See also final report at https://library.modot.mo.gov/RDT/reports/TR201714/cmr18-008.pdf

Appendix A – Existing Bridge Documentation

Detailed description of contents is included in Section 3.2.

Driven Pile excerpt from 1961 MoDOT Standard Specifications (7 sheets, 2 pages/sheet)
Final plans for existing bridge A2141 (CIP piles) (4 sheets)
As-built ("Finished") plan sheet for existing bridge A2141 (CIP piles) (1 sheet)
MoDOT standards sheet for CIP piles from 1962 (1 sheet)
Record of pile driving for CIP piles (1 page)
Historical boring logs for existing bridge A2141 (CIP piles) (7 pages)
MoDOT geotechnical report for replacement of existing CIP pile bridge (41 pages)
Final plan sheet for existing bridge N0771 (Precast piles) (1 sheet)
As-built ("Finished") plans for existing bridge N0771 (Precast piles) (4 sheets)
MoDOT standards sheet for precast piles from 1962 (1 sheet)
MoDOT geotechnical report for replacement of existing precast pile bridge (14 pages)

STANDARD SPECIFICATIONS

for

State Roads, Materials, Bridges, Culverts and Incidental Structures

Edition of 1961

MISSOURI
STATE HIGHWAY COMMISSION
JEFFERSON CITY, MISSOURI

Price $3.00

51.6.2. Pedestal pile will be paid for at the contract unit price per linear foot under

Item 51006: Pedestal Pile, per linear foot.

NOTE: The third and forth digit of the item number indicates the pile diameter.

SECTION 52
BEARING PILE

52.1. Description.

52.1.1. Bearing pile shall consist of furnishing and driving concrete, steel and timber piles to the bearing and penetration required, at the location shown on the plans. When designated in the contract, the Commission will furnish the piles, otherwise they shall be furnished by the contractor.

52.2. Materials.

52.2.1. *Precast concrete piles* shall be manufactured of Class A Concrete to the shape and size shown on the plans or to an approved equivalent section. Piles shall be cast with a driving point, and if required, shall be shod with a metal shoe of approved pattern. All materials, proportioning, air-entrainment, mixing and transporting of portland cement concrete shall be in accordance with Sec. 47. Precast piles shall be straight, with a center line variation of not more than ½ inch per 25-foot length of pile. The removing of forms, curing, storing, transporting, and handling of precast piles shall be done in a manner to avoid excessive bending stresses, cracking, spalling, and other damaging affects. Precast concrete piles shall be lifted and handled by a suitable bridle attached to the pile at points shown on the plans. They shall not be moved from the forming bed for at least 48 hours after casting and not until the concrete has attained a flexural strength of 500 pounds per square inch. This removal will be permitted only when 1/8 point (4 equi-distant points) pickup is used for all piles 18 feet or greater in length, 1/4 point pickup for all other lengths, and further provided that piles are moved without vibration or impact to a curing bed which provides uniform support the full length of the piles. Curing shall be maintained for at least 24 hours after concrete has reached a flexural strength of 750 pounds per square inch. Piling shall not be subjected to transportation stresses or driven until the above specified flexural strength has been attained.

52.2.2. *Cast-in-place concrete piles* shall consist of Class

A Concrete cast in pre-driven metal shells. The metal shells shall conform to the shape, size, and minimum shell thickness shown on the plans or to an approved equivalent section. All materials, proportioning, air-entrainment, mixing and transporting of portland cement concrete shall be in accordance with Sec. 47. Metal shells driven by core or mandrel shall be of sufficient thickness, and shall be reinforced so that they will hold their original form without distortion after being driven and the core withdrawn. Metal shells driven without a core or mandrel shall be of sufficient thickness and shall be reinforced so that they will hold their original form without distortion after being driven. Unless otherwise noted in the contract, cast-in-place concrete piles will not require reinforcing steel inside the shells.

52.2.3. *Structural steel piles* shall be of the series rolled as H-bearing piles and shall conform to A.S.T.M. A 7-58T. They shall be of the size, weight, and structural shape designated on the plans. Piles shall not have a camber or sweep in excess of permitted mill tolerance. Steel piles shall be stored on platforms, skids or other supports at the site of the work and shall be supported at frequent intervals.

52.2.4. *State furnished steel piles* will be furnished at points on or near the project for handling and driving by the contractor. The contractor shall make a physical inspection and inventory of the steel bearing pile material furnished to him by the Commission in stock pile or f.o.b. cars at delivery point. He shall verify, in the presence of the engineer, all quantities, sizes, lengths, and condition of the material. He shall accept custody of this steel bearing pile material and shall furnish the engineer with a signed recept for each lot accepted. After the specified examination and acceptance of the material, the contractor shall be fully responsible for it. This responsibility shall include the proper stockpiling and protection at or near the bridge sites pending use, and the safeguarding of cutoffs and unused lengths until after all pile driving and other work on the project is completed, or until such time as he may be relieved of responsibility for portions of cutoffs and other material taken over by the Commission.

52.2.5. *Treated timber piles* shall be Southern Pine. *Untreated timber piles* for use in unexposed locations or in temporary bridges shall be Southern Pine or other species approved by the engineer. Grade and treatment of timber piles shall be as specified in Sec. 150. Timber piles shall be pointed when required by soil conditions and when necessary, piles shall be shod with metal shoes of approved design. The points of the piles shall be shaped to secure an even uniform bearing on the shoes. Special care shall be taken to avoid breaking the surface of treated piles; cant hooks, dogs, and pike poles shall not be used. All cuts and abrasions made after treatment shall be given 2 brush coats of hot creosote. The first coat shall be allowed to dry before the second coat is applied. Material surrounding all holes in treated timber piles shall be thoroughly saturated with hot creosote.

52.2.6. The pile lengths shown on the plans are approximate. Lengths of precast concrete, steel, and timber piles to be furnished by the contractor necessary to obtain the required bearing and penetration will be authorized by the engineer. A tolerance of one foot will be allowed on lengths furnished under the engineer's authorization. For cast-in-place concrete piles payment will be made only for pile lengths in place, and the contractor shall be fully responsible for the lengths he furnishes for driving to obtain the specified bearing and penetration. Subsurface investigations made by the Commission for design purposes only, are available for the contractor's review in accordance with Sec. 2.4.

52.2.7. *Test piles* shall be the same material and size as the permanent piles, except that if treated timber piles are specified for the structure, untreated timber test piles may be used if not driven in a permanent location. Test piles of concrete and steel shall, in general, be driven in the place of foundation piles. Test piles shall be of such length as to permit driving the tips to an elevation 10 feet below that indicated by plan lengths unless otherwise specified.

52.3. Equipment.

52.3.1. The pile driving equipment shall be adequate for driving piles 10 feet longer than the longest length authorized. Piles shall be driven with gravity or power-driven hammers, or by a combination of hammer and water jets. Power-driven hammers are defined as hammers operated by steam, air, or diesel power. For determining the energy per blow of diesel power hammers, 75 percent of the manufacturer's energy rating for the hammer will apply. If the contractor desires to check his diesel power hammer against an approved steam hammer on a specified type of pile at a particular site, he may do so at his expense, and the checked rating of the diesel powered hammer will be used in determination of pile bearing values at that site.

52.3.2. *Pile driver leads* shall be constructed in such a manner as to afford freedom of movement of the hammer, and they shall be held in position by guys or stiffener braces to insure support to the pile during driving. The leads shall be of sufficient length so that the use of a follower will not be necessary. Inclined leads shall be used for the driving of battered piles.

52.3.3. *Followers* may be used in the driving of piles only with the written permission of the engineer. When used, one pile of every group of 10 shall be a long pile driven without a follower to determine the available bearing value of the group.

52.3.4. *Water jets* used to aid in driving piles shall be sufficient in number to deliver a volume and pressure of water at the jet nozzles that will freely erode the material adjacent to the pile. The use of water jets shall be discontinued before the final penetration is reached and the piles shall be driven to secure a final penetration of not less than 2 feet when the nature of the soil permits.

52.3.5. *Precast concrete piles* shall be driven with a power-driven hammer developing an energy per blow of not less than 3500 foot pounds per cubic yard of concrete in the pile being driven. The total energy developed by the hammer shall be not less than 8000 foot pounds per blow.

52.3.6. *Shells for cast-in-place concrete piles* driven without a mandrel shall be driven with a power-driven hammer developing an energy per blow of not less than 7000 foot pounds. Shells driven with a core or mandrel shall be driven with a power-driven hammer developing an energy per blow of not less than 10,000 foot pounds.

52.3.7. *Structural steel piles* shall, in general, be driven with power-driven hammers developing an energy per blow of not less than 7000 foot pounds.

52.3.8. *Timber piles* may be driven with a gravity hammer or a power-driven hammer. Power-driven hammers shall develop an energy of not less than 4100 foot pounds per blow at each full stroke of the piston. It is preferable that gravity hammers weigh 3000 pounds but they shall weigh not less than 2000 pounds. The weight of the hammer shall be

verified. The fall of the hammer shall be regulated to avoid injury to the piles, and shall not exceed 20 feet.

52.4. Construction Procedure

52.4.1. The contractor shall furnish and drive test piles at locations designated. Where required, test piles shall be driven full length or to refusal, or to a capacity 50 percent greater than that required on the design plans. They shall be driven with the same type of equipment as will be used for driving the permanent piles. Before driving test piles, the excavation shall be completed to an elevation not more than 2 feet above the proposed grade at the point where a test pile is to be driven. Test piles not driven in a permanent location shall be cut off, or pulled and backfilled as directed by the engineer.

52.4.2. The contractor shall not proceed with pile driving until the type and weight of the hammer to be used has been approved. Foundation piles shall not be driven until after the excavation of the footing has been completed. The heads of all precast concrete and timber piles shall be protected, when the nature of the driving is such as to unduly damage them, by a cap of an approved design having a cushion made of wood, rope, or other suitable material next to the pile head, and fitting into a casting which in turn supports a timber shock block. A suitable cap may be required to distribute the blow of the hammer throughout the cross section of the pile when the area of the head of any timber pile is greater than the face of the hammer. Pile collars or dished metal caps to protect timber piles from splitting or shattering shall be used where necessary. Broomed, crushed, or splintered piles shall be replaced. A cast or structural steel driving head shall be used to prevent excessive upsetting of the pile head of steel piles when required by extremely hard driving conditions. The procedure incident to the driving of piles, whether of timber, concrete, or steel, shall not subject them to excessive and undue abuse. Any pile broken by reason of internal defects or by improper driving, or driven out of its proper location shall be removed and replaced, or a second pile may be driven adjacent thereto if this can be done without detriment to the structure.

52.4.3. *Final position of piles driven* shall not be more than ¼ inch per foot from the vertical or from the battered line indicated on the plans. The maximum variation of the head of the pile from the position shown on the plans shall be not more than 2 inches for trestle piling and 6 inches for foundation piling. Timber piles driven below the elevation shown on the plans shall be withdrawn and replaced by longer piles at the expense of the contractor. Precast concrete piles driven below the elevation shown on the plans shall be extended by build-up construction. All piles pushed up by the driving of adjacent piles, or by any other cause, shall be redriven to required bearing and penetration. Metal pile shells shall be free from water, soil, and other deleterious matter when concrete is cast in them. The contractor shall maintain on the job at all times prior to and during the filling of the shells, a light suitable for use in their inspection. Improperly driven, broken, or otherwise defective shells shall be removed and replaced, or otherwise corrected to the satisfaction of the engineer.

52.4.4. *Pre-bored holes* may be required when piles are to be driven through compacted embankments more than 5 feet deep. For piles other than cast-in-place concrete piles, the holes shall be bored to a diameter equal to or less than that of the pile. Metal shells for cast-in-place piles will require holes equal to or larger than the size of the shell. The space remaining around any type pile after it is driven shall be completely filled with sand or other approved material.

52.4.5. *Extending and splicing of piles* is not desirable, and full length piles should be driven wherever possible and practicable. The number of splices used shall be held to a minimum. Splicing of timber piles and more than one splice per pile for other types shall not be made without permission from the engineer. All splices of steel shells or steel bearing piles shall be made by properly qualified welding operators, with welding operations witnessed by the engineer. Welding shall be done by the electric arc process in accordance with A.W.S. Specifications. When permitted or required by the engineer, extensions and splices shall be made as follows.

52.4.5.1. Precast concrete piles shall be extended after driving is completed. The pile shall be extended by having the concrete at the end of the pile cut away leaving the reinforcing bars exposed for a length of 40 diameters. The final cut shall be at right angles to the axis of the pile. Reinforcing bars shall be lapped 32 diameters and fastened to the projecting steel. If the bars are butt welded instead of lapped, the concrete at the end of the piles shall be cut away as described above to expose at least 12 inches of the main reinforcing bars. The concrete for the extension shall be of the same class as used in the pile. Just prior to placing the concrete, the top of the pile shall be thoroughly wetted and covered with a thin coating of 1:2 cement mortar. The forms shall remain in place at least 24 hours, and the extended section of the pile shall be finished as specified in Sec. 53.4.7.

52.4.5.2. Cast-in-place concrete pile shells shall be spliced as shown on the plans. Metal shell sections used for splicing shall be at least 5 feet in length, and not more than 2 splices per pile shell will be permitted.

52.4.5.3. Structural steel piles shall be spliced with a butt-weld as shown on the plans.

52.4.5.4. Timber pile splices shall be of the butt-joint type, and the added pieces shall conform closely in diameter to that of the main pile at the point of splice. Piles shall be sawed square and the butt joints shall bear evenly over the entire surface. The joint shall be banded with a 4-foot length of iron pipe at least 12 inches in diameter centered on the joint and held in position by 6-5/8 inch lag screws 6 inches long; 3 lag screws in the pile and 3 in the splice. The sawed and trimmed surfaces of treated piling shall be given 2 heavy brush coatings of hot creosote before the splice is assembled. Hot creosote shall be poured in all holes for lag screws.

52.4.6. Tops of all piles shall be cut off square at cut-off elevations. Pile tops which support timber caps or grillages shall conform to the plane of the bottom of the superimposed structure. The heads of all treated timber piles shall be given 2 coats of hot creosote, and in addition, trestle piles shall be covered with a protective cap made by applying a coat of hot roofing pitch and a sheet of 24-gauge galvanized iron. The cap material shall measure at least 6 inches more in each direction than the diameter of the pile, and shall be bent down over the pile, and the edges trimmed in a neat manner and secured with large head galvanized or copper nails.

52.4.7. Piles for steel pile end bents shall be coated with a heavy coating of an approved bituminous paint applied for a length of 3 feet below the bottom of the concrete cap. Where exposed steel piles extend into the ground, the portion of the pile 3 feet below and one foot above the finished ground line shall be coated. Before the coating is applied, the steel shall be thoroughly cleaned. Coating below the water line will not be required. All metal shells, after driving, shall be protected with a heavy coat of an approved bituminous paint as specified for steel piles.

52.4.8. *Concrete footings on cast-in-place piles* shall not be placed until at least 12 hours after the last pile in the footing has been cast. No piling shall be driven within a radius of 20 feet of concrete that has taken initial set and has not obtained at least 50 percent of the flexural strength specified in Sec. 53.4.9.1.

52.4.9. *The bearing value of piles* shall be determined by actual load tests when called for on the plans or ordered by the engineer. The test shall consist of the application of a load placed upon a suitable platform supported by the pile, with suitable apparatus for accurately measuring the test load and the settlement of the pile under each increment of load. Hydraulic jacks with suitable yokes and pressure gauges may be used in lieu of the loaded platform. The test load shall be applied to exert a uniform pressure over the pile or piles being tested. The driven pile shall not be disturbed for at least 24 hours prior to the application of any portion of the test load. The load shall be applied in 25 percent increments of the total load, allowing rest periods of 6, 12, and 6 hours respectively between the increment of loadings. The safe allowable load per pile shall be considered as 50 percent of that load which, after remaining in place for 48 hours, produces a permanent settlement not greater than ¼ inch, measured at the top of the pile.

52.4.10. The following formulae will be used as a guide to determine the safe bearing value of piles when loading tests are not required:

$$P = \frac{2WH}{S + 1.0} \text{ for gravity hammers.}$$

$$P = \frac{2WH}{S + 0.1} \text{ for single acting hammers.}$$

$$P = \frac{2E}{S + 0.1} \text{ for double acting hammers.}$$

$$P = \frac{1.5E}{S + 0.1} \text{ for diesel powered hammers unless tested as described in Sec. 52.3.1.}$$

P = safe allowable bearing value, in pounds.

W = weight of striking parts of hammer, in pounds.

H = height of fall, in feet.

E = manufacturer's rated energy in foot-pounds per blow at manufacturer's rated speed.

S = average penetration, in inches per blow, for 5 to 10 consecutive blows for gravity hammers, or 10 to 20 consecutive blows for power-driven hammers.

52.4.10.1. The above formulae are applicable only when:

1. The piles are driven in a vertical position.
2. The hammer has a free fall.
3. The pile head is not broomed, crushed, or splintered.
4. There is no appreciable bounce of the hammer after striking the pile.
5. The penetration is at a uniform or uniformly decreasing rate.
6. The fall of the gravity hammer is limited to 15 feet.

52.4.10.2. For piles driven to a batter, the safe bearing value of the pile shall be taken as $P_B = KP$.

$$K = \frac{.25\,(4 - m)}{(1 + m^2)} \text{ for gravity hammers.}$$

$$\text{or } K = \frac{.1\,(10 - m)}{(1 + m^2)} \text{ for power-driven hammers.}$$

P_B = safe allowable bearing value in pounds for batter pile.

m = the tangent of the angle of batter.

K = numerical constant.

52.4.11. The penetration of piles shall be such that the bearing value determined in accordance with Sec. 52.4.10 is not less than that shown on the plans. In general, timber piles shall not be driven to a bearing value in excess of 5 tons, nor precast concrete piles to a bearing value in excess of 10 tons over the specified bearing value. Piles shall also be driven to the minimum penetration indicated on the plans. If no required minimum penetration is specified, they shall have a minimum penetration of 10 feet in firm material below the bottom of the footing for foundation pile or below the natural ground line for other piles.

52.5. Method of Measurement.

52.5.1. Piles in place shall be the actual length of all piles, except test piles, measured to the nearest foot for each pile that remains permanently in the structure.

52.5.2. Test piles will be measured to the nearest linear foot of pile authorized and driven.

52.5.3. Pile cut-offs shall be the actual length, measured to the nearest foot for each pile furnished by the contractor, less the lengths of the piles permanently remaining in place. All cut-off material, except steel shell cut-offs shall become the property of the Commission, and shall be disposed of as directed by the engineer. Steel shell cut-offs will remain the property of the contractor.

52.5.4. Precast concrete piles with cast-in-place extensions will be considered single piles and measured as such; no measurement being made for the length of pile destroyed when making the extension.

52.5.5. No measurement will be made of any excavation required to apply the protective coating below ground line to steel piles or metal shells of cast-in-place concrete piles.

52.6. Basis of Payment.

52.6.1. Payment for any type of pile in place will be made at the contract unit price per linear foot. The extra cost of the material, when the required or authorized length of piles exceeds by more than 10 feet the length originally shown on the plans, will be allowed upon submittal of documentary evidence establishing the extra cost per linear foot of the longer piles. No direct payment will be made for furnishing and placing protective caps for timber piles or the protective coating for steel piles and metal shells.

52.6.2. Payment for test piles will be made at the contract unit price per linear foot. Test piles when driven and used as permanent piles in place will be paid for as test piles and not as piles in place.

52.6.3. Pile cut-offs of timber, precast concrete and contractor furnished steel piles will be paid for at the contract

unit price per linear foot. No payment will be made for cut-offs of (1) steel shells for cast-in-place concrete piles, (2) state furnished steel piles, and (3) test piles.

52.6.4. Payment for loading tests will be made at the contract unit price per test.

52.6.5. Pile splices, when authorized, will be paid for as an additional 8 feet of pile in place at the contract unit price per linear foot for the type of pile spliced. No payment will be made for any splices in metal shells for cast-in-place concrete piles.

52.6.6. Metal shoes for timber piles, where specified, will be paid for as an additional 5 feet of pile in place at the contract unit price per linear foot for timber piles.

52.6.7. Payment will be made under

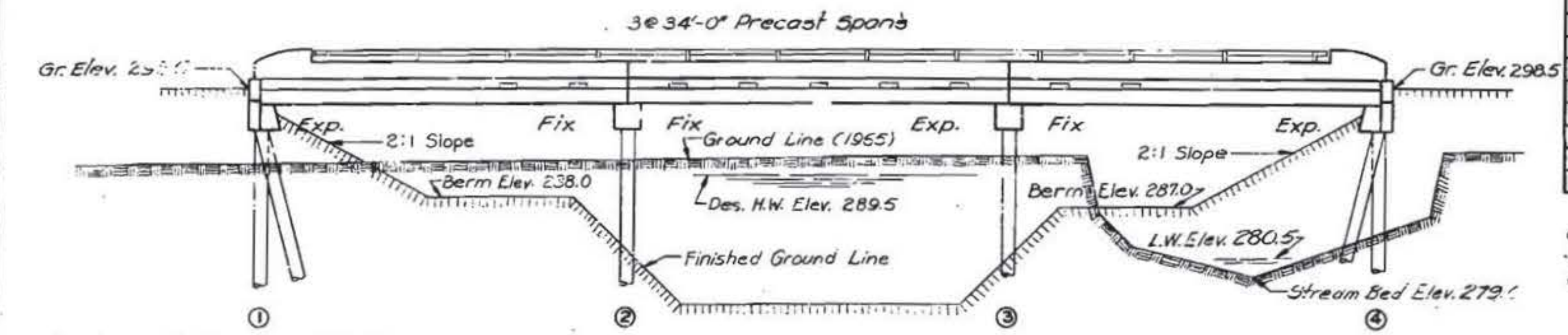
- Item 52000: Untreated Timber Piles in Place, per linear foot.
- Item 52010: Untreated Timber Pile Cut-Offs, per linear foot.
- Item 52001: Treated Timber Piles in Place, per linear foot.
- Item 52011: Treated Timber Pile Cut-Offs, per linear foot.
- Item 52002: Precast Concrete Piles in Place, per linear foot.
- Item 52012: Precast Concrete Pile Cut-Offs, per linear foot.
- Item 52003: Cast-in-Place Concrete Piles, per linear foot.
- Item 52004: Steel Piles in Place (State Furnished), per linear foot.
- Item 52005: Steel Piles in Place, per linear foot.
- Item 52015: Steel Pile Cut-Offs, per linear foot.
- Item 52008: Test Piles, per linear foot.
- Item 52009: Loading Tests, each.

NOTE: The third digit of the item number indicates size of pile:

0—10″ Pile
2—12″ Pile
4—14″ Pile

MISSOURI STATE HIGHWAY DEPARTMENT

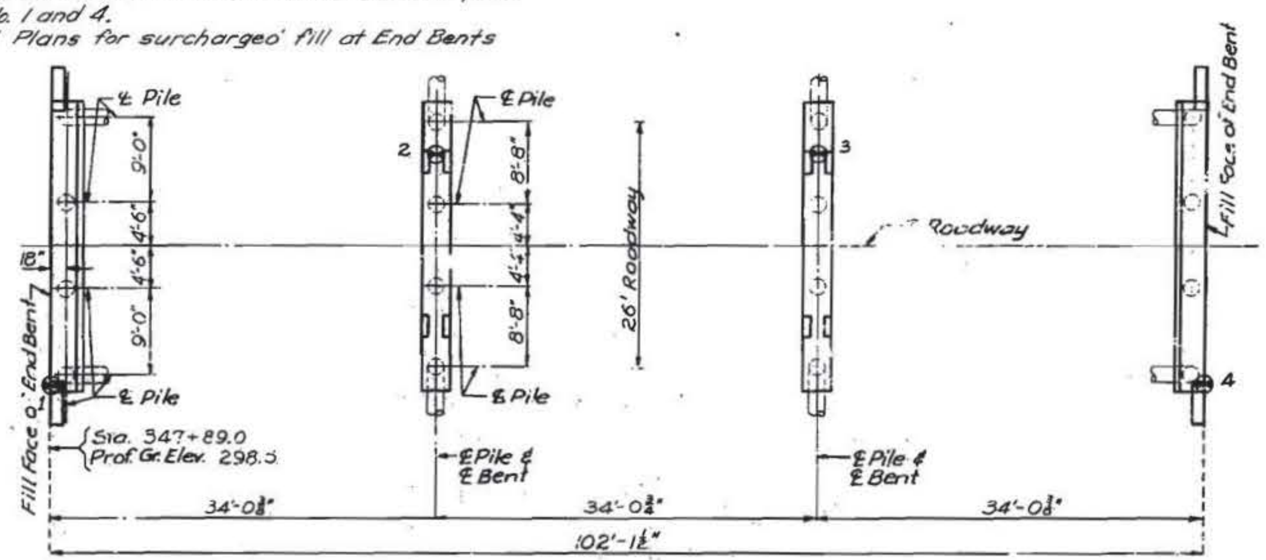
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19		43



ELEVATION

Note: Compacted roadway fill (full roadway width) shall be placed up to elevation of bottom of concrete beam in front of and not less than 25'-0" in back of End Bents No. 1 and 4 before piles are driven for End Bents No. 1 and 4. Pre-bore holes for piles at End Bents No. 1 and 4.

See Road Plans for surcharged fill at End Bents No. 1 and 4.



PLAN

PILE DATA					
Bent No.		1	2	3	4
Type		CIP	CIP	CIP	CIP
Kind		Trestle	Trestle	Trestle	Trestle
Number		4	4	4	4
Approximate Length	Ft.	55	60	70	70
Design Bearing	Tons	30	30	30	30
Min. Tip Penetration	Elev.	265.0	260.0	260.0	265.0
Pile Standard		52.02	52.02	52.02	52.02
Hammer Energy Req'd.	Ft. Lbs.	8000	8000	8000	8000

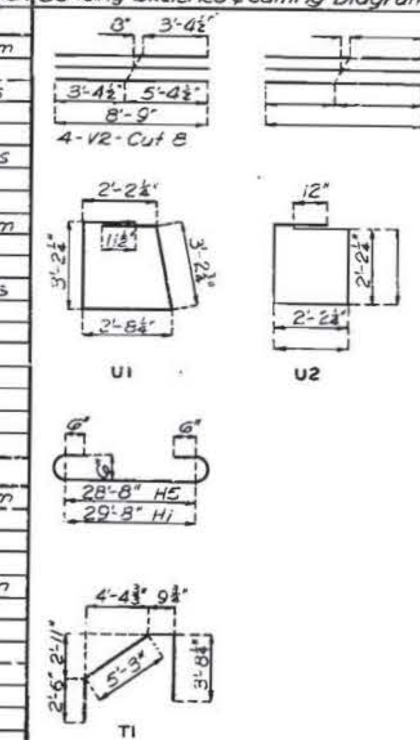
Minimum energy requirement of hammer based on plan length of piles.

All pile shall be driven to the minimum penetrations and to not less than the design bearings noted.

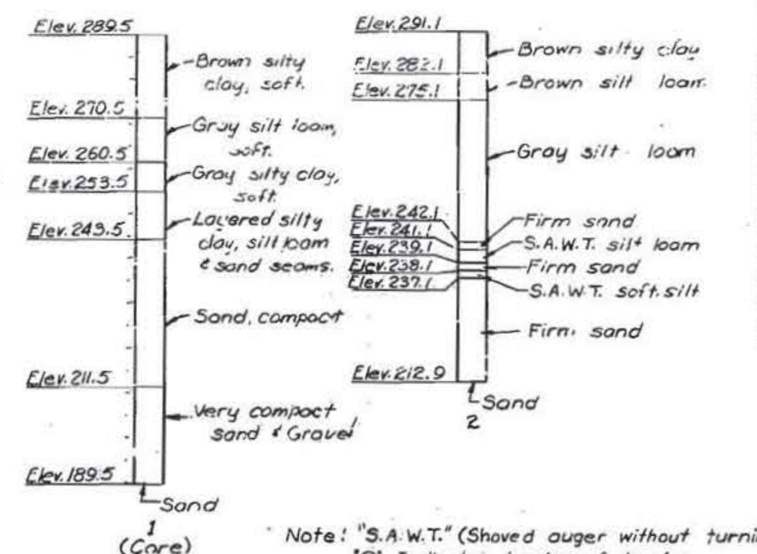
BILL OF REINFORCING STEEL – SUBSTRUCTURE

No.	Size	Length	Mark	Location
End Bents No. 1 & 4				
16	#6	32'-3"	H1	Beam
4	#6	30'-3"	H2	"
24	#5	5'-9"	H4	Wings
8	#6	12'-3"	T1	Wings
56	#4	12'-3"	U1	Beam
8	#4	8'-9"	V2	Wings
Int. Bents No. 2 & 3				
16	#6	31'-3"	H5	Beam
4	#6	29'-3"	H6	"
56	#4	9'-9"	U2	Beam

Bending Sketches & Cutting Diagrams

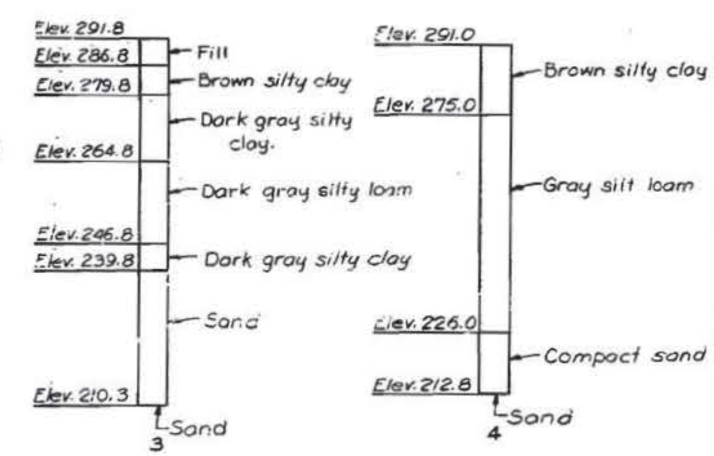


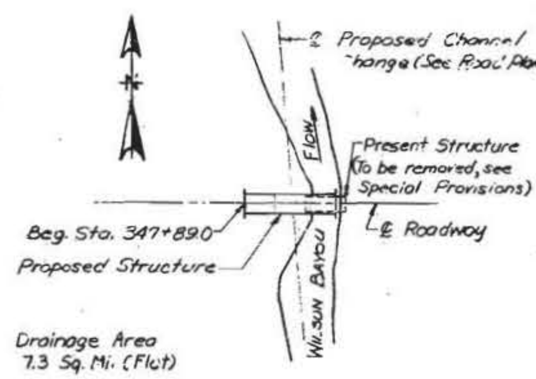
Note: See Sheet No. 3 of 4 for Bill of Reinforcing Steel for superstructure.



Note: "S.A.W.T." (Shoved auger without turning.) "⊖" Indicates location of boring.

BORING DATA





LOCATION SKETCH

ESTIMATED QUANTITIES				
ITEM		SUBSTR.	SUPERSTR.	TOTAL
Bituminous Surface	Sq. Yds.		295	295
Cast-in-Place Concrete Piles	Lin. Ft.	1020		1020
Class A or Class X Concrete	Cu. Yds.		88.5	88.5
Class B Concrete	Cu. Yds.	38.6	15.4	54.0
Reinforcing Steel	Lbs.	3050	37,240	40,290
Bridge Rail (Single tube type)	Lin. Ft.		183	183

Note: Cost of ⅝" coil ties, 1"ø tie bolts and washers shall be included in price bid for precast units.

Cost of any required excavation for bridge will be included in price bid for other items.

GENERAL NOTES:

Design Specifications: A.A.S.H.O. - 1965

Design Loading:

H15-44

Earth 120# Equivalent Fluid Pressure 30#.

Design Unit Stresses:

Class A or Class X Concrete (Precast Units) f'c = 1,500 psi

Class B Concrete (substructure, superstructure curb, parapet and end posts) f'c = 1,200 psi

Reinforcing Steel fs = 20,000 psi

Structural Steel (A.S.T.M. A36-66) fs = 20,000 psi

Painting:

All bolts and washers for holding precast concrete units together shall be cleaned and painted three coats or galvanized.

All exposed surfaces of steel shells for cast-in-place piles shall be painted. Payment to be included in price bid for items painted.

B.M. #35 Elev. 289.24 N.I.W.R. 40" Sycamore 104' Rt. Sta. 348+21

BRIDGE OVER WILSON BAYOU

STATE ROAD FROM NEW MADRID NORTHEASTERLY

ABOUT 6.5 MILES N.E. OF NEW MADRID

PROJECT NO. C072-WW(1) SWW STA. 347+89.0

NEW MADRID COUNTY

SUBMITTED BY D. B. Gentner BRIDGE ENGINEER DATE 12/7/67

APPROVED BY M. J. Mulder CHIEF ENGINEER DATE 12/7/67

STD. 52.02

A-2141

117

No. 5140.1	Revised
June 1965	Jan. 1966

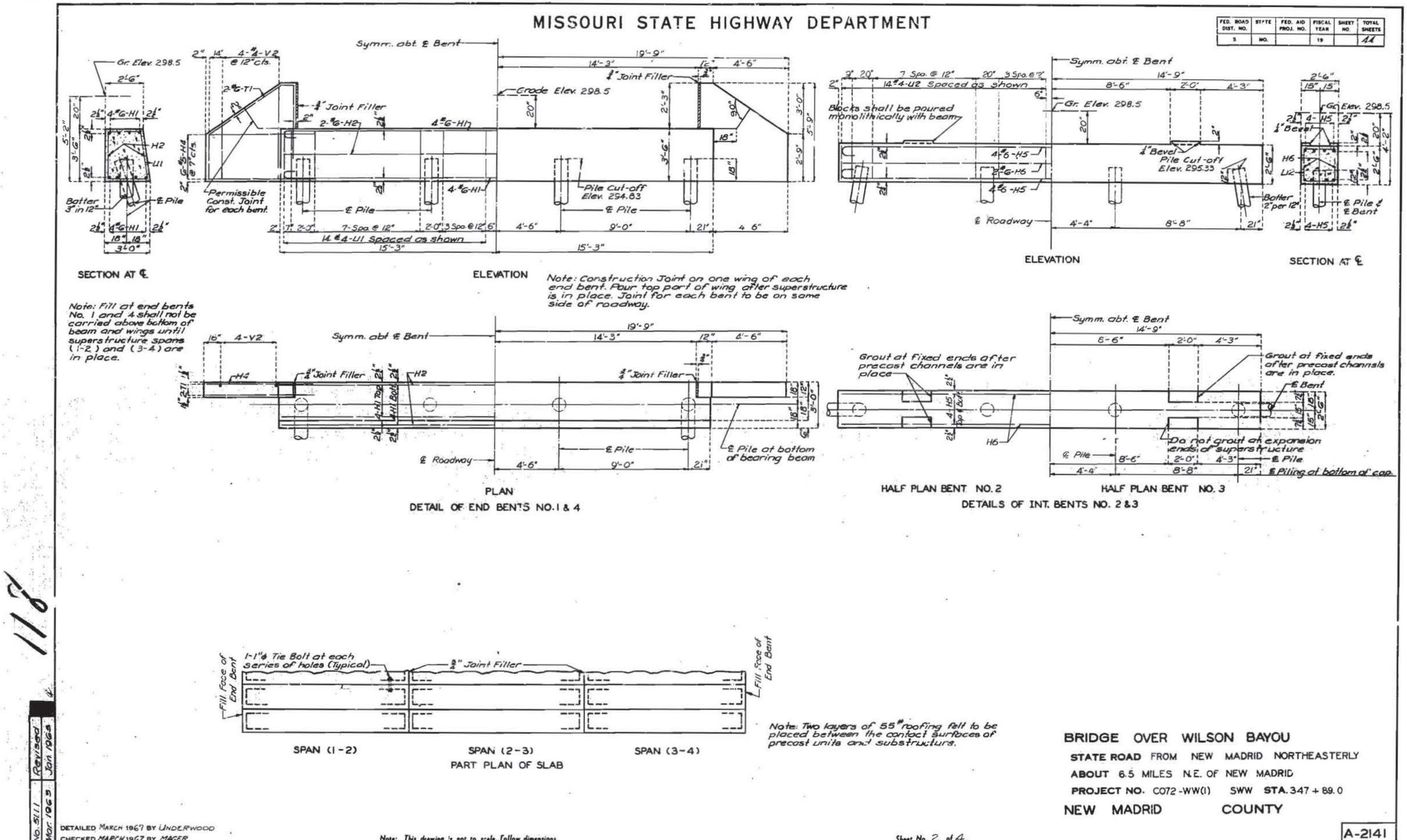
DESIGNED MARCH 1967 BY UNDERWOOD

DETAILED MARCH 1967 BY UNDERWOOD

CHECKED MARCH 1967 BY MAGER

Note: This drawing is not to scale. Follow dimensions.

Sheet No. 1 of 4.


MISSOURI STATE HIGHWAY DEPARTMENT
FED. ROAD DIST. NO. 5 | STATE MO. | FED. AID PROJ. NO. | FISCAL YEAR 19 | SHEET NO. | TOTAL SHEETS 44
SECTION AT ℄
ELEVATION
Note: Construction Joint on one wing of each end bent. Pour top part of wing after superstructure is in place. Joint for each bent to be on same side of roadway.
Note: Fill at end bents No. 1 and 4 shall not be carried above bottom of beam and wings until superstructure spans (1-2) and (3-4) are in place.
Symm. abt. ℄ Bent
Grade Elev. 298.5
Gr. Elev. 298.5
¾" Joint Filler
Permissible Const. Joint for each bent.
Pile Cut-off Elev. 294.83
℄ Pile
14 #4-U1 Spaced as shown
Batter 3" in 12"
Blocks shall be poured monolithically with beam
14 #4-U2 Spaced as shown
Pile Cut-off Elev. 295.33
Batter 2" per 12"
℄ Pile & ℄ Bent
℄ Roadway
PLAN
DETAIL OF END BENTS NO. 1 & 4
℄ Pile at bottom of bearing beam
Grout at fixed ends after precast channels are in place
Do not grout at expansion ends of superstructure
℄ Piling at bottom of cap.
HALF PLAN BENT NO. 2
HALF PLAN BENT NO. 3
DETAILS OF INT. BENTS NO. 2 & 3
1-1"φ Tie Bolt at each series of holes (Typical)
½" Joint Filler
Fill Face of End Bent
SPAN (1-2)
SPAN (2-3)
SPAN (3-4)
PART PLAN OF SLAB
Note: Two layers of 55# roofing felt to be placed between the contact surfaces of precast units and substructure.
BRIDGE OVER WILSON BAYOU
STATE ROAD FROM NEW MADRID NORTHEASTERLY
ABOUT 6.5 MILES N.E. OF NEW MADRID
PROJECT NO. C072-WW(1) SWW STA. 347 + 89.0
NEW MADRID COUNTY
A-2141
DETAILED MARCH 1967 BY UNDERWOOD
CHECKED MARCH 1967 BY MAGER
Note: This drawing is not to scale. Follow dimensions.
Sheet No. 2 of 4.
No. 5111 Mar. 1963 Revised Jan 1965
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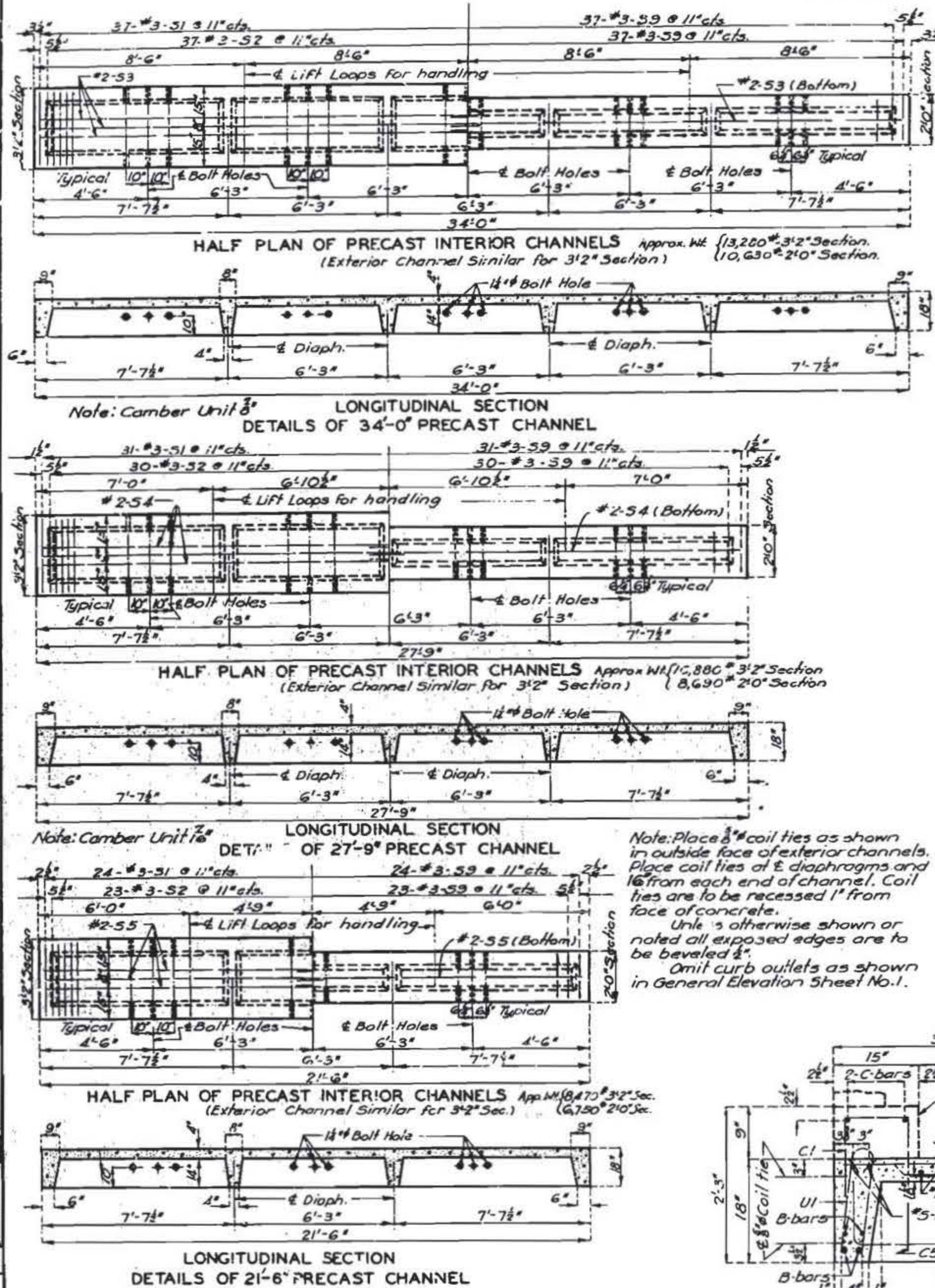
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19		45

MISSOURI STATE HIGHWAY DEPARTMENT



HALF PLAN OF PRECAST INTERIOR CHANNELS Approx. Wt. {13,260# 3'-2" Section. 10,630# 2'-0" Section.
(Exterior Channel Similar for 3'-2" Section)

LONGITUDINAL SECTION

Note: Camber Unit 3/8"

DETAILS OF 34'-0" PRECAST CHANNEL

HALF PLAN OF PRECAST INTERIOR CHANNELS Approx. Wt. {10,880# 3'-2" Section 8,690# 2'-0" Section
(Exterior Channel Similar for 3'-2" Section)

LONGITUDINAL SECTION

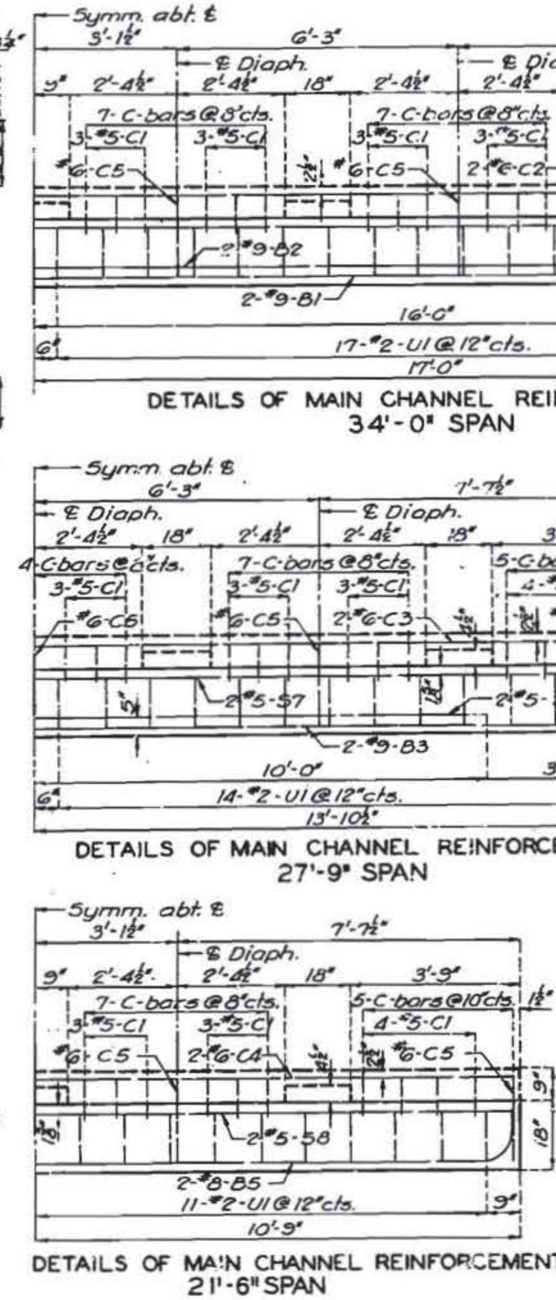
Note: Camber Unit 7/8"

DETAILS OF 27'-9" PRECAST CHANNEL

HALF PLAN OF PRECAST INTERIOR CHANNELS Approx. Wt. {8,470# 3'-2" Sec. 6,750# 2'-0" Sec.
(Exterior Channel Similar for 3'-2" Sec.)

LONGITUDINAL SECTION

DETAILS OF 21'-6" PRECAST CHANNEL



DETAILS OF MAIN CHANNEL REINFORCEMENT
34'-0" SPAN

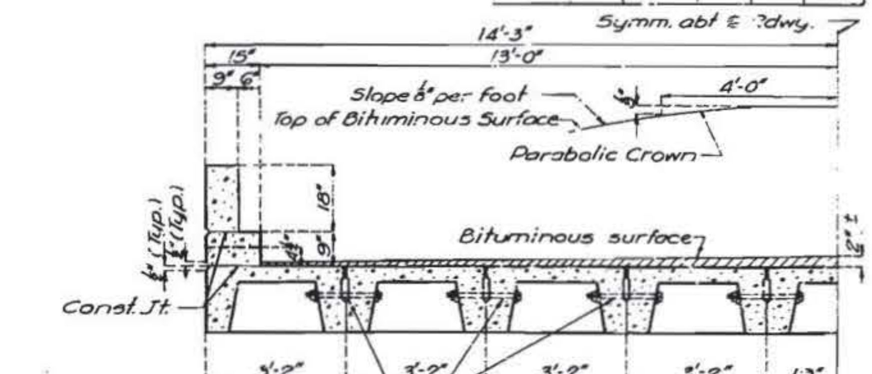
DETAILS OF MAIN CHANNEL REINFORCEMENT
27'-9" SPAN

DETAILS OF MAIN CHANNEL REINFORCEMENT
21'-6" SPAN

Note: Place 3/8" coil ties as shown in outside face of exterior channels. Place coil ties at ℄ diaphragms and 16" from each end of channel. Coil ties are to be recessed 1" from face of concrete.

Unless otherwise shown or noted all exposed edges are to be beveled 3/4".

Omit curb outlets as shown in General Elevation Sheet No. 1.



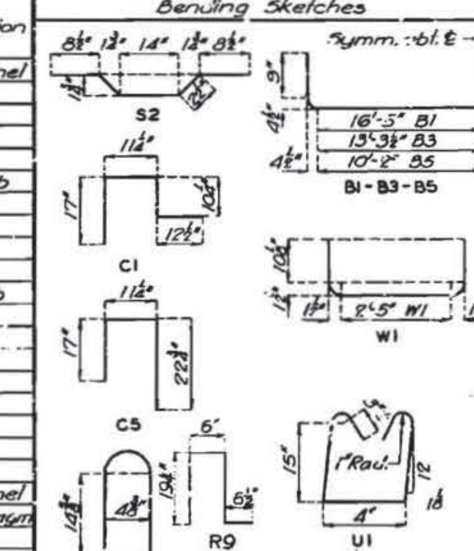
Joints to be filled with dry packed cement mortar to within 1" of top surface. Top 1" to be filled with joint seal.

1 1/4" Tie Bolts spaced as shown on channel plan. Bevel washer under Hd. & Nut. One bolt required at each set of 3 holes.

HALF SECTION THRU SPAN

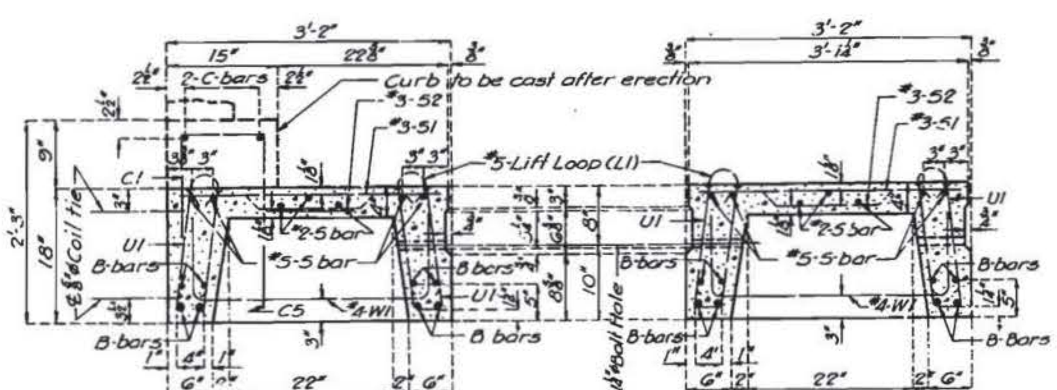
BILL OF REINFORCING STEEL FOR SUPERSTRUCTURE							
Total No.	One Span 21'-6" No.	One Span 27'-9" No.	One Span 34'-0" No.	Size	Length	Mark	Location
108			36	#9	35'-6"	B1	Channel
108			36	#9	32'-0"	B2	"
		36		#9	29'-5"	B3	"
		36		#5	20'-0"	B4	"
	36			#8	23'-0"	B5	"
192	40	52	64	#5	4'-3"	C1	Curb
12			4	#6	33'-9"	C2	"
		4		#6	27'-6"	C3	"
	4			#6	21'-3"	C4	"
36	8	10	12	#6	4'-3"	C5	"
999	216	279	333	#3	3'-0"	S1	Slab
999	207	270	333	#3	3'-0"	S2	"
162			54	#2	17'-3"	S3	"
		54		#2	14'-0"	S4	"
	54			#2	11'-0"	S5	"
108			36	#5	33'-9"	S6	"
		36		#5	27'-6"	S7	"
	36			#5	21'-3"	S8	"
1836	378	504	612	#2	3'-9"	U1	Channel
162	36	45	54	#4	4'-0"	W1	Diaphragm
108	36	36	36	#5	3'-0"	L1	Channel
8	–	–	–	#5	4'-9"	R1	End Post
4	–	–	–	#5	5'-6"	R2	" "
4	–	–	–	#5	6'-0"	R3	" "
4	–	–	–	#5	6'-6"	R4	" "
4	–	–	–	#5	7'-0"	R5	" "
8	–	–	–	#5	7'-0	R6	" "
208	–	–	–	#5	5'-3"	R7	Parapet
24	–	–	–	#5	33'-9"	R8	"
18				#5	4'-3	R9	"

Bending Sketches



Bar	a	b	c
R2	2'-1 1/4"	6"	22 1/4"
R3	2'-4 3/4"	6"	2'-1 1/4"
R4	2'-7 3/4"	6"	2'-4 1/4"
R5	2'-10 1/4"	6"	2'-7 1/4"
R6	2'-10 3/4"	6"	2'-7 3/4"
R7	2'-0 1/2"	6"	21 1/2"

R2-R3-R4-R5-R6-R7



TYPICAL SECTION THRU EXT. CHANNEL

TYPICAL SECTION THRU INT. CHANNEL

Note: #5 Lift loops to be cut off in field after units have been set in final position at bridge site and wells to be filled with grout containing iron oxide (Embeco or an approved equivalent) by Contractor, except lift loops under curbs.

BRIDGE OVER WILSON BAYOU

STATE ROAD FROM NEW MADRID NORTHEASTERLY
ABOUT 6.5 MILES N.E. OF NEW MADRID
PROJECT NO. C072-WW(1) SWW STA. 347+89.0
NEW MADRID COUNTY

No. 51265	Nov. 1964
Revised	July 1968

DETAILED MARCH 1967 BY UNDERWOOD
CHECKED MARCH 1967 BY MAGER

Note: This drawing is not to scale. Follow dimensions.

Sheet No. 3 of 4.

A-2141

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FED ROAD DIST NO	STATE	FED AID PROJ NO	FISCAL YEAR	SHEET NO	TOTAL SHEETS
5	MO		19		46

MISSOURI STATE HIGHWAY DEPARTMENT

GENERAL HANDRAIL NOTES:

All handrail posts shall be set normal to grade.

Aluminum tube handrail shall be bent to conform to vertical and horizontal alignment of parapet.

Aluminum washer shims between top of parapet and post base may be used for adjusting handrail alignment. Maximum thickness of shims to be 1/8". Where more tilting of post is required for proper alignment, concrete bearing areas shall be ground down.

All parts of handrail, except anchor bolts, nuts, washers, and set screws are to be of aluminum material.

The contract unit price per linear foot of "Bridge Rail" shall include furnishing and erecting the handrail complete with anchor bolts, shims and insulating compound.

All fillets 1/4" except as noted.

All drafts 3° except as noted.

Pipe rail to be fabricated in a minimum of 2 panel lengths.

Omit set screw on side adjacent to filled joint in parapet and curb at all expansion posts.

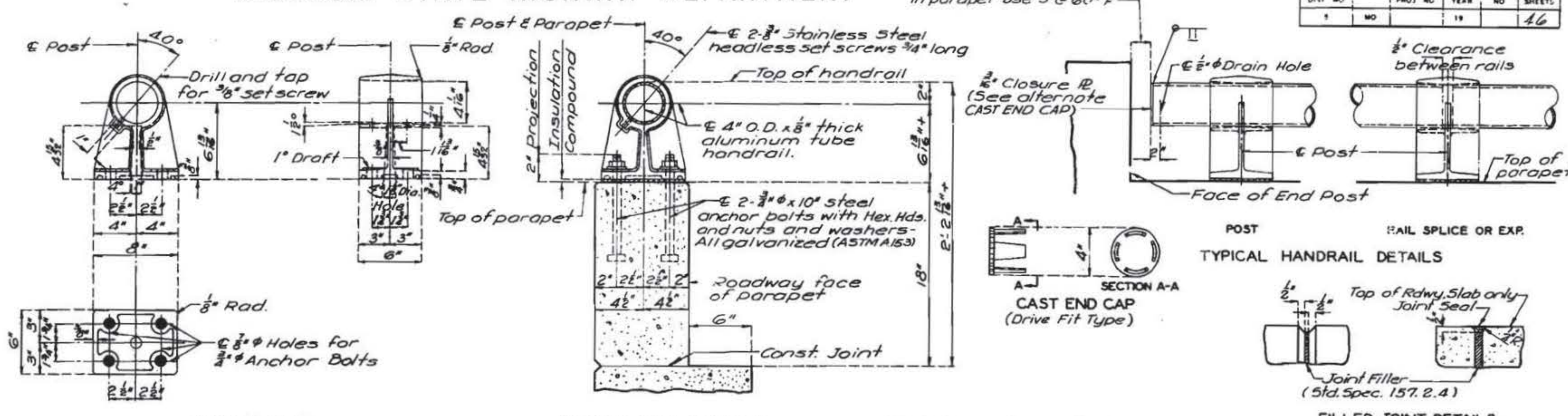
Top of curbs and parapets to be built parallel to grade with curb and parapet joints (except at end posts) normal to grade.

Concrete end posts to be vertical.

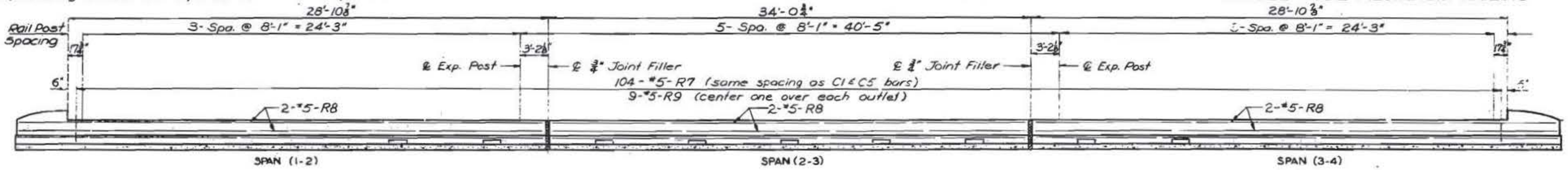
All exposed edges of end posts shall have 1/2" bevel. All exposed edges of curbs and parapets shall have 1/2" radius or 1/8" bevel unless otherwise noted.

If the contractor desires, he may use drive fit cast aluminum end caps in lieu of welded aluminum closure plates.

Integrally cast test coupons and a coat of clear lacquer specified in Std. Spec. 56.2.4 and 56.3.5 respectively will not be required for these rail posts.

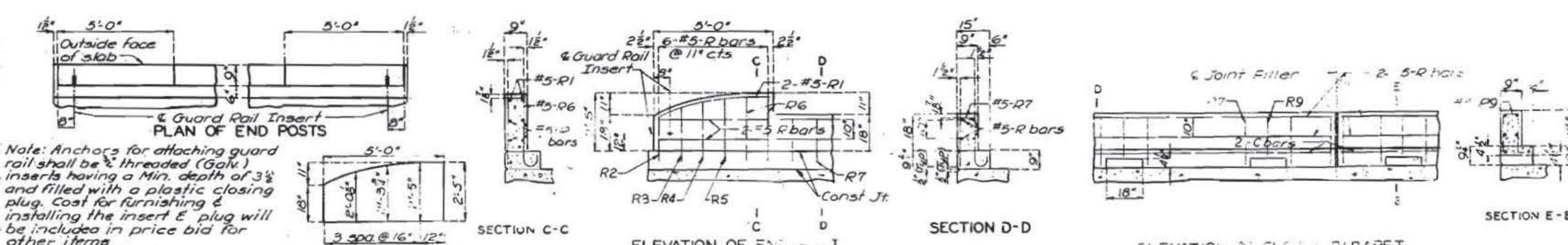


SINGLE TUBE ALUMINUM RAILING



SECTION NEAR LEFT CURB AND PARAPET

Note: See Sheet No. 3 of 4 for dimensioning of curb outlets.



Note: Anchors for attaching guard rail shall be 3/4" threaded (Galv.) inserts having a Min. depth of 3 1/2" and filled with a plastic closing plug. Cost for furnishing & installing the insert & plug will be included in price bid for other items

Note: For horizontal curb and parapet bars use a minimum lap of 15" for #5 and 18" for #6

BRIDGE OVER WILSON BAYOU

STATE ROAD FROM NEW MADRID NORTHEASTERLY

ABOUT 6.5 MILES N. E. OF NEW MADRID

PROJECT NO. C072-WW(1) SWW STA. 347 + 89.0

NEW MADRID COUNTY

DETAILED MARCH 1967 BY UNDERWOOD

CHECKED MARCH 1967 BY MAGER

Note: This drawing is not to scale. Follow dimensions

Sheet No. 4 of 4

A-2141

STD. 15.2 SQ. | MAR. 1964 | REVISED JAN. 1967

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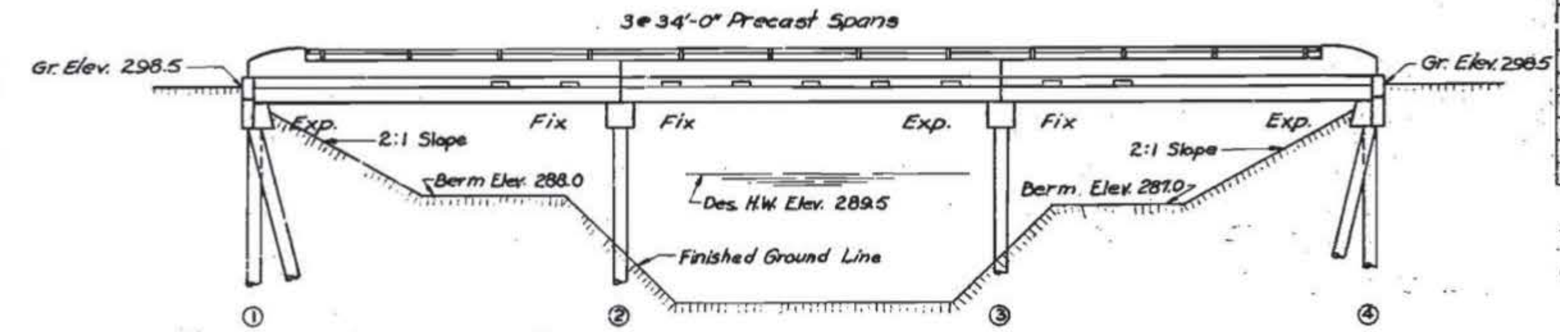
MISSOURI STATE HIGHWAY DEPARTMENT

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19		43



Note: Compacted roadway fill (full roadway width) placed up to elevation of bottom of concrete beam in front of and not less than 25'-0" in back of End Bents No. 1 and 4 before piles were driven for End Bents No. 1 and 4. Pre-bored holes for piles at End Bents No. 1 and 4.

ELEVATION

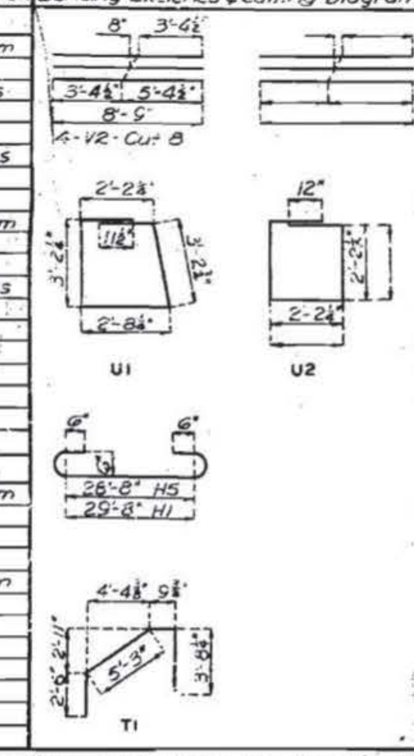
PILE DATA					
Bent No.		1	2	3	4
Type		CIP	CIP	CIP	CIP
Kind		Trestle	Trestle	Trestle	Trestle
Number		4	4	4	4
Approximate Length	Ft.	55	60	70	70
Design Bearing	Tons	30	30	30	30
Min. Tip Penetration	Elev.	265.0	260.0	260.0	265.0
Pile Standard		52.02	52.02	52.02	52.02
Hammer Energy Req'd.	Ft. Lbs.	8000	8000	8000	8000

Minimum energy requirement of hammer was based on plan length of piles.

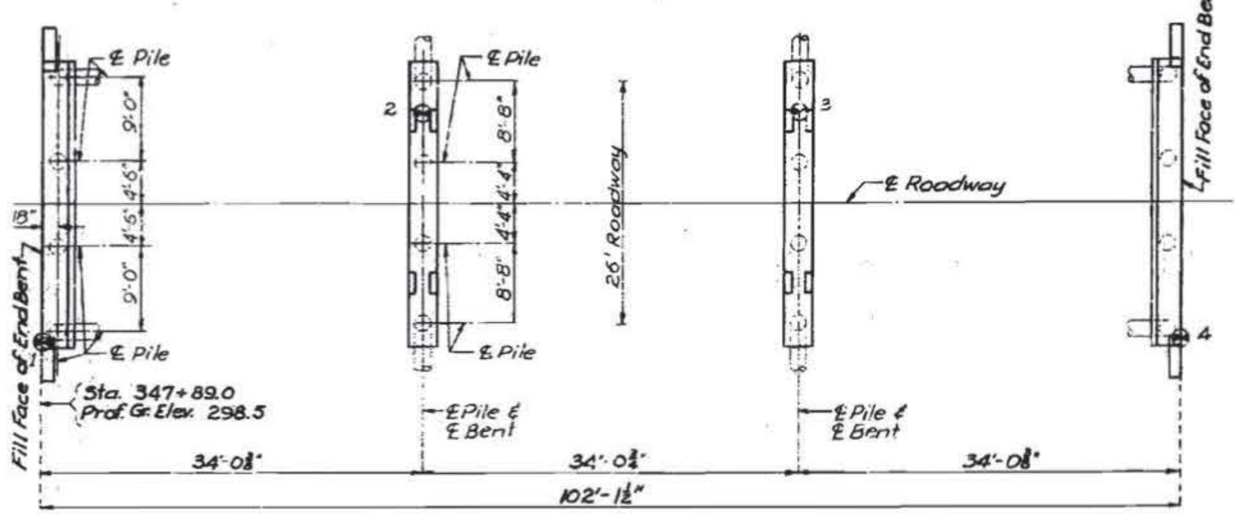
All pile were driven to the minimum penetrations and to not less than the design bearings noted.

BILL OF REINFORCING STEEL - SUBSTRUCTURE				
No.	Size	Length	Mark	Location
End Bents No. 1 & 4				
16	#6	32'-3"	H1	Beam
4	#6	30'-3"	H2	"
24	#5	5'-9"	H4	Wings
8	#6	12'-3"	T1	Wings
56	#4	12'-3"	U1	Beam
8	#4	8'-9"	T2	Wings
Int. Bents No. 2 & 3				
16	#6	31'-3"	H5	Beam
4	#6	29'-3"	H5	"
56	#4	9'-9"	U2	Beam

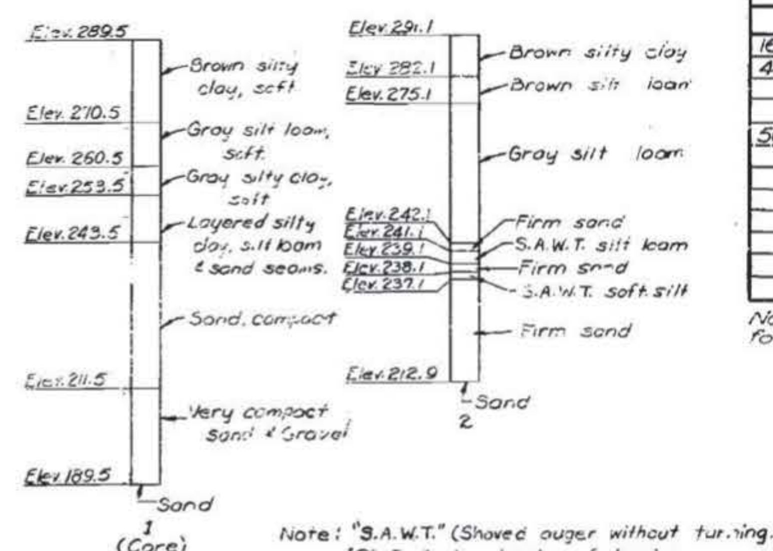
Bending Sketches & Cutting Diagrams



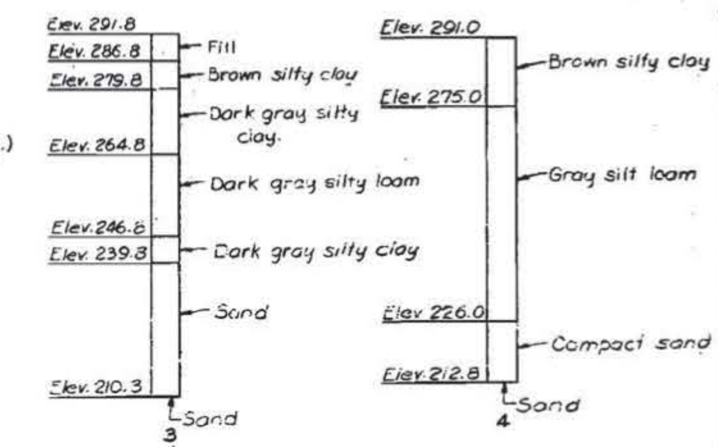
Note: See Sheet No. 3 of 4 for Bill of Reinforcing Steel for superstructure.



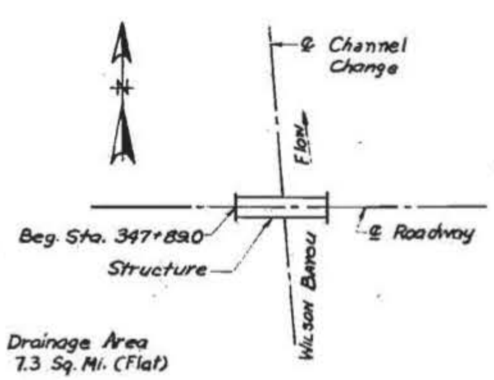
PLAN



Note: "S.A.W.T." (Shoved auger without turning.)
"⊖" Indicates location of boring.



BORING DATA



Drainage Area
7.3 Sq. Mi. (Flat)

LOCATION SKETCH

QUANTITIES				
ITEM		SUBSTR.	SUPERSTR.	TOTAL
Bituminous Surface	Sq. Yds.		295	295
Cast-in-Place Concrete Piles	Lin. Ft.	1034		1034
Class X Concrete	Cu. Yds.		88.5	88.5
Class B Concrete	Cu. Yds.	38.6	15.4	54.0
Reinforcing Steel	Lbs.	3050	37240	40290
Bridge Rail (Single tube type)	Lin. Ft.		183	183.

Note: Cost of ⅝" coil ties, 1" ϕ tie bolts and washers was included in price bid for precast units.
Cost of any required excavation for bridge was included in price bid for other items.

GENERAL NOTES:

Design Specifications: A.A.S.H.O. - 1965

Design Loading:
H15-44
Earth 120 # Equivalent Fluid Pressure 30 #.

Design Unit Stresses:
Class X Concrete (Precast Units) fc = 1,500 psi
Class B Concrete (Substructure, superstructure curb, parapet and end posts) fc = 1,200 psi
Reinforcing Steel fs = 20,000 psi
Structural Steel (A.S.T.M. A36-66) fs = 20,000 psi

Painting:
All bolts and washers for holding precast concrete units together were cleaned and painted three coats.
All exposed surfaces of steel shells for cast-in-place piles were painted. Payment was included in price bid for items painted.

B.M. #35B Elev. 299.02 L on N.E. wingwall 152' Lt. Sta. 348+91

BRIDGE OVER WILSON BAYOU

STATE ROAD FROM NEW MADRID NORTHEASTERLY
ABOUT 6.5 MILES N.E. OF NEW MADRID
PROJECT NO. C072-WW(1) SWW STA. 347+89.0
NEW MADRID COUNTY

FINISHED

SUBMITTED BY [illegible] BRIDGE ENGINEER
APPROVED BY [illegible] CHIEF ENGINEER 12/7/67

STD. 52.02

A-2141

121

No. 51.40.1 Revised June 1965 Jan. 1965

DESIGNED MARCH 1967 BY UNDERWOOD
DETAILED MARCH 1967 BY UNDERWOOD
CHECKED MARCH 1967 BY MAGER

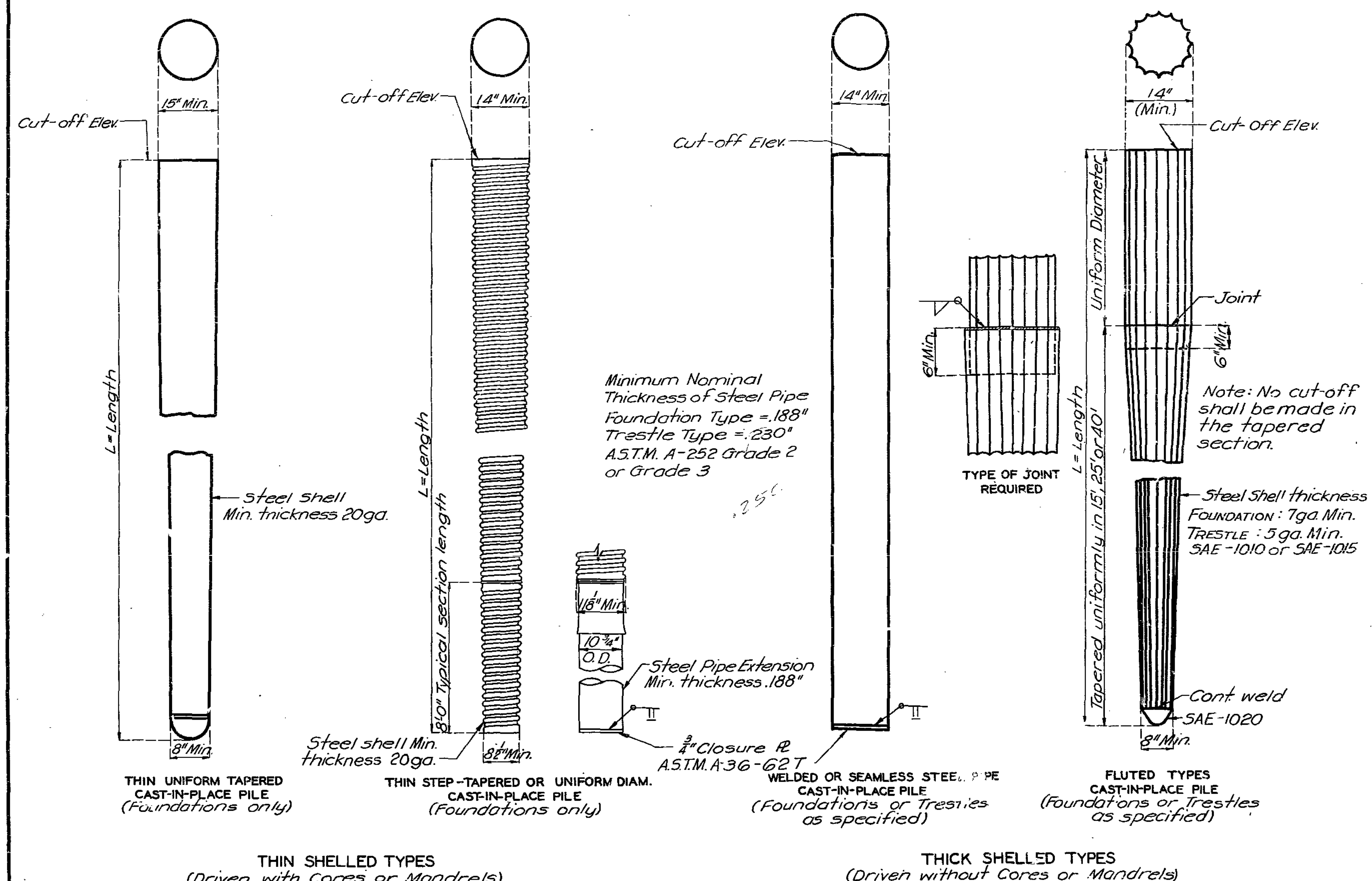
Note: This drawing is not to scale. Follow dimensions.

Sheet No. 1A of 1

FINAL PLANS

MISSOURI STATE HIGHWAY DEPARTMENT

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19		



GENERAL NOTES:

All concrete for cast-in-place piles shall be class A.

Thin Shelled Types, driven with cores or mandrels, shall have a minimum nominal average thickness of 20 ga. and shall, in every case, have such additional thickness as may be required to provide sufficient strength to withstand driving without injury and to resist harmful distortion or buckling due to soil pressure after being driven and the mandrel removed.

Thick Shelled Types, driven without cores or mandrels, Welded or Seamless steel pipes, shall meet the requirements of A.S.T.M. Specification A-252, Grade 2 or Grade 3, and the ¾" closure plates shall meet the requirements of A.S.T.M. Specification A36-62 T. Where Trestle Type Thick Shelled pile are specified, these pipes shall have a nominal average thickness of .23 inches minimum, and where Foundation Type Thick Shelled pile are specified, they shall have a nominal average thickness of .188 inches minimum.

Thick Shelled Types, driven without cores or mandrels, Fluted pipes, shall meet the requirements of Specification SAE-1010 or SAE-1015 and the forged steel tips or noses shall meet the requirements of SAE-1020. Where Trestle Type Thick Shelled pile are specified the fluted pile shall have a nominal thickness of 5 ga. minimum and where Foundation Type Thick Shelled pile are specified, they shall have a nominal thickness of 7 ga. minimum.

The minimum wall thickness of any spot or local area of any type shell shall not be more than 12.5% under the specified nominal average thickness.

If practicable, the contractor shall furnish notarized mill test reports in duplicate covering the chemical and physical properties of all steel shells. When shells are fabricated from plate material in stock which cannot be identified as to heat number, the contractor shall furnish in duplicate, a notarized statement from the shell fabricator certifying that the material used was purchased to meet a specification which fully complies with the requirements of our specifications.

Where ¾" closure plates are required for tips of pipe piles they shall not project beyond the outside diameter of the pipe piles. Satisfactory weldments may be made by beveling tip ends of pipe or by use of inside backing rings. In either case proper gaps shall be used to obtain weld penetration full thickness of pipe.

Splice details for cast-in-place concrete piles shall be in accordance with the manufacturers recommendations, subject to the approval of the engineer. All field splices to be made by qualified welders.

All splices of shells for cast-in-place concrete piles shall be made watertight and to the full strength of the shell above and below the splice to permit hard driving without damage. All shells damaged during driving shall be replaced without cost to the State. Shell sections used for splicing shall be at least 5'-0" in length and not more than two splices per pile will be permitted. The splice at top of tapered section shall be at least 3'-0" below stream bed for intermediate trestle type bents.

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APPROVED TYPES OF CAST-IN-PLACE CONCRETE PILES

SUBMITTED BY D.B. Jenkins, BRIDGE ENGINEER, DATE 5-7-1962

APPROVED BY J.J. Corbett, CHIEF ENGINEER, DATE 5-7-1962

Drawn Sept. 1959 by W.G.S.
Checked Sept. 1959 by J.E.L.

Note: This drawing is not to scale. Follow dimensions.

Revised 6-62 | 9-62 | 9-63

Sheet No. 1 of 1

52.02

FORM B-714
20M-4-61

MISSOURI STATE HIGHWAY COMMISSION

PILE DRIVING DATA

Sheet 1 of 1

County New Madrid Route WW Kind of Hammer Vulcan (1) Single Action Substr. Conc Cap on Trestle pile
Project CO 72-WW(1) Station 347+89 Weight of Hammer 5,000 Lb. Superstr. 3 @ 34'0" Precast Spans
Bridge A2141 Size Piles 14" Piles Furnished By Contractor Inspector [illegible]

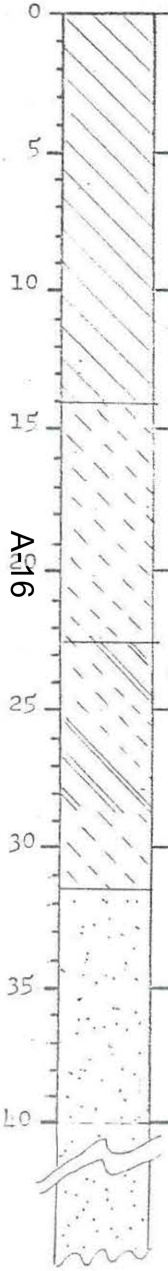
Date	Bent No.	Ftg. No.	Pile No.	Lgth. Ord.	Lgth. Used	Lgth. Spl.	In Pla.	Cut-off* Lgth.	Cut-off* Used	Cut-off* Salv.	Cut-off* Scrap	No. Blow	Av. Drop	Av. Pen.	Brg. Ton	Remarks
9-12-68	1		1	55	55		55					20	3'	.36"	30.0	battered 3"/ft
9-12-68	1		2	55	55		55					20	3'	.35"	33.3	
9-12-68	1		3	55	55		55					20	3'	.26"	41.7	
9-12-68	1		4	55	55		55					20	3'	.25"	39.5	battered 3"/ft
8-22-68	2		5	60	60	4	64					20	3'	.23"	48.7	battered 2"/ft
8-20-68	2		6	60	60		60					10	3'	.36"	32.6	
8-21-68	2		7	60	60		60					20	3'	.38"	31.3	
8-23-68	2		8	60	60	2	62					20	3'	.20"	48.0	battered 2"/ft
8-30-68	3		9	70	70	20+4	74					10	3'	.38	30.0	battered 2"/ft
9-3-68	3		10	70	70	20+3	73					20	3'	.16	57.7	
8-30-68	3		11	70	70	20+2	72					20	3'	.25	45.9	
9-3-68	3		12	70	70	20+1	71					20	3'	.28	37.9	battered 2"/ft
9-10-68	4		13	70	70	20+0	70					20	3'	.20	46.0	battered 3"/ft
9-10-68	4		14	70	70	20+0	70					20	3'	.28	39.5	
9-10-68	4		15	70	70	20+0	68	2	2 bent 3 pile 11			20	3'	.38	31.3	
9-10-68	4		16	70	70	20+0	70					20	3'	.34	36.4	battered 3"/ft

Extra Piles
Total
Splices @ 8' each
Total Pay Quantity 1034'

* If cutoff is used, show bent and pile number in "Used" column. Show cutoffs left on hand in "Salvage" column if 10' or longer, in "Scrap" column if less than 10'. If pile is used from another bridge, show in "Remarks" column. Show sketch of footings on reverse side. If freeze test is made, give time elapsed and revised bearing.
Left over piles stored at ____

12-0255

001928



Depth & Description	Wn	LL	PI	% Silt	% Clay	% -200	Torvane, TSF	* Pock. Pen., TSF	*** qu/2 psf	ǂ ø°	ǂ c psf	Consolidation Data: Cc	C_v 10^{-2} ft²/D	P_1 ksf	P_c ksf	P_2 ksf	e_o	e_c	e_2
285.9																			
0-14±' Brown to gray clay, mottled.																			
@ 6'	33.8							2.0	1100										
@ 11'		85	55					2.0	215*										
14±'-22.5' Mottled gray to gray silt. @ 16'	28.6	(Non-plastic)						0.3	180										
@ 21'	30.0	(Non-plastic)						0.4	1130										
22.5-31.5' Sandy loam and dense silt. @ 26'	29.3	(Non-plastic)					0.08	0.25-0.5	600										
@ 28'	40.9																		
31.5' Sand. Discontinued in sand at 51.5'. @ 31'	14.3 23.3																		

Project No. C072-WW(1)
County New Madrid Br. No. A-2140
Route WW
Station 237+10, 14' Lt.
Fill Height 20'

* Pocket penetrometer reading, TSF
** Standard penetration test
*** Unconfined compression test
ǂ Direct shear test
▽ Water table
* Slickensided failure plane

Figure 1 of 2

Depth & Description 289.5	Wn	LL	PI	% Silt	% Clay	% -200	** n blows per foot	* Pock. Pen., TSF	*** qu/2 psf	ǂ ø°	ǂ c psf	Consolidation Data Cc	C_v 10^{-2} ft^2/D	P_1 ksf	P_c ksf	P_2 ksf	e_o	e_c	e_2
0-19±' Brown silty clay grading to silt loam with depth with some clay and sand streaks.																			
@ 6'	30.2							.75	1640										
@ 11' W.L. @ 12'	41.1 43.9	67	43					1.5	680										
@ 16'	36.4 41.1							0.5	465										
19±-28±' Gray silt loam. @ 21'	35.3 32.5	32	5					0.5	680										
@ 26'	33.6 35.5							0.8	580										
28±-36' Gray silty clay.																			
@ 31'	43.1 33.6	50	29					0.75	920										
@ 36' 36-45' Gray silty clay loam with silt and sand seams.	38.9 40.8								750										

(continued)

Project No. C072-WW(1) Br. No. A-2141

County New Madrid

Route WW

Station 347+89, 14' Rt.

Fill Height 11' (15' max.)

* Pocket penetrometer reading, TSF
** Standard penetration test
*** Unconfined compression test
ǂ Direct shear test
∇ Water table

Page 1 of 2

Figure 2 of 2

B-705 3M SETS 5-64

LOG OF SOUNDINGS

Sheet 1 of 2

NEW MADRID County. Route WW Design No. A-2140

Project No. C072-WW(1) Over East Bayou

Soundings by Klick, Thomas & Shouse Date of Report February 21, 1967

Station	Loc. †	Surf. Elev.	Depth	Elev. Bott.	Bottom on	Log of Materials*
237+10	14'Lt.	285.9	51.5	234.4	Sand	0-14'± Brown and gray mottled clay, medium. 14±-31.5' Mottled to gray silt loam, loose to firm. 31.5-51.5' Sand.
Opr.: Hoecker						
Water at 9' after 20 hrs.						
Standard Penetration Tests						
Depth	Blows/6"					
35'	23/22/26					
40'	20/26/34					
45'	24/29/34					
50'	28/45/47					
237+75	9'Lt.	276.3	78.2	198.1	Sand with some gravel	0-13' Brown clay, soft. 13-19' Sand and silt layers. 19-65' Firm blue sand. 65-78.2' Sand with some gravel.
237+75	9'Rt.	274.6	94.0	180.6	Sand with some gravel	0-11' Brown silt loam, soft. 11-18' Gray silt and small sand layers. 18-64' Sand. 64-94' Coarse sand with some gravel.
239+24	14'Lt.	277.5	68.2	209.3	Sand and gravel	0-7.5' Brown silty clay, (auger shoved). 7.5-15' Gray silt loam, (auger shoved). 15-18.9' Gray silt loam, (auger shoved intermittentl 18.9-65.0' Gray sand, medium. 65.0-68.2' Sand and gravel.

†Distances given from centerline are perpendicular thereto unless otherwise noted.

Instructions to Reporter: Describe equipment used, and where, and give accurate log of operations.

BLUE – BRIDGE OFFICE
BUFF – B.P.R.
WHITE – PROJECT ENGINEER
PINK – DISTRICT OFFICE

***Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgment of the operator. Log of operations is an integral part of this information. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.**

TRI-COUNTY PUBLICATIONS, BELLE, MO.

B-705 3M SETS 5-64

LOG OF SOUNDINGS

NEW MADRID County. Route WW Design No. A-2140

Project No. 0072-WW(1) Over East Bayou

Soundings by Klick, Thomas & Shouse Date of Report February 21, 1967

Station	Loc. †	Surf. Elev.	Depth	Elev. Bott.	Bottom on	Log of Materials*
230.87	9' Rt.	275.6	67.5	208.1	Sand and gravel	0-9' Brown silty clay, (auger shoved). 9-19.5' Gray silt, (auger shoved intermittently). 19.5-64.3' Gray sand. 64.3-67.5' Sand and gravel.

NOTE: South side of Bayou not accessible to core drill due to soft ground. Penetrations could not be secured on this side.

†Distances given from centerline are perpendicular thereto unless otherwise noted.

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TRI-COUNTY PUBLICATIONS, BELLE, MO.

B-705 3M SETS 5-64

LOG OF SOUNDINGS

NEW MADRID County. Route WW Design No. A-2141

Project No. S072-WW(1) Over Wilson Bayou

Soundings by Klick, Thomas & Thouse Date of Report February 21, 1967

Station	Loc. †	Surf. Elev.	Depth	Elev. Bott.	Bottom on	Log of Materials*
347+89	14'Rt.	289.5	100.0	189.5	Sand	0-19'± Brown silty clay, soft.
Opr.: Moecker						19±-29'± Gray silt loam, soft.
Core No.: M-67-16						29±-36' Gray silty clay, soft.
Standard Penetration Test						36-46' Layered silty clay, silt loam and sand seams.
Depth	Blows/6"					46-78' Sand, compact.
						78-100' Very compact sand and gravel.
50'	17/20/27					
55'	19/28/36					
60'	27/52/59					
65'	32/48/55					
NOTE: Undisturbed samples lifted from 5-44'.						
347+89	14'Lt.	289.3	78.2	211.1	Sand	0-9' Brown silty clay, S.A.W.T.
Opr.: Klick						9-16' Brown silt loam, S.A.W.T., free water at 16'.
3" Auger						16-47.9' Gray silt loam, S.A.W.T.
						42.9-43.9' Firm layer, no S.A.W.T.
						43.9-47.6' Firm layer, S.A.W.T.
						47.6-48.6' Firm layer, no S.A.W.T.
						48.6-51.9' Firm layer, S.A.W.T.
						51.9-63.0' Firm sand.
						63.0-65.0' Sand and few layers of gravel.
						65.0-78.2' Compact sand.

†Distances given from centerline are perpendicular thereto unless otherwise noted.

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TRI-COUNTY PUBLICATIONS, BELLE, MO.

B-705 3M SETS 5-64

LOG OF SOUNDINGS

NEW MADRID County. Route WW Design No. A-2141

Project No. C072-WW(1) Over Wilson Bayou

Soundings by Klick, Thomas & Shouse Date of Report February 21, 1967

Station	Loc. †	Surf. Elev.	Depth	Elev. Bott.	Bottom on	Log of Materials*
348+23	9'Lt.	291.1	78.2	212.9	Sand	0-9.0' Brown silty clay. 9.0-16.0' Brown silt loam. 16.0-49.0' Gray silt loam. 49.0-50.0' Firm sand. 50.0-52.0' S.A.W.T. silt loam. 52.0-53.0' Firm sand. 53.0-54.0' S.A.W.T. soft silt 54.0-78.2' Firm sand.
348+57	9'Lt.	291.8	81.5	210.3	Sand	0-5.0' Fill. 5.0-12.0' Brown silty clay. 12.0-27.0' Dark gray silty clay. 27.0-45.0' Dark gray silt loam. 45.0-52.0' Dark gray silty clay. 52.0-81.5' Sand.

Sta. 348+23:

Opr.: Klick 3" Auger

NOTE: S.A.W.T. - Shoved auger without turning.

Sta. 348+57:

Core No.: F-7-7

Opr.: McBride

Standard Penetration Tests

Depth	Blows/6"
10'	0/0/1
15'	1/1½/1½
20'	1/1½/1½
25'	0/0/1
30'	1/1/1½
35'	1½/1/1½
40'	1½/1/1½
45'	2/3/2
50'	3/2/3
55'	4/4/8
60'	5/5/8
65'	6/10/11
70'	16/22/26
75'	25/31/32
80'	30/43/30

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WHITE – PROJECT ENGINEER
PINK – DISTRICT OFFICE

TRI-COUNTY PUBLICATIONS, BELLE, MO.

B-705 3M SETS 5-64

LOG OF SOUNDINGS

Sheet 3 of 3

NEW MADRID County. Route WW Design No. A-2141

Project No. C072-3W(1) Over Wilson Bayou

Soundings by Klick, Thomas & Thomas Date of Report February 21, 1967

Station	Loc. †	Surf. Elev.	Depth	Elev. Bott.	Bottom on	Log of Materials*
349+03	12'Rt.	292.1	100.0	192.1	Sand	0-14.9' Brown silty clay. 14.9-51.6' Blue silty clay. 51.6-66.5' Sand layers and clay layers. 66.5-70.0' Clay. 70.0-73.0' Sand. 73.0-80.0' Fine sand. 80.0-80.3' Layer of small gravel. 80.3-85.0' Coarse sand. 85.0-94.5' Coarse sand with small gravel. 94.5-100.0' Coarse sand with small gravel.
Core No.:	K-7-B					
Opr.: McBride						
Standard Penetration Test						
Depth	Blows/6"					
70'	5/4/2					
75'	12/13/12					
80'	36/33/23					
85'	30/38/39					
90'	32/40/41					
NOTE: Omitted penetrations to 70 feet in view of low blow counts on bent 3 and ability to shove augers without turning and with little resistance.						
348+94	14'Rt.	291.0	78.2	212.8	Sand	0-16.0' Brown silty clay. 16.0-65.0' Gray silt loam. 65.0-78.2' Compact sand.
Opr.: Klick						
3" Auger						
NOTE: S.A.W.T. to 65.0'.						

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TRI-COUNTY PUBLICATIONS, BELLE, MO.

MEMORANDUM

Missouri Department of Transportation

Construction - Materials
Central Laboratory

TO: Michele Atkinson-br

CC/ATT: Bill Dunn-br
Andrew Meyer-se/cm
Kevin Plott-se/cm
Corbin Carlton-se/cm

FROM: Thomas W. Fennessey
Geotechnical Engineer

DATE: March 21, 2016

SUBJECT: Materials
Geotechnical Section
Foundation Investigation for
Structure No. A8472
Job No. J9S3146
Route WW, New Madrid County

General - A foundation investigation has been performed for the above referenced structure as requested in an email from Michelle Atkinson dated November 16, 2015. This project site is located in New Madrid County where Route WW crosses over Wilson Bayou about 6.5 miles northeast of New Madrid, Missouri.

While no formal Sounding Request has been provided, it is understood that the existing 102-foot long bridge at this site, Structure No. A2141 is to be replaced on essentially the same grade and alignment by a proposed similar length bridge, Structure No. A8472. Per existing bridge plans, the existing structure is supported on pile foundations. It is anticipated that the proposed structure will similarly be supported on pile foundations.

Field Investigation – As indicated in Table 1 below, subsurface exploration was recently performed at two locations at this site. One cone penetration test (CPT) boring, H-16-22, was performed near the west end of the existing structure using Hogentogler CPT track-mounted equipment. One standard penetration test (SPT) boring, A-16-14, was performed near the east end of the existing structure using Failing 1500 truck-mounted equipment.

Table 1 – 2016 Subsurface Exploration Locations

Subsurface Exploration Location	Comment
Sta. 347+97.0, 23.0R, Elev. 288.9 ft.	CPT Boring, H-16-22, Northing: 280453.1, Easting: 1138135.7
Sta. 907+63.9, 16.4R, Elev. 279.9 ft.	SPT Boring, A-16-14, Northing: 280465.6, Easting: 1138267.6

A review of this recent subsurface exploration data indicated somewhat different subsurface conditions exist at these two locations. At the west end of the site, generally soft cohesive soils were found to overlie dense sand at about Elev. 235 ft. while at the east end of the site, generally soft cohesive soils were found to overlie dense sands at about Elev. 222 ft. Accordingly, the subsurface exploration performed in 1967 for the existing bridge was also reviewed and found to generally agree with and complement the recent subsurface exploration. The combined data generally indicate that the dense sand is higher to the west of about Sta. 348+60 and lower to the east of about Sta. 348+90.

Therefore, previous subsurface exploration data from the additional six locations listed in Table 2 below are included in this report. However, elevations of these previous borings were adjusted to match recent survey data from this site.

Table 2 – 1967 Subsurface Exploration Locations

Subsurface Exploration Location	Comment
Sta. 347+89.0, 14.0L, Elev. 288.8 ft.	Northing: 280493.2, Easting: 1138125.4
Sta. 347+89.0, 14.0R, Elev. 289.0 ft.	Northing: 280465.2, Easting: 1138126.1
Sta. 348+23.0, 9.0L, Elev. 290.6 ft.	Northing: 280489.2, Easting: 1138159.5
Sta. 348+57.0, 9.0L, Elev. 291.3 ft.	Northing: 280489.8, Easting: 1138193.5
Sta. 348+94.0, 14.0R, Elev. 290.5 ft.	Northing: 280467.7, Easting: 1138231.1
Sta. 349+03.0, 12.0R, Elev. 291.6 ft.	Northing: 280493.9, Easting: 1138239.5

The subsurface exploration locations for both the 2016 and the 1967 site investigations are shown with respect to the site on Figure 1 – Subsurface Exploration Location Aerial. A subsurface diagram showing the subsurface exploration conditions encountered in 2016 with respect to stationing is attached as Figure 2 – Subsurface Diagram – 2016 Data. For comparison, a subsurface diagram showing the subsurface exploration conditions encountered in 1967 with respect to stationing is attached as Figure 3 – Subsurface Diagram – 1967 Data. Summary sheets providing input parameters for software programs LPile and Driven are provided for the two recent subsurface exploration locations. Logs of the individual 2016 and 1967 subsurface exploration locations are also attached along with a recent grain size distribution graph.

