Hudson River Park Trust

PARKWIDE MARINE INSPECTION, ENGINEERING,
DESIGN, CONSTRUCTION & ADMINISTRATION SERVICES
CONTRACT No. A3056 - PHASE 47

CONDITION MONITORING INSPECTION

PIER 40

MARCH 2015

HUDSON RIVER PARK TRUST

PARKWIDE MARINE INSPECTION, ENGINEERING, DESIGN, CONSTRUCTION & ADMINISTRATION SERVICES
CONTRACT NO. A3056 – PHASE 47

CONDITION MONITORING INSPECTION

PIER 40

MARCH 2015

Prepared by:

Halcrow

22 Cortlandt Street, 31st Floor

New York, NY 10007

NYCEDC
New York City Economic Development Corp.

March 4th, 2015

Madelyn Wils
President
Hudson River Park Trust
Pier 40, 2nd Floor
353 West Street
New York, NY 10014

Reference: **Pier 40 – Halcrow Rehabilitation Cost Estimate Review**

Dear Ms. Wils:

The New York City Economic Development Corporation (NYCEDC) is in agreement with Halcrow's February 26th, 2015 estimate.

At the request of the Hudson River Park Trust (HRPT), NYCEDC reviewed the Pier 40 Halcrow inspection report dated December 2014, as well as the corresponding Excel cost estimate back up, dated September 18th, 2014. During the initial review NYCEDC decided to bring in one of our waterfront construction management teams to assist with the review. Following our team's review of both documents we compiled a list of comments and concerns, which were discussed between all parties involved on January 28th, 2015. Following our meeting, HRPT directed Halcrow to make several revisions to their estimate to reflect the comments and discussions.

Additionally, both the inspection methodology and the rehabilitation details used by Halcrow in their report appear to be consistent with those used by NYCEDC for similar inspections and types of work.

If you have any questions regarding the above information, please feel free to contact me directly.

Regards,

Richard A. Cote
Executive Vice President, Asset Management
New York City Economic Development Corporation
Tel: (212) 312-4269
rcote@nycedc.com

EXECUTIVE SUMMARY

From December 2013 to February 2014, Halcrow performed a condition monitoring inspection of Pier 40 on the eastern shore of the Hudson River in Manhattan, New York. The scope of work included an above water and underwater inspection of the steel H-piles, concrete piles caps, pile cap beams, deck soffit, cathodic protection, and fender system, The purpose of the inspection was to provide a general condition assessment and load rating of the pier in its current condition, to evaluate and grade the condition of structural elements of the pier, and to develop repair recommendations with an associated order-of-magnitude cost estimate.

The methodology used to assign both the general condition assessment rating and the structural element grading is described in Section 1.2 of this report. Based on this methodology and the inspection results, Pier 40 is in overall **Poor** condition. The allowable uniform live load rating for the pier is significantly less than its original design live load capacity.

The repair recommendations and cost estimate contained in this report have been developed to maintain a 100 psf live load rating sufficient for public assembly use over the entire pier structure. The repair recommendations are also intended to provide enough lateral load capacity to resist ice, wind, wave, current, and mooring loads typical to this riverfront location, and to maintain sufficient structural capacity for fire truck access to the Court Yard (currently used as an athletic field) and the perimeter Pier Shed. The repair recommendations are not intended to bring the pier back to the original design live load capacity when the pier was first constructed.

As depicted in Table 1 below, the steel H-piles supporting the pier are in overall **Poor** condition with 35% graded Severe and 22% graded Major. Deterioration of the H-piles is typically due to corrosion within the splash zone and at Mean Low Water (MLW) for those piles with no prior channel repairs. In general, the condition of the steel H-piles at and below MLW, with the exception of H-piles without prior channel repairs, has little bearing on the overall pile condition rating. This is because they exhibit only minor to moderate deterioration in those areas, and appear to be adequately protected by the sacrificial anodes of the existing cathodic protection system. A full visual inspection of the underlying steel of the H-piles was not possible in every instance as some of the piles were previously repaired with an epoxy coating, and other H-piles above the high water line were previously repaired with both welded steel plates and epoxy coating. For those H-piles that received a more intensive Level II/III inspection, the epoxy coating was removed in small sections to reveal the underlying steel. For those inspected H-piles with no prior welded steel plate repair showing rust staining through the epoxy coating, the typical condition revealed was a Severe pile section. Therefore, all H-piles that exhibited rust staining through an epoxy coating application, but which did not have a prior welded steel plate repair, were graded Severe. H-piles with visible rust staining through epoxy coating which had prior welded steel plate repair were graded Major.

A summary of the H-pile ratings are provided in Table 1.

Table 1 Summary of Pile Conditions

Location	No. of H-piles	Minor		Moderate		Major		Severe	
		No.	%	No.	%	No.	%	No.	%
Pier Shed	2,845	505	18%	669	24%	698	25%	973	34%
Court Yard	483	39	8%	255	53%	50	10%	139	29%
Finger Pier	135	8	6%	17	13%	25	19%	85	63%
Total	**3,463**	**552**	**16%**	**941**	**27%**	**773**	**22%**	**1,197**	**35%**

The concrete pile caps under the Pier Shed and Court Yard are generally in **Fair** condition with corrosion cracks on the cap soffits that extend from the flange tips of the steel H-piles to the bottom corners of the pile caps. At a number of locations, these corrosion cracks have either extended along the vertical faces of the concrete pile caps or have resulted in spalls along the bottom corners of the caps. In the Court Yard, the pile caps typically have hairline map cracks on the vertical faces with efflorescence.

The concrete pile cap beams under the Pier Shed and Court Yard are in **Fair** condition with typical rust staining and opposing longitudinal corrosion cracks that have resulted in delaminations along the beam soffits. In isolated locations, the delaminations along the beam soffits have developed into spalls with exposed steel reinforcement.

The longitudinal concrete and transverse concrete beams at the Finger Pier Extension are in overall **Fair** condition. Similar to the beams under the Pier Shed and Court Yard, the beams exhibit rust staining and delaminations in the beam soffits with isolated spalls with exposed steel reinforcing.

The concrete underdeck at Pier 40 is in overall **Fair** condition with minor hairline cracks. There are isolated spalls up to 3 in. deep with exposed prestressing strands and reinforcing steel throughout the pier.

The Finger Pier Extension concrete underdeck is in overall **Fair** condition with areas of shallow concrete cover and spalls with exposed steel reinforcing.

The current vertical live load capacity of the pier is summarized in Table 2

Table 2 Summary of Load Ratings for Pier 40

Structure	Governing Structural Element	Current Allowable Uniform Live Load	Fire Truck Access
Pier Shed	Steel H-piles rated Severe	100 psf (with a 2% overstress)	No Restrictions
Court Yard	Concrete pile cap beam with exposed steel reinforcing and Severe piles	150 psf	No Restrictions
Finger Pier Extension	Deteriorated beams and deck	0 psf (with the possibility of ice loading)	Not Applicable
		100 psf (with no possibility of ice loading)	

The Pier Shed is capable of supporting a uniform live load of 100 psf in its existing condition, and it is also capable of supporting a fire truck with a 24 kip wheel load and a Rescue Truck with a 26 kip wheel load. The results of the analysis indicate that the design ice load, which is based on an 8 inch thick layer of ice, is very demanding on the pier structure and is the controlling load within Load Combination 8 and Load Combination 9 (see Table 4-1).

While the steel H-piles under the Pier Shed with a pile grade of Severe are currently sufficient to support the current pier usage, the level of corrosion on the piles has reached a critical level. Any notable amount of additional section loss, especially at the tops of the piles, could result in public access restrictions on the pier because the resulting allowable live load at the Pier Shed would likely be lower than the 100 psf needed for public assembly.

The allowable uniform live load rating for the Court Yard is currently 150 psf, and is governed by the structural capacity of the concrete pile cap beams with exposed reinforcing. Although the extent of deterioration on the concrete beams varies throughout the entire Court Yard, the structure was conservatively rated based on the lowest load rating determined for the analyzed structural elements, which is 150 psf.

At the Finger Pier Extension, the steel H-piles are capable of supporting a uniform live load of 0 psf in their current condition based upon the loading combinations outlined in the HRPT Design Guide. This rating is governed by the Ice Load outlined in Load Combination 8 and Load Combination 9 (see Table 4-1). Since the controlling load is an ice load, it is recommended that access to the Finger Pier Extension be restricted whenever there is the possibility of ice loading. When there is no possibility of ice load on the Finger Pier the allowable live load remains at 100 psf.

Recommendations made in this report are grouped into the following three levels of importance. The definition of each level of importance is taken from the New York City Economic Development Corporation's (NYCEDC) Waterfront Facilities Maintenance Management System Inspection Guidelines Manual. **"Immediate"** level actions are recommended to be completed as soon as possible to prevent unsafe conditions. **"Priority"** level actions are intended to maintain the structure in a safe operating condition and/or prevent deterioration from continuing to a point where the future repairs will be significantly more costly. **"Routine"** level actions are intended to be undertaken as part of a scheduled maintenance program. They should be undertaken in accordance with good engineering and industry practice to maintain the structure and reduce future capital expenses.

A cost effective repair plan that addresses all areas of deterioration without the need for future phased repair efforts (aside from routine inspections and regular maintenance) was developed. Because of the size of the pier and complexity of the work, a single continuous design and repair effort extending over an approximately nine and one-half year period likely represents the most cost effective approach to maintaining current usage.

The order-of-magnitude cost estimate for the recommended repair is summarized in Table 3. It is assumed that a detailed design and engineering effort will take approximately 2 years and precede the physical repair work. Because of the condition of the H-piles, we recommend that this effort commence as soon as practicable.

Recommendations and assumptions regarding the timing and cost of repairs consider the extent of the repairs based on inspected conditions, the size of Pier 40 and difficulty in accessing the middle portion of the pier, environmental constraints, and lack of head room under the pier, particularly under the Court Yard, which forces more of the work to be performed underwater. Repairs specified at the interior/middle piles will therefore be significantly more labor intensive and costly than repairs specified along the pier perimeter. Environmental obstacles include pouring encasements in the winter months, which is not possible/recommended. Finally, large areas of the pier, especially under the Court Yard, have little to no headroom during some or all of the tide cycle. All of these factors contribute to the difficulty associated with a widespread and comprehensive repair effort, and, while cost effective, the recommended repair is nevertheless more costly than what would ordinarily be the case for a more typical pier structure.

Table 1 Cost Estimate for the Pier 40 Repair

<table>
<tr><th rowspan="2">Repair Importance Level</th><th colspan="2">Number of Piles to Repair</th><th rowspan="2">Timber Fender System Replacement (LF)</th><th rowspan="2">Area of Beam and Deck to Repair (SF)</th><th colspan="2">Cost Estimate</th></tr>
<tr><th>Structural Enc.</th><th>Non-Structural Enc.</th><th>Sub-Total</th><th>TOTAL</th></tr>
<tr><td>Immediate</td><td>110</td><td>0</td><td>0</td><td>0</td><td>$3.96M</td><td rowspan="3">$80.6M</td></tr>
<tr><td>Priority</td><td>1,671</td><td>1,124</td><td>0</td><td>0</td><td>$61.1M</td></tr>
<tr><td>Routine</td><td>168</td><td>390</td><td>2,405</td><td>1,730</td><td>$15.46M</td></tr>
<tr><td colspan="7">Escalation to Mid-Point of Construction – 7.5 Year Construction Period</td></tr>
<tr><td colspan="3">Escalation to 2016 (2 Year Design Period)</td><td>6.09%</td><td colspan="2">$4.9M</td><td rowspan="2">$14.9M</td></tr>
<tr><td colspan="3">Escalation to 2020 (Mid-Point of Construction)</td><td>11.72%</td><td colspan="2">$10M</td></tr>
<tr><td colspan="3">Total Project Construction Cost With Escalation</td><td colspan="4">$95.5M</td></tr>
</table>

Note: Owner's costs, not included in the base pricing, totals $9.1M. For detailed breakdown of costs please refer to Appendix C.

The recommended repair is a comprehensive program that encases every pile in a continuous repair program with an assumed nine and one-half year design and construction period. Fifty-six (56%) percent of the piles are recommended for structural encasement and forty-four (44%) are recommended for non-structural encasement. The resulting repaired structure should be relatively low maintenance and sufficient to support the current use requirements for the foreseeable future. In addition, any unknown deficiencies that exist now will likely be discovered and remedied during such a widespread rehabilitation effort. The $95.5 million cost estimate includes both expected contractor costs, such as insurance, overhead and profit, industry standard contingencies for design and construction, and annual escalations of 3%.

Not included in the cost estimate are typical owner costs estimated at approximately $9.1 million which would be expended during both the design and engineering phase and during construction. Owner costs include items such as design services, construction administration, diving and controlled inspections. The order-of-magnitude total for both contractor and owner costs is therefore estimated at approximately $104.6 million.

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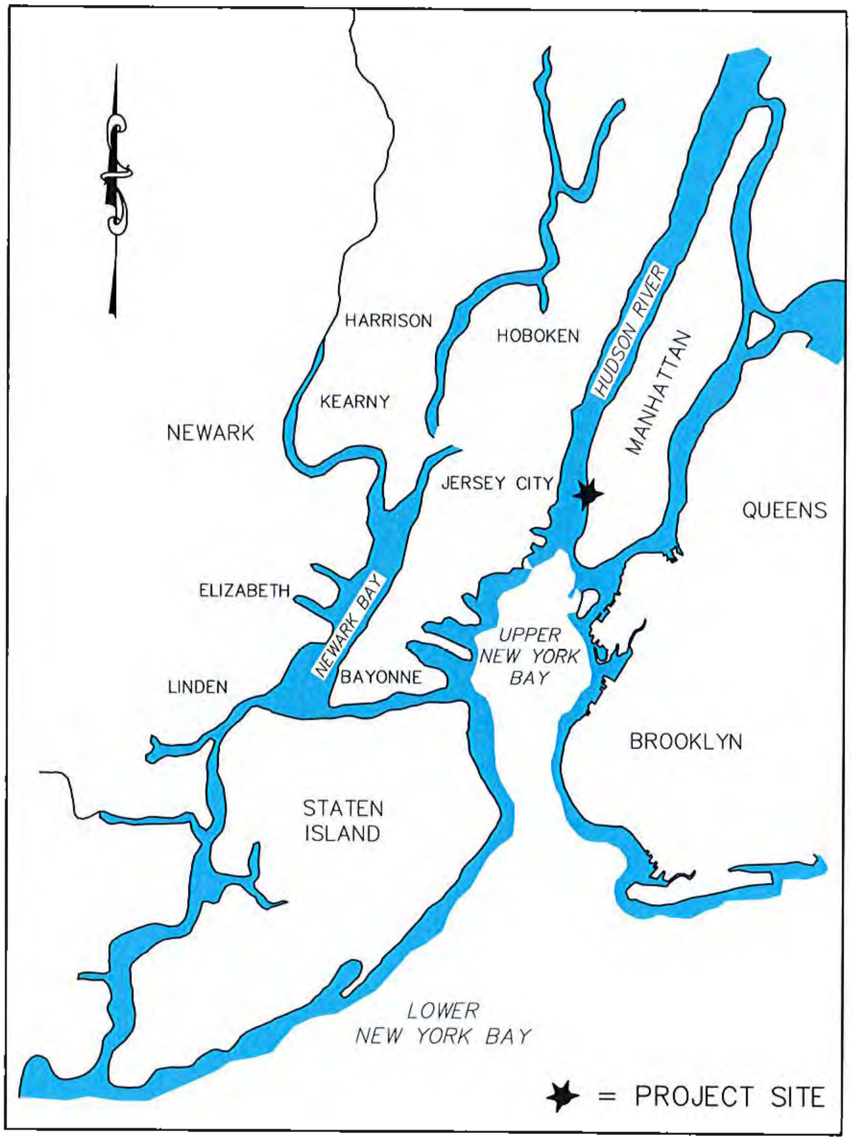
1. INTRODUCTION

From December 2013 to February 2014, at the request of the Hudson River Park Trust (Trust or HRPT), Halcrow performed a condition monitoring inspection of Pier 40 which is located at the terminus of West Houston Street, along the western shore of Manhattan in New York. A vicinity map and location plan are presented on Figures 1-1 and 1-2, respectively.

The scope of work included an above water and underwater inspection of the facility's foundation structural and non-structural components, which includes steel H-piles, concrete pile caps, pile cap beams, deck soffit, the top surface of the pier apron, cathodic protection, and fender system. The purpose of the inspection was to provide a general condition assessment and load rating of the pier in its current condition, and to develop repair recommendations with an associated order-of-magnitude cost estimate. The repair recommendations and cost estimate were developed based upon HRPT's request for a repair plan to address all areas of deterioration without the need for future phased maintenance or repair efforts. Immediate and Priority repair recommendations were developed to maintain a 100 psf live load rating over the entire pier structure and sufficient lateral load capacity to resist, ice, wind, wave, current, and mooring loads typical to the geographical area. Considerations for fire truck loading and access to all areas within the Court Yard and the perimeter Pier Shed were also included in the repair recommendations.

A history of all previous inspections is provided in Table 1-1.



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VICINITY MAP

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FIG 1–1

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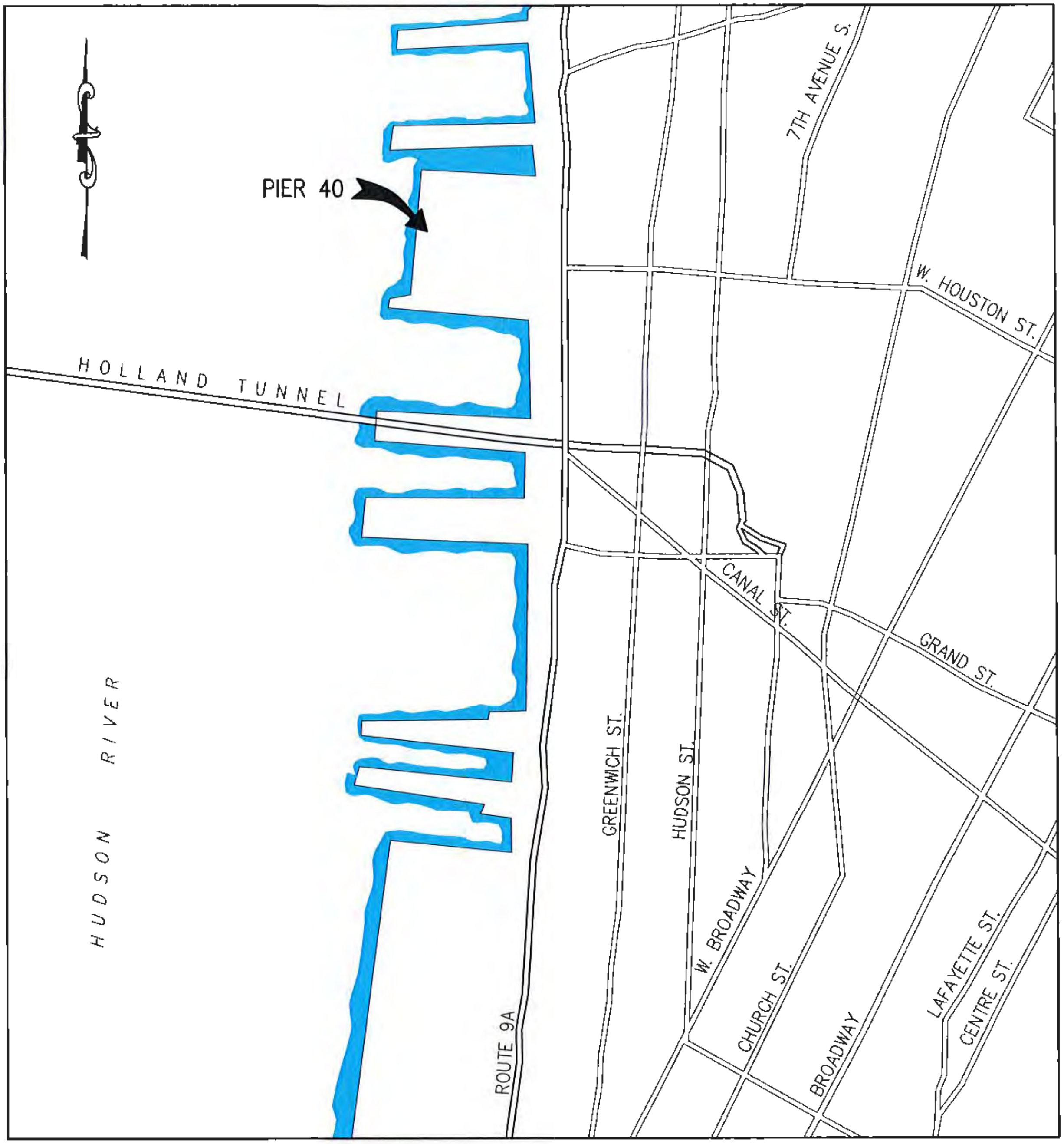

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LOCATION PLAN

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FIG 1–2

Table 1-1 Previous Inspections of Pier 40

Report Date	Inspection Date	Inspection Firm	Client
2009 Jun.	Feb. to Apr. 2009	HPA (Halcrow)	HRPT
2004 Aug.	2004 May	HPA (Halcrow)	HRPT
	2002 Dec. to Jan. 2003	HPA (Halcrow)	HRPT
2001 Jun.	1999 Apr. to 2001 Jan.	Goodkind and O'Dea	NYSDOT
1998 Dec.	1998 Apr. to Jul.	Goodkind and O'Dea	NYSDOT
1996 Aug.	1999 May to Jun.	Goodkind and O'Dea	NYSDOT
1994 Dec.	1994 Sept.	Han-Padron Associates (Halcrow)	Port Authority
1993 Aug.	1992 Sept. to Nov.	Han-Padron Associates (Halcrow)	Port Authority
1988 March	1987	Parsons Brinckerhoff, Quade and Douglas Inc.	Port Authority
1985 March	1984	Parsons Brinckerhoff, Quade and Douglas Inc.	Port Authority

1.1 Inspection and Methodology

The inspection was conducted by a three-person team lead by an engineer-diver. The underwater inspection of Pier 40 was performed using surface-supplied diving equipment staged from a van parked on the perimeter apron. The diving equipment included a Superlite 17B diving helmet, a three-part umbilical with continuous hardwire communications, and other commercial diving gear. Above water, the inspection was performed using a 12 ft aluminum boat outfitted with an outboard motor. Various hand tools, photographic equipment, and measuring devices were utilized to assess the condition of the structural components.

The underwater inspection included a 100% visual (Level I) inspection of all steel H-piles below mean low water (MLW) and a detailed (Level II/III) inspection on approximately 10% of the steel H-piles. The Level II/III detailed inspection was conducted on the same H-piles that received a Level II/III detailed inspection during the 2009 Halcrow inspection. The detailed inspection was performed by using an ultrasonic thickness (UT) gauge to collect thickness measurements (readings) of the pile webs and flanges at the mudline and at mid-pile elevations. The mid-pile readings were taken at either above existing channel repairs on the H-piles, at the top set of bolts connecting the repairs to the H-piles, within the areas of severe deterioration at MLW, at the bottom set of bolts connecting the repairs to the H-piles, or below the channel repairs. Pile ratings were based on a combination of visual assessment and UT measurements. The presence and condition of the sacrificial anodes connected to the H-piles were also noted and assessed.

The above water inspection included a visual (Level I) inspection of the pile caps, beams, and deck soffits, and the portions of the steel H-piles above MLW. A detailed inspection of the above water portions of the same 10% of steel H-piles selected for the underwater detailed inspection was also performed. Thickness measurements of the pile web and flange were taken using the ultrasonic measuring device or by a micrometer.

1.2 Rating Criteria

The general condition assessment ratings for the entire pier and element groups are based on a six point assessment scale developed by the American Society of Civil Engineers (ASCE). The six condition ratings are:

- Good: No visible damage, or only minor damage is noted. Structural elements may show very minor deterioration, but no overstressing is observed. No repairs are required.

- Satisfactory: Limited minor to moderate defects or deterioration are observed, but no overstressing is observed. No repairs are required.

- Fair: All primary structural elements are sound, but minor to moderate defects or deterioration are observed. Localized areas of moderate to advanced deterioration may be present but do not significantly reduce the load-bearing capacity of the structure. Repairs are recommended, but the priority of the recommended repairs is low.

- Poor: Advanced deterioration or overstressing is observed on widespread portions of the structure but does not significantly reduce the load-bearing capacity of the structure. Repairs may be carried out with moderate urgency.

- Serious: Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of primary structural components. Local failures are possible and loading restrictions may be necessary. Repairs may need to be carried out on a high-priority basis with urgency.

- Critical: Very advanced deterioration, overstressing, or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and load restrictions should be implemented as necessary. Repairs may need to be carried out on a very priority basis with strong urgency.

The pile rating criteria are based on a five point assessment scale defined by average residual thickness of the pile flange and web. The ratings are listed and defined below:

- No Damage: Protective coating intact. No apparent loss of material.

- Minor: Less than 50 percent of perimeter or circumference affected by corrosion at any elevation or cross section. Remaining thickness greater than 0.500 in. (1/2 in.).

- Moderate: Over 50 percent of perimeter or circumference affected by corrosion at any elevation or cross section. Remaining thickness between 0.375 in. (3/8 in.) and 0.500 in. (1/2 in.).

- Major: Partial loss of flange edges or visible reduction of wall thickness on piles. Remaining thickness between 0.250 in. (1/4 in.) and 0.375 in. (3/8 in.).

- Severe: Structural bends or buckling and breakage. Remaining thickness of 0.250 in. (1/4 in.) or less.

Corrosion of steel elements is classified using the following assessment terms:

- Minor (or Light): A light surface corrosion with no apparent loss of section.

- Moderate: Corrosion that is loose and flaking with some pitting. The scaling or exfoliation can be removed with some effort by use of a scraper or chipping hammer. The element exhibits measurable but no significant loss of section.

- Severe: Heavy, stratified corrosion or corrosion scales with extensive pitting. Removal requires exerted effort and may require mechanical means. Significant loss of section.

The damage grades used to describe the concrete beams and pile caps are based on a four point assessment scale and are listed and defined below:

- No Damage: Good original surface, hard material, sound.

- Minor: Mechanical abrasion or impact dents up to 1 in. General cracks up to 1/16 in. Occasional corrosion stains but no exposed reinforcing. Small shallow pop-out spalls.

- Moderate: Structural cracks up to 1/8 in. Corrosion stains and cracks up to 1/4 in. wide and open spalls with no exposed steel reinforcing. Signs of chemical deterioration.

- Severe: Structural cracks wider than 1/8 in. or complete breakage. Complete loss of concrete cover due to corrosion of reinforcing steel with over 30 percent of diameter loss for any main reinforcing bar. Loss of concrete cover (exposed steel) due to chemical deterioration.

Chemical deterioration in the above context can include sulfate attack, alkali-silica reaction, or ettringite distress.

Cracking of concrete elements, defined as a separation into two or more parts as identified by the space between fracture surfaces, is categorized using the following assessment terms:

- Hairline - Crack width less than 1/32 in.

- Fine - Crack width between 1/32 in. and 1/16 in.

- Medium - Crack width between 1/16 in. and 1/8 in.

- Wide - Crack width greater than 1/8 in.

The types of cracks identified include overstressing, corrosion, and general cracking. An overstressing crack results from external loads which cause high internal stresses that exceed the strength of the concrete member. Corrosion cracks are the result of the expansion of chemical products generated by the corrosion of the steel reinforcement. General cracks typically include shrinkage, thermal and chemical reaction cracks caused by the expansion of concrete, which occurs during chemical reaction between concrete constituents or these constituents and the environment.

2. FACILITY DESCRIPTION

2.1 PIER DESCRIPTION

Pier 40 was constructed between 1958 and 1962 and is an approximately 810 ft by 810 ft steel pile-supported high-level platform, and includes a typically 175-ft-wide two-story shed along the pier perimeter surrounding an approximately 400-ft-long by 410-ft-wide Court Yard (Photo B-1). The pier shed is currently being used as the project office and maintenance facility for the Hudson River Park Trust, a public parking facility, and various spaces for other commercial and not-for-profit uses, while the Court Yard currently serves as an athletic field. Along the pier perimeter, a 20-ft-wide apron serves as a public esplanade. At the southwest corner of the structure, there is a 55-ft-wide by 142-ft-long finger pier extension supporting a one-story shed (Photo B-2).

The main pier, which includes the Pier Shed and Court Yard, and the Finger Pier Extension are supported on 3,328 and 135 14BP89 steel H-piles, respectively. The area referred to as the Court Yard in this report is identified as the Truck Court in previous inspection reports. The H-piles are driven to bedrock (as indicated on Roberts and Schaefer Co. Drawing F-7). The pier's main deck consists of prestressed concrete panels supported on cast-in-place reinforced concrete beams and pile caps above the steel H-piles. Pile clusters are located along the building column lines under the Pier Shed and are spaced 50 ft to 55 ft on center in the east-west direction, and between 40 ft and 55 ft in the north-south direction (Photo B-3). There are no pile clusters under the Court Yard, which is supported by 483 single steel H-piles (Photo B-4).

The reinforced concrete pile caps over the single steel H-piles typically measure 3 ft long by 3 ft wide by 4 ft deep, while the pile caps over the pile clusters are typically 6 ft deep with variable length and width dimensions. The pile embedment within the concrete pile caps is 1 ft and the pile caps do not contain steel reinforcing around the H-piles.

The reinforced concrete pile cap beams that span between the pile caps in the north-south direction are typically 16 in. wide by 30 in. deep within the Pier Shed and 16 in. wide by 36 in. deep within the Court Yard. The concrete pile cap beams that abut the concrete edge beams at the bents with H-piles clusters are typically 42 in. deep. The pile cap beams generally span north to south, except in the area directly west of the

Court Yard where the pile cap beams span east to west. One foot thick precast concrete panels with two layers of prestressing strands and a 2 in. thick wearing surface span between the pile cap beams to form the deck of the pier structure.

The 135 steel 14BP89 H-piles supporting the Finger Pier Extension are primarily distributed along the north and south edges of the pier with a batter pile and plumb pile spaced approximately 5 ft on center (Photo B-5). There are a total of 18 H-piles located along the centerline of the Finger Pier Extension that support transverse concrete beams (oriented north-south) which span between three longitudinal concrete beams (oriented east-west) that run along the northern edge, southern edge, and center of the Finger Pier Extension. The longitudinal concrete beams also serve as pile caps for the steel H-piles. A reinforced concrete deck spans over the transverse and longitudinal concrete beams.

A total of 11 bollards are located along the southern edge of the pier, seven bollards and two cleats are located along the western edge of the pier, and eight bollards and eight cleats are located along the northern edge of the pier. There are also four bollards and two cleats on the Finger Pier Extension.

Four vessels: Hornblower Hybrid, Hornblower Infinity, Hornblower Serenity, and Vista Jubilee moor along the northern side of the pier (Photo B-6). A timber fender system comprises timber H-piles, wales, and chocks extends the length of the northern edge of the pier, and additional timber fender pile clusters are located at the northwestern corner of the pier and at the offshore corners of the Finger Pier Extension (Photos B-7 and 8). Four steel pipe H-piles with rubber fender blocks are located between Bents N and P-bar and serve as the fender system for the Hornblower vessels.

Originally, an impressed current cathodic protection system provided protection to the H-piles and was operational for approximately ten years before it was abandoned due to maintenance problems. In 1989, a sacrificial anode system replaced the original impressed current system. A new sacrificial anode system with 33 test stations was installed by the Trust in 2000.

2.2 EXISTING PILE REPAIRS

A total of four types of repairs were implemented on the steel H-piles at Pier 40 during two repair phases in the late 1980s. Of these repair types, three are located at the tops of the H-piles and one is located at MLW. At the tops of the H-piles, either a protective epoxy coating, welded steel plates with a protective epoxy coating (steel plate and epoxy repair), or concrete jackets (encasement repair) were installed (Photos B-9 and B-10). These repairs were installed to protect or structurally reinforce the H-piles within the splash zone, which is where a typical zone of severe corrosion is located. The repair at MLW, consisting of C12x30 or MC12x35 channels with 12 or 20 bolts, respectively, functions to span a typical zone of severe corrosion at the MLW elevation and structurally connect the H-pile pile sections above and below the corrosion (channel repair) (Photo B-11 and 12).

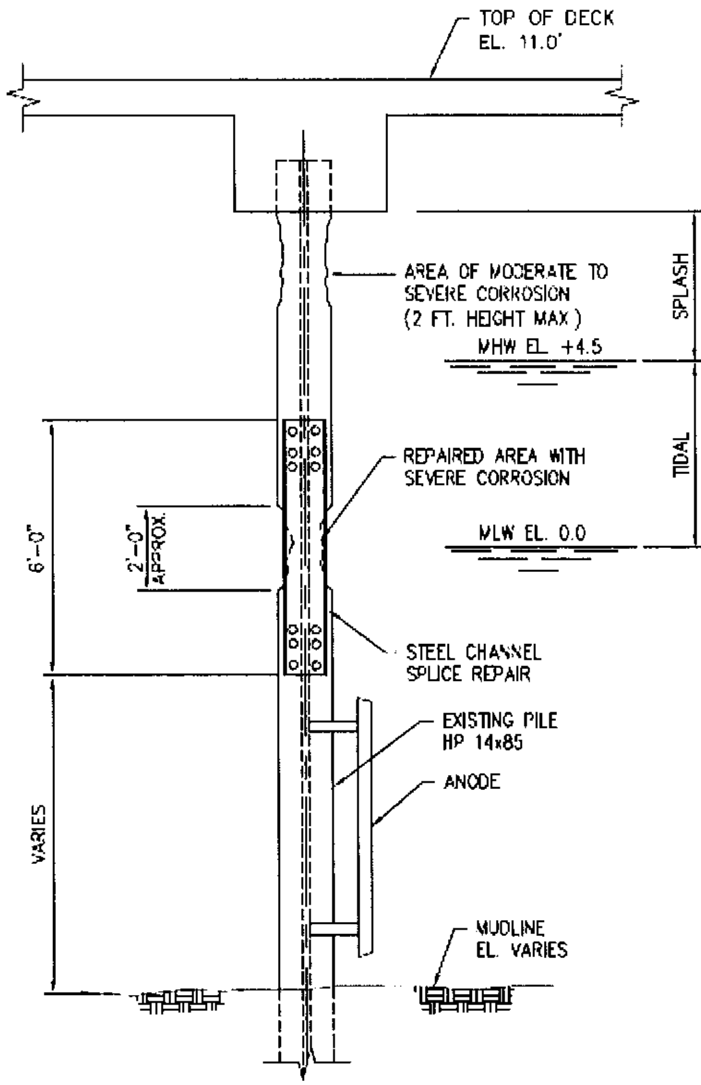
Nearly all H-piles have at least one sacrificial anode installed. A typical elevation view of a pile with a channel repair is shown on Figure 2-1.

2.2.1 Pier Shed

Nearly all H-piles under the Pier Shed have channel repairs, and the majority of H-piles from Pile Bents F through S and between Pile Rows 4 and 19 have either a steel plate with epoxy coating or an encasement repair installed above MLW. The steel plate and epoxy repairs consist of welded steel plates from approximately 3 ft above MLW to within a few inches of the underside of the pile caps. The epoxy coating extends to the pile cap. Throughout the Pier Shed, a total of 438 H-piles were observed with steel plate and epoxy coating repairs. A total of 382 H-piles have protective epoxy coating without steel plates.

Encasement repairs are generally 3 ft to 4 ft in length and extend down from the pile caps. A total of 254 concrete encasements were observed on H-piles under the Pier Shed. The greatest concentration of encasement repairs are located under the Pier Shed, west of the Court Yard.

In addition to these short encasement repairs, approximately 34 steel H-piles contain full-length concrete encasements or are encased in concrete for most of their length. These H-piles are located mostly under the Pier Shed in an area to the west of the Court Yard.



TYPICAL STEEL H-PILE
SCALE: 1/4"=1'-0"

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TYPICAL CHANNEL REPAIRS

FIG 2-1

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Halcrow

2.2.2 Court Yard

The H-piles under the Court Yard only have channel repairs. Unlike the Pier Shed, where all of the H-piles have repairs, only the most severely deteriorated H-piles under the Court Yard were previously addressed. According to calculations performed by Parsons Brinckerhoff and Quade and Douglas Inc. for the repairs installed in the late 1980s, H-piles with a remaining flange thickness of 1/4 in. were determined to be more than adequate to structurally support the required HS20-44 live loads without the need for repair. Thus, the criterion used for recommending repairs to H-piles under the Court Yard included a flange thickness of 1/4 in. or less.

2.2.3 Finger Pier Extension

The majority of the H-piles under the Finger Pier Extension have channel repairs at MLW or are encased in concrete with fabric stay-in-place formwork. A total of 32 H-piles under the Finger Pier Extension are encased in concrete for all or part of their length.

3. INSPECTION FINDINGS

Pier 40 is in overall **Poor** condition with 22% of the H-piles rated Major and 35% of the H-piles rated Severe, primarily due to severe corrosion within the splash zone at the top of the H-piles.

The Pier Shed and Court Yard pile and underdeck conditions are presented in Appendix A, Figures A-1 through A-6 and Figures A-7 through A-12, respectively. The Finger Pier Extension pile and underdeck conditions are presented on Figure A-13. Photographs of observed conditions are presented in Appendix B, and the cost estimate breakdown is presented in Appendix C. Structural calculations are included in Appendix D.

3.1 STEEL H-PILES

The steel H-piles are in overall **Poor** condition with 35% of the H-piles rated Severe due to severe corrosion within the splash zone and at MLW on H-piles with no channel repairs. The edges of the H-pile flanges with severe corrosion are typically between 1/4 in. thick and knife-edged.

In general, the conditions of the steel H-piles at MLW, and below MLW, with the exception of H-piles without channel repairs, have little bearing on the overall pile ratings because they exhibit only minor to moderate deterioration, and appear to be adequately protected by the sacrificial anodes. Typically, the channel repairs and associated bolts exhibit minor pitting and the steel H-pile sections within these channel repair areas have adequate section remaining (Photo B-13).

A full visual inspection of the underlying steel H-piles was not possible at H-piles with epoxy coating or at H-piles with steel plate and epoxy coating repairs. However, the epoxy coating was broken off in small sections at H-piles that received a Level II/III inspection to reveal the underlying steel. At the epoxy coated H-piles strengthened by a welded steel plate, the underlying steel (original steel under the epoxy coating) exhibits moderate to severe deterioration beneath the epoxy coating. Based on the distribution of the observed deterioration, and the large percentage of the surface area of the H-pile that is hidden by epoxy coating, H-piles that contain a steel plate and epoxy coating with visible rust staining were conservatively graded as Major. It should be noted that the top 1 in. to 3 in., approximately, of steel H-pile between the top of the welded steel plates and the pile cap remains unreinforced and exhibits rust staining and cracking of the epoxy coating at the pile flange edges. At Level II/III inspection areas on steel H-piles with only epoxy coating (without a steel plate repair), removal of the epoxy coating typically revealed a severe pile section. Therefore, all H-piles that exhibited rust staining through an epoxy coating repair (with no welded steel plate) were graded Severe during the inspection. The rust stains bleeding through the epoxy coating, as well as blistering of the coating, suggests that the underlying H-piles continue to corrode (Photo B-14 and 15).

The encasement repairs typically exhibit minor spalling and rust staining, therefore, steel H-piles with encasements were generally rated Minor during the inspection.

The pile condition rating plans are presented on Figures A-1 through A-6 for the Pier Shed and Court Yard and Figure A-13 for the Finger Pier Extension. A summary of the pile ratings are provided in Table 3-1.

Table 3-1 Summary of Pile Conditions

Location	No. of H-piles	Minor		Moderate		Major		Severe	
		No.	%	No.	%	No.	%	No.	%
Pier Shed	2,845	505	18%	669	24%	698	25%	973	34%
Court Yard	483	39	8%	255	53%	50	10%	139	29%
Finger Pier	135	8	6%	17	13%	25	19%	85	63%
Total	**3,463**	**552**	**16%**	**941**	**27%**	**773**	**22%**	**1,197**	**35%**

3.1.1 Pier shed

A total of 2,845 steel H-piles were inspected under the Pier Shed. Of these, 34% are rated Severe due to severe corrosion and section loss at the tops of the H-piles (Photo B-15 to B-21). The non-cluster H-piles exhibit the greatest deterioration because the tops of these H-piles are located within the splash zone, where the sacrificial anodes do not provide protection. In contrast, the pile caps at the cluster pile locations are typically 2 ft lower in elevation and the H-piles are fully submerged at MHW, thereby, receiving intermittent periods of cathodic protection from the sacrificial anodes.

This observation is substantiated by examining the conditions of cluster H-piles at Pile Rows 7 and 17, Bents Q and R. These H-piles are exposed within the splash zone, due to the upwards slope of the deck, and are generally the only cluster H-piles rated Major to Severe.

The cluster H-piles at the center of Bent A/B exhibit severe corrosion with knife-edging of the flanges and have channel repairs that extend to the tops of the H-piles. At Bent A/B, the H-piles at Cluster 18 are grouped together in an encasement that extends to the mudline.

At the third single pile north of Cluster Pile 4 in Bent P, there is one loose bolt at the top of its channel repair. At H-piles 12 and 14 in Bent U, the encasement repairs have full circumference spalls with all of the longitudinal reinforcing bars exposed.

Ultrasonic thickness readings indicate that the H-piles are mostly deteriorated at the pile tops, within the splash zone. It should be noted that the steel is typically heavily pitted above water, which makes it difficult to obtain accurate thickness readings. Above water readings could not be collected on some H-piles using standard inspection equipment. Use of special equipment, e.g. a hand-held grinder, would be required to remove the pitting for accurate readings on these H-piles. A summary of the obtained readings are provided in Table 3-2.

Table 3-2 Pier Shed - Average and Minimum Thickness Measurements on H-piles

Reading Location	Avg. Flange Thickness (Minimum Thickness)	Avg. Flange % Loss	Avg. Web Thickness (Minimum Thickness)	Avg. Web % Loss
Top of Pile	0.371 in. (hole)	40 (100)	0.509 in. (hole)	17 (100)
Mid-pile	0.481 in. (0.273 in.)	22 (56)	0.276 in. (hole)	55 (100)
Mudline	0.497 in. (0.245 in.)	19 (60)	0.473 in. (0.196 in.)	23 (68)

Notes: Section loss is based on the original flange and web thickness of 0.615 in.
The top value is the average of thickness reading, and the number in parenthesis is the minimum thickness reading recorded.

3.1.2 Court Yard

A total of 483 steel H-piles were inspected under the Court Yard. Of these, 29% are rated Severe primarily due to severe corrosion and section loss within the tidal zone at H-piles without channel repairs. There are 152 H-piles without channel repairs. Similar to the cluster H-piles under the Pier Shed, the H-piles under the Court Yard are fully submerged at MHW, thereby, receiving some level of protection from the sacrificial anodes (Photo B-22). There is only one encasement repair and only four steel plate and epoxy repairs on H-piles under the Court Yard.

Ultrasonic thickness readings indicate that the H-piles are the most deteriorated at mid-pile elevation. It should be noted that the steel is typically heavily pitted above water, which makes it difficult, and sometimes not possible, to obtain accurate thickness readings with standard inspection equipment. A summary of the obtained readings are provided in Table 3-3.

Table 3-3 Court Yard - Average and Minimum Thickness Measurements on H-piles

Reading Location	Avg. Flange Thickness (Minimum Thickness)	Avg. Flange % Loss	Avg. Web Thickness (Minimum Thickness)	Avg. Web % Loss
Top of Pile	0.463 in. (0.250 in.)	25 (59)	0.497 in. (0.227 in.)	19 (63)
Mid-pile	0.384 in. (0.162 in.)	38 (74)	0.379 in. (0.120 in.)	38 (81)
Mudline	0.526 in. (0.273 in.)	15 (56)	0.507 in. (0.258 in.)	18 (58)

Notes: Section loss is based on the original flange and web thickness of 0.615 in.
The top value is the average of thickness reading, and the number in parenthesis is the minimum thickness reading recorded.

3.1.3 Finger Pier

Of the 135 steel H-piles supporting the Finger Pier Extension, 63% are rated Severe due to severe corrosion and section loss with knife-edged flanges at the tops of the H-piles without encasement repair, and due to severely damaged encasements with the underlying steel H-piles exposed. The steel H-piles with severely damaged encasements are conservatively rated Severe because a full visual inspection was not possible during the inspection due to the presence of timber spacers located between the pile flanges and the remaining concrete comprising the encasements. Of the 32 encasements, 23 have severe damage which has exposed the steel H-piles and or steel cage reinforcement.

Ultrasonic thickness readings indicate that the H-piles are the most deteriorated at mid-pile elevation. It should be noted that the steel is typically heavily pitted above water, which makes it difficult, and sometimes not possible, to obtain accurate thickness readings with standard inspection equipment. A summary of the obtained readings is provided in Table 3-4.

Table 3-4 Finger Pier - Average and Minimum Thickness Measurements on H-piles

Reading Location	Avg. Flange Thickness (Minimum Thickness)	Avg. Flange % Loss	Avg. Web Thickness (Minimum Thickness)	Avg. Web % Loss
Top of Pile	0.359 in. (hole)	42 (100)	0.528 in. (0.253 in.)	14 (59)
Mid-pile	0.453 in. (0.368 in.)	26 (40)	0.396 in. (0.263 in.)	36 (57)
Mudline	0.443 in. (0.367 in.)	28 (40)	0.434 in. (0.245 in.)	30 (60)

Notes: Section loss is based on the original flange and web thickness of 0.615 in.
The top value is the average of thickness reading, and the number in parenthesis is the minimum thickness reading recorded.

3.2 CONCRETE PILE CAP

3.2.1 Pier Shed and Court Yard

The concrete pile caps under the Pier Shed and Court Yard are generally in **Fair** condition with corrosion cracks on the cap soffits that extend from the flange tips of the steel H-piles to the bottom corners of the pile caps. At a number of locations, these corrosion cracks have either extended along the vertical faces of the concrete pile caps at a 45 to 60 degree angle, or have resulted in spalls up to 2 ft high along the bottom corners of the caps (Photos B-14 and B-15). These deficiencies do not directly affect the load bearing capacity of the H-piles, however they could affect the integrity of the connections between the steel H-piles and the concrete caps. In the Court Yard, the pile caps typically have hairline map cracks on the vertical faces with efflorescence (Photo B-23).

At the pile caps along the edge beam of the Pier Shed, there are vertical cracks up to 1/2 in. wide and spalls due to severe corrosion of the bollard through-bolts at isolated locations. The vertical cracks and spalls on the pile caps due to corrosion of the bollard through-bolts are shown in Appendix A, Figures A-7 to A-12.

3.3 CONCRETE PILE CAP BEAM AND EDGE BEAM

3.3.1 Pier Shed and Court Yard

The concrete pile cap beams under the Pier Shed and Court Yard are in **Fair** condition with typical rust staining and opposing longitudinal corrosion cracks that have resulted in delaminations along the beam soffits (Photo B-24 and B-25). In isolated locations, the delaminations along the beam soffits have developed into spalls with exposed steel reinforcement for lengths ranging from 1 ft to nearly 15 ft across the entire width of the beams (Photo B-26 and B-27). Under the Court Yard, the pile cap beams typically have hairline map cracks on the vertical faces with efflorescence (Photo B-24).

There are typical vertical cracks up to 1/16 in. wide at the joints between the offshore pile cap beams and edge beams, and also at the mid-span locations at isolated pile cap beams. At the pile cap beam in Bent K, between Piles 14 and 14-bar, a 1/8 in. wide diagonal crack runs from a closed spall at the top of the beam and extends through the full depth of the beam. Between Piles 7-bar and 8 in Bent N, exposed reinforcing and foam is located in an area along the top edge of the beam where additional concrete was placed after the original casting of the pile cap beam. Additionally, steel reinforcement is protruding from the top surface of the pile cap at Pile Row 16.

A concrete closure wall extends around the south, west, and north perimeter of the Court Yard. The closure wall is located above the Court Yard pile cap beams along Pile Row 7-bar, on the south side, Pile Row 16-bar, on the north side, and along Pile Bent P-bar, on the west side, and typically exhibits cracking up to 1/16" and moderate spalling. At Bent P-bar, between Pile Rows 15-bar and 16, there are gaps at the joints between the concrete closure wall and a deck panel for the Pier Shed. Between Bent I and I-bar, along Pile Row 7-bar, a large crack up to 3/4 in. wide is located in the concrete closure wall where the wall is visibly deflected (Photo B-28 and B-29).

At Bent 18 there is a pile cap beam that runs east-west from Pile U to T. This beam exhibits cracking and delamination along the bottom edge, and there is a 2.5 in. gap between the top of the beam and the concrete deck slab above.

The concrete edge beams are in overall **Fair** condition with general areas of minor erosion up to 1 in. deep along the length of the beams. There are isolated delaminations and spalls with exposed reinforcing steel along the top and bottom edges of the edge beams ranging in length from 1 ft to 5 ft and up to 6 in. in depth. The largest area of erosion is located on the western edge beam between Bents 7 and 8-bar, where the full height of the beam is eroded away up to 1 in. deep with exposed reinforcing steel (Photo B-30).

The deficiencies on the pile cap beams and edge beams are shown in Figures A-7 through A-12.

3.3.2 Finger Pier Extension

The longitudinal concrete and transverse concrete beams at the Finger Pier Extension are in overall **Fair** condition. Similar to the beams under the Pier Shed and Court Yard, the beams exhibit rust staining and delaminations in the beam soffits (Photo B-31) with isolated spalls with exposed steel reinforcing.

The fascia of the concrete beams around the perimeter of the Finger Pier exhibit general areas of surface erosion up to 1/2 in. deep along the bottom and top edges. On the exterior face of the northern longitudinal beam at Bent V, there is a full height, 1/8 in. wide vertical crack due to corrosion of the cleat hardware. Also, along the exterior face of northern longitudinal beam, between W.4 and W.6, a 10 ft long by 1/8 in. wide horizontal crack is located at the longitudinal centerline of the beam.

The concrete beam deficiencies are included on Figure A-13.

3.4 CONCRETE UNDERDECK

3.4.1 Pier Shed and Court Yard

The concrete underdeck at Pier 40 is in overall **Fair** condition with minor hairline cracks (Photo B-3 and B-4). Isolated spalls up to 3 in. deep with exposed prestressing strands and reinforcing steel are located throughout the pier (Photo B-32 through B-34).

In general, these spalls are less than 2 sq ft in area with only one or two partially exposed steel reinforcing bars or prestressing strands.

At the deck panel south of Cluster Pile 11 in Bent E, multiple prestressing strands are exposed. In addition, at the deck panel east of Pile 9 East in Bent F, multiple prestressing strands are exposed and are broken (Photo B-35).

An approximately 30 sq ft spalled area with exposed steel reinforcing is located at a cast-in-place portion of the deck, west of the pile cap beam located south of Pile Cluster 7, in Bent F.

Between Bents 16 and 16-bar, several hangers are attached to the concrete underdeck and support a partially collapsed 48 in. diameter concrete outfall. At the offshore end of the pier, the outfall has completely collapsed and is supported by steel beams attached to the H-piles. Several pipe hangers remain attached to the underdeck and no longer support the outfall.

The locations of the deck panel spalls are included on the underdeck deficiency plans in Figures A-7 through A-12.

3.4.2 Finger Pier Extension

The Finger Pier Extension concrete underdeck is in overall **Fair** condition with areas of shallow cover and spalls with exposed steel reinforcing (Photo B-36 and B-37). The underdeck at the southwestern bay of the pier has an approximate 40 sq ft spall with two broken reinforcing bars and additional exposed steel reinforcing in both directions.

3.5 PIER APRON

The top of deck around the Pier Shed is generally in **Satisfactory** condition with cracks in the asphalt surface and typical deterioration of the expansion joints. There is a dislodged cleat at Bent 12 along the western edge of the pier, and a cleat with a broken horn near Bent J along the northern edge of the pier.

3.6 FENDER SYSTEM

The fender systems for the four Hornblower vessels, along the northern edge of the pier, are in overall **Good** condition. The timber fender system along the north side of the pier is approximately 300 ft long and exhibits minor deterioration and abrasion (Photo B-8 and B-38). To accommodate the Hornblower vessels that berth along the north side of the pier, steel pipe H-piles with tires and rubber fender blocks are attached to the concrete edge beam and serve as a fender system. These piles exhibit 100% coating loss and minor corrosion.

There is no functioning fender system along the southern and western edges of Pier 40. At the southern edge of the pier, portions of deteriorated timber chocks and rubber blocks remain attached to the fascia of the concrete edge beam.

The fender system on the Finger Pier Extension is also non-functional with only remnants of timber chocks attached to the fascia of the exterior longitudinal beams.

The timber fender cluster piles located at the western corners of the Finger Pier Extension and the northwest corner of Pier 40 are in **Poor** condition. All of the accessible timber piles are split or broken within the tidal zone (Photo B-39 and B-40).

3.7 CATHODIC PROTECTION SYSTEM

The cathodic protection system consists of sacrificial anodes attached to the north and/or south faces of each Pile with one to five sacrificial anodes per pile. At least two generations of sacrificial anodes exist at most pile locations under the Pier Shed and one generation of sacrificial anodes exists under the Court Yard. Based on field observations, it appears that both generations of anodes are of similar size. The anodes installed as part of the most recent cathodic protection efforts include a 144 lb anode located just below the mid-tide elevation, and a 115 lb anode located just above the mudline elevation. Details of the most recent anode installation efforts are provided in Appendix F. As outlined in the 2000 cathodic protection system plans, two sacrificial anodes were installed per pile with additional anodes installed as necessary. Additionally, 33 test stations were installed throughout the pier.

As part of this inspection, an estimated section loss of each anode was recorded and used in conjunction with electrical potential readings recorded at the 33 test stations to estimate the remaining functional life of the cathodic protection system. Table 3-5, below, presents the electrical potential readings collected in 2005, 2006, 2008, and 2014.

Pier 40 - Cathodic Protection Performance Evaluation

Pile (+) to Silver/Silver Chloride Reference (-) Potential - Volts												
Measurement Location	13-Sep-05			13-Oct-06			21-Nov-08			27-Feb-14		
	1' Below Water Line	Half-Way to Mudline	At Mudline	1' Below Water Line	Half-Way to Mudline	At Mudline	1' Below Water Line	Half-Way to Mudline	At Mudline	At MLW	Half-Way to Mudline	At Mudline
Test Location 1	-0.913	-0.916	-0.918	-0.900	-0.890	-0.900	-0.911	-0.904	-0.902	-0.844	-0.850	-0.856
Test Location 2	-0.980	-1.022	-1.050	-0.920	-0.925	-0.940	-0.918	-0.922	-0.905	-0.860	-0.913	-0.890
Test Location 3	-1.022	-1.045	-1.095	-0.963	-0.975	-0.970	-0.942	-0.955	-0.924	-0.920	-0.970	-0.928
Test Location 4	-1.031	-1.055	-1.070	-0.928	-0.930	-0.920	-0.913	-0.918	-0.911	-0.910	-0.926	-0.912
Test Location 5	-1.016	-1.050	-1.035	-0.885	-0.901	-0.903	-0.881	-0.893	-0.895	-0.874	-0.885	-0.864
Test Location 6	-1.055	-1.065	-1.058	-0.952	-0.940	-0.935	-0.937	-0.939	-0.934	-0.954	-0.938	-0.920
Test Location 7	-1.068	-1.070	-1.065	-0.948	-0.950	-0.943	-0.923	-0.932	-0.926	-0.973	-0.930	-0.848
Test Location 8	-1.080	-1.086	-1.058	-0.948	-0.920	-0.952	-0.935	-0.930	-0.924	-0.864	-0.919	-0.884
Test Location 9	-1.017	-1.020	-1.021	-0.870	-0.870	-0.850	-0.873	-0.864	-0.852	-0.925	-0.929	-0.896
Test Location 10	-1.100	-1.105	-1.109	-0.902	-0.910	-0.904	-0.895	-0.900	-0.887	-0.928	-0.929	-0.889
Test Location 11	-1.160	-1.190	-1.230	-0.860	-0.880	-0.860	-0.875	-0.873	-0.874	-0.877	-0.880	-0.860
Test Location 12	-1.058	-1.100	-1.130	-0.925	-0.930	-0.910	-0.912	-0.926	-0.924	-0.882	-0.880	-0.866
Test Location 13	-1.080	-1.108	-1.120	-0.925	-0.940	-0.890	-0.916	-0.924	-0.922	-0.970	-0.937	-0.905
Test Location 14	-1.054	-1.112	-1.115	-0.900	-0.930	-0.890	-0.887	-0.908	-0.903	-0.858	-0.888	-0.850
Test Location 15	-1.130	-1.140	-1.140	-0.880	-0.880	-0.860	-0.874	-0.894	-0.891	-0.866	-0.868	-0.864
Test Location 16	-1.072	-1.085	-1.110	-0.890	-0.880	-0.860	-0.895	-0.906	-0.890	-0.875	-0.889	-0.907
Test Location 17	-1.104	-1.110	-1.120	-0.890	-0.890	-0.880	-0.913	-0.926	-0.906	-0.836	-0.839	-0.803
Test Location 18	-1.056	-1.068	-1.065	-0.880	-0.890	-0.880	-0.883	-0.905	-0.902	-0.898	-0.836	-0.797
Test Location 19	-1.120	-1.150	-1.158	-0.910	-0.910	-0.900	-0.895	-0.898	-0.886	-0.815	-0.780	-0.746
Test Location 20	-2.150	-2.210	-2.200	-0.904	-0.915	-0.843	-0.904	-0.908	-0.904	-0.847	-0.858	-0.843
Test Location 21	-2.004	-2.140	-2.110	-0.890	-0.900	-0.880	-0.890	-0.897	-0.894	-0.795	-0.792	-0.778
Test Location 22	-1.860	-1.880	-1.880	-0.890	-0.910	-0.900	-0.887	-0.897	-0.888	-0.875	-0.870	-0.857
Test Location 23	-1.630	-1.650	-1.650	-0.912	-0.908	-0.900	-0.916	-0.905	-0.911	-0.820	-0.840	-0.811
Test Location 24	-1.084	-1.122	-1.108	-0.905	-0.920	-0.904	-0.892	-0.897	-0.893	-0.905	-0.910	-0.886
Test Location 25	-1.420	-1.460	-1.450	-0.880	-0.900	-0.890	-0.902	-0.914	-0.907	-0.900	-0.879	-0.863
Test Location 26	-1.220	-1.360	-1.340	-0.900	-0.910	-0.890	-0.894	-0.903	-0.902	-0.897	-0.896	-0.878
Test Location 27	-1.198	-1.208	-1.210	-0.930	-0.940	-0.890	-0.927	-0.933	-0.902	-0.953	-0.955	-0.917
Test Location 28	-1.088	-1.113	-1.124	-0.902	-0.924	-0.930	-0.898	-0.903	-0.891	-0.930	-0.913	-0.877
Test Location 29	-1.092	-1.108	-1.110	-0.920	-0.933	-0.925	-0.902	-0.908	-0.903	-0.947	-0.951	-0.940
Test Location 30	-1.087	-1.108	-1.134	-0.928	-0.925	-0.901	-0.914	-0.925	-0.915	-0.966	-0.980	-0.959
Test Location 31	-1.109	-1.120	-1.180	-0.911	-0.923	-0.917	-0.914	-0.915	-0.912	-0.954	-0.947	-0.919
Test Location 32	-1.034	-1.074	-1.100	-0.890	-0.910	-0.890	-0.883	-0.885	-0.902	-0.910	-0.919	-0.967
Test Location 33	-1.030	-1.065	-1.055	-0.904	-0.904	-0.903	-0.887	-0.897	-0.884	-0.890	-0.891	-0.874
Minimum Potential	**-0.913**			**-0.843**			**-0.852**			**-0.746**		
Maximum Potential	**-2.210**			**-0.975**			**-0.955**			**-0.980**		
Average Potential	**-1.208**			**-0.907**			**-0.905**			**-0.888**		

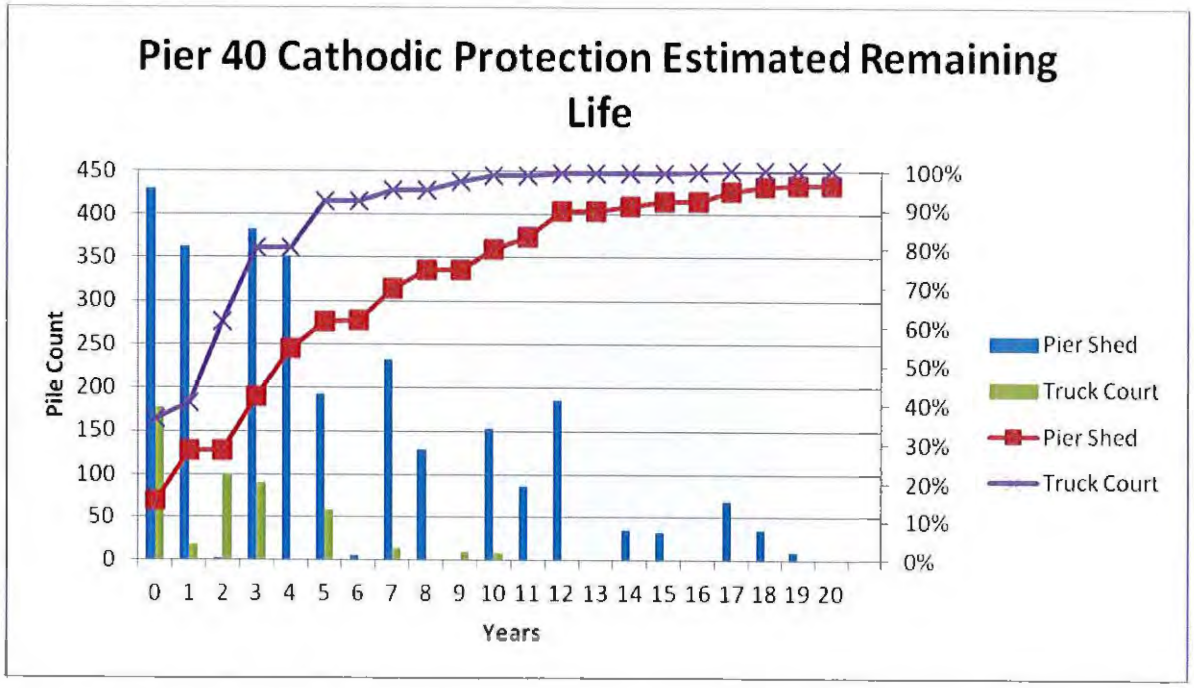
A comparison of these electrical potential readings shows that the current readings are slightly less negative than what were observed in earlier inspections. This slight, but notable decline in potential readings, combined with current observed section loss quantities on the anodes is a good indicator that the cathodic protection system is working to protect the underwater portions of the H-piles. It should be noted when reviewing these potential readings that the minimum threshold for cathodic protection, using a silver/silver-chloride reference cell in a 3.5m solution of KCl (Potassium Chloride), is -0.750V. As shown in Table 3-5, the average potential measured in 2014 is -0.888V, which is slightly less negative than the average measured in 2008, -0.905V. A reading of -0.746V was recorded at Test Location 19. Although this is below the minimum threshold of -0.750V, the difference is negligible and this pile can still be considered as protected. The anodes at this particular pile are likely close to the end of their functional life.

In addition to the remaining anode material, water depths (wetted surface areas of the H-piles) play a factor in the remaining functional life of the system. Average submerged pile depths of 15 ft and 20 ft were used for the Court Yard and the Pier Shed, respectively. The table below shows the estimated remaining life of the cathodic protection system.

Table **3-6** Pier 40 – Remaining Useful Life of Cathodic Protection System

Estimated Years Remaining	**Pier Shed**		**Court Yard**	
	Pile Count	Cumulative % of H-piles	Pile Count	Cumulative % of H-piles
0	430	15%	177	37%
1	363	28%	19	41%
2	3	28%	101	61%
3	383	42%	91	80%
4	352	55%	0	80%
5	193	61%	58	92%
6	6	62%	0	92%
7	233	70%	14	95%
8	129	75%	0	95%
9	1	75%	10	97%
10	153	80%	8	99%
11	87	83%	0	99%
12	186	90%	2	99%
13	1	90%	0	99%
14	36	91%	1	100%
15	33	92%	0	100%
16	0	92%	1	100%
17	68	95%	1	100%
18	35	96%	0	100%
19	10	96%	0	100%
20	1	96%	0	100%
More	12	100%	0	100%

The Court Yard H-piles are typically protected by two anodes per pile. The remaining useful life of these anodes ranges between 0 years and 17 years, with a median remaining useful life of 1 year. The Pier Shed H-piles are protected by two to five anodes per pile. The remaining useful life of these anodes ranges between 0 years and more than 20 years, with a median remaining useful life of 3.5 years. The graph below illustrates the information provided in the table above.



As illustrated in the graph above, a steep increase in the number of unprotected H-piles is expected over the next 3 years under the Court Yard. The Pier Shed is expected to experience a steady increase in unprotected H-piles over the next 10 years.

Overall, the electrical potential readings presented in Table 3-5 suggest that all H-piles are currently protected by the existing cathodic protection system; however, the estimated remaining life calculations suggest that the sacrificial anode system is nearing the end of its useful life, and is possibly no longer adequately protecting the H-piles in many areas. The following is a list of items that were considered when developing recommendations for the cathodic protection system.

- Potential readings were recorded at 33 locations throughout the pier, which represent only 1 percent of the total H-piles supporting the pier. A more extensive survey of electrical potentials would provide a more accurate analysis of the estimated remaining life of the system.
- The original pier cathodic protection system was an impressed current-type system and all H-piles are likely electrically bonded. This bonding allows H-piles with excess anode material to protect nearby H-piles with little or no anode material. The amount of protection offered, however, cannot be confirmed without a more extensive potential survey and therefore, was not included in this evaluation.
- The estimated remaining life calculations are reliant on visual observations and estimates of remaining anode sections.

While it is estimated that the majority of the pier is protected at the time of this report, most of the sacrificial cathodic protection system appears to be at the end of its useful life. Considering the items presented in the list above, and the degenerative effects of saltwater on an uncoated steel structure, it is recommended that action be taken to protect the steel H-piles supporting Pier 40.It is important to note that the data presented does not reflect the entire pier structure and that the items listed above introduce additional uncertainties regarding the overall protection of the pier.

In addition, the reference electrodes at the pier's potential test stations are no longer functioning and it is recommended that they be abandoned.

4. PIER LOAD RATING AND STRUCTURAL ANALYSIS

4.1 PURPOSE OF ANALYSIS

A structural analysis was performed to determine the overall load rating of the pier and the capacity of the piles to resist lateral loads. The load rating portion of the analysis was based upon load combinations provided in the Trust's 2001 Structural Design Guidelines, included in Appendix E. The analysis was performed using Allowable Stress Design (ASD) criteria. Evaluation of the steel and concrete elements was performed in accordance with the American Institute of Steel Construction (AISC) Steel Construction Manual and American Concrete Institute (ACI) 318.

The structural calculations are provided in Appendix D.

4.2 OVERALL LOAD RATING

The overall load rating for the pier is governed by the element with the minimum remaining live load capacity, based upon the levels of deterioration observed during the inspection. This load rating is based upon a Routine inspection, which includes a swim-by inspection of 90 percent of the structural elements and a hands-on (Level 2) inspection of 10 percent of the structural elements. Since this inspection is general in nature, the load rating contains generalizations of conditions and assigns a single load rating to large areas of the pier.

The structural elements that were analyzed included the steel H-piles, concrete pile cap beams, and the concrete deck. The allowable live load capacity of the pier was determined by subtracting the dead loads from the remaining capacity of the elements.

In addition to determining the overall allowable uniform live load capacity, each structural element was analyzed to check two different fire truck specifications within an allowable stress of 130% in the element. The details of each fire truck are presented below:

- Fire Truck: 68,000 lb vehicle load with a 24 kip wheel load (48 kip rear axle load) and a 37ft – 9in. wheel base

- Rescue Truck: 74,800 lb vehicle load with a 26 kip wheel load (52 kip rear axle load) and a 17ft – 2in. wheel base.

4.2.1 Steel H-Piles

(A) METHODOLOGY

The condition ratings for the steel H-piles under the Pier Shed, Court Yard and Finger Pier Extension were primarily dictated by the level of deterioration found within the splash zone. For the analysis, each pile was broken up into four zones: top, mid-pile, bottom (mudline), and below the mudline. The ultrasonic thickness measurement data for the piles identified the top zone as the most extensively deteriorated and this zone was conservatively estimated to include the top 4.75 ft of the piles. Below this zone, the mid-pile zone, is where the majority of the piles previously received a structural repair (Parsons 1989), consisting of C-channels bolted to each flange of the H-Piles. From the bottom of the 1989 repair to the mudline is the mudline zone. The thickness measurements within this zone suggest only minor to moderate pile deterioration. Below the mudline, the piles were assumed to have their original cross section, or to be very close to their original cross section.

To analyze the piles using these distinct deterioration zones, STAAD, a finite element based structural analysis program, was utilized. The thickness readings within each pile grade (Minor, Moderate, Major, and Severe) were averaged and examined for both the 2009 and 2014 inspections. The lower (more conservative value) was taken and used as the representative flange and web thickness for each pile grade and zone.

To determine the overall capacity of each representative pile, three checks were performed. The first check was to evaluate the global stability of each pile. Due to the complexity of the H-pile section, a non-standard approach was needed to calculate the capacity of a deteriorated pile. In STAAD, an iterative buckling analysis was performed to determine the Euler buckling factor for a given pile grade. From the Euler buckling factor, the Euler buckling critical stress was determined. This value was then utilized to calculate the overall pile capacity.

The second check, which was generally found to be the controlling failure catalyst for most representative pile grades, was for local buckling of the pile flanges. As the H-pile flanges deteriorate and lose cross sectional area, they become slender and therefore, are more likely to fail due to buckling. To check for local buckling of the flange, the Euler buckling critical stress was applied for evaluation through AISC Chapter E, section 7.

The third check for the piles was made to evaluate the most severe corrosion observations and representative pile sections for local crushing/yielding of the steel. Results from this check revealed that local crushing of the steel was not a controlling failure mechanism for the piles.

An evaluation of lateral loads on the pier structure was also included in the structural analysis to determine the capacity of the piles to resist ice, wind, wave, current, and mooring loads. The overall analysis, and determination of the remaining allowable live load for the pier, was performed by using the loading combinations included in the HRPT Structural Design Guidelines. The applicable load combinations and loads are provided in Table 4-1. The alpha-numeric load combination designations correspond to those identified in the HRPT Structural Design Guidelines.

Table 4-1 Load Combinations for Lateral Load Analysis

Load	Load Combination					
	S1	S3	S5	S7	S8	S9
Dead	1	1	1	1	1	1
Live	1	1	-	-	1	-
Current	-	1	1	-	-	1
Wind on Structure	-	0.3	1	-	-	1
Earthquake	-	-	-	1	-	-
Wave*	-	0.3	1	-	-	-
Ice	-	-	-	-	1	1
% Allowable Stress	100	125	140	133	140	150

* Wave loads were not included in the HRPT Structural Design Guidelines Load Combinations, however, they have been included in this analysis.

Given the complexity and size of Pier 40, the structural analysis was primarily performed by utilizing a three-dimensional model in a structural analysis program.

Considering that the pier is structurally separated by expansion joints, the 300 ft by 220 ft northwestern corner of the structure was determined to be the most critical portion due to its location and vulnerability to lateral loads, as well as the exposed height of the steel H-piles supporting it. This portion of the pier was modeled from the steel H-piles to the two-story concrete building on top of the pier deck. The steel H-piles were entered into the model using their original cross-sectional area. The mudline along the exterior and interior piles was assumed to be 15 ft and 12 ft, respectively, below MLW. The Court Yard was not analyzed for lateral loads. It was assumed that all lateral loads are resisted by the Pier Shed structure.

The Finger Pier Extension was modeled separately in a structural analysis program, as it is a completely independent structure and is located where the greatest water depths were identified beneath Pier 40. The mudline along at the Finger Pier Extension was assumed to be 30 ft below MLW.

The loading combinations included in Table 4-7 were applied to the structural models of the northwest corner of Pier 40 and the Finger Pier Extension. Using the results from the structural analysis for the entire pier structure, the bending moments incurred due to the lateral loads were determined for each individual steel H-pile. For the purposes of this analysis, the worst-case bending moment from each applied lateral load (wind, current, ice, earthquake, wave) was reapplied to a STAAD model of a single pile with a reduced cross section, based on its pile grade. From this, the steel utilization ratio was determined. The steel utilization ratio, combined with the axial capacity of the pile, was then used to calculate the axial steel utilization ratio for the pile. These values were then combined using the axial/flexure interaction equation in AISC Chapter H, section 1 to determine whether the deteriorated pile could resist the combined bending and axial loads.

It should be noted that this analysis is conservatively based since it assumes that every pile under the pier is resisting the worst case combination of moment and axial loads, and that there is no redistribution of the moments to adjacent piles.

ASD was used for this analysis and a factor of safety of 1.67 was included in all calculations.

(B) DESIGN VALUES

The typical pile length for the Pier Shed, Court Yard, and Finger Pier Extension were based on the longest exposed pile height observed during the inspection with a point of pile fixity located 12 ft below the mudline. The point of fixity below the mudline was identified by using "L-pile," a pile analysis program, and through geotechnical information obtained from Mueser Rutledge's Compilation of Available Geotechnical Data Report, dated December 1997. The results of the L-Pile analysis are included in Appendix E of this report.

A summary of the steel H-pile design values used for the STAAD program are as listed in Table 4-2. It should be noted that no level II piles were rated as minor under the Court Yard, as noted in Table 4-3, Minor Pile.

Table 4-2 Summary of Steel H-Pile Design Values

Location	Pile Length (ft)	Fy (ksi)	Dead Load (kips)	Tributary Area (SF)
Pier Shed	37	36	76	312.5*
Truck Court	30		103	441
Finger Pier	46		80.4	225

* The tributary area for the piles supporting the Pier Shed was based on the section of the pier between Bents R and T, and Pile Rows 18 and 19. The length and width of the area is 12.5 ft and 25 ft, respectively.

Table 4-3 Summary of Steel H-Pile Design Values

		Severe Pile		
		Finger Pier (in.)	Pier Shed (in.)	Truck Court (in.)
Top	Flange	0.259	0.274	0.447
	Web	0.407	0.398	0.459
Mid	Flange	0.454	0.271	0.243
	Web	0.384	0.27	0.267
Bottom	Flange	0.418	0.425	0.494
	Web	0.411	0.473	0.502

		Major Pile		
		Finger Pier (in.)	Pier Shed (in.)	Truck Court (in.)
Top	Flange	0.39	0.414	0.452
	Web	0.445	0.441	0.49
Mid	Flange	0.453	0.227	0.326
	Web	0.417	0.286	0.326
Bottom	Flange	0.412	0.465	0.488
	Web	0.44	0.476	0.504

		Moderate Pile		
		Finger Pier (in.)	Pier Shed (in.)	Truck Court (in.)
Top	Flange	0.449	0.459	0.428
	Web	0.43	0.448	0.448
Mid	Flange	0.478	0.325	0.516
	Web	0.275	0.257	0.462
Bottom	Flange	0.42	0.503	0.447
	Web	0.42	0.472	0.472

		Minor Pile		
		Finger Pier (in.)	Pier Shed (in.)	Truck Court (in.)
Top	Flange	0.611	0.525	N/A
	Web	0.608	0.483	N/A
Mid	Flange	0.421	0.224	N/A
	Web	0.493	0.322	N/A
Bottom	Flange	0.52	0.484	N/A
	Web	0.545	0.485	N/A

4.3 LATERAL LOAD ANALYSIS

4.3.1 Loads

The following loads were applied to the structural models for the single pile analyses, the northwest corner of Pier 40, and the Finger Pier Extension.

(A) DEAD LOAD

The dead load for the single pile analysis of a typical non-cluster steel H-pile under the Pier Shed is 76 kips, and is based on a tributary area of approximately 312 sq ft. The dead load on a single typical steel H-pile under the Finger Pier is 80 kips, and is based on a tributary area of 225 sq ft. An additional superimposed dead load of 30 psf was added to the Finger Pier to account for the existing structure since this structure bears on the deck of the Finger Pier.

(B) LIVE LOAD

After all of the aforementioned loads were applied, based on the load combinations, the remaining capacities of the piles were used to determine the maximum allowable live load for the pier.

(C) CURRENT

Based on historical design data for the Hudson River, a current velocity of 2.0 knots was used in the analysis, which results in a uniform line load of 13.2 pounds per linear foot. This was applied over the length of each pile, from MHW to the mudline.

(D) WIND ON STRUCTURE

Based on the wind load criteria provided in the HPRT Structural Design Guidelines, a wind velocity of 110 mph, Exposure C, with an importance factor of 1.0 was applied over the entire structure.

(E) EARTHQUAKE

Although seismic loading was not included in the original scope of work, earthquake loading was applied to the structural models. The following parameters were used for earthquake loading.

- NYC Building Code 2008

- $A = 0.15$
- Site Coefficient, $S = 1.2$
- Importance Factor, $I_e = 1.0$
- Response Modification Factor, $R_w = 5$ for main pier, $R_w = 3$ for finger pier

(F) WAVE

Based on the extreme wave conditions data provided in the HRPT Structural Design Guidelines, a wave force acting on the concrete edge beam was calculated. The wave force was based on a southerly wave with a significant wave height of 10 ft, a wave period of 5.4 seconds, and wavelength of 218 ft. Using these parameters, the wave load applied to the vertical face of the edge beam was calculated to be 223 psf. Uplift due to waves, and waves from the westerly direction were not considered in the analysis.

(G) ICE

Based on ice load data provided in the HRPT Structural Design Guidelines, an ice thickness of 8 in. with a crushing strength of 100 psi was used. The dynamic ice load acting on a pile is 14 kips, with an abrasion load of 1.54 kips located along the sides of the pile. These two ice loads were applied concurrently.

(H) VESSEL MOORING

A mooring analysis for the Hornblower Infinity was performed using OPTIMOOR mooring analysis software. The purpose of the mooring analysis was to determine the lateral loads imposed on the pier structure as a result of wind, current, and waves pushing the moored vessel onto the pier. Although the current fender system for the Hornblower Infinity includes steel pipe piles with rubber blocks; for the purposes of this analysis, a timber fender system comprised of timber piles spaced 8 ft on-center, with timber wales and chocks attached to the concrete edge beam was assumed. Vessel dimensions for the Hornblower Infinity are included in Table 4-4.

Table 4-4 Vessel Particulars for the Hornblower Infinity

Parameter	Vessel Particular
Length (LOA)	205.3
Beam	46.4 ft
Depth	23.3 ft
Draft (Lightship / Full Load)	7.11 / 8.6 ft
Displacement (Lightship / Full Load)	799 / 1034 Long Ton

A summary of the environmental conditions used in the mooring analysis is provided in Table 4-5. All parameters were applied in a north-south (upriver-downriver) direction to estimate the most severe loads on the vessel.

Table 4-5 Summary of Mooring Analysis Environmental Parameters

Parameter	Design Value
Wind Speed	110 mph (96 knots)
Current Velocity	2 knots
Significant Wave Height	10 ft
Wave Period	5.4 seconds

Since the purpose of the analysis was to determine the lateral loads imposed on the steel H-piles by the moored vessel, an assumed mooring arrangement for the Hornblower Infinity was used. The capacities of the mooring lines and bollards were assumed to be sufficient to support the applied loads and were not evaluated in the analysis.

Based on the results of the mooring analysis, the total lateral load imposed on the fender system and pier structure over the 190 ft length (LBP) of the vessel is 253 kips, or 1.3 kips per ft along the length of the concrete edge beam. The loadings imposed by the Hornblower Infinity is comparable to the combined wind and wave loading along the same length of pier, and is therefore not included in the loading combination.

4.3.2 Load Analysis Results

(A) PIER SHED

The results of the structural analysis are summarized in Table 4-6. The table lists each loading combination, the minimum pile grade required to resist each loading combination, and the associated allowable live load for the Pier Shed. It should be noted that, in general, the edge piles, which have their stronger axis oriented in the direction of the imposed lateral loads, resist a larger proportion of the loads and thus are subject to a higher bending moment. The interior piles, including the cluster piles, are typically subject to far less bending moment than the edge piles, with the exception of bending moments incurred do to seismic loading.

Table 4-6 Summary of Results for the Lateral Load Analysis of the Pier Shed

Loading Combination	Applied Loadings	Minimum Required Pile Rating	Allowable Live Load
1	DL + LL	Severe	535 psf
3	DL + LL + C + 0.3W+ 0.3Wa	Severe	255 psf
5	DL + C + W + Wa	Severe	N/A
7	DL + E	Moderate	N/A
8	DL + LL + Ice	Severe	100 psf (with a 2% overstress)
9	DL + C + W + Ice	Severe (with a 2% overstress)	N/A

DL = dead load ; LL = live load ; C = current ;
W = wind ; Wa = wave ; E = earthquake

Based on the results in Table 4-6, Load Combination 8 is the critical load combination and the Pier Shed has an allowable live load of 100 psf in its current condition. Analysis results for severe piles under the Pier Shed subject to a 100 psf public assembly live load indicate that these piles are overstressed 2 percent above their allowable stress. The results of the analysis indicate that the design ice load, which is based on an 8 inch thick layer of ice, is very demanding on the pier structure and is the controlling load within Load Combination 8, as well as Load Combination 9.

It should be noted that Load Combination 7, which requires piles to be in Moderate or better condition, is based on the current seismic code. It is unlikely that Pier 40 was originally designed to resist seismic loads on the order of magnitude that is required by the current code. Therefore, this load combination was not considered to be the critical load combination for this analysis.

A seismic analysis was only performed to determine the adequacy of the existing steel H-piles. No other structural components supporting the pier, or its associated structures, were analyzed for seismic loads.

The capacity of the steel H-piles supporting the column foundations around the Pier Shed was assumed to be the same as the H-piles supporting the deck. It was assumed that the columns for the Pier Shed transfer the Pier Shed loads directly to the column clusters; however, since no building column load take-down was performed, no further evaluation the clusters was performed.

Table 4-7 Summary of Load Ratings for Pile Cluster Steel H-Piles

Location	Pile Live Load Capacity Based on Pile Rating (kips)		
	Moderate	**Major**	**Severe**
Pier Shed	278 kips	268 kips	167 kips

(B) FINGER PIER EXTENSION

The results of the structural analysis are summarized in Table 4-8. The table lists each loading combination, the minimum pile grade required to resist each loading combination, and the associated allowable live load for the Finger Pier Extension.

Table 4-8 Summary of Results for the Lateral Load Analysis of the Finger Pier

Loading Combination	Applied Loadings	Minimum Required Pile Rating	Allowable Live Load
1	DL + LL	Severe	519
3	DL + LL + C + 0.3W+ 0.3Wa	Severe	419
5	DL + C + W + Wa	Severe	N/A
7	DL + E	Moderate	N/A
8	DL + LL + Ice	Severe	0
9	DL + C + W + Ice	Moderate	N/A

DL = dead load ; LL = Live Load ; C = current ;
W = wind ; Wa = wave ; E = earthquake

Based on the results in Table 4-8, Load Combination 8 is the critical load combination and the Finger Pier substructure has an allowable live load of 0 psf in its current condition. The results of the analysis indicate that the design ice load, which is based on an 8 inch thick layer of ice, is very demanding on the pier structure and is the controlling load within Load Combination 8, as well as and Load Combination 9.

It should be noted that Load Combination 7, which requires piles to be in Moderate or better condition, is based on the current seismic code. It is unlikely that Pier 40, or the Finger Pier, were originally designed to resist seismic related loads on the order of magnitude that is required by the current code. Therefore, this load combination was not considered to be the critical load combination for this analysis.

(C) COURT YARD

The Court Yard is assumed to resist vertical loads only (all lateral loads are assumed to be resisted by the Pier Shed). The Court Yard deck is set lower than the surrounding Pier Shed, which is beneficial when examining the load capacity of the Court Yard. First, lower deck elevation of the Court Yard requires a shorter exposed pile length. This, combined with an overall shallower mudline beneath the center of the pier, greatly increases the buckling capacity of the piles. Second, since the deck is set lower than the surrounding Pier Shed, the piles under the Court Yard are submerged for a greater percentage of the tide cycle, allowing the cathodic protection system to better protect the piles. Considering these advantages, the piles under the Court Yard are generally in better condition than the piles under the Pier Shed and the Finger Pier. Consequently, the concrete pile caps and concrete beams under the Court Yard are the controlling structural elements when examining its allowable live load capacity. Since severe defects, such as spalling with exposed reinforcement, are randomly distributed throughout the Court Yard, the allowable live load capacity of a pile cap graded severe is assumed to govern. Based on this, the Court Yard is capable of supporting 150 psf.

5. CONCLUSIONS, RECOMMENDATIONS, COST ESTIMATES

5.1 CONCLUSIONS

In general, Pier 40 is in overall **Poor** condition. The allowable uniform live load rating for the pier has been significantly reduced from its original design live load capacity. Based on its current usage, the overall operational condition of Pier 40 should be maintained to support a minimum allowable live load rating of 100 psf for general assembly, for fire truck loads in all public areas, and to be able to resist all of the aforementioned lateral loads.

The current vertical live load capacity of the pier is summarized in Table 5-1.

Table 5-1 Summary of Load Ratings for Pier 40

Structure	Governing Structural Element	Allowable Uniform Live Load	Fire Truck Access
Pier Shed	Steel H-piles rated Severe	100 psf (with a 2% overstress)	No Restrictions
Court Yard	Concrete pile cap beam with exposed steel reinforcing and Severe piles	150 psf	No Restrictions
Finger Pier Extension	Deteriorated beams and deck	0 psf (with the possibility of ice loading)	Not Applicable
		100 psf (with no possibility of ice loading)	

The Pier Shed is capable of supporting a uniform live load of 100 psf in its existing condition and it is also capable of supporting a fire truck with a 24 kip wheel load and a Rescue Truck with a 26 kip wheel load. The results of the analysis indicate that the design ice load, which is based on an 8 inch thick layer of ice, is very demanding on the pier structure and is the controlling load within Load Combination 8, as well as Load Combination 9.

Earlier inspection reports had discounted ice loading conditions as being low probability. With the continued deterioration of the pier substructure and the severe winter of 2013/2014, the ice loads specified in the HRPT Design Guide have now been included in the load combinations, greatly reducing the allowable live loads.

While the steel H-piles with a pile grade of Severe under the Pier Shed and are currently sufficient to support the current usage on the pier, the level of corrosion on the piles has reached a critical level. Any notable amount of additional section loss, especially at the tops of the piles, could possibly result in load restrictions on the pier.

The allowable uniform live load rating for the Court Yard is 150 psf, and is governed by the structural capacity of the concrete pile cap beams with exposed reinforcing. Although the extent of deterioration on the concrete beams varies throughout the entire Court Yard, the structure was conservatively rated based on the lowest load rating determined for the analyzed structural elements.

At the Finger Pier Extension, the steel H-piles are capable of supporting a uniform live load of 0 psf in their current condition, based upon the loading combinations outlined in the HRPT Design Guide. It is important to note that the allowable uniform live load rating of 0 psf is governed by the Ice Load outlined in Load Combination 8 and Load Combination 9. Since the controlling load is an ice load, it is recommended that access to the Finger Pier Extension be restricted anytime that the possibility for an ice load exists. When there is no possibility of ice load on the Finger Pier the allowable live load is 100 psf.

The Finger Pier Extension was not analyzed for fire truck loads due to its size, and limited access to large vehicular traffic. It should be noted that fire truck access on the apron around the Pier Shed is somewhat restricted due to gated entrances and limited apron deck space.

With the installation of a proper fender system, vessels similar in size to the Hornblower Infinity could also moor along the south and west sides of the pier, in addition to the currently occupied north side. Vessels significantly larger than the Hornblower Infinity would require additional mooring analyses to determine whether the piles are capable of resisting the associated lateral loads. Existing mooring line and mooring hardware capacities associated with the Hornblower Infinity were not evaluated.

5.2 RECOMMENDED REPAIR

A cost effective repair plan that addresses all areas of deterioration without the need for future phased repair efforts (aside from routine inspections and regular maintenance) was developed.

The controlling defect on the steel H-Piles is the corrosion at the tops of the piles, above the MHW line. An epoxy coating often obscures this corrosion, however, selective removal of the epoxy coating at Level II/III locations has confirmed that corrosion of the underlying steel H-Piles exists. As this corrosion extends to the very tops of the piles, to just below the concrete pile cap, the integrity of the load transfer/bearing between the piles and the pile caps can become compromised, and the associated repair must reestablish these connections, as well as strengthen the deteriorated areas of the piles.

Although the pile flanges were previously repaired/strengthened with C-Channels within the mid-pile zone, at approximately MLW, the H-piles webs were not repaired within this zone and are very thin. The H-pile webs should be strengthened as well. Therefore, it is recommended that reinforced concrete encasements, with structural connections to the pile caps, be installed. These encasements should extend down into the mudline.

At pile locations that do not require structural repairs to maintain the aforementioned minimum operational capacity, it is recommended that non-structural encasements be installed as a preventative measure to inhibit corrosion and preserve the pile in its existing condition. These non-structural repairs should extend from the tops of the piles and into the mudline. They would effectively replace the function of a cathodic protection system. It is recommended that these be epoxy grout encasements.

Since the recommended repairs are widespread, there are several obstacles to the swift and efficient rehabilitation of the pier. These obstacles include pier size, environmental obstacles, and lack of head room. Pier 40 is a large pier and repairs specified at the interior/middle piles will be significantly more labor intensive than repairs specified along the pier perimeter. Environmental obstacles include pouring encasements in the winter months, which is not possible/recommended. Finally, large areas of the pier, especially the Court Yard, have little to no headroom during some or all of the tide cycle. All of these factors contribute to the difficulty associated with a widespread and comprehensive repair effort and cause repair costs to increase.

5.3 RECOMMENDATIONS

Recommendations made in this report are grouped into the following three levels of importance. The definition of each level of importance is taken from the New York City Economic Development Corporation's (NYCEDC) Waterfront Facilities Maintenance Management System Inspection Guidelines Manual.

- **"Immediate"** actions are taken to prevent unsafe conditions and are intended to return structural capacity.

- **"Priority"** repairs do not require immediate action, are intended to maintain the structure in a safe operating condition and/or prevent deterioration from continuing to a point where the future repairs will be significantly more costly. Based on general assumptions of production rates and construction crew size, it is expected that Priority repairs will take approximately 6 years to complete.

- **"Routine"** actions are those to be undertaken as part of a scheduled maintenance program. Postponing recommended Routine Level actions will not compromise the structural integrity of the facility or significantly increase the cost to repair the structure. Based on general assumptions of production rates and construction crew size, it is expected that Routine repairs will take approximately 1.5 years to complete.

5.3.1 Repair Description

(A) IMMEDIATE ACTIONS

At the Finger Pier Extension, the steel H-piles are capable of supporting a uniform live load of 0 psf in their current condition, based upon the loading combinations outlined in the HRPT Design Guide. It is important to note that the allowable uniform live load rating of 0 psf is governed by the Ice Load outlined in Load Combination 8 and Load Combination 9. Since the controlling load is an ice load, it is recommended that access to the Finger Pier Extension be restricted anytime that the possibility for an ice load exists and that repairs to restore its structural capacity be implemented on an Immediate basis.

