Appendix D -
Existing Conditions Waterfront Inspections Report
NEW JERSEY TRANSIT

TASK ORDER 12 AS PART OF CONTRACT 13-002D

ENVIRONMENTAL CONSULTING SERVICES TASK ORDER CONTRACT TO PERFORM A FEASIBILITY STUDY AND ENVIRONMENTAL IMPACT STATEMENT FOR REBUILD BY DESIGN’S “RESIST, DELAY, STORE, DISCHARGE” PROJECT

TASK 2—WATERFRONT STRUCTURES INSPECTION
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INTRODUCTION

Boswell Engineering (Boswell) was tasked to perform a waterfront structures inspection in conjunction with New Jersey Transit’s Task Order 12 as part of Contract 13-002D Environmental Consulting Services Task Order Contract for Rebuild by Design’s “Resist, Delay, Store, Discharge” project. Boswell’s waterfront structures inspection included evaluating of 14 specific segments along the City of Hoboken’s waterfront and preparing load rating calculations for each of these. The following report summarizes the findings by segment.
BACKGROUND

Boswell’s task began by performing a cursory site visit from the waterside. In addition, all publicly available reports for waterfront inspections and recent repairs to the waterfront structures for the entire length of the City of Hoboken (City) waterfront were reviewed. Areas that did not have load calculations performed to date were identified.

Boswell was then provided a map indicating the location of 14 segments of interest along Hoboken’s waterfront with a total length not to exceed 2,000 linear feet (see Appendix A). These waterfront areas are both public and privately owned. Underwater inspections were performed on each of these segments following the American Society of Civil Engineers (ASCE) manual guidelines for “Underwater Inspections – Standard Practice Manual”. Following these inspections, load rating calculations were performed to determine the maximum surcharge load for each of the structures. A summary of these load rating calculations is included in Appendix D. Sketches of each segment can be found in Appendix B. The following is a description of the findings per segment:

SEGMENT 1

Structure Description

Segment 1 extends for approximately 39 feet on NJ Transit property. There was no available information regarding the construction of this structure. The structure type is anchored steel sheet piling. The anchors are located approximately 4 feet from the top. The anchors are connected directly to the sheets using beveled washers, with two anchors provided for each sheet pair. Based on the measurements taken during the diving inspection, the sheeting size appears to be PZ 27. The maximum exposure height of the sheet piles is 18 feet. (See Dwg. 1.1).

Inspection Findings

The sheeting appears to be in good condition. There is a 6 foot high band in the tidal zone with coating loss and minor surface corrosion. The section losses vary along the structure and through the sections. Based on D-meter readings, the minimum thickness of the web is 0.345 inches. This compares with the .375 inch original thickness of a PZ 27 section. (See Photos 1 and 2).

Ratings Results

The calculations were performed based on the assumptions that the steel grade of the piles is ASTM A572 grade 50, and the sheet piles are properly tied-back for this type of structure. Based on the inspection, the members were analyzed using 11% section loss. The load ratings for the structure indicate a reserve surcharge capacity of more than 400 lb/sf.
SEGMENT 2

Structure Description

Segment 2 extends for approximately 126 feet on NJ Transit property. There was no available information regarding the construction of this structure. The structure type is anchored steel sheet piling. The anchors are located approximately 2.7 feet from the top, and are connected directly to the sheets with two anchors provided for each sheet pair. Based on the measurements taken during the diving inspection, the sheeting size appears to be PZ 22. The maximum exposed height of the sheet piles is 16 feet. (See Dwg. 2.1).

Inspection Findings

The sheeting appears to be in good condition with minor coating loss along the top 2 feet. The section losses vary along the structure and through the sections. Based on D-meter readings, the minimum thickness of the web is 0.305 inches. This compares with the .375 inch original thickness of a PZ 22 section. (See Photos 3 and 4).

Ratings Results

The calculations were performed based on the assumptions that the steel grade of the piles is ASTM A572 grade 50, and the sheet piles are properly tied-back for this type of structure. Based on the inspection, the members were analyzed using 20% section loss. The load ratings for the structure indicate a reserve surcharge capacity of more than 400 lb/sf.

SEGMENT 3

Structure Description

Segment 3 extends for approximately 400 feet on City property. There are two structure types along this segment. The southernmost 285 feet is high level platform comprised of steel pipe piles (14 inch diameter) supporting longitudinal and transverse concrete cap beam and concrete deck. The remaining 335 feet is an approximately 18 feet high granite block retaining wall bulkhead supported on timber cribbing. There was no available information regarding the construction of these structures. (See Dwgs. 3.1 through 3.4).

Inspection Findings

The platform structure exhibits evidence of rust staining and coating loss at the top 2 feet of the piles. The granite block showed signs of minor grout loss, estimated to be 3 inches in depth. The crib wall was exposed for approximately 20 feet, showing a 20% section loss. (See Photos 5 and 6).
Ratings Results

Calculations for the platform structure were performed based on the following assumptions: concrete members have minimum reinforcement; concrete strength is 4000 psi, precast deck thickness based on the span to depth ratio. Based on the beam span, a 10 in hollow-core precast deck was assumed. The overall rating of the platform is based on the member with the lowest capacity, which controls the rating for the entire structure. The maximum surcharge load capacity is 260 lb/sf as governed by the capacity of the concrete beam.

The following assumptions were made for the granite block retaining wall: granite stone blocks have a depth of 3 ft. and 6 ft., block unit weight of 140 lb/ft³, and backfill unit weight of 120 lb/ft³. The stability analysis for the wall indicates there is no surcharge capacity based on the assumed parameters. However, there are no visible signs of distress. Further investigation of the structure’s construction would be necessary in order determine a surcharge capacity.

SEGMENT 4

Structure Description

Segment 4 extends for approximately 101 feet on City property. The structure consists of an approximate 14 feet high granite block retaining wall bulkhead. There was no available information regarding the construction of this structure, however, it was assumed this wall is also supported on timber cribbing, similar to the Segment 3 block wall section. (See Dwgs. 4.1 and 4.2).

Inspection Findings

The block sizes along the bulkhead vary in size, with some blocks completely missing throughout the structure. Widespread mortar loss was noted for the entire length of the segment. Voids in the wall varied, in some areas measuring 4 to 5 feet deep. There were no signs, however, of structural distress. (See photos 7 and 8).

Ratings Results

The calculations were performed based on an assumed average size and depth of block due to their various sizes. Based on the inspection, the conventional analysis indicates the structure should be failing, however, there appears to be load distribution occurring which explains why there are no signs of overstress at the time of the inspection.
SEGMENT 5

Structure Description

Segment 5 extends for approximately 100 feet on City property. From the underwater inspection, no structures were found, and the rip rap was noted to be sloping into the water. Previous inspection findings indicated a masonry seawall on timber cribbing. The general condition of the exposed timber was fair due to marine borer attack. These prior findings also revealed large voids and widespread mortar loss within the tidal zone for the length of the wall (See photo 9).

SEGMENT 6

Structure Description

Segment 6 extends for approximately 245 feet on City property. This pier consists of concrete-encased steel H-piles supporting cast-in-place concrete pile caps and a concrete deck. There was no available information regarding the construction of this structure. (See Dwg 6.1 through 6.3).

Inspection Findings

The concrete encasement at some of the H-piles was missing. The exposed H-pile sections showed signs of corrosion with maximum section loss of 50%. Some piles appeared bent with cracks in the concrete encasement. The concrete members were in good condition, with isolated spalled areas where some section loss of the steel was observed. (See photo 10 and 11).

Ratings Results

Calculations for the platform structure were performed based on the following assumptions: concrete members had minimum reinforcement, with 10% section loss for the deck, and 20% section loss for the beams. A concrete strength of 4,000 psi was used. The maximum surcharge load capacity is 110 lb/sf as governed by the capacity of the concrete beam.

SEGMENT 7

Structure Description

Segment 7 extends for approximately 222 feet on both City property and property owned by Shipyard Associates. The structure type is steel sheet pile bulkhead located at the land side of the pier. There was some available information regarding the construction of this structure. (See Appendix F). The segment consists of single and double steel sheet pile bulkhead. The sheet pile sections are PZ 22 and PZ 27.
Inspection Findings

Based on D-meter readings, a maximum section loss of 50% and 25% was recorded for the front and back sheets, respectively (See photo 12 and 13).

Ratings Results

The load ratings were performed based on the assumption that steel grade of the piles is ASTM A572 grade 50, and assumed properties of the soil. Based on these assumptions and the inspection findings, the reserve surcharge capacity for this segment is 250 lb/sf.

SEGMENT 8

Structure Description

Segment 8 extends for approximately 277 feet on Shipyard Associates property. The structure consists of two older platforms and one newer platform. There was some available information regarding the construction of the structures along this segment. (See Appendix F). One of the two older platforms is a high-level platform supported by 41 steel H-piles, 23 of which have concrete encasements that extend into the mudline. The remaining piles were not encased, or have partial encasements leaving portions of the steel H-pile exposed. The second older platform is a high-level timber platform which consists of timber piles supporting timber beams and concrete deck. The newer platform is oriented north to south and consists of 30 steel H-piles supporting cast-in-place concrete pile caps and a precast deck. (See Dwgs. 8.1 through 8.5).

Inspection Findings

During the inspections of the older platforms, isolated areas of spalling in the concrete pile caps and deck were found; also section losses of 50% to 100% on the steel piles were noted, with a maximum exposed length of 14.3 ft. The deterioration consisted of loss of bearing, loss of cross-sectional area, and splits in the piles. The concrete members of the newer platform are in good condition. The piles show a maximum section loss of 20%. (See photos 14 through 16).

Ratings Results

The following assumptions were made for these structures: concrete members have minimum reinforcement, concrete strength is 4000 psi, pile cap beams span at least 3 spans, timber grade used for caps is Southern Pine No.2, Southern Pine for the pile, and an assumed unit weight for the fill above the deck. As some of the piles at the older platform exhibited 100% section loss, the concrete beams were analyzed for the increased span length that resulted in an overstress of the beam. The newer platform was analyzed based on the inspection findings and available plans. The surcharge rating for the newer platform is 140 lb/sf as governed by the capacity of the concrete deck. The surcharge rating for the older platform is 100 lb/sf as governed by the capacity of the concrete deck.
SEGMENT 9

Structure Description

Segment 9 extends for 114 feet on Shipyard Associates property. The structure is a high-level timber platform which consists of timber piles supporting timber beams and concrete deck. There was no available information regarding the construction of this structure. (See Dwgs 9.1 and 9.2).

Inspection Findings

The general condition of the timber piles is fair. Of the 489 previously inspected piles, 32 piles have deterioration classified as severe and 15 have defects classified as moderate. The deterioration consists of loss of bearing, loss of cross-sectional area, and splits. Some attempts have been made to repair some of the deficiencies, including use of foam to fill voids, but this has been inadequate to restore loss of cross-sectional area or bearing. There was also some spalling with corroded reinforcing at the underside of the deck observed. (See Photos 17 and 18).

Ratings Results

The rating of this segment was performed based on the following assumptions: concrete members have minimum reinforcement, concrete strength is 4000 psi, pile cap beams span at least 3 spans, timber grade used for caps is Southern Pine No.2, and Southern Pine for the pile. The piles were analyzed using 25% section loss. The surcharge rating for this platform is 100 lb/sf as governed by the capacity of the concrete deck.

SEGMENT 10

Structure Description

Segment 10 extends approximately 50 feet on Shipyard Associates property. The structure consists of steel pipe piles with concrete pile caps supporting concrete deck planks and beams. There was some available information regarding the construction of this structure along this segment. (See Appendix F and see Dwgs. 10.1 and 10.2).

Inspection Findings

The overall condition of the concrete members was good; the pipe piles exhibit moderate corrosion, with minor pitting along the top 3 feet of the piles. (See photo 19).
Ratings Results

Since no losses to the concrete members were recorded during the inspection, the analysis was performed based on the design section. The piles were analyzed using 45% section loss. The load ratings indicate the structure has a surcharge capacity of 225 lb/sf as controlled by the concrete beams.

SEGMENT 11

Structure Description

Segment 11 extends 60 feet on Shipyard Associates property. The structure is a steel pipe pile supported high-level platform. The structure consists of 24 16-inch diameter steel pipe piles driven in 12 clusters of 2 piles each supporting cast-in-place concrete pile caps. The pile caps in turn support precast concrete beams and deck planks. There was some available information regarding the construction of this structure. (See Appendix F and see Dwg. 10.1 and 10.2).

Inspection Findings

The overall condition of the concrete members was good. Minor corrosion of the pipe piles was observed. (See photo 20).

Ratings Results

The analysis was performed based on the design section for the concrete members, and 10% section loss for the piles. The load ratings indicate the structure has a surcharge capacity of 255 lb/sf as controlled by the concrete beams.

SEGMENT 12

Structure Description

Segment 12 extends approximately 109 feet on Toll Brothers property. The structure is a 7.2’ high concrete seawall on a low level timber platform supported by timber piles at 5 ft. spacing in the longitudinal direction. There was no available information regarding the construction of this structure. (See Dwg. 12.1).

Inspection Findings

The timber piles were in satisfactory condition with minor marine borer infestation. The timber pile caps had 30% - 40% section loss near the bottom 4 inches, resulting in up to 50% loss of bearing. The piles had approximately 10% section loss. The concrete seawall appeared to be in
good condition. The inspection could not determine the extent of the structure in the transverse direction, or the next pile row in that direction. The transverse distance from the outer face of the pile to the mud line was 5 ft. (See Photos 21 and 22).

Ratings Result

Due to the limited information from the diving inspection, the analysis was performed based on the following assumptions: the sea wall is 3 feet wide at the top and 5 feet wide at the bottom, the spacing between the piles in transverse direction is 5 ft, the timber grade for the deck is Southern Pine No. 2 for the cap pile, and Southern Pine for the pile, the pile cap beam and decking span at least 3 spans, and assumed properties for the backfill soil. Based on the above, the analysis shows the timber cap beams are overstressed. However, there were no visible signs of distress.

SEGMENT 13

Segment 13 extends 75 feet on City property. No visible structure was noted. Only a rip-rap bank sloping into the water was observed. (See photo 23).

SEGMENT 14

Segment 14 extends approximately 92 feet on both City and City of Weehawken property. No visible structure was noted. Only a rip-rap bank sloping into the water was observed. (See photo 24).
APPENDIX A
SEGMENT PLANS
APPENDIX B
SKETCHES
SEGMENT 1 TYPICAL SECTION

PAVER WALKWAY
FILL
SHEET PILE PZ 27
ANCHOR BEARING PLATE
TIDAL (VARY)
MUDLINE

RAILING

4' ±

18' ±
SEGMENT 2 TYPICAL SECTION
NOTES:

1. ALL ACCESSIBLE PILES HAVE MINOR TO MODERATE COATING LOSS WITH MINOR CORROSION, BLISTERING, AND RUST STAINING IN THE TOP 2 TO 3'.
NOTES:
1. INTERMITTENT GROUT LOSS WAS FOUND IN THE BOTTOM 2' TO 3' OF THE VERTICAL JOINTS THROUGHOUT THE MASONRY GRAVITY WALL

LEGEND:
☐ TIMBER CRIBBING EXPOSED BELOW

CITY OF HOBOKEN
LOAD EATING WATERFRONT STRUCTURES
SEGMENT 3-MASONRY GRAVITY WALL PLAN VIEW

TIMBER CRIBBING EXPOSED BELOW MASONRY GRAVITY WALL
UP TO 0.5' VERTICAL EXPOSURE

FACE OF MASONRY GRAVITY WALL

HUDSON RIVER

FLOOD
SECTION B-B

PAVER WALKWAY

STEEL RAILING

CONCRETE BEAM

FILL

2 OF MASONRY 2'Hx3'W (BLOCK 1)

VARIES (TIDAL)

4 OF MASONRY 2'Hx6'W (BLOCK 2)

BLOCK 2 (TYP.)

TIMBER CRIBBING

17.5'±
NOTES:

1. THERE IS WIDESPREAD GROUT LOSS AND MISSING AND DISPLACED MASONRY UNITS WITHIN THE TIDAL ZONE.

LEGEND:

- MASONRY GRAVITY WALL WITH WIDESPREAD GROUT LOSS, LOOSE AND MISSING MASONRY UNITS
SECTION C-C

- Steel Railing
- Concrete Beam 6'H x 3'W
- 1 of Masonry 2'H x 3'W (Block 1)
- Fill
- Varies (Tidal)
- 4 of Masonry 2'H x 6'W (Block 2)
- Paver Walkway
WEST PIER TYPICAL SECTION

12" THICK CONCRETE DECK

CONCRETE BEAM
24" x 36" (TYP.)

RAILING

MUDLINE

ABANDONED CONCRETE
ENCASED STEEL H-PILE (TYP.)

RE-ENCASED CONCRETE
STEEL H-PILE: HPI2×5 (TYP.)

12.5' ±

TIDAL (VARIES)

12'
SECTION G-G

TIMBER PILECAP 1'HX1'W (TYP.)

5" THICK CONCRETE DECK

RAILING

TIDAL (VARIIES)

15'±

TIMBER PILE #12 (TYP.)
NOTE:
1. ALL PIPE PILES HAVE RANDOM, INTERMITTENT COATING LOSS AND MINOR CORROSION WITHIN THE TOLERABLE ZONE.
2. HIGH-LEVEL PIPE PILE PLATFORM WAS BUILT OVER A PARTIALLY COLLAPSED STEEL H-FRAME SUPPORTED PLATFORM, NOT SHOWN FOR CLARITY.
SECTION H-H

CONCRETE CAP BEAM 4"X4" W (TYP.)

15" THICK CONCRETE DECK

CONCRETE BEAM 3"X4" W (TYP.)

FILL

RAILING

STEEL PILE (TYP.)

STEEL PILE HSS16X0.375

TIDAL (VARIES)

2.2" max

20'

6'±
APPENDIX C

PHOTOS
Photo 1- Segment 1 General View Looking Northwest

Photo 2- Segment 1 Moderate Corrosion & Coating Loss
Photo 3- Segment 2 General View Looking Northwest

Photo 4- Segment 2 Moderate Corrosion & Coating Loss
Photo 5- Segment 3 General View Looking Southwest

Photo 6- Segment 3 Minor Grout Loss in Granite Block Up to 3” Deep
Photo 7- Segment 4 General View Looking Northwest

Photo 8- Segment 4 Close-up Severe Block Displacement and Grout Loss
With Voids Exceeding 5' Deep
Photo 9- Segment 5 General View Looking North

Photo 10- Segment 6 Encased Steel H-Piles
Photo 11- Segment 6 Encased Steel H-Piles and Exposed Steel

Photo 12- Segment 7 Steel Sheet Pile Bulkhead Looking North from North Edge of Pier
Photo 13 - Segment 7 Double Sheet Pile Bulkhead Looking South

Photo 14 - Segment 8 Corroded H-Steel Pile - Old Platform
Photo 15- Segment 8 Spalling Area of the Deck-Old Platform

Photo 16- Segment 8 Concrete Deck and Beams with Steel H-Piles-New Platform
Photo17-Segment 9 General View Looking North

Photo18-Segment 9 Severe Deterioration and Loss of Bearing on Timber Pile
Photo23- Segment 13 General View

Photo24- Segment 14 General View
APPENDIX D

LOAD RATING SUMMARY TABLE
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<th>Segment</th>
<th>Lot/Block</th>
<th>Owner</th>
<th>Available Information</th>
<th>Structure Type</th>
<th>Load Ratings</th>
<th>Drawing Reference</th>
<th>Photo Reference</th>
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<tbody>
<tr>
<td>1</td>
<td>1/01/139</td>
<td>NJ Transit</td>
<td>None</td>
<td>Anchored steel sheet pile bulkhead</td>
<td>400 PSF</td>
<td>1.1</td>
<td>1, 2</td>
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<tr>
<td>2</td>
<td>3/139</td>
<td>NJ Transit</td>
<td>None</td>
<td>Anchored steel sheet pile bulkhead</td>
<td>400 PSF</td>
<td>2.1</td>
<td>3, 4</td>
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<td>3</td>
<td>4/231.05</td>
<td>City of Hoboken</td>
<td>Previously inspected as part of waterfront inspection of City-owned properties.</td>
<td>Section 1: Concrete deck planks on 14&quot; steel pipe piles with concrete pile caps. Section 2: Masonry Gravity Wall on timber crib structure</td>
<td>165 PSF</td>
<td>3.1 - 3.4</td>
<td>5, 6</td>
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<tr>
<td>4</td>
<td>1/233</td>
<td>City of Hoboken</td>
<td>Previously inspected as part of waterfront inspection of City-owned properties.</td>
<td>Masonry block retaining wall on timber cribbing</td>
<td>Severe, extensive deterioration.</td>
<td>4.1, 4.2</td>
<td>7, 8</td>
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<tr>
<td>5</td>
<td>1/233</td>
<td>City of Hoboken</td>
<td>None</td>
<td>Rip-Rap on grade. No structure visible by diving inspection.</td>
<td>No structure to rate.</td>
<td>......</td>
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<td>6</td>
<td>1/261.05</td>
<td>City of Hoboken</td>
<td>Previously inspected as part of waterfront inspection of privately-owned waterfront properties (PT Maxwell West Pier section).</td>
<td>Concrete deck on concrete pile caps supported by concrete encased steel H-piles</td>
<td>110 PSF</td>
<td>6.1 - 6.3</td>
<td>10, 11</td>
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<tr>
<td>7</td>
<td>1/261.05; 1/262.01</td>
<td>City of Hoboken/Shipyard Assoc.</td>
<td>Previously inspected as part of waterfront inspection of privately-owned waterfront properties (PT Maxwell North Pier section). Some plan information available.</td>
<td>Steel sheet pile bulkhead</td>
<td>250 PSF</td>
<td>7.1, 7.2</td>
<td>12, 13</td>
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<td>8</td>
<td>1/262.01</td>
<td>Shipyard Assoc.</td>
<td>Previously inspected as part of waterfront inspection of privately-owned waterfront properties (Shipyard section partial). Some plan information available.</td>
<td>Old section 1: High level platform supported by concrete encased H-piles. Old section 2: High level platform consisting of concrete deck and timber pile caps supported by timber piles. New section. Precast concrete deck on CIP pile caps supported by steel H-piles (some encased in concrete)</td>
<td>100 PSF</td>
<td>8.1 - 8.5</td>
<td>14-16</td>
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<td>9</td>
<td>1/264.01</td>
<td>Shipyard Assoc.</td>
<td>Previously inspected as part of waterfront inspection of privately-owned waterfront properties (NY Waterway section)</td>
<td>Concrete deck on high-level timber platform supported by timber piles</td>
<td>100 PSF</td>
<td>9.1, 9.2</td>
<td>17, 18</td>
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<td>10</td>
<td>1/264.01</td>
<td>Shipyard Assoc.</td>
<td>Some plan information available.</td>
<td>Concrete deck planks/beams on steel pipe piles with concrete pile caps</td>
<td>225 PSF</td>
<td>10.1, 10.2</td>
<td>19</td>
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<td>11</td>
<td>1/264.01</td>
<td>Shipyard Assoc.</td>
<td>Previously inspected as part of waterfront inspection of privately-owned waterfront properties (Sovereign at Shipyard section). Some plan information available.</td>
<td>Precast concrete deck planks/beams on 16&quot; steel pipe piles with CIP concrete pile caps</td>
<td>255 PSF</td>
<td>10.1, 10.2</td>
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<td>12</td>
<td>4/268.01</td>
<td>Toll Brothers</td>
<td>None</td>
<td>Concrete seawall on low level timber platform supported by timber piles</td>
<td>100 PSF (Rating of fascia pile cap only)</td>
<td>12.1</td>
<td>21, 22</td>
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<td>City of Hoboken</td>
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<td>Rip-Rap on grade. No structure visible by diving inspection.</td>
<td>No structure to rate.</td>
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<td>1/267; 2/1</td>
<td>City of Hoboken/ Twp. of Weehawken</td>
<td>None</td>
<td>Rip-Rap on grade. No structure visible by diving inspection.</td>
<td>No structure to rate.</td>
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APPENDIX E
CALCULATIONS
All references shall be AASHTO LRFD Bridge Design Specifications unless otherwise noted.

### Wall Geometry

- **Design Height (Top of Sheet Piles to Dredge Line):** \( H = 18.00 \text{ ft} \)
- **Anchor Location (From top of Sheet Pile):** \( h = 4.00 \text{ ft} \)

### Slope at Top of Wall

- **Angle of Fill:** \( \beta = \tan(\text{H/V}) = 0.0 \text{ degrees} \)

### Slope at Bot. of Wall

- **Angle of Fill:** \( \beta' = \tan(\text{V/H}) = 0.0 \text{ degrees} \)

### Angle of Back Face of Wall

- \( \theta = 90.0 \text{ degrees} \)

### Soil 1

- **Retained Soil From Top of Wall to Dredge Line**
  - **Unit Weight:** \( \gamma_1 = 0.120 \text{ kips/ft}^3 \)
  - **Effective Angle of Internal Friction:** \( \phi'_{11} = 28.0 \text{ degrees} \)

### Tab. 3.11.5.3-1

- **Friction Angle Between Fill and Wall:** Steel sheet piles against silty sand, gravel or sand mixed with silt or clay
  - **Intermediate Calculated Value:** \( \delta_i = 17.0 \text{ degrees} \)

### Tab. 3.11.5.3-2

- **Coulomb Active Pressure:** \( K_{a1} = 0.32 \text{ kips} \)

### Tab. 3.11.5.6

- **Active Earth Pressure:** \( K_{a1} \cdot \gamma_1 = 0.039 \text{ ksf/ft} \)

### Soil 2

- **Soil Below Dredge Line**
  - **Unit Weight:** \( \gamma_2 = 0.120 \text{ kips/ft}^3 \)
  - **Effective Angle of Internal Friction:** \( \phi'_{22} = 28.0 \text{ degrees} \)

### Tab. 3.11.5.3-1

- **Friction Angle Between Fill and Wall:** Steel sheet piles against silty sand, gravel or sand mixed with silt or clay
  - **Intermediate Calculated Value:** \( \delta_i = 14.0 \text{ degrees} \)

### Tab. 3.11.5.3-2

- **Coulomb Active Pressure:** \( K_{a2} = 0.33 \text{ kips} \)

### Tab. 3.11.5.6

- **Active Earth Pressure:** \( K_{a2} \cdot \gamma_2 = 0.039 \text{ ksf/ft} \)

### Additional Loads

- **Live Load Surcharge:** \( q_L = 0.40 \text{ kip/ft}^2 \)
- **Soil Load Surcharge:** \( q_S = 0.00 \text{ kip/ft}^2 \)

### Forces on Sheet Pile

- **Design Limit State** = Strength I

#### Load Category

- **\( P_{L1} = PL \cdot K_{a1} \cdot \gamma_1 \cdot H \)** = 4.07 = 4.07 kips
- **\( P_{L2} = PL \cdot K_{a1} \cdot \gamma_1 \cdot D_b \)** = 0.23 \* \( D_b \) = 2.03 kips
- **\( P_{a1} = ES_{max} \cdot K_{a1} \cdot \gamma_1 \cdot H \)** = 0.00
- **\( P_{a2} = ES_{max} \cdot K_{a1} \cdot \gamma_1 \cdot D_b \)** = 0.00 \* \( D_b \) = 0.00 kips
- **\( P_{v1} = EH_{max} \cdot 1/2 \cdot K_{a1} \cdot \gamma_1 \cdot H^2 \)** = 9.41 = 9.41 kips
- **\( P_{v2} = EH_{max} \cdot 1/2 \cdot K_{a1} \cdot \gamma_1 \cdot D_b \)** = 1.06 \* \( D_b \) = 9.38 kips
- **\( P_{v3} = EH_{max} \cdot 1/2 \cdot K_{a1} \cdot \gamma_1 \cdot D_b^2 \)** = 0.03 \* \( D_b^2 \) = 2.31 kips
- **\( P_{f} = EH_{max} \cdot 1/2 \cdot K_{a1} \cdot \gamma_1 \cdot D_b^2 \)** = 0.22 \* \( D_b^2 \) = 17.67 kips

### Moments Taken About Anchor

#### Overturning Moments

- **\( M_{L1} = P_{L1} \cdot (H/2 - h) \)** = 20.34 = 20.3 kip*ft
- **\( M_{L2} = P_{L2} \cdot (H - h + D_b/2) \)** = 3.20 \* \( D_b \) = 37.3 kip*ft
- **\( M_{a1} = P_{a1} \cdot (H/2 - h) \)** = 0.00 + 0.00 \* \( D_b \) = 0.0 kip*ft
- **\( M_{a2} = P_{a2} \cdot (H - h + D_b/2) \)** = 0.00 \* \( D_b \) = 0.0 kip*ft
- **\( M_{v1} = P_{v1} \cdot (H + h + D_b/2) \)** = 75.30 = 75.3 kip*ft
- **\( M_{v2} = P_{v2} \cdot (H - h + D_b/2) \)** = 14.80 \* \( D_b \) = 172.8 kip*ft
- **\( M_{v3} = P_{v3} \cdot (H - h + D_b/2) \)** = 0.41 \* \( D_b^2 \) + 0.02 \* \( D_b^3 \) = 46.0 kip*ft

- **Total Overturning Moments** = 95.64 + 18.00 \* \( D_b \) + 1.05 \* \( D_b^2 \) + 0.02 \* \( D_b^3 \) = 351.8 kip*ft

#### Resisting Moments

- **\( M_{r} = P_{f} \cdot (H - h + 2/3 \cdot D_b) \)** = 3.15 \* \( D_b^2 \) + 0.15 \* \( D_b^3 \) = 104.4 kip*ft

#### Total Moments

- **Total Overturning Moments \( \cdot M \)** = 0 = 95.64 + 18.00 \* \( D_b \) + 0.0 \* \( D_b^2 \) - 2.09 \* \( D_b^2 \) - 0.13 \* \( D_b^3 \) = 0.0 kip*ft
### Anchor Force

Total Driving Forces
\[ P_1 = 27.19 \text{ kips} \]

Total Resisting Forces
\[ P_2 = 17.67 \text{ kips} \]

Anchor Force
\[ P_1 - P_2 = A_1 = 9.52 \text{ kips/ft} \]

### Calculate Location of Zero Shear

\[
P_{d1x} = \frac{P_L}{E} \cdot K_{x1} \cdot \psi_{1} \cdot x \\
P_{d2x} = \frac{P_{L}}{E} \cdot K_{x2} \cdot \psi_{2} \cdot x \\
P_{d3x} = \frac{P_{L}}{E} \cdot \gamma_{1} \cdot \psi_{1} \cdot x^2 \\
\text{Anchor Force:} -9.52 \\
\text{Depth of Zero Shear:} \quad 0 = -9.52 + 0.23 \cdot x + 0.03 \cdot x^2 \\
\quad \quad x = 14.63 \text{ ft} \\
\]

### Shear and Moment Tables

<table>
<thead>
<tr>
<th>Location</th>
<th>Distance from top of Sheet</th>
<th>Individual Shear Forces</th>
<th>Individual Moments</th>
<th>Total Shear</th>
<th>Total Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Sheet Pile</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
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<td>-8.15</td>
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<tr>
<td>Zero Shear</td>
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<td>3.31</td>
<td>6.22</td>
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<tr>
<td>Dredge Line</td>
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<td>-9.52</td>
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<tr>
<td>Maximum Shear</td>
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<td></td>
<td></td>
<td>0.00</td>
<td>8.15</td>
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<tr>
<td>Total Moment</td>
<td></td>
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<td></td>
<td>0.00</td>
<td>46.72</td>
</tr>
</tbody>
</table>

### Structural Steel Properties

- **Rolled Section Structural Steel Material:** ASTM A572 Grade 50
- **Modulus of Elasticity of Steel Beam:** \( E_s = 29,000 \text{ ksi} \)
- **Resistance Factor for Flexure and Shear in Steel:** \( \Phi_s = 1.00 \)
- **Hybrid Factor:** \( R_h = 1.0 \)

### Steel Sheet Pile Size

- **Sheet Pile Width:** \( w = 18 \text{ in} \)  
- **Sheet Pile Height:** \( d = 12 \text{ in} \)  
- **Flange Thickness:** \( t_f = 0.375 \text{ in} \)  
- **Web Thickness:** \( t_w = 0.375 \text{ in} \)  
- **Weight / Pile:** 40.5 lb/ft  
- **Weight / Foot:** 27 lb/ft^2
### Sheet Pile Loads

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design Moment</td>
<td>$M_u = 46.7$ kip-ft</td>
</tr>
<tr>
<td></td>
<td>Design Maximum Flexural Stress in Sheet Pile</td>
<td>$M_u / S = f_{ux} = 18.56$ ksi</td>
</tr>
</tbody>
</table>

#### Check Strength Limit State

**Flexural Design for Continuously Braced Flanges in Tension and Compression**

6.10.8.1.3-1

- **Design Strength for Continuously Braced Flanges**
  - $f_{ux} = 18.56 \leq \Phi_{b} \cdot R_{n} \cdot F_{y} = 50.00$
  - **OK Flexure**
  - **Utilization Percentage**
    - $37.1\%$

**Sheet Pile Loads**

- **Steel Elastic Section Modulus per foot**
  - Measured Section Loss 11%
  - $S = 26.9$ in$^3$/ft
- **Design Maximum Flexural Stress in Sheet Pile Based Upon Deteriorated Steel Section**
  - $M_u / S = f_{ux} = 20.86$ ksi

#### Check Strength Limit State

**Flexural Design for Continuously Braced Flanges in Tension and Compression**

- **Design Strength for Continuously Braced Flanges**
  - $f_{ux} = 20.86 \leq \Phi_{b} \cdot R_{n} \cdot F_{y} = 50.00$
  - **OK Flexure**
- **Utilization Percentage**
  - $41.7\%$
All references shall be AASHTO LRFD Bridge Design Specifications unless otherwise noted.