Analyses - Attached are plots of preliminary pile capacity graphs for 14-in. and 16-in. diameter cast-in-place steel pipe piles showing ultimate pile capacity and factored pile capacity. For H-16-22 at the west end of the site, these graphs are based upon CPeT-IT v.1.7.6.42 software using LCPC Method and a resistance factor of 0.45. For A-16-14, these graphs are based upon DRIVEN 1.2 software using α Method and a resistance factor of 0.35 for cohesive soils and Nordlund Method and a resistance factor of 0.45 for non-cohesive soils. These pile capacity graphs assume the top of pile is at existing ground surface at these locations and do not account for any soil loss due to scour.

cs
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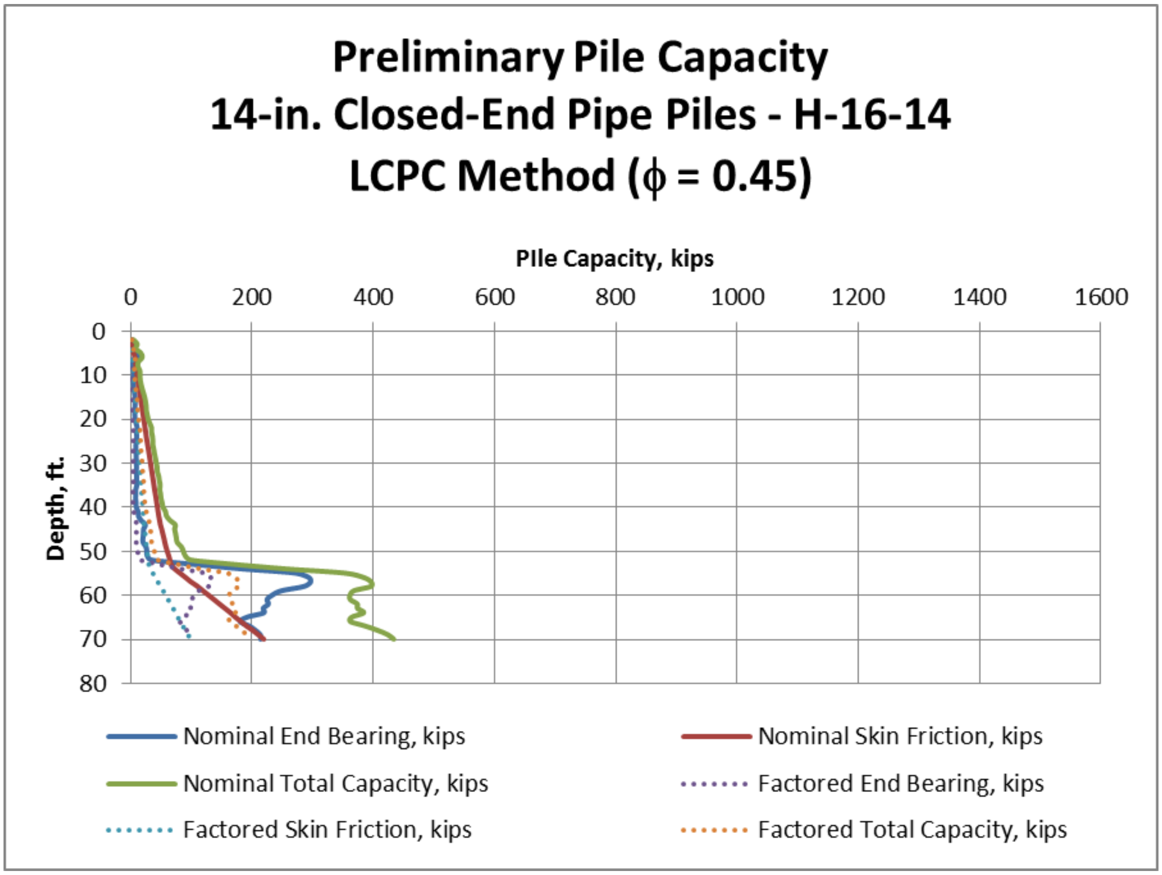
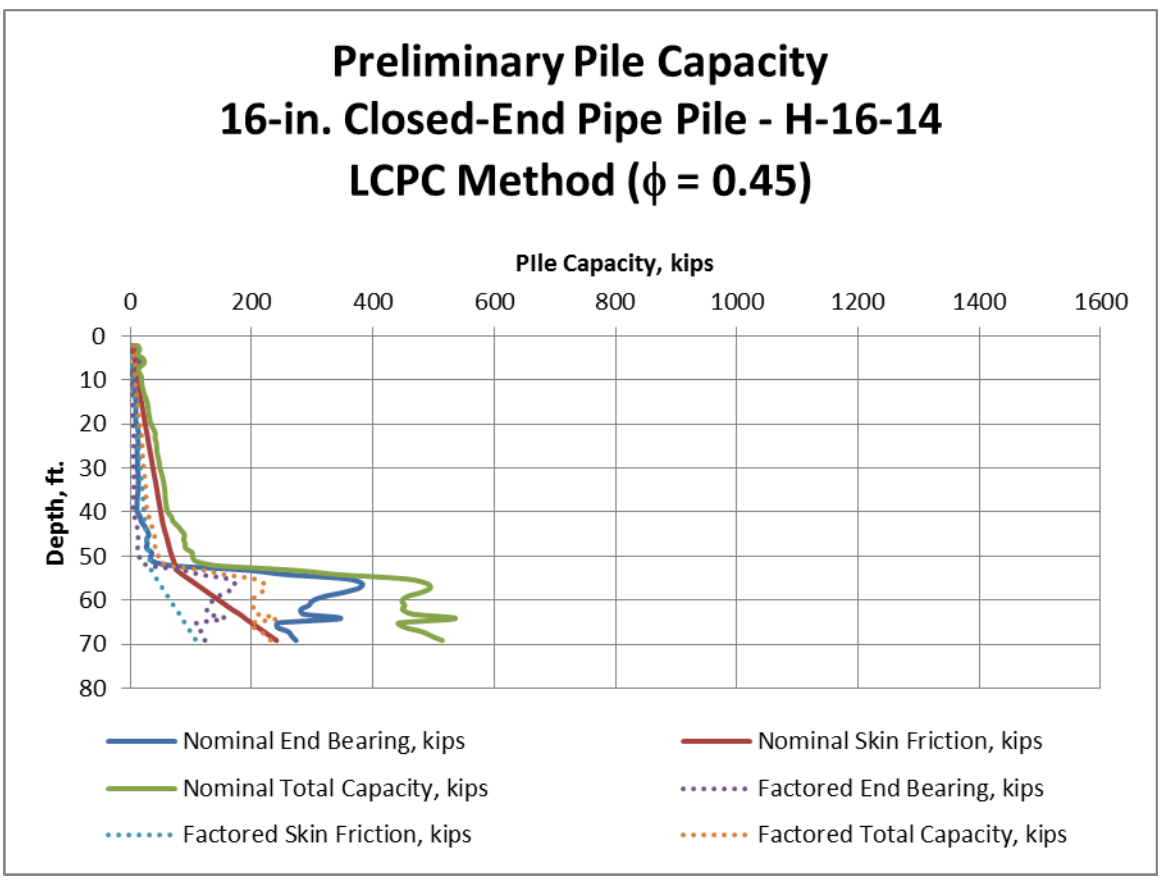
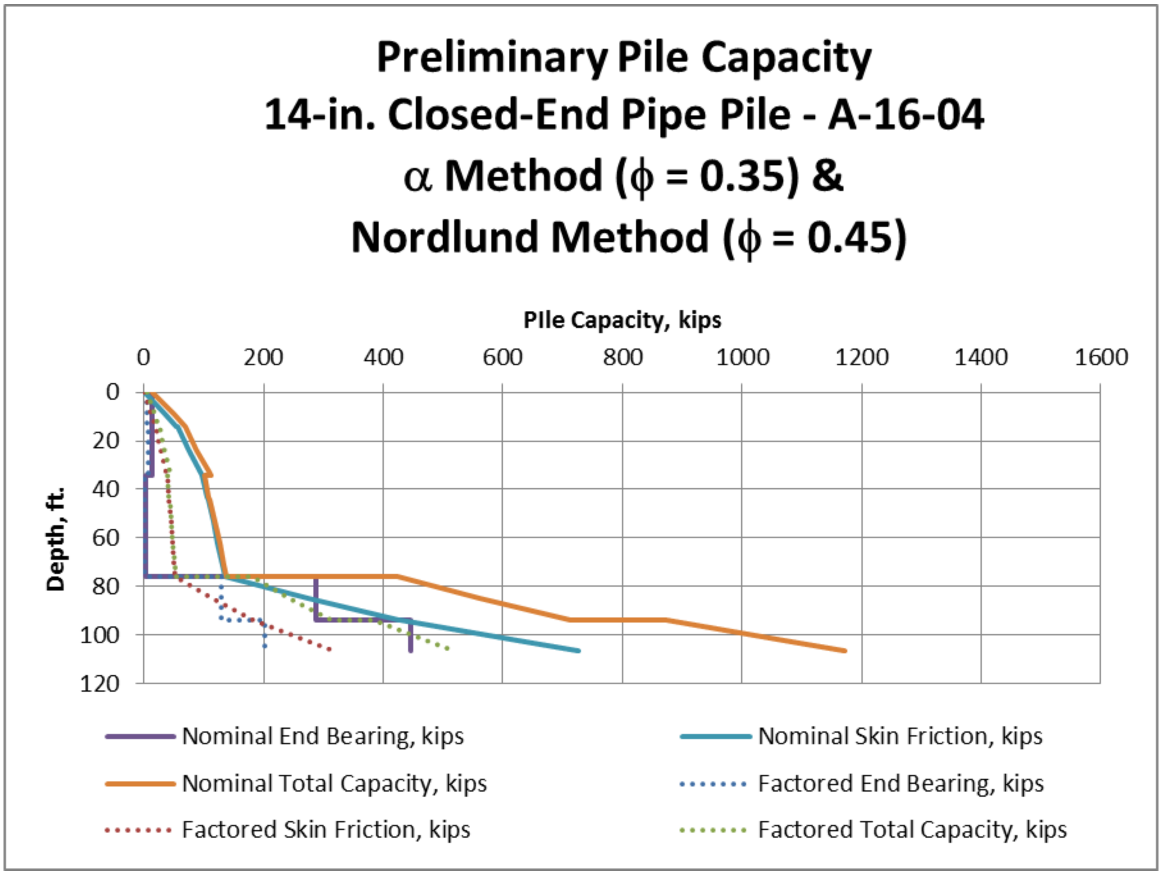
Attachments


Preliminary Pile Capacity
14-in. Closed-End Pipe Piles - H-16-14
LCPC Method (ϕ = 0.45)
PIle Capacity, kips
0
200
400
600
800
1000
1200
1400
1600
Depth, ft.
0
10
20
30
40
50
60
70
80
Nominal End Bearing, kips
Nominal Skin Friction, kips
Nominal Total Capacity, kips
Factored End Bearing, kips
Factored Skin Friction, kips
Factored Total Capacity, kips


Preliminary Pile Capacity
16-in. Closed-End Pipe Pile - H-16-14
LCPC Method (ϕ = 0.45)
PIle Capacity, kips
0
200
400
600
800
1000
1200
1400
1600
Depth, ft.
0
10
20
30
40
50
60
70
80
Nominal End Bearing, kips
Nominal Skin Friction, kips
Nominal Total Capacity, kips
Factored End Bearing, kips
Factored Skin Friction, kips
Factored Total Capacity, kips


Preliminary Pile Capacity
14-in. Closed-End Pipe Pile - A-16-04
α Method (ϕ = 0.35) &
Nordlund Method (ϕ = 0.45)
PIle Capacity, kips
0
200
400
600
800
1000
1200
1400
1600
Depth, ft.
0
20
40
60
80
100
120
Nominal End Bearing, kips
Nominal Skin Friction, kips
Nominal Total Capacity, kips
Factored End Bearing, kips
Factored Skin Friction, kips
Factored Total Capacity, kips

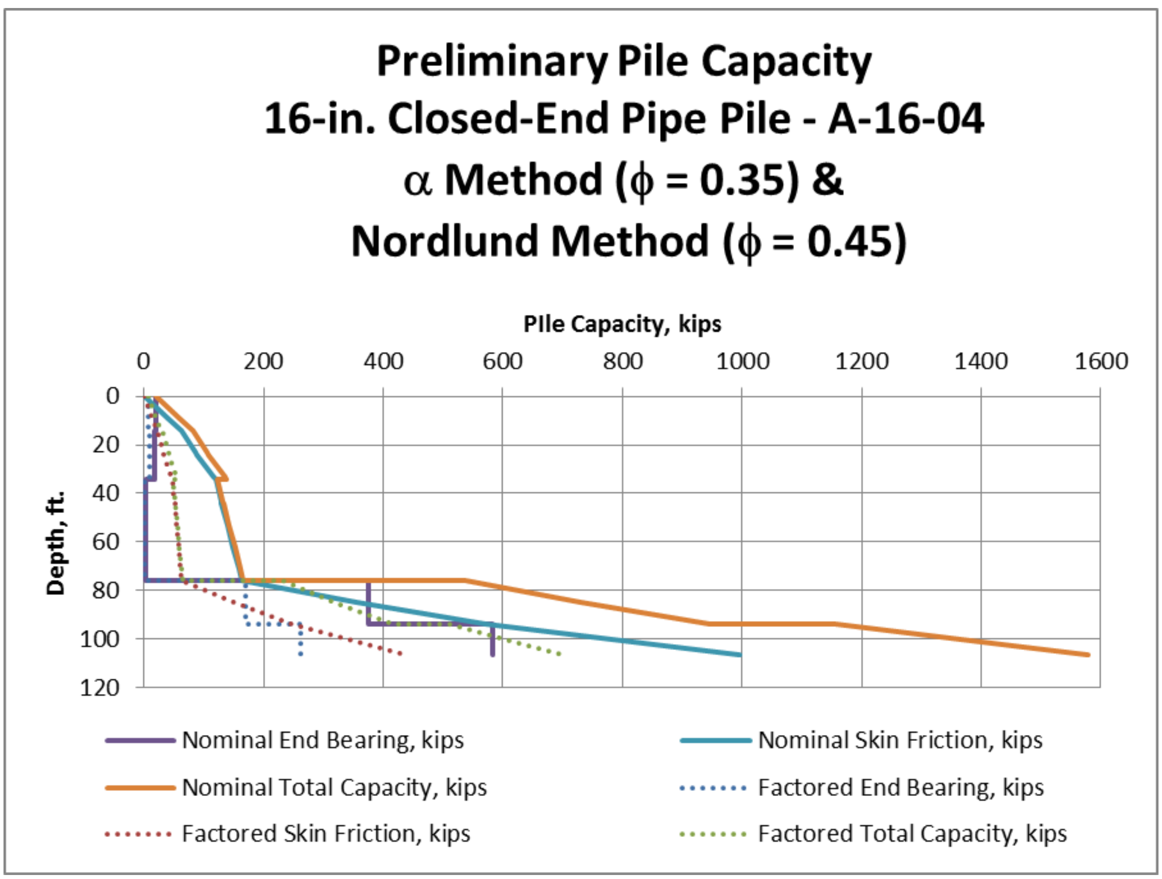

Preliminary Pile Capacity
16-in. Closed-End Pipe Pile - A-16-04
α Method (ϕ = 0.35) &
Nordlund Method (ϕ = 0.45)
PIle Capacity, kips
0
200
400
600
800
1000
1200
1400
1600
Depth, ft.
0
20
40
60
80
100
120
Nominal End Bearing, kips
Nominal Skin Friction, kips
Nominal Total Capacity, kips
Factored End Bearing, kips
Factored Skin Friction, kips
Factored Total Capacity, kips

Figure 1 - Subsurface Exploration Location Aerial

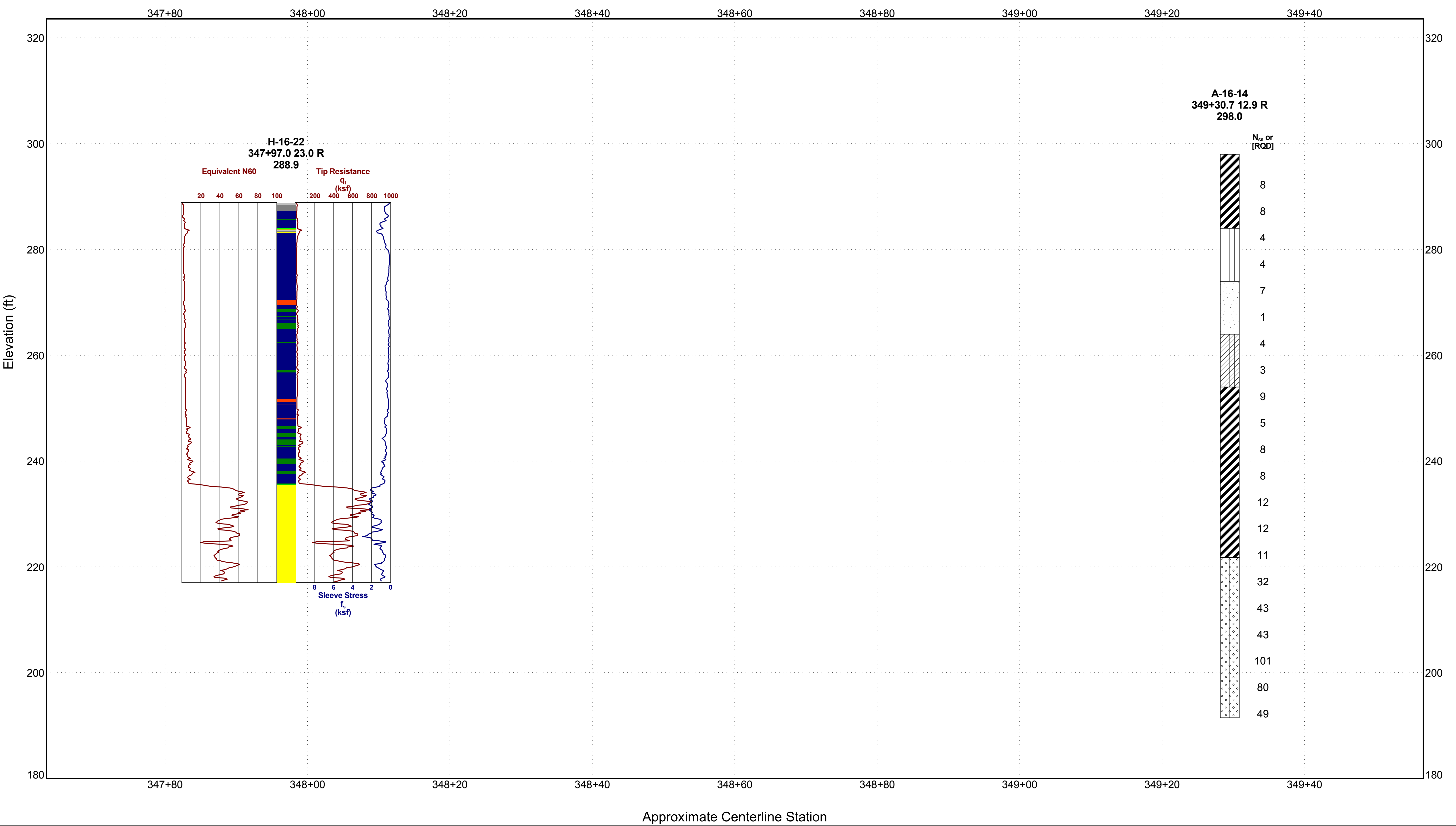
MoDOT - Geotechnical Section
1617 Missouri Boulevard
Jefferson City, Missouri 65109

CPT MATERIAL GRAPHICS
Sensitive, Fine Grained Soils
Organic Soils, Peats
Clays-Clay to Silty Clay
Silt Mixtures-Clay Silt to Silty Clay
Sand Mixtures-Silty Sand to Sandy Silt
Sands-Clean Sand to Silty Sand
Gravelly Sand to Sand
Very Stiff Clay to Clayey Sand
Very Stiff Fine Grained Soils
Robertson et al (1990) Q_t vs F_r - MAI =

FIGURE 2 - SUBSURFACE DIAGRAM - 2016 DATA

PROJECT NAME A8472
PROJECT LOCATION Rt. WW over Wilson Bayou
CLIENT MoDOT Bridge Division - EFK Moen, LLC
PROJECT NUMBER J9S3146

USCS High Plasticity Clay
USCS Low Plasticity Silty Clay
USCS Silt
USCS Well-graded Sand with Silt
USCS Poorly-graded Sand

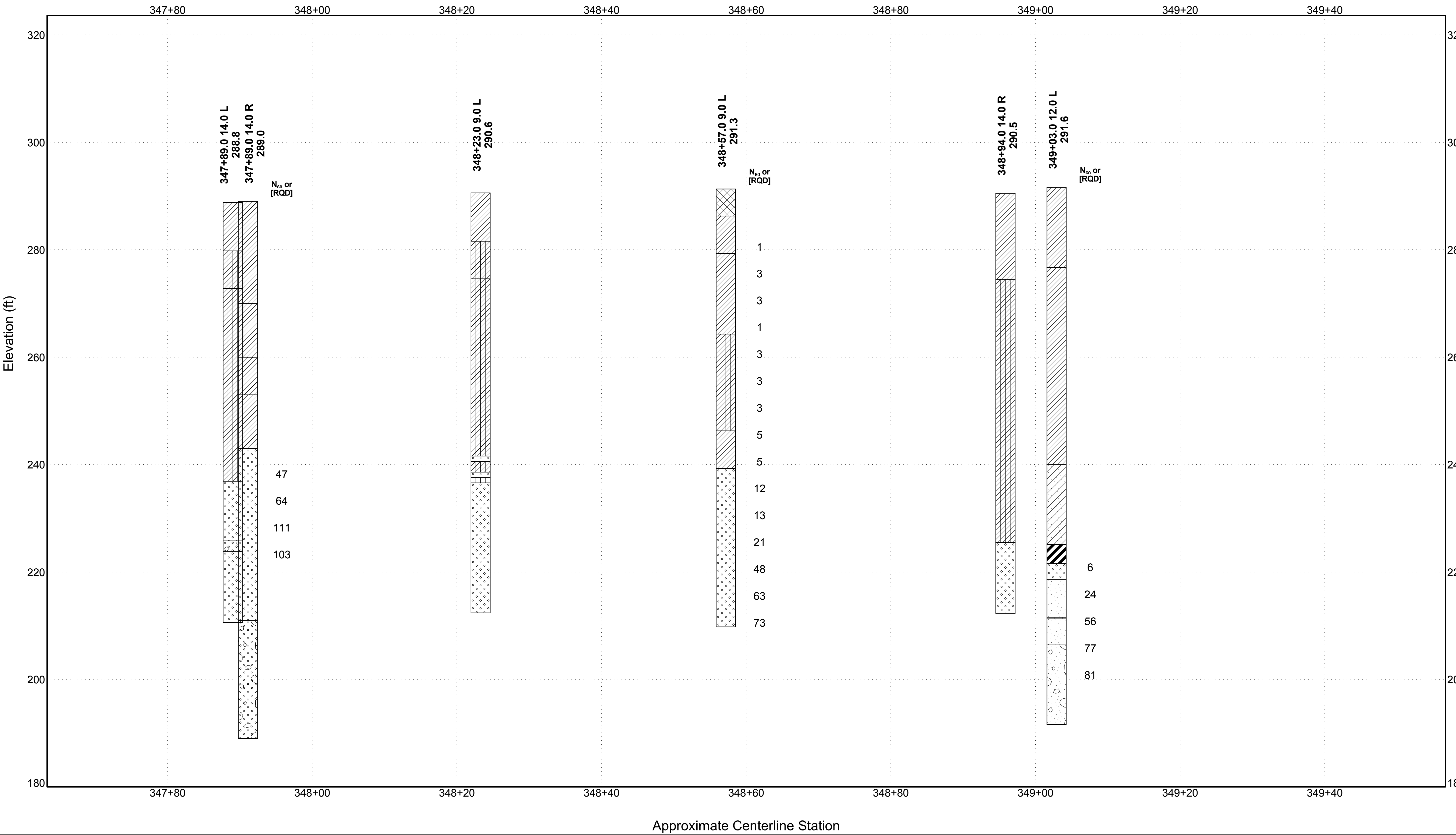


MoDOT - Geotechnical Section
1617 Missouri Boulevard
Jefferson City, Missouri 65109

FIGURE 3 - SUBSURFACE DIAGRAM - 1967 DATA

PROJECT NAME A8472
PROJECT LOCATION Rt. WW over Wilson Bayou
CLIENT MoDOT Bridge Division - EFK Moen, LLC
PROJECT NUMBER J9S3146

USCS Low Plasticity Clay
USCS Well-graded Gravelly Sand
USCS Clayey Sand
USCS Poorly-graded Gravel
USCS Low Plasticity Silty Clay
USCS Silt
USCS High Plasticity Clay
USCS Poorly-graded Gravelly Sand
USCS Well-graded Sand
Fill (made ground)
USCS Poorly-graded Sand



MODOT DYNAMIC FENCE - MODOT 20150728.GDT - 3/16/16 11:28 - J:\SGIGINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

Missouri Department of Transportation
Construction and Materials

BORING NO. H-16-22

Job No.: J9S3146	**County:** New Madrid	**Route:** WW	**Logged By:** Ricardo Todd
Design: A8472	**Skew:**	**Coordinate Units:** U.S. Survey Feet	**Operator:** Mike Donahoe
Bent:	**Location:** Rt. WW over Wilson Bayou	**Coordinate Proj. Factor:**	**Date of Work:** 01/27/16
Station: 347+97.0	**Northing:** 280453.1	**Coordinate System:** U.S. State Plane 1983	**Depth to Water:**
Offset: 23.0 R	**Easting:** 1138135.7	**Coordinate Datum:** NAD 83 (CONUS)	**Time Change:**
Elevation: 288.9	**Drilling Method:**	**Coordinate Zone:** Missouri East	**Depth Hole Open:**
Drill No.: G-8929	**Hammer Efficiency:**	**Equipment:** Hogentogler CPT ,	

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle φ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
0		0 - 48.2' Soft clay																
							111(1)	111(1)	600(1)			5(1)		0.015	75			
10			280				111(1)	111(1)	175(1)			2(1)		0.02	30			
	▽(1)						111(1)	49(1)	375(1)			3(1)		0.02	30			
			270	yes	yes	yes												
20							111(1)	49(1)	400(1)			3(1)		0.02	30			
							111(1)	49(1)	400(1)			3(1)		0.02	30			
30			260				114(1)	52(1)	450(1)			4(1)		0.15	50			

LPILE SUMMARY - MODOT_20150812.GDT - 3/15/16 12:34 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

(1) = Assumed, (2) = Actual, (3) = Phi' 0

(Continued Next Page)

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW	**Logged By:** Ricardo Todd
Design: A8472	**Skew:**	**Coordinate Units:** U.S. Survey Feet	**Operator:** Mike Donahoe
Bent:	**Location:** Rt. WW over Wilson Bayou	**Coordinate Proj. Factor:**	**Date of Work:** 01/27/16
Station: 347+97.0	**Northing:** 280453.1	**Coordinate System:** U.S. State Plane 1983	**Depth to Water:**
Offset: 23.0 R	**Easting:** 1138135.7	**Coordinate Datum:** NAD 83 (CONUS)	**Time Change:**
Elevation: 288.9	**Drilling Method:**	**Coordinate Zone:** Missouri East	**Depth Hole Open:**
Drill No.: G-8929	**Hammer Efficiency:**	**Equipment:** Hogentogler CPT ,	

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle φ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
		0 - 48.2' Soft clay *(continued)*					114(1)	52(1)	450(1)			4(1)		0.15	50			
			250															
40							111(1)	49(1)	500(1)			5(1)		0.15	65			
				yes	yes	yes												
							114(1)	52(1)	1200(1)			7(1)		0.008	400			
		48.2 - 53.7' Stiff clay with free water	240															
50							114(1)	52(1)	1600(1)			10(1)		0.007	600			
				yes	yes	yes												
		53.7 - 71.9' Sand					127(1)	65(1)				63(1)	44(1)		125			
			230															
60							124(1)	62(1)				42(1)	41(1)		125			
				no	no	yes												
							124(1)	62(1)				43(1)	40(1)		125			
			220															
70																		

(1) = Assumed, (2) = Actual, (3) = Phi' 0

(Continued Next Page)

Missouri Department of Transportation
Construction and Materials

BORING NO. H-16-22

Job No.:	J9S3146	**County:**	New Madrid	**Route:**	WW	**Logged By:**	Ricardo Todd
Design:	A8472	**Skew:**		**Coordinate Units:**	U.S. Survey Feet	**Operator:**	Mike Donahoe
Bent:		**Location:**	Rt. WW over Wilson Bayou	**Coordinate Proj. Factor:**		**Date of Work:**	01/27/16
Station:	347+97.0	**Northing:**	280453.1	**Coordinate System:**	U.S. State Plane 1983	**Depth to Water:**	
Offset:	23.0 R	**Easting:**	1138135.7	**Coordinate Datum:**	NAD 83 (CONUS)	**Time Change:**	
Elevation:	288.9	**Drilling Method:**		**Coordinate Zone:**	Missouri East	**Depth Hole Open:**	
Drill No.:	G-8929	**Hammer Efficiency:**		**Equipment:**	Hogentogler CPT ,		

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle ϕ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
70		53.7 - 71.9' Sand *(continued)*	– –	no	no	yes	124(1)	62(1)				42(1)	40(1)		125			
		Bottom of borehole at 71.9 feet.																

LPILE SUMMARY - MODOT_20150812.GDT - 3/15/16 12:34 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

(1) = Assumed, (2) = Actual, (3) = Phi' 0

Missouri Department of Transportation
Construction and Materials

BORING NO. A-16-14

Job No.: J9S3146 | **County:** New Madrid | **Route:** WW | **Logged By:** Sheri Lamberson

Design: A8472 | **Skew:** | **Coordinate Units:** U.S. Survey Feet | **Operator:** Kenny Mathews

Bent: | **Location:** Rt. WW over Wilson Bayou | **Coordinate Proj. Factor:** | **Date of Work:** 02/03/16-02/03/16

Station: 349+30.7 | **Northing:** 280465.6 | **Coordinate System:** U.S. State Plane 1983 | **Depth to Water:** 14

Offset: 12.9 R | **Easting:** 1138267.6 | **Coordinate Datum:** NAD 83 (CONUS) | **Time Change:** Adjacent Stream/Lake

Elevation: 298 | **Drilling Method:** Mud Rotary | **Coordinate Zone:** Missouri East | **Depth Hole Open:**

Drill No.: G-7887 | **Hammer Efficiency:** 79% | **Equipment:** Failing 1500 ,Split-Spoon Sampler

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle φ'	Soil/Rock Strain (ϵ_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
0		0 - 14' Stiff clay without free water		yes	yes	yes												
							120(1)	120(1)	1500(PP)	1.5		2-2-4 (8)		0.007	500			
			290															
10							120(1)	120(1)	1750(PP)	1.75		2-3-3 (8)		0.0065	675			
		▽(1) 14 - 24' Silt		yes	yes	yes												
							95(1)	33(1)	500(PP)	0.5		1-1-2 (4)	29(1)		20			
			280															
20							95(1)	33(1)				2-1-2 (4)	29(1)		20			
		24 - 34' Sand		no	yes	yes												
							105(1)	43(1)	0(PP)	0.0		2-2-3 (7)	30(1)		20			
			270															
30							75(1)	13(1)	0(PP)	0.0		2-1-0 (1)	26(1)		20			
		34 - 44' Soft clay		yes	yes	yes												

(1) = Assumed, (2) = Actual, (3) = Phi' 0

(Continued Next Page)

Missouri Department of Transportation
Construction and Materials

BORING NO. A-16-14

Job No.: J9S3146
Design: A8472
Bent:
Station: 349+30.7
Offset: 12.9 R
Elevation: 298
Drill No.: G-7887

County: New Madrid
Skew:
Location: Rt. WW over Wilson Bayou
Northing: 280465.6
Easting: 1138267.6
Drilling Method: Mud Rotary
Hammer Efficiency: 79%

Route: WW
Coordinate Units: U.S. Survey Feet
Coordinate Proj. Factor:
Coordinate System: U.S. State Plane 1983
Coordinate Datum: NAD 83 (CONUS)
Coordinate Zone: Missouri East
Equipment: Failing 1500 ,Split-Spoon Sampler

Logged By: Sheri Lamberson
Operator: Kenny Mathews
Date of Work: 02/03/16-02/03/16
Depth to Water: 14
Time Change: Adjacent Stream/Lake
Depth Hole Open:

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle φ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
		34 - 44' Soft clay *(continued)*					115$^{(1)}$	53$^{(1)}$	0$^{(PP)}$	0.0		2-1-2 (4)		0.02	30			
			260	yes	yes	yes												
40							110$^{(1)}$	48$^{(1)}$	250$^{(PP)}$	0.25		3-1-1 (3)		0.02	30			
		44 - 76.2' Soft clay					120$^{(1)}$	58$^{(1)}$	250$^{(PP)}$	0.25		3-3-4 (9)		0.02	30			
			250															
50							115$^{(1)}$	53$^{(1)}$	250$^{(PP)}$	0.25		2-2-2 (5)		0.02	30			
							120$^{(1)}$	58$^{(1)}$	250$^{(PP)}$	0.25		2-3-3 (8)		0.02	30			
			240	yes	yes	yes												
60							120$^{(1)}$	58$^{(1)}$	250$^{(PP)}$	0.25		2-3-3 (8)		0.02	30			
							120$^{(1)}$	58$^{(1)}$	250$^{(PP)}$	0.25		3-4-5 (12)		0.02	30			
			230															
70																		

(1) = Assumed, (2) = Actual, (3) = Phi' 0

(Continued Next Page)

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW	**Logged By:** Sheri Lamberson
Design: A8472	**Skew:**	**Coordinate Units:** U.S. Survey Feet	**Operator:** Kenny Mathews
Bent:	**Location:** Rt. WW over Wilson Bayou	**Coordinate Proj. Factor:**	**Date of Work:** 02/03/16-02/03/16
Station: 349+30.7	**Northing:** 280465.6	**Coordinate System:** U.S. State Plane 1983	**Depth to Water:** 14
Offset: 12.9 R	**Easting:** 1138267.6	**Coordinate Datum:** NAD 83 (CONUS)	**Time Change:** Adjacent Stream/Lake
Elevation: 298	**Drilling Method:** Mud Rotary	**Coordinate Zone:** Missouri East	**Depth Hole Open:**
Drill No.: G-7887	**Hammer Efficiency:** 79%	**Equipment:** Failing 1500 ,Split-Spoon Sampler	

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle φ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
70		44 - 76.2' Soft clay *(continued)*		yes	yes	yes	120(1)	58(1)	250(PP)	0.25		2-4-5 (12)		0.02	30			
							120(1)	58(1)	250(PP)	0.25		9-2-6 (11)		0.02	30			
		76.2 - 106.5' Sand	220	no	no	yes												
80							122(1)	60(1)				10-8-16 (32)	36(1)		125			
							130(1)	68(1)				15-17-16 (43)	38(1)		125			
			210															
90							130(1)	68(1)				15-18-15 (43)	38(1)		125			
							150(1)	88(1)				30-38/0.5'	43(1)		125			
			200															
100							142(1)	80(1)				18-30-31 (80)	41(1)		125			

(1) = Assumed, (2) = Actual, (3) = Phi' 0 *(Continued Next Page)*

Missouri Department of Transportation
Construction and Materials

BORING NO. A-16-14

Job No.: J9S3146 **County:** New Madrid **Route:** WW **Logged By:** Sheri Lamberson

Design: A8472 **Skew:** **Coordinate Units:** U.S. Survey Feet **Operator:** Kenny Mathews

Bent: **Location:** Rt. WW over Wilson Bayou **Coordinate Proj. Factor:** **Date of Work:** 02/03/16-02/03/16

Station: 349+30.7 **Northing:** 280465.6 **Coordinate System:** U.S. State Plane 1983 **Depth to Water:** 14

Offset: 12.9 R **Easting:** 1138267.6 **Coordinate Datum:** NAD 83 (CONUS) **Time Change:** Adjacent Stream/Lake

Elevation: 298 **Drilling Method:** Mud Rotary **Coordinate Zone:** Missouri East **Depth Hole Open:**

Drill No.: G-7887 **Hammer Efficiency:** 79% **Equipment:** Failing 1500 ,Split-Spoon Sampler

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Scour Parameters: Downdrag	Scour Parameters: Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle ϕ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
		76.2 - 106.5' Sand *(continued)*		no	no	yes	135(1)	73(1)				20-17-20 (49)	39(1)		125			
		Bottom of borehole at 106.5 feet.																

LPILE SUMMARY - MODOT_20150812.GDT - 3/15/16 13:35 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

(1) = Assumed, (2) = Actual, (3) = Phi' 0

A8472
Rt. WW over Wilson Bayou
Project Number:J9S3146

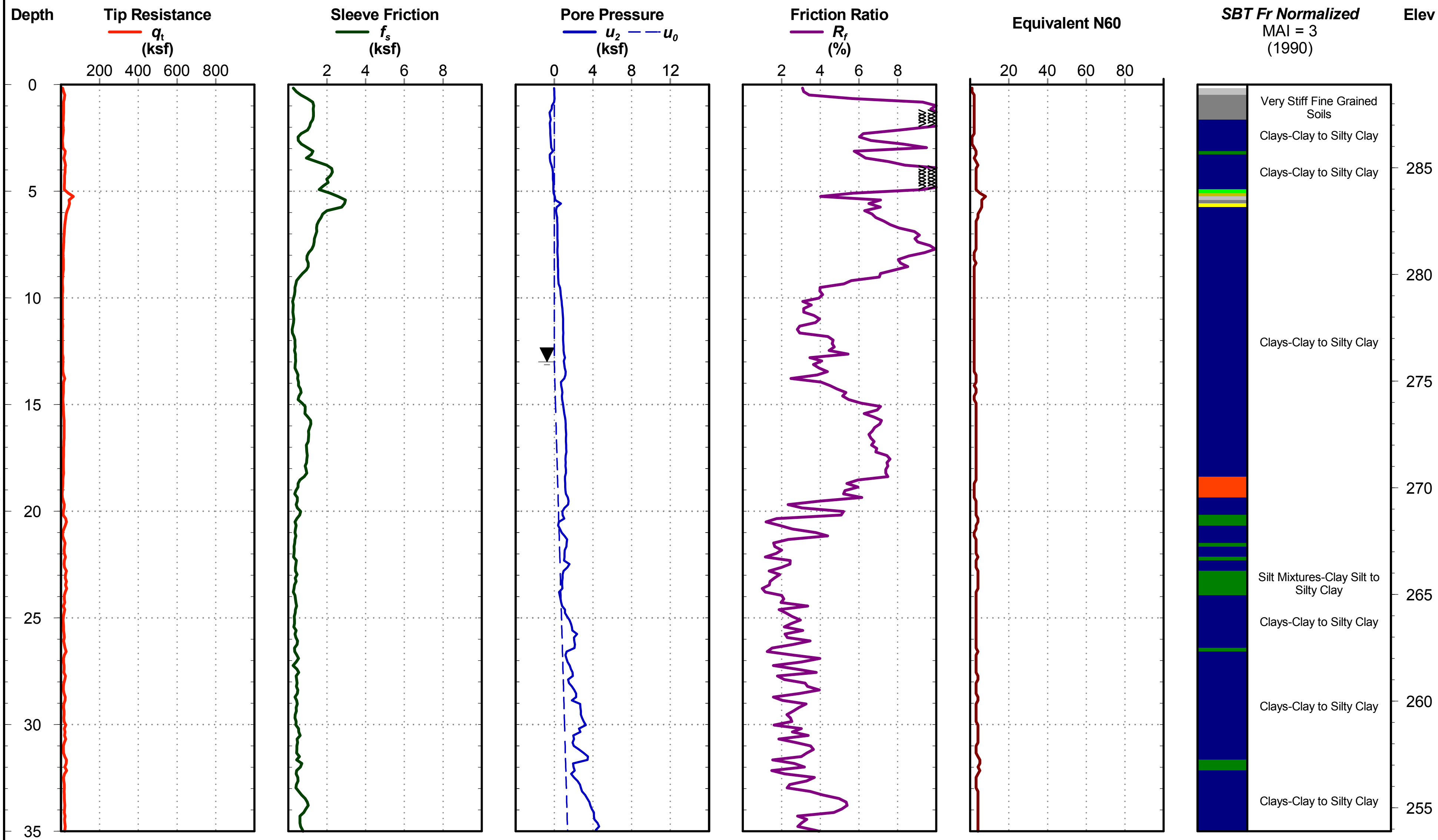
Cone Penetration Test

H-16-22

Date: Jan. 27, 2016
Estimated Water Depth: 13
Rig/Operator: Ricardo Todd

Northing: 280453.1
Easting: 1138135.7
Elevation: 288.9 NAD 83 (CONUS)

Total Depth: 71.9
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD - MODOT 20150728.GDT - 3/16/16 12:44 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

H-16-22

Electronic File Name: J9S3146-A8472

A8472
Rt. WW over Wilson Bayou
Project Number:J9S3146

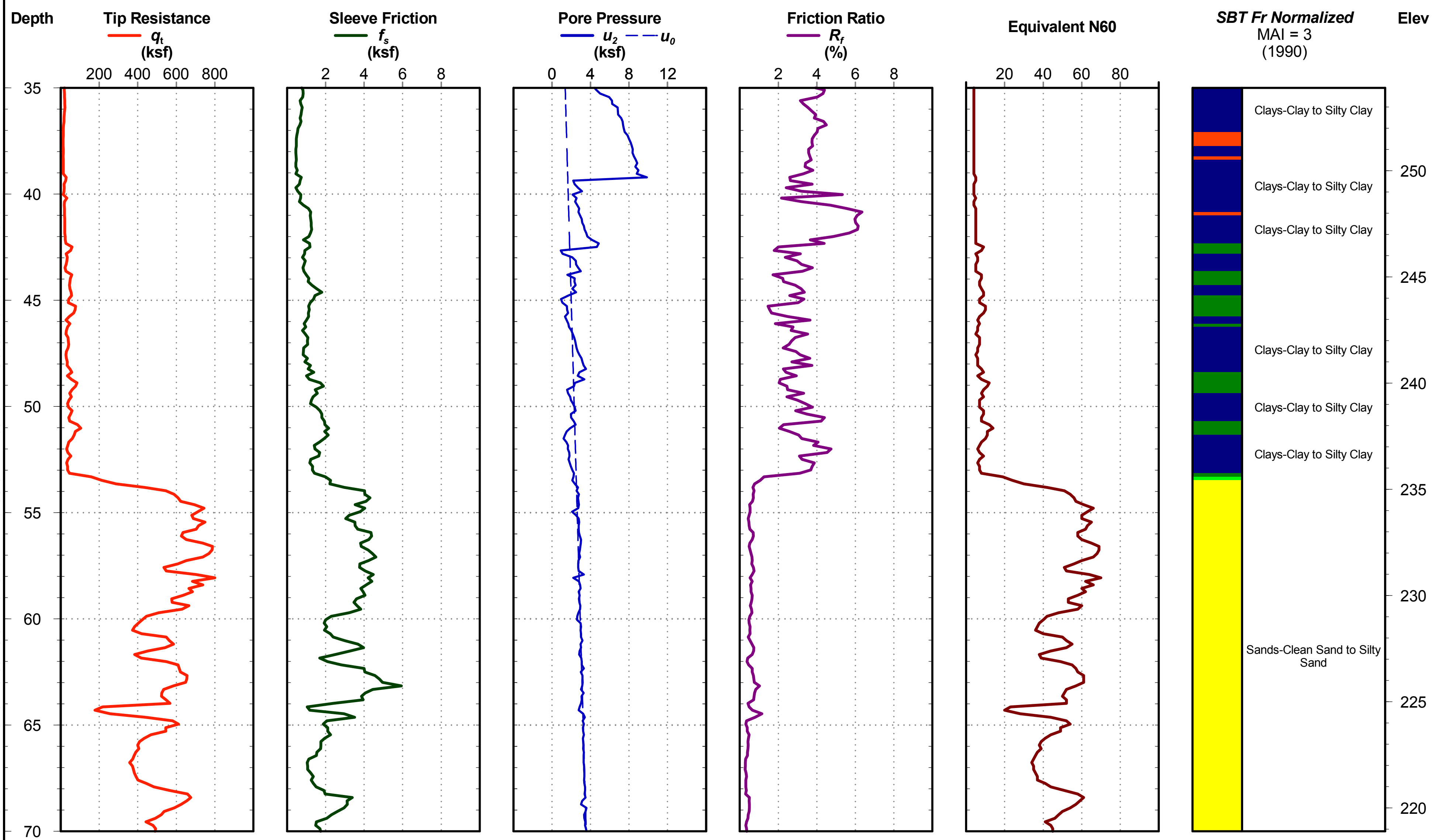
Cone Penetration Test

H-16-22

Date: Jan. 27, 2016
Estimated Water Depth: 13
Rig/Operator: Ricardo Todd

Northing: 280453.1
Easting: 1138135.7
Elevation: 288.9 NAD 83 (CONUS)

Total Depth: 71.9
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD - MODOT 20150728.GDT - 3/16/16 12:44 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

H-16-22

Electronic File Name: J9S3146-A8472

A8472
Rt. WW over Wilson Bayou
Project Number:J9S3146

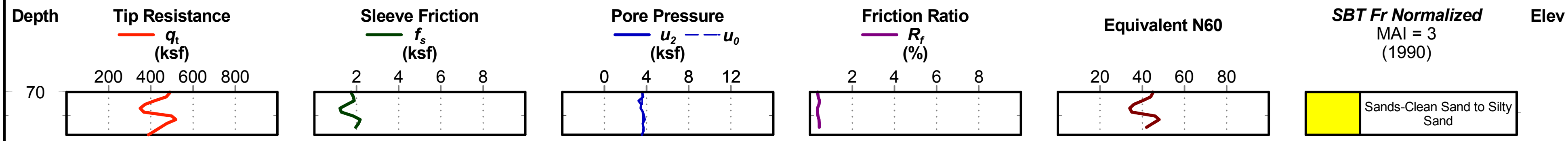
Cone Penetration Test

H-16-22

Date: Jan. 27, 2016
Estimated Water Depth: 13
Rig/Operator: Ricardo Todd

Northing: 280453.1
Easting: 1138135.7
Elevation: 288.9 NAD 83 (CONUS)

Total Depth: 71.9
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD - MODOT 20150728.GDT - 3/16/16 12:44 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

H-16-22

Electronic File Name: J9S3146-A8472

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A8472	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:** Sheri Lamberson	**Operator:** Kenny Mathews
Station: 349+30.7	**Northing:** 280465.6	**Date of Work:** 02/03/16-02/03/16
Offset: 12.9 R	**Easting:** 1138267.6	**Depth to Water:** 14.0
Elevation: 298.0	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:** Adjacent Stream/Lake
Requested Offset:	**Equipment:** Failing 1500 ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.: G-7887	**Hammer Efficiency:** 79%	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-14.0' Dark gray, FAT CLAY, medium stiff to stiff, moist							
			295						
5									
				X	100	2-2-4 (8)		PP = 1.50 tsf	MC = 42.4% γ_{sat} = 111 pcf(1)
			290						
10									
				X	100	2-3-3 (8)		PP = 1.75 tsf	MC = 44.4% γ_{sat} = 110 pcf(1)
			285						
15		14.0-24.0' Greenish gray and brown, SILT, soft, wet							
				X	100	1-1-2 (4)		PP = 0.50 tsf	MC = 38.8% γ_{sat} = 114 pcf(1)
			280						
20									
				X	100	2-1-2 (4)			
			275						
25		24.0-34.0' Dark gray, SAND scattered silt, loose to very loose, wet, fine grained							
				X	100	2-2-3 (7)		PP = 0.00 tsf	MC = 34.2% γ_{sat} = 117 pcf(1)
			270						
30									
				X	100	2-1-0 (1)		PP = 0.00 tsf	LL = NP MC = 35.7% γ_{sat} = 116 pcf(1)
			265						
35									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983	**Coordinate Zone:** Missouri East	**Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS)	**Coordinate Units:** U.S. Survey Feet	

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146 | **County:** New Madrid | **Route:** WW
Design: A8472 | **Skew:** | **Location:** Rt. WW over Wilson Bayou
Bent: | **Logged By:** Sheri Lamberson | **Operator:** Kenny Mathews
Station: 349+30.7 | **Northing:** 280465.6 | **Date of Work:** 02/03/16-02/03/16
Offset: 12.9 R | **Easting:** 1138267.6 | **Depth to Water:** 14.0
Elevation: 298.0 | **Requested Northing:** | **Depth Hole Open:**
Requested Station: | **Requested Easting:** | **Time Change:** Adjacent Stream/Lake
Requested Offset: | **Equipment:** Failing 1500 ,Split-Spoon Sampler
Requested Elevation: | **Location Note:**
Drill No.: G-7887 | **Hammer Efficiency:** 79% | **Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
35		34.0-44.0' Dark gray, LEAN CLAY to silt, soft to very soft *(continued)*		X	100	2-1-2 (4)		PP = 0.00 tsf	
			260						
40				X	100	3-1-1 (3)		PP = 0.25 tsf	
			255						
45		44.0-76.2' Dark gray, FAT CLAY, soft to medium stiff, wet		X	100	3-3-4 (9)		PP = 0.25 tsf	
			250						
50				X	100	2-2-2 (5)		PP = 0.25 tsf	
			245						
55				X	100	2-3-3 (8)		PP = 0.25 tsf	
			240						
60				X	100	2-3-3 (8)		PP = 0.25 tsf	
			235						
65				X	100	3-4-5 (12)		PP = 0.25 tsf	
			230						
70									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 | **Coordinate Zone:** Missouri East | **Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS) | **Coordinate Units:** U.S. Survey Feet

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146 | **County:** New Madrid | **Route:** WW
Design: A8472 | **Skew:** | **Location:** Rt. WW over Wilson Bayou
Bent: | **Logged By:** Sheri Lamberson | **Operator:** Kenny Mathews
Station: 349+30.7 | **Northing:** 280465.6 | **Date of Work:** 02/03/16-02/03/16
Offset: 12.9 R | **Easting:** 1138267.6 | **Depth to Water:** 14.0
Elevation: 298.0 | **Requested Northing:** | **Depth Hole Open:**
Requested Station: | **Requested Easting:** | **Time Change:** Adjacent Stream/Lake
Requested Offset: | **Equipment:** Failing 1500 ,Split-Spoon Sampler
Requested Elevation: | **Location Note:**
Drill No.: G-7887 | **Hammer Efficiency:** 79% | **Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
70		44.0-76.2' Dark gray, FAT CLAY, soft to medium stiff, wet *(continued)*			100	2-4-5 (12)		PP = 0.25 tsf	
			225						
75					67	9-2-6 (11)		PP = 0.25 tsf	
		76.2-106.5' Dark gray, SILTY SAND with clay seams, scattered gravel, dense to very dense, wet, fine to coarse grained, coarser with depth	220						
80					100	10-8-16 (32)			
			215						
85					100	15-17-16 (43)			
			210						
90					100	15-18-15 (43)			
			205						
95					95	30-38/0.5'			
			200						
100					67	18-30-31 (80)			
			195						
105									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 | **Coordinate Zone:** Missouri East | **Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS) | **Coordinate Units:** U.S. Survey Feet

Missouri Department of Transportation
Construction and Materials

Job No.:	J9S3146	**County:**	New Madrid	**Route:**	WW
Design:	A8472	**Skew:**		**Location:**	Rt. WW over Wilson Bayou
Bent:		**Logged By:**	Sheri Lamberson	**Operator:**	Kenny Mathews
Station:	349+30.7	**Northing:**	280465.6	**Date of Work:**	02/03/16-02/03/16
Offset:	12.9 R	**Easting:**	1138267.6	**Depth to Water:**	14.0
Elevation:	298.0	**Requested Northing:**		**Depth Hole Open:**	
Requested Station:		**Requested Easting:**		**Time Change:**	Adjacent Stream/Lake
Requested Offset:		**Equipment:**	Failing 1500 ,Split-Spoon Sampler		
Requested Elevation:		**Location Note:**			
Drill No.:	G-7887	**Hammer Efficiency:**	79%	**Drilling Method:**	Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
105									
					100	20-17-20 (49)			
		Bottom of borehole at 106.5 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System:	U.S. State Plane 1983	**Coordinate Zone:**	Missouri East	**Coordinate Proj. Factor:**	
Coordinate Datum:	NAD 83 (CONUS)	**Coordinate Units:**	U.S. Survey Feet		

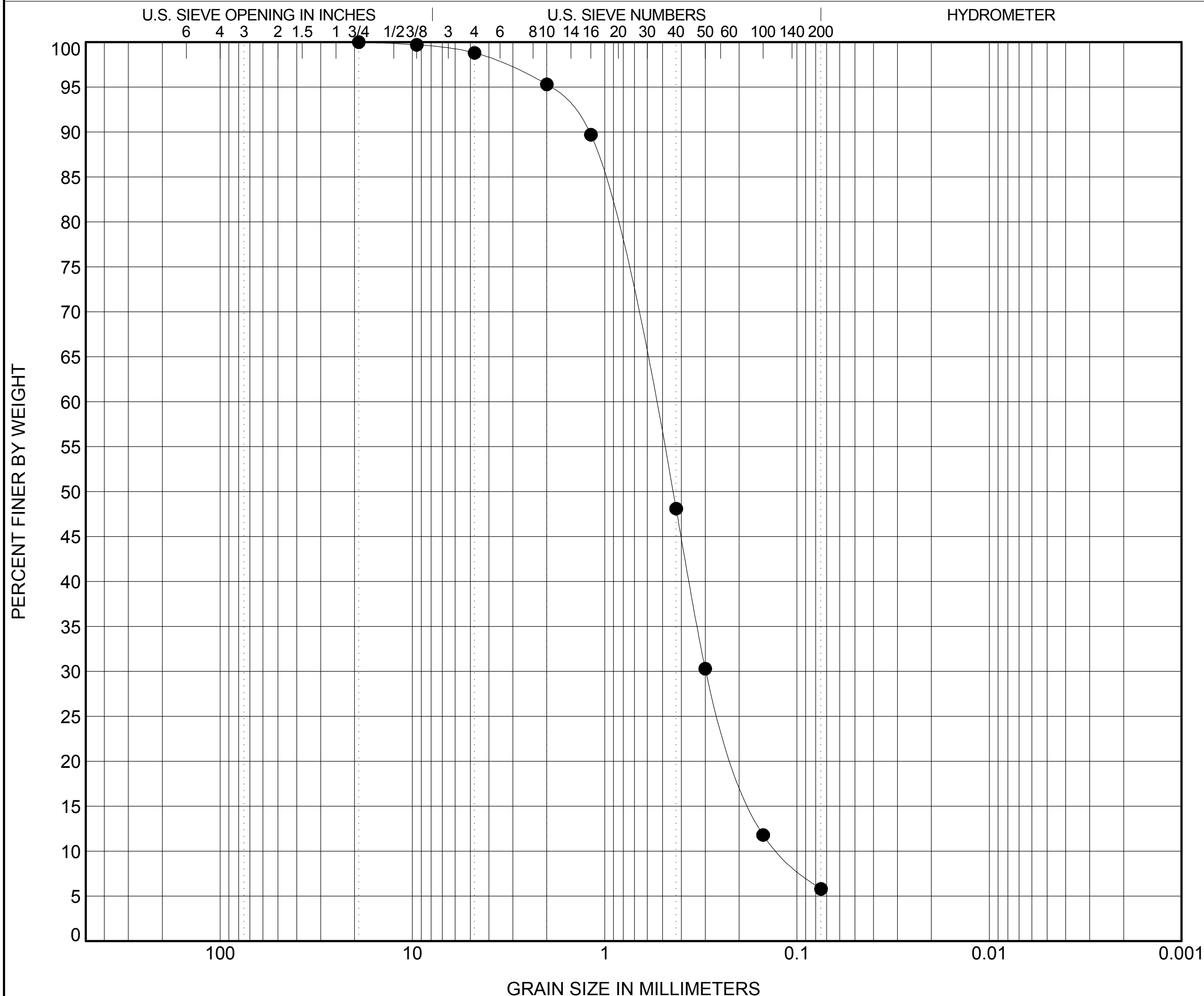
* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

MoDOT - Geotechnical Section
1617 Missouri Boulevard
Jefferson City, Missouri 65109

GRAIN SIZE DISTRIBUTION

CLIENT MoDOT Bridge Division - EFK Moen, LLC **PROJECT NAME** A8472

PROJECT NUMBER J9S3146 **PROJECT LOCATION** Rt. WW over Wilson Bayou



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		ASTM Classification	LL	PL	PI	Cc	Cu
●	**A-16-14**	**30.0**					**1.27**	**4.67**

	Specimen Identification		D95	D90	D84	D50	%Gravel	%Sand	%Silt	%Clay
●	**A-16-14**	**30.0**	**1.94**	**1.21**	**1.03**	**0.445**	**1.2**	**93.0**	**5.8**	

GRAIN SIZE - MODOT - MODOT 20150728.GDT - 3/16/16 12:51 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 347+89.0	**Northing:** 280493.2	**Date of Work:** 02/21/67-02/21/67
Offset: 14.0 L	**Easting:** 1138125.4	**Depth to Water:** 16.0
Elevation: 288.8	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:** 0 hours
Requested Offset:	**Equipment:** ,	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-9.0' Brown, LEAN CLAY, soft							
			285						
5									
			280						
10		9.0-16.0' Brown, SILT to lean clay, soft							
			275						
15									
		16.0-51.9' Gray, SILT to lean clay, soft							
			270						
20									
			265						
25									
			260						
30									
			255						
35									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983   **Coordinate Zone:** Missouri East   **Coordinate Proj. Factor:**

Coordinate Datum: NAD 83 (CONUS)   **Coordinate Units:** U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146
County: New Madrid
Route: WW
Design: A2141
Skew:
Location: Rt. WW over Wilson Bayou
Bent:
Logged By:
Operator:
Station: 347+89.0
Northing: 280493.2
Date of Work: 02/21/67-02/21/67
Offset: 14.0 L
Easting: 1138125.4
Depth to Water: 16.0
Elevation: 288.8
Requested Northing:
Depth Hole Open:
Requested Station:
Requested Easting:
Time Change: 0 hours
Requested Offset:
Equipment: ,
Requested Elevation:
Location Note:
Drill No.:
Hammer Efficiency:
Drilling Method: Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
35		16.0-51.9' Gray, SILT to lean clay, soft *(continued)*							
			250						
40									
		42.9-43.9' stiff	245						
45									
		47.6-48.6' stiff	240						
50									
		51.9-63.0' SAND, medium dense							
			235						
55									
			230						
60									
		63.0-65.0' SAND and gravel	225						
65		65.0-78.2' SAND, dense							
			220						
70									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983
Coordinate Zone: Missouri East
Coordinate Proj. Factor:
Coordinate Datum: NAD 83 (CONUS)
Coordinate Units: U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146
County: New Madrid
Route: WW
Design: A2141
Skew:
Location: Rt. WW over Wilson Bayou
Bent:
Logged By:
Operator:
Station: 347+89.0
Northing: 280493.2
Date of Work: 02/21/67-02/21/67
Offset: 14.0 L
Easting: 1138125.4
Depth to Water: 16.0
Elevation: 288.8
Requested Northing:
Depth Hole Open:
Requested Station:
Requested Easting:
Time Change: 0 hours
Requested Offset:
Equipment: ,
Requested Elevation:
Location Note:
Drill No.:
Hammer Efficiency:
Drilling Method: Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
70		65.0-78.2' SAND, dense *(continued)*							
			215						
75									
		Bottom of borehole at 78.2 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983
Coordinate Zone: Missouri East
Coordinate Proj. Factor:
Coordinate Datum: NAD 83 (CONUS)
Coordinate Units: U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 347+89.0	**Northing:** 280465.2	**Date of Work:** 02/21/67-02/21/67
Offset: 14.0 R	**Easting:** 1138126.1	**Depth to Water:**
Elevation: 289.0	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-19.0' Brown, LEAN CLAY, soft							
5			285						
10			280						
15			275						
20		19.0-29.0' Gray, SILT to lean clay, soft	270						
25			265						
30		29.0-36.0' Gray, LEAN CLAY, soft	260						
35			255						

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983	**Coordinate Zone:** Missouri East	**Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS)	**Coordinate Units:** U.S. Survey Feet	

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146
County: New Madrid
Route: WW
Design: A2141
Skew:
Location: Rt. WW over Wilson Bayou
Bent:
Logged By:
Operator:
Station: 347+89.0
Northing: 280465.2
Date of Work: 02/21/67-02/21/67
Offset: 14.0 R
Easting: 1138126.1
Depth to Water:
Elevation: 289.0
Requested Northing:
Depth Hole Open:
Requested Station:
Requested Easting:
Time Change:
Requested Offset:
Equipment: ,Split-Spoon Sampler
Requested Elevation:
Location Note:
Drill No.:
Hammer Efficiency:
Drilling Method: Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
35		29.0-36.0' Gray, LEAN CLAY, soft *(continued)*							
		36.0-46.0' LEAN CLAY, with silt and sand seams							
40			250						
45			245						
		46.0-78.0' SAND, dense to very dense							
50			240	X		17-20-27 (47)			
55			235	X		19-28-36 (64)			
60			230	X		27-52-59 (111)			
65			225	X		32-48-55 (103)			
70			220						

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983
Coordinate Zone: Missouri East
Coordinate Proj. Factor:
Coordinate Datum: NAD 83 (CONUS)
Coordinate Units: U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 347+89.0	**Northing:** 280465.2	**Date of Work:** 02/21/67-02/21/67
Offset: 14.0 R	**Easting:** 1138126.1	**Depth to Water:**
Elevation: 289.0	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
70		46.0-78.0' SAND, dense to very dense *(continued)*							
75			215						
80		78.0-100.0' SAND and gravel, very dense	210						
85			205						
90			200						
95			195						
100			190						
		Bottom of borehole at 100.0 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 **Coordinate Zone:** Missouri East **Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS) **Coordinate Units:** U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 348+23.0	**Northing:** 280489.2	**Date of Work:** 02/21/67-02/21/67
Offset: 9.0 L	**Easting:** 1138159.5	**Depth to Water:**
Elevation: 290.6	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-9.0' Brown, LEAN CLAY	290						
5			285						
10		9.0-16.0' Brown, SILT and lean clay	280						
15			275						
		16.0-49.0' Gray, SILT and lean clay							
20			270						
25			265						
30			260						
35									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983   **Coordinate Zone:** Missouri East   **Coordinate Proj. Factor:**

Coordinate Datum: NAD 83 (CONUS)   **Coordinate Units:** U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.:	J9S3146	**County:**	New Madrid	**Route:**	WW
Design:	A2141	**Skew:**		**Location:**	Rt. WW over Wilson Bayou
Bent:		**Logged By:**		**Operator:**	
Station:	348+23.0	**Northing:**	280489.2	**Date of Work:**	02/21/67-02/21/67
Offset:	9.0 L	**Easting:**	1138159.5	**Depth to Water:**	
Elevation:	290.6	**Requested Northing:**		**Depth Hole Open:**	
Requested Station:		**Requested Easting:**		**Time Change:**	
Requested Offset:		**Equipment:**	,		
Requested Elevation:		**Location Note:**			
Drill No.:		**Hammer Efficiency:**		**Drilling Method:**	Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
35		16.0-49.0' Gray, SILT and lean clay *(continued)*	255						
40			250						
45			245						
		49.0-50.0' SAND, medium dense							
50		50.0-52.0' SILT and lean clay, soft	240						
		52.0-53.0' SAND, medium dense							
		53.0-54.0' SILT, soft							
		54.0-78.2' SAND, medium dense							
55			235						
60			230						
65			225						
70									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System:	U.S. State Plane 1983	**Coordinate Zone:**	Missouri East	**Coordinate Proj. Factor:**	
Coordinate Datum:	NAD 83 (CONUS)	**Coordinate Units:**	U.S. Survey Feet		

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 348+23.0	**Northing:** 280489.2	**Date of Work:** 02/21/67-02/21/67
Offset: 9.0 L	**Easting:** 1138159.5	**Depth to Water:**
Elevation: 290.6	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
70		54.0-78.2' SAND, medium dense *(continued)*	220						
75			215						
		Bottom of borehole at 78.2 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 **Coordinate Zone:** Missouri East **Coordinate Proj. Factor:**

Coordinate Datum: NAD 83 (CONUS) **Coordinate Units:** U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 348+57.0	**Northing:** 280489.8	**Date of Work:** 02/21/67-02/21/67
Offset: 9.0 L	**Easting:** 1138193.5	**Depth to Water:**
Elevation: 291.3	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-5.0' Fill	290						
5		5.0-12.0' Brown, LEAN CLAY, very soft	285						
10			280			0-0-1 (1)			
		12.0-27.0' Dark gray, LEAN CLAY, soft to very soft							
15			275			1-1-2 (3)			
20			270			1-1-2 (3)			
25			265			0-0-1 (1)			
		27.0-45.0' Dark gray, SILT and lean clay, soft							
30			260			1-1-2 (3)			
35									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 **Coordinate Zone:** Missouri East **Coordinate Proj. Factor:**

Coordinate Datum: NAD 83 (CONUS) **Coordinate Units:** U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 348+57.0	**Northing:** 280489.8	**Date of Work:** 02/21/67-02/21/67
Offset: 9.0 L	**Easting:** 1138193.5	**Depth to Water:**
Elevation: 291.3	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
35		27.0-45.0' Dark gray, SILT and lean clay, soft *(continued)*	255			1-1-2 (3)			
40			250			1-1-2 (3)			
45		45.0-52.0' Dark gray, LEAN CLAY, medium stiff	245			2-3-2 (5)			
50			240			3-2-3 (5)			
		52.0-81.5' SAND, medium dense to very dense							
55			235			4-4-8 (12)			
60			230			5-5-8 (13)			
65			225			6-10-11 (21)			
70									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983	**Coordinate Zone:** Missouri East	**Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS)	**Coordinate Units:** U.S. Survey Feet	

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Missouri Department of Transportation
Construction and Materials

Job No.:	J9S3146	**County:**	New Madrid	**Route:**	WW
Design:	A2141	**Skew:**		**Location:**	Rt. WW over Wilson Bayou
Bent:		**Logged By:**		**Operator:**	
Station:	348+57.0	**Northing:**	280489.8	**Date of Work:**	02/21/67-02/21/67
Offset:	9.0 L	**Easting:**	1138193.5	**Depth to Water:**	
Elevation:	291.3	**Requested Northing:**		**Depth Hole Open:**	
Requested Station:		**Requested Easting:**		**Time Change:**	
Requested Offset:		**Equipment:**	,Split-Spoon Sampler		
Requested Elevation:		**Location Note:**			
Drill No.:		**Hammer Efficiency:**		**Drilling Method:**	Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
70		52.0-81.5' SAND, medium dense to very dense *(continued)*	220			16-22-26 (48)			
75			215			25-31-32 (63)			
80			210			30-43-30 (73)			
		Bottom of borehole at 81.5 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 **Coordinate Zone:** Missouri East **Coordinate Proj. Factor:**

Coordinate Datum: NAD 83 (CONUS) **Coordinate Units:** U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 348+94.0	**Northing:** 280467.7	**Date of Work:** 02/21/67-02/21/67
Offset: 14.0 R	**Easting:** 1138231.1	**Depth to Water:**
Elevation: 290.5	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-16.0' Brown, LEAN CLAY, soft	290						
5			285						
10			280						
15			275						
		16.0-65.0' Gray, SILT and lean clay, soft							
20			270						
25			265						
30			260						
35									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983	**Coordinate Zone:** Missouri East	**Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS)	**Coordinate Units:** U.S. Survey Feet	

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 348+94.0	**Northing:** 280467.7	**Date of Work:** 02/21/67-02/21/67
Offset: 14.0 R	**Easting:** 1138231.1	**Depth to Water:**
Elevation: 290.5	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
35		16.0-65.0' Gray, SILT and lean clay, soft *(continued)*	255						
40			250						
45			245						
50			240						
55			235						
60			230						
65		65.0-78.2' SAND, dense	225						
70									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 **Coordinate Zone:** Missouri East **Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS) **Coordinate Units:** U.S. Survey Feet

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Missouri Department of Transportation
Construction and Materials

Job No.:	J9S3146	**County:**	New Madrid	**Route:**	WW
Design:	A2141	**Skew:**		**Location:**	Rt. WW over Wilson Bayou
Bent:		**Logged By:**		**Operator:**	
Station:	348+94.0	**Northing:**	280467.7	**Date of Work:**	02/21/67-02/21/67
Offset:	14.0 R	**Easting:**	1138231.1	**Depth to Water:**	
Elevation:	290.5	**Requested Northing:**		**Depth Hole Open:**	
Requested Station:		**Requested Easting:**		**Time Change:**	
Requested Offset:		**Equipment:**	,		
Requested Elevation:		**Location Note:**			
Drill No.:		**Hammer Efficiency:**		**Drilling Method:**	Continuous Flight Auger

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
70		65.0-78.2' SAND, dense *(continued)*	220						
75			215						
		Bottom of borehole at 78.2 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System:	U.S. State Plane 1983	**Coordinate Zone:**	Missouri East	**Coordinate Proj. Factor:**	
Coordinate Datum:	NAD 83 (CONUS)	**Coordinate Units:**	U.S. Survey Feet		

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

LETTER BOREHOLE - MODOT 20150728.GDT - 3/16/16 12:52 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 349+03.0	**Northing:** 280493.9	**Date of Work:** 02/21/67-02/21/67
Offset: 12.0 L	**Easting:** 1138239.5	**Depth to Water:**
Elevation: 291.6	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-14.9' Brown, LEAN CLAY, soft	290						
5			285						
10			280						
15		14.9-51.6' Blue, LEAN CLAY, soft	275						
20			270						
25			265						
30			260						
35									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983   **Coordinate Zone:** Missouri East   **Coordinate Proj. Factor:**

Coordinate Datum: NAD 83 (CONUS)   **Coordinate Units:** U.S. Survey Feet

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

LETTER BOREHOLE - MODOT 20150728.GDT - 3/16/16 12:52 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146	**County:** New Madrid	**Route:** WW
Design: A2141	**Skew:**	**Location:** Rt. WW over Wilson Bayou
Bent:	**Logged By:**	**Operator:**
Station: 349+03.0	**Northing:** 280493.9	**Date of Work:** 02/21/67-02/21/67
Offset: 12.0 L	**Easting:** 1138239.5	**Depth to Water:**
Elevation: 291.6	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.:	**Hammer Efficiency:**	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
35		14.9-51.6' Blue, LEAN CLAY, soft *(continued)*	255						
40			250						
45			245						
50									
		51.6-66.5' SAND layers, very loose to loose, and FAT CLAY layers, soft	240						
55			235						
60			230						
65									
		66.5-70.0' FAT CLAY, soft	225						
70									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983	**Coordinate Zone:** Missouri East	**Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS)	**Coordinate Units:** U.S. Survey Feet	

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Missouri Department of Transportation
Construction and Materials

Job No.: J9S3146 **County:** New Madrid **Route:** WW
Design: A2141 **Skew:** **Location:** Rt. WW over Wilson Bayou
Bent: **Logged By:** **Operator:**
Station: 349+03.0 **Northing:** 280493.9 **Date of Work:** 02/21/67-02/21/67
Offset: 12.0 L **Easting:** 1138239.5 **Depth to Water:**
Elevation: 291.6 **Requested Northing:** **Depth Hole Open:**
Requested Station: **Requested Easting:** **Time Change:**
Requested Offset: **Equipment:** ,Split-Spoon Sampler
Requested Elevation: **Location Note:**
Drill No.: **Hammer Efficiency:** **Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
70		70.0-73.0' SAND, loose	220	X		5-4-2 (6)			
75		73.0-80.0' SAND, medium dense to dense, fine grained	215	X		12-12-12 (24)			
80		80.0-80.3' GRAVEL, very dense, fine grained 80.3-85.0' SAND, very dense, coarse grained	210	X		36-33-23 (56)			
85		85.0-100.0' SAND with fine gravel, very dense, coarse grained	205	X		30-38-39 (77)			
90			200	X		32-40-41 (81)			
95			195						
100		Bottom of borehole at 100.0 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 **Coordinate Zone:** Missouri East **Coordinate Proj. Factor:**
Coordinate Datum: NAD 83 (CONUS) **Coordinate Units:** U.S. Survey Feet

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

KEY TO SYMBOLS

MoDOT - Geotechnical Section
1617 Missouri Boulevard
Jefferson City, Missouri 65109

CLIENT MoDOT Bridge Division - EFK Moen, LLC
PROJECT NAME A8472
PROJECT NUMBER J9S3146
PROJECT LOCATION Rt. WW over Wilson Bayou

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

- CH: USCS High Plasticity Clay
- CL: USCS Low Plasticity Clay
- CL-ML: USCS Low Plasticity Silty Clay
- FILL: Fill (made ground)
- GP: USCS Poorly-graded Gravel
- ML: USCS Silt
- SC: USCS Clayey Sand
- SP: USCS Poorly-graded Sand
- SPG: USCS Poorly-graded Gravelly Sand
- SW: USCS Well-graded Sand
- SWG: USCS Well-graded Gravelly Sand
- SW-SM: USCS Well-graded Sand with Silt

SAMPLER SYMBOLS

- Split-Spoon Sampler

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

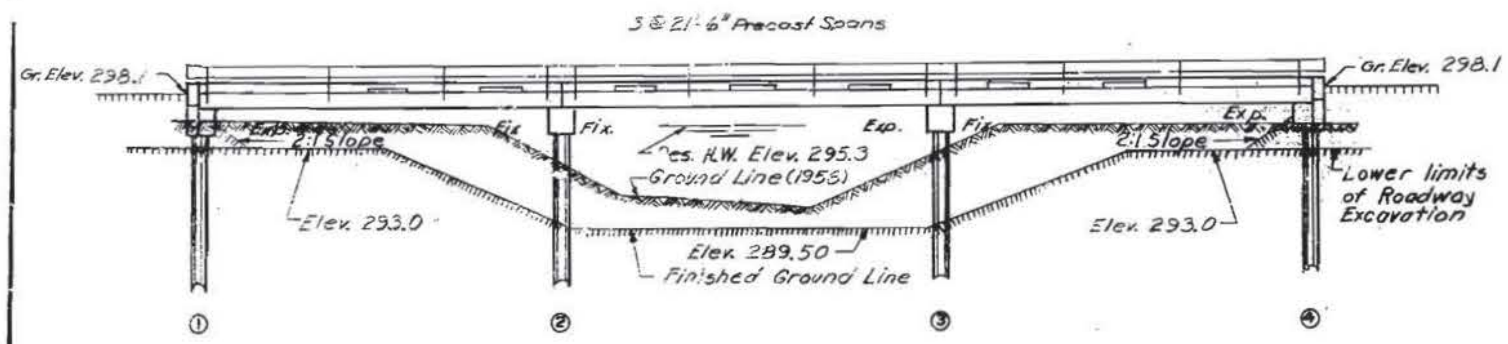
LL	- LIQUID LIMIT (%)
PI	- PLASTIC INDEX (%)
W	- MOISTURE CONTENT (%)
DD	- DRY DENSITY (PCF)
NP	- NON PLASTIC
-200	- PERCENT PASSING NO. 200 SIEVE
PP	- POCKET PENETROMETER (TSF)
Qu	- UNCONFINED COMPRESSIVE STRENGTH (PSF)
TV	- TORVANE
PID	- PHOTOIONIZATION DETECTOR
UC	- UNCONFINED COMPRESSION
ppm	- PARTS PER MILLION

- ▽ Water Level at Time of Drilling
- ▼ Water Level at End of Drilling
- ▽ Water Level after Drilling

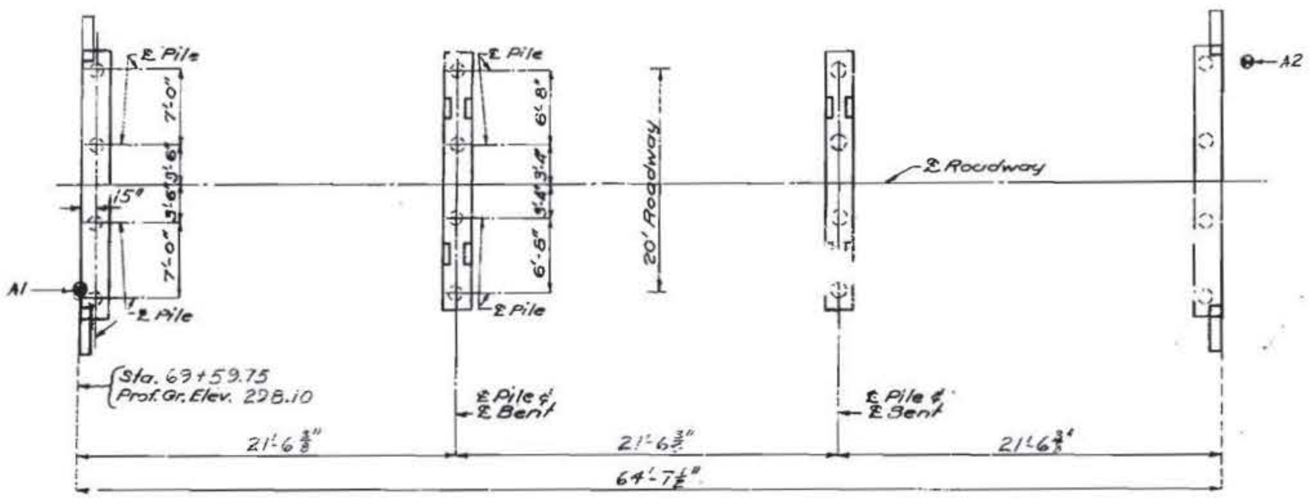
KEY TO SYMBOLS - MODOT 20150728.GDT - 3/16/16 12:52 - J:\SG\GINT\PROJECT FILES\J9S3146-A8472-A2141.GPJ

MISSOURI STATE HIGHWAY DEPARTMENT

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19	16	



ELEVATION



PLAN

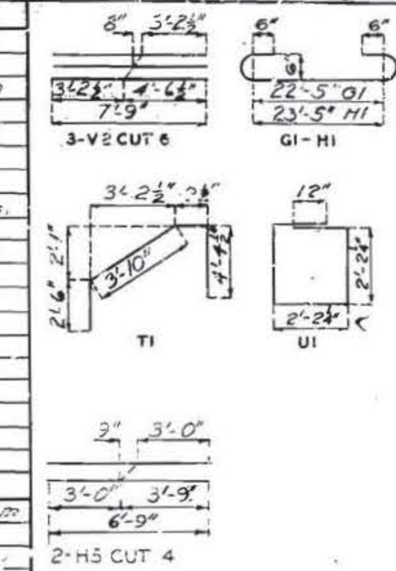
PILE DATA				
Bent No.	1	2	3	4
Pile Type	Precast	Precast	Precast	Precast
Number	4	3	4	4
Aproximate Length Ft.	30	30	30	30
"Plan" Capacity T/Pile	21.0	22.6	22.6	21.0
"Computed" Capacity T/Pile	14.0	18.8	18.8	14.0
Min. Penetration (Pile Tip Elev.)	275.0	270.0	270.0	275.0
Pile Standard	52.01	52.01	52.01	52.01
See Standard Specifications 52.2.6				

Note: One concrete test pile shall be driven in permanent position for bent No. 2.

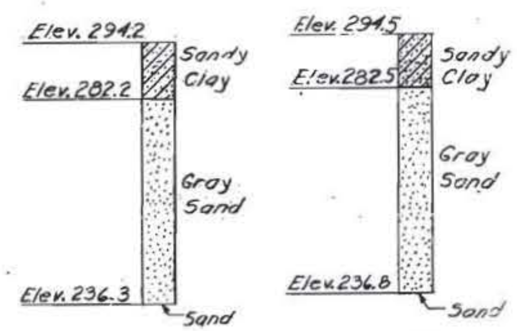
All piles shall be driven to the minimum penetrations noted and to not less than the specified "Plan" capacities unless the pile lengths authorized and furnished fail to give "Plan" capacities in which cases not less than the "Computed" capacities shall be obtained.

BILL OF REINFORCING STEEL - SUBSTRUCTURE				
No.	Size	Length	Mark	Location
End Bents No. 1 & 4				
8	#6	11'-6"	T1	Wing
16	#6	26'-0"	H1	Beam
4	#6	23'-9"	H2	"
20	#5	5'-0"	H4	Wing
4	#5	6'-3"	H5	"
40	#4	9'-9"	U1	Beam
6	#4	7'-9"	V2	Wing
Int. Bents No. 2 & 3				
40	#4	9'-9"	U1	Beam
16	#6	25'-0"	G1	Beam
4	#6	22'-9"	G2	"

Bending Sketches & Cutting Diagrams



Note: See Sheet No. 3 of 3 for Bill of Reinforcing Steel for superstructure



Note: Soundings taken with an auger. Location of soundings marked thus ⊖ on plans.

LOG OF SOUNDINGS

GENERAL NOTES:

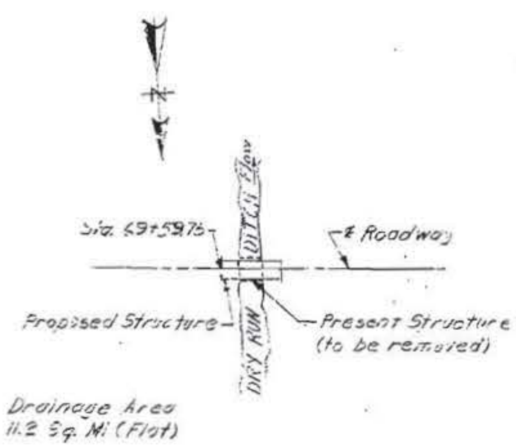
Design Specifications: A.A.S.H.O. 1961
Loading: H15-44 (one lane)
Structural Steel Stress: 20,000 psi
Reinforcing Steel Stress: 20,000 psi
Concrete, Class A Stress: 1,500 psi
Concrete, Class B Stress: 1,200 psi
Concrete, Class X Stress: 1,500 psi

Concrete for Substructure and Superstructure curbs shall be Class B.

Concrete for precast superstructure units shall be Class A or Class X Concrete.

Where joint filler is specified on the plans it shall conform to Standard Specification 157.2.5.

All bridge guardrail, guardrail posts together with bolts for holding posts in place and bolts and washers for holding precast concrete units together shall be cleaned and painted in accordance with Standard Specification 86.4.2. or galvanized in accordance with Standard Specification 55.2.8.



LOCATION SKETCH

ESTIMATED QUANTITIES				
Item		Substr.	Superstr.	Total
Bituminous Surface	Sq. Yds.		144	144
Precast Concrete Piles in Place	Lin. Ft.	405		405
Precast Concrete Pile Cut-offs	Lin. Ft.	45		45
Class A or Class X Concrete	Cu. Yds.		43.8	43.8
Class B Concrete	Cu. Yds.	24.2	4.6	28.8
Reinforcing Steel	Lbs.	2,330	10,140	12,470
Bridge Guard Rail (Steel Single Rail)	Lin. Ft.		130	130
Test Pile	Lin. Ft.	40		40

Note: All excavation under structure will be made and paid for as Roadway Excavation.

B.M. #5 Elev. 295.80 N.I.W.R. 36" Pecan 170' Lt. Sta. 66+50

BRIDGE OVER DRY RUN DITCH

STATE ROAD FROM NEW MADRID NORTHEASTERLY

ABOUT 8.0 MILES N.E. OF LILBOURN

PROJECT NO. Sec. 72(1) (SU) **STA.** 69+59.75

NEW MADRID **COUNTY**

SUBMITTED BY D.B. Jenkins BRIDGE ENGINEER DATE 9/24/62

APPROVED BY J.J. Corbett CHIEF ENGINEER DATE 9/24/62

STD. 52.01
STD. 86.00
N-771

No. 311	Dec. 1961	Revised
		Assembled 9-11-62

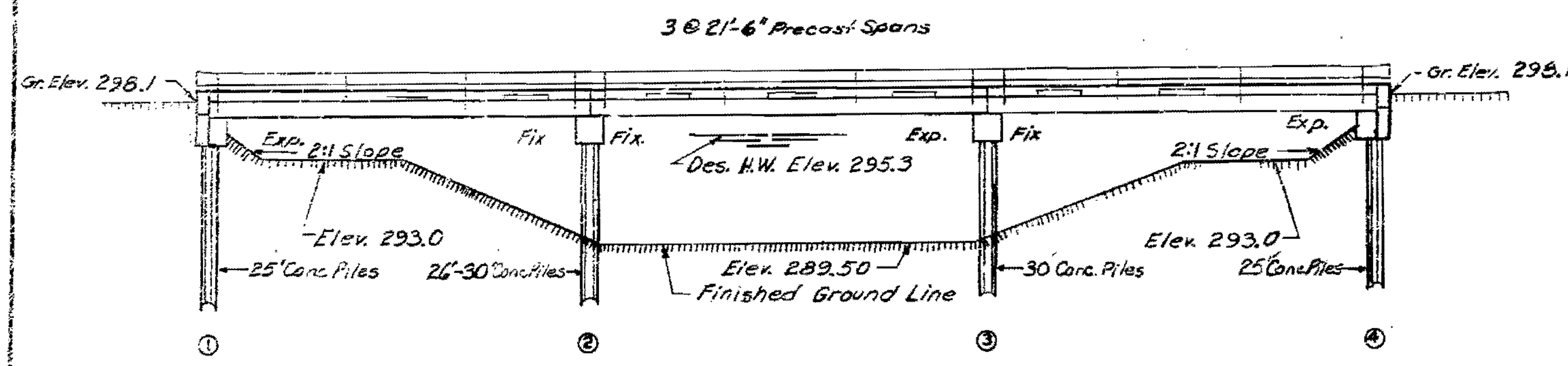
Assembled Sept. 1962 by P.C.L.
Checked Sept. 1962 by L.G.S.

Note: This drawing is not to scale. Follow dimensions.

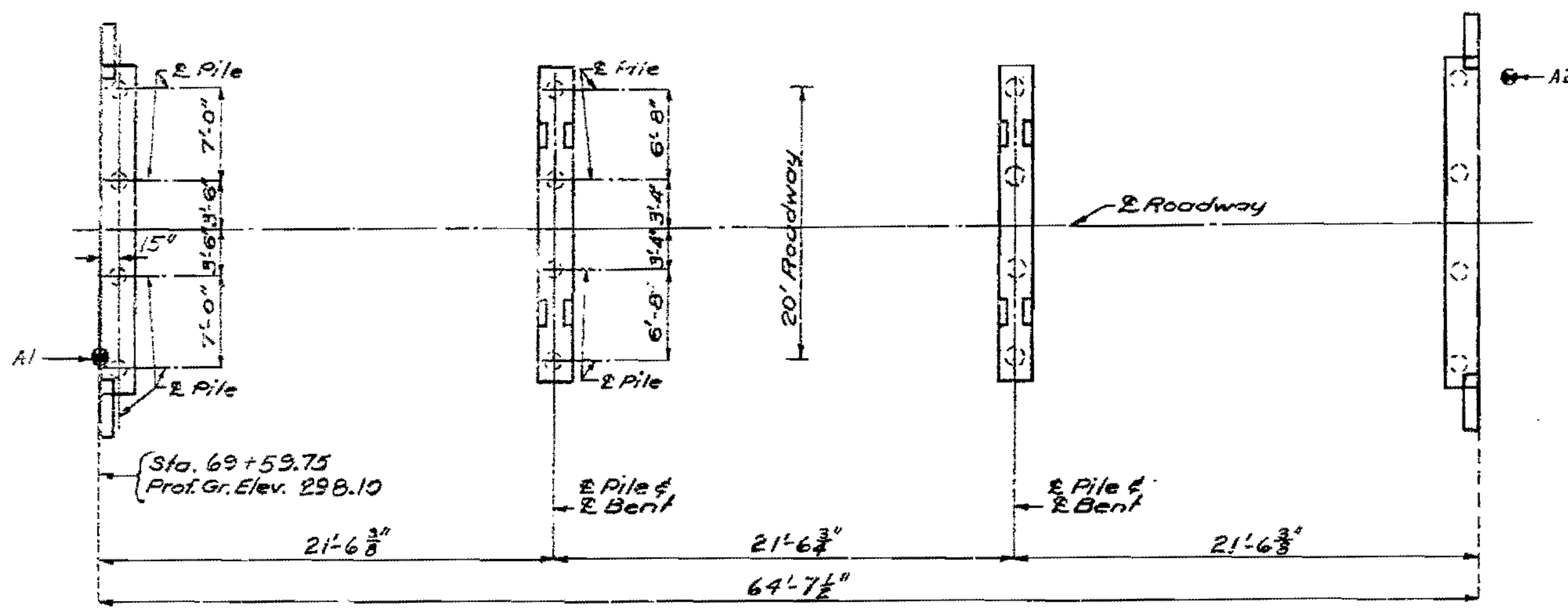
Sheet No. 1 of 3.

MISSOURI STATE HIGHWAY DEPARTMENT

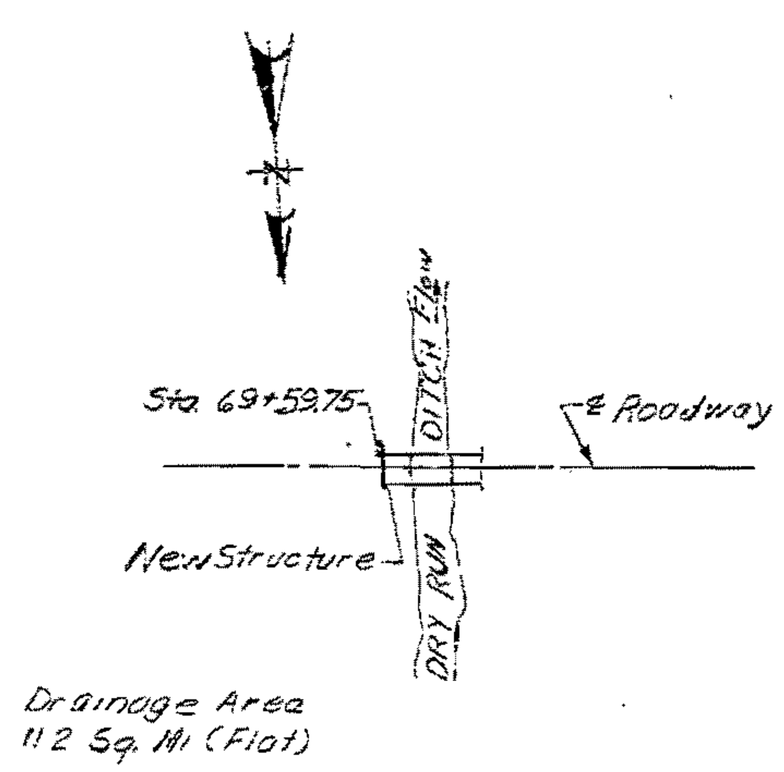
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19		



ELEVATION



PLAN



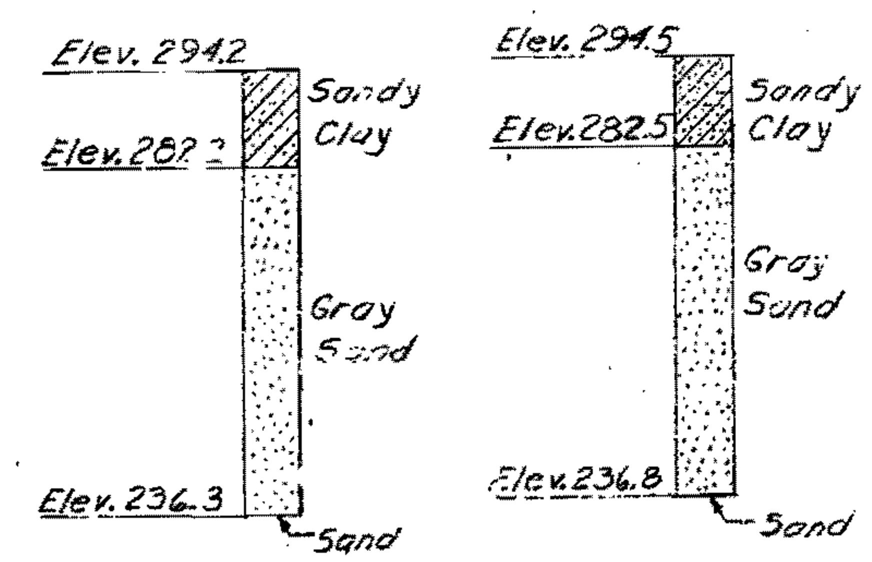
LOCATION SKETCH

FINAL QUANTITIES				
Item		Substr.	Superstr.	Total
Bituminous Surface	Sq. Yds.		144	144
Precast Concrete Piles in Place	Lin. Ft.	343		343
Precast Concrete Pile Cut-offs	Lin. Ft.	67		67
Class X Concrete	Cu. Yds.		43.8	43.8
Class B Concrete	Cu. Yds.	24.2	4.6	28.8
Reinforcing Steel	Lbs.	2330	10140	12470
Bridge Guard Rail (Steel Single Rail)	Lin. Ft.		130	130
Test Pile	Lin. Ft.	40		40

Note: All excavation under structure was made and paid for as Roadway Excavation.

PILE DATA				
Bent No.	1	2	3	4
Pile Type	Precast	Precast	Precast	Precast
Number	4	3	4	4
Aproximate Length Ft.	30	30	30	30
"Plan" Capacity T/Pile	21.0	22.6	22.6	21.0
"Computed" Capacity T/Pile	14.0	18.8	18.8	14.0
Min. Penetration (Pile Tip Elev.)	275.0	270.0	270.0	275.0
Pile Standard	52.01	52.01	52.01	52.01
See Standard Specifications 52.2.6				

Note: One concrete test pile was driven in permanent position for bent No. 2.
All piles were driven to the minimum penetrations noted and to not less than the specified "Plan" capacities.

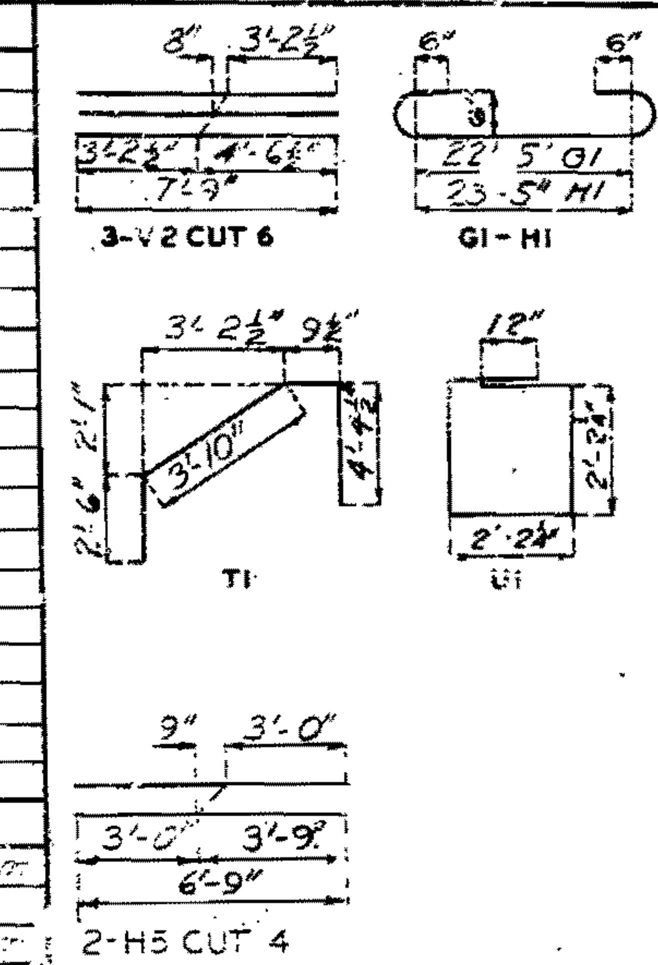


Note: Soundings taken with an auger. Location of soundings marked thus ● on plans.

LOG OF SOUNDINGS

BILL OF REINFORCING STEEL - SUBSTRUCTURE				
No.	Size	Length	Mark	Location
End Bents No. 1 & 4				
8	#6	11'-6"	T1	Wing
16	#6	26'-0"	H1	Beam
4	#6	23'-9"	H2	"
20	#5	5'-0"	H4	Wing
4	#5	6'-9"	H5	"
40	#4	9'-9"	U1	Beam
6	#4	7'-9"	V2	Wing
Int. Bents No. 2 & 3				
40	#4	9'-9"	U1	Beam
16	#6	25'-0"	G1	Beam
4	#6	22'-5"	G2	"

Bending Sketches & Cutting Diagrams



Note: See Sheet No. 3 of 3 for Bill of Reinforcing Steel for superstructure

GENERAL NOTES:

Design Specifications: A.A.S.H.O. 1961
Loading: H15-44 (one lane)
Structural Steel Stress: 20,000 psi
Reinforcing Steel Stress: 20,000 psi
Concrete, Class A Stress: 1,500 psi
Concrete, Class B Stress: 1,200 psi
Concrete, Class X Stress: 1,500 psi

Concrete for Substructure and Superstructure curbs was Class B.

Concrete for precast superstructure units was Class X Concrete.

Where joint filler is specified on the plans it did conform to Standard Specification 157.2.5.

All bridge guardrail, guardrail posts together with bolts for holding posts in place and bolts and washers for holding precast concrete units together was cleaned and painted in accordance with Standard Specification 86.4.2. or galvanized in accordance with Standard Specification 55.2.8.

FINISHED

B.M. #5-A Elev. 299.00
"N.E. Cor. of Wingwall 13' Rt. Sta. 69+92

BRIDGE OVER DRY RUN. DITCH

STATE ROAD FROM NEW MADRID NORTHEASTERLY

ABOUT 8.0 MILES N.E. OF LILBOURN

PROJECT NO. Sec. 72(1) (SU) **STA.** 69+59.75

NEW MADRID **COUNTY**

FINISHED

FINISHED

SUBMITTED BY D.B. Jenkins BRIDGE ENGINEER DATE 9/24/62

APPROVED BY J. J. Corbett ENGINEER DATE 9/24/62

STD. 52.01

STD. 86.00

N-771

447

No.	Revised	
No. 311	Dec. 1961	

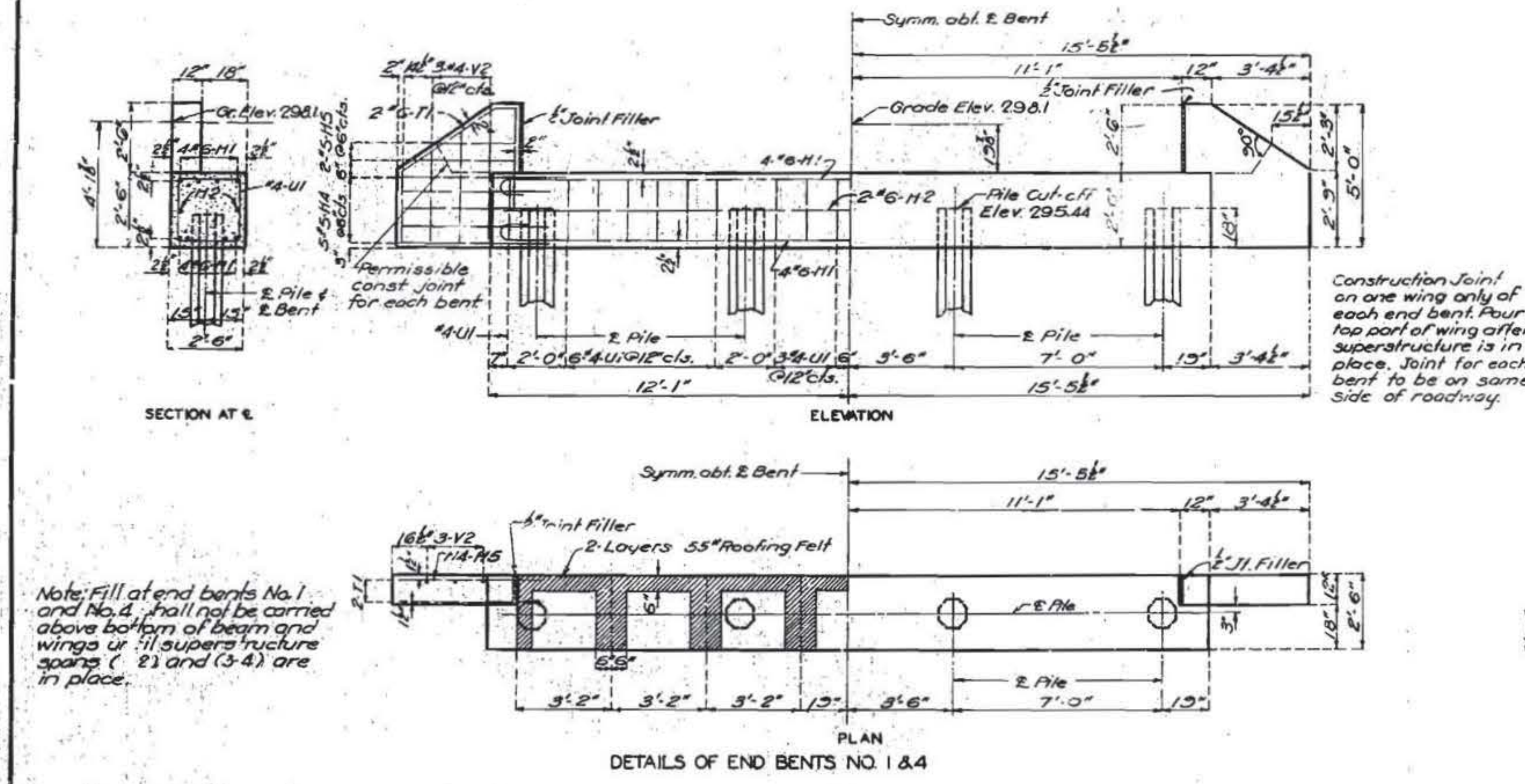
Assembled Sept. 1962 by R.C.
Checked Sept. 1962 by L.G.S.

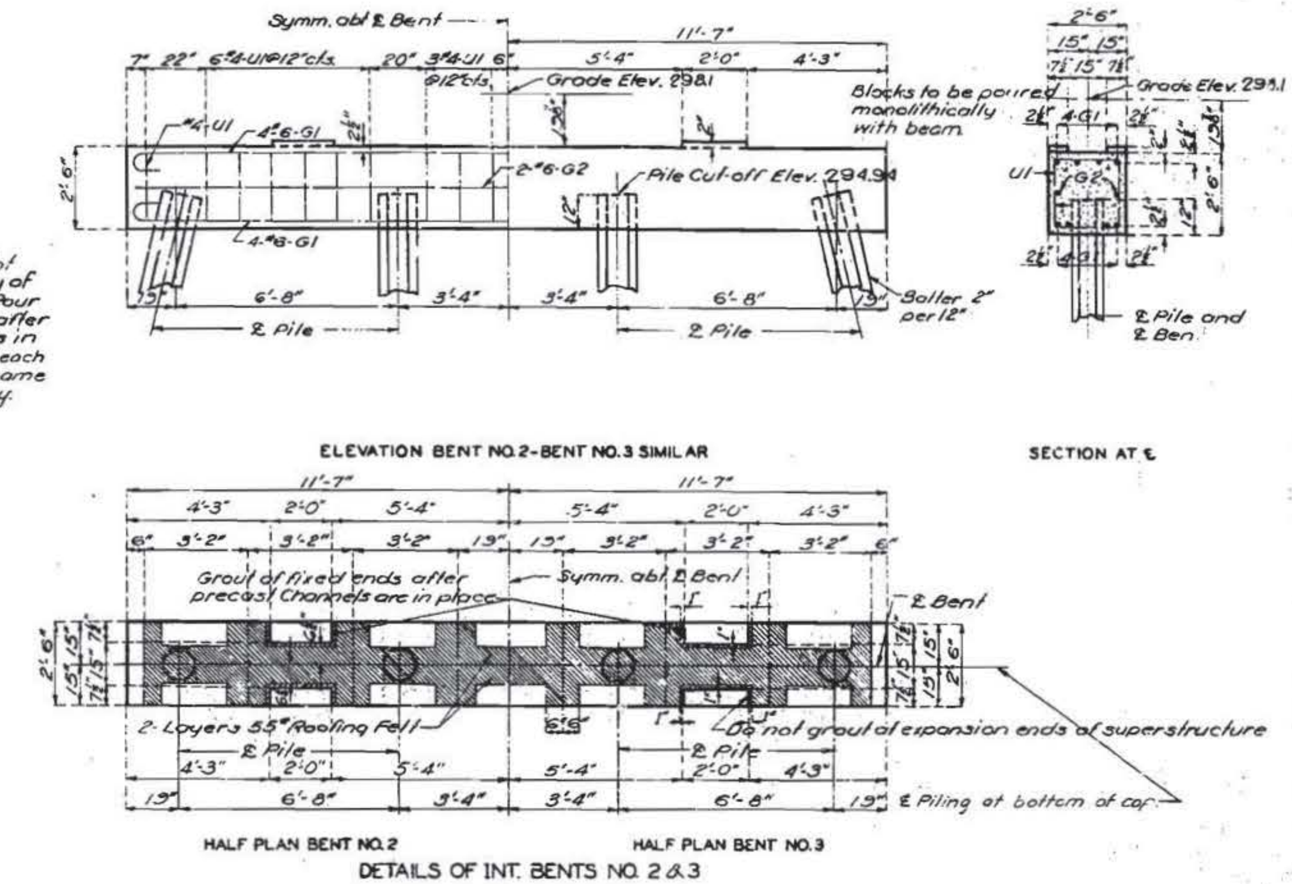
Note: This drawing is not to scale. Follow dimensions.

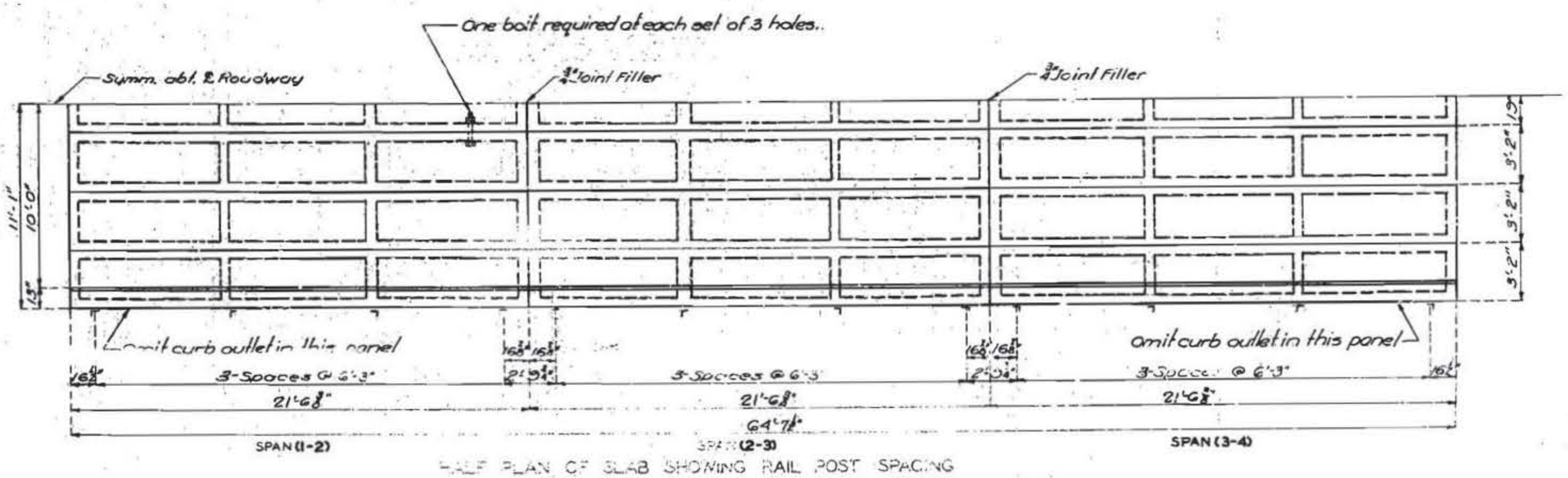
Sheet No. 1 of 1.