(B) PRIORITY ACTIONS

While the steel H-piles with a pile grade of Severe under the Pier Shed are currently sufficient to support the current usage on the pier, the level of corrosion on the piles has reached a critical level. Any notable amount of additional section loss, especially at the tops of the piles, could possibly result in load restrictions on the pier. Since the most severe corrosion is located at the tops of the piles, where nearly 25 percent of the H-piles (830 piles) have an epoxy coating that conceals this corrosion from view, it is difficult to determine when each pile would reach its critical load capacity. For this reason, and to better ensure uninterrupted use of the facility, it is recommended that all steel H-piles graded Severe and Major be repaired with structural concrete encasements on a Priority basis.

While Pier Shed piles graded Severe and Major are currently sufficient to support the current usage, non-structural encasement, which will "freeze" the pile in its existing condition, are not recommended. Structural encasements are recommended instead because these piles have little allowance for additional deterioration if the current load rating is required. Further, this load rating analysis is based on a Routine inspection, which is a general inspection intended to assess the general overall condition of the structure. If, in the future, a Repair Design Inspection is conducted, which is intended to record relevant attributes of each defect to be repaired, this additional information could be used to possible reduce the number of recommended structural encasements.

The existing cathodic protection system appears to be at the end of its useful life. Considering the data collected, and the high cost/risk of leaving an uncoated steel structure in an aggressive saltwater environment, it is recommended that any pile not recommended for encasement on an Immediate or Priority basis and with anodes having 5 years or less of estimated remaining life, be encased with a non-structural encasement on a Priority basis.

(C) ROUTINE ACTIONS

All steel H-Piles are in an active state of corrosion and are currently protected by a cathodic protection system that is likely at the end of its useful life. Further, this cathodic protection system does not protect the most aggressive zone of corrosion, the splash zone, located just above MHW. To keep future maintenance efforts to a minimum, it is recommended that all piles currently graded Moderate or Minor be repaired with a full-height protective encasement on a Routine basis.

The concrete pile cap beams with exposed steel reinforcing should be repaired by replacing the deteriorated steel reinforcing bars, including the stirrups, which are typically more heavily corroded than the longitudinal reinforcing bars. After the existing reinforcing is cleaned and replaced, with all unsound concrete removed a minimum of 1 in. behind the reinforcing, the beam cross section should be restored by a form and pour/pump concrete repair technique. It is recommended that all spalls with exposed reinforcement, all structural cracks, and all large cracks be repaired on a Routine basis.

It should be noted that concrete pile cap beams with fine cracks and delaminated soffits are currently not recommended for repair.

The precast prestressed deck planks with exposed prestressing strands should be repaired by cleaning the exposed strands and forming and pouring/pumping a concrete patch to protect the remaining strands. Analysis results of the precast planks indicate that a plank can support a uniform live load of 300 psf, and a fire truck wheel load assuming 60% section loss in the bottom layer of prestressing strands.

The exposed steel reinforcing on the deck soffit at the Finger Pier Extension should be repaired by replacing any broken reinforcing bars and forming and pouring/pumping a concrete patch.

It should be noted that delaminated concrete on the deck soffit is currently not recommended for repair.

Table 5-2 Cost Estimate for Pier 40 Recommended Repair

Repair Type	Structure	Number of Piles to Repair		Timber Fender System Replacement (LF)	Area of Beam and Deck to Repair (SF)	Cost Estimate	
		Structural Encasements	Non-Structural Encasements			Subtotal	TOTAL
Immediate	Pier Shed	--	--	--	--	--	$80.6M
	Court Yard	--	--	--	--	--	
	Finger Pier Ext.	110	--	--	--	$3.96M	
Priority	Pier Shed	1,671	676	--	--	54.41M	
	Court Yard	--	444	--	--	$6.63M	
	Finger Pier Ext.	--	4	--	--	$104,307	
Routine	Pier Shed	168	330	2,405	950	$13.49M	
	Court Yard	--	39	--	500	$1.12M	
	Finger Pier Ext.	--	21	--	280	$848k	
Escalation to Mid-Point of Construction – 7.5 Year Construction Period							
Escalation to 2016 (2 Year Design Period)		6.09%		$4.9M		$14.9M	
Escalation to 2020 (Mid-Point of Construction)		11.72%		$10M			
Total Project Construction Cost With Escalation				**$95.5M**			

Note: Owner's costs, not included in the base pricing, totals $9.1M. For detailed breakdown of costs please refer to Appendix C.

The strengths of this repair recommendation are that it is comprehensive and addresses all notable defects. The resulting repaired structure should be relatively low maintenance, sufficient to support the current use requirements for the foreseeable future, and any unknown deficiencies that exist now will likely be discovered and remedied during such a widespread rehabilitation effort.

5.3.2 Miscellaneous Repairs

Although these repairs will not affect the structural capacity of the pier, it is recommended that these repairs be completed to ensure public safety.

The concrete closure wall, between the deck planks of the Court Yard and the Pier Shed, has several areas of severe deterioration with through spalls/voids and large width cracks. It is recommended that these severe defects be repaired on a Routine basis to prevent access to the pier substructure from the Court Yard.

5.3.3 Additional Investigation

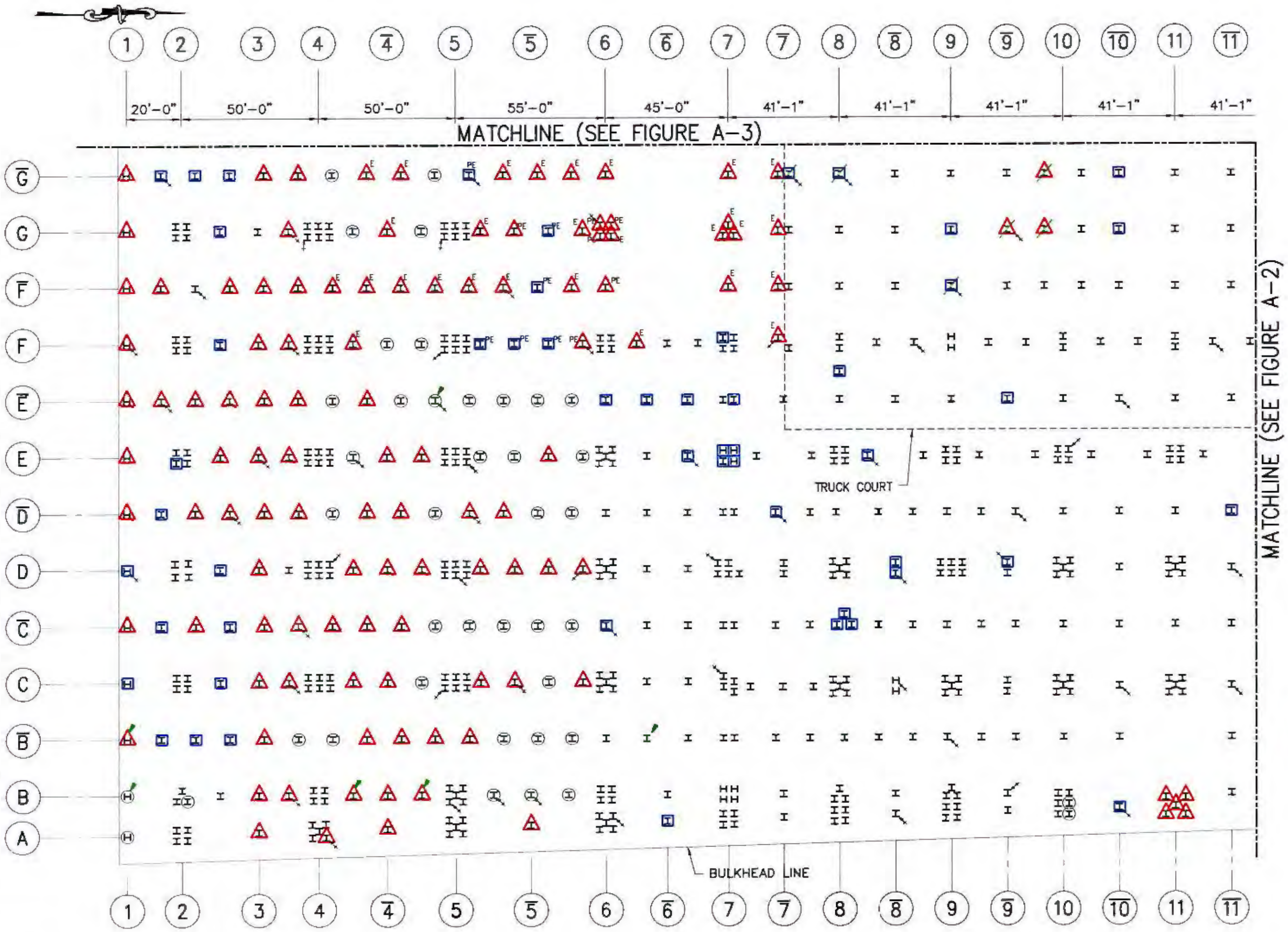
As per the American Society of Civil Engineers Underwater Inspections Guidelines Manual, steel pile supported structures in **Poor** condition that are located in aggressive environments should be inspected every 4 years.

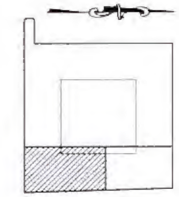
5.4 COST ESTIMATE

The estimated cost for the recommended repair is presented in Appendix C. Included in the cost estimate, but not shown in Table 5-2, are typical owner costs estimated at approximately $9.1 million which would be expended during both the design and engineering phase and during construction. Owner costs include items such as design services, construction administration, diving and controlled inspections. The order-of-magnitude total for both contractor and owner costs is therefore estimated at approximately $104.6 million.

APPENDIX A

FIGURES





KEY PLAN
N.T.S.

LEGEND

- STEEL H-PILE RATED MINOR OR MODERATE
- STEEL H-PILE RATED MAJOR
- STEEL H-PILE RATED SEVERE
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR ABOVE THE CHANNEL REPAIR
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR WITHOUT A CHANNEL REPAIR (NUMBER AT BOTTOM LEFT CORNER INDICATES THE DISTANCE IN FEET BETWEEN THE BOTTOM OF THE ENCASEMENT AND THE MUDLINE
- STEEL H-PILE WITH A STEEL PLATE AND EPOXY COATING REPAIR
- STEEL H-PILE WITH AN EPOXY COATING REPAIR
- STEEL H-PILE WITHOUT AN ANODE
- STEEL H-PILE WITHOUT A CHANNEL REPAIR
- LEVEL II INSPECTION LOCATION
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED MAJOR
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED SEVERE

1"=40' 0 40 80 FT.

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PARTIAL PILE PLAN
SOUTH END OF BENTS A TO G-BAR

FIG A-1

Halcrow

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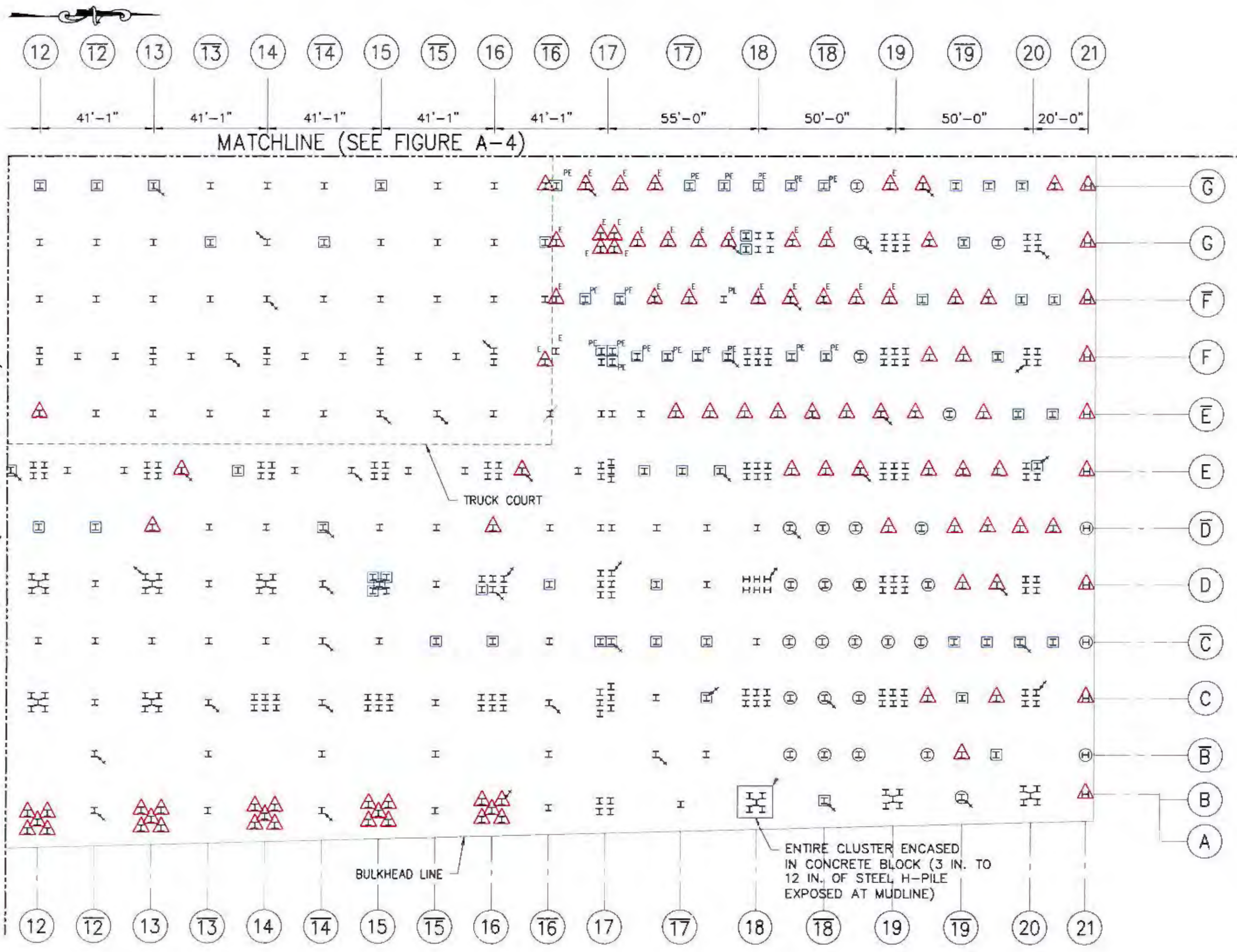

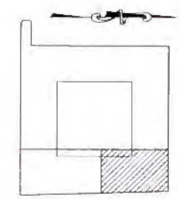

KEY PLAN
N.T.S.

LEGEND

- STEEL H-PILE RATED MINOR OR MODERATE
- STEEL H-PILE RATED MAJOR
- STEEL H-PILE RATED SEVERE
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR ABOVE THE CHANNEL REPAIR
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR WITHOUT A CHANNEL REPAIR (NUMBER AT BOTTOM LEFT CORNER INDICATES THE DISTANCE IN FEET BETWEEN THE BOTTOM OF THE ENCASEMENT AND THE MUDLINE
- STEEL H-PILE WITH A STEEL PLATE AND EPOXY COATING REPAIR
- STEEL H-PILE WITH AN EPOXY COATING REPAIR
- STEEL H-PILE WITHOUT AN ANODE
- STEEL H-PILE WITHOUT A CHANNEL REPAIR
- LEVEL II INSPECTION LOCATION
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED MAJOR
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED SEVERE

0 40 80 FT.
1"=40'

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PARTIAL PILE PLAN
NORTH END OF BENTS A TO G-BAR

FIG A-2

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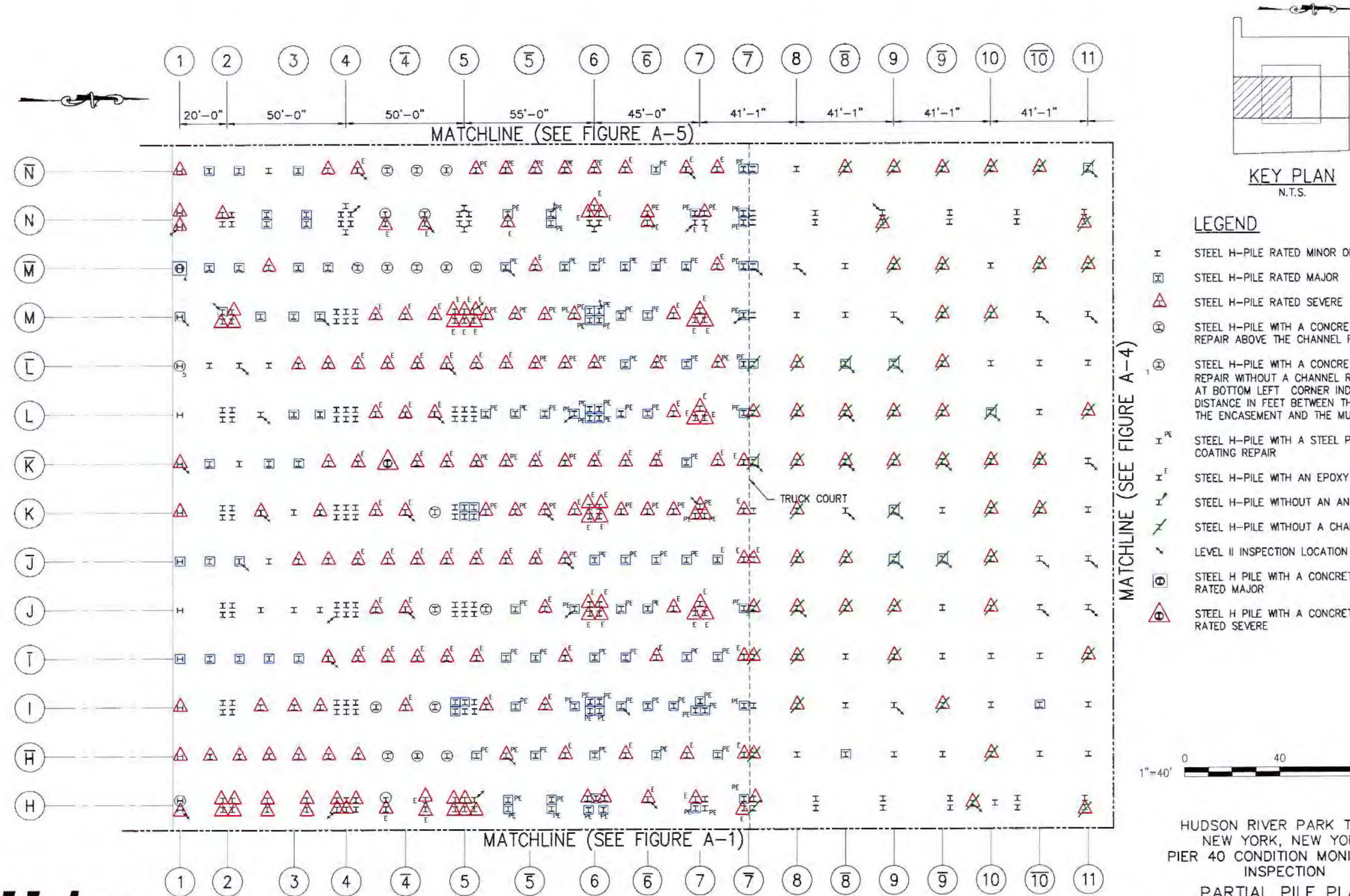

LEGEND

- STEEL H-PILE RATED MINOR OR MODERATE
- STEEL H-PILE RATED MAJOR
- STEEL H-PILE RATED SEVERE
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR ABOVE THE CHANNEL REPAIR
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR WITHOUT A CHANNEL REPAIR (NUMBER AT BOTTOM LEFT CORNER INDICATES THE DISTANCE IN FEET BETWEEN THE BOTTOM OF THE ENCASEMENT AND THE MUDLINE
- STEEL H-PILE WITH A STEEL PLATE AND EPOXY COATING REPAIR
- STEEL H-PILE WITH AN EPOXY COATING REPAIR
- STEEL H-PILE WITHOUT AN ANODE
- STEEL H-PILE WITHOUT A CHANNEL REPAIR
- LEVEL II INSPECTION LOCATION
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED MAJOR
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED SEVERE

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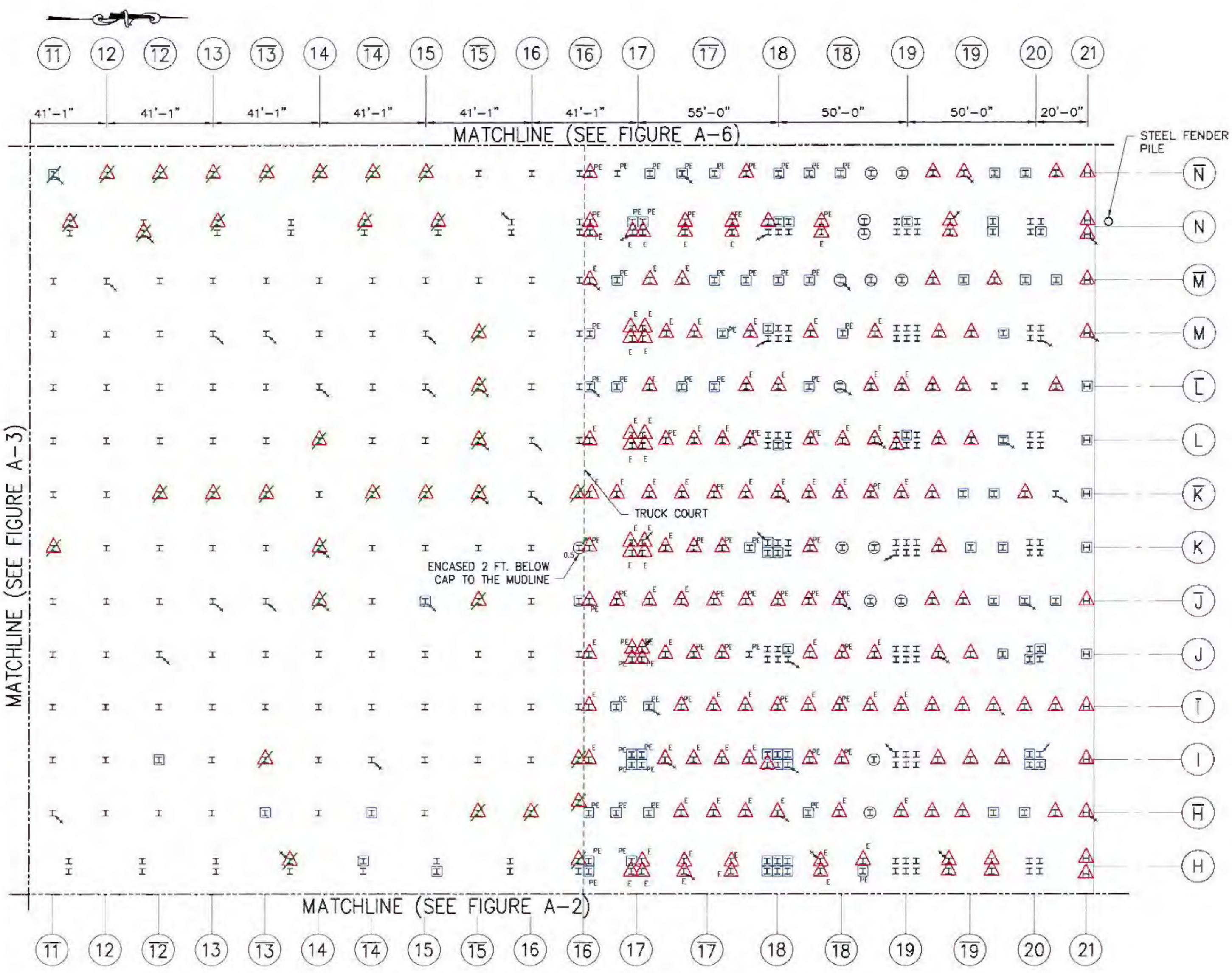
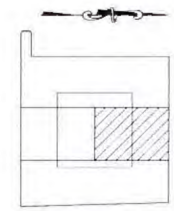
PARTIAL PILE PLAN
BENTS H TO N-BAR

FIG A-3





KEY PLAN
N.T.S.

LEGEND

- STEEL H-PILE RATED MINOR OR MODERATE
- STEEL H-PILE RATED MAJOR
- STEEL H-PILE RATED SEVERE
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR ABOVE THE CHANNEL REPAIR
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR WITHOUT A CHANNEL REPAIR (NUMBER AT BOTTOM LEFT CORNER INDICATES THE DISTANCE IN FEET BETWEEN THE BOTTOM OF THE ENCASEMENT AND THE MUDLINE
- STEEL H-PILE WITH A STEEL PLATE AND EPOXY COATING REPAIR
- STEEL H-PILE WITH AN EPOXY COATING REPAIR
- STEEL H-PILE WITHOUT AN ANODE
- STEEL H-PILE WITHOUT A CHANNEL REPAIR
- LEVEL II INSPECTION LOCATION
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED MAJOR
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED SEVERE

1"=40' 0 40 80 FT.

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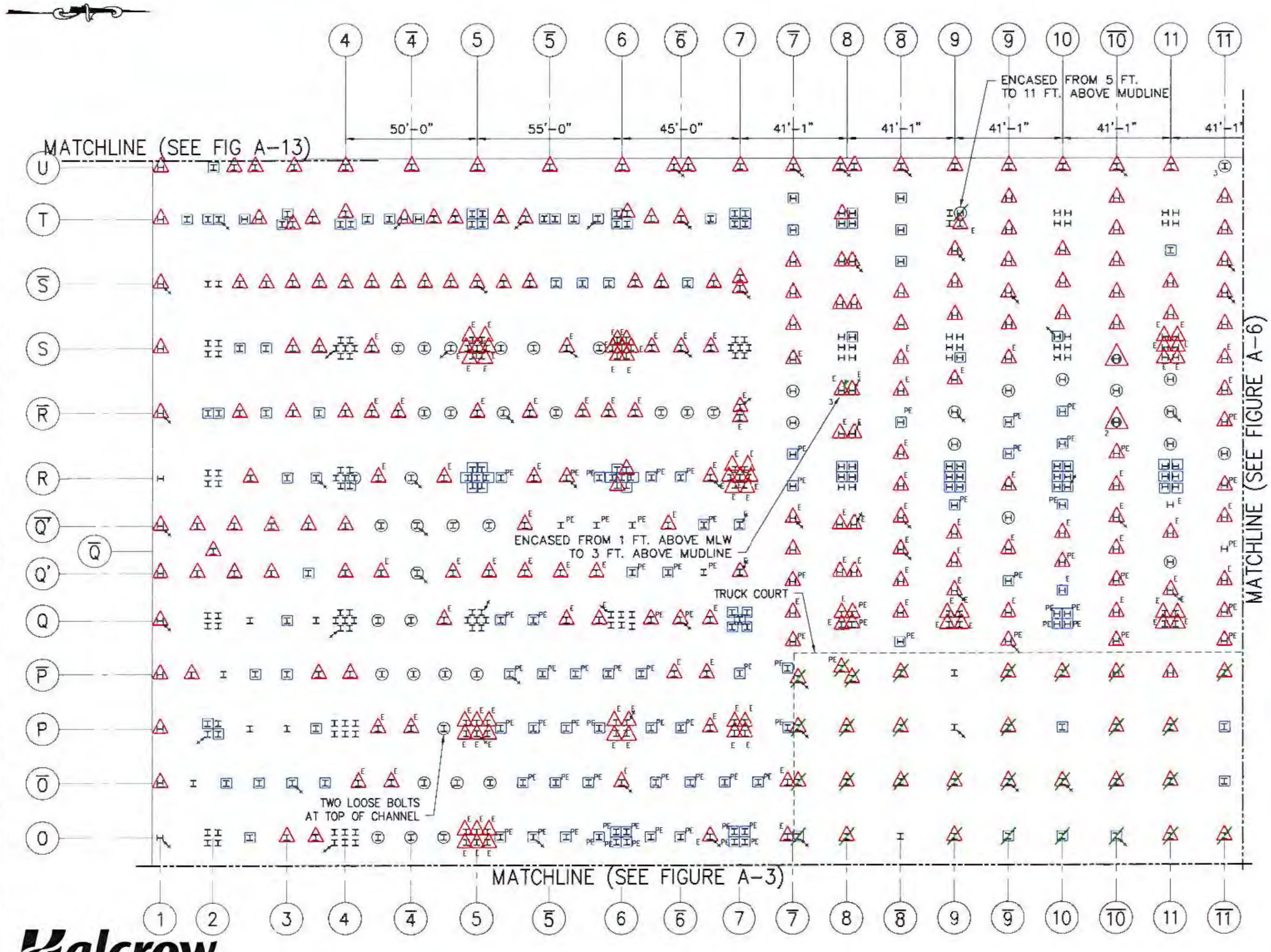
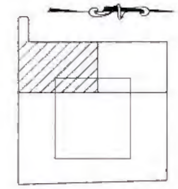
PIER 40 CONDITION MONITORING INSPECTION
PARTIAL PILE PLAN
NORTH END OF BENTS H TO N-BAR
FIG A-4





KEY PLAN

N.T.S.

LEGEND

- STEEL H-PILE RATED MINOR OR MODERATE
- STEEL H-PILE RATED MAJOR
- STEEL H-PILE RATED SEVERE
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR ABOVE THE CHANNEL REPAIR
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR WITHOUT A CHANNEL REPAIR (NUMBER AT BOTTOM LEFT CORNER INDICATES THE DISTANCE IN FEET BETWEEN THE BOTTOM OF THE ENCASEMENT AND THE MUDLINE
- STEEL H-PILE WITH A STEEL PLATE AND EPOXY COATING REPAIR
- STEEL H-PILE WITH AN EPOXY COATING REPAIR
- STEEL H-PILE WITHOUT AN ANODE
- STEEL H-PILE WITHOUT A CHANNEL REPAIR
- LEVEL II INSPECTION LOCATION
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED MAJOR
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED SEVERE

1"=40' 0 40 80 FT.

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PIER 40 CONDITION MONITORING INSPECTION

PARTIAL PILE PLAN
SOUTH END OF BENTS O TO Z
FIG A-5

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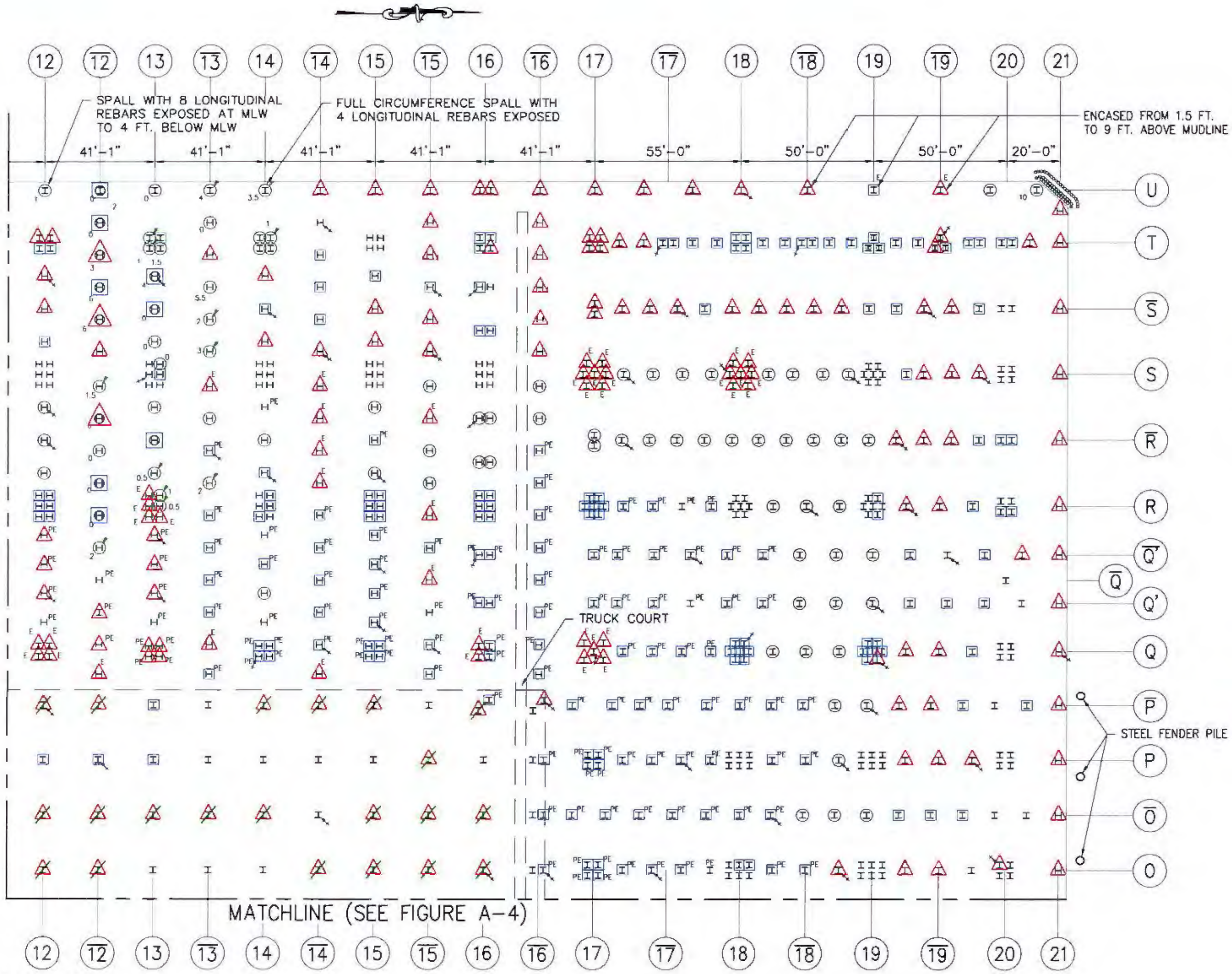

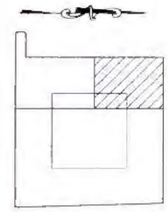

KEY PLAN
N.T.S.

LEGEND

- STEEL H-PILE RATED MINOR OR MODERATE
- STEEL H-PILE RATED MAJOR
- STEEL H-PILE RATED SEVERE
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR ABOVE THE CHANNEL REPAIR
- STEEL H-PILE WITH A CONCRETE ENCASEMENT REPAIR WITHOUT A CHANNEL REPAIR (NUMBER AT BOTTOM LEFT CORNER INDICATES THE DISTANCE IN FEET BETWEEN THE BOTTOM OF THE ENCASEMENT AND THE MUDLINE
- STEEL H-PILE WITH A STEEL PLATE AND EPOXY COATING REPAIR
- STEEL H-PILE WITH AN EPOXY COATING REPAIR
- STEEL H-PILE WITHOUT AN ANODE
- STEEL H-PILE WITHOUT A CHANNEL REPAIR
- LEVEL II INSPECTION LOCATION
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED MAJOR
- STEEL H PILE WITH A CONCRETE ENCASEMENT RATED SEVERE

1"=40' 0 40 80 FT.

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PARTIAL PILE PLAN
NORTH END OF BENTS O TO Z

FIG A-6

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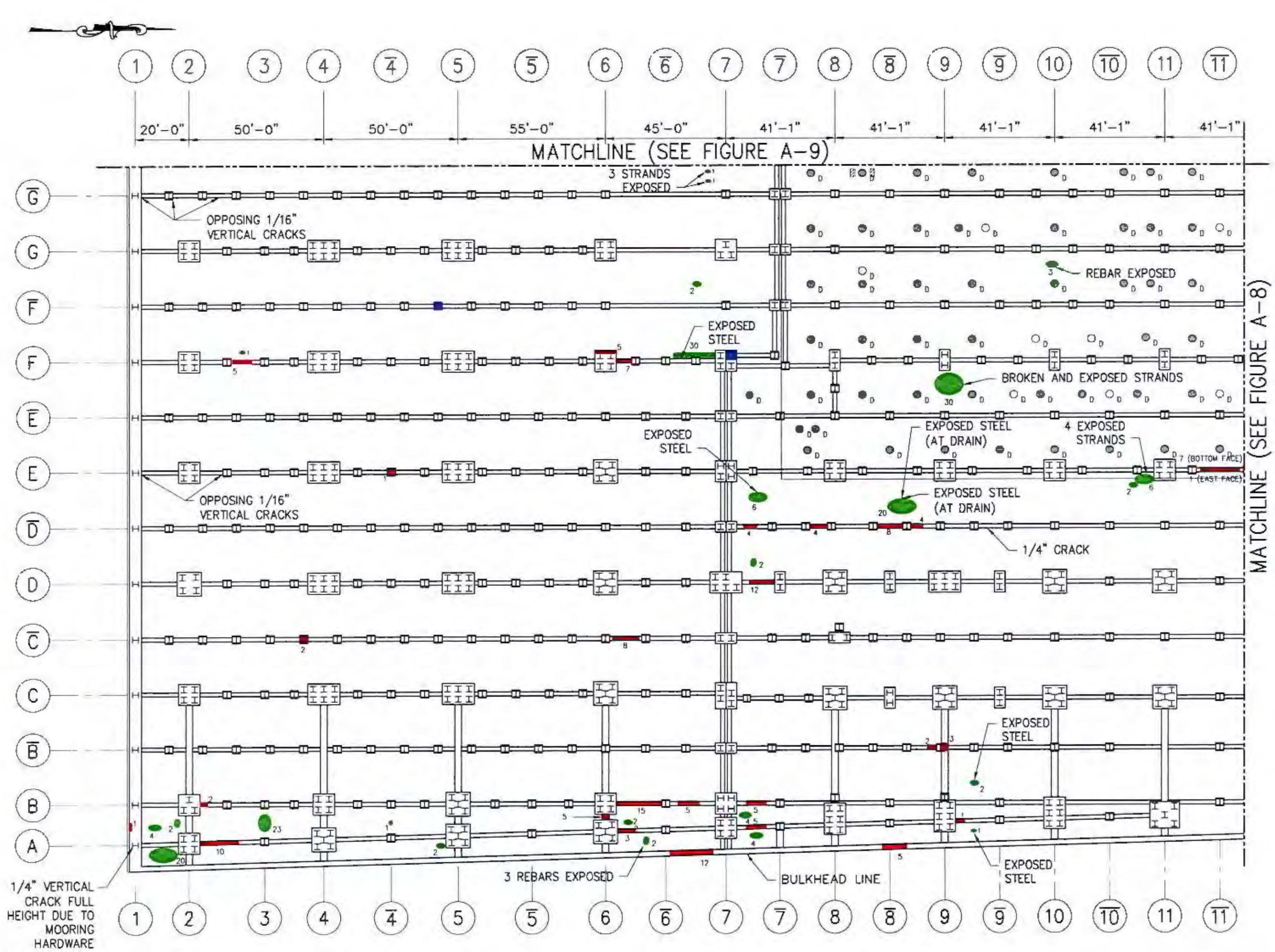

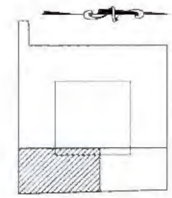

KEY PLAN

N.T.S.

LEGEND

- DECK SLAB DRAIN THAT APPEARS TO BE FUNCTIONING
- DECK SLAB DRAIN THAT IS NONFUNCTIONAL OR FUNCTIONALITY CANNOT BE DETERMINED
- 36" x 18" GRATING
- PILE CAP OR BEAM SPALL WITH REBARS EXPOSED (APPROXIMATE LOCTION) - NO. INDICATES THE AREA OF THE SPALL IN SQUARE FEET
- DECK PANEL SPALL (APPROXIMATE LOCATION) NO. INDICATES THE AREA OF THE SPALL IN SQUARE FEET
- PILE CAP SPALL 6 IN. DEEP AND GREATER WITH NO REBARS EXPOSED

NOTE:

1. PILE CAPS AND BEAMS ARE NOT SHOWN TO SCALE
2. SPALLS WITHOUT REBARS EXPOSED, DELAMINATION AND CRACKS ON PILE CAPS, PILE CAP BEAMS, AND DECK PANELS ARE NOT SHOWN UNLESS OTHERWISE NOTED.

1"=40' 0 40 80 FT.

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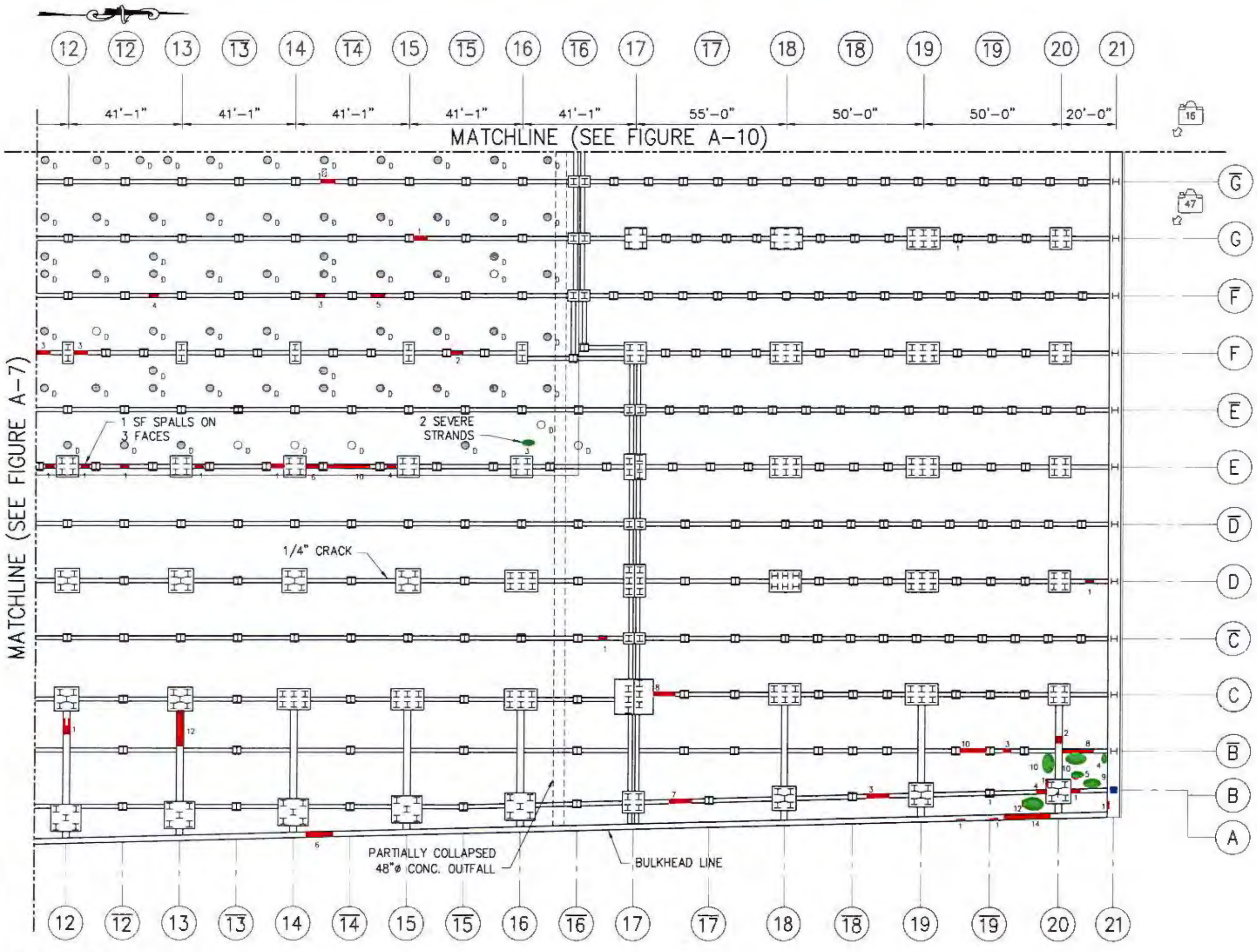
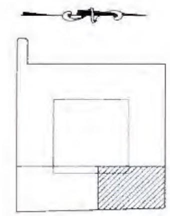
PARTIAL UNDERDECK PLAN
SOUTH END OF BENTS A TO G-BAR
FIG A-7





LEGEND

- STEEL H-PILE IN MINOR OR MODERATE CONDITION
- STEEL H-PILE IN MAJOR CONDITION
- STEEL H-PILE IN SEVERE CONDITION
- STEEL H-PILE WITH CONCRETE BAG REPAIR
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED SEVERE
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED MAJOR
- SPALLS WITH REBARS EXPOSED
- AREAS WITH RUST STAINING DUE TO SHALLOW CONCRETE COVER
- LEVEL II INSPECTION LOCATION

NOTE:

1. PILE CAPS AND BEAMS ARE NOT SHOWN TO SCALE
2. SPALLS WITHOUT REBARS EXPOSED, DELAMINATION AND CRACKS ON PILE CAPS, PILE CAP BEAMS, AND DECK PANELS ARE NOT SHOWN UNLESS OTHERWISE NOTED.

1"=40' 0 40 80 FT.

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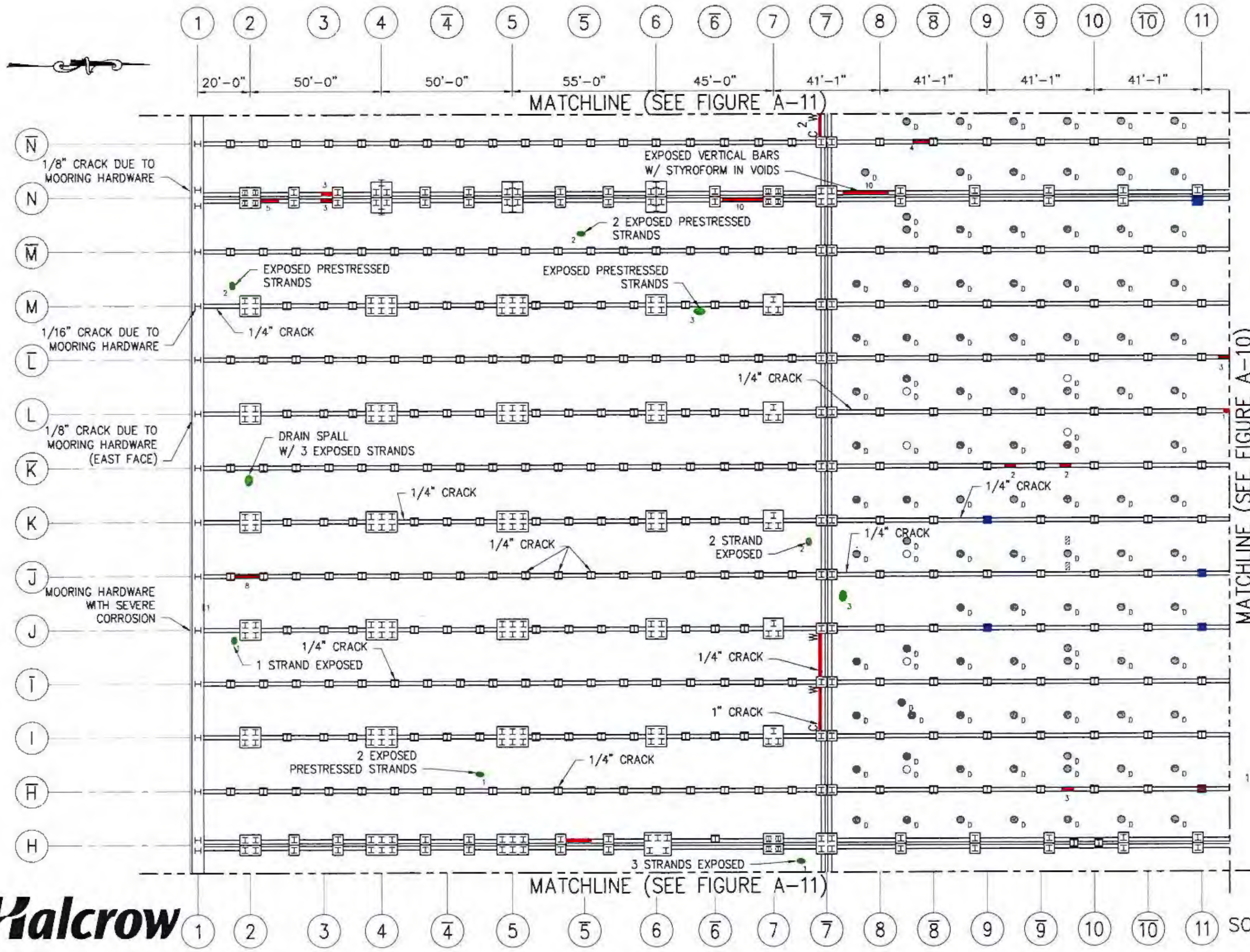
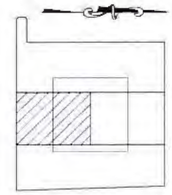
PIER 40 CONDITION MONITORING INSPECTION

PARTIAL UNDERDECK PLAN
NORTH END OF BENTS A TO G-BAR

FIG A-8





KEY PLAN

N.T.S.

LEGEND

- STEEL H-PILE IN MINOR OR MODERATE CONDITION
- STEEL H-PILE IN MAJOR CONDITION
- STEEL H-PILE IN SEVERE CONDITION
- STEEL H-PILE WITH CONCRETE BAG REPAIR
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED SEVERE
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED MAJOR
- SPALLS WITH REBARS EXPOSED
- AREAS WITH RUST STAINING DUE TO SHALLOW CONCRETE COVER
- LEVEL II INSPECTION LOCATION

NOTE:

1. PILE CAPS AND BEAMS ARE NOT SHOWN TO SCALE
2. SPALLS WITHOUT REBARS EXPOSED, DELAMINATION AND CRACKS ON PILE CAPS, PILE CAP BEAMS, AND DECK PANELS ARE NOT SHOWN UNLESS OTHERWISE NOTED.

0 40 80 FT.

1"=40'

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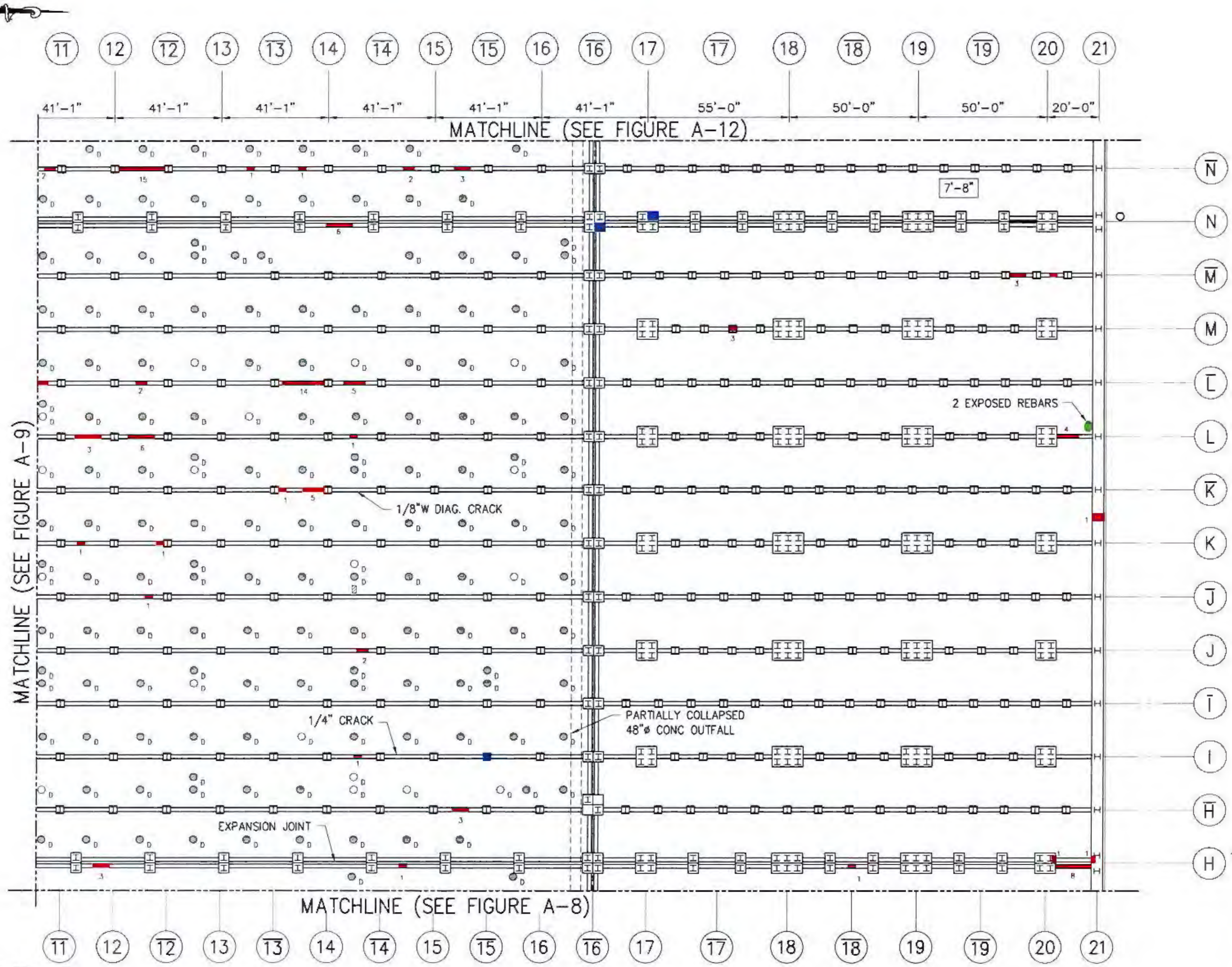
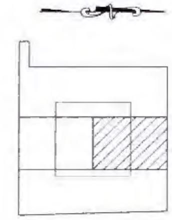
PIER 40 CONDITION MONITORING INSPECTION

PARTIAL UNDERDECK PLAN
SOUTH END OF BENTS H TO N-BAR

FIG A-9





KEY PLAN
N.T.S.

LEGEND

- STEEL H-PILE IN MINOR OR MODERATE CONDITION
- STEEL H-PILE IN MAJOR CONDITION
- STEEL H-PILE IN SEVERE CONDITION
- STEEL H-PILE WITH CONCRETE BAG REPAIR
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED SEVERE
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED MAJOR
- SPALLS WITH REBARS EXPOSED
- AREAS WITH RUST STAINING DUE TO SHALLOW CONCRETE COVER
- LEVEL II INSPECTION LOCATION

NOTE:

1. PILE CAPS AND BEAMS ARE NOT SHOWN TO SCALE
2. SPALLS WITHOUT REBARS EXPOSED, DELAMINATION AND CRACKS ON PILE CAPS, PILE CAP BEAMS, AND DECK PANELS ARE NOT SHOWN UNLESS OTHERWISE NOTED.

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PIER 40 CONDITION MONITORING INSPECTION

PARTIAL UNDERDECK PLAN
NORTH END OF BENTS H TO N-BAR
FIG A-10

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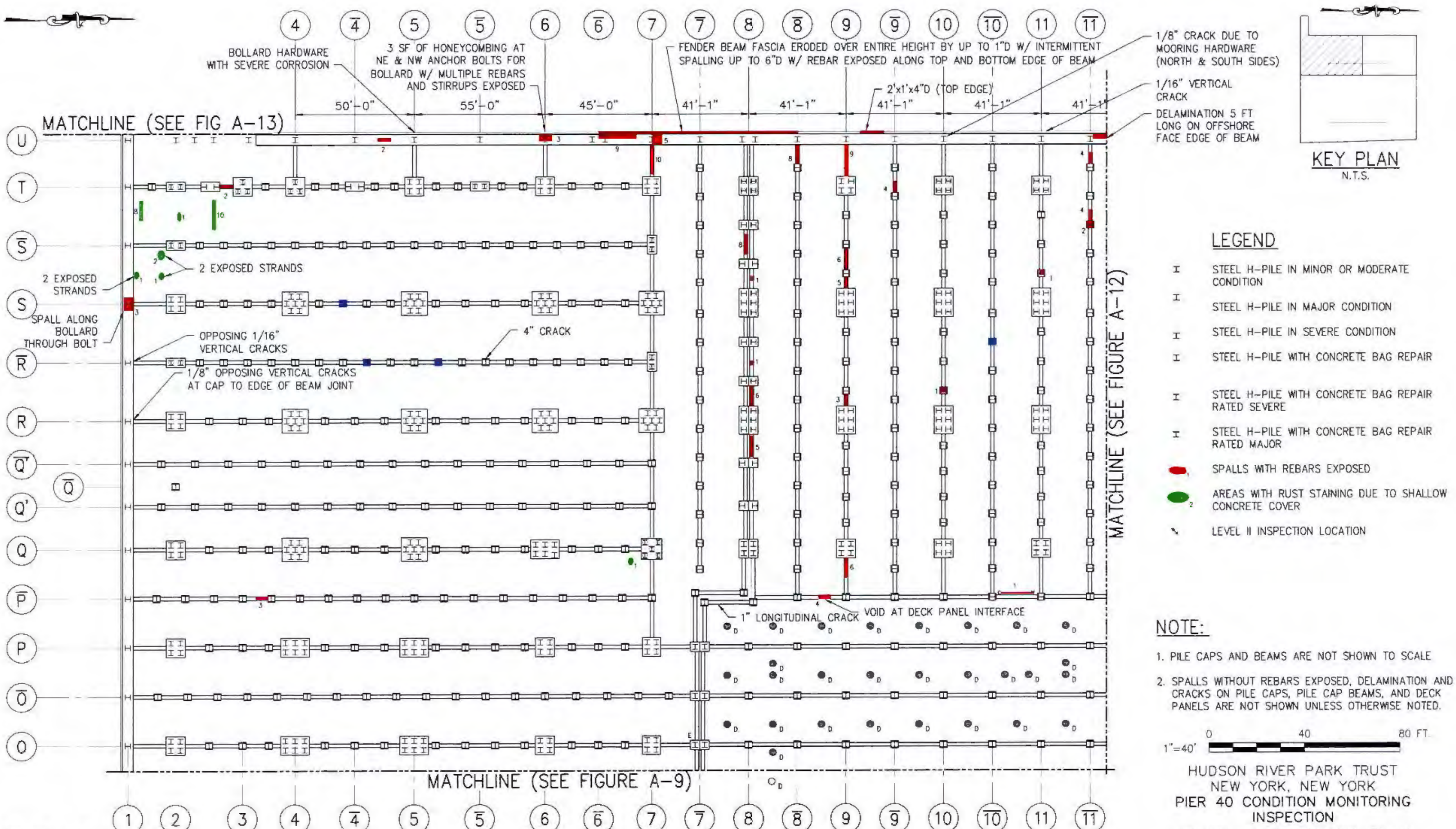

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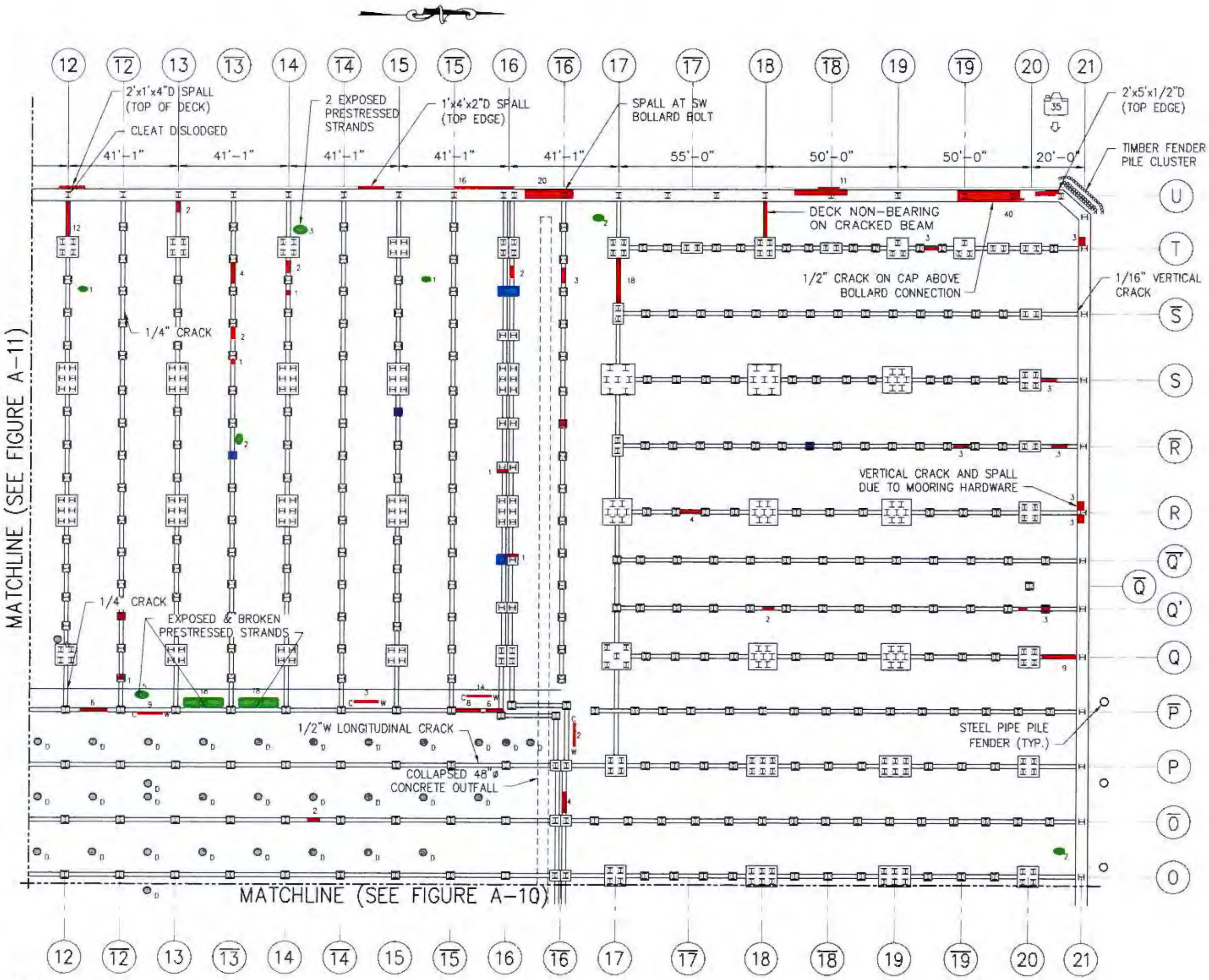
PIER 40 CONDITION MONITORING INSPECTION

PARTIAL UNDERDECK PLAN
SOUTH END OF BENTS O TO Z
FIG A-11



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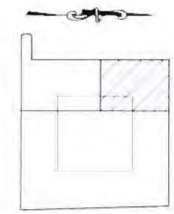

KEY PLAN
N.T.S.

LEGEND

- STEEL H-PILE IN MINOR OR MODERATE CONDITION
- STEEL H-PILE IN MAJOR CONDITION
- STEEL H-PILE IN SEVERE CONDITION
- STEEL H-PILE WITH CONCRETE BAG REPAIR
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED SEVERE
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED MAJOR
- SPALLS WITH REBARS EXPOSED
- AREAS WITH RUST STAINING DUE TO SHALLOW CONCRETE COVER
- LEVEL II INSPECTION LOCATION

NOTE:

1. PILE CAPS AND BEAMS ARE NOT SHOWN TO SCALE
2. SPALLS WITHOUT REBARS EXPOSED, DELAMINATION AND CRACKS ON PILE CAPS, PILE CAP BEAMS, AND DECK PANELS ARE NOT SHOWN UNLESS OTHERWISE NOTED.

0 40 80 FT.
1"=40'

HUDSON RIVER PARK TRUST
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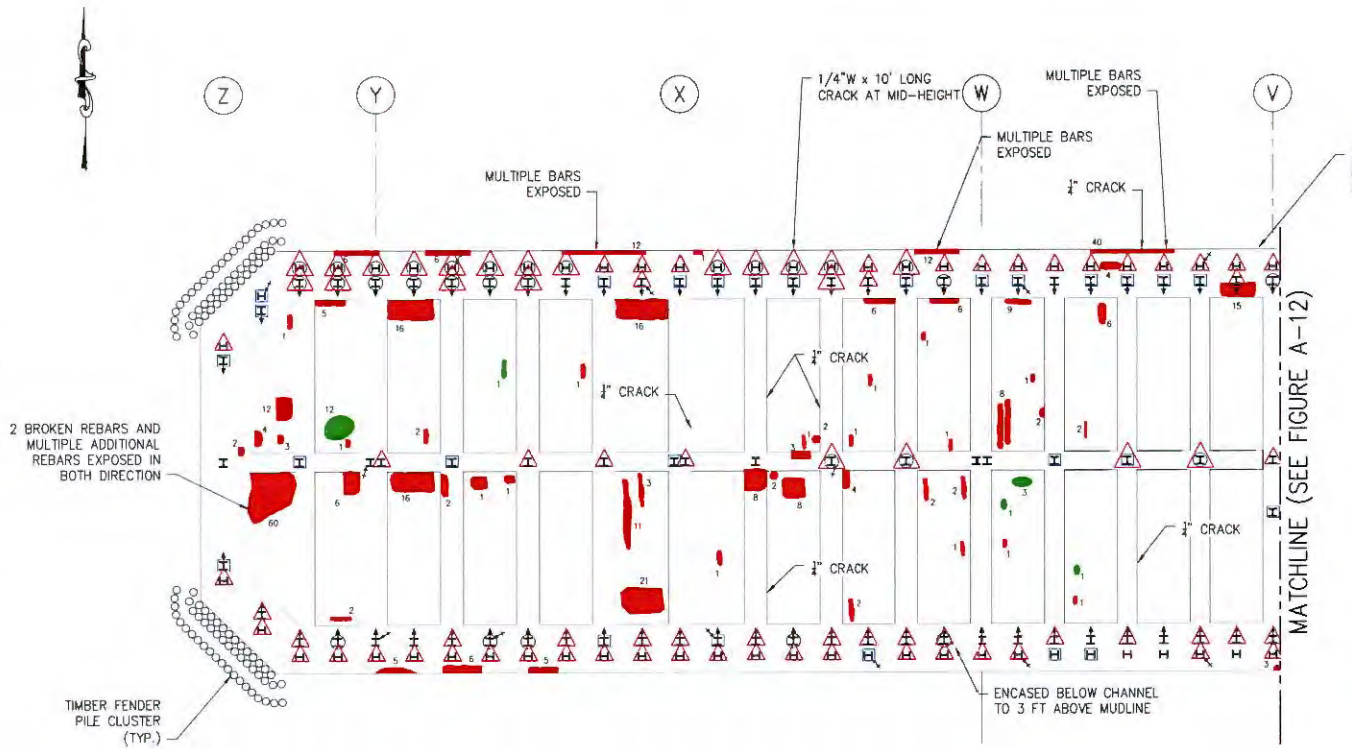
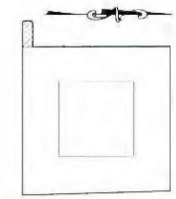
PIER 40 CONDITION MONITORING INSPECTION
PARTIAL UNDERDECK PLAN
NORTH END OF BENTS O TO Z
FIG A-12





KEY PLAN

N.T.S.

LEGEND

- STEEL H-PILE IN MINOR OR MODERATE CONDITION
- STEEL H-PILE IN MAJOR CONDITION
- STEEL H-PILE IN SEVERE CONDITION
- STEEL H-PILE WITH CONCRETE BAG REPAIR
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED SEVERE
- STEEL H-PILE WITH CONCRETE BAG REPAIR RATED MAJOR
- SPALLS WITH REBARS EXPOSED
- AREAS WITH RUST STAINING DUE TO SHALLOW CONCRETE COVER
- LEVEL II INSPECTION LOCATION

1/16"=1'-0" 0 10 20 30 FT.

HUDSON RIVER PARK TRUST
NEW YORK, NEW YORK
PIER 40 CONDITION MONITORING INSPECTION

PILE AND UNDERDECK PLAN
FINGER PIER EXTENSION

FIG A-13

Halcrow

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APPENDIX B

PHOTOGRAPHS

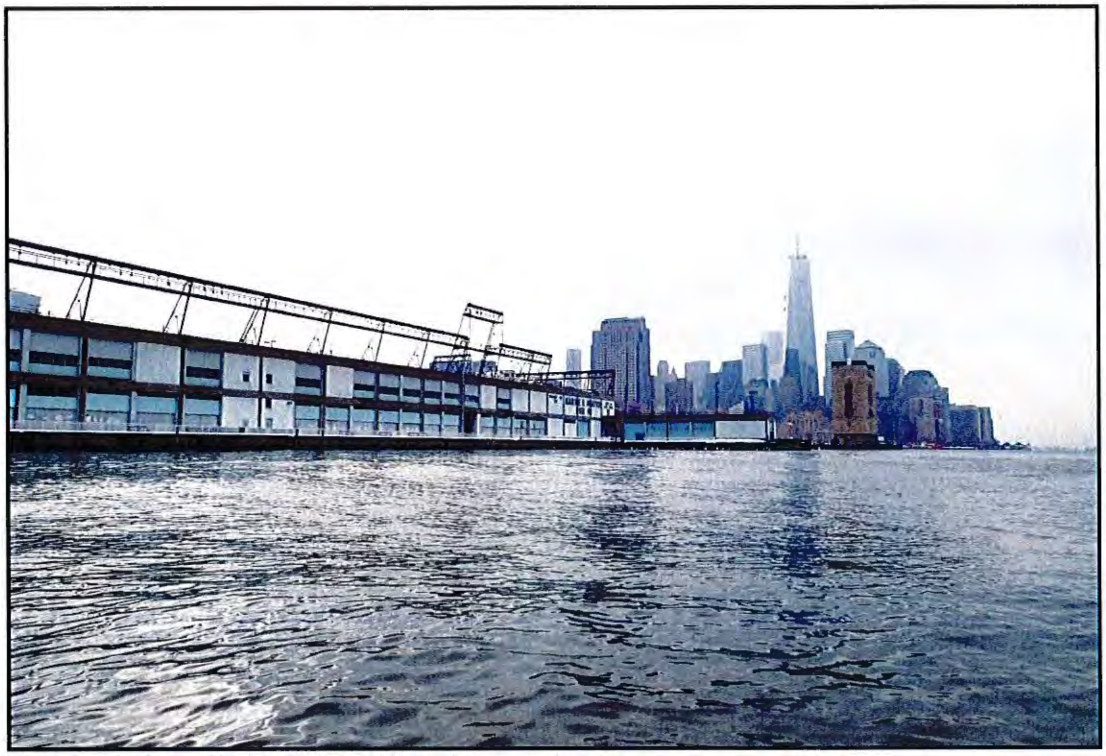


Photo B-1: Overall view of Pier 40, looking southeast.

Photo B-2: Overall view of the Finger Pier Extension with one-story shed, looking southeast.

Photo B-3: View of typical Pier Shed substructure, looking north at Bent F-bar.

Photo B-4: Typical Court Yard substructure. Note water seeping through deck planks at icicle location.

Photo B-5: View of typical Finger Pier Extension substructure, looking south at the southern edge beam of the pier.

Photo B-6: View of north side of Pier 40 looking southwest. Note Hornblower vessels berthed at the northwest corner of the pier.

Photo B-7: View of north side of Pier 40 looking southeast. Note the timber fender system along the edge beam and additional floating fenders.

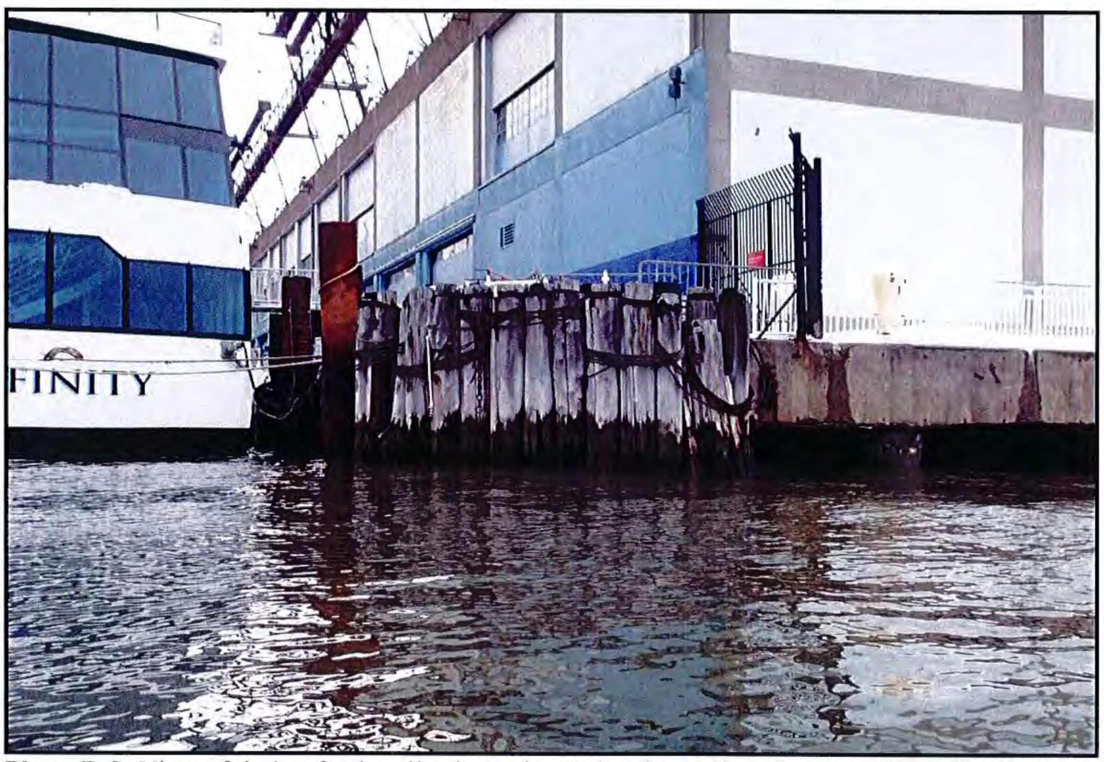


Photo B-8: View of timber fender pile cluster located at the northwest corner of the Pier Shed. Note steel fender piles with full coverage coating loss.

Photo B-9: Typical concrete encasement at the top of a steel H-pile in Bent F-bar (foreground) and typical epoxy coating at top of steel H-pile in Bent F-bar (background).

Photo B-10: Close-up of typical concrete encasement at the top of a steel H-pile at Bent S Pile Row 4.7.

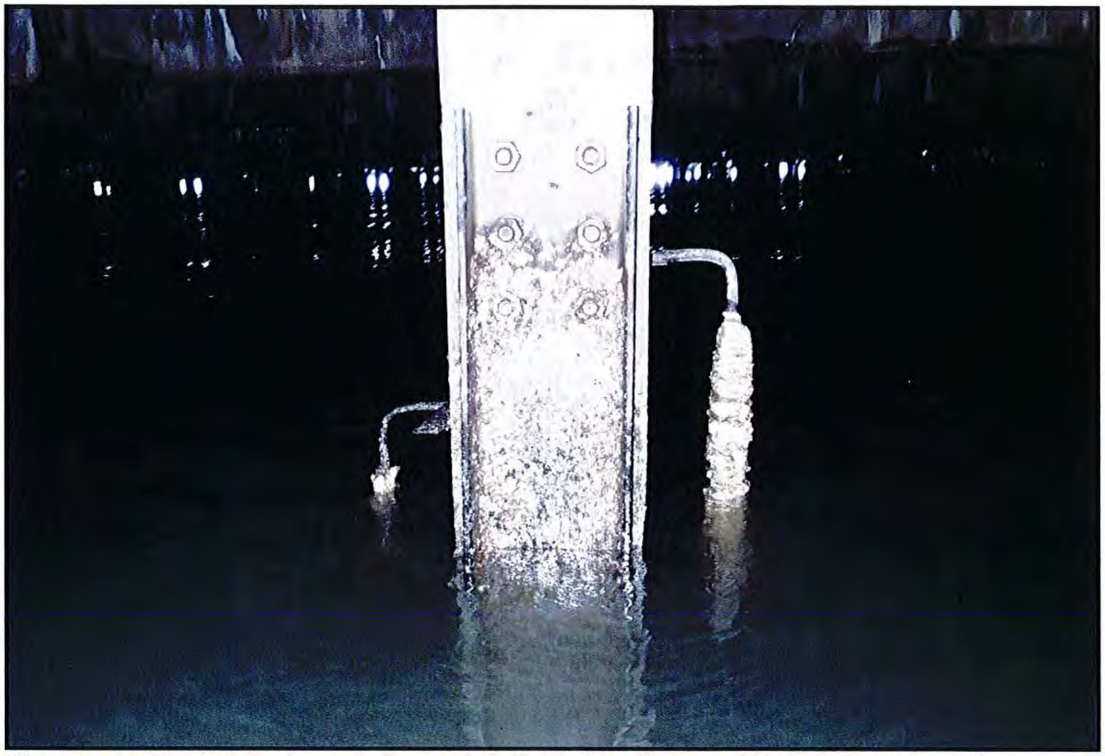


Photo B-11: Typical C-channel pile repair bolted to the flanges of H-pile I-bar 9-bar, view looking west. Note the depleted anodes attached to each side of the H-pile.

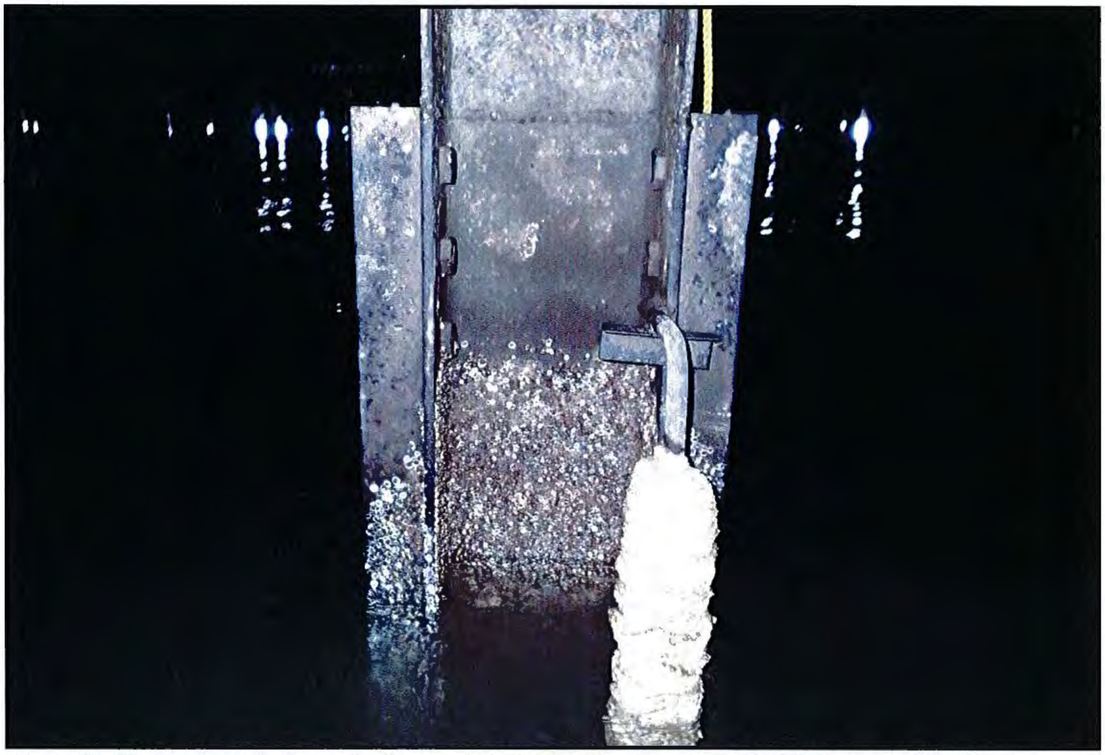


Photo B-12: Typical C-channel pile repair bolted to the flanges of H-pile I-bar 9-bar, view looking south.

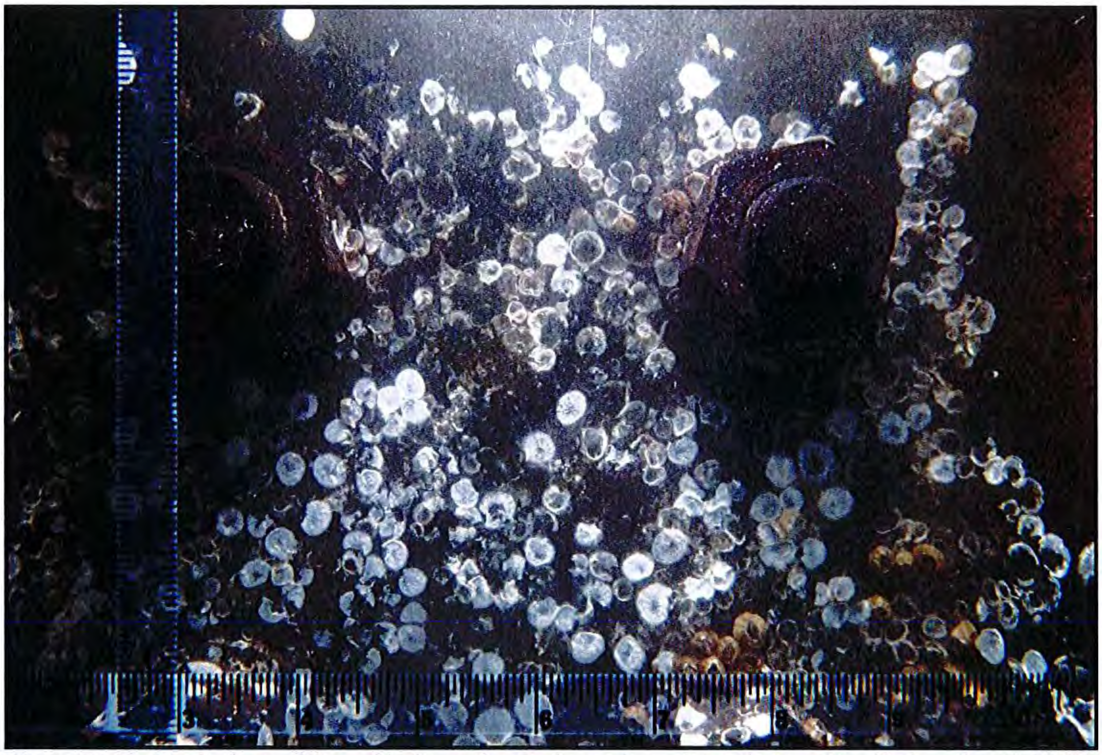


Photo B-13: Typical condition of bolts at the C-channel repairs. Photo at H-pile location T 18.75 in the Pier Shed.

Photo B-14: Typical rust staining and delamination of epoxy coating at Bent Q.25, Pile Row 16S. Note,corrosion of the H-pile flange has caused a large crack in the concrete pile cap.

Photo B-15: Typical rust staining and delamination of epoxy coating at Bent R.75, Pile Row 13-bar. Note, corrosion of the flange has caused a large spall in the concrete pile cap.

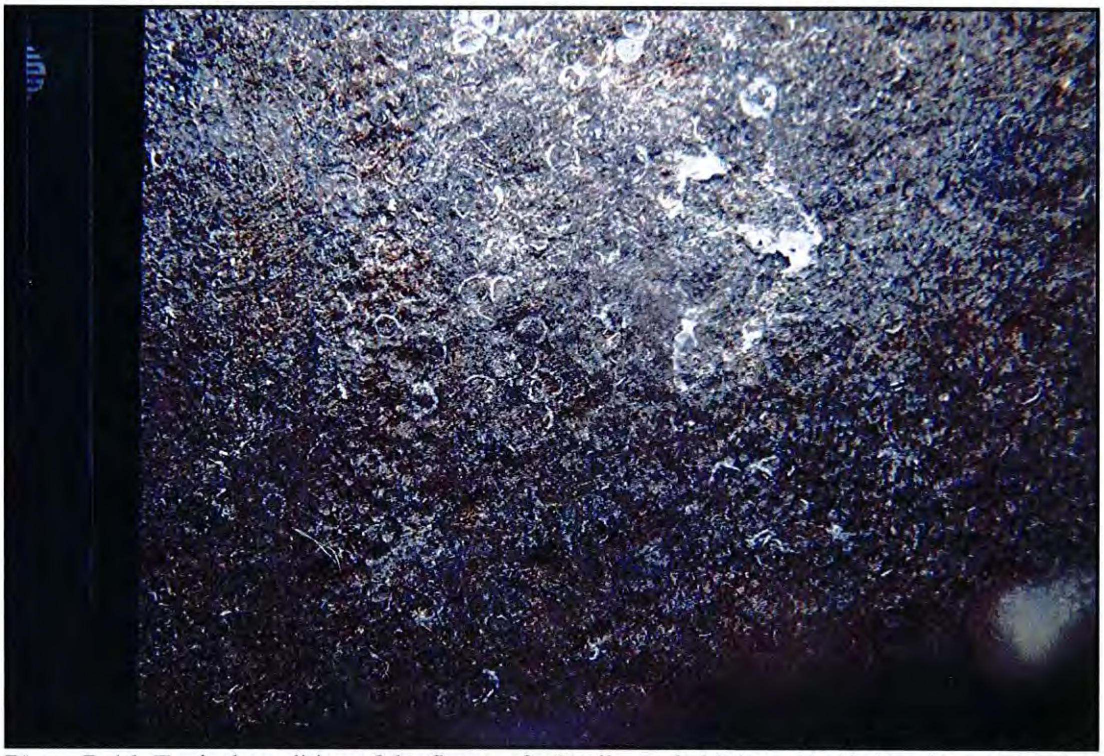


Photo B-16: Typical condition of the flange of a H-pile graded Minor. H-pile location at Bent S-bar and Pile Row 19.

Photo B-17: Typical condition of the flange edge of a H-pile graded Minor. H-pile location at Bent S-bar and Pile Row 19.

Photo B-18: Typical condition of the flange edge of a H-pile graded Moderate. H-pile location at Bent T and Pile Row 21.

Photo B-19: Typical condition of the flange edge and flange face of a H-pile graded Major. H-pile location is the center-west H-pile of the pile cluster at Bent L and Pile Row 19.

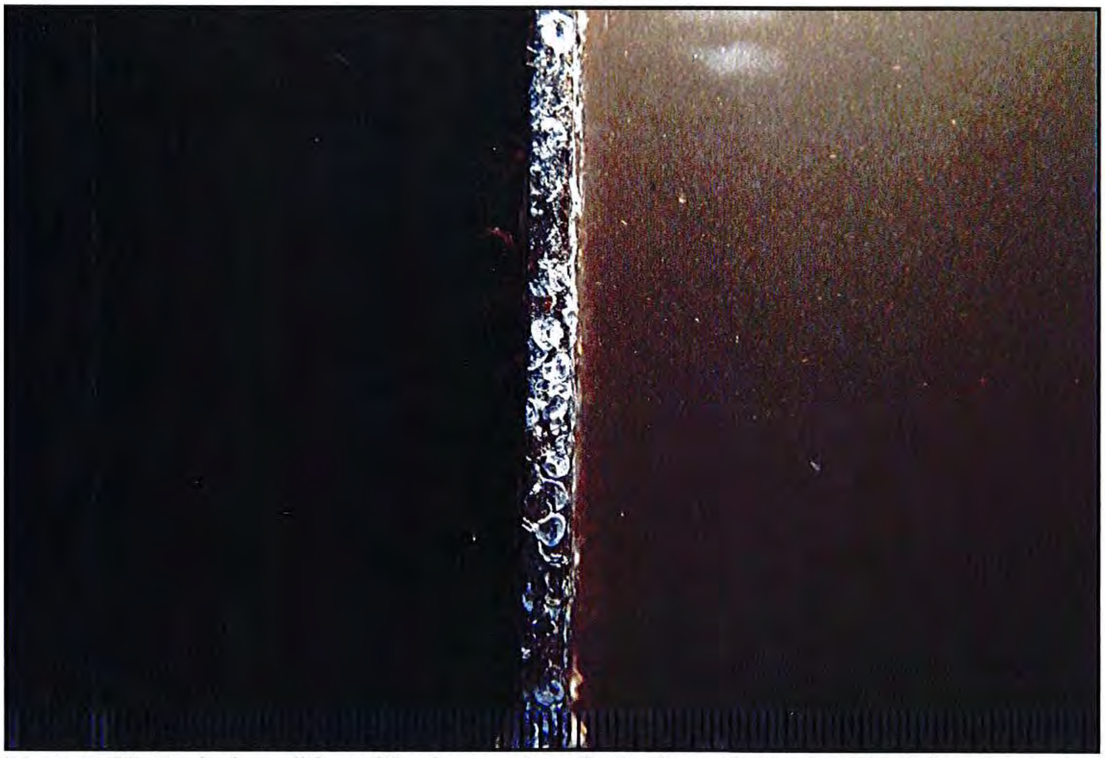


Photo B-20: Typical condition of the flange edge of a H-pile graded Major. H-pile location is the center-west H-pile of the pile cluster at Bent L and Pile Row 19.

Photo B-21: Typical condition of the flange edge of a H-pile graded Severe. H-pile location at Bent K-bar and Pile Row 16-bar South.

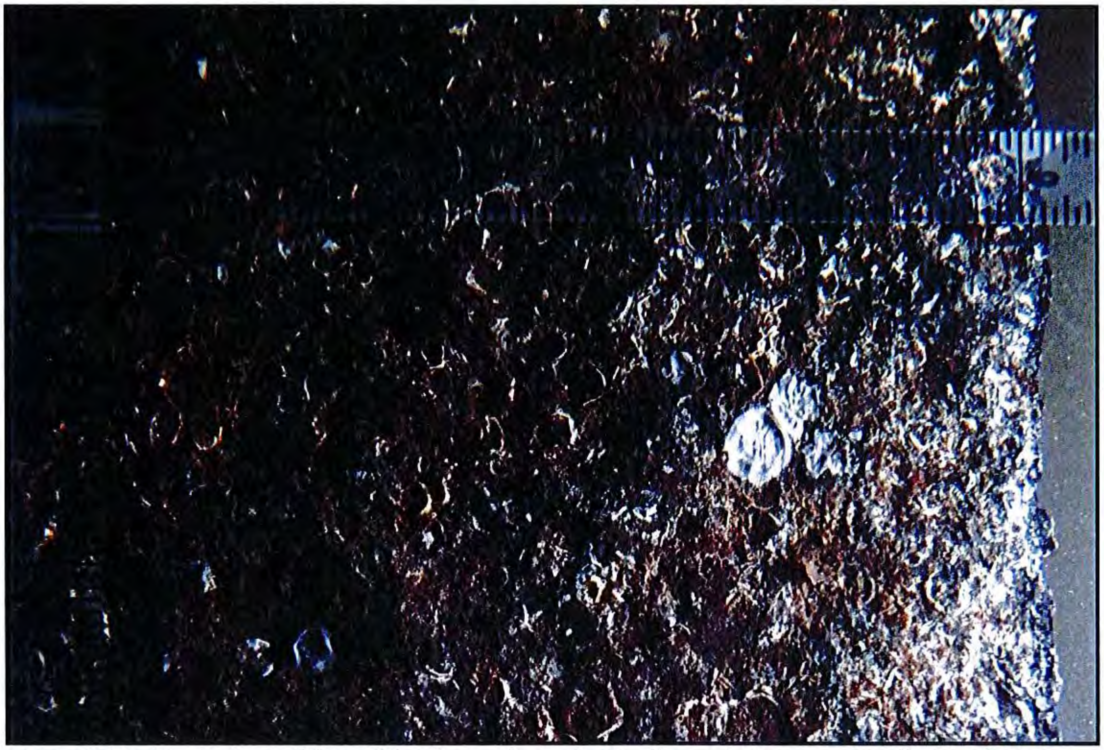


Photo B-22: Typical condition of the flange face of a H-pile graded Severe. H-pile location at Bent K-bar and Pile Row 16-bar South.

