GEOTECHNICAL DESIGN REPORT
REPLACEMENT OF SEWALL'S BRIDGE OVER YORK RIVER
MAINEDOT WIN 12665.00, BRIDGE NO. 3096
ORAGANUG ROAD – YORK, MAINE

by

Haley & Aldrich, Inc.
Portland, Maine

for

Maine Department of Transportation
Augusta, Maine

File No. 36654-001
15 February 2012

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15 February 2012
File No. 36654-001

Maine Department of Transportation
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Attention: Kate Maguire, P.E.
Geotechnical Engineer

Subject: Geotechnical Design Report
Replacement of Sewall's Bridge over York River
MaineDOT WIN 12665.00, Bridge No. 3096
Organug Road - York, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report titled, "Geotechnical Design Report, Replacement of Sewall's Bridge over York River," prepared in accordance with our proposal, dated 23 February 2011 and with the provisions of our GCA Agreement with MaineDOT, No. CT20070810000000003861.

This Geotechnical Design Report (GDR) is a compilation of the results of subsurface investigations and geotechnical laboratory testing programs and provides geotechnical design recommendations in support of the subject project.

The remainder of this report is divided into the following sections:

- Report Text: Introduction, Geologic Setting, Subsurface Investigation Program, Generalized Subsurface Conditions, Laboratory Testing Program, Geotechnical Design Recommendations, Construction Considerations, and Limitations of Recommendations.
- Sheets: Project Locus, Site and Subsurface Exploration Location Plan, Geologic Profile, Table I – Summary of Subsurface Explorations.
- Appendices: Test Boring Logs, Laboratory Test Results, Calculations, and Historic Information, Existing Timber Piles.

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Thank you for the opportunity to help support MaineDOT on this significant bridge project. We look forward to providing continued assistance to the Department during the bidding and construction phases of the project.

Sincerely yours,
HALEY & ALDRICH, INC.

Erin F. Wood

Erin F. Wood, P.E.
Project Manager/Senior Geotechnical Engineer

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Enclosures

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TABLE OF CONTENTS

1.	**INTRODUCTION**	**1**
1.1	Existing Site Conditions	1
1.1.1	Existing Bridge Structure	1
1.1.2	Terrain	1
1.2	Proposed Bridge Structure	2
1.3	Horizontal Coordinate System and Elevation Datum	2
2.	**GEOLOGIC SETTING**	**3**
3.	**SUBSURFACE INVESTIGATION PROGRAM**	**4**
3.1	Preliminary Phase Explorations	4
3.2	Design Phase Explorations	5
4.	**GENERALIZED SUBSURFACE CONDITIONS**	**6**
4.1	Soil Unit and Bedrock Descriptions	6
4.2	Groundwater Conditions	7
5.	**LABORATORY TESTING PROGRAM**	**8**
5.1	Preliminary Phase Laboratory Testing & Results	8
5.2	Design Phase Laboratory Testing & Results	8
6.	**GEOTECHNICAL DESIGN RECOMMENDATIONS**	**9**
6.1	Approach Embankment Design Considerations	9
6.2	Seismic Design	9
6.3	Bridge Abutment and Pier Foundation Design Recommendations	9
6.3.1	Reuse of Existing Timber Piles	10
6.3.2	New Timber Piles	10
6.3.3	Pipe Piles	10
6.3.4	Pile Demand	11
6.3.5	Axial Compression Pile Resistance	12
6.3.6	Lateral Pile Capacity and Pile Group Evaluations	12
6.3.7	Pile Settlement and Elastic Pile Compression	14
6.3.8	Pile Tip Elevations	14
6.3.9	Corrosion and Deterioration	15
6.4	Abutment and Wall Design Recommendations	15
6.4.1	Frost Protection	15
7.	**CONSTRUCTION CONSIDERATIONS**	**16**
7.1	Temporary Earth Support	16
7.2	Conflicts with Existing Piles	16

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TABLE OF CONTENTS
(continued)

		Page
7.3	Dynamic Pile Load Testing Program	16
7.4	Reuse of Excavated On-Site Soils	16
8.	**LIMITATIONS OF RECOMMENDATIONS**	**17**
REFERENCES		**18**

Sheets

Sheet 1 – Project Locus

Sheet 2 – Site and Subsurface Exploration Location Plan

Sheet 3 – Geologic Profile A-A

Sheet 4 – Table I - Summary of Subsurface Explorations

Appendices

Appendix A – Test Boring Logs

Appendix B – Laboratory Test Results

Appendix C – Calculations

Appendix D – Historic Information, Existing Timber Piles

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1. INTRODUCTION

This report present the results of preliminary and final design phase geotechnical investigations, laboratory testing, engineering evaluations and geotechnical recommendations conducted by Haley & Aldrich, Inc. (Haley & Aldrich) for the Maine Department of Transportation (MaineDOT) for the proposed replacement of Sewall's Bridge over York River in York, Maine (see Sheet 1, Project Locus).

1.1 Existing Site Conditions

1.1.1 Existing Bridge Structure

The existing Sewall's Bridge is a 255-ft long, 14-span timber structure that carries Organug Road across the York River. The present bridge was constructed in 1934, but it was designed and constructed to resemble the original Sewall's Bridge that was built in approximately the same location in 1761. The bridge structure consists of timber stringers and planking supported by timber pile bent piers consisting of timber piles, the upper portions of which were rough-hewn square timbers, with diagonal timber bracing and timber pile caps. The historic bridge plans (see Sheet 17-158 in Appendix D) show that the timber piles, bents, floor and stringers are long leaf Southern Yellow Pine. The timber piles were pressure treated with creosote oil. The original bridge abutments consisted of tierod-supported timber walls and were also supported on timber piles. The south abutment was replaced in about 1980 of similar construction as the 1934 abutment, but is supported by steel H-piles. The existing bridge is designated as a Historic Civil Engineering Landmark.

1.1.2 Terrain

The existing ground surface is relatively level along the approaches and abutments at approximately El. 10. The mudline between the abutments slopes from approximately El. -7 at the south abutment to El. -20 near Bent 7 and up to El. 0 at the north abutment.

As provided by Vanasse Hangen Brustlin, Inc. (VHB), the following flood levels in the river are based on NOAA National Ocean Service Tidal Reference Station #8423898 located at Fort Point, Newcastle Island, New Hampshire. Elevations from the National Tidal Datum EPOCH (NTDE) of 1983 to 2001 have been adjusted to the project vertical datum of NAVD 88.

Discharge	Headwater (ft, NAVD 88)
Mean Low Water (MLW)	El. -4.63
Mean High Water (MHW)	El. 4.00
Q_{10}	El. 7.74
Q_{50}	El. 8.54
Q_{100}	El. 8.84

Based on their scour assessment, VHB estimated total scour of 10 ft at each pier bent, unless cohesive, non-scour susceptible material was encountered at a depth of less than 10 ft, in which case scour was assumed to extend to the top of the cohesive material (marine clay).

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1.2 Proposed Bridge Structure

To maintain the historic character of the bridge, the existing bridge will be replaced "in kind." The replacement bridge will be constructed along the same alignment as the existing bridge using timber pile caps and a timber deck. The bridge bents will be constructed using concrete filled fiberglass members (designed to have the appearance of timber) in place of timber members. As discussed below, foundation piles below MLW will consist of concrete filled steel pipe piles driven closed-ended with a conical tip. Existing timber piles will be removed and new pipe piles will be installed in the locations of the old timber piles. Negligible changes in grade are proposed. A plan and profile of the existing bridge is shown on Sheet 2, Site and Subsurface Exploration Location Plan, and Sheet 3, Geologic Profile A-A.

The structural design of the replacement bridge has been performed by VHB.

1.3 Horizontal Coordinate System and Elevation Datum

Plan locations of test borings are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 West Zone. The project elevation datum and elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

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2. GEOLOGIC SETTING

According to Maine Geological Survey's York Harbor Quadrangle, Maine (1999), surficial geologic units mapped within the site vicinity and the region consist largely of Marine Nearshore Deposits and Presumpscot Formation. Surficial geologic units encountered in subsurface explorations at the site also included Artificial Fill, Harbor Bottom and Glacial Till Deposits.

Artificial Fill soils were encountered in the north and south abutment borings at the surface. Fill soils consisted of sand, silt and gravel overlying naturally deposited Harbor Bottom Deposits. Test borings drilled in the York River also encountered tidally influenced Harbor Bottom Deposits at the mudline surface consisting of organic silt and sand and containing marine shell fragments, typical of the coastal marine environment.

Glaciomarine sediments consisting of varying thicknesses of silt, sand and clay of the Presumpscot Formation were consistently encountered beneath the Harbor Bottom Deposits throughout the site. This formation typically overlies dense Glacial Till sediment composed of a heterogeneous mix of sand, silt, clay and gravel. Glacial Till is typically found overlying the bedrock at the site, except where the bedrock surface is shallow and the bedrock contour is steep. Glacial Till was likely eroded during deglaciation at these locations leaving Presumpscot soils directly overlying the bedrock.

Bedrock at the site is mapped as Late Ordovician to Early Silurian age rocks of the Kittery Formation, part of the Merrimack Group. These rocks are composed of metamorphosed shale, siltstone and sandstone, with variably thin to thick bedding and other well developed structural features such as folding and graded bedding. Metasiltstone and quartzite were encountered in subsurface explorations at the site (Hussey 1962). There are no mapped faults within the site and vicinity (Bedrock Geology of the Kittery 1:100,000 Quadrangle, Maine and New Hampshire, 2008).

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3. SUBSURFACE INVESTIGATION PROGRAM

3.1 Preliminary Phase Explorations

Haley & Aldrich conducted a preliminary phase geotechnical investigation program at the site. All test borings were drilled by Maine Test Borings of Brewer, Maine. In total, six test borings were drilled along the existing and proposed bridge alignment in order to identify general subsurface conditions. "As-drilled" locations of the test borings are shown on Sheet 2. Coordinate location data and ground surface elevations at exploration locations are provided on individual test boring logs provided in Appendix A and are listed in Table I (Sheet 4). All soil and bedrock samples were classified in accordance with MaineDOT classification system and were preserved in glass jars and wooden boxes. The samples that were not submitted for laboratory testing are available for review upon request. Soil and bedrock samples are being stored at the Haley & Aldrich laboratory facility in Portland, Maine.

A total of six test borings, designated BB-YSL-101 through BB-YSL-106, were drilled along and immediately west of the existing bridge structure. Test boring locations were laid out in the field by Haley & Aldrich by taping/pacing distances from existing site features. "As-drilled" test boring locations and ground surface elevations were determined in the field by MaineDOT using optical survey equipment.

Subsurface explorations were drilled using either a truck-mounted Mobile B53 drill rig (BB-YSL-101 and BB-YSL-106) or a CME 45 skid-mounted drill rig placed on an anchored barge (BB-YSL-102 through BB-YSL-105). Test borings were drilled to depths ranging from approximately 36.5 to 71 ft below ground surface (BGS) using 3.0-in. (NW-size) inside diameter (ID) steel casing. Soil samples were generally collected at standard, 5-ft intervals by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. All drilling and sampling was performed in accordance with MaineDOT specifications.

Each drill rig was equipped with a standard rope and cathead and safety hammer per MaineDOT requirements (Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). A theoretical hammer efficiency factor of 0.6 was assumed for the rope and cathead/safety hammer system for the all of the drill rigs.

The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6 (i.e., 60 percent theoretical hammer efficiency). Both the raw blow count data and the corrected N-values are shown on the boring logs.

In-situ vane shear tests were attempted within the marine clay deposit in test boring BB-YSL-101 but the vane was unable to be pushed to the full test depth due to the presence of a sand layer.

Each test boring was advanced either 5 ft or 10 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel.

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3.2 Design Phase Explorations

Preliminary engineering evaluations were conducted to assess how the subsurface conditions affected the overall design and construction of the proposed replacement bridge. Subsequently, it was determined that additional (design phase) explorations were required in order to better define the steeply sloping bedrock surface and update the preliminary engineering analyses.

The final design phase borings were drilled by New Hampshire Boring, Inc. of Derry, New Hampshire. In total, three additional test borings were drilled along the bridge alignment in order to identify general subsurface conditions. "As-drilled" locations of the test borings are shown on Sheet 2. Coordinate location data and ground surface elevations at exploration locations are provided on individual test boring logs provided in Appendix A and are listed in Table I (Sheet 4). All soil and bedrock samples were classified in accordance with MaineDOT classification system and were preserved in glass jars and wooden boxes. Soil and bedrock samples are being stored at the Haley & Aldrich laboratory facility in Portland, Maine and are available for review upon request.

A total of three test borings, designated BB-YSL-201 through BB-YSL-203, were drilled along and immediately east of the existing bridge structure. Test boring locations were laid out in the field by Haley & Aldrich by estimating distances from existing site features. "As-drilled" test boring locations and mudline elevations were determined in the field by MaineDOT using optical survey equipment.

Subsurface explorations were drilled using a CME 25 skid-mounted drill rig placed on an anchored barge. Test borings were drilled to depths ranging from approximately 27.5 to 58.5 ft below ground surface (BGS) using 4.0-in. (HW-size) inside diameter (ID) steel casing. Soil samples were generally collected continuously in the upper 10 ft, and at standard, 5-ft intervals thereafter, by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. All drilling and sampling was performed in accordance with MaineDOT specifications.

The drill rig was equipped with a standard rope and cathead and safety hammer per MaineDOT requirements (Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). A theoretical hammer efficiency factor of 0.6 was assumed for the rope and cathead/safety hammer system for the all of the drill rigs.

The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6 (i.e., 60 percent theoretical hammer efficiency). Both the raw blow count data and the corrected N-values are shown on the boring logs.

Each test boring was advanced 5 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel.

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4. GENERALIZED SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site consist of the following geologic units presented in order of increasing depth below ground surface: bituminous concrete/fill, harbor bottom deposit, marine deposits, glacial till and bedrock. Refer to Sheet 3 for a graphic interpretation of the subsurface soil conditions along the proposed project alignment and Table I (Sheet 4) for a summary of the soil units and encountered thicknesses. A description of each soil unit is provided separately, below. Detailed soil descriptions are provided on the test boring logs in Appendix A.

Please note that soil descriptions provided on the test boring logs, summarized below and shown on the geologic profile (Sheet 3) do not represent actual field conditions other than at the specific test boring locations. The actual conditions will vary from those described and shown herein.

4.1 Soil Unit and Bedrock Descriptions

Bituminous Concrete/Fill

A layer of man-placed fill soils was encountered in the two land borings behind the abutments (BB-YSL-101 and BB-YSL-106). The layer ranged in thickness from approximately 12 to 14 ft. The fill soils were typically medium dense to very dense or medium stiff.

Harbor Bottom Deposit

Harbor bottom deposits were encountered in each test boring. The deposit was approximately 3 to 14 ft thick and was encountered beneath a surficial layer of fill at the abutments and at the mudline within the river. The soil unit generally consisted of organic silt to silty fine sand with varying amounts of shell fragments. The soil was generally very soft in the river and medium stiff to stiff near the abutments.

Marine Clay Deposit

A marine clay deposit was encountered only in test borings BB-YSL-101, BB-YSL-102 and BB-YSL-203, at the north end of the bridge. The encountered thickness of the clay ranges from approximately 10 to 19 ft and likely pinches out in the vicinity of exiting bridge Bent 9. The clay deposit is typically very soft to medium stiff and in BB-YSL-203 contained zones of clayey sand with clay layers.

Marine Sand Deposit

A deposit of fine sand with varying amounts of silt was encountered in each test boring with the exception of BB-YSL-101 and BB-YSL-203 at the north end of the bridge. The marine sand was encountered directly beneath the marine clay layer (where present) or harbor bottom deposit and ranged in thickness from approximately 9 to 33 ft, generally increasing in thickness towards the center of the river. The marine sand was typically loose to dense. A 12 in. boulder was encountered in BB-YSL-105 at mid-depth of the deposit.

Glacial Till

A deposit of coarse to fine sand with silt and gravel was encountered in each test boring with the exception of BB-YSL-101, BB-YSL-102 and BB-YSL-106. Cobbles and boulders are often present

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within the glacial till deposit and cobbles were encountered as noted on the test boring logs. In general, glacial till was encountered directly beneath the marine sand layer. The deposit ranged in thickness from approximately 4 to 18 ft.

Bedrock

Bedrock was sampled in each test boring. The top of bedrock surface ranged from approximately 23 to 62 ft BGS. The bedrock surface slopes down significantly towards the center of the river. Bedrock encountered at the site consists of moderately hard to hard, fresh to moderately weathered, gray METASILTSTONE and QUARTZITE. At some test boring locations (BB-YSL-106 and BB-YSL-105), up to 2 ft of weathered or fractured bedrock was encountered overlying more competent bedrock.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of recovered bedrock. RQD values for bedrock encountered at the site ranged from 19 to 88 percent. The RQD of the upper core sample (1.9 ft) from BB-YSL-105 was 0 percent.

4.2 Groundwater Conditions

Water levels observed during drilling varied with tidal fluctuations. Water levels in the river, as determined by VHB, are Mean Low Water (MLW) El. -4.63, Mean High Water (MHW) El. 4.00.

Groundwater levels can be expected to fluctuate, subject to seasonal variation, local soil conditions, topography and precipitation. Water levels encountered during construction may differ from those observed in the test borings.

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5. LABORATORY TESTING PROGRAM

A laboratory testing program was undertaken to assist in soil classification/identification, determination of engineering properties, and for use in scour evaluations. In general, laboratory testing was performed on disturbed soil samples collected during SPT sampling or rock core samples. All laboratory soil testing was performed by GeoTesting Express of Boxborough, Massachusetts. Geotechnical laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures. All soil samples were transported to GeoTesting Express by Haley & Aldrich personnel. Laboratory testing and results are summarized below. All laboratory test results are provided in Appendix B.

5.1 Preliminary Phase Laboratory Testing & Results

A preliminary laboratory testing program was conducted upon completion of the geotechnical investigation. The testing program included nine natural water content tests, three Atterberg Limits tests, five grain size analyses (sieve and hydrometer), and eleven grain size analyses (sieve only). A summary of laboratory test results is provided below.

- Natural Water Content:
  - Harbor Bottom Deposit 27% to 47%
  - Marine Clay 32% to 38%
  - Marine Sand 11%

- Atterberg Limits:

	Harbor Bottom	Marine Clay
Liquid Limit (LL):	NP[1] to 30	35
Plastic Limit (PL):	NP[1] to 22	18
Plasticity Index (PI):	NP[1] to 8	17

[1]- NP = non-plastic

The testing program on rock consisted of two unconfined compressive strength tests including total unit weight and elastic modulus determination. Peak compressive stress of the tested rock core samples ranged from 37,432 psi to 48,338 psi.

5.2 Design Phase Laboratory Testing & Results

We determined that additional laboratory testing was not required to finalize the geotechnical design recommendations. Therefore, laboratory testing was not performed during the final design phase investigation.

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6. GEOTECHNICAL DESIGN RECOMMENDATIONS

Geotechnical design recommendations for the subject project, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Fifth Edition, 2010 and
- MaineDOT Bridge Design Guide (BDG), August 2003.

Engineering calculations are provided for reference in Appendix C.

6.1 Approach Embankment Design Considerations

There are negligible raises in grade proposed for the north and south bridge approaches (less than 0.5 ft over limited areas). Because of this minor grade change, we have concluded that there are no special settlement or stability considerations.

6.2 Seismic Design

Based on the corrected SPT blow count (granular soils) and undrained shear strength (cohesive soils) obtained from the test borings, the site is considered Site Class "E" in accordance with Table 3.10.3.1-1. The following provides the seismic design parameters, in accordance with the United States Geologic Survey software Seismic Design Parameters Version 2.10:

Parameter	Design Value
Fpga	2.5
Fa	2.5
Fv	3.5
As (0 second period)	0.242
SDs (0.2 second period)	0.466
SD1 (1 second period)	0.154

Refer to Appendix C, Calculations, for supporting documentation.

Due to the high silt content of the in-situ soils, it is our opinion that the soils at the site are not susceptible to liquefaction.

6.3 Bridge Abutment and Pier Foundation Design Recommendations

As shown on the interpretive geologic profile (Sheet 3), the subsurface conditions along the proposed bridge alignment consist primarily of variable thicknesses of harbor bottom deposits, marine clay, marine sand, glacial till and bedrock. The glacial till soils and bedrock are considered suitable for support of the bridge superstructure. Based on the depth to the suitable foundation bearing strata and the magnitude of the design loads, we consider driven pile foundations as the most practicable foundation alternative to support the new bridge piers. It is our understanding that the substructures of the existing abutments will be reused. Because of this, no technical evaluations were required specific to determining the support recommendations for the bridge abutments.

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6.3.1 Reuse of Existing Timber Piles

Based on historic drawings for the bridge, the existing timber piles were driven between approximately 9 and 28 ft below mudline and gain their support by a combination of side friction in the harbor bottom deposits and end bearing in the marine sand (see information provided in Appendix D). The factored resistance of existing piles, calculated in accordance with LRFD, ranges from 3 to 5 kips with no scour considerations. The calculated factored resistance decreases to 1 kip if 8 to 10 ft of scour is included. Refer to Appendix C, Calculations, for supporting documentation.

Although the existing timber piles appear to sufficiently support the current bridge structure, the calculated resistances per LRFD requirements are not sufficient to support the factored self weight of the bridge. Furthermore, under a scour event, the piles would have minimal (or no) embedment and provide little support. Therefore we recommend supporting the replacement bridge on new pile foundations.

6.3.2 New Timber Piles

To replace the existing bridge "in kind" and maintain the historic character of the bridge, new timber piles were initially the preferred foundation alternative. Therefore the structural and geotechnical resistance of new timber piles was evaluated. Based on the anticipated pile demand as determined by VHB, new timber piles would need to be driven deeper than the existing timber piles, into the marine sand deposits, and possibly several feet into the glacial till deposit (in areas where the depth to glacial till is relatively shallow). Further embedment into glacial till is not feasible for driveability reasons.

The structural resistance of new timber piles is highly influenced by the unsupported length of the piles, which varies along the length of the bridge. For unsupported lengths ranging from approximately 8 to 35 ft, the factored structural resistance ranged from 12 to 100 kips for the Strength I condition. Refer to Appendix C, Calculations, for supporting documentation.

Multiple methods available in LRFD to determine the geotechnical resistance resulted in a range of resistance values for timber piles (there are no specific guidelines or resistance factors for timber piles in LRFD). Based on our engineering judgment and experience from other projects where timber piles were used, the factored geotechnical resistance of new timber piles driven into the marine sand deposit is approximately 20 to 25 kips (based on resistance factor equal to 0.25 to 0.35). Refer to Appendix C, Calculations, for supporting documentation.

VHB indicated that a factored resistance of approximately 60 to 80 kips per pile is required. Both the calculated structural and geotechnical resistance values for new timber piles were significantly less than required. Therefore it is our opinion that the use of new timber piles to support the replacement bridge is not feasible.

6.3.3 Pipe Piles

To achieve higher resistances, we considered several driven pile foundation alternatives, with piles cut off above the mudline and below MLW so they would not be visible. A fiberglass encased concrete column (designed to have the appearance of timber) would then be structurally connected to the top of the pile and extend to the bridge deck to maintain the historic appearance of the bridge.

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Our evaluations considered various pipe pile sizes (24-in. and 30-in. diameter), wall thickness (½-in. and ¾-in.) and driving conditions (open and closed-ended). Based on the results of our evaluations, 30-in. diameter pipe piles with ½-in. wall thickness driven closed-ended were determined to be the most efficient and cost effective pile type to support the pier bents. Although other pile types are also feasible (H-piles, precast prestressed concrete piles), the pipe pile was preferred because: 1) pipe piles readily allow the concrete-filled fiberglass columns to be attached using a mechanical connection; and 2) a stiff foundation support member is required at the base of the concrete-filled fiberglass columns, even under the scour conditions, to provide the appropriate support condition for column design.

Specific pipe pile design recommendations are provided below (all articles and tables referenced below refer to AASHTO LRFD Bridge Design Specifications).

6.3.4 Pile Demand

Design pile loading information developed by VHB was provided to Haley & Aldrich on 17 August 2011 and is included in Appendix C. Loads were provided for two Service Limit Cases and ten Strength Limit Cases. Initial lateral pile evaluations identified the two Strength Limit Cases that resulted in the largest bending stresses in the longitudinal and transverse directions and the Service Limit Case that resulted in the greatest pile head deflections. These three load cases, summarized below, were then applied throughout the final evaluations.

Pier Bents 1-10, 13							
Load Case	P_v (kips)	At Top of Pile		Applied to Substructure			
				Pipe Piles		Square Columns	
		V_T (kips)	V_l (kips)	w_{T2} (klf)	w_{L2} (klf)	w_{T1} (klf)	w_{L1} (klf)
Strength III (Wind at 0°)	22.7	1.7	0.0	0.03	0.05	0.03	0.04
Strength V (Wind at 60°)	83.8	0.3	1.0	0.03	0.05	0.02	0.04
Service I (Wind at 60°)	62.2	0.3	0.8	0.03	0.05	0.02	0.04

Pier Bents 11 and 12							
Load Case	P_v (kips)	At Top of Pile		Applied to Substructure			
				Pipe Piles		Square Columns	
		V_T (kips)	V_l (kips)	w_{T2} (klf)	w_{L2} (klf)	w_{T1} (klf)	w_{L1} (klf)
Strength III (Wind at 0°)	34.3	2.5	0.0	0.03	0.05	0.03	0.04
Strength V (Wind at 60°)	126.2	0.5	1.2	0.03	0.05	0.02	0.04
Service I (Wind at 60°)	93.8	0.4	1.0	0.03	0.05	0.01	0.04

Legend:

P_v = Factored Vertical Load
V_T = Factored Transverse Load
V_l = Factored Longitudinal Load
w_{T2} = Distributed Factored Longitudinal Load on the pipe pile above mudline
w_{L2} = Distributed Factored Transverse Load on the pipe pile above mudline

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w_{T1} = Distributed Factored Longitudinal Load on the square column
w_{L1} = Distributed Factored Transverse Load on the square column
The loads indicated in the tables above were provided by VHB for each bent, expressed as the load applied to each of the four vertical foundation elements. Refer to Appendix C for illustration.

6.3.5 Axial Compression Pile Resistance

The maximum factored vertical pile load provided by VHB is 155 kips (Strength I load case) at Bents 11 and 12 and 105 kips (Strength I load case) at the remaining bents. Based on the results of our engineering evaluations, the required geotechnical resistance could be obtained by driving 30-in. diameter pipe piles, closed-ended, only a few feet into the glacial till or to shallow bedrock where overburden soils are thin. Refer to Appendix C, Calculations, for supporting documentation.

However, as described below, lateral pile capacity controls the pile design, necessitating a greater embedment depth than required based on axial compressive requirements alone. Therefore piles should be driven to the minimum tip elevations shown in the table below (see Pile Tip Elevations section) or deeper and should achieve a nominal resistance equal to or greater than the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 (Table 10.5.5.2.3-1).

In order to determine whether the piles can be installed to the minimum depth required, preliminary driveability evaluations were conducted using the computer program GRL WEAP 2005 developed by GRL Engineers, Inc. The driveability analyses were conducted to determine whether the likely hammer/pile combinations used by local contractors are capable of driving the piles to the required ultimate capacity without exceeding the allowable structural capacity (i.e. damaging or overstressing the piles). Results of the preliminary pile driveability evaluations suggest that 30-in. diameter piles with a ½-in. thick wall can be driven to the depth required without overstressing (damaging) the piles. Refer to Appendix C, Calculations, for supporting documentation.

Downdrag loading on piles is not anticipated to be a significant concern since the proposed raises in grade at the abutments are minimal.

Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with a minimum yield strength of 45 ksi. We recommend that the tips of the piles be outfitted with a conical driving point to protect the tips of the piles during hard driving in the dense/very dense glacial till soils. The entire length of the piles should be filled with Class A concrete (minimum compressive strength of 4,350 psi).

6.3.6 Lateral Pile Capacity and Pile Group Evaluations

Pile design, considering the effects of combined axial and flexural loading, is required to ensure that the pile demand is less than or equal to the pile capacity while maintaining bending stresses within the allowable limits as outlined in AASHTO (0.66 Fy).

The design approach considers the non-linear response of soil with lateral displacement of the piles. Soil-structure interaction, considering this non-linear response, can be modeled using several types of computer software. Haley & Aldrich conducted the lateral pile evaluations using the computer program FB-MultiPier, Version 4 developed by Florida Bridge Software Institute. These evaluations are necessary to confirm that pile tip fixity is achieved and pile deflections are within tolerable limits based on the proposed pile configuration and loading conditions.

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As discussed above, the axial compressive loads could generally be supported by piles bearing in the glacial till deposit. However, our lateral pile evaluations indicate that fixity is not achieved at this depth. Therefore, we modeled the piles penetrating the full depth of overburden soils, to the top of bedrock, in our lateral evaluations.

The following properties for the in-situ soils were used in our lateral pile evaluations:

Soil Unit	Total Unit Weight (pcf)	Angle of Internal Friction (ϕ, °)	Undrained Shear Strength (su, psf)	Soil-Modulus Parameter (k, pci)	Strain at 50% Stress (ε_{50})
Harbor Bottom Deposit	105	-	200	-	0.02
Marine Clay Deposit	120	-	800	-	0.01
Marine Sand Deposit	120	30	-	60	-
Glacial Till	130	38	-	125	-
Bedrock	-	-	-	-	-

The soil properties were established in part based on SPT N-values, natural water content tests and Atterberg Limits tests.

At each bent, we modeled four vertical piles consisting of 30-in. diameter pipe piles with ½-in. wall thickness, driven closed-ended. Where possible, we did not include battered piles in the model. Where we determined that additional lateral resistance in the transverse direction was required (Bents 1 through 3, as noted below), we included battered piles that also consisted of 30-in. diameter pipe piles with ½-in. wall thickness. Given that the piles are located in a marine environment, we included a 1/8-in. reduction in pile cross sectional area to account for steel degradation during the bridge design life.

Our model included 12 in. by 12 in. square concrete members above the top of the pipe piles. The presence of the fiberglass form was excluded from our model since the fiberglass is not expected to provide significant stiffness over the life of the bridge. The top of the pipe pile was modeled at El. -7 for vertical piles and El. -10 for battered piles.

Based on information provided by VHB, 10 ft of scour was accounted for at each pier, unless cohesive material was encountered at a shallower depth, in which case we modeled scour to the top of the cohesive material.

The resulting deflections and moments were provided to VHB to confirm that they were in conformance with the requirements of LRFD, and are also included in Appendix C, Calculations.

The following summarizes our conclusions:

- Bent 1 – Battered piles and scour countermeasures are required. We recommend that ripap be placed within the area defined by the right of way limits (to the east and west), Bent 3 (to the north), and Abutment 1 (to the south); See recommended limits provided by email on 6 October 2011 included in Appendix C.

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- Bent 2 and 3 – Battered piles are required. Scour countermeasures are not required.

- Bent 4 through 13 – Neither battered piles or scour countermeasures are required. Based on discussions with VHB, short battered piles, consisting of steel H-piles, will be included for aesthetic purposes (driven to a minimum of 10 ft below scour depth), but they are not required to provide lateral resistance to resist design loads provided by VHB.

Plumb and "structural" batter piles (at Bents 1 through 3) should be driven to bedrock, except at Bents 8 and 9 where the glacial till is very thick and piles should be driven to the minimum tip elevations indicated in the table below.

6.3.7 Pile Settlement and Elastic Pile Compression

Pile settlement due to elastic shortening of the steel pipe piles as well as pile tip settlement was evaluated based on the maximum factored Service Limit State loads provided by VHB. The estimate of elastic pile compression for an individual pile is less than ¼ in. Pile tip settlement is anticipated to be negligible since the piles will likely penetrate through any fractured, weathered or decomposed bedrock that would otherwise be present at the pile tip. The elastic shortening of the piles is anticipated to occur primarily during construction, soon after the superstructure loads are applied.

6.3.8 Pile Tip Elevations

Plumb Piles

As discussed previously, most of the plumb piles (with the exception of the piles driven for Bents 8 and 9) will be driven to bedrock to meet lateral capacity demands. We do not anticipate that the piles will penetrate an appreciable depth into the bedrock, therefore the recommended tip elevations for estimating pile lengths are based on interpolated bedrock elevations encountered in the test borings. All plumb pipe piles should be driven to the minimum pile tip elevation shown in the table below or deeper and should achieve a minimum nominal resistance (ultimate capacity) as shown in the table below.

Substructure	Minimum Pile Tip Elevation	Minimum Nominal Pile Resistance (kips)
Bent 1	El. -32	161.5
Bent 2	El. -40	161.5
Bent 3	El. -47	161.5
Bent 4	El. -52	161.5
Bent 5	El. -54	161.5
Bent 6	El. -55	161.5
Bent 7	El. -64	161.5
Bent 8	El. -68	161.5
Bent 9	El. -68	161.5
Bent 10	El. -67	161.5
Bent 11	El. -60	238.5
Bent 12	El. -48	238.5
Bent 13	El. -42	161.5
Abutment No. 2	El. -31	161.5

We recommend that the order lengths of the piles reflect a minimum additional 5 ft of length in order to accommodate dynamic pile testing instrumentation.

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Batter Piles

As described above, the battered piles at Bents 1 through 3 should be driven to bedrock to meet lateral capacity demands. The remaining battered piles should be included for aesthetic purposes and driven to a minimum of 10 ft below scour depth, but they are not required to provide lateral resistance to resist design loads provided by VHB. The following table summarizes driving criteria for the battered piles:

Substructure	Minimum Pile Tip Elevation	Minimum Nominal Pile Resistance (kips)	Pile Type
Bent 1	El. -32	161.5	pipe pile
Bent 2	El. -40	161.5	pipe pile
Bent 3	El. -47	161.5	pipe pile
Bent 4	El. -37	-	H-pile
Bent 5	El. -39.5	-	H-pile
Bent 6	El. -41	-	H-pile
Bent 7	El. -42	-	H-pile
Bent 8	El. -39.5	-	H-pile
Bent 9	El. -37	-	H-pile
Bent 10	El. -35.5	-	H-pile
Bent 11	El. -32	-	H-pile
Bent 12	El. -25	-	H-pile
Bent 13	El. -20	-	H-pile

6.3.9 Corrosion and Deterioration

In accordance with the BDG, we recommend that the piles be protected with a fusion bonded epoxy (FBE) coating. We recommend that the FBE coating extend from the top of the pile (cutoff elevation) to either 10 ft below the existing mudline elevation or 2 ft below the total predicted scour depth, whichever is deeper. In addition, we recommend that the Contractor use special precautions during pile handling, storage and installation to prevent damage to the FBE coating prior to pile installation.

Furthermore, due to deleterious effects of the marine environment combined with the corrosive potential of existing fill and harbor bottom soils we recommend that aluminum alloy cathodic protection anodes be used in conjunction with the FBE coating to prevent steel degradation and extend the design life of the proposed pier expansion.

6.4 Abutment and Wall Design Recommendations

There are no new abutment or wall designs on the project. Some upper stones on the existing masonry wall and some metal tie rods will be replaced in-kind. Based on correspondence with VHB, abutment and wall design recommendations are not required.

6.4.1 Frost Protection

The minimum depth of embedment/cover for walls or other below grade structures was evaluated in accordance with Section 5.2.1 of the MaineDOT BDG and Sections 10.6.1.2 and 11.10.2.2 of the LRFD Bridge Design Specifications. Based on a design freezing index equal to 1,000 freezing degree days, we recommend that the footings and walls bear a minimum of 5 ft below the lowest adjacent ground surface exposed to freezing. Refer to Appendix C, Calculations, for supporting documentation.

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7. CONSTRUCTION CONSIDERATIONS

7.1 Temporary Earth Support

Depending on space limitations, sloped open-cut excavations may be feasible for the limited excavation required for the project. If space limitations necessitate lateral support, steel sheeting or soldier piles and lagging systems appear feasible.

In general, temporary earth support system(s) are the responsibility of the Contractor and should be designed by a Licensed Professional Engineer in the State of Maine. We recommend that temporary earth support system(s) be designed to support all appropriate combinations of earth, water and surcharge loads (from traffic, construction equipment, material stockpiles and other sources) imposed on the system(s) during all phases of the construction period. The Contractor is responsible for choosing an applicable factor of safety for the earth support system(s). The Contractor's design shall also consider the means and methods and construction sequencing proposed by the Contractor. We recommend that design calculations and shop drawings be prepared by the Contractor and stamped by a Licensed Professional Engineer in the State of Maine and be submitted to MaineDOT for review prior to construction.

7.2 Conflicts with Existing Piles

The existing timber piles will conflict with the proposed pipe piles at each bent location, since the proposed bents will be constructed in the same plan location as the existing bridge (to preserve the historic nature of the bridge). Therefore we recommend that the existing timber piles be extracted to avoid interference with the new piles. We anticipate that a vibratory hammer could be used to extract the piles. The plans/specifications should be set up so that removal of existing piles is considered incidental to pile installation.

7.3 Dynamic Pile Load Testing Program

We recommend that the final factored structural resistances be confirmed in the field using dynamic methods. The piles should be driven to a nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 (Table 10.5.5.2.3-1). We recommend that the Contractor perform one dynamic pile load test with 24-hour (minimum) restrike tests at each of the following locations: Bent 1, 8, 11 and 13. We recommend that the dynamic pile load tests be completed on the first piles driven at each of these four bents. In addition, we recommend that CAPWAP analyses be performed on each of these four piles.

As noted above, piles should be driven to the minimum pile tip elevation shown in the table or deeper <u>and</u> should achieve a nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65.

7.4 Reuse of Excavated On-Site Soils

Based on our current understanding of the project, we do not anticipate that significant volumes of on-site soils will need to be excavated to construct the replacement bridge.

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8. LIMITATIONS OF RECOMMENDATIONS

This report is prepared for the exclusive use of MaineDOT relative to Replacement of Sewall's Bridge over York River, in York, Maine. There are no intended beneficiaries other than MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from MaineDOT and from Haley & Aldrich indicating that the report is adequate for such other use. Use of this report by such other person or entity without the written authorization of MaineDOT and Haley & Aldrich shall be at such other person's or entities sole risk, and shall be without legal exposure or liability to Haley & Aldrich.

The analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report.

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REFERENCES

1. Bothner, Wallace A., Hussey, Arthur M. and Peter J. Thompson (2008), Kittery Quadrangle, Maine-New Hampshire, Bedrock Geology, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File Report No. 08-78 2008.

2. Cameron, Cornelia C., Clinch, J. Michael and Patrick B. O'Toole (1999), York Harbor Quadrangle, Maine, Surficial Geology, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File Report No. 99-107 1999.

3. Hussey, Arthur M. (1962), The Geology of Southern York County, Maine, Maine Geological Survey, Department of Economic Development, Augusta, Maine, Special Geologic Studies Series No. 4,1962.

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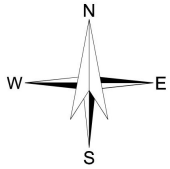
SHEETS

SITE COORDINATES:43°8'10"N 70°39'39"W



U.S.G.S. QUADRANGLE: YORK HARBOR, ME

HALEY & ALDRICH

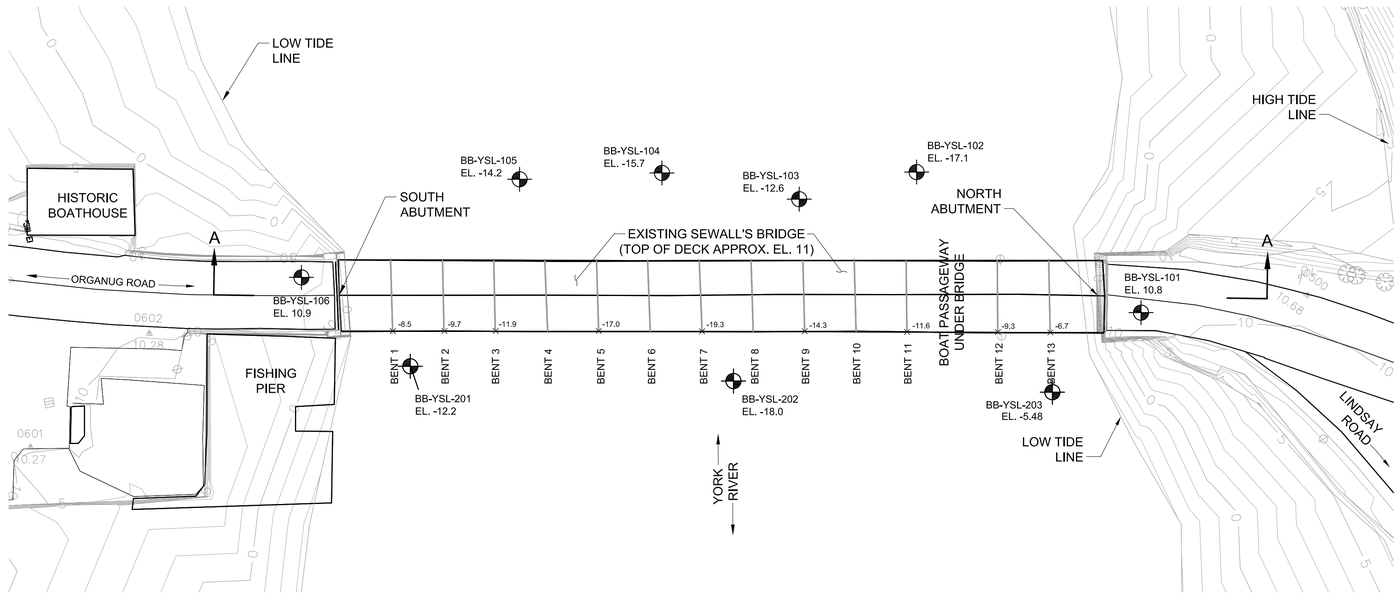
PROPOSED SEWALL'S BRIDGE REPLACEMENT
ORGANUG ROAD, YORK, MAINE
MAINEDOT WIN 12665.00

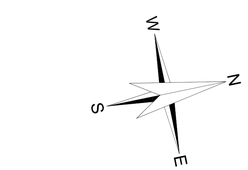
PROJECT LOCUS

SCALE: 1:24,000
DECEMBER 2011

SHEET 1

36654 2.PDF





0 25 50 75 100

SCALE IN FEET

LEGEND:

BB-YSL-201 EL. -12.2 — DESIGNATION, LOCATION AND GROUND SURFACE ELEVATION OF TEST BORING DRILLED BY NEW HAMPSHIRE BORING OF LONDONDERRY, NEW HAMPSHIRE IN JUNE 2011

BB-YSL-101 EL. 10.8 — DESIGNATION, LOCATION AND GROUND SURFACE ELEVATION OF TEST BORING DRILLED BY MAINE TEST BORINGS OF BREWER, MAINE IN NOVEMBER AND DECEMBER 2009

5 — ELEVATION CONTOUR OF EXISTING GROUND SURFACE

×-8.5 — MUDLINE SPOT ELEVATION BASED ON FIELD MEASUREMENTS BY VHB (SEE NOTE 2)

A A — DESIGNATION, LOCATION AND ORIENTATION OF SUBSURFACE PROFILE

NOTES:

1. EXISTING SITE CONDITIONS, CONTOURS OF EXISTING GROUND SURFACE ELEVATIONS AND LOCATION AND ORIENTATION OF EXISTING SITE FEATURES ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY MAINEDOT, DATED 21 DECEMBER 2009.
2. MUDLINE SPOT ELEVATIONS SHOWN BELOW BRIDGE BENTS BASED ON INSPECTION NOTES PROVIDED BY VHB DATED 5 OCTOBER 2009. ELEVATIONS BASED ON ASSUMED TYPICAL PILE BUTT LEVEL AT EL. 10.
3. AS-DRILLED LOCATIONS AND GROUND SURFACE/MUDLINE ELEVATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING OPTICAL SURVEY EQUIPMENT.
4. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
5. SUBSURFACE EXPLORATIONS WERE MONITORED IN THE FIELD BY HALEY & ALDRICH PERSONNEL.
6. REFER TO APPENDIX A FOR LOGS OF TEST BORINGS.

PRELIMINARY PLANS

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1266(500)X

WIN
BRIDGE NO. 3096 12665.00 BRIDGE PLANS

PROJ. MANAGER	N. BENOIT	BY	DATE
DESIGN-DETAILED	KFP	KFP	12/21/11
CHECKED-REVIEWED	EFW	WAC	12/21/11
DESIGN2-DETAILED2	...	...	...
DESIGN3-DETAILED3	...	...	...
REVISIONS 1	...		...
REVISIONS 2	...		...
REVISIONS 3	...		...
REVISIONS 4	...		...
FIELD CHANGES	...		...

SIGNATURE

P.E. NUMBER

DATE

SEWALL'S BRIDGE
YORK RIVER
YORK COUNTY
YORK

SITE & SUBSURFACE EXPLORATION PLAN

SHEET NUMBER
2
OF 4

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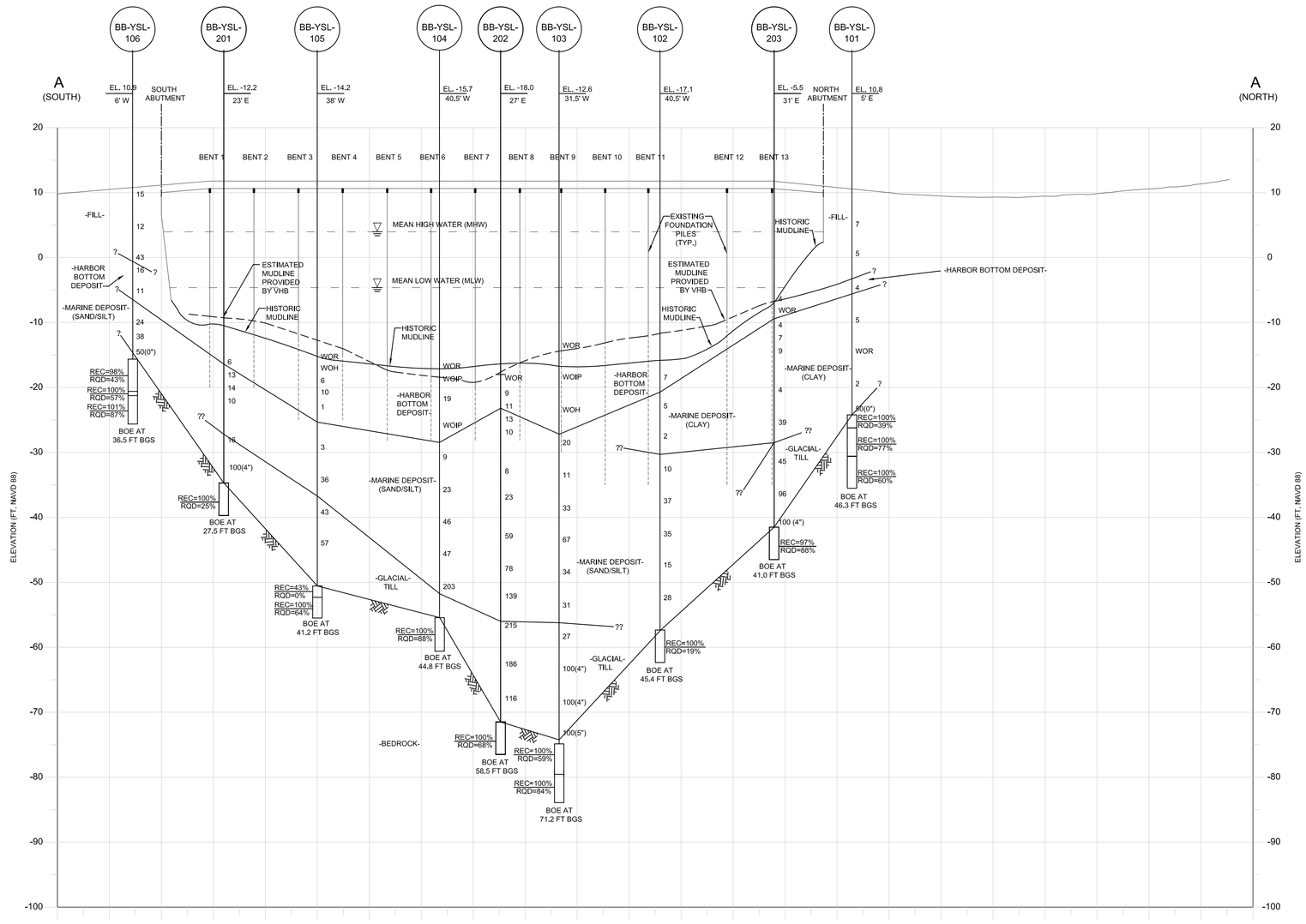

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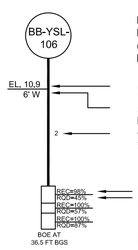

DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY MAINE TEST BORINGS OF BREWER, MAINE BETWEEN 11 NOVEMBER AND 10 DECEMBER 2009 (BB-YSL-100 SERIES) AND BY NEW HAMPSHIRE BORING OF LONDONDERRY, NEW HAMPSHIRE BETWEEN 8 AND 15 JUNE 2011 (BB-YSL-200 SERIES)."

GROUND SURFACE / MUDLINE ELEVATION AT TEST BORING LOCATION

APPROXIMATE PERPENDICULAR OFFSET DISTANCE AND DIRECTION MEASURED FROM THE SUBSURFACE PROFILE LINE (CONSTRUCTION CENTERLINE)
CORRECTED STANDARD PENETRATION TEST (SPT) N-VALUE (N_{60} BLOWS PER FOOT);
WOR = WEIGHT OF RODS; WOH = WEIGHT OF HAMMER; WO1P = WEIGHT OF ONE PERSON

PERCENT RECOVERY
PERCENT ROCK QUALITY DESIGNATION (RQD)

DENOTES BOTTOM OF EXPLORATION AT APPROXIMATE DEPTH SHOWN

NOTES:

1. SEE BORING LOCATION PLAN FOR THE LOCATION AND ORIENTATION OF THE SUBSURFACE PROFILE (EXISTING BRIDGE CENTERLINE).
2. THE LOCATION AND ORIENTATION OF THE EXISTING BRIDGE DECK AND BENTS, FOUNDATION PILE LENGTHS AND EMBEDMENTS, AND ESTIMATED MUDLINE (ALONG EAST END OF BRIDGE) ARE TAKEN FROM THE ELECTRONIC PDF FILE ENTITLED "PARTIAL ELEVATION", PROVIDED TO HALEY & ALDRICH, INC. BY VHB ON 4 JANUARY 2010.
3. THE HISTORIC MUDLINE, GROUND SURFACE OF THE APPROACH EMBANKMENTS AND ABUTMENT GEOMETRY WERE SCALED FROM HISTORIC DRAWINGS OF EXISTING BRIDGE PROVIDED BY MAINEDOT, AND SHOULD BE CONSIDERED APPROXIMATE.
4. MEAN ELEVATIONS ARE BASED ON NOAA NATIONAL OCEAN SERVICE TIDAL REFERENCE STATION #8423898 LOCATED AT FORT POINT, NEWCASTLE ISLAND, NEW HAMPSHIRE. ELEVATIONS FROM THE NATIONAL TIDAL DATUM EPOCH (NTDE) OF 1983 TO 2001 HAVE BEEN ADJUSTED TO THE PROJECT VERTICAL DATUM OF NAVD 88.
5. "AS-DRILLED" LOCATIONS AND GROUND SURFACE / MUDLINE ELEVATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING OPTICAL SURVEY EQUIPMENT.
6. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
7. LINES REPRESENTING CHANGES IN STRATA SHOWN ON THE PROFILE ARE BASED ON LINEAR INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS AND DEPICT ANTICIPATED SUBSURFACE CONDITIONS ALONG THE EXISTING BRIDGE CENTERLINE. THE ACTUAL CONDITIONS WILL DIFFER FROM THOSE SHOWN.

HORIZ. 0 25 50 75 100
VERT. 0 10 20 30 40
SCALE IN FEET

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1266(500)X

WIN 12665.00
BRIDGE NO. 3096

BRIDGE PLANS

	N. BENOIT	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	KFP	KFP	12/21/11
CHECKED-REVIEWED	EFW	WAC	12/21/11
DESIGN2-DETAILED2	. . .	. . .	. . .
DESIGN3-DETAILED3	. . .	. . .	. . .
REVISIONS 1	. . .	. . .	. . .
REVISIONS 2	. . .	. . .	. . .
REVISIONS 3	. . .	. . .	. . .
REVISIONS 4	. . .	. . .	. . .
FIELD CHANGES	. . .	. . .	. . .

SIGNATURE
P.E. NUMBER
DATE

SEWALL'S BRIDGE
YORK RIVER
YORK COUNTY
YORK

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

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TABLE I
Summary of Subsurface Explorations
Proposed Sewall's Bridge Rehabilitation
Organug Road - York, Maine

MaineDOT Win: 12665.00, Bride No. 3096
Haley & Aldrich File No.: 36654-001

LOCATION DATA:

Test Boring No.[1]	Coordinates[2]	
	Easting	Northing
BB-YSL-101	908,043.6	110,629.6
BB-YSL-102	907,983.2	110,566.4
BB-YSL-103	907,984.1	110,526.7
BB-YSL-104	907,966.5	110484.1
BB-YSL-105	907,959.0	110,437.7
BB-YSL-106	907,976.2	110,360.6
BB-YSL-201	908,012.2	110,389.8
BB-YSL-202	908,038.5	110,493.3
BB-YSL-203	908,063.4	110,595.6

SUBSURFACE DATA:

Test Boring No.[1]	Ground Surface Elevation[3,4]	Approximate Strata Thickness[5] (ft)					Approximate Elevation of Top of Bedrock[3,4]	Approximate Elevation of Bottom of Exploration[3,4]
		Bituminous Concrete / Fill	Harbor Bottom Deposit	Marine Deposit		Glacial Till		
				Clay	Sand / Silt			
BB-YSL-101	10.8	13.5	3.0	18.5	NE	NE	-24.2	-35.5
BB-YSL-102	-17.1	NE	3.5	9.5	27.4	NE	-57.5	-62.5
BB-YSL-103	-12.6	NE	14.0	NE	29.5	18.0	-74.1	-83.8
BB-YSL-104	-15.7	NE	13.0	NE	23.0	3.7	-55.4	-60.5
BB-YSL-105	-14.2	NE	11.0	NE	12.0	13.1	-50.3	-55.4
BB-YSL-106	10.9	11.5	5.5	NE	8.7	NE	-14.8	-25.6
BB-YSL-201	-12.2	NE	4.2	NE	10.8	7.5	-34.7	-39.7
BB-YSL-202	-18.0	NE	5.2	NE	32.8	15.5	-71.5	-76.5
BB-YSL-203	-5.5	NE	4.0	19.0	NE	13.0	-41.5	-46.5

Notes:
[1] Test boring locations are shown on Sheet 2, Site and Subsurface Exploration Plan.
[2] As-drilled coordinates of test borings were determined by MaineDOT using optical survey equipment and reference the NAD83, Maine 2000 West Zone coordinate system.
[3] Ground surface/mudline elevations at test boring locations were determined in the field by MaineDOT using optical survey equipment.
[4] Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[5] "NE" indicates stratum was not encountered in test boring.

	Individual	Date
Prepared By:	EFW	12/19/2011
Checked By:	BCS	12/20/2011
Reviewed By:	WAC	12/23/2011

APPENDIX A

Test Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-101
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.8	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	11/12/09-11/12/09	**Drilling Method:**	SSA to 5', NW to 35'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110629.6, E908043.6)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	12/4	0.8 - 1.8	10/50			SSA	10.10	-Bituminous Concrete- 0.7	
									Dark brown, dry, very dense, medium SAND, some fine gravel, little coarse to fine sand -FILL-(SW) Note: Brick fragments in wash water.	
5	2D	24/10	5.0 - 7.0	5/4/3/2	7	7	14	5.80	5.0 Dark brown, wet, medium stiff, sandy SILT, little fine gravel -FILL-(ML)	
							15			
							10			
							8			
							14			
10	3D	24/5	10.0 - 12.0	3/2/3/9	5	5	12		Dark brown, wet, medium stiff, sandy SILT, some gravel, trace brick -FILL-(ML)	
							18			
							40			
							24	-2.70	13.5	
							16			
15	4D	24/18	15.0 - 17.0	3/2/2/5	4	4	17		Gray-brown, wet, loose, silty SAND with organics, organic odor, trace shells, trace brick fragments -HARBOR BOTTOM DEPOSIT-(SM)	
							20	-5.70	16.5 Gray-brown, wet, medium stiff, mottled, silty CLAY with black organic pockets -MARINE DEPOSIT-(CL)	
							23			
							37			
							55			
20	5D	24/20	20.0 - 22.0	3/2/3/2	5	5	37		Brown-gray, wet, medium stiff, mottled silty CLAY with frequent fine sand seams and layers -MARINE DEPOSIT-(CL)	WC=31.9%
							36			
							35			
							34			
25							26			

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-101
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.8	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	11/12/09-11/12/09	**Drilling Method:**	SSA to 5', NW to 35'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110629.6, E908043.6)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.0 - 27.0	1/WOR/WOR/WO1P			30		Gray, wet, very soft, silty CLAY, with frequent fine sand seams -MARINE DEPOSIT-(CL) Note: Attempted vane shear test at 27.0 ft. Unable to push vane to full depth of test due to probable sand layer.	WC=32.4% LL=35, PL=18 PI=17
							37			
							20			
							26			
							22		Note: Sand and gravel observed in wash water at 29.0 ft.	
30	7D	24/5	30.0 - 32.0	1/1/1/1	2	2	31		Gray, wet, soft, silty CLAY, one gravel piece -MARINE DEPOSIT-(CL)	WC=38.3%
							25			
							25			
							22			
							25			
35								-24.20	35.0	
	MD/R1	23/23	35.0 - 36.9	50(0") RQD = 39%					Top of Bedrock at El.-24.2 R1: Gray, aphanitic, fine-grained METASILTSTONE, hard, fresh, joints dipping at horizontal to moderate angles, occasional steeply dipping joints, very close to closely spaced, tight to open, one infilled joint with gray silty fine sand coating, slight oxidation on some joint surfaces. Rock Mass Quality=Poor -KITTERY FORMATION- R1 Core Times (min:sec): 35.0-36.0'(4:00), 36.0-36.9'(3:00)	UC#9592 qp=48,338 psi
	R2	53/53	36.9 - 41.3	RQD = 77%					R2: Gray, aphanitic, fine-grained METASILTSTONE, hard, fresh, joints dipping at moderate to steep angles, very close to moderately close, occasional calcite coatings on some joint surfaces, one healed, near vertical secondary joint from approximately 39.8 to 41.0 ft infilled with quartz/calcite. Rock Mass Quality=Good -KITTERY FORMATION- R2 Core Times (min:sec): 36.9-37.9'(3:00), 37.9-38.9'(3:00), 38.9-39.9'(4:00), 39.9-40.9'(4:00)	
40	R3	60/60	41.3 - 46.3	RQD = 60%					R3: Gray, aphanitic, METASILTSTONE, hard, fresh, joints dipping at low to steep angles, very close to moderately close, occasional oxidation and/or calcite coatings on some joint surfaces. Highly fractured, slightly weathered zone from approximately 45.9 to 46.3 ft. Rock Mass Quality=Fair -KITTERY FORMATION- R3 Core Times (min:sec): 41.3-42.3'(4:00), 42.3-44.3'(4:00), 44.3-45.3'(4:00), 45.3-46.3'(4:00)	
45								-35.50	46.3	
50									**Bottom of Exploration at 46.3 feet below ground surface.**	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-102
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-17.1	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/10/09-12/10/09	**Drilling Method:**	SSA to 5', NW to 39.8'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110566.4, E907983.2)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.0 - 2.0	3/3/4/5	7	7	SSA		Gray with dark-gray, wet, medium stiff, CLAY, little organic silt, trace fine gravel and medium sand, little fine sand, shell fragments throughout, organic odor -HARBOR BOTTOM DEPOSIT-(CL)	G#172171 A-4, CL WC=30.3%
								-20.60	3.5	
5	2D	24/6	5.0 - 7.0	3/2/3/3	5	5	6		Gray, wet, medium stiff, CLAY, trace silt -MARINE DEPOSIT-(CL)	WC=37.4%
							6			
							8			
							9			
							13			
10	MD		10.0 - 12.0	4/1/1/4	2	2	6		No Recovery, spoon and casing action indicate presence of sand/gravel layers, probable that sampler pushed on gravel.	
							15			
								-30.10	13.0	
							18		Note: Wash water changed to brown at 13.8 ft.	
							16			
15	3D	24/12	15.0 - 17.0	4/4/6/8	10	10	16		Brown, wet, loose, coarse to fine SAND, trace silt, trace coarse gravel -MARINE DEPOSIT-(SM/ML)	
							24			
							23			
							31			
							37			
20	4D	24/16	20.0 - 22.0	4/18/19/24	37	37	48	-37.10	20.0 Brown, wet, dense, fine SAND, some silt -MARINE DEPOSIT-(SM/ML)	
							52			
							95			
							98			
25							113			

Remarks:

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2. Hammer consisted of rope and cathead and safety hammer.

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Boring No.: BB-YSL-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-102
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-17.1	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/10/09-12/10/09	**Drilling Method:**	SSA to 5', NW to 39.8'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110566.4, E907983.2)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/16	25.0 - 27.0	16/20/15/13	35	35	67			Brown, wet, dense, fine SAND, some silt -MARINE DEPOSIT-(SM)	G#172176 A-2-4, SM
							72				
							104				
							123	-45.60		28.5	
							144				
30	6D	24/12	30.0 - 32.0	WOH/6/9/9	15	15	19			Gray, wet, medium dense, fine SAND, trace medium sand, little silt -MARINE DEPOSIT-(SM)	G#172177 A-2-4, SM
							24				
							31				
							41				
							42				
35	7D	24/10	35.0 - 37.0	17/15/13/13	28	28	24			Gray, wet, medium dense, medium to fine SAND, trace silt, trace fine gravel (SW-SM)	
							20				
							61				
							126				
							150*				
40	R1	60/60	40.4 - 45.4	RQD = 19%				-57.50		40.4	
										Top of Bedrock at El.-57.5 R1: Gray, aphanitic, METASILTSTONE. Moderately hard, moderately to slightly weathered. Primary joints dipping at moderate angles, close, open to healed, smooth to rough, planar to stepped, slightly to moderately weathered with calcification and silt infilling present on some joint surfaces. Secondary joints dipping at vertical angles, close, open to healed, rough, undulating, slightly to moderately weathered, with calcification present. Rock Mass Quality=Very Poor -KITTERY FORMATION- R1 Core Times (min:sec): 40.4-41.4'(4:00), 41.4-42.4'(4:00), 42.4-43.4'(4:00), 43.4-44.4'(5:00), 44.4-45.4'(5:00)	
45								-62.50		45.4	
										Bottom of Exploration at 45.4 feet below ground surface.	
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-103
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-12.6	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/07/09-12/08/09	**Drilling Method:**	SSA to 5', NW to 61.8'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110526.7, E907984.1)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/1	0.0 - 2.0	WOR/WOR/WOR/ WOR			SSA			Dark gray, wet, very soft, ORGANIC SILT, trace fine sand, strong organic odor, shell fragments throughout -HARBOR BOTTOM DEPOSIT-(OL-OH)	
5	2D	24/8	5.0 - 7.0	WO1P/WO1P/WO1P/5						Dark gray, wet, very loose, silty fine SAND, trace medium to coarse sand, trace gravel, strong organic odor, shell fragments throughout -HARBOR BOTTOM DEPOSIT-(SC)	G#172172 A-4, SC WC=47.1% LL=30, PL=22 PI=8
							3				
							4				
							4				
10	3D	24/14	10.0 - 12.0	WOH/WOH/WOH/1			5			Gray to gray-brown, wet, very loose, silty fine SAND, some organic clay, trace medium to coarse sand, trace gravel, shell fragments in top 4 in. of recovery. Wood and plant fibers in bottom 10 in., organic odor -HARBOR BOTTOM DEPOSIT-(SC)	
							6				
							6				
							13				
							25	-26.60		14.0 Note: Drill action indicates stratum change at 14.0 ft.	
15	4D	24/4	15.0 - 17.0	18/13/7/4	20	20	22			Gray, wet, medium dense, coarse to fine SAND, trace silt, trace fine gravel -MARINE DEPOSIT-(SW-SM)	
							21				
							24				
							35				
							39				
20	5D	24/8	20.0 - 22.0	9/4/7/13	11	11	28	-32.60		20.0 Gray, wet, stiff, SILT, little fine sand -MARINE DEPOSIT-(ML)	G#172178 A-4, ML
							34				
							46				
							67				
25							78				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-103
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-12.6	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/07/09-12/08/09	**Drilling Method:**	SSA to 5', NW to 61.8'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110526.7, E907984.1)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
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LL = Liquid Limit
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PI = Plasticity Index
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.0 - 27.0	5/15/18/19	33	33	43	-37.60		Gray, wet, dense, fine SAND, little silt -MARINE DEPOSIT-(SM) (25.0)	G#172179 A-2-4, SM
							64				
							104				
							182				
							163				
30	7D	24/16	30.0 - 32.0	17/30/37/36	67	67	164			Gray, wet, very dense, fine SAND, little silt -MARINE DEPOSIT-(SM)	
							203				
							234				
							268				
							301				
35	8D	24/14	35.0 - 37.0	8/15/19/22	34	34	180			Gray, wet, dense, fine SAND, little silt -MARINE DEPOSIT-(SM)	
							244				
							293				
							350			Note: Washed ahead to 40.0 ft, then drove casing.	
							180				
40	9D	24/16	40.0 - 42.0	4/13/18/22	31	31	107			Gray, wet, dense, fine SAND, little silt -MARINE DEPOSIT-(SM)	
							115				
							90				
							58	-56.10		(43.5)	
							67				
45	10D	24/18	45.0 - 47.0	4/8/19/34	27	27	100			Gray, wet, medium dense, silty fine SAND, trace medium to coarse sand, little gravel, occasional pockets of medium sand and fine gravel, occasional weak bonding -GLACIAL TILL-(SC) Note: Washed ahead of casing to 50.0 ft.	G#172180 A-4, SC
							107				
							142				
							150				
50							246				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-103
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-12.6	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/07/09-12/08/09	**Drilling Method:**	SSA to 5', NW to 61.8'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110526.7, E907984.1)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	10/6	50.0 - 50.8	84/100(4")			43			Gray, wet, very dense, fine SAND, little medium to coarse sand, little coarse to fine gravel, some silt, bonded -GLACIAL TILL-(SM) Note: Washed ahead of casing to 55.0 ft.	G#172181 A-2-4, SM
							34				
							44				
							49				
							164				
55	12D	10/8	55.0 - 55.8	90/100(4")			36			Gray, wet, very dense, coarse to fine SAND, little silt, little coarse to fine gravel, occasional bonding and stratification -GLACIAL TILL-(SW-SM) Note: Washed ahead of casing to 60.0 ft.	
							44				
							52				
							60	-71.10		58.5	
							42	-72.30		-COBBLES/GRAVEL- 59.7	
60	13D	11/6	60.0 - 60.9	70/100(5")			36			Gray, wet, very dense, coarse to fine SAND, little silt, little coarse to fine gravel, bonded -GLACIAL TILL-(SW-SM)	
	R1	54/54	61.9 - 66.4	RQD = 59%			245/(6")	-74.10		61.5 Top of Bedrock at El.-74.1 R1: Gray, aphanitic, QUARTZITE. Moderately hard, slightly to moderately weathered. Primary joints dipping at moderate to steep angles, very close to close, open to healed, smooth to rough, planar to stepped, slightly to moderately weathered with silt infilling and occasional calcification of joint surfaces. Secondary joints dipping at low to horizontal angles, wide, open, rough, stepped, slightly weathered. Rock Mass Quality=Fair -KITTERY FORMATION- R1 Core Times (min:sec): 61.9-62.9'(4:00), 62.9-63.9'(3:00), 63.9-64.9'(3:00), 64.9-65.9'(3:00), 65.9-66.4'(2:00)	
65											
	R2	57/57	66.4 - 71.2	RQD = 84%						R2: Gray, aphanitic, QUARTZITE. Hard to moderately hard, slightly weathered. Primary joints dipping at steep angles, close, open to healed, rough, planar to undulating, slightly weathered with calcification present on joint surfaces. Secondary joints wide, open, rough, stepped, slightly weathered. Rock Mass Quality=Good -KITTERY FORMATION- R2 Core Times (min:sec): 66.4-67.4'(3:00), 67.4-68.4'(3:00), 68.4-69.4'(3:00), 69.4-70.4'(4:00), 70.4-71.2'(5:00)	
70											
								-83.80		71.2 **Bottom of Exploration at 71.2 feet below ground surface.**	
75											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-104
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-15.7	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/4/09-12/4/09	**Drilling Method:**	SSA to 10', NW to 39.7'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110484.1, E907966.5)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD		0.0 - 2.0	WOR/WOR/WOR/ WO1P			SSA			No Recovery	
	1D	24/2	2.0 - 4.0	WO1P/WO1P/WO1P/ WO1P						Dark gray, wet, very soft, organic SILT, little medium to fine sand, shells throughout, organic odor -HARBOR BOTTOM DEPOSIT-(OL-OH)	
5	2D	24/8	5.0 - 7.0	16/14/5/3	19	19				Dark gray to gray, wet, very stiff, SILT, little fine sand, trace medium to coarse sand, shells throughout, organic odor -HARBOR BOTTOM DEPOSIT-(ML)	G#172173 A-4, ML WC=27.3% NP
10	3D	24/20	10.0 - 12.0	WOR/WOR/WO1P/ WO1P			3			Gray, wet, very soft, organic SILT, little fine sand, trace medium to coarse sand, wood fragments and plant fibers throughout top 16 in. of recovery, slight organic odor -HARBOR BOTTOM DEPOSIT-(ML)	
							3				
							6	-28.70		13.0	
							42				
							37				
15	4D	24/6	15.0 - 17.0	22/7/2/2	9	9	8			Gray, wet, loose, coarse to fine SAND, some fine gravel, little silt, grading from coarse to fine -MARINE DEPOSIT-(SM)	G#172182 A-1-b, SM
							6				
							11	-33.70		18.0	
							25				
							29				
20	5D	24/4	20.0 - 22.0	WOH/6/17/25	23	23	19			Gray, wet, medium dense, fine SAND, some silt, little medium sand, trace coarse sand, trace coarse gravel, piece of gravel lodged in tip of spoon -MARINE DEPOSIT-(SM)	
							40				
							59				
							64				
25							99				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. NP=non plastic determined from Atterberg limits laboratory test.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-104
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-15.7	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/4/09-12/4/09	**Drilling Method:**	SSA to 10', NW to 39.7'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110484.1, E907966.5)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/20	25.0 - 27.0	WOH/18/28/28	46	46	79			Gray, wet, dense, fine SAND, some silt -MARINE DEPOSIT-(SM)	G#172183 A-2-4, SM
							70				
							145				
							150				
							220				
30	7D	24/18	30.0 - 32.0	2/21/26/39	47	47	135			Gray, wet, dense, fine SAND, little silt -MARINE DEPOSIT-(SM)	
							180				
							250				
							300				
							320			Note: Low blow counts in top 6 in. of 5D to 8D likely due to saturated loose sands beneath casing.	
35	8D	24/16	35.0 - 37.0	WOR/153/50/59	203	203	28	-51.70		36.0	
							121			Gray, wet, very dense, coarse to fine SAND, trace coarse to fine gravel, trace silt, weakly bonded in bottom 6 in. of recovery -GLACIAL TILL-(SW/SM)	
							220				
							316				
	R1	60/60	39.8 - 44.8	RQD = 88%			325(7")	-55.40		39.7	
40										Top of Bedrock at El.-55.4 R1: Gray, fine-grained to aphanitic QUARTZITE. Hard to moderately hard, very slightly weathered, steep angle joints, moderately close, open to healed, rough, undulating, discolored with calcite coating on joint surface. Rock Mass Quality=Good -KITTERY FORMATION- R1 Core Times (min:sec): 39.8-40.8'(3:00), 40.8-41.8'(3:00), 41.8-42.8'(3:00), 42.8-43.8'(3:00), 43.8-44.8'(4:00)	
								-60.50		44.8	
45										**Bottom of Exploration at 44.8 feet below ground surface.**	
50											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. NP=non plastic determined from Atterberg limits laboratory test.

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* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-105
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-14.2	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/2/09-12/3/09	**Drilling Method:**	SSA to 10', NW to 35.7'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110437.7, E907959.0)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD		0.0 - 2.0	WOR/WOR/WOR/ WO1P			SSA		No Recovery	
	1D	24/2	2.0 - 4.0	W01P/WOH/WOH/ WOH					Gray, wet, very soft, organic SILT, little fine sand, shell fragments throughout, organic odor -HARBOR BOTTOM DEPOSIT-(OL-OH)	
	2D	24/2	4.0 - 6.0	WOH/2/4/5	6	6			Dark gray, wet, medium stiff, organic SILT, little fine sand, trace medium sand, shell fragments throughout, organic odor (OL-OH)	
5	MD		6.0 - 8.0	5/7/3/3	10	10			No Recovery Residue on spoon similar to above	
	3D	24/10	8.7 - 10.7	1/WOH/1/1	1	1			Dark gray, wet, very soft, fine sandy organic SILT, trace fine gravel, little medium to coarse sand, shell fragments throughout, organic odor -HARBOR BOTTOM DEPOSIT-(OL-OH)	G#172174 A-4, OL-OH WC=34.0%
10							2			
							3	-25.20	11.0	
							30			
							36			
	4D	24/8	14.0 - 16.0	6/3/WOH/3	3	3	19		Gray, wet, loose, coarse to fine SAND, some silt, trace gravel -MARINE DEPOSIT-(SM)	G#172184 A-2-4, SM WC=10.6%
15							19			
							100(6")	-30.70	16.5	
							64	-31.70	Note: Advanced borehole from 16.5 to 17.5 ft through boulder with roller bit. -BOULDER-	
							43		17.5	
	5D	24/12	19.0 - 21.0	10/18/18/15	36	36	67		Brown, wet, fine SAND, some silt -MARINE DEPOSIT-(SM)	G#172185 A-2-4, SM
20							39			
							60			
							83		Note: Drill action indicates stratum change at 23.0 ft.	
							61	-37.20	23.0	
25	6D	24/8	24.0 - 26.0	18/21/22/19	43	43	120		Gray, wet, dense, coarse to fine SAND, little coarse to fine gravel, trace silt, moderately bonded -GLACIAL TILL-(SW-SM)	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-105
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	-14.2	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	B. Babcock	**Rig Type:**	CME 45 Barge	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	12/2/09-12/3/09	**Drilling Method:**	SSA to 10', NW to 35.7'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110437.7, E907959.0)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							71				
							69				
							70				
							75				
	7D	24/6	29.0 - 31.0	34/38/19/25	57	57	63			Gray, wet, very dense, coarse to fine GRAVEL, little coarse to fine sand, little silt, bonded -GLACIAL TILL-(GM)	G#172186 A-1-b, GM
30							68				
							84				
							115				
							126				
							139				
35							100(8")			Note: Rig chatter from 33.8 to 35.7 ft. Probable gravel/cobble layer.	
								-50.30		36.1	
	R1	23/10	36.1 - 38.0	RQD = 0%						Top of Bedrock at El.-50.3 R1: Recovery highly fractured, including heterogenous mix of gravel. One 3 in. piece of gray, fine grained, QUARTZITE. Moderately hard, slightly weathered. One steep angle joint, slightly weathered with silt coating on joint surface, open. Core run probably through glacial till layer. Rock Mass Quality=Very Poor -KITTERY FORMATION (possible glacial till)- R1 Core Times (min:sec): 36.1-37.1'(4:00), 37.1-38.0'(5:00)	
	R2	38/38	38.0 - 41.2	RQD = 64%							
40											
								-55.40		R2: Gray, fine-grained to aphanitic, QUARTZITE. Moderately hard, slightly weathered. Primary joints dipping at low to moderate angles, close, open to healed, smooth, planar to undulating, discolored to slightly weathered with calcite coatings on some joint surfaces. One vertical joint at 39.1 ft, healed. Rock Mass Quality=Fair -KITTERY FORMATION- R2 Core Times (min:sec): 38.0-39.0'(5:00), 39.0-40.0'(6:00), 40.0-41.2'(6:00)	
45										41.2	
50										**Bottom of Exploration at 41.2 feet below ground surface.** Note: Unable to advance casing beyond 36.1 ft. Borehole collapsing after completion of R2.	

Remarks:

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2. Hammer consisted of rope and cathead and safety hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-106
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.9	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	11/11/09-11/11/09	**Drilling Method:**	SSA to 5', NW to 25'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110360.6, E907976.2)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.5 - 2.5	6/7/8/8	15	15	SSA	10.40	-Bituminous Concrete- 0.5 Brown, dry, medium dense, medium to fine SAND, little coarse sand, little fine gravel, one piece of coarse gravel lodged in tip of sampler -FILL-(SW)	
5	2D	24/12	5.0 - 7.0	3/6/6/5	12	12	10		Brown, damp to wet at tip, medium dense, medium to fine SAND, some coarse sand, some fine gravel, trace silt -FILL-(SW)	
							16			
							6			
							14			
							28			
10	3D	24/3	10.0 - 12.0	15/21/22/17	43	43	12		Gray, wet, dense, coarse to fine gravel pieces, wood fragments	
							27	-0.60	11.5 Dark brown, wet, stiff, organic SILT, trace fine sand, organic odor -HARBOR BOTTOM DEPOSIT-(ML) Note: No recovery from 12 to 14 ft except for wood fragments in tip of sampler.	
	MD	24/0	12.0 - 14.0	6/10/6/5	16	16	27			
							18			
							12			
15	4D	24/12	15.0 - 17.0	4/4/7/13	11	11	23	-4.10	15.0 Dark gray, wet, medium dense, fine SAND with organics, little silt, trace shells, few brick fragments, organic odor -HARBOR BOTTOM DEPOSIT-(SP-SM)	
							23			
							50	-6.10	17.0	
							72			
							49			
20	MD	24/0	20.0 - 22.0	12/14/10/9	24	24	39		Note: Pushed on gravel, no recovery.	
							33			
	5D	24/4	22.0 - 24.0	12/20/18/20	38	38	53		Brown to gray, wet, dense, mottled coarse to fine SAND, some silt, trace fine gravel, gravel piece lodged in tip of sampler -MARINE DEPOSIT-(SM)	G#172175 A-2-4, SM
							56			
25							50			

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-106
PIN: 12665.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.9	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	140/30-SS, 300/16-NW
Date Start/Finish:	11/11/09-11/11/09	**Drilling Method:**	SSA to 5', NW to 25'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110360.6, E907976.2)	**Casing ID/OD:**	NW 3.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	8/4	25.0 - 25.7	4/3(2")/50(0")				-14.80		Brown, wet, hard, mottled SILT, little fine sand, little coarse sand, little fine gravel -MARINE DEPOSIT-(ML)	
	R1	60/59	26.4 - 31.4	RQD = 43%				-15.50		25.7 Top of Bedrock at El.-14.8 -WEATHERED BEDROCK-	UC qp=37,432 psi
										26.4 R1: Moderately hard, fresh, gray, aphanitic to fine grained, METASILTSTONE. Primary joints dipping at low to moderate angles, very close to close, tight to partly open, planar to undulating, smooth to rough, slight oxidation on some joint surfaces, occasional calcite coatings on some joint surfaces, pyrite mineralization, occasional calcite veins. One near vertical secondary joint, slightly weathered, undulating, rough, open, calcite coating. Rock Mass Quality=Poor -KITTERY FORMATION- R1 Core Times (min:sec): 26.4-27.4'(4:00), 27.4-28.4'(4:00), 28.4-29.4'(3:00), 29.4-30.4'(3:00), 30.4-31.4'(3:00)	
30	R2	7/7	31.4 - 32.0	RQD = 57%							
	R3	54/54	32.0 - 36.5	RQD = 87%							
35								-25.60		R2: Moderately hard, fresh, gray, aphanitic to fine grained, METASILTSTONE, joints dipping at moderate to high angles, very close to close, tight to partly open, planar to undulating, smooth to rough, one silt seam (in-filled joint). Rock Mass Quality=Fair -KITTERY FORMATION- R2 Core Times (min:sec): 31.4-32.0'(5:00) R3: Moderately hard, fresh, gray, aphanitic to fine grained METASILTSTONE. Primary joints dipping at moderate to high angles, close, tight, calcite coatings on some joint surfaces. Quartz intrusion from approximately 34.5 to 35.0 ft. Rock Mass Quality=Good -KITTERY FORMATION- R3 Core Times (min:sec): 32.0-33.0'(5:00), 33.0-34.0'(3:00), 34.0-35.0'(4:00), 35.0-36.0'(4:00)	
40										36.5 **Bottom of Exploration at 36.5 feet below ground surface.**	
45											
50											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine 2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-201
PIN: 12665.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	-12.2	**Auger ID/OD:**	--
Operator:	S. Cooley	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	T. Erickson	**Rig Type:**	CME 25 Barge	**Hammer Wt./Fall:**	140/24-SS, 300/30-HW
Date Start/Finish:	6/14/11-6/15/11	**Drilling Method:**	HW to 22.5', NQ to 27.5'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110389.8, E908012.2)	**Casing ID/OD:**	HW-4.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/0	0.0 - 2.0	WOR/WOR/WOR/ WOR					No Recovery	
	2D	24/14	3.0 - 5.0	6/2/4/4	6	6			Note: Split spoon taken from 3.0 to 5.0 ft due to very soft soils at mudline surface. Black and dark gray, wet, medium stiff, SILT, some fine sand, little clay, shell material, weakly cemented -HARBOR BOTTOM DEPOSIT-(ML)	
								-16.40	4.2	
5	3D	24/22	5.0 - 7.0	5/6/7/7	13	13		-17.70	Gray, wet, loose, silty SAND, little clay, trace shell material, poorly graded, slightly cemented -MARINE DEPOSIT-(SM)	
									5.5	
	4D	24/16	7.0 - 9.0	7/7/7/6	14	14		-19.70	Olive, wet, stiff, SILT, some fine sand, little silty clay, moderate to strong cementation, occasional clay seams -MARINE DEPOSIT-(ML)	
									7.5	
	5D	24/14	9.0 - 11.0	4/5/5/7	10	10			Olive-brown, wet, medium dense, silty SAND, trace clay, poorly graded, moderately cemented -MARINE DEPOSIT-(SM)	
10									Olive-brown, wet, loose, silty SAND, poorly graded, slightly cemented, occasional stratification -MARINE DEPOSIT-(SM)	
15								-27.20	15.0	
	6D	24/7	15.0 - 17.0	5/5/11/6	16	16			Gray-brown, wet, medium dense, SAND, some gravel, little silt, poorly graded, moderately well bonded -GLACIAL TILL-(SP-SM)	
20	7D	4/4	20.0 - 20.3	100(4.0)					Gray-brown, wet, very dense, gravelly SAND, little silt, poorly graded, moderately well bonded -GLACIAL TILL-(SP-SM)	
	R1	60/60	22.5 - 27.5	RQD = 25%				-34.70	22.5	
25									Top of Bedrock at El.-34.7 R1: Dark gray, light gray and reddish-brown streaks, aphanitic to fine grained, METASILTSTONE and QUARTZITE, moderately hard to hard. Primary joint set is dipping vertical, present from 23.5 to 26.8 ft and from 27.2 to 27.5 ft, planar to undulating, smooth to rough, tight to open, minor	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-201
PIN: 12665.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	-12.2	**Auger ID/OD:**	--
Operator:	S. Cooley	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	T. Erickson	**Rig Type:**	CME 25 Barge	**Hammer Wt./Fall:**	140/24-SS, 300/30-HW
Date Start/Finish:	6/14/11-6/15/11	**Drilling Method:**	HW to 22.5', NQ to 27.5'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110389.8, E908012.2)	**Casing ID/OD:**	HW-4.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										calcite infilling. Secondary joint set is dipping horizontal to low angle, close to moderately close, planar, smooth, tight to open, trace of calcite present. Rock Mass Quality=Very Poor -KITTERY FORMATION- R1 Core Times (min:sec): 22.5-23.5' (6:00), 23.5-24.5' (6:00), 24.5-25.5' (6:00), 25.5-26.5' (6:30), 26.5-27.5' (7:00)	
								-39.70		27.5	
										Bottom of Exploration at 27.5 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-202
PIN: 12665.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	-18.0	**Auger ID/OD:**	--
Operator:	S. Cooley	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	T. Erickson	**Rig Type:**	CME 25 Barge	**Hammer Wt./Fall:**	140/24-SS, 300/30-HW
Date Start/Finish:	6/10/11-6/13/11	**Drilling Method:**	HW to 53.5', NQ to 58.5'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110493.3, E908038.5)	**Casing ID/OD:**	HW-4.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.0 - 2.0	WOR/WOR/WOR/ WOR						Black and dark gray, wet, very loose, silty SAND with little shell material, poorly graded, weakly cemented -HARBOR BOTTOM DEPOSIT-(SM)	
	2D	24/10	2.0 - 4.0	4/4/5/7	9	9				Black and dark gray, wet, loose, silty SAND with little shell material, poorly graded, weakly cemented -HARBOR BOTTOM DEPOSIT-(SM)	
	3D	24/20	4.0 - 6.0	6/6/5/4	11	11					
5								-23.20		5.2 Gray, wet, medium dense, SAND, little silt, trace wood, poorly graded -MARINE DEPOSIT-(SP-SM)	
	4D	24/24	6.0 - 8.0	2/4/9/15	13	13				Gray, wet, medium dense, SAND, little gravel, trace silt, poorly graded -MARINE DEPOSIT-(SP)	
	5D	24/15	8.0 - 10.0	6/5/5/4	10	10				Gray, wet, loose, fine SAND, poorly graded -MARINE DEPOSIT-(SP)	
10											
								-32.00		14.0 Gray, wet, loose, SAND, some silt, trace clay, poorly graded, weakly cemented -MARINE DEPOSIT-(SM)	
	6D	24/20	14.0 - 16.0	3/4/4/7	8	8					
15											
								-36.00		18.0 Gray, wet, medium dense, fine SAND, trace clay, poorly graded -MARINE DEPOSIT-(SP)	
	7D	24/18	18.0 - 20.0	11/14/9/14	23	23					
20											
25	8D	24/24	24.0 - 26.0	19/22/37/47	59	59				Gray, wet, very dense, fine SAND, poorly graded -MARINE DEPOSIT-(SP)	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-202
PIN: 12665.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	-18.0	**Auger ID/OD:**	--
Operator:	S. Cooley	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	T. Erickson	**Rig Type:**	CME 25 Barge	**Hammer Wt./Fall:**	140/24-SS, 300/30-HW
Date Start/Finish:	6/10/11-6/13/11	**Drilling Method:**	HW to 53.5', NQ to 58.5'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110493.3, E908038.5)	**Casing ID/OD:**	HW-4.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
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q_p = Unconfined Compressive Strength (ksf)
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Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
	9D	24/20	29.0 - 31.0	29/36/42/52	78	78				Gray, wet, very dense, fine SAND, trace silt, poorly graded -MARINE DEPOSIT-(SP)	
30											
	10D	18/15	33.5 - 35.0	14/37/102	139	139				Note: Sample 10D taken at a different sampling interval due to tidal changes. Gray-brown, wet, very dense, fine SAND, poorly graded -MARINE DEPOSIT-(SP)	
35											
										Note: Drilling action indicated a strata change at 38.0 ft.	
								-56.00		38.0	
	11D	18/6	38.0 - 39.5	80/104/111	215	215				Gray, wet, very dense, gravelly SAND, trace silt, poorly graded, loosely bonded -GLACIAL TILL-(SP)	
40											
	12D	17/8	44.0 - 45.4	23/86/100(5.0)	186	186				Gray, wet, very dense, SAND, little gravel, trace silt, poorly graded -GLACIAL TILL-(SP)	
45											
50	13D	24/17	49.0 - 51.0	66/73/43/52	116	116				Gray, wet, very dense, gravelly SAND, trace silt, poorly graded, weak to moderate bonding	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-202
PIN: 12665.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	-18.0	**Auger ID/OD:**	--
Operator:	S. Cooley	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	T. Erickson	**Rig Type:**	CME 25 Barge	**Hammer Wt./Fall:**	140/24-SS, 300/30-HW
Date Start/Finish:	6/10/11-6/13/11	**Drilling Method:**	HW to 53.5', NQ to 58.5'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110493.3, E908038.5)	**Casing ID/OD:**	HW-4.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										-GLACIAL TILL-(SP)	
	R1	60.0/60.0	53.5 - 58.5	RQD = 68%				-71.50		53.5	
										Top of Bedrock at El. -71.5 R1: Gray with white streaks, aphanitic to fine grained METASILTSTONE, moderately hard to hard. Primary joint set dipping horizontal to low angle, very close to moderately close, planar to undulating, smooth to rough, tight to open, calcite infilling present at 55.5 and 56.0 ft, up to 1/8 in. thick, along joint surfaces. Secondary joint set dipping steep to vertical, very close to wide, open, traces of white calcite present along joint surfaces. Zone of low angle foliation quartz/calcite banding at approximately 55.5 ft. Rock Mass Quality=Fair	
55											
								-76.50		-KITTERY FORMATION- R1 Core Times (min:sec): 53.5-54.5' (6:00), 54.5-55.5' (7:00), 56.5-57.5' (12:00), 57.5-58.5' (11:00), 58.5-59.5' (10:00)	
60										58.5	
										Bottom of Exploration at 58.5 feet below ground surface.	
65											
70											
75											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine West Zone coordinate system.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-203
PIN: 12665.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	-5.5	**Auger ID/OD:**	--
Operator:	S. Cooley	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	T. Erickson	**Rig Type:**	CME 25 Barge	**Hammer Wt./Fall:**	140/24-SS, 300/30-HW
Date Start/Finish:	6/8/11-6/9/11	**Drilling Method:**	HW to 36.0', NQ to 41.0'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110595.6, E908063.4)	**Casing ID/OD:**	HW-4.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.0 - 2.0	1/2/2/2	4	4			Gray, wet, very loose, SAND, some silt, trace clay, poorly graded, traces of wood and shell material -HARBOR BOTTOM DEPOSIT-(SM)	
	MD	24/0	2.0 - 4.0	WOR/WOR/WOR/ WOR					No Recovery	
								-9.50	4.0	
	3D	24/14	4.0 - 6.0	3/2/2/3	4	4			Gray, wet, soft, silty CLAY, some fine sand, trace silt -MARINE DEPOSIT-(CL)	
5								-11.50	6.0	
	4D	24/20	6.0 - 8.0	2/3/4/7	7	7			Gray, wet, loose, clayey SAND, trace silt, poorly graded, occasional clay layers -MARINE DEPOSIT-(SC)	
	5D	24/22	8.0 - 10.0	2/4/5/7	9	9			Gray, wet, loose, SAND, some silty clay, trace silt, poorly graded, silty clay from 9.6 to 10.0 ft -MARINE DEPOSIT-(SC)	
10										
	6D	24/22	14.0 - 16.0	3/3/1/2	4	4			Yellow-brown (14.0 to 14.6 ft), gray, wet, very loose SAND, some silty clay, trace silt, poorly graded, silty clay (14.0 to 14.6 ft), occasional clay layers -MARINE DEPOSIT-(SC)	
15										
	7D	24/24	19.0 - 21.0	19/17/22/26	39	39			Gray (19.0 to 19.4 ft) to gray-brown, wet, dense SAND, some silty clay, trace silt, poorly graded, occasional silty clay layers (19.0 to 20.0 ft), sand from 20.0 to 21.0 ft -MARINE DEPOSIT-(SC)	
20									Note: Rollerbit action indicated a strata change at 23.0 ft.	
								-28.50	23.0	
25										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine West Zone coordinate system.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sewall's Bridge Replacement
Location: York, Maine

Boring No.: BB-YSL-203
PIN: 12665.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	-5.5	**Auger ID/OD:**	--
Operator:	S. Cooley	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	T. Erickson	**Rig Type:**	CME 25 Barge	**Hammer Wt./Fall:**	140/24-SS, 300/30-HW
Date Start/Finish:	6/8/11-6/9/11	**Drilling Method:**	HW to 36.0', NQ to 41.0'	**Core Barrel:**	NQ-2.0
Boring Location:	See Plan (N110595.6, E908063.4)	**Casing ID/OD:**	HW-4.0	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	8D	24/8	25.0 - 27.0	11/24/21/13	45	45				Gray, wet, dense, gravelly SAND, trace silt, poorly graded, loosely bonded -GLACIAL TILL-(SP)	
30	9D	24/18	30.0 - 32.0	42/49/47/48	96	96				Gray-brown, wet, very dense, gravelly SAND, little silt, poorly graded, well bonded -GLACIAL TILL DEPOSIT-(SP-SM)	
35	10D	10/5	35.0 - 35.8	22/100(4.0)				-40.50 -41.00 -41.50		35.0 Brown, wet, very dense, gravelly SAND, little silt, poorly graded, well bonded -GLACIAL TILL-(SP-SM) 35.5 Gray, wet, very dense, SAND, some gravel, little silt, poorly graded, well bonded -GLACIAL TILL-(SP-SM) 36.0	
40	R1	60/58	36.0 - 41.0	RQD = 88%				-46.50		Top of Bedrock at El. -41.5 R1: Gray with green streaks, aphanitic to fine grained METASILTSTONE, moderately hard to hard. Primary joint set dipping horizontal to low angle, only present at depth 39.8 and 40.1 ft, planar, smooth to rough, tight to open, traces of calcite infilling. One calcite infilled, healed vertical joint. Rock Mass Quality=Good -KITTERY FORMATION- R1 Core Times (min:sec): 36.0-37.0' (6:00), 37.0-38.0' (6:30), 38.0-39.0' (6:00), 39.0-40.0' (6:00), 40.0-41.0' (6:30) 41.0	
45										**Bottom of Exploration at 41.0 feet below ground surface.**	
50											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83 Maine West Zone coordinate system.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

Descriptive Term	Portion of Total
trace	0% - 10%
little	11% - 20%
some	21% - 35%
adjective (e.g. sandy, clayey)	36% - 50%

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnai
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

PIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

APPENDIX B

Laboratory Test Results

GeoTesting
express
a subsidiary of Geocomp Corporation

Boston
Atlanta
New York

www.geocomp.com/geotesting

February 2, 2010

Andy Blaisdell
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

RE: Proposed Sewall's Bridge Rehabilitation, York, ME (GTX-9592)

Dear Andy Blaisdell:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received 22 samples (20 soil and 2 rock core) from you on January 11, 2010. See the attached copy of your test for sample identifications.