### Wall Geometry
- Design Height (Top of Sheet Piles to Dredge Line) \( H = 16.00 \ \text{ft} \)
- Anchor Location (From top of Sheet Pile) \( h = 2.66 \ \text{ft} \)

#### Slope at Top of Wall
- Angle of Fill \( \beta = \tan(H/V) = 0.0 \ \text{degrees} \)
- Angle of Back Face of Wall \( \theta = 90.0 \ \text{degrees} \)

### Soil 1 - Retained Soil From Top of Wall to Dredge Line
- Unit Weight \( \gamma_{1} = 0.120 \ \text{ksf/ft}^3 \)
- Friction Angle Between Fill and Wall \( \delta_1 = 17.0 \ \text{degrees} \)
- Coulomb Active Pressure \( K_{A1} = 0.32 \)
- Active Earth Pressure \( K_{a1} \gamma_{1} = 0.039 \ \text{ksf/ft} \)

### Soil 2 - Soil Below Dredge Line
- Unit Weight \( \gamma_{2} = 0.120 \ \text{ksf/ft}^3 \)
- Friction Angle Between Fill and Wall \( \delta_2 = 14.0 \ \text{degrees} \)
- Coulomb Active Pressure \( K_{A2} = 0.33 \)
- Active Earth Pressure \( K_{a2} \gamma_{2} = 0.039 \ \text{ksf/ft} \)

### Additional Loads
- Live Load Surcharge \( q_L = 0.40 \ \text{kip/ft}^2 \)
- Soil Load Surcharge \( q_S = 0.00 \ \text{kip/ft}^2 \)

### Forces on Sheet Pile
#### Load Category
- **PH**: \( P_{PH} = \ \text{max} \)
- **EH**: \( P_{EH} = \ \text{max} \)

### Moments Taken About Anchor

#### Overturning Moments
- \( M_{L1} = P_{L1} (H/2 - h) = 19.3 \ \text{kip*ft} \)
- \( M_{L2} = P_{L2} (H - h + D_h/2) = 32.5 \ \text{kip*ft} \)
- \( M_{L3} = P_{L3} (H/2 - h) = 0.00 \ \text{kip*ft} \)
- \( M_{L4} = P_{L4} (H - h + D_h/2) = 59.55 \ \text{kip*ft} \)
- \( M_{L5} = P_{L5} (H + h + D_h/2) = 133.9 \ \text{kip*ft} \)
- \( M_{L6} = P_{L6} (H + h + D_h/2) = 36.9 \ \text{kip*ft} \)

#### Total Overturning Moments
- Total Overturning Moments = 282.2 \ \text{kip*ft}

#### Resisting Moments
- \( M_{R} = P_{R} (H - h + 2/3 * D_h) = 81.8 \ \text{kip*ft} \)
- Total Moments = 0 \ \text{kip*ft}
Solve for D

\[ D_h = 8.2 \text{ ft} \]

\[ D = 1.2 \times D_h = 9.8 \text{ ft} \]

**Anchor Force**

Total Driving Forces

\[ P_1 = 22.57 \text{ kips} \]

Total Resisting Forces

\[ P_2 = 15.02 \text{ kips} \]

Anchor Force

\[ P_1 - P_2 = A_1 = 7.55 \text{ kips/ft} \]

**Calculate Location of Zero Shear**

\[ P_{a1} = \frac{PL}{ES} \times K_{u1} \times q_1 \times x \quad 0.23 \times x \]

\[ P_{a2} = \frac{EH_{max}}{1/2} \times K_{u1} \times q_1 \times x^2 \quad 0.03 \times x^2 \]

Anchor Force

\[-7.55 \]

Depth of Zero Shear

\[ 0 = -7.55 + 0.23 \times x + 0.03 \times x^2 \quad x = 12.69 \text{ ft} \]

**Shear and Moment Tables**

<table>
<thead>
<tr>
<th>Location</th>
<th>Distance from top of Sheet</th>
<th>P1,1</th>
<th>P1,2</th>
<th>P3,1</th>
<th>P3,2</th>
<th>P4,1</th>
<th>P4,2</th>
<th>P4,3</th>
<th>P4,4</th>
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<td>Top of Sheet Pile</td>
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<th>P1,1</th>
<th>P1,2</th>
<th>P3,1</th>
<th>P3,2</th>
<th>P4,1</th>
<th>P4,2</th>
<th>P4,3</th>
<th>P4,4</th>
<th>Anchor</th>
<th>Total Moment</th>
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<td>0.00</td>
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<td>0.00</td>
<td>0.00 kip*ft</td>
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<td>0.00</td>
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<td>0.98 kip*ft</td>
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<td></td>
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<td></td>
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<td>37.73 kip*ft</td>
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</tbody>
</table>

**Structural Steel Properties**

Rolled Section Structural Steel Material

\[ F_y = F_{wy} = F_{st} = 50 \text{ ksi} \]

Modulus of Elasticity of Steel Beam

\[ E_t = 29,000 \text{ ksi} \]

Resistance Factor for Flexure and Shear in Steel

\[ F_r = F_p = 1.00 \]

Hybrid Factor

\[ R_s = 1.0 \]

**Steel Sheet Pile Size**

Sheet Pile Width

\[ w = 22 \text{ in} \]

Steel Area per foot

\[ A = 6.47 \text{ in}^2/\text{ft} \]

Sheet Pile Height

\[ d = 9 \text{ in} \]

Steel Elastic Section Modulus per foot

\[ S = 18.1 \text{ in}^3/\text{ft} \]

Flange Thickness

\[ t_f = 0.375 \text{ in} \]

Steel Plastic Section Modulus per foot

\[ Z = 21.79 \text{ in}^3/\text{ft} \]

Web Thickness

\[ t_w = 0.375 \text{ in} \]

Steel Moment of Inertia per foot

\[ I = 84.38 \text{ in}^4/\text{ft} \]

Weight / Pile

\[ 40.3 \text{ lb/ft} \]

Weight / Foot

\[ 22 \text{ lb/ft}^2 \]
<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet Pile Loads</td>
<td>Design Moment</td>
<td>( M_u = 37.7 \text{ kip-ft} )</td>
</tr>
<tr>
<td></td>
<td>Design Maximum Flexural Stress in Sheet Pile</td>
<td>( M_u / S = f_{u_s} = 25.01 \text{ ksi} )</td>
</tr>
<tr>
<td>Check Strength Limit State</td>
<td>Flexural Design for Continuously Braced Flanges in Tension and Compression</td>
<td>Design Strength for Continuously Braced Flanges</td>
</tr>
<tr>
<td></td>
<td>Utilization Percentage</td>
<td>OK Flexure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50.0%</td>
</tr>
<tr>
<td>Sheet Pile Loads</td>
<td>Steel Elastic Section Modulus per foot</td>
<td>Measured Section Loss</td>
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<tr>
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<td>Design Maximum Flexural Stress in Sheet Pile Based Upon Deteriorated Steel Section</td>
<td>( M_u / S = f_{u_s} = 31.27 \text{ ksi} )</td>
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<tr>
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<td>Utilization Percentage</td>
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<tr>
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<td>62.5%</td>
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</tbody>
</table>
### References

- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

### Beam Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>ℓ</td>
<td>15.00 ft</td>
</tr>
<tr>
<td>Beam Depth</td>
<td>h</td>
<td>36.00 in</td>
</tr>
<tr>
<td>Minimum Beam Depth for Deflection</td>
<td>h&lt;sub&gt;min&lt;/sub&gt; = ℓ / 20.0</td>
<td>9.00 in</td>
</tr>
<tr>
<td>Beam Width</td>
<td>b</td>
<td>36.00 in</td>
</tr>
<tr>
<td>Tributary Load Width</td>
<td>B</td>
<td>3.00 ft</td>
</tr>
</tbody>
</table>

### Dead Loads

#### Uniform Dead Loads

- Steel Railing: D<sub>1</sub> = 30 lb/ft
- Concrete Parapet: D<sub>2</sub> = 900 lb/ft²
- Miscellaneous: D<sub>3</sub> = 2,700 lb/ft

#### Weight of Beam

- b * h * w<sub>c</sub> = 1,350 lb/ft
- Total Dead Load: D = 4,080 lb/ft²

### Live Loads

#### Design Floor Load

- L<sub>0</sub> = 260 lb/ft²
- Tributary Area for Floor: A<sub>T</sub> = 45 ft²

#### Reduced Live Load

- L<sub>L</sub> = L<sub>0</sub> * (0.25 + 15 / √(K<sub>LL</sub> * A<sub>T</sub>))
- Reduced Live Load: L<sub>L</sub> = 46,080 lb
- Design Moment (Factored Load): M<sub>u</sub> = 172,800 lb*ft

### Concrete Properties

- Unit Weight of Concrete: w<sub>c</sub> = 150 pcf
- Specified Concrete Compressive Strength: f<sub>c</sub> = 4,000 psi

#### Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth

- K<sub>L</sub> = 1.00

### Load Combinations

- Design Load: 1.2 * D + 1.6 * L = 6,144 lb/ft
- Deflection Load, Dead + Live: D + L = 4,860 lb/ft
- Deflection Load, Live: L = 780 lb/ft

### Design Shear and Moment Assuming Simple Span

- Design Shear (Factored Load): V<sub>s</sub> = 46,080 lb
- Design Moment (Factored Load): M<sub>s</sub> = 172,800 lb*ft

### Steel Reinforcing Properties

- Minimum Specified Yield Stress of Steel: f<sub>y</sub> = 60,000 psi
- Assumed Reinforcing Steel Configuration
  - Clear Cover: d = 2.00 in
  - Tension Steel Reinforcement
    - #6 # of Bars = 3
    - d<sub>6</sub> = 0.750 in, bar area = 0.44 in²
    - A<sub>s</sub> = 1.32 in²
  - Number of shear legs per stirrup: 2
  - Shear Reinforcement
    - #4
    - s = 12 in
    - d<sub>s</sub> = 0.500 in, bar area = 0.20 in²
    - A<sub>s</sub> = 0.40 in²

### Flexural Design Capacity

- Minimum Design Reinforcement Steel Ratio: A<sub>s</sub> = 0.85 * f<sub>y</sub> / f<sub>c</sub> * d = 0.00098
- Minimum Design Reinforcement Steel Area: A<sub>s</sub> = 36.00 * 33.13 = 1.17 in²
- Minimum Reinforcement of Flexural Members: Λ<sub>s,min</sub> = 3 * (V<sub>f</sub> / f<sub>c</sub> * b * d) = 3.77 in²

#### Depth of Equivalent Rectangular Stress Block

- a = A<sub>s</sub> * f<sub>c</sub> / (0.85 * f<sub>y</sub> * b) = 0.65 in
- Distance From Extreme Compression Fiber to Neutral axis: c = a / b = 0.14 in
- Nominal Moment Strength: M<sub>n</sub> = A<sub>s</sub> * f<sub>c</sub> * (d - a / 2) = 2,597,876 lb*ft
- Design Moment Strength: M<sub>Lu</sub> = 172,800 lb*ft ≤ ΦM<sub>n</sub>

**OK Flexure**
Shear Design Capacity

ACI 318 11-3
Nominal Concrete Shear Strength
\[ V_c = 2 \lambda \sqrt{f'} b d = 150,841 \text{ lb} \]

ACI 318 11-15
Nominal Reinforcing Steel Shear Strength
\[ V_s = A_s f_y d / s = 66,250 \text{ lb} \]

Design Shear Strength
\[ V_a = 46,080 \text{ lb} \leq \Phi V_c \]
\[ \Phi V_c = 162,818 \text{ lb} \]

Check Deflection
Beam Span Length (Center to Center of Support)
\[ l = 180.00 \text{ in} \]
Moment of Inertia
\[ I = 139,968 \text{ in}^4 \]
Modulus of Elasticity * Moment of Inertia
\[ E_c I = 5.37 \times 10^{11} \text{ lb}^*\text{in}^2 \]

Allowable Span Deflection
Floor Members, Dead + Live Load
\[ \Delta \leq l / 240 = 0.750 \text{ in} \]
Floor Members, Live Load
\[ \Delta \leq l / 360 = 0.500 \text{ in} \]

Calculated Deflection
Span Dead + Live Load Deflection
\[ \Delta = 5 \times w_2 \times l^3 / (384 \times E \times I) = 0.010 \text{ in} \]
OK Deflection

Span Live Load Deflection
\[ \Delta = 5 \times w_3 \times l^3 / (384 \times E \times I) = 0.002 \text{ in} \]
OK Deflection
Hoboken Waterfront Evaluation: Segment 3-Transverse Beam

References
American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

Beam Properties
Beam Span Length (Center to Center of Support) \( \ell \) = 12.00 ft
Beam Depth \( h \) = 36.00 in
Minimum Beam Depth for Deflection Solid One-way Slabs Simply Supported \( h_{min} = \ell / 20.0 \) = 7.20 in
Beam Width \( b \) = 36.00 in
Tributary Load Width \( B \) = 26.30 ft

Dead Loads
Uniform Dead Loads
Fill Above Deck \( D_1 \) = 1 * 140 = 140 lb/ft²
Concrete Deck Hollow Core Planks \( D_2 \) = 65 lb/ft²
Total Uniform Loads \( D_T \) = 205 lb/ft²
Uniform Dead Loads per Beam \( b * h * w_c \) = 1,350 lb/ft
Total Dead Load \( D \) = 6,783 lb/ft

Live Loads
Design Floor Load \( L_0 \) = 260 lb/ft²
Is Floor Load Reducible No
Tributary Area for Floor \( A_T \) = 318 ft²

Load Combinations
Design Load \( 1.2 * D + 1.6 * L \) = 114,978 lb
Deflection Load, Dead + Live \( D + L \) = 8,900 lb/ft
Deflection Load, Live \( L \) = 6,000 lb/ft

Design Shear and Moment Assuming Simple Span
Design Shear (Factored Load) \( V_u \) = 564,934 lb
Design Moment (Factored Load) \( M_u \) = 17,669,346 lb*ft

Concrete Properties
Unit Weight of Concrete \( w_c \) = 150 pcf
Specified Concrete Compressive Strength \( f'c \) = 6,000 psi
Modification factor reflecting the reduced mechanical properties of lightweight concrete Normal Weight Concrete
Modulus of Concrete \( E_c \) = 4.000 ksi

Steel Reinforcing Properties
Minimum Specified Yield Stress of Steel ASTM A615 Grade 60 \( f_y \) = 60,000 psi
Assumed Reinforcing Steel Configuration
Clear Cover = 2.00 in
Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement \( d \) = 32.94 in
Tension Steel Reinforcement # of Bars = 4 \( d_t \) = 1.128 in bar area = 1.00 in² \( A_t \) = 4.00 in²
Number of Shear legs per stirrup = 2
Shear Reinforcement # of Shears = 6 \( d_s \) = 0.500 in bar area = 0.20 in² \( A_s \) = 0.40 in²

Flexural Design Capacity
\( R_u = M_u / (b * d^2) \) = 4,139,208 / (36.00 * 32.94²) = 106 psi
Minimum Design Reinforcement Steel Ratio \( 0.85 * f_y / f_{y_e} * [1 - (1 - 2 * A_e / (\Phi * 0.85 * f_y))] \) = 0.00200
Minimum Design Reinforcement Steel Area \( A_e = \rho * b * d \) = 3.75 in²
Minimum Reinforcement of Flexural Members \( A_{e,min} = 3 * \sqrt{\frac{R_u}{f_y}} / f_{y_e} * b * d \) = 260 in²
Depth of Equivalent Rectangular Stress Block \( a = A_e * f_y / (0.85 * f_{y_e} * b) \) = 1.96 in
Distance From Extreme Compression Fiber to Neutral axis \( c = a / f_y \) = 2.31 in
Nominal Moment Strength \( M_n = A_e * f_y * (d - a / 2) \) = 7,669,346 lb*ft
Design Moment Strength \( M_u = 344,934 lb*ft \leq \Phi M_n \) = OK Flexure

ANSWER

**CALCULATIONS**

<table>
<thead>
<tr>
<th>REFERENCES</th>
<th>CALCULATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>IBC Table 1607.9.1</td>
<td>Live Load Element Factor for Girder ( K_{LL} ) = 1.00</td>
</tr>
<tr>
<td>IBC Eq. 16-25</td>
<td>Reduced Live Load ( L = L_0 * B ) = 13,673 lb/ft</td>
</tr>
<tr>
<td>IBC Eq. 16-2</td>
<td>Design Load ( 1.2 * D + 1.6 * L ) = 19,163 lb/ft</td>
</tr>
<tr>
<td>IBC Table 1607.9.1</td>
<td>Deflection Load, Dead + Live ( D + L ) = 13,673 lb/ft</td>
</tr>
<tr>
<td>IBC Table 1607.9.1</td>
<td>Deflection Load, Live ( L ) = 6,890 lb/ft</td>
</tr>
<tr>
<td>IBC Eq. 16-2</td>
<td>Design Shear (Factored Load) ( V_u ) = 4,139,208 lb</td>
</tr>
<tr>
<td>IBC Eq. 16-2</td>
<td>Design Moment (Factored Load) ( M_u ) = 17,669,346 lb*ft</td>
</tr>
<tr>
<td>IBC Eq. 16-25</td>
<td>Design Floor Load ( L_0 ) = 260 lb/ft²</td>
</tr>
<tr>
<td>IBC Table 1607.9.1</td>
<td>Tributary Area for Floor ( A_T ) = 318 ft²</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Flexural Design Capacity ( R_u ) = 4,139,208 / (36.00 * 32.94²) = 106 psi</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Minimum Design Reinforcement Steel Ratio ( 0.85 * f_y / f_{y_e} * [1 - (1 - 2 * A_e / (\Phi * 0.85 * f_y))] ) = 0.00200</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Minimum Design Reinforcement Steel Area ( A_e = \rho * b * d ) = 3.75 in²</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Minimum Reinforcement of Flexural Members ( A_{e,min} = 3 * \sqrt{\frac{R_u}{f_y}} / f_{y_e} * b * d ) = 260 in²</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Depth of Equivalent Rectangular Stress Block ( a = A_e * f_y / (0.85 * f_{y_e} * b) ) = 1.96 in</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Distance From Extreme Compression Fiber to Neutral axis ( c = a / f_y ) = 2.31 in</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Nominal Moment Strength ( M_n = A_e * f_y * (d - a / 2) ) = 7,669,346 lb*ft</td>
</tr>
<tr>
<td>ACI 318 EQ. 10-3</td>
<td>Design Moment Strength ( M_u = 344,934 lb*ft \leq \Phi M_n ) = OK Flexure</td>
</tr>
</tbody>
</table>
Subject: Hoboken Waterfront Evaluation: Segment 3-Transverse Beam

<table>
<thead>
<tr>
<th>Shear Design Capacity</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318 11-3</td>
<td>Nominal Concrete Shear Strength</td>
</tr>
<tr>
<td></td>
<td>( V_c = 2 \cdot \lambda \cdot \sqrt{(f'_c)} \cdot b \cdot d )</td>
</tr>
<tr>
<td></td>
<td>= 149,980 lb</td>
</tr>
<tr>
<td>ACI 318 11-15</td>
<td>Nominal Reinforcing Steel Shear Strength</td>
</tr>
<tr>
<td></td>
<td>( V_s = A_v \cdot f_y \cdot d / s )</td>
</tr>
<tr>
<td></td>
<td>= 131,744 lb</td>
</tr>
<tr>
<td>Design Shear Strength</td>
<td>( V_a = \Phi (V_n + V_s) )</td>
</tr>
<tr>
<td></td>
<td>= 211,293 lb</td>
</tr>
<tr>
<td></td>
<td>OK Shear</td>
</tr>
</tbody>
</table>

Check Deflection

- Beam Span Length (Center to Center of Support)
  \( l = 144.00 \text{ in} \)
- Moment of Inertia
  \( I = 139,968 \text{ in}^4 \)
- Modulus of Elasticity * Moment of Inertia
  \( E_c \cdot I = 5.37E+11 \text{ lb*in}^2 \)
- Allowable Span Deflection
  - Floor Members, Dead + Live Load
    \( \Delta \leq l / 240 = 0.600 \text{ in} \)
  - Floor Members, Live Load
    \( \Delta \leq l / 360 = 0.400 \text{ in} \)

Calculated Deflection

- Span Dead + Live Load Deflection
  \( \Delta = 5 \cdot w_2 \cdot l^3 / (384 \cdot E \cdot I) = 0.012 \text{ in} \)
  OK Deflection
- Span Live Load Deflection
  \( \Delta = 5 \cdot w_3 \cdot l^3 / (384 \cdot E \cdot I) = 0.006 \text{ in} \)
  OK Deflection
Pile Analysis

Spacing Between Piles
- Parallel to Pile Cap
- Perpendicular to Pile Cap

Dead Load
| Concrete Pile Cap | 3.00 ft * 3.00 ft * 12.00 ft * 150.00 lb/ft³ = 16,200 lb |
| Hollowcore Deck Planks | ft * 12.00 ft * 26.50 ft * 65.00 lb/ft³ = 20,670 lb |
| Fill Above Deck | 1.00 ft * 12.00 ft * 26.50 ft * 140.00 lb/ft³ = 44,520 lb |
| Total | 81,390 lb |

Live Load
- Yards and Terraces, Pedestrian
  | 12.00 ft * 26.50 ft * 260.00 lb/ft² = 82,680 lb |

Load Combinations
- IBC EQ 16-1
  | Load 1 | 1.4 * D = 113,946 lb |
  | Load 2 | 1.2 * D + 1.6 * L = 229,956 lb |

All reference AISC 360-05 Specification for Structural Steel Buildings unless otherwise noted.