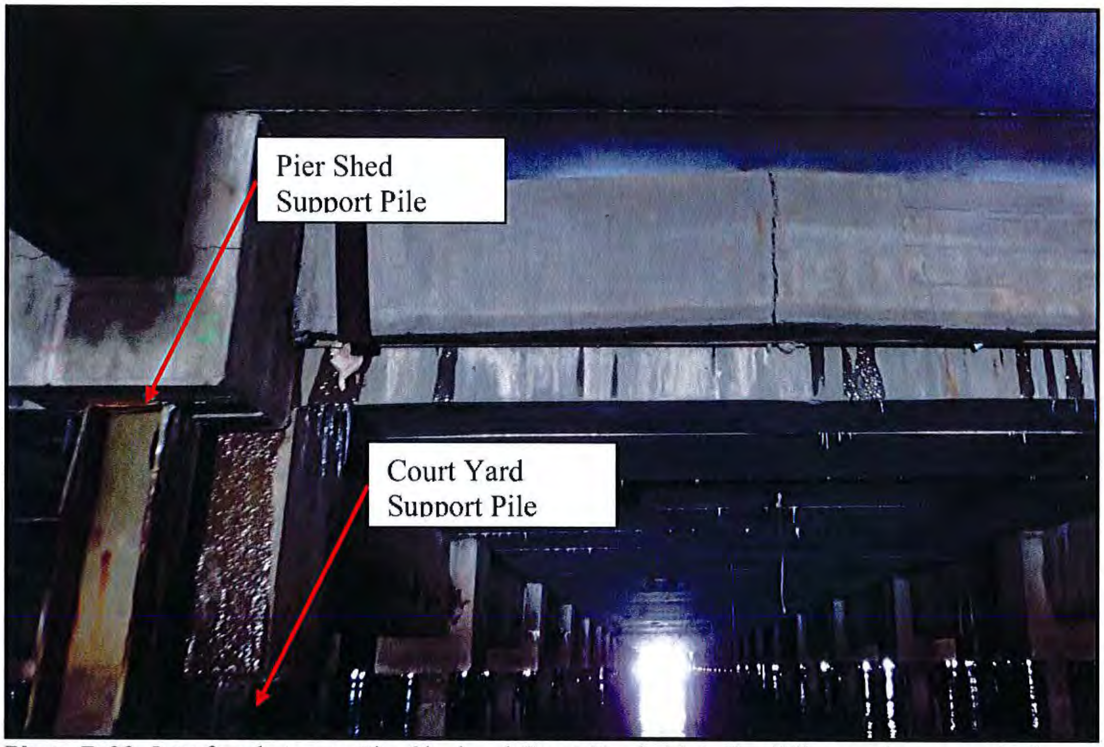


Photo B-23: Interface between Pier Shed and Court Yard. Note the difference in exposed pile length.

Photo B-24: Typical condition of concrete pile cap in the Court Yard. View looking west at Bent I-bar, Pile Row 9-bar.

Photo B-25: Typical condition of concrete beam in the Court Yard with efflorescence and light cracking. View looking west at Bent I-bar, between Pile Row 9 and 9-bar.

Photo B-26: Typical condition of concrete beam in the Pier Shed with light cracking and light rust staining. View looking north at Bent C, between Pile Row 17 and 17.3.

Photo B-27: Concrete beam in the Pier Shed with a full length spall and exposed reinforcement. View looking west at Bent T.5 and Pile Row 8-bar.

Photo B-28: Concrete beam in the Pier Shed with a full length spall and exposed reinforcement. View looking west at Bent D-bar between Pile Row 8.5 and 8-bar.


Photo B-29: Concrete closure wall with large vertical crack and deflection View looking north at Pile Row 7-bar between Bent I and I-bar.


Photo B-30: Concrete closeure wall with large vertical crack and deflection View looking east at Pile Row 7-bar between Bent I and I-bar.

Photo B-31: Erosion up to 1 in. deep and exposed reinforcing along the full height of the western edge beam between Pile Rows 7 and 8-bar.

Photo B-32: Rust staining and delamination of the soffit of the first transverse beam north of Bent W at the Finger Pier Extension.

Photo B-33: Typical condition of Court Yard deck soffit with light shrinkage cracks. View looking west between Bent I and I-bar at Pile Row 9-bar.

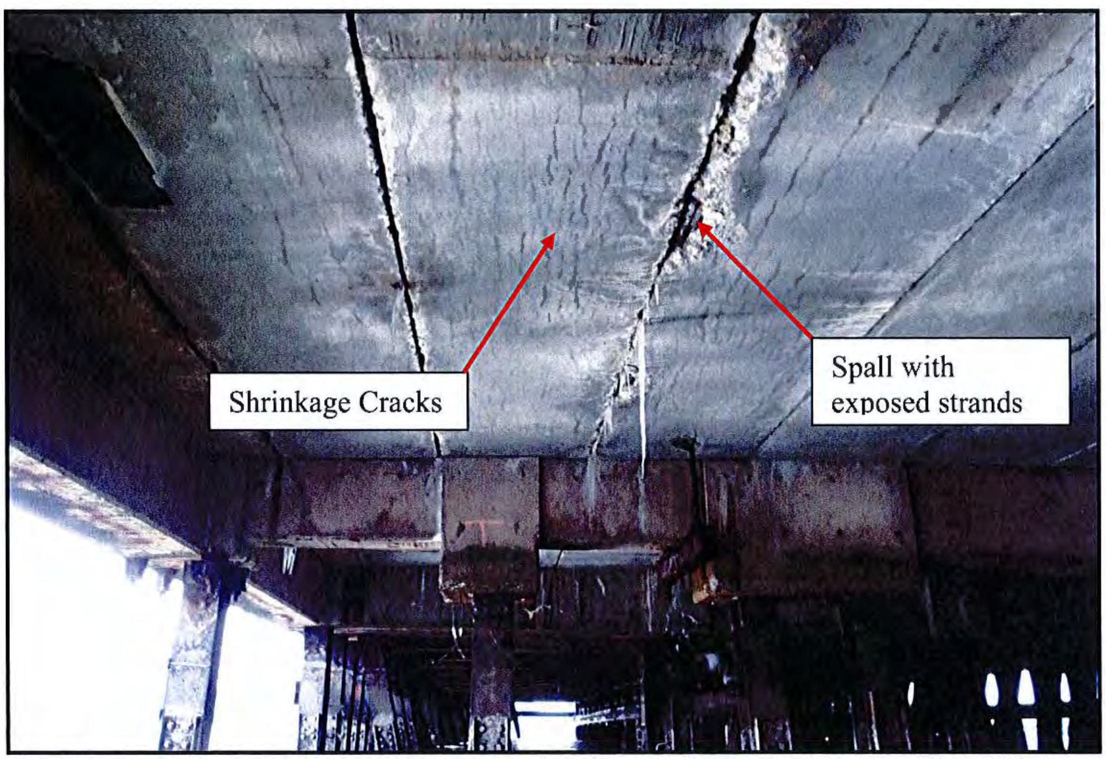


Photo B-34: Typical condition of Pier Shed deck soffit with light effloresense and light shrinkage cracks. Also note the spall in the deck plank with exposed strands.

Photo B-35: Large spall at Pier Shed deck soffit with exposed reinforcement. View looking north between Bents A and B at Pile Row 3.

Photo B-36: Precast deck plank located east of Pile 9 in Bent F with multiple exposed and broken prestressed strands.

Photo B-37: Spall along the north cap of the Finger Pier Extension between Bent Z and Bent Z.2. View looking north.

Photo B-38: Spall along the deck soffit of the Finger Pier Extension between Bent X.7 and Bent Y adjacent to the north cap. View looking north.

Photo B-39: Fender system along the north side of the Pier. Note the two steel fender piles located north of the timber fender system.

Photo B-40: Pile cluster at the north corner of the Finger Pier Extension. Note the severe damage at the MLW elevation and the loose wire rope wraps at the top of the cluster.

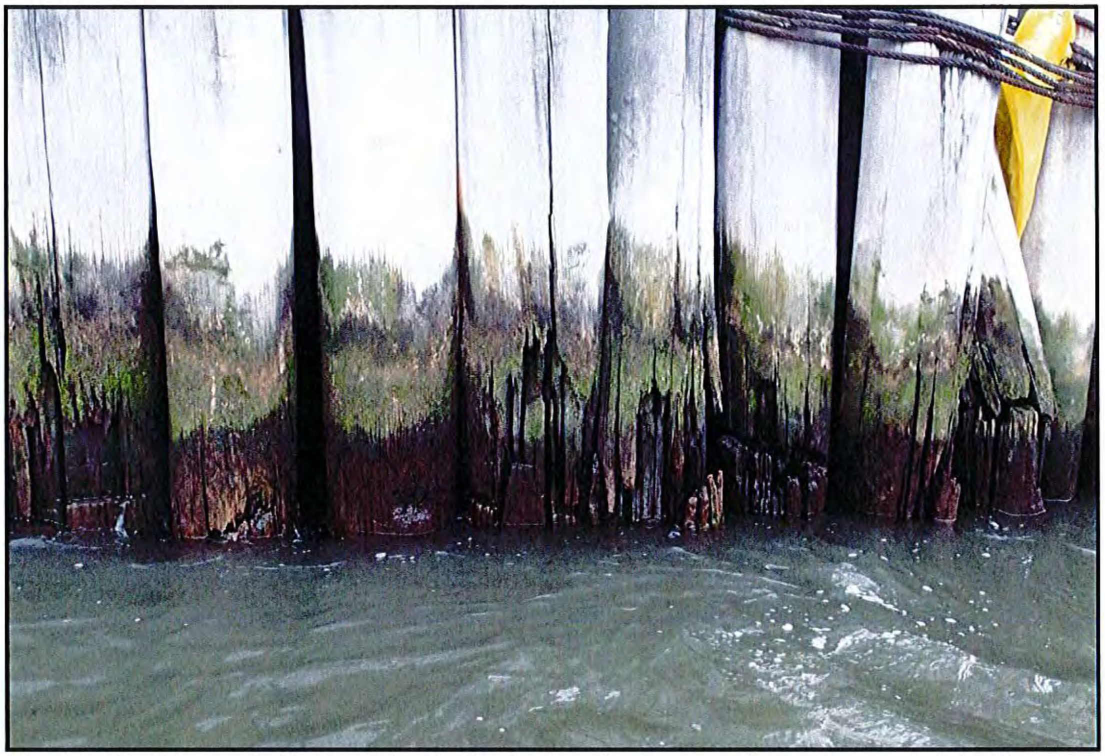


Photo B-41: Close-up of broken piles in the fender pile cluster at the north corner of the Finger Pier Extension.

APPENDIX C

COST ESTIMATE BREAKDOWN

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

CH2MHILL
Job No.: 483869.AD.01
13-Mar-15

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Repair Program				80,566,503
Immediate Repairs				3,961,988
Priority Repairs				61,148,531
Routine Repairs				15,455,984
		Sub-Total		**80,566,503**
		General Conditions	8.0%	Included
		Mobilization	8.0%	Included
		Overhead	10.0%	Included
		Profit	10.0%	Included
		Design Contingency	10.0%	Included
		Construction Contingency	10.0%	Included
		Bonding and Insurance - Contractor to Provide	4.3%	Included
		Repair Program		*80,566,503*
		Structural Encasement Unit Cost per Linear Foot		**1,525**
		Non-Structural Encasement Unit Cost per Linear Foot		**966**
		Highest Unit Rate Used for Budgetary Purposes		

Escalation to Mid Point of Construction - 7.5 Year Construction Period

Repair Program		*80,566,503*
Escalations to 2016 start (2 year Design Period)	6.09%	*4,906,500*
Repair Program Cost at Construction Start		*85,473,003*
Escalations to 2020 Midpoint of Construction (3.75 years)	11.72%	*10,019,341*
Total Project Costs With Escalations*		**95,492,344**

***Assumes work commences in 2 years and then proceeds for 7.5 years**
Rates Based on Expected Construction Cost Index

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

CH2MHILL
Job No.: 483869.AD.01
13-Mar-15

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Owner's Costs Not Included in Base Pricing				
Total Owner's Costs With Escalations				**9,095,642**
Design Costs Including Design Survey Inspections			*1.75%*	1,409,914
Client Contract Administration Costs (Including Escalation)			*0.75%*	728,726
Construction Manager Costs (Including Escalation)			*4.00%*	3,819,694
Inspections - Diving and Controlled (Including Escalation)			*2.50%*	2,387,309
Special Inspections (Including Escalation)			*0.93%*	750,000
Total Cost with Escalation and Owners Costs				**104,587,986**

Note:
Accuracy of Estimate Should be Considered to be +/- 25%

Markups Included or Specified	
General Conditions	8.0%
Mobilization	8.0%
Overhead	10.0%
Profit	10.0%
Design Contingency	10.0%
Construction Contingency	10.0%
Escalation to 2020 (Mid-Point of Construction)	16.8%

Exclusions and / or Clarifications
Permitting and Regulatory Approvals
Mitigation Requirements
Financing or Cost of Money

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

CH2MHILL
Job No.: 483869.AD.01
13-Mar-15

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Repair Program				80,566,503
Immediate Repairs				3,961,988
Finger Pier Extension				3,961,988
Structural Concrete Encasement at Steel H-Piles	3,300	ft		3,961,988
Priority Repairs				61,148,531
Pier Shed				54,414,432
Structural Concrete Encasement at Steel H-Piles	28,407	ft		43,315,904
Non-Structural Epoxy Grout Encasement at Steel H-Piles	11,492	ft		11,098,528
Truck Court				6,629,792
Non-Structural Epoxy Grout Encasement at Steel H-Piles	6,660	ft		6,629,792
Finger Pier Extension				104,307
Non-Structural Epoxy Grout Encasement at Steel H-Piles	120	ft		104,307
Routine Repairs				15,455,984
Pier Shed				10,793,288
Structural Concrete Encasement at Steel H-Piles	2,856	ft		4,355,946
Non-Structural Epoxy Grout Encasement at Steel H-Piles	5,610	ft		5,417,920
Concrete Spall Repairs - Beam and Deck	950	sq ft		1,019,422
Truck Court				1,118,884
Non-Structural Epoxy Grout Encasement at Steel H-Piles	585	ft		582,347
Concrete Spall Repairs - Beam and Deck	500	sq ft		536,538
Finger Pier Extension				848,074
Non-Structural Epoxy Grout Encasement at Steel H-Piles	630	ft		547,612
Concrete Spall Repairs - Beam and Deck	280	sq ft		-

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Timber Fender System				**2,695,738**
Timber Fender System	*2,405*	*ft*		**2,695,738**
		Sub-Total		**80,566,503**
		General Conditions	8.0%	Included
		Mobilization	8.0%	Included
		Overhead	10.0%	Included
		Profit	10.0%	Included
		Design Contingency	10.0%	Included
		Construction Contingency	10.0%	Included
		Bonding and Insurance - Contractor to Provide	4.3%	Included
		Repair Program		***80,566,503***
		Structural Encasement Unit Cost per Linear Foot		**1,525**
		Non-Structural Encasement Unit Cost per Linear Foot		**966**
		Highest Unit Rate Used for Budgetary Purposes		

Escalation to Mid Point of Construction - 7.5 Year Construction Period

	Repair Program		***80,566,503***
Escalations to 2016 start (2 year Design Period)		6.09%	*4,906,500*
	Repair Program Cost at Construction Start		***85,473,003***
Escalations to 2020 Midpoint of Construction (3.75 years)		11.72%	*10,019,341*
	Total Project Costs With Escalations*		**95,492,344**

***Assumes work commences in 2 years and then proceeds for 7.5 years**
Rates Based on Expected Construction Cost Index

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

CH2MHILL
Job No.: 483869.AD.01
13-Mar-15

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Owner's Costs Not Included in Base Pricing				
Total Owner's Costs With Escalations				**9,095,642**
Design Costs Including Design Survey Inspections			*1.75%*	1,409,914
Client Contract Administration Costs (Including Escalation)			*0.75%*	728,726
Construction Manager Costs (Including Escalation)			*4.00%*	3,819,694
Inspections - Diving and Controlled (Including Escalation)			*2.50%*	2,387,309
Special Inspections (Including Escalation)			*0.93%*	750,000
Total Cost with Escalation and Owners Costs				**104,587,986**

Note:
Accuracy of Estimate Should be Considered to be +/- 25%

Markups Included or Specified	
General Conditions	8.0%
Mobilization	8.0%
Overhead	10.0%
Profit	10.0%
Design Contingency	10.0%
Construction Contingency	10.0%
Escalation to 2020 (Mid-Point of Construction)	16.8%

Exclusions and / or Clarifications
Permitting and Regulatory Approvals
Mitigation Requirements
Financing or Cost of Money

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Repair Program				**80,566,503**
Immediate Repairs				**2,224,388**
Finger Pier Extension				**2,224,388**
Structural Concrete Encasement at Steel H-Piles	***3,300***	***ft***	**674**	**2,224,388**
Clean Steel Piles; Remove Epoxy Coating as Needed	110	loc	2,126	233,885
Excavate at Mudline	110	loc	1,276	140,331
Encasement of Steel Piles	3,300	ft	319	1,052,481
Install "C" Channels Bolted to Web	1,100	ft	354	389,327
Fiberglass Formwork	22,462	sq ft	11	247,086
Reinforcing Steel / Wire Mesh	22	ton	1,200	26,620
Concrete Fill	451	cu yd	250	112,659
Bolts	1,100	ea	20	22,000
Priority Repairs				**34,330,764**
Pier Shed				**30,550,023**
Structural Concrete Encasement at Steel H-Piles	***28,407***	***ft***	**856**	**24,318,950**
Clean Steel Piles; Remove Epoxy Coating as Needed	1,671	loc	2,126	3,552,921
Excavate at Mudline	1,671	loc	1,276	2,131,753
Encasement of Steel Piles	28,407	ft	319	9,059,949
Install "C" Channels Bolted to Web	16,710	ft	354	5,914,232
Fiberglass Formwork	193,360	sq ft	11	2,126,963
Reinforcing Steel / Wire Mesh	191	ton	1,200	229,146
Concrete Fill	3,879	cu yd	250	969,786
Bolts	16,710	ea	20	334,200
Non-Structural Epoxy Grout Encasement at Steel H-Piles	***11,492***	***ft***	**542**	**6,231,073**
Clean Steel Piles	676	ea	2,126	1,437,328
Encasement of Steel Piles	11,492	ft	319	3,665,186
Fiberglass Encasement	72,206	sq ft	11	794,270
Epoxy Grout Fill	1,337	cu yd	250	334,289
Truck Court				**3,722,180**
Non-Structural Epoxy Grout Encasement at Steel H-Piles	***6,660***	***ft***	**559**	**3,722,180**
Clean Steel Piles	444	ea	2,126	944,044
Encasement of Steel Piles	6,660	ft	319	2,124,098
Fiberglass Encasement	41,846	sq ft	11	460,306
Epoxy Grout Fill	775	cu yd	250	193,732
Finger Pier Extension				**58,561**
Non-Structural Epoxy Grout Encasement at Steel H-Piles	***120***	***ft***	**488**	**58,561**
Clean Steel Piles	4	ea	2,126	8,505
Encasement of Steel Piles	120	ft	319	38,272
Fiberglass Encasement	754	sq ft	11	8,294
Epoxy Grout Fill	14	cu yd	250	3,491
Routine Repairs				**8,677,490**
Pier Shed				**6,039,701**
Structural Concrete Encasement at Steel H-Piles	***2,856***	***ft***	**856**	**2,445,569**
Clean Steel Piles; Remove Epoxy Coating as Needed	168	loc	2,126	357,206
Excavate at Mudline	168	loc	1,276	214,323
Encasement of Steel Piles	2,856	ft	319	910,875
Install "C" Channels Bolted to Web	1,680	ft	354	594,609
Fiberglass Formwork	19,440	sq ft	11	213,842

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Reinforcing Steel / Wire Mesh	20	ton	1,200	23,614
Concrete Fill	390	cu yd	250	97,501
Bolts	1,680	ea	20	33,600
Non-Structural Epoxy Grout Encasement at Steel H-Piles	***5,610***	***ft***	**542**	**3,041,796**
Clean Steel Piles	330	ea	2,126	701,654
Encasement of Steel Piles	5,610	ft	319	1,789,218
Fiberglass Encasement	35,249	sq ft	11	387,735
Epoxy Grout Fill	653	cu yd	250	163,188
Concrete Spall Repairs - Beam and Deck	***950***	***sq ft***	**602**	**572,336**
Chip and Clean Spall Areas to Sound Concrete	950	sq ft	159	151,494
Replace Steel Reinforcing as Needed	950	sq ft	107	101,706
Form and Pump Concrete Spall Repairs	950	sq ft	336	319,137
Truck Court				**628,178**
Non-Structural Epoxy Grout Encasement at Steel H-Piles	***585***	***ft***	**559**	**326,948**
Clean Steel Piles	39	ea	2,126	82,923
Encasement of Steel Piles	585	ft	319	186,576
Fiberglass Encasement	3,676	sq ft	11	40,432
Epoxy Grout Fill	68	cu yd	250	17,017
Concrete Spall Repairs - Beam and Deck	***500***	***sq ft***	**602**	**301,230**
Chip and Clean Spall Areas to Sound Concrete	500	sq ft	159	79,733
Replace Steel Reinforcing as Needed	500	sq ft	107	53,529
Form and Pump Concrete Spall Repairs	500	sq ft	336	167,967
Finger Pier Extension				**476,136**
Non-Structural Epoxy Grout Encasement at Steel H-Piles	***630***	***ft***	**488**	**307,447**
Clean Steel Piles	21	ea	2,126	44,651
Encasement of Steel Piles	630	ft	319	200,928
Fiberglass Encasement	3,958	sq ft	11	43,542
Epoxy Grout Fill	73	cu yd	250	18,326
Concrete Spall Repairs - Beam and Deck	***280***	***sq ft***	**602**	**168,689**
Chip and Clean Spall Areas to Sound Concrete	280	sq ft	159	44,651
Replace Steel Reinforcing as Needed	280	sq ft	107	29,976
Form and Pump Concrete Spall Repairs	280	sq ft	336	94,061
Timber Fender System				**1,513,474**
Timber Fender System	***2,405***	***ft***	**629**	**1,513,474**
Timber Piles - Installation	302	ea	1,566	472,859
Timber Wale Fender Rail - 10" x 10"	4,810	ft	69.27	333,205
Timber Chocks	604	ea	418	252,191
Anchor and Through Bolts	1,812	ea	25.00	45,300
Timber Piles - Material	302	ea	1,320	398,640
Timber Wale and Fender 10" x 10" - Material	3,340	bd ft	3.00	10,021
Timber Chock Material	419	bd ft	3.00	1,258
		Sub-Total		**45,232,642**
		General Conditions	8.0%	3,618,611
		Mobilization	8.0%	3,908,100
		Overhead	10.0%	5,275,935
		Profit	10.0%	5,803,529
		Design Contingency	10.0%	6,383,882
		Construction Contingency	10.0%	7,022,270

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
Bonding and Insurance - Contractor to Provide			4.3%	3,321,534
Repair Program				***80,566,503***
Structural Encasement Unit Cost per Linear Foot				**1,525**
Non-Structural Encasement Unit Cost per Linear Foot				**966**
Highest Unit Rate Used for Budgetary Purposes				
Escalation to Mid Point of Construction - 7.5 Year Construction Period				
Repair Program				***80,566,503***
Escalations to 2016 start (2 year Design Period)		6.09%		*4,906,500*
Repair Program Cost at Construction Start				***85,473,003***
Escalations to 2020 Midpoint of Construction (3.75 years)		11.72%		*10,019,341*
Total Project Costs With Escalations*				**95,492,344**
***Assumes work commences in 2 years and then proceeds for 7.5 years**				
Rates Based on Expected Construction Cost Index				
Owner's Costs Not Included in Base Pricing				
Total Owner's Costs With Escalations				**9,095,642**
Design Costs Including Design Survey Inspections			*1.75%*	1,409,914
Client Contract Administration Costs (Including Escalation)			*0.75%*	728,726
Construction Manager Costs (Including Escalation)			*4.00%*	3,819,694
Inspections - Diving and Controlled (Including Escalation)			*2.50%*	2,387,309
Special Inspections (Including Escalation)			*0.93%*	750,000
Total Cost with Escalation and Owners Costs				**104,587,986**

Repair Construction Cost Estimate

ITEM DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost

Note:
Accuracy of Estimate Should be Considered to be +/- 25%

Markups Included or Specified

General Conditions	8.0%
Mobilization	8.0%
Overhead	10.0%
Profit	10.0%
Design Contingency	10.0%
Construction Contingency	10.0%
Escalation to 2020 (Mid-Point of Construction	16.8%

Exclusions and / or Clarifications
Permitting and Regulatory Approvals
Mitigation Requirements
Financing or Cost of Money

Repair Construction Cost - Detailed Breakdown

Structural Concrete Encasement at Steel H-Piles - Immediate Repair, Finger Pier Extension

ITEM DESCRIPTION	Quantity	Unit	Material Unit Cost	Total Material Cost	Labor Unit Cost	Total Labor Cost	Equipment Unit Cost	Total Equipment Cost	Unit Cost	Total Cost
Structural Concrete Encasement at Steel H-Piles	*3,300.00*	*ft*	*135*	*446,864*	*408*	*1,347,665*	*130*	*429,858*	*674*	2,224,388
Clean Steel Piles; Remove Epoxy Coating as Needed	110.00	loc	-	-	1,612.04	177,324	514.18	56,560	2,126.22	233,885
Excavate at Mudline	110.00	loc	-	-	967.22	106,395	308.51	33,936	1,275.73	140,331
Encasement of Steel Piles	3,300.00	ft	-	-	241.81	797,960	77.13	254,521	318.93	1,052,481
Install "C" Channels Bolted to Web	1,100.00	ft	35.00	38,500	241.81	265,987	77.13	84,840	353.93	389,327
Fiberglass Formwork	22,462.39	sq ft	11.00	247,086	-	-	-	-	11.00	247,086
Reinforcing Steel / Wire Mesh	22.18	ton	1,200.00	26,620	-	-	-	-	1,200.00	26,620
Concrete Fill	450.63	cu yd	250.00	112,659	-	-	-	-	250.00	112,659
Bolts	1,100.00	ea	20.00	22,000	-	-	-	-	20.00	22,000
									Sub-Total	**2,224,388**

Material Cost Basis

"C" Channels Bolted to Web	$ 35	ft	Published Info, Qoutes and Previous Bid Information
Fiberglass Formwork	$ 11	sq ft	Published Info, Qoutes and Previous Bid Information
Reinforcing Steel / Wire Mesh	$ *1,200*	ton	Published Info, Qoutes and Previous Bid Information
Concrete Fill	$ 250	cu yd	Published Info, Qoutes and Previous Bid Information
Bolts	$ 20	ea	Published Info, Qoutes and Previous Bid Information

General Conditions	8.0%	177,951
Mobilization	8.0%	192,187
Overhead	10.0%	259,453
Profit	10.0%	285,398
Construction Contingency	10.0%	313,938
Design Contingency	10.0%	345,331
Bonding and Insurance	4.3%	163,342
Structural Concrete Encasement at Steel H-Piles	**$**	**3,961,988**
Total Unit Cost Per Foot	***$***	***1,201***

Labor & Equipment Cost and Productivity Per Task - Structural Concrete Encasement - Immediate Repair, Finger Pier Extension

Dive Crew Cost Per Shift: Labor and Equipment $ 12,757
Hours Per Shift 8.00

Work Task	Quantity	Unit	Production Per Day	Labor Cost Per Shift	Equip. Cost Per Shift	No. of Shifts Per Task	Total Man Hours Per Task	Unit Labor Cost Per Task	Unit Equipment Cost Per Task	Total Labor Cost Per Task	Total Equipment Cost Per Task
Clean Steel Piles; Remove Epoxy Coating as Needed	110.00	loc	6.0	9,672	3,085	18.33	146.67	1,612	514	177,324	56,560
Excavate at Mudline	110.00	loc	10.0	9,672	3,085	11.00	88.00	967	309	106,395	33,936
Encasement of Steel Piles	3,300.00	ft	40.0	9,672	3,085	82.50	660.00	242	77	797,960	254,521
Install "C" Channels Bolted to Web	1,100.00	ft	40.0	9,672	3,085	27.50	220.00	242	77	265,987	84,840
						139.33	**1,114.67**			**1,347,665**	**429,858**

Total Labor Cost Per Structural Encasement *$ 1,347,665*
Total Equipment Cost Per Structural Encasement *$ 429,858*

Total Labor and Equipment Cost **$ 1,777,524**

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

Job No.: 483869.AD.01
26-Feb-15

Repair Construction Cost - Detailed Breakdown

Non-Structural Concrete Encasement at Steel H-Piles - Priority Repair, Pier Shed

ITEM DESCRIPTION	Quantity	Unit	Material Unit Cost	Total Material Cost	Labor Unit Cost	Total Labor Cost	Equipment Unit Cost	Total Equipment Cost	Unit Cost	Total Cost
Non-Structural Grout Encasement at Steel H-Piles	*11,492*	*ft*	*98*	*1,128,559*	*337*	*3,868,574*	*107*	*1,233,940*	*542*	6,231,073
Clean Steel Piles	676	ea	-	-	1,612.04	1,089,739	514.18	347,589	2,126.22	1,437,328
Encasement of Steel Piles Including Excavation	11,492	ft	-	-	241.81	2,778,835	77.13	886,351	318.93	3,665,186
Fiberglass Encasement with Wire Mesh	72,206	sq ft	11.00	794,270	-	-	-	-	11.00	794,270
Concrete Grout Fill	1,337	cu yd	250.00	334,289	-	-	-	-	250.00	334,289

Sub-Total		**6,231,073**
General Conditions	8.0%	498,486
Mobilization	8.0%	538,365
Overhead	10.0%	726,792
Profit	10.0%	799,472
Construction Contingency	10.0%	879,419
Design Contingency	10.0%	967,361
Bonding and Insurance	4.3%	457,562
Non-Structural Grout Encasement at Steel H-Piles		**$ 11,098,528**
Total Unit Cost Per Foot		***$ 966***

Material Cost Basis

Fiberglass Encasement with Wire Mesh	$ 11.00	sq ft	Published Info, Qoutes and Previous Bid Information
Concrete Grout Fill	$ 250.00	cu yd	Published Info, Qoutes and Previous Bid Infonnation

Labor & Equipment Cost and Productivity Per Task - Non-Structural Encasement - Priority Repair, Pier Shed

Dive Crew Cost Per Shift: Labor and Equipment $ 12,757
Hours Per Shift 8.00

Work Task	Quantity	Unit	Production Per Day	Labor Cost Per Shift	Equip. Cost Per Shift	No. of Shifts Per Task	Total Man Hours Per Task	Unit Labor Cost Per Task	Unit Equipment Cost Per Task	Total Labor Cost Per Task	Total Equipment Cost Per Task
Clean Steel Piles	676	ea	6.0	9,672	3,085	112.67	901.33	1,612	514	1,089,739	347,588.80
Encasement of Steel Piles Including Excavation	11,492	ft	40.0	9,672	3,085	287.30	2,298.40	242	77	2,778,835	886,351.44
						399.97	**3,199.73**			**3,868,574**	***1,233,940***

Total Labor Cost Per Non-Structural Encasement ***$ 3,868,574***
Total Equipment Cost Per Non-Structural Encasement ***$ 1,233,940***

Total Labor and Equipment Cost **$ 5,102,514**

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

Job No.: 483869.AD.01
26-Feb-15

Repair Construction Cost - Detailed Breakdown

Concrete Spall Repairs - Beam and Under Deck - Routine Repair, Pier Shed

ITEM DESCRIPTION	Quantity	Unit	Material Unit Cost	Total Material Cost	Labor Unit Cost	Total Labor Cost	Equipment Unit Cost	Total Equipment Cost	Unit Cost	Total Cost
Concrete Spall Repairs - Beam and Deck	***950***	***sq ft***	***22***	***20,900***	***440***	***418,083***	***140***	***133,354***	***602***	572,336
Chip and Clean Spall Areas to Sound Concrete	950	sq ft	-	-	120.90	114,858	38.56	36,636	159.47	151,494
Replace Steel Reinforcing as Needed	950	sq ft	5.00	4,750	77.38	73,509	24.68	23,447	107.06	101,706
Form and Pump Concrete Spall Repairs	950	sq ft	17.00	16,150	241.81	229,716	77.13	73,271	335.93	319,137

Material Cost Basis

Replace Steel Reinforcing as Needed	$ 5.00	sq ft	Published Info, Qoutes and Previous Bid Information
Form and Pump Concrete Spall Repairs	$ 17.00	sq ft	Published Info, Qoutes and Previous Bid Information

Sub-Total		**572,336**
General Conditions	8.0%	45,787
Mobilization	8.0%	49,450
Overhead	10.0%	66,757
Profit	10.0%	73,433
Construction Contingency	10.0%	80,776
Design Contingency	10.0%	88,854
Bonding and Insurance	4.3%	42,028
Concrete Spall Repairs - Beam and Deck	**$**	**1,019,422**
Total Unit Cost Per Square Foot	***$***	***1,073***

Labor & Equipment Cost and Productivity Per Task - Concrete Spall Repair

Dive Crew Cost Per Shift: Labor and Equipment	$	12,757
Hours Per Shift		8.00

Work Task	Quantity	Unit	Production Per Day	Labor Cost Per Shift	Equip. Cost Per Shift	No. of Shifts Per Task	Total Man Hours Per Task	Unit Labor Cost Per Task	Unit Equipment Cost Per Task	Total Labor Cost Per Task	Total Equipment Cost Per Task
Chip and Clean Spall Areas to Sound Concrete	950.00	sq ft	80.0	9,672	3,085	11.88	95.00	121	39	114,858	36,635.65
Replace Steel Reinforcing as Needed	950.00	sq ft	125.0	9,672	3,085	7.60	60.80	77	25	73,509	23,446.82
Form and Pump Concrete Spall Repairs	950.00	sq ft	40.0	9,672	3,085	23.75	190.00	242	77	229,716	73,271.31
						43.23	**345.80**			***418,083***	***133,354***

Total Labor Cost Per Spall Repair	***$***	***418,083***
Total Equipment Cost Per Spall Repair	***$***	***133,354***
Total Labor and Equipment Cost	**$**	**551,436**

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

CH2MHILL
Job No.: 483869.AD.01
26-Feb-15

Repair Construction Cost - Detailed Breakdown

Timber fender System - Sample Cost Work-Up

ITEM DESCRIPTION	Quantity	Unit	Material Unit Cost	Total Material Cost	Labor Unit Cost	Total Labor Cost	Equipment Unit Cost	Total Equipment Cost	Unit Cost	Total Cost
Timber Fender System	*2,405*	*ft*	*189*	*455,219*	*255.00*	*613,268*	*185*	*444,987*	*629*	1,513,474
Timber Piles - Installation	302	ea	-	-	907.37	274,026	658.39	198,833	1,565.76	472,859
Timber Wale Fender Rail - 10" x 10"	4,810	ft	-	-	40.14	193,095	29.13	140,110	69.27	333,205
Timber Chocks	604	ea	-	-	241.97	146,147	175.57	106,044	417.54	252,191
Anchor and Through Bolts	1,812	ea	25.00	45,300	-	-	-	-	25.00	45,300
Timber Piles - Material; 8' O.C.	302	ea	1,320.00	398,640	-	-	-	-	1,320.00	398,640
Timber Wale and Fender 10" x 10" - Material	3,340	bd ft	3.00	10,021	-	-	-	-	3.00	10,021
Timber Chock Material	419	bd ft	3.00	1,258	-	-	-	-	3.00	1,258
									Sub-Total	**1,513,474**

Material Cost Basis

Anchor and Through Bolts	$	25.00	sq ft	Published Info, Qoutes and Previous Bid Information
Timber Piles - Material; 8' O.C.	$	1,320.00	sq ft	Published Info, Qoutes and Previous Bid Information
Timber Wale and Fender 10" x 10" - Material	$	3.00	sq ft	Published Info, Qoutes and Previous Bid Information
Timber Chock Material	$	3.00	sq ft	Published Info, Qoutes and Previous Bid Information

General Conditions	8.0%	121,078
Mobilization	8.0%	130,764
Overhead	10.0%	176,532
Profit	10.0%	194,185
Construction Contingency	10.0%	213,603
Design Contingency	10.0%	234,964
Bonding and Insurance	4.3%	111,138
Timber Fender System	**$**	**2,695,738**
Total Unit Cost Per Foot	***$***	***1,121***

Labor & Equipment Cost and Productivity Per Task - Timber fender System

Pile Crew: Labor and Equipment $ 12,526
Hours Per Shift 8.00

Work Task	Quantity	Unit	Production Per Day	Labor Cost Per Shift	Equip. Cost Per Shift	No. of Shifts Per Task	Total Man Hours Per Task	Unit Labor Cost Per Task	Unit Equipment Cost Per Task	Total Labor Cost Per Task	Total Equipment Cost Per Task
Timber Piles - Installation	302.00	ea	8	7,259	5,267	37.75	302.00	907	658	274,026	198,833.08
Timber Wale Fender Rail - 10" x 10"	4,810.00	ft	181	7,259	5,267	26.60	212.81	40	29	193,095	140,109.91
Timber Chocks	604.00	ea	30	7,259	5,267	20.13	161.07	242	176	146,147	106,044.31
						84.48	**675.87**			*613,268*	*444,987*

Total Labor Cost Per 2405' Timber Fender System ***$ 613,268***
Total Equipment Cost Per 2405' Timber Fender System ***$ 444,987***

Total Labor and Equipment Cost **$ 1,058,255**

Pier 40 - Repair Cost Estimate
HRPT
New York, NY

Labor Crew and Equipment

Pile Crew Labor Work-Up

Trade Position		Number of Persons	Unit Labor Rate	Shift Hours	Cost Per Shift	Labor Cost Basis
Foreman		1	$ 108.43	8.00	867.44	Local Union Contracts and Received Bid and Billing
Heavy Equip Operator (Crane, Excavator, Driver)		2	$ 110.99	8.00	1,775.84	Local Union Contracts and Received Bid and Billing
Dockbuilder Or Skilled Tradesman		4	$ 103.99	8.00	3,327.68	Local Union Contracts and Received Bid and Billing
Unskilled Laborer		2	$ 80.50	8.00	1,288.00	Local Union Contracts and Received Bid and Billing

Total Crew	9	**Workers**
Total Labor Cost Per Shift	$ 7,259	
Shift Period	8.00	**Hours**
Total Labor Man Hours Per Shift	72.00	**Hours**

Pile Crew Equipment

		Number of Pieces	Cost Per Hour	Shift Length, Hours	Cost Per Shift	Equipment Basis
250 Ft X 76 Ft Deck Barge		1.00	$ 178.85	8.00	$ 1,430.77	Based on Blue Book look up and local contractor bids and billing
180 Ft X 35 Ft Deck Barge		1.00	$ 28.85	8.00	$ 230.77	Based on Blue Book look up and local contractor bids and billing
Linkbelt Ls-278 (250 Ton) Crawler Crane		1.00	$ 250.25	8.00	$ 2,001.97	Based on Blue Book look up and local contractor bids and billing
25 Ton Crane		1.00	$ 73.75	8.00	$ 590.03	Based on Blue Book look up and local contractor bids and billing
90 T Vibratory Hammer		1.00	$ 70.61	8.00	$ 564.89	Based on Blue Book look np and local contractor bids and billing
60 Kw Generator		1.00	$ 13.73	8.00	$ 109.85	Based on Blue Book look up and local contractor bids and billing
400 Amp Welder		1.00	$ 9.23	8.00	$ 73.85	Based on Blue Book look up and local contractor bids and billing
375 Cfm Compressor		1.00	$ 17.12	8.00	$ 136.98	Based on Blue Book look up and local contractor bids and billing
Stage Float		2.00	$ 16.00	8.00	$ 128.00	Based on Blue Book look up and local contractor bids and billing

Total Equipment Cost Per Shift $ 5,267

Quantity Breakdown

Dimensions Are In Sta

2014 Report

Repair Scenario 'A'

Immediate Repairs

Finger Pier Extension

Structural Concrete Encasement at Steel H-Piles

Number of Locations	110.00	loc
Total Repair Lengths	3,300.00	ft
Length of Each Encasement	30.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.17	ft
Area of Fiberglass per Encasement	204.20	sq ft
Install Vertical Reinforcing Steel Bars	8.00	ea
Length per Encasement	240.00	ft
Weight of Bar, #6	1.50	lbs/ft
Weight of Vertical Bar per Encasement	360.48	lbs
Reinforcing Spiral Spacing	1.00	ft
Number of Reinforcing Spirals per Encasement	30.00	ea
Circumference of Each Reinforcing Spiral	2.14	ft
Length of Reinforcing Loops per Encasement	64.14	ft
Weight of Bar, #4	0.67	lbs/ft
Weight of Reinforcing Spirals per Encasement	42.85	lbs
Reinforcing Steel per Encasement	0.20	ton
Volume of Fill per Encasement	110.61	cu ft
Install "C" Channel Bolted to Web/ Length per Encasement	10.00	ft
Bolt Spacing	1.00	ft
Number of Bolts per Encasement	10.00	ea
Fiberglass Formwork	22,462.39	sq ft
Reinforcing Steel / Wire Mesh	22.18	ton
Concrete Fill	451	cu yd
Excavate at Mudline	110.00	loc
Clean Steel Piles; Remove Epoxy Coating as Needed	110.00	loc
Encasement of Steel Piles	3,300.00	ft
Install "C" Channels Bolted to Web	1,100.00	ft
Bolts	1,100.00	ea

Clean Steel Piles; Remove Epoxy Coating as Needed	**110**
Excavate at Mudline	**110**
Encasement of Steel Piles	**3,300**
Install "C" Channels Bolted to Web	**1,100**
Fiberglass Formwork	**22,462**
Reinforcing Steel / Wire Mesh	**22**
Concrete Fill	**451**
Bolts	**1,100**
Install Encasement - 15.5 ft Each	**110.00**

Priority Repairs

Pier Shed

Structural Concrete Encasement at Steel H-Piles

Number of Locations	1,671.00	loc
Total Repair Lengths	28,407.00	ft
Length of Each Encasement	17.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.17	ft
Area of Fiberglass per Encasement	115.72	sq ft
Install Vertical Reinforcing Steel Bars	8.00	ea
Length per Encasement	136.00	ft
Weight of Bar, #6	1.50	lbs/ft
Weight of Vertical Bar per Encasement	204.27	lbs
Reinforcing Spiral Spacing	1.00	ft
Number of Reinforcing Spirals per Encasement	17.00	ea
Circumference of Each Reinforcing Spiral	2.14	ft
Length of Reinforcing Loops per Encasement	36.35	ft
Weight of Bar, #4	0.67	lbs/ft
Weight of Reinforcing Spirals per Encasement	24.28	lbs
Reinforcing Steel per Encasement	0.11	ton
Volume of Fill per Encasement	62.68	cu ft
Install "C" Channel Bolted to Web/ Length per Encasement	10.00	ft
Bolt Spacing	1.00	ft
Number of Bolts per Encasement	10.00	ea
Fiberglass Formwork	193,360.32	sq ft
Reinforcing Steel / Wire Mesh	190.95	ton
Concrete Fill	3,879	cu yd
Excavate at Mudline	1,671.00	loc
Clean Steel Piles; Remove Epoxy Coating as Needed	1,671.00	loc
Encasement of Steel Piles	28,407.00	ft
Install "C" Channels Bolted to Web	16,710.00	ft

Clean Steel Piles; Remove Epoxy Coating as Needed	**1,671**
Excavate at Mudline	**1,671**
Fiberglass Formwork	**193,360**
Reinforcing Steel / Wire Mesh	**191**
Concrete Fill	**3,879**
Encasement of Steel Piles	**28,407**
Install "C" Channels Bolted to Web	**16,710**
Bolts	**16,710**
Install Encasement - 15.5 ft Each	**1,671.00**

Bolts	16,710.00	ea

Non-Structural Epoxy Grout Encasement at Steel H-Piles

Number of Encasements	676.00	ea
Length of Each Encasement	17.00	ft
Total Length of Encasements	11,492.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	106.81	sq ft
Volume of Fill per Encasement	53.41	cu ft
Fiberglass Encasement	72,206.37	sq ft
Epoxy Grout Fill	1,337	cu yd
Clean Steel Piles	676.00	ea
Encasement of Steel Piles	11,492.00	ft

Clean Steel Piles	**676.00**
Encasement of Steel Piles	**11,492.00**
Fiberglass Encasement	**72,206.37**
Epoxy Grout Fill	**1,337.15**
Encasement of Steel Piles	**676.00**

Truck Court

Non-Structural Epoxy Grout Encasement at Steel H-Piles

Number of Encasements	444.00	ea
Length of Each Encasement	15.00	ft
Total Length of Encasements	6,660.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	94.25	sq ft
Volume of Fill per Encasement	47.12	cu ft
Fiberglass Encasement	41,846.01	sq ft
Epoxy Grout Fill	775	cu yd
Clean Steel Piles	444.00	ea
Encasement of Steel Piles	6,660.00	ft

Clean Steel Piles	**444.00**
Fiberglass Encasement	**41,846.01**
Epoxy Grout Fill	**774.93**
Encasement of Steel Piles	**6,660.00**
Encasement of Steel Piles	**444.00**

Finger Pier Extension

Non-Structural Epoxy Grout Encasement at Steel H-Piles

Number of Encasements	4.00	ea
Length of Each Encasement	30.00	ft
Total Length of Encasements	120.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	188.50	sq ft
Volume of Fill per Encasement	94.25	cu ft
Fiberglass Encasement	753.98	sq ft
Epoxy Grout Fill	14	cu yd
Clean Steel Piles	4.00	ea
Encasement of Steel Piles	120.00	ft

Clean Steel Piles	**4.0**
Fiberglass Encasement	**753.98**
Epoxy Grout Fill	**13.96**
Encasement of Steel Piles	**120.00**
Encasement of Steel Piles	**4.00**

Routine Repairs

Pier Shed

Structural Concrete Encasement at Steel H-Piles

Number of Locations	168.00	loc
Total Repair Lengths	17.00	ft
Length of Each Encasement	2,856.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.17	ft
Area of Fiberglass per Encasement	19,440.18	sq ft
Install Vertical Reinforcing Steel Bars	8.00	ea
Length per Encasement	136.00	ft
Weight of Bar, #6	1.50	lbs/ft
Weight of Vertical Bar per Encasement	204.27	lbs
Reinforcing Spiral Spacing	1.00	ft
Number of Reinforcing Spirals per Encasement	21.00	ea
Circumference of Each Reinforcing Spiral	2.14	ft
Length of Reinforcing Loops per Encasement	44.90	ft
Weight of Bar, #4	0.67	lbs/ft
Weight of Reinforcing Spirals per Encasement	29.99	lbs
Reinforcing Steel per Encasement	0.12	ton

Clean Steel Piles; Remove Epoxy Coating as Needed	**168**
Excavate at Mudline	**168**
Fiberglass Formwork	**19,440**
Reinforcing Steel / Wire Mesh	**20**
Concrete Fill	**390**
Encasement of Steel Piles	**2,856**
Install "C" Channels Bolted to Web	**1,680**
Bolts	**1,680**
Install Encasement - 15.5 ft Each	**168.00**

Volume of Fill per Encasement	62.68	cu ft
Install "C" Channel Bolted to Web/ Length per Encasement	10.00	ft
Bolt Spacing	1.00	ft
Number of Bolts per Encasement	10.00	ea
Fiberglass Formwork	19,440.18	sq ft
Reinforcing Steel / Wire Mesh	19.68	ton
Concrete Fill	390	cu yd
Excavate at Mudline	168.00	loc
Clean Steel Piles; Remove Epoxy Coating as Needed	168.00	loc
Encasement of Steel Piles	17.00	ft
Install "C" Channels Bolted to Web	1,680.00	ft
Bolts	1,680.00	ea

Non-Structural Epoxy Grout Encasement at Steel H-Piles

Clean Steel Piles	**330.00**
Fiberglass Encasement	**35,248.67**
Epoxy Grout Fill	**652.75**
Encasement of Steel Piles	**5,610.00**
Encasement of Steel Piles	**330.00**

Number of Encasements	330.00	ea
Length of Each Encasement	17.00	ft
Total Length of Encasements	5,610.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	106.81	sq ft
Volume of Fill per Encasement	53.41	cu ft
Fiberglass Encasement	35,248.67	sq ft
Epoxy Grout Fill	653	cu yd
Clean Steel Piles	330.00	ea
Encasement of Steel Piles	5,610.00	ft

Concrete Spall Repairs - Beam and Deck

Chip and Clean Spall Areas to Sound Concrete	**950.00**
Replace Steel Reinforcing as Needed	**950.00**
Form and Pump Concrete Spall Repairs	**950.00**

Chip and Clean Spall Areas to Sound Concrete	950.00	sq ft
Replace Steel Reinforcing as Needed	950.00	sq ft
Form and Pump Concrete Spall Repairs	950.00	sq ft

Truck Court

Non-Structural Epoxy Grout Encasement at Steel H-Piles

Clean Steel Piles	**39.00**
Fiberglass Encasement	**3,675.66**
Epoxy Grout Fill	**68.07**
Encasement of Steel Piles	**585.00**
Encasement of Steel Piles	**39.00**

Number of Encasements	39.00	ea
Length of Each Encasement	15.00	ft
Total Length of Encasements	585.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	94.25	sq ft
Volume of Fill per Encasement	47.12	cu ft
Fiberglass Encasement	3,675.66	sq ft
Epoxy Grout Fill	68	cu yd
Clean Steel Piles	39.00	ea
Encasement of Steel Piles	585.00	ft

Concrete Spall Repairs - Beam and Deck

Chip and Clean Spall Areas to Sound Concrete	**500.00**
Replace Steel Reinforcing as Needed	**500.00**
Form and Pump Concrete Spall Repairs	**500.00**

Chip and Clean Spall Areas to Sound Concrete	500.00	sq ft
Replace Steel Reinforcing as Needed	500.00	sq ft
Form and Pump Concrete Spall Repairs	500.00	sq ft

Finger Pier Extension

Non-Structural Epoxy Grout Encasement at Steel H-Piles

Clean Steel Piles	**21.00**
Fiberglass Encasement	**3,958.41**
Epoxy Grout Fill	**73.30**
Encasement of Steel Piles	**630.00**
Encasement of Steel Piles	**21.00**

Number of Encasements	21.00	ea
Length of Each Encasement	30.00	ft
Total Length of Encasements	630.00	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	188.50	sq ft
Volume of Fill per Encasement	94.25	cu ft
Fiberglass Encasement	3,958.41	sq ft
Epoxy Grout Fill	73	cu yd
Clean Steel Piles	21.00	ea
Encasement of Steel Piles	630.00	ft

Concrete Spall Repairs - Beam and Deck

Chip and Clean Spall Areas to Sound Concrete	**280.00**

Quantity Breakdown

Dimensions Are In Sta

Chip and Clean Spall Areas to Sound Concrete	280.00	sq ft	
Replace Steel Reinforcing as Needed	280.00	sq ft	
Form and Pump Concrete Spall Repairs	280.00	sq ft	

Replace Steel Reinforcing as Needed	**280.00**
Form and Pump Concrete Spall Repairs	**280.00**

Timber Fender System

Timber Fender System

Length	2,405.00	ft	
Timber Pile Spacing	8.00	ft	
Timber Piles - Installation	302.00	ea	300.63
Timber Wale Fender Rail - 10" x 10"	4,810.00	ft	
Timber Chocks	604.00	ea	
Anchor and Through Bolts	1,812.00	ea	
Timber Piles - Material	302.00	ea	
Timber Wale and Fender 10" x 10" - Material	3,340	bd ft	
Timber Chock Material	419	bd ft	

Timber Piles - Installation	**302.00**
Timber Wale Fender Rail - 10" x 10"	**4,810.00**
Timber Chocks	**604.00**
Anchor and Through Bolts	**1,812.00**
Timber Piles - Material	**302.00**
Timber Wale and Fender 10" x 10" - Material	**3,340.26**
Timber Chock Material	**419.44**

Pile Encasements 2009

Pile Rating Basis

Pier Shed Piles	2,846.00	ea	
Truck Court Piles	482.00	ea	
Finger Pier Piles	135.00	ea	
Total Number of Piles	3,463.00	ea	
Piles Rated Serve or Major	40%		
Piles Rated Moderate or Minor	60%		
Piles Rated Serve or Major	1,385.00	ea	1,385.20
Piles Rated Moderate or Minor	2,078.00	ea	2,077.80

Pile Length Basis - 2009 Report

Pier Shed Effective Length	33.00	ft
Truck Court Effective Length	27.00	ft
Finger Pier Effective Length	50.00	ft
Fixity Below Mudline	12.00	ft
Embedment in Pile Cap	1.00	ft
Exposed Lengths (Effective Less Embedments)		
Pier Shed Exposed Length	20.00	ft
Truck Court Exposed Length	14.00	ft
Finger Pier Exposed Length	37.00	ft
Encasement Mudline Embedment	2.00	ft
Encasement Lengths, Each		
Pier Shed Encasement Length	22.00	ft
Truck Court Encasement Length	16.00	ft
Finger Pier Encasement Length	39.00	ft
Pier Shed Piles	2,846.00	ea
Truck Court Piles	482.00	ea
Finger Pier Piles	135.00	ea
Encasement Lengths, Total		
Pier Shed Length	62,612.00	ft
Truck Court Length	7,712.00	ft
Finger Pier Length	5,265.00	ft
Total Encasement Length	75,589.00	ft
Average Encasement Length	21.83	ft

Structural Concrete Encasement at Steel H-Piles

Number of Encasements	1,385.00	ea
Length of Each Encasement	21.83	ft
Total Length of Encasements	30,231.23	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	137.15	sq ft
Install Vertical Reinforcing Steel Bars	8.00	ea
Length per Encasement	174.62	ft
Weight of Bar, #6	1.50	lbs/ft
Weight of Vertical Bar per Encasement	262.28	lbs
Reinforcing Spiral Spacing	1.00	ft
Number of Reinforcing Spirals per Encasement	21.83	ea
Circumference of Each Reinforcing Spiral	2.09	ft
Length of Reinforcing Loops per Encasement	45.72	ft
Weight of Bar, #4	0.67	lbs/ft
Weight of Reinforcing Spirals per Encasement	30.54	lbs

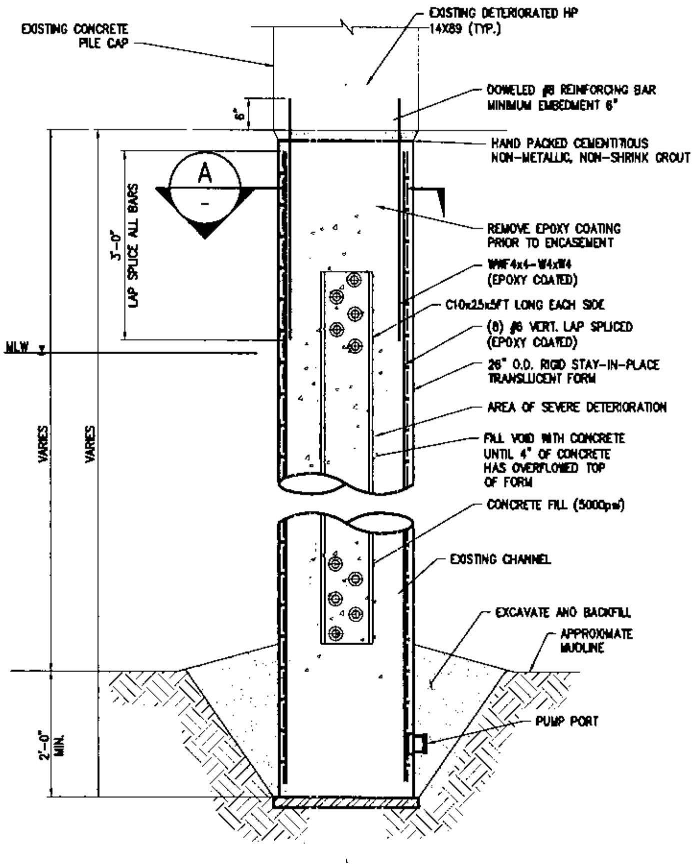
Clean Steel Piles	**1,385.00**	**ea**
Encasement of Steel Piles	**30,231.23**	**ft**
Fiberglass Encasement	**189,948.45**	**sq ft**
Reinforcing Steel / Wire Mesh	**202.78**	**ton**
Concrete Fill	**3,517.56**	**cu yd**
Install "C" Channels Bolted to Web	**13,850.00**	**ft**
Bolts	**13,850.00**	**ea**
Cathodic Protection - Sacrificial Zinc Mesh	**180,451.03**	**sq ft**

Reinforcing Steel per Encasement	0.15	ton
Volume of Fill per Encasement	68.57	cu ft
Install "C" Channel Bolted to Web/ Length per Encasement	10.00	ft
Bolt Spacing	1.00	ft
Number of Bolts per Encasement	10.00	ea
Fiberglass Encasement	189,948.45	sq ft
Reinforcing Steel / Wire Mesh	202.78	ton
Concrete Fill	3,518	cu yd
Clean Steel Piles	1,385.00	ea
Encasement of Steel Piles	30,231.23	ft
Install "C" Channels Bolted to Web	13,850.00	ft
Bolts	13,850.00	ea
Cathodic Protection - Sacrificial Zinc Mesh	180,451.03	sq ft

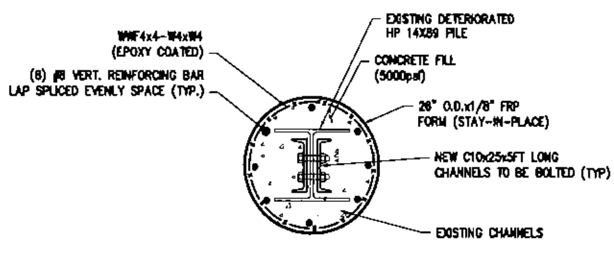
Non-Structural Concrete Encasement at Steel H-Piles

Number of Encasements	2,078.00	ea
Length of Each Encasement	21.83	ft
Total Length of Encasements	45,357.77	ft
Steel H-Pile Size	14" x 14"	ft
Diameter of Encasement	2.00	ft
Area of Fiberglass per Encasement	137.15	sq ft
Volume of Fill per Encasement	68.57	cu ft
Fiberglass Encasement	284,991.25	sq ft
Concrete Fill	5,278	cu yd
Clean Steel Piles	2,078.00	ea
Encasement of Steel Piles	45,357.77	ft
Cathodic Protection - Sacrificial Zinc Mesh	43,089.88	sq ft

Clean Steel Piles	**2,078.00**	**ea**
Encasement of Steel Piles	**45,357.77**	**ft**
Fiberglass Encasement	**284,991.25**	**sq ft**
Concrete Fill	**5,277.62**	**cu yd**
Cathodic Protection - Sacrificial Zinc Mesh	**43,089.88**	**sq ft**



TYPICAL STRUCTURAL CONCRETE ENCASEMENT DETAIL

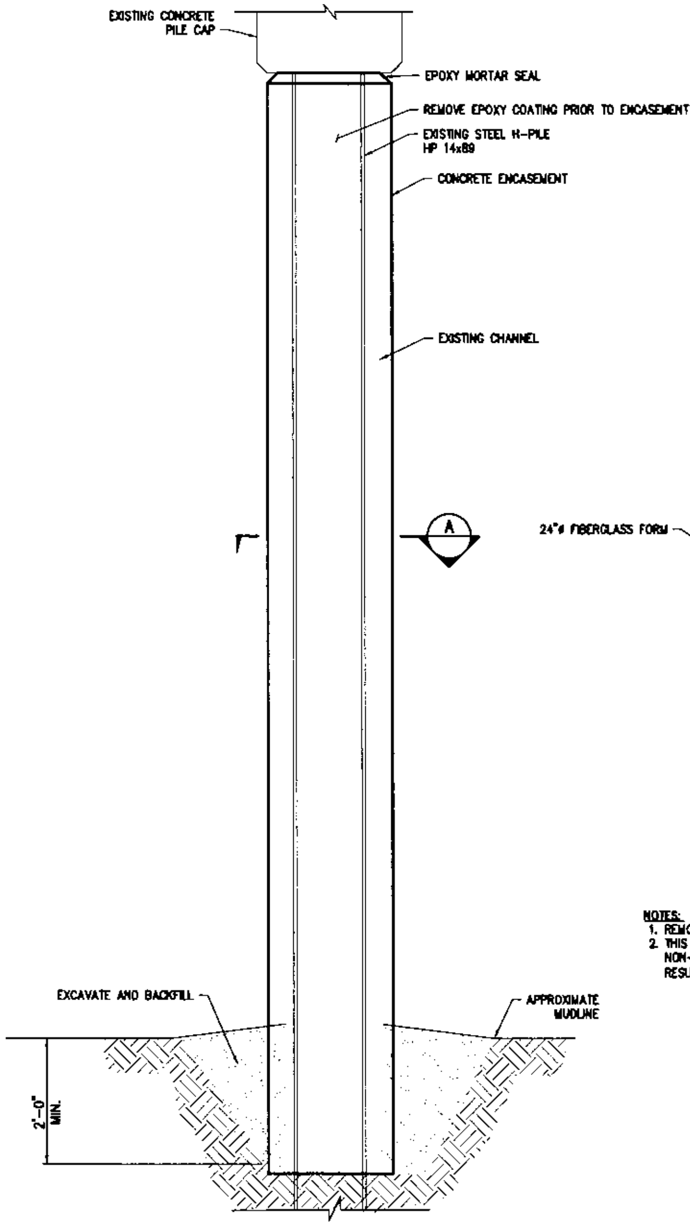


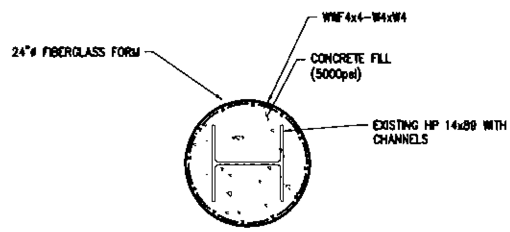
NOTES:
1. REMOVE TWO TO FOUR ANODES PER PILE PRIOR TO ENCASEMENT.
2. THIS DETAIL IS A CONCEPTUAL REPRESENTATION OF A TYPICAL STRUCTURAL ENCASEMENT. LOCATION SPECIFIC CONDITIONS MAY RESULT IN VARIATIONS DURING THE DESIGN/CONSTRUCTION PHASE.

A SECTION

3/8"=1'-0" 0 1 5 FT.

PURPOSE: PIER 40 CONDITION INSPECTION PREPARED BY HALCROW ENGINEERS, P.C. NEW YORK, NY	PIER 40 - CONCEPTUAL REPAIR SKETCHES HUDSON RIVER NEW YORK, NY SKETCH A HUDSON RIVER PARK TRUST 353 WEST ST., PIER 40, 2nd FLOOR NEW YORK, NY	PROPOSED: REPAIR AND UPGRADE CITY: NEW YORK COUNTY: NEW YORK SHEET 1 OF 4 DATE: AUGUST 2014





A - SECTION

NOTES:
1. REMOVE TWO TO FOUR ANODES PER PILE PRIOR TO ENCASEMENT.
2. THIS DETAIL IS A CONCEPTUAL REPRESENTATION OF A TYPICAL NON-STRUCTURAL ENCASEMENT. LOCATION SPECIFIC CONDITIONS MAY RESULT IN VARIATIONS DURING THE DEISGN/CONSTRUCTION PHASE.

TYPICAL NON-STRUCTURAL ENCASEMENT DETAIL

0 1 5 FT.
3/8"=1'-0"

PURPOSE: PIER 40 CONDITION INSPECTION PREPARED BY HALCROW ENGINEERS, P.C. NEW YORK, NY	PIER 40 - CONCEPTUAL REPAIR SKETCHES HUDSON RIVER NEW YORK, NY SKETCH B HUDSON RIVER PARK TRUST 353 WEST ST., PIER 40, 2nd FLOOR NEW YORK, NY	PROPOSED: REPAIR AND UPGRADE CITY: NEW YORK COUNTY: NEW YORK SHEET 2 OF 4 DATE: AUGUST 2014

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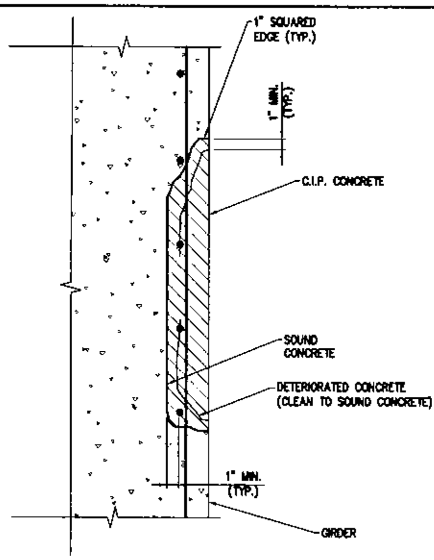
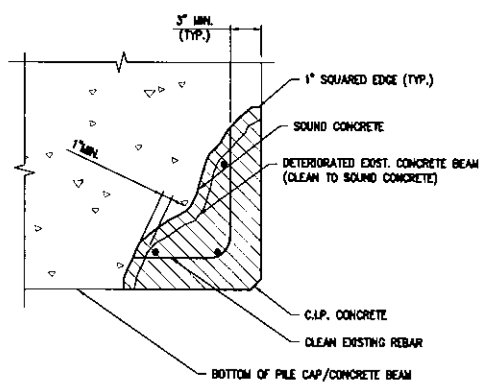


SPALL REPAIR – VERTICAL FACE



CONCRETE BEAM CORNER SPALL REPAIR

NOTE:
THIS DETAIL IS A CONCEPTUAL REPRESENTATION OF A TYPICAL CONCRETE SPALL/DELAMINATION REPAIR. LOCATION SPECIFIC CONDITIONS MAY RESULT IN VARIATIONS TO THIS DETAIL DURING THE DESIGN/CONSTRUCTION PHASE.

3/4"=1'-0" 0 1 2 3 FT.

PURPOSE: PIER 40 CONDITION INSPECTION PREPARED BY HALCROW ENGINEERS, P.C. NEW YORK, NY	PIER 40 – CONCEPTUAL REPAIR SKETCHES HUDSON RIVER NEW YORK, NY SKETCH C HUDSON RIVER PARK TRUST 353 WEST ST., PIER 40, 2nd FLOOR NEW YORK, NY	PROPOSED: REPAIR AND UPGRADE CITY: NEW YORK COUNTY: NEW YORK SHEET 3 OF 4 DATE: AUGUST 2014

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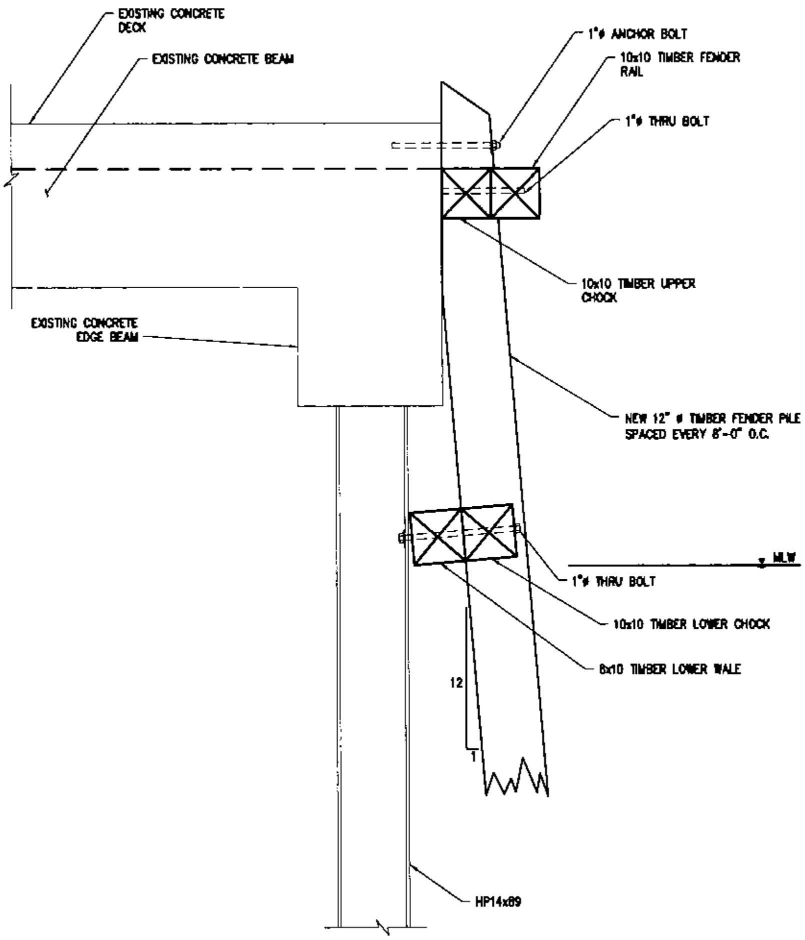


TYPICAL FENDER SYSTEM REPLACEMENT DETAIL

NOTE:
THIS DETAIL IS A CONCEPTUAL REPRESENTATION OF A TYPICAL FENDER SYSTEM. LOCATION SPECIFIC CONDITIONS MAY RESULT IN VARIATIONS TO THIS DETAIL DURING THE DESIGN/CONSTRUCTION PHASE.

3/8"=1'-0" 0 1 5 FT.

PURPOSE: PIER 40 CONDITION INSPECTION PREPARED BY HALCROW ENGINEERS, P.C. NEW YORK, NY	PIER 40 – CONCEPTUAL REPAIR SKETCHES HUDSON RIVER NEW YORK, NY SKETCH D HUDSON RIVER PARK TRUST 353 WEST ST., PIER 40, 2nd FLOOR NEW YORK, NY	PROPOSED: REPAIR AND UPGRADE CITY: NEW YORK COUNTY: NEW YORK SHEET 4 OF 4 DATE: AUGUST 2014

APPENDIX D

STRUCTURAL CALCULATIONS

APPENDIX D

STRUCTURAL CALCULATIONS

STEEL PILE PROPERTIES

STEEL CONSTRUCTION

A Manual for Architects, Engineers and Fabricators of Buildings and Other Steel Structures

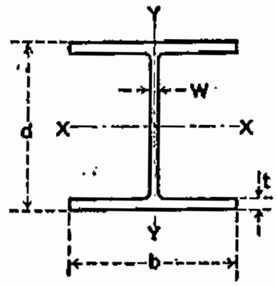
Fifth Edition

Twenty-third Printing

1958

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
NEW YORK, N. Y.

Printed in the United States of America



H BEARING PILES

DIMENSIONS AND PROPERTIES FOR DESIGNING

Section Number and Nominal Size	Weight per Foot	Area	Depth	Flange		Web Thickness	Axis X-X			Axis Y-Y		
				Width	Thickness							
		A	d	b	t	W	I	S	r	I′	S′	r′
	Lb.	In.²	In.	In.	In.	In.	In.⁴	In.³	In.	In.⁴	In.³	In.
	117	34.44	14.234	14.885	.805	.805	1228.5	172.6	5.97	443.1	59.5	3.59
BP 14	102	30.01	14.032	14.784	.704	.704	1055.1	150.4	5.93	379.6	51.3	3.56
14x14½	89	26.19	13.856	14.696	.616	.616	909.1	131.2	5.89	326.2	44.4	3.53
	73	21.46	13.636	14.586	.506	.506	733.1	107.5	5.85	261.9	35.9	3.49
BP 12	74	21.76	12.122	12.217	.607	.607	566.5	93.5	5.10	184.7	30.2	2.91
12 x 12	53	15.58	11.780	12.046	.436	.436	394.8	67.0	5.03	127.3	21.2	2.86
BP 10	57	16.76	10.012	10.224	.564	.564	294.7	58.9	4.19	100.6	19.7	2.45
10 x 10	42	12.35	9.720	10.078	.418	.418	210.8	43.4	4.13	71.4	14.2	2.40
BP 8 8 x 8	36	10.60	8.026	8.158	.446	.446	119.8	29.9	3.36	40.4	9.9	1.95

PART IV

STANDARD SPECIFICATIONS AND CODES

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings; as revised June 1949. (For brevity this document is referred to in the Manual as the A.I.S.C. Specification.)

Code of Standard Practice; as revised April 26, 1956.

AMERICAN INSTITUTE OF BOLT, NUT AND RIVET MANUFACTURERS

Tentative Specifications for Cold Riveted Construction; September, 1942.

AMERICAN SOCIETY FOR TESTING MATERIALS

Specifications for Steel for Bridges and Buildings. A.S.T.M. Designation A7-46.

Specifications for Structural Rivet Steel. A.S.T.M. Designation A141-39.

AMERICAN WELDING SOCIETY

Application of and Extracts from Code for Arc and Gas Welding in Building Construction.

UNITED STATES DEPARTMENT OF COMMERCE

Minimum Design Loads in Buildings and other Structures; as sponsored by the National Bureau of Standards and adopted by American Standards Association, A58.1—1945.

(3) BENDING.

Tension on extreme fibers of rolled sections, plate girders, and built-up members.

(See Section 26 (a)) 20,000

Compression on extreme fibers of rolled sections plate girders, and built-up members.

With $\frac{ld}{bt}$ not in excess of 600, ... 20,000

With $\frac{ld}{bt}$ in excess of 600, ... $\dfrac{12{,}000{,}000}{\frac{ld}{bt}}$

in which l is the unsupported length and d the depth, of the member; b is the width, and t the thickness, of its compression flange; all in inches; except that l shall be taken as twice the length of the compression flange of a cantilever beam not fully stayed at its outer end against translation or rotation.

Stress on extreme fibers of pins.......... 30,000

Fiber stresses in butt welds, due to bending, shall not exceed the values prescribed for tension and compression, respectively.

Fully continuous beams and girders may be proportioned for negative moments which are maximum at interior points of support, at a unit bending stress 20 percent higher than above stated; provided that the section modulus used over supports shall not be less than that required for the maximum positive moments in the same beam or girder, and provided that the compression flange shall be regarded as unsupported from the support to the point of contraflexure.

For columns proportioned for combined axial and bending stresses, the maximum unit bending stress Fb, Sect. 12 (a) may be taken at 24,000 pounds per square inch, when this stress is induced by the gravity loading of fully or partially restrained beams framing into the columns.

(4) SHEARING.

Rivets.......... 15,000

Pins, and turned bolts in reamed or drilled holes.......... 15,000

Unfinished bolts.......... 10,000

Webs of beams and plate girders, gross section.......... 13,000

Weld Metal

on section through throat of fillet weld, or on faying surface area of plug or slot weld.......... 13,600

on section through throat of butt weld.......... 13,000

(Stress in a fillet weld shall be considered as shear on the throat, for any direction of applied stress. Neither plug nor slot welds shall be assigned any values in resistance to stresses other than shear.)

Table 1.1a (Cont'd.)
Historical Summary of ASTM Specifications for Structural Steel

Date	Specification	Material	Yield Point, ksi	Tensile Strength†, ksi
1949	A6-49T* * Issued as a tentative standard covering delivery requirements for A7 steel.			
	A7-49T	Structural Steel	½ Tensile Str. ≥33	60/72
	A141-49T	Rivet Steel	28	52/62
1958	A373-58T	Structural Steel	32	58-75
1961	A7-61T	Structural Steel		
		All shapes	33	60/75
		Plates/bars to 1½ in.	33	60/72
		Plates/bars over 1½ in.	33	60/75
1962	A36-62T	Structural Steel		
		All shapes	36	58/80
		Plates to 8 in.	36	58/80
		Bars to 4 in.	36	58/80
1963	A242-63T	HSLA Steel:		
		Group 1 shapes & plates/bars to ¾ in.	50	70
		Group 2 shapes & plates/bars over ¾ to 1½ in.	46	67
		Group 3 shapes & plates/bars over 1½ to 4 in.	42	63
	A440-63T	High-Strength Steel:		
		Group 1 shapes & plates/bars to ¾ in.	50	70
		Group 2 shapes & plates/bars over ¾ to 1½ in.	46	67
		Group 3 shapes & plates over 1½ to 4 in.	42	63

L-PILE ANALYSIS
(PILE POINT OF FIXITY CALCULATION)

Halcrow

PROJECT

SUBJECT L-Pile Analysis

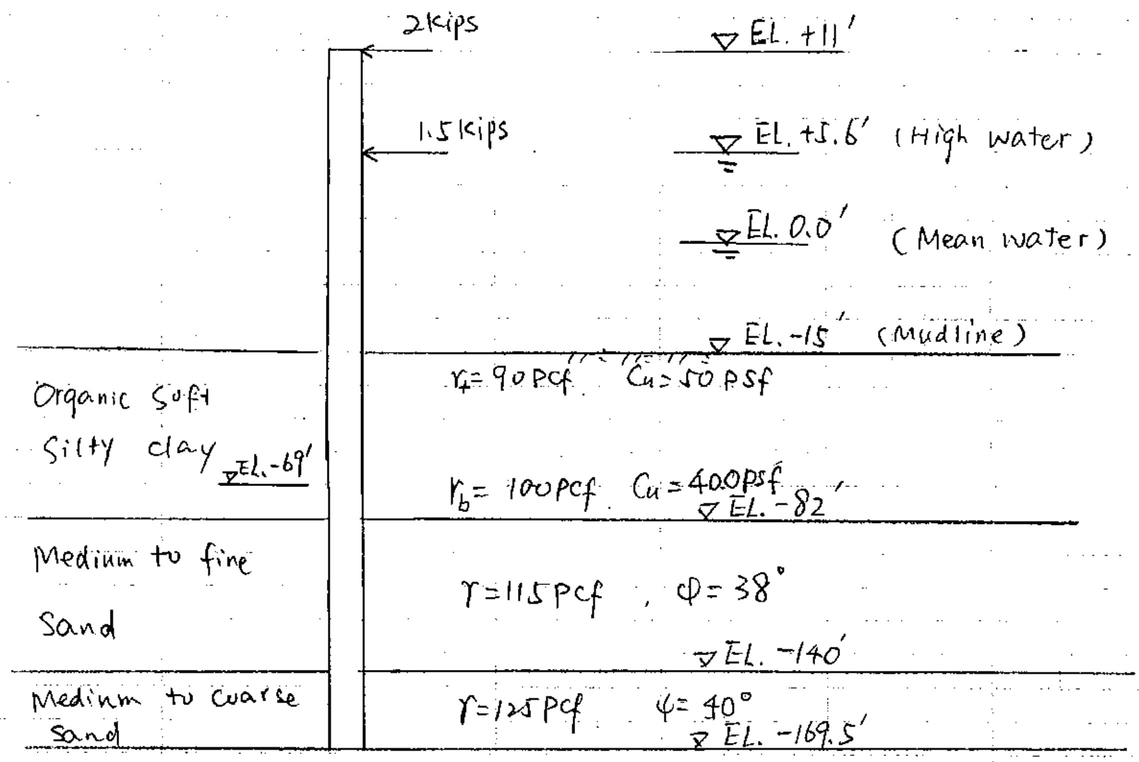
SHEET NO. OF

JOB NO.

MADE BY M, Hu DATE 4/23/09

CHKD. BY DATE

HP 14x89 , $I = 326 in^4$
(13.83"x14.7")



From L-Pile analysis, It's found that the fixity depth will be (38'-26') = 12' from the mudline.