MISSOURI STATE HIGHWAY DEPARTMENT

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19	17	







FINISHED

BRIDGE OVER DRY RUN DITCH

STATE ROAD FROM NEW MADRID NORTHEASTERLY
ABOUT 8.0 MILES N.E. OF LILBOURN
PROJECT NO. Sec. 72 (1) (SU) STA. 69+59.75

FINISHED

NEW MADRID COUNTY

FINISHED

N-771

Assembled Oct. 1958 by R.C.L. & B.E.G.
Checked Mar 1959 by M.E.E. & L.G.S.

Note: This drawing is not to scale. Follow dimensions.

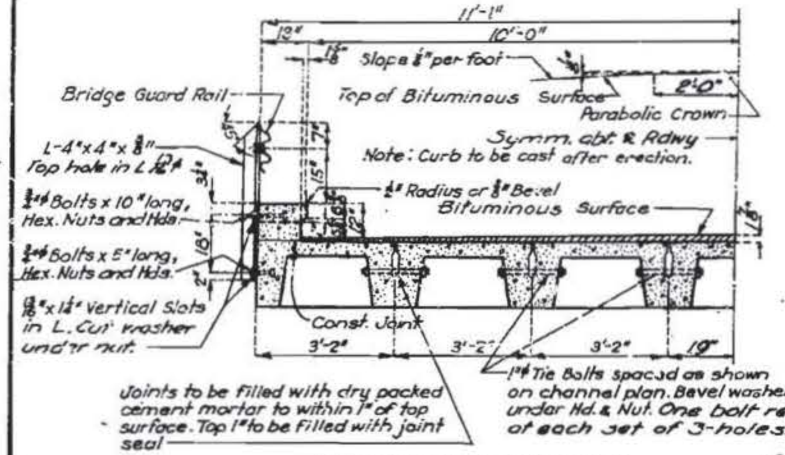
Sheet No. 2 of 3

NO CONSTRUCTION CHANGES

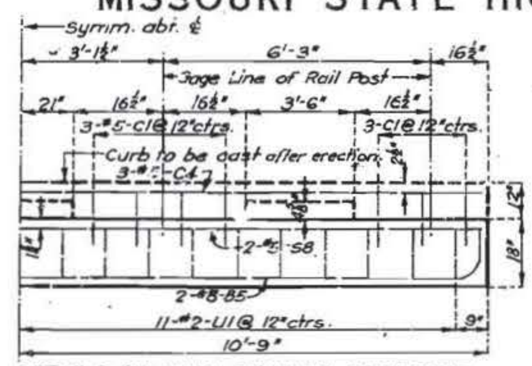
445

MISSOURI STATE HIGHWAY DEPARTMENT

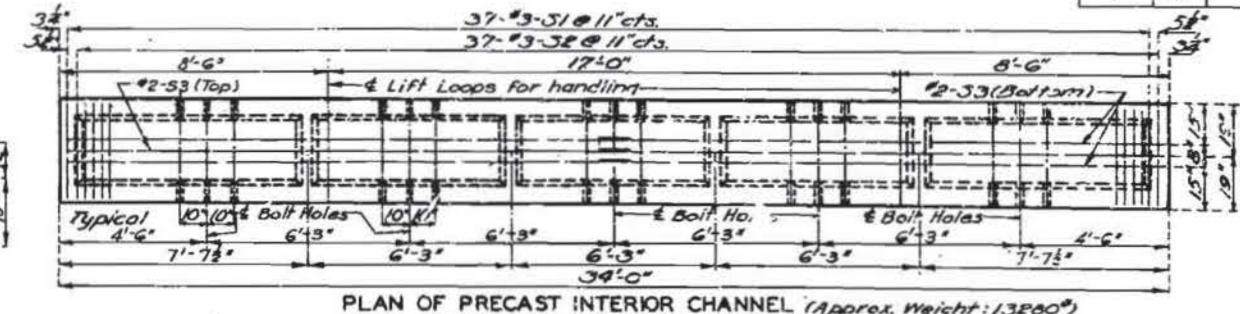
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19	13	



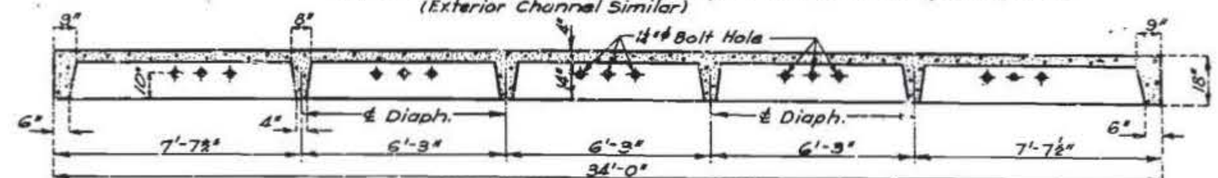
HALF SECTION THRU SPAN



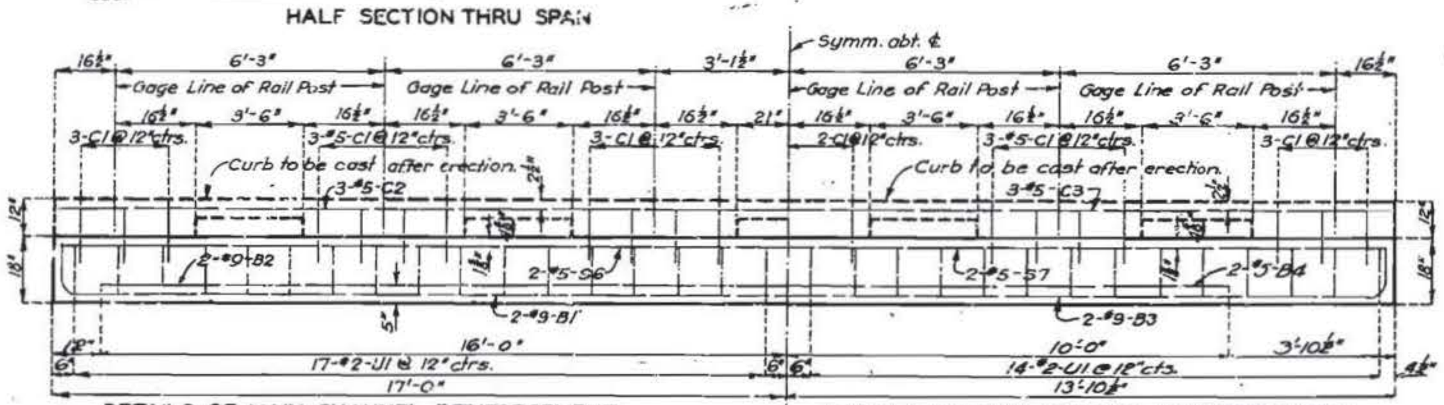
DETAILS OF MAIN CHANNEL REINFORCEMENT FOR 21'-6" SPAN



PLAN OF PRECAST INTERIOR CHANNEL (Approx. Weight: 13280#)
(Exterior Channel Similar)

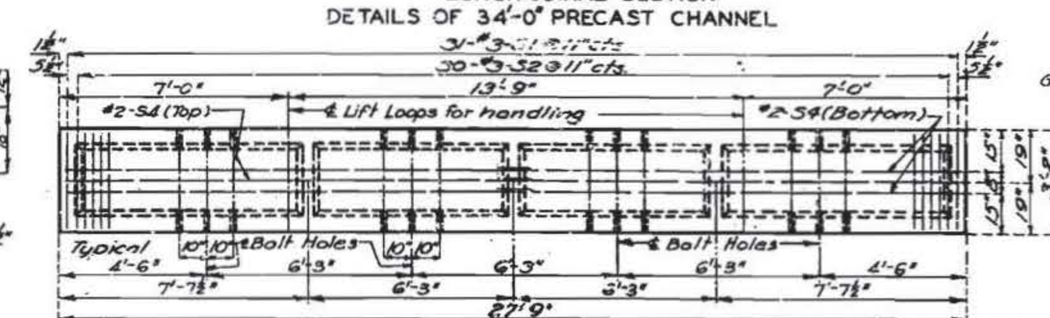


LONGITUDINAL SECTION
DETAILS OF 34'-0" PRECAST CHANNEL

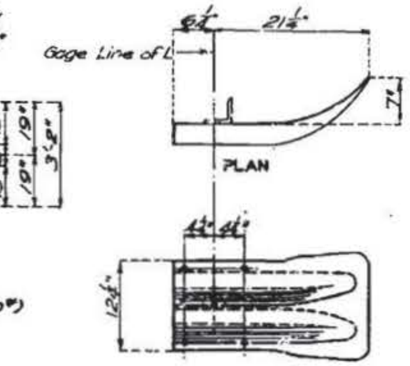


DETAILS OF MAIN CHANNEL REINFORCEMENT FOR 34'-0" SPAN

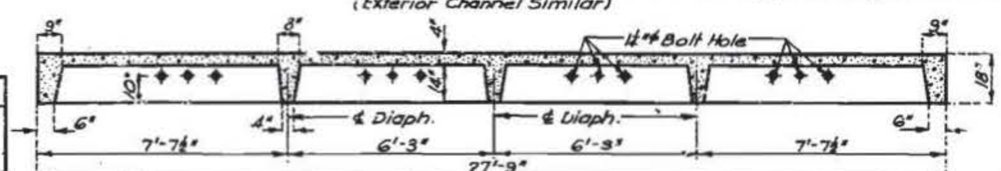
DETAILS OF MAIN CHANNEL REINFORCEMENT FOR 27'-9" SPAN



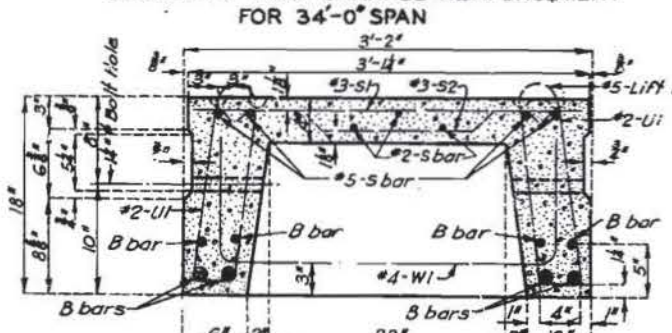
PLAN OF PRECAST INTERIOR CHANNEL (Approx. Weight: 10880#)
(Exterior Channel Similar)



DETAILS OF FLARED ENDS



LONGITUDINAL SECTION
DETAILS OF 27'-9" PRECAST CHANNEL



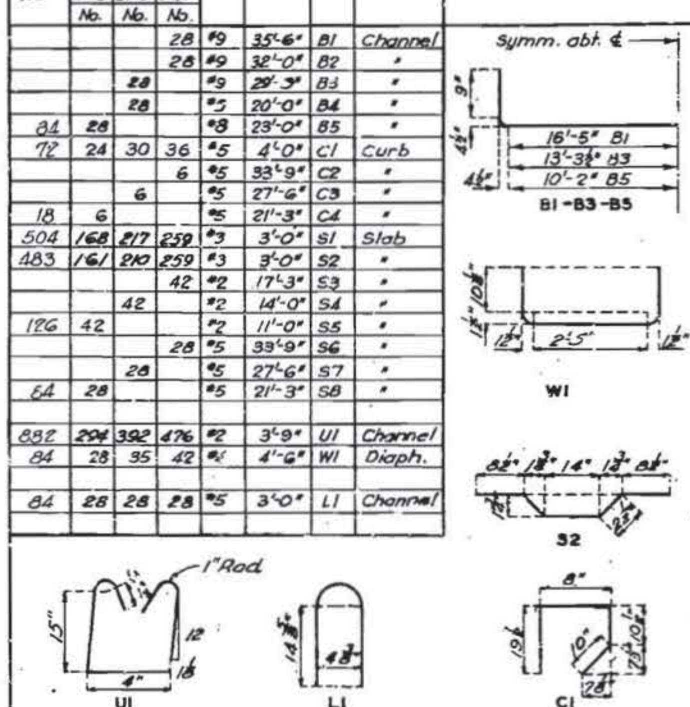
TYPICAL SECTION THRU INT. CHANNEL

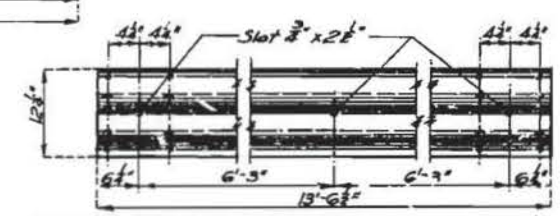
Note: #5 Lift Loops to be cut off in field after units have been set in final position at bridge site and wells to be filled with grout containing iron oxide (Embeco or an approved equivalent) by Contractor, except lift loops under curbs.

BILL OF REINFORCING STEEL FOR SUPERSTRUCTURE

Total No.	One Span 21'-6" No.	One Span 27'-9" No.	One Span 34'-0" No.	Size	Length	Mark	Location
			28	#9	35'-6"	B1	Channel
			28	#9	32'-0"	B2	"
		28		#9	29'-3"	B3	"
		28		#5	20'-0"	B4	"
84	28			#8	23'-0"	B5	"
72	24	30	36	#5	4'-0"	C1	Curb
			6	#5	33'-9"	C2	"
		6		#5	27'-6"	C3	"
18	6			#5	21'-3"	C4	"
504	168	217	259	#3	3'-0"	S1	Slab
483	161	210	259	#3	3'-0"	S2	"
			42	#2	17'-3"	S3	"
		42		#2	14'-0"	S4	"
126	42			#2	11'-0"	S5	"
			28	#5	33'-9"	S6	"
		28		#5	27'-6"	S7	"
84	28			#5	21'-3"	S8	"
882	294	392	476	#2	3'-9"	U1	Channel
84	28	35	42	#4	4'-6"	W1	Diaph.
84	28	28	28	#5	3'-0"	L1	Channel

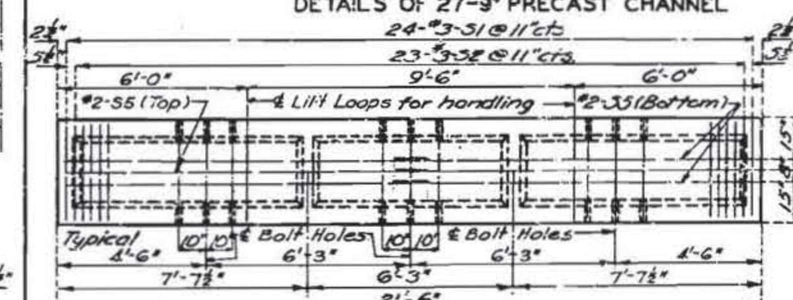
Bending Sketches



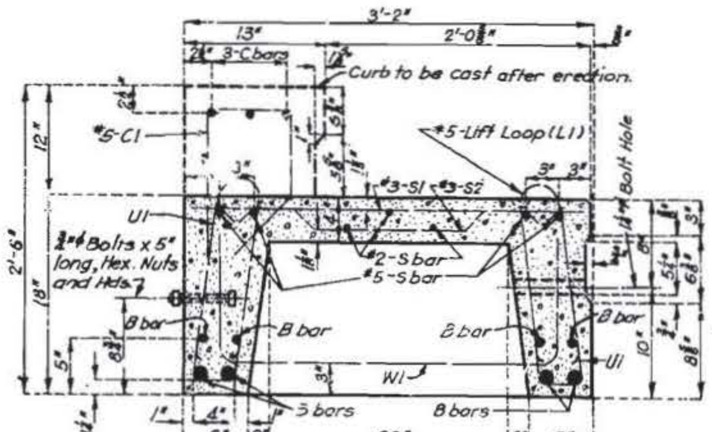


ELEVATION OF AN INTERMEDIATE PANEL
TYPICAL DETAILS OF RAIL

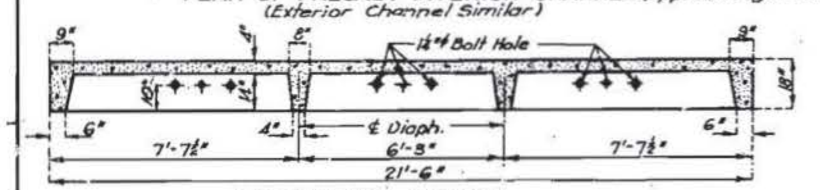
Note: Guard rail to be of 12 gage high carbon steel sheet. Rail to be secured to post by one 5/8"# heat treated bolt. Sheets to be lapped 12 1/2" and connected by 6-5/8"# heat treated bolts. All bolt holes to be 11/16"x1 1/2" except as noted. Details of rail sheet and splices shall conform with the details shown on Missouri State Highway Guard Rail Std. 86.00



PLAN OF PRECAST INTERIOR CHANNEL (Approx. Weight: 8470#)
(Exterior Channel Similar)



TYPICAL SECTION THRU EXT. CHANNEL



LONGITUDINAL SECTION
DETAILS OF 21'-6" PRECAST CHANNEL

Note: Unless otherwise shown or noted all exposed edges are to be beveled 1/2".

FINISHED

BRIDGE OVER DRY RUN DITCH

STATE ROAD FROM NEW MADRID NORTHEASTERLY ABOUT 8.0 MILES N.E. OF LILBOURN

PROJECT NO. (SU) STA. 69+59.75 FINISHED

NEW MADRID COUNTY

FINISHED

446

No. 31.203 Revised Dec. 1961

Assembled Sept. 1942 By R.C.L.
Checked Sept. 1942 By L.G.S.

Note: This drawing is not to scale. Follow dimensions.

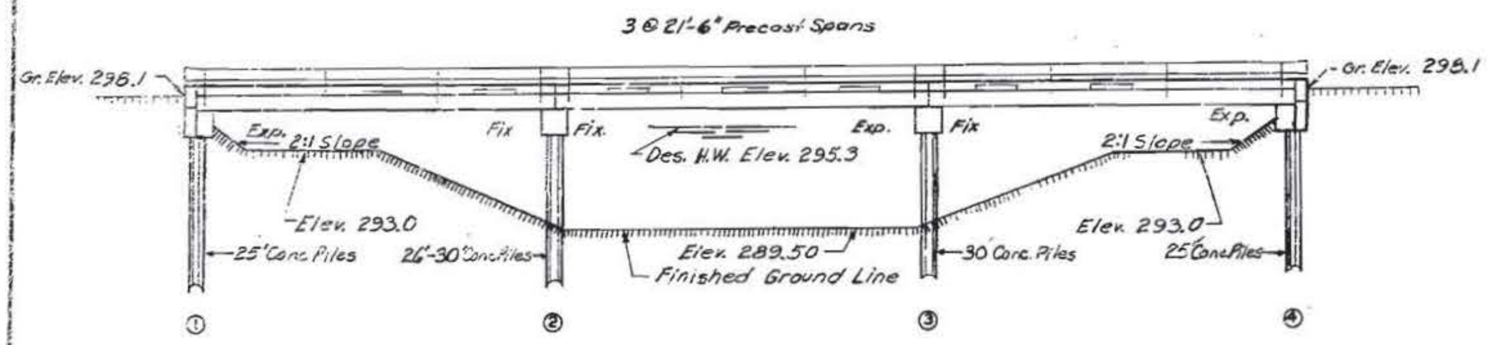
Sheet No. 3 of 3.

N-771

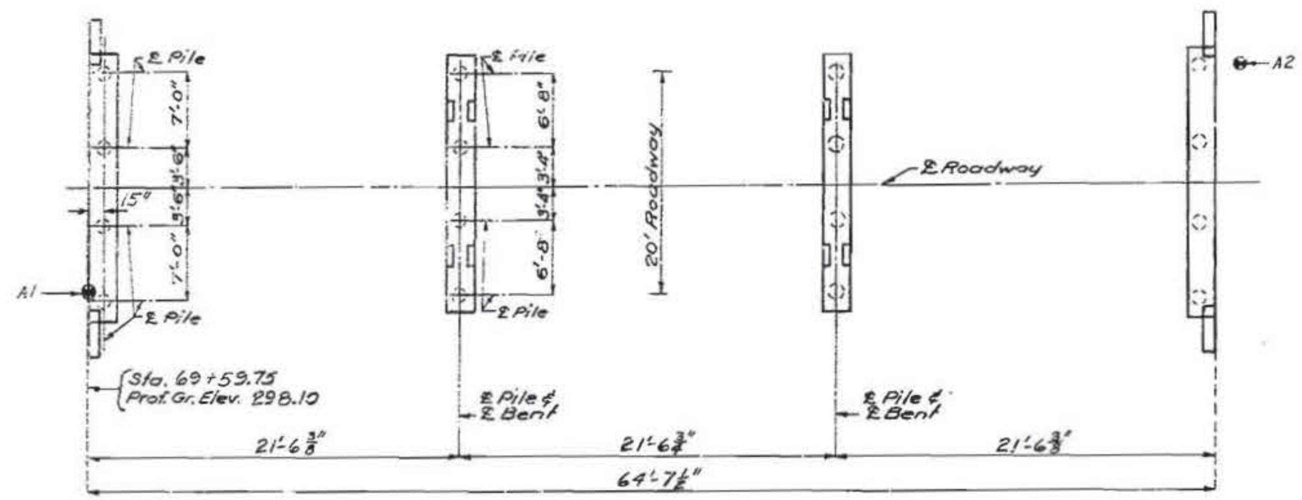
NO CONSTRUCTION CHANGES

MISSOURI STATE HIGHWAY DEPARTMENT

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19		



ELEVATION



PLAN

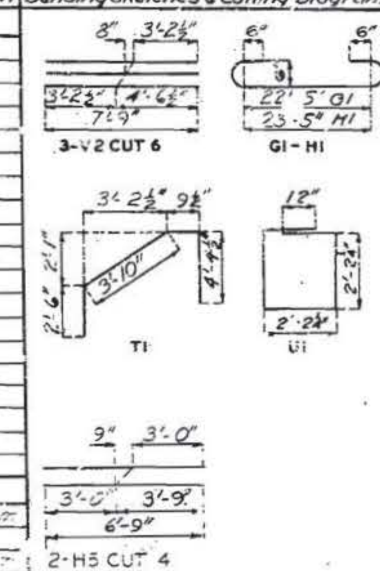
PILE DATA				
Bent No.	1	2	3	4
Pile Type	Precast	Precast	Precast	Precast
Number	4	3	4	4
Aproximate Length Ft.	30	30	30	30
"Plan" Capacity T/Pile	21.0	22.6	22.6	21.0
"Computed" Capacity T/Pile	14.0	18.8	18.8	14.0
Min. Penetration (Pile Tip Elev.)	275.0	270.0	270.0	275.0
Pile Standard	52.01	52.01	52.01	52.01
See Standard Specifications 52.2.6				

Note: One concrete test pile was driven in permanent position for bent No. 2.

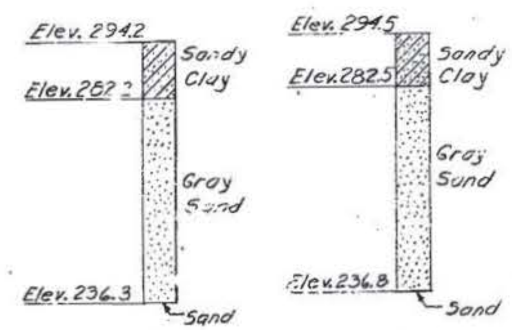
All piles were driven to the minimum penetrations noted and to not less than the specified "Plan" capacities.

BILL OF REINFORCING STEEL - SUBSTRUCTURE				
No.	Size	Length	Mark	Location
End Bents No. 1 & 4				
8	#6	11'-6"	T1	Wing
16	#6	26'-0"	H1	Beam
4	#6	23'-9"	H2	"
20	#5	5'-0"	H4	Wing
4	#5	6'-9"	H5	"
40	#4	9'-9"	U1	Beam
6	#4	7'-9"	V2	Wing
Int. Bents No. 2 & 3				
40	#4	9'-9"	U1	Beam
16	#6	25'-0"	G1	Beam
4	#6	22'-[illegible]	G2	"

Bending Sketches & Cutting Diagrams



Note: See Sheet No. 3 of 3 for Bill of Reinforcing Steel for superstructure



Note: Soundings taken with an auger. Location of soundings marked thus ● on plans.

LOG OF SOUNDINGS

GENERAL NOTES:

Design Specifications: A.A.S.H.O. 1961
Loading: H15-44 (one lane)
Structural Steel Stress: 20,000 psi
Reinforcing Steel Stress: 20,000 psi
Concrete, Class A Stress: 1,500 psi
Concrete, Class B Stress: 1,200 psi
Concrete, Class X Stress: 1,500 psi

Concrete for Substructure and Superstructure curbs was Class B.

Concrete for precast superstructure units was Class X Concrete.

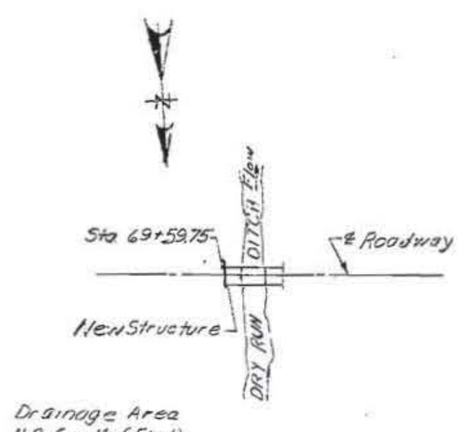
Where joint filler is specified on the plans it did conform to Standard Specification 1E7.2.E.

All bridge guardrail, guardrail posts together with bolts for holding posts in place and bolts and washers for holding precast concrete units together was cleaned and painted in accordance with Standard Specification 86.4.2. or galvanized in accordance with Standard Specification 55.2.8.

FINAL QUANTITIES				
Item		Substr.	Superstr.	Total
Bituminous Surface	Sq. Yds.		144	144
Precast Concrete Piles in Place	Lin. Ft.	343		343
Precast Concrete Pile Cut-offs	Lin. Ft.	67		67
Class X Concrete	Cu. Yds.		43.8	43.8
Class B Concrete	Cu. Yds.	24.2	4.6	28.8
Reinforcing Steel	Lbs.	2330	10140	12470
Bridge Guard Rail (Steel Single Rail)	Lin. Ft.		130	130
Test Pile	Lin. Ft.	40		40

Note: All excavation under structure was made and paid for as Roadway Excavation.

447



Drainage Area 11.2 Sq. Mi. (Flat)

LOCATION SKETCH

FINISHED

B.M. #5-A Elev. 299.00
"N.E. Cor. of Wingwall 13' Rt. Sta. 69+92

BRIDGE OVER DRY RUN DITCH

STATE ROAD FROM NEW MADRID NORTHEASTERLY

ABOUT 8.0 MILES N.E. OF LILBOURN

PROJECT NO. Sec. 72(1) (SU) **STA.** 69+59.75

NEW MADRID **COUNTY**

FINISHED

FINISHED

SUBMITTED BY D.B. Jenkins BRIDGE ENGINEER DATE 9/24/62

APPROVED BY J. J. Corbett ENGINEER DATE 9/24/62

STD. 32.01
STD. 86.00
N-771

No. 511	Dec. 1961	Revised	

Assembled Sept. 1962 by R.C.
Checked Sept. 1962 by L.G.S.

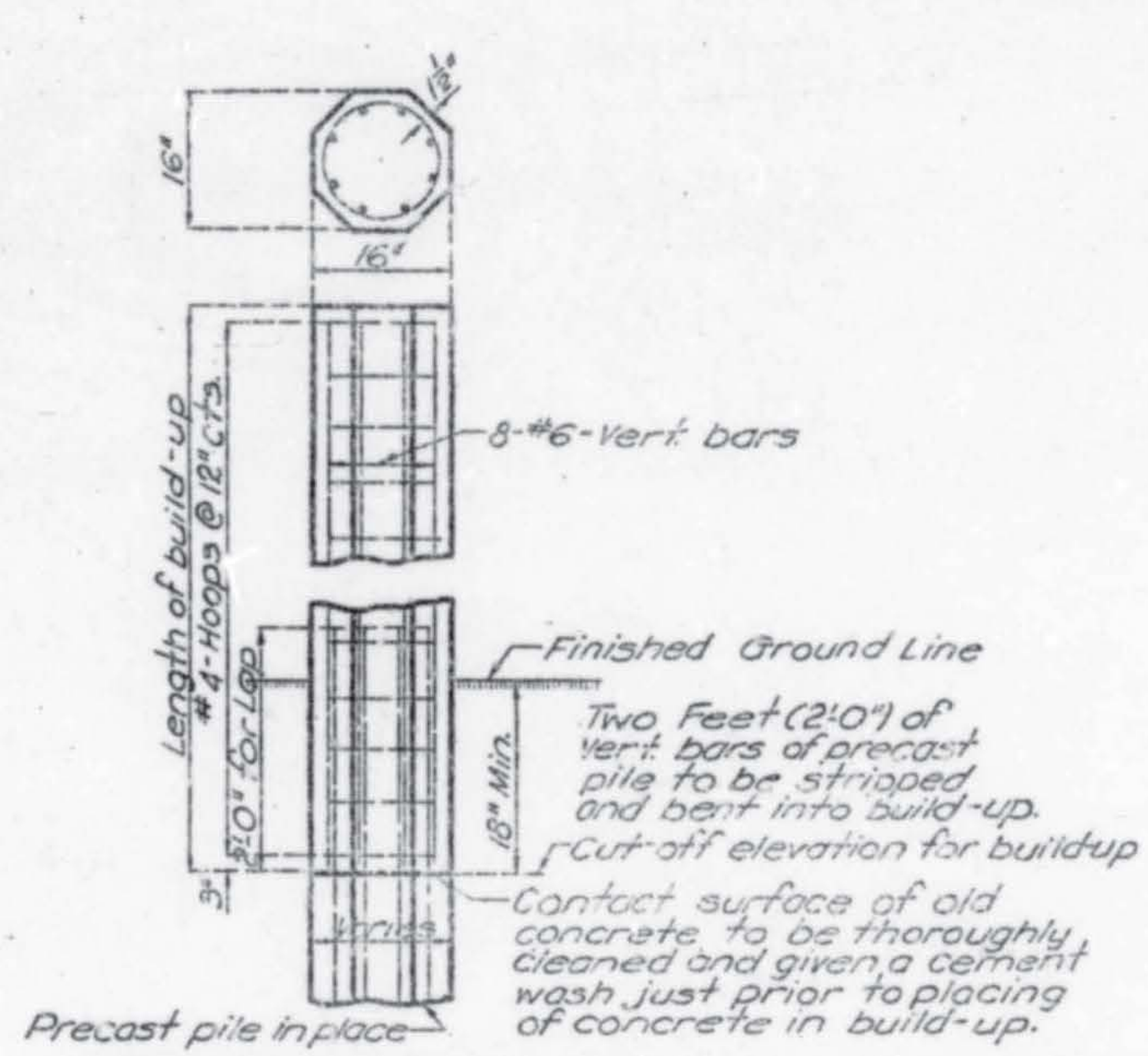
Note: This drawing is not to scale. Follow dimensions.

Sheet No. 1 of 1.

MISSOURI STATE HIGHWAY DEPARTMENT

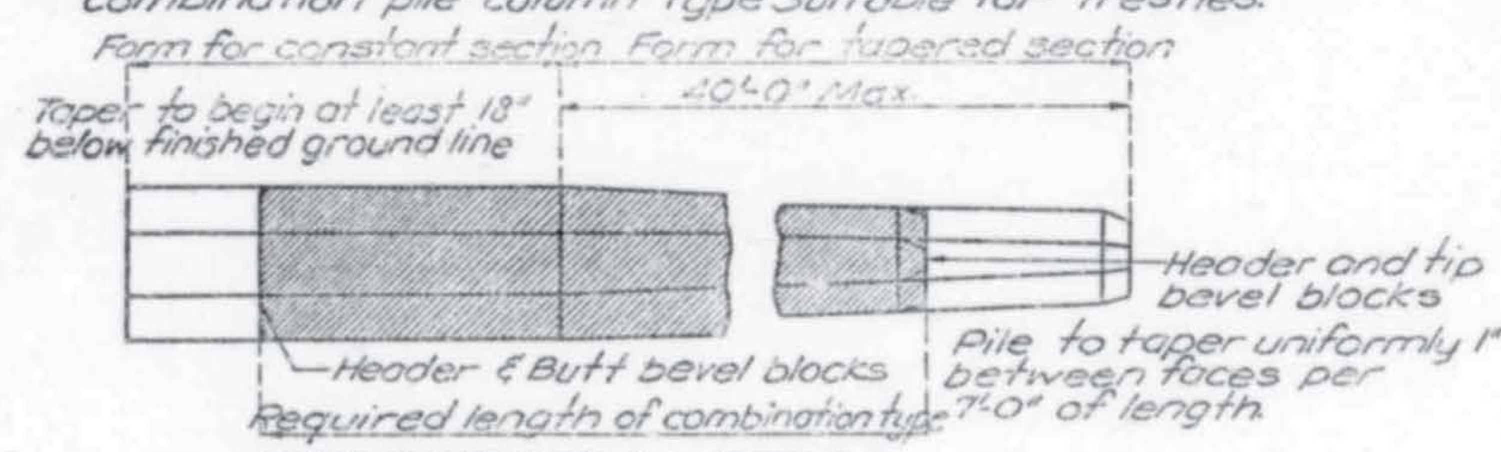
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	MO.		19		

FINAL PLANS

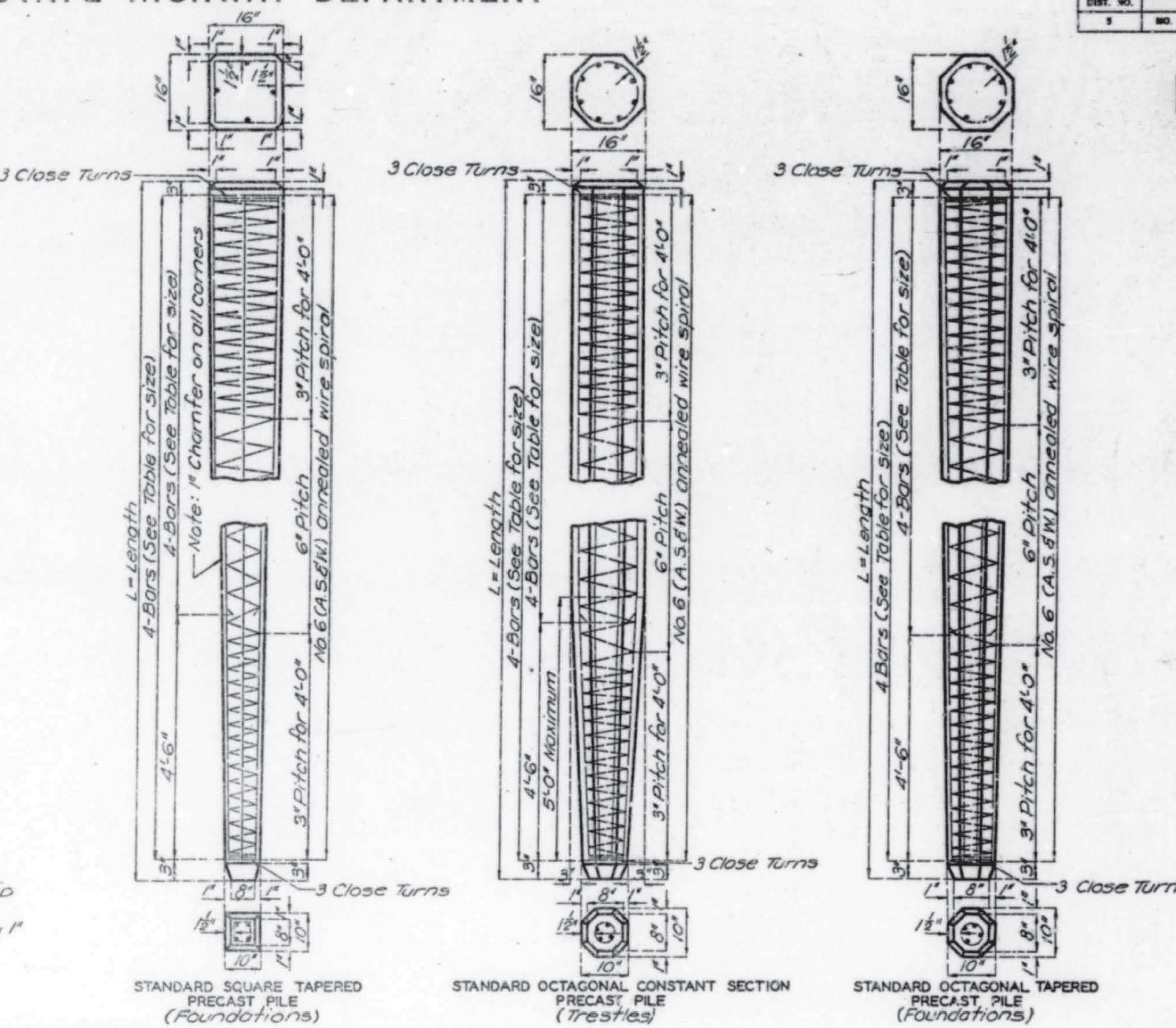


COMBINATION PILE BUILD UP WITH PRECAST PILE (WITHOUT DRIVING)
(Trestles)

Note: To standardize and simplify forming octagonal tapered piles a standard taper 1" between faces per 7'-0" of length may be used for all lengths up to and including 40'-0". All these piles shall have a 16" octagonal butt and this butt shall be extended as a 16" octagonal constant section for lengths over 40'-0". This will permit using the same forms for casting piles of any length with tips in all cases larger than the minimum allowed. These same forms may also be used for casting the combination pile-column type suitable for trestles.



FORMS SUITABLE FOR ALL LENGTHS & COMBINATIONS OF OCTAGONAL PILES



STANDARD SQUARE TAPERED PRECAST PILE
(Foundations)

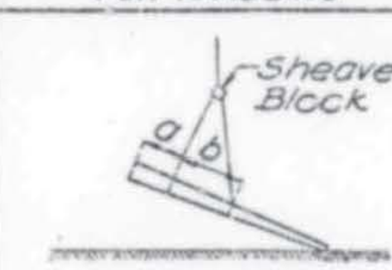
STANDARD OCTAGONAL CONSTANT SECTION PRECAST PILE
(Trestles)

STANDARD OCTAGONAL TAPERED PRECAST PILE
(Foundations)

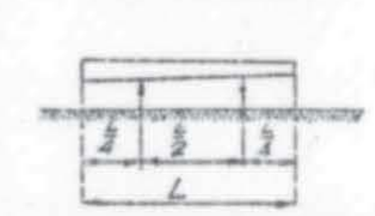
TABLE OF PILE DATA

SQUARE TAPERED				OCTAGONAL CONSTANT SECTION				OCTAGONAL TAPERED			
Length of Piling	Long. Reinf.	Pick-Up a	Pick-Up b	Length of Piling	Long. Reinf.	Pick-Up a	Pick-Up b	Length of Piling	Long. Reinf.	Pick-Up a	Pick-Up b
15'-0" to 25'-0", Incl.	8-#5	2'-0"	0	15'-0" to 26'-0", Incl.	8-#5	2'-0"	0	15'-0" to 25'-0", Incl.	8-#5	2'-0"	0
26'-0" to 36'-0", Incl.	8-#5	L/4	0	27'-0" to 36'-0", Incl.	8-#5	L/4	0	26'-0" to 37'-0", Incl.	8-#5	L/4	0
37'-0" to 39'-0", Incl.	8-#6	L/2	0	37'-0" to 41'-0", Incl.	8-#6	L/2	0	38'-0" to 40'-0", Incl.	8-#6	L/4	0
40'-0"	8-#6	10'-3"	0	42'-0" to 44'-0", Incl.	8-#7	L/4	0	41'-0"	8-#6	10'-6"	0
41'-0"	8-#6	10'-9"	0	45'-0" to 47'-0", Incl.	8-#8	L/2	0	42'-0"	8-#6	11'-0"	0
42'-0"	8-#6	11'-3"	0	48'-0" & 49'-0"	8-#8	13'-0"	0	43'-0"	8-#6	11'-6"	0
43'-0"	8-#6	11'-9"	0	50'-0"	8-#8	14'-0"	0	44'-0"	8-#6	12'-0"	0
44'-0"	8-#7	11'-9"	0	51'-0" & 52'-0"	8-#8	15'-0"	0	45'-0"	8-#7	12'-3"	0
45'-0"	8-#7	12'-3"	0	53'-0" to 56'-0", Incl.	8-#8	13'-0"	8'-0"	46'-0"	8-#7	12'-6"	0
46'-0"	8-#7	12'-6"	0	57'-0" to 59'-0", Incl.	8-#8	13'-0"	12'-0"	47'-0"	8-#7	12'-9"	0
47'-0"	8-#8	12'-6"	0	60'-0"	8-#8	13'-0"	16'-0"	48'-0"	8-#7	13'-3"	0
48'-0"	8-#8	13'-0"	0					49'-0"	8-#8	13'-6"	0
49'-0"	8-#8	13'-6"	0					50'-0"	8-#8	13'-9"	0
50'-0" to 56'-0", Incl.	8-#8	12'-0"	10'-0"					51'-0"	8-#8	14'-0"	0
57'-0" to 60'-0", Incl.	8-#8	12'-0"	14'-0"					52'-0" to 57'-0", Incl.	8-#8	13'-0"	8'-0"
								53'-0" to 60'-0" Incl.	8-#8	13'-0"	12'-0"

POINTS OF SUPPORT FOR HANDLING



PICK UP POINTS FOR UP ENDING



POINTS OF SUPPORT FOR HORIZONTAL HANDLING

GENERAL NOTES:

All concrete shall be Class A

Only Standard Octagonal Constant Section piling or combination pile build-ups shall be used as trestle piles for intermediate bents except for that portion of pile which lies 18" or more below the finished ground line.

Sections of precast concrete pile shall not be spliced together but build-ups may be used as follows: Where no additional driving is required, either of the build-ups (without driving) shown on Sheets 1 or 2 may be used. If additional driving is required, the build-up (with driving) shown on sheet 2 shall be used.

APPROVED TYPES OF 16" PRECAST CONCRETE PILES

SUBMITTED BY D. B. Jenkins BRIDGE ENGINEER DATE 5-15-1962

APPROVED BY M. J. Snider CHIEF ENGINEER DATE 1/2/63

Drawn Sept 1959 by W.G.S.
Checked Sept 1959 by J.E.L.

Note: This drawing is not to scale. Follow dimensions.

Revised		
5-62		

Sheet No. 1 of 2

52.01

MEMORANDUM

Missouri Department of Transportation

Construction - Materials
Central Laboratory

TO: Dean Franke-br

CC/ATT: Bill Dunn-br
Andrew Meyer-se/cm
Kevin Plott-se/cm
Corbin Carlton-se/cm

FROM: Paul Hilchen
Geotechnical Engineer

DATE: March 7, 2016

SUBJECT: Materials
Geotechnical Section
Foundation Investigation for
Structure No. A8414
Job No. J9S3034
Route U, New Madrid County

General - A foundation investigation has been performed for the above referenced structure. This project site is located in New Madrid County where Route U crosses over Dry Run Ditch about 2.9 miles Northeast of New Madrid, Missouri.

A formal Sounding Request has been provided for this site, and it is understood that this proposed structure is included in the STIP with a 2017 letting date. Based upon the available information, it is anticipated that the existing 65-foot long bridge at this site, Structure No. N0771 will be replaced on essentially the same grade and alignment by a proposed similar length bridge, Structure No. A8414. Per existing bridge plans, the existing structure is supported on pile foundations. It is anticipated that the proposed structure will similarly be supported on pile foundations.

Field Investigation – As indicated in Table 1 below, subsurface exploration was performed at two locations at this site. One cone penetration test (CPT) boring, H-16-12, was performed near the east end of the existing structure using Hogentogler CPT track-mounted equipment. One standard penetration test (SPT) boring, A-16-03, was performed near the west end of the existing structure using Failing 1500 truck-mounted equipment. The subsurface exploration locations for this site investigation are shown with respect to the site on Figure 1 – Subsurface Exploration Location Aerial Map.

A subsurface diagram showing the subsurface exploration conditions encountered with respect to stationing is attached as Figure 2 - Subsurface Diagram. Logs of the individual subsurface exploration locations are attached, as are summary sheets providing input parameters for software programs LPile and Driven.

Table 1 – Subsurface Exploration Locations

Subsurface Exploration Location	Comment
Sta. 69+44.1, 17.9L, Elev. 297.0 ft.	CPT Boring, H-16-12, Northing: 290628.8, Easting: 1109128.7
Sta. 70+40.1, 10.0R, Elev. 298.1 ft.	SPT Boring, A-16-03, Northing: 290656.9, Easting: 1109032.7

Analyses - Also attached are preliminary pile capacity graphs for 14-in. and 16-in. diameter cast-in-place steel pipe piles showing ultimate pile capacity and factored pile capacity. For A-16-03, these graphs are based upon SPT data, Nordlund Analysis using DRIVEN 1.2 software, and a resistance factor of 0.45. For H-16-13, these graphs are based upon CPT data, LCPC Analysis using CPeT-IT v.1.7.6.42 software, and a resistance factor of 0.45.

cs
j:\sublec\paul\a8414_j9s3034_ltr.doc
Attachments

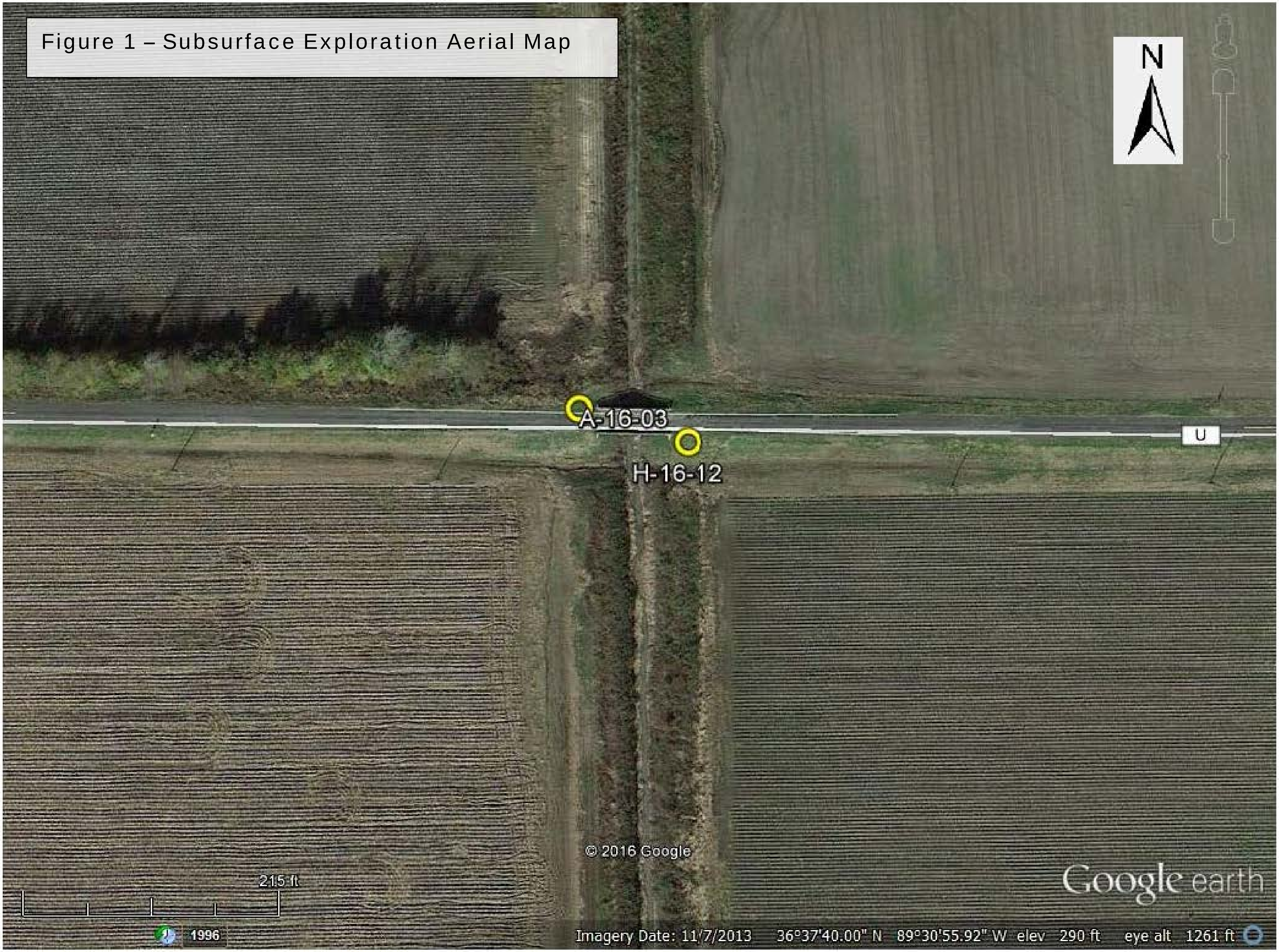
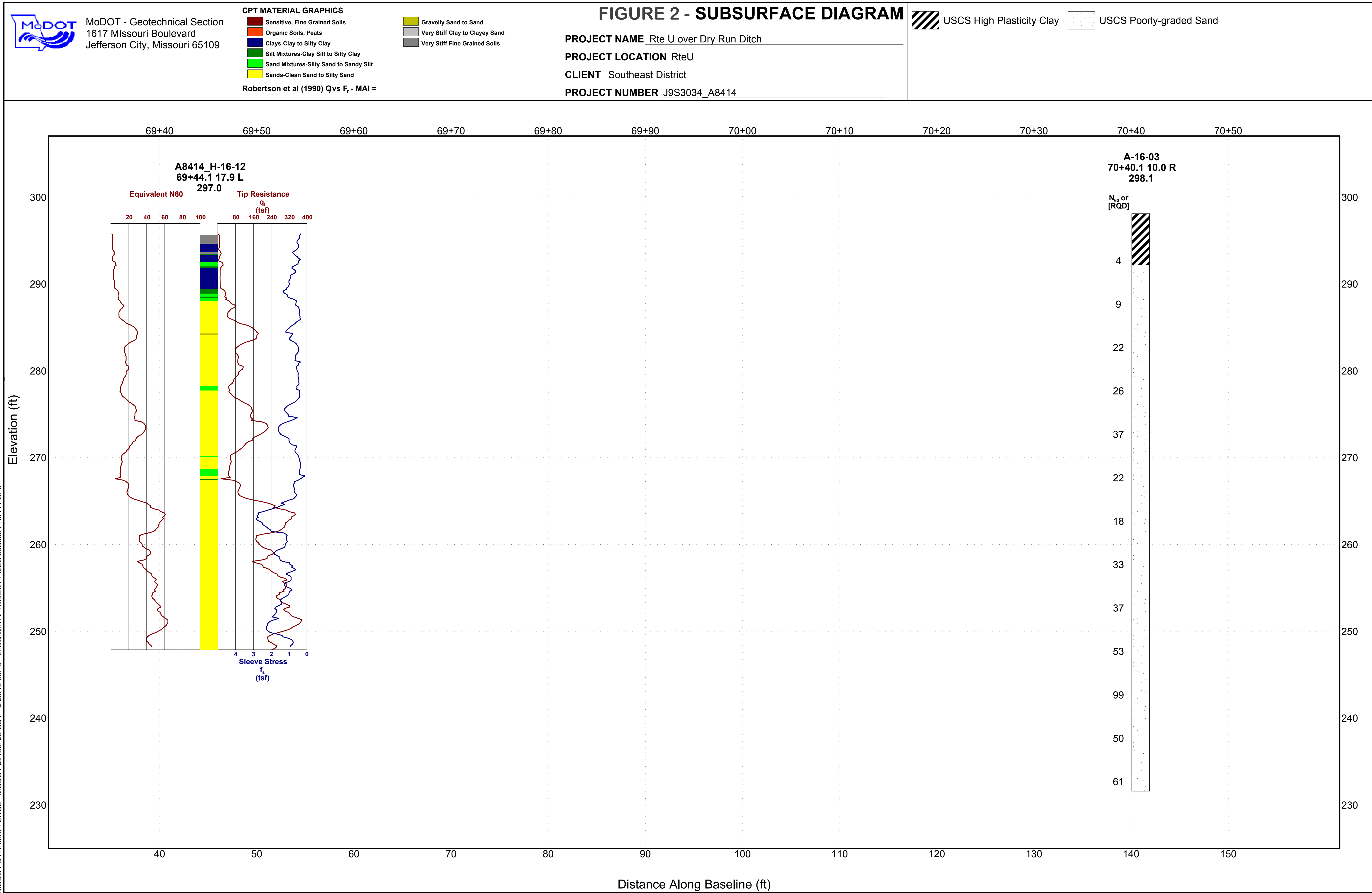


Figure 1 – Subsurface Exploration Aerial Map
N
A-16-03
H-16-12
U
© 2016 Google
215 ft
Google earth
1996
Imagery Date: 11/7/2013 36°37'40.00" N 89°30'55.92" W elev 290 ft eye alt 1261 ft

MoDOT - Geotechnical Section
1617 Missouri Boulevard
Jefferson City, Missouri 65109

FIGURE 2 - SUBSURFACE DIAGRAM

PROJECT NAME Rte U over Dry Run Ditch
PROJECT LOCATION RteU
CLIENT Southeast District
PROJECT NUMBER J9S3034_A8414



MODOT DYNAMIC FENCE - MODOT 20150728.GDT - 2/29/16 08:10 - J:\SG\GINT\PROJECT FILES\J9S3034-A8414.GPJ

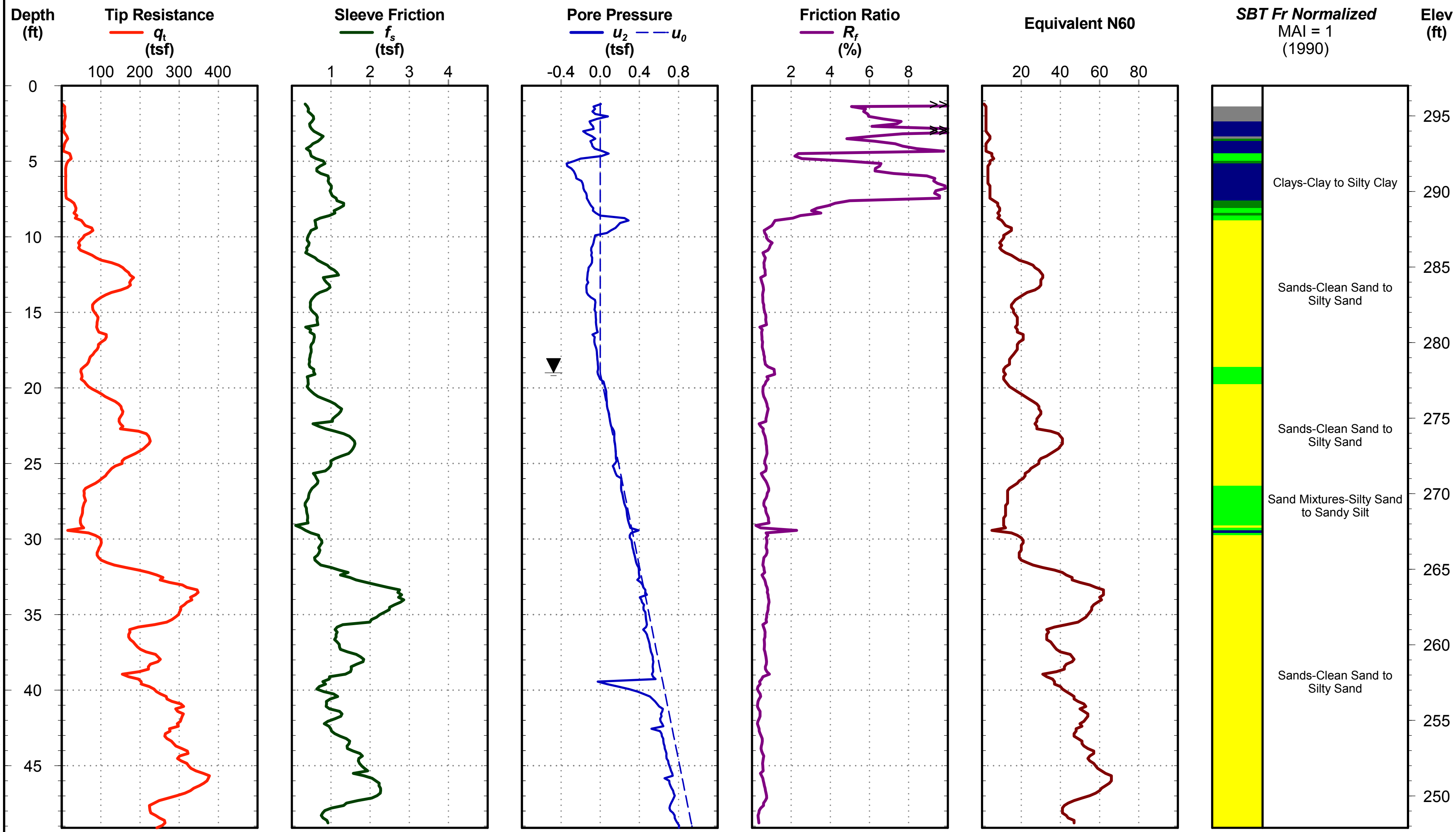
Cone Penetration Test A8414_H-16-12

Rte U over Dry Run Ditch
New Madrid Co. (MO)
Project Number:J9S3034

Date: Jan. 12, 2016
Estimated Water Depth: 19 ft
Rig/Operator: Hilchen

Northing: 290628.8
Easting: 1109128.7
Elevation: 297.0 NAD 83 (CONUS)

Total Depth: 49.1 ft
Termination Criteria:
Cone Size:



CPT REPORT - DYNAMIC - MODOT 20150728.GDT - 3/2/16 16:07 - J:\SG\GINT\PROJECT FILES\J9S3034-A8414.GPJ

A8414_H-16-12

Page 1 of 1

Electronic File Name: A8414_H-16-12.cpt

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3034	**County:** New Madrid	**Route:** U
Design: A8414	**Skew:** Right angles	**Location:** New Madrid Co.
Bent: 4	**Logged By:** George Davis	**Operator:** Kenny Mathews
Station: 70+40.1	**Northing:** 290656.935	**Date of Work:** 01/06/16-01/06/16
Offset: 10.0 R	**Easting:** 1109032.739	**Depth to Water:** 19.0
Elevation: 298.1	**Requested Northing:**	**Depth Hole Open:**
Requested Station:	**Requested Easting:**	**Time Change:**
Requested Offset:	**Equipment:** Failing 1500 ,Split-Spoon Sampler	
Requested Elevation:	**Location Note:**	
Drill No.: G-7887	**Hammer Efficiency:** 79%	**Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
0		0.0-5.9' Gray and brown, FAT CLAY scattered sand, soft to medium stiff, moist							
					53	2-1-2 (4)			LL = 60 PL = 20 MC = 26.6% γ_{sat} = 123 pcf(1)
		5.9-66.5' Gray, SAND, medium dense to very dense, poorly graded	290						
10					67	3-3-4 (9)			
					67	6-10-7 (22)			
			280						
20	▽				53	11-10-10 (26)			
					100	9-12-16 (37)			
			270						
30					100	5-8-9 (22)			
					100	3-6-8 (18)			
			260						
40					100	10-13-12 (33)			
					100	10-12-16 (37)			
			250						
50									

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983   **Coordinate Zone:** Missouri East   **Coordinate Proj. Factor:** 1.000000

Coordinate Datum: NAD 83 (CONUS)   **Coordinate Units:** U.S. Survey Feet

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

Missouri Department of Transportation
Construction and Materials

Job No.: J9S3034 | **County:** New Madrid | **Route:** U
Design: A8414 | **Skew:** Right angles | **Location:** New Madrid Co.
Bent: 4 | **Logged By:** George Davis | **Operator:** Kenny Mathews
Station: 70+40.1 | **Northing:** 290656.935 | **Date of Work:** 01/06/16-01/06/16
Offset: 10.0 R | **Easting:** 1109032.739 | **Depth to Water:** 19.0
Elevation: 298.1 | **Requested Northing:** | **Depth Hole Open:**
Requested Station: | **Requested Easting:** | **Time Change:**
Requested Offset: | **Equipment:** Failing 1500 ,Split-Spoon Sampler
Requested Elevation: | **Location Note:**
Drill No.: G-7887 | **Hammer Efficiency:** 79% | **Drilling Method:** Mud Rotary

Depth (ft)	Graphic	Description	Elevation (ft)	Sample Type	REC % (RQD %)	Blow Counts (N_{60})	Shear Data	Field Tests	Index Tests
50		5.9-66.5' Gray, SAND, medium dense to very dense, poorly graded *(continued)*		X	100	14-19-21 (53)			
				X	100	15-20-55 (99)			
			240						
60				X	100	11-19-19 (50)			
				X	100	12-20-26 (61)			
		Bottom of borehole at 66.5 feet.							

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency; Em - Measured hammer efficiency in percent; Nm - Observed N-value
(1) = Assumed, (2) = Actual

Coordinate System: U.S. State Plane 1983 | **Coordinate Zone:** Missouri East | **Coordinate Proj. Factor:** 1.000000
Coordinate Datum: NAD 83 (CONUS) | **Coordinate Units:** U.S. Survey Feet

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgement of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

MoDOT - Geotechnical Section
1617 MIssouri Boulevard
Jefferson City, Missouri 65109

KEY TO SYMBOLS

CLIENT Southeast District
PROJECT NAME Rte U over Dry Run Ditch
PROJECT NUMBER J9S3034
PROJECT LOCATION New Madrid Co.

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

CH: USCS High Plasticity Clay

SP: USCS Poorly-graded Sand

SAMPLER SYMBOLS

Split-Spoon Sampler

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)
Qu - UNCONFINED COMPRESSIVE STRENGTH (PSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION

▽ Water Level at Time of Drilling
▼ Water Level at End of Drilling
▼ Water Level after Drilling

KEY TO SYMBOLS - MODOT 20150728.GDT - 3/2/16 16:05 - J:\SG\GINT\PROJECT FILES\J9S3034-A8414.GPJ

Missouri Department of Transportation
Construction and Materials

BORING NO. A8414_H-16-12

Job No.: J9S3034
County: New Madrid
Route: U
Logged By: Hilchen
Design: A8414
Skew: Right angles
Coordinate Units: U.S. Survey Feet
Operator: Mike Donahoe
Bent: 1
Location:
Coordinate Proj. Factor: 1.000000
Date of Work: 01/12/16
Station: 69+44.1
Northing: 290628.774
Coordinate System: U.S. State Plane 1983
Depth to Water:
Offset: 17.9 L
Easting: 1109128.663
Coordinate Datum: NAD 83 (CONUS)
Time Change:
Elevation: 297
Drilling Method:
Coordinate Zone: Missouri East
Depth Hole Open:
Drill No.: G-8929
Hammer Efficiency:
Equipment: Hogentogler CPT ,

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle ϕ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
0		0 - 8' Soft clay	290	yes	no	yes	111[1]	111[1]	700[1]			5	16[1]	0.01	50	140		
10		8 - 21' Sand		no	no	yes	121[1]	121[1]				11	38[1]		92			
			280				121[1]	121[1]				16	38[1]		92			
20	▽(1)						121[1]	59[1]				16	38[1]		55			
		21 - 26' Sand		no	no	yes	124[1]	62[1]				39	42[1]		125			
30		26 - 32' Sand	270	no	no	yes	121[1]	59[1]				11	34[1]		55			
		32 - 49.1' Sand		no	no	yes												

LPILE SUMMARY - MODOT_20150812.GDT - 3/1/16 13:41 - J:\SG\GINT\PROJECT FILES\J9S3034-A8414.GPJ

(1) = Assumed, (2) = Actual, (3) = Phi' 0 *(Continued Next Page)*

Missouri Department of Transportation
Construction and Materials

BORING NO. A8414_H-16-12

Job No.: J9S3034	**County:** New Madrid	**Route:** U	**Logged By:** Hilchen
Design: A8414	**Skew:** Right angles	**Coordinate Units:** U.S. Survey Feet	**Operator:** Mike Donahoe
Bent: 1	**Location:**	**Coordinate Proj. Factor:** 1.000000	**Date of Work:** 01/12/16
Station: 69+44.1	**Northing:** 290628.774	**Coordinate System:** U.S. State Plane 1983	**Depth to Water:**
Offset: 17.9 L	**Easting:** 1109128.663	**Coordinate Datum:** NAD 83 (CONUS)	**Time Change:**
Elevation: 297	**Drilling Method:**	**Coordinate Zone:** Missouri East	**Depth Hole Open:**
Drill No.: G-8929	**Hammer Efficiency:**	**Equipment:** Hogentogler CPT ,	

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Scour Parameters: Downdrag	Scour Parameters: Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle ϕ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
		32 - 49.1' Sand *(continued)*	260				124(1)	62(1)				54	43(1)		125			
40				no	no	yes	127(1)	65(1)				52	43(1)		125			
			250				124(1)	62(1)				41	42(1)		125			
		Bottom of borehole at 49.1 feet.																

LPILE SUMMARY - MODOT_20150812.GDT - 3/1/16 13:41 - J:\SG\GINTPROJECT FILES\J9S3034-A8414.GPJ

(1) = Assumed, (2) = Actual, (3) = Phi' 0

BORING NO. A-16-03

Job No.: J9S3034_A8414
County: New Madrid
Route: U
Logged By: George Davis
Design: A8414
Skew: Right angles
Coordinate Units: U.S. Survey Feet
Operator: Kenny Mathews
Bent: 4
Location: RteU
Coordinate Proj. Factor: 1.000000
Date of Work: 01/06/16-01/06/16
Station: 70+40.1
Northing: 290656.935
Coordinate System: U.S. State Plane 1983
Depth to Water: 19
Offset: 10.0 R
Easting: 1109032.739
Coordinate Datum: NAD 83 (CONUS)
Time Change:
Elevation: 298.1
Drilling Method: Mud Rotary
Coordinate Zone: Missouri East
Depth Hole Open:
Drill No.: G-7887
Hammer Efficiency: 79%
Equipment: Failing 1500 ,Split-Spoon Sampler

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle ϕ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
0		0 - 5.9' Soft clay		yes	no	yes	120(1)	120(1)	600(1)			2-1-2 (4)	25(1)	0.01	50	140		
		5.9 - 19' Sand	290	no	no	yes	99(1)	99(1)				3-3-4 (9)	29(1)		25			
10							112(1)	112(1)				6-10-7 (22)	33(1)		158			
20		19 - 66.5' Sand	280	no	no	yes	129(1)	67(1)				11-10-10 (26)	34(1)		90			
			270				137(1)	75(1)				9-12-16 (37)	37(1)		125			
30							128(1)	66(1)				5-8-9 (22)	33(1)		90			

LPILE SUMMARY - MODOT_20150812.GDT - 3/1/16 10:17 - J:\SG\GINT\PROJECT FILES\J9S3034-A8414.GPJ

(1) = Assumed, (2) = Actual, (3) = Phi' 0 *(Continued Next Page)*

Missouri Department of Transportation
Construction and Materials

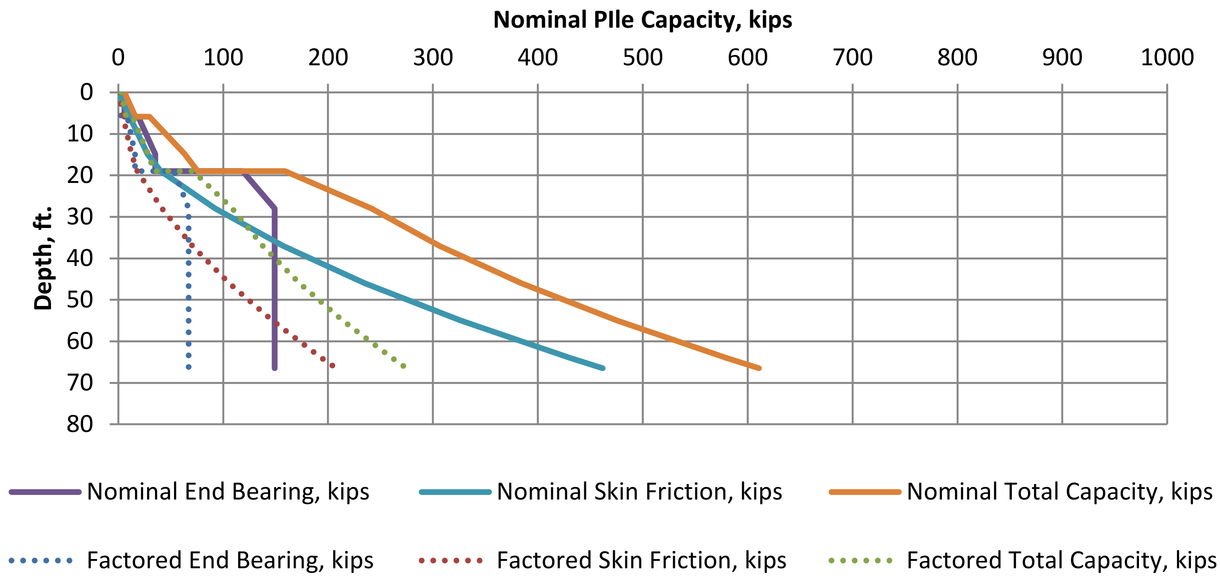
Job No.: J9S3034_A8414 **County:** New Madrid **Route:** U **Logged By:** George Davis
Design: A8414 **Skew:** Right angles **Coordinate Units:** U.S. Survey Feet **Operator:** Kenny Mathews
Bent: 4 **Location:** RteU **Coordinate Proj. Factor:** 1.000000 **Date of Work:** 01/06/16-01/06/16
Station: 70+40.1 **Northing:** 290656.935 **Coordinate System:** U.S. State Plane 1983 **Depth to Water:** 19
Offset: 10.0 R **Easting:** 1109032.739 **Coordinate Datum:** NAD 83 (CONUS) **Time Change:**
Elevation: 298.1 **Drilling Method:** Mud Rotary **Coordinate Zone:** Missouri East **Depth Hole Open:**
Drill No.: G-7887 **Hammer Efficiency:** 79% **Equipment:** Failing 1500 ,Split-Spoon Sampler

Depth (ft)	Graphic	Description	Elevation (ft)	Scour Parameters: Compressible	Downdrag	Scour	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Shear Strength Su or c (psf)	Pocket Pen (tsf)	Torvane (tsf)	Blow Counts (N_{60})	Internal Friction Angle ϕ'	Soil/Rock Strain (ε_{50}/k_{rm})	Lateral Subgrade Modulus K_f (pci)	Young's Modulus E_s (ksf)	Rock Quality Designation (RQD) (%)	Unconfined Compressive Strength Qu (ksf)
		19 - 66.5' Sand *(continued)*					127(1)	65(1)				3-6-8 (18)	33(1)		55			
			260															
40							132(1)	70(1)				10-13-12 (33)	36(1)		90			
							133(1)	71(1)				10-12-16 (37)	37(1)		125			
			250															
50				no	no	yes	135(1)	73(1)				14-19-21 (53)	40(1)		125			
							138(1)	76(1)				15-20-55 (99)	45(1)		150			
			240															
60							135(1)	73(1)				11-19-19 (50)	39(1)		125			
							136(1)	74(1)				12-20-26 (61)	39(1)		125			
		Bottom of borehole at 66.5 feet.																

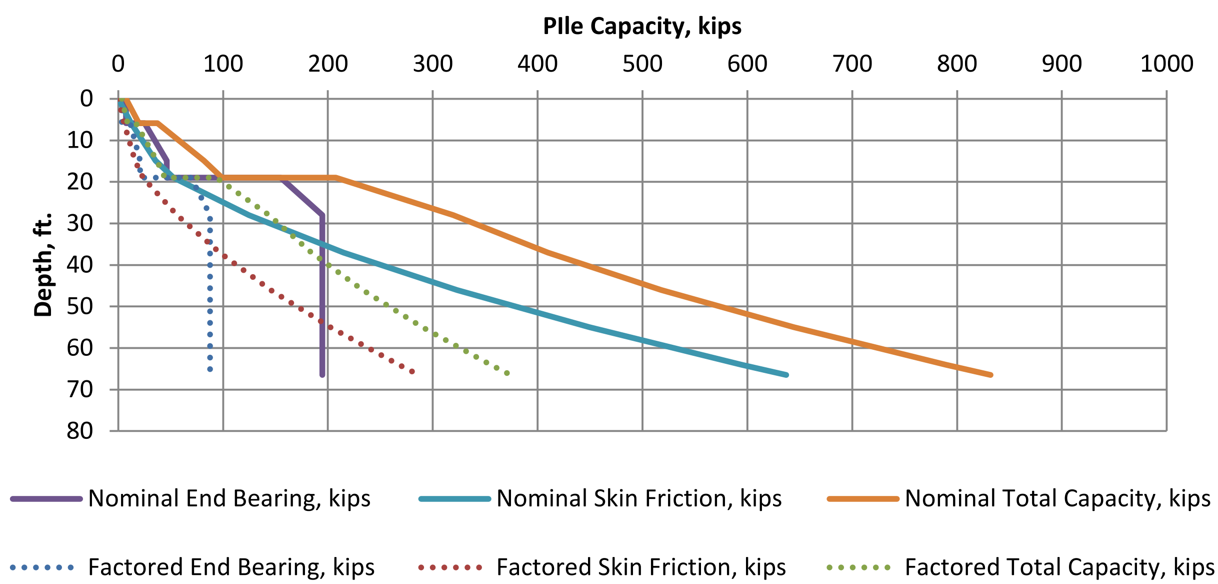
LPILE SUMMARY - MODOT_20150812.GDT - 3/1/16 10:17 - J:\SG\GINT\PROJECT FILES\J9S3034-A8414.GPJ

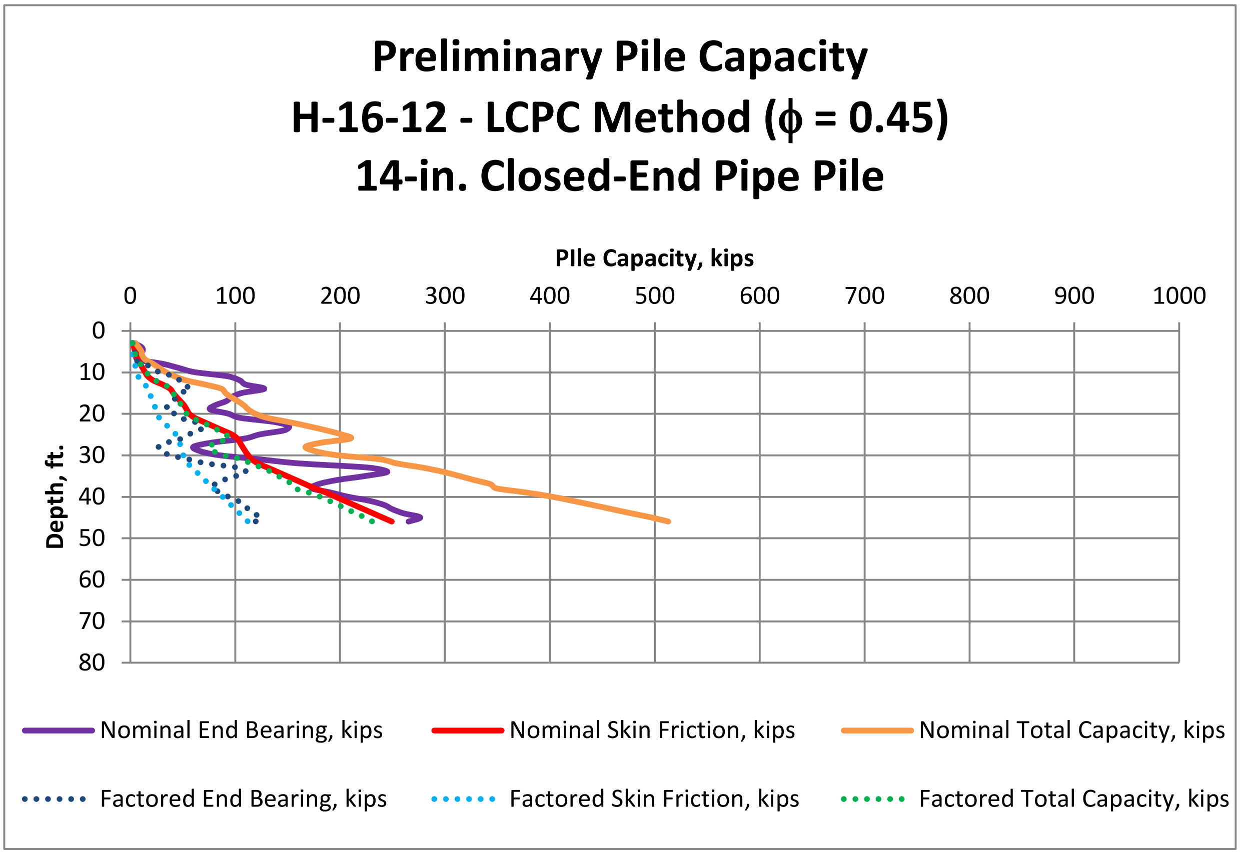
(1) = Assumed, (2) = Actual, (3) = Phi' 0

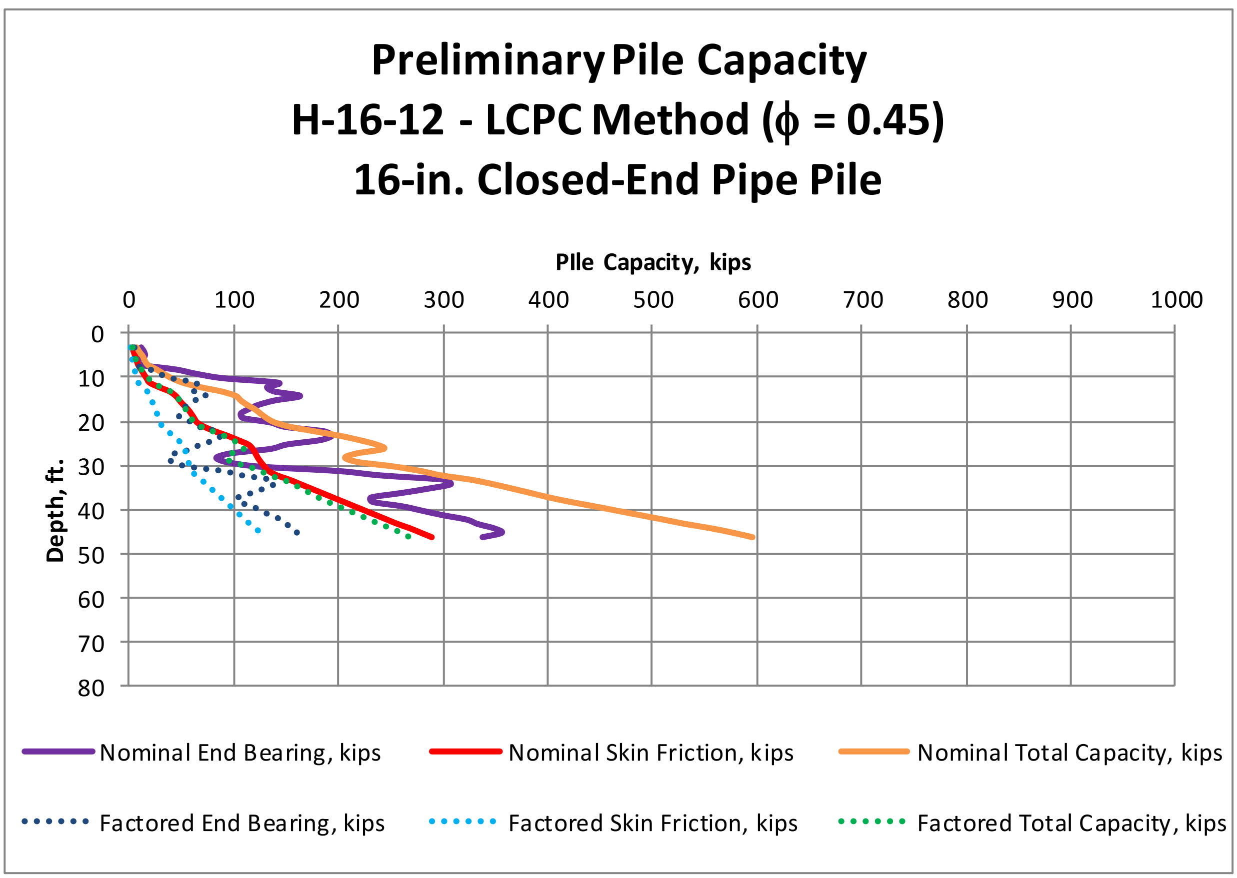
Preliminary Pile Capacity
A-16-03 - Nordlund Method (ϕ = 0.45)
14-in. Closed-End Pipe Pile



Preliminary Pile Capacity
A-16-03 - Nordlund Method (ϕ = 0.45)
16-in. Closed-End Pipe Pile




Preliminary Pile Capacity
H-16-12 - LCPC Method (ϕ = 0.45)
14-in. Closed-End Pipe Pile
PIle Capacity, kips
0
100
200
300
400
500
600
700
800
900
1000
Depth, ft.
0
10
20
30
40
50
60
70
80
Nominal End Bearing, kips
Nominal Skin Friction, kips
Nominal Total Capacity, kips
Factored End Bearing, kips
Factored Skin Friction, kips
Factored Total Capacity, kips


Preliminary Pile Capacity
H-16-12 - LCPC Method (ϕ = 0.45)
16-in. Closed-End Pipe Pile
PIle Capacity, kips
0
100
200
300
400
500
600
700
800
900
1000
Depth, ft.
0
10
20
30
40
50
60
70
80
Nominal End Bearing, kips
Nominal Skin Friction, kips
Nominal Total Capacity, kips
Factored End Bearing, kips
Factored Skin Friction, kips
Factored Total Capacity, kips

Appendix B – CPT Soundings and Downhole Boring Logs

Downhole Profiles

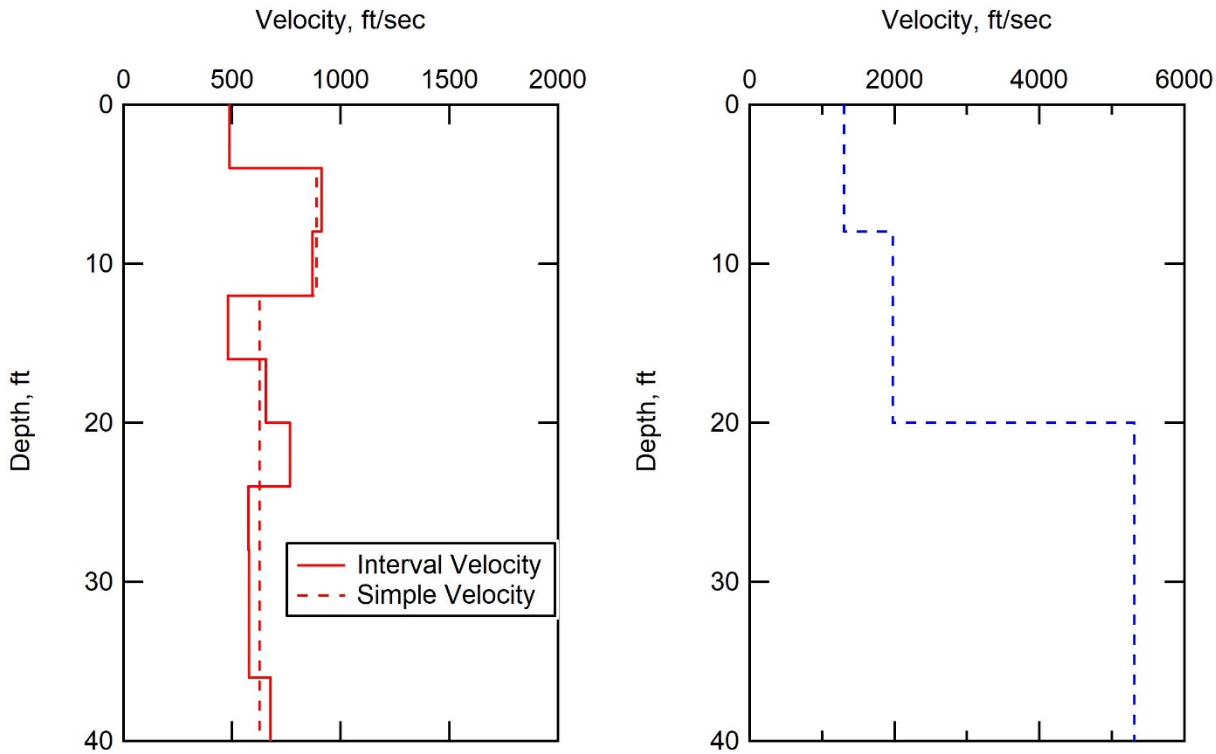


Figure B-1 Downhole profiles for shear wave (left) and compression waves (right) from Route U

Table B-1 Downhole Profile Values from Route U Site

S-wave Interval	
Depth (ft)	**Velocity (fps)**
0-4	489
4-8	912
8-12	869
12-16	481
16-20	657
20-24	767
24-28	575
28-32	577
32-36	578
36-40	676

S-wave Simple	
Depth (ft)	**Velocity (fps)**
0-4	489
4-12	889
12-40	626

P-wave	
Depth (ft)	**Velocity (fps)**
0-8	1305
8-20	1975
20-40	5307

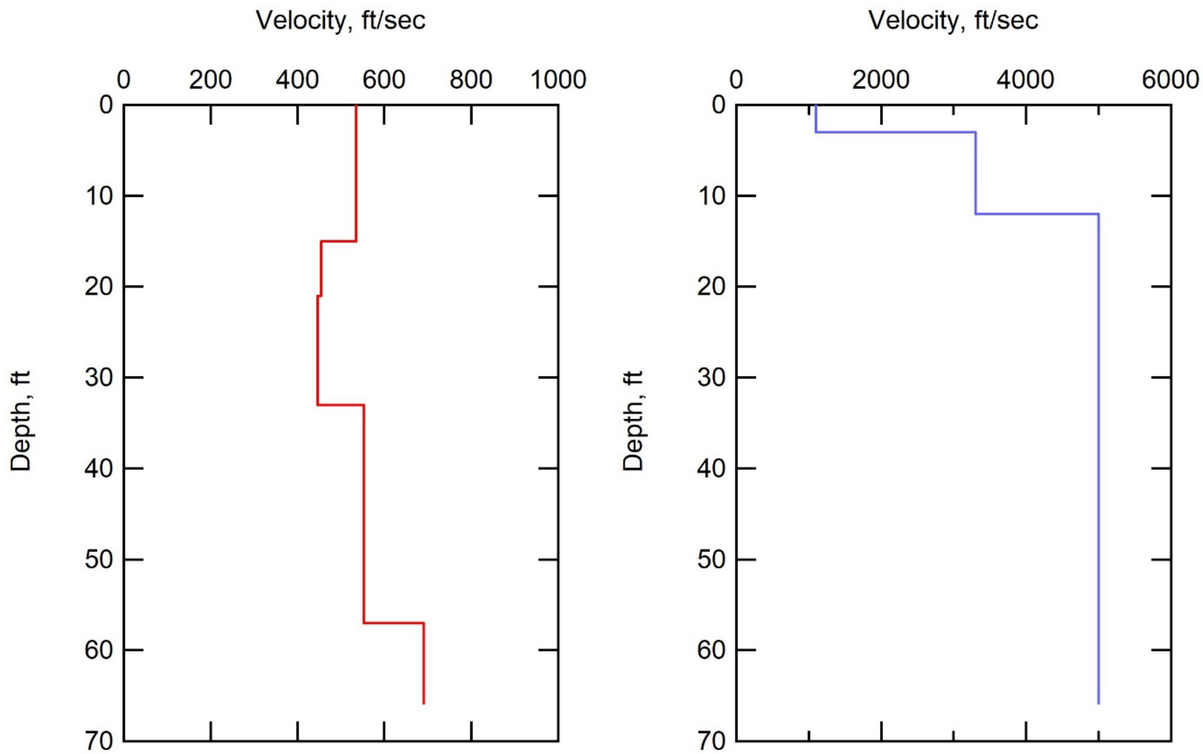


Figure B-2 Downhole profiles for shear wave (left) and compression waves (right) from Route WW

Table B-2 Downhole Profile Values from Route WW Site

S-wave	
Depth (ft)	**Velocity (fps)**
0-15	535
15-21	454
21-33	446
33-57	552
57-66	690

P-wave	
Depth (ft)	**Velocity (fps)**
0-3	1100
3-12	3300
12-66	~5000

Cone Penetration Soundings from:

H-16-71

H-16-72

H-16-73

H-16-74

H-16-75

H-16-76

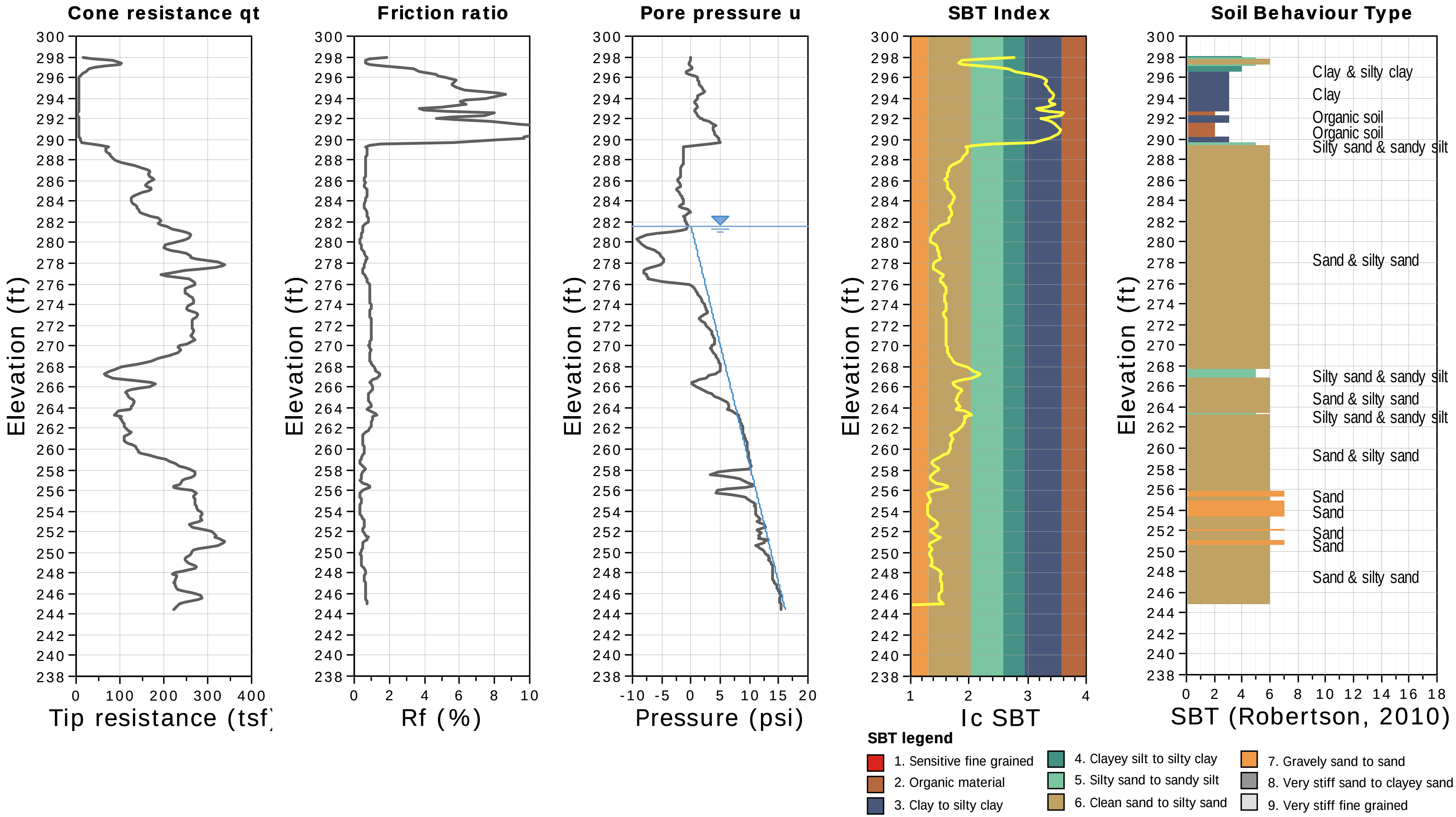
H-16-77

H-16-78

Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

CPT: N0771_H-16-71
Total depth: 53.64 ft, Date: 12/13/2016
Surface Elevation: 298.10 ft
Coords: X:0.00, Y:0.00
Cone Operator: Uknown

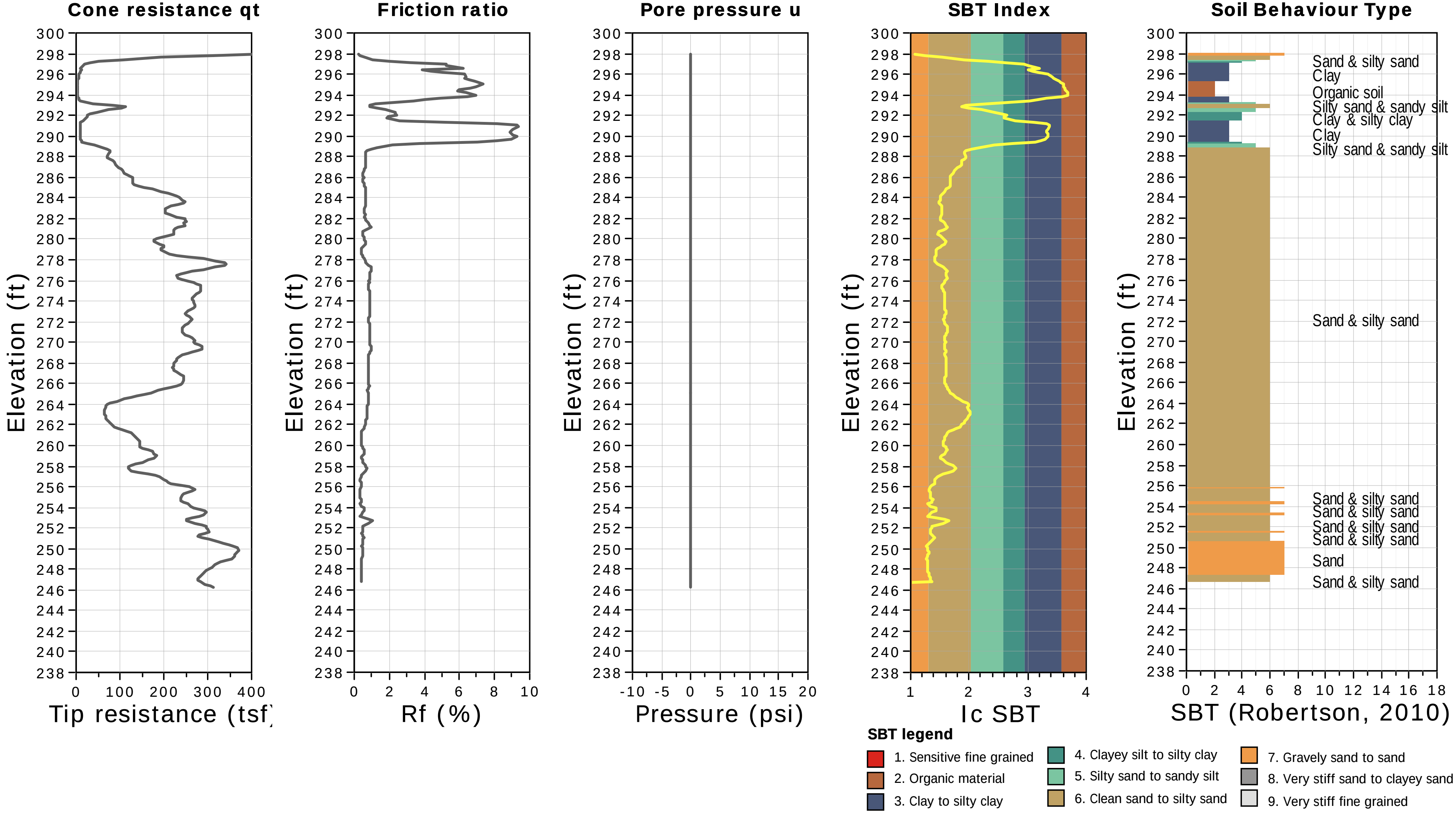
Project: **N0771 Foundation Reuse**
Location: **3 miles north of New Madrid, MO on Rte U**



Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

CPT: N0771_H-16-72
Total depth: 51.84 ft, Date: 12/13/2016
Surface Elevation: 298.10 ft
Coords: X:0.00, Y:0.00
Cone Operator: Uknown

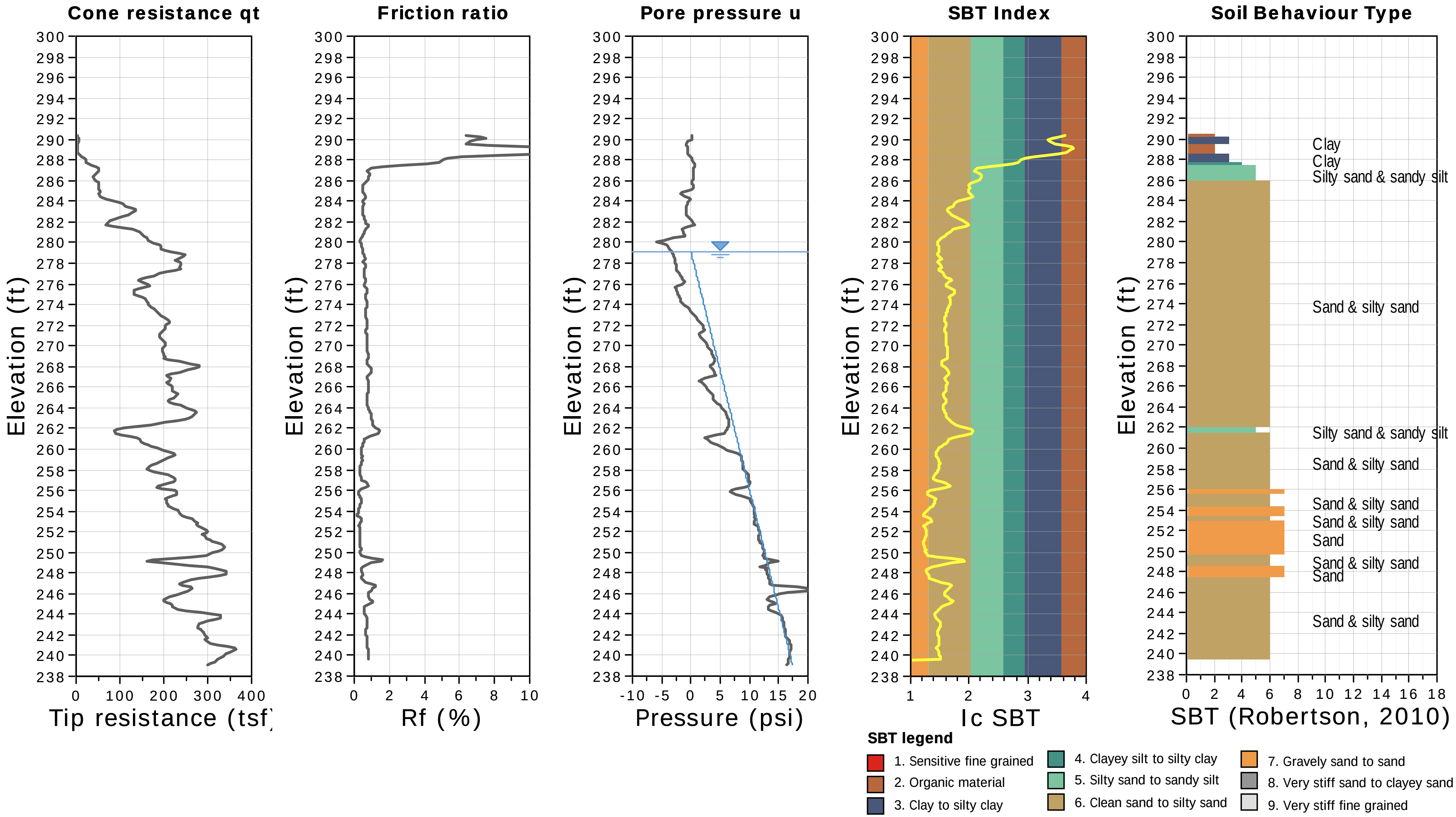
Project: N0771 Foundation Reuse
Location: 3 miles north of New Madrid, MO on Rte U



Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

Project: N0771 Foundation Reuse
Location: 3 miles north of New Madrid, MO on Rte U

