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COMPTON DRIVE WASTEWATER TREATMENT PLANT FLOOD PROTECTION IMPROVEMENTS – PHASE 1

Study and Alternative Analysis

B&V PROJECT NO. 192653
B&V FILE NO. 41.0000

PREPARED FOR

City of Branson, Missouri

9 NOVEMBER 2016

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Executive Summary

In the previous December, the Compton Drive Wastewater Treatment Plant (CDWWTP) was near inundated with the flood event; Lake Taneycomo experienced high water levels as a result of the Army Corps of Engineers operation of the Table Rock Lack Dam. The high water surface elevation reached within inches of overtopping the eastern and western berm at the CDWWTP. In the past eight years, it is estimated that high water levels equivalent to a 50-year flood event have impacted the CDWWTP three times. The City retained Black & Veatch (B&V) to analyze the impact of various storm events that could cause physical damage to the CDWWTP assets or treatment processes that could cause untreated wastewater flows into Lake Taneycomo.

The scope of the Flood Protection Improvements Study included evaluating several storm events and selecting two storms to analyze the impacts at the CDWWTP. Once the storm events and corresponding flood elevations were determined, temporary and permanent flood protection alternatives would be assessed and cost estimates prepared at a conceptual level. The values presented in this report should be considered conceptual and will be further refined through the design process and with additional geotechnical exploration.

1.0 Introduction

1.1 STUDY OBJECTIVE

The objective of this conceptual study was to determine two storm events and define the corresponding flood elevations; and assess and evaluate flood protection alternatives for those storm events, which include temporary flood walls and permanent flood walls alternatives. At the conclusion of the conceptual study, the City of Branson will have viable alternatives to provide some level of flood protection for the Compton Drive WWTP.

1.2 EVALUATION OF EXISTING DATA

Part of the study's evaluation included choosing two storm events to analysis and determine the appropriate flood water elevations associated with the storm events. The approach to determining the storm events involved a step-wise process. First, the FEMA Maps of the area around the Compton Drive Wastewater Treatment Plant were studied to determine local FEMA Stations with established flow rates and high water surface elevations for various storm events. Two stations were identified, K River Station, 517.817 River Mile (located downstream of CDWWTP) and L River Station, 518.836 River Mile (located upstream of CDWWTP). The United States Army Corps of Engineers (USACE) developed these storm events, flow and water surface elevation values, associated with each River Station. FEMA adapted the USACE's results as the values found along the White River and Lake Taneycomo. The storm events' associated flowrate and high water surface elevation on the White River were estimated by interpolating those previously mentioned values at the CDWWTP. These values can be seen in Table 1.1.

Table 1-1 Army Core of Engineers Storm Events, Flow and Water Surface Elevations

STORM EVENT (YEAR)	FLOW (CFS)	WATER SURFACE ELEVATION - NAVD88 (FT)
19-YR	32,800	705.7
20-YR	36,900	706.5
24-YR	43,100	707.6
27-YR	48,200	708.4
29-YR	53,300	709.2
36-YR	63,500	710.6
42-YR	73,700	712.1
50-YR	84,000	713.4
59-YR	94,200	714.8
69-YR	104,000	716.0
83-YR	115,000	717.1
100-YR	127,000	718.5
125-YR	145,000	720.4

STORM EVENT (YEAR)	FLOW (CFS)	WATER SURFACE ELEVATION - NAVD88 (FT)
149-YR	166,000	722.4
167-YR	186,000	724.2
200-YR	206,000	726.0
240-YR	227,000	727.7
294-YR	257,000	730.1
385-YR	307,000	733.7
500-YR	358,000	736.9

CDWWTP drawings were provided to B&V. These drawings were examined to determine the potential areas were a high water event could impact each building and basin at the CDWWTP. The lowest personnel entry point, top of concrete, bottom of concrete and lowest bearing elevation were evaluated and summarized for each building. Additionally, the bottom of concrete, top of concrete, lowest bearing elevation and lowest personnel entry elevation were evaluated and summarized for the basin structures at the CDWWTP. If the presence of pressure relief valves was shown on the drawings, it was noted in the summary.

The City of Branson recently hired a firm to perform a GPS survey with sufficient resolution to produce 2-foot contours. The 2-foot contours have a NAD 1988 vertical datum. B&V used these existing contours to evaluate topography of the site in relation to various storm events. To verify the drawings datum, B&V spoke to the City of Branson to obtain a dozen GPS survey points. Many of these points included top of concrete of basins, top of slabs and the west and east side top of berms elevations of the CDWWTP. These survey points are approximately consistent with the drawings elevations, and the elevations from the drawings were used for the conceptual evaluation.

B&V determined that two storm events should be evaluated based on Compton Drive WWTP's critical elevations (buildings lowest entry point and top of basins). The 100-year storm event, which produces a flood elevation of El. 718.5 at CDWWTP, would be evaluated based on the regulatory significance. It was determined that any protection lower than the 50-year flood event El. 713.4 could breach critical elevations of many of CDWWTP treatment processes and that at least temporary flood protection should be provided. Appendix A shows the flood inundation maps for the 24-year, 50-year, 69-year, 100-year and 125-year flood events. Even at the 24-year flood event, a large portion of the CDWWTP is at the flood elevation of 707.6. At the 50-year flood event, almost all of the topo of the CDWWTP is under the flood elevation of 713.4 except for a few structures on the south portion of the CDWWTP. Additional discussion on flood protection improvements for the 50-year and 100-year flood event including dewatering wells, building/basin modifications and topography modifications can be found in Section 3.3 of this report.

For the purpose of the conceptual design, the 100-year storm event was used to evaluate the permanent flood wall alternative with a top of wall elevation at 720.0, which provides 1.5 feet of free board. The 50-year storm event was used to evaluate all of the temporary flood wall protection alternatives with a target top of wall elevation at 715.0, which provides 1.6 feet of freeboard. The top of wall elevations and the amount of freeboard required will be finalized during detailed design.

A graphical summary of the various elevations found at the CDWWTP for the building structures is shown in Figure 1.1; a graphical summary of the bottom of concrete and top of concrete can be found in Figure 1.2. For a key of building letters and basin numbers, refer to Appendix B.

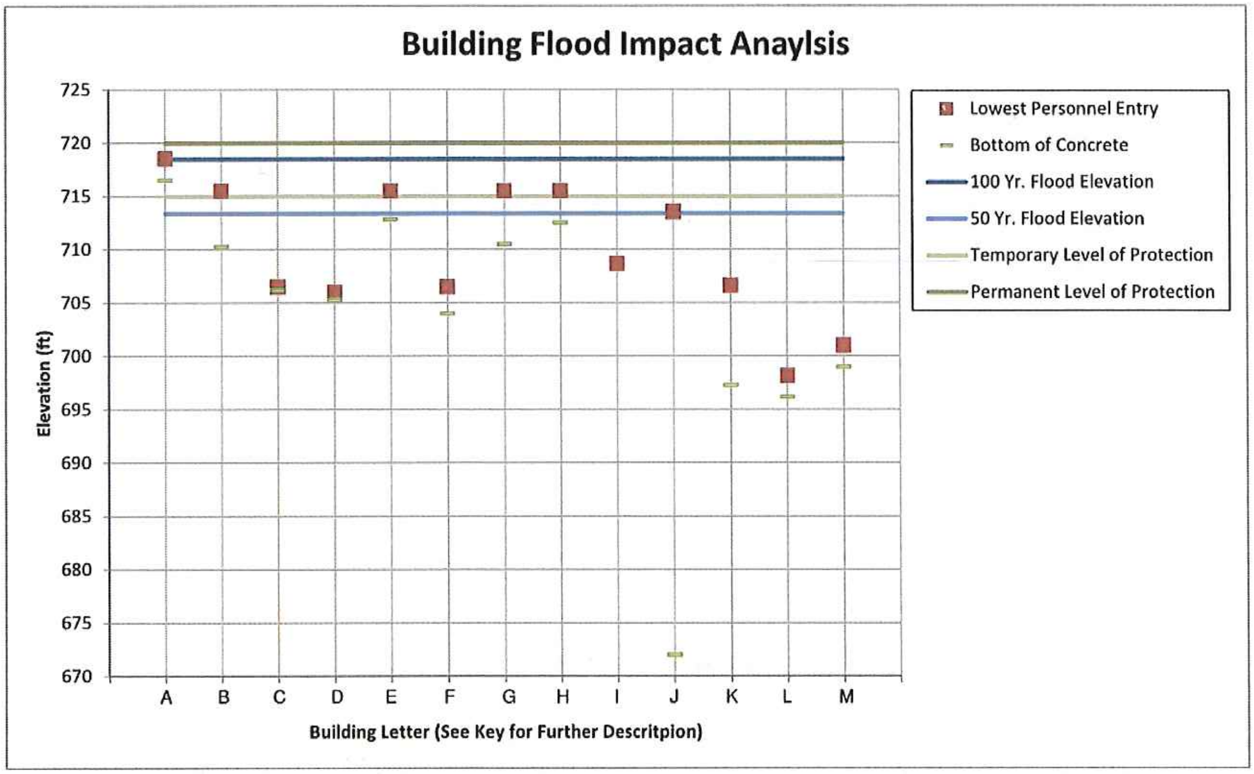

Figure 1-1 Graph of Building Elevations at Compton Drive WWTP

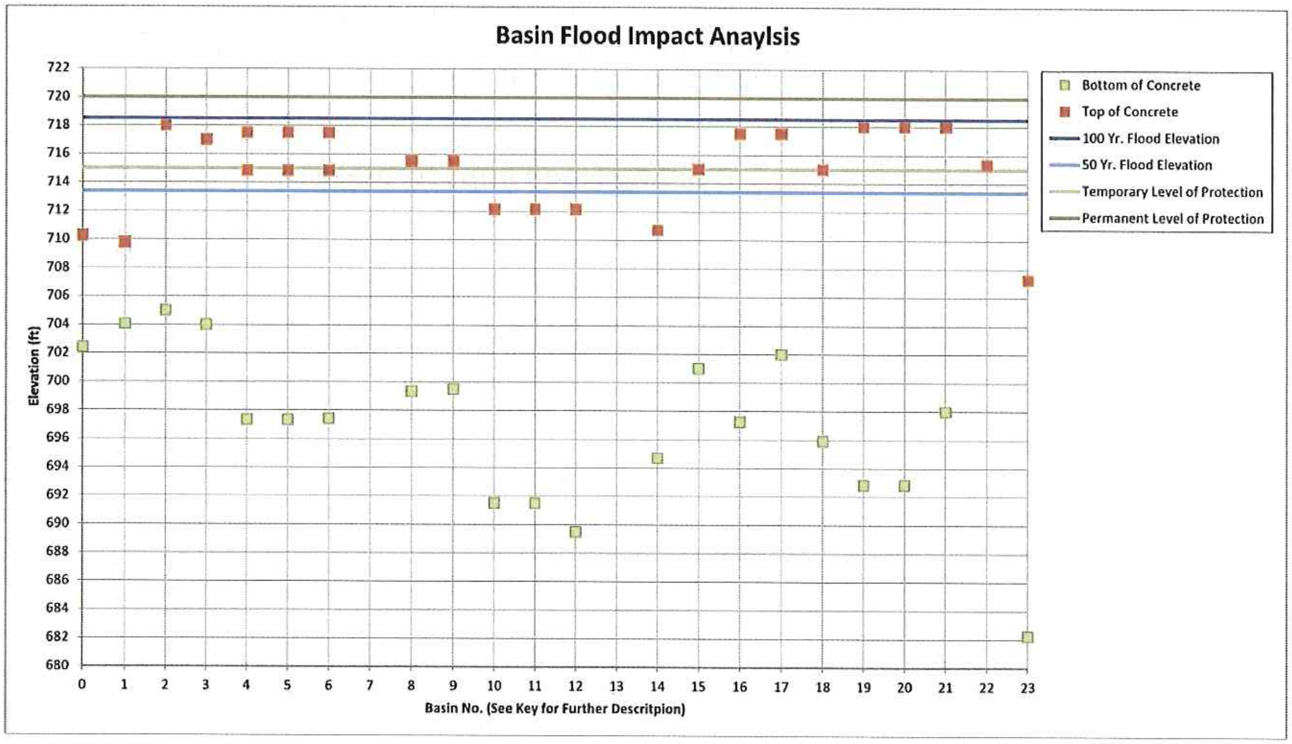

Figure 1-2 **Graph of Basin Elevations at Compton Drive WWTP**

The two previous figures show that all buildings and basins would be impacted by a 100-year flood event without a permanent flood wall. The several of the basins and buildings would be impacted by a 50-year flood event without temporary flood walls and dewatering wells.