I1

==

LPILE Plus for Windows, Version 4.0 (4.0.10)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This program is licensed to:

humi
HALCROW

```
Path to file locations:      C:\Documents and Settings\HuMi\Desktop\
Name of input data file:     HP.lpd
Name of output file:         HP.lpo
Name of plot output file:    HP.lpp
Name of runtime file:        HP.lpr
```

Time and Date of Analysis

Date: April 23, 2009   Time: 16:25:35

Problem Title

HP PILE

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip

I2

```
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments              =        128
- Maximum number of iterations allowed =        100
- Deflection tolerance for convergence =   1.0000E-05 in
- Maximum allowable deflection           =   1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) =  1


-------------------------------------------------------------------------------
                  Pile Structural Properties and Geometry
-------------------------------------------------------------------------------

Pile Length                                 =    2166.00 in
Depth of ground surface below top of pile =     312.00 in
Slope angle of ground surface               =        .00 deg.

Structural properties of pile defined using  2 points

Point     Depth          Pile       Moment of       Pile      Modulus of
            X          Diameter      Inertia        Area      Elasticity
           in             in          in**4         Sq.in     lbs/Sq.in
-----  ---------   -----------  ----------    ----------  -----------
  1       0.0000        13.830    326.0000      203.3000  29000000.000
  2    2166.0000        13.830    326.0000      203.3000  29000000.000


-------------------------------------------------------------------------------
                     Soil and Rock Layering Information
-------------------------------------------------------------------------------

The soil profile is modelled using  3 layers

Layer  1 is soft clay, p-y criteria by Matlock, 1970
Distance from top of pile to top of layer    =      312.000 in
Distance from top of pile to bottom of layer =     1116.000 in

Layer  2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =     1116.000 in
Distance from top of pile to bottom of layer =     1812.000 in
p-y subgrade modulus k for top of soil layer =      125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      125.000 lbs/in**3

Layer  3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =     1812.000 in
Distance from top of pile to bottom of layer =     2166.000 in
```

p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

(Depth of lowest layer extends .00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 6 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	312.00	.01600
2	1116.00	.02180
3	1116.00	.03040
4	1812.00	.03040
5	1812.00	.03620
6	2166.00	.03620

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	312.000	.34700	.00	.02000	.0
2	1116.000	2.77800	.00	.02000	.0
3	1116.000	.00000	38.00	------	------
4	1812.000	.00000	38.00	------	------
5	1812.000	.00000	40.00	------	------
6	2166.000	.00000	40.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

Distributed Lateral Loading

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lbs/in
1	58.800	-125.00000
2	70.800	-125.00000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = -2000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = .000 lbs

(Zero slope for this load indicates fixed-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = -2000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = .000 lbs

(Zero slope for this load indicates fixed-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	-5.002	917084.6109	-2000.0000	2.624E-17	19452.8837	0.0000
16.922	-4.988	883240.8609	-2000.0000	.001611	18735.0017	0.0000

I5

33.844	-4.947	849397.1109	-2000.0000	.003162	18017.1197	0.0000
50.766	-4.881	815553.3609	-2000.0000	.004652	17299.2377	0.0000
67.688	-4.790	781709.6109	-3057.6172	.006081	16581.3557	0.0000
84.609	-4.675	712072.1292	-4115.2344	.007418	15104.2294	0.0000
101.531	-4.539	642434.6476	-4115.2344	.008631	13627.1030	0.0000
118.453	-4.383	572797.1659	-4115.2344	.009718	12149.9767	0.0000
135.375	-4.210	503159.6842	-4115.2344	.010681	10672.8504	0.0000
152.297	-4.021	433522.2025	-4115.2344	.011519	9195.7240	0.0000
169.219	-3.820	363884.7208	-4115.2344	.012233	7718.5977	0.0000
186.141	-3.607	294247.2391	-4115.2344	.012822	6241.4713	0.0000
203.063	-3.386	224609.7574	-4115.2344	.013286	4764.3450	0.0000
219.984	-3.158	154972.2757	-4115.2344	.013626	3287.2187	0.0000
236.906	-2.925	85334.7940	-4115.2344	.013841	1810.0923	0.0000
253.828	-2.689	15697.3123	-4115.2344	.013932	332.9660	0.0000
270.750	-2.453	-53940.1693	-4115.2344	.013897	1144.1603	0.0000
287.672	-2.219	-123577.6510	-4115.2344	.013738	2621.2867	0.0000
304.594	-1.988	-193215.1327	-4115.2344	.013455	4098.4130	0.0000
321.516	-1.764	-262852.6144	-4002.5901	.013047	5575.5394	13.3135
338.438	-1.547	-328677.7898	-3728.1576	.012517	6971.8004	19.1218
355.359	-1.340	-389027.4495	-3355.9820	.011875	8251.9166	24.8657
372.281	-1.145	-442256.8056	-2888.1895	.011131	9380.9994	30.4227
389.203	-.963250	-486774.6128	-2329.0260	.010300	10325.2959	35.6649
406.125	-.796334	-521079.7794	-1684.9423	.009398	11052.9653	40.4595
423.047	-.645200	-543799.3802	-991.3057	.008445	11534.8856	41.5216
439.969	-.510537	-554629.2819	-290.7035	.007462	11764.6058	41.2827
456.891	-.392673	-553637.8781	400.9258	.006470	11743.5765	40.4611
473.813	-.291578	-541060.4509	1073.4688	.005490	11476.7884	39.0269
490.734	-.206871	-517307.6682	1716.2004	.004543	10972.9525	36.9377
507.656	-.137832	-482977.7932	2317.4288	.003648	10244.7590	34.1217
524.578	-.083423	-438877.1868	2863.6491	.002823	9309.3121	30.4362
541.500	-.042306	-386061.1699	3337.1411	.002084	8188.9969	25.5259
558.422	-.012883	-325935.8173	3705.5469	.001447	6913.6386	18.0161
575.344	.006668	-260651.5671	3729.8179	9.221E-04	5528.8515	-15.1475
592.266	.018324	-199704.7925	3414.1307	5.101E-04	4236.0694	-22.1637
609.188	.023931	-145104.5820	3012.8490	2.015E-04	3077.9085	-25.2639
626.109	.025144	-97738.6854	2572.8599	-1.583E-05	2073.1994	-26.7385
643.031	.023395	-58029.3534	2117.0478	-1.552E-04	1230.8987	-27.1340
659.953	.019890	-26089.8476	1661.7243	-2.305E-04	553.4089	-26.6808
676.875	.015594	-1790.3722	1220.2079	-2.555E-04	37.9768	-25.5022
693.797	.011244	15206.5623	804.1218	-2.435E-04	322.5564	-23.6751
710.719	.007354	25424.1238	424.0006	-2.071E-04	539.2878	-21.2515
727.641	.004235	29556.3335	89.6791	-1.579E-04	626.9388	-18.2621
744.563	.002010	28459.2006	-189.1928	-1.060E-04	603.6668	-14.6979
761.484	6.48E-04	23153.3385	-401.3488	-5.978E-05	491.1207	-10.3769
778.406	-1.29E-05	14876.0509	-461.3911	-2.575E-05	315.5457	3.2805
795.328	-2.23E-04	7538.1320	-367.7597	-5.688E-06	159.8963	7.7858
812.250	-2.05E-04	2429.6832	-235.9200	3.232E-06	51.5376	7.7963
829.172	-1.14E-04	-446.2865	-113.9522	5.008E-06	9.4665	6.6191
846.094	-3.59E-05	-1426.8873	-18.1563	3.331E-06	30.2666	4.7031
863.016	-1.18E-06	-1060.7622	38.1817	1.105E-06	22.5005	1.9555
879.938	1.45E-06	-134.6741	32.7552	3.490E-08	2.8567	-2.5969
896.859	1.22E-09	47.7979	3.9805	-4.285E-08	1.0139	-.8040
913.781	-1.05E-11	.040893	-1.4123	-3.597E-11	8.674E-04	.1666
930.703	-4.32E-16	-3.478E-04	-.001208	3.113E-13	7.377E-06	1.452E-04
947.625	3.53E-18	-1.449E-08	1.028E-05	1.276E-17	3.074E-10	-1.214E-06

I6

964.547	1.46E-22	1.165E-10	4.282E-10	-1.043E-19	2.472E-12	-5.143E-11
981.469	-1.13E-24	4.901E-15	-3.443E-12	-4.320E-24	1.040E-16	4.069E-13
998.391	0.000	-3.736E-17	-1.448E-16	0.0000	7.926E-19	1.738E-17
1015.	0.000	-1.585E-21	1.104E-18	0.0000	3.362E-23	-1.305E-19
1032.	0.000	1.148E-23	4.683E-23	0.0000	2.436E-25	-5.615E-24
1049.	0.000	0.0000	-3.393E-25	0.0000	0.0000	0.0000
1066.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1083.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1100.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1117.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1134.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1151.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1168.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1185.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1201.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1218.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1235.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1252.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1269.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1286.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1303.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1320.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1337.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1354.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1371.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1388.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1405.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1421.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1438.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1455.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1472.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1489.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1506.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1523.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1540.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1557.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1574.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1591.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1608.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1625.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1641.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1658.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1675.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1692.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1709.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1726.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1743.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1760.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1777.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1794.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1811.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1828.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1844.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1861.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1878.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000

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1895.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1912.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1929.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1946.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1963.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1980.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
1997.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2014.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2031.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2048.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2064.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2081.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2098.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2115.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2132.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2149.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000
2166.	0.000	0.0000	0.0000	0.0000	0.0000	0.0000

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =    -5.00169272 in
Computed slope at pile head      =    2.62435E-17
Maximum bending moment           =     917084.611 lbs-in
Maximum shear force              =      -4115.234 lbs
Depth of maximum bending moment  =          0.000 in
Depth of maximum shear force     =        270.750 in
Number of iterations             =             25
Number of zero deflection points =             21
```

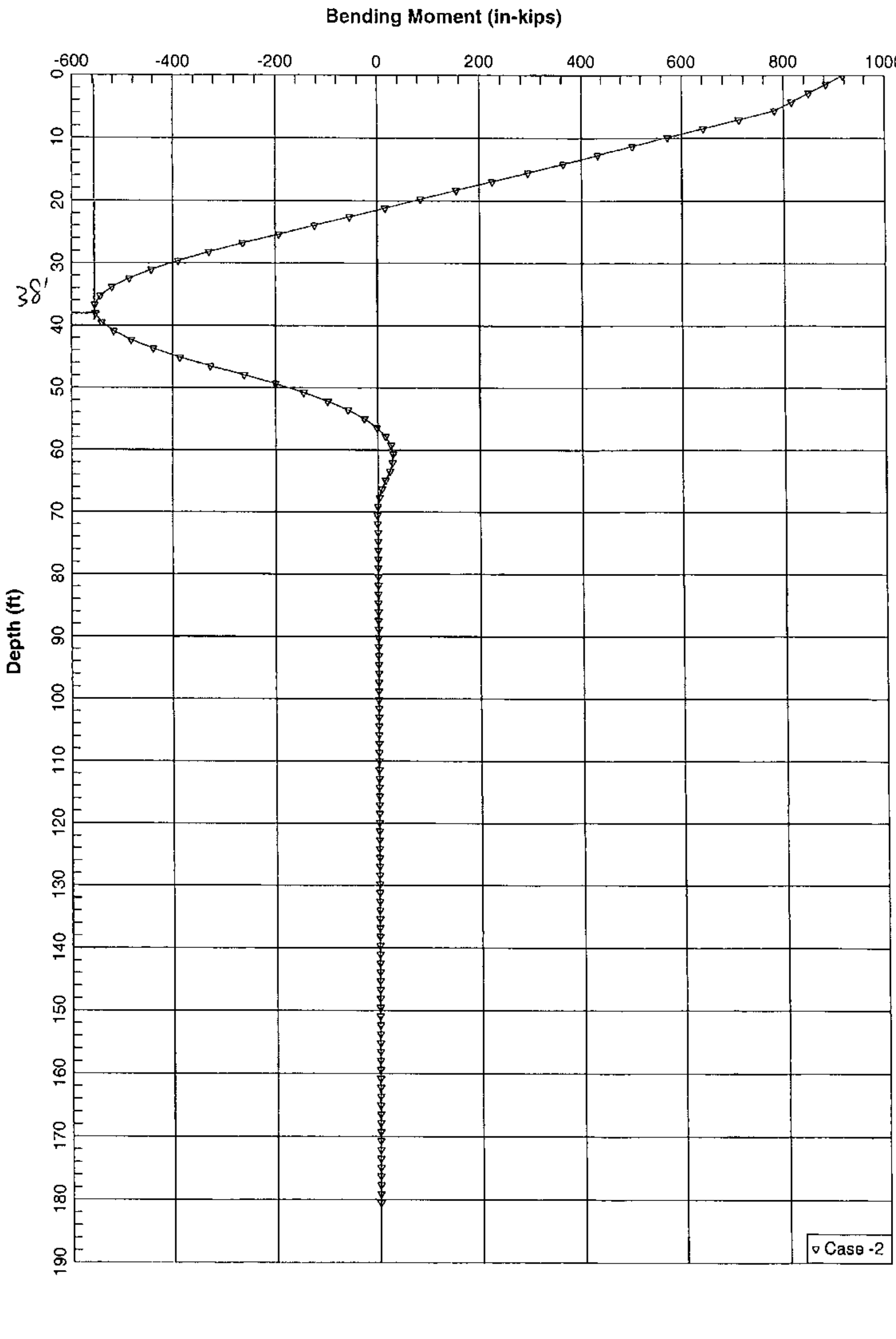
Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

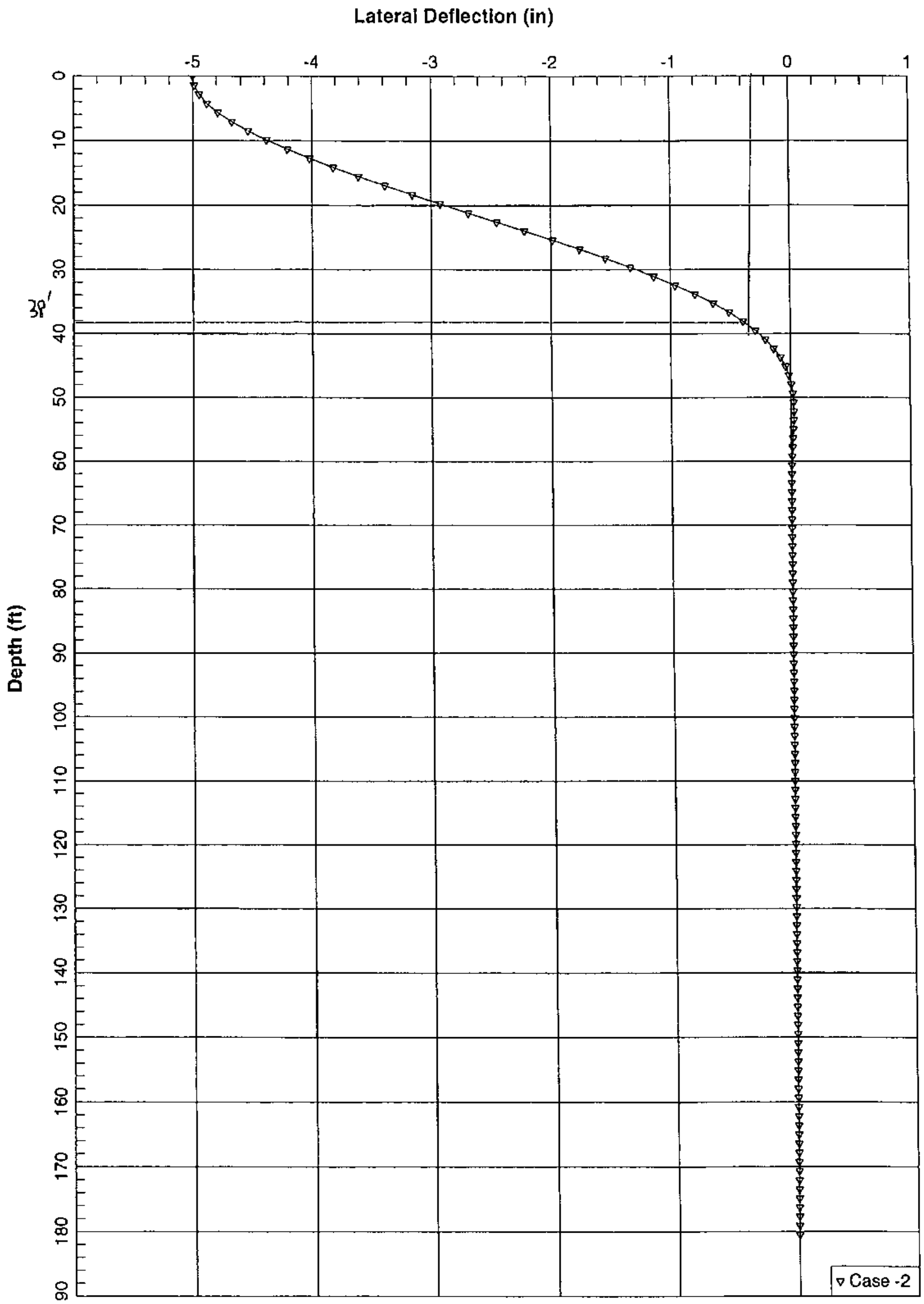
BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
2	V= -2000.000	S= 0.000	0.0000	-5.0017	917084.6109	-4115.2344

The analysis ended normally.



Bending Moment (in-kips)
-600
-400
-200
0
200
400
600
800
1000
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
38
Case -2

I10



Lateral Deflection (in)
-5
-4
-3
-2
-1
0
1
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
Case -2

I11

DEAD LOADS

Halcrow NOAM CALCULATION SHEET	No: 1	Rev:
Project: PIER 40 CONDITION SURVEY	By: JC	Date: 5/29/2009
Subject: DEAD LOAD CALCULATIONS FOR STEEL H-PILE LOAD RATING CALCS.	Check By:	Date: 5/30/2009

References/Results

PIER SHED

DEAD LOAD FOR THE PIER SHED INCLUDES THE DECK PLANKS WITH 2 IN. WEARING SURFACE, PILE CAPS, PILE CAPS BEAMS, AND SELF-WEIGHT OF PILE

ELEMENT	VOLUME	CONC. UNIT WEIGHT (KIPS/CF)	LOAD (KIPS)
DECK PLANKS	364.6	0.15	54.7
PILE CAPS	45.0	0.15	6.8
BEAMS	75.0	0.15	11.3
SELF-WEIGHT	-	-	2.9
TOTAL			**75.6**

TRIBUTARY AREA

SPAN 12.5 FT

WIDTH 25 FT — AREA 312.5 SF

DECK PLANKS

DECK THICKNESS 12 IN.

WEARING SURFACE 2 IN.

TOTAL THICKNESS 1.2 FT — VOLUME 364.6 CF

PILE CAPS

WIDTH 3 FT

LENGTH 3 FT

HEIGHT 5 FT — VOLUME 45.0 CF

PILE CAPS BEAMS

WIDTH 2 FT

HEIGHT 3 FT — VOLUME 75.0 CF

SELF-WEIGHT

PILE LENGTH 33 FT

LBS./FT 89 LBS./FT — LOAD 2.9 KIPS

Halcrow NOAM CALCULATION SHEET

No: 2	Rev:		
Project: PIER 40 CONDITION SURVEY	By: JC	Date: 5/29/2009	
Subject: DEAD LOAD CALCULATIONS FOR STEEL H-PILE LOAD RATING CALCS.	Check By:	Date: 5/30/2009	

References/Results

TRUCK COURT

DEAD LOAD FOR THE TRUCK COURT INCLUDES THE DECK PLANKS WITH 2 IN. WEARING SURFACE, PILE CAPS, PILE CAPS BEAMS, AND SELF-WEIGHT OF PILE

ELEMENT	VOLUME	CONC. UNIT WEIGHT (KIPS/CF)	LOAD (KIPS)
DECK PLANKS	504.7	0.15	75.7
PILE CAPS	45.0	0.15	6.8
BEAMS	123.6	0.15	18.5
SELF-WEIGHT	-	-	2.4
TOTAL			103.4

TRIBUTARY AREA

SPAN 20.6 FT

WIDTH 21 FT — AREA 432.6 SF

DECK PLANKS

DECK THICKNESS 12 IN.

WEARING SURFACE 2 IN.

TOTAL THICKNESS 1.2 FT — VOLUME 504.7 CF

PILE CAPS

WIDTH 3 FT

LENGTH 3 FT

HEIGHT 5 FT — VOLUME 45.0 CF

PILE CAPS BEAMS

WIDTH 2 FT

HEIGHT 3 FT — VOLUME 123.6 CF

SELF-WEIGHT

PILE LENGTH 27 FT

LBS./FT 89 LBS./FT — LOAD 2.4 KIPS

Halcrow — **NOAM CALCULATION SHEET**

Project: PIER 40 CONDITION SURVEY		No: 3	Rev:
Subject: DEAD LOAD CALCULATIONS FOR STEEL H-PILE LOAD RATING CALCS.		By: JC	Date: 5/29/2009
		Check By:	Date: 5/30/2009

References/Results

FINGER PIER

DEAD LOAD FOR THE TRUCK COURT INCLUDES THE DECK PLANKS WITH 2 IN. WEARING SURFACE, LONG. BEAM, TRANS. BEAMS, AND SELF-WEIGHT OF PILE

ELEMENT	VOLUME	CONC. UNIT WEIGHT (KIPS/CF)	LOAD (KIPS)
DECK PLANKS	262.5	0.15	39.4
LONG. BEAM	168.8	0.15	25.3
TRANS. BEAM	75.0	0.15	11.3
SELF-WEIGHT	-	-	4.5
TOTAL			80.4

TRIBUTARY AREA

SPAN	22.5	FT		
WIDTH	10	FT	AREA	225.0 SF

DECK PLANKS

DECK THICKNESS	12	IN.		
WEARING SURFACE	2	IN.		
TOTAL THICKNESS	1.2	FT	VOLUME	262.5 CF

LONG. BEAM

WIDTH	2.5	FT		
HEIGHT	3	FT	VOLUME	168.8 CF

TRANS. BEAM

WIDTH	2.5	FT		
HEIGHT	3	FT	VOLUME	75.0 CF

SELF-WEIGHT

PILE LENGTH	50	FT		
LBS./FT	89	LBS./FT	LOAD	4.5 KIPS

PILE LOAD RATINGS – PIER SHED

Halcrow

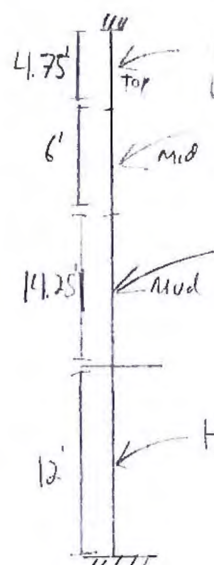
PROJECT: HRPT (Severe Pile Ag...)
SUBJECT: Pier Shed Compressive Capa...

SHEET NO. 1 OF
JOB NO.
MADE BY: BC DATE: 5/14
CHKD. BY: MWan DATE: 4/8/14

Calc. Global Buckling Capacity (severe piles)

Trib Area: 312 sqft ✓

Dead Load: 75.6 Kip ✓



Flange 0.274 ✓ Web: 0.378 ✓ A_s: 13.32 in² ✓

Flange .271 ✓ Web: .270 ✓ A_s = 11.54 in² ✓

Flange 0.425 ✓ Web 0.415 ✓ A_s: 18.62 ✓

Assume: 50% Moment transfer at the top

Fully fixed at the Mud line

Per Inspection: Minor axis buckling will not control as major axis is controlled by local buckling

From Stadd

Applied load -170 kip ✓

Buckling factor: 7.02 ✓

170 kip · 7.02 = 1193.4 kip ✓ (Euler Buckling Load)

Calc F_e @ Critical section

Critical section happends in the mud line section where buckling is critical (at mud pile)

A_s = 18.62 in² ✓

$$F_e = \frac{1193.4\,k}{18.62\,in^2} = 64.09\ ksi\ ✓$$

Halcrow

PROJECT HRPT
SUBJECT Pier Shed

SHEET NO. 2 OF
JOB NO.
MADE BY BC DATE 5/14
CHKD. BY Murdoch DATE 4/8/14

Find $\left(\frac{K}{r}\right)$ for system

$L = 444''$ ✓

$F_e = \frac{\pi^2 \cdot E}{\left(\frac{K \cdot L}{r}\right)^2} \Rightarrow \quad \frac{K}{r} = \frac{\sqrt{\frac{\pi^2 \cdot E}{F_e}}}{L} \Rightarrow 0.15$ ✓

$F_y = 36$ ksi ✓

$\frac{KL}{r} = 0.150 \cdot 444 = 66.82$ ✓

$4.71 \cdot \sqrt{\frac{29000}{36\ ksi}} = 133.7$ ✓

$\frac{KL}{r} < 4.71 \sqrt{\frac{29000}{36\ ksi}}$ ✓

AISC E3-2 $F_{cr} = \left[0.658^{36/64.0}\right] \cdot 36 = 28.4$ ✓

Pile will buckle at critical stress w/ $A_s = 18.62\ in^2$ ✓

$P_n = 28.4$ ksi $\cdot$ $18.62\ in^2$ $= 528.8$ kip ✓

$\Omega = 1.67$ ✓

$\boxed{P_n = 316.32 \text{ kip}}$ ✓

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PROJECT P/S

SUBJECT

SHEET NO. 3 OF

JOB NO.

MADE BY BC DATE 5/14

CHKD. BY Mushl DATE 4/8/14

Consider slender elements over the <u>Avg.</u> amount of section loss

Section is thinest at the top of the pile

$b = 14.7/2 = 7.35''$ ✓

$t = 0.274''$ ✓

$b/t = 26.8$ ✓

<u>AISC E-7</u>

$0.56 \cdot \sqrt{\frac{E}{F_y}} = 15.894$ ✓

$1.03 \cdot \sqrt{\frac{E}{F_y}} = 29.23$ ✓

$\underbrace{b/t}_{26.8} < 29.23 \quad \therefore$ Use (E 7-5)

$Q_s = 1.415 - 0.74 (\underbrace{b/t}_{26.8}) \cdot \underbrace{\sqrt{\frac{F_y}{E}}}_{0.035}$ ✓

$\boxed{Q_s = 0.716}$ ✓

Apply to AISC (E 3-2) - cons.

$F_{cr} = Q_s \left[0.658^{\frac{F_y}{F_e}} \right] \cdot F_y$ ✓

$= 21.78$ ksi ✓

Apply 'Q' factor @ critical section of pile

$A_s = 18.62\ in^2$ ✓

$P_n = 18.62\ in^2 \cdot 21.78$ ksi ✓

$P_n = 405.5$ kip ✓

$\boxed{\frac{P_n}{\Omega} = \frac{405.5 \text{ kip}}{1.67} = 242.8 \text{ kip}}$ ✓

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Tuesday, May 06, 2014, 11:53 AM

PAGE NO. 1

MB BC 5/14
CH MHall 4/8/14

```
****************************************************
*                                                  *
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*           Version  20.07.09.31                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    MAY  5, 2014                  *
*           Time=    12:13: 1                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
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    1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Avg(Severe).STD
    2. START JOB INFORMATION
    3. ENGINEER DATE 12-MARCH-2014
    4. END JOB INFORMATION
    5. INPUT WIDTH 79
    6. UNIT FEET KIP
    7. JOINT COORDINATES
    8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
    9. 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
   10. MEMBER INCIDENCES
   11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
   12. 25 19 20; 26 20 5
   13. START USER TABLE
   14. TABLE 2
   15. UNIT INCHES KIP
   16. WIDE FLANGE
   17. TOP
   18. 13.3299 13.8 0.398 14.7 0.274 445.686 145.131 0.480084 5.4924 8.0556
   19. MID
   20. 11.5471 13.8 0.27 14.7 0.271 417.059 143.495 0.28203 3.726 7.9674
   21. MUDLINE
   22. 18.6203 13.8 0.473 14.7 0.425 644.601 225.118 1.20911 6.5274 12.495
   23. 50%PILE
   24. 14.5475 13.8 0.575 13.8 0.25 429.481 109.714 0.986568 7.935 6.9
   25. 50%FLG12
   26. 13.6475 13.8 0.575 12 0.25 388.166 72.2107 0.967818 7.935 6
   27. 33%PILE
   28. 12.245 13.8 0.475 14.7 0.2 367.152 106.004 0.557101 6.555 5.88
   29. END
   30. UNIT FEET KIP
   31. DEFINE MATERIAL START
   32. ISOTROPIC STEEL
   33. E 4.176E+006
   34. POISSON 0.3
   35. DENSITY 0.489024
   36. ALPHA 6.5E-006
   37. DAMP 0.03
   38. END DEFINE MATERIAL
```

```
39. UNIT INCHES KIP
40. CONSTANTS
41. BETA 0 MEMB 8 9
42. MATERIAL STEEL ALL
43. MEMBER PROPERTY AMERICAN
44. 9 UPTABLE 2 TOP
45. 7 8 UPTABLE 2 MID
46. 20 TO 23 UPTABLE 2 MUDLINE
47. MEMBER PROPERTY AMERICAN
48. 4 24 TO 26 TABLE ST HP14X89
49. SUPPORTS
50. 4 FIXED
51. 10 FIXED BUT FY
52. MEMBER RELEASE
53. 9 END MPY .5 MPZ .5
54. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
55. JOINT LOAD
56. 10 FY -170
57. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	NUMBER OF MEMBERS	11
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 61

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	7.02252
2	13.31198
3	18.92311
4	26.81848

58. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 5,2014 TIME= 12:13: 1 ****

```
**************************************************************
*        For questions on STAAD.Pro, please contact          *
*        Bentley Systems or Partner offices                   *
*                                                             *
*             Telephone                Web / Email            *
*  USA        +1  (714) 974-2500                              *
*  UK         +44 (0) 808 101 9246                            *
*  SINGAPORE +65 6225-6158                                    *
*  FRANCE     +33 (0) 1 55238400                              *
*  GERMANY    +49 0931 40468                                  *
*  INDIA      +91 (033) 4006-2021                             *
*  JAPAN      +81 (03)5952-6500    http://www.ctc-g.co.jp     *
*  CHINA      +86 21 6288 4040                                *
*  THAILAND   +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                             *
*  Worldwide  http://selectservices.bentley.com/en-US/        *
*                                                             *
**************************************************************
```

CH2MHILL.

Job Name: HRPT (PS) Local Crushing [Assuming Avg Section]
Subject:

Job No.:
Sheet No.: 1
Date:
Computed By: BC 5/14
Checked By: MWM 4/8/14

Local Crushing: (Severe Pile)

w/ Avg Section [illegible]

F: 0.[illegible]

W: 0.[illegible]

A_s = 13.29 in² (From [illegible])

36 ksi × 13.29 in² = 478.44 kip

$\frac{P_n}{\Omega} = \frac{478.44 \text{ kip}}{1.67} = \boxed{286.49 \text{ kip}}$

Major Pile

F: 0.414

W: 0.441

A_s = 17.89

$\frac{P_n}{\Omega} = \boxed{385.6 \text{ kip}}$

Moderate Pile

F: 0.459

W: 0.442

A_s = 19.24 in²

$\frac{P_n}{\Omega} = \boxed{415.19 \text{ kip}}$

Minor Pile

F: 0.525

W: 0.433

A_s = 21.59 in²

$\frac{P_n}{\Omega} = \boxed{465.41 \text{ kip}}$

CH2MHILL.

Job No.

Sheet No. 1

Job Name HRPT Pier 40

Date

Subject P/S Lateral (Severe)

Computed By 5/14

Checked By MWahl 4/8/14

LC 3

$D_L + L_L + C + 0.3W + 0.3W_a$ ✓

12.5% Allowable Over Stress ✓

D_L } From axial analysis ✓

~~L_L~~ → 100psf ✓

Current ML [illegible] ✓

Typ [illegible] ✓

Wind ML 555 [illegible] ✓

Typ 135 [illegible] ✓

Wave: [illegible] ✓

Typ [illegible] ✓

Lateral U/R

LC 3]

Avg Severe pile [For Bending U/R see STAAD output]

U/R $\boxed{0.44}$ ✓ @ Typ

LC 5

$D_L + C + W + W_v$ ✓ Typ

Allowable Overstress 140% ✓

U/R [illegible] ✓ @ Typ

CH2MHILL.

Job Name HRPT Pier 40

Subject P/S Lateral Severe Pile

Job No.

Sheet No. 2

Date

Computed By BL 5/14

Checked By MWOM 4/8/14

LC 7

$D_L + E$

Allowable overstress: 133% ✓

X-axis: U/R 1.[illegible] @ Top ✓

Y-axis: U/R 1[illegible] @ Top ✓

LC 8

$D_L + L_L + Ice$

Allowable overstress: 140% ✓

U/R: [illegible] @ Top ✓

LC 9

$D_L + L_L + C + W + Ice$

Allowable overstress: 150% ✓

U/R: [illegible] @ Top ✓

Halcrow

PROJECT HRPT
SUBJECT P/S [illegible] Summary

SHEET NO. 4 OF
JOB NO.
MADE BY BC DATE 3/14
CHKD. BY. MWahl DATE 4/8/14

Assume 100 PSF L_L ✓

Trib area 312 sq ft ✓ 100 #/sq ft = 31.2 kip for L_L ✓

Total Axial load $L_L + D_L$ = 106.8 kip ✓

HRPT
Load
Case over
Stress factor

	(D_L+L_L)	U/R for Axial force (FLB Controls) 242.2 kip (Flange Local Buckling)
LC3 125% =	85.44 kip $\left(\frac{106.8}{1.25}\right)$ ✓	0.351 $\left(\frac{85.44}{242.2}\right)$ ✓
LC7 133% =	80.3 kip $\left(\frac{106.8}{1.33}\right)$ ✓	0.330 $\left(\frac{80.3}{242.2}\right)$ ✓
LC5 LC8 140% =	76.28 kip $\left(\frac{106.8}{1.40}\right)$ ✓	0.314 $\left(\frac{76.28}{242.2}\right)$ ✓
LC9 150% =	71.2 kip $\left(\frac{106.8}{1.5}\right)$ ✓	0.293 $\left(\frac{71.2}{242.2}\right)$ ✓
LC1 100% =	106.8 kip $\left(\frac{106.8}{1.0}\right)$ ✓	0.439 $\left(\frac{106.8}{242.2}\right)$ ✓

LC1: Assume 300 PSF Total Axial 169.2 [illegible] 300 PSF OK

USE
AISC
H 1-1A

LC3: Add [illegible]

[illegible] * [illegible] ✓ ✓ ✓ ✓

* Includes L/C [illegible] from [illegible] ✓

[illegible] ✓ [illegible] ✓ ok [illegible] 100 psf

calc?

LC5
U/R Dead [illegible] (see [illegible] PS)

0.~~0~~14 0.~~0~~49 [illegible] 0.8 ✓ ok w/ 100 psf

LC7 [illegible]

LC8 [illegible] 0.~~0~~38 [illegible] 1.04 ✓ NG ✓ [illegible]

LC9 0.~~0~~07 [illegible] 0.083 [illegible] 1.026 ✓ NG ✓
U/R Dead Moment [illegible]

CH2MHILL SUBJECT: Aditional Axial load due to lateral BY ______ SHEET NO. 5 of ______ DATE ______ PROJECT NO. ______

MB BC 5/14

Aditional Axial load

LC5

C + W + Wa

$1\,kip + 10.6\,kip + 5.2\,kip = \frac{(16.8\,kip \div 1.4)}{242.8} = 0.049$ ✓

U/R LC5

LC8

Ice

U/R LC8

$1.4 \div \left(\frac{13.2\,kip}{242.8\,kip}\right) = 0.038$ ✓

LC9

C + W + Ice

U/R LC9

$1 + 10.6 + 18.5 = \frac{(30.1 \div 1.5)}{242.8} = 0.083$ ✓

MB B 5/14
Ch Mwahl 4/8/14

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.31                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    APR 30, 2014                  *
*           Time=    13:31:43                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

Severe

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending_Pier Shed_LC3.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 19.7975 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
    10. MEMBER INCIDENCES
    11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
    12. 25 19 20; 26 20 5
    13. START USER TABLE
    14. TABLE 2
    15. UNIT INCHES KIP
    16. WIDE FLANGE
    17. 20%TOP
    18. 8.4175 13.8 0.35 14.7 0.125 244.377 66.226 0.212793 4.83 3.675
    19. 20%MID
    20. 7.49875 13.8 0.175 14.7 0.175 274.277 92.6546 0.0765497 2.415 5.145
    21. 20%MUDLINE
    22. 17.285 13.8 0.425 14.7 0.4 605.874 211.851 0.959851 5.865 11.76
    23. TOPAVG
    24. 13.3299 13.8 0.398 14.7 0.274 445.686 145.131 0.480084 5.4924 8.0556
    25. MIDAVG
    26. 11.5471 13.8 0.27 14.7 0.271 417.059 143.495 0.28203 3.726 7.9674
    27. MUDLINEAVG
    28. 18.6203 13.8 0.473 14.7 0.425 644.601 225.118 1.20911 6.5274 12.495
    29. END
    30. UNIT FEET KIP
    31. DEFINE MATERIAL START
    32. ISOTROPIC STEEL
    33. E 4.176E+006
    34. POISSON 0.3
    35. DENSITY 0.489024
    36. ALPHA 6.5E-006
    37. DAMP 0.03
    38. END DEFINE MATERIAL
```

```
39. UNIT INCHES KIP
40. CONSTANTS
41. BETA 0 MEMB 8 9
42. MATERIAL STEEL ALL
43. MEMBER PROPERTY AMERICAN
44. 9 UPTABLE 2 TOPAVG
45. 20 TO 23 UPTABLE 2 MUDLINEAVG
46. MEMBER PROPERTY AMERICAN
47. 4 24 TO 26 TABLE ST HP14X89
48. MEMBER PROPERTY AMERICAN
49. 7 8 UPTABLE 2 MIDAVG
50. SUPPORTS
51. 4 10 PINNED
52. LOAD 3 LOADTYPE WIND  TITLE WIND (MAJOR AXIS)
53. JOINT LOAD
54. 4 MZ 595
55. 10 MZ 493
56. LOAD 5 LOADTYPE FLUIDS  TITLE CURRENT (MAJOR AXIS)
57. JOINT LOAD
58. 4 MZ 49.2
59. 10 MZ 75.6
60. *MEMBER LOAD
61. *7 8 20 TO 23 UNI GZ 0.00157
62. LOAD 7 LOADTYPE WIND  TITLE WAVE (MAJOR AXIS)
63. JOINT LOAD
64. 4 MZ 458.4
65. 10 MZ 355.2
66. LOAD COMB 4 HRPT LOAD CASE 3
67. 3 0.24 5 0.8 7 0.24
68. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	NUMBER OF MEMBERS	11
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 3, TOTAL DEGREES OF FREEDOM = 66

```
***WARNING - INSTABILITY AT JOINT [illegible] DIRECTION [illegible]
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG=  6.5424866E+02 L-MATRIX DIAG= -1.7053026E-13 EQN NO      35
[illegible]
```

```
69. PARAMETER 1
70. CODE AISC
```

```
71. FYLD 36 ALL
72. LX 206.4 MEMB 7 TO 9 20 22
73. UNB 206.4 MEMB 7 TO 9 20 22
74. UNT 206.4 MEMB 7 TO 9 20 22
75. LX 237.6 MEMB 4 21 23 TO 26
76. UNB 237.6 MEMB 4 21 23 TO 26
77. UNT 237.4 MEMB 4 21 23 TO 26
78. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
```
***********************
```

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
4 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.211	3
		0.00 T	0.00	595.00	0.00
7 ST	MIDAVG		(UPT)		
		PASS	AISC- H1-3	0.321	3
		0.00 T	0.00	-331.27	63.00
8 ST	MIDAVG		(UPT)		
		PASS	AISC- H1-3	0.342	3
		0.00 T	0.00	-353.32	9.00
9 ST	TOPAVG		(UPT)		
		PASS	AISC- H1-3	0.444	3
		0.00 T	0.00	-493.00	57.00
20 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.090	3
		0.00 T	0.00	176.89	0.00
21 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.070	3
		0.00 T	0.00	-137.38	50.82
22 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.037	3
		0.00 T	0.00	72.14	0.00
23 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.123	3
		0.00 T	0.00	-242.14	42.75
24 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.179	3
		0.00 T	0.00	506.78	0.00
25 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.148	3
		0.00 T	0.00	418.57	0.00
26 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.117	3
		0.00 T	0.00	330.35	0.00

79. LOAD LIST
80. FINISH

```
*********** END OF THE STAAD.Pro RUN ***********

  **** DATE= APR 30,2014   TIME= 13:31:44 ****

************************************************************
*      For questions on STAAD.Pro, please contact          *
*      Bentley Systems or Partner offices                  *
*                                                          *
*            Telephone              Web / Email            *
* USA       +1  (714) 974-2500                             *
* UK        +44 (0) 808 101 9246                           *
* SINGAPORE +65 6225-6158                                  *
* FRANCE    +33 (0) 1 55238400                             *
* GERMANY   +49 0931 40468                                 *
* INDIA     +91 (033) 4006-2021                            *
* JAPAN     +81 (03)5952-6500    http://www.ctc-g.co.jp    *
* CHINA     +86 21 6288 4040                               *
* THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
* Worldwide  http://selectservices.bentley.com/en-US/      *
*                                                          *
************************************************************
```

MB 5/14
Ch MWall
4/8/14

severe

```
*****************************************************
*                                                   *
*          STAAD.Pro V8i SELECTseries4              *
*          Version  20.07.09.31                     *
*          Proprietary Program of                   *
*          Bentley Systems, Inc.                    *
*          Date=    APR 30, 2014                    *
*          Time=    13:31: 3                        *
*                                                   *
*      USER ID: CH2M HILL                           *
*****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending_Pier Shed_LC5.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 19.7975 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
    10. MEMBER INCIDENCES
    11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
    12. 25 19 20; 26 20 5
    13. START USER TABLE
    15. TABLE 2
    16. UNIT INCHES KIP
    17. WIDE FLANGE
    18. TOPAVG
    19. 13.3299 13.8 0.398 14.7 0.274 445.686 145.131 0.480084 5.4924 8.0556
    20. MIDAVG
    21. 11.5471 13.8 0.27 14.7 0.271 417.059 143.495 0.28203 3.726 7.9674
    22. MUDLINEAVG
    23. 18.6203 13.8 0.473 14.7 0.425 644.601 225.118 1.20911 6.5274 12.495
    25. END
    26. UNIT FEET KIP
    27. DEFINE MATERIAL START
    28. ISOTROPIC STEEL
    29. E 4.176E+006
    30. POISSON 0.3
    31. DENSITY 0.489024
    32. ALPHA 6.5E-006
    33. DAMP 0.03
    34. END DEFINE MATERIAL
    35. UNIT INCHES KIP
    36. CONSTANTS
    37. BETA 0 MEMB 8 9
    38  MATERIAL STEEL ALL
    39. MEMBER PROPERTY AMERICAN
    40. 9 UPTABLE 2 TOPAVG
```

```
42. 20 TO 23 UPTABLE 2 MUDLINEAVG
43. MEMBER PROPERTY AMERICAN
44. 4 24 TO 26 TABLE ST HP14X89
45. MEMBER PROPERTY AMERICAN
46. 7 8 UPTABLE 2 MIDAVG
47. SUPPORTS
48. 4 10 PINNED
49. LOAD 3 LOADTYPE WIND  TITLE WIND (MAJOR AXIS)
50. JOINT LOAD
51. 4 MZ 595
52. 10 MZ 493
53. LOAD 5 LOADTYPE FLUIDS  TITLE CURRENT (MAJOR AXIS)
54. JOINT LOAD
55. 4 MZ 49.2
56. 10 MZ 75.6
57. *MEMBER LOAD
58. *7 8 20 TO 23 UNI GZ 0.00157
59. LOAD 7 LOADTYPE WIND  TITLE WAVE (MAJOR AXIS)
60. JOINT LOAD
61. 4 MZ 458.4
62. 10 MZ 355.2
63. LOAD COMB 6 HRPT LOAD CASE 5
64. 3 0.714 5 0.714 7 0.714
65. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	NUMBER OF MEMBERS	11
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 3, TOTAL DEGREES OF FREEDOM = 66

```
***WARNING - INSTABILITY AT JOINT [illegible] DIRECTION = [illegible]
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG= 6.5424866E+02 L-MATRIX DIAG= -1.7053026E-13 EQN NO 35
[illegible]
```

```
66. PARAMETER 1
67. CODE AISC
68. FYLD 36 ALL
69. LX 206.4 MEMB 7 TO 9 20 22
70. UNB 206.4 MEMB 7 TO 9 20 22
71. UNT 206.4 MEMB 7 TO 9 20 22
72. LX 237.6 MEMB 4 21 23 TO 26
73. UNB 237.6 MEMB 4 21 23 TO 26
```

```
74. UNT 237.6 MEMB 4 21 23 TO 26
75. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
4 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.279	6
		0.00 T	0.00	787.26	0.00
7 ST	MIDAVG		(UPT)		
		PASS	AISC- H1-3	0.431	6
		0.00 T	0.00	-444.52	63.00
8 ST	MIDAVG		(UPT)		
		PASS	AISC- H1-3	0.459	6
		0.00 T	0.00	-473.85	9.00
9 ST	TOPAVG		(UPT)		
		PASS	AISC- H1-3	0.594	6
		0.00 T	0.00	-659.59	57.00
20 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.122	6
		0.00 T	0.00	239.22	0.00
21 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.091	6
		0.00 T	0.00	-178.70	50.82
22 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.051	6
		0.00 T	0.00	99.92	0.00
23 ST	MUDLINEAVG		(UPT)		
		PASS	AISC- H1-3	0.162	6
		0.00 T	0.00	-318.01	42.75
24 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.237	6
		0.00 T	0.00	669.94	0.00
25 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.196	6
		0.00 T	0.00	552.63	0.00
26 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1 3	0.154	6
		0.00 T	0.00	435.32	0.00

76. LOAD LIST
77. FINISH

```
           *********** END OF THE STAAD.Pro RUN ***********

             **** DATE= APR 30,2014   TIME= 13:31: 3 ****

      ************************************************************
      *          For questions on STAAD.Pro, please contact      *
      *          Bentley Systems or Partner offices              *
      *                                                          *
      *              Telephone              Web / Email          *
      *  USA        +1  (714) 974-2500                           *
      *  UK         +44 (0) 808 101 9246                         *
      *  SINGAPORE +65 6225-6158                                 *
      *  FRANCE     +33 (0) 1 55238400                           *
      *  GERMANY    +49 0931 40468                               *
      *  INDIA      +91 (033) 4006-2021                          *
      *  JAPAN      +81 (03)5952-6500     http://www.ctc-g.co.jp *
      *  CHINA      +86 21 6288 4040                             *
      *  THAILAND   +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
      *                                                          *
      *  Worldwide  http://selectservices.bentley.com/en-US/     *
      *                                                          *
      ************************************************************
```

MB BC 5/14
ch MWall
4/8/14

```
****************************************************
*                                                  *
*          STAAD.Pro V8i SELECTseries4             *
*          Version  20.07.09.31                    *
*          Proprietary Program of                  *
*          Bentley Systems, Inc.                   *
*          Date=    APR 30, 2014                   *
*          Time=    13:36:35                       *
*                                                  *
*     USER ID: CH2M HILL                           *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending_Pier Shed_LC8.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 20.4075 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
    10. MEMBER INCIDENCES
    11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
    12. 25 19 20; 26 20 5
    13. START USER TABLE
    15. TABLE 2
    16. UNIT INCHES KIP
    17. WIDE FLANGE
    18. TOP
    19. 13.3299 13.8 0.398 14.7 0.274 445.686 145.131 0 480084 5.4924 8.0556
    20. MID
    21. 11.5471 13.8 0.27 14.7 0.271 417.059 143.495 0.28203 3.726 7.9674
    22. MUDLINE
    23. 18.6203 13.8 0.473 14.7 0.425 644.601 225.118 1.20911 6.5274 12.495
    25. END
    26. UNIT FEET KIP
    27. DEFINE MATERIAL START
    28. ISOTROPIC STEEL
    29. E 4.176E+006
    30. POISSON 0.3
    31. DENSITY 0.489024
    32. ALPHA 6.5E-006
    33. DAMP 0.03
    34. END DEFINE MATERIAL
    35. UNIT INCHES KIP
    36. CONSTANTS
    37. BETA 0 MEMB 8 9
    38. MATERIAL STEEL ALL
    39. MEMBER PROPERTY AMERICAN
    40. 9 UPTABLE 2 TOP
```

```
42. 20 TO 23 UPTABLE 2 MUDLINE
43. MEMBER PROPERTY AMERICAN
44. 4 24 TO 26 TABLE ST HP14X89
45. MEMBER PROPERTY AMERICAN
46. 7 8 UPTABLE 2 MID
47. SUPPORTS
48. 4 10 PINNED
49. LOAD 4 LOADTYPE ICE  TITLE ICE (MAJOR AXIS)
50. JOINT LOAD
51. 4 MZ 1146
52. 10 MZ 861.6
53. LOAD COMB 7 HRPT LOAD CASE 8
54. 4 0.714
55. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	NUMBER OF MEMBERS	11
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 66

```
***WARNING - INSTABILITY AT JOINT [illegible] DIRECTION = [illegible]
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG=  7.2487886E+02 L-MATRIX DIAG=  5.6843419E-14 EQN NO       35
[illegible]
```

```
56. PARAMETER 1
57. CODE AISC
58. FYLD 36 ALL
59. LOAD LIST
60. PARAMETER 2
61. CODE AISC
62. LX 244.8 MEMB 4 21 23 TO 26
63. UNB 244.8 MEMB 4 21 23 TO 26
64. UNT 244.8 MEMB 4 21 23 TO 26
65. LX 199.1 MEMB 7 TO 9 20 22
66. UNB 199.1 MEMB 7 TO 9 20 22
67. UNT 199.1 MEMB 7 TO 9 20 22
68. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
4 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.406	4
		0.00 T	0.00	1146.00	0.00
7 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.546	4
		0.00 T	0.00	-563.17	63.00
8 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.585	4
		0.00 T	0.00	-603.87	9.00
9 ST	TOP		(UPT)		
		PASS	AISC- H1-3	0.776	4
		0.00 T	0.00	-861.60	57.00
20 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.142	4
		0.00 T	0.00	278.31	0.00
21 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.154	4
		0.00 T	0.00	-301.59	58.14
22 ST	MUDLINE		(UPT)		
		PASS	SHEAR -Y	0.048	4
		0.00 T	0.00	85.01	0.00
23 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.252	4
		0.00 T	0.00	-494.89	42.75
24 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.348	4
		0.00 T	0.00	983.22	0.00
25 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.291	4
		0.00 T	0.00	820.44	0.00
26 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.233	4
		0.00 T	0.00	657.66	0.00

69. FINISH

```
*********** END OF THE STAAD.Pro RUN ***********

     **** DATE= APR 30,2014   TIME= 13:36:36 ****

************************************************************
*      For questions on STAAD.Pro, please contact          *
*      Bentley Systems or Partner offices                  *
*                                                          *
*            Telephone              Web / Email            *
*  USA       +1  (714) 974-2500                            *
*  UK        +44 (0) 808 101 9246                          *
*  SINGAPORE +65 6225-6158                                 *
*  FRANCE    +33 (0) 1 55238400                            *
*  GERMANY   +49 0931 40468                                *
*  INDIA     +91 (033) 4006-2021                           *
*  JAPAN     +81 (03)5952-6500    http://www.ctc-g.co.jp   *
*  CHINA     +86 21 6288 4040                              *
*  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
*  Worldwide  http://selectservices.bentley.com/en-US/     *
*                                                          *
************************************************************
```

CH2MHILL.

Job No.
Sheet No. 1
Job Name HRPT
Date
Subject PS Mayor Pile (Euler Buckling)
Computed By BC 5/14
Checked By MWAM 4/8/14

Calc Global Buckling Major Capacity

Trib area: 312 ft² √

DL: 75.6 kip √

Top	Mid	Mud Line
F: .414" √	.227" √	.497 √
W: .441" √	.281" √	.478 √

From STADD

Applied load: −170 kip √

BF = 8.23 √

Euler Buckling load = 1399.1 kip √

Calc Fe @ critical section @ Node 15 (Mudline pile section)

$A_s = 20.73 \, in^2$ √

$f_e = \frac{1399.1 \, kip}{20.285 \, in^2} = 68.97 \, ksi$ √

$L = 444"$ √

$\frac{k}{r} = \frac{\sqrt{\frac{\pi^2 E}{F_e}}}{L} \Rightarrow 0.145$ √

$F_y = 36 \, ksi$ √

$\frac{KL}{r} = 64.38$ √

Critical section √ $A_s = 20.73 \, in^2$

$F_{cr} = \left[0.658^{36/68.97} \right] \cdot 36 = 28.9 \, ksi$ √

$\frac{P_n}{\Omega} = 28.9 \, ksi \cdot 20.73 \, in^2 = \boxed{359.17 \, kip}$ √

CH2MHILL.

Job No. ____

Sheet No. 2

Job Name HRPT Pier 40

Date ____

Subject PS Major Pile Local Buckling

Computed By BC 5/14

Checked By MWahl 4/8/14

Local buckling "will happen" @ top of pile (Most critical ...)

$b = 7.35''$ ✓

$t = .414''$ ✓

$b/t = \frac{7.35}{.414} = 17.75$ ✓

$\lambda_r = 0.56 \cdot \sqrt{\frac{E}{F_y}} => 15.89 < b/t$ ✓ ∴ Flange @ top is slender ✓

Use AISC (E 7-5)

$Q_s = 1.415 - 0.74 \cdot (b/t) \cdot \sqrt{\frac{F_y}{E}}$

$= 1.415 - 0.74 \cdot (17.75) \cdot \sqrt{\frac{35}{29000}}$

$= 0.96$ ✓

Apply Q to (E 3-2) ✓

Apply Q @ critical

$F_{cr} = Q_s \left[0.658^{\left(\frac{Q F_y}{F_e}\right)}\right] F_y$ ✓ Conserv. Section $A_s = 20.77 in^2$ ✓

$= 27.778$ ksi ✓

$\frac{P_n}{\Omega} = \boxed{344.8 \text{ kip}}$ ✓

CH2MHILL.

Job No. ____________
Sheet No. ____________
Job Name: HRPT
Date: ____________
Subject: PS Major Pile lateral
Computed By: ~~[illegible]~~ BC 5/14
Checked By: MWahl 4/8/14

L/C 3

Ok w/ sauce pile ✓

L/C 5

Ok w/ sauce pile ✓

L/C 7

Y Axis

Fails @ Mid pile ✓

U/R = [illegible] ✓

X-Axis

Fails @ Mid pile

U/R @ 1.464 ✓

L/C 8

[illegible] By [illegible] ✓

[illegible] ✓

[illegible] ✓

[illegible] 0.055 [illegible] 0.125 ok ✓

[illegible] ✓

U/R w/ 100 PSF L_L

D_L = 75.6 kip ✓

L_L w/100 psf = [illegible] kip ✓

Total Axial [illegible] ✓

	D_L +L_L	U/R
140%	76.22 kip ✓	0.221 ✓
150%	71.2 kip ✓	[illegible] ✓

FLB controls 344.8 kip

MB B 5/14
ch MWahl
4/8/14

```
****************************************************
*                                                  *
*          STAAD.Pro V8i SELECTseries4             *
*          Version  20.07.09.31                    *
*          Proprietary Program of                  *
*          Bentley Systems, Inc.                   *
*          Date=    MAY  2, 2014                   *
*          Time=    11:55:19                       *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Major.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
    10. MEMBER INCIDENCES
    11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
    12. 25 19 20; 26 20 5
    13. START USER TABLE
    14. TABLE 2
    15. UNIT INCHES KIP
    16. WIDE FLANGE
    17. TOP
    18. 17.8923 13.8 0.441 14.7 0.414 625.635 219.273 1.06624 6.0858 12.1716
    19. MID
    20. 10.4908 13.8 0.286 14.7 0.227 364.056 120.204 0.218702 3.9468 6.6738
    21. MUDLINE
    22. 20.7331 13.8 0.478 14.7 0.497 730.417 263.239 1.66929 6.5964 14.6118
    23. END
    24. UNIT FEET KIP
    25. DEFINE MATERIAL START
    26. ISOTROPIC STEEL
    27. E 4.176E+006
    28. POISSON 0.3
    29  DENSITY 0.489024
    30. ALPHA 6.5E-006
    31. DAMP 0.03
    32. END DEFINE MATERIAL
    33. UNIT INCHES KIP
    34. CONSTANTS
    35. BETA 0 MEMB 8 9
    36. MATERIAL STEEL ALL
    37. MEMBER PROPERTY AMERICAN
    38. 9 UPTABLE 2 TOP
```

```
39. *9 UPTABLE 2 50%PILE
40. *9 UPTABLE 2 33%PILE
41. *9 UPTABLE 2 50%FLG12
42. 7 8 UPTABLE 2 MID
43. 20 TO 23 UPTABLE 2 MUDLINE
44. MEMBER PROPERTY AMERICAN
45. 4 24 TO 26 TABLE ST HP14X89
46. SUPPORTS
47. 4 FIXED
48. 10 FIXED BUT FY
49. MEMBER RELEASE
50. 9 END MPY 0.5 MPZ 0.5
51. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
52. JOINT LOAD
53. 10 FY -169.2
54. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	NUMBER OF MEMBERS	11
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 61

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	8.23859
2	14.45484
3	21.40278
4	29.36024

55. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 2,2014 TIME= 11:55:19 ****

```
 ***********************************************************
 *        For questions on STAAD.Pro, please contact       *
 *        Bentley Systems or Partner offices               *
 *                                                         *
 *            Telephone            Web / Email             *
 *  USA       +1  (714) 974-2500                           *
 *  UK        +44 (0) 808 101 9246                         *
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 *  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
 *                                                         *
 *  Worldwide  http://selectservices.bentley.com/en-US/    *
 *                                                         *
 ***********************************************************
```

MB Bz 5/14

Ch HWW 4/8/14

Major Pile

```
****************************************************
*                                                  *
*          STAAD.Pro V8i SELECTseries4             *
*          Version  20.07.09.31                    *
*          Proprietary Program of                  *
*          Bentley Systems, Inc.                   *
*          Date=    APR 30, 2014                   *
*          Time=    14:59: 6                       *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending_Pier Shed_LC9.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 19.7975 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
    10. MEMBER INCIDENCES
    11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
    12. 25 19 20; 26 20 5
    13. START USER TABLE
    14. TABLE 2
    15. UNIT INCHES KIP
    16. WIDE FLANGE
    17. TOP
    18. 17.8923 13.8 0.441 14.7 0.414 625.635 219.273 1.06624 6.0858 12.1716
    19. MID
    20. 10.2505 13.8 0.268 14.7 0.227 360.49 120.2 0.200263 3.6984 6.6738
    21. MUDLINE
    22. 20.7331 13.8 0.478 14.7 0.497 730.417 263.239 1.66929 6.5964 14.6118
    23. END
    24. UNIT FEET KIP
    25. DEFINE MATERIAL START
    26. ISOTROPIC STEEL
    27. E 4.176E+006
    28. POISSON 0.3
    29. DENSITY 0.489024
    30. ALPHA 6.5E-006
    31. DAMP 0.03
    32. END DEFINE MATERIAL
    33. UNIT INCHES KIP
    34. CONSTANTS
    35. BETA 0 MEMB 8 9
    36. MATERIAL STEEL ALL
    37. MEMBER PROPERTY AMERICAN
    38. 9 UPTABLE 2 TOP
```

```
40. 20 TO 23 UPTABLE 2 MUDLINE
41. MEMBER PROPERTY AMERICAN
42. 4 24 TO 26 TABLE ST HP14X89
43. MEMBER PROPERTY AMERICAN
44. 7 8 UPTABLE 2 MID
45. SUPPORTS
46. 4 10 PINNED
47. LOAD 3 LOADTYPE WIND  TITLE WIND (MAJOR AXIS)
48. JOINT LOAD
49. 4 MZ 595.2
50. 10 MZ 463.2
51. LOAD 4 LOADTYPE ICE  TITLE ICE (MAJOR AXIS)
52. JOINT LOAD
53. 4 MZ 1146
54. 10 MZ 861.6
55. LOAD 5 LOADTYPE FLUIDS  TITLE CURRENT (MAJOR AXIS)
56. JOINT LOAD
57. 4 MZ 75.6
58. 10 MZ 49.2
59. LOAD COMB 9 HRPT LOAD CASE 9
60. 5 0.67 3 0.67 4 0.67
61. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	**NUMBER OF MEMBERS**	11
NUMBER OF PLATES	0	**NUMBER OF SOLIDS**	0
NUMBER OF SURFACES	0	**NUMBER OF SUPPORTS**	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 3, TOTAL DEGREES OF FREEDOM = 66

```
***WARNING - INSTABILITY AT JOINT [illegible] DIRECTION = MX
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG=  9.0325176E+02 L-MATRIX DIAG=  1.1368684E-13 EQN NO      35
[illegible]
```

```
62. PARAMETER 1
63. CODE AISC
64. FYLD 36 ALL
65. LOAD LIST
66. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

```
MEMBER     TABLE        RESULT/   CRITICAL COND/     RATIO/     LOADING/
                          FX          MY              MZ        LOCATION
=======================================================================

   4 ST   HP14X89                  (AISC SECTIONS)
                         PASS      AISC- H1-3        0.400          9
                        0.00 T        0.00         1217.26        0.00
   7 ST   MID                      (UPT)
                         PASS      AISC- H1-3        0.771          9
                        0.00 T        0.00         -602.79       63.00
   8 ST   MID                      (UPT)
                         PASS      AISC- H1-3        0.827          9
                        0.00 T        0.00         -646.13        9.00
   9 ST   TOP                      (UPT)
                         PASS      AISC- H1-3        0.488          9
                        0.00 T        0.00         -920.58       57.00
  20 ST   MUDLINE                  (UPT)
                         PASS      AISC- H1-3        0.128          9
                        0.00 T        0.00          299.45        0.00
  21 ST   MUDLINE                  (UPT)
                         PASS      AISC- H1-3        0.136          9
                        0.00 T        0.00          318.06       50.82
  22 ST   MUDLINE                  (UPT)
                         PASS      SHEAR  -Y         0.051          9
                        0.00 T        0.00           93.61        0.00
  23 ST   MUDLINE                  (UPT)
                         PASS      AISC- H1-3        0.224          9
                        0.00 T        0.00         -523.90       42.75
  24 ST   HP14X89                  (AISC SECTIONS)
                         PASS      AISC- H1-3        0.343          9
                        0.00 T        0.00         1043.92        0.00
  25 ST   HP14X89                  (AISC SECTIONS)
                         PASS      AISC- H1-3        0.286          9
                        0.00 T        0.00          870.58        0.00
  26 ST   HP14X89                  (AISC SECTIONS)
                         PASS      AISC- H1-3        0.229          9
                        0.00 T        0.00          697.24        0.00

  67. PARAMETER 2
  68. CODE AISC
  69. LX 206 MEMB 7 TO 9 20 22
  70. UNB 206 MEMB 7 TO 9 20 22
  71. UNT 206 MEMB 7 TO 9 20 22
  72. LX 237.58 MEMB 4 21 23 TO 26
  73. UNB 237.58 MEMB 4 21 23 TO 26
  74. UNT 237.58 MEMB 4 21 23 TO 26
  75. FINISH
```

```
*********** END OF THE STAAD.Pro RUN ***********

  **** DATE= APR 30,2014   TIME= 14:59: 6 ****

************************************************************
*        For questions on STAAD.Pro, please contact        *
*        Bentley Systems or Partner offices                *
*                                                          *
*              Telephone              Web / Email          *
* USA        +1  (714) 974-2500                            *
* UK         +44 (0) 808 101 9246                          *
* SINGAPORE +65 6225-6158                                  *
* FRANCE     +33 (0) 1 55238400                            *
* GERMANY    +49 0931 40468                                *
* INDIA      +91 (033) 4006-2021                           *
* JAPAN      +81 (03)5952-6500    http://www.ctc-g.co.jp   *
* CHINA      +86 21 6288 4040                              *
* THAILAND   +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
* Worldwide  http://selectservices.bentley.com/en-US/      *
*                                                          *
************************************************************
```

CH2MHILL.

Job Name: HRPT Pier 40	Job No.
Subject: PS Moderate Pile (Euler Buckling)	Sheet No. 1
	Date
	Computed By: [illegible] 5/14
	Checked By: MWoM 4/3/14

Calc G B Of Pile (Moderate Pile)

Top	Mid	Bottom
F: .459"	F .325"	F: [illegible]
W: .448"	W .257"	W: .472
A_s = 19.27 sq in	A_s = 12.93 in²	A_s = [illegible]

From stadd

Applied Load -170 kip

BF [illegible]

E. Buckling Load = [illegible] kip

F_e @ Critical section (Node 15)

F_e = [illegible] = [illegible]

L = 444"

$\frac{K}{r}$ = [illegible]

$\frac{KL}{r}$ = [illegible]

F_{cr} = [illegible]

$\frac{P_n}{\Omega}$ = [illegible] kip

MB BC 5/14
CH MWan1

```
 ****************************************************
 *                                                  *
 *           STAAD.Pro V8i SELECTseries4            *
 *           Version  20.07.09.31                   *
 *           Proprietary Program of                 *
 *           Bentley Systems, Inc.                  *
 *           Date=    MAY  2, 2014                  *
 *           Time=    12: 6:55                      *
 *                                                  *
 *      USER ID: CH2M HILL                          *
 ****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed_Moderate.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
    10. MEMBER INCIDENCES
    11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
    12. 25 19 20; 26 20 5
    13. START USER TABLE
    14. TABLE 2
    15. UNIT INCHES KIP
    16. WIDE FLANGE
    17. TOP
    18. 19.2657 13.8 0.448 14.7 0.459 680.495 243.101 1.33378 6.1824 13.4946
    19. MID
    20. 12.9345 13.8 0.257 14.7 0.325 482.523 172.08 0.410821 3.5466 9.555
    21. MUDLINE
    22. 20.827 13.8 0.472 14.7 0.503 736.36 266.411 1.69563 6.5136 14.7882
    23. END
    24. UNIT FEET KIP
    25. DEFINE MATERIAL START
    26. ISOTROPIC STEEL
    27. E 4.176E+006
    28. POISSON 0.3
    29. DENSITY 0.489024
    30. ALPHA 6.5E-006
    31. DAMP 0.03
    32. END DEFINE MATERIAL
    33. UNIT INCHES KIP
    34. CONSTANTS
    35. BETA 0 MEMB 8 9
    36. MATERIAL STEEL ALL
    37. MEMBER PROPERTY AMERICAN
    38. 9 UPTABLE 2 TOP
```

```
39. *9 UPTABLE 2 50%PILE
40. *9 UPTABLE 2 33%PILE
41. *9 UPTABLE 2 50%FLG12
42. 7 8 UPTABLE 2 MID
43. 20 TO 23 UPTABLE 2 MUDLINE
44. MEMBER PROPERTY AMERICAN
45. 4 24 TO 26 TABLE ST HP14X89
46. SUPPORTS
47. 4 FIXED
48. 10 FIXED BUT FY
49. MEMBER RELEASE
50. 9 END MPY 0.5 MPZ 0.5
51. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
52. JOINT LOAD
53. 10 FY -169.2
54. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	NUMBER OF MEMBERS	11
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 61

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	8.65055
2	16.36930
3	22.07895
4	32.08441

55. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 2,2014 TIME= 12: 6:56 ****

```
***********************************************************
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*                                                         *
***********************************************************
```

CH2MHILL.

Job Name: HRPT Pier 40

Subject: PS Minor Pile (EB)

Job No.:

Sheet No.: 1

Date:

Computed By: BC 5/14

Checked By: MWdh 4/8/14

Calc GB of Pile (Minor)

Section

Top	Mid	Bottom
F: .525" ✓	F: .225" ✓	F: .484 ✓
W: .483" ✓	W: .322" ✓	W: .465 ✓
$A_s = 21.59\ in^2$ ✓	$A_s = 10.88\ in^2$ ✓	$A_s = 20.45\ in^2$ ✓

From STADD

BF = 8.54 ✓ (Buckling Factor)

E. Buckling Load = 1451.8 kip ✓

Critical Section (Node 15) (at mudline)

$A_s = 20.45\ in^2$ ✓

$F_e = 70.99$ ksi ✓

$\frac{K}{r} = .143$ ✓, $\frac{KL}{r} = 63.49$ ✓ Use (E3-2)

$F_{cr} = 29.11$ ksi ✓

$\boxed{\frac{P}{\Omega} = 356.53\ kip}$ ✓

Check Flange Slenderness @ thinnest point (Assume repair only)

$b/t = \frac{7.35}{.484} = 15.186$ ✓ $< \lambda_r$ → 15.39 ✓ ∴ Flange is not slender ✓

MB
DC 5/14
Ch. MWahl
4/8/14

```
****************************************************
*                                                  *
*          STAAD.Pro V8i SELECTseries4             *
*          Version  20.07.09.31                    *
*          Proprietary Program of                  *
*          Bentley Systems, Inc.                   *
*          Date=    APR 16, 2014                   *
*          Time=    14:38:19                       *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
    1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Minor.STD
    2. START JOB INFORMATION
    3. ENGINEER DATE 12 MARCH-2014
    4. END JOB INFORMATION
    5. INPUT WIDTH 79
    6. UNIT FEET KIP
    7. JOINT COORDINATES
    8. 4 0 0 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
    9. 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0; 18 0 3 0; 19 0 6 0; 20 0 9 0
   10. MEMBER INCIDENCES
   11. 4 4 18; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5; 24 18 19
   12. 25 19 20; 26 20 5
   13. START USER TABLE
   14. TABLE 2
   15. UNIT INCHES KIP
   16. WIDE FLANGE
   17. TOP
   18  21.5932 13.8 0.483 14.7 0.525 763.79 278.065 1.89697 6.6654 15.435
   19. MID
   20. 10.8849 13.8 0.322 14.7 0.224 367.344 118.627 0.258737 4.4436 6.5856
   21. MUDLINE
   22. 20 4531 13.8 0.485 14.7 0.484 716.458 256.362 1.5991 6.693 14.2296
   23. END
   24. UNIT FEET KIP
   25. DEFINE MATERIAL START
   26. ISOTROPIC STEEL
   27. E 4.176E+006
   28. POISSON 0.3
   29. DENSITY 0.489024
   30  ALPHA 6.5E-006
   31  DAMP 0.03
   32. END DEFINE MATERIAL
   33. UNIT INCHES KIP
   34. CONSTANTS
   35. BETA 0 MEMB 8 9
   36. MATERIAL STEEL ALL
   37. MEMBER PROPERTY AMERICAN
   38. 9 UPTABLE 2 TOP
```

```
39. 7 8 UPTABLE 2 MID
40. 20 TO 23 UPTABLE 2 MUDLINE
41. MEMBER PROPERTY AMERICAN
42. 4 24 TO 26 TABLE ST HP14X89
43. SUPPORTS
44. 4 FIXED
45. 10 FIXED BUT FY
46. MEMBER RELEASE
47. 9 END MPY 0.5 MPZ 0.5
48. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
49. JOINT LOAD
50. 10 FY -169.2
51. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	12	NUMBER OF MEMBERS	11
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 61

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	8.54468
2	15.04405
3	22.22354
4	30.02812

52. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= APR 16,2014 TIME= 14:38:19 ****

```
************************************************************
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*                                                          *
************************************************************
```

CH2MHILL SUBJECT ____ BY ____
SHEET NO. ____ of ____ DATE ____
PROJECT NO. ____

MB BC 5/16

Pier Shed Short EL

Original Calcs took Pile length to be [illegible] (longest pile + 12')
This is too Conservative for most of the Pier Shed
The max mesured length on the first third of the Pier is 19 which gives a total Pile length of 31

Same Cross sectional properties as previous severe pile
Same assumptions as before.
Applied load -170 kip

From Stadd

Buckling factor 9.54

$170k \cdot 10.57 = 1621.8\,k$ (Euler Buckling)

Calc F_e @ Critical Section

$A_s = 18.62\,in^2$

$$\frac{1621.8\,k}{18.62\,in^2} = 87.1\,ksi = f_e$$

$$\frac{Kl}{r} = \sqrt{\frac{\pi^2 \cdot E}{F_e}} = 57.32$$

$$F_{cr} = \left[0.658^{36/87.1}\right] 36 = 30.28\,ksi$$

Pile will buckle at critical section $A_s = 18.62\,in^2$

$$P_n = \frac{30.28\,ksi \cdot 18.62\,in}{1.67} = 337.625$$

CH2MHILL SUBJECT ____ BY ____
SHEET NO. ____ of ____ DATE ____
PROJECT NO. ____

MB: DC 5/16

Section loss at the top of the pile will be the same with the short and long piles

$\therefore \frac{b}{t}$ ratio will be the same

$Q_s = 0.716$

Apply to (E 3-2)

$F_{cr} = Q_s \cdot \left[0.658^{Q F_y / F_e}\right] \cdot F_y$

$= 0.716 \cdot \left[0.658^{\frac{.716 \cdot 36}{87.1}}\right] \cdot 36 = 22.77$ ksi

$P_n = \frac{18.62 \cdot 22.77 \text{ ksi}}{1.67} = 253.913$ kip

LC 8

U/R Axial

$\left(\underbrace{\frac{106.8 \text{ k}}{253.91 \text{ k}}}_{.301} \cdot 1.4\right) + 0.037 + \left(8/9 \cdot \overbrace{.776}^{.689}\right) = 1.027$

LC 9

$\left(\underbrace{\frac{75.6 \text{ k}}{253.91 \text{ k}}}_{0.198} \cdot 1.5\right) + \quad + \left(8/9 \cdot \overbrace{.829}^{0.736}\right) = 1.013$

PILE LOAD RATINGS – TRUCK COURT

Halcrow

PROJECT HRPT Truck Court
SUBJECT Compressive Capacity (Severe)

SHEET NO. 1 OF
JOB NO.
MADE BY BC DATE 4/8/14
CHKD. BY MWahl DATE 4/8/14

Calc Global Buckling Capacity (Severe)

Trib area: 432 sqft ✓

Dead load: 103 kip ✓

El.: 30' ✓

(Section in Model)

Top

F: 0.400" ✓

W: 0.425" ✓

$A_s = 17.285 \text{ in}^2$ ✓

Assume 50% Moment release at the top of pile. Full fixed at the mudline ✓

Mid

F: 0.200 ✓

W: 0.225 ✓

Mud

F: 0.400 ✓

W: 0.500 ✓

$A_s = 18.26 \text{ in}^2$ ✓

From Staad:

Applied load: 167.8 kip ✓

Buckling Factor: 10.51 ✓ - see Staad output

Euler Buckling Load →
$$\frac{1763.58 \text{ kip}}{18.26 \text{ in}^2} = 96.58^{ksi} = f_c \checkmark$$

Find Approx k/r

$L = 360''$ ✓

$$\frac{k}{r} = \frac{\sqrt{\frac{\pi^2 \cdot E}{F_c}}}{L} = 0.151 \checkmark$$

Halcrow

PROJECT HRPT TC
SUBJECT Axial

SHEET NO. ______ OF ______
JOB NO. ______
MADE BY Bz DATE 5/8/14
CHKD. BY MWAN DATE 4/2/14

$\frac{KL}{r} = 360'' \cdot 0.151 = 54.36 < 4.71 \cdot \sqrt{\frac{29{,}000}{36}} = 133.7$

USE AISC E3-2

$F_{cr} = \left[0.658^{36/96.58}\right] 36 = 30.79\ ksi$

Pile will buckle at critical section $A_s = 18.26\ in^2$ (Bottom Section)

$P_n = 18.26\ in^2 \cdot 30.79\ ksi = 562.4\ kip$

$\boxed{\frac{P_n}{\Omega} = 336.76\ kip}$ $\Omega = 1.67$

Consider Slender Elements at the top of the pile

$b = 7.35''$ (A Avg. flange reading is used as FIB will happen over a larger area of the flange)

$t = 0.45''$

$b/t = 16.3$

$b/t > 0.56 \times \sqrt{\frac{29000}{36}} = 15.9$

$b/t < 1.03 \cdot \sqrt{\frac{29000}{36}} = 29.2$

Apply (E 7-5)

$Q_s = 1.415 - 0.74 \left(\overset{16.3}{b/t}\right) \cdot \overset{.035}{\sqrt{\frac{F_y}{E}}}$

$Q_s = .992$

Apply to Eqn E 3-2

$F_{cr} = .992 \left[0.658^{.992 \cdot 36/96.58}\right] 36$

$F_{cr} = 30.6\ ksi$

A_s @ bot of the of Pile = $18.26\ in^2$

$P_n = 558.75\ kip$ $\boxed{\frac{P_n}{\Omega} = 334.6\ kip}$

conserv as global and local buckling will not occur in same location

www.halcrow.com

Halcrow

PROJECT	T/C	SHEET NO.	3 OF
SUBJECT		JOB NO.	
		MADE BY	[illegible] DATE 4/8/14
		CHKD. BY	[illegible] DATE 4/8/14

Check Local crushing of min section of the flange

$A_s = 17.285 \text{ in}^2$ √

$P_n = 17.285 \text{ in} \times 36 \text{ ksi} = 622.26$ √

$\frac{P_n}{\Omega} = 372.6 \text{ kip}$ √

Local Buckling Controls √

Capacity 334.6 kip √

Allowable Live load: $\underbrace{334.6 \text{ k}}_{\text{Capacity}} - \underbrace{103 \text{ kip}}_{D_L} = 231.6 \text{ kip}$ √

$\frac{231.6 \text{ kip}}{432 \text{ sqft}} = .536 \text{ k/sf} = \boxed{536.1 \text{ psf}}$ √

*No Lateral Load on truck court √

MB BC 5/8/14

Tuesday, May 06, 2014, 12:47 PM

CM. Muthl
4/8/14

```
****************************************************
*                                                  *
*          STAAD.Pro V8i SELECTseries4             *
*          Version  20.07.09.31                    *
*          Proprietary Program of                  *
*          Bentley Systems, Inc.                   *
*          Date=    MAY  6, 2014                   *
*          Time=    11:16: 6                       *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
    1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Severe Truck Court.STD
    2. START JOB INFORMATION
    3. ENGINEER DATE 12-MARCH-2014
    4. END JOB INFORMATION
    5. INPUT WIDTH 79
    6. UNIT FEET KIP
    7. JOINT COORDINATES
    8. 4 0 7 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
    9. 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
   10. MEMBER INCIDENCES
   11. 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5
   12. START USER TABLE
   13. TABLE 1
   14. UNIT FEET KIP
   15. WIDE FLANGE
   16. ORIGINAL
   17. 0.179247 1.15 0.05125 1.225 0.05125 0.0428327 0.0157136 0.000156934 -
   18. 0.0589375 0.125563
   19. 0.25_THICK
   20. 0.0741319 1.15 0.0208333 1.225 0.0208333 0.0186353 0.00638368 1.07251E-005 -
   21. 0.0239583 0.0510417
   22. 0.375_THICK
   23. 0.110547 1.15 0.03125 1.225 0.03125 0.027312 0.00957706 3.59853E-005 -
   24. 0.0359375 0.0765625
   25. 0.4375_THICK
   26. 0.128592 1.15 0.0364583 1.225 0.0364583 0.0314958 0.0111744 5.69753E-005 -
   27. 0.0419271 0.0893229
   28. 0.5_THICK
   29. 0.146528 1.15 0.0416667 1.225 0.0416667 0.0355786 0.0127722 8.4796E-005 -
   30. 0.0479167 0.102083
   31. 0.3125_THICK
   32. 0.0923948 1.15 0.026042 1.225 0.026042 0.023026 0.0079803 2.0887E-005 -
   33. 0.0299483 0.0638029
   34. 0.125_THICK
   35. 0.0372842 1.15 0.010417 1.225 0.010417 0.00953593 0.00319164 1.34862E-006 -
   36. 0.0119796 0.0255217
   37. TABLE 2
   38. UNIT INCHES KIP
```

```
39. WIDE FLANGE
40. 20%TOP
41. 17.285 13.8 0.425 14.7 0.4 605.874 211.851 0.959851 5.865 11.76
42. 20%MID
43. 26.535 13.8 0.225 14.7 0.2 641.025 116.177 0.129278 3.105 5.88
44. 20%MUDLINE
45. 18.26 13.8 0.5 14.7 0.4 619.605 211.904 1.16887 6.9 11.76
46. 50%PILE
47. 14.5475 13.8 0.575 13.8 0.25 429.481 109.714 0.986568 7.935 6.9
48. 50%FLG12
49. 13.6475 13.8 0.575 12 0.25 388.166 72.2107 0.967818 7.935 6
50. 33%PILE
51. 12.245 13.8 0.475 14.7 0.2 367.152 106.004 0.557101 6.555 5.88
52. END
53. UNIT FEET KIP
54. DEFINE MATERIAL START
55. ISOTROPIC STEEL
56. E 4.176E+006
57. POISSON 0.3
58. DENSITY 0.489024
59. ALPHA 6.5E-006
60. DAMP 0.03
61. END DEFINE MATERIAL
62. UNIT INCHES KIP
63. CONSTANTS
64. BETA 0 MEMB 8 9
65. MATERIAL STEEL ALL
66. *9 UPTABLE 2 50%PILE
67. *9 UPTABLE 2 33%PILE
68. *9 UPTABLE 2 50%FLG12
69. MEMBER PROPERTY AMERICAN
70. 9 UPTABLE 2 20%TOP
71. 20 TO 23 UPTABLE 2 20%MUDLINE
72. MEMBER PROPERTY AMERICAN
73. 7 8 UPTABLE 2 20%MID
74. 4 TABLE ST HP14X89
75. SUPPORTS
76. 4 FIXED
77. 10 FIXED BUT FY
78. MEMBER RELEASE
79. 9 END MPY 0.5 MPZ 0.5
80. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
81. JOINT LOAD
82. 10 FY -167.8
83. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	9	NUMBER OF MEMBERS	8
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 43

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	10.51111
2	18.35732
3	28.93738
4	36.68349

84. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 6,2014 TIME= 11:16: 7 ****

```
*************************************************************
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*    Worldwide  http://selectservices.bentley.com/en-US/    *
*                                                           *
*************************************************************
```

CH2MHILL.

Job Name HRPT
Subject T/C Mayor Pile Capacity

Job No.
Sheet No. 1
Date
Computed By RC 4/8/14
Checked By MWahl 4/8/14

Global buckling (Mayor Pile)

Top	Mid	Bottom
F: .452	F: [illegible]	F: .481
W: .49	W: .326	W: .54
A_s = 19.6	A_s = 13.8	A_s = 20.81

From Staad

Applied Load = [illegible] kip

BF = [illegible].43

EBL = 2085.75 kip

$$F_e = \frac{2085.75\,kip}{20.81\,in^2} = 100.228\ ksi$$

Critical Section @ Node 16

L = 360"

$$\frac{kL}{r} = \sqrt{\frac{\pi^2 E}{F_e}} = 53.4[illegible]$$

F_{cr} = 27.15 ksi

$$\frac{P_n}{\Omega} = \boxed{338.3\ kip}$$

Check FLB

b/t = 16.26

Q_s = 1.415 − 0.74(16.26)[illegible]

Q_s = .991

F_{cr} = 30.73

$$\frac{P_n}{\Omega} = \boxed{382.[illegible]}$$

MB BC 5/8/14

CH. MWahl 4/8/14

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 ****************************************************
 *                                                  *
 *           STAAD.Pro V8i SELECTseries4            *
 *           Version  20.07.09.31                   *
 *           Proprietary Program of                 *
 *           Bentley Systems, Inc.                  *
 *           Date=    APR 16, 2014                  *
 *           Time=    16:57:38                      *
 *                                                  *
 *      USER ID: CH2M HILL                          *
 ****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Major Truck Court.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 7 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
    10. MEMBER INCIDENCES
    11. 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5
    12. START USER TABLE
    13. TABLE 2
    14. UNIT INCHES KIP
    15. WIDE FLANGE
    16. TOP
    17. 19.6078 13.8 0.49 14.7 0.452 679.715 239.424 1.41072 6.762 13.2888
    18. MID
    19. 13.8706 13.8 0.326 14.7 0.326 496.841 172.629 0.491372 4.4988 9.5844
    20. MUDLINE
    21. 20.8105 13.8 0.504 14.7 0.488 724.476 258.494 1.68616 6.9552 14.3472
    22. END
    23. UNIT FEET KIP
    24. DEFINE MATERIAL START
    25. ISOTROPIC STEEL
    26. E 4.176E+006
    27. POISSON 0.3
    28. DENSITY 0.489024
    29. ALPHA 6.5E-006
    30. DAMP 0.03
    31. END DEFINE MATERIAL
    32. UNIT INCHES KIP
    33. CONSTANTS
    34. BETA 0 MEMB 8 9
    35. MATERIAL STEEL ALL
    36. MEMBER PROPERTY AMERICAN
    37. 9 UPTABLE 2 TOP
    38. 20 TO 23 UPTABLE 2 MUDLINE
```

```
35. MEMBER PROPERTY AMERICAN
40. 7 8 UPTABLE 2 MID
41. 4 TABLE ST HP14X89
42. SUPPORTS
43. 4 FIXED
44. 10 FIXED BUT FY
45. MEMBER RELEASE
46. 9 END MPY 0.5 MPZ 0.5
47. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
48. JOINT LOAD
49. 10 FY -167.8
50. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	9	NUMBER OF MEMBERS	8
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 43

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	12.43459
2	22.83334
3	31.42424
4	45.25114

51. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= APR 16,2014 TIME= 16:57:38 ****

```
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 *         Bentley Systems or Partner offices               *
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 *                                                          *
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 *                                                          *
 ************************************************************
```

CH2MHILL.

Job Name

Subject T/C Moderate Pile

Job No.

Sheet No. 1

Date

Computed By B 5/8/14

Checked By

Global Buckling Moderate Pile

Top	Mid	Bot
F .465	F .516	F .447
W .447	W .460	W .472
A_s = 15.79	A_s = 21.07	A_s = 19.23

From STAAD

Applied Load 157.8

BF: 12.68

LBL: 2000.58 kip

F_e = 107.155 ksi

L = 360"

$\frac{KL}{r} = \sqrt{\frac{\pi^2 E}{F_e}}$ = 51.62

$F_{cr} = \left[0.658^{36/107.1}\right] 36 = 31.27$ ksi

$\frac{P_n}{\Omega}$ = 360.14 kip

Check FLB (Local Buckling)

$b/t = \frac{7.35}{.465}$ = 17.1 Top flange is slender

Q_s = 1.415 − 0.74 (17.1) $\sqrt{F_y/E}$

= 0.967

A_s = 17.33 Apply to (E 7-2)

$F_{cr} = .967\left[.658^{\frac{.967 \cdot 36}{107.15}}\right] 36$

= 30.3 ksi

$\frac{P_n}{\Omega}$ = $\boxed{349.2 \text{ kip}}$

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.31                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    APR 17, 2014                  *
*           Time=    10:28:21                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Moderate Truck Court.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 4 0 7 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0; 9 0 32.25 0; 10 0 37 0
     9. 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
    10. MEMBER INCIDENCES
    11. 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15; 23 17 5
    12. START USER TABLE
    13. TABLE 2
    14. UNIT INCHES KIP
    15. WIDE FLANGE
    16. TOP
    17. 18.3821 13.8 0.448 14.7 0.428 643.66 226.689 1.1563 6.1824 12.5832
    18. MID
    19. 21.0692 13.8 0.462 14.7 0.516 749.733 273.286 1.76609 6.3756 15.1704
    20. MUDLINE
    21. 19.2334 13.8 0.472 14.7 0.447 670.577 236.764 1.32766 6.5136 13.1418
    22. END
    23. UNIT FEET KIP
    24. DEFINE MATERIAL START
    25. ISOTROPIC STEEL
    26. E 4.176E+006
    27. POISSON 0.3
    28. DENSITY 0.489024
    29. ALPHA 6.5E-006
    30. DAMP 0.03
    31. END DEFINE MATERIAL
    32. UNIT INCHES KIP
    33. CONSTANTS
    34. BETA 0 MEMB 8 9
    35. MATERIAL STEEL ALL
    36. MEMBER PROPERTY AMERICAN
    37. 9 UPTABLE 2 TOP
    38. 20 TO 23 UPTABLE 2 MUDLINE
```

```
39. MEMBER PROPERTY AMERICAN
40. 7 8 UPTABLE 2 MID
41. 4 TABLE ST HP14X89
42. SUPPORTS
43. 4 FIXED
44. 10 FIXED BUT FY
45. MEMBER RELEASE
46. 9 END MPY 0.5 MPZ 0.5
47. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
48. JOINT LOAD
49. 10 FY -167.8
50. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	9	NUMBER OF MEMBERS	8
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 43

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	12.28600
2	25.04458
3	31.40695
4	47.45310

51. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= APR 17,2014 TIME= 10:28:22 ****

```
**************************************************************
*        For questions on STAAD.Pro, please contact          *
*        Bentley Systems or Partner offices                  *
*                                                            *
*              Telephone                Web / Email          *
*  USA        +1  (714) 974-2500                             *
*  UK         +44 (0) 808 101 9246                           *
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*  INDIA      +91 (033) 4006-2021                            *
*  JAPAN      +81 (03)5952-6500     http://www.ctc-g.co.jp   *
*  CHINA      +86 21 6288 4040                               *
*  THAILAND   +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                            *
*  Worldwide  http://selectservices.bentley.com/en-US/       *
*                                                            *
**************************************************************
```

CH2MHILL.

Job Name: HKPT TC Local Crushing [w/ Avg pile]

Subject:

Job No.:

Sheet No.:

Date:

Computed By: BC 5/8/14

Checked By: MWahl 4/8/14

Local Crushing

Truck Court Severe Pile

F: .447 in

W: .459 in

$A_s = 19.07\ in^2$

36 ksi · 19.07 in² = 686.52 kip

$\frac{P_n}{\Omega} = \boxed{411.09\ kip}$

Major Pile

F: 0.452"

W: 0.490"

$A_s = 19.60\ in^2$

$\frac{P_n}{\Omega} = \boxed{422.51}$

Moderate Pile

F: 0.428

W: 0.448

$A_s = 18.38\ in^2$

$\frac{P_n}{\Omega} = \boxed{396.21}$

Minor Pile

No data

PILE LOAD RATINGS – FINGER PIER

CH2MHILL.

Job No. ____________
Sheet No. ____________
Job Name: HRPT Finger Pier [illegible]
Date ____________
Subject ____________
Computed By: [illegible] 5/8/14
Checked By: MWd 5/8/14

Global Buckling (severe)

Top	Mid	Bot
F .259	F .454	F .41[illegible]
W .407	W .354	W .311
A_s 13.02	A_s 18.29	A_s 17.61

From STADD

Applied Load: 175.6 kip

BF: 4.48

EBL: 785.69 kip

$$F_e = \frac{785.69 \text{ kip}}{17.61} = 44.6 \text{ ksi}$$

L = 552"

$$\frac{KL}{r} = \sqrt{\frac{\pi^2 E}{F_e}} = 80.109$$

$$F_{cr} = \left[0.658^{F_y/44.6}\right] 36 = 25.70 \text{ ksi}$$

$A = 17.61 \text{ in}^2$

$P_n = 452.7$ kip

$$\boxed{\frac{P_n}{\Omega} = 271.07 \text{ kip}}$$

CH2MHILL SUBJECT HPP1 F/P [illegible] BY
SHEET NO [illegible] of ____ DATE
PROJECT NO

BC 5/8/2014
Ch MWahl
5/8/14

Check FLB (Flange Local Buckling)

$b/t = \frac{7.35''}{0.259} = 28.37$ ✓

$28.37 < 1.03 \cdot \sqrt{\frac{29000}{36}}$ ✓

$Q_s = 1.415 - 0.74 \cdot (28.37) \cdot \sqrt{\frac{36}{29000}}$ ✓

$\boxed{Q_s = 0.675}$ ✓

Apply to AISC E 7-2) conserv.

$F_{cr} = 0.675 \cdot (0.658^{[illegible]/[illegible]}) \cdot 36$ ✓

$F_{cr} = 19.345$ ksi ✓

$P_n = 19.345$ ksi $\cdot$ 17.61 in² = 340,665 ✓

$\boxed{\frac{P_n}{\Omega} = 204 \text{ kip}}$ ✓

MB BC 5/8/2014

CH MWahl
5/8/14

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.31                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    MAY  6, 2014                  *
*           Time=    11:41:53                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Avg Severe Finger Pier.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0
     9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
    10. 18 0 -6 0; 19 0 -3 0; 20 0 0 0
    11. MEMBER INCIDENCES
    12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
    13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
    14. START USER TABLE
    15. TABLE 2
    16. UNIT INCHES KIP
    17. WIDE FLANGE
    18. TOP
    19. 13.0204 13.8 0.407 14.7 0.259 428.563 137.195 0.468752 5.6166 7.6146
    20. MID
    21. 18.2981 13.8 0.384 14.7 0.454 663.15 240.418 1.16038 5.2992 13.3476
    22. MUDLINE
    23. 17.6174 13.8 0.411 14.7 0.418 624.984 221.373 1.01575 5.6718 12.2892
    25. END
    26. UNIT FEET KIP
    27. DEFINE MATERIAL START
    28. ISOTROPIC STEEL
    29. E 4.176E+006
    30. POISSON 0.3
    31. DENSITY 0.489024
    32. ALPHA 6.5E-006
    33. DAMP 0.03
    34. END DEFINE MATERIAL
    35. UNIT INCHES KIP
    36. CONSTANTS
    37. BETA 0 MEMB 8 9
    38. MATERIAL STEEL ALL
    40. MEMBER PROPERTY AMERICAN
```

```
41. 1 24 TO 26 TABLE ST HP14X89
42. MEMBER PROPERTY AMERICAN
43. 9 UPTABLE 2 TOP
44. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
45. MEMBER PROPERTY AMERICAN
46. 7 8 UPTABLE 2 MID
47. SUPPORTS
48. 1 FIXED
49. 10 FIXED BUT FY
50. MEMBER RELEASE
51. 9 END MPY 0.5 MPZ 0.5
52. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
53. JOINT LOAD
54. 10 FY -175.6
55. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	15	NUMBER OF MEMBERS	14
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 79

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	4.48064
2	9.25367
3	12.28145
4	18.32388

56. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 6,2014 TIME= 11:41:53 ****

```
**********************************************************
*        For questions on STAAD.Pro, please contact      *
*        Bentley Systems or Partner offices              *
*                                                        *
*            Telephone              Web / Email          *
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*  CHINA     +86 21 6288 4040                            *
*  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                        *
*  Worldwide  http://selectservices.bentley.com/en-US/   *
*                                                        *
**********************************************************
```

CH2MHILL.

Job Name HRPT FF LC w/Avg Pile

Subject

Job No.

Sheet No.

Date

Computed By BC 5/8/2014

Checked By MWH 5/8/14

Local Crushing

Finger Pier (Severe Pile)

F: 0.254" ✓

W: 0.407" ✓

A_s: 13.02 in^2 ✓

36 ksi 13.02 in^2 = 468.72 kip ✓

$\frac{P_n}{1.67}$: $\boxed{280.67 \text{ kip}}$ ✓

Major Pile

F: .390" ✓

W: .445 ✓

A_s = 17.26 in^2 ✓

$\frac{P_n}{\Omega}$ = $\boxed{378.53 \text{ kip}}$ ✓

Moderate Pile

F: .449" ✓

W: .430" ✓

A_s = 18.75 in^2 ✓

$\frac{P_n}{\Omega}$ $\boxed{404.2 \text{ kip}}$ ✓

Minor Pile

F: 0.611" ✓

W: 0.608" ✓

A_s = 25.6 in^2 ✓

$\frac{P_n}{\Omega}$ = $\boxed{551 \text{ kip}}$ ✓

CH2MHILL SUBJECT FP Lateral [illegible] BY
SHEET NO 1 of DATE
PROJECT NO

MB RC 5/8/14
ch MWahl 5/8/14

Load Case [illegible] [Moment U/R From STADD See attached Output] ✓

U/R [illegible] 0.192 ✓

Load Case 5

U/R : 0.214 ✓

Load Case

U/R 0.627 @ Top ✓

Load Case 8

U/R 0.861 @ Top ✓

Load Case 7

U/R 0.895 @ Top ✓

Combined Axial & Moment ✓

Load Case 1

Total Axial [illegible] = [illegible] kip ✓

U/R [illegible]

Load Case ✓

Additional Axial

1.4 + 2 + 1.4 = 4.8 k - see attached Etab output

$$\left[\frac{\underset{L_L}{[illegible]} + \underset{D_L}{80.4} + \underset{L_L}{[illegible]} + \underset{\text{Addtl}}{[illegible]}}{204\ kip}\right] + \left[8/9 \cdot 0.198\right] = 0.624\ ✓$$

For Service Pile

F.L.D. Controls

204 k - Capacity ✓

Live Load

100 PSF : [illegible] kip ✓

300 PSF : 67.5 kip

D_L = 80.4 kip ✓

Superimposed D_L (to account for structure on service pile)

[illegible] = [illegible] 15 kip ✓

CH2MHILL SUBJECT ____ BY ____

SHEET NO 2 of ____ DATE ____ BC MB 5/8/2014

PROJECT NO. ____

Ch MWahl 5/8/14

Load Case 5

$\left[\frac{6.75+8 \text{ [illegible]} 5 \text{ [illegible]}}{2.04\text{ k}}\right] \cdot 1.4 + 8/9 \text{ [illegible]} = 0.590$ ✓

OK w/ 100 PSF ✓

Load Case 7

$\left[\frac{6.75\text{k} + \text{[illegible]} + 7.3\text{k}}{2.04\text{k}}\right] \div 1.33 + 8/9 \cdot 0.627$ .91 ✓ OK ✓

Load Case 8

~~other page / Said 0.949~~ Corrected BC 5/13

$\left[\frac{\underset{D_1}{8\text{[illegible]}} + \underset{D_2}{6.75\text{k}} + \underset{M\text{[illegible]}}{35\text{k}} + \underset{L_1}{\text{[illegible]}5\text{k}}}{2.04\text{k}}\right] \div 1.4 + 8/9 \cdot 0.861 = 1.27$ N/G

Load Case 9

$\left[\frac{80.4 + 6.75 + (1.4 \cdot 2.0) + 35}{2.04}\right] \div 1.5 + 8/9 \cdot .895 = 1.21$ ✓ N/G ✓

0.410

* For L/C 8 & 9 Please see Appendix 1

MB Bc 5/14
Ch MW11
5/X/14

Severe Pile

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.31                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    MAY  6, 2014                  *
*           Time=    15:50:10                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
    1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending__Finger Pier LC3.STD
    2. START JOB INFORMATION
    3. ENGINEER DATE 12-MARCH-2014
    4. END JOB INFORMATION
    5. INPUT WIDTH 79
    6. UNIT FEET KIP
    7. JOINT COORDINATES
    8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9.89 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0
    9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
   10. 18 0 -6 0; 19 0 -3 0; 20 0 0 0
   11. MEMBER INCIDENCES
   12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
   13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
   14. START USER TABLE
   15. TABLE 1
   16. UNIT FEET KIP
   17. WIDE FLANGE
   18. ORIGINAL
   19. 0.179247 1.15 0.05125 1.225 0.05125 0.0428327 0.0157136 0.000156934 -
   20. 0.0589375 0.125563
   21. 0.25_THICK
   22. 0.0741319 1.15 0.0208333 1.225 0.0208333 0.0186353 0 00638368 1.07251E-005
   23. 0.0239583 0.0510417
   24. 0.375_THICK
   25. 0.110547 1.15 0.03125 1.225 0.03125 0.027312 0.00957706 3.59853E-005 -
   26. 0.0359375 0.0765625
   27. 0.4375_THICK
   28. 0.128592 1.15 0.0364583 1.225 0.0364583 0.0314958 0.0111744 5.69753E-005 -
   29. 0.0419271 0.0893229
   30. 0.5_THICK
   31. 0.146528 1.15 0.0416667 1.225 0.0416667 0.0355786 0.0127722 8.4796E-005
   32. 0.0479167 0.102083
   33. 0.3125_THICK
   34. 0.0923948 1.15 0.026042 1.225 0.026042 0.023026 0.0079803 2.0887E-005 -
   35. 0.0299483 0.0638029
   36. 0.125_THICK
   37. 0.0372842 1.15 0.010417 1.225 0.010417 0.00953593 0 00319164 1.34862E-006 -
   38. 0.0119796 0.0255217
```

```
39. TABLE 2
40. UNIT INCHES KIP
41. WIDE FLANGE
42. TOP
43. 13.0204 13.8 0.407 14.7 0.259 428.563 137.195 0.468752 5.6166 7.6146
44. MID
45. 18.2981 13.8 0.384 14.7 0.454 663.15 240.418 1.16038 5.2992 13.3476
46. MUDLINE
47. 17.6174 13.8 0.411 14.7 0.418 624.984 221.373 1.01575 5.6718 12.2892
48. 50%PILE
49. 14.5475 13.8 0.575 13.8 0.25 429.481 109.714 0.986568 7.935 6.9
50. 50%FLG12
51. 13.6475 13.8 0.575 12 0.25 388.166 72.2107 0.967818 7.935 6
52. 33%PILE
53. 12.245 13.8 0.475 14.7 0.2 367.152 106.004 0.557101 6.555 5.88
54. END
55. UNIT FEET KIP
56. DEFINE MATERIAL START
57. ISOTROPIC STEEL
58. E 4.176E+006
59. POISSON 0.3
60. DENSITY 0.489024
61. ALPHA 6.5E-006
62. DAMP 0.03
63. END DEFINE MATERIAL
64. UNIT INCHES KIP
65. CONSTANTS
66. BETA 0 MEMB 8 9
67. MATERIAL STEEL ALL
68. MEMBER PROPERTY AMERICAN
69. 1 24 TO 26 TABLE ST HP14X89
70. MEMBER PROPERTY AMERICAN
71. 9 UPTABLE 2 TOP
72. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
73. MEMBER PROPERTY AMERICAN
74. 7 8 UPTABLE 2 MID
76. SUPPORTS
77. 1 10 PINNED
79. LOAD 3 LOADTYPE WIND  TITLE WAVE
80. JOINT LOAD
81. 1 MZ 204
82. 10 MZ 204
84. LOAD 4 LOADTYPE WIND  TITLE WIND Y AXIS
85. JOINT LOAD
86. 1 MZ 80.4
87. 10 MZ 79.2
89. LOAD 5 LOADTYPE WIND  TITLE CURRENT
90. JOINT LOAD
91. 1 MZ 19.2
92. 10 MZ 25.2
94. LOAD COMB 6 HRPT LOAD CASE 3
95. 3 0.24 4 0.24 5 0.8
97. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	15	NUMBER OF MEMBERS	14
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 3, TOTAL DEGREES OF FREEDOM = 84

```
***WARNING- INSTABILITY AT JOINT [illegible] DIRECTION = MY
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG=  5.5741491E+02 L-MATRIX DIAG=  3.1263880E-13 EQN NO      14
***NOTE - VERY WEAK SPRING ADDED FOR STABILITY
```

```
 98. PARAMETER 1
 99. CODE AISC
100. FYLD 36 ALL
101. LX 325.3 MEMB 4 7 TO 9 20 TO 23
102. UNB 325.3 MEMB 4 7 TO 9 20 TO 23
103. UNT 325.3 MEMB 4 7 TO 9 20 TO 23
104. LX 226.7 MEMB 1 TO 3 24 TO 26
105. UNB 226.7 MEMB 1 TO 3 24 TO 26
106. UNT 226.7 MEMB 1 TO 3 24 TO 26
107. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.072	3
		0.00 T	0.00	204.00	0.00
2 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.052	3
		0.00 T	0.00	97.57	0.00
3 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.038	3
		0.00 T	0.00	70.96	0.00
4 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.022	3
		0.00 T	0.00	36.45	0.00
7 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.088	3
		0.00 T	0.00	-155.22	63.00
8 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.091	3
		0.00 T	0.00	-161.87	9.00
9 ST	TOP		(UPT)		
		PASS	AISC- H1-3	0.198	3
		0.00 T	0.00	-204.00	57.00
20 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.066	3
		0.00 T	0.00	108.65	0.00
21 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.027	3
		0.00 T	0.00	45.46	0.00
22 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.046	3
		0.00 T	0.00	77.05	0.00
23 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.011	3
		0.00 T	0.00	17.74	42.75
24 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.063	3
		0.00 T	0.00	177.39	0.00
25 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.053	3
		0.00 T	0.00	150.78	0.00
26 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.044	3
		0.00 T	0.00	124.17	0.00