Member Properties

| HSS14x0.625 | A36 |

Axial Load Design Compression

Full Section

<table>
<thead>
<tr>
<th>Slenderness Ratio</th>
<th>K * L / r_y</th>
</tr>
</thead>
<tbody>
<tr>
<td>OK</td>
<td>32.8</td>
</tr>
<tr>
<td>AISC E3-4</td>
<td>4.71 * (E / F_y) = 265.36</td>
</tr>
<tr>
<td>AISC E3-2</td>
<td>F_a = F_y * [0.658 * (F / F_y)]</td>
</tr>
<tr>
<td>AISC E3-3</td>
<td>F_a = 0.877 * F_a</td>
</tr>
<tr>
<td>AISC E3-1</td>
<td>P_n = A_y * F_a = 833.3 kip</td>
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<tr>
<td>Resistance Factor for Compression</td>
<td>Φ_a = 0.85</td>
</tr>
<tr>
<td>Φ_a * P_n ≥ P_u</td>
<td>708.3 kip</td>
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<tr>
<td>Deteriorated Section (≤20% Section Loss)</td>
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<tr>
<td>Slenderness Ratio</td>
<td>K * L / r_y</td>
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<tr>
<td>OK</td>
<td>33.1</td>
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<tr>
<td>AISC E3-4</td>
<td>4.71 * (E / F_y) = 260.49</td>
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<tr>
<td>AISC E3-2</td>
<td>F_a = F_y * [0.658 * (F / F_y)]</td>
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<tr>
<td>AISC E3-3</td>
<td>F_a = 0.877 * F_a</td>
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<tr>
<td>AISC E3-1</td>
<td>P_n = A_y * F_a = 660.3 kip</td>
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<tr>
<td>Resistance Factor for Compression</td>
<td>Φ_a = 0.85</td>
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<tr>
<td>Φ_a * P_n ≥ P_u</td>
<td>561.2 kip</td>
</tr>
</tbody>
</table>
**Hoboken Waterfront Evaluation: Segment 3-Ground Stone Wall**

### References
- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08
- American Concrete Institute (ACI) Building Code Requirements for Masonry Structures ACI-530-05

### Soil Properties - All soil assumptions to be field verified prior to construction.

**Slope at Top of Wall**

<table>
<thead>
<tr>
<th>Retained Soil</th>
<th>$\gamma_{soil,1}$</th>
<th>$\Phi_1$</th>
<th>$K_a$</th>
<th>$\beta = \tan^{-1} (V / H) = 0.0$ degrees</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{soil,1}$</td>
<td>120 lb/ft$^3$</td>
<td>30 degrees</td>
<td>0.333</td>
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<tr>
<td>Foundation Soil</td>
<td>$\gamma_{soil,2}$</td>
<td>$\Phi_2$</td>
<td>$K_a$</td>
<td>$\mu = 0.58$</td>
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<tr>
<td>$\gamma_{soil,2}$</td>
<td>120 lb/ft$^3$</td>
<td>30 degrees</td>
<td>0.333</td>
<td></td>
</tr>
</tbody>
</table>

**Surcharge Loading**

$S = 0$ lb/ft$^2$

**Equivalent Fluid Pressure**

$\gamma_{soil} = \gamma_{soil,1} \times K_a = 120 \times 0.333 = 40.0$ lb/ft$^2$

$\gamma_s = S \times K_a = 0 \times 0.333 = 0.0$ lb/ft$^2$

**Height of Submerge Soil**

$h = 5.00$ ft

### Wall Properties

**Unit Weight of Modular Blocks (Concrete)** $\gamma = 150$ lb/ft$^3$

**Unit Weight of Modular Blocks (Granite Stone)** $\gamma = 165$ lb/ft$^3$

### Wall Batter

**Wall Geometry**

### Wall Loading

<table>
<thead>
<tr>
<th>Block Weight</th>
<th>Centroid of Block From Front Face of Wall</th>
<th>Retained Soil Width</th>
<th>Retained Soil Load</th>
<th>Centroid of Retained Soil From Front Face of Wall</th>
<th>Resisting Moment Modular Block</th>
<th>Resisting Moment Retained Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Course #1</td>
<td>1,980</td>
<td>3.00</td>
<td>0.50</td>
<td>120</td>
<td>6.25</td>
<td>5,940</td>
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<tr>
<td>Wall Course #2</td>
<td>1,980</td>
<td>3.17</td>
<td>0.33</td>
<td>80</td>
<td>6.33</td>
<td>6,270</td>
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<tr>
<td>Wall Course #3</td>
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<td>Wall Course #4</td>
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<tr>
<td>Wall Course #5</td>
<td>990</td>
<td>2.17</td>
<td>2.3</td>
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<td>5.08</td>
<td>2,145</td>
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<tr>
<td>Wall Course #6</td>
<td>990</td>
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<td>2.67</td>
<td>640</td>
<td>5.17</td>
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<tr>
<td>Wall Course #7</td>
<td>1,650</td>
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<td>2,310</td>
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<td>Wall Course #8</td>
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<td>0.00</td>
<td>0.00</td>
<td>0</td>
<td>0.00</td>
<td>0</td>
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<tr>
<td><strong>Total</strong></td>
<td>11,550</td>
<td>3,870</td>
<td></td>
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<td>33,495</td>
<td>19,249</td>
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<tr>
<td>REF.</td>
<td>WALL COURSE</td>
<td>DEPTH TO TOP OF BLOCK</td>
<td>DEPTH TO BOTTOM OF BLOCK</td>
<td>SOIL PRESSURE TOP</td>
<td>SOIL PRESSURE BOTTOM</td>
<td>FORCE ON BLOCK</td>
</tr>
<tr>
<td>------</td>
<td>-------------</td>
<td>-----------------------</td>
<td>--------------------------</td>
<td>------------------</td>
<td>---------------------</td>
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<tr>
<td></td>
<td></td>
<td>15.50</td>
<td>17.50</td>
<td>620.0</td>
<td>700.0</td>
<td>1,320.0</td>
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<td></td>
<td>Wall Course #2</td>
<td>13.50</td>
<td>15.50</td>
<td>540.0</td>
<td>620.0</td>
<td>1,160.0</td>
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<td></td>
<td>Wall Course #3</td>
<td>11.50</td>
<td>13.50</td>
<td>460.0</td>
<td>540.0</td>
<td>1,000.0</td>
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<td></td>
<td>Wall Course #4</td>
<td>9.50</td>
<td>11.50</td>
<td>380.0</td>
<td>460.0</td>
<td>840.0</td>
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<td>Wall Course #5</td>
<td>7.50</td>
<td>9.50</td>
<td>300.0</td>
<td>380.0</td>
<td>680.0</td>
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<td></td>
<td>Wall Course #6</td>
<td>5.50</td>
<td>7.50</td>
<td>220.0</td>
<td>300.0</td>
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<td></td>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6,125.0</td>
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</tbody>
</table>

**Total Wall Load Summary**

- **Total Wall Weight (Wall + Retained Soil)**
  
  \( W = \text{Total Wall Load} \)

- **Driving Force**
  
  \( P_a = P_a \)

- **Resisting Moment**
  
  \( M_r = M_r \)

- **Overturning Moment**
  
  \( M_o = M_o \)

**Check Wall Stability**

- **Factor of Safety for Overturning**
  
  \[ F_{So} = \frac{M_o}{M_r} = \frac{52,744}{35,729} = 1.48 \]

  \( F_{So} \geq 1.5 \quad \text{CHECK} \)

- **Force resisting**
  
  \[ W \mu = 15,420 \times 0.58 = 8,903 \text{ lb} \]

- **Factor of Safety for Sliding**
  
  \[ F_{Ss} = \frac{W \mu}{P_a} = \frac{8,903}{6,125} = 1.45 \]

  \( F_{Ss} \geq 1.5 \quad \text{CHECK} \)
### References
- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08
- American Concrete Institute (ACI) Building Code Requirements for Masonry Structures ACI-530-05

### Soil Properties - All soil assumptions to be field verified prior to construction.

#### Slope at Top of Wall
\[ \beta = \tan^{-1}(V / H) = 0.0 \ degrees \]

<table>
<thead>
<tr>
<th>Unit Weight of Soil</th>
<th>Effective Angle of Internal Friction</th>
<th>Coefficient of Friction</th>
<th>Rankine Active Pressure</th>
<th>Allowable Soil Bearing Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retained Soil</td>
<td>( \gamma_{\text{soil-1}} = 120 ) lb/ft(^3)</td>
<td>( \Phi_1 = 30 ) degrees</td>
<td>( K_a = 0.333 )</td>
<td></td>
</tr>
<tr>
<td>Foundation Soil</td>
<td>( \gamma_{\text{soil-2}} = 120 ) lb/ft(^3)</td>
<td>( \Phi_2 = 30 ) degrees</td>
<td>( \mu = 0.58 )</td>
<td>( 4,000 ) lb/sf</td>
</tr>
</tbody>
</table>

#### Surcharge Loading
\[ S = 25 \] lb/ft\(^2\)

#### Equivalent Fluid Pressure
\[ \gamma_s = \gamma_{\text{soil-1}} \times K_a = 120 \times 0.333 = 40.0 \] lb/sf/ft
\[ \gamma_s = S \times K_a = 25 \times 0.333 = 8.3 \] lb/sf

#### Height of Submerge Soil
\[ h = 5.00 \] ft

### Wall Properties
- Unit Weight of Modular Blocks(Concrete) \( \gamma = 150 \) lbs/ft\(^3\)
- Unit Weight of Modular Blocks(Granite Stone) \( \gamma = 165 \) lbs/ft\(^3\)

### Modular Block Size

<table>
<thead>
<tr>
<th>Type</th>
<th>Length</th>
<th>Depth</th>
<th>Height</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Beam</td>
<td>6.0</td>
<td>3.0</td>
<td>6.0</td>
<td>2,700</td>
</tr>
<tr>
<td>Block #1</td>
<td>6.0</td>
<td>3.0</td>
<td>2.0</td>
<td>990</td>
</tr>
<tr>
<td>Block #2</td>
<td>6.0</td>
<td>6.0</td>
<td>2.0</td>
<td>1,980</td>
</tr>
<tr>
<td>None</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Wall Batter
0.0 inches per Block

### Wall Geometry

<table>
<thead>
<tr>
<th>Modular Block Type (Bottom of Wall)</th>
<th>Block Height</th>
<th>Bottom of Block Elev</th>
<th>Top of Block Elev</th>
<th>Block Depth</th>
<th>Location Front of Block</th>
<th>Location Back of Block</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Course #1</td>
<td>Block #2</td>
<td>2.00</td>
<td>0.00</td>
<td>2.00</td>
<td>6.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Wall Course #2</td>
<td>Block #2</td>
<td>2.00</td>
<td>2.00</td>
<td>4.00</td>
<td>6.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Wall Course #3</td>
<td>Block #2</td>
<td>2.00</td>
<td>4.00</td>
<td>6.00</td>
<td>0.00</td>
<td>6.00</td>
</tr>
<tr>
<td>Wall Course #4</td>
<td>Block #2</td>
<td>2.00</td>
<td>6.00</td>
<td>8.00</td>
<td>6.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Wall Course #5</td>
<td>Block #1</td>
<td>2.00</td>
<td>8.00</td>
<td>10.00</td>
<td>3.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Wall Course #6</td>
<td>Concrete Beam</td>
<td>6.00</td>
<td>10.00</td>
<td>16.00</td>
<td>3.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Wall Course #7</td>
<td>None</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Wall Course #8</td>
<td>None</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

#### Wall Design Height
\[ H = 16.00 \] ft

#### Width of Bottom Wall (Block and Retained Soil)
\[ B = 6.00 \] ft

### Wall Loading

<table>
<thead>
<tr>
<th>Block Weight</th>
<th>Centroid of Block From Front Face of Wall</th>
<th>Retained Soil Width</th>
<th>Retained Soil Load</th>
<th>Centroid of Retained Soil From Front Face of Wall</th>
<th>Resisting Moment Modular Block</th>
<th>Resisting Moment Retained Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Course #1</td>
<td>1,980</td>
<td>3.00</td>
<td>0.00</td>
<td>0</td>
<td>6.00</td>
<td>5,940</td>
</tr>
<tr>
<td>Wall Course #2</td>
<td>1,980</td>
<td>3.00</td>
<td>0.00</td>
<td>0</td>
<td>6.00</td>
<td>5,940</td>
</tr>
<tr>
<td>Wall Course #3</td>
<td>1,980</td>
<td>3.00</td>
<td>0.00</td>
<td>0</td>
<td>6.00</td>
<td>5,940</td>
</tr>
<tr>
<td>Wall Course #4</td>
<td>1,980</td>
<td>3.00</td>
<td>0.00</td>
<td>0</td>
<td>6.00</td>
<td>5,940</td>
</tr>
<tr>
<td>Wall Course #5</td>
<td>990</td>
<td>1.50</td>
<td>3.00</td>
<td>720</td>
<td>4.50</td>
<td>1,485</td>
</tr>
<tr>
<td>Wall Course #6</td>
<td>2,970</td>
<td>1.50</td>
<td>3.00</td>
<td>2,160</td>
<td>4.50</td>
<td>4,455</td>
</tr>
<tr>
<td>Wall Course #7</td>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wall Course #8</td>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>11,880</td>
<td>2,880</td>
<td>29,700</td>
<td>12,960</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Hoboken Waterfront Evaluation: Segment 4-Stone Wall

**REF.**

<table>
<thead>
<tr>
<th>Wall Course</th>
<th>Depth to Top of Block</th>
<th>Depth to Bottom of Block</th>
<th>Soil Pressure Top</th>
<th>Soil Pressure Bottom</th>
<th>Force on Block</th>
<th>Location of Force From Bottom</th>
<th>Overturning Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>14.00</td>
<td>16.00</td>
<td>568.3</td>
<td>648.3</td>
<td>1,216.7</td>
<td>0.98</td>
<td>1,190.0</td>
</tr>
<tr>
<td>#2</td>
<td>12.00</td>
<td>14.00</td>
<td>488.3</td>
<td>568.3</td>
<td>1,056.7</td>
<td>2.97</td>
<td>3,143.3</td>
</tr>
<tr>
<td>#3</td>
<td>10.00</td>
<td>12.00</td>
<td>408.3</td>
<td>488.3</td>
<td>896.7</td>
<td>4.97</td>
<td>4,456.7</td>
</tr>
<tr>
<td>#4</td>
<td>8.00</td>
<td>10.00</td>
<td>328.3</td>
<td>408.3</td>
<td>736.7</td>
<td>6.96</td>
<td>5,130.0</td>
</tr>
<tr>
<td>#5</td>
<td>6.00</td>
<td>8.00</td>
<td>248.3</td>
<td>328.3</td>
<td>576.7</td>
<td>8.95</td>
<td>5,163.3</td>
</tr>
<tr>
<td>#6</td>
<td>0.00</td>
<td>6.00</td>
<td>8.3</td>
<td>248.3</td>
<td>770.0</td>
<td>12.06</td>
<td>9,290.0</td>
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<tr>
<td>#7</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>#8</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
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<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5,253.3</td>
<td>28,373.3</td>
<td></td>
</tr>
</tbody>
</table>

**Total Wall Load Summary**

- **Total Wall Weight (Wall + Retained Soil)**
  
- **Driving Force**: $P_a = 5,253\text{ lbs}$
- **Resisting Moment**: $M_r = 42,660\text{ lb*ft}$
- **Overturning Moment**: $M_o = 28,373\text{ lb*ft}$

**Check Wall Stability**

- **Factor of Safety for Overturning**: $FS_o = M_r / M_o = 42,660 / 28,373 = 1.5$  
  
- **Force resisting**: $W \times \mu = 14,760 \times 0.58 = 8,522\text{ lb}$
- **Factor of Safety for Sliding**: $FS_s = W \times \mu / P_a = 8,522 / 5,253 = 1.62$

- $FS_o \geq 1.5$: OK
- $FS_s \geq 1.5$: OK
**References**

American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

**Slab Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab Span Length (Center to Center of Support)</td>
<td>12.00 ft</td>
</tr>
<tr>
<td>Slab Depth</td>
<td>12.00 in</td>
</tr>
<tr>
<td>Minimum Slab Depth for Deflection</td>
<td>7.20 in</td>
</tr>
<tr>
<td>Slab Span Length</td>
<td>20.00 ft</td>
</tr>
<tr>
<td>Slab Unit Width</td>
<td>12.00 in</td>
</tr>
</tbody>
</table>

**Dead Loads**

- **Uniform Dead Load**  
  - Slab Load: \( D_1 = \) lb/ft
  - Miscellaneous: \( D_2 = \) lb/ft
  - Total Uniform Loads: \( D_T = \) lb/ft

- **Weight of Slab per foot**  
  - \( b \times h \times w_c = \) lb/ft

- **Total Dead Load**: \( D = \) lb/ft

**Live Loads**

- **Design Floor Live Load**  
  - \( L_0 = \) lb/ft

- **Is Floor Load Reducible**

- **Tributary Area for Floor**  
  - \( A_T = \) ft²

- **Design Floor Live Load, Factored Load**  
  - \( V = w_1 \times \ell / 2 = \) lb

- **Design Moment, Factored Load**  
  - \( M = w_1 \times \ell^2 / 8 = \) lb*ft

**Concrete Properties**

- **Unit Weight of Concrete**  
  - \( w_c = \) pcf

- **Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth**  
  - \( \beta_1 = \) 0.85

- **Modulus of Concrete**  
  - \( E_c = \) psi

- **Strength-Reduction Factor - Shear**  
  - \( \Phi = \) 0.75

- **Strength-Reduction Factor - Flexural sections in pretensioned members**  
  - \( \Phi = \) 0.90

**Stein Reinforcing Properties**

- **Minimum Specified Yield Stress of Steel**  
  - ASTM A615 Grade 60

- **Assumed Reinforcing Steel**  
  - \( f_y = \) 60,000 psi

- **Assuming 10% Steel Section Loss**  
  - \( A_{sl} = \) in²

- **Nominal Moment Strength**  
  - \( M_n = \) lb*ft

- **Design Moment Strength**  
  - \( \Phi M_n = \) lb*ft

**Flexural Design Capacity**

- **Flexural Design Capacity**  
  - \( R_e = M_i / (b^2 d^2) = 76,896 \) psi

- **Minimum Design Reinforcement Steel Ratio**  
  - \( \rho = 0.00180 \) in

- **Minimum Reinforcement of Flexural Members**  
  - \( A_{s, min} = 3 \times \sqrt{d} / f_y = 0.00180 \times 12.00 \times 8.19 = 0.18 \) in²

- **OK Flexure**

**Load Combinations**

- **Design Load**  
  - \( L = w_1 \times \ell / 2 = \) lb

- **Deflection Load**  
  - \( L = w_3 = \) lb/ft

**Design Shear and Moment Assuming Simple Span**

- **Design Shear (Factored Load)**  
  - \( V = \) lb

- **Design Moment (Factored Load)**  
  - \( M = \) lb*ft

**Calculation Details**

- **Design Load 1.2 * D + 1.6 * L = w**  
  - \( w_1 = 356 \) lb/ft

- **Deflection Load, Dead + Live D + L = w**  
  - \( w_2 = 20.00 \) lb/ft

- **Deflection Load, Live L = w**  
  - \( w_3 = 110 \) lb/ft

**Assumptions and Design Values**

- **Concrete Unit Weight**  
  - \( w_c = 150 \) pcf

- **Slab Span**  
  - \( \ell = 12.00 \) ft

- **Slab Depth**  
  - \( h = 12.00 \) in

- **Minimum Slab Depth for Deflection**  
  - \( h_{min} = \ell / 20.00 = 7.20 \) in

- **Slab Span Length**  
  - \( \ell = 20.00 \) ft

- **Slab Unit Width**  
  - \( b = 12.00 \) in

- **Design Floor Live Load**  
  - \( L_0 = 110 \) lb/ft

- **Tributary Area for Floor**  
  - \( A_T = 12 \) ft²

- **Live Load Element Factor for Slab**  
  - \( K_{LL} = 1.00 \)

- **Reduced Live Load**  
  - \( L_1 = L_0 \times (0.25 + 15 / \sqrt{K_{LL} \times A_T}) = 110 \) lb/ft

- **Design Moment, Factored Load**  
  - \( M = w_1 \times \ell^2 / 8 = 6,408 \) lb*ft

**Variables and Formulas**

- **Concrete Properties**  
  - \( w_c = 4,000 \) psi

- **Steel Reinforcing Properties**  
  - \( f_y = 60,000 \) psi

**Design Shear and Moment Assuming Simple Span**

- **Design Moment, Factored Load**  
  - \( M_0 = w_1 \times \ell^2 / 8 = 6,408 \) lb*ft

- **Minimum Reinforcement of Flexural Members**  
  - \( A_s, min = 0.28 \) in²

- **Nominal Moment Strength**  
  - \( M_n = 133,625 \) lb*ft

**OK Flexure**
**Hoboken Waterfront Evaluation: Segment 6- Slab**

**Shear Design Capacity**

**ACI 318 11-3**
Nominal Concrete Shear Strength

\[ V_C = 2 \lambda \lambda' \sqrt{f_c'} b d = 12,428 \text{ lb} \]

**ACI 318 11-15**
Nominal Reinforcing Steel Shear Strength

\[ V_s = \Phi \Phi(V_c + V_s) = 9,321 \text{ lb} \]

Design Shear Strength

\[ V_a = 2,136 \text{ lb} \leq \Phi V_s \]

**OK Shear**

**Check Deflection**

Beam Span Length (Center to Center of Support)

\[ \ell = 144.00 \text{ in} \]

Moment of Inertia

\[ I = 1,728 \text{ in}^4 \]

Modulus of Elasticity * Moment of Inertia

\[ E_c I = 6.63 \times 10^9 \text{ lb*in}^2 \]

**IBC Table 1604.3**

Floor Members, Dead + Live Load

\[ \Delta \leq \ell \ell / 240 = 0.600 \text{ in} \]

Floor Members, Live Load

\[ \Delta \leq \ell / 360 = 0.400 \text{ in} \]

**Calculated Deflection**

Span Dead + Live Load Deflection

\[ \Delta = 5w \ell^3 / (384 E I) = 0.018 \text{ in} \]

Span Live Load Deflection

\[ \Delta = 5w_3 \ell^3 / (384 E I) = 0.008 \text{ in} \]

**OK Deflection**
Beam Properties

Beam Span Length (Center to Center of Support) \( l = 18.00 \) ft

Beam Depth \( h = 24.00 \) in

ACI Table 9.5(a) Minimum Beam Depth for Deflection \( h_{\text{min}} = \frac{l}{20.0} = 10.80 \) in

Beam Width \( b = 36.00 \) in

Tributary Load Width \( B = 12.00 \) ft

Dead Loads

Uniform Dead Loads

Floor Load \( D_1 = \) lb/ft

Miscellaneous \( D_2 = \) lb/ft

Total Uniform Loads \( D_T = \) lb/ft

Uniform Dead Loads per Beam \( b \times h \times w_c = \) lb/ft

Total Dead Load \( D = \) lb/ft

Live Loads

Design Floor Live Load \( L_0 = \) lb/ft

Is Floor Load Reducible

Tributary Area for Floor \( A_T = \) ft²

Live Load Element Factor for Girder \( K_{LL} = 1.00 \)

Load Combinations

Design Load \( 1.2 \times D + 1.6 \times L = w_1 = 5352 \) lb/ft

Deflection Load, Dead + Live \( D + L = w_2 = 4020 \) lb/ft

Deflection Load, Live \( L = w_3 = 1320 \) lb/ft

Design Shear and Moment Assuming Simple Span

Design Shear (Factored Load) \( V_u = w_1 \times \frac{l}{2} = 48168 \) lb

Design Moment (Factored Load) \( M_u = w_1 \times \frac{l^2}{8} = 216756 \) lb*ft

Concrete Properties

Unit Weight of Concrete \( w_c = 150 \)pcf

Specified Concrete Compressive Strength \( f'_c = 4000 \) psi

Steel Reinforcing Properties

Minimum Specified Yield Stress of Steel \( f_y = 60000 \) psi

Assumed Reinforcing Steel

Clear Cover = 2.00 in

Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement \( d = 20.94 \) in

Tension Steel Reinforcement

\# of Bars = 3 \( d_b = 1.128 \) in \( A_s = 3.00 \) in²

Shear Reinforcement

\# of Bars = 2 \( d_b = 0.500 \) in \( A_s = 0.40 \) in²

Assuming 20% Steel Section Loss \( A_{s,\text{min}} = 2.40 \) in²

Assuming 20% Steel Section Loss \( A_{s,\text{min}} = 0.32 \) in²

Flexural Design Capacity

\( \frac{R_u}{M_u} = \frac{M_u}{l/b^2} = 2601.072 \times \frac{0.360 \times 20.94}{20.94} = 165 \) psi

Minimum Design Reinforcement Steel Ratio \( 0.85 \times f'_y / f_y = 1 = 0.00314 \)

Minimum Design Reinforcement Steel Area \( A_n = 0.00314 \times 36.00 \times 20.94 = 2.37 \) in²

Minimum Reinforcement of Flexural Members

\( A_{s,\text{min}} = 3 \times \frac{f'_y}{f_y} \times b \times d = 200 \) in²

Depth of Equivalent Rectangular Stress Block \( a = A_n \times f_y / (0.85 \times f'_y) = 1.18 \) in

Distance From Extreme Compression Fiber to Neutral axis \( c = a / f_y = 1.38 \) in

Nominal Moment Strength \( M_n = A_n \times f_y \times (d - a / 2) = 2930.078 \) lb*ft

Design Moment Strength \( \Phi M_n = 219756 \) lb*ft

Answer: OK Flexure
### Shear Design Capacity

**Nominal Concrete Shear Strength**

\[ V_c = 2 \lambda \sqrt{f'_c} b d \]

**Nominal Reinforcing Steel Shear Strength**

\[ V_s = A_v f_y d / s \]

**Design Shear Strength**

\[ \Phi V_n = \Phi (V_c + V_s) \]

\[ V_n = 48,168 \text{ lb} \leq \Phi V_n \]

**OK Shear**

### Check Deflection

**Beam Span Length (Center to Center of Support)**

\[ l = 216.00 \text{ in} \]

**Moment of Inertia**

\[ I = 41,472 \text{ in}^4 \]

**Modulus of Elasticity * Moment of Inertia**

\[ E_c I = 1.59E+11 \text{ lb*in}^2 \]

**Allowable Span Deflection**

Floor Members, Dead + Live Load

\[ \Delta \leq l / 240 = 0.900 \text{ in} \]

Floor Members, Live Load

\[ \Delta \leq l / 360 = 0.600 \text{ in} \]

**Calculated Deflection**

**Span Dead + Live Load Deflection**

\[ \Delta = 5 \times w_2 \times l^3 / (384 \times E \times I) = 0.060 \text{ in} \]

**OK Deflection**

**Span Live Load Deflection**

\[ \Delta = 5 \times w_3 \times l^3 / (384 \times E \times I) = 0.020 \text{ in} \]

**OK Deflection**
### Pile Analysis

**Spacing Between Piles**
- Parallel to Pile Cap: 12.00 ft
- Perpendicular to Pile Cap: 18.00 ft

**Dead Load**
- Concrete Pile Cap: $2 \text{ ft} \times 3.00 \text{ ft} \times 12.00 \text{ ft} = 150.00 \text{ lb/ft}^3 = 10,800 \text{ lb}$
- Prestressed Deck Planks: $1.00 \text{ ft} \times 12.00 \text{ ft} \times 18.00 \text{ ft} = 150.00 \text{ lb/ft}^3 = 32,400 \text{ lb}$
- Soil Fill: $0.00 \text{ ft} \times 12.00 \text{ ft} \times 18.00 \text{ ft} = 120.00 \text{ lb/ft}^3 = 43,200 \text{ lb}$

**Total Dead Load**
- IBC Tab. 1607.1: $12.00 \text{ ft} \times 18.00 \text{ ft} = 100.00 \text{ lb/ft}^2 = 21,600 \text{ lb}$

**Live Load**
- Yards and Terraces, Pedestrian: $12.00 \text{ ft} \times 18.00 \text{ ft} = 100.00 \text{ lb/ft}^2 = 21,600 \text{ lb}$

**Load Combinations**
- Load 1: $1.4 \text{ } D = 60,480 \text{ lb}$
- Load 2: $1.2 \text{ } D + 1.6 \text{ } L = 86,400 \text{ lb}$

All reference AISC 360-05 Specification for Structural Steel Buildings unless otherwise noted

### Effective Length Factor

- $K_e = 1.00$

### Unbraced Column

- $L_{eu} = 17.50 \text{ ft}$

### Design Axial Load

- $P_u = 86.4 \text{ kip}$

### Member Properties

<table>
<thead>
<tr>
<th>Member</th>
<th>HP12x53</th>
<th>A36</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Section</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Specified minimum yield stress</td>
<td>$F_y = 36 \text{ ksi}$</td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity of Steel</td>
<td>$E = 29,000 \text{ ksi}$</td>
<td></td>
</tr>
<tr>
<td>Beam depth</td>
<td>$d = 11.78 \text{ in}$</td>
<td></td>
</tr>
<tr>
<td>Web thickness</td>
<td>$t_w = 0.435 \text{ in}$</td>
<td></td>
</tr>
<tr>
<td>Flange width</td>
<td>$b_f = 12.045 \text{ in}$</td>
<td></td>
</tr>
<tr>
<td>Flange Thickness</td>
<td>$t_f = 0.435 \text{ in}$</td>
<td></td>
</tr>
</tbody>
</table>

| Deteriorated Section (~50% Section Loss) | | |
| Beam depth | $d = 11.32 \text{ in}$ | |
| Web thickness | $t_w = 0.205 \text{ in}$ | |
| Flange width | $b_f = 14.388 \text{ in}$ | |
| Flange Thickness | $t_f = 0.2175 \text{ in}$ | |

### Axial Load Design Compression

**Full Section**

| Slenderness Ratio | $K^* \frac{L}{r_f}$ | 73.4 |
| | $K^* \frac{L}{r_f} \leq 200$ | OK |
| | $4.71 \sqrt[3]{\left(\frac{E}{F_y}\right)} = 133.7$ | |
| AISC E3-4 | $F_a = \frac{(\pi^2 \cdot E)}{(K^* \frac{L}{r_f})^2}$ | 53.09 |
| AISC E3-2 | if $K^* \frac{L}{r_f} \leq 4.71 \sqrt[3]{\left(\frac{E}{F_y}\right)}$ | $F_{cr} = F_y * [0.658 \left(\frac{F_y}{F_e}\right)]$ |
| AISC E3-3 | if $K^* \frac{L}{r_f} > 4.71 \sqrt[3]{\left(\frac{E}{F_y}\right)}$ | $F_{cr} = 0.877 \cdot F_a$ |

| Nominal Compression Axial Strength | $P_n = A_f \cdot F_{cr}$ | 420.1 kip |
| Resistance Factor for Compression | $\Phi = 0.85$ | |
| $\Phi P_n \geq P_u$ | 357.1 kip | OK |

**Deteriorated Section**

| Slenderness Ratio | $K^* \frac{L}{r_f}$ | 59.2 |
| | $K^* \frac{L}{r_f} \leq 200$ | OK |
| | $4.71 \sqrt[3]{\left(\frac{E}{F_y}\right)} = 133.7$ | |
| AISC E3-4 | $F_a = \frac{(\pi^2 \cdot E)}{(K^* \frac{L}{r_f})^2}$ | 81.69 |
| AISC E3-2 | if $K^* \frac{L}{r_f} \leq 4.71 \sqrt[3]{\left(\frac{E}{F_y}\right)}$ | $F_{cr} = F_y * [0.658 \left(\frac{F_y}{F_e}\right)]$ |
| AISC E3-3 | if $K^* \frac{L}{r_f} > 4.71 \sqrt[3]{\left(\frac{E}{F_y}\right)}$ | $F_{cr} = 0.877 \cdot F_a$ |

| Nominal Compression Axial Strength | $P_n = A_f \cdot F_{cr}$ | 256.8 kip |
| Resistance Factor for Compression | $\Phi = 0.85$ | |
| $\Phi P_n \geq P_u$ | 218.3 kip | OK |
Slope at Top of Wall \( \beta = \tan(V/H) = 0.0 \) degrees

Angle of Fill \( \beta' = \tan(V/H) = 0.0 \) degrees

Angle of Back Face of Wall \( \theta = 90.0 \) degrees

Soil 1

Unit Weight \( \gamma_1 = 0.120 \) kip/ft

Effective Angle of Internal Friction \( \Phi_{11} = 30.0 \) degrees

Steel sheet piles against silty sand, gravel or sand mixed with silt or clay

Friction Angle Between Fill and Wall \( \delta_1 = 17.0 \) degrees

Coulomb Active Pressure \( K_{a1} = 0.30 \)

Active Earth Pressure \( K_{e1} \gamma_1 = 0.036 \) kips

Soil 2

Unit Weight \( \gamma_2 = 0.120 \) kip/ft

Effective Angle of Internal Friction \( \Phi_{12} = 30.0 \) degrees

Steel sheet piles against silty sand, gravel or sand mixed with silt or clay

Friction Angle Between Fill and Wall \( \delta_2 = 14.0 \) degrees

Coulomb Active Pressure \( K_{a2} = 0.30 \)

