MAMARONECK & SHELDRAKE RIVERS
NEW YORK
FLOOD RISK MANAGEMENT
GENERAL REEVALUATION REPORT
FOR
THE VILLAGE OF MAMARONECK

APPENDIX C4: STRUCTURAL

US Army Corps of Engineers®
New York District

JANUARY 2016

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1.0 *STRUCTURAL INTRODUCTION*

The structural portion of this project includes the replacement of the Waverly Place Bridge, the structural stability of the bypass culvert, and the stability of the channel improvements including concrete and sheet pile retaining wall designs. The project includes four distinct areas or sections of the rivers: Lower Mamaroneck; Upper Mamaroneck; Harbor Heights; and Sheldrake. These sections of the project will be referred to in this report and analysis. The Lower Mamaroneck is the section of the Mamaroneck River south of the Rail Road Bridge. The Upper Mamaroneck is the section of the Mamaroneck River north of the Rail Road Bridge and south of the New England Thruway (I-95) Bridge. Harbor Heights is the section of the Mamaroneck River north of the I-95 Bridge. The Sheldrake section is the portion of the Sheldrake River involved in this project. These areas are identified in the following graphic, Figure 1. See Table 1 below for the approximate station numbers of the respective reaches. Note that there are two reference lines used in this project: one for the Mamaroneck River; and one for the Sheldrake River. Also note that there are three channel design alternatives for a few portions of the project, which will be identified as "1F", "1M", and "1Z". The difference between these alternatives is changes in the channel widths and channel extents, which is represented in the civil drawings. Also, channel modifications to the Harbor Heights area is only included in alternative "1M".

Table 1: Approximate Stationing of River Sections

Channel Section	Starting Station	Ending Station	Length	Alternatives
Lower Mamaroneck	74+40	94+08	1,968 LF	1F, 1M, & 1Z
Upper Mamaroneck	46+20	74+40	2,820 LF	1F & 1M
Upper Mamaroneck	47+00	74+40	2,740 LF	1Z
Harbor Heights	2+20	16+20	1,400 LF	1M
Sheldrake River	0+00 Sheldrake	34+36 Sheldrake	3,436 LF	1F, 1M, & 1Z

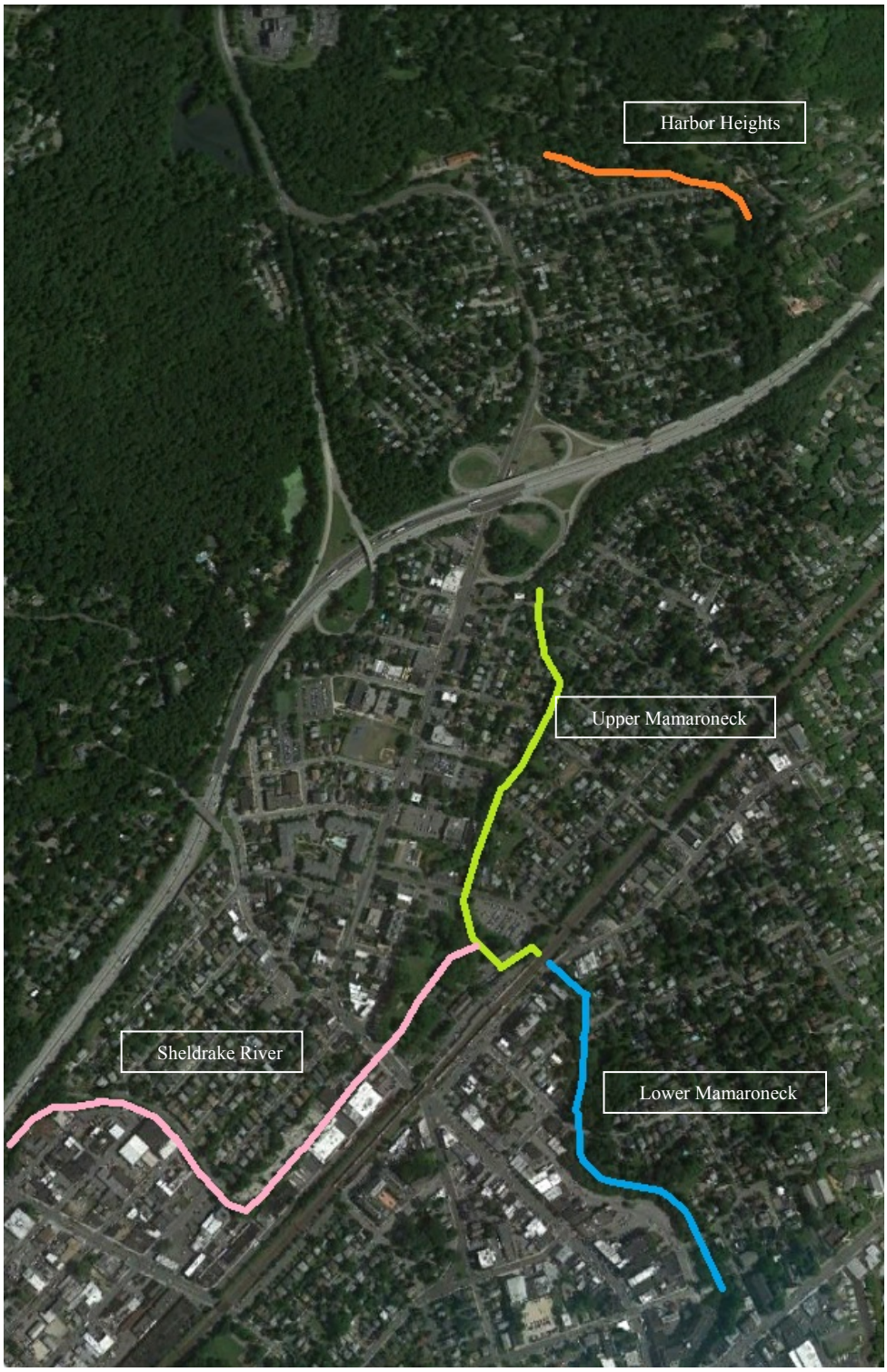
Harbor Heights
Upper Mamaroneck
Sheldrake River
Lower Mamaroneck

Figure 1: Aerial Photograph showing the project locations of the Mamaroneck and Sheldrake Rivers, Westchester County, NY

This structural appendix will include discussions of where structural measures are required and sample calculations used to create the designs for preliminary cost estimates. All subsurface geotechnical data was estimated based on existing boring logs. Taking additional borings in critical areas will improve confidence in the geotechnical estimates and lead to the most appropriate structural designs. All structural designs presented are preliminary and may change as the design moves forward. Table 2 identifies the preliminary geotechnical properties that were utilized in the structural analyses. All properties were provided by the geotechnical engineer.

Table 2: Preliminary Geotechnical Properties for Project Area

SOIL	UNIT WEIGHT (SAT) (lbs/ft^3)	UNIT WEIGHT (MOIST) (lbs/ft^3)	PHI ANGLE, φ (°)	COHESION (psf)
FILL	122	120	32	50
SW	122	120	33	0
SM	122	120	32	100
ML	120	115	31	100
GM	125	122	34	100
GP	125	122	34	0
GW	125	122	36	0
BEDROCK	165	165	45	10,000

2.0 *STRUCTURAL ANALYSIS*

2.1 Waverly Place Bridge

The Waverly Place Bridge is a roadway bridge traversing the Sheldrake River within the project limits. For this project, the Sheldrake River channel bottom will be deepened and widened, which will require this existing structure to be replaced.

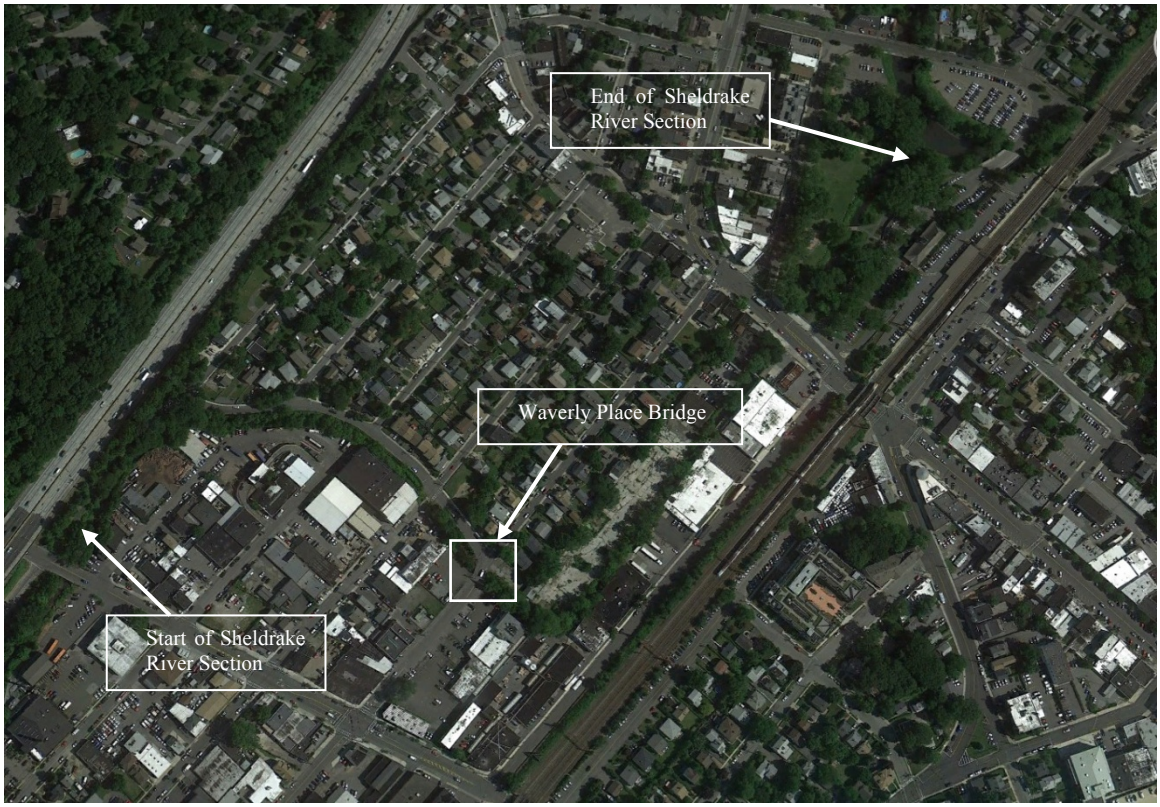

Figure 2: Waverly Place Bridge Location on the Sheldrake River

The existing Waverly Place Bridge consists of precast concrete bridge girders and a concrete deck supported on concrete abutments. The bridge shows significant wear and deterioration, including concrete spalling, cracking, and rusting, and adjacent retaining walls along the river edge demonstrate undermining. With the addition of the planned channel modifications, it is likely that this bridge structure would become unstable. Replacement is highly recommended as a life-safety issue.

Figure 3: Waverly Place Bridge Structure (4 Sept. 2014)

Figure 4: Waverly Place Bridge Structure and Adjacent Wall (4 Sept. 2014)

The span of the new bridge design was coordinated with the hydraulic model of the river to ensure the new structure does not impede flow or cause flooding. The structure of a center bridge pier will also be avoided for this reason. For the concept design of the structure, a simple span steel bridge girder with concrete deck was assumed. Geotechnical information at the bridge location was approximated based on the available boring log information previously collected. For further designs, the geotechnical information will require updating in order to determine the rock level beneath the bridge and the appropriate foundation system that should be used. The design will be further detailed and analyzed to consider alternate bridge designs. Other bridge design options include precast concrete bridge girders instead of steel bridge girders. A cost analysis may need to be completed to determine any cost savings between the alternatives, based on material prices and availability.

Figure 5 below represents the approximated geotechnical data used for the bridge design. Note that the new channel bottom elevation is within the area that is currently estimated as rock. Because the rock table location will impact the final abutment and foundation system designs, additional geotechnical investigation will be needed in the area around the Waverly Place Bridge.

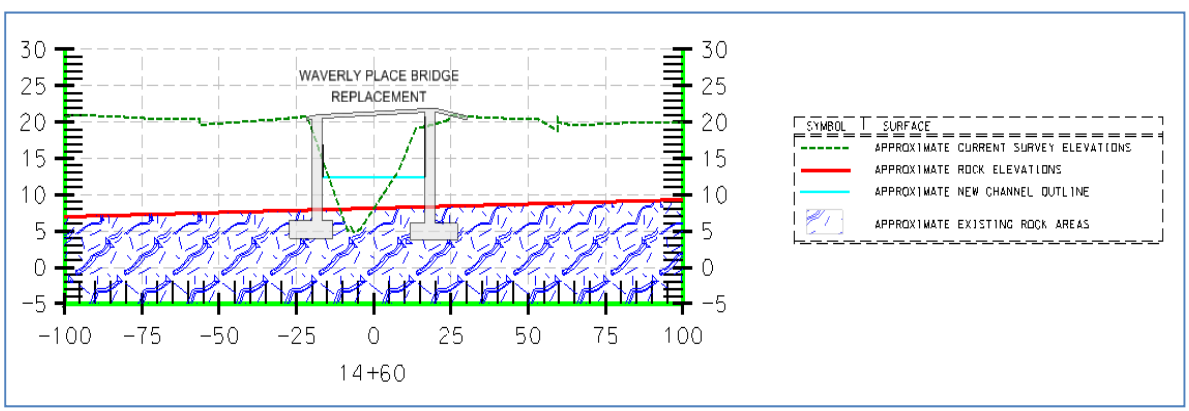

Figure 5: Waverly Place Bridge, Approximate Subsurface

The design criteria in Table 3 were assumed during the preliminary design of the Waverly Place Bridge. The bridge will support two lanes of traffic (one lane in each direction), and will include sidewalks and safety railings along both edges. See the attached calculations and concept drawings for the bridge in the calculation section.

Table 3: Waverly Place Bridge Preliminary Design Assumptions

Design Assumptions	**Alternatives 1F & 1M**
Bridge Span	38 feet
Bridge Width	34 feet
ASSHTO Bridge Loading Case	HS20-44
Bridge Design Method	LFD (Load Factor Design)
# of Steel Stringers	5 stringers
Thickness Concrete Deck Slab	8 inches
Prelim. Stringer Size	W24x76

2.2 Retaining Walls

A major aspect of this project is channelization work in the Mamaroneck and Sheldrake Rivers. Channelization involves deepening and widening channels in order to increase hydraulic capacities with the end goal of reducing overtopping and flooding. Ideally, an improved channel is designed to slope up to existing grade, but in some cases existing structures and infrastructure along the river will prevent construction of stable slopes. In these areas, retaining walls must be constructed along the channel to support the soil and surcharge loads adjacent to the rivers. The team determined which locations in the project area required retaining walls after visiting the site on 4 September 2014 and consulting Google Earth imagery.

Cantilever steel sheet pile retaining walls were preferred, but concrete retaining walls were also considered for locations where the rock table is shallow or the exposed wall height was too tall for cantilever walls. The following information, limitations, and assumptions were used for the analysis of the project retaining walls:

- Soil Profiles used were created by the civil engineer based on the existing geotechnical and boring information.

- The cross-sections for wall elevations were based on the topographical survey and channel modification surfaces created by the civil engineer in coordination with the hydraulics model.
- Surcharge loads were estimated for this phase of the project. In areas where surcharge loads were considered, they were estimated to be 300 psf due to traffic loading. Additional assessment will be required for walls adjacent to buildings.
- The soil properties presented in Table 2 were utilized in the basic analysis of the retaining walls.
- Construction, wind, and earthquake loads were not considered at this phase in the project. Additional geotechnical information will be required before finalizing designs in this manner.
- Stability of the wall will need to be determined in a future phase including seepage, settlement, and bearing capacity.
- For concrete retaining walls:
    - Overturning stability = 75% of footing base in compression
    - Sliding factor of safety = 1.5
    - Bearing capacity factor of safety = 3.0
- For cantilever steel sheet pile walls:
    - All designs were done with CWALSHT
    - Deep-seated failure was not assessed
    - Factor of safety for active soil pressure = 1.0
    - Factor of safety for passive soil pressure = 1.5
- The wall alignments will be refined in the next phase to ensure the maximization of the available real estate and channel area.
- The necessary transitions between the sloped channel sections and the retaining wall will be developed in the next phase when the retaining wall alignment is set and the retaining wall profiles are developed.

- The number of transitions between the sloped channel section and the retaining walls will be fine-tuned and minimized in the next phase to ensure logical constructability.

Figure 6 shows the proposed channel stabilization plan and distinguishes the retaining walls by size and material.

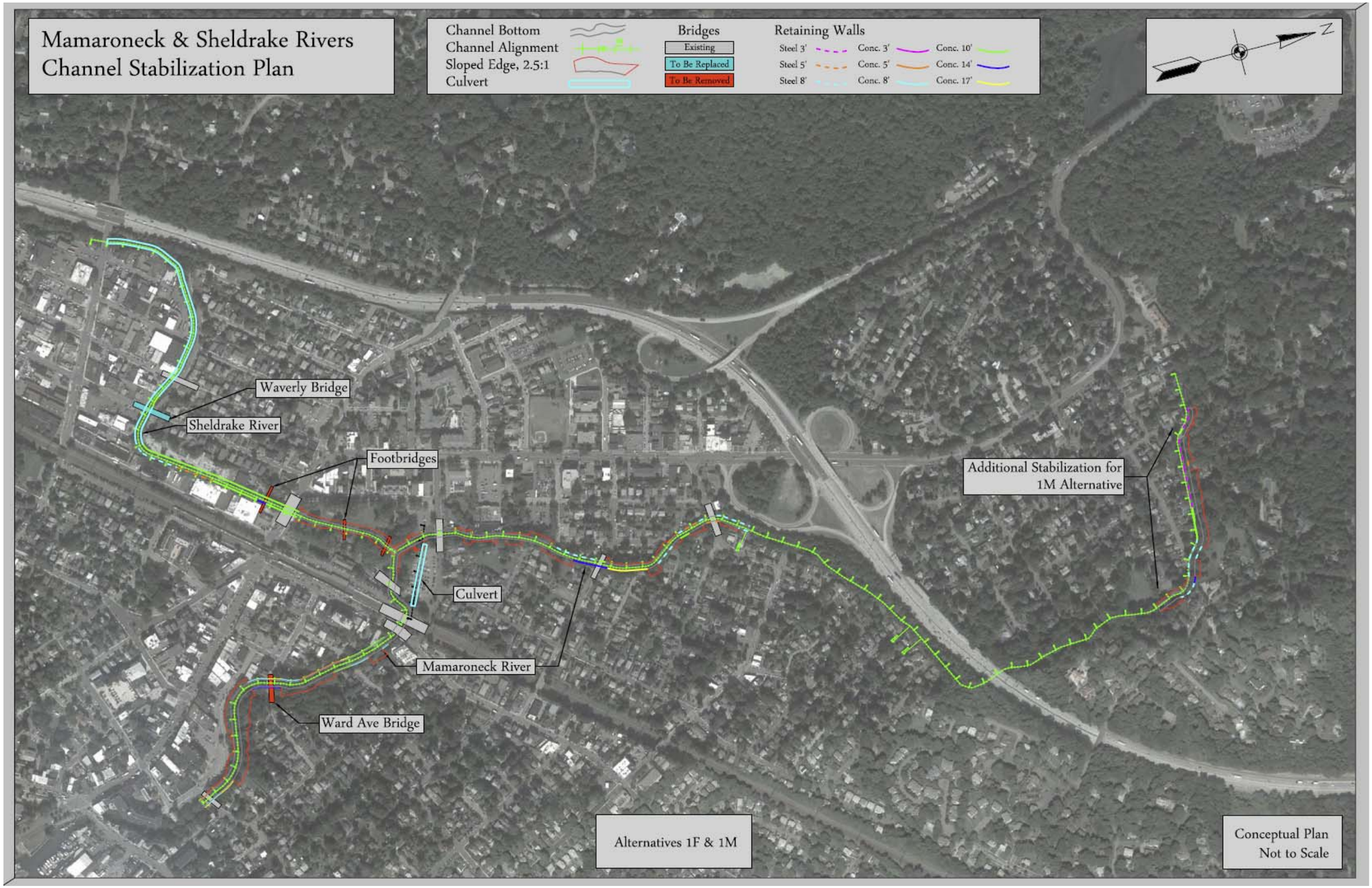

Figure 6: Channel Stabilization Plan with Approx. Locations of Walls and Slopes for Alternatives 1F & 1M

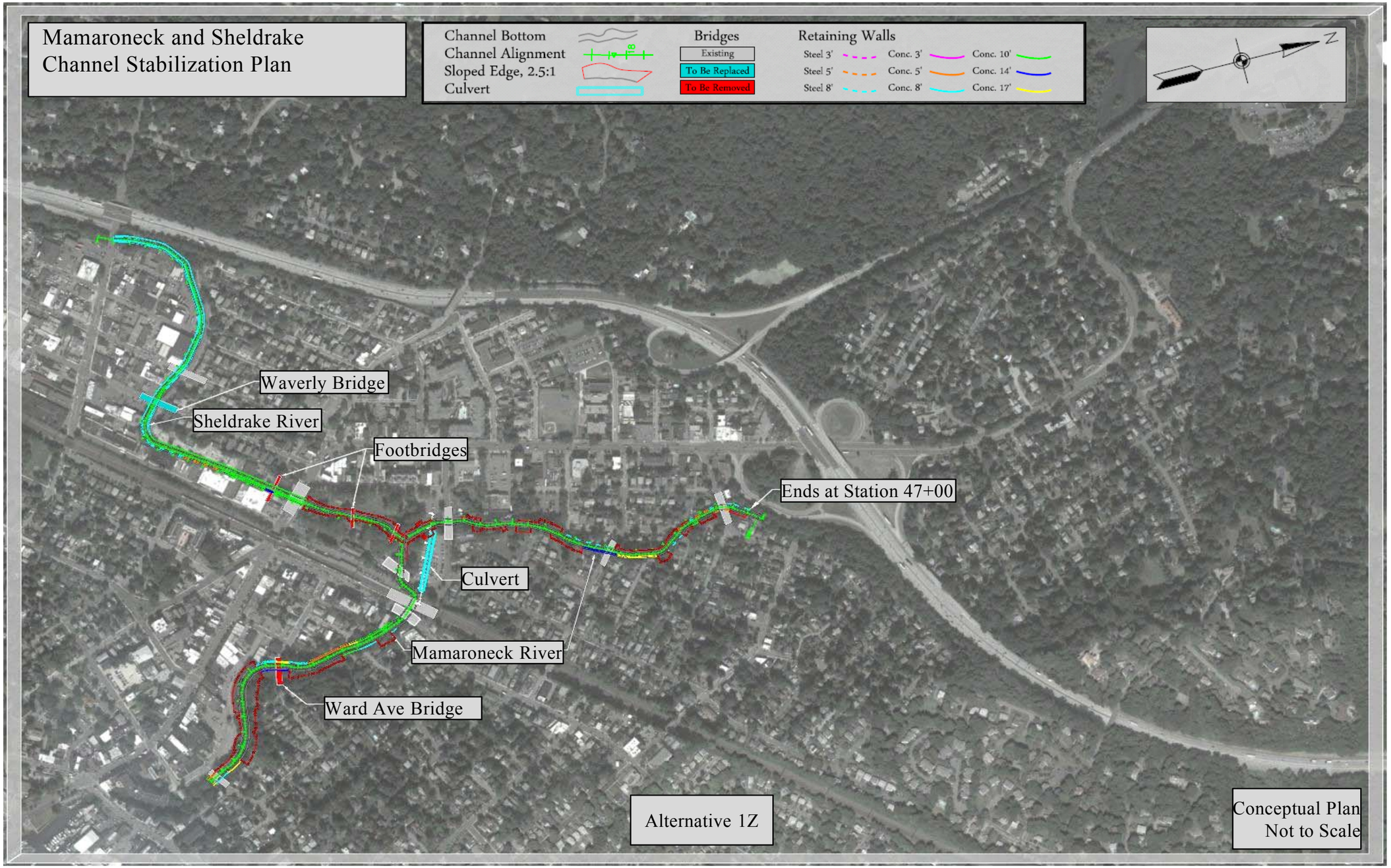

Figure 7: Channel Stabilization Plan with Approx. Locations of Walls and Slopes for Alternatives 1Z

Preliminary designs were created for concrete and sheet pile retaining walls at several heights, as shown below in Table 4 and Table 5. The retaining wall quantities for each design alternative and river section are summarized in Table 6. Refer to the subsequent discussions of each reach for additional details on existing conditions and other considerations in wall design. All of the retaining wall will require safety handrails to be affixed to the top of the walls. This is for operational public safety and will be further detailed in the next phase of the design.

Table 4: Concrete Retaining Wall Designs and Quantities

Wall Types	Exposed Height	Total Height	Wall Thickness	Footing Length	Footing Thickness
C-1	3’	6’	12”	6’	18”
C-2	5’	8’	12”	7’	18”
C-3	8’	11’	12”	10’	18”
C-4	10’	13’	18”	10’	24”
C-5	14’	17’	18”	12’	24”
C-6	17’	20’	24”	14’	24”

Table 5: Steel Sheet Pile Retaining Wall Designs and Quantities

Wall Types	Exposed Height	Min. Embedment	Min. I (in^4)	Min. s (in^3/ft)	Recommended Pile
S-1	3’	9.14’	6.01	1.43	SCZ 14, 12.25’
S-2	5’	15.0’	30.1	6.03	SCZ 14, 20.00’
S-3	8’	23.1’	263	22.0	SKZ 22, 31.25’

Table 6: Alternative 1F, Retaining Wall Quantities by Type and Reach

Wall Type	Mamaroneck (Upper)	Mamaroneck (Lower)	Harbor Heights*	Sheldrake	Total
C-1	0	0	0	0	**0**
C-2	0	355	0	0	**355**
C-3	0	515	0	3285	**3,800**
C-4	100	375	0	1490	**1,965**
C-5	220	190	0	60	**470**
C-6	250	280	0	0	**530**
S-1	0	0	0	0	**0**
S-2	255	0	0	240	**495**
S-3	855	0	0	325	**1,180**
Total	**1,680**	**1,715**	**0**	**5,400**	**8,795**

Table 7: Alternative 1M, Retaining Wall Quantities by Type and Reach

Wall Type	Mamaroneck (Upper)	Mamaroneck (Lower)	Harbor Heights	Sheldrake	Total
C-1	0	0	0	0	**0**
C-2	0	355	0	0	**355**
C-3	0	515	0	3285	**3,800**
C-4	100	375	245	1490	**2,210**
C-5	220	190	50	60	**520**
C-6	250	280	0	0	**530**
S-1	0	0	595	0	**595**
S-2	255	0	55	240	**550**
S-3	855	0	470	325	**1,650**
Total	**1,680**	**1,715**	**1,415**	**5,400**	**10,210**

Table 8: Alternative 1Z, Retaining Wall Quantities by Type and Reach

Wall Type	Mamaroneck (Upper)	Mamaroneck (Lower)	Harbor Heights	Sheldrake	Total
C-1	0	0	0	0	**0**
C-2	0	355	0	0	**355**
C-3	0	515	0	3285	**3,800**
C-4	100	375	0	1490	**1,965**
C-5	220	190	0	60	**470**
C-6	250	280	0	0	**530**
S-1	0	0	0	0	**0**
S-2	255	0	0	240	**495**
S-3	720	0	0	325	**1,045**

Total	**1,545**	**1,715**	**0**	**5,400**	**8,660**

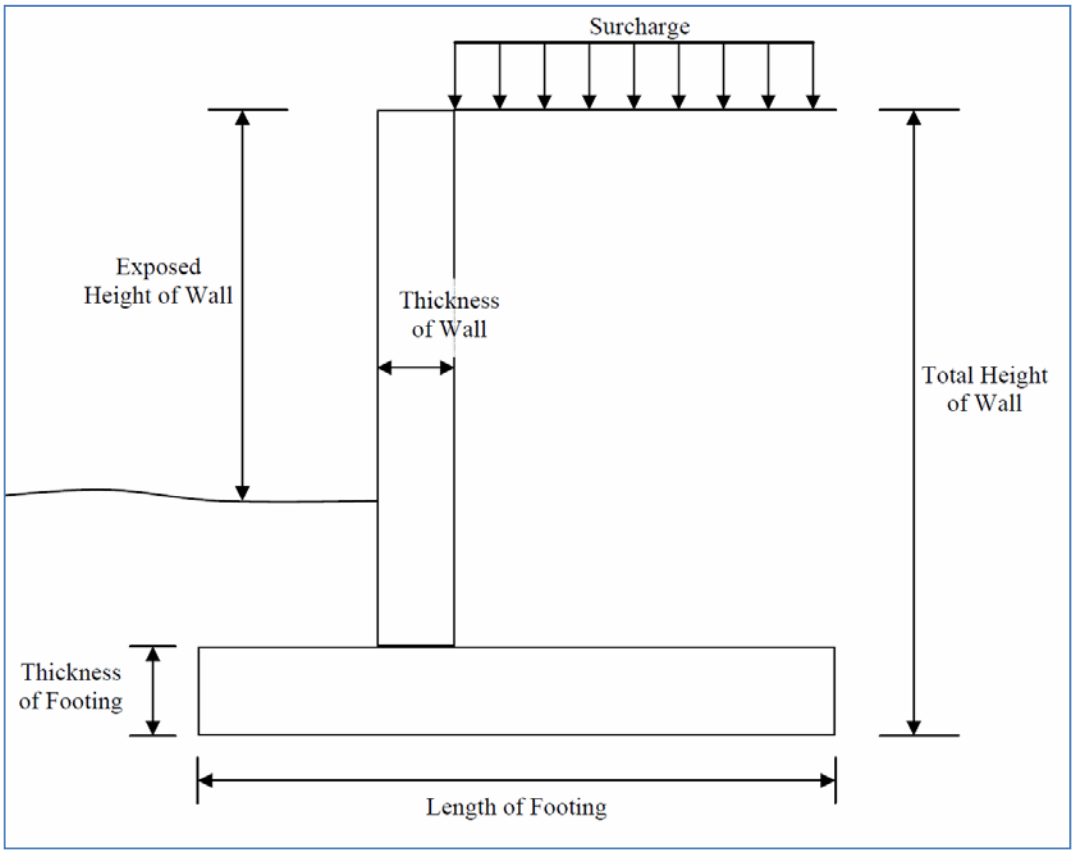

Figure 8: Preliminary Wall Section Diagram

2.2.1 Lower Mamaroneck River

The Lower Mamaroneck section is highly populated with both residential and commercial uses. In many locations, the adjacent real estate is too tight to provide the required space for stable side-slopes, and therefore retaining walls will be required. The following sample cross-sections show the channel conditions along the Lower Mamaroneck Reach. Note the shallow rock table. This soil condition precludes sheet pile walls, so all walls in this area will be concrete anchored into the bedrock.

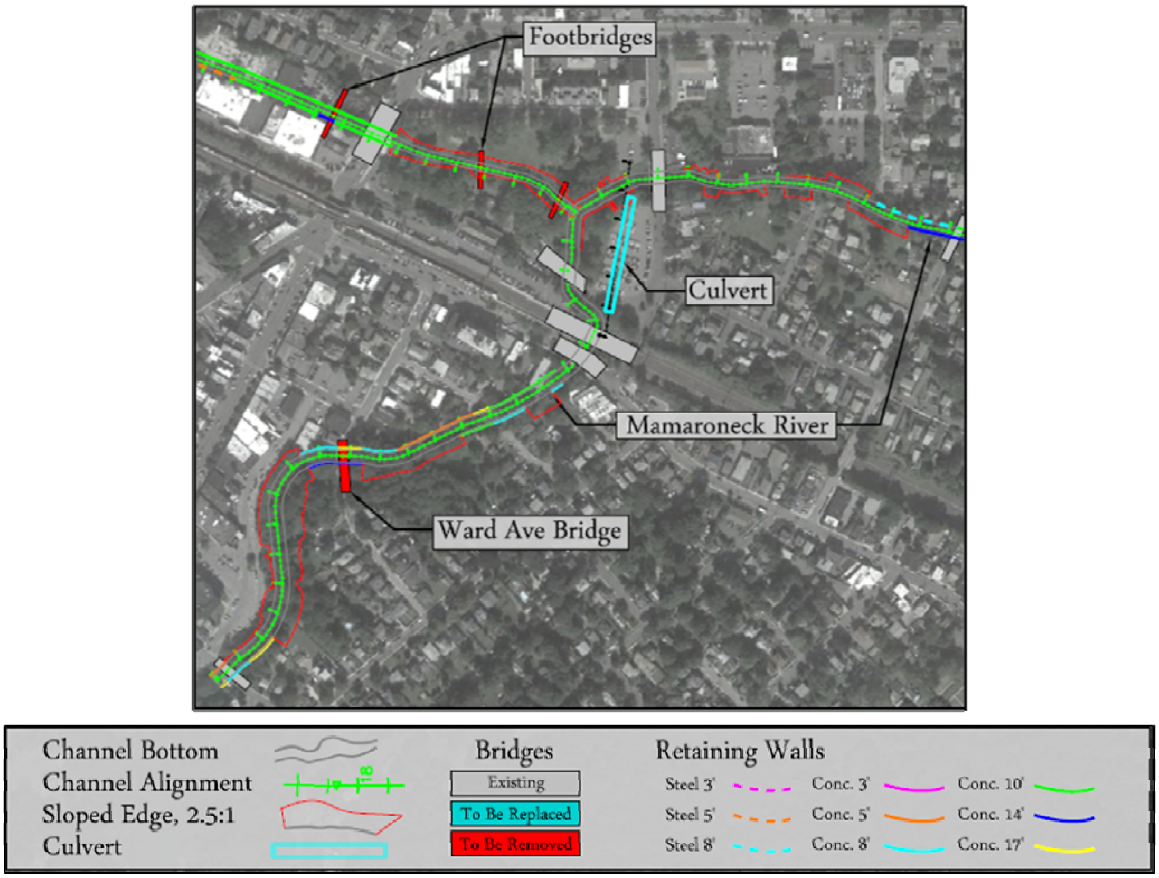

Figure 9: Lower Mamaroneck Plan View

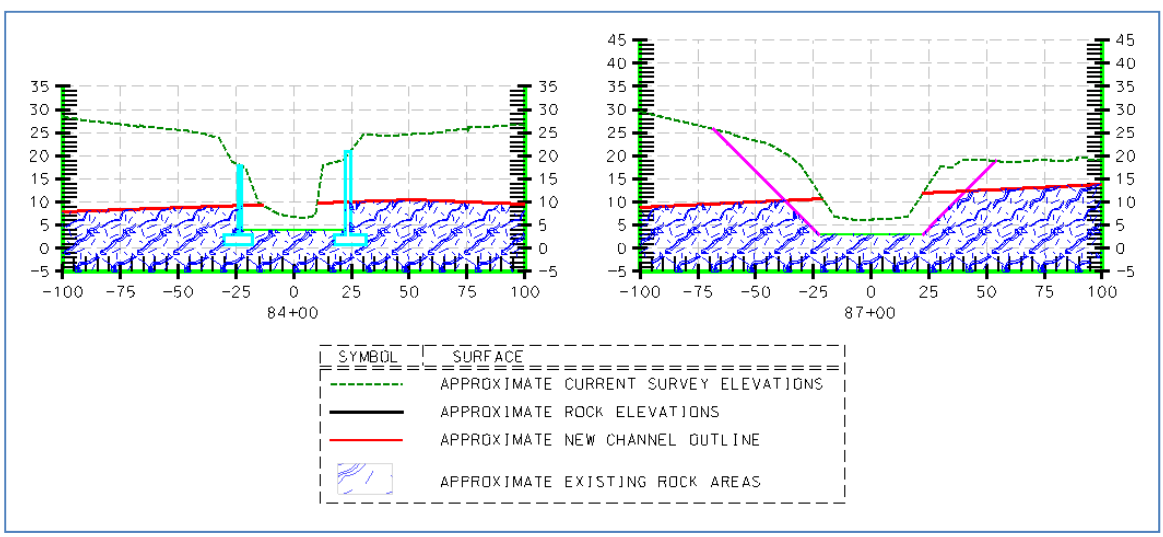

Figure 10: Lower Mamaroneck Sample Cross-Sections

Figure 11: Visible Rock Table in Lower Mamaroneck Section

Table 9: Lower Mamaroneck Retaining Walls

Wall Type	Wall Height	Wall Length on this River Section
C-2	5’	355 LF
C-3	8’	515 LF
C-4	10’	375 LF
C-5	14’	190 LF
C-6	17’	280 LF

2.2.2 Upper Mamaroneck River

The Upper Mamaroneck section is mainly bordered with residential properties, although the end portion borders an existing parking lot and train station. The Alternative 1M & 1Z channels are slightly wider than the Alternative 1F channel in this reach; for this reason, it may have longer slopes and higher walls. For comparison, Figure 13 shows cross-sections of existing conditions and proposed channelization for both plans. By and large, however, the slopes and retaining walls are similarly sized and located in each plan. For the sake of simplicity, wall placement and design was based on the 1M & 1Z channel layout. As the design progresses, the wall heights required at individual sections will need to be refined. In order to get to this level of detail, a more detailed survey of the channel and surrounding slopes will be needed. This is the case throughout the project.

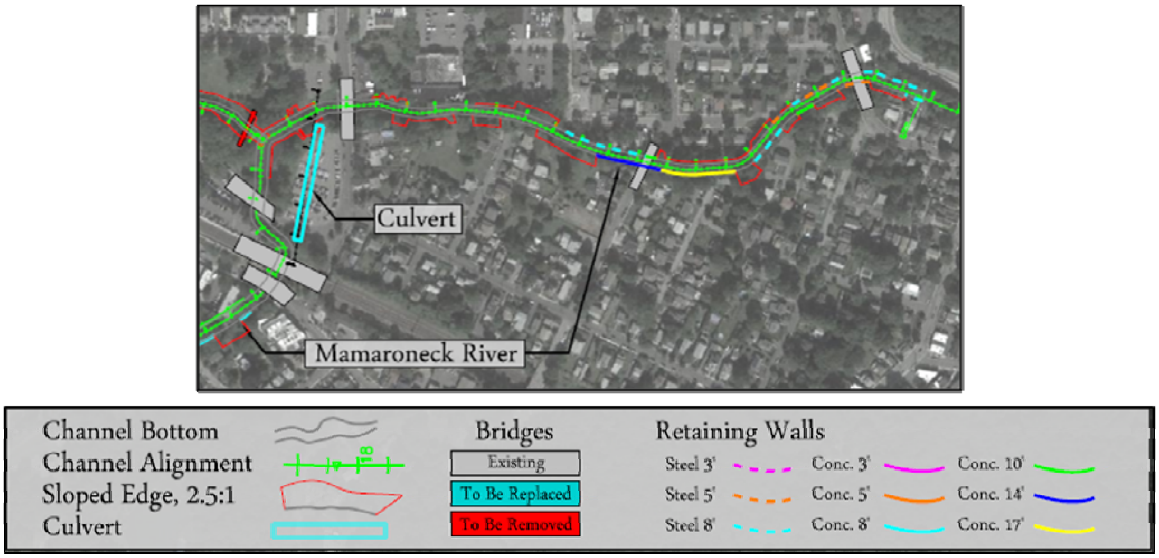

Figure 12: Upper Mamaroneck Plan View

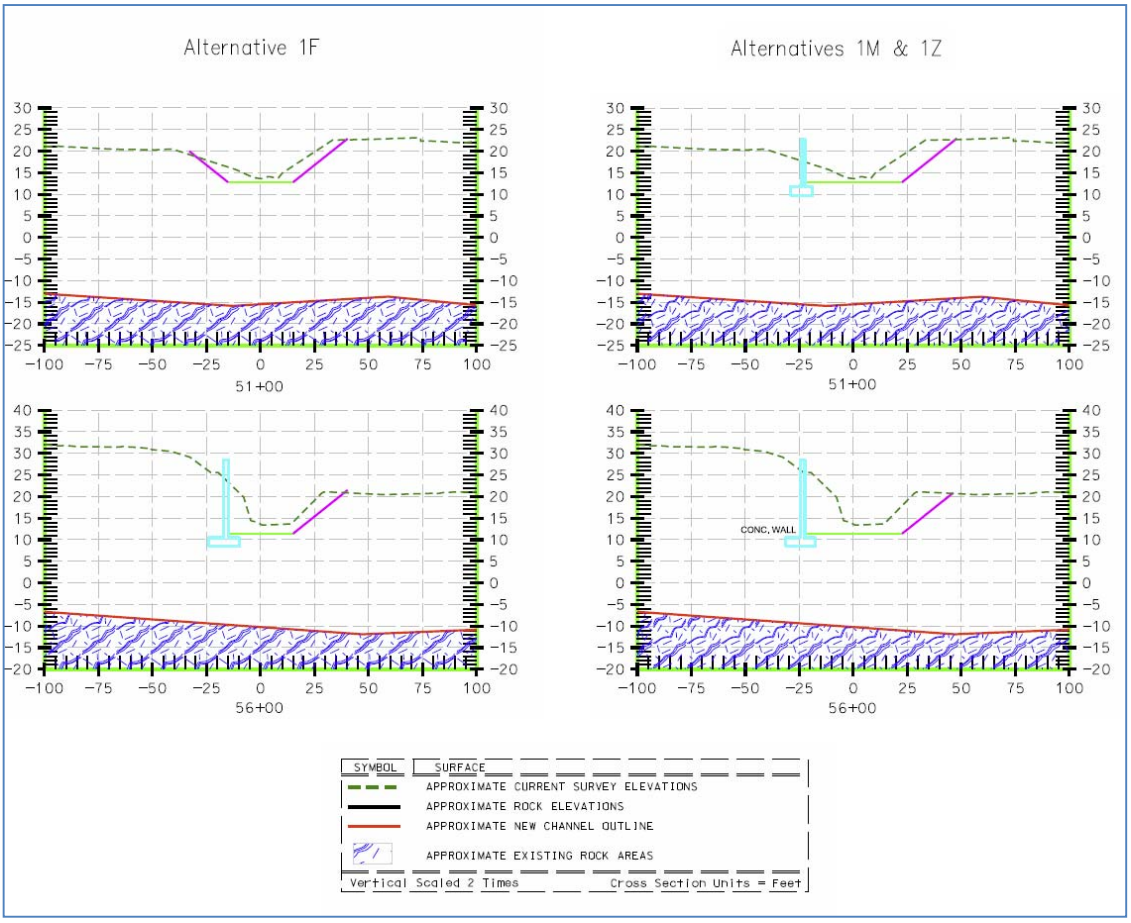

Figure 13: Upper Mamaroneck Sample Cross-Sections

Table 10: Upper Mamaroneck Retaining Walls

Wall Type	Wall Height	Wall Length on this River Section	
		1M	1Z
C-4	10	100 LF	100 LF
C-5	14’	220 LF	220 LF
C-6	17’	250 LF	250 LF
S-2	5’	255 LF	255 LF
S-3	8’	855 LF	720 LF

2.2.3 Harbor Heights – Mamaroneck River

Channel modifications in Harbor Heights are only included in the Alternative 1M Plan. This section of the Mamaroneck River is mainly bordered with residential properties. To provide the necessary channel capacity while minimizing impacts to the residential properties along the right bank of the river, the channel will be realigned and the centerline pushed towards the existing left bank.