108. LOAD LIST 6
109. FINISH

```
*********** END OF THE STAAD.Pro RUN ***********

  **** DATE= MAY  6,2014   TIME= 15:50:10 ****

************************************************************
*        For questions on STAAD.Pro, please contact        *
*        Bentley Systems or Partner offices                *
*                                                          *
*             Telephone              Web / Email           *
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*  UK         +44 (0) 808 101 9246                         *
*  SINGAPORE +65 6225-6158                                 *
*  FRANCE     +33 (0) 1 55238400                           *
*  GERMANY    +49 0931 40468                               *
*  INDIA      +91 (033) 4006-2021                          *
*  JAPAN      +81 (03)5952-6500    http://www.ctc-g.co.jp  *
*  CHINA      +86 21 6288 4040                             *
*  THAILAND   +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
*  Worldwide  http://selectservices.bentley.com/en-US/     *
*                                                          *
************************************************************
```

Severe Pile

MB BC 5/14

(+) Mwill

5/8/14

```
****************************************************
*                                                  *
*          STAAD.Pro V8i SELECTseries4             *
*          Version  20.07.09.31                    *
*          Proprietary Program of                  *
*          Bentley Systems, Inc.                   *
*          Date=    MAY  6, 2014                   *
*          Time=     15:53:32                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending__Finger Pier_LC5.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 11.16 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0
     9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
    10. 18 0 -6 0; 19 0 -3 0; 20 0 0 0
    11. MEMBER INCIDENCES
    12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
    13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
    14. START USER TABLE
    15. TABLE 1
    16. UNIT FEET KIP
    17. WIDE FLANGE
    18. ORIGINAL
    19. 0.179247 1.15 0.05125 1.225 0.05125 0.0428327 0.0157136 0.000156934 -
    20. 0.0589375 0.125563
    21. 0.25_THICK
    22. 0.0741319 1.15 0.0208333 1.225 0.0208333 0.0186353 0.00638368 1.07251E-005 -
    23. 0.0239583 0.0510417
    24. 0.375_THICK
    25. 0.110547 1.15 0.03125 1.225 0.03125 0.027312 0.00957706 3.59853E-005 -
    26. 0.0359375 0.0765625
    27. 0.4375_THICK
    28. 0.128592 1.15 0.0364583 1.225 0.0364583 0.0314958 0.0111744 5.69753E-005 -
    29. 0.0419271 0.0893229
    30. 0.5_THICK
    31. 0.146528 1.15 0.0416667 1.225 0.0416667 0.0355786 0.0127722 8.4796E-005 -
    32. 0.0479167 0.102083
    33. 0.3125_THICK
    34. 0.0923948 1.15 0.026042 1.225 0.026042 0.023026 0.0079803 2.0887E-005 -
    35. 0.0299483 0.0638029
    36. 0.125_THICK
    37. 0.0372842 1.15 0.010417 1.225 0.010417 0.00953593 0.00319164 1.34862E-006 -
    38. 0.0119796 0.0255217
```

```
39. TABLE 2
40. UNIT INCHES KIP
41. WIDE FLANGE
42. TOP
43. 13.0204 13.8 0.407 14.7 0.259 428.563 137.195 0.468752 5.6166 7.6146
44. MID
45. 18.2981 13.8 0.384 14.7 0.454 663.15 240.418 1.16038 5.2992 13.3476
46. MUDLINE
47. 17.6174 13.8 0.411 14.7 0.418 624.984 221.373 1.01575 5.6718 12.2892
48. 50%PILE
49. 14.5475 13.8 0.575 13.8 0.25 429.481 109.734 0.986568 7.935 6.9
50. 50%FLG12
51. 13.6475 13.8 0.575 12 0.25 388.166 72.2107 0.967818 7.935 6
52. 33%PILE
53. 12.245 13.8 0.475 14.7 0.2 367.152 106.004 0.557101 6.555 5.88
54. END
55. UNIT FEET KIP
56. DEFINE MATERIAL START
57. ISOTROPIC STEEL
58. E 4.176E+006
59. POISSON 0.3
60. DENSITY 0.489024
61. ALPHA 6.5E-006
62. DAMP 0.03
63. END DEFINE MATERIAL
64. UNIT INCHES KIP
65. CONSTANTS
66. BETA 0 MEMB 8 9
67. MATERIAL STEEL ALL
68. *9 UPTABLE 2 50%PILE
69. *9 UPTABLE 2 33%PILE
70. *9 UPTABLE 2 50%FLG12
71. MEMBER PROPERTY AMERICAN
72. 1 24 TO 26 TABLE ST HP14X89
73. MEMBER PROPERTY AMERICAN
74. 9 UPTABLE 2 TOP
75. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
76. MEMBER PROPERTY AMERICAN
77. 7 8 UPTABLE 2 MID
78. SUPPORTS
79. 1 10 PINNED
81. LOAD 3 LOADTYPE WIND  TITLE WAVE
82. JOINT LOAD
83. 1 MZ 204
84. 10 MZ 204
86. LOAD 4 LOADTYPE WIND  TITLE WIND Y AXIS
87. JOINT LOAD
88. 1 MZ 80.4
89. 10 MZ 79.2
91. LOAD 5 LOADTYPE WIND  TITLE CURRENT
92. JOINT LOAD
93. 1 MZ 19.2
94. 10 MZ 25.2
95. LOAD COMB 6 HRPT LOAD CASE 5
96. 5 0.714 3 0.714 4 0.714
98. PERFORM ANALYSIS
```

```
         P R O B L E M   S T A T I S T I C S
         -----------------------------------

   NUMBER OF JOINTS          15  NUMBER OF MEMBERS       14
   NUMBER OF PLATES           0  NUMBER OF SOLIDS         0
   NUMBER OF SURFACES         0  NUMBER OF SUPPORTS       2

        SOLVER USED IS THE IN-CORE ADVANCED SOLVER

   TOTAL PRIMARY LOAD CASES =     3, TOTAL DEGREES OF FREEDOM =      84

***WARNING - INSTABILITY AT JOINT   [illegible]   DIRECTION = MY
 PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
 K-MATRIX DIAG=  4.9767912E+02 L-MATRIX DIAG=  2.5579538E-13 EQN NO      14
 ***NOTE - VERY WEAK SPRING ADDED FOR STABILITY

   99. PARAMETER 1
  100. CODE AISC
  101. FYLD 36 ALL
  102. LX 310.1 MEMB 4 7 TO 9 20 TO 23
  103. UNB 310.1 MEMB 4 7 TO 9 20 TO 23
  104. UNT 310.1 MEMB 4 7 TO 9 20 TO 23
  105. LX 241.9 MEMB 1 TO 3 24 TO 26
  106. UNB 241.9 MEMB 1 TO 3 24 TO 26
  107. UNT 241.9 MEMB 1 TO 3 24 TO 26
  108. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.077	6
		0.00 T	0.00	216.77	0.00
2 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.054	6
		0.00 T	0.00	102.78	0.00
3 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.039	6
		0.00 T	0.00	74.28	0.00
4 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.015	6
		0.00 T	0.00	25.26	0.00
7 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.092	6
		0.00 T	0.00	-167.95	63.00
8 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.096	6
		0.00 T	0.00	-175.08	9.00
9 ST	TOP		(UPT)		
		PASS	AISC- H1-3	0.214	6
		0.00 T	0.00	-220.20	57.00
20 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.069	6
		0.00 T	0.00	118.08	0.00
21 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.030	6
		0.00 T	0.00	50.40	0.00
22 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.049	6
		0.00 T	0.00	84.24	0.00
23 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.010	3
		0.00 T	0.00	-17.74	42.75
24 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.067	6
		0.00 T	0.00	188.27	0.00
25 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.057	6
		0.00 T	0.00	159.77	0.00
26 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.046	6
		0.00 T	0.00	131.28	0.00

109. LOAD LIST 6
110. FINISH

```
*********** END OF THE STAAD.Pro RUN ***********

  **** DATE= MAY  6,2014   TIME= 15:53:33 ****

************************************************************
*        For questions on STAAD.Pro, please contact        *
*        Bentley Systems or Partner offices                *
*                                                          *
*              Telephone              Web / Email          *
*  USA        +1  (714) 974-2500                           *
*  UK         +44 (0) 808 101 9246                         *
*  SINGAPORE +65 6225-6158                                 *
*  FRANCE     +33 (0) 1 55238400                           *
*  GERMANY    +49 0931 40468                               *
*  INDIA      +91 (033) 4006-2021                          *
*  JAPAN      +81 (03)5952-6500    http://www.ctc-g.co.jp  *
*  CHINA      +86 21 6288 4040                             *
*  THAILAND   +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
*  Worldwide  http://selectservices.bentley.com/en-US/     *
*                                                          *
************************************************************
```

Severe Pile

MB BC 5/14

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.31                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    MAY  1, 2014                  *
*           Time=    14:50:25                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
    1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending__Finger Pier_LC7.STD
    2. START JOB INFORMATION
    3. ENGINEER DATE 12-MARCH-2014
    4. END JOB INFORMATION
    5. INPUT WIDTH 79
    6. UNIT FEET KIP
    7. JOINT COORDINATES
    8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9.69 0; 5 0 12.23 0; 7 0 26.25 0; 8 0 31.5 0
    9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
   10. 18 0 -6 0; 19 0 -3 0; 20 0 0 0
   11. MEMBER INCIDENCES
   12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
   13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
   14. START USER TABLE
   16. TABLE 2
   17. UNIT INCHES KIP
   18. WIDE FLANGE
   19. TOP
   20. 13.0204 13.8 0.407 14.7 0.259 428.563 137.195 0.468752 5.6166 7.6146
   21. MID
   22. 18.2981 13.8 0.384 14.7 0.454 663.15 240.418 1.16038 5.2992 13.3476
   23. MUDLINE
   24. 17.6174 13.8 0.411 14.7 0.418 624.984 221.373 1.01575 5.6718 12.2892
   26. END
   28. UNIT FEET KIP
   29. DEFINE MATERIAL START
   30. ISOTROPIC STEEL
   31. E 4.176E+006
   32. POISSON 0.3
   33. DENSITY 0.489024
   34. ALPHA 6.5E-006
   35. DAMP 0.03
   36. END DEFINE MATERIAL
   38. UNIT INCHES KIP
   39. CONSTANTS
   40. BETA 0 MEMB 8 9
   41. MATERIAL STEEL ALL
   43. MEMBER PROPERTY AMERICAN
```

```
44. 1 24 TO 26 TABLE ST HP14X89
45. MEMBER PROPERTY AMERICAN
46. 9 UPTABLE 2 TOP
47. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
48. MEMBER PROPERTY AMERICAN
49. 7 8 UPTABLE 2 MID
51. SUPPORTS
52. 1 10 PINNED
54. LOAD 1 LOADTYPE ICE  TITLE EARTHQUAKE (MAJOR AXIS)
55. JOINT LOAD
56. 1 MZ 876
57. 10 MZ 860.4
58. LOAD COMB 2 HRPT LOAD CASE 7
59. 1 0.75
61. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	15	NUMBER OF MEMBERS	14
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 84

```
***WARNING - INSTABILITY AT JOINT [illegible] DIRECTION = MY
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG=  5.5741491E+02 L-MATRIX DIAG=  1.1368684E-13 EQN NO      14
[illegible]
```

```
62. PARAMETER 1
63. CODE AISC
64. FYLD 36 ALL
65. LX 297.2 MEMB 7 TO 9 20 TO 23
66. UNB 297.2 MEMB 7 TO 9 20 TO 23
67. UNT 297.2 MEMB 7 TO 9 20 TO 23
68. LX 254.76 MEMB 1 TO 4 24 TO 26
69. UNB 254.76 MEMB 1 TO 4 24 TO 26
70. UNT 254.76 MEMB 1 TO 4 24 TO 26
71. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.310	1
		0.00 T	0.00	876.00	0.00
2 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.224	1
		0.00 T	0.00	423.03	0.00
3 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.164	1
		0.00 T	0.00	309.78	0.00
4 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.086	1
		0.00 T	0.00	162.94	0.00
7 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.348	1
		0.00 T	0.00	-652.79	63.00
8 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.363	1
		0.00 T	0.00	-681.10	9.00
9 ST	TOP		(UPT)		
		PASS	AISC- H1-3	0.836	1
		0.00 T	0.00	-860.40	57.00
20 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.261	1
		0.00 T	0.00	454.61	0.00
21 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.106	1
		0.00 T	0.00	185.66	0.00
22 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.184	1
		0.00 T	0.00	320.13	0.00
23 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.043	1
		0.00 T	0.00	-74.61	39.99
24 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.270	1
		0.00 T	0.00	762.76	0.00
25 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.230	1
		0.00 T	0.00	649.51	0.00
26 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.190	1
		0.00 T	0.00	536.27	0.00

73. FINISH

```
           *********** END OF THE STAAD.Pro RUN ***********

             **** DATE= MAY  1,2014   TIME= 14:50:25 ****

      ************************************************************
      *         For questions on STAAD.Pro, please contact        *
      *         Bentley Systems or Partner offices                *
      *                                                           *
      *             Telephone            Web / Email              *
      *  USA       +1  (714) 974-2500                             *
      *  UK        +44 (0) 808 101 9246                           *
      *  SINGAPORE +65 6225-6158                                  *
      *  FRANCE    +33 (0) 1 55238400                             *
      *  GERMANY   +49 0931 40468                                 *
      *  INDIA     +91 (033) 4006-2021                            *
      *  JAPAN     +81 (03)5952-6500     http://www.ctc-g.co.jp   *
      *  CHINA     +86 21 6288 4040                               *
      *  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
      *                                                           *
      *  Worldwide  http://selectservices.bentley.com/en-US/      *
      *                                                           *
      ************************************************************
```

CH2MHILL.

Job Name HRPT Pier 40

Subject Finger Pier Major Pile

Job No.

Sheet No. 1

Date

Computed By BC 5/14

Checked By MW 5/8/14

Calc GB of Pile (Global Buckling) ✓

Section (Major Pile)

Top	Mid	Bottom
F: 0.39" /	F .453 ✓	F .412 ✓
w: 0.485" /	w .417" ✓	w .440 ✓
A_s = 17.78 in ✓	A_s = 18.69 in² ✓	A_s = 17.82 in² ✓

Assume 50% / cons Moment release @ top, fy = rel @ Mid

From STADD

Applied Load 175.5 kip tot

Mid Pile (Conservative)

BF. 4.86 (Buckling factor) ✓

Critical Section @ (Node 17) ✓ Mid pile.

E.B. Load = 853.4 kip (Euler Buckling)

F_e @ critical section

$$F_e = \frac{853.4\,kip}{17.82\,in^2} = 47.89\ ksi ✓$$

$$\frac{K}{r} = \frac{\sqrt{\frac{\pi^2 \cdot E}{F_e}}}{L} \Rightarrow \frac{\sqrt{\frac{\pi^2 \cdot 29000}{47.89}}}{552''} \Rightarrow 0.140 /$$

$$\frac{KL}{r} = 77.30 ✓ < 133.6 ✓ \text{ (AISC E3-2)}$$

$$F_{cr} = 0.658^{(36/47.89)} \cdot 36 ✓$$

$$= 26.2\ ksi ✓$$

$$\frac{P_n}{\Omega_c} = \boxed{279.57\ kip} /$$

1.67

CH2MHILL.

Job Name HKPT Pier 40

Subject FP Mayor Pile (FLB)

Job No.

Sheet No. 2

Date

Computed By BC 5/14

Checked By CH MWM 5/8/14

Flange Local buckling

Check F/LB @ Top of the pile

$b/t = \dfrac{7.35''}{0.39} = 18.84$ ✓ flange is slender ✓

*Use A_s of critical section

Use E 7-5

$Q_s = 1.415 - 0.74\,(18.84)\sqrt{\dfrac{36}{29000}}$

$= 0.923$ ✓

Apply Q_s to E(7-2) (Conserv.) ✓

$F_{cr} = 0.923 \cdot \left[0.658^{(0.923 \cdot 36/47.84)}\right] 36$ ✓

$= 24.85$ ksi ✓

$\dfrac{P_n}{\Omega} = \dfrac{24.85 \cdot 17.82\,in^2}{1.67} = \boxed{265.1 \text{ kip}}$ ✓

CH2MHILL SUBJECT ________ BY ________

SHEET NO. ____ of ____ DATE ________

PROJECT NO. ________

MB BC 5/15/2014

L/C 8 & 9 Please see Appendix 1

L/C 3, 5 ok w/ Severe Piles, hence ok w/ major

L/C 7 ok by inspection, slight over stress w/ severe
hence ok w/ Major.

MB Bc 5/14

Ch MWahl
5/8/14

```
*****************************************************
*                                                   *
*           STAAD.Pro V8i SELECTseries4             *
*           Version  20.07.09.31                    *
*           Proprietary Program of                  *
*           Bentley Systems, Inc.                   *
*           Date=    APR 16, 2014                   *
*           Time=    15:12:27                       *
*                                                   *
*      USER ID: CH2M HILL                           *
*****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Major Finger Pier.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9 0; 5 0 12 0; 7 0 26.25 0; 8 0 3[illegible].5 0
     9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
    10. 18 0 -6 0; 19 0 -3 0; 20 0 0 0
    11. MEMBER INCIDENCES
    12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 1[illegible]
    13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
    14. START USER TABLE
    15. TABLE 2
    16. UNIT INCHES KIP
    17. WIDE FLANGE
    18. TOP
    19. 17.7807 13.8 0.485 14.7 0.39 604.828 206.598 1.07645 6.693 11.466
    20. MID
    21. 18.695 13.8 0.417 14.7 0.453 667.855 239.905 1.22266 5.7546 13.3182
    22. MUDLINE
    23. 17.8222 13.8 0.44 14.7 0.412 623.053 218.213 1.05381 6.072 12.1128
    24. END
    25. UNIT FEET KIP
    26. DEFINE MATERIAL START
    27. ISOTROPIC STEEL
    28. E 4.176E+006
    29. POISSON 0.3
    30. DENSITY 0.489024
    31. ALPHA 6.5E-006
    32. DAMP 0.03
    33. END DEFINE MATERIAL
    34. UNIT INCHES KIP
    35. CONSTANTS
    36. BETA 0 MEMB 8 9
    37. MATERIAL STEEL ALL
    38. MEMBER PROPERTY AMERICAN
```

```
39. 1 24 TO 26 TABLE ST HP14X89
40. MEMBER PROPERTY AMERICAN
41. 9 UPTABLE 2 TOP
42. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
43. MEMBER PROPERTY AMERICAN
44. 7 8 UPTABLE 2 MID
45. SUPPORTS
46. 1 FIXED
47. 10 FIXED BUT FY
48. MEMBER RELEASE
49. 9 END MPY 0.5 MPZ 0.5
50. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
51. JOINT LOAD
52. 10 FY -175.6
53. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	15	NUMBER OF MEMBERS	14
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 79

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	4.86092
2	9.59280
3	13.18304
4	19.07985

54. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= APR 16,2014 TIME= 15:12:28 ****

```
************************************************************
*          For questions on STAAD.Pro, please contact      *
*          Bentley Systems or Partner offices              *
*                                                          *
*              Telephone              Web / Email          *
*   USA      +1  (714) 974-2500                            *
*   UK       +44 (0) 808 101 9246                          *
*   SINGAPORE +65 6225-6158                                *
*   FRANCE   +33 (0) 1 55238400                            *
*   GERMANY  +49 0931 40468                                *
*   INDIA    +91 (033) 4006-2021                           *
*   JAPAN    +81 (03)5952-6500     http://www.ctc-g.co.jp  *
*   CHINA    +86 21 6288 4040                              *
*   THAILAND +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
*   Worldwide  http://selectservices.bentley.com/en-US/    *
*                                                          *
************************************************************
```

CH2MHILL.

Job Name: HRPT P-40	Job No.
Subject: FP Mid Pile	Sheet No. 1
	Date
	Computed By: BC 5/14
	Checked By: CH MCCALL 5/8/14

Calc GB of Pile (Moderate Pile) ✓

Section

Top	Mid	Bot
F: .449 ✓	F: .478 ✓	F: .420 ✓
W: .43 ✓	W: .275 ✓	W: .420 ✓
A_s = 17.75 in ✓	A_s = 17.58 ✓	A_s = 17.79 ✓

From STAAD

Applied Load: 175.6 kip ✓

BF: 5.03 (Buckling Factor) ✓

EB Load: 883.27 kip ✓ [Critical section @ node 17 (Bottom section)]

mid pile

$F_e = \frac{883.26 \text{ kip}}{17.79 \text{ in}} = 49.65 \text{ ksi}$ ✓

$\frac{K}{r} = 0.140$ ✓

$\frac{KL}{r} = 75.92$ ✓

$F_{cr} = 0.658^{36/49.65} \cdot 36$

$= 26.5 \text{ ksi}$ ✓

$\frac{P_n}{\Omega} = \boxed{282.29 \text{ kip}}$ ✓

CH2MHILL.

Job No.

Sheet No. 2

Job Name

Date

Subject FP Mod Pile

Computed By

Checked By MWahl 5/8/14

MB BC 5/8/14

Check FLB @ Thin Flange (Bottom)

$b/t = \frac{7.35}{0.420} = 17.5 > .56 \cdot \sqrt{\frac{29000}{36}}$

$Q_s = 1.415 - 0.74\,(17.5)\,\sqrt{\frac{36}{29000}}$

$= .958$

Apply Q_s to (E 7.2)

$\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{QF_y}}$

$F_{cr} = .958\left[0.658^{\left(.958 \cdot 36/49.35\right)}\right] 36$

$= 25.79$ ksi

$\frac{P_n}{\Omega} = \boxed{271.73 \text{ kip}}$

L/C 4, 6, 11 → Figure 1

L/C 3, 5, 7, OK w/ Major or Scour pile

⇒ hence ok for moderate pile

Ck MWH
5/8/14

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.31                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    APR 16, 2014                  *
*           Time=    15:48:39                      *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Moderate Finger Pier.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0
     9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
    10. 18 0 -6 0; 19 0 -3 0; 20 0 0 0
    11. MEMBER INCIDENCES
    12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
    13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
    14. START USER TABLE
    15. TABLE 2
    16. UNIT INCHES KIP
    17. WIDE FLANGE
    18. TOP
    19. 18.7485 13.8 0.43 14.7 0.449 665.43 237.795 1.22902 5.934 13.2006
    20. MID
    21. 17.7912 13.8 0.42 14.7 0.42 629.017 222.437 1.04612 5.796 12.348
    22. MUDLINE
    23. 17.7912 13.8 0.42 14.7 0.42 629.017 222.437 1.04612 5.796 12.348
    24. END
    25. UNIT FEET KIP
    26. DEFINE MATERIAL START
    27. ISOTROPIC STEEL
    28. E 4.176E+006
    29. POISSON 0.3
    30. DENSITY 0.489024
    31. ALPHA 6.5E-006
    32. DAMP 0.03
    33. END DEFINE MATERIAL
    34. UNIT INCHES KIP
    35. CONSTANTS
    36. BETA 0 MEMB 8 9
    37. MATERIAL STEEL ALL
    38. MEMBER PROPERTY AMERICAN
```

```
39. 1 24 TO 26 TABLE ST HP14X89
40. MEMBER PROPERTY AMERICAN
41. 9 UPTABLE 2 TOP
42. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
43. MEMBER PROPERTY AMERICAN
44. 7 8 UPTABLE 2 MID
45. SUPPORTS
46. 1 FIXED
47. 10 FIXED BUT FY
48. MEMBER RELEASE
49. 9 END MPY 0.5 MPZ 0.5
50. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
51. JOINT LOAD
52. 10 FY -175.6
53. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	15	NUMBER OF MEMBERS	14
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 79

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	5.03402
2	9.84173
3	13.40354
4	19.42029

54. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= APR 16,2014 TIME= 15:48:40 ****

```
**************************************************************
*         For questions on STAAD.Pro, please contact         *
*         Bentley Systems or Partner offices                 *
*                                                            *
*             Telephone               Web / Email            *
*  USA        +1  (714) 974-2500                             *
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*  SINGAPORE  +65 6225-6158                                  *
*  FRANCE     +33 (0) 1 55238400                             *
*  GERMANY    +49 0931 40468                                 *
*  INDIA      +91 (033) 4006-2021                            *
*  JAPAN      +81 (03)5952-6500    http://www.ctc-g.co.jp    *
*  CHINA      +86 21 6288 4040                               *
*  THAILAND   +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                            *
*  Worldwide  http://selectservices.bentley.com/en-US/       *
*                                                            *
**************************************************************
```

CH2MHILL.

Job Name HRPT Pier 40 [illegible]

Subject Minor Pile

Job No.

Sheet No. 1

Date

Computed By

Checked By MW [illegible] 5/8/14

MB DC 5/8/14

Calc Global Buckling [illegible] Pile (Minor Pile)

Section

Top	Mid	Bot
F: .511" √	F: .421 √	F: .52 √
W: .608" √	W: .493" √	W: .545" √
A_s = 15.61 in √	A_s = 18.76 in √	A_s = 22.24 in √

From Staad

Applied: 175.6 kip tot √

BF: 5.91 √

E.B.L = 1037.8 kip √

F_e @ Critical Section (Node 17) [illegible] at mid pile where global buckling is most critical

$$F_e = \frac{1037.8 \text{ kip}}{22.24 \text{ in}^2} = 46.66 \text{ ksi} \checkmark$$

$$\frac{KL}{r} = \sqrt{\frac{\pi^2 E}{F_e}} = 78.32 \checkmark$$

Use (E 3-2) √

$$F_{cr} = 0.658^{(36/46.66)} \cdot 36 = 26.06 \checkmark$$

$$\frac{P_n}{\Omega} = 347.1 \text{ kip} \checkmark$$

Check FLB

$$b/t = \frac{7.35}{.52} = 14.13 < \lambda_r \therefore \text{No FLB} \checkmark$$

*Did not consider Moment/bending as all L/C are satisfied with Major or Sem Piles √

ch MW-1
5/8/14

```
 ****************************************************
 *                                                  *
 *           STAAD.Pro V8i SELECTseries4            *
 *           Version  20.07.09.31                   *
 *           Proprietary Program of                 *
 *           Bentley Systems, Inc.                  *
 *           Date=    APR 16, 2014                  *
 *           Time=    16: 9:52                      *
 *                                                  *
 *      USER ID: CH2M HILL                          *
 ****************************************************
```

```
    1. STAAD SPACE
INPUT FILE: Pier40_Individual_Column_Shed Minor Finger Pier.STD
    2. START JOB INFORMATION
    3. ENGINEER DATE 12-MARCH-2014
    4. END JOB INFORMATION
    5. INPUT WIDTH 79
    6. UNIT FEET KIP
    7. JOINT COORDINATES
    8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0
    9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
   10. 18 0 -6 0; 19 0 -3 0; 20 0 0 0
   11. MEMBER INCIDENCES
   12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
   13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
   14. START USER TABLE
   15. TABLE 2
   16. UNIT INCHES KIP
   17  WIDE FLANGE
   18. TOP
   19. 25.6108 13.8 0.608 14.7 0.611 882.563 323.711 3.1777 8.3904 17.9634
   20. MID
   21. 18.7657 13.8 0.493 14.7 0.421 643.452 223.015 1.24882 6.8034 12.3774
   22. MUDLINE
   23. 22.2422 13.8 0.545 14.7 0 52 768.742 275.471 2.06648 7.521 15.288
   24. END
   25. UNIT FEET KIP
   26. DEFINE MATERIAL START
   27. ISOTROPIC STEEL
   28. E 4.176E+006
   29. POISSON 0.3
   30. DENSITY 0.489024
   31. ALPHA 6.5E-006
   32. DAMP 0.03
   33. END DEFINE MATERIAL
   34. UNIT INCHES KIP
   35. CONSTANTS
   36. BETA 0 MEMB 8 9
   37. MATERIAL STEEL ALL
   38. MEMBER PROPERTY AMERICAN
```

```
39. 1 24 TO 26 TABLE ST HP14X89
40. MEMBER PROPERTY AMERICAN
41. 9 UPTABLE 2 TOP
42. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
43. MEMBER PROPERTY AMERICAN
44. 7 8 UPTABLE 2 MID
45. SUPPORTS
46. 1 FIXED
47. 10 FIXED BUT FY
48. MEMBER RELEASE
49. 9 END MPY 0.5 MPZ 0.5
50. LOAD 1 LOADTYPE DEAD  TITLE DEAD+WEARING SURFACE
51. JOINT LOAD
52. 10 FY -175.6
53. PERFORM BUCKLING ANALYSIS MAXSTEP 200
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	15	NUMBER OF MEMBERS	14
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 79

CALCULATED BUCKLING FACTORS FOR LOAD CASE 1

MODE	BUCKLING FACTOR
1	5.91744
2	11.78776
3	15.70296
4	22.69195

54. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= APR 16,2014 TIME= 16: 9:52 ****

```
**********************************************************
*        For questions on STAAD.Pro, please contact      *
*        Bentley Systems or Partner offices              *
*                                                        *
*            Telephone            Web / Email            *
*  USA       +1  (714) 974-2500                          *
*  UK        +44 (0) 808 101 9246                        *
*  SINGAPORE +65 6225-6158                               *
*  FRANCE    +33 (0) 1 55238400                          *
*  GERMANY   +49 0931 40468                              *
*  INDIA     +91 (033) 4006-2021                         *
*  JAPAN     +81 (03)5952-6500    http://www.ctc-g.co.jp *
*  CHINA     +86 21 6288 4040                            *
*  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                        *
*  Worldwide  http://selectservices.bentley.com/en-US/   *
*                                                        *
**********************************************************
```

CH2MHILL SUBJECT Appendix 1 BY
SHEET NO. __ of __ DATE
PROJECT NO.

MB BC 5/15/2014

Finger Pier
Ice load w/New loads

New loads from Alex W are Based On Most stressed Pile in Sap 2000 Model

L/C 8

Top of Pile: 714 kin
Bot of Pile: 373 kin
Axial: -6.2 kip

Both load Cases include D_L and HRPT Allowable Over Stress

L/C 9

Top of Pile: 684.5 kin
Bot of pile: 378 kin
Axial: -8.2 kip

L/C 8

	Lateral	Axial
Severe Pile: U/R	2.309 @ Top	N/G
Mayor Pile: U/R	1.246 @ Top	N/G 0.918
Mod. Pile: U/R	1.033 @ Top	$\left(\frac{6.2}{274}\right) = .022 + \left(\frac{1}{1.9}\right)(1.033) = 0.94$ Mod Pile ok
Minor Pile: U/R	0.945 @ Mid Pile	

L/C 9

	Lateral	Axial
Severe Pile U/R	2.214 @ Top	N/G
Mayor Pile U/R	1.195 @ Top	N/G
Mod Pile U/R	0.989 @ Top	$\frac{8.2k}{274k} = 0.029$ ok w/ Moderate pile
Minor Pile U/R	0.905 @ Mid	

Mod Pile

MB BC 5/15/2014

```
*****************************************************
*                                                   *
*          STAAD.Pro V8i SELECTseries4              *
*          Version  20.07.09.31                     *
*          Proprietary Program of                   *
*          Bentley Systems, Inc.                    *
*          Date=    MAY 15, 2014                    *
*          Time=    14:16:57                        *
*                                                   *
*     USER ID: CH2M HILL                            *
*****************************************************

    1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending_ Finger Pier_LC9.STD
    2. START JOB INFORMATION
    3. ENGINEER DATE 12-MARCH-2014
    4. END JOB INFORMATION
    5. INPUT WIDTH 79
    6. UNIT FEET KIP
    7. JOINT COORDINATES
    8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9.89 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0
    9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
   10. 18 0 -6 0; 19 0 -3 0; 20 0 0.83 0
   11. MEMBER INCIDENCES
   12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
   13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
   14. START USER TABLE
   15. TABLE 1
   16. UNIT FEET KIP
   17. WIDE FLANGE
   18. ORIGINAL
   19. 0.179247 1.15 0.05125 1.225 0.05125 0.0428327 0.0157136 0.000156934 -
   20. 0.0589375 0.125563
   21. 0.25_THICK
   22. 0.0741319 1.15 0.0208333 1.225 0.0208333 0.0186353 0.00638368 1.07251E-005 -
   23. 0.0239583 0.0510417
   24. 0.375_THICK
   25. 0.110547 1.15 0.03125 1.225 0.03125 0.027312 0.00957706 3.59853E-005 -
   26. 0.0359375 0.0765625
   27. 0.4375_THICK
   28. 0.128592 1.15 0.0364583 1.225 0.0364583 0.0314958 0.0111744 5.69753E-005 -
   29. 0.0419271 0.0893229
   30. 0.5_THICK
   31. 0.146528 1.15 0.0416667 1.225 0.0416667 0.0355786 0.0127722 8.4796E-005 -
   32. 0.0479167 0.102083
   33. 0.3125_THICK
   34. 0.0923948 1.15 0.026042 1.225 0.026042 0.023026 0.0079803 2.0887E-005 -
   35. 0.0299483 0.0638029
   36. 0.125_THICK
   37. 0.0372842 1.15 0.010417 1.225 0.010417 0.00953593 0.00319164 1.34862E-006 -
   38. 0.0119796 0.0255217
```

```
39. TABLE 2
40. UNIT INCHES KIP
41. WIDE FLANGE
42. TOP
43. 18.7485 13.8 0.43 14.7 0.449 665.43 237.795 1.22902 5.934 13.2006
44. MID
45. 17.5853 13.8 0.275 14.7 0.478 672.35 253.085 1.15935 3.795 14.0532
46. MUDLINE
47. 17.7912 13.8 0.42 14.7 0.42 629.017 222.437 1.04612 5.796 12.348
48. 50%PILE
49. 14.5475 13.8 0.575 13.8 0.25 429.481 109.714 0.986568 7.935 6.9
50. 50%FLG12
51. 13.6475 13.8 0.575 12 0.25 388.166 72.2107 0.967818 7.935 6
52. 33%PILE
53. 12.245 13.8 0.475 14.7 0.2 367.152 106.004 0.557101 6.555 5.88
54. END
55. UNIT FEET KIP
56. DEFINE MATERIAL START
57. ISOTROPIC STEEL
58. E 4.176E+006
59. POISSON 0.3
60. DENSITY 0.489024
61. ALPHA 6.5E-006
62. DAMP 0.03
63. END DEFINE MATERIAL
64. UNIT INCHES KIP
65. CONSTANTS
66. BETA 0 MEMB 8 9
67. MATERIAL STEEL ALL
68. MEMBER PROPERTY AMERICAN
69. 1 24 TO 26 TABLE ST HP14X89
70. MEMBER PROPERTY AMERICAN
71. 9 UPTABLE 2 TOP
72. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
73. MEMBER PROPERTY AMERICAN
74. 7 8 UPTABLE 2 MID
75. SUPPORTS
76. 1 10 PINNED
77. LOAD 3 LOADTYPE WIND  TITLE WAVE
78. JOINT LOAD
79. 1 MX 378
80. 10 MX 684
82. *LOAD 4 LOADTYPE WIND  TITLE WIND
83. *JOINT LOAD
84. *1 MZ 66
85. *10 MZ 77
86. *LOAD 5 LOADTYPE WIND  TITLE CURRENT
87. *JOINT LOAD
88. *1 MZ 20
89. *10 MZ 35
90. *LOAD 7 LOADTYPE WIND  TITLE ICE
91. *JOINT LOAD
92. *1 MZ 49.2
93. *10 MZ 844.8
94. *LOAD COMB 2 HRPT LOAD CASE 9
95. *3 0.67 4 0.67 5 0.67 7 0.67
```

```
 96. PERFORM ANALYSIS


        P R O B L E M   S T A T I S T I C S
        -----------------------------------

  NUMBER OF JOINTS          15  NUMBER OF MEMBERS       14
  NUMBER OF PLATES           0  NUMBER OF SOLIDS         0
  NUMBER OF SURFACES         0  NUMBER OF SUPPORTS       2


       SOLVER USED IS THE IN-CORE ADVANCED SOLVER

  TOTAL PRIMARY LOAD CASES =      1, TOTAL DEGREES OF FREEDOM =      84

***WARNING    INSTABILITY AT JOINT       3   DIRECTION   RY
 PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
 K-MATRIX DIAG=  5.7408111E+02 L-MATRIX DIAG=  3.9790393E-13 EQN NO       14
 ***NOTE    VERY WEAK SPRING ADDED FOR STABILITY


  97. PARAMETER 1
  98. CODE AISC
  99. FYLD 36 ALL
 100. LX 372 MEMB 3 4 7 TO 9 20 TO 23
 101. UNB 372 MEMB 3 4 7 TO 9 20 TO 23
 102. UNT 372 MEMB 3 4 7 TO 9 20 TO 23
 103. LX 180 MEMB 1 2 24 TO 26
 104. UNB 180 MEMB 1 2 24 TO 26
 105. UNT 180 MEMB 1 2 24 TO 26
 106. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.330	3
		0.00 T	-378.00	0.00	0.00
2 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.159	3
		0.00 T	-100.96	0.00	0.00
3 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.092	3
		0.00 T	58.11	0.00	46.68
4 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.169	3
		0.00 T	106.83	0.00	25.32
7 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.732	3
		0.00 T	557.02	0.00	63.00
8 ST	MID		(UPT)		
		PASS	AISC- H1-3	0.755	3
		0.00 T	574.34	0.00	9.00
9 ST	TOP		(UPT)		
		PASS	AISC- H1-3	0.989	3
		0.00 T	684.00	0.00	57.00
20 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.688	3
		0.00 T	435.82	0.00	0.00
21 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.429	3
		0.00 T	271.32	0.00	0.00
22 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.558	3
		0.00 T	353.57	0.00	0.00
23 ST	MUDLINE		(UPT)		
		PASS	AISC- H1-3	0.299	3
		0.00 T	189.07	0.00	0.00
24 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.270	3
		0.00 T	-308.74	0.00	0.00
25 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.209	3
		0.00 T	239.48	0.00	0.00
26 ST	HP14X89		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.132	3
		0.00 T	-151.06	0.00	0.00

107. *LOAD LIST 6
108. FINISH

```
 *********** END OF THE STAAD.Pro RUN ***********

   **** DATE= MAY 15,2014   TIME= 14:16:58 ****

************************************************************
*          For questions on STAAD.Pro, please contact       *
*          Bentley Systems or Partner offices               *
*                                                           *
*              Telephone              Web / Email           *
*  USA       +1  (714) 974-2500                             *
*  UK        +44 (0) 808 101 9246                           *
*  SINGAPORE +65 6225-6158                                  *
*  FRANCE    +33 (0) 1 55238400                             *
*  GERMANY   +49 0931 40468                                 *
*  INDIA     +91 (033) 4006-2021                            *
*  JAPAN     +81 (03)5952-6500    http://www.ctc-g.co.jp    *
*  CHINA     +86 21 6288 4040                               *
*  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                           *
*  Worldwide  http://selectservices.bentley.com/en-US/      *
*                                                           *
************************************************************
```

Mod Pile

MB BC 5/15/2014

```
****************************************************
*                                                  *
*          STAAD.Pro V8i SELECTseries4             *
*          Version  20.07.09.31                    *
*          Proprietary Program of                  *
*          Bentley Systems, Inc.                   *
*          Date=    MAY 15, 2014                   *
*          Time=    14:20:26                       *
*                                                  *
*      USER ID: CH2M HILL                          *
****************************************************
```

```
     1. STAAD SPACE
INPUT FILE: Pier40_Individual_Bending__Finger Pier_LC8.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 12-MARCH-2014
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT FEET KIP
     7. JOINT COORDINATES
     8. 1 0 -9 0; 2 0 3 0; 3 0 6 0; 4 0 9.89 0; 5 0 12 0; 7 0 26.25 0; 8 0 31.5 0
     9. 9 0 32.25 0; 10 0 37 0; 15 0 19.125 0; 16 0 22.6875 0; 17 0 15.5625 0
    10. 18 0 -5.53 0; 19 0 -3 0; 20 0 -0.27 0
    11. MEMBER INCIDENCES
    12. 1 1 18; 2 2 3; 3 3 4; 4 4 5; 7 7 8; 8 8 9; 9 9 10; 20 7 16; 21 15 17; 22 16 15
    13. 23 17 5; 24 18 19; 25 19 20; 26 20 2
    14. START USER TABLE
    16. TABLE 2
    17. UNIT INCHES KIP
    18. WIDE FLANGE
    19. TOP
    20. 18.7485 13.8 0.43 14.7 0.449 665.43 237.795 1.22902 5.934 13.2006
    21. MID
    22. 17.5853 13.8 0.275 14.7 0.478 672.35 253.085 1.15935 3.795 14.0532
    23. MUDLINE
    24. 17.7912 13.8 0.42 14.7 0.42 629.017 222.437 1.04612 5.796 12.348
    26. END
    27. UNIT FEET KIP
    28. DEFINE MATERIAL START
    29. ISOTROPIC STEEL
    30. E 4.176E+006
    31. POISSON 0.3
    32. DENSITY 0.489024
    33. ALPHA 6.5E-006
    34. DAMP 0.03
    35. END DEFINE MATERIAL
    36. UNIT INCHES KIP
    37. CONSTANTS
    38. BETA 0 MEMB 8 9
    39. MATERIAL STEEL ALL
    41. MEMBER PROPERTY AMERICAN
```

```
42. 1 24 TO 26 TABLE ST HP14X89
43. MEMBER PROPERTY AMERICAN
44. 9 UPTABLE 2 TOP
45. 2 TO 4 20 TO 23 UPTABLE 2 MUDLINE
46. MEMBER PROPERTY AMERICAN
47. 7 8 UPTABLE 2 MID
48. SUPPORTS
49. 1 10 PINNED
50. LOAD 4 LOADTYPE ICE  TITLE ICE (MAJOR AXIS)
51. JOINT LOAD
52. 1 MX 393
53. 10 MX 714
55. *1 MZ 376
56. *10 MZ 840
58. LOAD COMB 7 HRPT LOAD CASE 8
59. 4 1
60. *4 0.714
62. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS	15	NUMBER OF MEMBERS	14
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 84

```
***WARNING- INSTABILITY AT JOINT [illegible] DIRECTION = RY
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG= 5.7408111E+02 L-MATRIX DIAG= 1.0231815E-12 EQN NO 14
***NOTE - VERY WEAK SPRING ADDED FOR STABILITY
```

```
63. PARAMETER 1
64. CODE AISC
65. FYLD 36 ALL
66. LX 372 MEMB 9 3 4 23 21 22 20 7
67. UNB 372 MEMB 9 3 4 23 21 22 20 7
68. UNT 372 MEMB 9 3 4 23 21 22 20 7
69. LX 180 MEMB 2 26 25 24 1
70. UNB 180 MEMB 2 26 25 24 1
71. UNT 180 MEMB 2 26 25 24 1
72. CHECK CODE ALL
```

STAAD.Pro CODE CHECKING - (AISC 9TH EDITION) v1.0
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

```
MEMBER     TABLE       RESULT/   CRITICAL COND/     RATIO/     LOADING/
                         FX            MY              MZ       LOCATION
=======================================================================

     1 ST   HP14X89                   (AISC SECTIONS)
                         PASS     AISC- H1-3         0.343          4
                       0.00 T      -393.00           0.00        0.00
     2 ST   MUDLINE                   (UPT)
                         PASS     AISC- H1-3         0.165          4
                       0.00 T      -104.22           0.00        0.00
     3 ST   MUDLINE                   (UPT)
                         PASS     AISC- H1-3         0.097          4
                       0.00 T        61.59           0.00       46.68
     4 ST   MUDLINE                   (UPT)
                         PASS     AISC- H1-3         0.177          4
                       0.00 T       112.37           0.00       25.32
     7 ST   MID                       (UPT)
                         PASS     AISC- H1-3         0.765          4
                       0.00 T       581.64           0.00       63.00
     8 ST   MID                       (UPT)
                         PASS     AISC- H1-3         0.788          4
                       0.00 T       599.69           0.00        9.00
*    9 ST   TOP                       (UPT)
                         FAIL     AISC- H1-3         1.033          4
                       0.00 T       714.00           0.00       57.00
    20 ST   MUDLINE                   (UPT)
                         PASS     AISC- H1-3         0.719          4
                       0.00 T       455.30           0.00        0.00
    21 ST   MUDLINE                   (UPT)
                         PASS     AISC- H1-3         0.448          4
                       0.00 T       283.83           0.00        0.00
    22 ST   MUDLINE                   (UPT)
                         PASS     AISC- H1-3         0.584          4
                       0.00 T       369.57           0.00        0.00
    23 ST   MUDLINE                   (UPT)
                         PASS     AISC- H1-3         0.313          4
                       0.00 T       198.10           0.00        0.00
    24 ST   HP14X89                   (AISC SECTIONS)
                         PASS     AISC- H1-3         0.270          4
                       0.00 T      -309.49           0.00        0.00
    25 ST   HP14X89                   (AISC SECTIONS)
                         PASS     AISC- H1-3         0.217          4
                       0.00 T      -248.61           0.00        0.00
    26 ST   HP14X89                   (AISC SECTIONS)
                         PASS     AISC- H1-3         0.160          4
                       0.00 T      -182.91           0.00        0.00

  73. FINISH
```

```
*********** END OF THE STAAD.Pro RUN ***********

  **** DATE= MAY 15,2014   TIME= 14:20:27 ****

************************************************************
*        For questions on STAAD.Pro, please contact        *
*        Bentley Systems or Partner offices                *
*                                                          *
*             Telephone              Web / Email           *
*  USA       +1  (714) 974-2500                            *
*  UK        +44 (0) 808 101 9246                          *
*  SINGAPORE +65 6225-6158                                 *
*  FRANCE    +33 (0) 1 55238400                            *
*  GERMANY   +49 0931 40468                                *
*  INDIA     +91 (033) 4006-2021                           *
*  JAPAN     +81 (03)5952-6500    http://www.ctc-g.co.jp   *
*  CHINA     +86 21 6288 4040                              *
*  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
*  Worldwide  http://selectservices.bentley.com/en-US/     *
*                                                          *
************************************************************
```

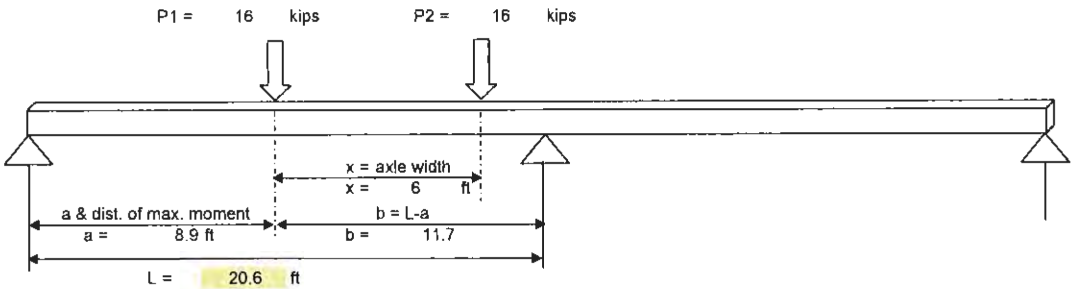
BEAM LOAD RATINGS – NOTES TO USER

Halcrow

Vehicle Particulars

Gross Vehicle Weight =	40,000 lbs	Front to Rear Axle Spacing =	14 ft
Rear Axle Weight =	32,000 lbs	Axle Width =	6 ft
Rear Axle Wheel Load =	16,000 lbs		
Front Axle Wheel Load =	4,000 lbs		

For Maximum Moment Due to Vehicle Load @ P1 Location

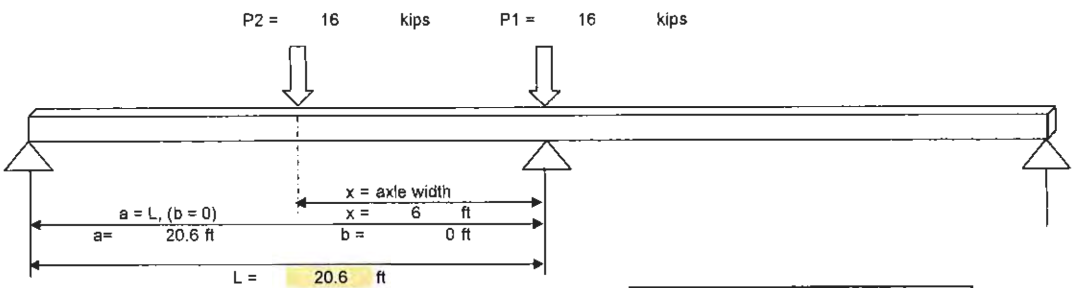


$$M_{P1}(@P_1) = \frac{P_1 ab}{4L^3}\left(4L^2 - a(L+a)\right)$$

$$M_{P2}(@P_1) = R1(a) = \frac{P_2(L-(a+x))}{4L^3}\left(4L^2 - (a+x)(L+a+x)\right)$$

Moment due to Vehicle Load Only	
M(P1) =	68.36799 k-ft
M(P2) =	62.78059 k-ft
Unfactored M_{max} =	131.15 k-ft
Factored M_{max} =	209.84 k-ft

For Maximum Shear Due to Vehicle Load @ Middle Support



$$V_{P1}(@\,P1) = \frac{P_1 a}{4L^3}\left(4L^2 + b(L+a)\right)$$

$$V_{P2}(@\,P1) = \frac{P_2(L-x)}{4L^3}\left(4L^2 + x(2L-x)\right)$$

Shear Due to Vehicle Load Only	
V(P1)	16 kips
V(P2)	12.75073 kips
Unfactored V_{max} =	28.75 kips
Factored V_{max} =	46.00 kips

BEAM LOAD RATINGS – PIER SHED

PIER SHED
300 PSF

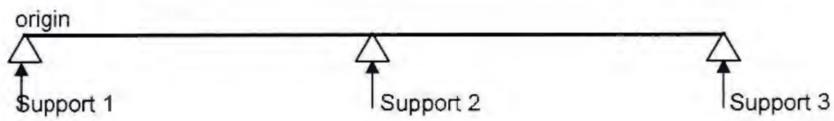
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	12.5
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	30
SDL (lb/ft^2)	0
LL (lb/ft^2)	300
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	0
Impact Factor	15%
Loc. of Max. Moment (ft)	0.0
Unfactored point load 2 (k)	0
Impact Factor	15%
Dist. from Max. Moment (ft)	6
Location from origin (ft)	6.0
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	12.5
Dist. of P2 from P1(ft)	6.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	45.9375	3.675	55.13	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	6.25	0.50	7.50	0.60
DL due to line load	0.00	0.00	0.00	0.00
Total DL	52.19	4.18	62.63	5.01
Total LL	78.75	6.30	126.00	10.08
Total Loading	130.9375	10.475	**188.625**	**15.09**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	115.08	165.78
Max negative moment (k-ft):	-204.59	-294.73
Maximum Shear (k):	55.65	80.17

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

PIER SHED
300 PSF

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	30	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	1	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	2.1592	
Es	0.0328	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	11	1.41	1.56	N/A	0.5
2	3	9	0	0	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

PIER SHED
300 PSF

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	4.68	2.34	Area of steel in bottom layer
$As_2 (in^2)$	0.00	0.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	4.68	2.34	Total area of steel
e (in)	0.71	0.71	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	25.80	25.80	Depth from top compression fiber to center of steel
a (in)	3.67	1.84	Depth of compression section
M_n (k*in)	4485.26	2328.52	
ΦM_n (k*in)	4036.73	2095.67	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	50.55	50.55
V_s (k)	117.92	58.96
$V_n = V_c + V_s$ (k)	168.47	109.51
ΦV_n (k)	126.35	82.13

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}\, bd \text{ [ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \text{ [ACI 318-05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
336.39	174.64	165.78	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
126.35	82.13	80.166	N/A

Halcrow

PIER SHED
H20 TRUCK

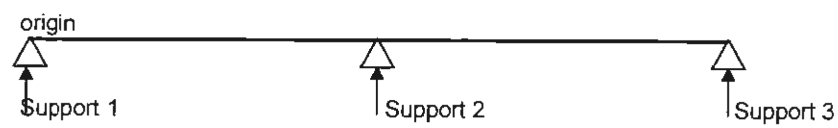
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	12.5
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	30
SDL (lb/ft^2)	0
LL (lb/ft^2)	0
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	16
Impact Factor	15%
Loc. of Max. Moment (ft)	5.4
Unfactored point load 2 (k)	16
Impact Factor	15%
Dist. from Max. Moment (ft)	6
Location from origin (ft)	11.4
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	12.5
Dist. of P2 from P1(ft)	6.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	45.9375	3.675	55.13	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	6.25	0.50	7.50	0.60
DL due to line load	0.00	0.00	0.00	0.00
Total DL	52.19	4.18	62.63	5.01
Total LL	0.00	0.00	0.00	0.00
Total Loading	52.1875	4.175	**62.625**	**5.01**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	98.07	138.64
Max negative moment (k-ft):	-76.61	-89.96
Maximum Shear (k):	48.02	67.96

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

PIER SHED
H20 TRUCK

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	30	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	1	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]

Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	

c	2.1592
Es	0.0328

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	11	1.41	1.56	N/A	0.5
2	3	9	0	0	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

PIER SHED
H20 TRUCK

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	4.68	2.34	Area of steel in bottom layer
$As_2 (in^2)$	0.00	0.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	4.68	2.34	Total area of steel
e (in)	0.71	0.71	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	25.80	25.80	Depth from top compression fiber to center of steel
a (in)	3.67	1.84	Depth of compression section
M_n (k*in)	4485.26	2328.52	
ΦM_n (k*in)	4036.73	2095.67	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	50.55	50.55
V_s (k)	117.92	58.96
$V_n = V_c + V_s$ (k)	168.47	109.51
ΦV_n (k)	126.35	82.13

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}bd \text{ [ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \text{ [ACI 318-05 (11.5.7.2)]}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
336.39	174.64	138.64	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
126.35	82.13	67.956	N/A

Halcrow

PIER SHED
FIRE TRUCK

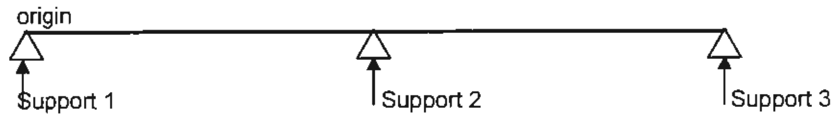
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	12.5
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	30
SDL (lb/ft^2)	0
LL (lb/ft^2)	0
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	24
Impact Factor	0%
Loc. of Max. Moment (ft)	5.4
Unfactored point load 2 (k)	24
Impact Factor	0%
Dist. from Max. Moment (ft)	8
Location from origin (ft)	13.4
Location btwn supports	Btwn 2&3

Loc. of Max. Shear (ft)	12.5
Dist. of P2 from P1(ft)	8.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	45.9375	3.675	55.13	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	6.25	0.50	7.50	0.60
DL due to line load	0.00	0.00	0.00	0.00
Total DL	52.19	4.18	62.63	5.01
Total LL	0.00	0.00	0.00	0.00
Total Loading	52.1875	4.175	**62.625**	**5.01**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	112.77	162.19
Max negative moment (k-ft):	-76.36	-89.56
Maximum Shear (k):	56.70	81.85

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	30	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	1	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	2.1592	
Es	0.0328	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	11	1.41	1.56	N/A	0.5
2	3	9	0	0	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	4.68	2.34	Area of steel in bottom layer
$As_2 (in^2)$	0.00	0.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	4.68	2.34	Total area of steel
e (in)	0.71	0.71	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	25.80	25.80	Depth from top compression fiber to center of steel
a (in)	3.67	1.84	Depth of compression section
M_n (k*in)	4485.26	2328.52	
ΦM_n (k*in)	4036.73	2095.67	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	50.55	50.55
V_s (k)	117.92	58.96
$V_n = V_c + V_s$ (k)	168.47	109.51
ΦV_n (k)	126.35	82.13

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$V_c = 2\sqrt{f'_c}\,bd \text{ [ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \text{ [ACI 318-05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
336.39	174.64	162.19	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
126.35	82.13	81.848	N/A

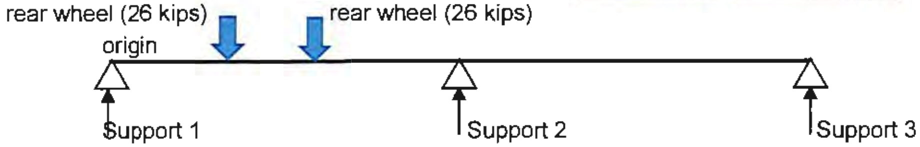
Halcrow

PIER SHED
PILE CAP CHECK
RESCUE TRUCK REAR AXLE WHEELS ON PILE CAP

Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	12.5
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16 BEAM B-11
Beam depth (in)	30 BEAM B-11
SDL (lb/ft^2)	0
LL (lb/ft^2)	0
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	45.9375	3.675	55.13	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	6.25	0.50	7.50	0.60
DL due to line load	0.00	0.00	0.00	0.00
Total DL	52.19	4.18	62.63	5.01
Total LL	0.00	0.00	0.00	0.00
Total Loading	52.1875	4.175	**62.625**	**5.01**



Point Loads	
Unfactored point load 1 (k)	26
Impact Factor	0%
Loc. of Max. Moment (ft)	5.4
Unfactored point load 2 (k)	26
Impact Factor	0%
Dist. from Max. Moment (ft)	8
Location from origin (ft)	13.4
Location btwn supports	Btwn 2&3

Loc. of Max. Shear (ft)	12.5
Dist. of P2 from P1(ft)	8.0

Rear wheels spaced 8 ft apart on the beam. Fire truck wheelbase is 17.2 ft.
Axle load = 52 kips so each wheel is 26 kips

Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	107.13	153.19
Max negative moment (k-ft):	-87.16	-106.83
Maximum Shear (k):	59.58	86.45

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	30	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	2.1592	
Es	0.0328	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	11	1.41	1.56	N/A	0.5
2	0	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^{2})	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

PIER SHED
PILE CAP CHECK
RESCUE TRUCK REAR AXLE WHEELS ON PILE CAP

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	4.68	2.34	Area of steel in bottom layer
$As_2 (in^2)$	0.00	0.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	4.68	2.34	Total area of steel
e (in)	0.71	0.71	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	25.80	25.80	Depth from top compression fiber to center of steel
a (in)	3.67	1.84	Depth of compression section
M_n (k*in)	4485.26	2328.52	
ΦM_n (k*in)	4036.73	2095.67	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	50.55	50.55
V_s (k)	117.92	58.96
$V_n = V_c + V_s$ (k)	168.47	109.51
ΦV_n (k)	126.35	82.13

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f_c'}\,bd \text{ [ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \text{ [ACI 318-05 (11.5.7.2)]}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
336.39	174.64	153.19	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
126.35	82.13	86.450	5.26%

Halcrow

PIER SHED
PILE CAP CHECK
RESCUE TRUCK REAR AXLE STRADDLING PILE CAP

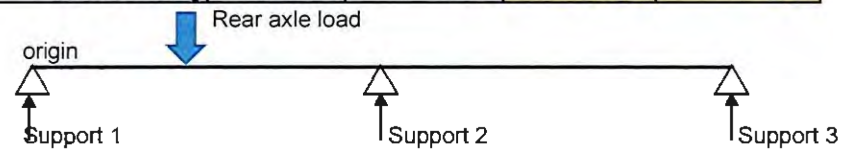
Loading Module

General Inputs		
Tributary width for beam (ft)	21	
Clear span of beam (ft)	12.5	
Weight of concrete (lb/ft^3)	150	
Slab thickness (in)	14	
Beam width (in)	16	Beam B-11
Beam depth (in)	30	Beam B-11
SDL (lb/ft^2)	0	
LL (lb/ft^2)	0	
Dead line load (k/ft)	0	
Dead Load Factor	1.2	
Live Load Factor	1.6	

Point Loads	
Unfactored point load 1 (k)	45
Impact Factor	15%
Loc. of Max. Moment (ft)	5.4
Unfactored point load 2 (k)	0
Impact Factor	0%
Dist. from Max. Moment (ft)	0
Location from origin (ft)	5.4
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	12.5
Dist. of P2 from P1(ft)	0.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	45.9375	3.675	55.13	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	6.25	0.50	7.50	0.60
DL due to line load	0.00	0.00	0.00	0.00
Total DL	52.19	4.18	62.63	5.01
Total LL	0.00	0.00	0.00	0.00
Total Loading	52.1875	4.175	**62.625**	**5.01**



Rear wheels spaced 8 ft apart on the beam. Fire truck wheelbase is 17.2 ft.
Axle load = 52 kips so each wheel is 26 kips
Actual rear axle wheel loads (spaced 8 ft apart) coming into beam is 42.1 kips - say 45 kips (see sketch)

Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	179.33	268.80
Max negative moment (k-ft):	-81.54	-97.85
Maximum Shear (k):	67.18	98.62

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	30	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	2.5910	
Es	0.0269	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	11	1.41	1.56	N/A	0.6
2	0	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

PIER SHED
PILE CAP CHECK
RESCUE TRUCK REAR AXLE STRADDLING PILE CAP

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	4.68	2.81	Area of steel in bottom layer
$As_2 (in^2)$	0.00	0.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	4.68	2.81	Total area of steel
e (in)	0.71	0.71	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	25.80	25.80	Depth from top compression fiber to center of steel
a (in)	3.67	2.20	Depth of compression section
M_n (k*in)	4485.26	2773.61	
ΦM_n (k*in)	4036.73	2496.25	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	50.55	50.55
V_s (k)	117.92	58.96
$V_n = V_c + V_s$ (k)	168.47	109.51
ΦV_n (k)	126.35	82.13

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}\,bd \text{ [ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \text{ [ACI 318-05 (11.5.7.2)]}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
336.39	208.02	268.80	29.22%
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
126.35	82.13	98.616	20.07%

BEAM LOAD RATINGS – TRUCK COURT

Halcrow

TRUCK COURT
150 PSF

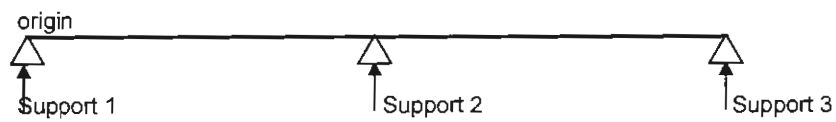
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	20.6
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	36
SDL (lb/ft^2)	0
LL (lb/ft^2)	150
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	0
Impact Factor	15%
Loc. of Max. Moment (ft)	0.0
Unfactored point load 2 (k)	0
Impact Factor	15%
Dist. from Max. Moment (ft)	6
Location from origin (ft)	6.0
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	20.6
Dist. of P2 from P1(ft)	6.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	75.705	3.675	90.85	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	12.36	0.60	14.83	0.72
DL due to line load	0.00	0.00	0.00	0.00
Total DL	88.07	4.28	105.68	5.13
Total LL	64.89	3.15	103.82	5.04
Total Loading	152.955	7.425	**209.502**	**10.17**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	221.54	303.45
Max negative moment (k-ft):	-393.86	-539.47
Maximum Shear (k):	73.32	100.43

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

TRUCK COURT
150 PSF

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	36	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]

Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	

c	4.1522
Es	0.0190

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	9	1.128	1	N/A	0.5
2	3	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

TRUCK COURT
150 PSF

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	3.00	1.50	Area of steel in bottom layer
$As_2 (in^2)$	3.00	3.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	6.00	4.50	Total area of steel
e (in)	1.63	1.98	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	30.87	30.52	Depth from top compression fiber to center of steel
a (in)	4.71	3.53	Depth of compression section
M_n (k*in)	6844.57	5175.47	
ΦM_n (k*in)	6160.12	4657.93	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	60.50	59.80
V_s (k)	141.13	69.75
$V_n = V_c + V_s$ (k)	201.63	129.56
ΦV_n (k)	151.22	97.17

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$V_c = 2\sqrt{f'_c}\,bd \ \text{[ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \ \text{[ACI 318-05 (11.5.7.2)]}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
513.34	388.16	303.45	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
151.22	97.17	100.429	3.36%

Halcrow

TRUCK COURT
H20 TRUCK
(rear axle at P1)

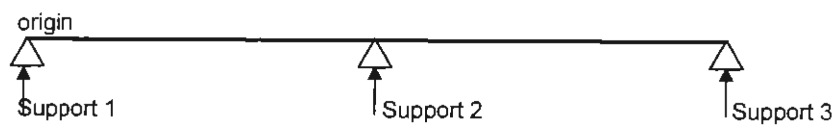
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	20.6
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	36
SDL (lb/ft^2)	0
LL (lb/ft^2)	0
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	27.4
Impact Factor	15%
Loc. of Max. Moment (ft)	8.9
Unfactored point load 2 (k)	0
Impact Factor	15%
Dist. from Max. Moment (ft)	6
Location from origin (ft)	14.9
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	20.6
Dist. of P2 from P1(ft)	6.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	75.705	3.675	90.85	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	12.36	0.60	14.83	0.72
DL due to line load	0.00	0.00	0.00	0.00
Total DL	88.07	4.28	105.68	5.13
Total LL	0.00	0.00	0.00	0.00
Total Loading	88.065	4.275	105.678	5.13



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	260.97	366.77
Max negative moment (k-ft):	-226.77	-272.12
Maximum Shear (k):	69.62	94.50

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

TRUCK COURT
H20 TRUCK
(rear axle at P1)

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	36	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]

Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	

c	4.1522
Es	0.0190

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	9	1.128	1	N/A	0.5
2	3	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in$^{2)}$	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

TRUCK COURT
H20 TRUCK
(rear axle at P1)

Beam Moment Capacity			
	Original	Reduced	
$As_1(in^2)$	3.00	1.50	Area of steel in bottom layer
$As_2\ (in^2)$	3.00	3.00	Area of steel in second layer
$As_3\ (in^2)$	0.00	0.00	Area of steel in third layer
$As_T\ (in^2)$	6.00	4.50	Total area of steel
e (in)	1.63	1.98	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	30.87	30.52	Depth from top compression fiber to center of steel
a (in)	4.71	3.53	Depth of compression section
M_n (k*in)	6844.57	5175.47	
ΦM_n (k*in)	6160.12	4657.93	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	60.50	59.80
V_s (k)	141.13	69.75
$V_n = V_c + V_s$ (k)	201.63	129.56
ΦV_n (k)	151.22	97.17

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f_c'}\,bd \ \ [\text{ACI 318-05 (11.3.1.1)}]$$

$$V_s = \frac{A_s f_y d}{s} \ \ [\text{ACI 318-05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
513.34	388.16	366.77	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
151.22	97.17	94.499	N/A

Halcrow

TRUCK COURT
H20 TRUCK
(both rear wheels on Span 1)

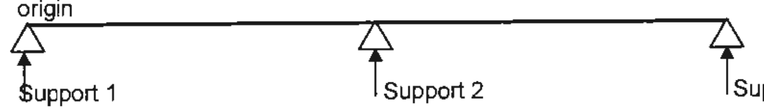
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	20.6
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	36
SDL (lb/ft^2)	0
LL (lb/ft^2)	0
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	16
Impact Factor	15%
Loc. of Max. Moment (ft)	8.9
Unfactored point load 2 (k)	16
Impact Factor	15%
Dist. from Max. Moment (ft)	6
Location from origin (ft)	14.9
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	20.6
Dist. of P2 from P1(ft)	6.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	75.705	3.675	90.85	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	12.36	0.60	14.83	0.72
DL due to line load	0.00	0.00	0.00	0.00
Total DL	88.07	4.28	105.68	5.13
Total LL	0.00	0.00	0.00	0.00
Total Loading	88.065	4.275	**105.678**	**5.13**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	236.50	327.50
Max negative moment (k-ft):	-195.58	-222.21
Maximum Shear (k):	70.97	96.66

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

TRUCK COURT
H20 TRUCK
(both rear wheels on Span 1)

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	36	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	4.1522	
Es	0.0190	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	9	1.128	1	N/A	0.5
2	3	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

TRUCK COURT
H20 TRUCK
(both rear wheels on Span 1)

Beam Moment Capacity			
	Original	Reduced	
As_1 (in^2)	3.00	1.50	Area of steel in bottom layer
As_2 (in^2)	3.00	3.00	Area of steel in second layer
As_3 (in^2)	0.00	0.00	Area of steel in third layer
As_T (in^2)	6.00	4.50	Total area of steel
e (in)	1.63	1.98	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	30.87	30.52	Depth from top compression fiber to center of steel
a (in)	4.71	3.53	Depth of compression section
M_n (k*in)	6844.57	5175.47	
ΦM_n (k*in)	6160.12	4657.93	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	60.50	59.80
V_s (k)	141.13	69.75
$V_n = V_c + V_s$ (k)	201.63	129.56
ΦV_n (k)	151.22	97.17

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}\,bd \quad \text{[ACI 318 - 05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \quad \text{[ACI 318 - 05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
513.34	388.16	327.50	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
151.22	97.17	96.660	N/A

Halcrow

TRUCK COURT
FIRE TRUCK
(rear axle at P1)

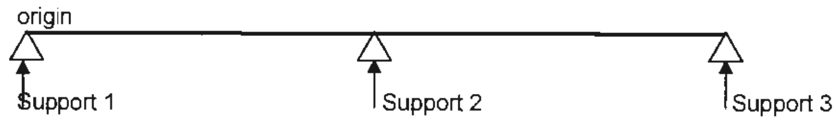
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	20.6
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	36
SDL (lb/ft^2)	0
LL (lb/ft^2)	0
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	38.7
Impact Factor	0%
Loc. of Max. Moment (ft)	8.9
Unfactored point load 2 (k)	0
Impact Factor	0%
Dist. from Max. Moment (ft)	6
Location from origin (ft)	14.9
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	20.6
Dist. of P2 from P1(ft)	6.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	75.705	3.675	90.85	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	12.36	0.60	14.83	0.72
DL due to line load	0.00	0.00	0.00	0.00
Total DL	88.07	4.28	105.68	5.13
Total LL	0.00	0.00	0.00	0.00
Total Loading	88.065	4.275	**105.678**	**5.13**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	291.54	415.73
Max negative moment (k-ft):	-226.77	-272.12
Maximum Shear (k):	80.92	112.58

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

TRUCK COURT
FIRE TRUCK
(rear axle at P1)

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	36	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	4.1522	
Es	0.0190	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	9	1.128	1	N/A	0.5
2	3	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

TRUCK COURT
FIRE TRUCK
(rear axle at P1)

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	3.00	1.50	Area of steel in bottom layer
$As_2 (in^2)$	3.00	3.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	6.00	4.50	Total area of steel
e (in)	1.63	1.98	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	30.87	30.52	Depth from top compression fiber to center of steel
a (in)	4.71	3.53	Depth of compression section
M_n (k*in)	6844.57	5175.47	
ΦM_n (k*in)	6160.12	4657.93	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	60.50	59.80
V_s (k)	141.13	69.75
$V_n = V_c + V_s$ (k)	201.63	129.56
ΦV_n (k)	151.22	97.17

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f_c'}bd \text{ [ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \text{ [ACI 318-05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
513.34	388.16	415.73	7.10%
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
151.22	97.17	112.579	15.86%

Halcrow

TRUCK COURT
FIRE TRUCK
(both rear wheel on Span 1)

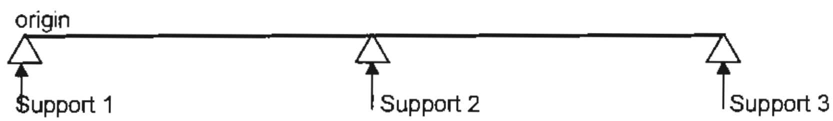
Loading Module

General Inputs	
Tributary width for beam (ft)	21
Clear span of beam (ft)	20.6
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	16
Beam depth (in)	36
SDL (lb/ft^2)	0
LL (lb/ft^2)	0
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	24
Impact Factor	0%
Loc. of Max. Moment (ft)	8.9
Unfactored point load 2 (k)	24
Impact Factor	0%
Dist. from Max. Moment (ft)	8
Location from origin (ft)	16.9
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	20.6
Dist. of P2 from P1(ft)	8.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	75.705	3.675	90.85	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	12.36	0.60	14.83	0.72
DL due to line load	0.00	0.00	0.00	0.00
Total DL	88.07	4.28	105.68	5.13
Total LL	0.00	0.00	0.00	0.00
Total Loading	88.065	4.275	**105.678**	**5.13**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	253.10	354.12
Max negative moment (k-ft):	-202.73	-233.65
Maximum Shear (k):	83.19	116.22

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

TRUCK COURT
FIRE TRUCK
(both rear wheel on Span 1)

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	36	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	4.1522	
Es	0.0190	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	9	1.128	1	N/A	0.5
2	3	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

TRUCK COURT
FIRE TRUCK
(both rear wheel on Span 1)

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	3.00	1.50	Area of steel in bottom layer
$As_2 (in^2)$	3.00	3.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	6.00	4.50	Total area of steel
e (in)	1.63	1.98	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	30.87	30.52	Depth from top compression fiber to center of steel
a (in)	4.71	3.53	Depth of compression section
M_n (k*in)	6844.57	5175.47	
ΦM_n (k*in)	6160.12	4657.93	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	60.50	59.80
V_s (k)	141.13	69.75
$V_n = V_c + V_s$ (k)	201.63	129.56
ΦV_n (k)	151.22	97.17

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}\, bd \text{ [ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \text{ [ACI 318-05 (11.5.7.2)]}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
513.34	388.16	354.12	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
151.22	97.17	116.221	19.61%

Halcrow

TRUCK COURT
PILE CAP CHECK
RESCUE TRUCK REAR AXLE STRADDLING PILE CAP

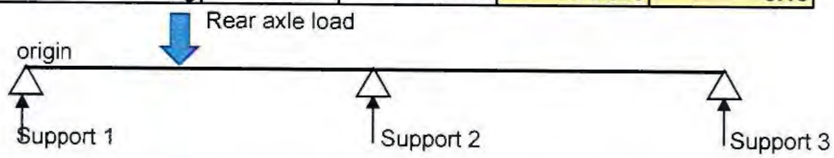
Loading Module

General Inputs		
Tributary width for beam (ft)	21	
Clear span of beam (ft)	20.6	
Weight of concrete (lb/ft^3)	150	
Slab thickness (in)	14	
Beam width (in)	16	Beam B-3
Beam depth (in)	36	Beam B-3
SDL (lb/ft^2)	0	
LL (lb/ft^2)	0	
Dead line load (k/ft)	0	
Dead Load Factor	1.2	
Live Load Factor	1.6	

Point Loads	
Unfactored point load 1 (k)	45
Impact Factor	15%
Loc. of Max. Moment (ft)	8.9
Unfactored point load 2 (k)	0
Impact Factor	0%
Dist. from Max. Moment (ft)	0
Location from origin (ft)	8.9
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	20.6
Dist. of P2 from P1(ft)	0.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	75.705	3.675	90.85	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	12.36	0.60	14.83	0.72
DL due to line load	0.00	0.00	0.00	0.00
Total DL	88.07	4.28	105.68	5.13
Total LL	0.00	0.00	0.00	0.00
Total Loading	88.065	4.275	**105.678**	**5.13**



Rear wheels spaced 8 ft apart on the beam. Fire truck wheelbase is 17.2 ft.
Axle load = 52 kips so each wheel is 26 kips
Actual rear axle wheel loads (spaced 8 ft apart) coming into beam is 42.1 kips - say 45 kips (see sketch)

Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	347.08	504.70
Max negative moment (k-ft):	-226.77	-272.12
Maximum Shear (k):	87.22	122.66

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

TRUCK COURT
PILE CAP CHECK
RESCUE TRUCK REAR AXLE STRADDLING PILE CAP

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	36	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]

Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	

c	4.1522
Es	0.0190

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	9	1.128	1	N/A	0.5
2	3	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

Halcrow

TRUCK COURT
PILE CAP CHECK
RESCUE TRUCK REAR AXLE STRADDLING PILE CAP

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	3.00	1.50	Area of steel in bottom layer
$As_2 (in^2)$	3.00	3.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	6.00	4.50	Total area of steel
e (in)	1.63	1.98	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	30.87	30.52	Depth from top compression fiber to center of steel
a (in)	4.71	3.53	Depth of compression section
M_n (k*in)	6844.57	5175.47	
ΦM_n (k*in)	6160.12	4657.93	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	60.50	59.80
V_s (k)	141.13	69.75
$V_n = V_c + V_s$ (k)	201.63	129.56
ΦV_n (k)	151.22	97.17

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f_c'}\,bd \ \text{[ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \ \text{[ACI 318-05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
513.34	388.16	504.70	30.02%
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
151.22	97.17	122.659	26.24%

Halcrow

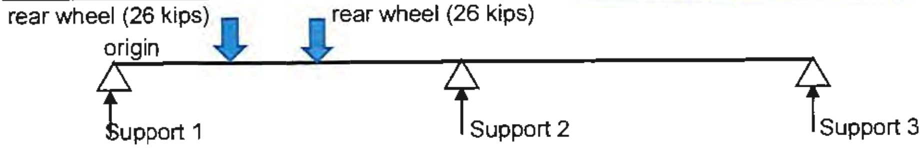
Loading Module

General Inputs		
Tributary width for beam (ft)	21	
Clear span of beam (ft)	20.6	
Weight of concrete (lb/ft^3)	150	
Slab thickness (in)	14	
Beam width (in)	16	BEAM B-3
Beam depth (in)	36	BEAM B-3
SDL (lb/ft^2)	0	
LL (lb/ft^2)	0	
Dead line load (k/ft)	0	
Dead Load Factor	1.2	
Live Load Factor	1.6	

Point Loads	
Unfactored point load 1 (k)	26
Impact Factor	0%
Loc. of Max. Moment (ft)	8.9
Unfactored point load 2 (k)	26
Impact Factor	0%
Dist. from Max. Moment (ft)	8
Location from origin (ft)	16.9
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	20.6
Dist. of P2 from P1(ft)	8.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	75.705	3.675	90.85	4.41
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	12.36	0.60	14.83	0.72
DL due to line load	0.00	0.00	0.00	0.00
Total DL	88.07	4.28	105.68	5.13
Total LL	0.00	0.00	0.00	0.00
Total Loading	88.065	4.275	**105.678**	**5.13**



Rear wheels spaced 8 ft apart on the beam. Fire truck wheelbase is 17.2 ft.
Axle load = 52 kips so each wheel is 26 kips

Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	263.60	370.93
Max negative moment (k-ft):	-200.72	-230.45
Maximum Shear (k):	86.61	121.68

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Halcrow

Concrete Beam Module

Beam Inputs		
b (in)	16	Width of beam
h (in)	36	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	2	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]
Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	
c	4.1522	
Es	0.0190	

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	3	9	1.128	1	N/A	0.5
2	3	9	1.128	1	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	3.5	0.5	0.2	0.5

TRUCK COURT
PILE CAP CHECK
RESCUE TRUCK REAR AXLE WHEELS ON PILE CAP

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	3.00	1.50	Area of steel in bottom layer
$As_2 (in^2)$	3.00	3.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	6.00	4.50	Total area of steel
e (in)	1.63	1.98	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	30.87	30.52	Depth from top compression fiber to center of steel
a (in)	4.71	3.53	Depth of compression section
M_n (k*in)	6844.57	5175.47	
ΦM_n (k*in)	6160.12	4657.93	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	60.50	59.80
V_s (k)	141.13	69.75
$V_n = V_c + V_s$ (k)	201.63	129.56
ΦV_n (k)	151.22	97.17

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}\,bd \quad \text{[ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \quad \text{[ACI 318-05 (11.5.7.2)]}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	**Reduced Moment Capacity (k-ft)**	**Factored Positive Moment (k-ft)**	**% Overstress**
513.34	388.16	370.93	N/A
Original Shear Capacity (k-ft)	**Reduced Shear Capacity (k)**	**Factored Shear (k)**	**% Overstress**
151.22	97.17	121.685	25.23%

BEAM LOAD RATINGS – FINGER PIER

Halcrow

FINGER PIER
275 PSF

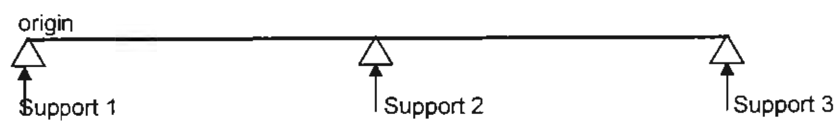
Loading Module

General Inputs	
Tributary width for beam (ft)	10
Clear span of beam (ft)	25.5
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	14
Beam width (in)	30
Beam depth (in)	48
SDL (lb/ft^2)	0
LL (lb/ft^2)	275
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	0
Impact Factor	0%
Loc. of Max. Moment (ft)	0.0
Unfactored point load 2 (k)	0
Impact Factor	0%
Dist. from Max. Moment (ft)	8
Location from origin (ft)	8.0
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	25.5
Dist. of P2 from P1(ft)	8.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	44.625	1.75	53.55	2.1
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	38.25	1.50	45.90	1.80
DL due to line load	0.00	0.00	0.00	0.00
Total DL	82.88	3.25	99.45	3.90
Total LL	70.13	2.75	112.20	4.40
Total Loading	153	6	**211.65**	**8.30**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	274.32	379.48
Max negative moment (k-ft):	-487.69	-674.63
Maximum Shear (k):	71.63	99.08

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Concrete Beam Module

Beam Inputs		
b (in)	30	Width of beam
h (in)	48	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	1	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]

Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	

c	1.4764
Es	0.0863

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	6	9	1.128	1	N/A	0.5
2	3	9	0	0	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	12	0.5	0.2	0.5

Halcrow

FINGER PIER
275 PSF

Beam Moment Capacity			
	Original	Reduced	
$As_1 (in^2)$	6.00	3.00	Area of steel in bottom layer
$As_2 (in^2)$	0.00	0.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	6.00	3.00	Total area of steel
e (in)	0.56	0.56	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	43.94	43.94	Depth from top compression fiber to center of steel
a (in)	2.51	1.25	Depth of compression section
M_n (k*in)	10243.46	5197.03	
ΦM_n (k*in)	9219.12	4677.32	

Beam Shear Capacity		
	Original	Reduced
V_c (k)	161.43	161.43
V_s (k)	58.58	29.29
$V_n = V_c + V_s$ (k)	220.01	190.72
ΦV_n (k)	165.01	143.04

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}\, bd \quad [\text{ACI 318-05 (11.3.1.1)}]$$

$$V_s = \frac{A_s f_y d}{s} \quad [\text{ACI 318-05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
768.26	389.78	379.48	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
165.01	143.04	99.081	N/A

DECK LOAD RATINGS – PRESTRESSED CONCRETE

Halcrow	**NOAM CALCULATION SHEET**	No: 1	Rev:
Project:	PIER 40 CONDITION SURVEY	By: MO	Date: 5/29/2009
Subject:	LOAD RATING FOR PRECAST PRESTRESSED PLANK WITH EXPOSED STRANDS	Check By: JC	Date: 5/30/2009

References/Results

MATERIAL PROPERITES

CONCRETE

F'c = 5000 PSI

REINFORCING DEFORMED BARS — DWG NO. F-23

Fy = 20000 PSI

HIGH TENSILE STEEL TENDONS - 7 WIRE

3/8 IN. DIAMETER A = 0.0799 in^2. — DWG NO. F-23

INITIAL PRESTRESS FORCE PER TENDON = 14000 LBS — DWG NO. F-23

DESIGN PRESTRESS FORCE PER TENDON 11000 LBS — DWG NO. F-23

STRESS IN TENDON AFTER PRESTRESS = 175219 PSI

ASSUME INITIAL PRESTRESS FORCE = 175219 PSI = 0.82Fpy — ACI 18.5.1

Fpy = 213681.7 PSI = 214 ksi

Fpy= 0.85Fpu Fpu = 251.4 ksi = 250 ksi

Fse = 137672.1 PSI = 137.7 ksi = EFFECTIVE STRESS IN PRESTRESSING

DETERMINE FLEXURAL CAPACITY OF PLANK

DUE TO THE SPALLS AND EXPOSED PRESTRESSING STRANDS ALONG THE BOTTOM OF THE PLANK, ASSUME ONLY THE SECOND LAYER OF PRESTRESSING REMAINS.

FOR THE PURPOSES OF THIS ANALYSIS, ASSUME PRECAST PLANK NUMBER P1 / P2 WHICH HAVE THE FOLLOWING PROPERTIES:

NUMBER OF STRANDS IN SECOND LAYER = 17 (assumes all strands in second layer are intact)

NUMBER OF STRANDS IN BOTTOM LAYER = 10 (assumes 10 out of 25 strands are still intact)

LENGTH OF PLANK = 20.5 FT

ORIGINAL DEPTH OF PLANK = 12 IN.

EFFECTIVE REMAINING DEPTH OF PLANK = 10.56 IN. (ADD 2 IN. TO CoG) — CoG of both layers of strands calculated to be 3.44 IN. from bottom of plank

CONCRETE COVER ON BOTTOM LAYER OF PRESTRESSING = 2 IN.

CONCRETE COVER ON SECOND LAYER OF PRESTRESSING = 4 IN.

B = WIDTH OF PLANK = 5 FT

Ap = AREA OF PRESTRESSING STEEL = 2.157 in^2. FOR BOTTOM AND SECOND LAYERS

DETERMINE THE RATIO OF EFFECTIVE PRESTRESS TO ULTIMATE STRENGTH OF STEEL

$\boxed{\dfrac{f_{pe}}{f_{pu}}}$ RATIO = 0.6 > 0.50, THEREFORE APPROXIMATE EQUATIONS BY ACI MAY BE USED — ACI 18.7.2

Halcrow NOAM CALCULATION SHEET	No: 2	Rev:
Project: PIER 40 CONDITION SURVEY	By: MO	Date: 5/29/2009
Subject: LOAD RATING FOR PRECAST PRESTRESSED PLANK WITH EXPOSED STRANDS	Check By: JC	Date: 5/30/2009

References/Results

CALCULATE THE SPAN TO DEPTH RATIO TO DETERMINE APPLICABLE EQUATION

ASSUME UNBONDED TNEDONS

SPAN TO DEPTH RATIO = 23.3 < 35 — ACI 18.7.2

$$f_{ps} = f_{se} + 10000 + \frac{f_c'}{100\rho_p}$$

stress in prestressing steel at nominal flexural strength — ACI EQUATION 18-4

fps shall not be taken greater than fpy and (fse+60000)

d_p = effective depth to prestressing steel = 8.56 IN.

ρ_p = A_p / bd_p = 0.0042 STEEL RATIO FAILURE

β_1 = 0.85 — ACI 10.2.7.3

f_{ps} = 159575.86 PSI = 159.6 KSI

CALCULATE NOMINAL FLEXURAL STRENGTH OF PLANK

$$M_n = A_p f_{ps}\left(d - \frac{a}{2}\right)$$

where $$a = \frac{A_p f_{ps}}{0.85 f_c' b}$$

a = 1.3500 in.

Mn = 2714.433 k-in = 226.2 ft-kips

a = 1.086 in.

Mn = 2759.944 k-in = 230 ft-kips

Φ = 0.9

DESIGN FLEXURAL STRENGTH = ΦM_n = 207.00 ft-kips — ΦMn = 207.00 ft-kips

DETERMINE REMAINING LIVE LOAD CAPACITY OF PLANK BASED ON FLEXURAL STRENGTH

DEAD LOAD =	15375 LBS	=	750	LB/FT	(12 in. PC PLANK ONLY)
			125	LB/FT	(2 IN. TOPPING)
TOTAL UNFACTORED DEAD LOAD =			875	LB/FT	
FACTORED DEAD LOAD w_D =			1050	LB/FT	(USE A FACTOR OF 1.2)

ACI 9.2

Halcrow	**NOAM CALCULATION SHEET**	No: 3	Rev:
Project:	PIER 40 CONDITION SURVEY	By: MO	Date: 5/29/2009
Subject:	LOAD RATING FOR PRECAST PRESTRESSED PLANK WITH EXPOSED STRANDS	Check By: JC	Date: 5/30/2009

References/Results

PRECAST PLANKS ARE SIMPLY SUPPORTED

DEAD LOAD MOMENT = $w_D L^2 / 8$ = 55157.81 ft-lbs
55.16 ft-kips

TOTAL REMAINING MOMENT CAPACITY IN PLANK = 151.84 ft-kips

UNFACTORED LIVE LOAD = 2.89 k/ft

Live Load Factor = 1.6

ALLOWABLE FACTORED LIVE LOAD = 1.81 k/ft

ALLOWABLE LIVE LOAD = 0.361 ksf = 361.3 PSF
say 360 PSF

ALLOWABLE LIVE LOAD BASED ON FLEXURE 360 PSF

ALLOWABLE POINT LOAD = 29.63 kips

CALCULATE THE SHEAR CAPACITY OF PLANK

$$\boxed{V_c = \left(0.6\sqrt{f_c'} + 700\frac{V_u d_p}{M_u}\right) b_w d}$$

ACI 11.4.2

Vc shall not be taken less than the following:

$\boxed{V_c = 2\sqrt{f_c'}\, b_w d}$ = 89604.57 LBS = 89.6 kips

DEAD LOAD SHEAR = $w_D L / 2$ = 10762.5 LBS = 10.76 kips

TOTAL REMAINING SHEAR CAPACITY IN PLANK = 78.84 kips

UNFACTORED LIVE LOAD = 7.69 k/ft

ALLOWABLE FACTORED LIVE LOAD = 4.81 k/ft > 2.89 k/ft

SINCE THE FACTORED LIVE LOAD BASED ON SHEAR IS HIGHER THAN THAT CALCULATED FOR FLEXURE, THE LIVE LOAD FOR FLEXURE GOVERNS.

CONCLUSIONS

ALTHOUGH THE PLANK HAS AN ALLOWABLE LIVE LOAD OF 360 PSF, THIS SHOULD BE REDUCED TO 300 PSF.

THIS APPLIES ONLY TO PRECAST PLANK NUMBERS P1 AND P2, AND PLANKS WITH SIMILAR PROPERTIES TO P1 AND P2.