GTX performed the following tests on these samples:

9 ASTM D 2216 - Moisture Content tests
5 ASTM D 422 - Grain Size Analyses with Hydrometer
11 ASTM D 422 - Grain Size Analyses (sieve only)
3 ASTM D 4318 - Atterberg Limits
2 ASTM D 7012 – Compressive Strength and Elastic Moduli of Rock

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

GeoTesting express
a subsidiary of Geocomp Corporation

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

2/2/2010

GTX-9592
Proposed Sewall's Bridge Rehabilitation

York, ME

Prepared for:

Haley & Aldrich, Inc.

GeoTesting
express
a subsidiary of Geocomp Corporation

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Transmittal

TO:
Andy Blaisdell
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 2/2/2010	GTX NO: 9592
RE: Proposed Sewall's Bridge Rehabilitation	

COPIES	DATE	DESCRIPTION
	2/2/2010	**January 2010 Laboratory Test Report**

REMARKS:

SIGNED: Joe Tomei, Laboratory Manager

CC:

APPROVED BY: Nancy Hubbard, Project Manager

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: ---	Sample Type: ---	Tested By:	mmd
Sample ID:---	Test Date: 01/18/10	Checked By:	jdt
Depth : ---	Sample Id: ---		

Moisture Content of Soil - ASTM D 2216-05

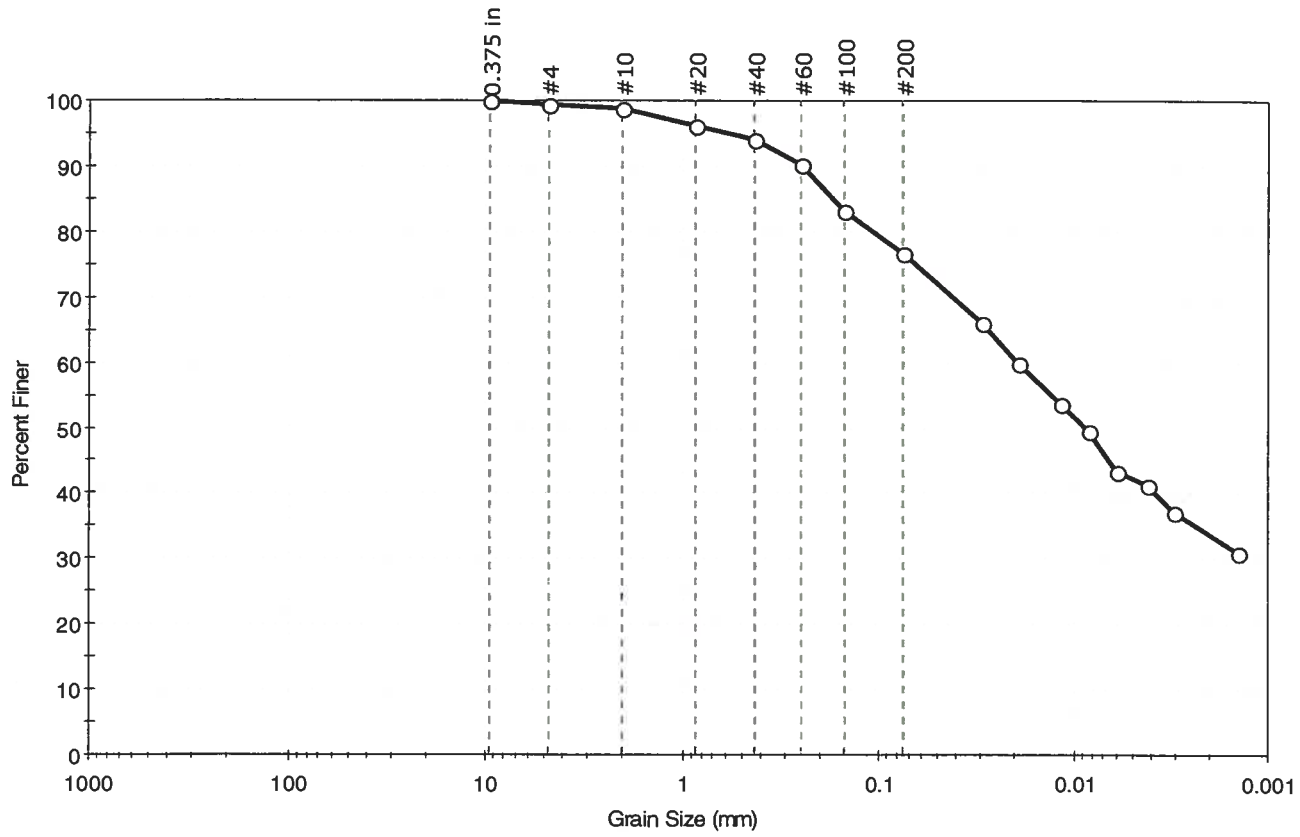
Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-YSL-101	5D	20.0-22.0 ft		31.9
BB-YSL-101	6D	25.0-27.0 ft		32.4
BB-YSL-101	7D	30.0-32.0 ft		38.3
BB-YSL-102	1D	0-2.0 ft		30.3
BB-YSL-102	2D	5.0-7.0 ft		37.4
BB-YSL-103	2D,3D	5.0-7.0,10.0-12.0 ft		47.1
BB-YSL-104	2D,3D	5.0-7.0,10.0-12.0 ft		27.3
BB-YSL-105	3D	8.7-10.7 ft		34
BB-YSL-105	4D	14.0-16.0 ft		10.6

Notes: Temperature of Drying : 110º Celsius

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-102	Sample Type: jar	Tested By:	jbr
Sample ID:1D	Test Date: 01/13/10	Checked By:	jdt
Depth : 0-2.0 ft	Test Id: 172171		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	0.6	22.8	76.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	99		
#20	0.85	96		
#40	0.42	94		
#60	0.25	90		
#100	0.15	83		
#200	0.075	77		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0293	66		
---	0.0193	60		
---	0.0116	54		
---	0.0083	50		
---	0.0060	43		
---	0.0042	41		
---	0.0031	37		
---	0.0014	31		

Coefficients

D_{85} = 0.1700 mm D_{30} = N/A
D_{60} = 0.0194 mm D_{15} = N/A
D_{50} = 0.0086 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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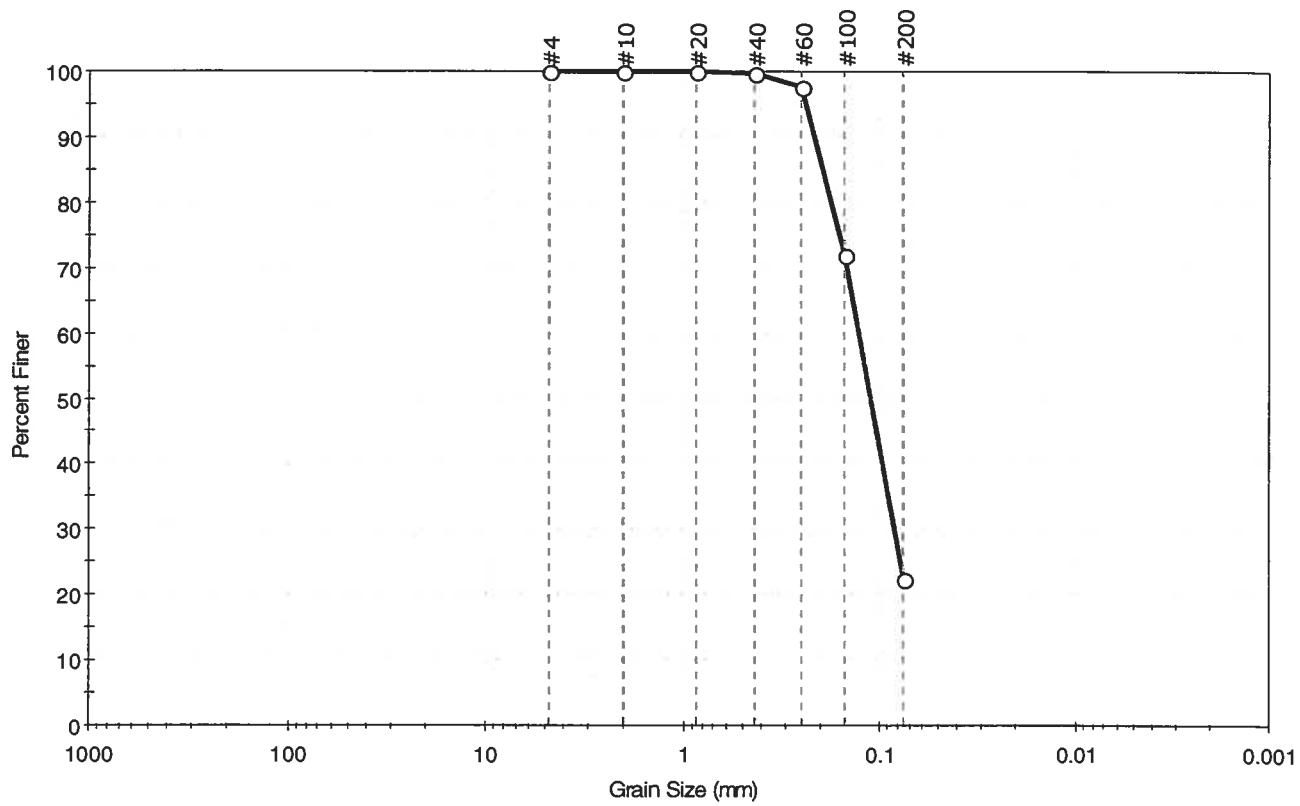
GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-102	Sample Type: jar	Tested By:	jbr
Sample ID:5D	Test Date: 01/12/10	Checked By:	jdt
Depth : 25.0-27.0 ft	Test Id: 172176		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	--	77.7	22.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	98		
#100	0.15	72		
#200	0.075	22		

Coefficients

D_{85} = 0.1947 mm D_{30} = 0.0835 mm
D_{60} = 0.1270 mm D_{15} = N/A
D_{50} = 0.1105 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

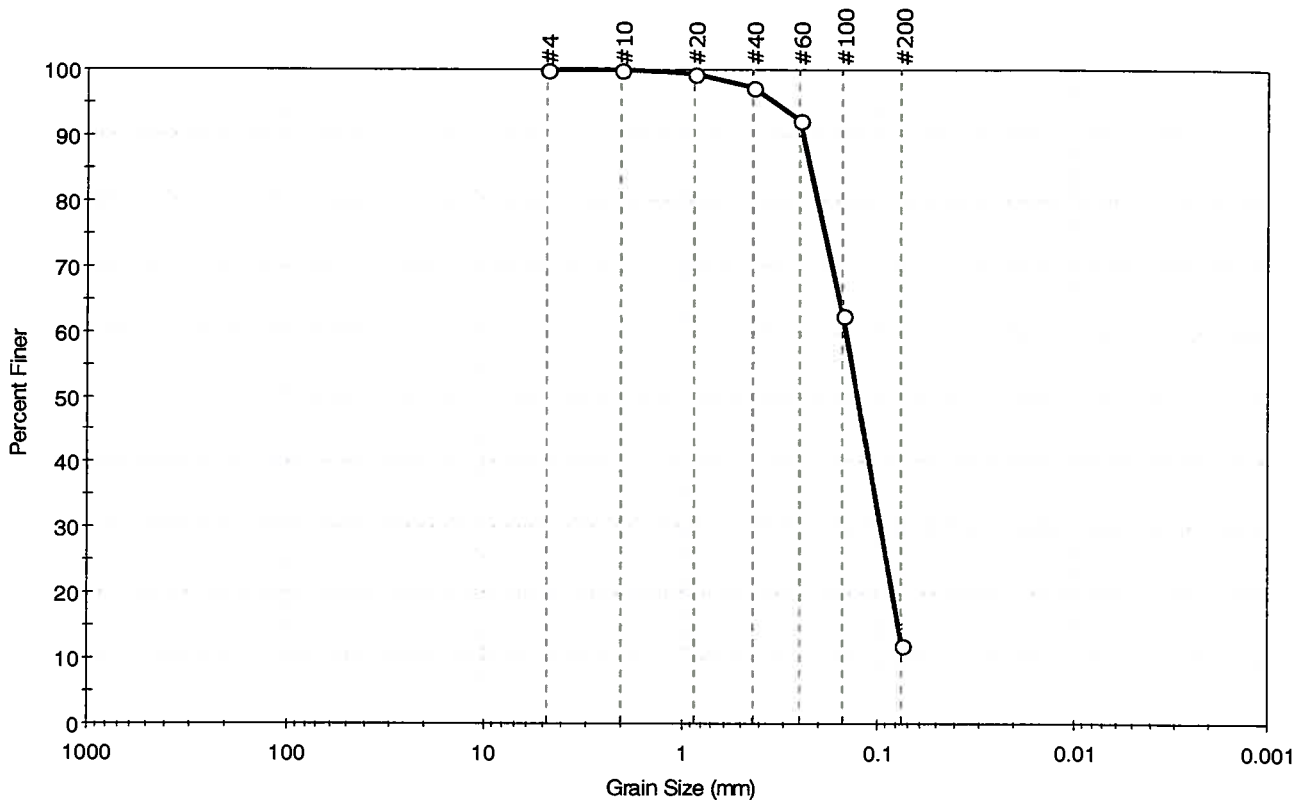
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-102	Sample Type: jar	Tested By:	jbr
Sample ID:6D	Test Date: 01/12/10	Checked By:	jdt
Depth : 30.0-32.0 ft	Test Id: 172177		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	87.8	12.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	97		
#60	0.25	92		
#100	0.15	62		
#200	0.075	12		

Coefficients

D_{85} = 0.2207 mm D_{30} = 0.0958 mm
D_{60} = 0.1449 mm D_{15} = 0.0779 mm
D_{50} = 0.1263 mm D_{10} = 0.0727 mm
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

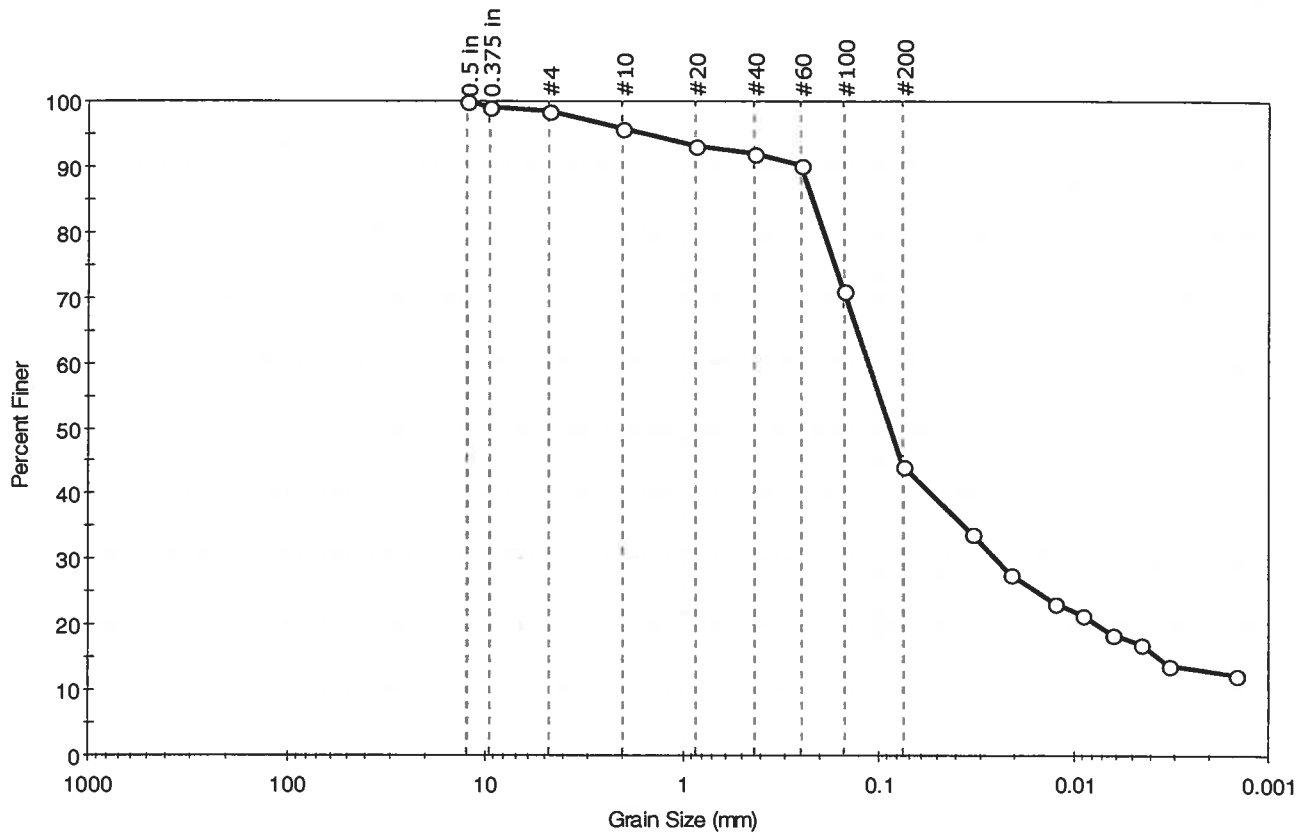
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-103	Sample Type: jar	Tested By:	jbr
Sample ID:2D,3D	Test Date: 01/12/10	Checked By:	jdt
Depth : 5.0-7.0,10.0-12.0 ft	Test Id: 172172		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.6	54.1	44.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	98		
#10	2.00	96		
#20	0.85	93		
#40	0.42	92		
#60	0.25	90		
#100	0.15	71		
#200	0.075	44		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0337	34		
---	0.0213	28		
---	0.0125	23		
---	0.0089	22		
---	0.0063	19		
---	0.0045	17		
---	0.0032	14		
---	0.0015	12		

Coefficients

D_{85} = 0.2170 mm D_{30} = 0.0250 mm
D_{60} = 0.1126 mm D_{15} = 0.0036 mm
D_{50} = 0.0869 mm D_{10} = 0.0004 mm
C_u = N/A C_c = N/A

Classification

ASTM Clayey sand (SC)

AASHTO Silty Soils (A-4 (1))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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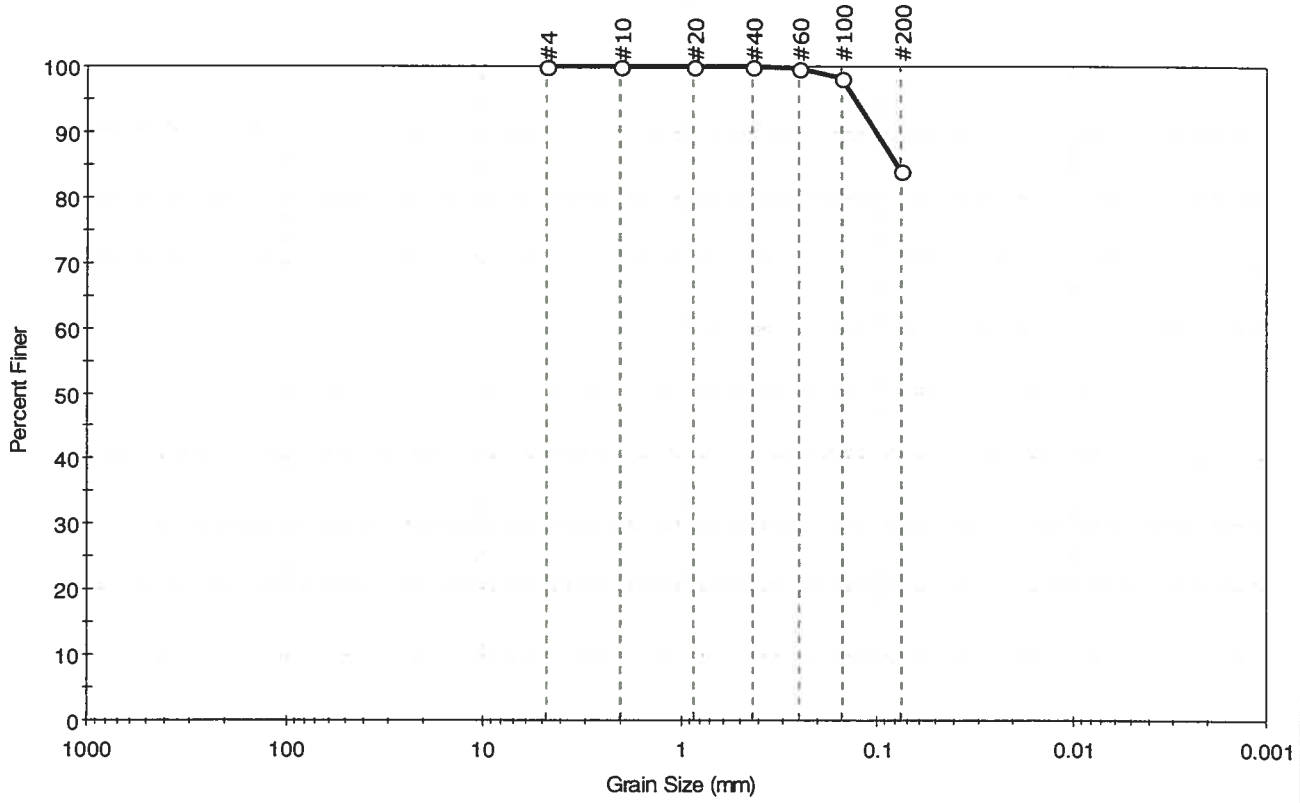
GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-103	Sample Type: jar	Tested By:	jbr
Sample ID:5D	Test Date: 01/12/10	Checked By:	jdt
Depth : 20.0-22.0 ft	Test Id: 172178		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	15.9	84.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	98		
#200	0.075	84		

Coefficients

D_{85} = 0.0784 mm D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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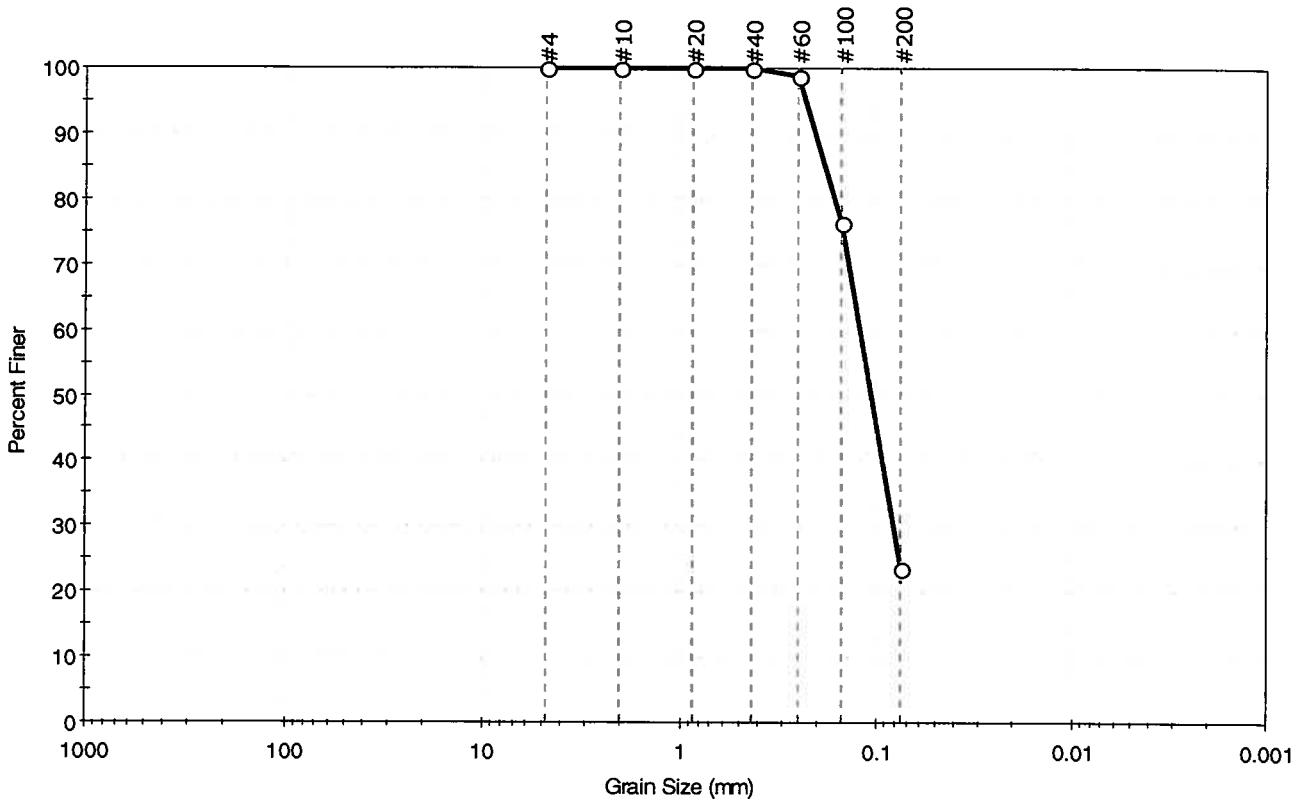
Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-103	Sample Type: jar	Tested By:	jbr
Sample ID:6D	Test Date: 01/12/10	Checked By:	jdt
Depth : 25.0-27.0 ft	Test Id: 172179		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	76.5	23.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	76		
#200	0.075	24		

Coefficients

D_{85} = 0.1831 mm D_{30} = 0.0817 mm
D_{60} = 0.1211 mm D_{15} = N/A
D_{50} = 0.1062 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

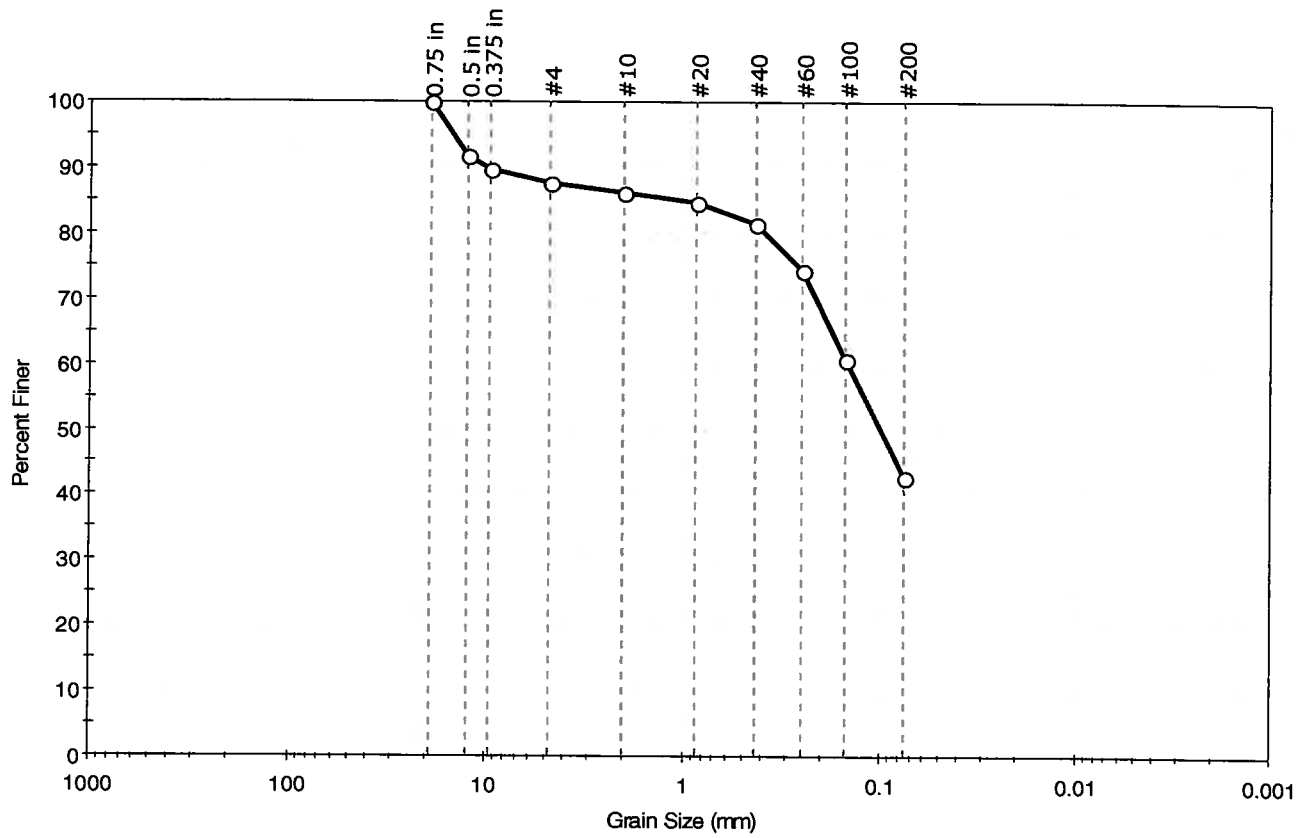
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-103	Sample Type: jar	Tested By:	jbr
Sample ID:10D	Test Date: 01/12/10	Checked By:	jdt
Depth : 45.0-47.0 ft	Test Id: 172180		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.4	44.8	42.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	92		
0.375 in	9.50	90		
#4	4.75	88		
#10	2.00	86		
#20	0.85	85		
#40	0.42	81		
#60	0.25	74		
#100	0.15	61		
#200	0.075	43		

Coefficients

D_{85} = 1.0702 mm D_{30} = N/A
D_{60} = 0.1457 mm D_{15} = N/A
D_{50} = 0.0991 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

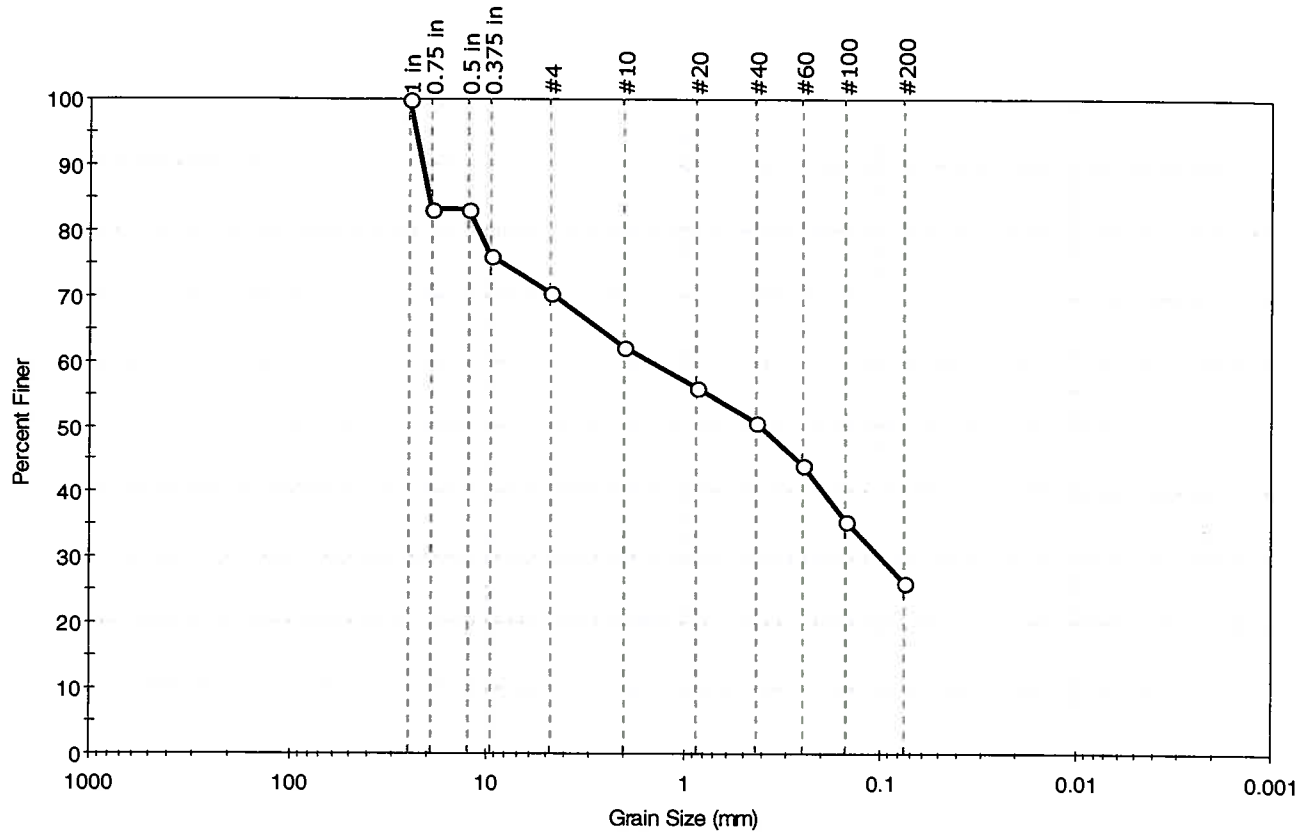
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-103	Sample Type: jar	Tested By:	jbr
Sample ID:11D	Test Date: 01/12/10	Checked By:	jdt
Depth : 50.0-50.8 ft	Test Id: 172181		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	29.4	44.4	26.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	83		
0.5 in	12.50	83		
0.375 in	9.50	76		
#4	4.75	71		
#10	2.00	62		
#20	0.85	56		
#40	0.42	51		
#60	0.25	44		
#100	0.15	36		
#200	0.075	26		

Coefficients

D_{85} = 19.6027 mm D_{30} = 0.0989 mm
D_{60} = 1.4539 mm D_{15} = N/A
D_{50} = 0.4045 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

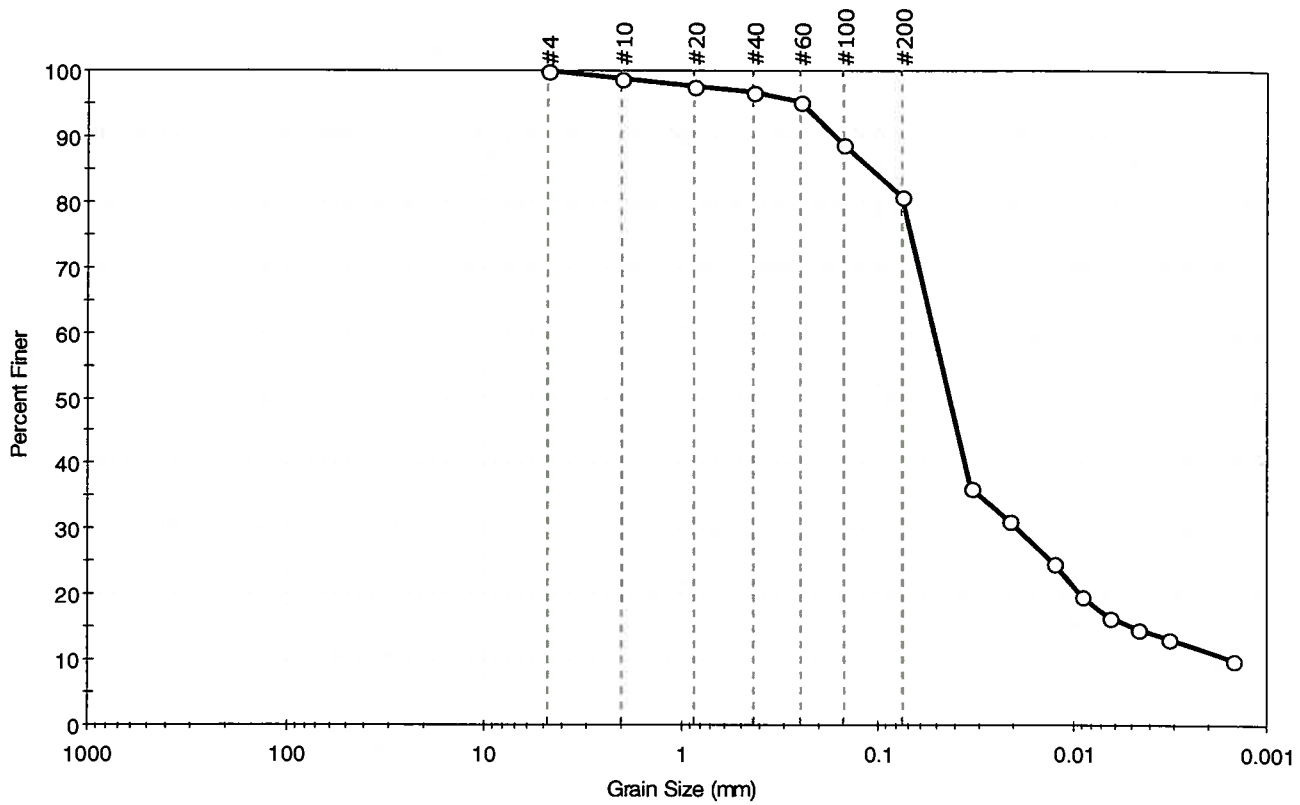
Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-104	Sample Type: jar	Tested By:	jbr
Sample ID:2D,3D	Test Date: 01/12/10	Checked By:	jdt
Depth : 5.0-7.0,10.0-12.0 ft	Test Id: 172173		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	19.3	80.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	98		
#40	0.42	97		
#60	0.25	95		
#100	0.15	89		
#200	0.075	81		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0337	36		
---	0.0212	31		
---	0.0125	25		
---	0.0090	20		
---	0.0064	16		
---	0.0046	15		
---	0.0033	13		
---	0.0015	10		

Coefficients

D_{85} = 0.1077 mm D_{30} = 0.0191 mm
D_{60} = 0.0517 mm D_{15} = 0.0047 mm
D_{50} = 0.0432 mm D_{10} = 0.0015 mm
C_u = N/A C_c = N/A

Classification

ASTM silt with sand (ML)

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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GeoTesting express
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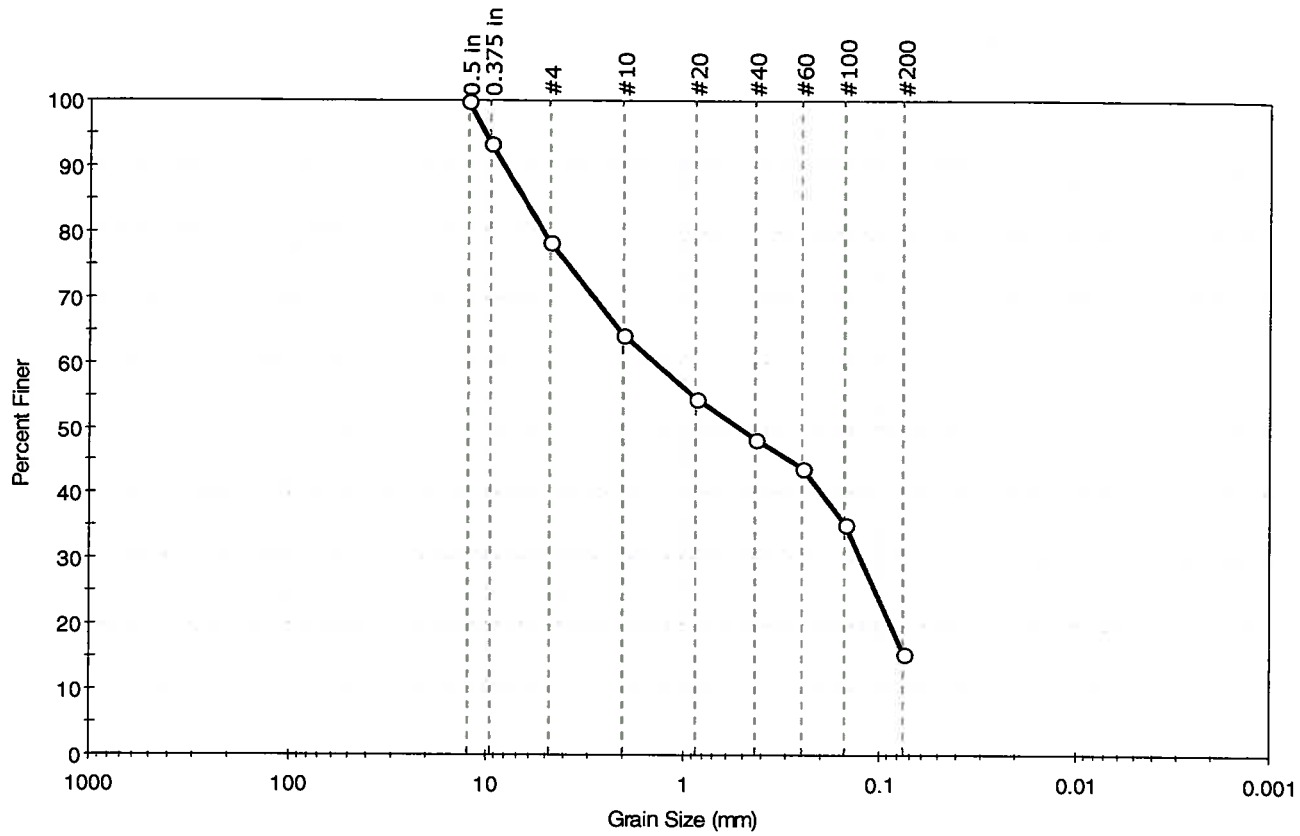
Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-104	Sample Type: jar	Tested By:	jbr
Sample ID:4D	Test Date: 01/12/10	Checked By:	jdt
Depth : 15.0-17.0 ft	Test Id: 172182		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	21.7	62.8	15.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	93		
#4	4.75	78		
#10	2.00	64		
#20	0.85	54		
#40	0.42	48		
#60	0.25	44		
#100	0.15	36		
#200	0.075	16		

Coefficients

D_{85} = 6.4480 mm D_{30} = 0.1238 mm
D_{60} = 1.3781 mm D_{15} = N/A
D_{50} = 0.5103 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

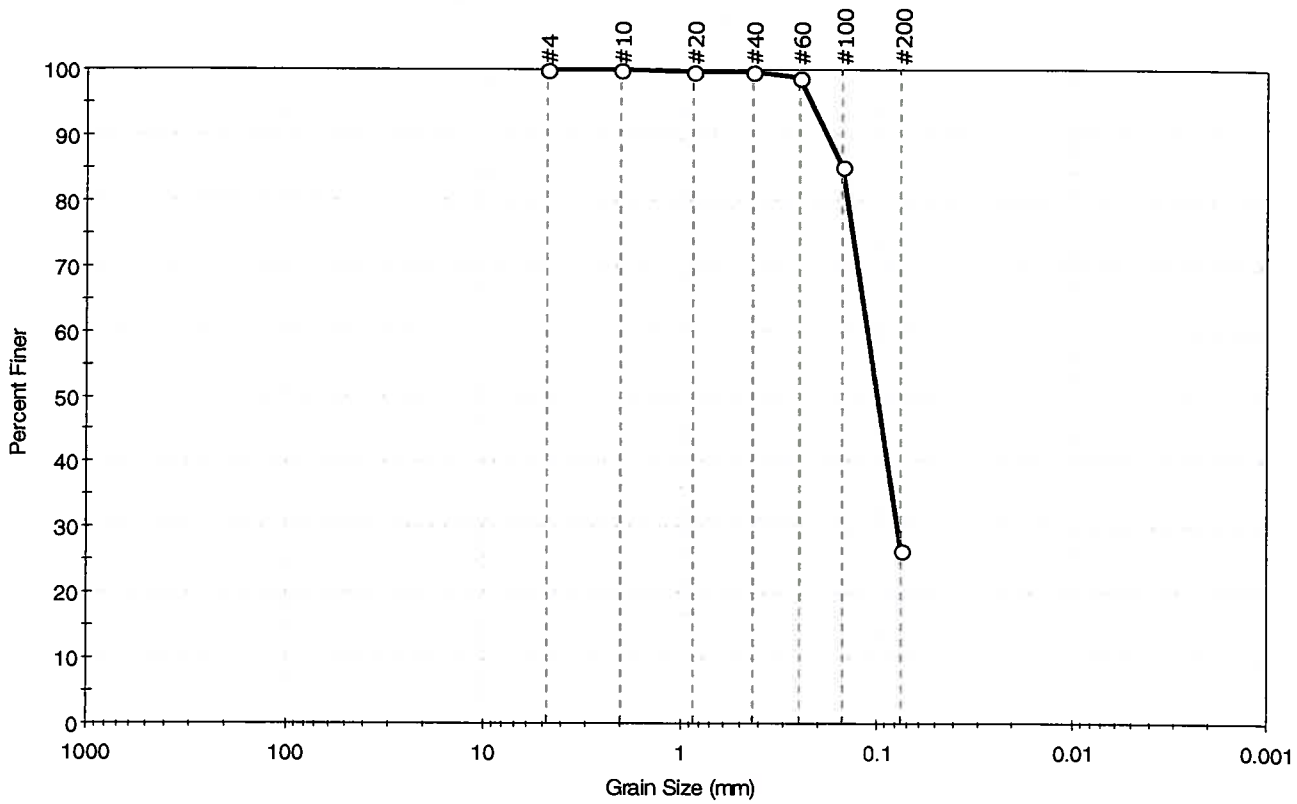
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-104	Sample Type: jar	Tested By:	jbr
Sample ID:6D	Test Date: 01/12/10	Checked By:	jdt
Depth : 25.0-27.0 ft	Test Id: 172183		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	73.6	26.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	85		
#200	0.075	26		

Coefficients

D_{85} = 0.1494 mm D_{30} = 0.0782 mm
D_{60} = 0.1113 mm D_{15} = N/A
D_{50} = 0.0990 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

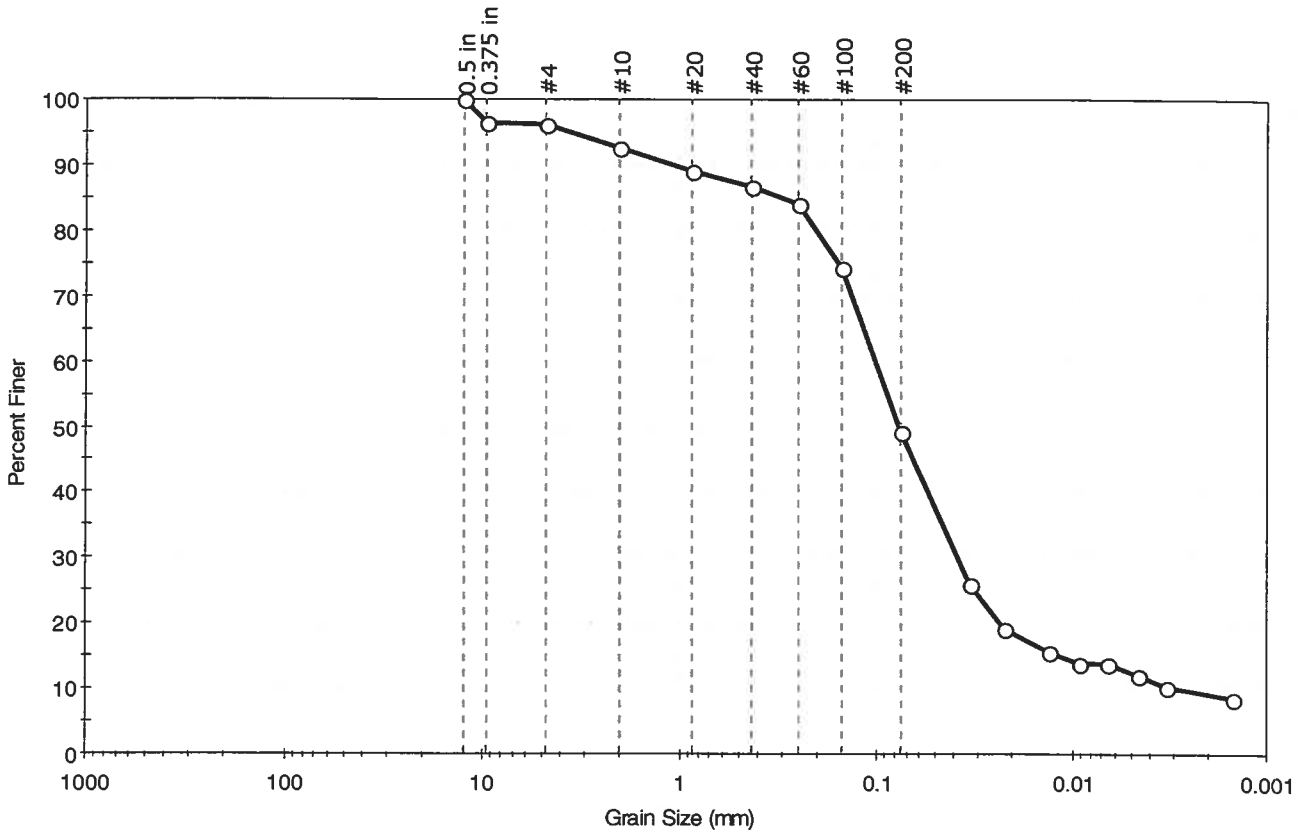
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-105	Sample Type: jar	Tested By:	jbr
Sample ID:3D	Test Date: 01/13/10	Checked By:	jdt
Depth : 8.7-10.7 ft	Test Id: 172174		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	3.8	46.9	49.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	96		
#4	4.75	96		
#10	2.00	93		
#20	0.85	89		
#40	0.42	87		
#60	0.25	84		
#100	0.15	74		
#200	0.075	49		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0338	26		
---	0.0220	19		
---	0.0130	16		
---	0.0092	14		
---	0.0065	14		
---	0.0046	12		
---	0.0033	10		
---	0.0015	9		

Coefficients

D_{85} = 0.2965 mm D_{30} = 0.0386 mm
D_{60} = 0.1008 mm D_{15} = 0.0114 mm
D_{50} = 0.0765 mm D_{10} = 0.0027 mm
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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GeoTesting express
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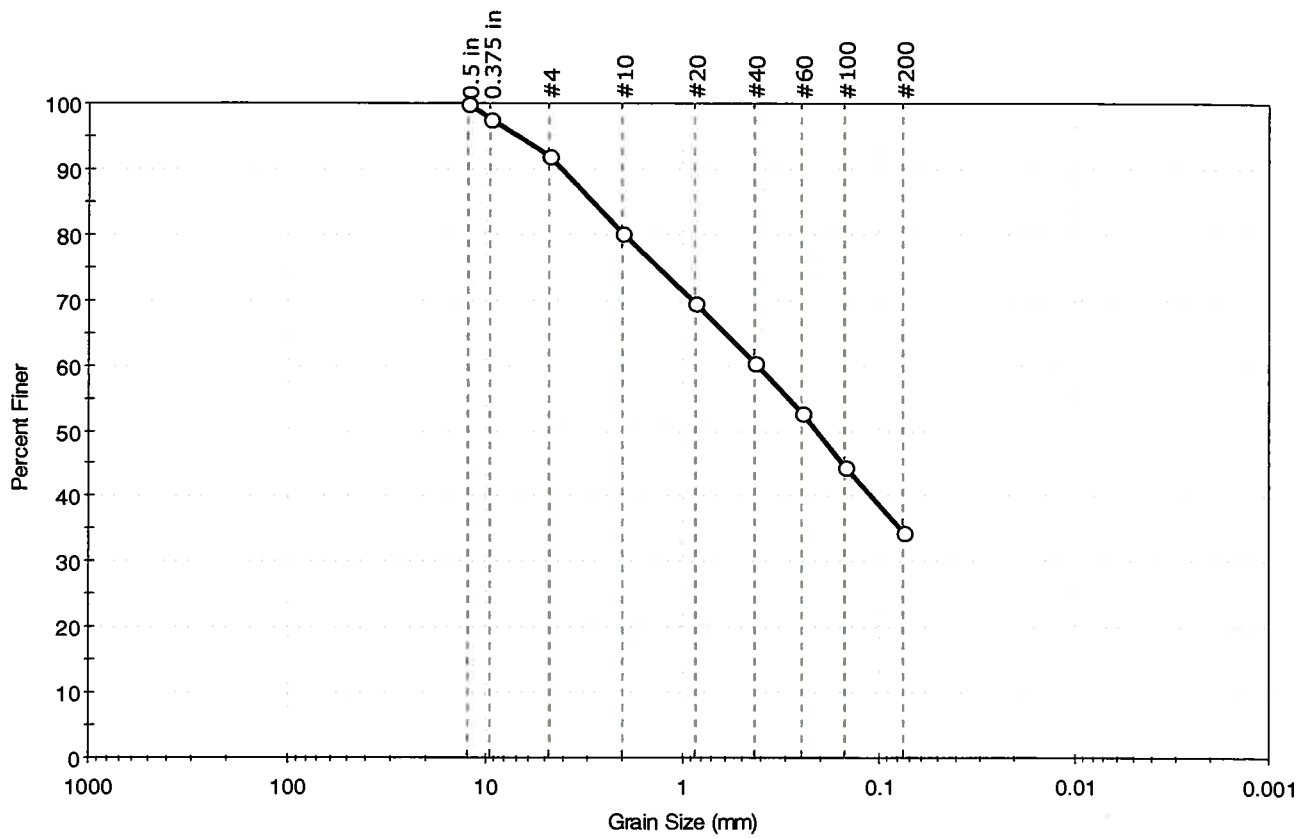
Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-105	Sample Type: jar	Tested By:	jbr
Sample ID:4D	Test Date: 01/12/10	Checked By:	jdt
Depth : 14.0-16.0 ft	Test Id: 172184		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	7.9	57.7	34.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Compiles
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	92		
#10	2.00	80		
#20	0.85	70		
#40	0.42	61		
#60	0.25	53		
#100	0.15	44		
#200	0.075	34		

Coefficients

D_{85} = 2.8198 mm D_{30} = N/A
D_{60} = 0.4085 mm D_{15} = N/A
D_{50} = 0.2101 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

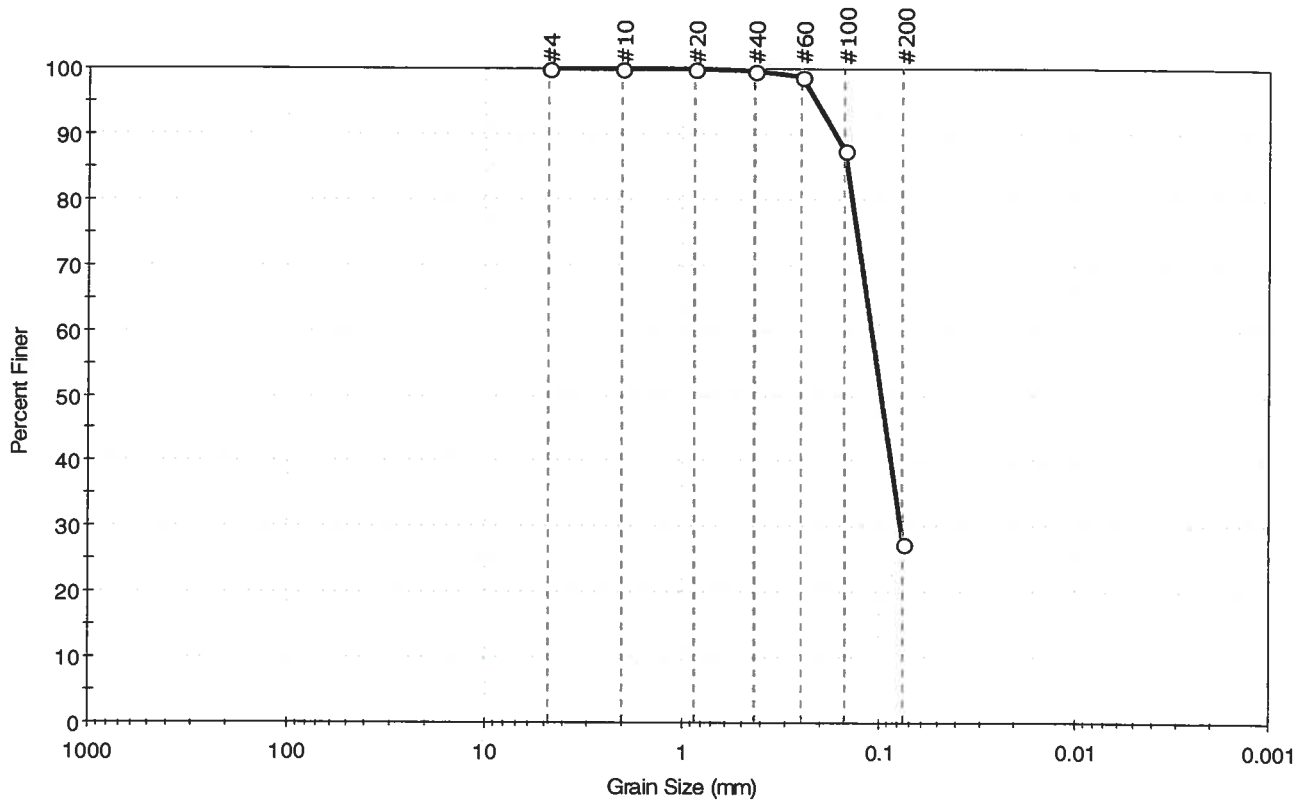
Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-105	Sample Type: jar	Tested By:	jbr
Sample ID:5D	Test Date: 01/12/10	Checked By:	jdt
Depth : 19.0-21.0 ft	Test Id: 172185		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	72.5	27.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	88		
#200	0.075	28		

Coefficients

D_{85} = 0.1454 mm D_{30} = 0.0772 mm
D_{60} = 0.1090 mm D_{15} = N/A
D_{50} = 0.0971 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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GeoTesting express
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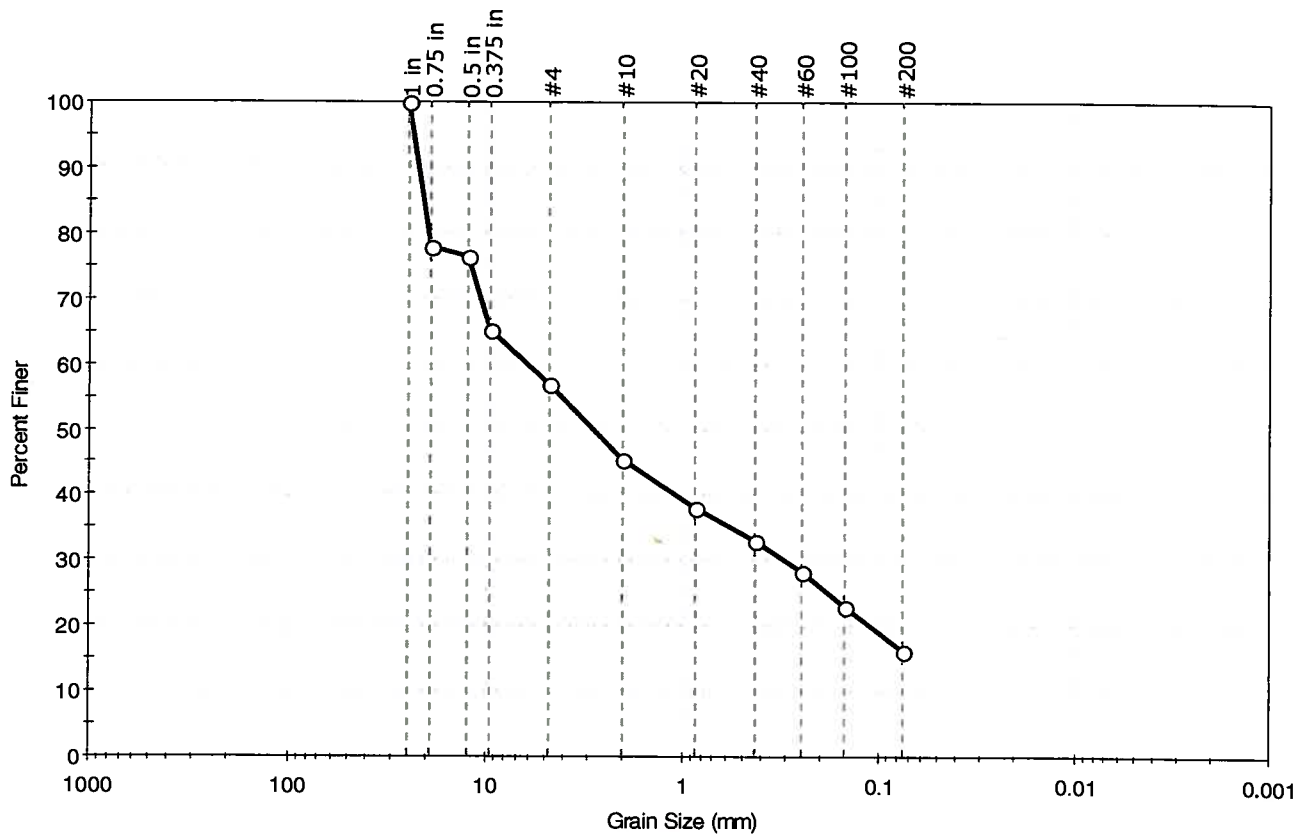
Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-105	Sample Type: jar	Tested By:	jbr
Sample ID:7D	Test Date: 01/12/10	Checked By:	jdt
Depth : 29.0-31.0 ft	Test Id: 172186		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	43.1	40.8	16.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	78		
0.5 in	12.50	76		
0.375 in	9.50	65		
#4	4.75	57		
#10	2.00	45		
#20	0.85	38		
#40	0.42	33		
#60	0.25	28		
#100	0.15	23		
#200	0.075	16		

Coefficients

D_{85} = 20.7429 mm D_{30} = 0.3047 mm
D_{60} = 6.1321 mm D_{15} = N/A
D_{50} = 2.8160 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

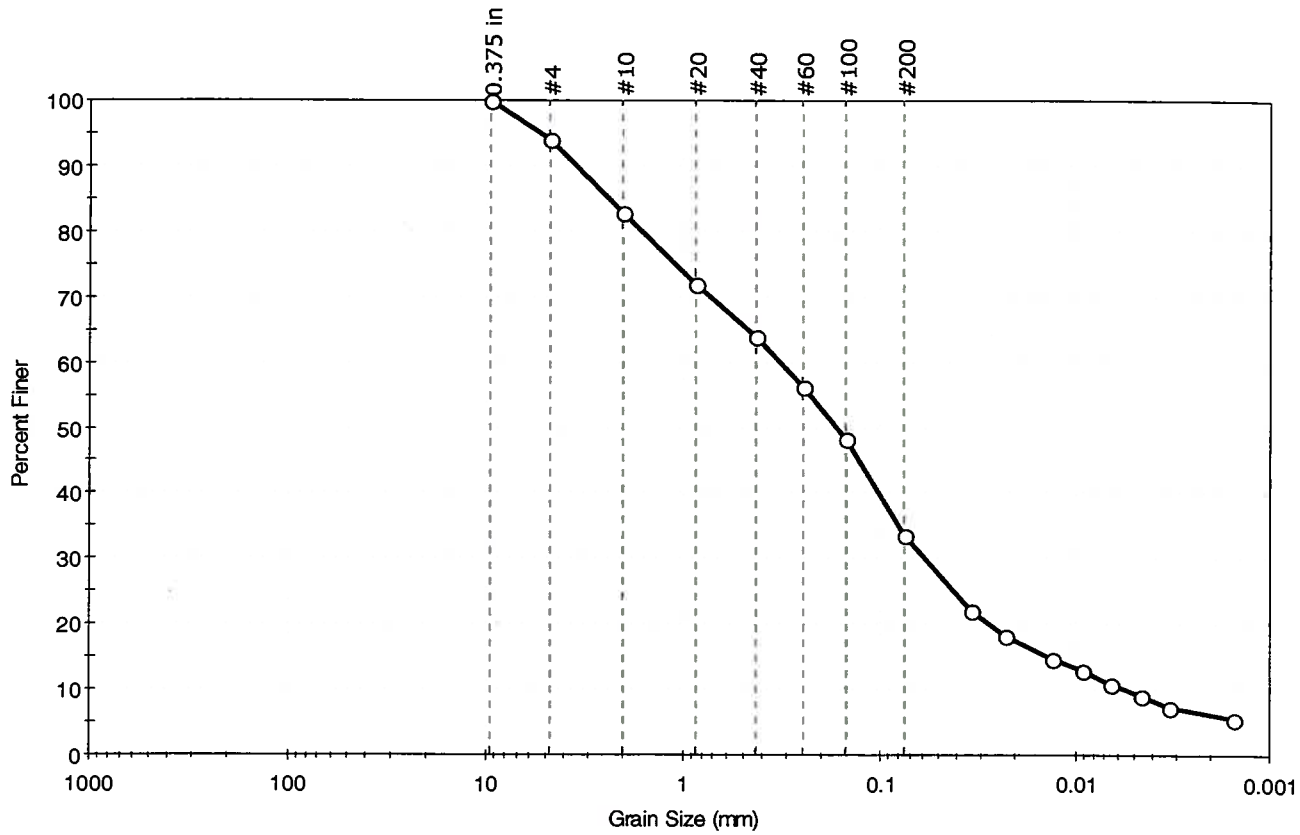
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL-106	Sample Type: jar	Tested By:	jbr
Sample ID:5D	Test Date: 01/13/10	Checked By:	jdt
Depth : 22.0-24.0 ft	Test Id: 172175		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.8	60.5	33.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	94		
#10	2.00	83		
#20	0.85	72		
#40	0.42	64		
#60	0.25	56		
#100	0.15	48		
#200	0.075	34		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0342	22		
---	0.0226	18		
---	0.0131	15		
---	0.0093	13		
---	0.0065	11		
---	0.0046	9		
---	0.0033	7		
---	0.0015	6		

Coefficients

D_{85} = 2.3472 mm D_{30} = 0.0585 mm
D_{60} = 0.3234 mm D_{15} = 0.0137 mm
D_{50} = 0.1659 mm D_{10} = 0.0054 mm
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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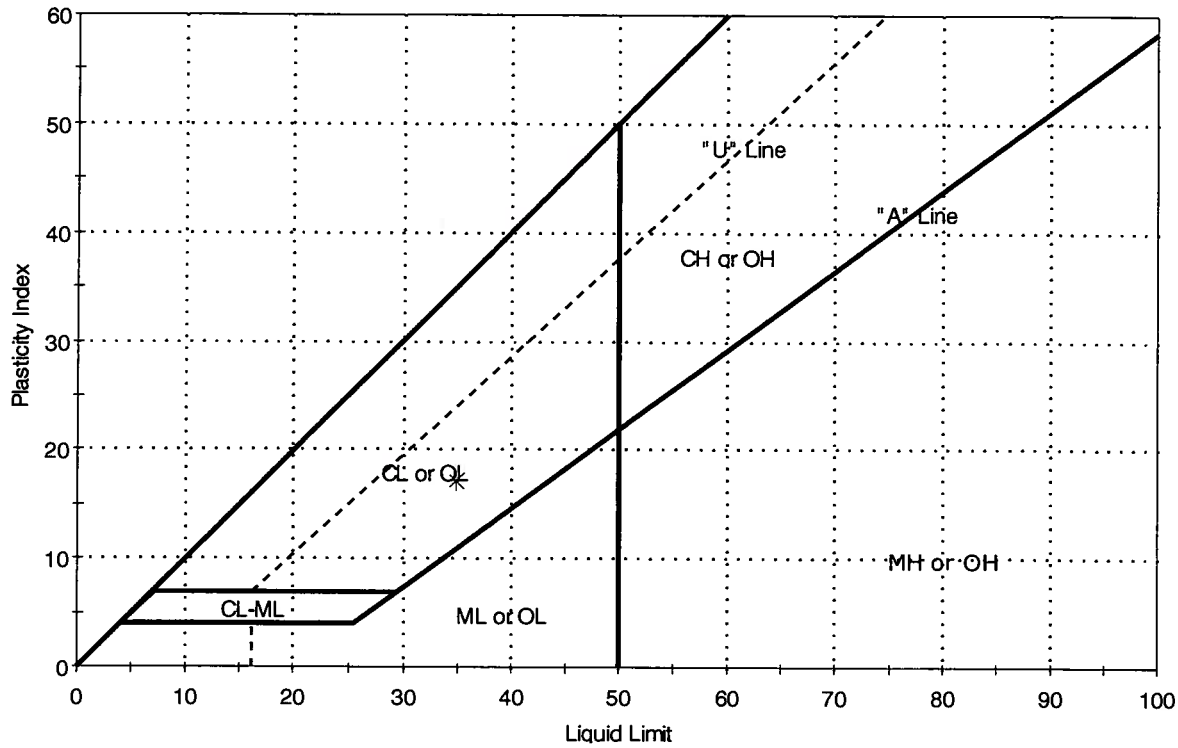
Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL -101	Sample Type: jar	Tested By:	cam
Sample ID:6D	Test Date: 01/13/10	Checked By:	jdt
Depth : 25.0- 27.0 ft	Test Id: 172168		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	6D	BB-YSL -101	25.0- 27.0 ft	32	35	18	17	1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilentancy: SLOW
Toughness: LOW

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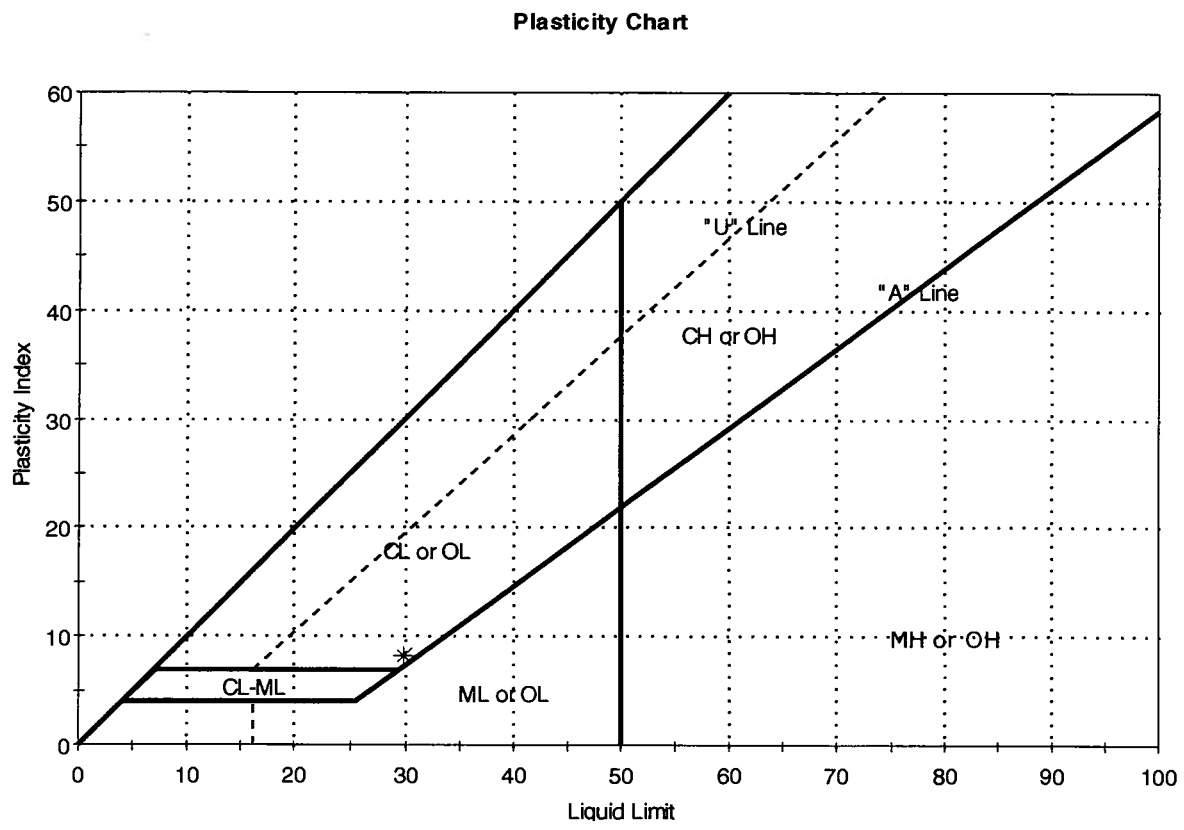
GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL -103	Sample Type: jar	Tested By:	cam
Sample ID:2D,3D	Test Date: 01/13/10	Checked By:	jdt
Depth : 5.0-7.0, 10.0-12.0 ft	Test Id: 172169		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	2D,3D	BB-YSL -103	5.0-7.0, 10.0-12.0 ft	47	30	22	8	3	Clayey sand (SC)

Sample Prepared using the WET method
8% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilentancy: SLOW
Toughness: LOW

GeoTesting express
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Client:	Haley & Aldrich, Inc.		
Project:	Proposed Sewall's Bridge Rehabilitation		
Location:	York, ME	Project No:	GTX-9592
Boring ID: BB-YSL -104	Sample Type: jar	Tested By:	cam
Sample ID:2D,3D	Test Date: 01/13/10	Checked By:	jdt
Depth : 5.0-7.0, 10.0-12.0 ft	Test Id: 172170		
Test Comment: ---			
Sample Description:			
Sample Comment: ---			

Atterberg Limits - ASTM D 4318-05

Sample Determined to be non-plastic

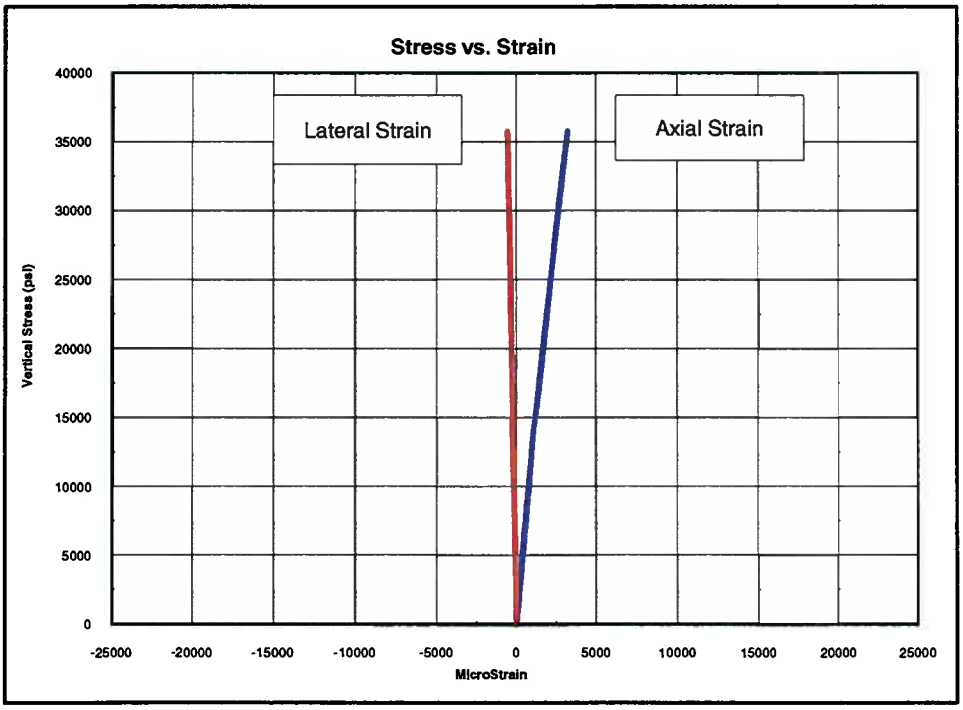
Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	2D,3D	BB-YSL -104	5.0-7.0, 10.0-12.0 ft	27	n/a	n/a	n/a	n/a	silt with sand (ML)

3% Retained on #40 Sieve
Dry Strength: LOW
Dilentancy: RAPID
Toughness: n/a
The sample was determined to be Non-Plastic

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.
Project Name:	Proposed Sewall's Bridge Rehabilitation
Project Location:	York, ME
GTX #:	9592
Test Date:	01/19/10
Tested By:	daa
Checked By:	jdt
Boring ID:	BB-YSL-106
Sample ID:	R1
Depth, ft:	26.64-27.01
Sample Type:	rock core
Sample Description:	See photographs

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 37,432 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-5000	13,000,000	0.20
5000-1000	13,000,000	0.20
10000-15000	12,000,000	0.18
15000-20000	9,600,000	0.14
20000-25000	11,000,000	0.17
25000-30000	10,000,000	0.17

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

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Client:	Haley & Aldrich, Inc.	Test Date:	1/15/2010
Project Name:	Proposed Sewall's Bridge Rehabilitation	Tested By:	daa
Project Location:	York, ME	Checked By:	mpd
GTX #:	9592		
Boring ID:	BB-YSL-106		
Sample ID:	R1		
Depth:	26.64-27.01 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.41	4.42	4.42
Specimen Diameter, in:	1.97	1.98	1.98
Specimen Mass, g:	605.03		
Bulk Density, lb/ft³	170		
Length to Diameter Ratio:	2.2		

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? ND

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00020	0.00020	0.00020	0.00030
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00020	-0.00020	-0.00020

Difference between max and min readings, in:

0° = 0.00050 90° = 0.00020

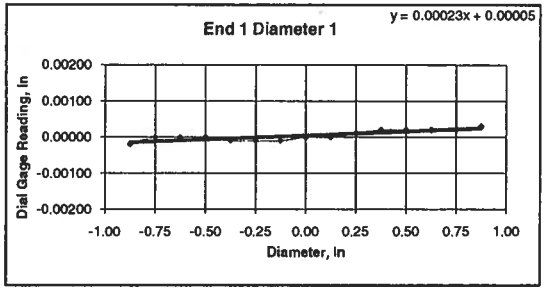
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020

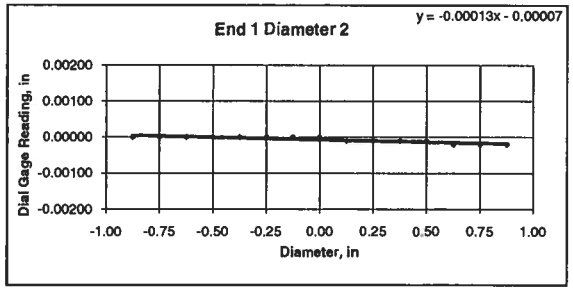
Difference between max and min readings, in:

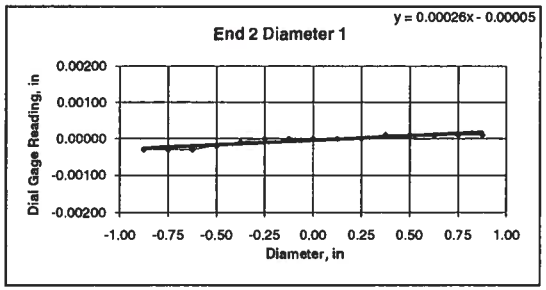
0° = 0.0004 90° = 0.0003

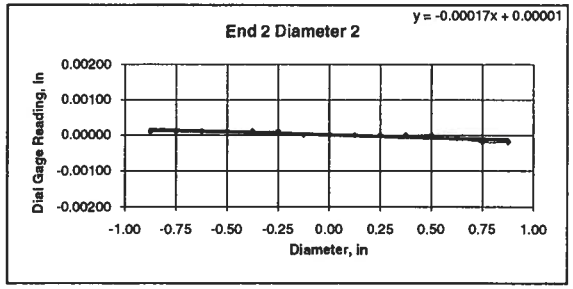
Maximum difference must be < 0.0020 in. Difference = ± 0.00025

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00023
Angle of Best Fit Line:	0.01318

End 2:

Slope of Best Fit Line	0.00026
Angle of Best Fit Line:	0.01490

Maximum Angular Difference: 0.00172

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	-0.00013
Angle of Best Fit Line:	-0.00745

End 2:

Slope of Best Fit Line	-0.00017
Angle of Best Fit Line:	-0.00974

Maximum Angular Difference: 0.00229

Parallelism Tolerance Met? YES

Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	1.975	0.00025	0.015	YES
Diameter 2, in (rotated 90°)	0.00020	1.975	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00040	1.975	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00030	1.975	0.00015	0.009	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

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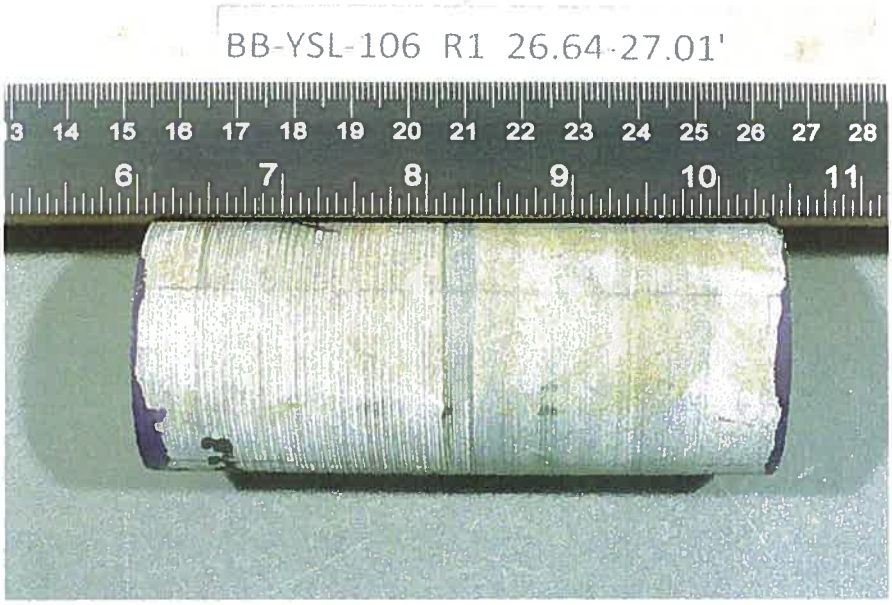
Client:	Haley & Aldrich, Inc.
Project Name:	Proposed Sewall's Bridge Rehabilitation
Project Location:	York, ME
GTX #:	9592
Test Date:	01/19/10
Tested By:	daa
Checked By:	jdt
Boring ID:	BB-YSL-106
Sample ID:	R1
Depth, ft:	26.64-27.01



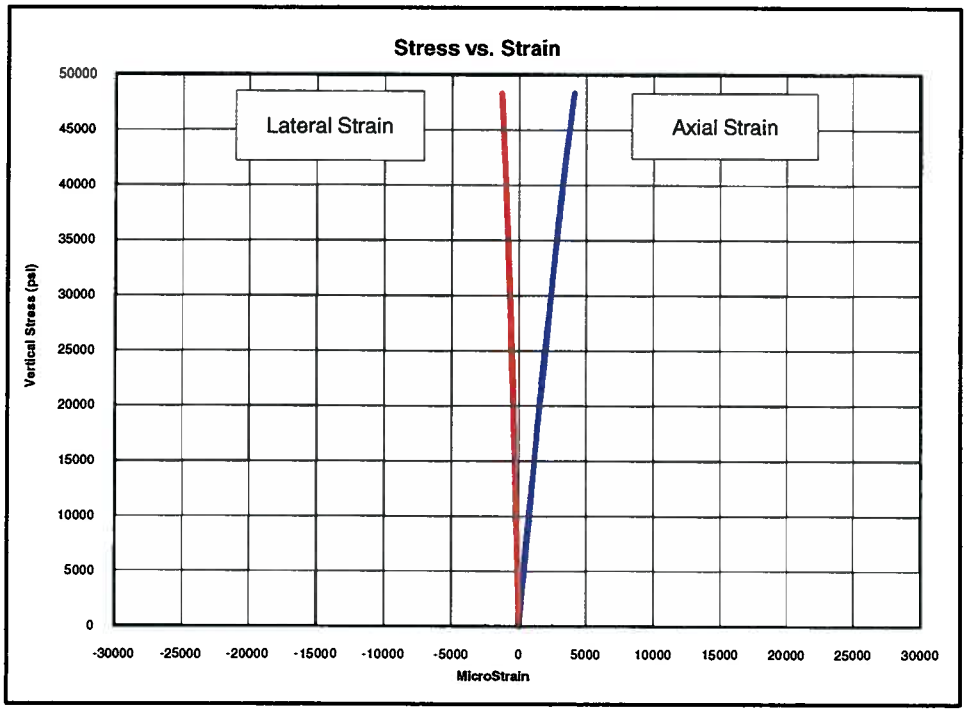
After cutting and grinding

After break

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.
Project Name:	Proposed Sewall's Bridge Rehabilitation
Project Location:	York, ME
GTX #:	9592
Test Date:	01/19/10
Tested By:	daa
Checked By:	jdt
Boring ID:	BB-YSL-101
Sample ID:	R1
Depth, ft:	35.88-36.24
Sample Type:	rock core
Sample Description:	See photographs

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 48,338 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-5000	14,000,000	0.27
5000-1000	13,000,000	0.27
10000-20000	13,000,000	0.28
20000-30000	12,000,000	0.30
30000-40000	11,000,000	0.35

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

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Client:	Haley Aldrich, Inc.	Test Date:	1/15/2010
Project Name:	Proposed Sewall's Bridge Rehabilitation	Tested By:	daa
Project Location:	York, ME	Checked By:	mpd
GTX #:	9592		
Boring ID:	BB-YSL-101		
Sample ID:	R1		
Depth:	35.88-36.24 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.30	4.31	4.31
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	636.78		
Bulk Density, lb/ft³	183		
Length to Diameter Ratio:	2.2		

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

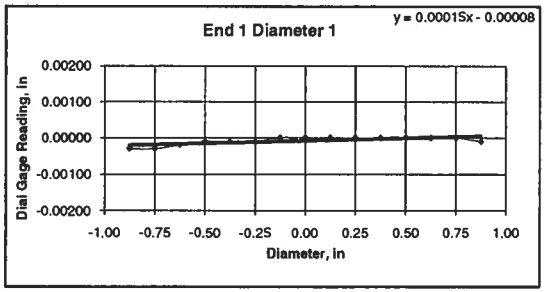
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00020	0.00020	0.00030	0.00030	0.00030	0.00030

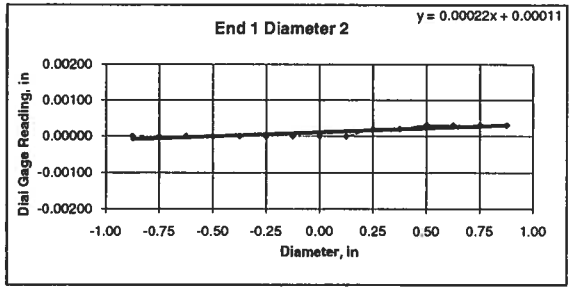
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00030

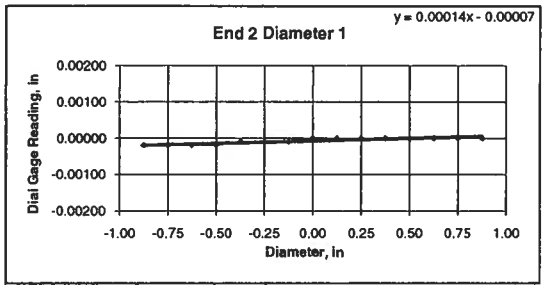
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00030	-0.00030	-0.00030	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010

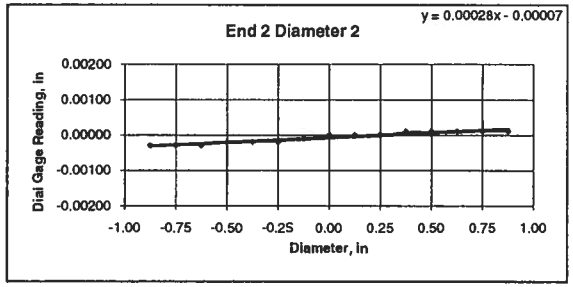
Difference between max and min readings, in:
0° = 0.0002 90° = 0.0004

Maximum difference must be < 0.0020 in. Difference = ± 0.00020
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00015
Angle of Best Fit Line: 0.00859

End 2:
Slope of Best Fit Line 0.00014
Angle of Best Fit Line: 0.00802

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00022
Angle of Best Fit Line: 0.01261

End 2:
Slope of Best Fit Line 0.00028
Angle of Best Fit Line: 0.01604

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	1.980	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00030	1.980	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

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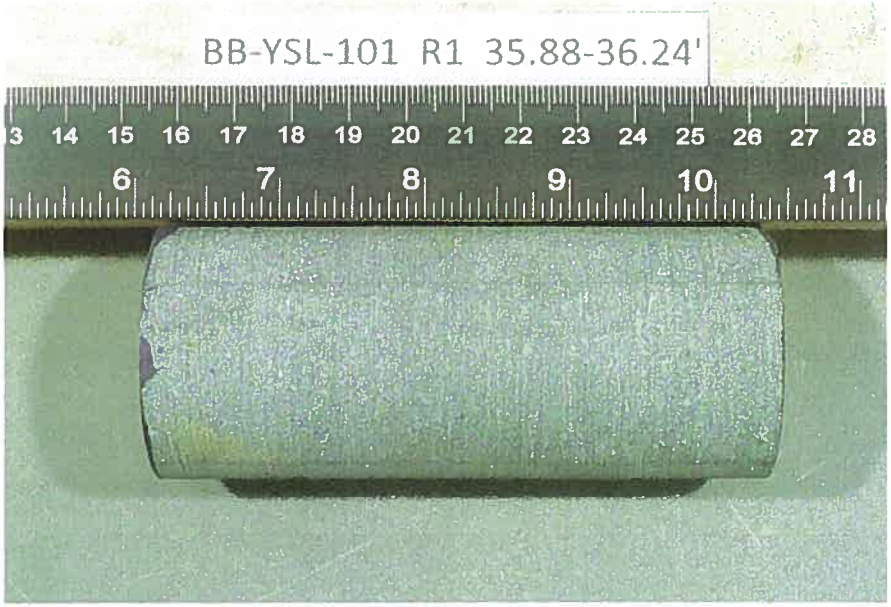
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.
Project Name:	Proposed Sewall's Bridge Rehabilitation
Project Location:	York, ME
GTX #:	9592
Test Date:	01/19/10
Tested By:	daa
Checked By:	jdt
Boring ID:	BB-YSL-101
Sample ID:	R1
Depth, ft:	35.88-36.24



After cutting and grinding

After break

GeoTesting express
A subsidiary of Geocomp Corporation

TO: Joe Tomei / Rob Turcotte

SOIL CHAIN OF CUSTODY & TEST REQUEST

CLIENT	
Company: Haley & Aldrich, Inc.	
Address: 75 Washington Ave., Ste 203	
City, State, Zip: Portland, ME 04101	
Contact: Andrew Blaisdell	E-mail:ablaisdell@haleyaldrich.com
Phone:207-482-4619	Fax:207-482-4669

PROJECT	
Project Name: Proposed Sewall's Bridge Rehabilitation	
Address: Organug Road	Client Project #:36654-000
City, State, Zip: York, Maine	
On-site Contact:	E-mail:
Phone:	Fax:

GeoTesting Express, Inc.
1145 Massachusetts Avenue
Boxborough, MA 01719
800 434 1062 Toll Free
978 635 0265 Fax

2662 Holcomb Bridge Road, Suite 310
Alpharetta, GA 30022
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

INVOICE (complete if different from client)	
Company:	
Address:	
City, State, Zip:	
Contact:	E-mail:
Phone:	Fax:

GENERAL	
Purchase Order #:	GTX Sales Order #:
Shipped By: Maura Foley	Date Shipped: 1/11/10
Mode of Shipment: Hand-deliver	Requested Turnaround: 1 week soil, 2 weeks lab
Send Results To:	☒ CLIENT OFFICE ☐ PROJECT OFFICE
Send Results Via:	☒ E-MAIL ☐ FAX ☐ VERBAL ☒ HARD COPY

SOIL *please include boring # and depth if known	Sample ID	Atterberg Limits (ASTM D 4318)	California Bearing Ratio (ASTM D 1883) *specify conditions below	USCS – Classification (ASTM D 2487)	Density (ASTM D 2937)	Direct Shear (ASTM D 3080) *specify conditions below	Direct Simple Shear (ASTM D 6528) *specify conditions below	Electrical Resistivity (ASTM G 57)	Grain Size (ASTM D 422) Sieve Only / Sieve & Hydrometer please circle one	Incremental Consolidation (ASTM D 2435)	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	Permeability/ Hydraulic Conductivity (Fixed Wall - ASTM D 2434) (Flexible Wall – ASTM D 5084) please circle one	Ph (ASTM D 4972)	Proctor Compaction (Standard – ASTM D 698) (Modified – ASTM D 1557) please circle one	Specific Gravity (ASTM D 854)	Triaxial Shear (UU – ASTM D 2850) (CU – ASTM D 4767) (CD – US COE EM1110) *specify conditions below please circle one	Unconfined Compression (ASTM D 2166)	Other: x-ray tube, tube log (total unit weights, lab vanes)	Other: rock unit weight, elastic modulus, unconfined compression test 350
1	BB-YSL-101, 5D, 20-22'										x									
2	BB-YSL-101, 6D, 25-27'	x									x									
3	BB-YSL-101, 7D, 30-32'										x									
4	BB-YSL-101, R1, 35.7-36.5'																			x
5	BB-YSL-102, 1D, 0-2'								x (s+h)		x									
6	BB-YSL-102, 2D, 5-7'										x									

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: [signature] PRINT NAME: Andy Blaisdell DATE: 1/11/10

Relinquished By: [signature]	DATE: 1/11/10	Received By: [signature]	DATE: 1/11/10
	TIME:		TIME:
Relinquished By: [signature]	DATE: 1/11/10	Received By: Robert Turcotte	DATE: 1/11/10
	TIME:		TIME: 12:30

GeoTesting express
a subsidiary of Geosamples Corporation

SOIL CHAIN OF CUSTODY & TEST REQUEST

CLIENT	
Company: Haley & Aldrich, Inc.	
Address: 75 Washington Ave., Ste 203	
City, State, Zip: Portland, ME 04101	
Contact: Andrew Blaisdell	E-mail:ablaisdell@haleyaldrich.com
Phone:207-482-4619	Fax:207-482-4669

PROJECT	
Project Name: Proposed Sewall's Bridge Rehabilitation	
Address: Organug Road	Address: Organug Road
City, State, Zip: York, Maine	
On-site Contact:	E-mail:
Phone:	Fax:

GeoTesting Express, Inc
1145 Massachusetts Avenue
Boxborough, MA 01719
800 434 1062 Toll Free
978 635 0266 Fax

2662 Holcomb Bridge Road, Suite 310
Alpharetta, GA 30022
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

INVOICE (complete if different from client)	
Company:	
Address:	
City, State, Zip:	
Contact:	E-mail:
Phone:	Fax:

GENERAL	
Purchase Order #:	GTX Sales Order #:
Shipped By: Maura Foley	Date Shipped: 1/11/10
Mode of Shipment: Hand-deliver	Requested Turnaround: 1 week soil, 2 weeks lab
Send Results To:	☒ CLIENT OFFICE ☐ PROJECT OFFICE
Send Results Via:	☒ E-MAIL ☐ FAX ☐ VERBAL ☒ HARD COPY

SOIL *please include boring # and depth if known	Sample ID	Atterberg Limits (ASTM D 4318)	California Bearing Ratio (ASTM D 1883) *specify conditions below	USCS – Classification (ASTM D 2487)	Density (ASTM D 2937)	Direct Shear (ASTM D 3080) *specify conditions below	Direct Simple Shear (ASTM D 6528) *specify conditions below	Electrical Resistivity (ASTM G 57)	Grain Size (ASTM D 422) Sieve Only / Sieve & Hydrometer please circle one	Incremental Consolidation (ASTM D 2435)	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	Permeability/ Hydraulic Conductivity (Fixed Wall - ASTM D 2434) (Flexible Wall – ASTM D 5084) please circle one	Ph (ASTM D 4972)	Proctor Compaction (Standard – ASTM D 698) (Modified – ASTM D 1557) please circle one	Specific Gravity (ASTM D 854)	Triaxial Shear (UU – ASTM D 2850) (CU – ASTM D 4767) (CD – US COE EM1110) *specify conditions below please circle one	Unconfined Compression (ASTM D 2166)	Other: x-ray tube, tube log (total unit weights, lab vanes)	Other: rock unit weight, elastic modulus, unconfined compression test
7	BB-YSL-102, 5D, 25-27'								x (s)											
8	BB-YSL-102, 6D, 30-32'								x (s)											
9	BB-YSL-103, 2D, 5-7'	x							x (s+h)		x									
10	BB-YSL-103, 3D, 10-12'																			
11	BB-YSL-103, 5D, 20-22'								x (s)											
12	BB-YSL-103, 6D, 25-27'								x (s)											
13	BB-YSL-103, 10D, 45-47'								x (s)											

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):
-Composite samples 2D and 3D from BB-YSL-103 for tests.