This section can accommodate many reaches of stable side-slopes, but some areas will require structural retaining walls. In this area, no geotechnical information was available to support retaining wall and slope stability designs. More geotechnical investigations will be required in order to understand the project area conditions, and the design will change as information becomes available. At this stage, it was assumed that bedrock was sufficiently deep (i.e. greater than 23' below the proposed channel) to allow for sheet pile retaining walls. Concrete walls will be used only in areas where the exposed wall height exceeds 8', and sheet pile walls will be used in all other locations. As geotechnical information becomes available, these designs may change.

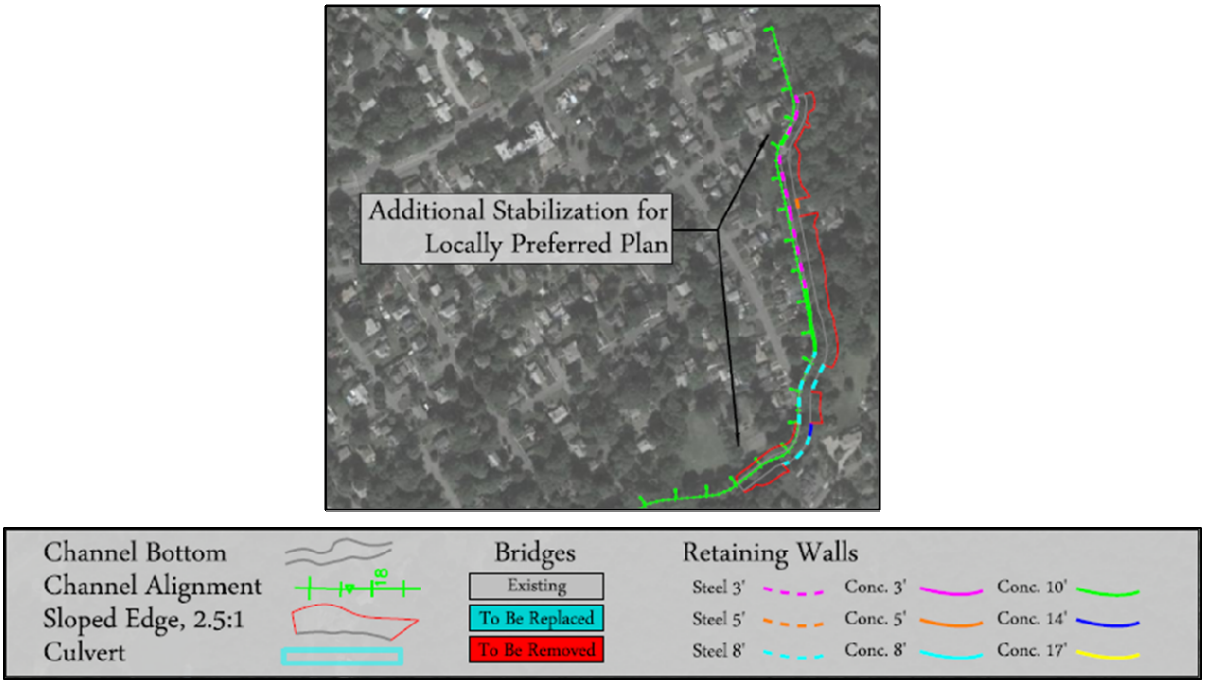

Figure 14: Harbor Heights Plan View

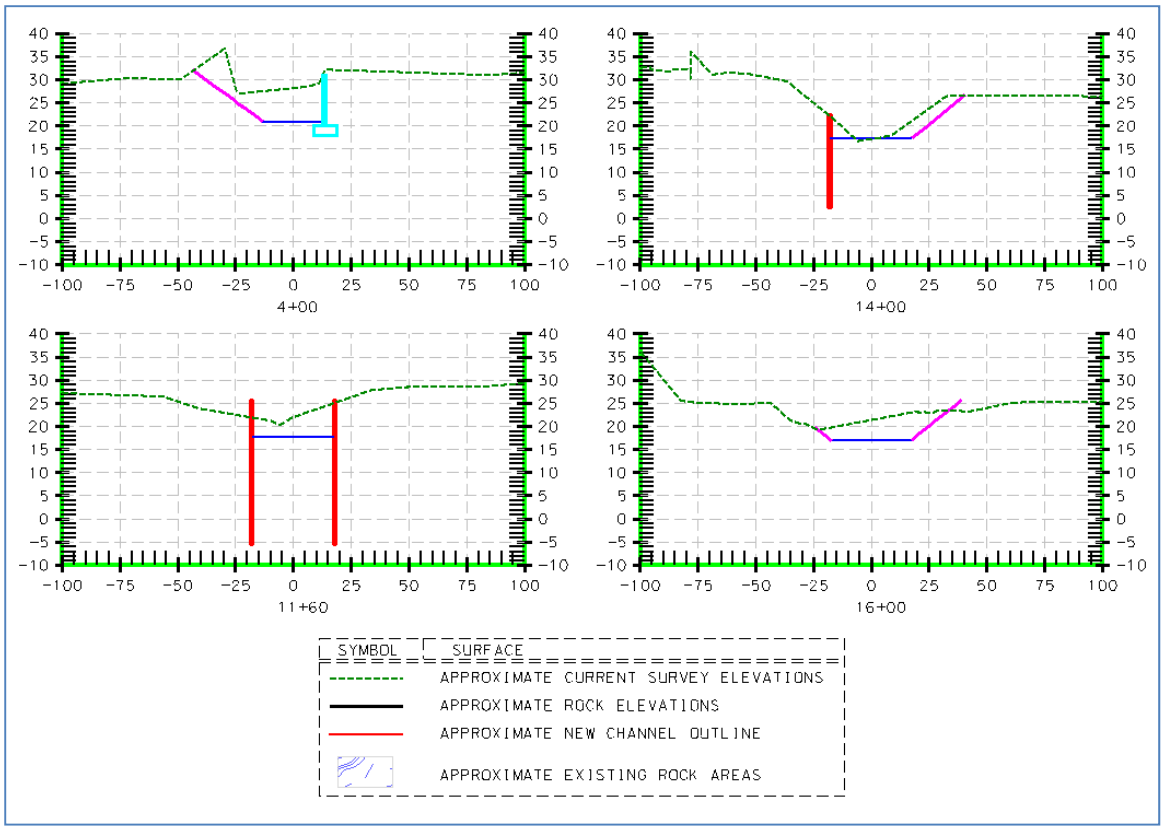

Figure 15: Harbor Heights Sample Cross-Sections

Table 11: Harbor Heights Retaining Walls

Wall Type	Wall Height	Wall Length on this River Section
C-4	10’	245 LF
C-5	14’	50 LF
S-1	3’	595 LF
S-2	5’	55 LF
S-3	8’	470 LF

2.2.4 Sheldrake River

The Sheldrake River section of the project is bordered with residential and industrial properties. The following plan view and sample cross-sections show the channel conditions along the Sheldrake River Reach. The replacement of the Waverly Place Bridge (discussed earlier) is part of this river section. New retaining walls will be constructed on both sides of the replacement bridge abutments, and the new structures will be tied together for uniformity. Many parts of this river section have industrial buildings near the river's edge. The adjacent retaining walls in these areas need to be designed for additional surcharge loads if those buildings prove to house heavy industrial functions.

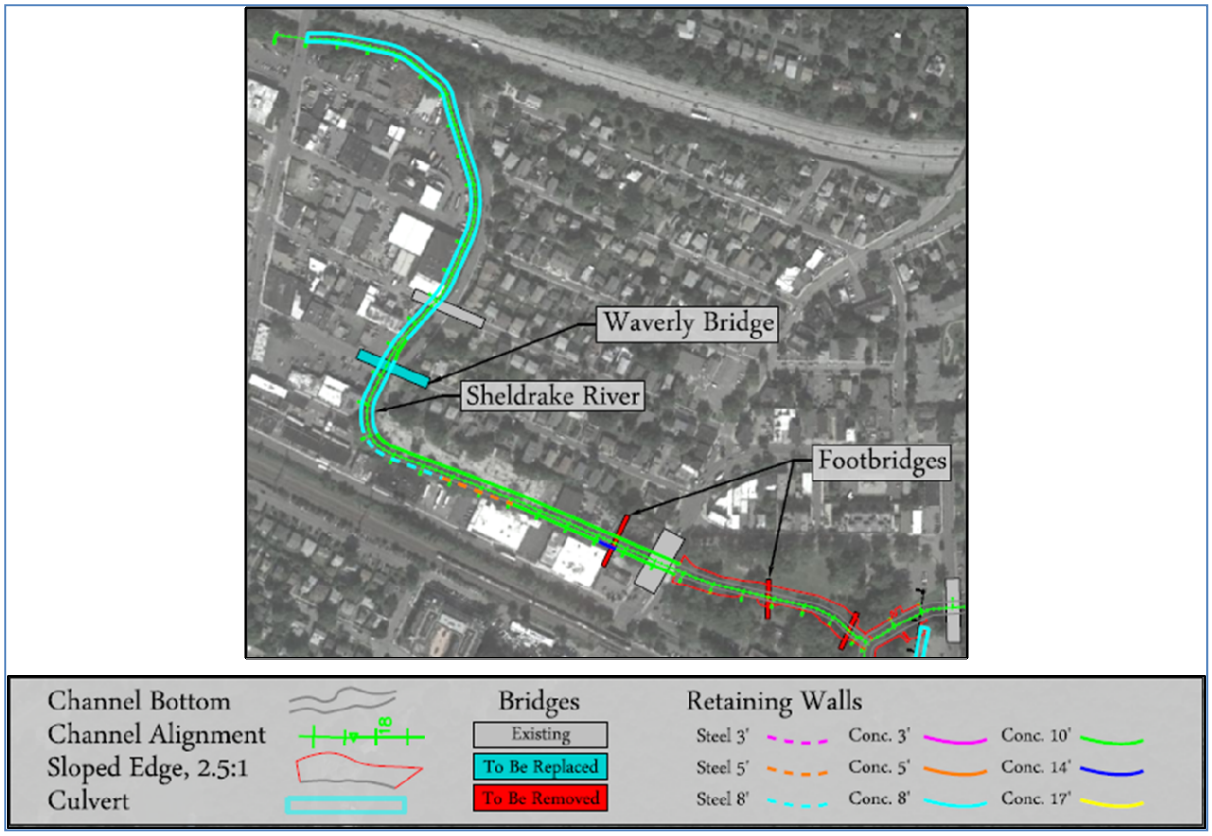

Figure 16: Sheldrake River Plan View

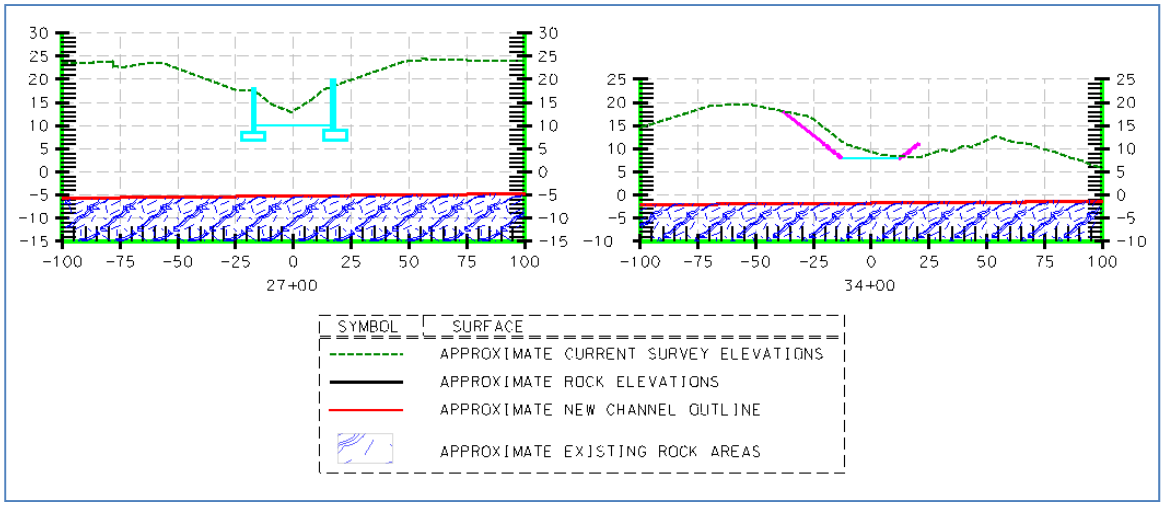

Figure 17: Sheldrake River Sample Cross-Sections

Table 12: Sheldrake River Retaining Walls

Wall Type	Wall Height	Wall Length on this River Section
C-3	8’	3285 LF
C-4	10’	1490 LF
C-5	14’	60 LF
S-2	5’	240 LF
S-3	8’	325 LF

2.3 Bypass Culvert – Upper Mamaroneck

Another major aspect of this project is the bypass culvert which will divert flow from the Upper Mamaroneck River directly into the river channel by the Railroad Station Bridge. This culvert system is being implemented in order to circumvent the river flow from the confluence between the Sheldrake and Mamaroneck Rivers, which occurs just north of the Railroad Station Bridge. This confluence is the source of much hydraulic backlog and bank overflow due to the tight meandering of the river at this location. The size of the culvert was determined from the Hydraulic modeling of the river.

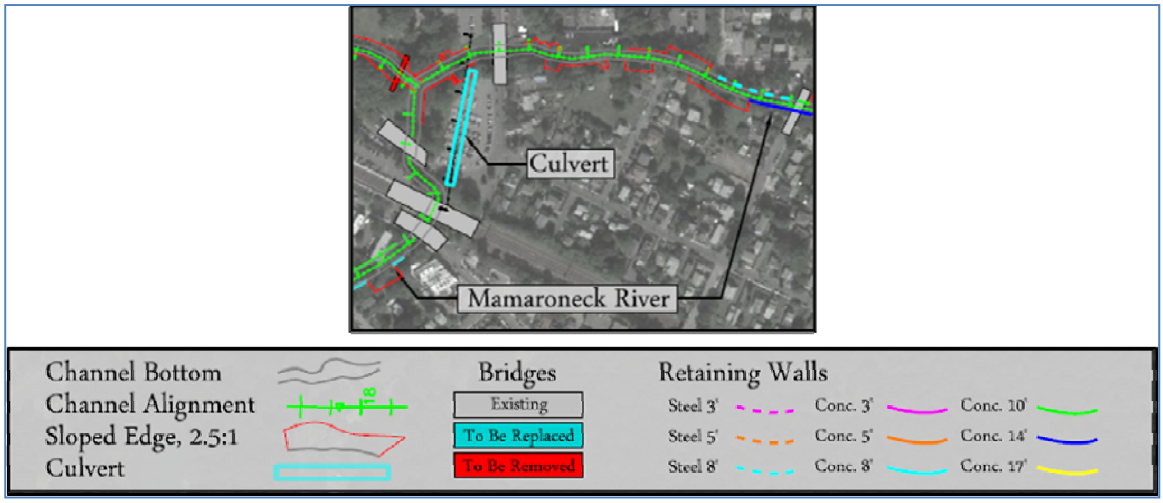

Figure 18: Bypass Culvert Plan View

The size of the culvert was determined from the Hydraulic model of the river. The culvert will be set at an elevation such that it is dry for most seasons and storm situations. The use of the culvert will only be activated once the river level increases to a specific flood elevation, which in this case is approximately 10.7 feet at the culvert inlet. The preliminary culvert design is of a precast concrete culvert. The preliminary geotechnical analysis determined that the culvert should not require a deep foundation. At the current location of the proposed culvert is an asphalt parking lot for the railroad station. To install this culvert the parking lot will be excavated, bedding material will be laid and compacted, and the culvert will be placed. The culvert will then be backfilled and the parking lot will be reconstructed on top of the culvert. The concrete culvert must be designed for this traffic loading.

Table 13: Bypass Culvert Preliminary Design Assumptions

Culvert Clear Height	8 feet
Culvert Clear Width	25 feet
Invert Elevation at Inlet	10.7 feet
Invert Elevation at Outlet	9.7 feet
Concrete Strength, f_c'	5000 psi
Fill Heights over Culvert	2.5 to 7 feet

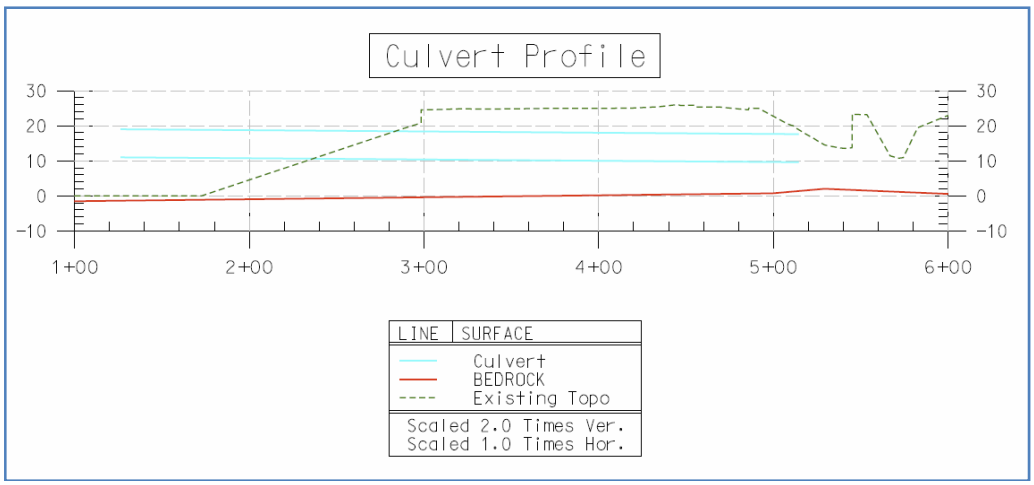

Figure 19: Bypass Culvert Profile View

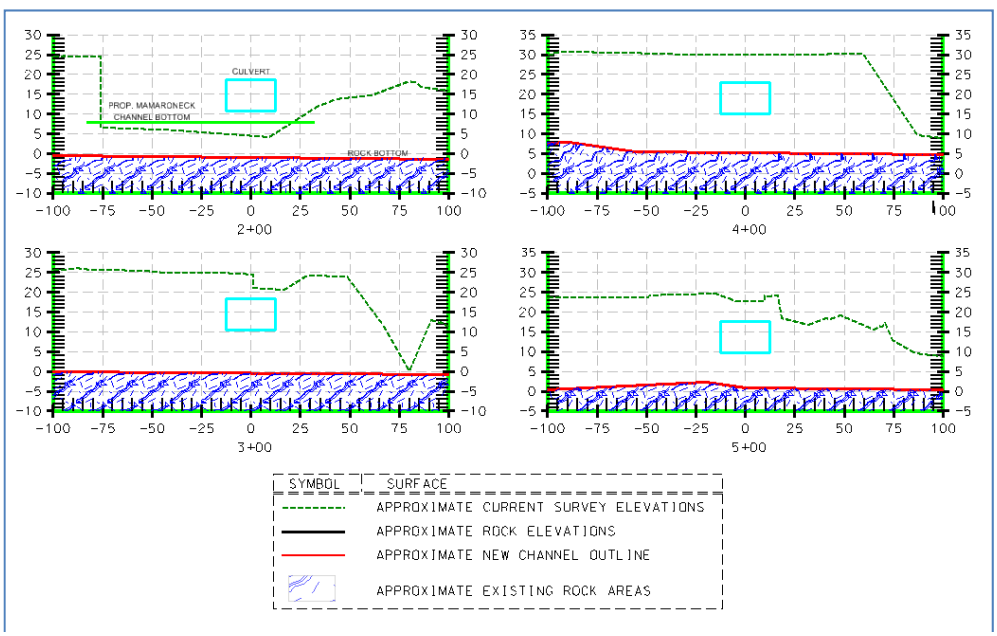

Figure 20: Bypass Culvert Sample Cross-Section

In addition to the details presented above, the culvert will also require walls at the inlet and outlet to retain the soil and traffic surcharge loads from the parking lot above. The outlet basin will need further designs and consideration in the future phases to ensure erosion prevention. Trash racks at the culvert inlet and outlet will also be required for safety reasons and to prevent debris accumulation. The trash racks would require regular maintenance, and would need to be designed for ease of use as well as hydraulic

loading. Safety railings would also need to be provided along the headwalls of the culvert's inlet and outlet for public safety.

2.4 Seismic Considerations

The seismic loading and impacts on the project structures were not evaluated at this phase. As the designs are refined and additional loading is added to the calculations, the seismic impacts and vibrations will be included. The following maps from the American Society of Civil Engineers (ASCE) 7-10, "Minimum Design Loads for Buildings and Other Structures" will be considered for the project based on the project location:

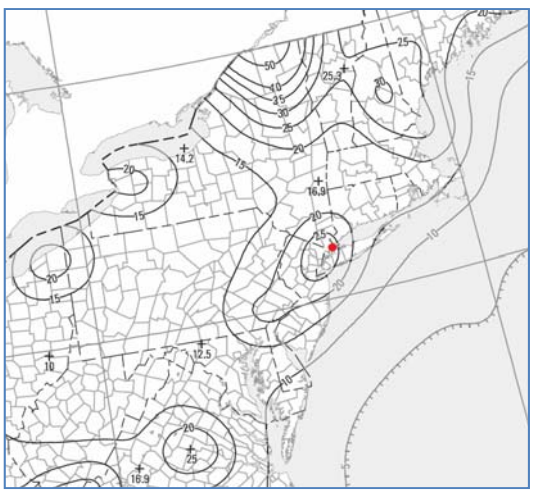

Figure 21: S_s Risk-Adjusted Maximum Considered Earthquake (MCER) Ground Motion Parameter for 0.2s Spectral Response Acceleration, Site Class B

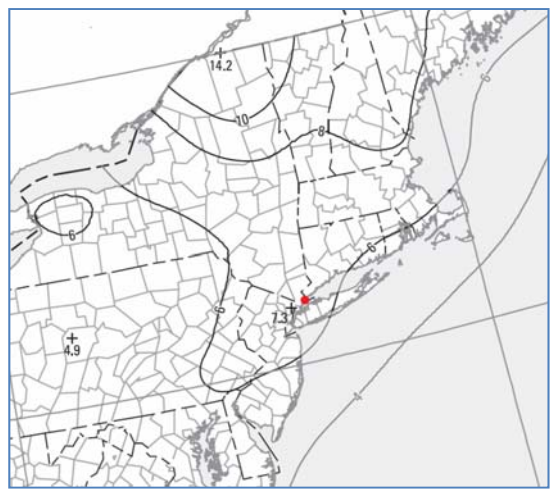

Figure 22: S_1 Risk-Adjusted Maximum Considered Earthquake (MCER) Ground Motion Parameter for 1.0 s Spectral Response Acceleration, Site Class B

The structures in this project are considered to fall within the risk category III as per the ASCE 7-10. Risk category III includes buildings and other structures with the "potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure," (ASCE 7-10, Table 1.5-1). The retaining walls, bridge, and culvert fall into this category for the significant flood that would result in a failure, as

well as the loss of roadways and buildings if river slopes collapsed. The following seismic loads will be considered for the project:

Table 14: Seismic Loads and Factors

Risk Category	III
Importance Factor, I_e	1.25
Design Spectral Acceleration Parameter, S_{DS}	0.27
Design Spectral Acceleration Parameter, S_{D1}	0.112
Site Classification	D
Seismic Design Category	B
Response Modification Coefficient, R	2
System Over-strength Factor, Ω_o	2
Deflection Amplification Factor, C_d	2

2.5 Existing Structures

There are several existing bridge structures, which cross the Mamaroneck and Sheldrake rivers within the project area. Those bridges are listed below, along with any required alterations to the bridge. As discussed in a previous section, the Waverly Place Bridge will be demolished and replaced in-kind. Other bridges have been constructed recently and do not significantly contribute to the hindering of river flow, and will therefore remain as constructed. The bridges that are marked for removal without replacement have been deemed by the local municipal authorities as non-essential.

Table 15: Existing Bridge Structures Crossing Project

Bridge	River Section	Approx. Station	Type of Opening	Project Impact
Barry Ave.	Upper Mamaroneck	48+80	Triple	None
Hillside Ave.	Upper Mamaroneck	57+00	Arch	None
Jefferson Ave.	Upper Mamaroneck	67+20	Single	None
Station Plaza	Upper Mamaroneck	72+20	Double	None
Railroad Station	Lower Mamaroneck	74+50	Arch-Double	Water Diverter
Halstead Ave.	Lower Mamaroneck	75+40	Single	None
Valley Place	Lower Mamaroneck	79+00	Arch-Double	None
Ward Ave.	Lower Mamaroneck	84+40	Arch	Removal
Tompkins Ave.	Lower Mamaroneck	93+70	Single-Single	None
Fenimore Rd.	Sheldrake	-0+30	Double	None
Pedestrian Bridge	Sheldrake	12+10	Single	Removal
Waverly Ave.	Sheldrake	14+60	Single	Replacement
Pedestrian Bridge	Sheldrake	25+40	Single	Removal
Mamaroneck Ave.	Sheldrake	27+00	Arch-Single	None
Pedestrian Bridge	Sheldrake	30+50	Single	Removal
Pedestrian Bridge	Sheldrake	33+40	Single	Removal

The Railroad Station Bridge (see Figure 23) is a double-arch stone bridge which carries the commuter trains to the adjacent railroad station. Since this bridge is a double-arch structure, the bridge has a supporting abutment in the middle of the river flow, as shown in the picture below. Due to the flat shape of the center pier, a water diverter or nosing will be added to this pier in order to channel the water flow around the pier and reduce the flow obstruction, as well as reduce the related scour.

Figure 23: Existing Railroad Station Bridge

In addition to the existing bridges along the project, there are several stretches of existing retaining walls along the project channels. These walls were built at different times by different entities, represent a variety of building materials and methods, and are all in various states of disrepair. Wall types observed in September 2014 included concrete retaining walls, loose stone gravity retaining walls, and grouted stone gravity retaining walls, (see Figure 24 and Figure 25). Some of these walls display significant deterioration such as undermining, erosion, cracking, and displaced stones: wall removal or stabilization may be required in some areas. Taking a detailed condition survey and finding additional information on the walls will be necessary to evaluate the full impact the walls may have on the project and to make any determinations regarding removal or stabilization.

Figure 24: Existing Retaining Walls, Example #1

Figure 25: Existing Retaining Walls, Example #2

3.0 *COORDINATION*

3.1 Project Coordination

The concepts presented in this appendix will be coordinated with all appropriate disciplines to ensure that the design remains feasible and logical. The retaining walls need to be reevaluated for stability with the addition of more geotechnical information and the foundation structure for the bypass culvert will need to be verified based on the site conditions. In locations where the project engineers have deemed it feasible to have side-slopes from the channel deepening, the geotechnical engineer will need to evaluate the slope stability based on the soil properties and available real estate for the slopes. If the soils cannot be stabilizes in some areas, this may result in additional retaining walls to support the channel banks. Any changes of this nature will require re-coordination with the hydraulic model, the civil engineering project layout, and the structural wall designs.

3.2 Path Ahead

The information and designs presented in this report are for conceptual purposes only. The details, assumptions, and decisions are subject to change with the addition of more project information and investigations. Further evaluation and analyses will assist in the solidification of the design plan and methods. The future phases of this project will require (but are not limited to) additional borings, field tests, an existing structure condition survey, and detailed site surveys of the channel area. The additional information will help refine the concepts, details, layouts, and cost estimate.

4.0 *CALCULATIONS*

Attached are the supporting calculations for the presented project data and designs. This calculation section includes bridge calculations, bridge plan sheets, various retaining wall designs, and project cross-sections. Calculations not provided in this section will be provided in a future design phase.

The structural elements of this project shall have designs that conform to the following list of standards and guides.

- USACE EM 1110-2-2502, Retaining and Flood Walls
- USACE EM 1110-2-2102, Water Stops and other Preformed Joint Materials for Civil Works Structures
- USACE EM 1110-2-2000, Standard Practice for Concrete for Civil Works Structures
- USACE EM 1110-2-2906, Design of Pile Foundations
- USACE EM 1110-2-2100, Stability Analysis of Concrete Structures
- ACI 318-11, Building Code Requirements for Concrete Structures
- AASHTO LRFD Bridge Design Specifications, 2012
- ASCE 7-10, Minimum Design Loads for Buildings and Other Structures

Mamaroneck & Sheldrake Rivers
Channel Stabilization Plan

Channel Bottom
Channel Alignment
Sloped Edge, 2.5:1
Culvert

Bridges
- Existing
- To Be Replaced
- To Be Removed

Retaining Walls
- Steel 3'
- Steel 5'
- Steel 8'
- Conc. 3'
- Conc. 5'
- Conc. 8'
- Conc. 10'
- Conc. 14'
- Conc. 17'

N

Alternatives 1F & 1M

Conceptual Plan
Not to Scale

Mamaroneck & Sheldrake Rivers
Channel Stabilization Plan

Channel Bottom
Channel Alignment
Sloped Edge, 2.5:1
Culvert

Bridges
- Existing
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Retaining Walls
- Steel 3'
- Steel 5'
- Steel 8'
- Conc. 3'
- Conc. 5'
- Conc. 8'
- Conc. 10'
- Conc. 14'
- Conc. 17'

N

Alternative 1Z

Conceptual Plan
Not to Scale

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★ LPP AND NED CONCEPT DESIGNS ★

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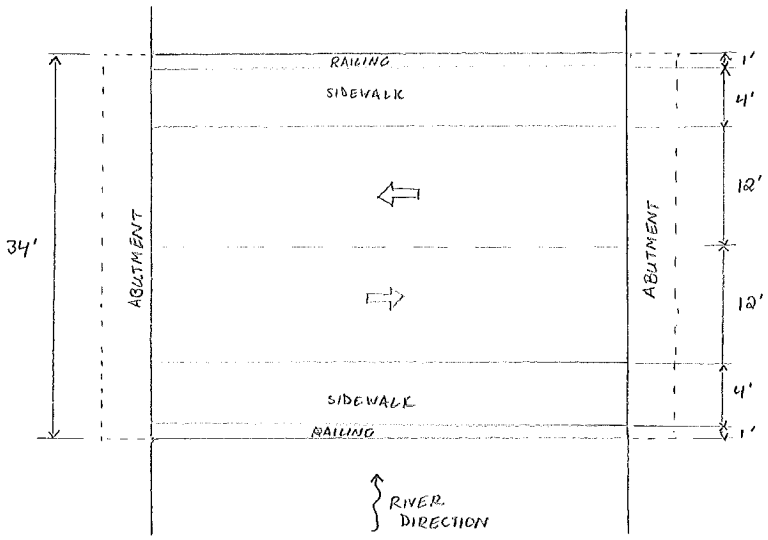
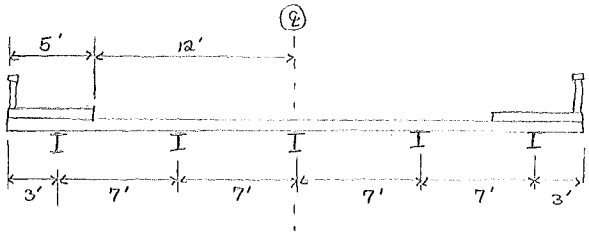
WAVERLY PLACE BRIDGE CONCEPT

- single span ; approximately 38'
- one lane of traffic in each direction
- two lane bridge
- sidewalks with railings on bridge
- frequent vehicle traffic between an industrial area and a residential area



- assumed properties:
  - $f_c' = 4.5$ ksi
  - $f_y = 60$ ksi
  - integrated wearing surface
  - 25 psf future wearing surface
- assume HS20-44 bridge loading case
- assume 8" reinforced conc slab
- assume a 34' wide bridge with the geometry below



↳ assume W36×150 stringers to start

- design of reinforced concrete deck slab (LFD Method: Load Factor Design)
  - compute effective span length, s

    slab supported on steel stringers and continuous over more than two supports

    s = distance between flanges + ½ (Top Flange Width)

    top flange width = 12" (for W36×150)

    $s = (7'-1') + \frac{1}{2}(1') = 6.5''$

  - compute moment due to dead load

    $DL = (8'')(1'/12'')(0.15\ k/ft^3) + (0.025\ k/ft^3)$

    $DL = 0.125\ k/ft$ (for 1 ft strip)

$$M_{DL} = \frac{ws^2}{10} = \frac{(0.125\ k/ft)(6.5')^2}{10} = 0.53\ k\cdot ft$$

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- compute moment due to live load + impact

$M_{LL} = 0.8\left(\frac{S+2}{32}\right)P$, P = live load, 16 kips for HS20 loading

$M_{LL} = 0.8\left(\frac{6.5+2}{32}\right)(16^k) = 3.4\ k\cdot ft$

use 30% impact magnifier factor

$M_{LL+I} = 1.3(3.4\ k\cdot ft) = 4.42\ k\cdot ft$

- compute total factored bending moment

$M_u = 1.3M_{DL} + 2.17\ M_{LL+I}$

$M_u = 1.3(0.53\ k\cdot ft) + 2.17(4.42\ k\cdot ft)$

$M_u = 10.28\ kip\cdot ft$

- compute effective depth of slab

slab depth = 8"

sacrificial wearing surface = ½"

conc cover to bottom of slab = 1"

#5 bar diameter = 0.625"

effective depth, $d = 8" - 0.5" - 1" - \frac{0.625"}{2} = 6.19"$

- compute required main reinforcement

try #5 @ 6" spacing

$A_s = (0.31\ in^2)(2) = 0.62\ in^2$ per ft

$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.62\ in^2)(60\ ksi)}{0.85(4.5\ ksi)(12")} = 0.81\ in$

$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(0.62)(60)(6.19 - \frac{0.81}{2})(1/12")$

$\phi M_n = 16.14\ k\cdot ft > M_u = 10.28\ k\cdot ft$

∴ OK

could increase bar spacing if desired

$(6")\left(\frac{\phi M_n}{M_u}\right) = 6"\left(\frac{16.14}{10.28}\right) = 9.42"$

⇒ use #5 @ 9" spacing

$A_s = (0.31)(12/9) = 0.413\ in^2$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.413)(60\ ksi)}{0.85(4.5\ ksi)(12'')} = 0.54\ in$$

$$\phi M_n = 0.9(0.413\ in^2)(60)(6.19 - 0.54/2)(1/12'')$$

$$\phi M_n = 11.0\ k\cdot ft \quad > \quad M_u = 10.28\ k\cdot ft$$

$\therefore$ ok

↳ use same reinforcement for both top and bottom of the slab in the main reinforcement direction

- compute distribution steel

$$D = \frac{220}{\sqrt{S}} = \frac{220}{\sqrt{6.5}} = 86.3\% \quad > \quad 67\% \quad \Rightarrow \quad \text{use } 67\%$$

distribution steel $= A_s(67\%) = (0.413\ in^2)(0.67) = 0.28\ in^2/ft$

$\Rightarrow$ use #5 bars @ 12" spacing ($A_s = 0.31\ in^2$)

◦ design of a composite steel-concrete stringer (LRFD Method)

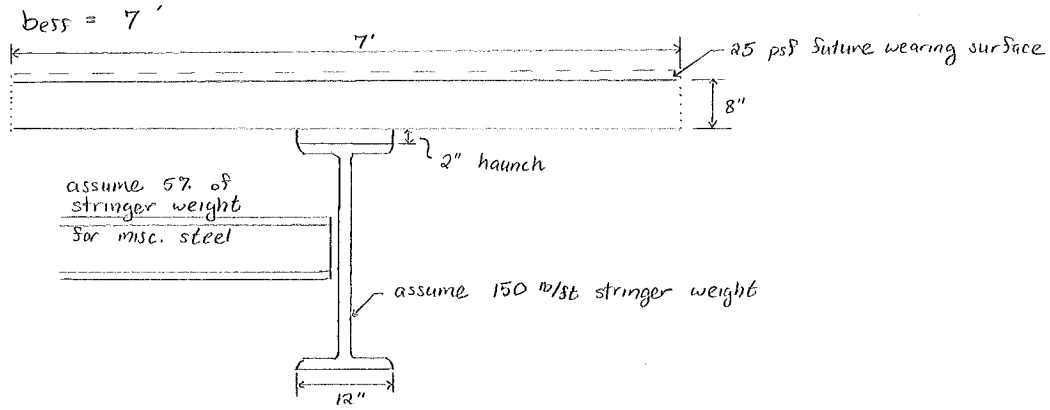
- assume: average haunch depth = 2"
  - unshored construction
  - simply supported span
  - design for a "case II" road type: ADTT = less than 2,500
    - truck loading stress cycles = 500,000
    - lane loading stress cycles = 100,000

- compute effective flange width

minimum of: 1/4 (span) = 1/4 (38') = 9.5'

center-to-center of stringers = 7'

12 (slab thickness) + 1/2 (flange width) = [12(7.5) + 1/2(9")] 1/12" = 7.88 ft



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- compute the dead load on non-composite section

$DC_{slab} = (b)(\text{slab thickness})(w_{conc}) = (7\,ft)(8''/12'')(0.15\ k/ft^3) = 0.7\ k/ft$

$DC_{haunch} = (\text{haunch width})(\text{haunch thickness})(w_{conc}) = (1\,ft)(8''/12'')(0.15\ k/ft^3) = 0.025\ k/ft$

$DC_{steel} = (\text{assumed stringer weight}) + \text{misc steel} = 0.15\ k/ft + 0.05(0.15) = 0.1575\ k/ft$

$DC_{1\,TOT} = 0.7 + 0.025 + 0.1575 = 0.8825\ k/ft$

$M_{DC_1} = \frac{w\ell^2}{8} = \frac{(0.8825)(38)^2}{8} = 159.3\ k\cdot ft$

$V_{DC_1} = \frac{w\ell}{2} = \frac{(0.8825)(38)}{2} = 16.77\ kip$

- compute the dead load on the composite section

parapet weight = steel construction = 0.2 k/ft

↳ two parapets on bridge distributed over all 5 stringers

$w = 2\ \text{parapets}\left(\frac{w_p}{\#\ \text{stringers}}\right) = 2\left(\frac{0.2}{5}\right) = 0.08\ k/ft$

$M_{DC_2} = \frac{w\ell^2}{8} = \frac{(0.08)(38)^2}{8} = 14.44\ k\cdot ft$

$V_{DC_2} = \frac{w\ell}{2} = \frac{(0.08)(38)}{2} = 1.52\ kip$

sidewalk weight = (5' wide)(0.5' high)(0.15 k/ft³) = 0.375 k/ft

↳ two sidewalks on bridge distributed over all 5 stringers

$w = 2\ \text{sidewalks}\left(\frac{w_s}{\#\ \text{stringers}}\right) = 2\left(\frac{0.375}{5}\right) = 0.15\ k/ft$

$M_{DC_3} = \frac{w\ell^2}{8} = \frac{(0.15)(38)^2}{8} = 27.1\ k\cdot ft$

$V_{DC_3} = \frac{w\ell}{2} = \frac{(0.15)(38)}{2} = 2.85\ kip$

- compute wearing surface on composite section

$DW = \frac{(\text{width of roadway})(\text{future wearing surface})}{\text{number of stringers}} = \frac{(24\,ft)(0.025\ k/ft^2)}{5\ \text{stringers}} = 0.12\ k/ft$

$M_{DW} = \frac{w\ell^2}{8} = \frac{(0.12)(38)^2}{8} = 21.66\ k\cdot ft$

$V_{DW} = \frac{w\ell}{2} = \frac{(0.12)(38)}{2} = 2.28\ kip$

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- compute live load distribution factor

assume a stringer size = W24×76

$A = 22.4\ in^2,\quad I = 2100\ in^4,\quad d = 23.92\ in^2$

distance between c.g. of stringer and c.g. of deck:

$$e_g = \frac{7.5}{2} + 2'' + \frac{23.92}{2} = 17.71''$$

for 4.5 ksi concrete, $n = 8$

$$K_g = n\left(I + Ae_g^2\right) = 8\left(2100 + (22.4)(17.71)^2\right) = 73{,}005\ in^4$$

live load distribution factor for moment (strength limit state) $\boxed{\text{for bending}}$

$$DF_m = 0.075 + \left[\frac{S}{9.5}\right]^{0.6}\left[\frac{S}{L}\right]^{0.2}\left[\frac{K_g}{12\,L\,t_s^3}\right]^{0.1}$$

$$DF_m = 0.075 + \left[\frac{7.0}{9.5}\right]^{0.6}\left[\frac{7.0}{38}\right]^{0.2}\left[\frac{73{,}005}{12(38)(7.5)^3}\right]^{0.1} = 0.539$$

live load distribution factor for shear (Strength Limit State)

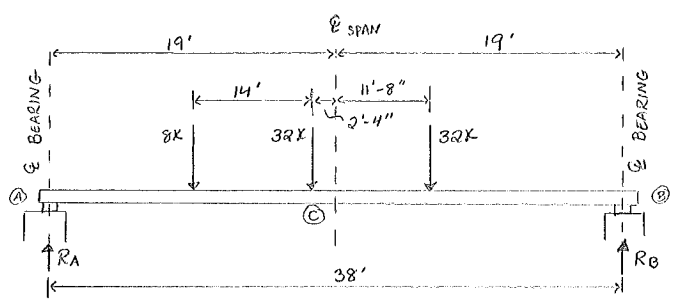
$$DF_s = 0.2 + \frac{S}{12} - \left[\frac{S}{35}\right]^{2.0}$$

$$DF_s = 0.2 + \frac{7.0}{12} - \left(\frac{7}{35}\right)^2 = 0.743$$

- compute live load moment and shear:

for short span bridges both HL-93 truck and tandem loads should be checked to determine the worst case

for truck load:



$$\Sigma M_A = 0:\quad 8^K(2.67') + 32(16.67') + 32(30.67') - R_B(38') = 0$$

$$R_B = 40.43\ kips$$

$$\Sigma F_y = 0:\quad 8 + 32 + 32 - 40.43 - R_A = 0 \quad \Rightarrow \quad R_A = 31.57\ kips$$

