertical Load	Shaft Diameter	Socket Diameter	Scour Condition	Min. Length of Rock Socket to Provide Fixity (Note 2)	Max. Head Deflection (Note 3)	Max. Resulting Bending Moment: Cased Section	Max. Resulting Bending Moment: Rock Socket	LPILE Run No.
	(ft)	(ft)	(ft)	(ft)		(kip)	(ft-kip)	(kip)	(ft)	(ft)		(ft)	(in)	(ft-kip)	(ft-kip)	
Pier 1	430	434	334	16	Free	250	6,000	12,000	8.5	8	No Scour	0	0.9	10,005	686	Run 1
											Max Scour	0	1.9	14,598	1,704	Run 2
									8	7.5	No Scour	0	1.1	10,083	523	Run 3
											Max Scour	0	2.4	15,070	1,598	Run 4
Pier 2	414.8	404.1	334.9	14	Free	250	6,000	12,000	8.5	8	No Scour	0	1.3	12,601	1,620	Run 5
											Max Scour	5	2.9	17,725	1,437	Run 6
									8	7.5	No Scour	0	1.5	12,782	1,665	Run 7
											Max Scour	5	3.6	18,293	647	Run 8
Piers 3&4	405.5	379.4	332	40	Fixed	1,500	*Note 1*	12,000	8.5	8	No Scour	8	4.3	57,350	36,853	Run 9
											Max Scour	24	5.4	63,330	52,651	Run 10
									8	7.5	No Scour	13	5.4	57,743	36,882	Run 11
											Max Scour	25	6.9	64,208	53,349	Run 12
Pier 5	414.8	415.2	392	17	Free	500	13,000	4,000	8.5	8	No Scour	10	0.7	23,535	23,535	Run 13
											Max Scour	15	0.8	24,751	24,751	Run 14
									8	7.5	No Scour	10	0.9	23,496	23,496	Run 15
											Max Scour	15	1.0	24,806	24,806	Run 16

Note 1: For Piers 3 & 4, since the shaft head is assumed to be fixed against rotation, no bending moment is applied as a load at the shaft head. The actual bending moment occuring at the head will be a resulting moment due to the head fixity and the shear force.

Note 2: For these calculations, the minimum length of rock socket to provide fixity is assumed to be a socket length beyond which increasing the socket length will have no significant effect on the shaft's behavior under lateral loads.

Note 3: The maximum head deflection is computed at the shaft head levels, which were assumed as follows based on discussions with the structural engineer:

Pier 1: Elev. 430 -- bottom of cap beam, approximately 4 feet below existing ground surface.
Piers 2 & 5: Elev. 414.8 -- bottom of cap beam according to Sheet S6 in the Preliminary Design Plans, December 2007.
Piers 3 & 4: Elev. 405.5 -- bottom of the tremie seal according to Sheet S2 in the Preliminary Design Plans, December 2007.

BY: MDU
DATE: 12/21/2007
CHK BY: ______
DATE: ______

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Boring Profile 12/10/2007

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 1 - STA 187+40, CL
Borings AC-1, 2, 3

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Classification)	Parameters	
434.0	0.0	Lean Clay (CL)	γ_t (lb/ft³) = 121 γ_e (lb/ft³) = 59 q_U (lb/ft²) = 1784 C_U (lb/ft²) = 892	K_S (lb/in³) = 100 D_{50}(mm) = 0.03 D_{95}(mm) = 0.19
418.3	15.7			
409.0	25.0	Sand with Silt (SM, SW-SM, and SP-SM)	γ_e (lb/ft³) = 55 ϕ' (°) = 32.5 K_S (lb/in³) = 60	D_{50}(mm) = 0.90 D_{95}(mm) = 20
379.0	55.0	Sand (SW-SM, SW and SP-SM)	γ_e (lb/ft³) = 64 ϕ' (°) = 34.8 K_S (lb/in³) = 60	D_{50}(mm) = 0.76 D_{95}(mm) = 8.00
334.0	100.0	Top of Rock Limestone(60%) Interbedded with Shale(40%). Limestone is gray, fine grained, thin bedded and argillaceous. Shale is gray, silty, and calcareous.	γ_t (lb/ft³) = 165 SDI (%) = 80 q_u(ton/ft²) = 411 c (lb/in²) = 300 ϕ' (°) = 28.0	
283.1	150.9			

4' BELOW GRADE
-4' (-48")
TOP OF SHAFT
430
MAX. SCOUR LINE
16' of scour
0' 0"
11.7' 140"
12' 144"
21' 252"
51' 612"
96' 1152"
111' 1332"
146.9' 1763"

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Boring Profiles
12/10/2007

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GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 2 - STA 189+65.5, CL
Borings AC-4, 5, B-1

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Description)	Parameters		
~~[illegible]~~	~~[illegible]~~	~~Sandy Lean Clay (CL)~~	~~γ (lb/ft³) = [illegible]~~ ~~K_S (lb/in³) = 38~~	~~D_{50}(mm) = 0.060~~ ~~D_{85}(mm) = 0.37~~	
~~[illegible]~~	~~[illegible]~~				
~~[illegible]~~	6.1	~~Sand with Silt~~ (SM)	~~γ_t (lb/ft³) = [illegible]~~ ~~γ' (lb/ft³) = 45~~ ~~ϕ' (°) = 28.0~~ ~~K_S (lb/in³) = 25 (Above Water Table)~~ K_S (lb/in³) = 20 (Below Water Table)	~~D_{50}(mm) = 0.11~~ ~~D_{85}(mm) = 0.65~~	TOP OF SHAFT EL. 414.8 0'	0"
~~[illegible]~~ 400.1	~~[illegible]~~	Sand (SW, SW-SM) MUDLINE	γ_e (lb/ft³)* = 68; ϕ' (°) = 35.2; K_S (lb/in³) = 60	D_{50}(mm) = 1.6; D_{85}(mm) = 19; 14' OF MAX. SCOUR	10.7' 128"; (24.7') (296")
379.9	49.0	Sand with gravel (SW-SM, SP, SP-SM, GP-GM,)	γ_e (lb/ft³)* = 65; ϕ' (°) = 35.8; K_S (lb/in³) = 60	D_{50}(mm) = 2.4; D_{85}(mm) = 18	34.9' 419"
334.9	94.0	Top of Rock. Limestone (55%) interbedded with Shale(45%). Limestone is gray, fine-grained, thin bedded, argillaceous and fossiliferous. Shale is gray, silty, laminated to thin bedded, calcareous and fossiliferous.	γ_t (lb/ft³) = 165; SDI (%) = 73; q_u(ton/ft²) = 563; c (lb/in²) = 300; ϕ' (°) = 28.0		79.9' 959"; 99.9' 1199"
282.6	146.3	Shale (70%) interbedded with Limestone (30%). Shale is gray, fine grained, thin bedded, silty, calcareous, fossiliferous. Limestone is gray, microcrystalline to fine grained, thin bedded, argillaceous, fossiliferous.	γ_t (lb/ft³) = 160; c (lb/in²) = 300; ϕ' (°) = 28.0		132.2' 1586"
275.9	153.0				

Boring Profiles
12/10/2007

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GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 3 - STA 193+77.5, CL
Borings AC-6, 7, 8, 9, B-2

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Description)	Parameters		
419.4	0.0	Water - Ohio River	TOP OF SHAFT = EL. 405.5'		z = 0' 0"
379.4	40.0	Sand (SW-SM, SP, SW)	γ_e (lb/ft³)* = 68; ϕ' (°) = 34.5; K_S (lb/in³) = 60	D_{50}(mm) = 0.98; D_{85}(mm) = 17; DRILLED SHAFT	26.1' 313"
365.4	54.0	Sand (SP-SM, SW-SM)	γ_e (lb/ft³)* = 67; ϕ' (°) = 36.9; K_S (lb/in³) = 60	D_{50}(mm) = 0.62; D_{85}(mm) = 18	40.1' 481"
354.4	65.0	Gravel (GW, GW-GM)	γ_e (lb/ft³)* = 71; ϕ' (°) = 38.0; K_S (lb/in³) = 125	D_{50}(mm) = 11; D_{85}(mm) = 30; MAX. 40' SCOUR.	51.1' 613"
339.4	80.0	Sand (SP-SM, SM)	γ_e (lb/ft³)* = 69; ϕ' (°) = 38.0; K_S (lb/in³) = 125	D_{50}(mm) = 1.3; D_{85}(mm) = 19	66.1' 793"
332.0	87.4	Top of Rock. Limestone (60%) interbedded with Shale (40%). Limestone is gray, microcrystalline to fine grained, thin bedded, fossiliferous and argillaceous. Shale is gray, silty, laminated to thin bedded, calcareous, fossiliferous.	γ_t (lb/ft³) = 164; SDI (%) = 67; q_u(ton/ft²) = 647; c (lb/in²) = 300; ϕ' (°) = 28.0	35' SOCKET	73.5' 882"; 108.5' 1302"
278.6	140.8				126.9 1523"

Boring Profiles
12/10/2007

9

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 4 - STA 206+12.5, CL
Borings AC-10, 11, 12, 13, B-3

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Description)	Parameters (Description STRATA)	
418.8	0.0	Water - Ohio River		
378.8	40.0	Gravel (GW, GP)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 35.5 K_S (lb/in^3) = 20	D_{50}(mm) = 9.1 D_{85}(mm) = 27
359.8	59.0	Sand (SP-SM, SW-SM, SP)	γ_e (lb/ft^3)* = 68 ϕ' (°) = 37.0 K_S (lb/in^3) = 125	D_{50}(mm) = 2.4 D_{85}(mm) = 18
343.8	75.0	Gravel (GP-GM, GM)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 3.9 D_{85}(mm) = 23
336.2	82.6	Top of Rock Limestone (60%) interbedded with Shale (40%). Limestone is gray, microcrystalline to fine grained, thin, wavy to nodular bedded, fossiliferous, and argillaceous. Shale is gray, silty, laminated to thin bedded, calcareous, and fossiliferous.	γ_t (lb/ft^3) = 165 SDI (%) = 74 q_u(ton/ft^2) = 550 c (lb/in^2) = 300 ϕ' (°) = 28.0	
282.8	136.0			

10

GENERAL SOIL AND BEDROCK PROFILE

I-265 Over Ohio River
Pier 5 - STA 210+24.5, CL
Borings AC-14, AC-15, B-4

AC-14

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Description)	Parameters (Description STRATA)	
436.0	0.0	(AC-14) Sandy Lean Clay (CL)	γ_t (lb/ft^3) = 120 q_u (lb/ft^2) = 2000 C_u (lb/ft^2) = 1000	K_S (lb/in^3) = 100 D_{50}(mm) = [illegible] D_{85}(mm) = 4.00
420.9	12.1	Top of Rock (AC-14) Limestone, gray, medium grained, thin bedded to medium bedded.	γ_t (lb/ft^3) = 164 q_u(ton/ft^2) = 600 c (lb/in^2) = 10 ϕ' (°) = 20.0	
417.0	19.0	(AC-14) Limestone (60%) interbedded with Shale (40%). Limestone is gray, fine to medium grained, very thin to medium wavy bedded. Shale is gray, silty	γ_t (lb/ft^3) = 164 SDI (%) = 98 q_u(ton/ft^2) = 650 c (lb/in^2) = 150 ϕ' (°) = 27.0	
407.0	29.0	(AC-14) Shale, gray to red, very thin bedded to thin bedded, silty	γ_t (lb/ft^3) = 140 SDI (%) = 0 q_u(ton/ft^2) = 18 c (lb/in^2) = 5 ϕ' (°) = 20.0	
404.0	32.0	(AC-14) Limestone, light gray, fine grained, thin bedded, shale stringers, streaks, and partings	γ_t (lb/ft^3) = 160 q_u(ton/ft^2) = 886 c (lb/in^2) = 150 ϕ' (°) = 32.0	
394.8	41.8	(AC-14) Dolomite Limestone, greenish gray, fine, thin bedded	γ_t (lb/ft^3) = 168 q_u(ton/ft^2) = 800 c (lb/in^2) = 150 ϕ' (°) = 32.0	
392.0	44.0	Limestone (30% - 60%) interbedded with Shale (40% - 70%). Limestone is gray, fine to medium crystalline grained, very thin to medium nodularly bedded, fossiliferous. Shale is gray, silty, laminated, calcareous, fossiliferous.	γ_t (lb/ft^3) = 168 SDI (%) = 97 q_u(ton/ft^2) = 782 c (lb/in^2) = 100 ϕ' (°) = 25.0	
342.0	94.0			

AC-15

Approximate Elevation (ft)	Approximate Depth (ft)	Description (USCS Description)	Parameters (Description Strata)	
418.4	0.0	Water - Ohio River		
410.2	4.2	Sandy Silt with Gravel (ML)	γ_e (lb/ft^3)* = [illegible] ϕ' (°) = 29.0 K_S (lb/in^3) = 20	D_{50}(mm) = 0.26 D_{85}(mm) = 10
409.4	10	Poorly graded gravel with silt and sand (GP-GM)	γ_e (lb/ft^3)* = 71 ϕ' (°) = 38.0 K_S (lb/in^3) = 125	D_{50}(mm) = 7.1 D_{85}(mm) = 27
392.0	27.4	Top of Rock		

TOP OF SHAFT ≃ 414.8'

-0.4' 0

-5'' 0''

5.4' 65''

MAX 17' OF SCOUR

16.6' 199''

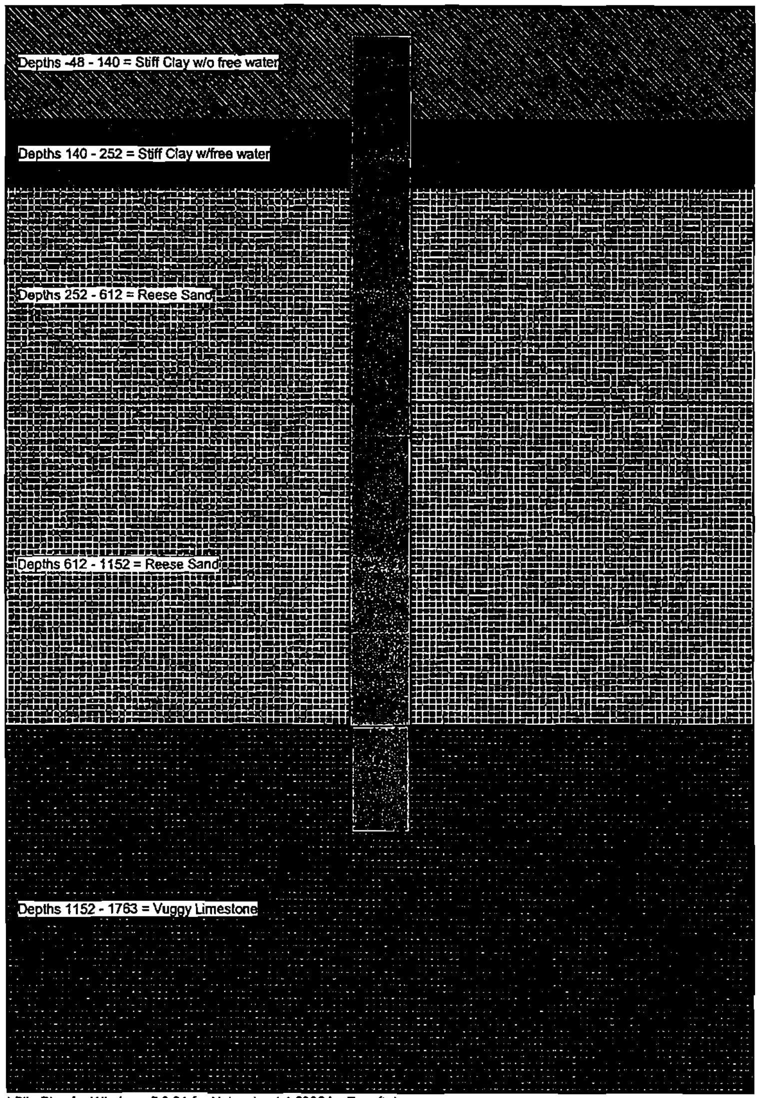
22.8' 274''

47.8' 574''

72.8' 874''

RUN 1: PIER 1
8.5' SHAFT, NO SCOUR

11



LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

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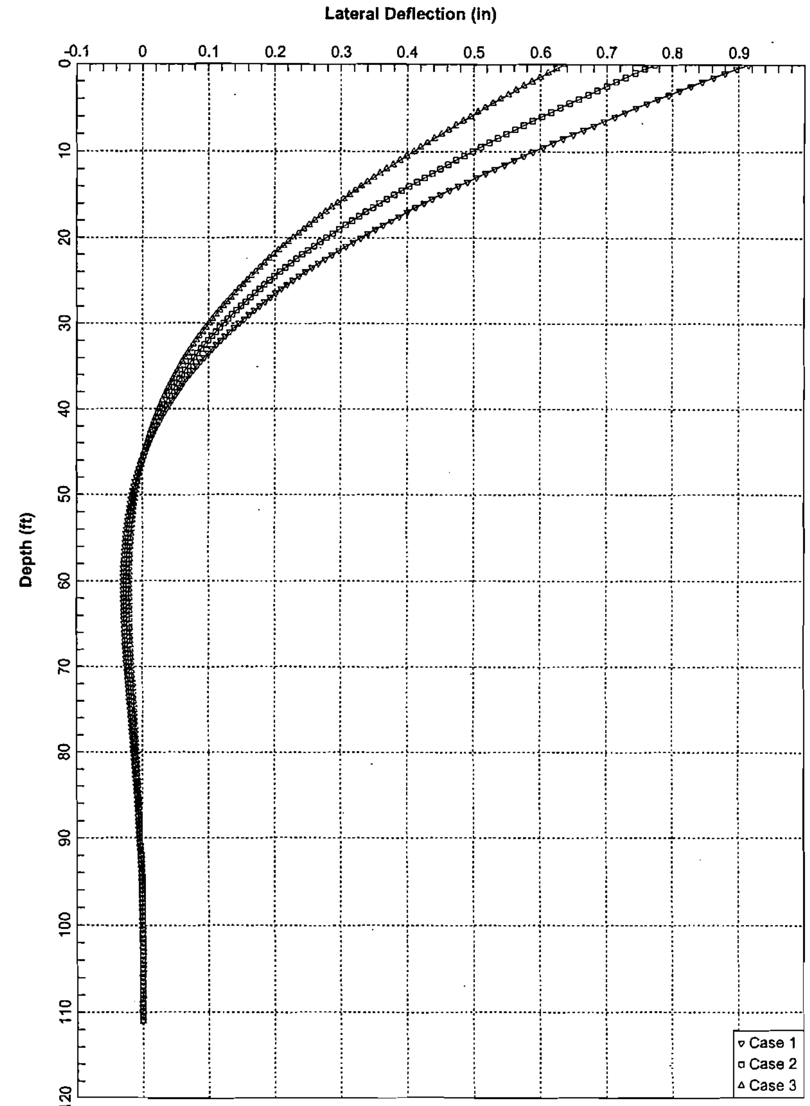

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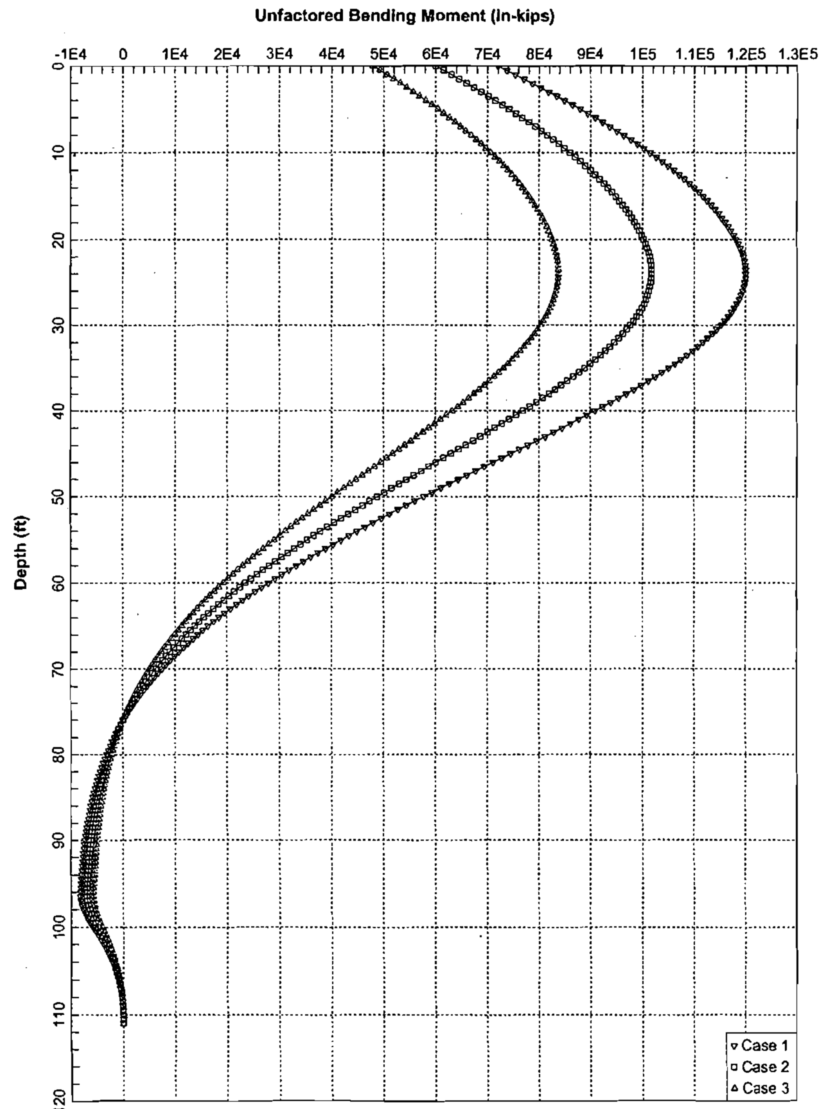
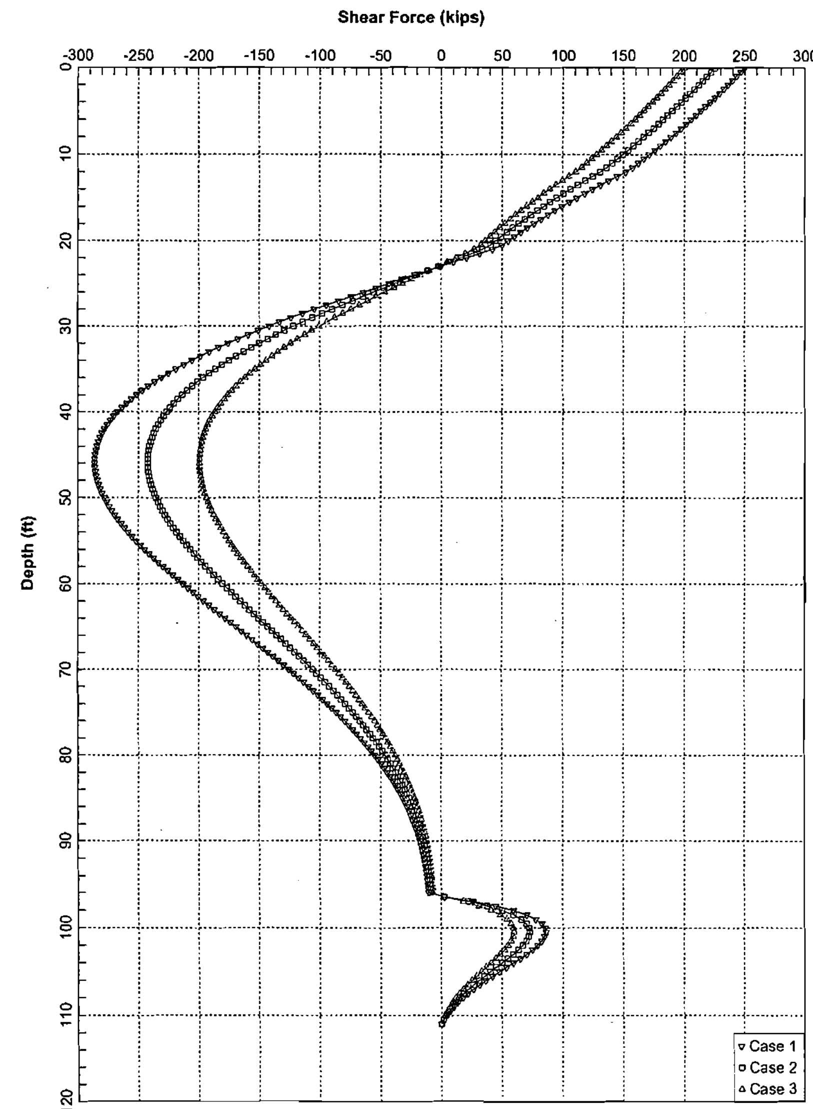


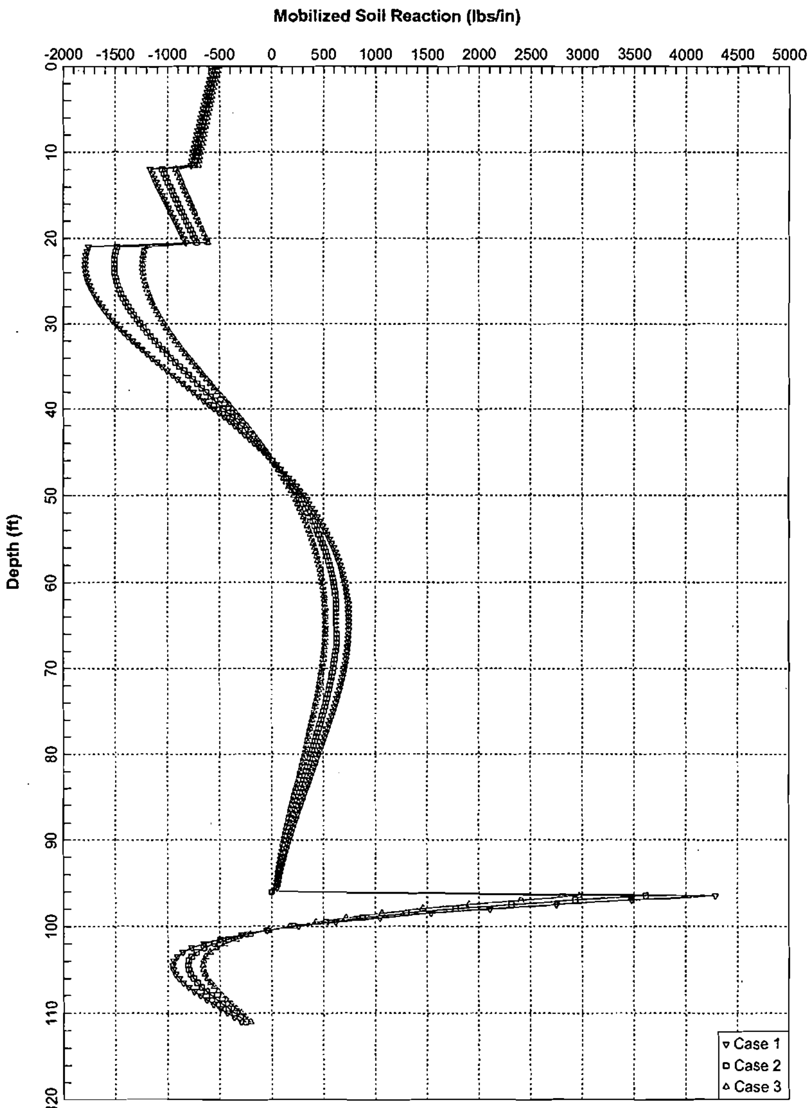
Unfactored Bending Moment (In-kips)
-1E4
0
1E4
2E4
3E4
4E4
5E4
6E4
7E4
8E4
9E4
1E5
1.1E5
1.2E5
1.3E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

14



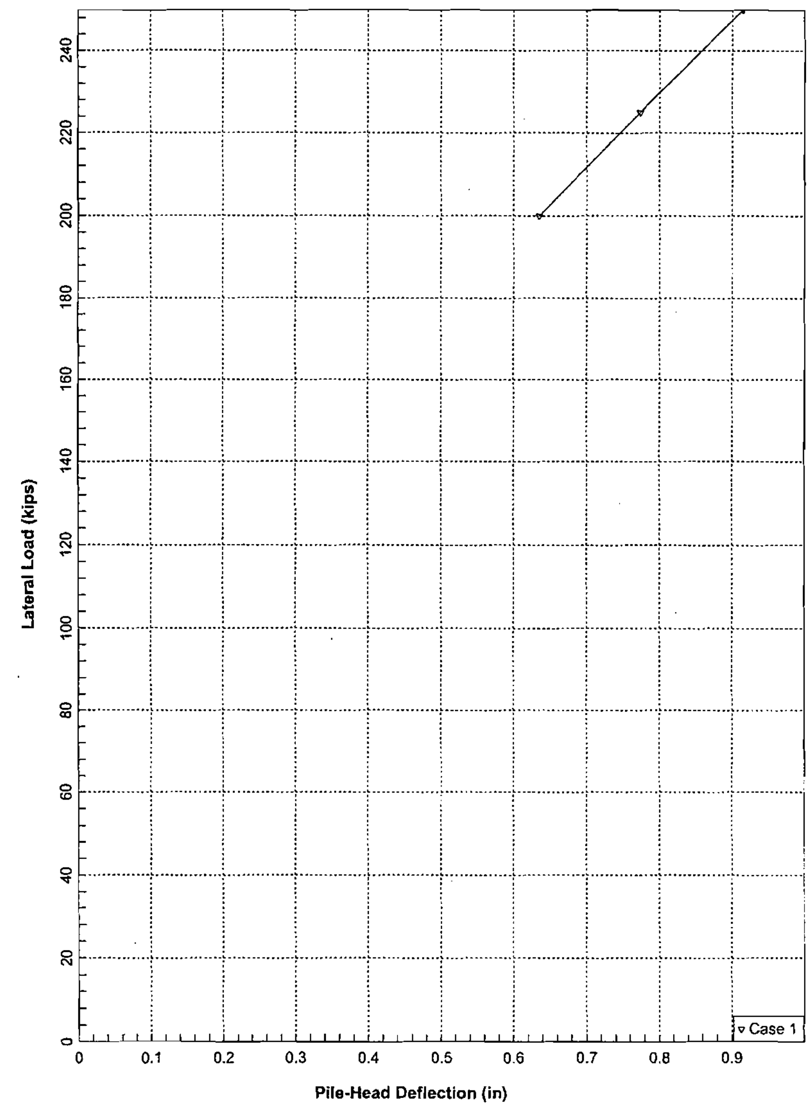
Shear Force (kips)
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

15

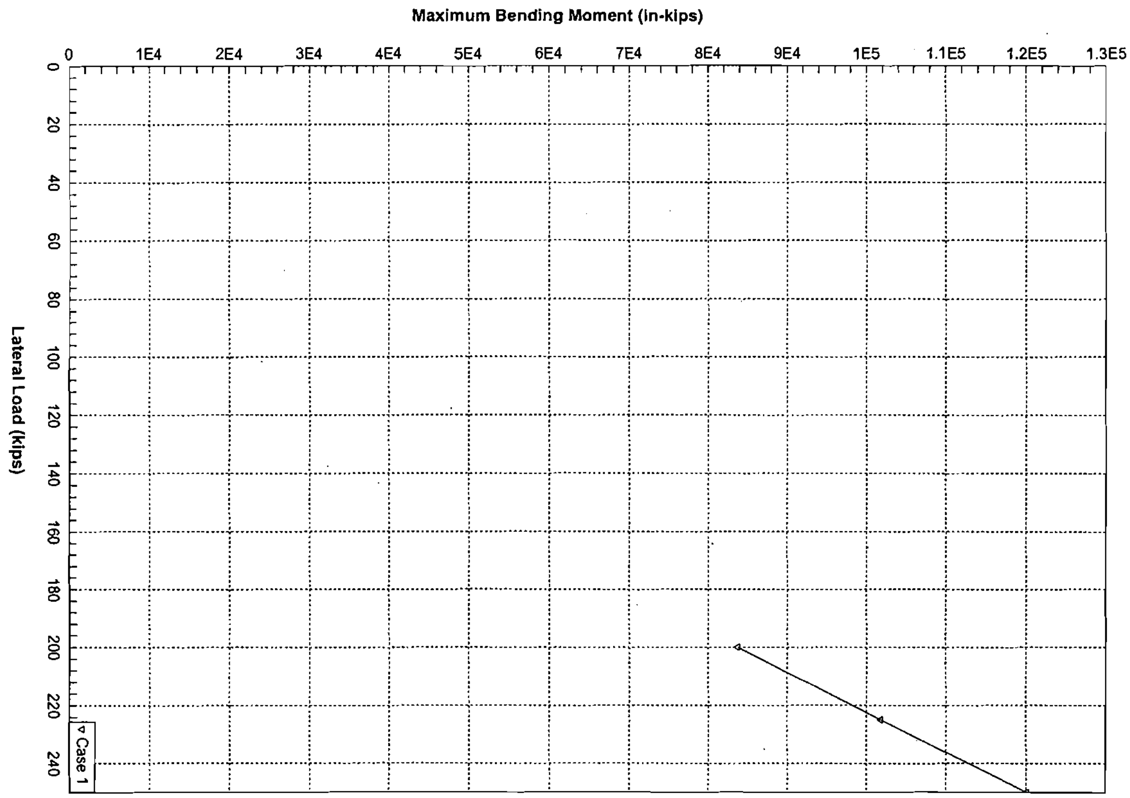


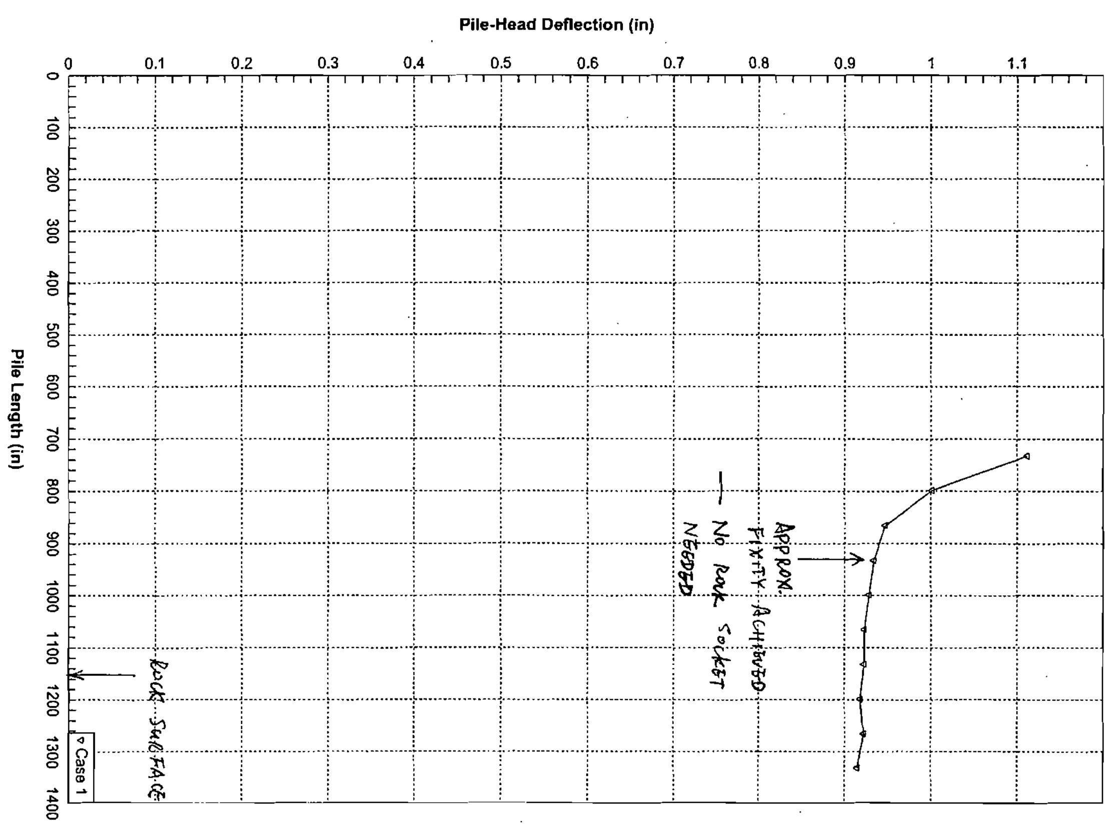
Mobilized Soil Reaction (lbs/in)
-2000
-1500
-1000
-500
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

16



Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Case 1


Maximum Bending Moment (in-kips)
0
1E4
2E4
3E4
4E4
5E4
6E4
7E4
8E4
9E4
1E5
1.1E5
1.2E5
1.3E5
Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Case 1


Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
Pile Length (in)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Case 1
APPROX. FIXITY ACHIEVED
— No Rock Socket NEEDED
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 1\
Name of input data file: Pier 1 - large - no scour.lpd
Name of output file: Pier 1 - large - no scour.lpo
Name of plot output file: Pier 1 - large - no scour.lpp
Name of runtime file: Pier 1 - large - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 10:39: 0

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 222
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1332.00 in
Depth of ground surface below top of pile = -48.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	5431065.	9630.7800	4074281.
2	1154.0000	102.00000	5431065.	9630.7800	4074281.
3	1154.0000	96.00000000	4169220.	7238.2300	4074281.
4	1332.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = -48.000 in
Distance from top of pile to bottom of layer = 140.000 in

Layer 2 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 140.000 in
Distance from top of pile to bottom of layer = 252.000 in
p-y subgrade modulus k for top of soil layer = 100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 100.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 252.000 in
Distance from top of pile to bottom of layer = 612.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 612.000 in
Distance from top of pile to bottom of layer = 1152.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 5 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 1152.000 in
Distance from top of pile to bottom of layer = 1763.000 in

(Depth of lowest layer extends 431.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-48.00	.07002
2	140.00	.07002
3	140.00	.03414
4	252.00	.03414
5	252.00	.03183
6	612.00	.03183
7	612.00	.03704
8	1152.00	.03704
9	1152.00	.05937
10	1763.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-48.000	6.19444	.00	.01000	.0
2	140.000	6.19444	.00	.01000	.0
3	140.000	6.19444	.00	.01000	.0
4	252.000	6.19444	.00	.01000	.0
5	252.000	.00000	32.50	-----	-----
6	612.000	.00000	32.50	-----	-----
7	612.000	.00000	34.80	-----	-----
8	1152.000	.00000	34.80	-----	-----
9	1152.000	4800.00000	.00	-----	-----
10	1763.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	-48.000	.7000	1.0000
2	1152.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	.9146662	1.2006E+08	-286089.
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	.7734806	1.0179E+08	-242428.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	.6356247	8.3767E+07	200000.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

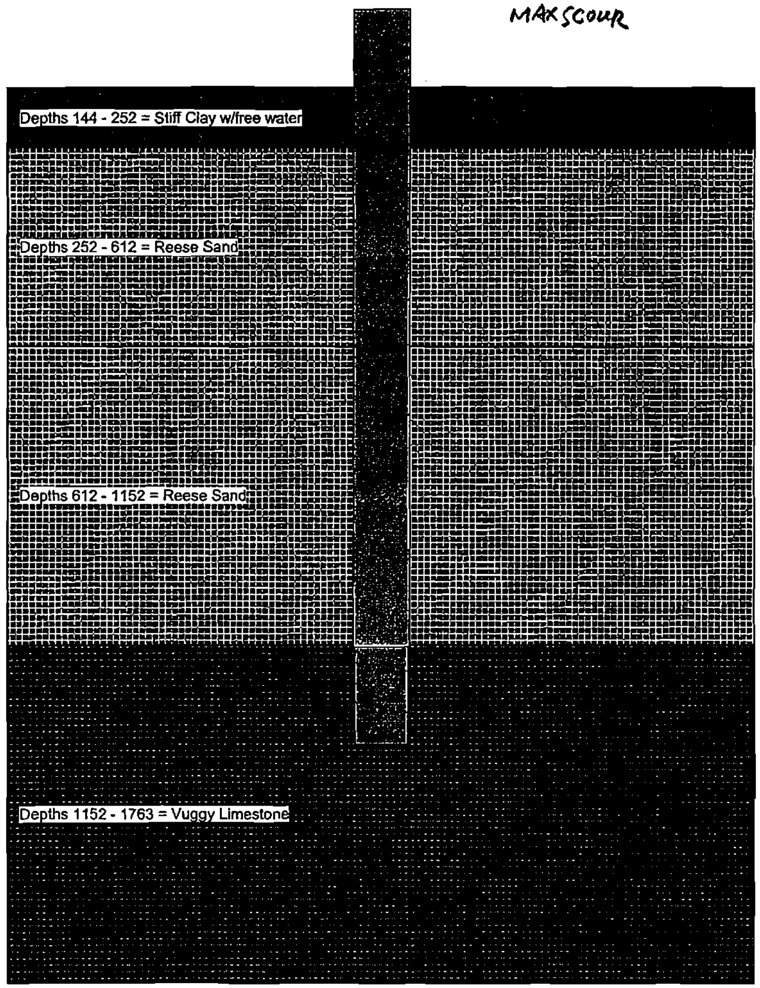
Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1332.000	.91466616	1.200599E+08	-286089.48519
1265.400	.92217197	1.200666E+08	-285185.88924
1198.800	.91823845	1.199067E+08	-285547.31703
1132.200	.92288781	1.200679E+08	-283873.63332
1065.600	.92302667	1.201358E+08	-283913.66905
999.000	.92822368	1.201524E+08	-287357.58700
932.400	.93379364	1.199860E+08	-298156.51257
865.800	.94732621	1.195767E+08	-320025.12083
799.200	1.00139299	1.191541E+08	-357492.91922
732.600	1.11208861	1.186604E+08	-409514.92889

The analysis ended normally.

RUN 2: PIER 1
8.5' SHAFT
MAX SCOUR

25



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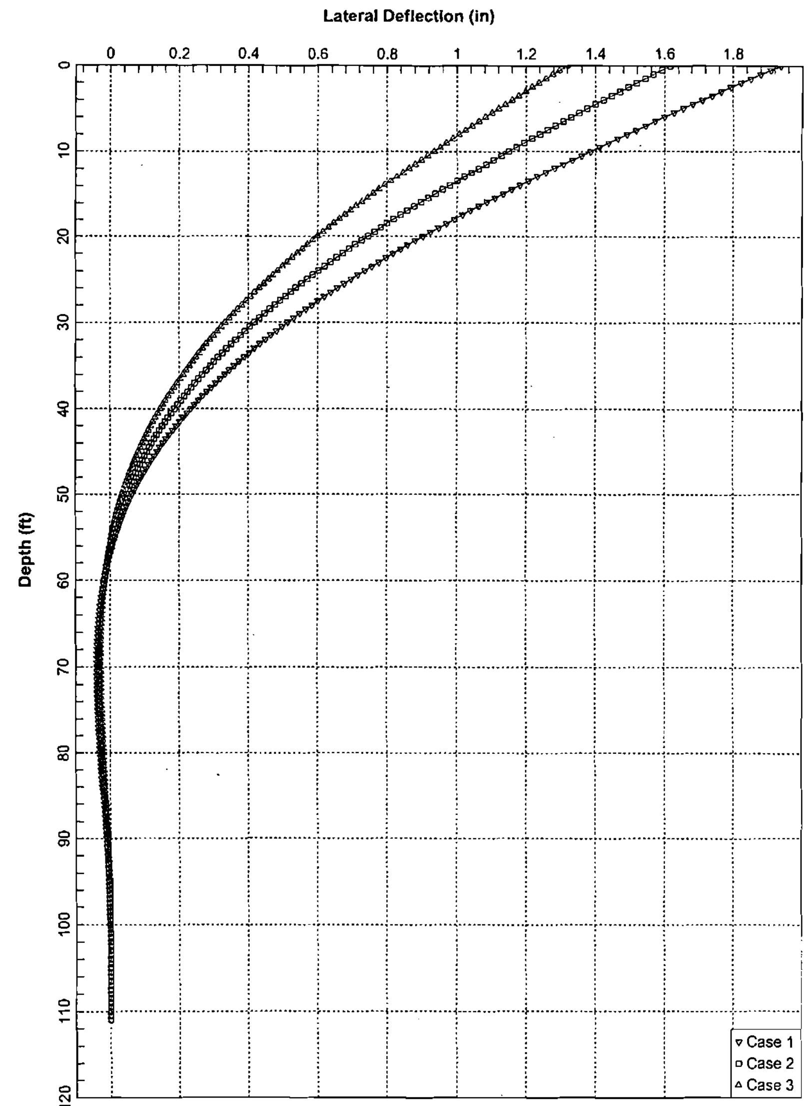

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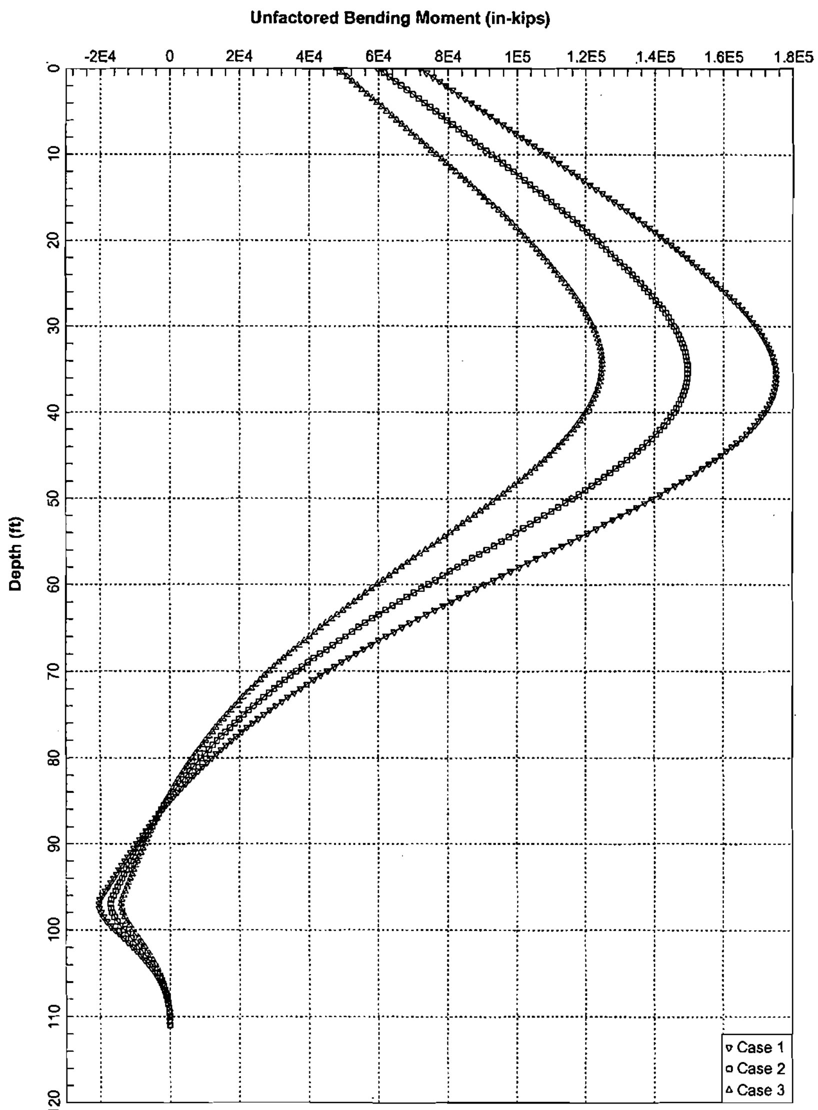
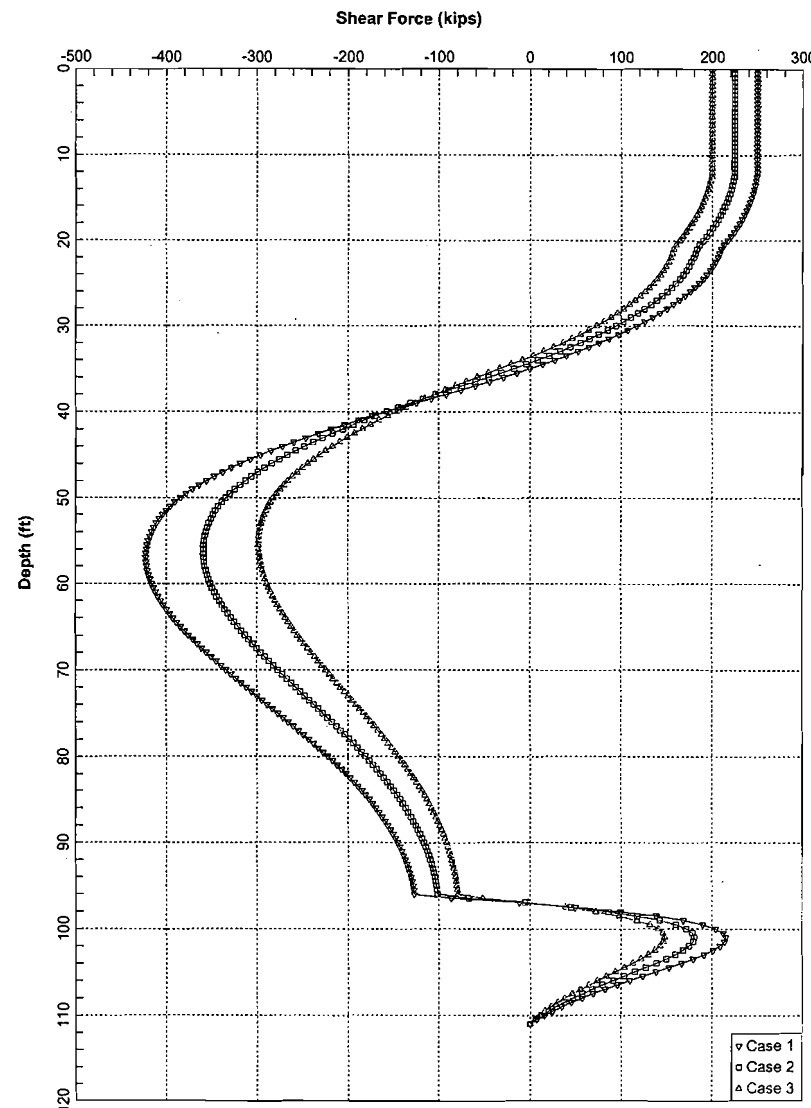


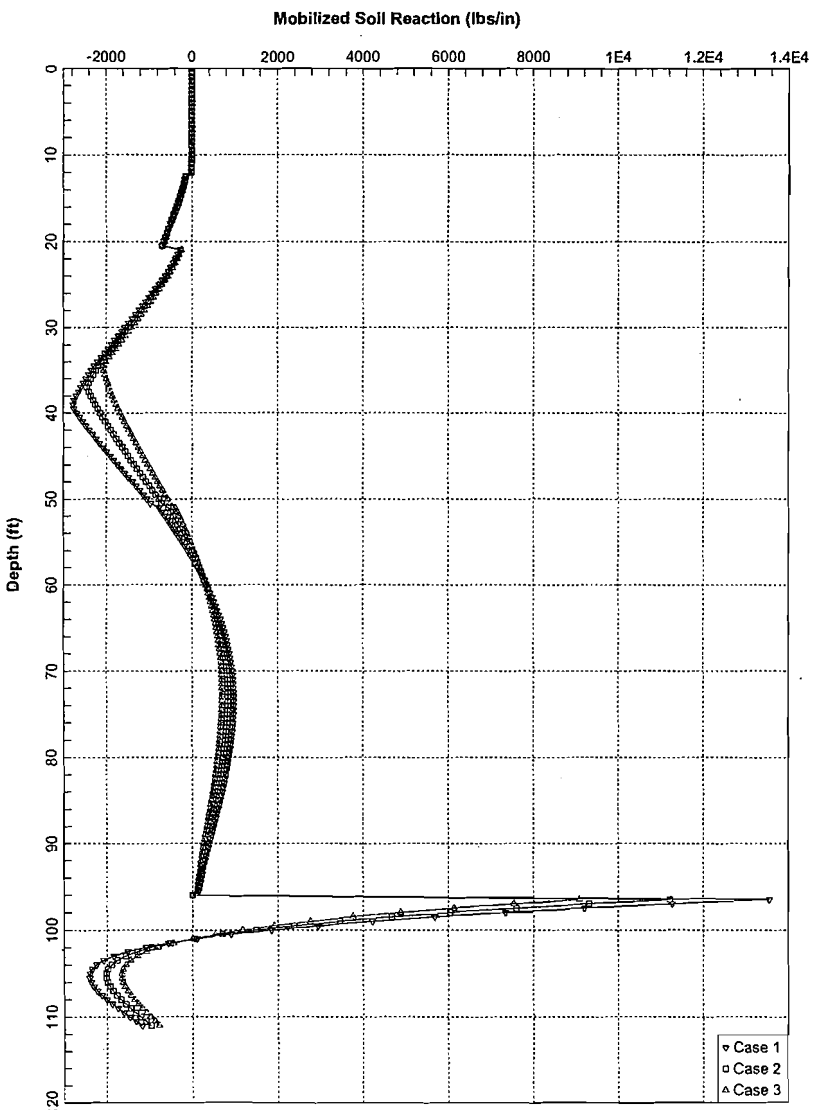
Unfactored Bending Moment (in-kips)
-2E4
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

28

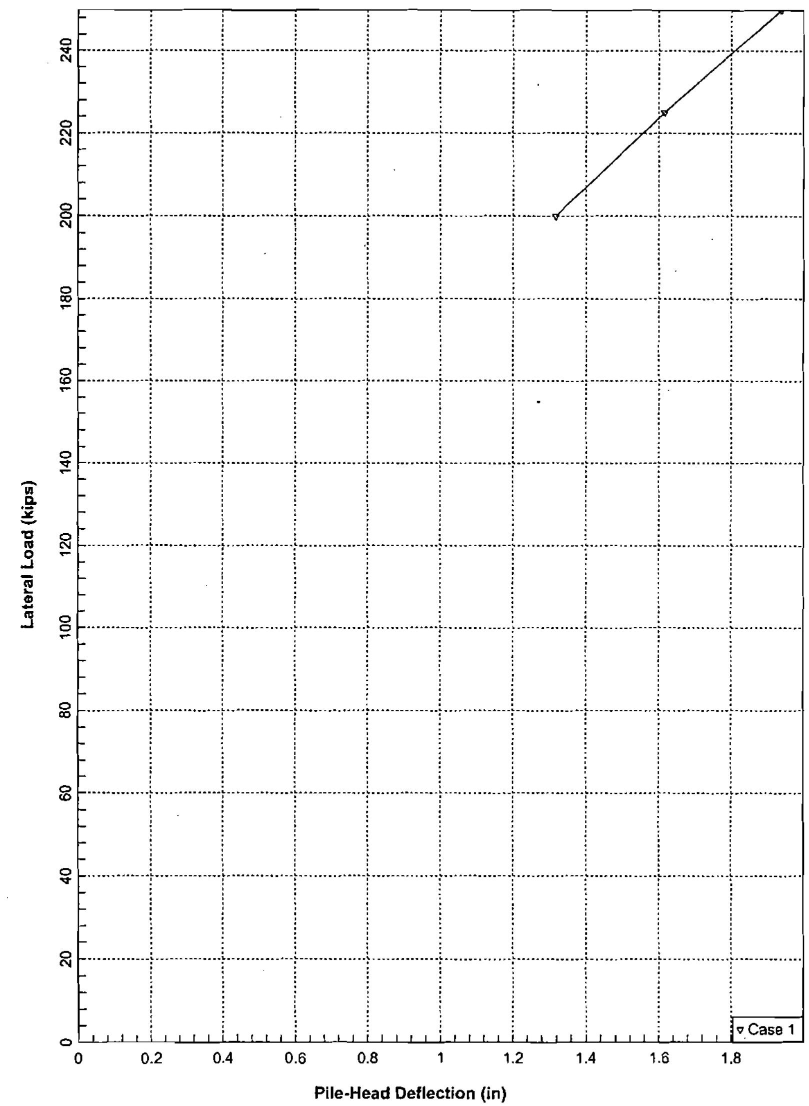


Shear Force (kips)
-500
-400
-300
-200
-100
0
100
200
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

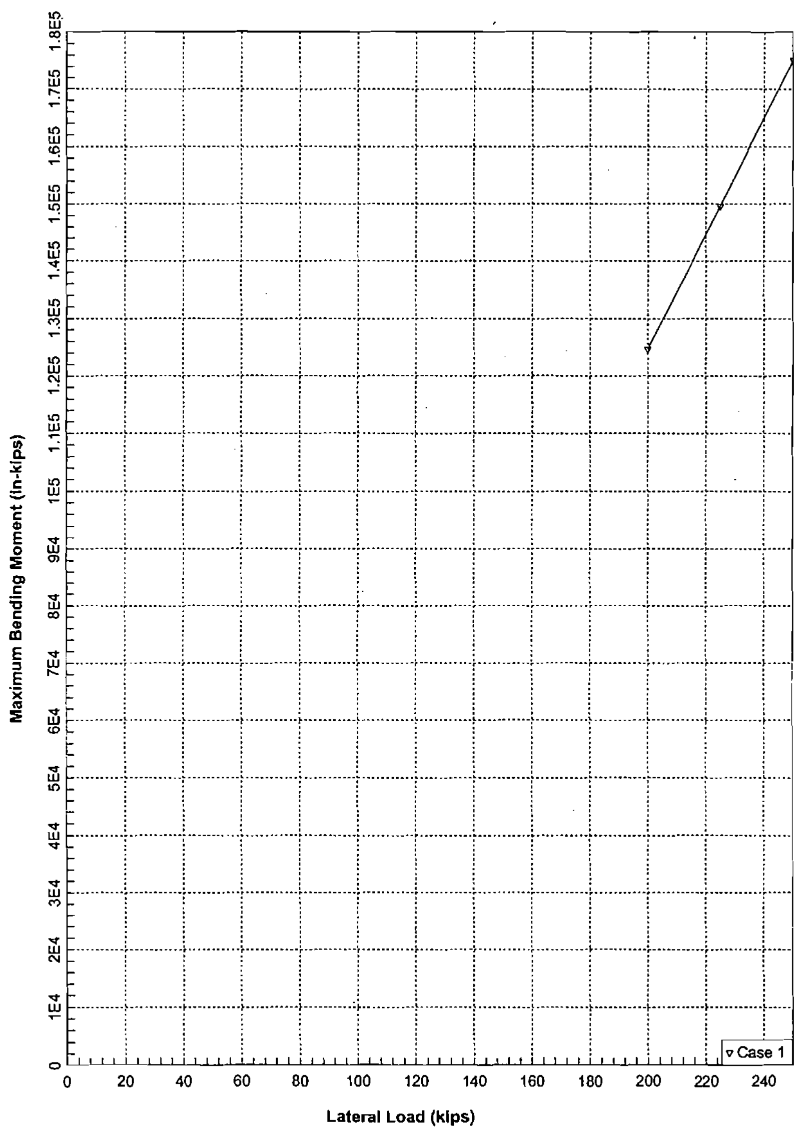
29


Mobilized Soil Reaction (lbs/in)
-2000
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

30

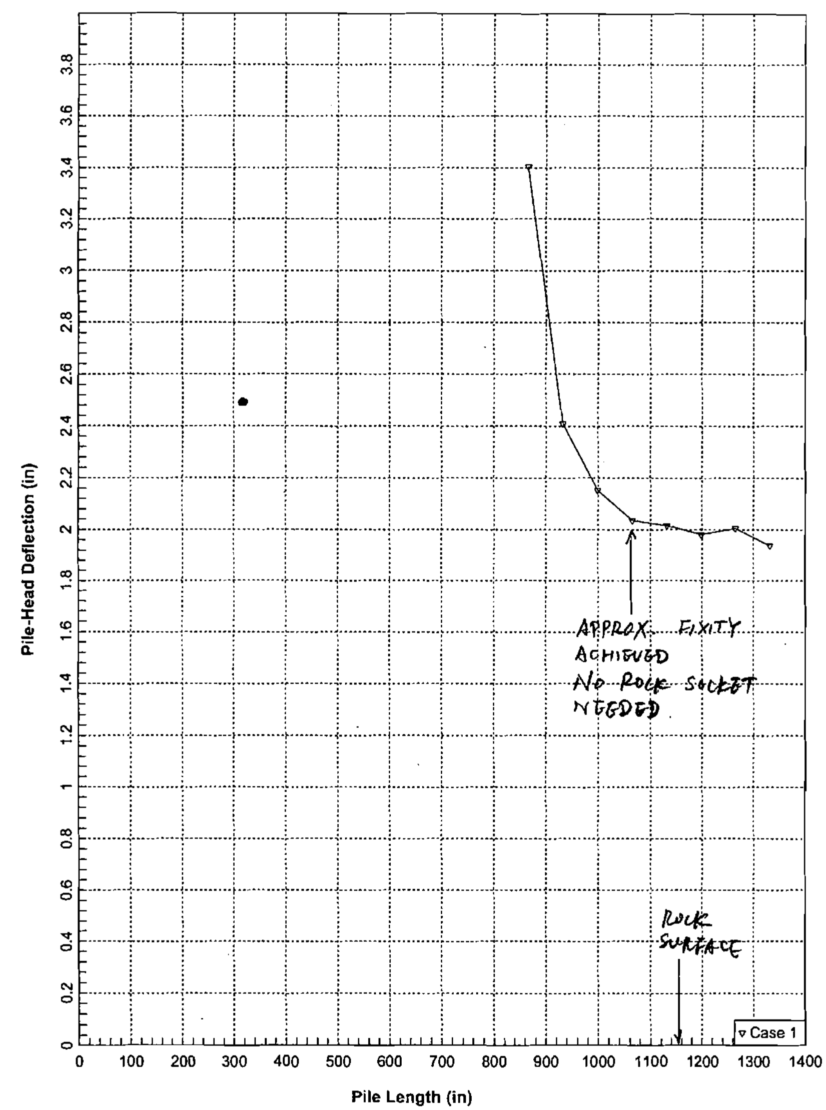

Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Pile-Head Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
Case 1

31



Maximum Bending Moment (in-kips)
Lateral Load (kips)
Case 1

32



Pile-Head Deflection (in)
Pile Length (in)
Case 1
APPROX. FIXITY ACHIEVED
NO ROCK SOCKET NEEDED.
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 1\
Name of input data file: Pier 1 - large - scour.lpd
Name of output file: Pier 1 - large - scour.lpo
Name of plot output file: Pier 1 - large - scour.lpp
Name of runtime file: Pier 1 - large - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 10:37:42

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 222
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1332.00 in
Depth of ground surface below top of pile = 144.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	5431065.	9630.7800	4074281.
2	1154.0000	102.00000	5431065.	9630.7800	4074281.
3	1154.0000	96.00000000	4169220.	7238.2300	4074281.
4	1332.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 144.000 in
Distance from top of pile to bottom of layer = 252.000 in
p-y subgrade modulus k for top of soil layer = 100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 100.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 252.000 in
Distance from top of pile to bottom of layer = 612.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 612.000 in
Distance from top of pile to bottom of layer = 1152.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 1152.000 in
Distance from top of pile to bottom of layer = 1763.000 in

(Depth of lowest layer extends 431.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 8 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	144.00	.03414
2	252.00	.03414
3	252.00	.03183
4	612.00	.03183
5	612.00	.03704
6	1152.00	.03704
7	1152.00	.05937
8	1763.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	144.000	6.19444	.00	.01000	.0
2	252.000	6.19444	.00	.01000	.0
3	252.000	.00000	32.50	------	------
4	612.000	.00000	32.50	------	------
5	612.000	.00000	34.80	------	------
6	1152.000	.00000	34.80	------	------
7	1152.000	4800.00000	.00	------	------
8	1763.000	4800.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	144.000	.7000	1.0000
2	1152.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs

Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs

Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2 lbs	Axial Load in	Pile-Head Deflection in-lbs	Maximum Moment lbs	Maximum Shear
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	1.9384	1.7518E+08	-422396.
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	1.6159	1.4963E+08	-359312.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	1.3174	1.2471E+08	-298222.

Pile-head Deflection vs. Pile Length

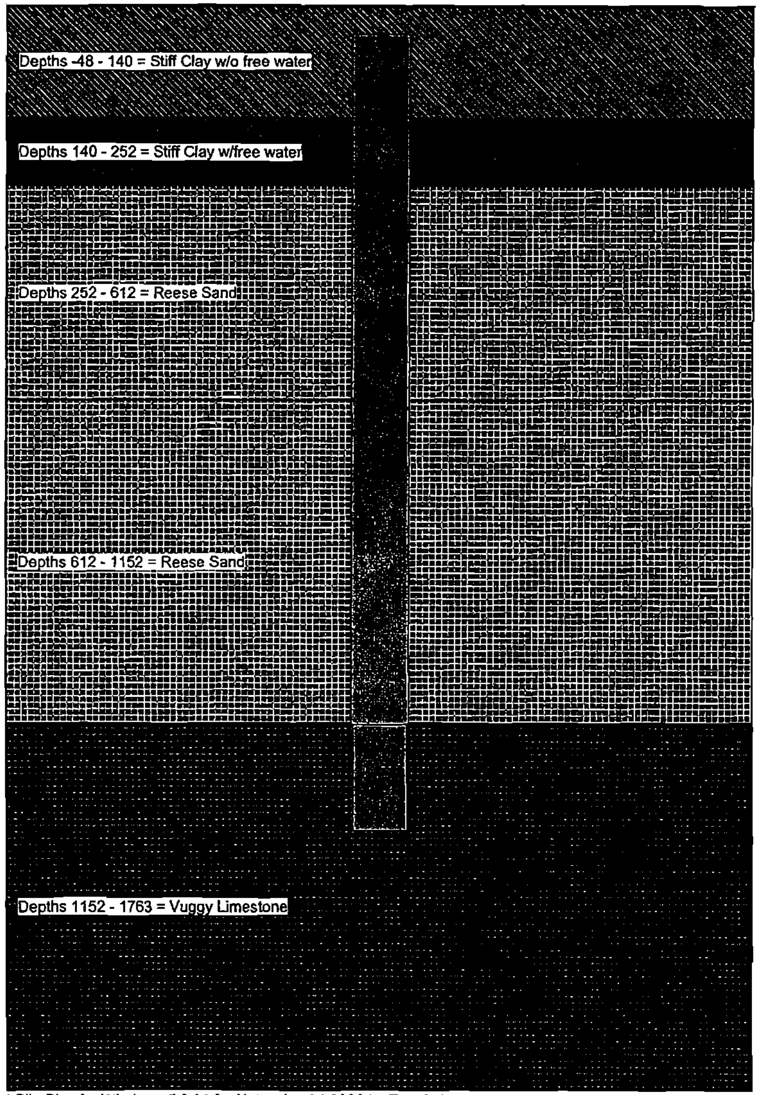
Boundary Condition Type 1, Shear and Moment

Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1332.000	1.93841170	1.751819E+08	-422396.38420
1265.400	2.00502177	1.771635E+08	-424749.06164
1198.800	1.98068023	1.761536E+08	-420394.97769
1132.200	2.01605069	1.769896E+08	-431347.86729
1065.600	2.03542113	1.770705E+08	-449396.42248
999.000	2.15287837	1.786902E+08	-496879.17729
932.400	2.40921630	1.806602E+08	-577989.08619
865.800	3.40552291	1.879045E+08	-743772.93729

The analysis ended normally.

