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.