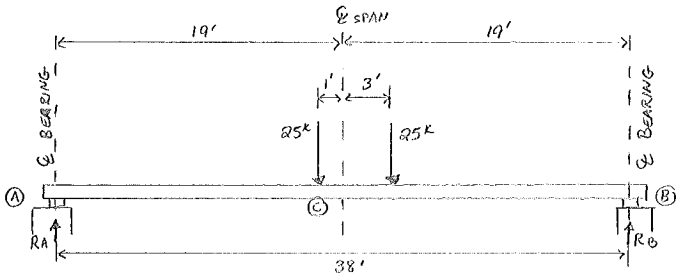
maximum live load moment:

$$M_{LL} = M_{max} = R_A(16.67) - 8^K(14') = 414.17\ k\cdot ft$$

maximum shear: (occurs when 32k axle is at the support

$$V_{LL} = 32 + \frac{32(24) + 8(10)}{38} = 54.32 \text{ kips}$$

for tandem load:



$\Sigma M_A = 0$: $25k(18') + 25k(22') - R_B(38') = 0 \Rightarrow R_B = 26.32\ k$

$\Sigma F_y = 0$: $25k + 25k - 26.32 - R_A = 0 \Rightarrow R_A = 23.68\ k$

$M_{LL} = R_A(18') = 23.68(18') = 426.24\ k\cdot ft$

$V_{LL} = 25 + \frac{25(34')}{38} = 47.37\ k$

$\Rightarrow$ controlling loads: $M_{LL} = 426.24\ k\cdot ft$, $V_{LL} = 54.32$ kips

for design lane load:

$$M_L = \frac{w\ell^2}{8} = \frac{(0.64\ k/ft)(38')^2}{8} = 115.52\ k\cdot ft$$

$$V_L = \frac{w\ell}{2} = \frac{(0.64\ k/ft)(38)}{2} = 12.16\ k\text{ip}$$

dynamic allowance, $IM = 0.33$ (applies only to truck or tandem loads)

live and impact applied to interior stringers:

$$M_{LL+I} = (0.539)[(1.33)(426.24) + 115.52] = 367.824\ k\cdot ft$$

$$V_{LL+I} = (0.743)[(1.33)(54.32) + 12.16] = 62.71 \text{ kips}$$

- compute factored moment and shear:
  - Strength I: $1.25DC + 1.50DW + 1.75(LL + IM)$

$$M_u = 1.25(159.3 + 14.44 + 27.1) + 1.50(21.66) + 1.75(367.824)$$

$$M_u = 927.23\ k\cdot ft$$

$$V_u = 1.25(16.77 + 1.52 + 2.85) + 1.50(2.28) + 1.75(62.71)$$

$$V_u = 139.6 \text{ kips}$$

- Strength III: 1.50(DC + DW)

$M_u = 1.50(159.3 + 14.44 + 27.1 + 21.66) = 333.75$ k·ft

$V_u = 1.50(16.77 + 1.52 + 2.85 + 2.28) = 35.13$ k

⇒ case I controls: $M_u = 927.23$ k·ft , $V_u = 139.6$ kips

- check prelim stringer section: W24x76 ($F_y = 50$ ksi)
  - calculate compression block height and plastic moment capacity for the composite section

$$a = \frac{A F_y}{0.85 f_c' b_{eff}} = \frac{(22.4 \text{ in}^2)(50 \text{ ksi})}{0.85(4.5 \text{ ksi})(7'(12''/1'))} = 3.5''$$

$$M_p = AF_y\left(d - \frac{a}{2}\right) = (22.4 \text{ in}^2)(50 \text{ ksi})\left[\frac{23.92}{2} + 2'' + 7.5'' - \frac{3.5''}{2}\right]\left(\frac{1'}{12''}\right)$$

$M_p = 1839.6$ k·ft

web, $D/t_w = \frac{21}{0.44} = 47.7 \le 150$ ⇒ section is compact

$D_p = a = 3.5''$

$D_t = 7.5'' + 2'' + 23.92 = 33.42''$

$D_p > 0.1 D_t$

$$M_n = M_p\left[1.07 - 0.7\left(\frac{D_p}{D_t}\right)\right] = 1839.6\left[1.07 - 0.7\left(\frac{3.5}{33.42}\right)\right] = 1833.5 \text{ k·ft}$$

$M_u + \frac{1}{3} f_\ell S_{xt}$ (where $f_\ell = 0$) ⇒ $M_u = 927.23$ k·ft $\le \phi M_n = 1833.5$ k·ft

∴ OK

ductility requirements, $D_p/D_t = 3.5/33.42 = 0.105 \le 0.42$ ∴ OK

- check shear capacity

$K = 5$ for unstiffened stringer

$$1.12\sqrt{\frac{KE}{F_y}} = 1.12\sqrt{\frac{5(29{,}000)}{50}} = 60.3 > D/t_w = 23.92/0.44 = 54.4$$

⇒ ∴ $C = 1.0$

$V_n = CV_p = C(0.58 F_y D t_w) = 1.0(0.58)(50 \text{ ksi})(23.92'')(0.44'') = 305.22$ k

$\phi_v V_n = 1.0(305.22) = 305.22$ kips $\ge V_u = 139.6$ kips

∴ OK

∴ prelim stringer section meets strength requirements

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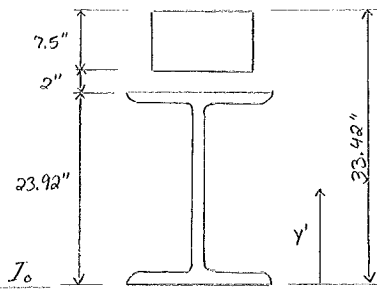
- compute composite section moment of inertia:
  - for superimposed dead loads



$$b_f = \frac{b_{eff}}{3n} = \frac{7'(12'')}{3(8)} = 3.5''$$

$$I_{deck} = \frac{(3.5'')(7.5'')^3}{12} = 123.05 \text{ in}^4$$

element	A (in²)	Y (in)	AY	AY²	I_o
W24×76	22.4	11.96	267.9	3204.1	2100
slab	26.25	29.67	778.84	23108.1	123.05
TOTALS	48.65		1046.74	26312.2	2223.05

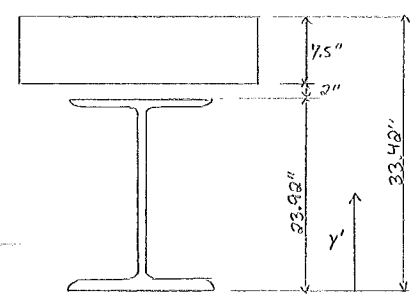
$$I_z = \Sigma I_o + \Sigma AY^2 = 2223.05 + 26312.2 = 28{,}535 \text{ in}^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{1046.74}{48.65} = 21.52 \text{ in}$$

$$I = I_z - (\Sigma A)(Y')^2 = 28{,}535 - (48.65)(21.52)^2 = 6004.68 \text{ in}^4$$

$$S = \frac{I}{Y'} = \frac{6004.68}{21.52} = 279.03 \text{ in}^3$$

  - for live loads



$$b_f = \frac{b_{eff}}{n} = \frac{7'(12'')}{8} = 10.5''$$

$$I_{deck} = \frac{(10.5'')(7.5'')^3}{12} = 369.14 \text{ in}^4$$

element	A (in²)	Y (in)	AY	AY²	I_o
W24×76	22.4	11.96	267.9	3204.1	2100
slab	78.75	29.67	2336.5	69324.3	369.14
TOTALS	101.15		2604.4	72528.4	2469.14

$$I_z = \Sigma I_o + \Sigma AY^2 = 2469.14 + 72528.4 = 74997.54 \text{ in}^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{2604.4}{101.15} = 25.75''$$

$$I = I_z - (\Sigma A)(Y')^2 = 74997.54 - (101.15)(25.75)^2 = 7928.77 \text{ in}^4$$

$$S = \frac{I}{Y'} = \frac{7928.77}{25.75} = 307.9 \text{ in}^3$$

- check service limit state

$$f_{DL} = \frac{M_{DC1}}{S} = \frac{159.3 \text{ k·ft } (12''/\text{ft})}{176 \text{ in}^3} = 10.86 \text{ ksi}$$

$$f_{SDL} = \frac{M_{DC2} + M_{DC3} + M_{DW}}{S} = \frac{(14.44 + 27.1 + 21.66)(12\ in/ft)}{279.03\ in^3} = 2.72\ ksi$$

$$f_{LL+I} = \frac{M_{LL+I}}{S} = \frac{367.824\ (12\ in/ft)}{307.9\ in^3} = 14.34\ ksi$$

service II load combination:

$$f_{bot} = 1.0(10.86 + 2.72) + 1.3(14.34) = 32.22\ ksi$$

$$max\ f \le 0.95 f_y = 0.95\ (50\ ksi) = 47.5\ ksi \quad \therefore \underline{OK} \checkmark$$

- compute shear range in the stringer

(for fatigue limit state, only one truck is loaded)

live load distribution factor for shear:

$$DF = \frac{1}{1.2}\left[0.36 + \frac{S}{25}\right] = \frac{1}{1.2}\left[0.36 + \frac{7}{25}\right] = 0.533$$

live load impact for fatigue: $IM = 0.15$

live load factor for fatigue limit state: $LF = 0.75$

$x=0$, positive shear

$$+V_{x=0} = (LF)(DF)(1+IM)\left[32 + 32\left(\frac{24}{38}\right) + 8\left(\frac{10}{38}\right)\right]$$

$$= 0.75\ (0.533)(1+0.15)\ [54.316] = 24.97\ kips$$

$x=0$, negative shear

$$-V_{x=0} = 0$$

$x=0$, total shear range

$$V_{x=0} = +V_{x=0} - (-V_{x=0}) = 24.97 - 0 = 24.97\ kips$$

$x=4.5$, positive shear

$$+V_{x=4.5} = 0.75\ (0.533)(1.15)\left[32\left(\frac{33.5}{38}\right) + 32\left(\frac{19.5}{38}\right) + 8\left(\frac{5.5}{38}\right)\right] = 21.05\ kip$$

$x=4.5$, negative shear

$$-V_{x=4.5} = LF\ (DF)(1+IM)\left[8\left(\frac{-4.5}{38}\right)\right] = 0.75(0.533)(1.15)(-0.95) = -0.44\ kip$$

$x=4.5$, total shear range

$$V_{x=4.5} = 21.05\ k - (-0.44) = 21.49\ kips$$

$x=9.0$, positive shear

$$+V_{x=9.0} = 0.75\ (0.533)(1.15)\left[32\left(\frac{29}{38}\right) + 32\left(\frac{15}{38}\right) + 8\left(\frac{1}{38}\right)\right] = 17.13\ kip$$

$x=9.0$, negative shear

$$-V_{x=9.0} = 0.75(0.533)(1.15)\left[32\left(\frac{-9}{38}\right) + 8\left(\frac{15}{38}\right)\right] = -2.03\ kip$$

$x = 9.0$, total shear range

$V_{x=9.0} = 17.13\,x - (-2.03\,x) = 19.16$ kips

$x = 19.0$, positive shear

$+V_{x=19} = (0.75)(0.533)(1.15)\left[32\left(\frac{19}{38}\right) + 32\left(\frac{5}{38}\right)\right] = 9.29\,x$

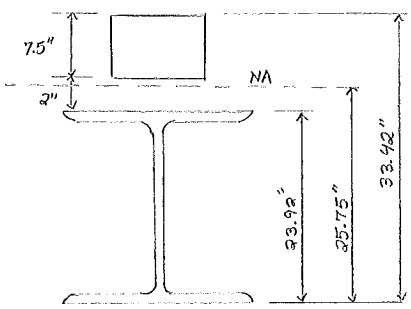
$x = 19.0$, negative shear

$-V_{x=19} = (0.75)(0.533)(1.15)\left[32\left(\frac{-5}{38}\right) + 32\left(\frac{-19}{38}\right) + 8\left(\frac{5}{38}\right)\right] = -8.81\,x$

$x = 19.0$, total shear range

$V_{x=19} = 9.29 - (-8.81) = 18.1$ kips

- shear studs based on fatigue criteria

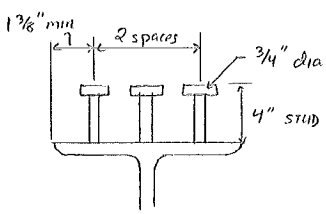


$I_x = 7928.77\ in^4$

Q = (Area of Steel)(distance NA to Stringer CG)

$Q = (22.4\ in^2)(25.75 - 11.96) = 308.9\ in^3$

$S_r = \dfrac{V_r Q}{I}$ and $P = \dfrac{(\#\ \text{studs})\ Z_r}{S_r}$



class II road, ADTT of 2500 (approx)

$s > 4d = 4(0.75) = 3''$

$b_f = 8.99''$ (flange width)

$s = \dfrac{b_f - (2 \times \text{edge distance})}{2\ \text{spaces}} = \dfrac{8.99 - 2(1.375'')}{2}$

$s = 3.12'' > 3''$ ∴ ok ✓

number of truck loading cycles (assuming 75 years):

$N = 365\,(75)\,n\,p\,(ADTT) = 365\,(75)(1.0)(1.0)(3000) = 82.1 \times 10^6$

$\alpha = 34.5 - 4.28 \log N = 34.5 - 4.28 \log(82.1 \times 10^6) = 0.627$

$Z_r = \alpha d^2 \Rightarrow \dfrac{5.5\,d^2}{2} = \dfrac{5.5\,(0.75)^2}{2} = 1.55$ k

spacing of shear connectors, $P = \dfrac{3 Z_r}{S_r} = \dfrac{3(1.55k)}{S_r} = \dfrac{4.65}{S_r}$

DESIGN WORK SHEET

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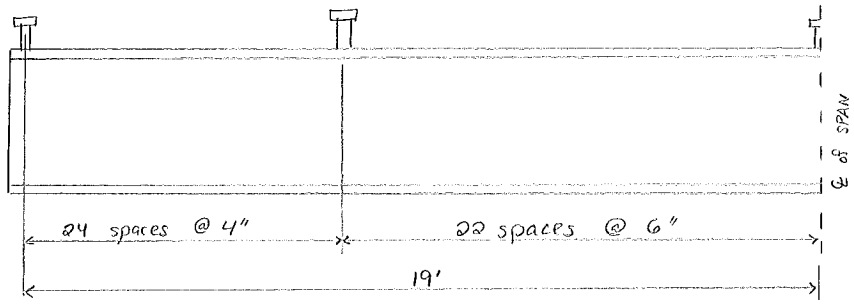
point (ft)	V_r (k)	Q (in^3)	I (in^4)	S_r (k/in)	P (in)
x = 0.0	24.97	308.9	7928.77	0.973	4.78
x = 4.5	21.49	308.9	7928.77	0.837	5.56
x = 9.0	19.16	308.9	7928.77	0.746	6.23
x = 19.0	18.10	308.9	7928.77	0.705	6.60



- shear studs based on strength limit state

$P_1 = A_s F_y = (22.4\ in^2)(50\ ksi) = 1120\ kips$

$P_2 = 0.85\ f_c'\ b_{eff}\ t = 0.85(4.5)(7' \times 12\ in/ft)(7.5'') = 2409.75\ k$

$P = \min(P_1, P_2) = 1120\ k$

$Q_n = 0.5\ A_{sc}\sqrt{f_c' E_c} \le A_{sc} F_u$

$A_{sc} = \frac{\pi (0.75)^2}{4} = 0.442\ in^2$

$E_c = 1820\sqrt{f_c'} = 1820\sqrt{4.5} = 3860\ ksi$

$0.5 A_{sc}\sqrt{f_c' E_c} = 0.5(0.442)\sqrt{(4.5)(3860)} = 29.13\ k$

$A_{sc} F_u = 0.442\ in^2\ (60\ ksi) = 26.52\ k$

$Q_n = 26.52\ k$

number of studs required $= \frac{P}{\phi Q_n} = \frac{1120\ k}{0.85(26.52)} = 50$

number of studs provided $= 3(24 + 22) = 138$ ∴ OK ✓

- check bridge deflections

$\Delta_{DL} = \frac{5 w L^4}{384 EI} = \frac{5(0.8825\ k/ft)(38')^4\ (1728\ in^3/ft^3)}{384\ (29{,}000\ ksi)(2100\ in^4)} = 0.68''$

$\Delta_{SDL} = \frac{5 w L^4}{384 EI} = \frac{5(0.35\ k/ft)(38)^4(1728)}{384\ (29000)(6004.68)} = 0.095''$

DESIGN WORK SHEET

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P = wheel load $(DF)(1+I) = 16^k(0.539)(1+0.3) = 11.22^k$

$$\Delta_{LL+I} = \frac{PL^3}{48EI} = \frac{(11.22)(38)^3(1728)}{48(29000)(7928.77)} = 0.10''$$

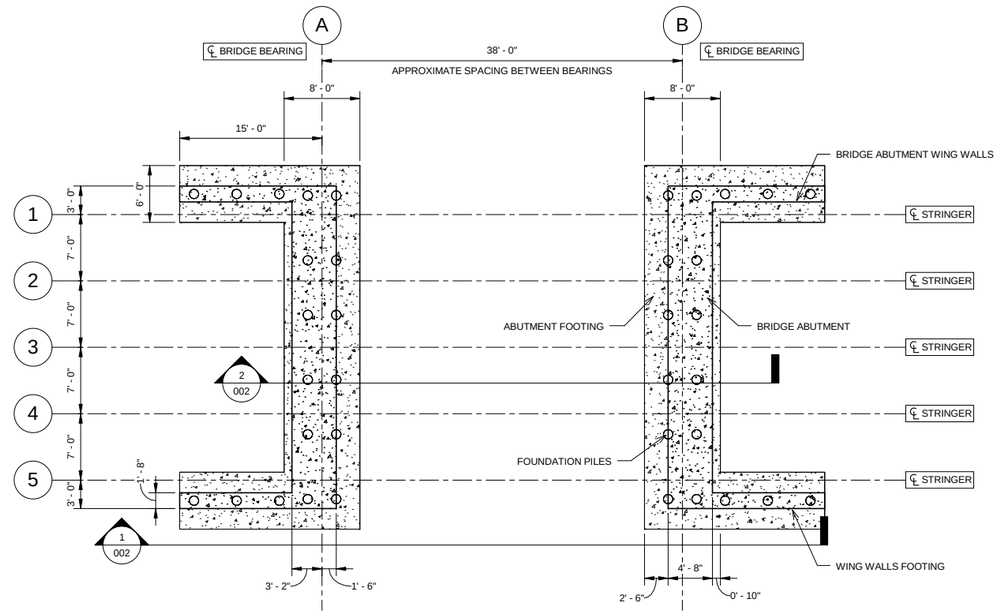
$$\Delta_{LL+I\,max} \leq \frac{L}{800} = \frac{38'(12''/ft)}{800} = 0.57'' > 0.10'' \quad \therefore \frac{OK}{?}$$

⇒ however to increase pedestrian comfort use:

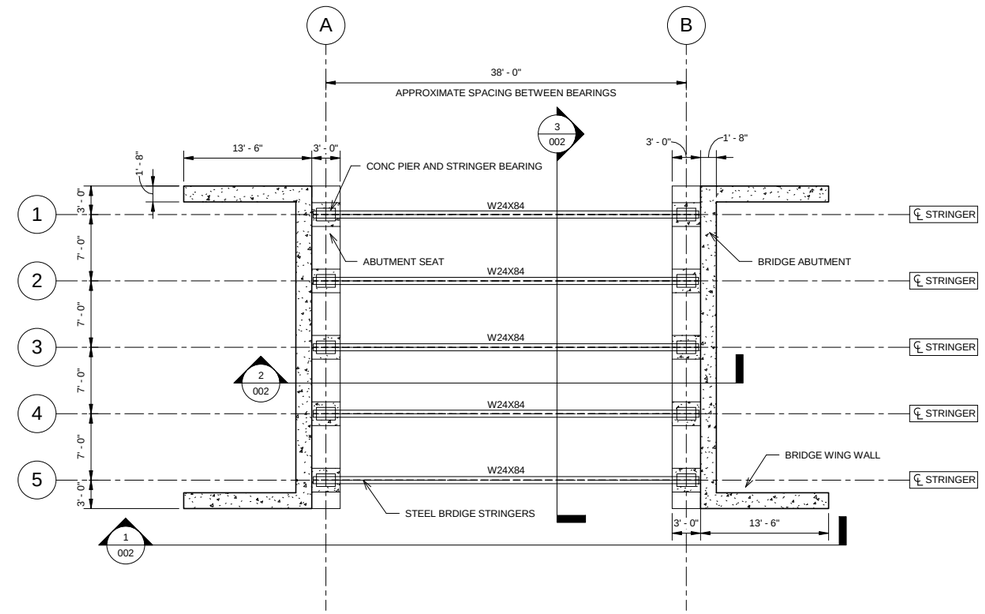
$$\Delta_{LL+I} \leq \frac{L}{1000} = \frac{38'(12''/ft)}{1000} = 0.456'' > 0.10'' \quad \therefore \frac{OK}{?}$$

- preliminary bridge design is sufficient for the bridge superstructure

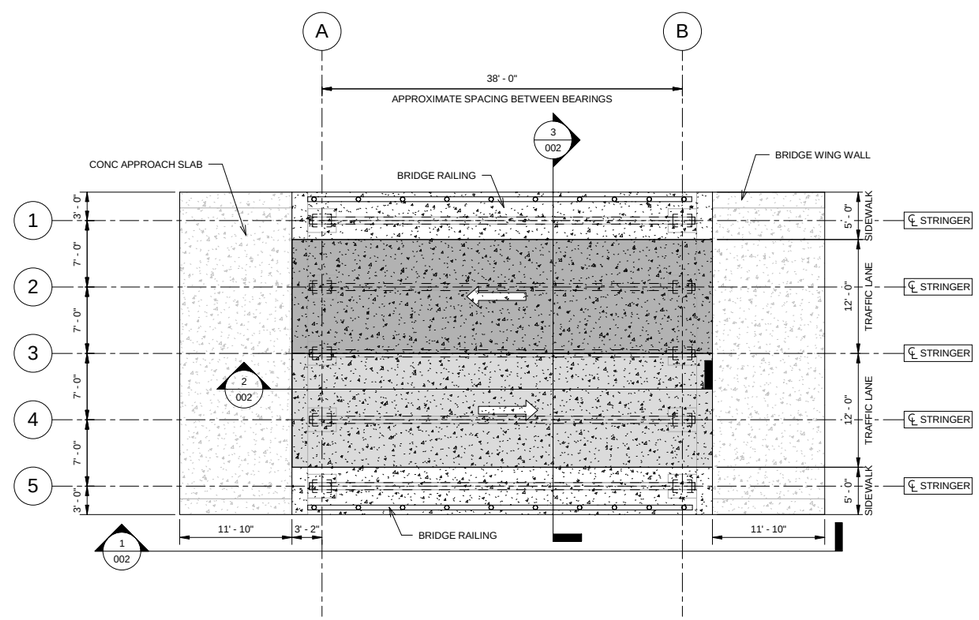
CONCEPT DRAWINGS - WAVERLY PLACE BRIDGE, ALTERNATIVES 1F & 1M



1 B/FTG
1/8" = 1'-0"



2 B/Bridge Chord
1/8" = 1'-0"



3 T/Slab
1/8" = 1'-0"

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SYMBOL	REVISION/ DESCRIPTION	DATE	APPR.

DESIGNED BY: AMP | DATE: 6 FEB 2015 | PROJECT STATUS: CONCEPTS DESIGNS
DWN BY: AMP | CKD BY: Checker | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: 1:1 | PLOT DATE: | FILE NUMBER:
SIZE: ANSI D | FILE NAME:

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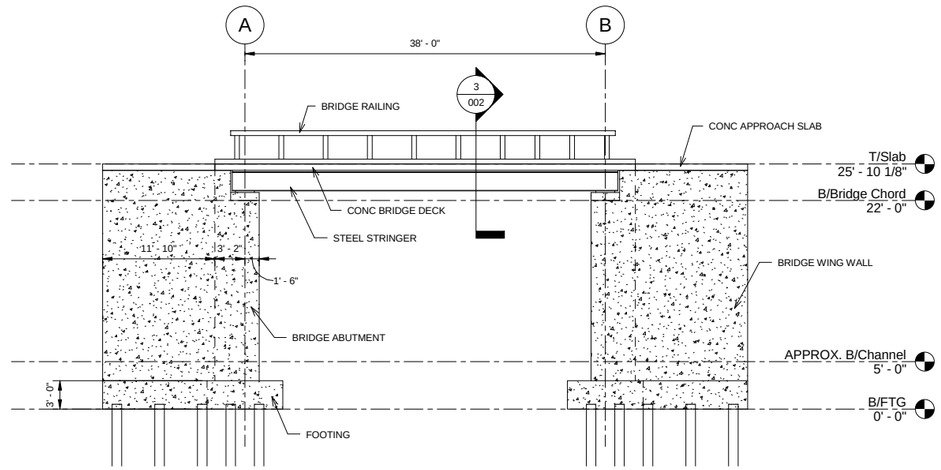
MAMARONECK
MAMARONECK & SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT
NEW YORK

STRUCTURAL CONCEPT DRAWINGS

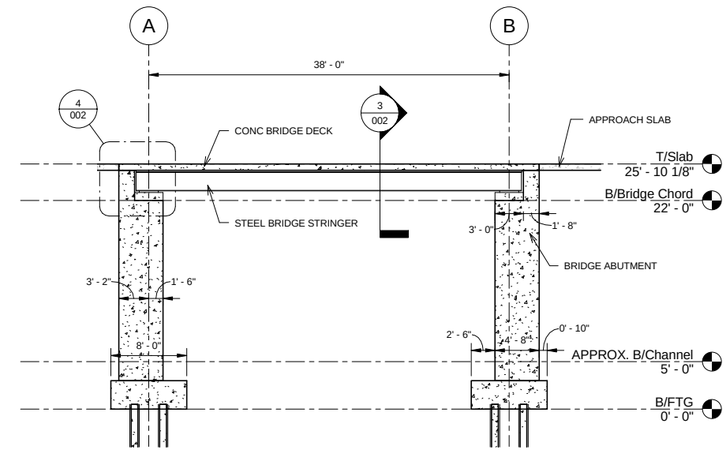
SHEET REFERENCE NUMBER:
001

FILE PATH: S:\fileroom\civil\Mamaroneck-Sheldrake Feasibility Study\STRUC\MODELS\Revit_Bridge Concept_Waverly Place.rvt
DATE: 1/29/2015 12:12:19 PM

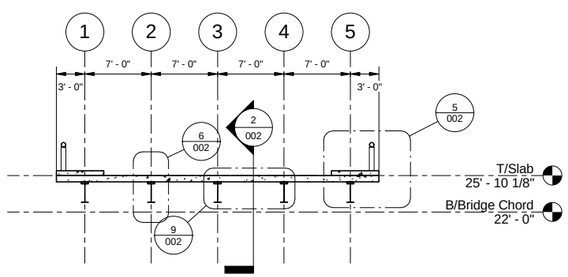
CONCEPT DRAWINGS - WAVERLY PLACE BRIDGE, ALTERNATIVES 1F & 1M



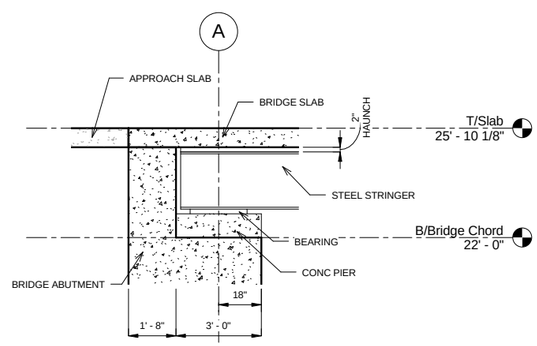
1 BRIDGE SECTION 1
1/8" = 1'-0"



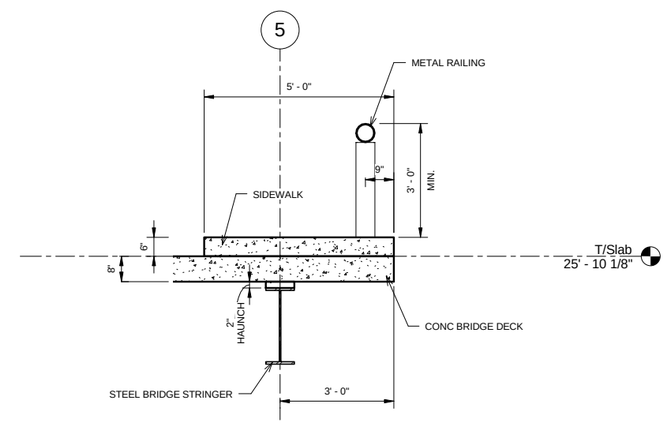
2 BRIDGE SECTION 2
1/8" = 1'-0"



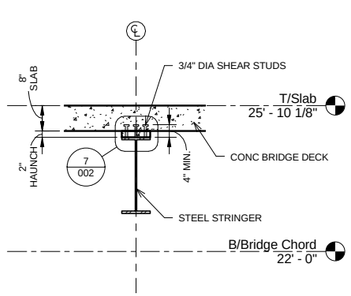
3 BRIDGE SECTION 3
1/8" = 1'-0"



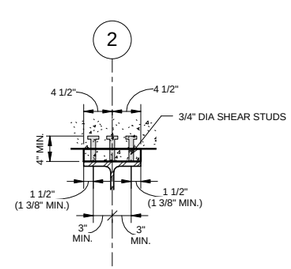
4 STRINGER BEARING DETAIL
3/8" = 1'-0"



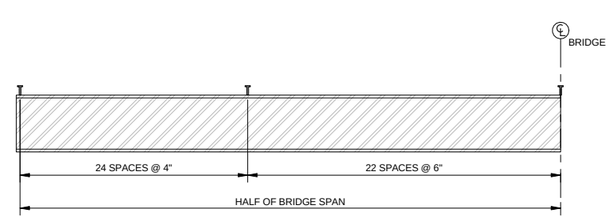
5 SIDEWALK & RAILING DETAIL
1/2" = 1'-0"



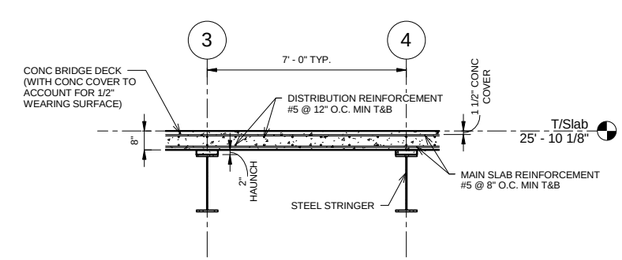
6 STRINGER DETAIL
1/2" = 1'-0"



7 SHEAR STUD DETAIL
1" = 1'-0"



8 SHEAR STUD SPACING DETAIL
3/8" = 1'-0"



9 SLAB REINFORCEMENT DETAIL
3/8" = 1'-0"

US Army Corps of Engineers
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SYMBOL	REVISION DESCRIPTION	DATE	APPR.

DESIGNED BY: AMP
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PLOT SCALE: As indicated
PLOT DATE:
SIZE: ANSI D
FILE NAME:
DATE: 6 FEB 2015
PROJECT STATUS: CONCEPTS DESIGNS
SOLICITATION NO.:
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MAMARONECK NEW YORK
MAMARONECK & SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT

STRUCTURAL CONCEPT DRAWINGS

SHEET REFERENCE NUMBER:
002

FILE PATH: S:\fileroom\civil\Mamaroneck-Sheldrake Feasibility Study\STRUC\MODELS\Revit_Bridge Concept_Waverly Place.rvt
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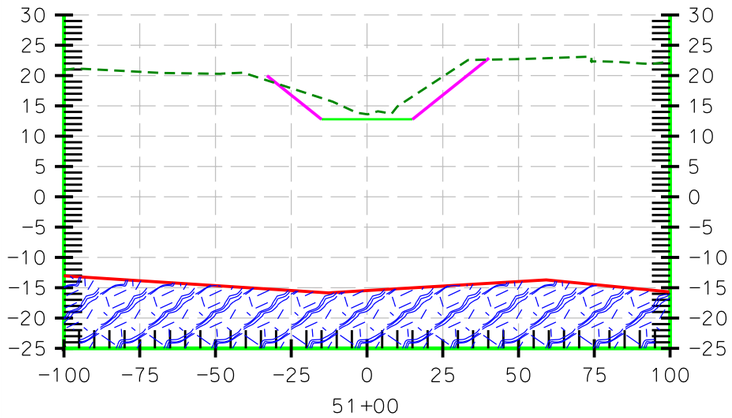
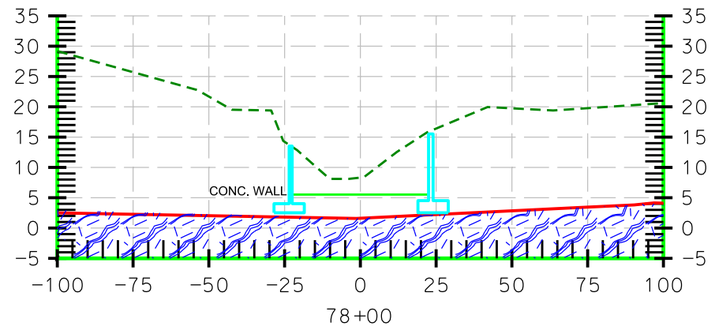
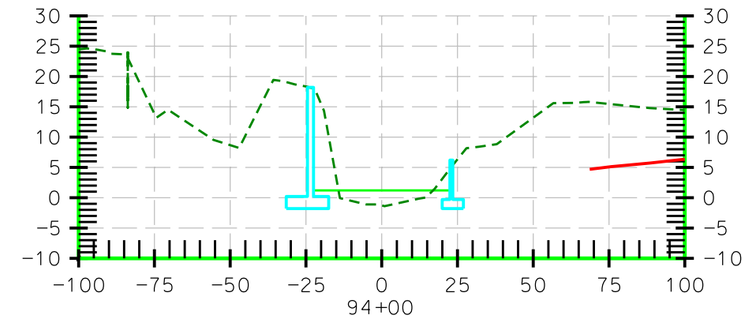
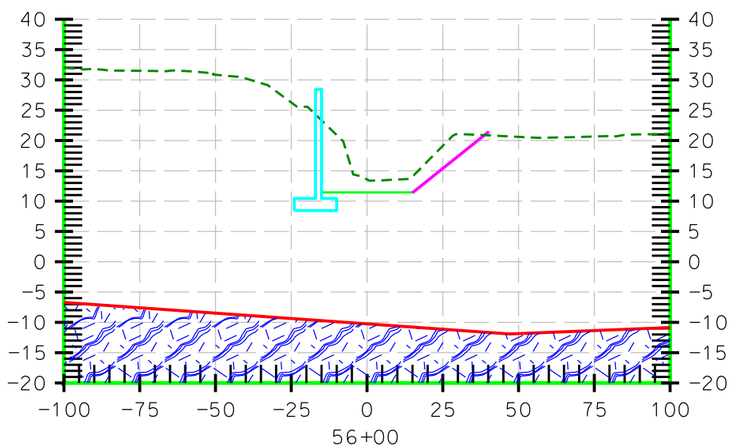
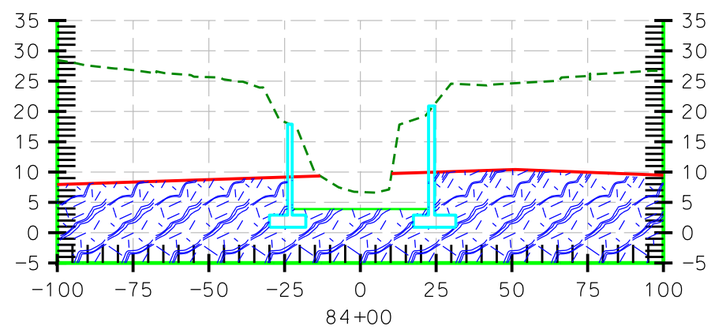
Alternative 1F Mamaroneck River X-Sections

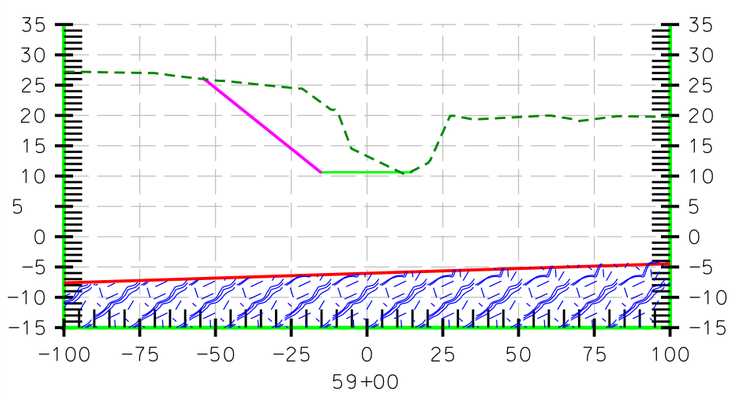


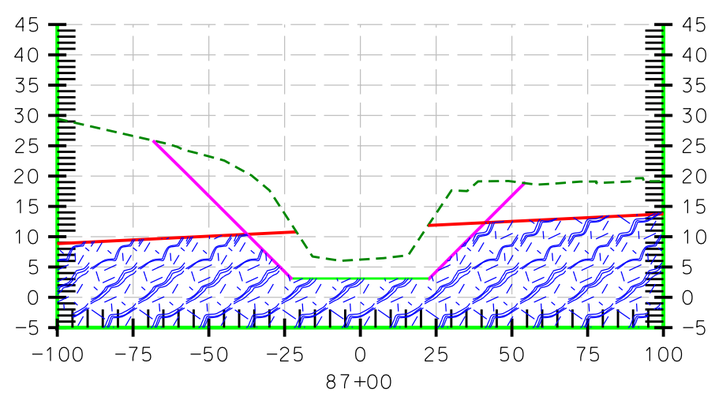












SYMBOL	SURFACE
	APPROXIMATE CURRENT SURVEY ELEVATIONS
	APPROXIMATE ROCK ELEVATIONS
	APPROXIMATE NEW CHANNEL OUTLINE
	APPROXIMATE EXISTING ROCK AREAS

Vertical Scaled 2 Times Cross Section Units = Feet

NOTES:

1. APPROXIMATE ROCK INFORMATION IS PROVIDED WHERE AVAILABLE. SEE LEGEND.
2. WALLS SHOWN ARE APPROXIMATE AND DO NOT REPRESENT ACCURATE DIMENSIONS. SEE WALL SECTIONS FOR DETAILS.

300 150 0 300 600FT
H. SCALE: 1"= 300'

150 75 0 150 300FT
V. SCALE: 1"= 150'

US Army Corps of Engineers®
New York District

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DESIGNED BY: DATE:
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U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK

MAMARONECK AND SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT
GENERAL RE-EVALUATION STUDY
V/L OF MAMARONECK, WESTCHESTER COUNTY, NY

STRUCTURAL CROSS-SECTIONS
MAMARONECK RIVER

SHEET IDENTIFICATION
S-501

SYMBOL	SURFACE
	APPROXIMATE CURRENT SURVEY ELEVATIONS
	APPROXIMATE ROCK ELEVATIONS
	APPROXIMATE NEW CHANNEL OUTLINE
	APPROXIMATE EXISTING ROCK AREAS

Vertical Scaled 2 Times — Cross Section Units = Feet

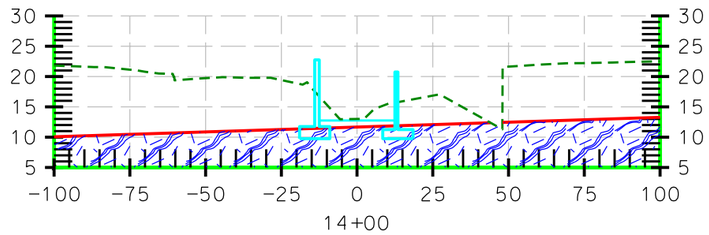
300 150 0 300 600FT
H. SCALE: 1"= 300'

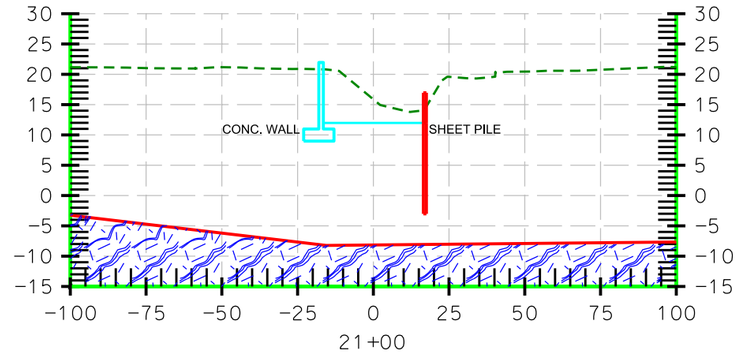
150 75 0 150 300FT
V. SCALE: 1"= 150'

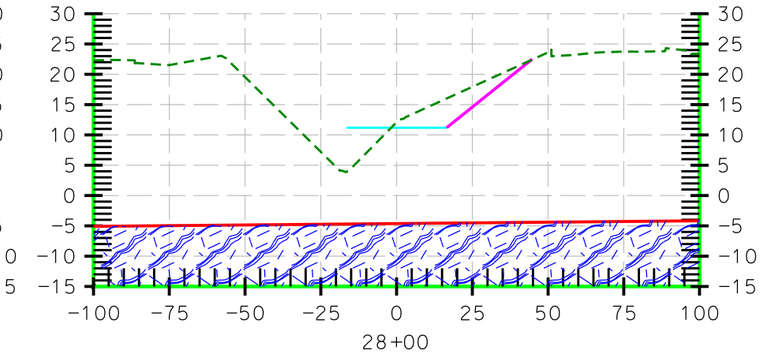
Alternative 1F Sheldrake River X-Sections

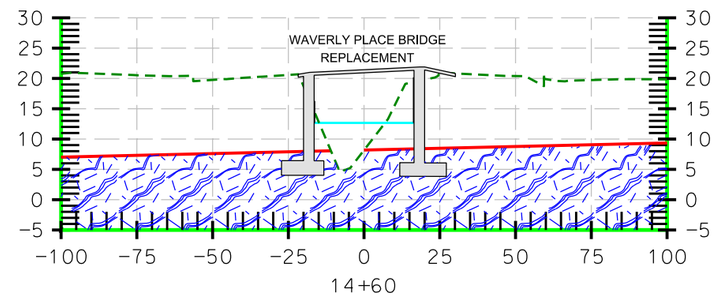
NOTES:

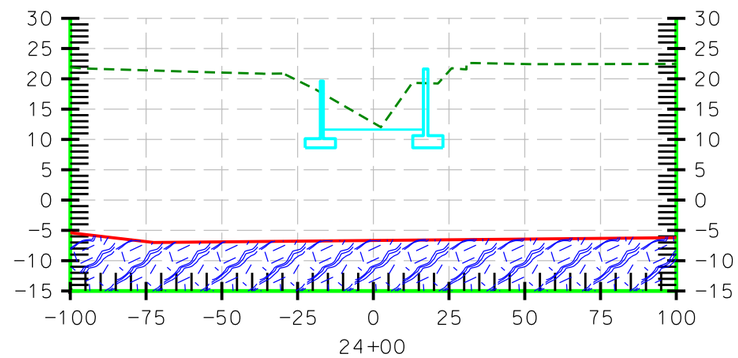
1. APPROXIMATE ROCK INFORMATION IS PROVIDED WHERE AVAILABLE. SEE LEGEND.
2. WALLS SHOWN ARE APPROXIMATE AND DO NOT REPRESENT ACCURATE DIMENSIONS. SEE WALL SECTIONS FOR DETAILS.