1.3 PROPOSED FLOOD WALL ALIGNMENTS

Based on the key vertical elevations of the structures and basins and the flood inundation maps (Refer to Appendix A), a proposed route of the flood wall was determined around the CDWWTP. By examining all the CDWWTP's vertical elevations, it was determined that most of the facilities could be protected using a permanent flood wall; due to the steep elevations around the polishing filters, the polishing filters would not receive permanent flood protection. To simplify the routing of the temporary flood wall solution, the temporary flood wall protection would exclude the Administration Building, Screening Building and the Selector Basin. However, these previously mentioned structures will receive individual modifications to achieve flood protection up to the 50 year storm event. Additionally, the polishing filters would not be enclosed in the temporary flood protection but since the top of the filter basin is Elevation 715, the polishing filters are anticipated to be protected from a 50 year storm event. Refer to Figure 1.3.1 for the two alignments proposed for the 100-year and 50-year for flood protection events.

As part of the Flood Protection Conceptual Study, the permanent flood wall alternatives, the temporary flood wall alternatives, seepage and geotechnical information were analyzed further in this report. The influence of seepage for the flood wall alternatives is discussed in Section 2.0.

N
W E
S

0 125
Feet

1 inch = 125 feet

LEGEND

- Permanent Flood Wall
- Temporary Flood Wall
- Additional Improvements

Permanent & Temporary Flood Wall Alignments

Flood Control for Compton WWTP

PN 192653

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FloodWallAlignments October 07, 2016

2.0 Conceptual Level Groundwater Seepage Analysis

The conceptual level seepage evaluation relies entirely on available data from three test borings drilled by Anderson Engineering in 1991 and one test boring drilled by Olsson in 2008. Additional data for the characteristics of the alluvium and bedrock beneath the site are needed to perform a more formal evaluation of seepage from Lake Taneycomo toward the CDWWTP site as flooding occurs. The intent of the desktop study is to provide initial, planning-level estimates for the range of seepage that may occur at the site to assist in the future evaluation and design of flood protection measures. Using data from these four test borings along with available topographic information for the CDWWTP and flood elevations for Lake Taneycomo, the conceptual-level groundwater model includes the following major assumptions:

- Groundwater seepage is from the lake toward the plant site. The evaluation does not consider surface water overtopping the berm and infiltrating vertically through the soils at the site;
- Only two storm events, the 50-year and 100-year, were analyzed for Lake Taneycomo, which are discussed in Section 1.0;
- Steady-state groundwater flow modeling was performed for the desktop study; more complex transient groundwater flow modeling is recommended for detailed design of any flood protection options;
- Due to the Army Core of Engineer's control of the dam releases, it is acknowledge that the Core could make a large release of flow for time periods longer than a week;
- The City GIS contours were used to determine a ground elevation across the site of 706 feet. The target for the maximum groundwater elevation is assumed to be between 702 and 703 feet;
- Aquifer layers were assumed based on boring information available. These layers are summarized in Figure 2-1.
- Hydraulic conductivities were assumed as follows:
  - Upper silty/clayey soils, K_h = 10 feet per day,
  - Silty sand layer, K_h = 40 feet per day,
  - Sand and gravel layer, K_h = max. 850 feet per day,
  - Carbonate bedrock, K_h = 5 feet per day;
- The carbonate bedrock and soils to the west of the CDWWTP contribute insignificant quantities of groundwater;
- Vertical hydraulic conductivity is 10-percent of the horizontal;
- The effective FEMA hydraulic model of Lake Taneycomo was used to obtain the elevation of the lake bottom near the CDWWTP;
- The permeability of the sediments at the bottom of Lake Taneycomo is 1 foot per day and a thickness of 2 feet; additional analysis will be required for detailed design of flood protection solutions;
- No data was discovered for the bottom elevation or properties of the drainage ditch located to the west of CDWWTP;

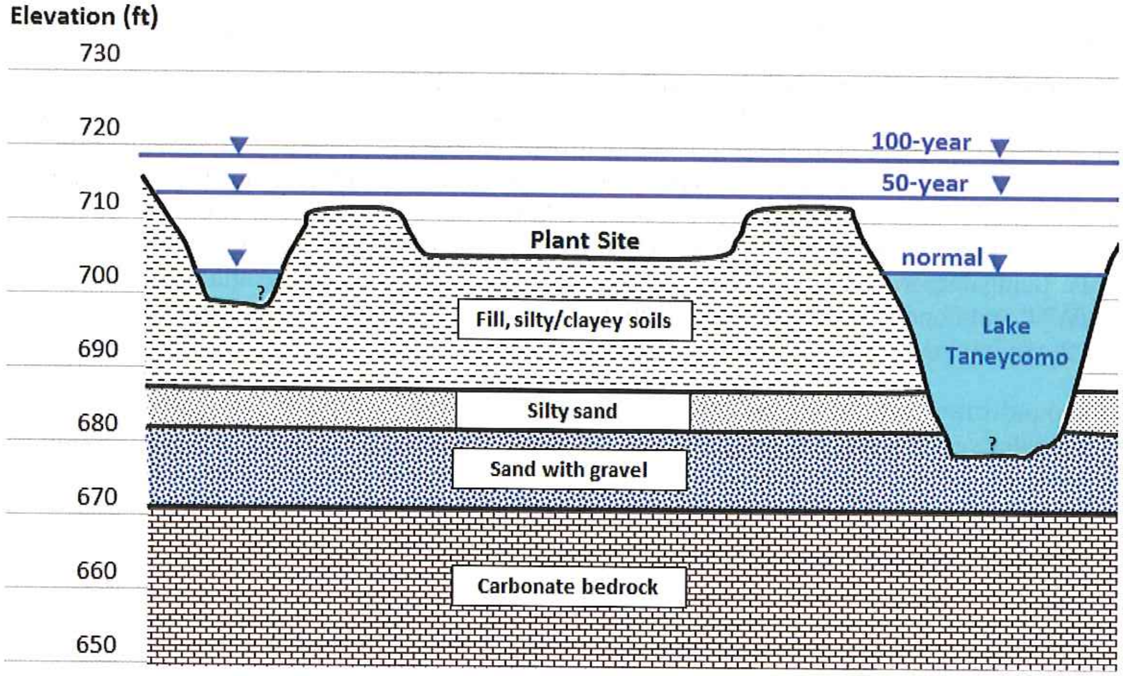

Figure 2-1 **Geologic Cross Sections at CDWWTP**

Several scenarios were modeled, using MODFLOW on Groundwater Modeling System 10.0, to evaluate the conceptual level groundwater seepage. A summary of the scenario results can be found in Table 2-1.

Table 2-1 Seepage Modeling Scenarios

SCENARIO	LAKE FLOOD ELEVATION (FEET)	HYDRAULIC CONDUCTIVITY OF SAND & GRAVEL LAYER (FT/DAY)	SHEET PILE CUTOFF SIMULATED? (ELEV OF BOTTOM, FEET)	TARGET MAXIMUM GROUNDWATER ELEVATION BENEATH PLANT SITE (FEET)	NUMBER OF WELLS, TOTAL PUMPING RATE (MGD)	RANGE OF INDIVIDUAL WELL PUMPING RATES (GPM)
1	718.5 (100-yr)	250	No (-)	702 – 703	9 wells, 3.74	200 – 550
2	718.5 (100-yr)	500	No (-)	702 – 703	9 wells, 6.05	350 – 900
3	713.4 (50-yr)	250	No (-)	702 – 703	9 wells, 2.52	100 – 400
4	713.4 (50-yr)	500	No (-)	702 – 703	9 wells, 4.10	150 – 650
5	718.5 (100-yr)	250	Yes (682, top sand & gravel)	702 – 703	9 wells, 3.17	175 – 450
6	718.5 (100-yr)	500	Yes (682, top sand & gravel)	702 – 703	9 wells, 5.40	300 – 850
7	713.4 (50-yr)	250	Yes (682, top sand & gravel)	702 – 703	9 wells, 2.23	125 – 300
8	713.4 (50-yr)	500	Yes (682, top sand & gravel)	702 – 703	9 wells, 3.67	200 – 550
9	718.5 (100-yr)	250	Yes (671, top bedrock)	702 – 703	6 wells, 0.35	40 each
10	718.5 (100-yr)	500	Yes (671, top bedrock)	702 – 703	6 wells, 0.35	40 each
11	713.4 (50-yr)	250	Yes (671, top bedrock)	702 – 703	6 wells, 0.22	25 each
12	713.4 (50-yr)	500	Yes (671, top bedrock)	702 – 703	6 wells, 0.22	25 each

An explanation of each scenario's results can be found in Appendix C. Records of historical pumping rates from several existing dewatering wells and groundwater levels within the plant were not available to calibrate the conceptual groundwater model results. A factor of safety is required for these initial findings to account for uncertainties due to a general lack of detailed geotechnical data. It was assumed that analysis methods can be expected to have uncertainty that ranges from approximately 0.5 to 2. During conceptual phases of a project, a factor of safety of 1.75 to 2 is often used. Additional site-specific aquifer testing will be required to determine several key parameters such as the hydraulic characteristics of the soils, hydraulic connection of the lake with the soils and interaction between the soils and the Ozark Aquifer beneath the site and toward the west of the site. Recommended testing includes borings with sieve analyses across the plant site, several hydraulic interval tests, and a full-scale constant rate pumping test. The field data would be used to refine this conceptual-level groundwater flow model in order to optimize the number and configuration of the dewatering wells and to aid in the design of the wells and any cutoff system that may be selected.

This preliminary analysis illustrates the benefits of having a seepage cutoff such as sheet piles. A sheet pile wall will have lower operational and maintenance costs than a system of dewatering wells. If sheet pile is installed properly to bedrock such that the seepage through the interlocking joints is minimized, it would significantly reduce the concerns about the uncertainty in the soil stratigraphy and heterogeneities in the hydraulic and storage characteristics of the soil layers. Due to the apparent proximity of the bottom of Lake Taneycomo with bedrock, upflow from carbonate bedrock from below any cutoff system could still be a concern; investigation of the degree of weathering of the uppermost bedrock will need to be considered and used to refine the estimates of underflow for the design of any seepage cutoff measure that may be selected.

The number of wells and pumping capacity required for each well were used for costing the flood protection alternatives in the proceeding sections of this report. Section 3.0 explains the permanent and temporary flood protection alternatives identified on the CDWWTP site.

3.0 Evaluation of Flood Protection Alternatives

As part of the Branson Flood Control Study, three temporary alternatives and one permanent (with varying materials of construction) flood protection alternative were investigated. The temporary alternatives have a varying product life up and would provide flood protection up to a 50 year flood event. The top of the wall of the temporary alternatives would be El. 715 providing a freeboard of approximately 1.6 feet. The permanent alternatives would consist of sheet piling a flood wall with access to the Compton Drive WWTP through flood gates. The top of wall of the permanent alternative would be at El. 720 and would provide flood protection for the 100 year flood event with 1.5 feet of freeboard. The flood protection alternatives are further discussed below.

3.1 PERMANENT FLOOD PROTECTION ALTERNATIVES

3.1.1 Modifications to Existing Structures

No modifications are necessary for the buildings and basins within the permanent flood protection alignment. Refer to Figure 1.3.1 for the proposed alignment. The polishing filters are currently outside the limits of the permanent and temporary flood protection alternatives alignments. These filters are discussed in more detail in Section 3.3.