ALLOWABLE LIVE LOAD

360 PSF

Halcrow	**NOAM CALCULATION SHEET**	No: 4	Rev:
Project:	PIER 40 CONDITION SURVEY	By: MO	Date: 5/29/2009
Subject:	LOAD RATING FOR PRECAST PRESTRESSED PLANK WITH EXPOSED STRANDS	Check By: JC	Date: 5/30/2009

CALCULATION OF CENTER OF GRAVITY OF PRESTRESSING STRAND LAYERS

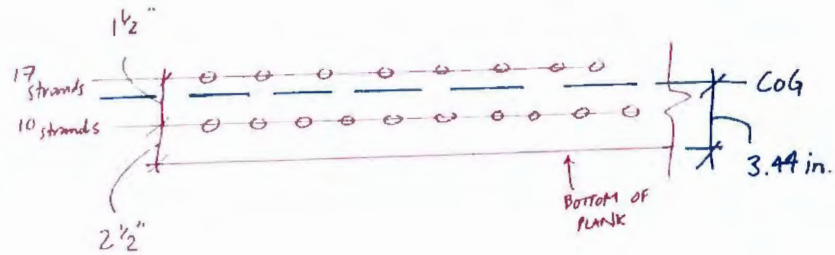
Assume plank's bottom layer of prestressing is half missing

No. of strands — area per strand per DWG F-23

2^{nd} layer = $17 \times 0.0799 = 1.3583$ in.^{2}

bot. layer = $10 \times 0.0799 = 0.799$ in.^{2}

2.1573 in^2



2^{nd} layer : $1.3583(4) = 5.4332$ in^3

bot layer : $0.799(2.5) = 1.9975$ in.^{3}

7.4307 in^3

$$\frac{7.4307}{2.1573} = 3.44 \text{ in. from bottom}$$

DECK LOAD RATINGS – CAST-IN-PLACE CONCRETE

Halcrow

FINGER PIER
DECK

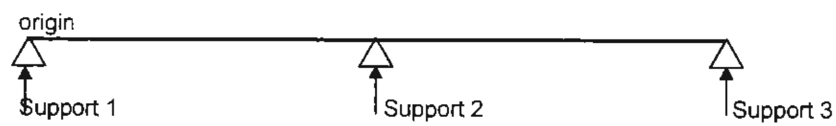
Loading Module

General Inputs	
Tributary width for beam (ft)	1
Clear span of beam (ft)	10.0
Weight of concrete (lb/ft^3)	150
Slab thickness (in)	0
Beam width (in)	12
Beam depth (in)	15
SDL (lb/ft^2)	0
LL (lb/ft^2)	275
Dead line load (k/ft)	0
Dead Load Factor	1.2
Live Load Factor	1.6

Point Loads	
Unfactored point load 1 (k)	0
Impact Factor	15%
Loc. of Max. Moment (ft)	0.0
Unfactored point load 2 (k)	0
Impact Factor	15%
Dist. from Max. Moment (ft)	6
Location from origin (ft)	6.0
Location btwn supports	Btwn 1&2

Loc. of Max. Shear (ft)	10.0
Dist. of P2 from P1(ft)	6.0

Loading	Unfactored Load		Factored Load	
	Total (k)	Distributed (k/ft)	Total (k)	Distributed (k/ft)
DL due to slab	0	0	0.00	0
DL due to SDL	0.00	0.00	0.00	0.00
DL due to self weght	1.88	0.19	2.25	0.23
DL due to line load	0.00	0.00	0.00	0.00
Total DL	1.88	0.19	2.25	0.23
Total LL	2.75	0.28	4.40	0.44
Total Loading	4.625	0.4625	**6.65**	**0.67**



Unfactored Moments	Unfactored	Factored
Max positive moment (k-ft):	3.25	4.67
Max negative moment (k-ft):	-5.78	-8.31
Maximum Shear (k):	2.31	3.33

Reactions at Support	Unfactored	Factored
Moment at Support 1 (k-ft):		
Moment at Support 2 (k-ft):		
Reaction at Support 1 (k):		
Reaction at Support 2 (k):		

Concrete Beam Module

Beam Inputs		
b (in)	12	Width of beam
h (in)	15	Depth of beam
f'c (ksi)	3.75	Strength of concrete
Wc (lb/ft^3)	150	Weight of concrete
fy (ksi)	40	Strength of steel reinforcing
Lc (bottom) (in)	3	Clear cover to bottom reinforcement
Lc (top) (in)	2	Clear cover to top reinforcement
εc	0.003	Max strain in concrete
εs	0.00138	Max strain in steel
β	0.85	[ACI 318-05 10.2.7.3]
Layers of rebar	1	

Φ (flexure)	0.9	[ACI 318-05 9.3.1]
Φ (shear)	0.75	[ACI 318-05 11.1]

Ec (ksi)	3712.5	[ACI 318-05 8.5.1]
Ey (ksi)	29000	

c	0.1907
Es	0.1730

Beam Longitudinal Reinforcing

Row of Reinforcing	Number of Bars	Bar Size	Reinforcing Bar Diameter (in.)	Area per Bar (in^2)	Clear Distance from Reinforcing Row Below (in.)	% Section Remaining (0≤x≤1)
1	1	5	0.625	0.31	N/A	0.5
2	3	3	0	0	1	1
3	0	0	0	0	1	1

Beam Stirrups

Bar Size	Distance o/c (in)	Diameter (in)	Area of steel (in^2)	% Section Remaining (0≤x≤1)
4	12	0.5	0.2	0.5

Beam Moment Capacity	Original	Reduced	
$As_1 (in^2)$	0.31	0.16	Area of steel in bottom layer
$As_2 (in^2)$	0.00	0.00	Area of steel in second layer
$As_3 (in^2)$	0.00	0.00	Area of steel in third layer
$As_T (in^2)$	0.31	0.16	Total area of steel
e (in)	0.31	0.31	Dist. to center of steel, 0 = bottom of long. reinforcement
d (in)	11.19	11.19	Depth from top compression fiber to center of steel
a (in)	0.32	0.16	Depth of compression section
M_n (k*in)	136.72	68.86	
ΦM_n (k*in)	123.04	61.97	

Beam Shear Capacity	Original	Reduced
V_c (k)	16.44	16.44
V_s (k)	14.92	7.46
$V_n = V_c + V_s$ (k)	31.36	23.90
ΦV_n (k)	23.52	17.93

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$V_c = 2\sqrt{f'_c}\,bd \ \text{[ACI 318-05 (11.3.1.1)]}$$

$$V_s = \frac{A_s f_y d}{s} \ \text{[ACI 318-05 (11.5.7.2)}$$

Results for Two Equal Span Continuous Beam			
Original Moment Capacity (k-ft)	Reduced Moment Capacity (k-ft)	Factored Positive Moment (k-ft)	% Overstress
10.25	5.16	4.67	N/A
Original Shear Capacity (k-ft)	Reduced Shear Capacity (k)	Factored Shear (k)	% Overstress
23.52	17.93	3.325	N/A

CURRENT AND ICE LOAD CALCULATION

Halcrow Yolles

Project: PIER 40				Project No.	No.
Design: AJW	Drawn:	Checked:	Date: 4/14/09	W.P. No.	Scale
Subject: CURRENT LOAD ON PILE				Reference	

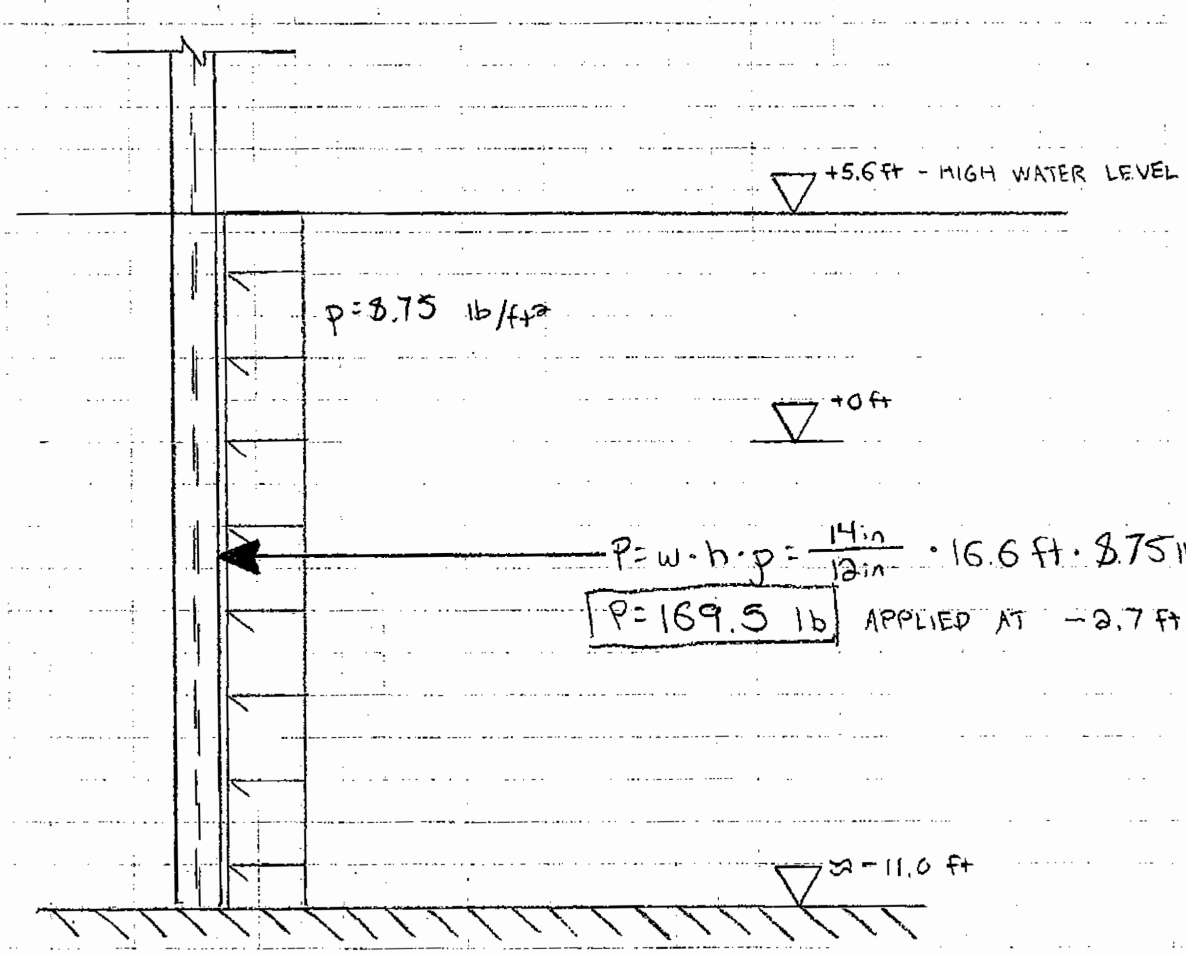
AS PER AASHTO STANDARD SPECIFICATIONS FOR BRIDGE DESIGN 2007 w/2008 REVISIONS

§ 3.7.3

$C_D = 1.4$

$V = 2.5\ ft/s$

$$p = \frac{C_D \cdot V^2}{1000} = \frac{1.4 \cdot (2.5\ ft/s)^2}{1000} = .00875\ K/ft^2 = 8.75\ lb/ft^2$$



$$P = w \cdot h \cdot p = \frac{14 in}{12 in} \cdot 16.6\ ft \cdot 8.75\ lb/ft^2$$

$\boxed{P = 169.5\ lb}$ APPLIED AT −2.7 ft

Halcrow Yolles

Project: PIER 40				Project No.	No.
Design: AJW	Drawn:	Checked:	Date: 4/13/09	W.P. No.	Scale
Subject: DYNAMIC ICE LOAD ON PILE				Reference	

AS PER AASHTO STANDARD SPECIFICATIONS FOR BRIDGE DESIGN 2007 w/ 2008 REVISIONS

§ 3.9.2.2

$w_{max} = 14 \text{ in} = 1\frac{1}{6} \text{ ft}$

$t = 8 \text{ in} = \frac{2}{3} \text{ ft}$

$p = 100 \text{ psi} = 14.4 \text{ ksf}$

$\frac{w}{t} = \frac{1\frac{1}{6} \text{ ft}}{\frac{2}{3} \text{ ft}} = 1.75 < 6$, THEREFORE $F = \min(F_c, F_B)$

$$F_c = C_a \cdot p \cdot t \cdot w$$

$$= \left(\frac{5t}{w+1}\right)^{.5} \cdot p \cdot t \cdot w$$

$$= \left(\frac{5 \cdot \frac{2}{3} \text{ ft}}{1\frac{1}{6} \text{ ft} + 1}\right)^{.5} \cdot 14.4 \text{ ksf} \cdot \frac{2}{3} \text{ ft} \cdot 1\frac{1}{6} \text{ ft}$$

$$= 13.89 \text{ K}$$

F_B = NEED NOT BE CONSIDERED BECAUSE $\alpha < 15$ AS PER C.3.9.2.2

$\boxed{F = 13.89 \text{ K}}$ AT +5.6 ft, HIGH WATER LEVEL

ICE ABRASION = $F_A = .11 \cdot F = .11 \cdot 13.89 = \boxed{1.53 \text{ K}}$

AT +5.6 ft, HIGH WATER LEVEL

WAVE LOAD CALCULATION

Halcrow, Inc
22 Cortlandt Street
New York, NY 10007
Tel (212) 608-3990 Fax (212) 566-5059
www.halcrow.com

Technical note

Project	Hudson River Park – Pier 40	**Date**	May 15, 2009
Note	Wave Forces on Pile Cap beam and piles at Pier 40 Expansion Bridge Structure	**Ref**	
Author	Furong Zhang, Atilla Bayram		

INTRODUCTION

This memorandum presents the results of wave force calculations on the edge beam and piles at the Pier 40 Finger Pier Extension in the Hudson River.

ENVIRONMENTAL PARAMETERS

Extreme wave climate at the site is provided in Hudson River Park Structural Design Guidelines Rev. B report prepared by Arup (Arup, 2001). Table 1 summarizes environmental parameters. Table 2 documents tidal planes published by NOAA for Battery Park City. It is based on 1983-2001 tidal epoch. Local water depth in front of the structure is taken as 9.14 m-MLLW (30 ft-MLLW) as it is given in project drawing. The deck level has been set at +3.42m-MLLW. The design water level is selected as+1.56 m-MLLW.

Table 1- Wave parameters for the Pier 40 Finger Pier Extension Structure

Direction	H_s (m)	T_p (s)	Local water depth (m-MLLW)
Southerly	3.04	5.40	9.14
Westerly	1.05	2.60	9.14

Table 2- Tidal Planes at Battery Park City Station

Tidal Planes	Water Level (m)
100 yr Flood, Level 3 Hurricane	3.62
Highest Observed Water Level (9/12/1960)	3.12
MHHW	**1.56**
MHW	1.46
NGVD 1929	0.57
MLW	0.07
MLLW	**0.00**
Lowest Observed (2/276)	-1.24

WAVE FORCES ON PILE CAP BEAM

The beams are 12.2 m in length (dimensions perpendicular to the direction of wave travel) with a cross-section of 1.3 m x 1.3 m (see Fig. 1). In order to calculate the forces on the beam the method by McConnell et al. (2004) was used. An excel spreadsheet was prepared to facilitate the calculations. Detailed calculations are attached. Table 3 summarizes predicted quasi-static and maximum (i.e. short duration impact) horizontal forces on the beam.

Uplift forces due to waves was not considered. The wave force on the Finger Pier Extension is considered to be the worst case condition and will therefore be applied to the Pier Shed structure as well.

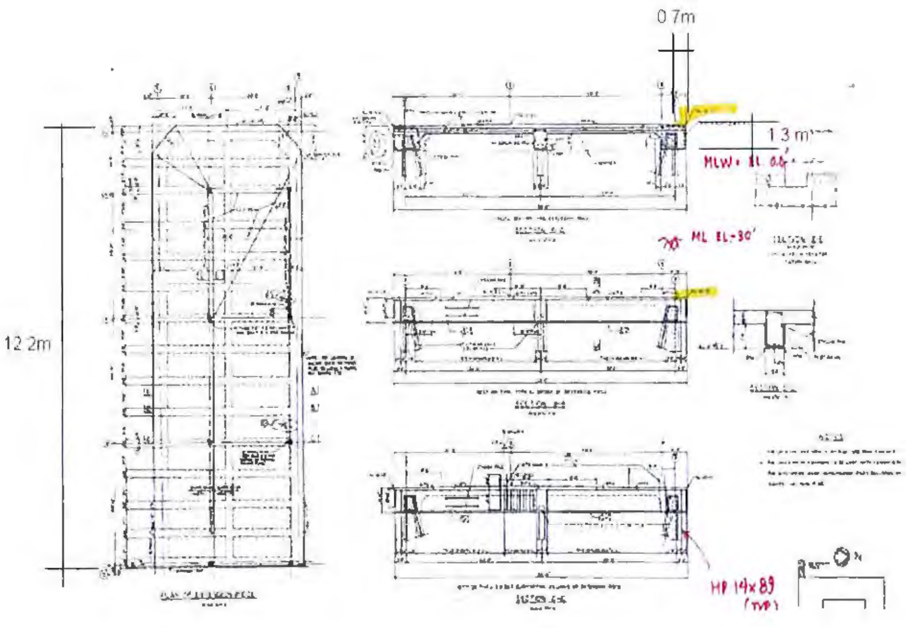


Fig. 1 Pier 40 Finger Pier Extension Structure Cross Section

Table 3- Total wave force on the edge beam (12.2 m x 1.3 x 1.3 m)

Force (kN)	Wave Directions	
	Southerly	Westerly
$F_{h, qs+}$	169.3 (38 kips)	2.8
$F_{h, max}$	423.2 (95 kips)	6.9

Use this for design

The wave force on the edge beam is distributed over contact area of 12.2 m x 1.3 m, or 15.86 m^2, which is equivalent to 171 SF. The wave force on the edge beam is therefore equal to 223 PSF.

A wave force of 223 PSF will be applied to the structural model. Because these extreme wave conditions are wind induced, the waves will be applied simultaneously with wind under the provided loading combinations.

WAVE FORCES ON PILE

Existing piles which support the deck slab are similar in size to HP14x89 steel H-piles. Force due to waves on the pile is estimated using Morrison equations as recommended in Shore Protection Manual (USACE SPM 1984). Pile cross section is assumed to be circular in order to simplify the calculations. Table 4 summarizes predicted total maximum force, and associated moment with moment arm with respect to the sea bed. Detailed calculations are attached. In short, the total maximum force on the pile is estimated to be 3915 N. The force is estimated to be applied at a point 8.2 m above the sea-bed.

Table 4- Wave forces on the pile

Directions	F_{max} (N)	Moment arm from mud line (m)
Southerly	3914.9 (0.88 kips)	8.2
Westerly	887.4 (0.2 kips)	10.3

Due to the relatively small load, these forces will not be applied to the structural model of the pile.

References

Arup, 2001, "Hudson River Park Structural Design Guidelines", Arup, NY, USA.

McConnell, K, William, Allsop, Ian, Cruickshank, 2004," Piers, Jetties and Related Structures Exposed to Waves - Guidelines for Hydraulic Loading, Thomas Telford, 250p.

USACE Shore Protection Guideline (SPM), 1984, Vol. II, Coastal Engineering Research Center, 19

Halcrow

Source: Pier, Jetties and Related Structures Exposed to Waves- Guidelines for Hydarulic Loading-2004 by McConnell at al.

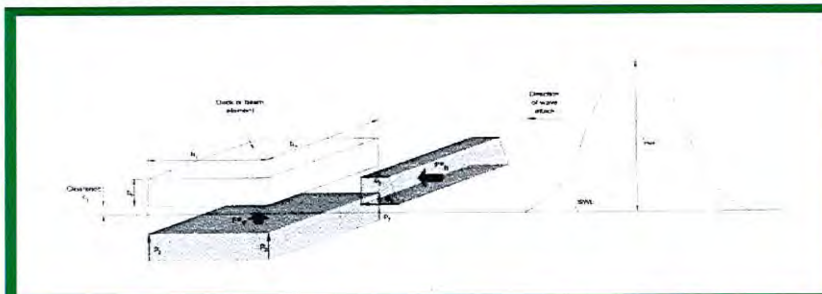


Fig. 5-5- Definitions of basic wave forces (after McConnell et al.)

INPUT DATA

Local Water Depth h_1 (m-MLLW)	9.14	
Design Water Level (m-MLLW)	1.56	
Deck Elev. (m-MLLW)	3.42	
H_s (m)	3.1	
h (m)	10.7	
T_m (s)	5.4	
N_z	660	Number of waves during the storm
B_w (m)	12.2	Deck width perpendicular dominant wave dir.
B_L (m)	1.3	Deck length (in direction of wave attack)
B_h (m)	1.3	Deck slab height
ρ_s (kg/m^3)	1030.0	Density of seawater
g (m/s^2)	9.8	Gravitational accleration
ρ_c (kg/m^3)	2400.0	Density of concrete
Coefficient for Vertical Force (quasi-static), C_v	1.0	Input- Table 5.5- Allsop et al. (2004)
Coefficient for Horizontal Force (quasi-static), C_h	1.0	Input- Table 5.7- Allsop et al. (2004)
$F_{h,max}$ / F_{vqs} Ratio	2.5	Input- Fig. 5-29 (There is also detail calc method to find Fmax but it requires prediction of natural period of oscillation for the structure and not included in here)
L_o (m)	45.5	Deep water wavelength
k_o (m^{-1})	0.1381	Deep water wave number
L (m)	42.0	Local wavelength
H_{max} (m)	5.49	Max. wave height
η_{max} (m)	3.84	Max. wave crest elevation (selected as 0.7*H_{max})
C_l (m)	0.56	Clearance between Design Water Level and Deck Bottom

Wave Forces on Deck-Southerly

p_1 (N/m^2)	20036.8		Hydrostatic pressure at the top of the deck element
p_2 (N/m^2)	33172.4	80.3 kips/ft^2	Hydrostatic pressure at the bottom of the deck element
$F_{h'}$ (kN)	421.9		
a	0.45	Input- Table 5.6- Allsop et al. (2004)	
b	1.56	Input- Table 5.6- Allsop et al. (2004)	
F_{hqs} (kN)	169.3	37.9 kips	Max. positive (seaward) quasi-static (pulsating) force
p_{hqs} (kN/m^2)	10.7	222.8 pound/ft^2	Max. positive (seaward) quasi-static (pulsating) pressure

Table 5.4. Coefficients for prediction of vertical wave forces using Equation (5.19)
(after McConnell at al., 2004)

Wave load and configuration	*a*	*b*
Upward vertical forces (seaward beam and deck)	0.82	0.61
Upward vertical forces (internal beam only)	0.84	0.66
Upward vertical forces (internal deck, two- and three-dimensional effects)	0.71	0.71
Downward vertical forces (seaward beam and deck)	-0.54	0.91
Downward vertical forces (internal beam only)	-0.35	1.12
Downward vertical forces (internal deck, two-dimensional effects)	-0.12	0.85
Downward vertical forces (internal deck, three-dimensional effects)	-0.80	0.34

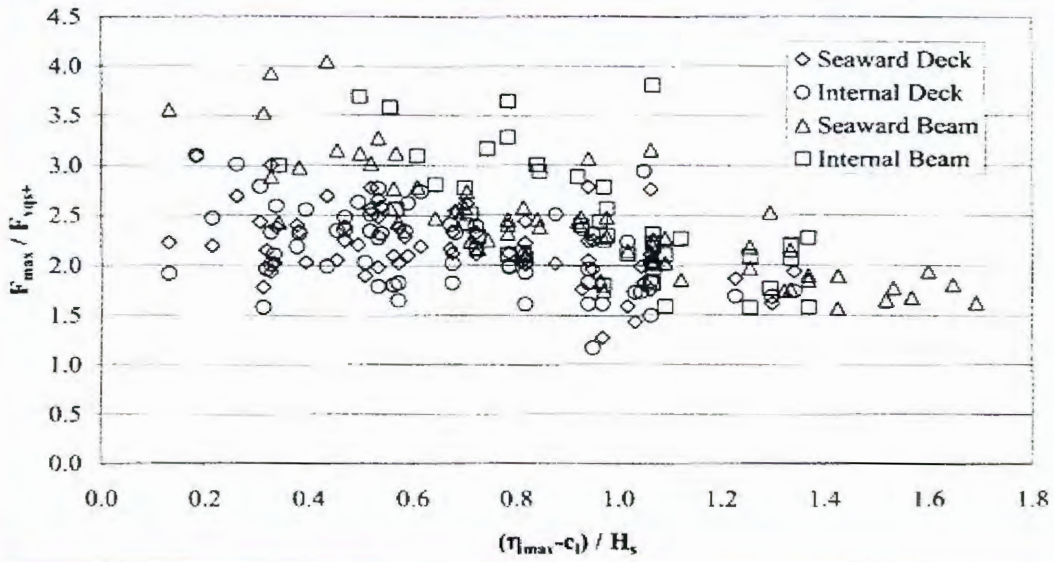
Table 5.5. Coefficients for upper and lower limits of test data
(after McConnell at al., 2004)

Wave load and configuration	C_{upper}	C_{lower}
Upward vertical forces (seaward beam and deck)	1.5	0.5
Upward vertical forces (internal beam only)	1.4	0.5
Upward vertical forces (internal deck, two- and three-dimensional effects)	2.2	0.1
Downward vertical forces (seaward beam and deck)	1.6	0.4
Downward vertical forces (internal beam only)	1.8	0.5
Downward vertical forces (internal deck, two-dimensional effects)	2.1	-
Downward vertical forces (internal deck, three-dimensional effects)	1.4	0.65

Halcrow

Table 5.7. Coefficients for upper and lower limits of test data
(after McConnell at al., 2004)

Wave load and configuration	C_{upper}	C_{lower}
Shoreward horizontal forces, F_{hqs-} (seaward beam)	2	0.25
Shoreward horizontal forces, F_{hqs+} (internal beam)	1.8	-
Seaward horizontal forces, F_{hqs-} (seaward beam)	2	0.15
Seaward horizontal forces, F_{hqs-} (internal beam)	3	-



Ratio of vertical impact forces to quasi-static forces (after McConnell et al. 2004)

Wave Forces on Deck-Southerly

Halcrow

Source: SPM 1984 (Morrsion Eq.)

INPUT DATA

Local Water Depth h_1 (m-MLLW)	9.14	
Design Water Level (m-MLLW)	1.56	
H_s (m)	3.05	
d (m)	10.70	
T_p (s)	5.94	
N_z	660.00	Number of waves during the storm
D (m)	0.35	
ρ_s (kg/m^3)	1030.00	Density of seawater
g (m/s^2)	9.81	Gravitational accleration
ρ_c (kg/m^3)	2400.00	Density of concrete
Drag Coefficient, C_D	0.70	Input- Fig.
Inertia Coefficient , C_M	1.50	Input- Fig

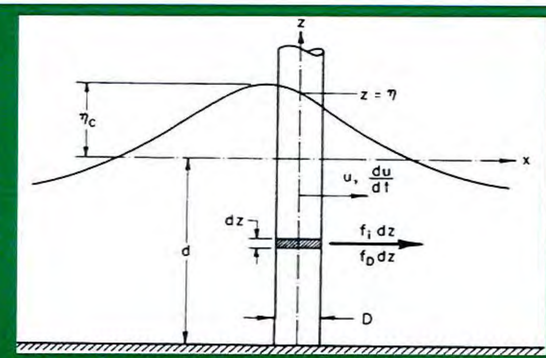


Fig. 1 Definition sketch of wave forces on a vertical cylinder (SPM 1984).

L_o (m)	55.0	Deep water wavelength
k_o (m^{-1})	0.1142	Deep water wave number
L (m)	48.6	Local wavelength
H_{max} (m)	5.49	Max. wave height

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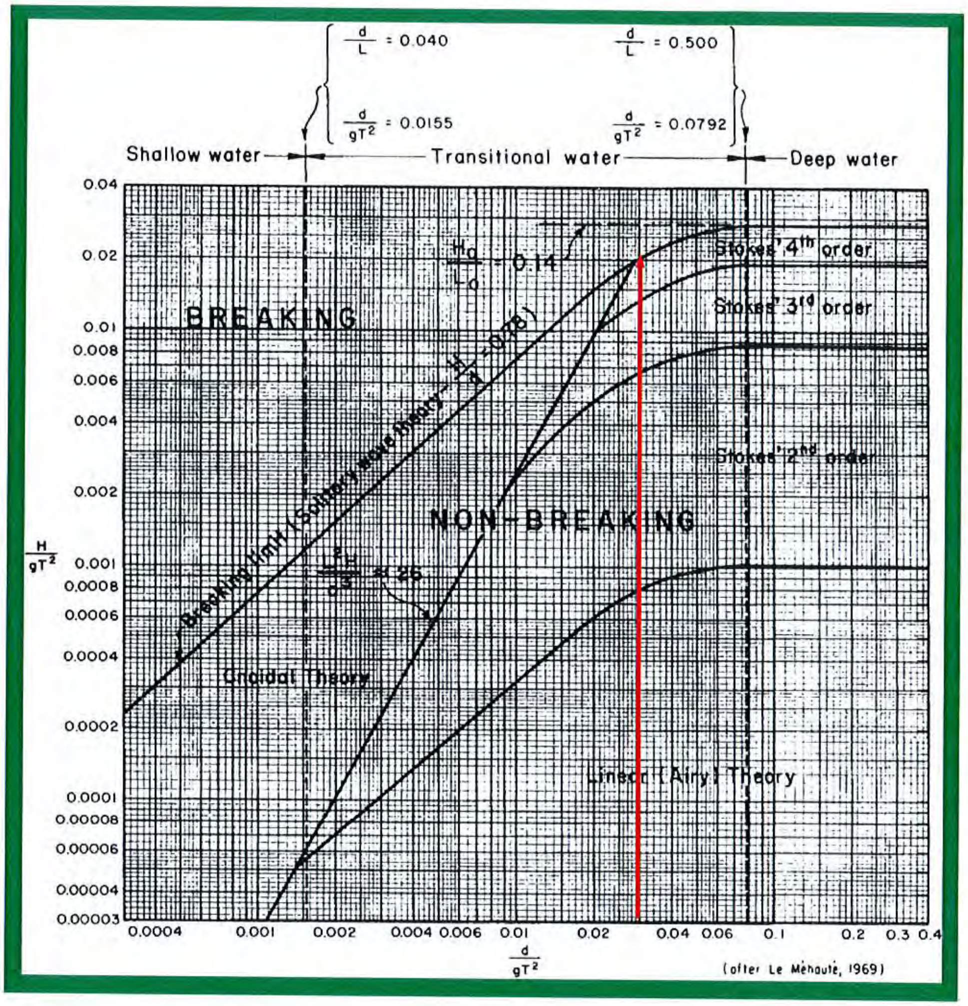
$d/(gT_p^2)$	0.0309								
$H_b/(gT_p^2)$	0.020	←	Read from Fig. 7.75						
H_b (m)	6.9								
$H_{m0}/(gT_p^2)$	0.0088								
K_{IM}	0.44	←	Read from Fig. 7.71	Using	$H_b/H=$	0.44	&	$d/(gT_p^2)$	0.0309
K_{DM}	0.29	←	Read from Fig. 7.72	Using	$H_b/H=$	0.44	&	$d/(gT_p^2)$	0.0309
F_{IM} (N)	1956.9								
F_{DM} (N)	3339.2								
w	0.25								
ϕ_m	0.17	←	Read from Fig. 7.76 ~ 7.79	Using	$H_{m0}/(gT_p^2)$	0.0088	&	$d/(gT_p^2)$	0.0309
F_{MAX} (N)	3914.9	or	880.1 lbf						
S_{IM}	0.58	←	Read from Fig. 7.73	Using	$H_b/H=$	0.44	&	$d/(gT_p^2)$	0.0309
S_{DM}	0.82	←	Read from Fig. 7.74	Using	$H_b/H=$	0.44	&	$d/(gT_p^2)$	0.0309
M_{IM} (N.m)	12,144.7								
M_{DM} (N.m)	29,298.0								
w	0.25								
α_m	0.13	←	Read from Fig. 7.80 ~ 7.83	Using	$H_{m0}/(gT_p^2)$	0.0088	&	$d/(gT_p^2)$	0.0309
M_{MAX} (N.m)	32,033.11	or	23,626.41 lbf x m						
r (m)	8.18	or	26.84 ft						



$\frac{d}{L} = 0.040$
$\frac{d}{L} = 0.500$
$\frac{d}{gT^2} = 0.0155$
$\frac{d}{gT^2} = 0.0792$
Shallow water
Transitional water
Deep water
$\frac{H_o}{L_o} = 0.14$
Stokes' 4th order
Stokes' 3rd order
Stokes' 2nd order
BREAKING
NON-BREAKING
Breaking limit (Solitary wave theory $\frac{H}{d} = 0.78$)
$\frac{L^2 H}{d^3} = 26$
Cnoidal Theory
Linear (Airy) Theory
$\frac{H}{gT^2}$
$\frac{d}{gT^2}$
(after Le Méhauté, 1969)

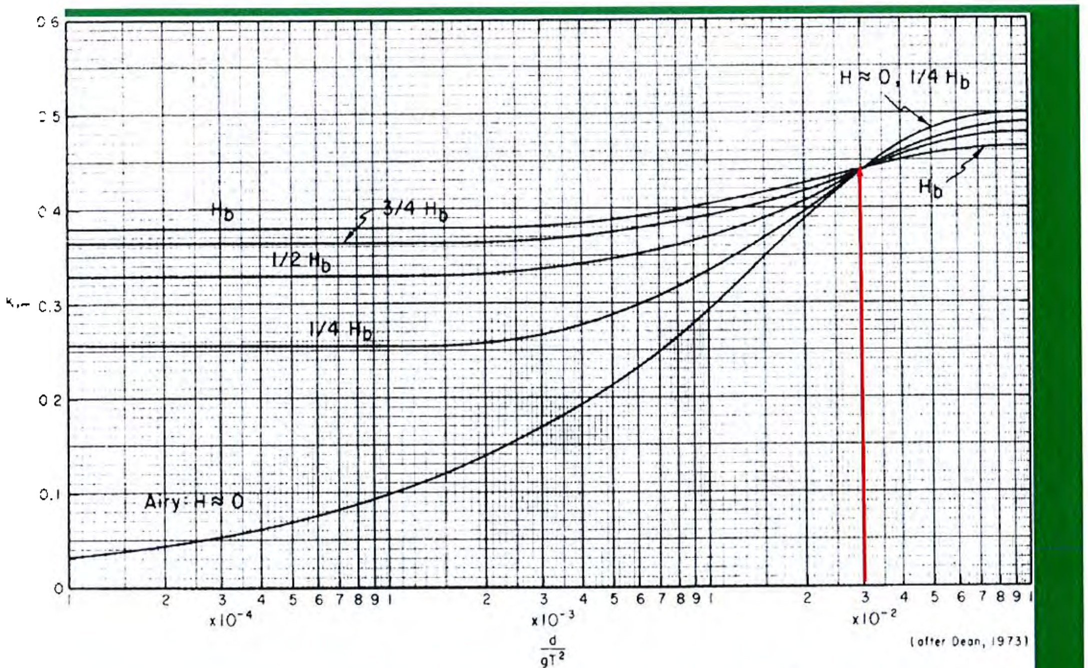


Figure 7-71. K_{im} versus relative depth, d/gT^2 .

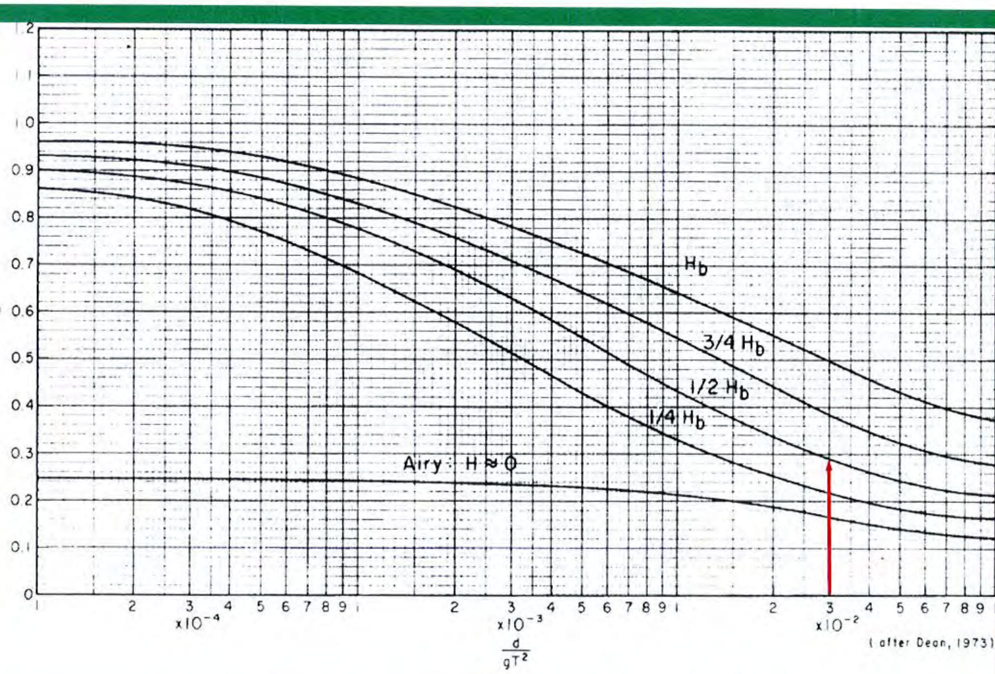


Figure 7-72. K_{Dm} versus relative depth, d/gT^2 .

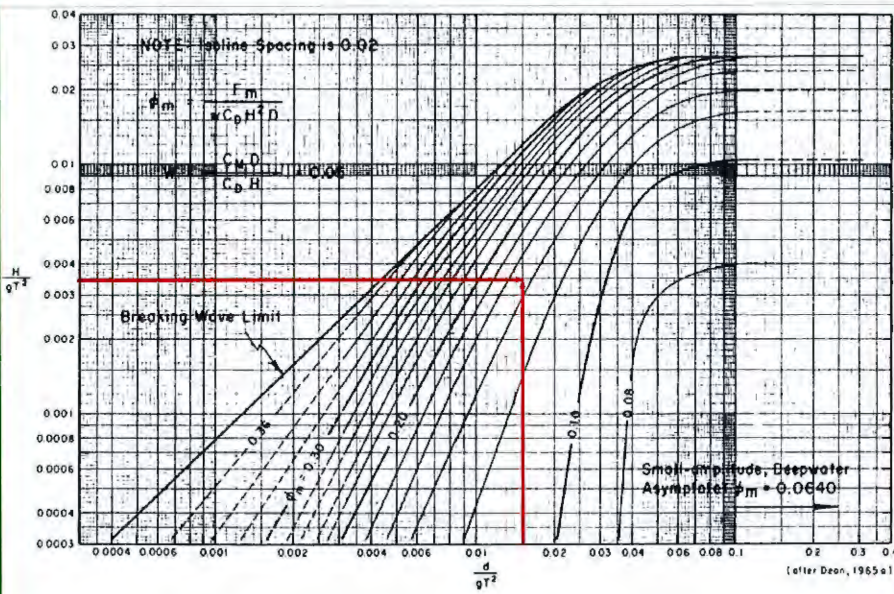


Figure 7-76. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.05) .

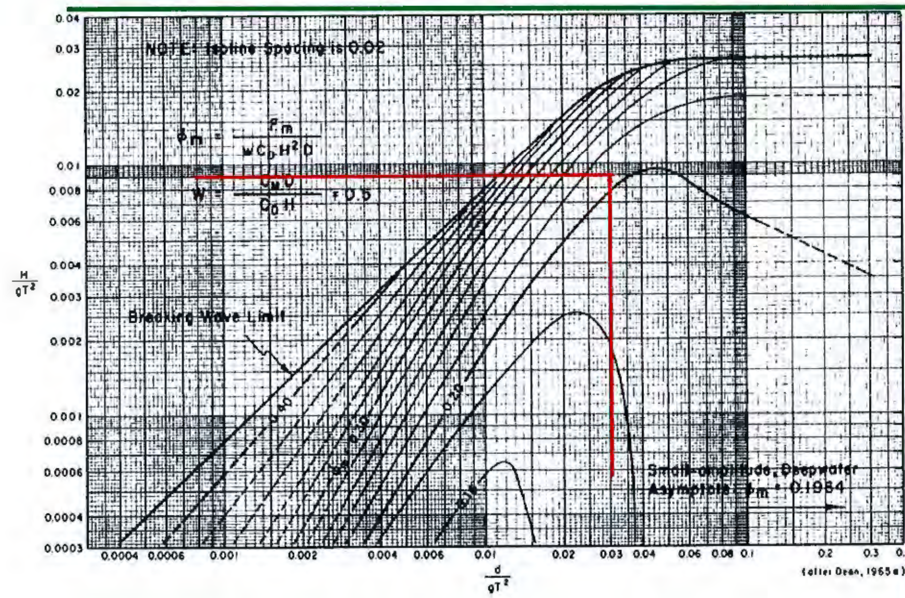


Figure 7-78. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.5) .

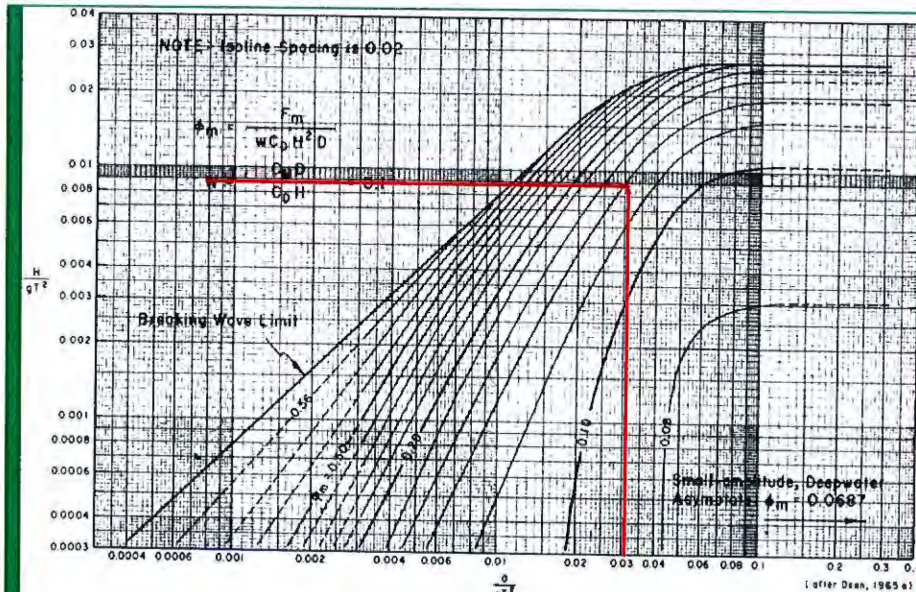


Figure 7-77. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.1) .

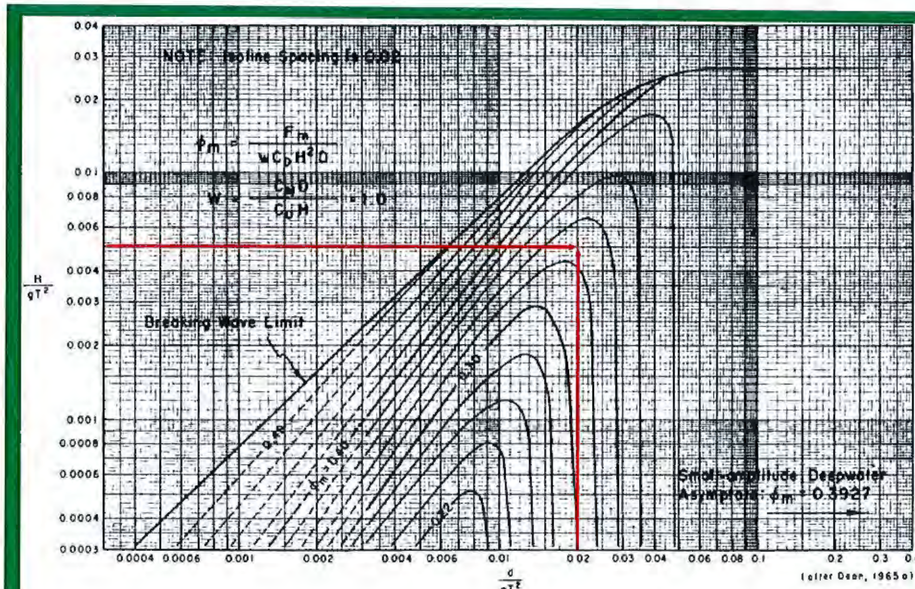


Figure 7-79. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 1.0) .

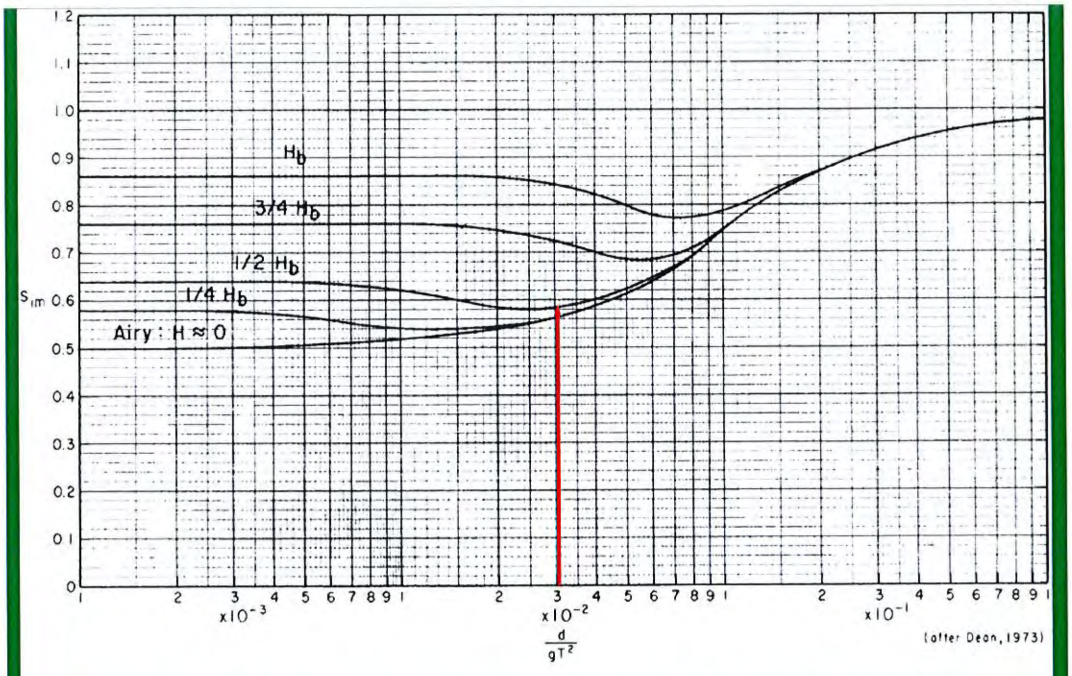


Figure 7-73. Inertia force moment arm, S_{im} , versus relative depth, d/gT^2 .

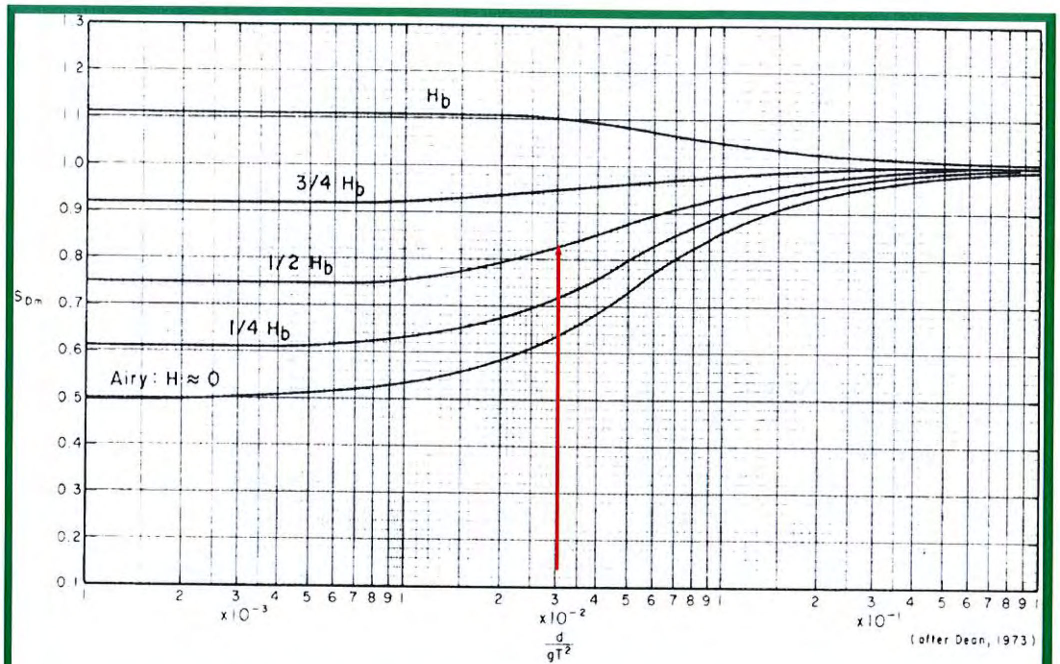


Figure 7-74. Drag force moment arm, S_{Dm}, versus relative depth, d/gT^2.

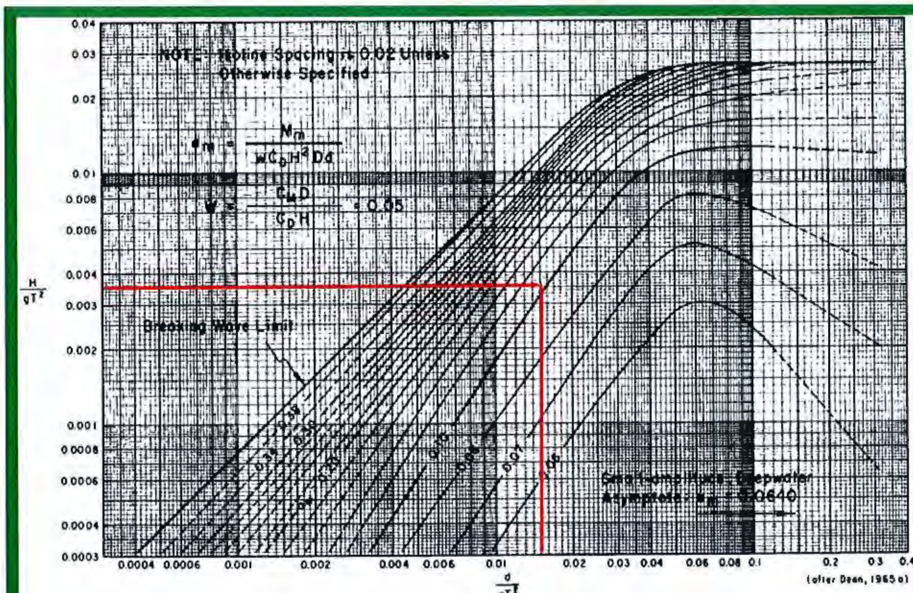


Figure 7-80. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.05) .

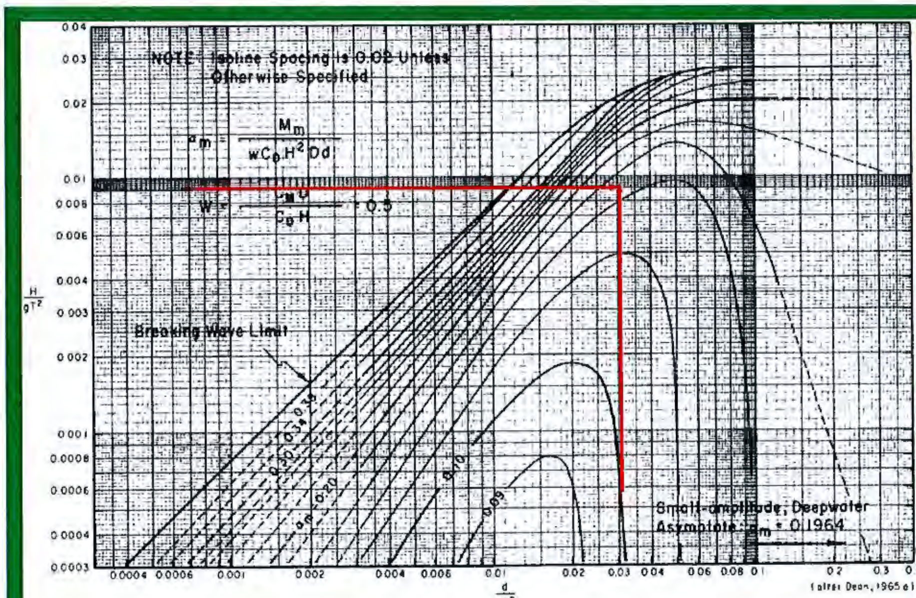


Figure 7-82. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.5) .

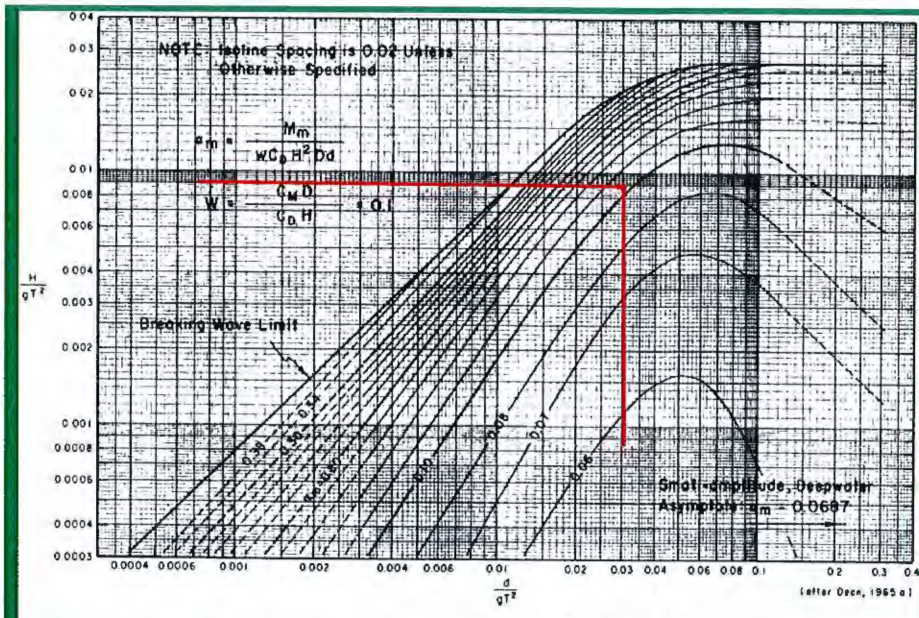


Figure 7-81. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.1) .

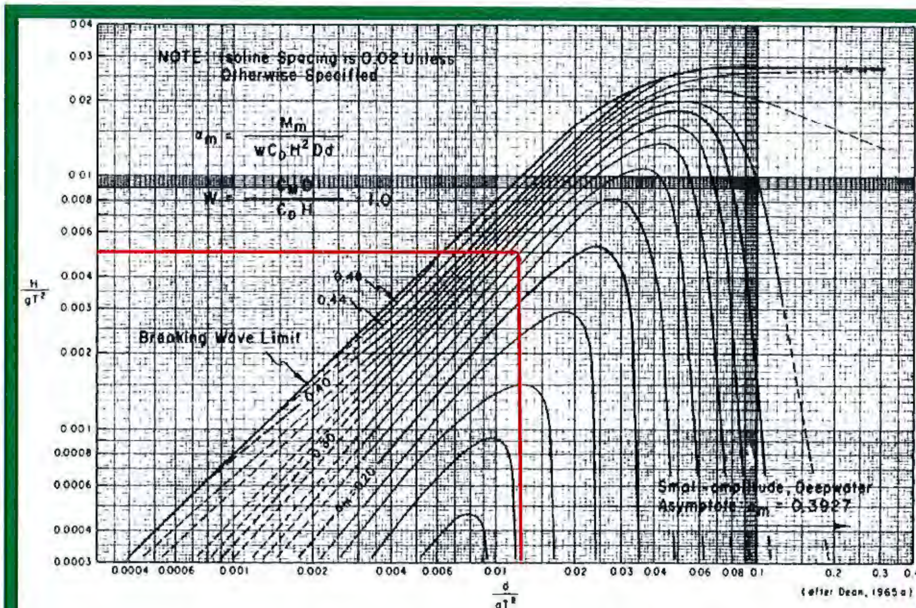
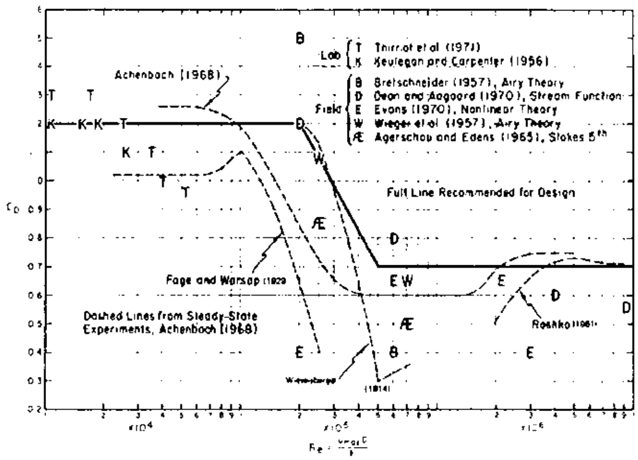


Figure 7-83. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 1.0) .



$$
\left.
\begin{aligned}
C_M &= 2.0 \quad \text{when} \quad R_e < 2.5 \times 10^5 \\
C_M &= 2.5 - \frac{R_e}{5 \times 10^5} \quad \text{when} \quad 2.5 \times 10^5 < R_e < 5 \times 10^5 \\
C_M &= 1.5 \quad \text{when} \quad R_e > 5 \times 10^5
\end{aligned}
\right\}
$$

Figure 7-85. Variation of drag coefficient C_D with Reynolds number R_e.

MOORING ANALYSIS

HALCROW, INC.

CONTRACT NO.:	D01080350	Calc.		Sheet	1	Cont'd	
Client:	Hudson River Park Trust	Project	HUDSON RIVER PARK BERTH 40				
Title:	MOORING ANALYSIS FOR HORNBLOWER INFINITY						

PURPOSE

To perform mooring analysis using quasi static method for preliminary design in order to predict maximum mooring load for safe mooring of Hornblower Infinity at Hudson River Park Berth 40. Compare the mooring results with previous mooring study done for LILAC.

DESIGN BASIS AND UNCONFIRMED ASSUMPTIONS

1. As per Reference 1, considered parameters are:
   Maximum wind speed = 110 MPH (~96 knots) and
   Current velocity = 2.0 knots
2. The location of the ship along the pier is assumed as shown through appendix 3.
3. Bollards are assumed to have 100 ton working capacity.
4. The mooring cables are assumed to have infinite strength.
5. Mooring line attachment points on the bollards are taken as effectively 1ft above the pier deck.
6. The pier height is considered 11 feet above low water datum (MLLW).
7. Future dredge depth is assumed as 50 ft from datum.
8. Each timber fender is assumed vertical, cylindrical and has constant c/s of 12" dia. The fenders are made of Southern Pine (Structural no. 2, medium grain).Elastic Modulus = 1400 ksi
9. C/C distance between timber fenders is 8 ft.
10. Fender piles fixity point is calculated 11 ft. below mudline. Mudline is at EL - 15.00'.
11. Type of mooring lines used in the analysis is Steel Wire of 143 kips strength.
12. The Mooring Arrangement Plan is attached with OPTIMOOR Report sheet (Appendix 5).
13. Water level (distance between MLLW and MHHW) = 5.12'
14. Width of estuary is considered as 2640 ft.
15. Distance between ship and fenders is 3'
16. Bollards are assumed to be 4 ft far from the fenders.
17. Fender Performance curve is generated based on calculations shown through appendix 4.
18. Ship impacts the fenders at 5ft above the datum (MLLW).

19. Port side of the vessel is facing the berth.
20. The vessel target is the midship of the vessel. Berth target is the bollard G (see Mooring Arrangement Plan, Appendix 5. The distance between the vessel and ship target is 50'.
21. As per reference 1, Significant wave coming from south : Wave height = 10.00' ; Wave period = 5.4 Secs
22. During regular tide, HWL EL.= 5.12' ; LWL EL.= 0.0' , Time lag at slack = 60 mins (assumed)
23. Flat side points of the vessels are assumed same as LBP points on aft and forward sides.
24. Tidal velocity = 0.2 Knots/ft tide for EBB and 0.6 Knots/ft tide for flood condition.
25. Ship windage area is based on calculations by OPTIMOOR.
26. The soil type under mud line is soft clay.

DESIGN PROCEDURES

Mooring analysis is performed using OPTIMOOR. Following case is studied:

CASE I: Analysis for USCGC LILAC (WAGL – 227)

SHIP DATA:

OVERALL LENGTH = 205.3'

LBP = 190' (assumed)

DEPTH = 22.8' (Draft + Free board (scaled from Fig in appendix 1)

BEAM = 46.4'

DRAFT = 7.9' (Light draft)

Trim (Draft aft minus draft forward) = 0.0' (assumed)

BODY OF CALCULATION

The analysis was performed for:

- Wind speed = 96 knots; Wind direction = 90^0 from North.
- Current velocity = 2.0 knots; Current direction = 90^0 from North
- Wave height = 10.00' ; Wave direction = 90^0 from North; Wave period = 5.4 Secs

CONCLUSIONS

The maximum lateral force (thrust) under wind, wave and current (including tidal current) actions is observed on fender gg (See Arrangement plan, appendix 2).

Maximum lateral load noticed on fender gg = 19 kips

Total lateral load on the deck = 253 kips

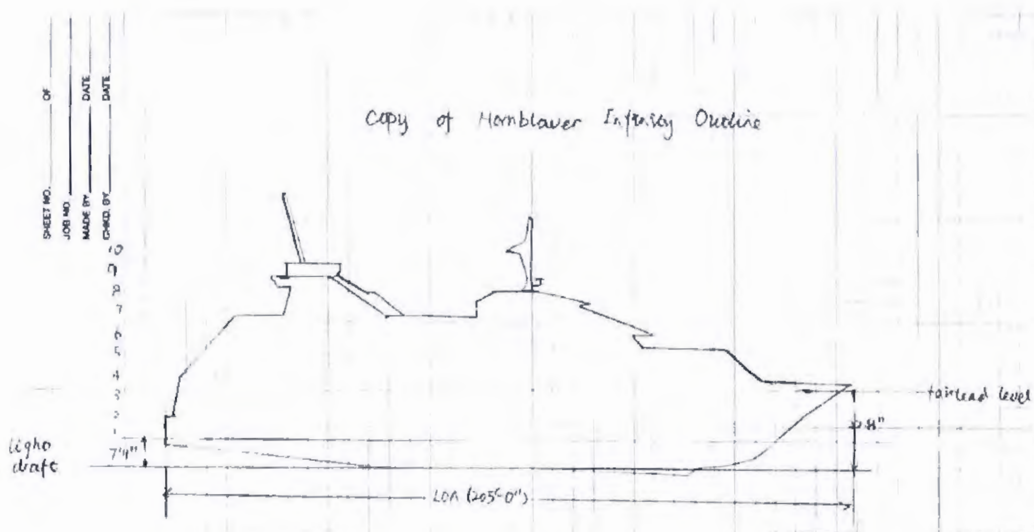
As the environmental loads push the vessel towards the vessel, two of the mooring lines are in slack and rest two don't experience high tensile forces.

Based on the results of the mooring analysis, the total lateral load imposed on the fender system and pier structure over the 190 ft length (LBP) of the vessel is 253 kips, or 1.2 kips per ft along the length of the concrete edge beam. Below table shows the comparison with the mooring loads generated by LILAC.

	LILAC	Hornblower Infinity
Vessel parameters		
LOA (ft)	173.3	205.3
Beam (ft)	32	46.4
Draft (ft)	11.25	7.9
Side windage area (ft^2) above deck	2702	3524
Mooring results		
Maximum fender load (kips)	19	19
Total lateral load on the deck (kips)	192	253
LPB (ft)	160	190
Pressure the concrete edge beam (kips/ft)	1.2	1.33
Fender number within parallel body	15	18

APPENDIX 1:

Geometry and mooring points location of Hornblower Infinity



205' → 34
⇒ 6.03' → 1
⇒ 1 grid = 36.35 sqft

over the deck

Side $96.97 \times 36.35 = 3524.8$

End-on: $3524.8 \times \frac{46.4}{205.3} = 796.7$

APPENDIX 2:
OPTIMOOR INPUT AND OUTPUT FILES

Berth Data for BERTH 40: HUDSON PAR

(file C:\OPTIMOOR\pier 40 rerun\Berth40_Hudson_River_Park.bth)
Units in ft & kips

Left to Right of Screen Site Plan Points:	0°
Width of Estuary (for Current):	2640
Pier Height (Fixed) above Datum:	11.0
Dredged Depth below Datum:	50.0
Dist of Berth Target to Right of Origin:	520.0
Wind Speed Specified at Height:	33.0
Current Specified at Depth:	mean

Hook/ Bollard	X-Dist to Origin	Dist to Fender Line	Ht above Berth	Allowable Load
A	20.0	4.0	1.0	220
B	90.0	4.0	1.0	220
C	180.0	4.0	1.0	220
D	270.0	4.0	1.0	220
E	350.0	4.0	1.0	220
F	430.0	4.0	1.0	220
G	520.0	4.0	1.0	220
H	600.0	4.0	1.0	220
I	700.0	4.0	1.0	220
J	800.0	4.0	1.0	220

Fender	X-Dist to Origin	Ht above Datum	Width Along Side	Face Contact Area (ft²)
aa	664.0	11.0	1.0	11.0
bb	672.0	11.0	1.0	11.0
cc	680.0	11.0	1.0	11.0
dd	688.3	11.0	1.0	11.0
ee	488.0	11.0	1.0	11.0
ff	496.0	11.0	1.0	11.0
gg	504.0	11.0	1.0	11.0
hh	512.0	11.0	1.0	11.0
ii	520.0	11.0	1.0	11.0
jj	528.0	11.0	1.0	11.0
kk	536.9	11.0	1.0	11.0
ll	544.0	11.0	1.0	11.0
mm	552.0	11.0	1.0	11.0
nn	560.0	11.0	1.0	11.0
oo	568.0	11.0	1.0	11.0
pp	576.0	11.0	1.0	11.0
qq	584.0	11.0	1.0	11.0
rr	592.0	11.0	1.0	11.0
ss	600.0	11.0	1.0	11.0
tt	608.0	11.0	1.0	11.0
uu	616.0	11.0	1.0	11.0
vv	623.9	11.0	1.0	11.0
ww	632.0	11.0	1.0	11.0
xx	640.0	11.0	1.0	11.0
yy	648.0	11.0	1.0	11.0
zz	656.0	11.0	1.0	11.0

Fender Load-Compression Data

Fender								
aa	0	100	200	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
bb	0	100	200	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft

HALCROW, INC.

CONTRACT NO.: D01080350 Calc. Sheet 6 Cont'd

Client: Hudson River Park Trust Project HUDSON RIVER PARK BERTH 40

Title: MOORING ANALYSIS

cc	0	100	200	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
dd	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
ee	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
ff	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
gg	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
hh	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
ii	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
jj	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
kk	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
ll	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
mm	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
nn	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
oo	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
pp	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
qq	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
rr	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
ss	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
tt	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
uu	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
vv	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
ww	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
xx	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
yy	0	100	300	300	400	500	600	kips
	0.03	2.08	4.17	6.25	8.34	10.43	12.50	ft
zz	0	100	300	300	400	500	600	kips

HALCROW, INC.

CONTRACT NO.:	D01080350	Calc.		Sheet 7	Cont'd
Client:	Hudson River Park Trust	Project	HUDSON RIVER PARK BERTH 40		
Title:	MOORING ANALYSIS				

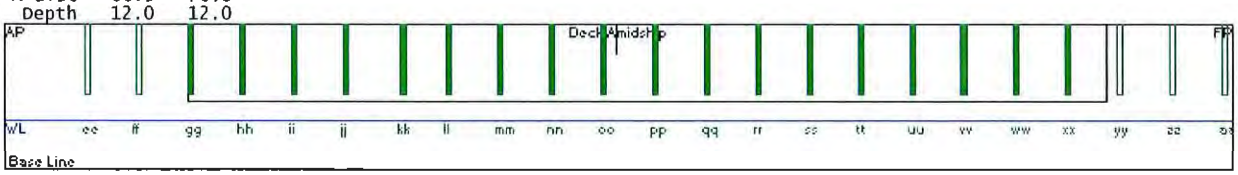
0.03 2.08 4.17 6.25 8.34 10.43 12.50 ft

Vessel Data for Hornblower infinity

(file C:\OPTIMOOR\pier 40 rerun\Berth40_Hornblowerinfinity.vsl)
Units in ft, inches, & kips
Longitudinal datum at Midship

```
                          LBP: 190.0
                      Breadth:  46.4
                        Depth:  22.8
                       Target:   0.0 fwd from midship and 0.0 above deck at side
 End-on projected windage area:  796 above deck level
   Side projected windage area: 3525 above deck level
      Fendering possible from: 0.350 LBP aft of midship
                           to: 0.400 LBP fwd of midship
    Current drag data based on:  OPTIMOOR (Generic Data)
       Wind drag data based on:  OCIMF Tanker (U-shaped Bow)
```

Flatside Contour		
X-dist	-66.5	76.0
Depth	12.0	12.0



Line No.	Fair-Lead X	Fair-Lead Y	Ht on Deck	Dist to Winch	Brake Limit	Pre-Tension	Line Size-Type-BL	Tail Segment-1 Lgth-Size-Type-BL
1	92.6	23.2	0.0	0.0		0.0	3.7 SW 143	
2	77.2	23.2	0.0	0.0		0.0	3.7 SW 143	
3	-49.9	23.2	0.0	0.0		0.0	3.7 SW 143	
4	-88.2	23.2	0.0	0.0		0.0	3.7 SW 143	

Codes for Types of Line:
SW: Steel Wire (steel core)

Static Mooring Response for Hornblower infinity at BERTH 40: HUDSON PAR

Units in ft & kips (file C:\OPTIMOOR\pier 40 rerun\Berth40_Hudson_River_Park.OPT)
Remarks:

```
         Static Time Sweep up to: 1141 Apr 24 2009
                     Water Level:   1.32 above Datum
                           Draft:   7.9
                            Trim:   0.0
                Bottom Clearance:  43.4

             Deck Level at Target:   5.2 above pier
              Significant Wave Ht:  10.01
                 Wave Mean Period:   5.4 sec
              Wave Direction True:   90°
    Wave Direction to Berth X-axis:   90°
                          Current:  2.82 knots
           Current Direction True:   90°
 Current Direction to Berth X-axis:   90°
                       Wind Speed:   96 knots
              Wind Direction True:   90°
    Wind Direction to Berth X-axis:   90°

     Fwd Offset of Vessel Target:  50.0 from Berth Target
         Total End-On Windage Area: 1487
           Total Side Windage Area: 6356
```

HALCROW, INC.

CONTRACT NO.: D01080350 Calc. Sheet 8 Cont'd

Client: Hudson River Park Trust Project HUDSON RIVER PARK BERTH 40

Title: MOORING ANALYSIS

	Longitudinal	Transverse	Yaw Moment/LBP
Wave Drift Force:	0.0	26.4	0.5
Current Drag Force:	0.0	21.1	0.0
Wind Drag Force:	-14.1	204.8	-11.2
Total Force:	-14.1	252.3	-10.7

Vessel Moves(at Target):	0.1 fwd	0.3 inw	-0.1° stbd	0.0 up
	0.0 fwd	0.3 inw	-0.1° stbd	0.0 up

Line to Bollard	Pull -in	Tot.Line Length	In-Line ±Motion	Winch Slippage	Worst Time	Line Tension	Percent Strength
1-I	0.00	37.8			1141	0.0	0%
2-G	0.00	127.3			1141	5.4	4%
3-H	0.00	80.1			1141	0.0	0%
4-F	0.00	52.1			1141	8.8	6%

Fender	Thrust	Compression	Pressure	Contact Area
gg	19	0.42	1.7	100%
hh	18	0.41	1.7	100%
ii	18	0.40	1.6	100%
jj	17	0.39	1.6	100%
kk	17	0.37	1.5	100%
ll	16	0.36	1.5	100%
mm	16	0.35	1.4	100%
nn	15	0.34	1.4	100%
oo	14	0.33	1.3	100%
pp	14	0.31	1.3	100%
qq	13	0.30	1.2	100%
rr	13	0.29	1.1	100%
ss	12	0.28	1.1	100%
tt	11	0.27	1.0	100%
uu	11	0.26	1.0	100%
vv	10	0.24	0.9	100%
ww	10	0.23	0.9	100%
xx	9	0.22	0.8	100%

Hook/ Bollard	X- Force	Y- Force	Other X-Load	Other Y-Load	Total Horiz Force	Direction in Plan	Uplift
F	8.8	0.6			8.8	86°	0.7
G	5.4	0.1			5.4	88°	0.2

Approximate natural periods
Surge: 4.0 Sway: 7.3 secs

OPTIMOOR Version 6.1.16 Tension Technology International © 29 June 2012

PIER SHED STRUCTURAL MODEL

Halcrow

PIER 40 ANALYSIS SUMMARY

TYPE OF CONSTRUCTION

Steel H piles (14BP89) were driven to the hard rock below the mud and silt soil. The mud elevation varies between 10 to 30 feet below Mean Sea Level (MSL). The concrete decks were built on the top of steel piles. The top elevation of decks is at 11' above MSL. The concrete decks consist of precast 12" solid concrete slabs and cast-in-place concrete girders. There is a 2" asphaltic concrete wearing surface on the top of slab. The referenced structural drawings are available by Roberts and Schaefer Co. dated April 1957.

A two-story building was built on the perimeter of the pier, creating an "O" shaped structure. The building serves as parking spacing in majority of the area, and an office space on the east end. The building is composed of 6" deep precast concrete slabs, "I" beams, reversed "T" girders and cast-in-place concrete columns. The referenced structural drawings are available by Roberts and Schaefer Co. dated February 1960.

The lateral system of building to resist the wind and earthquake is the concrete moment connection system. The first floor is about 20 feet in height, and the second floor is about 18 feet in height. There is a 3 foot high parapet at the top of the building perimeter.

LATERAL ANALYSIS ASSUMPTION

The following approaches and assumptions are taken in the modelling:

1) The fixed point of "H" pile at base is at the 12'-0" below the mud line. It was verified by the "LPILE" analysis based on soil report by Mueser Rutledge Consulting Engineers dated April 1979 with boring holes at the west side of the pier.
2) Top of "H" pile is treated as fixed point within the concrete deck systems (girders or slabs).
3) The portions of the pier divided by expansion joints act independently to the lateral loads.
4) Steel piles are of the yielding strength F_y = 36 ksi, based on bearing pile properties in "Steel Construction" AISC Fifth Edition 1958.
5) Average concrete strength is 4.375 ksi. Concrete is cracked. The properties for the concrete members of structure with cracked sections, base on ACI 318 - 05:
   Beams................................0.35 I_g
   Columns.............................0.70 I_g
   Slabs..................................0.25 I_g

DESIGN LOADS

1) Self – weight of structure: to be calculated by ETABS and SAP automatically.

2) Superimposed dead load, including topping: on building slab 20 psf and on pier slab 30 psf.

3) Wind: ASCE 7-98, 110 MPH, Exposure C and $I_w = 1.0$.

4) Wave: 223 PSF on edge beam (north-south direction only)

5) Seismic: NYC Building Code 2004, A = 0.15, site coefficient = 1.2 and $I_e = 1.0$.
   $R_w = 5$ for main pier, $R_w = 3$ for finger pier

6) Ice: 8" thick ice, and 14 Kips acting on piles and 1.54 Kips on side of piles. (north-south direction only)

7) Current: current velocity to be 1.5 knots and acting on piles with 10.2 plf.

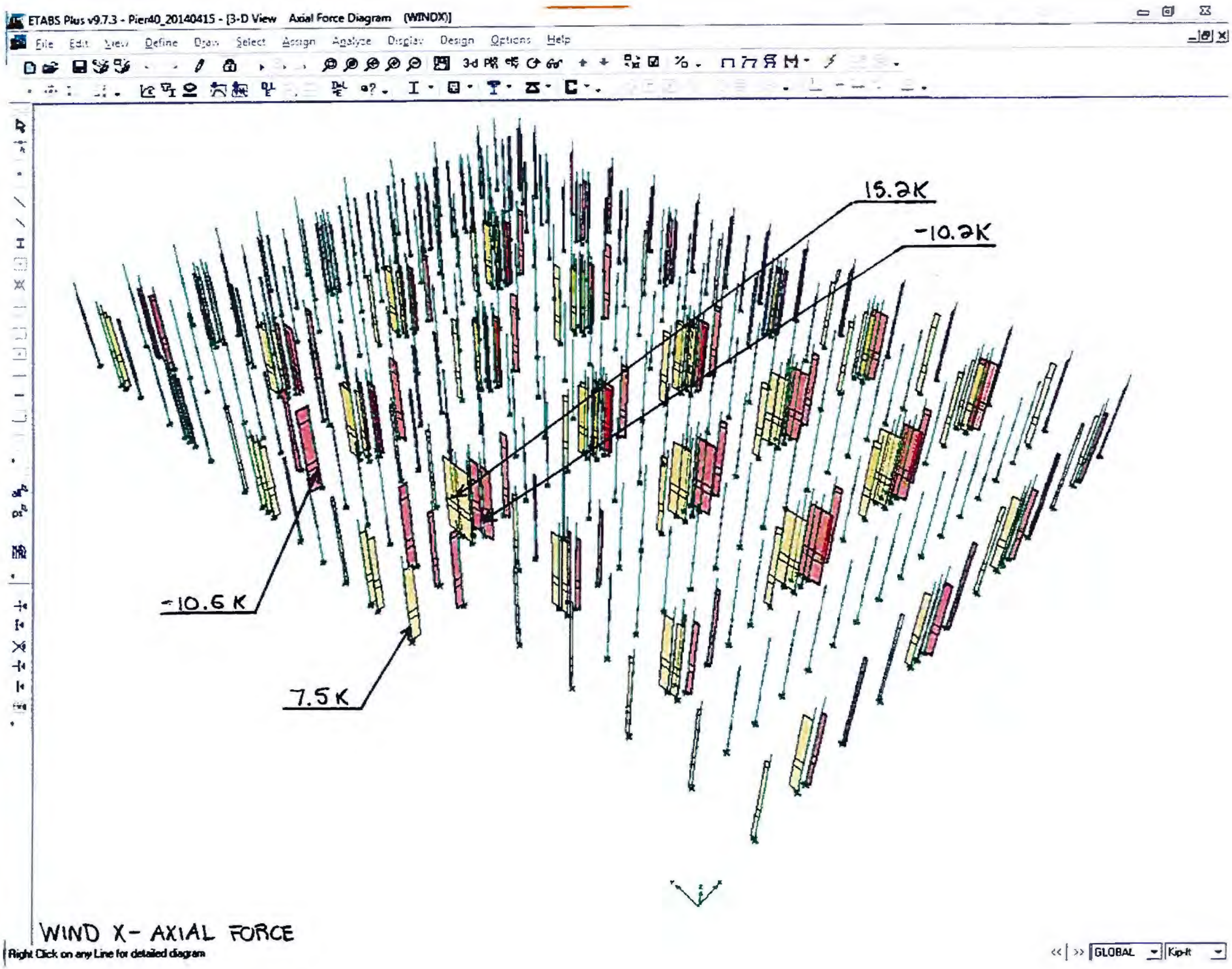
ANALYSIS PROCEDURE

Given the complexity of the structures, a series of three-dimensional computer analyses was conducted using the ETABS and SAP computer programs, both developed by Computers and Structures Inc., of Berkeley, California.

Previous experience with other structures had established the importance of incorporating gravity loads directly into the lateral analysis. Accordingly, ETABS and SAP were used to evaluate building and pier movements and the distribution of forces within the columns due to the gravity loads. P-Delta effects are automatically included in the ETABS and SAP analyses.

Two portions of the pier are taken to represent the actual building and pier situation. One is taken from the North – West corner of the pier with a 300x220 foot portion; and the other one is from the "finger pier," a narrow portion of pier at South – West corner jutting into deeper water. The finger pier has been modelled in SAP due to the geometry of the structure's battered piles.