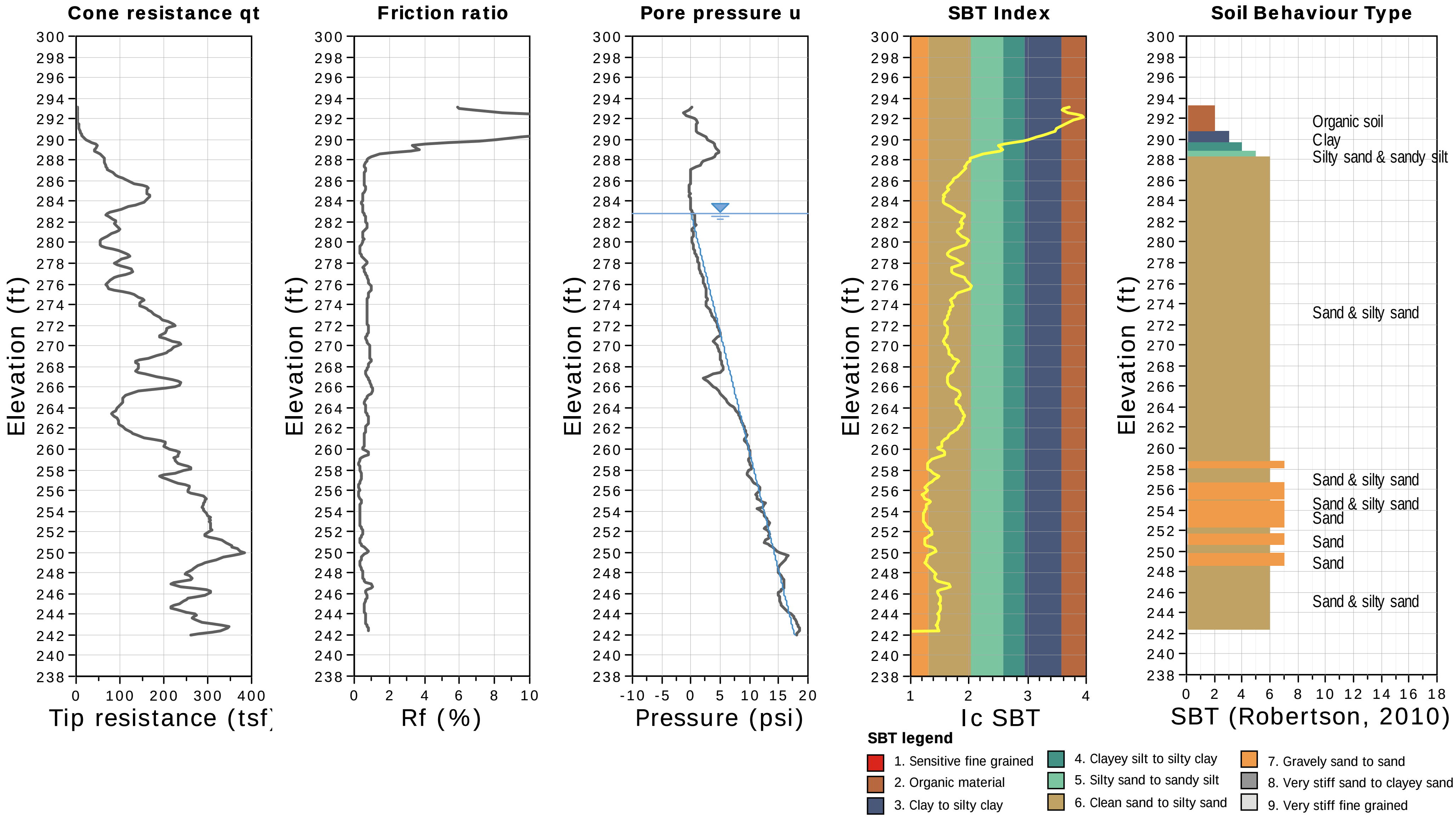
CPT: N0771_H-16-73
Total depth: 51.51 ft, Date: 12/14/2016
Surface Elevation: 290.60 ft
Coords: X:0.00, Y:0.00
Cone Operator: PEH



Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

CPT: N0771_H-16-74
Total depth: 51.35 ft, Date: 12/15/2016
Surface Elevation: 293.30 ft
Coords: X:0.00, Y:0.00
Cone Operator: PEH

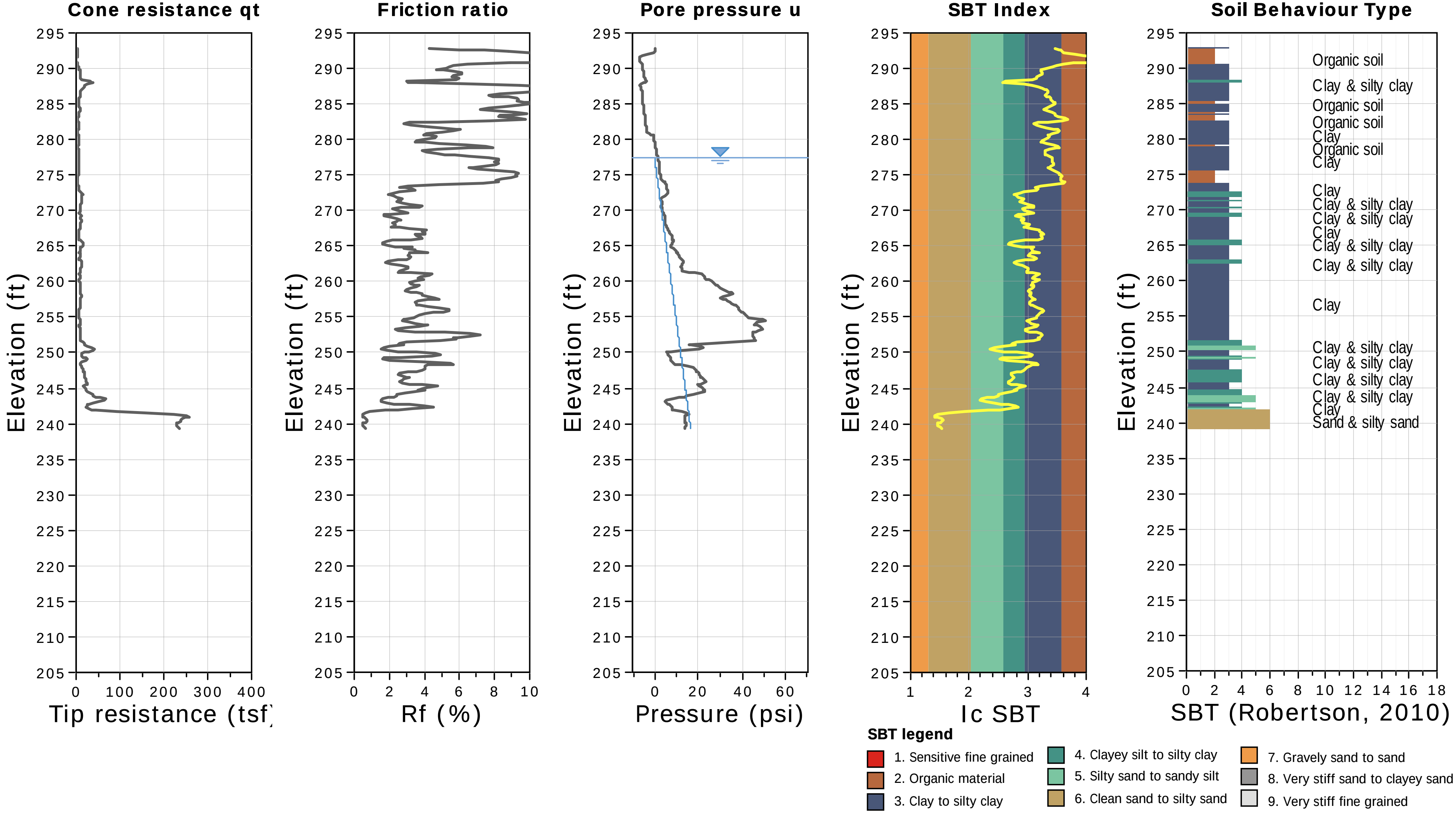
Project: **N0771 Foundation Reuse**
Location: **3 miles north of New Madrid, MO on Rte U**



Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

Project: **A2141 Foundation Reuse**
Location: **6 miles east of New Madrid, MO on Rte WW**

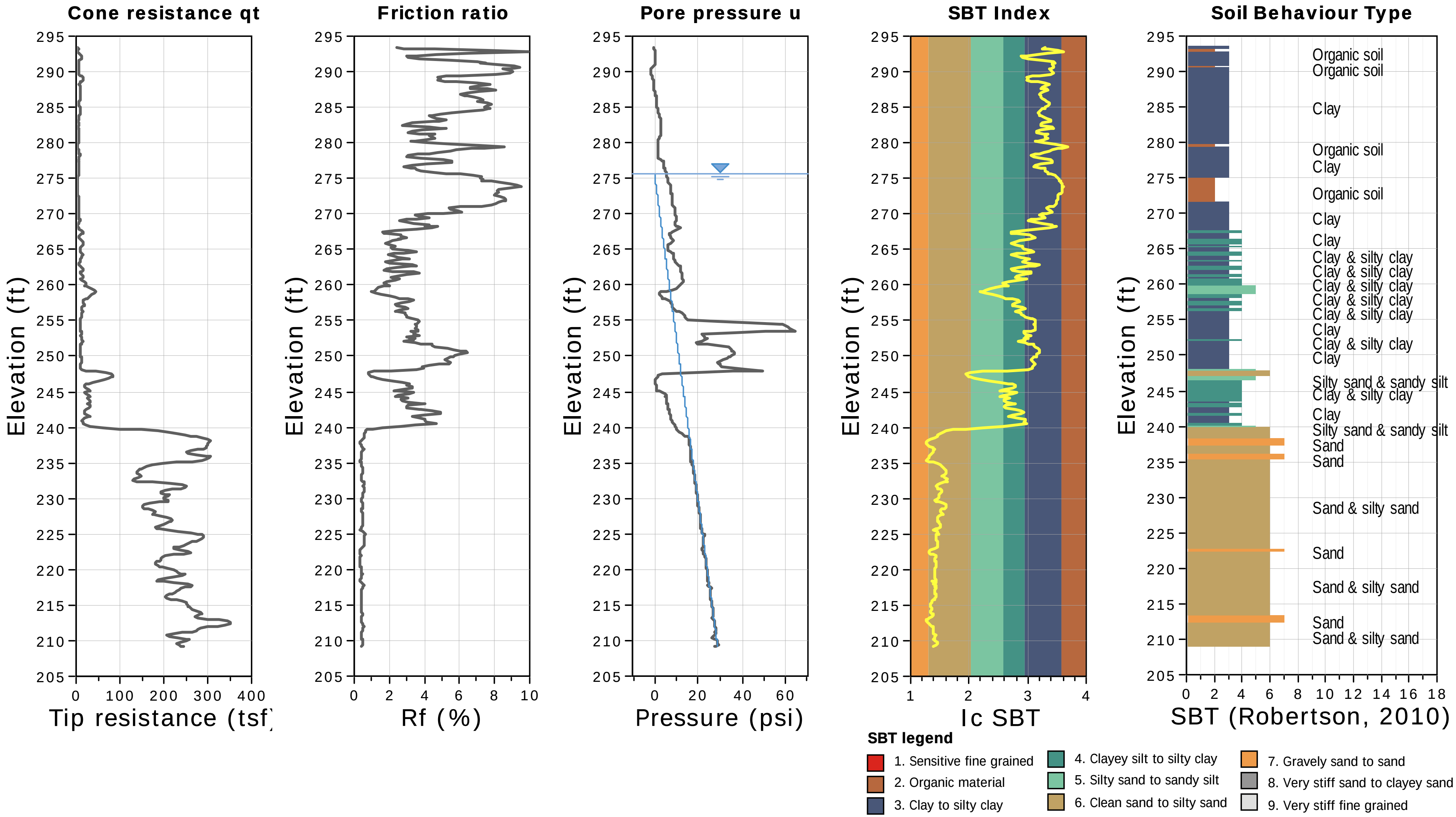
CPT: A2141_H-16-75
Total depth: 54.13 ft, Date: 12/22/2016
Surface Elevation: 293.50 ft
Coords: X:0.00, Y:0.00
Cone Operator: PEH



Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

CPT: A2141_H-16-76
Total depth: 84.48 ft, Date: 12/27/2016
Surface Elevation: 293.60 ft
Coords: X:0.00, Y:0.00
Cone Operator: PEH

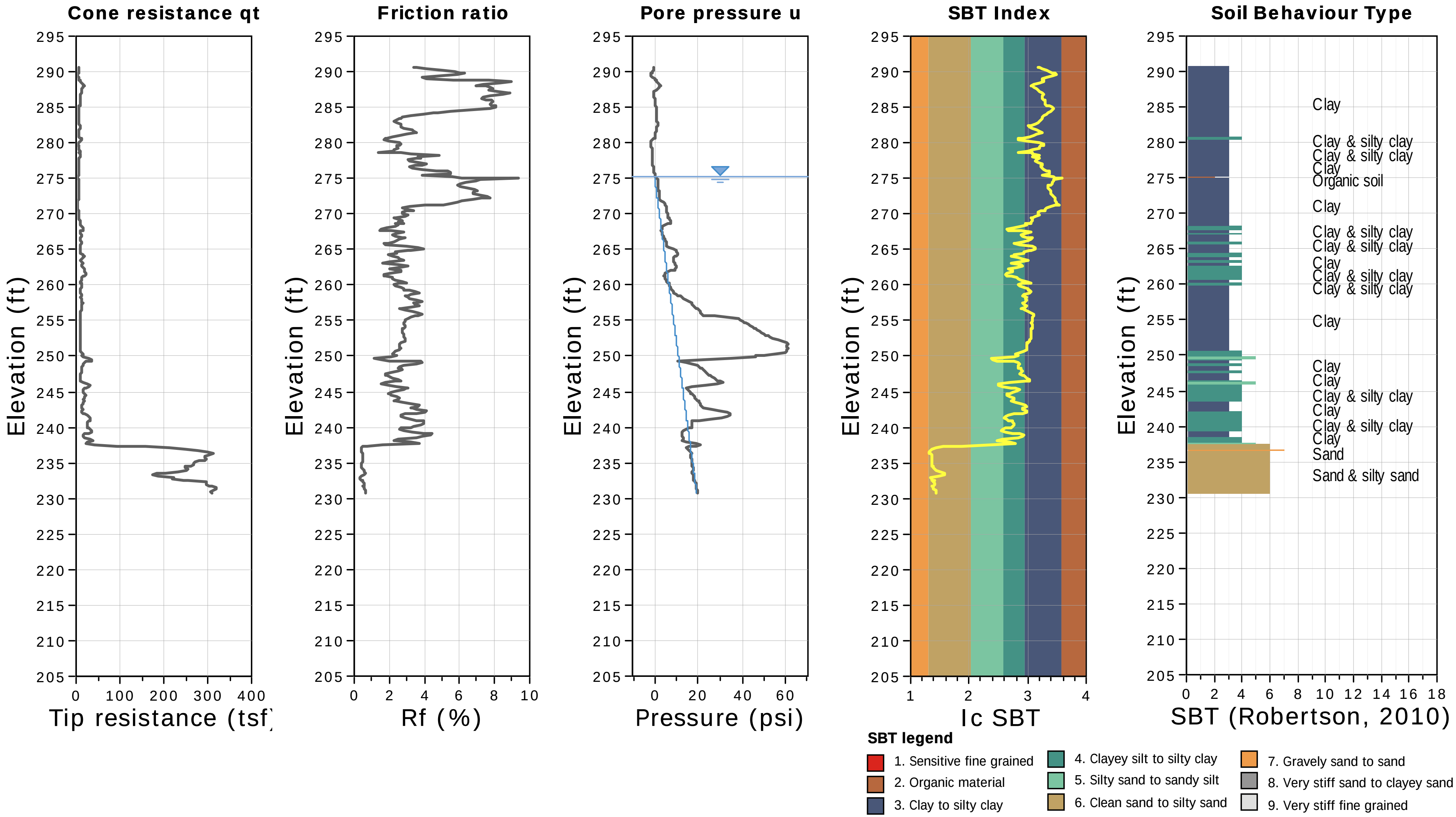
Project: **A2141 Foundation Reuse**
Location: **6 miles east of New Madrid, MO on Rte WW**



Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

Project: **A2141 Foundation Reuse**
Location: **6 miles east of New Madrid, MO on Rte WW**

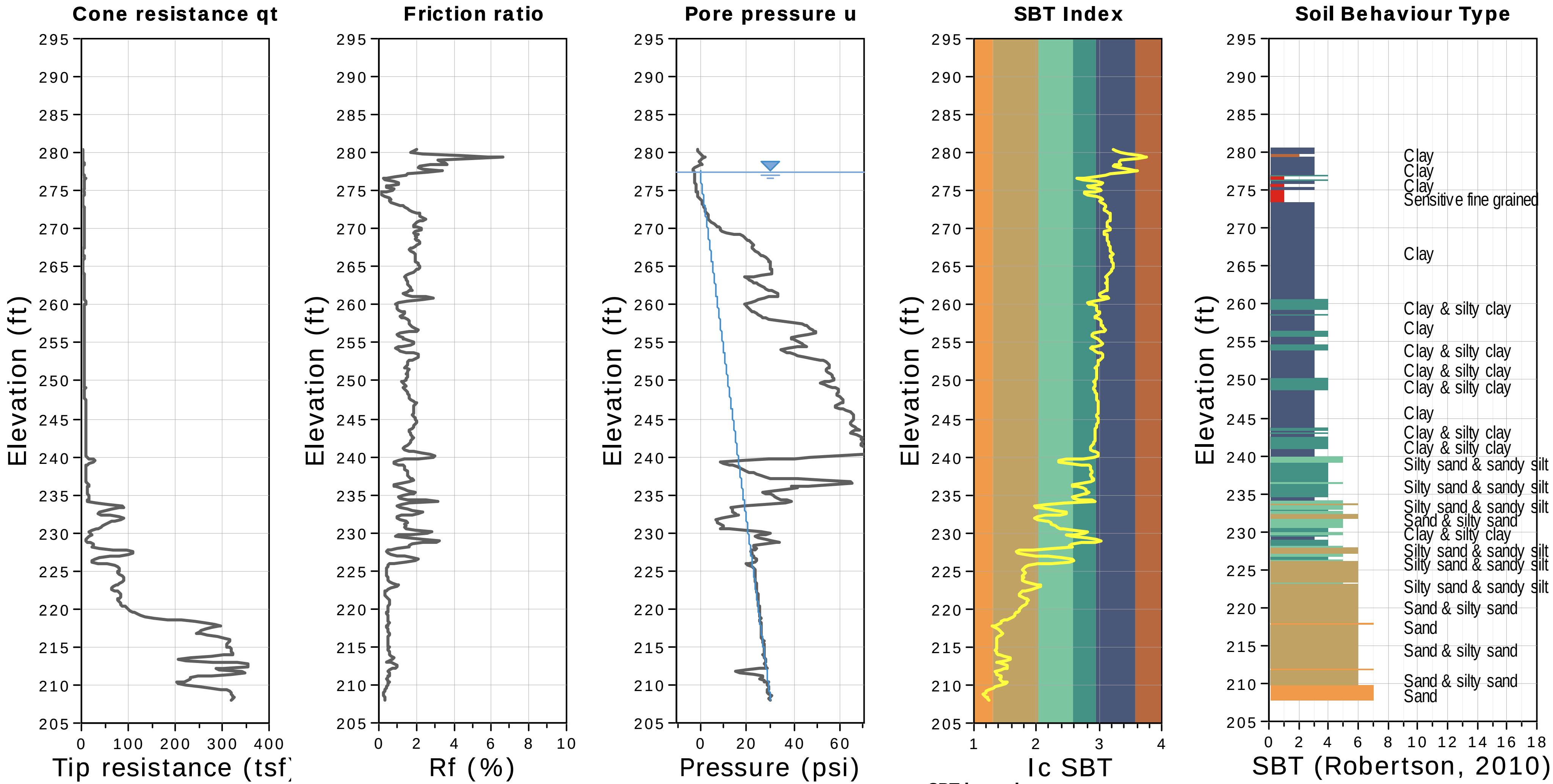
CPT: A2141_H-16-77
Total depth: 60.37 ft, Date: 12/28/2016
Surface Elevation: 291.20 ft
Coords: X:0.00, Y:0.00
Cone Operator: PEH



Geotechnical Section
Missouri DOT
1617 Missouri Blvd
Jefferson City, MO

CPT: A2141_H-16-78
Total depth: 72.51 ft, Date: 1/11/2017
Surface Elevation: 280.50 ft
Coords: X:0.00, Y:0.00
Cone Operator: PEH

Project: **A2141 Foundation Reuse**
Location: **6 miles east of New Madrid, MO on Rte WW**



SBT legend
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Appendix C – Pile Capacity Calculations

Discussion of contents is included in Section 3.5.

Objective Estimate pile capacity for existing piles at
CIP bridge (A2141)
Precast pile bridge (N0771)

Use static prediction methods. Capacity will be used
① To plan load test
② To compare with other prediction methods ← old design values (from plans), Restrike values, Unload test, Load test

CIP Bridge Route WW A2141

Overview Pile-supported bridge
4 bents - Bent 1 west to Bent 4 east End Bent: 1 - 2 - 3 - 4
4 piles/bent, "Design bearing" = 30T for all. Length, ft: 55 - 60 - 70 - 70
14"-OD closed end pipe piles driven to refusal in sand, then backfilled w/ concrete
50 to 70 feet of soft to stiff clay and silt over sand
Bents 1, 2, 3 — Bent 4 - original channel location before regrading for 1967 bridge construction
Outer piles at end bents are battered 1:4 (H:V) toward interior bents
(*) Piles at end bents were pre bored and holes backfilled w/ sand after driving per 1961 MoDOT specs.

Available Information Plans for 1967 bridge
"Finished" plans for 1967 - appear to be as builts, but "approximate" pile lengths from plans were not updated?
New CPT H-16-22 (Bent 1)
New Boring A-16-14 (Bent 4)
Old borings: 2 per end bent, 1 per interior bent
Lab data: old boring on West side included 6 unconfined compression tests on West end

Stratigraphy All boring and CPT information indicate soft to medium stiff clay and silt over sand, but depth of transition varies:

	Bent	Source	Sand EL	Ground EL
	West End ①	New CPT H-16-22	235	298.5 ← Bridge Surface EL all bents
347+89 14R		Old SPT South	243	
347+89 14L		North	237	
348+23 9L	West Interior ②	Old SPT	239	280 ±
348+57 9L	East Interior ③	Old SPT	240	280 ±
	East End ④	New SPT A-16-14	222	298.5
348+94 14R		Old SPT South	225	
348+94 12L		North	222	

Pile Lengths Assume piles were driven to refusal in sand. If "finished" plans lengths are correct:

	Bent	Top of Pile EL	Bottom EL	Corresponding Penetration into Sand	L ft
W	1	298.5	243.5	0	55
	2	298.5	238.5	0.5 ft	60
	3	298.5	228.5	11.5 ft	70
E	4	298.5	228.5	0	70

Note quantities called for 1020 ft of piling (= 4·(55+60+70+70))
Finished plans show 1034 ft of piles were installed → off by only 14' extra piling - lengths probably pretty close to plan values.

CIP Bridge cont'd

Material Properties

Information includes SPT data, CPT data, and some lab data.

SPT: 1967 data reported "shoved auger without turning" for much of clay.
for sand, highly variable {47, 64} Bent 1
And don't know hammer energy. {12, 13} Bent 3
{24, 56} Bent 4

only Bent 4 East

New boring (see stick log on 11x17): average N_{60} = { 8 above 284; 4 254–284; 8 234–254; 12 222–234 }

In sand, N_{60} = {32, 43, 43} in top 15' of sand

CPT: See Excel plots of CPT data. Bent 1 only. Follows similar trend to Bent 4 SPT data

West

EL	q_t tsf
above 282	8
270–282	5
246–270	9
235–246	23
225–235	340

Sheet 29

Lab: Old 1967 investigation included unconfined compression tests at West End Bent

Depth	EL	s_u from UC
5	283	1646 psf
11	278	680
16	273	465
21	268	680
26	263	580
31	259	920
36	253	750

Design Approximation

EL	s_u
Above 280	1500 psf
260–280	600
240–260	800

Pile Capacity Estimates

Original Plans

Design Bearing = 30 T = 60 kips

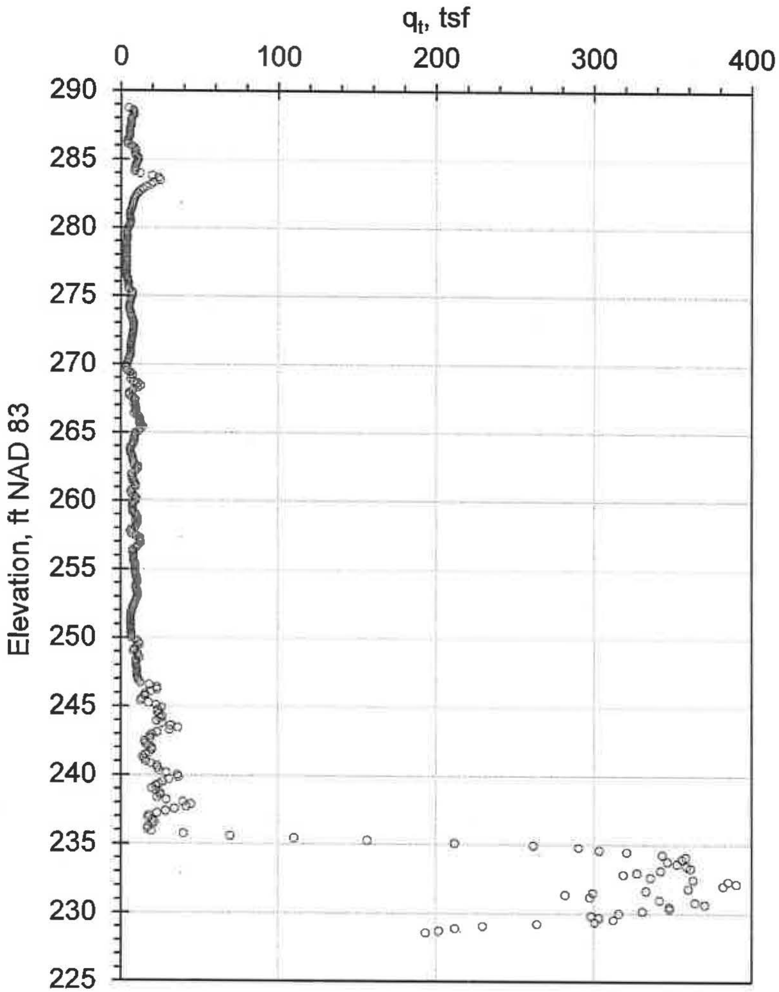
Presumably allowable, but FS is unknown.

CPT

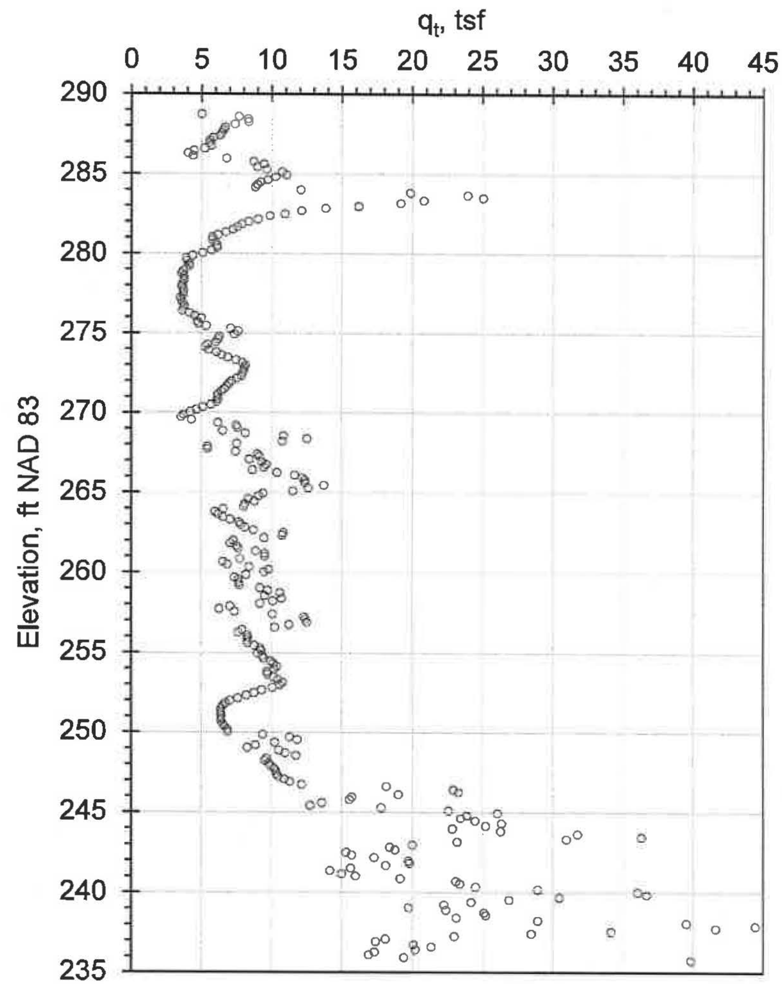
MoDOT predicted capacity based on West end Bent CPT sounding.
For LCPC method and 14-in. closed end pipe piles driven 55 ft
Nominal Skin Friction = 85 kips
Nominal End Resistance = 250 kips

∴ $\boxed{\text{Ultimate Capacity in Uplift = 85 kips; Ultimate Capacity in compression = 335 kips}}$

H-16-22: full-depth, full qt



H-16-22: qt < 45



CIP Pile Bridge

CIP Bridge contd | Pile Capacity Estimates cont'd | (SPT)

Meyerof (1976) -- reliability is questionable

$$q_t = 400\,\bar{N}_0' + \frac{(40\bar{N}_B' - 40\bar{N}_0')D_B}{b} \leq 400\,\bar{N}_B'$$

$\bar{N}_B'$ = avg corrected SPT N' value for bearing stratum, from tip to 3b below

⇒ Assume piles driven 3 ft into sand
only new SPT, corrected values are 32 bpf near top of sand and 43 bpf 5' lower — East Bent
say $\bar{N}_B' = 35$ bpf

$\bar{N}_0'$ = avg corrected SPT value for stratum overlying bearing stratum

⇒ Use $N_0' = 12$ bpf from avg in A-16-22 for 15 ft above sand

D_b = pile embedment depth in meters = Assume 1 m

b = pile diameter in meters = 0.36 m

$$q_t = 400(12) + \frac{[40(35) - 40(12)](1)}{0.36} \leq 400(35)$$

$$q_t = 7356 \leq 14000 \text{ kPa}$$

$$R_t = \left(7356\text{ kPa}\,\frac{20.89\text{ psf}}{1\text{ kPa}}\right)\left(14\text{ in}\cdot\frac{1\text{ ft}}{12\text{ in}}\right)^2\frac{\pi}{4} = 164{,}179\text{ lb}$$

$$R_t = 164\text{ kips}$$

$$f_s = \bar{N}' \leq 100\text{ kPa}$$

⇒ using formula for non-displacement piles since the piles were pre-bored

$$N' = [8(298.5-284) + 4(284-254) + 8(254-234) + 12(234-222)]/(298.5-222)$$

$N' = 7$ bpf above sand $N' = 32$ bpf in sand

$f_s = 7$ kPa $= 147$ psf above sand $f_s = 32$ kPa $= 668$ psf in sand

$$R_s = 147\text{ psf}\cdot\left(14\text{ in}\cdot\frac{1\text{ ft}}{12\text{ in}}\right)\pi\cdot(\cancel{298.5-222})\text{ ft}$$
limit to 67 ft per plans

$$+\,668\text{ psf}\cdot\left(14\text{ in}\cdot\frac{1\text{ ft}}{12\text{ in}}\right)\pi\cdot 3\text{ ft} = 43{,}421\text{ lb}$$

$$R_s = 43.4\text{ kips}$$

so

$$\boxed{\begin{array}{l}\text{Ultimate Capacity in Uplift} = 43\text{ kips}\\ \text{Ultimate Capacity in Compression} = 213\text{ kips}\end{array}}$$

CIP Bridge cont'd

Pile capacity Estimates cont'd

SPT

Nordlund End Resistance

$R_t = \alpha_t N'_q A_p \sigma'_p$

$D = 55$ ft $b = 14$ in $\Rightarrow \frac{D}{b} = 47$

Assume γ_{avg} for overburden is 115 pcf, and GWT at $z = 14'$ from new boring

$\sigma' = 115(55) - 62.4(55-14)$

$\sigma' = 3800$ psf $\pm$

$C_n = 0.77 \log \frac{40}{3.8} = 0.79$

If $N_{60} = 32$, $(N_1)_{60} = 25$ New Boring East Bent

If $N = 47$ $(N_1)_{60} = 37$ old Boring West Bent

For AASHTO $(N_1)_{60} - \phi$ correlation

$\phi = 36°$ for $(N_1)_{60} = 25$

$\phi = 39°$ for $(N_1)_{60} = 37$

For Kulhawy and Mayne (1990)

$\phi = 41°$ and $\phi = 45°$

Use $\phi = 36°$ and $\alpha_t = 0.70$ and $N'_q = 70$ and $q_L = 150$ ksf

R_t will depend on GWT at test date... assume same as above

$q_t = 0.7(70)(3.8\text{ ksf}) = 186$ ksf use q_L

$R_t = (150\text{ ksf})(\frac{14}{12}\text{ ft})^2 \frac{\pi}{4}$

$R_t = 160$ kips

Combine w/ α

uplift 174 kips

compression 174 + 160

334 kip

Brown (2001)

$f_s = F_{vs}^{?\,1.0}(A_b + B_b N_{60})$

F_{vs} = correction factor for piles installed w/ vibratory hammer = 1.0

A_b = 0.555 ksf compression
0.522 ksf tension

B_b = 0.040 ksf/bpf compression
0.0376 ksf/bpf tension

see p3 → N_{60} = 7 bpf El 298.5 to 222
32 bpf in sand -- 3' of pile assumed

f_s = 0.835 ksf El 298.5 to 222, compression
0.785 ksf " tension
1.835 ksf sand, compression
1.725 ksf " tension

$A_s = (14 \cdot \frac{1\text{ ft}}{12\text{ in}})\pi \cdot \ell = 3.66\ell$ ft²

$\ell = \begin{cases} 67 \text{ ft} \\ 3 \text{ ft} \end{cases}$ total 70' per plans

R_s = 0.835 ksf (3.66 (67) ft²) + 1.835 ksf (3.66 (3) ft²)
= 225 kips compression

R_s = 0.785 ksf (3.66 (67) ft²) + 1.725 ksf (3.66 (3) ft²)
= 211 kips tension

$q_t = 3.55 N_{60} = 3.55(32) = 113.6$ ksf

$R_t = 113.6\text{ ksf}\left(\frac{14\text{ in}}{12\text{ in/ft}}\right)^2 \frac{\pi}{4} = 121$ kips

Bent 4 East ∴

$\boxed{\text{Ultimate Capacity in uplift = 211 kips; Ultimate Capacity in Compression = 346 kips}}$

Lab

Alpha Method per Tomlinson's 1979 adhesion values

Bent 1 West

EL	d	Au	curve d/B	Ca
280-298.5	18.5	1500 psf	10	1.05 ksf
260-280	20	600 psf	40	0.6
240-260	20	800 psf	40	0.8

$D/b = \frac{40^+}{14/12} = 34^+$

$R_s = \frac{14\text{ in}}{12\text{ in/ft}} \pi (1.05 \cdot 18.5 + 0.6 \cdot 20 + 0.8 \cdot 20)$

$\boxed{R_s = 174 \text{ kips}}$

CIP Bridge cont'd

Pile capacity Estimates contd

MODOT – East Bent Boring

MoDOT used DRIVEN w/ α method for clay and Nordlund method in sand

Nominal End Bearing = 280 kips
Nominal Side Resistance = 200 kips

Ultimate Capacity in Uplift = 200 kips
Ultimate Capacity in Compression = 480 kips

Effect of Pre boring

Unknown. Not much literature available, if any!
Spec from 1961 requires prebore hole diameter equal to or larger than steel pipe diameter.
Would expect at least some reduction in side resistance, but effect may not be great.

Summary of Predictions

Method	Ult. Resistance (kips) Compression	Uplift	Notes
Original Plans	60		Likely allowable
LCPC (CPT)	335	85	Calc by MoDOT; only considers West End CPT
Meyerhoff (SPT)	213	43	Meyerhoff reliability questionable
Brown (SPT)	346	211	Not much better than Meyerhoff
Alpha + Nordlund (AZB)	334	174	Based on lab data in clay N is sand
MoDOT Static Also Alpha + Nordlund	480	200	Used alpha in clay and Nordlund for sand

Comments

1. CPT only new data at test bent.
2. MoDOT estimate was for design purposes, likely more conservatism than load test planning purposes.
3. Evaluating old SPT data is difficult. Not knowing hammer efficiency makes N_{60} unknown, adding uncertainty to already uncertain predictions of capacity and/or ϕ. Also GWT location.

PRECAST BRIDGE

Route U
N0771

OVERVIEW

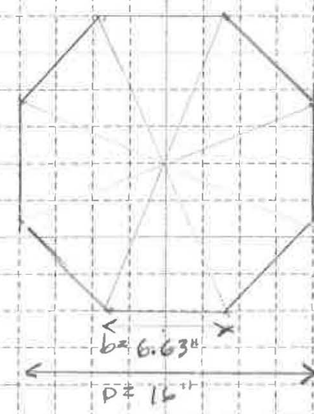
Pile supported bridge
4 bents - Bent 1 East to Bent 4 West
4 piles per bent w/ "plan capacity" of 21^T on end bents, 22.6^T interior
"computed capacity" of 14^T on end bents, 18.8^T interior
As built ("finished") plans note piles were driven to the minimum penetrations noted and to not less than the specified "Plan" capacities.
Piles are octogonal tapered piles driven to plan capacities above.
Pile tips are in sand.

from 8 triangles,

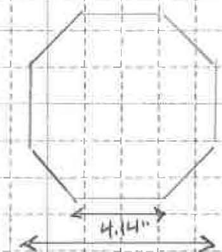
$A_{oct} = 8(\frac{1}{2}) b (\frac{D}{2})$
$= 2bD$

$A_{tip} = 2(4.14 in.)(10 in.)$
$= 83 in^2 = 0.58 ft^2$

→ Use 3-in. above tip as the area for bearing.



Top



3" Above Bottom

Side Area, A_s:

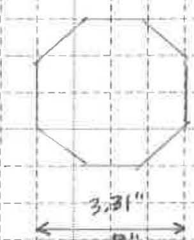
depth El, ft	b, in	A_s, in²/in.	ft²/ft
-11	6.63	53.0	4.4
0	5.71	45.7	3.8
5	5.29	42.3	3.5
11	4.79	38.3	3.2
18.75	4.14	33.1	2.8
19	3.31	26.5	2.2

reinforcement:
8 - #5s

→ Taper based on 30-ft piles per explanation on p7.

$\frac{6.63 - 4.14 \text{ in}}{29.75 \text{ ft}} = 0.0837 \text{ in/ft}$

→ Ab, not taper



Bottom

10 to 15 ft of sandy clay over sand.

Outer piles at interior bents are battered 1:6 (H:V) away from centerline of bridge.

Available Information

Plans dated 1962

"Finished" plans confirming pile driving to "plan" capacity values.
Does not list as-driven lengths, but quantities were updated from 405 ft (total) to 343 ft.

Finished = As-builts

New CPT H-16-12 at Bent 1
New Boring A-16-03 at Bent 4 -- SPT data
Old borings: A1 at Bent 1 and A2 at Bent 4
Only available info is stick logs on old plan sheets.

No lab data.

PRECAST BRIDGE cont'd

Stratigraphy: All explorations indicate fine grained material over sand, but depth of transition and description of fine grained material varies:

	Bent	Exploration	Fine Grained Description	EL Top Sand	EL Top Pile
East	1	CPT H-16-12 (New)	Clay to Silty clay	289	295.4
	1	Boring A1 1962	Sandy Clay	282	294.9
West	4	New Boring A-16-03	Fat Clay	292	294.9
	4	Boring A2	Sandy Clay	283	295.4

Note there is no information at the interior bents

Pile Lengths: "Finished" / as-built plans and final set of preconstruction plans note all piles are 30 ft long, but this might not account for cutoff?

"min. penetration" from plans

	Bent	No. Piles	EL Top Pile	EL Bottom Pile	L
East	1	4	295.4	275 -	20.4+
	2	4	294.9	270	24.9+
	3	4	294.9	270	24.9+
West.	4	4	295.4	275	20.4+

Total qty of piling to satisfy penetration req'd

Total length per table lengths $= 8(20.4+24.9) = 362.4$ ft

This is a lower bound - min req'd length to drive. Original plans estimated 405 ft installed and 45 ft cut off for 15 piles (test pile separate line).

$$\frac{405 \text{ ft} + 45 \text{ ft}}{15 \text{ piles}} = 30 \text{ ft/pile}$$

That is consistent w/ 30 ft long "approximate length."
Therefore, actual installed total pile length was 343 ft (replaced the 405 ft figure on original plans)

$$\text{avg driven pile length} = \frac{343 \text{ ft}}{15 \text{ piles}} = 22.8 \text{ ft}$$

The difference between installed lengths per the original plan (362.4 ft) and the total driven length (343 ft) corresponds to about 1 ft short per pile:

$$\frac{362.4-343}{15} = 1.3 \text{ ft}$$

So guess the end bent piles are $20.4 - 1.3 = 19$ ft long with corresponding El Top 295.4 to El Bottom 276.3 ft.

PRECAST BRIDGE contd

Design Profile

No lab data are available, so material properties are derived from SPT and CPT:

SPT Only data are from new SPT boring:

Mat'l	El Top	El Bottom	N_{60} Values
Soft Clay	-	292	4
Sand	292	?	9, 22, 26, 37, 22

See plot of N_{60} vs. depth. Average N_{60} along pile length:

Sheet 8a

$$N_{60,side} = \frac{(4\cdot3) + \frac{4+9}{2}\cdot5 + \frac{9+22}{2}\cdot5 + \frac{22+26}{2}\cdot5 + 26\cdot1}{19} = 14 \text{ bpf}$$

At pile tip and 3 diameters (~3 ft) below, use $\bar{N}_{60,tip} = 30$ bpf

CPT See plots of q_t vs. Elev. Above EL 290, say $q_t = 10$ tsf.
From EL 290 to pile tip (est. EL 276), average q_t is

$$\bar{q}_{t\,sand,side} = \frac{1}{290-276}\Big[\frac{10+75}{2}(290-287.5) + \frac{45+75}{2}(287.5-286.5) + \frac{175+45}{2}(286.5-284)$$
$$+ \frac{80+175}{2}(284-282.5) + \frac{115+80}{2}(282.5-279.5) + \frac{115+50}{2}(279.5-278)$$
$$+ \frac{150+50}{2}(278-276)\Big]$$

$$= \frac{1}{14}\left(42.5\cdot2.5 + 60\cdot1 + 110\cdot2.5 + 127.5\cdot1.5 + 97.5\cdot3 + 82.5\cdot1.5 + 100\cdot2\right)$$

$$\bar{q}_{t\,sand,side} = 89.2 \text{ tsf} \quad \text{say } 90 \text{ tsf}$$

For pile tip and 3 to 5 ft below, say $q_t = 150$ tsf

Considering stratigraphy info but emphasizing new boring (quality reasons), and considering SPT and CPT data, design profile is summarized in table below:

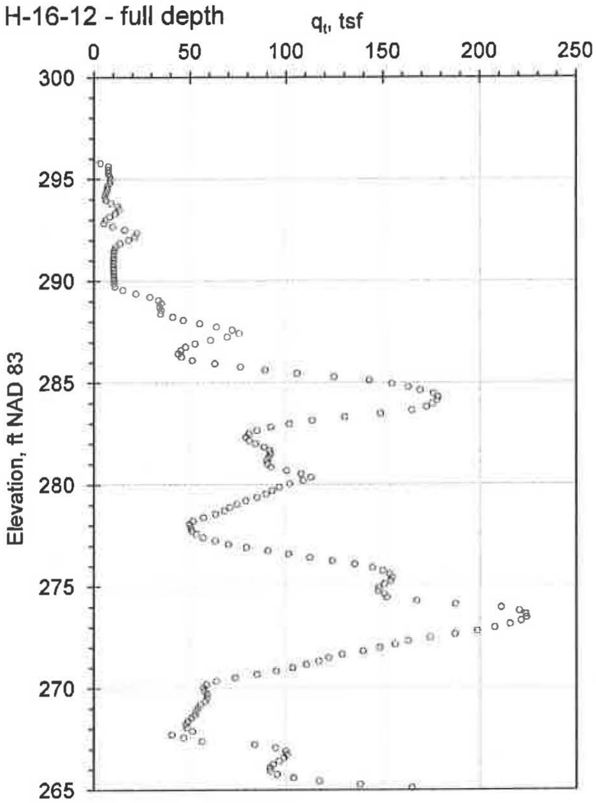
Elevation Top	Elevation Bottom	Material	N_{60} (bpf)	$N_{1,60}$ (bpf)	q_t (tsf)
295	290	clay	4	5.5	10
290	284	sand	9	10	90
284	276	sand	24	24	90
Pile Tip (276 ft)		Sand	30	28	150

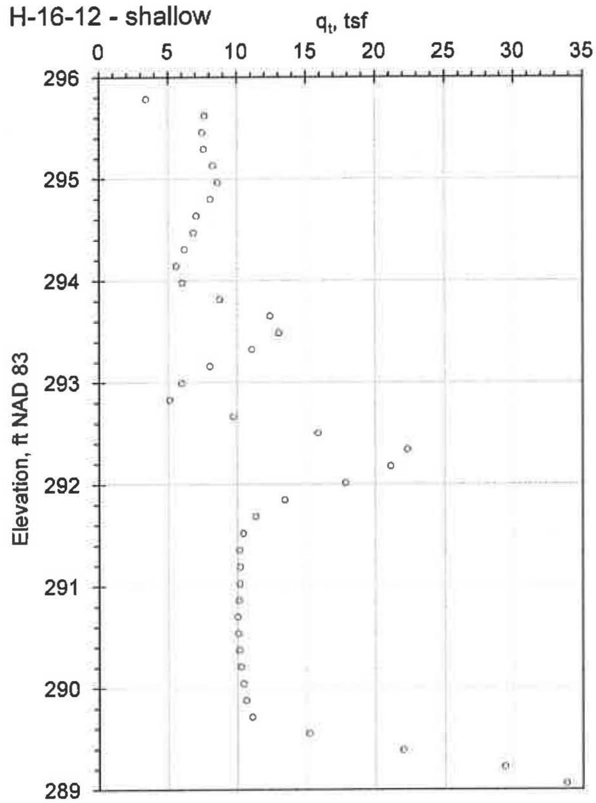
Guess undrained strength in the clay layer:

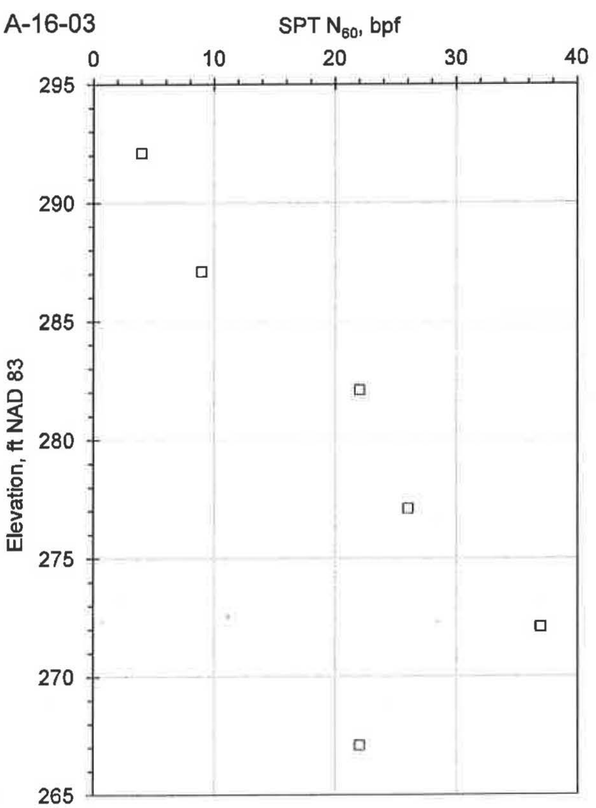
$$s_u = N_{60}\cdot150 = 600 \text{ psf}$$

$$s_u = \frac{q_t}{N_{kt}} = \frac{10 \text{ tsf}}{20} = 0.50 \text{ tsf} = 1000 \text{ psf}$$

say $s_u = 800$ psf







PRECAST PILE
BRIDGE

PRECAST BRIDGE cont'd

Design Profile cont'd

Estimate ϕ for sand by first correcting N_{60} values for overburden pressure. Guess unit weights as 110 pcf for clay and 120 pcf in sand. GWT at EL 278.6 at time of drilling. Assume unsaturated above GWT (no neg PWP). Calculations shown on SPT plot.

For AASHTO correlation ϕ for $N_{1,60} = 10$ is 30-35°
→ for EL 290 to 284
ϕ for $N_{1,60} \approx 24$ and 28 35-40°
→ for EL 284-276

From CPT, Robertson and Campanella

$\phi = \tan^{-1}\left(0.1 + 0.38 \log \frac{q_c}{\sigma_v'}\right)$

$\phi = \begin{cases} 43° \\ 40° \end{cases}$

For Kulhawy and Mayne,

$$\phi' = \tan^{-1}\left(\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma_v'}{P_a}\right)}\right)^{0.34}$$

for $N_{60} = 9$ and $\bar{\sigma} = 1260$, $\phi' = 35.5°$ EL 290 - 284

$N_{60} = \begin{cases} 22 \\ 26 \end{cases}$ $\bar{\sigma} = \begin{cases} 1260 \\ 2366 \end{cases}$ $\phi' = 42°$ 284-276

for design, say

$\phi = \begin{cases} 35° & 290\text{-}284 \\ 40° & 284 \downarrow \end{cases}$

Kulhawy and Chen, $\phi' = 27.5 + 9.2 \log N_{1,60} = \begin{cases} 36° & 290\text{-}284 \\ 41° & 284\text{-}276 \end{cases}$

Pile Capacity Estimates

Original Plans

Assume the "plan capacity" values listed on plans are design values. "computed capacity" could then be the computed loads? A bit of a misnomer...

Plan capacity = 21^T = 42 kip
Computed Capacity = 14^T = 28 kips

CPT MoDOT's report included CPT-based capacity predictions for sounding 11-16-12, with calculations by the CPT software.

For LCPC method and 14-in CIP pipe piles driven 20 ft

$A_t = 154 \text{ in}^2 = 1.07 \text{ ft}^2$
$A_s = 44 \text{ in}^2/\text{in} = 3.66 \text{ ft}^2/\text{ft}$

Nominal Skin Friction = 60 k
→ Avg side resistance, $f_s = \frac{60k}{3.66 \text{ ft}^2/\text{ft} \cdot 20 \text{ ft}} = 0.82$ ksf

Nominal End Resistance = 100k
→ Unit End Resistance, $q_t = \frac{100k}{1.07 \text{ ft}^2} = 93$ ksf

For the 16 in. octagonal piles
average $A_s \approx (44 + 28)/2 = 3.6 \text{ ft}^2/\text{ft}$ (linear taper)
$A_s = 19(3.6) \approx 68 \text{ ft}^2$
$R_s = (68 \text{ ft}^2)(0.82 \text{ ksf}) = 56$ kips

$A_t = 0.58 \text{ ft}^2$ (p6) so $R_t = 93 \text{ ksf}(0.58 \text{ ft}^2) = 54$ kips

PRECAST BRIDGE cont'd | Pile capacity Estimates cont'd | CPT cont'd

∴ Ultimate capacity in Uplift = 56 kips
Ultimate capacity in Compression = 110 kips

Other MoDOT

MoDOT used DRIVEN v. 1.2 with the SPT data and Nordlund's method.

For 14-in CIP pipe piles driven 19 ft: $A_t = 1.07\ ft^2$, $A_s = 3.66\ ft^2/ft$

Nominal skin friction = 45 kips

→ Avg side resistance, $f_s = \frac{45k}{3.66 \cdot 19} = 0.60$ ksf

Nominal End Resistance = 120 kips

→ Unit End Resistance ⇒ $q_t = \frac{120k}{1.07\ ft^2} = 112$ ksf

For the 16-in. octagonal piles

$R_s = (68\ ft^2)(0.60\ ksf) = 41$ kips

$R_t = (112\ ksf)(0.58\ ft^2) = 65$ kips

∴ Ultimate capacity in uplift = 41 kips
Ultimate capacity in compression = 106 kips

SPT

Meyerhoff (1976) — questionable reliability

for end bearing, don't have soft over stiff — so Meyerhoff equation unworkable - just predict uplift

$f_s = 2N'$ N' bpf f_s kPa   ∴ $f_s = 0.042\ N'$ f_s ksf

Layer Depths	z ft	avg a_s ft²/ft	A_s ft²	N_{60}	f_s ksf	$R_s = f_s A_s$ kips
0-5	5	3.65	18.3	4	0.168	3.1
5-11	6	3.35	20.1	9	0.378	7.6
11-19	8	3.0	24.0	24	1.01	24.2
					Σ	34.9 kips

Ultimate capacity in Uplift = 35 kips

Brown (2001)

$$f_s = F_{vs}^{1.0}(A_b + B_b N_{60})$$

ksf A_b = 0.555 compression
0.522 tension

ksf/bpf B_b = 0.040 compression
0.0376 tension

University of Missouri
Civil and Environmental Engineering

Project: MODOT Foundation Reuse

Date: 21 JUNE 2017 Sheet 11 of 12

By: ARB CHKD: Rev:

Task: ESTIMATE PILE CAPACITY

PRECAST BRIDGE cont'd

Pile Capacity Estimates cont'd

(SPT) cont'd

Brown (2001)

sec P.10

Layer depths	A_s ft²	N_{60}	Compression f_s ksf	Compression R_s k	Tension f_s ksf	Tension R_s k
0-5	18.3	4	0.715	13.1	0.672	12.3
5-11	20.1	9	0.915	18.4	0.860	17.3
11-19	24.0	24	1.515	36.4	1.424	34.2
			Σ	67.9 k	Σ	63.8 k

$q_t = 3.55 N_{60} = 3.55(30) = 106.5$ ksf

$R_t = 106.5$ ksf $(0.58 ft^2) = 61.8$ k

So $\boxed{\text{Ultimate capacity in uplift} = 64 \text{ kips} \quad \text{Ultimate capacity in Compression} = 130 \text{ kips}}$

alpha and Nordlund

For 5 ft of clay, use α mtd per Tomlinson

$D/b = 5/1.3 = 4$ -- use line for $D/b = 10$

and concrete piles For $S_u = 800$ psf, $C_a = 0.8$ ksf

$R_{s,clay} = (0.8 \text{ ksf})(18.3 ft^2) = 14.6$ kips

Nordlund - side

$$R_n = K_\delta C_F \sigma'_d \frac{\sin(\delta+\omega)}{\cos \omega} C_d A d$$

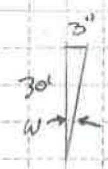
K_δ = coeff lat earth pressure at depth d

$= f(V, \omega, \phi)$

V = volume disp per ft = Avg x-sec area

$= 132 in^2 \cdot \frac{1}{144} \cdot \frac{\pi}{4} = 0.92 ft^2$

$$\phi = \begin{cases} 35° & \text{for 5 to 11 ft} \\ 40° & \text{for 11 to 19 ft} \end{cases}$$



ω = taper angle

$\tan \omega = \frac{3}{12 \cdot 30} = 0.0083$

$\omega = 0.48°$

$$\Rightarrow K_\delta = \begin{cases} 5.25 & \text{for } \phi = 35°, \text{ 5 to 11 ft} \\ 17 & \text{for } \phi = 40°, \text{ 11 to 19 ft} \end{cases}$$

δ = friction angle b/n pile and soil

for $V = 0.92 ft^3/ft$ and precast concrete piles,

$$\delta/\phi = 0.75 \Rightarrow \delta = \begin{cases} 26.25 & \text{5 to 11 ft} \\ 30 & \text{11 to 19 ft} \end{cases}$$

PRECAST BRIDGE

Pile Capacity Estimates contd

alpha and Nordlund contd

C_F = Correction factor for K_δ for $\delta \neq \phi$

$$C_F = \begin{cases} 0.88 & \text{for } \delta/\phi = 0.75, \phi = 35° \quad z = 5 \text{ to } 11 \text{ ft} \\ 0.83 & \text{for } \delta/\phi = 0.75, \phi = 40° \quad z = 11 \text{ to } 19 \text{ ft} \end{cases}$$

σ'_d = vert eff stress at ctr

See SPT plot

$$= \begin{cases} 1260 \text{ psf} & \text{for } z = 5 \text{ to } 11 \text{ ft} \\ 1860 + 2(120) = 2100 \text{ psf} & \text{for } z = 11 \text{ to } 19 \text{ ft} \end{cases}$$

Layer depths	K_δ	C_F	σ'_d ksf	δ	ω	C_d A_d ft²	R_n kips
5-11	5.25	0.88	1.26	26.25	0.48	20.1	526
11-19	17	0.83	2.10	30	0.48	24.0	361

Σ 413 kips

High .. check layer two

$$f_s = K_\delta C_F \sigma'_d \frac{\sin(\delta+\omega)}{\cos\omega} \qquad A_s = 8 \text{ ft}(3 \text{ ft}) = 24 \text{ ft}^2$$

$$= 17(0.83)(2.1 \text{ ksf}) \frac{\sin 30.48}{\cos 0.48}$$

$= 15.0$ ksf

$R_n = f_s A_s = 361$ kips ✓

Nordlund End Resistance

$$R_t = \alpha_t N'_q A_p \sigma'_p$$

$\sigma'_p = 2400$ psf at pile tip (see SPT plot)

$\phi = 40°$ per previous (p.9)

Say $\alpha_t = 0.7$ and $N'_q = 150$

$R_t = 0.7(150)(0.58 \text{ ft}^2)(2.4 \text{ ksf}) = 146$ kips

$q_t = 252$ ksf
$= 126$ ksf ✓

Totals

Ultimate Capacity in Uplift = ~~420~~ kips
Ultimate Capacity in Compression = 574 kips

too high - skin friction estimate includes effect of taper - neglect

Summary

Method	Ult Resistance (kips) Compression	Uplift	Notes
Original Plans	28 or 42	–	Presumably allowable; FS unknown
CPT/LCPC	110	56	By MoDOT Software
MoDOT	106	41	DRIVEN/Nordlund conservative parameters?
SPT - Meyerhoff		35	
Brown	130	64	
alpha and Nordlund	574		Only method to account for taper.

Appendix D – Reports from Dynamic Analysis of Restrike Tests

Discussion of contents is included in Sections 3.6 and 4.3.

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TRANSMITTAL

To: Dan Klaproth, P.E.	From: Travis Coleman, P.E.
Company: Koehler Engineering	No. of Sheets: 44
E-mail:dklaproth@koehlerengineering.com	Date: October 13, 2017

RE: CAPWAP Analyses
Route WW over Wilson Bayou, New Madrid County, MO

This transmittal summarizes our CAPWAP analyses of the dynamic load test data collected by Koehler Engineering. On September 14, 2017 GRL was contracted to perform the analyses. GRL waited to finalize these analyses pending information on the static load test results, which was requested by Koehler Engineering. On October 6, GRL was informed by University of Missouri and MODOT personnel that the intent of the test was to compare the analysis methods and that the static load test results would not be shared.

Testing objectives included mobilized pile capacity of cast in place piles from the existing bridge under re-construction. Koehler Engineering, using a Pile Driving Analyzer, acquired the dynamic data and provided testing details. Further evaluation of bearing capacity including an assessment of the soil resistance distribution was conducted by GRL Engineers, Inc. using the CAPWAP® Version 2014 program.

The tested piles were Bent 1 Pile 1, 2, 3, and 4, and Bent 2 Pile 5 and Pile 8. Bent 1 Pile 2 and Pile 3 were vertical piles, and the remaining tested piles were installed at a batter angle. It was reported to GRL Engineers, Inc. that the piles were 14 inch diameter with a wall thickness of 0.375 inches. GRL understands the piles were extracted following restrike testing; the tested Bent 1 piles had lengths from 50.9 to 55.9 feet and the tested Bent 2 piles had lengths of 62.4 and 64.5 feet. The reported restrike blow count for the piles ranged from 20 blows for one inch to 20 blows for ⅜ inches. The piles were restruck with a Delmag D-15 diesel hammer.

The CAPWAP analyses are summarize in the table on the following page. For each analysis, the resistance is separated into shaft resistance and end bearing components of the mobilized CAPWAP capacity. **Please note** – At blow counts greater than 10 blows per inch the full pile capacity, particularly at and near the pile toe, is not fully mobilized. To fully mobilize the capacity of these piles would have required a larger hammer.

October 13, 2017

Pile Number	Shaft Resistance (kips)	End Bearing (kips)	Total Capacity (kips)
Bent 1 Pile 1	250	18	268
Bent 1 Pile 2	221	80	301
Bent 1 Pile 3	223	78	301
Bent 1 Pile 4	228	40	268
Bent 2 Pile 5	221	22	243
Bent 2 Pile 8	195	34	229

GRL recommends a thorough review GRL's stated understanding of the reported pile details. Any discrepancies in the pile properties such as steel thickness, pile lengths, etc. have significant effects on the CAPWAP results. Please see the attached Appendix A for further discussion of dynamic testing and CAPWAP analysis. Please contact us if you have any questions regarding these results.

GRL Engineers, Inc.

Travis Coleman, P.E

Harry Weintraub

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Attachments: Appendix A (pages 3 – 14)
CAPWAP Analysis Results (pages 15 - 43)
Coleman PDCA Certificate (page 44)

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

1. BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during both design phase test programs as well as during production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore form an important part of a quality assurance program when deep foundations are constructed. Several dynamic pile testing methods exist. These methods have different benefits and limitations as well as different requirements for proper implementation.

The Case Method of dynamic pile testing, named after Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (e.g. a pile driving hammer or large drop weight) impacts the pile or shaft top such that a small permanent set is achieved. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® System (PDA).

The Case Method provides a simple closed-form solution for bearing capacity assessment. However, a more rigorous signal matching analysis method, CAPWAP® offers a more rigorous analysis of the dynamic test records than the Case Method solution and is therefore state-of-practice for final evaluation of the data to assess bearing capacity. A somewhat less rigorous signal matching analysis, called iCAP®, can be performed in real time on a construction site. However, iCAP results have not been as thoroughly correlated with static load test results as has been done with CAPWAP results. Therefore, iCAP results still require review by experienced testing and analysis engineers.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the "High Strain Test Method" of dynamic pile monitoring and dynamic load testing as standardized in ASTM D4945. Reference will also be made to the Rapid Load Test (or Force Pulse Test) as described in ASTM D7383. For completeness, three methods for deep foundation integrity assessments; the Pile Integrity Test™ (PIT), Cross Hole Sonic Logging with the Cross Hole Analyzer (CHA), and Thermal Integrity Profiling (TIP) are also discussed in Section 3.

2. RESULTS FROM PDA DYNAMIC TESTING

The primary objectives of high strain dynamic pile testing are either:

- *Dynamic Pile Monitoring, or*
- *Dynamic Load Testing*

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both drilled shafts and impact driven piles during restrike. With sufficient ram weight and impact cushioning, the duration of the dynamic load test force pulse can be lengthened such that a dynamic load test can satisfy Rapid Load Test requirements.

2.1 DYNAMIC PILE MONITORING

During pile installation, the sensors attached to the pile measure force and velocity near the pile top. A PDA provides signal conditioning, processes these signals, and calculates or evaluates by the Case Method:

- ***Bearing capacity*** at the time of testing, including an assessment of resistance distribution which is usually then related to blow count. This information supports formulation of a driving criterion.

- ***Dynamic pile stresses*** in both tension and compression, axial and averaged over the pile cross section, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- ***Pile integrity*** assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence for subsequently driven piles.

- ***Hammer performance*** parameters including the energy transferred to the pile, the hammer operating rate in blows per minute and the stroke of open ended diesel hammers

2.2 DYNAMIC PILE LOAD TESTING

Bearing capacity testing of either driven piles or drilled shafts (or bored piles and augercast piles) employs the basic measurement approach of dynamic pile monitoring. However, the test is often done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then, for sufficient soil resistance activation, its weight should be at least 1% of the test load for rock socketed piles and at least 2% for piles founded in gravelly materials. As an example, the ram weight should be at least 5 tons in favorable conditions and 10 tons in more energy absorbing soil conditions for a 500 ton test load. Ram weights larger than the minimum are acceptable. To satisfy rapid load test requirements, a ram weight of at least 5% of the test load is needed (e.g. minimum 25 ton ram for 500 ton test load).

For a successful test, it is most important that the test be conducted after a sufficient waiting time following pile installation so that soil strength properties approach their long term condition or in the case of cast-in-place concrete foundations that the concrete achieve sufficient strength and maturity. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain stresses within specified limits and for sufficient resistance activation. For dynamic load testing of drilled shafts, transferred energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles in sensitive soils require a warm pile hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with stress control and sufficient energy for resistance mobilization, the CAPWAP analysis provides the following results:

- *Bearing capacity i.e.* the mobilized capacity present at the time of testing

- *Resistance distribution* including shaft resistance and end bearing components

- *Stresses in pile or shaft* calculated at each point along the shaft for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or non-uniform contact stresses, e.g. when the pile toe is on uneven rock.

- *Shaft impedance vs. depth*; this is an estimate of the shaft shape if it differs substantially from the planned profile

- *Dynamic soil parameters* for shaft and toe, i.e. damping factors and quakes (quakes are related to the dynamic stiffness of the resistance at the pile/soil interface.)

3. FIELD MEASUREMENTS

The following is a general summary of dynamic measurements available to solve typical deep foundation problems.

3.1 PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed-form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below. Additional test details and procedures are described in ASTM D4945.

3.2 HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance

Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC.

3.3 SAXIMETER™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

3.4 PIT

The Pile Integrity Tester™ (PIT) helps in detecting major defects in concrete piles or shafts or in assessing the length of a variety of deep foundations, except steel piles. PIT performs the "Pulse-Echo Method" which only requires the measurement of motion (e.g., acceleration) at the pile top caused by a light hammer impact. PIT also supports the "Transient Response Method" which requires the additional measurement of the hammer force and an analysis in the frequency domain. PIT may also be used to evaluate the unknown length of deep foundations under existing structures. Additional test details and procedures are described in ASTM D5882.

3.5 CHA

This test requires that at least two tubes (typically steel tubes of at least 1.5 inch or 38 mm inside diameter) are installed vertically around the reinforcing cage in the shaft to be tested. A high frequency signal is generated in one of the water filled tubes and received in the other tube. The received signal strength and its First Arrival Time (FAT) yield important information about the concrete quality between the two tubes. The transmitting and recording of the signal is repeated typically every 2 inches or 50 mm starting at the shaft bottom and all records together establish a log or profile of the concrete quality between the two tubes and inside the reinforcing cage. The total number of tubes installed depends on the diameter of the drilled shaft. Generally one tube is installed for each foot (0.3 m) of shaft diameter. More tubes create more profiles for anomaly evaluation and delineation, if needed. Additional test details and procedures are described in ASTM D6760.

3.6 TIP

Thermal Integrity Profiling (TIP) can be used to assess the integrity, concrete cover, and concrete quality of concrete filled deep foundation elements by measuring the concrete temperature resulting from the heat of hydration. The test can be performed using Thermal Wire® cables embedded in the concrete or using Thermal Probes in access tubes similar to CHA. Analyzing the temperature vs. depth information leads to a 3-D pile volume image, including outside the reinforcing cage. Under favorable conditions, the volume vs depth information thus generated can be helpful when analyzing with CAPWAP the high strain records taken on cast-in-situ piles. Additional test details and procedures are described in ASTM D7949.

3.7 PIR-A

The Pile Installation Recorder for augered-cast-in-place (ACIP) or Continuous Flight Auger (CFA) piles, as a minimum, measures the amount of concrete or grout installed in the soil as a function of depth. As for the TIP results, under favorable conditions, the volume vs depth information thus generated can be helpful when analyzing with CAPWAP the high strain records taken on cast-in-situ piles.

4. ANALYTICAL SOLUTIONS

4.1 *BEARING CAPACITY*

4.1.1 WAVE EQUATION

The GRLWEAP program calculates a relationship between bearing capacity, pile stress, hammer stroke, and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph estimates a corresponding bearing capacity. This approach requires no field measurements other than blow count. However, it does require an accurate knowledge of the various parameters describing hammer, driving system, pile and soil. The wave equation is also very useful during the design stage of a project for the selection of hammer, cushion and pile size. Another option is the driveability analysis which predicts the blow count versus depth for a given hammer, pile and soil profile.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (Figure 1) is often performed by inputting the PDA and CAPWAP calculated parameters. With many of the dynamic parameters verified by the dynamic tests, the RWEA offers a more reliable basis for a safe and sufficient driving criterion.

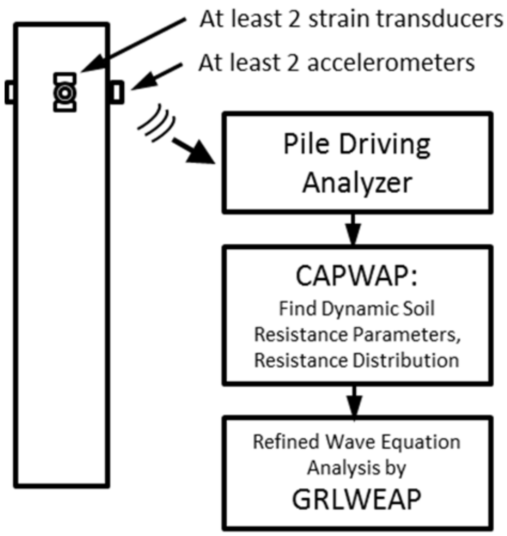


Figure 1. Block Diagram of Refined Wave Equation Analysis

4.1.2 CASE METHOD

The Case Method is a closed-form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force, F(t), and pile top velocity, v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \quad (1)$$

where

t = a point in time after impact
t_2 = time t + 2L/c
L = pile length below gages
$c = (E/\rho)^{1/2}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile (ρc^2)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \quad (2)$$

The dynamic component may be computed from a soil damping factor, J, and the calculated pile toe velocity, $v_{toe}(t)$. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \quad (3)$$

and, finally, to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 could be evaluated. Most commonly, t is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. Higher values are possible and lead to more conservative results. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used except when a correction is added as a result of “early unloading”) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity method, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed for one record. The capacities for other hammer blows are then quickly calculated for the thus selected Case Method and its associated damping factor.

The static resistance calculated by either Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set (permanent net displacement) has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The estimated static end bearing, EBR, is then calculated from the estimated static capacity and the shaft resistance estimate SFR.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDIPLOT program.

4.1.3 iCAP

iCAP is a signal matching program that works in parallel with the PDA software. iCAP allows signal matching based capacity assessments during data collection and/or data review for driven piles of known uniform geometry. iCAP performs a completely automatic signal match procedure, similar to the one available in the CAPWAP® program, but using faster algorithms. Depending on the blow rate of the hammer, and the level of iCAP computation, iCAP results will be a few blows behind the current PDA installation data. The following numeric results are available for each iCAP analyzed blow:

- RUC – total capacity by iCAP matching
- SFC – shaft resistance computed by iCAP
- EBC – end bearing computed by iCAP
- CSC – maximum compression stress
- BSC – max bottom compression stress
- TSC – maximum tension stress
- JC - correlating Case damping factor
- MQ - iCAP match quality

Since iCAP is fully automated, non-uniform piles, piles with (even minor) damage, concrete piles with minor cracking, or piles with uncertain properties cannot accurately be analyzed by iCAP. Larger open-end pipes (due to internal plug movements) or piles in unusual soils may pose extra difficulties. Also, the program only performs a limited data quality check. In addition, and as mentioned earlier, the iCAP signal matching procedure is not as thorough as what is done by CAPWAP and differences in results from these two types of signal matching analyses must be expected. Only CAPWAP has been extensively correlated with static load test results. A responsible engineer will therefore check the iCAP results thoroughly and compare them with CAPWAP, at least on a spot check basis, to determine reliable test results.

4.1.4 CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffness "quake" values. The method iteratively calculates a number of unknowns by signal matching.

While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program uses actual the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters based on the dynamic measurements. As a by-product, CAPWAP calculates tension and compression stresses along the length and provides a simulated static load test graph.

4.1.5 Capacity of damaged piles

Occasionally piles are damaged during driving and such damage may be indicated in the PDA collected records if it occurs below the sensor location. Damage on steel piles is often a broken splice, a collapsed pile bottom, a ripped of flange on an H-pile or a sharp bend (a very gradual dog leg is usually not recognized in the records). For concrete piles, among the problems encountered are cracks perpendicular to the pile axis, which deteriorate into a major damage, slabbing (loss of concrete cover) or a compressive failure at the bottom which in effect makes the pile shorter.

Damaged piles, with **BTA** values less than 0.8 should never be evaluated for bearing capacity by the Case Method or iCAP alone> Damaged piles are non-uniform piles which therefore violate the basic premise of the Case Method: a uniform, elastic pile. BTA is discussed more in Section 4.3.

Using the CAPWAP program, it is sometimes possible to obtain a reasonable match between computed and measured pile top quantities. In such an analysis the damaged section has to be modeled either by impedance reductions or by slacks. For piles with severe damage along their length it may be necessary to analyze a short pile. It should be born in mind, however, that such an analysis also violates the basic principles of the CAPWAP analysis, namely that the pile is elastic. Also, the nature of the damage is never known with certainty. For example, a broken splice could be a cracked weld either with the neighboring sections lining up well or shifted laterally. In the former case the compression stresses would be similar to those in the undamaged pile; in the latter situation, high stress concentrations would develop. In either case uplift is then uncertain or nonexistent. A sharp bend or toe damage present equally unpredictable situations under sustained loads which may cause further structural deterioration. If a short pile is analyzed then the lower section of the pile below the damage may offer unreliable end bearing and therefore should be discounted.

It is GRL's position that damaged piling should be replaced. Utilizing the CAPWAP calculated capacities should only be done after a very careful consideration of the effects of a loss of the foundation member while in service. Under no circumstances should the CAPWAP calculated capacity be utilized in the same manner in which the capacity of an undamaged pile be used. Under the best of circumstances the capacity should be used with an increased factor of safety and discounting all questionable capacity components. This evaluation cannot be made by GRL as it involves consideration of the type of structure, its seismic environment, the nature of the loads expected, the corrosiveness of the soil material, considerations of scour on the shortened pile, etc.

4.2 *STRESSES*

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from an individual strain transducer, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX, and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance, R(t), minus half the total shaft resistance, SFT. Again, for toe stress estimation, uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum net tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress, again averaged over the cross section and therefore not including bending stresses, can be computed from the pile top measurements by finding the maximum tension force in either traveling upward, $W_{ut,max}$, or downward, $W_{dt,max}$ waves and reducing it by the minimum compressive wave, $W_{oc,min}$, traveling in opposite direction, within the adjoining 2L/c period. The forces in the upward and downward waves can be calculated from the pile top measurements F(t) and v(t) from

$$W_u = ½[F(t) - Zv(t)] \tag{4a}$$

$$W_d = ½[F(t) + Zv(t)] \tag{4b}$$

The maximum tension due to an upward tension wave force Wu,t force is then

$$TSX = \max \left(\begin{array}{l} W_{dt,max} - W_{oc,min} \\ W_{ut,max} - W_{oc,min} \end{array} \right. \tag{5}$$

The simplified iCAP signal matching routine also calculates tensile and compressive stresses along the pile and, if it achieves a satisfactory signal match, more accurately than the PDA closed-form solution. iCAP calculated stresses from signal matching include **CSC** the maximum compression stress anywhere below the gage location, **BSC** the bottom (toe) compression stress, and **TSC** the maximum tension stress below the gage location. For non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA as well as the simplified signal matching results of iCAP may be in error. For piles with joints, cracks, or other discontinuities, CAPWAP provides the best analysis method for tensile and compressive stresses along the pile length.

4.3 *PILE INTEGRITY BY PDA*

Stress waves in a pile are reflected wherever the pile impedance, Z = EA/c = ρcA = A √(E ρ), changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the local relative decrease of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta = (1 + \alpha)/(1 - \alpha) \tag{6}$$

with

$$\alpha = W_{ut}/W_{di} \tag{7}$$

W_{ut} is the upwards traveling reflection wave (negative) due to the damage.

W_{di} is the maximum downward traveling wave due to impact (compressive and thus positive).

Actually, the formula used by the PDA is more complex as it also includes terms reflecting the effect of the soil resistance above the damage location which reduces both impact wave and reflection.

In addition to the quantification of damage, the PDA software also calculates the length to damage, **LTD**, from the time at which the BTA value has been determined.

It can be shown that the BTA calculation is quite meaningful as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections. However, because of the overlapping of waves limitation of Equation 6, when it comes to damage reflections occurring near the toe then either the toe resistance or the reflection of the impact wave tend to obscure the true magnitude of the damage reflection. In that case it is, however, sufficient to know that damage has occurred near the toe which can be assessed from the fact that the toe reflection appears too early (the pile appears to be short). The PDA software in that case displays an LTT (length to toe damage) but with no corresponding BTA value.

When testing or reviewing records with indicated pile damage, a decision has to be made as to what constitutes a serious damage and what could be dismissed as minor. Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.8, and that the pile is essentially broken if BTA is less than 0.6. While there are many reason why this very simplified approach is not a true representation of the strength of the pile portion at and below the damage, it is often useful as a preliminary criterion. The location of damage below the pile top should also be considered by the engineer-or-record when evaluating the acceptability of a damaged pile.

4.4 *HAMMER PERFORMANCE BY PDA*

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_0^t F(t)v(t)\,dt \qquad (9a)$$

The maximum of the E(t) curve is called **EMX** by the PDA but is also often called **ENTHRU**, for example, in GRLWEAP**;** it is the most important information for an overall evaluation of the performance of a hammer and its driving system. **ENTHRU** or **EMX** allow for a classification of the hammer's performance when presented as the transfer ratio, **ETR**, also reflecting the global effectiveness.

$$ETR = EMX/E_R \qquad (9b)$$

where

E_R is the hammer manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^2 - h_L \qquad (10)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and friction losses during impact (usually 0.3 ft or 0.1 m).

4.5 *DETERMINATION OF WAVE SPEED*

An important facet of dynamic pile testing is an assessment of pile material properties. Since, in most cases, force is determined from strain by multiplication with elastic modulus, **E**, and cross sectional area, **A**, the dynamic elastic modulus has to be determined for practically all pile materials. Even steel may have wave speed variations of 1 or 2%. In general, the records measured by the PDA clearly indicate a pile toe reflection in early easy to moderate blow count conditions.. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \qquad (11)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \quad (12)$$

Since the mass density of concrete or steel pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is then easily found from the thus measured wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static modulus and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from a PIT (Low Strain) test is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is non-uniform along the length then the wave speed c, according to Eq. 11, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c of the whole pile is lower than the wave speed at the pile top. It is therefore recommended to determine wave speed and E at the sensors in the beginning of pile driving and not adjust them when the average c changes during the pile installation.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption (e.g. previous experience with piles on site or by the same manufacturer) or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

5. DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the engineer performing PDA tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

5.1 PROPORTIONALITY

As long as there is only a wave traveling in one direction, as is the case during initial impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \quad (13a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \quad (13b)$$

or strain

$$\varepsilon = v / c \quad (13c)$$

This means that the early portion of strain times wave speed must be equal to the pile top particle velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

5.2 *NUMBER OF SENSORS*

Measurements are always taken at opposite sides of the pile so that the average force and velocity in the pile can be calculated. The velocities on the two sides of the pile are very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. In that case the averaging of the two strain signals does not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary and highly recommended to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top and for spiral welded piles with all strain sensors staying away from the welds a distance of a few centimeters or inches. On concrete piles it is critical to not place the strain transducer straddling a crack.

6. LIMITATIONS, OTHER CONSIDERATIONS

6.1 MOBILIZATION OF CAPACITY

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

6.2 TIME DEPENDENT and RATE DEPENDENT SOIL RESISTANCE EFFECTS

Static pile capacity from dynamic method calculations provides an estimate of the axial pile capacity in compression. Increases and decreases in the pile capacity with time typically occur as a result of soil setup or relaxation. Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

6.2.1 SOIL SETUP

Because excess positive pore pressures often develop during pile driving in fine grained soils (clays, silts or even fine sands), the capacity of a pile at the time of driving is often less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile shaft, thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, effective stresses increase and the soil resistance and hence axial pile capacity acting on the pile increases. This phenomena is routinely called soil setup or soil freeze. There are numerous other reasons for soil setup such as realignment of clay particles, arching that reduces effective stresses during pile installation in very dense sands, soil fatigue in over-consolidated clays but also in very dense sands, etc.

6.2.2 RELAXATION

Relaxation, which is capacity reduction with time, has been observed for piles driven into weathered shale, and may take several days to fully develop. Where relaxation occurs, pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically with particular emphasis on the first few “high energy” blows. Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. In general, relaxation occurs at the pile toe and is therefore relevant for end bearing piles. Restrike tests should be performed and compared with the records from early restrike blows in order to avoid dangerous overpredictions.

6.2.3 RATE EFFECTS

The CAPWAP soil model assesses rate effects (elevated resistance caused by a non-zero pile velocity) by identifying the velocity dependent resistance components (static resistance is total resistance minus damping factor times pile velocity). For certain highly plastic soils, however, experience has shown that additional rate effects exist. It is therefore recommended that at least one static test is performed in fine grained materials where no experience exists with the dynamic soil behavior. High unit end bearing in highly plastic soils should be viewed with caution.

6.3 CAPACITY RESULTS FOR OPEN PILE PROFILES

Open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions. The plug behavior may also be quite different for cohesive and non-cohesive materials.

6.4 CAPWAP ANALYSIS RESULTS

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the signal match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations. Further, uplift estimates from dynamic testing should be coupled with higher factors of safety and, for short piles, the shaft resistance may behave very differently and often be considerably smaller in uplift.

6.5 STRESSES

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or nonuniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90%	of yield the steel strength for steel piles
85%	of the concrete compressive strength - minus the effective prestress for concrete piles in compression
100%	of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension
70%	of the reinforcement strength for regularly reinforced concrete piles in tension
300%	of the static design allowable stress for Timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA or CAPWAP for other locations along the pile based on the pile top measurements. The above allowable stresses also apply to those calculated by wave equation.

6.6 ADDITIONAL DESIGN CONSIDERATIONS

Numerous factors have to be considered in pile foundation design. Some of these considerations include:

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements,
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,
- loss of shaft resistance due to scour or other effects,
- Liquefaction and seismic effects,
- loss of structural pile strength due to additional bending loads, buckling (the dynamic loads generally do not cause buckling even though they may exceed the buckling strength of the pile section), corrosion etc.,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

6.7 VIBRATIONS

In certain situations, pile driving can cause ground vibrations and/or vibration induced soil settlements that may adversely impact nearby structures, utilities, facility equipment, etc. Standard industry practice is to perform a preconstruction survey of the neighboring area prior to the commencement of pile driving operations to identify and determine the condition of nearby structures, facilities, and utilities and their susceptibility to potential vibrations. If vibration susceptible concerns are identified, vibration monitoring equipment is used to measure vibration levels associated with the pile driving operations and those measurements are evaluated by a knowledgeable vibration specialist. Vibration monitoring is <u>not</u> a service offered by GRL Engineers. Therefore pile driving vibrations and their effects have not been considered in our analysis of the dynamic test results. Preconstruction surveys, monitoring and mitigating vibration effects are the responsibility of the owner, contractor, and design engineer.

6.8 WAVE EQUATION ANALYSIS RESULTS

The results calculated by the wave equation analysis program depend on a variety of assumptions of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

7. FACTORS OF SAFETY OR RESISTANCE FACTORS

Static or dynamic load tests run to failure yield an ultimate pile bearing capacity, R_{ult}. If this failure load were applied to the pile, then excessive settlements would occur. Therefore, in allowable stress designs it is absolutely necessary that the actually applied load, also often called the design load, R_d (or working load or safe load), is less than R_{ult}. In most soils it is necessary that R_{ult} is at least 50% higher than R_d to limit settlements. This means that

$$R_{ult} \geq 1.5\, R_d, \tag{13}$$

or the Factor of Safety has to be at least 1.5.

Unfortunately, neither applied loads nor R_{ult} are exactly known. One static load test may be performed at a site, but that would not guarantee that all other piles have the same capacity and it is to be expected that a certain percentage of the production piles have lower capacities, either due to soil variability or due to pile damage. Uncertainty also exists because different types of tests and their interpretations present different bearing capacity results for the same pile.

Not only bearing capacity values of all piles are unknown, even loads vary considerably and occasional overloads must be expected. We would not want a structure to become unserviceable or useless because of either an occasional overload or a few piles with low capacity. For this reason, and to avoid being overly conservative which would mean excessive cost, modern safety concepts suggest that the overall factor of safety should reflect both the uncertainty in loads and resistance. Thus, if all piles were tested statically and if we carefully controlled the loads, we probably could live with F.S. = 1.5. However, in general, depending on the building type or load combinations and as a function of quality assurance of pile foundations, a variety of Factors of Safety have been proposed.

For highway bridge loads in the United States, AASHTO allowable stress design guideline specifications proposed the following Factors of Safety (prior to 2007):

F.S. = 1.90 for static load test with wave equation and dynamic test.

F.S. = 2.25 for dynamic testing with wave equation analysis.

F.S. = 2.50 for indicator piles with wave equation analysis.

F.S. = 2.75 for wave equation analysis.

F.S. = 3.50 for FHWA Modified Gates dynamic formula.

It should be mentioned that all of these methods should always be combined with soil exploration and static pile analysis. Also, specifications are occasionally updated and therefore the latest version should be consulted for the current guidance on factors of safety.

Codes and specifications (in the United States for example IBC, PDCA, ASCE, or other specifications issued by State Departments of Transportation) specify different factors of safety. However, the range of recommended factors of safety in the US typically varies between 1.9 and 6.0 for ASD design.

In 2007, Load and Resistance Factor Design (LRFD) was mandated for highway bridge design and construction in the United States. In LRFD, the sum of the factored loads must be less than the nominal resistance, R_n, multiplied by a resistance factor, ϕ.

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \tag{14}$$

The 2014 AASHTO LRFD design specifications recommend the following resistance factors, ϕ_{dyn}, be applied to the nominal resistance based on the selected construction control procedures.

ϕ_{dyn} = 0.80 for driving criteria established by static load test of 1 pile per site condition and dynamic testing with signal matching of at least 2 piles per site condition but no less than 2% of production piles.

ϕ_{dyn} = 0.75 for driving criteria established by successful static load test of 1 pile per site condition without dynamic testing.

ϕ_{dyn} = 0.75 for driving criteria established by dynamic testing with signal matching conducted on 100% of production piles.

ϕ_{dyn} = 0.65 for driving criteria developed by dynamic testing with signal matching, quality control by dynamic testing on 2 piles per site condition, but no less than 2% of production piles.

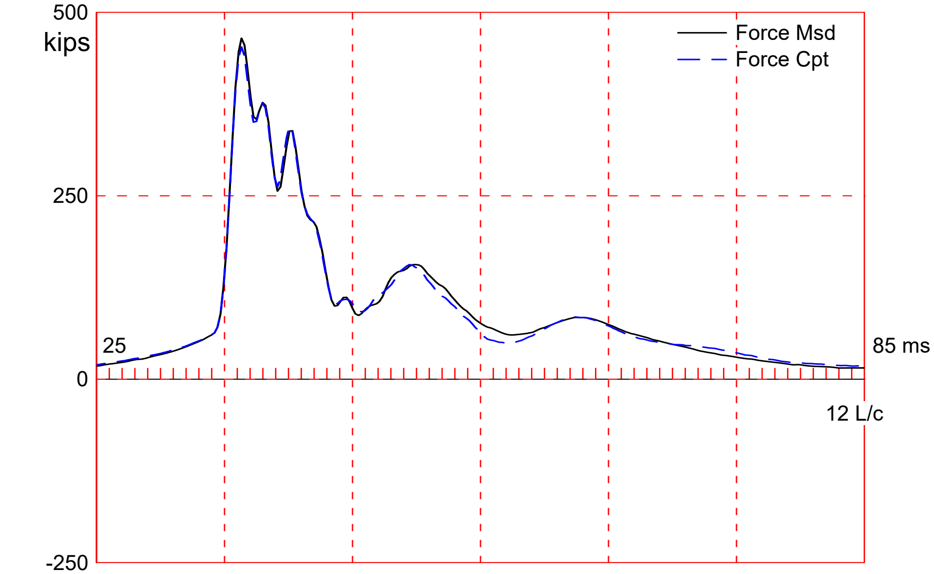
ϕ_{dyn} = 0.50 for wave equation analysis without dynamic measurements or load test but with field confirmation of hammer performance.

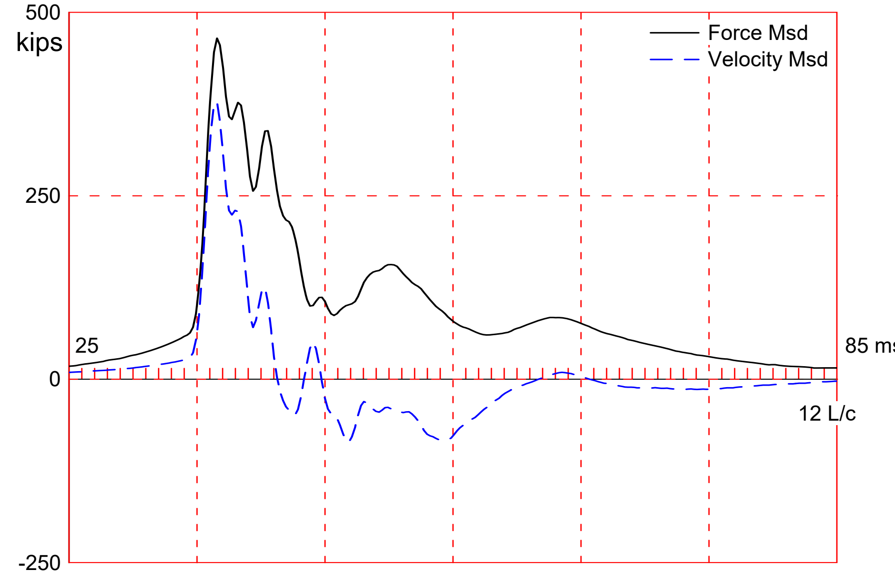
ϕ_{dyn} = 0.40 for FHWA modified Gates dynamic formula (end of drive condition only)

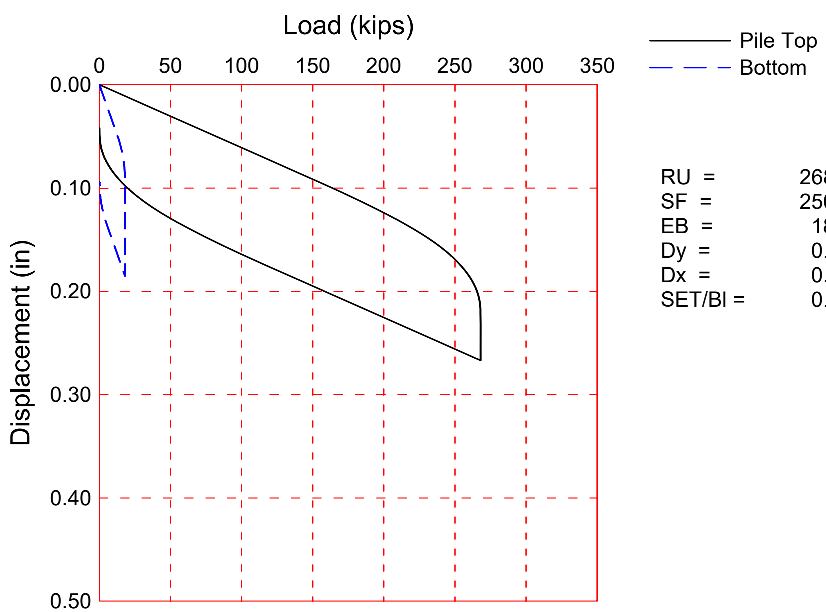
ϕ_{dyn} = 0.10 for Engineering News dynamic formula as defined in AASHTO 10.7.3.8.5 (end of drive conditions only)

In ASD, it is the designer's responsibility to identify the required ultimate capacity based on the design loads and the adopted factor of safety. Similarly in LRFD, it is the designer's responsibility to identify the required nominal resistance based on the factored loads and the construction control procedure and its resistance factor. The required factor of safety in ASD or resistance factor in LRFD should be included in the design drawings and specifications along with the testing requirements.

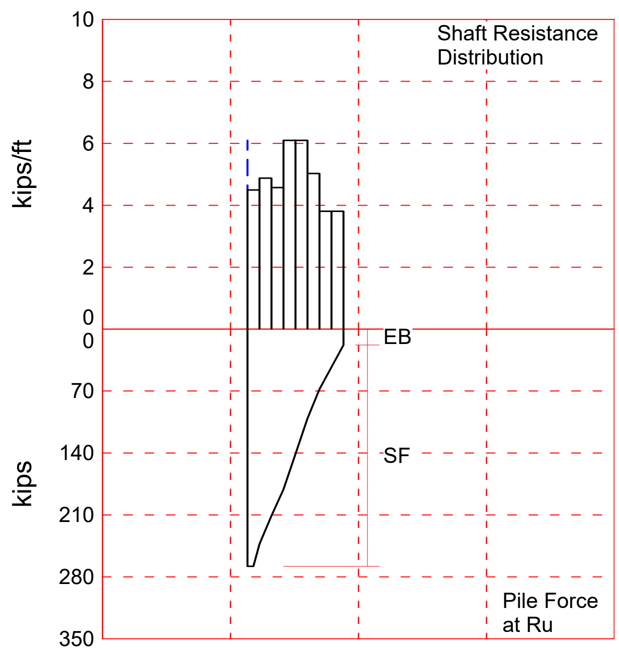
For optimal solutions it is always recommended that increased testing for lower ultimate pile capacities or reduced nominal resistances is considered. Frequent pile testing will also help reduce the confusion that often exists on construction sites as to foundation loads and bearing requirements. In any event, it cannot be expected that the test engineer is aware of and responsible for the variety of considerations that must be met for ASD or LRFD based foundation designs as well as to determine the appropriate factor of safety or resistance factor associated with the design.