Passive Pressure Coefficient \( K_p = 6.5 \)

Reduction Factor for \( K_p \) \( R = 0.726 \)

Passive Pressure Coefficient \( K_p \gamma_2 = 4.72 \)

Active Earth Pressure \( K_{e2} \gamma_2 = 0.036 \) ksf/ft

Live Load Surcharge \( q_L = 0.25 \) kip/ft

Soil Load Surcharge \( q_s = 0.00 \) kip/ft

Design Height \( H = 11.50 \) ft

**Figure 3.11.5.6-3 Unfactored Simplified Earth Pressure Distributions for Permanent Nongravity Cantilevered Walls with Continuous Vertical Wall Elements Embedded In Granular Soil Modified After Teng (1962).**

**Forces on Sheet Pile**

Design Limit State = Strength I

Load Category

| \( P_{Li} \) | \( PL \) | \( K_{a1} \gamma_1 \gamma_1 \gamma_1 \) | \( H \) | 1.51 | 1.51 kips |
| \( P_{L2} \) | \( PL \) | \( K_{a2} \gamma_2 \gamma_2 \gamma_2 \) | \( D_0 \) | 1.63 kips |
| \( P_{L1} \) | \( ES_{max} \) | \( K_{a1} \gamma_1 \) | \( H \) | 0.00 kips |
| \( P_{L2} \) | \( ES_{max} \) | \( K_{a2} \gamma_2 \) | \( D_0 \) | 0.00 kips |
| \( P_{H1} \) | \( EH_{max} \) | \( 1/2 \times K_{a1} \gamma_1 \) | \( 1/2 \times H \) | 3.56 kips |
| \( P_{H2} \) | \( EH_{max} \) | \( K_{a2} \gamma_2 \gamma_2 \gamma_2 \) | \( H \) | 7.64 kips |
| \( P_{H3} \) | \( EH_{max} \) | \( 1/2 \times K_{a2} \gamma_2 \gamma_2 \gamma_2 \) | \( D_0 \) | 4.13 kips |
| \( P_f \) | \( EH_{max} \) | \( 1/2 \times K_{a2} \gamma_2 \gamma_2 \gamma_2 \) | \( D_0 \) | 38.68 kips |

**Moments Taken About Tip of Sheet Pile**

**Overturning Moments**

| \( M_{i1} \) | \( P_{L1} \times (H/2 + D_0) \) | 8.66 + 1.51 \* \( D_0 \) | 27.2 kip*ft |
| \( M_{i2} \) | \( P_{L2} \times D_0 / 2 \) | 0.07 \* \( D_0 \) | 10.0 kip*ft |
| \( M_{i3} \) | \( P_{L1} \times (H/2 + D_0) \) | 0.00 + 0.00 \* \( D_0 \) | 0 kip*ft |
| \( M_{i2} \) | \( P_{L2} \times D_0 / 2 \) | 0.00 \* \( D_0 \) | 0 kip*ft |
| \( M_1 \) | \( P_{L1} \times (H/2 + D_0) \) | 13.66 + 3.56 \* \( D_0 \) | 57.6 kip*ft |
| \( M_2 \) | \( P_{L2} \times D_0 / 2 \) | 0.31 \* \( D_0 \) | 47.0 kip*ft |
| \( M_3 \) | \( P_{L2} \times D_0 / 2 \) | 0.01 \* \( D_0 \) | 17.0 kip*ft |
| Total Overturning Moments | = | 22.32 + 5.07 \* \( D_0 \) + 0.38 \* \( D_0 \) + 0.01 \* \( D_0 \) | 158.9 kip*ft |

**Resisting Moments**

| \( M_4 \) | \( P_f \times \) \( D/3 \) | 0.08 \* \( D_0 \) | 158.9 kip*ft |
## Hoboken Waterfront Evaluation: Segment 7.1-Sheet Pile Bulkhead

### Calculations

#### Total Moments

Total overturning moments \( M = 0 \) = \( 22.32 + 5.07 \times D_0 + 0.38 \times D_0^2 + 0.08 \times D_0^3 \) = 0 kip*ft

Solve for \( D_0 \)

\[
D_0 = 12.3 \text{ ft}
\]

Depth of Zero Shear

\[
0 = 2.07 + 0.75 \times x - 0.23 \times x^2
\]

\( x = 6.654 \text{ ft} \)

#### Moment @ Ground Line

\[
M_g = P_{a1} \times \frac{H}{3}
\]

#### Moment @ Zero Shear

\[
M_0 = \left\{ P_{a1} \times \frac{H}{3} + x \right\} + \left\{ K_{a1} \times \gamma_{1}' \times \frac{H}{2} \times x \right\} + \left\{ 0.5 \times \gamma_{2}' \times \frac{x^3}{3} \right\} + \left\{ P_{a2} \times \gamma_{1}' \times \frac{x^3}{3} \right\} - \left\{ 0.5 \times \gamma_{2}' \times \frac{x^3}{3} \right\}
\]

\( M_0 = \text{kip*ft} \)

### Structural Steel Properties

- Rolled Section Structural Steel Material: ASTM A572 Grade 50
- Modulus of Elasticity of Steel Beam: \( E_s = 29,000 \text{ ksi} \)
- Resistance Factor for Flexure and Shear in Steel: \( \Phi_f = 1.0 \)
- Hybrid Factor: \( R_h = 1.0 \)

### Sheet Pile Size

<table>
<thead>
<tr>
<th>Sheet Pile Width</th>
<th>w</th>
<th>22</th>
<th>in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet Pile Height</td>
<td>d</td>
<td>9</td>
<td>in</td>
</tr>
<tr>
<td>Flange Thickness</td>
<td>t_f</td>
<td>0.375</td>
<td>in</td>
</tr>
<tr>
<td>Web Thickness</td>
<td>t_w</td>
<td>0.375</td>
<td>in</td>
</tr>
<tr>
<td>Weight / Pile</td>
<td>40.3</td>
<td>lb/ft</td>
<td></td>
</tr>
<tr>
<td>Weight / Foot</td>
<td>22</td>
<td>lb/ft²</td>
<td></td>
</tr>
</tbody>
</table>

### Sheet Pile Loads

Design Moment

\( M_u = 50.4 \text{ kip-ft} \)

Design Maximum Flexural Stress in Sheet Pile

Utilization Percentage

\( f_{us} = 33.39 \leq \Phi_r \times R_h \times F_{yd} = 50.00 \) 

OK Flexure

66.8%

### Check Strength Limit State

#### Flexural Design for Continuously Braced Flanges in Tension and Compression

6.10.8.1.3-1

\[
f_{us} = 44.52 \leq \Phi_r \times R_h \times F_{yd} = 50.00
\]

OK Flexure

89.0%

### Sheet Pile Loads

Steel Elastic Section Modulus per foot

\( S = 13.6 \text{ in}^3/\text{ft} \)

Design Maximum Flexural Stress in Sheet Pile Based Upon Deteriorated Steel Section

Utilization Percentage

\( f_{us} = 44.52 \leq \Phi_r \times R_h \times F_{yd} = 50.00 \) 

OK Flexure

89.0%
Slope at Top of Wall: \( V \) = 1, \( H \) = 0

Angle of Fill: \( \beta = \tan(\frac{H}{V}) \) = 0.0 degrees

Slope at Bot. of Wall: \( V \) = 1, \( H \) = 0

Angle of Fill: \( \beta' = \tan(\frac{V}{H}) \) = 0.0 degrees

Angle of Back Face of Wall: \( \theta \) = 90.0 degrees

Soil 1
Unit Weight: \( \gamma \) = 1.20 kips/ft³

Effective Angle of Internal Friction: \( \Phi \) = 30.0 degrees

Steel sheet piles against silty sand, gravel or sand mixed with silt or clay

Friction Angle Between Fill and Wall: \( \delta \) = 17.0 degrees

Coulomb Active Pressure: \( K_a \)

Active Earth Pressure: \( K_p \)

Soil 2
Unit Weight: \( \gamma \) = 1.20 kips/ft³

Effective Angle of Internal Friction: \( \Phi \) = 30.0 degrees

Steel sheet piles against silty sand, gravel or sand mixed with silt or clay

Friction Angle Between Fill and Wall: \( \delta \) = 14.0 degrees

Coulomb Active Pressure: \( K_a \)

Active Earth Pressure: \( K_p \)

Reduction Factor for \( K_p \): \( R \) = 0.726

Passive Pressure Coefficient: \( K_0 \)

Live Load Surcharge: \( q_L \) = 0.25 kips/ft²

Soil Load Surcharge: \( q_S \) = 0.00 kips/ft²

Design Height: \( H \) = 9.50 ft

\[ \begin{align*}
M_{L1} &= P_{L1} \times (H/2 + D_0) = 5.91 + 1.24 \times D_0 = 19.0 \text{ kip*ft} \\
M_{L2} &= P_{L2} \times D_0/2 = 7.00 \times D_0 = 0.00 \text{ kip*ft} \\
M_{L3} &= P_{L3} \times (H/2 + D_0) = 0.00 + 0.00 \times D_0 = 0.00 \text{ kip*ft} \\
M_{L4} &= P_{L4} \times (H/2 + D_0) = 2.43 \times D_0 = 2.43 \text{ kip*ft} \\
M_{L5} &= P_{L5} \times (H/2 + D_0) = 5.11 \times D_0 = 5.11 \text{ kip*ft} \\
M_{L6} &= P_{L6} \times D_0/2 = 0.00 \times D_0 = 0.00 \text{ kip*ft} \\
M_{L7} &= P_{L7} \times D_0/2 = 0.26 \times D_0^2 = 0.26 \text{ kip*ft} \\
M_{L8} &= P_{L8} \times D_0/2 = 0.01 \times D_0^2 = 0.01 \text{ kip*ft} \\
M_{L9} &= P_{L9} \times D_0/2 = 3.68 \times D_0 = 3.68 \text{ kip*ft} \\
M_{L10} &= P_{L10} \times D_0/2 = 0.32 \times D_0^2 = 0.32 \text{ kip*ft} \\
M_{L11} &= P_{L11} \times D_0/2 = 0.01 \times D_0^2 = 0.01 \text{ kip*ft} \\
M_{L12} &= P_{L12} \times D_0/2 = 98.2 \times D_0 = 98.2 \times D_0 \text{ kip*ft} \\
\end{align*} \]
<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total Moments</strong></td>
<td>Total Overturning Moments - ( M = 0 ) = ( 13.61 + 3.68 \times D_0 + 0.32 \times D_0^2 + -0.08 \times D_0^3 ) = 0 kip*ft</td>
<td></td>
</tr>
<tr>
<td>Solve for ( D )</td>
<td>( D_0 = 10.5 ) ft</td>
<td></td>
</tr>
<tr>
<td>( D = 12.6 ) ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Depth of Zero Shear</strong></td>
<td>( M = 5.677 ) ft</td>
<td></td>
</tr>
<tr>
<td><strong>Moment @ Ground Line</strong></td>
<td>( M_g = P_a \times (H/3) ) = kip*ft</td>
<td></td>
</tr>
<tr>
<td><strong>Moment @ Zero Shear</strong></td>
<td>( M_0 = (P_a \times (H/2+x)) + {K_{a1} \times (H/2+x) + P_{a1} \times (H/2+x) + P_{a2} \times (H/2+x) + P_{a3} \times (x/2) } - {1/2 \times K_{a2} \times \gamma \times x^2 } ) = kip*ft</td>
<td></td>
</tr>
<tr>
<td><strong>Structural Steel Properties</strong></td>
<td>Rolled Section Structural Steel Material ASTM A572 Grade 50</td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity of Steel Beam</td>
<td>( E_s = 29,000 ) ksi</td>
<td></td>
</tr>
<tr>
<td>Resistance Factor for Flexure and Shear in Steel</td>
<td>( \Phi_v = \Phi_f = 1.00 )</td>
<td></td>
</tr>
<tr>
<td>Hybrid Factor</td>
<td>( R_h = 1.0 )</td>
<td></td>
</tr>
<tr>
<td><strong>Steel Sheet Pile Size</strong></td>
<td><strong>Sheet Pile Width</strong></td>
<td>( w = 18 ) in</td>
</tr>
<tr>
<td><strong>Sheet Pile Height</strong></td>
<td>( d = 12 ) in</td>
<td>Steel Elastic Section Modulus per foot ( S = 30.2 ) in(^3)/ft</td>
</tr>
<tr>
<td><strong>Flange Thickness</strong></td>
<td>( t_f = 0.375 ) in</td>
<td>Steel Plastic Section Modulus per foot ( Z = 36.49 ) in(^3)/ft</td>
</tr>
<tr>
<td><strong>Web Thickness</strong></td>
<td>( t_w = 0.375 ) in</td>
<td>Steel Moment of Inertia per foot ( I = 184.2 ) in(^4)/ft</td>
</tr>
<tr>
<td><strong>Weight / Pile</strong></td>
<td>40.5 lb/ft</td>
<td></td>
</tr>
<tr>
<td><strong>Weight / Foot</strong></td>
<td>27 lb/ft(^2)</td>
<td></td>
</tr>
<tr>
<td><strong>Sheet Pile Loads</strong></td>
<td>Design Moment ( M_{d} = 31.0 ) kip*ft</td>
<td></td>
</tr>
<tr>
<td>Design Maximum Flexural Stress in Sheet Pile ( M_{d} / S = f_{ux} = 12.31 ) ksi</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Check Strength Limit State</strong></td>
<td>Flexural Design for Continuously Braced Flanges in Tension and Compression</td>
<td></td>
</tr>
<tr>
<td>Design Strength for Continuously Braced Flanges ( f_{ux} = 12.31 \leq \Phi_v \times R_h \times F_{yd} = 50.00 ) OK Flexure</td>
<td>24.6%</td>
<td></td>
</tr>
<tr>
<td>Usage Percentage</td>
<td>6.10.8.1.3-1</td>
<td></td>
</tr>
<tr>
<td>Steel Elastic Section Modulus per foot</td>
<td>Measured Section Loss 50%</td>
<td>( S = 15.1 ) in(^3)/ft</td>
</tr>
<tr>
<td>Design Maximum Flexural Stress in Sheet Pile Based Upon Deteriorated Steel Section ( M_{d} / S = f_{ux} = 24.63 ) ksi</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Check Strength Limit State</strong></td>
<td>Flexural Design for Continuously Braced Flanges in Tension and Compression</td>
<td></td>
</tr>
<tr>
<td>Design Strength for Continuously Braced Flanges ( f_{ux} = 24.63 \leq \Phi_v \times R_h \times F_{yd} = 50.00 ) OK Flexure</td>
<td>49.3%</td>
<td></td>
</tr>
</tbody>
</table>
Slab Properties

- Slab Span Length (Center to Center of Support) $\ell = 9.50\ ft$
- Slab Depth $h = 24.00\ in$
- Minimum Slab Depth for Deflection $h_{\text{min}} = \frac{\ell}{20.0} = 5.70\ in$
- Slab unit Width $b = 12.00\ in$

Dead Loads

- Uniform Dead Loads $D_1 = \frac{\text{lb}}{\text{ft}}$
- Miscellaneous $D_2 = \frac{\text{lb}}{\text{ft}}$
- Total Uniform Loads $D_T = \frac{\text{lb}}{\text{ft}}$
- Weight of Slab per foot $b \cdot h \cdot w_c = \frac{\text{lb}}{\text{ft}}$
- Total Dead Load $D = \frac{\text{lb}}{\text{ft}}$

Live Loads

- Minimum Floor Live Load $L_0 = \frac{\text{lb}}{\text{ft}}$
- Is Floor Load Reducible $T$
- Tributary Area for Floor $A_T = \frac{\text{ft}^2}{\text{ft}^2}$
- Reduced Live Load $L_1 = L_0 \cdot (0.25 + 15 / \sqrt{K_{\text{LL}} \cdot A_T}) = \frac{\text{lb}}{\text{ft}}$
- Live Load Element Factor for Slab $K_{\text{LL}}$ = Reduced Live Load $L_1 \cdot b = \frac{\text{lb}}{\text{ft}}$

Load Combinations

- Design Load $1.2 \cdot D + 1.6 \cdot L = \frac{\text{lb}}{\text{ft}}$
- Deflection Load, Dead + Live $D + L = \frac{\text{lb}}{\text{ft}}$
- Deflection Load, Live $L = \frac{\text{lb}}{\text{ft}}$

Concrete Properties

- Unit Weight of Concrete $w_c = 150\ \text{pcf}$
- Specified Concrete Compressive Strength $f_{\text{c}}' = 4,000\ psi$
- Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth $\beta_1 = 0.85$
- Modification factor reflecting the reduced mechanical properties of lightweight concrete $\lambda = 1.00$
- Modulus of Concrete $E_c = \frac{w_c \cdot 33 \cdot \sqrt{f_{\text{c}}'}}{1,500} = 3.8E+6\ psi$
- Strength-Reduction Factor - Shear $\Phi = 0.75$
- Strength-Reduction Factor - Flexural sections in pretensioned members $\Phi = 0.90$

Steel Reinforcing Properties

- Minimum Specified Yield Stress of Steel $f_y = 60,000\ psi$
- Clear Cover $d = 3.00\ in$
- Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement $d = 20.25\ in$
- Tension Steel Reinforcement $#4\ \#12\ in\ \bar{A}_s = 0.500\ in\ \bar{A}_s = 0.20\ in^2\ \bar{A}_s = 0.20\ in^2$
- Secondary Reinforcement $#4\ \#12\ in\ \bar{A}_s = 0.500\ in\ \bar{A}_s = 0.20\ in^2\ \bar{A}_s = 0.20\ in^2$
- Assuming 10% Steel Section Loss $\bar{A}_s = 0.18\ in^2$

Flexural Design Capacity

- $R_u = M_u / (b \cdot d \cdot f_y')$
- $70.395 / (12.00 \cdot 20.25 \cdot 200) = 14\ psi$
- Minimum Design Reinforcement Steel Ratio $0.85 \cdot f_y / f_y' \cdot [1 - (1 - 2 \cdot R_u / (\phi \cdot 0.85 \cdot f_y'))] = \rho = 0.00027$
- Minimum Design Reinforcement Steel Area $A_{s\text{min}} = \frac{\rho \cdot b \cdot d}{f_y'} = 0.06\ in^2$

Nominal Moment Strength

- Minimum Reinforcement of Flexural Members $M_u = A_{s\text{min}} \cdot f_y' \cdot (d / 2) = 217,271\ lb*ft$
- $M_u = 5,866\ lb*ft \leq 0.6F_M = \text{OK Flexure}$
### SUBJECT
Hoboken Waterfront Evaluation: Segment 8 Platform 1 - Concrete Slab

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
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</thead>
<tbody>
<tr>
<td><strong>Shear Design Capacity</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ACT 318 11-3</td>
<td>Nominal Concrete Shear Strength</td>
<td>[ V_c = 2 \lambda \sqrt{f'c} b d ] = 30,737 lb</td>
</tr>
<tr>
<td>ACI 318 11-15</td>
<td>Nominal Reinforcing Steel Shear Strength</td>
<td>[ V_s = \text{lb} ]</td>
</tr>
<tr>
<td></td>
<td>Design Shear Strength</td>
<td>[ \Phi V_a = \Phi(V_c + V_s) ] = 23,053 lb</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[ V_a = 2,470 \text{ lb} \leq \Phi V_a ] OK Shear</td>
</tr>
<tr>
<td><strong>Check Deflection</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Beam Span Length (Center to Center of Support)</td>
<td>[ \ell = 114.00 \text{ in} ]</td>
</tr>
<tr>
<td></td>
<td>Moment of Inertia</td>
<td>[ I = 13,824 \text{ in}^4 ]</td>
</tr>
<tr>
<td></td>
<td>Modulus of Elasticity * Moment of Inertia</td>
<td>[ E_s * I = 5.30E+10 \text{ lb*in}^2 ]</td>
</tr>
<tr>
<td>IBC Table 1604.3</td>
<td>Allowable Span Deflection</td>
<td>[ \Delta \leq \ell / 240 = 0.475 \text{ in} ]</td>
</tr>
<tr>
<td></td>
<td>Floor Members, Live Load</td>
<td>[ \Delta \leq \ell / 360 = 0.317 \text{ in} ]</td>
</tr>
<tr>
<td></td>
<td>Calculated Deflection</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Span Dead + Live Load Deflection</td>
<td>[ \Delta = 5 * w_2 * \ell^3 / (384 * E * I) = 0.001 \text{ in} ] OK Deflection</td>
</tr>
<tr>
<td></td>
<td>Span Live Load Deflection</td>
<td>[ \Delta = 5 * w_s * \ell^3 / (384 * E * I) = 0.000 \text{ in} ] OK Deflection</td>
</tr>
</tbody>
</table>
**Hoboken Waterfront Evaluation: Segment 8 Platform 1 - Longitudinal Beam**

**Beam Properties**
- Beam Span Length (Center to Center of Support) \( t \) = 30.00 in
- Beam Depth \( h \) = 36.00 in
- Minimum Beam Depth for Deflection \( h_{\text{min}} \) = \( \frac{t}{20.0} \) = 1.50 in
- Beam Width \( b \) = 36.00 in
- Tributary Load Width \( B \) = 9.38 in

**Dead Loads**
- Uniform Dead Loads
  - Floor Load \( D_1 \) = 300 lb/ft²
  - Miscellaneous \( D_2 \) = 150 lb/ft²
  - Total Uniform Loads \( D_3 \) = 300 lb/ft²

**Live Loads**
- Minimum Floor Live Load \( L_0 \) = 0 lb/ft²
- Tributary Area for Floor \( A_r \) = 281 ft²

**Load Combinations**
- Design Load \( 1.5 \times D + 1.5 \times L \) = 4.995 lb/ft
- Deflection Load, Dead + Live \( D + L \) = 4.163 lb/ft
- Deflection Load, Live \( L \) = 0 lb/ft

**Concrete Properties**
- Unit Weight of Concrete \( w_c \) = 150 pcf
- Specified Concrete Compressive Strength \( f_{cy} \) = 4,000 psi

**Steel Reinforcing Properties**
- Minimum Specified Yield Stress of Steel \( f_y \) = 60,000 psi

**Assumed Reinforcing Steel**
- Clear Cover = 2.00 in
- Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement \( d \) = 33.00 in
- Tension Steel Reinforcement \# of Bars = 5, \( d_b \) = 1.000 in, bar area = 0.79 in², \( A_s \) = 3.95 in⁴
- Number of Shear legs per stirrup = 2
- Shear Reinforcement \# of Bars = 4, \( s \) = 12 in, \( d_b \) = 0.500 in, bar area = 0.20 in², \( A_s \) = 0.40 in⁴

**Design Shear and Moment Assuming Simple Span**
- Design Shear (Factored Load) \( V_0 = w_1 \times t / 2 \) = 74.925 lb
- Design Moment (Factored Load) \( M_0 = w_1 \times t^2 / 8 \) = 561,938 lb*ft

**Flexural Design Capacity**
- Minimum Design Reinforcement Steel Ratio \( A_s = \rho \times b \times d \) = 0.00328
- Minimum Design Reinforcement Steel Area \( A_{s,\text{min}} = 0.90 \times A_s \) = 3.51 in⁴
- Depth of Equivalent Rectangular Stress Block \( a = A_s \times f_y / (0.85 \times f_{cy} \times b) \) = 1.94 in
- Distance From Extreme Compression Fiber to Neutral axis \( c = a / \beta \) = 2.28 in
- Nominal Moment Strength \( M_n = A_s \times f_y \times (d - a / 2) \) = 7,591,551 lb*ft
- Design Moment Strength \( M_s = 561,938 \times \Phi M_0 \) = 569,366 lb*ft

**ANSWER**

<table>
<thead>
<tr>
<th>Formula</th>
<th>( \Phi M_0 )</th>
<th>OK Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_s = 561,938 \times \Phi M_0 )</td>
<td>569,366</td>
<td>OK Flexure</td>
</tr>
</tbody>
</table>
**Subject**: Hoboken Waterfront Evaluation: Segment 8 Platform 1 - Longitudinal Beam

### Shear Design Capacity

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318 11-3</td>
<td>Nominal Concrete Shear Strength: $V_c = 2 \times \lambda \times (f'_c) \times b \times d$</td>
<td>150,271 lb</td>
</tr>
<tr>
<td>ACI 318 11-15</td>
<td>Nominal Reinforcing Steel Shear Strength: $V_s = A_v \times f_y \times d / s$</td>
<td>66,000 lb</td>
</tr>
<tr>
<td></td>
<td>Design Shear Strength: $V_u = \Phi (V_c + V_s)$</td>
<td>162,204 lb</td>
</tr>
</tbody>
</table>

### Check Deflection

| Beam Span Length (Center to Center of Support) | $l$ = 360.00 in |
| Moment of Inertia | $I = 139,968 in^4$ |
| Modulus of Elasticity * Moment of Inertia | $E_c \times I = 5.37E+11 lb\text{in}^2$ |

### Allowable Span Deflection

| IBC Table 1604.3 |Floor Members, Dead + Live Load| $\Delta \leq l / 240 = 1.500$ in |
| Floor Members, Live Load | $\Delta \leq l / 360 = 1.000$ in |

### Calculated Deflection

| Span Dead + Live Load Deflection | $\Delta = 5 \times w \times l^4 / (384 \times E \times I)$ | 0.141 in | **OK Deflection** |
| Span Live Load Deflection       | $\Delta = 5 \times w \times l^4 / (384 \times E \times I)$ | 0.000 in | **OK Deflection** |
### Beam Properties

- **Beam Span Length (Center to Center of Support)**: \( l = 19.00 \) ft
- **Beam Depth**: \( h = 36.00 \) in
- **Minimum Beam Depth for Deflection**:
  - Solid One-way Slabs: \( h_{\text{min}} = \frac{l}{20.0} = 11.40 \) in
  - Simply Supported: \( h_{\text{min}} = \frac{l}{20.0} = 11.40 \) in
- **Beam Width**: \( b = 36.00 \) in
- **Tributary Load Width**: \( B = 15.00 \) ft
- **Concrete Properties**:
  - **Modulus of Concrete** \( E_c = 3.76 \times 10^6 \) psi
  - **Modulus of Concrete : Normal Weight Concrete** \( E_c = 4,000 \) psi
  - **Specified Concrete Compressive Strength** \( f'_c = 6,000 \) psi
  - **Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth** \( \lambda = 0.85 \) psi
  - **Min. Design Reinforcement Steel Ratio** \( A_{s,\text{min}} = 1.39 \) in

### Dead Loads

- **Uniform Dead Loads**
  - **Floor Load** \( D_1 = \) lb/ft²
  - **Miscellaneous** \( D_2 = \) lb/ft²
  - **Total Uniform Loads** \( D = B * D_1 + 0.85 * f'c * a \) lb/ft
- **Uniform Dead Loads per Beam** \( B * D = 0 \) lb/ft
- **Weight of Beam** \( b * h * w = 1.350 \) lb/ft²
- **Total Dead Load** \( D = 1.350 \) lb/ft²

### Live Loads

- **Minimum Floor Live Load**
  - **None**
- **Tributary Area for Floor** \( A_T = 285 \) ft²
- **Load Combinations**
  - **Design Load** \( 1.2 * D + 1.6 * L = w \) lb/ft²
  - **Deflection Load, Dead + Live** \( D + L = w \) lb/ft²
  - **Deflection Load, Live** \( L = w \) lb/ft²

### Factored Reactions from Longitudinal Beam

- **Reduction Load, Live** \( \frac{285}{285} = 1 \) lb/ft²
- **Deflection Load, Dead** \( \frac{1.62}{1.62} = 1 \) lb/ft²

### Design Shear and Moment Assuming Simple Span

- **Design Shear (Factored Load)** \( V_u = w_1 * (2.5 + 15 / 3.76) \) lb
- **Design Moment (Factored Load)** \( M_u = w_1 * (d - a / 2) \) lb*ft
- **Nominal Moment Strength** \( M_n = 429,001 \) lb*ft

### Flexural Design Capacity

- **Minimum Design Reinforcement Steel Ratio** \( \rho = 0.00249 \)
- **Minimum Design Reinforcement Steel Area** \( A_{s,\text{min}} = \rho * b * d \) in²
- **Minimum Reinforcement of Flexural Members** \( A_{s,\text{min}} = 3.76 \) in²

- **Depth of Equivalent Rectangular Stress Block** \( a = 1.64 \) in
- **Nominal Moment Strength** \( M_n = 5,512,174 \) lb*ft

### Assumption: Minimum Bottom Reinforcement Along Entire Length of Beam

- **Nominal Moment Strength** \( M_n = 429,001 \) lb*ft
Shear Design Capacity

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318 11-3</td>
<td>Nominal Concrete Shear Strength: $V_c = 2 \cdot \lambda \cdot \sqrt{(f'_c)} \cdot b \cdot d$</td>
<td>$150,271$ lb</td>
</tr>
<tr>
<td>ACI 318 11-15</td>
<td>Nominal Reinforcing Steel Shear Strength: $V_s = A_v \cdot f_y \cdot d / s$</td>
<td>$59,400$ lb</td>
</tr>
<tr>
<td></td>
<td>Design Shear Strength: $V_s = \Phi(V_c + V_s)$</td>
<td>$157,254$ lb</td>
</tr>
</tbody>
</table>

Calculated Deflection

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td>IBC Table 1604.3</td>
<td>Span Dead + Live Load Deflection: $\Delta = 5 \cdot w_s \cdot \ell / (384 \cdot E \cdot I)$</td>
<td>$0.007$ in</td>
</tr>
<tr>
<td></td>
<td>Span Live Load Deflection: $\Delta = 5 \cdot w_s \cdot \ell / (384 \cdot E \cdot I)$</td>
<td>$0.000$ in</td>
</tr>
</tbody>
</table>