RUN 3: PIER 1
8' SHAFT
NO SCOUR



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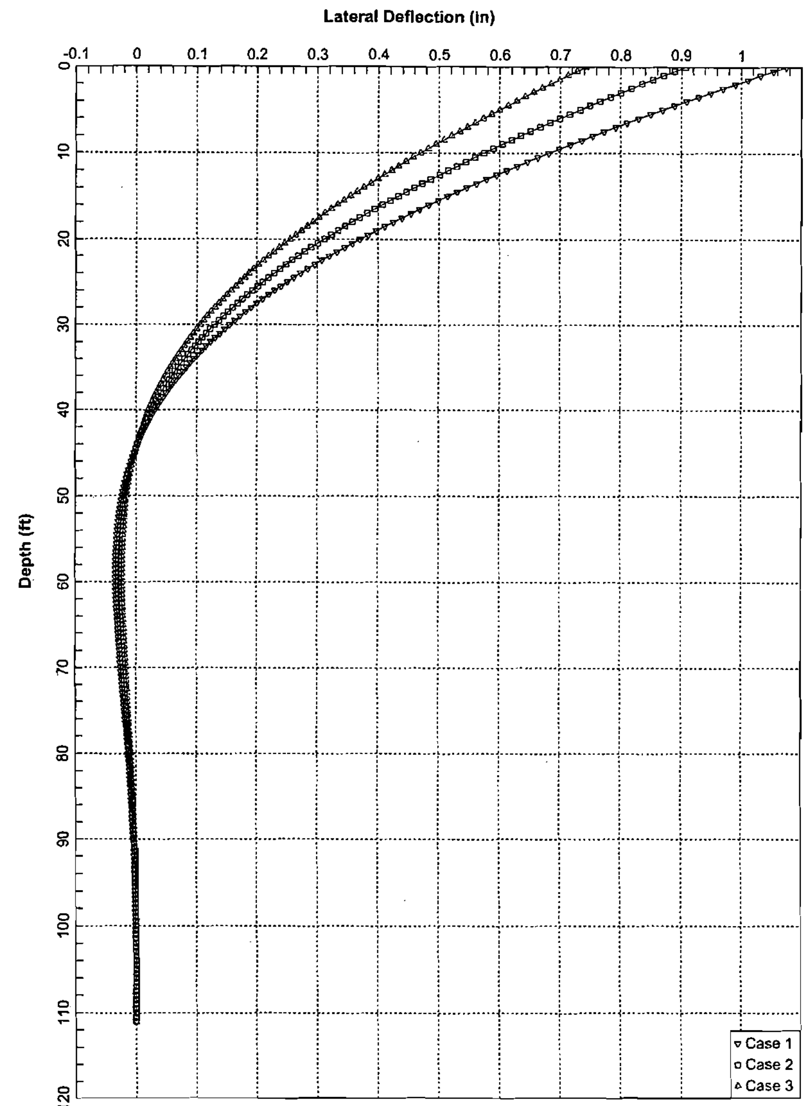

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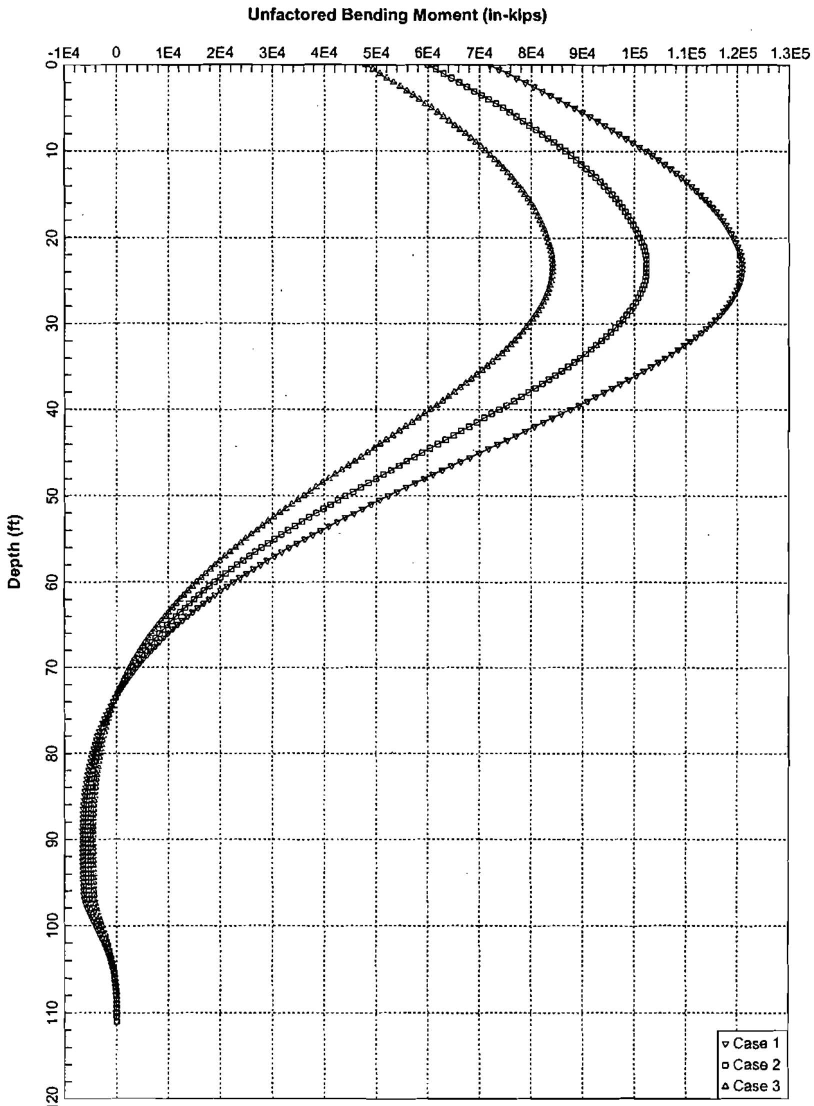
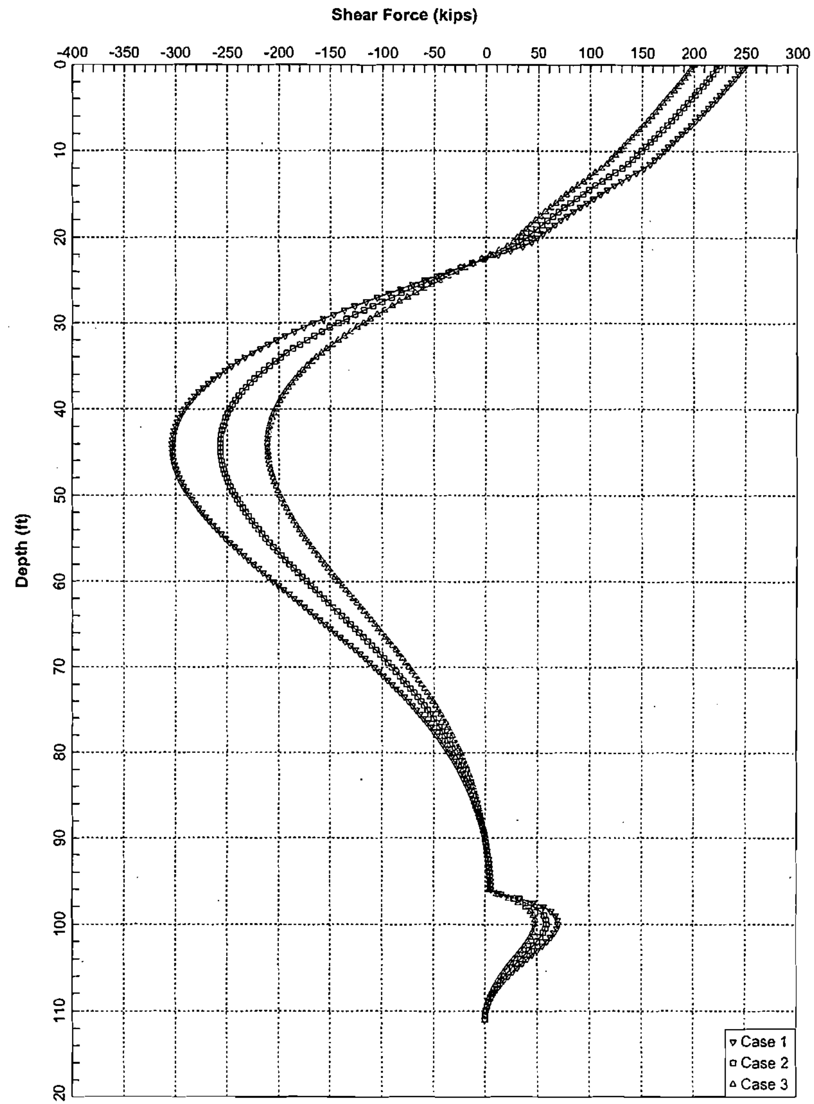


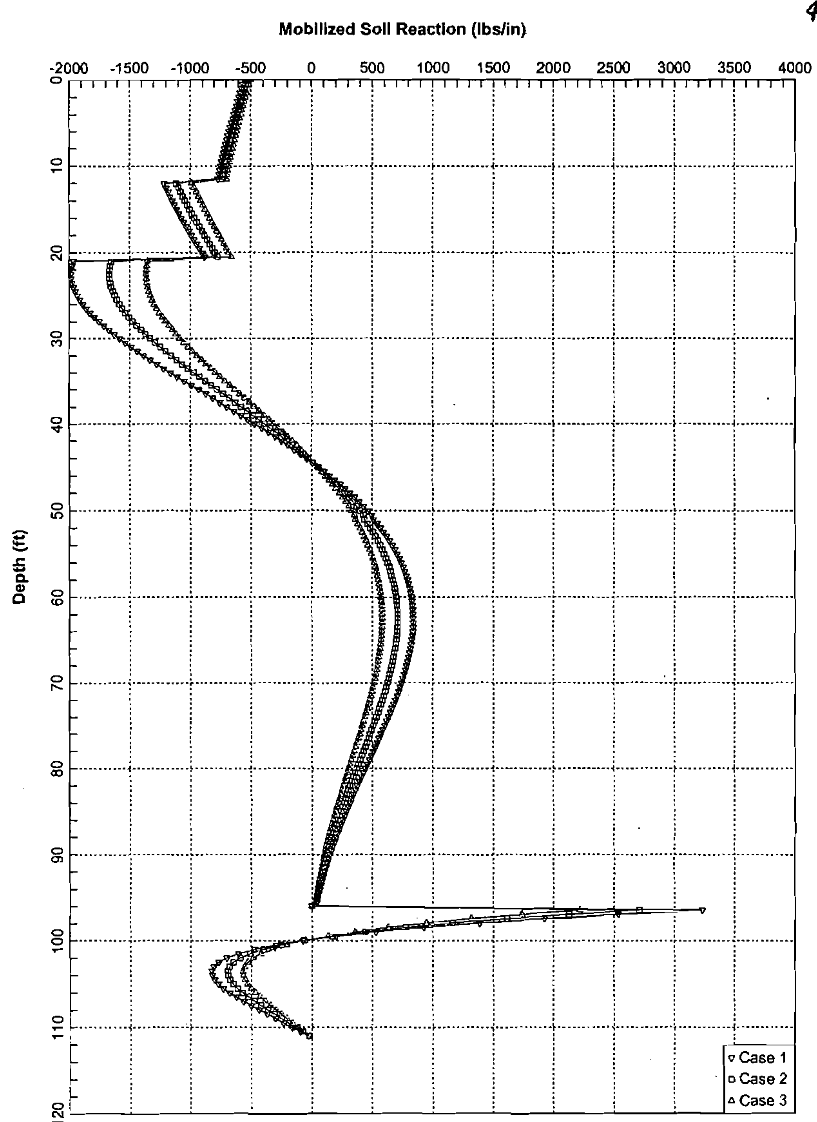
Unfactored Bending Moment (in-kips)
-1E4
0
1E4
2E4
3E4
4E4
5E4
6E4
7E4
8E4
9E4
1E5
1.1E5
1.2E5
1.3E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

42



Shear Force (kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

43



Mobilized Soil Reaction (lbs/in)
Depth (ft)
Case 1
Case 2
Case 3

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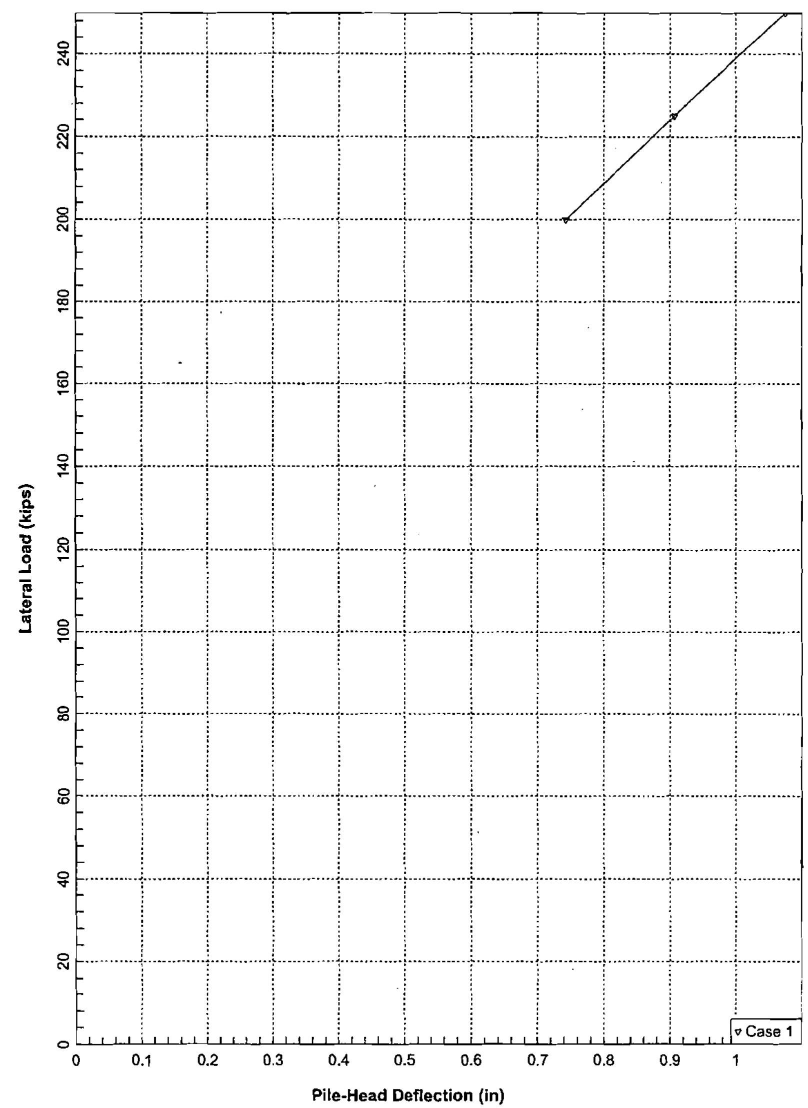
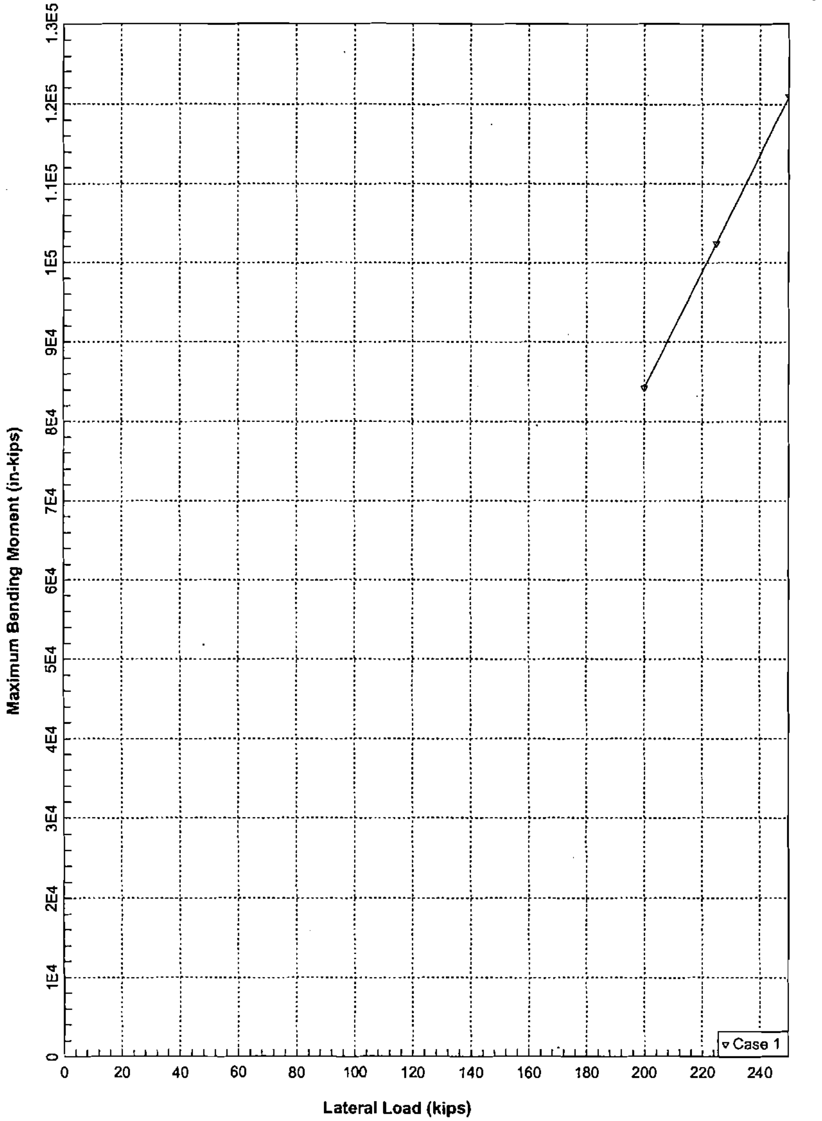


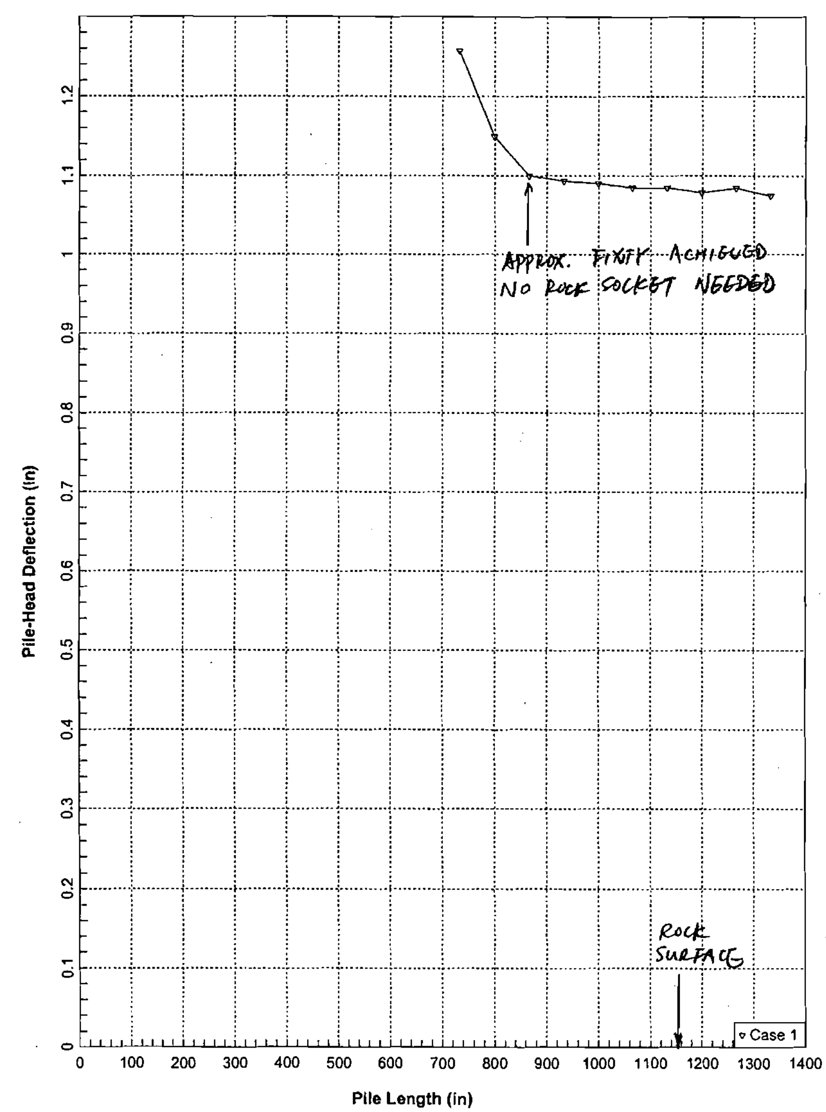
Lateral Load (kips)
Pile-Head Deflection (in)
Case 1

45



Maximum Bending Moment (in-kips)
0
1E4
2E4
3E4
4E4
5E4
6E4
7E4
8E4
9E4
1E5
1.1E5
1.2E5
1.3E5
Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Case 1

46



Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
Pile Length (in)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Case 1
APPROX. FIXITY ACHIEVED
NO ROCK SOCKET NEEDED
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 1\
Name of input data file: Pier 1 - small - no scour.lpd
Name of output file: Pier 1 - small - no scour.lpo
Name of plot output file: Pier 1 - small - no scour.lpp
Name of runtime file: Pier 1 - small - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 10:44:25

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 222
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1332.00 in
Depth of ground surface below top of pile = -48.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	1154.0000	96.00000000	4356263.	8611.2400	4074281.
3	1154.0000	90.00000000	3220623.	6361.7300	4074281.
4	1332.0000	90.00000000	3220623.	6361.7300	4074281.

ok!

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = -48.000 in
Distance from top of pile to bottom of layer = 140.000 in

Layer 2 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 140.000 in
Distance from top of pile to bottom of layer = 252.000 in
p-y subgrade modulus k for top of soil layer = 100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 100.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 252.000 in
Distance from top of pile to bottom of layer = 612.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 612.000 in
Distance from top of pile to bottom of layer = 1152.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 5 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 1152.000 in
Distance from top of pile to bottom of layer = 1763.000 in

(Depth of lowest layer extends 431.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-48.00	.07002
2	140.00	.07002
3	140.00	.03414
4	252.00	.03414
5	252.00	.03183
6	612.00	.03183
7	612.00	.03704
8	1152.00	.03704
9	1152.00	.05937
10	1763.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-48.000	6.19444	.00	.01000	.0
2	140.000	6.19444	.00	.01000	.0
3	140.000	6.19444	.00	.01000	.0
4	252.000	6.19444	.00	.01000	.0
5	252.000	.00000	32.50	------	------
6	612.000	.00000	32.50	------	------
7	612.000	.00000	34.80	------	------
8	1152.000	.00000	34.80	------	------
9	1152.000	4800.00000	.00	------	------
10	1763.000	4800.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	-48.000	.7000	1.0000
2	1152.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	1.0748	1.2099E+08	-303428.
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	.9065911	1.0246E+08	-256728.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	.7431125	8.4215E+07	-210896.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

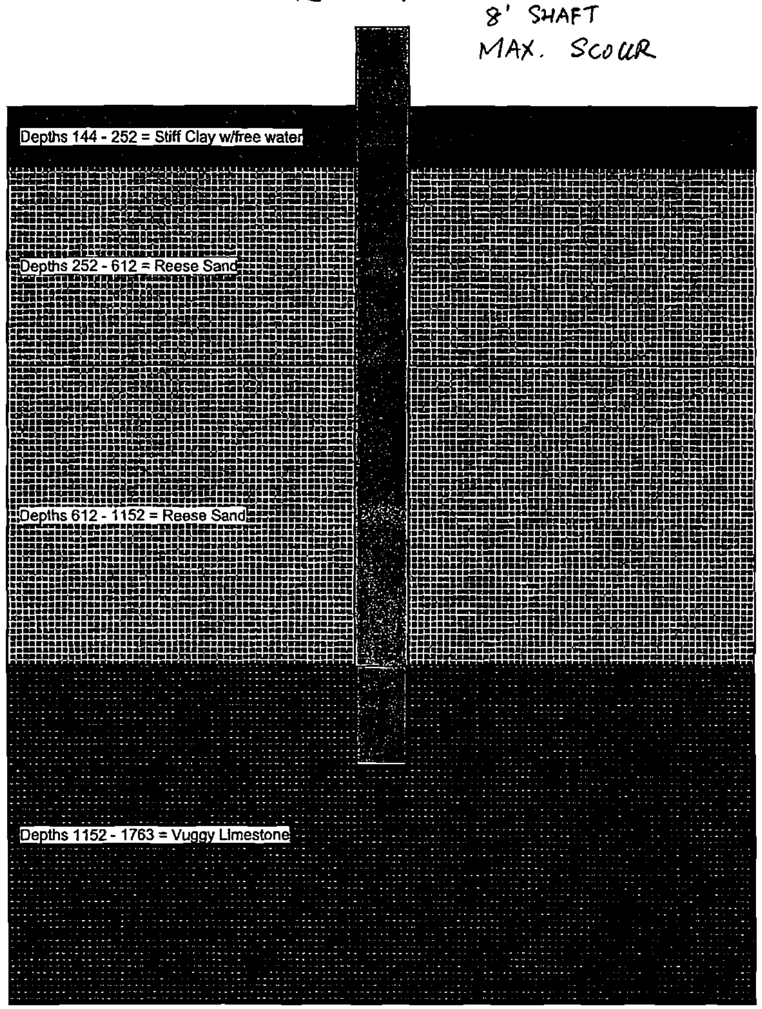
Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1332.000	1.07478736	1.209873E+08	-303428.18920
1265.400	1.08441722	1.210006E+08	-302540.92051
1198.800	1.07847510	1.208252E+08	-302824.03459
1132.200	1.08467978	1.210061E+08	-301426.80700
1065.600	1.08486950	1.210870E+08	-300580.79526
999.000	1.09025812	1.211327E+08	-301587.75245
932.400	1.09285159	1.210145E+08	-308215.12443
865.800	1.10004800	1.206962E+08	-325512.23721
799.200	1.14923412	1.204678E+08	-360611.13030
732.600	1.25660086	1.202211E+08	-414061.05258

The analysis ended normally.

RUN 4: PIER 1
8' SHAFT
MAX. SCOUR

53



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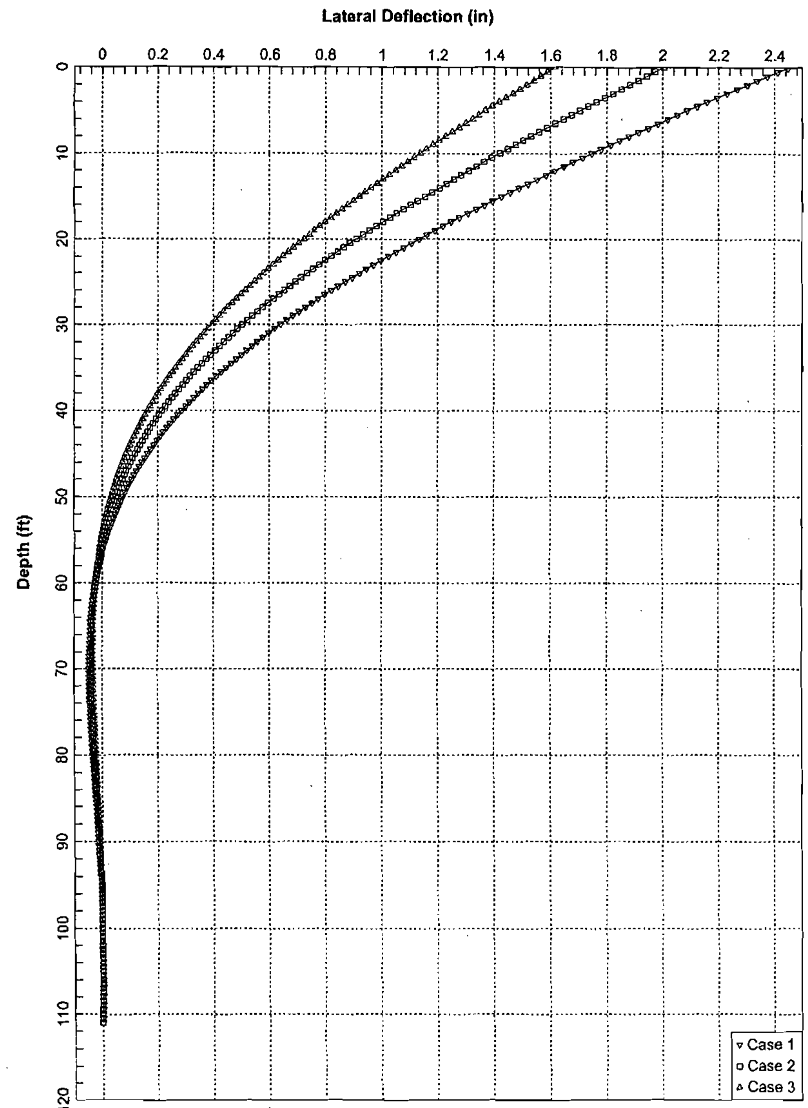
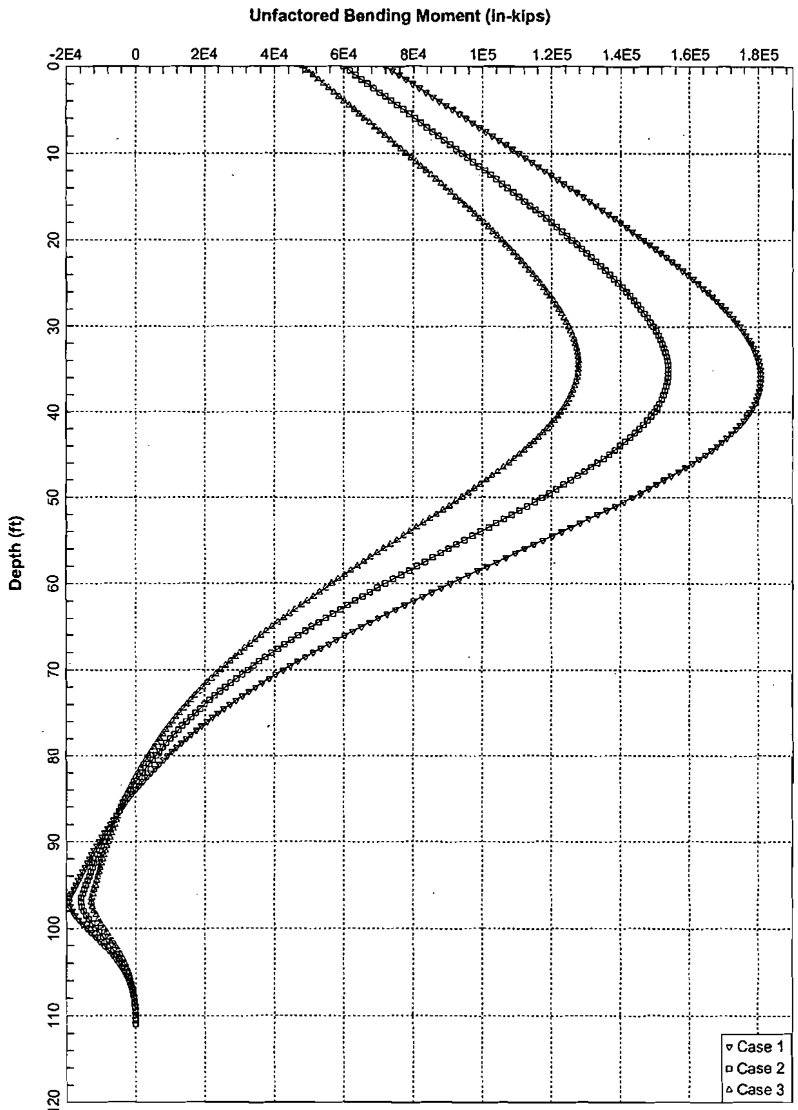
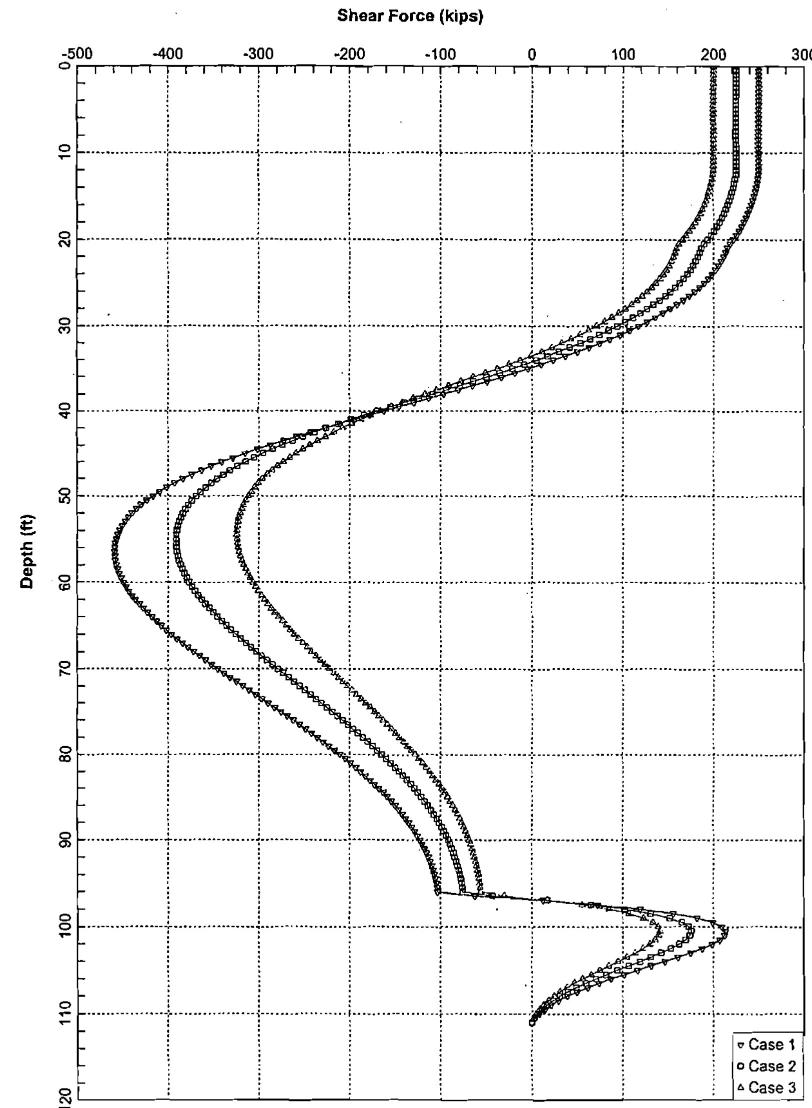


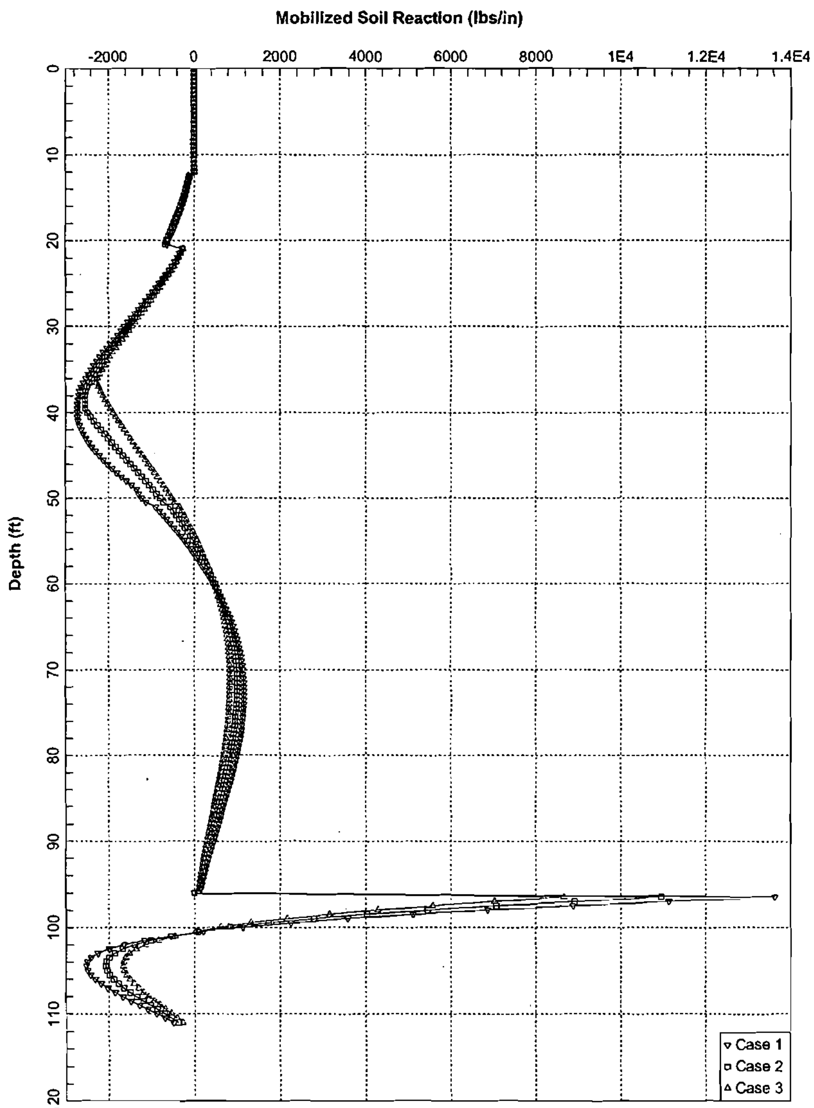
55


Unfactored Bending Moment (in-kips)
-2E4
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

56

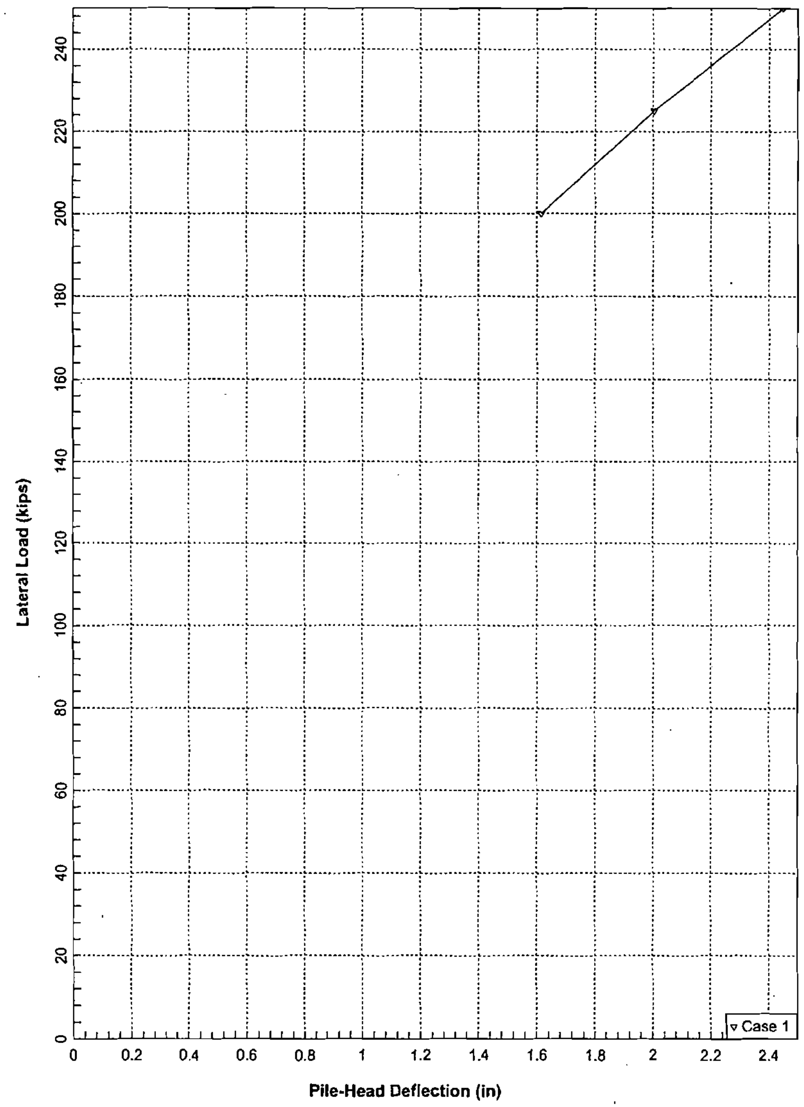

Shear Force (kips)
-500
-400
-300
-200
-100
0
100
200
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

57



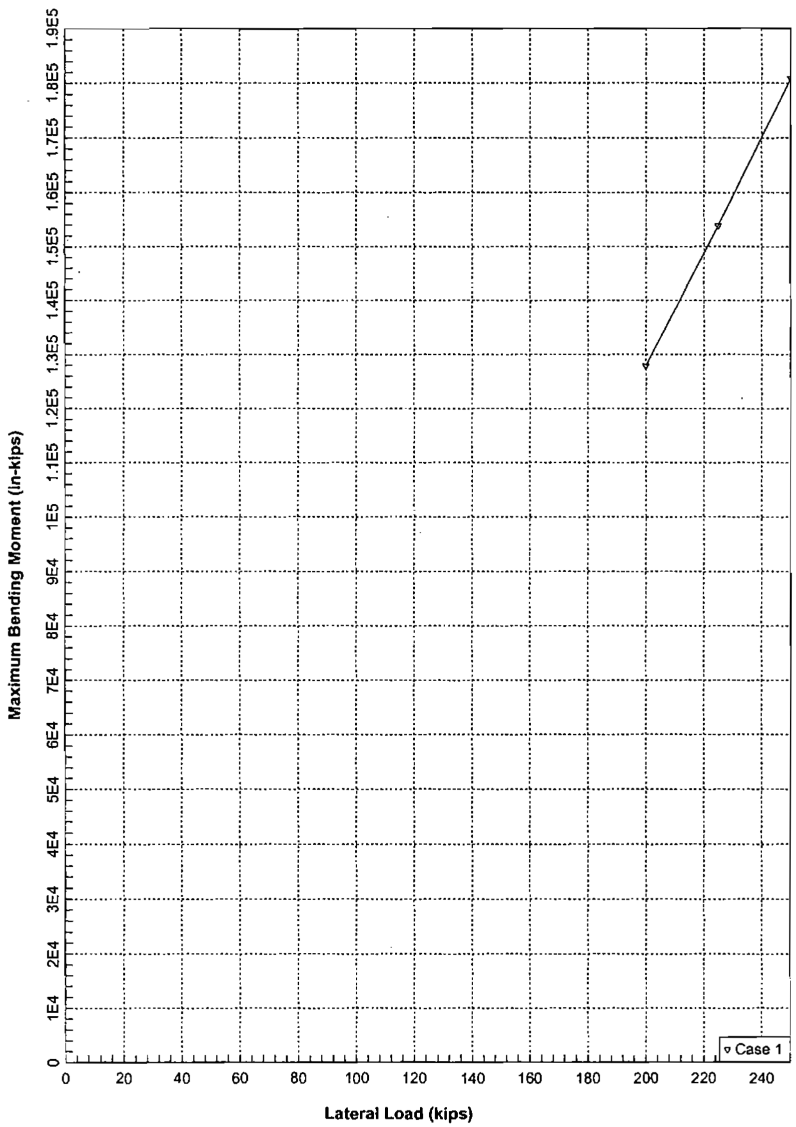
Mobilized Soil Reaction (lbs/in)
-2000
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Case 1
Case 2
Case 3

58



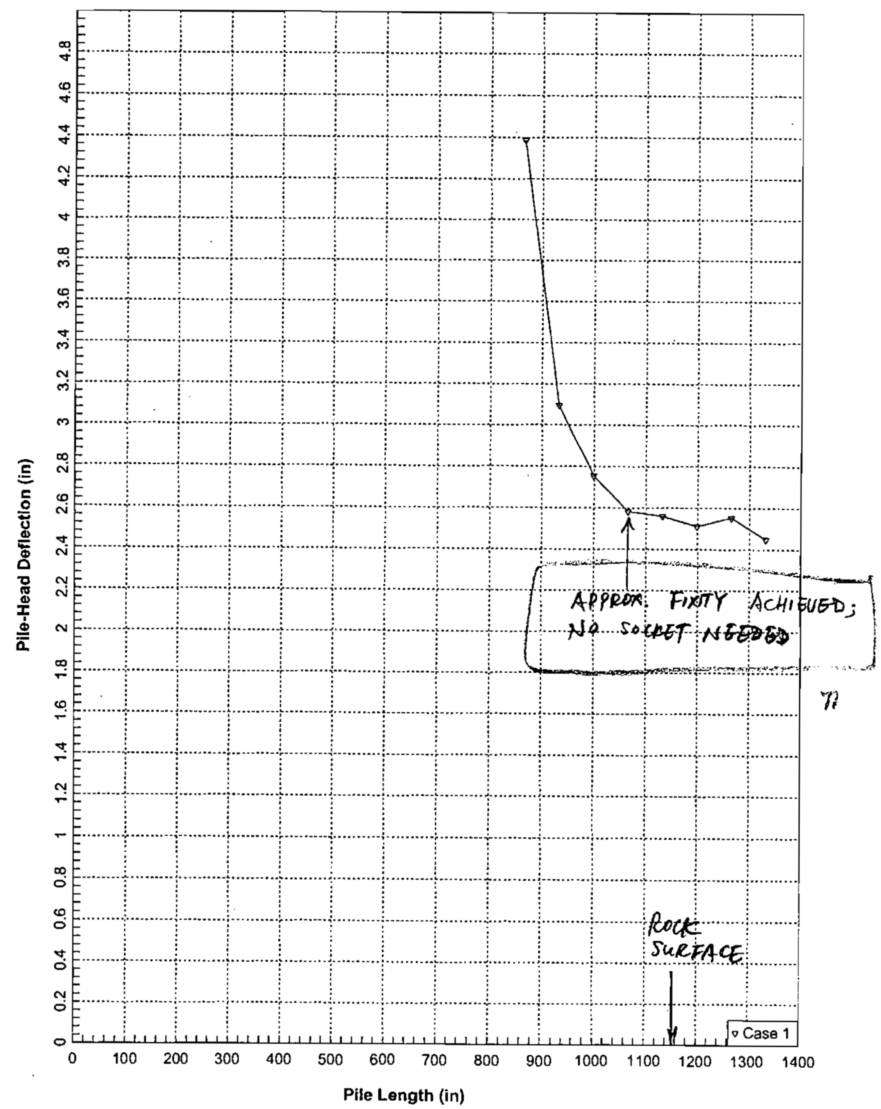
Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Pile-Head Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
Case 1

59



Maximum Bending Moment (in-kips)
0
1E4
2E4
3E4
4E4
5E4
6E4
7E4
8E4
9E4
1E5
1.1E5
1.2E5
1.3E5
1.4E5
1.5E5
1.6E5
1.7E5
1.8E5
1.9E5
Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Case 1

60



Pile-Head Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
2.6
2.8
3
3.2
3.4
3.6
3.8
4
4.2
4.4
4.6
4.8
Pile Length (in)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Case 1
APPROX. FIXITY ACHIEVED;
NO SOCKET NEEDED
ROCK SURFACE

71

61

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 1\
Name of input data file: Pier 1 - small - scour.lpd
Name of output file: Pier 1 - small - scour.lpo
Name of plot output file: Pier 1 - small - scour.lpp
Name of runtime file: Pier 1 - small - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 10:46:59

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

62

Solution Control Parameters:
- Number of pile increments = 222
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1332.00 in
Depth of ground surface below top of pile = 144.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	1154.0000	96.00000000	4356263.	8611.2400	4074281.
3	1154.0000	90.00000000	3220623.	6361.7300	4074281.
4	1332.0000	90.00000000	3220623.	6361.7300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 144.000 in
Distance from top of pile to bottom of layer = 252.000 in
p-y subgrade modulus k for top of soil layer = 100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 100.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 252.000 in
Distance from top of pile to bottom of layer = 612.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 612.000 in
Distance from top of pile to bottom of layer = 1152.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 1152.000 in
Distance from top of pile to bottom of layer = 1763.000 in

(Depth of lowest layer extends 431.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 8 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	144.00	.03414
2	252.00	.03414
3	252.00	.03183
4	612.00	.03183
5	612.00	.03704
6	1152.00	.03704
7	1152.00	.05937
8	1763.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	144.000	6.19444	.00	.01000	.0
2	252.000	6.19444	.00	.01000	.0
3	252.000	.00000	32.50	------	------
4	612.000	.00000	32.50	------	------
5	612.000	.00000	34.80	------	------
6	1152.000	.00000	34.80	------	------
7	1152.000	4800.00000	.00	------	------
8	1763.000	4800.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	144.000	.7000	1.0000
2	1152.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs

Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs

Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	2.4489	1.8084E+08	-458903.
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	2.0052	1.5391E+08	-391225.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	1.6154	1.2785E+08	-323505.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

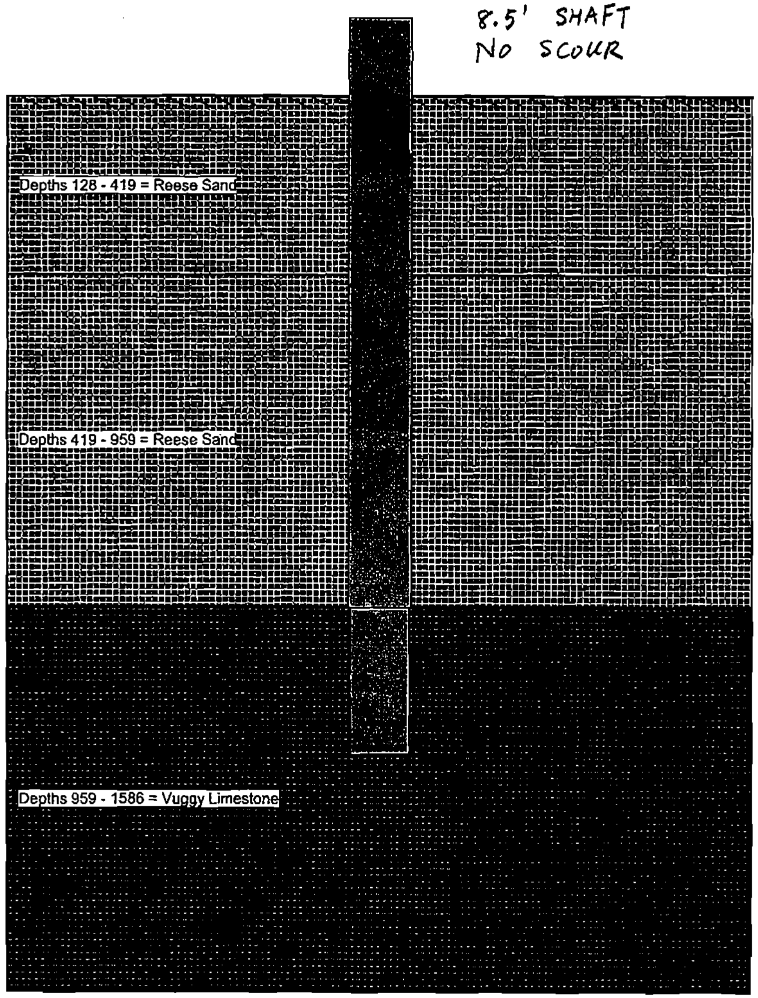
Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1332.000	2.44888051	1.808423E+08	-458902.73356
1265.400	2.55294061	1.835590E+08	-462582.19973
1198.800	2.51108107	1.822197E+08	-459153.77096
1132.200	2.55910013	1.832829E+08	-465262.77656
1065.600	2.58214127	1.833523E+08	-478867.04664
999.000	2.75458006	1.856269E+08	-525351.83987
932.400	3.09429337	1.882331E+08	-613926.82499
865.800	4.38630636	1.972995E+08	-805494.59291

The analysis ended normally.

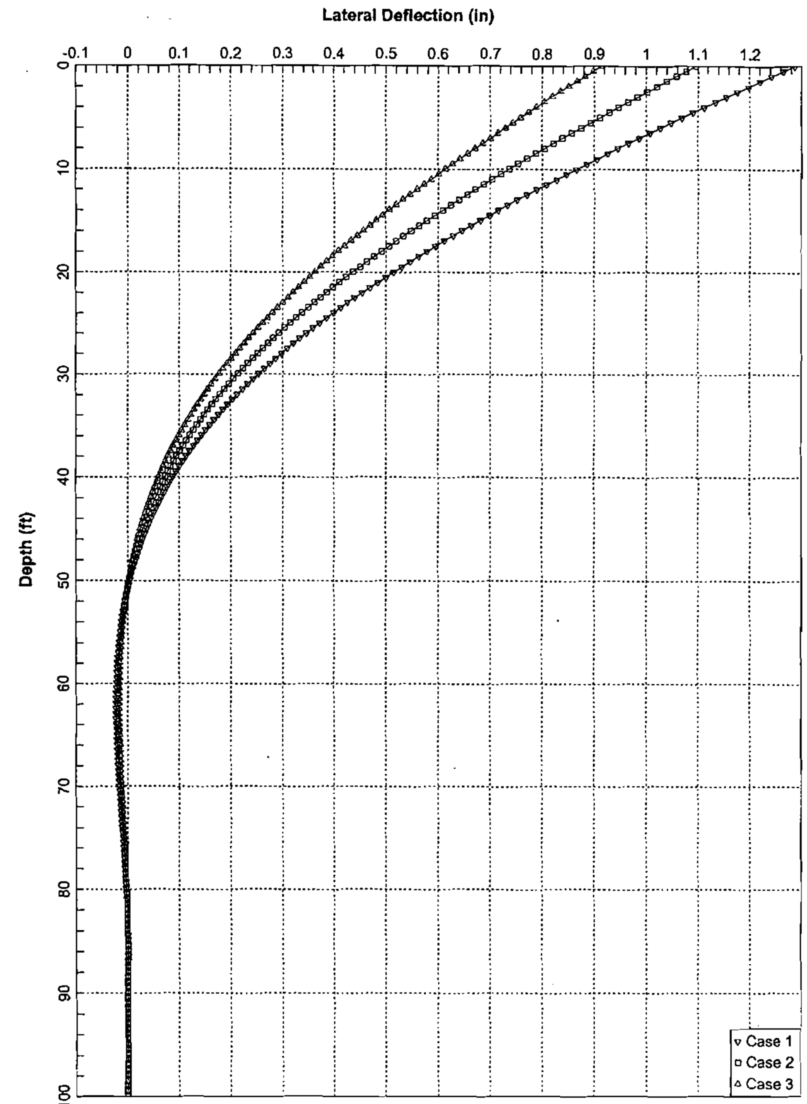
RUN 5: PIER 2 ✓
8.5' SHAFT
NO SCOUR

67

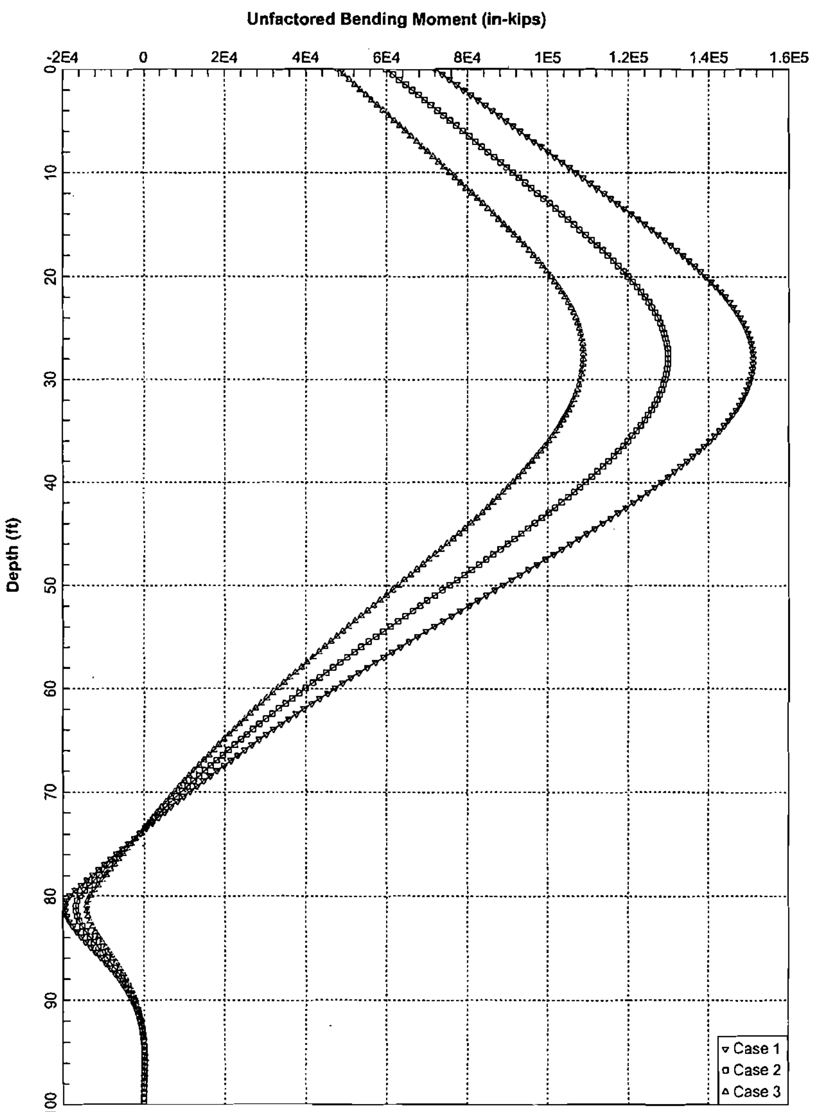


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

68

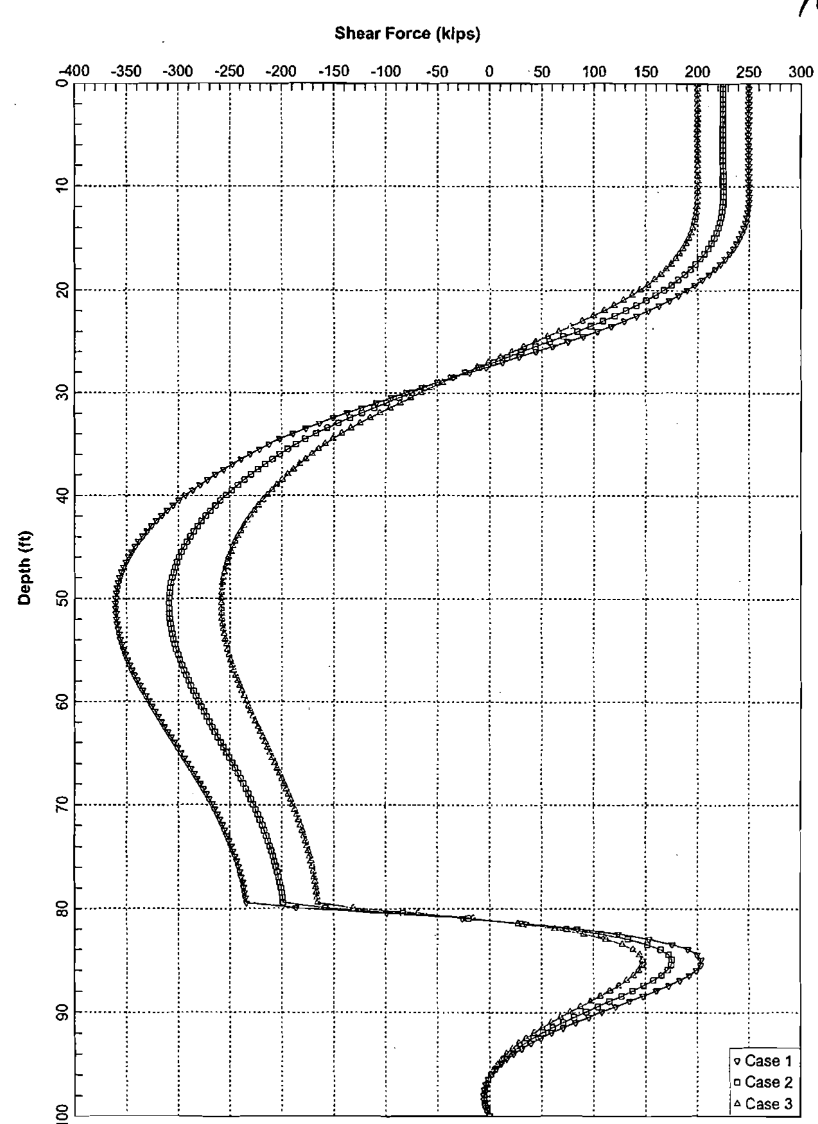


69



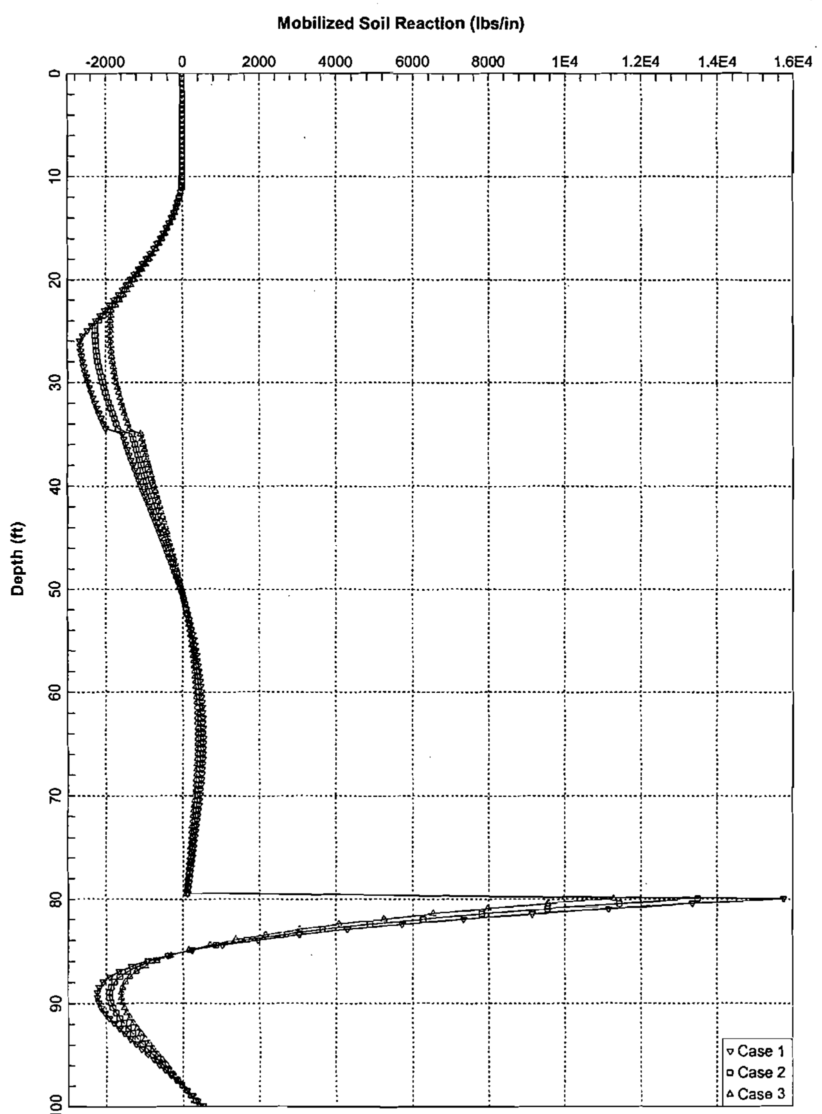
Unfactored Bending Moment (in-kips)
-2E4
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

70



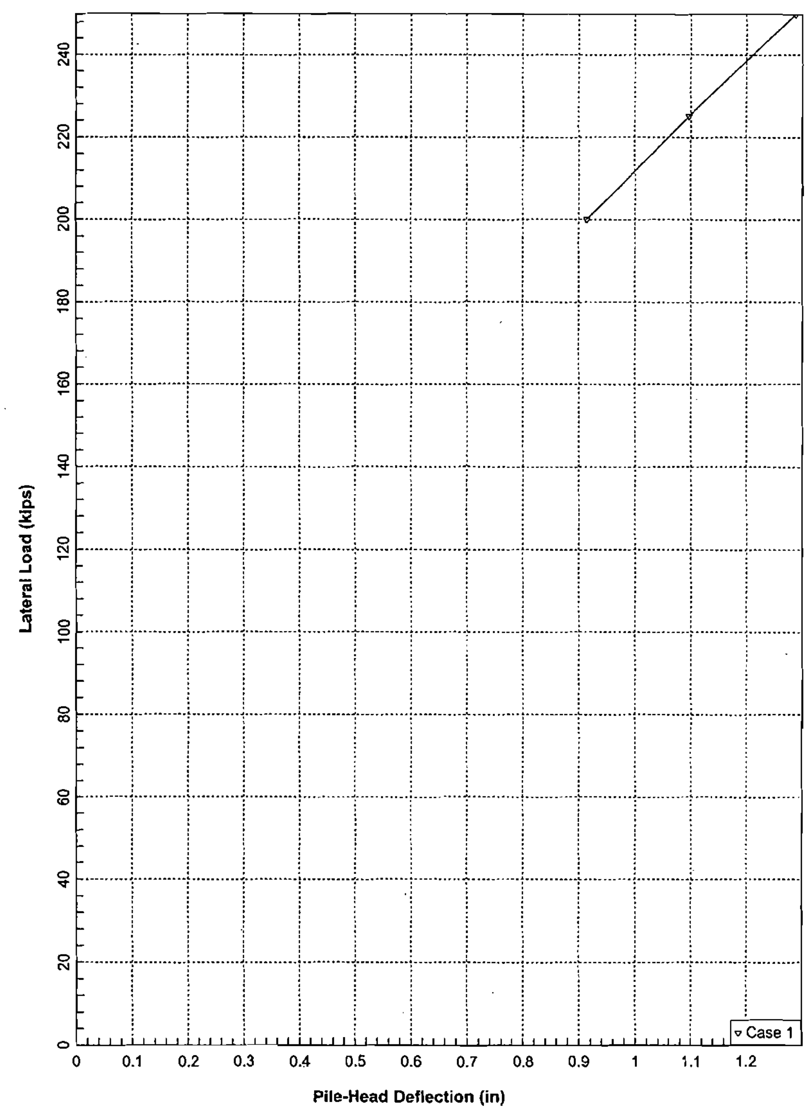
Shear Force (kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

71



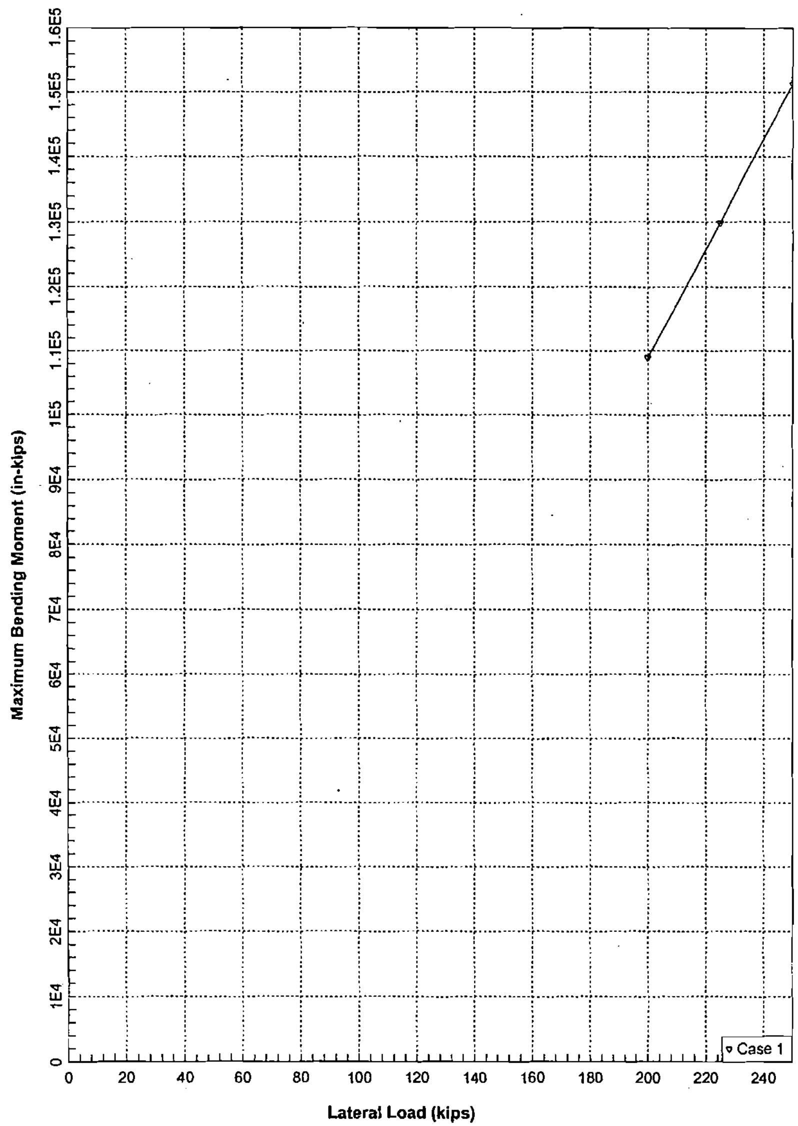
Mobilized Soil Reaction (lbs/in)
-2000
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
1.6E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

72

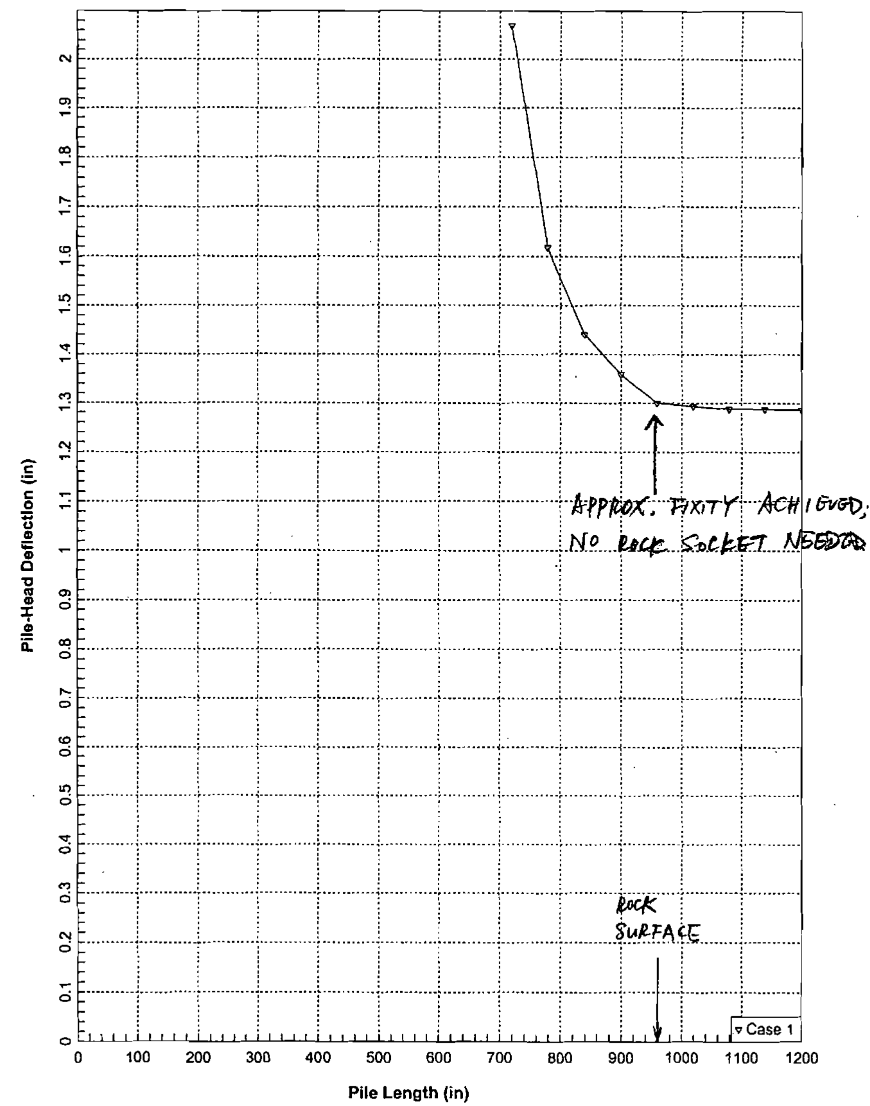


Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
Case 1

73


Maximum Bending Moment (in-kips)
Lateral Load (kips)
Case 1

74


Pile-Head Deflection (in)
Pile Length (in)
Case 1
APPROX. FIXITY ACHIEVED;
NO ROCK SOCKET NEEDED
ROCK
SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 2\
Name of input data file: Pier 2 - large - no scour.lpd
Name of output file: Pier 2 - large - no scour.lpo
Name of plot output file: Pier 2 - large - no scour.lpp
Name of runtime file: Pier 2 - large - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 10:52:35

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 200
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1199.00 in
Depth of ground surface below top of pile = 128.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	5431065.	9630.7800	4074281.
2	961.0000	102.00000	5431065.	9630.7800	4074281.
3	961.0000	96.00000000	4169220.	7238.2300	4074281.
4	1199.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 128.000 in
Distance from top of pile to bottom of layer = 419.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 419.000 in
Distance from top of pile to bottom of layer = 959.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 959.000 in
Distance from top of pile to bottom of layer = 1586.000 in

(Depth of lowest layer extends 387.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth

is defined using 6 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	128.00	.03935
2	419.00	.03935
3	419.00	.03762
4	959.00	.03762
5	959.00	.05937
6	1586.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	128.000	.00000	35.20	-----	-----
2	419.000	.00000	35.20	-----	-----
3	419.000	.00000	35.60	-----	-----
4	959.000	.00000	35.60	-----	-----
5	959.000	4800.00000	.00	-----	-----
6	1586.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	128.000	.7000	1.0000
2	959.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	1.2872	1.5121E+08	-360375.
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	1.0967	1.2990E+08	-308933.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	.9140806	1.0888E+08	-258627.