The models are dependent on the average mud line elevation. The outer piles in the N-W are assumed to 15'-0" below MSL (Mean See Level) and interior ones are 12'-0" below MSL. The mud line under the finger pier is taken as 30'-0" below MSL.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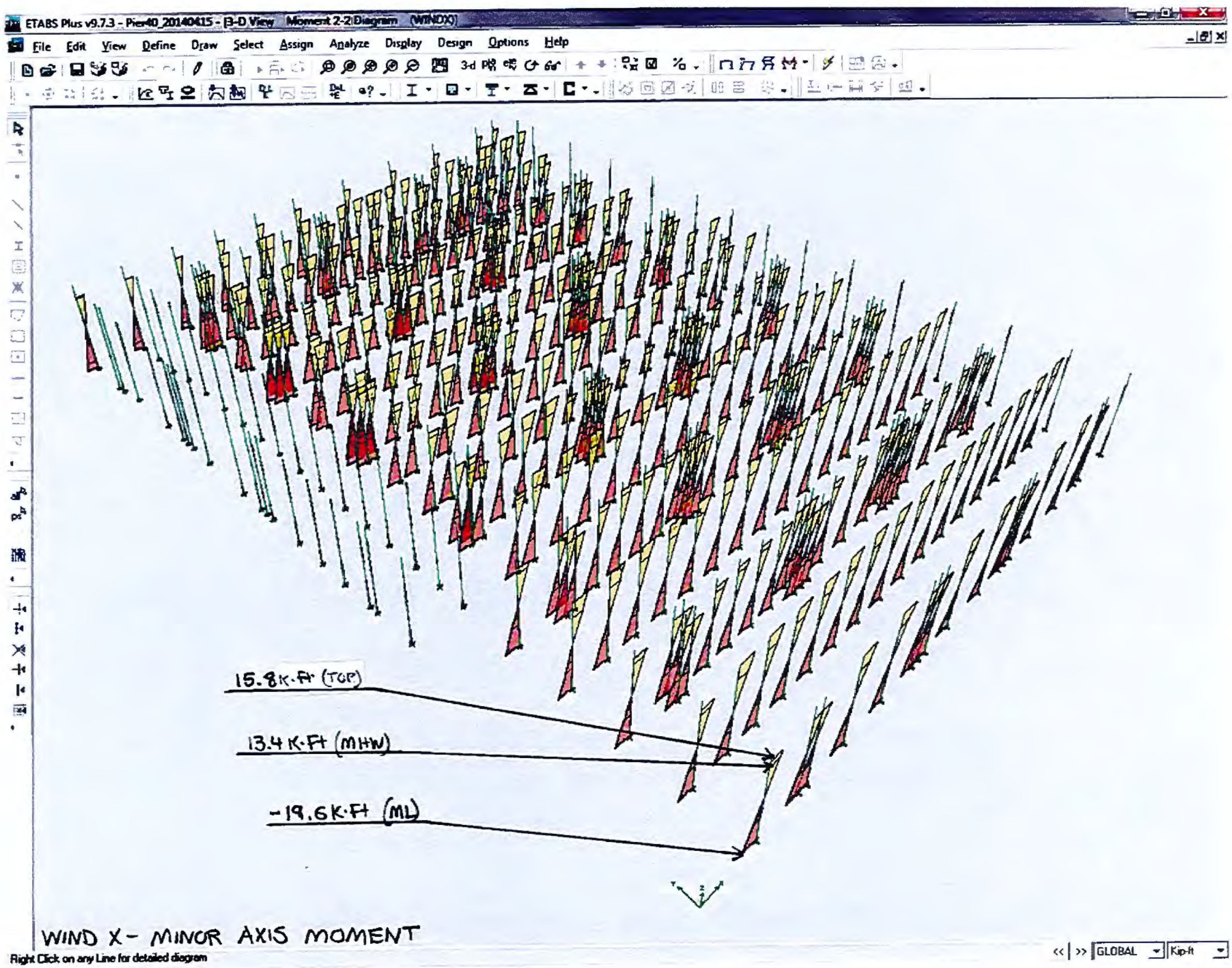
15.2K
-10.2K
-10.6 K
7.5 K
WIND X - AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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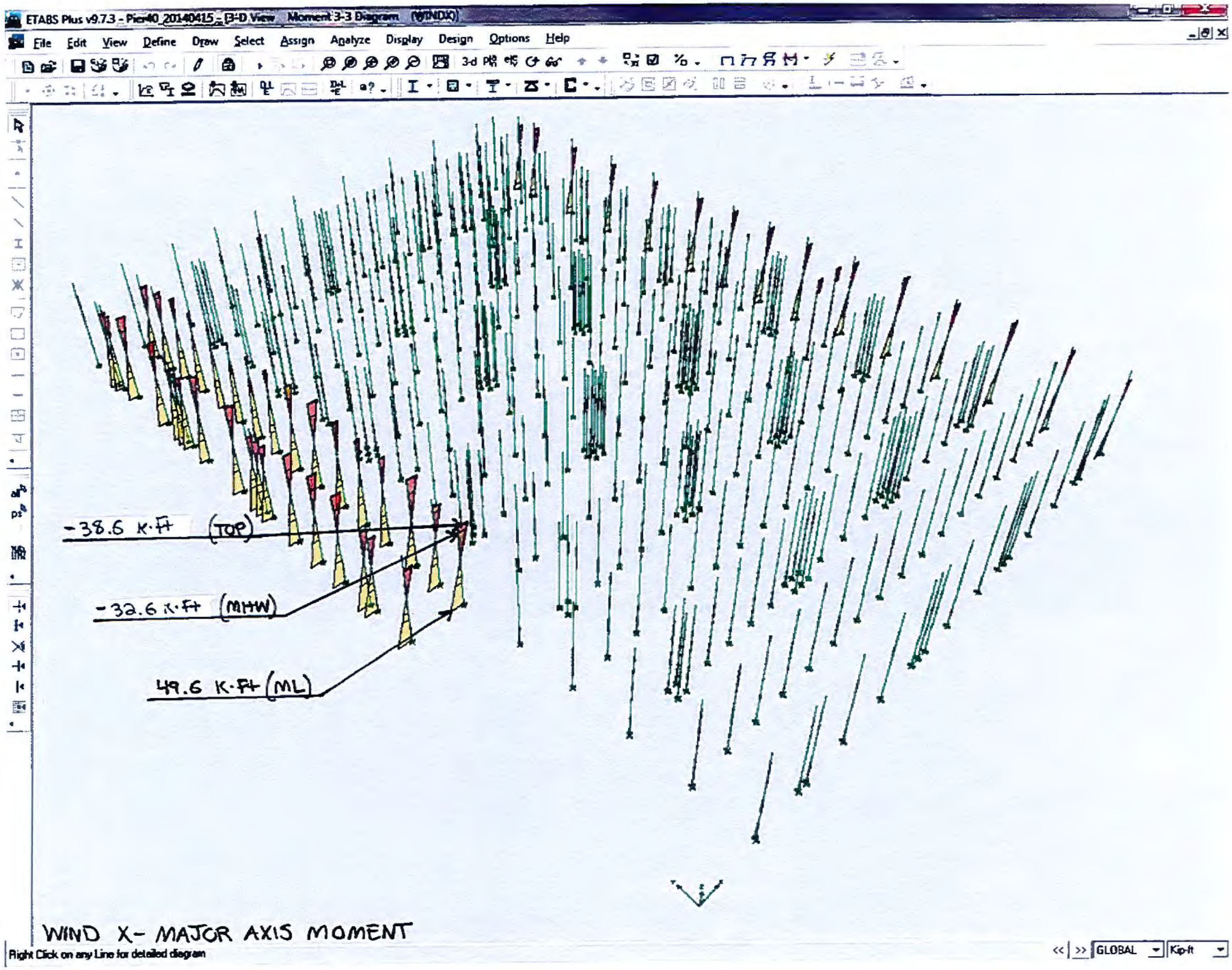
13.4 K·Ft (MHW)
-19.6K·Ft (ML)
WIND X - MINOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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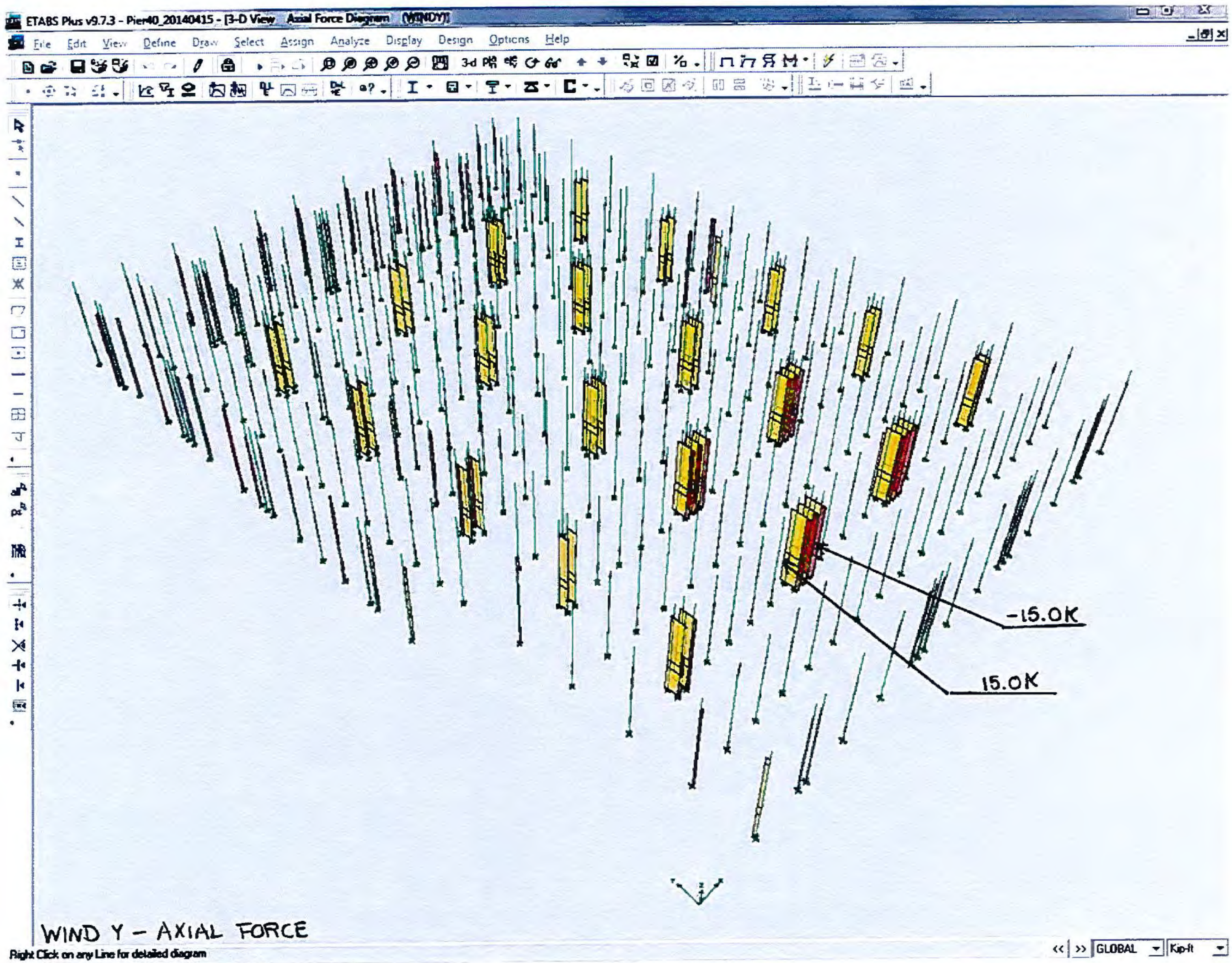
-32.6 K·Ft (MHW)
49.6 K·Ft (ML)
WIND X- MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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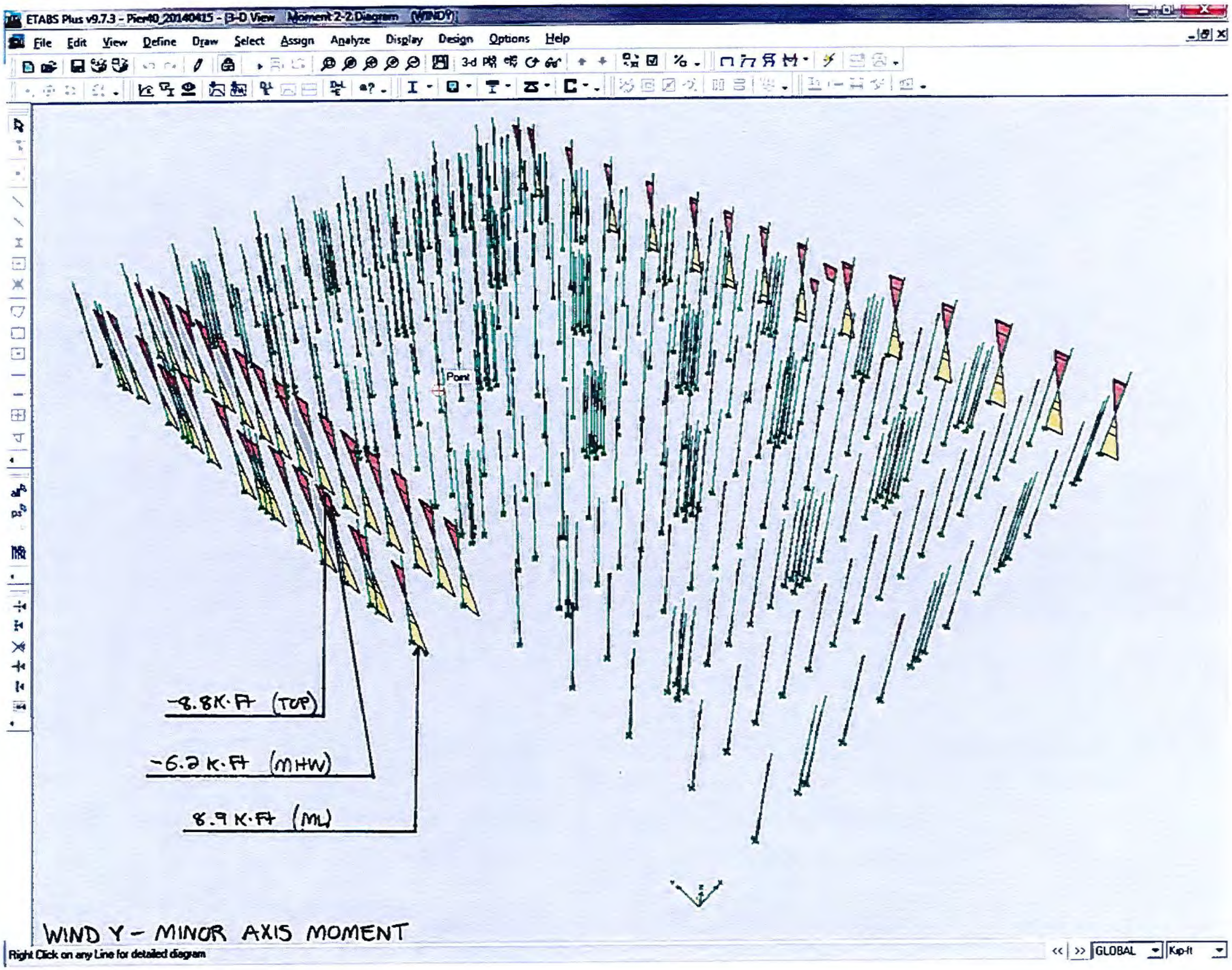
15.0K
WIND Y – AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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Point
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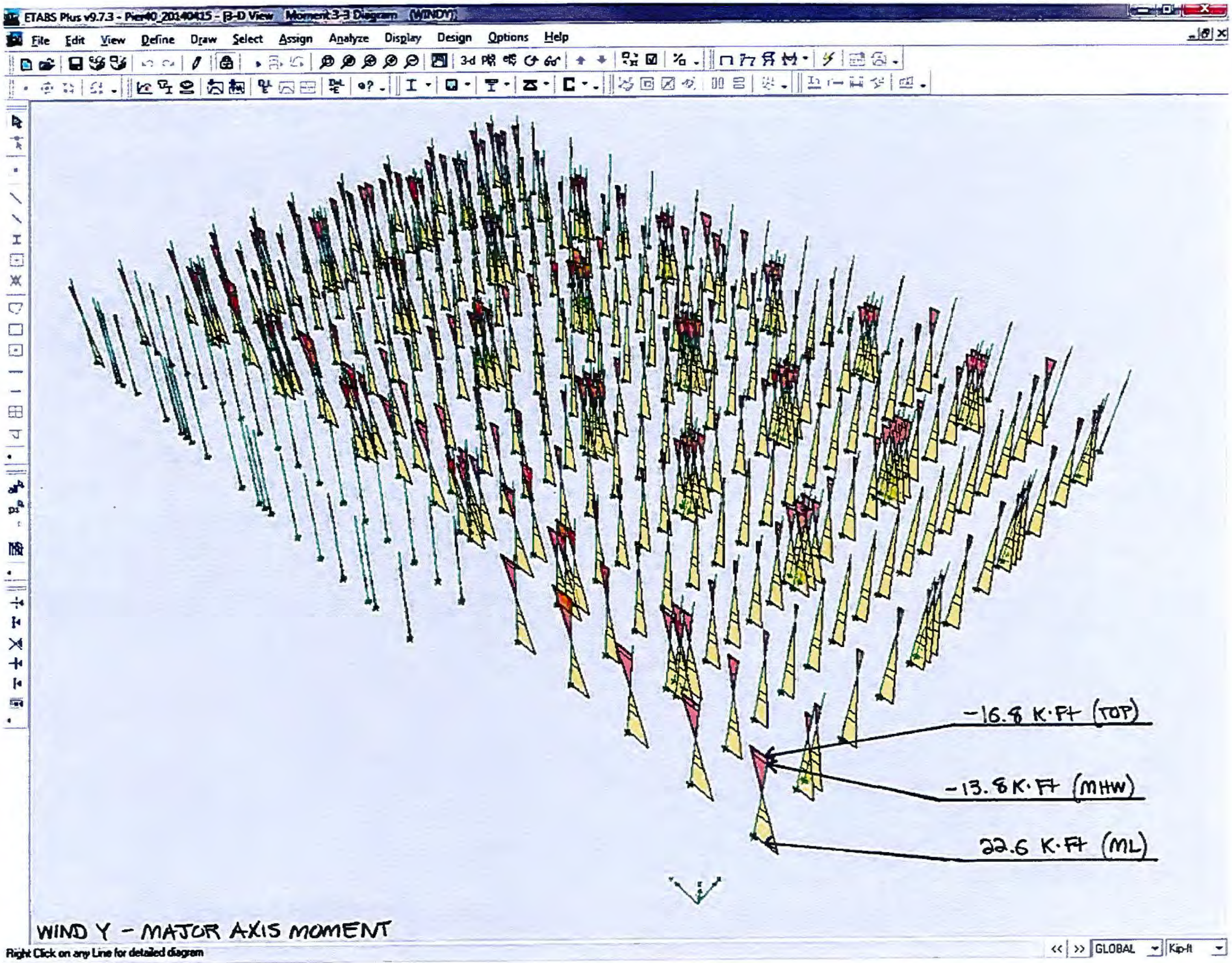
8.9 K·Ft (ML)
WIND Y - MINOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft



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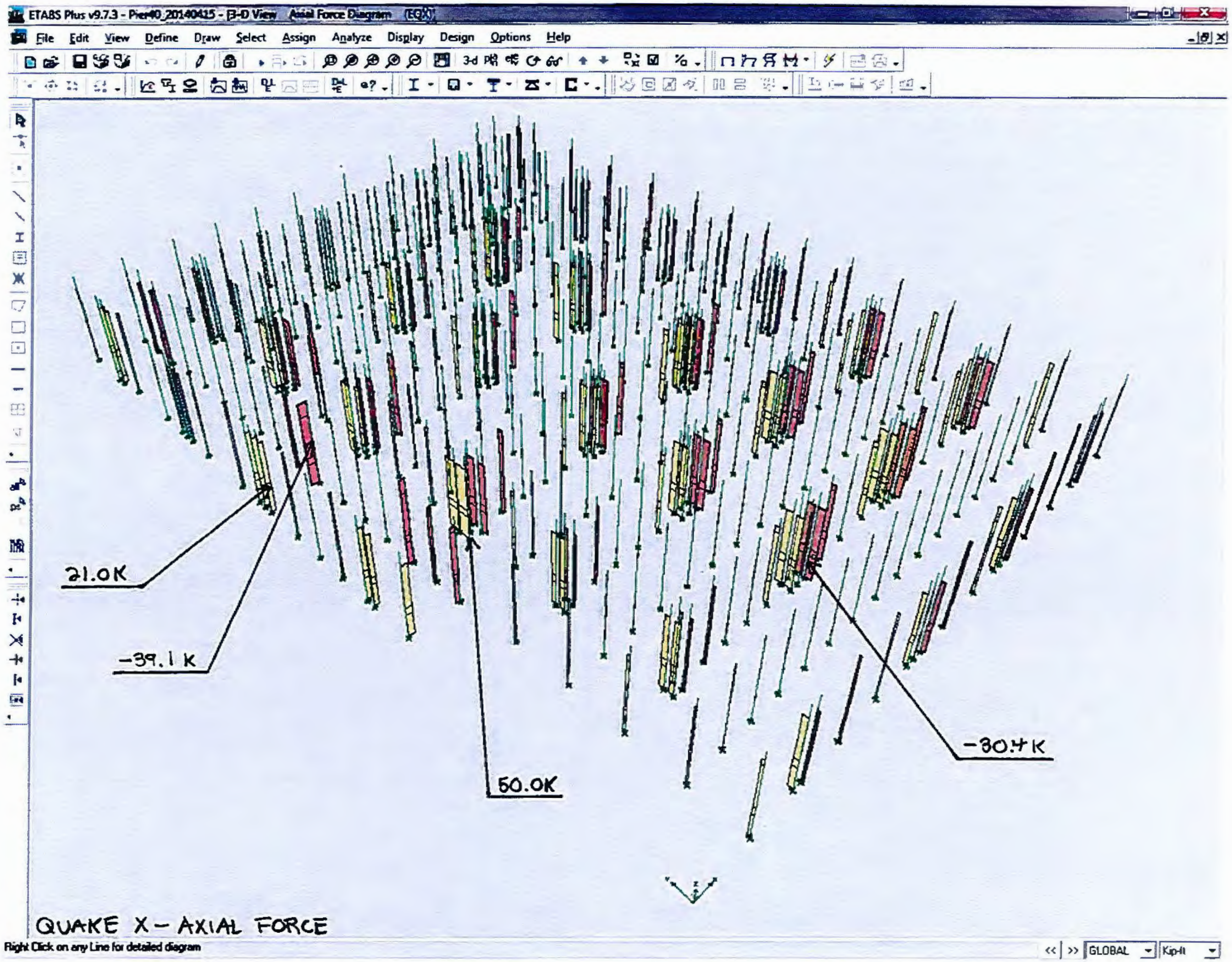
22.6 K·Ft (ML)
WIND Y - MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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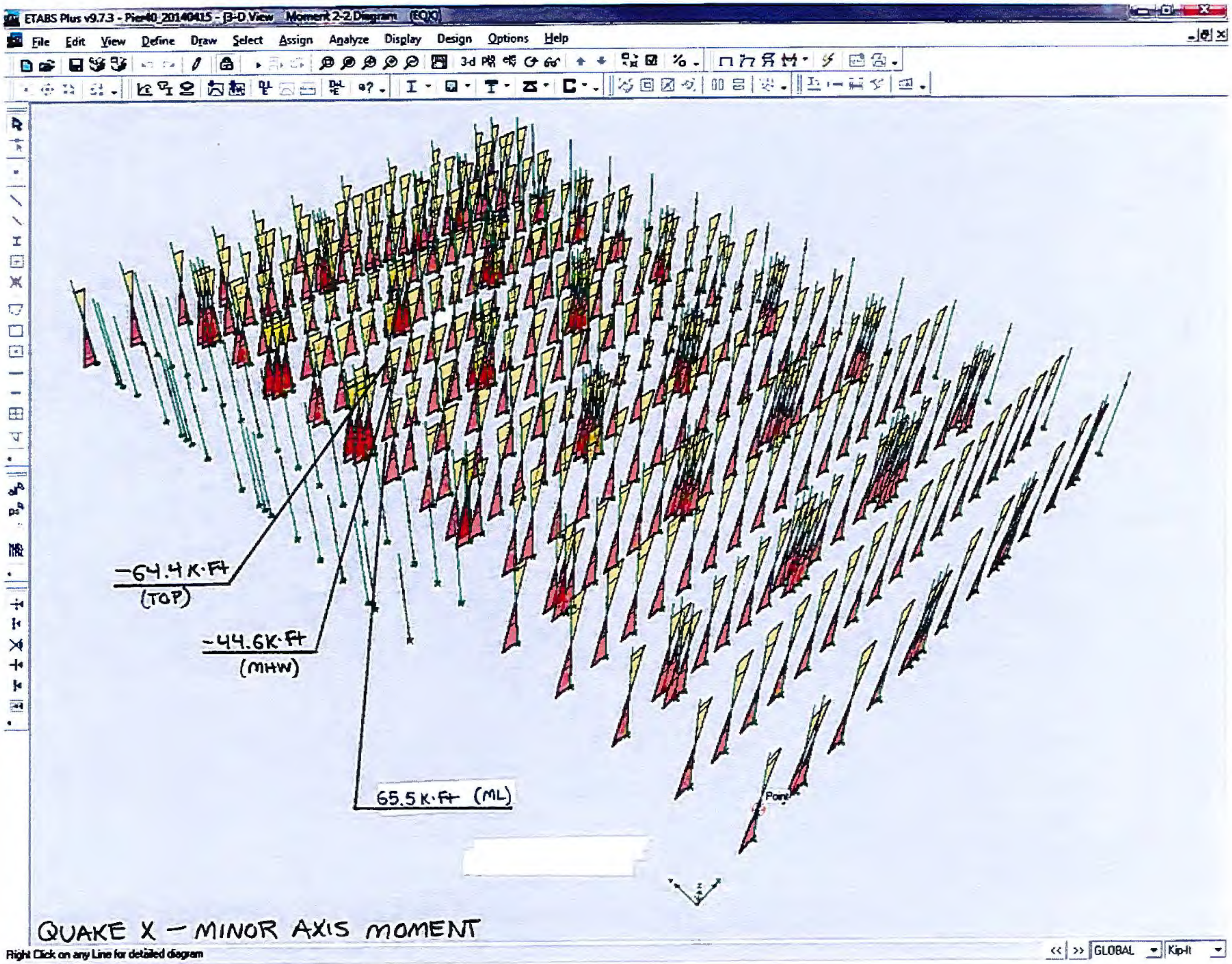
-39.1 K
50.0K
-30.4 K
QUAKE X – AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft



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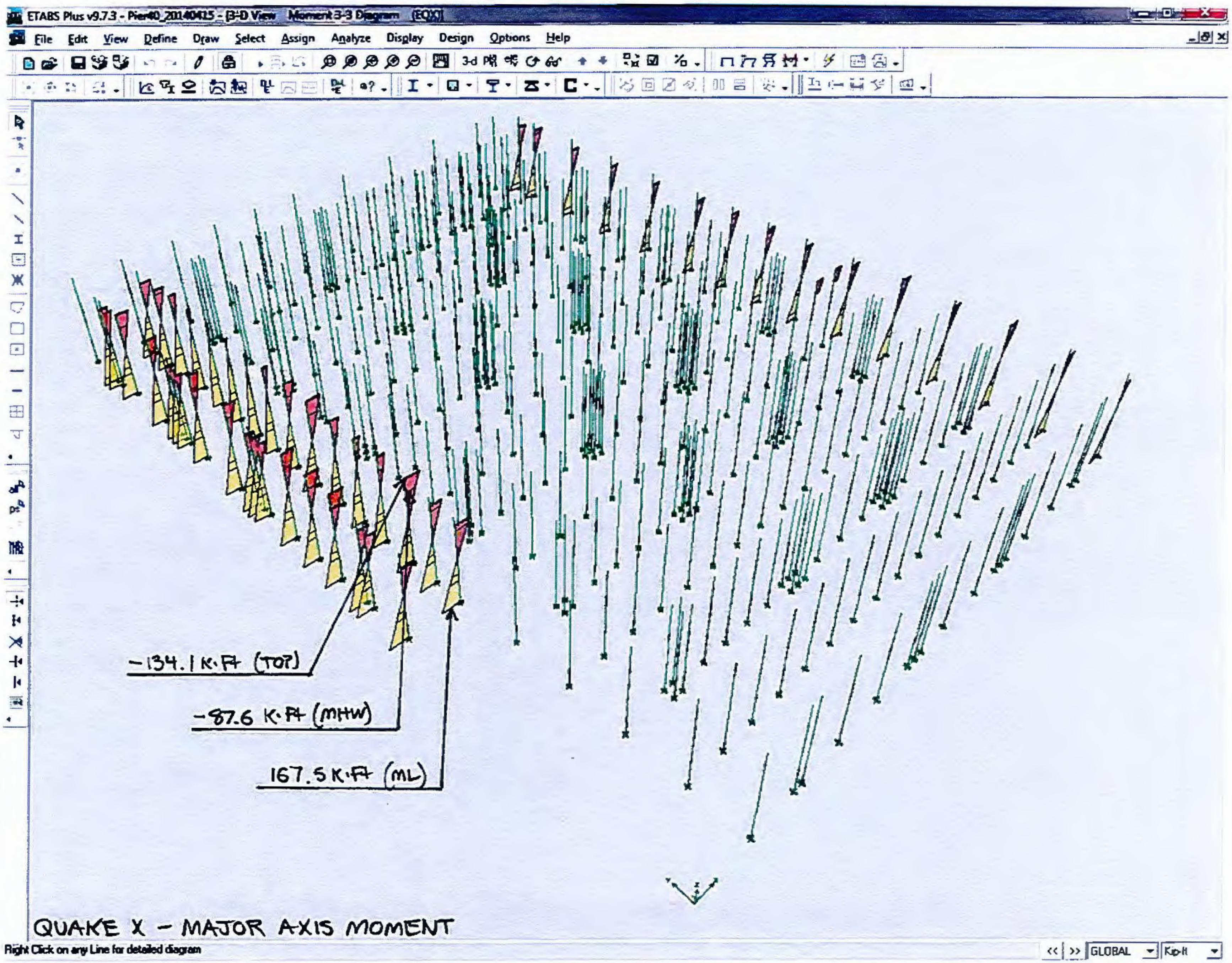
-64.4 K·Ft
(TOP)
-44.6K·Ft
(MHW)
65.5 K·Ft (ML)
Point
QUAKE X — MINOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft



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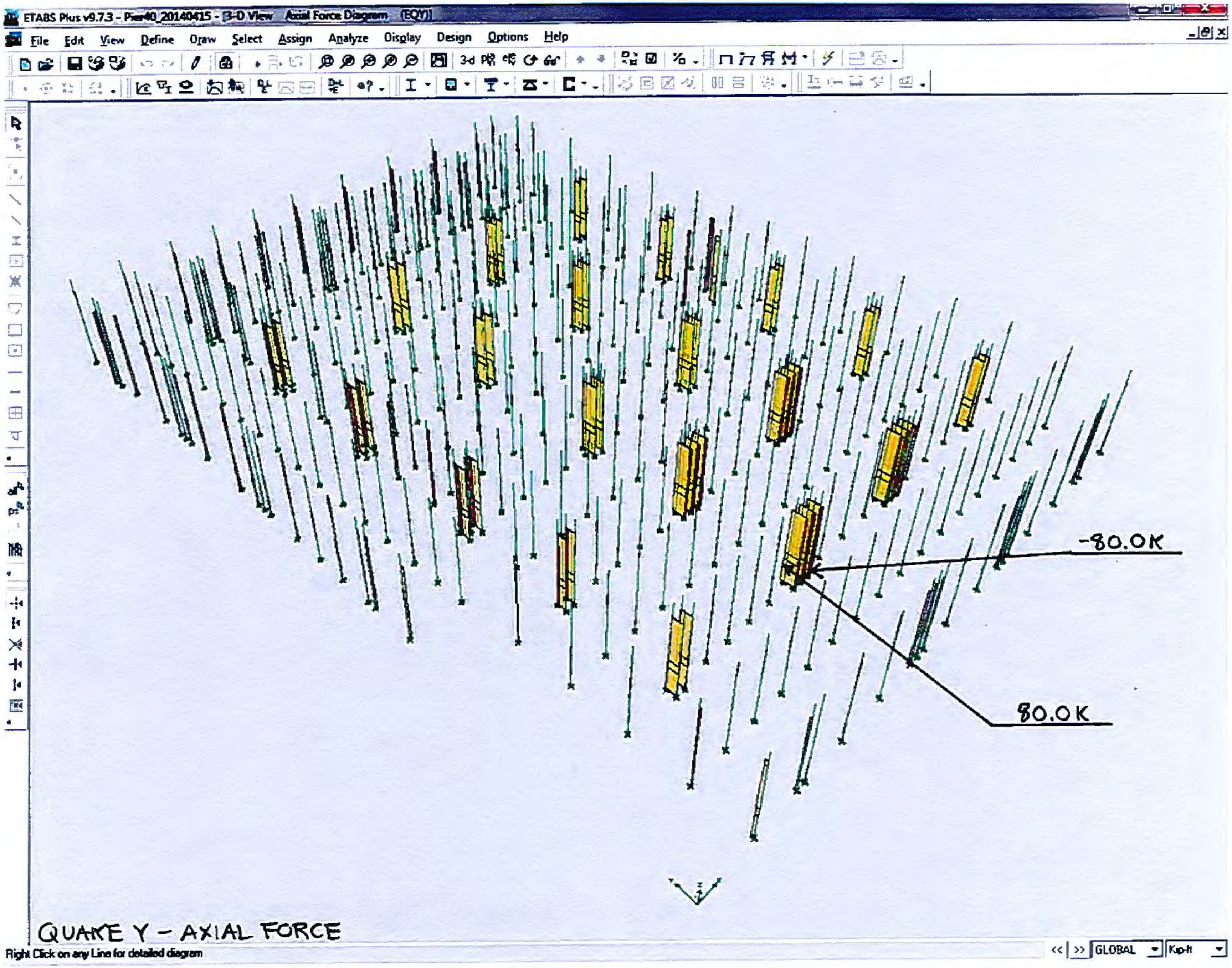
-87.6 K·Ft (MHW)
167.5 K·Ft (ML)
QUAKE X - MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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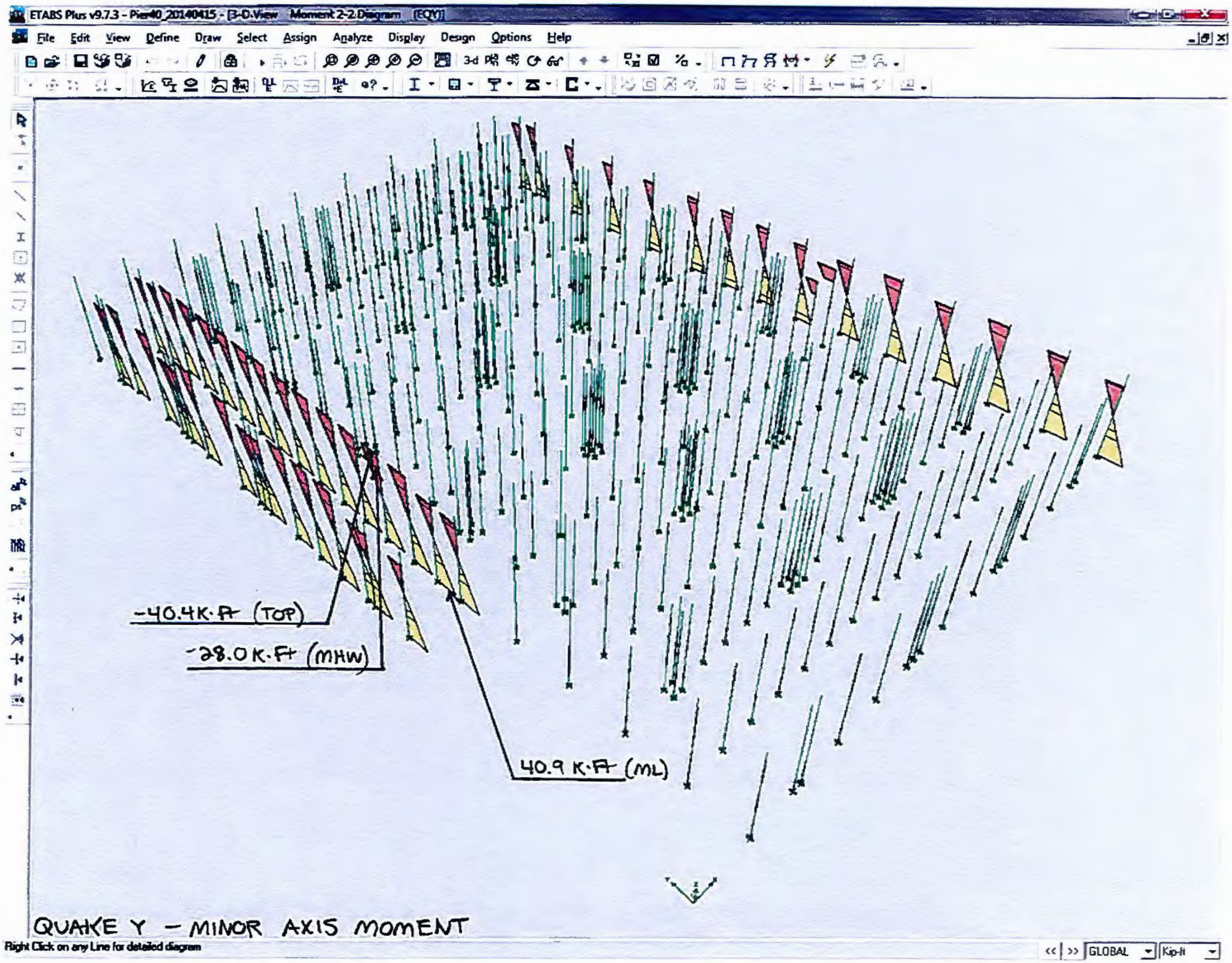
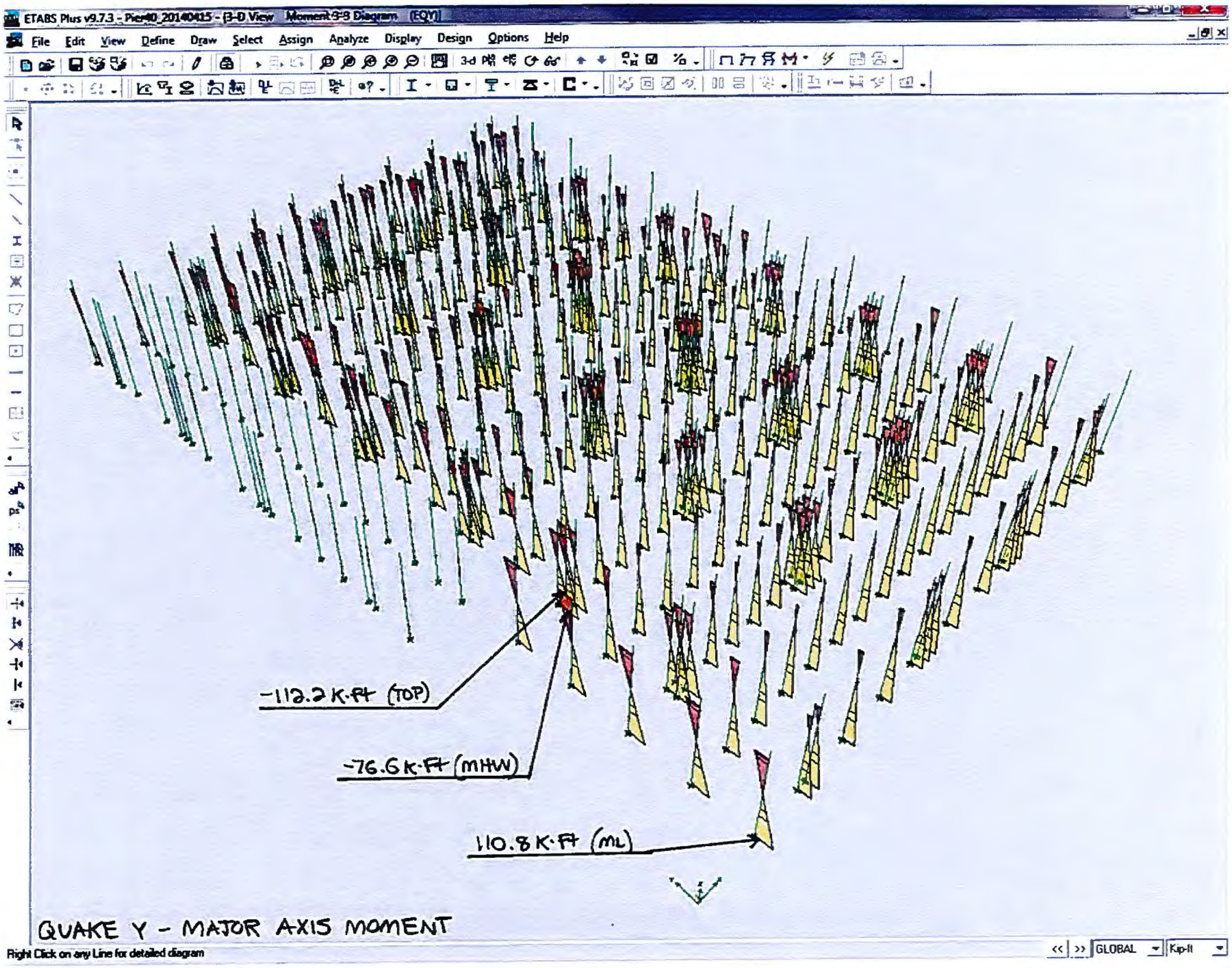
80.0K
QUAKE Y - AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft



QUAKE Y – MINOR AXIS MOMENT



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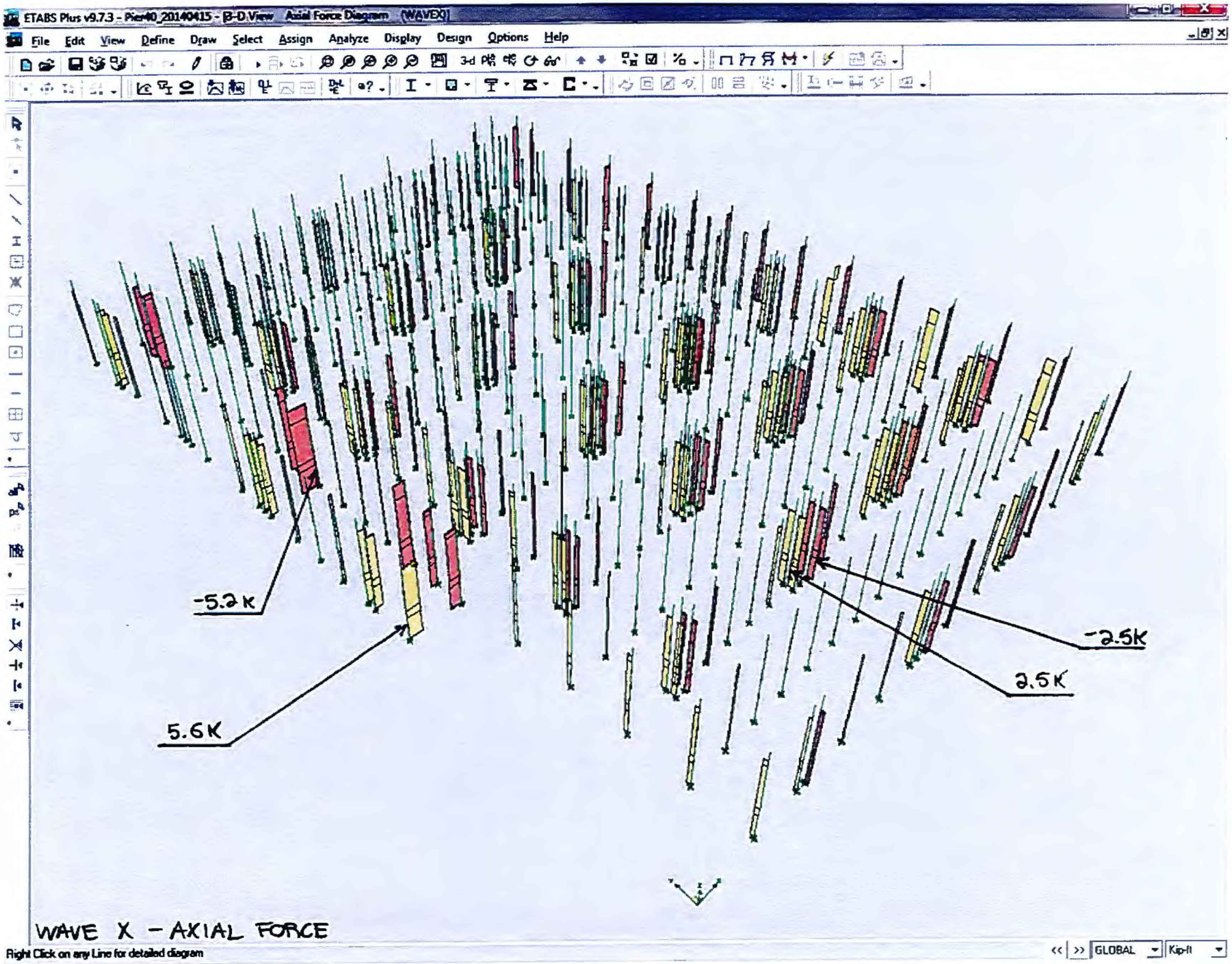
110.8 K·Ft (ML)
QUAKE Y - MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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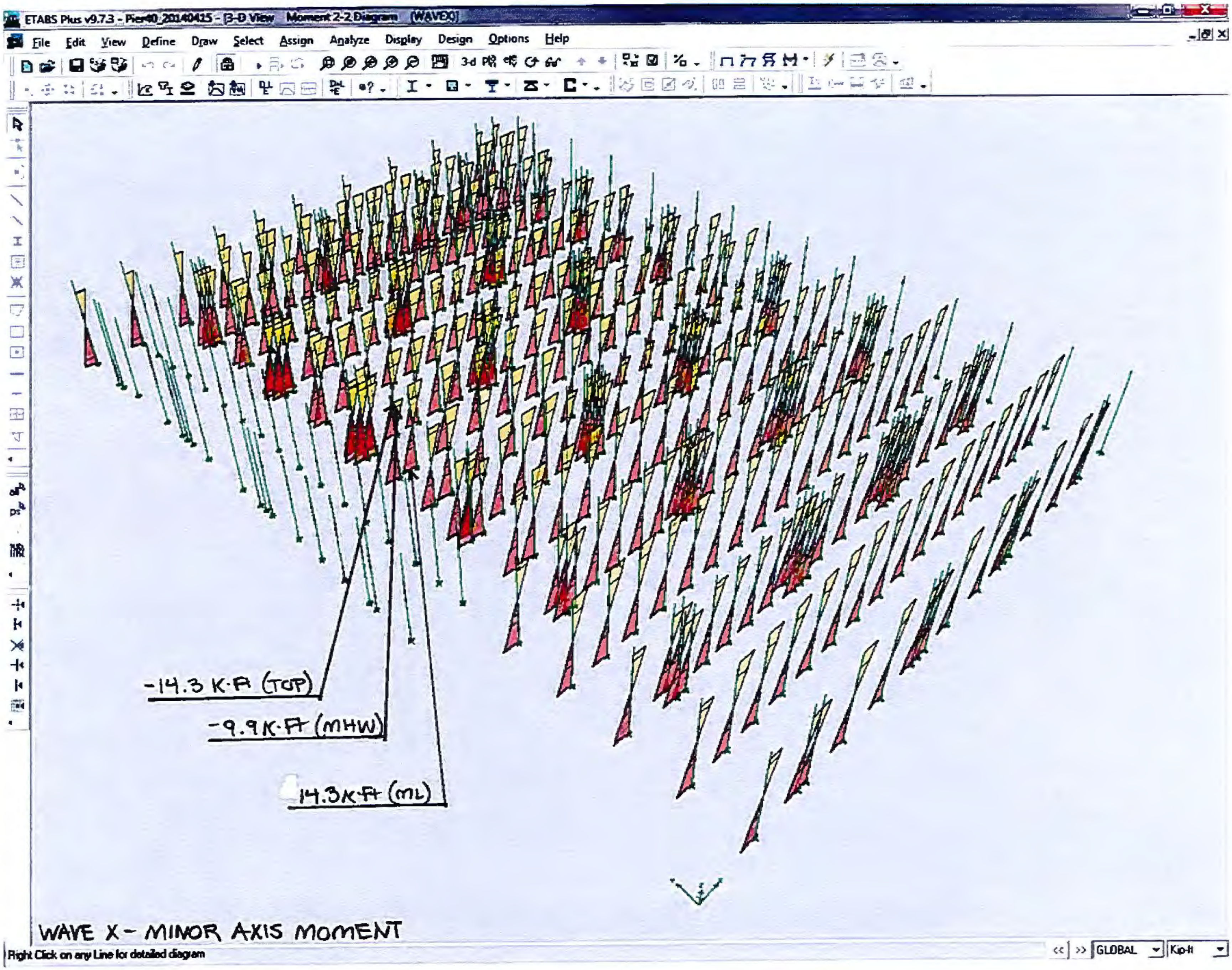
5.6K
-2.5K
2.5K
WAVE X - AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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14.3 K·Ft (ML)
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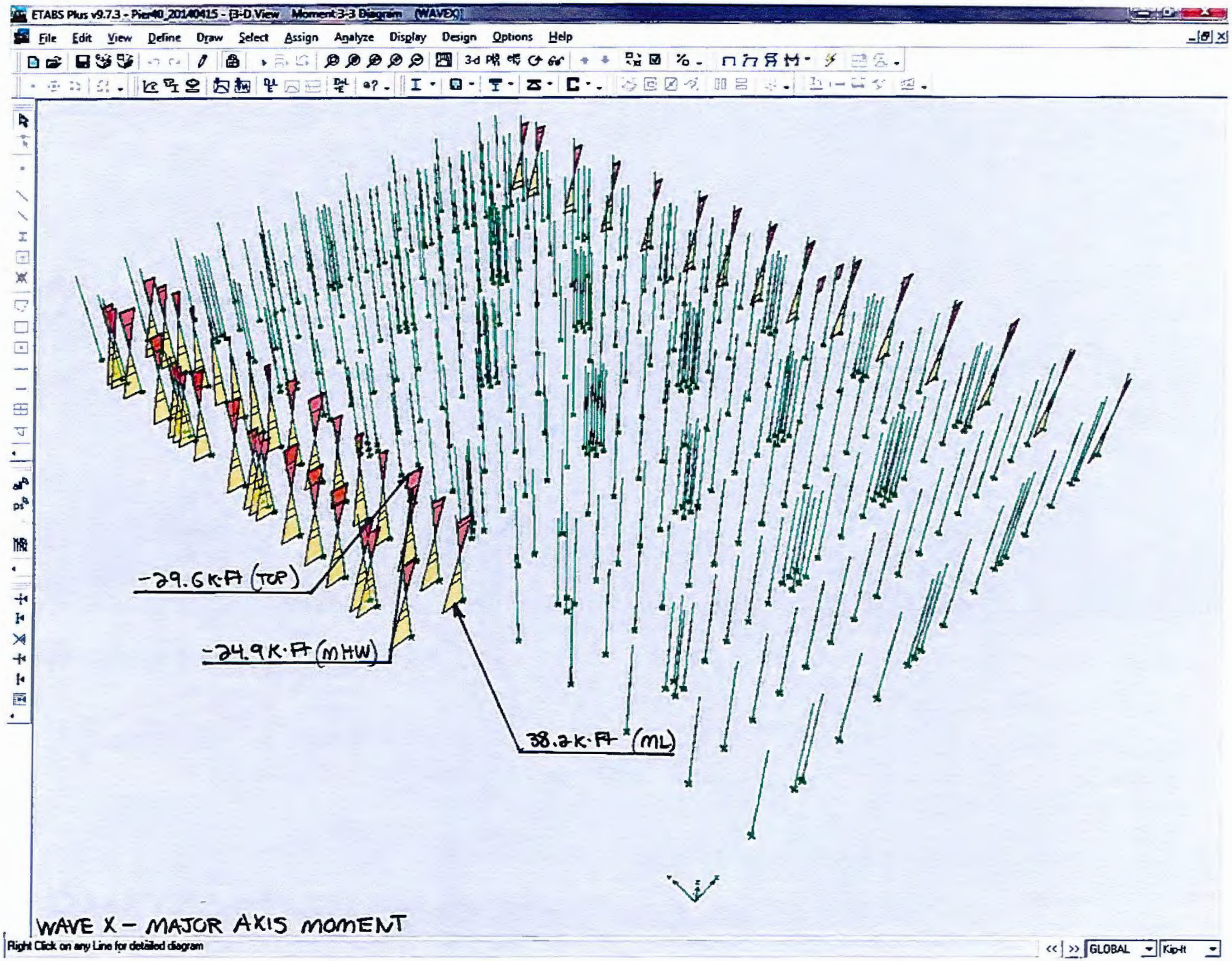
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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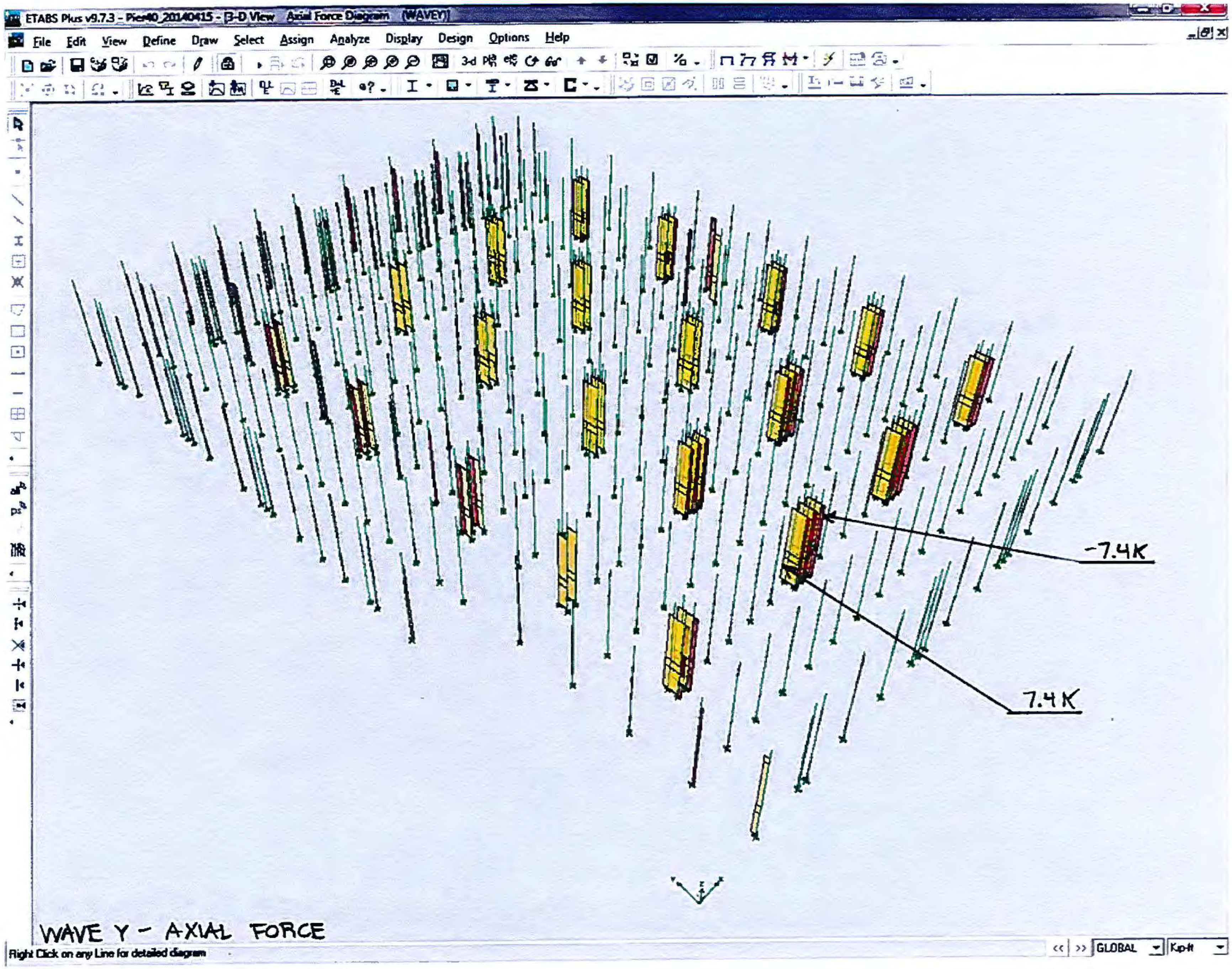
38.2K·Ft (ML)
WAVE X – MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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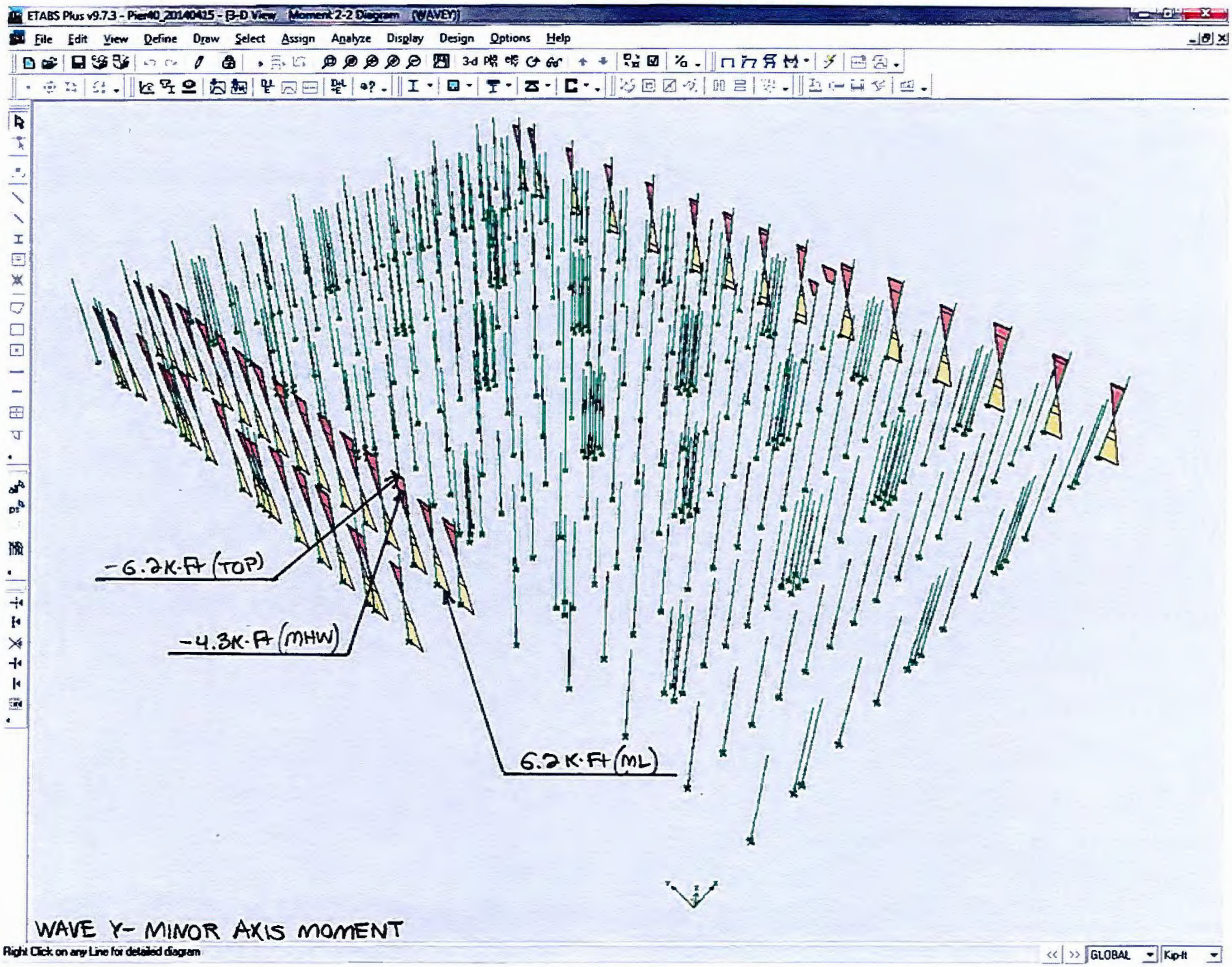
7.4K
WAVE Y - AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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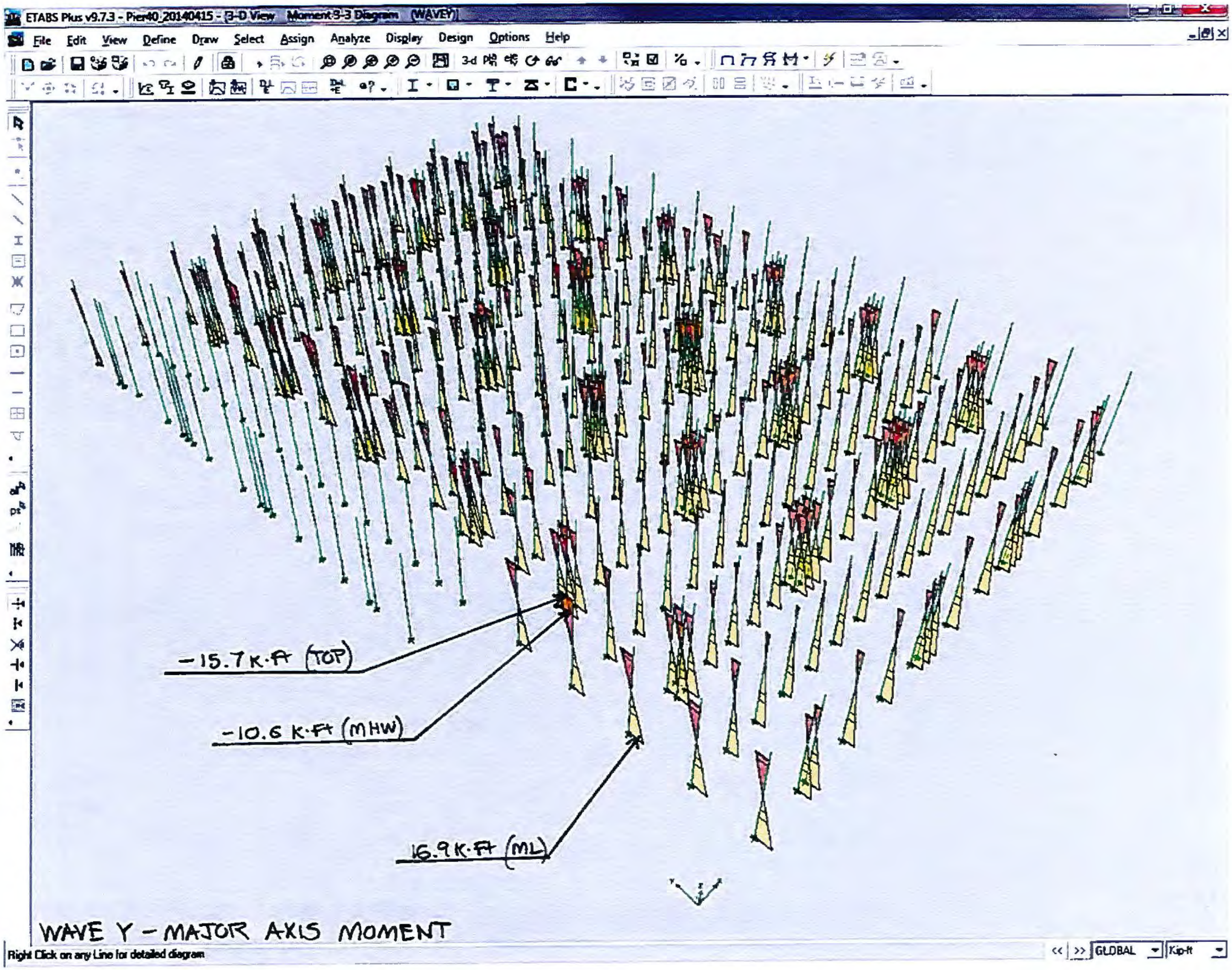
6.2K-Ft (ML)
WAVE Y- MINOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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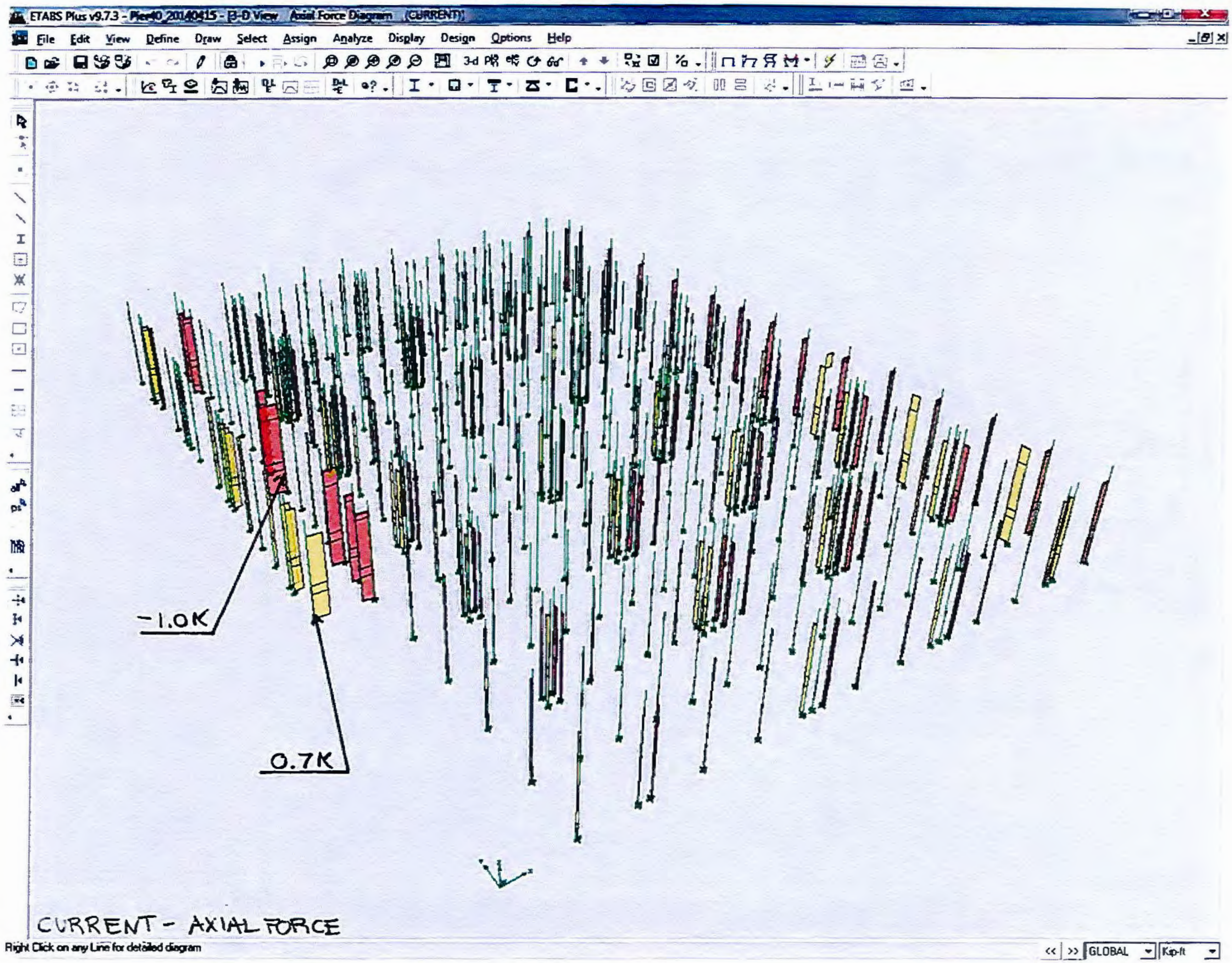
16.9 K·Ft (ML)
WAVE Y - MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft



ETABS Plus v9.7.3 - Pier40_20140415 - 3-D View Axial Force Diagram (CURRENT)
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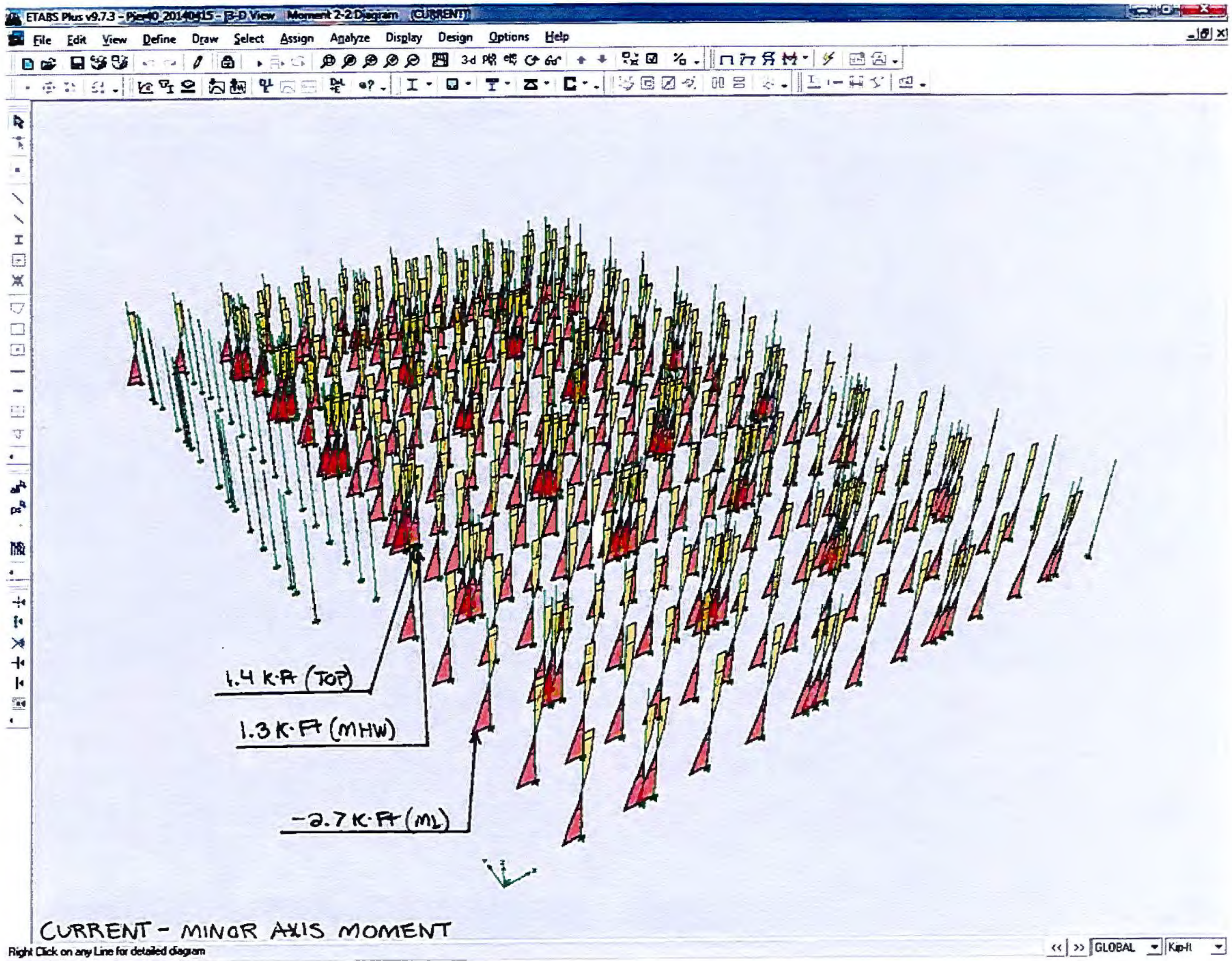
0.7K
CURRENT - AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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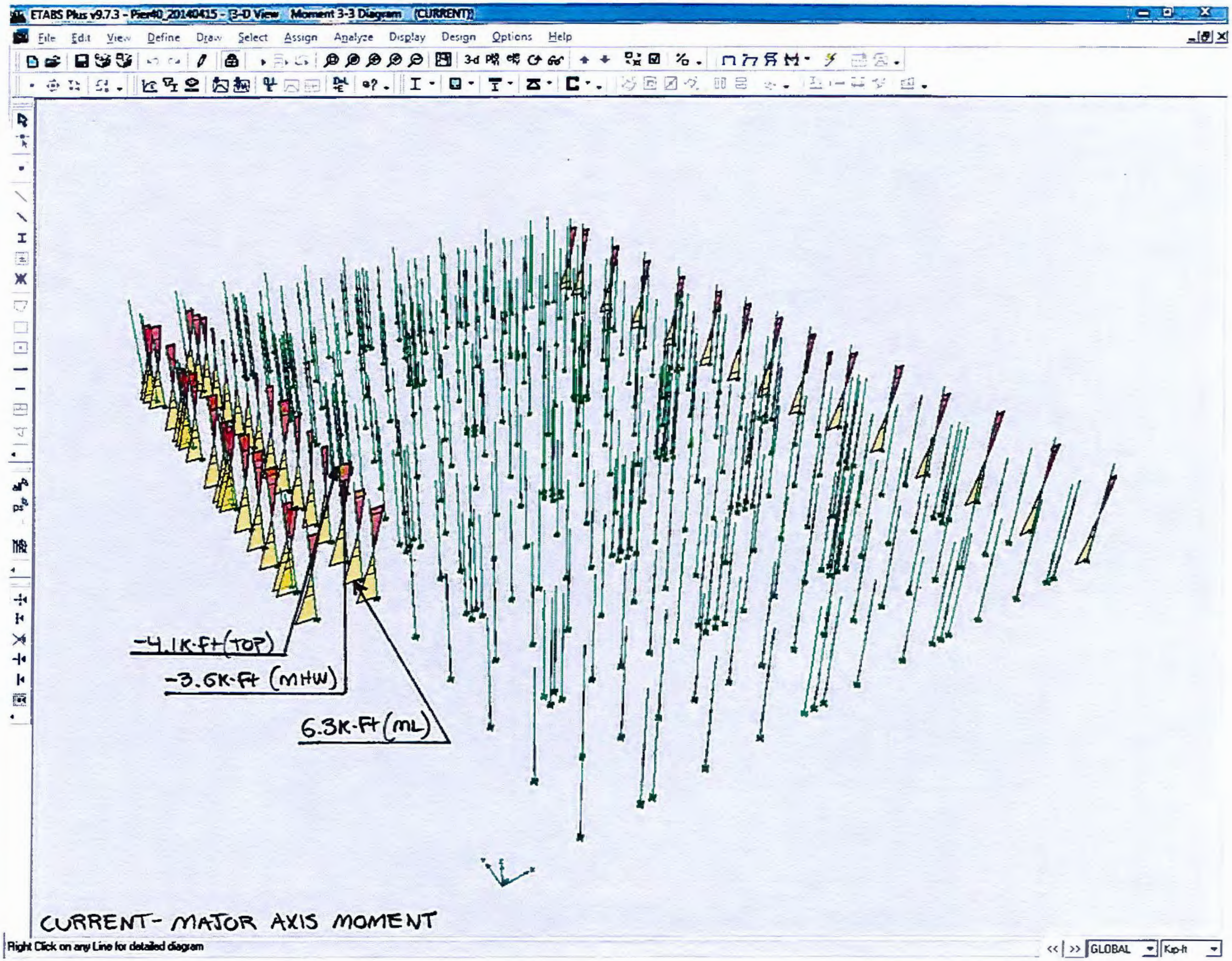
1.4 K·Ft (TOP)
1.3 K·Ft (MHW)
-2.7 K·Ft (ML)
CURRENT - MINOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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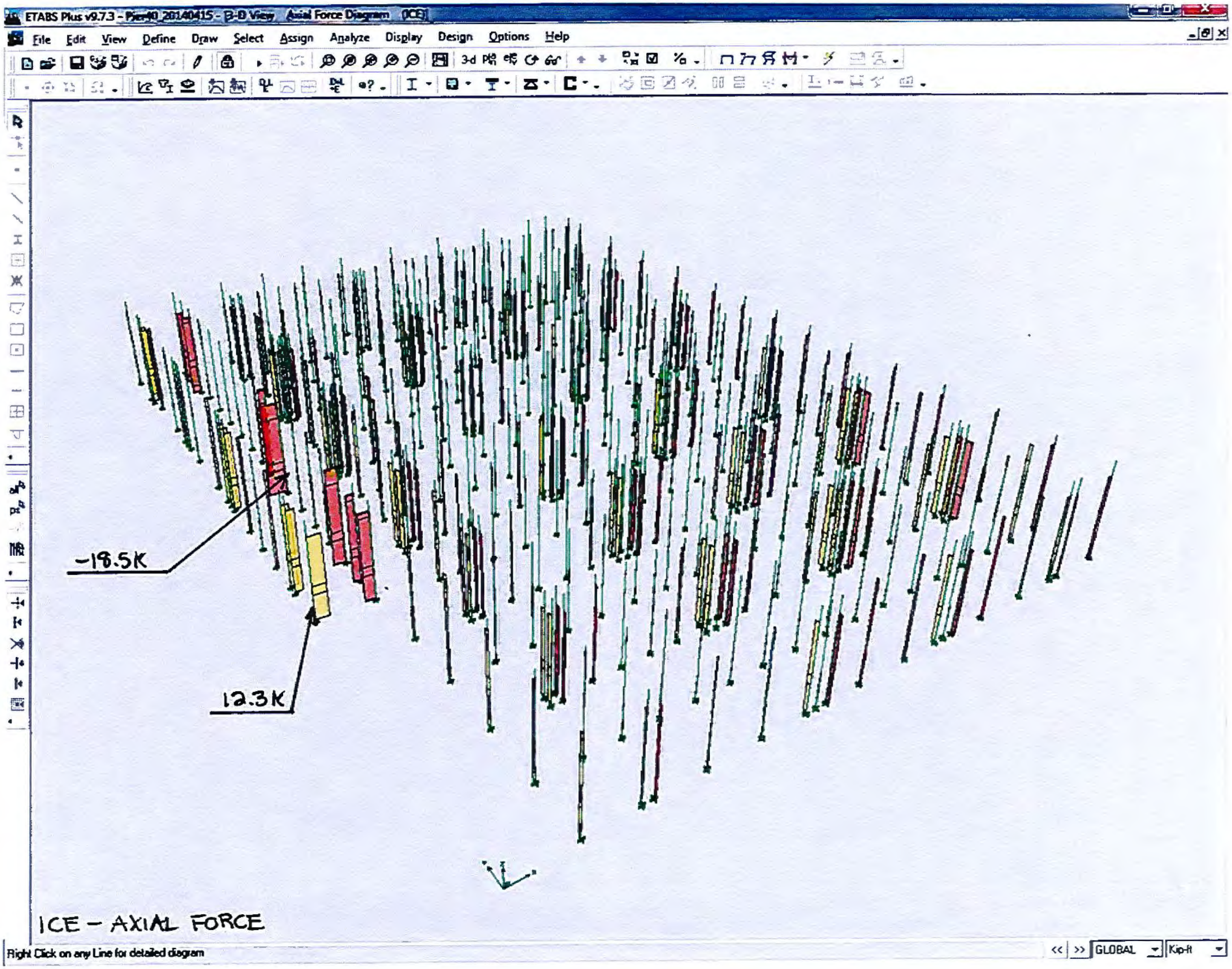
-3.6K-Ft (MHW)
6.3K-Ft (ML)
CURRENT- MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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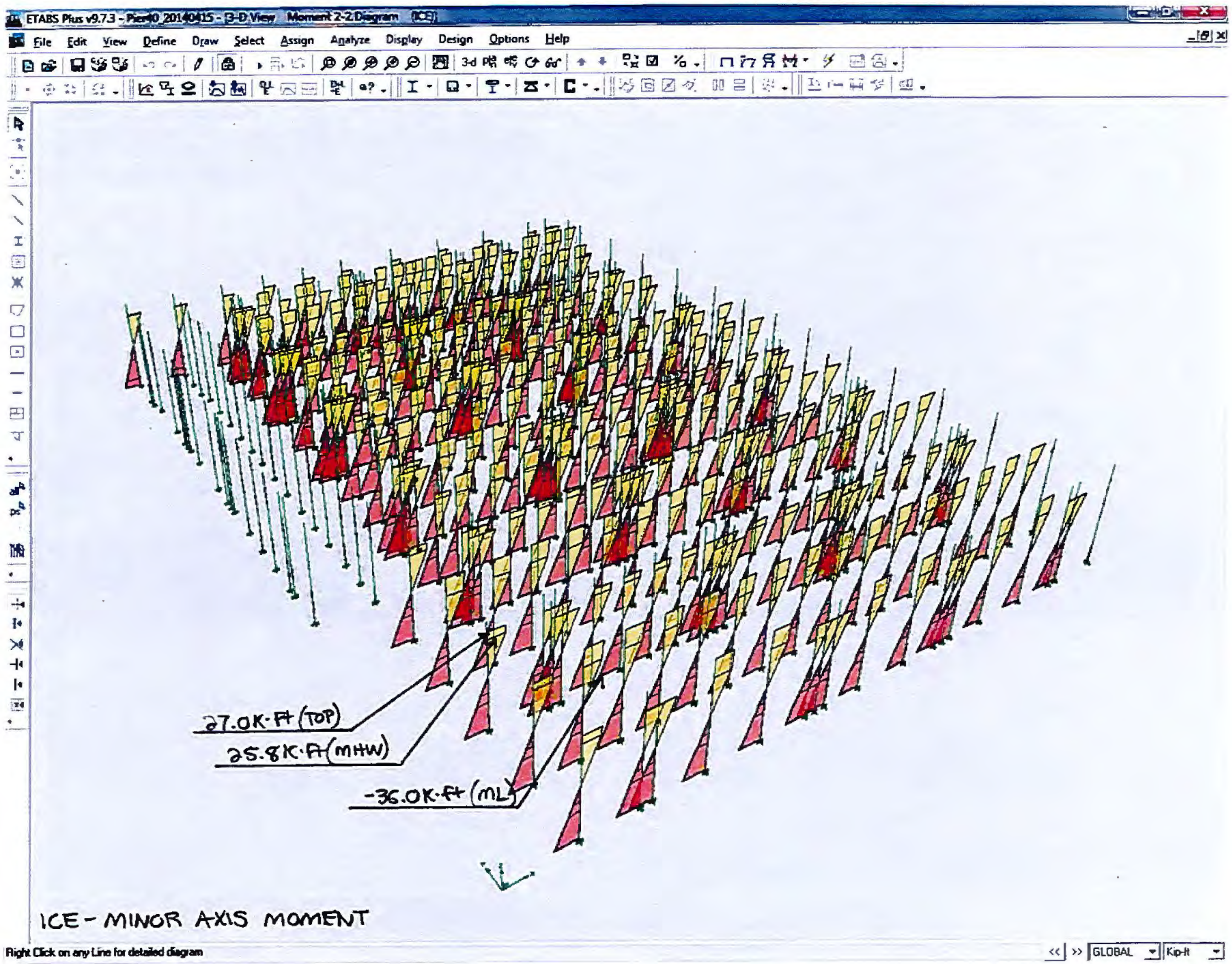
-18.5K
12.3K
ICE – AXIAL FORCE
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft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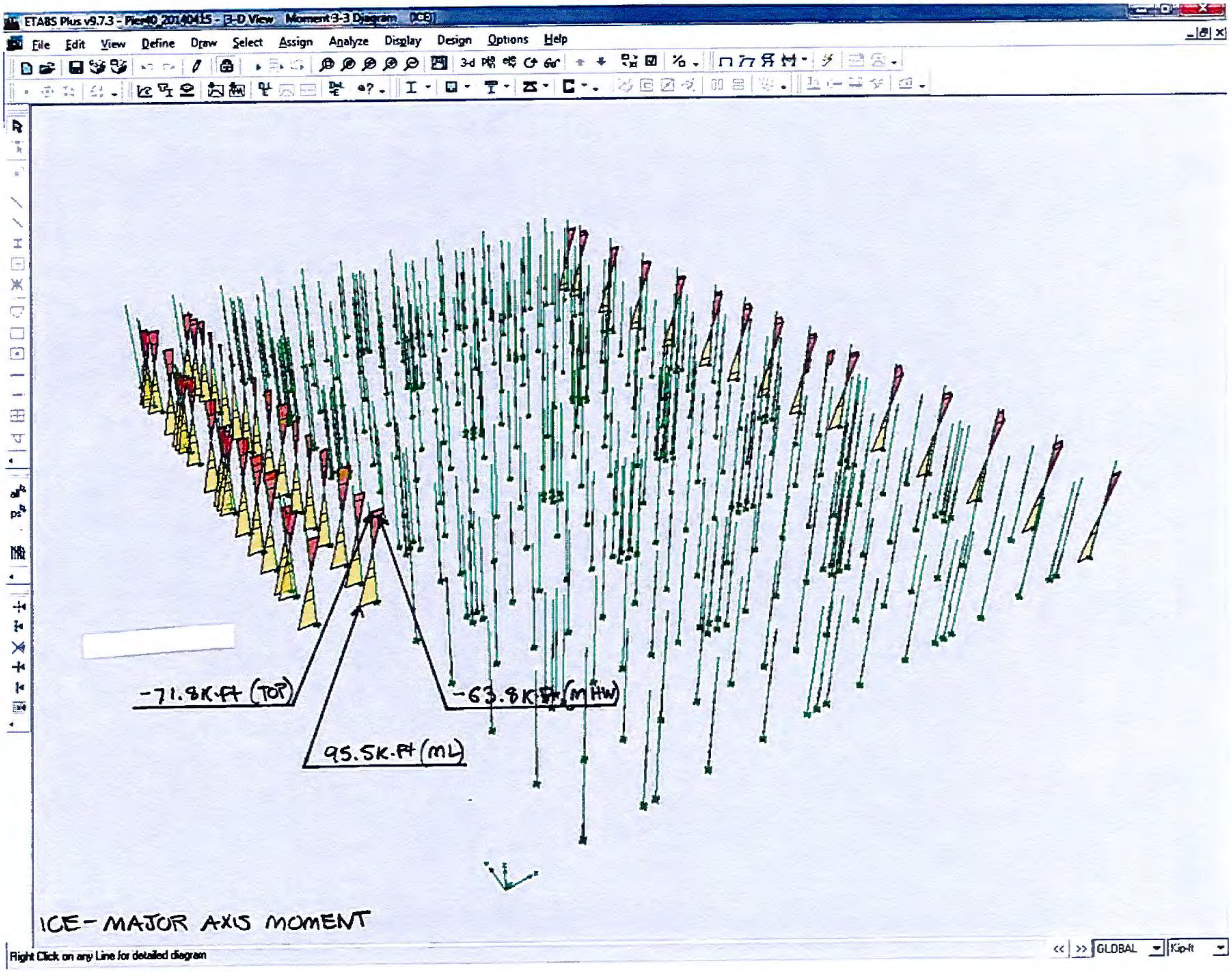
-36.0K-Ft (ML)
ICE - MINOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft



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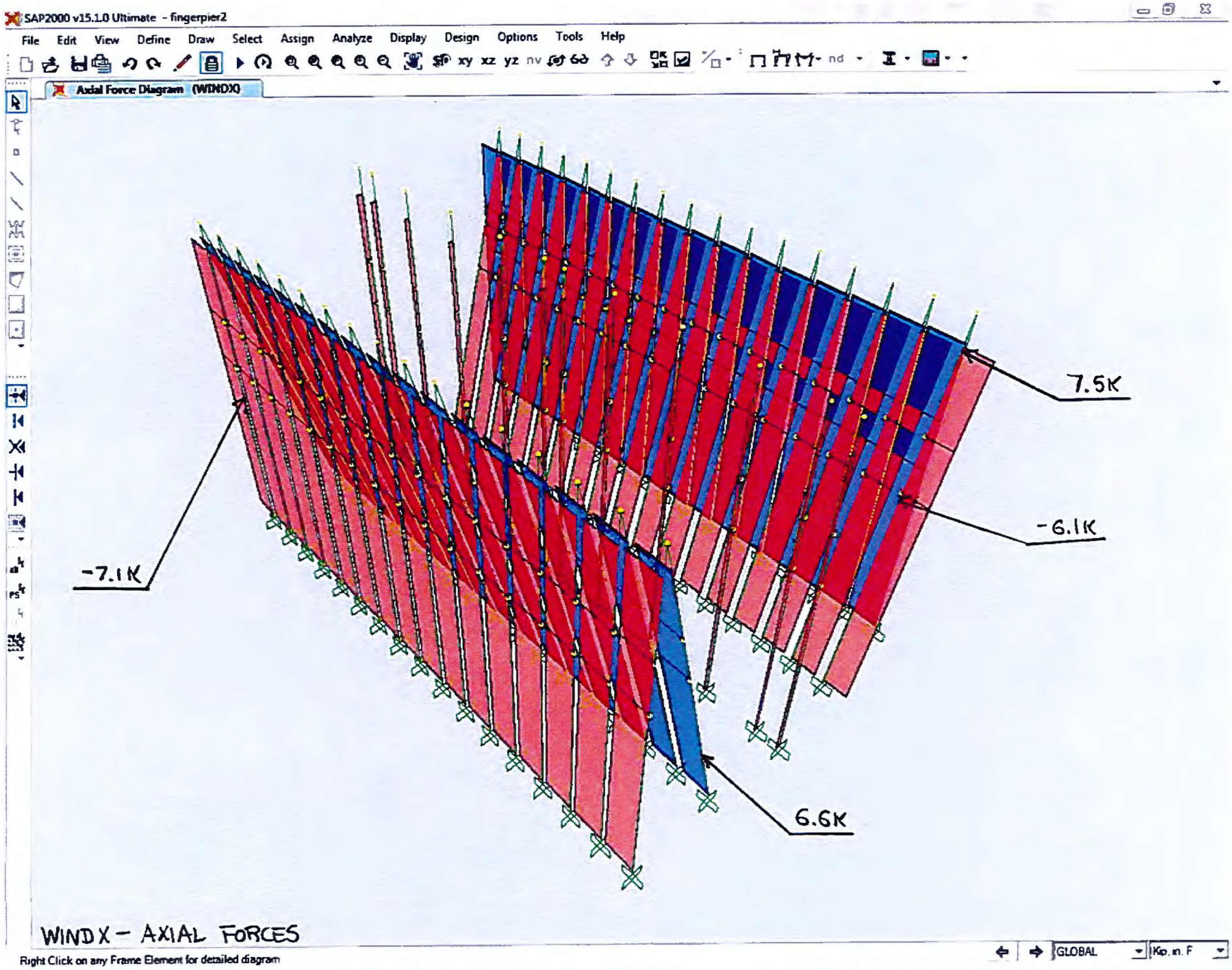
-63.8K-Ft (MHW)
95.5K-Ft (ML)
ICE - MAJOR AXIS MOMENT
Right Click on any Line for detailed diagram
GLOBAL
Kip-ft