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: [signature] PRINT NAME: Andy Blaisdell DATE: 1/11/10

Relinquished By: [signature]	DATE: 1/11/10	Received By: [signature]	DATE: 1/11/10
	TIME:		TIME:
Relinquished By: [signature]	DATE: 1/11/10	Received By: [signature]	DATE: 1/11/10
	TIME:		TIME: 12:30

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A subsidiary of Geotechnical Corporation

SOIL CHAIN OF CUSTODY & TEST REQUEST

CLIENT	
Company: Haley & Aldrich, Inc.	
Address: 75 Washington Ave., Ste 203	
City, State, Zip: Portland, ME 04101	
Contact: Andrew Blaisdell	E-mail:ablaisdell@haleyaldrich.com
Phone:207-482-4619	Fax:207-482-4669

PROJECT	
Project Name: Proposed Sewall's Bridge Rehabilitation	
Address: Organug Road	Address: Organug Road
City, State, Zip: York, Maine	
On-site Contact:	E-mail:
Phone:	Fax:

GeoTesting Express, Inc.
1145 Massachusetts Avenue
Boxborough, MA 01719
800 434 1062 Toll Free
978 635 0266 Fax

2662 Holcomb Bridge Road, Suite 310
Alpharetta, GA 30022
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

INVOICE (complete if different from client)	
Company:	
Address:	
City, State, Zip:	
Contact:	E-mail:
Phone:	Fax:

GENERAL	
Purchase Order #:	GTX Sales Order #:
Shipped By: Maura Foley	Date Shipped: 1/11/10
Mode of Shipment: Hand-deliver	Requested Turnaround: 1 week soil, 2 weeks lab
Send Results To: ☒ CLIENT OFFICE ☐ PROJECT OFFICE	
Send Results Via: ☒ E-MAIL ☐ FAX ☐ VERBAL ☒ HARD COPY	

SOIL *please include boring # and depth if known	Sample ID	Atterberg Limits (ASTM D 4318)	California Bearing Ratio (ASTM D 1883) *specify conditions below	USCS – Classification (ASTM D 2487)	Density (ASTM D 2937)	Direct Shear (ASTM D 3080) *specify conditions below	Direct Simple Shear (ASTM D 6528) *specify conditions below	Electrical Resistivity (ASTM G 57)	Grain Size (ASTM D 422) Sieve Only / Sieve & Hydrometer please circle one	Incremental Consolidation (ASTM D 2435)	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	Permeability/ Hydraulic Conductivity (Fixed Wall - ASTM D 2434) (Flexible Wall – ASTM D 5084) please circle one	Ph (ASTM D 4972)	Proctor Compaction (Standard – ASTM D 698) (Modified – ASTM D 1557) please circle one	Specific Gravity (ASTM D 854)	Triaxial Shear (UU – ASTM D 2850) (CU – ASTM D 4767) (CD – US COE EM1110) *specify conditions below please circle one	Unconfined Compression (ASTM D 2166)	Other: x-ray tube, tube log (total unit weights, lab vanes)	Other: rock unit weight, elastic modulus, unconfined compression test
14	BB-YSL-103, 11D, 50-50.8'								x (s)											
15	BB-YSL-104, 2D, 5-7'	x							x (s+h)		x									
16	BB-YSL-104, 3D, 10-12'																			
17	BB-YSL-104, 4D, 15-17'								x (s)											
18	BB-YSL-104, 6D, 25-27'								x (s)											
19	BB-YSL-105, 3D, 8.7-10.7								x (s+h)		x									
20	BB-YSL-105, 4D, 14-16'								x (s)		x									

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):
-Composite samples 2D and 3D from BB-YSL-104 for tests.

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: [signature] PRINT NAME: Andy Blaisdell DATE: 1/11/10

Relinquished By: [signature]	DATE: 1/11/10	Received By: [signature]	DATE: 1/11/10
	TIME:		TIME:
Relinquished By: [signature]	DATE: 1/11/10	Received By: Robert Nutter	DATE: 1/11/10
	TIME:		TIME: 12:30

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SOIL CHAIN OF CUSTODY & TEST REQUEST

CLIENT	
Company: Haley & Aldrich, Inc.	
Address: 75 Washington Ave., Ste 203	
City, State, Zip: Portland, ME 04101	
Contact: Andrew Blaisdell	E-mail:ablaisdell@haleyaldrich.com
Phone:207-482-4619	Fax:207-482-4669

PROJECT	
Project Name: Proposed Sewall's Bridge Rehabilitation	
Address: Organug Road	Address: Organug Road
City, State, Zip: York, Maine	
On-site Contact:	E-mail:
Phone:	Fax:

GeoTesting Express, Inc.
1145 Massachusetts Avenue
Boxborough, MA 01719
800 434 1062 Toll Free
978 635 0286 Fax

2662 Holcomb Bridge Road, Suite 310
Alpharetta, GA 30022
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

INVOICE (complete if different from client)	
Company:	
Address:	
City, State, Zip:	
Contact:	E-mail:
Phone:	Fax:

GENERAL	
Purchase Order #:	GTX Sales Order #:
Shipped By: Maura Foley	Date Shipped: 1/11/10
Mode of Shipment: Hand-deliver	Requested Turnaround: 1 week soil, 2 weeks lab
Send Results To:	☒ CLIENT OFFICE ☐ PROJECT OFFICE
Send Results Via:	☒ E-MAIL ☐ FAX ☐ VERBAL ☒ HARD COPY

SOIL *please include boring # and depth if known		Atterberg Limits (ASTM D 4318)	California Bearing Ratio (ASTM D 1883) *specify conditions below	USCS – Classification (ASTM D 2487)	Density (ASTM D 2937)	Direct Shear (ASTM D 3080) *specify conditions below	Direct Simple Shear (ASTM D 6528) *specify conditions below	Electrical Resistivity (ASTM G 57)	Grain Size (ASTM D 422) Sieve Only / Sieve & Hydrometer please circle one	Incremental Consolidation (ASTM D 2435)	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	Permeability/ Hydraulic Conductivity (Fixed Wall - ASTM D 2434) (Flexible Wall – ASTM D 5084) please circle one	Ph (ASTM D 4972)	Proctor Compaction (Standard – ASTM D 698) (Modified – ASTM D 1557) please circle one	Specific Gravity (ASTM D 854)	Triaxial Shear (UU – ASTM D 2850) (CU – ASTM D 4767) (CD – US COE EM1110) *specify conditions below please circle one	Unconfined Compression (ASTM D 2166)	Other: x-ray tube, tube log (total unit weights, lab vanes)	Other: rock unit weight, elastic modulus, unconfined compression test
21	BB-YSL-105, 5D, 19-21'								x (s)											
22	BB-YSL-105, 7D, 29-31'								x (s)											
23	BB-YSL-106, 5D, 22-24'								x (s+h)											
24	BB-YSL-106, R1, 26.6-27.3'																			x

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: [signature] PRINT NAME: Andy Blaisdell DATE: 1/11/10

Relinquished By: [signature]	DATE: 1/11/10 TIME:	Received By: [signature]	DATE: 1/11/10 TIME:
Relinquished By: [signature]	DATE: 1/11/10 TIME:	Received By: Robert Smith	DATE: 1/11/10 TIME: 12:30

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WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

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Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
C_c	coefficient of curvature, $(D_{30})^2 / (D_{10} \times D_{60})$
C_u	coefficient of uniformity, D_{60}/D_{10}
C_c	compression index for one dimensional consolidation
C_α	coefficient of secondary compression
c_v	coefficient of consolidation
c	cohesion intercept for total stresses
c'	cohesion intercept for effective stresses
D	diameter of specimen
D_{10}	diameter at which 10% of soil is finer
D_{15}	diameter at which 15% of soil is finer
D_{30}	diameter at which 30% of soil is finer
D_{50}	diameter at which 50% of soil is finer
D_{60}	diameter at which 60% of soil is finer
D_{85}	diameter at which 85% of soil is finer
d_{50}	displacement for 50% consolidation
d_{90}	displacement for 90% consolidation
d_{100}	displacement for 100% consolidation
E	Young's modulus
e	void ratio
e_c	void ratio after consolidation
e_o	initial void ratio
G	shear modulus
G_s	specific gravity of soil particles
H	height of specimen
PI	plasticity index
i	gradient
K_o	lateral stress ratio for one dimensional strain
k	permeability
LI	Liquidity Index
m_v	coefficient of volume change
n	porosity
PI	plasticity index
P_c	preconsolidation pressure
p	$(\sigma_1 + \sigma_3) / 2$, $(\sigma_v + \sigma_h) / 2$
p'	$(\sigma'_1 + \sigma'_3) / 2$, $(\sigma'_v + \sigma'_h) / 2$
p'_c	p' at consolidation
Q	quantity of flow
q	$(\sigma_1 - \sigma_3) / 2$
q_f	q at failure
q_o, q_i	initial q
q_c	q at consolidation
S	degree of saturation
SL	shrinkage limit
s_u	undrained shear strength
T	time factor for consolidation
T	temperature
t	time
U, UC	unconfined compression test
UU, Q	unconsolidated undrained triaxial test
u_a	pore gas pressure
u_e	excess pore water pressure
u, u_w	pore water pressure
V	total volume
V_g	volume of gas
V_s	volume of solids
V_v	volume of voids
V_w	volume of water
V_o	initial volume
v	velocity
W	total weight
W_s	weight of solids
W_w	weight of water
w	water content
w_c	water content at consolidation
w_f	final water content
w_l	liquid limit
w_n	natural water content
w_p	plastic limit
w_s	shrinkage limit
w_o, w_i	initial water content
α	slope of q_f versus p_f
α'	slope of q_f versus p_f'
γ_t	total unit weight
γ_d	dry unit weight
γ_s	unit weight of solids
γ_w	unit weight of water
ε	strain
ε_{vol}	volume strain
ε_h, ε_v	horizontal strain, vertical strain
μ	Poisson's ratio, also viscosity
σ	normal stress
σ'	effective normal stress
σ_c, σ'_c	consolidation stress in isotropic stress system
σ_h, σ'_h	horizontal normal stress
σ_v, σ'_v	vertical normal stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ	shear stress
φ	friction angle based on total stresses
φ'	friction angle based on effective stresses
φ'_r	residual friction angle
φ_{ult}	φ for ultimate strength

APPENDIX C

Calculations

1. **Seismic Design Calculations**
2. **Existing Timber Pile Evaluations**
3. **New Timber Piles – Structural Resistance**
4. **New Timber Piles – Geotechnical Resistance**
5. **Pipe Piles – Pile Demand from VHB**
6. **Pipe Piles – Geotechnical Resistance**
7. **Pipe Piles – Driveability Evaluations**
8. **Pipe Piles – Lateral Pile Capacity**
9. **Recommended Rip Rap Limits**
10. **Pipe Piles – Pile Settlement**
11. **Estimate of Frost Penetration**

1. Seismic Design Calculations

HALEY & ALDRICH

Calculations

Client	Maine DOT	File No.	36654-000
Project	Sewall's Bridge	Sheet	1 of 1
Subject	Seismic Design	Date	8/19/10
		Computed By	EFW
		Checked By	JLL

AASHTO 2010, Table 3.10.3.1-1 Site Class Definitions

Check Site Class F Criteria.

- Are there peats or highly organic clays greater than 10 ft?

  Harbor bottom deposits are present. Thickness ranges from 3 to 14 ft (average 8 ft) However it is not peat or highly organic clay <u>and</u> on average is less than 10 ft thick.
  ∴ No

- Are there highly plastic clays greater than 25 ft?
  No

- Are there soft/medium stiff clays greater than 120 ft?
  No.

Since no Site Class F Criteria are met, <u>use Site Class E</u>

Site Location:

43° 8' 10" N = 43.1361
70° 39' 39" W = 70.6608

Using USGS Seismic Design Parameters Software:

$A_s = 0.242$
$S_{DS} = 0.186$
$S_{D1} = 0.157$
$F_{pga} = 2.5$
$F_a = 2.5$
$F_v = 3.5$
$SD_S = 0.466$
$SD_1 = 0.157$

see attached output

CAL02AFH.FRP 20 November 1997

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 43.136100
Longitude = -070.660800
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.097	PGA - Site Class B
0.2	0.186	Ss - Site Class B
1.0	0.044	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 43.136100
Longitude = -070.660800
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.242	As - Site Class E
0.2	0.466	SDs - Site Class E
1.0	0.154	SD1 - Site Class E

2. Existing Timber Pile Evaluations

DESIGN AXIAL COMPRESSION PILE RESISTANCE CALCULATIONS - NORTH ABUTMENT
PROPOSED BRIDGE REHABILITATION
SEWALLS BRIDGE (PIN 12665.00)
YORK, MAINE

HALEY & ALDRICH, INC. FILE NO.: 36654-000

Beta Method, with Scour (assume 2 ft pile embedment, tip resistance only)

Embedment Depth (ft)	Total Unit Weight of Bearing Soils (pcf)	Effective Stress at Pile Tip (psf)	Diameter of Pile Tip (in.)	Area of Pile Tip (sqft.)	Nt	Reduction Factor	Nomrinal End Bearing Resist. (tons)	Factored End Bearing Resist. (tons)	Nominal Friction Resist. (tons)	Factored Friction Resist. (tons)	Total Nominal Resist. (tons)	Total Factored Resist. (tons)
2.0	118	110	10	0.545	30	0.25	0.9	0.2	0.0	0.0	0.9	0.2

Beta Method, without Scour (assume typical 11 ft pile embedment, tip resistance only)

Embedment Depth (ft)	Total Unit Weight of Bearing Soils (pcf)	Effective Stress at Pile Tip (psf)	Diameter of Pile Tip (in.)	Area of Pile Tip (sqft.)	Nt	Reduction Factor	Nomrinal End Bearing Resist. (tons)	Factored End Bearing Resist. (tons)	Nominal Friction Resist. (tons)	Factored Friction Resist. (tons)	Total Nominal Resist. (tons)	Total Factored Resist. (tons)
11.0	118	605	10	0.545	30	0.25	4.9	1.2	1.1	0.3	6.0	1.5

Nordlund-Thurman Method, with Scour (assume 2 ft pile embedment, tip resistance only)

Embedment Depth, D (ft)	Total Unit Weight of Bearing Soils (pcf)	Effective Stress at Pile Tip (psf)	Diameter of Pile Tip, b (in.)	Area of Pile Tip (sqft.)	D/b	αt	Nq	Reduction Factor	Nominal End Bearing Resist. (tons)	Factored End Bearing Resist. (tons)	Nominal Friction Resist. (tons)	Factored Friction Resist. (tons)	Total Nominal Resist. (tons)	Total Factored Resist. (tons)
2.0	118	110	10	0.545	2.4	0.6	40	0.45	0.7	0.3	0.0	0.0	0.7	0.3

Nordlund-Thurman Method, without Scour (assume typical 11 ft pile embedment, tip resistance only)

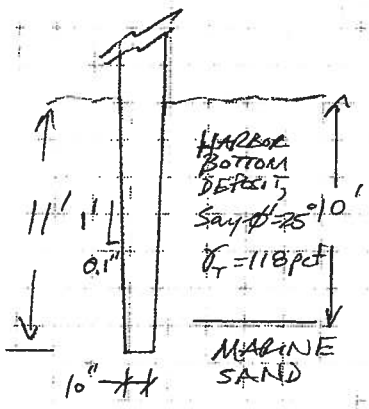
Embedment Depth, D (ft)	Total Unit Weight of Bearing Soils (pcf)	Effective Stress at Pile Tip (psf)	Diameter of Pile Tip, b (in.)	Area of Pile Tip (sqft.)	D/b	αt	Nq	Reduction Factor	Nominal End Bearing Resist. (tons)	Factored End Bearing Resist. (tons)	Nominal Friction Resist. (tons)	Factored Friction Resist. (tons)	Total Nominal Resist. (tons)	Total Factored Resist. (tons)
11.0	118	605	10	0.545	13.2	0.6	40	0.45	4.0	1.8	1.1	0.5	5.1	2.3

By: ARB
checked by: CLH

HALEY & ALDRICH	**Calculations**	File No.	36654-000
		Sheet	1 of
Client	MAINEDOT	Date	1/27/10
Project	SEWALLS RRIVSE	Computed By	ARB
Subject	Friction Capacity, Existing Piles	Checked By	CLH

→ Estimate skin friction of existing timber pile assuming less than full scour.

Current "Typical" Controlling Condition (Piers 1 & 8)



⇒ To estimate "current" capacity, neglect skin friction on upper 3' of pile, and assume all 8' of friction length is in harbor bottom deposit.

Beta Method

$R_{SIDE} = \beta \cdot \sigma_v' \cdot A$ → From Fig. 9.20 attached, say $\beta = 0.25$

$R_{SIDE} = 0.25 \times (4' \times (118 - 62.4)) \times [\pi \times (10.5''/12'') \times 8'] = 1{,}222$ lb $\cong$ 0.6 tons

Nordlund-Thurman Method

$R_{SIDE} = K_\delta \cdot C_F \cdot \sigma_v' \cdot \frac{\sin(\delta + \omega)}{\cos(\omega)} \times A$

→ Assume taper of 0.1 in/ft, Displaced volume (m^3/m) = $\pi/4 \left(\frac{10.5''}{12''}\right)^2 \times 1$ ft

$V = 0.60$ ft³/ft ∴ From Fig. 10.7.3.8.6f-6,

$\delta/\phi = 0.57$

→ From Fig. 10.7.3.8.6f-5, $C_F = 0.82$

→ From Fig. 10.7.3.8.6f-1, $K_\delta = 1.25$

$R_{SIDE} = 1.25 \times 0.82 \times (4' \times (118 - 62.4)) \times \frac{\sin(0.57 \times 25 + 0.48)}{\cos 0.48} \times [\pi \times (10.5''/12'') \times 8']$

$\cong 0.26 \times \sigma_v' \times A \cong$ 0.6 tons

CAL02AFH.FRP 20 November 1997

HALEY & ALDRICH

Calculations

Client	MAINEDOT	File No.	36684-000
Project	SEWALLS BRIDGE	Sheet	2 of
Subject	Friction Capacity, Existing Piles	Date	1/28/10
		Computed By	ARB
		Checked By	CLH

⇒ If embedment is full 11 ft,

$$R_{SIDE} = 0.6\text{ tons} \times \left[\frac{5.5'}{4'} \times \frac{11'}{8'}\right] \cong \underline{1.1\text{ tons}}$$ ← Max existing friction capacity

Deeper midpoint for effective stress

Longer pile length in friction

Using Nordlund-Thurman resistance factor, the design friction capacity is 0.45×1.1 tons = $\underline{0.5\text{ tons}}$

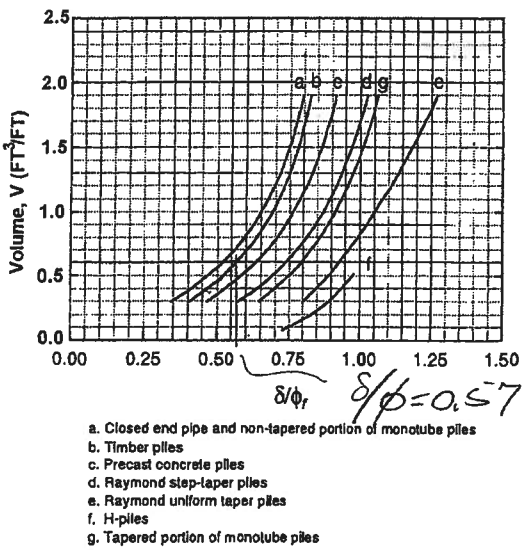


Figure 10.7.3.8.6f-6 Relation of δ/ϕ_f and Pile Displacement, *V*, for Various Types of Piles (*Hannigan et al., 2005* after *Nordlund, 1979*).

The nominal unit tip resistance, q_p, in ksf by the Nordlund/Thurman method shall be taken as:

$$q_p = \alpha_t N'_q \sigma'_v \leq q_L \qquad (10.7.3.8.6f\text{-}2)$$

where:

α_t = coefficient from Figure 7 (dim.)

N'_q = bearing capacity factor from Figure 8

σ'_v = effective overburden stress at pile tip (ksf) ≤3.2 ksf

q_L = limiting unit tip resistance from Figure 9

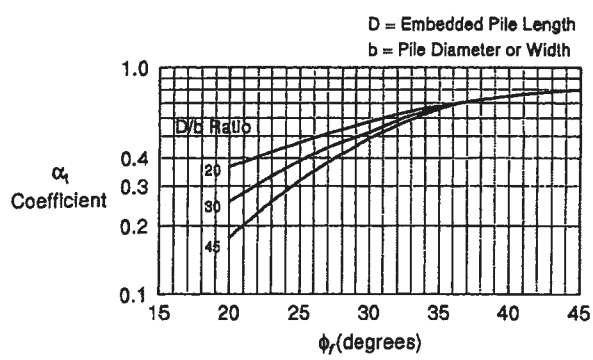


Figure 10.7.3.8.6f-7 α_t Coefficient (*Hannigan et al., 2005* modified after *Bowles, 1977*).

If the friction angle, ϕ_f, is estimated from average, corrected *SPT* blow counts, $N1_{60}$, the $N1_{60}$ values should be averaged over the zone from the pile tip to 2 diameters below the pile tip.

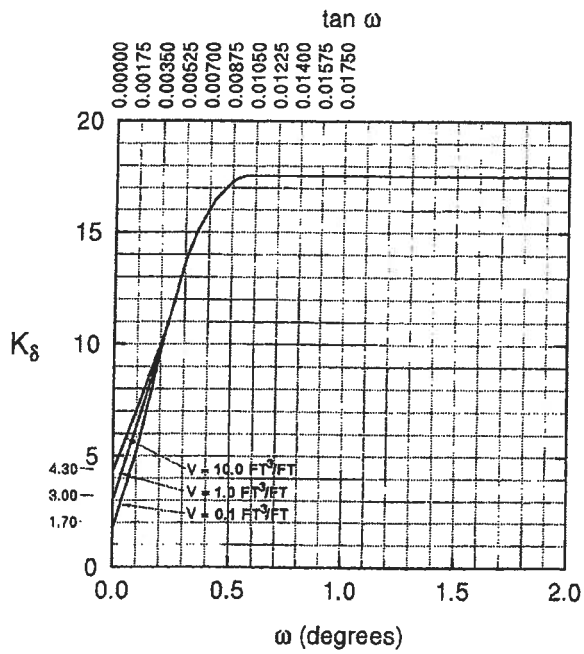


Figure 10.7.3.8.6f-4 Design Curve for Evaluating K_δ for Piles where $\phi_f = 40°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

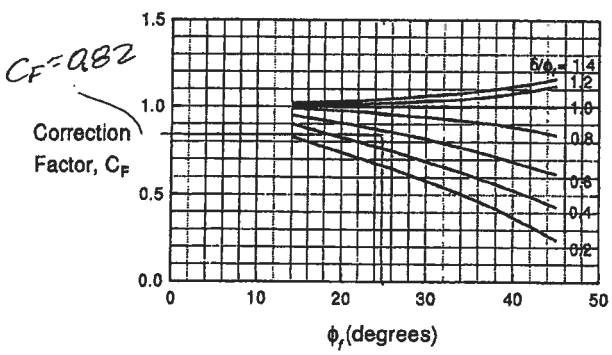


Figure 10.7.3.8.6f-5 Correction Factor for K_δ where $\delta \neq \phi_f$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

C_F = correction factor for K_δ when $\delta \neq \phi_f$, from Figure 5

σ'_v = effective overburden stress at midpoint of soil layer under consideration (ksf)

δ = friction angle between pile and soil obtained from Figure 6 (deg.)

ω = angle of pile taper from vertical (deg.)

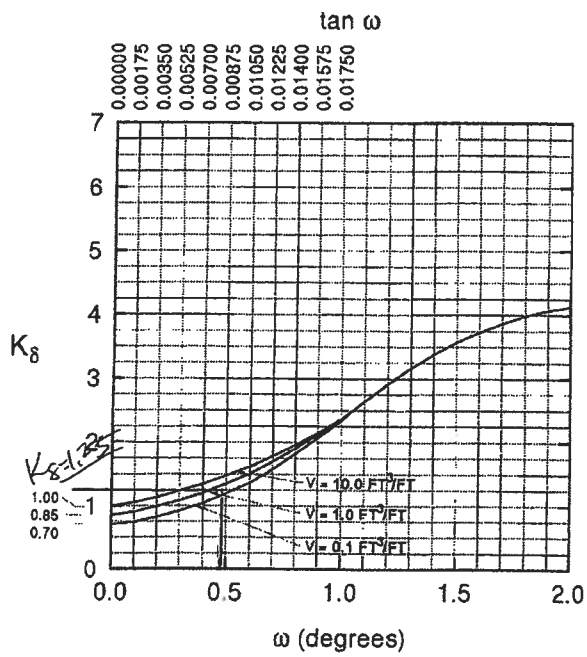
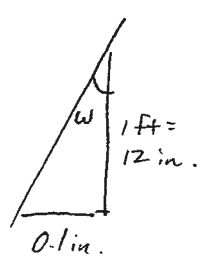


Figure 10.7.3.8.6f-1 Design Curve for Evaluating K_δ for Piles where $\phi_f = 25°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).



$$\omega = \tan^{-1}\left(\frac{0.1\text{ in}}{12\text{ in}}\right) = 0.48°$$

3. New Timber Piles – Structural Resistance

Checked by: JLL
03 MAR 2010

DESIGN AXIAL COMPRESSION PILE RESISTANCE CALCULATIONS - STRUCTURAL CAPACITY OF TIMBER PILES
PROPOSED BRIDGE REHABILITATION
SEWALLS BRIDGE (PIN 12665.00)
YORK, MAINE

HALEY & ALDRICH, INC. FILE NO.: 38654-000

NOTES:
1) Structural resistance of timber piles determined in accordance with AASHTO LRFD 2007 Specifications with interim revisions.
2) See calculations dated 1/28/10 for source and description of modification factors.

35'

Condition	Fco (ksi)	Ckf	Cm	Cf	Ci	Cl	Fc (ksi)	Lu (in)	d (in)	Lu/d	Le (in)	Kce	E (ksi)	d.top (in)	Fce	B	c	Co	d.tip (in)	A (in^2)	Pn (kips)	ϕ	Pr (kips)
Strength I	1.2	2.78	1	1	1	0.8	2.67	420	12	35.0		0.76	1500	12	0.27	0.10	0.85		8	50.3	13.6	0.9	12.2
Strength II/III	1.2	2.78	1	1	1	1	3.33	420	12	35.0		0.76	1500	12	0.27	0.08	0.85		8	50.3	13.6	0.9	12.3
Strength IV	1.2	2.78	1	1	1	0.6	2.00	420	12	35.0		0.76	1500	12	0.27	0.14	0.85		8	50.3	13.5	0.9	12.2
Extreme I	1.2	2.78	1	1	1	1	3.33	420	12	35.0		0.76	1500	12	0.27	0.08	0.85		8	50.3	13.6	0.9	12.3
Driving	1.2	2.17	1	1	1	1	2.61	420	12	35.0		0.76	1500	12	0.27	0.11	0.85		8	50.3	13.6	1.15	15.6

16.7'

Condition	Fco (ksi)	Ckf	Cm	Cf	Ci	Cl	Fc (ksi)	Lu (in)	d (in)	Lu/d	Le (in)	Kce	E (ksi)	d (in)	Fce	B	c	Co	d.tip (in)	A (in^2)	Pn (kips)	ϕ	Pr (kips)
Strength I	1.2	2.78	1	1	1	0.8	2.67	200	12	16.7		0.76	1500	12	1.21	0.45	0.85		8	50.3	55.1	0.9	49.6
Strength II/III	1.2	2.78	1	1	1	1	3.33	200	12	16.7		0.76	1500	12	1.21	0.36	0.85		8	50.3	56.6	0.9	50.9
Strength IV	1.2	2.78	1	1	1	0.6	2.00	200	12	16.7		0.76	1500	12	1.21	0.61	0.85		8	50.3	52.4	0.9	47.1
Extreme I	1.2	2.78	1	1	1	1	3.33	200	12	16.7		0.76	1500	12	1.21	0.36	0.85		8	50.3	56.6	0.9	50.9
Driving	1.2	2.17	1	1	1	1	2.61	200	12	16.7		0.76	1500	12	1.21	0.46	0.85		8	50.3	55.0	1.15	63.2

8.3'

Condition	Fco (ksi)	Ckf	Cm	Cf	Ci	Cl	Fc (ksi)	Lu (in)	d (in)	Lu/d	Le (in)	Kce	E (ksi)	d (in)	Fce	B	c	Co	d.tip (in)	A (in^2)	Pn (kips)	ϕ	Pr (kips)
Strength I	1.2	2.78	1	1	1	0.8	2.67	100	12	8.3		0.76	1500	12	4.15	1.55	0.85		8	50.3	113.6	0.9	102.2
Strength II/III	1.2	2.78	1	1	1	1	3.33	100	12	8.3		0.76	1500	12	4.15	1.24	0.85		8	50.3	132.7	0.9	119.4
Strength IV	1.2	2.78	1	1	1	0.6	2.00	100	12	8.3		0.76	1500	12	4.15	2.07	0.85		8	50.3	90.2	0.9	61.2
Extreme I	1.2	2.78	1	1	1	1	3.33	100	12	8.3		0.76	1500	12	4.15	1.24	0.85		8	50.3	132.7	0.9	119.4
Driving	1.2	2.17	1	1	1	1	2.61	100	12	8.3		0.76	1500	12	4.15	1.59	0.85		8	50.3	111.7	1.15	128.5

varies

By: ARB
Checked by: JLL

Printed
7/30/2010

HALEY & ALDRICH	Calculations	File No.	36654-000
		Sheet	1 of 2
Client	MAINEDOT	Date	1/28/10
Project	SEWALLS BRIDGE	Computed By	ABB
Subject	Timber Pile - structural Capacity	Checked By	JLL

Determine structural capacity of timber piles in accordance with 2007 AASHTO LRFD (w/ Interim revisions).

- Per Article 10.7.3.13.3, nominal axial compression resistance determined in accordance w/ Art. 8.8.2.
- Per Art. 8.8.2, nominal resistance, P_n, in compression parallel to grain is.

$$P_n = F_c A C_p$$

F_c ← Adjusted design value per Art. 8.4.4

- Per Art. 8.4.4, Adjusted design value, F_c:

$$F_c = F_{co} \cdot C_{KF} \, C_M \cdot C_F \cdot C_i \cdot C_\lambda$$

F_{co} ← reference design value in wood in compression parallel to grain

$C_{KF}, C_M, C_F, C_i, C_\lambda$ ← adjustment factors

▷ Per Art. 8.4.1.3, Table 8.4.1.3-1, $F_{co} = 1.20$ ksi for Southern Pine

▷ Per Art. 8.4.4.2, $C_{KF} = 2.5/\phi$, where ϕ = resistance factor for compression parallel to grain OR resistance during pile driving. Per Art. 8.5.2.2:

$\phi = 0.90$ for compression parallel to grain ∴ $C_{KF} = 2.78$

$\phi = 1.15$ for resistance during pile driving ∴ $C_{KF} = 2.17$

▷ Per Art. 8.4.4.3, C_M (wet service factor), reduction not required for southern pine greater than 5"x5" ∴ $C_M = 1.0$

▷ Per Art. 8.4.4.4, C_F (size factor for sawn lumber), $C_F = 1.0$

▷ Per Art. 8.4.4.7, C_i (incising factor) NA, not incised $C_i = 1.0$

▷ Per Art. 8.4.4.9 C_λ (time effect factor) varies by strength limit state:

Strength I: $C_\lambda = 0.8$

Strength II, III, Extreme I: $C_\lambda = 1.0$

Strength IV: $C_\lambda = 0.6$

HALEY & ALDRICH

Calculations

Client	MAINE DOT	File No.	36654-000
Project	SEWALLS BRIDGE	Sheet	2 of 2
Subject	Timber Pile Capacity	Date	1/28/10
		Computed By	ARB
		Checked By	JLL

* Resulting F_c values:

CONDITION	F_{co}	C_{KF}	C_d	F_c
Strength I	↑	2.78	0.8	2.67 ksi
Strength II/III		2.78	1.0	3.37 ksi
Strength IV	1.2	2.78	0.6	2.00 ksi
Extreme I	ksi	2.78	1.0	3.37 ksi
Driving	↓	2.17	1.0	2.60 ksi

- Nominal Resistance, $P_n = F_c \cdot A \cdot C_p$, say $A = 50.3\ in^2$ for 8"ϕ tip
  - ▷ Eq. 8.8.2-2: $C_p = \frac{1+B}{c} - \sqrt{\frac{(1+B)^2}{c} - \frac{B}{c}}$, $c = 0.85$ for timber piles
  - ▷ Eq. 8.8.2-3; $B = \frac{F_{cE}}{F_c}$
  - ▷ Eq. 8.8.2-4: $F_{cE} = \frac{K_{cE} E d^2}{L_e^2}$, $K_{cE} = 0.76$ for round piles
    - E = adjusted mod. of Elast. (8.4.4.1)
    - d = diameter
    - L_e = effective length (KL, in.)

UNCLEAR IF APPLIED CORRECTLY

- □ $E = E_o C_M C_i = E_o = 1{,}500$ ksi (Table 8.4.1.3-1, Southern Pine)
- □ Say $L_u = 35$ ft (max unsupported length) = 420 in.
- □ Say $d = 12$ in. in unsupported zone

$$\cancel{F_{cE} = \frac{0.76 \times 1500\ ksi \times (12\ in)^2}{(420\ in)^2} = 0.93}$$

From Art. 8.6.2,
- If $L_u/d < 7$, $L_e = 2.06 L_u$
- If $7 \le L_u/d < 14.3$, $L_e = 1.63 L_u + 3d$
- If $L_u/d > 14.3$, $L_e = 1.84 L_u$

* Resulting F_{cE}, P_n, and P_r ($P_r = \phi \cdot P_n$) values presented on Excel spreadsheet.

FINDING: Unsupported length DRAMATICALLY impacts allowable load, confirm w/ VHB prior to determining design pile resistance

CAL02AFH.FRP 20 November 1997

4. New Timber Piles – Geotechnical Resistance

Project: Sewalls Bridge
Subject: Bent 1-3 timber pile capacity
β-method, ASD

Printed: 02 AUG 2012
JLL

FS
2

Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Dr %	K	β	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	n	N'q	N'gamma	Unit Tip Resistance (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Allowable Resistance (kips)	
								Side Resistance - Beta Method - Bhushan (1982)					Tip Resistance - Hansen Method								
-10	0	Harbor Bottom Deposit	118	0	1	28	25	0.70	0.34	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-11	1	Harbor Bottom Deposit	118	55	1	28	23	0.69	0.33	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-12	2	Harbor Bottom Deposit	120	112	1	28	22	0.67	0.32	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-13	3	Harbor Bottom Deposit	120	169	1	28	20	0.66	0.31	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-14	4	Harbor Bottom Deposit	120	226	1	28	19	0.66	0.31	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-15	5	Harbor Bottom Deposit	120	283	1	28	18	0.65	0.30	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-16	6	Harbor Bottom Deposit	120	340	1	28	18	0.64	0.29	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-17	7	Harbor Bottom Deposit	120	397	1	28	17	0.64	0.29	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-18	8	Harbor Bottom Deposit	120	454	1	28	16	0.63	0.29	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-19	9	Harbor Bottom Deposit	120	511	1	28	16	0.63	0.28	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-20	10	Harbor Bottom Deposit	120	568	1	28	15	0.62	0.28	0	10	0	0.6	14.72	10.94	0	8	0	0	0	
-21	11	Marine Deposit	120	625	29	35	80	1.14	0.70	437	10	1	0.6	33.30	33.92	13137	8	5	5	3	
-22	12	Marine Deposit	120	682	29	35	78	1.12	0.68	467	10	2	0.6	33.30	33.92	14276	8	5	7	3	
-23	13	Marine Deposit	120	739	29	35	76	1.10	0.67	496	10	3	0.6	33.30	33.92	15415	8	5	8	4	
-24	14	Marine Deposit	120	796	29	35	74	1.09	0.66	525	10	4	0.6	33.30	33.92	16553	8	6	10	5	
-25	15	Marine Deposit	120	853	29	35	72	1.08	0.65	552	10	6	0.6	33.30	33.92	17692	8	6	12	6	
-26	16	Marine Deposit	120	910	29	35	70	1.06	0.64	579	10	7	0.6	33.30	33.92	18831	8	7	14	7	
-27	17	Marine Deposit	120	967	29	35	69	1.05	0.63	606	10	9	0.6	33.30	33.92	19970	8	7	16	8	
-28	18	Marine Deposit	120	1024	29	35	67	1.04	0.62	632	10	10	0.6	33.30	33.92	21108	8	7	18	9	
-29	19	Marine Deposit	120	1081	29	35	66	1.03	0.61	658	10	12	0.6	33.30	33.92	22247	8	8	20	10	
-30	20	Marine Deposit	120	1138	29	35	65	1.02	0.60	683	10	14	0.6	33.30	33.92	23386	8	8	22	11	560 280
-31	21	Glacial Till	125	1200	100	38	118	1.44	0.94	1133	10	16	0.6	48.93	56.17	36404	8	13	29	14	
-32	22	Glacial Till	125	1262	100	38	115	1.42	0.93	1172	10	19	0.6	48.93	56.17	38224	8	13	33	16	
-33	23	Glacial Till	125	1324	100	38	113	1.40	0.91	1211	10	22	0.6	48.93	56.17	40045	8	14	36	18	
-34	24	Glacial Till	125	1386	100	38	111	1.39	0.90	1250	10	26	0.6	48.93	56.17	41865	8	15	40	20	
-35	25	Glacial Till	125	1448	100	38	109	1.37	0.89	1287	10	29	0.6	48.93	56.17	43685	8	15	44	22	
-36	26	Glacial Till	125	1510	100	38	107	1.36	0.88	1324	10	32	0.6	48.93	56.17	45506	8	16	48	24	
-37	27	Glacial Till	125	1572	100	38	105	1.34	0.87	1361	10	36	0.6	48.93	56.17	47326	8	17	52	26	
-38	28	Glacial Till	125	1634	100	38	104	1.33	0.85	1397	10	39	0.6	48.93	56.17	49146	8	17	57	28	
-39	29	Glacial Till	125	1696	100	38	102	1.32	0.84	1432	10	43	0.6	48.93	56.17	50967	8	18	61	30	
-40	30	Glacial Till	125	1758	100	38	101	1.31	0.83	1467	10	47	0.6	48.93	56.17	52787	8	18	65	33	

approx. bearing elev.

By: JLL
Checked by: EFW

22 MAR 2010

Project: Seawalls Bridge
Subject: Timber Pile Capacity
DM-7, ASD

Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Dr %	K	β	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	Nq	Unit Tip Resistance (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Allowable Resistance (kips)	
								Side Resistance - K and tan d Values from DM7					Tip Resistance					FS 2	
-10	0	Harbor Bottom Deposit	118	0	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-11	1	Harbor Bottom Deposit	118	55	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-12	2	Harbor Bottom Deposit	120	112	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-13	3	Harbor Bottom Deposit	120	169	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-14	4	Harbor Bottom Deposit	120	226	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-15	5	Harbor Bottom Deposit	120	283	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-16	6	Harbor Bottom Deposit	120	340	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-17	7	Harbor Bottom Deposit	120	397	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-18	8	Harbor Bottom Deposit	120	454	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-19	9	Harbor Bottom Deposit	120	511	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-20	10	Harbor Bottom Deposit	120	568	1	28		2.00	0.77	0	10	0	15	0	8	0	0	0	
-21	11	Marine Deposit	120	625	29	35		2.00	0.99	616	10	1	50	31250	8	11	12	6	
-22	12	Marine Deposit	120	682	29	35		2.00	0.99	673	10	2	50	34100	8	12	14	7	
-23	13	Marine Deposit	120	739	29	35		2.00	0.99	729	10	4	50	36950	8	13	17	9	
-24	14	Marine Deposit	120	796	29	35		2.00	0.99	785	10	6	50	39800	8	14	20	10	
-25	15	Marine Deposit	120	853	29	35		2.00	0.99	841	10	8	50	42650	8	15	23	12	
-26	16	Marine Deposit	120	910	29	35		2.00	0.99	898	10	11	50	45500	8	16	27	13	
-27	17	Marine Deposit	120	967	29	35		2.00	0.99	954	10	13	50	48350	8	17	30	15	
-28	18	Marine Deposit	120	1024	29	35		2.00	0.99	1010	10	16	50	51200	8	18	34	17	
-29	19	Marine Deposit	120	1081	29	35		2.00	0.99	1066	10	18	50	54050	8	19	37	19	
approx. bearing el. -30	20	Marine Deposit	120	1138	29	35		2.00	0.99	1122	10	21	50	56900	8	20	41	21	869 435
-31	21	Glacial Till	125	1200	100	38		2.00	1.09	1303	10	24	86	103200	8	36	60	30	
-32	22	Glacial Till	125	1262	100	38		2.00	1.09	1370	10	28	86	108532	8	38	66	33	
-33	23	Glacial Till	125	1324	100	38		2.00	1.09	1438	10	32	86	113864	8	40	71	36	
-34	24	Glacial Till	125	1386	100	38		2.00	1.09	1505	10	35	86	119196	8	42	77	39	
-35	25	Glacial Till	125	1448	100	38		2.00	1.09	1572	10	40	86	124528	8	43	83	41	
-36	26	Glacial Till	125	1510	100	38		2.00	1.09	1640	10	44	86	129860	8	45	89	45	
-37	27	Glacial Till	125	1572	100	38		2.00	1.09	1707	10	48	86	135192	8	47	95	48	
-38	28	Glacial Till	125	1634	100	38		2.00	1.09	1774	10	53	86	140524	8	49	102	51	
-39	29	Glacial Till	125	1696	100	38		2.00	1.09	1842	10	57	86	145856	8	51	108	54	
-40	30	Glacial Till	125	1758	100	38		2.00	1.09	1909	10	62	86	151188	8	53	115	58	

By: JLL
Checked by: EFW

22 MAR 2010

Project: Sewalls Bridge
Subject: Bent 1-3 timber pile capacity
β-method, LRFD

Res. Factor 0.25 AASHTO

									Side Resistance - Beta Method - Bhushan (1982)						Tip Resistance - Thurman Method								
Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Dr %	K	β	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	s'v used (psf)	α	N'q	Unit Tip Resistance (psf)	Limiting Tip Resistance (psf)	Tip Resistance Used (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Factored Resistance (kips)	
-10	0	Harbor Bottom Deposit	118	0	1	28	25	0.70	0.34	0	10	0	0	0.5	22	0	0	0	8	0	0	0	
-11	1	Harbor Bottom Deposit	118	55	1	28	23	0.69	0.33	0	10	0	55	0.5	22	605	0	0	8	0	0	0	
-12	2	Harbor Bottom Deposit	120	112	1	28	22	0.67	0.32	0	10	0	112	0.5	22	1232	0	0	8	0	0	0	
-13	3	Harbor Bottom Deposit	120	169	1	28	20	0.66	0.31	0	10	0	169	0.5	22	1859	0	0	8	0	0	0	
-14	4	Harbor Bottom Deposit	120	226	1	28	19	0.66	0.31	0	10	0	226	0.5	22	2486	0	0	8	0	0	0	
-15	5	Harbor Bottom Deposit	120	283	1	28	18	0.65	0.30	0	10	0	283	0.5	22	3113	0	0	8	0	0	0	
-16	6	Harbor Bottom Deposit	120	340	1	28	18	0.64	0.29	0	10	0	340	0.5	22	3740	0	0	8	0	0	0	
-17	7	Harbor Bottom Deposit	120	397	1	28	17	0.64	0.29	0	10	0	397	0.5	22	4367	0	0	8	0	0	0	
-18	8	Harbor Bottom Deposit	120	454	1	28	16	0.63	0.29	0	10	0	454	0.5	22	4994	0	0	8	0	0	0	
-19	9	Harbor Bottom Deposit	120	511	1	28	16	0.63	0.28	0	10	0	511	0.5	22	5621	0	0	8	0	0	0	
-20	10	Harbor Bottom Deposit	120	568	1	28	15	0.62	0.28	0	10	0	568	0.5	22	6248	0	0	8	0	0	0	
-21	11	Marine Deposit	120	625	29	35	80	1.14	0.70	437	10	1	625	0.7	65	28438	100000	28438	8	10	10	3	
-22	12	Marine Deposit	120	682	29	35	78	1.12	0.68	467	10	2	682	0.7	65	31031	100000	31031	8	11	13	3	
-23	13	Marine Deposit	120	739	29	35	76	1.10	0.67	496	10	3	739	0.7	65	33625	100000	33625	8	12	15	4	
-24	14	Marine Deposit	120	796	29	35	74	1.09	0.66	525	10	4	796	0.7	65	36218	100000	36218	8	13	17	4	
-25	15	Marine Deposit	120	853	29	35	72	1.08	0.65	552	10	6	853	0.7	65	38812	100000	38812	8	14	19	5	
-26	16	Marine Deposit	120	910	29	35	70	1.06	0.64	579	10	7	910	0.7	65	41405	100000	41405	8	14	22	5	
-27	17	Marine Deposit	120	967	29	35	69	1.05	0.63	606	10	9	967	0.7	65	43999	100000	43999	8	15	24	6	
-28	18	Marine Deposit	120	1024	29	35	67	1.04	0.62	632	10	10	1024	0.7	65	46592	100000	46592	8	16	27	7	
-29	19	Marine Deposit	120	1081	29	35	66	1.03	0.61	658	10	12	1081	0.7	65	49186	100000	49186	8	17	29	7	
-30	20	Marine Deposit	120	1138	29	35	65	1.02	0.60	683	10	14	1138	0.7	65	51779	100000	51779	8	18	32	8	560 140
-31	21	Glacial Till	125	1200	100	38	118	1.44	0.94	1133	10	16	1200	0.72	105	90720	250000	90720	8	32	48	12	
-32	22	Glacial Till	125	1262	100	38	115	1.42	0.93	1172	10	19	1262	0.72	105	95407	250000	95407	8	33	53	13	
-33	23	Glacial Till	125	1324	100	38	113	1.40	0.91	1211	10	22	1324	0.72	105	100094	250000	100094	8	35	57	14	
-34	24	Glacial Till	125	1386	100	38	111	1.39	0.90	1250	10	26	1386	0.72	105	104782	250000	104782	8	37	62	16	
-35	25	Glacial Till	125	1448	100	38	109	1.37	0.89	1287	10	29	1448	0.72	105	109469	250000	109469	8	38	67	17	
-36	26	Glacial Till	125	1510	100	38	107	1.36	0.88	1324	10	32	1510	0.72	105	114156	250000	114156	8	40	72	18	
-37	27	Glacial Till	125	1572	100	38	105	1.34	0.87	1361	10	36	1572	0.72	105	118843	250000	118843	8	41	77	19	
-38	28	Glacial Till	125	1634	100	38	104	1.33	0.85	1397	10	39	1634	0.72	105	123530	250000	123530	8	43	83	21	
-39	29	Glacial Till	125	1696	100	38	102	1.32	0.84	1432	10	43	1696	0.72	105	128218	250000	128218	8	45	88	22	
-40	30	Glacial Till	125	1758	100	38	101	1.31	0.83	1467	10	47	1758	0.72	105	132905	250000	132905	8	46	93	23	

approx. bearing el

By: JLL
Checked by: EFW

2010 MAR 22

Project: Sewalls Bridge
Subject: Bent 1-3 Timber Pile Capacity
DM-7, LRFD

Res. Factor 0.25 AASHTO

Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Dr %	Side Resistance - K and tan d Values from DM7: K	β	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	Tip Resistance - Thurman Method: s'v used (psf)	α	N'q	Unit Tip Resistance (psf)	Limiting Tip Resistance (psf)	Tip Resistance Used (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Factored Resistance (kips)		
-10	0	Harbor Bottom Deposit	118	0	1	28		2.00	0.77	0	10	0	0	0.5	22	0	0	0	8	0	0	0		
-11	1	Harbor Bottom Deposit	118	55	1	28		2.00	0.77	0	10	0	55	0.5	22	605	0	0	8	0	0	0		
-12	2	Harbor Bottom Deposit	120	112	1	28		2.00	0.77	0	10	0	112	0.5	22	1232	0	0	8	0	0	0		
-13	3	Harbor Bottom Deposit	120	169	1	28		2.00	0.77	0	10	0	169	0.5	22	1859	0	0	8	0	0	0		
-14	4	Harbor Bottom Deposit	120	226	1	28		2.00	0.77	0	10	0	226	0.5	22	2486	0	0	8	0	0	0		
-15	5	Harbor Bottom Deposit	120	283	1	28		2.00	0.77	0	10	0	283	0.5	22	3113	0	0	8	0	0	0		
-16	6	Harbor Bottom Deposit	120	340	1	28		2.00	0.77	0	10	0	340	0.5	22	3740	0	0	8	0	0	0		
-17	7	Harbor Bottom Deposit	120	397	1	28		2.00	0.77	0	10	0	397	0.5	22	4367	0	0	8	0	0	0		
-18	8	Harbor Bottom Deposit	120	454	1	28		2.00	0.77	0	10	0	454	0.5	22	4994	0	0	8	0	0	0		
-19	9	Harbor Bottom Deposit	120	511	1	28		2.00	0.77	0	10	0	511	0.5	22	5621	0	0	8	0	0	0		
-20	10	Harbor Bottom Deposit	120	568	1	28		2.00	0.77	0	10	0	568	0.5	22	6248	0	0	8	0	0	0		
-21	11	Marine Deposit	120	625	29	35		2.00	0.99	616	10	1	625	0.7	65	28438	100000	28438	8	10	11	3		
-22	12	Marine Deposit	120	682	29	35		2.00	0.99	673	10	2	682	0.7	65	31031	100000	31031	8	11	13	3		
-23	13	Marine Deposit	120	739	29	35		2.00	0.99	729	10	4	739	0.7	65	33625	100000	33625	8	12	16	4		
-24	14	Marine Deposit	120	796	29	35		2.00	0.99	785	10	6	796	0.7	65	36218	100000	36218	8	13	19	5		
-25	15	Marine Deposit	120	853	29	35		2.00	0.99	841	10	8	853	0.7	65	38812	100000	38812	8	14	22	5		
-26	16	Marine Deposit	120	910	29	35		2.00	0.99	898	10	11	910	0.7	65	41405	100000	41405	8	14	25	6		
-27	17	Marine Deposit	120	967	29	35		2.00	0.99	954	10	13	967	0.7	65	43999	100000	43999	8	15	28	7		
-28	18	Marine Deposit	120	1024	29	35		2.00	0.99	1010	10	16	1024	0.7	65	46592	100000	46592	8	16	32	8		
-29	19	Marine Deposit	120	1081	29	35		2.00	0.99	1066	10	18	1081	0.7	65	49186	100000	49186	8	17	36	9		
-30	20	Marine Deposit	120	1138	29	35		2.00	0.99	1122	10	21	1138	0.7	65	51779	100000	51779	8	18	39	10	869	217
-31	21	Glacial Till	125	1200	100	38		2.00	1.09	1303	10	24	1200	0.72	105	90720	250000	90720	8	32	56	14		
-32	22	Glacial Till	125	1262	100	38		2.00	1.09	1370	10	28	1262	0.72	105	95407	250000	95407	8	33	61	15		
-33	23	Glacial Till	125	1324	100	38		2.00	1.09	1438	10	32	1324	0.72	105	100094	250000	100094	8	35	67	17		
-34	24	Glacial Till	125	1386	100	38		2.00	1.09	1505	10	35	1386	0.72	105	104782	250000	104782	8	37	72	18		
-35	25	Glacial Till	125	1448	100	38		2.00	1.09	1572	10	40	1448	0.72	105	109469	250000	109469	8	38	78	19		
-36	26	Glacial Till	125	1510	100	38		2.00	1.09	1640	10	44	1510	0.72	105	114156	250000	114156	8	40	84	21		
-37	27	Glacial Till	125	1572	100	38		2.00	1.09	1707	10	48	1572	0.72	105	118843	250000	118843	8	41	90	22		
-38	28	Glacial Till	125	1634	100	38		2.00	1.09	1774	10	53	1634	0.72	105	123530	250000	123530	8	43	96	24		
-39	29	Glacial Till	125	1696	100	38		2.00	1.09	1842	10	57	1696	0.72	105	128218	250000	128218	8	45	102	26		
-40	30	Glacial Till	125	1758	100	38		2.00	1.09	1909	10	62	1758	0.72	105	132905	250000	132905	8	46	109	27		

approx. bearing el.

22 MAR 2010

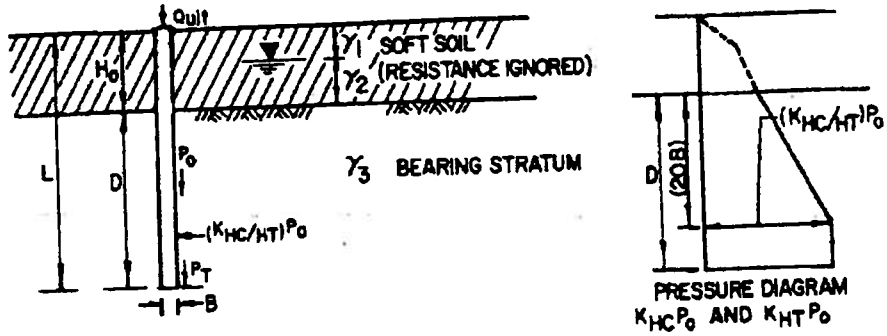
Project: Sewalls Bridge
Subject: Bent 1-3 Timber Pile Capacity
Nordlund / Thurman Method

							Side Resistance - Nordlund Method								Tip Resistance - Thurman Method								Res. Factor 0.35	NCHRP		
Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Kd	Cf	δ	βeff	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	s'v used (psf)	α	N'q	Unit Tip Resistance (psf)	Limiting Tip Resistance (psf)	Tip Resistance Used (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Factored Resistance (kips)		
-10	0	Harbor Bottom Deposit	118	0	1	28	2.06	0.78	14	0.40	0	10	0	0	0.5	22	0	0	0	8	0	0	0		
-11	1	Harbor Bottom Deposit	118	55	1	28	2.06	0.78	14	0.40	0	10	0	55	0.5	22	605	0	0	8	0	0	0		
-12	2	Harbor Bottom Deposit	120	112	1	28	2.06	0.78	14	0.40	0	10	0	112	0.5	22	1232	0	0	8	0	0	0		
-13	3	Harbor Bottom Deposit	120	169	1	28	2.06	0.78	14	0.40	0	10	0	169	0.5	22	1859	0	0	8	0	0	0		
-14	4	Harbor Bottom Deposit	120	226	1	28	2.06	0.78	14	0.40	0	10	0	226	0.5	22	2486	0	0	8	0	0	0		
-15	5	Harbor Bottom Deposit	120	283	1	28	2.06	0.78	14	0.40	0	10	0	283	0.5	22	3113	0	0	8	0	0	0		
-16	6	Harbor Bottom Deposit	120	340	1	28	2.06	0.78	14	0.40	0	10	0	340	0.5	22	3740	0	0	8	0	0	0		
-17	7	Harbor Bottom Deposit	120	397	1	28	2.06	0.78	14	0.40	0	10	0	397	0.5	22	4367	0	0	8	0	0	0		
-18	8	Harbor Bottom Deposit	120	454	1	28	2.06	0.78	14	0.40	0	10	0	454	0.5	22	4994	0	0	8	0	0	0		
-19	9	Harbor Bottom Deposit	120	511	1	28	2.06	0.78	14	0.40	0	10	0	511	0.5	22	5621	0	0	8	0	0	0		
-20	10	Harbor Bottom Deposit	120	568	1	28	2.06	0.78	14	0.40	0	10	0	568	0.5	22	6248	0	0	8	0	0	0		
-21	11	Marine Deposit	120	625	29	35	5.40	0.65	18	1.08	678	10	1	625	0.7	65	28438	100000	28438	8	10	11	4		
-22	12	Marine Deposit	120	682	29	35	5.40	0.65	18	1.08	740	10	3	682	0.7	65	31031	100000	31031	8	11	14	5		
-23	13	Marine Deposit	120	739	29	35	5.40	0.65	18	1.08	802	10	5	739	0.7	65	33625	100000	33625	8	12	16	6		
-24	14	Marine Deposit	120	796	29	35	5.40	0.65	18	1.08	863	10	7	796	0.7	65	36218	100000	36218	8	13	20	7		
-25	15	Marine Deposit	120	853	29	35	5.40	0.65	18	1.08	925	10	9	853	0.7	65	38812	100000	38812	8	14	23	8		
-26	16	Marine Deposit	120	910	29	35	5.40	0.65	18	1.08	987	10	12	910	0.7	65	41405	100000	41405	8	14	26	9		
-27	17	Marine Deposit	120	967	29	35	5.40	0.65	18	1.08	1049	10	14	967	0.7	65	43999	100000	43999	8	15	30	10		
-28	18	Marine Deposit	120	1024	29	35	5.40	0.65	18	1.08	1111	10	17	1024	0.7	65	46592	100000	46592	8	16	34	12		
-29	19	Marine Deposit	120	1081	29	35	5.40	0.65	18	1.08	1173	10	20	1081	0.7	65	49186	100000	49186	8	17	37	13		
-30	20	Marine Deposit	120	1138	29	35	5.40	0.65	18	1.08	1234	10	23	1138	0.7	65	51779	100000	51779	8	18	41	15	956	335
-31	21	Glacial Till	125	1200	100	38	12.36	0.65	19	2.68	3218	10	29	1200	0.72	105	90720	250000	90720	8	32	61	21		
-32	22	Glacial Till	125	1262	100	38	12.36	0.65	19	2.68	3385	10	38	1262	0.72	105	95407	250000	95407	8	33	71	25		
-33	23	Glacial Till	125	1324	100	38	12.36	0.65	19	2.68	3551	10	47	1324	0.72	105	100094	250000	100094	8	35	82	29		
-34	24	Glacial Till	125	1386	100	38	12.36	0.65	19	2.68	3717	10	56	1386	0.72	105	104782	250000	104782	8	37	93	33		
-35	25	Glacial Till	125	1448	100	38	12.36	0.65	19	2.68	3883	10	66	1448	0.72	105	109469	250000	109469	8	38	105	37		
-36	26	Glacial Till	125	1510	100	38	12.36	0.65	19	2.68	4050	10	77	1510	0.72	105	114156	250000	114156	8	40	117	41		
-37	27	Glacial Till	125	1572	100	38	12.36	0.65	19	2.68	4216	10	88	1572	0.72	105	118843	250000	118843	8	41	129	45		
-38	28	Glacial Till	125	1634	100	38	12.36	0.65	19	2.68	4382	10	99	1634	0.72	105	123530	250000	123530	8	43	142	50		
-39	29	Glacial Till	125	1696	100	38	12.36	0.65	19	2.68	4549	10	111	1696	0.72	105	128218	250000	128218	8	45	155	54		
-40	30	Glacial Till	125	1758	100	38	12.36	0.65	19	2.68	4715	10	123	1758	0.72	105	132905	250000	132905	8	46	169	59		

By: JLL
Checked by: EFW

22 MAR 2010

DM7



(A) ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = P_T N_q A_T + \sum_{H=H_o}^{H=H_o+D} (K_{HC})(P_o)(\tan\delta)(S)$$

WHERE Q_{ult} = ULTIMATE LOAD CAPACITY IN COMPRESSION

P_T = EFFECTIVE VERTICAL STRESS AT PILE TIP (SEE NOTE 1)

N_q = BEARING CAPACITY FACTOR (SEE TABLE, FIGURE 1 CONTINUED)

A_T = AREA OF PILE TIP

K_{HC} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN COMPRESSION.

P_o = EFFECTIVE VERTICAL STRESS OVER LENGTH OF EMBEDMENT, D (SEE NOTE 1)

δ = FRICTION ANGLE BETWEEN PILE AND SOIL (SEE TABLE, FIGURE 1 CONTINUED)

S = SURFACE AREA OF PILE PER UNIT LENGTH

FOR CALCULATING Q_{all}, USE F_S OF 2 FOR TEMPORARY LOADS, 3 FOR PERMANENT LOADS. (SEE NOTE 2)

(B) ULTIMATE LOAD CAPACITY IN TENSION

$$T_{ult} = \sum_{H=H_o}^{H=H_o+D} (K_{HT})(P_o)(\tan\delta)(S)(H)$$

WHERE: T_{ult} = ULTIMATE LOAD CAPACITY IN TENSION, PULLOUT

K_{HT} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN TENSION

FOR CALCULATING T_{all}, USE $F_S = 3$ ON T_{ult} PLUS THE WEIGHT OF THE PILE (W_P), THUS $T_{all} = \frac{T_{ult}}{3} + W_P$ (SEE NOTE 2)

NOTE-1: EXPERIMENTAL AND FIELD EVIDENCE INDICATE THAT BEARING PRESSURE AND SKIN FRICTION INCREASE WITH VERTICAL EFFECTIVE STRESS P_o UP TO A LIMITING DEPTH OF EMBEDMENT, DEPENDING ON THE RELATIVE DENSITY OF THE GRANULAR SOIL AND POSITION OF THE WATER TABLE. BEYOND THIS LIMITING DEPTH (10B ± TO 40B ±) THERE IS VERY LITTLE INCREASE IN END BEARING, AND INCREASE IN SIDE FRICTION IS DIRECTLY PROPORTIONAL TO THE SURFACE AREA OF THE PILE. THEREFORE, IF D IS GREATER THAN 20 B, LIMIT P_o AT THE PILE TIP TO THAT VALUE CORRESPONDING TO D = 20B.

NOTE-2: IF BUILDING LOADS AND SUBSURFACE CONDITION ARE WELL DOCUMENTED IN THE OPINION OF THE ENGINEER, A LESSER FACTOR OF SAFETY CAN BE USED BUT NOT LESS THAN 2.0 PROVIDED PILE CAPACITY IS VERIFIED BY LOAD TEST AND SETTLEMENTS ARE ACCEPTABLE.

FIGURE 1

Load Carrying Capacity of Single Pile in Granular Soils

BEARING CAPACITY FACTORS - Nq

[phi] [*] (DEGREES]	26	28	30	31	32	33	34	35	36	37	38	39	40
$N_{\lceil q \rceil}$ (DRIVEN PILE DISPLACE MENT)	10	15	21	24	29	35	42	50	62	77	86	120	145
$N_{\lceil q \rceil}$ [**] (DRILLED PIERS)	5	8	10	12	14	17	21	25	30	38	43	60	72

EARTH PRESSURE COEFFICIENTS $K_{\lceil}HC_{\rceil}$ AND $K_{\lceil}HT_{\rceil}$

PILE TYPE	$K_{\lceil}HC_{\rceil}$	$K_{\lceil}HT_{\rceil}$
DRIVEN SINGLE H-PILE	0.5 - 1.0	0.3 - 0.5
DRIVEN SINGLE DISPLACEMENT PILE	1.0 - 1.5	0.6 - 1.0
DRIVEN SINGLE DISPLACEMENT TAPERED PILE	1.5 - 2.0	1.0 - 1.3
DRIVEN JETTED PILE	0.4 - 0.9	0.3 - 0.6
DRILLED PILE (LESS THAN 24" DIAMETER)	0.7	0.4

* used 2.0 for timber

FRICTION ANGLE - [delta]

PILE TYPE	[delta]
STEEL	20 deg.
CONCRETE	3/4 [phi]
TIMBER	3/4 [phi]

* used $\delta = \frac{3}{4}\phi$ for timber

[*] LIMIT [phi] TO 280 IF JETTING IS USED

[**] (A) IN CASE A BAILER OR GRAB BUCKET IS USED BELOW GROUND WATER TABLE, CALCULATE END BEARING BASED ON [phi] NOT EXCEEDING 28 deg.

(B) FOR PIERS GREATER THAN 24-INCH DIAMETER, SETTLEMENT RATHER THAN BEARING

CAPACITY USUALLY CONTROLS THE DESIGN. FOR ESTIMATING SETTLEMENT, TAKE 50% OF THE SETTLEMENT FOR AN EQUIVALENT FOOTING RESTING ON THE SURFACE OF COMPARABLE GRANULAR SOILS. (CHAPTER 5, DM-7.1).

FIGURE 1 (continued)
Load Carrying Capacity of Single Pile in Granular Soils

Bowles (1986)

used. The computed point bearing capacity varies widely because there is little agreement on what numerical values to use for the bearing-capacity factors N_i.

We will look at several of the more popular values, but no special recommendation is given for the "best" values since local practice or individual designer preference usually governs the values selected/used.

As previously stated, the soil parameters may be derived from laboratory tests on "undisturbed" samples but more often are unconfined compression data from an SPT or cone penetration test data. In general, the point capacity is computed as

$$P_{pu} = A_p(cN_c'd_cs_c + \eta\bar{q}N_q'd_qs_q + \tfrac{1}{2}\gamma' B_pN_\gamma s_\gamma) \tag{16-6}$$

where A_p = area of pile point effective in bearing, i.e., generally include any "plug." Use actual steel area for point bearing **HP** piles founded on rock, giving simply

$$P_{pu} = A_{steel} \times q_{ult}; \text{ see Sec. 4-16 for rock } q_{ult}$$

c = cohesion of soil beneath pile point (or s_u)

B_p = width of pile point (including "plug")—usually used only when point is enlarged

N_c' = bearing capacity factor for cohesion as previously defined in Chap. 4 but not computed the same way. Use

$$d_c = 1 + 0.4\tan^{-1}(L/B)$$

And when $\phi = 0$; $c = s_u$; $N_c' \approx$ **9.0**

N_q' = bearing capacity factor (may include overburden effects)

$$\text{Use } d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\tan^{-1} L/B$$

The following depth factors are representative:

L/B	d_c	d_q; $\phi = 36°$
10	1.59	$1.36 = 1 + 0.247\tan^{-1} 10$
20	1.61	1.38
40	1.62	1.38
100	1.62	1.39

N_γ' = bearing capacity factor for base width = N_γ since it is not affected by depth

$\bar{q} = \gamma L$ = effective vertical (or overburden) pressure at pile point

η = **1.0** for all except the Vesić (1975*a*) N_i factors where

$$\eta = \frac{1 + 2K_o}{3}$$

K_o = at-rest earth pressure coefficient defined in Chap. 2.

When making point resistance computations, keep in mind that these bearing-capacity factors are based on the initial in situ soil parameters and not on any soil parameters revised to include driving effects. Initially, of course, any revised values would not be known.

Neglecting the N_γ term and making adjustment for pile weight, we may rewrite Eq. (16-6) as follows:

$$P_{pu} = A_p[cN'_c d_c + \eta\bar{q}(N'_q - 1)d_q] \tag{16-6a}$$

For $c = s_u$ and $\phi = 0$, the value of $N'_q = 1$ and

$$P_{pu} = A_p(9s_u) \tag{16-6b}$$

Most designers use N'_q, not $(N'_q - 1)$, for piles (but not piers of Chap. 19) when $\phi > 0$ since the factor reduced by 1 is a substantial refinement not justified by estimated soil parameters. The ultimate point capacity is divided by an SF on the order of 1.5 to 3.

Based on results obtained by Coyle and Castello (1981), who back-computed point capacities of a large number of piles in sand, the Hansen bearing-capacity factors of Table 4-4 can be used together with the shape and depth factors of Table 4-5 with a reliability about as good as any other procedure.

The Terzaghi bearing-capacity equation and factors (Table 4-3) are often used even though they are strictly valid only for $L \leq B$. They seem to give about the same point capacity as the Hansen equation for pile depths on the order of 10 to 20 m—probably because the Hansen $N_q d_q$ term equates to the larger Terzaghi N_q factor.

The depth factor d_c was previously shown to give a limiting value on the order of 1.62; the depth factor d_q depends on both the pile depth ratio L/B and ϕ but from the typical values previously given we see that it can be computed to give a limit on the N'_q term as well. From this we see that using any of Eqs. (16-6) gives an unlimited ultimate point resistance P_u but at a decreasing rate. The point capacity increase at a decreasing rate with increasing L/B seems to be approximately what occurs with actual piles, and for this reason *critical depth* methods such as that of Meyerhof (1976), which adjusts both bearing-capacity factors N'_c, N'_q using a critical depth ratio of L_c/B that was dependent on the ϕ angle of the soil, are not suggested for use.

The Vesić Method

According to Vesić (1975*a*) the bearing-capacity factors N'_i of Eq. (16-6) can be computed based on the following:

$$N'_q = \frac{3}{3 - \sin\phi}\left\{\exp\left[\left(\frac{\pi}{2} - \phi\right)\tan\phi\right]\tan^2\left(45° + \frac{\phi}{2}\right)I_{rr}^{\frac{1.333\sin\phi}{1+\sin\phi}}\right\} \tag{16-7}$$

The reduced rigidity index I_{rr} in this equation is computed using the volumetric strain ϵ_v [see Eq. (*d*) of Sec. 2-14] as

$$I_{rr} = \frac{I_r}{1 + \epsilon_v I_r} \tag{c}$$

The rigidity index I_r is computed using the shear modulus G' and soil cohesion and shear strength s (or τ) as

Bowles (1986)

scatter it does not seem so great as in using other methods, including both the α and λ method, according to Esrig and Kirby (1979).

See Fellenius (2001) for counter

Most authorities agree that f_s in Eq. (16-11) does not increase indefinitely with depth but rather, beyond some critical L/B ratio, increases at an ever-decreasing rate. Bhushan (1982) suggests for large-displacement piles (closed-end pipe, solid concrete, possibly open-end pipe with a plug) that K and β can be estimated as follows:

$$\beta = K \tan\delta = 0.18 + 0.0065D_r$$

and

$$K = 0.50 + 0.008D_r$$

where D_r is the relative density (as a *percent*) previously defined in Chap. 2. We might use SPT correlations (see Table 3-4) to obtain D_r at increasing depths.

Zeitlen and Paikowsky (1982) suggest that the *limiting* f_s is automatically accounted for by the decrease in ϕ' with effective normal confining pressure. To obtain ϕ' at some depth of interest when a reference value of ϕ_o is available as from a triaxial test using an effective normal pressure of $\bar{q}_o$ (and refer to Fig. 2-31) the following equation is suggested

$$\phi' = \phi_o - 5.5 \log \frac{\eta \bar{q}}{q_o} \tag{16-15}$$

where ϕ' = angle of internal friction for design and is computed from the actual effective normal pressure $\eta\bar{q}$ existing at the depth of interest (along pile shaft or point). Use this angle in Eq. (16-6) and Eq. (16-6*a*).

ϕ_o = reference angle of internal friction measured at some effective normal pressure q_o in a laboratory test.

We must also make a decision on what to use for $\tan\delta$. Some persons suggest a maximum for δ on the order of 0.5 to $0.75\phi'$, whereas others routinely use the effective value ϕ'. It has already been pointed out that δ is dependent on the normal pressure at the interface of soil and pile.

Finally, there is a question of what to use for the lateral earth-pressure coefficient K that will give a consistent pile capacity estimation for design within, say, a ± 20 percent error. Several choices for K, given in the text, have been suggested by different authorities; however, although they tend to provide reasonable (after the fact) computations for their authors, for others they have the nasty habit of giving unpredictable results.

It appears that K values are very likely to be both site- and pile-type-specific. Table 16-3 tabulates a number of values of K found from several pile test programs. From this table one can readily see that there is not very good agreement on what to use for K.

It appears that the pile weight was not included in at least some of the pullout tests; and little to no consideration was given to stratification, to changes in the soil parameters with depth, or to effective normal confining pressure. Note, too, that a significant variation in K can be created by the assumption of how much of the load is carried by the point.

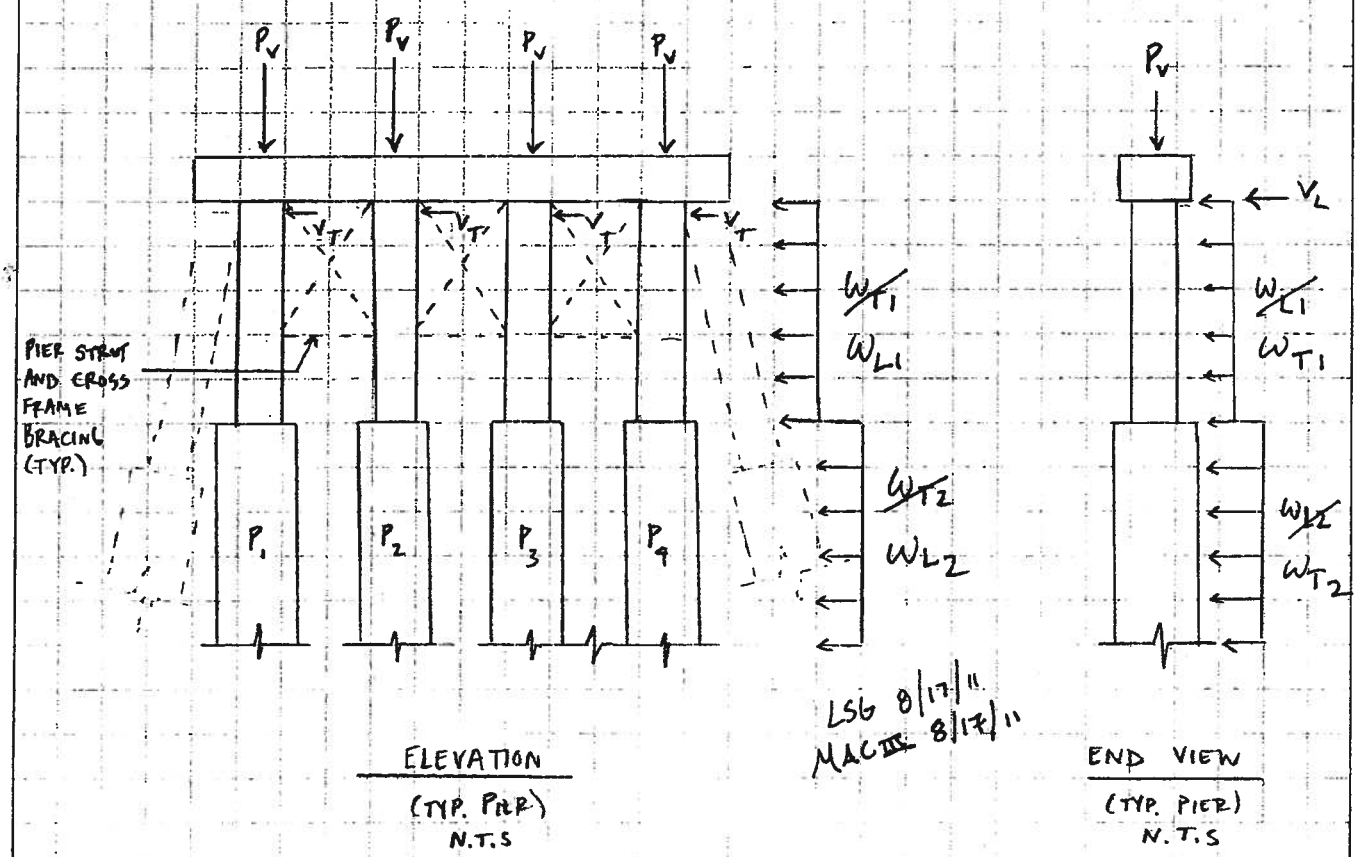
The major error in the foregoing back-computations for K was in obtaining a single value for the full pile depth rather than dividing the pile shaft into lengths of ΔL and using Eq. (16-5*a*) for compression and Eq. (16-5*b*) for tension tests.

5. Pipe Piles – Pile Demand from VHB

VHB **Computations**

Project:	SEWALL'S BRIDGE	Project #	51987
Location:	YORK, ME	Sheet	1 of 2
Calculated by:	LSG	Date:	8/4/11
Checked by:	MAC III	Date:	8/4/11
Title	LOADS TO PIER		

LOADS TO PIER SUMMARY



LSG 8/17/11
MAC III 8/17/11

NOTES:

1. CONSERVATIVLY, LOADS ONLY DISTRIBUTED TO (4) VERTRAL PILES, (NEGLECT BATTERED PILES)
2. SELF WEIGHT OF PILES, PIER CROSS BRACING & STRUTS NOT INCLUDED.
3. ASSUME ABUTMENT 2 PILE BENT IS SIMILAR TO BENT No. 13 LOADS. (CONSERVATIVE)

Transportation
Land Development
Environmental
Services

Vanasse Hangen Brustlin, Inc.

Computations

Project:	Sewall's Bridge	Project #:	51987.00
Location:	York, ME	Sheet:	
Calculated by:	LSG	Date:	8/17/11
Checked by:	MACIII	Date:	8/17/11
Title:	Loads to Pier Summary		

Loads Summary

Pier Bents 1-10, 13

Load Combination	P_v (kips)	At Top of Pile		Applied to Substructure			
				Pipe Piles		Square Columns	
		V_T (kips)	V_L (kips)	w_{T2} (klf)	w_{L2} (klf)	w_{T1} (klf)	w_{L1} (klf)
Strength I (Max)	102.6	0.0	0.7	0.03	0.05	0.01	0.04
Strength I (Min)	11.1	0.0	0.0	0.03	0.05	0.01	0.04
Strength III (Max) Wind at 0°	22.7	1.7	0.0	0.03	0.05	0.03	0.04
Strength III (Min) Wind at 0°	7.0	1.7	0.0	0.03	0.05	0.03	0.04
Strength III (Max) Wind at 60°	19.0	0.6	0.9	0.03	0.05	0.02	0.05
Strength III (Min) Wind at 60°	10.8	0.6	0.9	0.03	0.05	0.02	0.05
Strength V (Max) Wind at 0°	84.4	0.9	0.6	0.03	0.05	0.02	0.04
Strength V (Min) Wind at 0°	10.1	0.9	0.0	0.03	0.05	0.02	0.04
Strength V (Max) Wind at 60°	83.8	0.3	1.0	0.03	0.05	0.02	0.04
Strength V (Min) Wind at 60°	10.8	0.3	0.4	0.03	0.05	0.02	0.04
Service I - Wind at 0°	62.8	0.8	0.4	0.03	0.05	0.02	0.04
Service I - Wind at 60°	62.2	0.3	0.8	0.03	0.05	0.02	0.04

Loads Summary

Pier Bents 11 & 12

Load Combination	P_v (kips)	At Top of Pile		Applied to Substructure			
				Pipe Piles		Square Columns	
		V_T (kips)	V_L (kips)	w_{T2} (klf)	w_{L2} (klf)	w_{T1} (klf)	w_{L1} (klf)
Strength I (Max)	154.5	0.0	0.7	0.03	0.05	0.01	0.04
Strength I (Min)	17.4	0.0	0.0	0.03	0.05	0.01	0.04
Strength III (Max) Wind at 0°	34.3	2.5	0.0	0.03	0.05	0.03	0.04
Strength III (Min) Wind at 0°	11.5	2.5	0.0	0.03	0.05	0.03	0.04
Strength III (Max) Wind at 60°	29.0	0.8	1.2	0.03	0.05	0.02	0.05
Strength III (Min) Wind at 60°	16.8	0.8	1.2	0.03	0.05	0.02	0.05
Strength V (Max) Wind at 0°	127.4	1.5	0.6	0.03	0.05	0.02	0.04
Strength V (Min) Wind at 0°	15.7	1.5	0.0	0.03	0.05	0.02	0.04
Strength V (Max) Wind at 60°	126.2	0.5	1.2	0.03	0.05	0.02	0.04
Strength V (Min) Wind at 60°	16.8	0.5	0.6	0.03	0.05	0.02	0.04
Service I - Wind at 0°	94.9	1.3	0.4	0.03	0.05	0.02	0.04
Service I - Wind at 60°	93.8	0.4	1.0	0.03	0.05	0.01	0.04

6. Pipe Piles – Geotechnical Resistance

Res. Factor 0.25 AASHTO & NCHRP

Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Dr %	K	β	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	s'v used (psf)	α	N'q	Unit Tip Resistance (psf)	Limiting Tip Resistance (psf)	Tip Resistance Used (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Factored Resistance (kips)
								Side Resistance - Beta Method - Bhushan (1982)					Tip Resistance - Thurman Method									
-9	0	Harbor Bottom Deposit	105	0	1	28	25	0.70	0.34	0	30	0	0	0.26	22	0	20000	0	30	0	0	0
-10	1	Harbor Bottom Deposit	105	42	1	28	24	0.69	0.33	0	30	0	42	0.26	22	240	20000	240	30	0	0	0
-11	2	Harbor Bottom Deposit	105	84	1	28	22	0.68	0.33	0	30	0	84	0.26	22	480	20000	480	30	0	0	0
-12	3	Harbor Bottom Deposit	105	126	1	28	21	0.67	0.32	0	30	0	126	0.26	22	721	20000	721	30	0	0	0
-13	4	Harbor Bottom Deposit	105	168	1	28	20	0.66	0.31	0	30	0	168	0.26	22	961	20000	961	30	0	0	0
-14	5	Harbor Bottom Deposit	105	210	1	28	20	0.66	0.31	0	30	0	210	0.26	22	1201	20000	1201	30	0	0	0
-15	6	Harbor Bottom Deposit	105	252	1	28	19	0.65	0.30	0	30	0	252	0.26	22	1441	20000	1441	30	0	0	0
-16	7	Marine Deposit	120	309	29	30	97	1.28	0.81	0	30	0	309	0.3	30	2781	20000	2781	30	0	0	0
-17	8	Marine Deposit	120	366	29	30	93	1.25	0.79	0	30	0	366	0.3	30	3294	20000	3294	30	0	0	0
-18	9	Marine Deposit	120	423	29	30	90	1.22	0.76	0	30	0	423	0.3	30	3807	20000	3807	30	0	0	0
-19	10	Marine Deposit	120	480	29	30	87	1.19	0.74	0	30	0	480	0.3	30	4320	20000	4320	30	0	0	0
-20	11	Marine Deposit	120	537	29	30	84	1.17	0.73	389	30	2	537	0.3	30	4833	20000	4833	30	24	25	6
-21	12	Marine Deposit	120	594	29	30	81	1.15	0.71	421	30	5	594	0.3	30	5346	20000	5346	30	26	31	8
-22	13	Marine Deposit	120	651	29	30	79	1.13	0.69	451	30	8	651	0.3	30	5859	20000	5859	30	29	37	9
-23	14	Marine Deposit	120	708	29	30	77	1.11	0.68	480	30	12	708	0.3	30	6372	20000	6372	30	31	43	11
-24	15	Marine Deposit	120	765	29	30	75	1.10	0.67	509	30	16	765	0.3	30	6885	20000	6885	30	34	49	12
-25	16	Marine Deposit	120	822	29	30	73	1.08	0.65	537	30	20	822	0.3	30	7398	20000	7398	30	36	56	14
-26	17	Marine Deposit	120	879	29	30	71	1.07	0.64	565	30	24	879	0.3	30	7911	20000	7911	30	39	63	16
-27	18	Glacial Till	130	946	100	38	129	1.53	1.02	961	30	30	946	0.36	105	35759	260000	35759	30	176	206	51
-28	19	Glacial Till	130	1013	100	38	125	1.50	0.99	1008	30	38	1013	0.36	105	38291	260000	38291	30	188	226	56
-29	20	Glacial Till	130	1080	100	38	122	1.48	0.98	1054	30	46	1080	0.36	105	40824	260000	40824	30	200	246	62
-30	21	Glacial Till	130	1147	100	38	120	1.46	0.96	1098	30	54	1147	0.36	105	43357	260000	43357	30	213	267	67
-31	22	Glacial Till	130	1214	100	38	117	1.44	0.94	1142	30	63	1214	0.36	105	45889	260000	45889	30	225	288	72
-32	23	Glacial Till	130	1281	100	38	115	1.42	0.92	1184	30	72	1281	0.36	105	48422	260000	48422	30	238	310	78
		Bedrock						see Bowles for K and beta					see AASHTO-LRFD for Thurman Method parameters									

Need 105 Kips ∴ bear on bedrock. where thin overburden.

Bent 1 Geotechnical Resistance

LRFD - Beta / Thurman method

Computed by: JLL
Checked by: EFW
12/21/11

							Side Resistance - Nordlund Method							Tip Resistance - Thurman Method										Res. Factor 0.35 NCHRP
Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Kd	Cf	δ	βeff	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	s'v used (psf)	α	N'q	Unit Tip Resistance (psf)	Limiting Tip Resistance (psf)	Tip Resistance Used (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Factored Resistance (kips)
-9	0	Harbor Bottom Deposit	105	0	1	28	0.89	1.00	27	0.40	0	30	0	0	0.26	22	0	20000	0	30	0	0	0
-10	1	Harbor Bottom Deposit	105	42	1	28	0.89	1.00	27	0.40	17	30	0	42	0.26	22	240	20000	240	30	0	0	0
-11	2	Harbor Bottom Deposit	105	84	1	28	0.89	1.00	27	0.40	33	30	0	84	0.26	22	480	20000	480	30	0	0	0
-12	3	Harbor Bottom Deposit	105	126	1	28	0.89	1.00	27	0.40	50	30	0	126	0.26	22	721	20000	721	30	0	0	0
-13	4	Harbor Bottom Deposit	105	168	1	28	0.89	1.00	27	0.40	67	30	0	168	0.26	22	961	20000	961	30	0	0	0
-14	5	Harbor Bottom Deposit	105	210	1	28	0.89	1.00	27	0.40	84	30	0	210	0.26	22	1201	20000	1201	30	0	0	0
-15	6	Harbor Bottom Deposit	105	252	1	28	0.89	1.00	27	0.40	100	30	0	252	0.26	22	1441	20000	1441	30	0	0	0
-16	7	Marine Deposit	120	309	29	30	0.98	1.00	29	0.47	144	30	0	309	0.30	30	2735	20000	2735	30	0	0	0
-17	8	Marine Deposit	120	366	29	30	0.98	1.00	29	0.47	171	30	0	366	0.30	30	3239	20000	3239	30	0	0	0
-18	9	Marine Deposit	120	423	29	30	0.98	1.00	29	0.47	198	30	0	423	0.30	30	3744	20000	3744	30	0	0	0
-19	10	Marine Deposit	120	480	29	30	0.98	1.00	29	0.47	224	30	0	480	0.30	30	4248	20000	4248	30	0	0	0
-20	11	Marine Deposit	120	537	29	30	0.98	1.00	29	0.47	251	30	2	537	0.30	30	4752	20000	4752	30	23	25	9
-21	12	Marine Deposit	120	594	29	30	0.98	1.00	29	0.47	278	30	4	594	0.30	30	5257	20000	5257	30	26	30	10
-22	13	Marine Deposit	120	651	29	30	0.98	1.00	29	0.47	304	30	6	651	0.30	30	5761	20000	5761	30	28	35	12
-23	14	Marine Deposit	120	708	29	30	0.98	1.00	29	0.47	331	30	9	708	0.30	30	6266	20000	6266	30	31	39	14
-24	15	Marine Deposit	120	765	29	30	0.98	1.00	29	0.47	358	30	11	765	0.30	30	6770	20000	6770	30	33	45	16
-25	16	Marine Deposit	120	822	29	30	0.98	1.00	29	0.47	384	30	14	822	0.30	30	7275	20000	7275	30	36	50	18
-26	17	Marine Deposit	120	879	29	30	0.98	1.00	29	0.47	411	30	17	879	0.30	30	7779	20000	7779	30	38	56	19
-27	18	Glacial Till	130	946	100	38	1.91	1.00	36	1.13	1065	30	23	946	0.36	105	35759	260000	35759	30	176	199	70
-28	19	Glacial Till	130	1013	100	38	1.91	1.00	36	1.13	1140	30	32	1013	0.36	105	38291	260000	38291	30	188	220	77
-29	20	Glacial Till	130	1080	100	38	1.91	1.00	36	1.13	1215	30	41	1080	0.36	105	40824	260000	40824	30	200	242	85
-30	21	Glacial Till	130	1147	100	38	1.91	1.00	36	1.13	1291	30	51	1147	0.36	105	43357	260000	43357	30	213	264	92
-31	22	Glacial Till	130	1214	100	38	1.91	1.00	36	1.13	1366	30	61	1214	0.36	105	45889	260000	45889	30	225	287	100
-32	23	Glacial Till	130	1281	100	38	1.91	1.00	36	1.13	1442	30	72	1281	0.36	105	48422	260000	48422	30	238	310	109

see AASHTO-LRFD for Nordlund/Thurman parameters

Bent 1 Geotechnical Resistance

LRFD - Nordlund / Thurman Method

Computed by : JLL
Checked by : EFW
12/21/11

Need 105 kips
∴ may achieve capacity 5 to 6 ft into glacial till. where thin overburden.