Pile-head Deflection vs. Pile Length

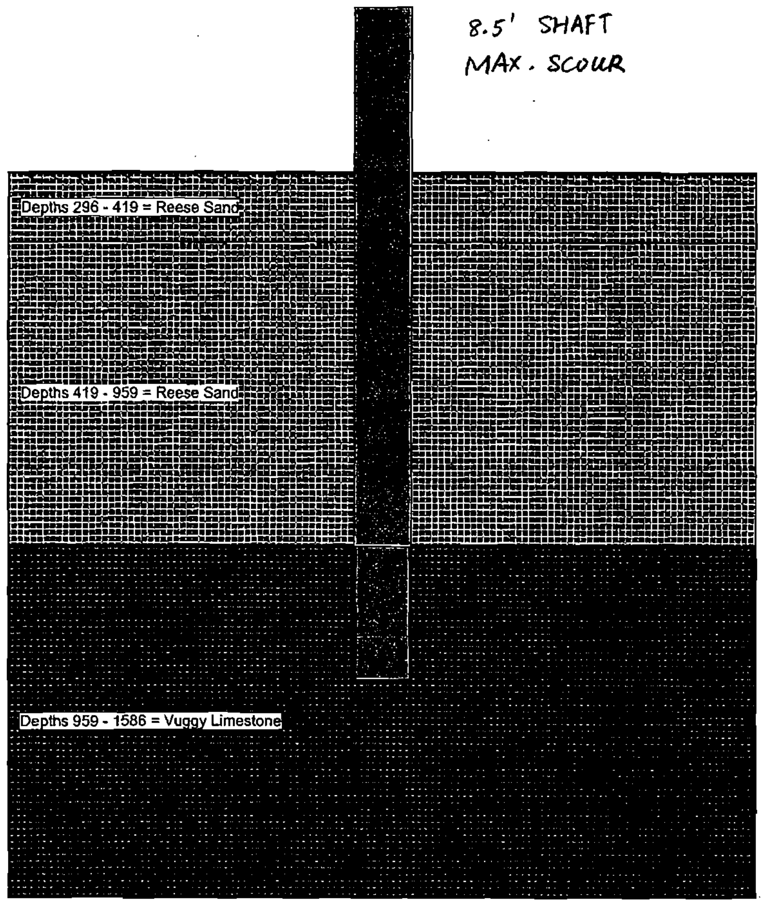
Boundary Condition Type 1, Shear and Moment

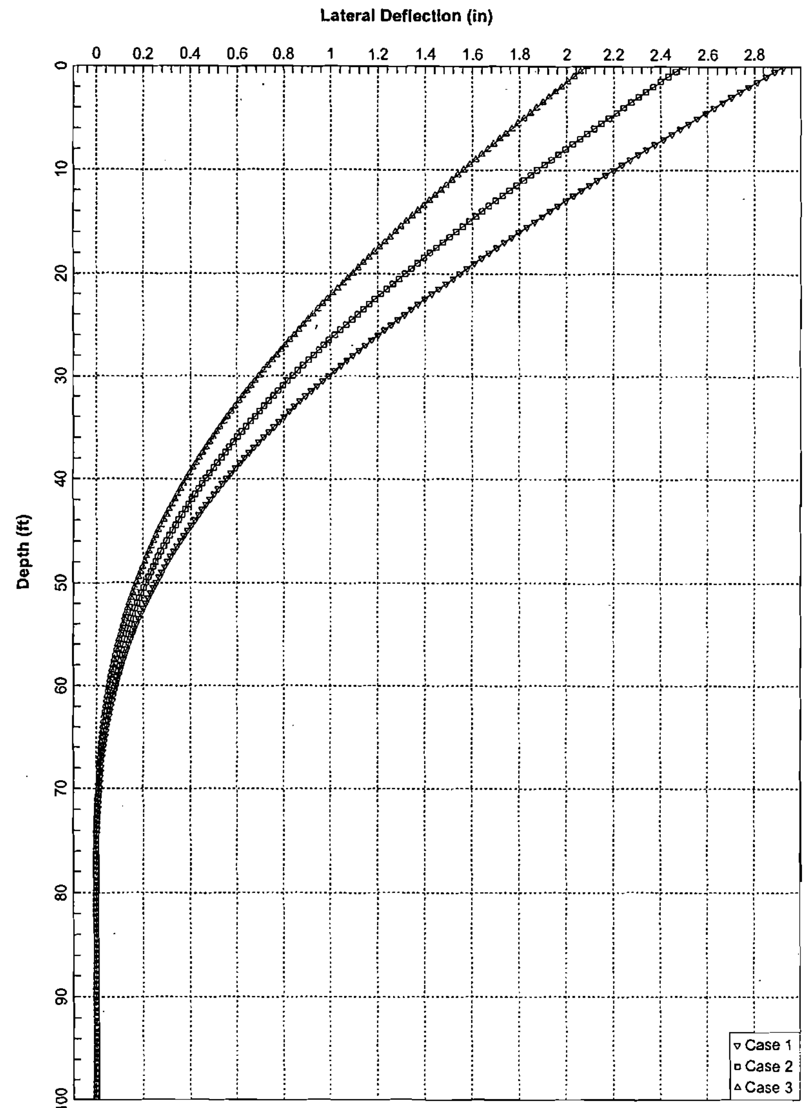
Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1199.000	1.28718309	1.512138E+08	-360375.14573
1139.050	1.28723977	1.512192E+08	-359958.68978
1079.100	1.28766398	1.512142E+08	-359657.62608
1019.150	1.29290143	1.512797E+08	-351597.46655
959.200	1.29977987	1.513060E+08	-351553.60531
899.250	1.35931331	1.513533E+08	-409833.39872
839.300	1.43976476	1.516033E+08	-456810.97735
779.350	1.61777236	1.523813E+08	-525798.76283
719.400	2.06960325	1.547844E+08	-637243.61161

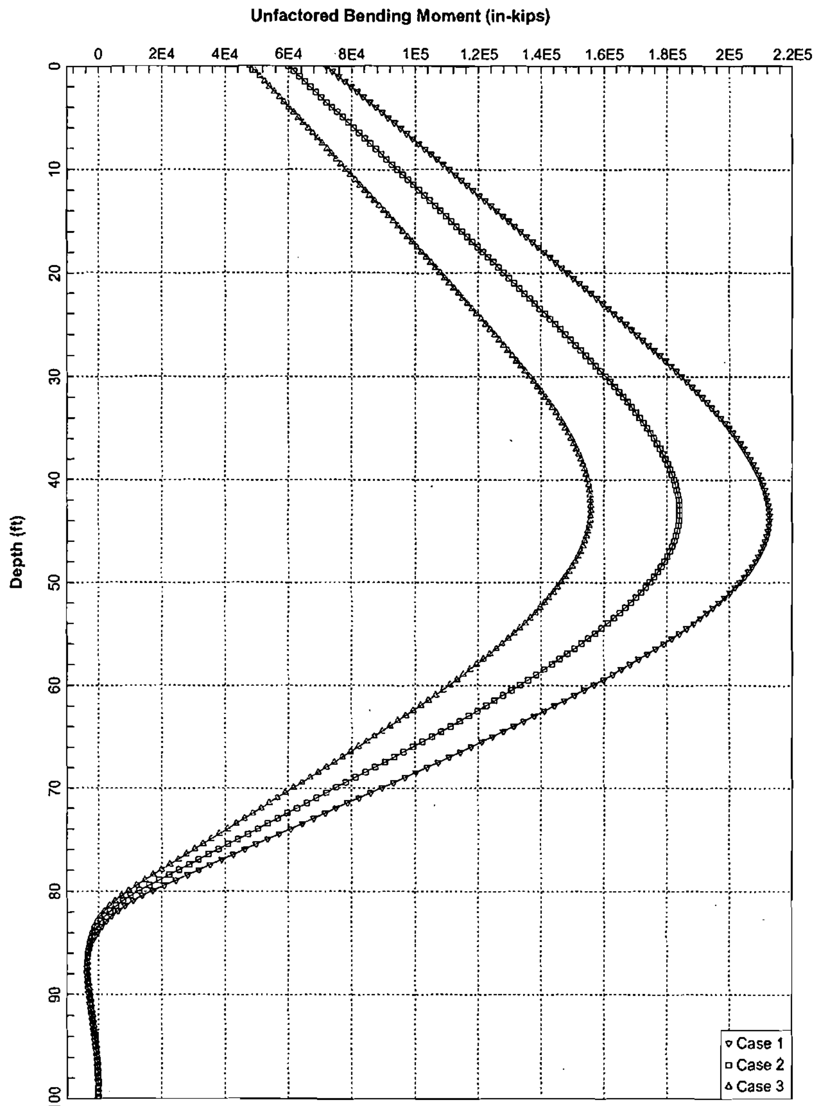
The analysis ended normally.

RUN 6: PIER 2
8.5' SHAFT
MAX. SCOUR



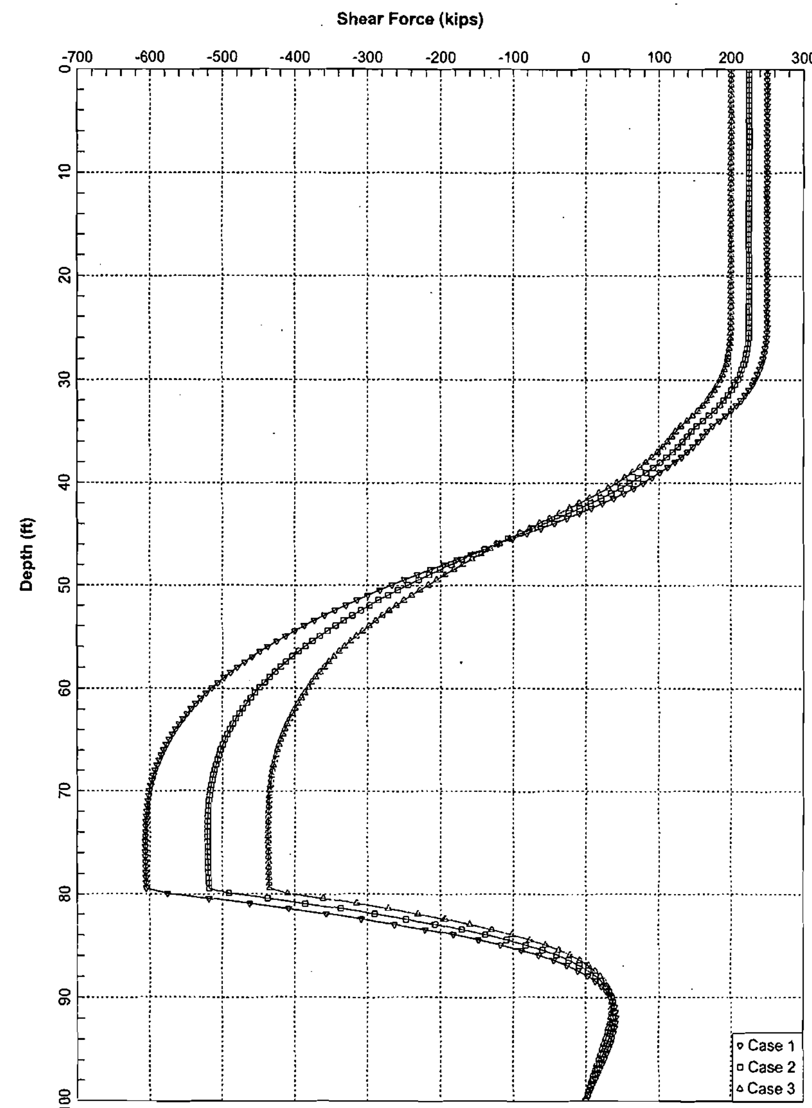


83



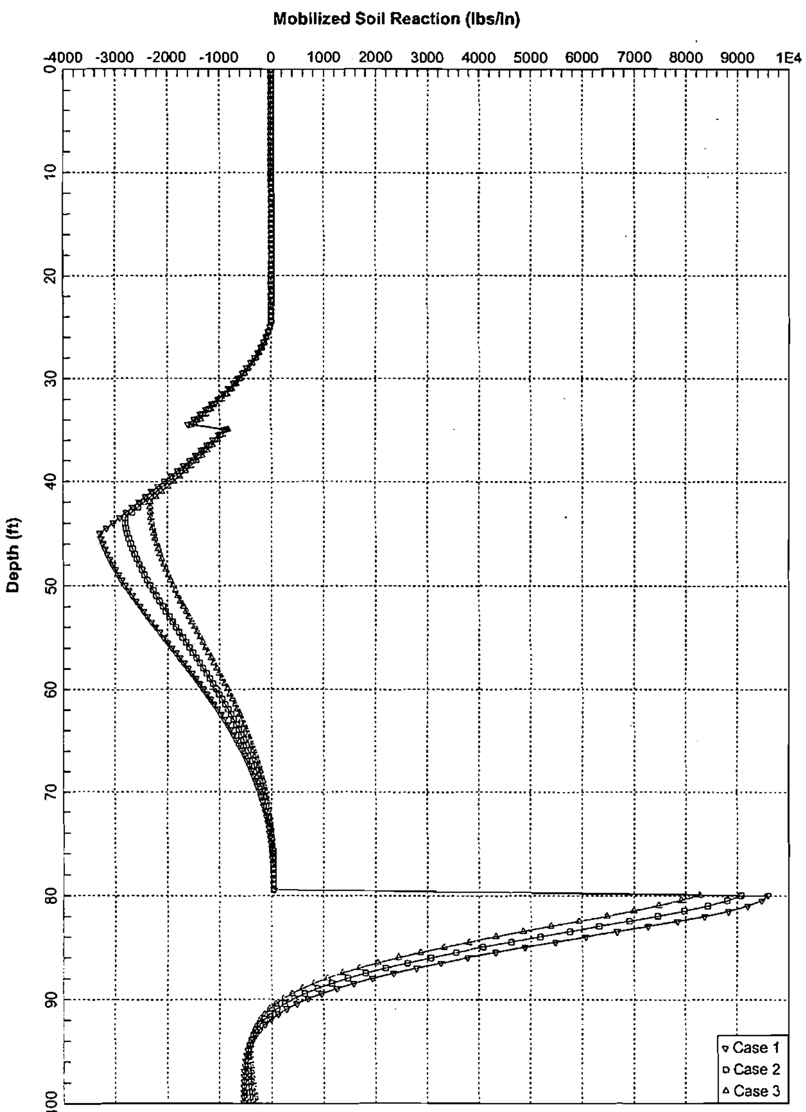
Unfactored Bending Moment (in-kips)
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
2E5
2.2E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

84



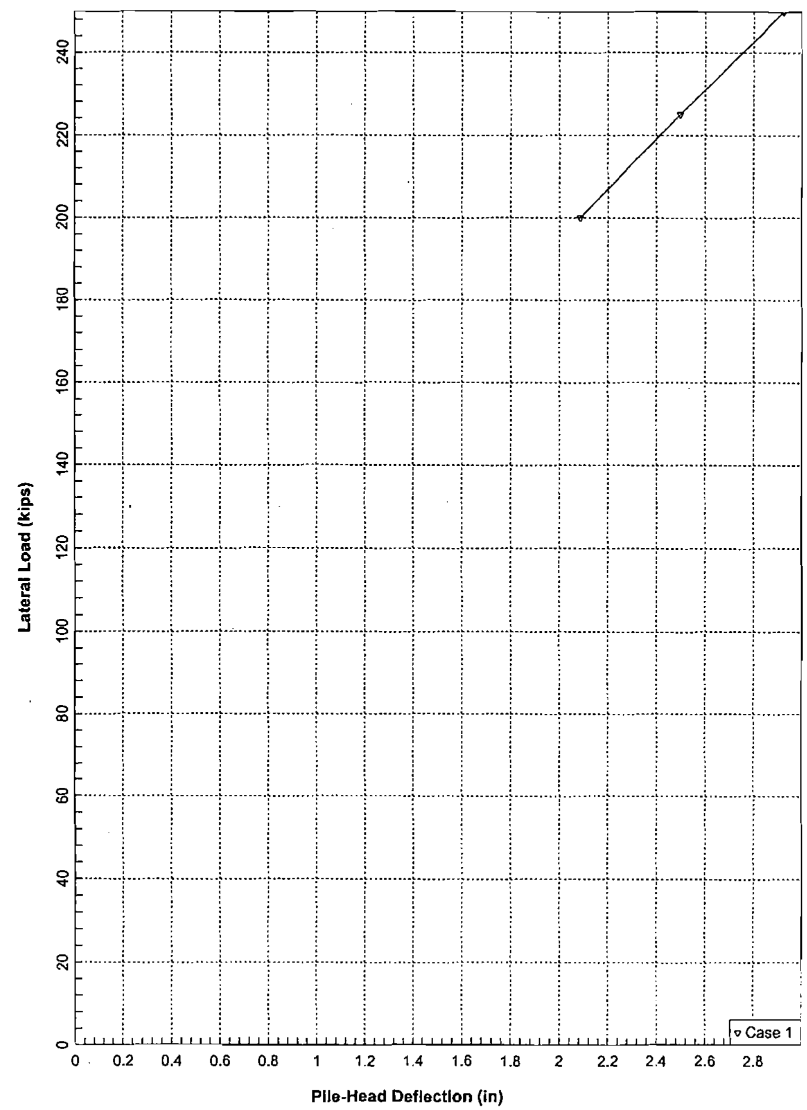
Shear Force (kips)
-700
-600
-500
-400
-300
-200
-100
0
100
200
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

85



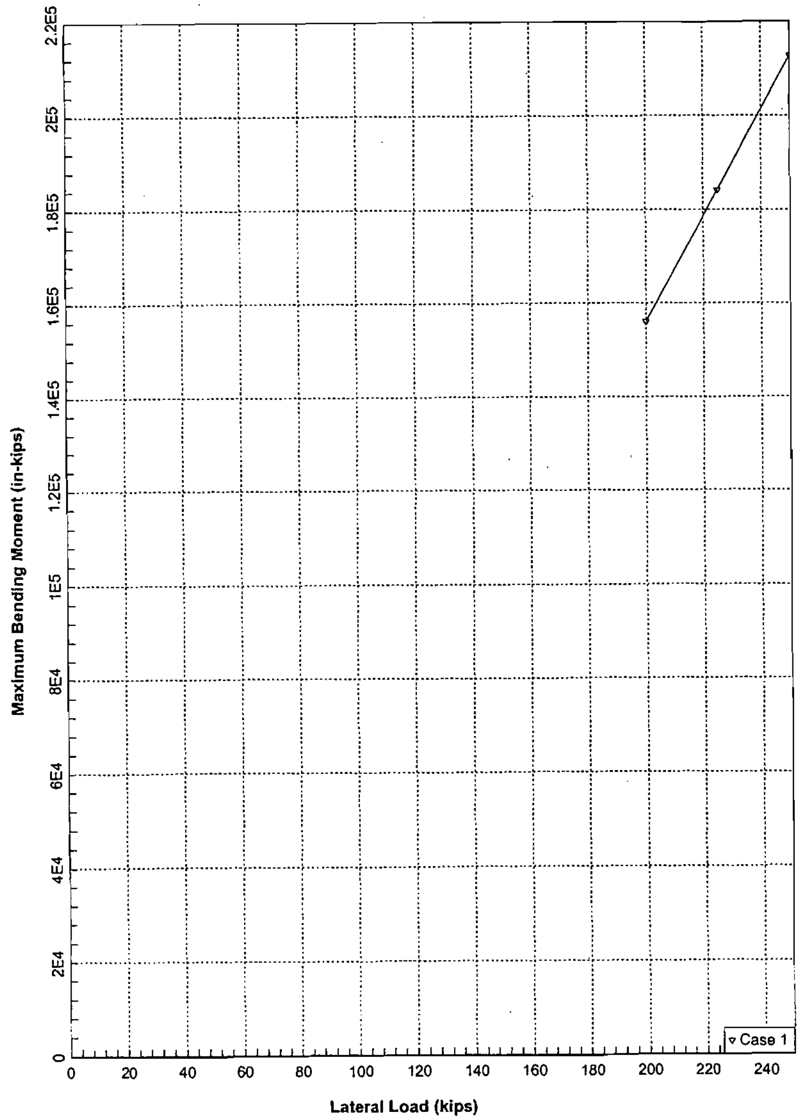
Mobilized Soil Reaction (lbs/in)
-4000
-3000
-2000
-1000
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
1E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

86



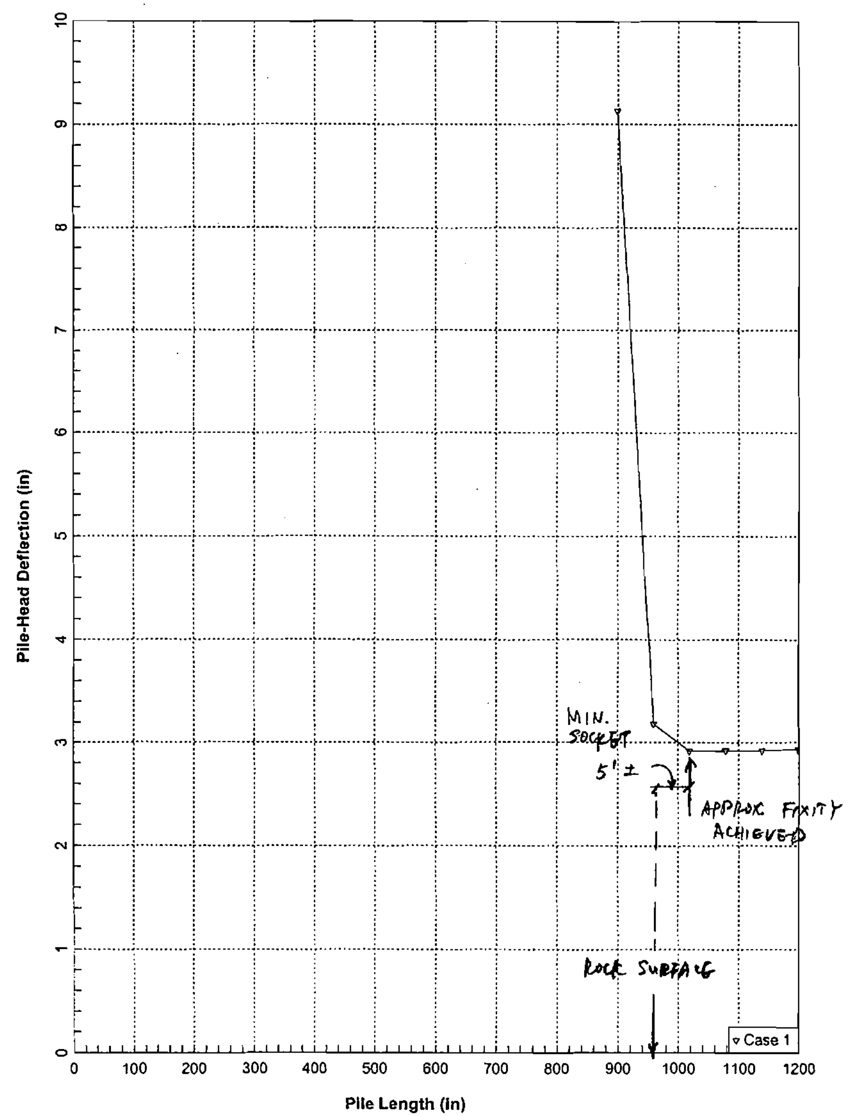
Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Case 1
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
2.6
2.8
Pile-Head Deflection (in)

87



Maximum Bending Moment (in-kips)
Lateral Load (kips)
Case 1

88



Pile-Head Deflection (in)
Pile Length (in)
Case 1
MIN. SOCKET
5' ±
APPROX FIXITY ACHIEVED
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 2\
Name of input data file: Pier 2 - large - scour.lpd
Name of output file: Pier 2 - large - scour.lpo
Name of plot output file: Pier 2 - large - scour.lpp
Name of runtime file: Pier 2 - large - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 10:54:42

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 200
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1199.00 in
Depth of ground surface below top of pile = 296.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	6431065.	9630.7800	4074281.
2	961.0000	102.00000	5431065.	9630.7800	4074281.
3	961.0000	96.00000000	4169220.	7238.2300	4074281.
4	1199.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 296.000 in
Distance from top of pile to bottom of layer = 419.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 419.000 in
Distance from top of pile to bottom of layer = 959.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 959.000 in
Distance from top of pile to bottom of layer = 1586.000 in

(Depth of lowest layer extends 387.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth

is defined using 6 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	296.00	.03935
2	419.00	.03935
3	419.00	.03762
4	959.00	.03762
5	959.00	.05937
6	1586.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	296.000	.00000	35.20	-----	-----
2	419.000	.00000	35.20	-----	-----
3	419.000	.00000	35.60	-----	-----
4	959.000	.00000	35.60	-----	-----
5	959.000	4800.00000	.00	-----	-----
6	1586.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	296.000	.7000	1.0000
2	959.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	2.9258	2.1270E+08	-606173. ✓
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	2.4977	1.8416E+08	-520244.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	2.0859	1.5594E+08	-436727.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

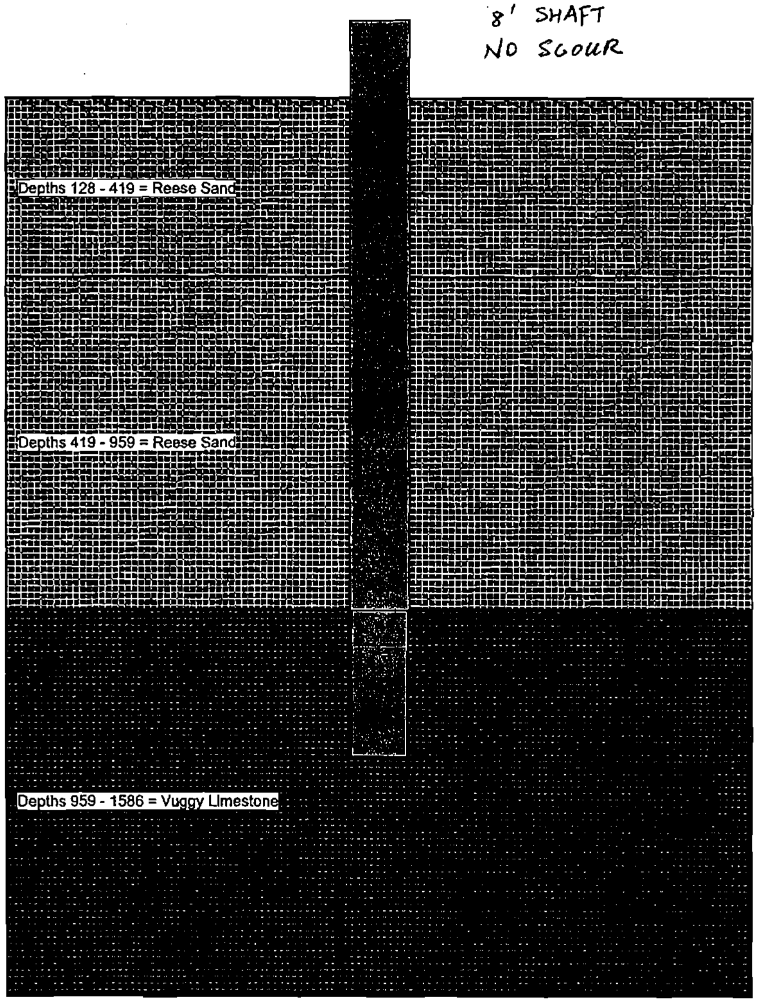
Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1199.000	2.92577295	2.126992E+08	-606173.25751
1139.050	2.92031775	2.124984E+08	-603503.80458
1079.100	2.92131969	2.125477E+08	-602494.24367
1019.150	2.91678142	2.123129E+08	-603326.63438
959.200	3.17899500	2.139229E+08	-673618.34493
899.250	9.12669375	2.568029E+08	-1295352.

The analysis ended normally.

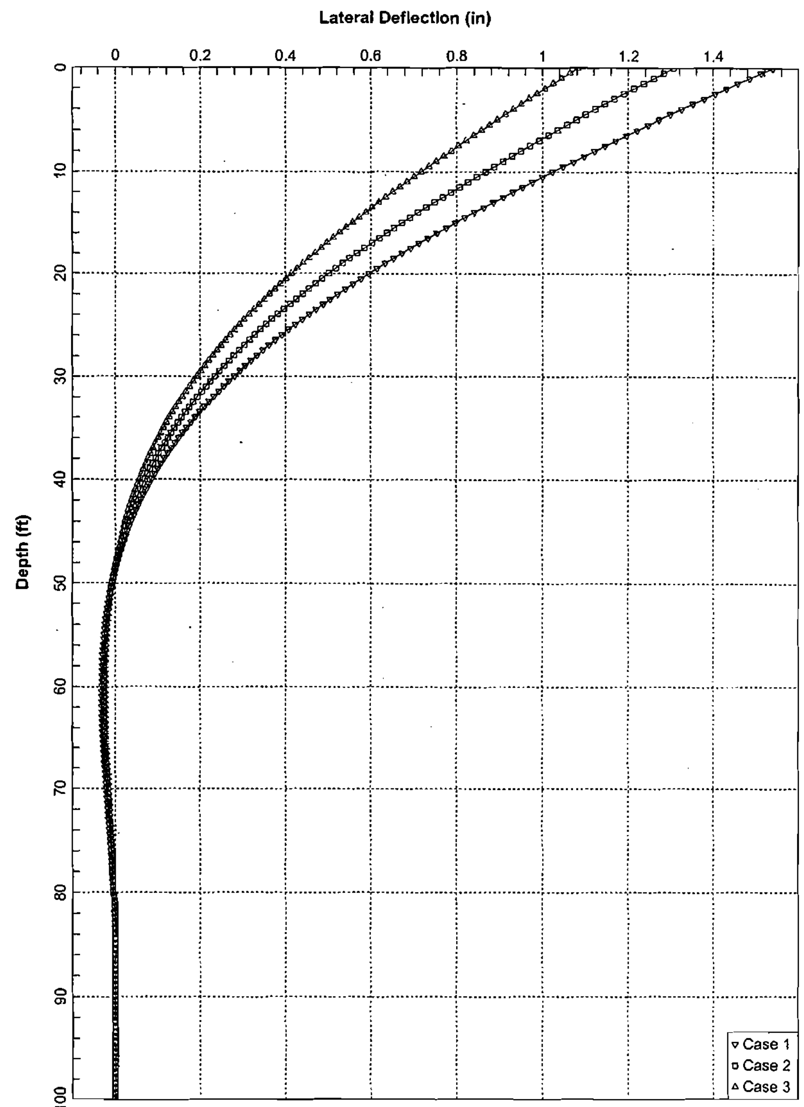
RUN 7: PIER 2
8' SHAFT
NO SCOUR

95

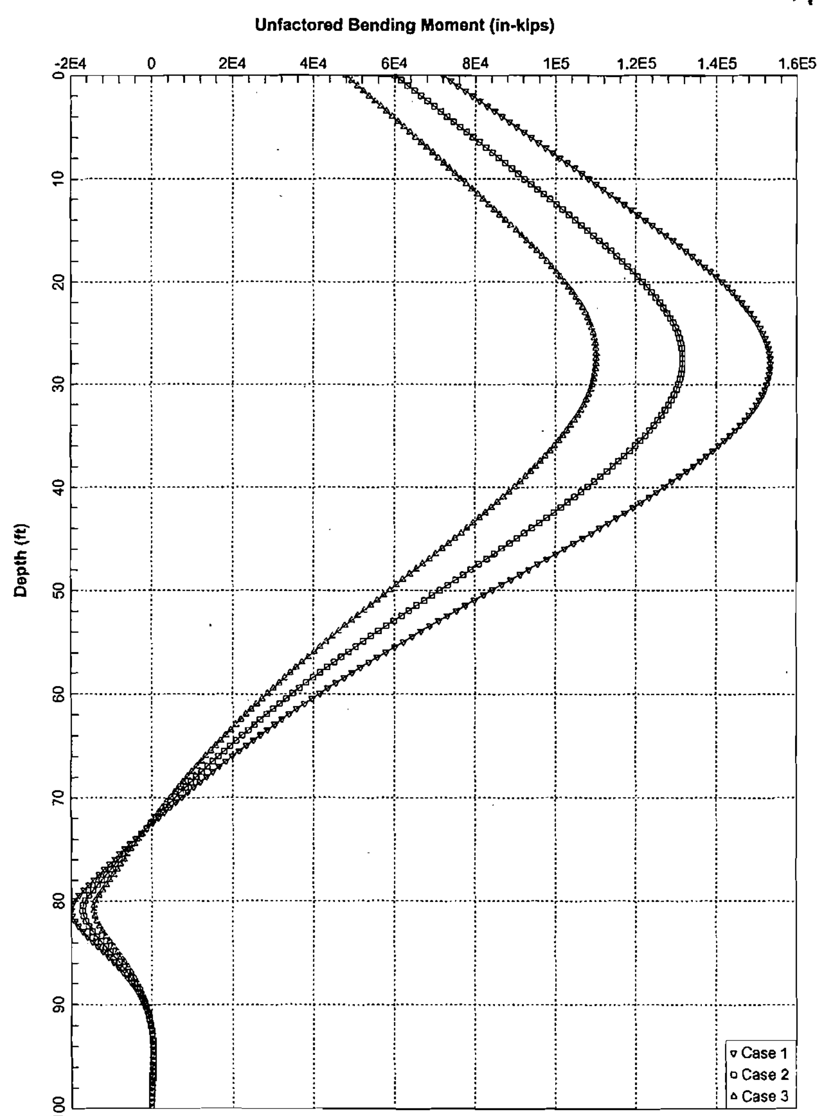


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

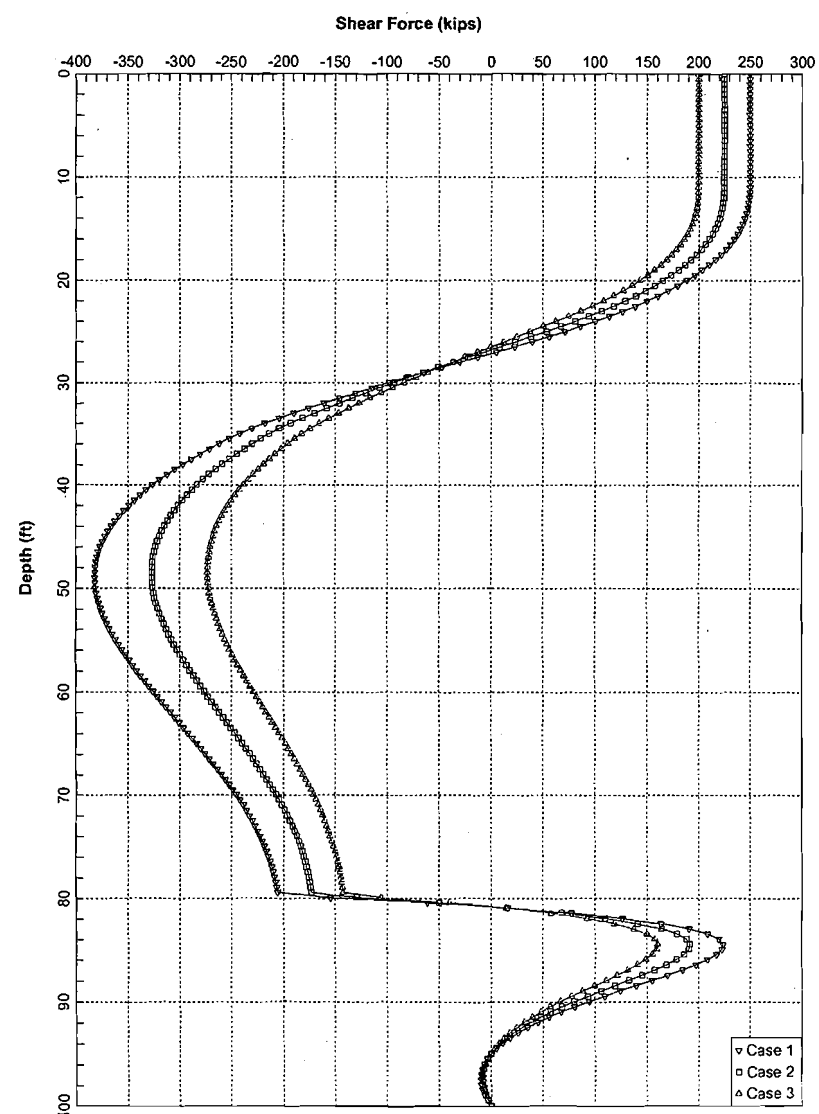
96



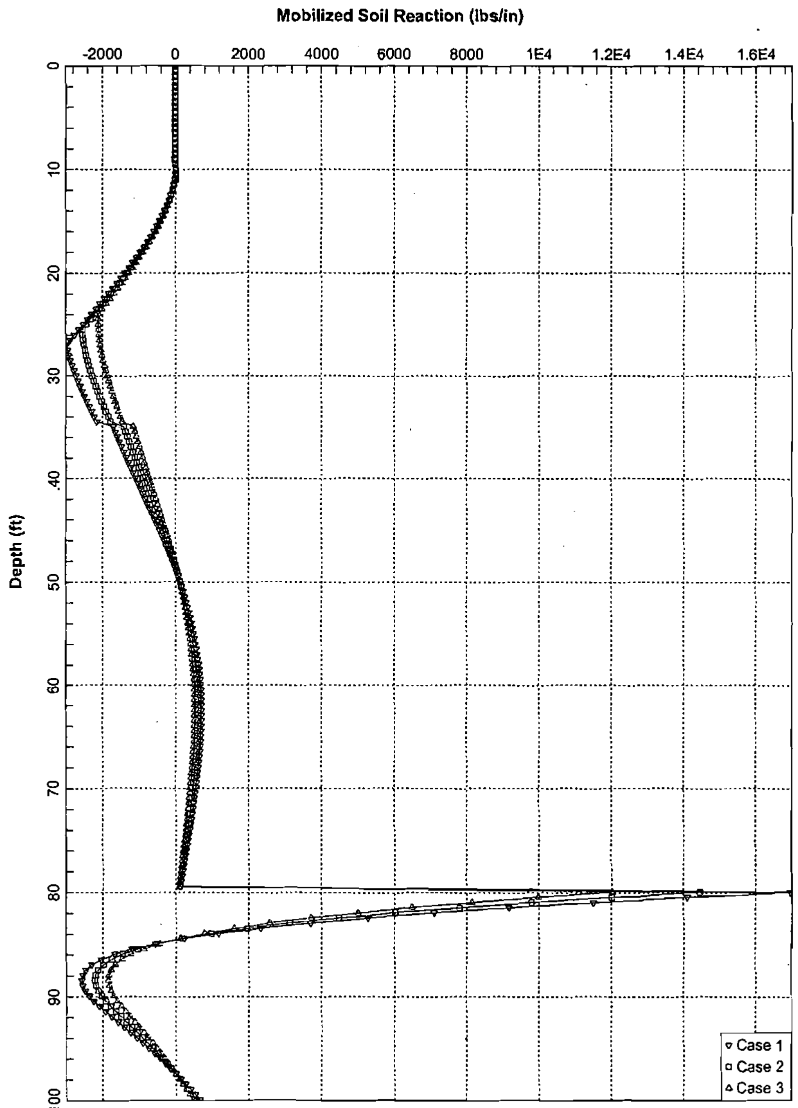
97


Unfactored Bending Moment (in-kips)
-2E4
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

98

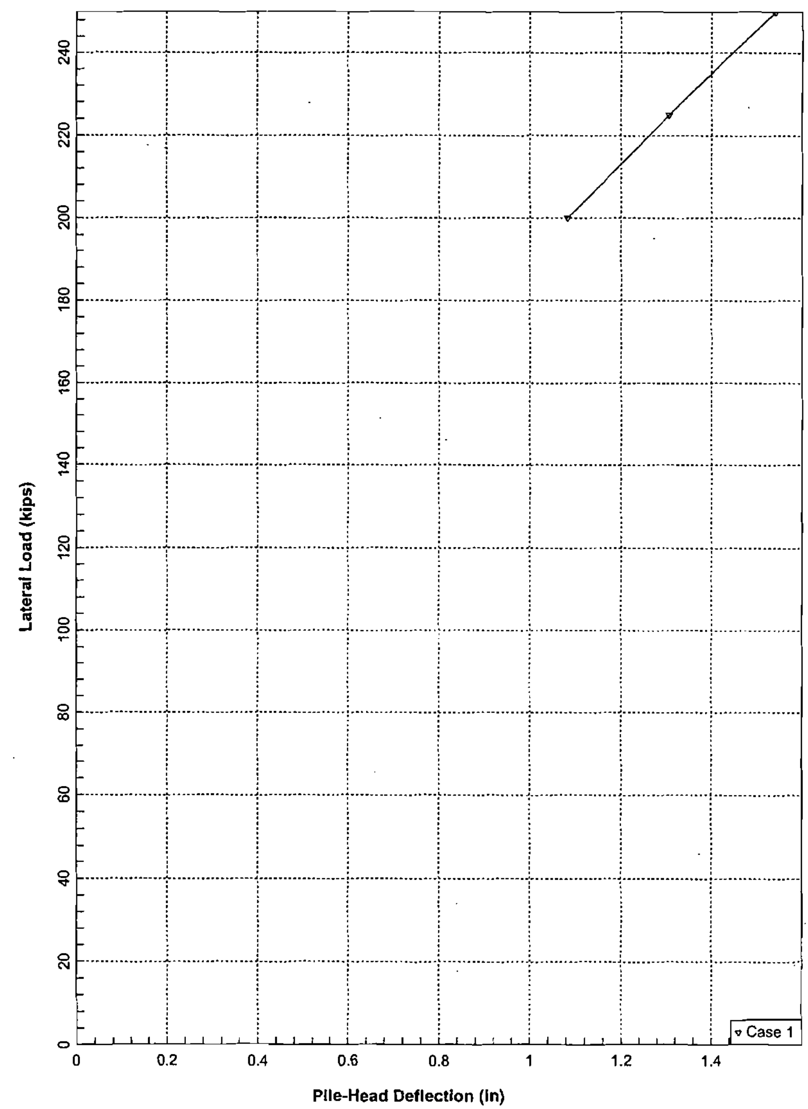

Shear Force (kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

99



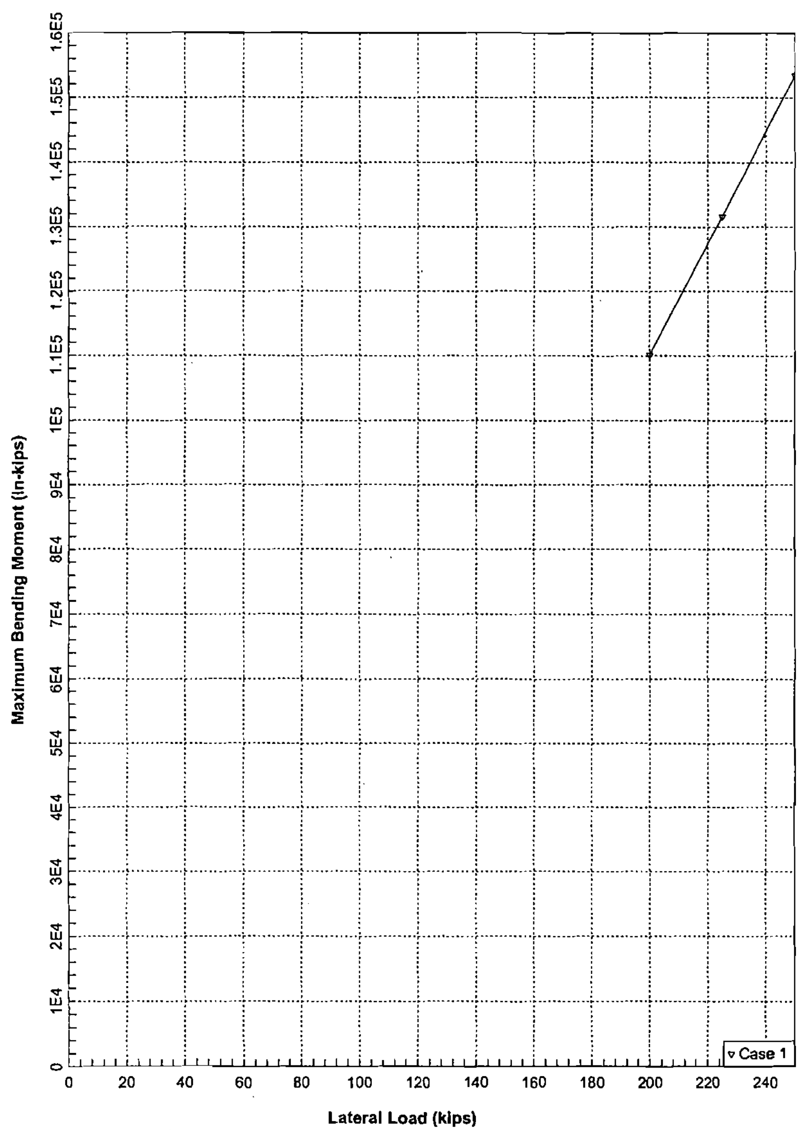
Mobilized Soil Reaction (lbs/in)
-2000
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
1.6E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

100



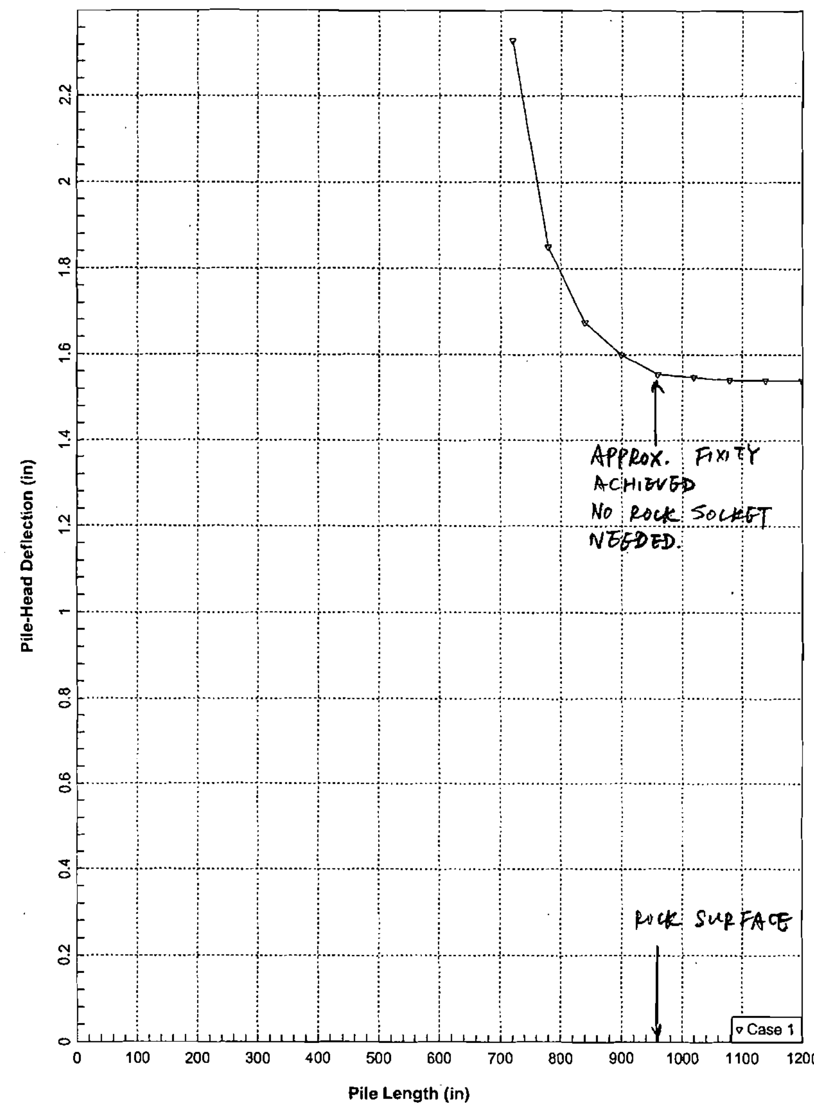
Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Pile-Head Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Case 1

101



Maximum Bending Moment (in-kips)
1.6E5
1.5E5
1.4E5
1.3E5
1.2E5
1.1E5
1E5
9E4
8E4
7E4
6E4
5E4
4E4
3E4
2E4
1E4
0
0 20 40 60 80 100 120 140 160 180 200 220 240
Lateral Load (kips)
Case 1

102



Pile-Head Deflection (in)
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0 100 200 300 400 500 600 700 800 900 1000 1100 1200
Pile Length (in)
Case 1
APPROX. FIXITY ACHIEVED
NO ROCK SOCKET NEEDED.
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 2\
Name of input data file: Pier 2 - small - no scour.lpd
Name of output file: Pier 2 - small - no scour.lpo
Name of plot output file: Pier 2 - small - no scour.lpp
Name of runtime file: Pier 2 - small - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 10:58: 0

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 200
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1199.00 in
Depth of ground surface below top of pile = 128.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	961.0000	96.00000000	4356263.	8611.2400	4074281.
3	961.0000	90.00000000	3220623.	6361.7300	4074281.
4	1199.0000	90.00000000	3220623.	6361.7300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 128.000 in
Distance from top of pile to bottom of layer = 419.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 419.000 in
Distance from top of pile to bottom of layer = 959.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 959.000 in
Distance from top of pile to bottom of layer = 1586.000 in

(Depth of lowest layer extends 387.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth

is defined using 6 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	128.00	.03935
2	419.00	.03935
3	419.00	.03762
4	959.00	.03762
5	959.00	.05937
6	1586.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	128.000	.00000	35.20	-----	-----
2	419.000	.00000	35.20	-----	-----
3	419.000	.00000	35.60	-----	-----
4	959.000	.00000	35.60	-----	-----
5	959.000	4800.00000	.00	-----	-----
6	1586.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	128.000	.7000	1.0000
2	959.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	1.5403	1.5338E+08	-381977.
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	1.3066	1.3162E+08	-326981.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	1.0835	1.1015E+08	-273155.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

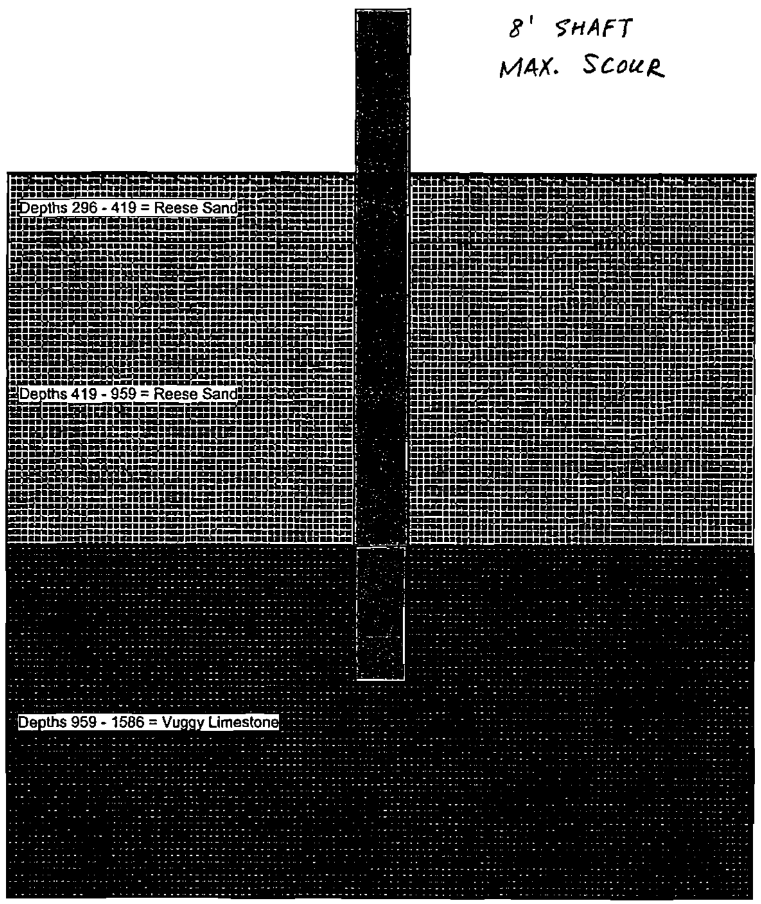
Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1199.000	1.54025774	1.533832E+08	-381976.86233
1139.050	1.54042835	1.533858E+08	-381763.04202
1079.100	1.54061706	1.533877E+08	-381740.33547
1019.150	1.54720160	1.534454E+08	-376821.30325
959.200	1.55410056	1.534928E+08	-374715.81621
899.250	1.59958144	1.535706E+08	-420208.58376
839.300	1.67329427	1.538236E+08	-464595.35213
779.350	1.85014296	1.546273E+08	-534814.55629
719.400	2.32913888	1.572057E+08	-651558.90676

The analysis ended normally.

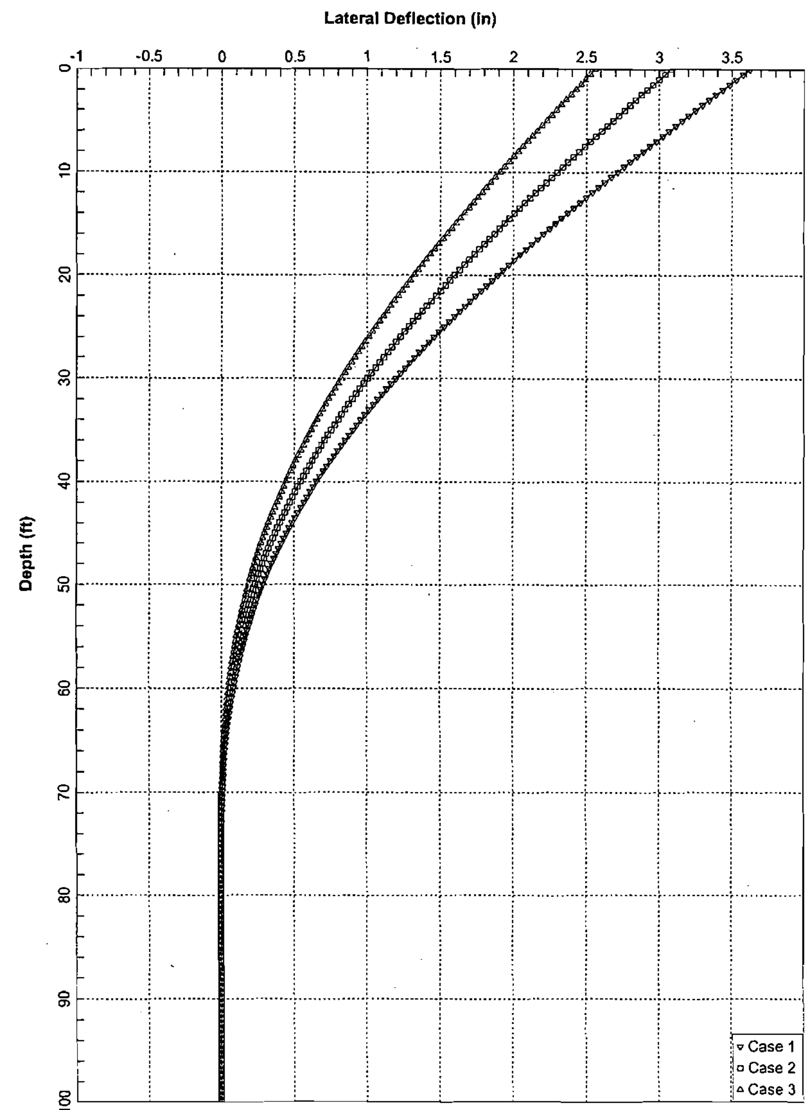
109

RUN 8: PIER 2
8' SHAFT
MAX. SCOUR

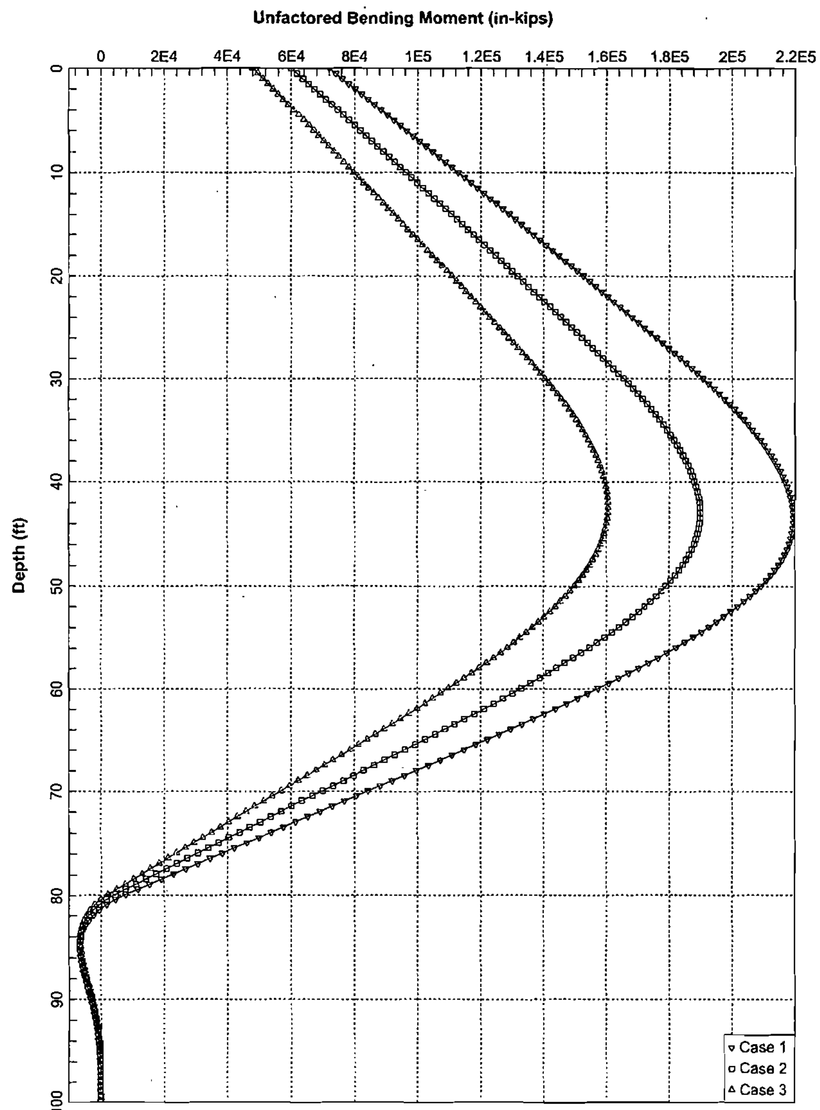


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

110

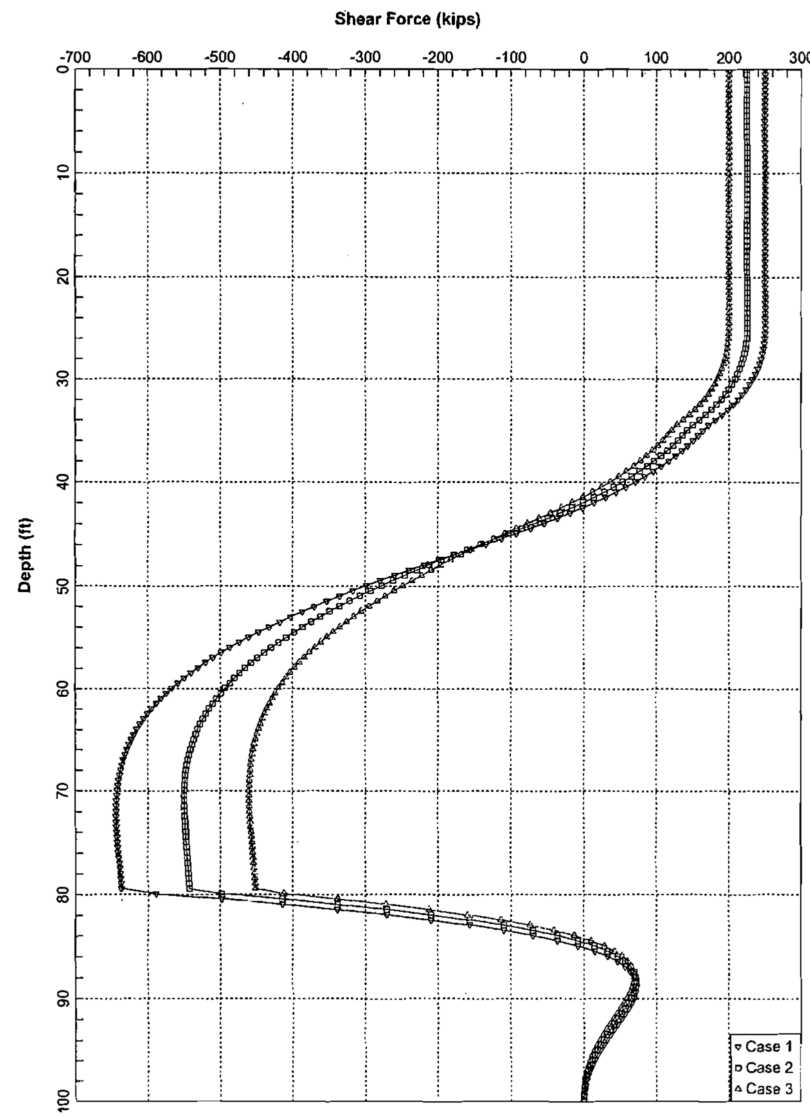


111

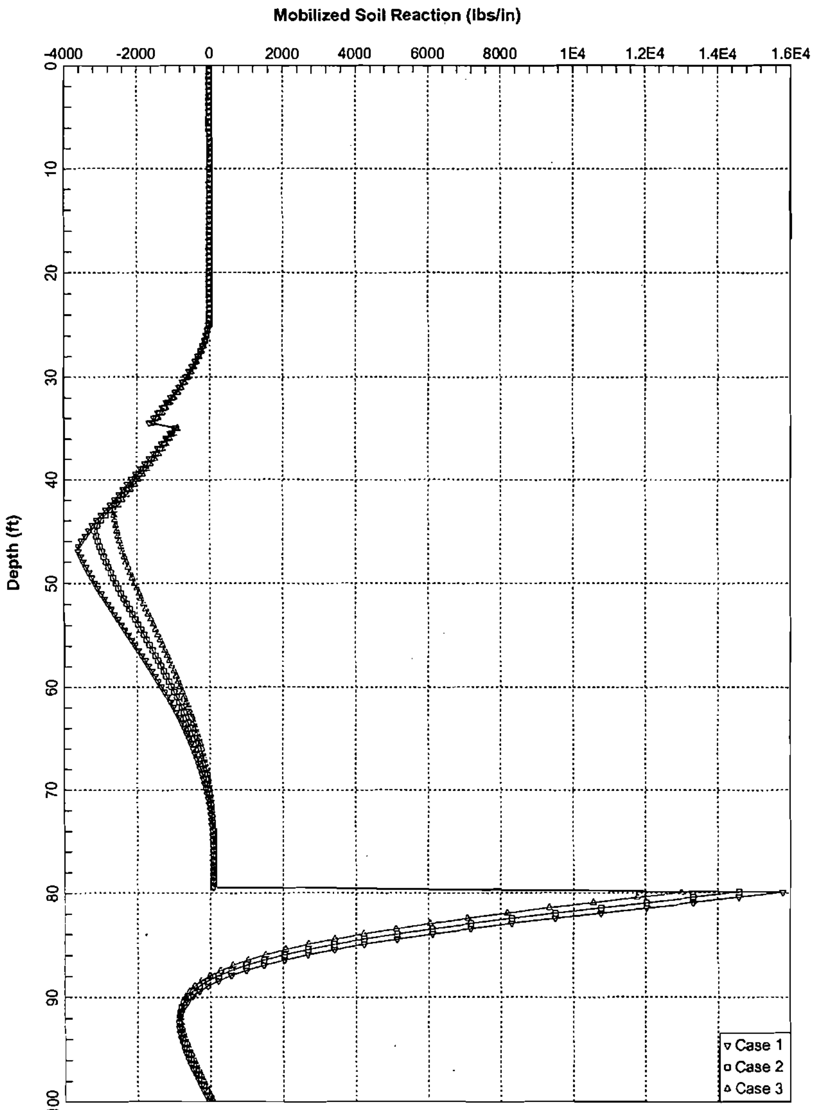


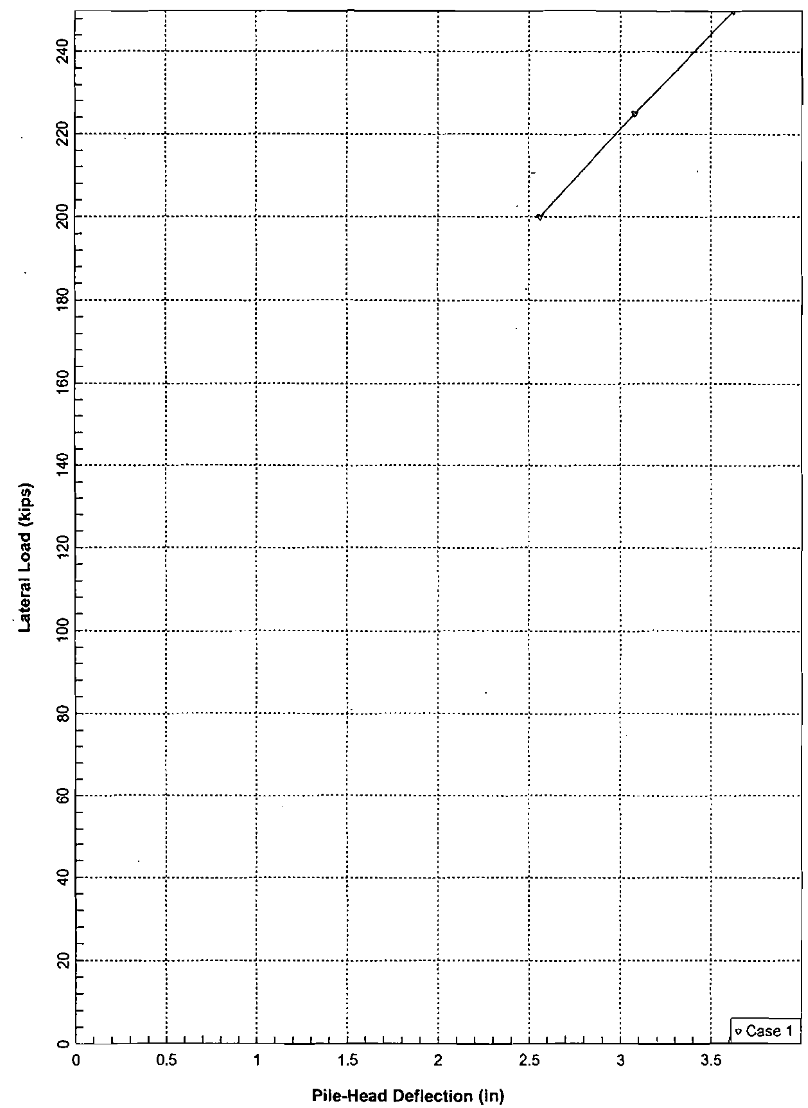
Unfactored Bending Moment (in-kips)
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
2E5
2.2E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

112

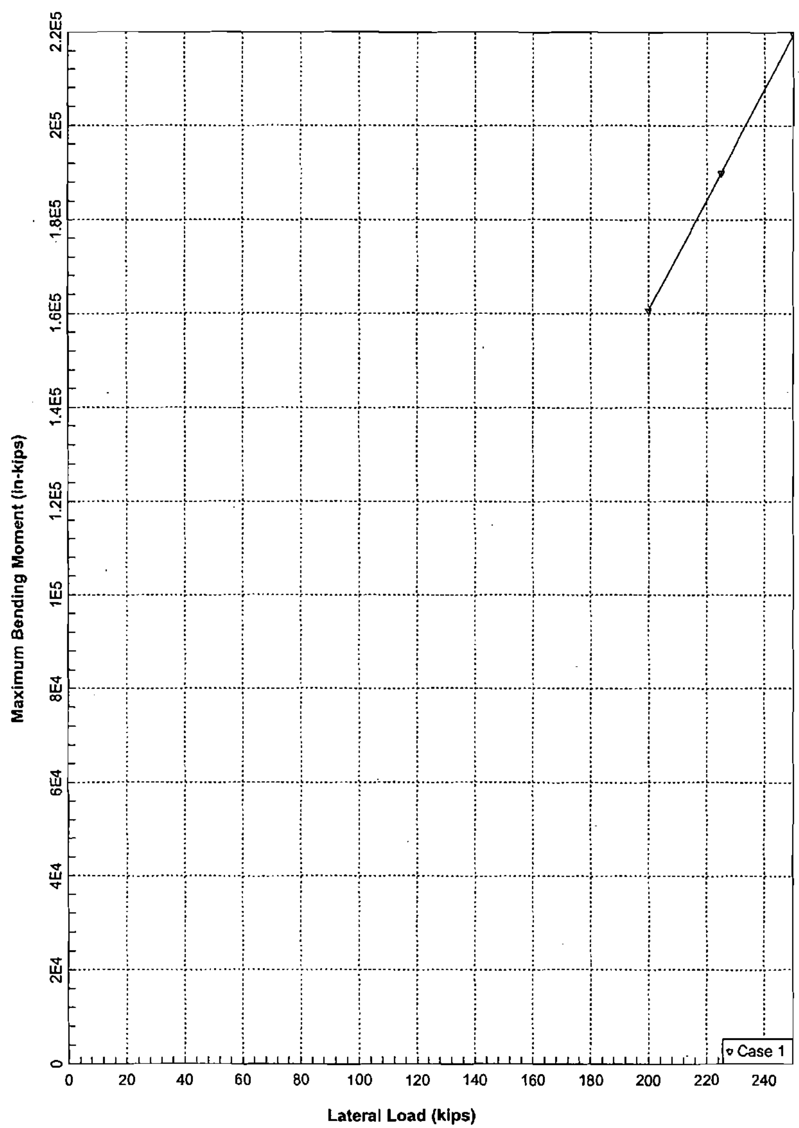


Shear Force (kips)
-700
-600
-500
-400
-300
-200
-100
0
100
200
300
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3


Mobilized Soil Reaction (lbs/in)
-4000
-2000
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
1.6E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
Case 1
Case 2
Case 3

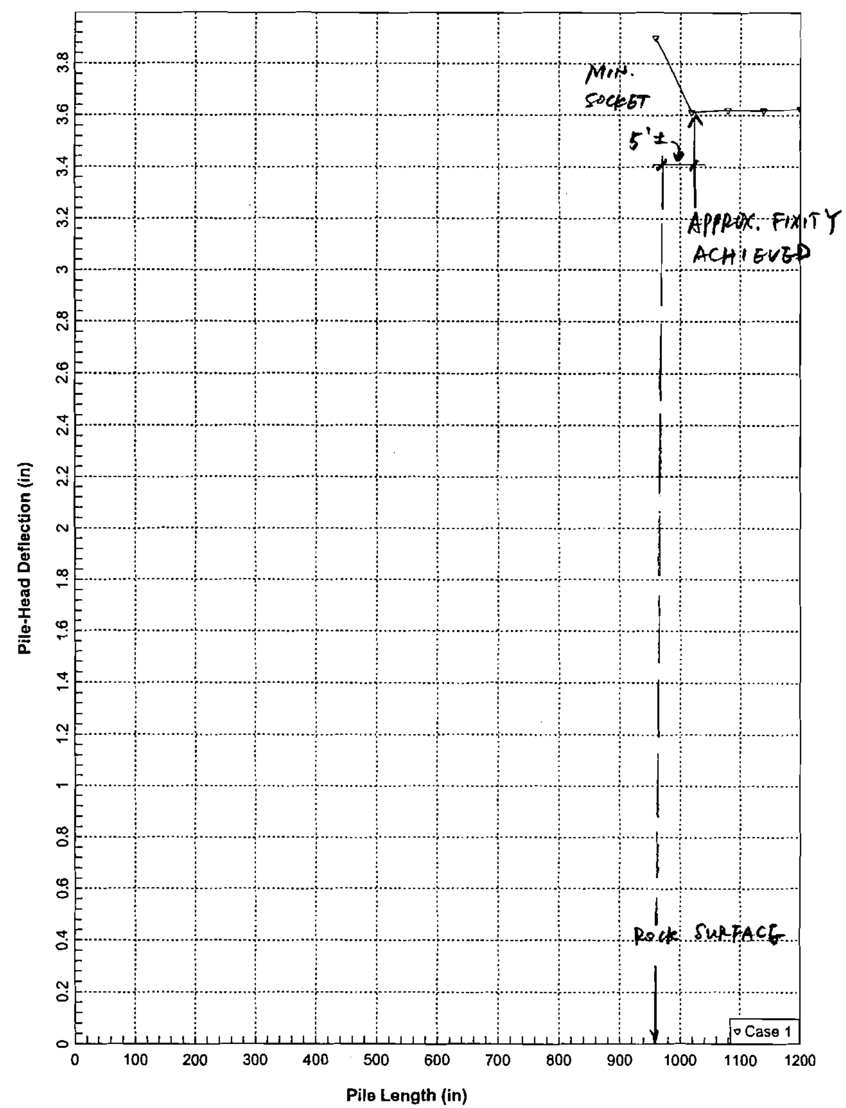

Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
Pile-Head Deflection (in)
0
0.5
1
1.5
2
2.5
3
3.5
Case 1

115



Maximum Bending Moment (in-kips)
Lateral Load (kips)
Case 1

116



Pile-Head Deflection (in)
Pile Length (in)
Case 1
MIN. SOCKET
5'±
APPROX. FIXITY ACHIEVED
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 2\
Name of input data file: Pier 2 - small - scour.lpd
Name of output file: Pier 2 - small - scour.lpo
Name of plot output file: Pier 2 - small - scour.lpp
Name of runtime file: Pier 2 - small - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:0:45

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 200
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1199.00 in
Depth of ground surface below top of pile = 296.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	961.0000	96.00000000	4356263.	8611.2400	4074281.
3	961.0000	90.00000000	3220623.	6361.7300	4074281.
4	1199.0000	90.00000000	3220623.	6361.7300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 296.000 in
Distance from top of pile to bottom of layer = 419.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 419.000 in
Distance from top of pile to bottom of layer = 959.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 959.000 in
Distance from top of pile to bottom of layer = 1586.000 in

(Depth of lowest layer extends 387.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth

is defined using 6 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	296.00	.03935
2	419.00	.03935
3	419.00	.03762
4	959.00	.03762
5	959.00	.05937
6	1586.00	.05937

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	296.000	.00000	35.20	------	------
2	419.000	.00000	35.20	------	------
3	419.000	.00000	35.60	------	------
4	959.000	.00000	35.60	------	------
5	959.000	4800.00000	.00	------	------
6	1586.000	4800.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	296.000	.7000	1.0000
2	959.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 250000.000 lbs
Bending moment at pile head = 72000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 225000.000 lbs
Bending moment at pile head = 60000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head. = 200000.000 lbs
Bending moment at pile head = 48000000.000 in-lbs
Axial load at pile head = 12000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 250000.000 lbs
Specified moment at pile head = 72000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 225000.000 lbs
Specified moment at pile head = 60000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 200000.000 lbs
Specified moment at pile head = 48000000.000 in-lbs
Specified axial load at pile head = 12000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2.50E+05	M= 7.20E+07	1.2000E+07	3.6255	2.1951E+08	-643575.
1	V= 2.25E+05	M= 6.00E+07	1.2000E+07	3.0807	1.8985E+08	-550048.
1	V= 2.00E+05	M= 4.80E+07	1.2000E+07	2.5621	1.6059E+08	-459990.