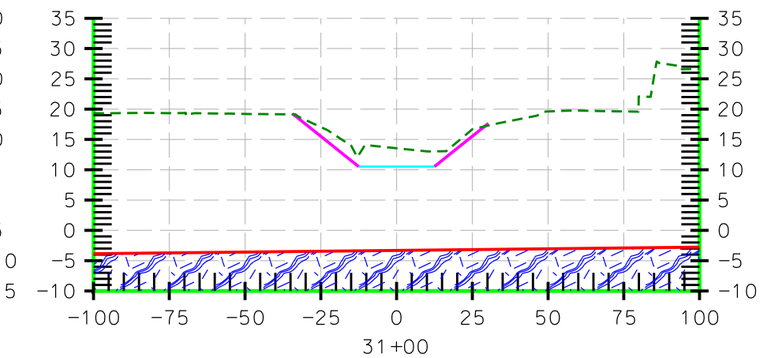


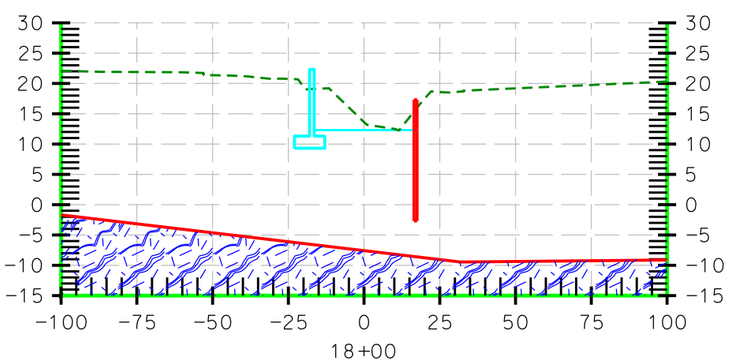


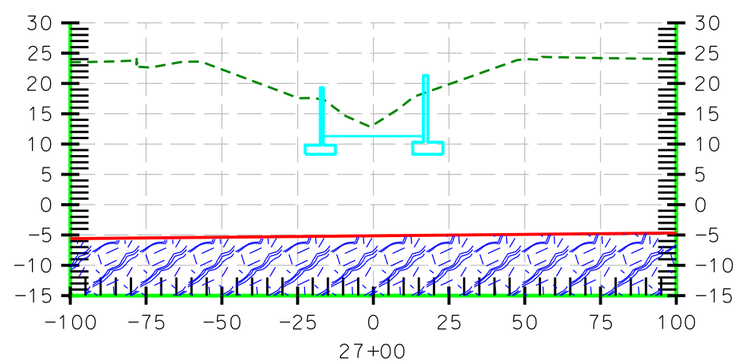


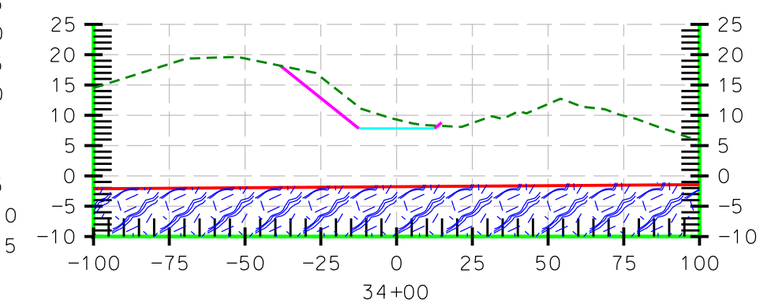












US Army Corps of Engineers®
New York District

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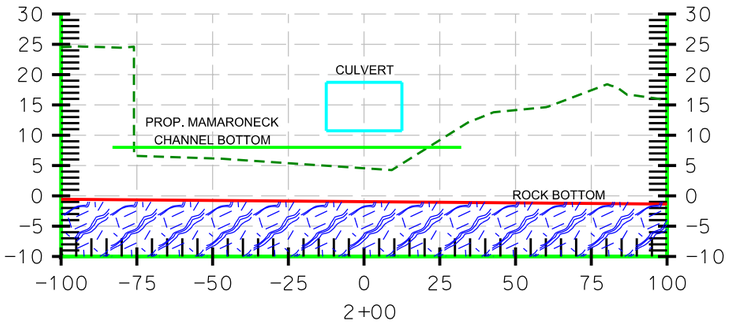
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK

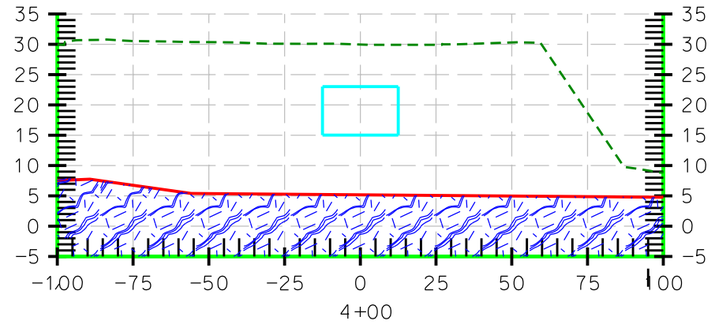
MAMARONECK AND SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT
GENERAL RE-EVALUATION STUDY
VIL. OF MAMARONECK, WESTCHESTER COUNTY, NY

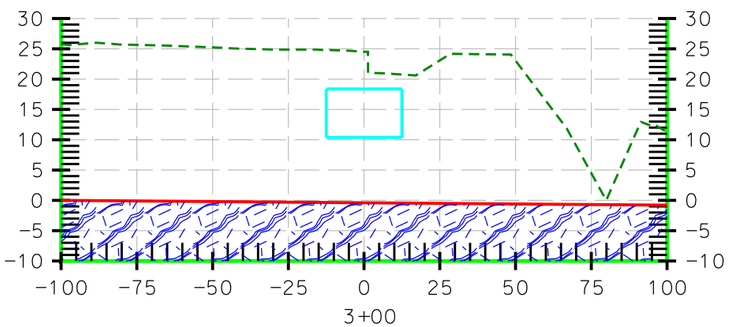
STRUCTURAL CROSS-SECTIONS
SHELDRAKE RIVER

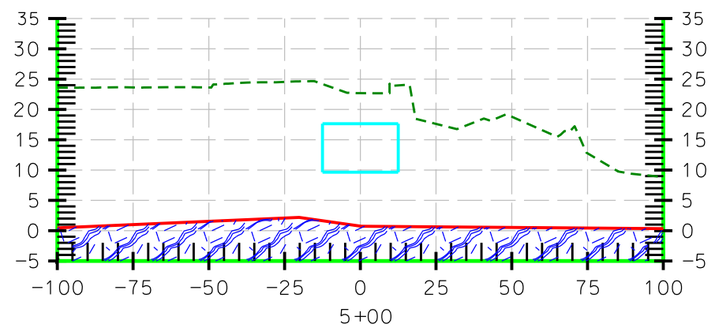
SHEET IDENTIFICATION
S-502

Culvert X-Sections (Alternative 1F, 1M & 1Z)









SYMBOL	SURFACE
	APPROXIMATE CURRENT SURVEY ELEVATIONS
	APPROXIMATE ROCK ELEVATIONS
	APPROXIMATE NEW CHANNEL OUTLINE
	APPROXIMATE EXISTING ROCK AREAS

Vertical Scaled 2 Times — Cross Section Units = Feet

NOTES:

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300 150 0 300 600FT
H. SCALE: 1"= 300'

150 75 0 150 300FT
V. SCALE: 1"= 150'

US Army Corps of Engineers®
New York District

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U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK

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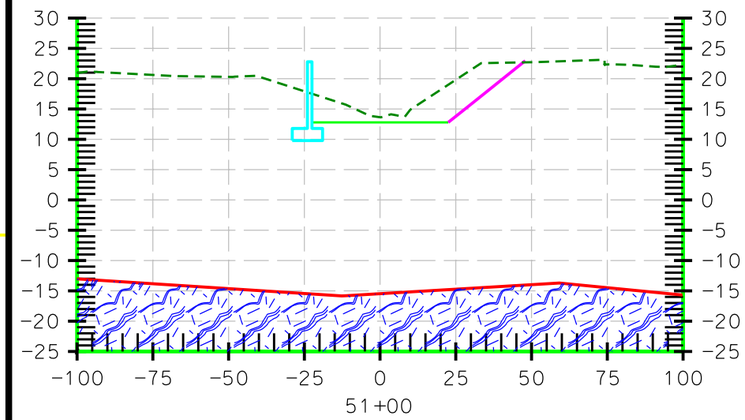
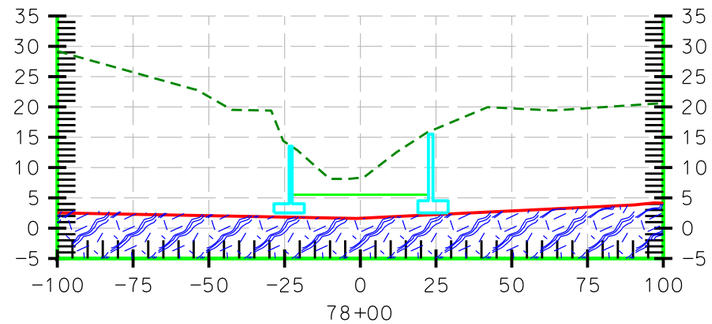
MAMARONECK AND SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT
GENERAL RE-EVALUATION STUDY
VIL. OF MAMARONECK, WESTCHESTER COUNTY, NY

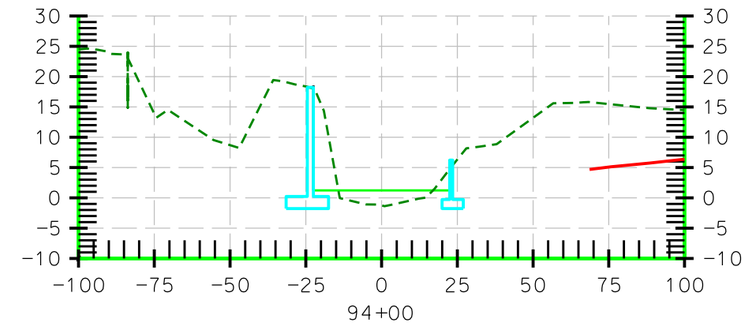
STRUCTURAL CROSS-SECTIONS
CULVERT (NED & LPP)

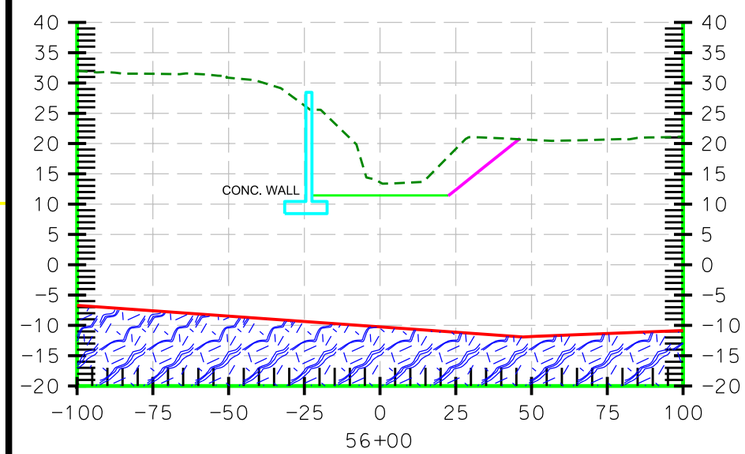
SHEET IDENTIFICATION
S-503

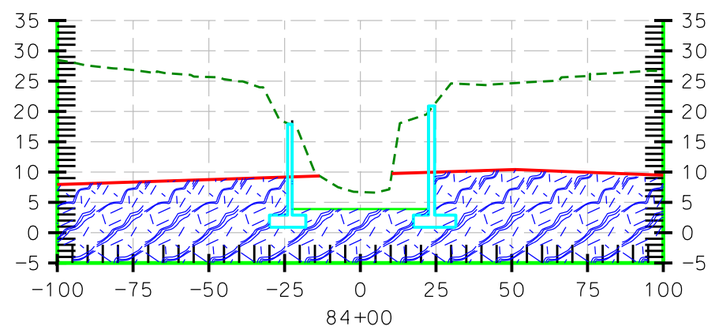
Alternatives 1M & 1Z Mamaroneck River X-Sections

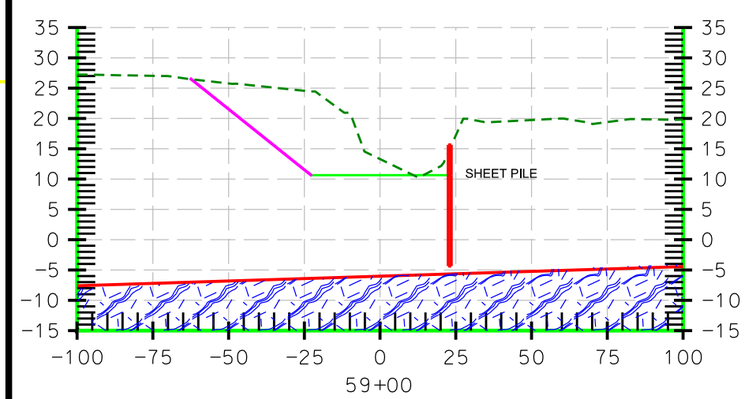


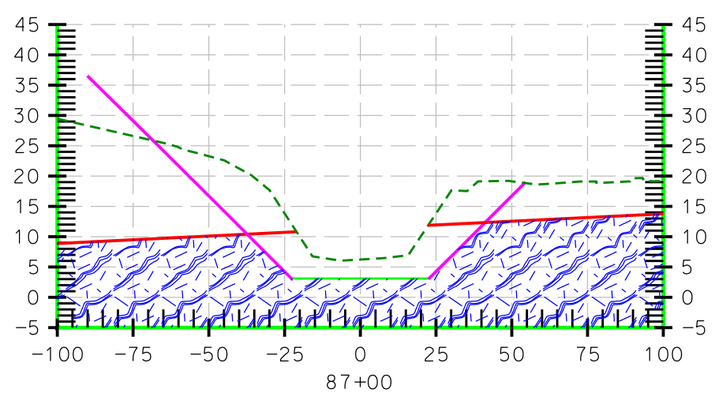












SYMBOL	SURFACE
	APPROXIMATE CURRENT SURVEY ELEVATIONS
	APPROXIMATE ROCK ELEVATIONS
	APPROXIMATE NEW CHANNEL OUTLINE
	APPROXIMATE EXISTING ROCK AREAS

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NOTES:

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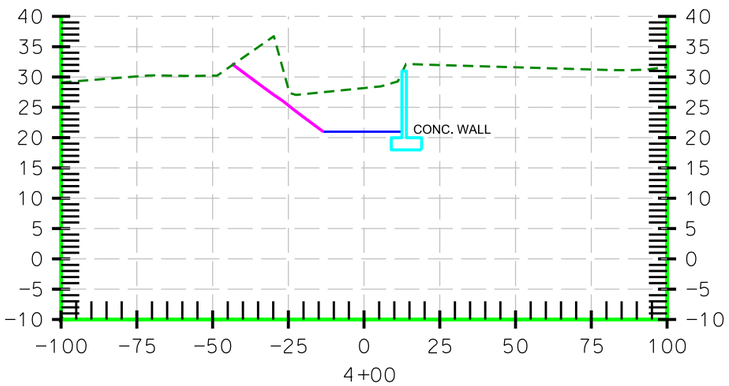
NEW YORK DISTRICT
NEW YORK, NEW YORK

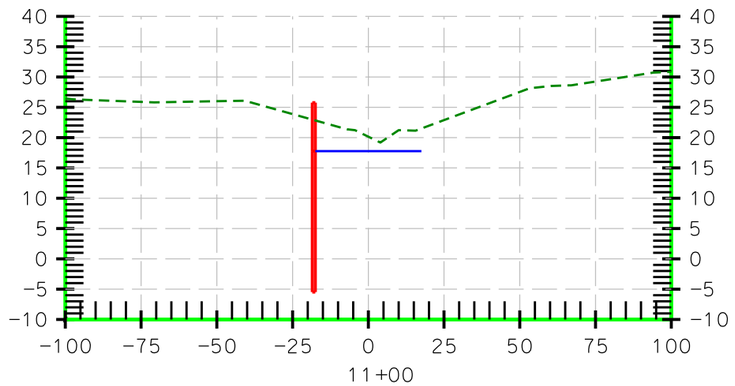
MAMARONECK AND SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT
GENERAL RE-EVALUATION STUDY
VIL. OF MAMARONECK, WESTCHESTER COUNTY, NY

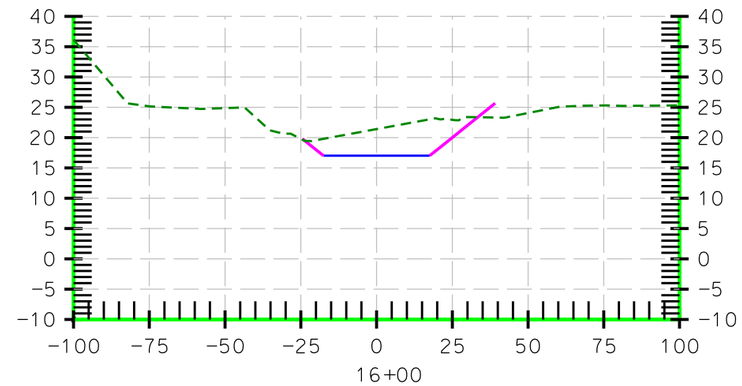
STRUCTURAL CROSS-SECTIONS
MAMARONECK RIVER (LPP)

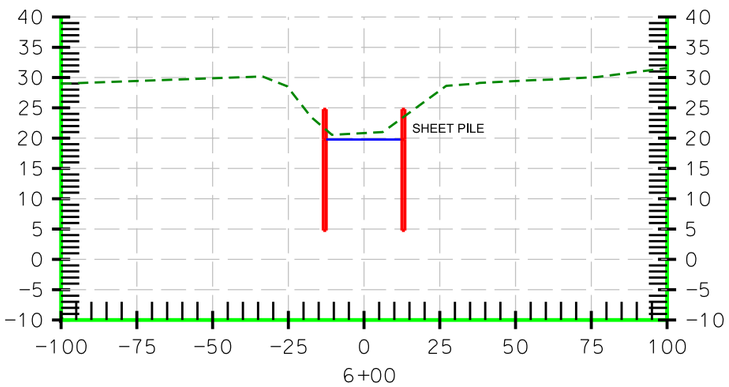
SHEET IDENTIFICATION
S-504

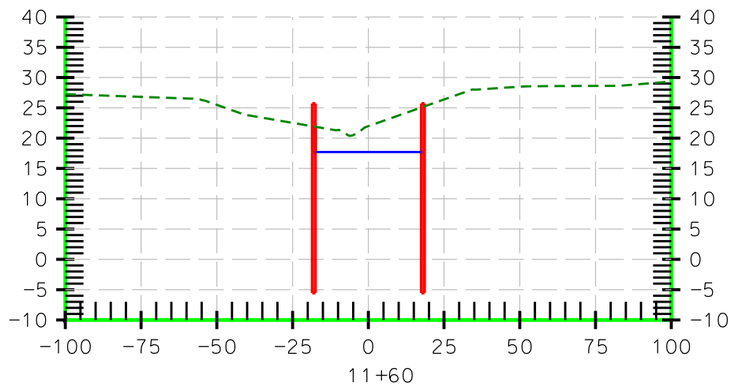
Alternative 1M Mamaroneck River Harbor Heights X-Sections

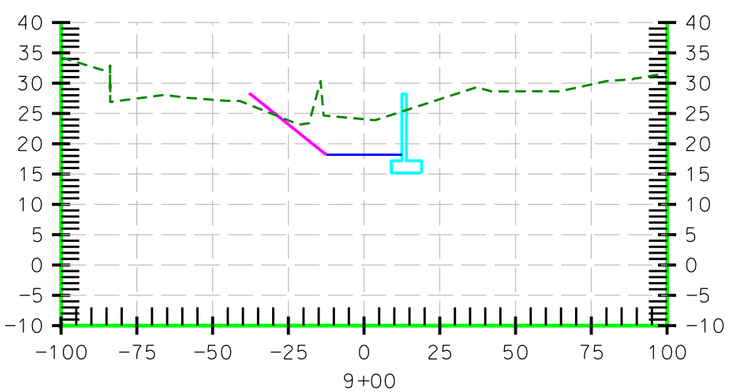


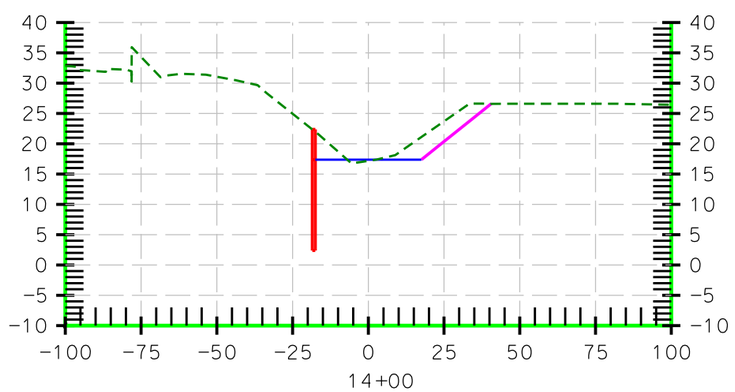












SYMBOL	SURFACE
	APPROXIMATE CURRENT SURVEY ELEVATIONS
	APPROXIMATE ROCK ELEVATIONS
	APPROXIMATE NEW CHANNEL OUTLINE
	APPROXIMATE EXISTING ROCK AREAS

Vertical Scaled 2 Times Cross Section Units = Feet

NOTES:

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U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK

MAMARONECK AND SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT
GENERAL RE-EVALUATION STUDY
VIL. OF MAMARONECK, WESTCHESTER COUNTY, NY

STRUCTURAL CROSS-SECTIONS
MAMARONECK RIVER HH (LPP)

SHEET IDENTIFICATION
S-505

SYMBOL	SURFACE
	APPROXIMATE CURRENT SURVEY ELEVATIONS
	APPROXIMATE ROCK ELEVATIONS
	APPROXIMATE NEW CHANNEL OUTLINE
	APPROXIMATE EXISTING ROCK AREAS

Vertical Scaled 2 Times — Cross Section Units = Feet

300 150 0 300 600FT
H. SCALE: 1"= 300'

150 75 0 150 300FT
V. SCALE: 1"= 150'

Alternatives 1M & 1Z
Sheldrake River X-Sections

NOTES:

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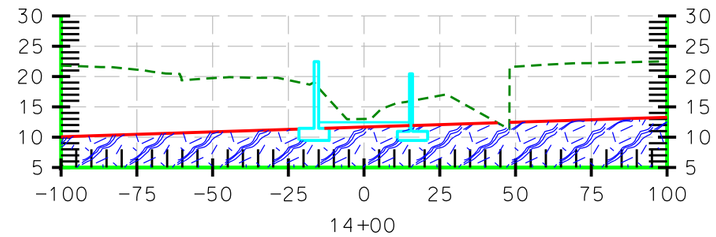

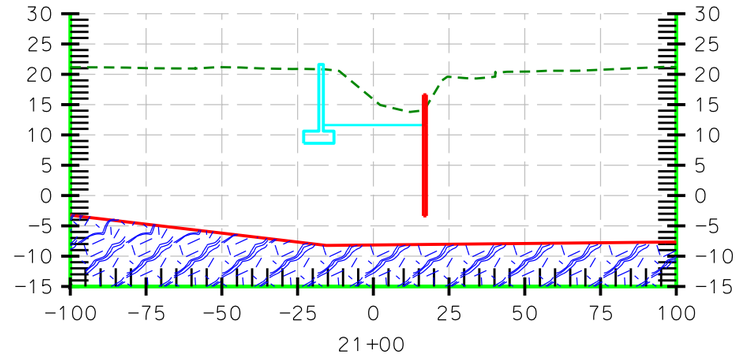

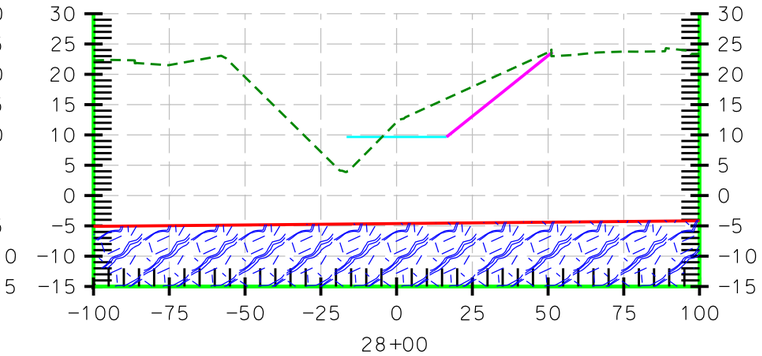

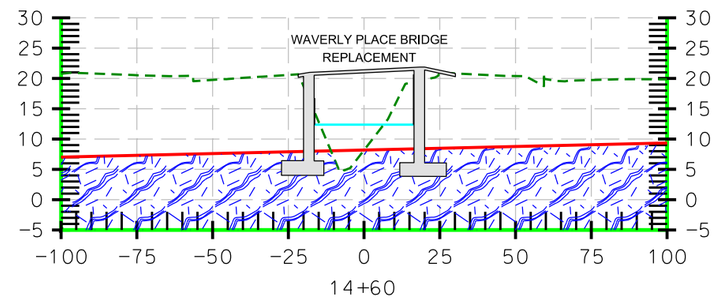

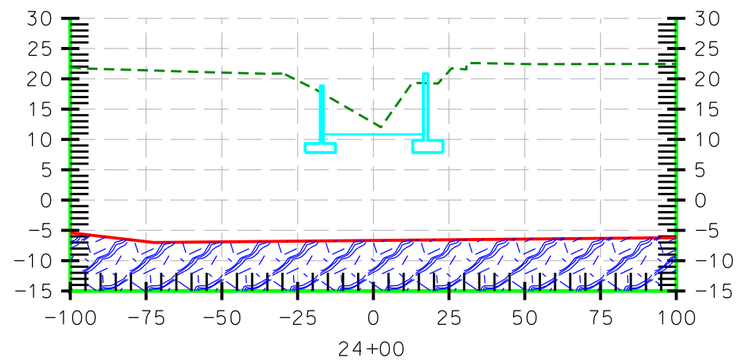

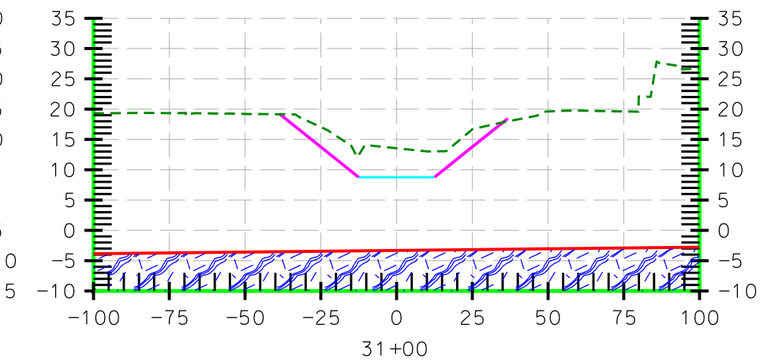

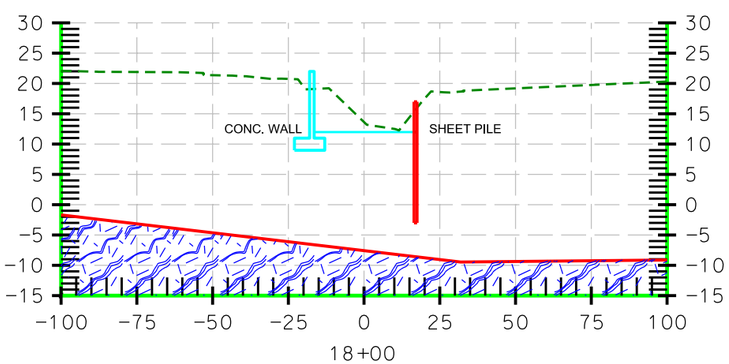

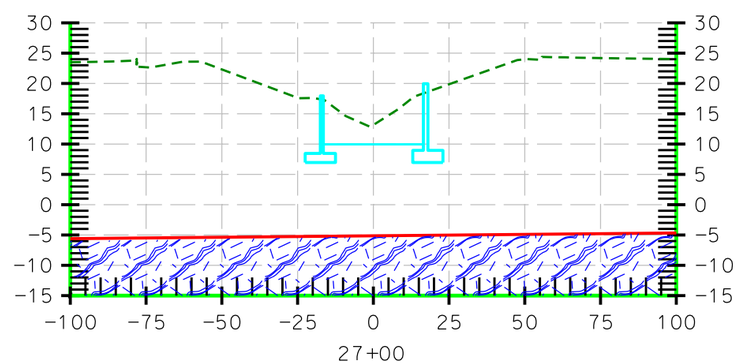

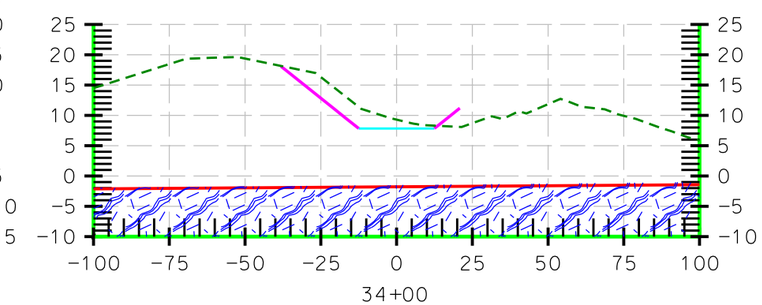

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New York District

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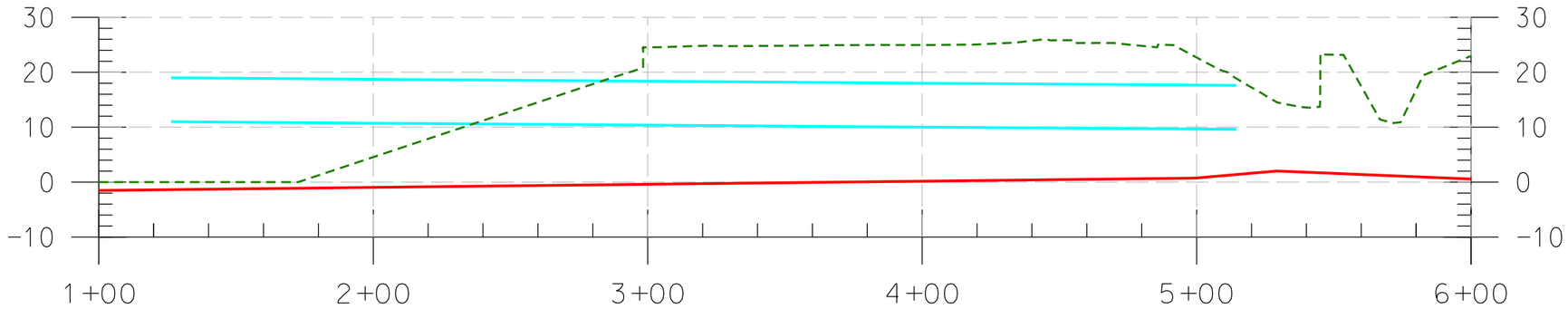
MAMARONECK AND SHELDRAKE RIVER BASIN
FLOOD RISK MANAGEMENT
GENERAL RE-EVALUATION STUDY
VIL. OF MAMARONECK, WESTCHESTER COUNTY, NY

STRUCTURAL CROSS-SECTIONS
SHELDRAKE RIVER (LPP)

SHEET IDENTIFICATION
S-506

Culvert Profile



LINE	SURFACE
———	Culvert
———	BEDROCK
- - - -	Existing Topo

Scaled 2.0 Times Ver.
Scaled 1.0 Times Hor.

Retaining Wall Design

	= Assumed Data
	= Check Value
	= Input Information

Retaining Wall Type C-1

Basic Wall Data

f_c'	3000	psi
fy	60000	psi
qa	3000	psf

wall height above ground	3	feet
wall height below ground	3	feet
Total Wall Height	6	feet

Surcharge loads:		
traffic	300	psf
buildings	0	psf
total surcharge	300	psf

Soil Estimates:		
unit weight	120	psf
phi angle	32	degrees
Ka	0.29	
Kp	3.46	

Horizontal Pressures:			
pa	208.18	lb/ft	
Ha	624.53	lbs.	*(H 1)*
Hp	1867.63	lbs.	

Surcharge Load Pressure:			
equiv. height	2.50	feet	
equiv total hgt	8.50	feet	
equiv pressure	86.74	lb/ft	
Surcharge Force	520.44	lbs.	*(H 2)*

Assumed Wall Dimensions:		
wall thickness	0.42	ft
actual	1	ft
Footing thickness	0.42	ft
actual	1.5	ft
height of stem	4.5	ft
width of footing	1.76	ft
actual	6	ft
x	3	ft

Factor of Safety against Overturning

Overturning Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
H1	624.53	2.00	1249.07	
H2	520.44	3	1561.33	
		Total	2810.40	lb-ft

W	1020
x	1.17

Righting Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
W1	1350.00	3.00	4050.00	
W2	675.00	3.5	2362.50	
W3	1080.00	5	5400.00	
Rv =	3105.00	M =	11812.50	lb-ft

Safety Factor Against Overturning =	4.20	>	2

Factor of Safety against Sliding

Sliding Forces	1144.98	lbs
Resisting Forces	1552.5	lbs

Safety Factor for Sliding =	1.36	>	1.5

Footing Soil Pressure

Rv =	3105.00	located ẍ from the toe of the footing
ẍ =	2.90	feet
A =	6	ft^2
I =	18.00	ft^4
f_{toe} =	-569.65033	psf
f_{heel} =	-465.35	psf

Design of Stem for Moment

**lateral forces applied to stem are calculated using a load factor of 1.6*

soil load pressure	249.81	lb/ft
H1	562.08	lbs.
surcharge pressure	138.79	lb/ft
H2	624.53	lbs.

Mu =	2248.32	ft-lb

Concrete Design

ρ =	0.0090		
Mu/(Φbd^2) =	482.6	psi	(from Table A.12)
b =	12	inches	
d =	2.28	inches	
h =	4.78	inches	
Actual h =	12	inches	
Actual d =	9.5	inches	

Mu/(Φbd^2) =	27.68	psi	
ρ =	0.0033	(min)	(from Table A.12)
As =	0.3762	in^2	
Use #6 @ 12" spacing			
Actual As =	0.44	in^2	
Actual ρ =	0.0038596		
min. ρ (ACI Section 14.3) =			0.0015
min. Horizontal As =	0.36	in^2	
say, 1/3 inside face =	0.12	in^2	
say, 2/3 outside face =	0.24	in^2	
Inside Face: Use #4 @ 12" spacing			
Actual As =	0.2	in^2	
Outside Face: Use #5 @ 12" spacing			
Actual As =	0.31	in^2	

Design of Stem for Shear

Vu =	1186.61	lbs.
ΦVc =	9366.06	lbs
ΦVc > Vu	YES	

Design of Heel

footing t =	18	inches
b =	12	inches
d =	14.5	inches

Vu =	2988	lbs
ΦVc =	14295.56	llbs
ΦVc > Vu	YES	

Mu @ face of stem	2988	ft-lb
Mu/(Φbd^2) =	15.79	psi
ρ =	0.0033	(Table A.12)
As =	0.574	in^2
Use #7 @ 12" spacing		
As =	0.6	in^2
d bar =	0.88	in
Ld =	47.93	inches
L avail =	48	inches
Ld < L avil	YES	
(Ld from back of wall into toe)		

temp. steel = #4 @ 12" spacing

Design of Toe

factored f_{toe} =	-911.44	psf
factored f_{heel} =	-744.56	psf
length of toe =	3	ft
f @ wall =	828.00	psf
Vu1 =	1367.16	lbs.
Vu2 =	1242.00	lbs.
Vu =	2609.16	lbs.
Mu @ wall =	3976.32	ft-lbs
Mu/(Φbd^2) =	21.01	psi
ρ =	0.0033	(Table A.12)
As =	0.5742	in^2
Use #7 @ 12" spacing		
As =	0.6	in^2

Retaining Wall Design

	= Assumed Data
	= Check Value
	= Input Information

Retaining Wall Type C-2

Basic Wall Data

f_c'	3000	psi
fy	60000	psi
qa	3000	psf

wall height above ground	5	feet
wall height below ground	3	feet
Total Wall Height	8	feet

Surcharge loads:		
traffic	300	psf
buildings	0	psf
total surcharge	300	psf

Soil Estimates:		
unit weight	120	psf
phi angle	32	degrees
Ka	0.29	
Kp	3.46	

Horizontal Pressures:			
pa	277.57	lb/ft	
Ha	1110.28	lbs.	*(H 1)*
Hp	1867.63	lbs.	

Surcharge Load Pressure:			
equiv. height	2.50	feet	
equiv total hgt	10.50	feet	
equiv pressure	86.74	lb/ft	
Surcharge Force	693.93	lbs.	*(H 2)*

Assumed Wall Dimensions:		
wall thickness	0.56	ft
actual	1	ft
Footing thickness	0.56	ft
actual	1.5	ft
height of stem	6.5	ft
width of footing	2.26	ft
actual	7	ft
x	4	ft

Factor of Safety against Overturning

Overturning Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
H1	1110.28	2.67	2960.75	
H2	693.93	4	2775.71	
		Total	5736.46	lb-ft

W	1260
x	1.51

Righting Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
W1	1575.00	3.50	5512.50	
W2	975.00	3.5	3412.50	
W3	2340.00	5.5	12870.00	
Rv =	4890.00	M =	21795.00	lb-ft

Safety Factor Against Overturning =	3.80	>	2

Factor of Safety against Sliding

Sliding Forces	1804.21	lbs
Resisting Forces	2445	lbs

Safety Factor for Sliding =	1.36	>	1.5

Footing Soil Pressure

Rv =	4890.00	located ẍ from the toe of the footing
ẍ =	3.28	feet
A =	7	ft^2
I =	28.58	ft^4
f_{toe} =	-827.93368	psf
f_{heel} =	-569.21	psf

Design of Stem for Moment

**lateral forces applied to stem are calculated using a load factor of 1.6*

soil load pressure	360.84	lb/ft
H1	1172.74	lbs.
surcharge pressure	138.79	lb/ft
H2	902.10	lbs.

Mu =	5472.77	ft-lb

Concrete Design

ρ =	0.0090		
Mu/(Φbd^2) =	482.6	psi	(from Table A.12)
b =	12	inches	
d =	3.55	inches	
h =	6.05	inches	
Actual h =	12	inches	
Actual d =	9.5	inches	

Mu/(Φbd^2) =	67.38	psi	
ρ =	0.0033	(min)	(from Table A.12)
As =	0.3762	in^2	
Use #6 @ 12" spacing			
Actual As =	0.44	in^2	
Actual ρ =	0.0038596		
min. ρ (ACI Section 14.3) =			0.0015
min. Horizontal As =	0.36	in^2	
say, 1/3 inside face =	0.12	in^2	
say, 2/3 outside face =	0.24	in^2	
Inside Face: Use #4 @ 12" spacing			
Actual As =	0.2	in^2	
Outside Face: Use #5 @ 12" spacing			
Actual As =	0.31	in^2	

Design of Stem for Shear

Vu =	2074.84	lbs.
ΦVc =	9366.06	lbs
ΦVc > Vu	YES	

Design of Heel

footing t =	18	inches
b =	12	inches
d =	14.5	inches

Vu =	5346	lbs
ΦVc =	14295.56	llbs
ΦVc > Vu	YES	

Mu @ face of stem	8019	ft-lb
Mu/(Φbd^2) =	42.38	psi
ρ =	0.0033	(Table A.12)
As =	0.574	in^2
Use #7 @ 12" spacing		
As =	0.6	in^2
d bar =	0.88	in
Ld =	47.93	inches
L avail =	48	inches
Ld < L avil	YES	
(Ld from back of wall into toe)		

temp. steel = #4 @ 12" spacing

Design of Toe

factored f_{toe} =	-1324.69	psf
factored f_{heel} =	-910.73	psf
length of toe =	3	ft
f @ wall =	1147.28	psf
Vu1 =	1987.04	lbs.
Vu2 =	1720.92	lbs.
Vu =	3707.97	lbs.
Mu @ wall =	5695.01	ft-lbs
Mu/(Φbd^2) =	30.10	psi
ρ =	0.0033	(Table A.12)
As =	0.5742	in^2
Use #7 @ 12" spacing		
As =	0.6	in^2

Retaining Wall Design

	= Assumed Data
	= Check Value
	= Input Information

Retaining Wall Type C-3

Basic Wall Data

f_c'	3000	psi
fy	60000	psi
qa	3000	psf

wall height above ground	8	feet
wall height below ground	3	feet
Total Wall Height	11	feet

Surcharge loads:		
traffic	300	psf
buildings	0	psf
total surcharge	300	psf

Soil Estimates:		
unit weight	120	psf
phi angle	32	degrees
Ka	0.29	
Kp	3.46	

Horizontal Pressures:			
pa	381.66	lb/ft	
Ha	2099.13	lbs.	*(H 1)*
Hp	1867.63	lbs.	