									Side Resistance - Beta Method - Bhushan (1982)					Tip Resistance - Thurman Method									
																							Res. Factor 0.25 AASHTO & NCHRP
Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Dr %	K	β	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	s'v used (psf)	α	N'q	Unit Tip Resistance (psf)	Limiting Tip Resistance (psf)	Tip Resistance Used (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Factored Resistance (kips)	
-19	0	Harbor Bottom Deposit	105	0	1	28	25	0.70	0.34	0	30	0	0	0.52	22	0	20000	0	30	0	0	0	
-20	1	Harbor Bottom Deposit	105	42	1	28	24	0.69	0.33	14	30	0	42	0.52	22	480	20000	480	30	0	0	0	
-21	2	Harbor Bottom Deposit	105	84	1	28	22	0.68	0.33	27	30	0	84	0.52	22	961	20000	961	30	0	0	0	
-22	3	Harbor Bottom Deposit	105	126	1	28	21	0.67	0.32	40	30	0	126	0.52	22	1441	20000	1441	30	0	0	0	
-23	4	Harbor Bottom Deposit	105	168	1	28	20	0.66	0.31	53	30	0	168	0.52	22	1922	20000	1922	30	0	0	0	
-24	5	Harbor Bottom Deposit	105	210	1	28	20	0.66	0.31	65	30	0	210	0.52	22	2402	20000	2402	30	0	0	0	
-25	6	Harbor Bottom Deposit	105	252	1	28	19	0.65	0.30	76	30	0	252	0.52	22	2883	20000	2883	30	0	0	0	
-26	7	Harbor Bottom Deposit	105	294	1	28	18	0.65	0.30	88	30	0	294	0.52	22	3363	20000	3363	30	0	0	0	
-27	8	Harbor Bottom Deposit	105	336	1	28	18	0.64	0.30	99	30	0	336	0.52	22	3844	20000	3844	30	0	0	0	
-28	9	Harbor Bottom Deposit	105	378	1	28	17	0.64	0.29	110	30	0	378	0.52	22	4324	20000	4324	30	0	0	0	
-29	10	Marine Deposit	120	435	29	30	89	1.21	0.76	331	30	0	435	0.59	30	7700	20000	7700	30	0	0	0	
-30	11	Marine Deposit	120	492	29	30	86	1.19	0.74	364	30	3	492	0.59	30	8708	20000	8708	30	43	45	11	
-31	12	Marine Deposit	120	549	29	30	83	1.17	0.72	396	30	6	549	0.59	30	9717	20000	9717	30	48	53	13	
-32	13	Marine Deposit	120	606	29	30	81	1.15	0.70	427	30	9	606	0.59	30	10726	20000	10726	30	53	62	15	
-33	14	Marine Deposit	120	663	29	30	78	1.13	0.69	457	30	12	663	0.59	30	11735	20000	11735	30	58	70	18	
-34	15	Marine Deposit	120	720	29	30	76	1.11	0.68	487	30	16	720	0.59	30	12744	20000	12744	30	63	79	20	
-35	16	Marine Deposit	120	777	29	30	74	1.09	0.66	515	30	20	777	0.59	30	13753	20000	13753	30	68	88	22	
-36	17	Marine Deposit	120	834	29	30	72	1.08	0.65	543	30	24	834	0.59	30	14762	20000	14762	30	72	97	24	
-37	18	Marine Deposit	120	891	29	30	71	1.07	0.64	570	30	29	891	0.59	30	15771	20000	15771	30	77	106	26	
-38	19	Marine Deposit	120	948	29	30	69	1.05	0.63	597	30	33	948	0.59	30	16780	20000	16780	30	82	116	29	
-39	20	Marine Deposit	120	1005	29	30	68	1.04	0.62	623	30	38	1005	0.59	30	17789	20000	17789	30	87	125	31	
-40	21	Marine Deposit	120	1062	29	30	66	1.03	0.61	649	30	43	1062	0.59	30	18797	20000	18797	30	92	135	34	
-41	22	Marine Deposit	120	1119	29	30	65	1.02	0.60	674	30	48	1119	0.59	30	19806	20000	19806	30	97	145	36	
-42	23	Marine Deposit	120	1176	29	30	64	1.01	0.59	699	30	54	1176	0.59	30	20815	20000	20000	30	98	152	38	
-43	24	Marine Deposit	120	1233	29	30	63	1.00	0.59	724	30	59	1233	0.59	30	21824	20000	20000	30	98	157	39	
-44	25	Marine Deposit	120	1290	29	30	62	0.99	0.58	748	30	65	1290	0.59	30	22833	20000	20000	30	98	163	41	
-45	26	Marine Deposit	120	1347	29	30	60	0.98	0.57	772	30	71	1347	0.59	30	23842	20000	20000	30	98	169	42	
-46	27	Marine Deposit	120	1404	29	30	59	0.98	0.57	795	30	77	1404	0.59	30	24851	20000	20000	30	98	175	44	
-47	28	Marine Deposit	120	1461	29	30	59	0.97	0.56	819	30	83	1461	0.59	30	25860	20000	20000	30	98	182	45	
-48	29	Marine Deposit	120	1518	29	30	58	0.96	0.55	842	30	90	1518	0.59	30	26869	20000	20000	30	98	188	47	
-49	30	Marine Deposit	120	1575	29	30	57	0.95	0.55	865	30	97	1575	0.59	30	27878	20000	20000	30	98	195	49	
-50	31	Marine Deposit	120	1632	29	30	56	0.95	0.54	887	30	103	1632	0.59	30	28886	20000	20000	30	98	202	50	
-51	32	Marine Deposit	120	1689	29	30	55	0.94	0.54	909	30	111	1689	0.59	30	29895	20000	20000	30	98	209	52	
-52	33	Marine Deposit	120	1746	29	30	54	0.94	0.53	931	30	118	1746	0.59	30	30904	20000	20000	30	98	216	54	
-53	34	Marine Deposit	120	1803	29	30	54	0.93	0.53	953	30	125	1803	0.59	30	31913	20000	20000	30	98	223	56	
-54	35	Marine Deposit	120	1860	29	30	53	0.92	0.52	975	30	133	1860	0.59	30	32922	20000	20000	30	98	231	58	
-55	36	Glacial Till	130	1927	100	38	97	1.27	0.81	1560	30	143	1927	0.72	105	145681	260000	145681	30	715	858	214	
-56	37	Glacial Till	130	1994	100	38	95	1.26	0.80	1596	30	155	1994	0.72	105	150746	260000	150746	30	740	895	224	
-57	38	Glacial Till	130	2061	100	38	94	1.25	0.79	1632	30	168	2061	0.72	105	155812	260000	155812	30	765	933	233	
-58	39	Glacial Till	130	2128	100	38	93	1.24	0.78	1667	30	181	2128	0.72	105	160877	260000	160877	30	790	970	243	
-59	40	Glacial Till	130	2195	100	38	92	1.23	0.78	1702	30	194	2195	0.72	105	165942	260000	165942	30	815	1009	252	
-60	41	Glacial Till	130	2262	100	38	90	1.22	0.77	1737	30	207	2262	0.72	105	171007	260000	171007	30	839	1047	262	
-61	42	Glacial Till	130	2329	100	38	89	1.21	0.76	1771	30	221	2329	0.72	105	176072	260000	176072	30	864	1086	271	
-62	43	Glacial Till	130	2396	100	38	88	1.21	0.75	1805	30	235	2396	0.72	105	181138	260000	181138	30	889	1124	281	
-63	44	Glacial Till	130	2463	100	38	87	1.20	0.75	1838	30	250	2463	0.72	105	186203	260000	186203	30	914	1164	291	
-64	45	Glacial Till	130	2530	100	38	86	1.19	0.74	1871	30	264	2530	0.72	105	191268	260000	191268	30	939	1203	301	
-65	46	Glacial Till	130	2597	100	38	85	1.18	0.73	1904	30	279	2597	0.72	105	196333	260000	196333	30	964	1243	311	

Bent 7 Geotechnical Resistance

LRFD- Beta/Thurman Method

Computed by: JLL
Checked by: EFW
12/21/11

Need 105 kips to 155 kips (Bent 11:12) ∴ require minimal embedment into glacial till where thick overburden

Res. Factor 0.35 NCHRP

							Side Resistance - Nordlund Method							Tip Resistance - Thurman Method										
Elevation (ft)	Depth (ft)	Description	Unit Weight (pcf)	Effective Vertical Stress (psf)	SPT N	phi (deg)	Kd	Cf	δ	βeff	fs (psf)	Pile Diameter (in)	Cumulative Nominal Friction Resistance (kips)	s'v used (psf)	α	N'q	Unit Tip Resistance (psf)	Limiting Tip Resistance (psf)	Tip Resistance Used (psf)	Tip Diameter (in)	Nominal Tip Resistance (kips)	Total Nominal Resistance (kips)	Factored Resistance (kips)
-19	0	Harbor Bottom Deposit	105	0	1	28	0.89	1.00	27	0.40	0	30	0	0	0.52	22	0	20000	0	30	0	0	0
-20	1	Harbor Bottom Deposit	105	42	1	28	0.89	1.00	27	0.40	17	30	0	42	0.52	22	480	20000	480	30	0	0	0
-21	2	Harbor Bottom Deposit	105	84	1	28	0.89	1.00	27	0.40	33	30	0	84	0.52	22	961	20000	961	30	0	0	0
-22	3	Harbor Bottom Deposit	105	126	1	28	0.89	1.00	27	0.40	50	30	0	126	0.52	22	1441	20000	1441	30	0	0	0
-23	4	Harbor Bottom Deposit	105	168	1	28	0.89	1.00	27	0.40	67	30	0	168	0.52	22	1922	20000	1922	30	0	0	0
-24	5	Harbor Bottom Deposit	105	210	1	28	0.89	1.00	27	0.40	84	30	0	210	0.52	22	2402	20000	2402	30	0	0	0
-25	6	Harbor Bottom Deposit	105	252	1	28	0.89	1.00	27	0.40	100	30	0	252	0.52	22	2883	20000	2883	30	0	0	0
-26	7	Harbor Bottom Deposit	105	294	1	28	0.89	1.00	27	0.40	117	30	0	294	0.52	22	3363	20000	3363	30	0	0	0
-27	8	Harbor Bottom Deposit	105	336	1	28	0.89	1.00	27	0.40	134	30	0	336	0.52	22	3844	20000	3844	30	0	0	0
-28	9	Harbor Bottom Deposit	105	378	1	28	0.89	1.00	27	0.40	151	30	0	378	0.52	22	4324	20000	4324	30	0	0	0
-29	10	Marine Deposit	120	435	29	30	0.98	1.00	29	0.47	203	30	0	435	0.59	30	7700	20000	7700	30	0	0	0
-30	11	Marine Deposit	120	492	29	30	0.98	1.00	29	0.47	230	30	2	492	0.59	30	8708	20000	8708	30	43	44	16
-31	12	Marine Deposit	120	549	29	30	0.98	1.00	29	0.47	257	30	4	549	0.59	30	9717	20000	9717	30	48	51	18
-32	13	Marine Deposit	120	606	29	30	0.98	1.00	29	0.47	283	30	6	606	0.59	30	10726	20000	10726	30	53	58	20
-33	14	Marine Deposit	120	663	29	30	0.98	1.00	29	0.47	310	30	8	663	0.59	30	11735	20000	11735	30	58	66	23
-34	15	Marine Deposit	120	720	29	30	0.98	1.00	29	0.47	337	30	11	720	0.59	30	12744	20000	12744	30	63	73	26
-35	16	Marine Deposit	120	777	29	30	0.98	1.00	29	0.47	363	30	13	777	0.59	30	13753	20000	13753	30	68	81	28
-36	17	Marine Deposit	120	834	29	30	0.98	1.00	29	0.47	390	30	16	834	0.59	30	14762	20000	14762	30	72	89	31
-37	18	Marine Deposit	120	891	29	30	0.98	1.00	29	0.47	417	30	19	891	0.59	30	15771	20000	15771	30	77	97	34
-38	19	Marine Deposit	120	948	29	30	0.98	1.00	29	0.47	443	30	23	948	0.59	30	16780	20000	16780	30	82	105	37
-39	20	Marine Deposit	120	1005	29	30	0.98	1.00	29	0.47	470	30	26	1005	0.59	30	17789	20000	17789	30	87	114	40
-40	21	Marine Deposit	120	1062	29	30	0.98	1.00	29	0.47	497	30	30	1062	0.59	30	18797	20000	18797	30	92	123	43
-41	22	Marine Deposit	120	1119	29	30	0.98	1.00	29	0.47	523	30	34	1119	0.59	30	19806	20000	19806	30	97	131	46
-42	23	Marine Deposit	120	1176	29	30	0.98	1.00	29	0.47	550	30	38	1176	0.59	30	20815	20000	20000	30	98	137	48
-43	24	Marine Deposit	120	1233	29	30	0.98	1.00	29	0.47	577	30	43	1233	0.59	30	21824	20000	20000	30	98	141	49
-44	25	Marine Deposit	120	1290	29	30	0.98	1.00	29	0.47	603	30	48	1290	0.59	30	22833	20000	20000	30	98	146	51
-45	26	Marine Deposit	120	1347	29	30	0.98	1.00	29	0.47	630	30	52	1347	0.59	30	23842	20000	20000	30	98	151	53
-46	27	Marine Deposit	120	1404	29	30	0.98	1.00	29	0.47	657	30	57	1404	0.59	30	24851	20000	20000	30	98	156	54
-47	28	Marine Deposit	120	1461	29	30	0.98	1.00	29	0.47	683	30	63	1461	0.59	30	25860	20000	20000	30	98	161	56
-48	29	Marine Deposit	120	1518	29	30	0.98	1.00	29	0.47	710	30	68	1518	0.59	30	26869	20000	20000	30	98	166	58
-49	30	Marine Deposit	120	1575	29	30	0.98	1.00	29	0.47	736	30	74	1575	0.59	30	27878	20000	20000	30	98	172	60
-50	31	Marine Deposit	120	1632	29	30	0.98	1.00	29	0.47	763	30	80	1632	0.59	30	28886	20000	20000	30	98	178	62
-51	32	Marine Deposit	120	1689	29	30	0.98	1.00	29	0.47	790	30	86	1689	0.59	30	29895	20000	20000	30	98	184	64
-52	33	Marine Deposit	120	1746	29	30	0.98	1.00	29	0.47	816	30	92	1746	0.59	30	30904	20000	20000	30	98	190	67
-53	34	Marine Deposit	120	1803	29	30	0.98	1.00	29	0.47	843	30	99	1803	0.59	30	31913	20000	20000	30	98	197	69
-54	35	Marine Deposit	120	1860	29	30	0.98	1.00	29	0.47	870	30	105	1860	0.59	30	32922	20000	20000	30	98	204	71
-55	36	Glacial Till	130	1927	100	38	1.91	1.00	36	1.13	2169	30	117	1927	0.72	105	145681	260000	145681	30	715	832	291
-56	37	Glacial Till	130	1994	100	38	1.91	1.00	36	1.13	2244	30	135	1994	0.72	105	150746	260000	150746	30	740	875	306
-57	38	Glacial Till	130	2061	100	38	1.91	1.00	36	1.13	2319	30	153	2061	0.72	105	155812	260000	155812	30	765	917	321
-58	39	Glacial Till	130	2128	100	38	1.91	1.00	36	1.13	2395	30	171	2128	0.72	105	160877	260000	160877	30	790	961	336
-59	40	Glacial Till	130	2195	100	38	1.91	1.00	36	1.13	2470	30	190	2195	0.72	105	165942	260000	165942	30	815	1005	352
-60	41	Glacial Till	130	2262	100	38	1.91	1.00	36	1.13	2546	30	210	2262	0.72	105	171007	260000	171007	30	839	1049	367
-61	42	Glacial Till	130	2329	100	38	1.91	1.00	36	1.13	2621	30	230	2329	0.72	105	176072	260000	176072	30	864	1094	383
-62	43	Glacial Till	130	2396	100	38	1.91	1.00	36	1.13	2696	30	251	2396	0.72	105	181138	260000	181138	30	889	1140	399
-63	44	Glacial Till	130	2463	100	38	1.91	1.00	36	1.13	2772	30	272	2463	0.72	105	186203	260000	186203	30	914	1187	415
-64	45	Glacial Till	130	2530	100	38	1.91	1.00	36	1.13	2847	30	295	2530	0.72	105	191268	260000	191268	30	939	1233	432
-65	46	Glacial Till	130	2597	100	38	1.91	1.00	36	1.13	2923	30	317	2597	0.72	105	196333	260000	196333	30	964	1281	448

Bent 7 Geotechnical Resistance
LRFD - Nordlund/Thurman Method

Computed by: JLL
Checked by: EFW
12/21/11

Need 105 kips to 155 kips (Bent 11 & 12) ∴ require minimal embedment into glacial till where thick overburden

LRFD Resistance Factor			
Static Method	AASHTO	NCHRP	Remarks
Beta/Thurman	0.25	0.25	NCHRP factor is for pipe pile in mixed soils
Nordlund Thurman	0.45	0.35	NCHRP factor is for pipe pile in mixed soils

Assumptions for End-Bearing Component in Open-Ended Piles
- Assume that plug unit resistance is 25% of annulus unit resistance.
- Above assumptions based on findings in Paik et al. (2003)

NCHRP Report 507

TABLE 25 Recommended resistance and efficiency factors for static analyses of driven piles

Pile Type	Soil Type	Design Method	Resistance Factor ϕ		ϕ/λ	
			Redundant	Non-redundant	Redundant	Non-redundant
Concrete Pile	Mixed	SPT-97 mob	0.70	0.50	0.40	0.29
	Clay	α-API	0.50	0.40	0.67	0.55
		λ-Method			0.63	0.55
	Sand	β-Method			0.46	0.34
		SPT-97 mob			0.42	0.31
	Mixed	FHWA CPT			0.60	0.48
		β-Method/Thurman			0.51	0.39
		αTomlinson/Nordlund/Thurman	0.40	0.30	0.41	0.30
	Sand	Nordlund			0.42	0.31
	Clay	α-Tomlinson	0.35	0.25	0.41	0.30
	Mixed	α-API/Nordlund/Thurman			0.41	0.30
	Sand	Meyerhof	0.20	0.15	0.32	0.22
Pipe Pile	Sand	SPT-97 mob	0.55	0.45	0.38	0.28
		Nordlund			0.38	0.27
	Mixed	SPT-97 mob	0.40	0.30	0.51	0.40
		α-API/Nordlund/Thurman	0.35	0.25	0.44	0.31
	Sand	β-Method			0.31	0.21
	Clay	α-API	0.30	0.20	0.36	0.26
	Sand	Meyerhof			0.33	0.23
	Mixed	αTomlinson/Nordlund/Thurman	0.25	0.15	0.32	0.23
		β-Method/Thurman			0.41	0.30
	Clay	α-Tomlinson			0.40	0.29
		λ-Method			0.36	0.25
H Piles	Mixed	SPT-97 mob	0.55	0.45	0.45	0.33
	Sand	SPT-97 mob			0.46	0.35
		Nordlund	0.45	0.35	0.49	0.37
		Meyerhof			0.51	0.39
	Clay	α-API			0.48	0.37
		α-Tomlinson	0.40	0.30	0.49	0.37
		λ-Method			0.50	0.39
	Mixed	α-API/Nordlund/Thurman	0.35	0.25	0.45	0.34
		αTomlinson/Nordlund/Thurman	0.30		0.51	0.39
	Sand	β-Method			0.39	0.28
	Mixed	β-Method/Thurman	0.20	0.15	0.42	0.31

used this for N/ method

used th for β/ Thurm method

Notes: ϕ/λ = efficiency factor, evaluating the relative economic performance of each method (higher ratios indicate a more economical solution).
λ = bias = K_{ss} = Mean of measured over predicted.
ϕ/λ values relate to the exact calculated ϕ and λ and not to the assigned ϕ values in the table
Redundant = Five piles or more under one pile cap ($\beta = 2.33$ $p_f = 1.0\%$)
Non-Redundant = Four or fewer piles under one pile cap ($\beta = 3.0$ $p_f = 0.1\%$)

TABLE 26 Recommended resistance factors for static analysis of nontapered driven piles under pullout

Soil Type	Design Method	Pile Type	ϕ (resistance factor)	
			Redundant $\beta = 2.33$	Non-Redundant $\beta = 3.00$
Clay	α-API, λ αTomlinson	H, Pipe, PPC	0.25[1]	0.20
Sand	β	H	0.15	0.10
		Pipe, PPC	0.25	0.20
	SPT-97	H, Pipe, PPC	0.25	0.20
Mixed	α-API/Nordlund	H, Pipe, PPC	0.20	0.15

[1]Higher values may be applicable for PPC piles but no sufficient data were available to support this.

Table 10.5.5.2.3-1 Resistance Factors for Driven Piles (continued).

Condition/Resistance Determination Method		Resistance Factor
Nominal Resistance of Single Pile in Axial Compression—Static Analysis Methods, φ_{stat}	Skin Friction and End Bearing: Clay and Mixed Soils	
	α-method (*Tomlinson, 1987*; *Skempton, 1951*)	0.35
	β-method (*Esrig & Kirby, 1979*; *Skempton, 1951*)	0.25
	λ-method (*Vijayvergiya & Focht, 1972*; *Skempton, 1951*)	0.40
	Skin Friction and End Bearing: Sand	
	Nordlund/Thurman Method (*Hannigan et al., 2005*)	0.45
	SPT-method (Meyerhof)	0.30
	CPT-method (Schmertmann)	0.50
	End bearing in rock (*Canadian Geotech. Society, 1985*)	0.45
Block Failure, φ_{bl}	Clay	0.60
Uplift Resistance of Single Piles, φ_{up}	Nordlund Method	0.35
	α-method	0.25
	β-method	0.20
	λ-method	0.30
	SPT-method	0.25
	CPT-method	0.40
	Load test	0.60
Group Uplift Resistance, φ_{ug}	Sand and clay	0.50
Horizontal Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles — See the provisions of Article 6.5.4.2	
	Concrete piles — See the provisions of Article 5.5.4.2.1	
	Timber piles — See the provisions of Article 8.5.2.2 and 8.5.2.3	
Pile Drivability Analysis, φ_{da}	Steel piles — See the provisions of Article 6.5.4.2	
	Concrete piles — See the provisions of Article 5.5.4.2.1	
	Timber piles — See the provisions of Article 8.5.2.2	
	In all three Articles identified above, use φ identified as "resistance during pile driving"	

used for β-me

AASH LRFD resist fact (but used NCHR resist fact

Table 10.5.5.2.3-2 Relationship between Number of Static Load Tests Conducted per Site and φ (after *Paikowsky et al., 2004*).

Number of Static Load Tests per Site	Resistance Factor, φ — Site Variability[a]		
	Low[a]	Medium[a]	High[a]
1	0.80	0.70	0.55
2	0.90	0.75	0.65
3	0.90	0.85	0.75
≥4	0.90	0.90	0.80

a See commentary.

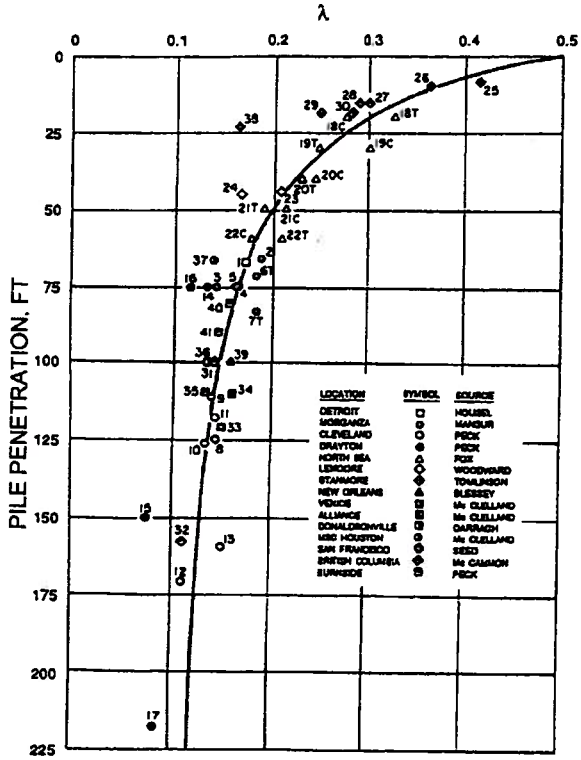


Figure 10.7.3.8.6d-1 λ Coefficient for Driven Pipe Piles after Vijayvergiya and Focht (*1972*).

10.7.3.8.6e Tip Resistance in Cohesive Soils

The nominal unit tip resistance of piles in saturated clay, in ksf, shall be taken as:

$$q_p = 9S_u \qquad (10.7.3.8.6e\text{-}1)$$

where:

S_u = undrained shear strength of the clay near the pile base (ksf)

10.7.3.8.6f Nordlund/Thurman Method in Cohesionless Soils

This effective stress method should be applied only to sands and nonplastic silts. The nominal unit side resistance, q_s, for this method, in ksf, shall be taken as:

$$q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta+\omega)}{\cos\omega} \qquad (10.7.3.8.6f\text{-}1)$$

where:

K_δ = coefficient of lateral earth pressure at mid-point of soil layer under consideration from Figures 1 through 4 (dim.)

C10.7.3.8.6f

Detailed design procedures for the Nordlund/Thurman method are provided in Hannigan et al., (*2005*). This method was derived based on load test data for piles in sand. In practice, it has been used for gravelly soils as well.

The effective overburden stress is not limited in Eq. 1.

For H-piles, the perimeter or "box" area should generally be used to compute the surface area of the pile side.

C_F = correction factor for K_δ when $\delta \neq \phi_f$, from Figure 5

σ'_v = effective overburden stress at midpoint of soil layer under consideration (ksf)

δ = friction angle between pile and soil obtained from Figure 6 (deg.)

ω = angle of pile taper from vertical (deg.)

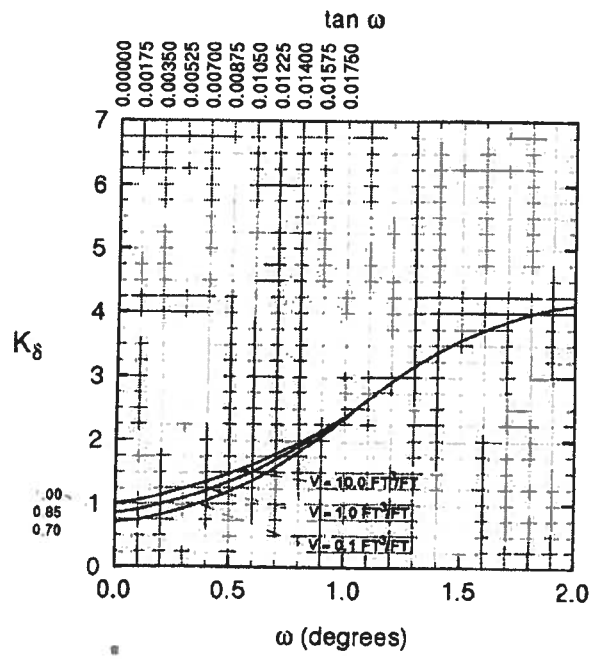


Figure 10.7.3.8.6f-1 Design Curve for Evaluating K_δ for Piles where $\phi_f = 25°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

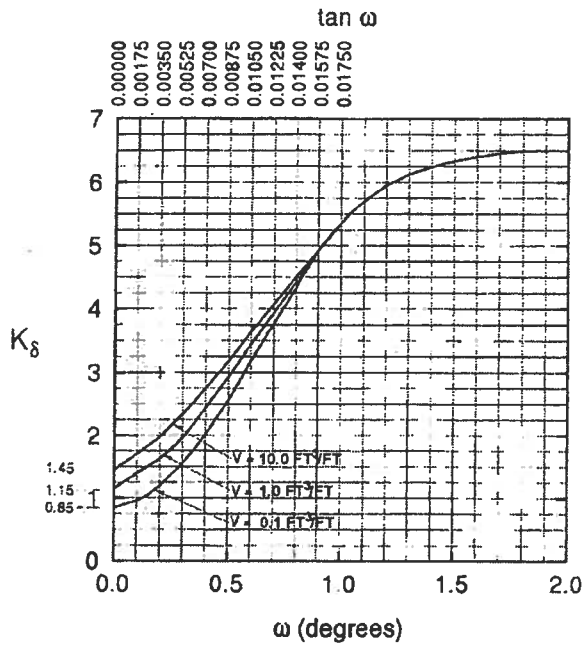


Figure 10.7.3.8.6f-2 Design Curve for Evaluating K_δ for Piles where $\phi_f = 30°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

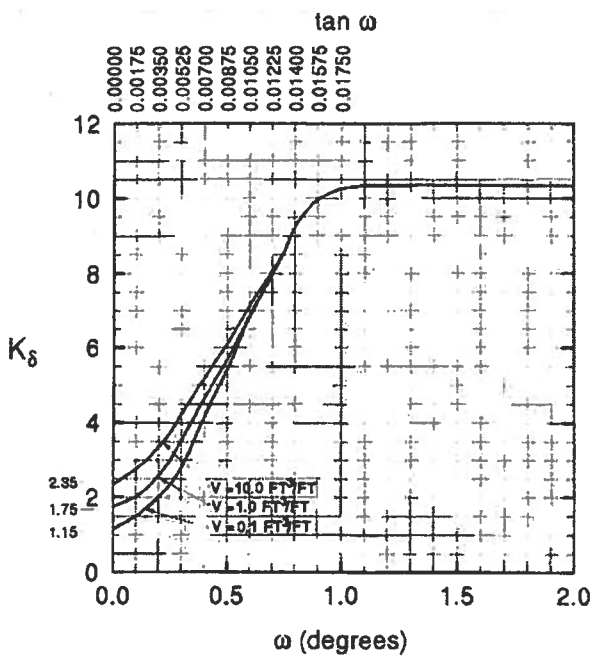


Figure 10.7.3.8.6f-3 Design Curve for Evaluating K_δ for Piles where $\phi_f = 35°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

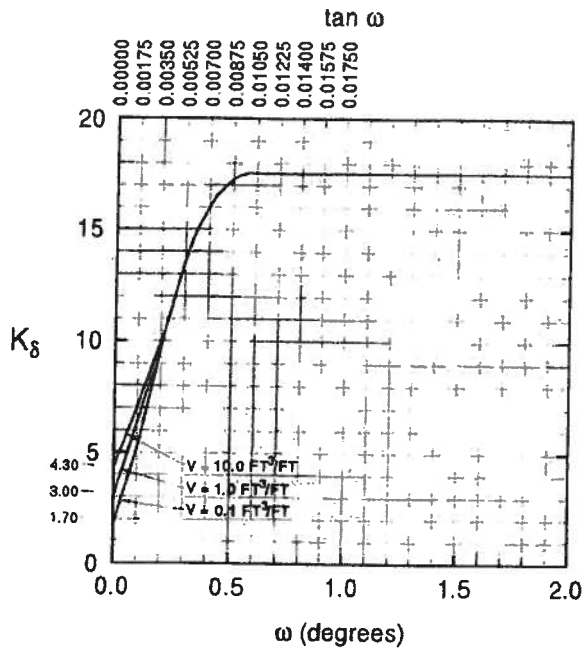


Figure 10.7.3.8.6f-4 Design Curve for Evaluating K_δ for Piles where $\phi_f = 40°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

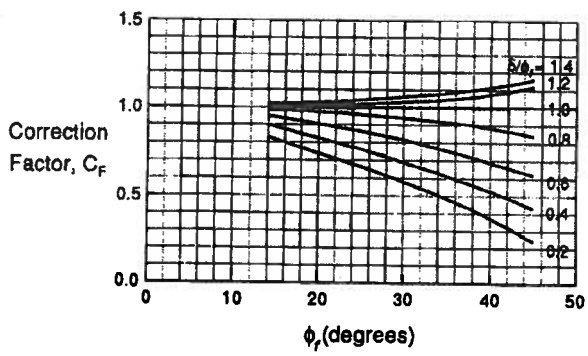
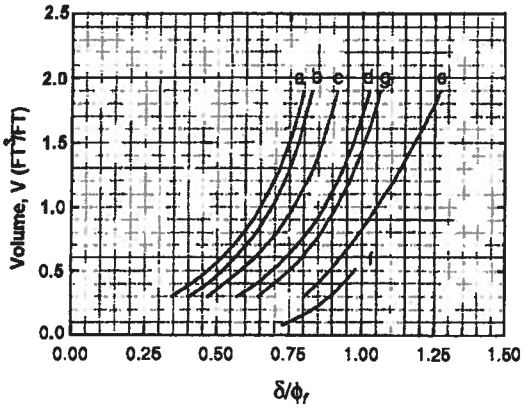


Figure 10.7.3.8.6f-5 Correction Factor for K_δ where $\delta \neq \phi_f$ (*Hannigan et al., 2005* after *Nordlund, 1979*).



a. Closed end pipe and non-tapered portion of monotube piles
b. Timber piles
c. Precast concrete piles
d. Raymond step-taper piles
e. Raymond uniform taper piles
f. H-piles
g. Tapered portion of monotube piles

Figure 10.7.3.8.6f-6 Relation of δ/ϕ_f and Pile Displacement, *V*, for Various Types of Piles (*Hannigan et al., 2005* after *Nordlund, 1979*).

The nominal unit tip resistance, q_p, in ksf by the Nordlund/Thurman method shall be taken as:

$$q_p = \alpha_t N'_q \sigma'_v \leq q_L \qquad (10.7.3.8.6f\text{-}2)$$

where:

α_t = coefficient from Figure 7 (dim.)

N'_q = bearing capacity factor from Figure 8

σ'_v = effective overburden stress at pile tip (ksf) $\leq$3.2 ksf

q_L = limiting unit tip resistance from Figure 9

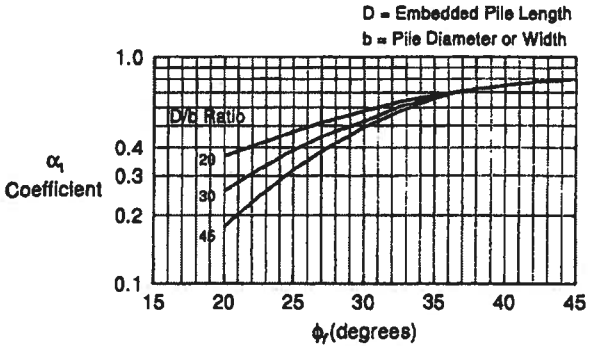


Figure 10.7.3.8.6f-7 α_t Coefficient (*Hannigan et al., 2005* modified after *Bowles, 1977*).

If the friction angle, ϕ_f, is estimated from average, corrected *SPT* blow counts, $N1_{60}$, the $N1_{60}$ values should be averaged over the zone from the pile tip to 2 diameters below the pile tip.

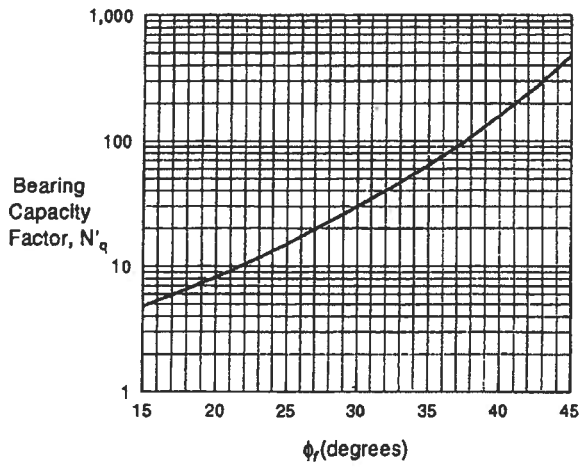


Figure 10.7.3.8.6f-8 Bearing Capacity Factor, N'_q, (*Hannigan et al., 2005* modified after *Bowles, 1977*).

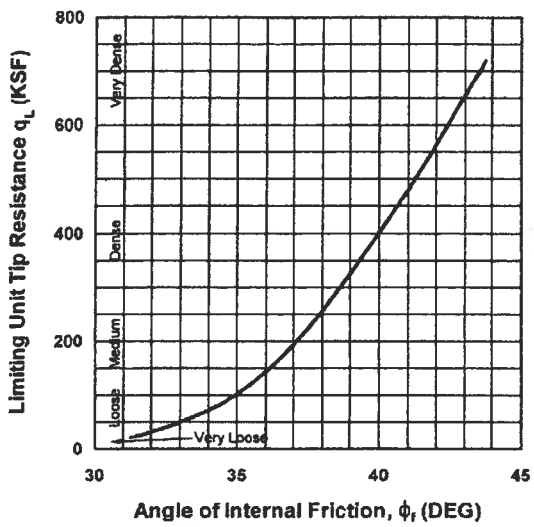


Figure 10.7.3.8.6f-9 Limiting Unit Pile Tip Resistance (*Hannigan et al., 2005* after *Meyerhof, 1976*).

10.7.3.8.6g Using SPT or CPT in Cohesionless Soils

These methods shall be applied only to sands and nonplastic silts.

The nominal unit tip resistance for the Meyerhof method, in ksf, for piles driven to a depth D_b into a cohesionless soil stratum shall be taken as:

$$q_p = \frac{0.8(N1_{60})D_b}{D} \leq q_\ell \qquad (10.7.3.8.6g\text{-}1)$$

C10.7.3.8.6g

In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In-situ test parameters may be used to estimate the tip resistance and skin friction of piles.

Two frequently used in-situ test methods for predicting pile axial resistance are the standard penetration test (*SPT*) method (*Meyerhof, 1976*) and the cone penetration test (*CPT*) method (*Nottingham and Schmertmann, 1975*).

where:

$N1_{60}$ = representative *SPT* blow count near the pile tip corrected for overburden pressure as specified in Article 10.4.6.2.4 (blows/ft.)

D = pile width or diameter (ft.)

D_b = depth of penetration in bearing strata (ft.)

q_ℓ = limiting tip resistance taken as eight times the value of $N1_{60}$ for sands and six times the value of $N1_{60}$ for nonplastic silt (ksf)

The nominal skin friction of piles in cohesionless soils for the Meyerhof method, in ksf, shall be taken as:

- For driven displacement piles:

$$q_s = \frac{\bar{N}1_{60}}{25} \qquad (10.7.3.8.6g\text{-}2)$$

- For nondisplacement piles, e.g., steel H-piles:

$$q_s = \frac{\bar{N}1_{60}}{50} \qquad (10.7.3.8.6g\text{-}3)$$

where:

q_s = unit skin friction for driven piles (ksf)

$\bar{N}1_{60}$ = average corrected *SPT*-blow count along the pile side (blows/ft.)

Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration. Nondisplacement piles usually have relatively small cross-sectional areas, e.g., steel H-piles and open-ended pipe piles that have not yet plugged. Plugging occurs when the soil between the flanges in a steel H-pile or the soil in the cylinder of an open-ended steel pipe pile adheres fully to the pile and moves down with the pile as it is driven.

Tip resistance, q_p, for the Nottingham and Schmertmann method, in ksf, shall be determined as shown in Figure 1.

In which:

$$q_p = \frac{q_{c1} + q_{c2}}{2} \qquad (10.7.3.8.6g\text{-}4)$$

where:

q_{c1} = average q_c over a distance of yD below the pile tip (path a-b-c); sum q_c values in both the downward (path a-b) and upward (path b-c) directions; use actual q_c values along path a-b and the minimum path rule along path b-c; compute q_{c1} for y-values from 0.7 to 4.0 and use the minimum q_{c1} value obtained (ksf)

CPT may be used to determine:

- The cone penetration resistance, q_c, which may be used to determine the tip resistance of piles, and
- Sleeve friction, f_s, which may be used to determine the skin friction resistance.

7. Pipe Piles – Driveability Evaluations

HALEY & ALDRICH

Calculations

Client	MAINE DOT	File No.	36654-001
Project	SEWALL'S BRIDGE - YORK, MAINE	Sheet	1 of
Subject	DRIVABILITY EVALUATIONS	Date	8/19/2011 12/13/11
		Computed By	BCS
		Checked By	GTW

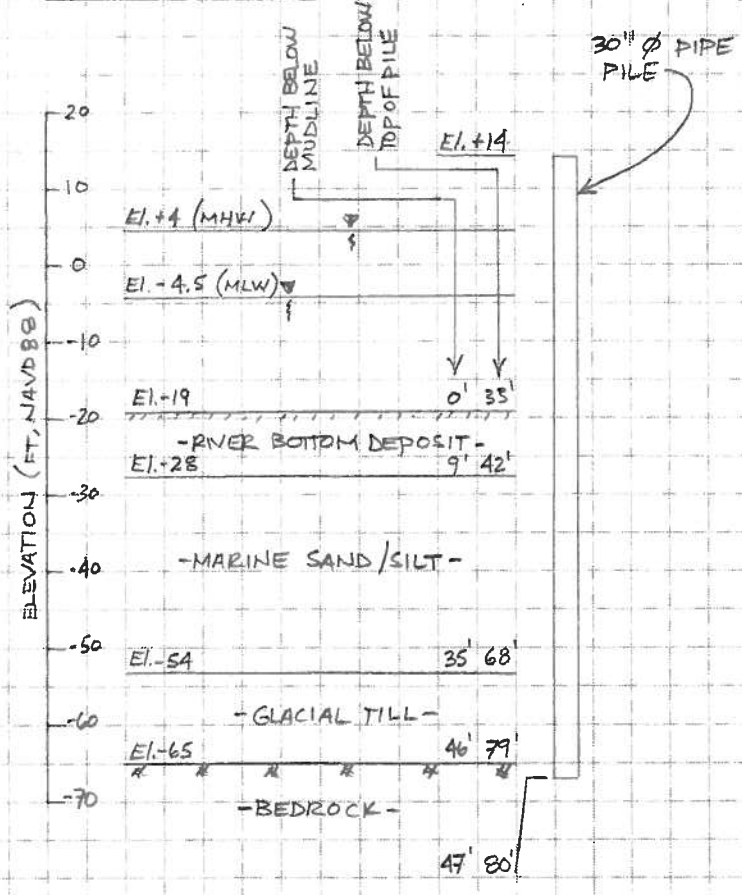
OBJECTIVE: COMPLETE DRIVABILITY EVALUATIONS USING GRL WEAP SOFTWARE TO DETERMINE AN APPROPRIATE PILE-HAMMER SYSTEM CAPABLE OF INSTALLING PROPOSED PILES TO THE REQUIRED DEPTHS/RESISTANCES WITHOUT DAMAGE (OVERSTRESS).

APPROACH:

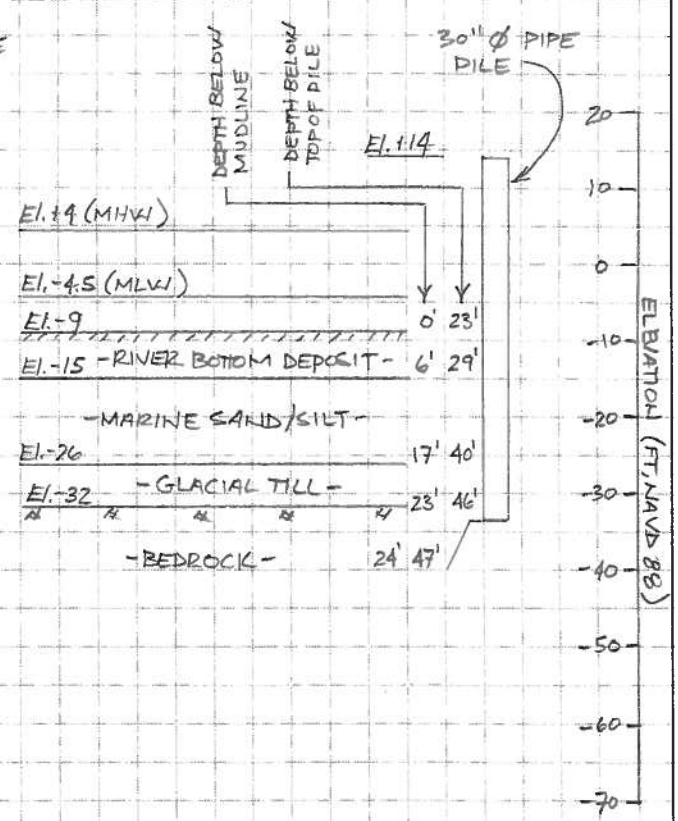
1. MODEL TWO SOIL PROFILES REPRESENTING LOCATIONS WHERE OVERBURDEN THICKNESS IS ANTICIPATED TO BE MAXIMUM (BENT NO.7) AND MINIMUM (BENT NO.1).
2. USE SIDE AND END BEARING RESISTANCES DETERMINED FROM STATIC PILE RESISTANCE CALCULATIONS.
3. MODEL TWO DIFFERENT "READILY AVAILABLE" OED PILE HAMMERS TO DETERMINE WHICH ONE IS MOST APPROPRIATE TO DRIVE THE PILES TO THE REQ'D DEPTH/RESISTANCE W/O DAMAGE.

SUBSURFACE PROFILES:

PIER BENT NO.7: BB-YSL-104, 202



PIER BENT NO.1: BB-YSL-201



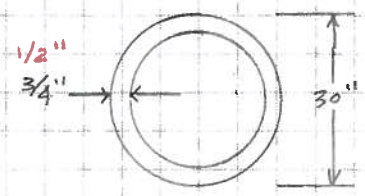
HALEY & ALDRICH

Calculations

Client	MAINE DOT	File No.	36654-001
Project	SEWALL'S BRIDGE - YORK, MAINE	Sheet	2 of
Subject	DRIVABILITY EVALUATIONS	Date	12/13/11 8/19/2011
		Computed By	DCS
		Checked By	EFW

PILE PROPERTIES:

CONSIDER A PIPE PILE DRIVEN OPEN-ENDED (CLOSED-END) WITH THE FOLLOWING SECTIONAL AND MATERIAL PROPERTIES....



D_o = 30 IN.

D_i = 30 IN − (2)(0.75 IN.) = 28.5 IN. (0.5 IN.) 29.0 IN.

$A_o = (\pi/4)(30\text{ IN.})^2 = 706.86\text{ IN}^2 = 4.91\text{ FT}^2$

$A_i = (\pi/4)(28.5\text{ IN.})^2 = 637.94\text{ IN}^2$ (29.0 IN.) 660.52 IN^2

$A_s = A_o - A_i = 706.86\text{ IN}^2 - 637.94\text{ IN}^2 = 68.92\text{ IN}^2 = 68.9\text{ IN}^2 = 0.48\text{ FT}^2$

$660.52\text{ IN}^2 = 46.34\text{ IN}^2 = 46.3\text{ IN}^2 = 0.32\text{ FT}^2$

$P = (\pi)(D_o) = (\pi)(30\text{ IN.}) = 94.25\text{ IN} = 94.3\text{ IN.} = 7.85\text{ FT}$

$L_{BENT\ 1}$ = 47 FT ; $L_{BENT\ 7}$ = 80 FT (SEE SUBSURFACE PROFILES)

F_y = 45 KSI (ASTM A252 GRADE 3)

SOIL PROPERTIES:

SUMMARIZED FROM STATIC PILE RESISTANCE CALCULATIONS....

	RESISTANCE	PIER BENT NO. 1	PIER BENT NO. 7	
RIVER BOTTOM	UNIT SIDE TOP	0 PSF	0 PSF	NEGLECT CONTRIBUTION OF THIS DEPOSIT
	UNIT SIDE BOTTOM	0 PSF	0 PSF	
	END TOP	0 KIP	0 KIP	
	END BOTTOM	0 KIP	0 KIP	
MARINE SAND/SILT	UNIT SIDE TOP	144 PSF	203 PSF	
	UNIT SIDE BOTTOM	411 PSF	870 PSF	
	END TOP	1.31 KIP	3.68 KIP	
	END BOTTOM	3.72 KIP	9.57 KIP	
GLACIAL TILL	UNIT SIDE TOP	1,065 PSF	2,169 PSF	
	UNIT SIDE BOTTOM	1,442 PSF	2,923 PSF	
	END TOP	57 KIP	251 KIP	
	END BOTTOM	77 KIP	311 KIP	
ROCK	UNIT SIDE	5,000 PSF	5,000 PSF	ASSUMED
	END	345 KIP	345 KIP	ASSUMED BASED ON COMP. STRENGTH OF ROCK = 5,000 PSI ∴ (5 KSI)(68.9 IN²) = 345 KIP

SINCE ROCK << STEEL
(5 KSI) (45 KSI)

HALEY & ALDRICH

Calculations

File No.	36654-001		
Sheet	3 of 3		
Client	MAINE DOT	Date	12/13/11 8/19/2011
Project	SEWALL'S BRIDGE - YORK, MAINE	Computed By	BCS
Subject	DRIVABILITY EVALUATIONS	Checked By	EFW

HAMMER SYSTEM PROPERTIES:

CONSIDER TWO POTENTIAL PILE HAMMER SYSTEMS....

1. APE D19-42 (E_{MAX} = 42,800 FT-LB)
2. APE D36-26 (E_{MAX} = 89,300 FT-LB)

SEE ATTACHED FOR PILE HAMMER DATA SHEETS.

RESULTS:

PIER BENT NO. 1: MIN. REQ'D CAPACITY DEVELOPED AT APPROXIMATE TOP OF BEDROCK ELEVATION WITH AN APE D19-42 AT A PENETRATION RESISTANCE OF 5 BPI.

PIER BENT NO. 7: MIN. REQ'D CAPACITY DEVELOPED AT APPROXIMATE TOP OF GLACIAL TILL ELEVATION WITH AN APE D19-42 AT A PENETRATION RESISTANCE OF 5 BPI.

∴ APE D19-42 CAPABLE OF DRIVING PROPOSED PILES TO MIN. REQ'D NOMINAL RESISTANCE AND/OR MINIMUM TIP ELEVATION WITHOUT OVERSTRESS.

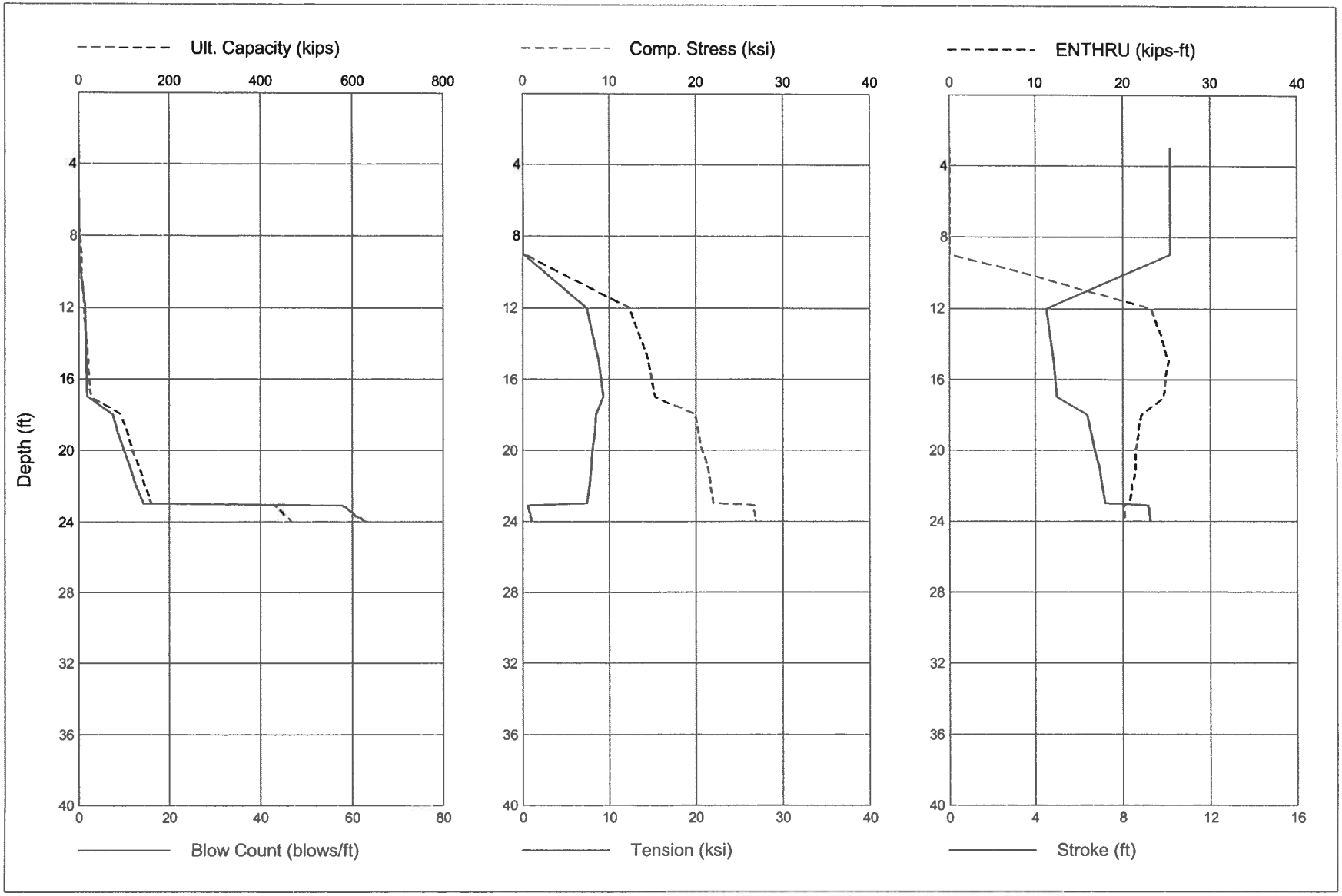
Haley & Aldrich, Inc.

Dec 20 2011

Bent 1 - Drivability - APE D19-42

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

GRLWEAP(TM) Version 2005



12/20/2011

Haley & Aldrich, Inc.
Bent 1 - Drivability - APE D19-42

Dec 20 2011
GRLWEAP(TM) Version 2005

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
3.0	0.0	0.0	0.0	0.0	0.000	0.000	10.17	0.0
6.0	0.0	0.0	0.0	0.0	0.000	0.000	10.17	0.0
9.0	6.2	4.2	2.0	0.0	0.000	0.000	10.17	0.0
12.0	12.8	10.2	2.6	1.5	12.464	-7.454	4.52	23.3
15.0	21.2	17.9	3.3	1.7	14.604	-8.843	4.85	25.2
17.0	27.7	24.0	3.7	1.9	15.331	-9.300	4.97	24.7
18.0	92.9	32.6	60.3	7.6	19.861	-8.510	6.38	22.1
19.0	105.4	41.7	63.7	8.7	20.349	-8.326	6.55	21.8
20.0	118.3	51.3	67.0	10.0	20.795	-8.095	6.71	21.5
21.0	131.8	61.4	70.3	11.4	21.394	-7.996	6.93	21.5
22.0	145.7	72.0	73.7	12.7	21.708	-7.720	7.07	21.1
23.0	160.1	83.1	77.0	14.2	22.069	-7.422	7.19	20.9
23.1	431.2	86.2	345.0	SBP! 57.9	26.706	-0.594	9.15	20.1
23.2	434.7	89.7	345.0	58.3	26.683	-0.639	9.16	20.1
23.2	437.9	92.9	345.0	58.7	26.739	-0.694	9.17	20.1
23.3	441.0	96.0	345.0	59.1	26.740	-0.741	9.18	20.1
23.4	444.2	99.2	345.0	59.5	26.728	-0.782	9.19	20.1
23.5	447.7	102.7	345.0	60.1	26.796	-0.837	9.20	20.2
23.6	450.8	105.8	345.0	60.4	26.774	-0.874	9.21	20.2
23.7	454.0	109.0	345.0	60.9	26.840	-0.916	9.22	20.2
23.8	457.5	112.5	345.0	61.3	26.812	-0.949	9.23	20.2
23.8	460.7	115.7	345.0	61.8	26.877	-0.978	9.24	20.3
23.9	463.8	118.8	345.0	62.2	26.868	-0.999	9.25	20.3
24.0	467.3	122.3	345.0	62.8	26.903	-1.015	9.26	20.3

GLACIAL TILL

BEDROCK

Total Continuous Driving Time 3.00 minutes; Total Number of Blows 129

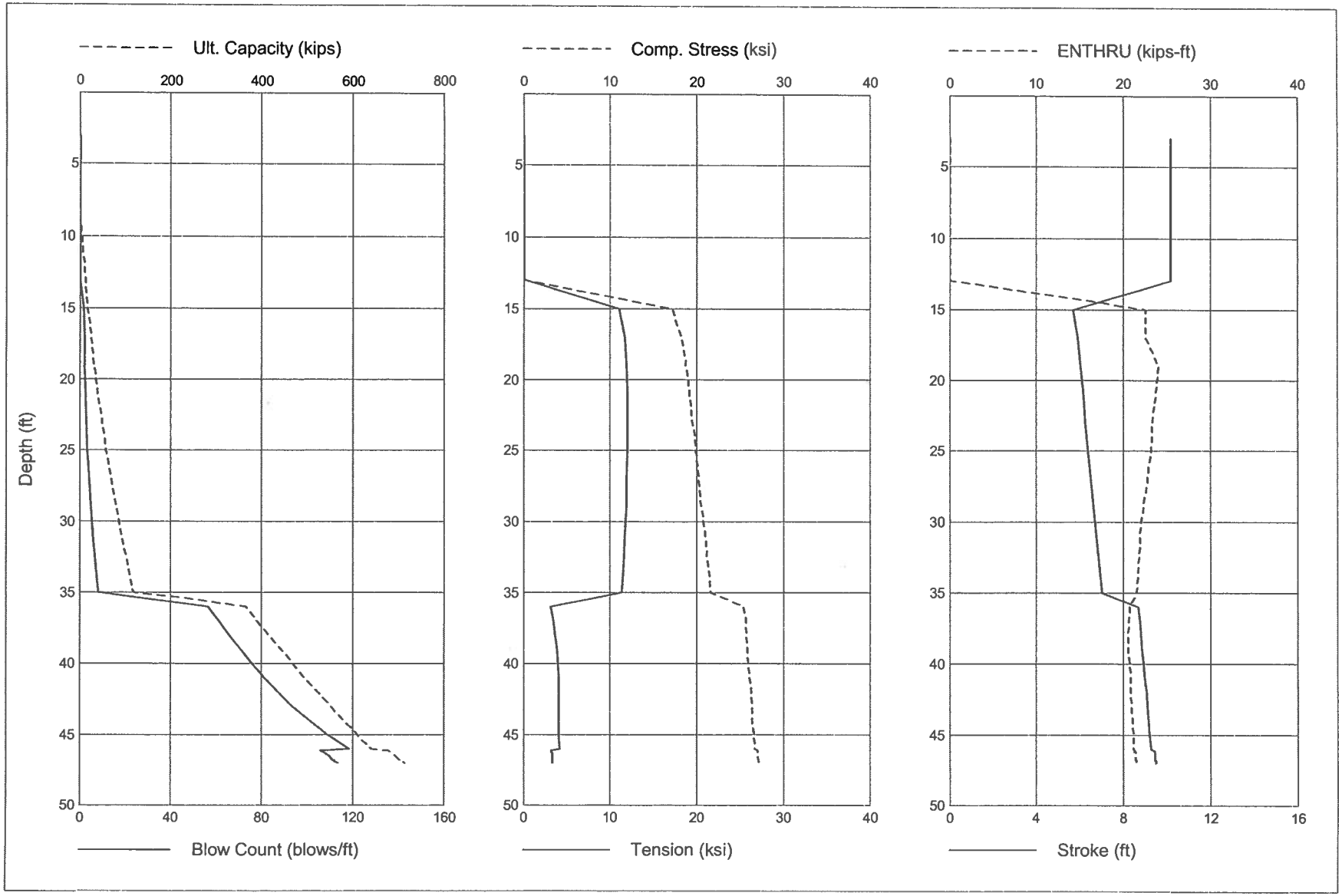
Haley & Aldrich, Inc.

Dec 20 2011

Bent 7 - Drivability - APE D19-42

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

GRLWEAP(TM) Version 2005



12/20/2011

Haley & Aldrich, Inc.
Bent 7 - Drivability - APE D19-42

Dec 20 2011
GRLWEAP(TM) Version 2005

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

	Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
	3.0	0.0	0.0	0.0	0.0	0.000	0.000	10.17	0.0
	6.0	0.0	0.0	0.0	0.0	0.000	0.000	10.17	0.0
	9.0	0.0	0.0	0.0	0.0	0.000	0.000	10.17	0.0
MARINE SAND /SILT	11.0	7.7	3.6	4.1	0.0	0.000	0.000	10.17	0.0
	13.0	12.6	8.0	4.6	0.0	0.000	0.000	10.17	0.0
	15.0	18.2	13.2	5.0	2.0	17.270	-11.075	5.70	22.6
	17.0	24.7	19.2	5.5	2.2	18.182	-11.676	5.89	22.6
	19.0	32.0	26.0	5.9	2.4	18.757	-11.874	6.03	24.0
	21.0	40.0	33.6	6.4	2.6	19.137	-12.036	6.15	23.7
	23.0	48.9	42.1	6.9	2.9	19.478	-12.023	6.25	23.3
	25.0	58.6	51.3	7.3	3.4	19.863	-12.013	6.38	23.2
	27.0	69.1	61.3	7.8	4.1	20.212	-11.941	6.51	22.8
	29.0	80.4	72.2	8.2	4.9	20.584	-11.847	6.63	22.3
	31.0	92.5	83.8	8.7	5.8	20.922	-11.701	6.77	21.9
	33.0	105.4	96.3	9.1	6.9	21.250	-11.532	6.89	21.7
	35.0	119.1	109.6	9.6	8.2	21.627	-11.388	7.02	21.5
	36.0	365.1	126.9	238.3	56.4	25.437	-3.193	8.69	20.7
GLACIAL TILL	37.0	390.3	144.7	245.5	5BPI 61.2	25.612	-3.417	8.76	20.6
	38.0	415.9	163.1	252.8	65.8	25.712	-3.651	8.80	20.5
	39.0	442.1	182.0	260.1	70.5	25.843	-3.864	8.87	20.5
	40.0	468.8	201.5	267.4	75.8	25.975	-3.996	8.92	20.7
	41.0	496.1	221.5	274.6	81.3	26.121	-4.080	8.99	20.8
	42.0	523.9	242.0	281.9	87.2	26.254	-4.148	9.06	20.9
	43.0	552.2	263.0	289.2	93.4	26.397	-4.140	9.12	21.0
	44.0	581.1	284.6	296.5	100.9	26.418	-4.124	9.16	21.1
	45.0	610.5	306.8	303.7	109.1	26.593	-4.158	9.21	21.2
	46.0	640.4	329.4	311.0	10BPI 118.4	26.680	-4.203	9.26	21.2
BEDROCK	46.1	677.6	332.6	345.0	105.9	27.026	-3.269	9.41	21.3
	46.2	681.1	336.1	345.0	106.7	27.004	-3.282	9.43	21.3
	46.2	684.2	339.2	345.0	107.1	27.070	-3.305	9.43	21.4
	46.3	687.4	342.4	345.0	107.8	27.078	-3.321	9.44	21.4
	46.4	690.5	345.5	345.0	108.6	27.067	-3.337	9.44	21.4
	46.5	694.1	349.1	345.0	109.5	27.034	-3.349	9.45	21.4
	46.6	697.2	352.2	345.0	110.2	27.067	-3.356	9.45	21.4
	46.7	700.3	355.3	345.0	110.6	27.127	-3.375	9.46	21.4
	46.8	703.9	358.9	345.0	111.5	27.126	-3.384	9.46	21.4
	46.8	707.0	362.0	345.0	111.9	27.165	-3.393	9.48	21.5
	46.9	710.2	365.2	345.0	112.6	27.173	-3.406	9.48	21.5

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000 (Continued)

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
47.0	713.7	368.7	345.0	113.6	27.132	-3.411	9.49	21.5

Total Continuous Driving Time 27.00 minutes; Total Number of Blows 1058

8. Pipe Piles – Lateral Pile Capacity

HALEY & ALDRICH	CALCULATIONS	File No.	36654-001
		Sheet	
Client	VHB	Date	10OCT2011
Project	Sewalls Bridge, York, Maine	Computed by	JLL
Subject	Pile Section Properties for FB-MultiPier Analyses	Checked by	EFW

Objective:

Perform lateral analysis of select pile bents.

Approach:

- Use FB-MultiPier to model/analyze concrete-filled fiberglass and pipe pile group.
- Apply updated loads provided by VHB (in email dated 17AUG2011).

Assumptions:

- Piles are linear-elastic
- Scour depth is 10 ft (but scour does not impact cohesive soils)
- 1/8 inch corrosion reduction applied to outside of pile
- Piles are closed-ended and will be concrete-filled

Model Parameters:

Pile Section Parameters

				Gross Section			Corroded Section		
Component	Diam. (in)	t (in)	tcorr (in)	I (in^4)	A (in^2)	E (lbs/in^2)	I (in^4)	A (in^2)	J (in^4)
Steel Pipe	30	0.5	0.125	5042	46.3	29000000	3733	35	7467
Concrete	29	N/A	N/A	34719	661	3604997	N/A	N/A	
Composite	30	N/A	N/A	9358	128	29000000	8049	117	7467

J calculated from steel only

- Corrosion taken only on the outside of the section.
- 100% of concrete moment of inertia taken in the composite section.

For Moment Capacity Check (consider corroded section steel only)

fy =	45	ksi
0.66*fy =	29.7	ksi
Mall =	7392	kip*in
Mall =	616	kip*ft
Scorr =	249	in^3

Concrete-Filled Fiberglass Section (if we take concrete only)

Component	Width (in)	I (in^4)	A (in^2)	E (lbs/in^2)	J (in^4)
Concrete	11.25	1335	126.6	3604997	2256

dimension of concrete according to Steve Hodgdon email dated 11AUG2011

Input cells in this sheet

Used in FB-MultiPier

HALEY & ALDRICH	CALCULATIONS	File No.	36654-001
		Sheet	
Client	VHB	Date	AUG2011
Project	Sewalls Bridge, York, Maine	Computed by	JLL
Subject	Pile Section Properties for FB-MultiPier Analyses	Checked by	EFW

P-multipliers (static):

S/D	R1	R2	R3 and on
3	0.7	0.5	0.35
5	1	0.85	0.7
2.8	0.67	0.47	0.32
3.5	0.78	0.59	0.44

P-multipliers for 2.5D extrapolated (see AASHTO re: interpolation)
P-multipliers from Table 10.7.2.4-1 of AASHTO LRFD 2007

For Sewall's bridge, 30 in diameter piles spaced 7 ft = 2.8D

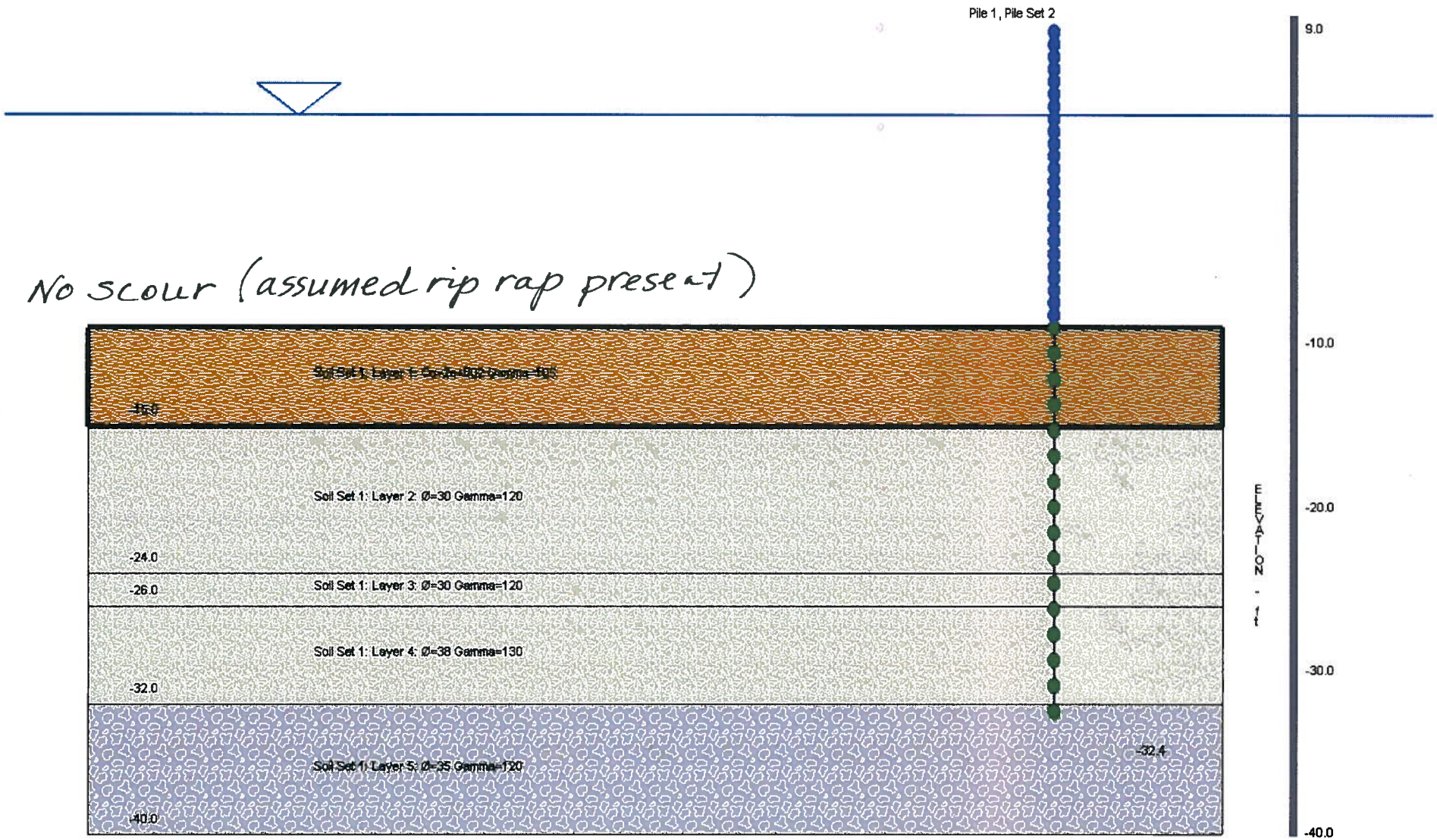
For Sewall's bridge, 24 in diameter piles spaced 7 ft = 3.5D

FB-Multipier File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-D1

File View Control Wizard Help

Pier #1 | 1 | Load Case

Pier 1



No scour (assumed rip rap present)

HALEY & ALDRICH	CALCULATIONS	File No.	36654-001
		Sheet	1 of 1
Client	VHB	Date	18OCT2011
Project	Sewall's Bridge	Computed by	JLL
Subject	**Bent 1 Summary of Results**	Checked by	EFW

Files: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML
G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-D1.XML
G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-D1.in.XML

FB-MultiPier Results for Bent 1

SUMMARY OF RESULTS

AASHTO-LRFD Limit State	Plumb Pile/ Concrete Fill	Batt. Pile/ Concrete Fill	Scour	δx (1) [in]	δy (1) [in]	$V_{x\,max}$ [kip]	$V_{y\,max}$ [kip]	$M_{x\,max}$ (2) [kip-ft]	$M_{y\,max}$ (2) [kip-ft]	N_{max} (3) [kip]	N_{min} (4) [kip]
SER I Wind at 60	30x0.5/ yes	30x0.5/ yes	none	0.05	0.29	4.4	1.7	19	12	55	22
STR III Wind at 0	30x0.5/ yes	30x0.5/ yes	none	0.07	0.06	2.3	0.5	6	20	23	-1
STR V Wind at 60	30x0.5/ yes	30x0.5/ yes	none	0.06	0.35	6.4	2.1	23	17	75	30

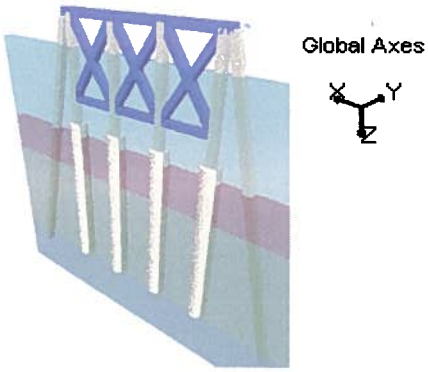
x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

SER I Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)



Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
Top of Cap Beam	2	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	3	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	4	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	5	0.30	0.80	62.20	0.00	0.00	0.00
Midheight of 12"x12"	78	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	117	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	156	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	195	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	89	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	128	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	167	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	206	0.10	0.06	0.00	0.00	0.00	0.00

x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

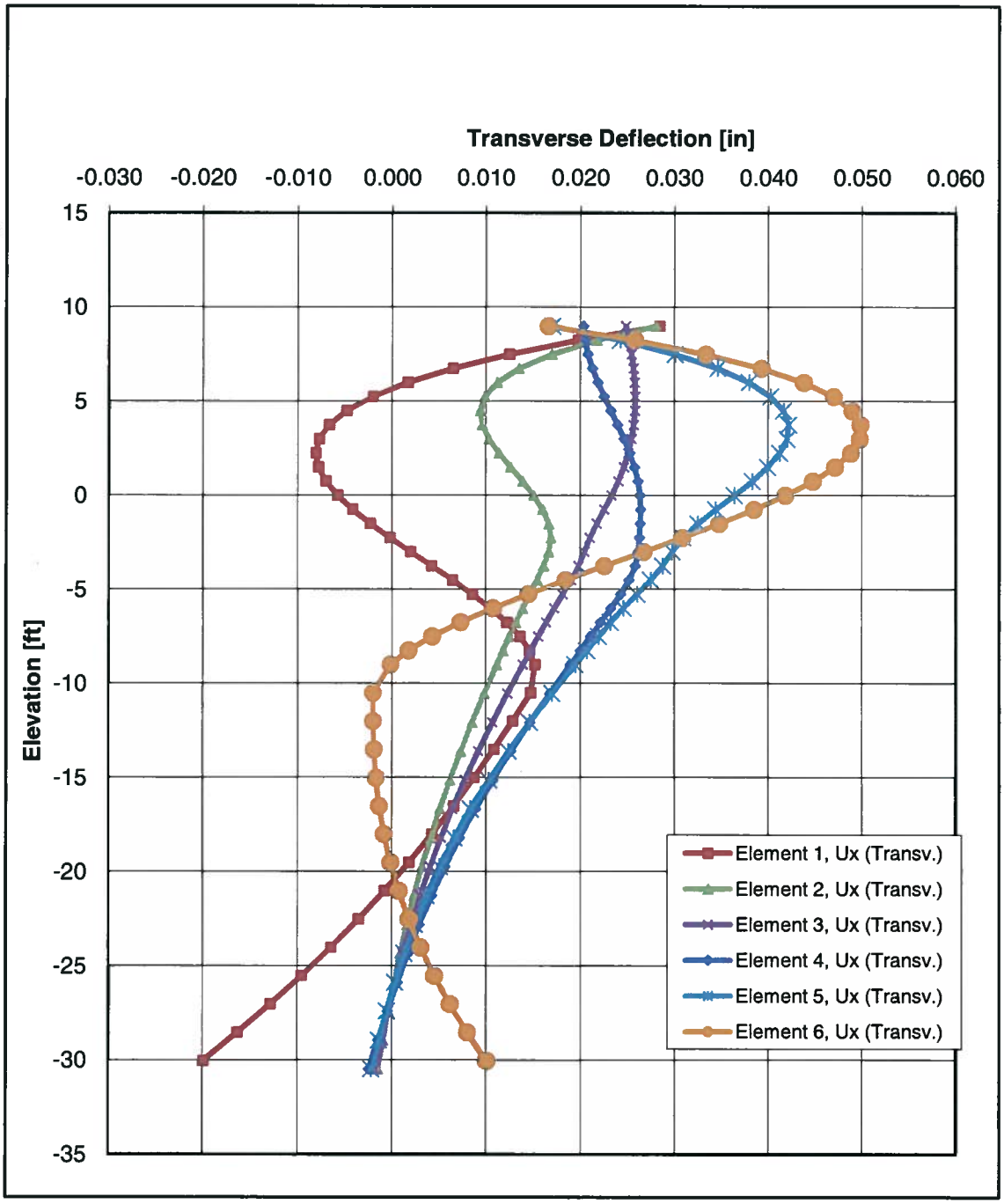
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

SER I Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-[



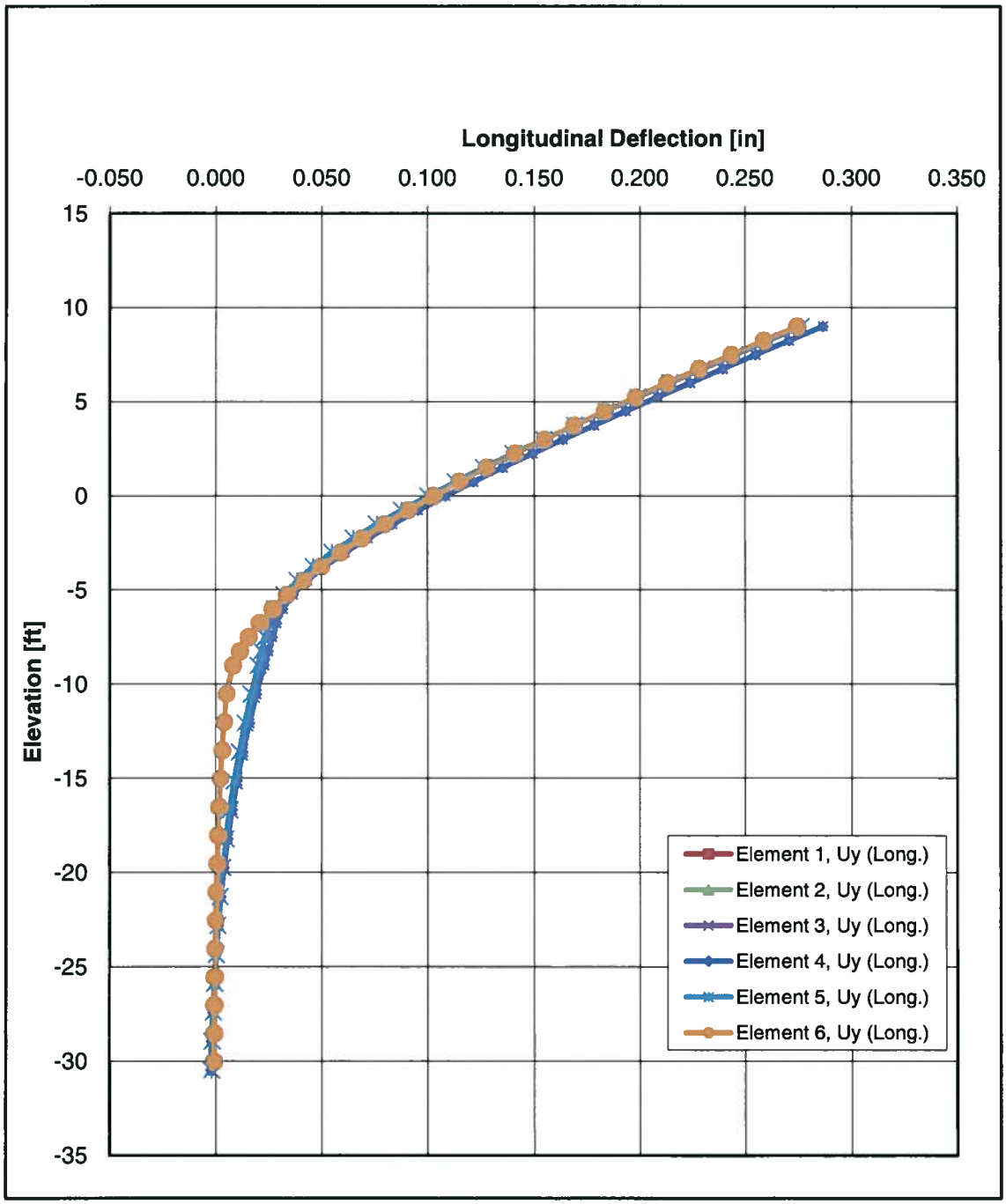
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

SER I Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-[



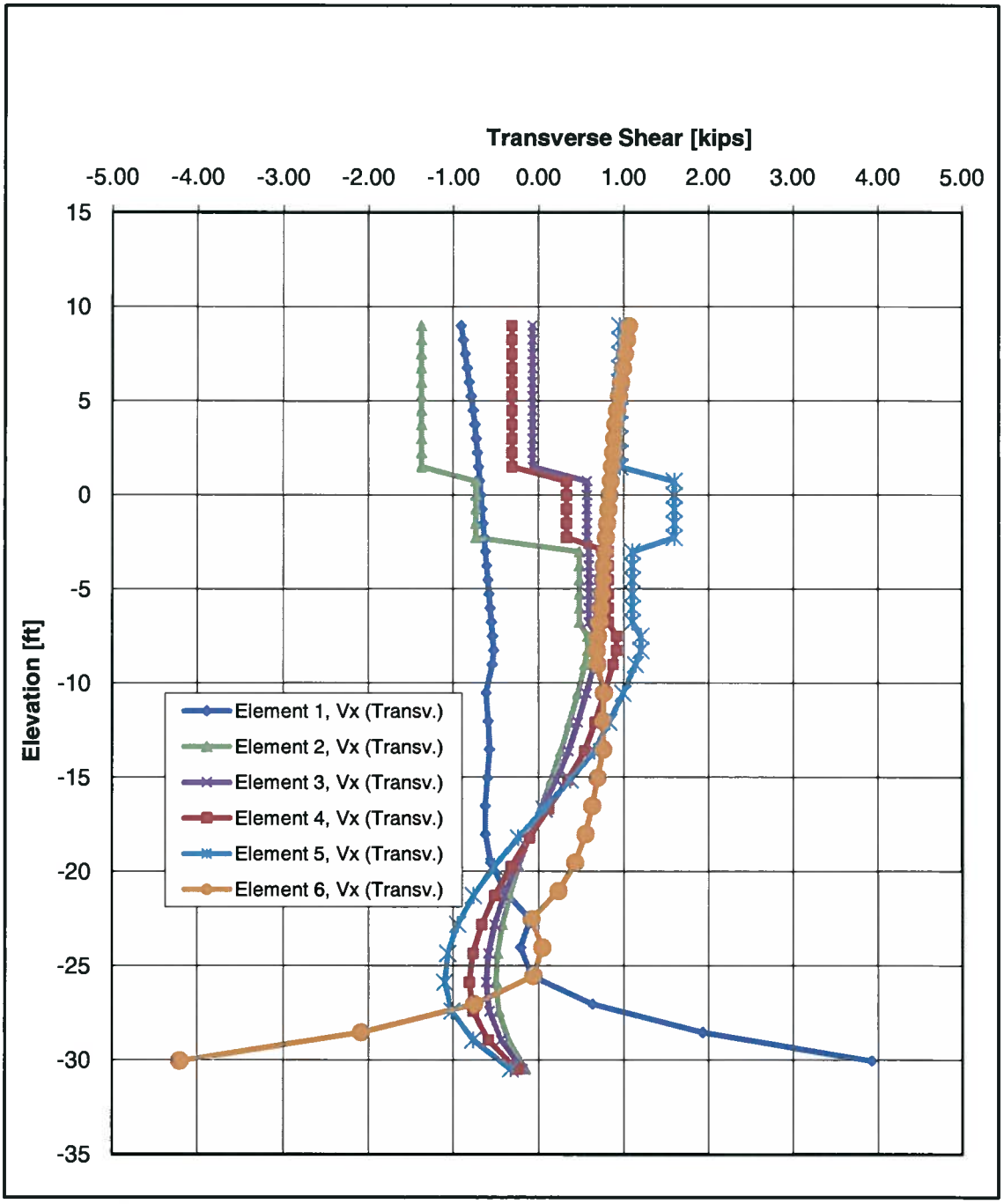
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

SER I Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-[



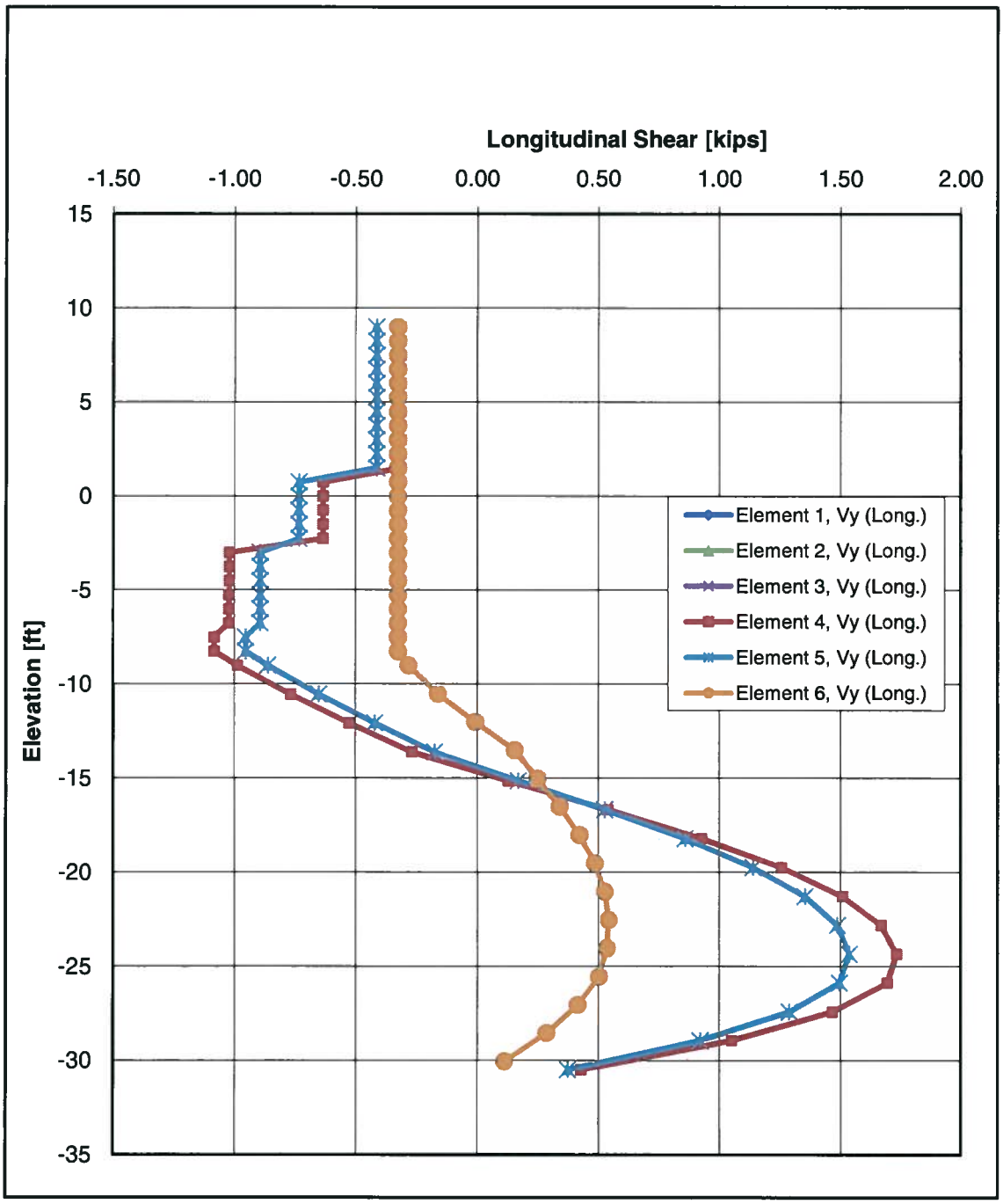
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

SER I Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-[



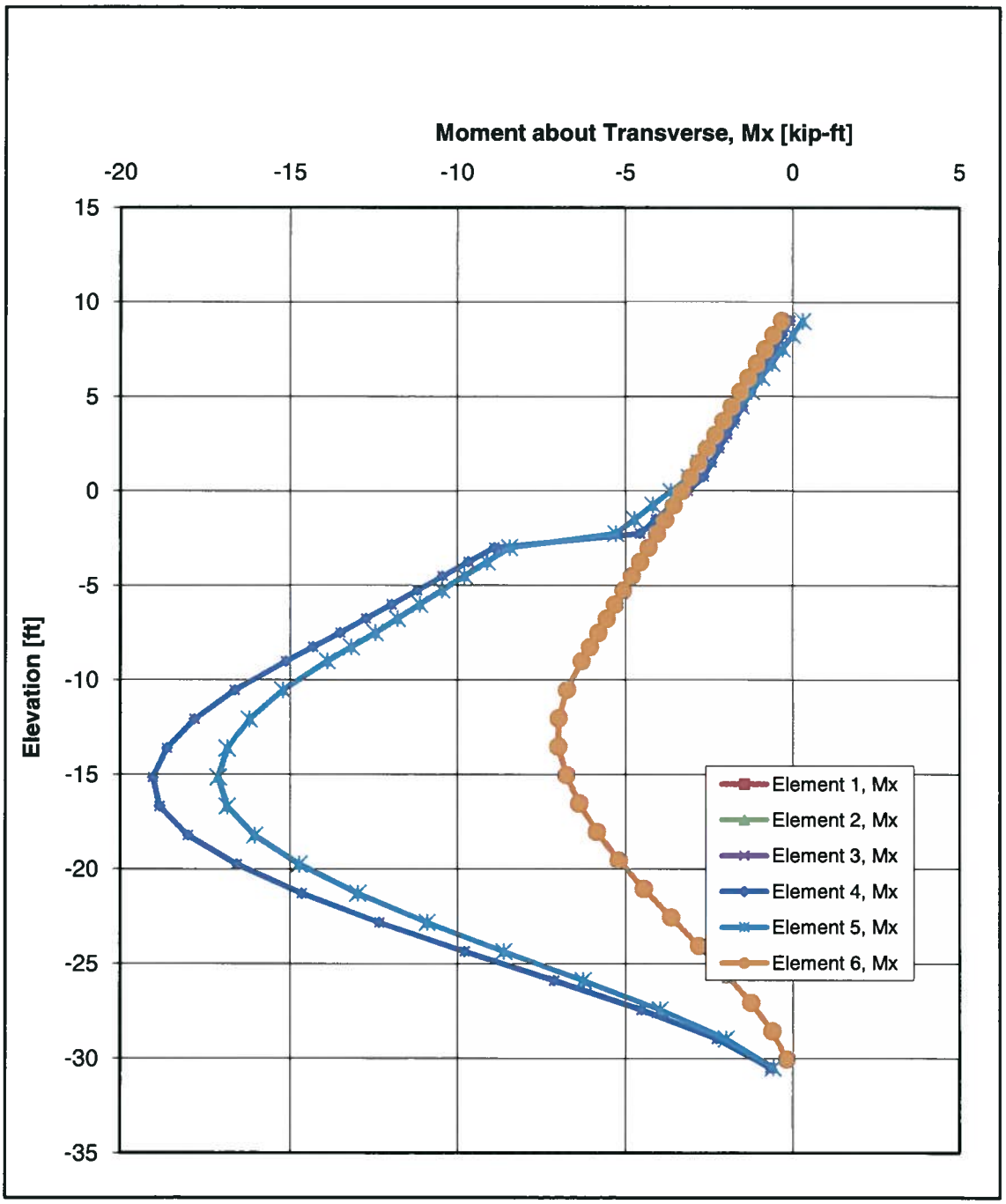
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

SER I Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-I



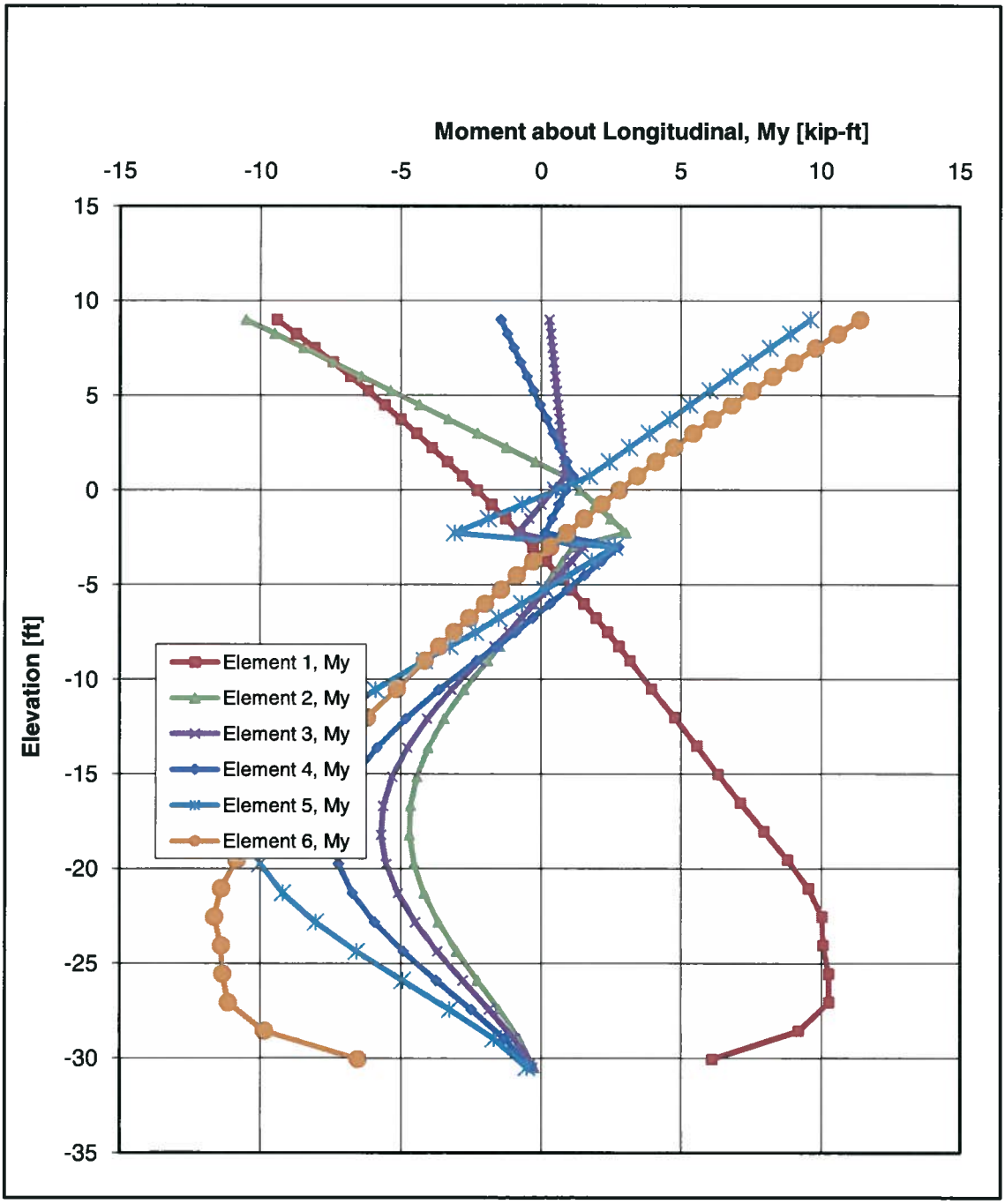
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