Pile-head Deflection vs. Pile Length

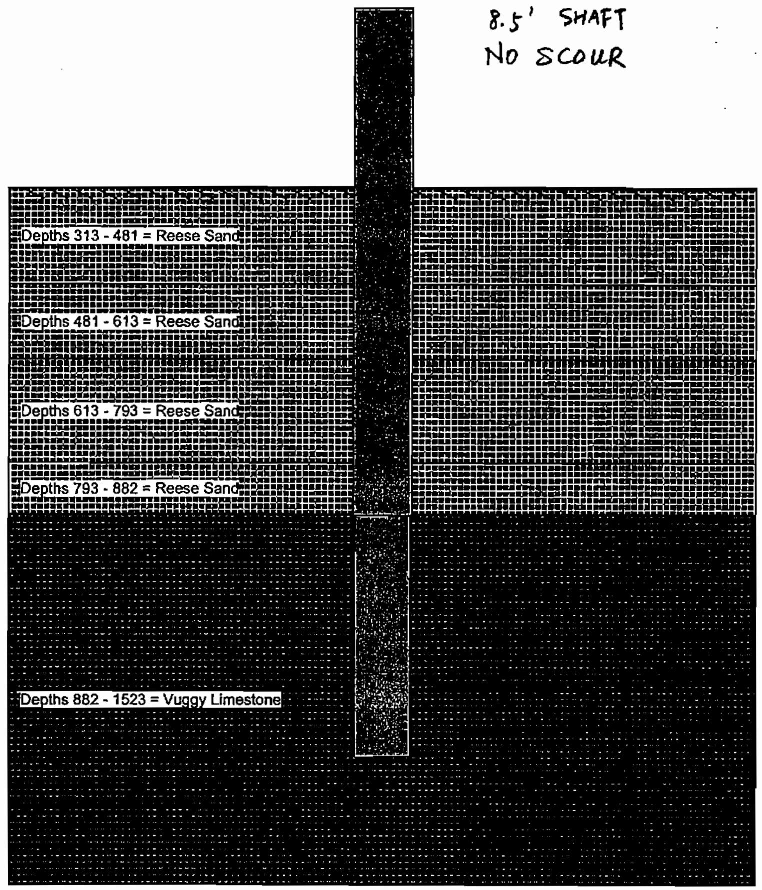
Boundary Condition Type 1, Shear and Moment

Shear = 250000. lbs
Moment = 72000000. in-lbs
Axial Load = 12000000. lbs

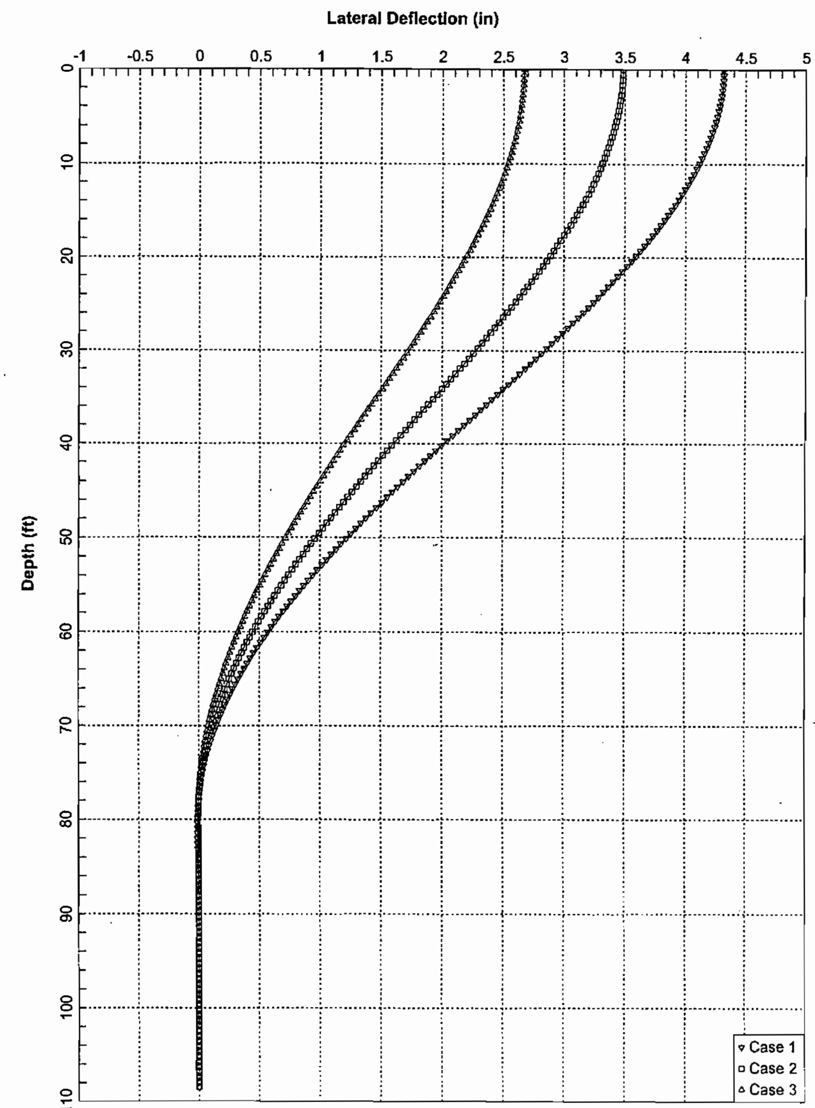
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1199.000	3.62548701	2.195124E+08	-643574.69084
1139.050	3.61847282	2.192932E+08	-639916.37554
1079.100	3.61999056	2.193456E+08	-639473.68340
1019.150	3.61095055	2.191259E+08	-623858.42249
959.200	3.90079836	2.210345E+08	-697692.25936

The analysis ended normally.

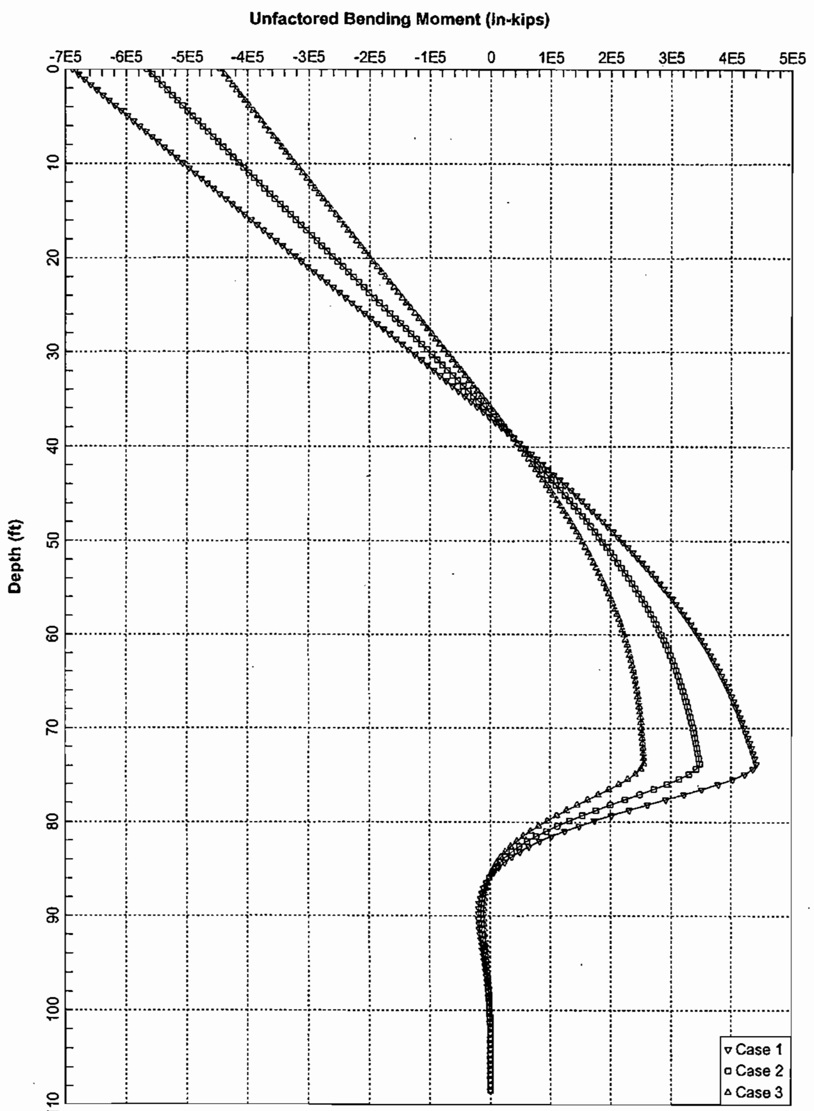
RUN 9: PIERS 3 & 4
8.5' SHAFT
NO SCOUR



LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

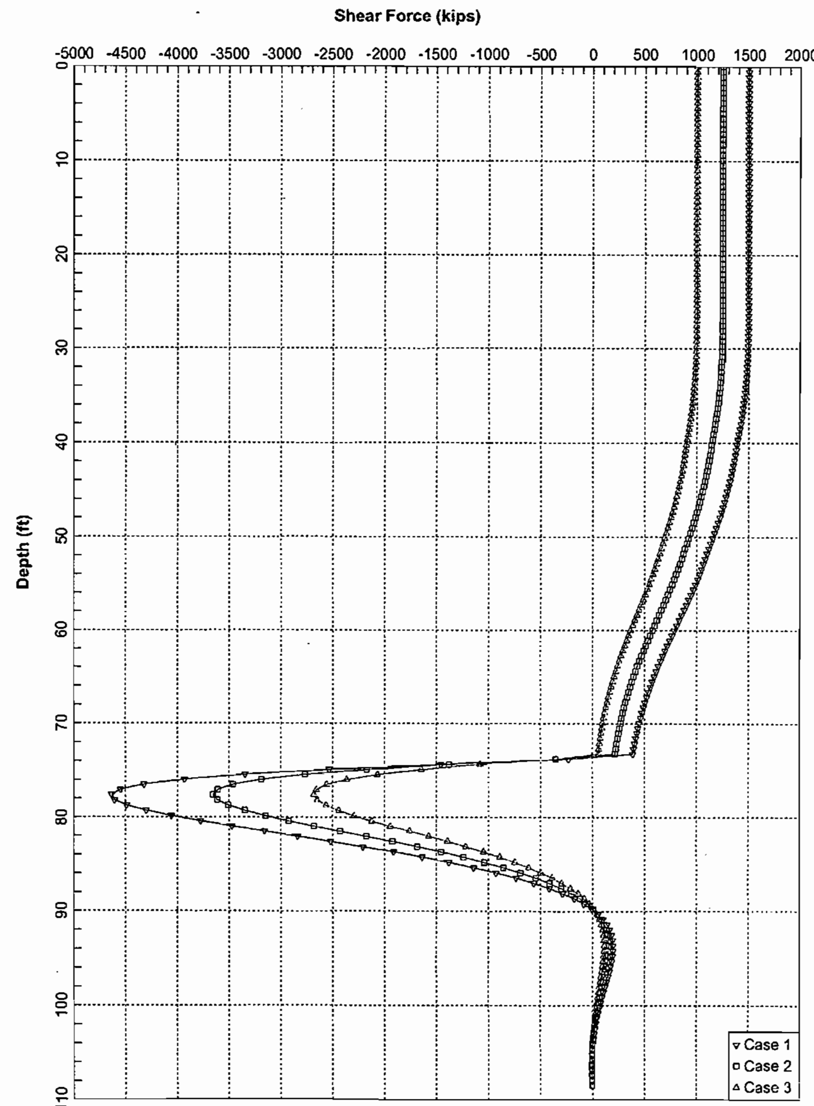


125



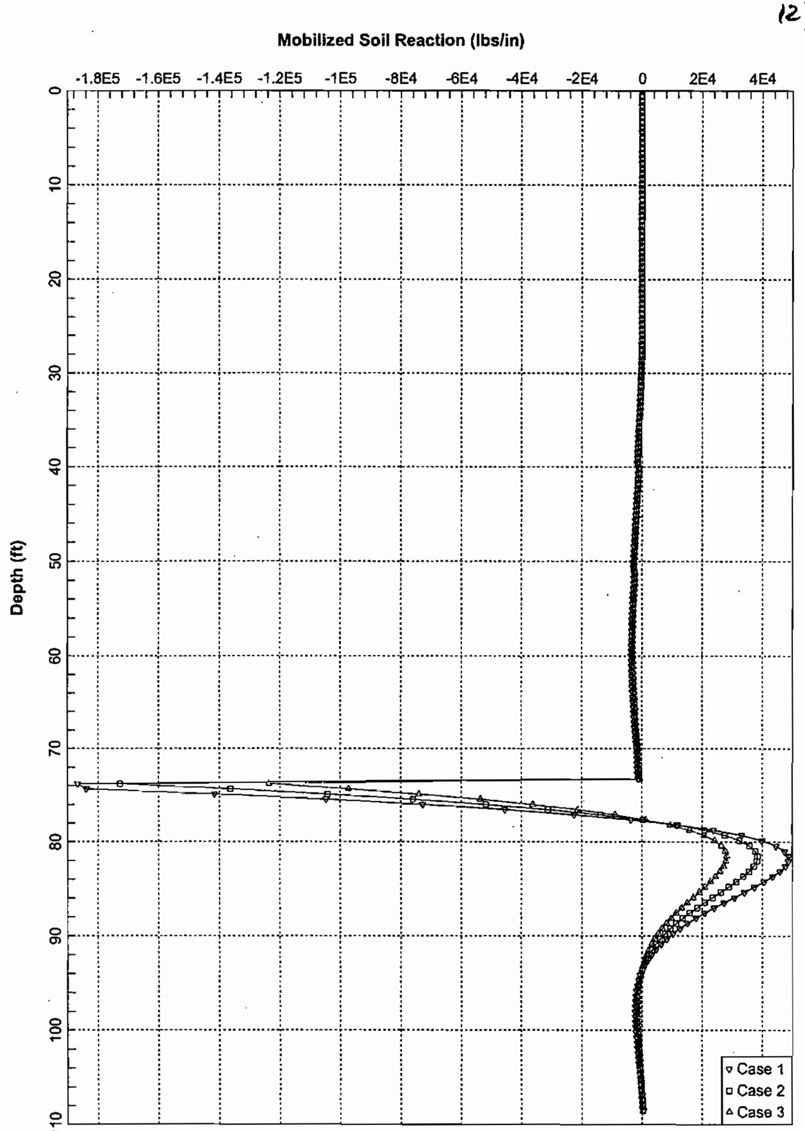
Unfactored Bending Moment (in-kips)
-7E5
-6E5
-5E5
-4E5
-3E5
-2E5
-1E5
0
1E5
2E5
3E5
4E5
5E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

126



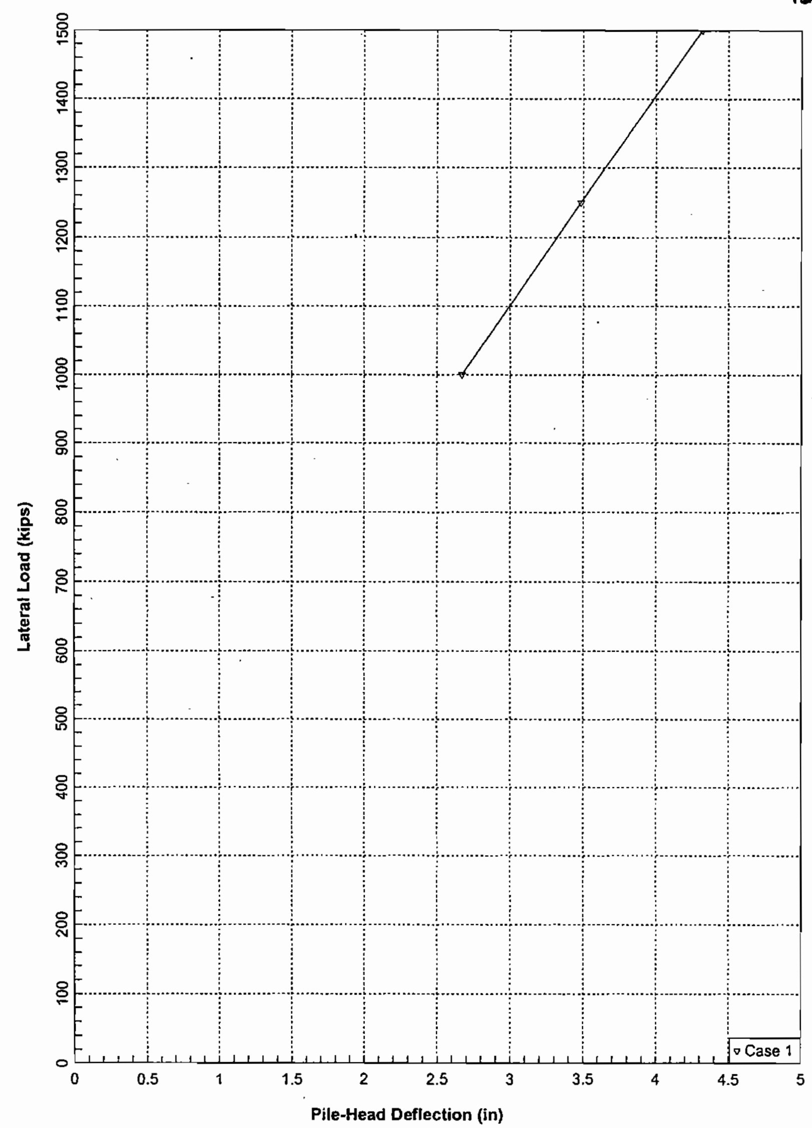
Shear Force (kips)
-5000
-4500
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

127



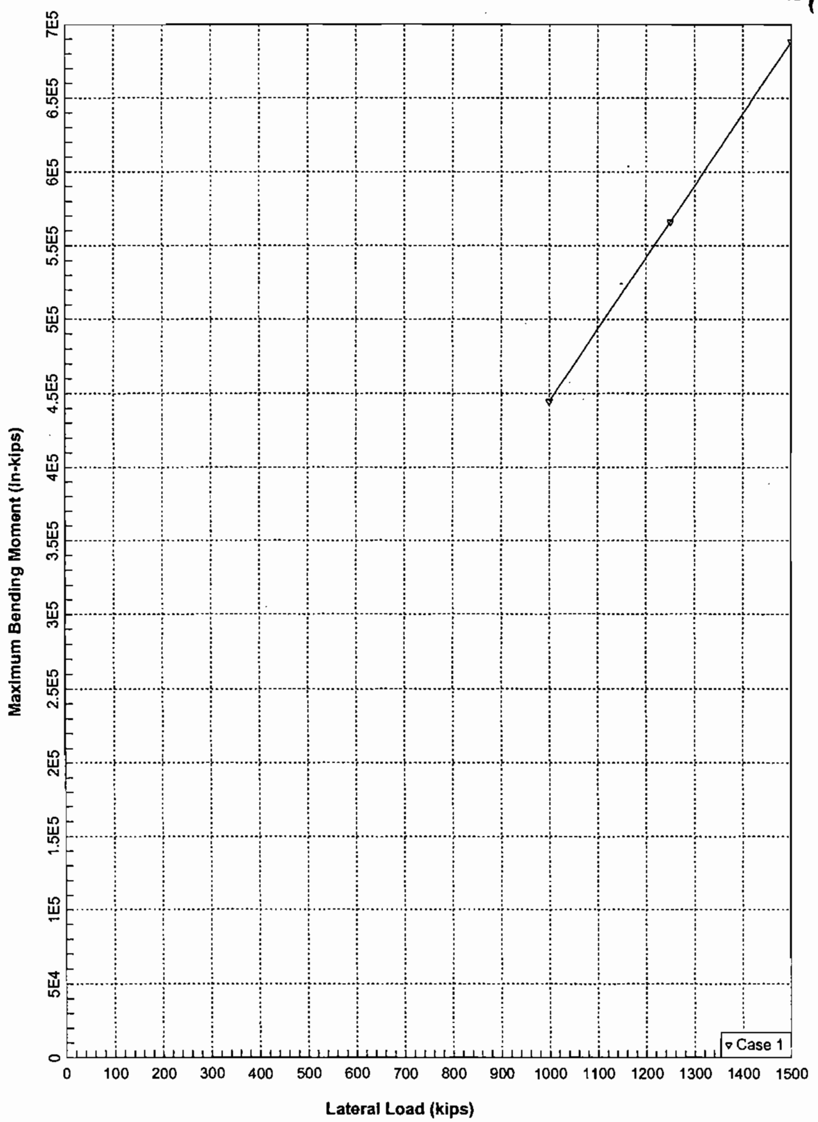
Mobilized Soil Reaction (lbs/in)
-1.8E5
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

128



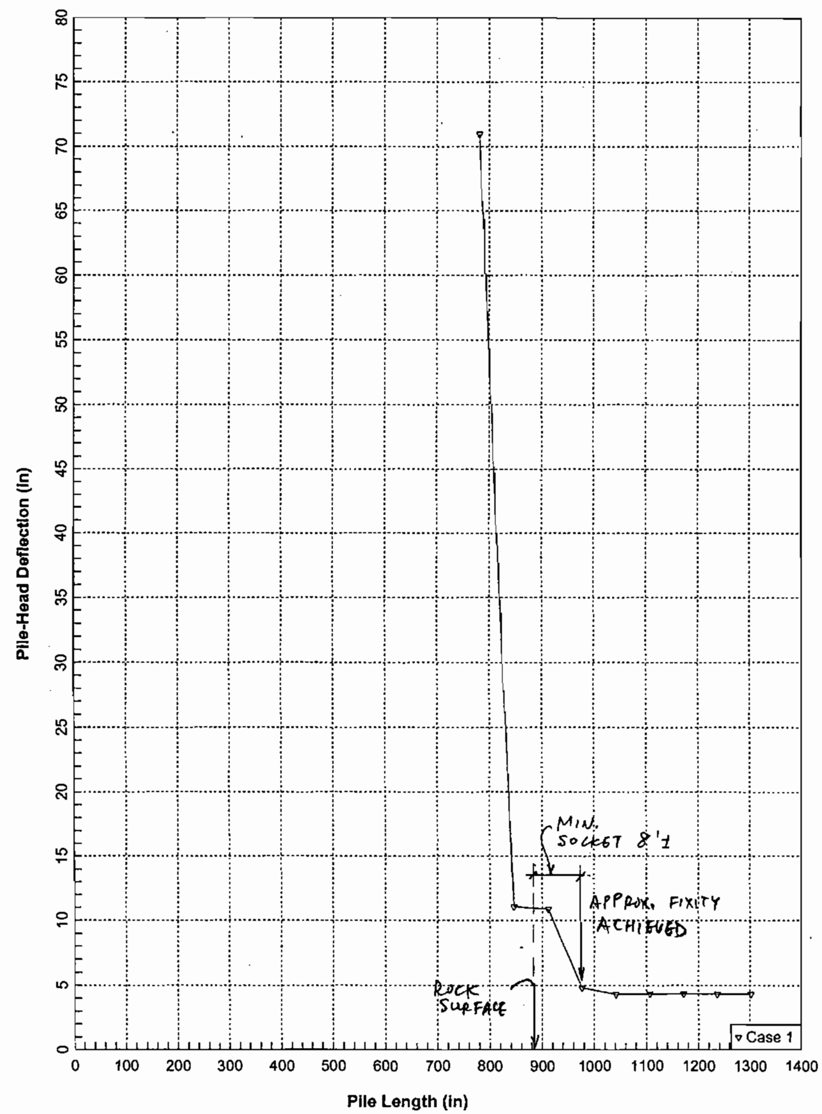
Lateral Load (kips)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
Pile-Head Deflection (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Case 1

129



Maximum Bending Moment (in-kips)
0
5E4
1E5
1.5E5
2E5
2.5E5
3E5
3.5E5
4E5
4.5E5
5E5
5.5E5
6E5
6.5E5
7E5
Case 1
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500
Lateral Load (kips)

130



Pile-Head Deflection (in)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
MIN. SOCKET 8'±
APPROX. FIXITY ACHIEVED
ROCK SURFACE
Case 1
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400
Pile Length (in)

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Piers 3&4\
Name of input data file: Piers 3&4 - large - no scour.lpd
Name of output file: Piers 3&4 - large - no scour.lpo
Name of plot output file: Piers 3&4 - large - no scour.lpp
Name of runtime file: Piers 3&4 - large - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:3:45

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 197
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1302.00 in
Depth of ground surface below top of pile = 313.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	5431065.	9630.7800	4074281.
2	884.0000	102.00000	5431065.	9630.7800	4074281.
3	884.0000	96.00000000	4169220.	7238.2300	4074281.
4	1302.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 313.000 in
Distance from top of pile to bottom of layer = 481.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 481.000 in
Distance from top of pile to bottom of layer = 613.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 613.000 in
Distance from top of pile to bottom of layer = 793.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 793.000 in
Distance from top of pile to bottom of layer = 882.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 882.000 in
Distance from top of pile to bottom of layer = 1523.000 in

(Depth of lowest layer extends 221.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	313.00	.03819
2	481.00	.03819
3	481.00	.03877
4	613.00	.03877
5	613.00	.04109
6	793.00	.04109
7	793.00	.03993
8	882.00	.03993
9	882.00	.05880
10	1523.00	.05880

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	313.000	.00000	34.50	-----	-----
2	481.000	.00000	34.50	-----	-----
3	481.000	.00000	36.90	-----	-----
4	613.000	.00000	36.90	-----	-----
5	613.000	.00000	38.00	-----	-----
6	793.000	.00000	38.00	-----	-----
7	793.000	.00000	38.00	-----	-----
8	882.000	.00000	38.00	-----	-----
9	882.000	4800.00000	.00	-----	-----
10	1523.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	313.000	.3500	1.0000
2	882.000	.3500	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1500000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1250000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1000000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1500000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1250000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1000000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in

Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
2	V= 1.50E+06	S= 0.000	1.2000E+07	4.3195	-6.8820E+08	-4632701.
2	V= 1.25E+06	S= 0.000	1.2000E+07	3.4848	-5.6584E+08	-3654073.
2	V= 1.00E+06	S= 0.000	1.2000E+07	2.6711	-4.4466E+08	-2678877.

Pile-head Deflection vs. Pile Length

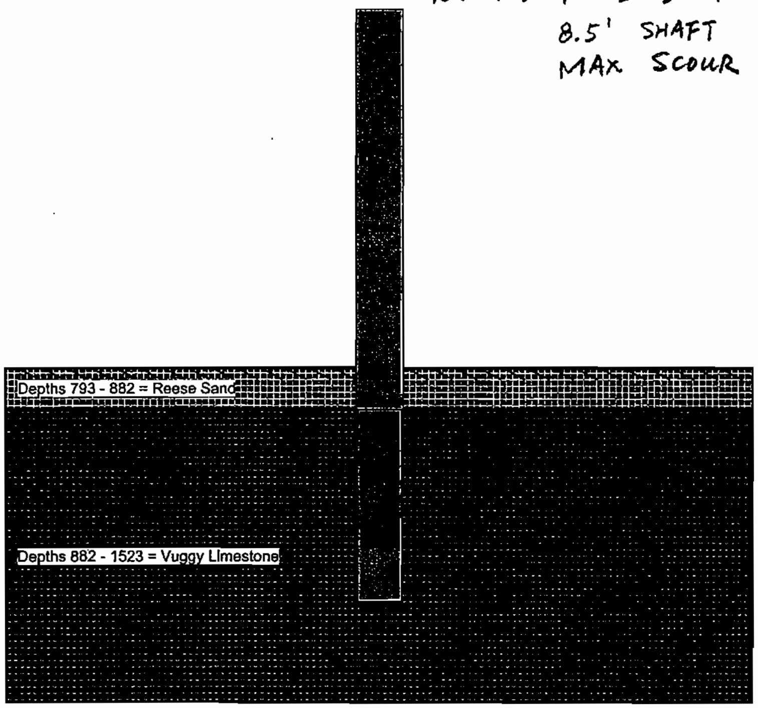
Boundary Condition Type 2, Shear and Slope

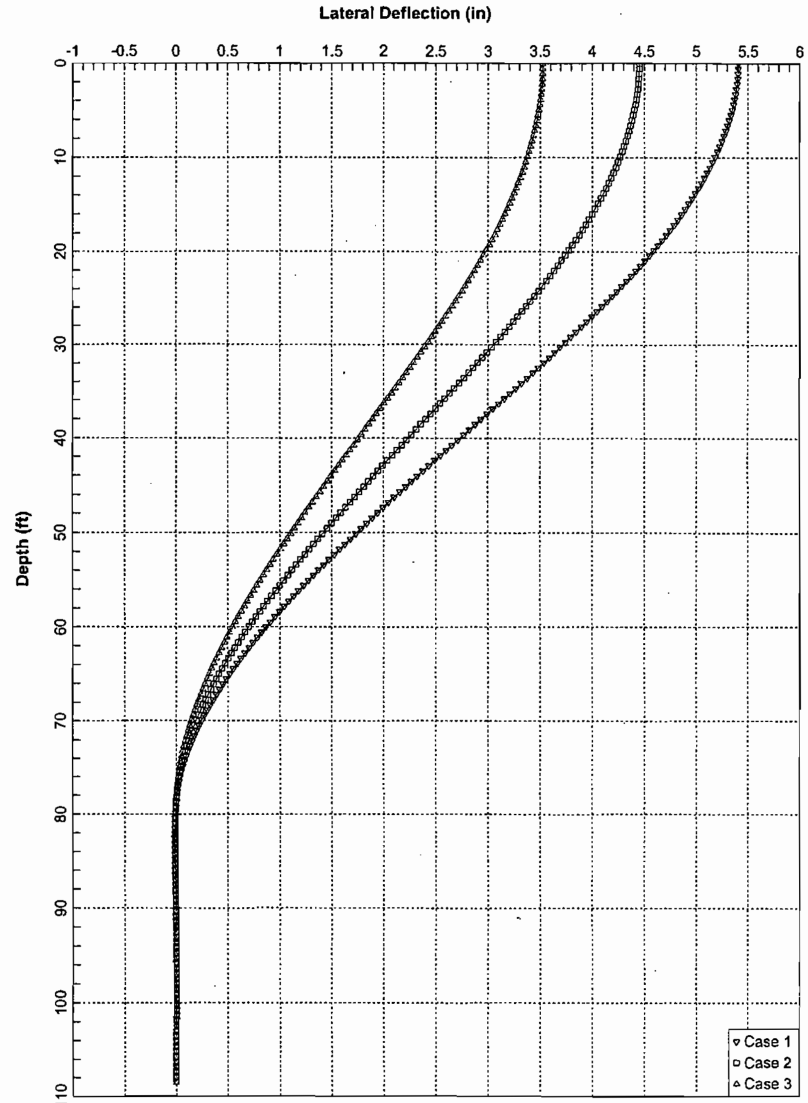
Shear = 1500000. lbs
Slope = .00000
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1302.000	4.31950401	-6.881955E+08	-4632701.
1236.900	4.31692165	-6.880399E+08	-4628097.
1171.800	4.32757523	-6.885533E+08	-4630273.
1106.700	4.34196800	-6.892204E+08	-4594601.
1041.600	4.29332090	-6.868497E+08	-4727464.
976.500	4.79857211	-7.130581E+08	-6993924.
911.400	10.86484965	-1.062344E+09	-2558967.
846.300	11.05195011	-1.123888E+09	1500000.
781.200	70.93442059	-1.084775E+09	1500016.

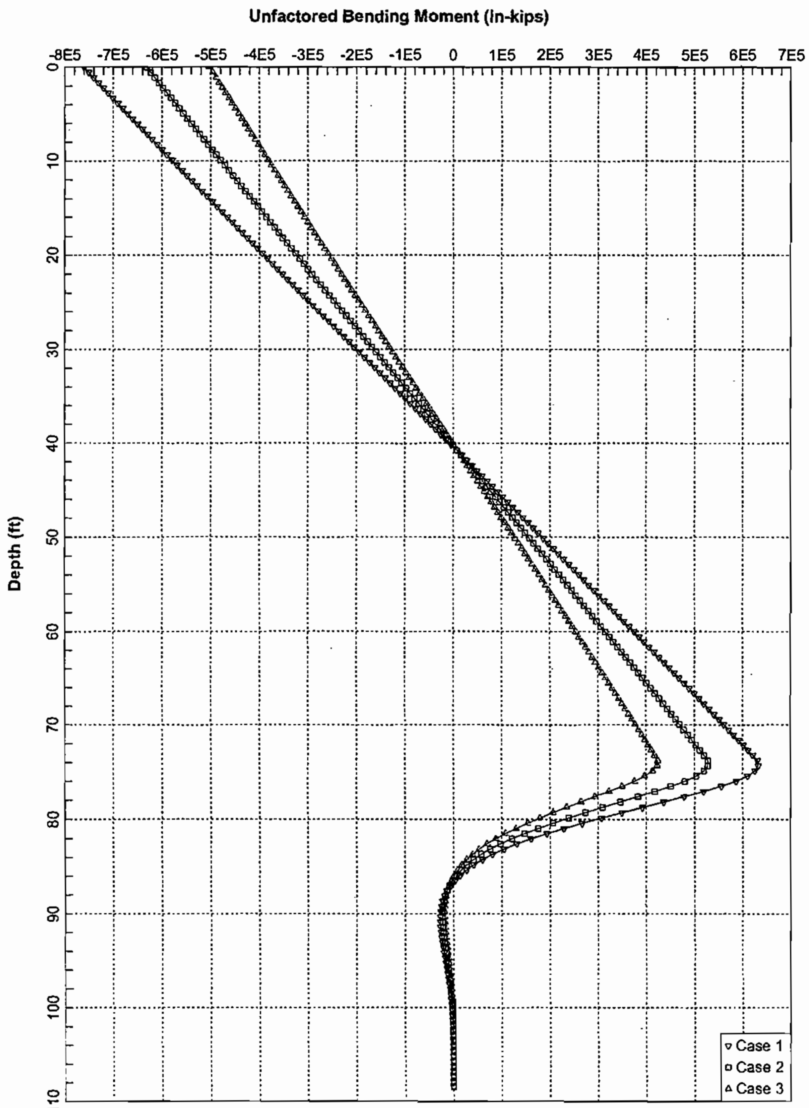
The analysis ended normally.

RUN 10: PIERS 3 & 4
8.5' SHAFT
MAX SCOUR



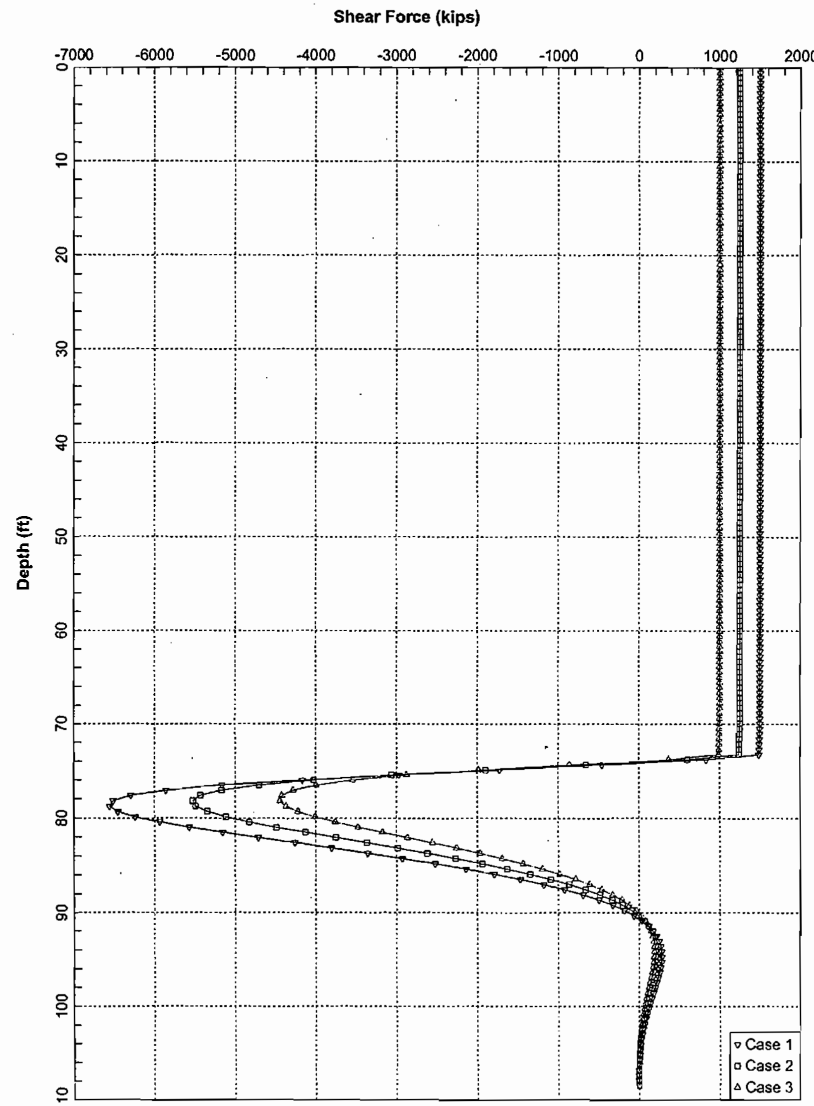


139



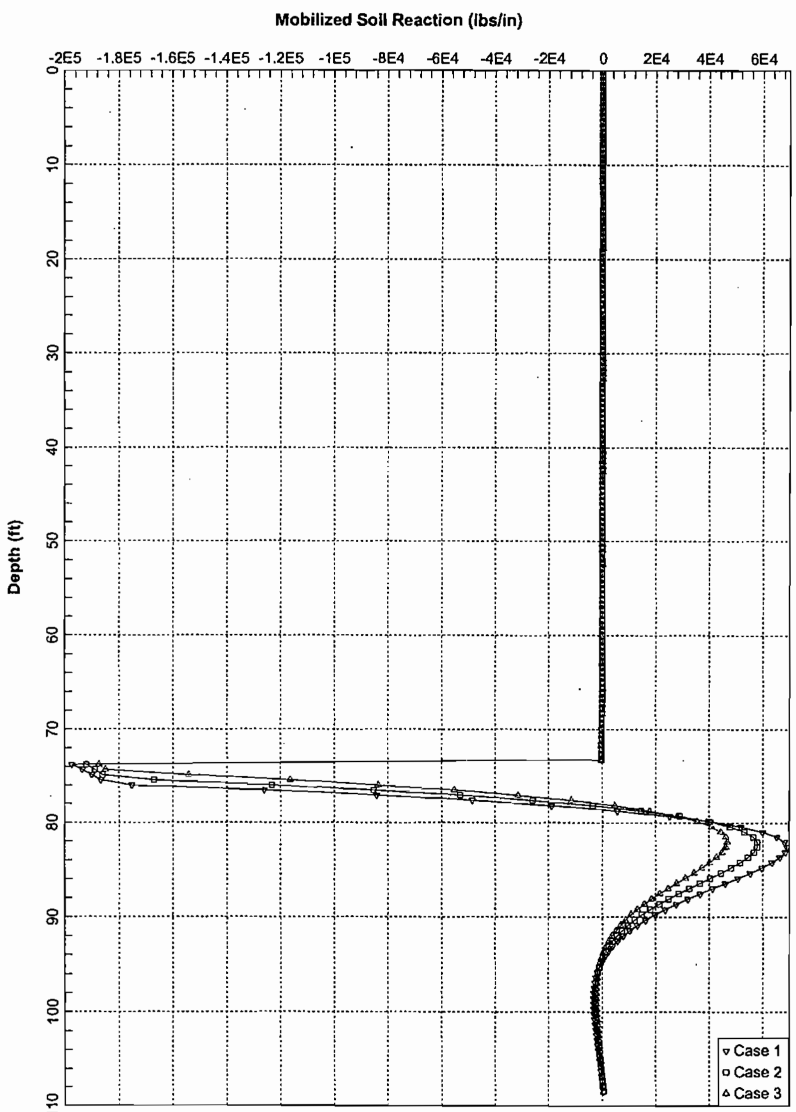
Unfactored Bending Moment (in-kips)
-8E5
-7E5
-6E5
-5E5
-4E5
-3E5
-2E5
-1E5
0
1E5
2E5
3E5
4E5
5E5
6E5
7E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

140



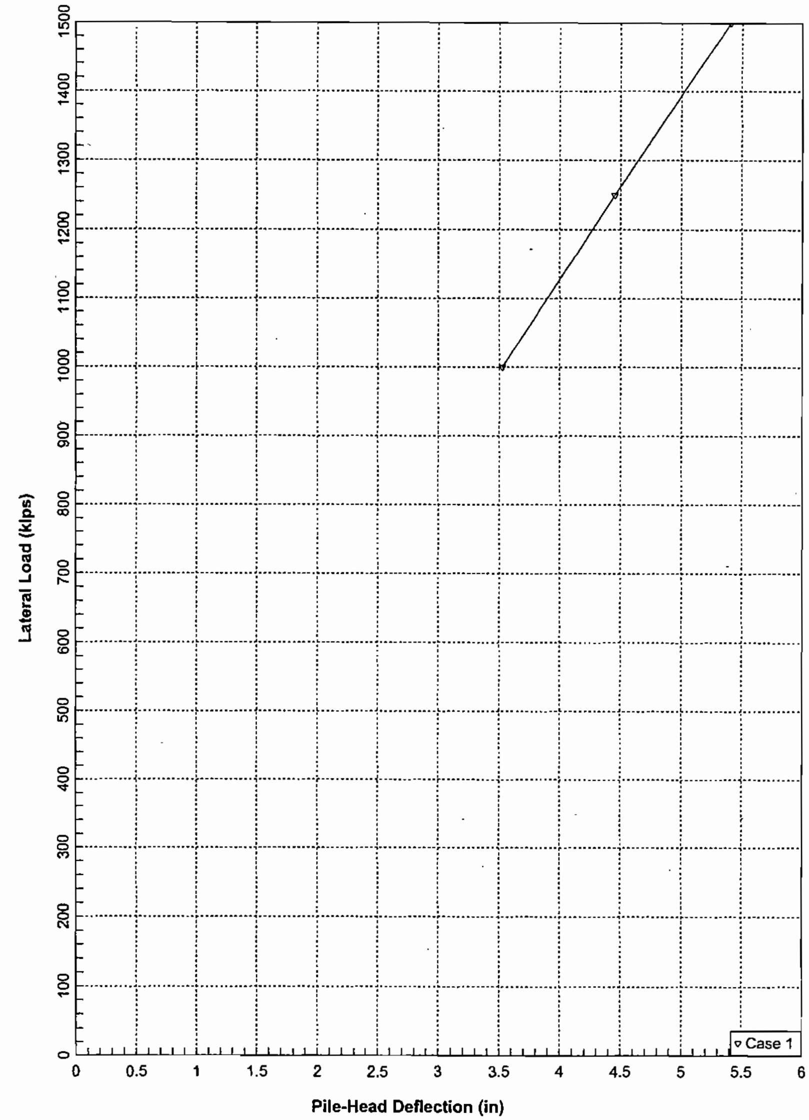
Shear Force (kips)
-7000
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

141



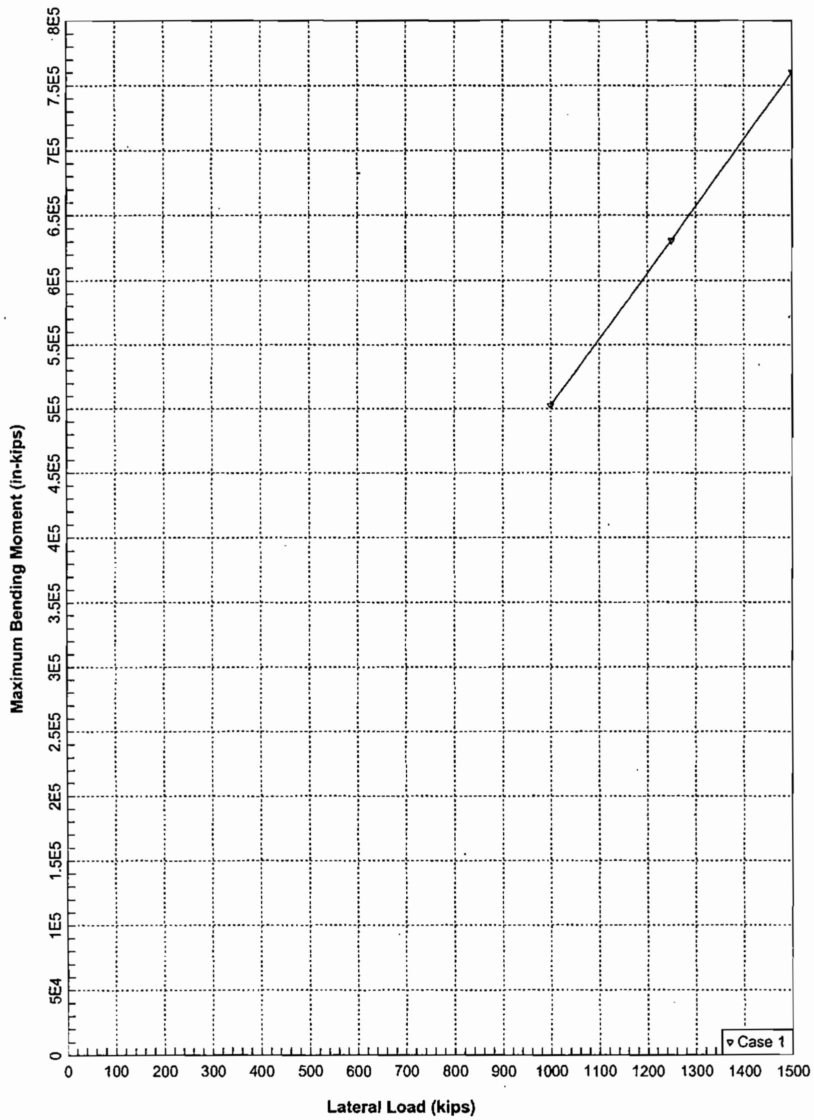
Mobilized Soil Reaction (lbs/in)
-2E5
-1.8E5
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
6E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

142

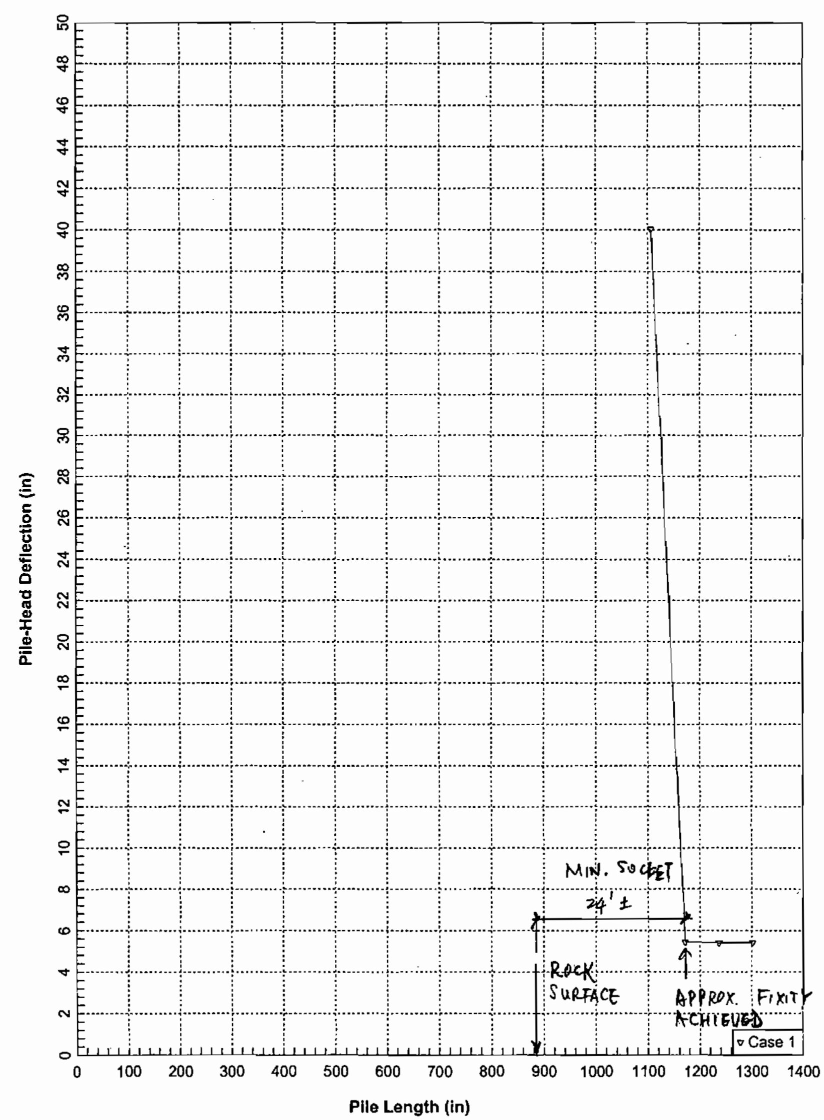


Lateral Load (kips)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
Pile-Head Deflection (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
Case 1

143


Maximum Bending Moment (in-kips)
0
5E4
1E5
1.5E5
2E5
2.5E5
3E5
3.5E5
4E5
4.5E5
5E5
5.5E5
6E5
6.5E5
7E5
7.5E5
8E5
Case 1
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
Lateral Load (kips)

144


Pile-Head Deflection (in)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
MIN. SOCKET
24' ±
ROCK
SURFACE
APPROX. FIXITY
ACHIEVED
Case 1
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Pile Length (in)

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Piers 3&4\
Name of input data file: Piers 3&4 - large - scour.lpd
Name of output file: Piers 3&4 - large - scour.lpo
Name of plot output file: Piers 3&4 - large - scour.lpp
Name of runtime file: Piers 3&4 - large - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11: 7: 9

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 197
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1302.00 in
Depth of ground surface below top of pile = 793.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	5431065.	9630.7800	4074281.
2	884.0000	102.00000	5431065.	9630.7800	4074281.
3	884.0000	96.00000000	4169220.	7238.2300	4074281.
4	1302.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 793.000 in
Distance from top of pile to bottom of layer = 882.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 2 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 882.000 in
Distance from top of pile to bottom of layer = 1523.000 in

(Depth of lowest layer extends 221.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	793.00	.03993

2	882.00	.03993
3	882.00	.05880
4	1523.00	.05880

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	793.000	.00000	38.00	-----	-----
2	882.000	.00000	38.00	-----	-----
3	882.000	4800.00000	.00	-----	-----
4	1523.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	793.000	.3500	1.0000
2	882.000	.3500	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1500000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1250000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1000000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1500000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1250000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1000000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
2	V= 1.50E+06	S= 0.000	1.2000E+07	5.4085	-7.5996E+08	-6559964.
2	V= 1.25E+06	S= 0.000	1.2000E+07	4.4548	-6.3067E+08	-5523668.
2	V= 1.00E+06	S= 0.000	1.2000E+07	3.5260	-5.0261E+08	-4445360.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 2, Shear and Slope

Shear = 1500000. lbs
Slope = .00000
Axial Load = 12000000. lbs

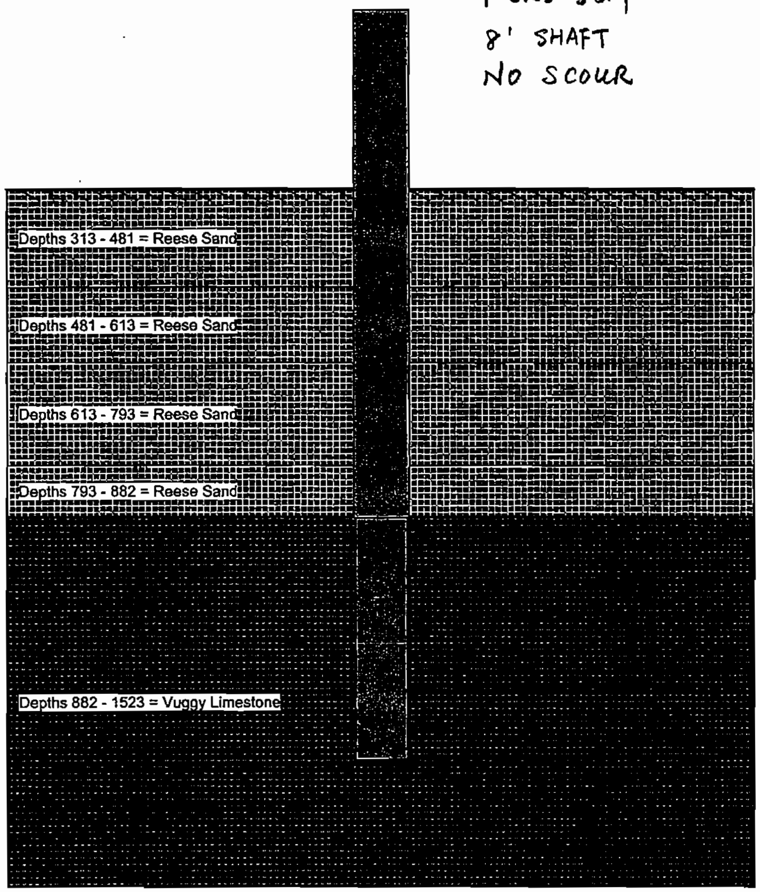
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1302.000	5.40852186	-7.599557E+08	-6559964.

1236.900	5.40454225	-7.597610E+08	-6563469.
1171.800	5.42720926	-7.608789E+08	-6572847.
1106.700	40.03260229	-2.112945E+09	1500000.

The analysis ended normally.