Surcharge Load Pressure:		
equiv. height	2.50	feet
equiv total hgt	13.50	feet
equiv pressure	86.74	lb/ft
Surcharge Force	954.15	lbs. *(H 2)*

Assumed Wall Dimensions:		
wall thickness	0.77	ft
actual	1	ft
Footing thickness	0.77	ft
actual	1.5	ft
height of stem	9.5	ft
width of footing	3.00	ft
actual	10	ft
x	6	ft

Factor of Safety against Overturning

Overturning Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
H1	2099.13	3.67	7696.80	
H2	954.15	5.5	5247.82	
		Total	12944.62	lb-ft

W	1620
x	2.00

Righting Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
W1	2250.00	5.00	11250.00	
W2	1425.00	4.5	6412.50	
W3	5700.00	7.5	42750.00	
Rv =	9375.00	M =	60412.50	lb-ft

Safety Factor Against Overturning =	4.67	>	2

Factor of Safety against Sliding

Sliding Forces	3053.28	lbs
Resisting Forces	4687.5	lbs

Safety Factor for Sliding =	1.54	>	1.5

Footing Soil Pressure

Rv =	9375.00	located ẍ from the toe of the footing
ẍ =	5.06	feet
A =	10	ft^2
I =	83.33	ft^4
f_{toe} =	-901.92715	psf
fheel =	-973.07	psf

Design of Stem for Moment

**lateral forces applied to stem are calculated using a load factor of 1.6*

soil load pressure	527.38	lb/ft
H1	2505.07	lbs.
surcharge pressure	138.79	lb/ft
H2	1318.46	lbs.

Mu =	14195.42	ft-lb

Concrete Design

ρ =	0.0090		
Mu/(Φbd^2) =	482.6	psi	(from Table A.12)
b =	12	inches	
d =	5.72	inches	
h =	8.22	inches	
Actual h =	12	inches	
Actual d =	9.5	inches	

Mu/(Φbd^2) =	174.77	psi	
ρ =	0.0033	(min)	(from Table A.12)
As =	0.3762	in^2	
Use #6 @ 12" spacing			
Actual As =	0.44	in^2	
Actual ρ =	0.0038596		
min. ρ (ACI Section 14.3) =			0.0015
min. Horizontal As =	0.36	in^2	
say, 1/3 inside face =	0.12	in^2	
say, 2/3 outside face =	0.24	in^2	
Inside Face: Use #4 @ 12" spacing			
Actual As =	0.2	in^2	
Outside Face: Use #5 @ 12" spacing			
Actual As =	0.31	in^2	

Design of Stem for Shear

Vu =	3823.53	lbs.
ΦVc =	9366.06	lbs
ΦVc > Vu	YES	

Design of Heel

footing t =	18	inches
b =	12	inches
d =	14.5	inches

Vu =	11070	lbs
ΦVc =	14295.56	llbs
ΦVc > Vu	YES	

Mu @ face of stem	27675	ft-lb
Mu/(Φbd^2) =	146.25	psi
ρ =	0.0033	(Table A.12)
As =	0.574	in^2
Use #7 @ 12" spacing		
As =	0.6	in^2
d bar =	0.88	in
Ld =	47.93	inches
L avail =	60	inches
Ld < L avil	YES	
(Ld from back of wall into toe)		

temp. steel = #4 @ 12" spacing

Design of Toe

factored f_{toe} =	-1443.08	psf
factored fheel =	-1556.92	psf
length of toe =	4	ft
f @ wall =	1488.62	psf
Vu1 =	2886.17	lbs.
Vu2 =	2977.23	lbs.
Vu =	5863.40	lbs.
Mu @ wall =	11666.09	ft-lbs
Mu/(Φbd^2) =	61.65	psi
ρ =	0.0033	(Table A.12)
As =	0.5742	in^2
Use #7 @ 12" spacing		
As =	0.6	in^2

Retaining Wall Design

	= Assumed Data
	= Check Value
	= Input Information

Retaining Wall Type C-4

Basic Wall Data

f_c'	3000	psi
fy	60000	psi
qa	3000	psf

wall height above ground	10	feet
wall height below ground	3	feet
Total Wall Height	13	feet

Surcharge loads:		
traffic	300	psf
buildings	0	psf
total surcharge	300	psf

Soil Estimates:		
unit weight	120	psf
phi angle	32	degrees
Ka	0.29	
Kp	3.46	

Horizontal Pressures:			
pa	451.05	lb/ft	
Ha	2931.84	lbs.	*(H 1)*
Hp	1867.63	lbs.	

Surcharge Load Pressure:			
equiv. height	2.50	feet	
equiv total hgt	15.50	feet	
equiv pressure	86.74	lb/ft	
Surcharge Force	1127.63	lbs.	*(H 2)*

Assumed Wall Dimensions:		
wall thickness	0.91	ft
actual	1.5	ft
Footing thickness	0.91	ft
actual	2	ft
height of stem	11	ft
width of footing	3.48	ft
actual	10	ft
x	6.5	ft

Factor of Safety against Overturning

Overturning Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
H1	2931.84	4.33	12704.64	
H2	1127.63	6.5	7329.60	
		Total	20034.23	lb-ft

W	1860
x	2.32

Righting Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
W1	3000.00	5.00	15000.00	
W2	2475.00	4.25	10518.75	
W3	6600.00	7.5	49500.00	
Rv =	12075.00	M =	75018.75	lb-ft

Safety Factor Against Overturning =	3.74	>	2

Factor of Safety against Sliding

Sliding Forces	4059.47	lbs
Resisting Forces	6037.5	lbs

Safety Factor for Sliding =	1.49	>	1.5

Footing Soil Pressure

Rv =	12075.00	located ẍ from the toe of the footing
ẍ =	4.55	feet
A =	10	ft^2
I =	83.33	ft^4
f_{toe} =	-1530.929	psf
f_{heel} =	-884.07	psf

Design of Stem for Moment

**lateral forces applied to stem are calculated using a load factor of 1.6*

soil load pressure	610.66	lb/ft
H1	3358.60	lbs.
surcharge pressure	138.79	lb/ft
H2	1526.64	lbs.

Mu =	20711.39	ft-lb

Concrete Design

ρ =	0.0090		
Mu/(Φbd^2) =	482.6	psi	(from Table A.12)
b =	12	inches	
d =	6.91	inches	
h =	9.41	inches	
Actual h =	18	inches	
Actual d =	15.5	inches	

Mu/(Φbd^2) =	95.79	psi	
ρ =	0.0033	(min)	(from Table A.12)
As =	0.6138	in^2	
Use #8 @ 12" spacing			
Actual As =	0.79	in^2	
Actual ρ =	0.0042473		
min. ρ (ACI Section 14.3) =			0.0015
min. Horizontal As =	0.54	in^2	
say, 1/3 inside face =	0.18	in^2	
say, 2/3 outside face =	0.36	in^2	
Inside Face: Use #4 @ 12" spacing			
Actual As =	0.2	in^2	
Outside Face: Use #5 @ 9" spacing			
Actual As =	0.41	in^2	

Design of Stem for Shear

Vu =	4885.24	lbs.
ΦVc =	15281.46	lbs
ΦVc > Vu	YES	

Design of Heel

footing t =	24	inches
b =	12	inches
d =	20.5	inches

Vu =	12960	lbs
ΦVc =	20210.96	llbs
ΦVc > Vu	YES	

Mu @ face of stem	32400	ft-lb
Mu/(Φbd^2) =	85.66	psi
ρ =	0.0033	(Table A.12)
As =	0.812	in^2
Use #8 @ 9" spacing		
As =	1.05	in^2
d bar =	1	in
Ld =	54.77	inches
L avail =	60	inches
Ld < L avil	YES	
(Ld from back of wall into toe)		

temp. steel = #4 @ 12" spacing

Design of Toe

factored f_{toe} =	-2449.49	psf
factored f_{heel} =	-1414.51	psf
length of toe =	3.5	ft
f @ wall =	2087.25	psf
Vu1 =	4286.60	lbs.
Vu2 =	3652.68	lbs.
Vu =	7939.28	lbs.
Mu @ wall =	14263.53	ft-lbs
Mu/(Φbd^2) =	37.71	psi
ρ =	0.0033	(Table A.12)
As =	0.8118	in^2
Use #8 @ 9" spacing		
As =	1.05	in^2

Retaining Wall Design

	= Assumed Data
	= Check Value
	= Input Information

Retaining Wall Type C-5

Basic Wall Data

f_c'	3000	psi
fy	60000	psi
qa	3000	psf

wall height above ground	14	feet
wall height below ground	3	feet
Total Wall Height	17	feet

Surcharge loads:		
traffic	300	psf
buildings	0	psf
total surcharge	300	psf

Soil Estimates:		
unit weight	120	psf
phi angle	32	degrees
Ka	0.29	
Kp	3.46	

Horizontal Pressures:			
pa	589.84	lb/ft	
Ha	5013.62	lbs.	*(H 1)*
Hp	1867.63	lbs.	

Surcharge Load Pressure:			
equiv. height	2.50	feet	
equiv total hgt	19.50	feet	
equiv pressure	86.74	lb/ft	
Surcharge Force	1474.59	lbs.	*(H 2)*

Assumed Wall Dimensions:		
wall thickness	1.19	ft
actual	1.5	ft
Footing thickness	1.19	ft
actual	2	ft
height of stem	15	ft
width of footing	4.44	ft
actual	12	ft
x	7	ft

Factor of Safety against Overturning

Overturning Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
H1	5013.62	5.67	28410.50	
H2	1474.59	8.5	12534.05	
		Total	40944.55	lb-ft

W	2340
x	2.96

Righting Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
W1	3600.00	6.00	21600.00	
W2	3375.00	5.75	19406.25	
W3	9900.00	9.25	91575.00	
Rv =	16875.00	M =	132581.25	lb-ft

Safety Factor Against Overturning =	3.24	>	2

Factor of Safety against Sliding

Sliding Forces	6488.21	lbs
Resisting Forces	8437.5	lbs

Safety Factor for Sliding =	1.30	>	1.5

Footing Soil Pressure

Rv =	16875.00	located ẍ from the toe of the footing
ẍ =	5.43	feet
A =	12	ft^2
I =	144.00	ft^4
f_{toe} =	-1806.8042	psf
f_{heel} =	-1005.70	psf

Design of Stem for Moment

**lateral forces applied to stem are calculated using a load factor of 1.6*

soil load pressure	832.71	lb/ft
H1	6245.34	lbs.
surcharge pressure	138.79	lb/ft
H2	2081.78	lbs.

Mu =	46840.03	ft-lb

Concrete Design

ρ =	0.0090		
Mu/(Φbd^2) =	482.6	psi	(from Table A.12)
b =	12	inches	
d =	10.38	inches	
h =	12.88	inches	
Actual h =	18	inches	
Actual d =	15.5	inches	

Mu/(Φbd^2) =	216.63	psi	
ρ =	0.0038	(min)	(from Table A.12)
As =	0.7068	in^2	
Use #8 @ 12" spacing			
Actual As =	0.79	in^2	
Actual ρ =	0.0042473		
min. ρ (ACI Section 14.3) =			0.0015
min. Horizontal As =	0.54	in^2	
say, 1/3 inside face =	0.18	in^2	
say, 2/3 outside face =	0.36	in^2	
Inside Face: Use #4 @ 12" spacing			
Actual As =	0.2	in^2	
Outside Face: Use #5 @ 9" spacing			
Actual As =	0.41	in^2	

Design of Stem for Shear

Vu =	8327.12	lbs.
ΦVc =	15281.46	lbs
ΦVc > Vu	YES	

Design of Heel

footing t =	24	inches
b =	12	inches
d =	20.5	inches

Vu =	17424	lbs
ΦVc =	20210.96	llbs
ΦVc > Vu	YES	

Mu @ face of stem	47916	ft-lb
Mu/(Φbd^2) =	126.69	psi
ρ =	0.0033	(Table A.12)
As =	0.812	in^2
Use #8 @ 9" spacing		
As =	1.05	in^2
d bar =	1	in
Ld =	54.77	inches
L avail =	78	inches
Ld < L avil	YES	
(Ld from back of wall into toe)		

temp. steel = #4 @ 12" spacing

Design of Toe

factored f_{toe} =	-2890.89	psf
factored f_{heel} =	-1609.11	psf
length of toe =	5	ft
f @ wall =	2356.81	psf
Vu1 =	7227.22	lbs.
Vu2 =	5892.04	lbs.
Vu =	13119.25	lbs.
Mu @ wall =	33910.78	ft-lbs
Mu/(Φbd^2) =	89.66	psi
ρ =	0.0033	(Table A.12)
As =	0.8118	in^2
Use #8 @ 9" spacing		
As =	1.05	in^2

Retaining Wall Design

	= Assumed Data
	= Check Value
	= Input Information

Retaining Wall Type C-6

Basic Wall Data

f_c'	3000	psi
fy	60000	psi
qa	3000	psf

wall height above ground	17	feet
wall height below ground	3	feet
Total Wall Height	20	feet

Surcharge loads:		
traffic	300	psf
buildings	0	psf
total surcharge	300	psf

Soil Estimates:		
unit weight	120	psf
phi angle	32	degrees
Ka	0.29	
Kp	3.46	

Horizontal Pressures:			
pa	693.93	lb/ft	
Ha	6939.26	lbs.	*(H 1)*
Hp	1867.63	lbs.	

Surcharge Load Pressure:			
equiv. height	2.50	feet	
equiv total hgt	22.50	feet	
equiv pressure	86.74	lb/ft	
Surcharge Force	1734.82	lbs.	*(H 2)*

Assumed Wall Dimensions:		
wall thickness	1.4	ft
actual	1.75	ft
Footing thickness	1.4	ft
actual	2	ft
height of stem	18	ft
width of footing	5.15	ft
actual	14	ft
x	8	ft

Factor of Safety against Overturning

Overturning Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
H1	6939.26	6.67	46261.76	
H2	1734.82	10	17348.16	
		Total	63609.92	lb-ft

W	2700
x	3.43

Righting Moment				
Pressure	Force (lbs.)	Moment Arm (ft)	Moment (lb-ft)	
W1	4200.00	7.00	29400.00	
W2	4725.00	6.875	32484.38	
W3	13500.00	10.875	146812.50	
Rv =	22425.00	M =	208696.88	lb-ft

Safety Factor Against Overturning =	3.28	>	2

Factor of Safety against Sliding

Sliding Forces	8674.08	lbs
Resisting Forces	11212.5	lbs

Safety Factor for Sliding =	1.29	>	1.5

Footing Soil Pressure

Rv =	22425.00	located ẍ from the toe of the footing
ẍ =	6.47	feet
A =	14	ft^2
I =	228.67	ft^4
f_{toe} =	-1965.7055	psf
fheel =	-1237.87	psf

Design of Stem for Moment

**lateral forces applied to stem are calculated using a load factor of 1.6*

soil load pressure	999.25	lb/ft
H1	8993.29	lbs.
surcharge pressure	138.79	lb/ft
H2	2498.14	lbs.

Mu =	76442.93	ft-lb

Concrete Design

ρ =	0.0090		
Mu/(Φbd^2) =	482.6	psi	(from Table A.12)
b =	12	inches	
d =	13.27	inches	
h =	15.77	inches	
Actual h =	21	inches	
Actual d =	18.5	inches	

Mu/(Φbd^2) =	248.17	psi	
ρ =	0.0044	(min)	(from Table A.12)
As =	0.9768	in^2	
Use #8 @ 9" spacing			
Actual As =	1.053	in^2	
Actual ρ =	0.0047432		
min. ρ (ACI Section 14.3) =			0.0015
min. Horizontal As =	0.63	in^2	
say, 1/3 inside face =	0.21	in^2	
say, 2/3 outside face =	0.42	in^2	
Inside Face: Use #5 @ 12" spacing			
Actual As =	0.31	in^2	
Outside Face: Use #6 @ 12" spacing			
Actual As =	0.44	in^2	

Design of Stem for Shear

Vu =	11491.42	lbs.
ΦVc =	18239.16	lbs
ΦVc > Vu	YES	

Design of Heel

footing t =	24	inches
b =	12	inches
d =	20.5	inches

Vu =	22500	lbs
ΦVc =	20210.96	llbs
ΦVc > Vu	NO	

Mu @ face of stem	70312.5	ft-lb
Mu/(Φbd^2) =	185.90	psi
ρ =	0.0033	(Table A.12)
As =	0.812	in^2
Use #8 @ 9" spacing		
As =	1.05	in^2
d bar =	1	in
Ld =	54.77	inches
L avail =	93	inches
Ld < L avil	YES	
(Ld from back of wall into toe)		

temp. steel = #4 @ 12" spacing

Design of Toe

factored f_{toe} =	-3145.13	psf
factored fheel =	-1980.59	psf
length of toe =	6	ft
f @ wall =	2646.04	psf
Vu1 =	9435.39	lbs.
Vu2 =	7938.12	lbs.
Vu =	17373.50	lbs.
Mu @ wall =	53617.78	ft-lbs
Mu/(Φbd^2) =	141.76	psi
ρ =	0.0033	(Table A.12)
As =	0.8118	in^2
Use #8 @ 9" spacing		
As =	1.05	in^2

DESIGN WORK SHEET

US Army Corps of Engineers®

Project: MAMARONECK SHELDRAKE

Subject: SHEET PILE RETAINING WALLS

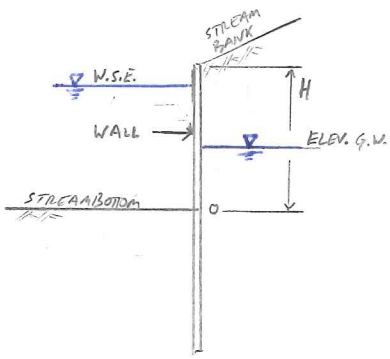
Page No. 1

Sheet No.

Office: Designed by: PUMO Date: 20 JULY 2015 Checked by: Date:

MAM-SHEL WALLS

↳ DESIGN CASES



DESIGN CASE	WSE*	ELEV. G.W.*
1- NO FLOW	Ø	Ø
2- NO FLOW, HIGH G.W.	Ø	½·H
3- FULL BANK	H	H
4- FULL BANK, LOW G.W	H	½·H

* ALL ELEVATIONS RELATIVE TO STREAM BOTTOM

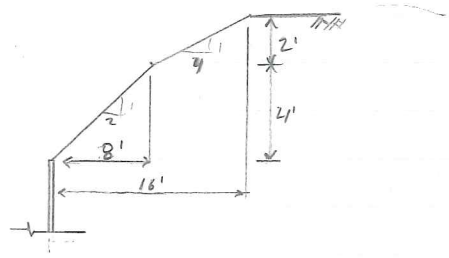
** EXPECT CASE 2 TO CONTROL AT ALL TIMES

ASSUME STREAMBANK SLOPES UP AWAY FROM WALL AS SHOWN AT RIGHT:

UP AT 1:2 FROM WALL TO 8' FROM WALL;

UP AT 1:4 FROM THENCE TO 16' FROM WALL;

LEVELS OUT THENCE.



[NEAR WORST CASE SCENARIO; NOTED AT HARBOR HEIGHTS STATION 14+00; WILL DESIGN WALLS FOR MORE SEVERE SLOPES LATER, IF NECESSARY.]

ADDITIONAL ASSUMPTIONS + CAVEATS

- SEEPAGE WAS NOT CONSIDERED IN ANY CASE
- ALL SOIL ASSUMED TO BE HOMOGENEOUS WITH THE FOLLOWING PROPERTIES:
  - $\gamma_{SAT} = 122$ pcf $\phi = 32°$
  - $\gamma_{moist} = 120$ pcf cohesion, $c = 50$ psf
  - → BASED ON PRELIM. GEOTECH
- DEEP-SEATED FAILURE WAS NOT ASSESSED
- FOR ALL CASES, FACTOR OF SAFETY = 1.0 FOR ACTIVE + 1.5 FOR PASSIVE SOIL PRESSURES
- ALL STREAMBANKS ASSUMED TO BE SUBJECT TO DISTRIBUTED LOADING = 300 PSF (ADDITIONAL CONSIDERATION MUST BE MADE LATER FOR WALLS NEAR BUILDINGS]

★ → ALL DESIGNS DONE USING THE SOFTWARE CWALSHT

DESIGN WORK SHEET

US Army Corps of Engineers®

Project: MAMARONECK SHELDRAKE

Subject: SHEET PILE RETAINING WALLS

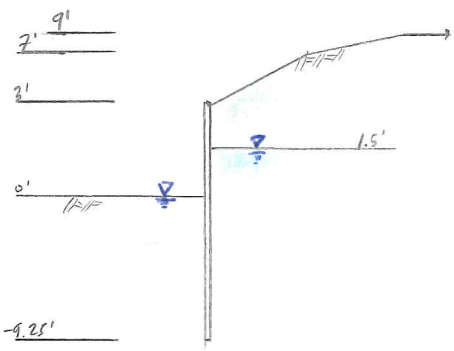
Page No. 2

Sheet No.

Office: Designed by: PUMO Date: JULY 2015 Checked by: Date:

WALL TYPE 1 – EXPOSED: HEIGHT UP TO 3'

SEE, FOR EXAMPLE, CONDITIONS AT HARBOR HEIGHTS STATION 6+00



DESIGN CASE	WSE	GW ELEV	(ft-lb/ft) M_{max}	(lb/ft) V_{max}	(lb-in³) Δ_{max} scaled	(ft) min. EMBED	(ft) min. LENGTH
1	0'	0'	1,923	-1195	1.147 E8	7.51	10.5
2	0'	1.5'	2,981	-1538	2.360 E8	9.16	12.25 ← CONTROLS
3	3'	3'	1,164	-868	5.227 E8	6.05	9.0
4	3'	1.5'	398.5	-430	9.565 E6	3.71	6.75

NOW, DETERMINE SHEET PILE SIZE (ASSUME GRADE 50 STEEL PILES)

TYPE 1 - CASE 2

DET. min section mod → $S_{min} = M_{max}/f_b$

- $f_b = f_y \times 0.5 = 50{,}000\ psi \times 0.5 = 25{,}000\ psi$
- $M_{max} = 2{,}981$ ft-lb/ft pile

↳ $S_{min} = \dfrac{2{,}981\ ft\text{-}lb/ft\ pile \times 12\ in/ft}{25{,}000\ lb/in^2} = 1.43\ in^3/ft\ wall$

Det. min Area for shear → $A_{v,min} = V_{max}/f_v$

- $f_v = f_y \times 0.33 = 50{,}000 \times 0.33 = 16{,}500\ psi$
- $V_{max} = 2{,}981$ lb/ft wall

↳ $A_{v,min} = \dfrac{2{,}981\ lb/ft\ wall}{16{,}500\ lb/in^2} = 0.181\ in^2/ft\ wall$

Det min I for Δ → $\Delta_{max} = \dfrac{\Delta_{scaled}}{EI} = 3\ in$

↳ $I_{min} = \dfrac{\Delta_{scaled}}{\Delta_{max} \cdot E} = \dfrac{5.227 \times 10^8\ lb\text{-}in^3}{3\ in \times 2.9 \times 10^7\ lb/in^2} = 6.01\ in^4$

⇒ TRY SCZ 14 ($S = 14.36\ in^3/ft$, $I = 71.82\ in^4$)

$A_v = t_w \times h/w = 0.250'' \times 10''/23.5'' \times 12''/ft\ wall$

$= 1.05\ in^2/ft\ pile > 0.181$ ∴ OK

∴ USE SCZ 14 PILES FOR WALL TYPE 1; min embed = 9.16', LENGTH = 12.25'

DESIGN WORK SHEET

US Army Corps of Engineers®

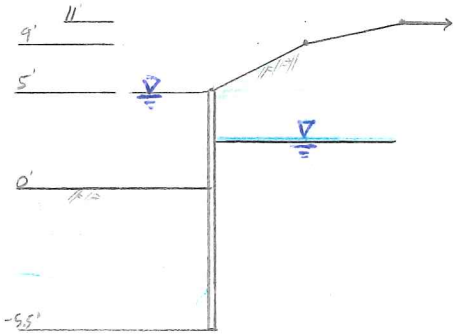
Project Mamaroneck Shoreline **Page No.** 3

Subject Sheet Pile Retaining Walls (Wall Type 2) **Sheet No.**

Office **Designed by** Pumo **Date** 20 July 2015 **Checked by** **Date**

WALL TYPE 2 – EXPOSED HEIGHT UP TO 5'

SEE, FOR EXAMPLE, CONDITIONS AT HARBOR HEIGHTS STATION 14+00



TYPE 2 – CASE 4

DESIGN CASE	WSE	OW ELEV	(ft-lb/ft) M_{max}	(lb/ft) V_{max}	(lb-in³) Δ_{max}^{scaled}	(ft) min EMBED.	(ft) min Length	
1	0'	0'	7923	-2950	1.233E9	12.19	17.5	
2	0	2.5'	12,560	-3766	2.618E9	14.96	20.0	← CONTROLS
3	5	5	4,561	-1974	5.137E8	9.67	15.0	
4	5	2.5'	1,132	-859	6.358E7	5.42	10.5	

NOW, DET. MINIMUM SIZE SHEET PILES

ASSUME GRADE 50 STEEL PILES

DET. min section mod. → $S_{min} = M_{max}/f_b$

- $f_b = f_y \times 0.5 = 50{,}000\ psi \times 0.5 = 25{,}000\ psi$
- $M_{max} = 12{,}560\ ft\text{-}lb/ft\ pile$

→ $S_{min} = \dfrac{12{,}560\ ft\text{-}lb/ft\ pile}{25{,}000\ lb/in^2} \times 12\ in/ft = 6.029\ in^3/ft\ wall$

Det. min Area for shear → $A_{v,min} = V_{max}/f_v$

- $f_v = f_y \times 0.33 = 50{,}000\ psi \times 0.33 = 16{,}500\ psi$
- $V_{max} = 3766\ lb/ft\ pile$

→ $A_{v,min} = \dfrac{3766\ lb/ft\ pile}{16{,}500\ lb/in^2} = 0.2282\ in^2/ft\ wall$

Det. min I for Δ → $\Delta_{max} = \dfrac{\Delta_{scaled}}{EI} = 3\ in.$

→ $I_{min} = \dfrac{\Delta_{scaled}}{\Delta_{max} \cdot E} = \dfrac{2.618 \times 10^9\ lb\text{-}in^3}{3\ in \times 2.9 \times 10^7\ lb/in^2} = 30.09\ in^4$

⇒ TRY SCZ 14 PILES ($S = 14.36\ in^3/ft > 6.029$; $I = 71.82\ in^4 > 30.09$)

$A_v = t_w \times h/w = 0.250" \times 10"/28.5" \times 12"/ft\ wall$

$= 1.05\ in^2/ft\ wall > 0.23$ ∴ OK

∴ USE SCZ14 PILES FOR WALL TYPE 2; min EMBED = 15', Length = 20'

DESIGN WORK SHEET

US Army Corps of Engineers®

Project: MAMARONECK SHELDRAKE

Subject: SHEET PILE RETAINING WALLS

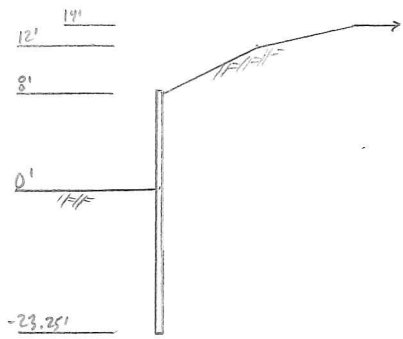
Page No. 4

Sheet No.

Office: Designed by: Pumo Date: JULY 2015 Checked by: Date:

WALL TYPE 3 – EXPOSED HEIGHT UP TO 8'

SEE, FOR EXAMPLE, CONDITIONS AT



DESIGN CASE	WDE	G.W. ELEV.	(ft-lb/ft) M_{max}	(lb/ft) V_{max}	(lb-in³) Δ_{max}	(ft) min. EMBED	(ft) min. LENGTH
1	0	0	28,330	-6623	1.052E10	18.73	
2	0	4	45,860	-8721	2.287E10	23.11	31.25' ← CONTROLS
3	8	8	15,370	-4394	4.035E9	14.60	22.75
4	8	4	2,638	-1463	3.100E8	7.21	

NOW, DET. MINIMUM SIZE SHEET PILES

ASSUME GRADE 50 STEEL PILES.

Det min. section mod. → $S_{min} = M_{max}/f_b$

- $f_b = f_y \times 0.5 = 50{,}000\ psi \times 0.5 = 25{,}000\ psi$
- $M_{max} = 45{,}860$ ft-lb/ft wall

↳ $S_{min} = \dfrac{45{,}860\ ft\text{-}lb/ft\ pile \times 12''/ft}{25{,}000\ psi} = 22.01\ in^3/ft\ pile$

Det. min A for shear → $A_{v,min} = V_{max}/f_v$

- $f_v = f_y \times 0.33 = 50{,}000\ psi \times 0.33 = 16{,}500\ psi$
- $V_{max} = 8721$ lb/ft pile

↳ $A_{v,min} = \dfrac{8721\ lb/ft\ pile}{16500\ psi} = 0.5285\ in^2/ft\ pile$

Det min I for Δ → $\Delta_{max} = \dfrac{\Delta_{scaled}}{EI} = 3''$

↳ $I_{min} = \dfrac{\Delta_{scaled}}{\Delta_{max} \cdot E} = \dfrac{2.287E10\ lb\text{-}in^3}{3\ in \times 2.9E7\ lb/in^2} = 262.9\ in^4$

⇒ TRY SKZ 22 PILES ($S = 33.43\ in^3/ft$; $I = 267.4\ in^4$)

$A_v = t_w \times h/w = 0.335'' \times 16''/28.5'' \times 12''/ft$

$= 2.257\ in^2/ft > 0.5285$ ∴ OK

∴ Use SKZ 22 PILES FOR WALL TYPE 3; MIN. EMBED = 23.11', LENGTH = 31.25'

DESIGN WORK SHEET

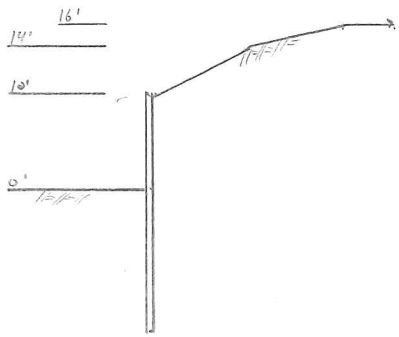
US Army Corps of Engineers®

Project: Mamaroneck Shelorane
Subject: Sheet Pile Retaining Walls
Page No. 5
Sheet No.

Office: Designed by: Pumo Date: July 2015 Checked by: Date:

WALL TYPE 4 – EXPOSED HEIGHT UP TO 10'

SEE, FOR EXAMPLE, CONDITIONS AT



DESIGN CASE	WSE	GW ELEV	M_{MAX}	V_{MAX}	Δ_{max}^{scaled}	min. EMBED	min. LENGTH
1	0	0	51,730	9764	2.880 e10	22.36	33.0'
2	0	5	84,410	13,350	6.341 e10	28.31	38.5' ← CONTROLS
3	10	10	27,250	6,008	1.063 e10	17.62	27.75'
4	10	5	3,800	1,827	6.079 e8	8.06	18.0'

NOW, DET. SIZE OF SHEET PILE

ASSUME GRADE 50 STEEL PILES

$S_{min} = M_{max} / f_b$

- $f_b = f_y \times 0.5 = 50{,}000 \text{ psi} \times 0.5 = 25{,}000 \text{ psi}$
- $M_{MAX} = 84{,}410 \text{ ft-lb/ft pile}$

$\hookrightarrow S_{min} = \dfrac{84{,}410 \text{ ft-lb/ft pile} \times 12''/\text{ft}}{25000 \text{ psi}} = 40.52 \text{ in}^3/\text{ft pile}$

$A_{v_{min}} = V_{MAX} / f_v$

- $f_v = f_y \times 0.33 = 50{,}000 \text{ psi} \times 0.33 = 16{,}500 \text{ psi}$
- $V_{MAX} = 13{,}350 \text{ lb/ft pile}$

$\hookrightarrow A_{v_{min}} = \dfrac{13{,}350 \text{ lb/ft pile}}{16{,}500 \text{ psi}} = 0.809 \text{ in}^2/\text{ft pile}$

$\Delta_{MAX} = \dfrac{\Delta_{SCALED}}{EI} = 3''$

$\hookrightarrow I_{min} = \dfrac{\Delta_{SCALED}}{\Delta_{MAX} \cdot E} = \dfrac{6.341 \times 10^{10} \text{ lb-in}^3}{3'' \times 2.7 \times 10^7 \text{ lb/in}^2} = 782.8 \text{ in}^4$

$\hookrightarrow$ TOO MUCH

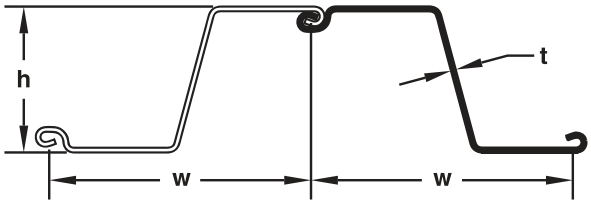
NO TYPICAL SHEET PILE CAN HANDLE THIS Δ (I of PZ40 = 491 in^4).

A SHEET PILE WALL OF THIS HEIGHT COULD FUNCTION IF DESIGNED WITH AN ANCHORING SYSTEM.

HOWEVER, BECAUSE THE CURRENT DESIGN OBJECTIVES ARE TO MINIMIZE ADDITION OF ELEMENTS AND, AS ALWAYS, TO MINIMIZE THE TOTAL COST, I RECOMMEND AGAINST USING SHEET PILE FOR TYPES 4, 5, + 6.

SCZ/SKZ

SCZ/SKZ Cold Formed Steel Sheet Pile



SECTION	Width (w) in (mm)	Height (h) in (mm)	Thickness (t) in (mm)	Cross Sectional Area in²/ft (cm²/m)	WEIGHT Pile lb/ft (kg/m)	WEIGHT Wall lb/ft² (kg/m²)	SECTION MODULUS Elastic in³/ft (cm³/m)	SECTION MODULUS Plastic in³/ft (cm³/m)	Moment of Inertia in⁴/ft (cm⁴/m)	COATING AREA Both Sides ft²/ft (m²/m)	COATING AREA Coating Area ft²/ft² (m²/m²)
SCZ 14	28.50 (723.9)	10.00 (254.0)	0.250 (6.4)	4.18 (88.48)	33.81 (50.31)	14.23 (69.50)	14.36 (772)	16.32 (877.4)	71.82 (9808)	6.10 (1.86)	1.28 (1.28)
SCZ 16	28.50 (723.9)	10.00 (254.0)	0.276 (7.0)	4.62 (97.79)	37.37 (55.61)	15.73 (76.82)	15.75 (847)	17.97 (965.9)	78.73 (10751)	6.10 (1.86)	1.28 (1.28)
SCZ 17N	28.50 (723.9)	10.00 (254.0)	0.295 (7.5)	4.95 (104.78)	40.03 (59.58)	16.86 (82.32)	16.87 (907)	19.21 (1033)	84.35 (11519)	6.10 (1.86)	1.28 (1.28)
SCZ 18N	28.50 (723.9)	10.00 (254.0)	0.317 (8.1)	5.31 (112.39)	42.94 (63.91)	18.08 (88.28)	18.10 (973)	20.61 (1108)	90.48 (12356)	6.10 (1.86)	1.28 (1.28)
SCZ 21N	28.50 (723.9)	10.00 (254.0)	0.375 (9.5)	6.29 (133.06)	50.84 (75.66)	21.41 (104.54)	21.43 (1152)	24.40 (1312)	107.13 (14629)	6.10 (1.86)	1.28 (1.28)
SKZ 20	28.50 (723.9)	16.00 (406.4)	0.315 (8.0)	6.00 (127.00)	48.24 (71.79)	20.31 (99.17)	31.69 (1704)	36.66 (1970.97)	253.51 (34618)	7.60 (2.32)	1.60 (1.60)
SKZ 22	28.50 (723.9)	16.00 (406.4)	0.335 (8.5)	6.30 (133.35)	51.30 (76.34)	21.60 (105.46)	33.43 (1797)	38.94 (2093.55)	267.40 (36515)	7.60 (2.32)	1.60 (1.60)
SKZ 23	28.50 (723.9)	16.00 (406.4)	0.354 (9.0)	6.70 (141.82)	54.20 (80.66)	22.82 (111.42)	35.61 (1915)	41.12 (2210.75)	284.90 (38905)	7.60 (2.32)	1.60 (1.60)
SKZ 24	28.50 (723.9)	16.00 (406.4)	0.375 (9.5)	7.10 (150.28)	57.43 (85.47)	24.18 (118.06)	37.73 (2028)	43.52 (2339.78)	301.80 (41213)	7.60 (2.32)	1.60 (1.60)
SKZ 25	28.50 (723.9)	16.00 (406.4)	0.399 (10.1)	7.60 (160.87)	61.10 (90.93)	25.73 (125.61)	40.14 (2158)	46.24 (2486.02)	321.12 (43851)	7.60 (2.32)	1.60 (1.60)
SKZ 31	28.50 (723.9)	18.00 (457.2)	0.450 (11.4)	9.07 (192.04)	73.82 (109.85)	31.08 (151.75)	51.56 (2772)	60.51 (3253.29)	464.05 (63369)	8.06 (2.46)	1.70 (1.70)
SKZ 33	28.50 (723.9)	18.00 (457.2)	0.475 (12.1)	9.40 (198.97)	77.64 (115.54)	32.69 (159.61)	54.89 (2951)	63.57 (3417.68)	494.03 (67462)	8.06 (2.46)	1.70 (1.70)
SKZ 34	28.50 (723.9)	18.00 (457.2)	0.500 (12.7)	9.89 (209.25)	81.42 (121.17)	34.28 (167.38)	57.62 (3098)	66.86 (3594.60)	518.62 (70821)	8.06 (2.46)	1.70 (1.70)
SKZ 36	28.50 (723.9)	18.00 (457.2)	0.535 (13.6)	10.78 (228.10)	86.81 (129.19)	36.55 (178.46)	60.71 (3264)	71.58 (3848.17)	546.43 (74619)	8.06 (2.46)	1.70 (1.70)
SKZ 38	28.50 (723.9)	18.00 (457.2)	0.550 (14.0)	11.07 (234.42)	88.95 (132.37)	37.45 (182.85)	62.32 (3350)	73.52 (3952.44)	560.85 (76588)	8.06 (2.46)	1.70 (1.70)

Interlock Compatibility

	SCZ 14	SCZ 16	SCZ 17N	SCZ 18N	SCZ 21N	SKZ 20	SKZ 22	SKZ 23	SKZ 24	SKZ 25	SKZ 31	SKZ 33	SKZ 34	SKZ 36	SKZ 38
SCZ 14	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SCZ 16	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SCZ 17N	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SCZ 18N	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SCZ 21N	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SKZ 20	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SKZ 22	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SKZ 23	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SKZ 24	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SKZ 25	●	●	●	●	●	●	●	●	●	●	○	○	○	○	○
SKZ 31	○	○	○	○	○	○	○	○	○	○	●	●	●	●	●
SKZ 33	○	○	○	○	○	○	○	○	○	○	●	●	●	●	●
SKZ 34	○	○	○	○	○	○	○	○	○	○	●	●	●	●	●
SKZ 36	○	○	○	○	○	○	○	○	○	○	●	●	●	●	●
SKZ 38	○	○	○	○	○	○	○	○	○	○	●	●	●	●	●

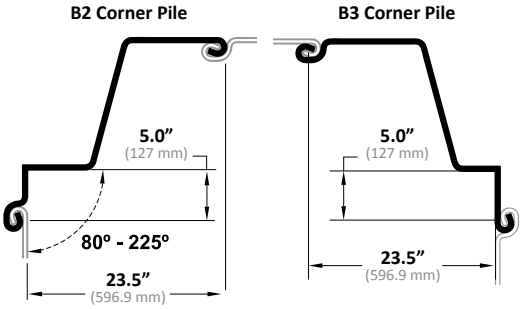
SCZ/SKZ

SCZ/SKZ Cold Formed Steel Sheet Pile

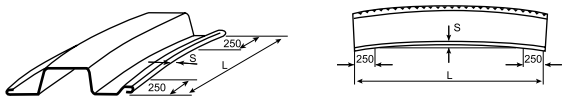
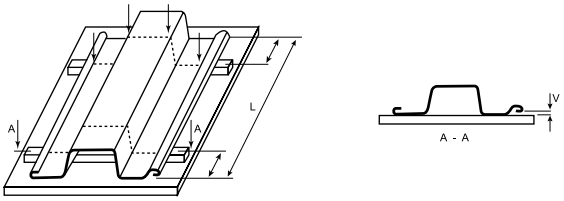
Available Steel Grades					
ASTM	YIELD STRENGTH		ASTM	YIELD STRENGTH	
	(ksi)	(MPa)		(ksi)	(MPa)
A 572 Grade 50	50	345	A 572 Grade 65 (Mod)**	80	555
A 572 Grade 55	55	380	A 588	50	345
A 572 Grade 60	60	415	A 690	50	345
A 572 Grade 65*	65	450			

*Not available for thicknesses ≥ 0.375" (9.525mm). **Not available for thicknesses > 0.276" (7.0mm).

Corner Piles



Delivery Conditions & Tolerances

	ASTM A6		EN 10249-2	
Mass	± 2.5%		± 7%	
Length	+ 5 inches	– 0 inches	± 50 mm	
Straightness				
Bending (S)			0.25% of the length	
Curving (C)			0.25% of the length	
Twisting (V)			2% of the length	

Maximum Rolled Lengths*

SKZ	70 feet	(21.3 m)

* Longer lengths may be possible upon request.