RU =	268.0 kips
SF =	250.0 kips
EB =	18.0 kips
Dy =	0.23 in
Dx =	0.27 in
SET/Bl =	0.04 in



Length b. Sensors	52.5 ft
Embedment	51.5 ft
Top Area	153.9 in^2
End Bearing Area	153.9 in^2
Top Perimeter	3.67 ft
Top E-Modulus	7826 ksi
Top Spec. Weight	185.0 lb/ft^3
Top Wave Spd.	14000 ft/s
Overall W.S.	14000 ft/s
Match Quality	1.84
Top Compr. Stress	3.0 ksi
Max Compr. Stress	3.1 ksi
Max Tension Stress	-0.02 ksi
Avg. Shaft Quake	0.11 in
Toe Quake	0.07 in
Avg. Shaft Smith Dpg.	0.35 s/ft
Toe Smith Damping	0.25 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 268.0; along Shaft 250.0; at Toe 18.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				268.0			
1	6.6	5.6	25.0	243.0	25.0	4.49	1.23
2	13.1	12.1	32.0	211.0	57.0	4.88	1.33
3	19.7	18.7	30.0	181.0	87.0	4.57	1.25
4	26.3	25.3	40.0	141.0	127.0	6.10	1.66
5	32.8	31.8	40.0	101.0	167.0	6.10	1.66
6	39.4	38.4	33.0	68.0	200.0	5.03	1.37
7	45.9	44.9	25.0	43.0	225.0	3.81	1.04
8	52.5	51.5	25.0	18.0	250.0	3.81	1.04
Avg. Shaft			31.3			4.85	1.32
Toe			18.0				16.84

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.35	0.25
Quake	(in)	0.11	0.07
Case Damping Factor		1.02	0.05
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	19	
Resistance Gap (included in Toe Quake) (in)			0.03
Soil Plug Weight	(kips)		0.063

CAPWAP match quality = 1.84 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.04 in; Blow Count = 288 b/ft
Computed: Final Set = 0.06 in; Blow Count = 193 b/ft

max. Top Comp. Stress = 3.0 ksi (T= 36.8 ms, max= 1.028 x Top)
max. Comp. Stress = 3.1 ksi (Z= 6.6 ft, T= 37.0 ms)
max. Tens. Stress = -0.02 ksi (Z= 42.7 ft, T= 45.0 ms)
max. Energy (EMX) = 4.9 kip-ft; max. Measured Top Displ. (DMX)= 0.20 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	464.3	0.0	3.0	0.00	4.9	4.4	0.19
2	6.6	477.1	0.0	3.1	0.00	4.8	4.2	0.18
3	9.8	437.9	0.0	2.8	0.00	4.3	4.1	0.18
4	13.1	450.5	0.0	2.9	0.00	4.3	3.9	0.18
5	16.4	401.0	0.0	2.6	0.00	3.6	3.8	0.17
6	19.7	413.9	0.0	2.7	0.00	3.6	3.6	0.17
7	23.0	373.6	0.0	2.4	0.00	3.1	3.5	0.17
8	26.3	386.3	0.0	2.5	0.00	3.1	3.3	0.17
9	29.5	332.7	0.0	2.2	0.00	2.4	3.2	0.16
10	32.8	343.0	0.0	2.2	0.00	2.4	3.1	0.16
11	36.1	291.0	-0.5	1.9	-0.00	1.7	3.0	0.16
12	39.4	296.6	0.0	1.9	0.00	1.7	3.0	0.16
13	42.7	239.2	-2.9	1.6	-0.02	1.1	3.1	0.16
14	45.9	210.1	0.0	1.4	0.00	1.1	3.8	0.16
15	49.2	117.7	-2.4	0.8	-0.02	0.7	4.2	0.16
16	52.5	84.7	0.0	0.6	0.00	0.3	4.3	0.16
Absolute	6.6			3.1			(T =	37.0 ms)
	42.7				-0.02		(T =	45.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	447.8	407.9	367.9	328.0	288.0	248.1	208.1	168.2	128.2	88.3
RX	449.7	410.0	371.2	332.4	295.1	259.7	235.2	218.5	205.1	195.9
RU	500.5	465.8	431.1	396.4	361.7	327.1	292.4	257.7	223.0	188.3

RAU = 190.0 (kips); RA2 = 341.8 (kips)

Current CAPWAP Ru = 268.0 (kips); Corresponding J(RP)= 0.45; J(RX) = 0.48

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
4.4	36.56	382.2	465.2	468.7	0.20	0.05	0.04	5.0	502.8	450

PILE PROFILE AND PILE MODEL

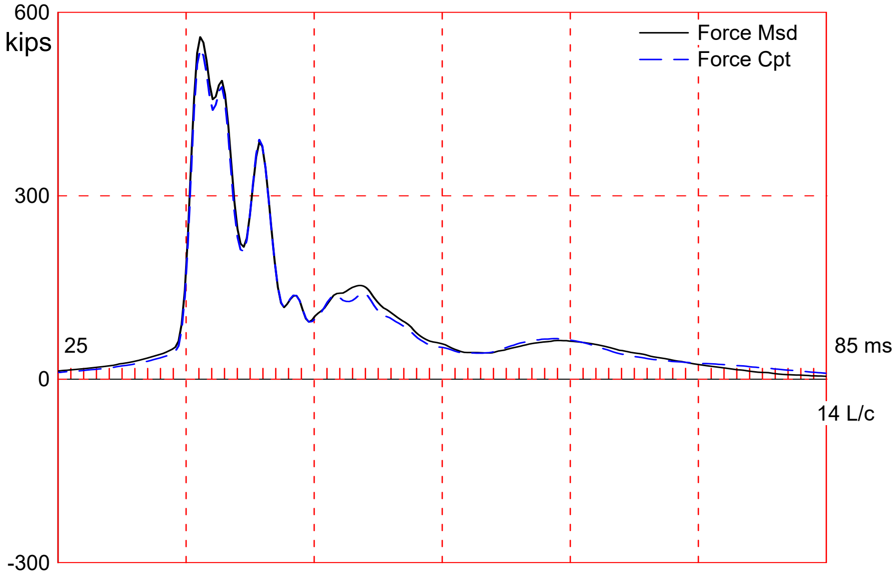
Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.0	153.9	7845.5	185.452	3.67
52.5	153.9	7845.5	185.452	3.67

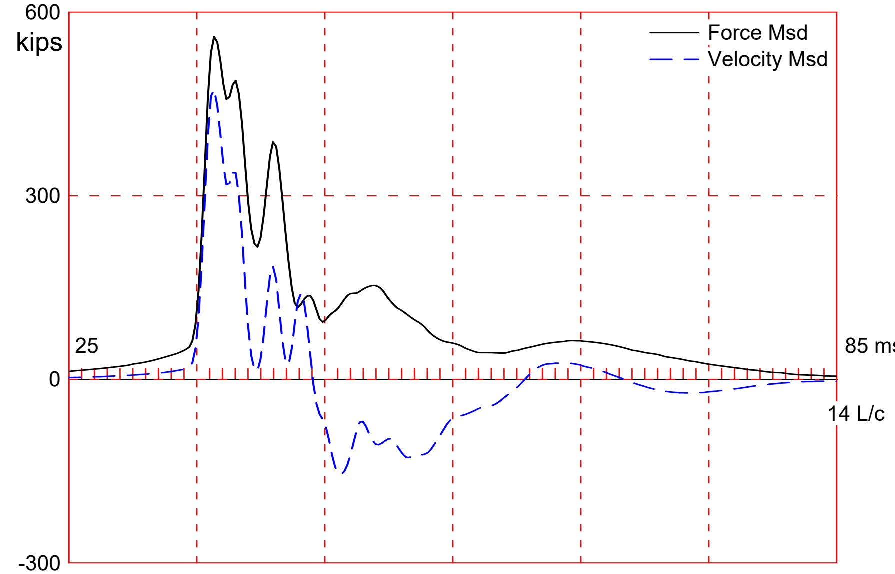
Toe Area 153.9 in²

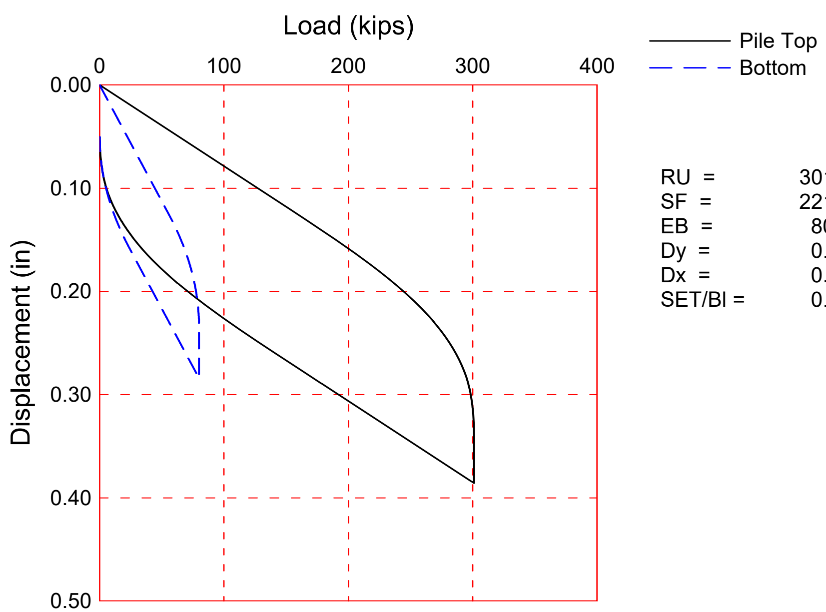
Top Segment Length 3.28 ft, Top Impedance 86 kips/ft/s

Wave Speed: Pile Top 14000.0, Elastic 14000.0, Overall 14000.0 ft/s

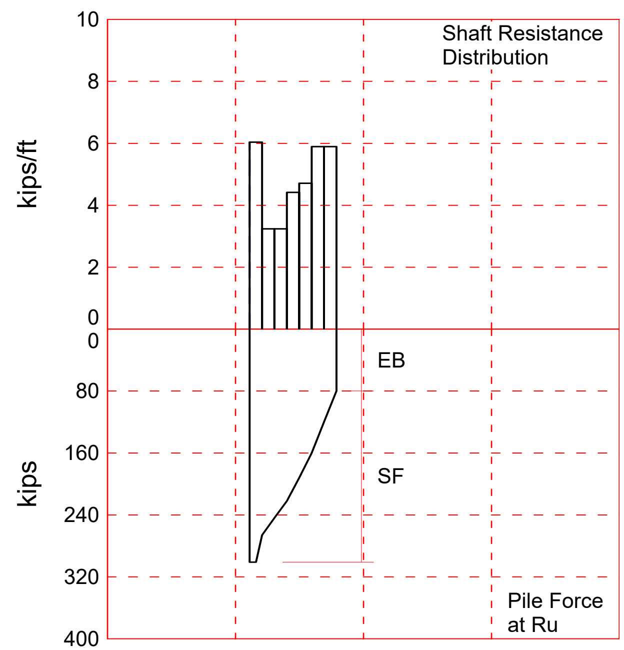
Pile Damping 2.00 %, Time Incr 0.234 ms, 2L/c 7.5 ms
Total volume: 56.123 ft^3; Volume ratio considering added impedance: 1.000







RU =	301.0 kips
SF =	221.0 kips
EB =	80.0 kips
Dy =	0.34 in
Dx =	0.39 in
SET/Bl =	0.05 in



Length b. Sensors	47.5 ft
Embedment	46.5 ft
Top Area	153.9 in^2
End Bearing Area	153.9 in^2
Top Perimeter	3.67 ft
Top E-Modulus	7846 ksi
Top Spec. Weight	185.5 lb/ft^3
Top Wave Spd.	14000 ft/s
Overall W.S.	14000 ft/s
Match Quality	1.83
Top Compr. Stress	3.6 ksi
Max Compr. Stress	3.7 ksi
Max Tension Stress	-0.03 ksi
Avg. Shaft Quake	0.16 in
Toe Quake	0.18 in
Avg. Shaft Smith Dpg.	0.32 s/ft
Toe Smith Damping	0.26 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

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CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 301.0; along Shaft 221.0; at Toe 80.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				301.0			
1	6.8	5.8	35.0	266.0	35.0	6.04	1.65
2	13.6	12.6	22.0	244.0	57.0	3.24	0.88
3	20.4	19.4	22.0	222.0	79.0	3.24	0.88
4	27.1	26.2	30.0	192.0	109.0	4.42	1.21
5	33.9	32.9	32.0	160.0	141.0	4.72	1.29
6	40.7	39.7	40.0	120.0	181.0	5.89	1.61
7	47.5	46.5	40.0	80.0	221.0	5.89	1.61
Avg. Shaft			31.6			4.75	1.30
Toe			80.0				74.84

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.32	0.26
Quake	(in)	0.16	0.18
Case Damping Factor		0.82	0.24
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	97	68
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	93	
Resistance Gap (included in Toe Quake) (in)			0.00
Soil Plug Weight	(kips)		0.015

CAPWAP match quality = 1.83 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.05 in; Blow Count = 240 b/ft
Computed: Final Set = 0.06 in; Blow Count = 190 b/ft

max. Top Comp. Stress = 3.6 ksi (T= 36.6 ms, max= 1.023 x Top)
max. Comp. Stress = 3.7 ksi (Z= 6.8 ft, T= 36.8 ms)
max. Tens. Stress = -0.03 ksi (Z= 6.8 ft, T= 187.1 ms)
max. Energy (EMX) = 8.1 kip-ft; max. Measured Top Displ. (DMX)= 0.25 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	555.5	-4.7	3.6	-0.03	8.1	5.5	0.26
2	6.8	568.6	-4.7	3.7	-0.03	8.1	5.4	0.26
3	10.2	505.4	-0.3	3.3	-0.00	6.9	5.2	0.25
4	13.6	515.0	-0.3	3.3	-0.00	6.8	5.1	0.25
5	17.0	480.5	-0.3	3.1	-0.00	6.0	5.0	0.24
6	20.4	491.1	-0.3	3.2	-0.00	6.0	4.9	0.24
7	23.8	460.8	-1.2	3.0	-0.01	5.3	4.7	0.23
8	27.1	472.5	-0.3	3.1	-0.00	5.3	4.6	0.22
9	30.5	430.2	-0.3	2.8	-0.00	4.4	4.4	0.22
10	33.9	441.8	-0.3	2.9	-0.00	4.3	4.7	0.21
11	37.3	388.7	-0.3	2.5	-0.00	3.4	4.7	0.20
12	40.7	354.5	-0.3	2.3	-0.00	3.4	5.3	0.20
13	44.1	209.8	-0.2	1.4	-0.00	2.3	5.9	0.20
14	47.5	199.4	-0.1	1.3	-0.00	1.3	6.0	0.20
Absolute	6.8			3.7			(T =	36.8 ms)
	6.8				-0.03		(T =	187.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	505.7	452.8	400.0	347.1	294.3	241.4	188.6	135.8	82.9	30.1
RX	509.3	462.6	428.7	394.8	361.3	327.9	294.7	274.1	273.4	272.7
RU	505.7	452.8	400.0	347.1	294.3	241.4	188.6	135.8	82.9	30.1

RAU = 271.8 (kips); RA2 = 390.1 (kips)

Current CAPWAP Ru = 301.0 (kips); Corresponding J(RP)= 0.39; J(RX) = 0.58

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
5.6	36.35	480.7	553.4	563.7	0.25	0.05	0.05	8.1	649.4	444

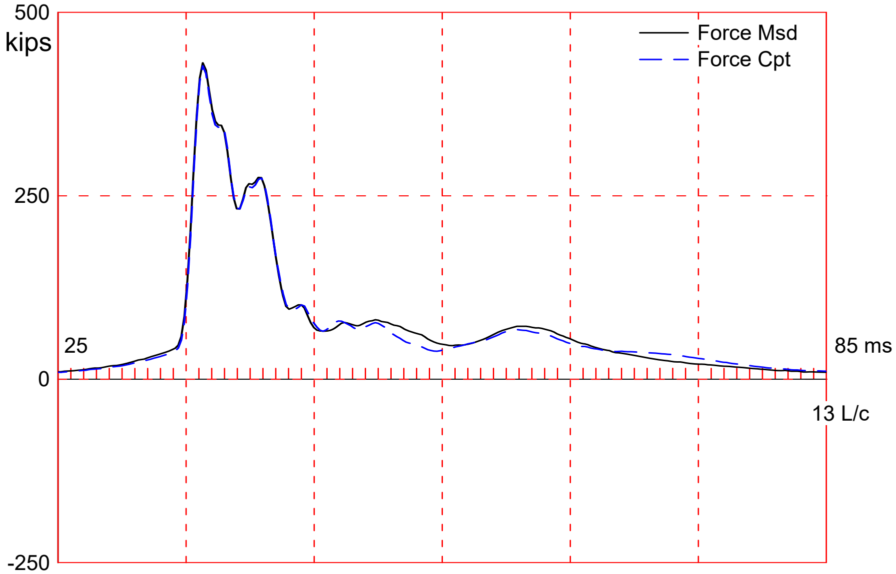
PILE PROFILE AND PILE MODEL

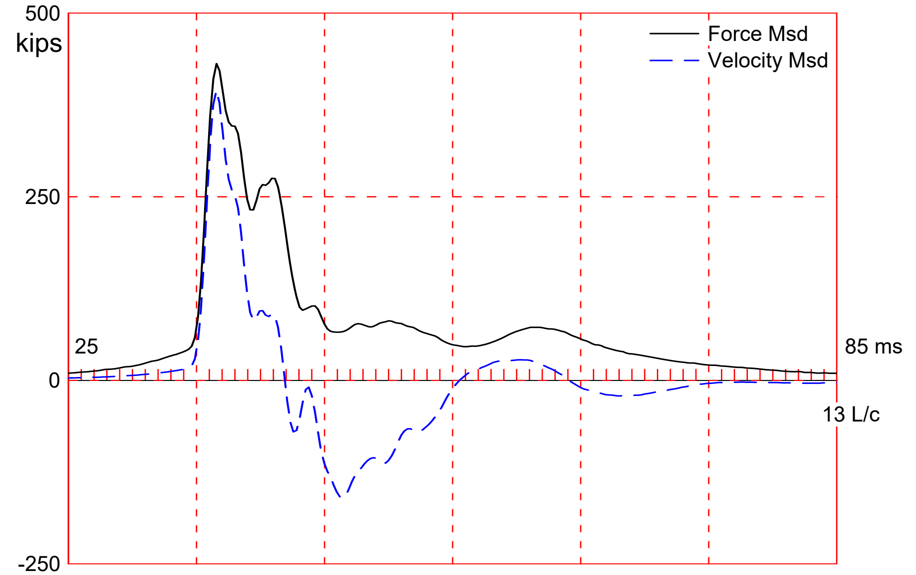
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.0	153.9	7845.5	185.452	3.67
47.5	153.9	7845.5	185.452	3.67

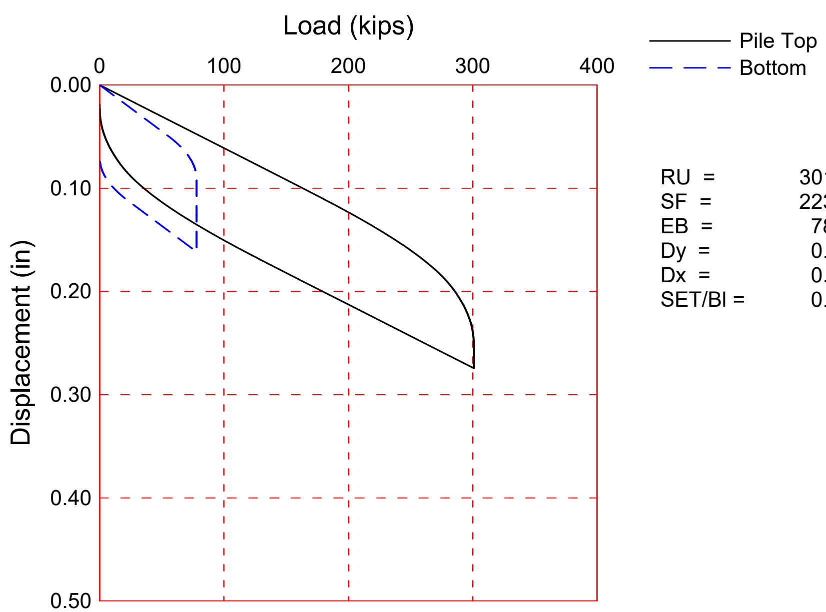
Toe Area 153.9 in^2

Top Segment Length 3.39 ft, Top Impedance 86 kips/ft/s

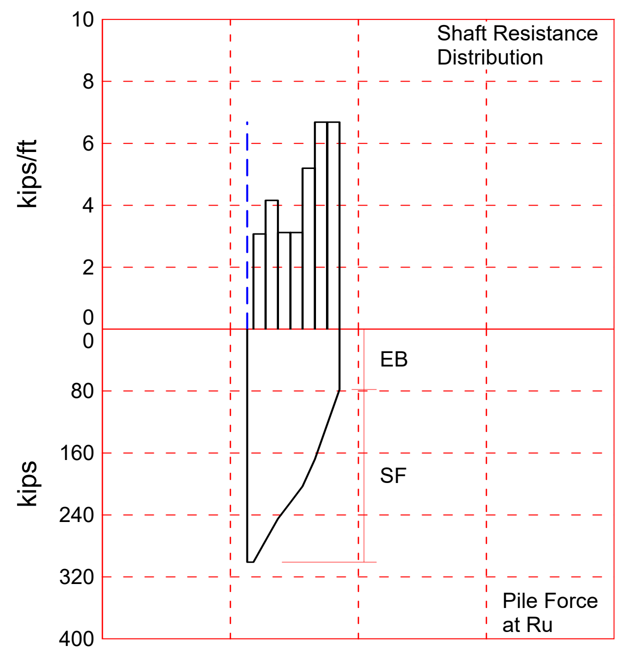
Wave Speed: Pile Top 14000.0, Elastic 14000.0, Overall 14000.0 ft/s
Pile Damping 2.00 %, Time Incr 0.242 ms, 2L/c 6.8 ms
Total volume: 50.778 ft^3; Volume ratio considering added impedance: 1.000







RU =	301.0 kips
SF =	223.0 kips
EB =	78.0 kips
Dy =	0.26 in
Dx =	0.27 in
SET/Bl =	0.02 in



Length b. Sensors	50.5 ft
Embedment	49.5 ft
Top Area	153.9 in^2
End Bearing Area	153.9 in^2
Top Perimeter	3.67 ft
Top E-Modulus	7846 ksi
Top Spec. Weight	185.5 lb/ft^3
Top Wave Spd.	14000 ft/s
Overall W.S.	14000 ft/s
Match Quality	2.02
Top Compr. Stress	2.8 ksi
Max Compr. Stress	3.0 ksi
Max Tension Stress	-0.03 ksi
Avg. Shaft Quake	0.11 in
Toe Quake	0.07 in
Avg. Shaft Smith Dpg.	0.36 s/ft
Toe Smith Damping	0.28 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 301.0; along Shaft 223.0; at Toe 78.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				301.0			
1	10.1	9.1	28.0	273.0	28.0	3.08	0.84
2	16.8	15.8	28.0	245.0	56.0	4.16	1.13
3	23.6	22.6	21.0	224.0	77.0	3.12	0.85
4	30.3	29.3	21.0	203.0	98.0	3.12	0.85
5	37.0	36.0	35.0	168.0	133.0	5.20	1.42
6	43.8	42.8	45.0	123.0	178.0	6.68	1.82
7	50.5	49.5	45.0	78.0	223.0	6.68	1.82
Avg. Shaft			31.9			4.51	1.23
Toe			78.0				72.96

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.36	0.28
Quake	(in)	0.11	0.07
Case Damping Factor		0.93	0.25
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	100	86
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	92	
Soil Plug Weight	(kips)		0.179

CAPWAP match quality = 2.02 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.02 in; Blow Count = 640 b/ft
Computed: Final Set = 0.03 in; Blow Count = 418 b/ft

max. Top Comp. Stress = 2.8 ksi (T= 36.8 ms, max= 1.056 x Top)
max. Comp. Stress = 3.0 ksi (Z= 10.1 ft, T= 37.3 ms)
max. Tens. Stress = -0.03 ksi (Z= 10.1 ft, T= 185.4 ms)
max. Energy (EMX) = 4.7 kip-ft; max. Measured Top Displ. (DMX)= 0.19 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	434.5	-4.1	2.8	-0.03	4.7	4.5	0.19
2	6.7	446.3	-4.2	2.9	-0.03	4.6	4.4	0.18
3	10.1	458.8	-4.4	3.0	-0.03	4.6	4.2	0.17
4	13.5	414.3	0.0	2.7	0.00	4.0	4.1	0.17
5	16.8	424.6	0.0	2.8	0.00	4.0	4.0	0.17
6	20.2	381.2	0.0	2.5	0.00	3.4	3.9	0.16
7	23.6	389.6	0.0	2.5	0.00	3.4	3.8	0.16
8	26.9	361.1	0.0	2.3	0.00	3.0	3.7	0.15
9	30.3	371.4	0.0	2.4	0.00	2.9	3.5	0.15
10	33.7	348.3	0.0	2.3	0.00	2.6	3.4	0.14
11	37.0	360.7	0.0	2.3	0.00	2.5	3.2	0.13
12	40.4	313.3	0.0	2.0	0.00	2.0	3.2	0.13
13	43.8	293.8	0.0	1.9	0.00	1.9	3.5	0.12
14	47.1	189.6	0.0	1.2	0.00	1.3	3.9	0.12
15	50.5	189.4	-2.0	1.2	-0.01	0.8	3.9	0.11
Absolute	10.1			3.0			(T =	37.3 ms)
	10.1				-0.03		(T =	185.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	470.7	434.4	398.1	361.8	325.4	289.1	252.8	216.4	180.1	143.8
RX	470.7	434.4	398.1	361.8	325.6	295.2	274.8	254.7	238.5	231.8
RU	505.1	472.3	439.4	406.5	373.6	340.7	307.8	274.9	242.0	209.2

RAU = 184.7 (kips); RA2 = 328.4 (kips)

Current CAPWAP Ru = 301.0 (kips); Corresponding J(RP)= 0.47; J(RX) = 0.48

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
4.6	36.55	398.4	435.6	435.6	0.19	0.02	0.02	4.8	560.2	1114

PILE PROFILE AND PILE MODEL

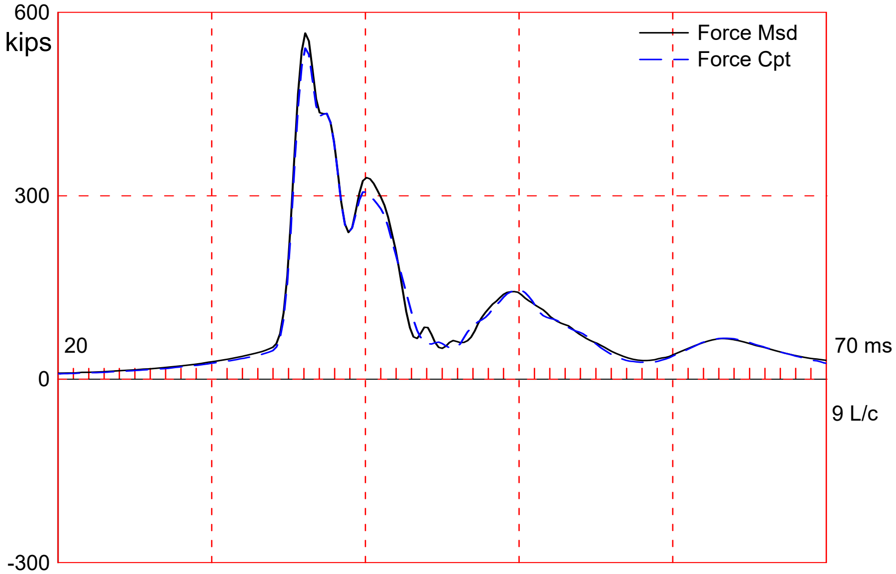
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.0	153.9	7845.5	185.452	3.67
50.5	153.9	7845.5	185.452	3.67

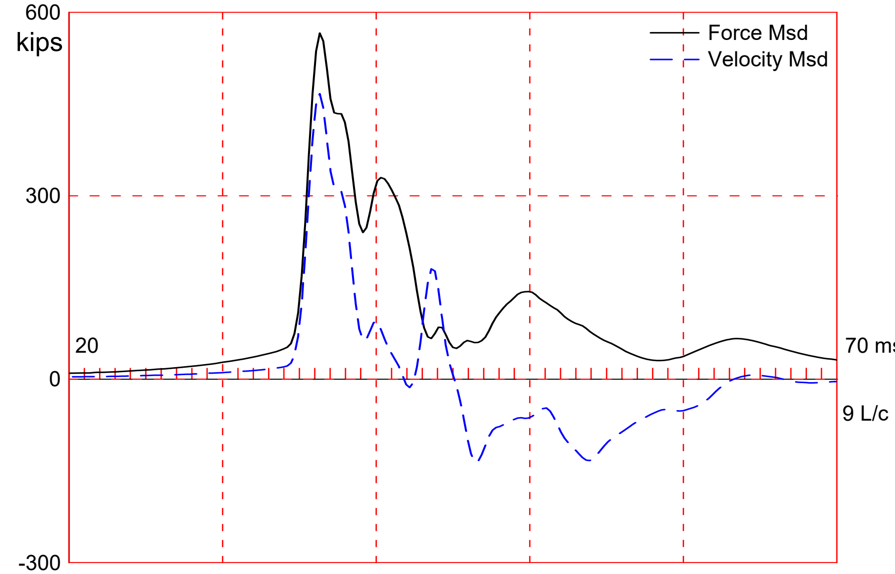
Toe Area 153.9 in^2

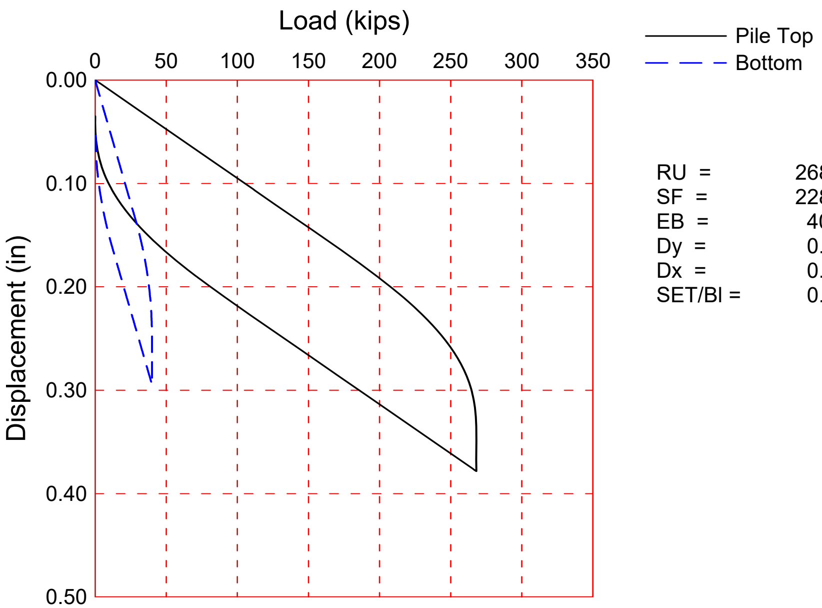
Top Segment Length 3.37 ft, Top Impedance 86 kips/ft/s

Wave Speed: Pile Top 14000.0, Elastic 14000.0, Overall 14000.0 ft/s
Pile Damping 2.00 %, Time Incr 0.240 ms, 2L/c 7.2 ms

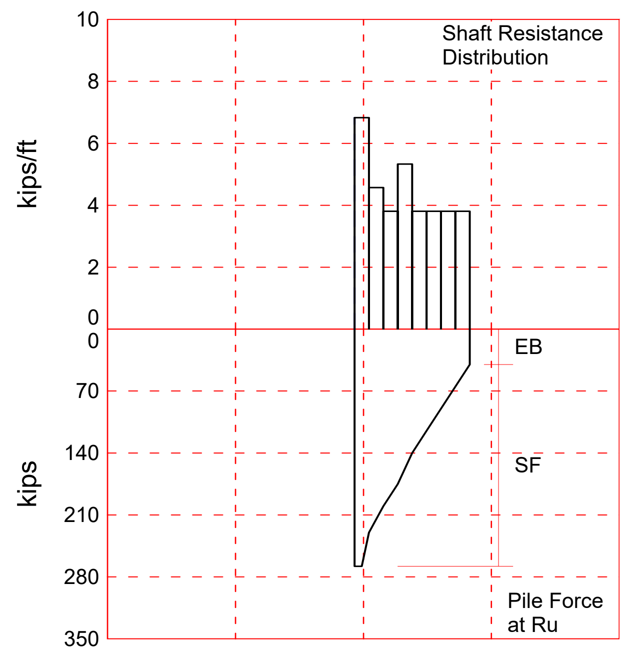
Total volume: 53.985 ft^3; Volume ratio considering added impedance: 1.000







RU =	268.0 kips
SF =	228.0 kips
EB =	40.0 kips
Dy =	0.34 in
Dx =	0.38 in
SET/Bl =	0.03 in



Length b. Sensors	52.5 ft
Embedment	51.5 ft
Top Area	153.9 in^2
End Bearing Area	153.9 in^2
Top Perimeter	3.67 ft
Top E-Modulus	7846 ksi
Top Spec. Weight	185.5 lb/ft^3
Top Wave Spd.	14000 ft/s
Overall W.S.	14000 ft/s
Match Quality	1.98
Top Compr. Stress	3.6 ksi
Max Compr. Stress	3.7 ksi
Max Tension Stress	-0.59 ksi
Avg. Shaft Quake	0.20 in
Toe Quake	0.19 in
Avg. Shaft Smith Dpg.	0.30 s/ft
Toe Smith Damping	0.32 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

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CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

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Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 268.0; along Shaft 228.0; at Toe 40.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				268.0			
1	6.6	5.6	38.0	230.0	38.0	6.83	1.86
2	13.1	12.1	30.0	200.0	68.0	4.57	1.25
3	19.7	18.7	25.0	175.0	93.0	3.81	1.04
4	26.3	25.3	35.0	140.0	128.0	5.33	1.46
5	32.8	31.8	25.0	115.0	153.0	3.81	1.04
6	39.4	38.4	25.0	90.0	178.0	3.81	1.04
7	45.9	44.9	25.0	65.0	203.0	3.81	1.04
8	52.5	51.5	25.0	40.0	228.0	3.81	1.04
Avg. Shaft			28.5			4.43	1.21
Toe			40.0				37.42

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.30	0.32
Quake	(in)	0.20	0.19
Case Damping Factor		0.79	0.15
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	92	101
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	51	
Resistance Gap (included in Toe Quake) (in)			0.03
Soil Plug Weight	(kips)	0.150	

CAPWAP match quality = 1.98 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.03 in; Blow Count = 348 b/ft
Computed: Final Set = 0.05 in; Blow Count = 225 b/ft

max. Top Comp. Stress = 3.6 ksi (T= 36.6 ms, max= 1.025 x Top)
max. Comp. Stress = 3.7 ksi (Z= 6.6 ft, T= 36.8 ms)
max. Tens. Stress = -0.59 ksi (Z= 36.1 ft, T= 41.5 ms)
max. Energy (EMX) = 7.0 kip-ft; max. Measured Top Displ. (DMX)= 0.24 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	557.1	0.0	3.6	0.00	7.0	5.5	0.26
2	6.6	571.0	0.0	3.7	0.00	7.0	5.3	0.26
3	9.8	508.6	-24.4	3.3	-0.16	5.9	5.2	0.25
4	13.1	519.4	-2.9	3.4	-0.02	5.9	5.1	0.25
5	16.4	473.8	-12.0	3.1	-0.08	5.0	4.9	0.25
6	19.7	484.7	0.0	3.1	0.00	5.0	4.8	0.24
7	23.0	452.2	-1.3	2.9	-0.01	4.3	4.7	0.24
8	26.3	463.4	0.0	3.0	0.00	4.3	4.5	0.23
9	29.5	416.7	-69.3	2.7	-0.45	3.5	4.4	0.23
10	32.8	428.7	-77.8	2.8	-0.51	3.4	4.2	0.23
11	36.1	396.3	-91.5	2.6	-0.59	2.9	4.1	0.23
12	39.4	397.0	-23.7	2.6	-0.15	2.8	4.8	0.22
13	42.7	343.9	-17.5	2.2	-0.11	2.1	5.1	0.22
14	45.9	292.2	-0.0	1.9	-0.00	2.1	5.7	0.22
15	49.2	160.8	-0.0	1.0	-0.00	1.4	6.4	0.22
16	52.5	117.8	-0.0	0.8	-0.00	0.7	6.6	0.22
Absolute	6.6			3.7			(T =	36.8 ms)
	36.1				-0.59		(T =	41.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	460.7	402.7	344.7	286.7	228.7	170.8	112.8	54.8	0.0	0.0
RX	465.6	409.2	353.1	298.9	267.4	240.7	226.7	217.0	208.8	205.5
RU	482.7	426.9	371.1	315.3	259.5	203.7	147.9	92.1	36.3	0.0

RAU = 203.3 (kips); RA2 = 307.4 (kips)

Current CAPWAP Ru = 268.0 (kips); Corresponding J(RP)= 0.33; J(RX) = 0.40

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
5.5	36.33	472.0	568.7	572.0	0.24	0.03	0.03	6.9	593.7	250

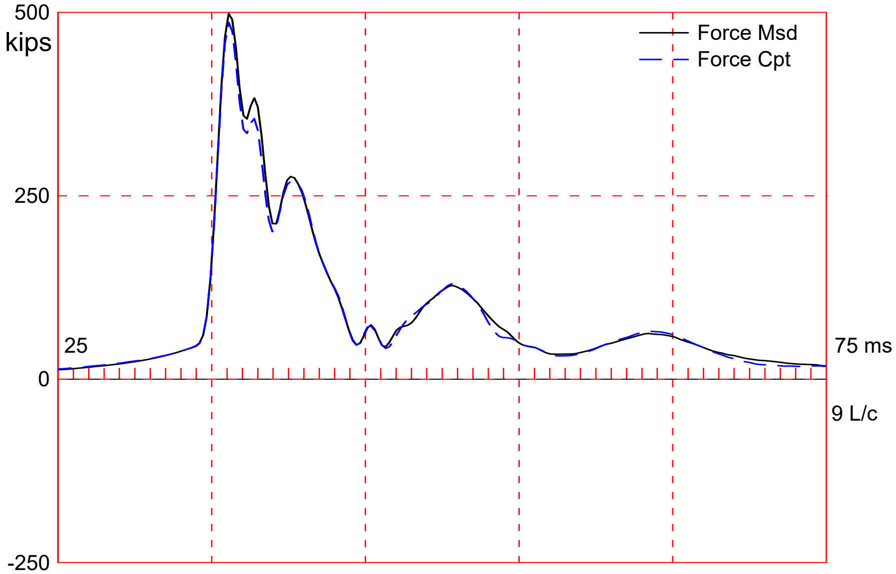
PILE PROFILE AND PILE MODEL

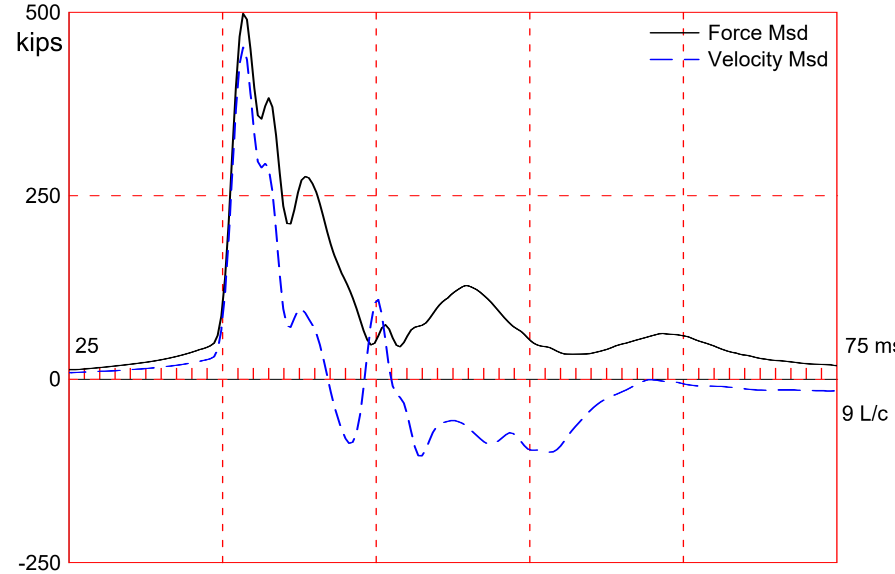
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.0	153.9	7845.5	185.452	3.67
52.5	153.9	7845.5	185.452	3.67

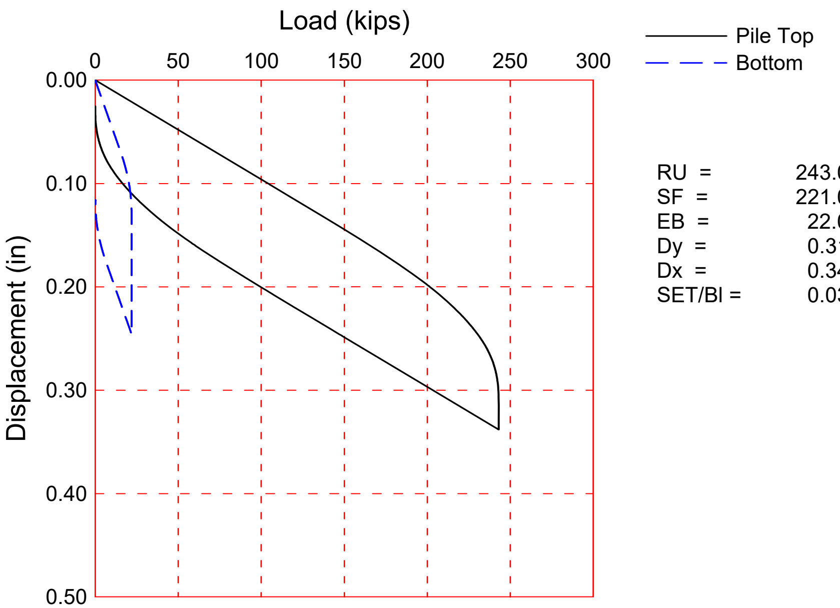
Toe Area 153.9 in^2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Wave Speed ft/s	Soil Plug kips
1	3.3	86.27	0.00	0.00	0.000	-0.00	0.000	3.67	14000.0	0.000
11	36.1	86.27	0.00	0.00	0.000	-0.00	0.000	3.67	14000.0	0.025
12	39.4	86.27	0.00	0.00	0.000	-0.00	0.000	3.67	14000.0	0.065
13	42.7	86.27	0.00	0.00	0.000	-0.00	0.000	3.67	14000.0	0.045
14	45.9	86.27	0.00	0.00	0.000	-0.00	0.000	3.67	14000.0	0.015
15	49.2	86.27	0.00	0.00	0.000	-0.00	0.000	3.67	14000.0	0.000
16	52.5	86.27	0.00	0.00	0.000	-0.00	0.000	3.67	14000.0	0.000

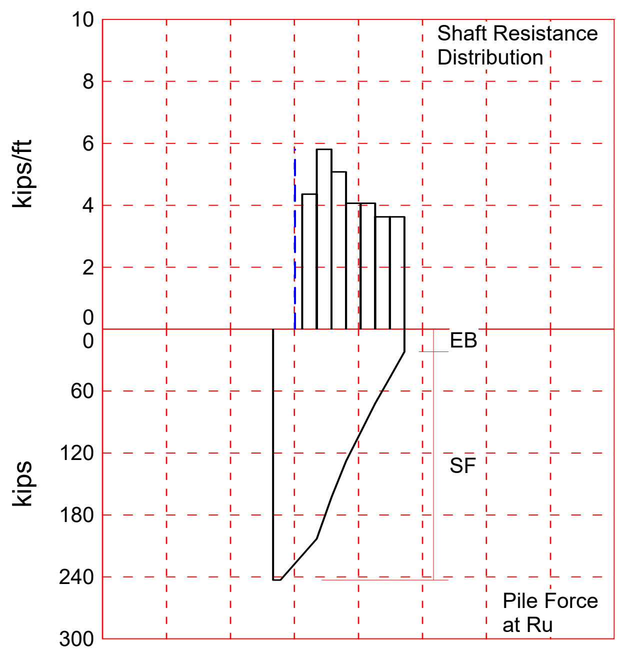
Wave Speed: Pile Top 14000.0, Elastic 14000.0, Overall 14000.0 ft/s
Pile Damping 2.00 %, Time Incr 0.234 ms, 2L/c 7.5 ms
Total volume: 56.123 ft^3; Volume ratio considering added impedance: 1.000







RU =	243.0 kips
SF =	221.0 kips
EB =	22.0 kips
Dy =	0.31 in
Dx =	0.34 in
SET/Bl =	0.03 in



Length b. Sensors	62.0 ft
Embedment	50.5 ft
Top Area	153.9 in^2
End Bearing Area	153.9 in^2
Top Perimeter	3.67 ft
Top E-Modulus	8416 ksi
Top Spec. Weight	185.5 lb/ft^3
Top Wave Spd.	14500 ft/s
Overall W.S.	14419 ft/s
Match Quality	1.30
Top Compr. Stress	3.2 ksi
Max Compr. Stress	3.5 ksi
Max Tension Stress	-0.27 ksi
Avg. Shaft Quake	0.17 in
Toe Quake	0.10 in
Avg. Shaft Smith Dpg.	0.35 s/ft
Toe Smith Damping	0.35 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 243.0; along Shaft 221.0; at Toe 22.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				243.0			
1	20.7	9.2	40.0	203.0	40.0	4.36	1.19
2	27.6	16.1	40.0	163.0	80.0	5.81	1.58
3	34.4	22.9	35.0	128.0	115.0	5.08	1.39
4	41.3	29.8	28.0	100.0	143.0	4.06	1.11
5	48.2	36.7	28.0	72.0	171.0	4.06	1.11
6	55.1	43.6	25.0	47.0	196.0	3.63	0.99
7	62.0	50.5	25.0	22.0	221.0	3.63	0.99
Avg. Shaft			31.6			4.38	1.19
Toe			22.0				20.58

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.35	0.35
Quake	(in)	0.17	0.10
Case Damping Factor		0.87	0.09
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	92	63
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	50	
Resistance Gap (included in Toe Quake) (in)			0.02

CAPWAP match quality = 1.30 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.03 in; Blow Count = 480 b/ft
Computed: Final Set = 0.04 in; Blow Count = 312 b/ft

max. Top Comp. Stress = 3.2 ksi (T= 36.6 ms, max= 1.092 x Top)
max. Comp. Stress = 3.5 ksi (Z= 20.7 ft, T= 37.8 ms)
max. Tens. Stress = -0.27 ksi (Z= 44.8 ft, T= 42.0 ms)
max. Energy (EMX) = 5.6 kip-ft; max. Measured Top Displ. (DMX)= 0.22 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	487.1	-5.0	3.2	-0.03	5.6	5.2	0.22
2	6.9	487.5	-10.6	3.2	-0.07	5.6	5.2	0.22
3	10.3	490.0	-13.2	3.2	-0.09	5.6	5.1	0.21
4	13.8	499.1	-5.0	3.2	-0.03	5.5	5.0	0.21
5	17.2	515.0	-5.1	3.3	-0.03	5.5	4.8	0.20
6	20.7	531.9	-5.1	3.5	-0.03	5.5	4.6	0.20
7	24.1	468.2	-2.8	3.0	-0.02	4.6	4.4	0.20
8	27.6	482.6	-2.8	3.1	-0.02	4.6	4.3	0.20
9	31.0	421.4	-0.8	2.7	-0.01	3.7	4.1	0.20
10	34.4	432.6	-0.9	2.8	-0.01	3.7	4.0	0.20
11	37.9	381.4	-34.0	2.5	-0.22	2.9	3.9	0.20
12	41.3	390.3	-32.5	2.5	-0.21	2.9	3.8	0.20
13	44.8	352.7	-41.8	2.3	-0.27	2.3	3.7	0.20
14	48.2	359.9	-4.9	2.3	-0.03	2.3	4.0	0.19
15	51.7	308.9	-24.1	2.0	-0.16	1.7	4.1	0.19
16	55.1	265.4	-7.7	1.7	-0.05	1.7	4.7	0.19
17	58.6	149.8	-5.6	1.0	-0.04	1.1	5.4	0.19
18	62.0	110.2	-4.3	0.7	-0.03	0.5	5.5	0.19
Absolute	20.7			3.5			(T =	37.8 ms)
	44.8				-0.27		(T =	42.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	449.3	398.3	347.3	296.3	245.2	194.2	143.2	92.2	41.2	0.0
RX	449.3	398.3	347.5	297.0	252.8	223.0	203.5	194.1	189.5	184.8
RU	495.6	449.2	402.9	356.5	310.1	263.7	217.3	170.9	124.5	78.2

RAU = 180.4 (kips); RA2 = 270.9 (kips)

Current CAPWAP Ru = 243.0 (kips); Corresponding J(RP)= 0.40; J(RX) = 0.43

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
5.1	36.34	455.9	503.5	503.5	0.22	0.03	0.03	5.7	563.0	275

PILE PROFILE AND PILE MODEL

Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.0	153.9	8415.9	185.452	3.67
62.0	153.9	8415.9	185.452	3.67

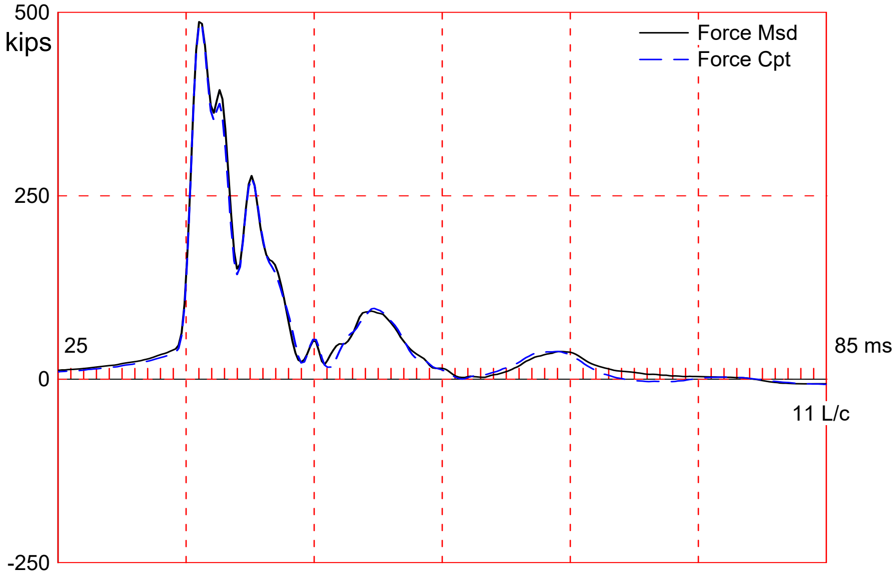
Toe Area 153.9 in²

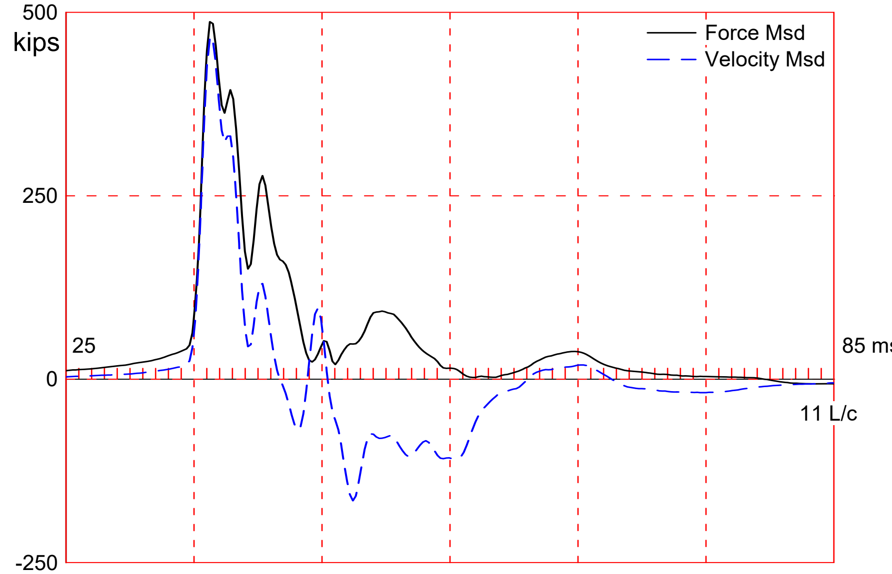
Top Segment Length 3.44 ft, Top Impedance 89 kips/ft/s

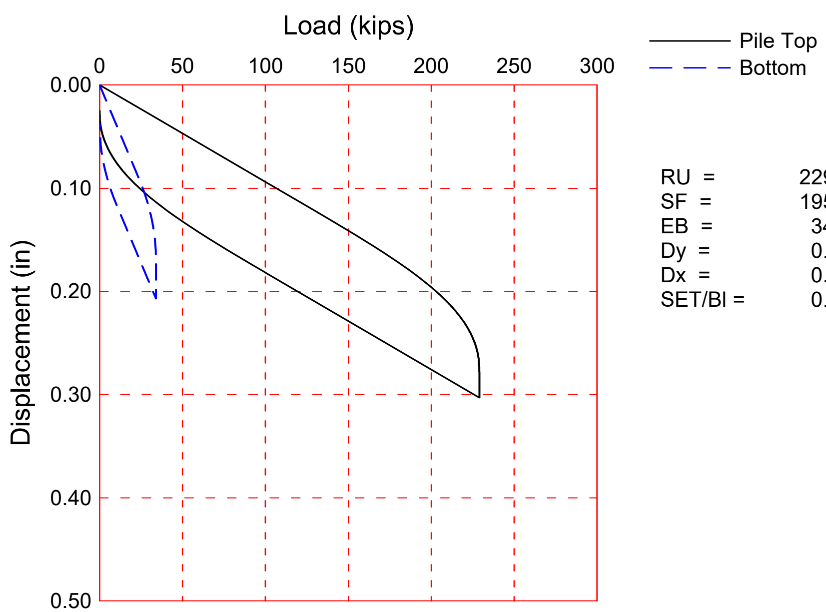
Wave Speed: Pile Top 14500.0, Elastic 14500.0, Overall 14418.6 ft/s

Pile Damping 2.00 %, Time Incr 0.238 ms, 2L/c 8.6 ms

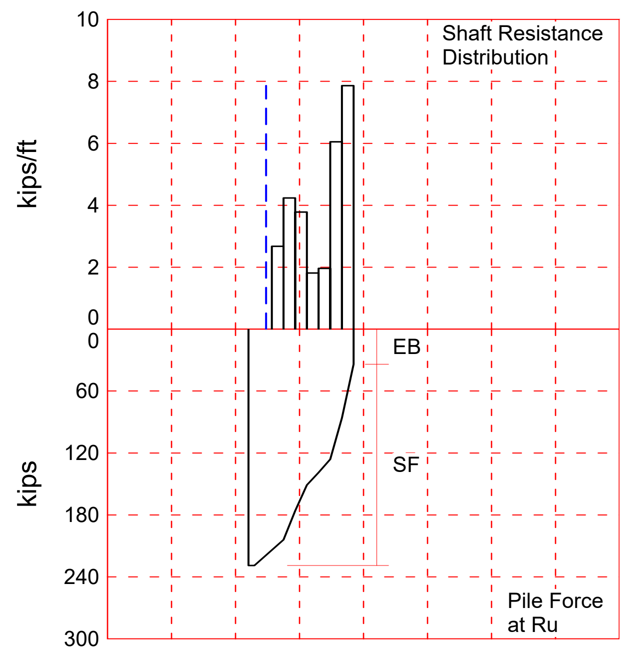
Total volume: 66.279 ft³; Volume ratio considering added impedance: 1.000







RU =	229.0 kips
SF =	195.0 kips
EB =	34.0 kips
Dy =	0.28 in
Dx =	0.30 in
SET/Bl =	0.03 in



Length b. Sensors	59.5 ft
Embedment	49.0 ft
Top Area	153.9 in²
End Bearing Area	153.9 in²
Top Perimeter	3.67 ft
Top E-Modulus	8416 ksi
Top Spec. Weight	185.5 lb/ft³
Top Wave Spd.	14500 ft/s
Overall W.S.	14337 ft/s
Match Quality	1.84
Top Compr. Stress	3.2 ksi
Max Compr. Stress	3.4 ksi
Max Tension Stress	-0.29 ksi
Avg. Shaft Quake	0.14 in
Toe Quake	0.13 in
Avg. Shaft Smith Dpg.	0.35 s/ft
Toe Smith Damping	0.32 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 229.0; along Shaft 195.0; at Toe 34.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				229.0			
1	19.8	9.3	25.0	204.0	25.0	2.68	0.73
2	26.4	15.9	28.0	176.0	53.0	4.24	1.16
3	33.1	22.6	25.0	151.0	78.0	3.78	1.03
4	39.7	29.2	12.0	139.0	90.0	1.82	0.50
5	46.3	35.8	13.0	126.0	103.0	1.97	0.54
6	52.9	42.4	40.0	86.0	143.0	6.05	1.65
7	59.5	49.0	52.0	34.0	195.0	7.87	2.15
Avg. Shaft			27.9			3.98	1.09
Toe			34.0				31.80

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.35	0.32
Quake	(in)	0.14	0.13
Case Damping Factor		0.76	0.12
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	92	45
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	49	

CAPWAP match quality = 1.84 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.03 in; Blow Count = 480 b/ft
Computed: Final Set = 0.05 in; Blow Count = 240 b/ft

max. Top Comp. Stress = 3.2 ksi (T= 36.5 ms, max= 1.070 x Top)
max. Comp. Stress = 3.4 ksi (Z= 19.8 ft, T= 37.8 ms)
max. Tens. Stress = -0.29 ksi (Z= 36.4 ft, T= 41.9 ms)
max. Energy (EMX) = 5.7 kip-ft; max. Measured Top Displ. (DMX)= 0.20 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	485.5	-13.5	3.2	-0.09	5.7	5.2	0.22
2	6.6	485.3	-32.8	3.2	-0.21	5.6	5.2	0.21
3	9.9	486.6	-29.6	3.2	-0.19	5.6	5.2	0.21
4	13.2	494.9	-26.1	3.2	-0.17	5.6	5.1	0.21
5	16.5	507.3	-28.9	3.3	-0.19	5.6	5.0	0.21
6	19.8	519.5	-31.9	3.4	-0.21	5.6	4.8	0.21
7	23.1	479.5	-27.4	3.1	-0.18	4.9	4.7	0.20
8	26.4	490.5	-27.1	3.2	-0.18	4.9	4.6	0.20
9	29.8	444.7	-21.0	2.9	-0.14	4.2	4.5	0.20
10	33.1	451.5	-21.1	2.9	-0.14	4.2	4.3	0.20
11	36.4	408.9	-44.1	2.7	-0.29	3.6	4.3	0.20
12	39.7	414.1	-34.3	2.7	-0.22	3.6	4.2	0.19
13	43.0	398.4	-17.1	2.6	-0.11	3.3	4.2	0.19
14	46.3	407.5	-16.9	2.6	-0.11	3.3	4.2	0.19
15	49.6	381.2	-16.3	2.5	-0.11	2.9	4.3	0.18
16	52.9	351.9	-16.2	2.3	-0.11	2.9	4.9	0.18
17	56.2	208.7	-11.6	1.4	-0.08	1.9	5.5	0.18
18	59.5	175.1	-12.3	1.1	-0.08	0.6	5.6	0.18
Absolute	19.8			3.4			(T =	37.8 ms)
	36.4				-0.29		(T =	41.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	449.5	397.6	345.7	293.8	241.9	190.0	138.1	86.2	34.3	0.0
RX	450.3	398.6	346.9	310.9	280.7	252.8	229.0	218.0	215.1	213.8
RU	492.1	444.5	396.8	349.2	301.5	253.9	206.2	158.6	111.0	63.3

RAU = 211.2 (kips); RA2 = 285.8 (kips)

Current CAPWAP Ru = 229.0 (kips); Corresponding J(RP)= 0.42; J(RX) = 0.60

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
5.3	36.25	474.5	494.1	494.3	0.20	0.03	0.03	5.7	597.5	262

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.0	153.9	8415.9	185.452	3.67
59.5	153.9	8415.9	185.452	3.67

Toe Area 153.9 in^2

Top Segment Length 3.31 ft, Top Impedance 89 kips/ft/s

Wave Speed: Pile Top 14500.0, Elastic 14500.0, Overall 14337.3 ft/s
Pile Damping 2.00 %, Time Incr 0.228 ms, 2L/c 8.3 ms
Total volume: 63.606 ft^3; Volume ratio considering added impedance: 1.000

GRL Engineers, Inc.

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Phone: (847) 221-2750 Fax: (847) 221-2752

TRANSMITTAL

To: Dan Klaproth, P.E.	From: Travis Coleman, P.E.
Company: Koehler Engineering	No. of Sheets: 40
E-mail:dklaproth@koehlerengineering.com	Date: October 16, 2017

RE: CAPWAP Analyses
Route U over Dry Run Ditch, New Madrid County, MO

This transmittal summarizes our CAPWAP analyses of the dynamic load test data collected by Koehler Engineering. On September 14, 2017 GRL was contracted to perform the analyses. GRL waited to finalize these analyses pending information on the static load test results, which was requested by Koehler Engineering. On October 6, GRL was informed by University of Missouri and MODOT personnel that the intent of the test was to compare the analysis methods and that the static load test results would not be shared.

Testing objectives included mobilized pile capacity of pre-cast piles from the existing bridge under re-construction. Koehler Engineering, using a Pile Driving Analyzer, acquired the dynamic data and provided testing details. Further evaluation of bearing capacity including an assessment of the soil resistance distribution was conducted by GRL Engineers, Inc. using the CAPWAP® Version 2014 program.

The tested piles were Bent 1 Pile 6, Bent 3 Pile 4 and Pile 5, and Bent 4 Pile 1, Pile 2, and Pile 3. Bent 3 Pile 4 was installed at a batter angle, the remaining piles were vertically oriented. It was reported to GRL Engineers, Inc. that the piles are octagonal pre-cast piles with a constant diameter of 16 inches, with a taper to a 10 inch diameter over the lower five feet of the pile length. GRL understands the piles were extracted following restrike testing; the reported pile lengths are included in the summary on the following page. The reported beginning of restrike blow count was 12 blows per inch for Bent 3 Pile 5. For the remainder of the piles the beginning of restrike blow count ranged from 10 blows for ¼ inch to 20 blows for ¼ inch. The piles were restruck with a Delmag D-15 diesel hammer.

The CAPWAP analyses are summarize in the table on the following page. For each analysis, the resistance is separated into shaft resistance and end bearing components of the mobilized CAPWAP capacity. **Please note** – At blow counts greater than 10 blows per inch the full pile capacity, particularly at and near the pile toe, is not fully mobilized. To fully mobilize the capacity of these piles would have required a larger hammer.

Pile Number	Reported Pile Length (feet)	Shaft Resistance (kips)	End Bearing (kips)	Total Capacity (kips)
Bent 1 Pile 6	21.3	187	60	247
Bent 3 Pile 4	30.1	265	70	335
Bent 3 Pile 5	30.2	236	65	301
Bent 4 Pile 1	23.5	181	65	246
Bent 4 Pile 2	25.1	144	62	206

For Bent 4 Pile 3, one strain gage was loose during the restrike event. Thus, the data is unsuitable for analysis.

GRL recommends a thorough review GRL's stated understanding of the reported pile details. Any discrepancies in the pile details, lengths, blow count, etc. have significant effects on the CAPWAP results. Please see the attached Appendix A for further discussion of dynamic testing and CAPWAP analysis. Please contact us if you have any questions regarding these results.

GRL Engineers, Inc.

Travis Coleman, P.E

Harry Weintraub

.

Attachments: Appendix A (pages 3 – 14)
CAPWAP Analysis Results (pages 15 - 39)
Coleman PDCA Certificate (page 40)

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

1. BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during both design phase test programs as well as during production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore form an important part of a quality assurance program when deep foundations are constructed. Several dynamic pile testing methods exist. These methods have different benefits and limitations as well as different requirements for proper implementation.

The Case Method of dynamic pile testing, named after Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (e.g. a pile driving hammer or large drop weight) impacts the pile or shaft top such that a small permanent set is achieved. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® System (PDA).

The Case Method provides a simple closed-form solution for bearing capacity assessment. However, a more rigorous signal matching analysis method, CAPWAP® offers a more rigorous analysis of the dynamic test records than the Case Method solution and is therefore state-of-practice for final evaluation of the data to assess bearing capacity. A somewhat less rigorous signal matching analysis, called iCAP®, can be performed in real time on a construction site. However, iCAP results have not been as thoroughly correlated with static load test results as has been done with CAPWAP results. Therefore, iCAP results still require review by experienced testing and analysis engineers.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the "High Strain Test Method" of dynamic pile monitoring and dynamic load testing as standardized in ASTM D4945. Reference will also be made to the Rapid Load Test (or Force Pulse Test) as described in ASTM D7383. For completeness, three methods for deep foundation integrity assessments; the Pile Integrity Test™ (PIT), Cross Hole Sonic Logging with the Cross Hole Analyzer (CHA), and Thermal Integrity Profiling (TIP) are also discussed in Section 3.

2. RESULTS FROM PDA DYNAMIC TESTING

The primary objectives of high strain dynamic pile testing are either:

- *Dynamic Pile Monitoring, or*
- *Dynamic Load Testing*

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both drilled shafts and impact driven piles during restrike. With sufficient ram weight and impact cushioning, the duration of the dynamic load test force pulse can be lengthened such that a dynamic load test can satisfy Rapid Load Test requirements.

2.1 DYNAMIC PILE MONITORING

During pile installation, the sensors attached to the pile measure force and velocity near the pile top. A PDA provides signal conditioning, processes these signals, and calculates or evaluates by the Case Method:

- ***Bearing capacity*** at the time of testing, including an assessment of resistance distribution which is usually then related to blow count. This information supports formulation of a driving criterion.

- ***Dynamic pile stresses*** in both tension and compression, axial and averaged over the pile cross section, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- ***Pile integrity*** assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence for subsequently driven piles.

- ***Hammer performance*** parameters including the energy transferred to the pile, the hammer operating rate in blows per minute and the stroke of open ended diesel hammers

2.2 DYNAMIC PILE LOAD TESTING

Bearing capacity testing of either driven piles or drilled shafts (or bored piles and augercast piles) employs the basic measurement approach of dynamic pile monitoring. However, the test is often done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then, for sufficient soil resistance activation, its weight should be at least 1% of the test load for rock socketed piles and at least 2% for piles founded in gravelly materials. As an example, the ram weight should be at least 5 tons in favorable conditions and 10 tons in more energy absorbing soil conditions for a 500 ton test load. Ram weights larger than the minimum are acceptable. To satisfy rapid load test requirements, a ram weight of at least 5% of the test load is needed (e.g. minimum 25 ton ram for 500 ton test load).

For a successful test, it is most important that the test be conducted after a sufficient waiting time following pile installation so that soil strength properties approach their long term condition or in the case of cast-in-place concrete foundations that the concrete achieve sufficient strength and maturity. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain stresses within specified limits and for sufficient resistance activation. For dynamic load testing of drilled shafts, transferred energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles in sensitive soils require a warm pile hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with stress control and sufficient energy for resistance mobilization, the CAPWAP analysis provides the following results:

- *Bearing capacity i.e.* the mobilized capacity present at the time of testing

- *Resistance distribution* including shaft resistance and end bearing components

- *Stresses in pile or shaft* calculated at each point along the shaft for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or non-uniform contact stresses, e.g. when the pile toe is on uneven rock.

- *Shaft impedance vs. depth*; this is an estimate of the shaft shape if it differs substantially from the planned profile

- *Dynamic soil parameters* for shaft and toe, i.e. damping factors and quakes (quakes are related to the dynamic stiffness of the resistance at the pile/soil interface.)

3. FIELD MEASUREMENTS

The following is a general summary of dynamic measurements available to solve typical deep foundation problems.

3.1 PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed-form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below. Additional test details and procedures are described in ASTM D4945.

3.2 HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance

Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC.

3.3 SAXIMETER™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

3.4 PIT

The Pile Integrity Tester™ (PIT) helps in detecting major defects in concrete piles or shafts or in assessing the length of a variety of deep foundations, except steel piles. PIT performs the "Pulse-Echo Method" which only requires the measurement of motion (e.g., acceleration) at the pile top caused by a light hammer impact. PIT also supports the "Transient Response Method" which requires the additional measurement of the hammer force and an analysis in the frequency domain. PIT may also be used to evaluate the unknown length of deep foundations under existing structures. Additional test details and procedures are described in ASTM D5882.

3.5 CHA

This test requires that at least two tubes (typically steel tubes of at least 1.5 inch or 38 mm inside diameter) are installed vertically around the reinforcing cage in the shaft to be tested. A high frequency signal is generated in one of the water filled tubes and received in the other tube. The received signal strength and its First Arrival Time (FAT) yield important information about the concrete quality between the two tubes. The transmitting and recording of the signal is repeated typically every 2 inches or 50 mm starting at the shaft bottom and all records together establish a log or profile of the concrete quality between the two tubes and inside the reinforcing cage. The total number of tubes installed depends on the diameter of the drilled shaft. Generally one tube is installed for each foot (0.3 m) of shaft diameter. More tubes create more profiles for anomaly evaluation and delineation, if needed. Additional test details and procedures are described in ASTM D6760.

3.6 TIP

Thermal Integrity Profiling (TIP) can be used to assess the integrity, concrete cover, and concrete quality of concrete filled deep foundation elements by measuring the concrete temperature resulting from the heat of hydration. The test can be performed using Thermal Wire® cables embedded in the concrete or using Thermal Probes in access tubes similar to CHA. Analyzing the temperature vs. depth information leads to a 3-D pile volume image, including outside the reinforcing cage. Under favorable conditions, the volume vs depth information thus generated can be helpful when analyzing with CAPWAP the high strain records taken on cast-in-situ piles. Additional test details and procedures are described in ASTM D7949.

3.7 PIR-A

The Pile Installation Recorder for augered-cast-in-place (ACIP) or Continuous Flight Auger (CFA) piles, as a minimum, measures the amount of concrete or grout installed in the soil as a function of depth. As for the TIP results, under favorable conditions, the volume vs depth information thus generated can be helpful when analyzing with CAPWAP the high strain records taken on cast-in-situ piles.

4. ANALYTICAL SOLUTIONS

4.1 *BEARING CAPACITY*

4.1.1 WAVE EQUATION

The GRLWEAP program calculates a relationship between bearing capacity, pile stress, hammer stroke, and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph estimates a corresponding bearing capacity. This approach requires no field measurements other than blow count. However, it does require an accurate knowledge of the various parameters describing hammer, driving system, pile and soil. The wave equation is also very useful during the design stage of a project for the selection of hammer, cushion and pile size. Another option is the driveability analysis which predicts the blow count versus depth for a given hammer, pile and soil profile.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (Figure 1) is often performed by inputting the PDA and CAPWAP calculated parameters. With many of the dynamic parameters verified by the dynamic tests, the RWEA offers a more reliable basis for a safe and sufficient driving criterion.

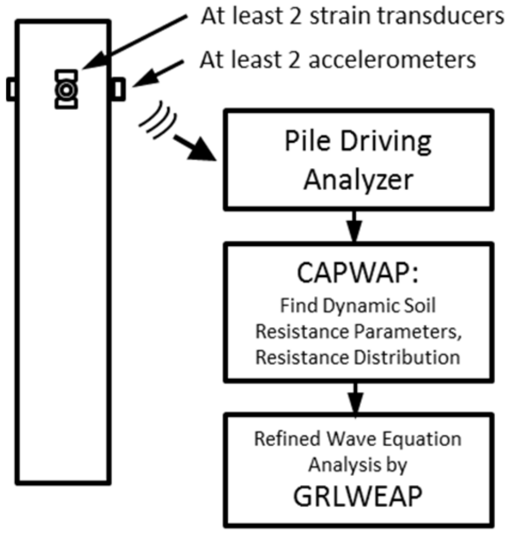


Figure 1. Block Diagram of Refined Wave Equation Analysis

4.1.2 CASE METHOD

The Case Method is a closed-form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force, F(t), and pile top velocity, v(t), the total soil resistance is

$$R(t) = ½\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \qquad (1)$$

where

t = a point in time after impact
t_2 = time t + 2L/c
L = pile length below gages
$c = (E/\rho)^{½}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile ($\rho\ c^2$)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \qquad (2)$$

The dynamic component may be computed from a soil damping factor, J, and the calculated pile toe velocity, $v_{toe}(t)$. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \qquad (3)$$

and, finally, to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 could be evaluated. Most commonly, t is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. Higher values are possible and lead to more conservative results. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used except when a correction is added as a result of "early unloading") requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity method, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed for one record. The capacities for other hammer blows are then quickly calculated for the thus selected Case Method and its associated damping factor.

The static resistance calculated by either Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set (permanent net displacement) has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The estimated static end bearing, EBR, is then calculated from the estimated static capacity and the shaft resistance estimate SFR.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDIPLOT program.

4.1.3 iCAP

iCAP is a signal matching program that works in parallel with the PDA software. iCAP allows signal matching based capacity assessments during data collection and/or data review for driven piles of known uniform geometry. iCAP performs a completely automatic signal match procedure, similar to the one available in the CAPWAP® program, but using faster algorithms. Depending on the blow rate of the hammer, and the level of iCAP computation, iCAP results will be a few blows behind the current PDA installation data. The following numeric results are available for each iCAP analyzed blow:

- RUC – total capacity by iCAP matching
- SFC – shaft resistance computed by iCAP
- EBC – end bearing computed by iCAP
- CSC – maximum compression stress
- BSC – max bottom compression stress
- TSC – maximum tension stress
- JC - correlating Case damping factor
- MQ - iCAP match quality

Since iCAP is fully automated, non-uniform piles, piles with (even minor) damage, concrete piles with minor cracking, or piles with uncertain properties cannot accurately be analyzed by iCAP. Larger open-end pipes (due to internal plug movements) or piles in unusual soils may pose extra difficulties. Also, the program only performs a limited data quality check. In addition, and as mentioned earlier, the iCAP signal matching procedure is not as thorough as what is done by CAPWAP and differences in results from these two types of signal matching analyses must be expected. Only CAPWAP has been extensively correlated with static load test results. A responsible engineer will therefore check the iCAP results thoroughly and compare them with CAPWAP, at least on a spot check basis, to determine reliable test results.

4.1.4 CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffness “quake” values. The method iteratively calculates a number of unknowns by signal matching.

While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program uses actual the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters based on the dynamic measurements. As a by-product, CAPWAP calculates tension and compression stresses along the length and provides a simulated static load test graph.

4.1.5 Capacity of damaged piles

Occasionally piles are damaged during driving and such damage may be indicated in the PDA collected records if it occurs below the sensor location. Damage on steel piles is often a broken splice, a collapsed pile bottom, a ripped of flange on an H-pile or a sharp bend (a very gradual dog leg is usually not recognized in the records). For concrete piles, among the problems encountered are cracks perpendicular to the pile axis, which deteriorate into a major damage, slabbing (loss of concrete cover) or a compressive failure at the bottom which in effect makes the pile shorter.

Damaged piles, with **BTA** values less than 0.8 should never be evaluated for bearing capacity by the Case Method or iCAP alone> Damaged piles are non-uniform piles which therefore violate the basic premise of the Case Method: a uniform, elastic pile. BTA is discussed more in Section 4.3.

Using the CAPWAP program, it is sometimes possible to obtain a reasonable match between computed and measured pile top quantities. In such an analysis the damaged section has to be modeled either by impedance reductions or by slacks. For piles with severe damage along their length it may be necessary to analyze a short pile. It should be born in mind, however, that such an analysis also violates the basic principles of the CAPWAP analysis, namely that the pile is elastic. Also, the nature of the damage is never known with certainty. For example, a broken splice could be a cracked weld either with the neighboring sections lining up well or shifted laterally. In the former case the compression stresses would be similar to those in the undamaged pile; in the latter situation, high stress concentrations would develop. In either case uplift is then uncertain or nonexistent. A sharp bend or toe damage present equally unpredictable situations under sustained loads which may cause further structural deterioration. If a short pile is analyzed then the lower section of the pile below the damage may offer unreliable end bearing and therefore should be discounted.

It is GRL's position that damaged piling should be replaced. Utilizing the CAPWAP calculated capacities should only be done after a very careful consideration of the effects of a loss of the foundation member while in service. Under no circumstances should the CAPWAP calculated capacity be utilized in the same manner in which the capacity of an undamaged pile be used. Under the best of circumstances the capacity should be used with an increased factor of safety and discounting all questionable capacity components. This evaluation cannot be made by GRL as it involves consideration of the type of structure, its seismic environment, the nature of the loads expected, the corrosiveness of the soil material, considerations of scour on the shortened pile, etc.

4.2 *STRESSES*

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from an individual strain transducer, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX, and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance, R(t), minus half the total shaft resistance, SFT. Again, for toe stress estimation, uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum net tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress, again averaged over the cross section and therefore not including bending stresses, can be computed from the pile top measurements by finding the maximum tension force in either traveling upward, $W_{ut,max}$, or downward, $W_{dt,max}$ waves and reducing it by the minimum compressive wave, $W_{oc,min}$, traveling in opposite direction, within the adjoining 2L/c period. The forces in the upward and downward waves can be calculated from the pile top measurements F(t) and v(t) from

$$W_u = ½[F(t) - Zv(t)] \qquad (4a)$$

$$W_d = ½[F(t) + Zv(t)] \qquad (4b)$$

The maximum tension due to an upward tension wave force Wu,t force is then

$$TSX = \max \begin{cases} W_{dt,max} - W_{oc,min} \\ W_{ut,max} - W_{oc,min} \end{cases} \qquad (5)$$

The simplified iCAP signal matching routine also calculates tensile and compressive stresses along the pile and, if it achieves a satisfactory signal match, more accurately than the PDA closed-form solution. iCAP calculated stresses from signal matching include **CSC** the maximum compression stress anywhere below the gage location, **BSC** the bottom (toe) compression stress, and **TSC** the maximum tension stress below the gage location. For non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA as well as the simplified signal matching results of iCAP may be in error. For piles with joints, cracks, or other discontinuities, CAPWAP provides the best analysis method for tensile and compressive stresses along the pile length.

4.3 *PILE INTEGRITY BY PDA*

Stress waves in a pile are reflected wherever the pile impedance, Z = EA/c = ρcA = A √(E ρ), changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the local relative decrease of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta = (1 + \alpha)/(1 - \alpha) \qquad (6)$$

with

$$\alpha = W_{ut}/W_{di} \qquad (7)$$

W_{ut} is the upwards traveling reflection wave (negative) due to the damage.

W_{di} is the maximum downward traveling wave due to impact (compressive and thus positive).

Actually, the formula used by the PDA is more complex as it also includes terms reflecting the effect of the soil resistance above the damage location which reduces both impact wave and reflection.

In addition to the quantification of damage, the PDA software also calculates the length to damage, **LTD**, from the time at which the BTA value has been determined.

It can be shown that the BTA calculation is quite meaningful as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections. However, because of the overlapping of waves limitation of Equation 6, when it comes to damage reflections occurring near the toe then either the toe resistance or the reflection of the impact wave tend to obscure the true magnitude of the damage reflection. In that case it is, however, sufficient to know that damage has occurred near the toe which can be assessed from the fact that the toe reflection appears too early (the pile appears to be short). The PDA software in that case displays an LTT (length to toe damage) but with no corresponding BTA value.

When testing or reviewing records with indicated pile damage, a decision has to be made as to what constitutes a serious damage and what could be dismissed as minor. Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.8, and that the pile is essentially broken if BTA is less than 0.6. While there are many reason why this very simplified approach is not a true representation of the strength of the pile portion at and below the damage, it is often useful as a preliminary criterion. The location of damage below the pile top should also be considered by the engineer-or-record when evaluating the acceptability of a damaged pile.

4.4 *HAMMER PERFORMANCE BY PDA*

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_0^t F(t)v(t)\,dt \qquad (9a)$$

The maximum of the E(t) curve is called **EMX** by the PDA but is also often called **ENTHRU**, for example, in GRLWEAP**;** it is the most important information for an overall evaluation of the performance of a hammer and its driving system. **ENTHRU** or **EMX** allow for a classification of the hammer's performance when presented as the transfer ratio, **ETR**, also reflecting the global effectiveness.

$$ETR = EMX/E_R \qquad (9b)$$

where

E_R is the hammer manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^2 - h_L \qquad (10)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and friction losses during impact (usually 0.3 ft or 0.1 m).

4.5 *DETERMINATION OF WAVE SPEED*

An important facet of dynamic pile testing is an assessment of pile material properties. Since, in most cases, force is determined from strain by multiplication with elastic modulus, **E**, and cross sectional area, **A**, the dynamic elastic modulus has to be determined for practically all pile materials. Even steel may have wave speed variations of 1 or 2%. In general, the records measured by the PDA clearly indicate a pile toe reflection in early easy to moderate blow count conditions.. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \qquad (11)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \qquad (12)$$

Since the mass density of concrete or steel pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is then easily found from the thus measured wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static modulus and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from a PIT (Low Strain) test is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is non-uniform along the length then the wave speed c, according to Eq. 11, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c of the whole pile is lower than the wave speed at the pile top. It is therefore recommended to determine wave speed and E at the sensors in the beginning of pile driving and not adjust them when the average c changes during the pile installation.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption (e.g. previous experience with piles on site or by the same manufacturer) or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

5. DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the engineer performing PDA tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

5.1 PROPORTIONALITY

As long as there is only a wave traveling in one direction, as is the case during initial impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \qquad (13a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \qquad (13b)$$

or strain

$$\varepsilon = v / c \qquad (13c)$$

This means that the early portion of strain times wave speed must be equal to the pile top particle velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

5.2 ***NUMBER OF SENSORS***

Measurements are always taken at opposite sides of the pile so that the average force and velocity in the pile can be calculated. The velocities on the two sides of the pile are very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. In that case the averaging of the two strain signals does not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary and highly recommended to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top and for spiral welded piles with all strain sensors staying away from the welds a distance of a few centimeters or inches. On concrete piles it is critical to not place the strain transducer straddling a crack.

6. LIMITATIONS, OTHER CONSIDERATIONS

6.1 MOBILIZATION OF CAPACITY

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

6.2 TIME DEPENDENT and RATE DEPENDENT SOIL RESISTANCE EFFECTS

Static pile capacity from dynamic method calculations provides an estimate of the axial pile capacity in compression. Increases and decreases in the pile capacity with time typically occur as a result of soil setup or relaxation. Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

6.2.1 SOIL SETUP

Because excess positive pore pressures often develop during pile driving in fine grained soils (clays, silts or even fine sands), the capacity of a pile at the time of driving is often less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile shaft, thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, effective stresses increase and the soil resistance and hence axial pile capacity acting on the pile increases. This phenomena is routinely called soil setup or soil freeze. There are numerous other reasons for soil setup such as realignment of clay particles, arching that reduces effective stresses during pile installation in very dense sands, soil fatigue in over-consolidated clays but also in very dense sands, etc.

6.2.2 RELAXATION

Relaxation, which is capacity reduction with time, has been observed for piles driven into weathered shale, and may take several days to fully develop. Where relaxation occurs, pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically with particular emphasis on the first few “high energy” blows. Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. In general, relaxation occurs at the pile toe and is therefore relevant for end bearing piles. Restrike tests should be performed and compared with the records from early restrike blows in order to avoid dangerous overpredictions.

6.2.3 RATE EFFECTS

The CAPWAP soil model assesses rate effects (elevated resistance caused by a non-zero pile velocity) by identifying the velocity dependent resistance components (static resistance is total resistance minus damping factor times pile velocity). For certain highly plastic soils, however, experience has shown that additional rate effects exist. It is therefore recommended that at least one static test is performed in fine grained materials where no experience exists with the dynamic soil behavior. High unit end bearing in highly plastic soils should be viewed with caution.

6.3 CAPACITY RESULTS FOR OPEN PILE PROFILES

Open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions. The plug behavior may also be quite different for cohesive and non-cohesive materials.

6.4 CAPWAP ANALYSIS RESULTS

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the signal match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations. Further, uplift estimates from dynamic testing should be coupled with higher factors of safety and, for short piles, the shaft resistance may behave very differently and often be considerably smaller in uplift.

6.5 STRESSES

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or nonuniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90%	of yield the steel strength for steel piles
85%	of the concrete compressive strength - minus the effective prestress for concrete piles in compression
100%	of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension
70%	of the reinforcement strength for regularly reinforced concrete piles in tension
300%	of the static design allowable stress for Timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA or CAPWAP for other locations along the pile based on the pile top measurements. The above allowable stresses also apply to those calculated by wave equation.

6.6 ADDITIONAL DESIGN CONSIDERATIONS

Numerous factors have to be considered in pile foundation design. Some of these considerations include:

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements,
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,
- loss of shaft resistance due to scour or other effects,
- Liquefaction and seismic effects,
- loss of structural pile strength due to additional bending loads, buckling (the dynamic loads generally do not cause buckling even though they may exceed the buckling strength of the pile section), corrosion etc.,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

6.7 VIBRATIONS

In certain situations, pile driving can cause ground vibrations and/or vibration induced soil settlements that may adversely impact nearby structures, utilities, facility equipment, etc. Standard industry practice is to perform a preconstruction survey of the neighboring area prior to the commencement of pile driving operations to identify and determine the condition of nearby structures, facilities, and utilities and their susceptibility to potential vibrations. If vibration susceptible concerns are identified, vibration monitoring equipment is used to measure vibration levels associated with the pile driving operations and those measurements are evaluated by a knowledgeable vibration specialist. Vibration monitoring is <u>not</u> a service offered by GRL Engineers. Therefore pile driving vibrations and their effects have not been considered in our analysis of the dynamic test results. Preconstruction surveys, monitoring and mitigating vibration effects are the responsibility of the owner, contractor, and design engineer.

6.8 WAVE EQUATION ANALYSIS RESULTS

The results calculated by the wave equation analysis program depend on a variety of assumptions of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

7. FACTORS OF SAFETY OR RESISTANCE FACTORS

Static or dynamic load tests run to failure yield an ultimate pile bearing capacity, R_{ult}. If this failure load were applied to the pile, then excessive settlements would occur. Therefore, in allowable stress designs it is absolutely necessary that the actually applied load, also often called the design load, R_d (or working load or safe load), is less than R_{ult}. In most soils it is necessary that R_{ult} is at least 50% higher than R_d to limit settlements. This means that

$$R_{ult} \geq 1.5\, R_d, \tag{13}$$

or the Factor of Safety has to be at least 1.5.

Unfortunately, neither applied loads nor R_{ult} are exactly known. One static load test may be performed at a site, but that would not guarantee that all other piles have the same capacity and it is to be expected that a certain percentage of the production piles have lower capacities, either due to soil variability or due to pile damage. Uncertainty also exists because different types of tests and their interpretations present different bearing capacity results for the same pile.

Not only bearing capacity values of all piles are unknown, even loads vary considerably and occasional overloads must be expected. We would not want a structure to become unserviceable or useless because of either an occasional overload or a few piles with low capacity. For this reason, and to avoid being overly conservative which would mean excessive cost, modern safety concepts suggest that the overall factor of safety should reflect both the uncertainty in loads and resistance. Thus, if all piles were tested statically and if we carefully controlled the loads, we probably could live with F.S. = 1.5. However, in general, depending on the building type or load combinations and as a function of quality assurance of pile foundations, a variety of Factors of Safety have been proposed.

For highway bridge loads in the United States, AASHTO allowable stress design guideline specifications proposed the following Factors of Safety (prior to 2007):

F.S. = 1.90 for static load test with wave equation and dynamic test.

F.S. = 2.25 for dynamic testing with wave equation analysis.

F.S. = 2.50 for indicator piles with wave equation analysis.

F.S. = 2.75 for wave equation analysis.

F.S. = 3.50 for FHWA Modified Gates dynamic formula.

It should be mentioned that all of these methods should always be combined with soil exploration and static pile analysis. Also, specifications are occasionally updated and therefore the latest version should be consulted for the current guidance on factors of safety.

Codes and specifications (in the United States for example IBC, PDCA, ASCE, or other specifications issued by State Departments of Transportation) specify different factors of safety. However, the range of recommended factors of safety in the US typically varies between 1.9 and 6.0 for ASD design.

In 2007, Load and Resistance Factor Design (LRFD) was mandated for highway bridge design and construction in the United States. In LRFD, the sum of the factored loads must be less than the nominal resistance, R_n, multiplied by a resistance factor, ϕ.

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \tag{14}$$

The 2014 AASHTO LRFD design specifications recommend the following resistance factors, ϕ_{dyn}, be applied to the nominal resistance based on the selected construction control procedures.

ϕ_{dyn} = 0.80 for driving criteria established by static load test of 1 pile per site condition and dynamic testing with signal matching of at least 2 piles per site condition but no less than 2% of production piles.

ϕ_{dyn} = 0.75 for driving criteria established by successful static load test of 1 pile per site condition without dynamic testing.

ϕ_{dyn} = 0.75 for driving criteria established by dynamic testing with signal matching conducted on 100% of production piles.

ϕ_{dyn} = 0.65 for driving criteria developed by dynamic testing with signal matching, quality control by dynamic testing on 2 piles per site condition, but no less than 2% of production piles.

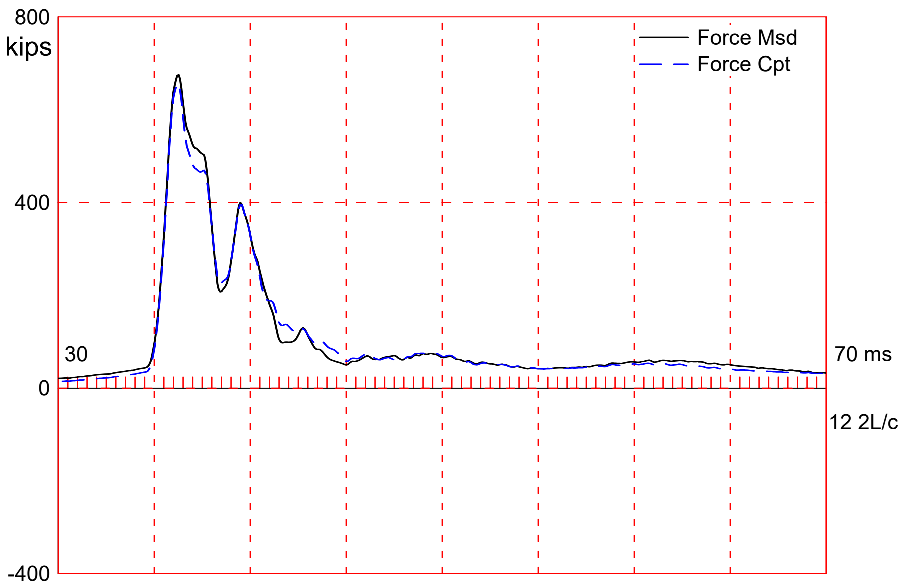
ϕ_{dyn} = 0.50 for wave equation analysis without dynamic measurements or load test but with field confirmation of hammer performance.

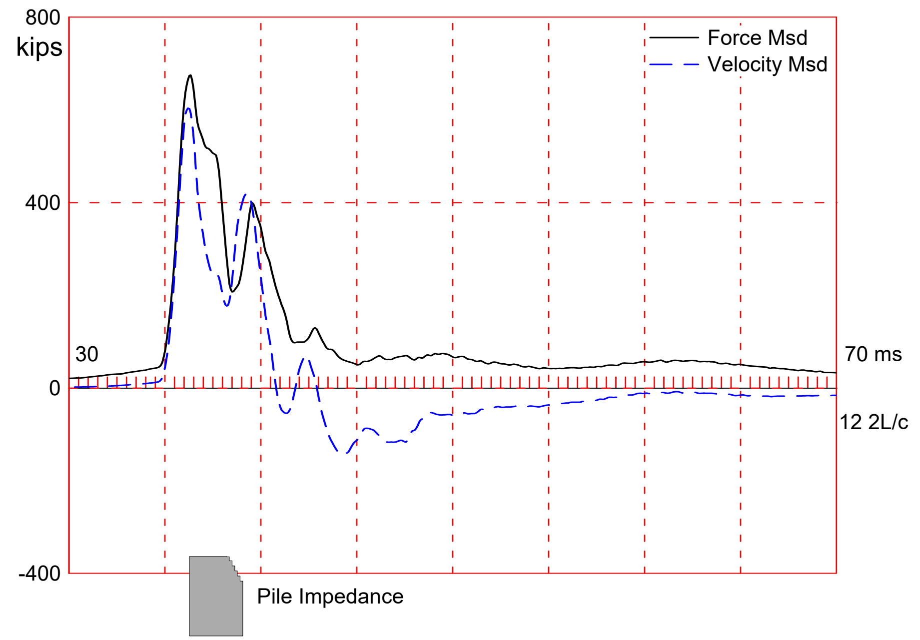
ϕ_{dyn} = 0.40 for FHWA modified Gates dynamic formula (end of drive condition only)

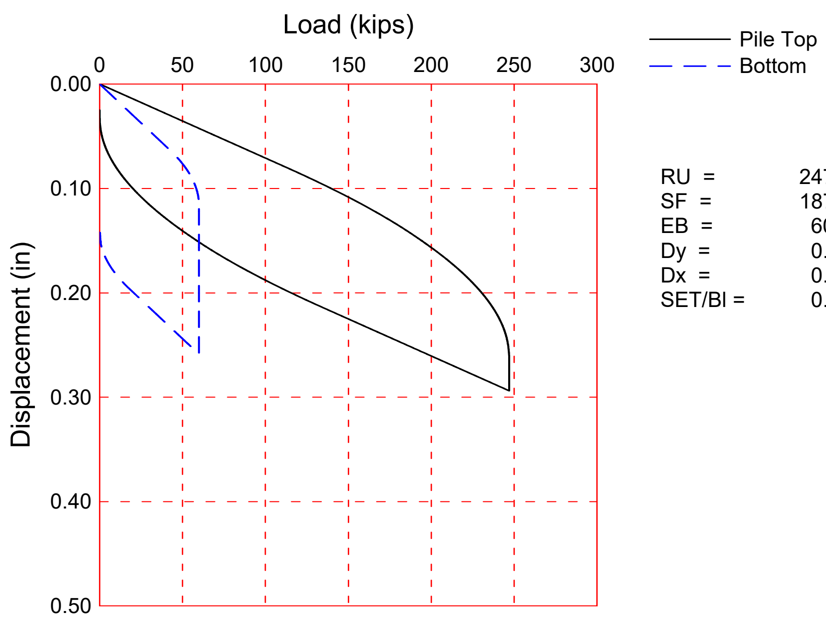
ϕ_{dyn} = 0.10 for Engineering News dynamic formula as defined in AASHTO 10.7.3.8.5 (end of drive conditions only)

In ASD, it is the designer's responsibility to identify the required ultimate capacity based on the design loads and the adopted factor of safety. Similarly in LRFD, it is the designer's responsibility to identify the required nominal resistance based on the factored loads and the construction control procedure and its resistance factor. The required factor of safety in ASD or resistance factor in LRFD should be included in the design drawings and specifications along with the testing requirements.

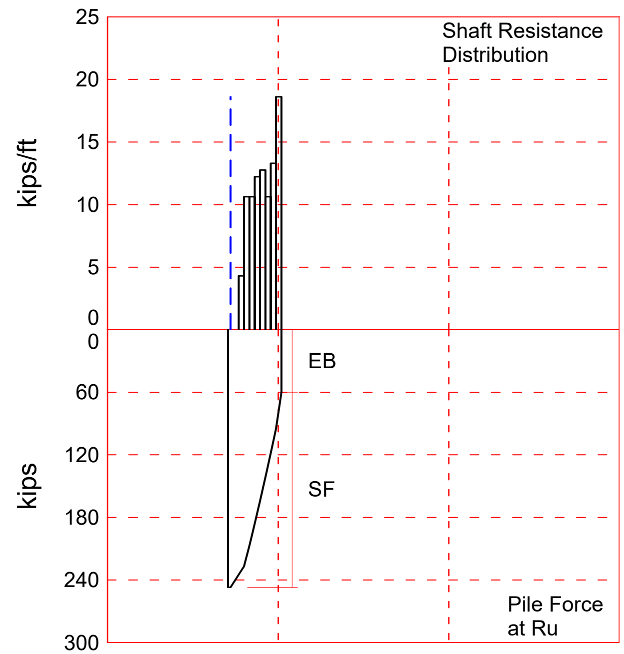
For optimal solutions it is always recommended that increased testing for lower ultimate pile capacities or reduced nominal resistances is considered. Frequent pile testing will also help reduce the confusion that often exists on construction sites as to foundation loads and bearing requirements. In any event, it cannot be expected that the test engineer is aware of and responsible for the variety of considerations that must be met for ASD or LRFD based foundation designs as well as to determine the appropriate factor of safety or resistance factor associated with the design.