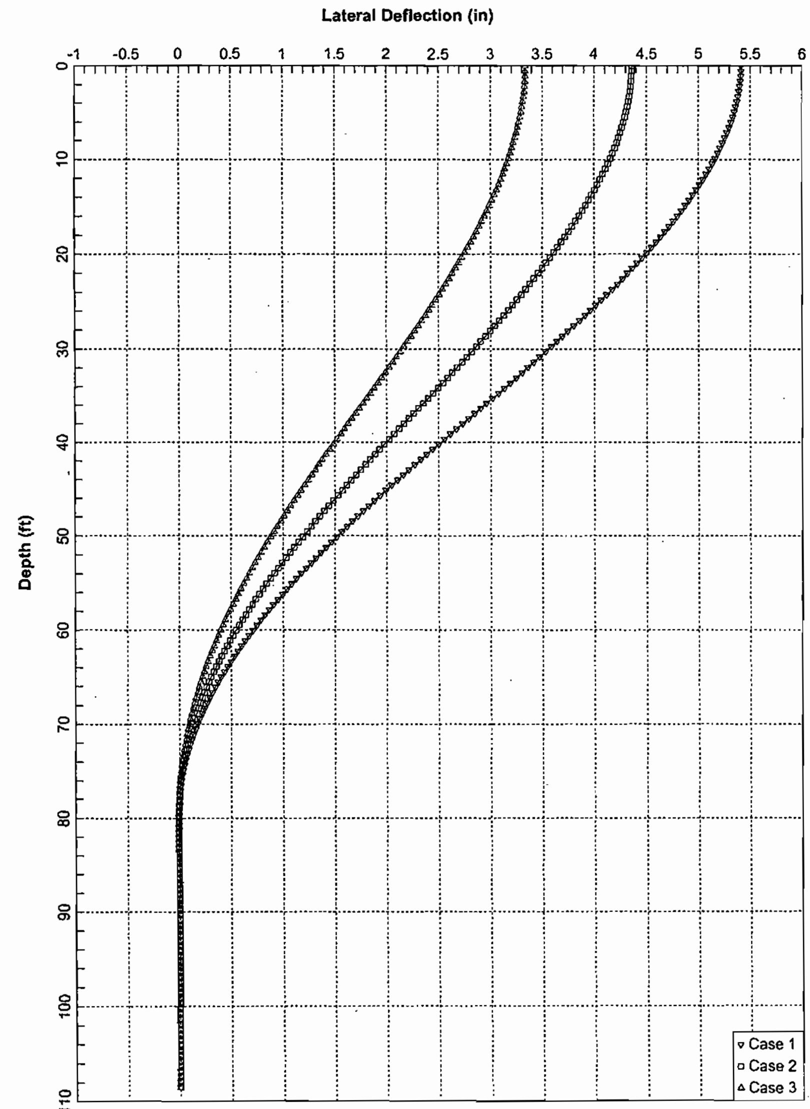
RUN 11:
PIERS 3&4
8' SHAFT
NO SCOUR

151

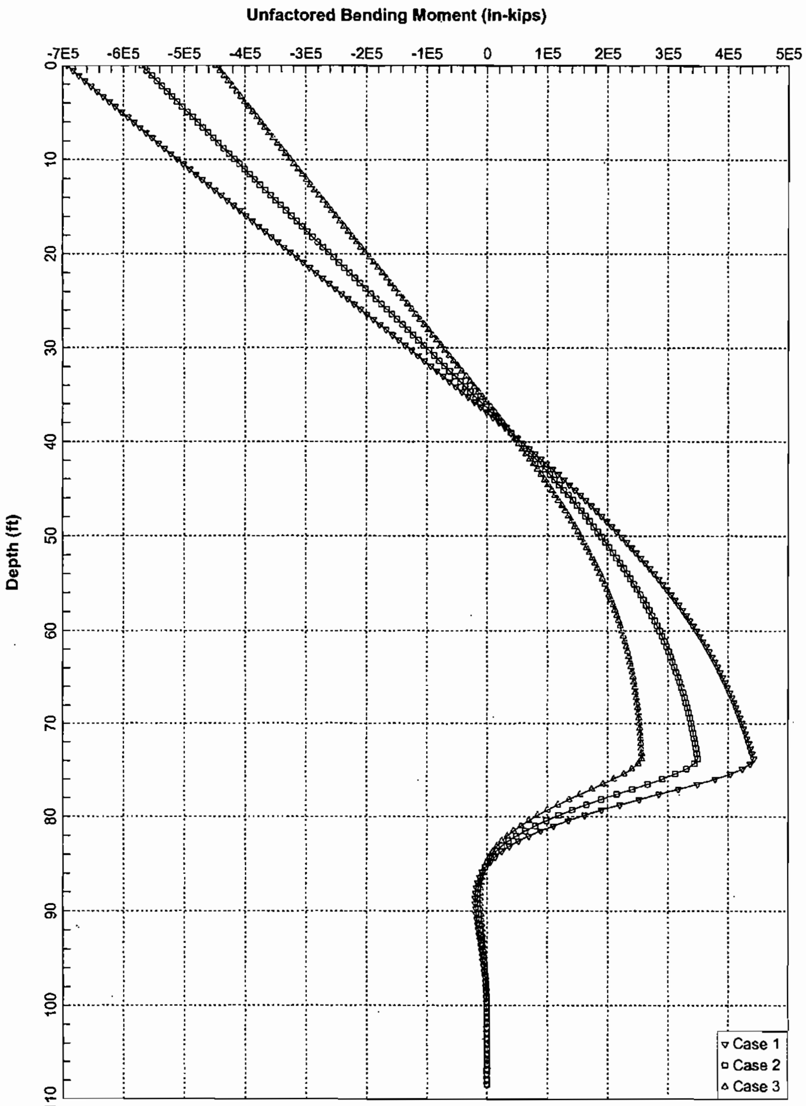


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

152

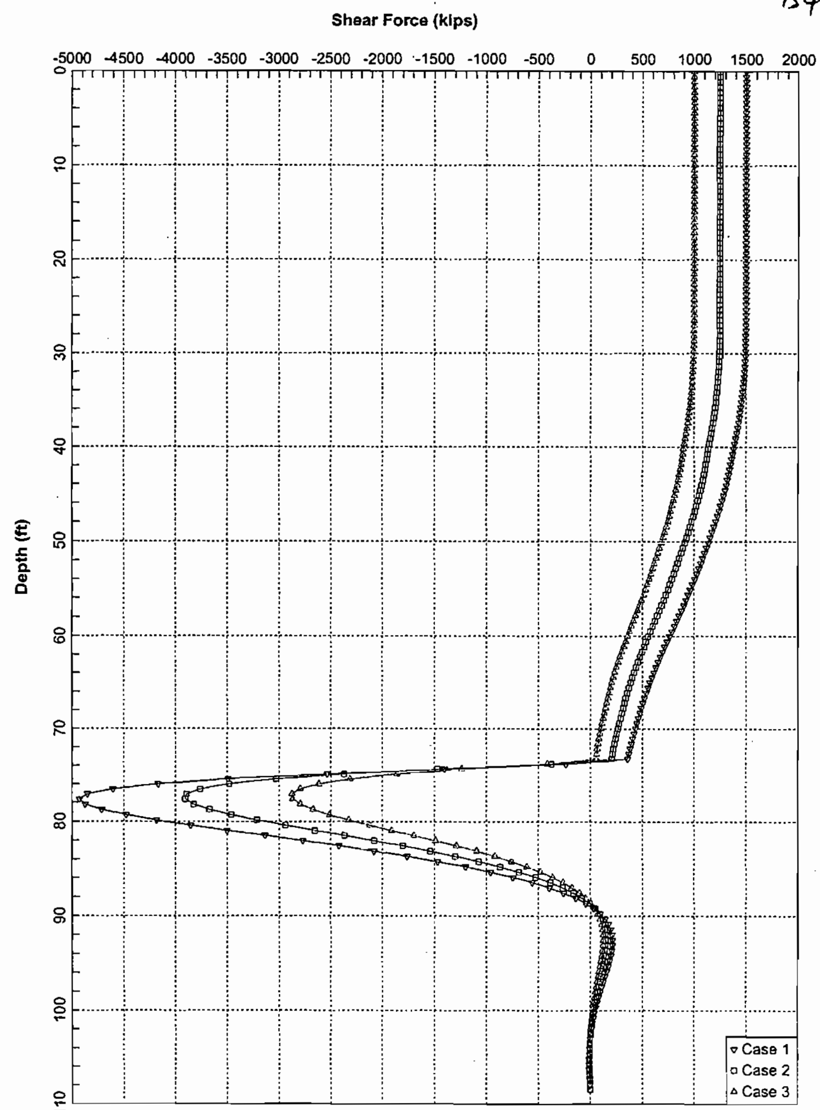


153



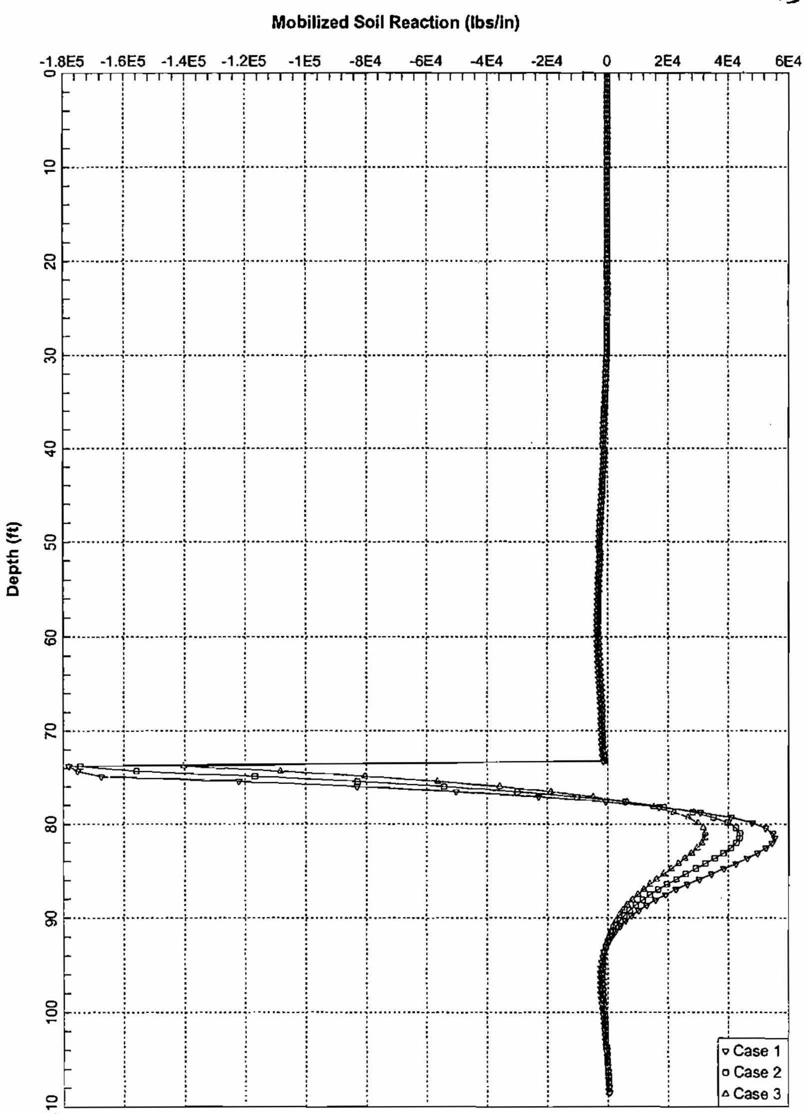
Unfactored Bending Moment (in-kips)
-7E5
-6E5
-5E5
-4E5
-3E5
-2E5
-1E5
0
1E5
2E5
3E5
4E5
5E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

154



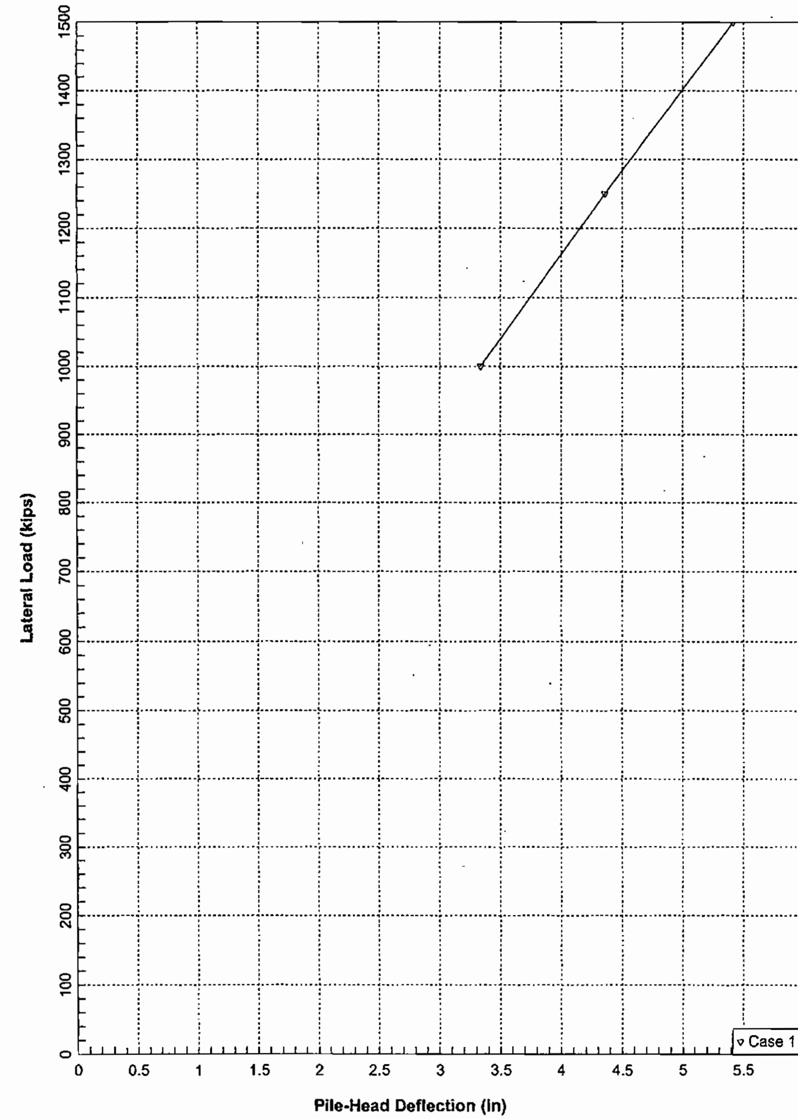
Shear Force (kips)
-5000
-4500
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

155



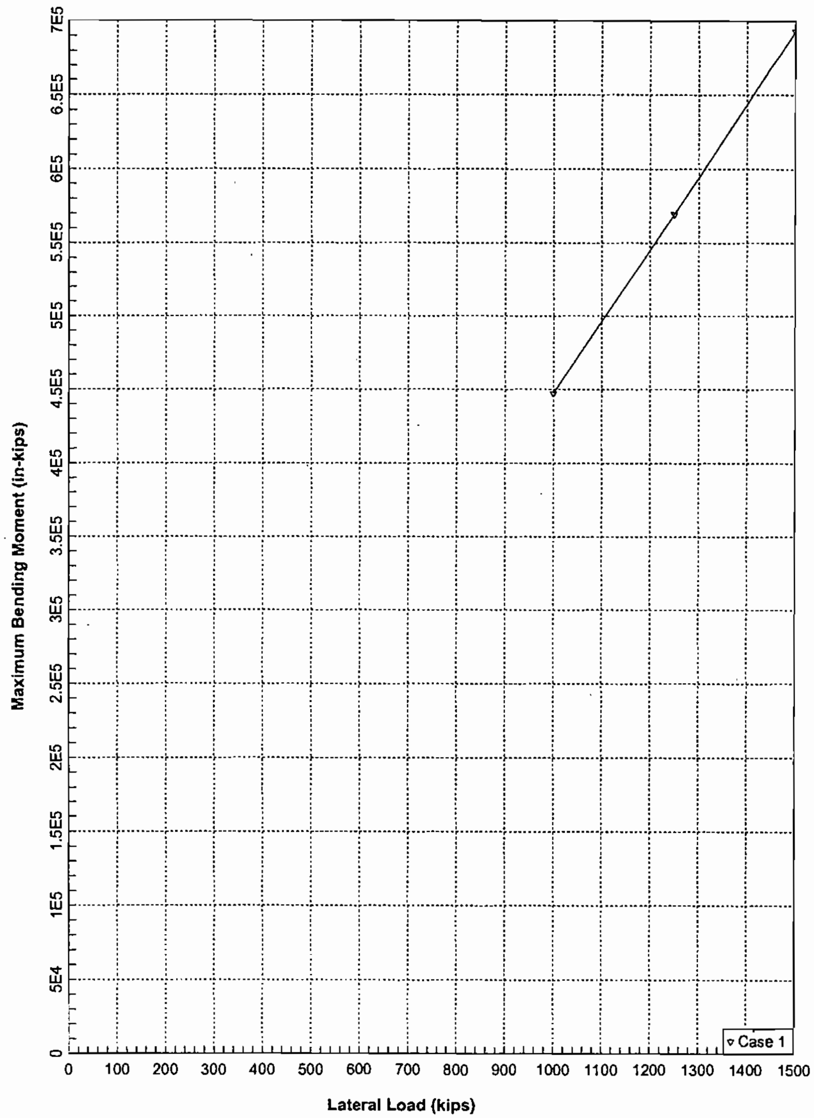
Mobilized Soil Reaction (lbs/in)
-1.8E5
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
6E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

156



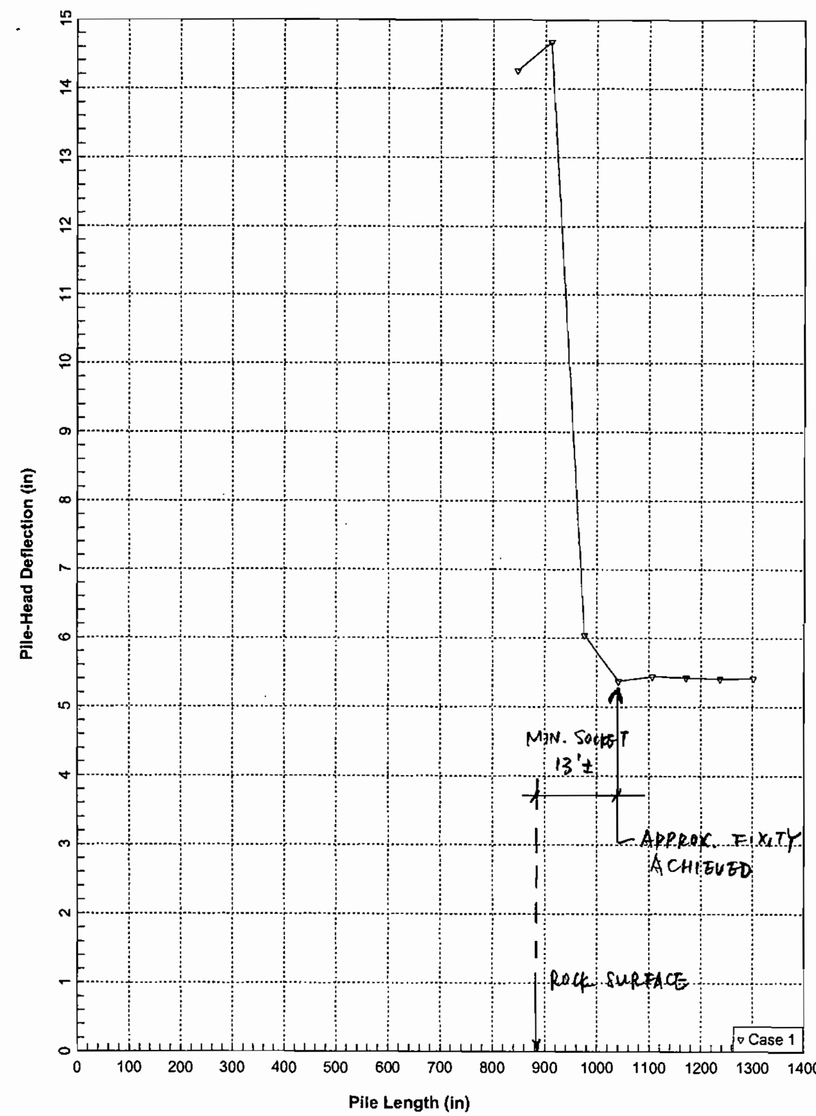
Lateral Load (kips)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
Pile-Head Deflection (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
Case 1

157



Maximum Bending Moment (in-kips)
0
5E4
1E5
1.5E5
2E5
2.5E5
3E5
3.5E5
4E5
4.5E5
5E5
5.5E5
6E5
6.5E5
7E5
Lateral Load (kips)
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500
Case 1

158



Pile-Head Deflection (in)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Pile Length (in)
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400
Case 1
MIN. SOCKET
13'±
APPROX. FIXITY
ACHIEVED
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Piers 3&4\
Name of input data file: Piers 3&4 - small - no scour.lpd
Name of output file: Piers 3&4 - small - no scour.lpo
Name of plot output file: Piers 3&4 - small - no scour.lpp
Name of runtime file: Piers 3&4 - small - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:10:14

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 197
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1302.00 in
Depth of ground surface below top of pile = 313.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	884.0000	96.00000000	4356263.	8611.2400	4074281.
3	884.0000	90.00000000	3220623.	6361.7300	4074281.
4	1302.0000	90.00000000	3220623.	6361.7300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 313.000 in
Distance from top of pile to bottom of layer = 481.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 481.000 in
Distance from top of pile to bottom of layer = 613.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 613.000 in
Distance from top of pile to bottom of layer = 793.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 793.000 in
Distance from top of pile to bottom of layer = 882.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 882.000 in
Distance from top of pile to bottom of layer = 1523.000 in

(Depth of lowest layer extends 221.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	313.00	.03819
2	481.00	.03819
3	481.00	.03877
4	613.00	.03877
5	613.00	.04109
6	793.00	.04109
7	793.00	.03993
8	882.00	.03993
9	882.00	.05880
10	1523.00	.05880

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	313.000	.00000	34.50	-----	-----
2	481.000	.00000	34.50	-----	-----
3	481.000	.00000	36.90	-----	-----
4	613.000	.00000	36.90	-----	-----
5	613.000	.00000	38.00	-----	-----
6	793.000	.00000	38.00	-----	-----
7	793.000	.00000	38.00	-----	-----
8	882.000	.00000	38.00	-----	-----
9	882.000	4800.00000	.00	-----	-----
10	1523.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	313.000	.3500	1.0000
2	882.000	.3500	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1500000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1250000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1000000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1500000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1250000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1000000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in

Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
2	V= 1.50E+06	S= 0.000	1.2000E+07	5.4153	-6.9291E+08	-4926894.
2	V= 1.25E+06	S= 0.000	1.2000E+07	4.3586	-5.6929E+08	-3904813.
2	V= 1.00E+06	S= 0.000	1.2000E+07	3.3370	-4.4717E+08	-2872895.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 2, Shear and Slope

Shear = 1500000. lbs
Slope = .00000
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1302.000	5.41530091	-6.929085E+08	-4926894.
1236.900	5.41116218	-6.927180E+08	-4917409.
1171.800	5.42442651	-6.932324E+08	-4922005.
1106.700	5.44201665	-6.938839E+08	-4897360.
1041.600	5.37695732	-6.913170E+08	-4966388.
976.500	6.04098692	-7.190738E+08	-7183168.
911.400	14.67646239	-1.127713E+09	-1594591.
846.300	14.25339352	-1.163675E+09	1500000.

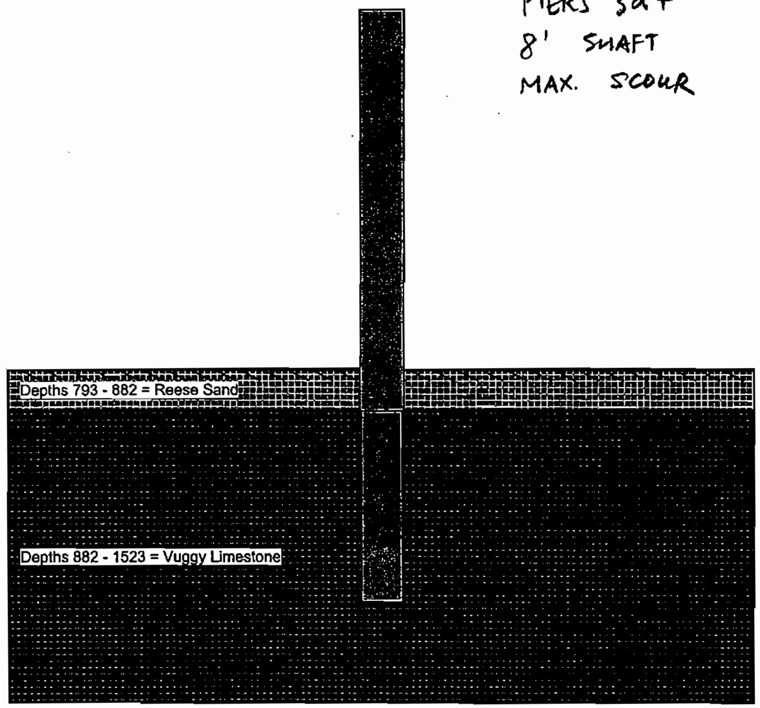
The analysis ended normally.

RUN 12: 165

PIERS 3&4

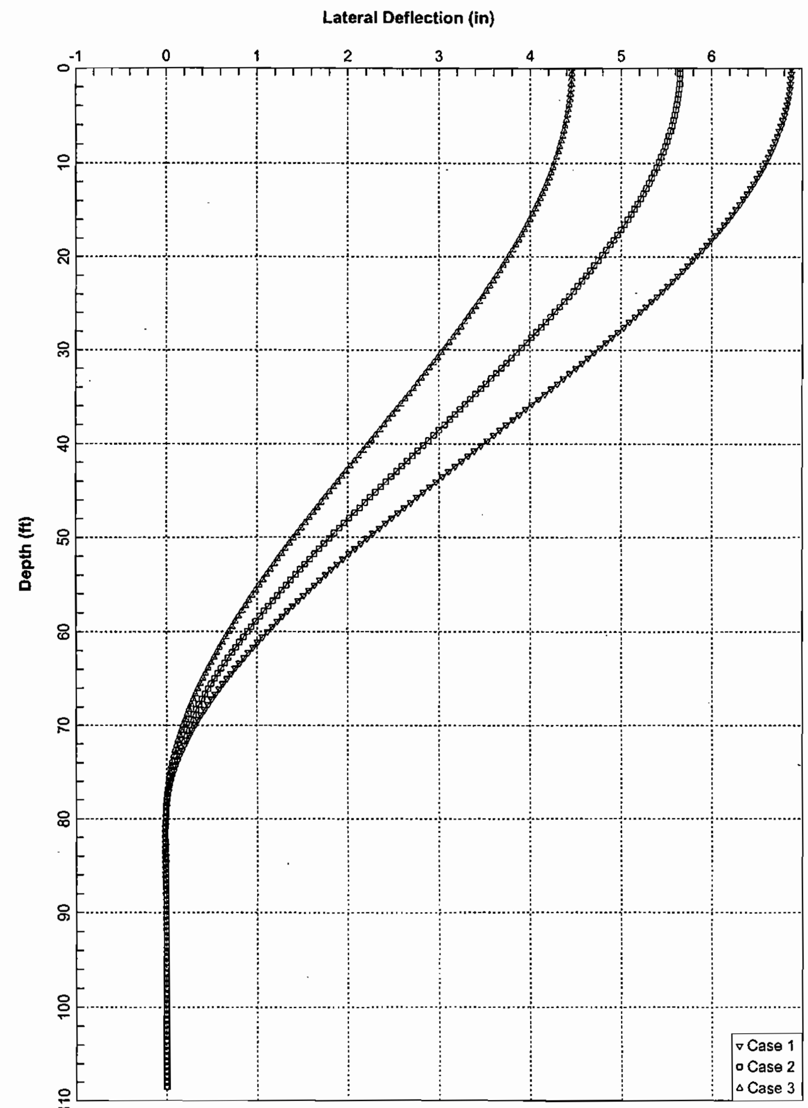
8' SHAFT

MAX. SCOUR

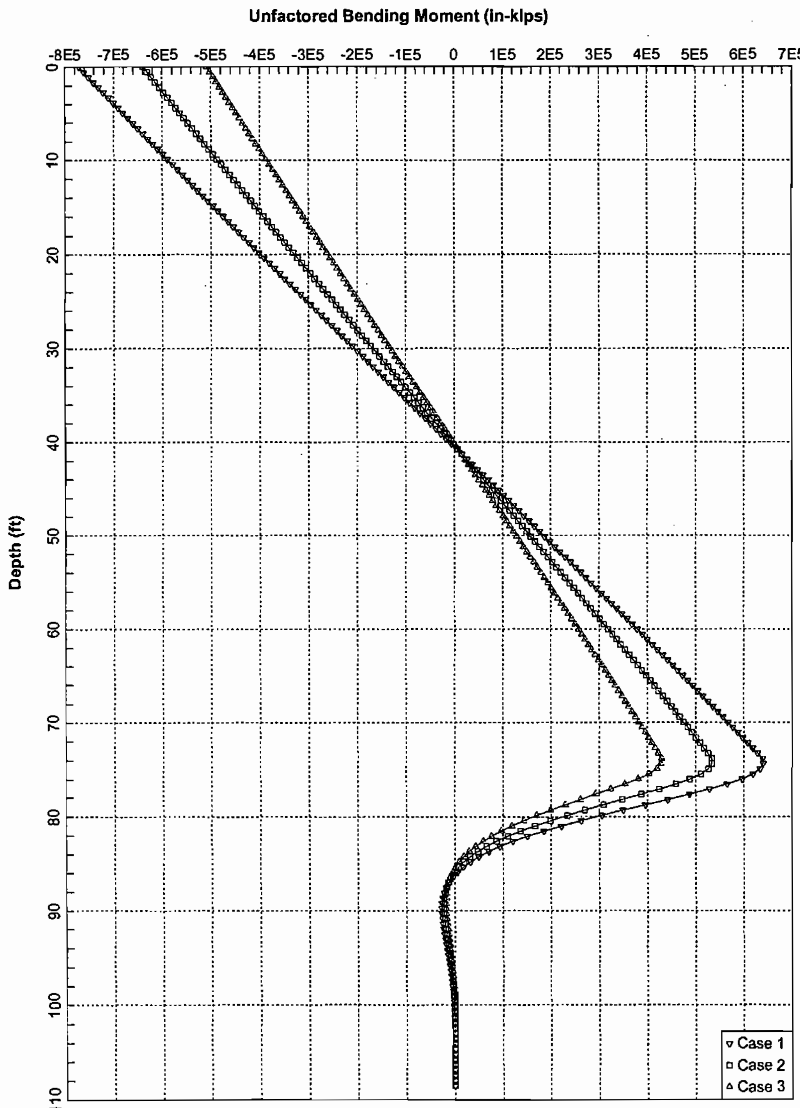


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

166

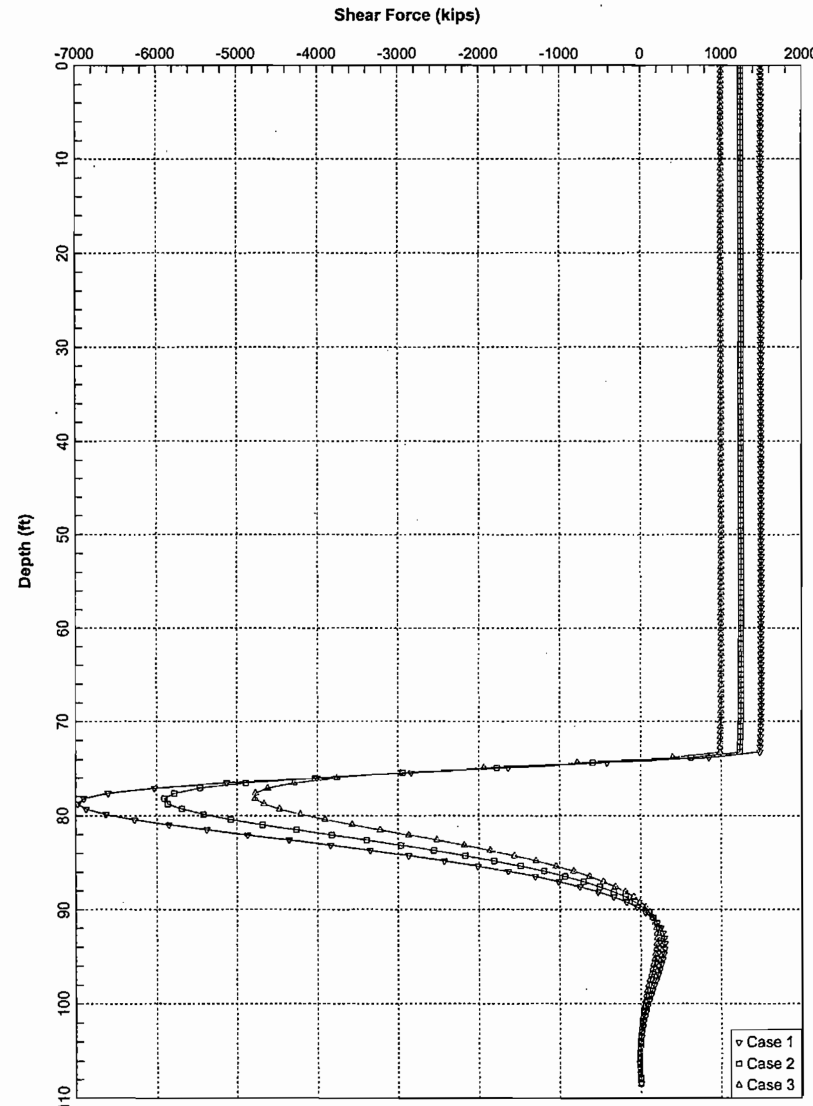


167



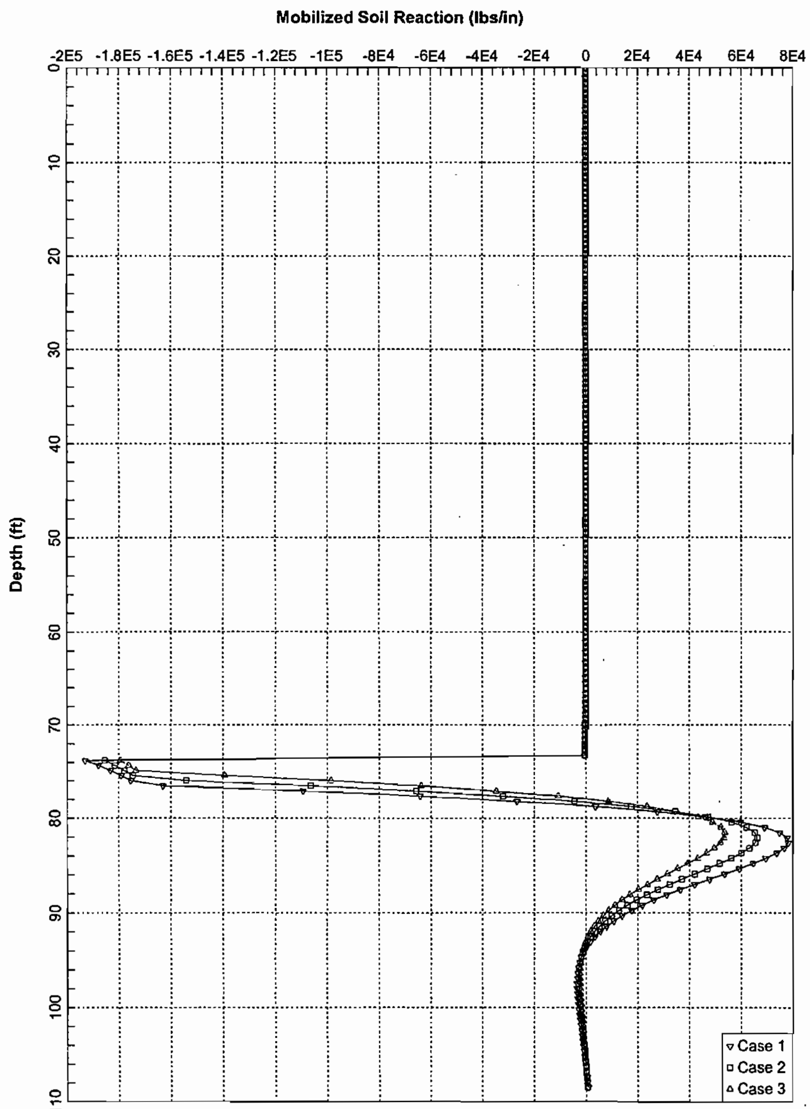
Unfactored Bending Moment (in-kips)
-8E5
-7E5
-6E5
-5E5
-4E5
-3E5
-2E5
-1E5
0
1E5
2E5
3E5
4E5
5E5
6E5
7E5
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

168



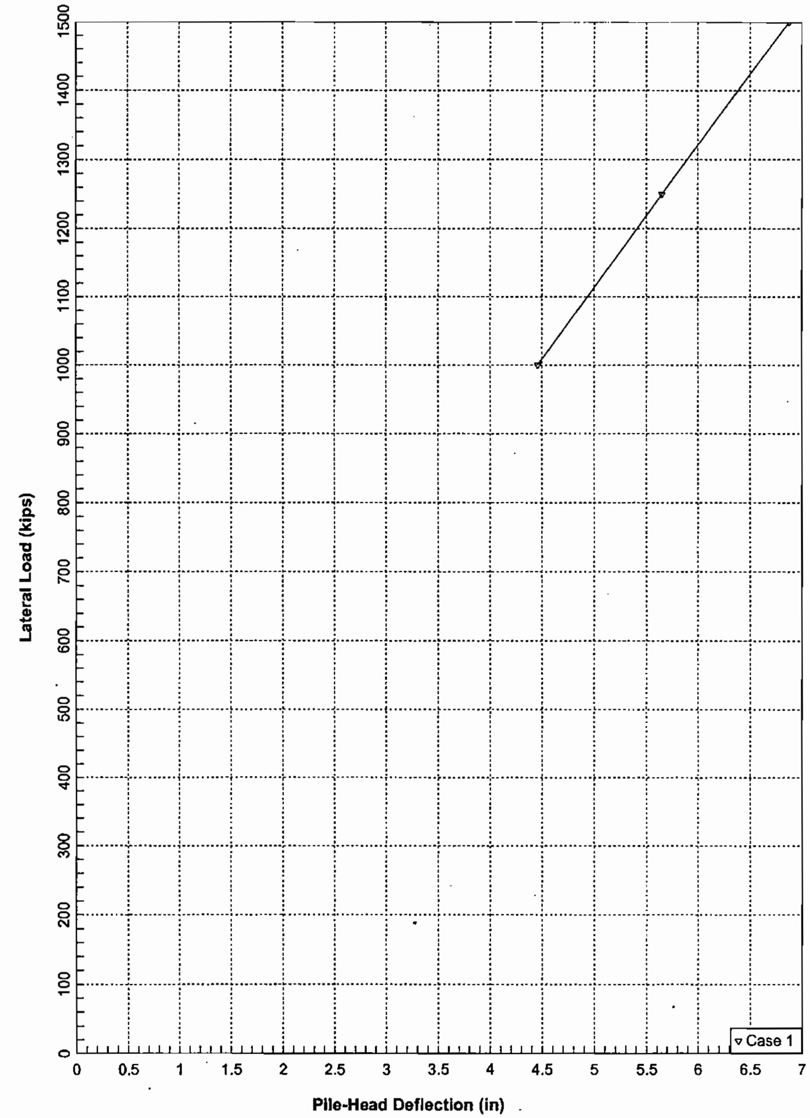
Shear Force (kips)
-7000
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

169



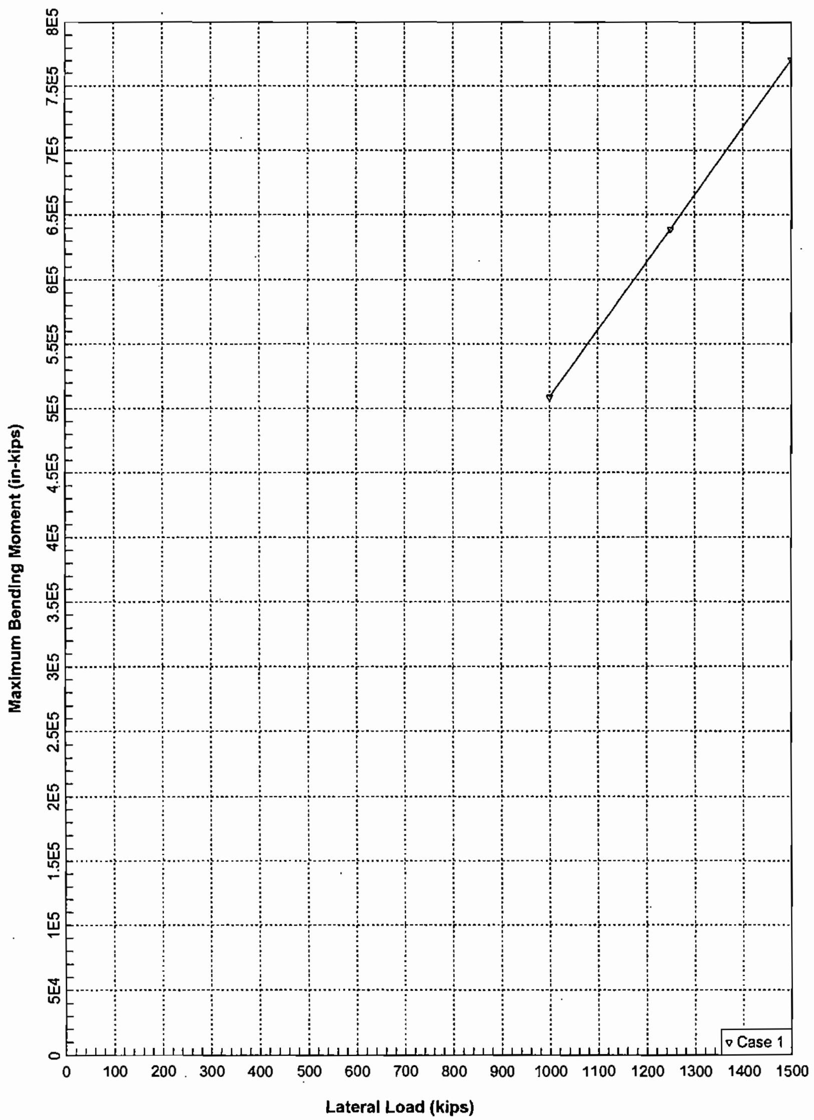
Mobilized Soil Reaction (lbs/in)
-2E5
-1.8E5
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
6E4
8E4
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
Case 1
Case 2
Case 3

170



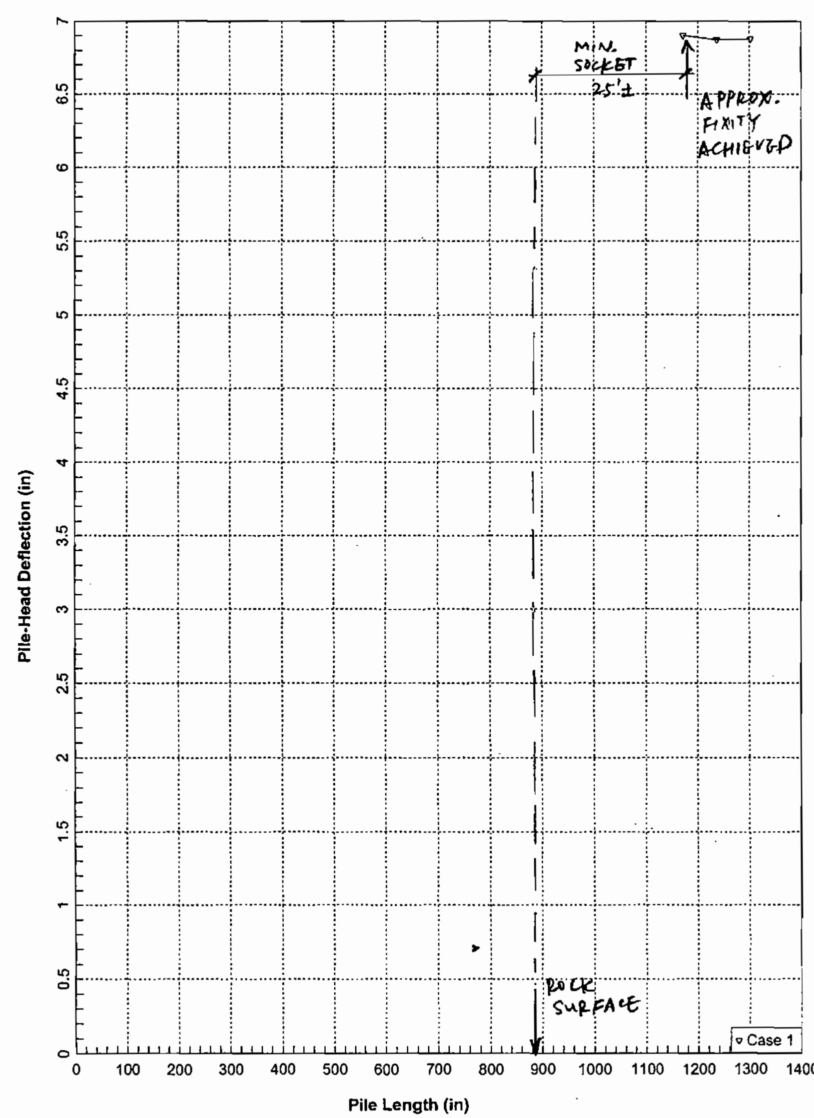
Lateral Load (kips)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
Pile-Head Deflection (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
Case 1

171



Maximum Bending Moment (in-kips)
0
5E4
1E5
1.5E5
2E5
2.5E5
3E5
3.5E5
4E5
4.5E5
5E5
5.5E5
6E5
6.5E5
7E5
7.5E5
8E5
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500
Lateral Load (kips)
Case 1

172



Pile-Head Deflection (in)
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400
Pile Length (in)
Case 1
MIN. SOCKET 25'±
APPROX. FIXITY ACHIEVED
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Piers 3&4\
Name of input data file: Piers 3&4 - small - scour.lpd
Name of output file: Piers 3&4 - small - scour.lpo
Name of plot output file: Piers 3&4 - small - scour.lpp
Name of runtime file: Piers 3&4 - small - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:38:35

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 197
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 1302.00 in
Depth of ground surface below top of pile = 793.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	884.0000	96.00000000	4356263.	8611.2400	4074281.
3	884.0000	90.00000000	3220623.	6361.7300	4074281.
4	1302.0000	90.00000000	3220623.	6361.7300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 793.000 in
Distance from top of pile to bottom of layer = 882.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 2 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 882.000 in
Distance from top of pile to bottom of layer = 1523.000 in

(Depth of lowest layer extends 221.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	793.00	.03993

2	882.00	.03993
3	882.00	.05880
4	1523.00	.05880

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	793.000	.00000	38.00	-----	-----
2	882.000	.00000	38.00	-----	-----
3	882.000	4800.00000	.00	-----	-----
4	1523.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	793.000	.3500	1.0000
2	882.000	.3500	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1500000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1250000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 1000000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1500000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1250000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 1000000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 12000000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2 lbs	Axial Load in	Pile-Head Deflection in-lbs	Maximum Moment lbs	Maximum Shear
2	V= 1.50E+06	S= 0.000	1.2000E+07	6.8777	-7.7050E+08	-6972826.
2	V= 1.25E+06	S= 0.000	1.2000E+07	5.6531	-6.3889E+08	-5901324.
2	V= 1.00E+06	S= 0.000	1.2000E+07	4.4594	-5.0852E+08	-4773386.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 2, Shear and Slope

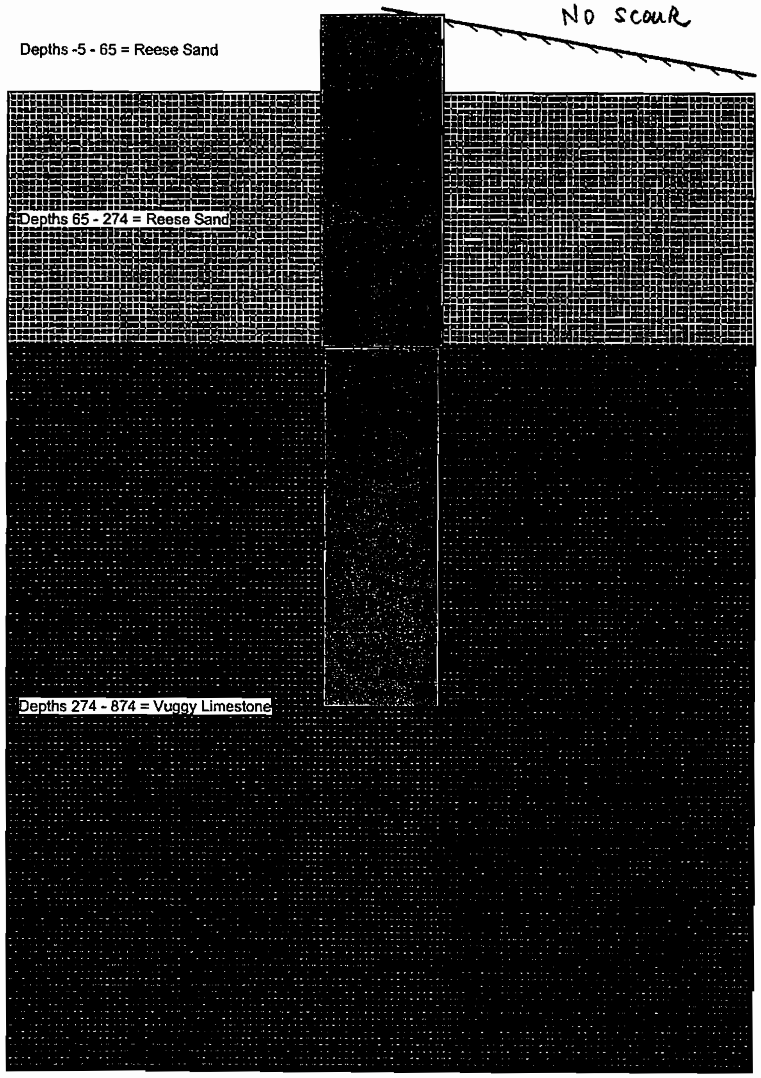
Shear = 1500000. lbs
Slope = .00000
Axial Load = 12000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1302.000	6.87767917	-7.704978E+08	-6972826.
1236.900	6.87306469	-7.703153E+08	-6964100.
1171.800	6.90136904	-7.714405E+08	-6983414.

The analysis ended normally.

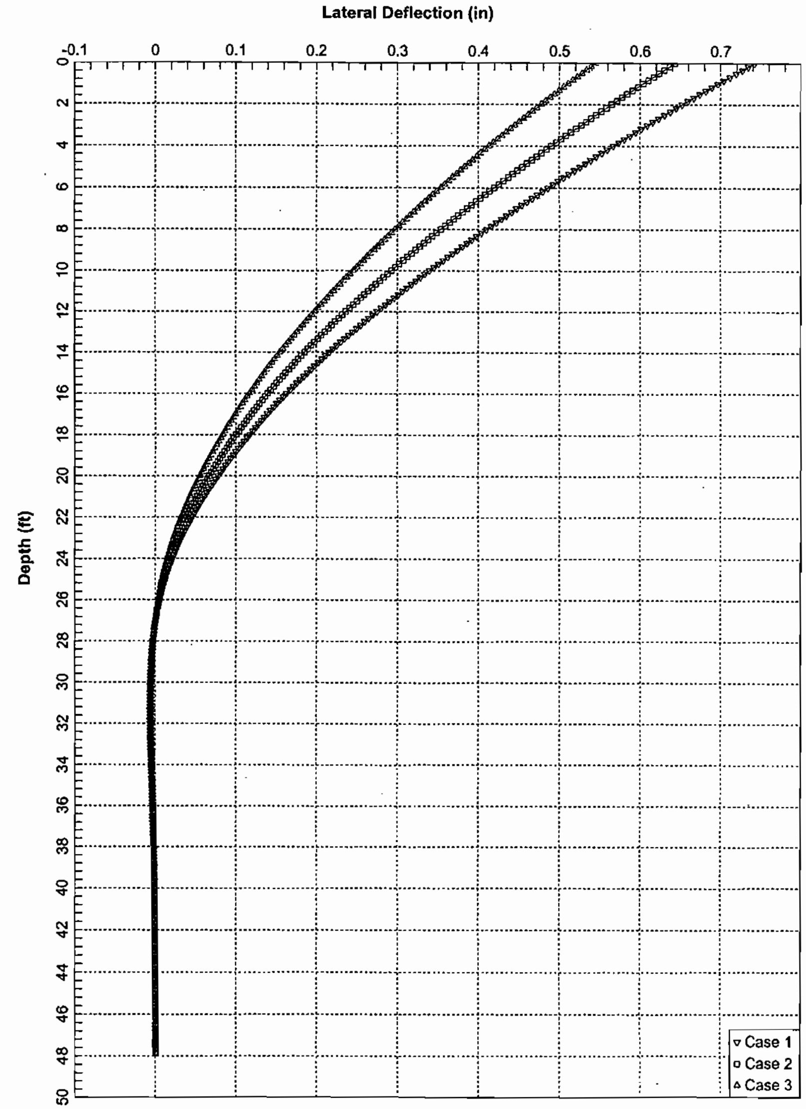
RUN 13: PIER 5
8.5' SHAFT
NO SCOUR

179

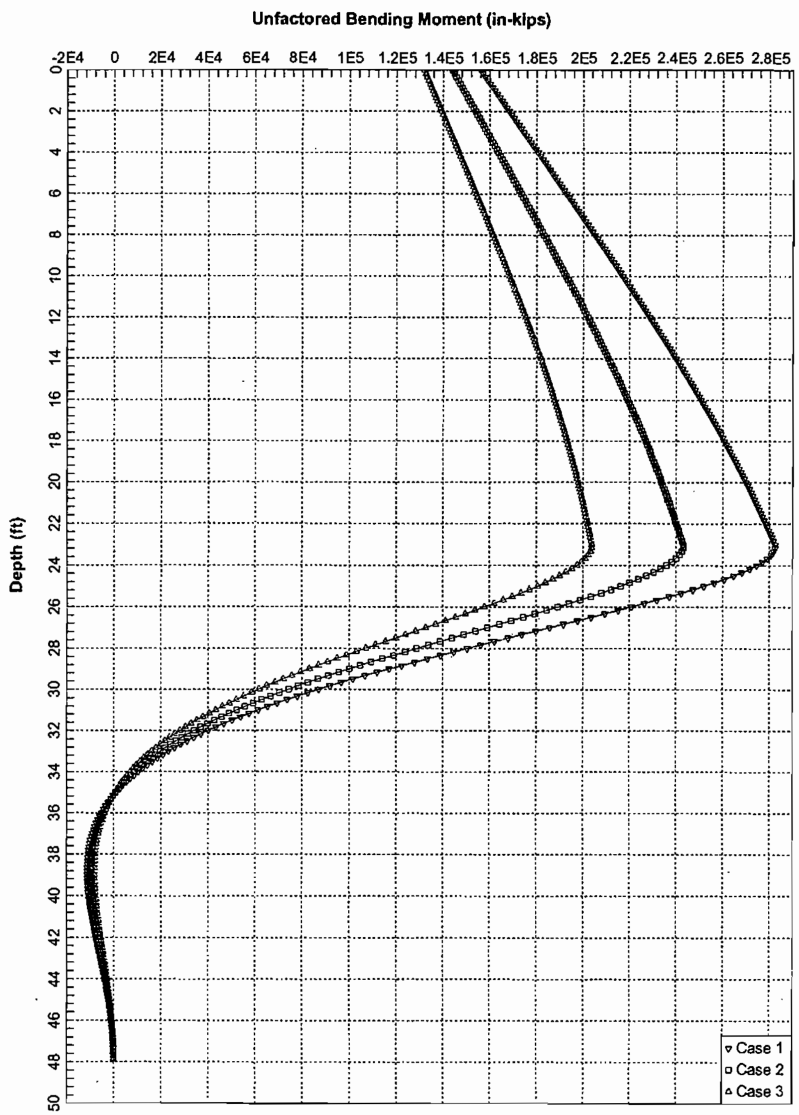


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

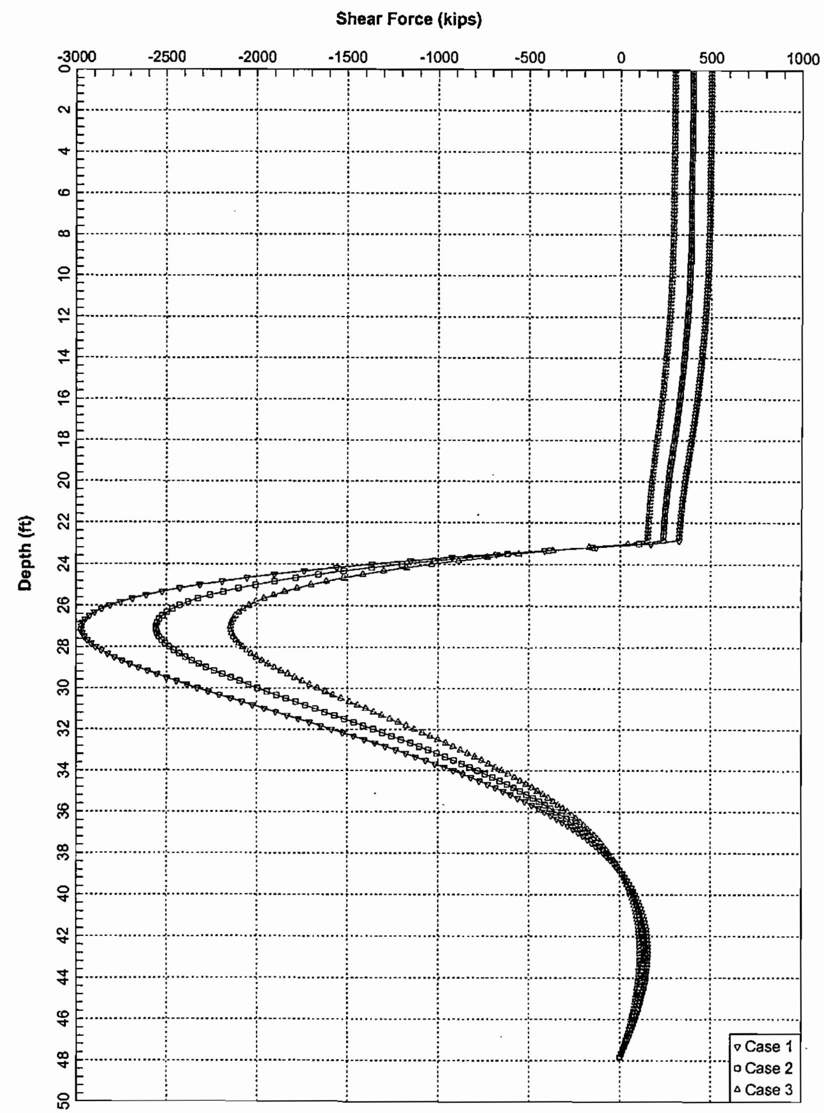
180



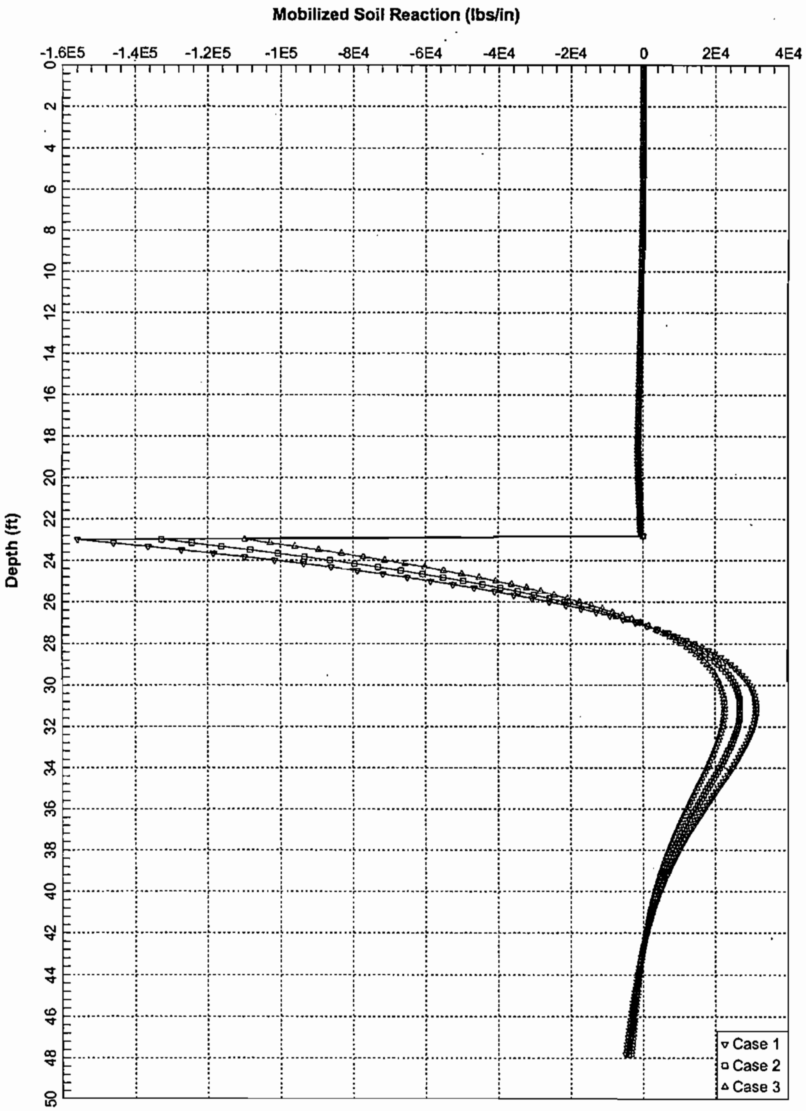
181


Unfactored Bending Moment (in-kips)
-2E4
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
2E5
2.2E5
2.4E5
2.6E5
2.8E5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

182

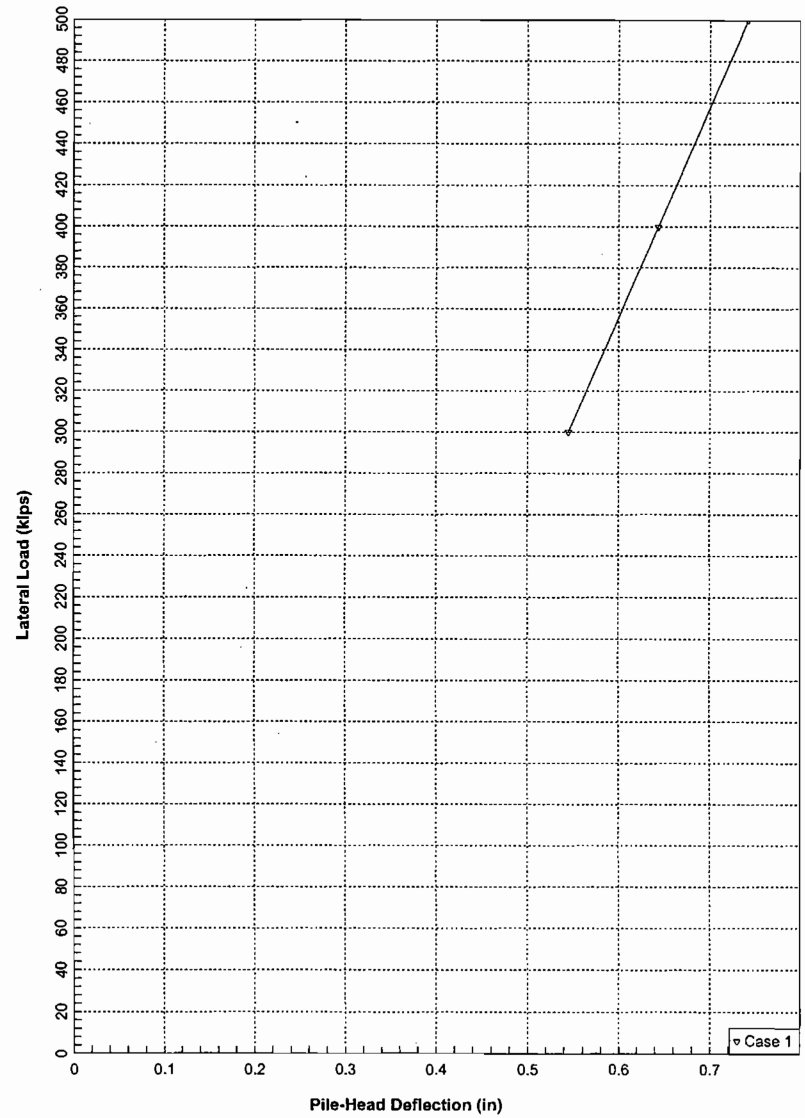

Shear Force (kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

183



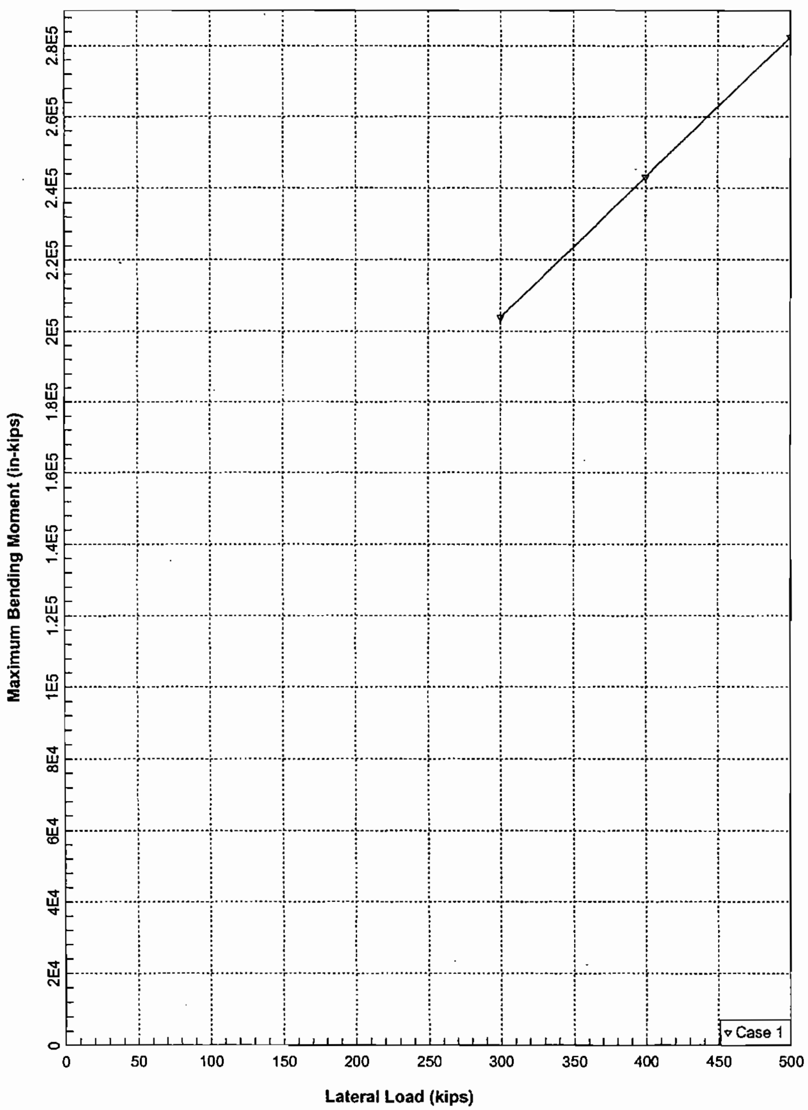
Mobilized Soil Reaction (lbs/in)
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

184

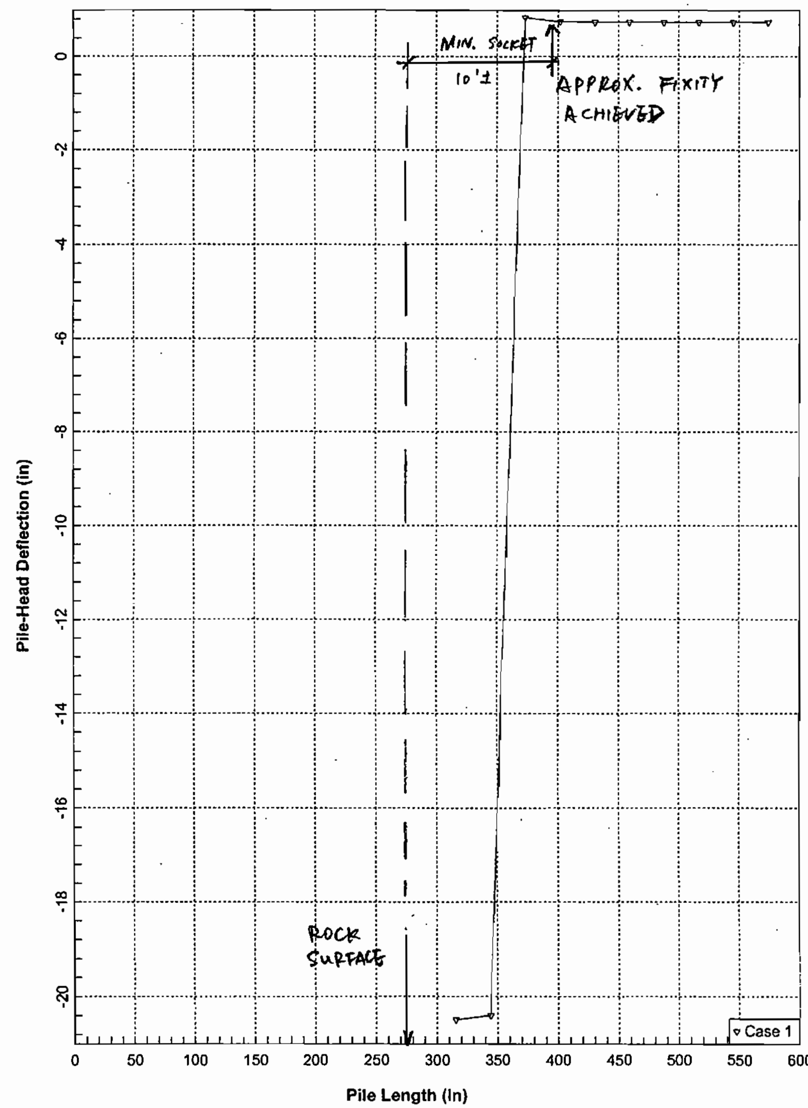


Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Case 1

185


Maximum Bending Moment (in-kips)
Lateral Load (kips)
Case 1
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
2E5
2.2E5
2.4E5
2.6E5
2.8E5
0
50
100
150
200
250
300
350
400
450
500

186


Pile-Head Deflection (in)
Pile Length (in)
Case 1
MIN. SOCKET
10'±
APPROX. FIXITY ACHIEVED
ROCK SURFACE
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
0
50
100
150
200
250
300
350
400
450
500
550
600

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 5\
Name of input data file: Pier 5 - large - no scour.lpd
Name of output file: Pier 5 - large - no scour.lpo
Name of plot output file: Pier 5 - large - no scour.lpp
Name of runtime file: Pier 5 - large - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:41:53

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 287
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 574.00 in
Depth of ground surface below top of pile = -5.00 in
Slope angle of ground surface = 10.00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	5431065.	9630.7800	4074281.
2	276.0000	102.00000	5431065.	9630.7800	4074281.
3	276.0000	96.00000000	4169220.	7238.2300	4074281.
4	574.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = -5.000 in
Distance from top of pile to bottom of layer = 65.000 in
p-y subgrade modulus k for top of soil layer = 20.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 20.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 65.000 in
Distance from top of pile to bottom of layer = 274.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 274.000 in
Distance from top of pile to bottom of layer = 874.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth

189

is defined using 6 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-5.00	.02257
2	65.00	.02257
3	65.00	.04109
4	274.00	.04109
5	274.00	.06111
6	874.00	.06111

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-5.000	.00000	29.50	-----	-----
2	65.000	.00000	29.50	-----	-----
3	65.000	.00000	38.00	-----	-----
4	274.000	.00000	38.00	-----	-----
5	274.000	4800.00000	.00	-----	-----
6	874.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	-5.000	.7000	1.0000
2	274.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

190

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 500000.000 lbs
Bending moment at pile head = 156000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 400000.000 lbs
Bending moment at pile head = 144000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 300000.000 lbs
Bending moment at pile head = 132000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 500000.000 lbs
Specified moment at pile head = 156000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 400000.000 lbs
Specified moment at pile head = 144000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 300000.000 lbs
Specified moment at pile head = 132000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 5.00E+05	M= 1.56E+08	4000000.	.7421650	2.8242E+08	-2970957. ✓
1	V= 4.00E+05	M= 1.44E+08	4000000.	.6435758	2.4309E+08	-2556757.
1	V= 3.00E+05	M= 1.32E+08	4000000.	.5452596	2.0392E+08	-2144841.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

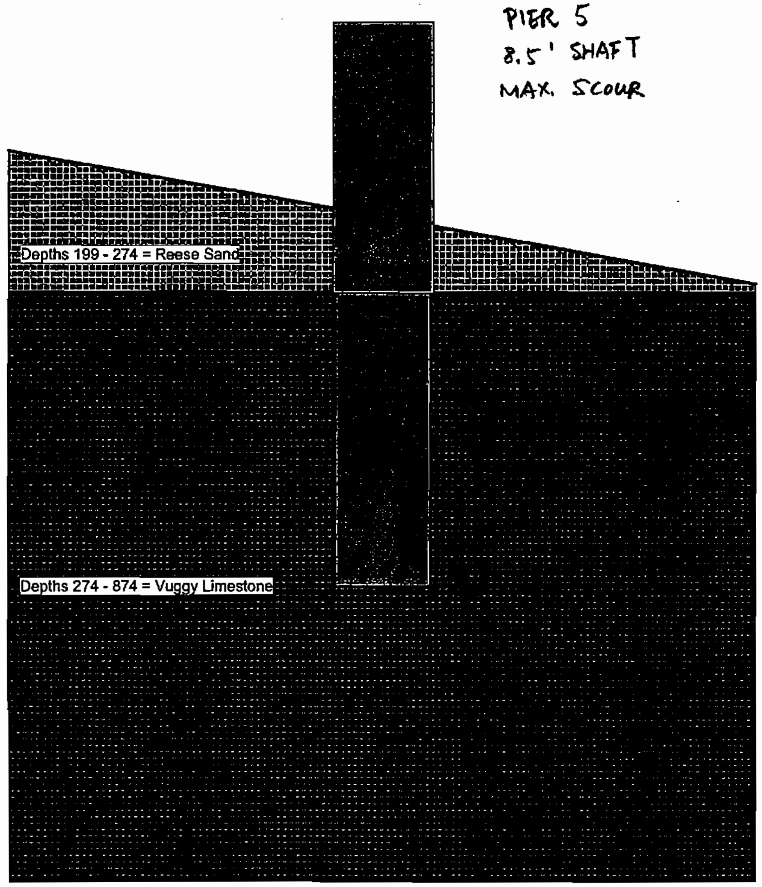
Shear = 500000. lbs
Moment = 156000000. in-lbs
Axial Load = 4000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
574.000	.74216496	2.824227E+08	-2970957.
545.300	.73806253	2.822625E+08	-2969977.
516.600	.73825730	2.822358E+08	-2965128.
487.900	.73893525	2.822644E+08	-2953055.
459.200	.73846295	2.822635E+08	-2950304.
430.500	.73570294	2.820191E+08	-3038641.
401.800	.75184784	2.818451E+08	-3411977.
373.100	.83438094	2.812793E+08	-4306664.
344.400	-20.39923880	1.560047E+08	559826.04440
315.700	-20.49212863	1.559568E+08	548191.45962

The analysis ended normally.