3.1.2 Sheet Piling

The existing topography from the GIS 2-foot contours and the previously mentioned four existing borings were used to develop a profile of the geotechnical information at or near the permanent flood wall alignment. Based on the limited geotechnical information available and the estimated length of the flood wall, additional geotechnical borings and laboratory testing along the proposed alignment is recommended prior to detailed design.

The wall elevation, flood elevation, ground surface profile on both sides of the flood wall, subsurface layers and geotechnical properties of each layer were used in the desktop analysis of the permanent flood wall. CWALSHT was used to model the flood wall at the various cross sections developed from the City's 2-foot contours. The model uses soil mechanics procedures to determine the required depth of penetration of a new flood wall and accesses the factors of safety of an existing wall. A final design product is reached when the values of wall penetration produce a pressure distribution where the sum of moments about any point and the sum of all horizontal forces are equal to zero. The penetration depth of the flood wall was increased by 30 percent to provide additional factor of safety against overturning. Based on the modeling, the minimum depth required is at El. 696, which produces a 24-foot tall flood wall. The flood wall could also be taken to bedrock, which produces a 49-foot tall flood wall, which would further reduce groundwater seepage during a flood event. Refer to Appendix D for more detail on the geotechnical analysis of the permanent flood wall.

The results above are based on using a Skyline Steel NZ19, Grade 60 sheet pile or equivalent. To prohibit corrosion it is recommended that the sheet piles be coated with an elastomeric sealant. Any additional, aesthetics would be at the discretion of the owner and are not included in the conceptual cost estimate. Alternatively, a composite sheet pile such as CMI Ultra Composite Sheet Pilling and Piles UC-95 could be used. The composite sheet pile would not require additional coating for corrosion protection. However, the composite sheet pile would be more susceptible to vandalism and less fire resistant than that the traditional metal sheet pile. Both types of sheet piles, could be driven to bedrock or El. 696 (El. 696 is dependent on the metal sheet pile calculations and additional computations for the composite sheet pile would be required during design) as noted above.

The site would be assessable through the south end through one flood gate near the Screening Building. Another flood gate on the north side near the Sludge Thickening Building would provide access to the polishing filters and CDWWTP docking station. The flood gates would extent just along the existing road and, similar to the sheet piles, would provide a flood protection up to an El. 720. Four types of flood gates were assessed during this study. The Presray Model CG3S and FB44 Preray Model CG3S is a sliding flood panel with a compression gasket, and Model FB44 is a side hinged aluminum panel with inflatable seals. Both Presray flood gate models would have to be encased in concrete. Hydrogate stop logs would be another option instead of a true gate. The logs would consist of approximately two to three carbon steel stacked stop logs. The guides would be embedded in a concrete structure. The Floodbreak Vehicular flood gate was also analyzed. This gate sits beneath the entrance and is flush with the roadway. The gate passively rises up from the ground when a flood event occurs. Product data and cut sheets of all the flood gate options can be found in Appendix E. To reduce seepage from ground water, all flood gate models will be positioned and permanently attached to the sheet pile below grade and will be attached to the adjacent sheet pile on the sides.

3.2 TEMPORARY FLOOD PROTECTION ALTERNATIVES

Several types of temporary flood protection alternatives exist, which are viable for protecting the Compton Drive WWTP up to the 50-year storm. The following temporary flood protection options were evaluated: TrapBag, HESCO Barrier and Muscle Wall. Product data and cut sheets of all the temporary flood protection alternatives can be found in Appendix E.

3.2.1 TrapBag

The trapbag system uses synthetic bags that are sewn accordion style together and have a rigid partition between the walls of the cells. For installation, the trapbag system is constructed so setup and placement can be completed using a front-end loader. The bags are then filled with granular material or flowable fill at the site and can be used for a permanent or semi-permanent installation.

The design life of a trapbag system is approximately four to five years per the manufacturer's recommendation. These bag systems are subject to ultraviolet degradation, though it is possible to extend the design life to 10 years by installing an ultraviolet guard or double front fabric layer on the top of the bags; adding the guard, will also prevent storm water from eroding away the fill material during a flood event. The trapbags are not restrained and could sustain damage from flood debris or plant side impacts. When using granular fill material, the trapbag system could be subject to internal settlement.

Trapbags come in heights of 2 feet, 4 feet and 6 feet. The footprint of the trap bags would limit the clearance on the perimeter roadway. The footprint is 8 feet wide for a 6 feet tall trapbag, 5 feet wide for a 4 feet tall trapbag and approximately 3 feet wide for a 2 feet tall trapbag.

The 4 feet trapbags with UV guards are approximately 55 dollars per linear ft (including installation and assuming granular fill material).

The trapbag system does not provide cutoff of under seepage through the subgrade. It is recommended that a dewatering well system be used in conjunction with this temporary flood protection system.

For the proposed temporary alternative route, extra trapbags would be secured to the end cell that will be able to close the road openings during a flood event. For anchorage, it is recommended that the end cell be filled with flowable fill. When a flood event occurs, the City would use the secured trapbags to close the opening and fill the cells. At the end of the storm event, the City would have to break, tear and destroy the trapbags to reopen roadway access. A new set of trapbags would then have to be sewn to the end cell and secured until the next flood event occurs. Section 4.0 explains the conceptual costs associated with the Trapbag system for temporary flood protection. Refer to Figure 3-1 and 3-2 for photos of the Trapbag system.

Figure 3-1 Photo of Trapbags with the UV guards installed

Figure 3-2 Photo of 4 ft. Trapbag with Granular Fill

3.2.2 HESCO Barrier

The HESCO barrier consist of synthetic bags that can be filled with granular material, sand, earth, crush rock or etc. and is protected by a welded wire mesh basket. The welded wire mesh has a one year warranty and the geotextile fabric has a five year life. The product life of the geotextile can be extended by coating it with elastomeric paint. Similarly to the trapbag system, the barrier blocks are sewn accordion style together and have a rigid partition between the walls of the cells. The HESCO barrier can be used for a permanent or semi-permanent installation.

The HESCO barrier is more resistant to external impacts than the trap bag system. Unfortunately, the HESCO barrier blocks are subject to internal settlement of the fill, which results in a loss in flood protection elevation.

The MIL-1 (recommended for this project) HESCO barrier blocks come in a height of 4'-6" and can be stacked to achieve an 8 feet high level of protection. The footprint of the trap bags would limit the clearance on the perimeter roadway. The footprint for the barrier blocks is roughly 3'-6".

The 4'-6" HESCO barrier system with the elastomeric paint are approximately 48 dollars per linear foot (including installation and assuming granular fill material).

The HESCO Barrier system does not provide cutoff of under seepage through the subgrade. It is recommended that a dewatering well system be used in conjunction with this temporary flood protection system.

For the proposed temporary alternative route, extra HESCO barrier blocks would be secured to the end cell that will be able to close the road openings during a flood event. When a flood event occurs, the City would use the secured HESCO barrier blocks to close the opening and fill the cells. At the end of the flood event, the City would have to break, tear and destroy the barrier bags to reopen roadway access. A new set of barrier would then have to be sewn to the end cell and secured until the next flood event occurs. Section 4.0 explains the conceptual costs associated with the HESCO Barrier system for temporary flood protection. Refer to Figure 3-3 and 3-4 for photos of the HESCO Barrier system.

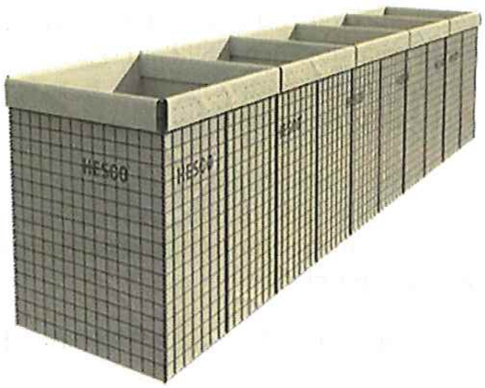

Figure 3-3 **Rendering of the HESCO barriers.**

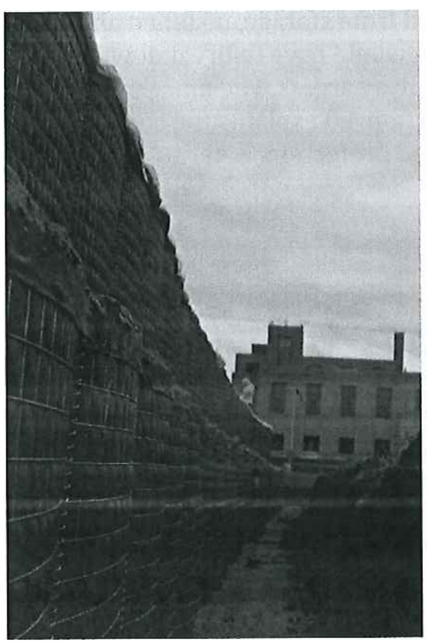

Figure 3-4 **Photo of HESCO barrier blocks used for Council Bluffs IA waterworks.**

3.2.3 Muscle Wall

The muscle wall is constructed from various low-density polyethylene block systems that can be deployed at the site and can be used for a permanent or semi-permanent installation. The systems utilize interconnecting blocks designed with a stabilizing geometry. The muscle wall is more resistant to external impacts and internal settlement (walls may need to be refilled with water due to evaporation). They have a warranty life of ten years without the UV linear, but the product life of the muscle wall can be extended by using a UV liner.

The muscle wall has various alternatives for installation. Installation for the site would include a combination of trench installation or liner deployment on asphalt. The trench installation includes a 16-inch deep trench that would be backfilled with soil once the liner is laid out. The linear deployment on asphalt consists of spraying foam on the ground under the liner. Both installation requirements require use of sandbags to aid in the stability of the muscle wall system.

The walls are available in heights of four or eight feet. The four foot muscle walls are to be filled with water at site and the eight feet muscle walls are prefilled with foam. The footprint of the muscle wall would limit the clearance on the perimeter roadway. The footprint is roughly 2'-6" for a 4 feet tall muscle wall and roughly 4 feet for an 8 feet tall muscle wall

The four feet muscle walls are approximately $223 per linear feet (excluding fill material).

The muscle wall system does not provide cutoff of under seepage through the subgrade. It is recommended that a dewatering well system be used in conjunction with this temporary flood protection system.

For the proposed temporary alternative route, extra muscle walls would have to be stored onsite. During a flood event, the muscle walls would have to be removed from storage, installed and filled (depending on the height). At the end of the flood event, the City would have to breakdown and drain (depending on the height) the muscle walls to reopen roadway access. The muscle walls would go back into storage until the next flood event occurs. Section 4.0 explains the conceptual costs associated with the Muscle Wall system for temporary flood protection. Refer to Figure 3-5 for photos of the HESCO Barrier system.

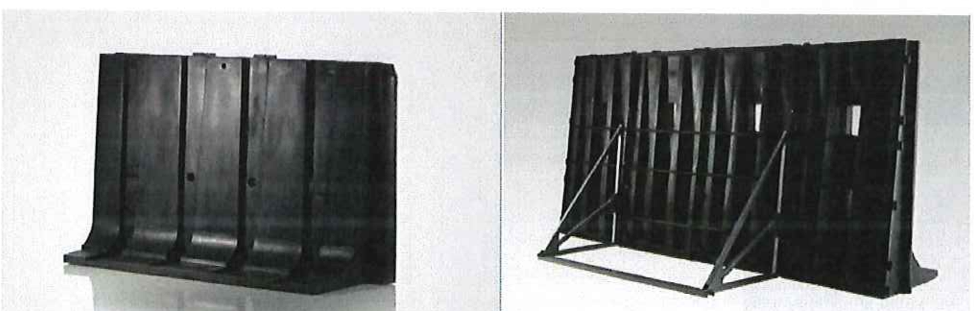

Figure 3-5 Photos of the 4 ft. and 8 ft. Muscle Wall, respectively.