SER I Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1010-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-SER I Wind at 60-WITH CONCRETE-[



HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-D1.XML

STR III Wind at 0 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Global Axes

Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
Top of Cap Beam	2	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	3	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	4	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	5	1.70	0.00	7.00	0.00	0.00	0.00
Midheight of 12"x12"	78	0.64	0.48	0.00	0.00	0.00	0.00
Midheight of 12"x12"	117	0.64	0.48	0.00	0.00	0.00	0.00
Midheight of 12"x12"	156	0.64	0.48	0.00	0.00	0.00	0.00
Midheight of 12"x12"	195	0.64	0.48	0.00	0.00	0.00	0.00
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Midheight of Exposed 30 in PP	128	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	167	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	206	0.10	0.06	0.00	0.00	0.00	0.00

x - Bridge transverse direction
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z - Vertical direction

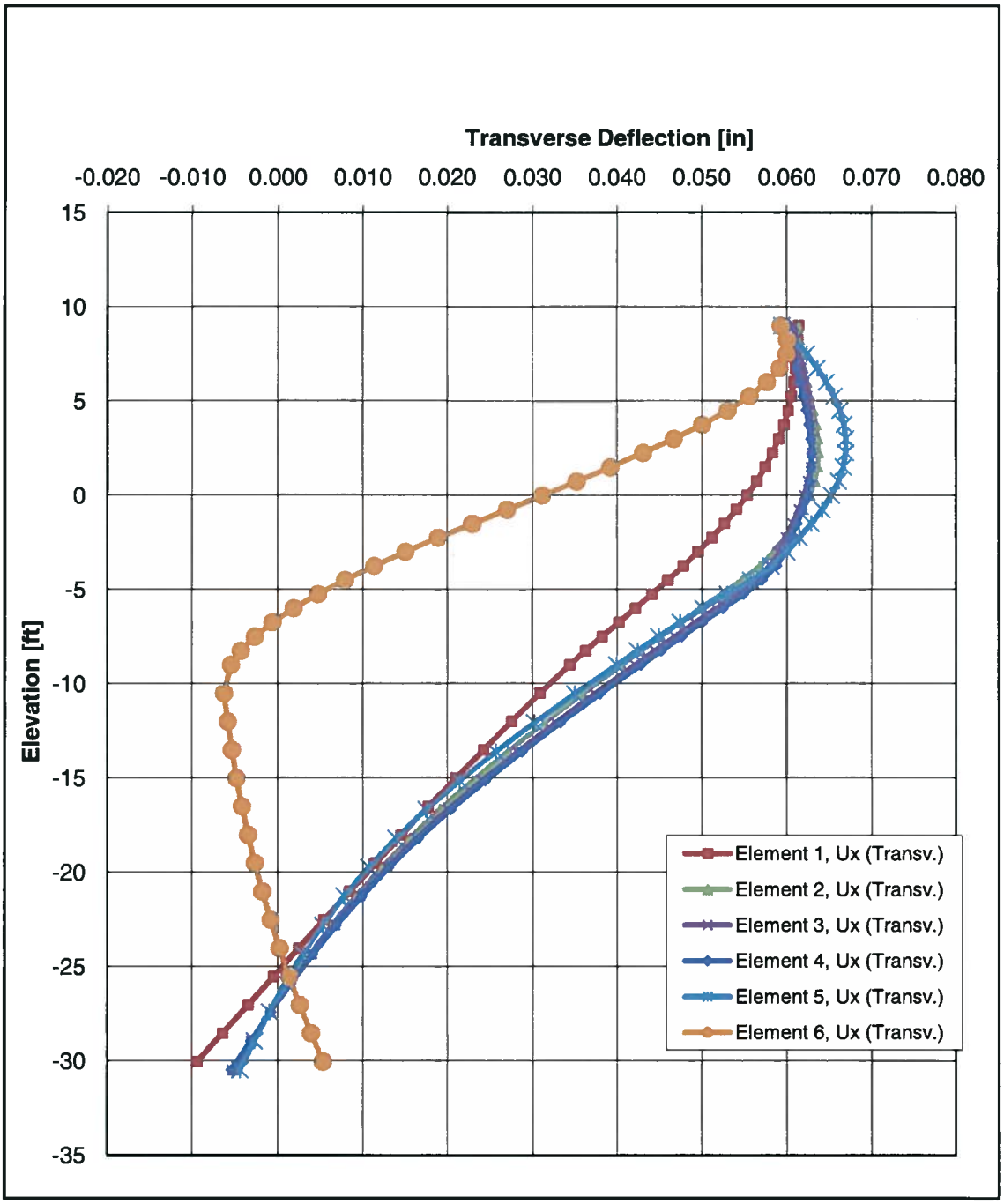
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR III Wind at 0 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-[



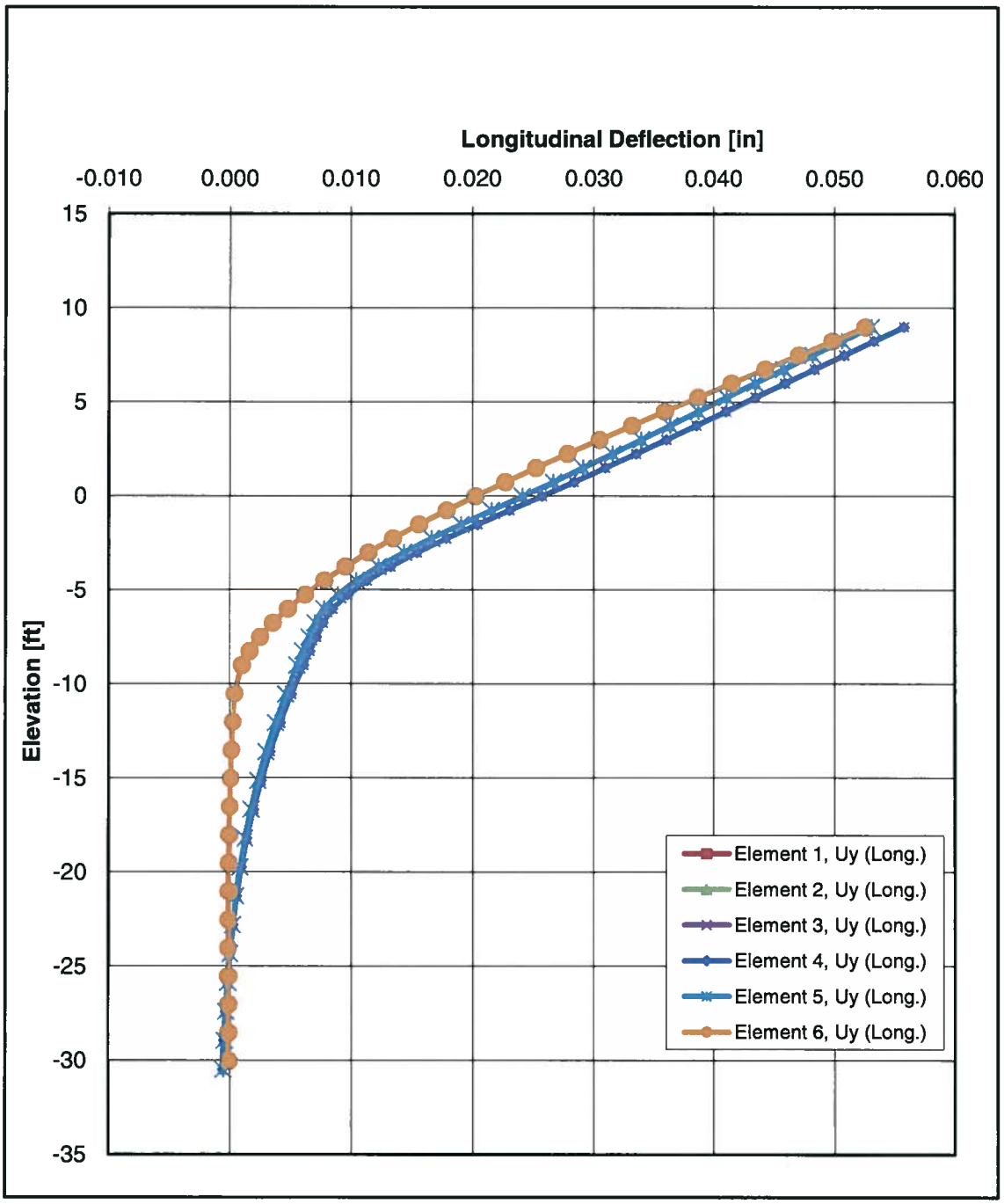
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR III Wind at 0 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-[



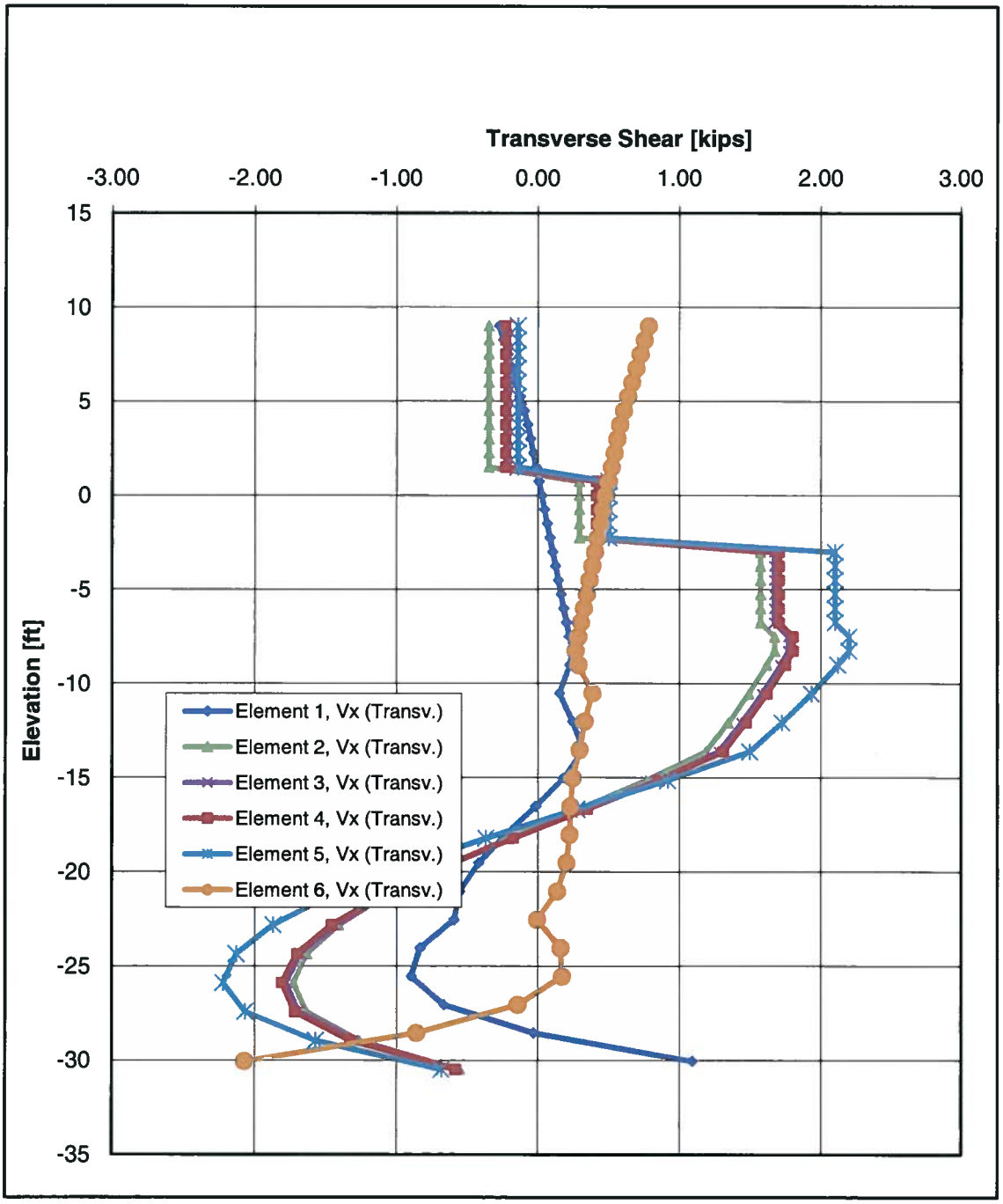
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR III Wind at 0 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-D



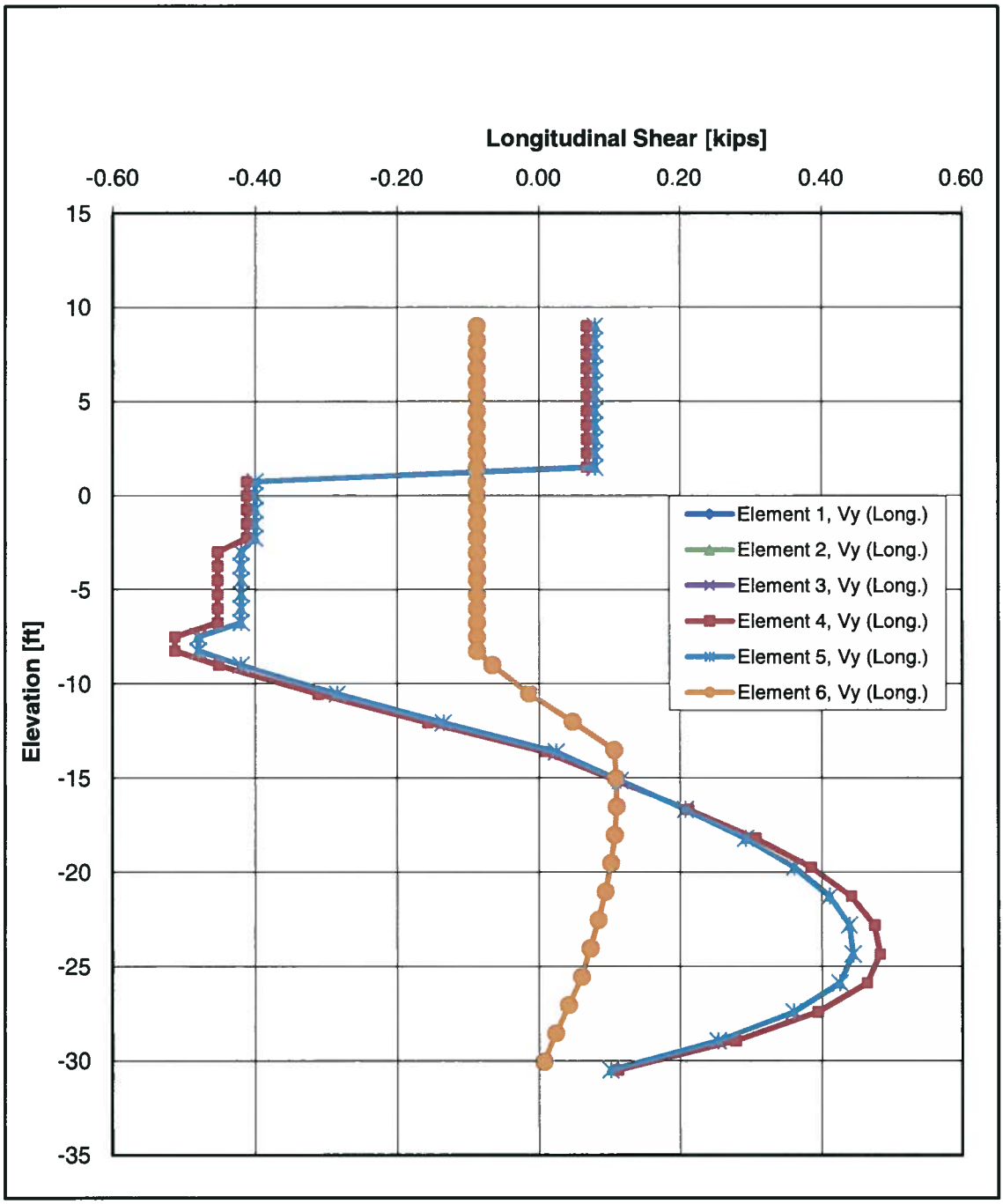
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR III Wind at 0 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-[



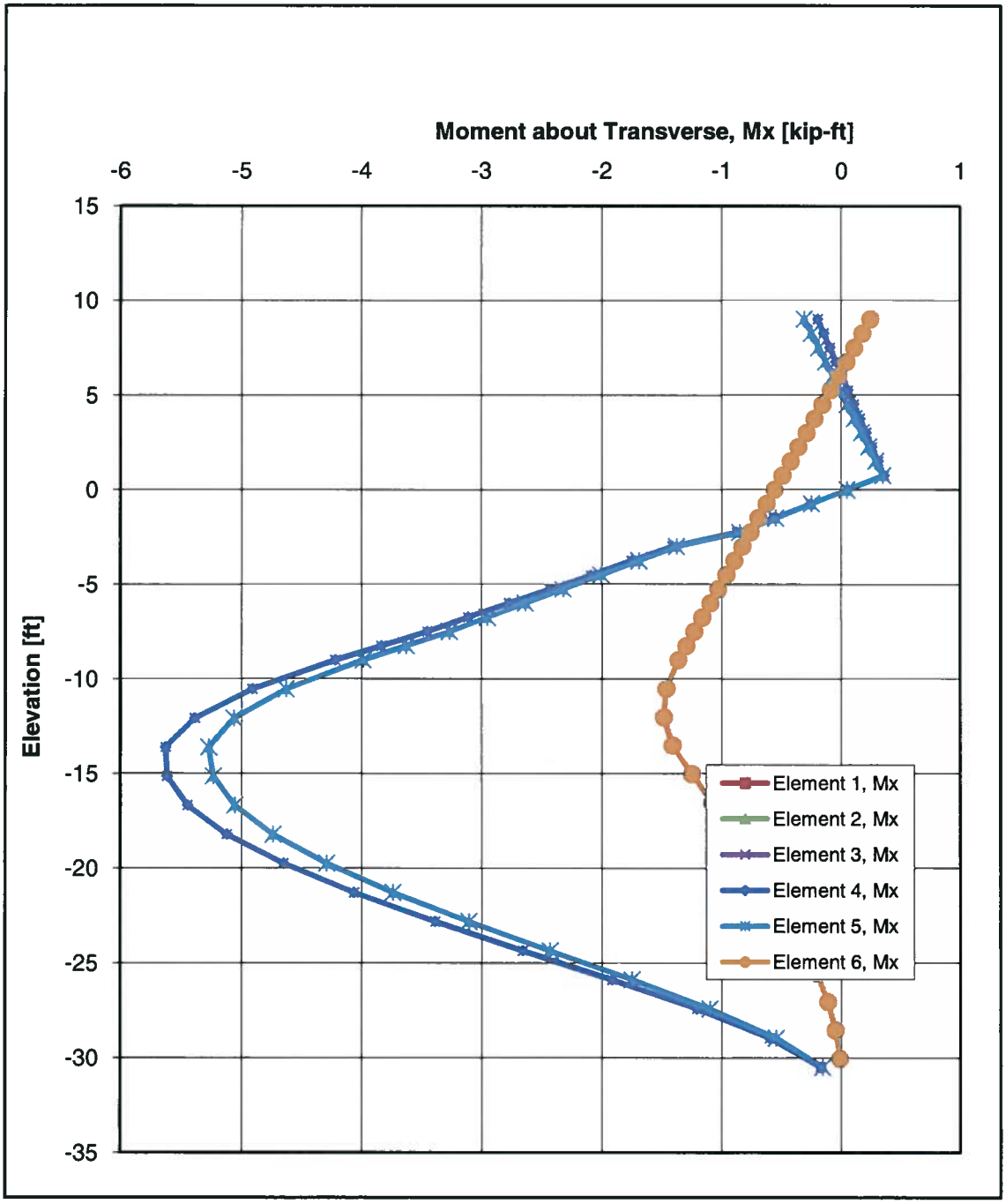
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR III Wind at 0 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-[



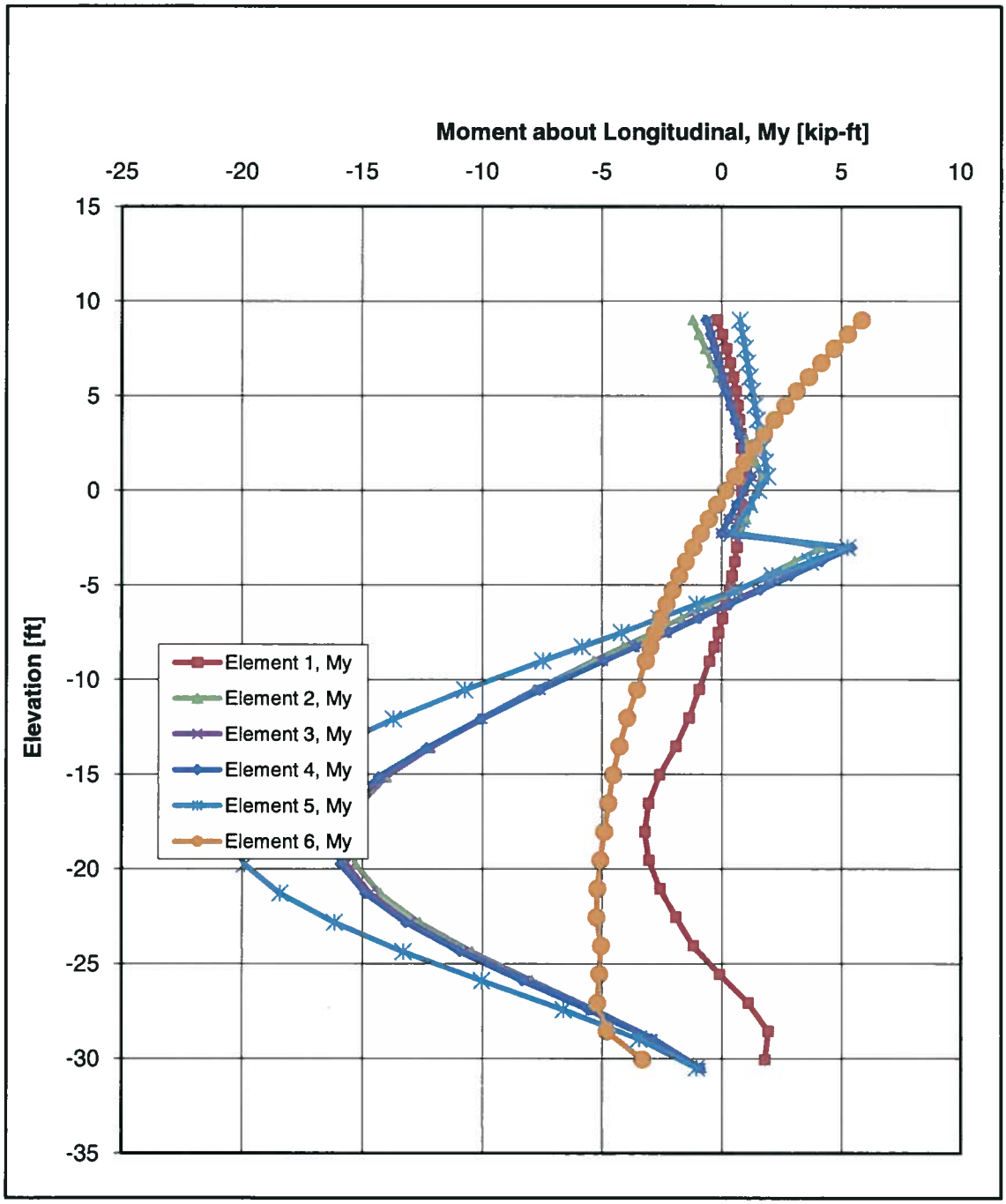
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR III Wind at 0 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR III Wind at 0-WITH CONCRETE-[

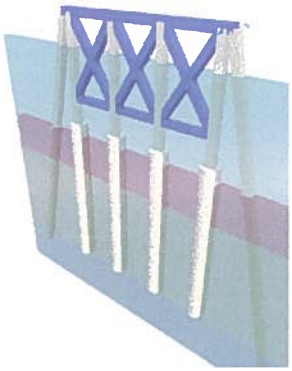


HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-D1.in.XML

STR V Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)



Global Axes

Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
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Midheight of 12"x12"	117	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	156	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	195	0.64	0.32	0.00	0.00	0.00	0.00
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Midheight of Exposed 30 in PP	167	0.10	0.06	0.00	0.00	0.00	0.00
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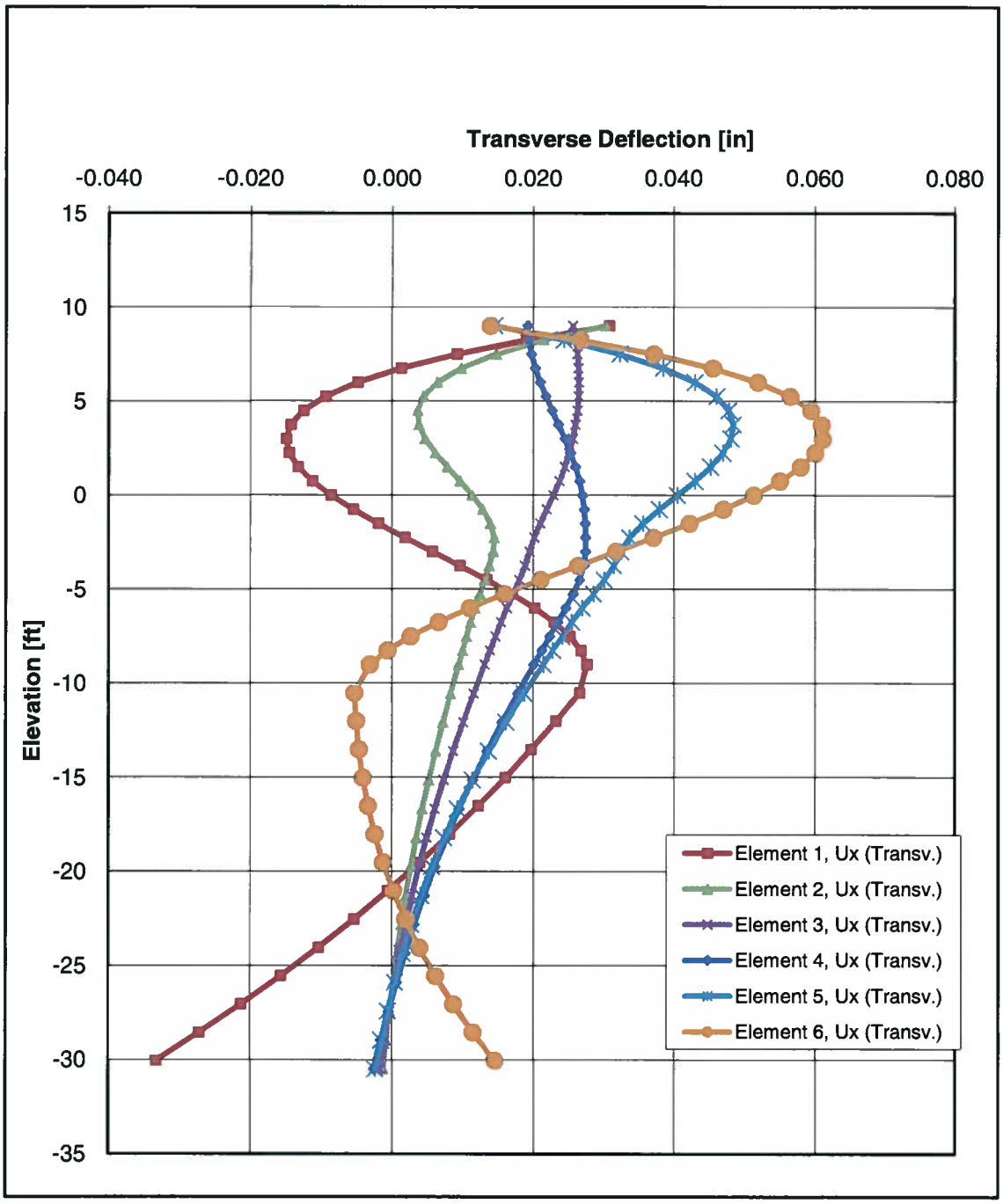
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR V Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-



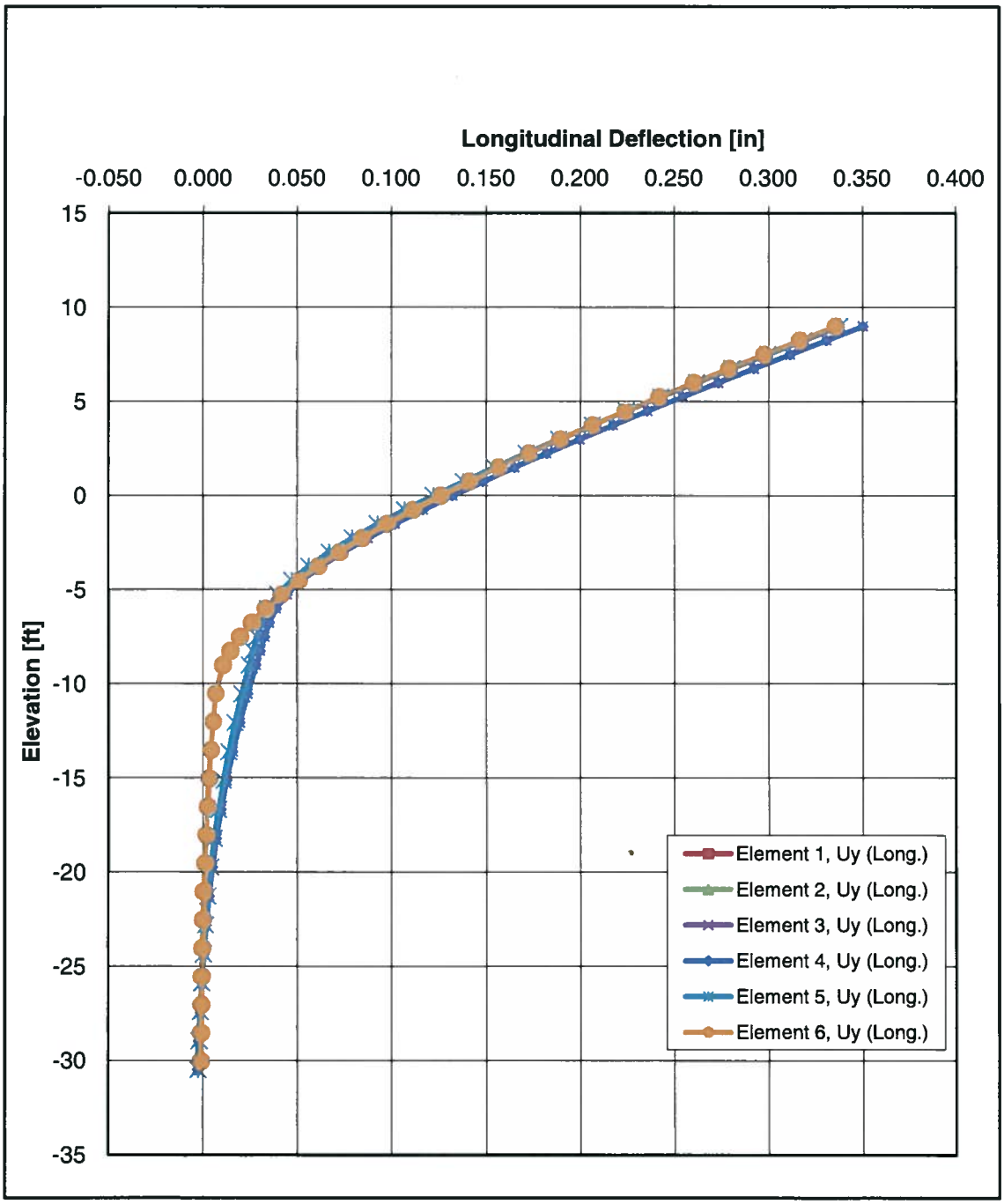
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR V Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-



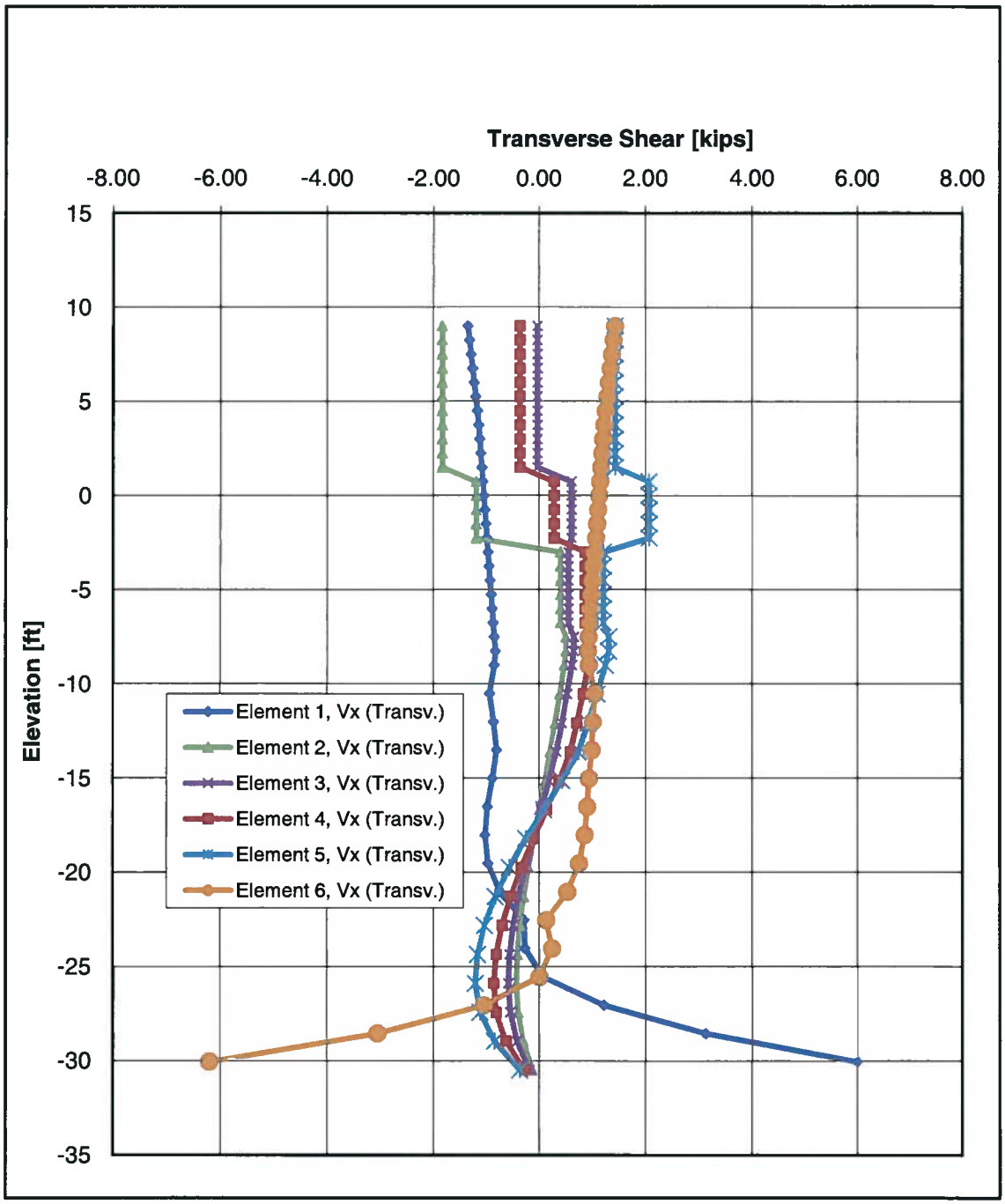
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR V Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-



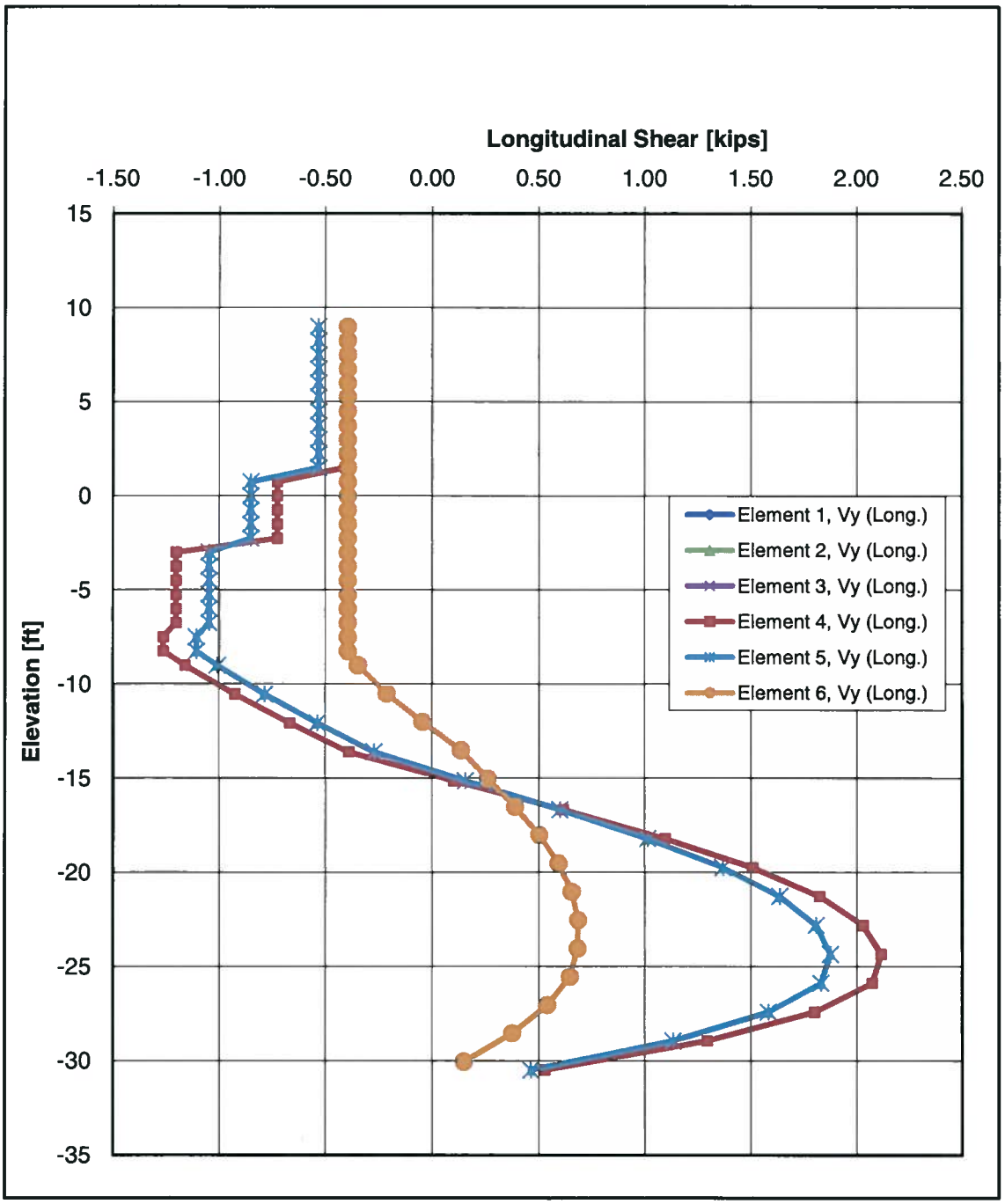
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR V Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-



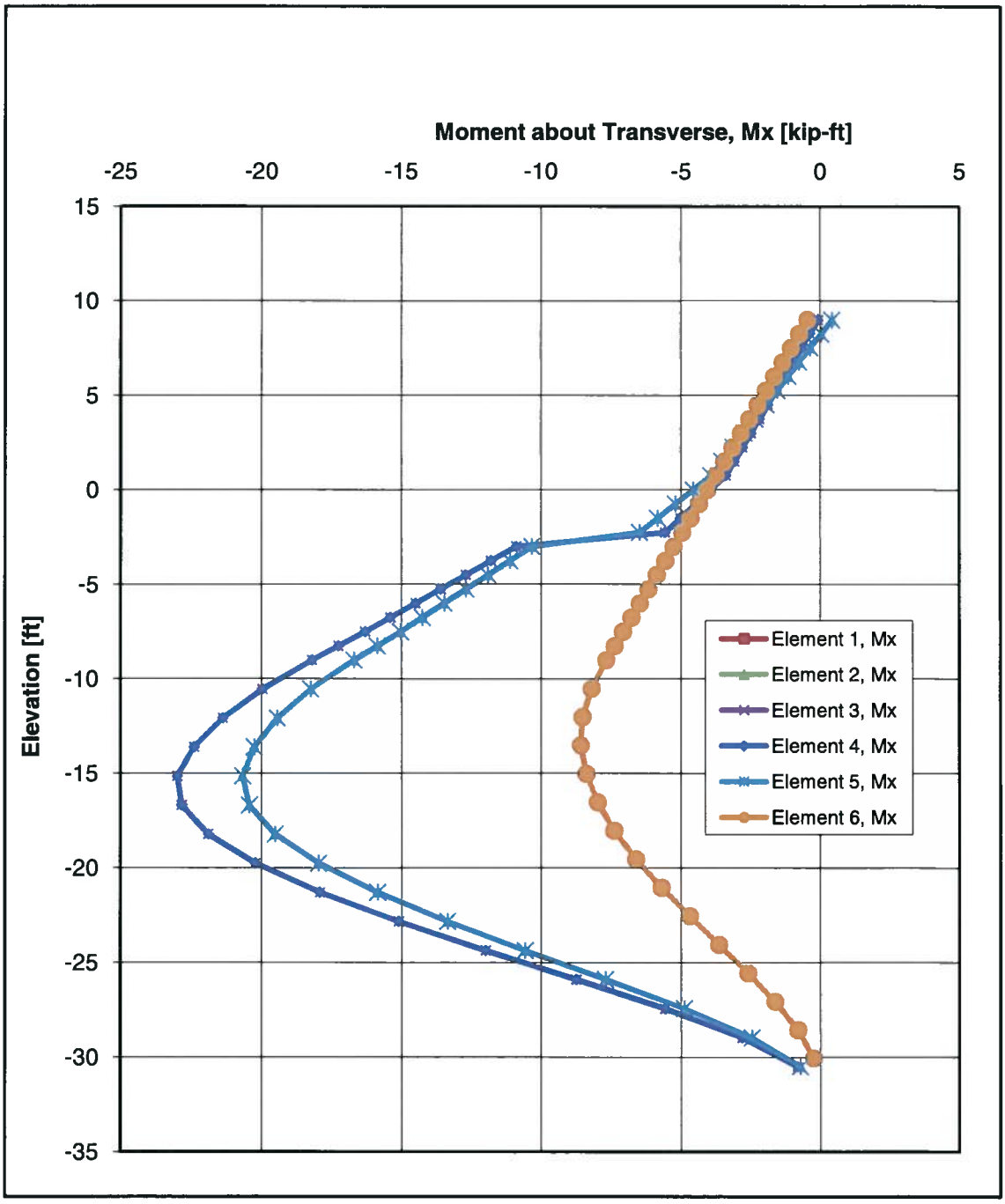
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR V Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-



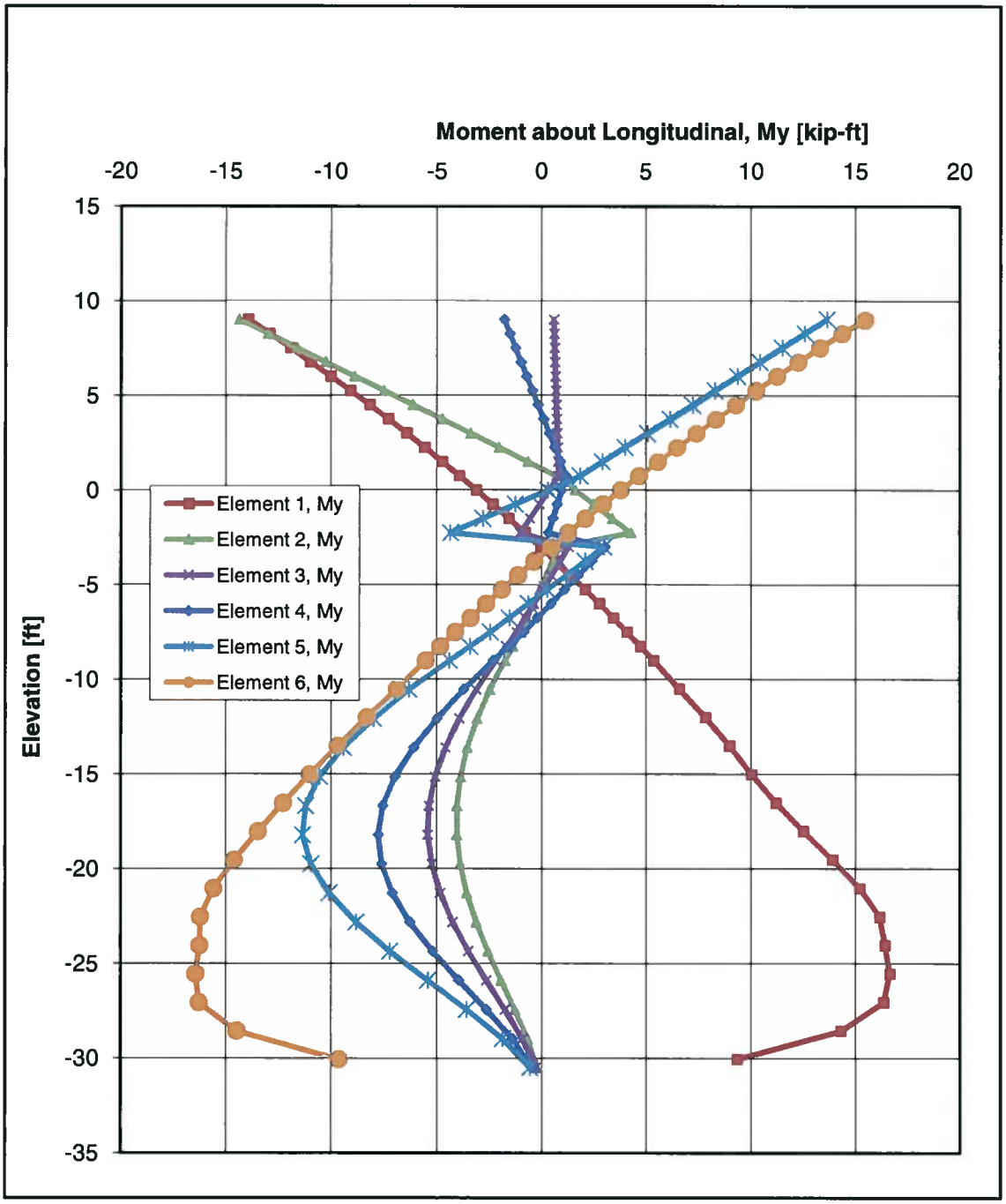
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

STR V Wind at 60 (NO SCOUR)

FB-MultiPier Results for Bent 1 (30x0.5 in PP & 30x0.5 in Battered PP, with concrete)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 1\2011-1018-HAI-Bent 1-30x0_5 in PP+Battered 30x0_5 PP-NO SCOUR-STR V Wind at 60-WITH CONCRETE-



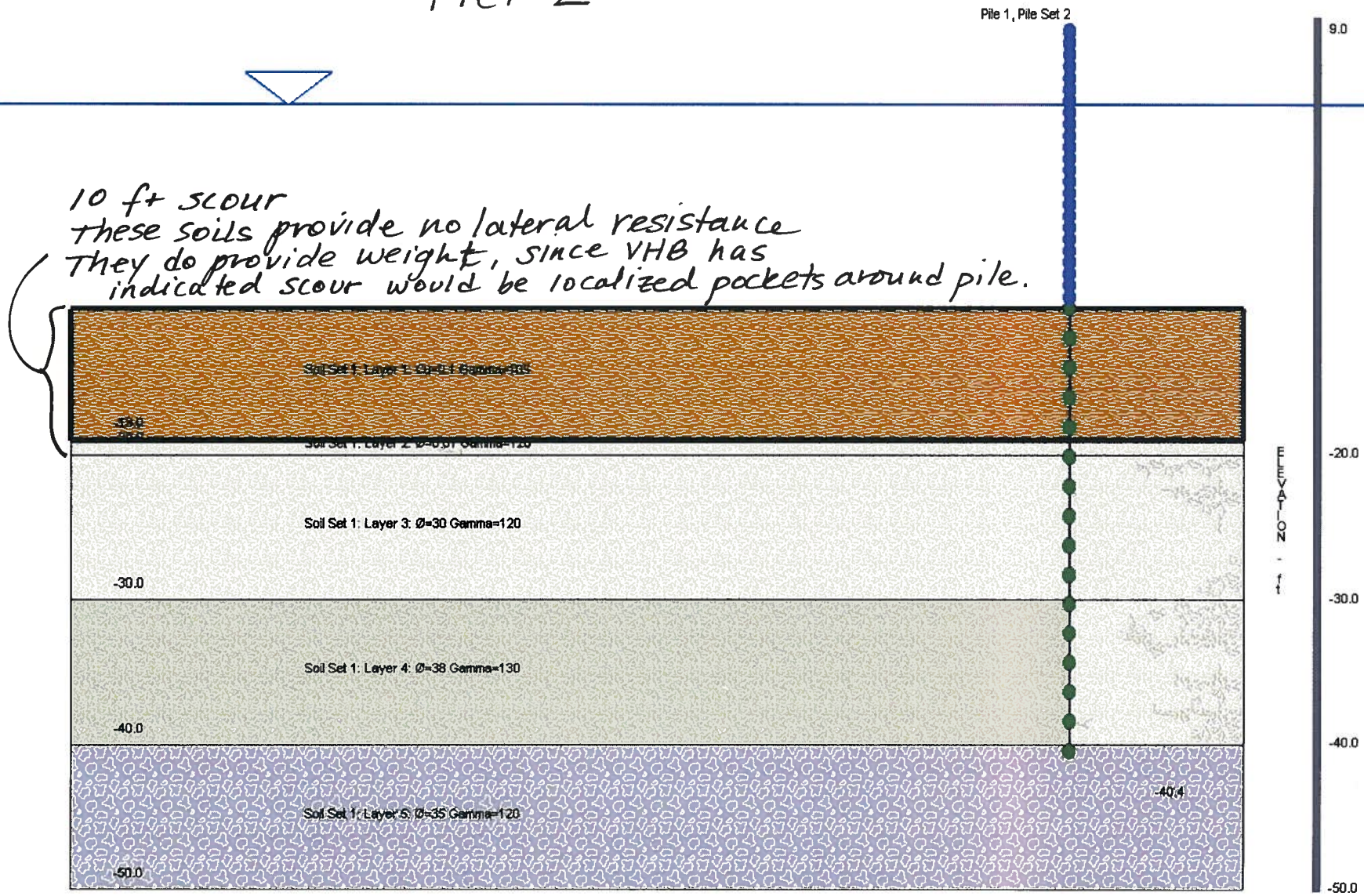
FB-Multipier File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CON

File View Control Wizard Help

Pier #1 | 1 | Load Case

Pier 2

10 ft scour
these soils provide no lateral resistance
They do provide weight, since VHB has
indicated scour would be localized pockets around pile.



HALEY & ALDRICH	CALCULATIONS	File No.	36654-001
		Sheet	1 of 1
Client	VHB	Date	18OCT2011
Project	Sewall's Bridge	Computed by	JLL
Subject	**Bent 2 Summary of Results**	Checked by	EFW

Files: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1010-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML
G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML
G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

FB-MultiPier Results for Bent 2

SUMMARY OF RESULTS

AASHTO-LRFD Limit State	Plumb Pile/ Concrete Fill	Batt. Pile/ Concrete Fill	Scour	δx (1) [in]	δy (1) [in]	$V_{x\,max}$ [kip]	$V_{y\,max}$ [kip]	$M_{x\,max}$ (2) [kip-ft]	$M_{y\,max}$ (2) [kip-ft]	N_{max} (3) [kip]	N_{min} (4) [kip]
SER I Wind at 60	30x0.5/ yes	30x0.5/ yes	10 ft	0.07	0.36	3.0	2.8	28	14	57	18
STR III Wind at 0	30x0.5/ yes	30x0.5/ yes	10 ft	0.12	0.10	3.1	1.1	11	27	23	-4
STR V Wind at 60	30x0.5/ yes	30x0.5/ yes	10 ft	0.08	0.45	4.3	3.3	34	16	78	28

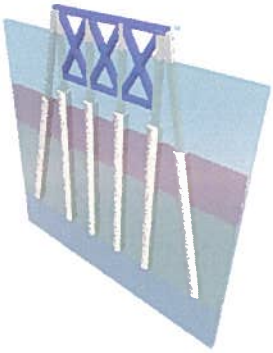
x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1010-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

SER I Wind
at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)



Global Axes

Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
Top of Cap Beam	2	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	3	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	4	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	5	0.30	0.80	62.20	0.00	0.00	0.00
Midheight of 12"x12"	77	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	116	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	155	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	194	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	89	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	128	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	167	0.10	0.06	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	206	0.10	0.06	0.00	0.00	0.00	0.00

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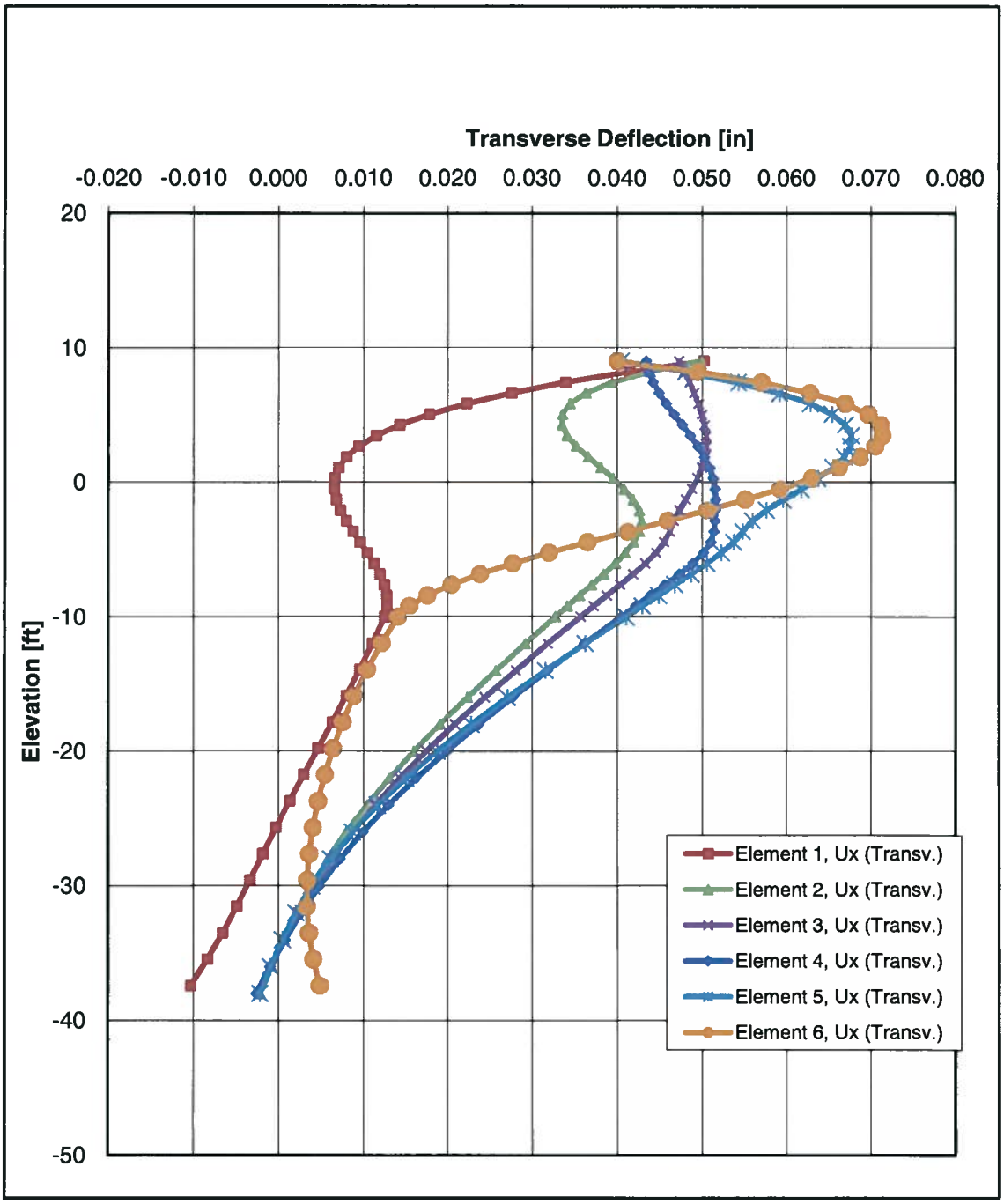
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

SER I Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1010-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-



SER I Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
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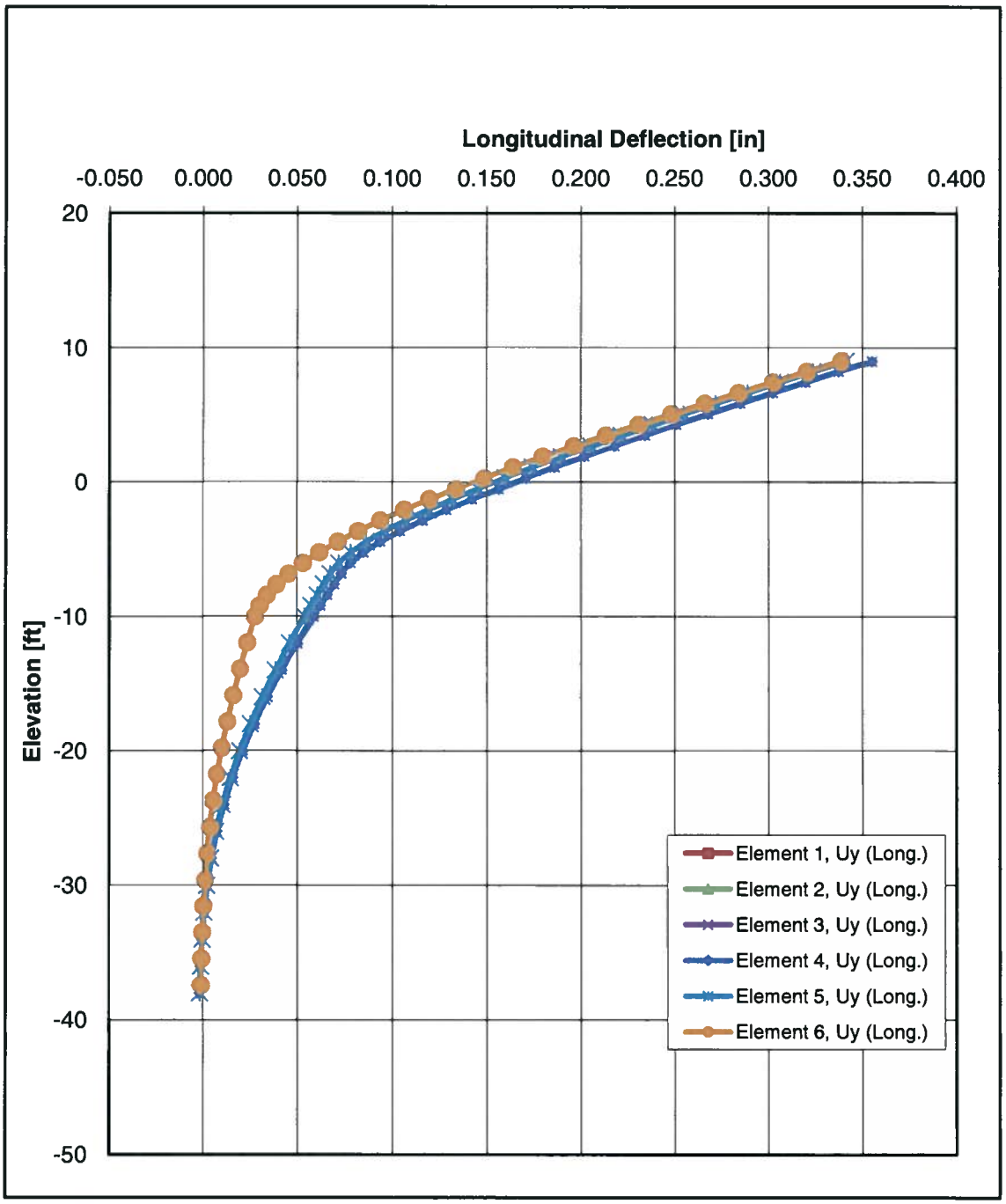
Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1010-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-



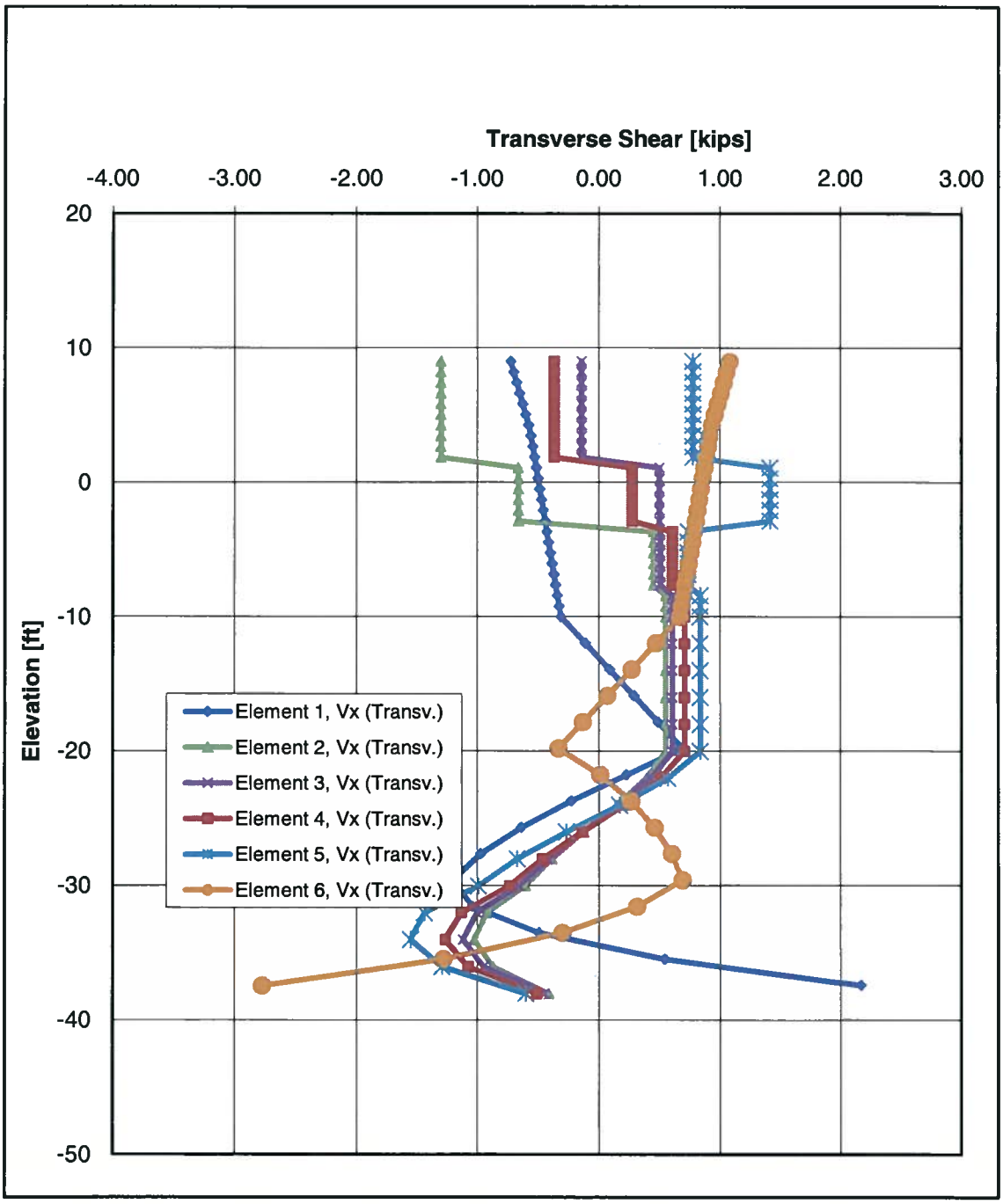
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

SER I Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1010-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-



SER I Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Shear vs. Elevation

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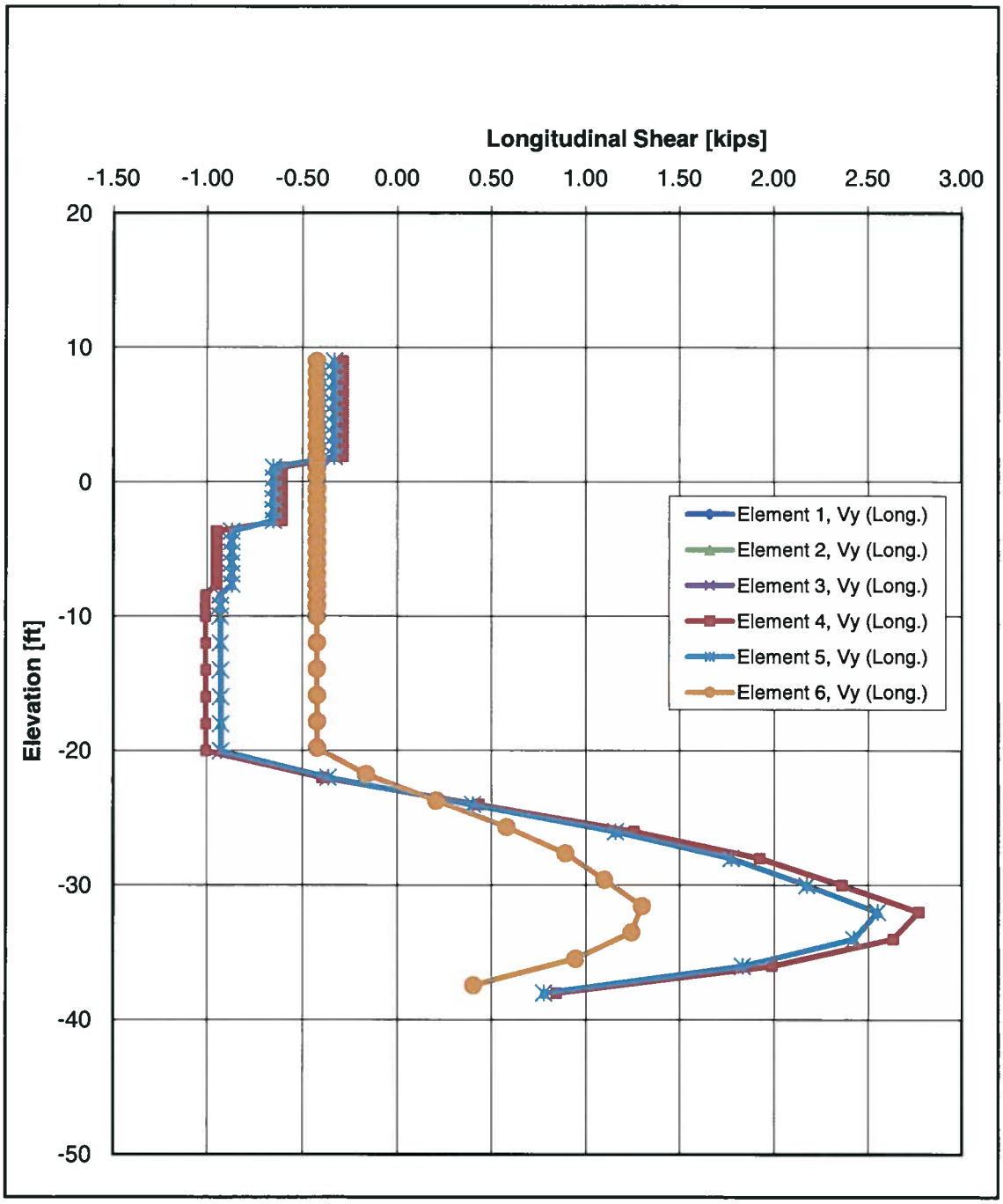


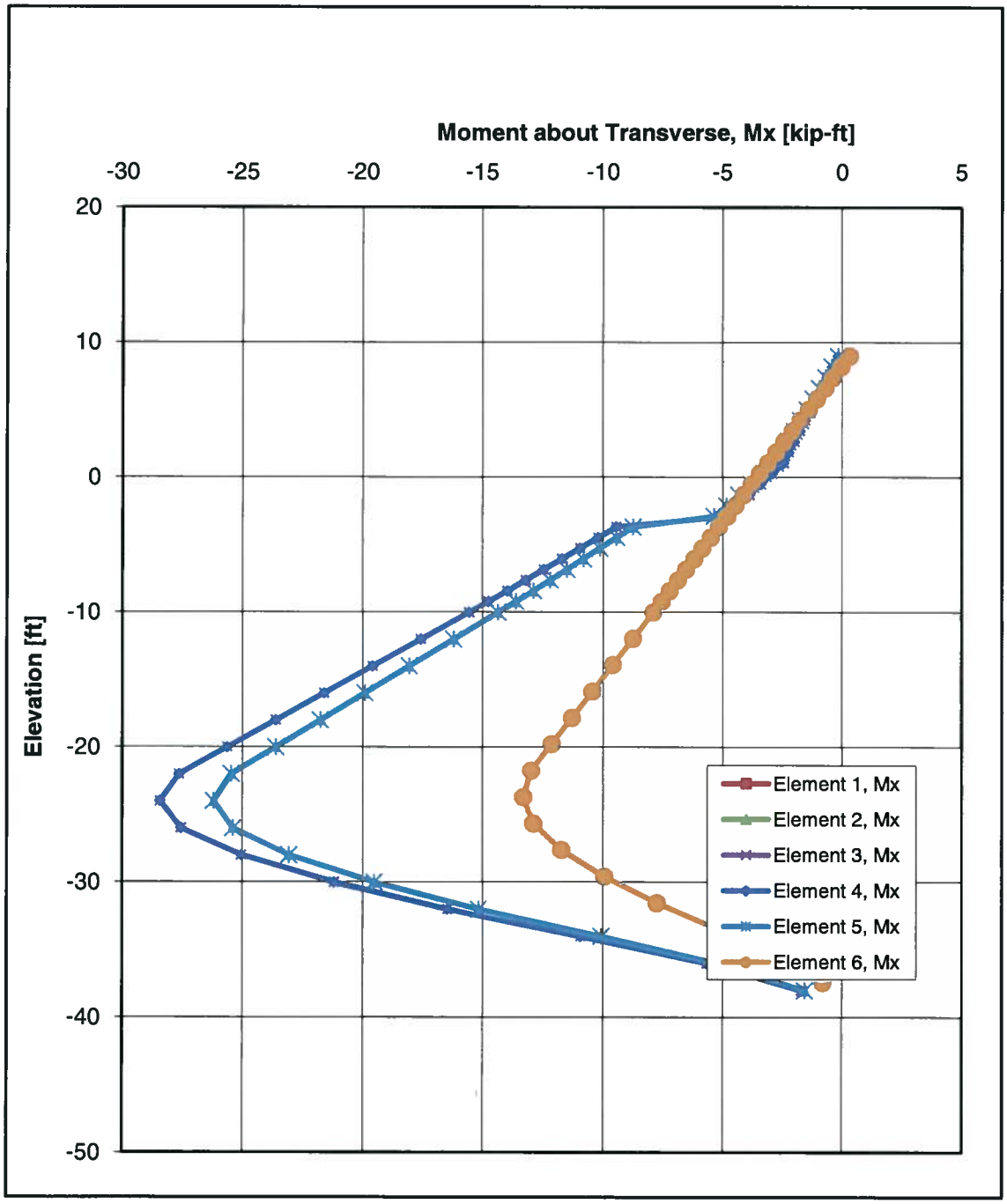
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

SER I Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1010-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-



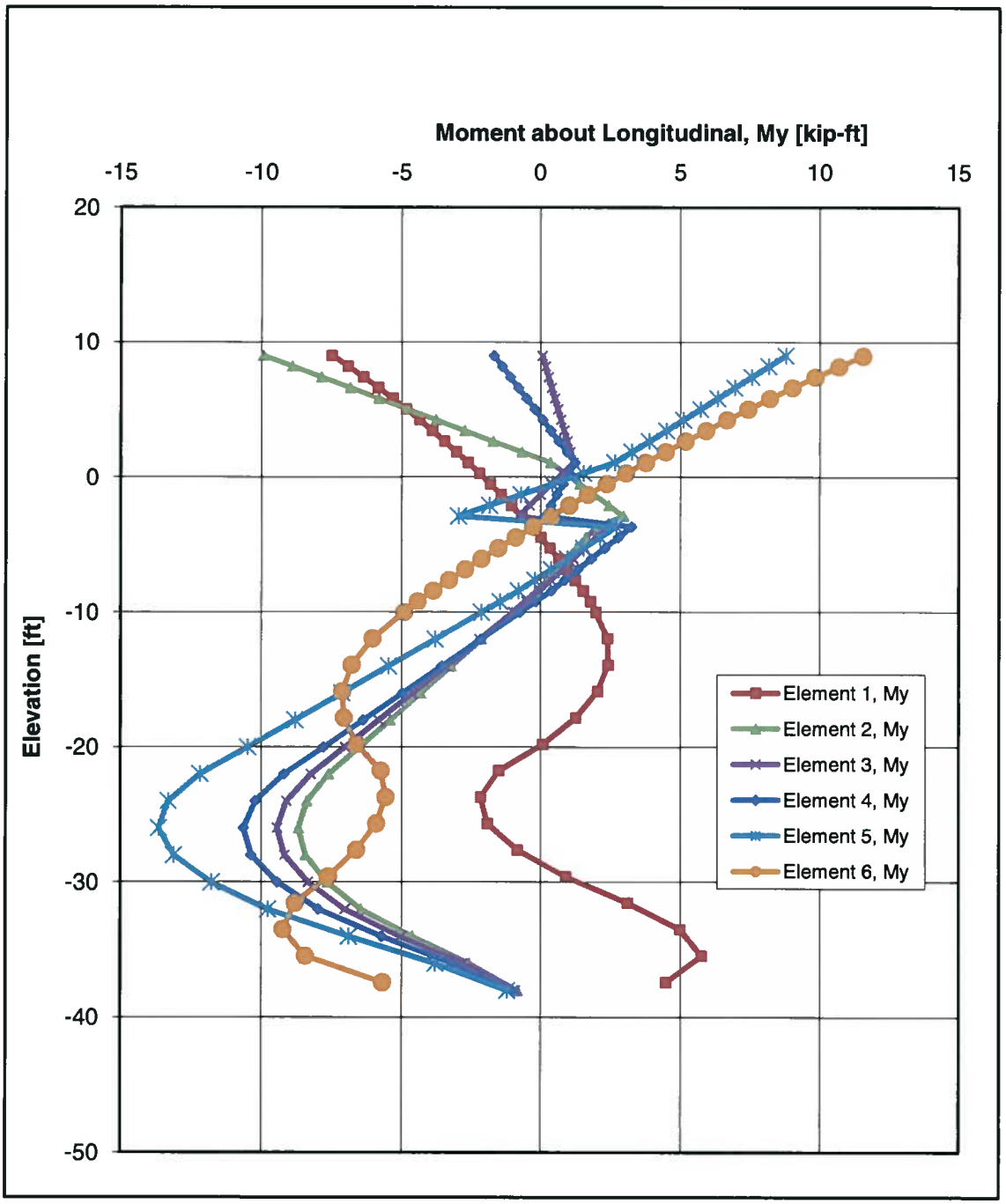
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

SER I Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1010-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-

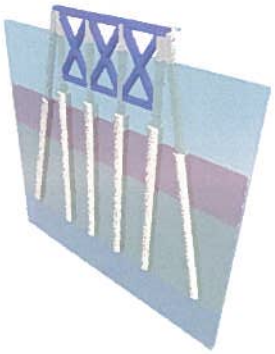


HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML

STR III (Min) Wind at 0 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)



Global Axes

Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
Top of Cap Beam	2	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	3	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	4	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	5	1.70	0.00	7.00	0.00	0.00	0.00
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Midheight of 12"x12"	116	0.64	0.48	0.00	0.00	0.00	0.00
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Midheight of Exposed 30 in PP	167	0.10	0.06	0.00	0.00	0.00	0.00
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z - Vertical direction

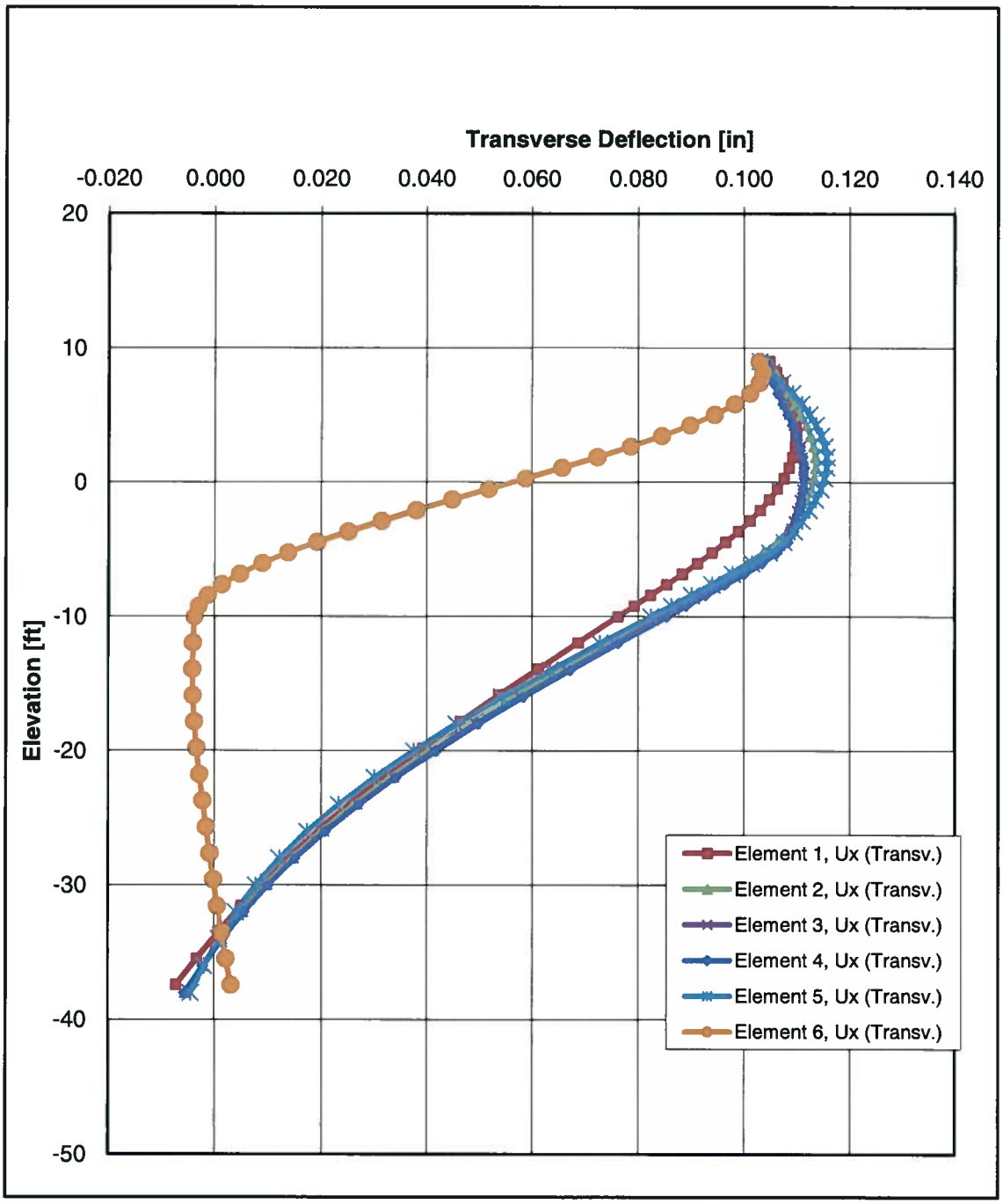
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR III (Min) Wind at 0 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRE



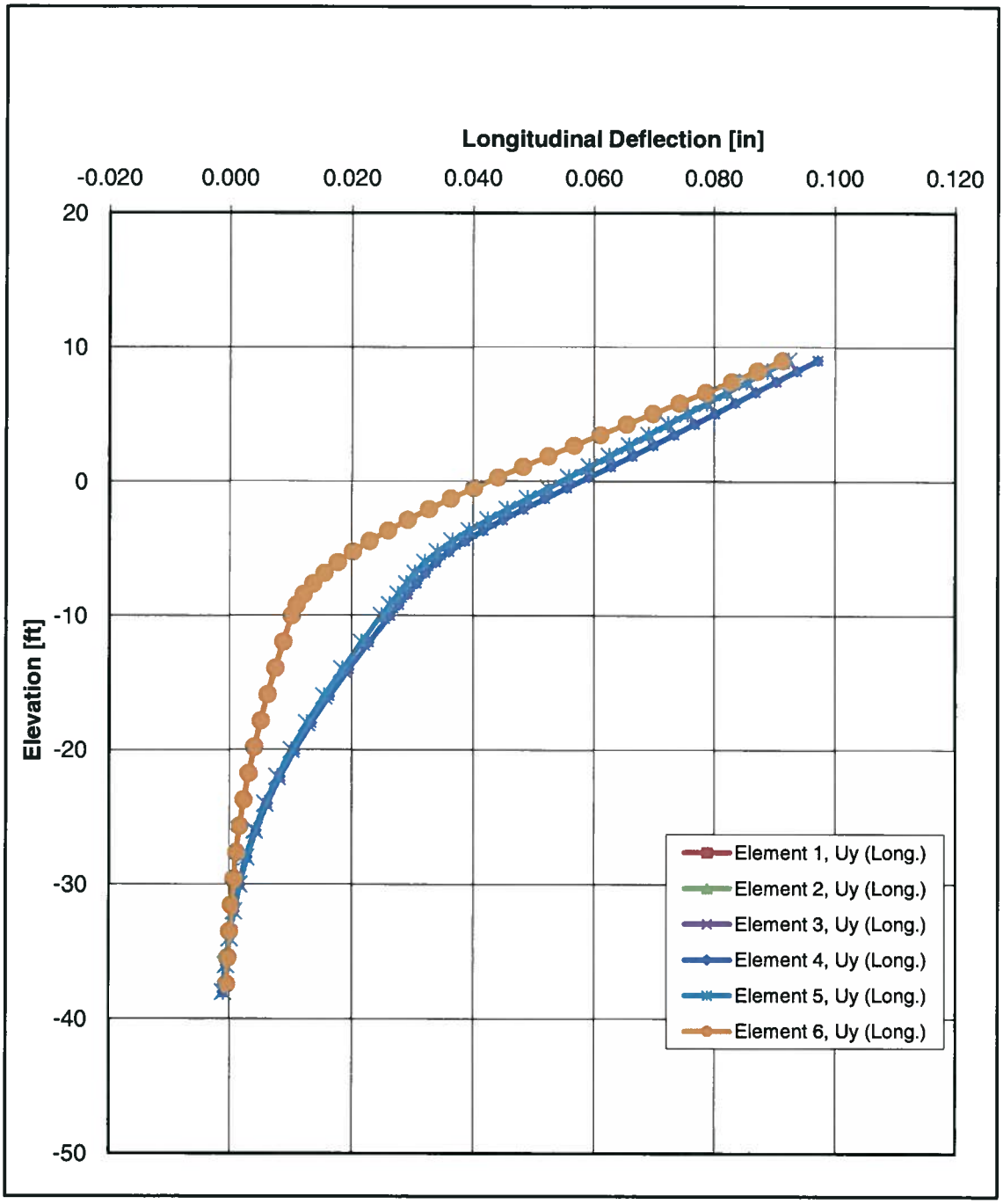
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR III (Min) Wind at 0 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRE



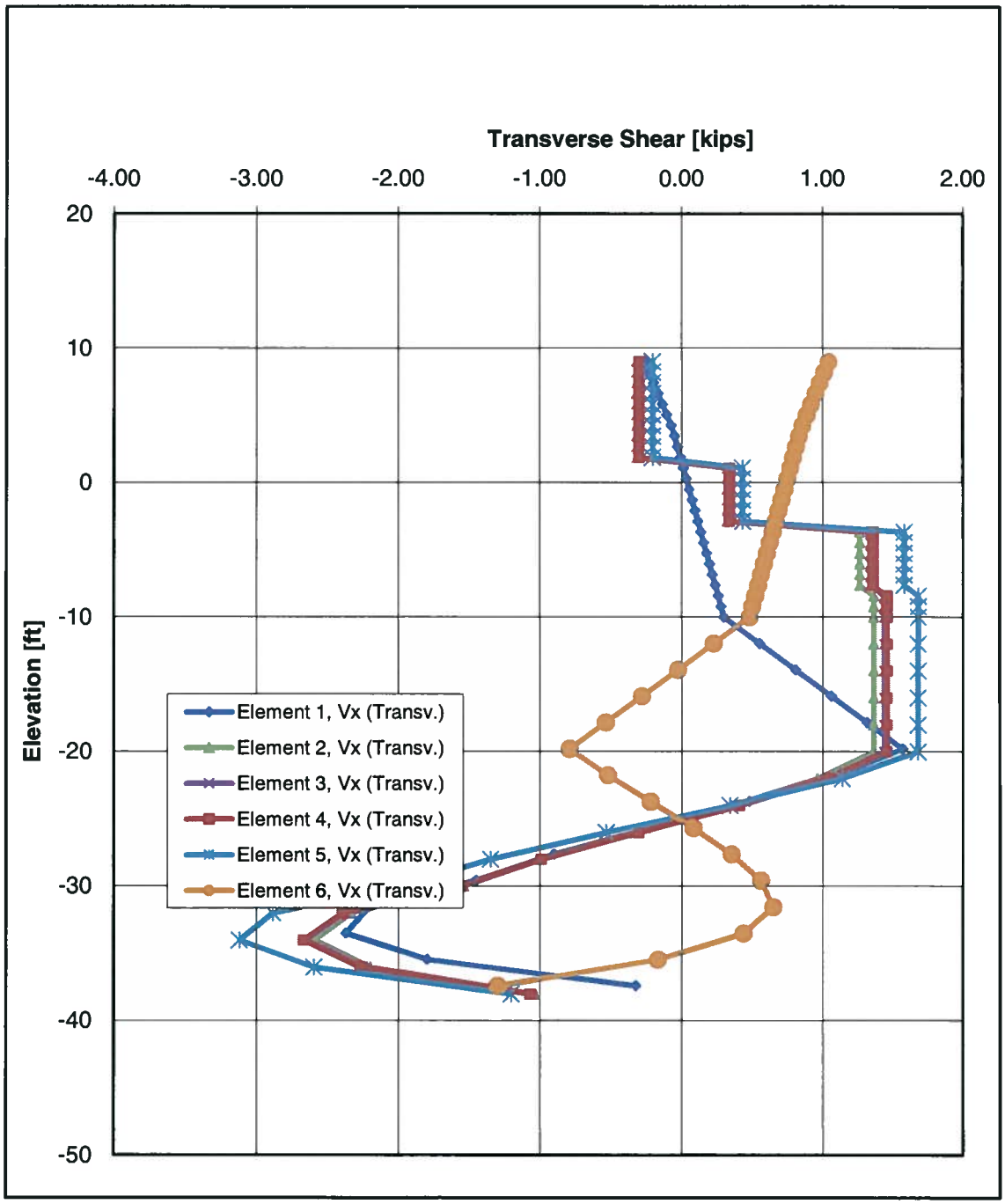
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR III (Min) Wind at 0 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRE

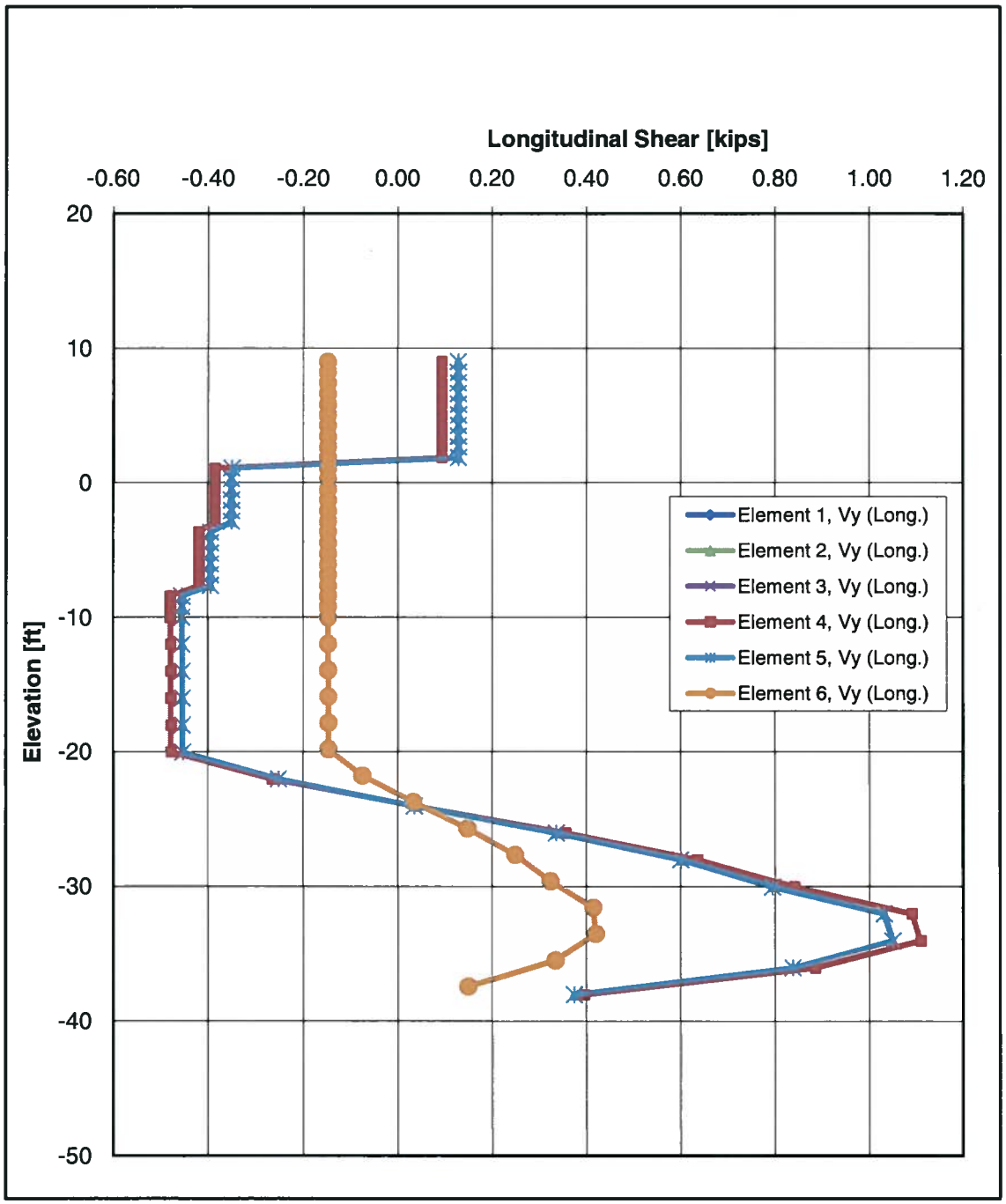


STR III (Min) Wind at 0 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRE



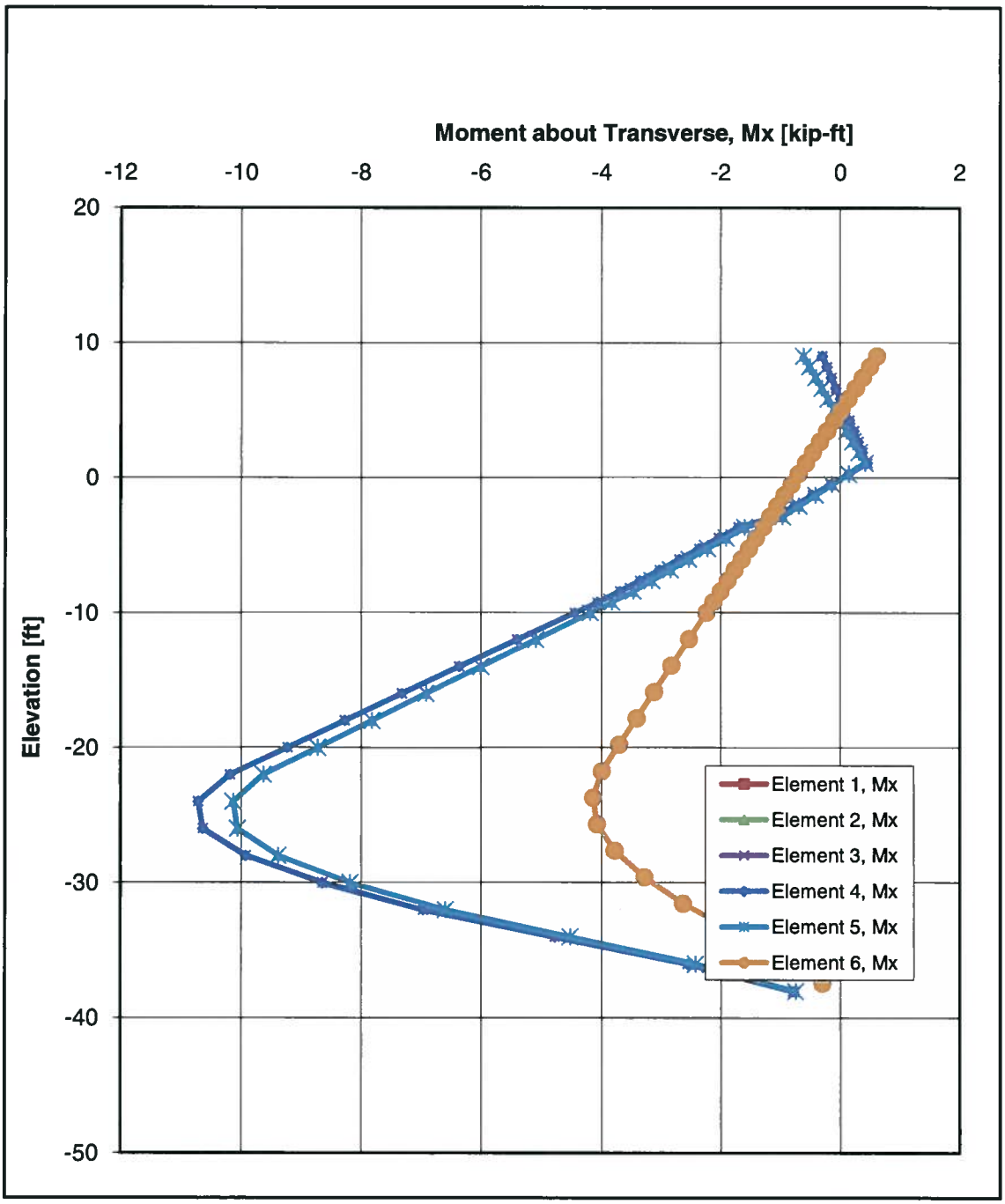
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR III (Min) Wind at 0 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRE



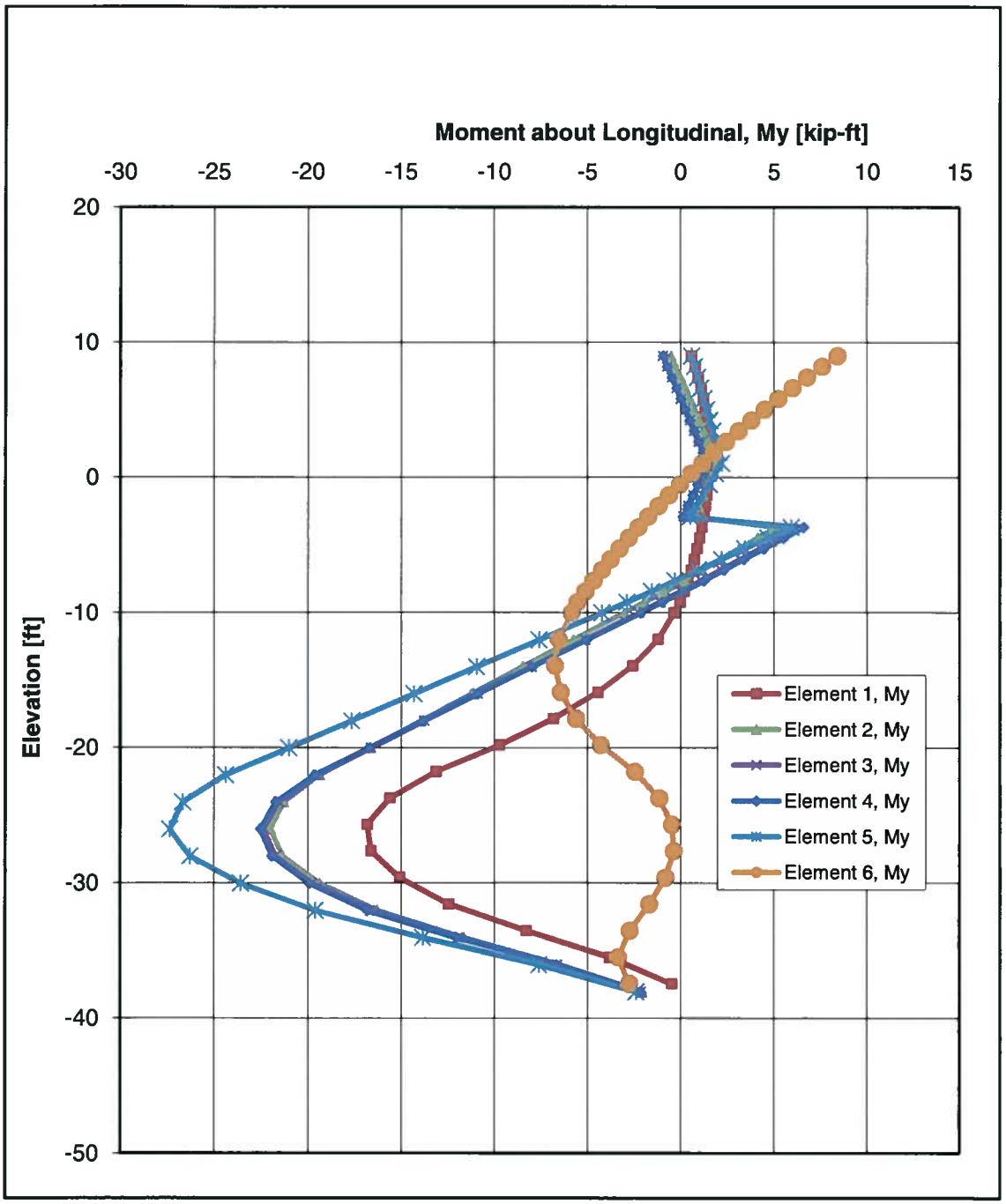
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR III (Min) Wind at 0 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRE

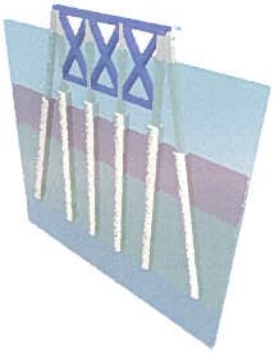


HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

STR V (Max) Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)



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x - Bridge transverse direction
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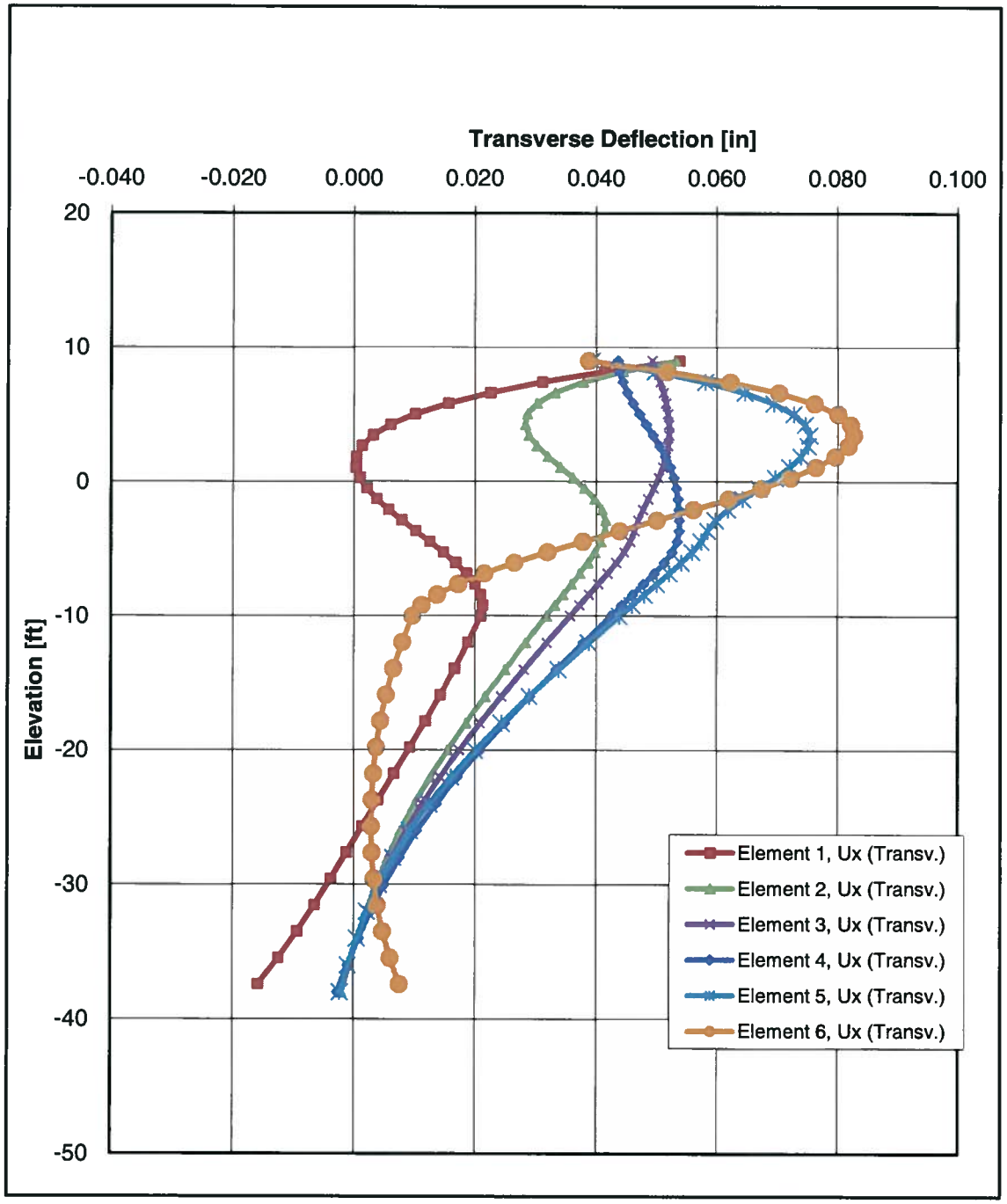
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR V (Max) Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCF

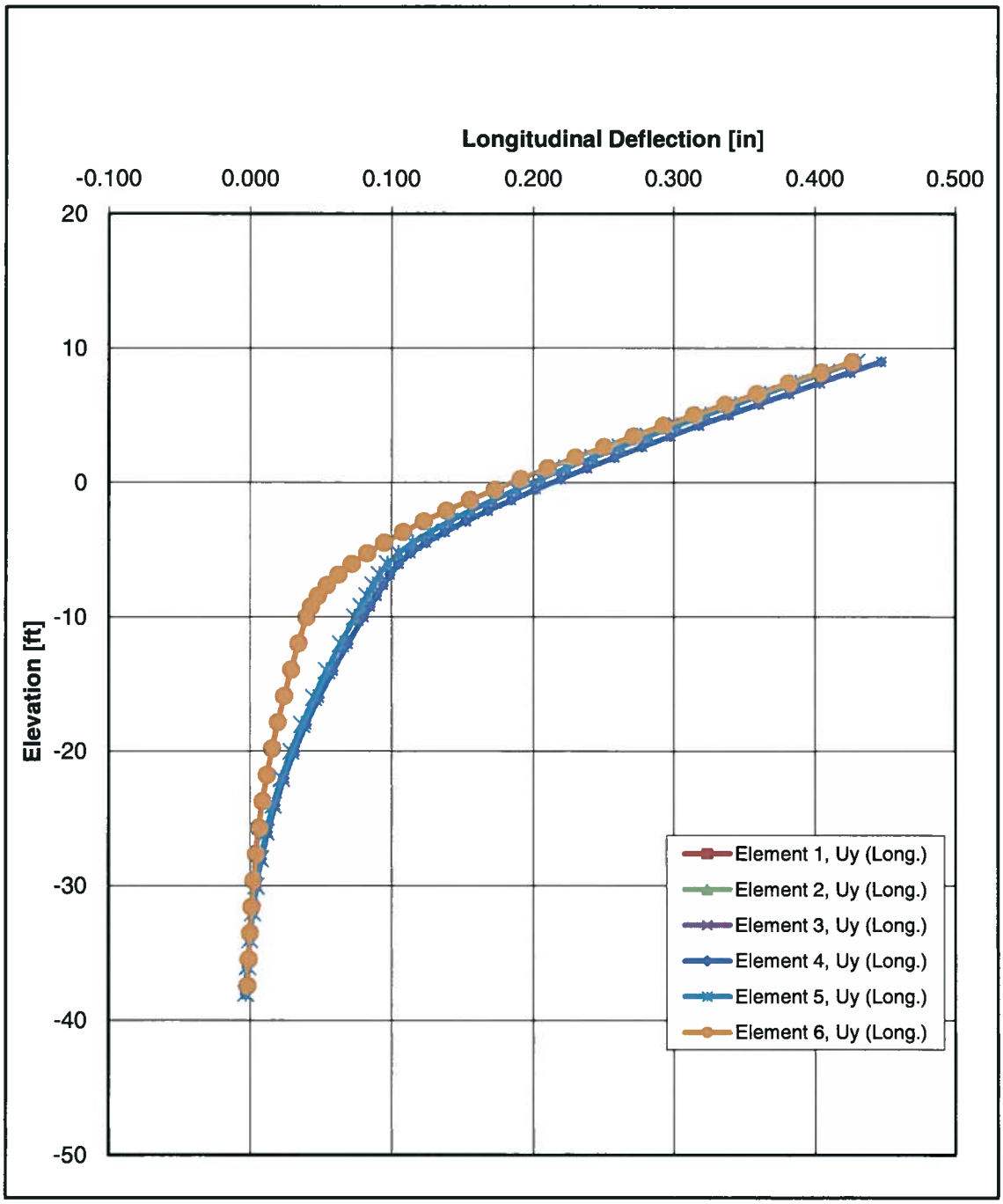


STR V (Max) Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCF



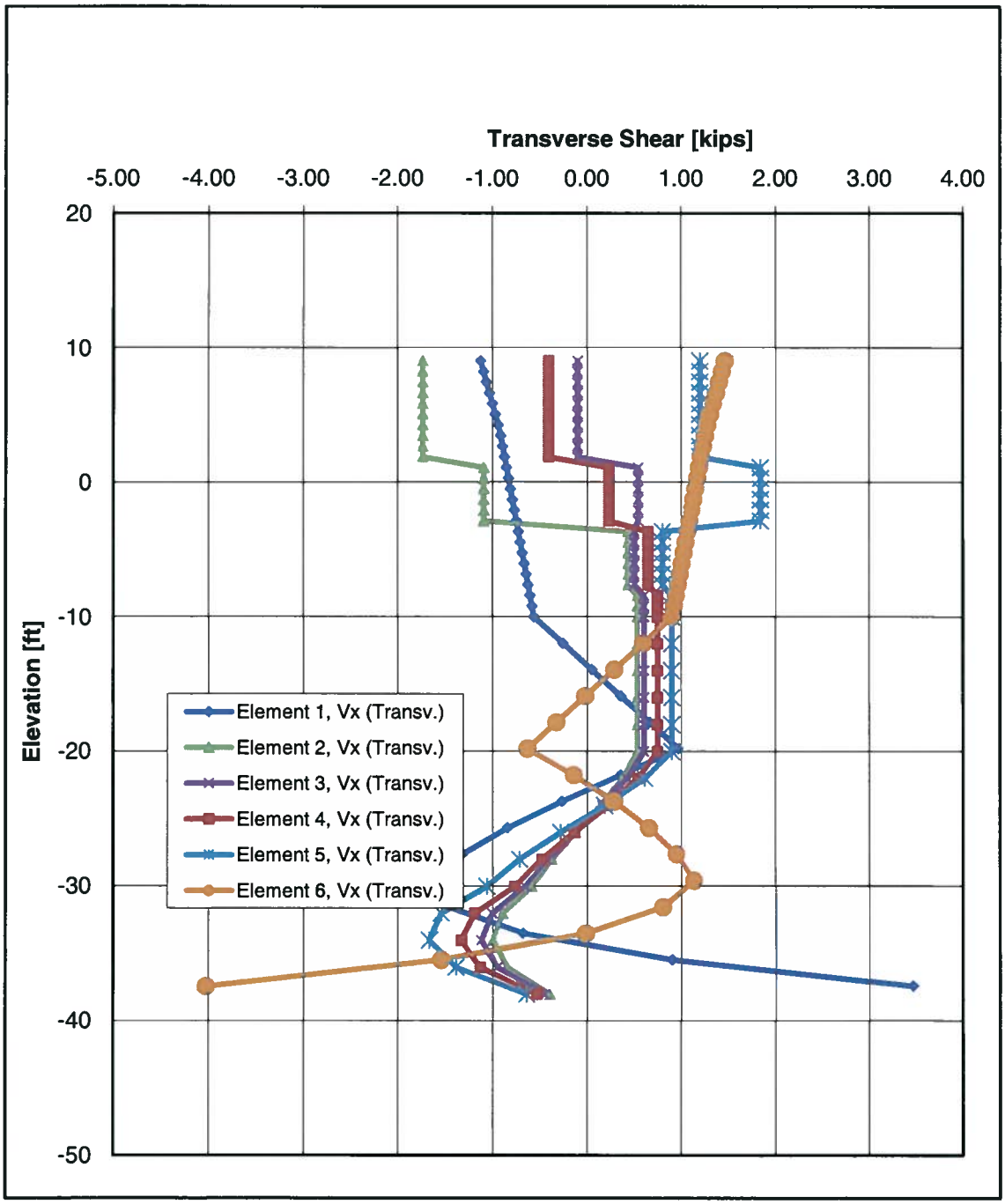
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR V (Max) Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCF

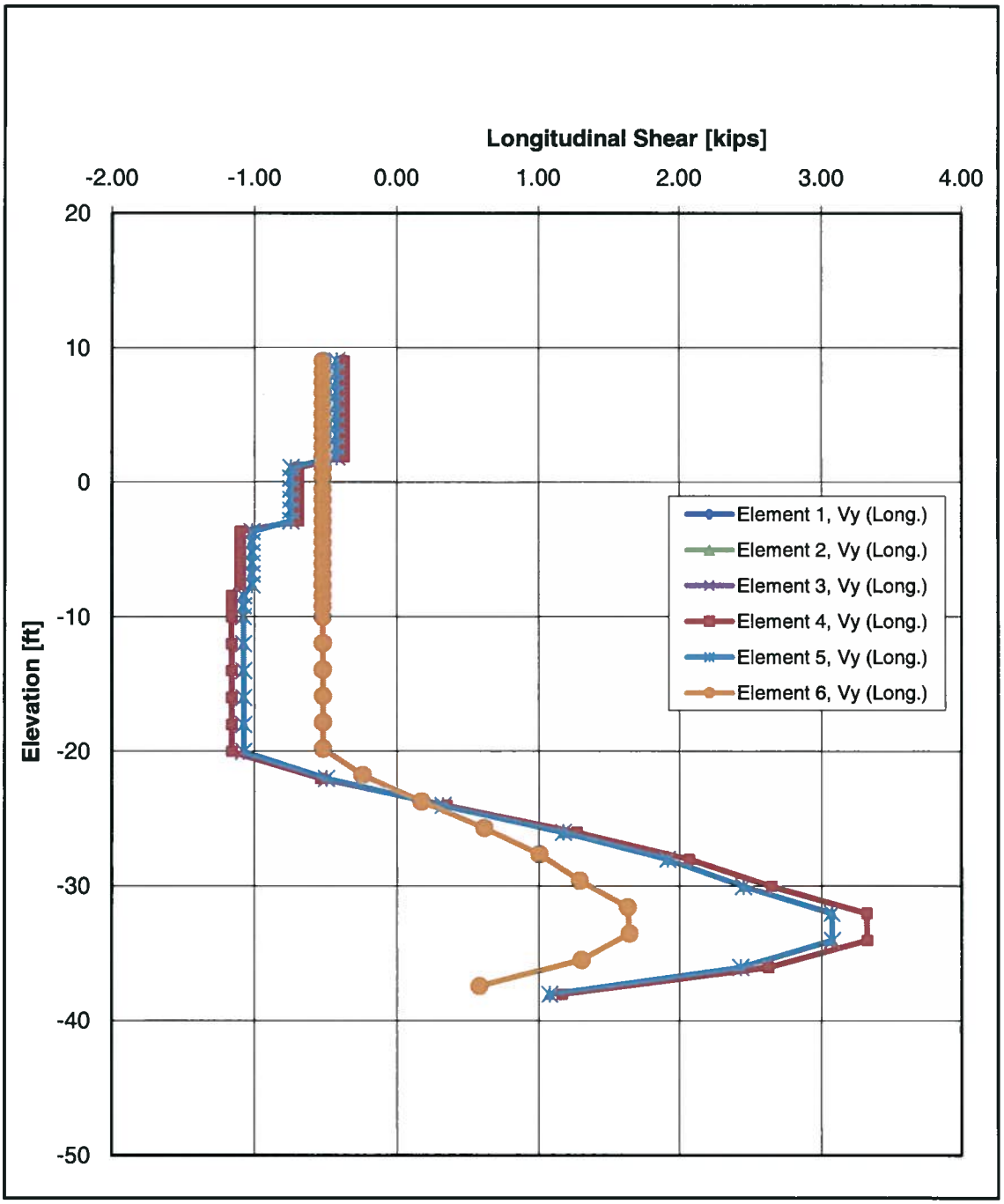


STR V (Max) Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCF

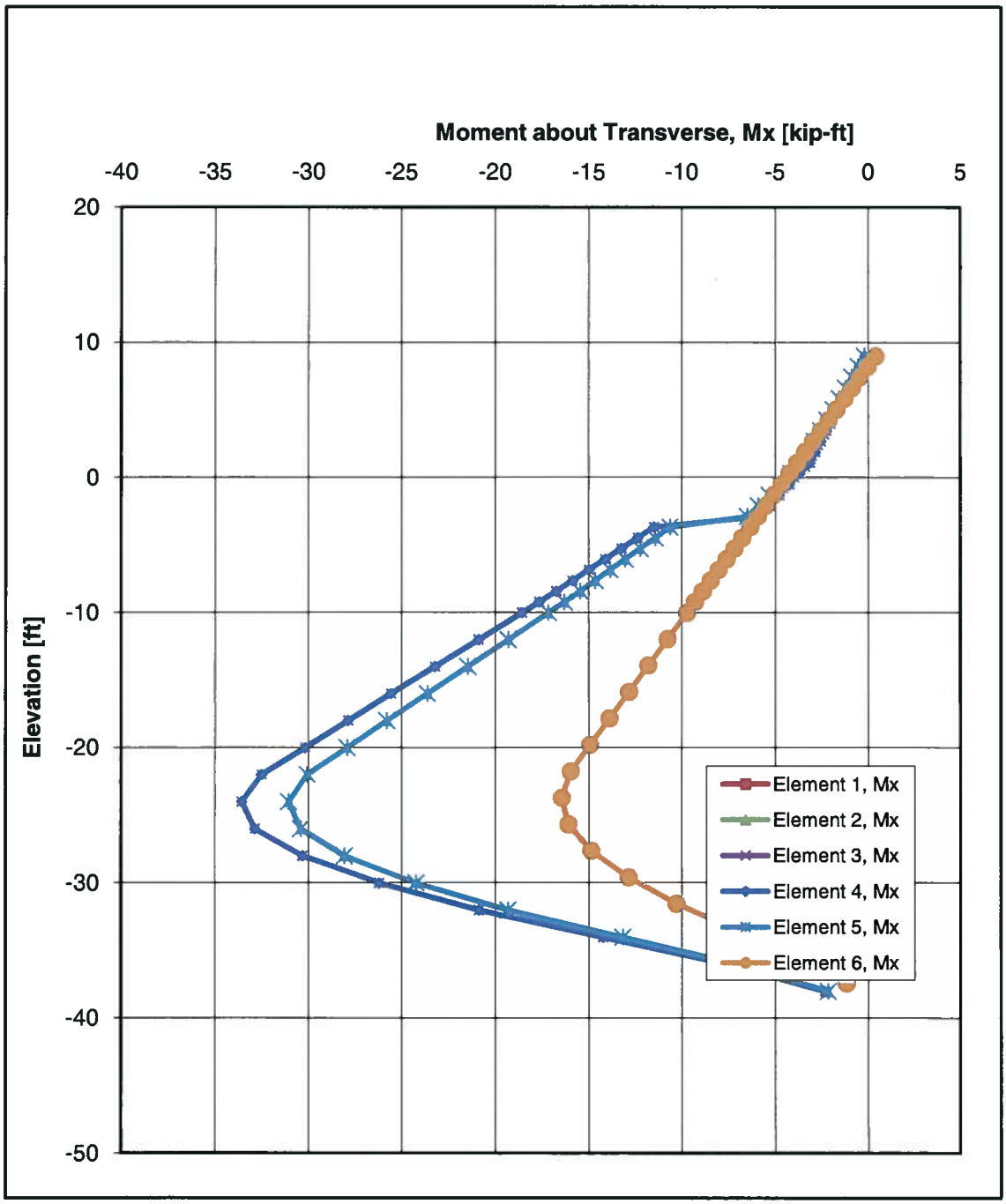


STR V (Max) Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCF



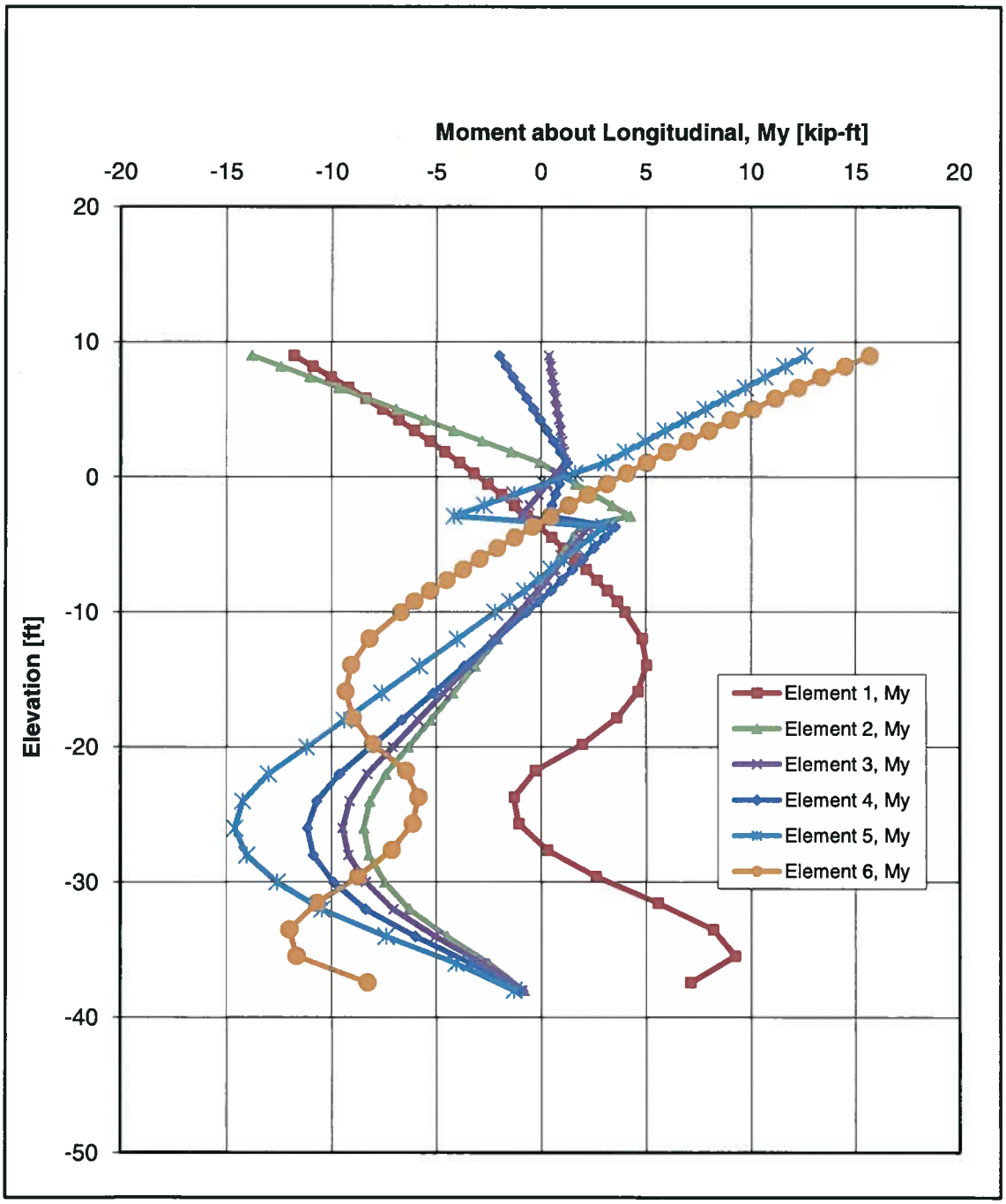
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (With Concrete)	Checked by	EFW

STR V (Max) Wind at 60 (10' SCOUR)

FB-MultiPier Results for Bent 2
(30x0.5 PP with 30x0.5 Battered PP, Piles are concrete-filled)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 2\2011-1018-HAI-Bent 2-30x0_5 in PP+30x0_5 in PP Batter-10 ft SCOUR-STR V Max Wind at 60-WITH CONCF

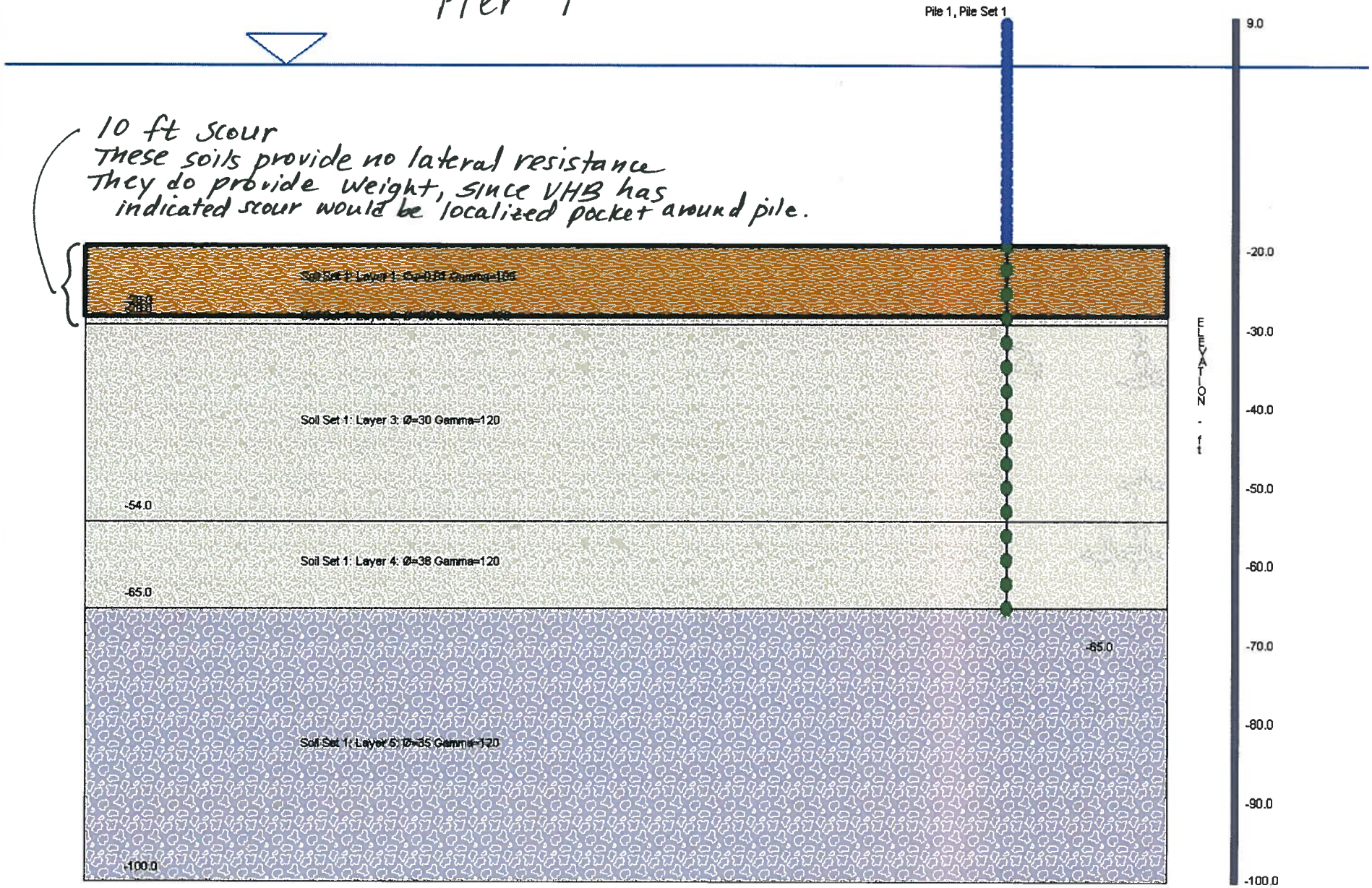


FB-MultiPier File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.in - [Soil]

File View Control Wizard Help

Pier #1 | 1 | Load Case

Pier 7



HALEY & ALDRICH	CALCULATIONS	File No.	36654-001
		Sheet	1 of 1
Client	VHB	Date	18OCT2011
Project	Sewall's Bridge	Computed by	JLL
Subject	**Bent 7 Summary of Results**	Checked by	EFW

Files: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML
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G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

FB-MultiPier Results for Bent 7

SUMMARY OF RESULTS

AASHTO-LRFD Limit State	Plumb Pile/ Concrete Fill	Batt. Pile/ Concrete Fill	Scour	δx (1) [in]	δy (1) [in]	$V_{x\,max}$ [kip]	$V_{y\,max}$ [kip]	$M_{x\,max}$ (2) [kip-ft]	$M_{y\,max}$ (2) [kip-ft]	N_{max} (3) [kip]	N_{min} (4) [kip]
SER I Wind at 60	30x0.5/ yes	none/NA	10 ft	0.19	0.70	2.6	3.4	50	38	85	38
STR III Min Wind at 0	30x0.5/ yes	none/NA	10 ft	0.38	0.22	5.0	1.5	23	76	34	4
STR V Max Wind at 60	30x0.5/ yes	none/NA	10 ft	0.19	0.83	2.6	3.9	58	39	106	47

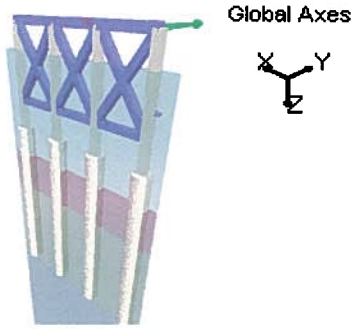
x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

SER I Wind
at 60 (10 ft SCOUR)

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)



Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
Top of Cap Beam	1	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	2	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	3	0.30	0.80	62.20	0.00	0.00	0.00
Top of Cap Beam	4	0.30	0.80	62.20	0.00	0.00	0.00
Midheight of 12"x12"	25	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	64	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	103	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	142	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	37	0.60	0.36	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	76	0.60	0.36	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	115	0.60	0.36	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	154	0.60	0.36	0.00	0.00	0.00	0.00

x - Bridge transverse direction
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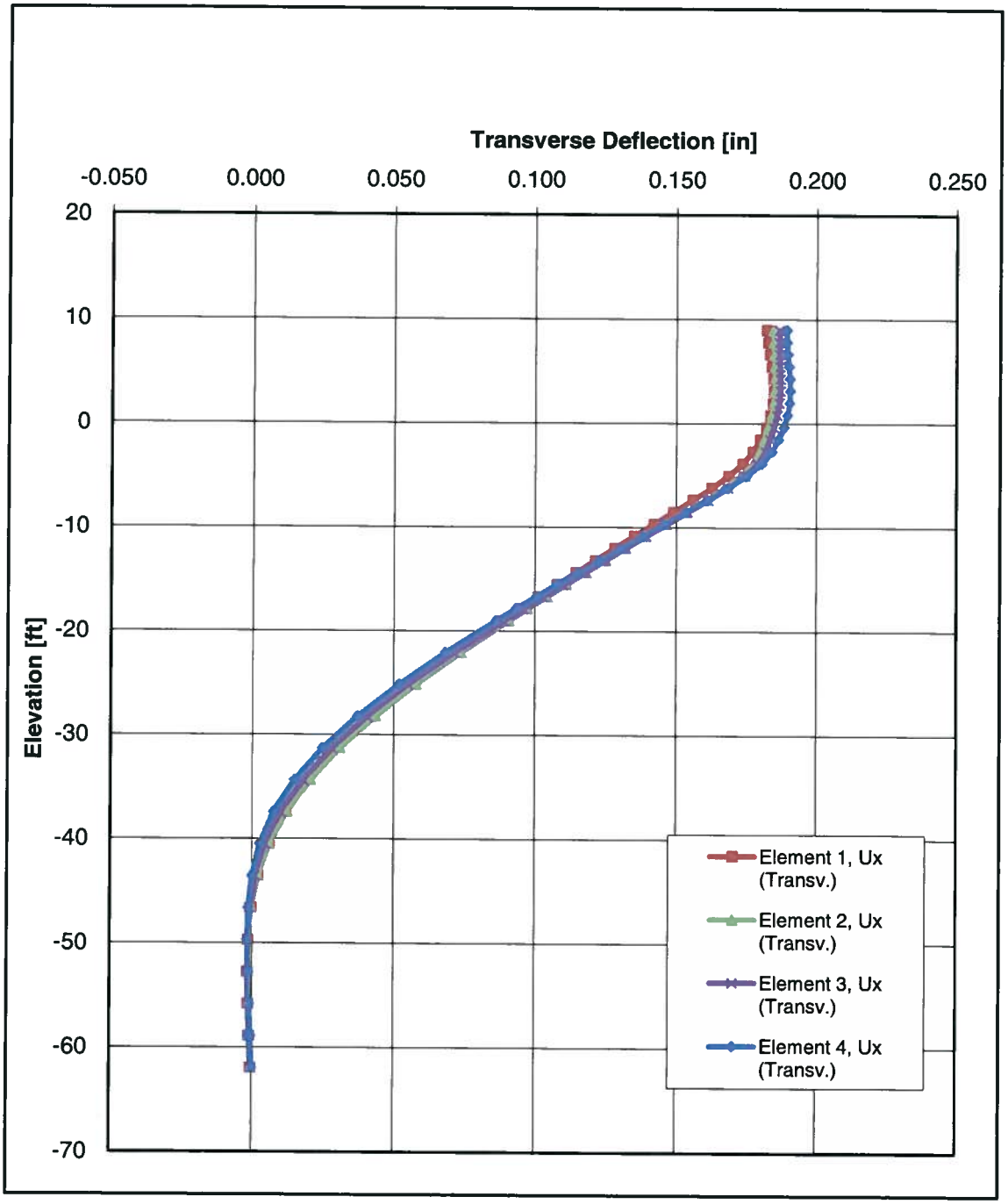
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

SER I Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML



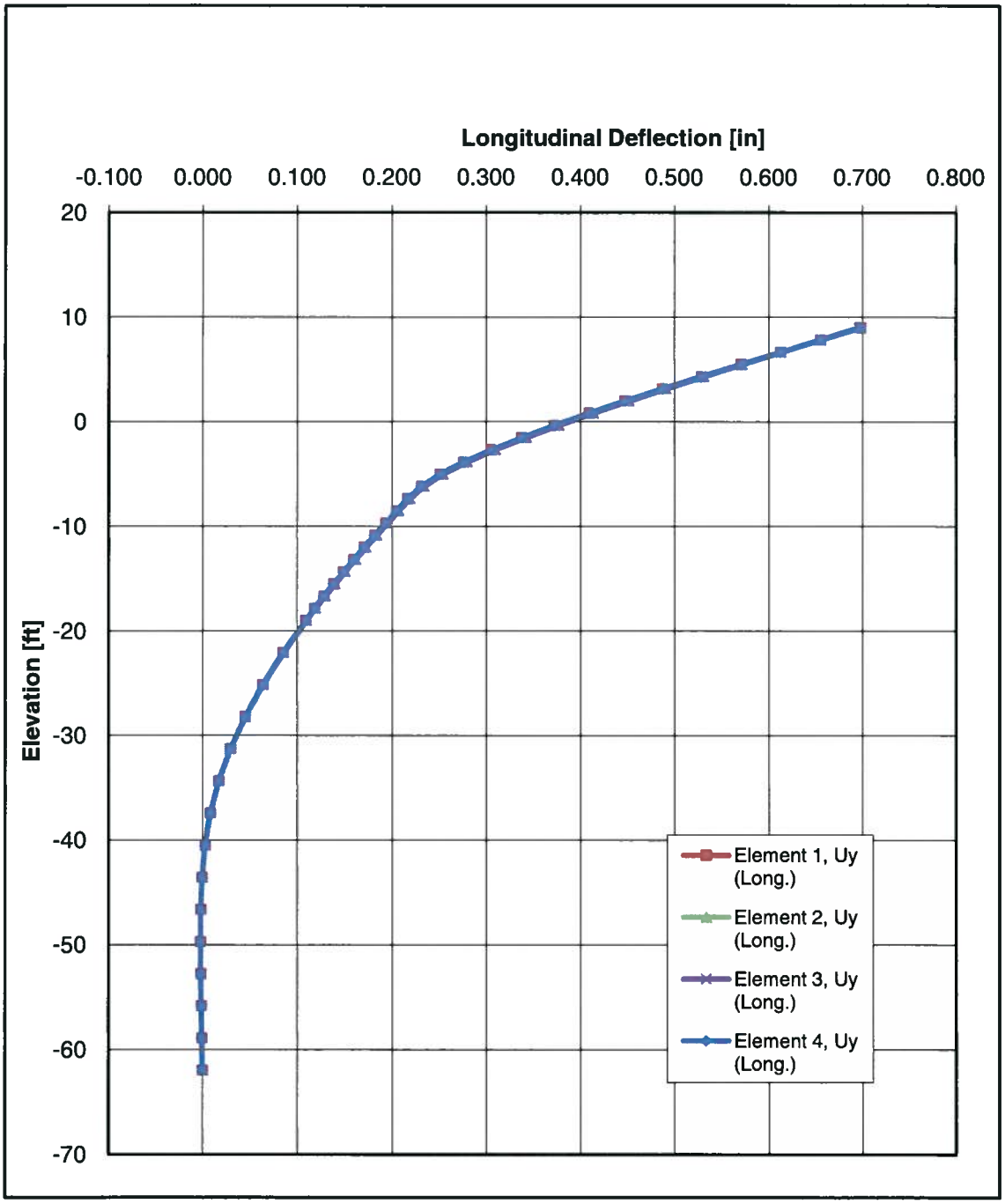
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EPW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

SER I Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML



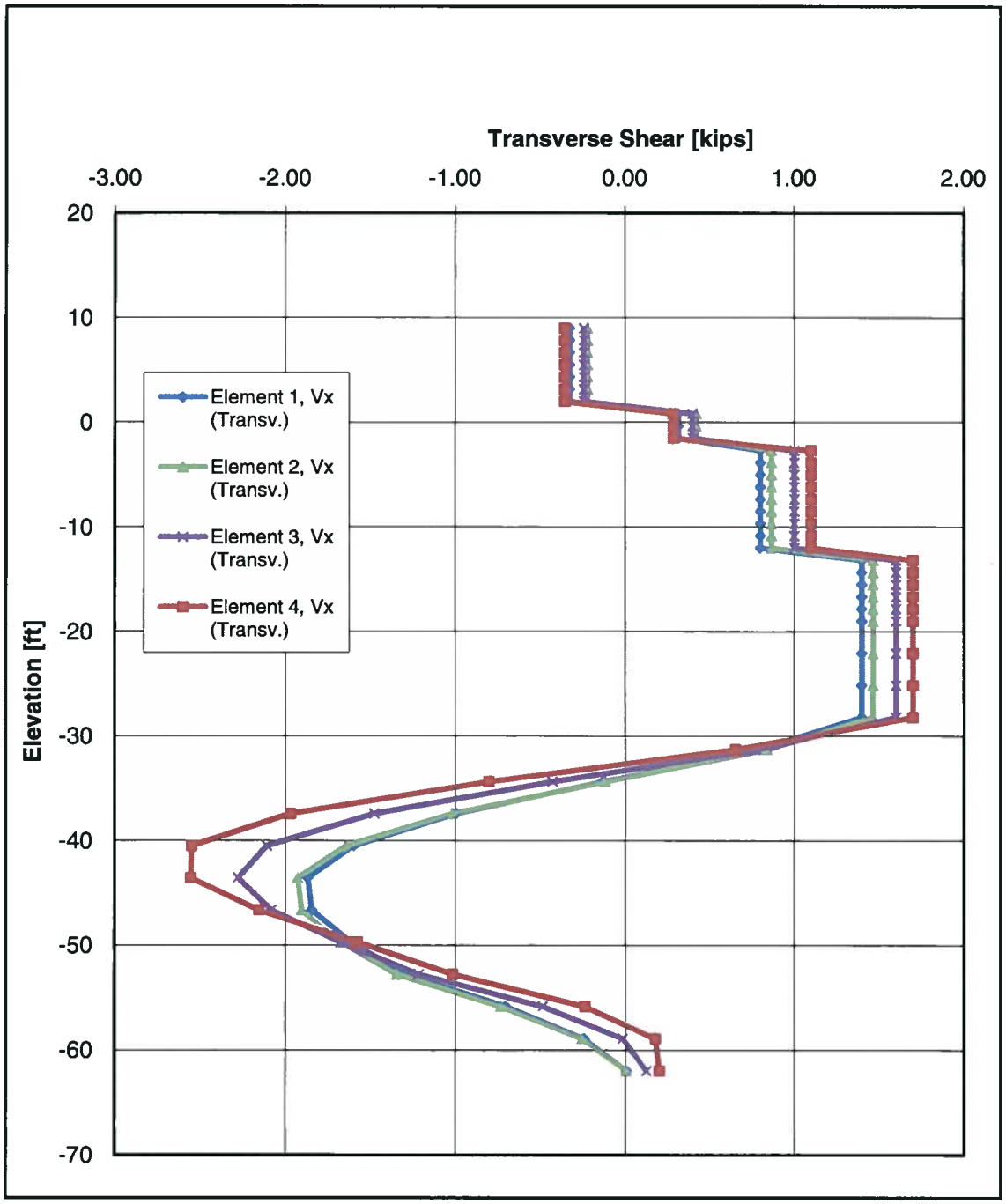
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

SER I Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML



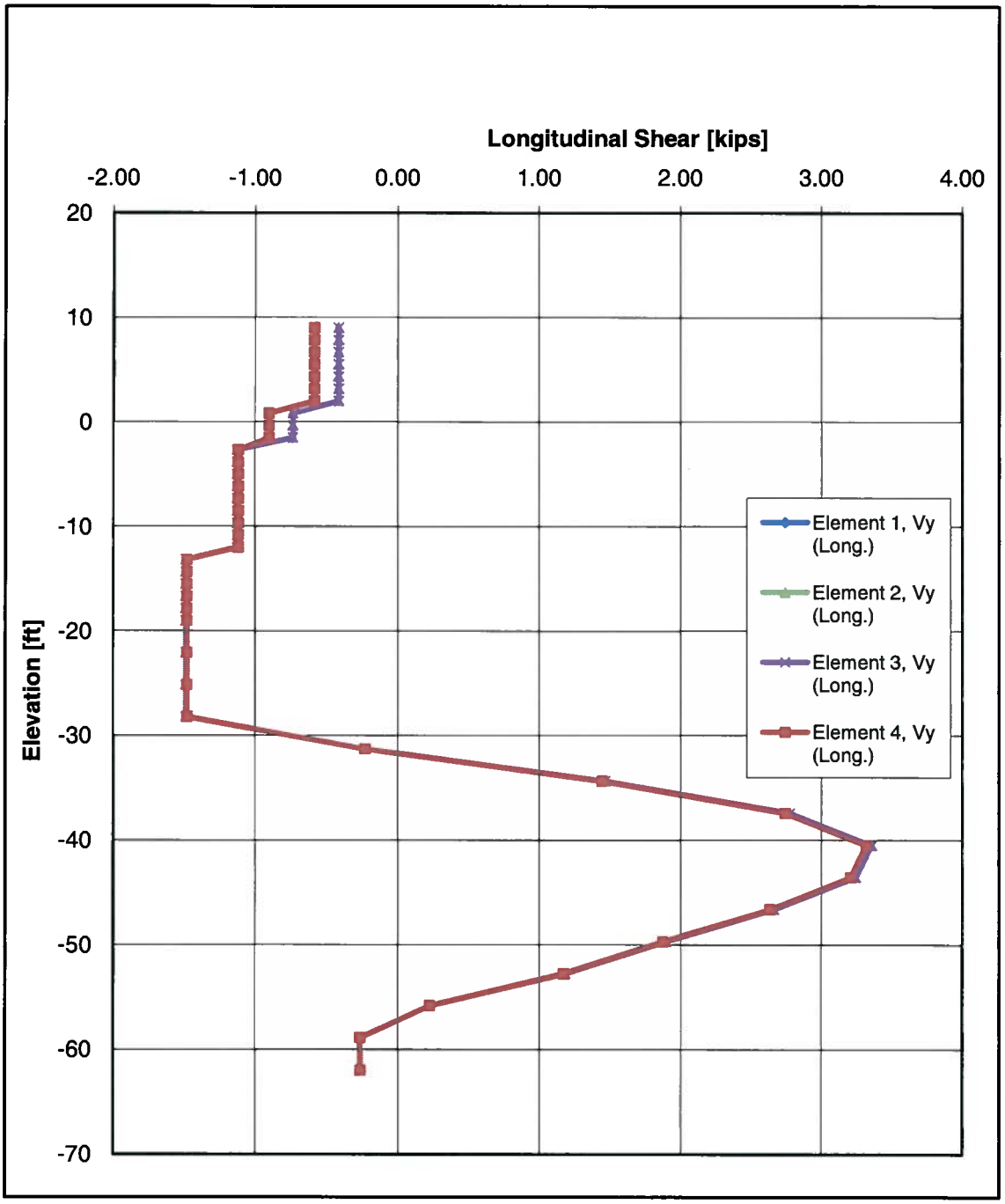
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

SER I Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML



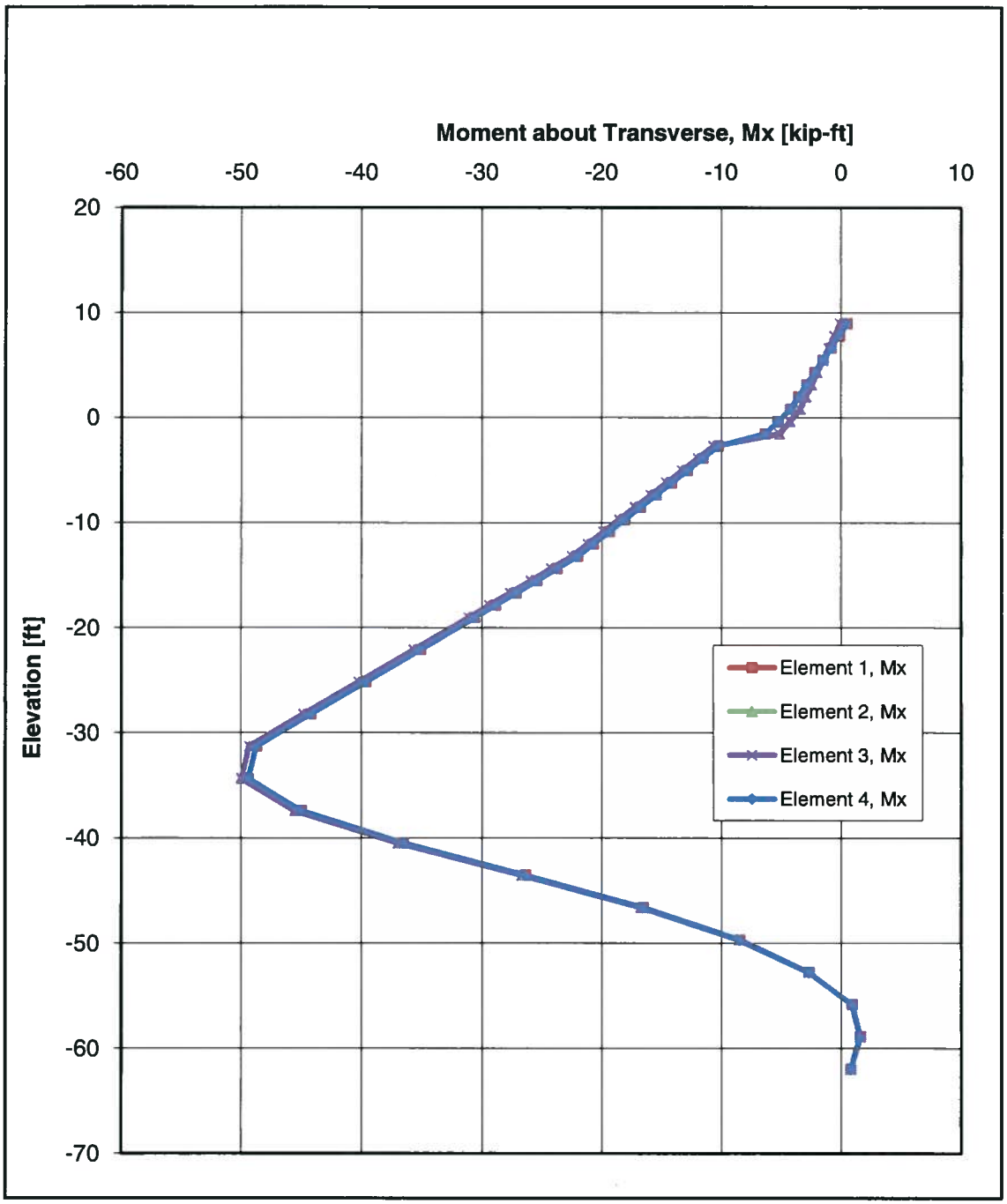
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

SER I Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

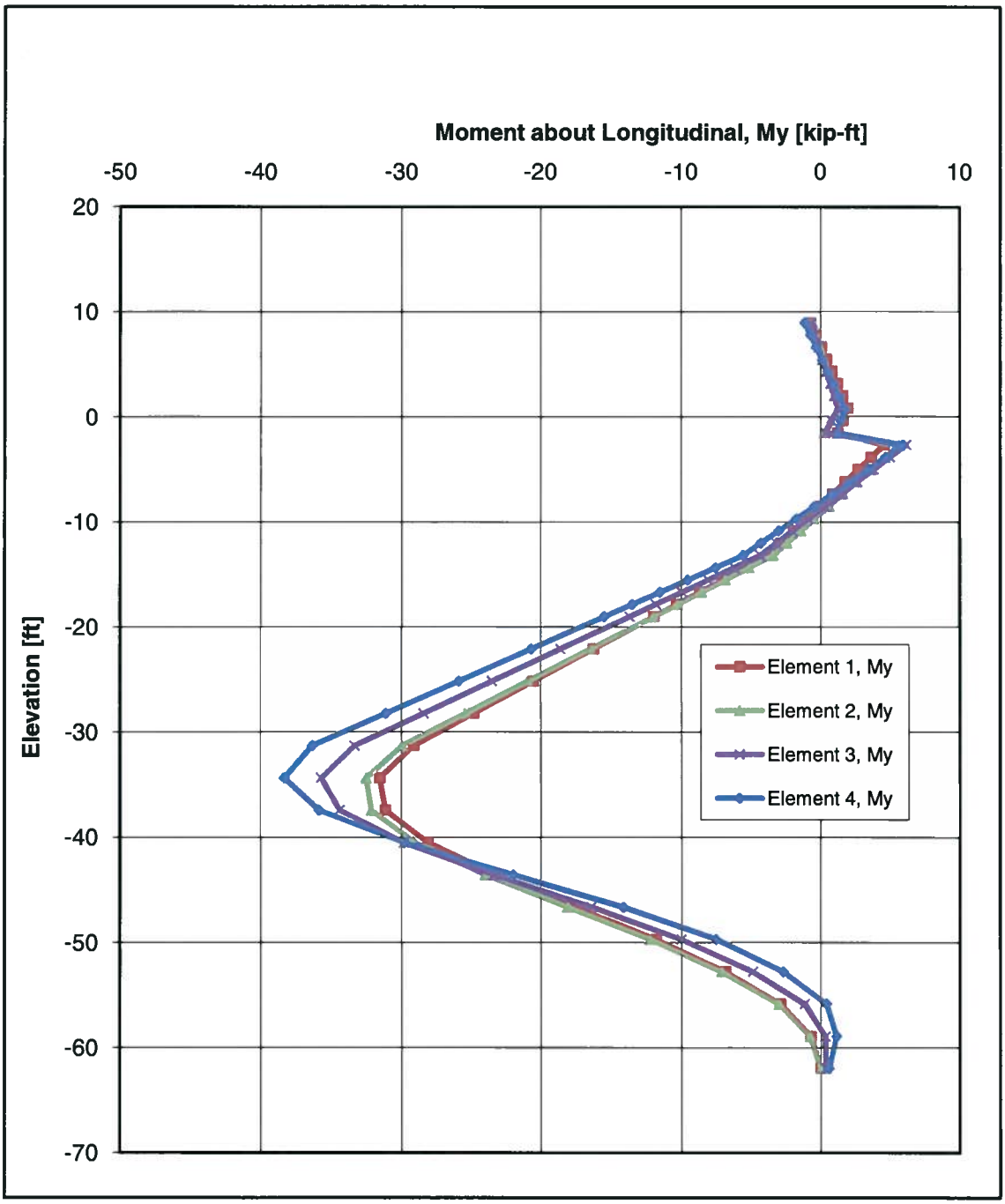


FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

SER I Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1010-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

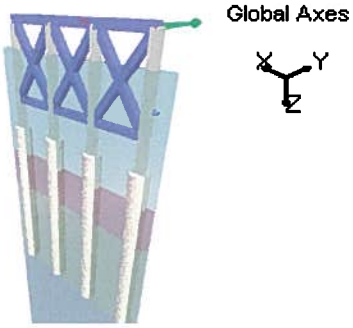


HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML

STR III Min Wind at 0 (10 ft SCOUR)

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)



Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
Top of Cap Beam	1	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	2	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	3	1.70	0.00	7.00	0.00	0.00	0.00
Top of Cap Beam	4	1.70	0.00	7.00	0.00	0.00	0.00
Midheight of 12"x12"	25	0.64	0.48	0.00	0.00	0.00	0.00
Midheight of 12"x12"	64	0.64	0.48	0.00	0.00	0.00	0.00
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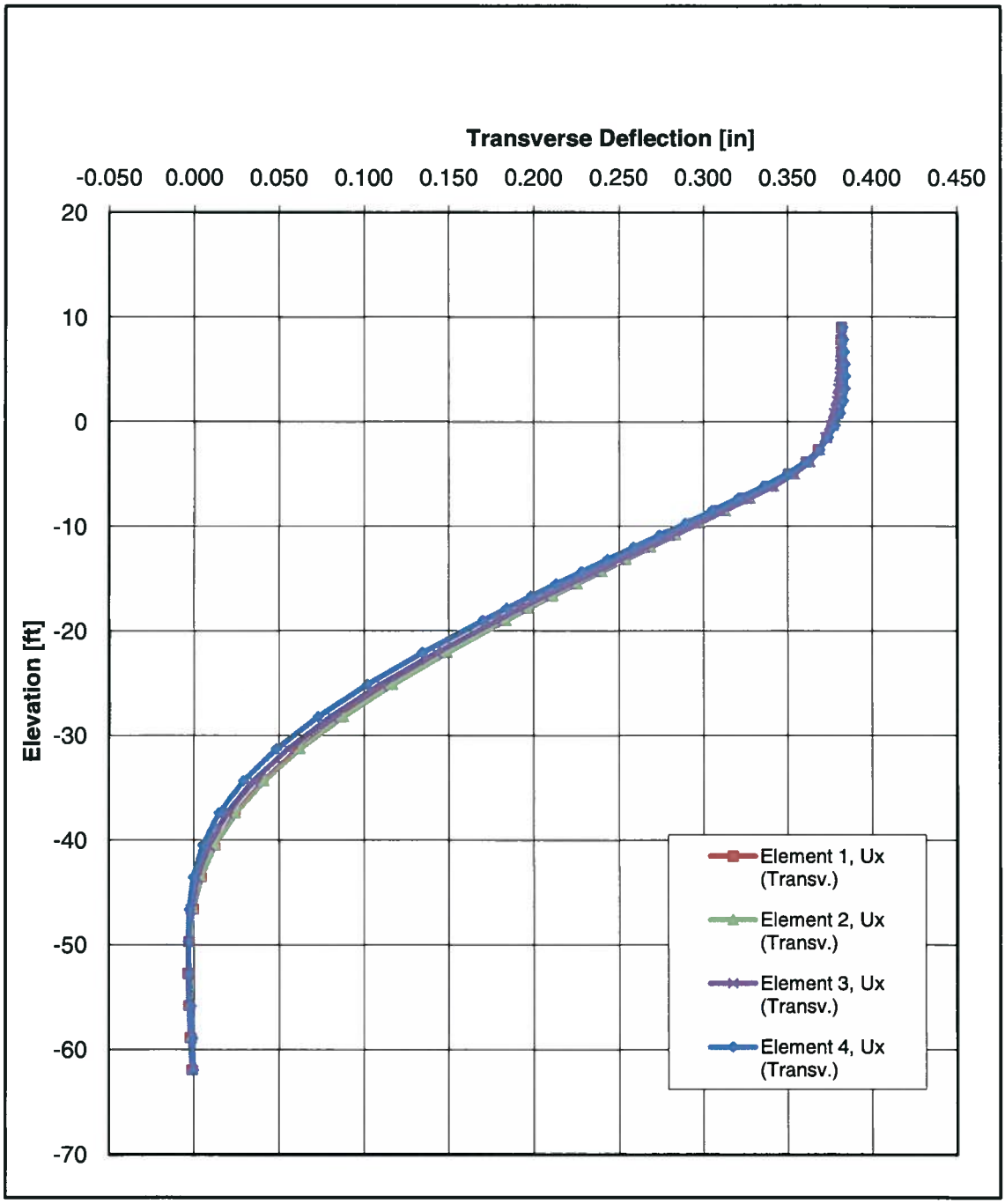
x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



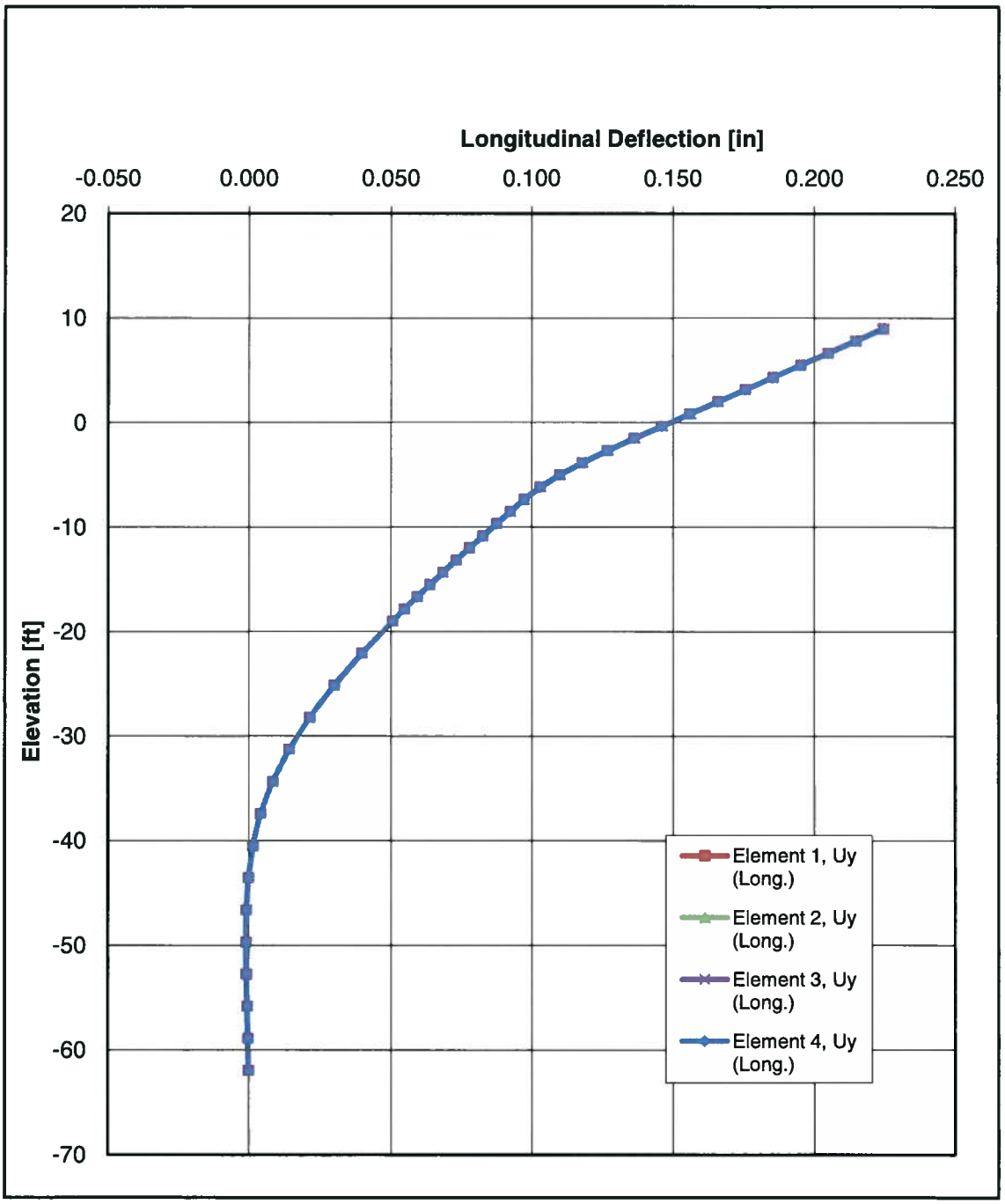
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML

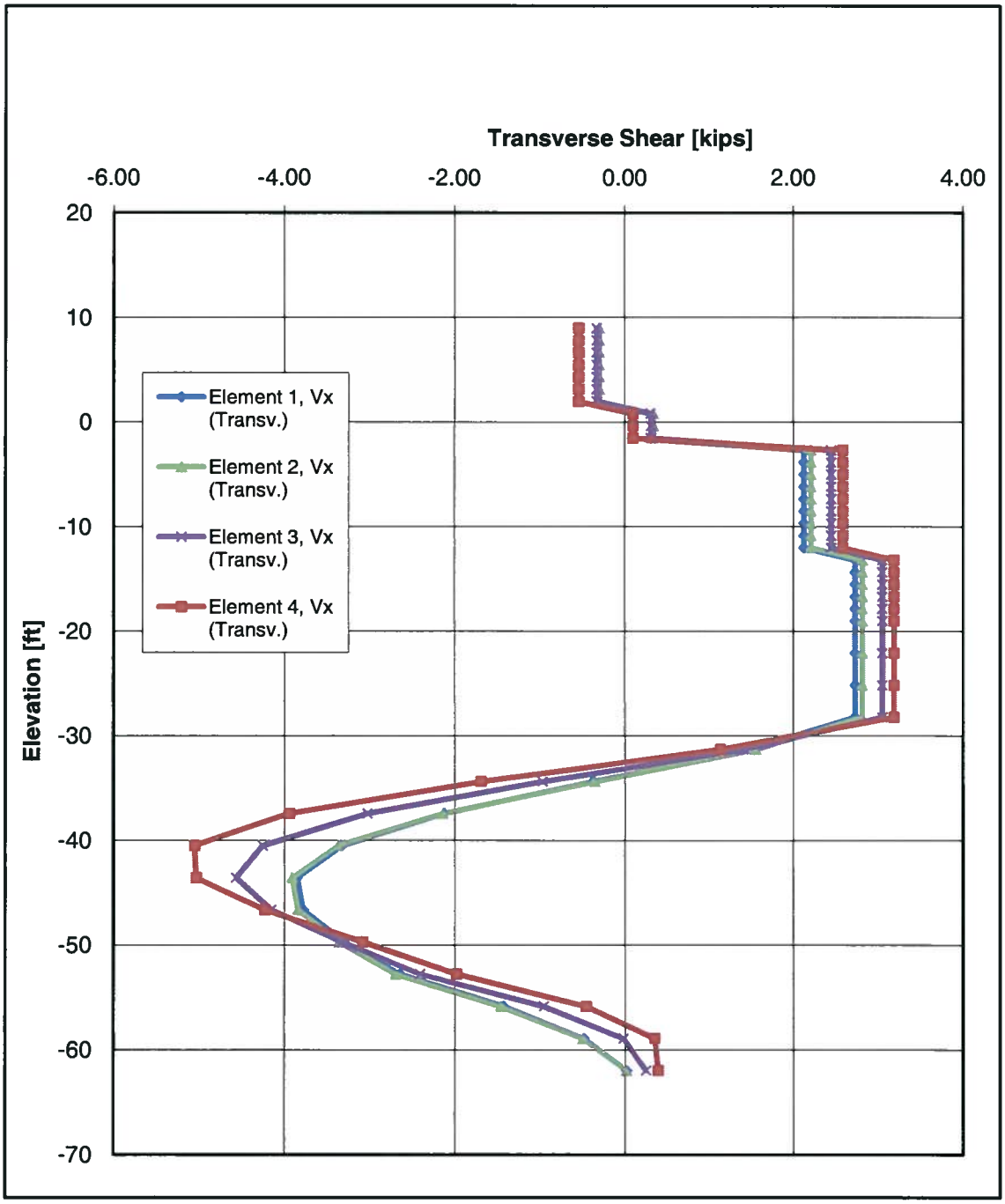


FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



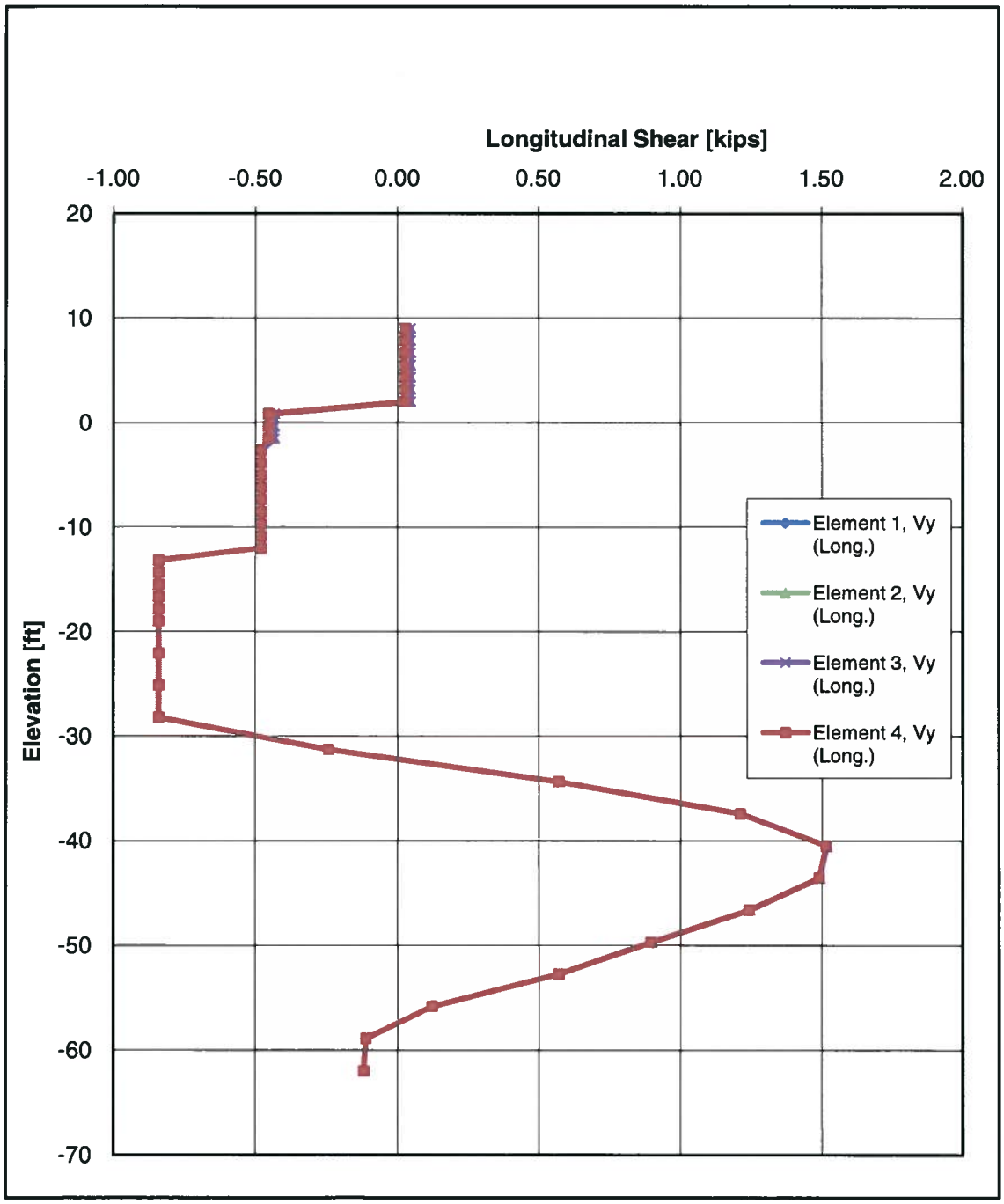
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



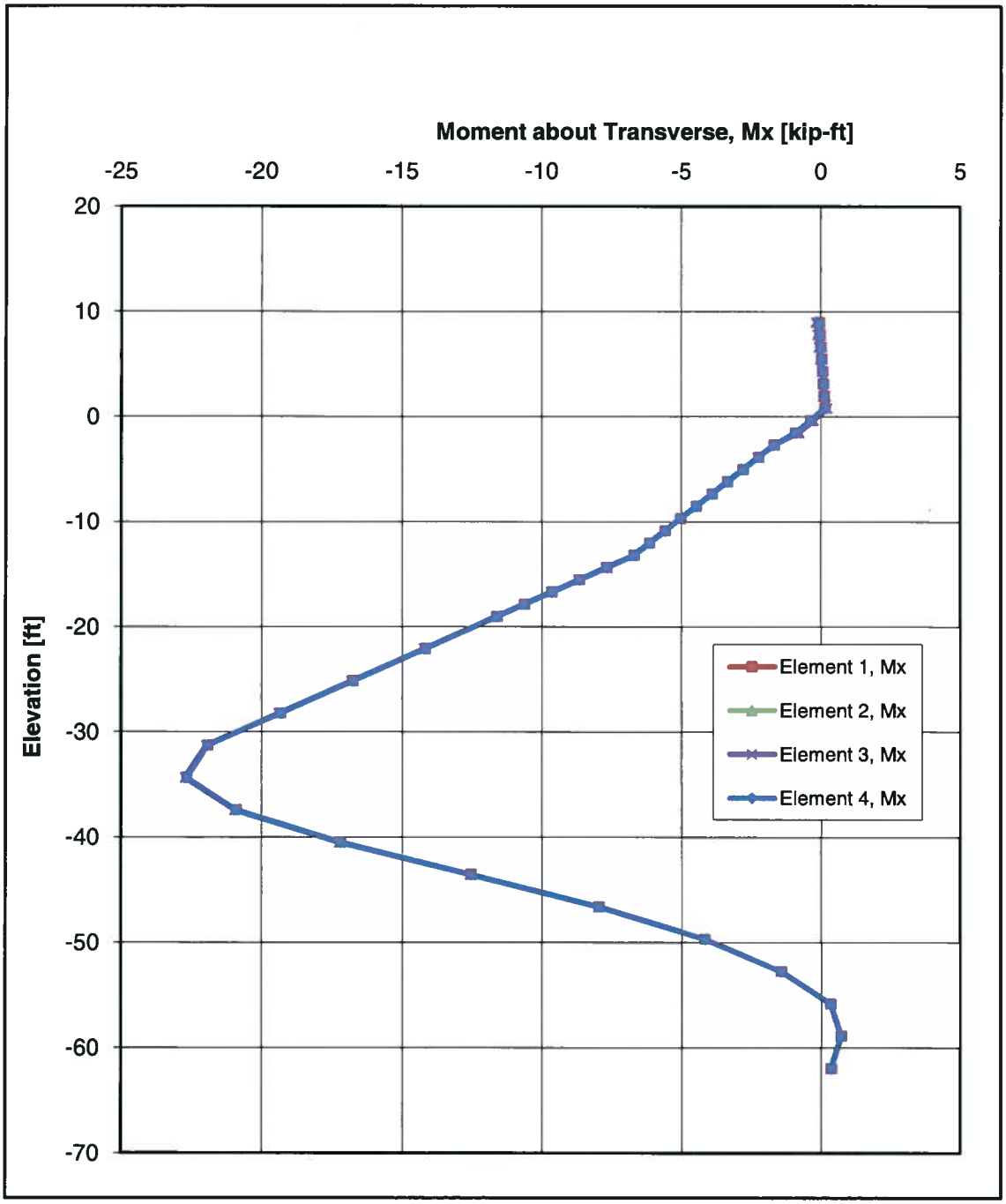
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



HALEY & ALDRICH	CALCULATIONS	File No.	
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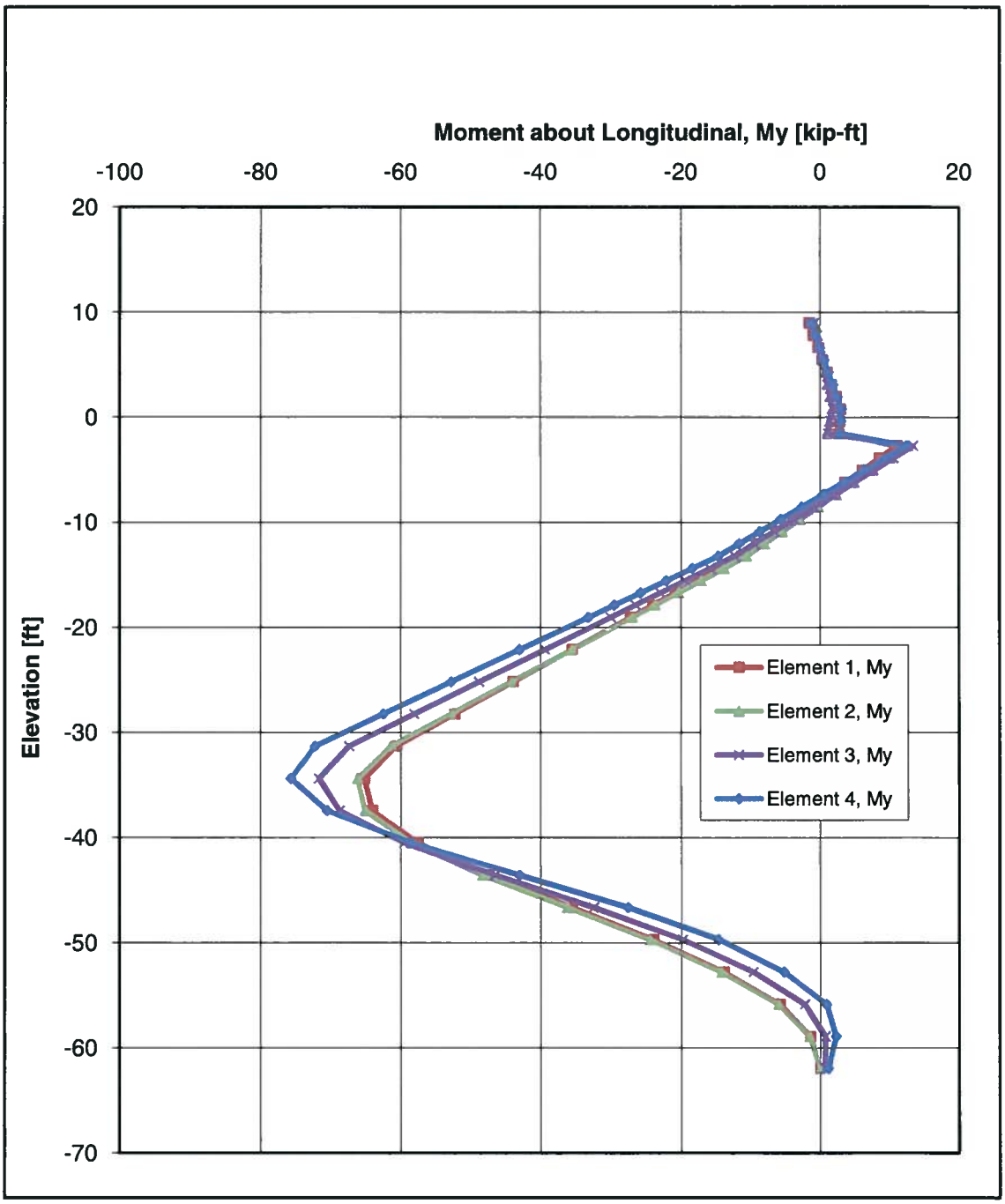
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML

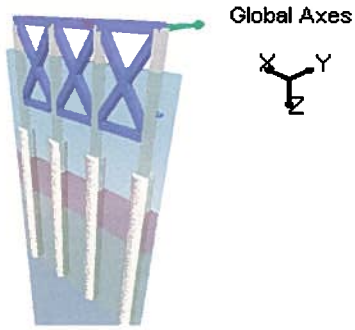


HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

STR V Max Wind at 60 (10 ft SCOUR)

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)



Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
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Top of Cap Beam	1	0.30	1.00	83.80	0.00	0.00	0.00
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x - Bridge transverse direction
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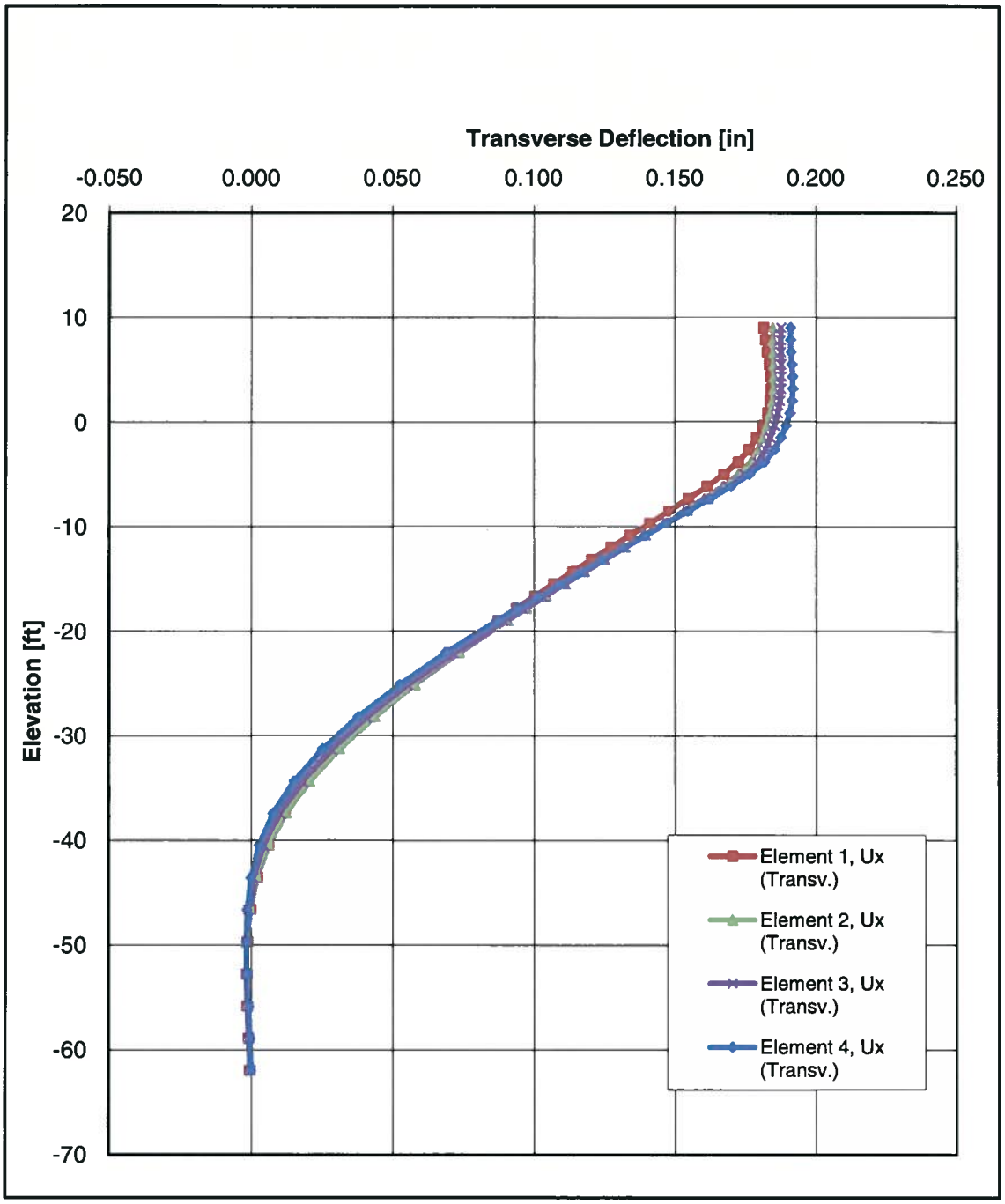
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



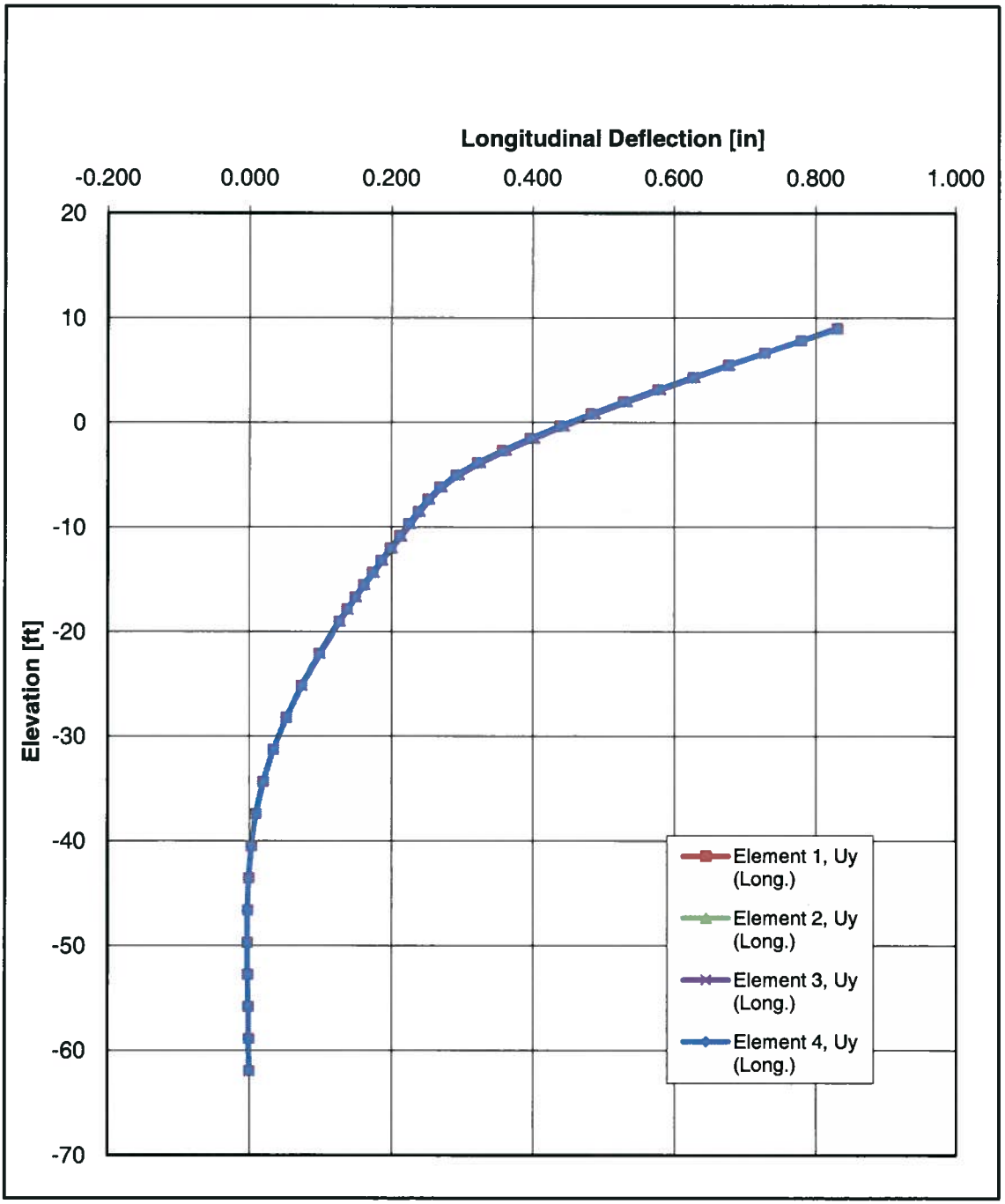
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



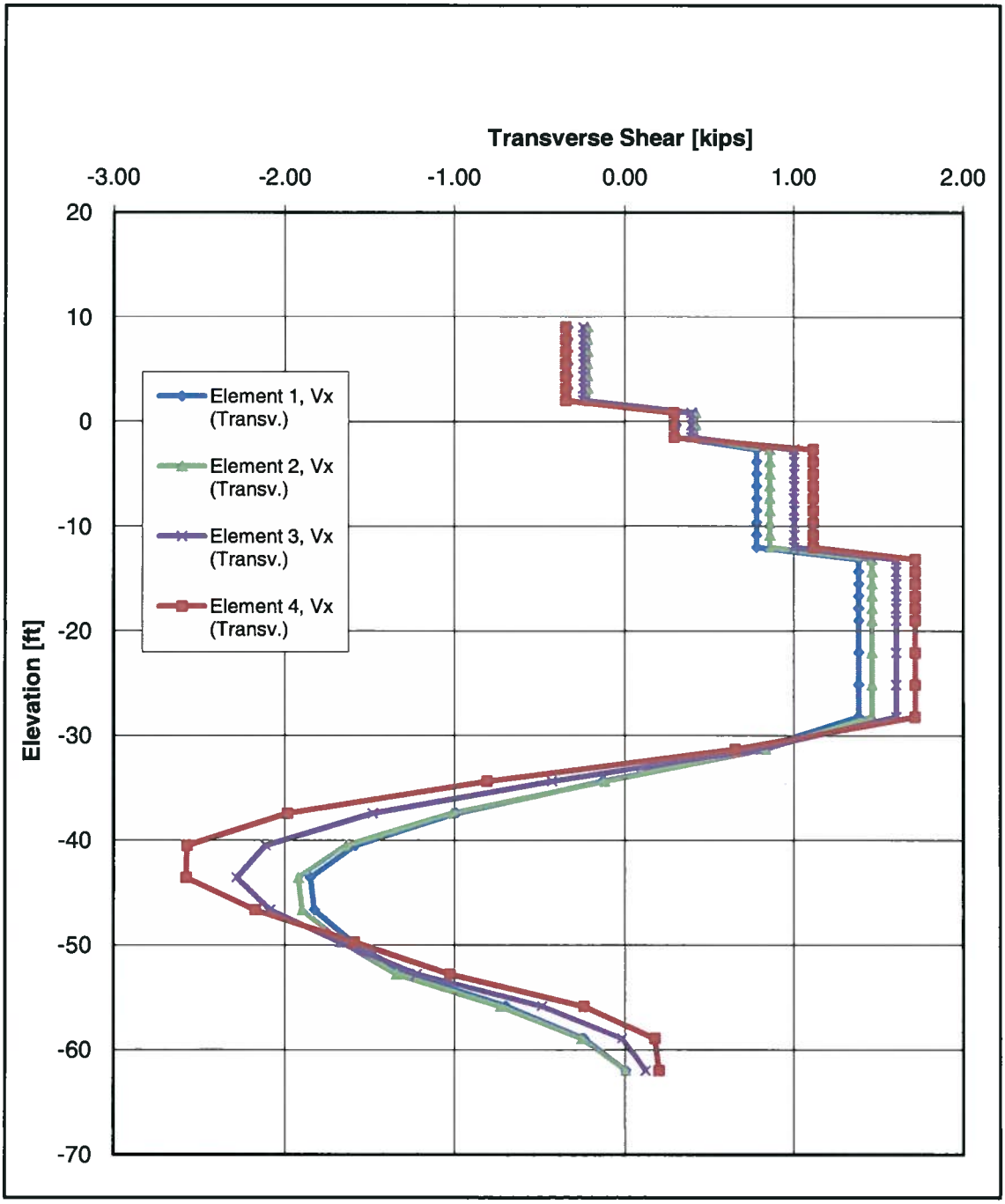
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