RU	=	247.0 kips
SF	=	187.0 kips
EB	=	60.0 kips
Dy	=	0.27 in
Dx	=	0.29 in
SET/Bl	=	0.03 in



Length b. Sensors	18.8 ft
Embedment	17.8 ft
Top Area	212.1 in²
End Bearing Area	212.1 in²
Top Perimeter	4.42 ft
Top E-Modulus	5901 ksi
Top Spec. Weight	150.0 lb/ft³
Top Wave Spd.	13500 ft/s
Overall W.S.	13500 ft/s
Match Quality	1.82
Top Compr. Stress	3.1 ksi
Max Compr. Stress	3.3 ksi
Max Tension Stress	-0.04 ksi
Avg. Shaft Quake	0.18 in
Toe Quake	0.09 in
Avg. Shaft Smith Dpg.	0.32 s/ft
Toe Smith Damping	0.20 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal "match quality" (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 247.0; along Shaft 187.0; at Toe 60.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				247.0			
1	5.6	4.6	20.0	227.0	20.0	4.31	0.98
2	7.5	6.5	20.0	207.0	40.0	10.64	2.41
3	9.4	8.4	20.0	187.0	60.0	10.64	2.41
4	11.3	10.3	23.0	164.0	83.0	12.23	2.77
5	13.2	12.2	24.0	140.0	107.0	12.77	2.89
6	15.0	14.0	20.0	120.0	127.0	10.64	2.41
7	16.9	15.9	25.0	95.0	152.0	13.30	3.01
8	18.8	17.8	35.0	60.0	187.0	18.62	4.21
Avg. Shaft			23.4			10.51	2.38
Toe			60.0				40.74

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.32	0.20
Quake	(in)	0.18	0.09
Case Damping Factor		0.65	0.13
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	97	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	27	
Soil Plug Weight	(kips)	0.167	

```
CAPWAP match quality     =    1.82         (Wave Up Match) ; RSA = 0
Observed: Final Set      =    0.03 in;     Blow Count      =     480 b/ft
Computed: Final Set      =    0.06 in;     Blow Count      =     195 b/ft

max. Top Comp. Stress    =     3.1 ksi      (T=  36.3 ms, max= 1.071 x Top)
max. Comp. Stress        =     3.3 ksi      (Z=   5.6 ft, T=  36.8 ms)
max. Tens. Stress        =   -0.04 ksi      (Z=  17.9 ft, T=  45.5 ms)
max. Energy (EMX)        =     8.4 kip-ft;  max. Measured Top Displ. (DMX)=  0.23 in
```

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	0.9	660.9	-0.6	3.1	-0.00	8.4	6.6	0.25
2	1.9	668.2	-0.6	3.2	-0.00	8.4	6.6	0.25
3	2.8	677.8	-0.6	3.2	-0.00	8.4	6.5	0.25
4	3.8	688.1	-0.6	3.2	-0.00	8.4	6.4	0.24
5	4.7	698.1	-0.6	3.3	-0.00	8.3	6.3	0.24
6	5.6	707.7	-0.6	3.3	-0.00	8.3	6.1	0.24
7	6.6	670.9	-0.6	3.2	-0.00	7.6	6.0	0.24
8	7.5	680.0	-0.6	3.2	-0.00	7.6	5.9	0.24
9	8.5	643.3	-0.6	3.0	-0.00	6.9	5.8	0.24
10	9.4	649.2	-0.6	3.1	-0.00	6.9	5.8	0.24
11	10.3	607.4	-0.5	2.9	-0.00	6.2	5.8	0.24
12	11.3	602.4	-0.6	2.8	-0.00	6.2	5.8	0.24
13	12.2	543.4	-0.5	2.6	-0.00	5.4	6.0	0.24
14	13.2	528.4	-0.5	2.5	-0.00	5.4	6.3	0.23
15	14.1	455.4	-0.5	2.2	-0.00	4.6	6.6	0.23
16	15.0	430.4	-0.5	2.1	-0.00	4.6	7.0	0.23
17	16.0	355.8	-0.5	1.9	-0.00	3.9	7.4	0.23
18	16.9	320.6	-0.6	1.8	-0.00	3.9	7.7	0.23
19	17.9	219.4	-6.2	1.4	-0.04	3.0	7.8	0.23
20	18.8	230.0	-4.9	1.6	-0.03	1.7	7.7	0.23
Absolute	5.6			3.3			(T =	36.8 ms)
	17.9				-0.04		(T =	45.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	566.8	496.0	425.2	354.4	283.6	212.9	142.1	71.3	0.5	0.0
RX	576.0	505.6	436.5	374.4	345.8	317.8	291.5	272.2	257.3	243.4
RU	566.8	496.0	425.2	354.4	283.6	212.9	142.1	71.3	0.5	0.0

RAU = 185.7 (kips); RA2 = 395.3 (kips)

Current CAPWAP Ru = 247.0 (kips); Corresponding J(RP)= 0.45; J(RX) = 0.87

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
6.5	36.35	605.7	668.9	678.5	0.23	0.03	0.03	8.3	785.4	670

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.0	212.1	5900.5	150.000	4.42

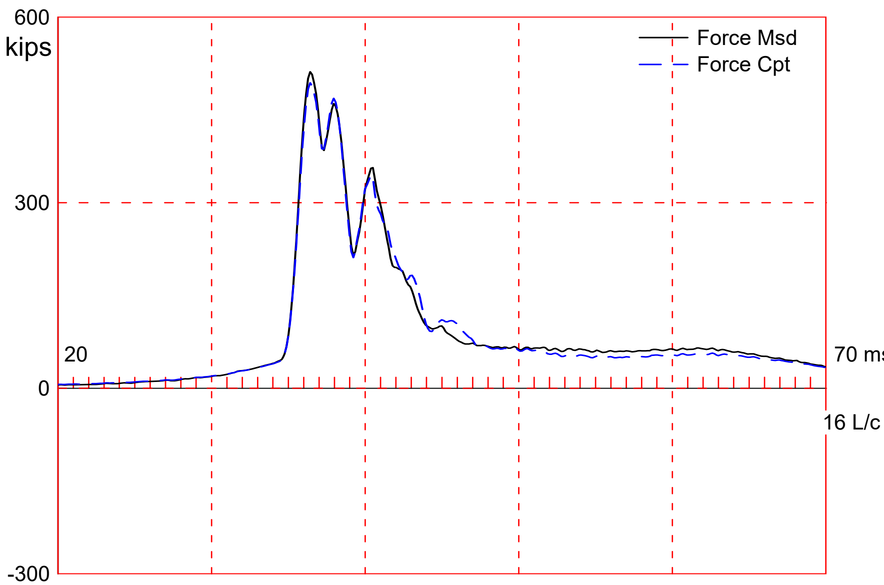
PILE PROFILE AND PILE MODEL

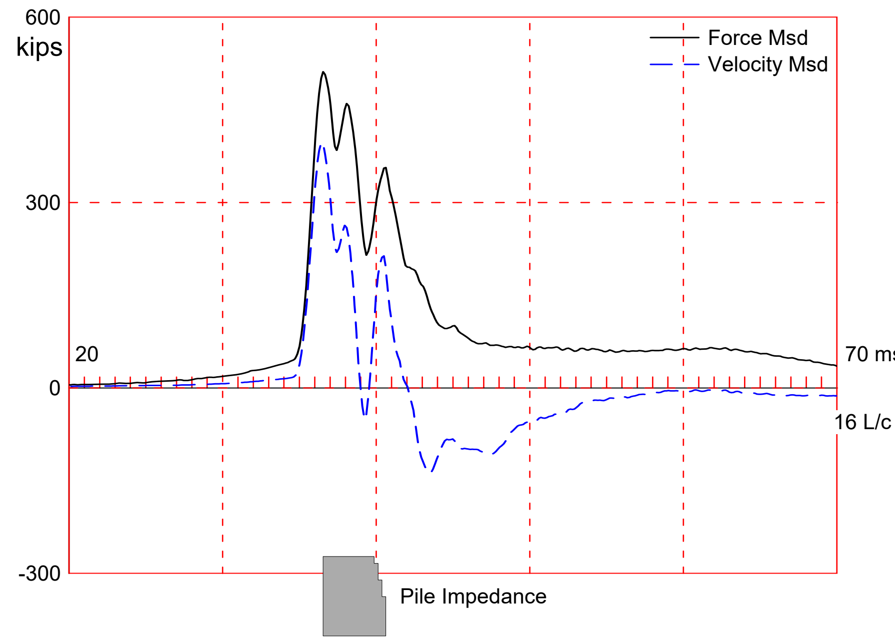
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
13.8	212.1	5900.5	150.000	4.42
18.8	139.7	5900.5	150.000	4.42

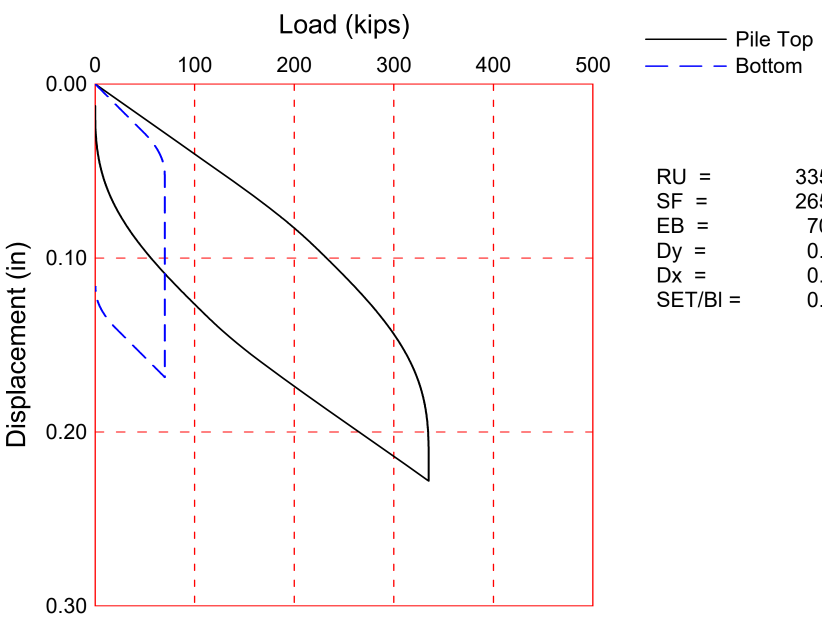
Toe Area 212.1 in^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Tension Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.	Wave Speed	Soil Plug
	ft	kips/ft/s	%	in		in		ft	ft/s	kips
1	0.9	92.69	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.000
15	14.1	92.39	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.000
16	15.0	87.82	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.000
17	16.0	81.88	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.016
18	16.9	75.93	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.042
19	17.9	69.98	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.055
20	18.8	64.04	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.055

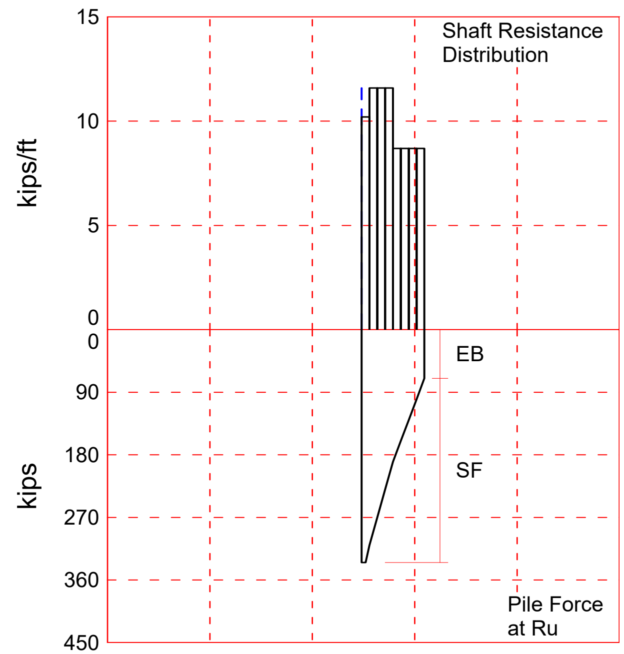
Wave Speed: Pile Top 13500.0, Elastic 13500.0, Overall 13500.0 ft/s
Pile Damping 2.00 %, Time Incr 0.070 ms, 2L/c 2.8 ms
Total volume: 26.432 ft^3; Volume ratio considering added impedance: 1.000







RU =	335.0 kips
SF =	265.0 kips
EB =	70.0 kips
Dy =	0.22 in
Dx =	0.23 in
SET/Bl =	0.01 in



Length b. Sensors	27.6 ft
Embedment	26.6 ft
Top Area	212.1 in^2
End Bearing Area	212.1 in^2
Top Perimeter	4.42 ft
Top E-Modulus	5901 ksi
Top Spec. Weight	150.0 lb/ft^3
Top Wave Spd.	13500 ft/s
Overall W.S.	13500 ft/s
Match Quality	2.40
Top Compr. Stress	2.4 ksi
Max Compr. Stress	2.4 ksi
Max Tension Stress	-0.04 ksi
Avg. Shaft Quake	0.12 in
Toe Quake	0.04 in
Avg. Shaft Smith Dpg.	0.30 s/ft
Toe Smith Damping	0.20 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

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Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 335.0; along Shaft 265.0; at Toe 70.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				335.0			
1	3.5	2.4	25.0	310.0	25.0	10.20	2.31
2	6.9	5.9	40.0	270.0	65.0	11.59	2.62
3	10.4	9.4	40.0	230.0	105.0	11.59	2.62
4	13.8	12.8	40.0	190.0	145.0	11.59	2.62
5	17.3	16.3	30.0	160.0	175.0	8.70	1.97
6	20.7	19.7	30.0	130.0	205.0	8.70	1.97
7	24.2	23.2	30.0	100.0	235.0	8.70	2.02
8	27.6	26.6	30.0	70.0	265.0	8.70	2.61
Avg. Shaft			33.1			9.96	2.34
Toe			70.0				47.53

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.30	0.20
Quake	(in)	0.12	0.04
Case Damping Factor		0.86	0.15
Damping Type		Viscous	Viscous
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	91	
Soil Plug Weight	(kips)	0.278	

```
CAPWAP match quality   =    2.40        (Wave Up Match) ; RSA = 0
Observed: Final Set    =    0.01 in;    Blow Count      =     960 b/ft
Computed: Final Set    =    0.02 in;    Blow Count      =     640 b/ft

max. Top Comp. Stress  =     2.4 ksi     (T=  36.7 ms, max= 1.027 x Top)
max. Comp. Stress      =     2.4 ksi     (Z=   3.5 ft, T=  36.8 ms)
max. Tens. Stress      =   -0.04 ksi     (Z=  25.9 ft, T=  45.5 ms)
max. Energy (EMX)      =     5.0 kip-ft;  max. Measured Top Displ. (DMX)=  0.17 in
```

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	1.7	505.8	-2.8	2.4	-0.01	5.0	4.4	0.17
2	3.5	519.3	-2.8	2.4	-0.01	5.0	4.2	0.16
3	5.2	490.3	-0.4	2.3	-0.00	4.5	4.1	0.16
4	6.9	503.6	-0.4	2.4	-0.00	4.5	3.9	0.15
5	8.6	453.2	-0.3	2.1	-0.00	3.8	3.8	0.15
6	10.4	464.8	-0.4	2.2	-0.00	3.8	3.7	0.15
7	12.1	416.9	-0.3	2.0	-0.00	3.2	3.5	0.14
8	13.8	426.6	-0.3	2.0	-0.00	3.1	3.4	0.14
9	15.5	379.0	-0.3	1.8	-0.00	2.6	3.3	0.13
10	17.3	381.7	-0.3	1.8	-0.00	2.5	3.3	0.13
11	19.0	336.7	-0.2	1.6	-0.00	2.1	3.4	0.13
12	20.7	324.7	-0.3	1.5	-0.00	2.1	3.6	0.13
13	22.4	262.2	-0.3	1.2	-0.00	1.7	3.9	0.13
14	24.2	237.2	-0.2	1.2	-0.00	1.7	4.1	0.13
15	25.9	174.1	-5.6	1.2	-0.04	1.3	4.2	0.13
16	27.6	177.0	-2.8	1.7	-0.03	1.0	4.1	0.12
Absolute	3.5			2.4			(T =	36.8 ms)
	25.9				-0.04		(T =	45.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	523.7	485.2	446.7	408.2	369.7	331.2	292.7	254.2	215.6	177.1
RX	543.6	507.8	472.0	436.2	400.4	364.5	328.7	308.1	293.5	284.8
RU	562.9	528.3	493.7	459.1	424.5	389.9	355.3	320.7	286.1	251.5

RAU = 277.6 (kips); RA2 = 445.0 (kips)

Current CAPWAP Ru = 335.0 (kips); Corresponding J(RP)= 0.49; J(RX) = 0.58

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
4.3	36.67	402.3	506.5	515.5	0.17	0.01	0.01	5.1	672.6	1750

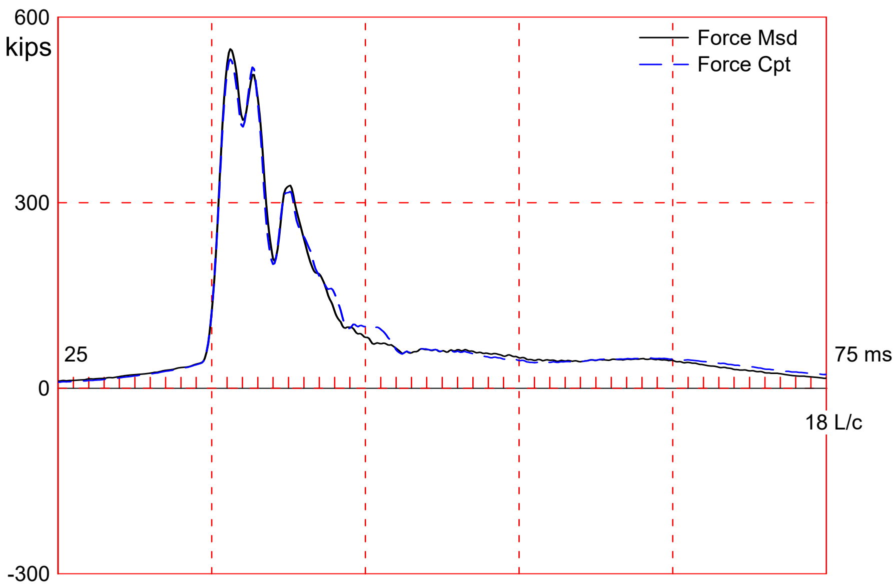
PILE PROFILE AND PILE MODEL

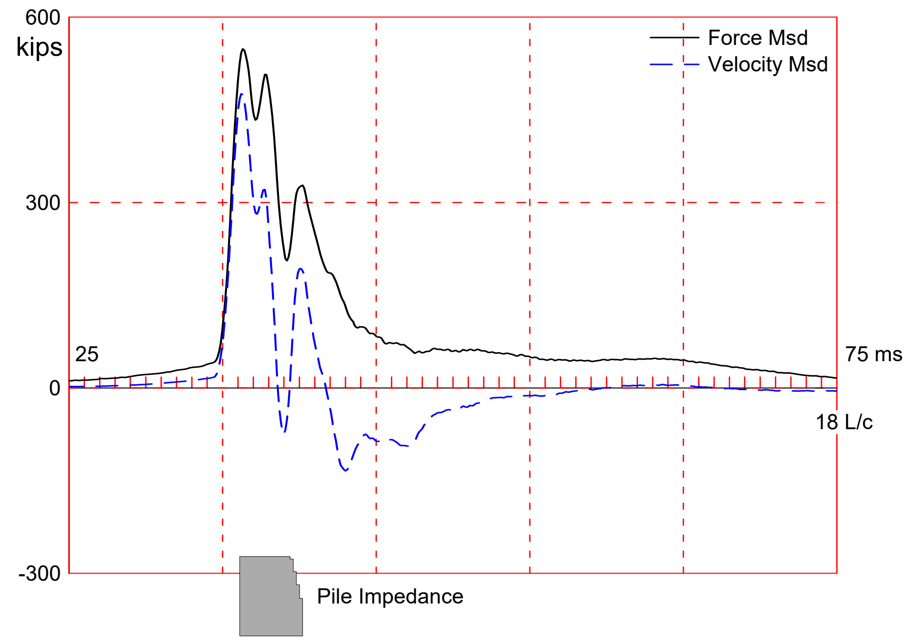
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.0	212.1	5900.5	150.000	4.42
22.6	212.1	5900.5	150.000	4.42
27.6	82.8	5900.5	150.000	2.76

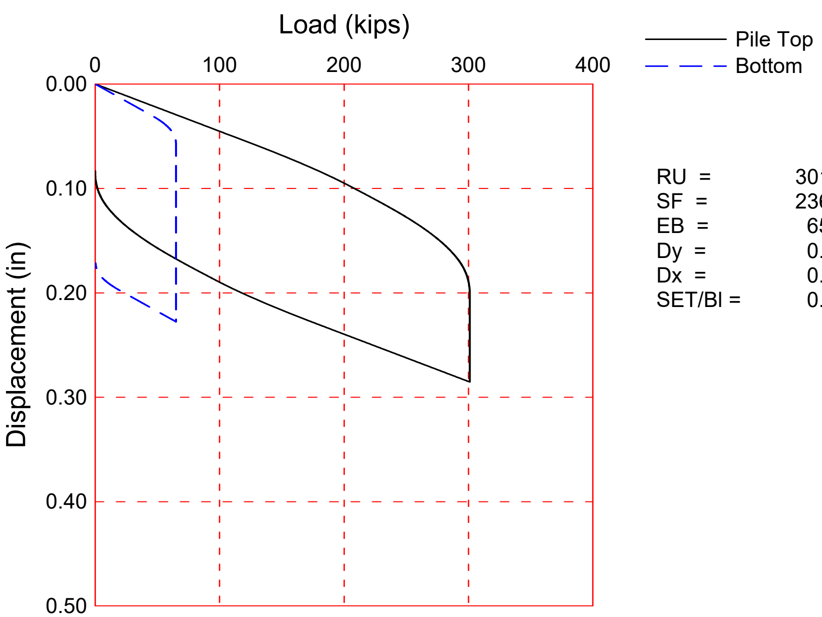
Toe Area 212.1 in^2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Wave Speed ft/s	Soil Plug kips
1	1.7	92.69	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.000
13	22.4	92.69	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.023
14	24.2	84.83	0.00	0.00	0.000	-0.00	0.000	4.19	13500.0	0.085
15	25.9	65.44	0.00	0.00	0.000	-0.00	0.000	3.62	13500.0	0.085
16	27.6	45.95	0.00	0.00	0.000	-0.00	0.000	3.05	13500.0	0.085

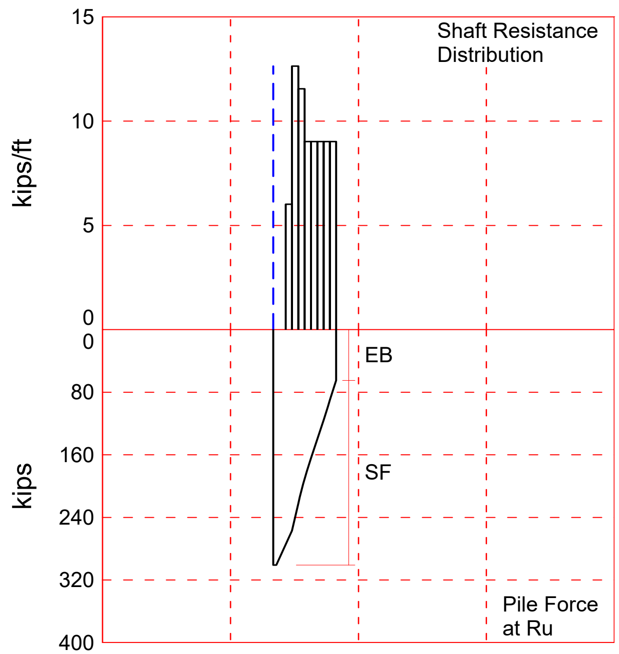
Wave Speed: Pile Top 13500.0, Elastic 13500.0, Overall 13500.0 ft/s
Pile Damping 2.00 %, Time Incr 0.128 ms, 2L/c 4.1 ms
Total volume: 38.405 ft^3; Volume ratio considering added impedance: 1.000







RU	=	301.0 kips
SF	=	236.0 kips
EB	=	65.0 kips
Dy	=	0.20 in
Dx	=	0.29 in
SET/Bl	=	0.08 in



Length b. Sensors	27.7 ft
Embedment	26.7 ft
Top Area	212.1 in^2
End Bearing Area	212.1 in^2
Top Perimeter	4.42 ft
Top E-Modulus	5901 ksi
Top Spec. Weight	150.0 lb/ft^3
Top Wave Spd.	13500 ft/s
Overall W.S.	13500 ft/s
Match Quality	2.36
Top Compr. Stress	2.5 ksi
Max Compr. Stress	2.9 ksi
Max Tension Stress	-0.05 ksi
Avg. Shaft Quake	0.12 in
Toe Quake	0.04 in
Avg. Shaft Smith Dpg.	0.34 s/ft
Toe Smith Damping	0.28 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 301.0; along Shaft 236.0; at Toe 65.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Quake in
				301.0				
1	8.3	7.3	44.0	257.0	44.0	6.02	1.36	0.12
2	11.1	10.1	35.0	222.0	79.0	12.64	2.86	0.12
3	13.9	12.9	32.0	190.0	111.0	11.55	2.61	0.12
4	16.6	15.6	25.0	165.0	136.0	9.03	2.04	0.12
5	19.4	18.4	25.0	140.0	161.0	9.03	2.04	0.12
6	22.2	21.2	25.0	115.0	186.0	9.03	2.04	0.12
7	24.9	23.9	25.0	90.0	211.0	9.03	2.19	0.11
8	27.7	26.7	25.0	65.0	236.0	9.03	2.80	0.11
Avg. Shaft			29.5			8.84	2.07	0.12
Toe			65.0				44.13	0.04

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.34	0.28
Case Damping Factor		0.87	0.20
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	80	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	50	
Resistance Gap (included in Toe Quake) (in)			0.00
Soil Plug Weight	(kips)	0.135	

CAPWAP match quality = 2.36 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.08 in; Blow Count = 144 b/ft
Computed: Final Set = 0.06 in; Blow Count = 211 b/ft

max. Top Comp. Stress = 2.5 ksi (T= 36.4 ms, max= 1.124 x Top)
max. Comp. Stress = 2.9 ksi (Z= 8.3 ft, T= 37.0 ms)
max. Tens. Stress = -0.05 ksi (Z= 27.7 ft, T= 49.1 ms)
max. Energy (EMX) = 6.0 kip-ft; max. Measured Top Displ. (DMX)= 0.18 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	1.4	538.9	-4.6	2.5	-0.02	6.0	5.2	0.18
2	2.8	548.5	-4.6	2.6	-0.02	6.0	5.1	0.18
3	4.2	562.6	-4.6	2.7	-0.02	5.9	5.0	0.18
4	5.5	577.9	-4.7	2.7	-0.02	5.9	4.9	0.18
5	6.9	592.8	-4.7	2.8	-0.02	5.9	4.7	0.17
6	8.3	605.8	-4.8	2.9	-0.02	5.8	4.5	0.17
7	9.7	531.8	0.0	2.5	0.00	4.9	4.4	0.16
8	11.1	542.6	0.0	2.6	0.00	4.9	4.3	0.16
9	12.5	487.4	0.0	2.3	0.00	4.2	4.2	0.16
10	13.9	495.7	0.0	2.3	0.00	4.1	4.1	0.15
11	15.2	445.2	-0.0	2.1	-0.00	3.5	4.0	0.15
12	16.6	447.9	-0.0	2.1	-0.00	3.5	4.0	0.15
13	18.0	401.6	0.0	1.9	0.00	3.0	4.0	0.15
14	19.4	393.5	0.0	1.9	0.00	3.0	4.2	0.15
15	20.8	333.5	-0.0	1.6	-0.00	2.6	4.4	0.15
16	22.2	313.7	-0.1	1.5	-0.00	2.6	4.6	0.15
17	23.5	247.1	-0.2	1.2	-0.00	2.1	4.7	0.14
18	24.9	235.9	-0.2	1.4	-0.00	2.1	4.8	0.14
19	26.3	196.3	-4.5	1.4	-0.03	1.6	4.7	0.14
20	27.7	200.5	-5.0	2.0	-0.05	1.2	4.6	0.14
Absolute	8.3			2.9			(T =	37.0 ms)
	27.7				-0.05		(T =	49.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	581.1	536.9	492.7	448.5	404.3	360.1	316.0	271.8	227.6	183.4
RX	592.1	549.1	506.1	463.1	420.1	377.1	348.9	323.9	301.6	291.1
RU	661.1	624.9	588.7	552.5	516.3	480.2	444.0	407.8	371.6	335.4

RAU = 256.7 (kips); RA2 = 468.2 (kips)

Current CAPWAP Ru = 301.0 (kips); Corresponding J(RP)= 0.63; J(RX) = 0.81

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
5.2	36.22	477.9	545.0	552.2	0.18	0.09	0.08	6.0	553.6	1625

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.0	212.1	5900.5	150.000	4.42

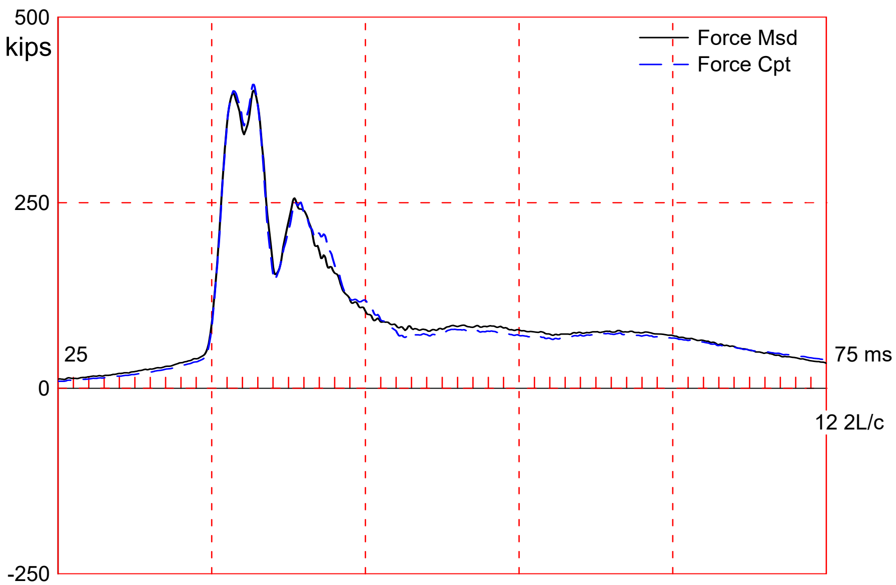
PILE PROFILE AND PILE MODEL

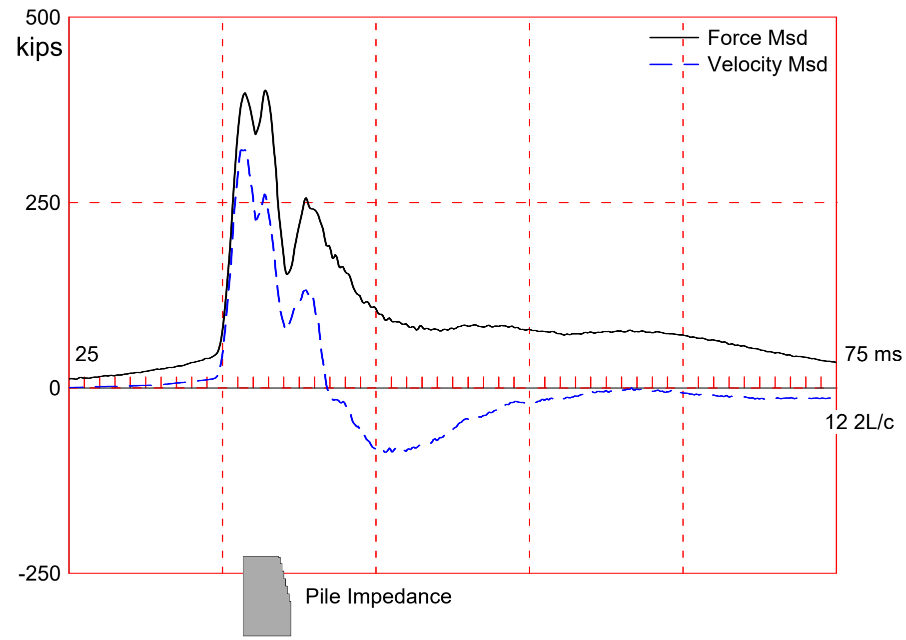
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
22.7	212.1	5900.5	150.000	4.42
27.7	82.8	5900.5	150.000	2.76

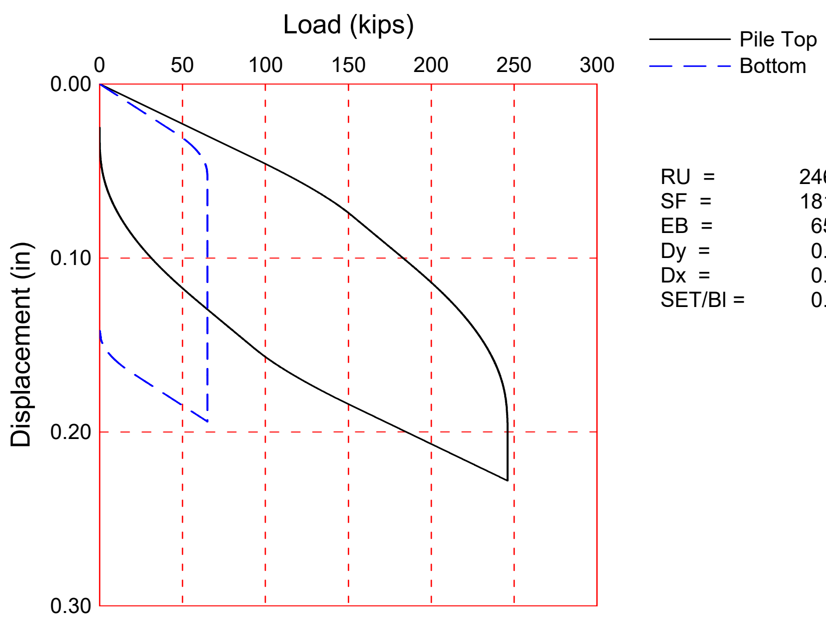
Toe Area 212.1 in^2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Wave Speed ft/s	Soil Plug kips
1	1.4	92.69	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0	0.000
17	23.5	89.78	0.00	0.00	0.000	-0.00	0.000	4.33	13500.0	0.000
18	24.9	75.32	0.00	0.00	0.000	-0.00	0.000	3.91	13500.0	0.015
19	26.3	59.68	0.00	0.00	0.000	-0.00	0.000	3.45	13500.0	0.055
20	27.7	44.03	0.00	0.00	0.000	-0.00	0.000	2.99	13500.0	0.065

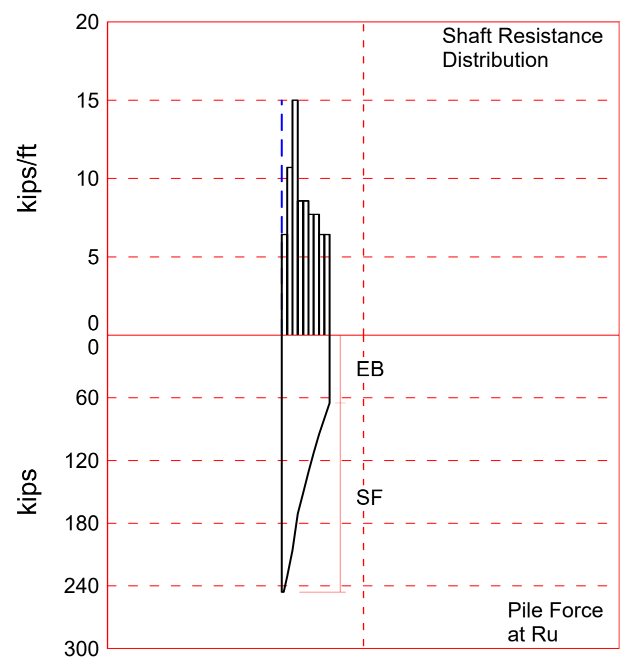
Wave Speed: Pile Top 13500.0, Elastic 13500.0, Overall 13500.0 ft/s
Pile Damping 2.00 %, Time Incr 0.103 ms, 2L/c 4.1 ms
Total volume: 38.552 ft^3; Volume ratio considering added impedance: 1.000







RU	=	246.0 kips
SF	=	181.0 kips
EB	=	65.0 kips
Dy	=	0.20 in
Dx	=	0.23 in
SET/Bl	=	0.03 in



Length b. Sensors	21.0 ft
Embedment	21.0 ft
Top Area	212.1 in^2
End Bearing Area	212.1 in^2
Top Perimeter	4.42 ft
Top E-Modulus	5901 ksi
Top Spec. Weight	150.0 lb/ft^3
Top Wave Spd.	13500 ft/s
Overall W.S.	13500 ft/s
Match Quality	2.80
Top Compr. Stress	1.9 ksi
Max Compr. Stress	2.0 ksi
Max Tension Stress	-0.05 ksi
Avg. Shaft Quake	0.13 in
Toe Quake	0.04 in
Avg. Shaft Smith Dpg.	0.33 s/ft
Toe Smith Damping	0.22 s/ft

About the CAPWAP Results

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The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 246.0; along Shaft 181.0; at Toe 65.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				246.0			
1	2.3	2.3	15.0	231.0	15.0	6.43	1.46
2	4.7	4.7	25.0	206.0	40.0	10.71	2.43
3	7.0	7.0	35.0	171.0	75.0	15.00	3.40
4	9.3	9.3	20.0	151.0	95.0	8.57	1.94
5	11.7	11.7	20.0	131.0	115.0	8.57	1.94
6	14.0	14.0	18.0	113.0	133.0	7.71	1.75
7	16.3	16.3	18.0	95.0	151.0	7.71	1.75
8	18.7	18.7	15.0	80.0	166.0	6.43	1.64
9	21.0	21.0	15.0	65.0	181.0	6.43	2.04
Avg. Shaft			20.1			8.62	2.04
Toe			65.0				44.13

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.33	0.22
Quake	(in)	0.13	0.04
Case Damping Factor		0.64	0.15
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	28	
Soil Plug Weight	(kips)		0.226

CAPWAP match quality = 2.80 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.03 in; Blow Count = 480 b/ft
Computed: Final Set = 0.01 in; Blow Count = 954 b/ft

max. Top Comp. Stress = 1.9 ksi (T= 37.8 ms, max= 1.017 x Top)
max. Comp. Stress = 2.0 ksi (Z= 2.3 ft, T= 36.7 ms)
max. Tens. Stress = -0.05 ksi (Z= 21.0 ft, T= 47.2 ms)
max. Energy (EMX) = 3.8 kip-ft; max. Measured Top Displ. (DMX)= 0.15 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	0.8	409.0	-4.2	1.9	-0.02	3.8	3.4	0.16
2	1.6	410.7	-4.2	1.9	-0.02	3.8	3.4	0.16
4	3.1	398.4	-2.5	1.9	-0.01	3.5	3.3	0.16
6	4.7	407.4	-2.6	1.9	-0.01	3.5	3.1	0.15
8	6.2	379.3	-0.7	1.8	-0.00	3.1	3.1	0.15
10	7.8	336.3	-0.2	1.6	-0.00	2.5	3.2	0.15
12	9.3	336.0	-0.1	1.6	-0.00	2.5	3.2	0.14
13	10.1	308.3	-0.1	1.5	-0.00	2.2	3.2	0.14
14	10.9	306.2	-0.2	1.4	-0.00	2.2	3.1	0.14
15	11.7	303.4	-0.2	1.4	-0.00	2.2	3.1	0.14
16	12.4	273.9	-0.3	1.3	-0.00	1.9	3.2	0.14
17	13.2	268.2	-0.3	1.3	-0.00	1.9	3.4	0.14
18	14.0	261.4	-0.3	1.2	-0.00	1.9	3.5	0.14
19	14.8	229.2	-0.5	1.1	-0.00	1.7	3.5	0.14
20	15.6	218.6	-0.5	1.0	-0.00	1.7	3.6	0.14
21	16.3	207.9	-0.5	1.0	-0.00	1.7	3.7	0.14
22	17.1	173.9	-0.4	0.9	-0.00	1.4	3.7	0.14
23	17.9	168.0	-0.3	1.0	-0.00	1.4	3.7	0.14
24	18.7	164.6	-0.3	1.1	-0.00	1.4	3.7	0.14
25	19.4	142.7	-3.8	1.1	-0.03	1.2	3.7	0.14
26	20.2	143.1	-4.1	1.3	-0.04	1.2	3.7	0.13
27	21.0	142.7	-4.2	1.5	-0.05	1.0	3.7	0.13
Absolute	2.3			2.0			(T =	36.7 ms)
	21.0				-0.05		(T =	47.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	387.7	355.8	323.9	292.1	260.2	228.3	196.5	164.6	132.7	100.9
RX	399.4	372.7	346.1	320.7	295.6	270.6	248.2	231.0	214.2	207.2
RU	387.7	355.8	323.9	292.1	260.2	228.3	196.5	164.6	132.7	100.9

RAU = 189.4 (kips); RA2 = 312.5 (kips)

Current CAPWAP Ru = 246.0 (kips); Corresponding J(RP)= 0.44; J(RX) = 0.61

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS	KEB
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips	kips/in
3.5	36.41	324.6	381.7	402.7	0.15	0.03	0.03	3.8	511.8	1625

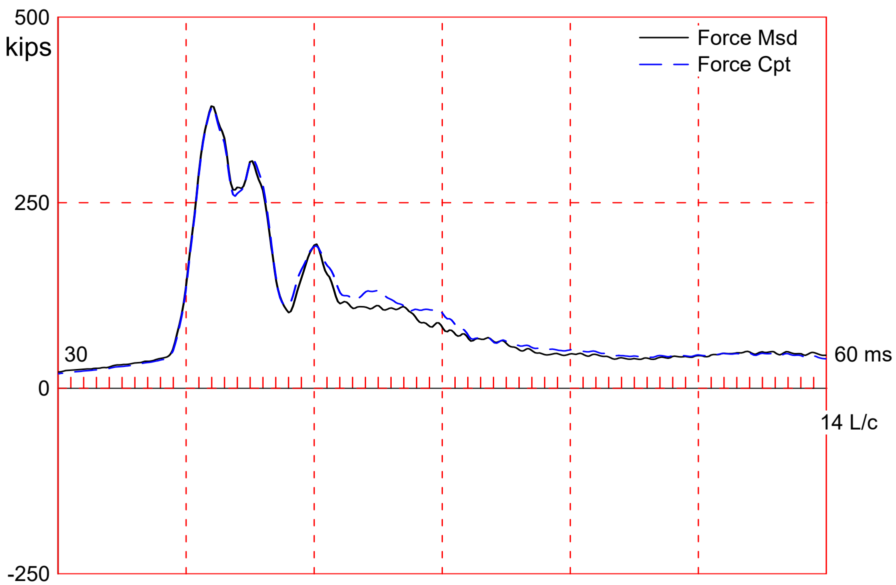
PILE PROFILE AND PILE MODEL

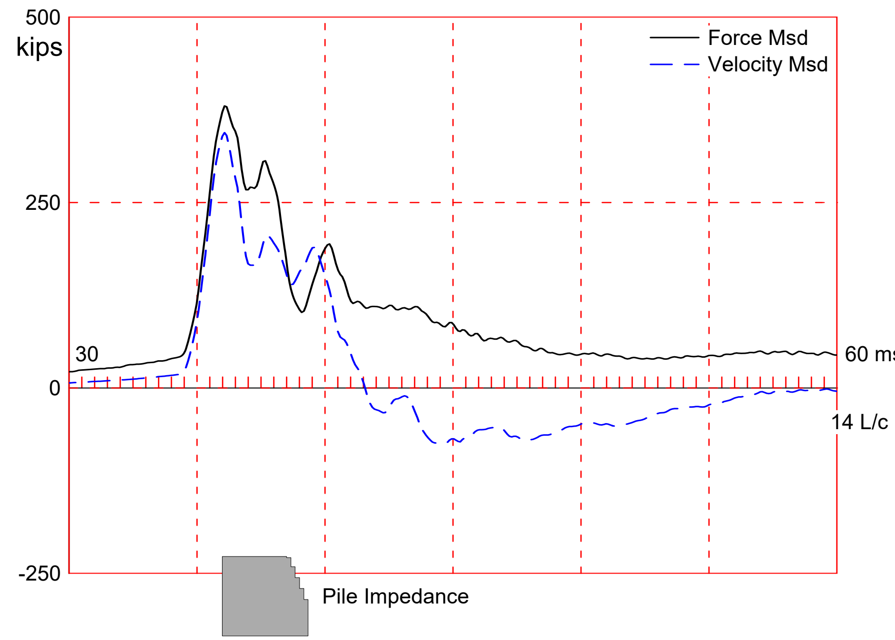
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in²	ksi	lb/ft³	ft
0.0	212.1	5900.5	150.000	4.42
16.0	212.1	5900.5	150.000	4.42
21.0	82.8	5900.5	150.000	2.76

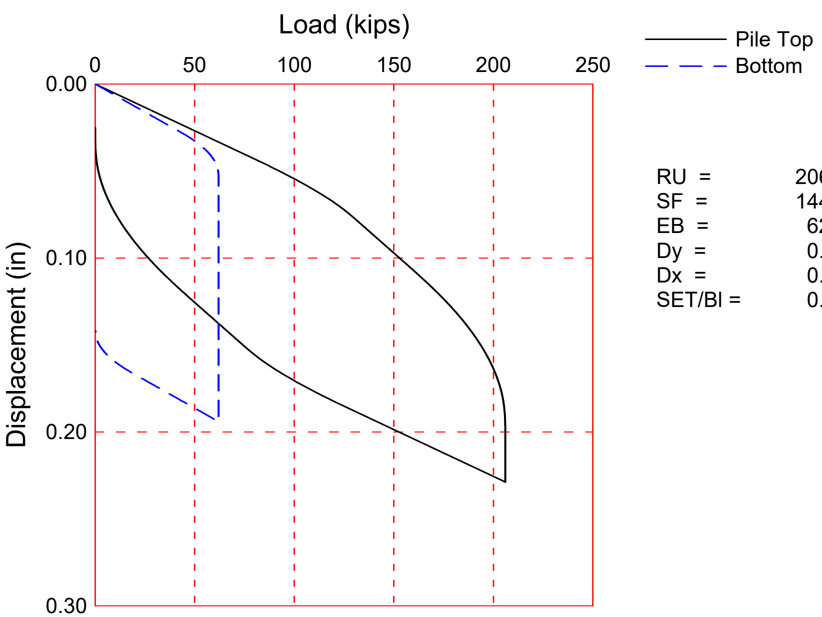
Toe Area 212.1 in²

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Tension Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.	Wave Speed
	ft	kips/ft/s	%	in		in		ft	ft/s
1	0.8	92.69	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0
21	16.3	91.89	0.00	0.00	0.000	-0.00	0.000	4.39	13500.0
22	17.1	84.54	0.00	0.00	0.000	-0.00	0.000	4.18	13500.0
23	17.9	75.75	0.00	0.00	0.000	-0.00	0.000	3.92	13500.0
24	18.7	66.96	0.00	0.00	0.000	-0.00	0.000	3.66	13500.0
25	19.4	58.17	0.00	0.00	0.000	-0.00	0.000	3.41	13500.0
26	20.2	49.39	0.00	0.00	0.000	-0.00	0.000	3.15	13500.0
27	21.0	40.60	0.00	0.00	0.000	-0.00	0.000	2.89	13500.0

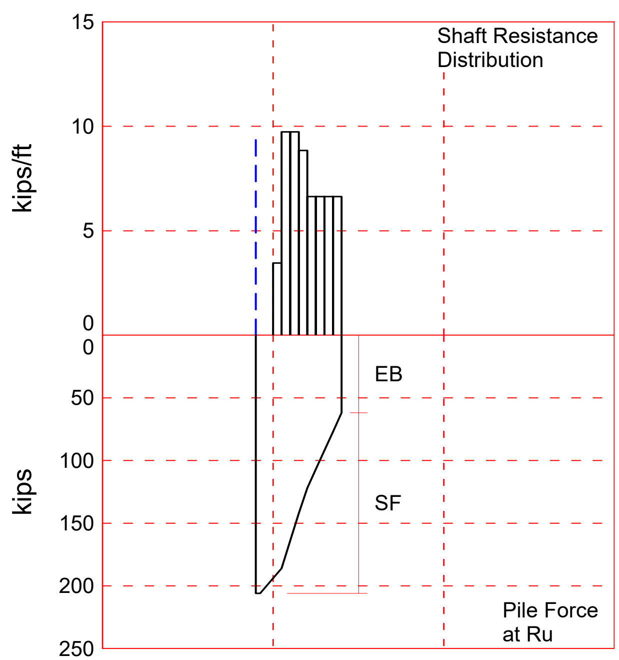
Wave Speed: Pile Top 13500.0, Elastic 13500.0, Overall 13500.0 ft/s
Pile Damping 2.00 %, Time Incr 0.058 ms, 2L/c 3.1 ms
Total volume: 28.685 ft³; Volume ratio considering added impedance: 1.000







RU =	206.0 kips
SF =	144.0 kips
EB =	62.0 kips
Dy =	0.20 in
Dx =	0.23 in
SET/Bl =	0.03 in



Length b. Sensors	22.6 ft
Embedment	21.6 ft
Top Area	212.1 in²
End Bearing Area	212.1 in²
Top Perimeter	4.42 ft
Top E-Modulus	5901 ksi
Top Spec. Weight	150.0 lb/ft³
Top Wave Spd.	13500 ft/s
Overall W.S.	13500 ft/s
Match Quality	2.23
Top Compr. Stress	1.8 ksi
Max Compr. Stress	1.9 ksi
Max Tension Stress	-0.03 ksi
Avg. Shaft Quake	0.13 in
Toe Quake	0.04 in
Avg. Shaft Smith Dpg.	0.30 s/ft
Toe Smith Damping	0.20 s/ft

About the CAPWAP Results

The CAPWAP program performs a signal matching or reverse analysis based on measurements taken on a deep foundation under an impact load. The program is based on a one-dimensional mathematical model. Under certain conditions, the model only crudely approximates the often complex dynamic situations.

The CAPWAP analysis relies on the input of accurately measured dynamic data plus additional parameters describing pile and soil behavior. If the field measurements of force and velocity are incorrect or were taken under inappropriate conditions (e.g., at an inappropriate time or with too much or too little energy) or if the input pile model is incorrect, then the solution cannot represent the actual soil behavior.

Generally the CAPWAP analysis is used to estimate the axial compressive pile capacity and the soil resistance distribution. The long-term capacity is best evaluated with restrike tests since they incorporate soil strength changes (set-up gains or relaxation losses) that occur after installation. The calculated load settlement graph does not consider creep or long term consolidation settlements. When uplift is a controlling factor in the design, use of the CAPWAP results to assess uplift capacity should be made only after very careful analysis of only good measurement quality, and further used only with longer pile lengths and with nominally higher safety factors.

CAPWAP is also used to evaluate driving stresses along the length of the pile. However, it should be understood that the analysis is one dimensional and does not take into account bending effects or local contact stresses at the pile toe.

Furthermore, if the user of this software was not able to produce a solution with satisfactory signal “match quality” (MQ), then the associated CAPWAP results may be unreliable. There is no absolute scale for solution acceptability but solutions with MQ above 5 are generally considered less reliable than those with lower MQ values and every effort should be made to improve the analysis, for example, by getting help from other independent experts.

Considering the CAPWAP model limitations, the nature of the input parameters, the complexity of the analysis procedure, and the need for a responsible application of the results to actual construction projects, it is recommended that at least one static load test be performed on sites where little experience exists with dynamic behavior of the soil resistance or when the experience of the analyzing engineer with both program use and result application is limited.

Finally, the CAPWAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors. The CAPWAP results should be reviewed by the Engineer of Record with consideration of applicable geotechnical conditions including, but not limited to, group effects, potential settlement from underlying compressible layers, soil resistances provided from any layers unsuitable for long term support, as well as effective stress changes due to soil surcharges, excavation or change in water table elevation.

The CAPWAP analysis software is one of many means by which the capacity of a deep foundation can be assessed. The engineer performing the analysis is responsible for proper software application and the analysis results. Pile Dynamics accepts no liability whatsoever of any kind for the analysis solution and/or the application of the analysis result.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 206.0; along Shaft 144.0; at Toe 62.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				206.0			
1	6.8	5.8	20.0	186.0	20.0	3.46	0.78
2	9.0	8.0	22.0	164.0	42.0	9.73	2.20
3	11.3	10.3	22.0	142.0	64.0	9.73	2.20
4	13.6	12.6	20.0	122.0	84.0	8.85	2.00
5	15.8	14.8	15.0	107.0	99.0	6.64	1.50
6	18.1	17.1	15.0	92.0	114.0	6.64	1.51
7	20.3	19.3	15.0	77.0	129.0	6.64	1.71
8	22.6	21.6	15.0	62.0	144.0	6.64	2.12
Avg. Shaft			18.0			6.67	1.58
Toe			62.0				42.10

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.30	0.20
Quake	(in)	0.13	0.04
Case Damping Factor		0.47	0.13
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	92	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	
Soil Plug Weight	(kips)		0.154

CAPWAP match quality = 2.23 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.03 in; Blow Count = 480 b/ft
Computed: Final Set = 0.02 in; Blow Count = 554 b/ft

max. Top Comp. Stress = 1.8 ksi (T= 36.2 ms, max= 1.069 x Top)
max. Comp. Stress = 1.9 ksi (Z= 6.8 ft, T= 36.7 ms)
max. Tens. Stress = -0.03 ksi (Z= 22.6 ft, T= 49.4 ms)
max. Energy (EMX) = 3.1 kip-ft; max. Measured Top Displ. (DMX)= 0.16 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	1.1	381.2	-0.6	1.8	-0.00	3.1	3.7	0.16
2	2.3	384.9	-0.5	1.8	-0.00	3.1	3.7	0.16
3	3.4	390.1	-0.6	1.8	-0.00	3.1	3.6	0.16
4	4.5	395.8	-0.8	1.9	-0.00	3.0	3.5	0.16
5	5.7	401.7	-0.8	1.9	-0.00	3.0	3.5	0.16
6	6.8	407.4	-0.8	1.9	-0.00	3.0	3.4	0.15
7	7.9	384.6	-0.6	1.8	-0.00	2.7	3.4	0.15
8	9.0	387.9	-0.7	1.8	-0.00	2.7	3.3	0.15
9	10.2	360.5	-0.5	1.7	-0.00	2.4	3.3	0.15
10	11.3	360.6	-0.5	1.7	-0.00	2.4	3.3	0.15
11	12.4	328.6	-0.5	1.5	-0.00	2.1	3.3	0.15
12	13.6	321.7	-0.5	1.5	-0.00	2.1	3.4	0.14
13	14.7	283.0	-0.4	1.3	-0.00	1.8	3.6	0.14
14	15.8	267.2	-0.5	1.3	-0.00	1.8	3.8	0.14
15	17.0	229.9	-0.6	1.1	-0.00	1.6	4.0	0.14
16	18.1	212.9	-0.5	1.0	-0.00	1.6	4.1	0.14
17	19.2	176.3	-0.5	1.0	-0.00	1.3	4.2	0.14
18	20.3	172.7	-0.4	1.1	-0.00	1.3	4.2	0.14
19	21.5	148.4	-3.3	1.2	-0.03	1.1	4.2	0.14
20	22.6	145.7	-2.9	1.5	-0.03	0.9	4.3	0.14
Absolute	6.8			1.9			(T =	36.7 ms)
	22.6				-0.03		(T =	49.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	340.2	301.3	262.4	223.5	184.6	145.7	106.8	67.9	29.0	0.0
RX	340.2	301.3	264.9	235.0	209.6	193.3	185.0	176.8	170.7	167.6
RU	340.2	301.3	262.4	223.5	184.6	145.7	106.8	67.9	29.0	0.0

RAU = 160.1 (kips); RA2 = 221.3 (kips)

Current CAPWAP Ru = 206.0 (kips); Corresponding J(RP)= 0.34; J(RX) = 0.42

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
3.7	36.08	345.4	383.8	383.8	0.16	0.02	0.03	3.1	404.7	1550

PILE PROFILE AND PILE MODEL

Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.0	212.1	5900.5	150.000	4.42

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in²	ksi	lb/ft³	ft
17.6	212.1	5900.5	150.000	4.42
22.6	82.8	5900.5	150.000	2.76

Toe Area 212.1 in²

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Slack	Tension Eff.	Slack	Compression Eff.	Perim.	Wave Speed
	ft	kips/ft/s	%	in		in		ft	ft/s
1	1.1	92.69	0.00	0.00	0.000	-0.00	0.000	4.42	13500.0
16	18.1	91.54	0.00	0.00	0.000	-0.00	0.000	4.38	13500.0
17	19.2	80.89	0.00	0.00	0.000	-0.00	0.000	4.07	13500.0
18	20.3	68.12	0.00	0.00	0.000	-0.00	0.000	3.70	13500.0
19	21.5	55.36	0.00	0.00	0.000	-0.00	0.000	3.32	13500.0
20	22.6	42.59	0.00	0.00	0.000	-0.00	0.000	2.95	13500.0

Wave Speed: Pile Top 13500.0, Elastic 13500.0, Overall 13500.0 ft/s
Pile Damping 2.00 %, Time Incr 0.084 ms, 2L/c 3.3 ms
Total volume: 31.041 ft³; Volume ratio considering added impedance: 1.000

Appendix E – Parallel Seismic Time Records

SCPT Parallel Seismic Time Records

Route U

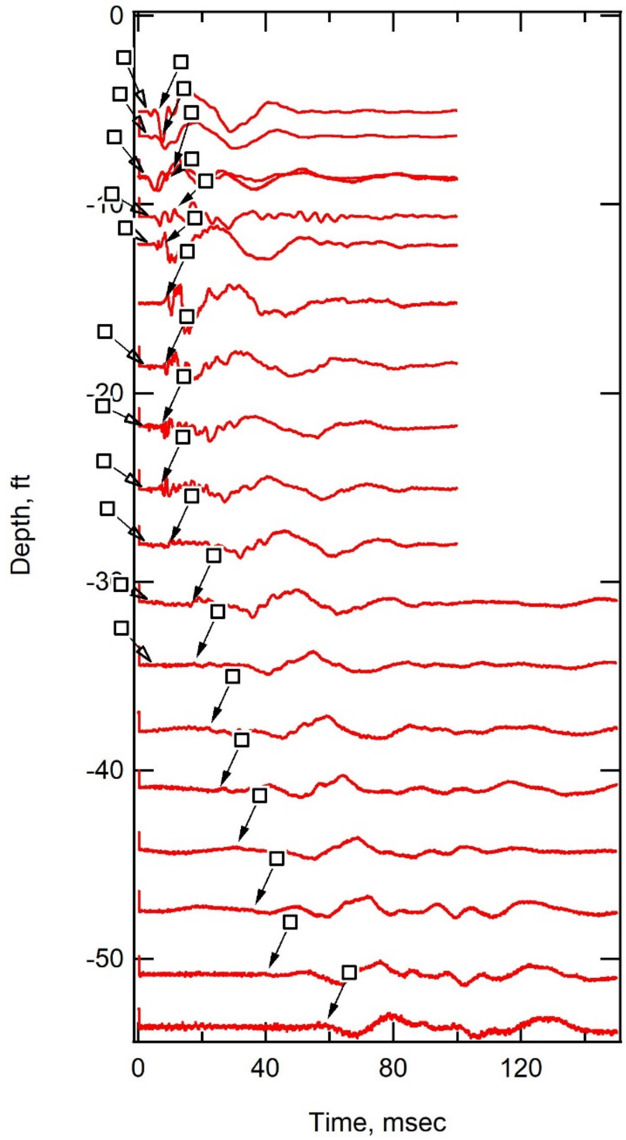


Figure E-1 Wave arrival picks at Route U using SCPT for Pile 1 for p-waves (open arrow) and s-waves (solid arrow).

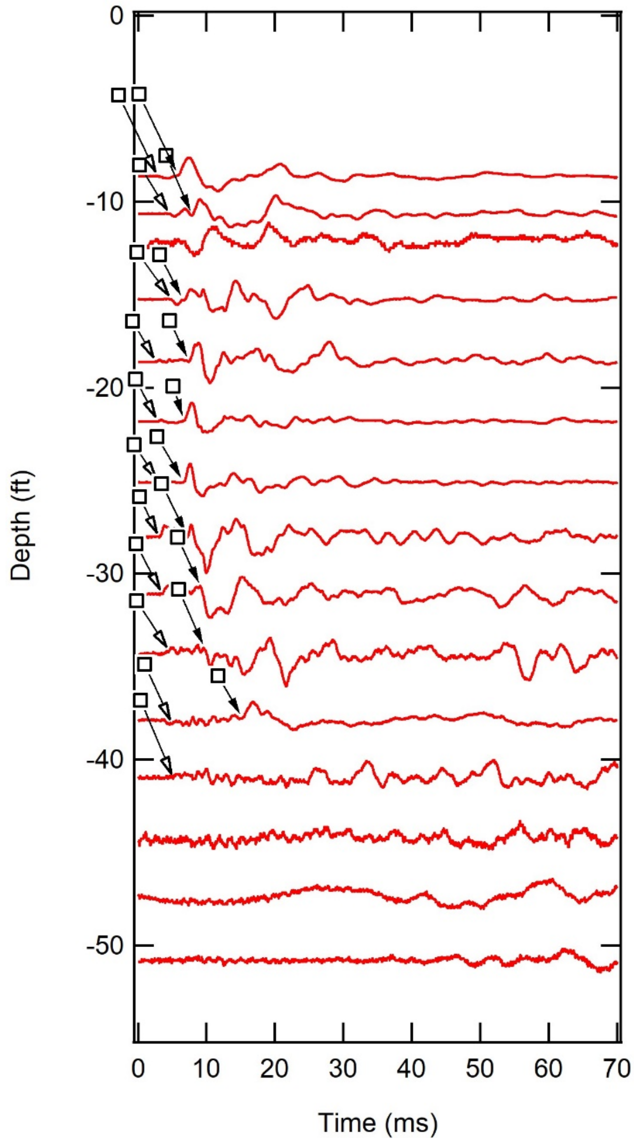


Figure E-2 Wave arrival picks at Route U using SCPT for Pile 2 for p-waves (open arrow) and s-waves (solid arrow).

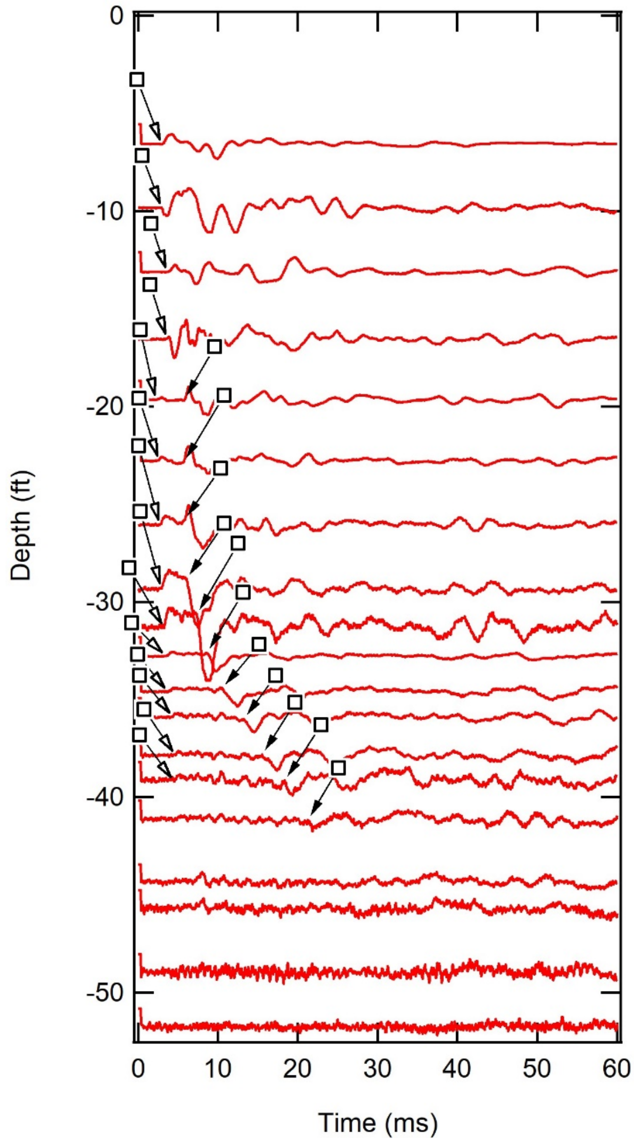


Figure E-3 Wave arrival picks at Route U using SCPT for Pile 3 for p-waves (open arrow) and s-waves (solid arrow).

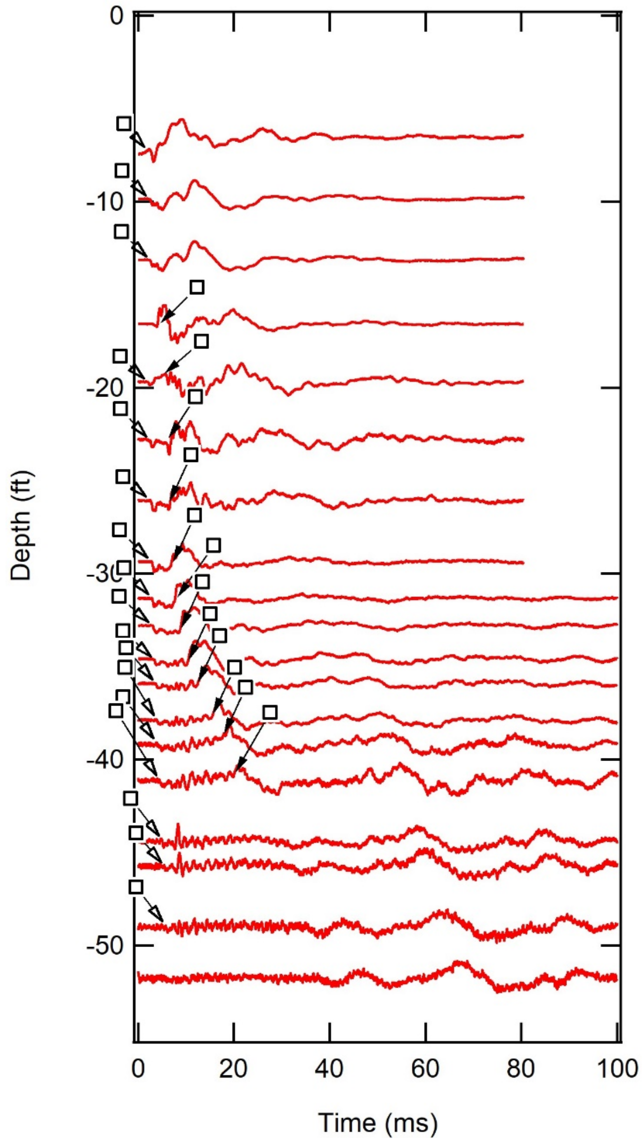


Figure E-4 Wave arrival picks at Route U using SCPT for Pile 4 for p-waves (open arrow) and s-waves (solid arrow).

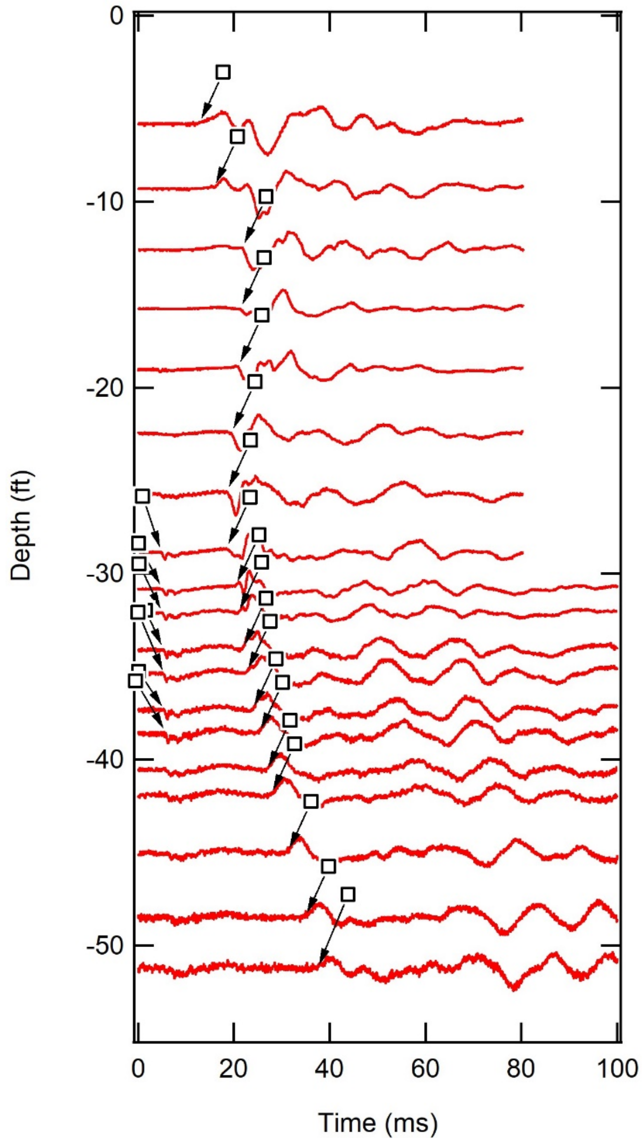


Figure E-5 Wave arrival picks at Route U using SCPT for Pile 5 for p-waves (open arrow) and s-waves (solid arrow).

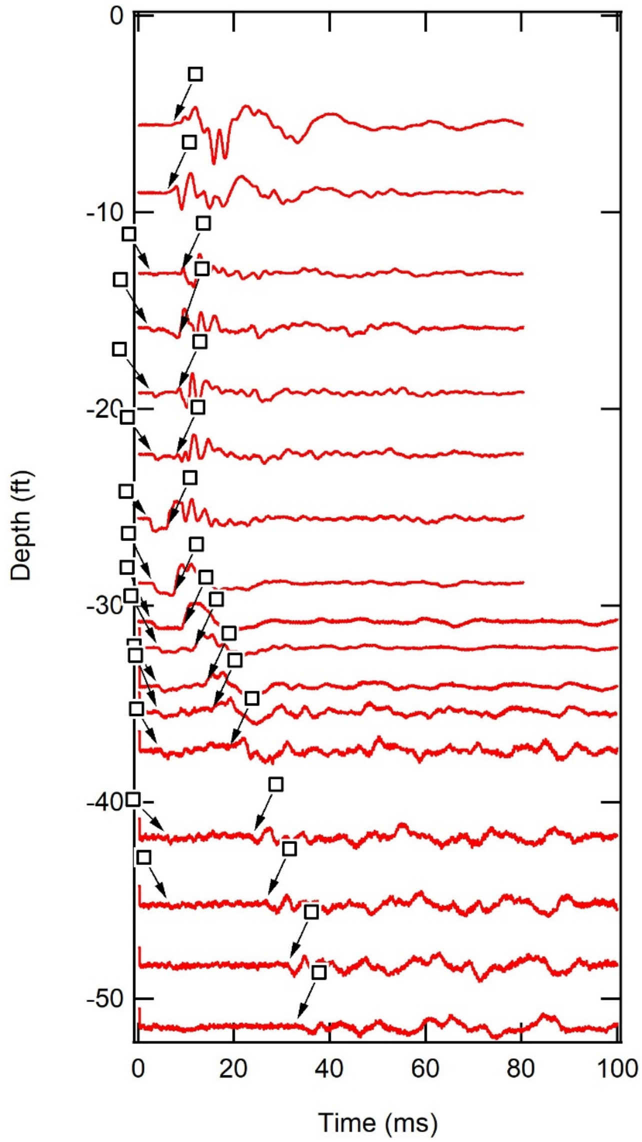


Figure E-6 Wave arrival picks at Route U using SCPT for Pile 8 for p-waves (open arrow) and s-waves (solid arrow).

SCPT Parallel Seismic Time Records

Route WW

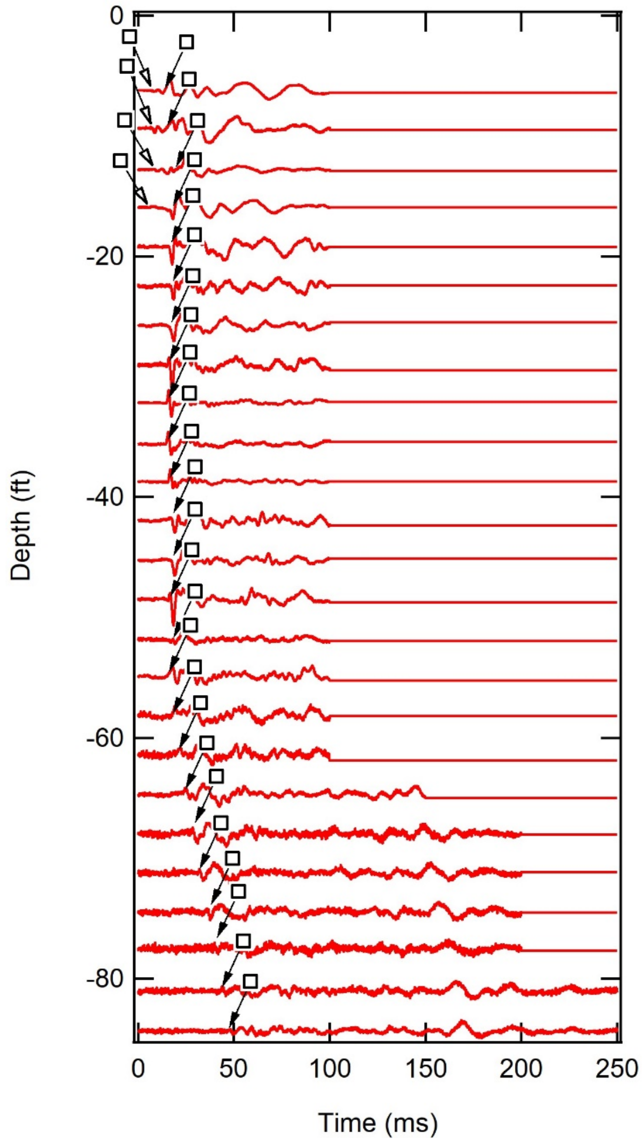


Figure E-7 Wave arrival picks at Route WW using SCPT for Pile 1 for p-waves (open arrow) and s-waves (solid arrow).

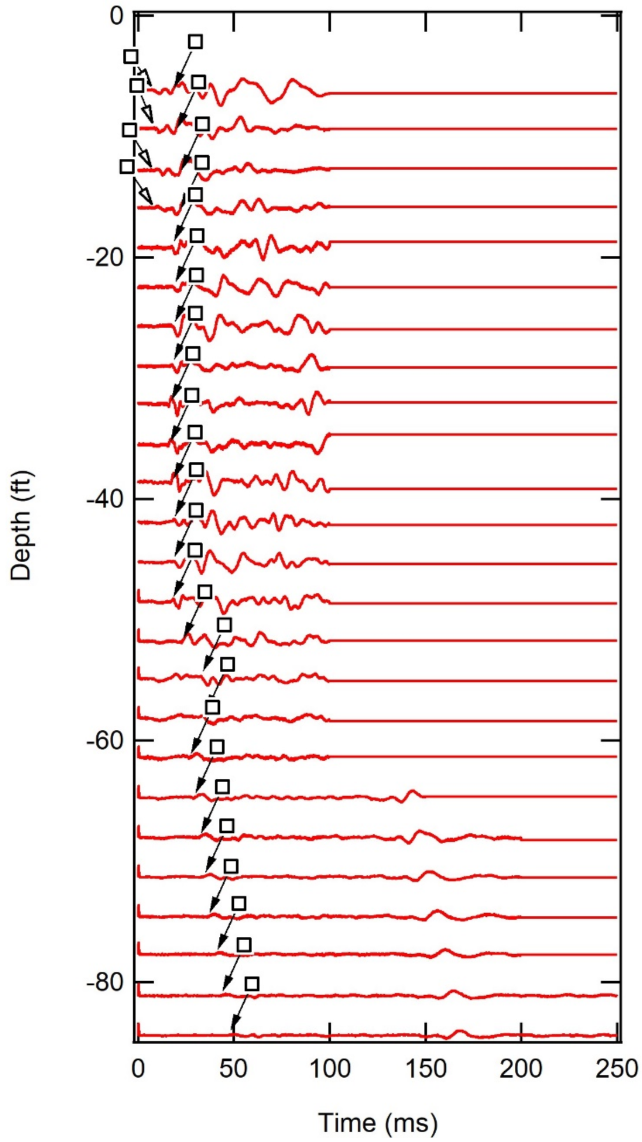


Figure E-8 Wave arrival picks at Route WW using SCPT for Pile 2 for p-waves (open arrow) and s-waves (solid arrow).

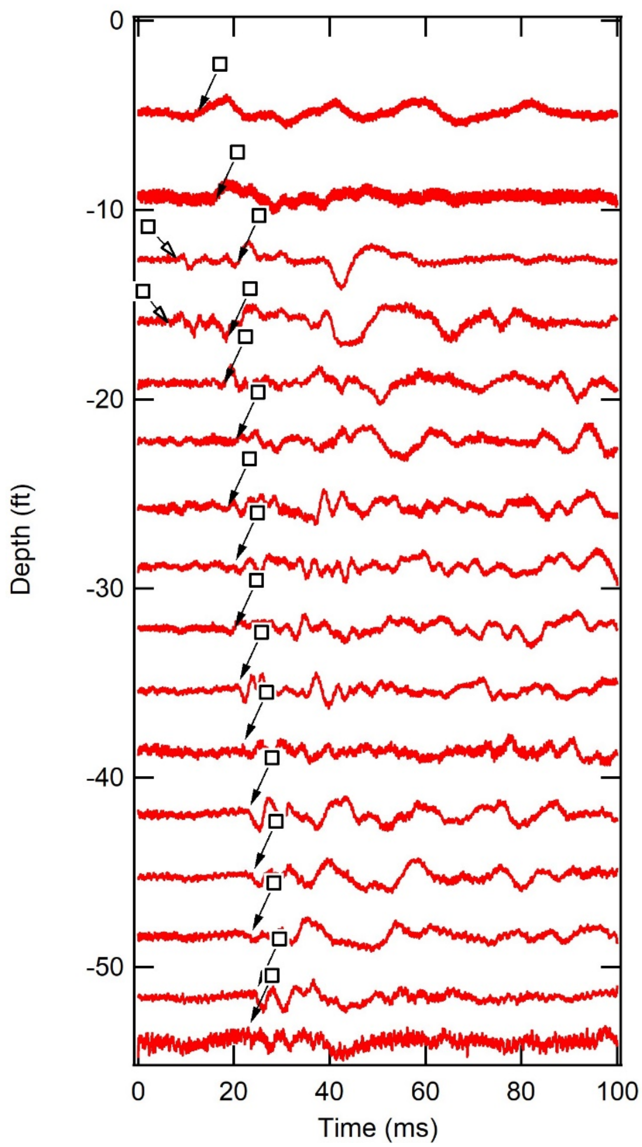


Figure E-9 Wave arrival picks at Route WW using SCPT for Pile 3 for p-waves (open arrow) and s-waves (solid arrow).

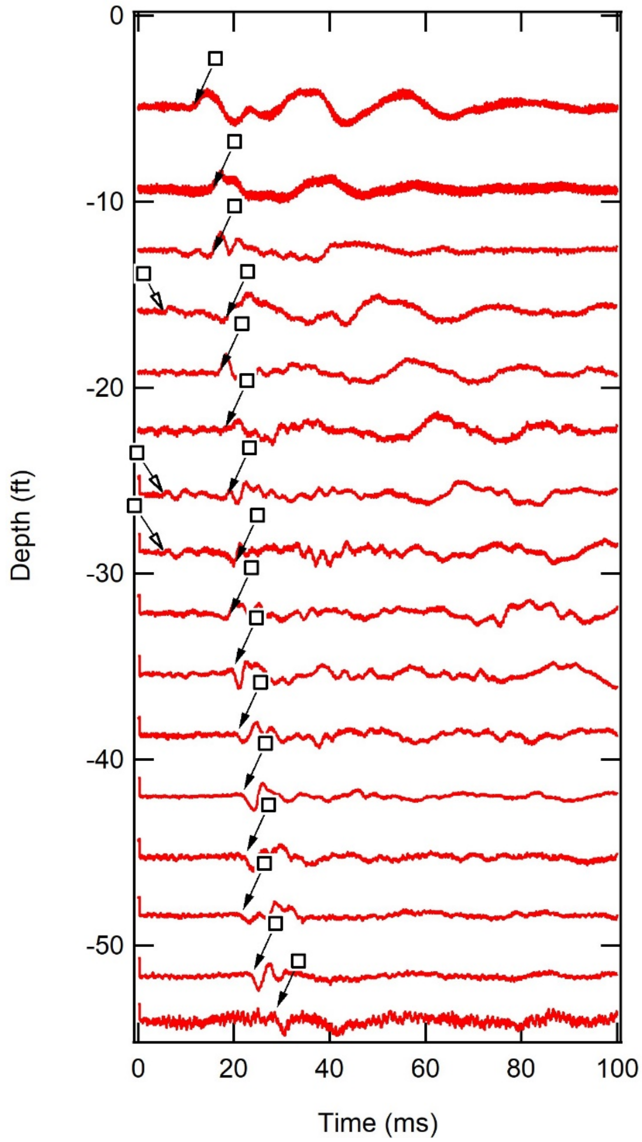


Figure E-10 Wave arrival picks at Route WW using SCPT for Pile 4 for p-waves (open arrow) and s-waves (solid arrow).

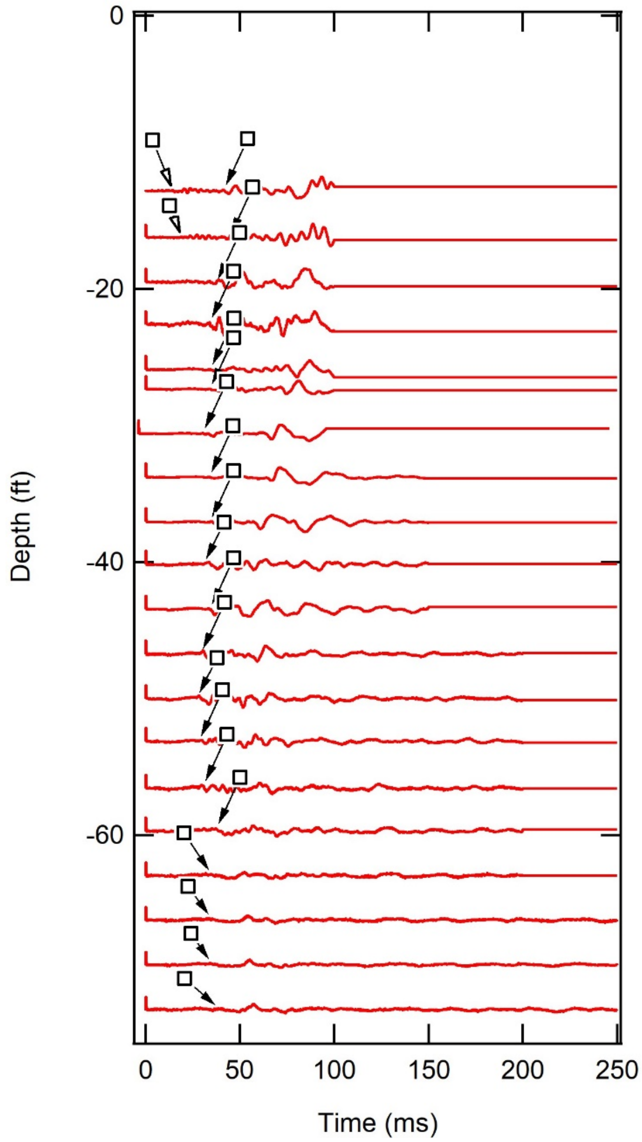


Figure E-11 Wave arrival picks at Route WW using SCPT for Pile 5 for p-waves (open arrow) and s-waves (solid arrow).

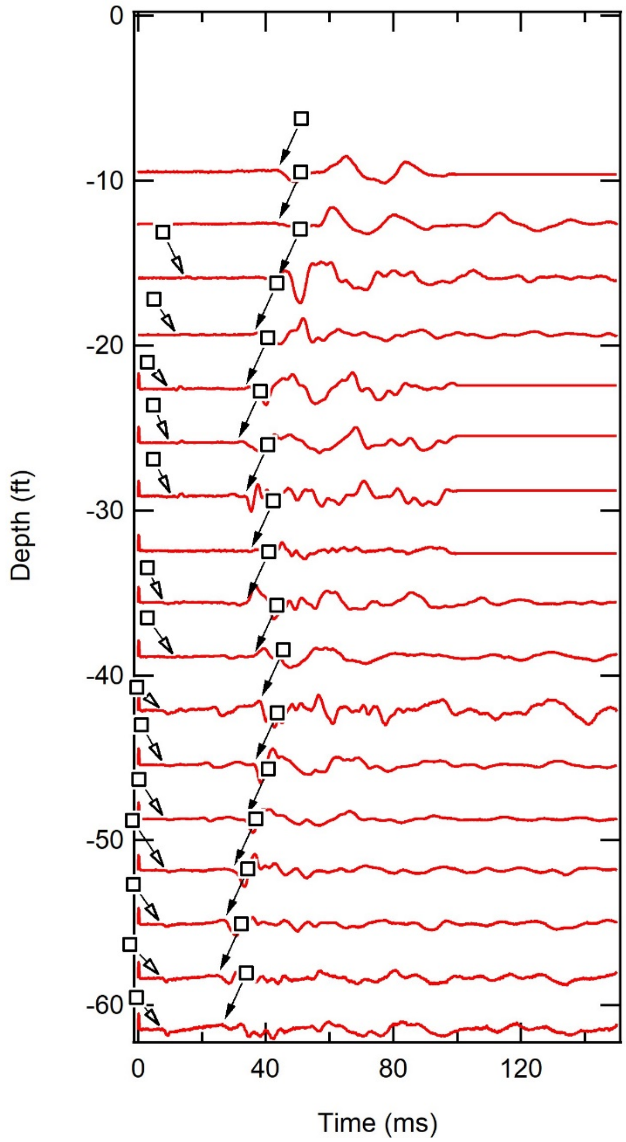


Figure E-12 Wave arrival picks at Route WW using SCPT for Pile 8 for p-waves (open arrow) and s-waves (solid arrow).

Borehole Parallel Seismic Time Records

Route U

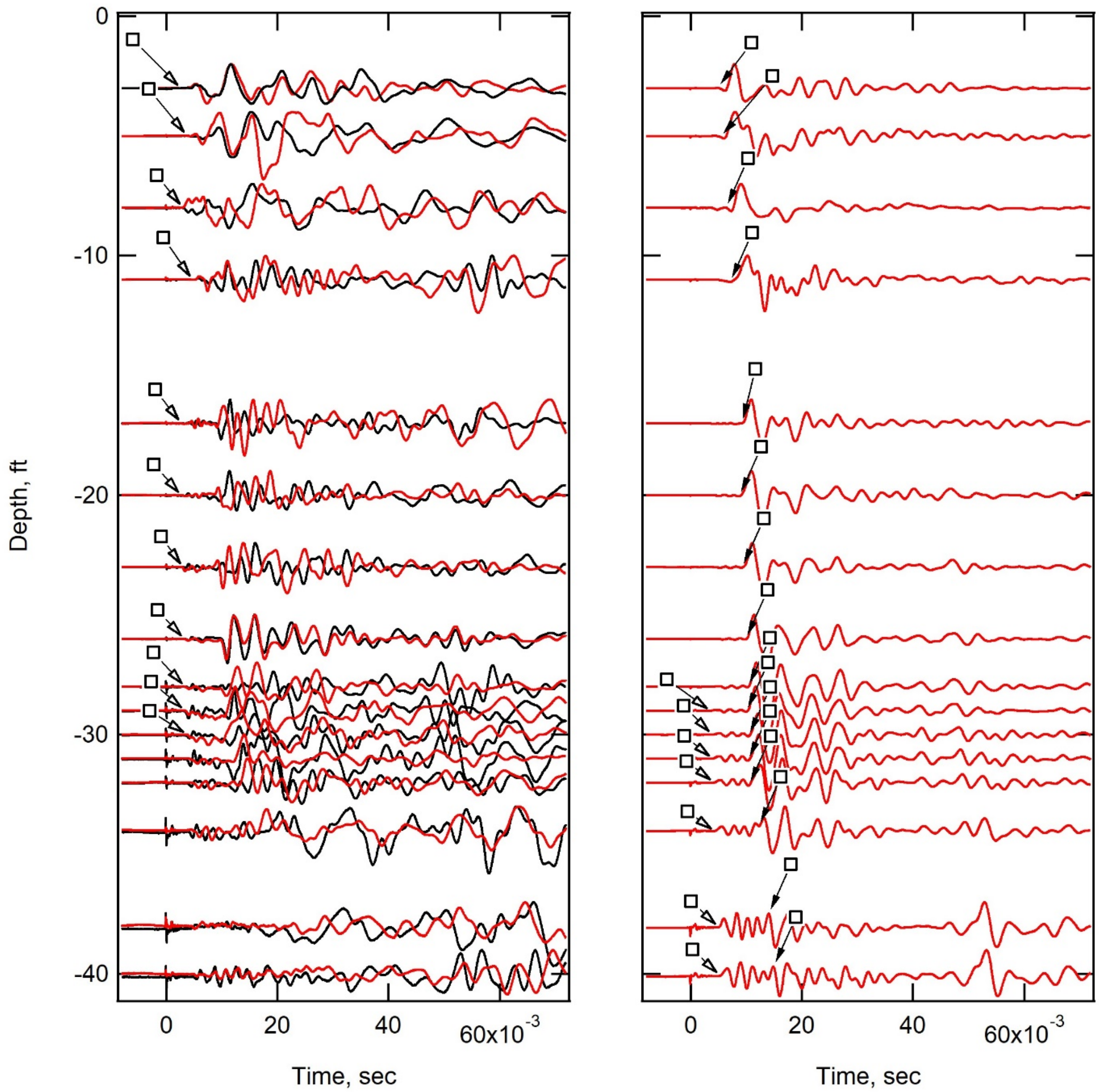


Figure E-13 Wave arrival picks at Route U Borehole 1 – Pile 1 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

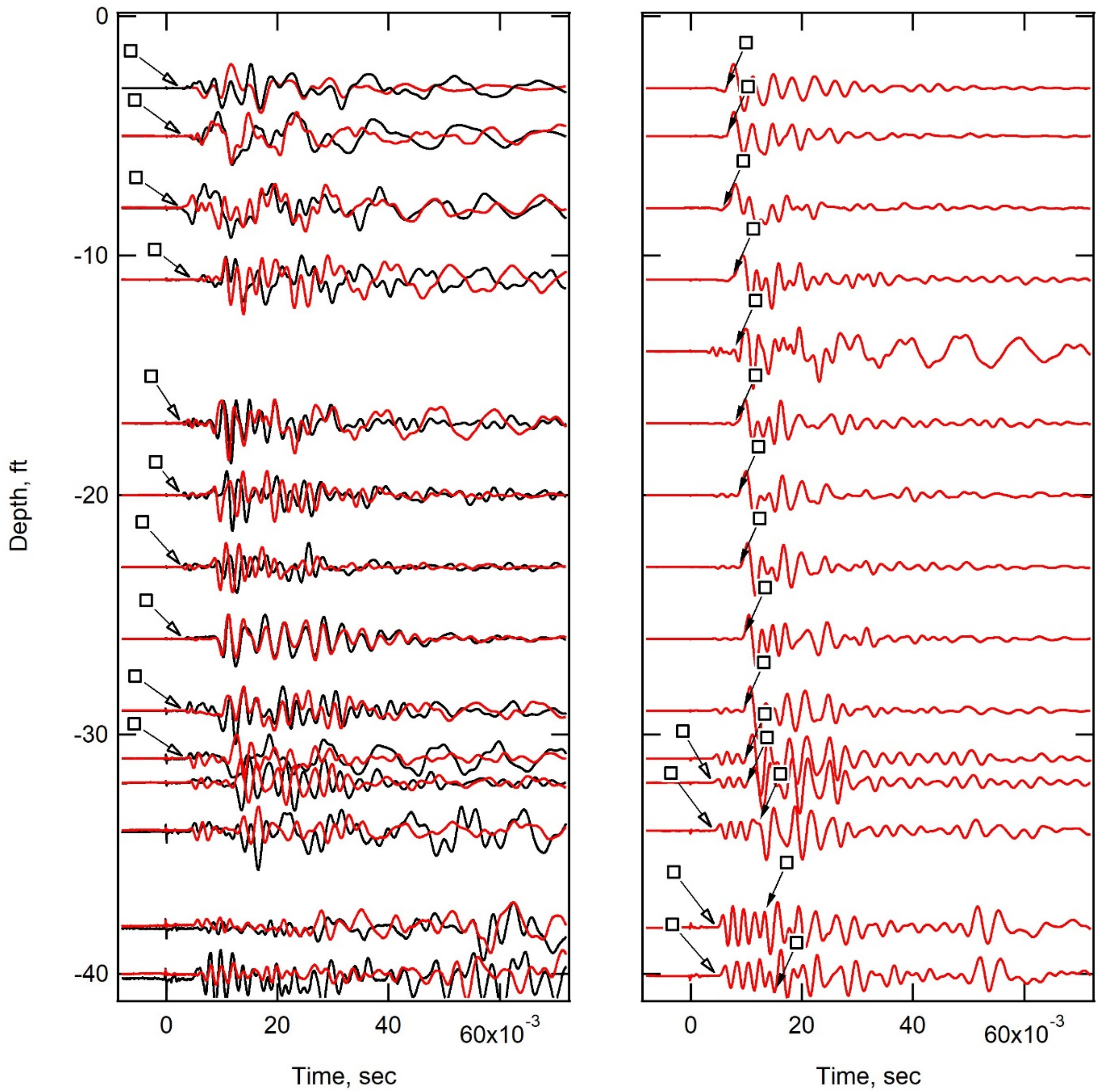


Figure E-14 Wave arrival picks at Route U Borehole 1 – Pile 2 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

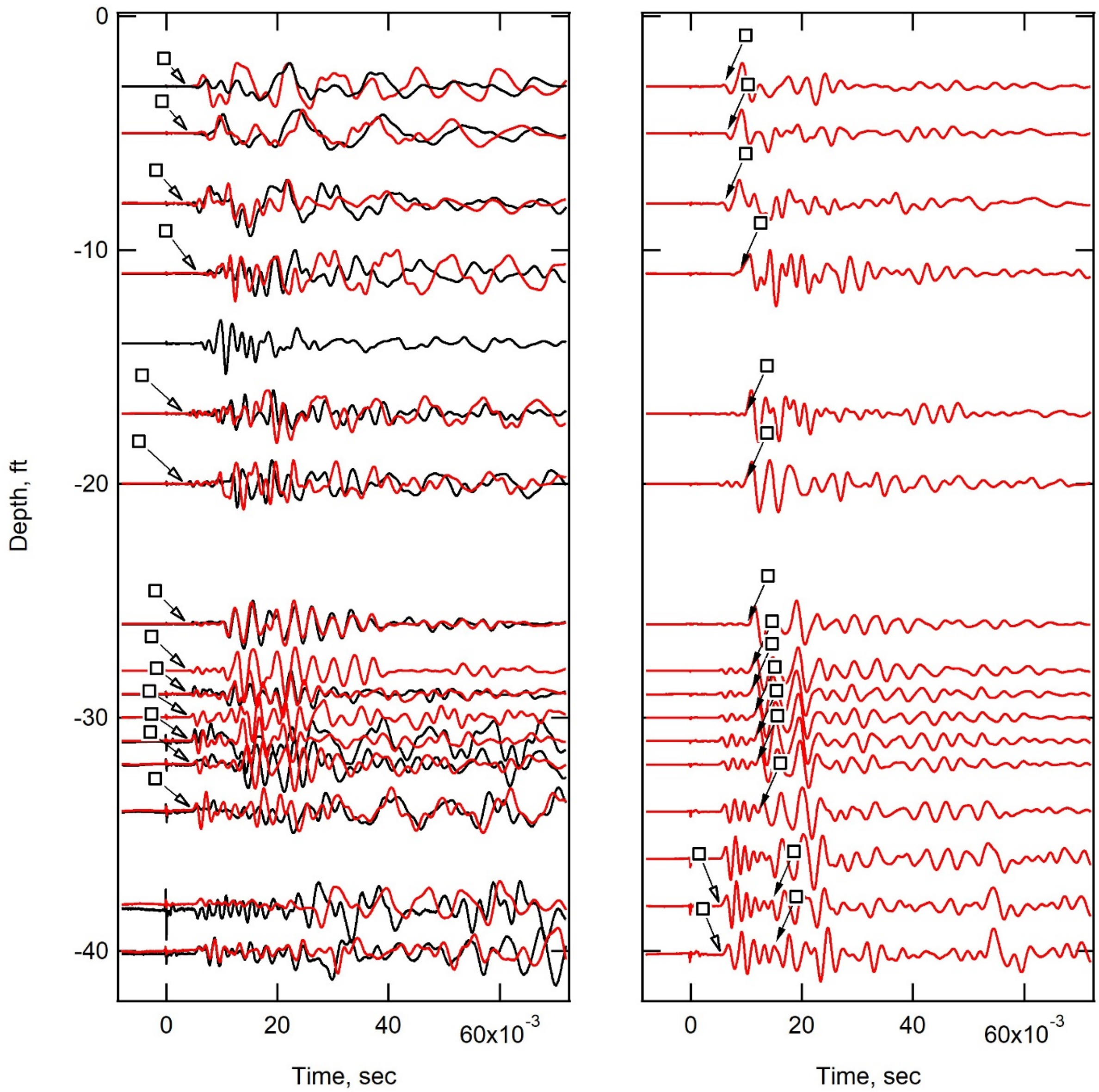


Figure E-15 Wave arrival picks at Route U Borehole 1 – Pile 3 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

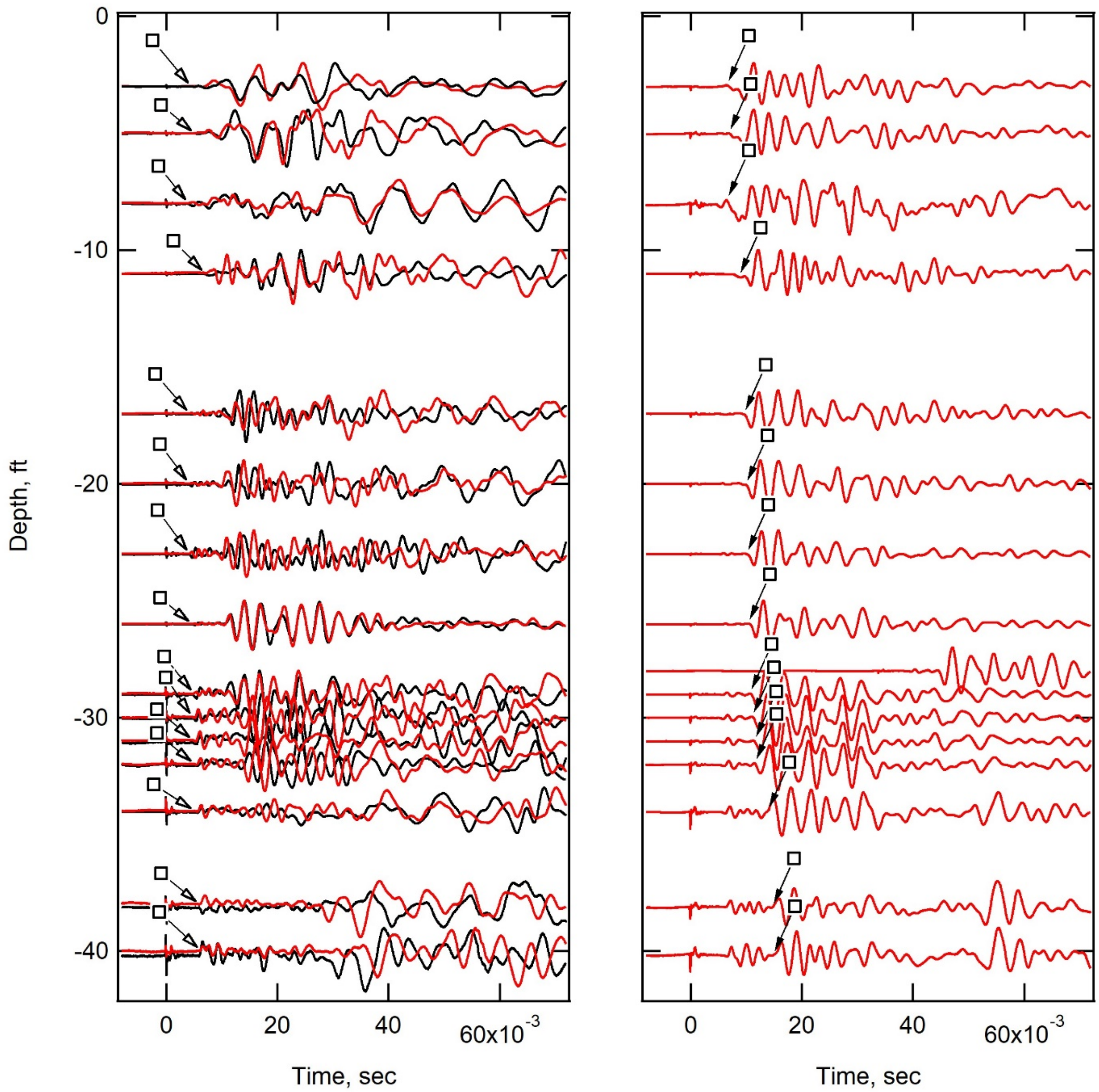


Figure E-16 Wave arrival picks at Route U Borehole 1 – Pile 4 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

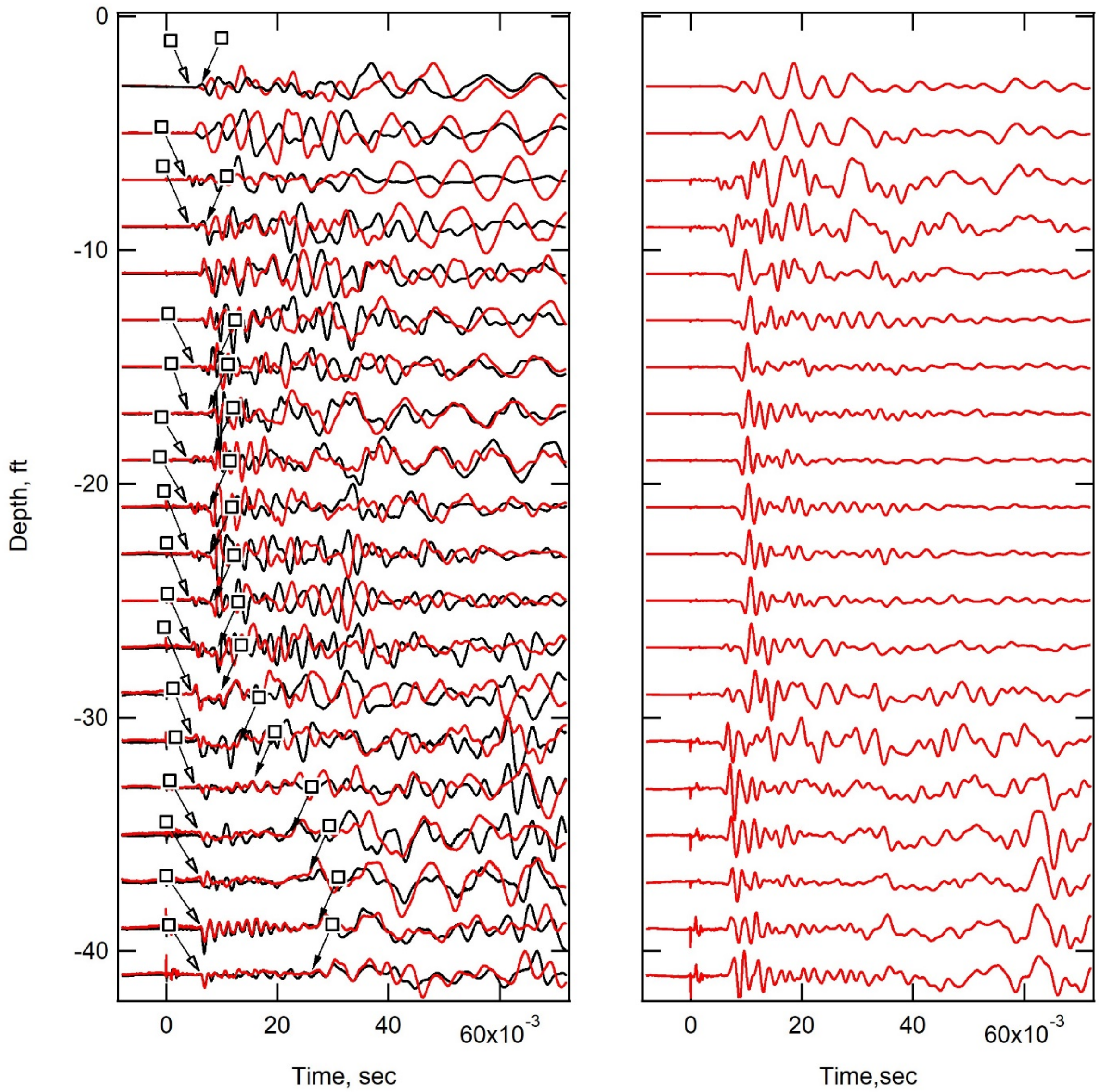


Figure E-17 Wave arrival picks at Route U Borehole 2 – Pile 1 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

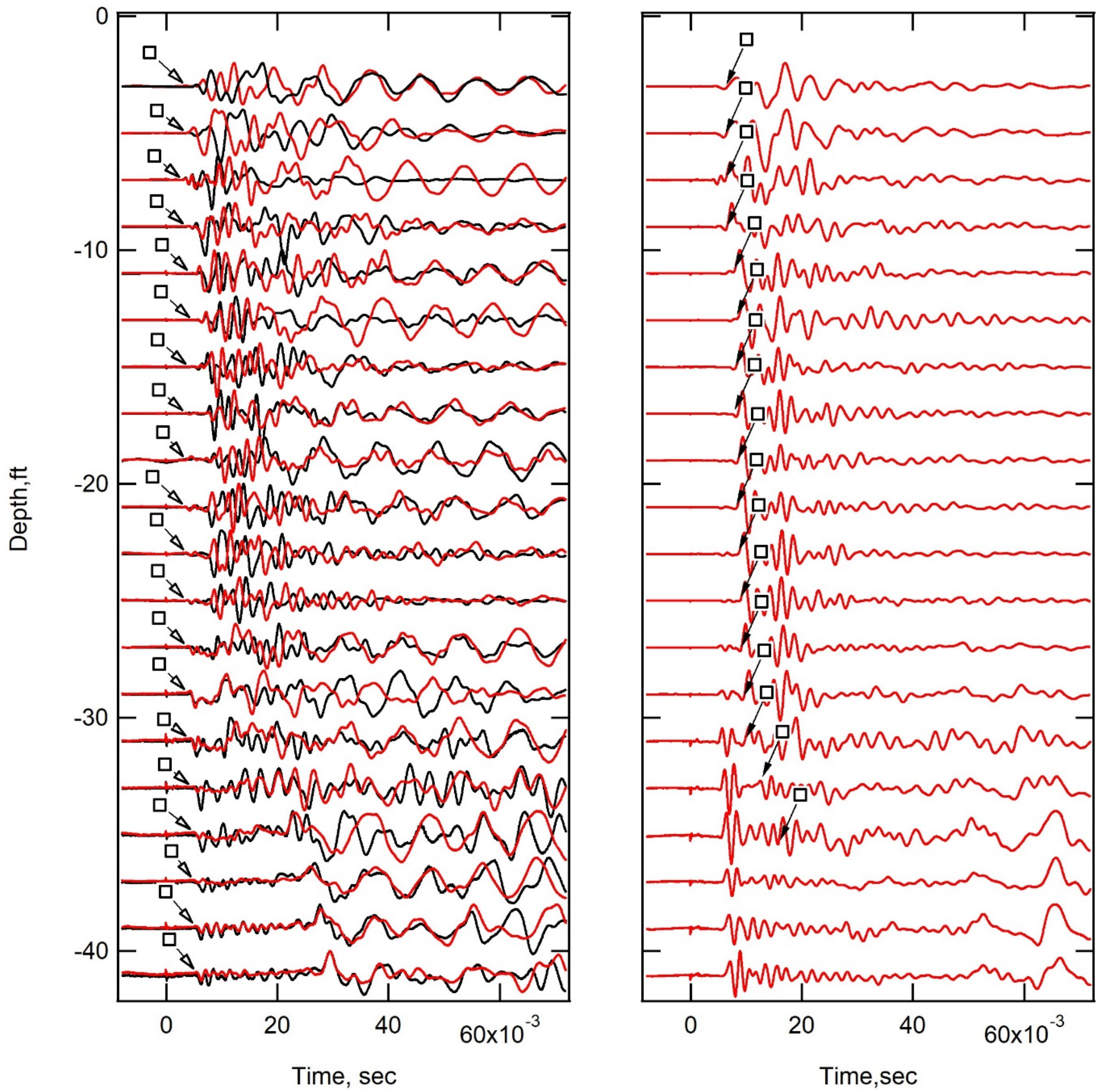


Figure E-18 Wave arrival picks at Route U Borehole 2 – Pile 2 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

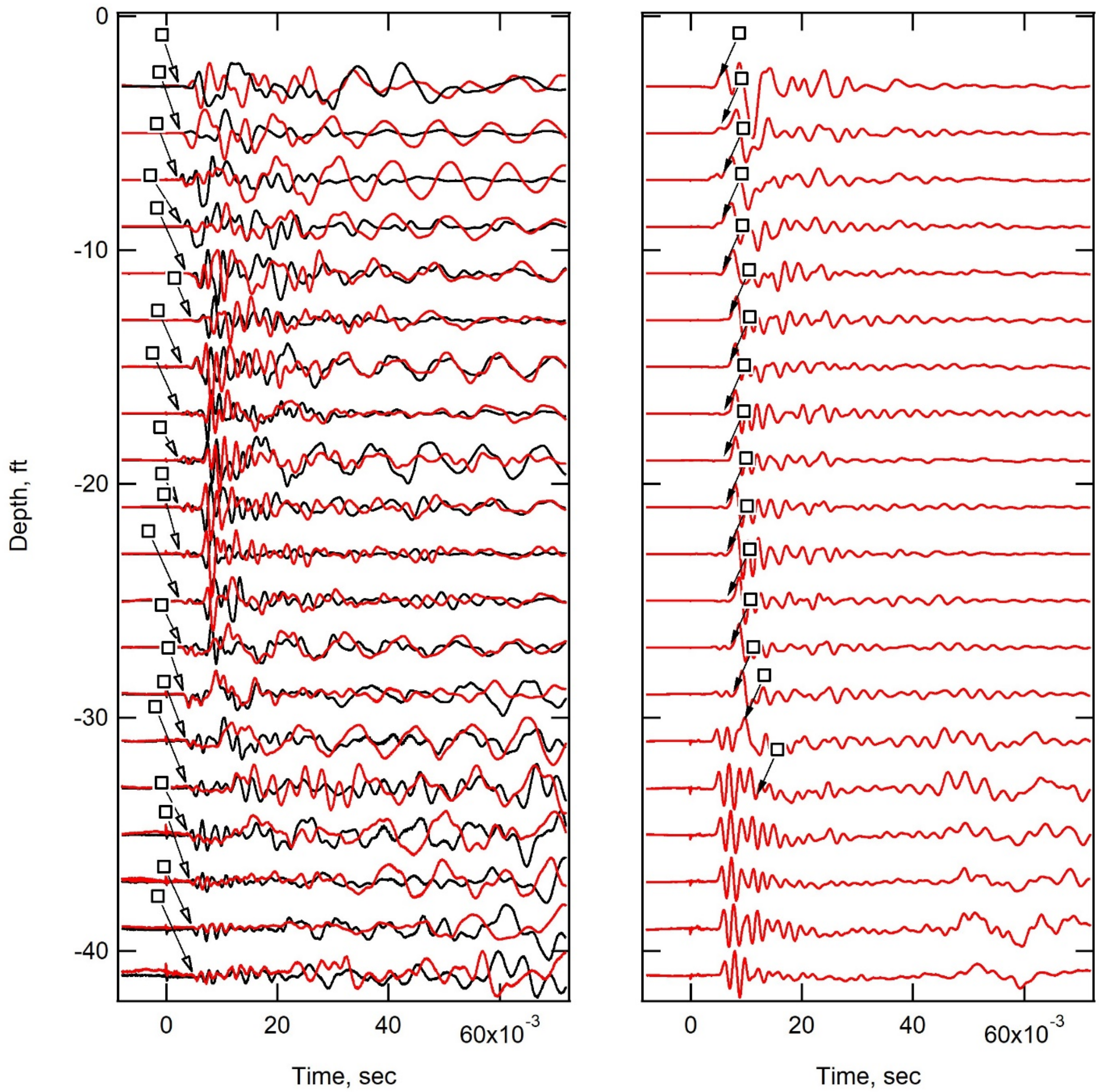


Figure E-19 Wave arrival picks at Route U Borehole 2 – Pile 3 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