Check Deflection

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>$l$</td>
<td>$228.00$ in</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>$I$</td>
<td>$139,968$ in$^4$</td>
</tr>
<tr>
<td>Modulus of Elasticity * Moment of Inertia</td>
<td>$E \cdot I$</td>
<td>$5.37 \times 10^11$ lb*in$^2$</td>
</tr>
<tr>
<td>Allowable Span Deflection</td>
<td>Floor Members, Dead + Live Load</td>
<td>$\Delta \leq l / 240$</td>
</tr>
<tr>
<td>Floor Members, Live Load</td>
<td>$\Delta \leq l / 360$</td>
<td>$0.633$ in</td>
</tr>
</tbody>
</table>
### Pile Analysis

**Spacing Between Piles**
- Parallel to Pile Cap
- Perpendicular to Pile Cap

**Dead Load**
- Concrete Pile Cap: 3 ft * 3.00 ft * 19.00 ft * 1500.00 lb/cft = 25,650 lb
- Concrete Deck: 2.00 ft * 19.00 ft * 15.00 ft * 1500.00 lb/cft = 85,500 lb
- Soil Fill: 0.00 ft * 19.00 ft * 15.00 ft * 1200.00 lb/cft = 0 lb
- Total: 111,150 lb

**Live Load**
- Yards and Terraces, Pedestrian: 19.00 ft * 15.00 ft * 100.00 lb/ft² = 28,500 lb

**Load Combinations**
- Load 1: 1.4 * D = 155,610 lb
- Load 2: 1.2 * D + 1.6 * L = 178,980 lb

---

### Member Properties

<table>
<thead>
<tr>
<th>HP14x102</th>
<th>A36</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slenderness Ratio</td>
<td>65.1</td>
</tr>
<tr>
<td>Cross sectional area</td>
<td>30 in²</td>
</tr>
<tr>
<td>AISC E3-2</td>
<td>28.3 ksi</td>
</tr>
<tr>
<td>AISC E3-1</td>
<td>73.4 kip</td>
</tr>
</tbody>
</table>

---

### Axial Load Design Compression

**Full Section**
- Slenderness Ratio: K * L / t ≤ 200 OK
- F_c = (τ² * E) / (K * L / t)^2
- F_c = 28.8 ksi
- P_n = A * F_c = 864.3 kip
- Resistance Factor for Compression: Φ = 0.85
- Φ * P_n = 734.6 kip
- Φ * P_n ≥ P_u OK

**Deteriorated Section**
- Slenderness Ratio: K * L / t ≤ 200 OK
- F_c = (τ² * E) / (K * L / t)^2
- F_c = 28.3 ksi
- P_n = A * F_c = 420.4 kip
- Resistance Factor for Compression: Φ = 0.85
- Φ * P_n = 357.3 kip
- Φ * P_n ≥ P_u OK
References

American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

Slab Properties

| Slab Span Length (Center to Center of Support) | ℓ | 22.50 ft |
| Slab Depth | h | 14.00 in |
| Minimum Slab Depth for Deflection | h<sub>min</sub> | ℓ / 20.0 = 13.50 in |
| Slab unit Width | b | 12.00 in |

Dead Loads

<table>
<thead>
<tr>
<th>Dead Loads</th>
<th>Unit</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform Dead Loads</td>
<td>D&lt;sub&gt;1&lt;/sub&gt;</td>
<td>4&quot; Concrete Pavers</td>
<td>50 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>D&lt;sub&gt;2&lt;/sub&gt;</td>
<td>15&quot; Sand Setting Bed for Pavers</td>
<td>150 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>D&lt;sub&gt;3&lt;/sub&gt;</td>
<td>4&quot; C.I.P. Concrete Topping</td>
<td>50 lb/ft²</td>
</tr>
<tr>
<td>Total Uniform Loads</td>
<td>D&lt;sub&gt;T&lt;/sub&gt;</td>
<td>250 lb/ft²</td>
<td></td>
</tr>
<tr>
<td>Weight of Slab per foot</td>
<td>b * h * w&lt;sub&gt;c&lt;/sub&gt;</td>
<td>125 lb/ft</td>
<td></td>
</tr>
<tr>
<td>Steel and Wood Guardrail</td>
<td>D&lt;sub&gt;r&lt;/sub&gt;</td>
<td>10 lb/ft</td>
<td></td>
</tr>
<tr>
<td>Total Dead Load</td>
<td>D&lt;sub&gt;T&lt;/sub&gt;</td>
<td>385 lb/ft</td>
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</tr>
</tbody>
</table>

Live Loads

<table>
<thead>
<tr>
<th>Live Loads</th>
<th>Unit</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Floor Live Load</td>
<td>L&lt;sub&gt;0&lt;/sub&gt;</td>
<td>40. Yards and Terraces, Pedestrians</td>
<td>100 lb/ft²</td>
</tr>
<tr>
<td>Is Floor Load Reducible</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tributary Area for Floor</td>
<td>A&lt;sub&gt;T&lt;/sub&gt;</td>
<td>23 ft²</td>
<td></td>
</tr>
<tr>
<td>Live Load Element Factor for Slab</td>
<td>K&lt;sub&gt;LL&lt;/sub&gt;</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Reduced Live Load</td>
<td>L&lt;sub&gt;1&lt;/sub&gt;</td>
<td>100 lb/ft²</td>
<td></td>
</tr>
</tbody>
</table>

Load Combinations

<table>
<thead>
<tr>
<th>Load Combinations</th>
<th>Factor</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Load</td>
<td>1.2 * D + 1.6 * L&lt;sub&gt;1&lt;/sub&gt;</td>
<td>622 lb/ft</td>
<td></td>
</tr>
<tr>
<td>Deflection Load, Dead + Live</td>
<td>D + L&lt;sub&gt;1&lt;/sub&gt;</td>
<td>485 lb/ft</td>
<td></td>
</tr>
<tr>
<td>Deflection Load, Live</td>
<td>L&lt;sub&gt;1&lt;/sub&gt;</td>
<td>100 lb/ft</td>
<td></td>
</tr>
</tbody>
</table>

Concrete Properties

<table>
<thead>
<tr>
<th>Concrete Properties</th>
<th>Unit</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Concrete</td>
<td>w&lt;sub&gt;c&lt;/sub&gt;</td>
<td>150 pcf</td>
<td></td>
</tr>
<tr>
<td>Specified Concrete Compressive Strength</td>
<td>f&lt;sub&gt;c&lt;/sub&gt;&lt;sup&gt;′&lt;/sup&gt;</td>
<td>4,000 psi</td>
<td></td>
</tr>
<tr>
<td>Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth</td>
<td>β&lt;sub&gt;1&lt;/sub&gt;</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Modulus of Concretes</td>
<td>E&lt;sub&gt;c&lt;/sub&gt;</td>
<td>3,846 psi</td>
<td></td>
</tr>
<tr>
<td>Strength-Reduction Factor - Shear</td>
<td>Φ</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>Normal Weight Concrete</td>
<td>E&lt;sub&gt;nc&lt;/sub&gt;</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Strength-Reduction Factor - Flexural sections in pretensioned members</td>
<td>Φ</td>
<td>0.90</td>
<td></td>
</tr>
</tbody>
</table>

Steel Reinforcing Properties

<table>
<thead>
<tr>
<th>Steel Reinforcing Properties</th>
<th>Unit</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Specified Yield Stress of Steel</td>
<td>f&lt;sub&gt;y&lt;/sub&gt;</td>
<td>60,000 psi</td>
<td></td>
</tr>
<tr>
<td>Clear Cover</td>
<td>1.50 in</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Distance from Extreme Compression Fiber to Center of Tension Reinforcement</td>
<td>d</td>
<td>11.63 in</td>
<td></td>
</tr>
<tr>
<td>Tension Steel Reinforcement</td>
<td>A&lt;sub&gt;s&lt;/sub&gt;</td>
<td>0.88 in²</td>
<td></td>
</tr>
<tr>
<td>Secondary Reinforcement</td>
<td>A&lt;sub&gt;v&lt;/sub&gt;</td>
<td>0.20 in²</td>
<td></td>
</tr>
</tbody>
</table>

Flexural Design Capacity

<table>
<thead>
<tr>
<th>Flexural Design Capacity</th>
<th>Factor</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>R&lt;sub&gt;N&lt;/sub&gt;</td>
<td>M&lt;sub&gt;j&lt;/sub&gt; / (b * d&lt;sup&gt;2&lt;/sup&gt;)</td>
<td>472,331 / (12.00 * 11.63)</td>
<td>291 psi</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Ratio</td>
<td>0.85 * f&lt;sub&gt;y&lt;/sub&gt; / f&lt;sub&gt;c&lt;/sub&gt;</td>
<td>0.00568</td>
<td>0.00568</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Area</td>
<td>A&lt;sub&gt;s&lt;/sub&gt;</td>
<td>0.44 in²</td>
<td></td>
</tr>
<tr>
<td>Minimum Reinforcement of Flexural Members</td>
<td>A&lt;sub&gt;s, min&lt;/sub&gt;</td>
<td>0.44 in²</td>
<td></td>
</tr>
<tr>
<td>Depth of Equivalent Rectangular Stress Block</td>
<td>a = A&lt;sub&gt;s&lt;/sub&gt; * f&lt;sub&gt;y&lt;/sub&gt; / (0.85 * f&lt;sub&gt;c&lt;/sub&gt; * b)</td>
<td>1.29 in</td>
<td></td>
</tr>
<tr>
<td>Nominal Moment Strength</td>
<td>M&lt;sub&gt;N&lt;/sub&gt;</td>
<td>579,635 lb*ft</td>
<td></td>
</tr>
<tr>
<td>Design Moment Strength</td>
<td>M&lt;sub&gt;u&lt;/sub&gt;</td>
<td>39,361 lb*ft</td>
<td></td>
</tr>
<tr>
<td>OK Flexure</td>
<td>43,473 lb*ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Shear Design Capacity

<table>
<thead>
<tr>
<th>ACT 318 11-3</th>
<th>Nominal Concrete Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_c = 2 \times \lambda \times \sqrt{f'_{c}} \times b \times d$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ACT 318 11-15</th>
<th>Nominal Reinforcing Steel Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_s = \Phi V_n$</td>
</tr>
</tbody>
</table>

### Design Shear Strength

$\Phi V_n = \Phi (V_c + V_s)$

$V_n = 6,998$ lb

### OK Shear

### Check Deflection

**Beam Span Length (Center to Center of Support)**

$\ell = 270.00$ in

**Moment of Inertia**

$I = 2,744$ in$^4$

**Modulus of Elasticity * Moment of Inertia**

$E_c \times I = 1.05 \times 10^5$ lb*in$^3$

### Allowable Span Deflection

Floor Members, Dead + Live Load

$\Delta \leq \ell / 240 = 1.125$ in

Floor Members, Live Load

$\Delta \leq \ell / 360 = 0.750$ in

### Calculated Deflection

**Span Dead + Live Load Deflection**

$\Delta = 5 \times w_2 \times \ell^3 / (384 \times E \times I) = 0.266$ in

**OK Deflection**

**Span Live Load Deflection**

$\Delta = 5 \times w_3 \times \ell^3 / (384 \times E \times I) = 0.055$ in

**OK Deflection**
References
American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

Beam Properties
Beam Span Length (Center to Center of Support) t = 5.00 ft
Beam Depth h = 36.00 in
Minimum Beam Depth for Deflection Solid One-way Slabs b_{min} = t / 20.0 = 3.00 in
Simply Supported
Beam Width b = 36.00 in
Tributary Load Width B = 11.25 in

Dead Loads
Uniform Dead Loads
4" Concrete Pavers D_1 = 50 lb/ft^2
15"(max) Sand Setting Bed for Pavers D_2 = 150 lb/ft^2
4" N.W. C.I.P. Concrete Topping D_3 = 50 lb/ft^2
10" Concrete Precast Slab D_4 = 150 lb/ft^2
Miscellaneous D_5 = 125 lb/ft^2
Total Uniform Loads B = 525 lb/ft^2
Uniform Dead Loads per Beam B * D_1 = 5,906 lb/ft
Weight of Beam b * h * w = 1,350 lb/ft
Steel and Wood Guardrail D_3 = 10 lb/ft
Total Dead Load D = 7,266 lb/ft

Live Loads
Minimum Floor Live Load 40. Yards and Terraces, Pedestrians L_{40} = 100 lb/ft^2
Is Floor Load Reducible A_{V} = 56 ft^2
Tributary Area for Floor K_{LL} = 1.00
IBC Table 1607.1
IBC Table 1607.9.1
IBC Eq. 16-25 Reduced Live Load
IBC Eq. 16-2
Design Load 1.2 * D + 1.6 * L = w_1 = 10,520 lb/ft
Deflection Load, Dead + Live D + L = w_2 = 8,391 lb/ft
Deflection Load, Live L = w_3 = 1,125 lb/ft

Design Shear and Moment Assuming Simple Span
Design Shear (Factored Load) V_{s} = w_1 * 1 / 2 = 26,299 lb
Design Moment (Factored Load) M_{M} = w_1 * 1 / 8 = 32,873 lb*ft

Concrete Properties
Unit Weight of Concrete w_{c} = 150 pcf
Specified Concrete Compressive Strength f_{c} = 6,000 psi
ACI 318 10.2.7.3 Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth \beta_{l} = 0.85
ACI 318 8.6.1 Modification factor reflecting the reduced mechanical properties of lightweight concrete Normal Weight Concrete \lambda_{C} = 1.00
ACI 318 8.5.1 Modulus of Concrete E_{c} = w_{c} * 33 * \lambda_{C} = 3.8E6 psi
ACI 318 9.3.2.3 Strength-Reduction Factor - Shear \Phi_{S} = 0.75
ACI 318 9.3.2.7 Strength-Reduction Factor - Flexural sections in pretensioned members \Phi_{F} = 0.90

Steel Reinforcing Properties
Minimum Specified Yield Stress of Steel ASTM A615 Grade 60 f_{y} = 60,000 psi
Assumed Reinforcing Steel
Clear Cover d = 2.00 in
Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement \beta_{l} = 32.94 in
Tension Steel Reinforcement #9 # of Bars = 3 d_{e} = 1.128 in bar area = 1.00 in^2 A_{s} = 3.00 in^2
Number of Shear legs per stirrup \# = 2
Shear Reinforcement #4 s = 12 in d_{s} = 0.500 in bar area = 0.20 in^2 A_{s} = 0.40 in^2

Flexural Design Capacity
R_{n} = M_{n} / (b * d)^{2} 394,481 / (36.00 * 32.94)^{2} = 10 psi
Minimum Design Reinforcement Steel Ratio 0.85 * f_{c} / f_{y} * [1 - \sqrt{1 - 2 * R_{n} / \Phi_{F} * \lambda_{C} * (1 + \lambda_{C} / \beta_{l})}] = 0.00019
Minimum Design Reinforcement Steel Area A_{s} = \rho * b * d 0.00019 * 36.00 * 32.94 = 0.22 in^2
Minimum Reinforcement of Flexural Members A_{s, min} = 3 * \sqrt{f_{y} / \rho} * b * d / \Phi_{F} 3.75 in^2
Depth of Equivalent Rectangular Stress Block a = A_{s} * f_{y} / (0.85 * f_{c} * b) = 1.47 in
Distance From Extreme Compression Fiber to Neutral axis c = a / \beta_{l} = 1.73 in
Nominal Moment Strength M_{n} = A_{s} * f_{y} * (d - a / 2) = 5,796,127 lb*ft
Design Moment Strength M_{d} = 32,873 lb*ft \leq \Phi M_{d} = \Phi 434,710 lb*ft OK Flexure
**Hoboken Waterfront Evaluation: Segment 8 Platform 2 - Concrete Beam**

### Shear Design Capacity

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318 11-3</td>
<td>Nominal Concrete Shear Strength</td>
<td>( V_c = 2 \times \lambda \times (f'_c) \times b \times d = 149,980 \text{ lb} )</td>
</tr>
<tr>
<td>ACI 318 11-15</td>
<td>Nominal Reinforcing Steel Shear Strength</td>
<td>( V_s = A_v \times f_y \times d / s = 65,872 \text{ lb} )</td>
</tr>
<tr>
<td></td>
<td>Design Shear Strength</td>
<td>( \Phi V_s = \Phi (V_c + V_s) = 161,889 \text{ lb} )</td>
</tr>
</tbody>
</table>

**Check Deflection**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>( l = 60.00 \text{ in} )</td>
<td>( E_c \times I = 5,37E+11 \text{ lb*in}^2 )</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>( I = 139,968 \text{ in}^4 )</td>
<td></td>
</tr>
</tbody>
</table>

**IBC Table 1604.3**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Allowable Span Deflection</td>
<td>Floor Members, Dead + Live Load</td>
<td>( \Delta \leq l / 240 = 0.250 \text{ in} )</td>
</tr>
<tr>
<td>Floor Members, Live Load</td>
<td>( \Delta \leq l / 360 = 0.167 \text{ in} )</td>
<td></td>
</tr>
</tbody>
</table>

**Calculated Deflection**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Dead + Live Load Deflection</td>
<td>( \Delta = 5 \times w_2 \times l^3 / (384 \times E \times I) = 0.000 \text{ in} )</td>
<td>OK Deflection</td>
</tr>
<tr>
<td>Span Live Load Deflection</td>
<td>( \Delta = 5 \times w_3 \times l^3 / (384 \times E \times I) = 0.000 \text{ in} )</td>
<td>OK Deflection</td>
</tr>
</tbody>
</table>
**Pile Analysis**

**Spacing Between Piles**
- Parallel to Pile Cap: 5.00 ft
- Perpendicular to Pile Cap: 22.50 ft

**Dead Load**
- Concrete Pile Cap: 3 ft * 3.00 ft * 5.00 ft * 150.00 lb/ft³ = 6,750 lb
- Concrete Deck: 0.83 ft * 5.00 ft * 22.50 ft * 150.00 lb/ft³ = 14,057 lb
- 4” N.W. C.I.P. Concrete Topping: 0.33 ft * 5.00 ft * 22.50 ft * 50.00 lb/ft³ = 949.22 lb
- 15”(max) Sand Setting Bed for Pavers: 1.25 ft * 22.50 ft * 22.50 ft * 150.00 lb/ft³ = 94,922 lb
- 4” Concrete Pavers: 0.33 ft * 0.00 ft * 22.50 ft * 50.00 lb/ft³ = 117,585 lb

**Total**

**Live Load**
- Yards and Terraces, Pedestrian: 5.00 ft * 22.50 ft * 100.00 lb/ft² = 11,250 lb

**Load Combinations**
- IBC EQ 16-1 Load 1: 1.4 * D = 164,619 lb
- IBC EQ 16-2 Load 2: 1.2 * D + 1.6 * L = 159,102 lb

All reference AISC 360-05 Specification for Structural Steel Buildings unless otherwise noted

---

**Member Properties**

### Full Section

**AISC Table C-2**

<table>
<thead>
<tr>
<th>Specification</th>
<th>HP14x89</th>
<th>A36</th>
</tr>
</thead>
</table>

---

**Effective Length Factor**

<table>
<thead>
<tr>
<th>Specified minimum yield stress</th>
<th>$F_y = 36$ ksi</th>
<th>Cross sectional area</th>
<th>$A = 26.1$ in²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity of Steel</td>
<td>$E = 29,000$ ksi</td>
<td>Moment of inertia</td>
<td>$I_y = 904$ in⁴</td>
</tr>
<tr>
<td>Beam depth</td>
<td>$d = 13.83$ in</td>
<td>Radius of Gyration</td>
<td>$r_y = 5.88$ in</td>
</tr>
<tr>
<td>Web thickness</td>
<td>$t_w = 0.615$ in</td>
<td>Moment of inertia</td>
<td>$I_y = 326$ in⁴</td>
</tr>
<tr>
<td>Flange width</td>
<td>$b_f = 14.685$ in</td>
<td>Radius of Gyration</td>
<td>$r_y = 3.53$ in</td>
</tr>
<tr>
<td>Flange Thickness</td>
<td>$t_f = 0.615$ in</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Axial Load Design Compression**

### Full Section

<table>
<thead>
<tr>
<th>Slenderness Ratio</th>
<th>$K * L / r_y = 49.3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistance Factor for Compression</td>
<td>$\Phi = 0.85$</td>
</tr>
<tr>
<td>$\Phi P_n \geq P_u = 702.8$ kip</td>
<td></td>
</tr>
</tbody>
</table>

### Deteriorated Section (~20% Section Loss)

<table>
<thead>
<tr>
<th>Slenderness Ratio</th>
<th>$K * L / r_y = 50.8$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistance Factor for Compression</td>
<td>$\Phi = 0.85$</td>
</tr>
<tr>
<td>$\Phi P_n \geq P_u = 556.7$ kip</td>
<td></td>
</tr>
</tbody>
</table>

---

**AISC E3-1 Nominal Compression Axial Strength**

| $P_n = A_y \times F_y = 826.8$ kip |

| $\Phi P_n \geq P_u = 702.8$ kip | OK |

---

**AISC E3-4**

| $F_y = (\pi^2 \times E) / (K * L / r_y)^2 = 117.80$ |

---

**AISC E3-2**

| $F_{cr} = F_y \times (0.658 \times F_y / F_y) = 839.7$ ksi |

| $\Phi P_n \geq P_u = 702.8$ kip | OK |

---

**AISC E3-3**

| $F_y = (\pi^2 \times E) / (K * L / r_y)^2 = 31.7$ ksi |

---

**AISC E3-1 Nominal Compression Axial Strength**

| $P_n = A_y \times F_y = 664.9$ kip |

| $\Phi P_n \geq P_u = 556.7$ kip | OK |

---

**AISC E3-4**

| $F_y = (\pi^2 \times E) / (K * L / r_y)^2 = 110.85$ |

---

**AISC E3-2**

| $F_{cr} = F_y \times (0.658 \times F_y / F_y) = 839.7$ ksi |

| $\Phi P_n \geq P_u = 702.8$ kip | OK |

---

**AISC E3-3**

| $F_y = (\pi^2 \times E) / (K * L / r_y)^2 = 31.4$ ksi |

---

**AISC E3-1 Nominal Compression Axial Strength**

| $P_n = A_y \times F_y = 664.9$ kip |

| $\Phi P_n \geq P_u = 556.7$ kip | OK |
## References
- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

### Slab Properties
- **Slab Span Length (Center to Center of Support)**
  \[ \ell = 8.43 \text{ ft} \]
- **Slab Depth**
  \[ h = 5.00 \text{ in} \]
- **Minimum Slab Depth for Deflection**
  \[ h_{\text{min}} = \ell / 20.0 = 5.06 \text{ in} \]
- **Slab unit Width**
  \[ b = 12.00 \text{ in} \]

### Dead Loads
- **Uniform Dead Loads**
  - **Slab Load**
    \[ D_1 = 63 \text{ lb/ft}^2 \]
  - **Miscellaneous Load**
    \[ D_2 = 63 \text{ lb/ft}^2 \]
  - **Total Uniform Load**
    \[ D_1 + D_2 = 63 \text{ lb/ft}^2 \]
- **Weight of Slab per foot**
  \[ w_c = \text{lb/ft} \]
- **Total Dead Load**
  \[ D = 63 \text{ lb/ft} \]

### Live Loads
- **Minimum Floor Live Load**
  \[ L_0 = 100 \text{ lb/ft}^2 \]
- **Is Floor Load Reducible**
  \[ A_T = 8 \text{ ft}^2 \]
- **Live Load Element Factor for Slab**
  \[ K_{LL} = 1.00 \]
- **Reduced Live Load**
  \[ L_1 = L_0 \times (0.25 + 15 / \sqrt{K_{LL} \times A_T}) = 991 \text{ lb} \]
  \[ L_1 \times b = 2,088 \text{ lb*ft} \]

### Consequences
- **Concrete Properties**
  - **Unit Weight of Concrete**
    \[ w_c = 150 \text{pcf} \]
  - **Specified Concrete Compressive Strength**
    \[ f'_c = 4,000 \text{ psi} \]
  - **Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth**
    \[ \beta_1 = 0.85 \]
  - **Modification factor reflecting the reduced mechanical properties of lightweight concrete**
    \[ \lambda = 1.00 \]
  - **Normal Weight Concrete**
    \[ E_c = w_c \times 33 \times \sqrt{f'_c} = 3.8E+6 \text{ psi} \]
  - **Strength-Reduction Factor - Shear**
    \[ \Phi = 0.75 \]
  - **Strength-Reduction Factor - Flexural sections in pretensioned members**
    \[ \Phi = 0.90 \]

### Steel Reinforcing Properties
- **Minimum Specified Yield Stress of Steel**
  \[ f_y = 60,000 \text{ psi} \]
- **Assumed Reinforcing Steel**
  - Clear Cover
    \[ d = 1.50 \text{ in} \]
  - **Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement**
    \[ d = 2.75 \text{ in} \]
  - **Tension Steel Reinforcement**
    \[ \#4 @ 12 \text{ in} \]
    \[ A_s = 0.20 \text{ in}^2 \]
    \[ A_{s,min} = 0.18 \text{ in}^2 \]
  - **Secondary Reinforcement**
    \[ 12 \text{ in} \]
    \[ A_s = 0.20 \text{ in}^2 \]
    \[ A_{s,min} = 0.18 \text{ in}^2 \]
  - **Assuming 10% Steel Section Loss**
    \[ A_{s,min} = 0.18 \text{ in}^2 \]
- **Flexural Design Capacity**
  \[ R_u = M_u / (b * d^2) \]
  \[ 25,050 / (12.00 - 2.75 - 2) = 276 \text{ psi} \]
- **Minimum Design Reinforcement Steel Ratio**
  \[ A_s \times \rho = 0.85 \times f_y / f_e \times \{1 - \sqrt{1 - 2 * R_u / (\Phi * 0.85 \times f_e)}\} \]
  \[ 0.00537 \times 12.00 \times 2.75 = 0.18 \text{ in}^2 \]

**OK Flexure**
### Shear Design Capacity

**ACT 318 11-3**
- Nominal Concrete Shear Strength
  \[ V_c = 2 \lambda \sqrt{f'c} b d \]
  \[ V_c = 4,174 \text{ lb} \]

**ACT 318 11-15**
- Nominal Reinforcing Steel Shear Strength
  \[ V_s = \Phi V_n \]
  \[ V_s = 3,131 \text{ lb} \]

- Design Shear Strength
  \[ \Phi V_n = (V_c + V_s) \]
  \[ \Phi V_n = 991 \text{ lb} \]

**OK Shear**

### Check Deflection

- Beam Span Length (Center to Center of Support)
  \[ \ell = 101.16 \text{ in} \]
- Moment of Inertia
  \[ I = 125 \text{ in}^4 \]
- Modulus of Elasticity * Moment of Inertia
  \[ E_c I = 4.79E+08 \text{ lb*in}^2 \]

**IBC Table 1604.3**
- Allowable Span Deflection
  - Floor Members, Dead + Live Load
    \[ \Delta \leq \ell / 240 = 0.422 \text{ in} \]
  - Floor Members, Live Load
    \[ \Delta \leq \ell / 360 = 0.281 \text{ in} \]

**Calculated Deflection**

- Span Dead + Live Load Deflection
  \[ \Delta = \frac{5 \times w_2 \times \ell^4}{(384 \times E \times I)} \]
  \[ \Delta = 0.039 \text{ in} \]

- Span Live Load Deflection
  \[ \Delta = \frac{5 \times w_3 \times \ell^4}{(384 \times E \times I)} \]
  \[ \Delta = 0.024 \text{ in} \]

**OK Deflection**
National Design Specifications (NDS) 2005
2005 ASD/LRFD Manual for Engineered Wood Construction

Beam Properties
- Beam Span Length (Center to Center of Support): $\ell = 4.70$ ft
- Beam Depth: $h = 12.00$ in
- Beam Width: $b = 12.00$ in
- Tributary Load Width: $B = 8.43$ ft

IBC Table 1607.9.1
- Uniform Dead Loads per Beam
  - Floor Load
  - Miscellaneous
- Total Uniform Loads

IBC Table 1607.9.1
- Uniform Dead Loads
- Miscellaneous
- Total Uniform Loads

Uniform Dead Loads
- $D_1 = 62$ lb/ft²
- $D_2 = 62$ lb/ft²

Total Uniform Loads
- $B \times D_1 = 518$ lb
- $b \times h \times w_1 = 38$ lb/ft
- $D = 557$ lb/ft

Load Combinations
- Dead + Live
- Reduced Live Load
  - $L_1 = L_2 \times (0.25 + 15 / \sqrt{K_{LL} \times A_T}) = 100$ lb/ft²
  - $L = L_1 \times B = 843$ lb/ft

Design Shear and Moment Assuming Simple Span
- Design Shear (Factored Load): $V_u = 0.6 \times w_1 \times \ell = 3.947$ lb
- Design Moment (Factored Load): $M_u = 0.1 \times w_1 \times \ell^2 = 3.092$ lb*ft

Timber Design Values
- Timber Species and Grade: Southern Pine No. 2
- Timber Specific Gravity: $G = 0.55$
- Moisture Content of Wood: $m.c. = 25%$
- Timber Density: $\rho = 38.2$ lb/ft³
- Member Depth: $b = 12$ in
- Member Thickness: $t = 38.2$ in

Wet Service - Moisture content will exceed 19% for an extended time period
- Is Wood Incised: No