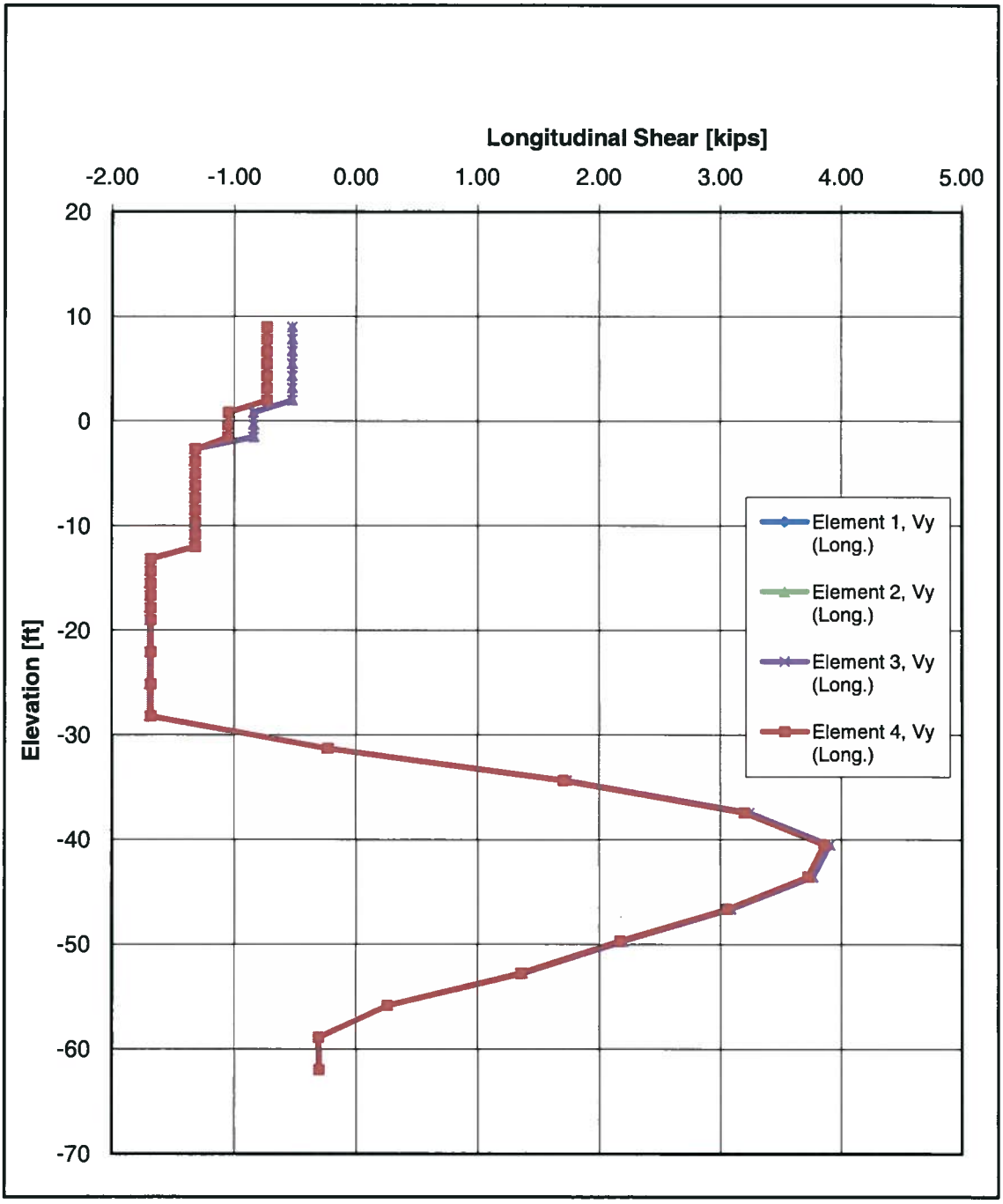


FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



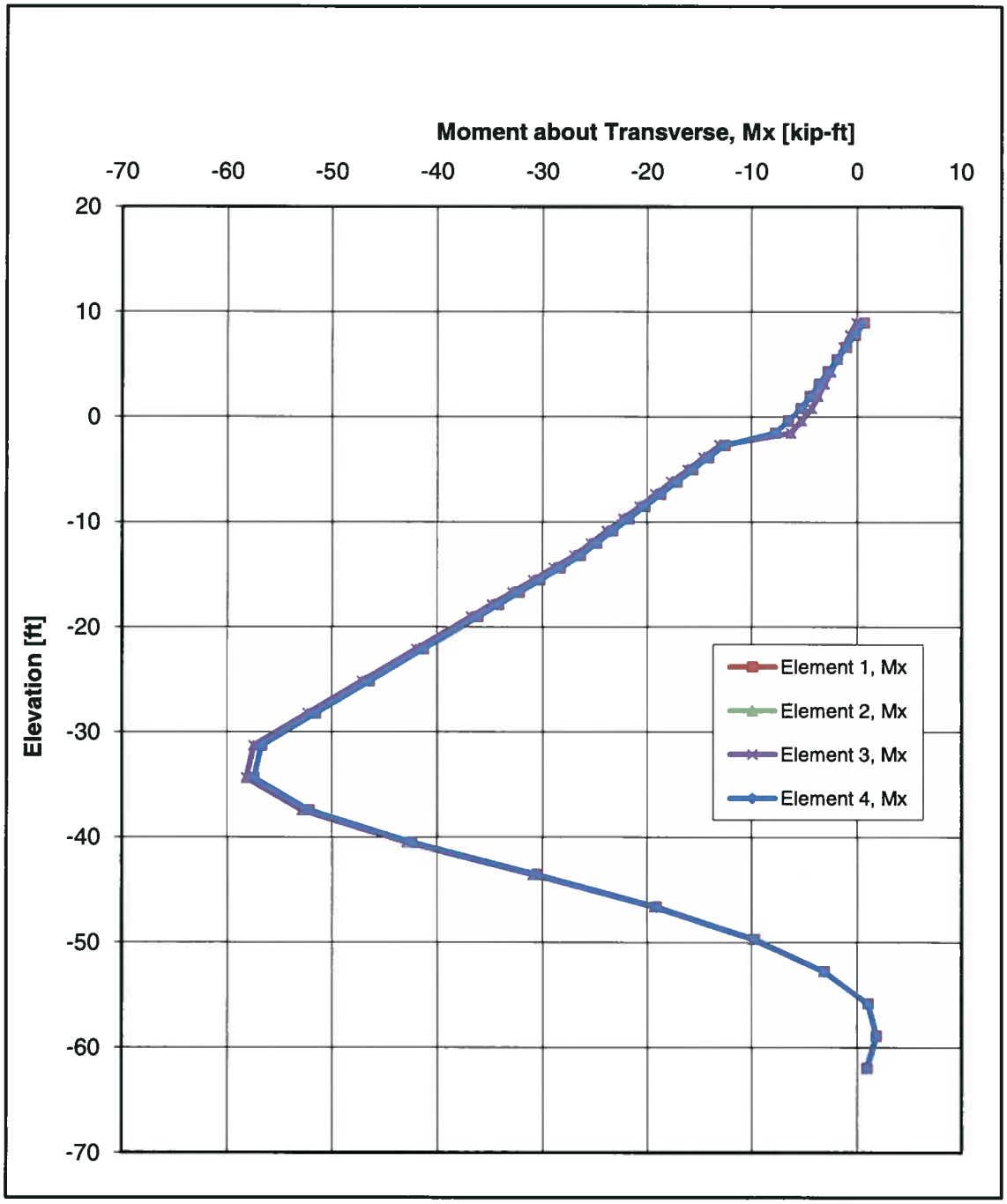
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



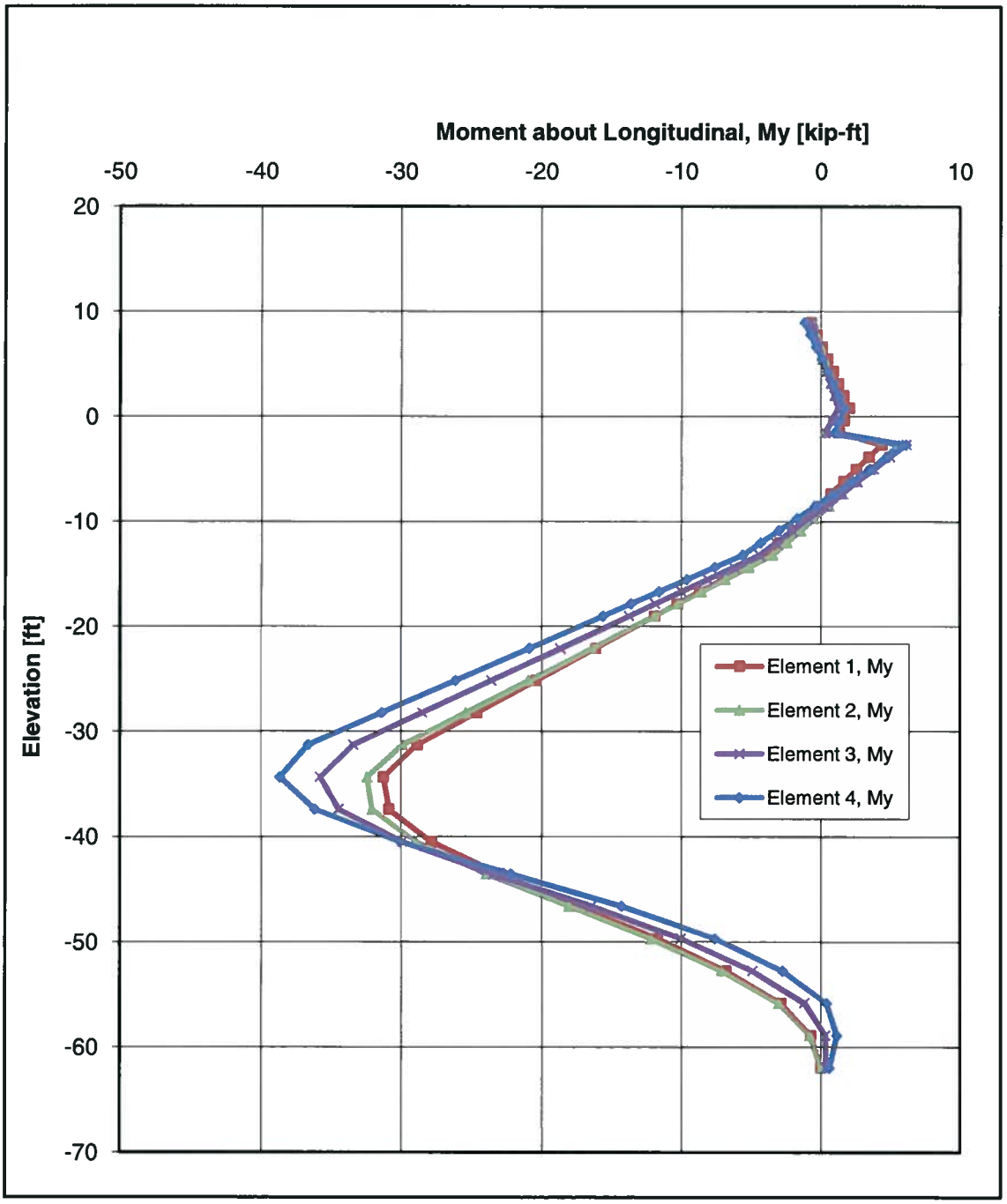
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (WITH Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 7 (30x0.5in PP with CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 7\2011-1018-HAI-Bent 7-30x0_5 in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



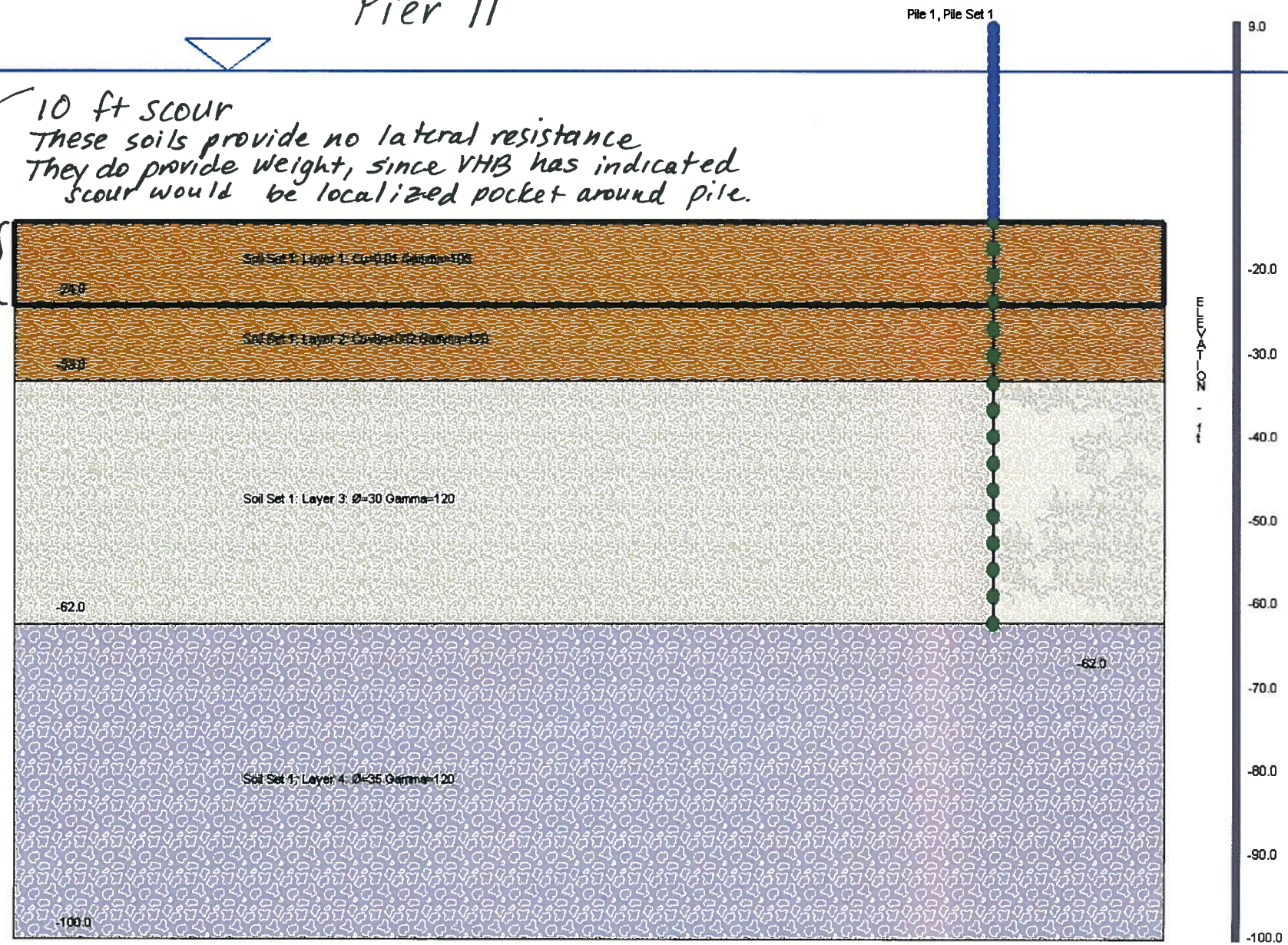
FB-Multipier File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.in - [Soi]

File View Control Wizard Help

Pier #1 | 1 | Load Case

Pier 11

10 ft scour
These soils provide no lateral resistance
They do provide weight, since VHB has indicated
scour would be localized pocket around pile.



HALEY & ALDRICH	CALCULATIONS	File No.	36654-001
		Sheet	1 of 1
Client	VHB	Date	18OCT2011
Project	Sewall's Bridge	Computed by	JLL
Subject	**Bent 11 Summary of Results**	Checked by	EFW

Files: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1010-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML
G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML
G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

FB-MultiPier Results for Bent 11

SUMMARY OF RESULTS

AASHTO-LRFD Limit State	Plumb Pile/ Concrete Fill	Batt. Pile/ Concrete Fill	Scour	δx (1) [in]	δy (1) [in]	$V_{x\,max}$ [kip]	$V_{y\,max}$ [kip]	$M_{x\,max}$ (2) [kip-ft]	$M_{y\,max}$ (2) [kip-ft]	N_{max} (3) [kip]	N_{min} (4) [kip]
SER I Wind at 60	30x0.5/ yes	none/NA	10 ft	0.10	0.52	2.5	4.0	44	32	107	48
STR III Min Wind at 0	30x0.5/ yes	none/NA	10 ft	0.31	0.12	5.6	1.5	17	79	38	6
STR V Max Wind at 60	30x0.5/ yes	none/NA	10 ft	0.11	0.69	2.7	4.1	56	35	146	72

x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

HALEY & ALDRICH

CALCULATIONS

File No.	
Sheet	
Date	
Computed by	JLL
Checked by	EFW

Client

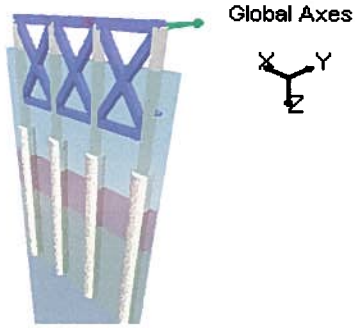
Project

Subject

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1005-HAI-Bent 11-24 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

SER I Wind
at 60 (10 ft SCOUR)

FB-MultiPier Results for Bent 11 (24 in PP WITH CONC)



Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
		[kip]	[kip]	[kip]	[kip-ft]	[kip-ft]	[kip-ft]
Top of Cap Beam	1	0.40	1.00	93.80	0.00	0.00	0.00
Top of Cap Beam	2	0.40	1.00	93.80	0.00	0.00	0.00
Top of Cap Beam	3	0.40	1.00	93.80	0.00	0.00	0.00
Top of Cap Beam	4	0.40	1.00	93.80	0.00	0.00	0.00
Midheight of 12"x12"	25	0.64	0.16	0.00	0.00	0.00	0.00
Midheight of 12"x12"	59	0.64	0.16	0.00	0.00	0.00	0.00
Midheight of 12"x12"	93	0.64	0.16	0.00	0.00	0.00	0.00
Midheight of 12"x12"	127	0.64	0.16	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	34	0.28	0.21	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	68	0.28	0.21	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	102	0.28	0.21	0.00	0.00	0.00	0.00
Midheight of Exposed 24 in PP	136	0.28	0.21	0.00	0.00	0.00	0.00

x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

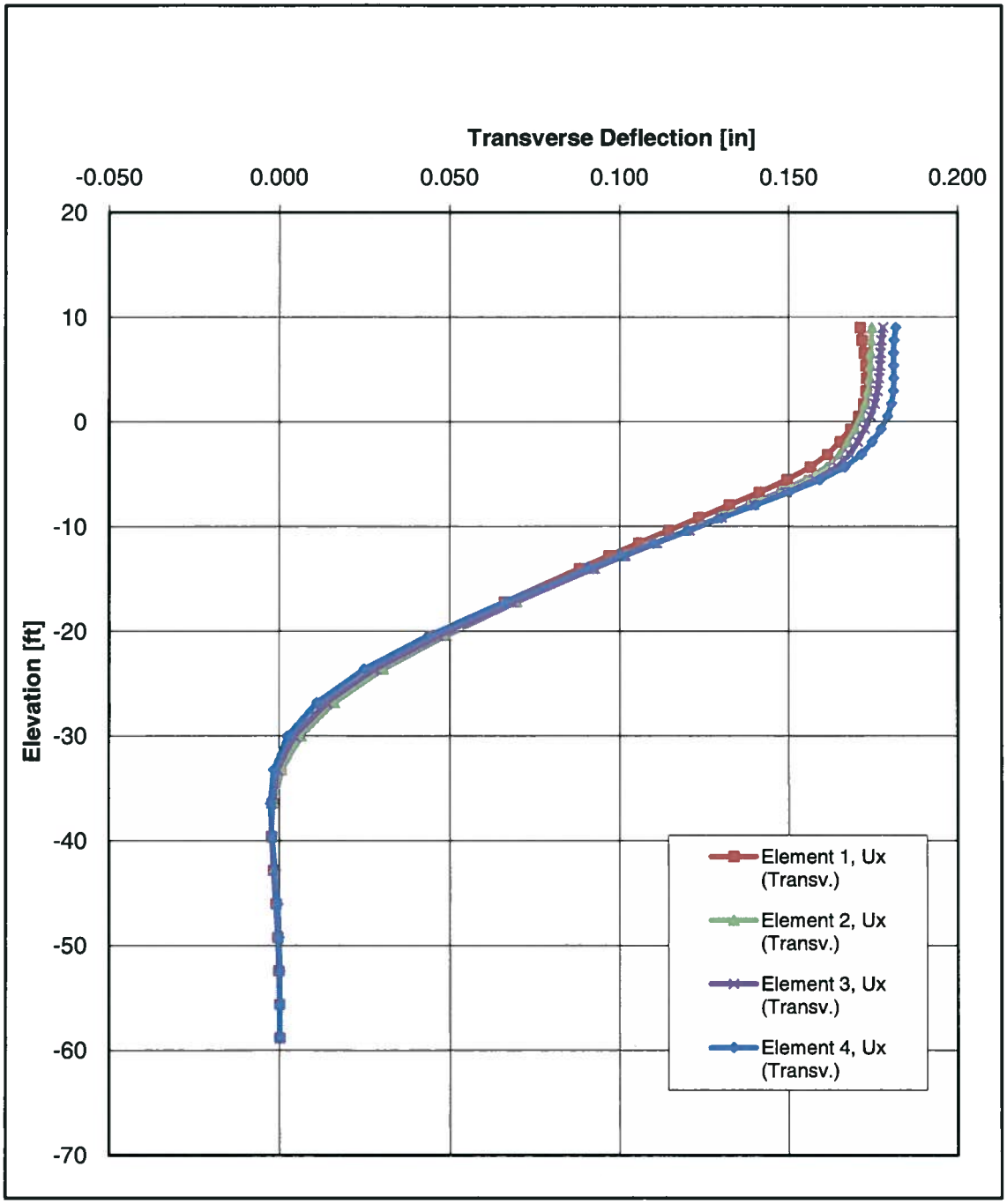
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project		Computed by	JLL
Subject		Checked by	EFW

FB-MultiPier Results for Bent 11 (24 in PP WITH CONC)

SER I Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1005-HAI-Bent 11-24 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

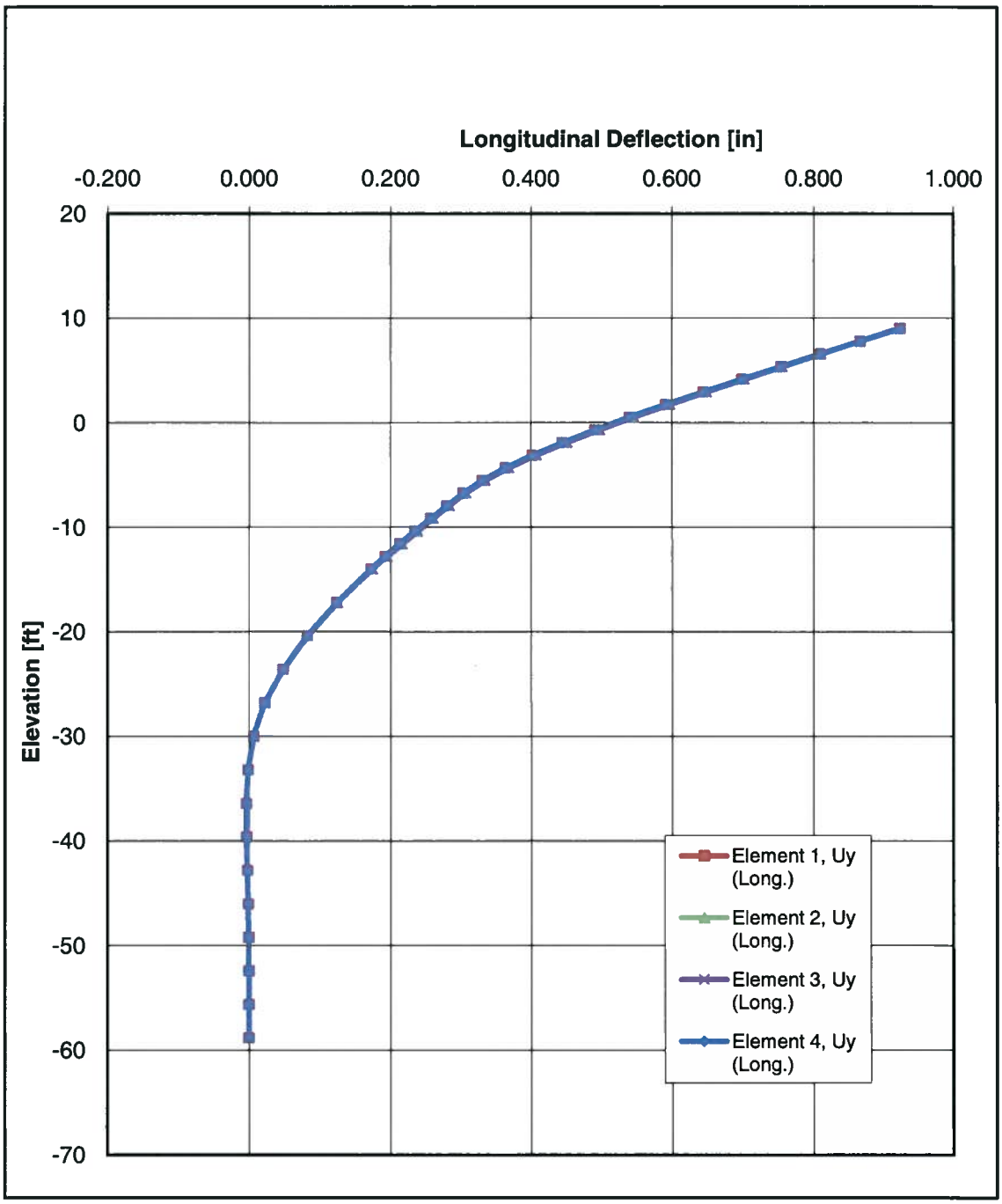


FB-MultiPier Results for Bent 11 (24 in PP WITH CONC)

SER I Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1005-HAI-Bent 11-24 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

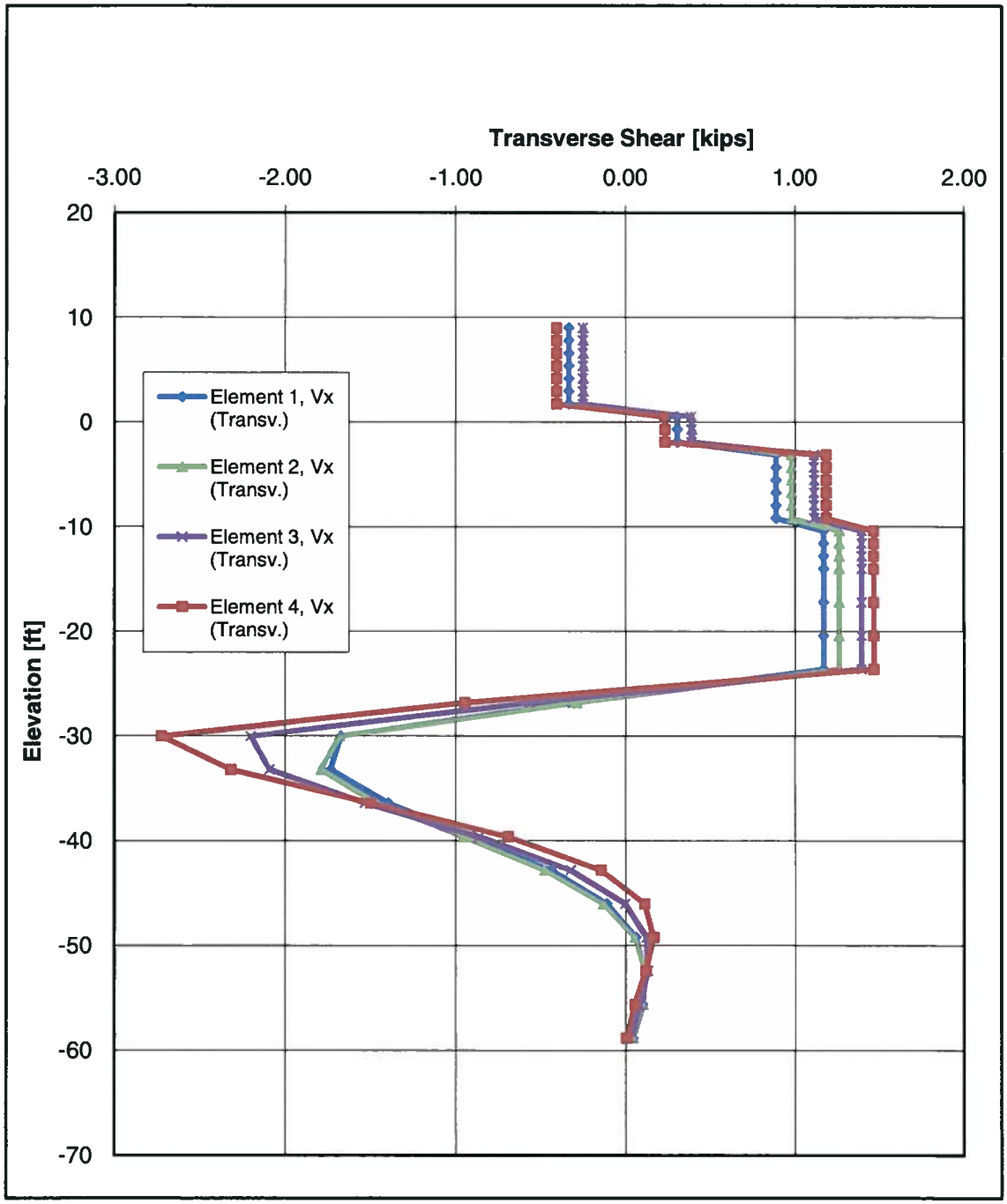


FB-MultiPier Results for Bent 11 (24 in PP WITH CONC)

SER I Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1005-HAI-Bent 11-24 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML



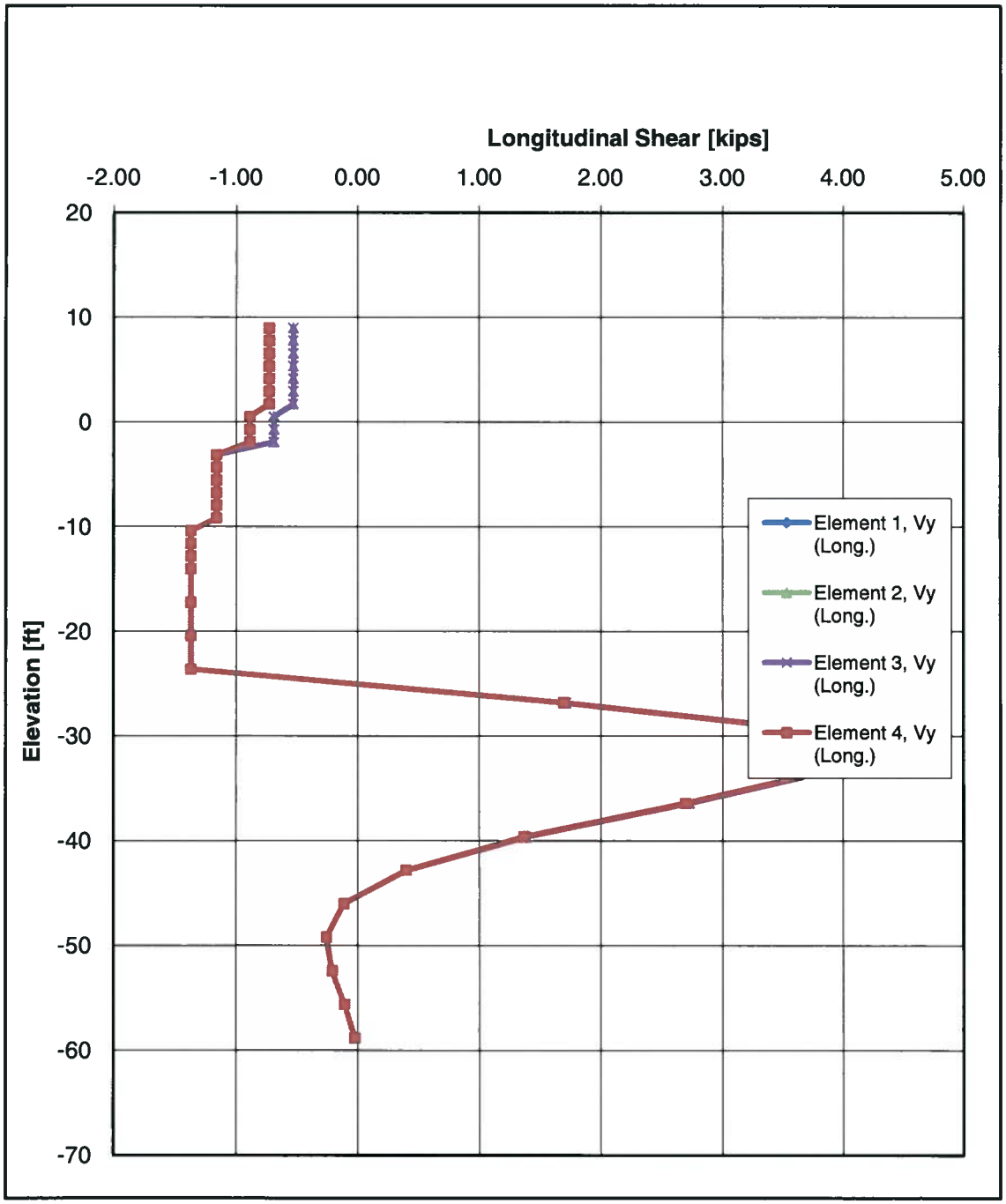
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project		Computed by	JLL
Subject		Checked by	EFW

FB-MultiPier Results for Bent 11 (24 in PP WITH CONC)

SER I Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1005-HAI-Bent 11-24 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML



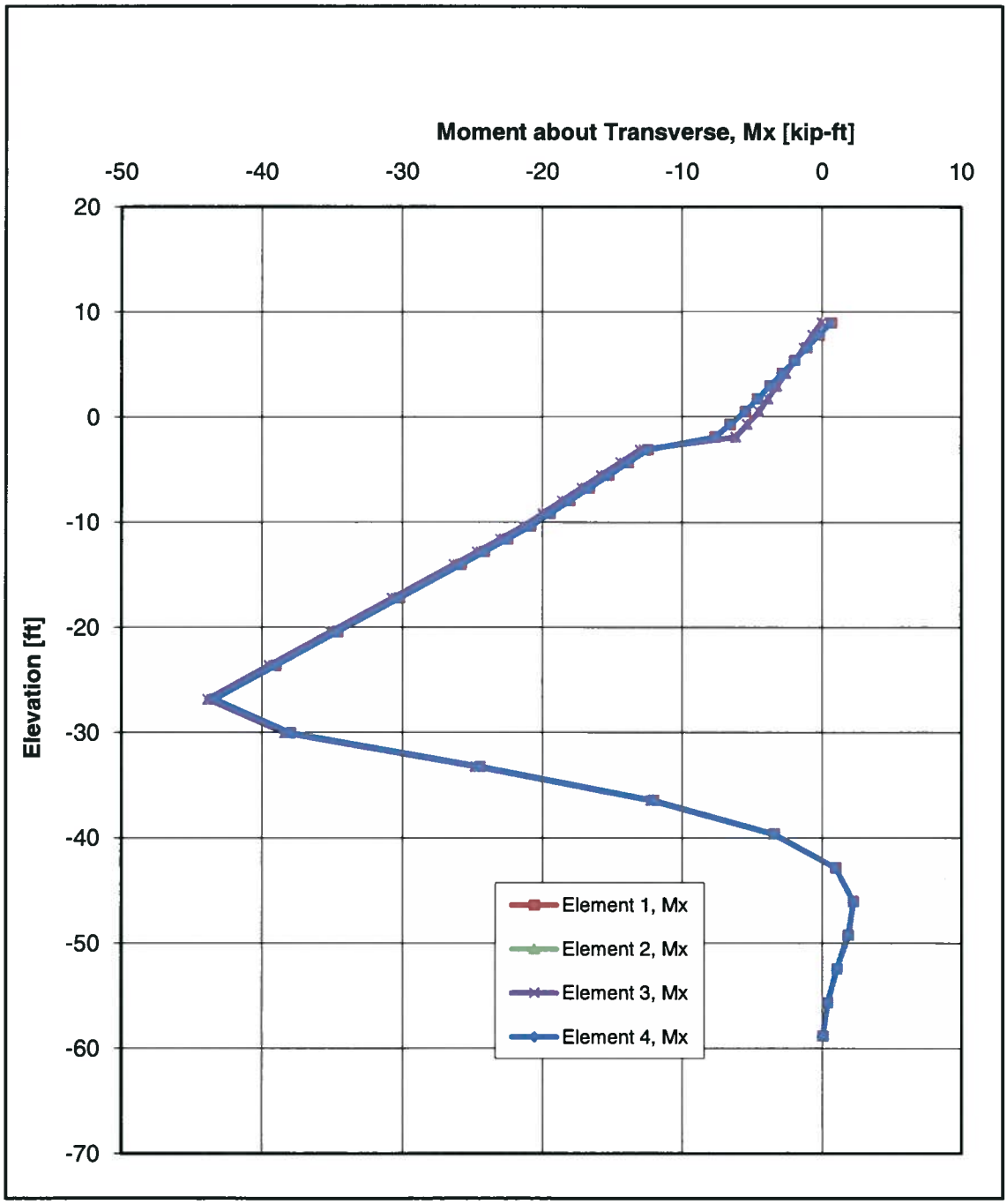
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project		Computed by	JLL
Subject		Checked by	EFW

FB-MultiPier Results for Bent 11 (24 in PP WITH CONC)

SER I Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1005-HAI-Bent 11-24 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML



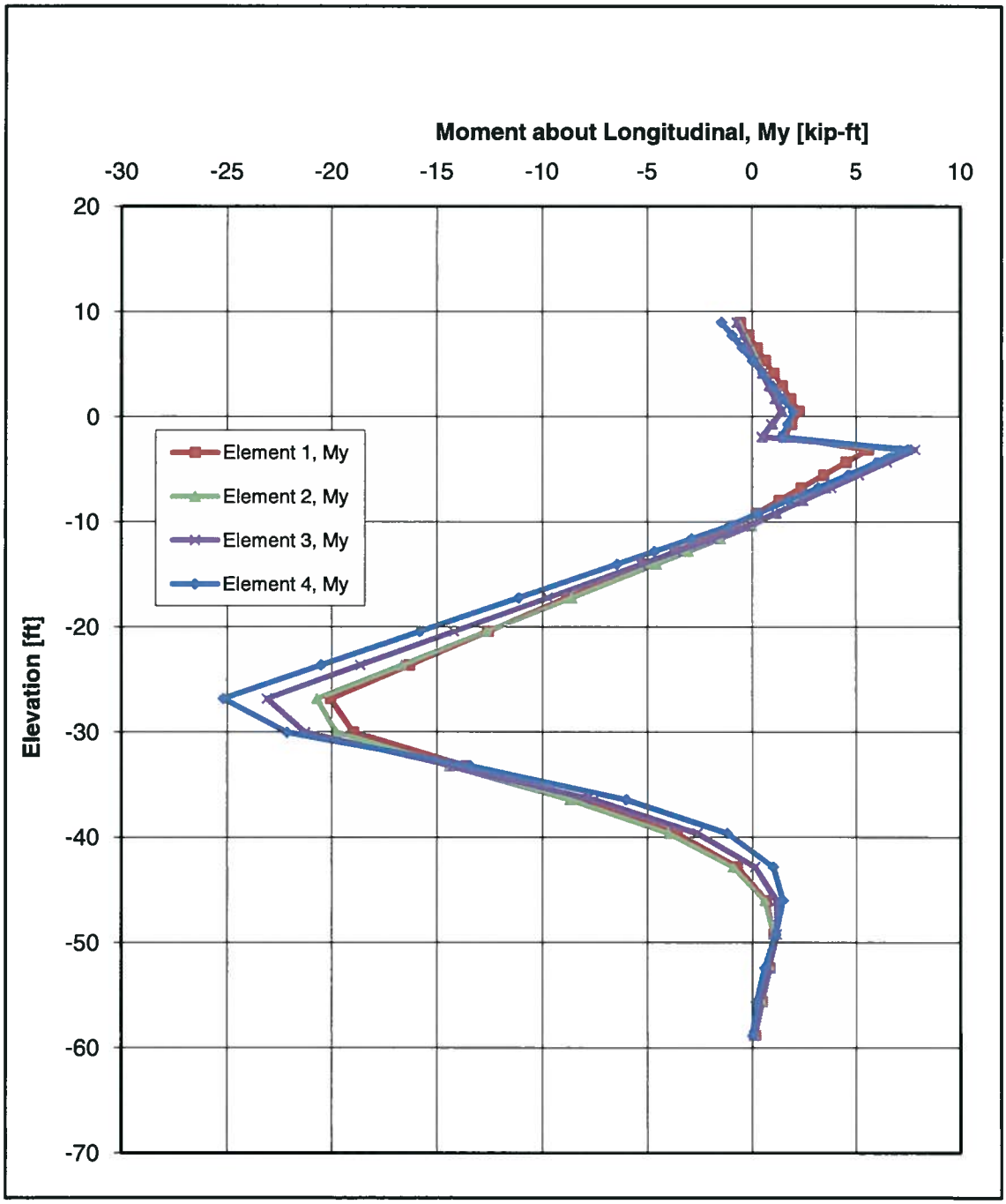
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project		Computed by	JLL
Subject		Checked by	EFW

FB-MultiPier Results for Bent 11 (24 in PP WITH CONC)

SER I Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1005-HAI-Bent 11-24 in PP-10 ft SCOUR-SER I Wind at 60-WITH CONCRETE-D1.XML

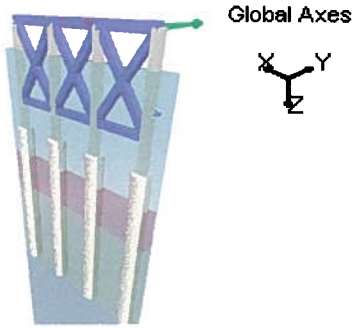


HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML

STR III Min Wind at 0 (10 ft SCOUR)

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)



Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
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Top of Cap Beam	1	2.50	0.00	11.50	0.00	0.00	0.00
Top of Cap Beam	2	2.50	0.00	11.50	0.00	0.00	0.00
Top of Cap Beam	3	2.50	0.00	11.50	0.00	0.00	0.00
Top of Cap Beam	4	2.50	0.00	11.50	0.00	0.00	0.00
Midheight of 12"x12"	25	0.64	0.48	0.00	0.00	0.00	0.00
Midheight of 12"x12"	59	0.64	0.48	0.00	0.00	0.00	0.00
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Midheight of Exposed 30 in PP	136	0.35	0.21	0.00	0.00	0.00	0.00

x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

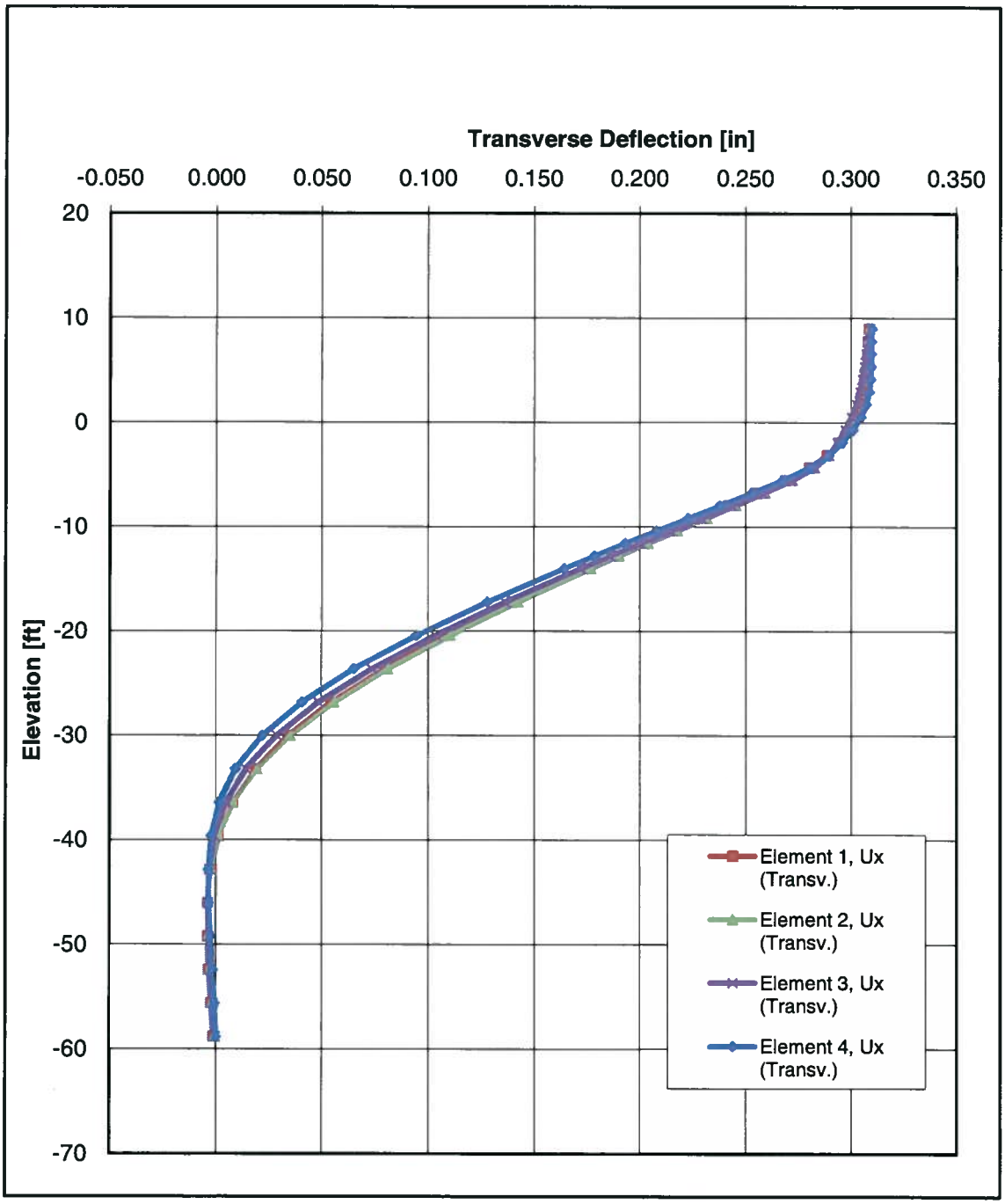
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



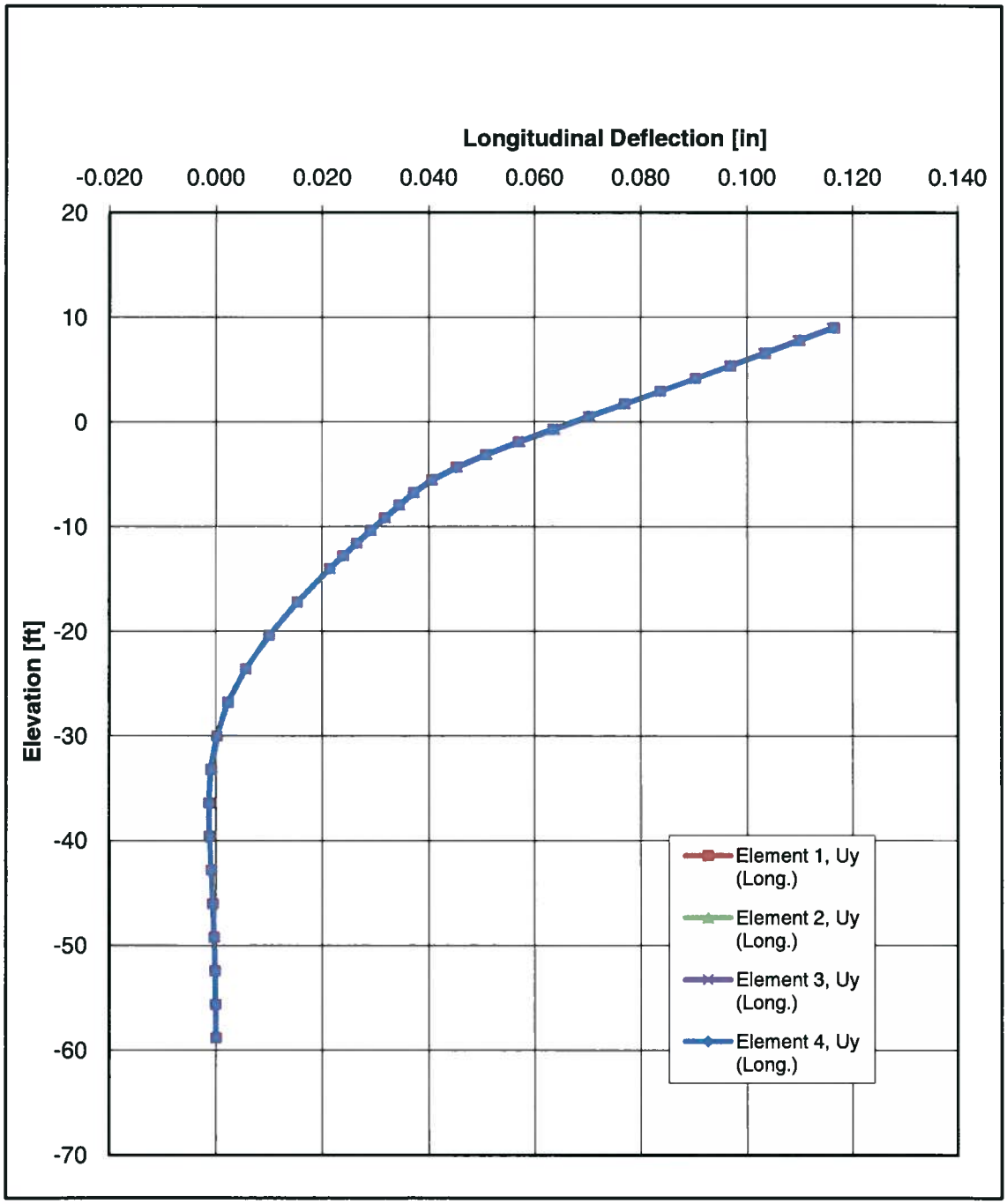
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML

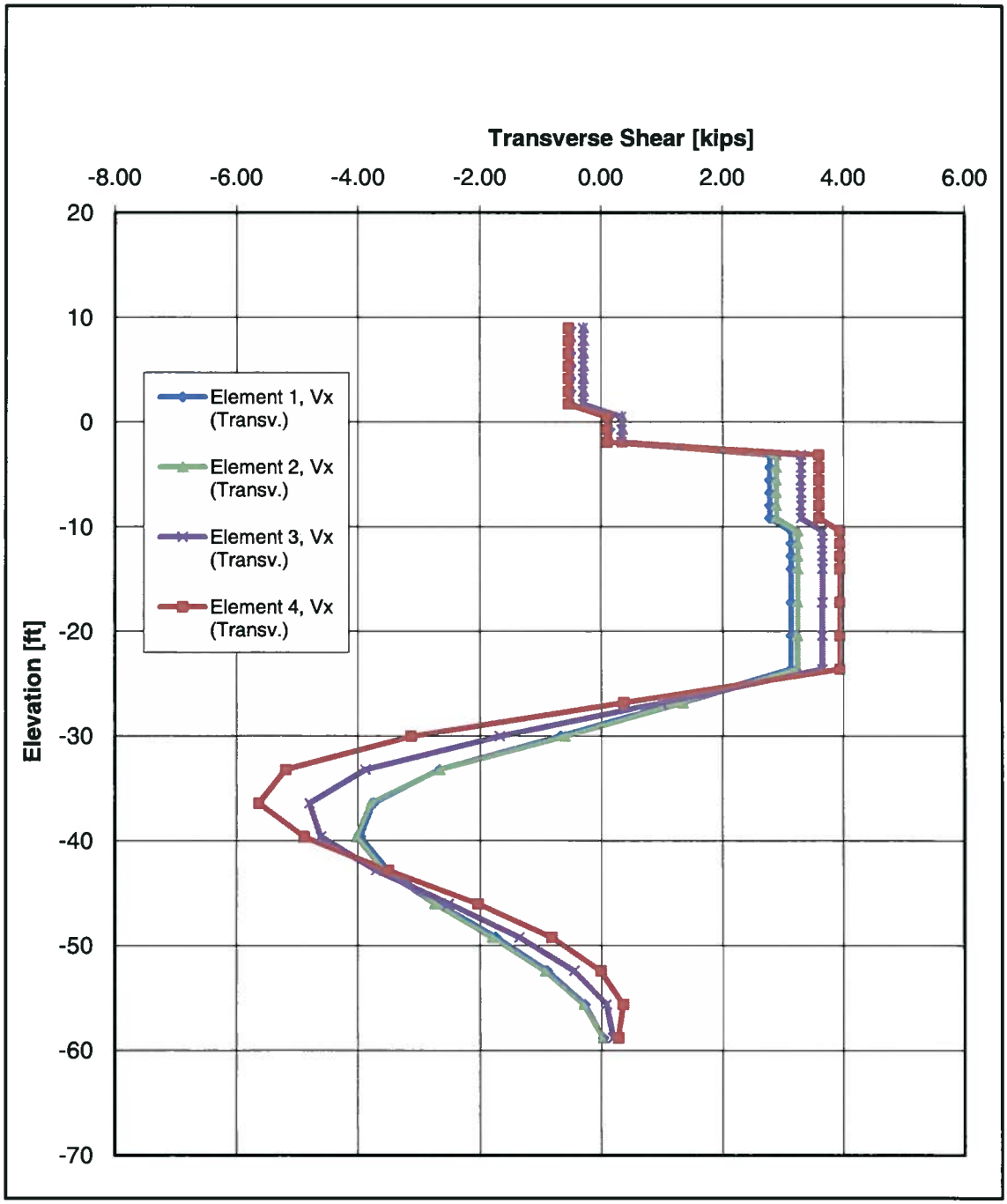


FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



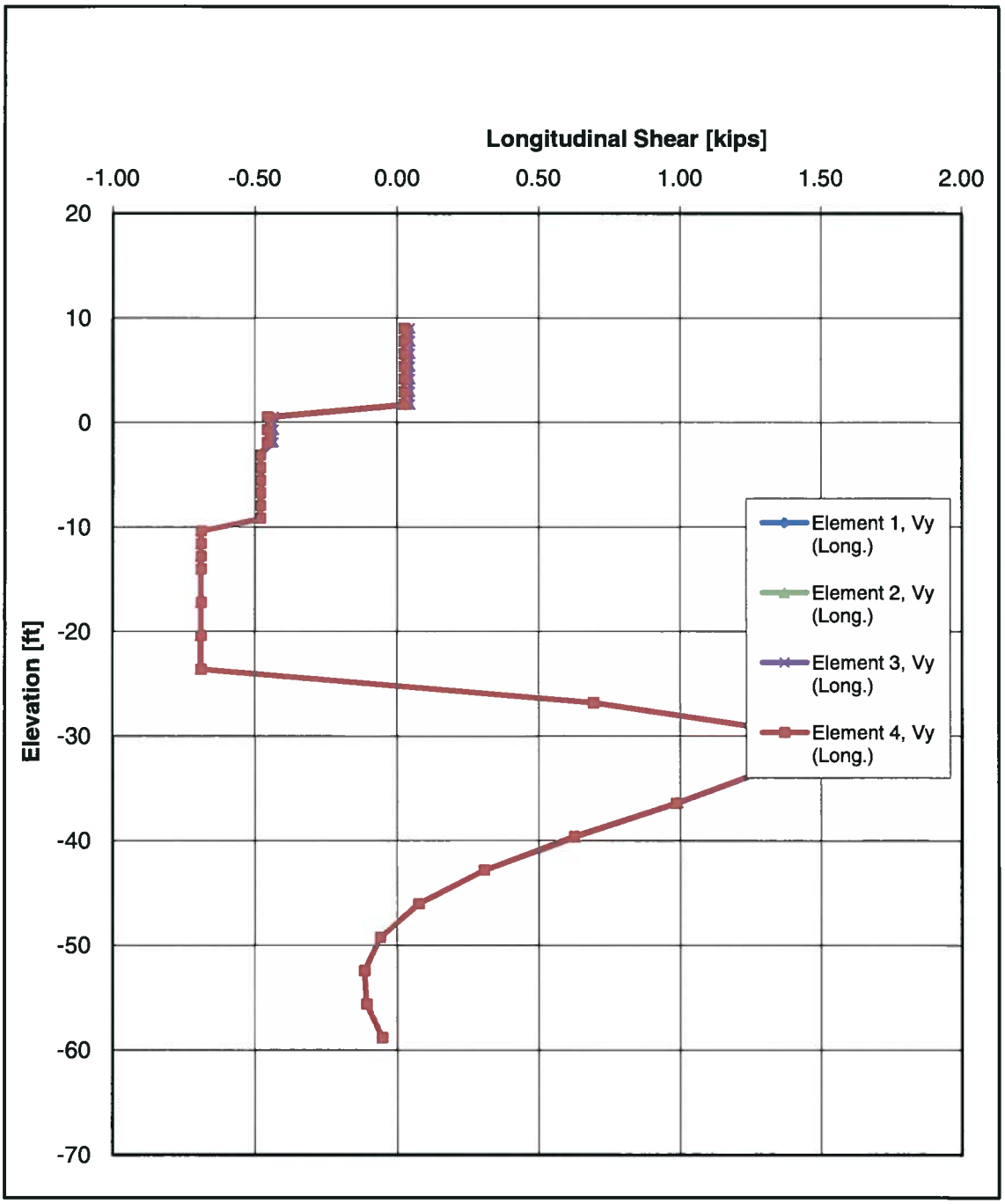
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



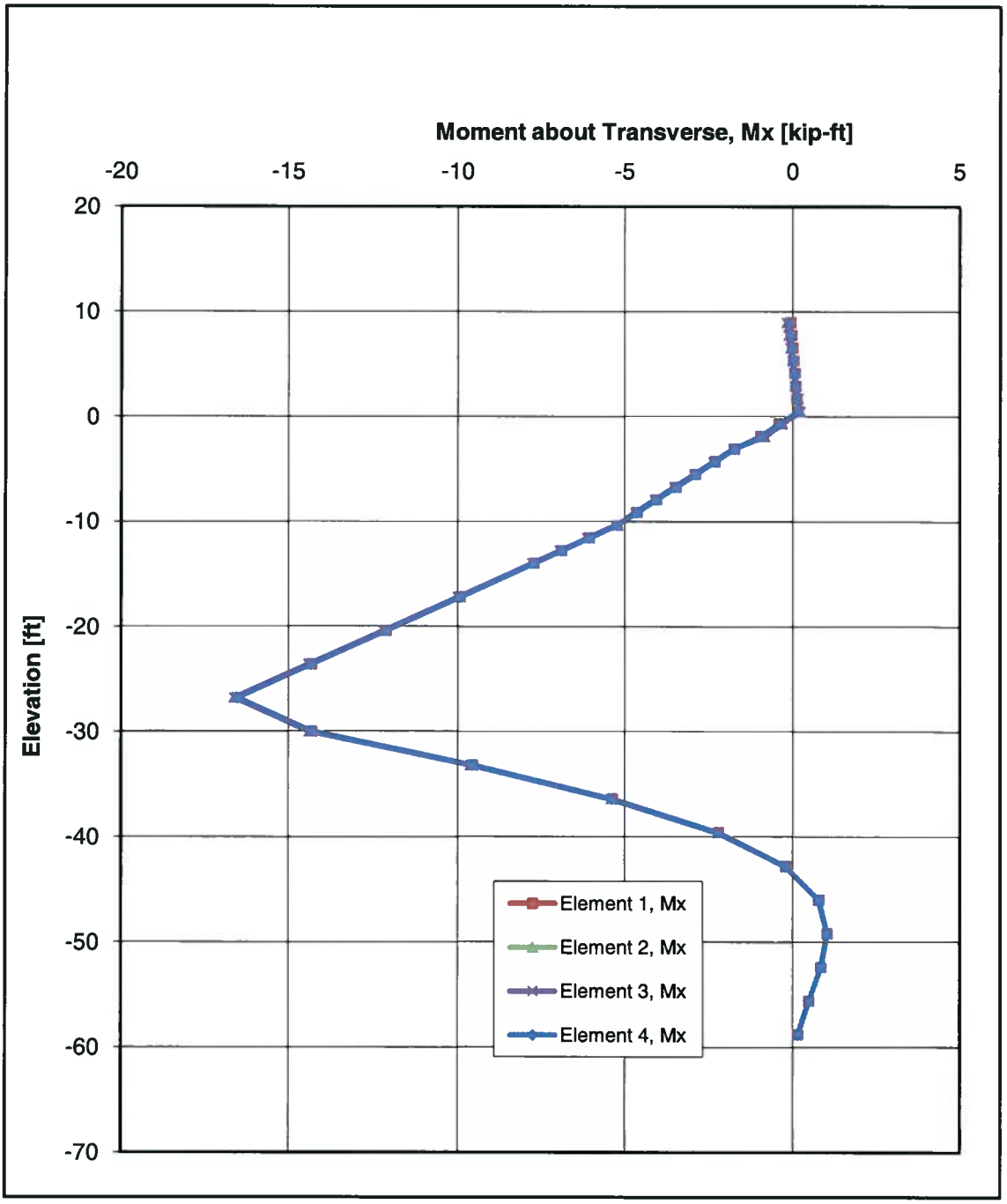
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML



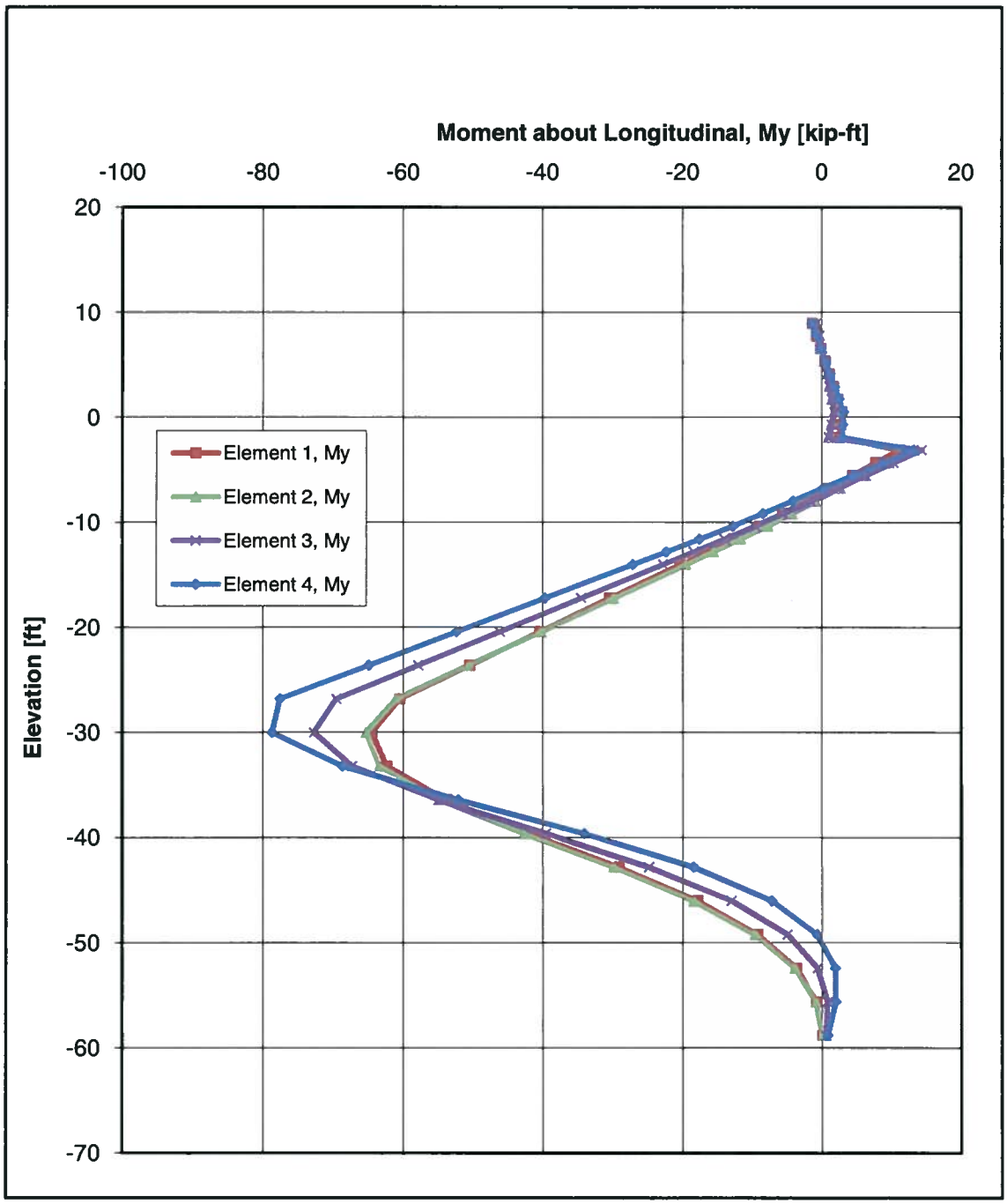
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR III Min Wind at 0 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR III Min Wind at 0-WITH CONCRETE-D1.XML

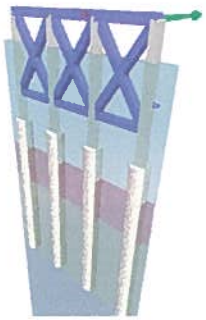


HALEY & ALDRICH	CALCULATIONS	File No.
		Sheet
Client		Date
Project	Sewall's Bridge	Computed by JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by EFW

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

STR V Max Wind at 60 (10 ft SCOUR)

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)



Global Axes

Loading Summary:

Location	Node	Fx	Fy	Fz	Mx	My	Mz
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Top of Cap Beam	1	0.50	1.20	126.20	0.00	0.00	0.00
Top of Cap Beam	2	0.50	1.20	126.20	0.00	0.00	0.00
Top of Cap Beam	3	0.50	1.20	126.20	0.00	0.00	0.00
Top of Cap Beam	4	0.50	1.20	126.20	0.00	0.00	0.00
Midheight of 12"x12"	25	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	59	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	93	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of 12"x12"	127	0.64	0.32	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	34	0.35	0.21	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	68	0.35	0.21	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	102	0.35	0.21	0.00	0.00	0.00	0.00
Midheight of Exposed 30 in PP	136	0.35	0.21	0.00	0.00	0.00	0.00

x - Bridge transverse direction
y - Bridge longitudinal direction
z - Vertical direction

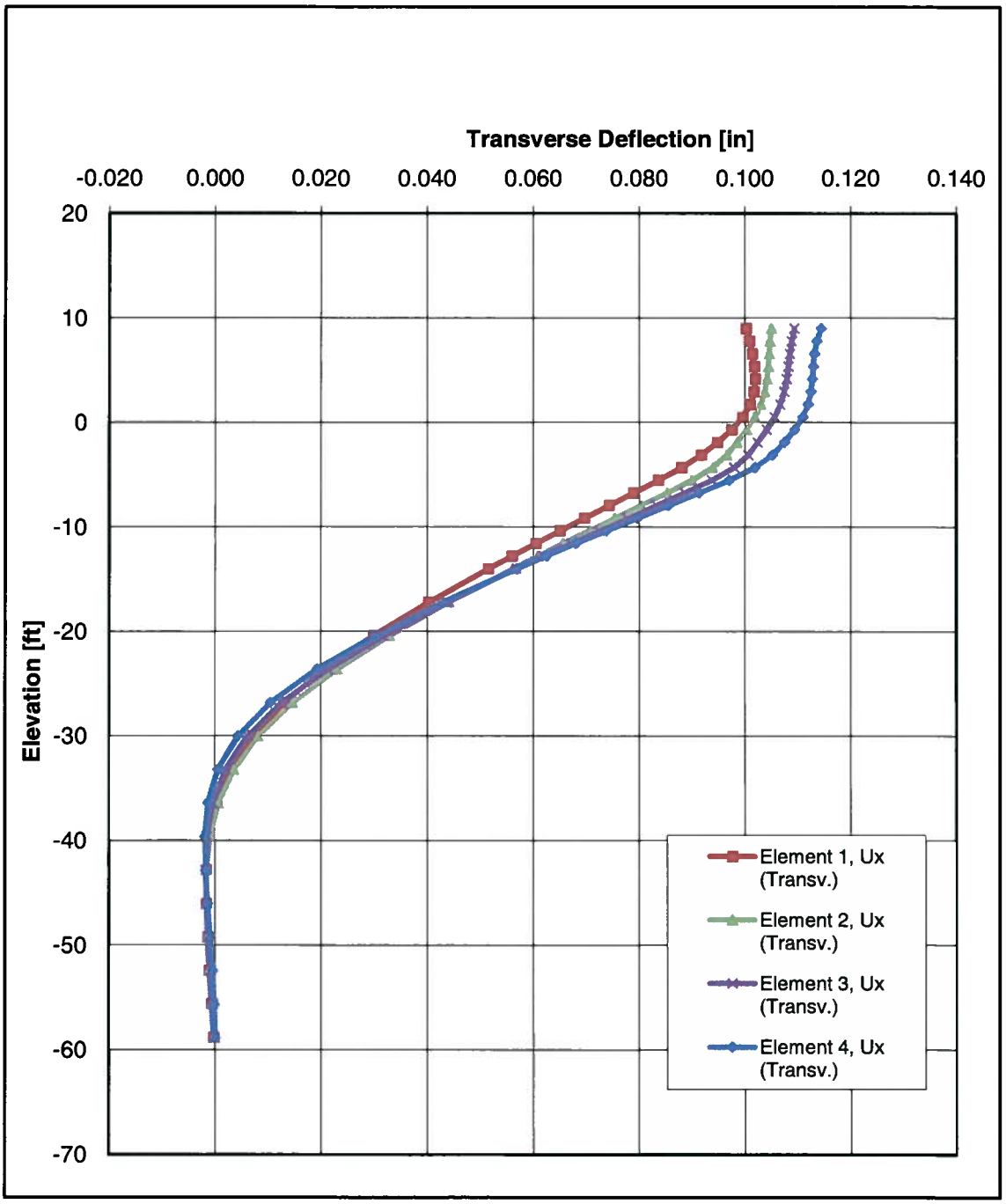
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	SFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



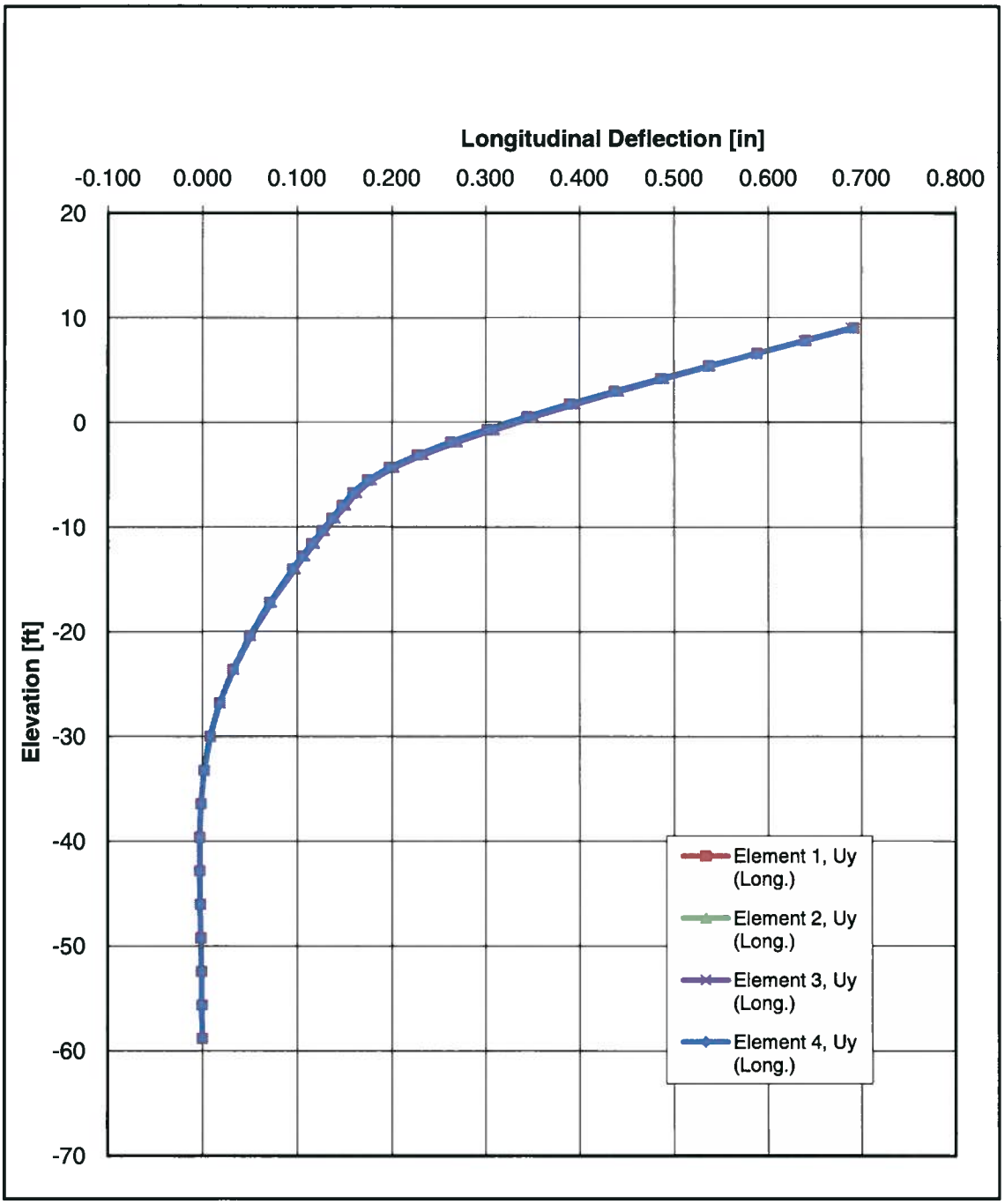
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	3 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Deflection vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



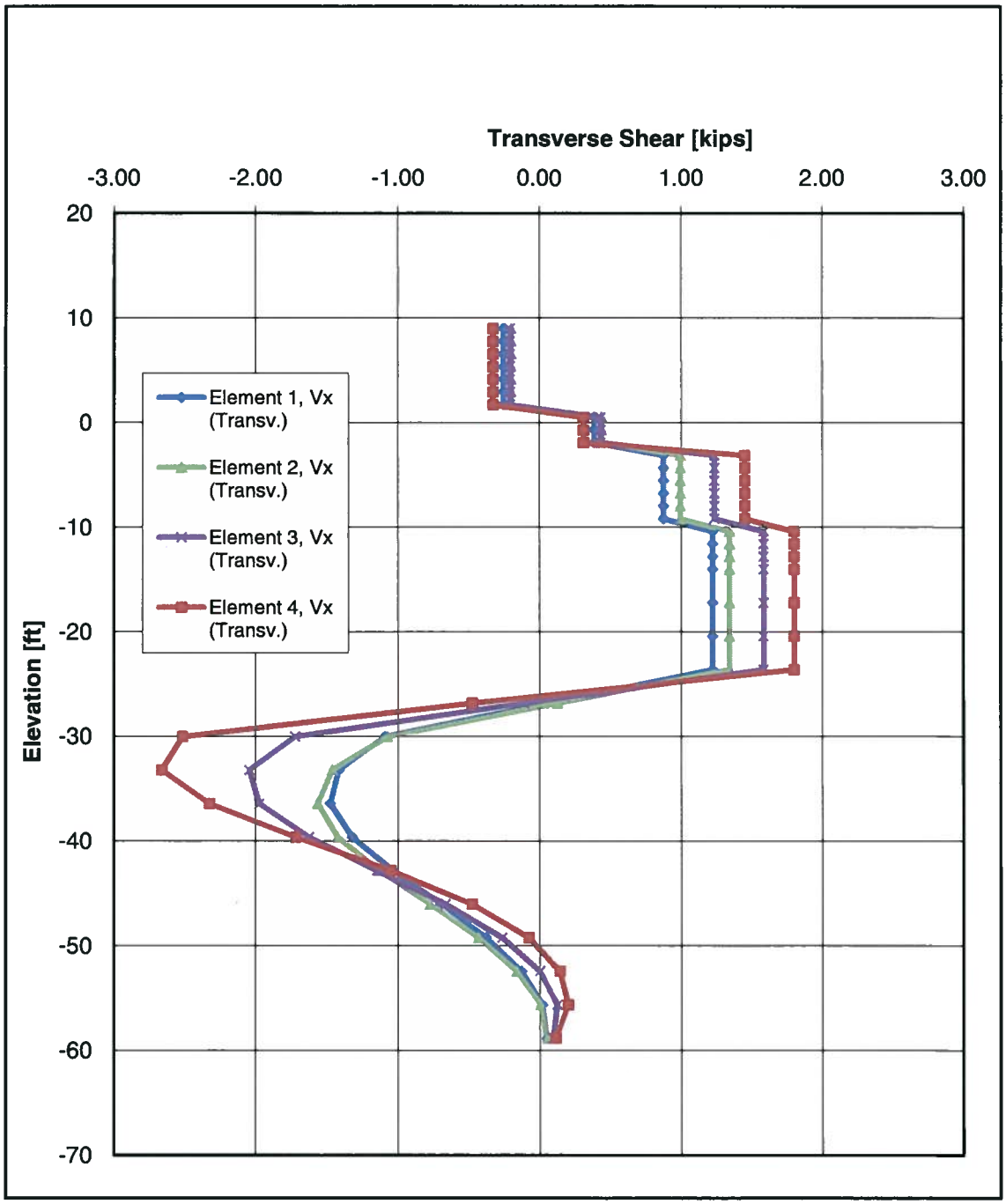
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	4 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML

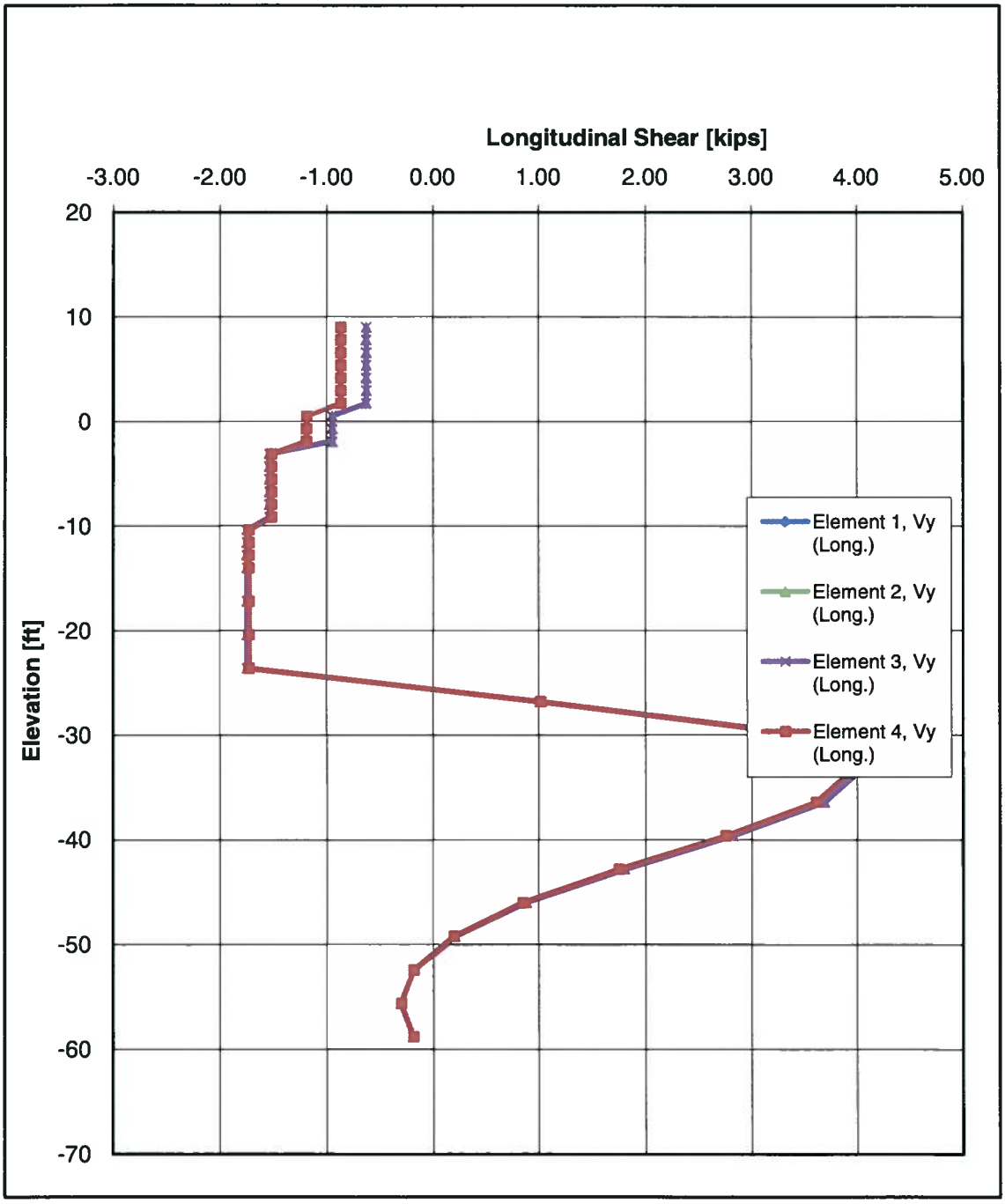


FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Shear vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



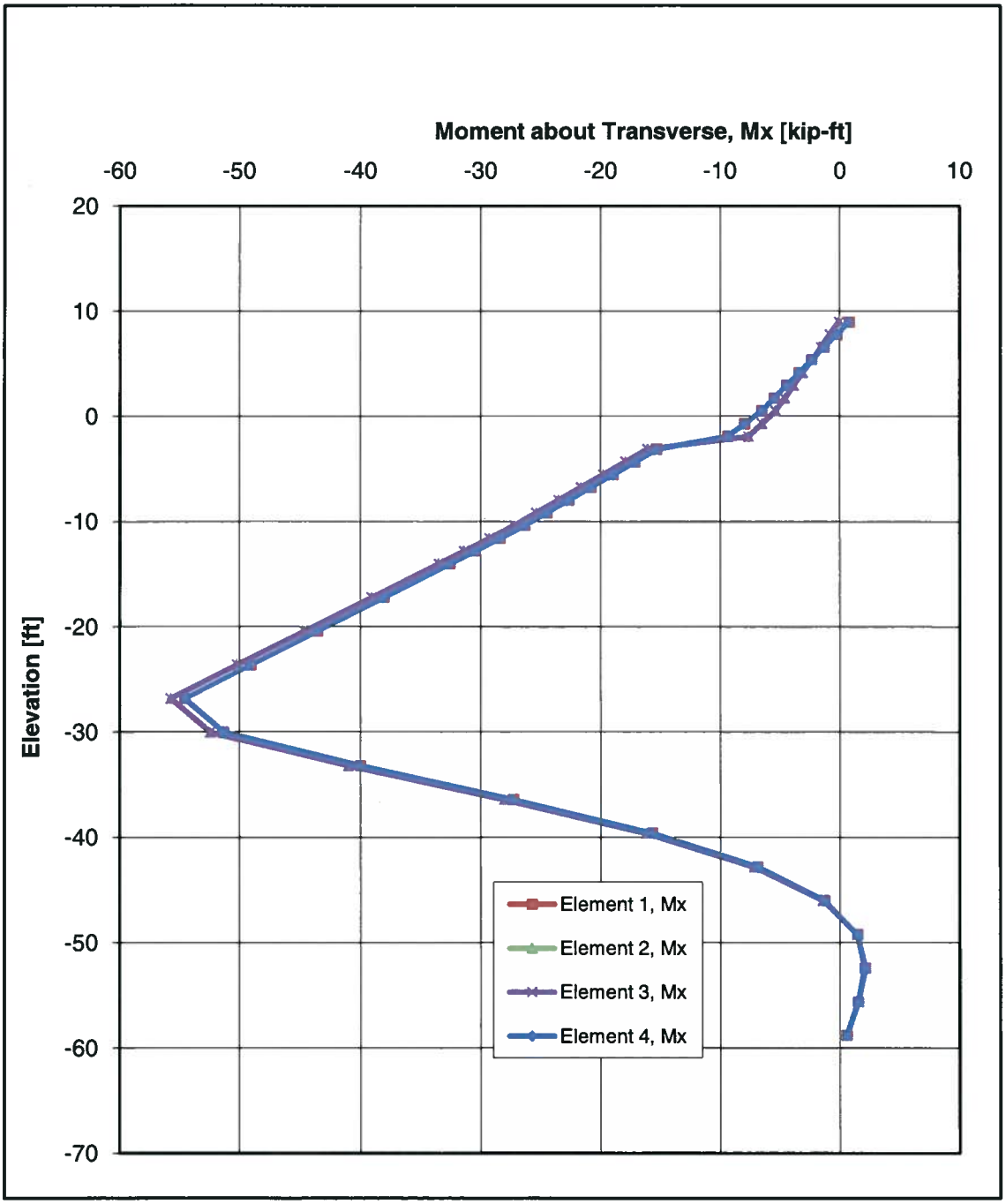
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

STR V Max Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



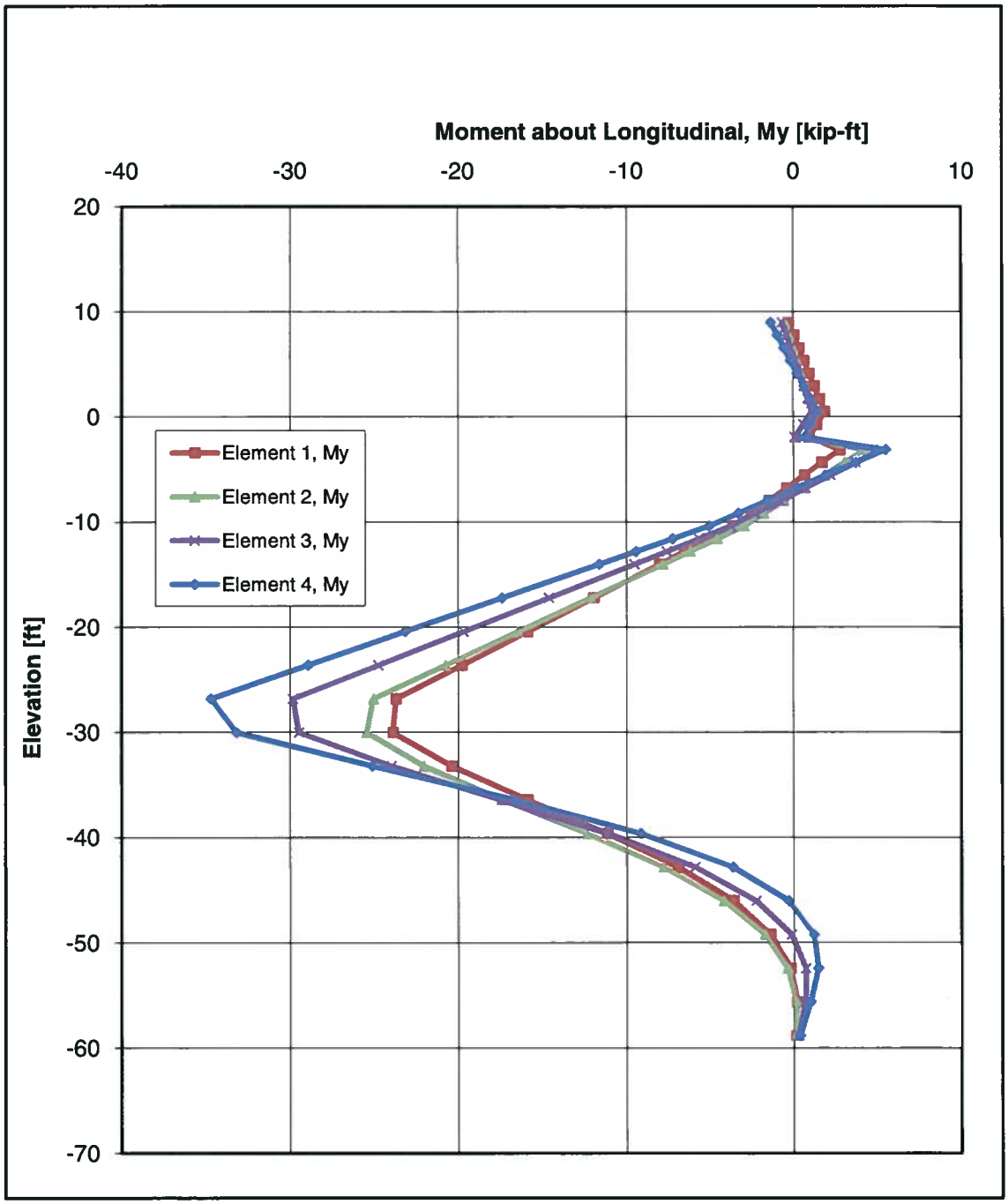
HALEY & ALDRICH	CALCULATIONS	File No.	
		Sheet	5 of 5
Client		Date	
Project	Sewall's Bridge	Computed by	JLL
Subject	Pile Group Analysis of 30 in Pipe Piles (No Concrete)	Checked by	EFW

FB-MultiPier Results for Bent 11 (30x0.5 in PP WITH CONC)

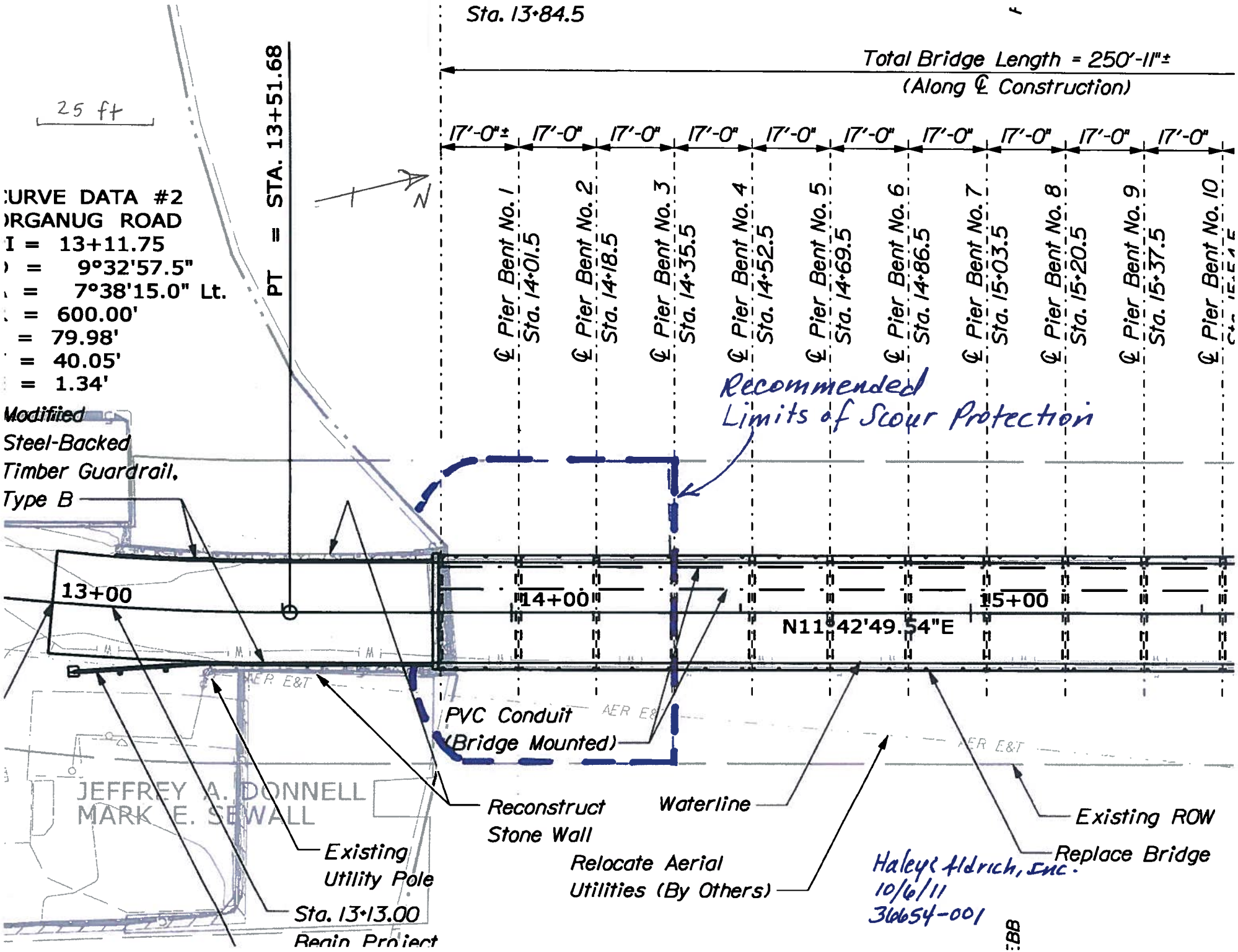
STR V Max Wind at 60 (10 ft SCOUR)

Bending Moment vs. Elevation

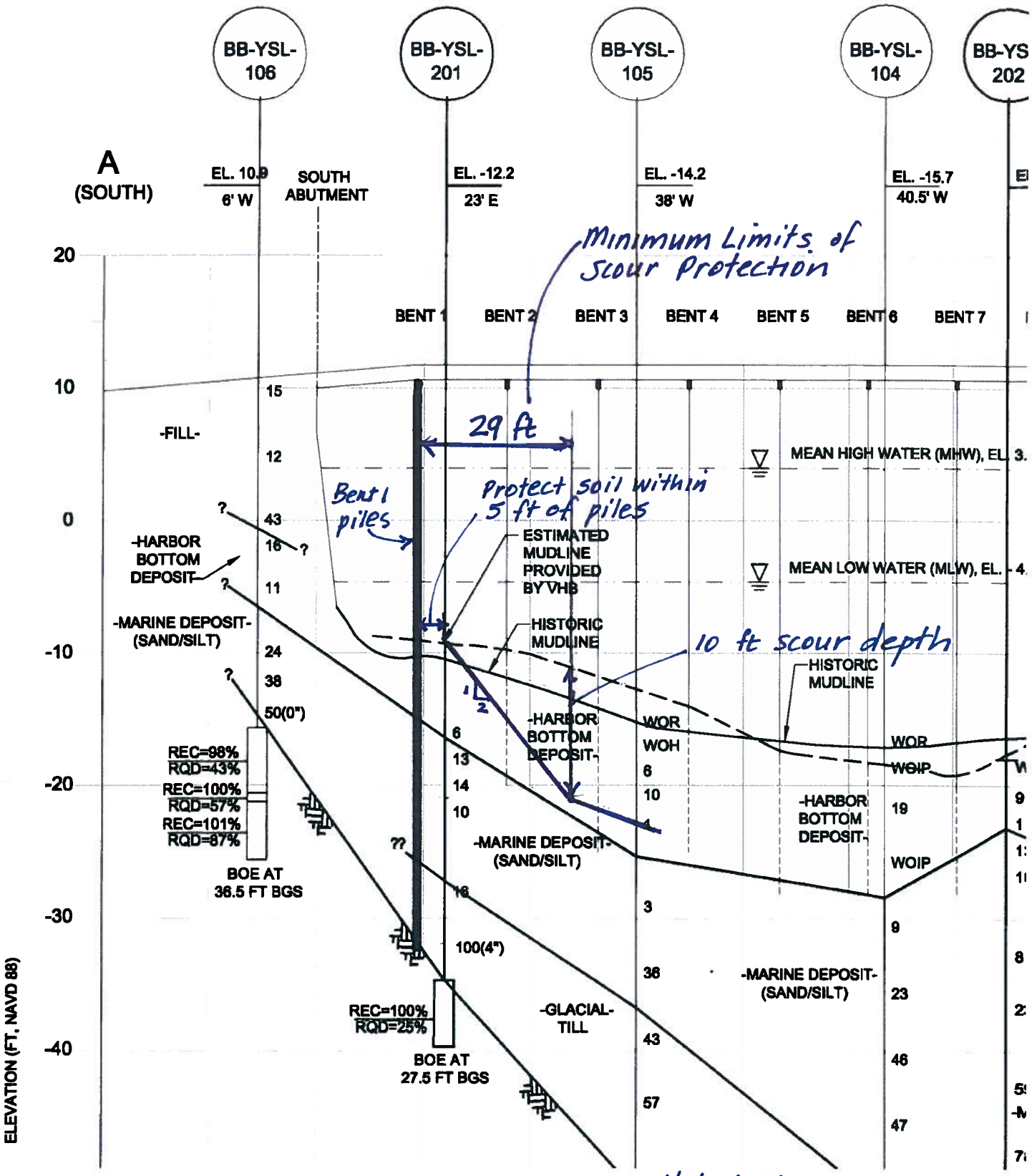
Data File: G:\36654-Sewalls Bridge\001\FB-MPier\Bent 11\2011-1018-HAI-Bent 11-30x0_5in PP-10 ft SCOUR-STR V Max Wind at 60-WITH CONCRETE-D1.XML



9. Recommended Rip Rap Limits


Sta. 13+84.5
Total Bridge Length = 250'-11"±
(Along ℄ Construction)
17'-0"± 17'-0" 17'-0" 17'-0" 17'-0" 17'-0" 17'-0" 17'-0" 17'-0" 17'-0"
℄ Pier Bent No. 1 Sta. 14+01.5
℄ Pier Bent No. 2 Sta. 14+18.5
℄ Pier Bent No. 3 Sta. 14+35.5
℄ Pier Bent No. 4 Sta. 14+52.5
℄ Pier Bent No. 5 Sta. 14+69.5
℄ Pier Bent No. 6 Sta. 14+86.5
℄ Pier Bent No. 7 Sta. 15+03.5
℄ Pier Bent No. 8 Sta. 15+20.5
℄ Pier Bent No. 9 Sta. 15+37.5
℄ Pier Bent No. 10
25 ft
PT = STA. 13+51.68
N
CURVE DATA #2
ORGANUG ROAD
I = 13+11.75
= 9°32'57.5"
= 7°38'15.0" Lt.
= 600.00'
= 79.98'
= 40.05'
= 1.34'
Modified Steel-Backed Timber Guardrail, Type B
Recommended Limits of Scour Protection
13+00
14+00
15+00
N11°42'49.54"E
AER E&T
PVC Conduit (Bridge Mounted)
JEFFREY A. DONNELL
MARK E. SEWALL
Reconstruct Stone Wall
Waterline
Existing ROW
Existing Utility Pole
Relocate Aerial Utilities (By Others)
Replace Bridge
Sta. 13+13.00 Begin Project
Haley & Aldrich, Inc.
10/6/11
316654-001

25 ft(H) / 10 ft(V)



Haley & Aldrich, Inc.
10/6/11
36654-001

10. Pipe Piles – Pile Settlement

HALEY & ALDRICH	CALCULATIONS	File No.	36654-001
		Sheet	1 of 1
Client	Maine Department of Transportation	Date	12/20/11
Project	Sewalls Bridge, York, Maine	Computed by	EFW
Subject	Pile Settlement and Elastic Pile Compression	Checked by	

Objective:

Determine pile settlement due to elastic shortening of the steel pipe piles as well as pile tip settlement

Assumptions:

- maximum axial load = 94.9 kips (Service I, Bents 11 & 12)
- pile consists of 30-in. diameter, 1/2-in. wall thickness pipe pile
- maximum pile length at Bents 11 & 12 = 53 ft

Calculations:

Elastic Compression:

Area = 46.34 in^2

E = 29000000 lb/in^2

$\delta = (P * L) / (A * E)$
$= (94.9 \text{ kips} * 1000 \text{ lb/kip} * 53 \text{ ft} * 12 \text{ in/ft}) / (46.34 \text{ in}^2 * 29{,}000{,}000 \text{ lb/in}^2)$
$= 0.045$ in.