Sheet Pile Wall – Design Case 2 System & Loads

We used the software CWALSHT to design steel sheet pile retaining walls. The system & loads shown below were used for walls of 3', 5', 8', and 10' exposed heights. What follows are the text and graphical design outputs for Design Case 2 – low channel level, high groundwater – which controlled the designs of all wall types.

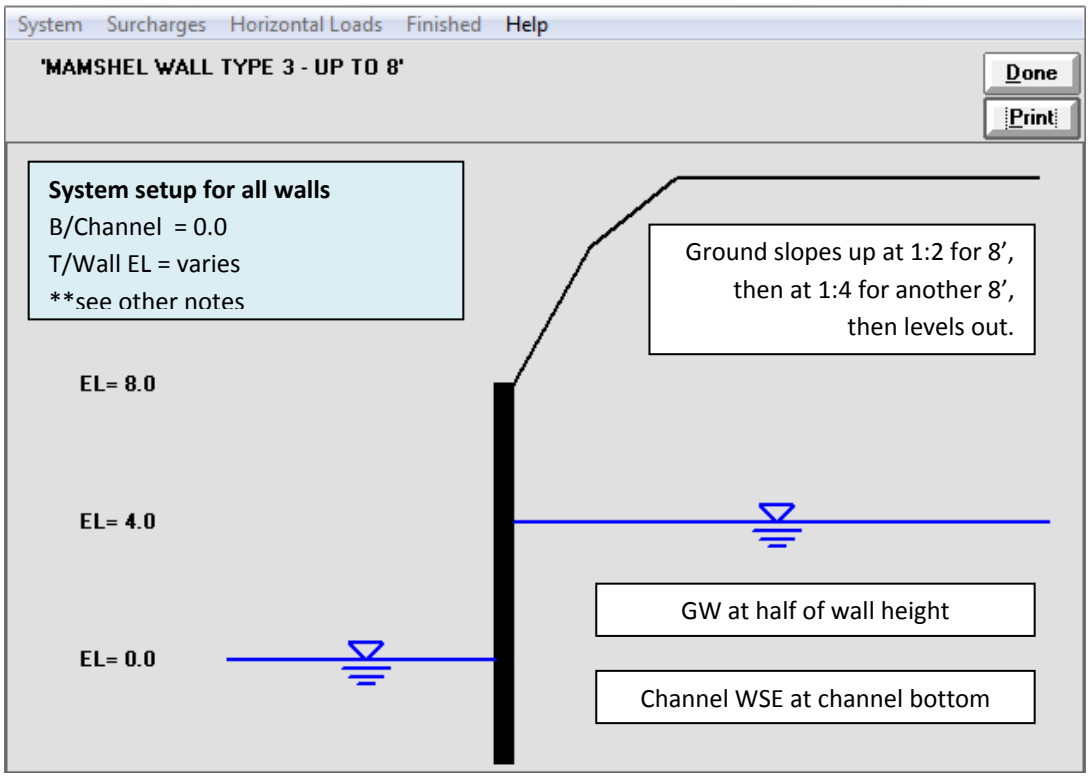


Figure 1 - System Setup

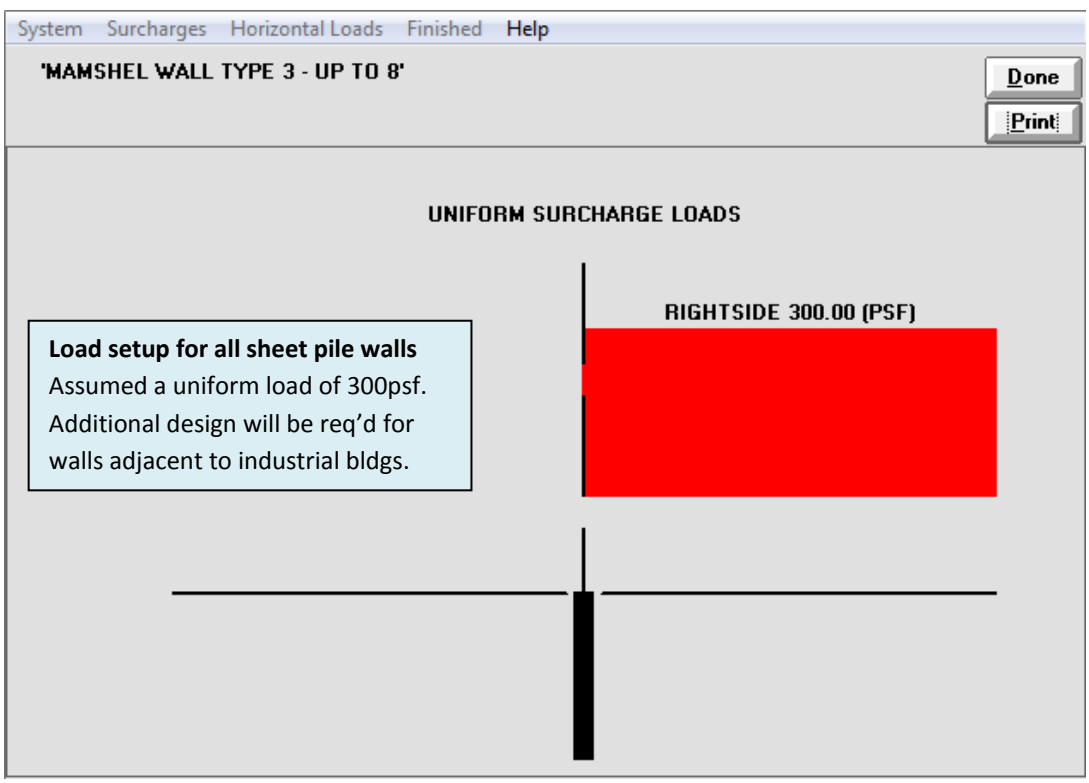


Figure 2 - Loading

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PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 20-JULY-2015                                          TIME: 14:51:55

                            ****************
                            *  INPUT DATA  *
                            ****************

      I.--HEADING
     'MAMSHEL WALL TYPE 1 - Up to 3'
     'SEE HH 6+00 FOR EXAMPLE
     'Case 2

     II.--CONTROL
        CANTILEVER WALL DESIGN
        FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
        FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

    III.--WALL DATA
        ELEVATION AT TOP OF WALL    = 3.00 FT.

     IV.--SURFACE POINT DATA

          IV.A.--RIGHTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00          3.00
                  8.00          7.00
                 16.00          9.00
                 50.00          9.00

          IV.B.--LEFTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00          0.00
                 25.00          0.00

      V.--SOIL LAYER DATA

         V.A.--RIGHTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

         V.B.--LEFTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

     VI.--WATER DATA
        UNIT WEIGHT         = 62.40 (PCF)
        RIGHTSIDE ELEVATION = 1.50 (FT)
        LEFTSIDE ELEVATION  = 0.00 (FT)
```

NO SEEPAGE

VII.--VERTICAL SURCHARGE LOADS

VII.A.--VERTICAL LINE LOADS
NONE

VII.B.--VERTICAL UNIFORM LOADS

LEFTSIDE (PSF)	RIGHTSIDE (PSF)
0.00	300.00

VII.C.--VERTICAL STRIP LOADS
NONE

VII.D.--VERTICAL RAMP LOADS
NONE

VII.E.--VERTICAL TRIANGULAR LOADS
NONE

VII.F.--VERTICAL VARIABLE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 14:52:04

```
*************************
*   SOIL PRESSURES FOR  *
* CANTILEVER WALL DESIGN *
*************************
```

I.--HEADING
'MAMSHEL WALL TYPE 1 - Up to 3'
'SEE HH 6+00 FOR EXAMPLE

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

		<---LEFTSIDE--->		<------NET------> (SOIL + WATER)		<--RIGHTSIDE--->	
ELEV. (FT)	NET WATER (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
3.0	0.0	0.0	0.0	43.8	974.4	43.8	974.4
2.0	0.0	0.0	0.0	105.7	2171.7	105.7	2171.7
1.5	0.0	0.0	0.0	130.4	2429.6	130.4	2429.6
1.0	31.2	0.0	0.0	182.1	2679.1	150.9	2647.9

0.0+	93.6	0.0	0.0	281.8	3138.7	188.2	3045.1
0.0-	93.6	100.0	0.0	181.8	3138.7	188.2	3045.1
-1.0	93.6	234.2	0.0	84.8	3530.7	225.3	3437.1
-1.9	93.6	353.2	0.0	0.0	3652.3	259.6	3558.7
-2.0	93.6	368.3	0.0	-10.7	3667.6	264.0	3574.0
-3.0	93.6	502.4	0.0	-108.0	3712.0	300.8	3618.4
-4.0	93.6	636.5	0.0	-212.6	3928.3	330.3	3834.7
-5.0	93.6	770.6	0.0	-321.3	4156.7	355.7	4063.1
-6.0	93.6	904.7	30.3	-430.3	4293.0	380.9	4229.7
-7.0	93.6	1038.8	71.3	-539.1	4236.4	406.1	4214.1
-8.0	93.6	1172.9	91.1	-647.9	4210.2	431.4	4207.7
-9.0	93.6	1307.1	109.4	-756.8	4302.1	456.7	4317.9
-10.0	93.6	1441.2	127.7	-865.6	4388.5	482.0	4422.6
-11.0	93.6	1575.3	146.0	-974.4	4495.5	507.3	4547.9
-12.0	93.6	1709.4	164.3	-1083.2	4613.4	532.6	4684.1
-13.0	93.6	1843.5	182.6	-1192.6	4712.9	557.3	4801.9
-14.0	93.6	1977.6	200.9	-1307.4	4806.1	576.6	4913.5
-15.0	93.6	2111.7	219.3	-1420.1	4917.1	598.1	5042.8
-16.0	93.6	2245.9	237.6	-1529.9	5033.7	622.4	5177.7
-17.0	93.6	2380.0	255.9	-1644.9	5150.3	641.5	5312.6
-18.0	93.6	2514.1	274.2	-1760.9	5266.9	659.5	5447.5
-19.0	93.6	2648.2	292.5	-1877.0	5383.5	677.6	5582.4
-20.0	93.6	2782.3	310.8	-1993.1	5500.0	695.6	5717.3
-21.0	93.6	2916.4	329.1	-2109.1	5616.6	713.7	5852.1
-22.0	93.6	3050.5	347.4	-2225.2	5722.1	731.7	5975.9
-23.0	93.6	3184.7	365.8	-2341.3	5822.6	749.8	6094.8
-24.0	93.6	3318.8	384.1	-2457.3	5934.0	767.8	6224.5
-25.0	93.6	3452.9	402.4	-2570.6	6049.9	788.7	6358.7
-26.0	93.6	3587.0	420.7	-2681.8	6165.8	811.6	6492.9
-27.0	93.6	3721.1	439.0	-2795.6	6281.7	831.9	6627.1
-28.0	93.6	3855.2	457.3	-2911.5	6397.6	850.1	6761.3
-29.0	93.6	3989.3	475.6	-3027.3	6513.5	868.4	6895.5
-30.0	93.6	4123.5	493.9	-3143.2	6629.4	886.6	7029.7
-31.0	93.6	4257.6	512.3	-3259.1	6745.3	904.9	7163.9

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 14:52:04

```
****************************
*  SUMMARY OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
****************************
```

I.--HEADING
'MAMSHEL WALL TYPE 1 - Up to 3'
'SEE HH 6+00 FOR EXAMPLE

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :       -9.16
      PENETRATION (FT)     :        9.16

MAX. BEND. MOMENT (LB-FT)  :  2.9814E+03
      AT ELEVATION (FT)    :       -5.37

MAX. SCALED DEFL. (LB-IN^3):  2.3602E+08
      AT ELEVATION (FT)    :        3.00
```

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 14:52:04

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****************************
* COMPLETE OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
****************************
```

I.--HEADING
'MAMSHEL WALL TYPE 1 - Up to 3'
'SEE HH 6+00 FOR EXAMPLE

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
3.00	0.0000E+00	0.	2.3602E+08	43.78
2.00	3.2216E+01	75.	2.0495E+08	105.73
1.50	8.3836E+01	134.	1.8943E+08	130.35
1.00	1.6917E+02	212.	1.7395E+08	182.08
0.00+	4.8872E+02	444.	1.4326E+08	281.83
0.00-	4.8872E+02	444.	1.4326E+08	181.84
-1.00	1.0073E+03	577.	1.1346E+08	84.76
-1.89	1.5421E+03	615.	8.8424E+07	0.00
-2.00	1.6109E+03	614.	8.5400E+07	-10.70
-3.00	2.2035E+03	555.	6.0126E+07	-107.98
-4.00	2.6869E+03	395.	3.8645E+07	-212.58
-5.00	2.9571E+03	128.	2.1775E+07	-321.30
-6.00	2.9059E+03	-248.	9.9699E+06	-430.26
-7.00	2.4244E+03	-733.	3.1236E+06	-539.09
-8.00	1.4039E+03	-1326.	3.8920E+05	-647.92
-8.32	9.4913E+02	-1538.	1.2253E+05	-682.51
-9.00	5.3783E+01	-629.	2.1528E+02	3346.84
-9.16	0.0000E+00	0.	0.0000E+00	4316.26

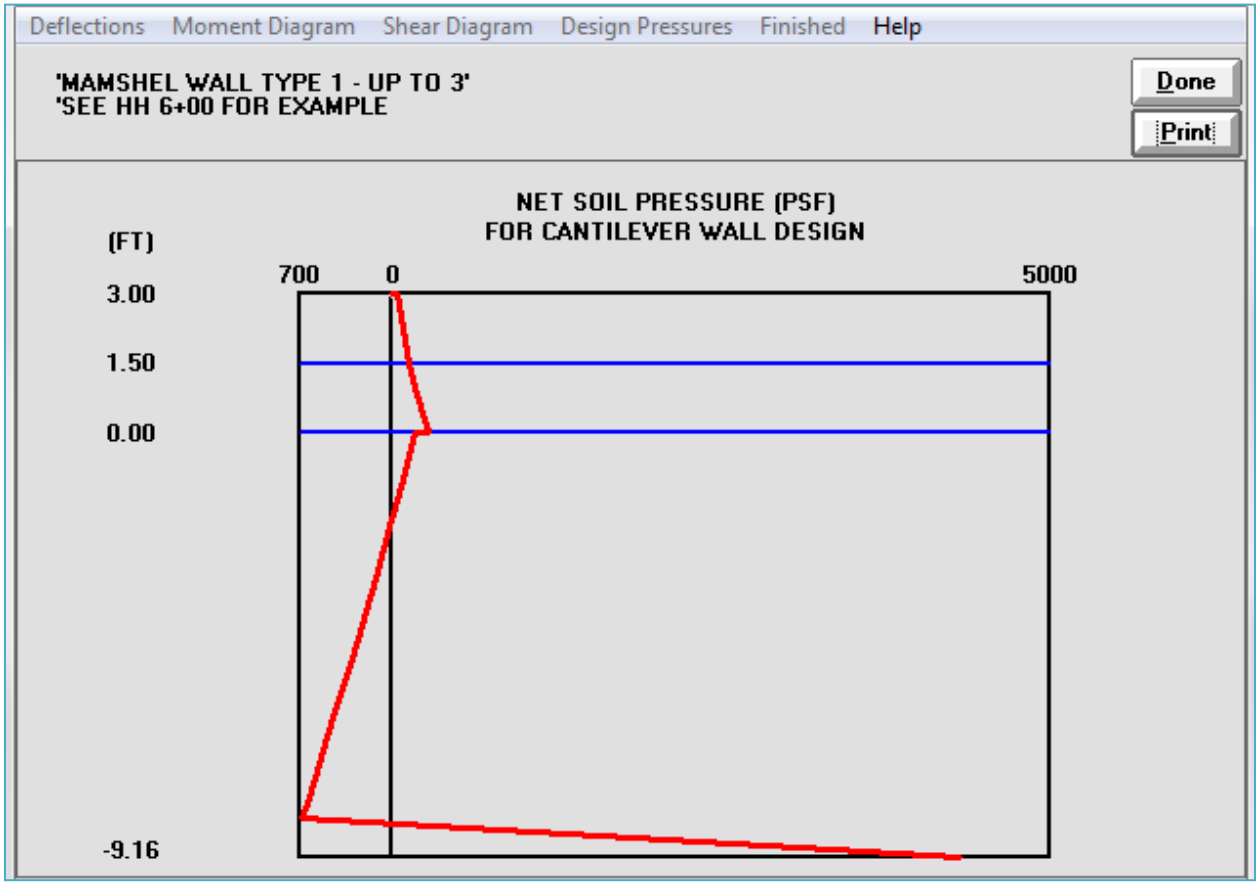
NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

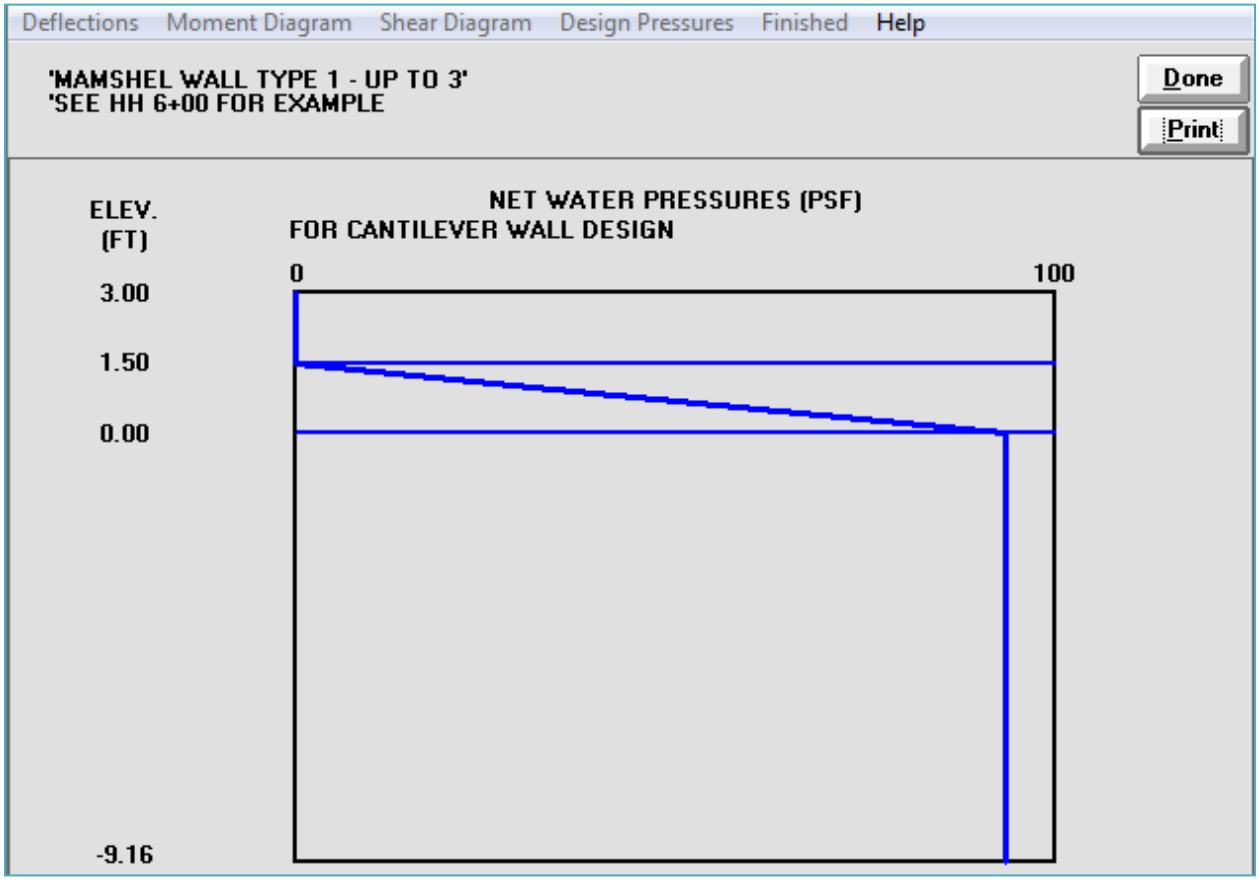
III.--WATER AND SOIL PRESSURES

<----------SOIL PRESSURES---------->

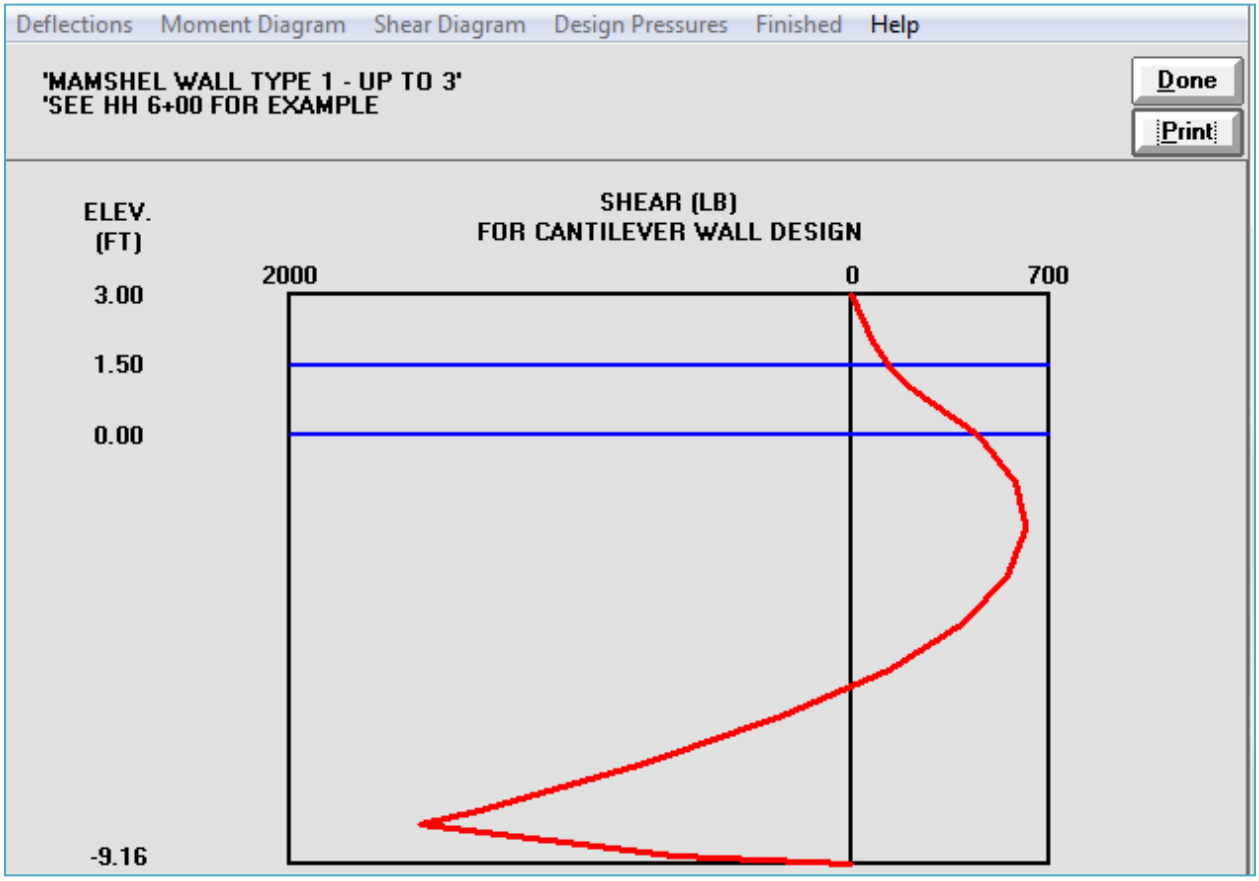
ELEVATION (FT)	WATER PRESSURE (PSF)	<----LEFTSIDE-----> PASSIVE (PSF)	ACTIVE (PSF)	<---RIGHTSIDE----> ACTIVE (PSF)	PASSIVE (PSF)
3.00	0.	0.	0.	44.	974.
2.00	0.	0.	0.	106.	2172.
1.50	0.	0.	0.	130.	2430.
1.00	31.	0.	0.	151.	2648.
0.00+	94.	0.	0.	188.	3045.
0.00-	94.	100.	0.	188.	3045.
-1.00	94.	234.	0.	225.	3437.
-1.89	94.	353.	0.	260.	3559.
-2.00	94.	368.	0.	264.	3574.
-3.00	94.	502.	0.	301.	3618.
-4.00	94.	636.	0.	330.	3835.
-5.00	94.	771.	0.	356.	4063.
-6.00	94.	905.	30.	381.	4230.
-7.00	94.	1039.	71.	406.	4214.
-8.00	94.	1173.	91.	431.	4208.
-8.32	94.	1216.	97.	439.	4243.
-9.00	94.	1307.	109.	457.	4318.
-9.16	94.	1441.	128.	482.	4423.
-11.00	94.	1575.	146.	507.	4548.

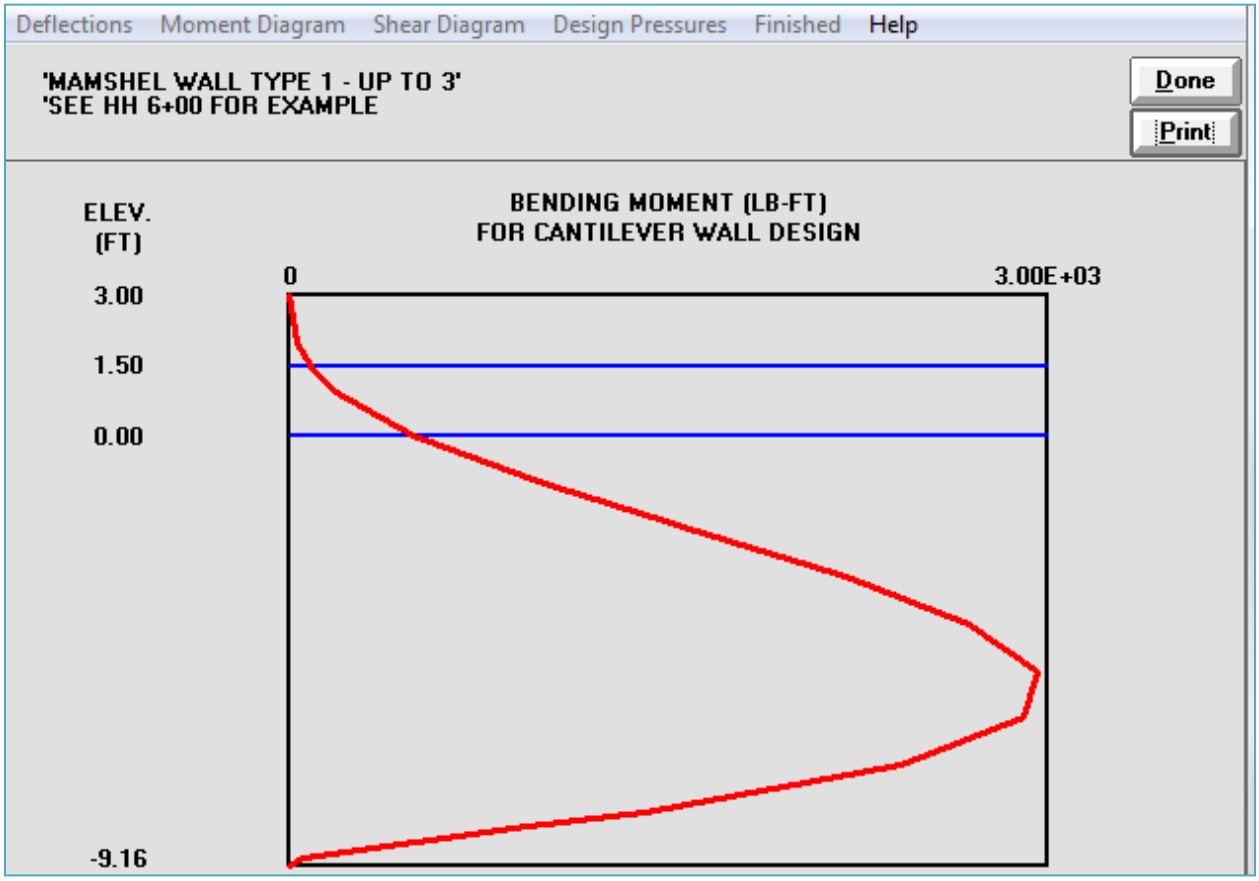
Sheet Pile Wall Type 1 (up to 3') – Design Case 2



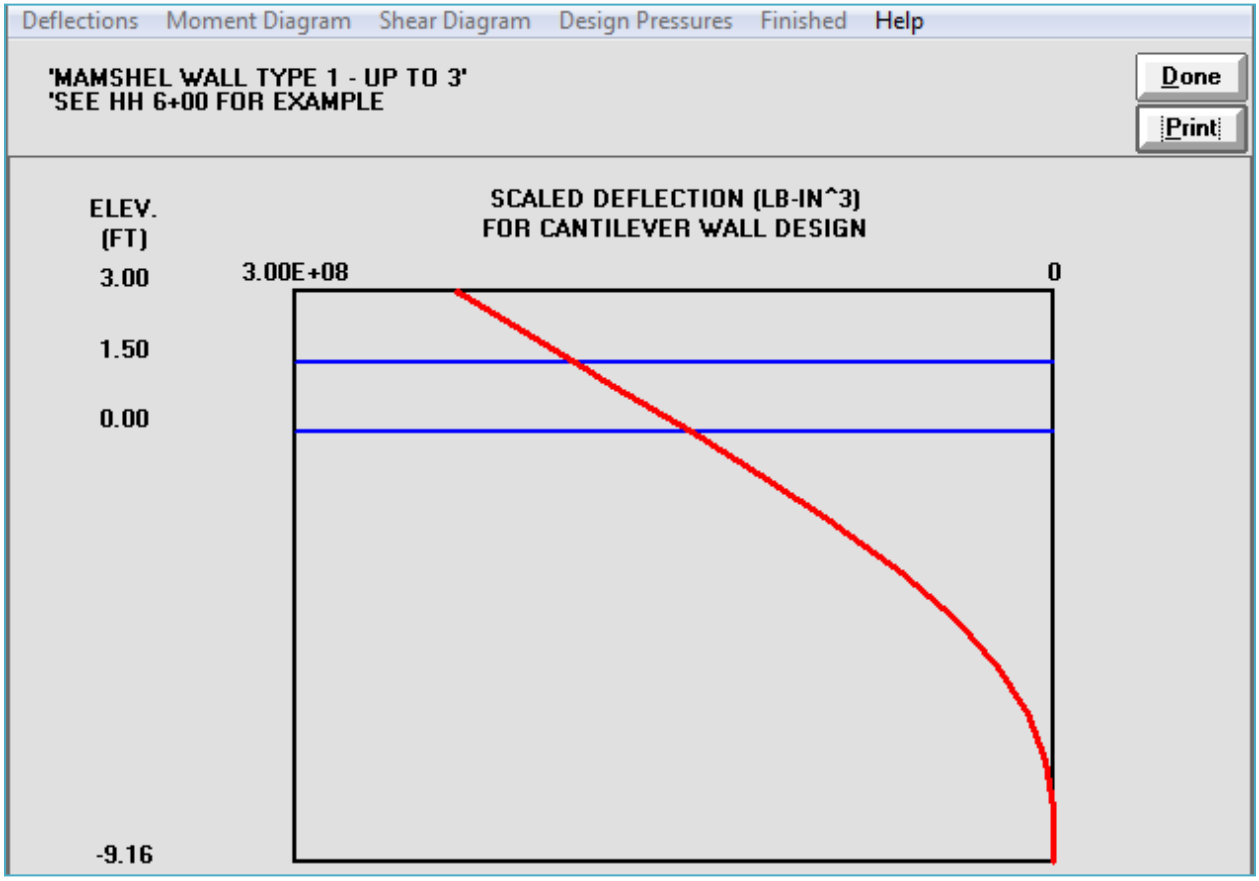


Sheet Pile Wall Type 1 (up to 3') – Design Case 2





Sheet Pile Wall Type 1 (up to 3') – Design Case 2



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PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 20-JULY-2015                                          TIME: 14:15:53

                            ****************
                            *  INPUT DATA  *
                            ****************

      I.--HEADING
     'MAMSHEL WALL TYPE 2 - Up to 5'
     'SEE HH 14+00 FOR EXAMPLE
     'Case 2

     II.--CONTROL
        CANTILEVER WALL DESIGN
        FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
        FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

    III.--WALL DATA
        ELEVATION AT TOP OF WALL    = 5.00 FT.

     IV.--SURFACE POINT DATA

          IV.A.--RIGHTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00          5.00
                  8.00          9.00
                 16.00         11.00
                 50.00         11.00

          IV.B.--LEFTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00          0.00
                 25.00          0.00

      V.--SOIL LAYER DATA

         V.A.--RIGHTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

         V.B.--LEFTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

     VI.--WATER DATA
        UNIT WEIGHT         = 62.40 (PCF)
        RIGHTSIDE ELEVATION = 2.50 (FT)
        LEFTSIDE ELEVATION  = 0.00 (FT)
```

NO SEEPAGE

VII.--VERTICAL SURCHARGE LOADS

VII.A.--VERTICAL LINE LOADS
NONE

VII.B.--VERTICAL UNIFORM LOADS

LEFTSIDE (PSF)	RIGHTSIDE (PSF)
0.00	300.00

VII.C.--VERTICAL STRIP LOADS
NONE

VII.D.--VERTICAL RAMP LOADS
NONE

VII.E.--VERTICAL TRIANGULAR LOADS
NONE

VII.F.--VERTICAL VARIABLE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 14:15:56

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**************************
*   SOIL PRESSURES FOR   *
* CANTILEVER WALL DESIGN *
**************************
```

I.--HEADING
'MAMSHEL WALL TYPE 2 - Up to 5'
'SEE HH 14+00 FOR EXAMPLE

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

		<--- LEFTSIDE --->		<------NET------> (SOIL + WATER)		<-- RIGHTSIDE -->	
ELEV. (FT)	NET WATER (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
5.0	0.0	0.0	0.0	43.8	974.4	43.8	974.4
4.0	0.0	0.0	0.0	105.7	2171.7	105.7	2171.7
3.0	0.0	0.0	0.0	159.8	2726.9	159.8	2726.9
2.5	0.0	0.0	0.0	185.3	2984.7	185.3	2984.7

2.0	31.2	0.0	0.0	237.5	3234.3	206.3	3203.1
1.0	93.6	0.0	0.0	336.9	3693.9	243.3	3600.3
0.0+	156.0	0.0	0.0	437.0	3875.2	281.0	3719.2
0.0-	156.0	100.0	0.0	337.0	3875.2	281.0	3719.2
-1.0	156.0	234.2	0.0	239.4	3906.8	317.6	3750.8
-2.0	156.0	368.3	0.0	137.0	4127.5	349.2	3971.5
-3.0	156.0	502.4	0.0	28.5	4357.9	374.9	4201.9
-3.3	156.0	537.1	0.0	0.0	4397.8	381.1	4241.8
-4.0	156.0	636.5	0.0	-81.7	4512.4	398.8	4356.4
-5.0	156.0	770.6	0.0	-190.5	4487.1	424.1	4331.1
-6.0	156.0	904.7	30.3	-299.3	4467.1	449.4	4341.4
-7.0	156.0	1038.8	71.3	-408.2	4531.5	474.7	4446.8
-8.0	156.0	1172.9	91.1	-517.0	4615.4	499.9	4550.4
-9.0	156.0	1307.1	109.4	-625.8	4732.6	525.2	4686.0
-10.0	156.0	1441.2	127.7	-734.7	4846.7	550.5	4818.4
-11.0	156.0	1575.3	146.0	-844.1	4940.0	575.2	4930.0
-12.0	156.0	1709.4	164.3	-958.6	5036.3	594.8	5044.6
-13.0	156.0	1843.5	182.6	-1068.6	5152.9	618.9	5179.5
-14.0	156.0	1977.6	200.9	-1178.4	5269.4	643.2	5314.4
-15.0	156.0	2111.7	219.3	-1296.0	5386.0	659.8	5449.3
-16.0	156.0	2245.9	237.6	-1412.0	5502.6	677.8	5584.2
-17.0	156.0	2380.0	255.9	-1528.1	5619.2	695.9	5719.1
-18.0	156.0	2514.1	274.2	-1644.2	5735.8	713.9	5854.0
-19.0	156.0	2648.2	292.5	-1760.2	5848.8	732.0	5985.3
-20.0	156.0	2782.3	310.8	-1876.3	5949.4	750.0	6104.2
-21.0	156.0	2916.4	329.1	-1992.4	6053.2	768.1	6226.3
-22.0	156.0	3050.5	347.4	-2107.8	6169.1	786.7	6360.5
-23.0	156.0	3184.7	365.8	-2219.0	6284.9	809.7	6494.7
-24.0	156.0	3318.8	384.1	-2330.6	6400.8	832.1	6628.9
-25.0	156.0	3452.9	402.4	-2446.5	6516.7	850.4	6763.1
-26.0	156.0	3587.0	420.7	-2562.4	6632.6	868.6	6897.3
-27.0	156.0	3721.1	439.0	-2678.2	6748.5	886.9	7031.5
-28.0	156.0	3855.2	457.3	-2794.1	6864.4	905.1	7165.7
-29.0	156.0	3989.3	475.6	-2910.0	6980.3	923.4	7299.9
-30.0	156.0	4123.5	493.9	-3025.8	7096.2	941.6	7434.1
-31.0	156.0	4257.6	512.3	-3141.7	7212.1	959.9	7568.3

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 14:15:56

```
****************************
*  SUMMARY OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
****************************
```

I.--HEADING
'MAMSHEL WALL TYPE 2 - Up to 5'
'SEE HH 14+00 FOR EXAMPLE

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :      -14.96
      PENETRATION (FT)     :       14.96

MAX. BEND. MOMENT (LB-FT)  :  1.2556E+04
      AT ELEVATION (FT)    :       -8.73

MAX. SCALED DEFL. (LB-IN^3):  2.6182E+09
      AT ELEVATION (FT)    :        5.00
```

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 14:15:56

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****************************
* COMPLETE OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
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I.--HEADING
'MAMSHEL WALL TYPE 2 - Up to 5'
'SEE HH 14+00 FOR EXAMPLE

II.--RESULTS

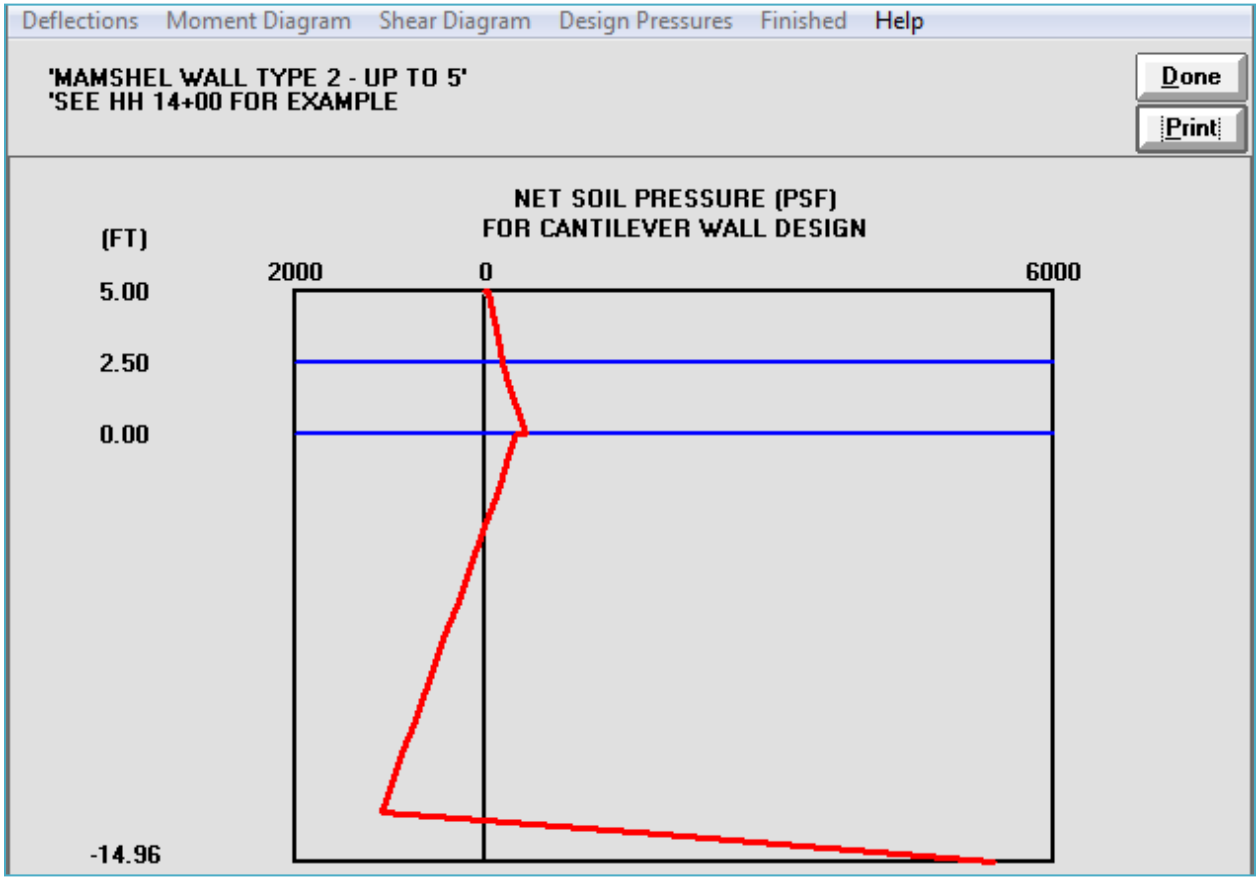
ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
5.00	0.0000E+00	0.	2.6182E+09	43.78
4.00	3.2216E+01	75.	2.4090E+09	105.73
3.00	1.6886E+02	208.	2.1998E+09	159.84
2.50	2.9367E+02	294.	2.0953E+09	185.27
2.00	4.6591E+02	400.	1.9909E+09	237.54
1.00	1.0008E+03	687.	1.7829E+09	336.87
0.00+	1.8726E+03	1074.	1.5767E+09	436.96
0.00-	1.8726E+03	1074.	1.5767E+09	336.97
-1.00	3.0985E+03	1362.	1.3737E+09	239.40
-2.00	4.5629E+03	1550.	1.1761E+09	136.96
-3.00	6.1633E+03	1633.	9.8648E+08	28.48
-3.26	6.5861E+03	1636.	9.3907E+08	0.00
-4.00	7.7919E+03	1606.	8.0747E+08	-81.68
-5.00	9.3390E+03	1470.	6.4192E+08	-190.51
-6.00	1.0696E+04	1225.	4.9247E+08	-299.34
-7.00	1.1753E+04	871.	3.6146E+08	-408.17
-8.00	1.2402E+04	409.	2.5071E+08	-517.00
-9.00	1.2534E+04	-163.	1.6131E+08	-625.84
-10.00	1.2040E+04	-843.	9.3474E+07	-734.67
-11.00	1.0812E+04	-1632.	4.6343E+07	-844.06
-12.00	8.7386E+03	-2534.	1.7773E+07	-958.65
-13.00	5.7073E+03	-3547.	4.1649E+06	-1068.61
-13.20	4.9672E+03	-3766.	2.7935E+06	-1090.84
-14.00	1.9284E+03	-3463.	2.8309E+05	1849.54
-14.96	0.0000E+00	0.	0.0000E+00	5381.12

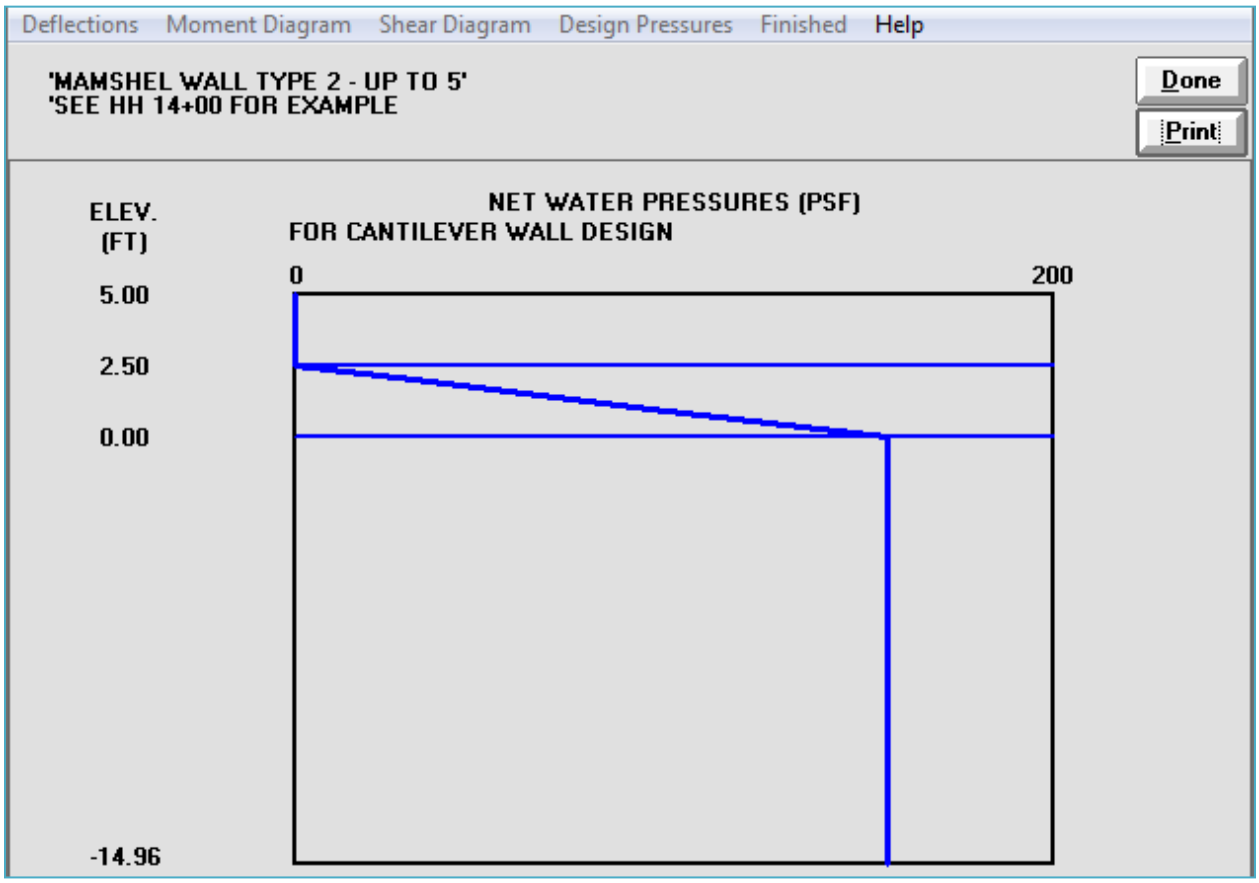
NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

		<----------SOIL PRESSURES---------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
5.00	0.	0.	0.	44.	974.
4.00	0.	0.	0.	106.	2172.
3.00	0.	0.	0.	160.	2727.
2.50	0.	0.	0.	185.	2985.
2.00	31.	0.	0.	206.	3203.
1.00	94.	0.	0.	243.	3600.
0.00+	156.	0.	0.	281.	3719.
0.00-	156.	100.	0.	281.	3719.
-1.00	156.	234.	0.	318.	3751.
-2.00	156.	368.	0.	349.	3971.
-3.00	156.	502.	0.	375.	4202.
-3.26	156.	537.	0.	381.	4242.
-4.00	156.	636.	0.	399.	4356.
-5.00	156.	771.	0.	424.	4331.
-6.00	156.	905.	30.	449.	4341.
-7.00	156.	1039.	71.	475.	4447.
-8.00	156.	1173.	91.	500.	4550.
-9.00	156.	1307.	109.	525.	4686.
-10.00	156.	1441.	128.	551.	4818.
-11.00	156.	1575.	146.	575.	4930.
-12.00	156.	1709.	164.	595.	5045.
-13.00	156.	1844.	183.	619.	5179.
-13.20	156.	1871.	186.	624.	5207.
-14.00	156.	1978.	201.	643.	5314.
-14.96	156.	2112.	219.	660.	5449.
-16.00	156.	2246.	238.	678.	5584.