FINGER PIER STRUCTURAL MODEL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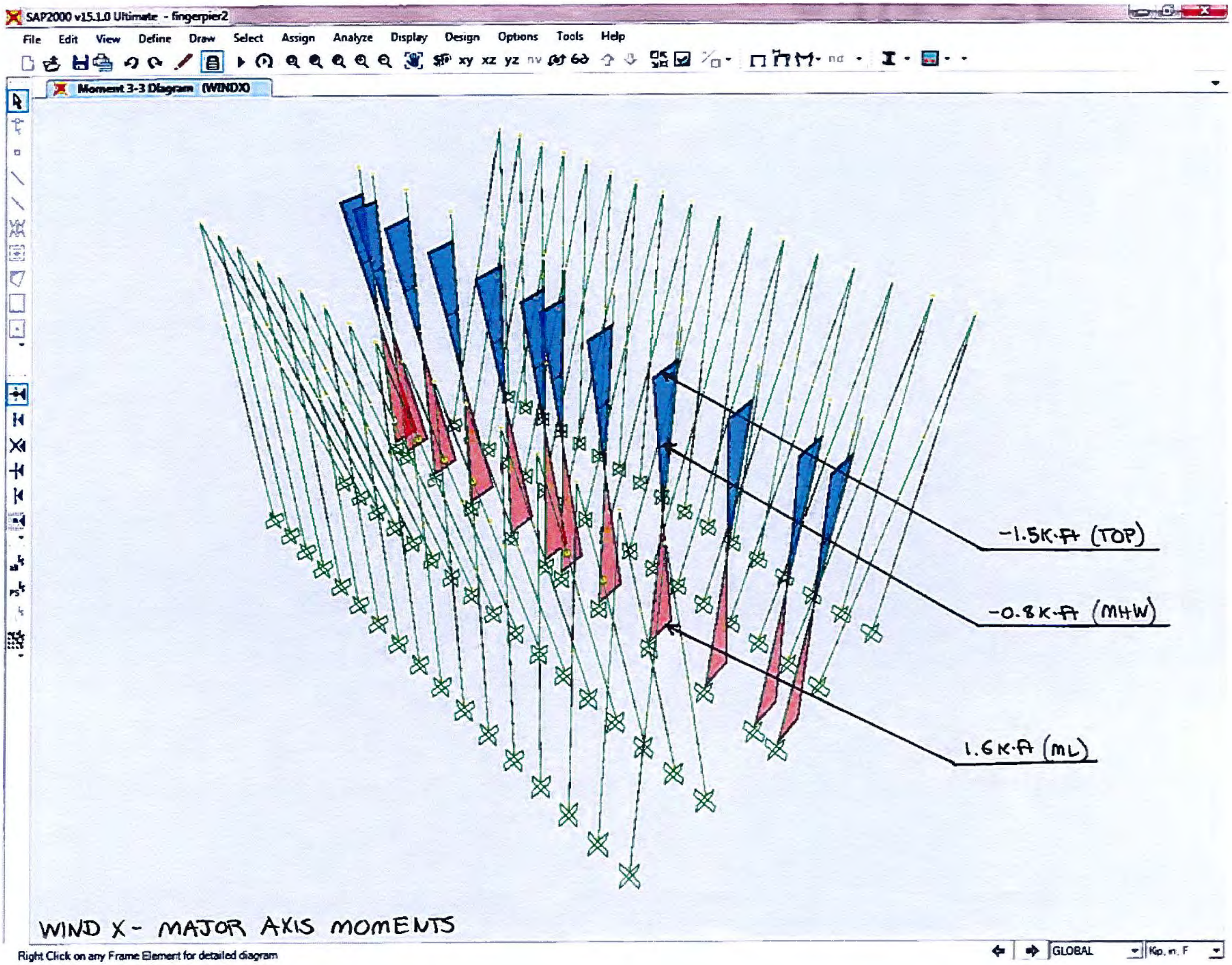
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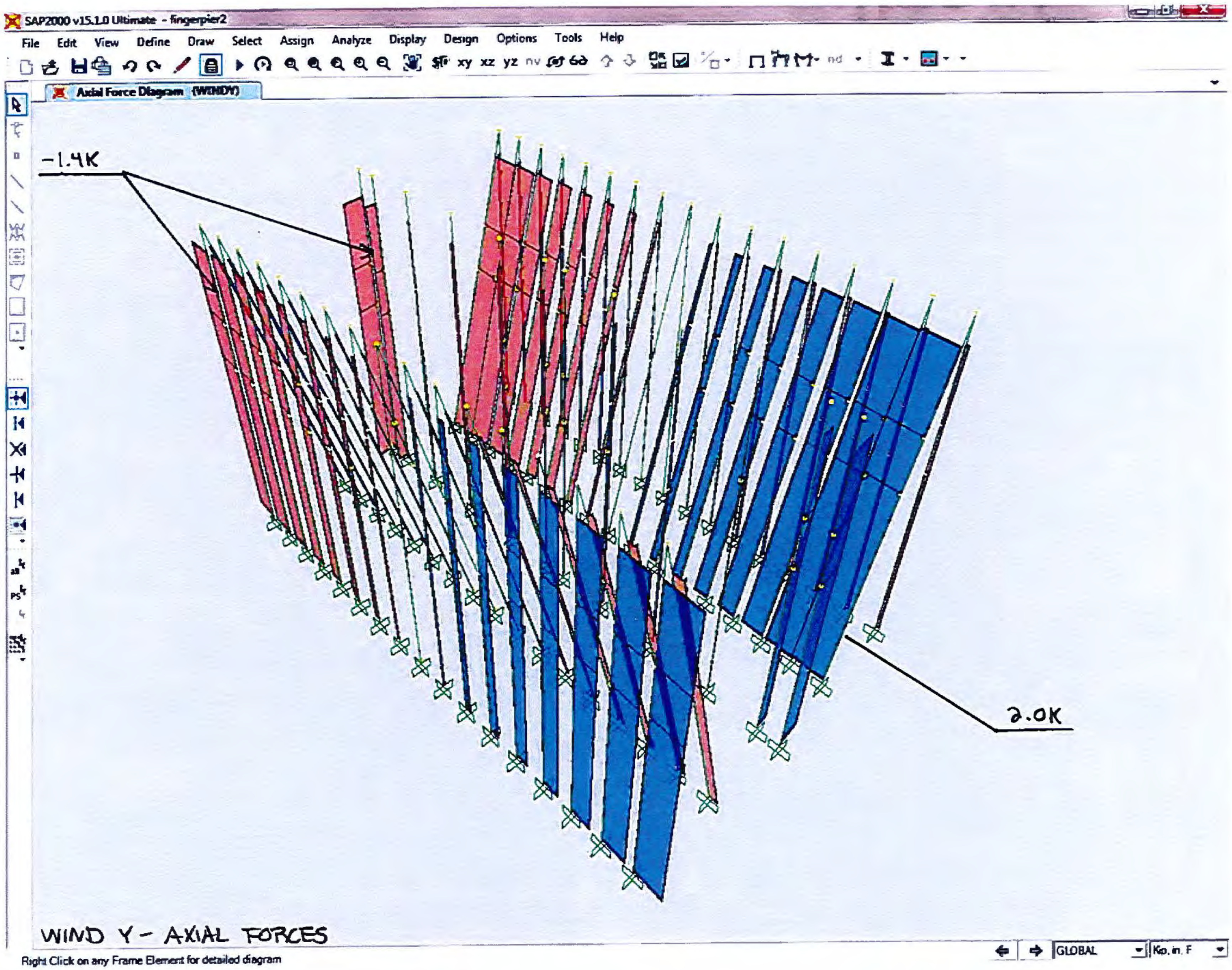
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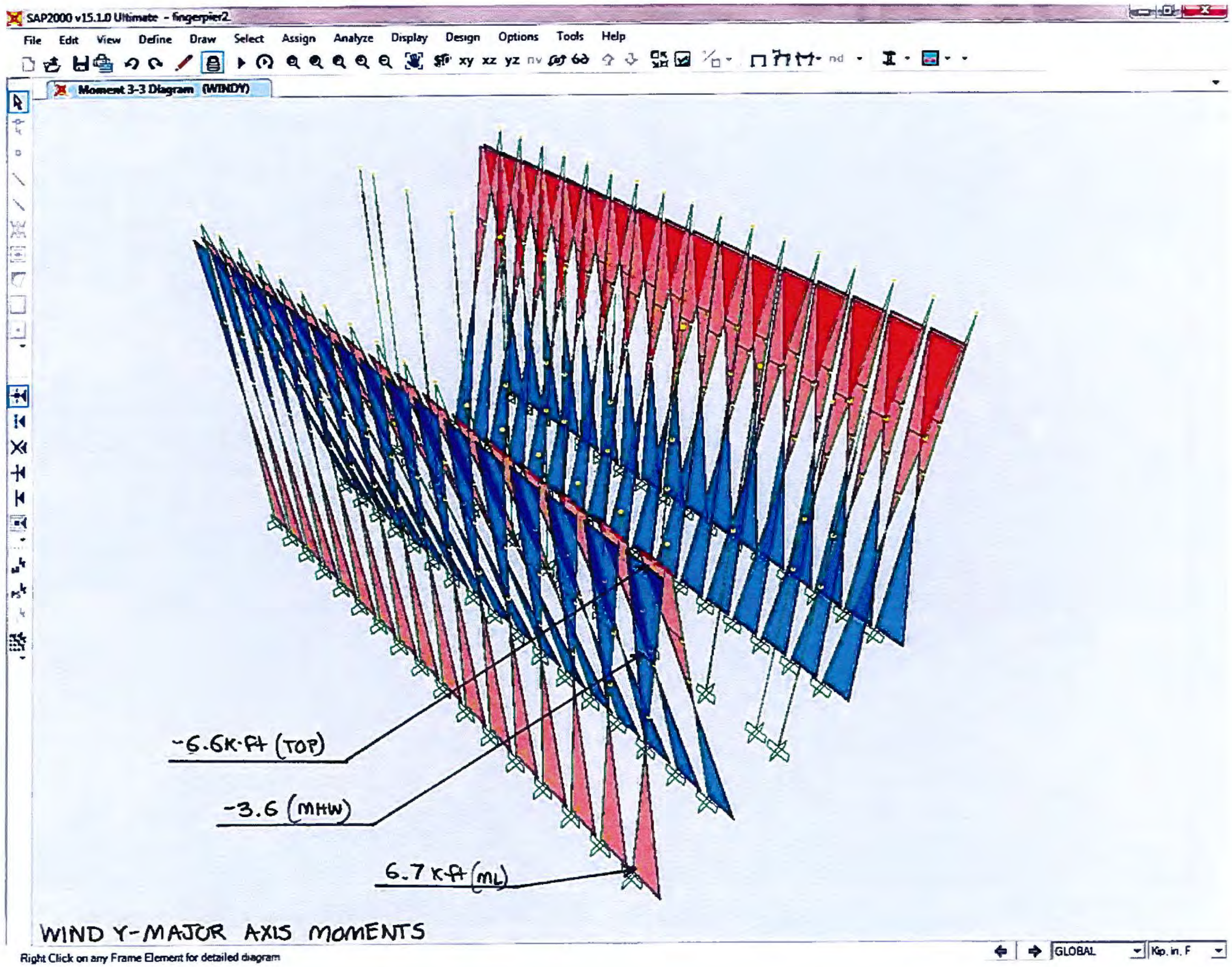
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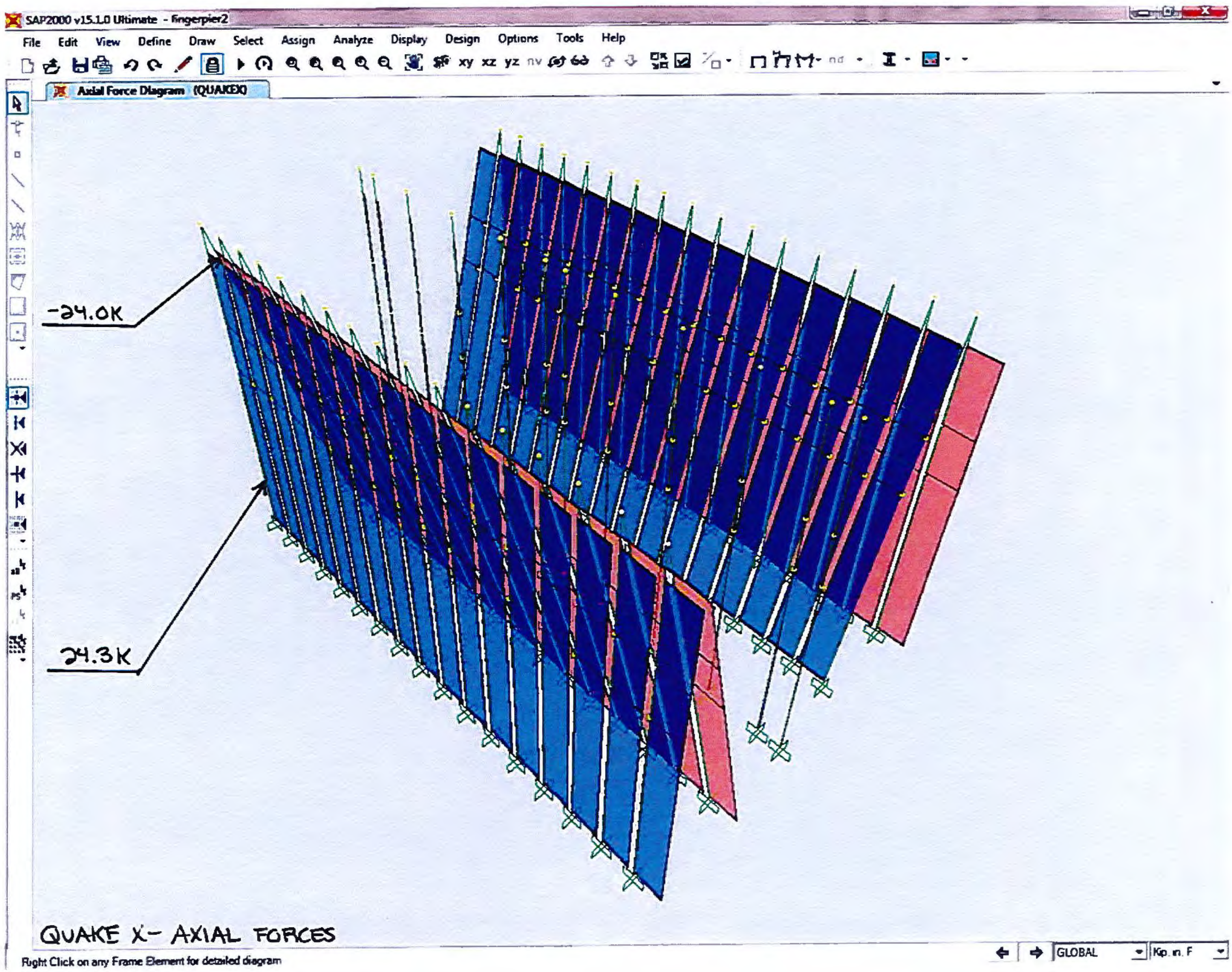
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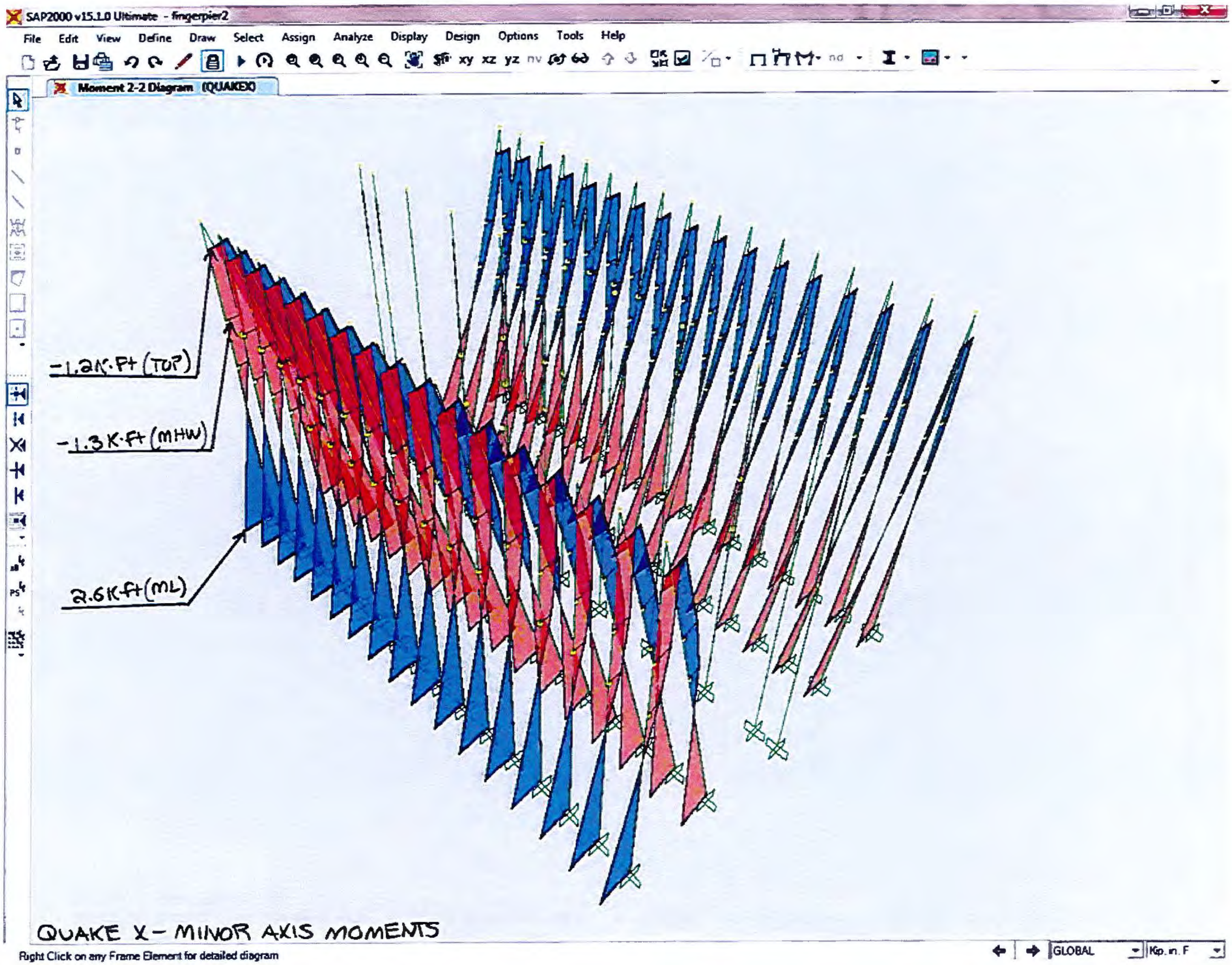
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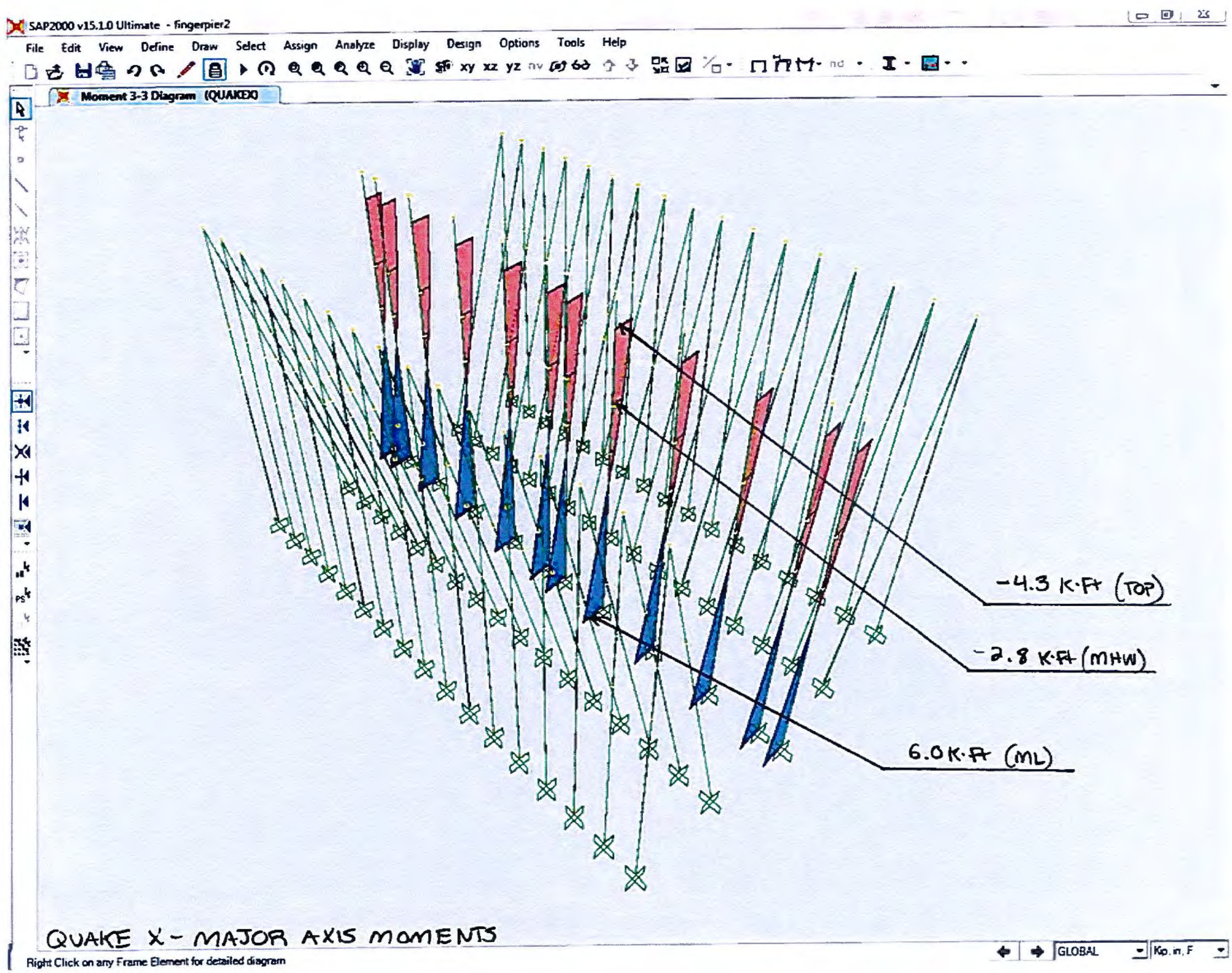
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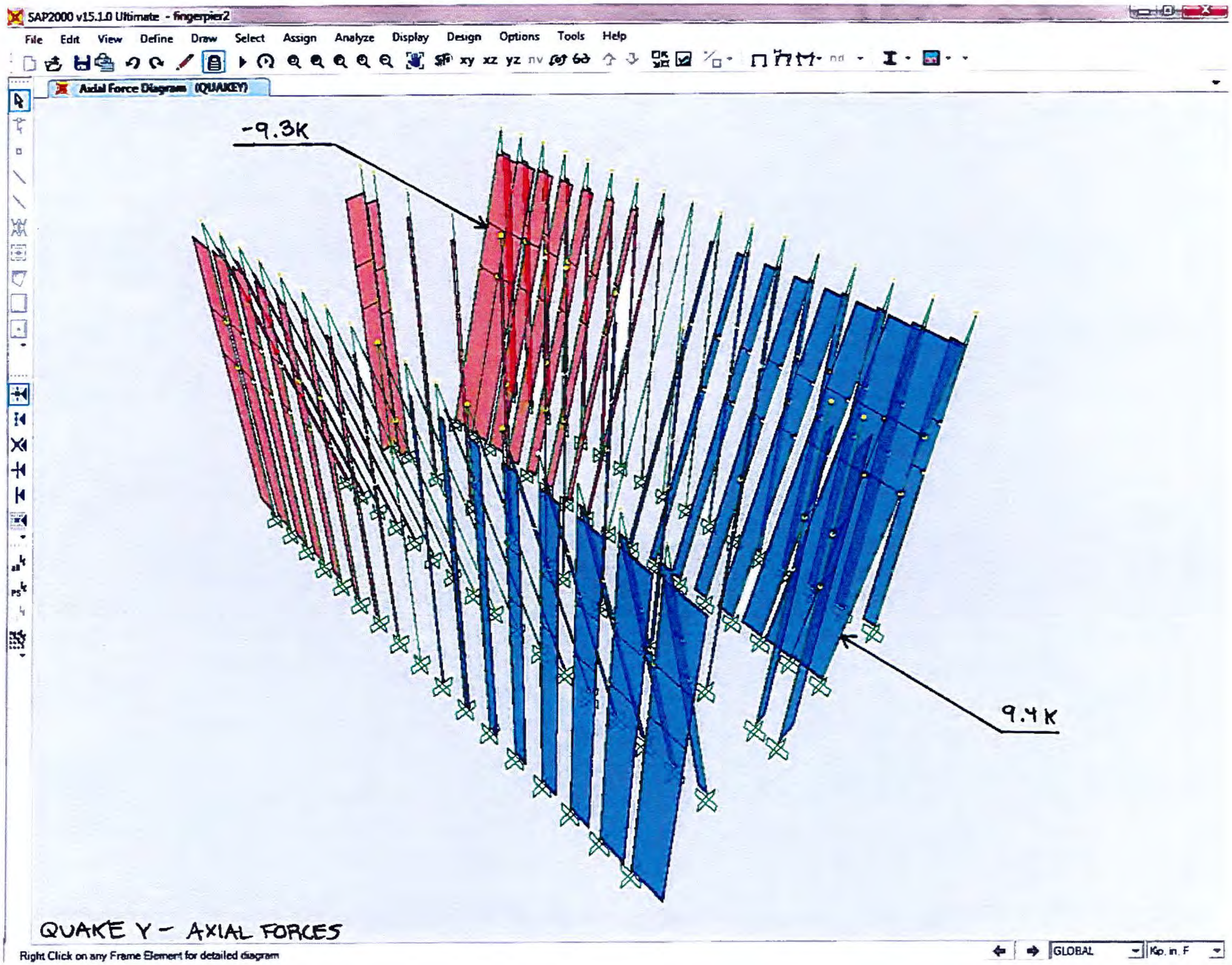
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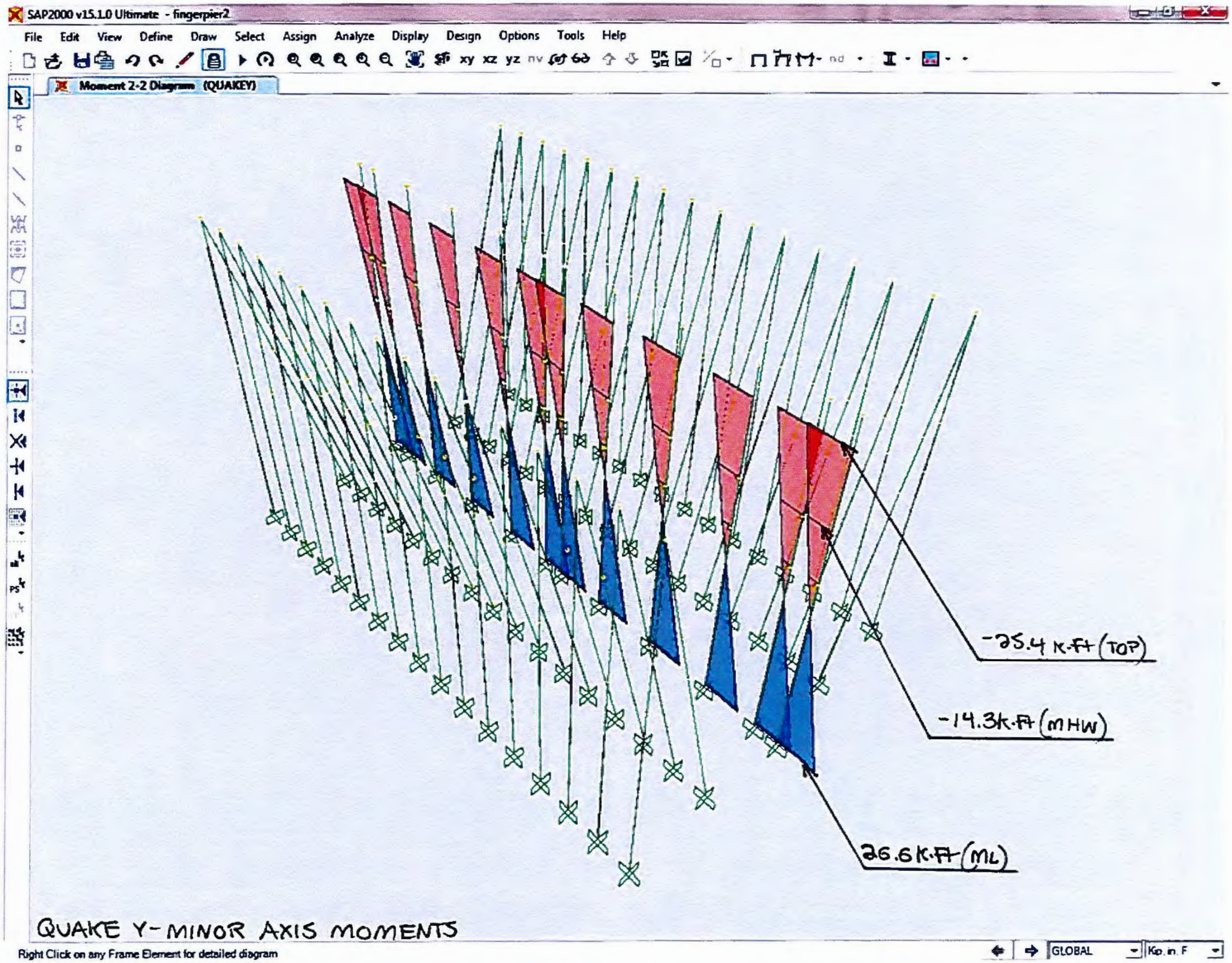

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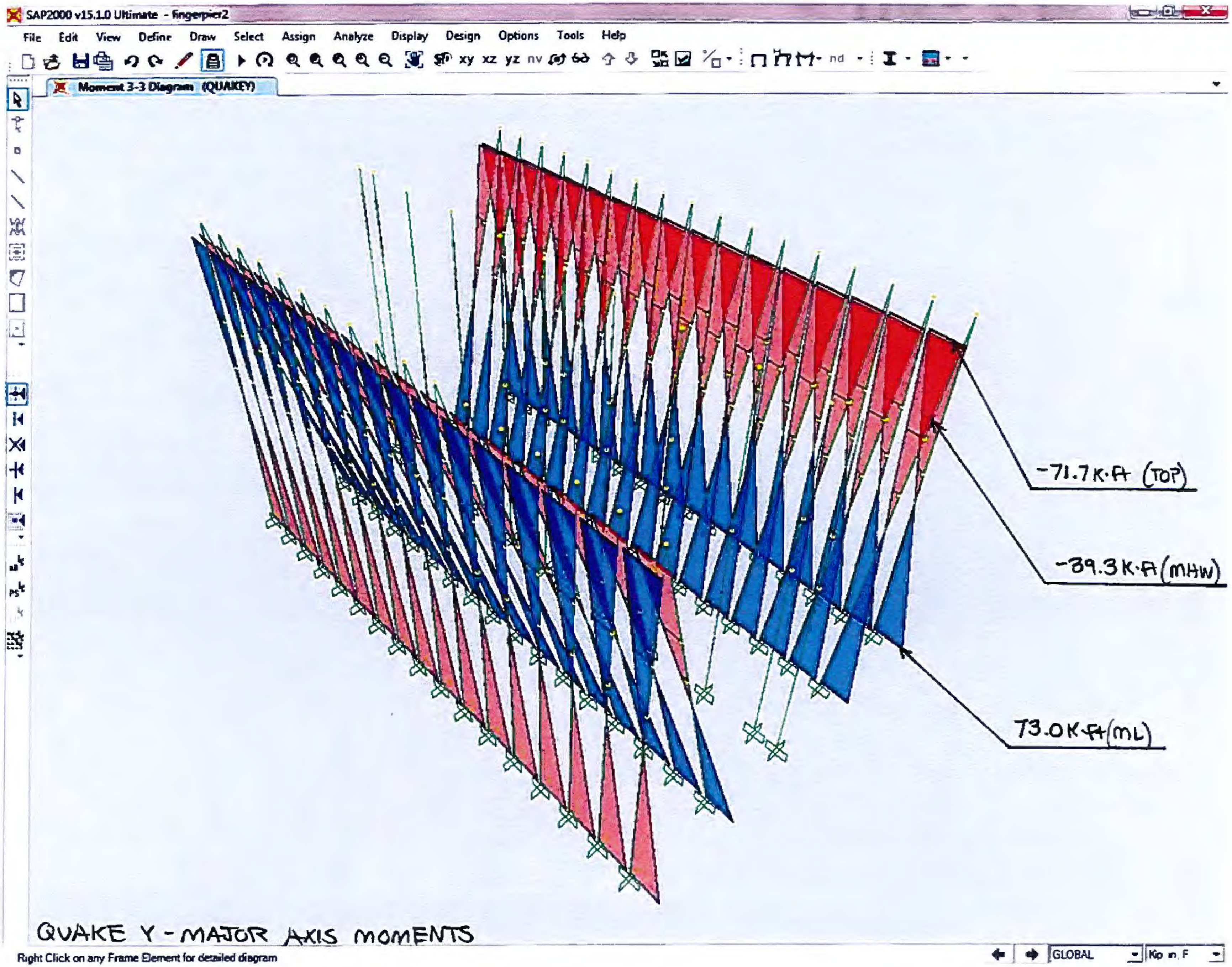
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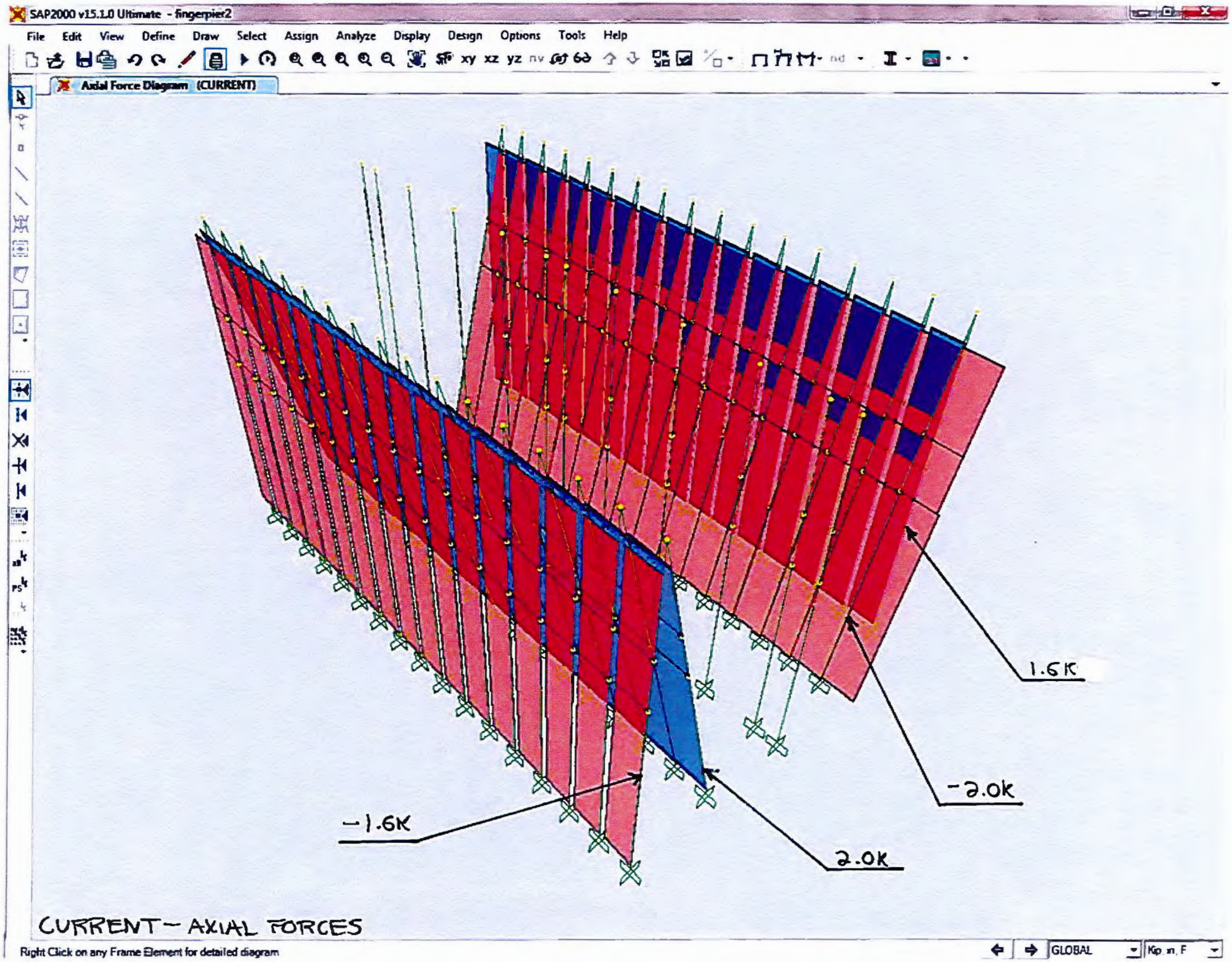

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CURRENT - AXIAL FORCES
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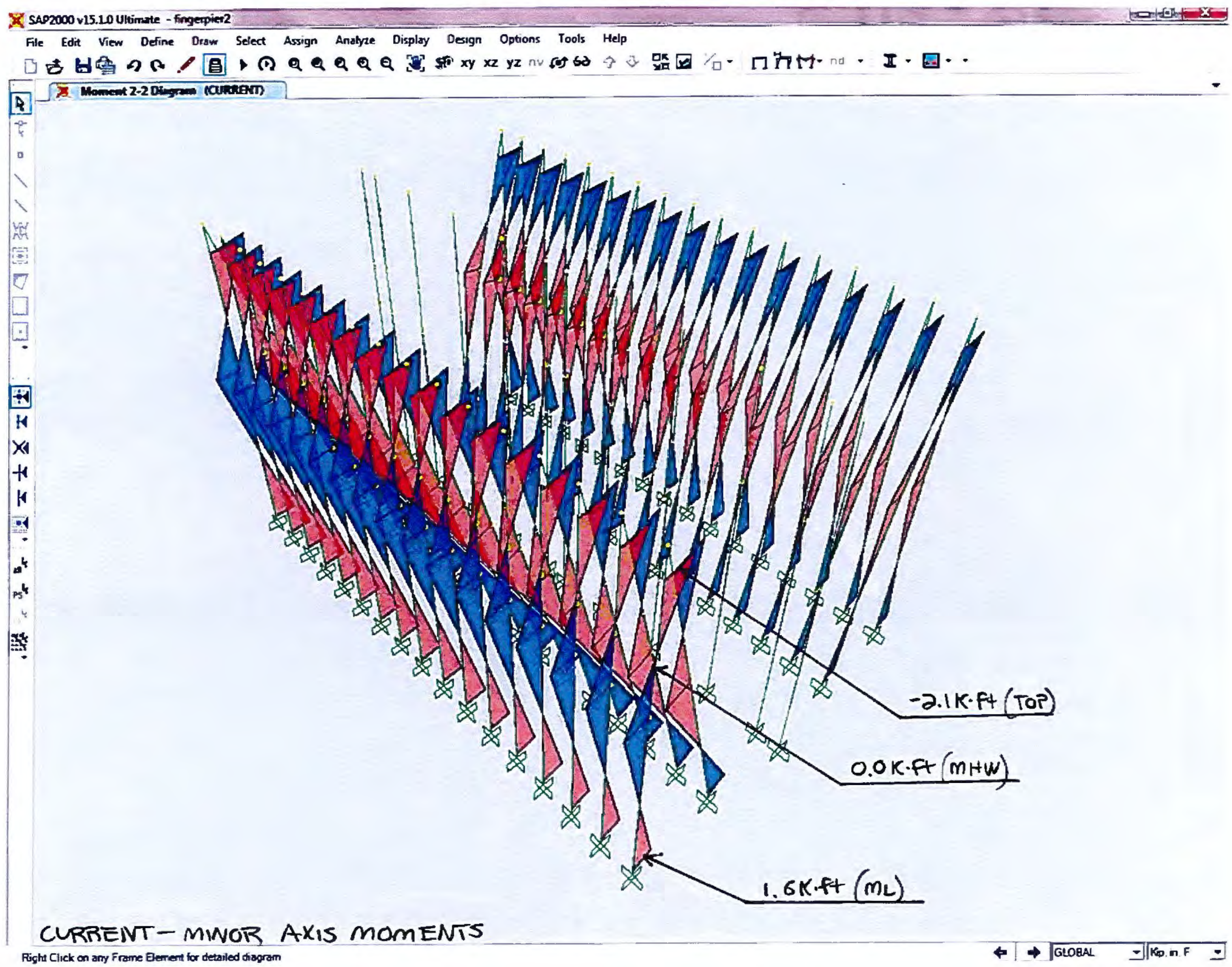
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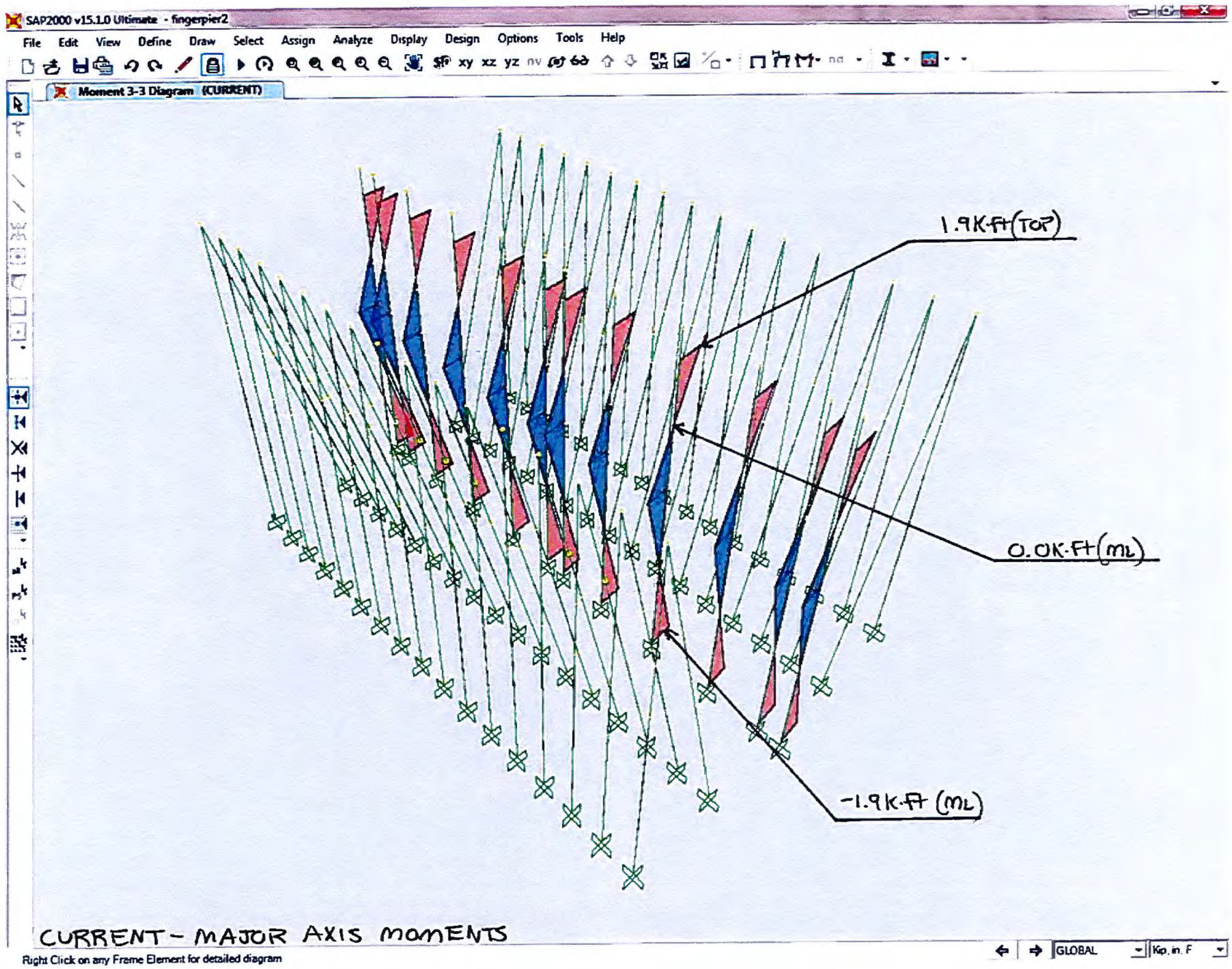
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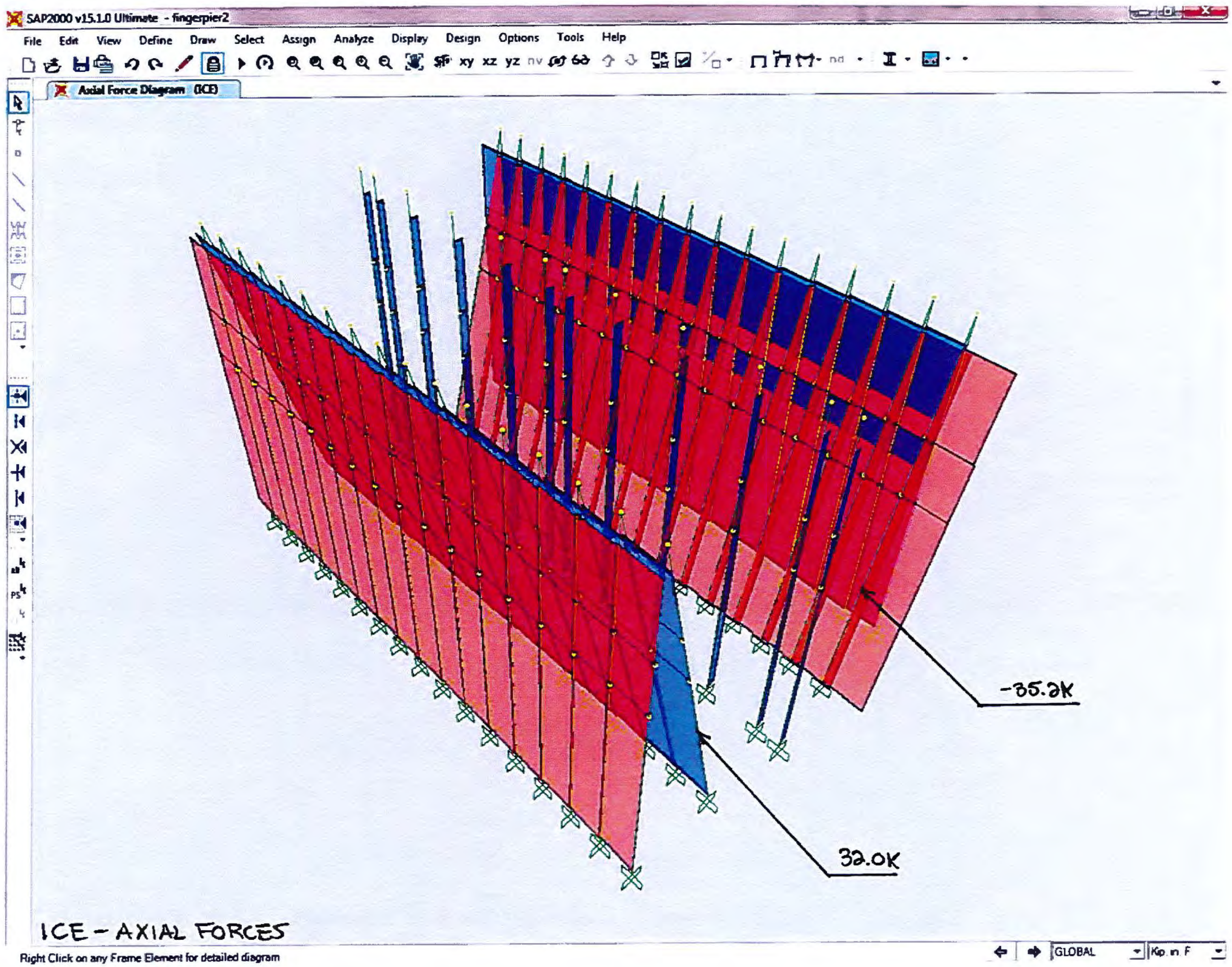
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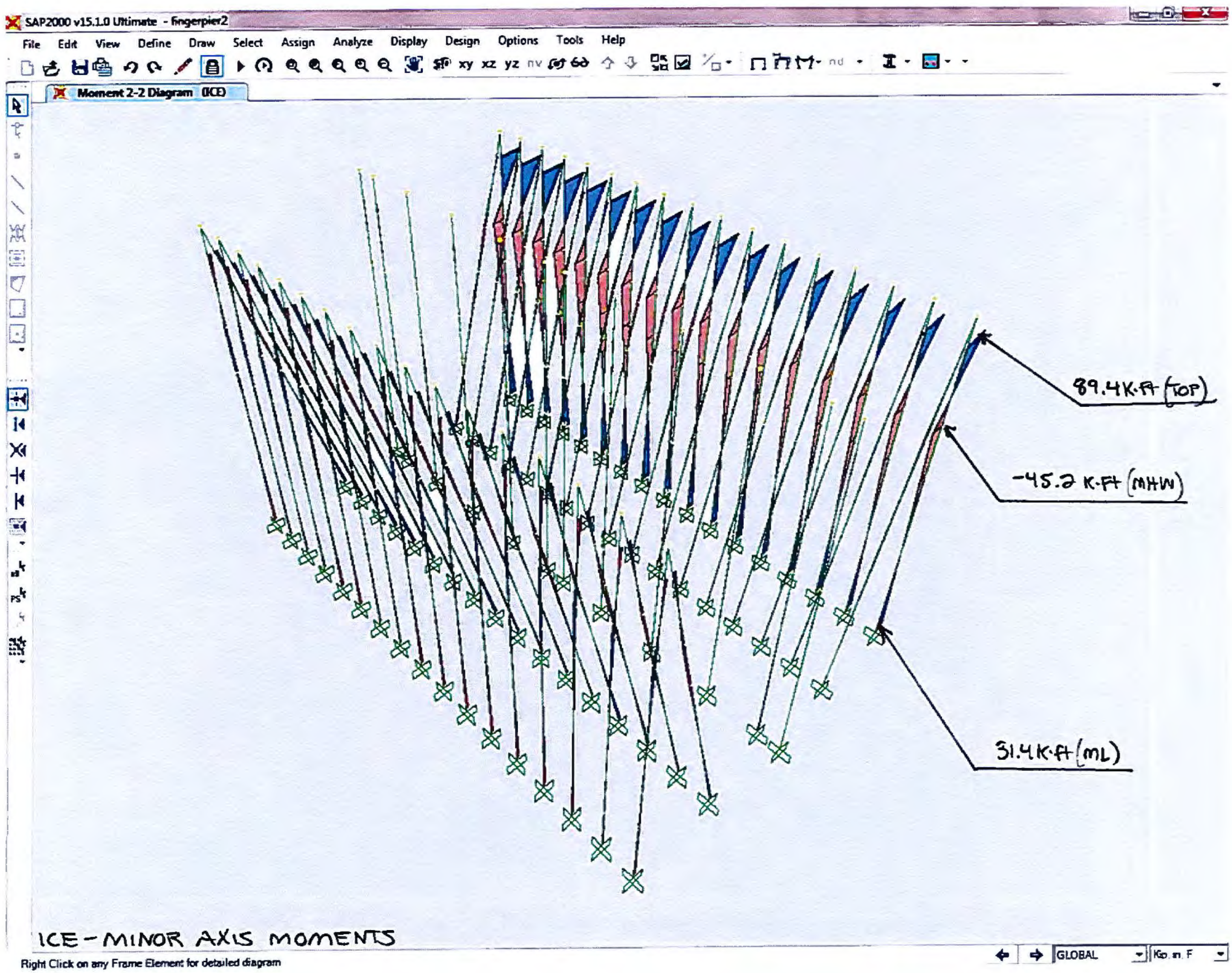
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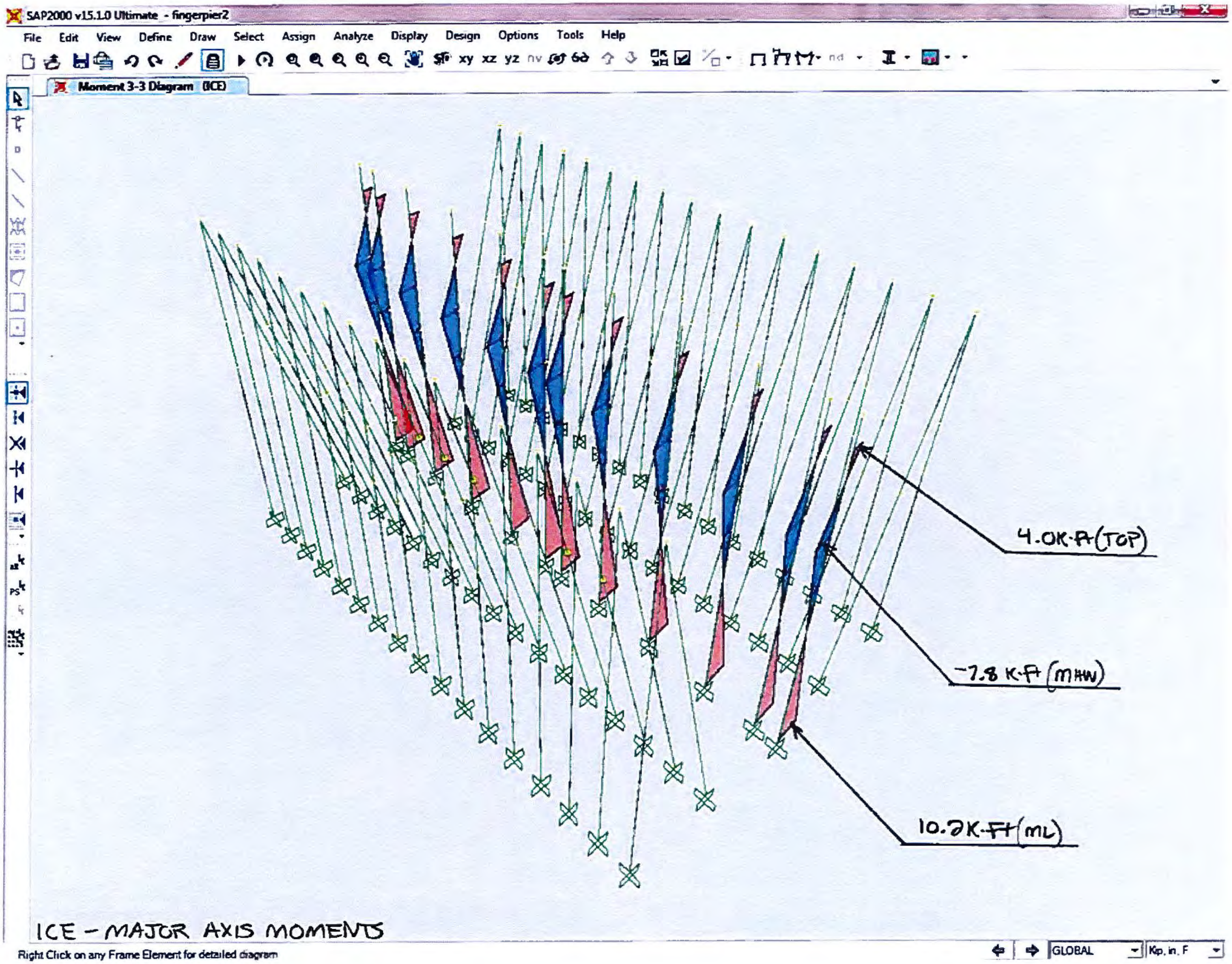
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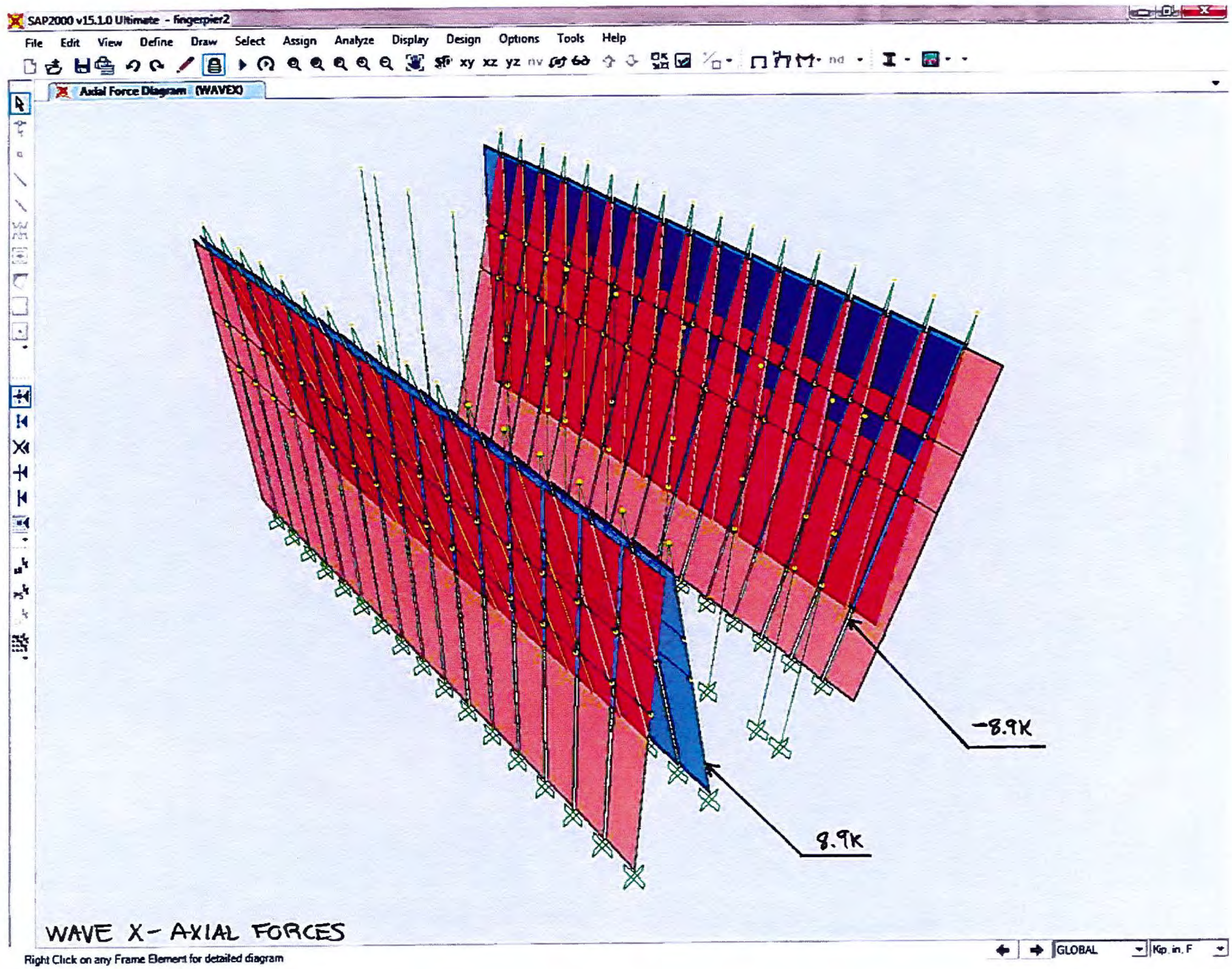

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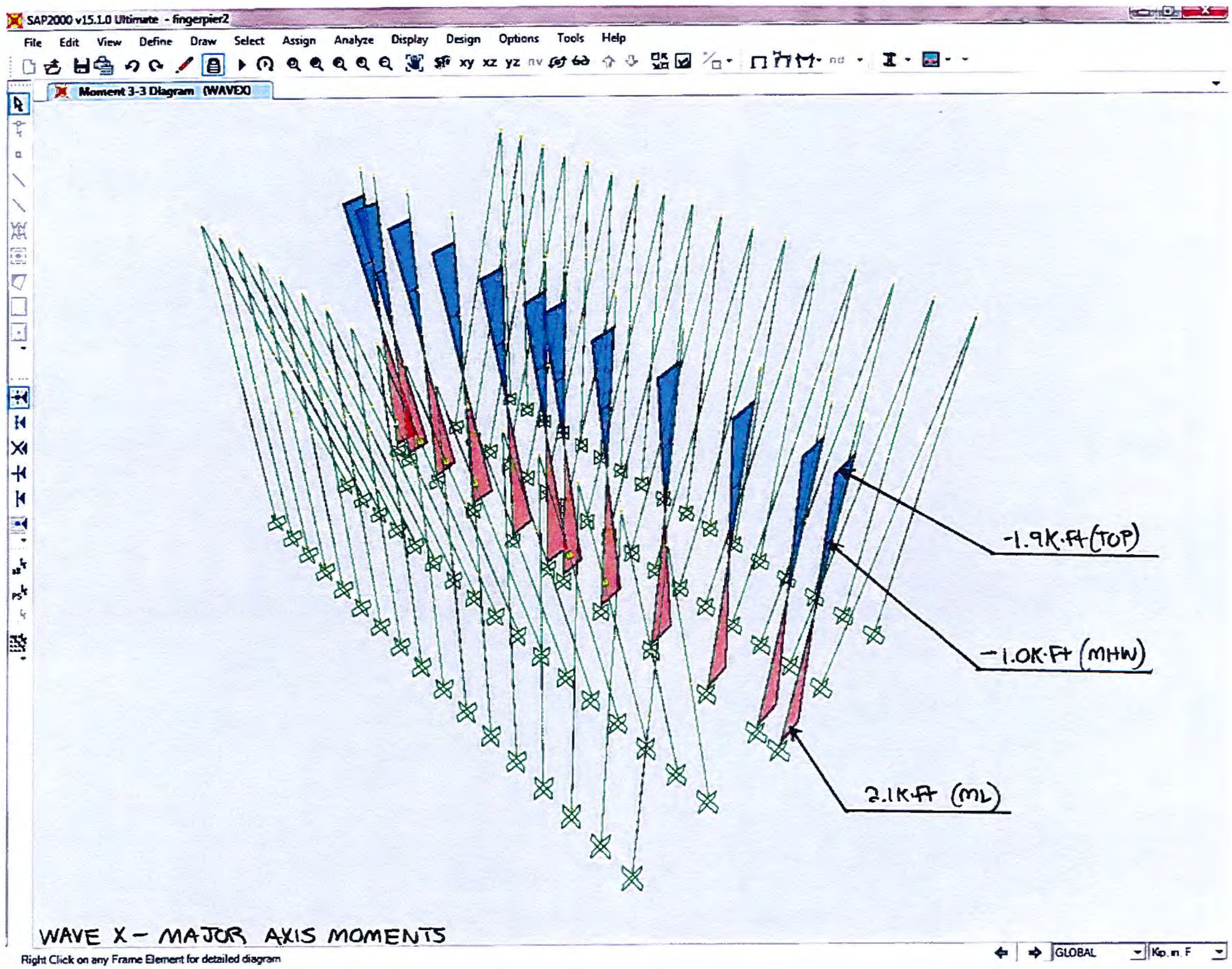

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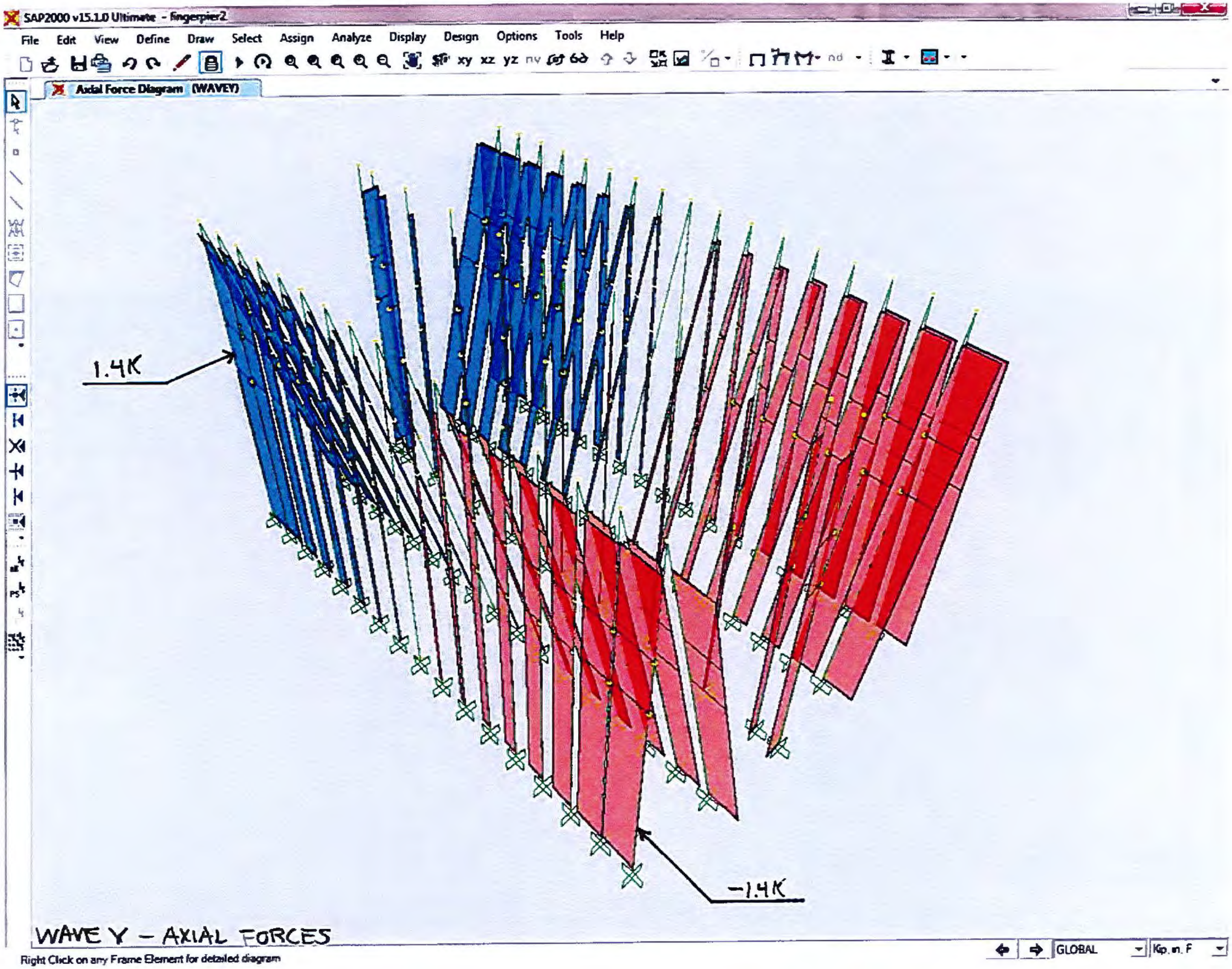
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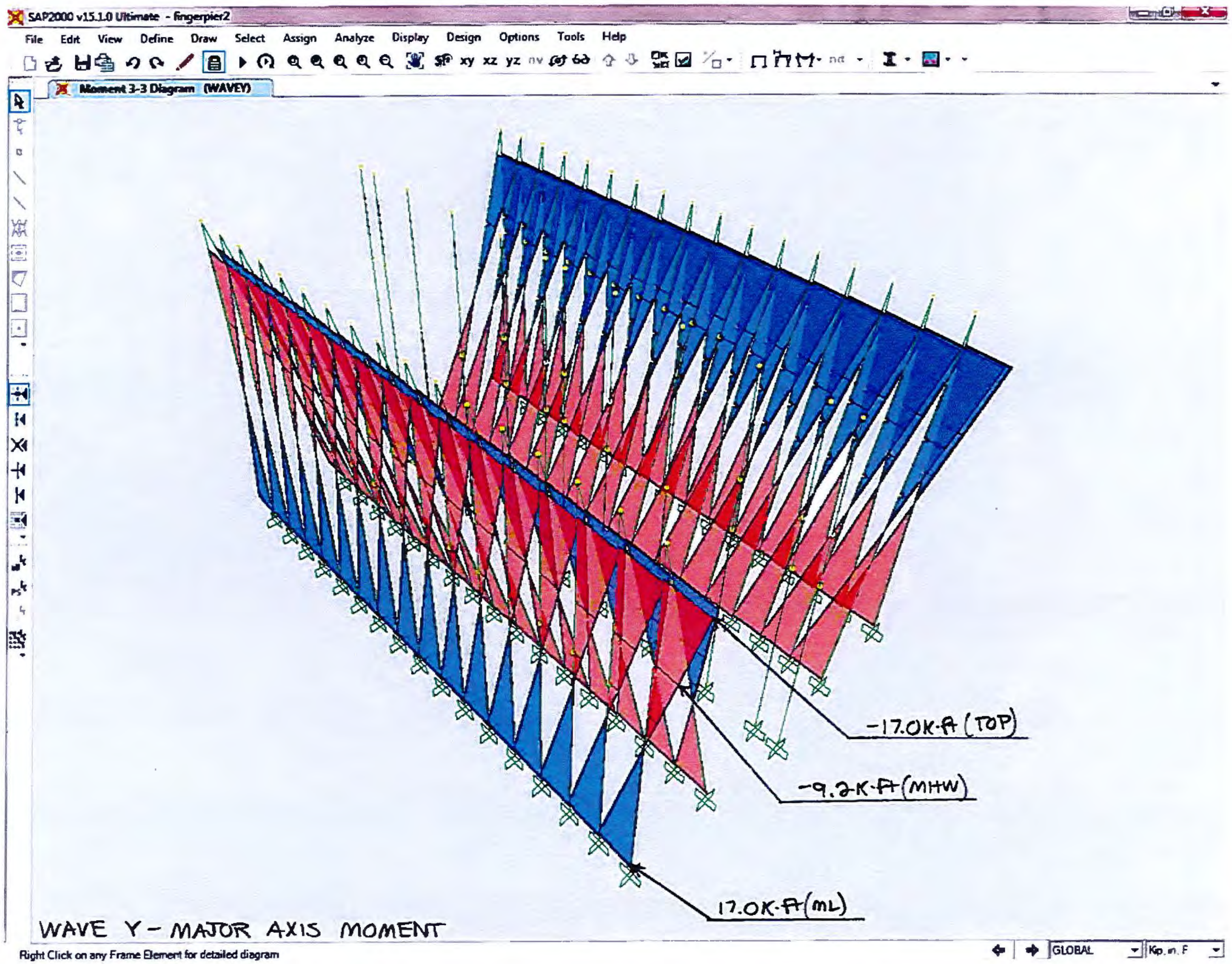
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17.0K-ft (ML)
WAVE Y - MAJOR AXIS MOMENT
Right Click on any Frame Element for detailed diagram
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APPENDIX E

HUDSON RIVER PARK TRUST
STRUCTURAL DESIGN GUIDELINES
(OCTOBER 2001)

Hudson River Park Trust

Hudson River Park

Structural Design
Guidelines

REV B

Hudson River Park Trust

Hudson River Park Trust

Hudson River Park

Structural Design Guidelines

October 2001

ARUP
155 Avenue of the Americas
New York, NY 10013

Job number 31853

ARUP

Document Verification

Job title	Hudson River Park	Job number	31853
Document title	Structural Design Guidelines	File reference	
Document ref			

Revision	Date	Filename			
Draft 1	6/15/01	Description	First draft		
			Prepared by	Checked by	Approved by
		Name	Cliff McMillan	Liam O'Hanlon	Cliff McMillan
		Signature			
Rev A	6/22/01		Strct_Des_Guilde_RevB.doc		
			Prepared by	Checked by	Approved by
			Cliff McMillan	Liam O'Hanlon	Liam O'Hanlon
		Signature			
Rev B	10/15/01	Filename	Strct_Des_Guilde_RevB.doc		
		Description			
			Prepared by	Checked by	Approved by
		Name	Cliff McMillan	Liam O'Hanlon	Cliff McMillan
		Signature			

Issue Document Verification with Document ☑

F8.5
QA Rev 1/00 1 October 2000

CONTENTS

1. INTRODUCTION

These guidelines have been prepared to establish structural design criteria and standards applicable to the whole of Hudson River Park, with the exception of Segment 4.

Design criteria for Segment 4 have been reviewed in compiling the Guidelines. In certain instances these guidelines differ from what has been applied in Segment 4.

2. STANDARDS AND CODES

2.1 General

The primary code applicable is the New York City Building Code (1999 Edition), except in specific areas where the Trust may direct that the latest New York State Building Code will apply.

2.1.1 Other Codes and Standards:

- NYC Fire Prevention Code
- NYC Local Laws
- Directives and Memoranda of the NYCBD
- Rules of the Board of Standards & Appeals
- NFPA where referenced by applicable codes
- The Americans With Disabilities Act (ADA)
- Occupational Safety and Health Administration (OSHA) Code of Federal Regulations - 29CFR 1919

2.2 Site Flood Conditions

Flood Hazard Areas are located on the Flood Insurance Rate Map contained in the NYCBC (RS 4-4). The design requirements for minimizing flood damage are given in Subchapter 4, Article 10 of the NYCDB and are referenced in FEMA 102/May 1986 Design Standards. Waterfront structures shall be designed to survive a 100 year flood and a Level 3 hurricane.

2.3 Structural

Design and construction shall be in accordance with the latest edition of the following (The applicable publication year is noted. Copies of these documents are kept in the Project Office for reference):

- New York City Building Code (1999 Edition), except in specific areas where the Trust may direct that the latest New York State Building Code will apply.
- Piers and marine structures shall be designed in accordance with the Department of the Army, Waterways Experiment Station, Corps of Engineers "Shore Protection Manual" (Volumes 1 and 2), 1984

- Pier Structures shall be designed for the most stringent of the above standards and also AASHTO - Standard Specifications for Highway Bridges (1996).
- Design of Coastal Revetments, Seawalls and Bulkheads. ACOE Publication EM 1110-2-1614, 1995
- British Standards Institute BS 6349.
- American Concrete Institute – Building Code Requirements for Reinforced Concrete. ACI 318 – 1999
- Precast Concrete Institute (PCI) - Recommended Practice for Design, Manufacture and Installation of Pre-stressed Concrete Piling; Prestressed Concrete Piling Interaction Diagrams.
- Structural Steel: AISC Steel Construction Manual (LRFD 2nd Edition or ASD 9th Edition)
- Stainless Steel: ASCE Standard – Specification for the Design of Cold-Formed Stainless Steel Structural Members – ANSI/ASCE-8-90.
- Aluminium: The Aluminium Association – Aluminium Design Manual (6th Edition, 1994) .
- Timber: National Design Specification for Wood Construction.
- Welding:
  - Steel: A.W.S. Structural Welding Code-Steel D1.1 - 1996.
  - Aluminium: A.W.S Structural Welding Code – Aluminium D1.2 - 1997
  - Stainless Steel: Welding of Stainless Steel –Nickel Development Institute (NIDI) - 1996
- Masonry: Building Code Requirements for Masonry Structure ACI-530 and ACI-531 for Concrete Masonry Structures.
- NAVFAC (Design Manual) DM-7.1 - Soil Mechanics.
- NAVFAC (Design Manual) DM-7.2 – Foundation and Soil Structures.
- ASCE Standard ANSIASCE-7-98. "Minimum Design Loads for Buildings and Other Structures".
- AWPA (American Wood-Preservers' Association) standards:

- C 1-00 - All Timber Products, Pressure Treatment

  C 2-00 -Lumber, Timbers, Bridge Ties and Mine Ties, Pressure Treatment

  C 18-99 -Material in Marine Construction, Pressure Treatment

  P 5-00 - Waterborne Preservatives

  P 13-95 - Creosote

  M 4-98 - Care of Pressure-Treated Wood Products

- ASTM Standards:
    - A 36-00 Specification for Carbon Structural Steel
    - A 153-00 Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware
    - A 185-97 Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement
    - A 193-01 Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service
    - A 194-01 Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure or High-Temperature Service, or Both
    - A 307-00 Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength
    - A 325-00 Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
    - A 416-99 Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
    - A 490-00 Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)
    - A 500-01 Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
    - A 572-00 Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
    - A 615-01 Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
    - A 653-00 Specification for Steel Sheet, Zinc-Coated (Galvanized) by the Hot -Dip Process
    - A 706-01 Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
    - A 775-01 Specification for Epoxy-Coated Reinforcing Steel Bars
    - A 884-01 Specification for Epoxy- Coated Steel Wire and Welded Wire Fabric for Reinforcement
    - A 924-99 Specification for General Requirements for Steel Sheet, Metallic-Coated by the Hot-Dip Process
    - A 992-00 Specification for Steel for Structural Shapes For Use in Building Framing
- B 221-00 Specification for Aluminum and Aluminum-Alloy Extruded Bars, Rods and Wire, Profiles and Tubes
- C 33-01 Specification for Concrete Aggregates
- C 62-01 Specification for Building Brick
- C 90-01 Specification for Loadbearing Concrete Masonry Units

- C 150-00 Specification for Portland Cement
- C 260-00 Specification for Air-Entraining Admixtures for Concrete
- C 476-01 Specification for Grout for Masonry
- C 857-95 Practice for Minimum Structural Design Loading for Underground Precast Utility Structures
- C 902-01 Specification for Pedestrian and Light Traffic Paving Brick
- D 25-99 Specification for Round Timber Piles
- D 1751-99 Specification for Preformed Expansion Joint Filler for Concrete Paving and Structural Construction
- D 1760-99 Specification for Pressure Treatment of Timber Products
- D 6109-97 Standard Test Methods for Flexural Properties of Unreinforced and Reinforced Plastic Lumber

2.4 References

- Eldridge "Tide and Pilot Book" 1996
- American Petroleum Institute. "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Working Stress Design"
- "Design of Highway Bridges" – R. M Barker and J.A. Puckett, 1997
- Tidal Data published by NOAA, date 10/28/85, for period 1960 – 1978
- " Breakwaters, Jetties, Bulkheads & Seawalls" – Pile Buck Inc. - 1992
- Naval Facilities Command, Military Handbooks:
  - Piers and Wharves MIL-HDBK-1025/1
  - Harbors DM 26.1
  - Coastal Protection DM 26.2

3. MATERIALS

The material properties to be used for design shall be as follows:

3.1 Reinforced Concrete

Cast-in-place Concrete and Non-Prestressed Precast Concrete:

- For all piers and structures subject to marine environment, minimum fc'= 5000 psi, normal weight concrete with Type II or Type II A cement to conform to ASTM C150. Water cement ratio (by weight) shall not be greater than 0.4. All exposed concrete shall be air-entrained and comply with ASTM C 260, with 5% to 8% air content. Admixtures containing chloride ions are prohibited. Aggregates shall conform to ASTM C33. Concrete shall be mixed and transported in accordance with ACI 318, 304, 301.

- High quality concrete with adequate thickness and cover shall be specified to provide 50-year life in marine environment. In chloride environment, consideration should be given to cement with at least 25% fly ash or 65% blast furnace slag.
- Reinforcing bars shall conform to ASTM A 615 Grade 60 Fy = 60 ksi. Consideration should be given to epoxy coating to comply with ASTM A 775. Protect coating from damage in accordance with the Specification. All damaged areas of epoxy coating shall be re-coated and cured to the satisfaction of the Owner's Representative prior to concrete encasement. Welding of ASTM A 615 reinforcing bars is prohibited.
- Reinforcing to be welded shall be approved by the Owner's Representative and conform to ASTM A 706. Welding shall conform to the requirements of AWS D1.4 – Structural Welding Code for Reinforcing Steel. Welding electrodes for reinforcing steel shall be E90XX unless otherwise noted.
- Welded wire fabric shall confonn to ASTM A 185 and shall have an ultimate tensile strength of 70 ksi, epoxy coated to comply with ASTM A 884.
- All finished surfaces shall be smooth troweled, unless otherwise noted on the Design Drawings.
- Pouring new concrete against existing concrete: clean and roughen existing concrete surface, just prior to concrete pouring, coat existing concrete with Sikadur 32 high- mod bonding agent or approved equivalent applied in accordance with manufacturer's instructions.
- Premolded expansion joint filler for concrete shall be bituminous type and shall be preformed in accordance with ASTM D1751.

3.2 Precast Prestressed Concrete:

- Minimum fc'= 6000 psi.
- Tendons shall be low relaxation 7-wire strand conform to ASTM A 416, Grade 270.
- 4 Tendon release shall not take place until the concrete strength exceeds 4000 psi.
- All surfaces shall be smooth finish, corners shall not be chamfered unless otherwise noted.
- Submit calculations to the owner's representative for review prior to ordering.
- The location and detail of lifting devices for the precast prestressed member shall be determined by the contractor, considering all loads imparted during transporting, handling and installation. All lifting devices shall be removed and concrete patched with epoxy grout as required after installation.

3.3 Structural Steel

- Shapes and Bars shall be ASTM A 572 and A 992 Grade 50 with Fy= 50 ksi and Fu=65 ksi, unless otherwise noted. Plates shall be ASTM A 36, with Fy=36 ksi and Fu=58ksi, unless otherwise noted.
- Pipes shall be ASTM A 500 type B, with Fy=42 ksi and Fu=58 ksi.
- Tubes shall be ASTM A 500 type B, with Fy=46 ksi and Fu=58 ksi.

- Bolts shall be High Strength Bolts ASTM A 325 or ASTM A 490 (for buildings only) or ASTM A 193 with hardened nuts and wasbers, galvanized in accordance with ASTM A153.
- Anchor bolts sball be ASTM A 307 or ASTM A193 or, for buildings, ASTM A 325 or ASTM A 490. Threaded fasteners shall be ASTM A 36, ASTM A193 or ASTM A 572 Grade 50.
- Welding electrodes shall be E70XX with the exception of welding to existing steel, where investigation sball be directed to determine the type of the existing base metals, its weldability and the filler metals to he used. Field welding is not permitted unless otherwise noted or unless approved in advance by owner's representative.
- Metal deck shall conform to ASTM A 653, Fy=33 ksi. Minimum 18 gage, hot-dip galvanized with minimum G115 for severe exposure conditions.
- All structural steel, bolts, hardware, etc., shall be galvanized in accordance with ASTM A 153. Chase all threads after galvanizing.

3.4 Treated Timber Fender Piles

- All treated timber piles shall be 12" minimum diameter, measured 3 ft from butts. Minimum compressive strength F'c = 1000 psi
- Timber piles shall be either Douglas Fir Larch or Southern Pine conforming to AWPA C1, C2 and C18 and sball be pressure impregnated with a preservative in accordance with AWPA P5, P13 and M4 and AASHTO M133.
- Mechanical fasteners in wood shall be hot-dip galvanized in accordance witb ASTM A 153.

3.5 Masonry

- Concrete block for load bearing masonry construction sball be Type 1 moisture controlled units (Grade N-I) that meets the requirements of ASTM C 90, minimum f 'm = 1200 psi.
- Brick for masonry construction shall be Grade SW and conform to requirements of ASTM C 62.
- Brick for use as paving material to support pedestrian and ligbt vehicular traffic shall be Class SX Type I and conform to requirements of ASTM C 902.
- Reinforcing steel used in the masonry construction shall conform to ASTM A 615 Grade 60.
- Mortar: the ingredients used in making mortar shall conform to the mortar specified in AASHTO Articles 14.2.3 and 14.4.2.
- Grout for filling voids in hollow masonry units shall conform to ASTM C 476, or the requirements of AASHTO Section 8 or Section 14.2.3. Admixture shall he used only when specified or approved by tbe owner's representative.

3.6 Aluminium

Aluminium shall be Alloy 6061-T6 in accordance with ASTM B221. Properties and tension tests are required.

3.7 Structural Timber

- Structural timber design and construction shall comply with the American Institute of Timber Construction Manual, second edition.
- All structural timber to be either douglas fir-larch or southern pine and shall be visually graded by authorized agency in accordance with ASTM D 245 and bears the official grade mark. Design values for the graded lumber sball comply with the applicable provisions of "National Design Specification for Wood Construction" by the American Forest and Paper Association (AF & PA).
- All structural timber shall be pressure treated with preservative materials and solutions in accordance with AWPA C1, C2 and C18 and a preservative in accordance with AWPA P5, P13 and M4 and AASHTO M133.
- Timber chocks and wales, diagonal bracing and low water bracing will be douglas fir-larch or southern yellow pine, Commercial Grade No. 2, treated per AWPA C2 with preservative treatment by pressure process (salt water use) with waterborne preservatives. Minimum retention of Chromated Copper Arsenate (CCA) 2.5 lb/cf. Commercial No. 2 timber will have the following minimum characteristics: Boards: Extreme fiber bending Fb = 1,200 psi Modulus of elasticity E = 1,600 ksi. Hardware will be ASTM A307, galvanized per ASTM A153. All bolts will include ogee washers and hex nuts. Hardware connections on outboard faces of fender piles and on chocks will be countersunk.

3.8 Timber Bearing Piles & Timber Pile Caps

- Timber bearing piles and timber pile caps will be Demarara Greenheart. The minimum diameter of these piles shall be 12", when measured 3 feet from the butt. Pile cap beams will be 12"x 12"(rough).
- Greenheart will have the following minimum characteristics:

Air- dried	
Modulus of Elasticity	3,100 ksi
Modulus of Rupture	17,900 psi
Bending Stress (allow)	3,800 psi
Compression Parallel to Grain (allow)	3,000 psi
Shear Parallel to Grain	380 psi

3.9 Recycled Plastic

- Subject to approval by HRPT and Department of Building Services Waterfront Permits Unit, materials for structural joist, girders, columns, and fender piles may be based on the use of acceptable quality recycled plastic lumber shapes which comply with ASTM D6109-97 Flexural Secant modulus @ 1% strain > 350,000 psi; and flexural stress @ 3% strain > 2,500 psi.
- Materials for decking shall be based on the use of acceptable quality recycled plastic lumber shapes which comply with ASTM D6109-97 having a minimum flexural secant modulus @ 1% strain 70,000 psi, and a flexural stress at 3% strain > 1,500 psi.
- Exposed surfaces of plastic lumher shall be non-skid.

- All fasteners for decking shall be stainless steel #10 square drive countersunk decking screws.

3.10 Backfill

Backfill or fill material shall conform with the requirements of the Project Specifications.

Design of structures and paving supported on such fill shall be in accordance with AASHTO (1996).

4. LOADS

4.1 Vertical loads

- Self-weight of structure.
- No increase in the historic load-bearing capacities of piers is permitted. Loads for which existing piers have been designed are given in Appendix A.
- New and re-built piers shall be designed for the superimposed dead loads applicable at the pier and for a live load of 100 pounds per square foot in public areas. To provide for future use adaptability, piers should generally be designed for a superimposed dead plus live load equal to 350 pounds per square foot, but not exceeding the historic loads. Design loads must be clearly stated on the drawings.
- Maximum vehicle loads will generally be H20 as defined in accordance with AASHTO, with 15% impact. Vehicles will be excluded from grating areas, unless otherwise noted.
- In locations directed by the New York City Planning Bureau, a fire truck must be accommodated, witb allowance of up to 30% overstress, with the following maximum loads and dimensions:

Total Weight	68 000 pounds
Length between axles	37' 9"
Rear Axle Weight	48,000 pounds
Vehicle Width	8' 0"

- Minimum live loads and dead loads to be applied when designing underground monolithic or sectional precast concrete utility structures shall conform to ASTM C 857.

4.2 Wind

Basic Wind Speed 110 MPH, Exposure C. Wind pressure shall be in accordance with ANSI/ASCE 7-98: Minimum Design Loads for Buildings and Other Structures, with an importance factor = 1.00.

4.3 Seismic Acceleration Coefficient A = 0.15

- Response Modification Factor R shall be in accordance with AASHTO Division I-A - Seismic Design Section 3.7.
- Site Coefficient S shall be in accordance with NYC Building Code section 27- 675 and its Reference Standard RS-9-6.

4.4 Snow

Ground Snow Loading (Pf) = 30 psf (drift and partial snow loads to be considered, where applicable).

Combination of snow load and snow clearing vehicle to be considered where applicable.

4.5 Temperature: (AASHTO)

Steel structures:	0 degrees to 120 degrees Fahrenheit.
Concrete structures:	Temperature Rise: 30 degrees Fahrenheit.
	Temperature Fall: 40 degrees Fahrenheit

4.6 Soil Pressure

Earth pressures shall be determined in accordance with AASHTO (1996), Division 1, Section 3.20.

4.7 Lateral loads on curbs, guardrail and vehicle barriers

Curbs, guardrails and vehicle barriers shall comply with AASHTO (1996) requirements. Railings should be designed for loads specified in the NYC Building Code.

4.8 Piers and Marine Structures

Piers and marine structures shall also be designed in accordance with the recommended procedures for such structures and for the following conditions:

4.8.1 Wind

Wind conditions defined in 4.2 shall apply.

4.8.2 Current

Design for actual anticipated current conditions at the particular pier, with a minimum current velocity of 1.5 knots (2.5 ft/sec)

4.8.3 Wave Conditions

- Average Wave Height H = 3.0 feet
- Average Wave Length L = 65 feet
- Average Wave Period T = 3.5 seconds

- Extreme Conditions

Southerly

- Fetch = 6.5 nautical miles
- Significant Wave Height = 10.0 feet
- Wave Period = 5.4 sec
- Wavelength = 218 feet

Westerly

- Fetch = 0.75 nautical miles
- Significant Wave Height = 3.4 feet
- Wave Period = 2.6 sec
- Wavelength = 106 feet

4.8.4 Ice

Ice Thickness: 8 inches

Design shall be for both static (crushing at structure sides) and dynamic (flowing) conditions and shall be in accordance with Design of Highway Bridges, Barker and Puckett, 1997, pp. 180-190

Ice Abrasion (side) Fa =0.11 * F

4.8.5 Vessel Loads

Berthing

Piers and bulkheads at which vessels will berth shall be designed for berthing loads to be determined based on BS 6349 – Part 4, using an approach velocity of 1.5 to 2.0 knots and an angle of approach of 15 and 10 degrees respectively, as measured from the longitudinal axis of the pier. The vessel size to be designed for shall be determined from the actual conditions applicable to the pier.

Mooring Loads

Mooring loads will be determined based on wind, wave and current loads. Winds will be applied from all possible directions. The analyses will include the elastic properties of the selected fender and the elasticity of the mooring lines.

4.8.6 Loading Combinations

The specified loads shall be considered in various combinations in accordance with the following table from the Military Handbook, MIL-HDBK-1025/1, Section3.4, Load Combinations:

Service Load Design

Load Combination:	S1	S2	S3	S4	S5	S6	S7	S8	S9

Service Load Design

Dead* (including superimposed dead load)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Live(Concentrated)+Impact or Live(Uniform)	1.0	0.1	1.0	1.0		1.0	**	1.0	
Buoyancy	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Vessel Berthing		1.0							
Current			1.0	1.0	1.0	1.0			1.0
Earth Pressure	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Earthquake							1.0		
Wind on Structure			0.3		1.0	0.3			1.0
Wind on Vessel			0.3		1.0	0.3			
Creep/Rib Shortening+Shrinkage+ Temperature				1.0	1.0	1.0			
Ice + Minimum Temperature								1.0	1.0
% Allowable Stress	100	100	125	125	140	140	133	140	150

Load Factor Design

Load Combination:	U1	U2	U3	U4	U5	U6	U7	U8	U9
Dead* (including superimposed dead load)	1.3	1.3	1.3	1.3	1.25	1.25	1.3	1.3	1.2
Live(Concentrated)+Impact or Live(Uniform)	1.7	0.17	1.3	1.3		1.25	**	1.3	
Buoyancy	1.3	1.3	1.3	1.3	1.25	1.25	1.3	1.3	1.2
Vessel Berthing		1.7							
Current			1.3	1.3	1.25	1.25			1.2
Earth Pressure	1.3	1.3	1.3	1.3	1.25	1.25	1.3	1.3	1.2
Earthquake							1.3		
Wind on Structure			0.3		1.25	0.3			1.2
Wind on Vessel			0.3		1.25	0.3			
Creep/Rib Shortening+Shrinkage+ Temperature				1.3	1.25	1.25			

Load Factor Design

Ice + Minimum Temperature	1.3	1.2

* 0.90 of dead load only (excluding superimposed dead load) for checking members for minimum axial load and maximum moment.

** 0.0, 0.10, or 0.20, depending on the live load assumed to be acting on pier for earthquake

Load calculations. See Earthquake Loads, paragraph 3.3.4, MIL-HDBK-1025/1.

5. SERVICEABILITY CRITERIA

5.1 Tidal Effects

Piers and bulkheads, including elevations, get downs, gangways, etc shall be designed to accommodate the applicable tidal data.

The following tidal data is published by NOAA (10/28/85) for Battery Park City (elevations in feet, referenced to Borough of Manhattan Highway Datum). For other locations designers should refer to the actual published data (See Reference in 2.4).

Tidal Data: Battery Park City

	Borough President of Manhattan Highway Datum	National Geodetic Vertical Datum of 1929 (NGVD'29)	MLLW Datum
100 yr. Flood, Level 3 Hurricane	+7.25	+10.00	+11.88
Highest Observed (9/12/60)	+5.60	+8.35	+10.23
Mean High High Water	+0.49	+3.24	+5.12
Mean High Water	+0.15	+2.90	+4.78
Manhattan Highway Datum	0.00	+2.75	+4.63
NGVD of 1929	-2.75	0.00	+1.88
Mean Low Water	-4.41	-1.66	+0.22
Mean Low Low Water	-4.63	-1.88	0.00
Lowest Observed (2/2/76)	-8.70	-5.95	-4.07

Northeaster of December 11, 1992 reached +7.68 above NGVD.

All elevations are in feet.

Elevations on the plans refer to the Borough of Manhattan Highway Datum.

The following design water levels shall apply, based on Manhattan Borough Datum:

- Extreme High Water +5.60 feet
- Mean High Water +0.15 feet
- Mean Low Water -4.41 feet
- Extreme Low Water -8.71 feet

5.2 Durability

All structures and their materials shall be designed for a 50-year service life, during which minimal maintenance or inspection will be required under normal use.

5.3 Deflection Criteria

Deflection of pier structures shall be limited as follows:

$\Delta_{cdl} + \Delta_{sdl} + \Delta_{ll}$ - Camber $< L/180$

Δ_{ll} $< L/240$ or 1", whichever is smaller.

where;

Δ_{cdl} = Construction dead load deflection,

Δ_{sdl} = Superimposed dead load deflection

Δ_{ll} = Live load deflection

L = Beam span. For cantilevers use twice the cantilever length.

The lateral deflection of new structures shall be limited as follows:

Δ_{Wind} $< h/500$

$\Delta_{Seismic}$ $< h/250$

h = height of pier (calculated from the theoretical point of fixity).

APPENDIX A

PIER LOADS

A1. PIER LOADS

PIER	ORIGINAL DESIGN LOAD PER ACOE (1) (psf)	CURRENT LOAD CAPACITY PER HRPT (2) (psf)	COMMENT BASED ON MASTER PLAN	
25		468	Pier to be reconstructed	
26		483	Pier to be reconstructed	
32		464	New get down + ecological pier	
40		508	Future redevelopment	
42			Pier to be reconstructed	One pier has been deleted - refer to Segment 4 drgs
45		468	Pier to be reconstructed	
46		468	Pier to be reconstructed	
51			Pier to be reconstructed	
52	None	Not inspected	Repair substructure	
53	None		Fire boat station - no work	
54		468	Part replace, part repair	
57			Part repair	
61	500		Chelsea Pier - no work	
62			Repair substructure	
63		356	Pier to be reconstructed	
64	None	468	Part replace, part repair	
66		Not inspected	Pile field - pier to be reconstructed	
72		Not rated	Pile field with getdown	
78	None		Ferry Terminal - no work	
81	500		World Yacht - no work	
83	500		Circle Line - no work	
84		415	Pier to be reconstructed	
86	None		Intrepid Pier - No work	
88	500 (lower)+100(2nd)+50(roof)		Passenger ship terminal - No work	
90	500		Passenger ship terminal - No work	
92	500		Passenger ship terminal - No work	
94	650		Conference Center - no work	
96	400		Boat launch- existing pile field	

PIER	ORIGINAL DESIGN LOAD PER ACOE (1) (psf)	CURRENT LOAD CAPACITY PER HRPT (2) (psf)	COMMENT BASED ON MASTER PLAN	
97	400	110 - unusual config.	Pier to be reconstructed	
98	500		Con Edison Pier - no work	

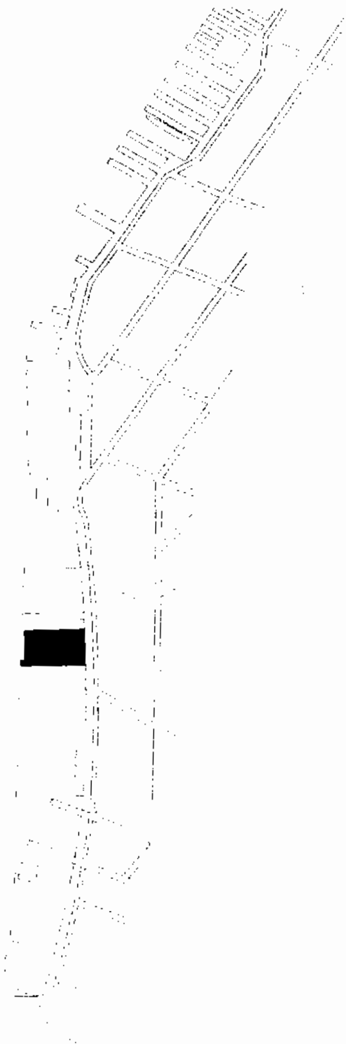
NOTE:

(1) U.S Army Corps of Engineers. Port Series No. 5, Revised 1999, ' The Port of New York, NY and NJ and Ports onLong Island, NY

(2) HRPT e-mail, per Goodkind and O'Dea inspection reports. Current pier load ratings (only for piers to be totally or partially re-built) to be totally or partially re-built)

APPENDIX F

CATHODIC PROTECTION SYSTEM DETAILS



HUDSON RIVER PARK

Pier 40 Cathodic Protection System

Contract No. C3019

Design prepared by

Norton
Corrosion
Limited
Woodinville, WA
(800) 426-3111

In association with

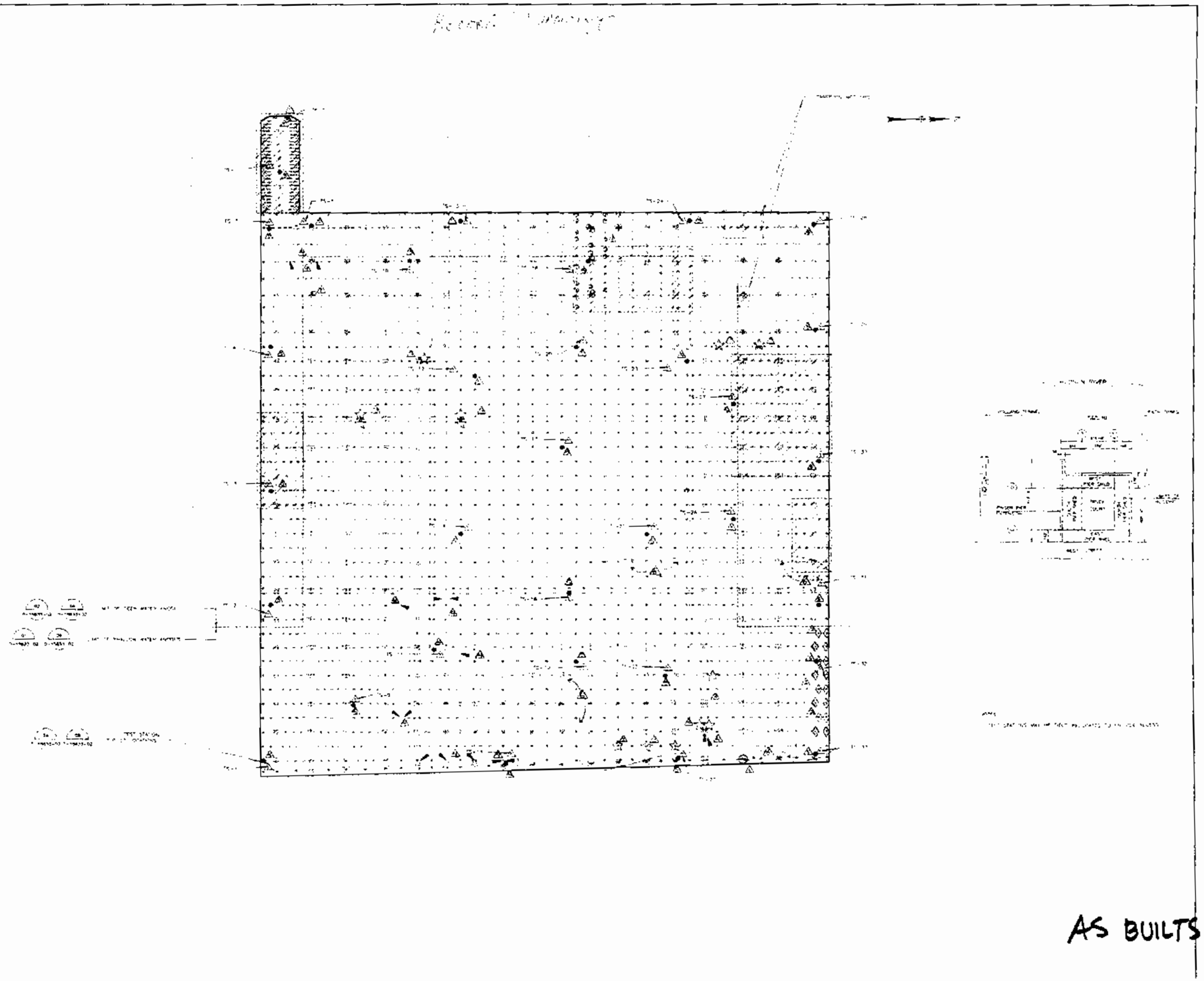
GOODKIND & O'DEA, INC.

Consulting Engineers and Planners
15 East 26th Street, New York, NY 10010-1505
(212) 685-0900

Hudson River Park Trust

Robert Balachandran — Date
President/CEO

AS. BUILTS



AS BUILTS

Hudson River Park Trust

The Honorable
George E. Pataki

The Honorable
Rudolph Giuliani

The Honorable
C. Virginia Fields

HUDSON RIVER PARK

Pier 40
Cathodic Protection System

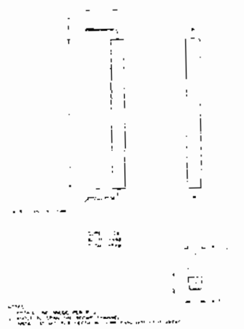
Norton Corrosion Limited

In association with
DORDINO AND CYDFA, INC
Consulting Engineers and Planners
15 East 26th Street, New York, NY 10010
(212) 685-0900

No.	Date	Revision

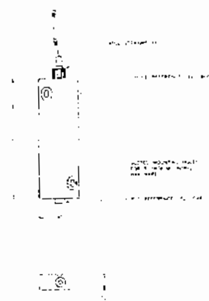
As Built Revisions

STRONGWORK DIVING

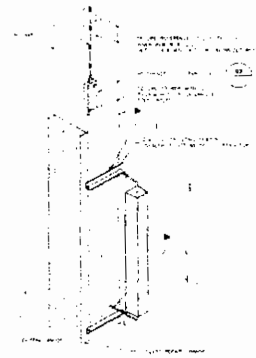


SHALLOW ANODE

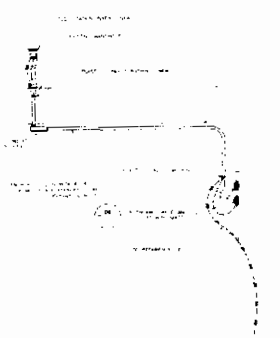
DEEP ANODE



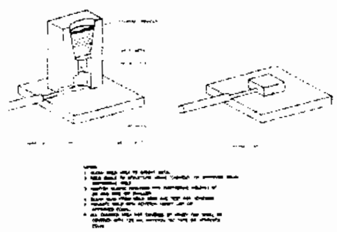
REFERENCE CELL



ANODE/REFERENCE CELL INSTALLATION



TYPICAL TEST STATION



EXOTHERMIC WELD

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