Tip Settlement:

Piles will be driven to a resistance greater than the Service Limit State and will likely penetrate fractured, weathered or decomposted rock.

Therefore pile tip movement is negligible.

11. Estimate of Frost Penetration

HALEY & ALDRICH

Calculations

Client	MAINEDOT	File No.	36654-000
Project	SEWALL'S BRIDGE - YORK, MAINE	Sheet	1 of 1
Subject	ESTIMATE OF FROST PENETRATION	Date	8/17/2010
		Computed By	BCS
		Checked By	EFW

OBJECTIVE: ESTIMATE MAXIMUM DEPTH OF FROST PENETRATION BASED ON GEOGRAPHIC LOCATION AND SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ENCOUNTERED IN SUBSURFACE EXPLORATIONS IN ACCORDANCE WITH THE MAINEDOT BDG.

REFERENCE:

1 MAINE DEPARTMENT OF TRANSPORTATION (MAINEDOT) BRIDGE DESIGN GUIDE (BDG), AUGUST 2003, WITH INTERIM REVISIONS THROUGH AUGUST 2008.

EVALUATION:

LOOK AT SUBSURFACE CONDITIONS ENCOUNTERED IN TEST BORINGS DRILLED AT THE NORTH AND SOUTH APPROACHES (BB-YSL-101 AND BB-YSL-106).

SUBSURFACE CONDITIONS CONSIST OF 0.5 TO 0.7 FT BITUMINOUS CONCRETE OVERLYING MAN-PLACED FILL. MAN-PLACED FILL GENERALLY CONSISTS OF THE FOLLOWING....

- DRY, MED. SAND, SOME F. GRAVEL, LITTLE C-F SAND (0.0'-2.0')
- WET, SANDY SILT, LITTLE F. GRAVEL (5.0'-7.0')
- WET, SANDY SILT, SOME GRAVEL, TRACE BRICK (10.0'-12.0')

} BB-YSL-101

- DRY, M-F SAND, LITTLE C. SAND, LITTLE F. GRAVEL (0.0'-2.0')
- DAMP TO WET, M-F SAND, SOME COARSE SAND, SOME F. GRAVEL, TRACE SILT (5.0'-7.0')

} BB-YSL-106

NOTE: NO LABORATORY GRAIN SIZE OR WATER CONTENT TESTS PERFORMED ON SOIL SAMPLES.

FROM FIGURE 5.1 (ATTACHED), ASSUME THE DESIGN FREEZING INDEX FOR YORK, MAINE IS APPROXIMATELY 1,000 FREEZING DEGREE DAYS.

BASED ON FIELD OBSERVATION OF COLLECTED SOIL SAMPLES (SUMMARIZED ABOVE), ASSUME WATER CONTENT OF PRIMARILY GRANULAR FILL SOILS IS 10% TO 20%.

FROM TABLE 5-1, MAXIMUM DEPTH OF FROST PENETRATION IS ANTICIPATED TO BE BETWEEN 4.5 FT AND 5.5 FT.

∴ RECOMMEND 5-FT MAX. DEPTH OF FROST PENETRATION

q_s	traffic live load surcharge pressure
Q_A	horizontal sliding force
Q_t	horizontal soil shear resistance along footing base (force)
$q_{allowable}$	allowable bearing pressure
$Q_{applied}$	applied footing load
R	resultant force at base of footing
R_g	beam/girder rotation (radians)
S_p	section modulus of the pile
t	footing thickness
W	total beam/girder live load, end span
W_{c1}, W_{c2}	weight of abutment wall, footing
W_s	weight of soil above heel
W_{toe}	weight of soil above toe
X_{DL}	distance from the point of interest to the dead load reaction (centerline of bearing)
X_{LL}	distance from the point of interest to the live load reaction (centerline of bearing)
X_{WS}	distance from the point of interest to the centroid of W_s
X_{WC1}	distance from the point of interest to the centroid of W_{c1}
X_{WC2}	distance from the point of interest to the centroid of W_{c2}
X_{wtoe}	distance from the point of interest to the centroid of W_{toe}
y	the depth of seal from top of seal to bottom of seal
z	the depth of water from water surface to bottom of sea
α	batter angle from the horizontal plane
β	backfill slope
δ	friction angle between soil/bedrock and concrete
γ	soil weight
ϕ	soil internal angle of friction
σ_p	pile stress
σ_v	vertical bearing stress at base of footing
τ	horizontal superstructure forces transmitted through bearing at wall top

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	[illegible]	54.2	46.6	42.2
1400	79.2	65.5	[illegible]	56.3	48.5	43.9
1500	82.1	67.9	[illegible]	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Note: Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

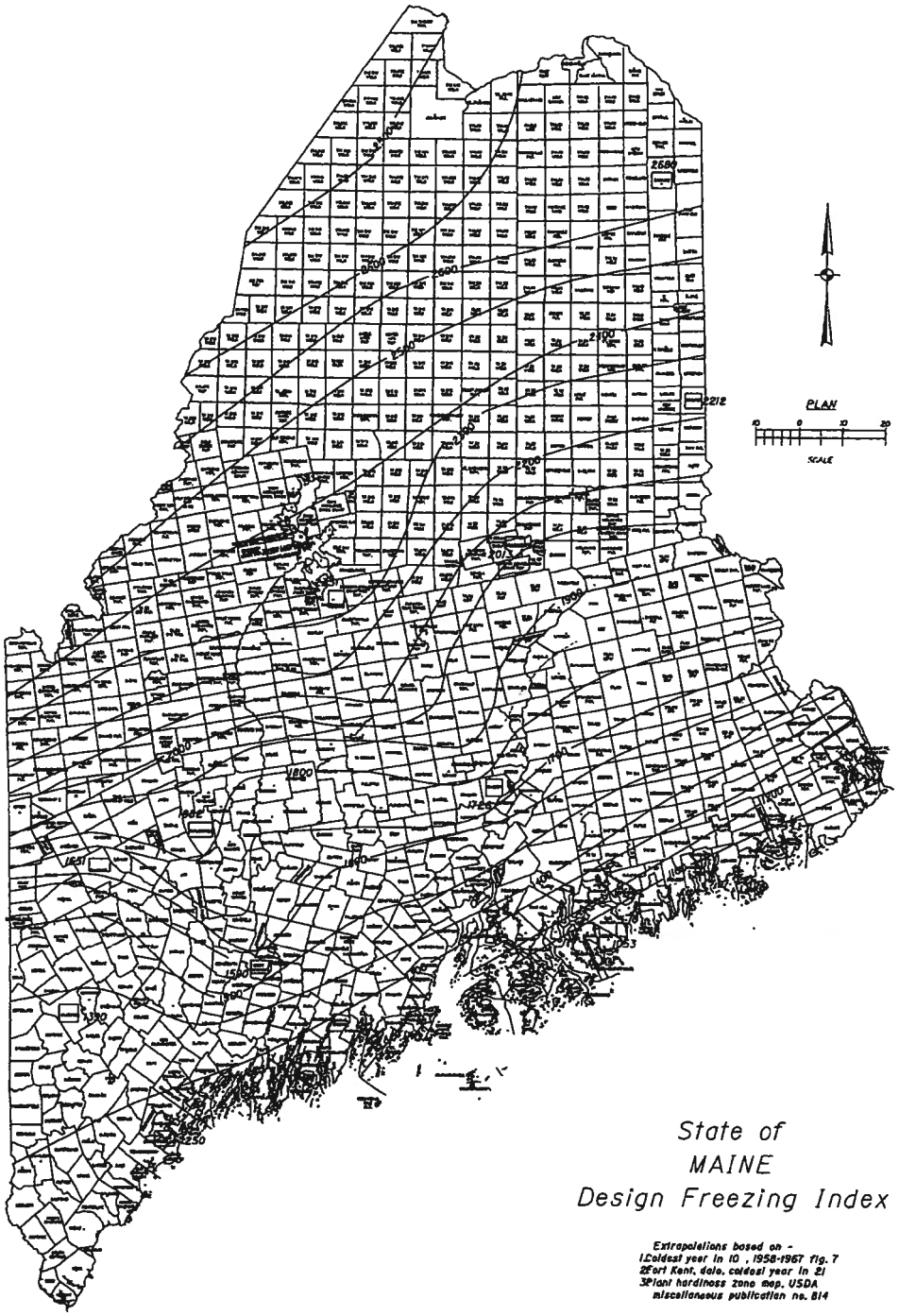


Figure 5-1 Maine Design Freezing Index Map

Example 5-1 illustrates how to use Table 5-1 and Figure 5-1 to determine the depth of frost penetration:

Example 5-1 Depth of Frost Penetration

Given: Site location is Freeport, Maine
Soil conditions: Silty fine to coarse Sand

Step 1. From Figure 5-1 Design Freezing Index = 1300 degree-days
Step 2. From laboratory results: soil water content = 28% and major constituent Sand
Step 3. From Table 5-1: Depth of frost penetration = 54 inches = 4.5 feet

Spread footings founded on bedrock require no minimum embedment depth. Pile supported footings will be embedded for frost protection. The minimum depth of embedment will be calculated using the techniques discussed in Example 5-1. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

The final depth of footing embedment may be controlled by the calculated scour depth and be deeper than the depth required for frost protection. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.2.2 Seal Cofferdams

Seal cofferdams are used when a substructure unit must be constructed with its foundation more than 4 feet below the water table, to counteract the buoyant forces produced during pumping of the cofferdam. Once the cofferdam is constructed, the seal is placed under water and water is then pumped out of the cofferdam. This provides a dry platform for construction of the spread footing, or in the case of a pile foundation, the distribution slab. When a seal is needed, the top of footing or distribution slab is located approximately at streambed, and the depth of seal is calculated based upon the buoyancy of the concrete under the expected water surface during construction. The following formula can be used:

$$150 \cdot y = 62.4 \cdot z$$

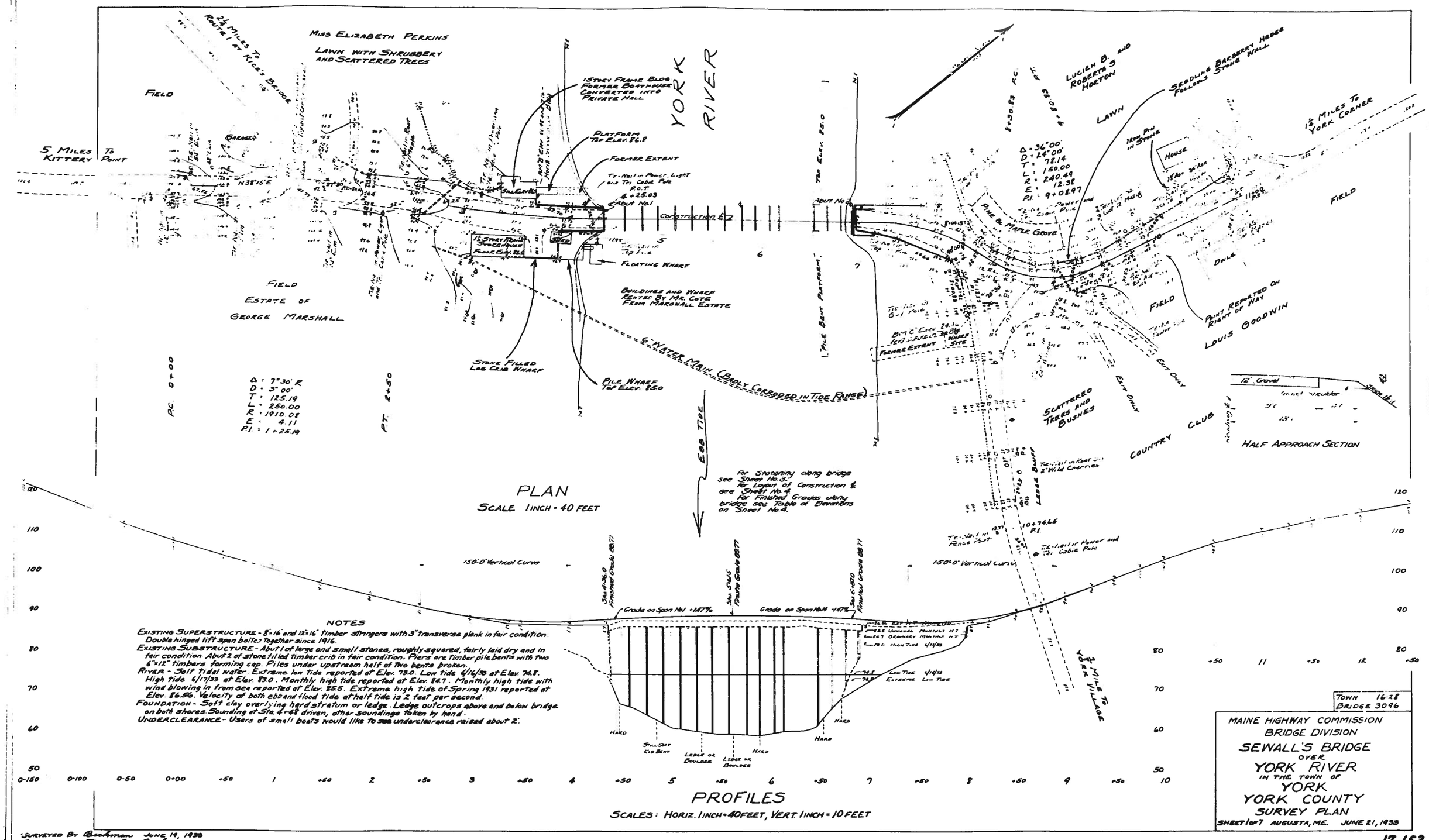
where:

150 lb/ft^3 =	unit weight of concrete
62.4 lb/ft^3 =	unit weight of water
y =	the depth of seal from top of seal to bottom of seal
z =	the depth of water from water surface to bottom of seal

Anchorage of the footing or distribution slab to the seal is required. For pile-supported foundations, this can be accomplished by extending the piles into

APPENDIX D

Historic Information, Existing Timber Piles



NOTES

EXISTING SUPERSTRUCTURE - 8"×16" and 12"×16" timber stringers with 3" transverse plank in fair condition. Double hinged lift span bolted together since 1916.

EXISTING SUBSTRUCTURE - Abut 1 of large and small stones, roughly squared, fairly laid dry and in fair condition. Abut 2 of stone filled timber crib in fair condition. Piers are timber pile bents with two 6"×12" timbers forming cap. Piles under upstream half of two bents broken.

RIVER - Salt tidal water. Extreme low tide reported at Elev. 73.0. Low tide 6/16/33 at Elev. 74.8. High tide 6/17/33 at Elev. 83.0. Monthly high tide reported at Elev. 84.7. Monthly high tide with wind blowing in from sea reported at Elev. 85.5. Extreme high tide of Spring 1931 reported at Elev. 86.56. Velocity of both ebb and flood tide at half tide is 2 feet per second.

FOUNDATION - Soft clay overlying hard stratum or ledge. Ledge outcrops above and below bridge on both shores. Sounding at Sta. 4+48 driven, other soundings taken by hand.

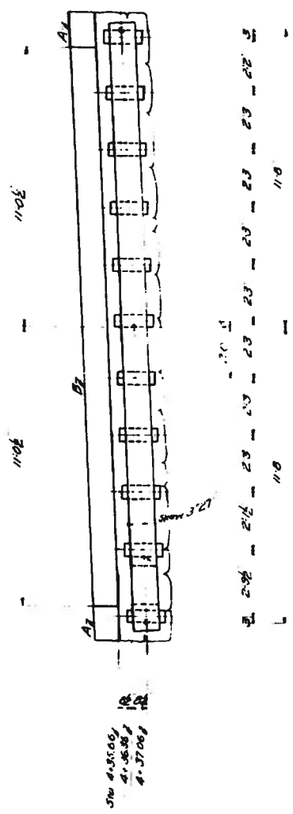
UNDERCLEARANCE - Users of small boats would like to see underclearance raised about 2'.

TOWN 16.28
BRIDGE 3096

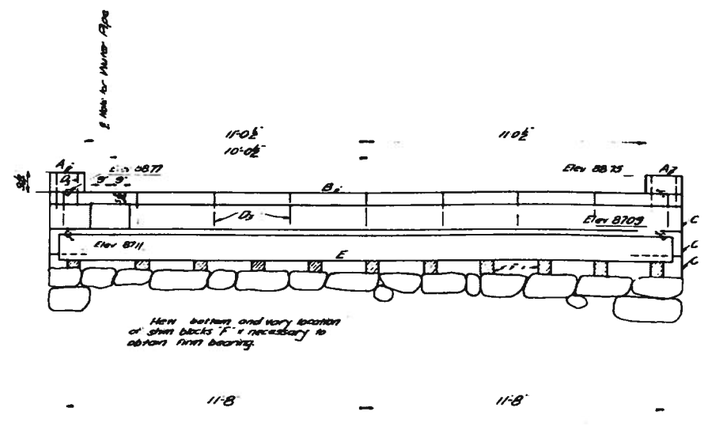
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
SURVEY PLAN
SHEET 1 of 7 AUGUSTA, ME. JUNE 21, 1933

SURVEYED BY Beckman JUNE 19, 1933
PLOTTED AND TRACED BY Beckman

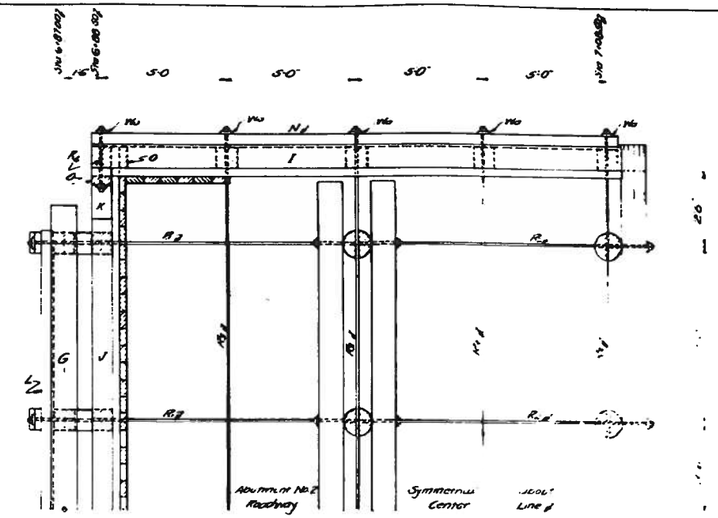
17-152



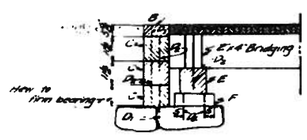
PLAN ABUTMENT No.1



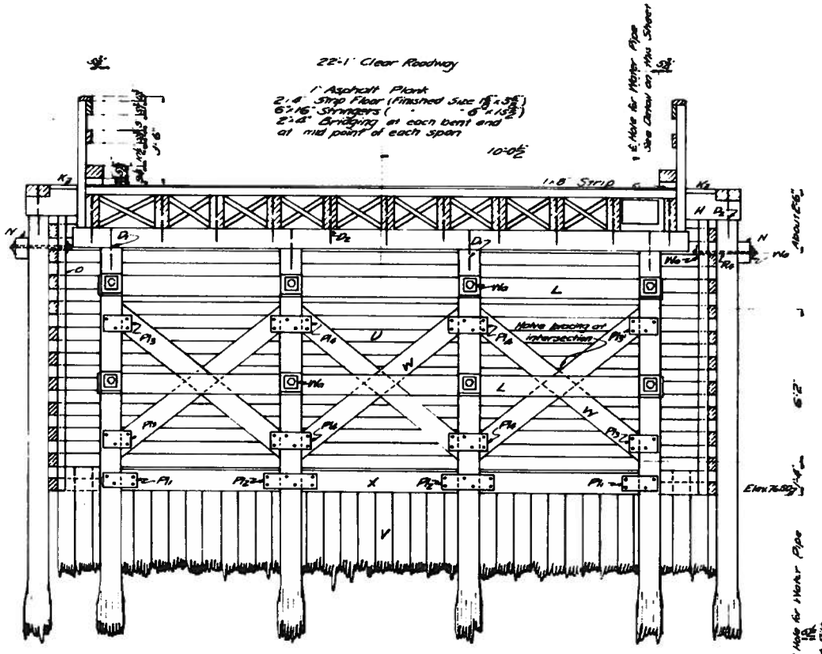
FRONT ELEVATION ABUTMENT No.1
PERPENDICULAR TO ROADWAY ℄



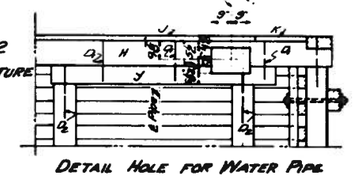
HALF PLAN ABUTMENT No.2



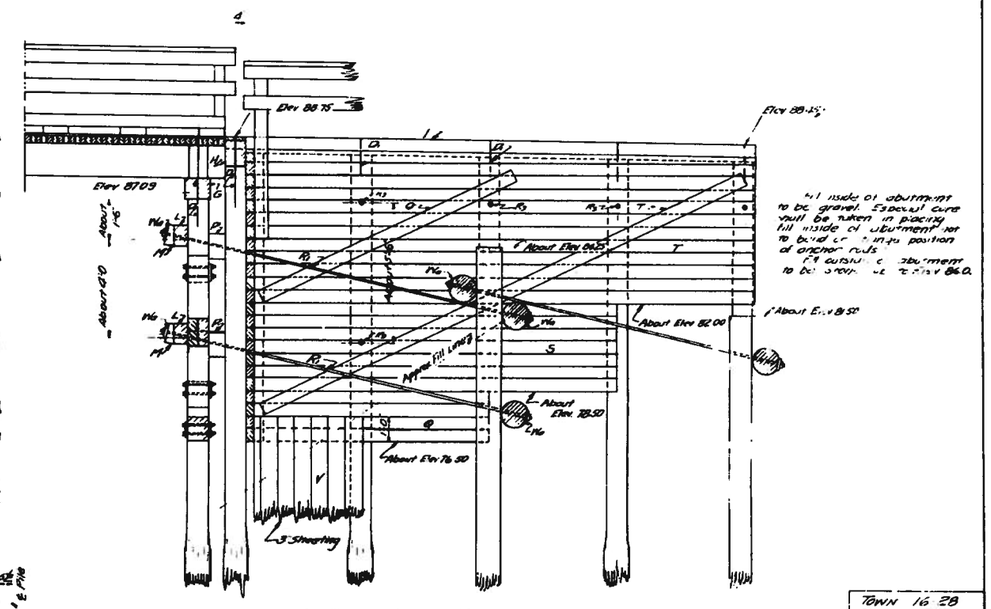
SECTION ABUTMENT No.1



FRONT ELEVATION ABUTMENT No.2
TRANSVERSE SECTION SUPERSTRUCTURE



DETAIL HOLE FOR WATER PIPE
ABUTMENT No.2



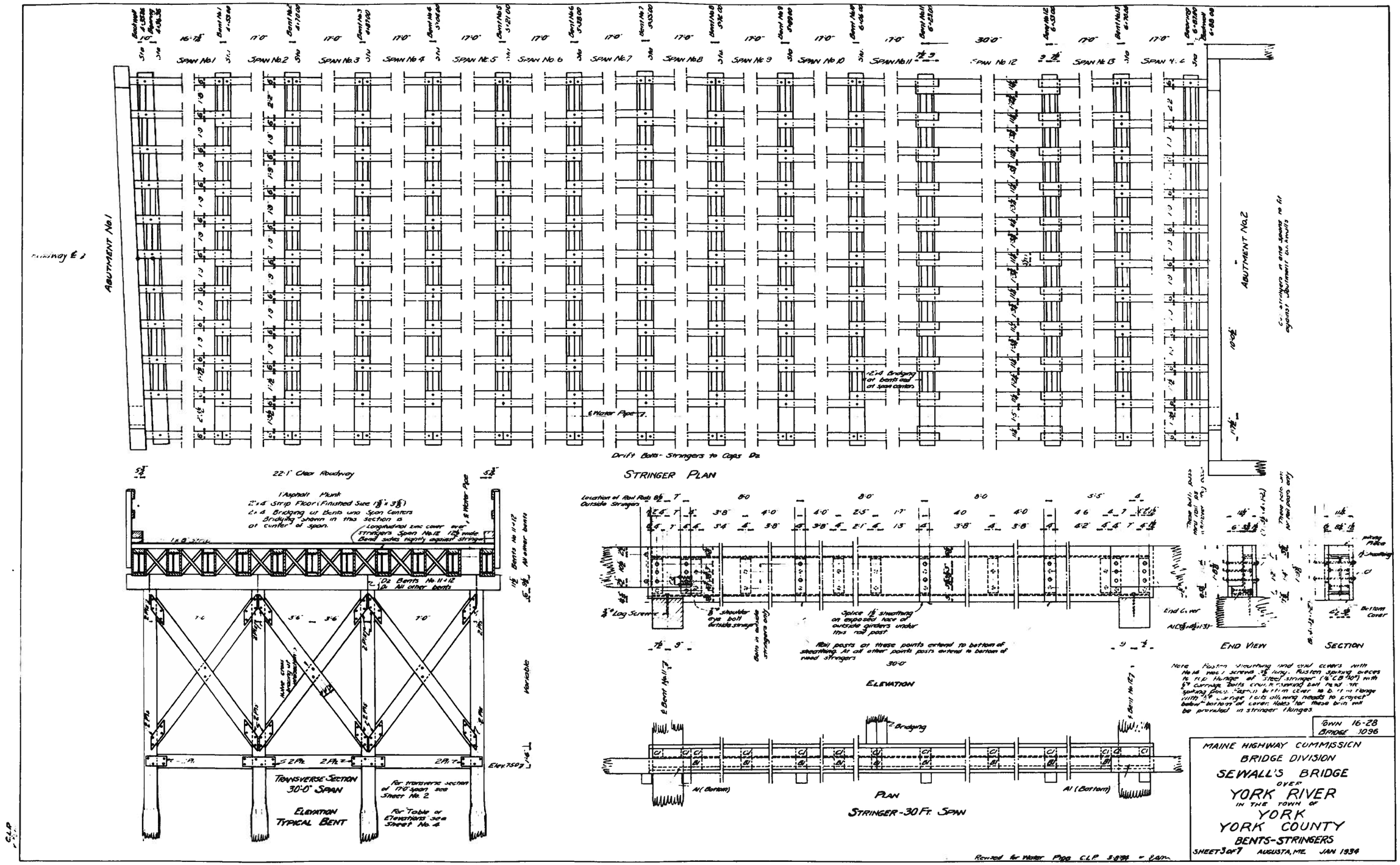
SECTION ALONG ROAD CENTER LINE
ABUTMENT No. 2

Revised for Water Pipe C.L.P. 3-8-34

TOWN 16 28
BRIDGE 3096

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
ABUTMENTS
SHEET 2 of 7 AUGUSTA, ME. JAN. 1934

17-153


ABUTMENT No.1
ABUTMENT No.2
SPAN No.1
SPAN No.2
SPAN No.3
SPAN No.4
SPAN No.5
SPAN No.6
SPAN No.7
SPAN No.8
SPAN No.9
SPAN No.10
SPAN No.11
SPAN No.12
SPAN No.13
SPAN No.14
Roadway ℄
Drift Bolts - Stringers to Caps
STRINGER PLAN
22.1' Clear Roadway
1" Asphalt Plank
2"x4" Strip Floor (Finished Size 1⅞"x3⅞")
2"x4" Bridging at Bents and Span Centers
Bridging shown in this section is at center of span.
Longitudinal zinc cover over stringers Span No.12 12" wide
Bent sides painted against stringer
Bents No.11+12 Heavier bents
Variable
TRANSVERSE SECTION 30'-0" SPAN
ELEVATION TYPICAL BENT
For transverse section of 17'-0" span see Sheet No. 2
For Table of Elevations see Sheet No. 4
Location of Rail Posts Outside Stringers
Log Screws
Splice ½" sheathing on exposed face of outside girders under this rail post
Rail posts at these points extend to bottom of sheathing. At all other points posts extend to bottom of wood stringers
ELEVATION
Bridging
Al (Bottom)
PLAN
STRINGER - 30 Ft. SPAN
End Cover
Bottom Cover
END VIEW
SECTION
Town 16-28
Bridge 1096
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
BENTS-STRINGERS
SHEET 3 of 7 AUGUSTA, ME. JAN 1934
Revised for Water Pipe C.L.P. 5-8-34

19-154

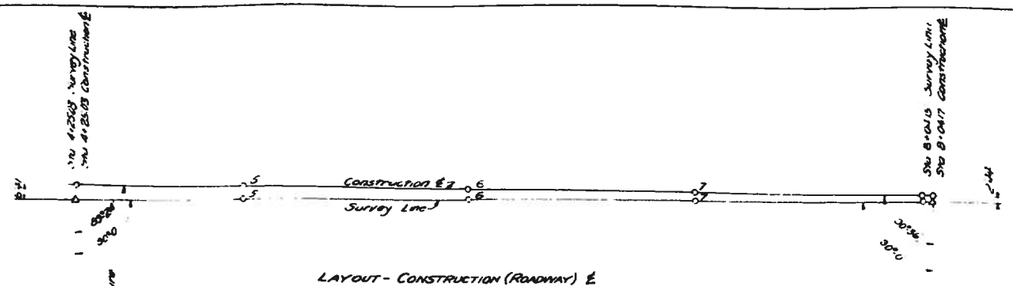
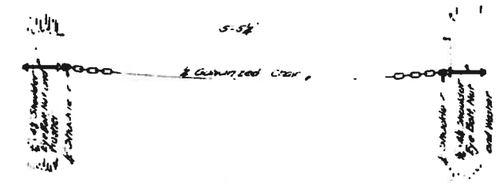
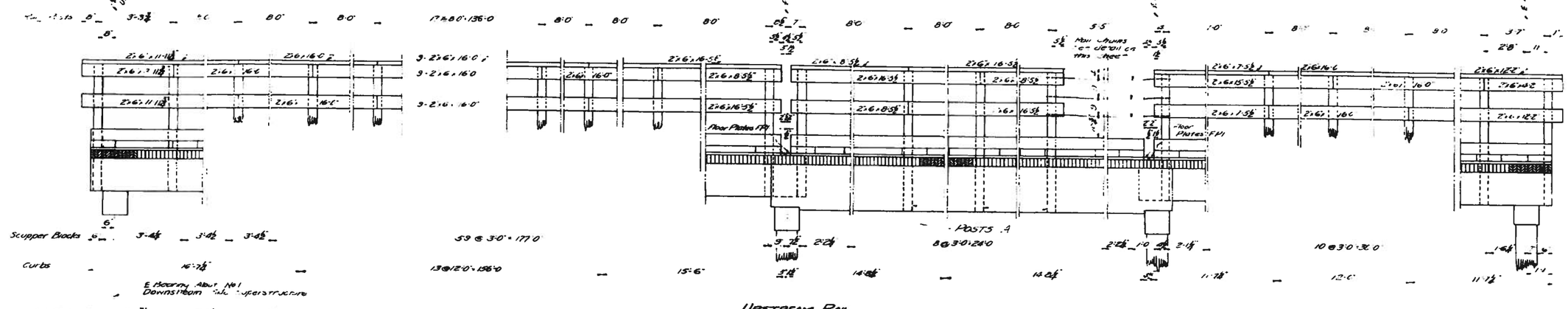
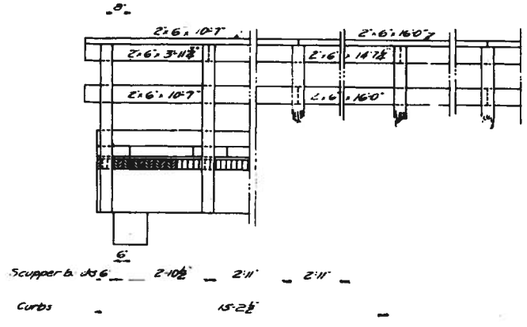


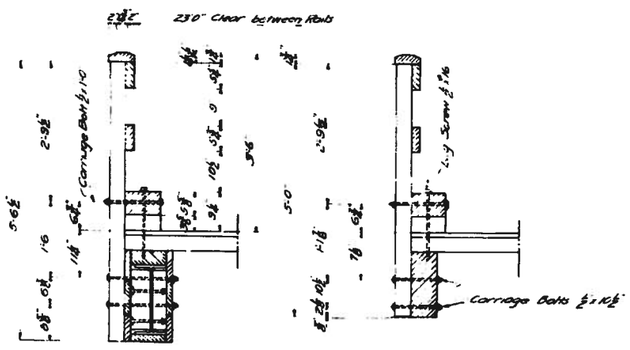
TABLE OF ELEVATIONS

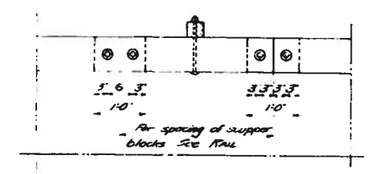
BENT No.	ELEVATIONS		
	GRADE	TOP OF CAP	TOP OF PILES
1	89.02	87.34	86.55
2	89.24	87.56	86.77
3	89.42	87.74	86.95
4	89.54	87.89	87.09
5	89.67	87.99	87.20
6	89.73	88.05	87.26
7	89.77	88.09	87.30
8	89.74	88.08	87.29
9	89.72	88.04	87.25
10	89.64	87.96	87.17
11	89.53	87.85	86.99
12	89.24	87.16	86.20
13	89.02	87.34	86.55

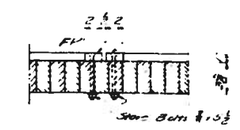










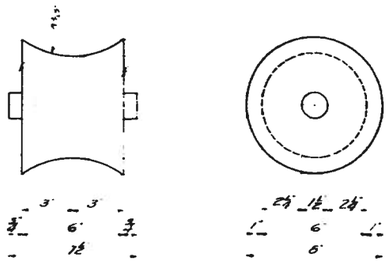


TOWN 16-28
BRIDGE 3096

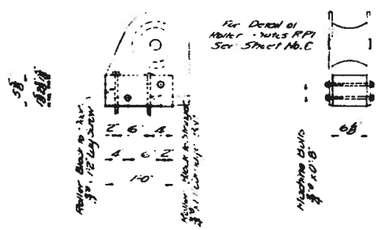
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
BRIDGE RAIL

SHEET 4 OF 7 AUGUSTA, ME., JAN. 1934

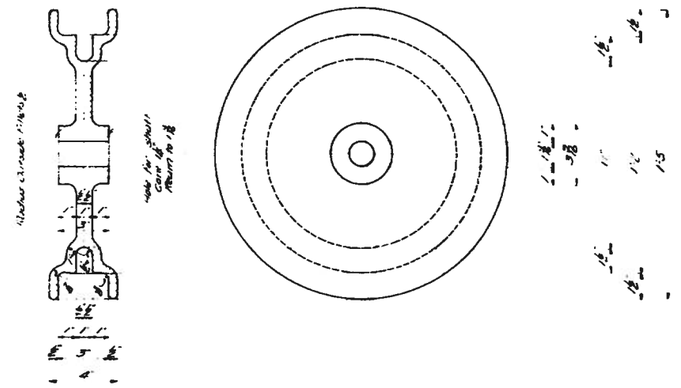
17-155



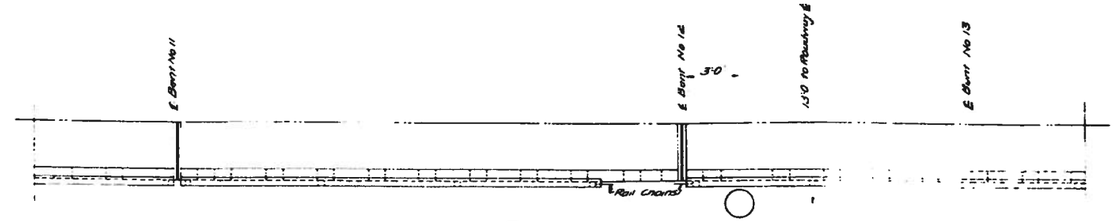
CAST IRON ROLLER
TWO REQUIRED



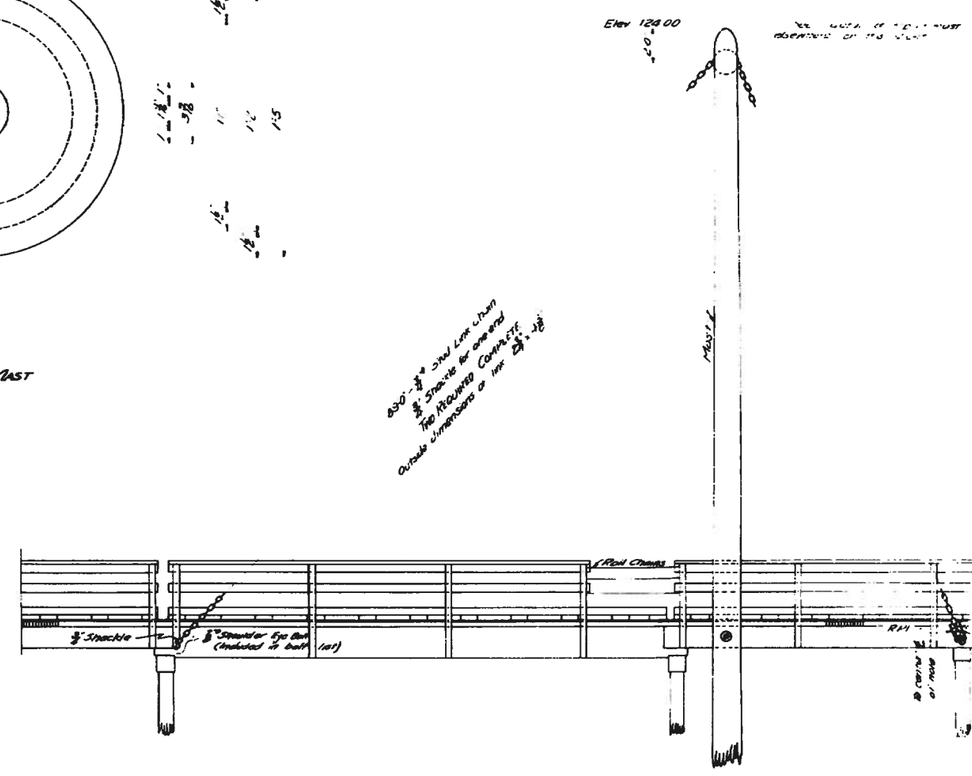
ROLLER BLOCK
5 3/8" x 6 1/8" x 1'-0"
See Timber Schedule



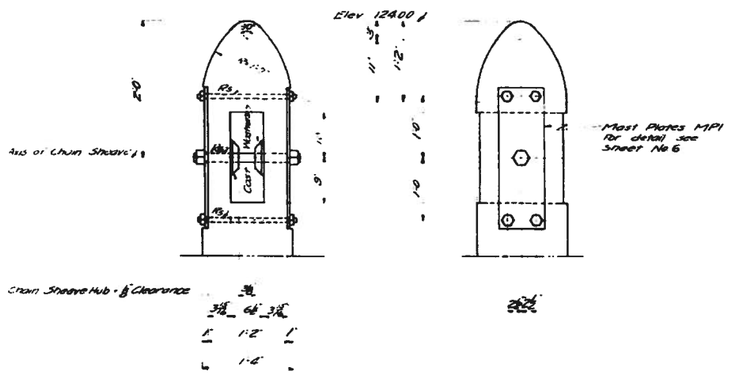
CHAIN SHEAVE FOR MAST
TWO REQUIRED
CAST IRON



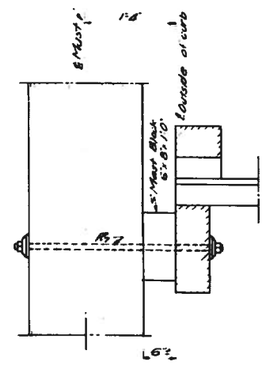
PLAN



ELEVATION



TOP OF MAST



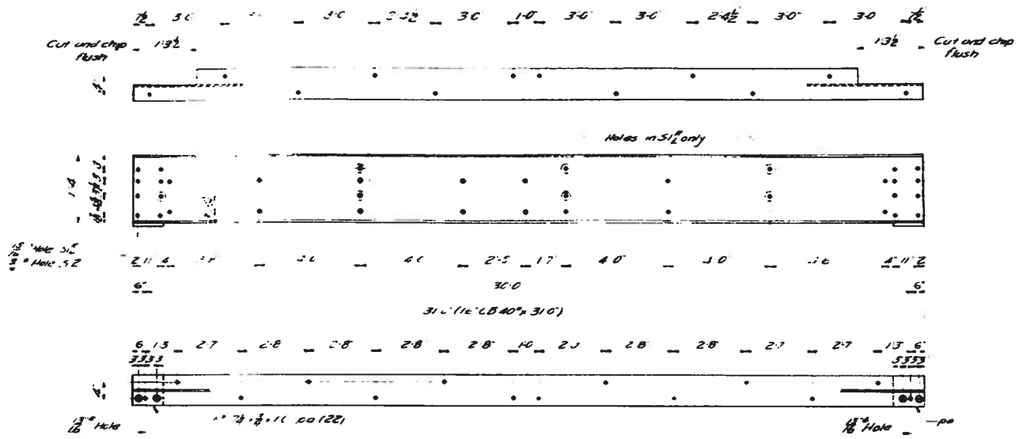
CONNECTION - MAST TO STRINGER

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
MAST - CHAIN DETAILS

SHEET 5 OF 7 AUGUSTA, ME. JAN 1936

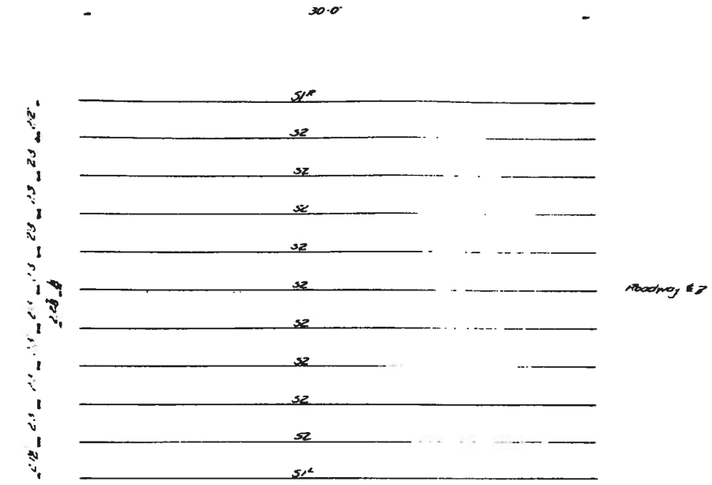
DWN. 6-28
J.O. NO. 3096

17-156

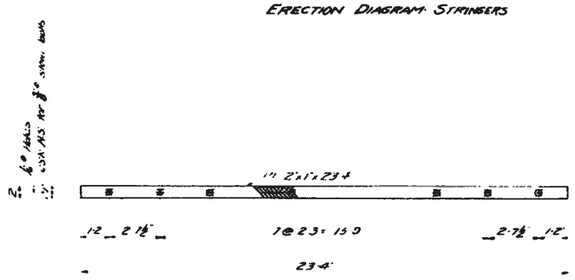


1 STRINGER S1R
1 STRINGER S1L OPP HAND
9 STRINGERS S2

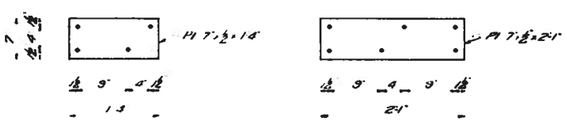
Holes 13/16" unless noted
Rivets 3/4"



ERECTION DIAGRAM STRINGERS

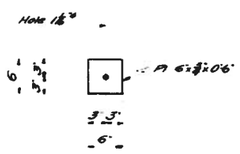


4 FLOOR PLATES FP1
Non-Skid
(30'-0" Span)

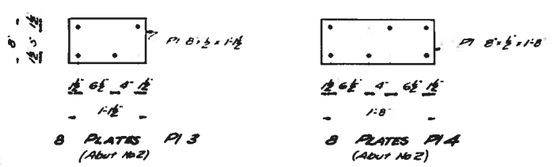


56 PLATES PI 1
(Bents and Abut No 2)

56 PLATES PI 2
Bents and Abut No 2

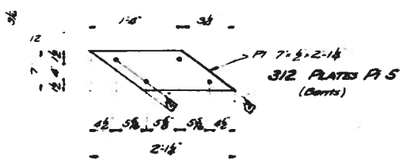


38 PLATES W6
(Abutment No 2)



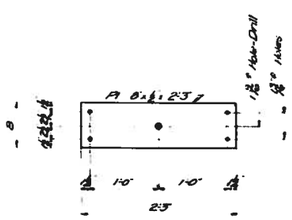
8 PLATES PI 3
(Abut No 2)

8 PLATES PI 4
(Abut No 2)

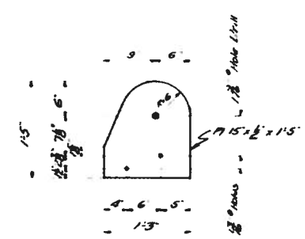


312 PLATES PI 5
(Bents)

All holes in Plates PI 1-5 13/16"



4 MAST PLATES MP1



4 ROLLER PLATES RP1

No structural steel necessary for installing water pipe, water pipe hangers or the walking floor is detailed on this sheet

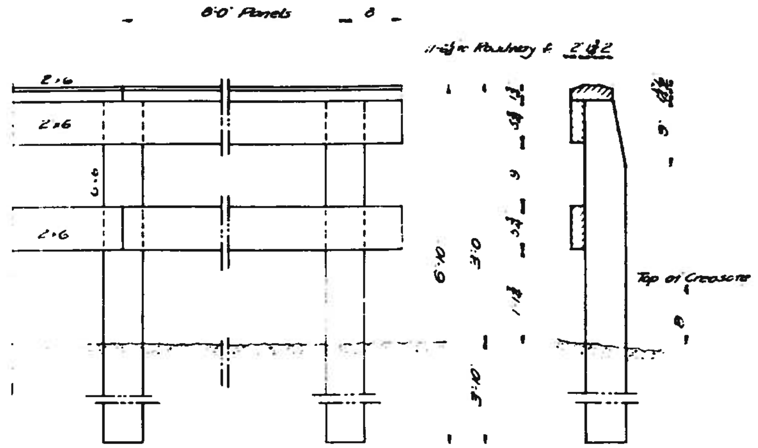
Revised Water Pipe C.L.P. Mar. 9 1934 — PMP.

Town	16-28
Bridge	3096

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
STRUCTURAL STEEL

SHEET 6 OF 7 AUGUSTA, ME., JAN. 1934

17-157

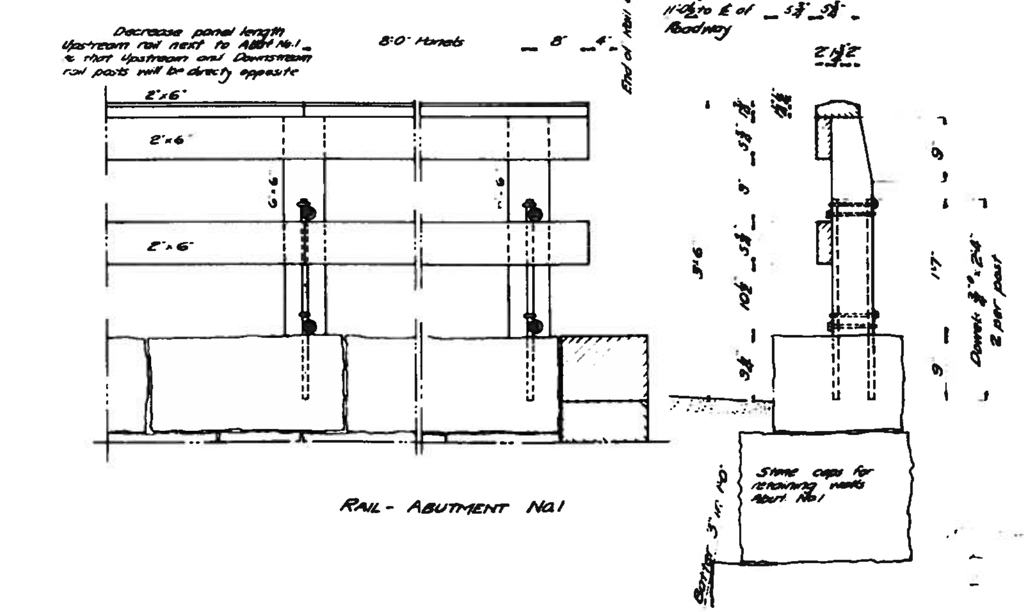


Guard Rail - Abut. No.2 Approach

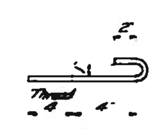
Posts for rail Abut. No 2 approach and on retaining walls of Abut. No1 shall be of seasoned cedar, oak or hackmatack. The posts are to be 6"x6" surfaced four sides. All posts to be set in the ground shall be creosoted up to 8" above ground elevation either by immersing for a period not less than six hours or by two brush treatments uniform, and thoroughly applied.

Cap and side rails shall be well seasoned spruce or other wood acceptable to the Engineer. All sides of rails shall be surfaced.

All posts where not creosoted and all rails shall be painted two coats of pure white lead paint.



Rail - Abutment No.1



Four of above bolts required for each post on retaining wall. Bolts to be ⅝" and furnished with a nut and washer for each bolt. End bolts protruding beyond nuts shall be cut off neatly.

Rods

Mark	Size	No.	Thread Ends	Remarks	
R1	1"x15'-6"	8	1'-0"	2 Nuts Each	Abut No2
R2	1"x14'-0"	4	1'-0"	2 Nuts Each	Abut No2
R3	1"x23'-6"	4	1'-0"	2 Nuts Each	Abut No2
R4	1"x2'-5"	2	0'-5	2 Nuts Each	Abut No2
R5	¾"x1'-5"	8	0'-3"	2 Nuts Each	Mast Plates
R6	1½"x1'-7"	2	0'-3"	2 Nuts+2 Cast Washers Each	Sheave Axis
R7	1"x3'-3"	2	0'-6	2 Nuts+2 Cast Washers Each	Mast to Stringer
R8	1"x1'-0	2	—		Chain Fasteners
R9	1"x28'-6"	1	1'-0"	2 Nuts Each	Abut No.2

Bolts

Kind	Size	No.	Plate Washers Each	Nuts Each	Remarks
Carriage	⅝"x1'-3	18	1	1	Thru rail posts and completely thru outside stringers Span 12
Carriage	⅝"x1'-1½"	4	1	1	Thru rail posts but not opposite sheathing outside stringers Span 12
Carriage	⅝"x1'-0	80	1	1	Completely thru stringers Span 12
Carriage	⅝"x0'-7⅜	250	1	1	Thru stringers but clear of sheathing Span 12
Carriage	⅝"x0'-9½	45	1	1	Thru stringers but clear of sheathing one side Span 12
Shoulder Eye Bolt	⅝"x1'-4"	2	1	1	Thru rail post and completely thru outside stringer Span 12
Carriage	⅝"x0'-4½	135	—	1	Top flange stringers Span 12
Carriage	⅝"x0'-5"	135	—	1	Bottom flange stringers Span 12
Carriage	⅝"x0'-6½	125	1	1	Rail posts to stringers except Span 12
Carriage	⅝"x1'-0"	70	1	1	Rail posts to curbs
Stove	⅜"x0'-5½	40	1	1	Floor plates
Machine	¾"x0'-8"	4	—	1	Roller blocks
Machine	¾"x1'-1"	975	—	1	Bracing, bents and abut. no 2
Carriage	¾"x1'-1"	2	1	1	Roller block to stringer
Carriage	¾"x0'-11"	85	1	1	Intersection of bracing
Carriage	¾"x0'-6½	4	1	1	Rail posts at end of chain

Lag Screws

Size	No.	Plate Washers Each	Remarks
⅝"x1'-4	350	1	Curbs and scupper blocks
¾"x0'-9	22	—	Stringers Span No.12
¾"x1'-2"	2	1	Roller blocks

Timber Schedule

Mark	Finished Size	No	Dress	Description	Remarks
				Rail and Curbs	
	3⅝"x5⅝"x5'-6½"	6	S4S	Posts A	
	3⅝"x3⅝"x5'-0"	64	S4S	Posts B	
	1⅝"x5⅝"x11'-11⅝"	2	S4S	Rails Upstream	
	1⅝"x5⅝"x3'-11⅝	2	S4S	Rails	
	1⅝"x5⅝"x19'-7	2	S4S	Rails Downstream	
	1⅝"x5⅝"x16'-5⅝	10	S4S	Rails	
	1⅝"x5⅝"x5'-5⅝	8	S4S	Rails	
	1⅝"x5⅝"x7'-5⅝	4	S4S	Rails	
	1⅝"x5⅝"x15'-5⅝	2	S4S	Rails	
	1⅝"x5⅝"x12'-2"	4	S4S	Rails	
	1⅝"x5⅝"x16'-7⅝	1	S4S	Rails Downstream	
	1⅝"x5⅝"x16'-0"	67	S4S	Rails	
	1⅝"x5⅝"x4'-2"	2	S4S	Rails	
	5⅝"x7½"x16'-7⅞	1	S4S	Curb - Upstream	
	5⅝"x7½"x15'-2⅞	1	S4S	Curb Downstream	
	5⅝"x7½"x12'-0"	28	S4S	Curbs	
	5⅝"x7½"x14'-8⅞	4	S4S	Curbs	
	5⅝"x7½"x11'-7⅞	4	S4S	Curbs	
	3⅝"x7½"x1'-0"	174	S4S	Scupper Blocks	
	3⅝"x7½"x15'-6"	2	S4S	Curbs	
				Miscellaneous	
	6"x8"x1'-0"	2	—	Mast Blocks	
	5⅝"x6½"x1'-0"	2	S4S	Roller Blocks	

Mark	Finished Size	No.	Dress	Description	Remarks
				Abutment No.1	
A	9½"x12"x3'-0	1	S2 1S	Parapets	Makes both
B	9⅝"x12"x25'-0"	1	S2 1S	Top Backwall Timbers	Cut
C	11½"x12"x25'-0"	3	S2 1S	Bottom Backwall Timbers	[illegible] bottom timber to [illegible]
E	11½"x12"x28'-0"	1	S2 1S	Bridge Seat	
F	6"x8"x18'-0"	1	—	Shim Blocks	[illegible]
				Abutment No.2	
G	9½"x12"x28'-0"	1	S2 1S	Bridge Seat	
H	10"x11½"x28'-0"	1	S2 0S	Front Cap	
I	7½"x11½"x21'-0"	2	S4S	Side Caps	
J	2"x10"x22'-0"	1	—	Top Wearing Strip	Cut
K	2"x10"x4'-0"	1	—	Wearing Strip Extensions	Makes both
L	6"x10"x22'-0"	2	—	Front Walers	
M	10"x10"x6'-0"	1	—	Beveled Washers	Makes all
N	6"x10"x21'-0"	2	—	Side Walers	
O	4"x10"x12'-0"	4	—	Vertical Spiking Pieces	
P	8"x10"x8'-0"	1	—	Compression Blocks	Makes all
Q	4"x6"x11'-0"	10	—	Side Planking	
S	4"x6"x16'-0"	4	—	Side Planking	
T	4"x6"x21'-0	26	—	Side Planking	
U	4"x6"x28'-0"	23	—	Front Planking	
V	3"x8"	55	—	Sheathing	Determine length field
W	10"x10"x10'-0"	6	—	Cross Bracing	
X	10"x10"x20'-0"	1	—	Struts	Makes all
Y	9½"x10"x10'-0"	1	S2S	Front Cap	
				Bents	
	11½"x12"x28'-0"	2	S2 1S	Caps Bents No11+12	
	9½"x12"x28'-0"	11	S2 1S	Caps All Other Bents	
	10"x10"x20'-0"	13	—	Struts	One piece makes struts for 1 bent
	10"x10"x12'-0"	78	—	Cross Bracing	
				Floor	
	6"x15½"x18'-0"	4	S2 6S	Stringers Span No1	Cut to fit against backwall
	6"x15½"x20'-0"	7	S2 6S	Stringers Span No1	Cut to fit against backwall
	6"x15½"x17'-0"	18	S2 6S	Outside Stringers Span Nos 2 to 10 Inc	
	6"x15½"x18'-0"	81	S2 6S	Inside Stringers Span Nos 2 to 10 Inc	
	6"x15½"x17'-9"	4	S2 6S	Outside Stringers Span Nos 11+13	
	6"x15½"x18'-3"	18	S2 6S	Inside Stringers Span Nos 11+13	
	6"x15½"x20'-0"	11	S2 6S	Stringers Span No 14	Cut to fit against backwall
	1⅝"x3⅝"x23'-4"	1900	S4S	Strip Floor	
	2"x4"x16'-0"	75	—	Bridging	Makes all
	1"x6" Strips	520 LF	S4S	Strips under Scupper Blocks	
				Stringer Encasement Span No 12	
	1½"x8"x16'-0"	66	—	Side Sheathing	Matched saw to uniform thickness
	1½"x8"x18'-0"	66	—	Side Sheathing	Matched saw to uniform thickness
	3⅝"x8½"x16'-0"	22	S4S	Spiking Pieces	
	2"x8½"x15'-0"	22	S2 2S	Bottom Cover	Saw to uniform thickness
	1½"x8½"x20'-0"	2	S2 2S	End Cover	Makes all
A1	3⅝"x4½"x14'-0"	2	S4S	Stringer Bearing Blocks	Makes all
B1	4"x4½"x12'-0"	10	S2 4S	Spacer Blocks	Makes all
C1	3½"x4"x14'-0"	14	S2 4S	Spacer Blocks	Makes all

Cap rails shall be surfaced to the finished size shown in Rail Sections on Sheet No.4 and all timber shall be sawed to the length given in the schedule and dressed if necessary before treatment.

All scheduled timber unless otherwise noted shall be long leaf Southern Yellow Pine of the grade specified.

Parapets for Abutment No.1, Caps and Wearing Strip Extensions for Abutment No.2, and all timber listed under Floor, Stringer Encasement Span No.12, Rail and Curbs and Miscellaneous shall be treated with ZMA salt. All other Southern Pine timber shall be creosoted.

1" Asphalt Plank

Reqd 5575 Sq.Ft

Pile Schedule

Location	No.	Length	Revised Length
Bent No1*	4	35'-0"	30'-0"
2*	4	35'-0"	30'-0"
3*	4	35'-0"	35'-0"
4*	4	40'-0"	35'-0"
5*	4	40'-0"	38'-0"
6*	4	40'-0"	38'-0"
7*	4	40'-0"	38'-0"
8*	4	45'-0"	38'-0"
9*	4	40'-0	40'-0"
10*	4	40'-0	45'-0"
11*	4	35'-0"	45'-0
12*	4	35'-0"	45'-0"
13*	4	30'-0"	45'-0"
Abut No.2*	18	28'-0"	35'-0"
Anchor Piling Abut No2	4	25'-0"	25'-0"
Anchor Piles Abut No2	4	22'-0	22'-0"
Anchor logs Abut No2	3	25'-0"	25'-0"
-	1	29'-0"	29'-0"

All piles, anchor piles and anchor logs to be long leaf Southern Yellow Pine pressure treated with creosote oil.

* Large end to be hewn to 10" square for a length of 20'-0" before treating.

Pile lengths given in schedule are for estimate only. Actual lengths shall be determined by driving test piles.

[redacted] Masts

Masts to be [redacted] Southern Pine or Western Fir. Tip diameter to be 16." [redacted] Masts to be straight. Bark to be entirely removed.

No materials necessary for installing water pipe, water pipe hangers or the including floor are scheduled on this sheet.

Drift Bolts

Mark	Size	No.
D1	¾"x1'-6"	73
D2	¾"x1'-9"	300
D3	¾"x1'-3"	13
D4	¾"x1'-0"	22

Town 1628
Bridge 3096

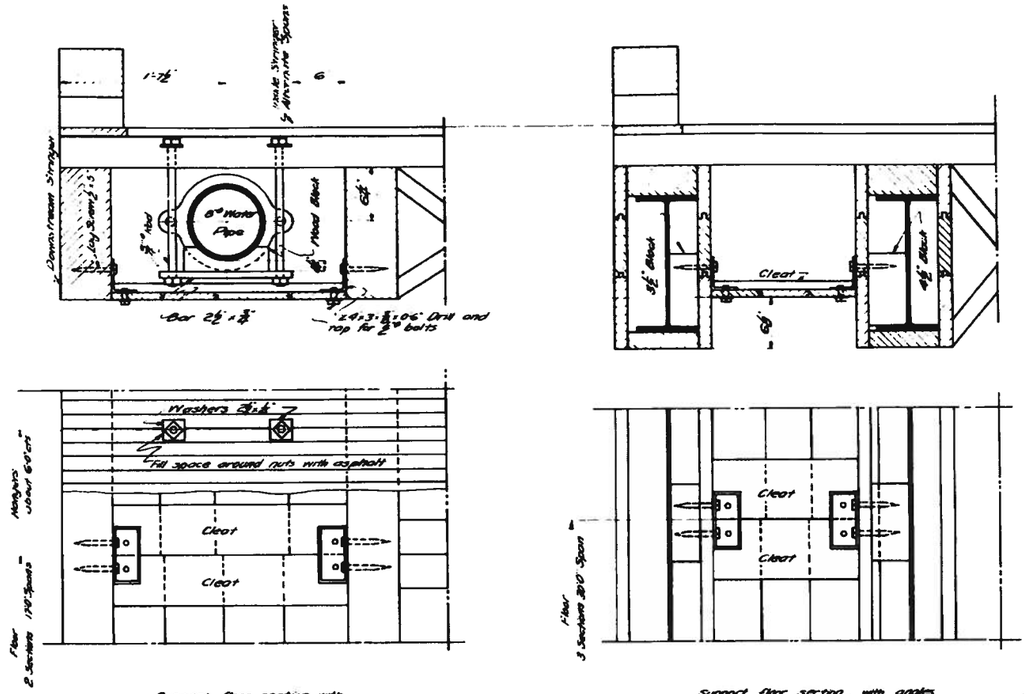
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION

SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
APPROACH RAILS-MATERIALS

SHEET 7 OF 7 AUGUSTA, ME. JAN. 1934

Revised Water Pipe C.L.P. Nov. 9 1934

17-158



Support floor section with angles at center and at ends. Space cleats at ends and approximately at third points. Cut and cleats to clear supporting angles. Hangers and cleats must not be located at the same points.

17'0" Spans

Support floor section with angles at ends and approximately at third points. Use five cleats per section. Hangers and cleats must not be located at the same points.
Details not shown are same as for 17'0" spans.

30'-0" Span

Water Pipe Hanger and Floor

All labor and materials necessary for installing water pipe, water pipe hangers and the enclosing floor will be supplied by the water company.

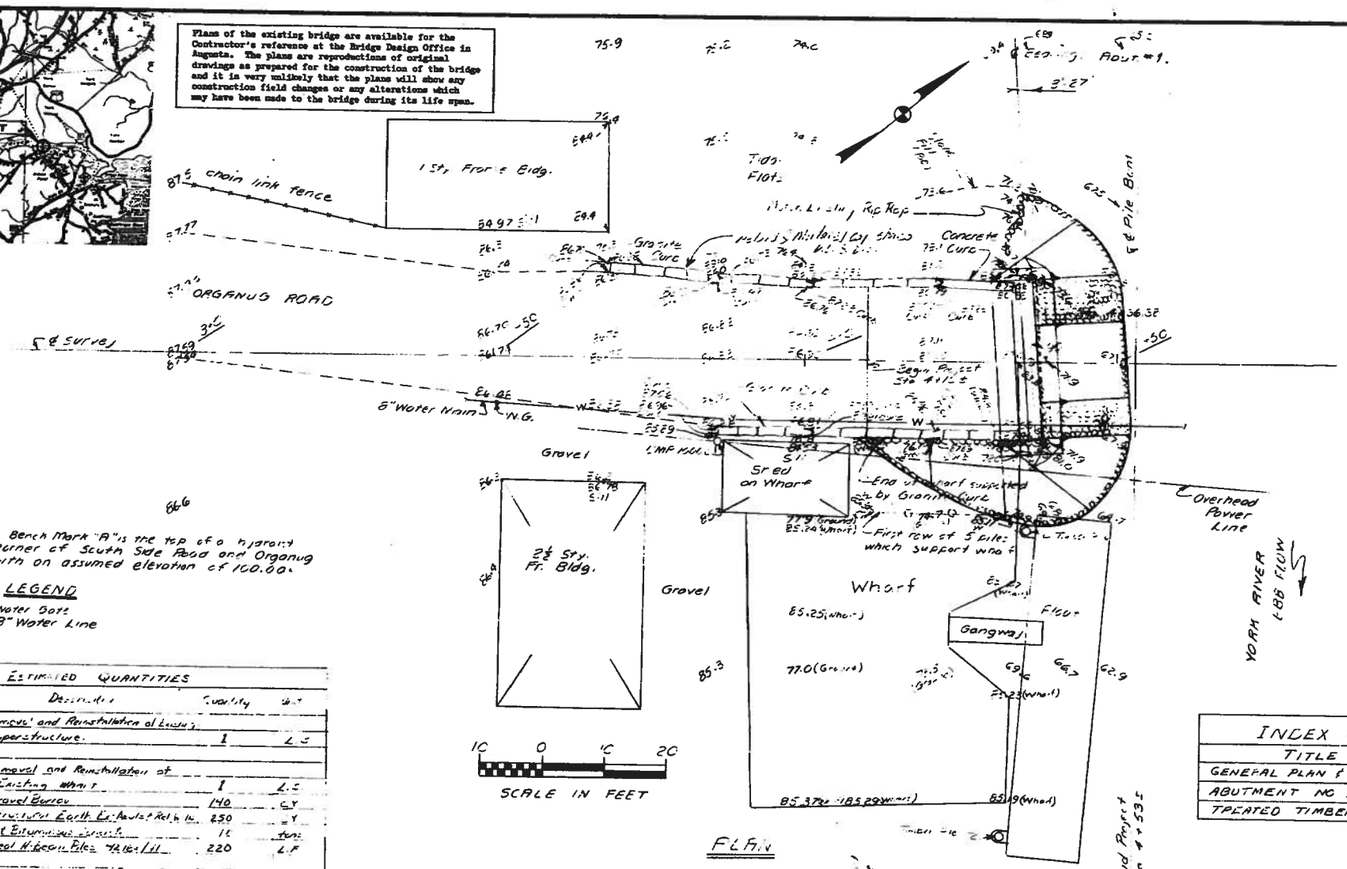
Town 16-28
Bridge 3036

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
SEWALL'S BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY
WATER PIPE SUPPORT
SHEET 7a of 7 AUGUSTA, ME., MAR 1934

17-159

F.H.W.A. REGN. NO.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	3096	1	3

Plans of the existing bridge are available for the Contractor's reference at the Bridge Design Office in Augusta. The plans are reproductions of original drawings as prepared for the construction of the bridge and it is very unlikely that the plans will show any construction field changes or any alterations which may have been made to the bridge during its life span.



NOTE: Bench Mark "A" is the top of a hydrant at the corner of South Side Road and Organus Road with an assumed elevation of 100.00'.

LEGEND

N.G. = Water Gate
—W— = 8" Water Line

ESTIMATED QUANTITIES

Item No	Description	Quantity	Unit
202.094	Removal and Reinstallation of Existing Superstructure.	1	L.S.
202.092	Removal and Reinstallation of Existing Wharf	1	L.S.
203.26	Gravel Borrow	140	C.Y.
206.06	Structural Earth Excavation Rel. to Ex.	250	C.Y.
460.62	Hot Bituminous Concrete	12	Ton
501.212	Steel H-Section Piles HP 14x117	220	L.F.
502.21	Structural Concrete, Abut. #1 Rel. to	52	C.Y.
502.22	Structural Concrete Abut. #2 Rel. to (Percent Under Water)	75	C.Y.
503.12	Reinforcing Steel, Fab. & Delivered	2535	Lb.
503.13	Reinforcing Steel, Placing	2535	Lb.
511.07	Cofferdams	1	L.S.
512.07	French Drains (Stones Only)	4	C.Y.
514.06	Curing Box for Concrete Cylinders	1	Ea.
515.20	Protective Coating for Conc. Sur.	20	S.Y.
610.08	Plain Riprap	100	C.Y.
629.05	Labor Straight Time	40	M.H.
631.10	Air Compressor (inc. operator)	5	Hr.
631.11	Air Tool (inc. operator)	5	Hr.
631.12	All Purpose Excavator (inc. operator)	20	Hr.
631.171	Truck (inc. operator)	15	Hr.
636.22	Front End Loader	5	Hr.
642.04	Warning Light	2	Stop
659.10	Field Office Type C	1	Ea.
653.3	Type 1 Barricade	4	Ea.
652.35	Construction Signs	30	S.F.
652.36	Maintenance of Traffic	70	L.S.
629.02	Mobilization	1	L.S.

GENERAL NOTES

1) Unless otherwise noted all utilities shall be adjusted by the respective utilities owners.

2) The roadway will be closed during construction for approved special provisions.

3) The roadway shall be transitioned from the new structure to match the existing roadway at sta. [illegible].

4) The new structure shall be opened to traffic upon completion of the structure. The posts shall be set so as not to damage the sidewalk curbs. Removal, storage and reinstallation of the guardrail will be considered incidental to Item 202.092.

5) The amount of stone riprap will be based on the estimated quantities. The rip rap shall be placed as necessary. The walls and curbs shall be constructed between the underwater portion and non-concrete work using the forms and other means from the equipment approved by the Engineer. The reconstruction of the stone [illegible] will end curbs with the established grade [illegible] paid for under the appropriate rear and equipment rate. Pay items.

Note: Grouted joints between all [illegible] shall be EWC #1 Mix. M.

6) All back fill and roadway base shall be gravel borrow.

7) No direct payment will be made for removing and disposing of excavation outside the limits of structure. Excavation, fill, back excavation will be considered incidental to Item 206.06 - Structural Earth Excavation, Abutments and Retaining Walls.

INDEX OF SHEETS	
TITLE	No
GENERAL PLAN & SURVEY	1
ABUTMENT NO 1	2
TREATED TIMBER JACKET	3

SPECIFICATIONS

Design: AASHTO Specifications for Highway Bridges 1977 with Interims 1978 & 1979.

Contract: State of Maine, State Highway Commission Standard Specifications, Highway and Bridges, Revision of June 1968.

Materials

Concrete: Seal, Class S
All Other; Class A

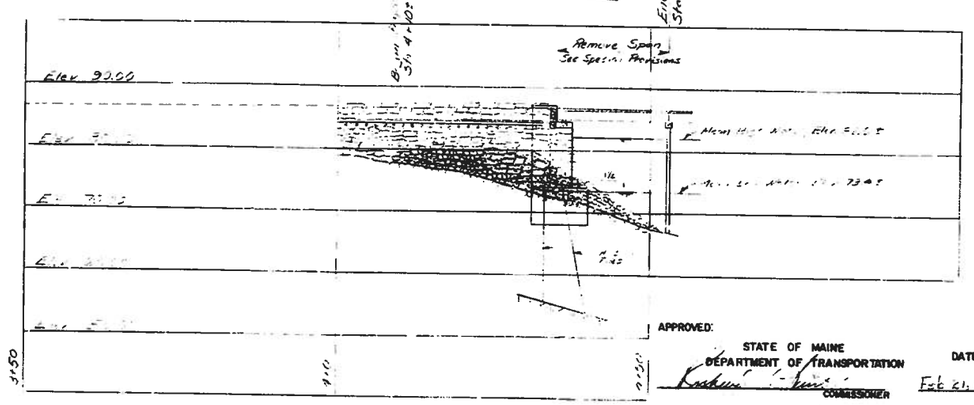
Reinforcing Steel: ASTM A615, Grade 60

Basic Allowable Stresses

Concrete: f_c = 1600 psi
Reinforcing Steel: f_s = 24,000 psi

Revised Date 3/23/81

BRIDGE NUMBER 3096



ELEVATION

Traffic Data

AADT (1982) = 460
AADT (2000) = 1440

APPROVED:

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

COMMISSIONER — DATE Feb 21, 1980

BUREAU DIRECTOR AND CHIEF ENGINEER — DATE Feb 21, 1980

DHV

FIELD BOOK 5606

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

SEWALLS BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY

GENERAL PLAN & SURVEY

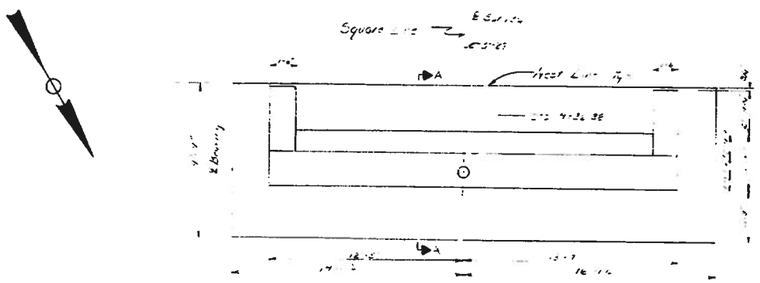
SHEET 1 OF 3 AUGUSTA, MAINE

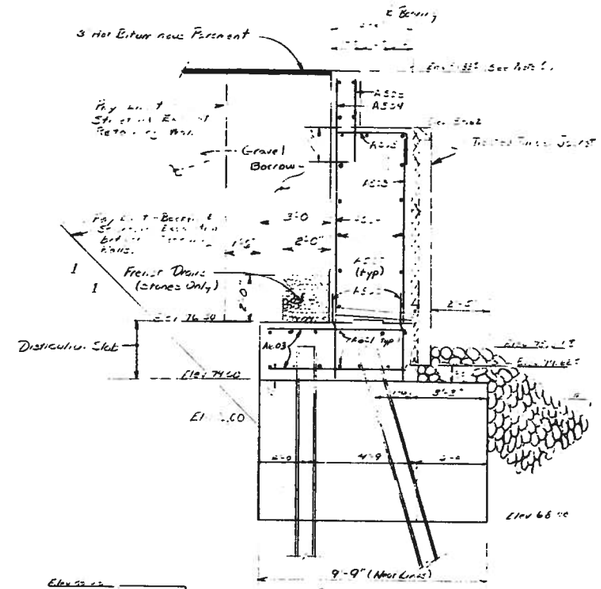
172-70

PROJECT BRIDGE ENGINEER | BY | DATE
DESIGN - DETAILED | JET | TJN
CHECKED | JET | JMO
REVISIONS
FIELD CHANGES
PLANS

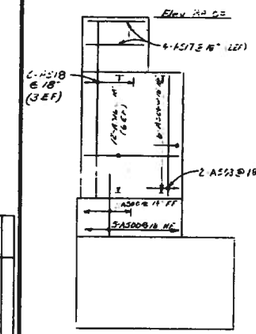
Survey by: Haggin & Crew Q Sep 1977
Survey Plot by: Y. Thomas Oct. Nov 1979
Survey Plot by: J.E.T. Nov 1979

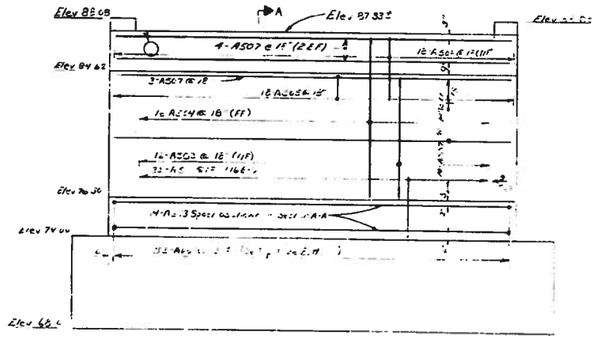
F.A.H.S. PROJ. NO.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	3096	2	3

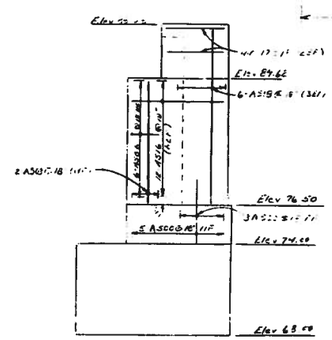


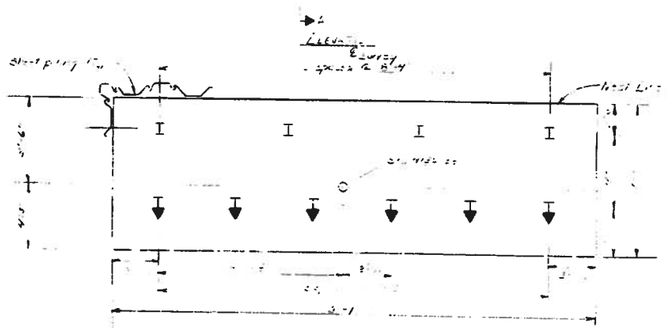


Instrument sleeve in backwall supplied by the utility. Exact location shall be determined in the field.









Section E-E

Notes

1) Piles shown for design to ledge or practical refusal.

2) All Tips shall use Pointed Reinforced Pile Tips as shown on Standard Detail BD104-77

3) Alternate types of Pointed Reinforced Pile Tips may be used if they have at the cross-sectional area of the Pointed Reinforced Pile Tip shown on the plans and is approved by the Engineer.

4) Estimated driven lengths of piles are determined from available soil information with no allowance for other pile penetration.

5) Embedment of piles in dense soil stratum varies between 1'-0" and 3'-0" and the total embedment length up to a maximum of 1'-6" will be included in the measurement for payment.

6) Pile marked thus H, shall be battered 3½" inches per foot in the direction of the arrow.

7) Maximum pile loads 54 tons

8) Following are pile locations, number of piles required, size of piles and estimated driven lengths:
Abutment Number 1 10 - 11" x 12" x 74 @ 22 feet

Abutment Notes (Cont.)

6) The existing 12"x12" timbers used as a bearing seat shall be retained as a bearing seat on the new abutment. The exact elevation of the seat will shall be determined in the field based on the height of the timber bearing seat, stringers, and decking

7) All reinforcing splices shall be 1'-6" minimum unless otherwise noted.

8) Protective Coating for Concrete Surfaces shall be applied to all exposed concrete areas.

PROJECT DESIGN ENGINEER	JEF	BY	DATE
PLANS — DESIGN - DETAILED		JEF	2/80
CHECKED		JCN	2/80
REVISIONS		JEF	2/80
FIELD CHANGES			

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

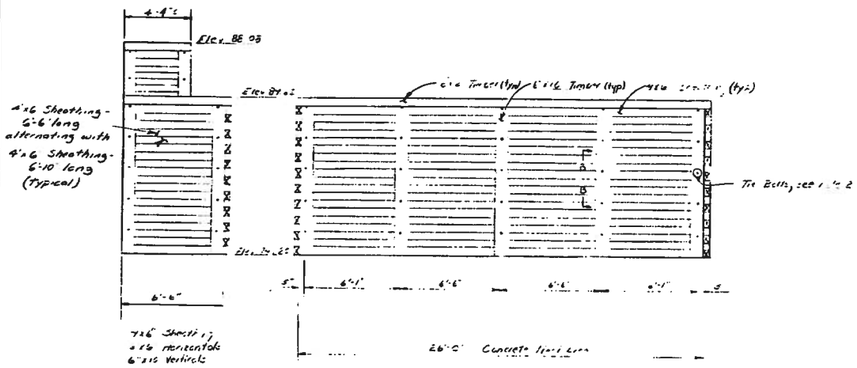
SEWALLS BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY

ABUTMENT NO. 1

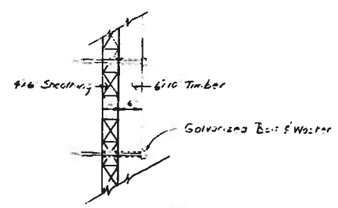
SHEET 2 OF 3 AUGUSTA, MAINE F.A.L. 1977

172-77

F.A.P. NO.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	3096	3	3



ELEVATION



Section "B-B"

Typical Sheathing Layout in Breastwall

TIMBER SCHEDULE

[illegible]	14	[illegible]	Wingwall
[illegible]	7	9'-0"	Breastwall Wing
[illegible]	7	3'-6"	Wingwall
[illegible]	2	12'-4"	Breastwall
[illegible]	2	6'-[illegible]	Wingwall
[illegible]	2	4'-6"	[illegible]
[illegible]	30	[illegible]	[illegible]
4"x6"	23	6'-6"	Wingwall
4"x6"	40	6'-10"	Breastwall & Wing
4"x6"	16	4'-6"	Wingwall

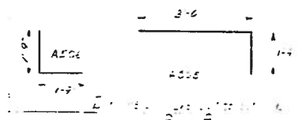
TIMBER NOTES

1) All timber shall be structural grade Southern Yellow Pine No 2, unplaned. See Special Provisions Section 502 - Treated Timber Jacket.

2) Locations of holes in beams for tie bolts shall be determined by the Contractor and shall be bored before treatment.

3) Holes for tie bolts in 4" planking shall be bored in the field as required.

REINFORCING STEEL SCHEDULE



[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	4'-9"	Bridge Seat
A506	5	16	3'-6"	Intersection of Wing & Breastwall
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A600	3	48	4'-3"	Distribution Slab
[illegible]	3	[illegible]	7'-10"	Breast Wall
A504	3	16	12'-5"	Breast Wall & Back wall
A501	3	17	25'-6"	Breast Wall & Back wall
A506	3	18	3'-9"	Back Wall
A510	5	24	6'-0"	Wings
A517	5	8	3'-10"	Wings
A518	5	12	11'-3"	Wings
A601	6	52	6'-0"	Distribution Slab
A603	6	14	25'-6"	Distribution Slab

Note: Reinforcing Steel shall conform to ASTM Designation A615, Grade 60

PROJECT DESIGN ENGINEER	JCF	BY	DATE
DESIGN - DETAILED	JCF	[illegible]	[illegible]
CHECKED	JCF		[illegible]
REVISIONS			
FIELD CHANGES			

PLANS

Revised as built (Nov. 21, 1961)

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

SEWALLS BRIDGE
OVER
YORK RIVER
IN THE TOWN OF
YORK
YORK COUNTY

TREATED TIMBER JACKET

SHEET 3 OF 3 AUGUSTA, MAINE Feb 1950

172-72

Transportation
Land Development
Environmental
Services

Vanasse Hangen Brustlin, Inc.

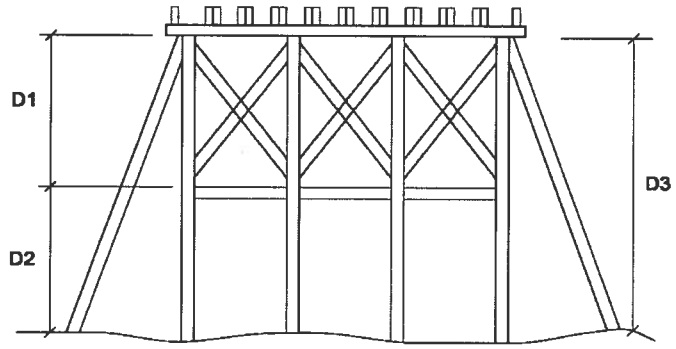
Computations

Project:	Sewalls Bridge	Project #:	51987.00
Location:	York, ME	Sheet:	
Calculated by:	LSG	Date:	12/18/09
Reviewed by:	KDW	Date:	12/22/09
Title:	Dimensions to Mudline		

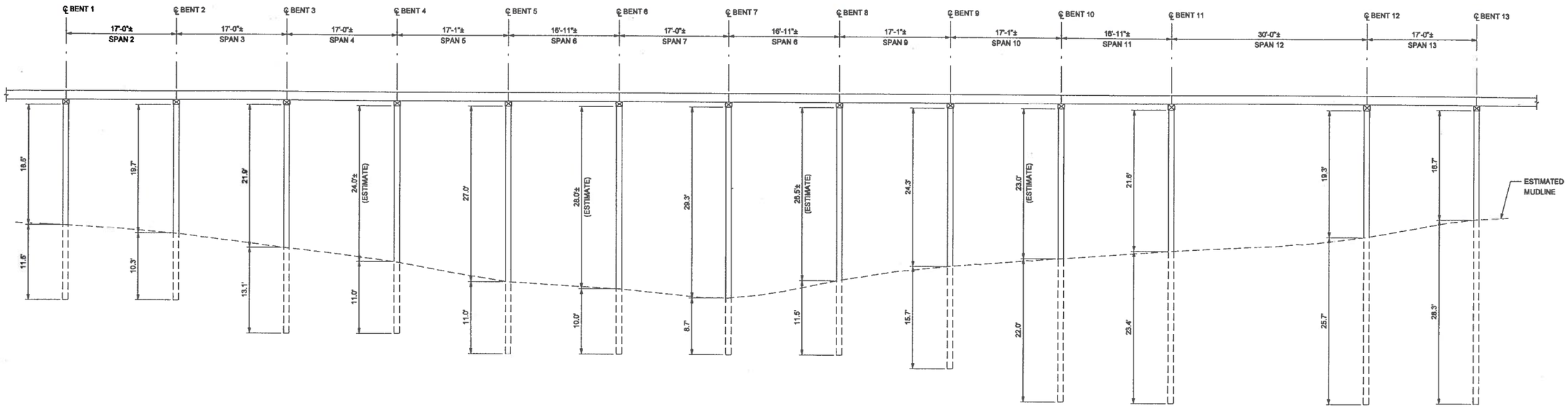
Dimensions to Mudline Reference Inspection Notes week of 10/5/09

Bent No.	Top of Pile to Top of Strut (D_1), (ft)	Top of Strut to Mudline (D_2), (ft)	Top of Pile to Mudline (D_3), (ft)	Pile Length from Existing Plans (ft)
1	10.5	8	18.5	30
2	10.7	9	19.7	30
3	10.9	11	21.9	35
4	11.0	-	-	35
5	11.0	16	27.0	38
6	11.3	-	-	38
7	11.3	18	29.3	38
8	11.2	-	-	38
9	11.3	13	24.3	40
10	11.4	-	-	45
11	10.6	11	21.6	45
12	10.3	9	19.3	45
13	10.7	6	16.7	45

Note: Where dimensions from top of strut to mudline were not taken, mudline will be extrapolated from adjacent bents



VHB PROJECT # 51987.00
DATE: 1/4/10



PARTIAL ELEVATION
(LOOKING UPSTREAM)
SCALE: 1/8" = 1'-0"

DRAFT

NOTES:

- PILE LENGTHS TAKEN FROM EXISTING PLANS (1934)
- DIMENSIONS TO MUDLINE WERE FIELD MEASURED 10/09 (BENTS 4, 6, 8, 10, ARE INTERPOLATED)

VHB

FIGURE 1

SEWALL'S BRIDGE
OVER
YORK RIVER
IN
YORK, MAINE