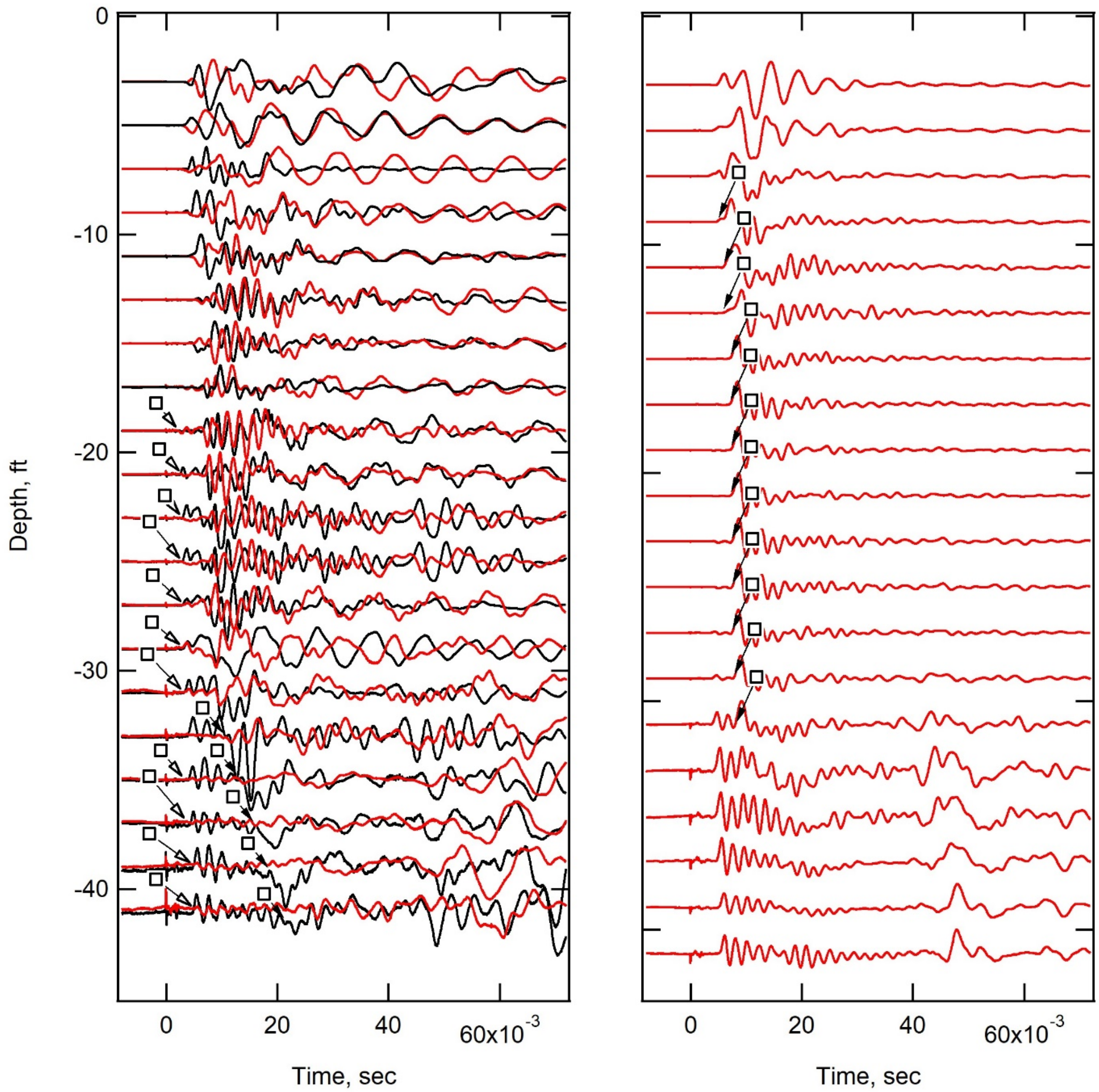


Figure E-20 Wave arrival picks at Route U Borehole 2 – Pile 4 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

Borehole Parallel Seismic Time Records

Route WW

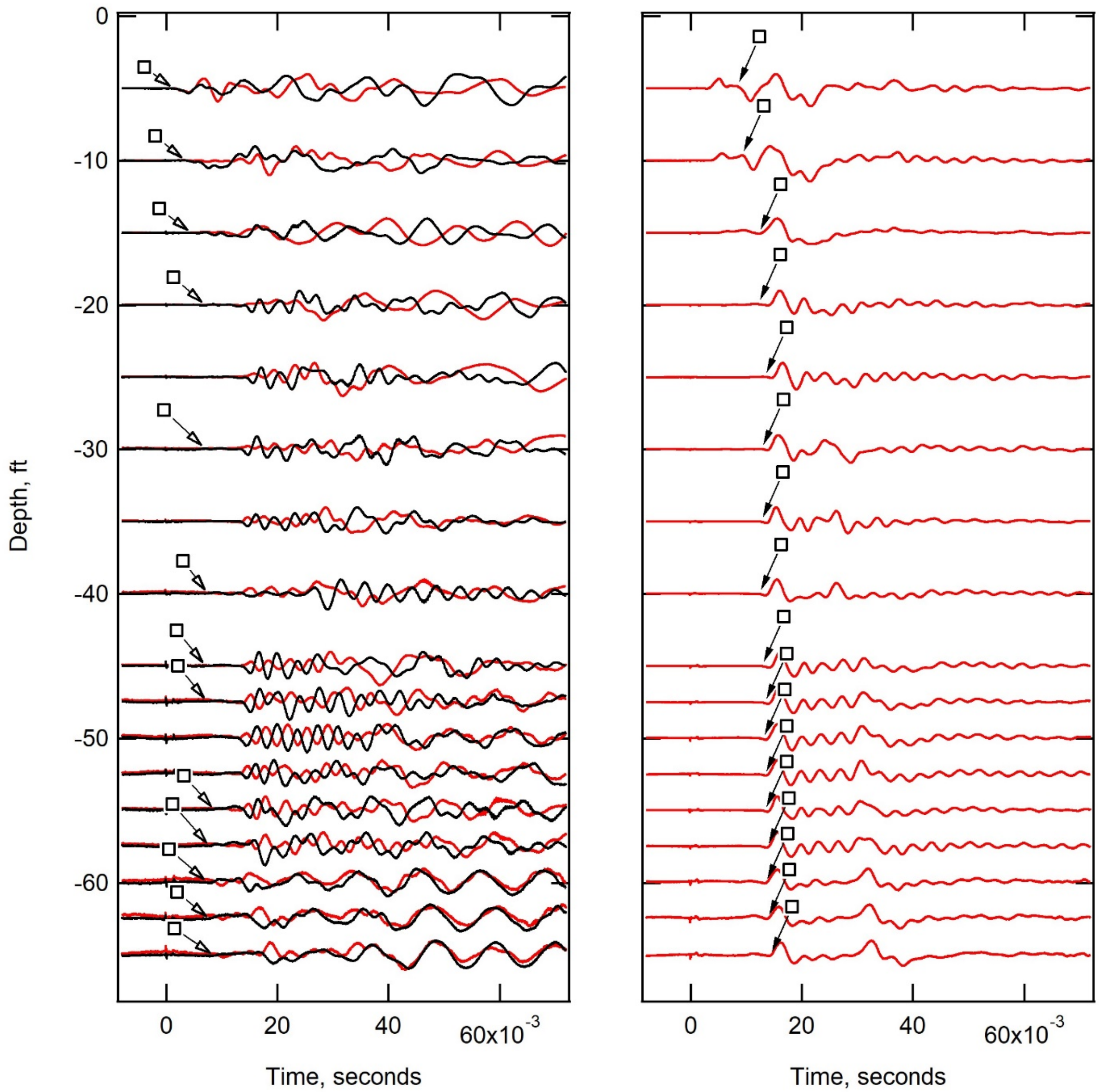


Figure E-21 Wave arrival picks at Route WW Borehole 1 – Pile 1 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

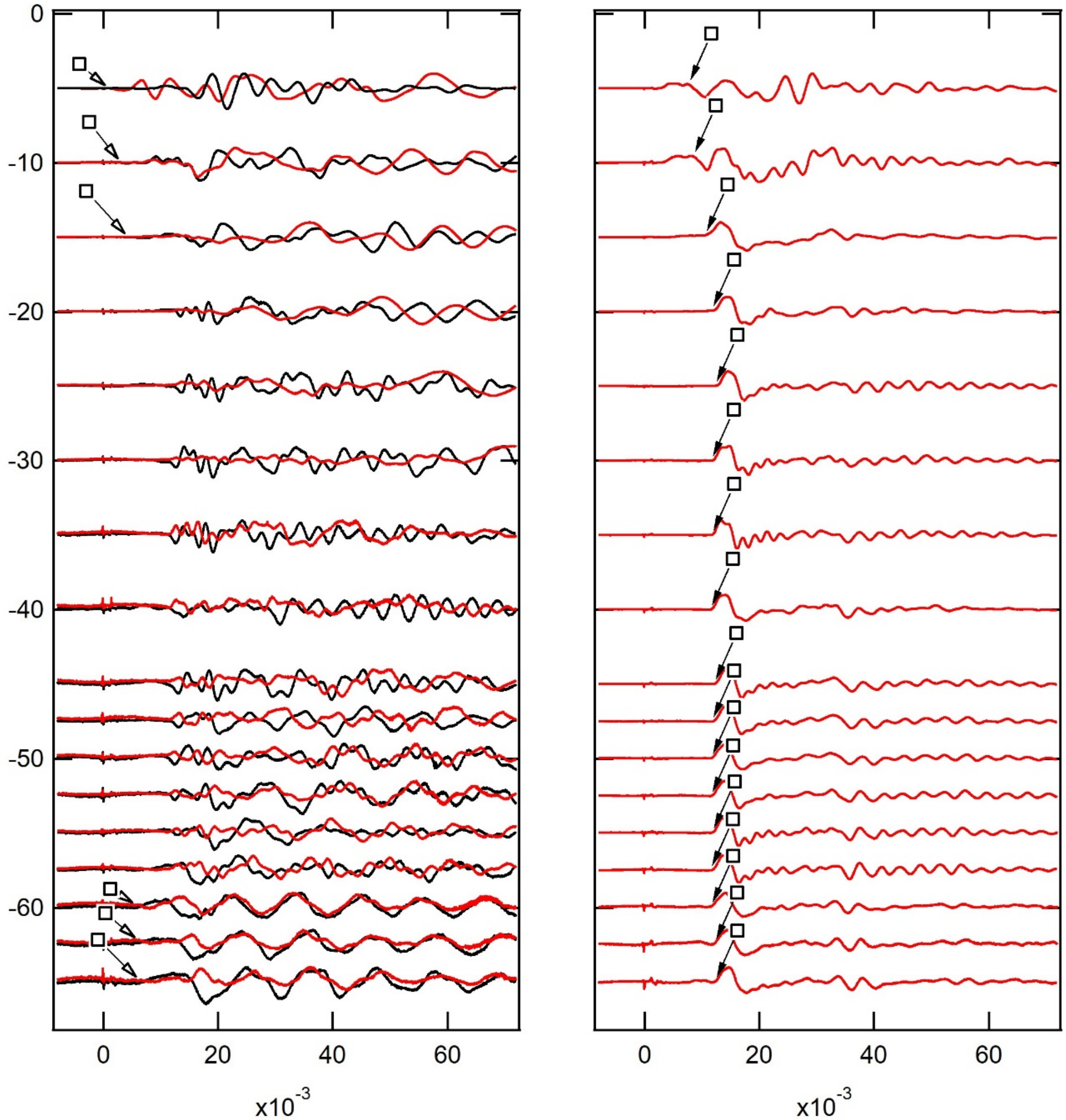


Figure E-22 Wave arrival picks at Route WW Borehole 1 – Pile 2 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

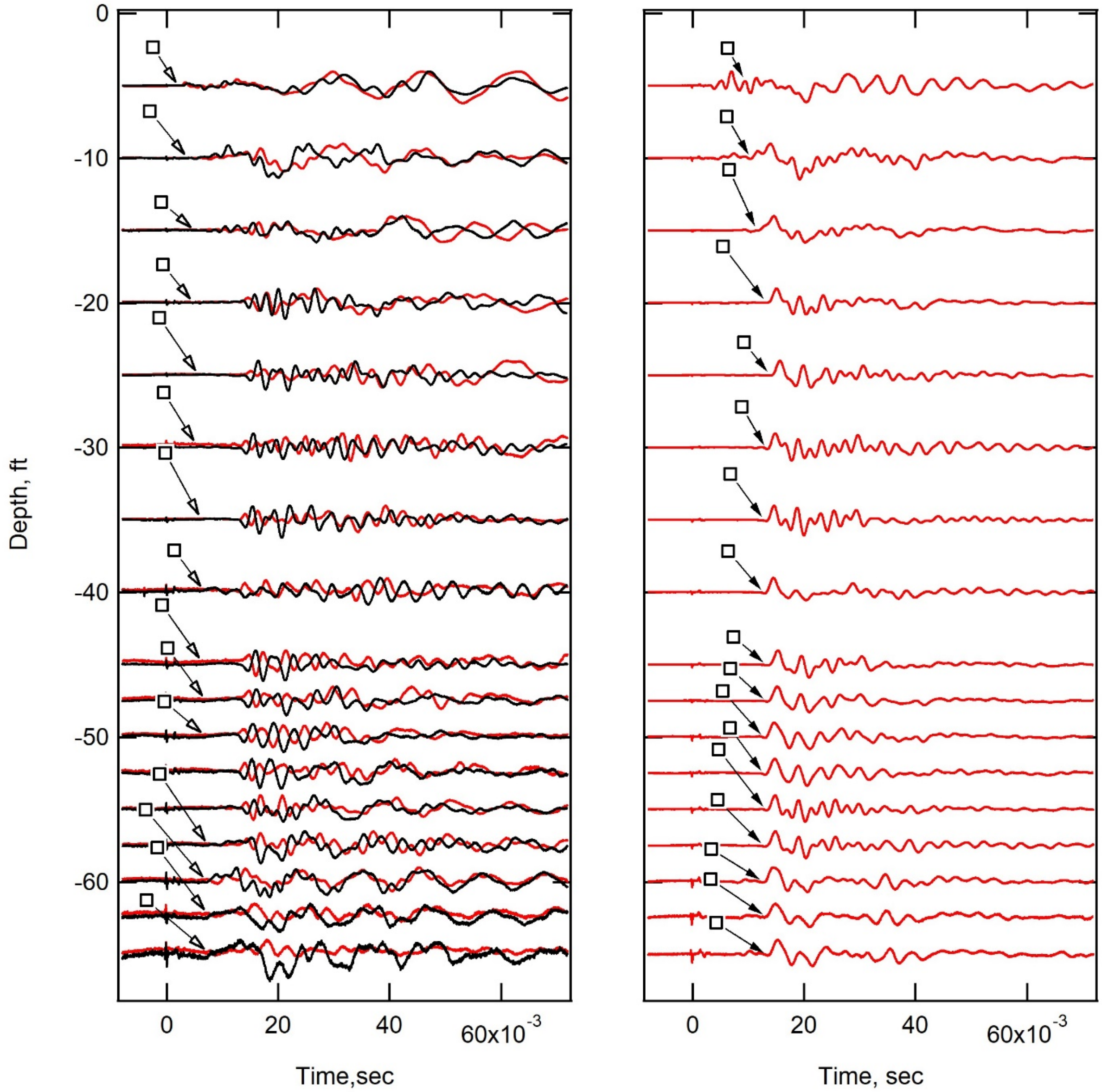


Figure E-23 Wave arrival picks at Route WW Borehole 1 – Pile 3 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

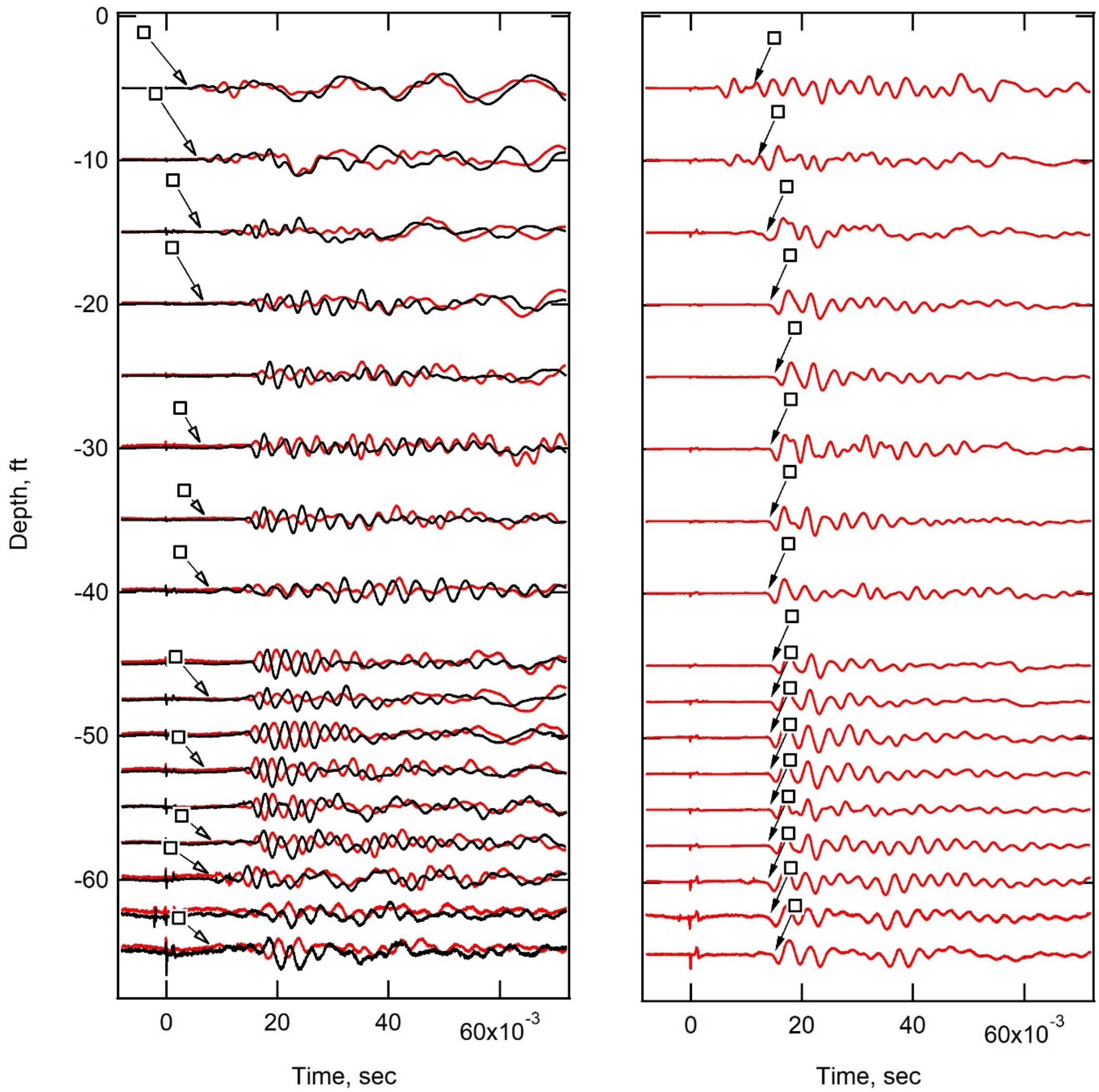


Figure E-24 Wave arrival picks at Route WW Borehole 1 – Pile 4 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

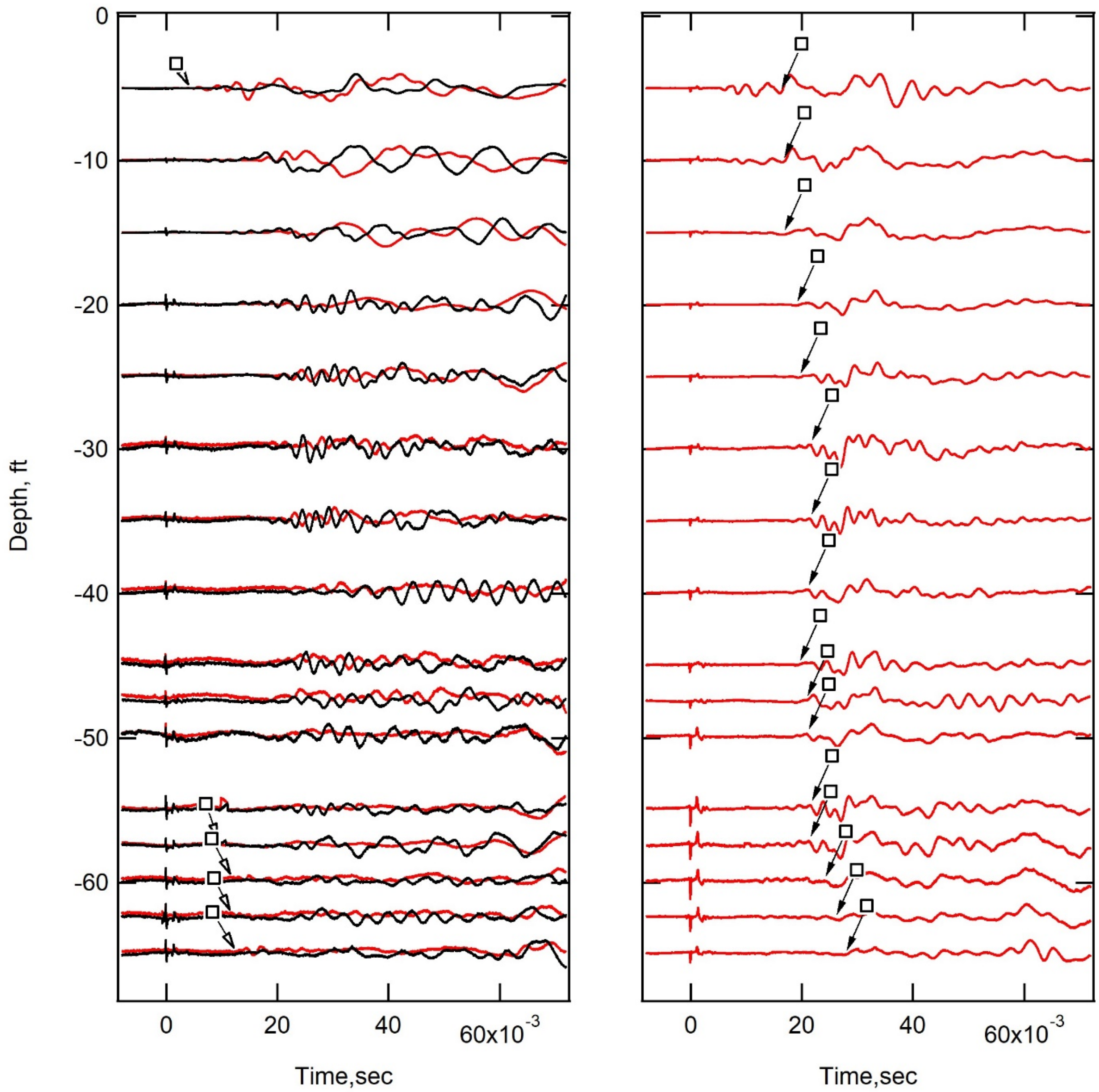


Figure E-25 Wave arrival picks at Route WW Borehole 1 – Pile 5 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

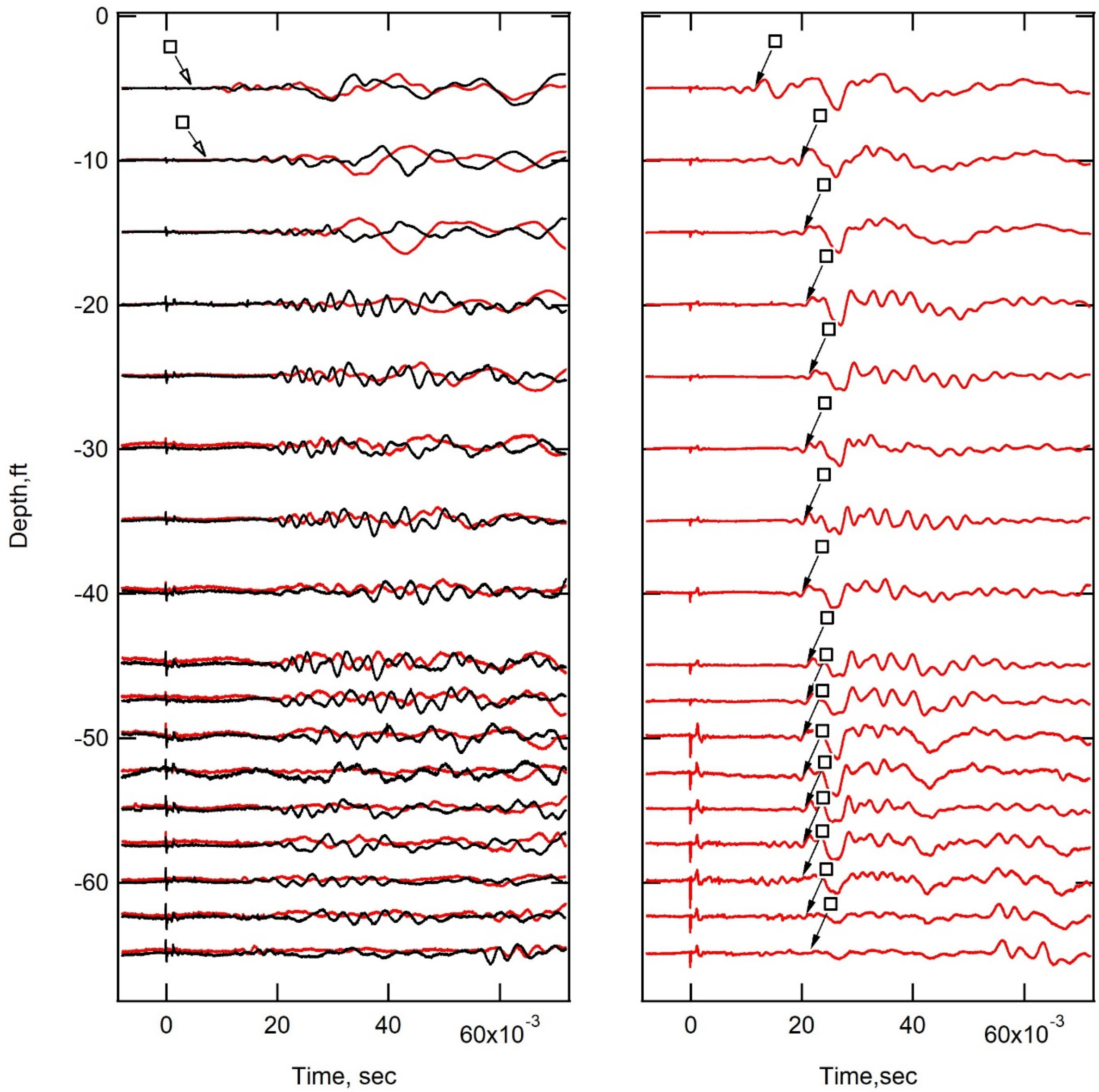


Figure E-26 Wave arrival picks at Route WW Borehole 1 – Pile 6 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

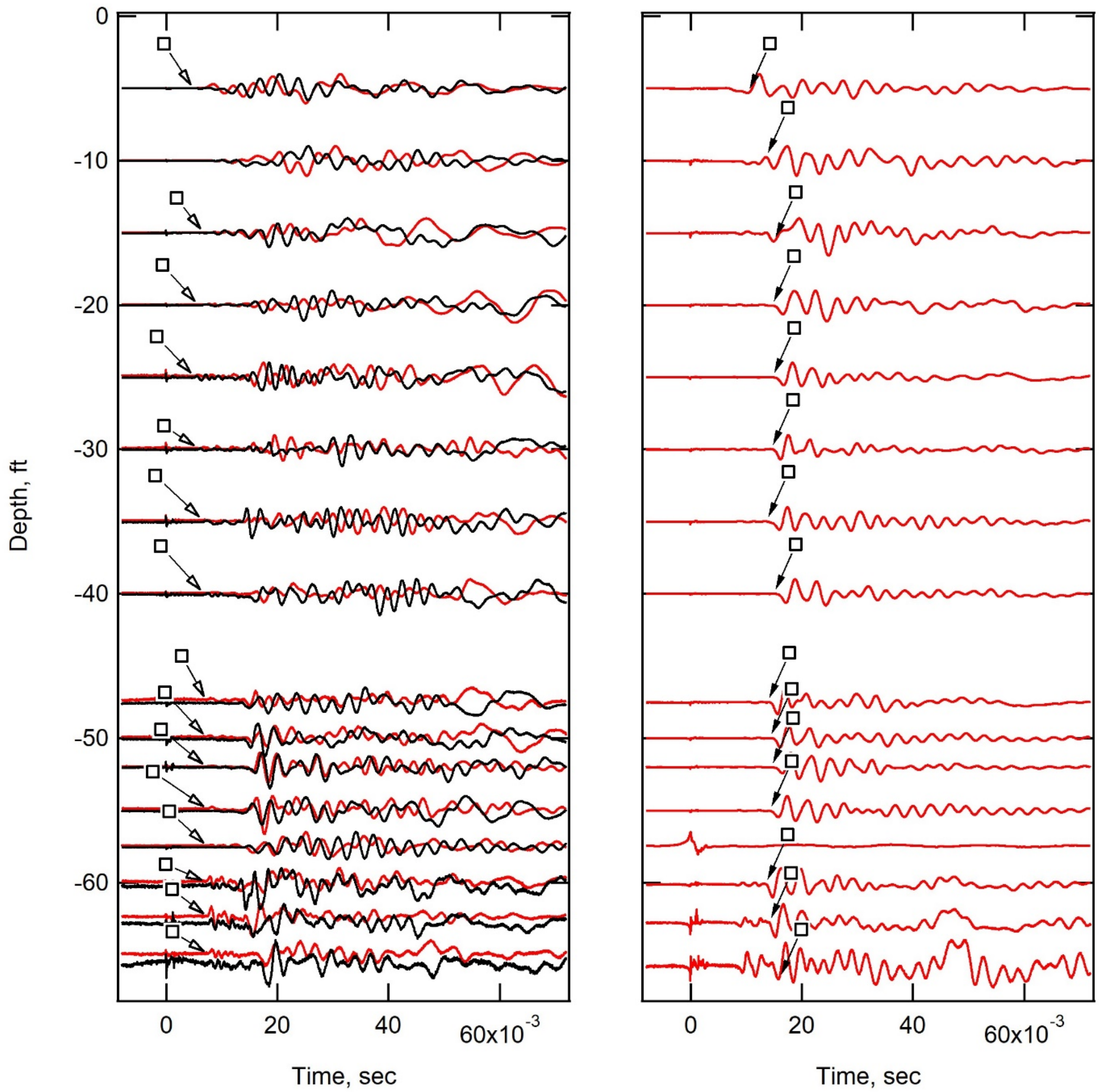


Figure E-27 Wave arrival picks at Route WW Borehole 2 – Pile 1 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

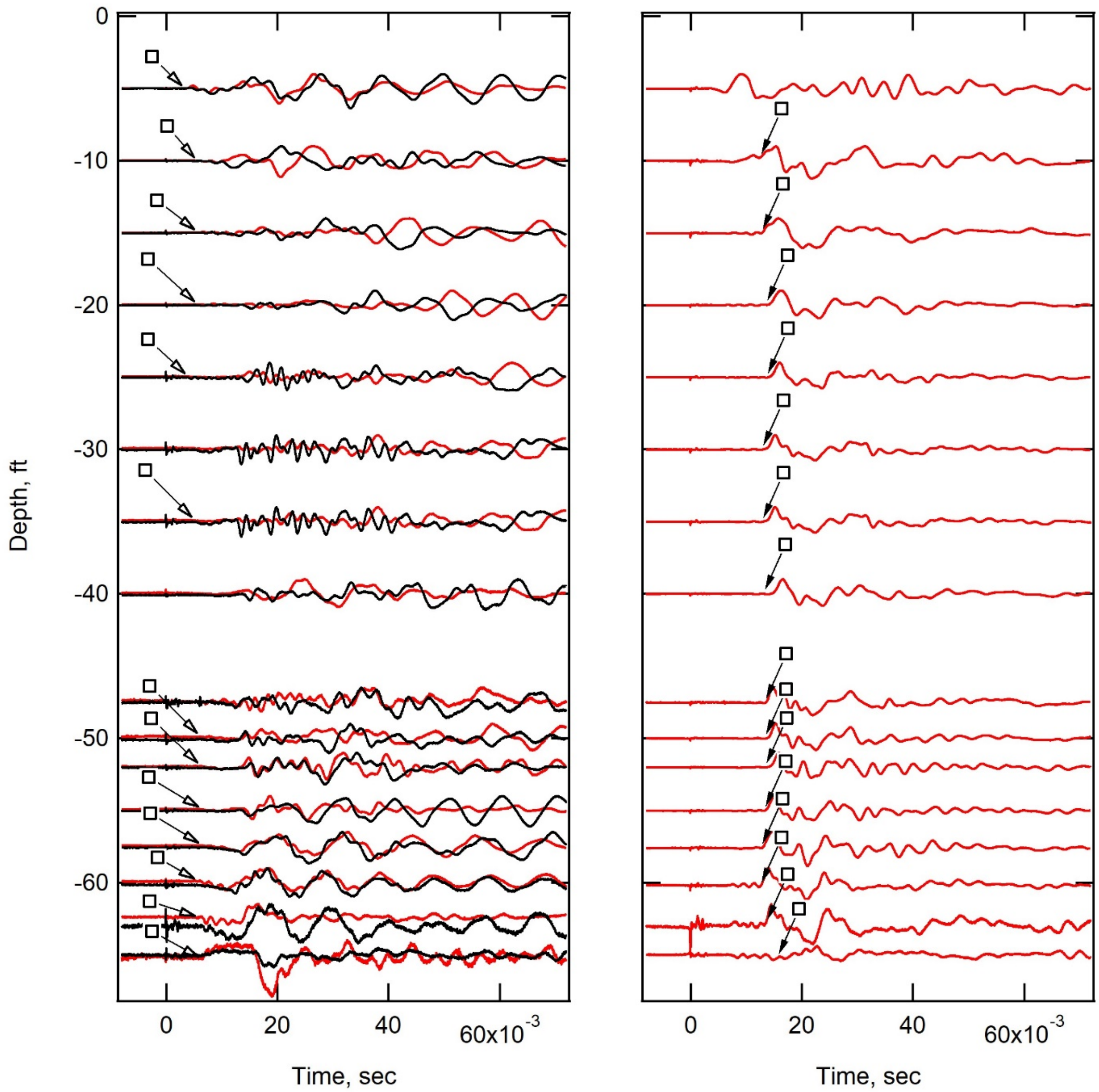


Figure E-28 Wave arrival picks at Route WW Borehole 2 – Pile 2 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

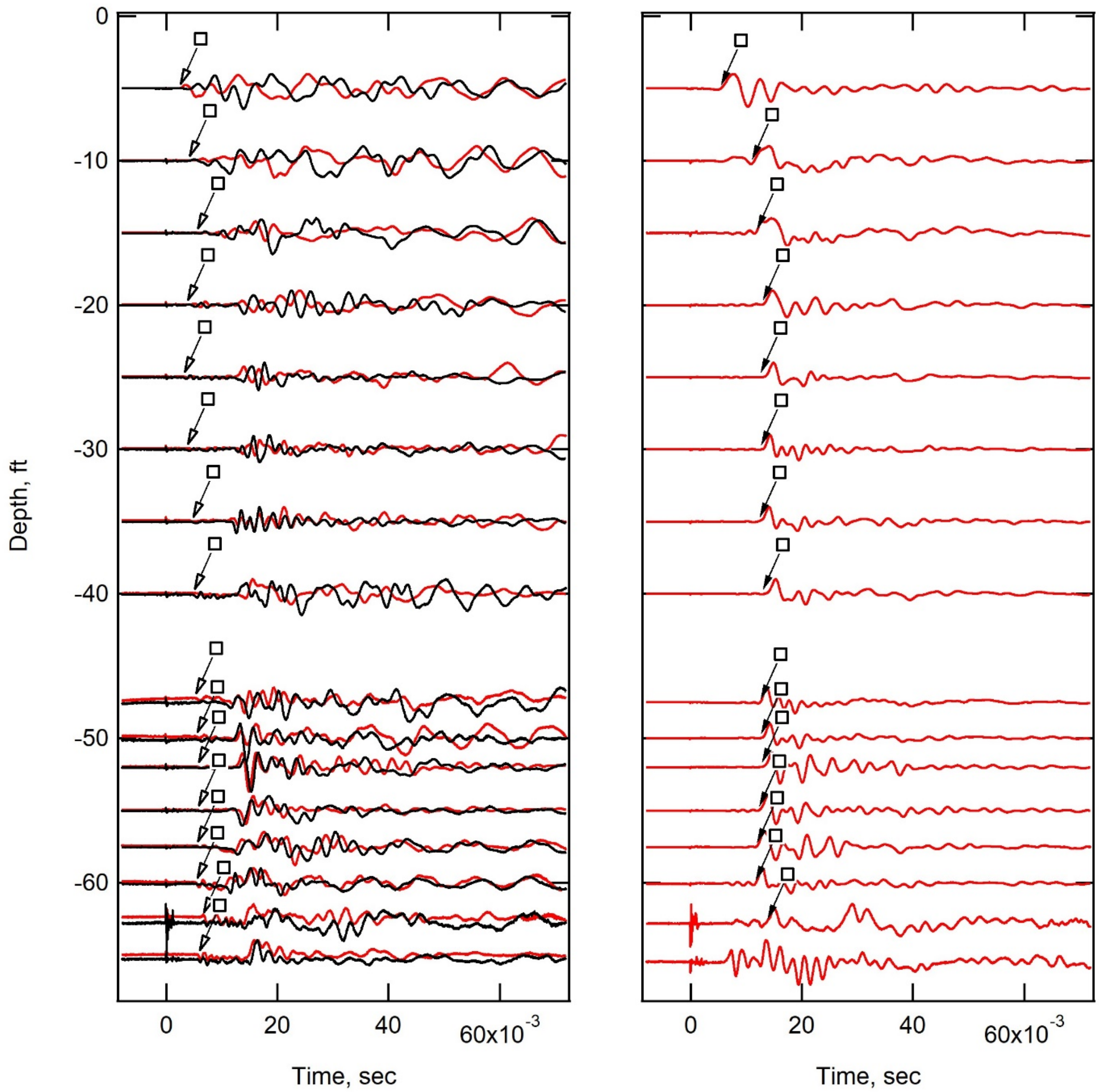


Figure E-29 Wave arrival picks at Route WW Borehole 2 – Pile 3 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

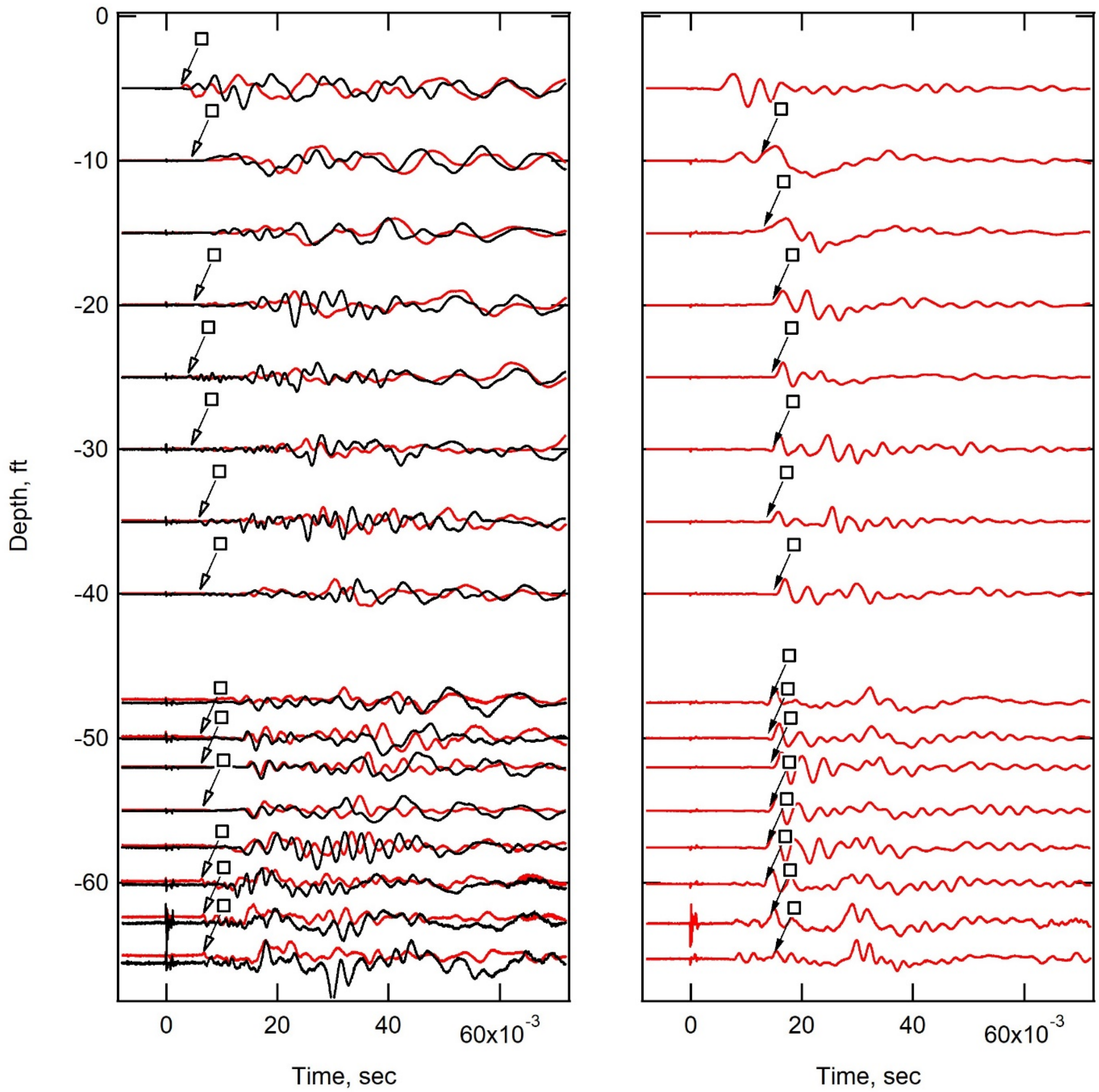


Figure E-30 Wave arrival picks at Route WW Borehole 2 – Pile 4 for p-waves (open arrow) and s-waves (solid arrow) from horizontal geophones (left) and vertical geophones (right).

Appendix F – Parallel Seismic Wave Arrival

Parallel Seismic : SCPT at Route U

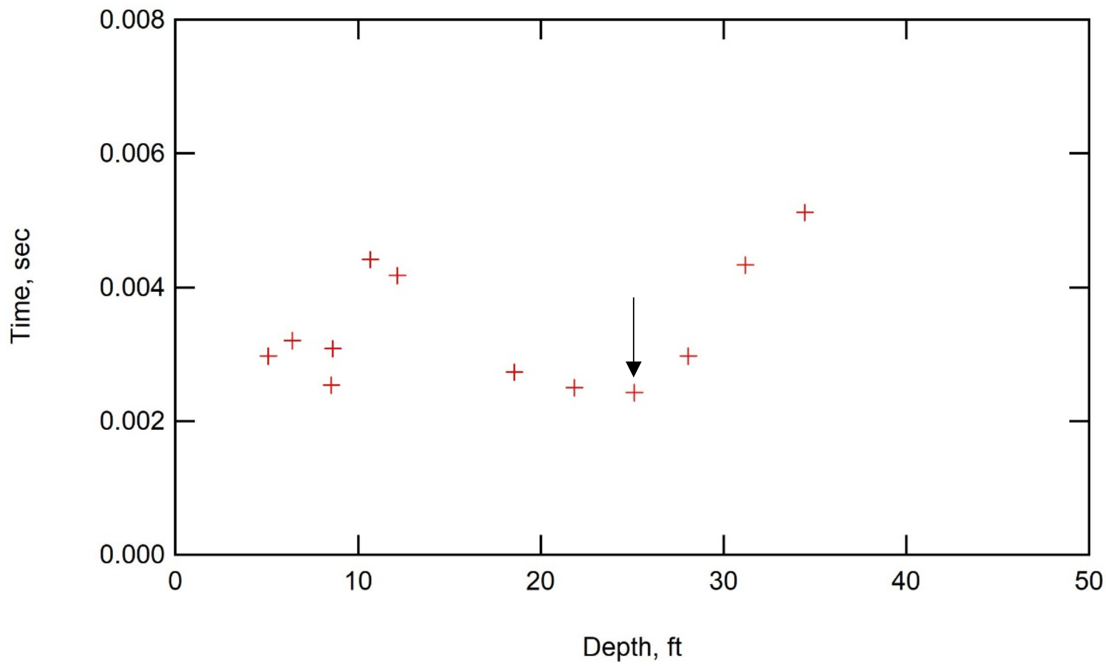


Figure F-1 Wave arrivals from p-p waves recorded with SCPT and striking horizontally on pile cap near Pile 1 at Route U. Depth interpreted from change in slope (arrow)

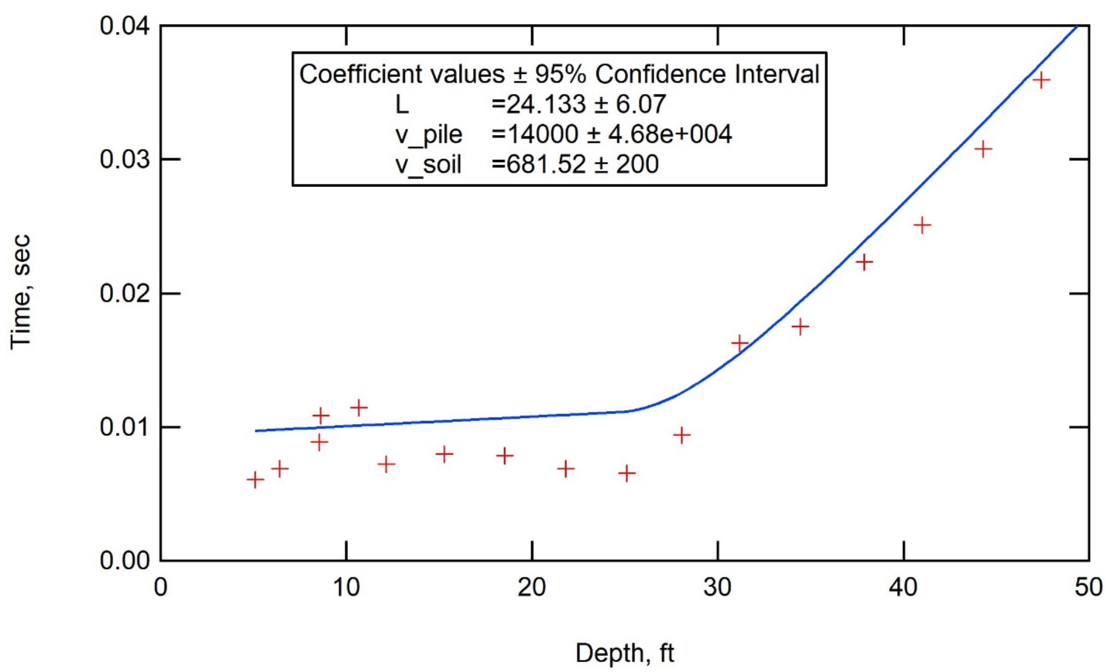


Figure F-2 Wave arrivals picks and model fit to data using p-s waves recorded with SCPT and striking vertically on bridge deck above Pile 1 at Route U.

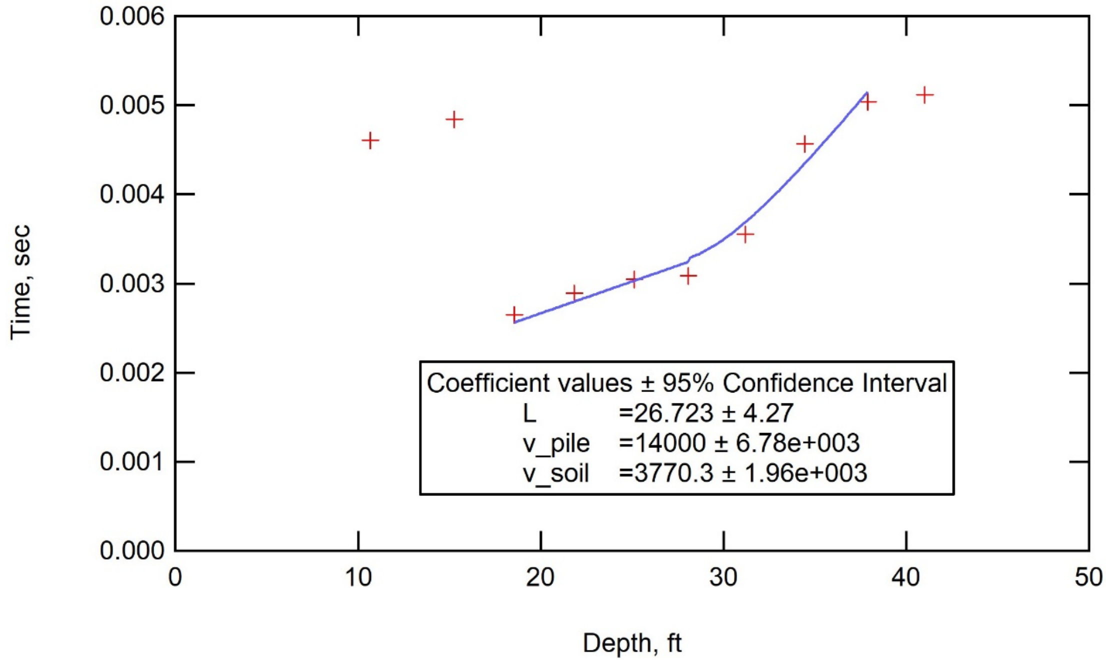


Figure F-3 Wave arrivals picks and model fit from p-p waves recorded with SCPT and striking vertically on road surface directly above Pile 2 at Route U.

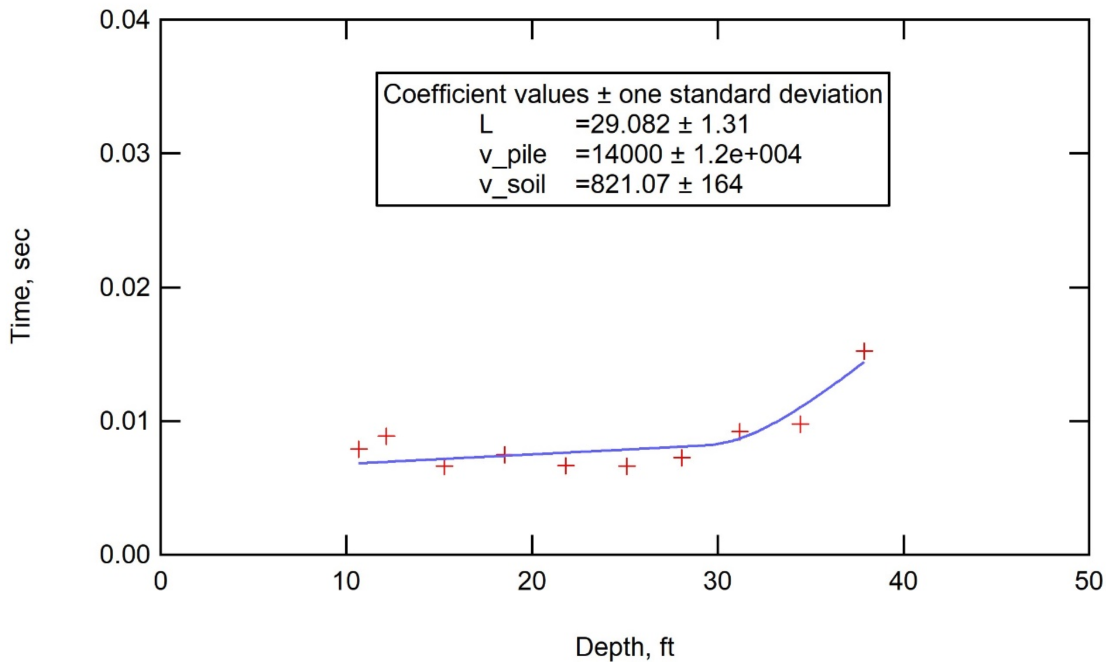


Figure F-4 Wave arrivals picks and model fit from p-s waves recorded with SCPT and striking vertically on road surface directly above Pile 2 at Route U.

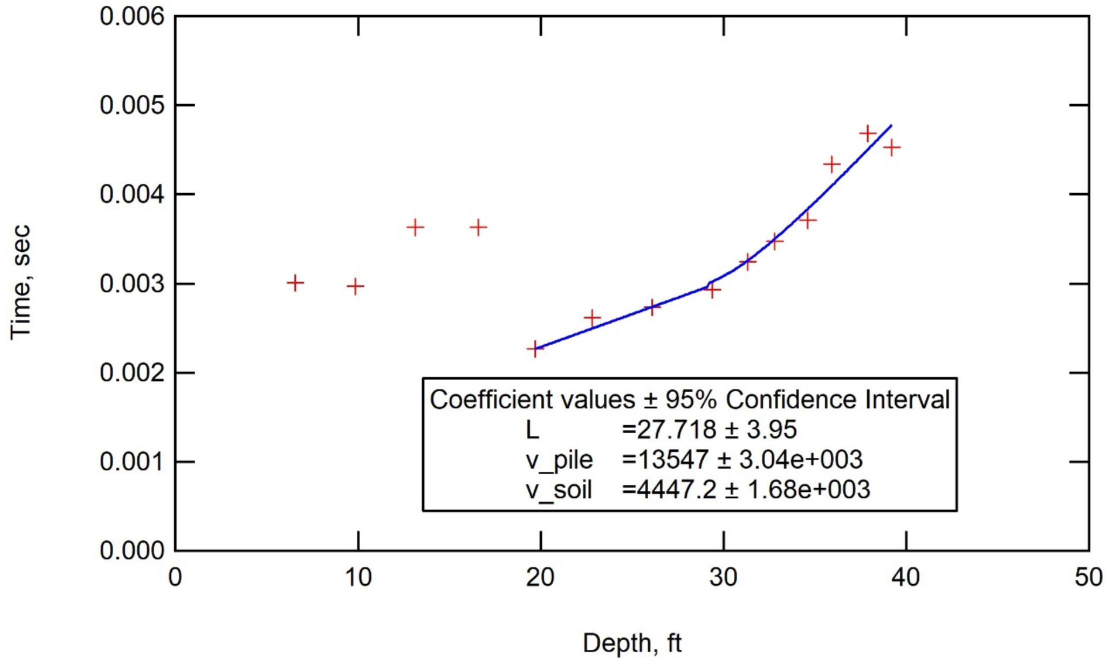


Figure F-5 Wave arrivals picks and model fit from p-p waves recorded with SCPT and striking vertically on road surface directly above Pile 3 at Route U

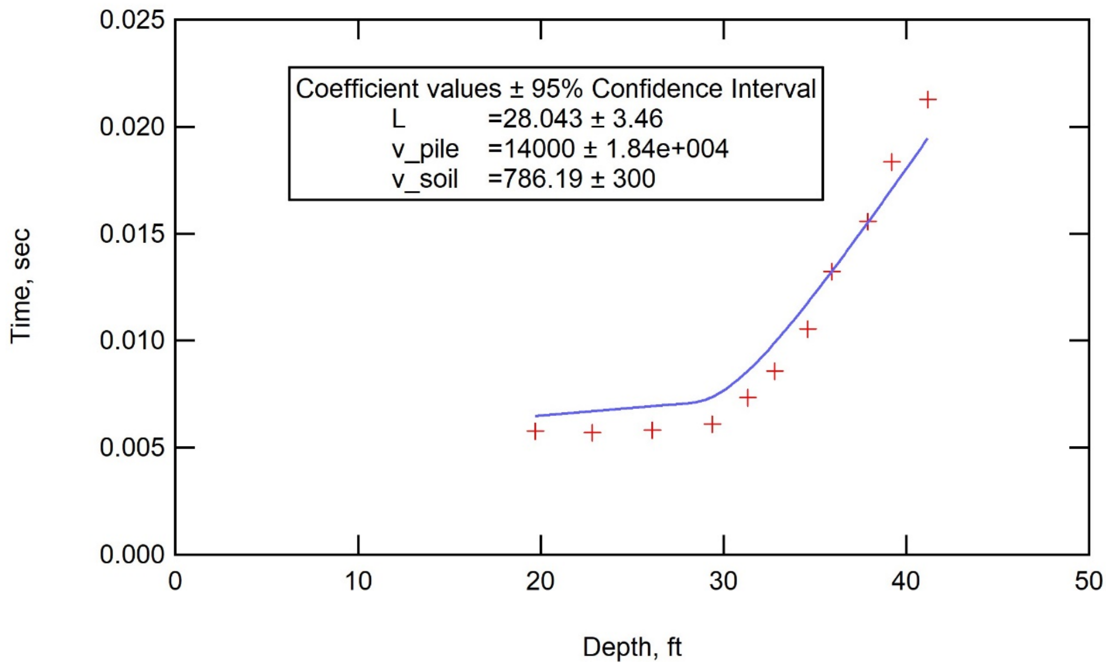


Figure F-6 Wave arrivals picks from p-s waves recorded with SCPT and striking vertically on road surface directly above Pile 3 at Route U.

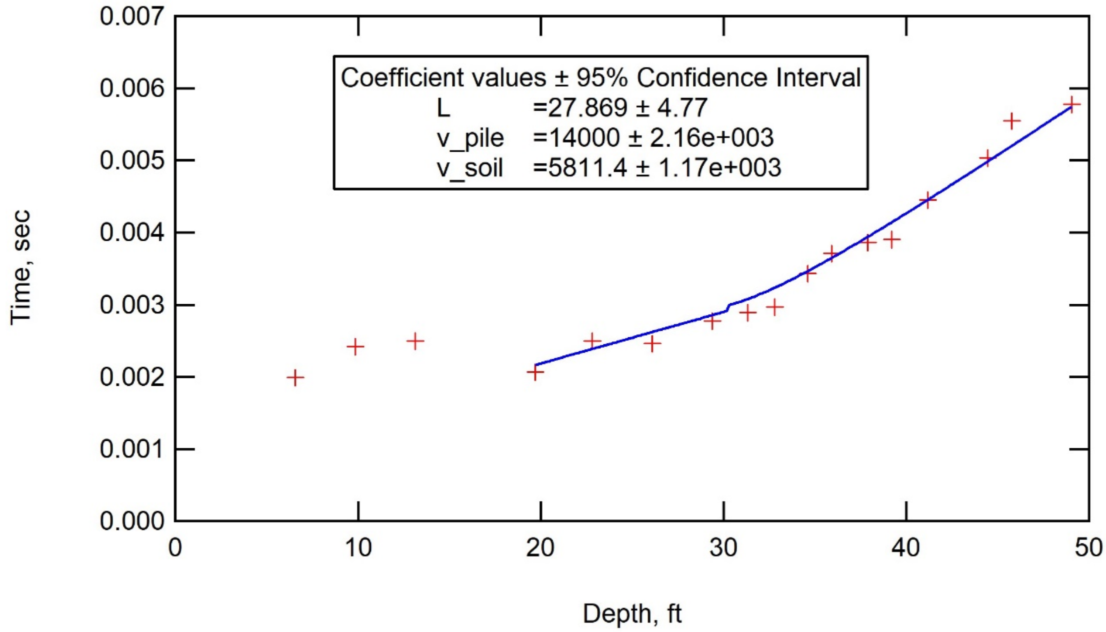


Figure F-7 Wave arrivals picks and model fit from p-p waves recorded with SCPT and striking vertically on road surface directly above Pile 4 at Route U

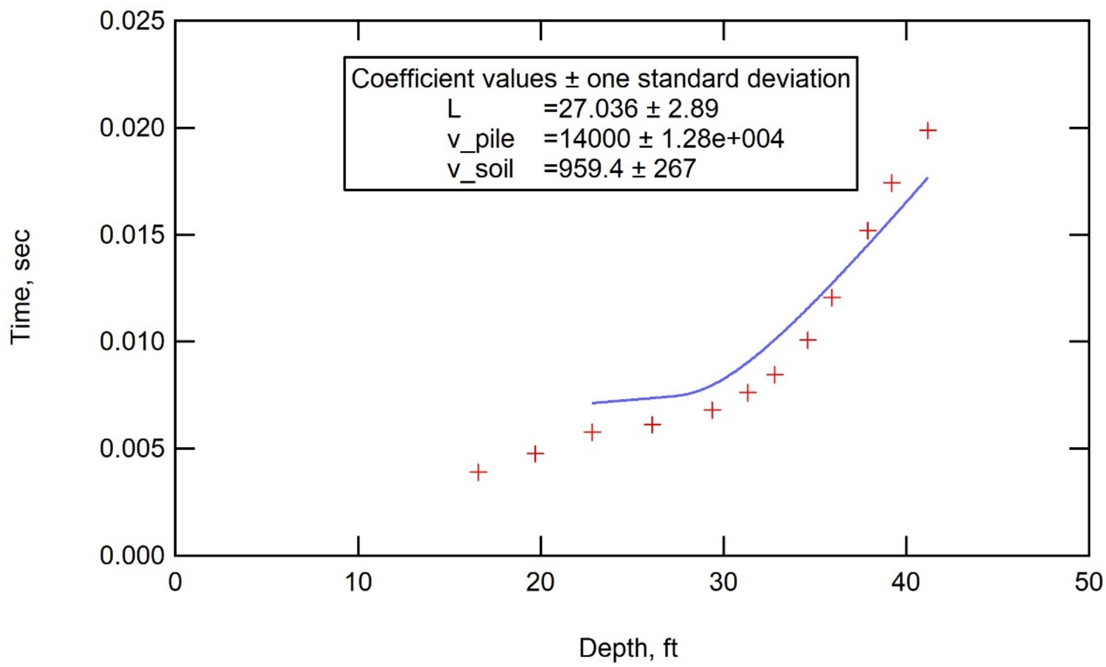


Figure F-8 Wave arrivals picks and model fit from p-s waves recorded with SCPT and striking vertically on road surface directly above Pile 4 at Route U.

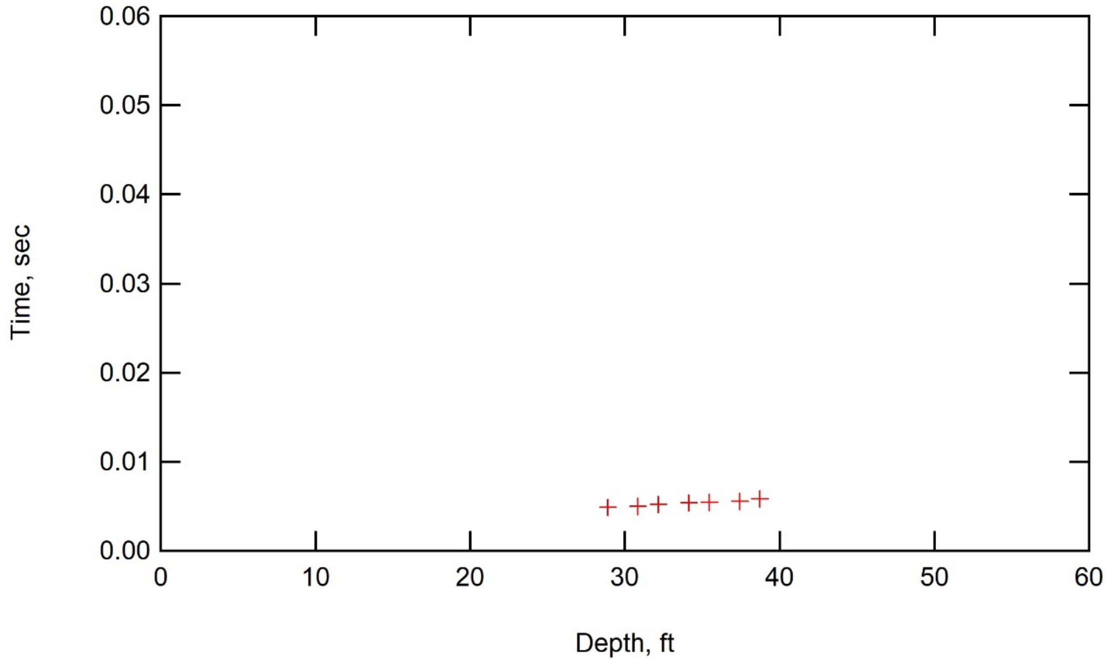


Figure F-9 Wave arrivals picks from p-p waves recorded with SCPT and striking vertically on road surface directly above Pile 5 at Route U. No fit was possible.

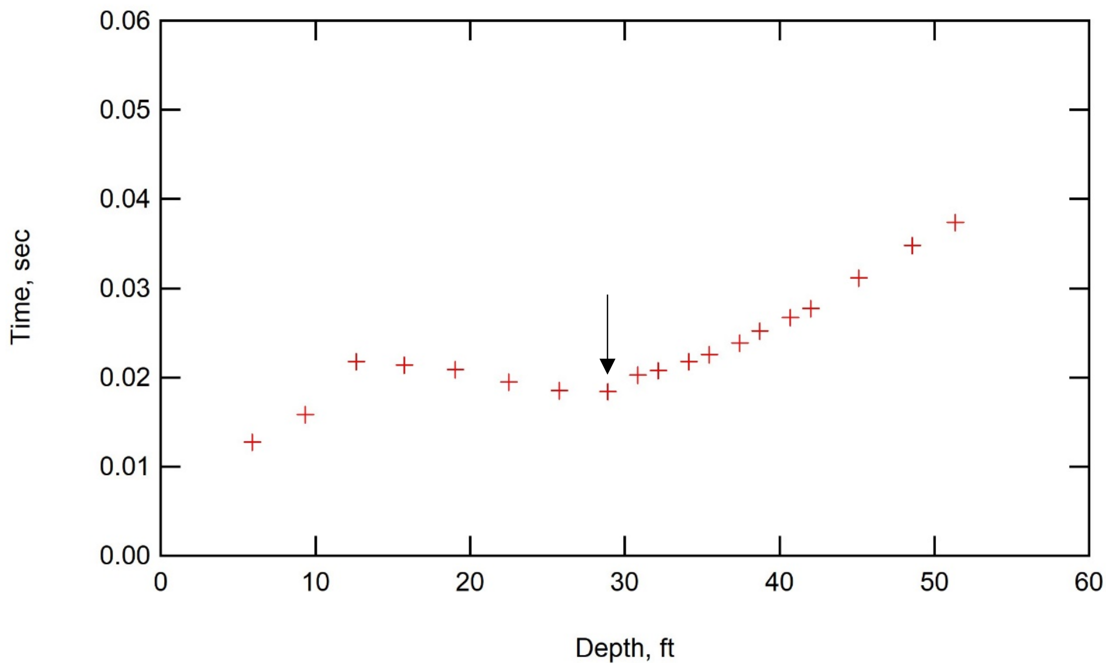


Figure F-10 Wave arrivals picks from p-s waves recorded with SCPT and striking vertically on road surface directly above Pile 5 at Route U. Depth interpreted from change in slope (arrow)

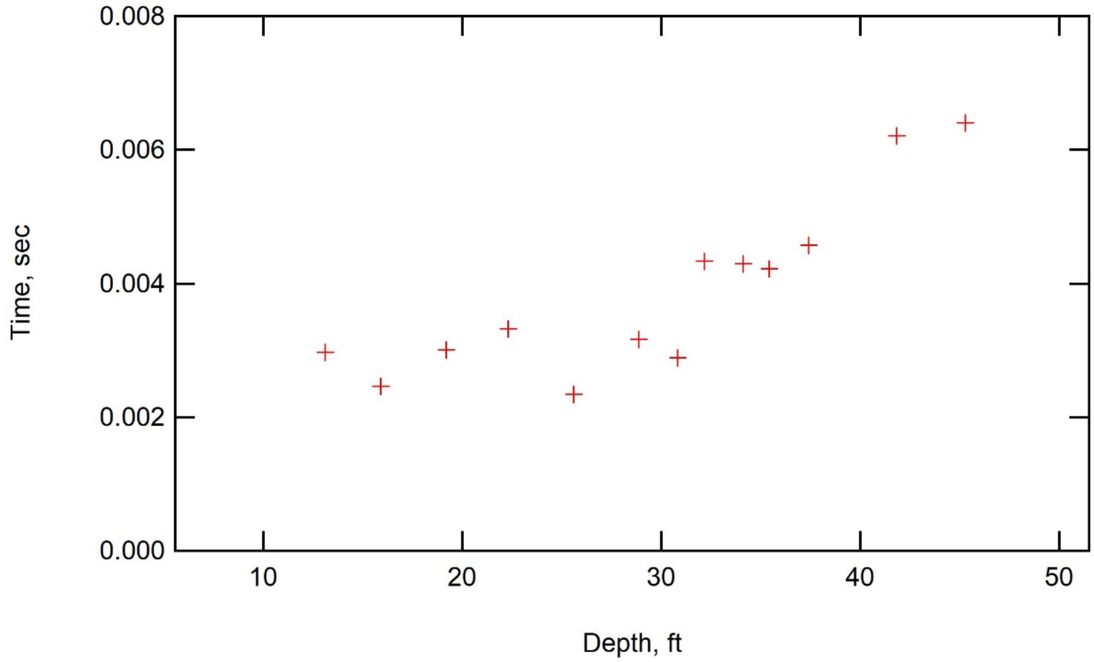


Figure F-11 Wave arrivals picks from p-p waves recorded with SCPT and striking vertically on road surface directly above Pile 8 at Route U. No fit was possible with this data.

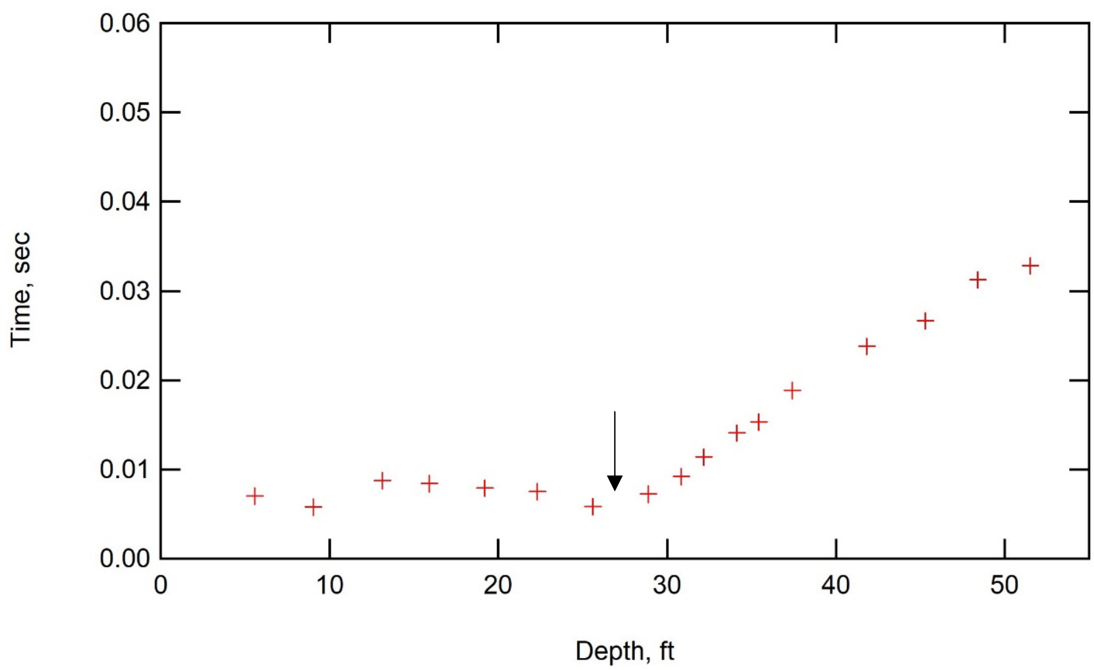


Figure F-12 Wave arrivals from p-s-waves recorded with SCPT and striking vertically on bridge deck above Pile 8 at Route U. Depth interpreted from change in slope (arrow)

Parallel Seismic : Borehole 1 at Route U

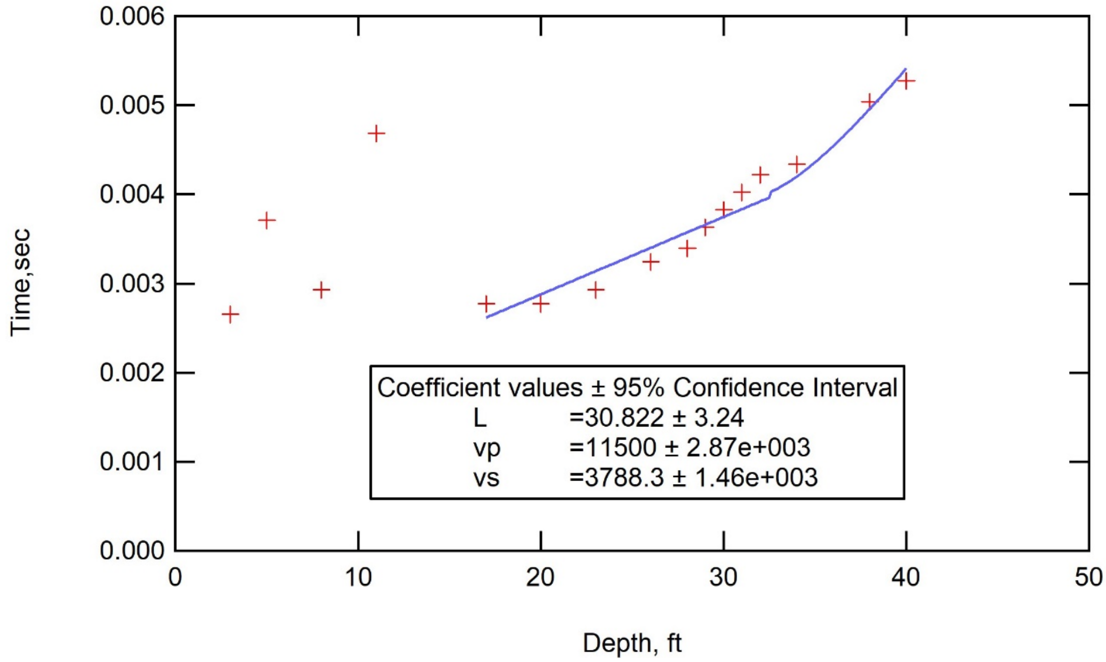


Figure F-13 Wave arrival picks and model fit using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 1.

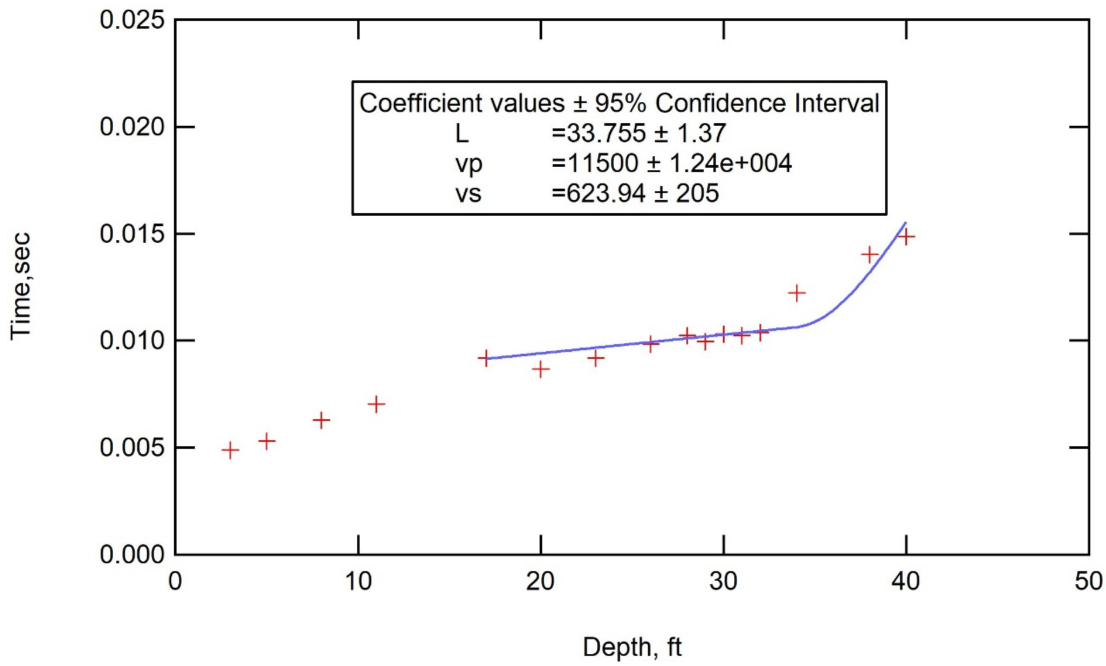


Figure F-14 Wave arrival picks and model fit using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 1.

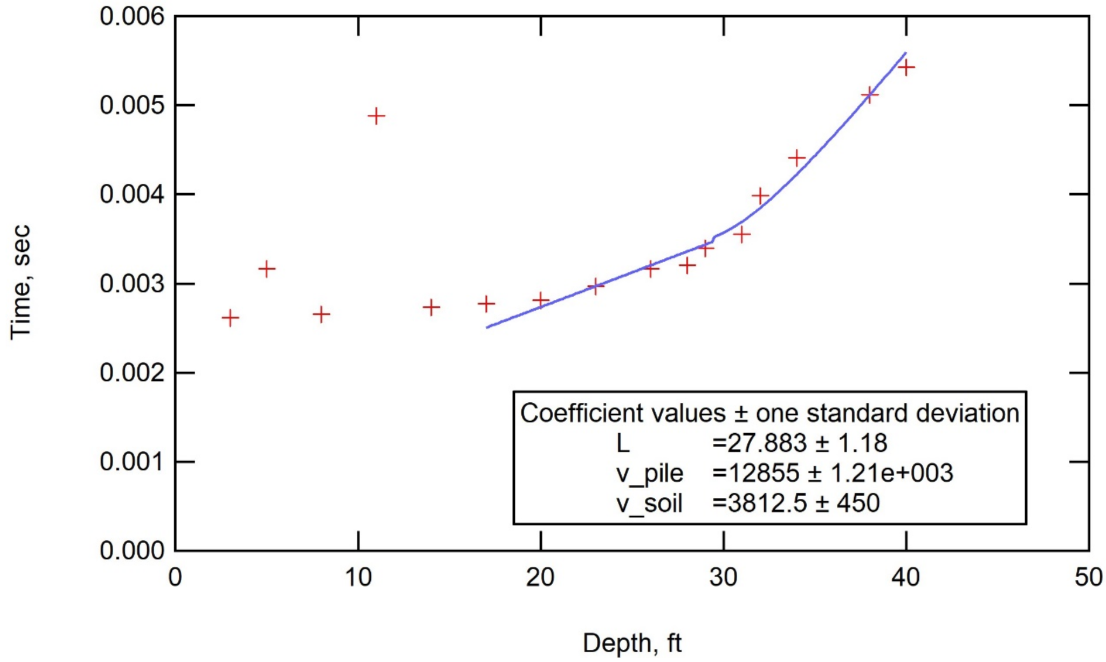


Figure F-15 Wave arrival picks and model fit using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 2.

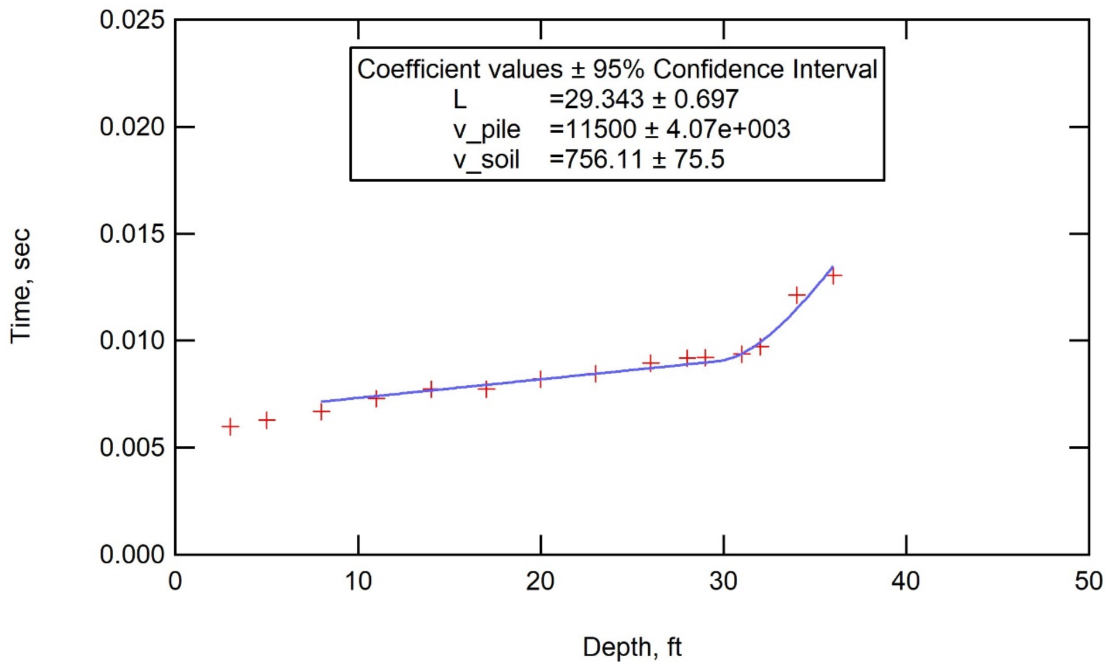


Figure F-16 Wave arrival picks and model fit using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 2.

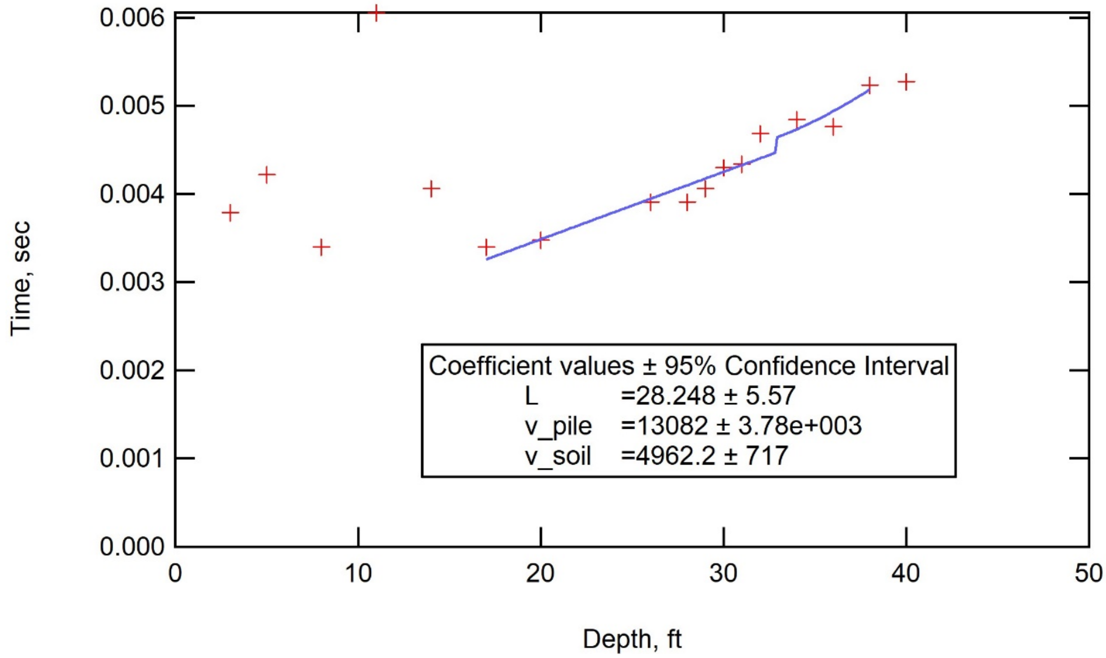


Figure F-17 Wave arrival picks and model fit using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 3

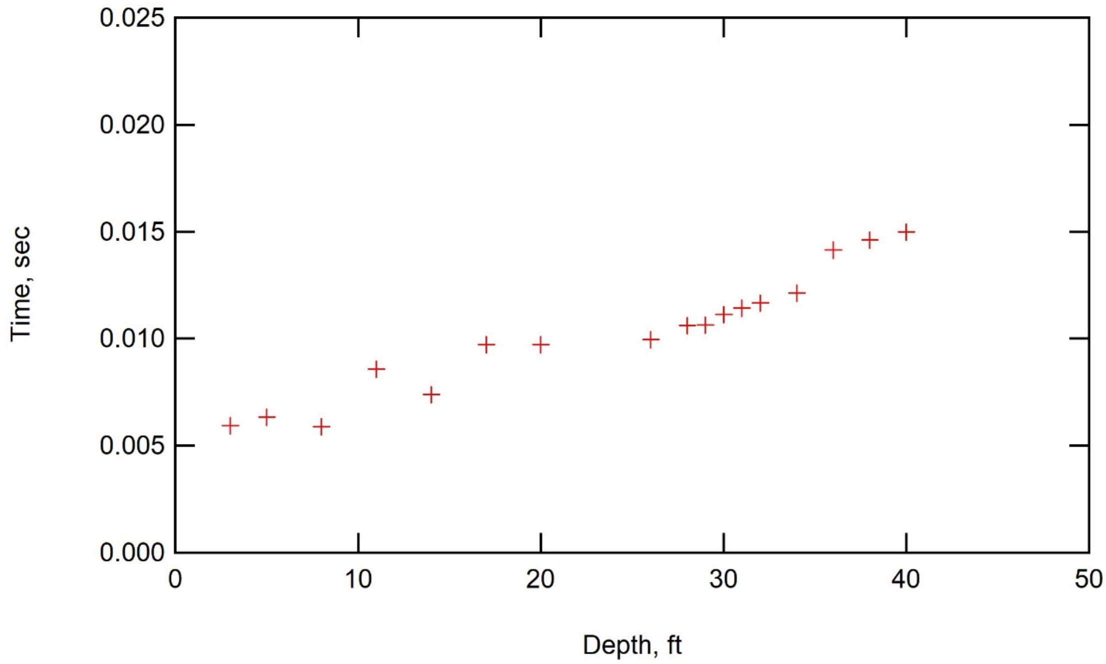


Figure F-18 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 3. No fit was possible with the data (wave arrivals are too early)

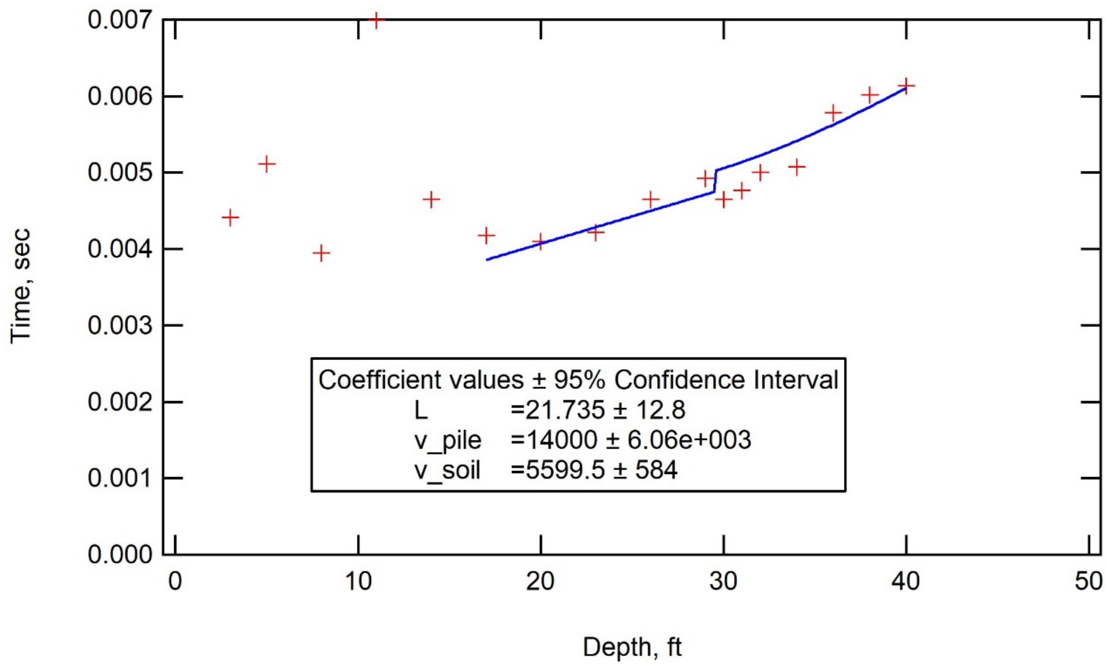


Figure F-19 Wave arrival picks and model fit using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 4.

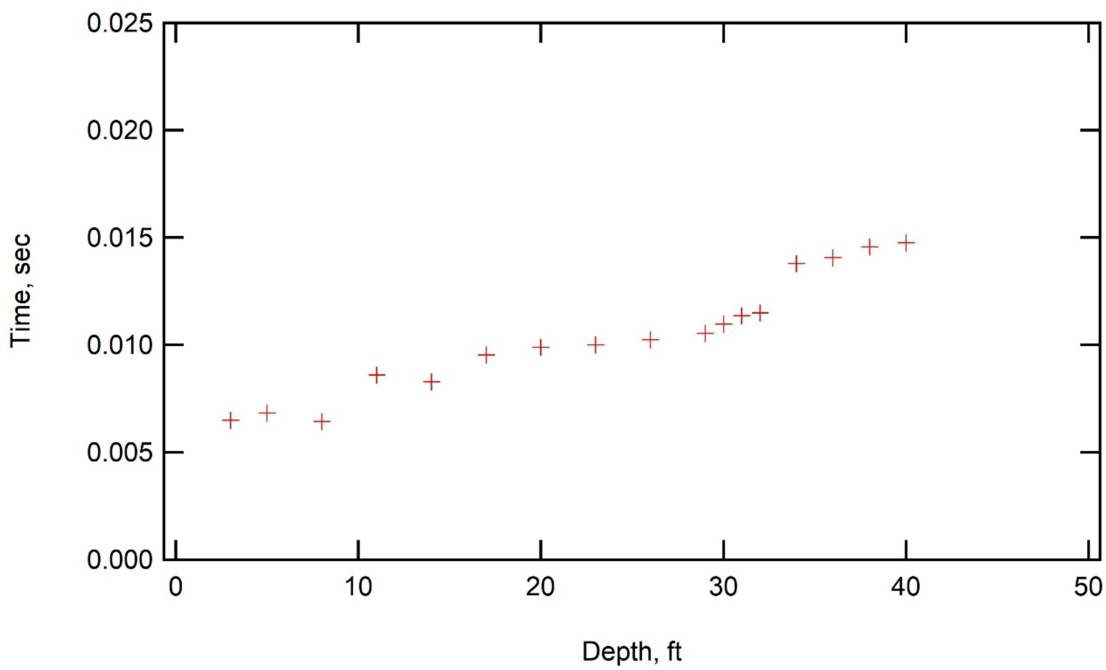


Figure F-20 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 4. No fit was possible with the data (wave arrivals are too early)

Parallel Seismic : Borehole 2 at Route U

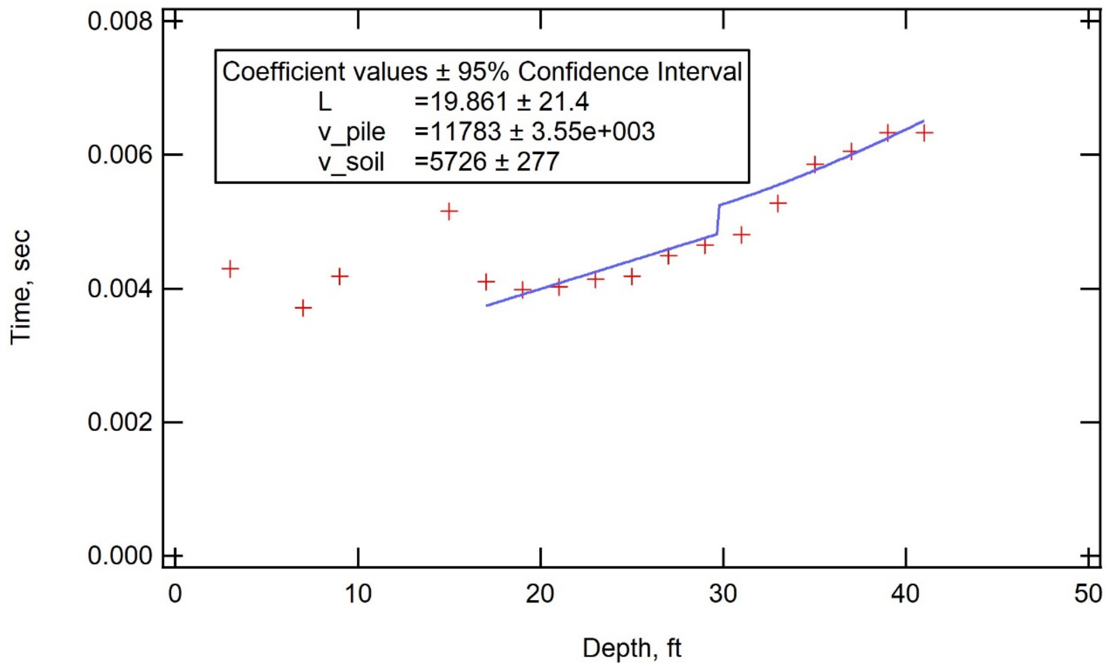


Figure F-21 Wave arrival picks and model fit using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 1

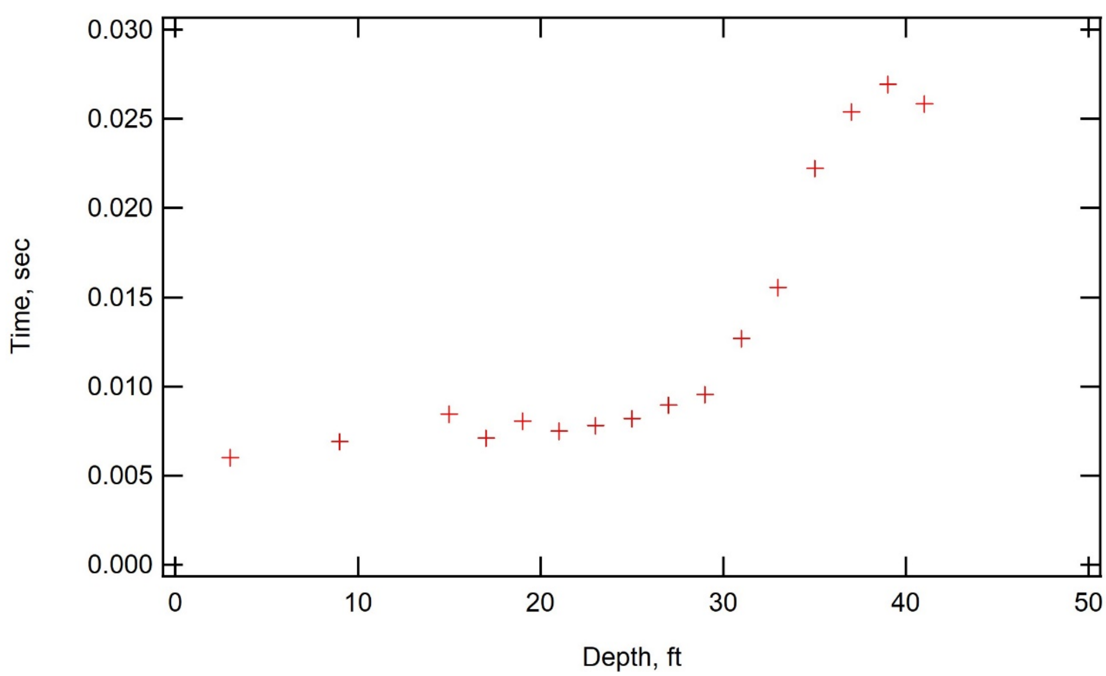


Figure F-22 Wave arrival picks using p-s waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 1. No fit was possible with the data (wave arrivals are too early).

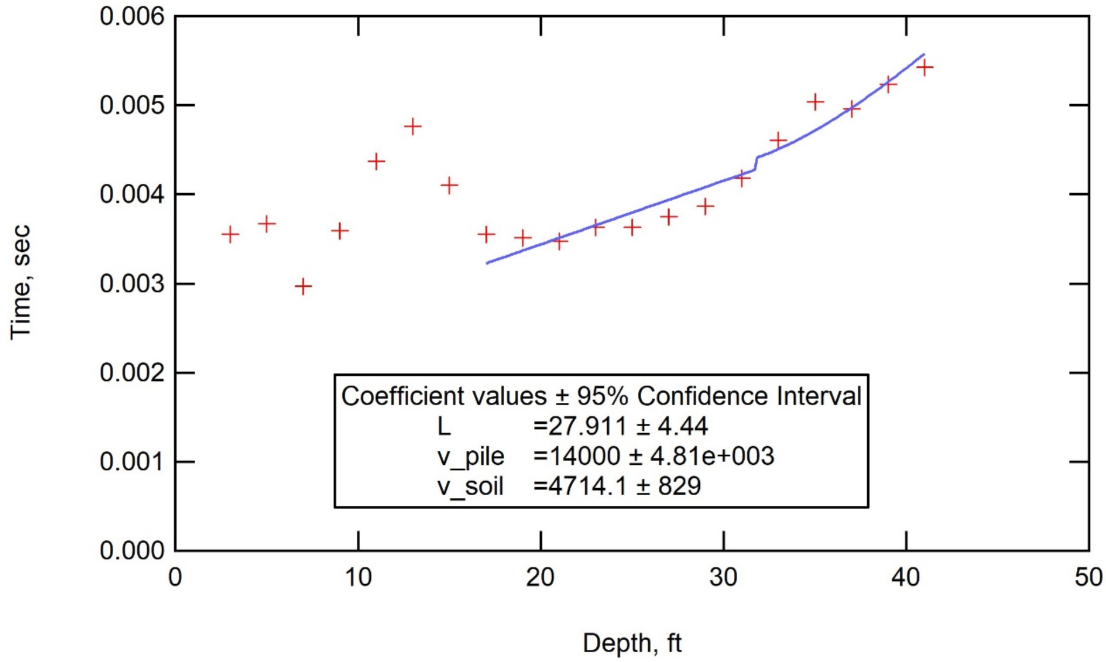


Figure F-23 Wave arrival picks and model fit using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 2.

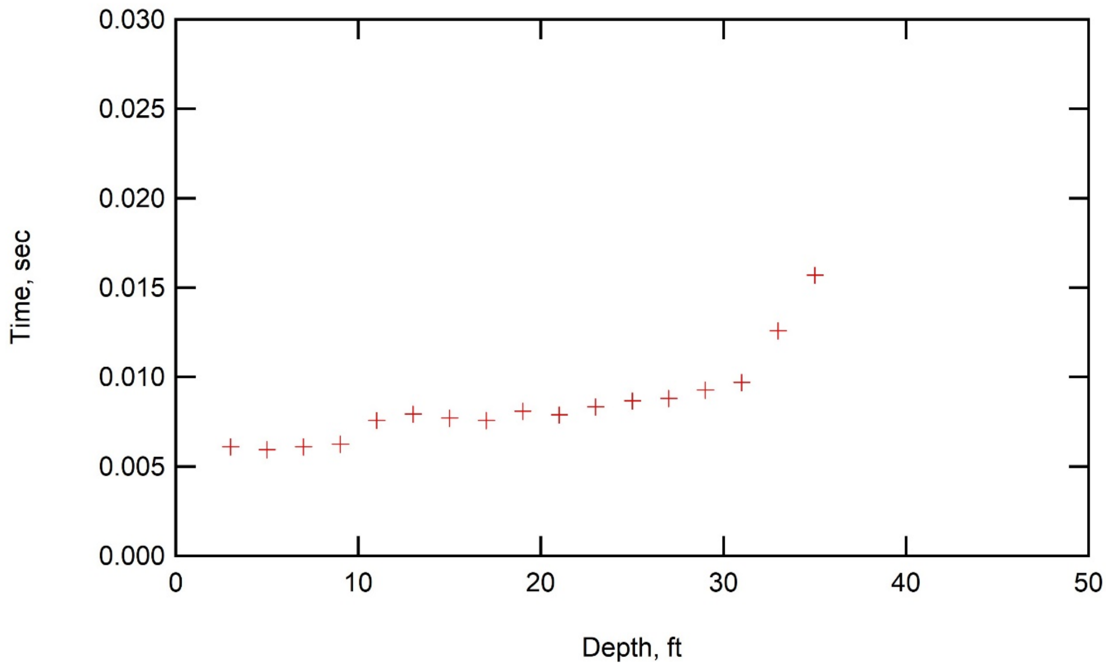


Figure F-24 Wave arrival picks using p-s waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 2. No fit was possible with the data (wave arrivals are too early)

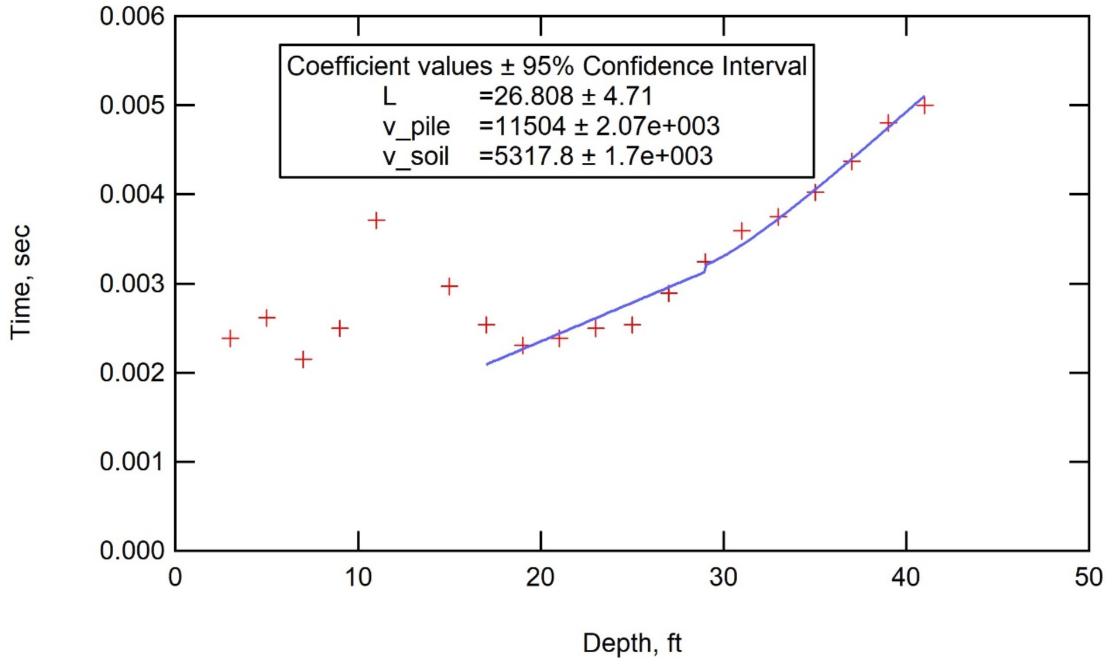


Figure F-25 Wave arrival picks and model fit using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 3.