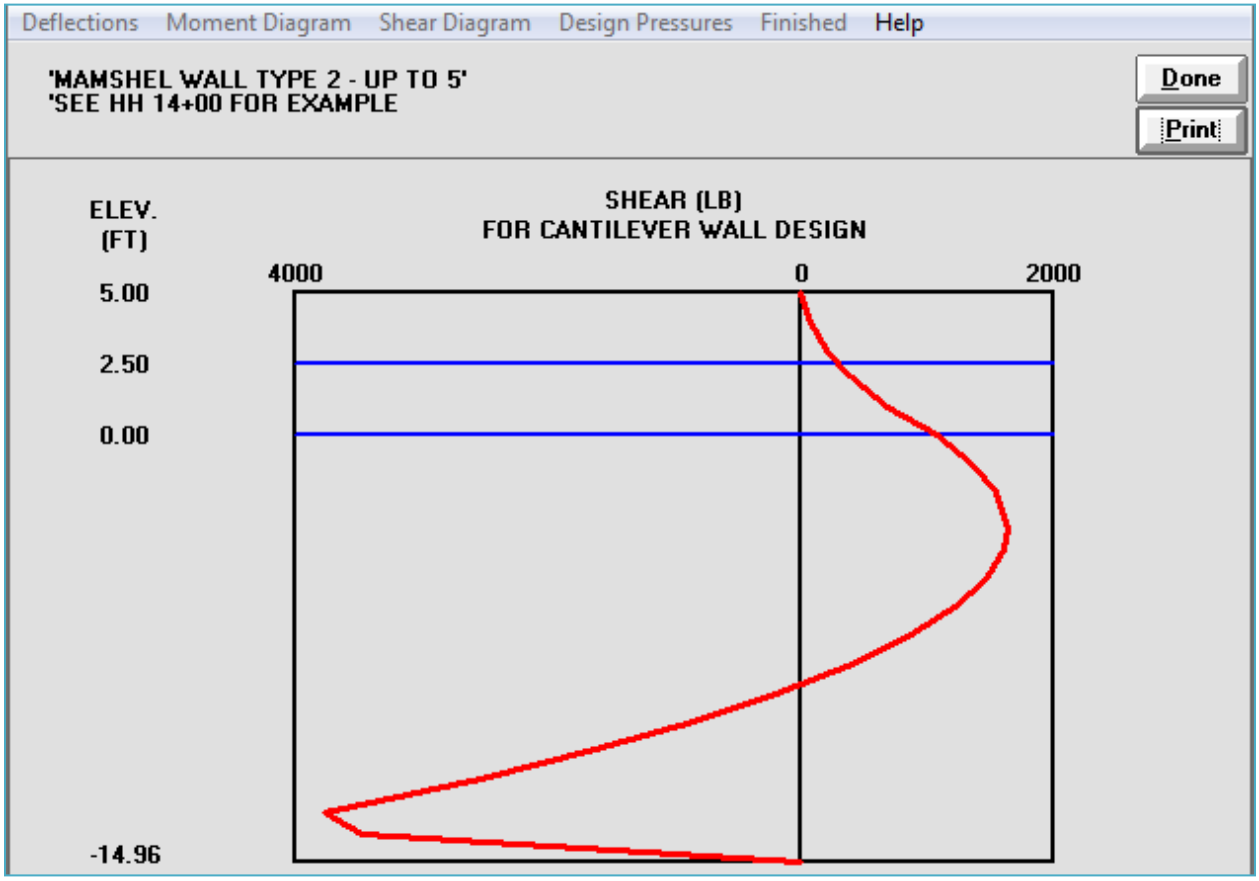
Sheet Pile Wall Type 2 (up to 5') – Design Case 2

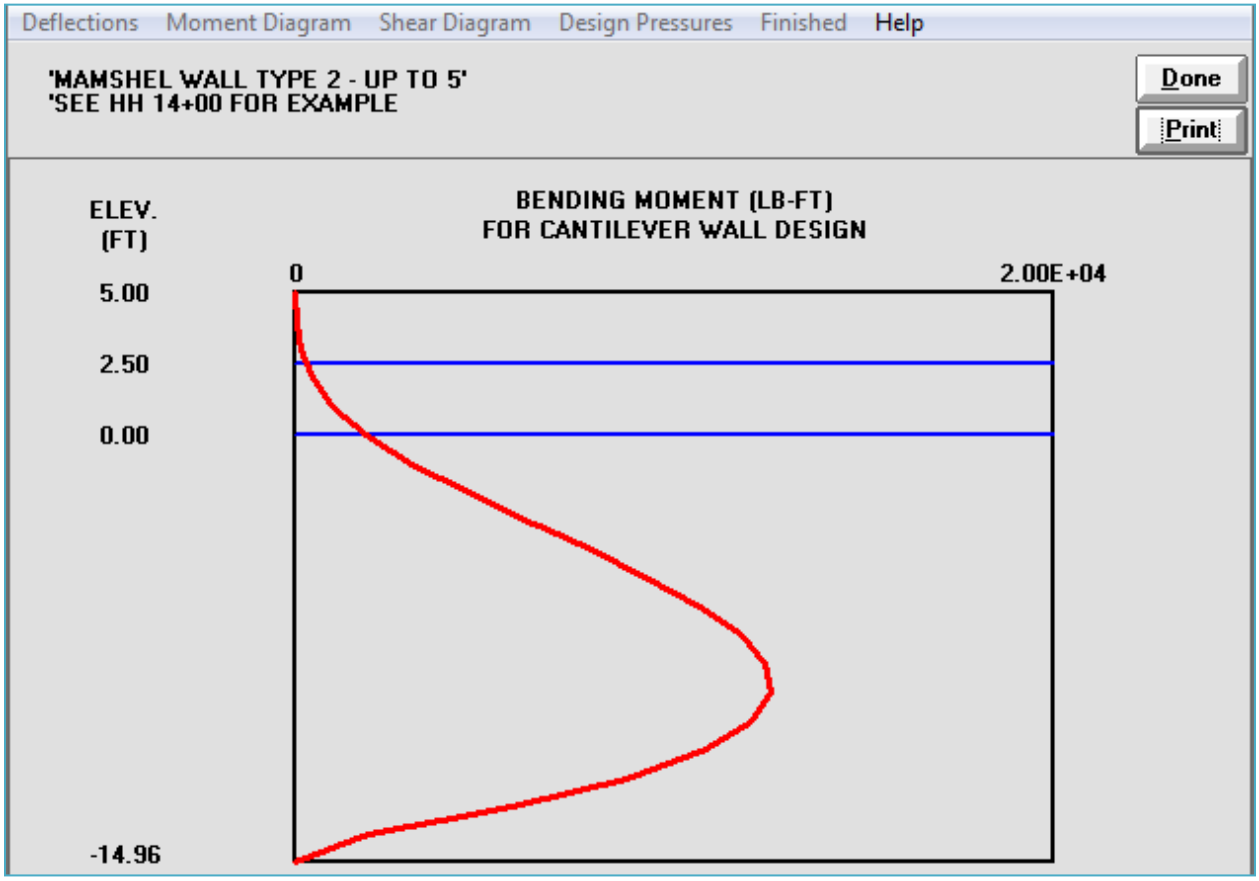




Type 2 Figures - p1

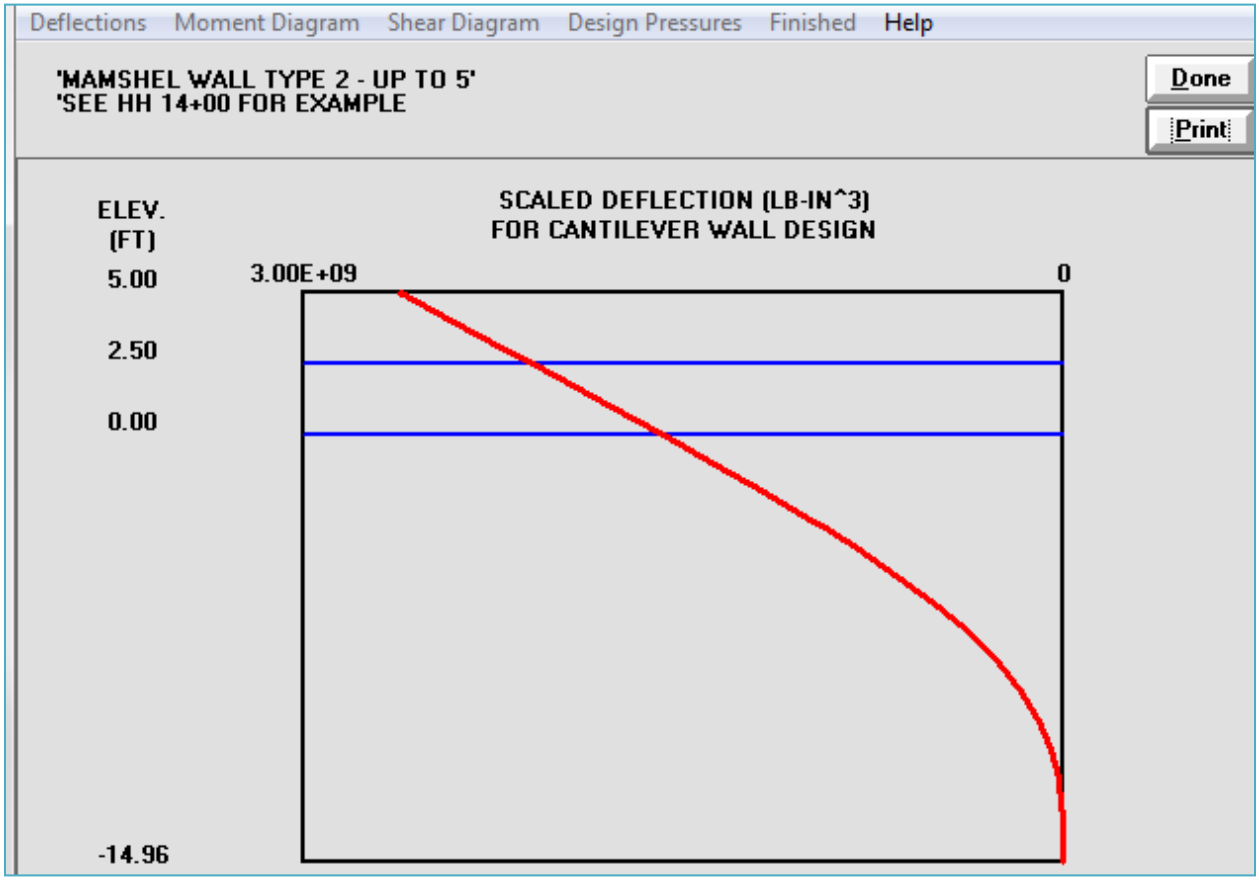
Sheet Pile Wall Type 2 (up to 5') – Design Case 2





Type 2 Figures - p2

Sheet Pile Wall Type 2 (up to 5') – Design Case 2



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PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 20-JULY-2015                                          TIME: 15:47:44

                            ****************
                            *  INPUT DATA  *
                            ****************

      I.--HEADING
     'MAMSHEL WALL TYPE 3 - UP TO 8'
     'Case 2

     II.--CONTROL
        CANTILEVER WALL DESIGN
        FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
        FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

    III.--WALL DATA
        ELEVATION AT TOP OF WALL    = 8.00 FT.

     IV.--SURFACE POINT DATA

          IV.A.--RIGHTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00          8.00
                  8.00         12.00
                 16.00         14.00
                 50.00         14.00

          IV.B.--LEFTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00          0.00
                 25.00          0.00

      V.--SOIL LAYER DATA

         V.A.--RIGHTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

         V.B.--LEFTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

     VI.--WATER DATA
        UNIT WEIGHT         = 62.40 (PCF)
        RIGHTSIDE ELEVATION = 4.00 (FT)
        LEFTSIDE ELEVATION  = 0.00 (FT)
        NO SEEPAGE
```

VII.--VERTICAL SURCHARGE LOADS

VII.A.--VERTICAL LINE LOADS
NONE

VII.B.--VERTICAL UNIFORM LOADS

LEFTSIDE (PSF)	RIGHTSIDE (PSF)
0.00	300.00

VII.C.--VERTICAL STRIP LOADS
NONE

VII.D.--VERTICAL RAMP LOADS
NONE

VII.E.--VERTICAL TRIANGULAR LOADS
NONE

VII.F.--VERTICAL VARIABLE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 20-JULY-2015 TIME: 15:47:47

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*************************
*   SOIL PRESSURES FOR   *
* CANTILEVER WALL DESIGN *
*************************
```

I.--HEADING
'MAMSHEL WALL TYPE 3 - UP TO 8'

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

				<------NET------>			
	NET	<---LEFTSIDE--->		(SOIL + WATER)		<--RIGHTSIDE--->	
ELEV.	WATER	PASSIVE	ACTIVE	ACTIVE	PASSIVE	ACTIVE	PASSIVE
(FT)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)
8.0	0.0	0.0	0.0	43.8	974.4	43.8	974.4
7.0	0.0	0.0	0.0	105.7	2171.7	105.7	2171.7
6.0	0.0	0.0	0.0	160.1	2726.9	160.1	2726.9
5.0	0.0	0.0	0.0	215.1	3282.1	215.1	3282.1
4.0	0.0	0.0	0.0	265.7	3794.2	265.7	3794.2
3.0	62.4	0.0	0.0	369.5	4005.4	307.1	3943.0

2.0	124.8	0.0	0.0	468.8	4084.2	344.0	3959.4
1.0	187.2	0.0	0.0	564.1	4368.5	376.9	4181.3
0.0+	249.6	0.0	0.0	651.8	4671.5	402.2	4421.9
0.0-	249.6	100.0	0.0	551.8	4671.5	402.2	4421.9
-1.0	249.6	234.2	0.0	441.2	4792.6	425.7	4543.0
-2.0	249.6	368.3	0.0	332.4	4762.4	451.0	4512.8
-3.0	249.6	502.4	0.0	223.5	4788.5	476.3	4538.9
-4.0	249.6	636.5	0.0	114.7	4891.6	501.6	4642.0
-5.0	249.6	770.6	0.0	5.9	5006.4	526.9	4756.8
-5.1	249.6	777.8	1.6	0.0	5012.1	528.2	4764.1
-6.0	249.6	904.7	30.3	-103.0	5112.3	552.2	4893.0
-7.0	249.6	1038.8	71.3	-211.8	5191.1	577.4	5012.7
-8.0	249.6	1172.9	91.1	-321.2	5281.8	602.2	5123.3
-9.0	249.6	1307.1	109.4	-432.0	5389.9	625.4	5249.7
-10.0	249.6	1441.2	127.7	-541.7	5506.5	649.8	5384.5
-11.0	249.6	1575.3	146.0	-655.1	5623.0	670.6	5519.4
-12.0	249.6	1709.4	164.3	-772.6	5739.6	687.2	5654.3
-13.0	249.6	1843.5	182.6	-888.6	5856.2	705.3	5789.2
-14.0	249.6	1977.6	200.9	-1004.7	5972.8	723.3	5924.1
-15.0	249.6	2111.7	219.3	-1120.8	6089.4	741.4	6059.0
-16.0	249.6	2245.9	237.6	-1236.8	6191.0	759.4	6179.0
-17.0	249.6	2380.0	255.9	-1352.9	6291.1	777.5	6297.4
-18.0	249.6	2514.1	274.2	-1469.0	6405.7	795.5	6430.3
-19.0	249.6	2648.2	292.5	-1580.7	6521.6	817.9	6564.5
-20.0	249.6	2782.3	310.8	-1691.7	6637.5	841.0	6698.7
-21.0	249.6	2916.4	329.1	-1806.9	6753.4	859.9	6832.9
-22.0	249.6	3050.5	347.4	-1922.8	6869.3	878.1	6967.1
-23.0	249.6	3184.7	365.8	-2038.7	6985.2	896.4	7101.3
-24.0	249.6	3318.8	384.1	-2154.5	7101.1	914.6	7235.5
-25.0	249.6	3452.9	402.4	-2270.4	7217.0	932.9	7369.7
-26.0	249.6	3587.0	420.7	-2386.3	7332.8	951.1	7503.9
-27.0	249.6	3721.1	439.0	-2502.2	7448.7	969.4	7638.1
-28.0	249.6	3855.2	457.3	-2618.0	7564.6	987.6	7772.3
-29.0	249.6	3989.3	475.6	-2733.9	7680.5	1005.9	7906.6
-30.0	249.6	4123.5	493.9	-2849.8	7796.4	1024.1	8040.8
-31.0	249.6	4257.6	512.3	-2965.6	7912.3	1042.3	8175.0

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 15:47:47

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****************************
*  SUMMARY OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
****************************
```

I.--HEADING
'MAMSHEL WALL TYPE 3 - UP TO 8'

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :      -23.11
      PENETRATION (FT)     :       23.11

MAX. BEND. MOMENT (LB-FT)  :  4.5856E+04
      AT ELEVATION (FT)    :      -13.45

MAX. SCALED DEFL. (LB-IN^3):  2.2868E+10
      AT ELEVATION (FT)    :        8.00
```

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 15:47:47

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****************************
* COMPLETE OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
****************************
```

I.--HEADING
'MAMSHEL WALL TYPE 3 - UP TO 8'

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
8.00	0.0000E+00	0.	2.2868E+10	43.78
7.00	3.2216E+01	75.	2.1699E+10	105.73
6.00	1.6890E+02	208.	2.0529E+10	160.08
5.00	4.6577E+02	395.	1.9360E+10	215.13
4.00	9.7703E+02	636.	1.8192E+10	265.71
3.00	1.7629E+03	953.	1.7026E+10	369.52
2.00	2.9175E+03	1372.	1.5862E+10	468.85
1.00	4.5403E+03	1889.	1.4704E+10	564.13
0.00+	6.7259E+03	2497.	1.3554E+10	651.82
0.00-	6.7259E+03	2497.	1.3554E+10	551.83
-1.00	9.4804E+03	2993.	1.2415E+10	441.19
-2.00	1.2676E+04	3380.	1.1293E+10	332.36
-3.00	1.6205E+04	3658.	1.0192E+10	223.53
-4.00	1.9956E+04	3827.	9.1201E+09	114.70
-5.00	2.3823E+04	3888.	8.0823E+09	5.86
-5.05	2.4032E+04	3888.	8.0275E+09	0.00
-6.00	2.7695E+04	3839.	7.0858E+09	-102.97
-7.00	3.1465E+04	3682.	6.1370E+09	-211.80
-8.00	3.5022E+04	3415.	5.2426E+09	-321.19
-9.00	3.8258E+04	3039.	4.4087E+09	-432.03
-10.00	4.1062E+04	2552.	3.6409E+09	-541.74
-11.00	4.3324E+04	1953.	2.9439E+09	-655.07
-12.00	4.4930E+04	1239.	2.3216E+09	-772.58
-13.00	4.5764E+04	409.	1.7769E+09	-888.65
-14.00	4.5709E+04	-538.	1.3112E+09	-1004.71
-15.00	4.4650E+04	-1601.	9.2429E+08	-1120.78
-16.00	4.2469E+04	-2779.	6.1438E+08	-1236.84

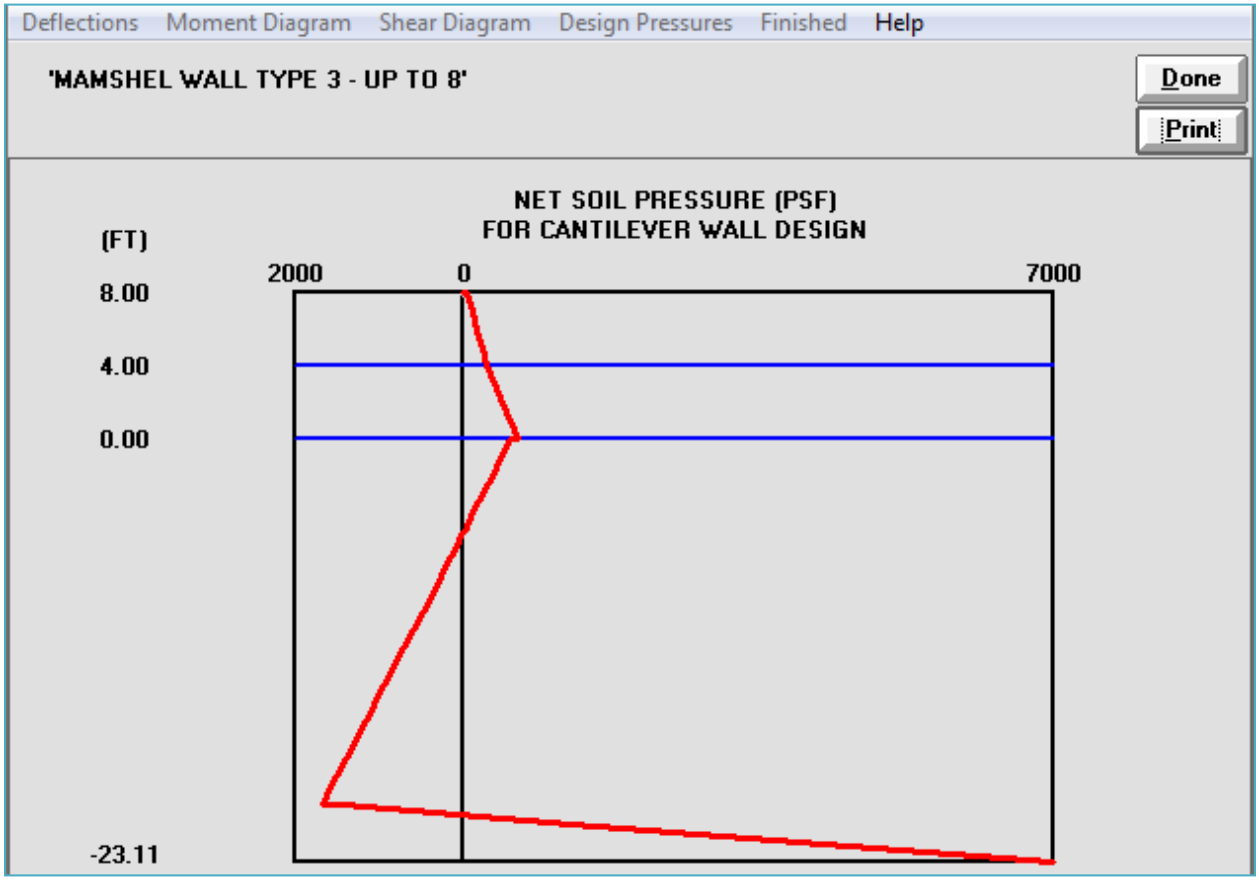
-17.00	3.9052E+04	-4074.	3.7768E+08	-1352.91
-18.00	3.4282E+04	-5485.	2.0827E+08	-1468.98
-19.00	2.8044E+04	-7010.	9.7883E+07	-1580.73
-19.91	2.0979E+04	-8498.	3.9587E+07	-1681.97
-20.00	2.0225E+04	-8635.	3.5728E+07	-1443.14
-21.00	1.1320E+04	-8721.	8.3403E+06	1271.67
-22.00	3.6875E+03	-6092.	6.9711E+05	3986.47
-23.00	4.1159E+01	-748.	7.1119E+01	6701.28
-23.11	0.0000E+00	0.	0.0000E+00	6997.83

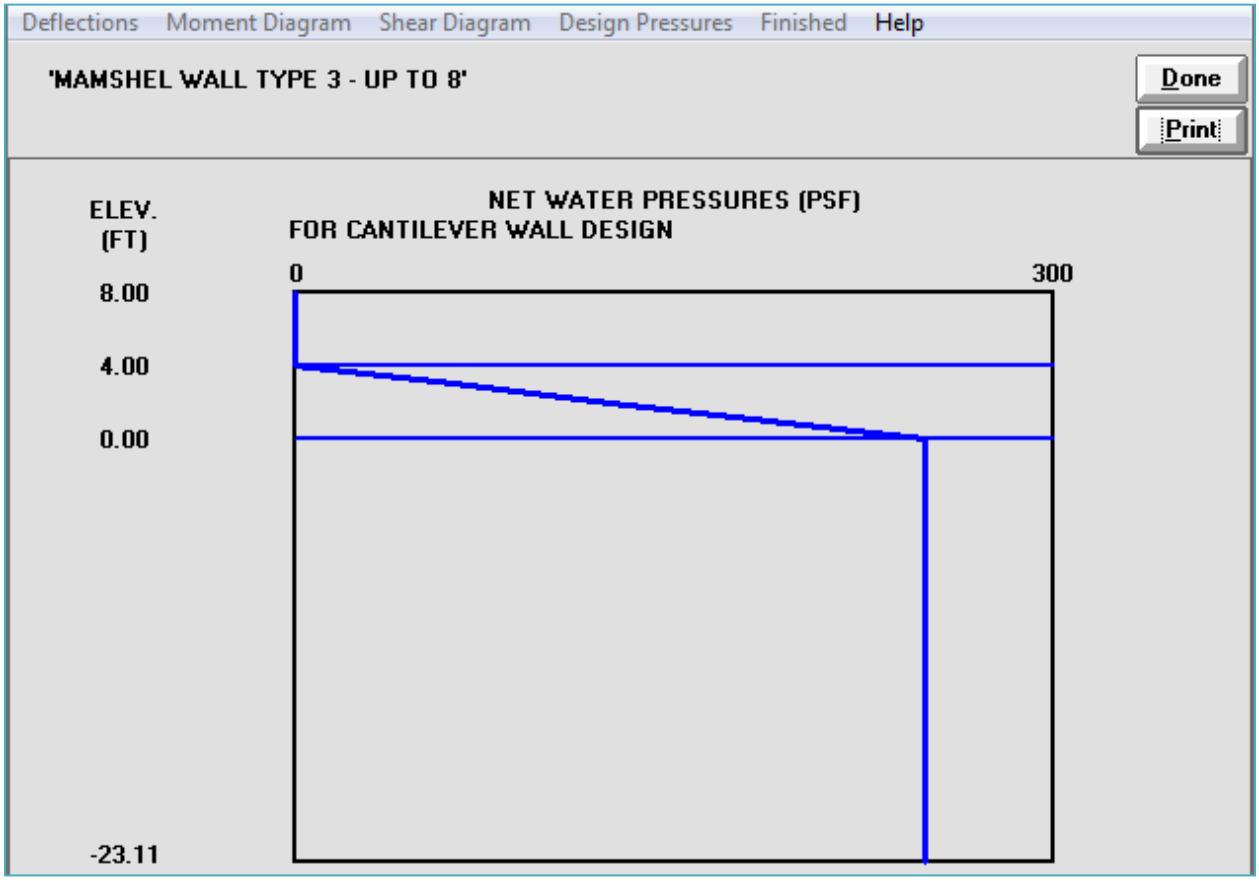
NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

		<----------SOIL PRESSURES---------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
8.00	0.	0.	0.	44.	974.
7.00	0.	0.	0.	106.	2172.
6.00	0.	0.	0.	160.	2727.
5.00	0.	0.	0.	215.	3282.
4.00	0.	0.	0.	266.	3794.
3.00	62.	0.	0.	307.	3943.
2.00	125.	0.	0.	344.	3959.
1.00	187.	0.	0.	377.	4181.
0.00+	250.	0.	0.	402.	4422.
0.00-	250.	100.	0.	402.	4422.
-1.00	250.	234.	0.	426.	4543.
-2.00	250.	368.	0.	451.	4513.
-3.00	250.	502.	0.	476.	4539.
-4.00	250.	636.	0.	502.	4642.
-5.00	250.	771.	0.	527.	4757.
-5.05	250.	778.	2.	528.	4764.
-6.00	250.	905.	30.	552.	4893.
-7.00	250.	1039.	71.	577.	5013.
-8.00	250.	1173.	91.	602.	5123.
-9.00	250.	1307.	109.	625.	5250.
-10.00	250.	1441.	128.	650.	5385.
-11.00	250.	1575.	146.	671.	5519.
-12.00	250.	1709.	164.	687.	5654.
-13.00	250.	1844.	183.	705.	5789.
-14.00	250.	1978.	201.	723.	5924.
-15.00	250.	2112.	219.	741.	6059.
-16.00	250.	2246.	238.	759.	6179.
-17.00	250.	2380.	256.	777.	6297.
-18.00	250.	2514.	274.	796.	6430.
-19.00	250.	2648.	293.	818.	6565.
-19.91	250.	2771.	309.	839.	6687.
-20.00	250.	2782.	311.	841.	6699.
-21.00	250.	2916.	329.	860.	6833.
-22.00	250.	3051.	347.	878.	6967.
-23.00	250.	3185.	366.	896.	7101.
-23.11	250.	3319.	384.	915.	7236.
-25.00	250.	3453.	402.	933.	7370.

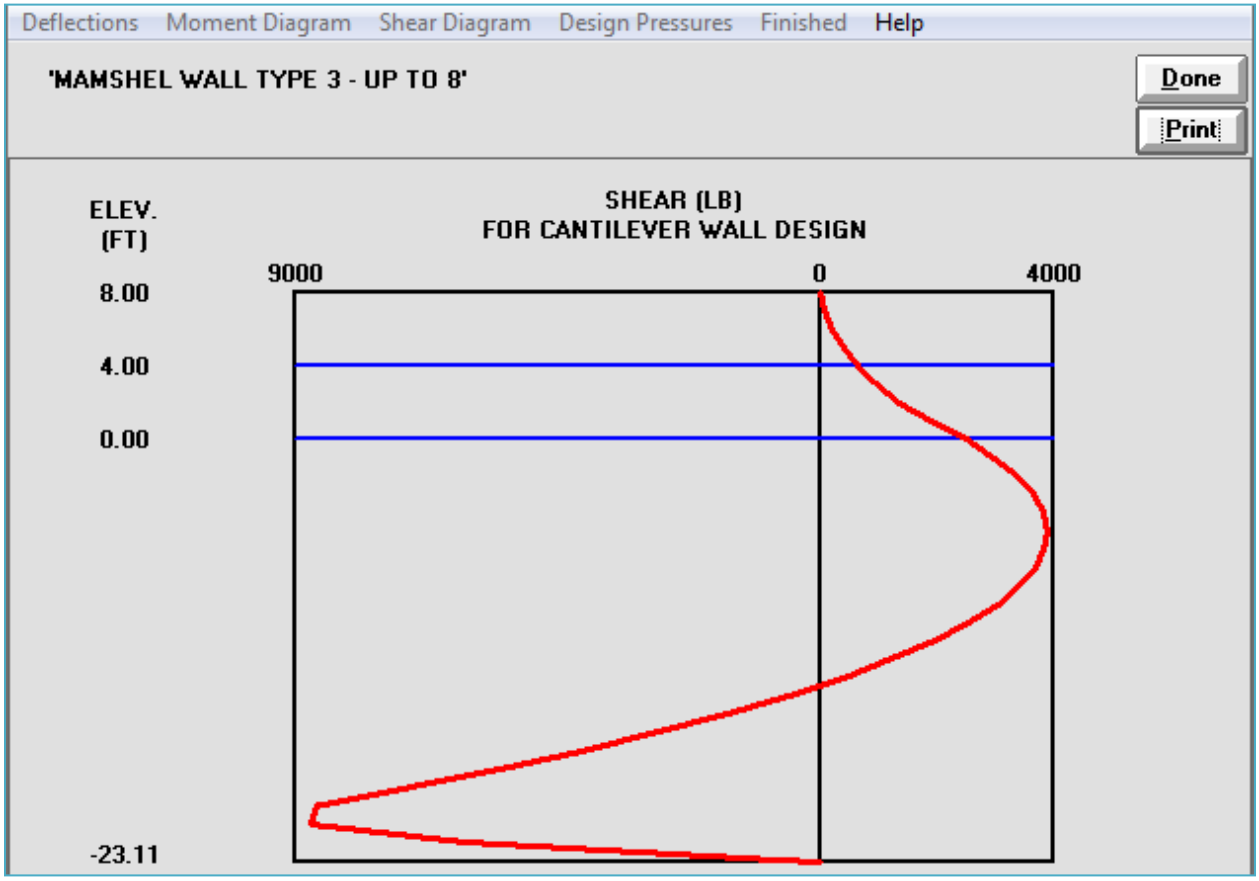
Sheet Pile Wall Type 3 (up to 8') – Design Case 2

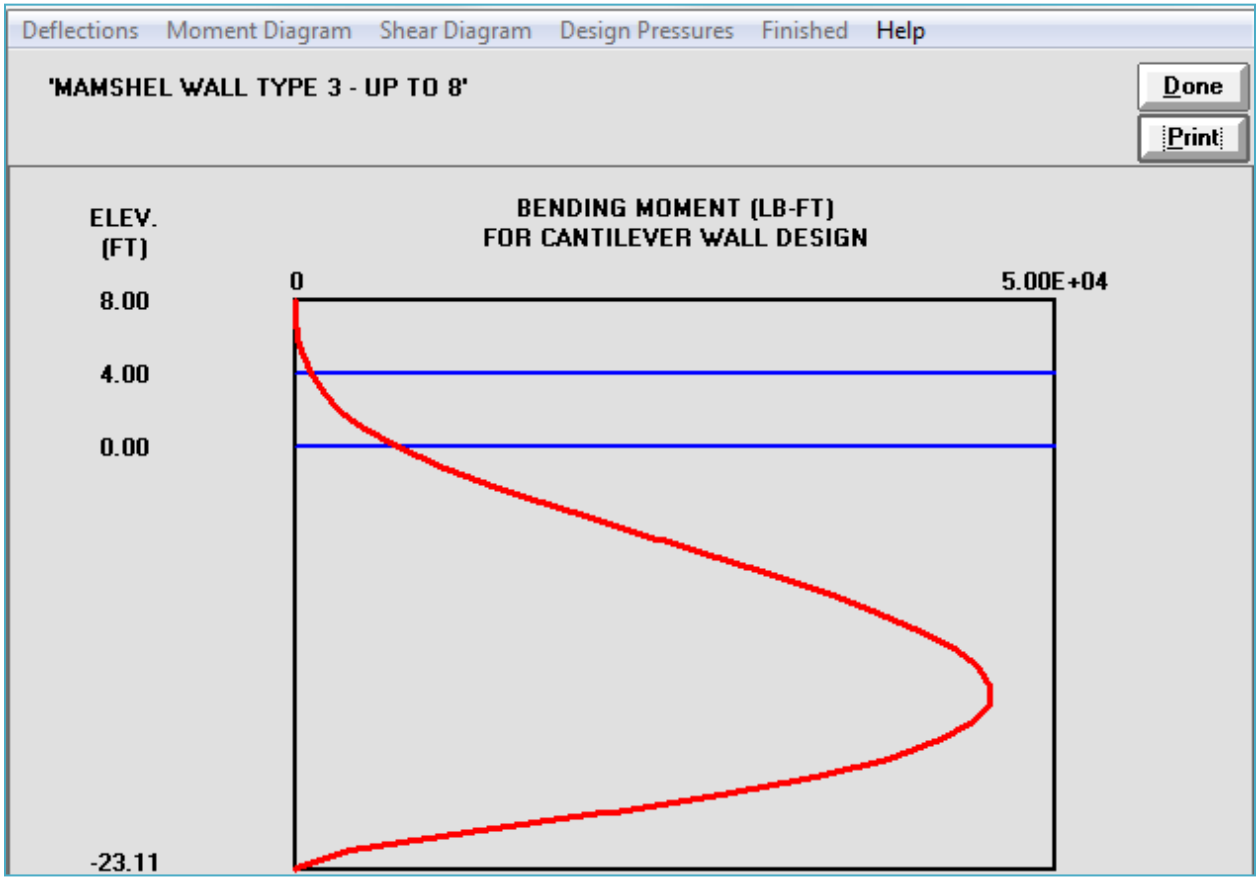




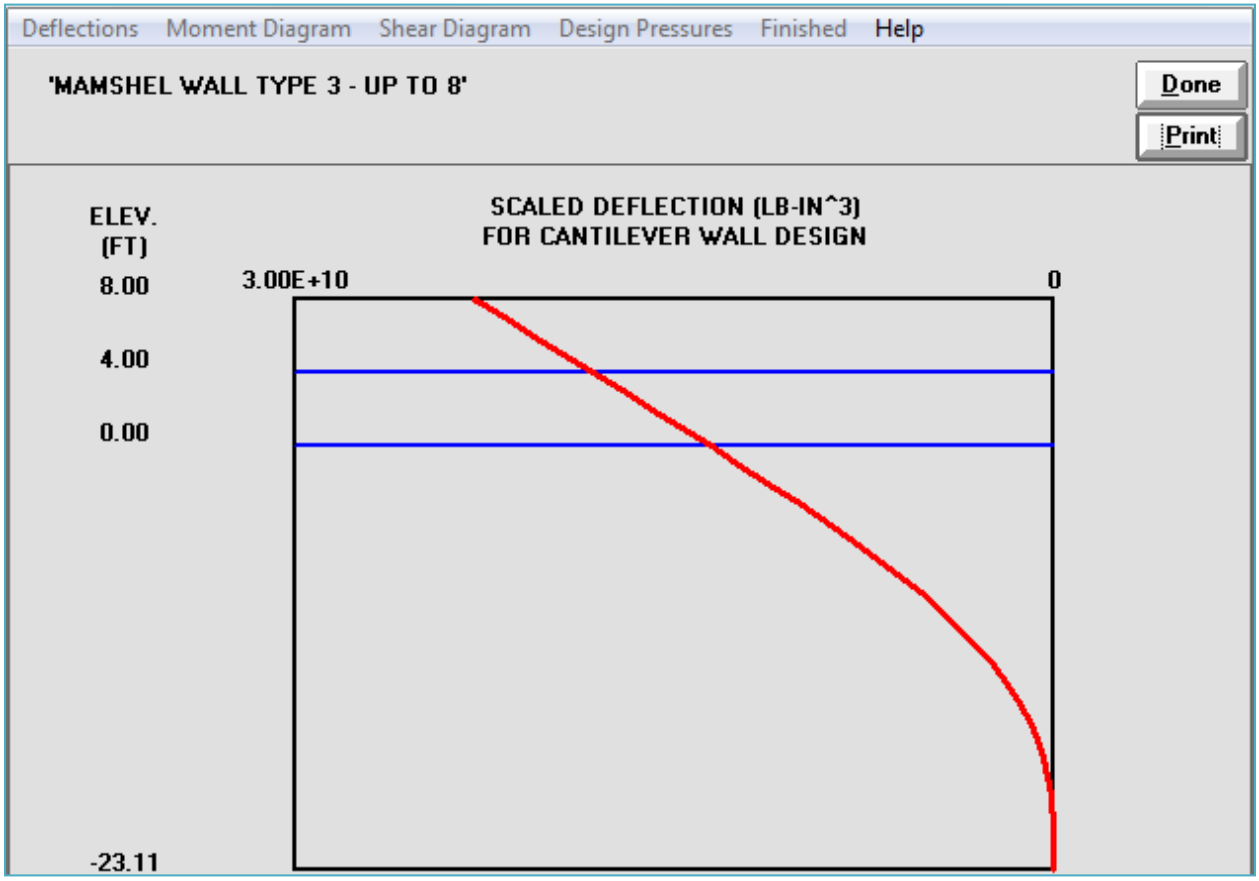
Type 3 Figures - p1

Sheet Pile Wall Type 3 (up to 8') – Design Case 2





Sheet Pile Wall Type 3 (up to 8') – Design Case 2



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PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 20-JULY-2015                                          TIME: 16:47:12

                            ****************
                            *  INPUT DATA  *
                            ****************

      I.--HEADING
     'MAMSHEL WALL TYPE 4 - UP TO 10'
     'Case 2

     II.--CONTROL
        CANTILEVER WALL DESIGN
        FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
        FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

    III.--WALL DATA
        ELEVATION AT TOP OF WALL    = 10.00 FT.

     IV.--SURFACE POINT DATA

          IV.A.--RIGHTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00         10.00
                  8.00         14.00
                 16.00         16.00
                 50.00         16.00

          IV.B.--LEFTSIDE
             DIST. FROM     ELEVATION
             WALL (FT)         (FT)
                  0.00          0.00
                 25.00          0.00

      V.--SOIL LAYER DATA

         V.A.--RIGHTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

         V.B.--LEFTSIDE
             LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
             LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF         ANGLE OF                        <-SAFETY->
  SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
122.00  120.00    32.00   50.00     0.00    0.00                  DEF  DEF

     VI.--WATER DATA
        UNIT WEIGHT         = 62.40 (PCF)
        RIGHTSIDE ELEVATION = 5.00 (FT)
        LEFTSIDE ELEVATION  = 0.00 (FT)
        NO SEEPAGE
```