3.3 OTHER CONSIDERATIONS FOR FLOOD PROTECTION

Based on the Drawings titled *Compton Drive WWTP Peak Flow Improvements*, 2009, the 100-year flood elevation is shown at 718.00. The Effluent Pump Station is capable of pumping flow up to El. 718.00 based on the hydraulic profile found within this drawing set. Verification of the pumping head at the Effluent Pumping Station should be conducted to ensure that flow can be conveyed out of the CDWWTP when the Lake is at the 718.50 flood elevation.

As mentioned in previous discussion within this report, the polishing filters top of concrete are at El. 715, which will protect it from the 50 year flood elevation but cannot be protected using a flood wall for the 100-year flood event due to the steep vertical drops in elevation. The polishing filters could remain in operation beyond the 50-year flood event, even with the lower hydraulic gradeline through the facility. This assumes that the wells within the temporary or permanent flood wall are capable of keeping groundwater out of the treatment process. Beyond the 59-year storm event, the water from the Lake will be overtopping the basin walls and entering the treatment process. After the surface water elevation from Lake Taneycomo recedes, the filter media and the polishing filters themselves will need to be replaced. This replacement cost is estimated to be approximately $900,000. Other options such as relocating the polishing filters within the plant or raising the filter top of concrete could be evaluated during a preliminary or detailed design phase.

For the temporary flood protection options, the south portion of the CDWWTP site is not protected with a temporary flood wall. The structures affected by this include: Administration Building, Screening Building and the Selector Basin. Conduit penetrations, doors, hatches and windows will need to be flood protected at least to the El. 715.0. Further discussion on the conceptual costs associated with these south structures is located in Section 4.0.

Insert regulatory issues.

4.0 Summary of Costs and Recommendations

A refined conceptual, order of magnitude cost estimate was developed for the sheet piling alternatives and temporary flood protection alternatives. Preliminary material quantities for each site were developed based on the typical wall sections, the GIS based ground surface elevation, top of flood protection elevation, site parameter and any other miscellaneous considerations to the CDWWTP site.

The cost estimates include the following assumptions:

- Costs are in 2016 dollars; 13-percent Contractor General Requirements; 35-percent Contingency; 20-percent Engineering and Design Fees;
- Mobilization of 3 months for Permanent and Temporary Alternatives; Sitework is 10-percent of the construction subtotal cost; permanent alternatives provide a level of flood protection to El. 720, and temporary alternatives provide a level of flood protection to El. 715;
- Route lengths, as noted previously in the report, are 1,650 feet and 2,200 feet, respectively, for the temporary and permanent alternatives;
- Floodgate cost vary by model– Presray Model CG3S used in cost estimate;
- Sheet piling cost assume use of the Skyline NZ19 for the metal sheet pile and UC-95 for the composite sheet pile;
- Dewatering costs assume a factor of safety of 1.5; PVC Wells are 50 feet in length with 15 feet stainless steel screens (Stainless steel wells and slotted PVC wells are also viable options); Sheet piles to bedrock use 6–inch diameter wells and sheet piles to El. 696 assume 24-inch diameter wells (12 inch diameter wells are a viable alternative pending further investigation); a standby well is not included in the cost estimates below;
- Building flood protection improvements assume closure of conduit penetrations and use of a floodbreak pedestrian gate 2'-0" high (refer to Appendix E for pictures) at each of the pedestrian doors and overhead doors in the Administration Building and Screening Building;
- The following costs are not included in the conceptual costs: permitting; additional surveying and geotechnical investigations; and hydrological investigation.

For a summary of costs for the sheet piling alternatives, which protect up to the 100-year storm, refer to Table 4.1 through Table 4.4 below:

Table 4-1 Metal Sheet Pile to Bedrock Conceptual Cost Estimate

METAL SHEET PILE TO BEDROCK CONCEPTUAL COST ESTIMATE				
Item	Quantity	Unit	Cost	Total
Mobilization	1	LS	$70,000	$70,000
Sitework	1	LS	$546,000	$546,000
Sheet pile (Material, Installation and Elastomeric Sealant)	1	LS	$3,606,000	$3,606,000
Sheet pile Aesthetic (At the Discretion of the Owner)	0	LS	$0	$0
Flood Gates	2	each	$777,000	$1,553,000
Dewatering Improvements (6 Wells)	1	LS	$302,000	$302,000
Construction Subtotal				$6,077,000
Contractor General Requirements			13%	$790,000
Contingency			35%	$2,127,000
Engineering and Design			20%	$1,215,000
Total Construction Cost				$10,209,000

Table 4-2 Composite Sheet Pile to Bedrock Conceptual Cost Estimate

COMPOSITE SHEET PILE TO BEDROCK CONCEPTUAL COST ESTIMATE				
Item	Quantity	Unit	Cost	Total
Mobilization	1	LS	$70,000	$70,000
Sitework	1	LS	$522,000	$522,000
Sheet pile (Material and Installation)	1	LS	$3,370,000	$3,370,000
Sheet pile Aesthetic (At the Discretion of the Owner)	0	LS	$0	$0
Flood Gates	2	each	$777,000	$1,553,000
Dewatering Improvements (6 Wells)	1	LS	$302,000	$302,000
Construction Subtotal				$5,817,000
Contractor General Requirements			13%	$756,000
Contingency			35%	$2,036,000
Engineering and Design			20%	$1,164,000
Total Construction Cost				$9,773,000

Table 4-3 Metal Sheet Pile to El. 696 Conceptual Cost Estimate

METAL SHEET PILE TO EL. 696 CONCEPTUAL COST ESTIMATE				
Item	Quantity	Unit	Cost	Total
Mobilization	1	LS	$70,000	$70,000
Sitework	1	LS	$414,000	$414,000
Sheet pile (Material, Installation and Elastomeric Sealant)	1	LS	$1,940,000	$1,940,000
Sheet pile Aesthetic (At the Discretion of the Owner)	0	LS	$0	$0
Flood Gates	2	each	$777,000	$1,553,000
Dewatering Improvements (9 Wells)	1	LS	$647,000	$647,000
Construction Subtotal				$4,624,000
Contractor General Requirements			13%	$601,000
Contingency			35%	$1,618,000
Engineering and Design			20%	$924,000
Total Construction Cost				**$7,767,000**

Table 4-4 Composite Sheet Pile to EL. 696 Conceptual Cost Estimate

COMPOSITE SHEET PILE TO EL. 696 CONCEPTUAL COST ESTIMATE				
Item	Quantity	Unit	Cost	Total
Mobilization	1	LS	$70,000	$70,000
Sitework	1	LS	$383,000	$383,000
Sheet pile (Material and Installation)	1	LS	$1,626,000	$1,626,000
Sheet pile Aesthetic (At the Discretion of the Owner)	0	LS	$0	$0
Flood Gates	2	each	$777,000	$1,553,000
Dewatering Improvements (9 Wells)	1	LS	$647,000	$647,000
Construction Subtotal				$4,279,000
Contractor General Requirements			13%	$556,000
Contingency			35%	$1,497,000
Engineering and Design			20%	$856,000
Total Construction Cost				**$7,188,000**

Tables 4-5 through 4-7 show the costs associated with the temporary flood wall options, which protect up to a 50-year storm.

Table 4-5 Muscle Wall Conceptual Cost Estimate

MUSCLE WALL CONCEPTUAL COST ESTIMATE				
Item	Quantity	Unit	Cost	Total
Mobilization	1	LS	$15,000	$15,000
Sitework	0	LS	$0	$0
Muscle Wall (Includes UV Guard Liner)	1	LS	$203,000	$203,000
Installation	1	LS	$27,000	$27,000
Fill Material	1	LS	$900	$900
Sand Bags	2750	Each	$50	$138,000
Foundation Support	0	LS	$0	$0
Dewatering Well Improvements (Include Sitework and Mobilization for this Work)	1	LS	$744,000	$744,000
Flood Gate (Not Included)	0	LS	$0	$0
Building Flood Protection Improvements (Administration Building and Screening Building)	1	LS	$101,000	$101,000
Construction Subtotal				$1,230,000
Contractor General Requirements		13%		$160,000
Contingency		35%		$430,000
Engineering and Design		20%		$246,000
Total Construction Cost				**$2,066,000**

Table 4-6 Trap Bags Conceptual Cost Estimate

TRAP BAGS CONCEPTUAL COST ESTIMATE				
Item	Quantity	Unit	Cost	Total
Mobilization	1	LS	$13,000	$13,000
Sitework	0	LS	$0	$0
Trap Bags (Includes UV Guards)	1	LS	$42,000	$42,000
Installation	1	LS	$5,000	$5,000
Fill Material	1	LS	$43,600	$43,600
Foundation Support	0	LS	$0	$0
Dewatering Well Improvements (Include Sitework and Mobilization for this Work)	1	LS	$744,000	$744,000
Flood Gate (Not Included)	0	LS	$0	$0
Building Flood Protection Improvements (Administration Building and Screening Building)	1	LS	$101,000	$101,000
Construction Subtotal				$950,000
Contractor General Requirements		13%		$123,000
Contingency		35%		$332,000
Engineering and Design		20%		$190,000
Total Construction Cost				**$1,595,000**

Table 4-7 HESCO Barrier Conceptual Cost Estimate

HESCO BARRIER CONCEPTUAL COST ESTIMATE				
Item	Quantity	Unit	Cost	Total
Mobilization	1	LS	$16,000	$16,000
Sitework	0	LS	$0	$0
HESCO Barrier (Including Elastomeric Coating)	1	LS	$71,700	$71,700
Installation	1	LS	$7,000	$7,000
Fill Material	1	LS	$116,000	$116,000
Foundation Support	0	LS	$0	$0
Dewatering Well Improvements (Include Sitework and Mobilization for this Work)	1	LS	$744,000	$744,000
Flood Gate (Not Included)	0	LS	$0	$0
Building Flood Protection Improvements (Administration Building and Screening Building)	1	LS	$101,000	$101,000
Construction Subtotal				$1,056,000
Contractor General Requirements		13%		$137,000
Contingency		35%		$370,000
Engineering and Design		20%		$211,000
Total Construction Cost				**$1,774,000**

APPENDIX A

FLOOD INUNDATION MAPS

LEGEND

Hydrants	Manholes
Large Pole	Catch Basin
Light Pole	Storm Inlet
Utility Pole	Storm Outfall
Utility Light Pole	Tanks
Water Valve	Fence

Natural Gas Lines
Sewer Mains
Storm Mains
Hydrology Lines
10 Ft Contours
Flood Event Inundation Zone

N W E S

0 60 120
Feet

1 inch = 240 feet

Inundation Limits for 24-Year Flood Event

Flood Control for Compton WWTP

192653

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LEGEND

- Hydrants
- Large Pole
- Light Pole
- Utility Pole
- Utility Light Pole
- Water Valve
- Manholes
- Catch Basin
- Storm Inlet
- Storm Outfall
- Tanks
- Fence
- Natural Gas Lines
- Sewer Mains
- Storm Mains
- Hydrology Lines
- 10 Ft Contours
- Flood Event Inundation Zone

N W E S

0 60 120 Feet

1 inch = 240 feet

Inundation Limits for 50-Year Flood Event

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LEGEND

N
W E
S

0 60 120
Feet

1 inch = 240 feet

- Hydrants
- Large Pole
- Light Pole
- Utility Pole
- Utility Light Pole
- Water Valve
- Manholes
- Catch Basin
- Storm Inlet
- Storm Outfall
- Tanks
- Fence
- Natural Gas Lines
- Sewer Mains
- Storm Mains
- Hydrology Lines
- 10 Ft Contours
- Flood Event Inundation Zone

Inundation Limits for 69-Year Flood Event

Flood Control for Compton WWTP

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LEGEND

Hydrants	Manholes	Natural Gas Lines
Large Pole	Catch Basin	Sewer Mains
Light Pole	Storm Inlet	Storm Mains
Utility Pole	Storm Outfall	Hydrology Lines
Utility Light Pole	Tanks	10 Ft Contours
Water Valve	Fence	Flood Event Inundation Zone