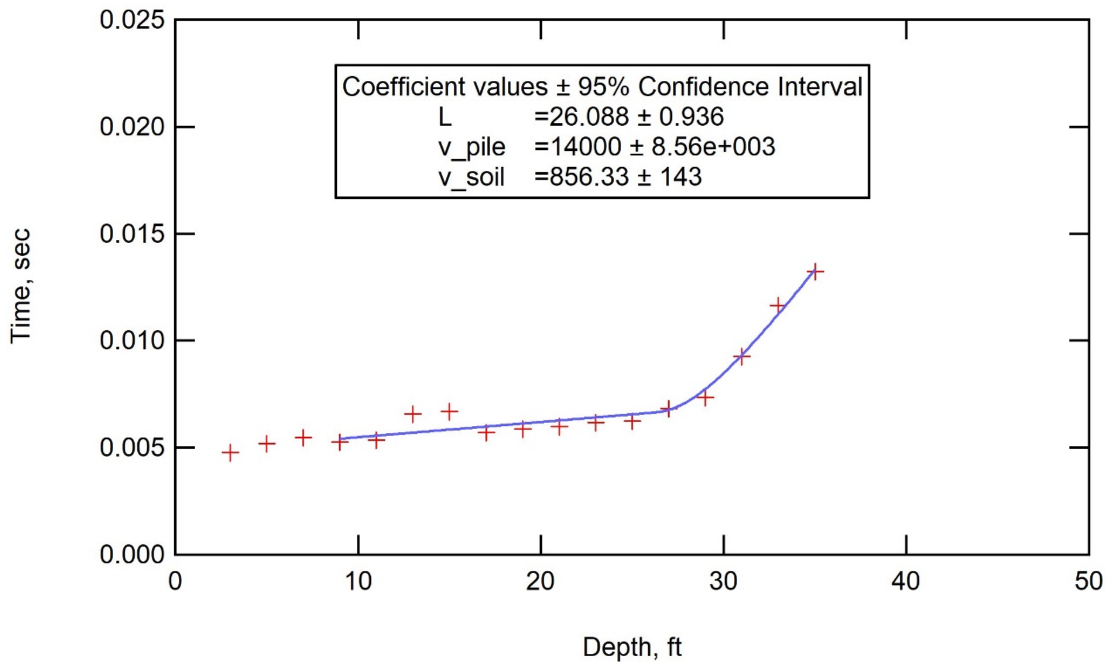


Figure F-26 Wave arrival picks and model fit using p-s waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 3.

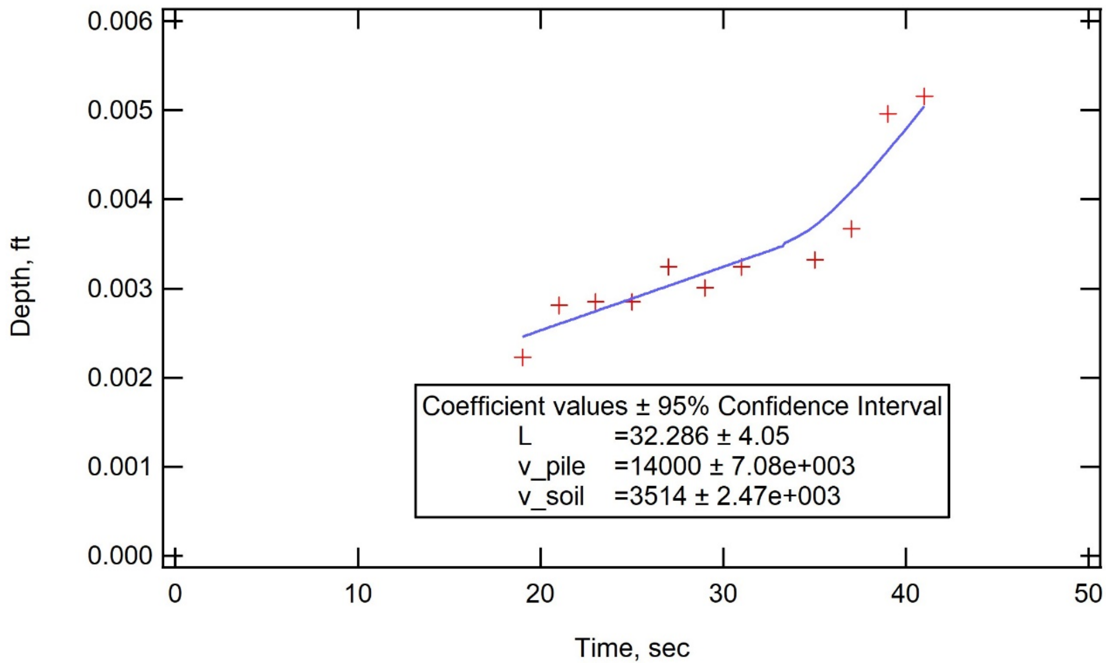


Figure F-27 Wave arrival picks and model fit using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 4.

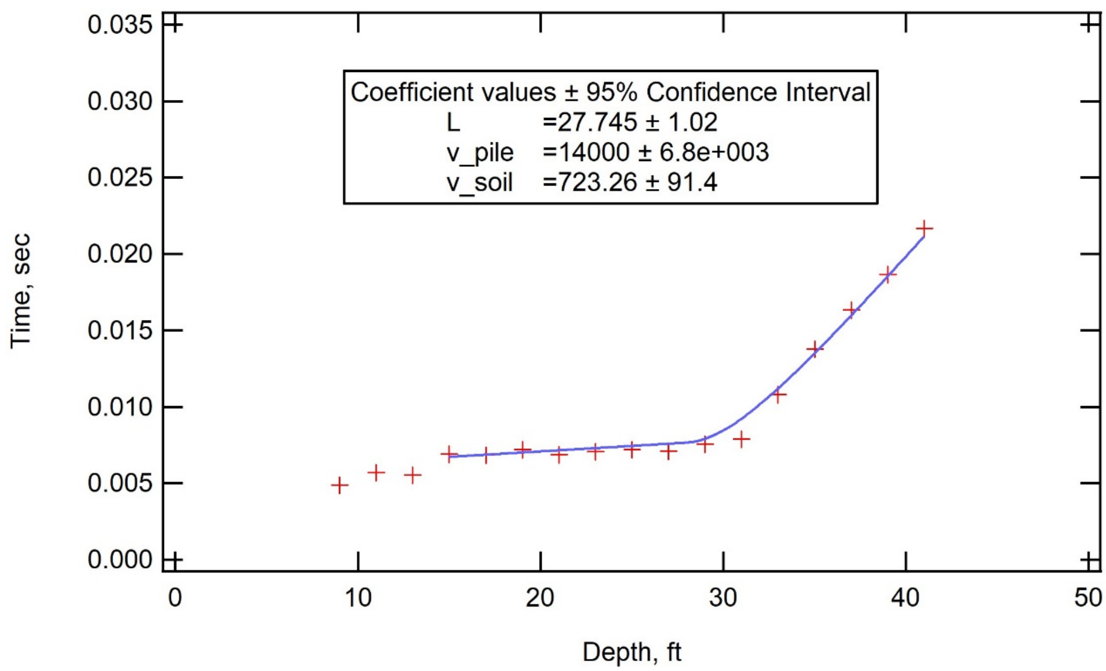


Figure F-28 Wave arrival picks and model fit using p-s waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 4.

Parallel Seismic : SCPT at Route WW

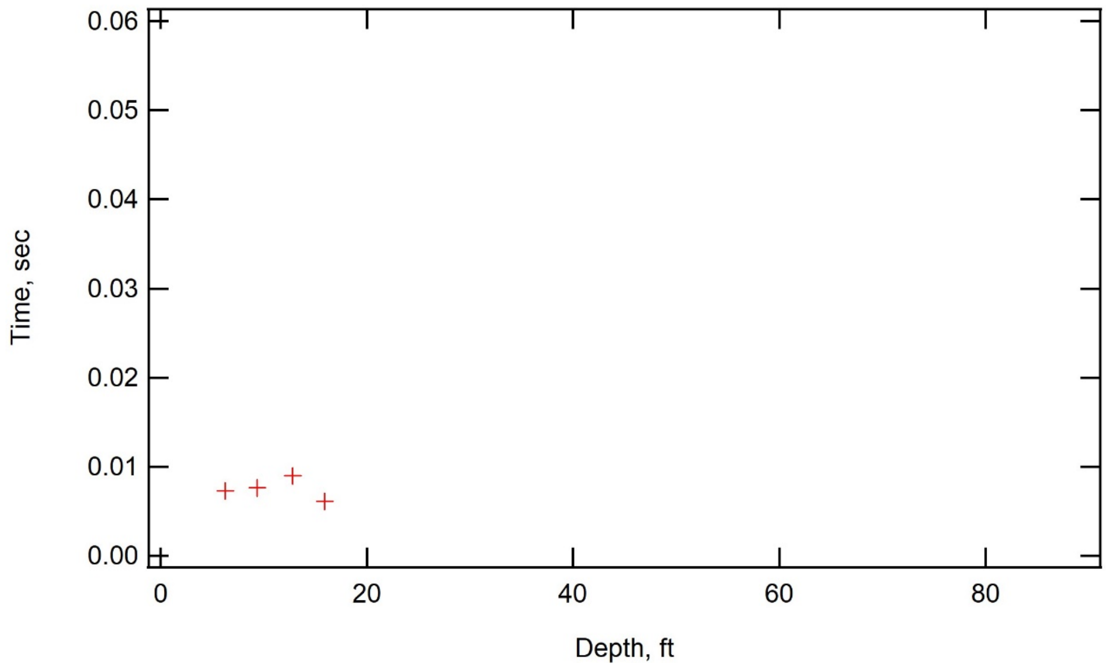


Figure F-29 Wave arrivals picks to data using p-p waves recorded with SCPT and striking vertically on bridge deck above Pile 1 at Route WW. No fit was possible with this data.

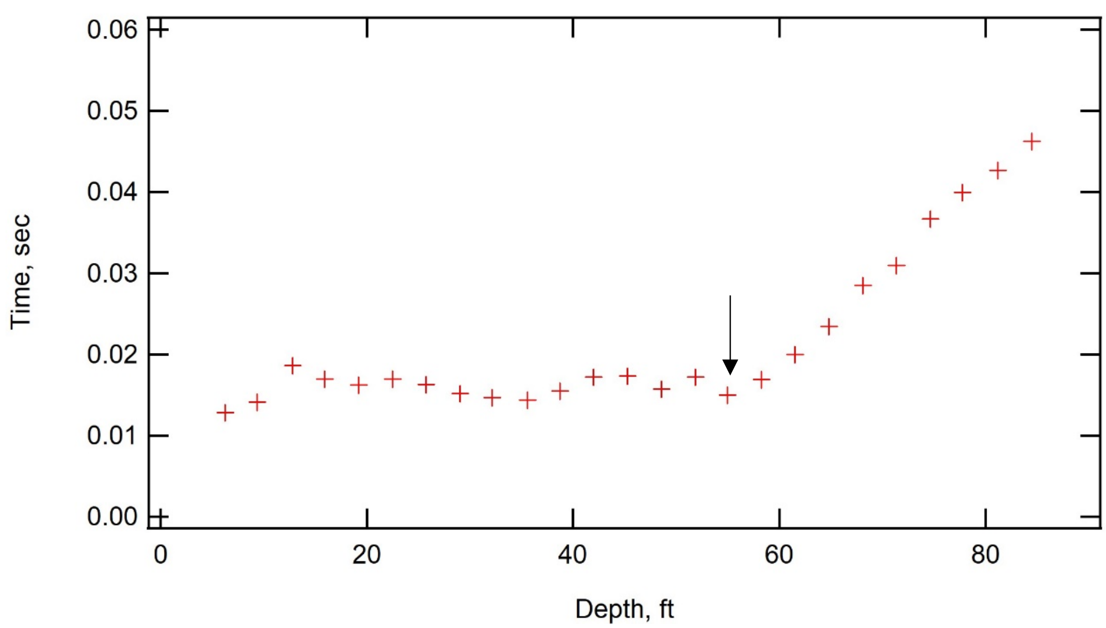


Figure F-30 Wave arrivals picks to data using p-s waves recorded with SCPT and striking vertically on bridge deck above Pile 1 at Route WW. Depth interpreted from change in slope (arrow)

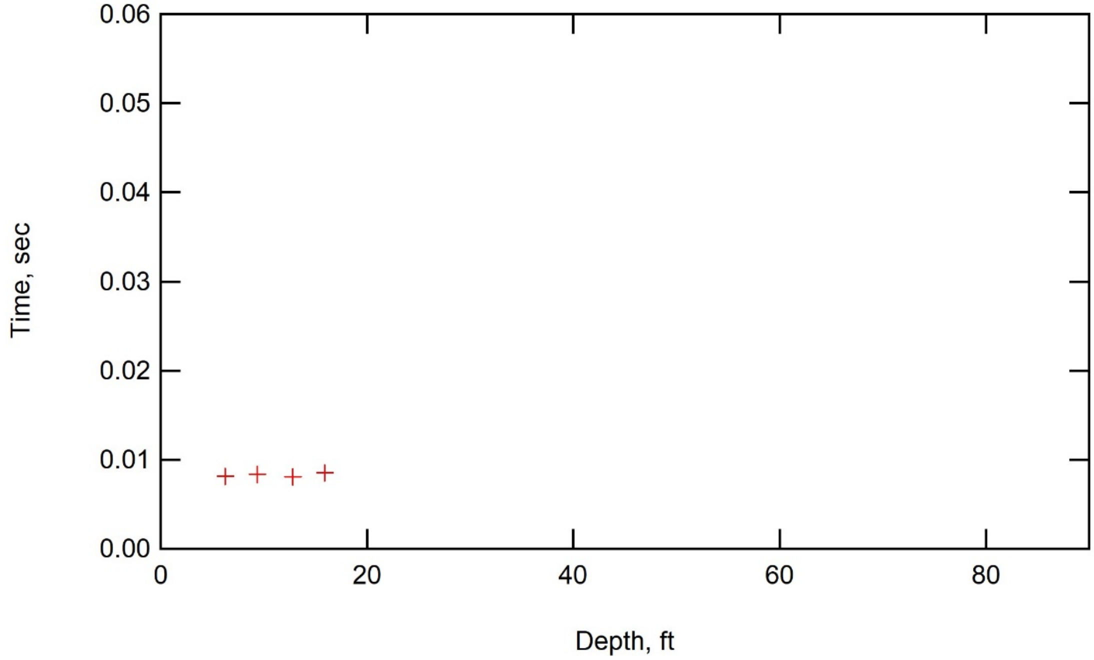


Figure F-31 Wave arrivals picks to data using p-p waves recorded with SCPT and striking vertically on bridge deck above Pile 2 at Route WW. No fit was possible with this data.

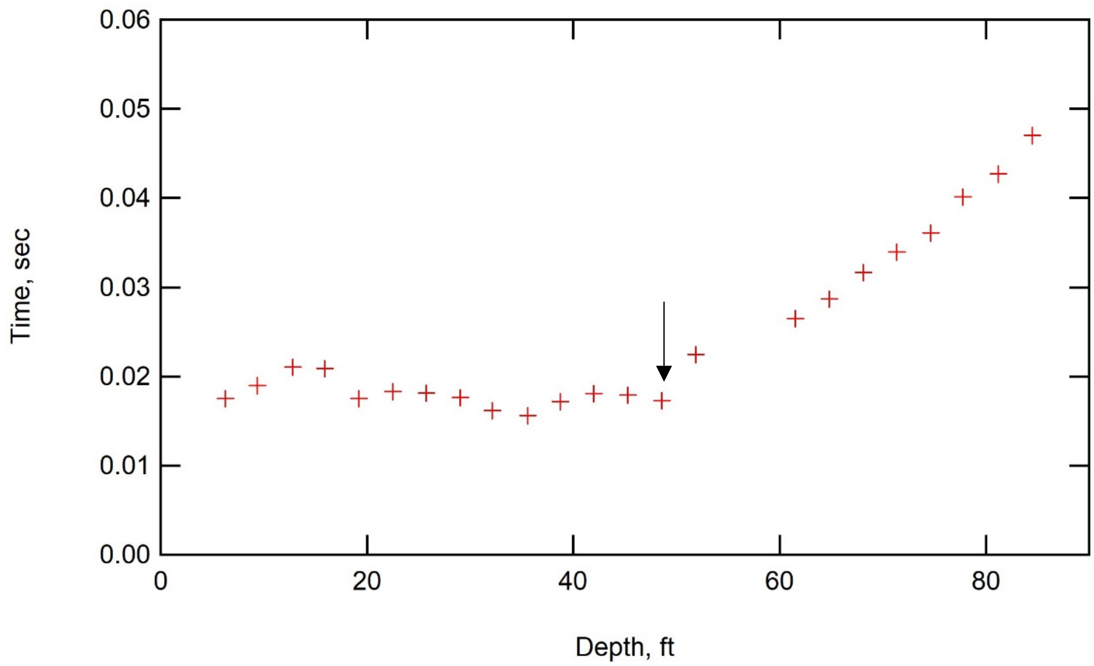


Figure F-32 Wave arrivals picks using p-s waves recorded with SCPT and striking vertically on bridge deck above Pile 2 at Route WW. Depth interpreted from change in slope (arrow)

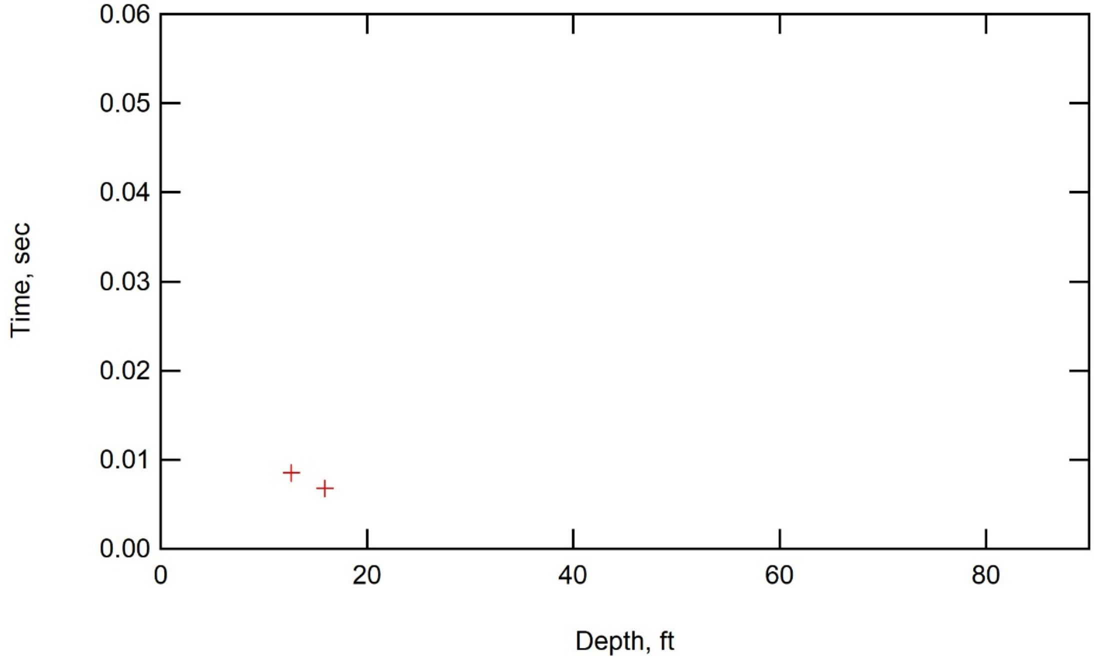


Figure F-33 Wave arrivals picks to data using p-p waves recorded with SCPT and striking vertically on bridge deck above Pile 3 at Route WW. No fit was possible with this data.

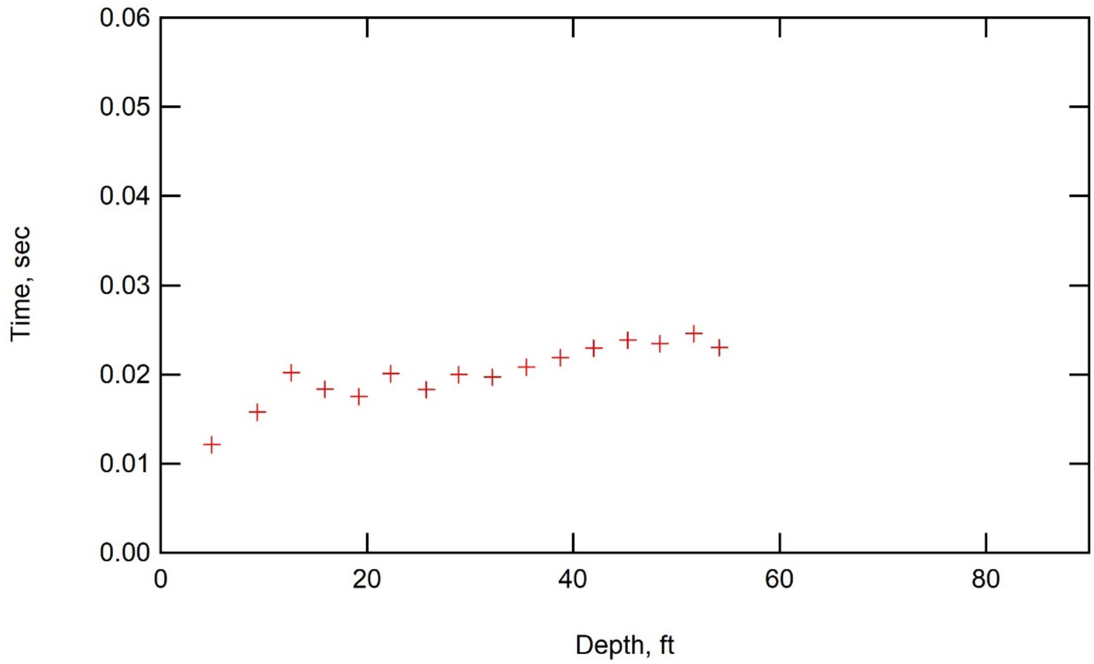


Figure F-34 Wave arrivals picks to data using p-s waves recorded with SCPT and striking vertically on bridge deck above Pile 3 at Route WW. No fit was possible with this data.

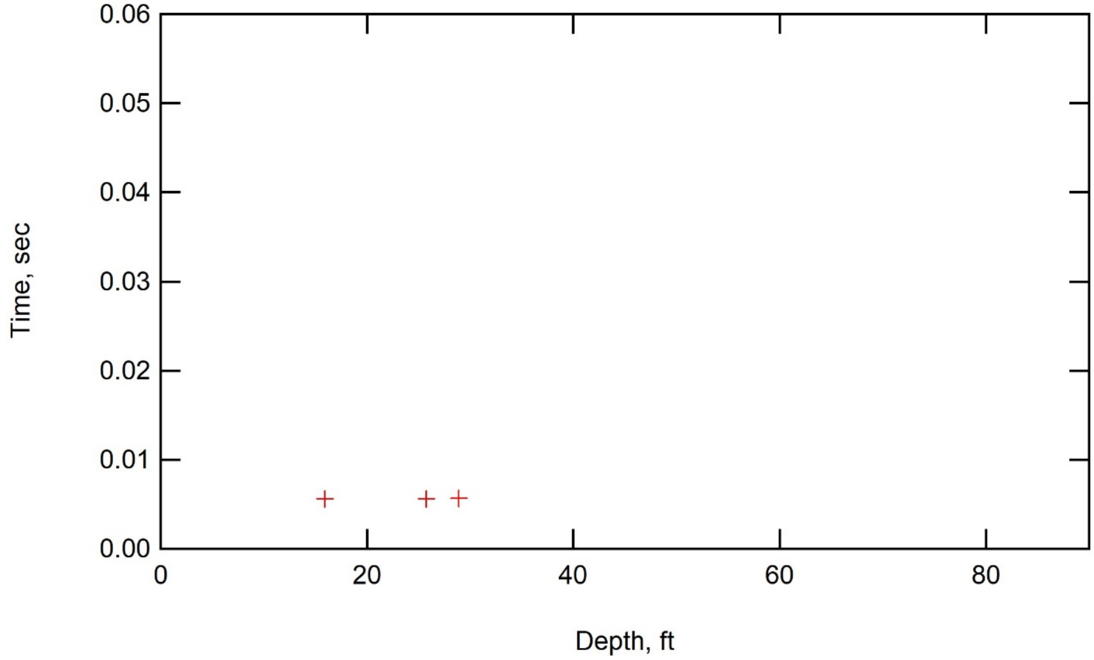


Figure F-35 Wave arrivals picks to data using p-p waves recorded with SCPT and striking vertically on bridge deck above Pile 4 at Route WW. No fit was possible with this data.

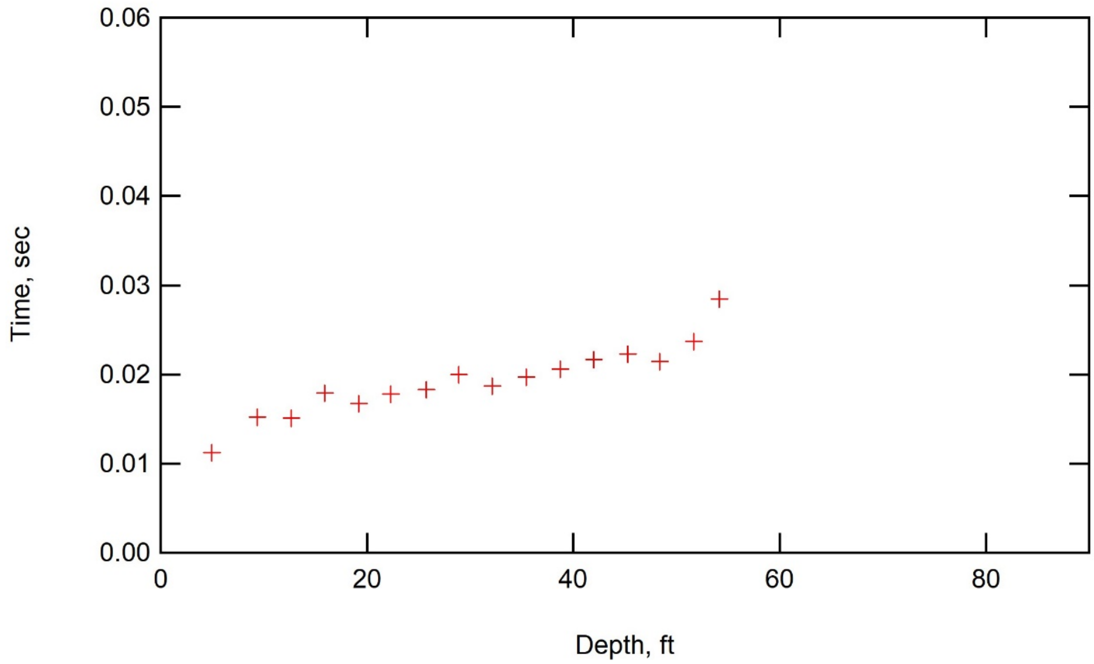


Figure F-36 Wave arrivals picks to data using p-s waves recorded with SCPT and striking vertically on bridge deck above Pile 4 at Route WW. No fit was possible with this data.

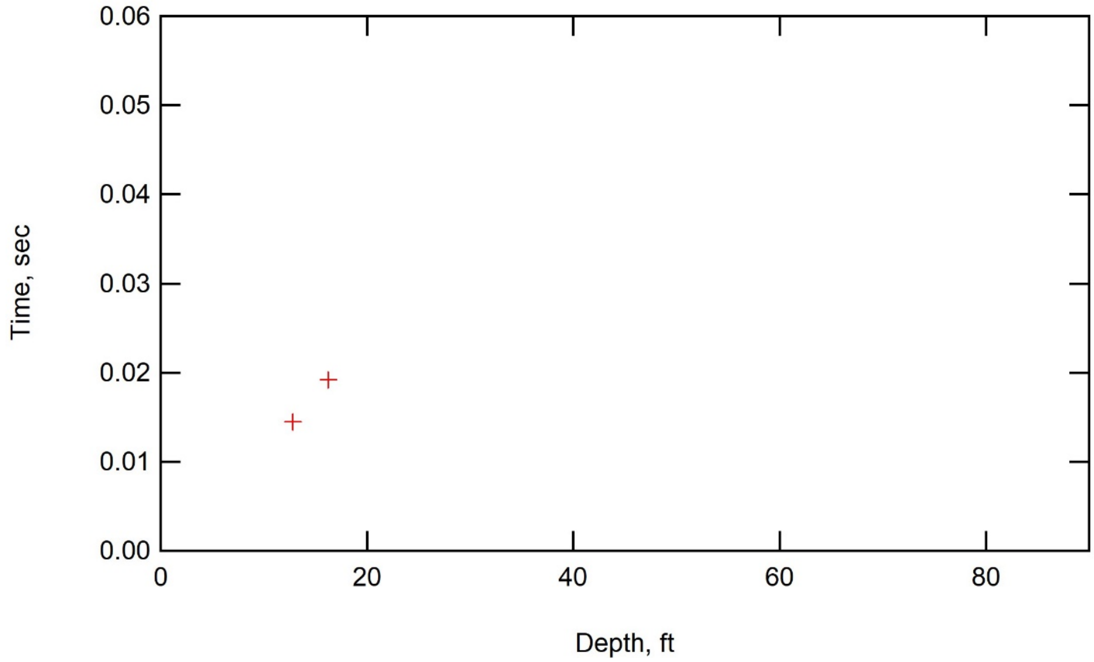


Figure F-37 Wave arrivals picks to data using p-p waves recorded with SCPT and striking vertically on bridge deck above Pile 5 at Route WW. No fit was possible with this data.

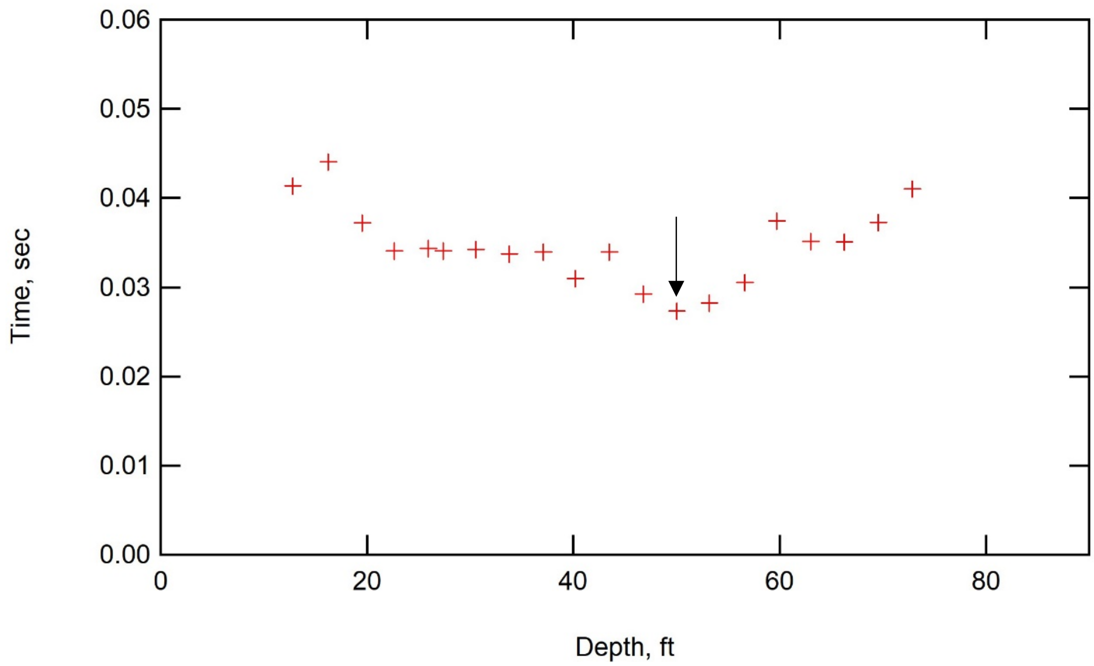


Figure F-38 Wave arrivals picks and model fit to data using p-s waves recorded with SCPT and striking vertically on bridge deck above Pile 5 at Route WW. Depth interpreted from change in slope (arrow)

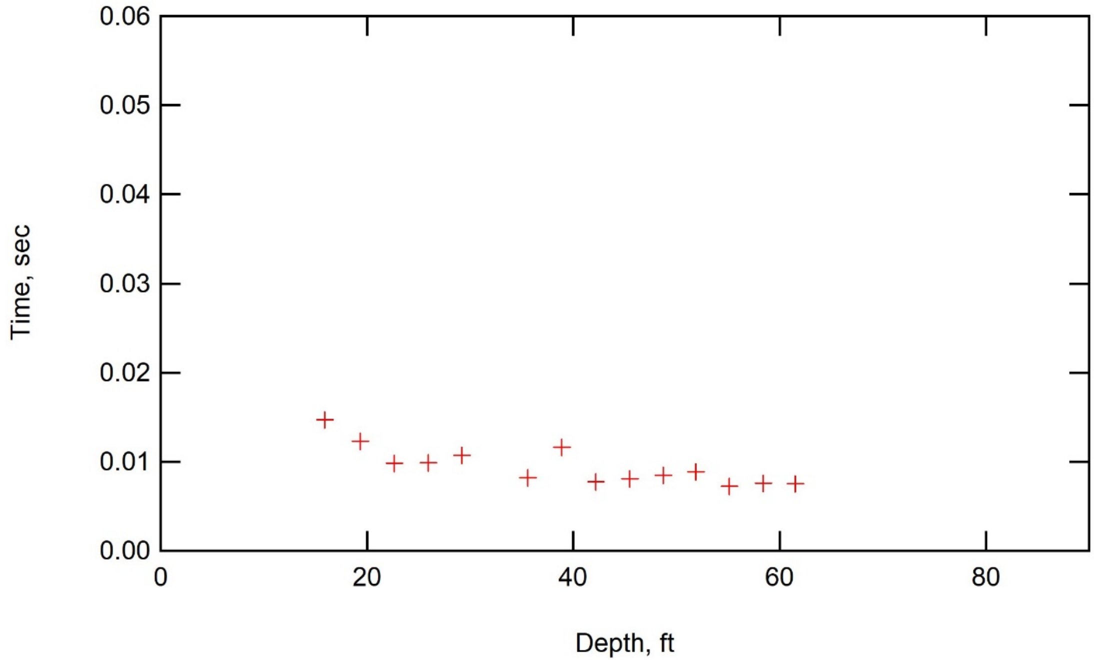


Figure F-39 Wave arrivals picks to data using p-p waves recorded with SCPT and striking vertically on bridge deck above Pile 8 at Route WW. No fit was possible with this data.

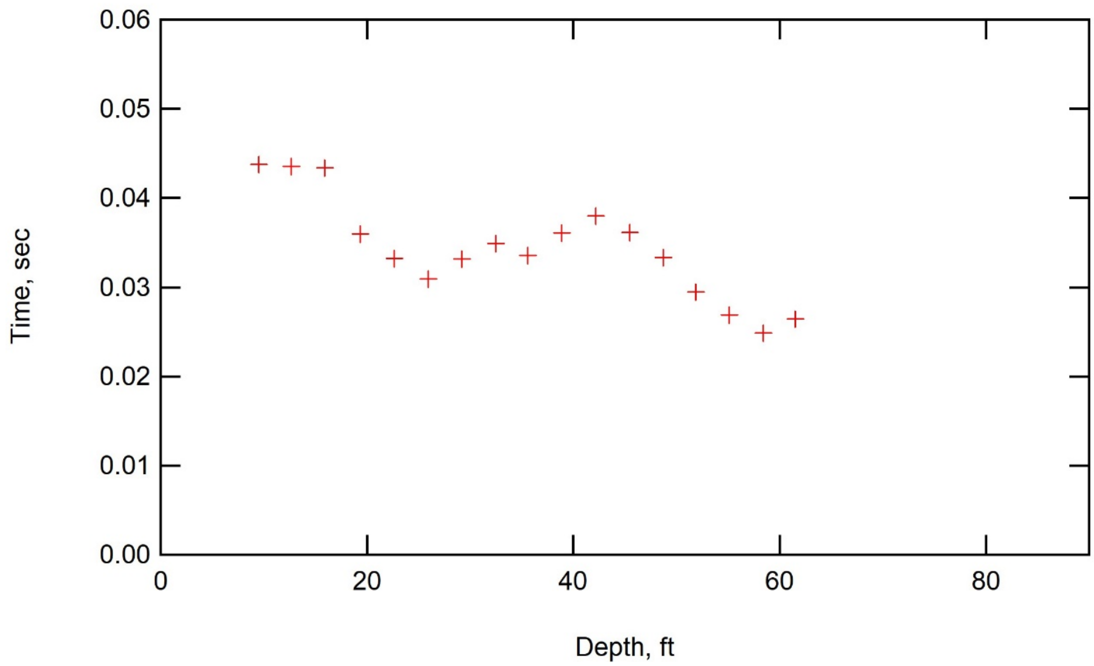


Figure F-40 Wave arrivals picks to data using p-s waves recorded with SCPT and striking vertically on bridge deck above Pile 8 at Route WW. No fit was possible with this data.

Parallel Seismic : Borehole 1 at Route WW

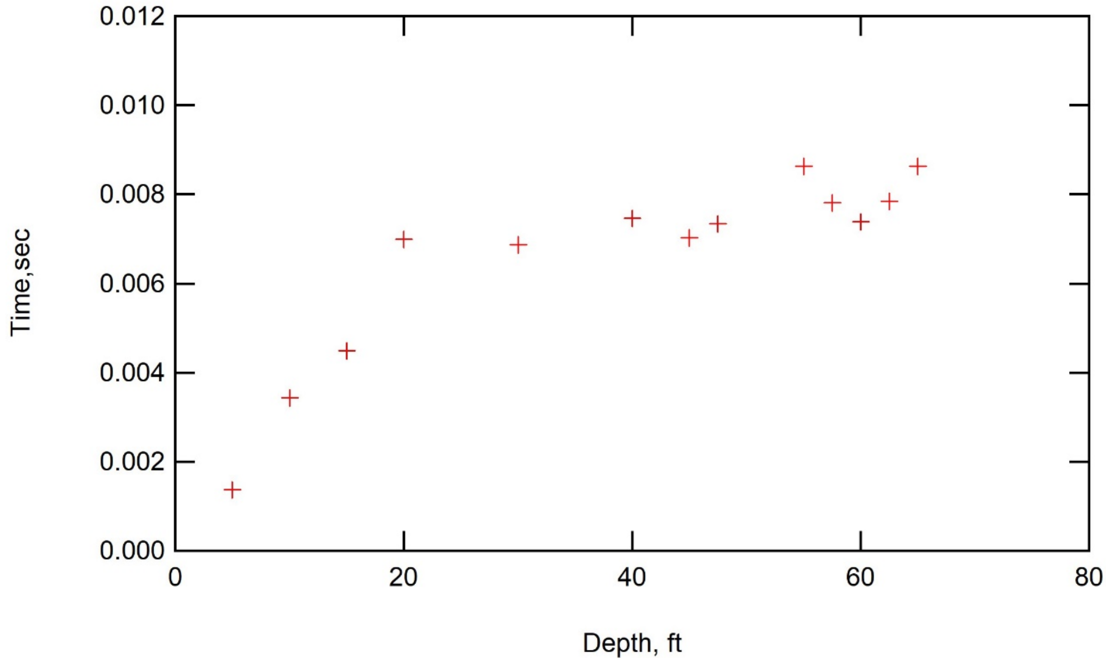


Figure F-41 Wave arrival picks using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 1. No fit was possible with this data.

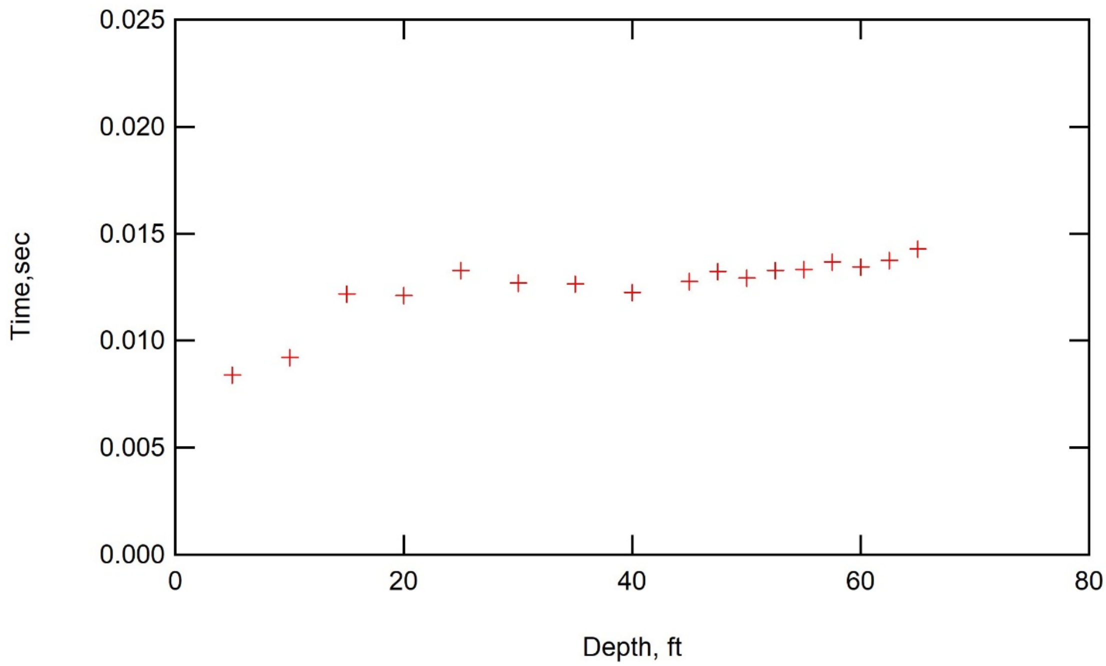


Figure F-42 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 1. No fit was possible with this data.

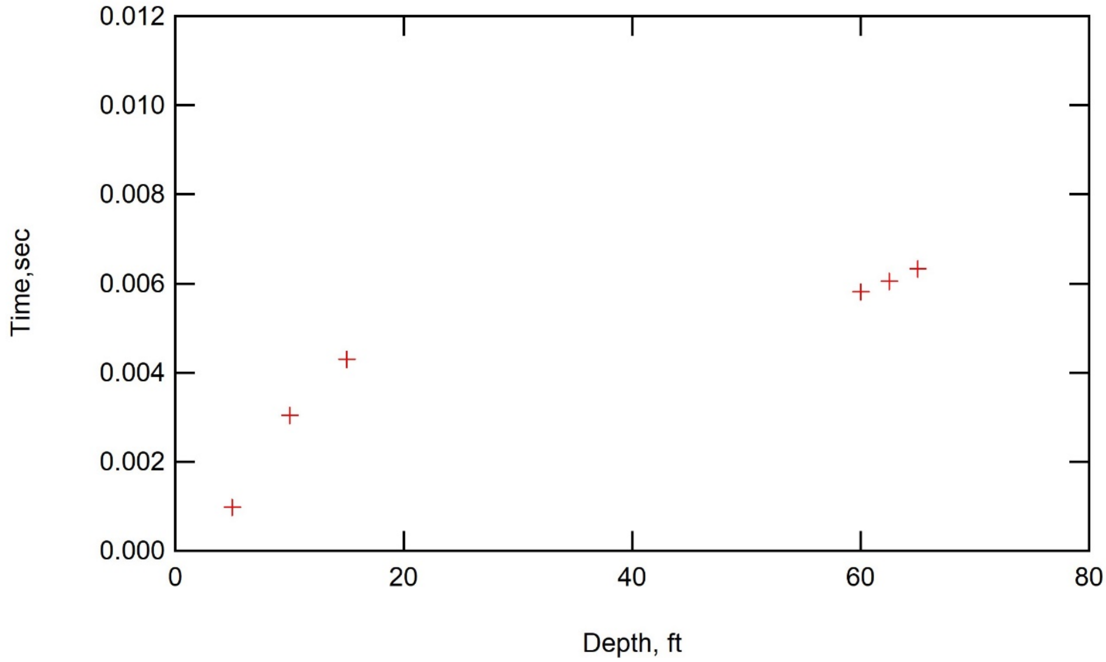


Figure F-43 Wave arrival picks using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 2. No fit was possible with this data.

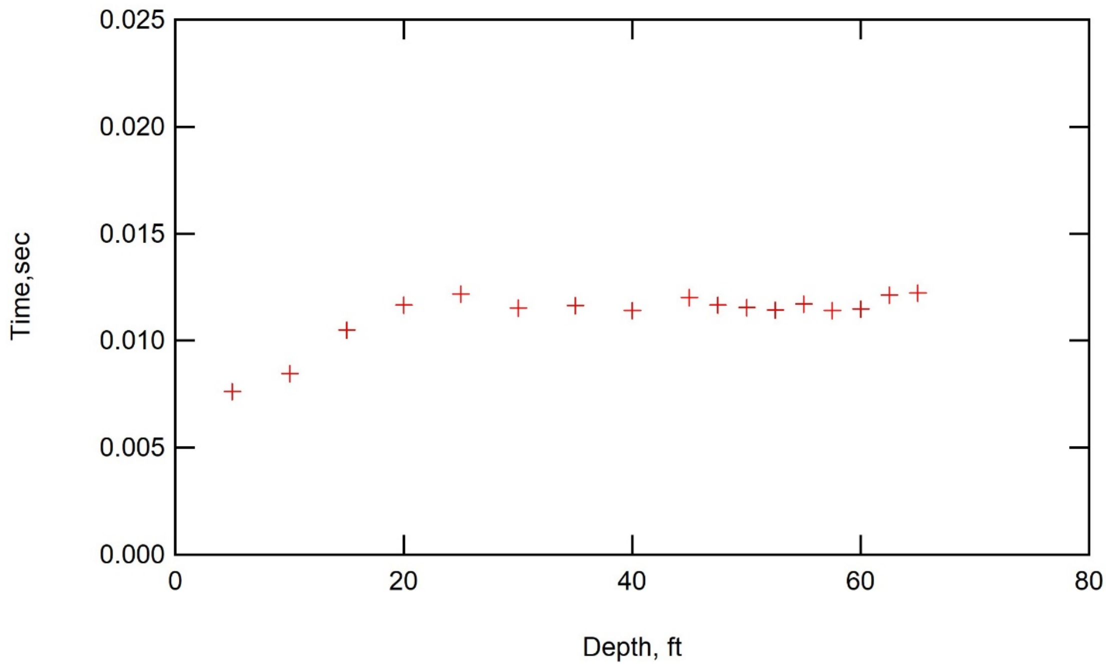


Figure F-44 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 2. No fit was possible with this data.

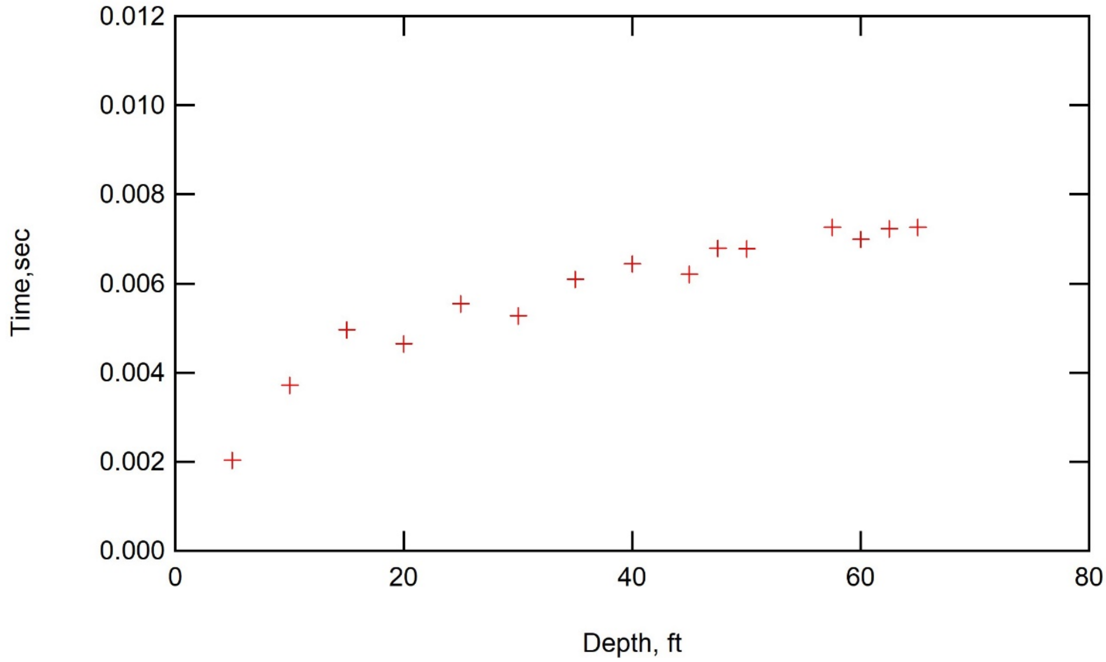


Figure F-45 Wave arrival picks using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 3. No fit was possible with this data.

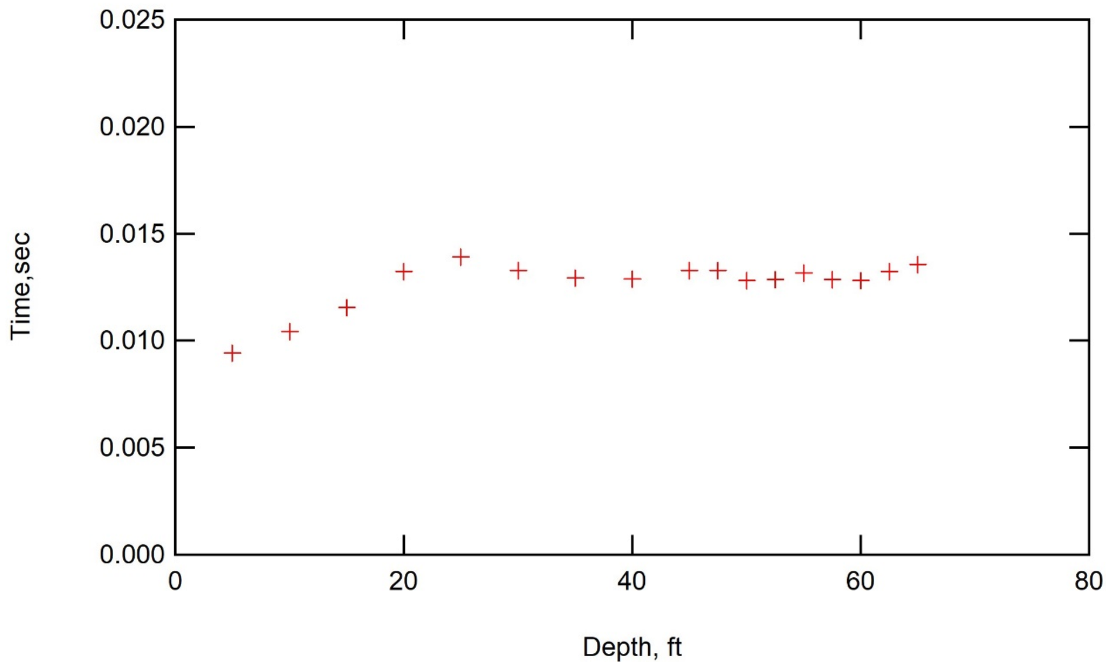


Figure F-46 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 3. No fit was possible with this data.

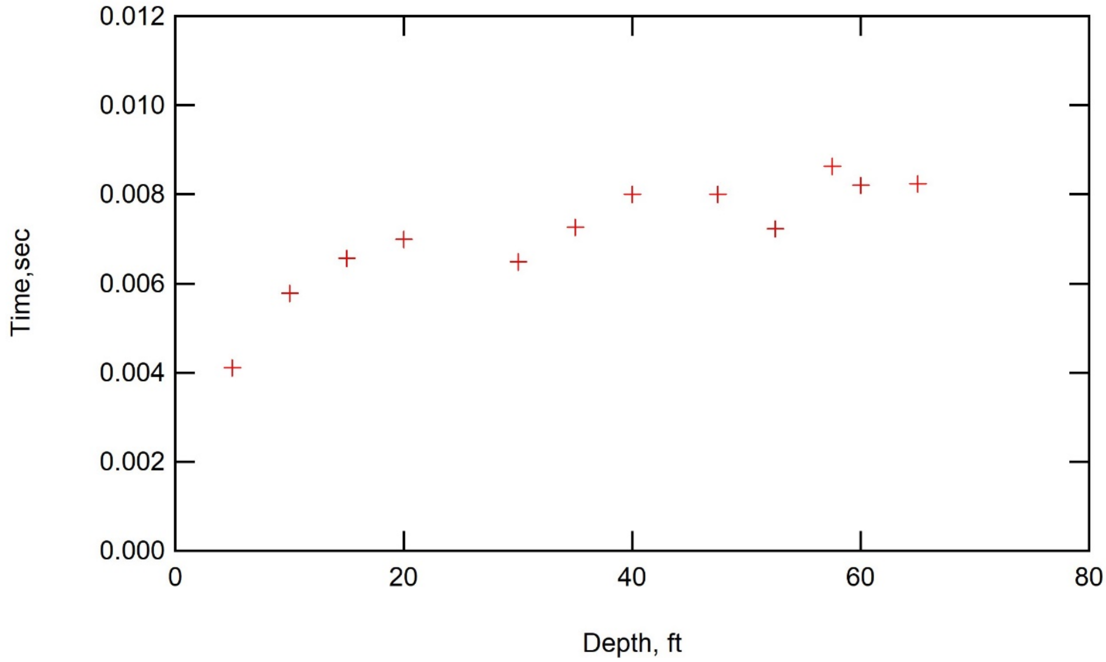


Figure F-47 Wave arrival picks using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 4. No fit was possible with this data.

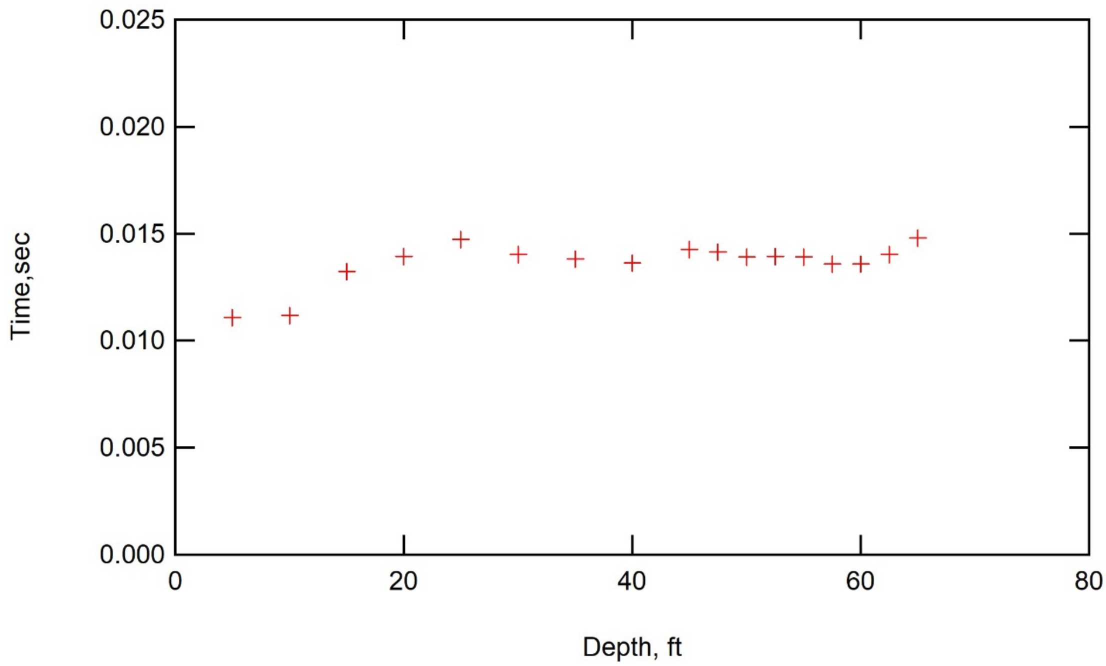


Figure F-48 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 4. No fit was possible with this data (wave arrivals are too early)

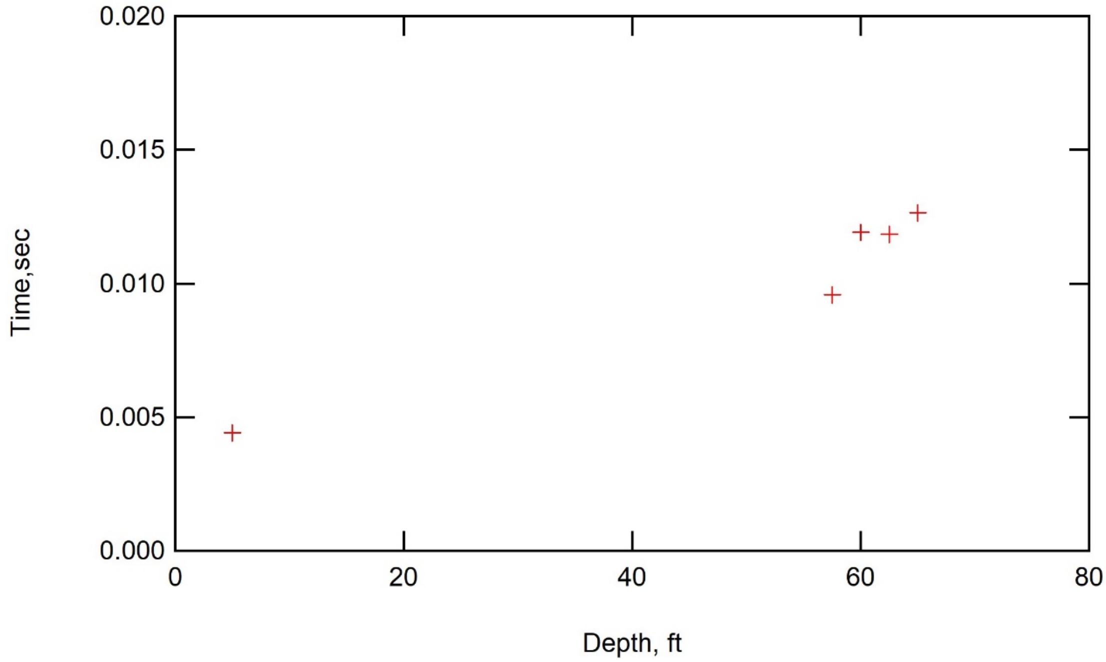


Figure F-49 Wave arrival picks using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 5. No fit was possible with this data.

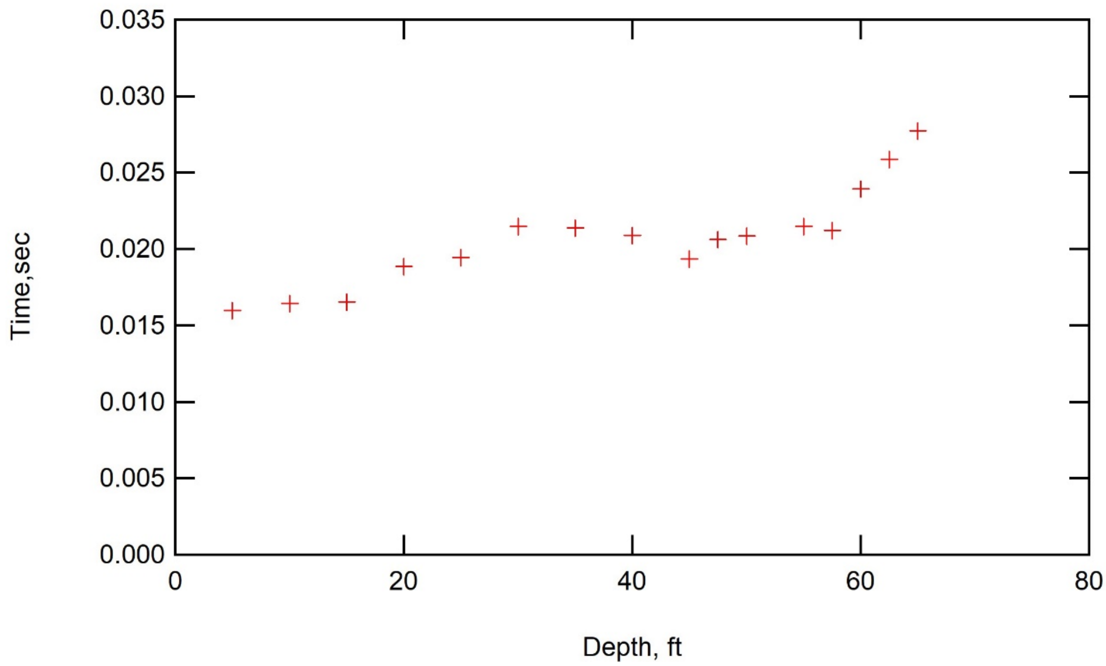


Figure F-50 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 5. No fit was possible with this data (wave arrivals are too early)

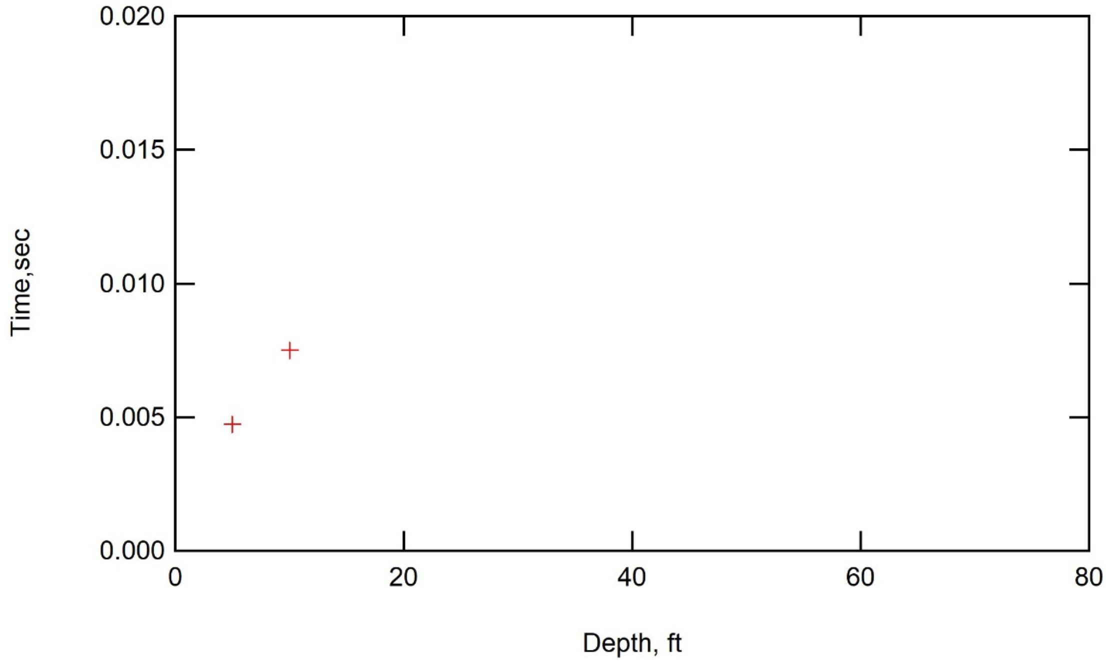


Figure F-51 Wave arrival picks using p-p waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 8. No fit was possible with this data.

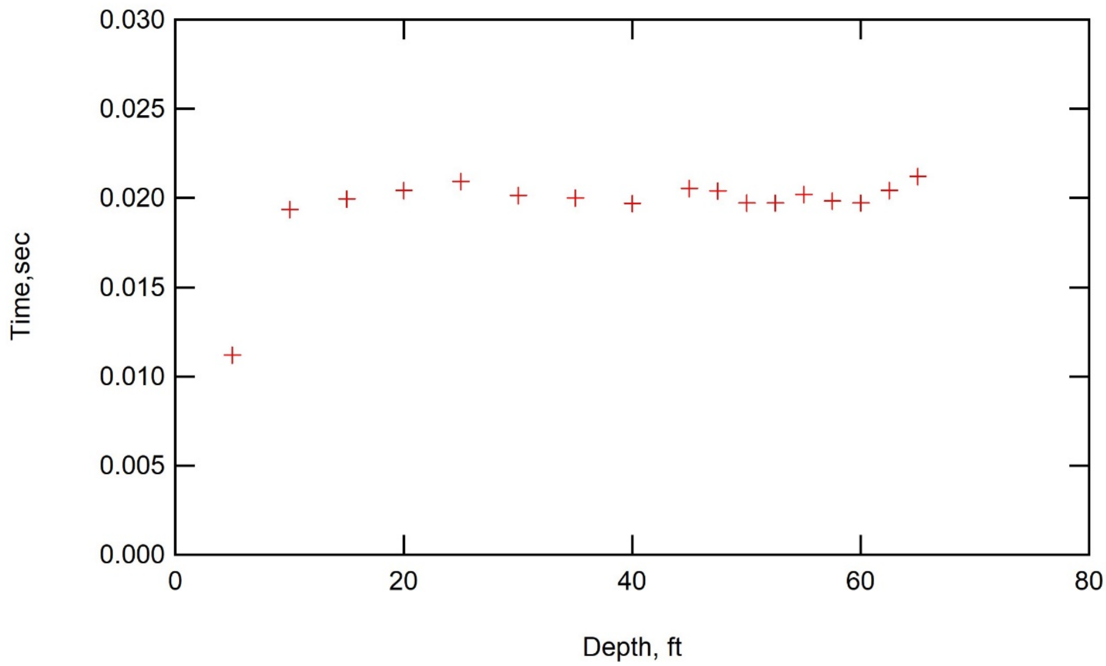


Figure F-52 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 8. No fit was possible with this data (wave arrivals are too early)

Parallel Seismic : Borehole 2 at Route WW

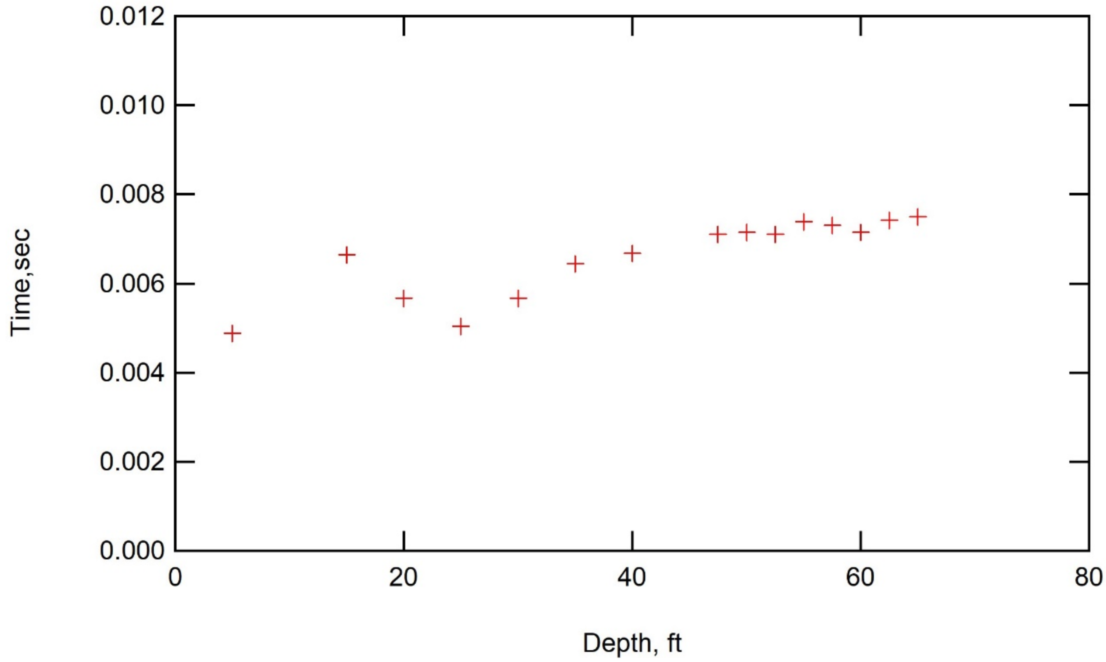


Figure F-53 Wave arrival picks using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 1. No fit was possible with this data.

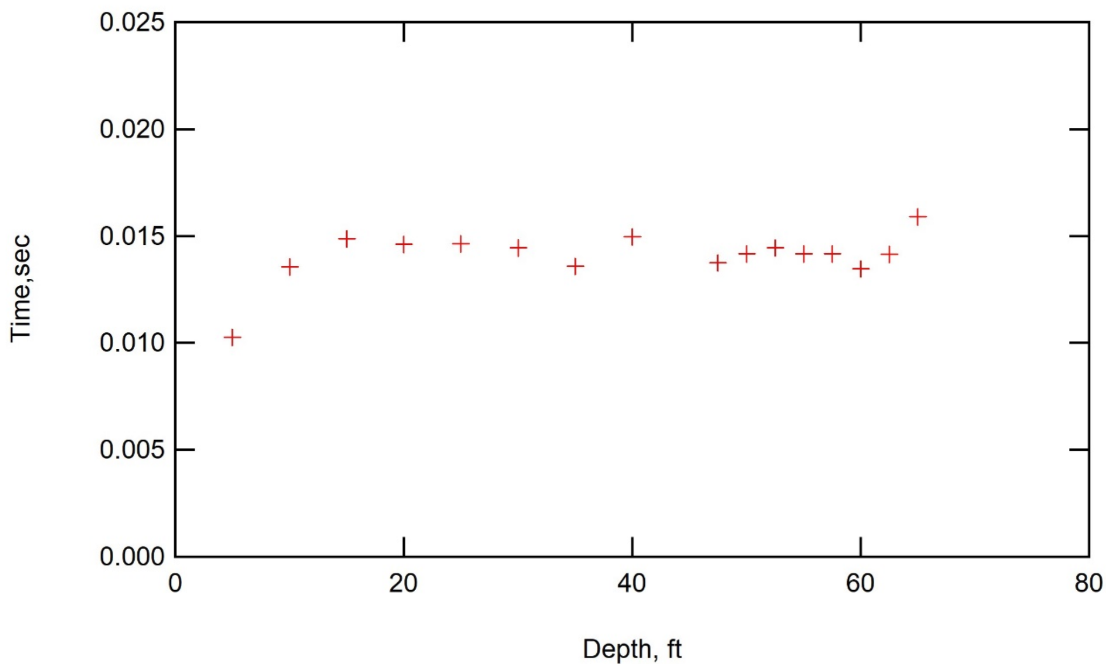


Figure F-54 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 1. No fit was possible with this data (wave arrivals are too early)

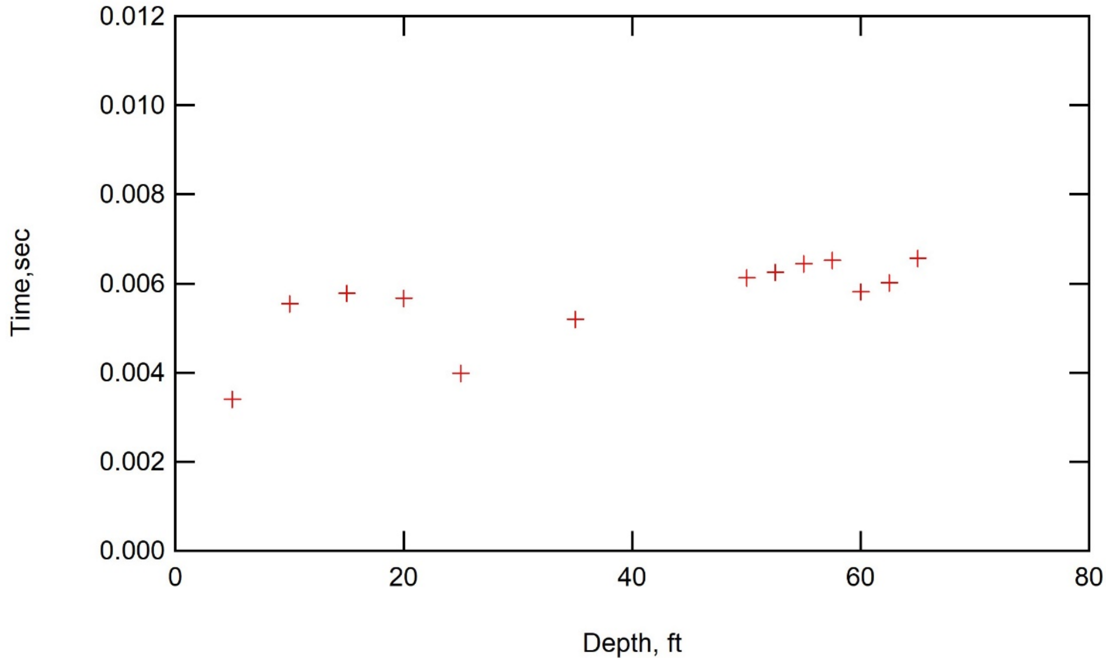


Figure F-55 Wave arrival picks using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 2. No fit was possible with this data.

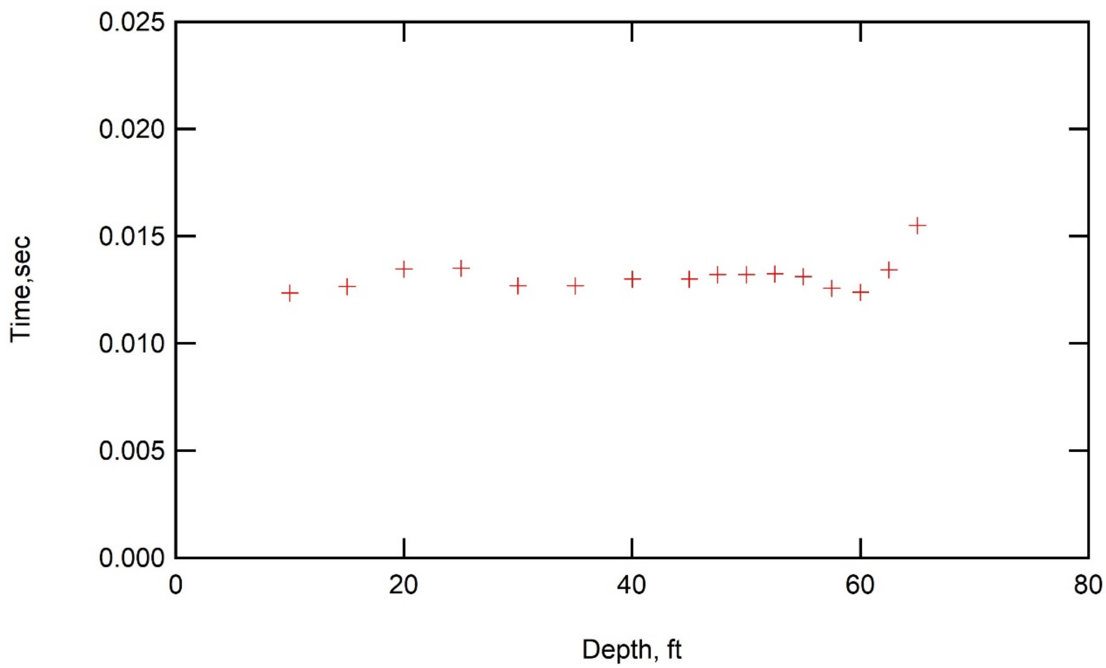


Figure F-56 Wave arrival picks using p-s waves recorded with borehole 1 sensor and striking vertically on bridge deck above Pile 2 No fit was possible with this data (wave arrivals are too early)

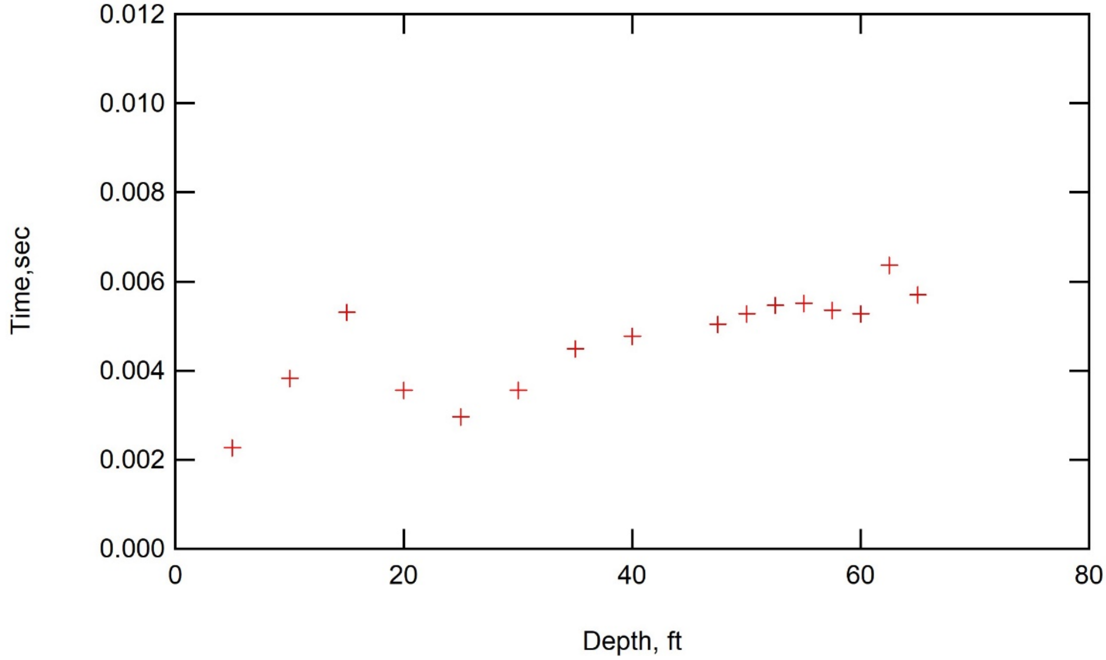


Figure F-57 Wave arrival picks using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 3. No fit was possible with this data.

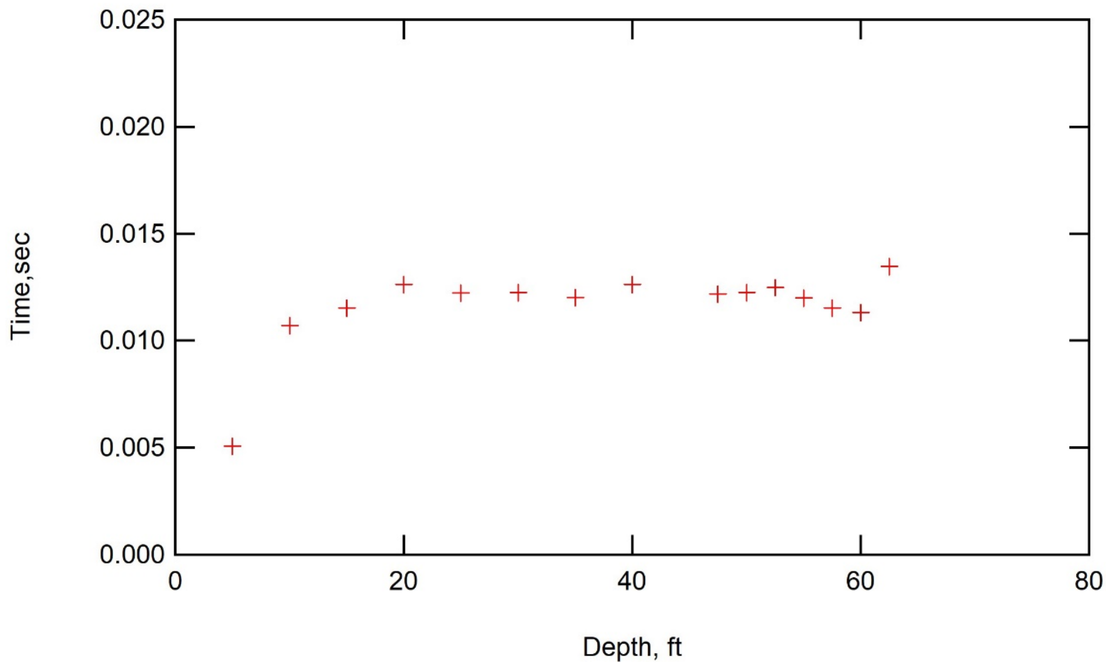


Figure F-58 Wave arrival picks using p-s waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 3. No fit was possible with this data.

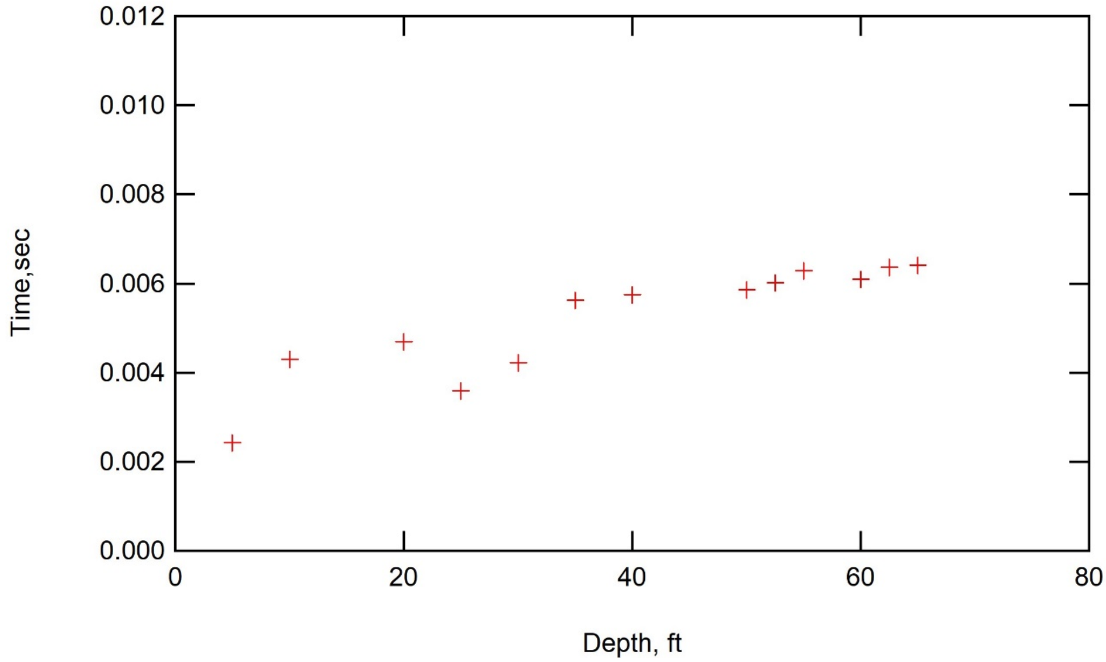


Figure F-59 Wave arrival picks and model fit using p-p waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 4. No fit was possible with this data.

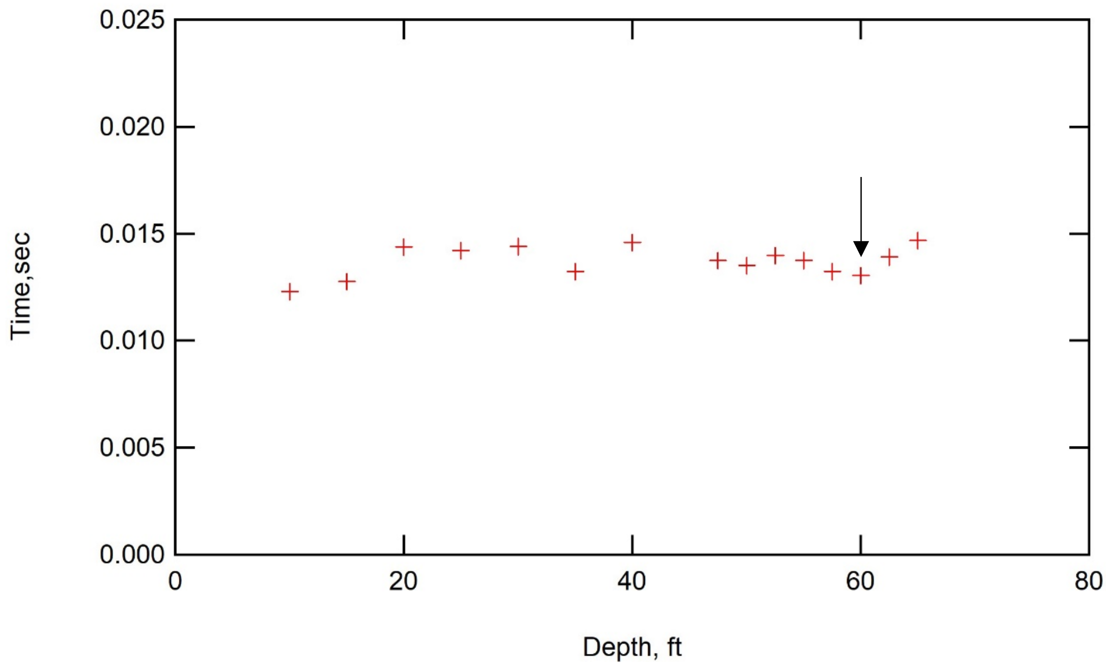


Figure F-60 Wave arrival picks using p-s waves recorded with borehole 2 sensor and striking vertically on bridge deck above Pile 4. Depth interpreted from change in slope (arrow)

Appendix G – Sonic Echo / Impulse Response Data

SE/IR at Route U

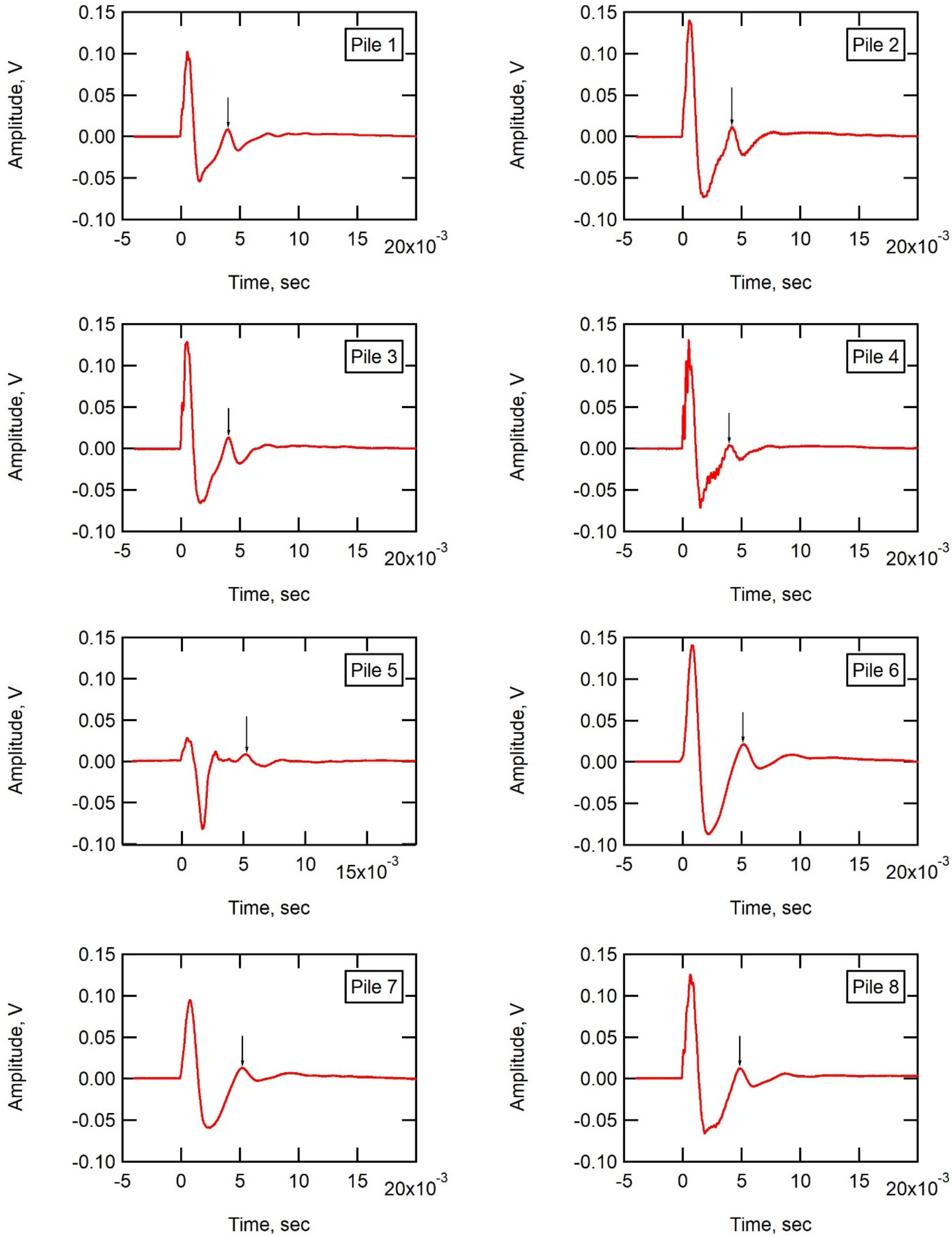


Figure G-1 Time records recorded from Sonic-Echo (SE) measurements at the Route U bridge site after bridge was removed. Reflected arrival is identified with an arrow.

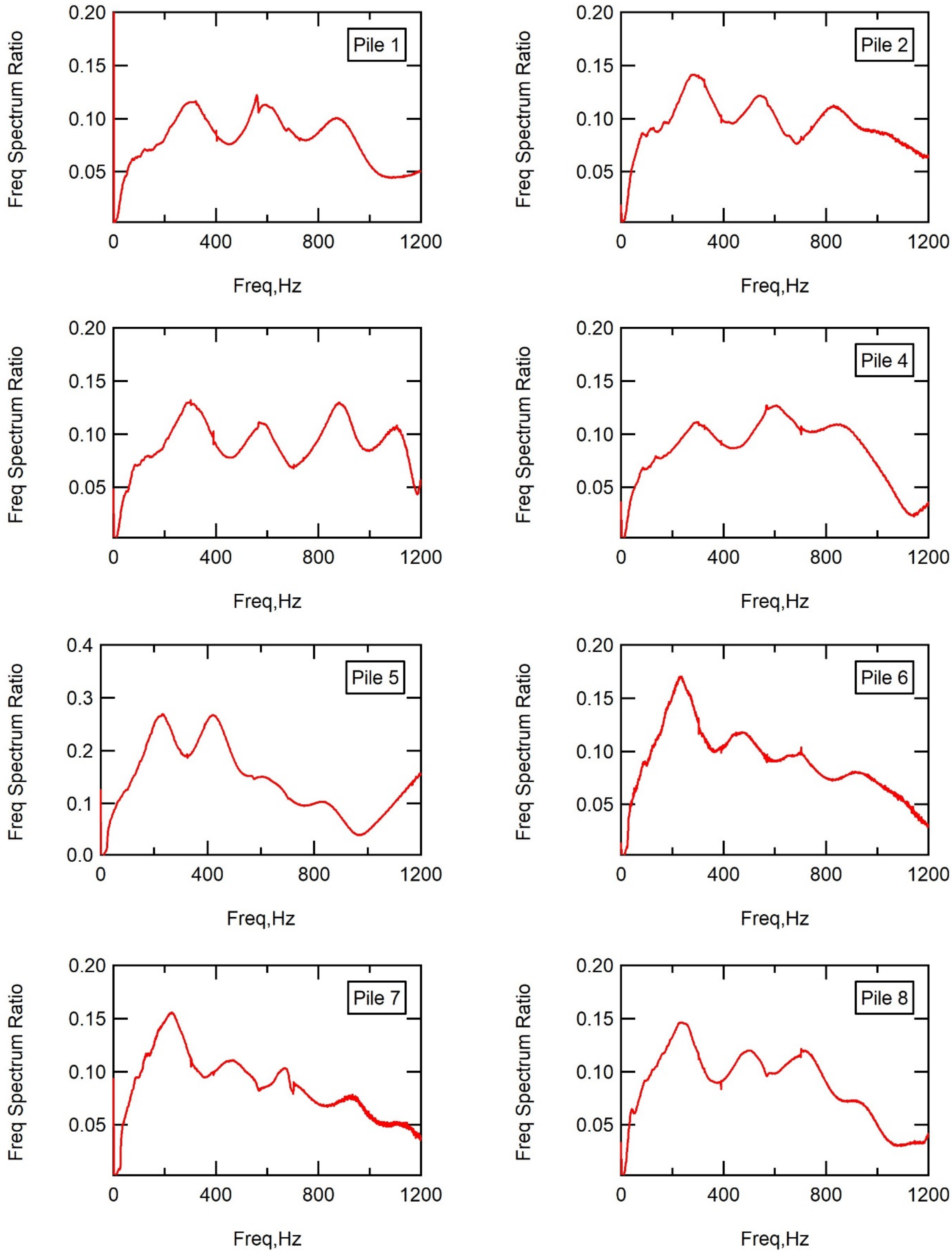


Figure G-2 Frequency spectra recorded from Impulse-Response (IR) measurements at the Route U bridge site after bridge was removed.

SE/IR at Route U

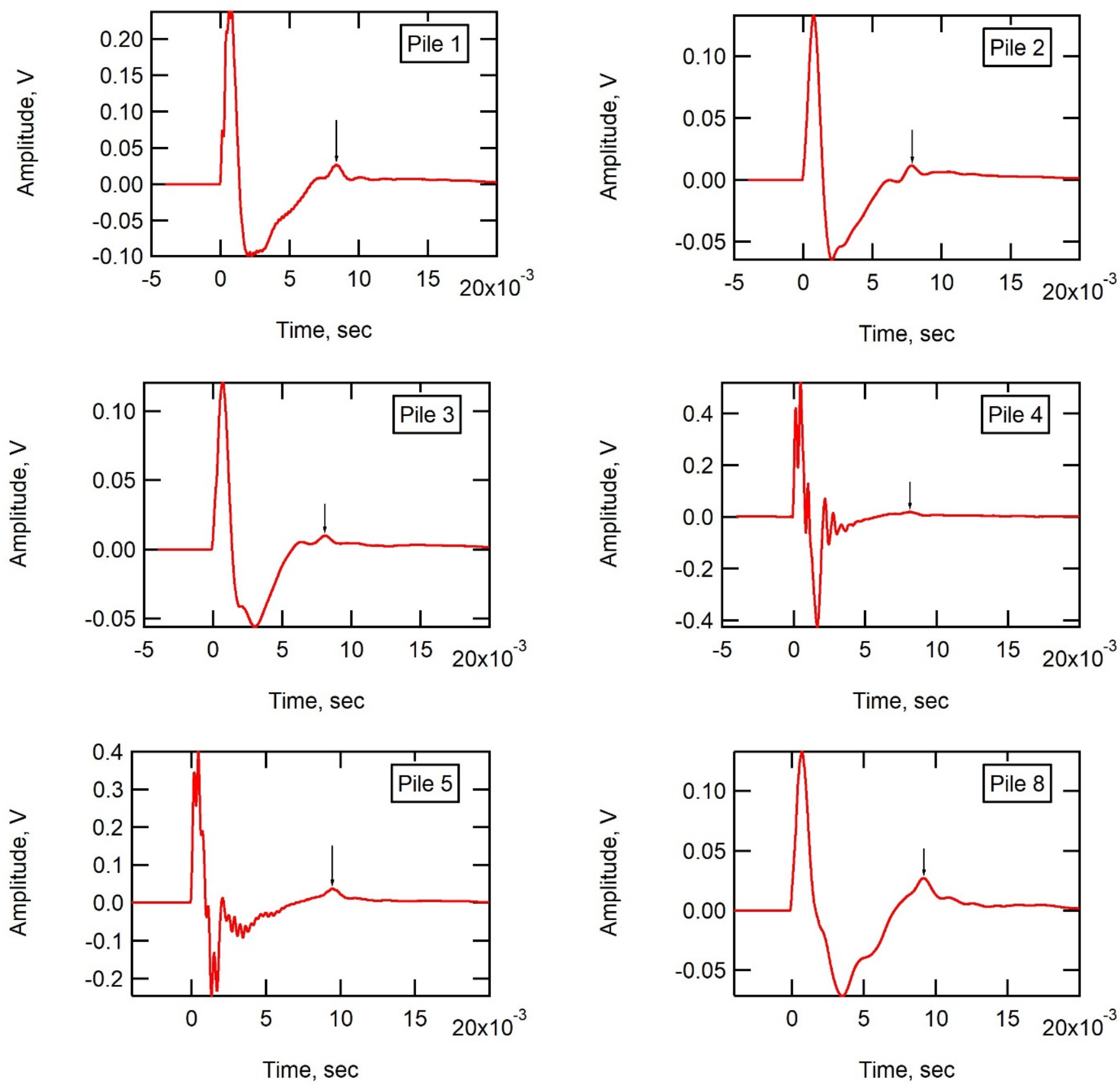


Figure G-3 Time records recorded from Sonic-Echo (SE) measurements at the Route WW bridge site after bridge was removed. Reflected arrival is identified with an arrow.

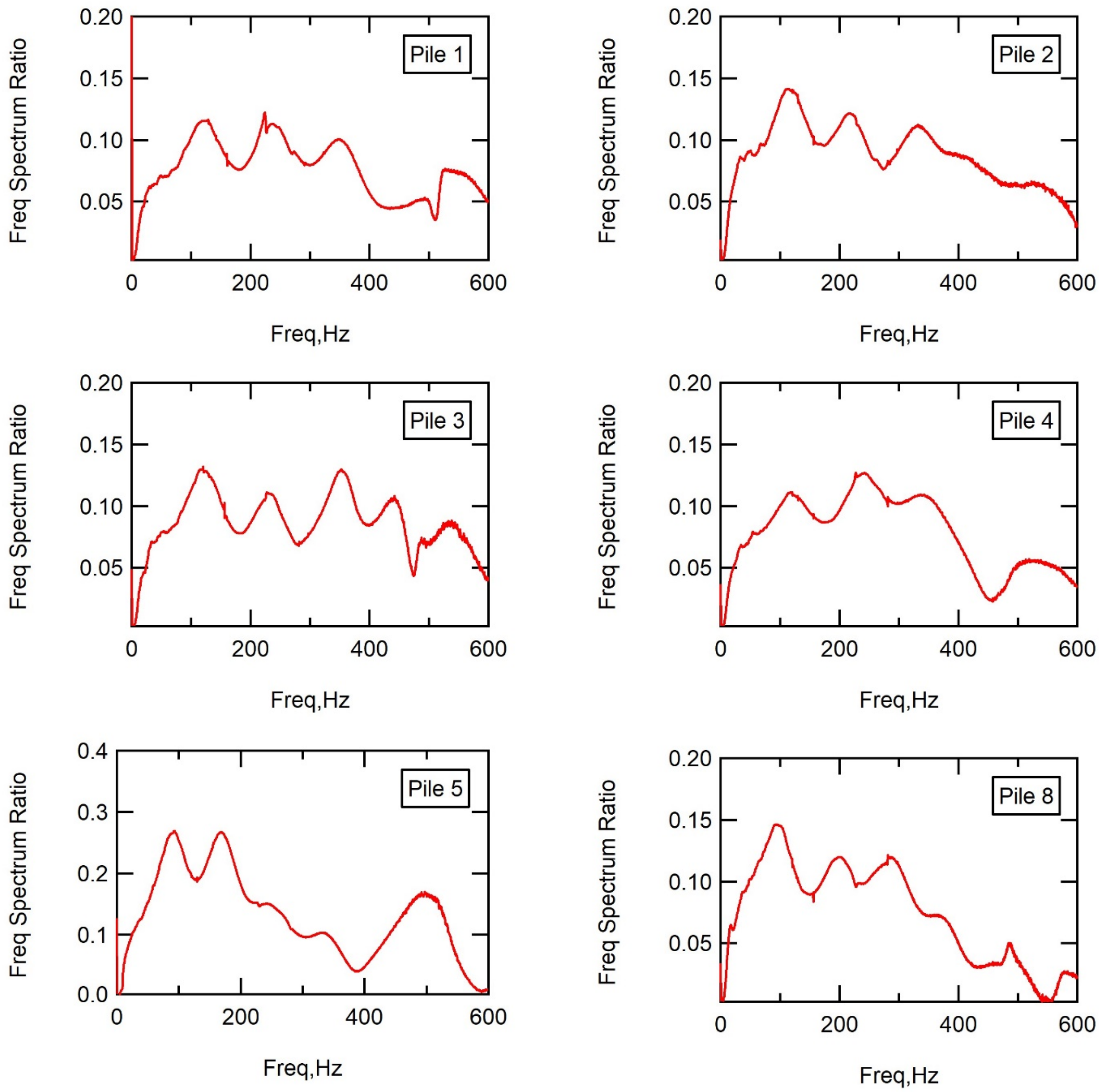


Figure G-4 Frequency spectra recorded from Impulse-Response (IR) measurements at the Route WW bridge site after bridge was removed