Loading Type
- Member Cross Sectional Area: $A = 144.00$ in²
- Member Bearing Length (perpendicular to grain): $L_b = 12.00$ in
- Member Bearing Area (perpendicular to grain): $A_b = 144.00$ in²
- Section Modulus: $S = 288.00$ in³
- Moment of Inertia: $I = 1,728.00$ in⁴
- Weight per foot of length: $w_0 = 38.18$ lb/ft

NDS Table 4A
- Allowable Strength (Bending): $F_b = 975$ psi
- Allowable Strength (Shear): $F_v = 165$ psi
- Allowable Strength (Compression perpendicular to the grain): $F_{c,\parallel} = 440$ psi
- Modulus of Elasticity: $E = 1,300,000$ psi
- Minimum Modulus of Elasticity: $E_{min} = 470,000$ psi

NDS Table M4.3
- Design Shear Values:
  - $F_{V,1} = 1.00$
  - $F_{V,2} = 1.00$
  - $F_{V,3} = 0.87$
  - $E = 1.00$

NDS Table M4.3
- Design Moment Values:
  - $C_{M,1} = 1.00$
  - $C_{M,2} = 1.00$
  - $C_{M,3} = 1.00$

NDS Table 4D
- Incising Factor: $C_i = 1.00$
- Size Factor: $C_d = 1.00$
- Stability Factor: $C_s = 1.00$
- Bearing Area Factor: $C_{B,A} = 1.00$
SUBJECT
Hoboken Waterfront Evaluation: Segment 8-Timber Beam Platform 3

Unbraced Beam Length
Effective Unbraced Length

\[ R_b = \sqrt{l_e \cdot d / b^2} = 1.4 \]  \[ F_b = 975 \text{ psi} \]

Modulus of Elasticity * Moment of Inertia

\[ E' \cdot I = 2,246,400 \text{ kips}^2 \text{ in}^2 \]

Assuming that all pile cap spans at least 3 spans

Allowable Load Bearing
\[ W_b = P_{\text{cap}} / L^2 \cdot 6 = 15,054 \text{ lb/ft} \]

Allowable Load Shear
\[ W_s = V' / L^6 = 5,617 \text{ lb/ft} \]

Allowable Load Moment
\[ W_m = M' / L^4 \cdot 1 = 10,591 \text{ lb/ft} \]

OK

OK

1.00 ft
24.7 in

280,750 lb/in
15,840 lb
42,451 lb
2,246,400 kips/in²

15,054 lb/ft
5,617 lb/ft
10,591 lb/ft
References

American Society of Civil Engineers ASCE/SEI 7-05 Minimum Design Loads for Building and Other Structures
American Forest & Paper Association (AF&PA) NDS-05 National Design Specifications (NDS) for Wood Construction

Spacing Between Piles
Parallel to Pile Cap

Perpendicular to Pile Cap

Dead Load
Timber Pile Cap

Concrete Deck

Total

Live Load

Yards and Terraces, Pedestrian

Load Combinations
Load 1

\[ P_u = D + L = \text{lb} \]

Timber Design Values

Pile Diameter

Timber Species & Grade
Southern Pine

Timber Specific Gravity

Moisture Content of Wood

Timber Density

Member Cross Sectional Area

Section Modulus

Moment of Inertia

A/2 * y = Q

Allowable Strength (Compression)

Allowable Strength (Bending)

Allowable Strength (Shear)

Allowable Strength (Compression perpendicular to the grain)

Modulus of Elasticity

Minimum Modulus of Elasticity

NDS Table 4D

NDS Table M6.3-1

Unbraced Column Length

Axial Loading Only

Euler-based ASD critical buckling value for Columns

NDS M3.6-2

Allowable Compression

Allowable Compression

Deteriorated Section (~75% Section Loss)

Unbraced Column Length

Axial Loading Only

Euler-based ASD critical buckling value for Columns

NDS M3.6-2

Allowable Compression

Allowable Compression

Deteriorated Section (~75% Section Loss)
### References

- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

### Slab Properties

<table>
<thead>
<tr>
<th>Slab Span Length (Center to Center of Support)</th>
<th>ℓ = 8.43 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab Depth</td>
<td>h = 5.00 in</td>
</tr>
</tbody>
</table>

ACI Table 9.5(a)

<table>
<thead>
<tr>
<th>Minimum Slab Depth for Deflection</th>
<th>Solid One-way Slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td>h_{min} = ℓ / 20.0             = 5.06 in</td>
<td></td>
</tr>
<tr>
<td>Slab unit Width</td>
<td>b = 12.00 in</td>
</tr>
</tbody>
</table>

### Dead Loads

- **Uniform Dead Loads**
  - Slab Load: \( D_1 \) = \( \text{lb/ft}^2 \)
  - Miscellaneous: \( D_2 \) = \( \text{lb/ft}^2 \)
  - Total Uniform Loads: \( D_T \) = \( \text{lb/ft}^2 \)

- **Weight of Slab per foot**
  - \( b \cdot h = \text{lb/ft} \)
  - Total Dead Load: \( D = \text{lb/ft} \)

### Live Loads

- **Minimum Floor Live Load**
  - \( \text{lb/ft}^2 \)
  - Is Floor Load Reducible: No
  - Tributary Area for Floor: \( A_T \) = \( \text{ft}^2 \)

IBC Table 1607.1

<table>
<thead>
<tr>
<th>Reduced Live Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_i = L_{d0} \cdot (0.25 + 15 / \sqrt{K_{LL} \cdot A_T}) = 100 \text{ lb/ft}^2 )</td>
</tr>
</tbody>
</table>
| \( L = L_i \cdot b \) = \( 100 \text{ lb/ft} \)

IBC Eq. 16-25

- **Design Shear and Moment Assuming Simple Span**
  - Design Shear (Factored Load): \( V_u = w_1 \cdot ℓ / 2 = 991 \text{ lb} \)
  - Design Moment (Factored Load): \( M_u = w_1 \cdot ℓ^2 / 8 = 2,088 \text{ lb*ft} \)

### Concrete Properties

- **Unit Weight of Concrete**: \( w_c = 150 \text{ pcf} \)
- **Specified Concrete Compressive Strength**: \( f'_{c,u} = 4,000 \text{ psi} \)
- **Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth**: \( \beta_1 = 0.85 \)
- **Modification factor reflecting the reduced mechanical properties of lightweight concrete**: \( \lambda = 1.00 \)
- **Modulus of Concrete**: \( E_c = w_c^{1.5} \cdot 33 \cdot \sqrt{f'_{c,u}} = 3.8E+6 \text{ psi} \)

ACI Table 9.5(a)

<table>
<thead>
<tr>
<th>Normal Weight Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_y = 0.90 \text{ psi} )</td>
</tr>
</tbody>
</table>

ACI 318 9.3.2.3

<table>
<thead>
<tr>
<th>Strength-Reduction Factor - Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Phi = 0.75 )</td>
</tr>
</tbody>
</table>

### Steel Reinforcing Properties

- **Minimum Specified Yield Stress of Steel**: \( f_y = 60,000 \text{ psi} \)

#### Assumed Reinforcing Steel

<table>
<thead>
<tr>
<th>Clear Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d = 1.50 \text{ in} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tension Steel Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4 12 in ( d_0 = 0.500 \text{ in} ) bar area = ( 0.20 \text{ in}^2 ) ( A_s = 0.20 \text{ in}^2 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Secondary Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4 12 in ( d_0 = 0.500 \text{ in} ) bar area = ( 0.20 \text{ in}^2 ) ( A_s = 0.20 \text{ in}^2 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Assuming 10% Steel Section Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_{sl} = 0.18 \text{ in}^2 )</td>
</tr>
</tbody>
</table>

#### Flexural Properties

- **Minimum Reinforcement of Flexural Members**
  - \( A_{min} = 3 \cdot \sqrt{L_{ci} / f_y \cdot b \cdot d / 200} \cdot b / d / f_y = 0.10 \text{ in}^2 \)

ACI 318 EQ. 10-3

<table>
<thead>
<tr>
<th>Minimum Reinforcement of Flexural Members</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_{min} = 0.00537 \cdot 12.00 = 0.18 \text{ in}^2 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Minimum Design Reinforcement Steel Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 0.85 \cdot f_y / f'<em>{c,u} \cdot [1 - (1 - \Phi \cdot 0.85 \cdot f_y / f'</em>{c,u})] = \rho = 0.00537 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Minimum Design Reinforcement Steel Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_s = 0.20 \text{ in}^2 )</td>
</tr>
</tbody>
</table>

### Flexural Design Capacity

<table>
<thead>
<tr>
<th>Flexural Design Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_u = 2,088 \text{ lb*ft} \leq \Phi M_b )</td>
</tr>
</tbody>
</table>

OK Flexure
### Shear Design Capacity

<table>
<thead>
<tr>
<th>ACI 318 11-3</th>
<th>Nominal Concrete Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_c = 2\cdot \lambda \cdot \sqrt{(f'_c)} \cdot b \cdot d$</td>
<td>4,174 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ACI 318 11-15</th>
<th>Nominal Reinforcing Steel Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_s = \phi V_n$</td>
<td>3,131 lb</td>
</tr>
</tbody>
</table>

### Design Shear Strength

<table>
<thead>
<tr>
<th>$\Phi V_n = \phi (V_n + V_s)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>OK Shear</td>
</tr>
</tbody>
</table>

### Check Deflection

#### Beam Span Length (Center to Center of Support)

<table>
<thead>
<tr>
<th>$\ell$</th>
</tr>
</thead>
<tbody>
<tr>
<td>101.16 in</td>
</tr>
</tbody>
</table>

#### Moment of Inertia

<table>
<thead>
<tr>
<th>$I$</th>
</tr>
</thead>
<tbody>
<tr>
<td>125 in$^4$</td>
</tr>
</tbody>
</table>

#### Modulus of Elasticity * Moment of Inertia

<table>
<thead>
<tr>
<th>$E \cdot I$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.79E+08 lb*in$^2$</td>
</tr>
</tbody>
</table>

#### Allowable Span Deflection

<table>
<thead>
<tr>
<th>$\Delta \leq \ell / 240$</th>
</tr>
</thead>
<tbody>
<tr>
<td>OK Deflection</td>
</tr>
</tbody>
</table>

#### Floor Members, Dead + Live Load

<table>
<thead>
<tr>
<th>$\Delta \leq \ell / 360$</th>
</tr>
</thead>
<tbody>
<tr>
<td>OK Deflection</td>
</tr>
</tbody>
</table>

### Calculated Deflection

#### Span Dead + Live Load Deflection

<table>
<thead>
<tr>
<th>$\Delta = 5 \cdot w \cdot \ell^3 / (384 \cdot E \cdot I)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.039 in</td>
</tr>
</tbody>
</table>

#### Span Live Load Deflection

<table>
<thead>
<tr>
<th>$\Delta = 5 \cdot w \cdot \ell^3 / (384 \cdot E \cdot I)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.024 in</td>
</tr>
</tbody>
</table>

### ACI 318 11-15

- IBC Table 1604.3
- Floor Members, Dead + Live Load: 0.422 in
- Floor Members, Live Load: 0.281 in

- OK Deflection
Hoboken Waterfront Evaluation: Segment 9-Timber Beam

### Beam Properties
- **Beam Span Length (Center to Center of Support)**: \( \ell = 9.50 \) ft
- **Beam Depth** (height): \( h = 12.00 \) in
- **Beam Width** (width): \( b = 12.00 \) in
- **Tributary Load Width**: \( B = 8.43 \) ft

### Dead Loads

#### IBC Table 1607.1
- **Uniform Dead Loads**
  - **Floor Load**: \( D_1 = 62 \) lb/ft²
  - **Miscellaneous**: \( D_2 = 62 \) lb/ft²

#### IBC Table 1607.9.1
- **Uniform Dead Loads per Beam**: \( B \cdot D_2 = 518 \) lb/ft

#### IBC Eq. 16-25
- **Total Uniform Loads**: \( D = 557 \) lb/ft

#### Floor Live Load
- **Minimum Floor Live Load**: 40. Yards and Terraces, Pedestrians
- **Is Floor Load Reducible**: No
- **Tributary Area for Floor**: \( A_T = 80 \) ft²

#### Live Loads
- **Live Load Element Factor for Girder**: \( K_{LL} = 1.00 \)
- **Reduced Live Load**: \( L_1 = L_0 \cdot (0.25 + 15 \sqrt[4]{K_{LL} \cdot A_T}) = 100 \) lb/ft²
- **Design Moment (Factored Load)**: \( M_u = 0.1 \cdot w_1 \cdot \ell^2 = 12,632 \) lb*ft

### Load Combinations
- **Dead + Live**: \( D + L = w_1 = 1,400 \) lb/ft

### Design Shear and Moment Assuming Simple Span
- **Design Shear (Factored Load)**: \( V_u = 0.6 \cdot w_1 \cdot \ell = 7,978 \) lb
- **Design Moment (Factored Load)**: \( M_u = 0.1 \cdot w_1 \cdot \ell^2 = 12,632 \) lb*ft

### Timber Design Values
- **Timber Species and Grade**: Southern Pine No. 2
- **Timber Specific Gravity**: \( G = 0.55 \)
- **Moisture Content of Wood**: \( m.c. = 25\% \)
- **Timber Density**: \( d = 12 \) lb/cf
- **Member Thickness**: \( b = 12 \) in
- **Wet Service - Moisture content will exceed 19% for an extended time period**: Yes
- **Is Wood Incised**: No

#### NDS Table 4A

<table>
<thead>
<tr>
<th>NDS Appendix B Load Duration Factor</th>
<th>NDS Appendix C Wet Service Factor</th>
<th>NDS Appendix C Temperature Factor</th>
<th>Table 4D Size Factor</th>
<th>NDS Sec. 4.3.8 Incising Factor</th>
<th>Lateral Stability Factor</th>
<th>Bearing Area Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_a )</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( F_v )</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( F_c )</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( F_{ci} )</td>
<td>0.67</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( E )</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**Values**: 974.83 psi, 165.00 psi, 294.80 psi, 1,300,000 psi
### Hoboken Waterfront Evaluation: Segment 9-Timber Beam

<table>
<thead>
<tr>
<th>REF.</th>
<th>CALCULATIONS</th>
<th>ANSWER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unbraced Beam Length</td>
<td>1.00 ft</td>
<td></td>
</tr>
<tr>
<td>Effective Unbraced Length</td>
<td>24.7 in</td>
<td></td>
</tr>
<tr>
<td>Slenderness ratio for bending member</td>
<td>R_B = \sqrt{(l_e \cdot d / b^2)} = 1.4</td>
<td></td>
</tr>
<tr>
<td>F_b = F_b * (product of all adjustment factors except C_fu and C_L)</td>
<td>F_b = 975 psi</td>
<td></td>
</tr>
<tr>
<td>Euler-based ASD critical buckling value for bending members</td>
<td>F_{ub} = 1.20 \cdot E'_{min} / R_B^2 = 273.786 psi</td>
<td></td>
</tr>
<tr>
<td>NDS M3.3-2</td>
<td>Allowable Bending Moment</td>
<td>M = F_b \cdot S = 280,750 lb\cdot in</td>
</tr>
<tr>
<td>NDS M3.4-3</td>
<td>Allowable Shear</td>
<td>V' = 2 \cdot F_b \cdot b^* \cdot d / 3 = 15,840 lb</td>
</tr>
<tr>
<td>NDS M3.10-2</td>
<td>Allowable Bearing (perpendicular to the grain)</td>
<td>P_{1'} = F_b \cdot A_n = 42,451 lb</td>
</tr>
<tr>
<td>Modulus of Elasticity * Moment of Inertia</td>
<td>E' \cdot I = 2,246,400 kips\cdot in^2</td>
<td></td>
</tr>
<tr>
<td>Main Span</td>
<td>L = 9.50 ft</td>
<td></td>
</tr>
<tr>
<td>Assuming that all pile cap spans at least 3 spans</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Allowable Load Bearing</td>
<td>W_b = P_{1'}/L^*6 = 7,448 lb/ft</td>
<td></td>
</tr>
<tr>
<td>Allowable Load Shear</td>
<td>W_s = V'/L^*6 = 2,779 lb/ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>W_s \geq W_1 = OK</td>
<td></td>
</tr>
<tr>
<td>Allowable Load Moment</td>
<td>W_m = M'/L^2*1 = 2,592 lb/ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>W_m \geq W_1 = OK</td>
<td></td>
</tr>
</tbody>
</table>
## References

- American Society of Civil Engineers ASCE/SEI 7-05 Minimum Design Loads for Building and Other Structures
- American Forest & Paper Association (AF&PA) NDS-05 National Design Specifications (NDS) for Wood Construction

## Spacing Between Piles

- Parallel to Pile Cap: 8.43 ft
- Perpendicular to Pile Cap: 9.50 ft

## Dead Load

<table>
<thead>
<tr>
<th>Material</th>
<th>Length</th>
<th>Width</th>
<th>Depth</th>
<th>Density</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber Pile Cap</td>
<td>1</td>
<td>1.00</td>
<td>9.50</td>
<td>38.20</td>
<td>363 lb</td>
</tr>
<tr>
<td>Concrete Deck</td>
<td>0.41</td>
<td>9.50</td>
<td>8.43</td>
<td>150.00</td>
<td>4,925 lb</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5,288 lb</td>
</tr>
</tbody>
</table>

## Live Load

- Yards and Terraces, Pedestrian: 9.50 ft * 8.43 ft * 100.00 lb/sf = 8,009 lb

## Load Combinations

<table>
<thead>
<tr>
<th>Load 1</th>
<th>$P_u = D + L$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>13,297 lb</td>
</tr>
</tbody>
</table>

## Timber Design Values

### Wind & Seismic

- Pile Diameter: 12.00 in

### Southern Pine

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber Specific Gravity G</td>
<td>0.55</td>
</tr>
<tr>
<td>Moisture Content of Wood m.c.</td>
<td>25%</td>
</tr>
<tr>
<td>Timber Density</td>
<td>38.2  lb/cf</td>
</tr>
<tr>
<td>Member Cross Sectional Area A</td>
<td>113.10 in²</td>
</tr>
<tr>
<td>Section Modulus S</td>
<td>169.6 in³</td>
</tr>
<tr>
<td>Moment of Inertia I</td>
<td>1017.9 in⁴</td>
</tr>
<tr>
<td>A/2 * y = Q</td>
<td>144.0  in³</td>
</tr>
</tbody>
</table>

### Allowable Strength

- (Compression): $P_{fc}$ = 1,200 psi
- (Bending): $P_{fb} = 2,400$ psi
- (Shear): $P_{fs} = 110$ psi
- (Perpendicular to the grain): $P_{fc,⊥} = 250$ psi

### Minimum Modulus of Elasticity

- $E_{min} = 790,000$ psi

### NDS Table 4D

<table>
<thead>
<tr>
<th>NDS Appendix B Load Duration Factor $C_D$</th>
<th>NDS Appendix C Temperature Factor $C_T$</th>
<th>Table 3.4 Untreated Factor $C_u$</th>
<th>NDS 4.3.2 Single Pile Factor $C_p$</th>
<th>Critical Section Factor $C_c$</th>
<th>Bearing Area Factor $C_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{cy}$ 1.60</td>
<td>1.0</td>
<td>1.18</td>
<td>0.80</td>
<td>0.65</td>
<td>1.00</td>
</tr>
<tr>
<td>$F_{by}$ 1.60</td>
<td>1.0</td>
<td>1.18</td>
<td>0.80</td>
<td>0.65</td>
<td>1.00</td>
</tr>
<tr>
<td>$F_{ay}$ 1.60</td>
<td>1.0</td>
<td>1.18</td>
<td>0.80</td>
<td>0.65</td>
<td>1.00</td>
</tr>
<tr>
<td>$F_{cy}$ 1.60</td>
<td>1.0</td>
<td>1.18</td>
<td>0.80</td>
<td>0.65</td>
<td>1.00</td>
</tr>
<tr>
<td>$F_{by}$ 1.60</td>
<td>1.0</td>
<td>1.18</td>
<td>0.80</td>
<td>0.65</td>
<td>1.00</td>
</tr>
<tr>
<td>$F_{ay}$ 1.60</td>
<td>1.0</td>
<td>1.18</td>
<td>0.80</td>
<td>0.65</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Unbraced Column Length

- $L = 20.00$ ft

### Axial Loading Only

- $P_a^* = F_{cy} \times (\text{product of all adjustment factors})$
- Euler-based ASD critical buckling value for Columns $F_{cl} = 0.822 \times \frac{P_{ay}^*}{1 / d l^2} = 1,812$ psi
- $E_{cl} = 1,623$ psi

### NDS Table M6.3-1

<table>
<thead>
<tr>
<th>NDS M3.6-2 Allowable Compression</th>
<th>$P = F_{cy} \times A = 133,601$ lb</th>
</tr>
</thead>
<tbody>
<tr>
<td>NDS M3.3-2 Allowable Bending Moment</td>
<td>$M = F_{by} \times S = 591,899$ lb/in</td>
</tr>
<tr>
<td>NDS M3.4-3 Allowable Shear</td>
<td>$V = F_{ay} \times 1 * b / Q = 17,616$ lb</td>
</tr>
<tr>
<td>NDS M3.6-2 Allowable Compression</td>
<td>$P = F_{cy} \times A = 53,382$ lb</td>
</tr>
</tbody>
</table>

### Deteriorated Section (~25% Section Loss)

- $P_a = 40,036$ lb

### Maximum Allowable Force per Pile (deflection)

- $P_d \geq P_u$
## Hoboken Waterfront Evaluation: Segment 10- Slab

### Slab Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab Span Length (Center to Center of Support)</td>
<td>18.00 ft</td>
</tr>
<tr>
<td>Slab Depth</td>
<td>15.00 in</td>
</tr>
<tr>
<td>Minimum Slab Depth for Deflection</td>
<td>Solid One-way Slabs</td>
</tr>
<tr>
<td>h&lt;sub&gt;min&lt;/sub&gt; = ℓ / 20.0</td>
<td>10.80 in</td>
</tr>
<tr>
<td>Slab unit Width</td>
<td>12.00 in</td>
</tr>
</tbody>
</table>

### References

- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

### Slab Loads

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Loads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniform Dead loads</td>
<td>Aggregate Base</td>
<td>D&lt;sub&gt;1&lt;/sub&gt; = 265 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>Miscellaneous</td>
<td>D&lt;sub&gt;2&lt;/sub&gt; = 265 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>Total Uniform Loads</td>
<td>D&lt;sub&gt;T&lt;/sub&gt; = 265 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>Weight of Slab per foot</td>
<td>b * h * w&lt;sub&gt;c&lt;/sub&gt; = 188 lb/ft</td>
</tr>
<tr>
<td></td>
<td>Total Dead Load</td>
<td>D = 453 lb/ft</td>
</tr>
</tbody>
</table>

### Load Combinations

- Design Load: 1.2 * D + 1.6 * L = w<sub>1</sub> = 903 lb/ft
- Deflection Load, Dead + Live: D + L = w<sub>2</sub> = 678 lb/ft
- Deflection Load, Live: L = w<sub>3</sub> = 225 lb/ft

## Design Shear and Moment Assuming Simple Span

### Concrete Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Concrete</td>
<td>130 pcf</td>
</tr>
<tr>
<td>Specified Concrete Compressive Strength</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth</td>
<td>β&lt;sub&gt;1&lt;/sub&gt; = 0.85</td>
</tr>
<tr>
<td>Factor modification reflecting the reduced mechanical properties of lightweight concrete</td>
<td>λ&lt;sub&gt;w&lt;/sub&gt; = 1.00</td>
</tr>
<tr>
<td>Modulus of Concrete</td>
<td>E&lt;sub&gt;c&lt;/sub&gt; = 3.8E+6 psi</td>
</tr>
<tr>
<td>Normal Weight Concrete</td>
<td>Φ = 0.75</td>
</tr>
<tr>
<td>Strength-Reduction Factor - Shear</td>
<td>Φ = 0.90</td>
</tr>
</tbody>
</table>

### Steel Reinforcing Properties

- Minimum Specified Yield Stress of Steel: f<sub>y</sub> = 60,000 psi
- Clear Cover: d = 2.00 in
- Tension Steel Reinforcement: #9 @ 12 in, d<sub>0</sub> = 1.128 in, A<sub>s</sub> = 1.00 in²
- Secondary Reinforcement: #4 @ 12 in, d<sub>s</sub> = 0.500 in, A<sub>s</sub> = 0.20 in²

### Flexural Design Capacity

- R<sub>s</sub> = M<sub>s</sub>/h<sub>b</sub>d<sub>T</sub> = 438.858 / (12.00 * 0.85 * 1.128) = 257 psi
- Minimum Design Reinforcement Steel Ratio: 0.85 * f<sub>y</sub> * f<sub>c</sub> * [1/(1-2 * R<sub>s</sub>/(Φ * 0.85 * f<sub>c</sub>))] = 0.00497
- Minimum Design Reinforcement Steel Area: A<sub>s</sub> = ρ * b * d = 0.00497 * 12.00 * 11.94 = 0.71 in²
- Minimum Reinforcement of Flexural Members: A<sub>s</sub> <sup>min</sup> = 3 * √(f<sub>c</sub> / f<sub>y</sub>) * b * d ≤ 200 * b * d / f<sub>y</sub> = 0.45 in²
- Depth of Equivalent Rectangular Stress Block: a = A<sub>s</sub> * f<sub>c</sub> / (0.85 * f<sub>y</sub> * b) = 1.47 in
- Distance From Extreme Compression Fiber to Neutral axis: c = a / β<sub>1</sub> = 1.73 in
- Nominal Moment: M<sub>N</sub> = A<sub>s</sub> * f<sub>c</sub> * (d - a / 2) = 672,042 lb*ft
- Design Moment: M<sub>u</sub> = 36,572 lb*ft ≤ ΦM<sub>N</sub> = OK Flexure
**Shear Design Capacity**

ACI 318 11-3
Nominal Concrete Shear Strength

\[ V_c = 2 \times \lambda \times \sqrt{f_c'} \times b \times d \]

= 18,118 lb

ACI 318 11-15
Nominal Reinforcing Steel Shear Strength

\[ V_s = \Phi \times V_c \]

\[ V_s = 13,588 \text{ lb} \]

**Design Shear Strength**

\[ V_n = \Phi (V_c + V_s) \]

\[ V_n = 8,127 \text{ lb} \]

\[ V_n \leq \Phi V_n \]

OK Shear

**Check Deflection**

Beam Span Length (Center to Center of Support)

\[ \ell = 216.00 \text{ in} \]

Moment of Inertia

\[ I = 3,375 \text{ in}^4 \]

Modulus of Elasticity * Moment of Inertia

\[ E_c \times I = 1.29 \times 10^9 \text{ lb} \times \text{in}^2 \]

IBC Table 1604.3

Allowable Span Deflection

- Floor Members, Dead + Live Load
  \[ \Delta \leq \ell / 240 = 0.900 \text{ in} \]

- Floor Members, Live Load
  \[ \Delta \leq \ell / 360 = 0.600 \text{ in} \]

**Calculated Deflection**

Span Dead + Live Load Deflection

\[ \Delta = 5 \times w_d \times \ell^3 / (384 \times E_c \times I) \]

\[ \Delta = 0.124 \text{ in} \]

OK Deflection

Span Live Load Deflection

\[ \Delta = 5 \times w_s \times \ell^3 / (384 \times E_c \times I) \]

\[ \Delta = 0.041 \text{ in} \]

OK Deflection

---

**Sub-Headings:**

- Nominal Concrete Shear Strength
- Nominal Reinforcing Steel Shear Strength
- Design Shear Strength
- Beam Span Length
- Moment of Inertia
- Modulus of Elasticity
- IBC Table 1604.3
- Allowable Span Deflection
- Calculated Deflection

---

**Notes:**

- \( f_c' \) is the compressive strength of concrete.
- \( \lambda \) is a factor related to the height-to-span ratio of the beam.
- \( \Phi \) is a strength reduction factor.
- \( w_d \) is the dead load per unit length.
- \( w_s \) is the live load per unit length.
### References
- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

### Beam Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>ℓ</td>
<td>20.00 ft</td>
</tr>
<tr>
<td>Beam Depth</td>
<td>h</td>
<td>36.00 in</td>
</tr>
<tr>
<td>Minimum Beam Depth for Deflection</td>
<td>h&lt;sub&gt;min&lt;/sub&gt; = ℓ / 20.0</td>
<td>12.00 in</td>
</tr>
<tr>
<td>Beam Width</td>
<td>b</td>
<td>48.00 in</td>
</tr>
<tr>
<td>Tributary Load Width</td>
<td>B</td>
<td>18.00 ft</td>
</tr>
</tbody>
</table>

### Dead Loads

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform Dead Loads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck Load</td>
<td>D&lt;sub&gt;1&lt;/sub&gt;</td>
<td>188 lb/ft²</td>
</tr>
<tr>
<td>Aggregate Base Above Deck</td>
<td>D&lt;sub&gt;2&lt;/sub&gt;</td>
<td>265 lb/ft²</td>
</tr>
<tr>
<td>Total Uniform Loads</td>
<td>D&lt;sub&gt;3&lt;/sub&gt;</td>
<td>453 lb/ft²</td>
</tr>
<tr>
<td>Uniform Dead Loads per Beam</td>
<td>B * D&lt;sub&gt;1&lt;/sub&gt;</td>
<td>8,145 lb/ft</td>
</tr>
<tr>
<td>Weight of Beam</td>
<td>b * h * w&lt;sub&gt;c&lt;/sub&gt;</td>
<td>1,800 lb/ft</td>
</tr>
<tr>
<td>Total Dead Load</td>
<td>D&lt;sub&gt;1&lt;/sub&gt;</td>
<td>9,945 lb/ft</td>
</tr>
</tbody>
</table>