N W E S

0 60 120
Feet

1 inch = 240 feet

Inundation Limits for 100-Year Flood Event

Flood Control for Compton WWTP

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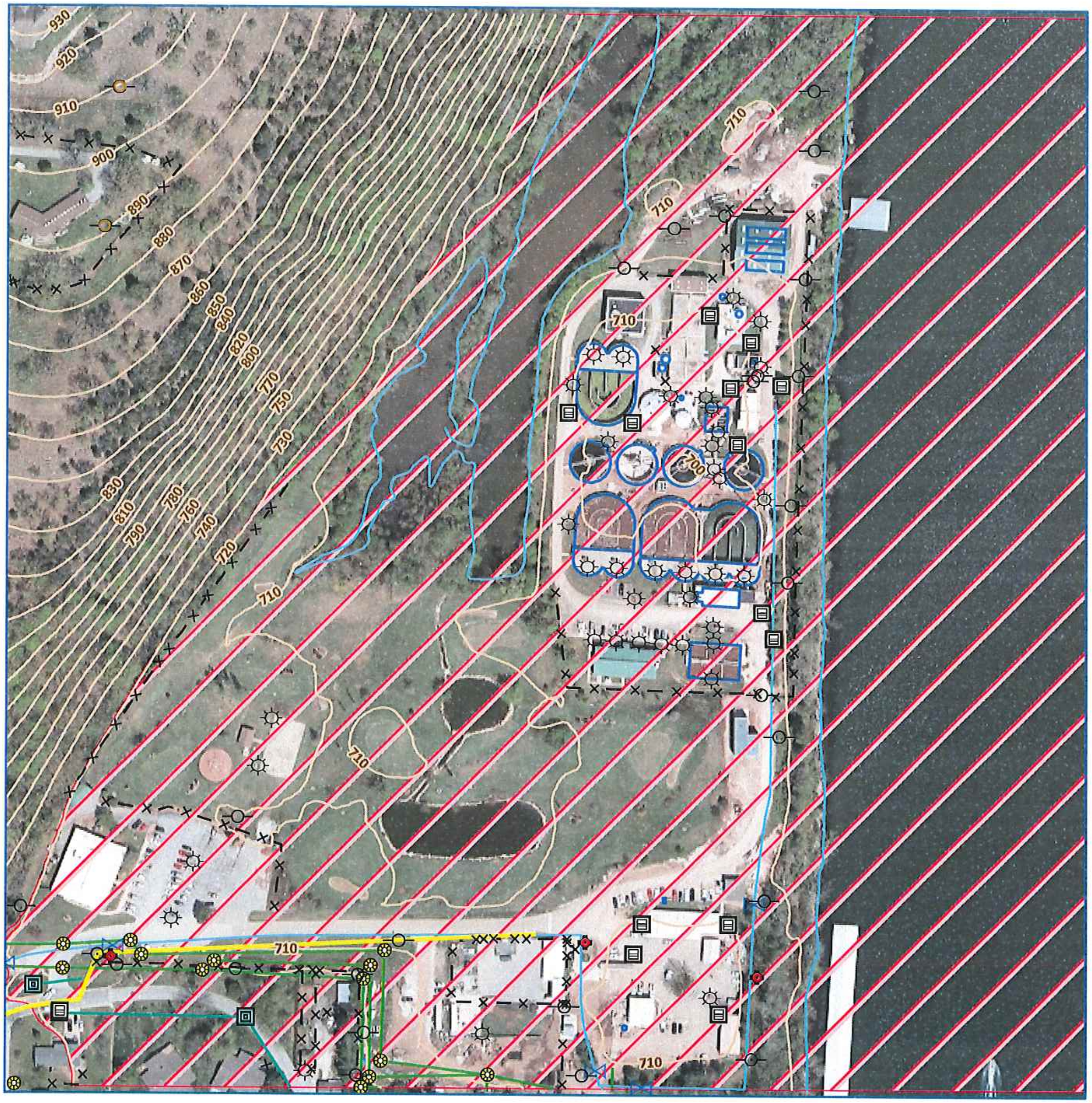

LEGEND

Hydrants	Manholes	Natural Gas Lines
Large Pole	Catch Basin	Sewer Mains
Light Pole	Storm Inlet	Storm Mains
Utility Pole	Storm Outfall	Hydrology Lines
Utility Light Pole	Tanks	10 Ft Contours
Water Valve	Fence	Flood Event Inundation Zone

N W E S

0 60 120
Feet

1 inch = 240 feet

Inundation Limits for 125-Year Flood Event

Flood Control for Compton WWTP

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APPENDIX B

BUILDING & BASIN KEY

N
W E
S

0 125
Feet
1 inch = 125 feet

LEGEND

Permanent Flood Wall
Temporary Flood Wall
Additional Improvements

Permanent & Temporary Flood Wall Alignments

Flood Control for Compton WWTP

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Appendix B - Building and Basin Key

BUILDING/ STRUCTURE	BUILDING/ STRUCTURE DESIGNATED NUMBER	LOWEST PERSONNEL ENTRY(*)	APPROXIMATE BOTTOM OF CONCRETE OF STRUCTURE (**)	PILE BEARING ELEVATION	ADDITIONAL STRUCTUAL OBSERVATIONS THAT COULD BE POTENTIALLY IMPACTED BY FLOOD EVENT
BUILDINGS					
Electrical Building and Generator	A	719'-0"	716'-6"	*(1)*	-
Screening Building	B	715'-6"	710'-3"	*(1)*	-
Equipment Building	C	706'-6"	706'-2"	702*(2)*	Underground Fuel Oil Tank (Pile Elevation 703*(2)*
Odor Control Station	D	706'-0"	705'-4"	*(1)*	-
Sludge Truck Loading Station	E	715'-6"	712'-10"	712'-4"*(2)*	-
Sludge Thickening Building	F	706'-6"	701'-6"	702'-6"*(2)*	-
Grit Building	G	715'-6"	710'-6"	709'-4"(2)	-
Administration Building / Laboratory	H	715'-6"	712'-6"	712'-10"(2)	-
Maintenance Building	I	708'-8"	*(3)*	*(1)*	-
Headworks Building /Influent Pump Station	J	713'-6"	672'-0"	*(1)*	-
Effluent Pump Station Building	K	706'-6"	697'-4"	698'-4"	-
Return Sludge Pump Station	L	698'-2"	696'-2"	697'-2"(2)	
Waste Sludge Pump Station	M	701'-0"	699'-0"	700'-0"(2)	

*Lowest point of entry refers to lowest door (entrance or overhead) opening elevation for buildings or top of concrete for equipment pads.

**Lowest point of concrete in the structure (i.e. bottom of lowest point of footing, sump structure, etc.)

(1)-Referenced drawings do not indicate structural pile elevations. It is underdetermined if structure is supported on piles.

(2)- Elevation indicates top of pile (does not include the pile cap elevation) or cutoff elevation per referenced drawings. Referenced drawings do not include linear length of pile to determine lowest bearing point of the structural pile.

(3) – Elevations could not be determined from referenced drawings.

BUILDING/ STRUCTURE	BUILDING/ STRUCTURE DESIGNATED NUMBER (REFER TO FIGURE XYZ)	TOP OF CONCRETE FOR WATER RETAINING STRUCTURES	LOWEST PERSONNEL ENTRY POINT(***)	APPROXIMATE BOTTOM OF CONCRETE OF STRUCTURE (**)	PILE BEARING ELEVATION	ADDITIONAL STRUCTUAL OBSERVATIONS THAT COULD BE POTENTIALLY IMPACTED BY FLOOD EVENT
BASINS						
Septage Receiving Station	1A	710'-3"	-	702'-5"	*(1)*	-
Septage Receiving Drain Basin	1B	709'-9"	-	704'-1"	*(1)*	-
Grit & Scum Removal Basins	2	718'-0"	713'-7"	705'-0"	705'-4"(2)	-
Influent Splitter Box	3	717'-0"	-	704'-0"	704'-4"(2)	-
Oxidation Basin No. 1	4	717'-6" (North Side) & 714'-10" (South Side)	707'-4"	697'-4"	700'-2"(2)	Pressure Release Valves
Oxidation Basin No. 2	5	717'-6" (North Side) & 714'-10" (South Side)	707'-4"	697'-4"	700'-2"(2)	Pressure Release Valves
Oxidation Basin No. 3	6	717'-6" (North Side) & 713'-10" (South Side)	701'-3"	697'-5"	671'-11"	Pressure Release Valves
Oxidation Basin No. 4	7	*(3)*	*(3)*	*(3)*	*(3)*	*(3)*
Oxidation Basin Effluent Weir and Splitter Box (Oxidation Basin No. 1 & No. 2)	8	715'-6"	707'-4"	699'-4"	*(1)*	-
Mixed Liquor Flow Splitter Box	9	715'-6"	706'-3"	699'-6"	*(1)*	-

Appendix B - Building and Basin Key

BASINS CONTINUED						
Final Clarifier No. 1	10	712'-2"	708'-8"	691'-6"	691'-8"[2]	Pressure Release Valves
Final Clarifier No. 2	11	712'-2"	708'-8"	691'-6"	691'-8"[2]	Pressure Release Valves
Final Clarifier No. 3	12	712'-2"	-	689'-6"	670'-11"	Pressure Release Valves
Final Clarifier No. 4	13	*(3)*	*(3)*	*(3)*	*(3)*	*(3)*
Chlorine Contact Basin	14	710'-8"	706'-8"	694'-8"	696'-8"[2]	Pressure Release Valves
Polishing Filters	15	715'-0"	709'-6"	701'-0"	702'-6"[2]	-
Post Aeration Basin	16	717'-6"	715'-9"	697'-3"	698'-4"	-
Selector Basin	17	717'-6"	714'-0"	702'-0"	*(1)*	-
Waste Sludge Holding Tank	18	715'-0"	-	695'-11"	695'-1"[2]	Pressure Release Valves
Sludge Holding Basin No. 1	19	718'-0"	706'-4"	692'-10"	697'-10"[2]	Pressure Release Valves
Sludge Holding Basin No. 2	20	718'-0"	706'-4"	692'-10"	697'-10"[2]	Pressure Release Valves
Sludge Holding Basin No. 3	21	718'-0"	714'-3"	698'-0"	697'-0"[2]	Pressure Release Valves

Appendix B - Building and Basin Key

BASINS CONTINUED

Waste Sludge Pump Station (Tank)	22	715'-4"	-	(3)	699'-6"(2)	Pressure Release Valves
Plant Sewer Pump Station	23	707'-3"	-	682'-3"	(1)	

* Structures many have access hatches or manhole rungs for maintenance but structures are not normally occupied. The start of the stair elevation is noted if applicable. Stairs elevations usually start at grade (polish filters structure is an exception. Stairs into the structure are supplied). Elevations not noted do not have stairs to access the structure.

**Lowest point of concrete in the structure (i.e. bottom of lowest point of footing, sump structure, drainage, etc.)

(1)-Referenced drawings do not indicate structural pile elevations. It is underdetermined if structure is supported on piles.

(2)- Elevation indicates top of pile (does not include the pile cap elevation) or cutoff elevation per referenced drawings. Referenced drawings do not include linear length of pile to determine lowest bearing point of the structural pile.

(3) – Elevations could not be determined from referenced drawings.

APPENDIX C

ADDITIONAL SEEPAGE ANALYSIS

DRAFT

COMPTON WASTEWATER TREATMENT PLANT

Conceptual-Level Groundwater Seepage Analysis

B&V PROJECT NO. 192653
B&V FILE NO. 41.0000

PREPARED FOR

City of Branson, Missouri

NOVEMBER 8 , 2016

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Project Area and Recent Flood Conditions

This technical memorandum summarizes the findings of a brief conceptual groundwater seepage analysis for the Compton Wastewater Treatment Plant (WWTP) owned and operated by the City of Branson, Missouri (City). The plant is located along Lake Taneycomo, a little over 10 miles downstream of Table Rock Dam as shown on Figure 1. When the US Army Corps of Engineers (USACE) makes significant releases from the dam, the water level of Lake Taneycomo rises, inducing groundwater seepage and requiring the operation of dewatering wells to lower the water table beneath the plant site. At the end of December 2015, the USACE released approximately 72,000 cubic feet per second (cfs) at the dam which was the maximum release ever recorded, exceeding the previous record of approximately 69,000 cfs in 2011.[1] The dam releases on these occasions caused lake levels to approach the top of the berm surrounding the WWTP, resulting in concerns about both groundwater and surface water flooding the plant. According to the effective FEMA Flood Insurance Study for Lake Taneycomo, these peak flowrates are approximately equal to a flood event with a 40-year return period.