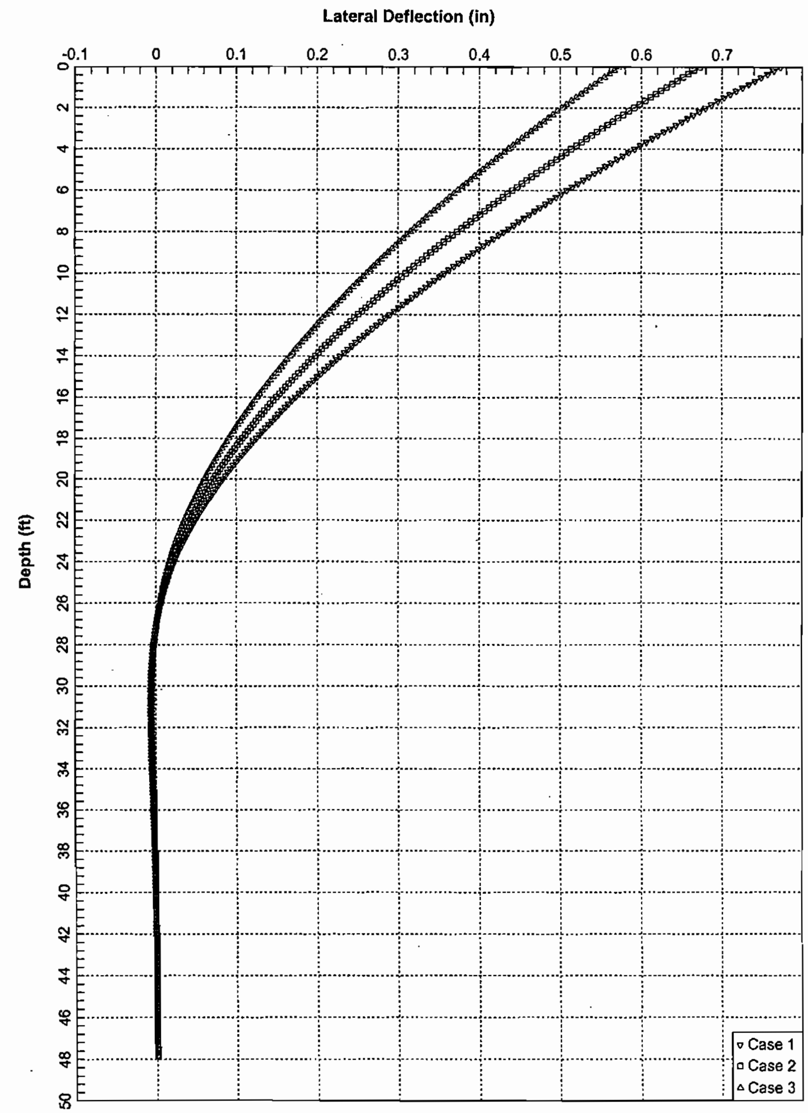
193

RUN 14:
PIER 5
8.5' SHAFT
MAX. SCOUR

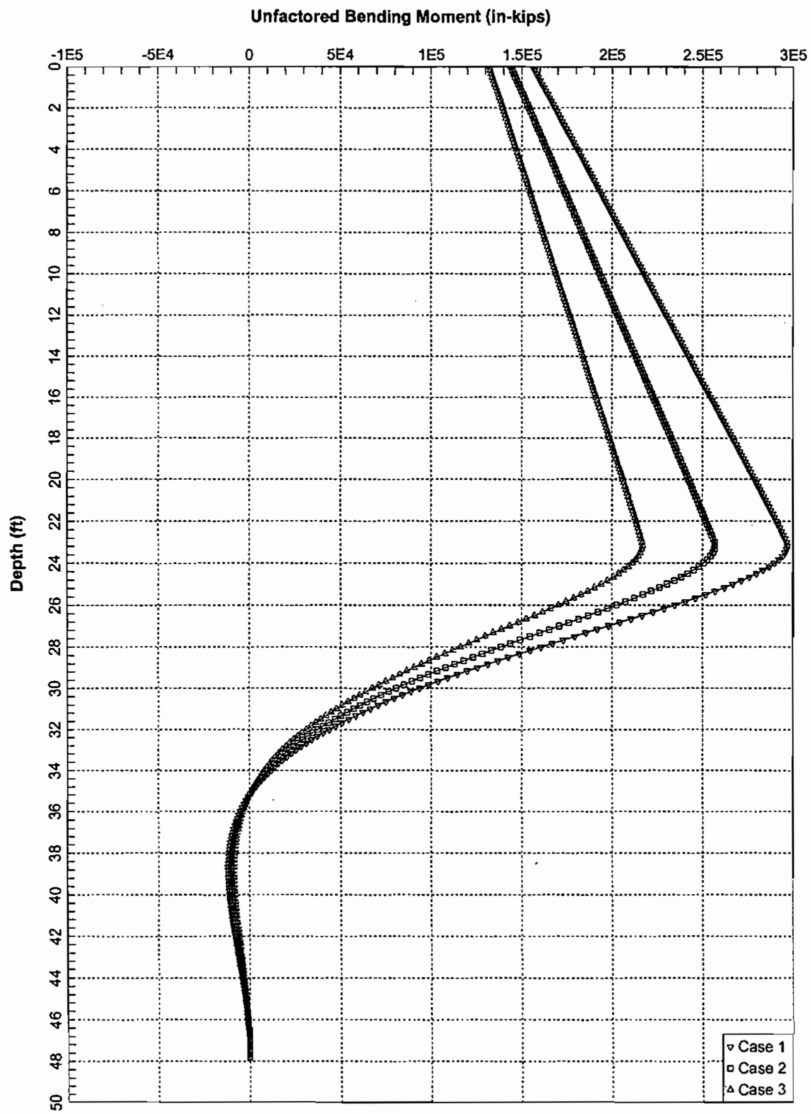


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

194

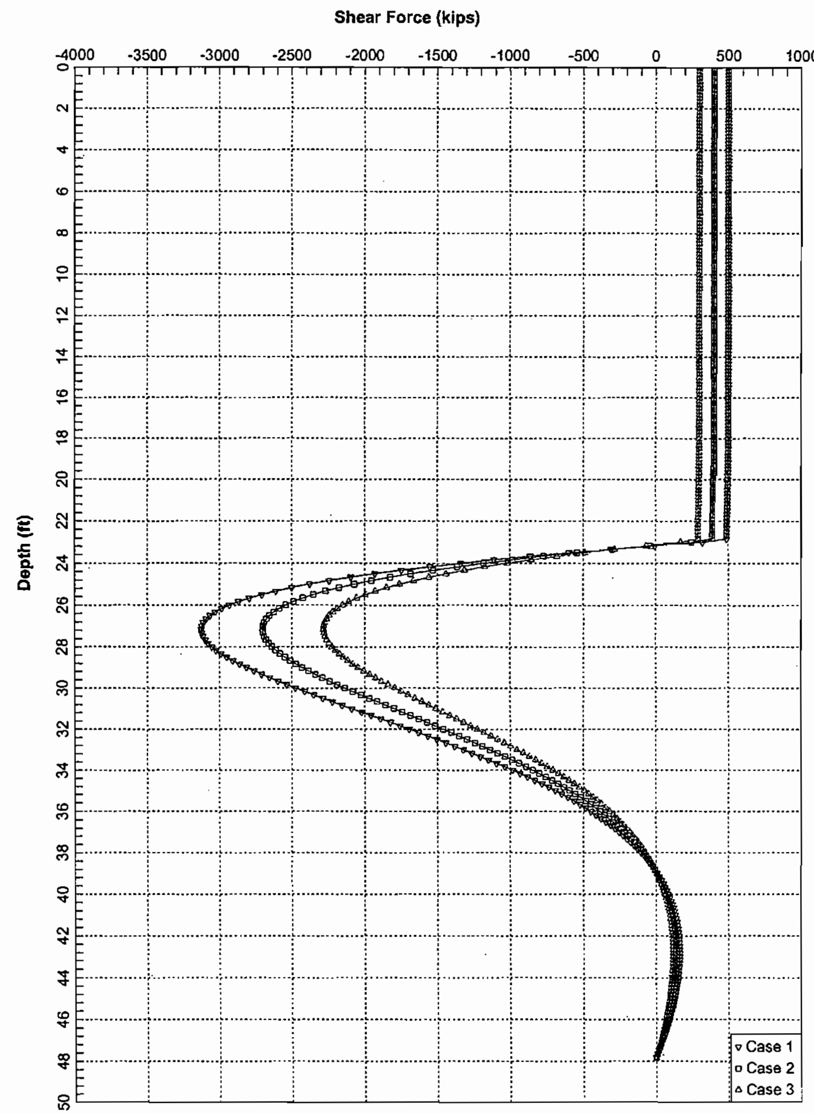


195



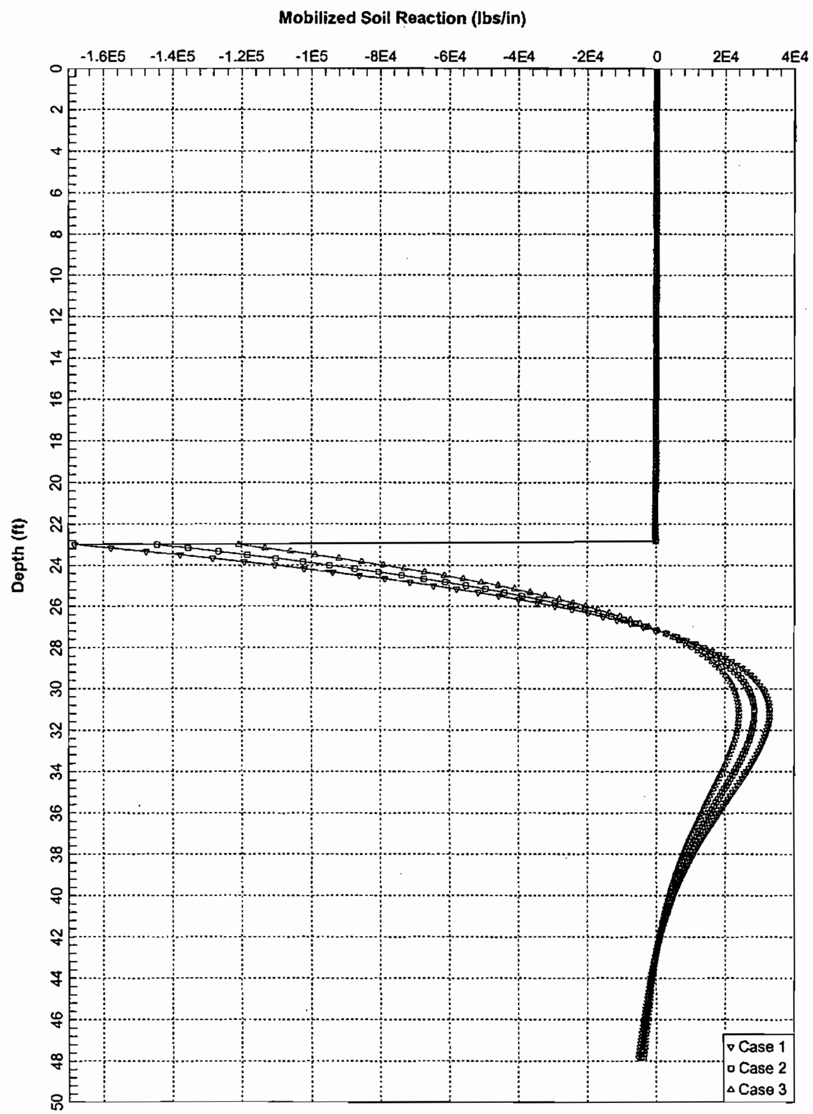
Unfactored Bending Moment (in-kips)
-1E5
-5E4
0
5E4
1E5
1.5E5
2E5
2.5E5
3E5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

196

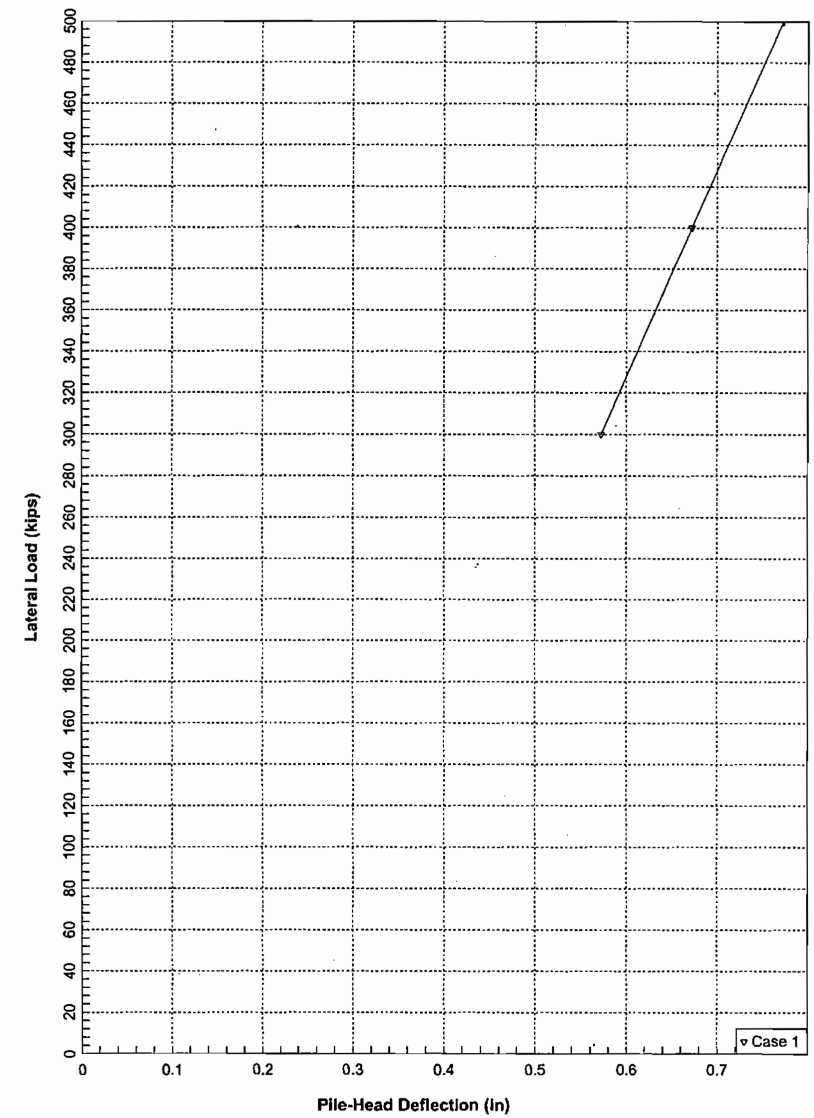


Shear Force (kips)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

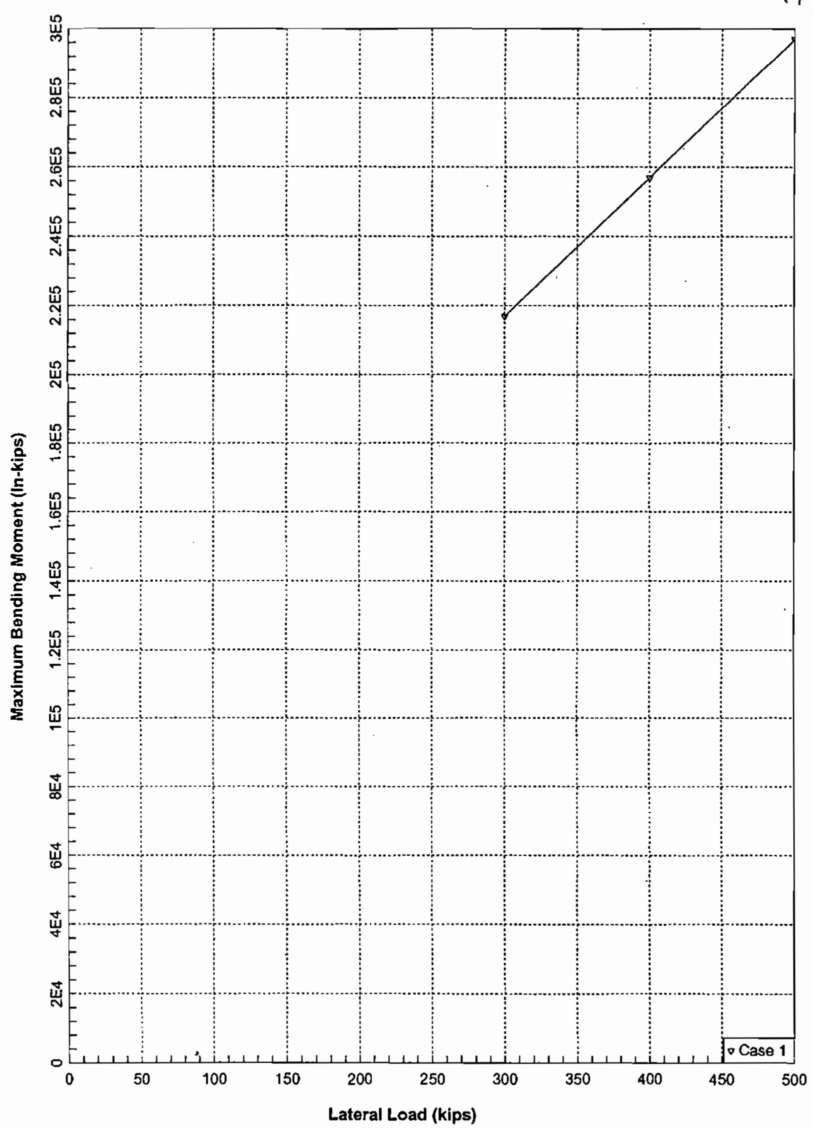
197


Mobilized Soil Reaction (lbs/in)
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

198

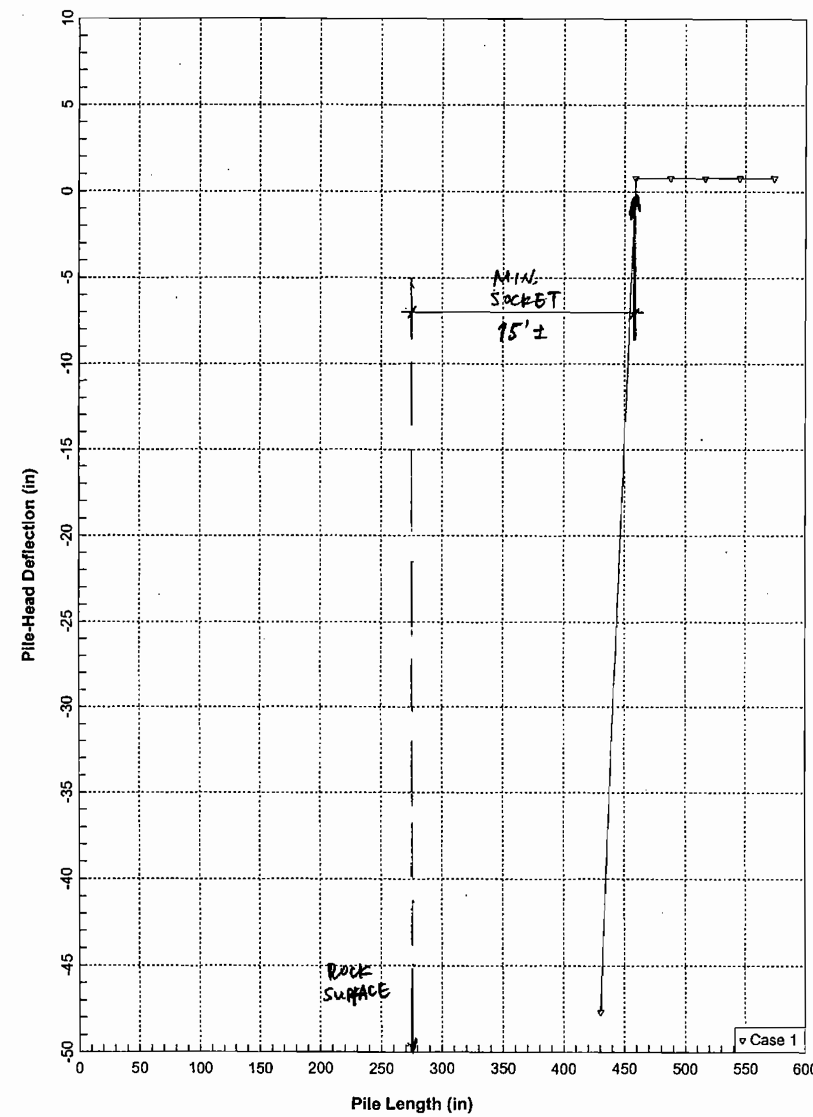

Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Case 1

199



Maximum Bending Moment (in-kips)
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
2E5
2.2E5
2.4E5
2.6E5
2.8E5
3E5
Lateral Load (kips)
0
50
100
150
200
250
300
350
400
450
500
Case 1

200



Pile-Head Deflection (in)
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
Pile Length (in)
0
50
100
150
200
250
300
350
400
450
500
550
600
Case 1
MIN. SOCKET
15' ±
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 5\
Name of input data file: Pier 5 - large - scour.lpd
Name of output file: Pier 5 - large - scour.lpo
Name of plot output file: Pier 5 - large - scour.lpp
Name of runtime file: Pier 5 - large - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:44:36

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 287
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 574.00 in
Depth of ground surface below top of pile = 199.00 in
Slope angle of ground surface = 10.00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	102.00000	5431065.	9630.7800	4074281.
2	276.0000	102.00000	5431065.	9630.7800	4074281.
3	276.0000	96.00000000	4169220.	7238.2300	4074281.
4	574.0000	96.00000000	4169220.	7238.2300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 199.000 in
Distance from top of pile to bottom of layer = 274.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 2 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 274.000 in
Distance from top of pile to bottom of layer = 874.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	199.00	.04109

2	274.00	.04109
3	274.00	.06111
4	874.00	.06111

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	199.000	.00000	38.00	-----	-----
2	274.000	.00000	38.00	-----	-----
3	274.000	4800.00000	.00	-----	-----
4	874.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	199.000	.7000	1.0000
2	274.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 500000.000 lbs
Bending moment at pile head = 156000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 400000.000 lbs
Bending moment at pile head = 144000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 300000.000 lbs
Bending moment at pile head = 132000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 500000.000 lbs
Specified moment at pile head = 156000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 400000.000 lbs
Specified moment at pile head = 144000000.000 in-lbs

Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 300000.000 lbs
Specified moment at pile head = 132000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 5.00E+05	M= 1.56E+08	4000000.	.7724881	2.9701E+08	-3124177.
1	V= 4.00E+05	M= 1.44E+08	4000000.	.6724177	2.5693E+08	-2702692.
1	V= 3.00E+05	M= 1.32E+08	4000000.	.5723577	2.1686E+08	-2281246.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

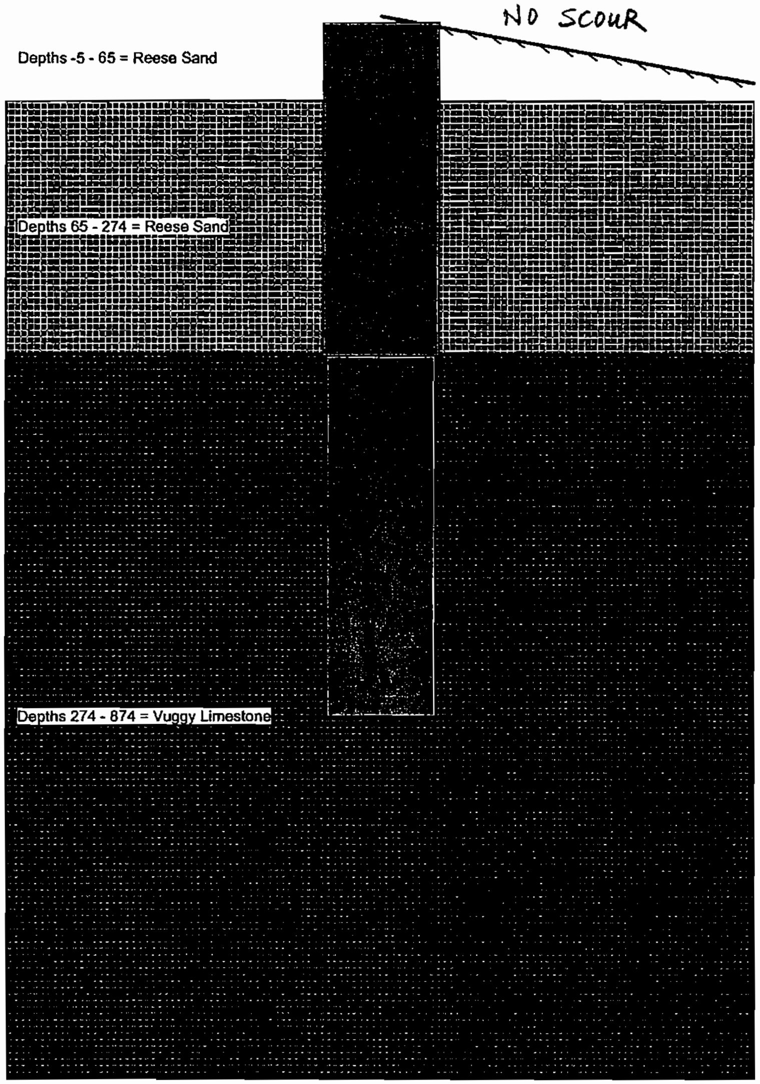
Shear = 500000. lbs
Moment = 156000000. in-lbs

Axial Load = 4000000. lbs

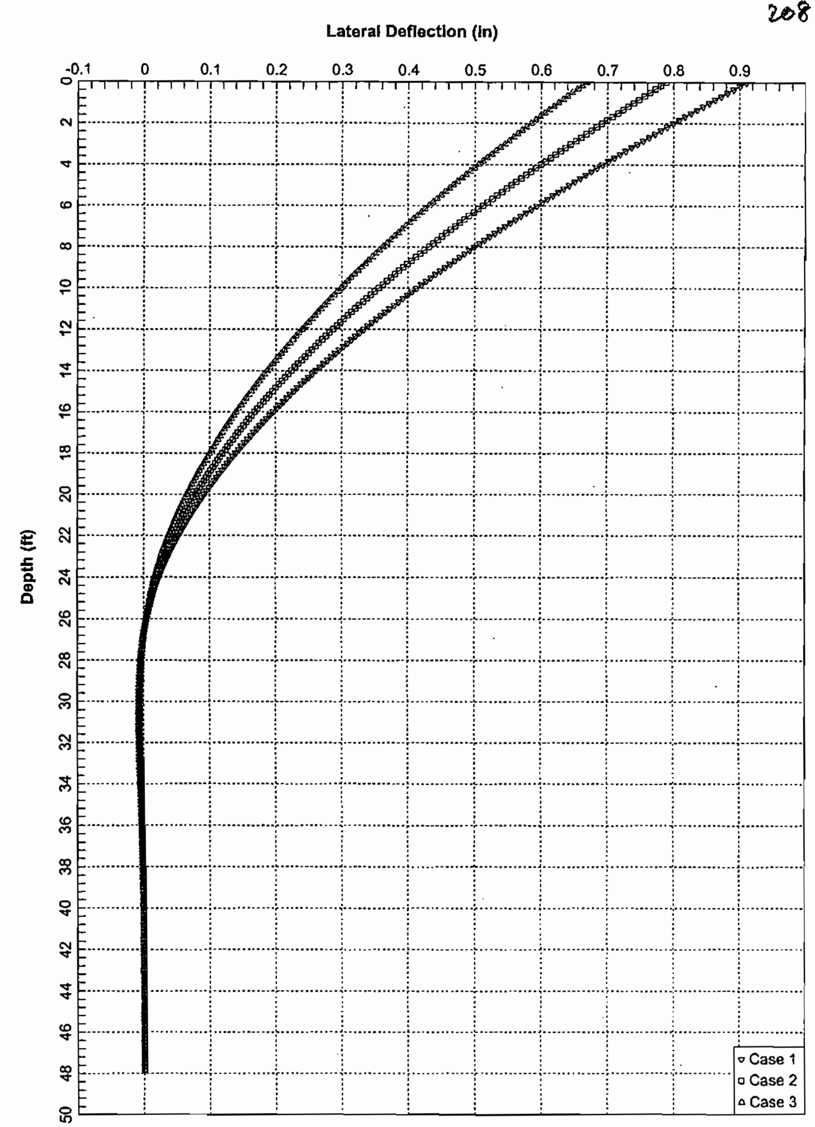
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
574.000	.77248808	2.970066E+08	-3124177.
545.300	.76813692	2.967591E+08	-3121602.
516.600	.76837472	2.967228E+08	-3116987.
487.900	.76908270	2.967346E+08	-3103833.
459.200	.76845103	2.966401E+08	-3101825.
430.500	-47.66998525	1.560000E+08	564666.42377

The analysis ended normally.

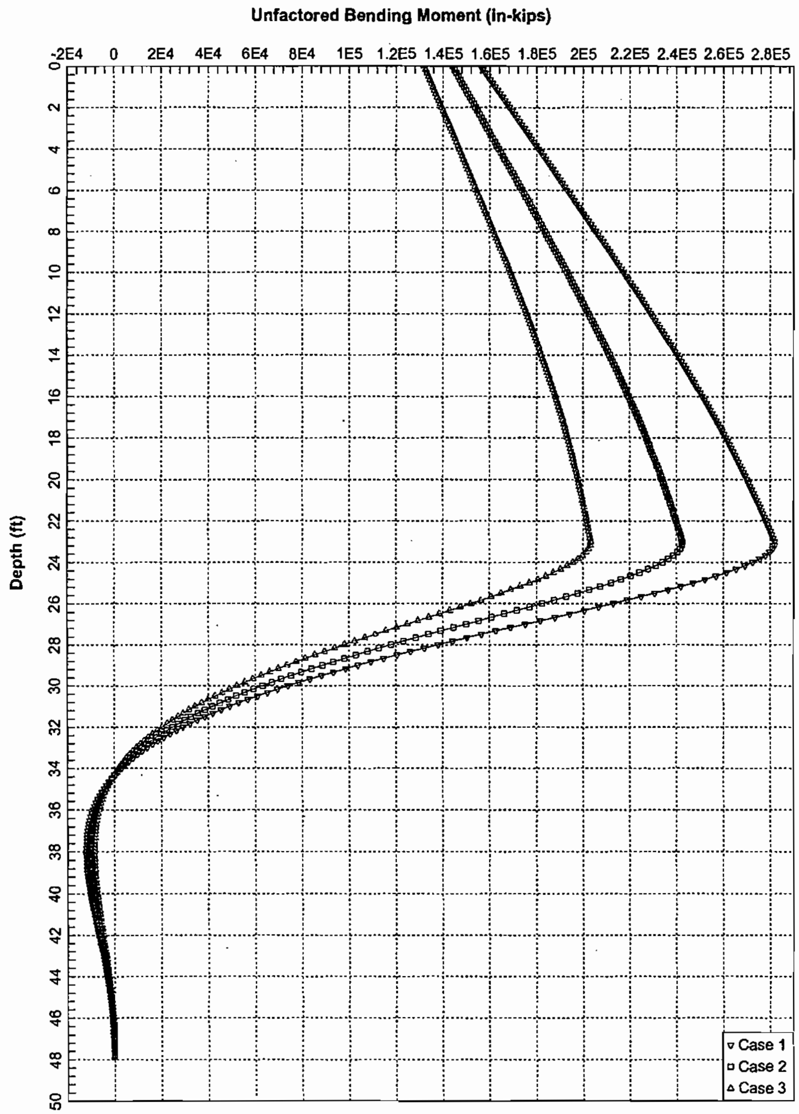
RUN 15: PIER 5
8' SHAFT
NO SCOUR



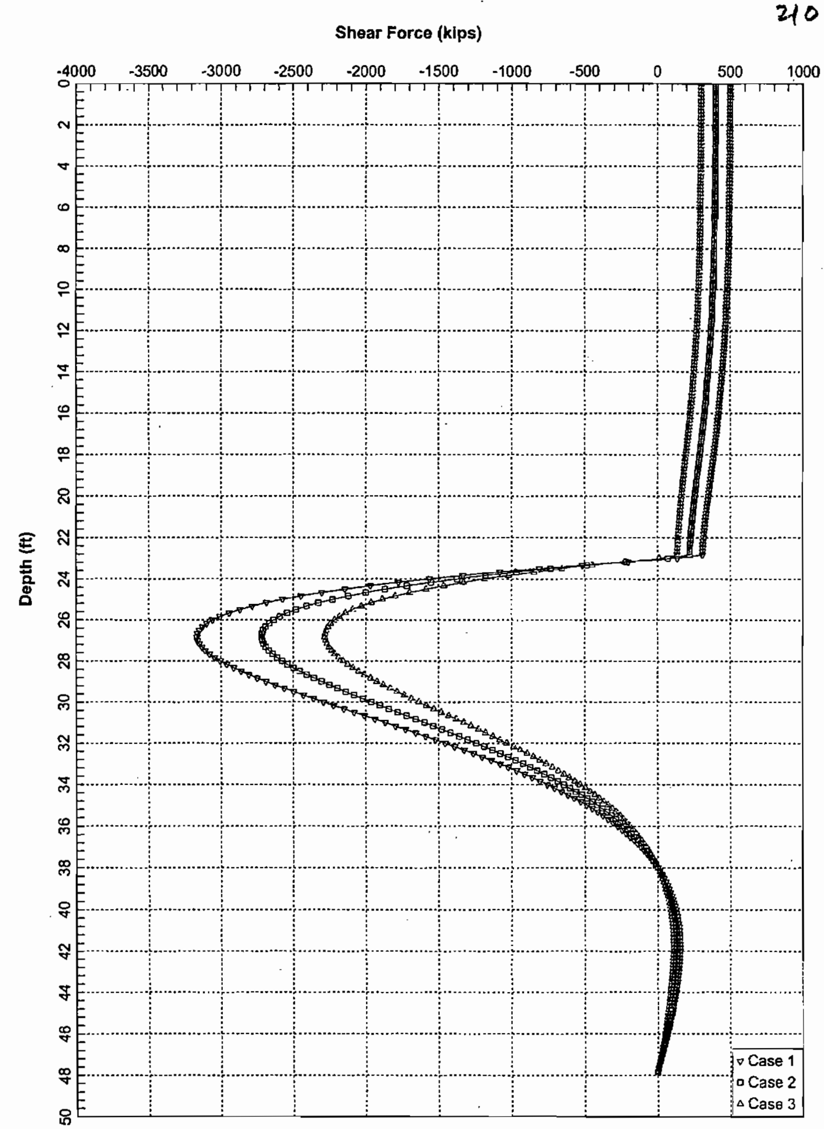
LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.



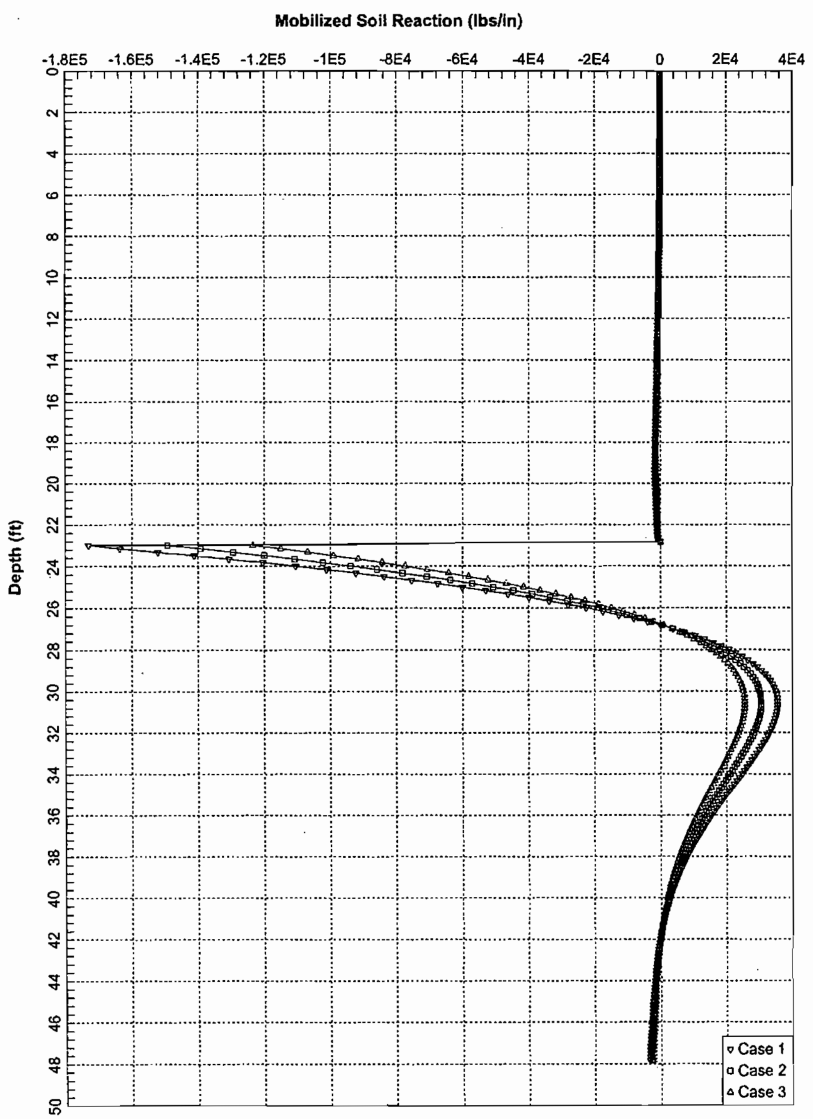
209


Unfactored Bending Moment (in-kips)
-2E4 0 2E4 4E4 6E4 8E4 1E5 1.2E5 1.4E5 1.6E5 1.8E5 2E5 2.2E5 2.4E5 2.6E5 2.8E5
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50
Case 1
Case 2
Case 3

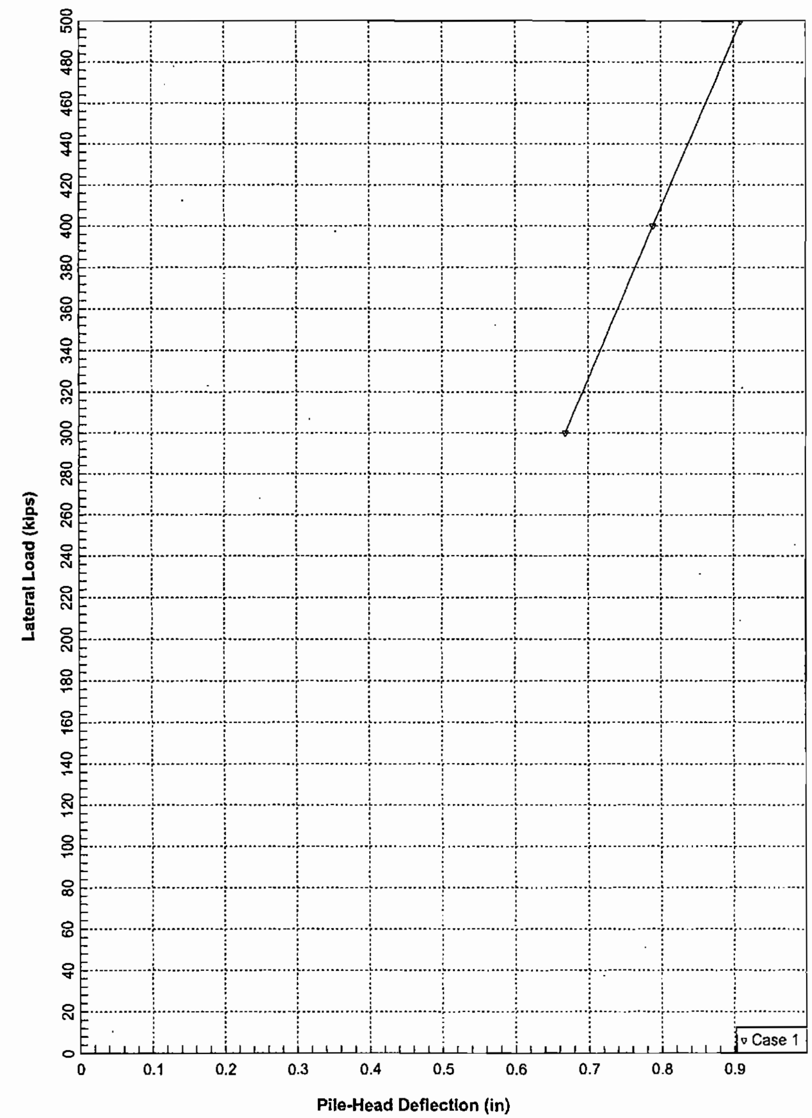
210


Shear Force (kips)
-4000 -3500 -3000 -2500 -2000 -1500 -1000 -500 0 500 1000
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50
Case 1
Case 2
Case 3

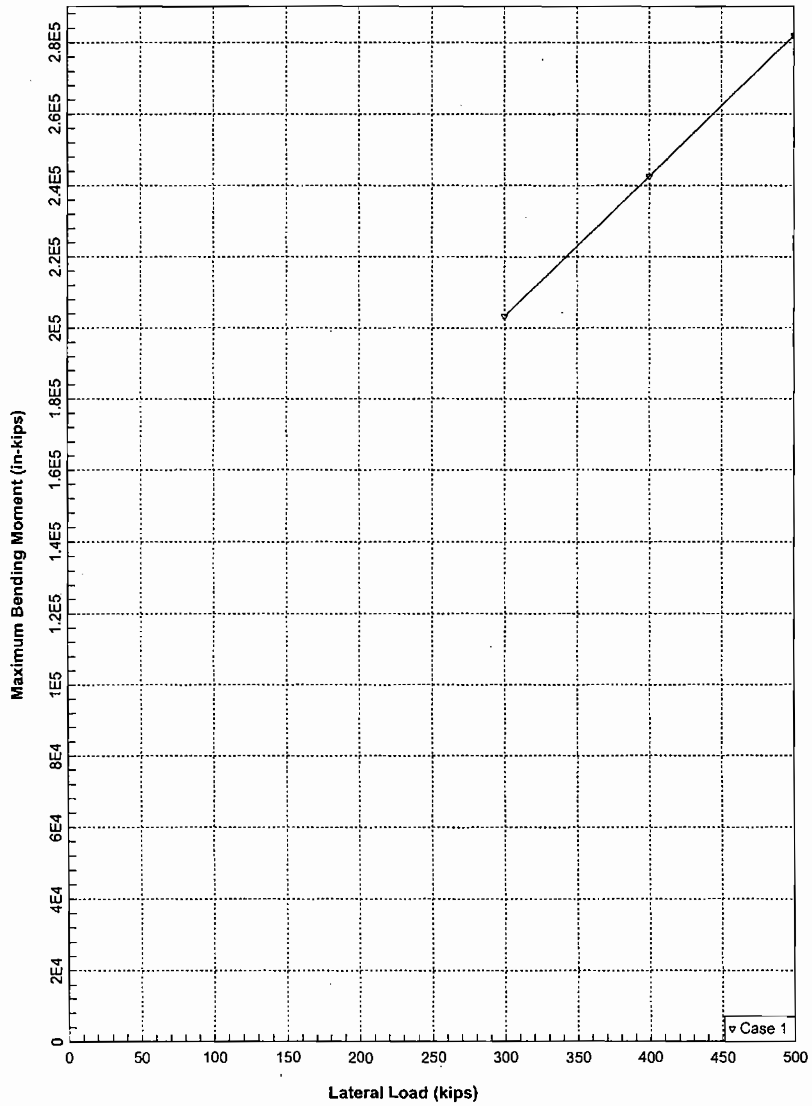
211


Mobilized Soil Reaction (lbs/in)
-1.8E5
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

212

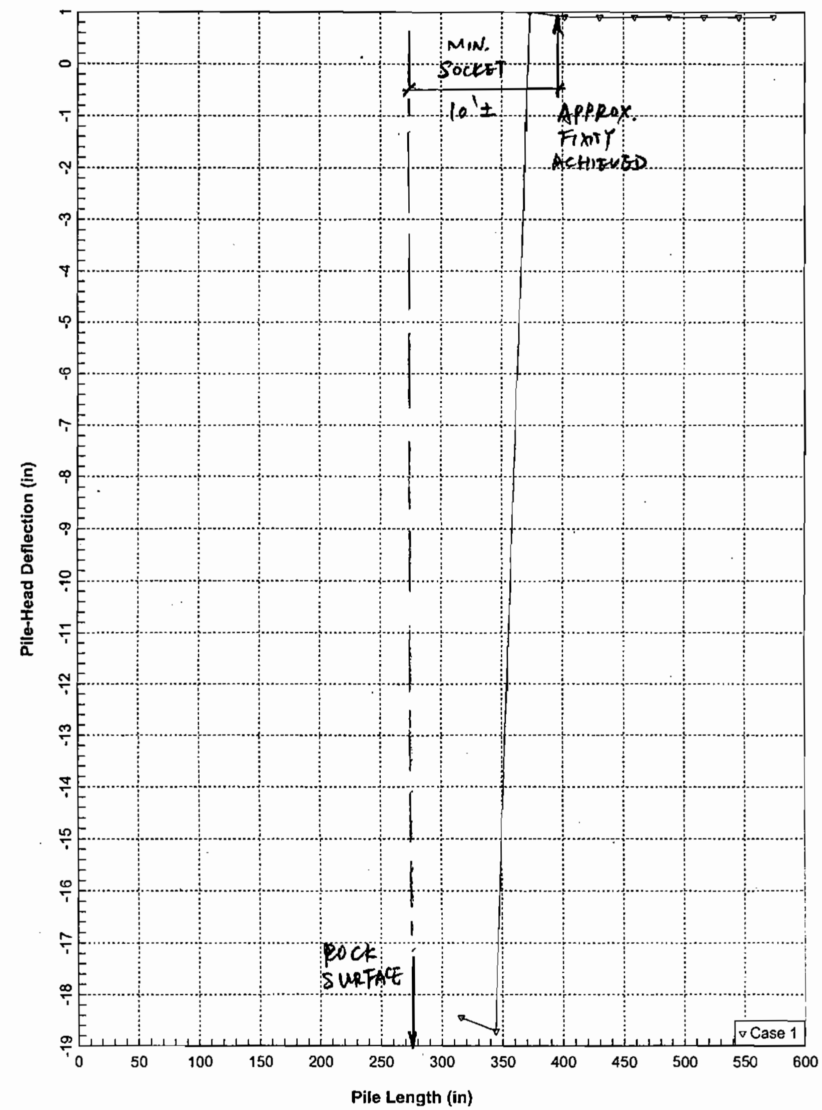

Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Case 1

213



Maximum Bending Moment (in-kips)
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
2E5
2.2E5
2.4E5
2.6E5
2.8E5
Lateral Load (kips)
0
50
100
150
200
250
300
350
400
450
500
▿ Case 1

214



Pile-Head Deflection (in)
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
-14
-15
-16
-17
-18
-19
Pile Length (in)
0
50
100
150
200
250
300
350
400
450
500
550
600
▿ Case 1
MIN. SOCKET
10'±
APPROX. FIXITY ACHIEVED
ROCK SURFACE

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 5\
Name of input data file: Pier 5 - small - no scour.lpd
Name of output file: Pier 5 - small - no scour.lpo
Name of plot output file: Pier 5 - small - no scour.lpp
Name of runtime file: Pier 5 - small - no scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:47:37

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 287
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 574.00 in
Depth of ground surface below top of pile = -5.00 in
Slope angle of ground surface = 10.00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	276.0000	96.00000000	4356263.	8611.2400	4074281.
3	276.0000	90.00000000	3220623.	6361.7300	4074281.
4	574.0000	90.00000000	3220623.	6361.7300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = -5.000 in
Distance from top of pile to bottom of layer = 65.000 in
p-y subgrade modulus k for top of soil layer = 20.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 20.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 65.000 in
Distance from top of pile to bottom of layer = 274.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 274.000 in
Distance from top of pile to bottom of layer = 874.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth

is defined using 6 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-5.00	.02257
2	65.00	.02257
3	65.00	.04109
4	274.00	.04109
5	274.00	.06111
6	874.00	.06111

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-5.000	.00000	29.50	-----	-----
2	65.000	.00000	29.50	-----	-----
3	65.000	.00000	38.00	-----	-----
4	274.000	.00000	38.00	-----	-----
5	274.000	4800.00000	.00	-----	-----
6	874.000	4800.00000	.00	-----	-----

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	-5.000	.7000	1.0000
2	274.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 500000.000 lbs
Bending moment at pile head = 156000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 400000.000 lbs
Bending moment at pile head = 144000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 300000.000 lbs
Bending moment at pile head = 132000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 500000.000 lbs
Specified moment at pile head = 156000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 400000.000 lbs
Specified moment at pile head = 144000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 300000.000 lbs
Specified moment at pile head = 132000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 5.00E+05	M= 1.56E+08	4000000.	.9103446	2.8195E+08	-3163369.
1	V= 4.00E+05	M= 1.44E+08	4000000.	.7891612	2.4256E+08	-2720995.
1	V= 3.00E+05	M= 1.32E+08	4000000.	.6685145	2.0331E+08	-2280187.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

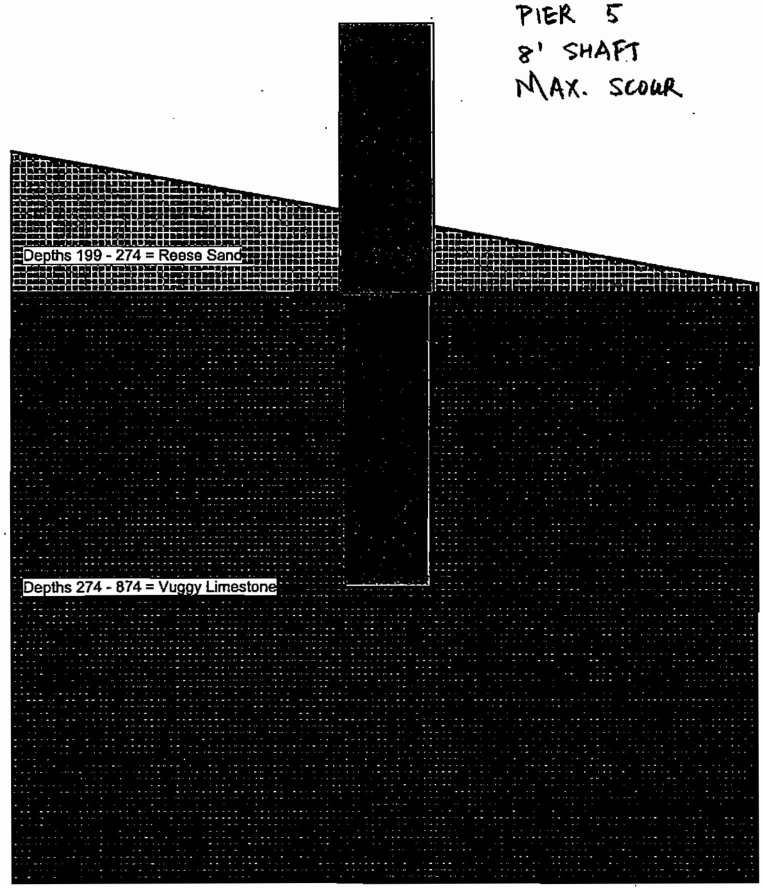
Shear = 500000. lbs
Moment = 156000000. in-lbs
Axial Load = 4000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
574.000	.91034464	2.819511E+08	-3163369.
545.300	.90451667	2.817655E+08	-3162515.
516.600	.90468259	2.817143E+08	-3160614.
487.900	.90565536	2.817463E+08	-3152314.
459.200	.90523040	2.817612E+08	-3137918.
430.500	.89986210	2.815449E+08	-3177504.
401.800	.91144062	2.814486E+08	-3465517.
373.100	.99508564	2.810168E+08	-4334205.
344.400	-18.71338255	1.560296E+08	545641.74483
315.700	-18.44260151	1.559964E+08	535018.64235

The analysis ended normally.

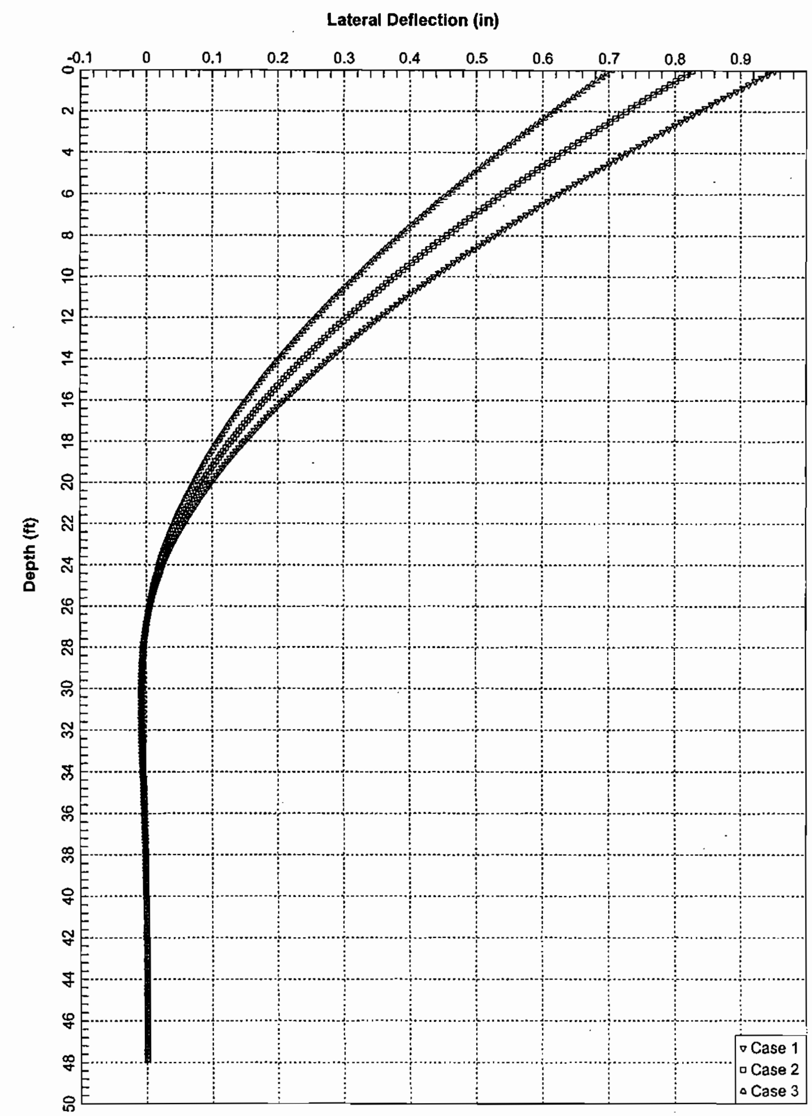
RUN 16:
PIER 5
8' SHAFT
MAX. SCOUR

221

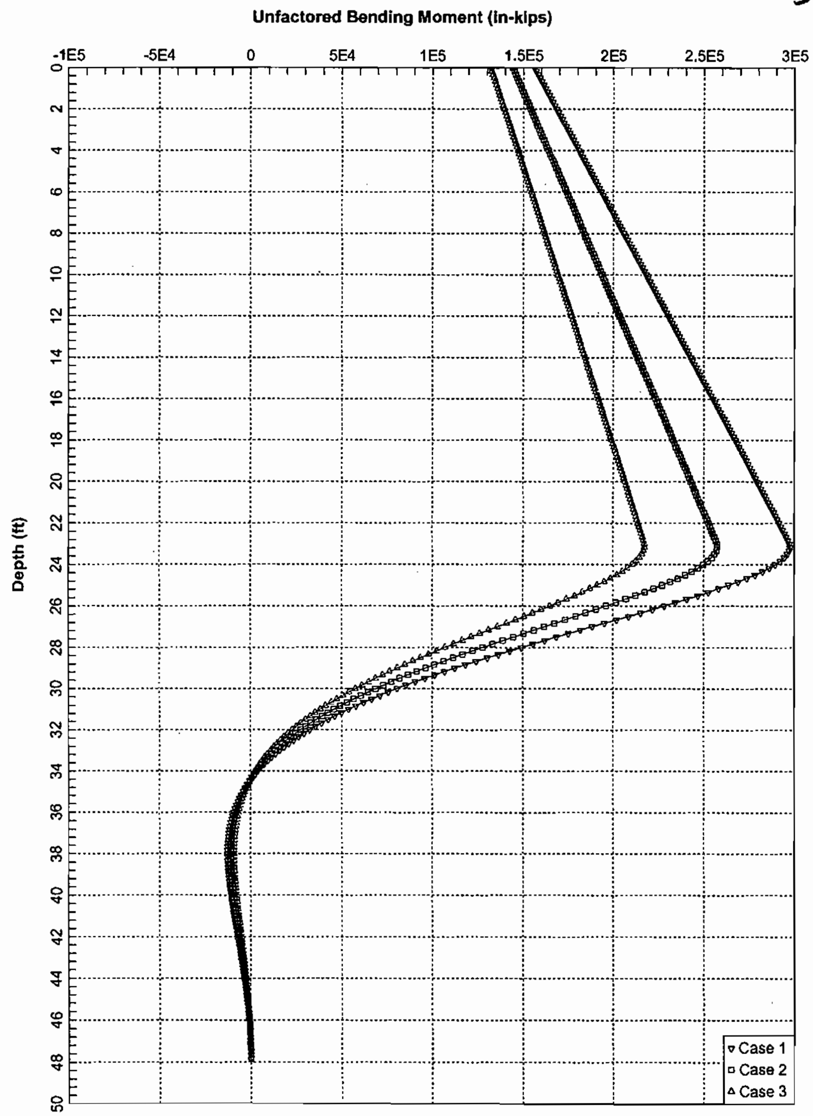


LPile Plus for Windows 5.0.31 for Networks, (c) 2006 by Ensoft, Inc.

222

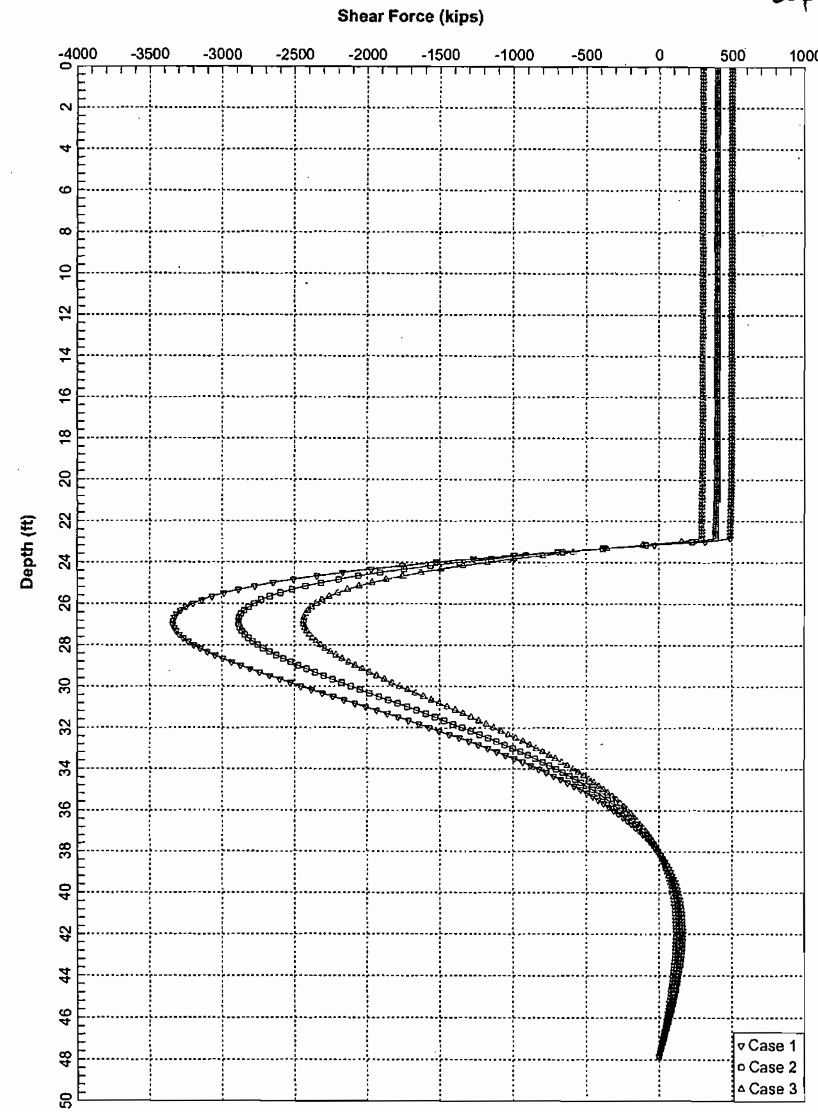


223



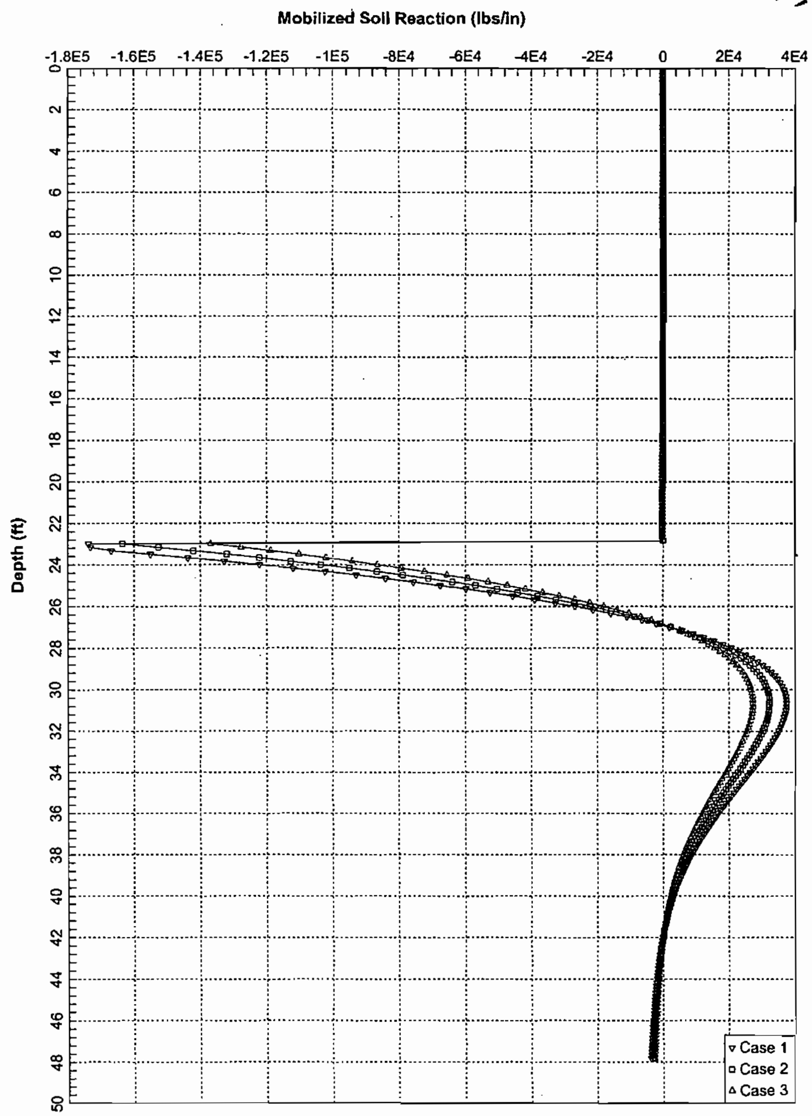
Unfactored Bending Moment (in-kips)
-1E5
-5E4
0
5E4
1E5
1.5E5
2E5
2.5E5
3E5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

224



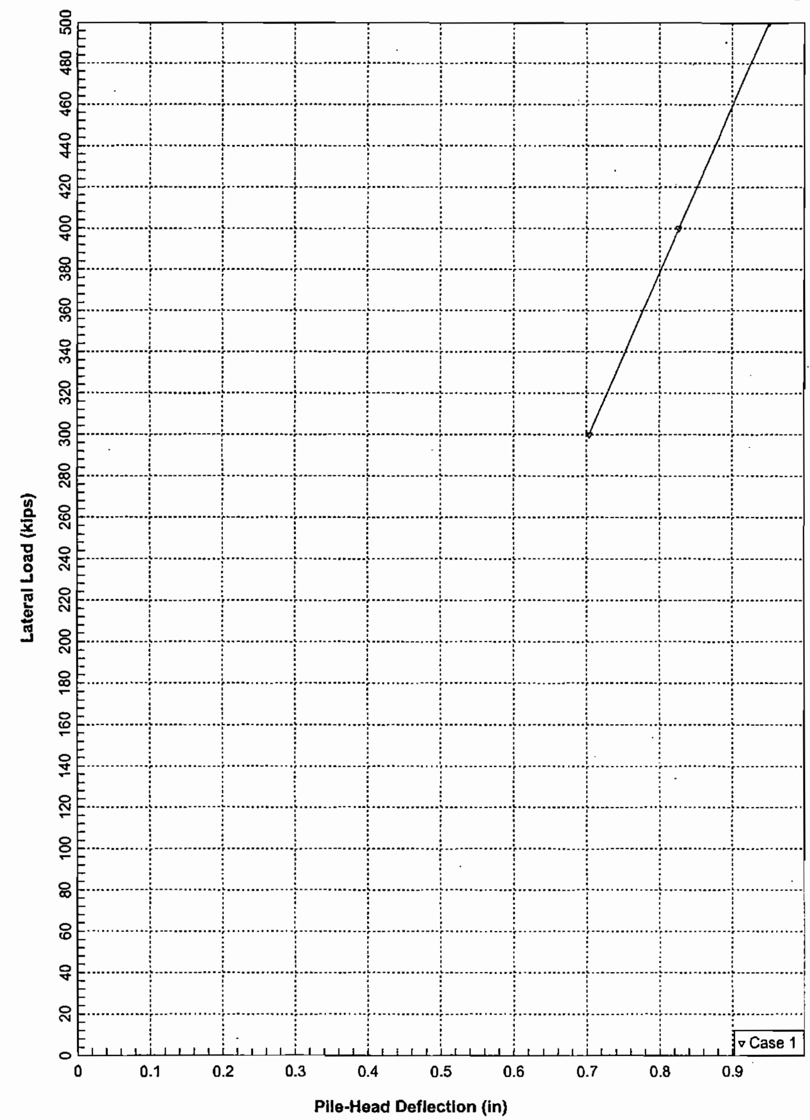
Shear Force (kips)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

225



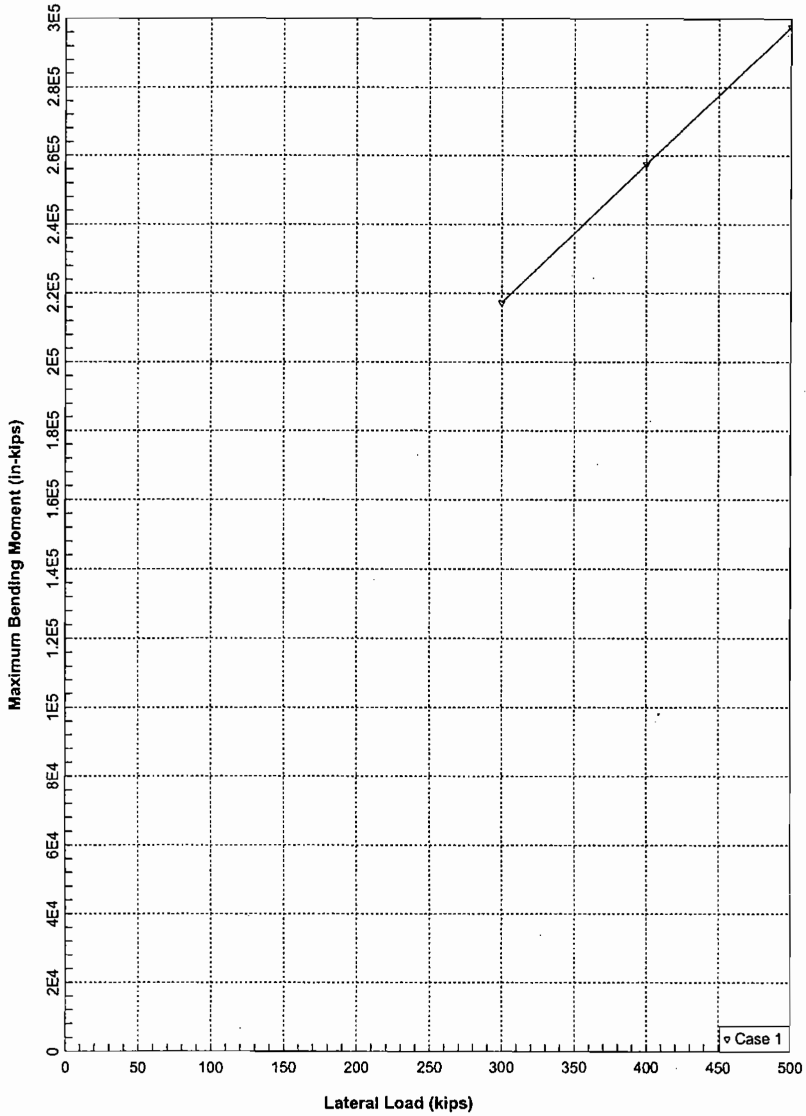
Mobilized Soil Reaction (lbs/in)
-1.8E5
-1.6E5
-1.4E5
-1.2E5
-1E5
-8E4
-6E4
-4E4
-2E4
0
2E4
4E4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1
Case 2
Case 3

226

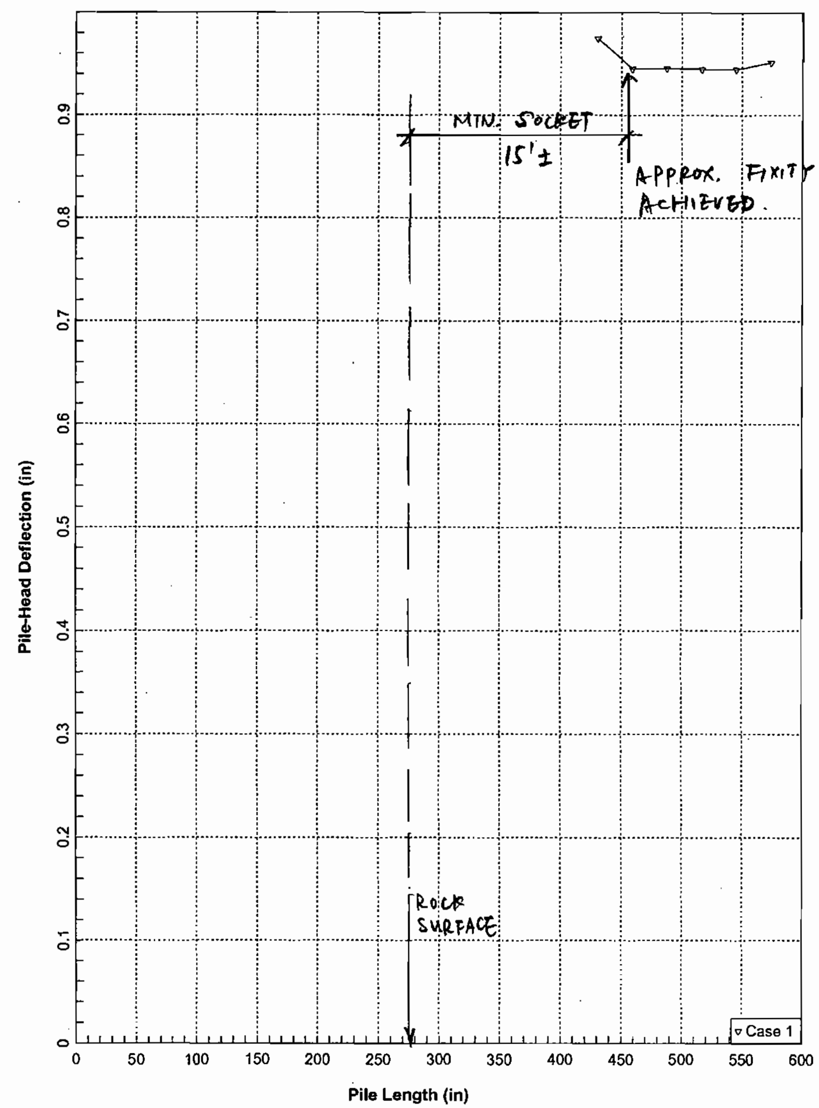


Lateral Load (kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Case 1

227


Maximum Bending Moment (in-kips)
0
2E4
4E4
6E4
8E4
1E5
1.2E5
1.4E5
1.6E5
1.8E5
2E5
2.2E5
2.4E5
2.6E5
2.8E5
3E5
Lateral Load (kips)
0
50
100
150
200
250
300
350
400
450
500
Case 1

228


Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Pile Length (in)
0
50
100
150
200
250
300
350
400
450
500
550
600
Case 1
MIN. SOCKET
15' ±
APPROX. FIXITY
ACHIEVED.
ROCK
SURFACE

229

LPILE Plus for Windows, Version 5.0 (5.0.31)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Mangtao Du
PB Americas, Inc.