### Live Loads

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Floor Live Load</td>
<td>I&lt;sub&gt;0&lt;/sub&gt;</td>
<td>225 lb/ft²</td>
</tr>
<tr>
<td>Is Floor Load Reducible</td>
<td></td>
<td>No</td>
</tr>
<tr>
<td>Tributary Area for Floor</td>
<td>A&lt;sub&gt;t&lt;/sub&gt;</td>
<td>360 ft²</td>
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### Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Load</td>
<td>1.2 * D + 1.6 * I&lt;sub&gt;0&lt;/sub&gt;</td>
<td>18,414 lb/ft</td>
</tr>
<tr>
<td>Deflection Load, Dead + Live</td>
<td>D&lt;sub&gt;L&lt;/sub&gt; = w&lt;sub&gt;L&lt;/sub&gt;</td>
<td>13,995 lb/ft</td>
</tr>
<tr>
<td>Deflection Load, Live</td>
<td>L&lt;sub&gt;L&lt;/sub&gt; = w&lt;sub&gt;L&lt;/sub&gt;</td>
<td>4,050 lb/ft</td>
</tr>
</tbody>
</table>

### Design Shear and Moment Assuming Simple Span

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Shear (Factored Load)</td>
<td>V&lt;sub&gt;s&lt;/sub&gt; = w&lt;sub&gt;L&lt;/sub&gt; * ℓ / 2</td>
<td>184,140 lb</td>
</tr>
<tr>
<td>Design Moment (Factored Load)</td>
<td>M&lt;sub&gt;s&lt;/sub&gt; = w&lt;sub&gt;L&lt;/sub&gt; * ℓ&lt;sup&gt;2&lt;/sup&gt; / 8</td>
<td>920,700 lb*ft</td>
</tr>
</tbody>
</table>

### Concrete Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Concrete</td>
<td>w&lt;sub&gt;c&lt;/sub&gt;</td>
<td>150 pcf</td>
</tr>
<tr>
<td>Specified Concrete Compressive Strength</td>
<td>f&lt;sub&gt;c'&lt;/sub&gt;</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>Normal Weight Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth</td>
<td>f&lt;sub&gt;β&lt;/sub&gt;</td>
<td>0.85</td>
</tr>
<tr>
<td>Modulus of Concrete</td>
<td>E&lt;sub&gt;c&lt;/sub&gt; = w&lt;sub&gt;c&lt;/sub&gt; * 33 * f&lt;sub&gt;c&lt;/sub&gt;'</td>
<td>3,8E+6 psi</td>
</tr>
<tr>
<td>Strength-Reduction Factor - Shear</td>
<td>Φ</td>
<td>0.75</td>
</tr>
<tr>
<td>Strength-Reduction Factor - Flexural sections in pretensioned members</td>
<td>Φ</td>
<td>0.90</td>
</tr>
</tbody>
</table>

### Steel Reinforcing Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Specified Yield Stress of Steel</td>
<td>f&lt;sub&gt;y&lt;/sub&gt;</td>
<td>60,000 psi</td>
</tr>
<tr>
<td>Clear Cover</td>
<td>d&lt;sub&gt;c&lt;/sub&gt;</td>
<td>2.00 in</td>
</tr>
<tr>
<td>Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement</td>
<td>d</td>
<td>32.74 in</td>
</tr>
<tr>
<td>Tension Steel Reinforcement</td>
<td># of Bars = 6</td>
<td>d&lt;sub&gt;u&lt;/sub&gt; = 1.270 in, A&lt;sub&gt;u&lt;/sub&gt; = 7.62 in²</td>
</tr>
<tr>
<td>Number of Shear legs per stirrup</td>
<td>#5</td>
<td>S = 12 in, d&lt;sub&gt;s&lt;/sub&gt; = 0.625 in, A&lt;sub&gt;s&lt;/sub&gt; = 0.62 in²</td>
</tr>
</tbody>
</table>

### Flexural Design Capacity

<table>
<thead>
<tr>
<th>Property</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>R&lt;sub&gt;u&lt;/sub&gt; = M&lt;sub&gt;u&lt;/sub&gt; / (b*b&lt;sub&gt;c&lt;/sub&gt;)</td>
<td>11,048,400 / 8.65</td>
<td>215 psi</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Ratio</td>
<td>0.85 * f&lt;sub&gt;c&lt;/sub&gt;' / f&lt;sub&gt;y&lt;/sub&gt; * [1 - (1 - 2 * R&lt;sub&gt;u&lt;/sub&gt; / (Φ * 0.85 * f&lt;sub&gt;c&lt;/sub&gt;'))]</td>
<td>0.00413</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Area</td>
<td>A&lt;sub&gt;u&lt;/sub&gt; = ρ * b * d&lt;sub&gt;u&lt;/sub&gt;</td>
<td>6.49 in²</td>
</tr>
<tr>
<td>Minimum Reinforcement of Flexural Members</td>
<td>A&lt;sub&gt;min&lt;/sub&gt; = 3 * f&lt;sub&gt;c&lt;/sub&gt;' / f&lt;sub&gt;y&lt;/sub&gt; * b * d</td>
<td>4.97 in²</td>
</tr>
<tr>
<td>If A&lt;sub&gt;u&lt;/sub&gt; is at least one-third greater than that required by analysis A&lt;sub&gt;min&lt;/sub&gt; need not be applied</td>
<td>A&lt;sub&gt;s&lt;/sub&gt; = 4/3 * A&lt;sub&gt;u&lt;/sub&gt;</td>
<td>8.65 in²</td>
</tr>
</tbody>
</table>

### Steel Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Weight Concrete</td>
<td>f&lt;sub&gt;c&lt;/sub&gt;</td>
<td>60,000 psi</td>
</tr>
<tr>
<td>Clear Cover</td>
<td>d&lt;sub&gt;c&lt;/sub&gt;</td>
<td>2.00 in</td>
</tr>
<tr>
<td>Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement</td>
<td>d</td>
<td>32.74 in</td>
</tr>
<tr>
<td>Tension Steel Reinforcement</td>
<td># of Bars = 6</td>
<td>d&lt;sub&gt;u&lt;/sub&gt; = 1.270 in, A&lt;sub&gt;u&lt;/sub&gt; = 7.62 in²</td>
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<tr>
<td>Number of Shear legs per stirrup</td>
<td>#5</td>
<td>S = 12 in, d&lt;sub&gt;s&lt;/sub&gt; = 0.625 in, A&lt;sub&gt;s&lt;/sub&gt; = 0.62 in²</td>
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### Flexural Design Capacity

<table>
<thead>
<tr>
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<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>R&lt;sub&gt;u&lt;/sub&gt; = M&lt;sub&gt;u&lt;/sub&gt; / (b*b&lt;sub&gt;c&lt;/sub&gt;)</td>
<td>11,048,400 / 8.65</td>
<td>215 psi</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Ratio</td>
<td>0.85 * f&lt;sub&gt;c&lt;/sub&gt;' / f&lt;sub&gt;y&lt;/sub&gt; * [1 - (1 - 2 * R&lt;sub&gt;u&lt;/sub&gt; / (Φ * 0.85 * f&lt;sub&gt;c&lt;/sub&gt;'))]</td>
<td>0.00413</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Area</td>
<td>A&lt;sub&gt;u&lt;/sub&gt; = ρ * b * d&lt;sub&gt;u&lt;/sub&gt;</td>
<td>6.49 in²</td>
</tr>
<tr>
<td>Minimum Reinforcement of Flexural Members</td>
<td>A&lt;sub&gt;min&lt;/sub&gt; = 3 * f&lt;sub&gt;c&lt;/sub&gt;' / f&lt;sub&gt;y&lt;/sub&gt; * b * d</td>
<td>4.97 in²</td>
</tr>
<tr>
<td>If A&lt;sub&gt;u&lt;/sub&gt; is at least one-third greater than that required by analysis A&lt;sub&gt;min&lt;/sub&gt; need not be applied</td>
<td>A&lt;sub&gt;s&lt;/sub&gt; = 4/3 * A&lt;sub&gt;u&lt;/sub&gt;</td>
<td>8.65 in²</td>
</tr>
</tbody>
</table>

### OK Minimum Steel

<table>
<thead>
<tr>
<th>Property</th>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Equivalent Rectangular Stress Block</td>
<td>a = A&lt;sub&gt;c&lt;/sub&gt;' * (0.85 * f&lt;sub&gt;c&lt;/sub&gt;' * b)</td>
<td>2.80 in</td>
</tr>
<tr>
<td>Distance From Extreme Compression Fiber to Neutral axis</td>
<td>c = a / b</td>
<td>3.30 in</td>
</tr>
<tr>
<td>Nominal Moment Strength</td>
<td>M&lt;sub&gt;n&lt;/sub&gt; = A&lt;sub&gt;s&lt;/sub&gt; * f&lt;sub&gt;y&lt;/sub&gt; / (d - a / 2)</td>
<td>14,328,312 lb*ft</td>
</tr>
<tr>
<td>Design Moment Strength</td>
<td>ΦM&lt;sub&gt;n&lt;/sub&gt; = M&lt;sub&gt;n&lt;/sub&gt; / (Ψ&lt;sub&gt;2&lt;/sub&gt; * (2/3) * A&lt;sub&gt;s&lt;/sub&gt;)</td>
<td>1,074,623 lb*ft</td>
</tr>
</tbody>
</table>
**Subject:** Hoboken Waterfront Evaluation: Segment 10- Beam

### Shear Design Capacity

**ACI 318 11-3**

Nominal Concrete Shear Strength

\[ V_c = 2 \cdot \lambda \cdot \sqrt{(f'_c)} \cdot b \cdot d \]

\[ V_c = 2 \cdot \lambda \cdot \sqrt{(f'_c)} \cdot b \cdot d = 198,783 \text{ lb} \]

**ACI 318 11-15**

Nominal Reinforcing Steel Shear Strength

\[ V_s = A_v \cdot f_y \cdot d / s \]

\[ V_s = A_v \cdot f_y \cdot d / s = 225,208 \text{ lb} \]

Design Shear Strength

\[ V_d = \Phi (V_c + V_s) \]

\[ V_d = \Phi (V_c + V_s) \leq 184,140 \text{ lb} \]

### Check Deflection

Beam Span Length (Center to Center of Support)

\[ l = 240.00 \text{ in} \]

Moment of Inertia

\[ I = 186,624 \text{ in}^4 \]

Modulus of Elasticity * Moment of Inertia

\[ E_c \cdot I = 7.16 \times 10^{11} \text{ lb*in}^2 \]

Allowable Span Deflection

Floor Members, Dead + Live Load

\[ \Delta \leq \frac{l}{240} = 1.000 \text{ in} \]

Floor Members, Live Load

\[ \Delta \leq \frac{l}{360} = 0.667 \text{ in} \]

### Calculated Deflection

Span Dead + Live Load Deflection

\[ \Delta = 5 \cdot w_2 \cdot \ell^3 / (384 \cdot E \cdot I) = 0.070 \text{ in} \]

Span Live Load Deflection

\[ \Delta = 5 \cdot w_3 \cdot \ell^3 / (384 \cdot E \cdot I) = 0.020 \text{ in} \]

OK Deflection
### References
- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

### Beam Properties
- Beam Span Length (Center to Center of Support) \( l = 4.50 \) ft
- Beam Depth \( h = 48.00 \) in
- Minimum Beam Depth for Deflection \( h_{min} = \frac{l}{20.0} = 2.70 \) in
- Beam Width \( b = 48.00 \) in
- Tributary Load Width \( B = 20.00 \) ft

### Dead Loads
- Uniform Dead Loads
  - Deck Load \( D_1 = 188 \) lb/ft²
  - Aggregate Base Above Deck \( D_2 = 265 \) lb/ft²
  - Concrete Beam \( D_3 = 450 \) lb/ft²
  - Total Uniform Loads \( D_4 = 903 \) lb/ft²
- Uniform Dead Loads per Beam \( B \times D_4 = 18,050 \) lb/ft
- Weight of Beam \( b \times h \times w_c = 2,400 \) lb/ft
- Total Dead Load \( D = 20,450 \) lb/ft

### Live Loads
- Design Floor Live Load \( L_{lf} = 225 \) lb/ft²
- Is Floor Load Reducible \( \text{No} \)
- Tributary Area for Floor \( A_T = 90 \) ft²
- IBC Table 1607.9.1
  - Live Load Element Factor for Girder \( K_L = 1.00 \)
- IBC Eq. 16-25
  - Reduced Live Load \( L_1 = L_{lf} \times (0.25 + 15 \sqrt{\frac{K_L}{A_T}}) = 225 \) lb/ft²
  - \( L_1 = L_{lf} + B = 4,500 \) lb/ft

### Load Combinations
- IBC Eq. 16-2
  - Design Load \( 1.2 \times D + 1.6 \times L_1 = w_s = 31,740 \) lb/ft
  - Deflection Load, Dead + Live \( D + L_1 = w = 24,950 \) lb/ft
  - Deflection Load, Live \( L_1 = w_s = 4,500 \) lb/ft

### Design Shear and Moment Assuming Simple Span
- Design Shear (Factored Load) \( V_s = w_s \times \frac{1}{2} = 71,415 \) lb
- Design Moment (Factored Load) \( M_s = w_s \times \frac{1}{8} = 80,342 \) lb*ft

### Concrete Properties
- Unit Weight of Concrete \( w_c = 150 \) pcf
- Specified Concrete Compressive Strength \( f' = 4,000 \) psi
- ACI 318 10.2.7.3
  - Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth \( \beta_s = 0.85 \)
- ACI 318 8.6.1
  - Modification factor reflecting the reduced mechanical properties of lightweight concrete \( \lambda = 1.00 \)
- ACI 318 8.5.1
  - Normal Weight Concrete \( E_s = w_c^{1/2} \times 33 \times \sqrt{\beta_s} = 3.86E+6 \) psi
  - Modulus of Concrete \( \Phi = 0.75 \)
- ACI 318 9.3.2.3
  - Strength-Reduction Factor - Shear \( \Phi = 0.90 \)

### Steel Reinforcing Properties
- Minimum Specified Yield Stress of Steel \( f_y = 60,000 \) psi
- ASTM A615 Grade 60
- Clear Cover \( d = 14.00 \) in
- Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement \( d_w = 32.74 \) in
- Tension Steel Reinforcement \#10 \# of Bars = 5 \( d_s = 1.270 \) in bar area = 1.27 in² \( A_s = 6.35 \) in²
- Number of Shear legs per stirrup = 2
- Shear Reinforcement \#5 \( s = 6 \) in \( d_h = 0.625 \) in bar area = 0.31 in² \( A_v = 0.62 \) in²

### Flexural Design Capacity
- Minimum Design Reinforcement Steel Ratio \( 0.85 \times \frac{f_y}{f' + 0.32 \times 0.85} \times \sqrt{\beta_s} = 0.00035 \)
- Minimum Design Reinforcement Steel Area \( A_s = \rho \times b \times d = 0.00035 \times 48.00 \times 32.74 = 0.55 \) in²
- ACI 318 EQ. 10.3
  - Minimum Reinforcement of Flexural Members \( \Lambda_s = 3 \times \frac{f_y}{f'} + b \times d \leq 200 \times b \times d / f' = 4.97 \) in²
- ACI 318 10.5.3
  - Flexural Design Capacity \( 902,190 \) lb*ft
  - OK Minimum Steel

### Design Moment Capacity
- Depth of Equivalent Rectangular Stress Block \( a = A_s \times f_y / (0.85 \times f' + b) = 2.33 \) in
- Distance From Extreme Compression Fiber to Neutral axis \( c = a / \beta_s = 2.75 \) in
- Nominal Moment Strength \( M_n = A_s \times f_y \times (d - a / 2) = 12,029,207 \) lb*ft
- Design Moment Strength \( M = 80,342 \) lb*ft \( \leq \Phi M_n = 902,190 \) lb*ft
  - OK Flexure
### Shear Design Capacity

<table>
<thead>
<tr>
<th>Description</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Concrete Shear Strength</td>
<td>$V_c = 2 \cdot \lambda \cdot \sqrt{f'c} \cdot b \cdot d$</td>
<td>198,783 lb</td>
</tr>
<tr>
<td>Nominal Reinforcing Steel Shear Strength</td>
<td>$V_s = A_v \cdot f_y \cdot d / s$</td>
<td>202,988 lb</td>
</tr>
<tr>
<td>Design Shear Strength</td>
<td>$\Phi V_n = \Phi(V_c + V_s)$</td>
<td>301,328 lb</td>
</tr>
<tr>
<td></td>
<td>$V_s = 71,415 \text{ lb} \leq \Phi V_n$</td>
<td>OK Shear</td>
</tr>
</tbody>
</table>

### Check Deflection

<table>
<thead>
<tr>
<th>Description</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>$\ell$</td>
<td>54.00 in</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>$I$</td>
<td>442,368 in$^4$</td>
</tr>
<tr>
<td>Modulus of Elasticity * Moment of Inertia</td>
<td>$E_c \cdot I$</td>
<td>1.70E+12 lb$^2$/in$^2$</td>
</tr>
<tr>
<td></td>
<td>$\Delta \leq \ell / 240 = 0.225 \text{ in}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\Delta \leq \ell / 360 = 0.150 \text{ in}$</td>
<td></td>
</tr>
</tbody>
</table>

### IBC Table 1604.3

<table>
<thead>
<tr>
<th>Description</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allowable Span Deflection</td>
<td>$\Delta \leq \ell / 240 = 0.225 \text{ in}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\Delta \leq \ell / 360 = 0.150 \text{ in}$</td>
<td></td>
</tr>
</tbody>
</table>

### Calculated Deflection

<table>
<thead>
<tr>
<th>Description</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Dead + Live Load Deflection</td>
<td>$\Delta = 5 \cdot w_2 \cdot \ell^3 / (384 \cdot E \cdot I)$</td>
<td>0.000 in</td>
</tr>
<tr>
<td>Span Live Load Deflection</td>
<td>$\Delta = 5 \cdot w_3 \cdot \ell^3 / (384 \cdot E \cdot I)$</td>
<td>0.000 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>OK Deflection</td>
</tr>
<tr>
<td></td>
<td></td>
<td>OK Deflection</td>
</tr>
</tbody>
</table>
Space Between Piles
- Parallel to Pile Cap 18 [ft]
- Perpendicular to Pile Cap 20 [ft]

LOAD

Dead Load
- Concrete Pile Caps 8.7"5"4"150/2 = 13050 [lb]
- Concrete Beam 4"3"20"150/2 = 18000 [lb]
- Prestressed Deck Planks 1.25"20"18"150/2 = 33750 [lb]
- Aggregate Base Above Deck 2.2"18"20"120/2 = 47520 [lb]
- Total = 112320 [lb]

Live Load
- Yards and Terraces, Pedestrians 225"20"18/2 = 40500 [lb]

Load Combinations
- Load 1 1.4*D = 157248 [lb]
- Load 2 1.2*D+1.6*L = 199584 [lb]

DESCRIPTION
- Check design compression strength of concrete-filled round HSS members to AISC LRFD 3rd Ed.
- Material assumed to conform to ASTM A500 Grade B
- Design Table 4-14 from AISC LRFD

INPUT PARAMETERS
- Factored Axial Load Pu = 199.584 [kip] 887 [kN]
- Member Unbraced Length L = 108.0 [in] 2743.2 [mm]
- Member Size Des = HSS16.000X0.375
- Concrete encased or filled? Stu = FILLED
- Area of longitudinal rebars Ar = 2.64 [in^2] 1703 [mm^2]
- Overall diameter including concrete Md = 16 [in] 406 [mm]
- Table Row row = Match(Des,section,0) = 1752

Boundary Conditions
- unbraced length factor Kx = 1.0 [ ] 1.0 [ ]
- unbraced length factor Ky = 1.0 [ ] 1.0 [ ]

MATERIAL
- Yield strength Fy = 36 [ksi] 248 [MPa]
- Elastic Modulus Es = 29000 [ksi] 200000 [MPa]
- Specified compressive resistance of concrete fc = 4 [ksi] 28 [MPa]
- Specified minimum yield stress of rebars Fyr = 60 [ksi] 414 [Mpa]
- Unit weight of Concrete wc = 150 [lb/ft^3] 1034
- Modulus of Elasticity of Concrete Ec = wc^1.5*sqrt(fc) = 3674.234614 [ksi] 25340 [Mpa]

Section Properties
- Member Designation As = index(table,row,4) = HSS16.000X0.375 [ ]  HSS406.4X9.5 [ ]
- Gross Area Dh = index(table,row,7) = 15.69 [ ]
- Radius of Gyration Rm = 5.48 [in] 139 [mm]
- Overall member depth or thickness Trm = index(table,row,15) = 0.195
- Diameter/Thickness Ratio dt = index(table,row,27) = 80.5 [ ] 80 [ ]

CALCULATIONS

SLENDERNESS
- Slenderness SL = max(Kx*L/Rm,Ky*L/Rm) = 19.71 [ ] 19.71 [ ]
- Check Slenderness if(SL>200,"NOT OK","OK") = OK [ ]

Section B7
### AXIAL COMPRESSION

<table>
<thead>
<tr>
<th>Numerical coefficient</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>if(Stu=&quot;FILLED&quot;,1.0,0.7)</td>
<td>1.00</td>
<td>0.85</td>
<td>0.40</td>
<td>Section I2</td>
</tr>
<tr>
<td>if(Stu=&quot;FILLED&quot;,0.85,0.6)</td>
<td>1.00</td>
<td>0.85</td>
<td>0.40</td>
<td>Section I3</td>
</tr>
<tr>
<td>if(Stu=&quot;FILLED&quot;,0.4,0.2)</td>
<td>1.00</td>
<td>0.85</td>
<td>0.40</td>
<td>Section I4</td>
</tr>
</tbody>
</table>

- **Numerical coefficient**
  - \( C_1 = \text{if}(\text{Stu}="\text{FILLED}" , 1.0 , 0.7) = 1.00 \) Section I2
  - \( C_2 = \text{if}(\text{Stu}="\text{FILLED}" , 0.85 , 0.6) = 0.85 \) Section I3
  - \( C_3 = \text{if}(\text{Stu}="\text{FILLED}" , 0.4 , 0.2) = 0.40 \) Section I4

- **Area of concrete**
  \( A_c = 2\pi(\frac{D_t}{2})^2((\pi T_m^2)/4) - A_s) = 183.85 \text{ [in}^2]  
  4669.89 \text{ [mm}^2]  \)

- **Modified Modulus of Elasticity**
  \( E_m = E + E_c C_3 \frac{A_c}{A_s} = 5744.03 \text{ [ksi]}  
  396158.90 \text{ [Mpa]}  \)

- **Limiting Slenderness Parameter**
  \( \lambda_r = \frac{0.1144 \pi}{E_m / F_{my}} = 56.27 \)

- **Column Slenderness Parameter**
  \( \lambda_c = \frac{5L}{(\pi)^2}(\pi r^2) \frac{F_{my}}{E_m r} = 0.285 \)

- **Effective Area Factor**
  \( Q = \frac{E(A_c + A_s)}{A_c(A_s + 2A_c) + 2A_s} = 0.895 \)

- **Critical Stress**
  \( F_{cr} = \frac{0.658 \pi^{\frac{1}{2}}}{(0.0379 \pi r^2)^{\frac{1}{2}}} F_{my} = 114.9 \text{ [ksi]}  
  792.6 \text{ [Mpa]}  \)

- **Elastic Buckling Strength**
  \( P_n = 0.85 A_c F_{cr} = 928 \text{ [kip]}  
  4125 \text{ [kN]}  \)

### DESIGN CHECKS

- **Check cross-sectional area**
  \( A_s < 0.04 (A_c + A_s) \Rightarrow \) NOT OK

- **Check concrete specified strength**
  \( f_c < 3.0 \Rightarrow \) TRUE \Rightarrow \) NOT OK

- **Check rebar specified strength**
  \( f_y > 60 \Rightarrow \) TRUE \Rightarrow \) NOT OK
### References
- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

### Slab Properties
- Slab Span Length (Center to Center of Support) \( \ell \)
- Slab Depth \( h \)
- Minimum Slab Depth for Deflection \( h_{\text{min}} \) = \( \ell / 20.0 \)
- Slab unit Width \( b \)

### Dead Loads
- Uniform Dead Loads
  - Aggregate Base \( D_1 \)
  - Miscellaneous \( D_2 \)
- Total Uniform Loads \( D_T \)
- Weight of Slab per foot \( b \times h \times w_c \)
- Total Dead Load \( D \)

### Live Loads
- Design Floor Live Load \( L_0 \)
- Is Floor Load Reducible
- Tributary Area for Floor \( A_T \)
- Live Load Element Factor for Slab \( K_{\text{LL}} \)
- Reduced Live Load \( L_1 \)
- Load Combinations
  - Design Load \( 1.2 \times D + 1.6 \times L \)
  - Deflection Load, Dead + Live \( D + L \)
  - Deflection Load, Live \( L \)

### Concrete Properties
- Unit Weight of Concrete \( w_c \)
- Specified Concrete Compressive Strength \( f_{c'} \)
- Modulus of Concrete \( E_c \)
- Strength-Reduction Factor - Shear \( \Phi \)
- Strength-Reduction Factor - Flexural sections in pretensioned members \( \Phi \)

### Steel Reinforcing Properties
- Minimum Specified Yield Stress of Steel \( f_y \)
- Clear Cover \( d \)
- Tension Steel Reinforcement
  - #9 \( 12 \) in \( d_9 = 1.128 \) in \( \text{bar area} = 1.00 \) in\(^2\) \( A_s \)
  - Secondary Reinforcement
  - #4 \( 12 \) in \( d_4 = 0.500 \) in \( \text{bar area} = 0.20 \) in\(^2\) \( A_s \)
- Flexural Design Capacity
- Minimum Reinforcement of Flexural Members \( A_{s,\text{min}} \)
- Minimum Reinforcement of Slab
- Design Moment Strength \( M_{\text{des}} \)

### Calculations
- \( w_c = 150 \) pcf
- \( f_{c'} = 4,000 \) psi
- \( 0.90 \)
- \( 0.75 \)
- \( 1.47 \) in
- \( 1.73 \) in
- \( 50,403 \)
- \( 672,042 \)
- \( 60,000 \) psi

### Notes
- OK Flexure
**Shear Design Capacity**

- **Nominal Concrete Shear Strength**
  \[ V_c = 2 \cdot \lambda \cdot \sqrt{(f'_c)} \cdot b \cdot d \]  
  \[ V_c = 18,118 \text{ lb} \]

- **Nominal Reinforcing Steel Shear Strength**
  \[ V_s \]

- **Design Shear Strength**
  \[ \Phi V_n = \Phi (V_n + V_s) \]
  \[ V_n = 9,510 \text{ lb} \]
  \[ \Phi V_n \leq 13,588 \text{ lb} \]  
  **OK Shear**

**Check Deflection**

- **Beam Span Length (Center to Center of Support)**
  \[ \ell = 240.00 \text{ in} \]

- **Moment of Inertia**
  \[ I = 3,375 \text{ in}^4 \]

- **Modulus of Elasticity * Moment of Inertia**
  \[ E \cdot I = 1.29 \times 10^{10} \text{ lb} \cdot \text{in}^2 \]

**IBC Table 1604.3**

- **Allowable Span Deflection**
  - Floor Members, Dead + Live Load
    \[ \Delta \leq \ell / 240 = 1.000 \text{ in} \]
  - Floor Members, Live Load
    \[ \Delta \leq \ell / 360 = 0.667 \text{ in} \]

**Calculated Deflection**

- **Span Dead + Live Load Deflection**
  \[ \Delta = 5 \cdot w_s \cdot \ell^3 / (384 \cdot E \cdot I) \]
  \[ \Delta = 0.197 \text{ in} \]  
  **OK Deflection**

- **Span Live Load Deflection**
  \[ \Delta = 5 \cdot w_s \cdot \ell^3 / (384 \cdot E \cdot I) \]
  \[ \Delta = 0.071 \text{ in} \]  
  **OK Deflection**
**References**


American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

**Beam Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>ℓ = 18.00 ft</td>
</tr>
<tr>
<td>Beam Depth</td>
<td>h = 36.00 in</td>
</tr>
<tr>
<td>Minimum Beam Depth for Deflection</td>
<td>h_{min} = ℓ / 20.0 = 10.80 in</td>
</tr>
<tr>
<td>Beam Width</td>
<td>b = 48.00 in</td>
</tr>
<tr>
<td>Tributary Load Width</td>
<td>B = 20.00 ft</td>
</tr>
</tbody>
</table>

**Beam Properties**

**Dead Loads**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform Dead Loads</td>
<td>D = 188 lb/ft²</td>
</tr>
<tr>
<td>Aggregate Base Above Deck</td>
<td>D₂ = 265 lb/ft²</td>
</tr>
<tr>
<td>Total Uniform Loads</td>
<td>D₁ = 453 lb/ft²</td>
</tr>
<tr>
<td>Uniform Dead Loads per Beam</td>
<td>B₁ * D₁ = 9.050 lb/ft</td>
</tr>
<tr>
<td>Weight of Beam</td>
<td>b₁ * h₁ * w₁ = 1.800 lb</td>
</tr>
<tr>
<td>Total Dead Load</td>
<td>D = 10.850 lb/ft</td>
</tr>
</tbody>
</table>

**Live Loads**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Floor Live Load</td>
<td>L₀ = 255 lb/ft²</td>
</tr>
<tr>
<td>Is Floor Load Reducible</td>
<td>No</td>
</tr>
<tr>
<td>Tributary Area for Floor</td>
<td>A_T = 360 ft²</td>
</tr>
</tbody>
</table>