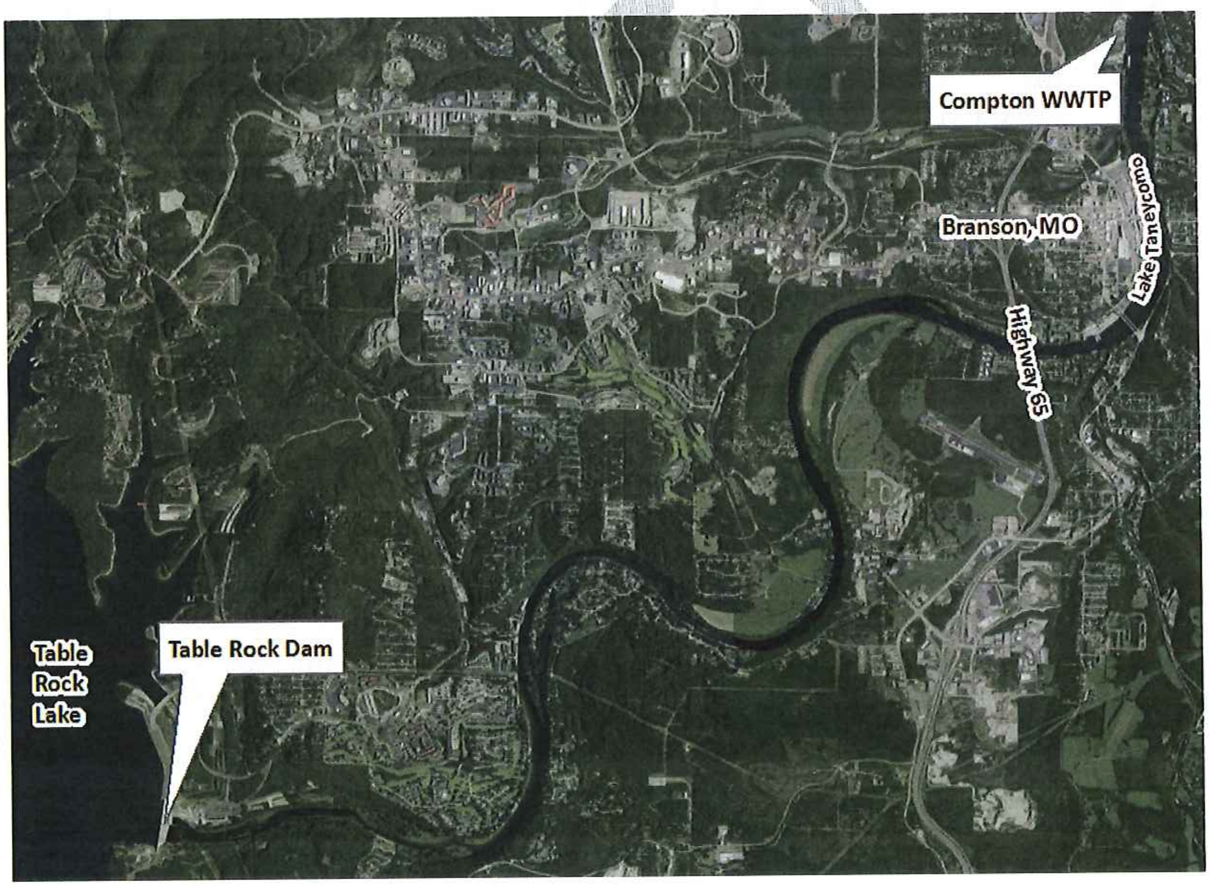

Figure 1 – Study Area

[1] http://www.news-leader.com/story/news/local/ozarks/2015/12/29/surge-water-pushes-table-rock-dam-release-into-record-books/78018342/

Figure 2 shows the estimated Lake Taneycomo water levels near the plant since 2004. These historical water levels were estimated by obtaining the historical lake levels from the US Geological Survey gage at Ozark Beach Dam located approximately 12 miles downstream of the WWTP[2], and adding 0.4 feet to account for the slope in the river profile across this distance as determined by the hydraulic grade line for the 2-year flood event from the effective FEMA hydraulic model of Lake Taneycomo.

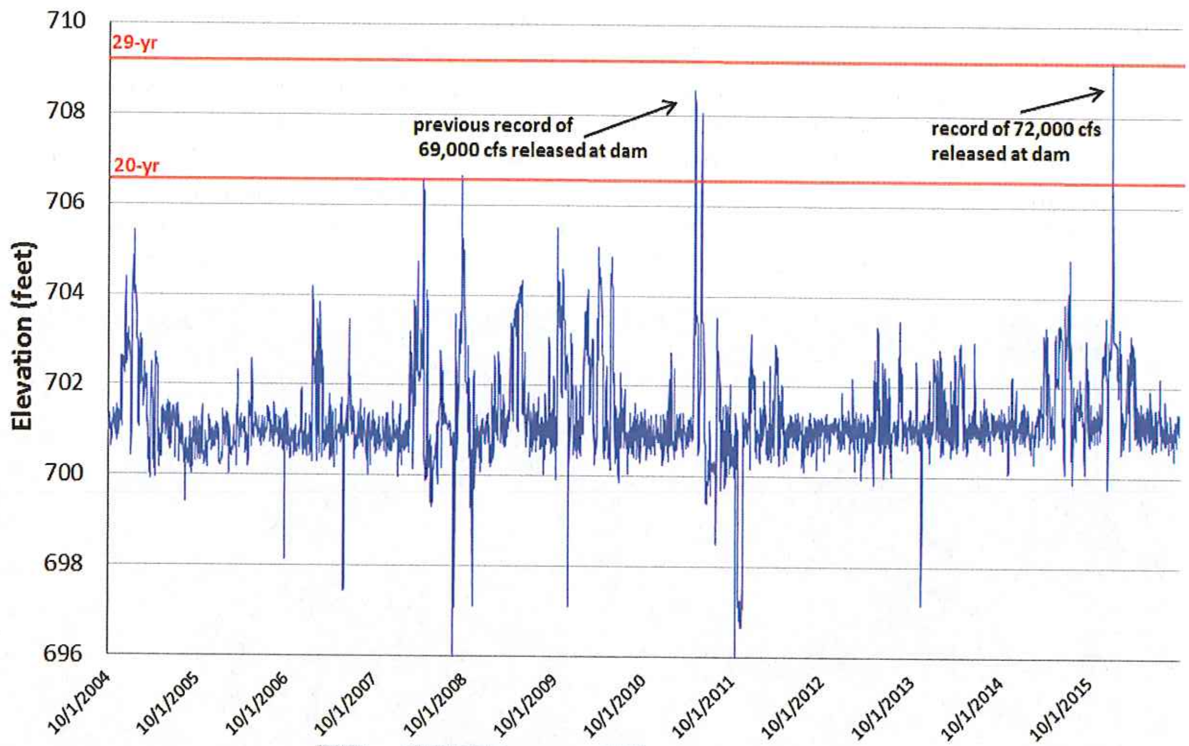

Figure 2 – Estimated Historical Lake Taneycomo Water Surface Elevation at the WWTP

(flood return period elevations from FEMA Flood Insurance Study; flowrates from http://www.news-leader.com/story/news/local/ozarks/2015/12/29/surge-water-pushes-table-rock-dam-release-into-record-books/78018342/)

Based on these estimated lake elevations at the WWTP, the December 2015 event was approximately equal to a FEMA flood event with a 29-year return period. It is noted, however, that the lake elevations at Ozark Beach Dam are recorded at 8 AM each morning, and these once-daily measurements likely missed the maximum lake elevation that day. So, the return period of the December 2015 event based on lake elevation was likely a closer match to the 40-year return period estimated from the documented flowrate released at the dam.

Plant personnel indicate the resulting water level in December 2015 was within inches of the top of the berm surrounding the west side of the plant and less than two feet of the top of the berm surrounding the east side of the plant. Figure 3 shows the topographic contours of the plant site obtained from the City's mapping. The lowest elevations along the top of the berm surrounding the plant are approximately 712 feet; this generally agrees with December 2015 observations of water levels

[2] http://waterdata.usgs.gov/mo/nwis/uv?site_no=07053820

approaching the top of the berm if some additional depth is added to the estimated elevation of 709.2 feet on December 29th to account for the fact the lake levels are only recorded once a day at the USGS gage. At an assumed minimum top of berm elevation of 712 feet, and until a more detailed topographic survey becomes available, it is estimated for purposes of this desktop study that the WWTP currently has approximately 40-year flood protection, with little if any freeboard above that level.

Figure 3 – Topographic Contours at WWTP (City GIS data)

Conceptual-Level Groundwater Seepage Model Assumptions

This conceptual-level seepage evaluation relies almost entirely on available data from three test borings drilled by Anderson Engineering in 1991[3] and one test boring drilled by Olsson in 2008[4]. It is clear that additional data for the characteristics of the alluvium and bedrock beneath the site are needed to perform a more formal evaluation of seepage from Lake Taneycomo toward the plant site as flooding occurs. With this in mind, the intent of this desktop study is to provide initial, planning-level estimates of the range of seepage that may occur at the site to assist in the future evaluation and design of alternative permanent flood protection measures. Using data from these four test borings along with available topographic information for the WWTP and flood elevations for Lake Taneycomo, the conceptual-level groundwater model includes the following major assumptions:

- This evaluation only considers groundwater seepage from the lake toward the plant site. It does not consider surface water overtopping the berm and infiltrating vertically through the soils within the site.
- Two flood return periods were selected for Lake Taneycomo in order to evaluate seepage beneath the berm and toward the plant site. These include the 50-year and 100-year FEMA flood elevations of 713.4 feet and 718.5 feet, respectively. Both of these lake flood elevations are higher than any known elevation that has occurred since Table Rock dam was completed in the 1950s.
- Steady-state groundwater flow modeling is performed for this desktop study. More complex transient groundwater flow modeling was not performed.
- Although complex transient modeling was not performed, it is recognized that the lake remains at elevated flood levels for only several days to about a week. As plant personnel know from recent historical dam releases, increases in the groundwater elevation during flooding and associated pumping to control these levels is temporary. In both 2011 and 2015, the lake elevation remained above 703 feet for approximately 7 consecutive days. It is acknowledged that, during future wet climate conditions which are more extreme than anything that has occurred since the 1950s, there is a possibility that the USACE could make large releases for longer periods of time.
- Based on the City's GIS as shown on the figure above, the ground elevation across much of the plant site is 706 feet, although there are lower sump areas near some of the facilities. The target maximum groundwater elevation beneath the plant footprint for this evaluation is assumed to be between about 702 and 703 feet, which is close to the normal lake elevation.
- The following elevations (with a simplified cross section shown on Figure 4) are assumed for the aquifer layers beneath the site:
    - Ground at plant site = El. 706 feet

[3] Burns & McDonnell, 1991, *Subsurface Information for the Wastewater Treatment Plant Expansion*, prepared for the City of Branson, Missouri, August.

[4] Olsson Associates, 2008, *Branson Compton Drive WWTP – Bar Screen Structure*, prepared for City of Branson, Missouri, January 12.