Path to file locations: L:\East End Bridge\Lateral Load Analyses\Pier 5\
Name of input data file: Pier 5 - small - scour.lpd
Name of output file: Pier 5 - small - scour.lpo
Name of plot output file: Pier 5 - small - scour.lpp
Name of runtime file: Pier 5 - small - scour.lpr

Time and Date of Analysis

Date: December 21, 2007 Time: 11:50:16

Problem Title

East End Bridge - Preliminary Design for Pier Foundation

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output summary table of values for pile-head deflection, maximum bending moment, and shear force only
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

230

Solution Control Parameters:
- Number of pile increments = 287
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Only summary tables of pile-head deflection, maximum bending moment, and maximum shear force are to be printed in output file.

Pile Structural Properties and Geometry

Pile Length = 574.00 in
Depth of ground surface below top of pile = 199.00 in
Slope angle of ground surface = 10.00 deg.

Structural properties of pile defined using 4 points

Point	Depth X (in)	Pile Diameter (in)	Moment of Inertia (in**4)	Pile Area (Sq.in)	Modulus of Elasticity (lbs/Sq.in)
1	0.0000	96.00000000	4356263.	8611.2400	4074281.
2	276.0000	96.00000000	4356263.	8611.2400	4074281.
3	276.0000	90.00000000	3220623.	6361.7300	4074281.
4	574.0000	90.00000000	3220623.	6361.7300	4074281.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 199.000 in
Distance from top of pile to bottom of layer = 274.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 2 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 274.000 in
Distance from top of pile to bottom of layer = 874.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 4 points

Point No.	Depth X (in)	Eff. Unit Weight (lbs/in**3)
1	199.00	.04109

2	274.00	.04109
3	274.00	.06111
4	874.00	.06111

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	199.000	.00000	38.00	------	------
2	274.000	.00000	38.00	------	------
3	274.000	4800.00000	.00	------	------
4	874.000	4800.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	199.000	.7000	1.0000
2	274.000	.7000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves

Number of cycles of loading = 30.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 500000.000 lbs
Bending moment at pile head = 156000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 400000.000 lbs
Bending moment at pile head = 144000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 300000.000 lbs
Bending moment at pile head = 132000000.000 in-lbs
Axial load at pile head = 4000000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 500000.000 lbs
Specified moment at pile head = 156000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 400000.000 lbs
Specified moment at pile head = 144000000.000 in-lbs

Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 300000.000 lbs
Specified moment at pile head = 132000000.000 in-lbs
Specified axial load at pile head = 4000000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output Verification:

Computed forces and moments are within specified convergence limits.

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 5.00E+05	M= 1.56E+08	4000000.	.9514358	2.9767E+08	-3338202.
1	V= 4.00E+05	M= 1.44E+08	4000000.	.8267117	2.5745E+08	-2887900.
1	V= 3.00E+05	M= 1.32E+08	4000000.	.7039511	2.1730E+08	-2437999.

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

Shear = 500000. lbs
Moment = 156000000. in-lbs

Axial Load = 4000000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
574.000	.95143581	2.976724E+08	-3338202.
545.300	.94446242	2.973789E+08	-3336915.
516.600	.94474182	2.973496E+08	-3335844.
487.900	.94579904	2.973685E+08	-3325298.
459.200	.94514739	2.972799E+08	-3310945.
430.500	.97490704	1.560000E+08	500000.05933

The analysis ended normally.

APPENDIX H-3
DRILLED SHAFT POINT OF FIXITY CALCULATIONS

PARSONS BRINCKERHOFF
COMPUTATION SHEET

Geotechnical & Tunneling Division

BY: M. Du **DATE:** 12/26/2007 **PROJECT:** East End Bridge
CHECKED BY: S. Malhotra **DATE:** 12/26/07 **PAGE** 1 **OF** 4
SUBJECT: Drilled Shaft Equivalent Point of Fixity Calculations for Pier 1
Preliminary Design

Purpose

To estimate equivalent point of fixity for drilled shaft, based on LPILE results, for Pier 1.

References

- Results of lateral load analyses with LPILE
- Elastic beam deflection equations

Index

- Calculations (pp. 1 ~ 4)

Functions to define beam end deflections under concentrated loads and bending moments:

Cantilever beam under concentrated shear at cantilever end:

$$\delta_P(P,L,E,I) := \frac{P \cdot L^3}{3E \cdot I}$$

Cantilever beam under concentrated bending moment at cantilever end:

$$\delta_M(M,L,E,I) := \frac{M \cdot L^2}{2E \cdot I}$$

Cantilever beam under combined concentrated shear and bending moment:

$$\delta_c(P,M,L,E,I) := \frac{P \cdot L^3}{3E \cdot I} + \frac{M \cdot L^2}{2E \cdot I}$$

Beam with one end fixed and one end free to translate but not rotate:

$$\delta_f(P,L,E,I) := \frac{P \cdot L^3}{12E \cdot I}$$

Drilled Shaft Cross-Section Properties

Young's Modulus: $E := 4074281\text{psi}$

8.5 ft Diameter Shaft: $I_{8.5} := 5431065\text{in}^4$

8 ft Diameter Shaft: $I_8 := 4356263\text{in}^4$

Shaft Head Conditions and Loading

Head Fixity: Free

Shaft Head Elevation: $EL_h := 430\text{ft}$

Shear:

$$P := \begin{pmatrix} 250 \\ 225 \\ 200 \end{pmatrix} \text{kip}$$

Moment:

$$M := \begin{pmatrix} 6000 \\ 5000 \\ 4000 \end{pmatrix} \text{kip} \cdot \text{ft}$$

Deflection computed by LPILE:

Run 1
8.5 ft Shaft
No scour,

$$\Delta_1 := \begin{pmatrix} 0.915 \\ 0.773 \\ 0.636 \end{pmatrix} \text{in}$$

Run 2
8.5 ft Shaft
Max scour,

$$\Delta_2 := \begin{pmatrix} 1.938 \\ 1.616 \\ 1.317 \end{pmatrix} \text{in}$$

Run 3
8 ft Shaft
No scour,

$$\Delta_3 := \begin{pmatrix} 1.075 \\ 0.907 \\ 0.743 \end{pmatrix} \text{in}$$

Run 4
8 ft Shaft
Max scour,

$$\Delta_4 := \begin{pmatrix} 2.449 \\ 2.005 \\ 1.615 \end{pmatrix} \text{in}$$

Equivalent Beam Lengths:

8.5 ft Shaft, No scour $L_1 := 42\text{ft}$ trial & error

8.5 ft Shaft, Max scour $L_2 := 55.6\text{ft}$ trial & error

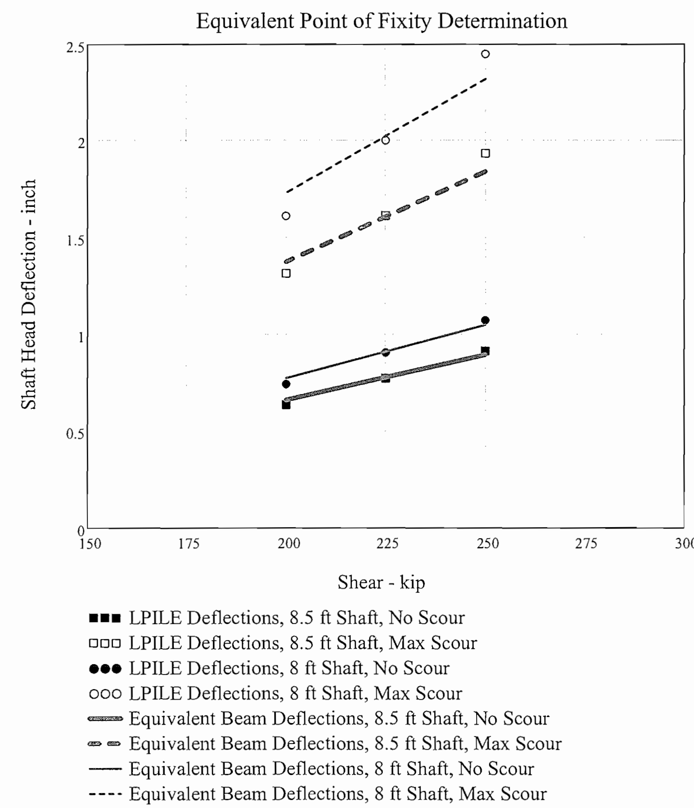
8 ft Shaft, No scour $L_3 := 41\text{ft}$ trial & error

8 ft Shaft, Max scour $L_4 := 55.8\text{ft}$ trial & error

Deflections of Equivalent Beams:

8.5 ft Shaft, No scour $\delta_1 := \delta_c(P, M, L_1, E, I_{8.5})$

8.5 ft Shaft, Max scour	$\delta_2 := \delta_c(P, M, L_2, E, I_{8.5})$
8 ft Shaft, No scour	$\delta_3 := \delta_c(P, M, L_3, E, I_8)$
8 ft Shaft, Max scour	$\delta_4 := \delta_c(P, M, L_4, E, I_8)$



Equivalent Point of Fixity -- Elevation:

8.5 ft Shaft, No scour	$EL_{f1} := EL_h - L_1$	$EL_{f1} = 388\ ft$
8.5 ft Shaft, Max scour	$EL_{f2} := EL_h - L_2$	$EL_{f2} = 374.4\ ft$
8 ft Shaft, No scour	$EL_{f3} := EL_h - L_3$	$EL_{f3} = 389\ ft$
8 ft Shaft, Max scour	$EL_{f4} := EL_h - L_4$	$EL_{f4} = 374.2\ ft$
	Bed Rock Elevation:	$EL_R := 334ft$

PARSONS BRINCKERHOFF
COMPUTATION SHEET
Geotechnical & Tunneling Division

BY: M. Du **DATE:** 12/26/2007 **PROJECT:** East End Bridge
CHECKED BY: S. Mallisther **DATE:** 12/27/07 **PAGE** 1 **OF** 4
SUBJECT: Drilled Shaft Equivalent Point of Fixity Calculations for Pier 2
Preliminary Design

Purpose

To estimate equivalent point of fixity for drilled shaft, based on LPILE results, for Pier 2.

References

- Results of lateral load analyses with LPILE
- Elastic beam deflection equations

Index

- Calculations (pp. 1 ~ 4)

Purpose

To estimate equivalent point of fixity for drilled shaft, based on LPILE results.

Functions to define beam end deflections under concentrated loads and bending moments:

Cantilever beam under concentrated shear at cantilever end:

$$\delta_P(P,L,E,I) := \frac{P \cdot L^3}{3E \cdot I}$$

Cantilever beam under concentrated bending moment at cantilever end:

$$\delta_M(M,L,E,I) := \frac{M \cdot L^2}{2E \cdot I}$$

Cantilever beam under combined concentrated shear and bending moment:

$$\delta_c(P,M,L,E,I) := \frac{P \cdot L^3}{3E \cdot I} + \frac{M \cdot L^2}{2E \cdot I}$$

Beam with one end fixed and one end free to translate but not rotate:

$$\delta_f(P,L,E,I) := \frac{P \cdot L^3}{12E \cdot I}$$

Drilled Shaft Cross-Section Properties

Young's Modulus: $E := 4074281 psi$

8.5 ft Diameter Shaft: $I_{8.5} := 5431065 in^4$

8 ft Diameter Shaft: $I_8 := 4356263 in^4$

Shaft Head Conditions and Loading

Head Fixity: Free

Shaft Head Elevation: $EL_h := 414.8 ft$

Shear: $P := \begin{pmatrix} 250 \\ 225 \\ 200 \end{pmatrix} kip$

Moment: $M := \begin{pmatrix} 6000 \\ 5000 \\ 4000 \end{pmatrix} kip \cdot ft$

Deflection computed by LPILE:

Run 1 8.5 ft Shaft No scour,	Run 2 8.5 ft Shaft Max scour,	Run 3 8 ft Shaft No scour,	Run 4 8 ft Shaft Max scour,
$\Delta_1 := \begin{pmatrix} 1.287 \\ 1.097 \\ 0.914 \end{pmatrix} in$	$\Delta_2 := \begin{pmatrix} 2.926 \\ 2.498 \\ 2.086 \end{pmatrix} in$	$\Delta_3 := \begin{pmatrix} 1.54 \\ 1.307 \\ 1.083 \end{pmatrix} in$	$\Delta_4 := \begin{pmatrix} 3.626 \\ 3.081 \\ 2.562 \end{pmatrix} in$

Equivalent Beam Lengths:

8.5 ft Shaft, No scour	$L_1 := 48.1 ft$	trial & error
8.5 ft Shaft, Max scour	$L_2 := 65.7 ft$	trial & error
8 ft Shaft, No scour	$L_3 := 47.2 ft$	trial & error
8 ft Shaft, Max scour	$L_4 := 65.4 ft$	trial & error

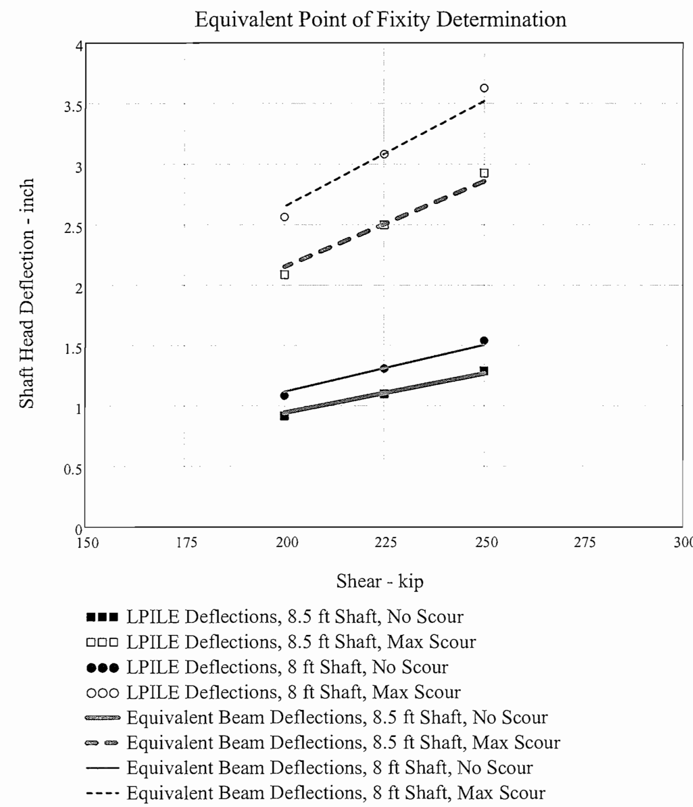
Deflections of Equivalent Beams:

8.5 ft Shaft, No scour $\delta_1 := \delta_c(P, M, L_1, E, I_{8.5})$

8.5 ft Shaft, Max scour $\delta_2 := \delta_c(P, M, L_2, E, I_{8.5})$

8 ft Shaft, No scour $\delta_3 := \delta_c(P, M, L_3, E, I_8)$

8 ft Shaft, Max scour $\delta_4 := \delta_c(P, M, L_4, E, I_8)$



<u>Equivalent Point of Fixity -- Elevation</u>:

8.5 ft Shaft, No scour	$EL_{f1} := EL_h - L_1$	$EL_{f1} = 366.7\,ft$
8.5 ft Shaft, Max scour	$EL_{f2} := EL_h - L_2$	$EL_{f2} = 349.1\,ft$
8 ft Shaft, No scour	$EL_{f3} := EL_h - L_3$	$EL_{f3} = 367.6\,ft$
8 ft Shaft, Max scour	$EL_{f4} := EL_h - L_4$	$EL_{f4} = 349.4\,ft$
	Bed Rock Elevation:	$EL_R := 334.9ft$

PARSONS BRINCKERHOFF
COMPUTATION SHEET
Geotechnical & Tunneling Division

BY: M. Du **DATE:** 12/26/2007 **PROJECT:** East End Bridge
CHECKED BY: S. Malhotra **DATE:** 12/27/07 **PAGE** 1 **OF** 4
SUBJECT: Drilled Shaft Equivalent Point of Fixity Calculations for Pier 3
Preliminary Design

Purpose

To estimate equivalent point of fixity for drilled shaft, based on LPILE results, for Pier 3.

References

- Results of lateral load analyses with LPILE
- Elastic beam deflection equations

Index

- Calculations (pp. 1 ~ 4)

Purpose

To estimate equivalent point of fixity for drilled shaft, based on LPILE results.

Functions to define beam end deflections under concentrated loads and bending moments:

Cantilever beam under concentrated shear at cantilever end:

$$\delta_P(P, L, E, I) := \frac{P \cdot L^3}{3E \cdot I}$$

Cantilever beam under concentrated bending moment at cantilever end:

$$\delta_M(M, L, E, I) := \frac{M \cdot L^2}{2E \cdot I}$$

Cantilever beam under combined concentrated shear and bending moment:

$$\delta_c(P, M, L, E, I) := \frac{P \cdot L^3}{3E \cdot I} + \frac{M \cdot L^2}{2E \cdot I}$$

Beam with one end fixed and one end free to translate but not rotate:

$$\delta_f(P, L, E, I) := \frac{P \cdot L^3}{12E \cdot I}$$

Drilled Shaft Cross-Section Properties

Young's Modulus: $E := 4074281\text{psi}$

8.5 ft Diameter Shaft: $I_{8.5} := 5431065\text{in}^4$

8 ft Diameter Shaft: $I_8 := 4356263\text{in}^4$

Shaft Head Conditions and Loading

Head Fixity: Fixed

Shaft Head Elevation: $EL_h := 405.5\text{ft}$

Shear:

$$P := \begin{pmatrix} 1500 \\ 1250 \\ 1000 \end{pmatrix} \text{kip}$$

Moment:

Not applied as the shaft head is fixed against rotation.

Deflection computed by LPILE:

Run 9
8.5 ft Shaft
No scour,

$$\Delta_1 := \begin{pmatrix} 4.319 \\ 3.485 \\ 2.671 \end{pmatrix} \text{in}$$

Run 10
8.5 ft Shaft
Max scour,

$$\Delta_2 := \begin{pmatrix} 5.409 \\ 4.455 \\ 3.526 \end{pmatrix} \text{in}$$

Run 11
8 ft Shaft
No scour,

$$\Delta_3 := \begin{pmatrix} 5.415 \\ 4.359 \\ 3.337 \end{pmatrix} \text{in}$$

Run 12
8 ft Shaft
Max scour,

$$\Delta_4 := \begin{pmatrix} 6.878 \\ 5.653 \\ 4.459 \end{pmatrix} \text{in}$$

Equivalent Beam Lengths:

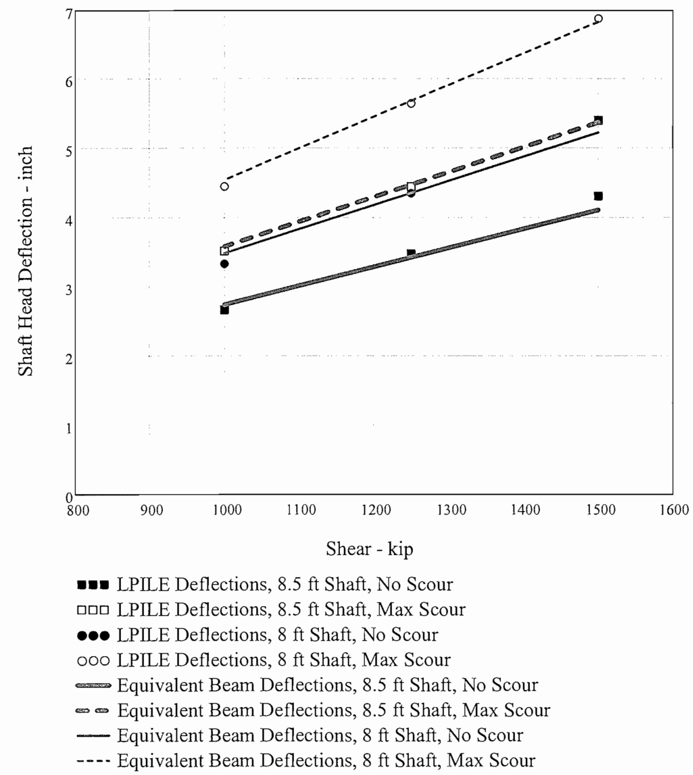
8.5 ft Shaft, No scour	$L_1 := 75\text{ft}$	trial & error
8.5 ft Shaft, Max scour	$L_2 := 82\text{ft}$	trial & error
8 ft Shaft, No scour	$L_3 := 75.5\text{ft}$	trial & error
8 ft Shaft, Max scour	$L_4 := 82.5\text{ft}$	trial & error

Deflections of Equivalent Beams:

8.5 ft Shaft, No scour $\delta_1 := \delta_f(P, L_1, E, I_{8.5})$

8.5 ft Shaft, Max scour	$\delta_2 := \delta_f(P, L_2, E, I_{8.5})$
8 ft Shaft, No scour	$\delta_3 := \delta_f(P, L_3, E, I_8)$
8 ft Shaft, Max scour	$\delta_4 := \delta_f(P, L_4, E, I_8)$

Equivalent Point of Fixity Determination



Equivalent Point of Fixity -- Elevation:

8.5 ft Shaft, No scour	$EL_{f1} := EL_h - L_1$	$EL_{f1} = 330.5\,ft$
8.5 ft Shaft, Max scour	$EL_{f2} := EL_h - L_2$	$EL_{f2} = 323.5\,ft$
8 ft Shaft, No scour	$EL_{f3} := EL_h - L_3$	$EL_{f3} = 330\,ft$
8 ft Shaft, Max scour	$EL_{f4} := EL_h - L_4$	$EL_{f4} = 323\,ft$

Bed Rock Elevation: $EL_R := 332ft$

PARSONS BRINCKERHOFF
COMPUTATION SHEET

Geotechnical & Tunneling Division

BY: M. Du	**DATE:** 12/26/2007	**PROJECT:** East End Bridge			
CHECKED BY: S. Malhotra	**DATE:** 12/27/07	**PAGE**	1	**OF**	4
SUBJECT: Drilled Shaft Equivalent Point of Fixity Calculations for Pier 5					
Preliminary Design					

Purpose

To estimate equivalent point of fixity for drilled shaft, based on LPILE results, for Pier 5.

References

- Results of lateral load analyses with LPILE
- Elastic beam deflection equations

Index

- Calculations (pp. 1 ~ 4)

Purpose

To estimate equivalent point of fixity for drilled shaft, based on LPILE results.

Functions to define beam end deflections under concentrated loads and bending moments:

Cantilever beam under concentrated shear at cantilever end:

$$\delta_p(P, L, E, I) := \frac{P \cdot L^3}{3E \cdot I}$$

Cantilever beam under concentrated bending moment at cantilever end:

$$\delta_M(M, L, E, I) := \frac{M \cdot L^2}{2E \cdot I}$$

Cantilever beam under combined concentrated shear and bending moment:

$$\delta_c(P, M, L, E, I) := \frac{P \cdot L^3}{3E \cdot I} + \frac{M \cdot L^2}{2E \cdot I}$$

Beam with one end fixed and one end free to translate but not rotate:

$$\delta_f(P, L, E, I) := \frac{P \cdot L^3}{12E \cdot I}$$

Drilled Shaft Cross-Section Properties

Young's Modulus: $E := 4074281\,psi$

8.5 ft Diameter Shaft: $I_{8.5} := 5431065\,in^4$

8 ft Diameter Shaft: $I_8 := 4356263\,in^4$

Shaft Head Conditions and Loading

Head Fixity: Free

Shaft Head Elevation: $EL_h := 414.8\,ft$

Shear:

$$P := \begin{pmatrix} 500 \\ 400 \\ 300 \end{pmatrix} kip$$

Moment:

$$M := \begin{pmatrix} 13000 \\ 12000 \\ 11000 \end{pmatrix} kip \cdot ft$$

Deflection computed by LPILE:

Run 1
8.5 ft Shaft
No scour,

$$\Delta_1 := \begin{pmatrix} 0.742 \\ 0.644 \\ 0.545 \end{pmatrix} in$$

Run 2
8.5 ft Shaft
Max scour,

$$\Delta_2 := \begin{pmatrix} 0.772 \\ 0.672 \\ 0.572 \end{pmatrix} in$$

Run 3
8 ft Shaft
No scour,

$$\Delta_3 := \begin{pmatrix} 0.91 \\ 0.789 \\ 0.669 \end{pmatrix} in$$

Run 4
8 ft Shaft
Max scour,

$$\Delta_4 := \begin{pmatrix} 0.951 \\ 0.827 \\ 0.704 \end{pmatrix} in$$

Equivalent Beam Lengths:

8.5 ft Shaft, No scour	$L_1 := 29ft$	trial & error
8.5 ft Shaft, Max scour	$L_2 := 29.5ft$	trial & error
8 ft Shaft, No scour	$L_3 := 28.7ft$	trial & error
8 ft Shaft, Max scour	$L_4 := 29.3ft$	trial & error

Deflections of Equivalent Beams:

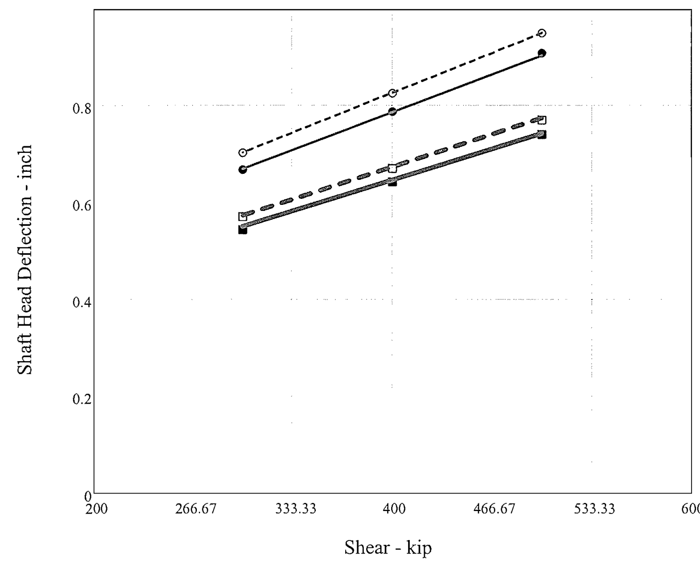
8.5 ft Shaft, No scour $\delta_1 := \delta_c(P, M, L_1, E, I_{8.5})$

8.5 ft Shaft, Max scour $\delta_2 := \delta_c(P, M, L_2, E, I_{8.5})$

8 ft Shaft, No scour $\delta_3 := \delta_c(P, M, L_3, E, I_8)$

8 ft Shaft, Max scour $\delta_4 := \delta_c(P, M, L_4, E, I_8)$

Equivalent Point of Fixity Determination



■■■ LPILE Deflections, 8.5 ft Shaft, No Scour
□□□ LPILE Deflections, 8.5 ft Shaft, Max Scour
●●● LPILE Deflections, 8 ft Shaft, No Scour
○○○ LPILE Deflections, 8 ft Shaft, Max Scour
Equivalent Beam Deflections, 8.5 ft Shaft, No Scour
Equivalent Beam Deflections, 8.5 ft Shaft, Max Scour
Equivalent Beam Deflections, 8 ft Shaft, No Scour
Equivalent Beam Deflections, 8 ft Shaft, Max Scour

Equivalent Point of Fixity – Elevation:

8.5 ft Shaft, No scour	$EL_{f1} := EL_h - L_1$	$EL_{f1} = 385.8\,ft$
8.5 ft Shaft, Max scour	$EL_{f2} := EL_h - L_2$	$EL_{f2} = 385.3\,ft$
8 ft Shaft, No scour	$EL_{f3} := EL_h - L_3$	$EL_{f3} = 386.1\,ft$
8 ft Shaft, Max scour	$EL_{f4} := EL_h - L_4$	$EL_{f4} = 385.5\,ft$
	Bed Rock Elevation:	$EL_R := 392\,ft$

APPENDIX H-4
CORRELATION OF SPT DATA

I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
	Depth of	Assumed	Vertical	SPT	SPT					Internal	Unit		Revised	
	Mid. Pt.	Estimated	Effective	N	N	Correction	Corrected	Relative		Angle of	Weight	Moisture	In-situ	Void
Soil	of Sample	Unit Weight	Stress	Value	Value	Factor	N-Value	Density	Unified Soil	Friction	Dry	Content	Unit Weight	Ratio
No.	(ft.)	(pcf)	(tsf)					(%)	Classification	(degrees)	(pcf)	(%)	(pcf)	
		γ_w	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e

NOTES:

C.	This spreadsheet has been designed such that an initial "Assumed Estimated Unit Weight" is placed into Column C.
E.	N_{80} is the blow count per foot as determined in the field using a automatic hammer.
F.	$N_{60} = (E_{AH}/60)\ N_{AH}$, where: E_{AH} = autohammer efficiency (80%): N_{AH} = blowcount from the autohammer, as referenced in (1)
	The autohammer efficiency is based on typical values of efficiencies (85 - 95) and actual testing preformed on FMSM hammers. SPT Analyzer equipment from
	Pile Dynamics Inc. was used to conduct the testing. An autohammer is more enegry efficient than a standard hammer.
	Hammer efficiency is a means of comparing the energy transferred from the hammer to the drill string during sampling.
G.	Correction Factor Based on 1/(square root of vertical effective stress). (Liao, S.C. and Whitman, R.V. 1985.
	"Overburden Correction Factors for SPT in Sand", JGED, ASCE, Vol. 112, No. 3, pp. 373-377; as referenced in (2).
	This correction factor is limited to vertical effective stresses greater than 0.25 tsf.
I.	Relative Density based on Tokimatsu, K. and Seed, H.B. 1988. "Evaluation of Settlements in Sands Due to Earthquake Shaking",
	JGED, ASCE, Vol. 113, No. 8, pp. 861-878; as referenced in (2).
J.	Classification based on field and laboratory data by FMSM.
K, L and O	Angle of Internal Friction (phi), Unit Weight Dry and Void Ratio based on NAVFAC 7.1 "Soil Mechanics", May 1982, page 7.1-149.
M.	Moisture content above the water table is based on laboratory testing of SPT samples by FMSM. Moisture contents below the water table are based upon correlations
	with limited testing by FMSM for Section 4 of the Louisville Bridges Project.
N.	In-situ unit weight is based on dry unit weight (L) times (1 + moisture content).
(1)	Goble, George, GRL Newsletter, December 1995 "SPT Improvements"
(2)	Seed and Harder, Volume 2 Memorial Symposium Proceedings, May 1990. "SPT Based Analysis of
	Cyclic Pore Pressure Generation and Undrained Residual Strength", pp. 361-362.

I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

		Depth of	Assumed	Vertical	SPT	SPT					Internal	Unit		Revised	
		Mid. Pt.	Estimated	Effective	N	N	Correction	Corrected	Relative		Angle of	Weight	Moisture	In-situ	Void
Sample		of Sample	Unit Weight	Stress	Value	Value	Factor	N-Value	Density	Unified Soil	Friction	Dry	Content	Unit Weight	Ratio
Interval		(ft.)	(pcf)	(tsf)					(%)	Classification	(degrees)	(pcf)	(%)	(pcf)	
			γ_w	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
					Input Required										
AC-1			water =	20.6	8/16/2007										
2.5 -	4.5	3.5	120	0.21	ST	NA	1.00	NA	NA	CL	NA	NA	19.1	NA	NA
5.0 -	7.0	6	120	0.36	ST	NA	1.00	NA	NA	CL	NA	NA	19.2	NA	NA
10.0 -	12.0	11	120	0.66	ST	NA	1.00	NA	NA	CL	NA	NA	23.4	NA	NA
15.0 -	17.0	16	120	0.96	ST	NA	1.00	NA	NA	CL	NA	NA	26.3	NA	NA
20.0 -	21.5	20.75	120	1.10	2	3	0.95	3	18	CL-ML	FALSE	NA	26.3	NA	NA
25.0 -	26.5	25.75	109	1.21	5	7	0.91	6	35	SM	30	94	16.0	109	0.78
30.0 -	31.5	30.75	113	1.34	11	15	0.86	13	52	SM	32	97	16.0	113	0.72
35.0 -	36.5	35.75	121	1.48	31	41	0.82	34	84	SM	35.5	104	16.0	121	0.61
40.0 -	41.5	40.75	110	1.60	9	12	0.79	10	44	SM	30.5	95	16.0	110	0.76
45.0 -	46.5	45.75	117	1.74	2	3	0.76	2	18	SW-SM	28.5	98	19.0	117	0.71
50.0	51.5	50.75	122	1.89	16	21	0.73	16	58	SW-SM	33.5	106	15.0	122	0.57
55.0 -	56.5	55.75	124	2.04	21	28	0.70	20	65	SW-SM	34.5	108	15.0	124	0.55
60.0 -	61.5	60.75	123	2.19	18	24	0.68	16	60	SW-SM	34	107	15.0	123	0.56
65.0 -	66.5	65.75	125	2.35	26	35	0.65	23	70	SW-SM	35	109	15.0	125	0.54
70.0 -	71.5	70.75	123	2.50	20	27	0.63	17	60	SW-SM	34	107	15.0	123	0.56
75.0 -	76.5	75.75	133	2.68	24	32	0.61	20	65	SW	35.5	115	16.0	133	0.45
80.0 -	81.5	80.75	132	2.86	24	32	0.59	19	63	SW	35	114	16.0	132	0.46
85.0 -	86.5	85.75	133	3.03	25	33	0.57	19	65	SW	35.5	115	16.0	133	0.45
90.0 -	91.5	90.75	133	3.21	26	35	0.56	19	65	SW	35.5	115	16.0	133	0.45
95.0 -	96.5	95.75	131	3.38	22	29	0.54	16	60	SW	35	114	16.0	132	0.46
100.0 -	100.3	100.15	140	3.55	50	67	0.53	35	86	SW	38.5	120.5	16.0	140	0.39

Project No: LX2005125
File Name: J:\2005proj\LX2005125 I-265 Bridge\Advance Contract\Analyses\Pier 1 SPT Correction

I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_t	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-2		water =	14.4	8/15/2007										
2.5 - 4.5	3.5	120	0.21	ST	NA	1.00	NA	NA	CL	NA	NA	18.2	NA	NA
5.0 - 7.0	6	120	0.36	ST	NA	1.00	NA	NA	CL	NA	NA	18.6	NA	NA
10.0 - 12.0	11	120	0.66	ST	NA	1.00	NA	NA	CL	NA	NA	19.9	NA	NA
15.0 - 16.5	15.75	120	0.80	2	3	1.00	3	18	CL	FALSE	NA	25.6	NA	NA
20.0 - 22.0	21	120	0.95	ST	NA	1.00	NA	NA	CL	NA	NA	23.6	NA	NA
25.0 - 26.5	25.75	104	1.05	2	3	0.98	3	18	SC	28	90	16.0	104	0.85
30.0 - 31.5	30.75	111	1.17	9	12	0.92	11	47	SC	31	96	16.0	111	0.74
35.0 - 36.5	35.75	127	1.33	23	31	0.87	27	75	SW-SM	36	110	15.0	127	0.52
40.0 - 41.5	40.75	121	1.48	11	15	0.82	12	52	SW-SM	33	105	15.0	121	0.59
45.0 - 46.5	45.75	122	1.63	14	19	0.78	15	56	SW-SM	33.5	106	15.0	122	0.57
50.0 - 51.5	50.75	124	1.78	19	25	0.75	19	65	SW-SM	34.5	108	15.0	124	0.55
55.0 - 56.5	55.75	125	1.94	25	33	0.72	24	73	SW-SM	35	109	15.0	125	0.54
60.0 - 61.5	60.75	116	2.07	22	29	0.69	20	67	SM	33.5	100	16.0	116	0.67
65.0 - 66.5	65.75	123	2.22	21	28	0.67	19	63	SP-SM	34	107	15.0	123	0.56
70.0 - 71.5	70.75	124	2.38	25	33	0.65	22	68	SP-SM	34.5	108	15.0	124	0.55
75.0 - 76.5	75.75	133	2.55	88	117	0.63	73	100	SP-SM	39	116	15.0	133	0.44
80.0 - 81.5	80.75	127	2.71	33	44	0.61	27	75	SP-SM	36	110	15.0	127	0.52
85.0 - 86.5	85.75	125	2.87	30	40	0.59	24	71	SW-SM	35	109	15.0	125	0.54
90.0 - 91.5	90.75	120	3.02	13	17	0.58	10	47	SW-SM	32	104	15.0	120	0.61
95.0 - 96.5	95.75	122	3.16	19	25	0.56	14	56	SW-SM	33.5	106	15.0	122	0.57
100.0 - 100.4	100.2	130	3.31	50	67	0.55	37	86	SW-SM	37	113	15.0	130	0.48

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_t	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-3		water =	12.0	8/24/2007										
2.5 - 4.5	3.5	120	0.21	ST	NA	1.00	NA	NA	CL	NA	NA	36.0	NA	NA
5.0 - 7.0	6	120	0.36	ST	NA	1.00	NA	NA	CL	NA	NA	36.1	NA	NA
10.0 - 12.0	11	120	0.66	ST	NA	1.00	NA	NA	CL	NA	NA	30.0	NA	NA
15.0 - 17.0	16	120	0.80	ST	NA	1.00	NA	NA	CL	NA	NA	24.0	NA	NA
20.0 - 21.5	20.75	120	0.94	2	3	1.00	3	18	CL	FALSE	NA	19.1	NA	NA
25.0 - 26.5	25.75	109	1.06	5	7	0.97	7	35	SM	30	94	16.0	109	0.78
30.0 - 31.5	30.75	117	1.19	18	24	0.92	22	70	SM	34	101	16.0	117	0.65
35.0 - 36.5	35.75	115	1.33	16	21	0.87	19	63	SM	33	99	16.0	115	0.69
40.0 - 41.5	40.75	121	1.47	12	16	0.82	13	53	SP-SM	33	105	15.0	121	0.59
45.0 - 46.5	45.75	123	1.62	17	23	0.78	18	60	SP-SM	34	107	15.0	123	0.56
50.0 - 51.5	50.75	123	1.77	17	23	0.75	17	60	SP-SM	34	107	15.0	123	0.56
55.0 - 56.5	55.75	121	1.92	13	17	0.72	13	52	SP-SM	33	105	15.0	121	0.59
60.0 - 61.5	60.75	128	2.08	36	48	0.69	33	84	SW-SM	36.5	111.5	15.0	128	0.5
65.0 - 66.5	65.75	127	2.25	30	40	0.67	27	75	SW-SM	36	110	15.0	127	0.52
70.0 - 71.5	70.75	125	2.40	30	40	0.65	26	74	SW-SM	35	109	15.0	125	0.54
75.0 - 76.5	75.75	127	2.56	35	47	0.62	29	79	SW-SM	36	110	15.0	127	0.52
80.0 - 81.5	80.75	122	2.71	18	24	0.61	15	56	SW-SM	33.5	106	15.0	122	0.57
85.0 - 86.5	85.75	123	2.86	22	29	0.59	17	60	SW-SM	34	107	15.0	123	0.56
90.0 - 91.5	90.75	131	3.03	22	29	0.57	17	60	SP	35	114	15.0	131	0.46
95.0 - 96.5	95.75	131	3.21	25	33	0.56	19	63	SP	35	114	15.0	131	0.46

I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_t	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-4		water =	0.0	6/15/2007	Bottom of River @		15.7							
15.7 - 17.2	16.45	124	0.02	16	21	1.00	21	68	SW-SM	34.5	108	15.0	124	0.55
21.7 - 23.2	22.45	125	0.21	18	24	1.00	24	73	SW-SM	35	109	15.0	125	0.54
26.7 - 28.2	27.45	123	0.36	13	17	1.00	17	60	SW-SM	34	107	15.0	123	0.56
31.7 - 33.2	32.45	129	0.53	10	13	1.00	13	53	SW	33.6	112	15.0	129	0.49
36.7 - 38.2	37.45	135	0.71	20	27	1.00	27	75	SW	37	117.5	15.0	135	0.42
41.7 - 43.2	42.45	137	0.90	25	33	1.00	33	84	SW	37.8	119	15.0	137	0.4
46.7 - 48.2	47.45	134	1.08	18	24	0.96	23	71	SP	36.2	116.5	15.0	134	0.43
51.7 - 53.2	52.45	132	1.25	18	24	0.89	22	68	SP	35.5	115	15.0	132	0.45
56.7 - 58.2	57.45	132	1.43	17	23	0.84	19	65	SP	35.5	115	15.0	132	0.45
61.7 - 63.2	62.45	135	1.61	15	20	0.79	16	58	GP-GM	34.8	117.5	15.0	135	0.42
66.7 68.2	67.45	148	1.82	65	87	0.74	64	100	GP-GM	41.6	129	15.0	148	0.29
71.7 - 73.2	72.45	131	1.99	48	64	0.71	45	93	SP-SM	38	114	15.0	131	0.47
76.7 - 78.2	77.45	127	2.15	34	45	0.68	31	79	SP-SM	36	110	15.0	127	0.52
81.7 - 83.2	82.45	131	2.33	52	69	0.66	46	93	SW-SM	38	114	15.0	131	0.47
86.7 - 87.4	87.05	122	2.46	50	67	0.64	43	91	SM	36.5	106	15.0	122	0.58
91.7 - 92.1	91.9	144	2.66	50	67	0.61	41	89	GM	39.3	125	15.0	144	0.34

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_t	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-5		water =	7.1	10/9/2007										
3.0 - 5.0	4	120	0.24	ST	NA	1.00	NA	NA	CL	NA	NA	9.2	NA	NA
5.0 - 7.0	6	120	0.36	ST	NA	1.00	NA	NA	SM	#N/A	#N/A	6.5	#N/A	#N/A
7.0 - 8.5	7.75	107	0.40	2	3	1.00	3	18	SM	28	90	19.0	107	0.85
10.0 - 11.5	10.75	107	0.47	2	3	1.00	3	18	SM	28	90	19.0	107	0.85
15.0 - 16.5	15.75	107	0.58	2	3	1.00	3	18	SM	28	90	19.0	107	0.85
20.0 - 21.5	20.75	135	0.76	18	24	1.00	24	73	SW	36.2	116.5	16.0	135	0.43
25.0 - 26.5	25.75	140	0.95	29	39	1.00	39	87	SW	38.5	120.5	16.0	140	0.39
30.0 - 31.5	30.75	132	1.13	14	19	0.94	18	60	SW	35	114	16.0	132	0.46
35.0 - 36.5	35.75	135	1.31	20	27	0.87	23	71	SW	36.2	116.5	16.0	135	0.43
40.0 - 41.5	40.75	136	1.49	25	33	0.82	27	77	SW	37	117.5	16.0	136	0.42
45.0 46.5	45.75	143	1.70	52	69	0.77	53	99	SW	40	123	16.0	143	0.36
50.0 - 51.5	50.75	125	1.85	24	32	0.73	24	71	SW-SM	35	109	15.0	125	0.54
55.0 - 56.5	55.75	125	2.01	24	32	0.71	23	70	SW-SM	35	109	15.0	125	0.54
60.0 - 61.5	60.75	125	2.17	25	33	0.68	23	70	SW-SM	35	109	15.0	125	0.54
65.0 - 66.5	65.75	122	2.32	16	21	0.66	14	56	SW-SM	33.5	106	15.0	122	0.57
70.0 - 71.5	70.75	124	2.47	25	33	0.64	21	68	SW-SM	34.5	108	15.0	124	0.55
75.0 - 76.5	75.75	125	2.63	27	36	0.62	22	70	SW-SM	35	109	15.0	125	0.54
80.0 - 81.5	80.75	125	2.79	28	37	0.60	22	70	SW-SM	35	109	15.0	125	0.54
85.0 - 86.5	85.75	125	2.94	30	40	0.58	23	71	SW-SM	35	109	15.0	125	0.54
90.0 - 91.5	90.75	124	3.10	27	36	0.57	21	67	SW-SM	34.5	108	15.0	124	0.55
95.0 - 96.5	95.75	124	3.25	27	36	0.55	20	67	SW-SM	34.5	108	15.0	124	0.55

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_w	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-6		water =	0.0	7/10/2007										
40.9 - 42.4	41.65	126	0.02	17	23	1.00	23	70	SW-SM	35	109	16.0	126	0.54
45.9 - 47.4	46.65	126	0.18	18	24	1.00	24	73	SW-SM	35	109	16.0	126	0.54
50.9 - 52.4	51.65	124	0.34	13	17	1.00	17	60	SW-SM	34	107	16.0	124	0.56
55.9 - 57.4	56.65	131	0.51	34	45	1.00	45	93	SP-SM	38	114	15.0	131	0.47
60.9 - 62.4	61.65	130	0.68	29	39	1.00	39	87	SP-SM	37	113	15.0	130	0.48
65.9 - 67.4	66.65	133	0.86	53	71	1.00	71	100	SP-SM	39	116	15.0	133	0.44
70.9 - 72.4	71.65	161	1.10	25	33	0.95	32	81	GW	41	140	15.0	161	0.2
75.9 - 77.4	76.65	164	1.36	39	52	0.86	45	93	GW	43	143	15.0	164	0.16
80.9 - 82.4	81.65	125	1.52	21	28	0.81	23	70	SP-SM	35	109	15.0	125	0.54
85.9 - 87.2	86.55	133	1.69	84	112	0.77	86	100	SP-SM	39	116	15.0	133	0.44

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_w	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-7		water =	0.0	7/3/2007										
40.5 - 42.0	41.25	119	0.02	7	9	1.00	9	44	SW-SM	31.5	103	16.0	119	0.62
45.5 - 47.0	46.25	126	0.18	19	25	1.00	25	74	SW-SM	35	109	16.0	126	0.54
50.5 - 52.0	51.25	125	0.34	15	20	1.00	20	67	SW-SM	34.5	108	16.0	125	0.55
55.5 - 57.0	56.25	129	0.51	25	33	1.00	33	84	SP-SM	36.5	111.5	16.0	129	0.5
60.5 - 62.0	61.25	126	0.67	19	25	1.00	25	74	SP-SM	35	109	16.0	126	0.54
65.5 - 67.0	66.25	169	0.93	52	69	1.00	69	100	GW	45	147	15.0	169	0.13
70.5 - 72.0	71.25	169	1.20	60	80	0.91	73	100	GW	45	147	15.0	169	0.13
75.5 - 77.0	76.25	161	1.45	31	41	0.83	34	84	GW	41	140	15.0	161	0.2
80.5 - 82.0	81.25	132	1.62	45	60	0.79	47	95	SP-SM	38.5	115	15.0	132	0.45
85.5 - 85.7	85.6	132	1.77	50	67	0.75	50	98	SP-SM	38.5	115	15.0	132	0.45
88.0 - 89.1	88.55	124	1.86	93	124	0.73	91	100	SM	38	108	15.0	124	0.55

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-8		water =	0.0	7/9/2007										
40.7 - 42.2	41.45	128	0.02	6	8	1.00	8	41	SP	32.3	110	16.0	128	0.52
45.7 - 47.2	46.45	135	0.21	17	23	1.00	23	70	SP	36.2	116.5	16.0	135	0.43
50.7 - 52.2	51.45	132	0.38	12	16	1.00	16	60	SP	35	114	16.0	132	0.46
55.7 - 57.2	56.45	128	0.55	25	33	1.00	33	84	SW-SM	36.5	111.5	15.0	128	0.5
60.7 - 62.2	61.45	132	0.72	38	51	1.00	51	98	SW-SM	38.5	115	15.0	132	0.45
65.7 - 67.2	66.45	133	0.90	63	84	1.00	84	100	SP-SM	39	116	15.0	133	0.44
70.7 - 72.2	71.45	133	1.08	65	87	0.96	84	100	SP-SM	39	116	15.0	133	0.44
75.7 - 77.2	76.45	131	1.25	36	48	0.90	43	91	SP-SM	38	114	15.0	131	0.47
80.7 - 82.2	81.45	128	1.41	31	41	0.84	35	84	SP-SM	36.5	111.5	15.0	128	0.5
85.7 - 86.5	86.1	132	1.57	50	67	0.80	53	99	SP-SM	38.5	115	15.0	132	0.45

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-9		water =	0.0	7/9/2007										
40.6 - 42.1	41.35	132	0.03	13	17	1.00	17	60	SW	35	114	16.0	132	0.46
45.6 - 47.1	46.35	133	0.20	16	21	1.00	21	68	SW	35.5	115	16.0	133	0.45
50.6 - 52.1	51.35	132	0.38	12	16	1.00	16	60	SW	35	114	16.0	132	0.46
55.6 - 57.1	56.35	127	0.54	22	29	1.00	29	79	SW-SM	36	110	15.0	127	0.52
60.6 - 62.1	61.35	127	0.70	21	28	1.00	28	77	SW-SM	36	110	15.0	127	0.52
65.6 - 67.1	66.35	169	0.97	78	104	1.00	104	100	GW-GM	45	147	15.0	169	0.13
70.6 - 72.1	71.35	169	1.23	73	97	0.90	88	100	GW-GM	45	147	15.0	169	0.13
75.6 - 77.1	76.35	133	1.41	50	67	0.84	56	100	SP-SM	39	116	15.0	133	0.44
80.6 - 82.1	81.35	127	1.57	26	35	0.80	28	77	SP-SM	36	110	15.0	127	0.52
85.6 - 86.2	85.9	132	1.73	50	67	0.76	51	98	SP-SM	38.5	115	15.0	132	0.45

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_w	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-10		water =	0.0	7/6/2007	Bottom of River @		44.0							
44.0 - 45.5	44.75	139	0.03	9	12	1.00	12	52	GP	34.5	120	16.0	139	120
49.0 - 50.5	49.75	120	0.17	4	5	1.00	5	32	GP	31.5	115	16.0	133	115
54.0 - 55.5	54.75	139	0.36	9	12	1.00	12	52	GP	34.5	120	16.0	139	120
59.0 - 60.5	59.75	131	0.54	32	43	1.00	43	91	SP-SM	38	114	15.0	131	0.47
64.0 - 65.5	64.75	133	0.71	53	71	1.00	71	100	SP-SM	39	116	15.0	133	0.44
69.0 - 70.5	69.75	127	0.87	22	29	1.00	29	79	SW-SM	36	110	15.0	127	0.52
74.0 - 75.5	74.75	127	1.03	20	27	0.98	26	75	SW-SM	36	110	15.0	127	0.52
79.0 - 80.5	79.75	123	1.19	13	17	0.92	16	58	SW-SM	33.5	106	16.0	123	0.57
84.0 - 84.4	84.2	133	1.34	50	67	0.86	58	100	SW-SM	39	116	15.0	133	0.44

Project No: LX2005125
File Name: J:\2005proj\LX2005125 I-265 Bridge\Advance Contract\Analyses\Pier 4 SPT Correction

I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_w	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-11		water =	0.0	6/28/2007	Bottom of River @		39.1							
41.7 - 43.2	42.45	151	0.15	9	12	1.00	12	52	GW	35.8	130	16.0	151	0.28
46.7 - 48.2	47.45	148	0.36	7	9	1.00	9	44	GW	34	127.5	16.0	148	0.31
51.7 - 53.2	52.45	152	0.59	11	15	1.00	15	56	GW	36.8	131	16.0	152	0.27
56.7 - 58.2	57.45	128	0.75	22	29	1.00	29	79	SP-SM	36	110	16.0	128	0.52
61.7 - 63.2	62.45	133	0.93	100	133	1.00	133	100	SP-SM	39	116	15.0	133	0.44
66.7 - 68.2	67.45	132	1.10	39	52	0.95	50	97	SP-SM	38.5	115	15.0	132	0.45
71.7 - 73.2	72.45	133	1.28	99	132	0.88	117	100	SP-SM	39	116	15.0	133	0.44
76.7 - 78.2	77.45	128	1.44	30	40	0.83	33	84	SP-SM	36.5	111.5	15.0	128	0.5
81.6 - 82.0	81.8	147	1.63	50	67	0.78	52	98	GM	41	127.5	15.0	147	0.31

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_w	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-12		water =	0.0	7/2/2007	Bottom of River @		40.5							
40.5 - 42.0	41.25	144	0.03	4	5	1.00	5	32	GW	32.2	124	16.0	144	0.34
45.5 - 47.0	46.25	154	0.26	12	16	1.00	16	60	GW	37.5	133	16.0	154	0.25
50.5 - 52.0	51.25	160	0.50	21	28	1.00	28	77	GW	40	138	16.0	160	0.21
55.5 - 57.0	56.25	133	0.68	16	21	1.00	21	68	SP	35.5	115	16.0	133	0.45
60.5 - 62.0	61.25	133	0.86	59	79	1.00	79	100	SP-SM	39	116	15.0	133	0.44
65.5 - 67.0	66.25	125	1.02	15	20	0.99	20	65	SP-SM	34.5	108	16.0	125	0.55
70.5 - 72.0	71.25	130	1.19	31	41	0.92	38	87	SP-SM	37	113	15.0	130	0.48
75.5 - 77.0	76.25	141	1.38	30	40	0.85	34	84	GP-GM	38.5	123	15.0	141	0.35
80.5 - 81.0	80.75	147	1.57	50	67	0.80	53	99	GP-GM	41	127.5	15.0	147	0.31

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_w	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-13		water =	0.0	6/21/2007	Bottom of River @		38.7							
42.0 - 43.5	42.75	151	0.18	9	12	1.00	12	52	GW	35.8	130	16.0	151	0.28
47.0 - 48.5	47.75	154	0.41	12	16	1.00	16	60	GW	37.5	133	16.0	154	0.25
52.0 - 53.5	52.75	125	0.57	16	21	1.00	21	68	SP-SM	34.5	108	16.0	125	0.55
57.0 - 58.5	57.75	133	0.74	55	73	1.00	73	100	SP-SM	39	116	15.0	133	0.44
62.0 - 63.5	62.75	133	0.92	55	73	1.00	73	100	SP-SM	39	116	15.0	133	0.44
67.0 - 68.5	67.75	124	1.08	15	20	0.96	19	65	SP-SM	34.5	108	15.0	124	0.55
72.0 - 73.5	72.75	127	1.24	23	31	0.90	28	77	SP-SM	36	110	15.0	127	0.52
77.0 - 78.5	77.75	148	1.45	95	127	0.83	105	100	GP-GM	41.6	129	15.0	148	0.29
82.0 - 83.0	82.5	147	1.65	50	67	0.78	52	98	GP-GM	41	127.5	15.0	147	0.31

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval		Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
			γ_e	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
					Input Required										
AC-15			water =	0.0	6/21/2007	Bottom of River @		4.2							
4.2 -	5.7	4.95	101	0.01	6	8	1.00	8	41	ML	30	87	16.0	101	0.93
6.7 -	8.2	7.45	99	0.06	4	5	1.00	5	32	ML	29	85	16.0	99	0.97
10.0 -	11.5	10.75	148	0.20	52	69	1.00	69	100	GP-GM	41.6	129	15.0	148	0.29
12.7 -	14.2	13.45	148	0.32	43	57	1.00	57	100	GP-GM	41.6	129	15.0	148	0.29
15.5 -	17.0	16.25	148	0.44	63	84	1.00	84	100	GP-GM	41.6	129	15.0	148	0.29
21.7 -	23.2	22.45	148	0.70	42	56	1.00	56	100	GP-GM	41.6	129	15.0	148	0.29
26.7 -	27.2	26.95	148	0.90	50	67	1.00	67	100	GP-GM	41.6	129	15.0	148	0.29

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval		Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
			γ_e	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
					Input Required										
AC-17			water =	DRY	9/26/2007										
0.0 -	1.5	0.75	139	0.05	36	48	1.00	48	95	GC	41	127.5	9.3	139	0.31

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_w	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-20		water =	DRY	9/26/2007										
0.0 - 1.5	0.75	150	0.06	49	65	1.00	65	100	GC	41.6	129	16.5	150	0.29

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_w	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
AC-23		water =	DRY	9/27/2007										
2.5 - 3.6	3.05	135	0.21	68	91	1.00	91	100	GC	41.6	129	5.0	135	0.29

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_e	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-26		water =	DRY	9/26/2007										
0.0 - 1.5	0.75	120	0.05	14	19	1.00	19	63	CL	FALSE	NA	10.2	NA	NA
5.0 - 6.5	5.75	120	0.35	41	55	1.00	55	99	CL	FALSE	NA	10.9	NA	NA

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I-265 OVER OHIO RIVER
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_e	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
AC-27		water =	DRY	9/26/2007										
0.0 - 1.2	0.6	120	0.04	60	80	1.00	80	100	CL	FALSE	NA	6.5	NA	NA

APPENDIX H-5
ROCK STABILITY ANALYSIS

PB AMERICAS
COMPUTATION SHEET

Subject	East End Bridge Rock Slope Stability in IN Abutment	Page	1 of 1
		Made by	KHC
		Date	12/25/07
		Checked by	EMD
		Date	12/27/07

FILENAME = I:\34676D 08.04 Geotechnical (EEB - Louisville, KY)\EEB work by KHC\[EEB FS Summary Table 122707.xls]FS Summary

[Summary of Factor of Safety (FS*)]

Case I: Sliding along Clay Seam

[H=13 ft for clay seam (EL. ~478)]

Stage	1. Preconstruction Case	2. After Construction	3. After Construction and Seismic Condition**
beta = 1 deg	12.447	2.132	1.378
beta = 3 deg	9.666	1.865	1.284
beta = 6 deg	8.132	1.606	1.186

Case II: Sliding along Upper Interface of Limestone and Shale Beds

[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

Stage	1. Preconstruction Case	2. After Construction	3. After Construction and Seismic Condition**
beta = 1 deg	8.85	2.169	1.358
beta = 3 deg	7.087	1.892	1.268
beta = 6 deg	5.985	1.626	1.175

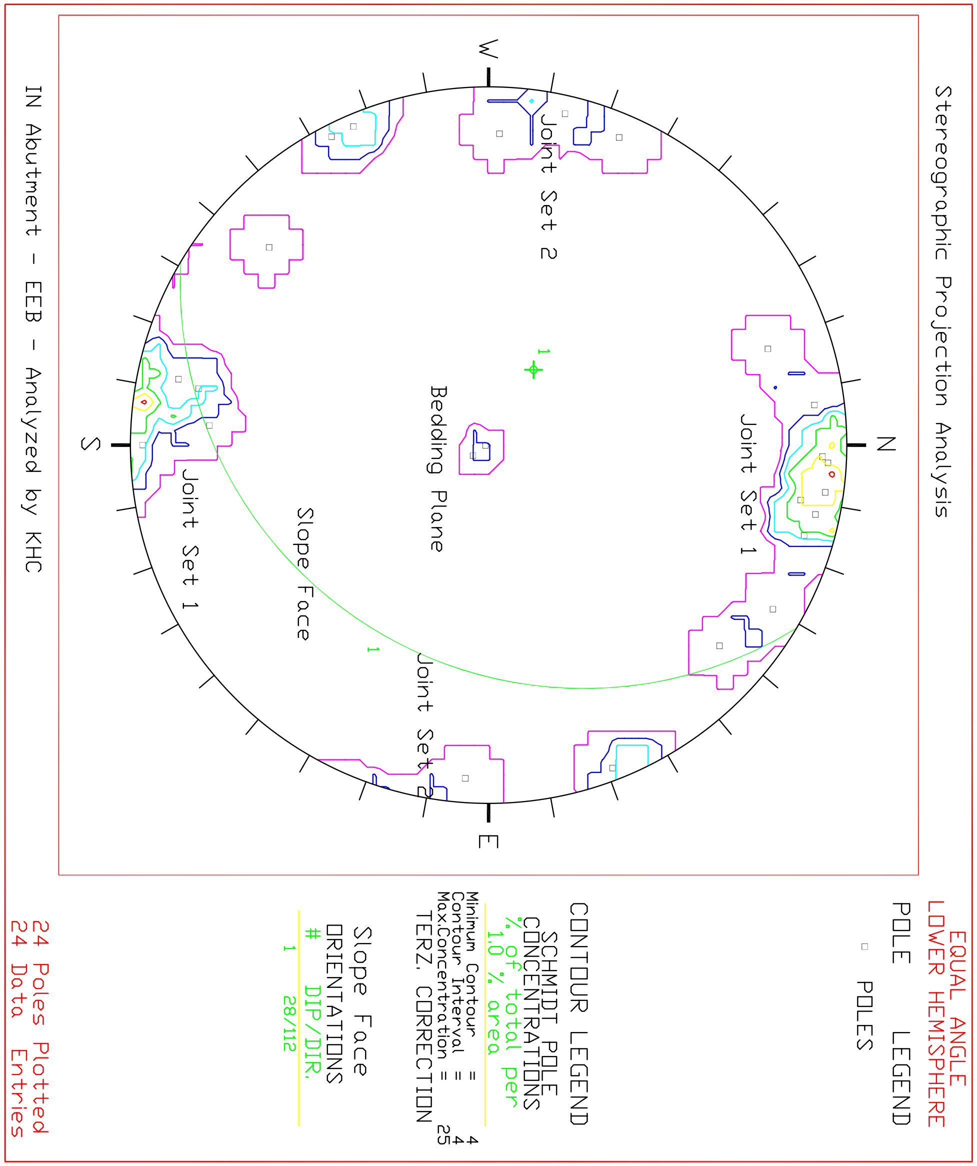
Case III: Sliding along Lower Interface of Limestone and Shale

[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

Stage	1. Preconstruction Case	2. After Construction	3. After Construction and Seismic Condition**
beta = 1 deg	3.83	2.02	1.183
beta = 3 deg	3.283	1.767	1.114
beta = 6 deg	2.86	1.529	1.046

* Factor of safety was calculated using the method by Kliche (1999).

** 0.1g of horizontal acceleration was applied to the factor of safety calculation.
The 0.1g of horizontal acceleration corresponds the 2/3 of 0.15g of the peak acceleration obtained from the site-specific response spectra as illustrated in Figure 6.