**Load Combinations**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Load</td>
<td>1.2 * D + 1.6 * L₁ = 21.180 lb/ft</td>
</tr>
<tr>
<td>Deflection Load, Dead + Live</td>
<td>D₁ + L₁ = 15.950 lb/ft</td>
</tr>
<tr>
<td>Deflection Load, Live</td>
<td>L₁ = 5.100 lb/ft</td>
</tr>
</tbody>
</table>

**Design Shear and Moment Assuming Simple Span**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Shear (Factored Load)</td>
<td>Vₙ = 190.620 lb</td>
</tr>
<tr>
<td>Design Moment (Factored Load)</td>
<td>Mₙ = 857,790 lb*ft</td>
</tr>
</tbody>
</table>

**Concrete Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Concrete</td>
<td>w_c = 150 pcf</td>
</tr>
<tr>
<td>Specified Concrete Compressive Strength</td>
<td>f'_c = 4,000 psi</td>
</tr>
<tr>
<td>Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth</td>
<td>β₁ = 0.85</td>
</tr>
<tr>
<td>Normal Weight Concrete</td>
<td>μ_c = 1.00</td>
</tr>
<tr>
<td>Moisture Modulus of Concrete</td>
<td>E_w = w_c * 33 * √ f'_c = 3.8E+6 psi</td>
</tr>
<tr>
<td>Strength-Reduction Factor - Shear</td>
<td>Φ₁ = 0.75</td>
</tr>
<tr>
<td>Strength-Reduction Factor - Flexural sections in pretensioned members</td>
<td>Φ₂ = 0.90</td>
</tr>
</tbody>
</table>

**Steel Reinforcing Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Specified Yield Stress of Steel</td>
<td>f_y = 60,000 psi</td>
</tr>
<tr>
<td>Clear Cover</td>
<td>d = 2.00 in</td>
</tr>
<tr>
<td>Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement</td>
<td>d₁ = 32.94 in</td>
</tr>
<tr>
<td>Tension Steel Reinforcement</td>
<td>Aₚ = 6.00 in²</td>
</tr>
<tr>
<td>Number of Shear legs per stirrup</td>
<td>a = 2</td>
</tr>
<tr>
<td>Shear Reinforcement</td>
<td>s = 14 in</td>
</tr>
<tr>
<td>Area of Tension Reinforcement</td>
<td>dₙ₁ = 0.500 in²</td>
</tr>
</tbody>
</table>

**Flexural Design Capacity**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Reinforcement of Flexural Members</td>
<td>A_{n, max} = 3 * √ f_y / b₁ * d₁ / 2 = 5.00 in²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Equivalent Rectangular Stress Block</td>
<td>a = A_n * f_y / (0.85 * f'_c * b₁) = 2.21 in</td>
</tr>
<tr>
<td>Distance From Extreme Compression Fiber to Neutral axis</td>
<td>c = a / b₁ = 2.60 in</td>
</tr>
<tr>
<td>Nominal Moment Strength</td>
<td>Mₚ = 11,459,901 lb*ft</td>
</tr>
<tr>
<td>Design Moment Strength</td>
<td>M_down = 857,790 lb*ft</td>
</tr>
</tbody>
</table>

**Answer**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>OK Flexure</td>
<td></td>
</tr>
<tr>
<td>Shear Design Capacity</td>
<td></td>
</tr>
<tr>
<td>-----------------------</td>
<td>--------------------</td>
</tr>
<tr>
<td><strong>Nominal Concrete Shear Strength</strong></td>
<td>( V_s = 2 \lambda \sqrt{f'} \cdot b \cdot d ) = 199,973 lb</td>
</tr>
<tr>
<td><strong>Nominal Reinforcing Steel Shear Strength</strong></td>
<td>( V_s = A_s \cdot f_y \cdot d / s ) = 56,462 lb</td>
</tr>
<tr>
<td><strong>Design Shear Strength</strong></td>
<td>( \Phi V_n = \Phi (V_n + V_s) ) = 192,326 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Check Deflection</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>( l ) = 216.00 in</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>( I ) = 186,624 in^4</td>
</tr>
<tr>
<td>Modulus of Elasticity * Moment of Inertia</td>
<td>( E_c \cdot I ) = 7.16E+11 lb*in^2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>IBC Table 1604.3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Allowable Span Deflection</td>
<td>Floor Members, Dead + Live Load ( \Delta \leq \frac{l}{240} ) = 0.900 in</td>
</tr>
<tr>
<td></td>
<td>Floor Members, Live Load ( \Delta \leq \frac{l}{360} ) = 0.600 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Calculated Deflection</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Dead + Live Load Deflection</td>
<td>( \Delta = 5 \cdot w_2 \cdot l^3 / (384 \cdot E \cdot I) ) = 0.053 in</td>
</tr>
<tr>
<td>Span Live Load Deflection</td>
<td>( \Delta = 5 \cdot w_3 \cdot l^3 / (384 \cdot E \cdot I) ) = 0.017 in</td>
</tr>
</tbody>
</table>

**OK Shear**
**References**


American Concrete Institute (ACI) Building Code Requirements for Structural Concrete ACI-318-08

**Beam Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span Length (Center to Center of Support)</td>
<td>ℓ</td>
<td>4.00 ft</td>
</tr>
<tr>
<td>Beam Depth</td>
<td>h</td>
<td>48.00 in</td>
</tr>
<tr>
<td>Minimum Beam Depth for Deflection</td>
<td>h&lt;sub&gt;min&lt;/sub&gt; = ℓ / 20.0</td>
<td>2.40 in</td>
</tr>
<tr>
<td>Beam Width</td>
<td>b</td>
<td>48.00 in</td>
</tr>
<tr>
<td>Tributary Load Width</td>
<td>B</td>
<td>18.00 ft</td>
</tr>
</tbody>
</table>

**Dead Loads**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform Dead Loads</td>
<td>Deck Load</td>
<td>D₁ = 188 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>Aggregate Base Above Deck</td>
<td>D₂ = 265 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>Concrete Beam</td>
<td>D₃ = 450 lb/ft²</td>
</tr>
<tr>
<td>Total Uniform Loads</td>
<td>D₄ = 903 lb/ft²</td>
<td></td>
</tr>
<tr>
<td>Uniform Dead Loads per Beam</td>
<td>B * D₄ = 16,245 lb/ft</td>
<td></td>
</tr>
<tr>
<td>Weight of Beam</td>
<td>b * h * w₄ = 2,400 lb/ft</td>
<td></td>
</tr>
<tr>
<td>Total Dead Load</td>
<td>D = 18,645 lb/ft</td>
<td></td>
</tr>
</tbody>
</table>

**Live Loads**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Floor Live Load</td>
<td>Is Floor Load Reduceable</td>
<td>No</td>
</tr>
<tr>
<td>Tributary Area for Floor</td>
<td>Aₓ</td>
<td>72 ft²</td>
</tr>
<tr>
<td>Live Load Element Factor for Girder</td>
<td>Kₓ</td>
<td>1.00</td>
</tr>
<tr>
<td>Reduced Live Load</td>
<td>L₁ = L₀ * (0.25 + 15 / √(Kₓ * Aₓ)) = 255 lb/ft²</td>
<td></td>
</tr>
<tr>
<td></td>
<td>L = L₁ * B = 4,590 lb/ft</td>
<td></td>
</tr>
</tbody>
</table>

**Load Combinations**

<table>
<thead>
<tr>
<th>Combination</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Load</td>
<td>1.2 * D + 1.6 * L = w₁</td>
<td>29,718 lb/ft</td>
</tr>
<tr>
<td>Deflection Load, Dead + Live</td>
<td>D + L = w₂</td>
<td>23,235 lb/ft</td>
</tr>
<tr>
<td>Deflection Load, Live</td>
<td>L = w₃</td>
<td>4,590 lb/ft</td>
</tr>
</tbody>
</table>

**Concrete Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Concrete</td>
<td>wₜ</td>
<td>150 pcf</td>
</tr>
<tr>
<td>Specified Concrete Compressive Strength</td>
<td>f&lt;sub&gt;c&lt;/sub&gt;</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>Factor relating breadth of equivalent rectangular compressive stress block to neutral axis depth</td>
<td>β₁</td>
<td>0.85</td>
</tr>
<tr>
<td>Modification factor reflecting the reduced mechanical properties of lightweight concrete</td>
<td>λ</td>
<td>1.00</td>
</tr>
<tr>
<td>Normal Weight Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ACI 318 10.2.7.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ACI 318 8.6.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ACI 318 8.5.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ACI 318 9.3.2.3</td>
<td></td>
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</tr>
<tr>
<td>ACI 318 9.3.2.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Reinforcing Properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Specified Yield Stress of Steel</td>
<td>fₚ</td>
<td>60,000 psi</td>
</tr>
<tr>
<td>Clear Cover</td>
<td>d</td>
<td>14.00 in</td>
</tr>
<tr>
<td>Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement</td>
<td>dₚ</td>
<td>32.74 in</td>
</tr>
<tr>
<td>Tension Steel Reinforcement</td>
<td>#10</td>
<td>1.270 in²</td>
</tr>
<tr>
<td>Number of Shear legs per stirrup</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Shear Reinforcement</td>
<td>#5</td>
<td>0.625 in²</td>
</tr>
<tr>
<td>Flexural Design Capacity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rₚ</td>
<td>M₀/(b*d&lt;sup&gt;2&lt;/sup&gt;)</td>
<td>713,232</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Ratio</td>
<td>0.85 * fₚ / f₀ * [1 / [(1 - 2 * Rₚ / (f₀ * 0.85 * fₚ))]</td>
<td>0.00026</td>
</tr>
<tr>
<td>Minimum Design Reinforcement Steel Area</td>
<td>A₀ = ρ * b * d</td>
<td>0.00026</td>
</tr>
<tr>
<td>Minimum Reinforcement of Flexural Members</td>
<td>A₀, min = 3 * √(fₚ / f₀) * b * d</td>
<td>4.97 in²</td>
</tr>
<tr>
<td>Depth of Equivalent Rectangular Stress Block</td>
<td>a = A₀ * fₚ / (0.85 * fₚ * b)</td>
<td>2.33 in</td>
</tr>
<tr>
<td>Distance From Extreme Compression Fiber to Neutral axis</td>
<td>c = a / β₁</td>
<td>2.75 in</td>
</tr>
<tr>
<td>Nominal Moment Strength</td>
<td>M₀ = A₀ * fₚ * (d - a / 2)</td>
<td>12,029,207 lb*in</td>
</tr>
<tr>
<td>Design Moment Strength</td>
<td>M₀ = 59,436 lb*ft</td>
<td>OK Flexure</td>
</tr>
</tbody>
</table>
### Shear Design Capacity

<table>
<thead>
<tr>
<th>Standard</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318 11-3</td>
<td>Nominal Concrete Shear Strength</td>
<td>$V_c = 2 \lambda \sqrt{f_{c}'} b d$</td>
</tr>
<tr>
<td>ACI 318 11-15</td>
<td>Nominal Reinforcing Steel Shear Strength</td>
<td>$V_s = A_v f_y d / s$</td>
</tr>
<tr>
<td>Design Shear Strength</td>
<td>$V_d = \Phi (V_c + V_s)$</td>
<td>301,328 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear Capacity</th>
<th>$V_s$</th>
<th>$\Phi V_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>59,436 lb</td>
<td>≤ OK Shear</td>
<td></td>
</tr>
</tbody>
</table>

### Check Deflection

- **Beam Span Length (Center to Center of Support):** $l = 48.00$ in
- **Moment of Inertia:** $I = 442,368$ in$^4$
- **Modulus of Elasticity * Moment of Inertia:** $E_c I = 1.70 \times 10^{12}$ lb*in$^2$

<table>
<thead>
<tr>
<th>Allowable Span Deflection</th>
<th>Floor Members, Dead + Live Load</th>
<th>$\Delta \leq \frac{l}{240}$</th>
<th>0.200 in</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Floor Members, Live Load</td>
<td>$\Delta \leq \frac{l}{360}$</td>
<td>0.133 in</td>
</tr>
</tbody>
</table>

### Calculated Deflection

<table>
<thead>
<tr>
<th>Deflection Type</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Dead + Live Load Deflection</td>
<td>$\Delta = 5 w_2 \ell^3 / (384 E I)$</td>
<td>0.000 in</td>
</tr>
<tr>
<td>Span Live Load Deflection</td>
<td>$\Delta = 5 w_3 \ell^3 / (384 E I)$</td>
<td>OK Deflection</td>
</tr>
</tbody>
</table>

IBC Table 1604.3:
- Floor Members, Dead + Live Load: 0.200 in
- Floor Members, Live Load: 0.133 in

ACI 318 11-15

ACI 318 11-3

IBC Table 1604.3

Floor Members, Dead + Live Load

0.200 in

Floor Members, Live Load

0.133 in

OK Deflection
COMPOSITE ROUND HSS SECTION COMPRESSION DESIGN (concrete filled only)

Space Between Piles
- Parallel to Pile Cap: 20 [ft]
- Perpendicular to Pile Cap: 26 [ft]

LOAD

Dead Load
- Concrete Pile Caps: 9.1"*6.1"*150/2 = 29142.75 [lb]
- Concrete Beam: 4.3"*18*150/2 = 16200 [lb]
- Pressurized Deck Planks: 1.25"*20*18*150/2 = 33750 [lb]
- Aggregate Base Above Deck: 2.2"*18*20*120/2 = 47520 [lb]
- Total: 126612.75 [lb]

Live Load
- Yards and Terraces, Pedestrians: 255"*20*18/2 = 45900 [lb]

Load Combinations
- Load 1: 1.4*D = 177257.85 [lb]
- Load 2: 1.2*D+1.6*L = 225375.3 [lb]

DESCRIPTION
- Check design compression strength of concrete-filled round HSS members to AISC LRFD 3rd Ed.
- Material assumed to conform to ASTM A500 Grade B
- Design Table 4-14 from AISC LRFD

INPUT PARAMETERS
- Factored Axial Load Pu = 225.3753 [kip] 1002 [kN]
- Member Unbraced Length L = 108.0 [in] 2743.2 [mm]
- Member Size Des = HSS16.000X0.375 []
- Concrete encased or filled? Stu = FILLED []
- Area of longitudinal rebars Ar = 2.64 [in^2] 1703 [mm^2]
- Overall diameter including concrete Md = 16 [in] 406 [mm]
- Table Row row = Match(Des,section,0) = 1752 []

- Boundary Conditions
  - unbraced length factor Kx = 1.0 []
  - unbraced length factor Ky = 1.0 []

- MATERIAL
  - Yield strength Fy = 36 [ksi] 248 [MPa]
  - Elastic Modulus Es = 29000 [ksi] 200000 [MPa]
  - Specified compressive resistance of concrete fc = 4 [ksi] 28 [MPa]
  - Specified minimum yield stress of rebars Fyr = 60 [ksi] 414 [MPa]
  - Unit weight of Concrete wc = 150 [lb/ft^3] 1034 [kN/m^3]

- Modulus of Elasticity of Concrete Ec = wc*1.5*sqrt(fc) = 3674.234614 [ksi] 25340 [Mpa]

Section Properties
- Member Designation As = index(table,row,4) = HSS16.000X0.375 [] HSS406.4X9.5 []
- Gross Area A = index(table,row,4) = 15.70 [in^2] 10129 [mm^2]
- HSS diameter Dh = index(table,row,7) = 15.94 [in]
- Radius of Gyration Rm = index(table,row,34)=0.3*Md.index(table,row,7)/3 = 0.3*Md.index(table,row,7) = 5.52 [in] 140 [mm]
- Overall member depth or thickness Trm = if(index(table,row,34)>0.3*Md,index(table,row,7),0.3*Md) = 15.94 [in]
- Wall thickness wt = index(table,row,15) = 0.320 [in]
- Diameter/Thickness Ratio dt = index(table,row,27) = 49.8 []

CALCULATIONS

- SLENDERNESS
  - Slenderness SL = max(Kx*L/Rm,Ky*L/Rm) = 19.57 []
  - Check Slenderness if(SL>200,NOT OK,"OK") = OK []

Section B7
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>-Numerical coefficient C1</td>
<td>1.00</td>
<td>1.00</td>
<td>Section I2</td>
</tr>
<tr>
<td>-Numerical coefficient C2</td>
<td>0.85</td>
<td>0.85</td>
<td>Section I3</td>
</tr>
<tr>
<td>-Numerical coefficient C3</td>
<td>0.40</td>
<td>0.40</td>
<td>Section I4</td>
</tr>
<tr>
<td>-Area of concrete Ac</td>
<td>183.90</td>
<td>in^2</td>
<td>4671.11 mm^2</td>
</tr>
<tr>
<td>-Modified Yield strength Fmy</td>
<td>85.92</td>
<td>ksi</td>
<td>592.52 Mpa</td>
</tr>
<tr>
<td>-Modified Modulus of Elasticity Em</td>
<td>46215.26</td>
<td>ksi</td>
<td>318725.91 Mpa</td>
</tr>
<tr>
<td>-Limiting Slenderness Parameter (\lambda_r)</td>
<td>61.32</td>
<td>kN</td>
<td>61.32</td>
</tr>
<tr>
<td>-Column Slenderness Parameter (\lambda_c)</td>
<td>0.269</td>
<td>kN</td>
<td>0.269</td>
</tr>
<tr>
<td>-Effective Area Factor Q</td>
<td>1.00</td>
<td></td>
<td>(4.2-4)</td>
</tr>
<tr>
<td>-Critical Stress For</td>
<td>83.4</td>
<td>kN</td>
<td>574.9</td>
</tr>
<tr>
<td>-Elastic Buckling Strength Pn</td>
<td>1112</td>
<td>kip</td>
<td>4944 kN</td>
</tr>
<tr>
<td>-Check for Elastic Buckling</td>
<td>OK</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-Check cross-sectional area</td>
<td>OK</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-Check concrete specified strength</td>
<td>OK</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-Check rebar specified strength</td>
<td>OK</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Soil Properties

- **Unit Weight of Soil**: 120 pcf
- **Unit Weight of Concrete**: 150 pcf
- **Effective angle of internal friction**: 30.0 degrees
- **Allowable Soil Bearing Pressure**: lb/sf
- **Coefficient of Friction**: μ = 0.58
- **Slope at Top of Wall**: 2
- **Angle of Back Face of Wall**: θ = 74.5 degrees
- **Effective angle of internal friction**: ϕ = 20.0 degrees
- **γ _soil_)**
- **γ _c_**
- **γ _f_**
- **γ _K_**
- **γ _L_**
- **γ _β_**

### Wall Properties

- **Width of top of Wall**: W₁ = 3.00 ft
- **Back Wall Slope**: S₁ = 1 H 4 V
- **Front Triangular base width**: W₂ = - ft
- **Front Wall Slope**: S₂ = 0 H 1 V
- **Back Triangular base width**: W₃ = 2.00 ft
- **Exposed Wall Height**: H₁ = 7.20 ft
- **Total Footing Width**: B = 5.00 ft
- **Below Grade Wall Height**: Hₑ = 7.20 ft
- **Total Wall Height**: H = 7.20 ft

### Calculations

#### Soil pressure

- Vertical: Pₐ₁ = 0.5 * γ _soil_ * Kₐ * (H)² = 1.337 lb
- Moment arm about Toe: 0.00 * 7.20 = 0.00 ft³

#### Resisting Forces and Moments

<table>
<thead>
<tr>
<th>Area A = Front Triangular Area</th>
<th>Section Area ft²</th>
<th>Unit Weight lb/ft²</th>
<th>Weight lb</th>
<th>Moment Arm ft</th>
<th>Moment ft²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area B = Center Rectangular Area</td>
<td>21.60</td>
<td>150</td>
<td>3,240</td>
<td>1.50</td>
<td>4,860</td>
</tr>
<tr>
<td>Area C = Back Triangular Area</td>
<td>7.20</td>
<td>150</td>
<td>1,080</td>
<td>3.67</td>
<td>3,960</td>
</tr>
<tr>
<td>Soil Vertical Loading</td>
<td>777</td>
<td>4.33</td>
<td>3,367</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surchage Vertical Loading</td>
<td>100</td>
<td>4.33</td>
<td>867</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| W = 5.297 | Mᵢ = 13,054 |

#### Overturning Forces and Moments

<table>
<thead>
<tr>
<th>Force lb</th>
<th>Moment Arm ft</th>
<th>Moment ft²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Pressure</td>
<td>1,088</td>
<td>2.40</td>
</tr>
<tr>
<td>Surchage Horizontal Loading</td>
<td>310</td>
<td>1.11</td>
</tr>
</tbody>
</table>

| Pᵢ | Mᵢ = 3.726 |

#### Check Wall Stability

- **Factor of Safety for Overturning**: FSₒ = Mᵢ / W * µ = 13,054 / 5.297 * 0.58 = OK
- **Force resisting**: 3,058 lb
- **Factor of Safety for Sliding**: FSₛ = W * µ / Pᵢ = 3,058 / 3,190 = OK

### Beam Properties
- **Beam Span Length (Center to Center of Support)**: \( \ell = 5.00 \text{ ft} \)
- **Beam Depth** : \( h = 12.00 \text{ in} \)
- **Beam Width** : \( b = 12.00 \text{ in} \)
- **Tributary Load Width** : \( B = 2.50 \text{ ft} \)

### Dead Loads
- **Uniform Dead Loads**
  - Seawall: \( D_1 = 864 \text{ lb/ft}^2 \)
  - Earth Fill: \( D_2 = 166 \text{ lb/ft}^2 \)
  - Timber Deck: \( D_3 = 13 \text{ lb/ft}^2 \)
- **Total Uniform Loads**: \( D_T = 1,042 \text{ lb/ft}^2 \)

### Live Loads
- **Minimum Floor Live Load**
  - 40. Yards and Terraces, Pedestrians
  - Is Floor Load Reducible: No
  - Tributary Area for Floor: \( A_T = 13 \text{ ft}^2 \)
  - Reduced Live Load
    \[ L_i = L_0 \times (0.25 + 15 \sqrt{K_{LL} \times A_T}) \]
  - Dead + Live
    \[ D + L = 2,894 \text{ lb/ft} \]

### Design Shear and Moment Assuming Simple Span
- **Design Shear (Factored Load)**
  \[ V_u = 0.6w_1 \times \ell = 8,681 \text{ lb} \]
- **Design Moment (Factored Load)**
  \[ M_u = 0.1w_1 \times \ell^2 = 7,234 \text{ lb/ft} \]

### Timber Design Values
- **Timber Species and Grade**: Southern Pine No. 2

### Timber Design Values
- **Timber Specific Gravity**: \( G = 0.55 \)
- **Moisture Content of Wood**: \( m.c. = 25\% \)
- **Timber Density**: \( d = 38.2 \text{ lb/cf} \)
- **Member Depth**: \( d = 12 \text{ in} \)
- **Member Thickness**: \( b = 12 \text{ in} \)
- **Wet Service - Moisture content will exceed 19\% for an extended time period**: Yes
- **Is Wood Incised**: No
- **Loading Type**: Floor Live Load
- **MemberCross Sectional Area**: \( A = 144.00 \text{ in}^2 \)
- **Member Bearing Length (perpendicular to grain)**: \( L_0 = 12.00 \text{ in} \)
- **Member Bearing Area (perpendicular to grain)**: \( A_T = 144.00 \text{ in}^2 \)
- **Section Modulus**: \( S = 288.00 \text{ in}^3 \)
- **Moment of Inertia**: \( I = 1,728.00 \text{ in}^4 \)
- **Weight per foot of length**: \( w_0 = 38.18 \text{ lb/ft} \)
- **Allowable Strength (Bending)**: \( F_b = 975 \text{ psi} \)
- **Allowable Strength (Shear)**: \( F_v = 165 \text{ psi} \)
- **Allowable Strength (Compression perpendicular to the grain)**: \( F_{Ck} = 440 \text{ psi} \)
- **Modulus of Elasticity**: \( E = 1,300,000 \text{ psi} \)
- **Minimum Modulus of Elasticity**: \( E_{min} = 470,000 \text{ psi} \)

### Calculation Table

<table>
<thead>
<tr>
<th>NDS Appendix B Load Duration Factor</th>
<th>Table 4D Wet Service Factor</th>
<th>NDS Appendix C Temperature Factor</th>
<th>Table 4D Size Factor</th>
<th>NDS Sec. 4.3.8 Incising Factor</th>
<th>Lateral Stability Factor</th>
<th>Bearing Area Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_a ) 1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( F_v ) 1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( F_{Ck} ) 0.87</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( E_{min} ) 1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>( E ) 1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**Answer**: Values are calculated based on the given data and calculations.
**SUBJECT**
Hoboken Waterfront Evaluation: Segment 12-Timber Pile Cap

| Unbraced Beam Length |  |  |  |  
|----------------------|---|---|---|---|
| Effective Unbraced Length |  |  |  |  
| Slenderness ratio for bending member |  |  |  |  
| $F_b = F_s \cdot (product\ of\ all\ adjustment\ factors\ except\ C_{fu} \ and\ C_L)$ |  |  |  |  
| Euler-based ASD critical buckling value for bending members |  |  |  |  
| $R_b = \sqrt{l_e \cdot d / b^2}$ |  |  |  |  
| $F_{bE} = 1.20 \cdot E'_{min} / R_b^2$ |  |  |  |  

- $F_{bE} = 1.20 \cdot E'_{min} / R_b^2$
- $F_{bE} = \psi_E\ Euler-based\ ASD\ critical\ buckling\ value\ for\ bending\ members$
- $F_{bE} = 1.20 \cdot E'_{min} / R_b^2$
- $F_{bE} = \psi_E$
- $E'_{min} = \psi_E\ Euler-based\ ASD\ critical\ buckling\ value\ for\ bending\ members$
- $E'_{min} = \psi_E$
- $E'_{min} = \psi_E$
- $E'_{min} = \psi_E$
- $E'_{min} = \psi_E$
- $E'_{min} = \psi_E$
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### References
- American Society of Civil Engineers ASCE/SEI 7-05 Minimum Design Loads for Building and Other Structures
- American Forest & Paper Association (AF&PA) NDS-05 National Design Specifications (NDS) for Wood Construction

### Spacing Between Piles
- Parallel to Pile Cap
- Perpendicular to Pile Cap

### Dead Load
- Timber Pile Cap: $2 \times 1.00 \text{ ft} \times 1.00 \text{ ft} \times 2.50 \text{ ft} \times 38.20 \text{ lb/ft}^3 = 191 \text{ lb}$
- Sea Wall: $7.20 \text{ ft} \times 2.50 \text{ ft} \times 5.00 \text{ ft} \times 150.00 \text{ lb/ft}^3 = 13,500 \text{ lb}$
- Total: $13,691 \text{ lb}$

### Live Load
- Yards and Terraces, Pedestrian: $2.50 \text{ ft} \times 5.00 \text{ ft} \times 100.00 \text{ lb/ft}^2 = 1,250 \text{ lb}$

### Load Combinations
- Load 1: $P_u = D + L = 14,941 \text{ lb}$

### Timber Design Values
- Pile Diameter
- Timber Species & Grade: Southern Pine
- Timber Specific Gravity: $G = 0.55$
- Moisture Content of Wood: $\text{m.c.} = 25\%$
- Timber Density: $\text{lb/ft}^3 = 38.2$
- Member Cross Sectional Area: $\text{in}^2 = 113.10$
- Section Modulus: $\text{in}^3 = 169.6$
- Moment of Inertia: $\text{in}^4 = 1017.9$
- $A/2 \times y = Q = 144.0$
- Allowable Strength (Compression): $F_{c} = 1,200 \text{ psi}$
- Allowable Strength (Bending): $F_b = 2,400 \text{ psi}$
- Allowable Strength (Shear): $F_v = 110 \text{ psi}$
- Allowable Strength (Compression perpendicular to the grain): $F_{c\perp} = 250 \text{ psi}$
- Modulus of Elasticity: $E = 1,500,000 \text{ psi}$
- Minimum Modulus of Elasticity: $E_{\text{min}} = 790,000 \text{ psi}$

### NDS Table 4D

<table>
<thead>
<tr>
<th>NDS Appendix B</th>
<th>NDS Appendix C</th>
<th>NDS 4.3.2</th>
<th>Single Pile Factor</th>
<th>Column Stability Factor</th>
<th>Critical Section Factor</th>
<th>Bearing Area Factor</th>
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</thead>
<tbody>
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<td>Load Duration Factor</td>
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<td>Untreated Factor</td>
<td>Size Factor</td>
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<td>$C_t$</td>
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</tr>
</tbody>
</table>

### Unbraced Column Length
- $L = 4.60 \text{ ft}$
- $c = 0.80$

### Axial Loading Only
- $F^* = F_L \times (\text{product of all adjustment factors})$
- Euler-based ASD critical buckling value for Columns: $F_{\text{cr}} = 30,689 \text{ psi}$
- $F_{\text{cr}} = 0.822 \times F_{\text{cr}} \times (L/D)^2$

### NDS Table M6.3-1

### Minimum Modulus of Elasticity
- $E_{\text{cr}} = 790,000 \text{ psi}$

### Axial Loading Only
- $P_a = F_L \times A$
- $P_a = 127,150 \text{ lb}$

### Allowable Compression
- $P = F_L \times A$
- $P = 33,364 \text{ lb}$

### Deteriorated Section (~10% Section Loss)
- $P_a = 30,027 \text{ lb}$
- $P_a \geq P_u \Rightarrow \text{OK}$
APPENDIX F

AS-BUILT DRAWINGS
Segment 8 As-builts
Segment 10 As-builts
Segment 11 As-builts