- Bottom of upper silty/clayey soils = El. 687 feet
- Bottom of silty sand layer (top of coarser sand and gravel) = El. 682 feet
- Bottom of coarser sand and gravel layer (top of bedrock) = El. 671 feet

- No published hydrogeologic reports were discovered for the alluvial soils formed by the former White River or the carbonate bedrock aquifer in this area; therefore, the following horizontal hydraulic conductivities are assumed:

  - Upper silty/clayey soils, K_h = 10 feet per day (ft/day) based on conservative values from Freeze and Cherry (1979)[5]

  - Silty sand layer, K_h = 40 ft/day, based on Hazen approximation of a soil sample collected from test boring B-2 drilled by Anderson Engineering in 1991

  - Sand and gravel layer, K_h = up to 850 ft/day based on Hazen approximation of a soil sample collected from test boring B-3 drilled by Anderson Engineering in 1991; however, a similar calculation for a sample collected from 1991 test boring B-1 reveals a hydraulic conductivity of only around 10 ft/day, suggesting significant heterogeneity in the soils identified as fine to coarse sand with gravel; for this analysis, and until the aquifer can be characterized further, somewhat average values of 250 ft/day and 500 ft/day were assumed for the sand and gravel layer in the conceptual-level model

  - Carbonate bedrock beneath the site[6], K_h = 5 ft/day; this is an assumption of the average permeability of the bedrock mass based on published values in Freeze and Cherry (1979)

- It is possible that the uppermost carbonate bedrock is weathered with significantly higher localized hydraulic conductivities. Well logs obtained from the Missouri Department of Natural Resources[7] indicate the uppermost bedrock beneath Branson is Jefferson City Dolomite, which is part of the Ozark Aquifer. This dolomite has minor water-bearing capabilities compared to other formations within the Ozark Aquifer[8]; registered irrigation and domestic wells in this area tap the bedrock aquifer.[9] Future site-specific testing should consider collecting some information about the permeability of the uppermost bedrock, specifically for any future design of a seepage cutoff system, since available information from the FEMA flood study suggests the bottom of Lake Taneycomo may be within several feet of the top of bedrock.

- For simplicity for this conceptual-level analysis, it is assumed that the carbonate bedrock and soils to the west of the WWTP where the ground surface rises steeply contribute insignificant quantities of groundwater toward the plant site when compared to quantity of groundwater contributed by the lake during flood conditions. If future testing shows bedrock is highly permeable and could transmit significant quantities of infiltration from the west during and following large rain events, and if no groundwater cutoff is included as part of a future flood control solution, this simplification should be revisited and modified.

---

[5] Freeze and Cherry, 1979, *Groundwater*, Prentice-Hill, Inc.
[6] Part of the Ozark Aquifer (USGS, 1997, *Ground Water Atlas of the United States, Segment 3, Kansas, Missouri, Nebraska*, Hydrologic Investigations Atlas 730-D)
[7] http://dnr.mo.gov/geology/wrc/logmain/?/env/wrc/logmain/
[8] http://pubs.usgs.gov/ha/ha730/ch_f/F-text6.html
[9] https://dnr.mo.gov/mowells/wimsSearchLanding.do

- It is assumed that the vertical hydraulic conductivity is 10 percent of the horizontal.

- The effective FEMA hydraulic model of Lake Taneycomo was used to obtain the elevation of the lake bottom near the plant; as shown on Figure 4, the bottom of the lake appears to be in hydraulic communication with the sand and gravel layer and silty sand layer. Plant personnel note that the groundwater levels respond fairly quickly (within hours) of rising lake levels, indicating seepage of lake water through these layers, through preferential flowpaths within the berm itself, or both.

- No information was discovered for the properties of the sediments at the bottom of Lake Taneycomo. With Table Rock dam located upstream, there may not be a thick layer of fine-grained sediments that significantly restricts seepage through the lake bed. It is assumed that the permeability of the sediments is 1 ft/day and the thickness is 2 feet. More conservative assumptions could be made for this parameter that would result in greater hydraulic connection between the lake and overburden soils beneath the plant site.

- No data was discovered for the bottom elevation or properties of the drainage ditch located to the west of the plant.

- Records of historical pumping rates from several existing dewatering wells and groundwater levels within the plant were not available to calibrate or verify the conceptual groundwater model.

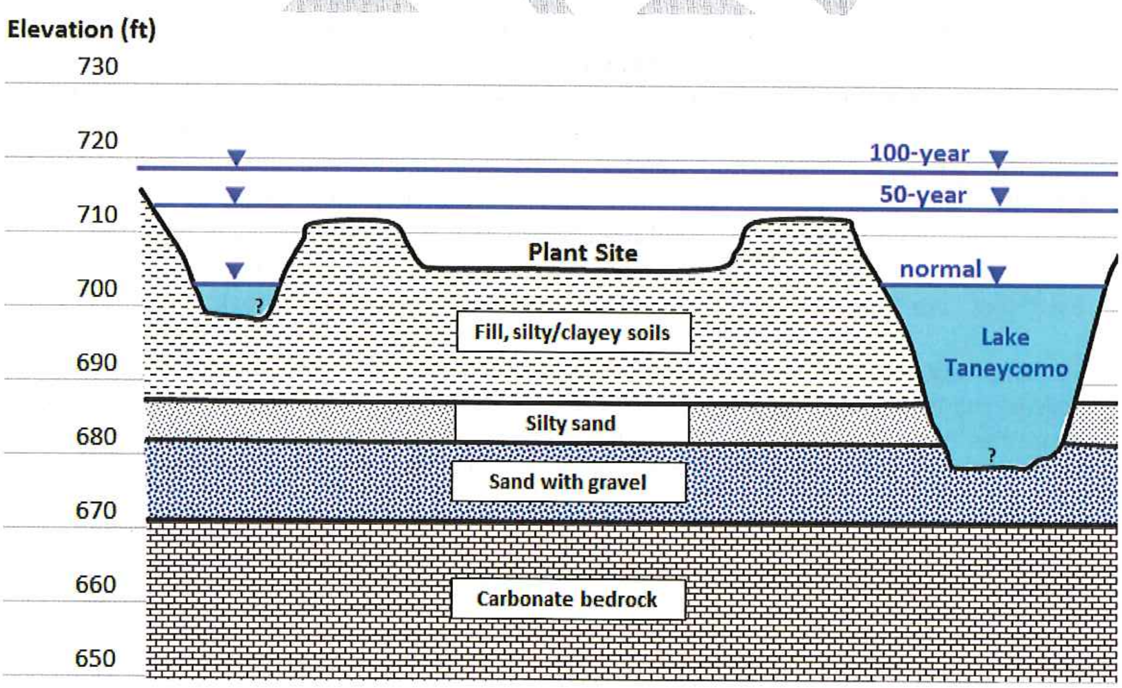

Figure 4 – Simplified Geologic Cross Section

Scenarios Simulated with the Conceptual Model

Table 1 provides a listing of the scenarios evaluated with the conceptual-level groundwater seepage model. Many other scenarios can be contemplated.

Table 1 – Seepage Scenarios

Scenario	Lake Flood Elevation (feet)	Hydraulic Conductivity of Sand & Gravel Layer (ft/day)	Sheet Pile Cutoff Simulated? (Elev of bottom, feet)	Target Maximum Groundwater Elevation beneath Plant Site (feet)
1	718.5 (100-yr)	250	No (-)	702 – 703
2	718.5 (100-yr)	500	No (-)	702 – 703
3	713.4 (50-yr)	250	No (-)	702 – 703
4	713.4 (50-yr)	500	No (-)	702 – 703
5	718.5 (100-yr)	250	Yes (682, top sand & gravel)	702 – 703
6	718.5 (100-yr)	500	Yes (682, top sand & gravel)	702 – 703
7	713.4 (50-yr)	250	Yes (682, top sand & gravel)	702 – 703
8	713.4 (50-yr)	500	Yes (682, top sand & gravel)	702 – 703
9	718.5 (100-yr)	250	Yes (671, top bedrock)	702 – 703
10	718.5 (100-yr)	500	Yes (671, top bedrock)	702 – 703
11	713.4 (50-yr)	250	Yes (671, top bedrock)	702 – 703
12	713.4 (50-yr)	500	Yes (671, top bedrock)	702 – 703

Results

SCENARIO 1

This scenario includes no seepage cutoff and maintains the lake at the 100-year flood level which is about 15 feet above normal lake pool and an estimated 8 feet above the December 2015 flood level. It also uses a hydraulic conductivity of 250 ft/day for the sand and gravel layer. Since this value is significantly higher than the hydraulic conductivities applied to the other layers, most of the seepage toward the site occurs through this layer, and future testing at the site should confirm the properties of these sandy layers in order to design any future dewatering wells. Aerial photography was used to locate potential viable sites for permanent wells, and a variety of simulations were performed to determine an adequate number of wells to achieve the target groundwater level with reasonable pumping rates per well. These simulations revealed advantages of placing dewatering wells around the perimeter and specifically within the corners of the plant site, along with several wells in the center of the site. The conceptual-level model indicates a total steady-state pumping rate of 3.74 million gallons per day (mgd) from a total of nine dewatering wells in order to achieve the target maximum groundwater elevation of approximately 702 feet beneath the plant site; the range of individual well pumping rates is 200 gallons per minute (gpm) to 550 gpm. The resulting groundwater elevation contours are provided on Figure 5, showing the critical areas are around the perimeter of the site and specifically within the corners. If piezometers are installed to monitor groundwater levels, wells could be designed to be operated to achieve the target elevation without lowering the water table too much in the center of the site.

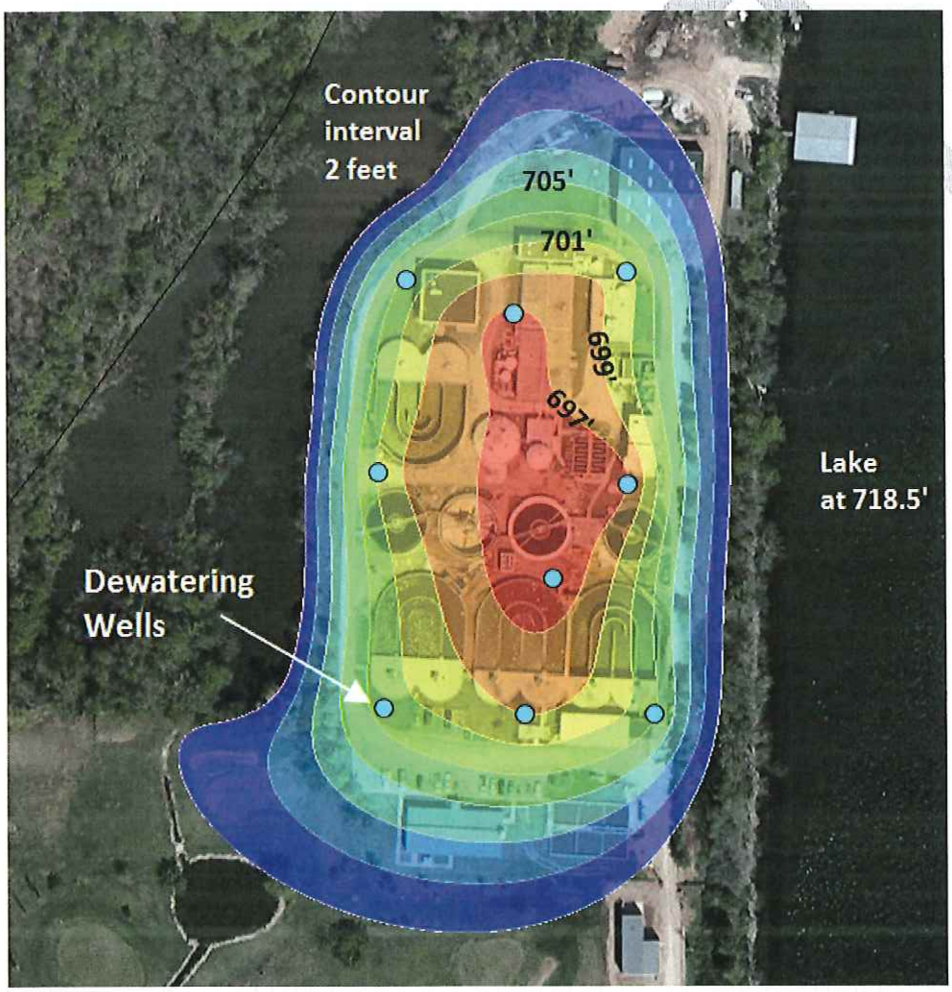

Figure 5 – Resulting Steady-State Groundwater Elevations for Scenario 1, without Cutoff

For comparability purposes, the nine selected well sites determined from this scenario are maintained for the remaining scenarios. Depending on the final decisions for flood protection at the site (e.g., depending on the selected flood event, with or without seepage cutoff, etc.) along with additional geotechnical information that becomes available between now and then, the number and configuration of dewatering wells will need to be refined.