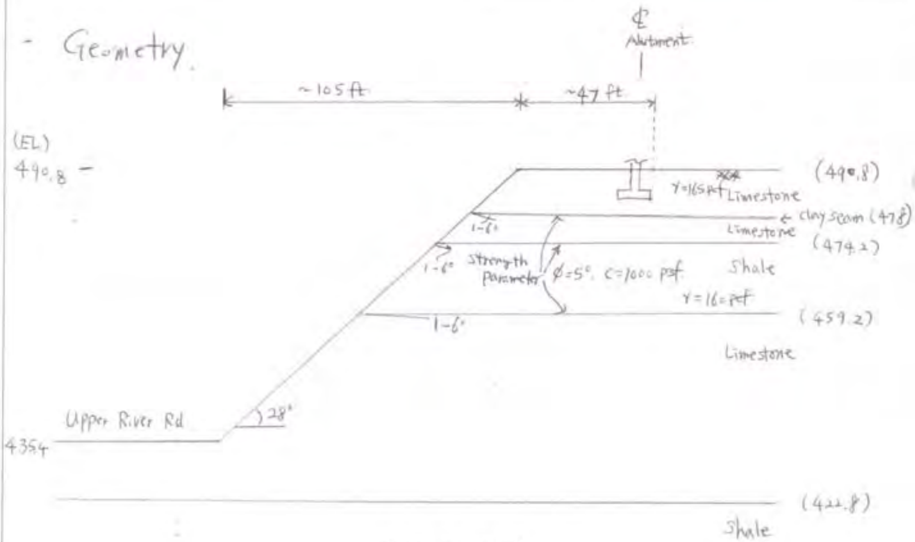


PB Americas, Inc.
COMPUTATION SHEET

Subject: Rock slope stability Analysis EEB

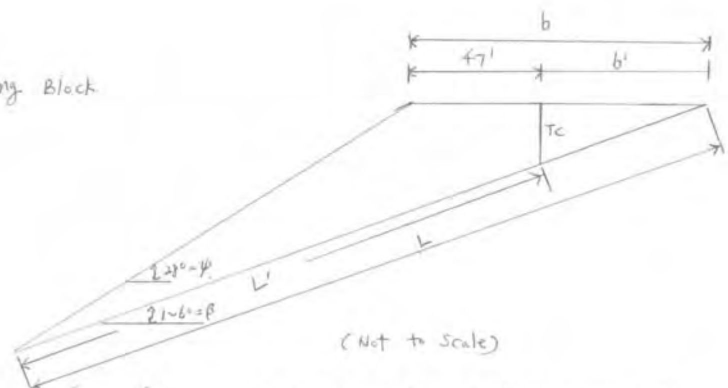
Page 1 of 2
Made by KHC
Date 12/26/07
Checked by EMD
Date 12/27/07

- Geometry



(Not to Scale)

- Sliding Block



(Not to Scale)

* See Fs calculation sheet for abbreviations

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COMPUTATION SHEET

Subject: Rock slope stability Analysis EEB

Page 2 of 2
Made by KHC
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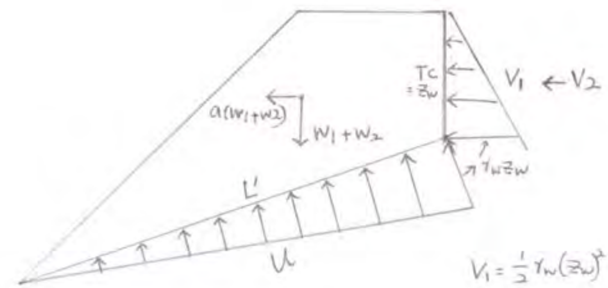
- Acting forces



$V_1 = \frac{1}{2}\gamma_w(Z_w)^2$

$U = \frac{1}{2}\gamma_w L' Z_w$

* See Fs calculation sheet for abbreviations

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

1. Preconstruction Case (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	720.3	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	673.3	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	744.9	ft	
Tension crack depth (TC) =	11.8	ft	
Height of water in vertical joint (zw) =	11.8	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	71.5		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	119,685.6	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	26,203.7	lb
Hor water pressure in tension crack (V1) =	4,309.6	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	79,630.8	lb
Driving Force (DF) =	6,397.8	lb
FS = (RF/DF) =	**12.447**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

1. Preconstruction Case (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	223.6	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	176.6	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	248.4	ft	
Tension crack depth (TC) =	9.3	ft	
Height of water in vertical joint (zw) =	9.3	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	71.5		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	104,964.7	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	20,660.9	lb
Hor water pressure in tension crack (V1) =	2,672.7	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	78,898.3	lb
Driving Force (DF) =	8,162.5	lb
FS = (RF/DF) =	**9.666**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

1. Preconstruction Case (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	99.2	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	52.2	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	124.4	ft	
Tension crack depth (TC) =	5.5	ft	
Height of water in vertical joint (zw) =	5.5	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	71.8		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	82,770.9	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	12,306.7	lb
Hor water pressure in tension crack (V1) =	940.5	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	77,959.6	lb
Driving Force (DF) =	9,587.3	lb
FS = (RF/DF) =	**8.132**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

2. After Construction (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	720.3	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	673.3	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	744.9	ft	
Tension crack depth (TC) =	11.8	ft	
Height of water in vertical joint (zw) =	11.8	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	71.5		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	119,685.6	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	26,203.7	lb
Hor water pressure in tension crack (V1) =	4,309.6	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	89,070.7	lb
Driving Force (DF) =	41,786.3	lb
FS = (RF/DF) =	**2.132**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

2. After Construction (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	223.6	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	176.6	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	248.4	ft	
Tension crack depth (TC) =	9.3	ft	
Height of water in vertical joint (zw) =	9.3	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	71.5		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	104,964.7	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	20,660.9	lb
Hor water pressure in tension crack (V1) =	2,672.7	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	88,224.4	lb
Driving Force (DF) =	47,295.0	lb
FS = (RF/DF) =	**1.865**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

2. After Construction (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	99.2	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	52.2	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	124.4	ft	
Tension crack depth (TC) =	5.5	ft	
Height of water in vertical joint (zw) =	5.5	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	71.8		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	82,770.9	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	12,306.7	lb
Hor water pressure in tension crack (V1) =	940.5	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	87,093.7	lb
Driving Force (DF) =	54,245.1	lb
FS = (RF/DF) =	**1.606**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

3. After Construction and Seismic Condition (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest =	47.0	ft	(assumed)
(Tension crack (vertical joint) behind spread footing)			
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry				
Length along the top of slope to the intersection of the bedding plane (b)				
b =	720.3	ft		
Hor dist from tension crack to intersection of bedding and surface (b')				
b' =	673.3	ft		
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)				
L =	744.9	ft		
Tension crack depth (TC) =	11.8	ft		
Height of water in vertical joint (zw) =	11.8	ft	zw/TC =	1.0
Length of bedding plane from daylight in the face to tension crack (L')				
L' =	71.5			

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	119,685.6	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	26,203.7	lb
Hor water pressure in tension crack (V1) =	4,309.6	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	89,035.8	lb
Driving Force (DF) =	64,601.3	lb
FS = (RF/DF) =	**1.378**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

3. After Construction and Seismic Condition (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest =	47.0	ft	(assumed)
(Tension crack (vertical joint) behind spread footing)			
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry				
Length along the top of slope to the intersection of the bedding plane (b)				
b =	223.6	ft		
Hor dist from tension crack to intersection of bedding and surface (b')				
b' =	176.6	ft		
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)				
L =	248.4	ft		
Tension crack depth (TC) =	9.3	ft		
Height of water in vertical joint (zw) =	9.3	ft	zw/TC =	1.0
Length of bedding plane from daylight in the face to tension crack (L')				
L' =	71.5			

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	104,964.7	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	20,660.9	lb
Hor water pressure in tension crack (V1) =	2,672.7	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	88,126.7	lb
Driving Force (DF) =	68,612.2	lb
FS = (RF/DF) =	**1.284**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case I: Sliding along Clay Seam
[H=13 ft for clay seam (EL. ~478)]

3. After Construction and Seismic Condition (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	13.0	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	99.2	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	52.2	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	124.4	ft	
Tension crack depth (TC) =	5.5	ft	
Height of water in vertical joint (zw) =	5.5	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	71.8		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	82,770.9	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	12,306.7	lb
Hor water pressure in tension crack (V1) =	940.5	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	86,918.8	lb
Driving Force (DF) =	73,267.4	lb
FS = (RF/DF) =	**1.186**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds
[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

1. Preconstruction Case (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	919.8	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	872.8	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	951.2	ft	
Tension crack depth (TC) =	15.2	ft	
Height of water in vertical joint (zw) =	15.2	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	78.2		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	162,678.1	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	37,185.3	lb
Hor water pressure in tension crack (V1) =	7,241.4	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	89,197.9	lb
Driving Force (DF) =	10,079.4	lb
FS = (RF/DF) =	**8.850**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds

[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

1. Preconstruction Case	(beta = 3 deg)		
<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	285.5	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	238.5	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	317.2	ft	
Tension crack depth (TC) =	12.5	ft	
Height of water in vertical joint (zw) =	12.5	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	78.3		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	145,035.2	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	30,549.3	lb
Hor water pressure in tension crack (V1) =	4,875.5	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	88,303.9	lb
Driving Force (DF) =	12,459.4	lb
FS = (RF/DF) =	**7.087**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds

[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

1. Preconstruction Case	(beta = 6 deg)		
<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	126.7	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	79.7	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	158.8	ft	
Tension crack depth (TC) =	8.4	ft	
Height of water in vertical joint (zw) =	8.4	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	78.7		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	118,435.8	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	20,560.7	lb
Hor water pressure in tension crack (V1) =	2,190.3	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	87,137.1	lb
Driving Force (DF) =	14,558.3	lb
FS = (RF/DF) =	**5.985**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds
[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

2. After Construction (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	
<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b) b =	919.8	ft	
Hor dist from tension crack to intersection of bedding and surface (b') b' =	872.8	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L) L =	951.2	ft	
Tension crack depth (TC) =	15.2	ft	
Height of water in vertical joint (zw) =	15.2	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L') L' =	78.2		
<Step 3> Calculation of weight of unstable block			
Weight (W1) =	162,678.1	lb	
<Step 4> Calculaiton of water pressure			
Uplift pressure along failure plane (U) =	37,185.3	lb	
Hor water pressure in tension crack (V1) =	7,241.4	lb	
<Step 5> Calculation FS			
Resisting Force (RF) =	98,637.9	lb	
Driving Force (DF) =	45,467.9	lb	
FS = (RF/DF) =	**2.169**		

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds
[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

2. After Construction (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	
<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b) b =	285.5	ft	
Hor dist from tension crack to intersection of bedding and surface (b') b' =	238.5	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L) L =	317.2	ft	
Tension crack depth (TC) =	12.5	ft	
Height of water in vertical joint (zw) =	12.5	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L') L' =	78.3		
<Step 3> Calculation of weight of unstable block			
Weight (W1) =	145,035.2	lb	
<Step 4> Calculaiton of water pressure			
Uplift pressure along failure plane (U) =	30,549.3	lb	
Hor water pressure in tension crack (V1) =	4,875.5	lb	
<Step 5> Calculation FS			
Resisting Force (RF) =	97,630.0	lb	
Driving Force (DF) =	51,591.9	lb	
FS = (RF/DF) =	**1.892**		

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds
[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

2. After Construction (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	126.7	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	79.7	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	158.8	ft	
Tension crack depth (TC) =	8.4	ft	
Height of water in vertical joint (zw) =	8.4	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	78.7		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	118,435.8	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	20,560.7	lb
Hor water pressure in tension crack (V1) =	2,190.3	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	96,271.2	lb
Driving Force (DF) =	59,216.1	lb
FS = (RF/DF) =	**1.626**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds
[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

3. After Construction and Seismic Condition (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	919.8	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	872.8	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	951.2	ft	
Tension crack depth (TC) =	15.2	ft	
Height of water in vertical joint (zw) =	15.2	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	78.2		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	162,678.1	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	37,185.3	lb
Hor water pressure in tension crack (V1) =	7,241.4	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	98,596.5	lb
Driving Force (DF) =	72,581.6	lb
FS = (RF/DF) =	**1.358**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds
[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

3. After Construction and Seismic Condition (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry

Length along the top of slope to the intersection of the bedding plane (b)
b = 285.5 ft

Hor dist from tension crack to intersection of bedding and surface (b')
b' = 238.5 ft

Length of bedding plane from daylight in the slope face to intersection to ground surface (L)
L = 317.2 ft

Tension crack depth (TC) = 12.5 ft

Height of water in vertical joint (zw) = 12.5 ft zw/TC = 1.0

Length of bedding plane from daylight in the face to tension crack (L')
L' = 78.3

<Step 3> Calculation of weight of unstable block

Weight (W1) = 145,035.2 lb

<Step 4> Calculaiton of water pressure

Uplift pressure along failure plane (U) = 30,549.3 lb

Hor water pressure in tension crack (V1) = 4,875.5 lb

<Step 5> Calculation FS

Resisting Force (RF) = 97,513.9 lb

Driving Force (DF) = 76,910.7 lb

FS = (RF/DF) = **1.268**

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case II: Sliding along Upper Interface of Limestone and Shale Beds
[H=16.6 ft for the upper interface of limestone and shale (EL. 474.2)]

3. After Construction and Seismic Condition (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	16.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry

Length along the top of slope to the intersection of the bedding plane (b)
b = 126.7 ft

Hor dist from tension crack to intersection of bedding and surface (b')
b' = 79.7 ft

Length of bedding plane from daylight in the slope face to intersection to ground surface (L)
L = 158.8 ft

Tension crack depth (TC) = 8.4 ft

Height of water in vertical joint (zw) = 8.4 ft zw/TC = 1.0

Length of bedding plane from daylight in the face to tension crack (L')
L' = 78.7

<Step 3> Calculation of weight of unstable block

Weight (W1) = 118,435.8 lb

<Step 4> Calculaiton of water pressure

Uplift pressure along failure plane (U) = 20,560.7 lb

Hor water pressure in tension crack (V1) = 2,190.3 lb

<Step 5> Calculation FS

Resisting Force (RF) = 96,063.7 lb

Driving Force (DF) = 81,785.3 lb

FS = (RF/DF) = **1.175**

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

1. Preconstruction Case (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest =	47.0	ft	(assumed)
(Tension crack (vertical joint) behind spread footing)			
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	1750.9	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	1703.9	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	1810.6	ft	
Tension crack depth (TC) =	29.7	ft	
Height of water in vertical joint (zw) =	29.7	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	106.4		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	383,682.3	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	98,778.5	lb
Hor water pressure in tension crack (V1) =	27,599.5	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	131,325.8	lb
Driving Force (DF) =	34,291.5	lb
FS = (RF/DF) =	**3.830**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

1. Preconstruction Case (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest =	47.0	ft	(assumed)
(Tension crack (vertical joint) behind spread footing)			
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	543.5	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	496.5	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	603.8	ft	
Tension crack depth (TC) =	26.0	ft	
Height of water in vertical joint (zw) =	26.0	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	106.6		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	351,018.2	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	86,529.1	lb
Hor water pressure in tension crack (V1) =	21,127.2	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	129,578.0	lb
Driving Force (DF) =	39,469.1	lb
FS = (RF/DF) =	**3.283**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

1. Preconstruction Case (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	0.0	lb	
Horizontal force by surcharge (V2) =	0.0	lb	
<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b) b =	241.2	ft	
Hor dist from tension crack to intersection of bedding and surface (b') b' =	194.2	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L) L =	302.3	ft	
Tension crack depth (TC) =	20.4	ft	
Height of water in vertical joint (zw) =	20.4	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L') L' =	107.0		
<Step 3> Calculation of weight of unstable block			
Weight (W1) =	301,772.2	lb	
<Step 4> Calculaiton of water pressure			
Uplift pressure along failure plane (U) =	68,159.9	lb	
Hor water pressure in tension crack (V1) =	13,001.6	lb	
<Step 5> Calculation FS			
Resisting Force (RF) =	127,192.1	lb	
Driving Force (DF) =	44,474.1	lb	
FS = (RF/DF) =	**2.860**		

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

2. After Construction (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	
<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b) b =	1750.9	ft	
Hor dist from tension crack to intersection of bedding and surface (b') b' =	1703.9	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L) L =	1810.6	ft	
Tension crack depth (TC) =	29.7	ft	
Height of water in vertical joint (zw) =	29.7	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L') L' =	106.4		
<Step 3> Calculation of weight of unstable block			
Weight (W1) =	383,682.3	lb	
<Step 4> Calculaiton of water pressure			
Uplift pressure along failure plane (U) =	98,778.5	lb	
Hor water pressure in tension crack (V1) =	27,599.5	lb	
<Step 5> Calculation FS			
Resisting Force (RF) =	140,765.7	lb	
Driving Force (DF) =	69,680.0	lb	
FS = (RF/DF) =	**2.020**		

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

2. After Construction (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry

Length along the top of slope to the intersection of the bedding plane (b) b =	543.5	ft	
Hor dist from tension crack to intersection of bedding and surface (b') b' =	496.5	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L) L =	603.8	ft	
Tension crack depth (TC) =	26.0	ft	
Height of water in vertical joint (zw) =	26.0	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L') L' =	106.6		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	351,018.2	lb
<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	86,529.1	lb
Hor water pressure in tension crack (V1) =	21,127.2	lb
<Step 5> Calculation FS		
Resisting Force (RF) =	138,904.1	lb
Driving Force (DF) =	78,601.7	lb
FS = (RF/DF) =	**1.767**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

2. After Construction (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.00	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry

Length along the top of slope to the intersection of the bedding plane (b) b =	241.2	ft	
Hor dist from tension crack to intersection of bedding and surface (b') b' =	194.2	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L) L =	302.3	ft	
Tension crack depth (TC) =	20.4	ft	
Height of water in vertical joint (zw) =	20.4	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L') L' =	107.0		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	301,772.2	lb
<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	68,159.9	lb
Hor water pressure in tension crack (V1) =	13,001.6	lb
<Step 5> Calculation FS		
Resisting Force (RF) =	136,326.3	lb
Driving Force (DF) =	89,132.0	lb
FS = (RF/DF) =	**1.529**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

3. After Construction and Seismic Condition (beta = 1 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	1.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	1750.9	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	1703.9	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	1810.6	ft	
Tension crack depth (TC) =	29.7	ft	
Height of water in vertical joint (zw) =	29.7	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	106.4		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	383,682.3	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	98,778.5	lb
Hor water pressure in tension crack (V1) =	27,599.5	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	140,690.5	lb
Driving Force (DF) =	118,890.7	lb
FS = (RF/DF) =	**1.183**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

3. After Construction and Seismic Condition (beta = 3 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	3.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest = (Tension crack (vertical joint) behind spread footing)	47.0	ft	(assumed)
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry			
Length along the top of slope to the intersection of the bedding plane (b)			
b =	543.5	ft	
Hor dist from tension crack to intersection of bedding and surface (b')			
b' =	496.5	ft	
Length of bedding plane from daylight in the slope face to intersection to ground surface (L)			
L =	603.8	ft	
Tension crack depth (TC) =	26.0	ft	
Height of water in vertical joint (zw) =	26.0	ft	zw/TC = 1.0
Length of bedding plane from daylight in the face to tension crack (L')			
L' =	106.6		

<Step 3> Calculation of weight of unstable block		
Weight (W1) =	351,018.2	lb

<Step 4> Calculaiton of water pressure		
Uplift pressure along failure plane (U) =	86,529.1	lb
Hor water pressure in tension crack (V1) =	21,127.2	lb

<Step 5> Calculation FS		
Resisting Force (RF) =	138,693.7	lb
Driving Force (DF) =	124,490.5	lb
FS = (RF/DF) =	**1.114**	

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

Case III: Sliding along Lower Interface of Limestone and Shale Beds
[H=31.6 ft for the lower interface of limestone and shale (EL. 459.2)]

3. After Construction and Seismic Condition (beta = 6 deg)

<Step 1> INPUT Data			
Dip of failure plane (beta) =	6.0	degree	
Dip of Slope face (psi) =	28.0	degree	
Cohesion (c) =	1000.0	psf	
Friction angle (phi) =	5.0	degree	
Unit Weight of Rock =	165.0	pcf	
Unit Weight of Water =	62.4	pcf	
Height of Slope (H)* =	31.6	ft	
Hor dist between tension crack and slope crest =	47.0	ft	(assumed)
(Tension crack (vertical joint) behind spread footing)			
Seismic acceleration (a) =	0.10	g	
Vertical weight by surcharge (W2) =	108,500.0	lb	
Horizontal force by surcharge (V2) =	33,500.0	lb	

<Step 2> Calculation of geometry

Length along the top of slope to the intersection of the bedding plane (b)

b = 241.2 ft

Hor dist from tension crack to intersection of bedding and surface (b')

b' = 194.2 ft

Length of bedding plane from daylight in the slope face to intersection to ground surface (L)

L = 302.3 ft

Tension crack depth (TC) = 20.4 ft

Height of water in vertical joint (zw) = 20.4 ft   zw/TC = 1.0

Length of bedding plane from daylight in the face to tension crack (L')

L' = 107.0

<Step 3> Calculation of weight of unstable block

Weight (W1) = 301,772.2 lb

<Step 4> Calculaiton of water pressure

Uplift pressure along failure plane (U) = 68,159.9 lb

Hor water pressure in tension crack (V1) = 13,001.6 lb

<Step 5> Calculation FS

Resisting Force (RF) = 135,951.1 lb

Driving Force (DF) = 129,934.4 lb

FS = (RF/DF) = **1.046**

Resisting force = cL' + [(W1+W2) (cos(beta) - a*sin(beta)) - (V1+V2) sin(beta) - U] tan(phi)

Driving force = (W1+W2) (sin(beta) + a*cos(beta)) + (V1+V2) cos(beta)

PB AMERICAS COMPUTATION SHEET

Page	1 of 1
Made by	KHC
Date	12/25/07
Checked by	EMD
Date	12/27/07

Subject: East End Bridge
Rock Slope Stability in IN Abutment

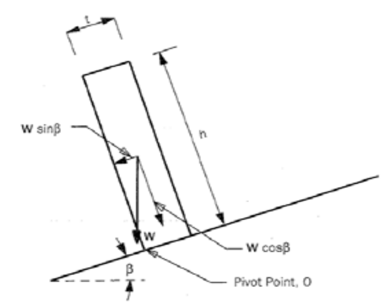
FILENAME = I:\34676D 08.04 Geotechnical (EEB - Louisville, KY)\EEB work by KHC\[EEB Toppling 122707.xls]Toppling

[Simplistic Factor of Safety Analysis for Toppling (Joint Set 2 is involved)]

FS=(t/h)/tan(beta)

t=width of block =	5	ft	0.5 to > 20 ft, typically 5 ft*
h=height of block-=	31.6	ft	(490.8-459.2)
beta=dip of bedding plane =	1	deg	1-6 deg
FS =	**9.06**		
beta=dip of bedding plane =	3	deg	1-6 deg
FS =	**3.02**		
beta=dip of bedding plane =	6	deg	1-6 deg
FS =	**1.51**		

[Sketch for General Model for Toppling Failure]
(Kliche, 1999)



APPENDIX H-6
ABUTMENT ANALYSIS

PB AMERICAS
COMPUTATION SHEET

Subject	East End Bridge	Page	1 of 2
	Abutment Foundation on Rock	Made by	EMD
		Date	12/22/07
		Checked by	KHC
		Date	4/23/08

FILENAME = I:\34676D 08.04 Geotechnical (EEB - Louisville, KY)\Calcs\[Abutment_Fndn__Analysis__EEB 5-2-08.xls]Abutment foundation

Abutment Foundation Analysis

Wall No. : EEB Indiana Abutment
Soil Boring : AC-20, 23, 26; See also Fig 5g General Soil and Bedrock Profile

Foundation Parameters: Refer to structural calculations, dmb 9-19-07 (attached)

	W=	108.46	klf
	B=	20	ft
	Df=	6	ft
	P/A=	5.14	ksf
	M/S=	5.03	ksf
Maximum applied pressure at toe	P/A+M/S=	10.16	ksf
	P/A-M/S=	0.11	ksf
Applied horizontal force	Fh=	33.54	klf

Rock Properties: Equivalent c and ϕ from Figure 5f General Soil and Rock Profile

Cohesion (c) =	2880	psf
Angle of Internal Friction (ϕ)=	22	deg.
Unit Weight (γ) =	160	pcf

Nominal sliding resistance parameters:
Adhesion $c\alpha$= 2016 psf
Concrete-Rock Friction δ= 14.7

Determine Factor of Safety against Sliding ($FS_{sliding}$)

$$FS_{sliding} = \frac{\sum P_{resisting}}{\sum P_{driving}} = \frac{W\tan\delta + c_a B}{P_a} =$$ 2.05 | > 1.5 | OK

Determine Factor of Safety against Overturning ($FS_{overturning}$)

Forces from structural calculations, dmb 9-19-07 (attached)

Before superstructure is in place:

		H	V	arm to toe ftg	resisting M	driving M
DL-super			52.2 k/ft	10.00'	neglect	
LL			4.8 k/ft	10.00'	neglect	
DL-sub	bk wall		3.4 k/ft	16.25'	55 k-ft/ft	
	pave seat		0.2 k/ft	17.25'	3 k-ft/ft	
	cap		6.8 k/ft	12.50'	84 k-ft/ft	
	stem		9.0 k/ft	10.00'	90 k-ft/ft	
	earth-heel		14.4 k/ft	16.00'	230 k-ft/ft	
	earth-toe		5.8 k/ft	9.00'	52 k-ft/ft	
	ftg		12.0 k/ft	10.00'	120 k-ft/ft	
	earth	30.4 k/ft		13.00'		395 k-ft/ft
	surcharge	3.1 k/ft		19.50'		neglect
		33.5 k/ft	108.5 k/ft		634 k-ft/'	395 k-ft/'

Fsoverturning=$M_{resisting}/M_{driving}$ = 1.60 | > 1.5 | OK

After superstructure is in place:

		H	V	arm to toe ftg	resisting M	driving M
DL-super			52.2 k/ft	10.00'	522 k-ft/ft	
LL			4.8 k/ft	10.00'	48 k-ft/ft	
DL-sub	bk wall		3.4 k/ft	16.25'	55 k-ft/ft	
	pave seat		0.2 k/ft	17.25'	3 k-ft/ft	
	cap		6.8 k/ft	12.50'	84 k-ft/ft	
	stem		9.0 k/ft	10.00'	90 k-ft/ft	
	earth-heel		14.4 k/ft	16.00'	230 k-ft/ft	
	earth-toe		5.8 k/ft	9.00'	52 k-ft/ft	
	ftg		12.0 k/ft	10.00'	120 k-ft/ft	
	earth	30.4 k/ft		13.00'		395 k-ft/ft
	surcharge	3.1 k/ft		19.50'		61 k-ft/ft
		33.5 k/ft	108.5 k/ft		1204 k-ft/'	456 k-ft/'

Fsoverturning=$M_{resisting}/M_{driving}$ = 2.64 | > 1.5 | OK

Determine Factor of Safety against Bearing Capacity Failure ($FS_{bearing}$)

1) Calculate a net allowable bearing capacity (q_{net})

Input Bearing Capacity Factors for ϕ=22°

Nc=	20.7
Nq=	9.5
Nγ=	6.9

$q_{net} = cN_c + \gamma Df(Nq-1) + 1/2BN\gamma$ = 67.85 ksf

FS for q_{all} 2.50

$q_{all} = q_{net}/FS$= 27.14 ksf

2) Compare maximum applied pressure at toe to net allowable bearing capacity
10.16 ksf | << 27.14 ksf | OK

3) Calculate FS against bearing capacity failure

$$FS_{bearing} = \frac{q_{net}}{\sigma_v} =$$ 6.67 | >> 2.5 | OK

4) Consider a reduction to allowable bearing pressure for settlement considerations
Rock formation contains clay seams.
Consider experience-based limit on bearing pressure to control foundation settlement.
Limit allow. bearing to 20 ksf
Resistance factor 0.45
Equiv. nominal resistance 44 ksf for use in LRFD analyses.

Therefore,
Based on sliding, overturning, and bearing capacity the abutment is STABLE
The heel of the footing needs to be extended in final design.
Final design analyses to be performed by AASHTO LRFD Methods.

PB AMERICAS
COMPUTATION SHEET

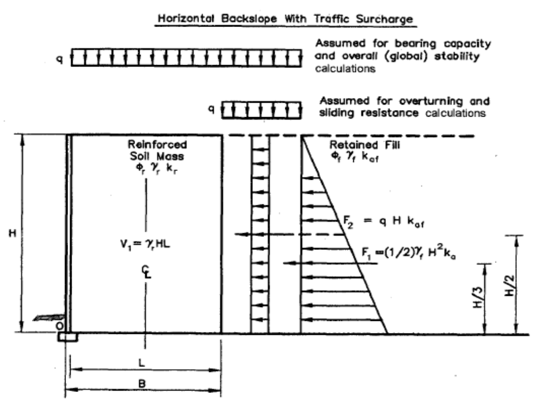
Page	1	of	4
Made by			KHC
Date	2/28/2008 rev 5/2/08		
Checked by			EMD
Date	4/23/2008 rev 5-2-08		

Subject East End Bridge
MSE Wall Analysis

FILENAME = I:\34676D 08.04 Geotechnical (EEB - Louisville, KY)\Calcs\[MSE Wall Analysis_EEB-LRFD 5-2-08.xls]LRFD B=0.75H

MSE Wall Analysis by LRFD

Geometry



q (traffic surcharge) = 0 psf (h_{eq} = 0 ft)
H (Wall Height) = 37.0 feet (Max. Height = 521 - 484 = 37)
* Passive pressure in front of the wall was neglected.

Soil/Rock Properties

	Reinforced	Retained	Foundation*	
Cohesion (c) =	0	0	2,880	psf
Angle of Internal Friction (ϕ)=	34	32	22	
Unit Weight (γ) =	125	125	160	pcf

* Rock

Unfactored Loads

		V (lb/ft)	M arm (ft)	M_O (lb/ft-ft)
P_{EV} =		128,344	13.88	1,780,770
P_{LSV} = qxL =		0	13.88	0

Ka =		0.307		
		H (lb/ft)	M arm (ft)	M_O (lb/ft-ft)
P_{EH} =		26,290	12.33	324,241
P_{LSH} = "P2 in ASD" =		0	18.5	0

Load Factors (NHI Course No. 130082A Table 6.2.9)

Group	r_{EV}	r_{EH}	r_{LS}
Strength I-a	1.00	1.50	1.75
Strength I-b	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(Strength 1-a and 1-b are used for minimum and maximum vertical load factors, respectively. AASHTO Table 3.4.1-2)

Factored Loads (lb/ft)

Group	P_{EV}	P_{LSV}	V_{total}
Unfactored	128,344	0	128,344
Strength I-a	128,344	0	128,344
Strength I-b	173,264	0	173,264
Service I	128,344	0	128,344

(lb/ft)

Group	P_{EH}	P_{LSH}	H_{total}
Unfactored	26,290	0	26,290
Strength I-a	39,435	0	39,435
Strength I-b	39,435	0	39,435
Service I	26,290	0	26,290

[Stability of MSE Wall]

Check Eccentricity (e)

B = 0.75(H)	27.8 ft	Min. = 0.7H
W =	128,344 lb/ft	

$$e = \frac{B}{2} - X_0 = \frac{B}{2} - \frac{\sum M_{EV} - \sum M_{htotal}}{\sum P_{EV}}$$

(No surcharge for M_{EV}, but surcharge for M_{htotal})

Group	P_{EV}	M_{EV}	M_{htotal}	X_0	e	$q_{uniform}$
Unfactored	128,344	1,780,770	324,241	11.35	2.53	5,655
Strength I-a	128,344	1,780,770	486,361	10.09	3.79	6,363
Strength I-b	173,264	2,404,039	486,361	11.07	2.81	7,827
Service I	128,344	1,780,770	324,241	11.35	2.53	5,655

e_{max} = B/4 = 6.94 Therefore $e < e_{max}$ **OK**
e *(Strength 1a)* = 3.79

(For comparison, FS by ASD = e_{max} = B/6 = 4.63 $e < e_{max}$ **OK, too**

$$FS_{overturning} = \frac{\sum M_{EV}}{\sum M_{htotal}} = 5.5 \quad > 2.0$$ **OK, too**

Determine Factor of Safety against Sliding ($FS_{sliding}$)

$$R_R = \varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep}$$

R_{ep} = 0 *(Neglect the passive resistance)*

φ_τ = 0.9 (AASHTO Table 10.5.5.2.2-1)

(No surcharge for P_{EV})

ϕ_f =	22 deg
C_a =	0 psf

(Consider only friction, conservatively neglect cohesion)

(ϕ_f = lesser of ϕ_r (reinforced fill) or ϕ_f (foundation soil))

$R_R = \varphi R_N = \varphi_\tau (P_{EV} \tan\phi_f + C_a B) =$ 46,669 > H_{total} = 39,435 lb/ft **OK**

(P_{EV} from Strength I-a) *(H_{total} from Strength I-a)*

(For comparison, FS by ASD =

$$FS_{sliding} = \frac{\sum P_{resisting}}{\sum P_{driving}} = \frac{R_\tau}{H_{total}} =$$ 1.97 > 1.5 **OK, too**

Determine Factor of Safety against Bearing Capacity Failure ($FS_{bearing}$)

$$q_R = \varphi_b \, q_n$$

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma \; B N_{\gamma m} C_{w\gamma}$$

- For the drained case only: *(Consider only friction, conservatively neglect cohesion)*

q_n (q_{net}) = $0.5B^*\gamma N_r + \gamma D_f (N_q - 1)$ =

18,053 psf

[If water table level is above footing base, reduce q_{net} by 50%.]

Water factor (0.5 if water, 1 if not) = 1

q_n (q_{net}) = 18,053 psf

B* =	20.17	ft
D_f =	6	ft
N_q =	16.88	for ϕ=22
N_q =	7.82	
N_r =	7.13	
(AASHTO Table 10.6.3.1.2a-1)		

Group	V_{total}	M_{vtotal}	M_{htotal}	X_0	e	$q_{uniform}$
Unfactored	128,344	1,780,770	324,241	11.35	2.53	5,655
Strength I-a	128,344	1,780,770	486,361	10.09	3.79	6,363
Strength I-b	173,264	2,404,039	486,361	11.07	2.81	7,827
Service I	128,344	1,780,770	324,241	11.35	2.53	5,655

φ_b = 0.45 (AASHTO Table 10.5.5.2.2-1)

$q_R = \varphi_b \, q_n$ = 8,124 > $q_{uniform}$ = 7,827 psf **OK**

($q_{uniform}$ from Strength 1b)

(For comparison, FS by ASD $FS_{bearing} = \frac{q_n}{q_{max}} =$ 2.52 psf > 2.0 **OK, too**

$$q_{max} = \frac{V_{total}}{B}\left(1 + \frac{6e}{B}\right) =$$ 7,151 psf

Therefore, the MSE wall is **STABLE** based upon the above LRFD analyses.

Revised Results of Supplemental Geotechnical Work

Indiana Abutment
I-265 Over the Ohio River
LSIORB, Section 5, Phase 4
Jefferson County, Kentucky
Item No. 5-118.00

Stantec Consulting Services Inc.
One Team. Infinite Solutions
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Prepared for:
Parsons Brinckerhoff, Inc.
Lexington, Kentucky

March 2, 2011

Stantec

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March 2, 2011 let_022_175565125

Mr. Steve Slade, PE, PLS
Parsons Brinckerhoff, Inc.
2333 Alumni Park Plaza, Suite 330
Lexington, Kentucky 40517

Re: Revised Results of Supplemental Geotechnical Work
Indiana Abutment
I-265 Over the Ohio River
LSIORB, Section 5, Phase 4
Jefferson County, Kentucky
Item No. 5-118.00

Dear Mr. Slade:

Submitted herein are the results of the supplemental geotechnical work for the Indiana abutment. The initial geotechnical engineering report was submitted May 12, 2008. This supplemental exploration at the Indiana abutment is to obtain more data regarding the presence and conditions of clay seams and orientation of bedding planes for evaluation of rock slope stability under the abutment loads. Stantec Consulting Services Inc. (Stantec) mobilized to the site and performed the fieldwork the week of August 16, 2010. The drilling and field work was conducted in accordance with the supplemental boring plan dated June 16, 2010.

Three rock core borings (AC-29, AC-30 and AC-31) were advanced using split tube barrels to the approximate bottom of hole elevation of 455 feet. The subsurface materials were visually described by the field representative in general accordance with the KYTC Geotechnical Manual. The boring logs are attached and summarized in Table 1.

Table 1. Summary of Borings

Hole No.	Station and Offset	Surface Elev.	Top of Rock Elev.	Bottom of Hole Elev.
AC-29	212+36, 44.0’ Lt.	494.2	491.3	454.2
AC-30	212+47, 13.0’ Lt.	496.2	492.8	454.6
AC-31	212+67, 40.0’ Lt.	498.2	486.9	455.1

Observation wells were installed in each core boring. They were installed to evaluate potential groundwater within the zone containing clay and soft shale seams and layers. The wells typically incorporated a 1-inch schedule 80 polyvinyl chloride (PVC) pipe with a 10-slot screen of varying length wrapped in a sand pack. A bentonite seal installed in the boring annulus created a seal above the monitored bedrock zone. The sand pack installed below the bentonite seal allowed a free exchange of water from the bedrock zone to the PVC screen. Because these wells were installed in rock core borings where water was introduced as part of the coring process, compressed nitrogen was used to evacuate (blow) water from the piezometers. All of the water could not be removed from the piezometer/core boring using this method. As such, the resulting water level readings may not be indicative of the actual groundwater surface. The observation wells monitored for a period of six months and end in February 2011. The water level readings obtained are attached.

During the field work, a geologist collected supplemental data relative to strike and dip of the near-horizontal bedding planes exposed in nearby rock cuts/rock quarries near the proposed abutment. Strike and dip measurements were taken at 22 locations using a Brunton compass and are included as an attachment to this letter. Also included in the attachment are stereonets developed from the field data using the computer program RockWorks98 developed by Rockware. An average dip of 3.1 degrees was calculated. The adjustment for magnetic declination (Utica, IN) used was 4 degrees 20 minutes or 4.33 degrees.

Table 2. Average Strike and Dip Results

	Dip Angle (degrees)	**Dip Direction (Azimuth)**	**Strike**
Unadjusted	3.1	276 (N84W)	N6E
Adjusted*		272 (N88W)	N2E

*Adjusted for magnetic declination (Utica, IN [4.33 degrees west]).

During the drilling process, selected samples of rock core were selected for potential laboratory testing. Identified samples generally consisted of clay partings, seams and layers of various thicknesses, in addition to shale layers. These samples were wrapped in cellophane, aluminum foil and then waxed to preserve the as-drilled condition. Samples were selected for direct shear and unconfined compression testing. Results of the laboratory testing are presented in the following tables.

Table 3. Results of Direct Shear Testing

Hole No.	Test Location Depth (Elevation) (feet)	Normal Stress (psi)	Peak Shear Stress (psi)	Post Peak Stress (psi)	Sample Material
AC-29	14.4 (479.8)	13.0	*	*	clay seam
AC-29	33.4 (460.8	32.5	17.2	12.4	weathered shale/clay seam
AC-30	17.3 (478.9)	16.8	13.6	9.6	clay seam
AC-30	36.3 (459.9)	35.3	24.0	21.8	weathered shale
AC-31	16.9 (481.3)	16.4	10.7	9.7	clay seam
AC-31	23.3 (474.9)	22.7	12.6	9.2	limestone/clay seam

*After completion of the test, the specimen was found to contain an ±¾ inch rock fragment, which may have affected the test. No apparent peak or post peak was observed.

Table 4. Results of Unconfined Compression Testing

Hole No.	Test Depth feet (Elevation)	Compressive Strength (tsf)	Sample Material
AC-29	25.3 – 25.7 (468.9 – 468.5)	68	weathered shale
AC-29	33.5 – 33.8 (460.7 – 460.4)	17	weathered shale
AC-30	35.7 – 36.1 (460.5 – 460.0)	64	weathered shale
AC-31	27.6 – 27.9 (470.6 – 470.3)	8	weathered shale
AC-31	35.3 – 35.7 (462.9 – 462.5)	23	weathered shale
AC-31	35.9 – 36.3 (462.3 – 461.9)	14	weathered shale

The unconfined compressive strength varied from 8 tsf to 68 tsf.

Stantec appreciates the opportunity to provide these services to you. If you need further assistance, please contact our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

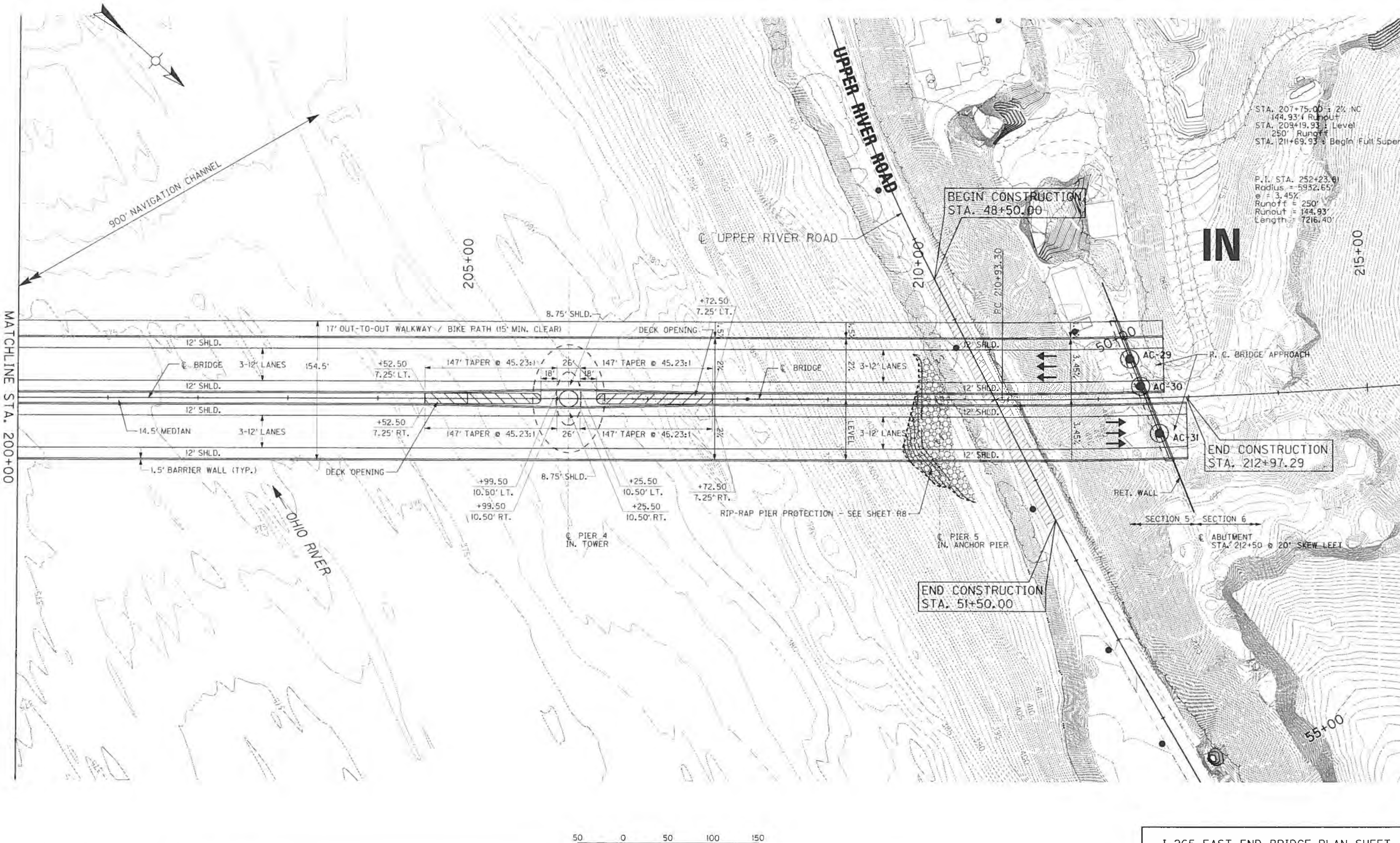
Donald Blanton, PE
Project Manager

/rdr

Boring Logs and
Boring Plan

DESIGN SPEED = 70 MPH

COUNTY OF	ITEM NO.	SHEET NO.
JEFFERSON	5-745.00	R5



CHECKED BY ____ DATE ____
APPROVED BY ____ DATE ____

E-SHEET NAME:

175565125/DRAFTING/LX2005125/CLIENT FILES/06-08-10/R5.DGN

I-265 EAST END BRIDGE PLAN SHEET
STA. 200+00 TO STA. 212+50

Drilling Firm: Stantec
For: Division of Structural Design
Geotechnical Branch

DRILLER'S SUBSURFACE LOG

Project ID: ***S-055-2008*** Item Number: ***05-118.00***	***Clark (Indiana) - I-265*** ***Ohio River***	Project Type: ***Structure*** Project Manager: _

Hole Number ***AC-29***	Immediate Water Depth ***19.8 (08/18/10)***	Start Date ***08/16/2010***	Hole Type ***NQ-3 core***
Surface Elevation ***494.2'***	Static Water Depth ***NA***	End Date ***08/16/2010***	Rig_Number ***CME 45 (811)***
Total Depth ***40.0'***	Driller ***D. Jessie***	Latitude(83) ***38.345011***	
Location ***212+36.00 44.0' Lt.***	Geologist ***Ben Halada (Eng)***	Longitude(83) ***-85.646206***	

Lithology: Elevation	Lithology: Depth	Description (Overburden / Rock Core)	Sample No. / Std/Ky RQD	Depth (ft) / Run (ft)	Rec. (ft) / Rec (ft)	SPT Blows / Rec (%)	Sample Type / SDI (JS)	Remarks
491.3	2.9	Medium dense, gray to brown, moist, clayey gravel with organics (gravel 1/2" to 1", with boulders 6" to 12").						(Begin Core)
			84 / 84	3.8	3.8	100		6.7
			80 / 80	10.0	9.9	99		16.7
472.0	22.2	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded).	77 / 70	10.0	10.0	100		26.7
460.2	34.0	Dark gray shale, (fine grained, soft to moderately hard, laminated to thin bedded).	79 / 64	10.1	10.1	100		36.8
454.2	40.0	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded).	100 / 100	3.2	3.2	100		40.0
		(Bottom of Hole 40.0') (Refusal @ 2.9)						

Remarks: **Shale (dark gray) with some clay (light brown) @ 9.4-10.4 clay seam (light brown), Water stained from 14.1 to 15.2 @ 14.1-14.6 healed vertical fracture @ 14.8-16.7 water stained @ 16.8-17.3 Sandstone (light brown), some clay in joint, soft @ 19.9-20 trace clay (<0.02') @ 20.9 clay seam (gray) @ 23-23.1 clay seam (gray, <0.05') @ 23.9 clay seam (dark gray, soft, <0.05') @ 24.9 clay seam @ 25.8-26.1 clay seam @ 29.3-29.4 clay seam (<0.1') @ 29.9 clay seam (<0.1') @ 30.7 shale / clay seam (weathered)**

Drilling Firm: Stantec
For: Division of Structural Design
Geotechnical Branch

DRILLER'S SUBSURFACE LOG

Project ID: S-055-2008 Item Number: 05-118.00	Clark (Indiana) - I-265 Ohio River	Project Type: Structure Project Manager: -

Hole Number AC-29	Immediate Water Depth 19.8 (08/18/10)	Start Date 08/16/2010	Hole Type NQ-3 core
Surface Elevation 494.2'	Static Water Depth NA	End Date 08/16/2010	Rig_Number CME 45 (811)
Total Depth 40.0'	Driller D. Jessie	Latitude(83) 38.345011	
Location 212+36.00 44.0' Lt.	Geologist Ben Halada (Eng)	Longitude(83) -85.646206	

Lithology: Elevation	Lithology: Depth	Description (Overburden / Rock Core)	Sample No. / Std/Ky RQD	Depth (ft) / Run (ft)	Rec. (ft) / Rec (ft)	SPT Blows / Rec (%)	Sample Type / SDI (JS)	Remarks
55 60		Observation well set. 1" PVC with a 10 foot screen (30 ft to 40 ft). Sand pack to ± 2 feet above screen. Remaining backfilled with pellets. 2X2 pad install with flushmount cover. The water depth recorded is the depth to water before attempting to evacuate the water from the piezometer.						@ 32.9-34 water stained @ 33.3-33.4 water stained @ 33.6-34.6 healed vertical fracture @ 38.7-38.8
65								
70								
75								
80								
85								
90								
95								
100								

Drilling Firm: Stantec
For: Division of Structural Design
Geotechnical Branch

DRILLER'S SUBSURFACE LOG

Project ID: S-055-2008 Item Number: 05-118.00	Clark (Indiana) - I-265 Ohio River	Project Type: Structure Project Manager: _

Hole Number AC-30	Immediate Water Depth 20.8 (08/18/10)	Start Date 08/17/2010	Hole Type NQ-3 core
Surface Elevation 496.2'	Static Water Depth NA	End Date 08/17/2010	Rig_Number CME 45 (811)
Total Depth 41.6'	Driller D. Jessie	Latitude(83) 38.345092	
Location 212+47.00 13.0' Lt.	Geologist Ben Halada (Eng)	Longitude(83) -85.646154	

Lithology: Elevation	Lithology: Depth	Description (Overburden / Rock Core)	Sample No. / Std/Ky RQD	Depth (ft) / Run (ft)	Rec. (ft) / Rec (ft)	SPT Blows / Rec (%)	Sample Type / SDI (JS)	Remarks
492.8	3.4	Medium stiff, brown, moist, clay with gravel (gravel is 1/2" to 1" with boulders 3" to 5").						(Begin Core)
481.6	14.6	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded, with broken joints and shale streaks).	78 / 78	4.1	4.1	100		7.5
			50 / 50	10.0	7.4	74		water stained @ 10.2-10.5 clay seam (<0.05') @ 10.4 water stained (<0.1') @ 10.7 water stained @ 11.7-11.8
480.6	15.6	(VOID).						
475.7	20.5	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded, with broken joints and shale streaks).	20 / 20	2.0	1.5	75		17.5 Clay, brown, 0.3' recovery, rest washed away @ 15.6-17.5
474.9	21.3	(VOID).						19.5 sporadic rod drops, solution features and vuggy @ 17.5-24 blocked off @ 19.5
471.1	25.1	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded, with broken joints and shale streaks).	47 / 40	10.0	6.8	68		core water return lost @ 20.5 clay seam (<0.05') @ 28
459.8	36.4	Dark gray shale, (fine grained, soft to moderately hard, laminated to thin bedded).	92 / 78	10.0	10.0	100		29.5 water stained @ 35-35.6 water stained @ 35.9-37 clay seam(<0.1') @ 36.3
454.6	41.6	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded, with broken joints and shale streaks).	90 / 90	2.1	2.1	100		39.5 water stained @ 38.6-39.5 stylolitic fracture, healed @ 41.4
		(Bottom of Hole 41.6') (Refusal @ 3.4)						41.6

DRILLER'S SUBSURFACE LOG

Project ID: S-055-2008 Item Number: 05-118.00	Clark (Indiana) - I-265 Ohio River	Project Type: Structure Project Manager: _

Hole Number AC-30	Immediate Water Depth 20.8 (08/18/10)	Start Date 08/17/2010	Hole Type NQ-3 core
Surface Elevation 496.2'	Static Water Depth NA	End Date 08/17/2010	Rig_Number CME 45 (811)
Total Depth 41.6'	Driller D. Jessie	Latitude(83) 38.345092	
Location 212+47.00 13.0' Lt.	Geologist Ben Halada (Eng)	Longitude(83) -85.646154	

Lithology: Elevation	Lithology: Depth	Description (Overburden / Rock Core)	Sample No. / Std/Ky RQD	Depth (ft) / Run (ft)	Rec. (ft) / Rec (ft)	SPT Blows / Rec (%)	Sample Type / SDI (JS)	Remarks
	55							55
	60	Observation well set. 1" PVC with a 25 foot screen (16.6 ft to 41.6 ft). Sand pack to ±2 feet above screen. Remaining backfilled with pellets. 2X2 pad install with flushmount cover. The water depth recorded is the depth to water before attempting to evacuate the water from the piezometer.						60
	65							65
	70							70
	75							75
	80							80
	85							85
	90							90
	95							95
	100							100

Drilling Firm: Stantec
For: Division of Structural Design
Geotechnical Branch

DRILLER'S SUBSURFACE LOG

Project ID: **S-055-2008** Item Number: **05-118.00**	**Clark (Indiana) - I-265** **Ohio River**	Project Type: **Structure** Project Manager: -

Hole Number **AC-31**	Immediate Water Depth **22.9 (08/18/10)**	Start Date **08/17/2010**	Hole Type **NQ-3 core**
Surface Elevation **498.2'**	Static Water Depth **NA**	End Date **08/17/2010**	Rig_Number **CME 45 (811)**
Total Depth **43.1'**	Driller **D. Jessie**	Latitude(83) **38.345233**	
Location **212+67.00 40.0' Lt.**	Geologist **Ben Halada (Eng)**	Longitude(83) **-85.646071**	

Lithology: Elevation	Lithology: Depth	Description (Overburden / Rock Core)	Sample No. / Std/Ky RQD	Depth (ft) / Run (ft)	Rec. (ft) / Rec (ft)	SPT Blows / Rec (%)	Sample Type / SDI (JS)	Remarks
486.9	11.3	Medium stiff, light brown to dark brown, moist, clay (with some chert fragments).						(Begin Core)
486.0	12.2	No recovery.	40 / 40	5.7	4.8	84		
473.7	24.5	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded).	73 / 62	10.0	10.0	100		17.0
461.9	36.3	Dark gray shale, (fine grained, soft to moderately hard, laminated to thin bedded).	98 / 74	10.0	10.0	100		27.0
455.1	43.1	Gray limestone, (microcrystalline to fine grained, moderately hard, thin to medium bedded).	100 / 100	6.1	6.1	100		37.0
		(Bottom of Hole 43.1') (Refusal @ 11.3)						43.1

Remarks:

boulders, washed away @ 11.3-12.2
water stained @ 12.2-14
clay seam (<0.02') @ 15.9
clay seam @ 16.79-17.2
water stained @ 16.8-17.8
clay seam @ 23.2-23.3
shale streaks @ 23.3-24.5
clay streak (<0.02') @ 25.1-25.3
clay / shale streak, weathered @ 26.3-27
clay seam (<0.05') @ 27.5
clay / shale seam, weathered @ 27.7-27.9
water stained @ 34.8-36.9
clay seam (<0.05') @ 35.1
shale / clay, lightly weathered @ 35.2-36.3
horizontal fracture @ 38.2
water stained

DRILLER'S SUBSURFACE LOG

Project ID: S-055-2008 Item Number: 05-118.00	Clark (Indiana) - I-265 Ohio River	Project Type: Structure Project Manager: -

Hole Number AC-31	Immediate Water Depth 22.9 (08/18/10)	Start Date 08/17/2010	Hole Type NQ-3 core
Surface Elevation 498.2'	Static Water Depth NA	End Date 08/17/2010	Rig_Number CME 45 (811)
Total Depth 43.1'	Driller D. Jessie	Latitude(83) 38.345233	
Location 212+67.00 40.0' Lt.	Geologist Ben Halada (Eng)	Longitude(83) -85.646071	

Lithology: Elevation	Lithology: Depth	Description (Overburden / Rock Core)	Sample No. / Std/Ky RQD	Depth (ft) / Run (ft)	Rec. (ft) / Rec (ft)	SPT Blows / Rec (%)	Sample Type / SDI (JS)	Remarks
								@ 39.1-43.1 stylolitic fracture @ 40.5
55								55
60		Observation well set. 1" PVC with a 30 foot screen (13.3 ft to 43.1 ft). Sand pack to ±2 feet above screen. Remaining backfilled with pellets. 2X2 pad install with flushmount cover. The water depth recorded is the depth to water before attempting to evacuate the water from the piezometer.						60
65								65
70								70
75								75
80								80
85								85
90								90
95								95
100								100

Observation Well Readings

Stantec

Water Level Readings

Project Name: I-265 East End Bridge - Indiana Abutment   Location: Charles Moore Property

Stantec Project No: 175565125

Ground Surface Elevations

ALL WATER LEVEL READINGS WERE TAKEN FROM THE TOP OF THE WELL COVER

AC-29 = 494.2 feet above Mean Sea Level
AC-30 = 496.2 feet above Mean Sea Level
AC-31 = 498.2 feet above Mean Sea Level

	Well Number - Elevation					
Reading Date	**AC-29**	**Elevation**	**AC-30**	**Elevation**	**AC-31**	**Elevation**
8/18/2010	19.8 ft	474.4 ft	20.8 ft	475.4 ft	22.9 ft	475.3 ft
9/17/2010	33.4 ft	460.8 ft	36.7 ft	459.5 ft	36.1 ft	462.1 ft
10/22/2010	33.2 ft	461.0 ft	33.3 ft	462.9 ft	35.4 ft	462.8 ft
11/18/2010	22.7 ft	471.5 ft	21.0 ft	475.2 ft	35.0 ft	463.2 ft
12/21/2010	20.0 ft	474.2 ft	20.6 ft	475.6 ft	30.2 ft	468.0 ft
1/25/2011	20.0 ft	474.3 ft	20.4 ft	475.8 ft	30.4 ft	467.8 ft
2/23/2011	20.8 ft	473.4 ft	21.4 ft	474.8 ft	30.8 ft	467.4 ft

Strike and Dip Measurements

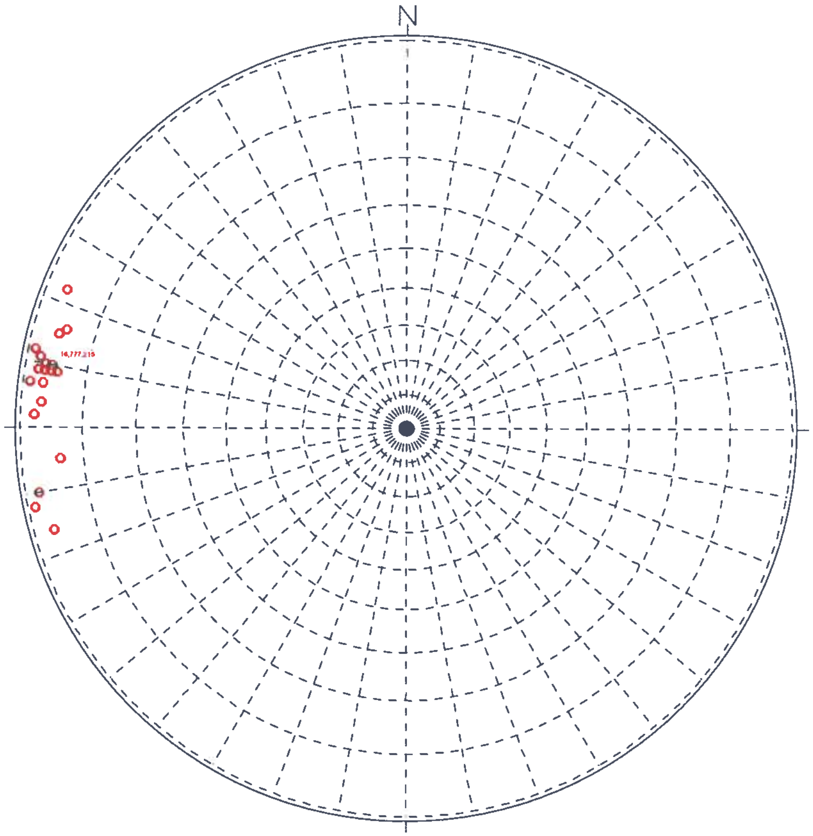
175565125-East End Approach

Stike and Dip Measurements Along Bedding Joints

1/19/2011

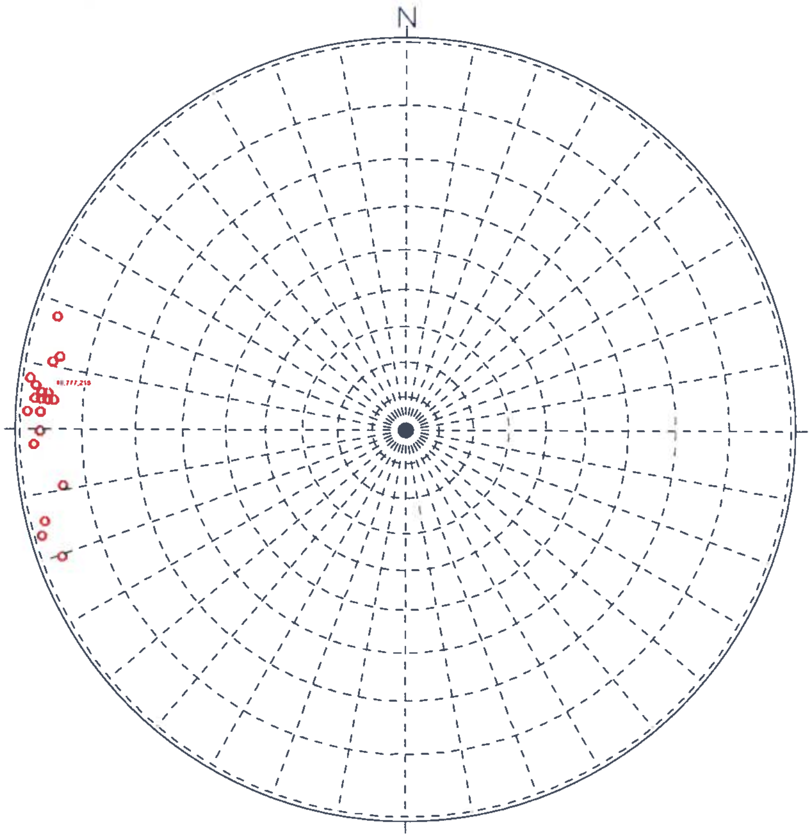
ID Site	Latitude	Longitude	Strike (In Azimuth)	Strike Magnetic Declination Adjusted	Strike (Quadrant)	Dip Angle (degrees)	Dip Direction (In Azimuth)	Dip Direction Magnetic Declination
15	N38.34513	W85.64682	10	5.7	N10E	4	280	275.7
17	N38.34586	W85.64644	9	4.7	N9E	5	279	274.7
19	N38.34586	W85.64652	9	4.7	N9E	5	279	274.7
20	N38.34581	W85.64617	9	4.7	N9E	3	279	274.7
21	N38.34585	W85.64625	16	11.7	N16E	5	286	281.7
25	N38.34588	W85.64618	9	4.7	N9E	4	279	274.7
27	N38.34583	W85.64581	22	17.7	N22E	3	292	287.7
28	N38.34568	W85.64571	9	4.7	N9E	2	279	274.7
29	N38.34575	W85.64580	11	6.7	N11E	2	281	276.7
30	N38.34490	W85.64756	7	2.7	N7E	1	277	272.7
31	N38.34471	W85.64718	10	5.7	N10E	4	280	275.7
32	N38.34466	W85.64715	12	7.7	N12E	1	282	277.7
34	N38.34495	W85.64762	4	359.7	N4E	3	274	269.7
35	N38.34444	W85.64648	7	2.7	N7E	3	277	272.7
37	N38.34452	W85.64651	348	343.7	N12W	1	258	253.7
38	N38.34439	W85.64650	344	339.7	N16W	3	254	249.7
39	N38.34433	W85.64645	350	345.7	N10W	2	260	255.7
42	N38.34491	W85.64574	2	357.7	N2E	2	272	267.7
43	N38.34474	W85.64582	15	10.7	N15E	4	285	280.7
44	N38.35074	W85.64895	10	5.7	N10E	3	280	275.7
45	N38.35207	W85.64795	355	350.7	N5W	6	265	260.7
46	N38.35060	W85.64630	350	345.7	N10W	2	260	255.7

175565125 Lineations Magnetic



Projection	Wulff (Equal Angle)
Number of Sample Points	22
Mean Lineation Azimuth	275.5
Mean Lineation Plunge	3.1
Great Circle Azimuth	215.5
Great Circle Plunge	3.6
1st Eigenvalue	0.972
2nd Eigenvalue	0.028
3rd Eigenvalue	0.001
LN (E1 / E2)	3.564
LN (E2 / E3)	3.924
(LN(E1/E2)] / (LN(E2/E3)) ..	0.908
Spherical variance	0.0143
Rbar	0.9857

175565125 Lineations Magnetic Declination Adjusted



Projection .... Wulff (Equal Angle)

Number of Sample Points 22

Mean Lineation Azimuth 271.2

Mean Lineation Plunge 3.1

Great Circle Azimuth 211.3

Great Circle Plunge 3.6

1st Eigenvalue 0.972

2nd Eigenvalue 0.028

3rd Eigenvalue 0.001

LN (E1 / E2) 3.564

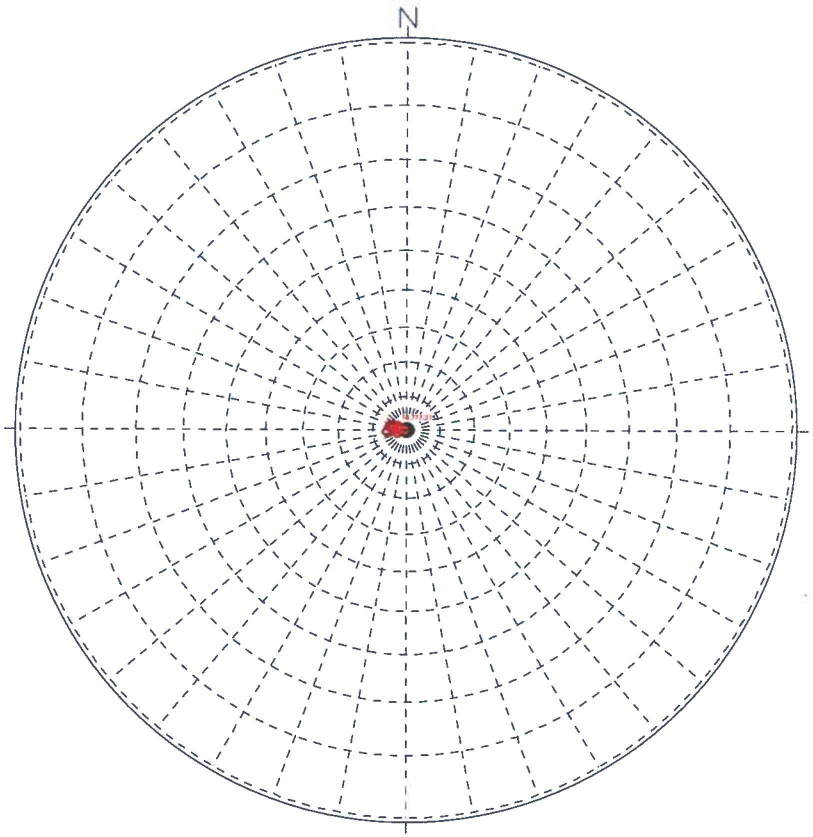
LN (E2 / E3) 3.924

(LN(E1/E2)] / (LN(E2/E3)) .. 0.908

Spherical variance 0.0143

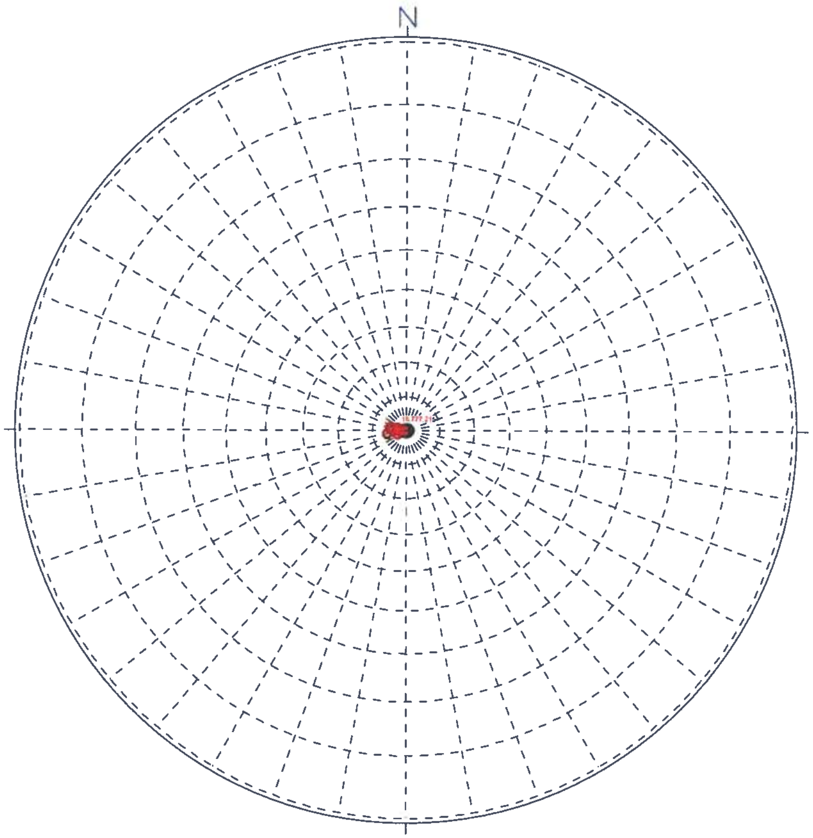
Rbar 0.9857

175565125 Planes Magnetic



Projection	Wulff (Equal Angle)
Number of Sample Points	22
Mean Lineation Azimuth	276.3
Mean Lineation Plunge	86.9
Great Circle Azimuth	99.7
Great Circle Plunge	89.8
1st Eigenvalue	0.999
2nd Eigenvalue	0.001
3rd Eigenvalue	0
LN (E1 / E2)	7.46
LN (E2 / E3)	2.026
(LN(E1/E2)] / (LN(E2/E3)) ..	3.681
Spherical variance	0.0003
Rbar	0.9997

175565125 Planes Magnetic Declination Adjusted



Projection	Wulff (Equal Angle)
Number of Sample Points	22
Mean Lineation Azimuth	272
Mean Lineation Plunge	86.9
Great Circle Azimuth	95.4
Great Circle Plunge	89.8
1st Eigenvalue	0.999
2nd Eigenvalue	0.001
3rd Eigenvalue	0
LN (E1 / E2)	7.46
LN (E2 / E3)	2.026
(LN(E1/E2)] / (LN(E2/E3)) ..	3.681
Spherical variance	0.0003
Rbar	0.9997

STANTEC
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1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
859-422-3000

Stantec

Addtional Geologic Mapping

I-265 over the Ohio River
Indiana Abutment
LSIORB, Section 5, Phase 4
Jefferson County, Kentucky

PROJECT NO. 175565125.205	
DATE	Jauary 2011
DRAWN BY	MAG
CHECKED BY	DB
CHECKED BY	
SCALE	1" = 250 '
REVISED	
1.	
2.	
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5.	
6.	
7.	
8.	
SHEET	1