VII.--VERTICAL SURCHARGE LOADS

VII.A.--VERTICAL LINE LOADS
NONE

VII.B.--VERTICAL UNIFORM LOADS

LEFTSIDE (PSF)	RIGHTSIDE (PSF)
0.00	300.00

VII.C.--VERTICAL STRIP LOADS
NONE

VII.D.--VERTICAL RAMP LOADS
NONE

VII.E.--VERTICAL TRIANGULAR LOADS
NONE

VII.F.--VERTICAL VARIABLE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 20-JULY-2015 TIME: 16:47:39

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**************************
*   SOIL PRESSURES FOR   *
* CANTILEVER WALL DESIGN *
**************************
```

I.--HEADING
'MAMSHEL WALL TYPE 4 - UP TO 10'

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

ELEV. (FT)	NET WATER (PSF)	<---LEFTSIDE---> PASSIVE (PSF)	ACTIVE (PSF)	<------NET------> (SOIL + WATER) ACTIVE (PSF)	PASSIVE (PSF)	<--RIGHTSIDE---> ACTIVE (PSF)	PASSIVE (PSF)
10.0	0.0	0.0	0.0	43.8	974.4	43.8	974.4
9.0	0.0	0.0	0.0	105.7	2171.7	105.7	2171.7
8.0	0.0	0.0	0.0	160.1	2726.9	160.1	2726.9
7.0	0.0	0.0	0.0	215.1	3282.1	215.1	3282.1
6.0	0.0	0.0	0.0	270.2	3832.1	270.2	3832.1
5.0	0.0	0.0	0.0	320.6	4054.2	320.6	4054.2

4.0	62.4	0.0	0.0	424.4	4167.2	362.0	4104.8
3.0	124.8	0.0	0.0	519.7	4451.5	394.9	4326.7
2.0	187.2	0.0	0.0	607.4	4755.7	420.2	4568.5
1.0	249.6	0.0	0.0	693.3	4921.1	443.7	4671.5
0.0+	312.0	0.0	0.0	781.0	4950.0	469.0	4638.0
0.0-	312.0	100.0	0.0	681.0	4950.0	469.0	4638.0
-1.0	312.0	234.2	0.0	572.1	4983.3	494.3	4671.3
-2.0	312.0	368.3	0.0	463.3	5085.5	519.5	4773.5
-3.0	312.0	502.4	0.0	354.4	5206.8	544.8	4894.8
-4.0	312.0	636.5	0.0	245.6	5343.1	570.1	5031.1
-5.0	312.0	770.6	0.0	136.8	5455.1	595.4	5143.1
-6.0	312.0	904.7	30.3	27.4	5534.7	620.1	5253.0
-6.3	312.0	938.6	40.7	0.0	5558.0	626.6	5286.6
-7.0	312.0	1038.8	71.3	-81.2	5627.1	645.7	5386.4
-8.0	312.0	1172.9	91.1	-190.7	5742.2	670.2	5521.3
-9.0	312.0	1307.1	109.4	-306.1	5858.8	688.9	5656.1
-10.0	312.0	1441.2	127.7	-423.7	5975.3	705.5	5791.0
-11.0	312.0	1575.3	146.0	-539.7	6091.9	723.6	5925.9
-12.0	312.0	1709.4	164.3	-655.8	6208.5	741.6	6060.8
-13.0	312.0	1843.5	182.6	-771.9	6319.6	759.7	6190.2
-14.0	312.0	1977.6	200.9	-887.9	6419.6	777.7	6308.6
-15.0	312.0	2111.7	219.3	-1004.0	6524.8	795.8	6432.1
-16.0	312.0	2245.9	237.6	-1118.4	6640.7	815.4	6566.3
-17.0	312.0	2380.0	255.9	-1229.4	6756.6	838.5	6700.5
-18.0	312.0	2514.1	274.2	-1342.0	6872.5	860.1	6834.7
-19.0	312.0	2648.2	292.5	-1457.8	6988.4	878.4	6968.9
-20.0	312.0	2782.3	310.8	-1573.7	7104.3	896.6	7103.1
-21.0	312.0	2916.4	329.1	-1689.6	7220.2	914.9	7237.3
-22.0	312.0	3050.5	347.4	-1805.4	7336.1	933.1	7371.5
-23.0	312.0	3184.7	365.8	-1921.3	7452.0	951.4	7505.7
-24.0	312.0	3318.8	384.1	-2037.2	7567.9	969.6	7639.9
-25.0	312.0	3452.9	402.4	-2153.0	7683.8	987.9	7774.1
-26.0	312.0	3587.0	420.7	-2268.9	7799.7	1006.1	7908.4
-27.0	312.0	3721.1	439.0	-2384.8	7915.5	1024.3	8042.6
-28.0	312.0	3855.2	457.3	-2500.6	8031.4	1042.6	8176.8
-29.0	312.0	3989.3	475.6	-2616.5	8147.3	1060.8	8311.0
-30.0	312.0	4123.5	493.9	-2732.4	8263.2	1079.1	8445.2
-31.0	312.0	4257.6	512.3	-2848.2	8379.1	1097.3	8579.4

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 16:47:40

```
****************************
*  SUMMARY OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
****************************
```

I.--HEADING
'MAMSHEL WALL TYPE 4 - UP TO 10'

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :      -28.31
      PENETRATION (FT)     :       28.31

MAX. BEND. MOMENT (LB-FT)  :  8.4410E+04
      AT ELEVATION (FT)    :      -16.39

MAX. SCALED DEFL. (LB-IN^3):  6.3407E+10
      AT ELEVATION (FT)    :       10.00
```

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 20-JULY-2015 TIME: 16:47:40

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****************************
* COMPLETE OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
****************************
```

I.--HEADING
'MAMSHEL WALL TYPE 4 - UP TO 10'

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
10.00	0.0000E+00	0.	6.3407E+10	43.78
9.00	3.2216E+01	75.	6.0777E+10	105.73
8.00	1.6890E+02	208.	5.8147E+10	160.08
7.00	4.6577E+02	395.	5.5517E+10	215.13
6.00	9.7778E+02	638.	5.2888E+10	270.19
5.00	1.7592E+03	933.	5.0260E+10	320.59
4.00	2.8701E+03	1306.	4.7636E+10	424.40
3.00	4.4040E+03	1778.	4.5017E+10	519.68
2.00	6.4563E+03	2341.	4.2405E+10	607.37
1.00	9.1157E+03	2992.	3.9805E+10	693.29
0.00+	1.2469E+04	3729.	3.7220E+10	780.98
0.00-	1.2469E+04	3729.	3.7220E+10	680.99
-1.00	1.6520E+04	4355.	3.4657E+10	572.11
-2.00	2.1143E+04	4873.	3.2123E+10	463.28
-3.00	2.6230E+04	5282.	2.9625E+10	354.44
-4.00	3.1671E+04	5582.	2.7173E+10	245.61
-5.00	3.7357E+04	5773.	2.4776E+10	136.78
-6.00	4.3181E+04	5855.	2.2443E+10	27.39
-6.25	4.4658E+04	5859.	2.1866E+10	0.00
-7.00	4.9032E+04	5828.	2.0185E+10	-81.17
-8.00	5.4801E+04	5692.	1.8011E+10	-190.70
-9.00	6.0379E+04	5444.	1.5932E+10	-306.15
-10.00	6.5650E+04	5079.	1.3957E+10	-423.66
-11.00	7.0498E+04	4597.	1.2096E+10	-539.73
-12.00	7.4806E+04	4000.	1.0357E+10	-655.80

-13.00	7.8459E+04	3286.	8.7463E+09	-771.86
-14.00	8.1339E+04	2456.	7.2714E+09	-887.93
-15.00	8.3332E+04	1510.	5.9370E+09	-1003.99
-16.00	8.4321E+04	449.	4.7464E+09	-1118.44
-17.00	8.4192E+04	-725.	3.7014E+09	-1229.45
-18.00	8.2833E+04	-2011.	2.8016E+09	-1341.96
-19.00	8.0132E+04	-3411.	2.0448E+09	-1457.83
-20.00	7.5973E+04	-4927.	1.4263E+09	-1573.70
-21.00	7.0240E+04	-6558.	9.3881E+08	-1689.56
-22.00	6.2818E+04	-8306.	5.7247E+08	-1805.43
-23.00	5.3590E+04	-10169.	3.1441E+08	-1921.30
-24.00	4.2441E+04	-12148.	1.4868E+08	-2037.16
-24.16	4.0447E+04	-12480.	1.2927E+08	-2055.93
-25.00	2.9506E+04	-13346.	5.6004E+07	-12.31
-26.00	1.6559E+04	-12139.	1.4316E+07	2426.15
-27.00	6.0395E+03	-8494.	1.5923E+06	4864.61
-28.00	3.8420E+02	-2410.	5.5120E+03	7303.07
-28.31	0.0000E+00	0.	0.0000E+00	8067.79

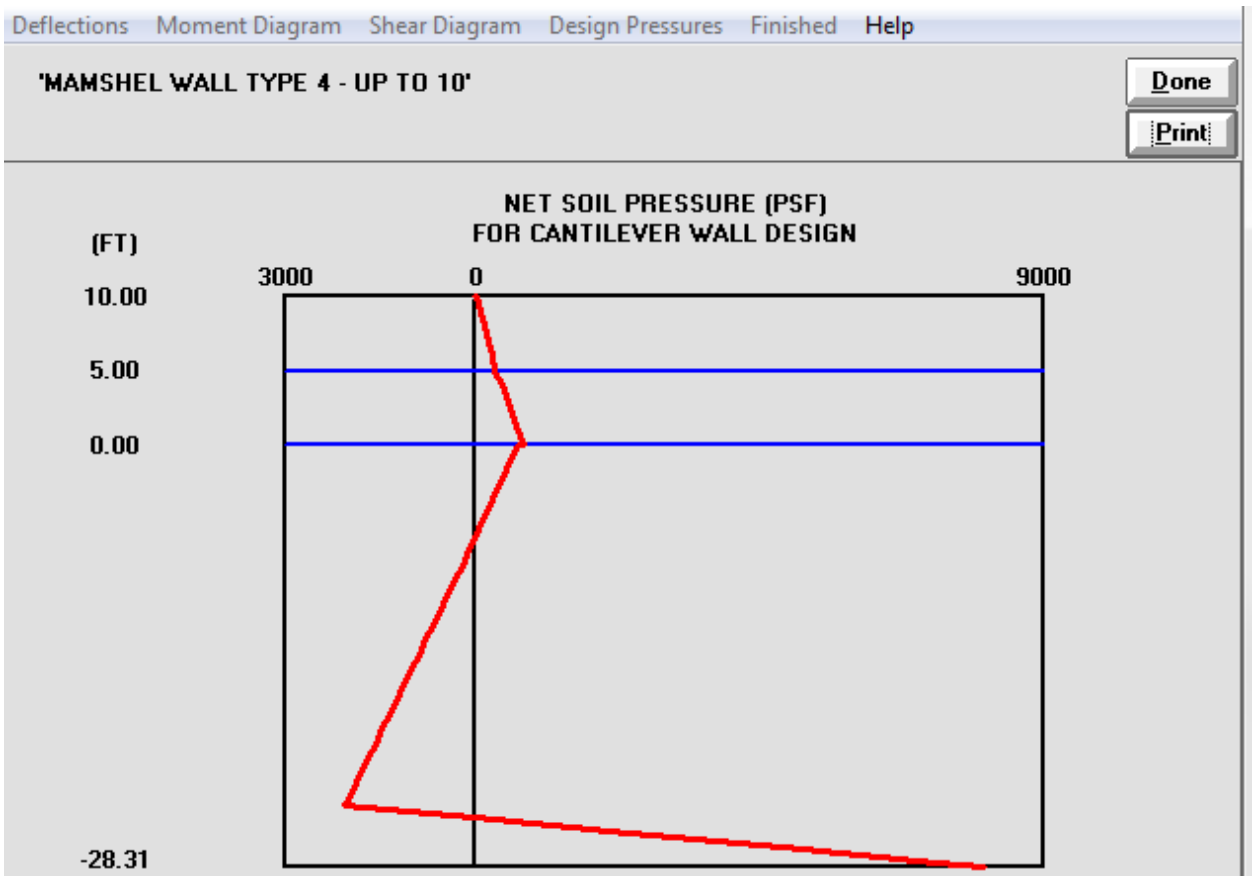
NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

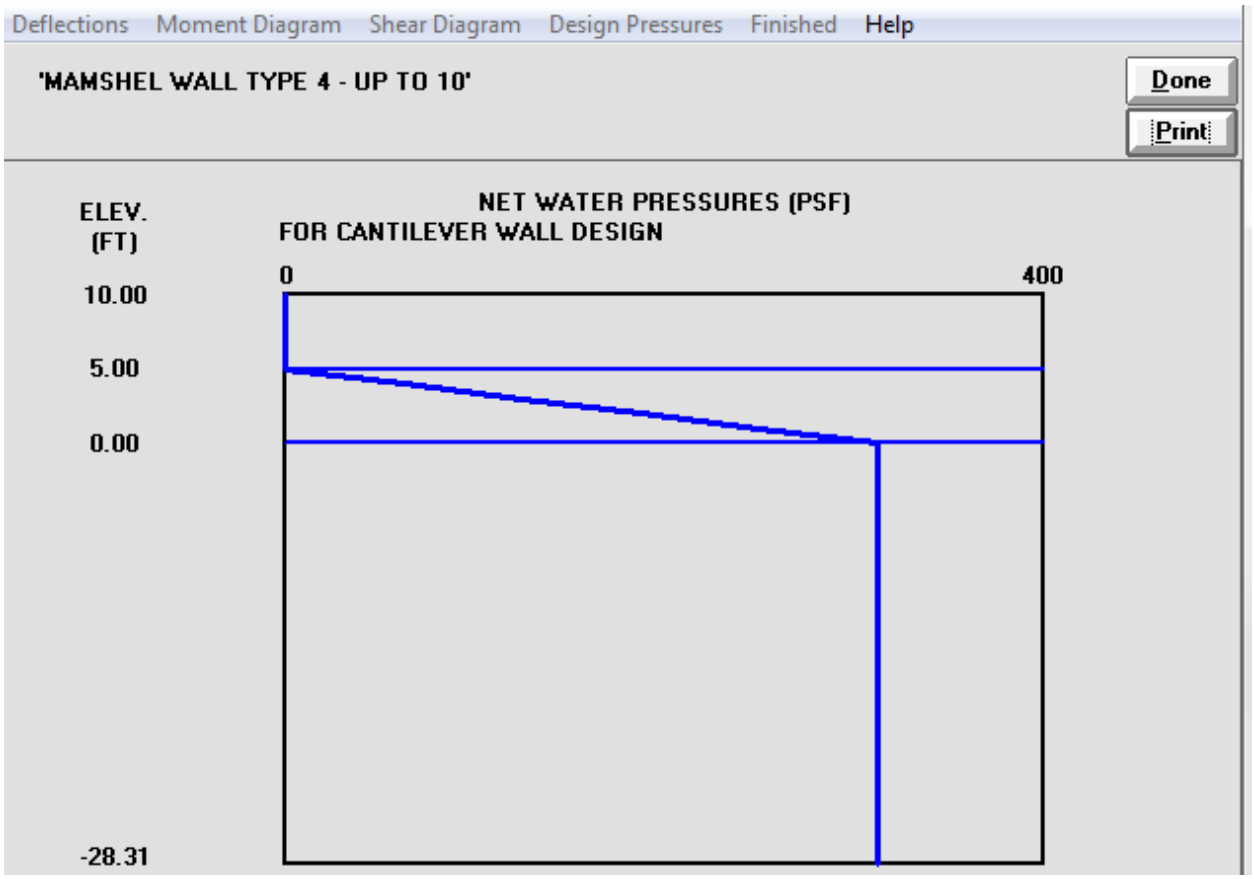
III.--WATER AND SOIL PRESSURES

		<----------SOIL PRESSURES---------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
10.00	0.	0.	0.	44.	974.
9.00	0.	0.	0.	106.	2172.
8.00	0.	0.	0.	160.	2727.
7.00	0.	0.	0.	215.	3282.
6.00	0.	0.	0.	270.	3832.
5.00	0.	0.	0.	321.	4054.
4.00	62.	0.	0.	362.	4105.
3.00	125.	0.	0.	395.	4327.
2.00	187.	0.	0.	420.	4569.
1.00	250.	0.	0.	444.	4672.
0.00+	312.	0.	0.	469.	4638.
0.00-	312.	100.	0.	469.	4638.
-1.00	312.	234.	0.	494.	4671.
-2.00	312.	368.	0.	520.	4773.
-3.00	312.	502.	0.	545.	4895.
-4.00	312.	636.	0.	570.	5031.
-5.00	312.	771.	0.	595.	5143.
-6.00	312.	905.	30.	620.	5253.
-6.25	312.	939.	41.	627.	5287.
-7.00	312.	1039.	71.	646.	5386.
-8.00	312.	1173.	91.	670.	5521.
-9.00	312.	1307.	109.	689.	5656.
-10.00	312.	1441.	128.	706.	5791.
-11.00	312.	1575.	146.	724.	5926.
-12.00	312.	1709.	164.	742.	6061.
-13.00	312.	1844.	183.	760.	6190.
-14.00	312.	1978.	201.	778.	6309.
-15.00	312.	2112.	219.	796.	6432.
-16.00	312.	2246.	238.	815.	6566.
-17.00	312.	2380.	256.	839.	6701.
-18.00	312.	2514.	274.	860.	6835.
-19.00	312.	2648.	293.	878.	6969.
-20.00	312.	2782.	311.	897.	7103.

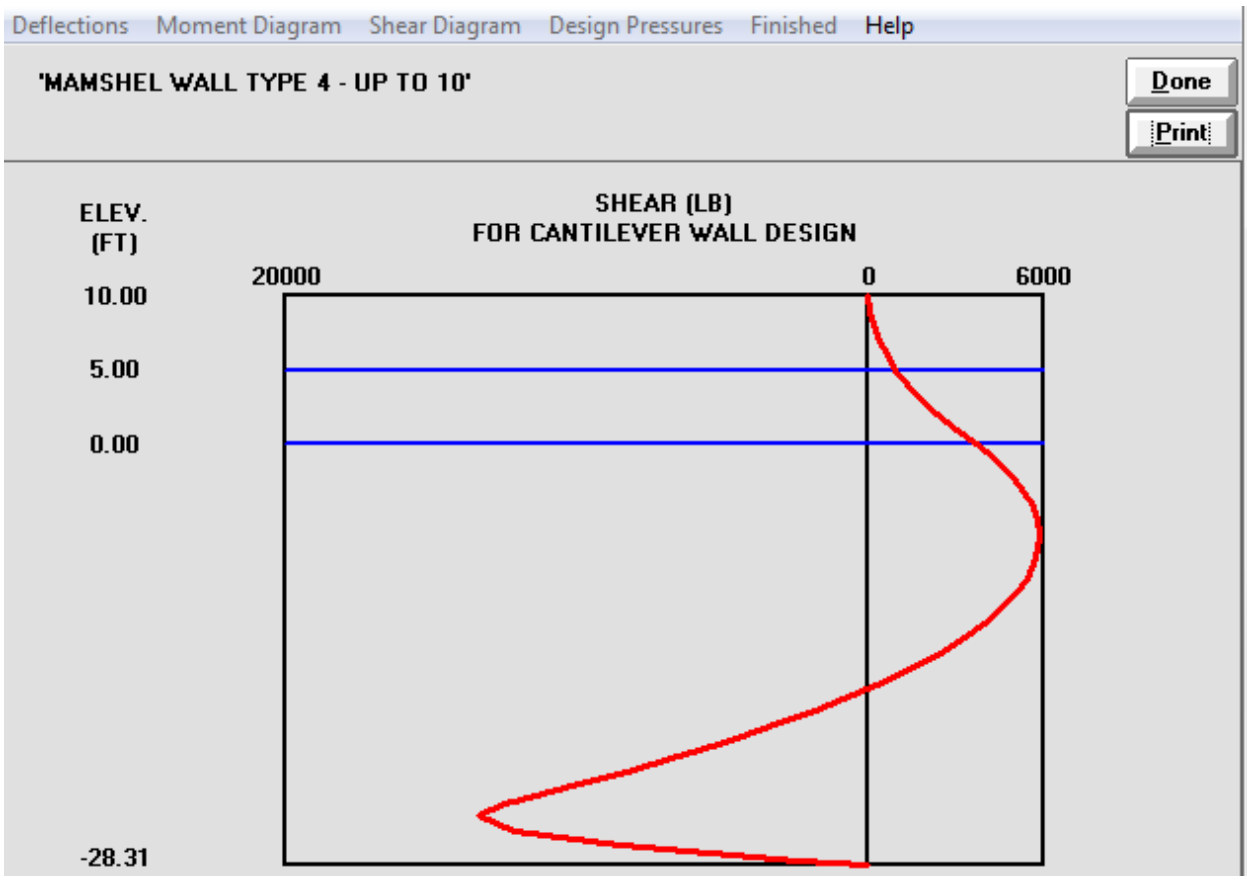
-21.00	312.	2916.	329.	915.	7237.
-22.00	312.	3051.	347.	933.	7372.
-23.00	312.	3185.	366.	951.	7506.
-24.00	312.	3319.	384.	970.	7640.
-24.16	312.	3340.	387.	973.	7662.
-25.00	312.	3453.	402.	988.	7774.
-26.00	312.	3587.	421.	1006.	7908.
-27.00	312.	3721.	439.	1024.	8043.
-28.00	312.	3855.	457.	1043.	8177.
-28.31	312.	3989.	476.	1061.	8311.
-30.00	312.	4123.	494.	1079.	8445.

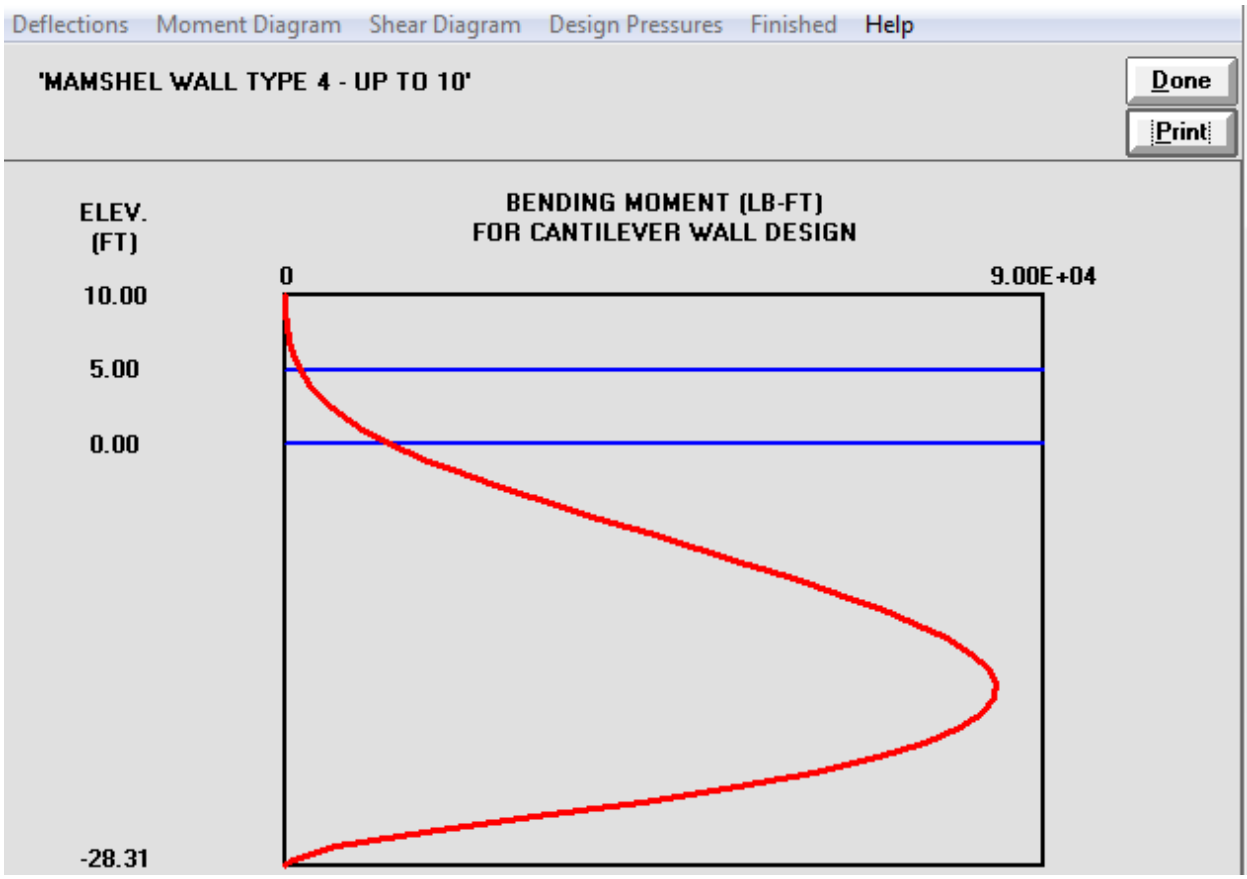
Sheet Pile Wall Type 4 (up to 10') – Design Case 2





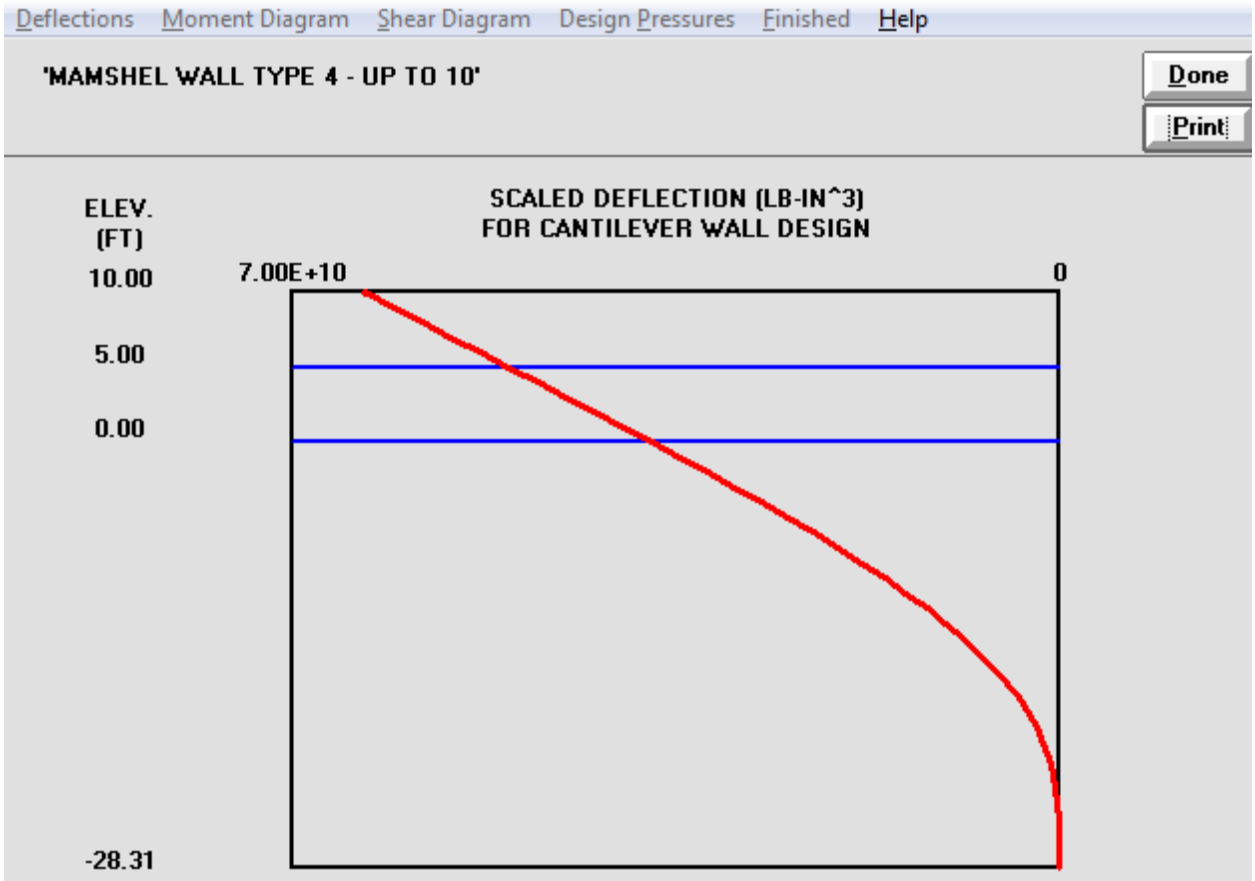
Sheet Pile Wall Type 4 (up to 10') – Design Case 2





Type 4 Figures - p2

Sheet Pile Wall Type 4 (up to 10') – Design Case 2



DESIGN WORK SHEET

US Army Corps of Engineers®

Project: Mamaroneck & Sheldrake Rivers
Subject: SEISMIC LOADING CONSIDERATIONS

Sheet No.

Office: Designed by: AMP Date: Nov 2014 Checked by: Date:

ASCE 7-10 : Chapter 15, SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES

spectral response acceleration parameter at short periods, S_s = 25 (%g)
spectral response acceleration parameter at 1 sec periods, S_1 = 7 (%g)

site classification = D (ASCE 7-10, Chapter 20)
site coefficient, F_a = 1.6 (Table 11.4-1)
site coefficient, F_v = 2.4 (Table 11.4-2)

MCE spectral response acceleration for short periods, $S_{MS} = F_a S_s = (1.6)(0.25) = 0.4$
MCE spectral response acceleration for 1 sec periods, $S_{M1} = F_v S_1 = (2.4)(0.07) = 0.168$

design spectral acceleration parameters :

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.4) = 0.27$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.168) = 0.112$

risk category = III (Table 1.5-1)
importance factor, I_e = 1.25
seismic design category :

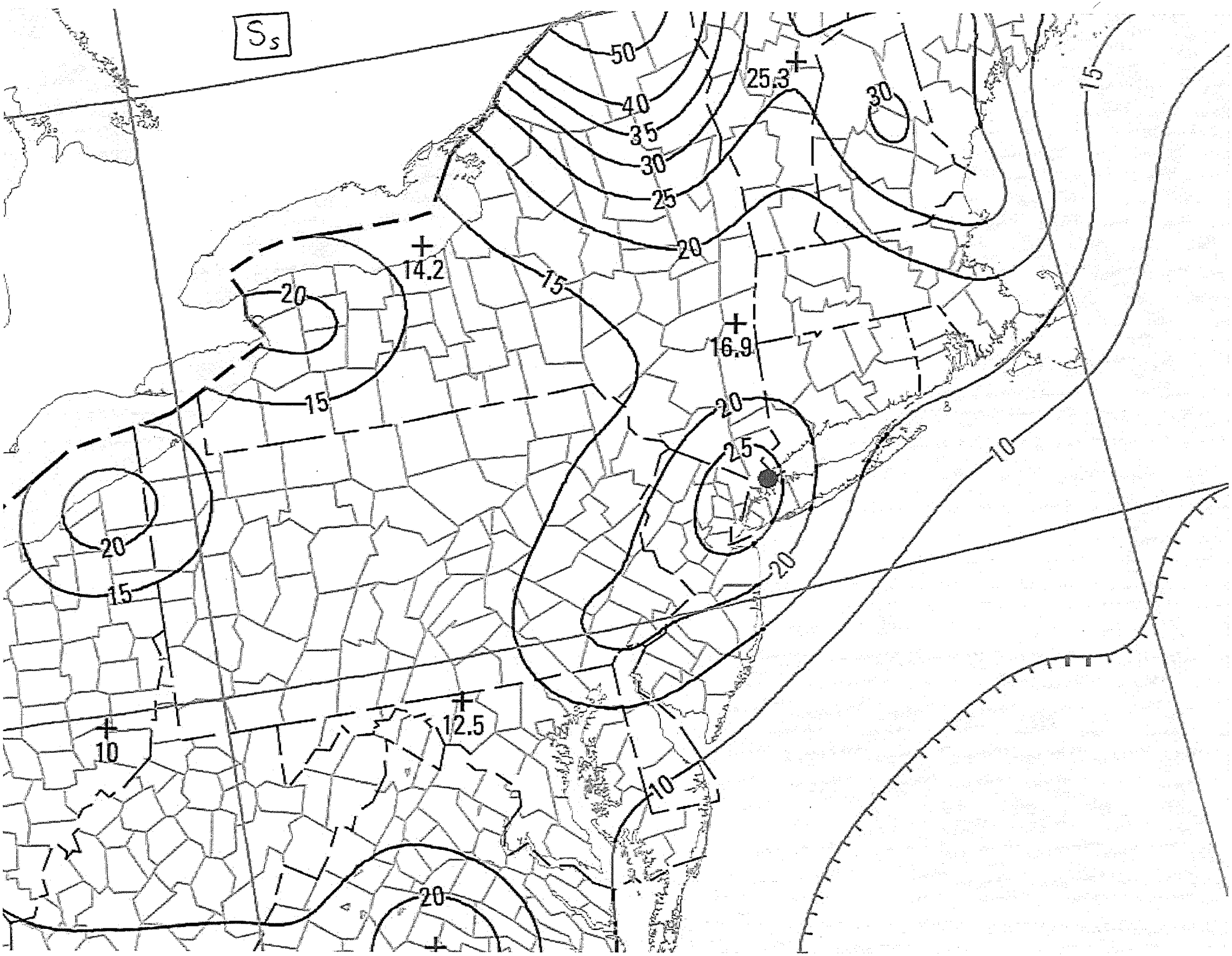
Table 11.6-1 : with $S_{DS} = 0.27$ → Design Category = B
Table 11.6-2 : with $S_{D1} = 0.112$ → Design Category = B
} Design Category = B

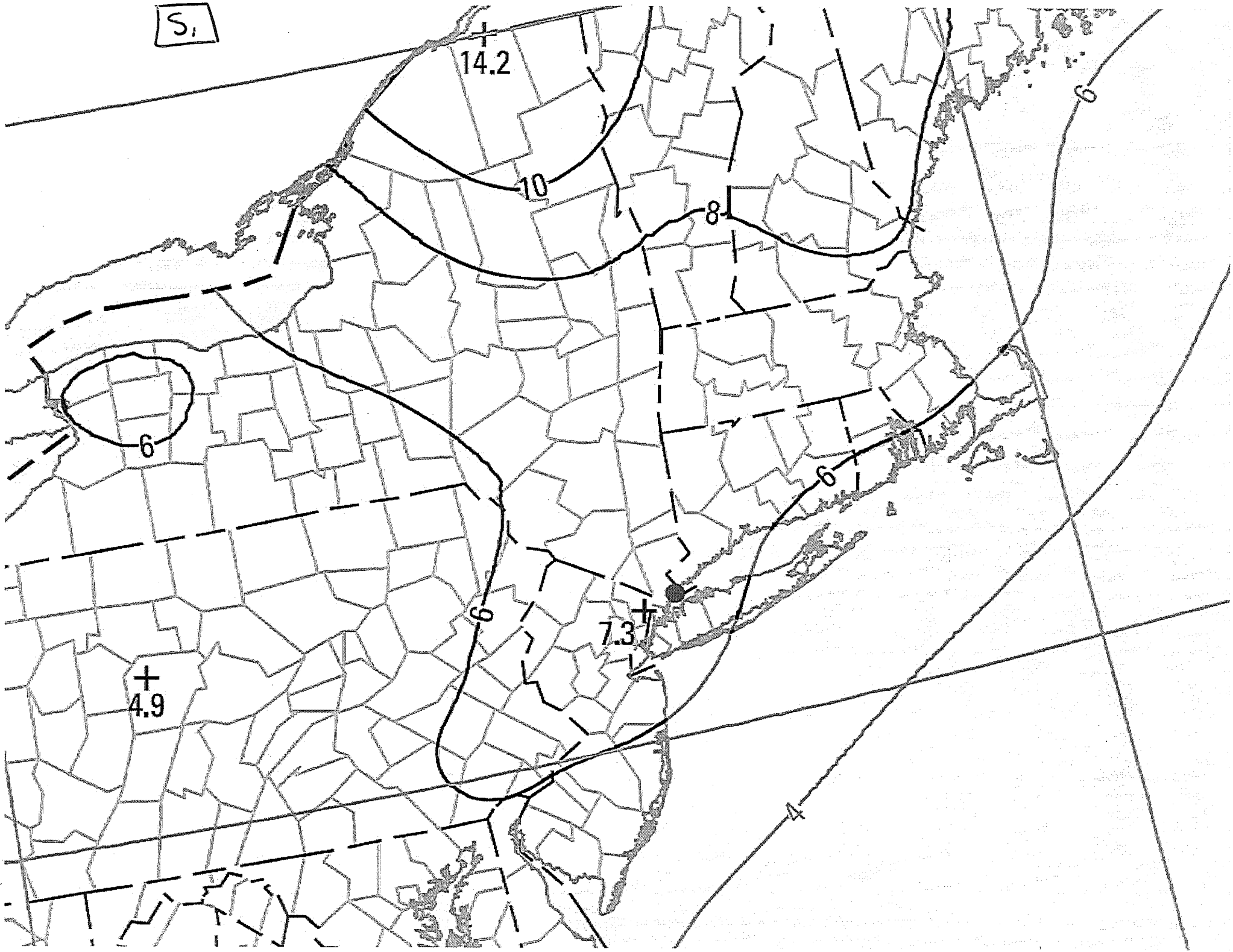
Table 15.4-2, Seismic Coefficients for Nonbuilding Structures not Similar to Buildings
- Steel and reinforced concrete distributed mass cantilever structures :
  - response modification coefficient, R = 2
  - system overstrength factor, Ω_0 = 2
  - deflection amplification factor, C_d = 2

seismic response coefficient, C_s

min $C_s = 0.044 S_{DS} I_e = 0.044(0.27)(1.25) = 0.015$

$$C_s = \frac{S_{DS}}{\left(R/I_e\right)} = \frac{0.27}{\left(2/1.25\right)} = 0.17$$


S_s
50
40
35
30
25
20
15
10
25.3
30
14.2
16.9
12.5


S1
14.2
10
8
6
6
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DESIGN WORK SHEET

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long period transition period, T_L = 6 sec (Figure 22-12)