SCENARIO 2

This scenario is the same as Scenario 1 but uses a higher hydraulic conductivity of 500 ft/day for the sand and gravel layer. As expected, the conceptual-level model shows that the total pumping rate from the 9 selected well sites increases to 6.05 mgd, with individual well rates ranging from 350 gpm to 900 gpm.

SCENARIOS 3 AND 4

Repeating Scenarios 1 and 2 (again without a groundwater cutoff), but lowering Lake Taneycomo to the 50-year flood elevation instead of the 100-yr elevation, the total pumping from 9 wells decreases to 2.52 mgd for Scenario 3 and 4.10 mgd for Scenario 4, or a reduction of approximately 33 percent for each. Individual well pumping rates range from 100 to 400 gpm for Scenario 3 and from 150 to 650 gpm for Scenario 4.

SCENARIO 5

Scenario 5 is similar to Scenario 1, but includes a seepage cutoff. The seepage cutoff was assumed to be sheet pile, with the bottom of the sheet pile extending to the top of the sand and gravel layer and terminating about 11 feet above bedrock, providing about 30 feet of embedment from the top of the existing berm. The sheet pile was simulated generally along the access road surrounding the perimeter of the interior of the plant. The exact location and depth of any sheet pile used in the future would need to be refined at a later date, but this selected location of the sheet pile provides an order-of-magnitude estimate of the reduction in seepage toward the plant site for the purposes of this preliminary analysis. For comparability purposes, this sheet pile alignment is held constant for Scenarios 5 – 12.

Some amount of seepage will occur through the interlocking joints of the sheet pile. Simple calculations were performed to provide a rough estimate of the total number of joints around the perimeter and the quantity of seepage that might occur through those joints. A model was calibrated to that seepage estimate to obtain the effective hydraulic properties to use for sheet pile for Scenarios 5 – 12. The results of Scenario 5 indicate the nine dewatering wells would need to pump a total of 3.17 mgd, which is a slight reduction of about 10 to 15 percent from Scenario 1 without sheet pile. This confirms that most of the seepage toward the site occurs through the coarser sands and gravels above bedrock. The individual well pumping rates range from 175 gpm to 450 gpm.

SCENARIO 6

Scenario 6 is the same as Scenario 5 but increasing the hydraulic conductivity of the sand and gravel from 250 ft/day to 500 ft/day. The partial-depth sheet pile reduces the total pumping of the dewatering wells (5.4 mgd) by about 10 to 15 percent when compared to Scenario 2. The individual well pumping rates range from 300 gpm to 850 gpm.

SCENARIOS 7 AND 8

Scenarios 7 and 8 repeat Scenarios 5 and 6, respectively, but with the lake reduced from the 100-year flood elevation to the 50-year flood elevation. Again, the partial depth sheet pile reduces the amount of pumping required by the dewatering wells by between 10 and 15 percent when compared to Scenarios 3 and 4 without sheet pile.

SCENARIO 9

By extending the sheet pile to the top of bedrock, the amount of pumping from the dewatering wells within the interior of the plant decreases dramatically by over 90 percent from Scenario 1, as would be expected. Only 6 wells were required at pumping rates of only 40 gpm per well, for a total of approximately 0.35 mgd. The resulting steady-state groundwater table contours from the conceptual-level model are illustrated on Figure 6.

Further analysis of the water budget from this scenario indicates most of the seepage into the interior of the sheet pile perimeter occurs as underflow through the carbonate bedrock (on the order of about 150 gpm with the hydraulic characteristics selected for bedrock). The actual amount of underflow beneath the sheet pile will depend on the actual degree of weathering of the uppermost carbonate bedrock, which is currently unknown.

SCENARIO 10

Repeating Scenario 9 with sheet pile to bedrock, but increasing the hydraulic conductivity of the sand and gravel layer from 250 ft/day to 500 ft/day, the total dewatering requirement from 6 wells within the interior of the plant remains the same (about 0.35 mgd) as Scenario 9. This illustrates how a well-constructed seepage cutoff to bedrock would nearly eliminate uncertainties of the heterogeneity of the soils beneath the site, leaving most of the uncertainty with the properties of the bedrock beneath the cutoff barrier.

SCENARIOS 11 AND 12

Scenarios 11 and 12 repeat Scenarios 9 and 10, respectively, but with the lake reduced from the 100-year flood elevation to the 50-year flood elevation. The total dewatering required from 6 wells is reduced to only approximately 0.2 mgd for each scenario with pumping rates of only 25 gpm per well.

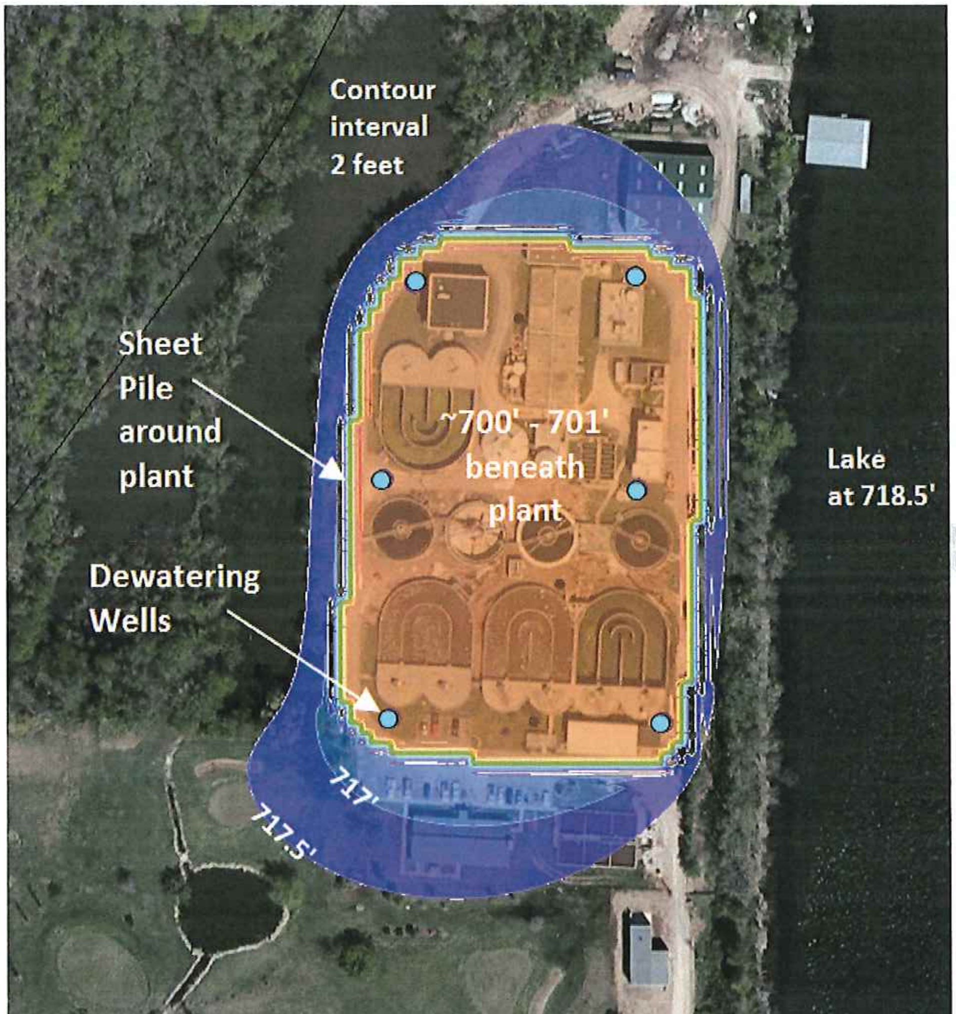

Figure 6 – Resulting Steady-State Groundwater Elevations for Scenario 9, with Cutoff

Summary and Recommendations

Table 2 provides a summary of the steady-state seepage analyses performed.

Table 2 – Summary of Conceptual-Level Model Results

Scenario	Lake Flood Elevation (feet)	Hydraulic Conductivity of Sand & Gravel Layer (ft/day)	Sheet Pile Cutoff Simulated? (Elev of bottom, feet)	Target Maximum Groundwater Elevation beneath Plant Site (feet)	Number of Wells, Total Pumping Rate (mgd)	Range of Individual Well Pumping Rates (gpm)
1	718.5 (100-yr)	250	No (-)	702 – 703	9 wells, 3.74	200 – 550
2	718.5 (100-yr)	500	No (-)	702 – 703	9 wells, 6.05	350 – 900
3	713.4 (50-yr)	250	No (-)	702 – 703	9 wells, 2.52	100 – 400
4	713.4 (50-yr)	500	No (-)	702 – 703	9 wells, 4.10	150 – 650
5	718.5 (100-yr)	250	Yes (682, top sand & gravel)	702 – 703	9 wells, 3.17	175 – 450
6	718.5 (100-yr)	500	Yes (682, top sand & gravel)	702 – 703	9 wells, 5.40	300 – 850
7	713.4 (50-yr)	250	Yes (682, top sand & gravel)	702 – 703	9 wells, 2.23	125 – 300
8	713.4 (50-yr)	500	Yes (682, top sand & gravel)	702 – 703	9 wells, 3.67	200 – 550
9	718.5 (100-yr)	250	Yes (671, top bedrock)	702 – 703	6 wells, 0.35	40 each
10	718.5 (100-yr)	500	Yes (671, top bedrock)	702 – 703	6 wells, 0.35	40 each
11	713.4 (50-yr)	250	Yes (671, top bedrock)	702 – 703	6 wells, 0.22	25 each
12	713.4 (50-yr)	500	Yes (671, top bedrock)	702 – 703	6 wells, 0.22	25 each

A factor of safety is required for these initial findings to account for uncertainties due to a general lack of information at this time. It is generally assumed that analysis methods can be expected to have uncertainty that ranges from approximately 0.5 to 2. To be conservative during preliminary planning phases of a project, a factor of safety of as much as 1.75 to 2 are often used when there is a general lack of data and when considering the consequences of failure with regard to damage to or shutdown of facilities, safety, and economics.[10] Additional site-specific aquifer testing will be required to determine several key parameters such as the hydraulic characteristics of the soils, hydraulic connection of the lake with the soils, and interaction between the soils and the Ozark Aquifer beneath the site and toward the west of the site. Recommended testing includes borings with sieve analyses across the plant site,

[10] Departments of the Army, the Navy, and the Air Force, 1983, *Dewatering and Groundwater Control*, Army TM 5-818-5, Navy NAVFAC P-418, Air Force AFM 88-5, Chapter 6, November.

several hydraulic interval tests, and a full-scale constant rate pumping test. The field data would be used to refine this conceptual-level groundwater flow model in order to optimize the number and configuration of the dewatering wells and to aid in the design of the wells and any cutoff system that may be selected.

This preliminary analysis illustrates the benefits of having a seepage cutoff such as sheet pile. A cutoff wall will have lower operational and maintenance costs than a system of dewatering wells. If sheet pile is installed properly to bedrock such that the seepage through the interlocking joints is minimized, it would significantly reduce the concerns about the uncertainty in the soil stratigraphy and heterogeneities in the hydraulic and storage characteristics of the soil layers. Due to the apparent proximity of the bottom of Lake Taneycomo with bedrock, upflow from carbonate bedrock from below any cutoff system could still be a concern; investigation of the degree of weathering of the uppermost bedrock will need to be considered and used to refine the estimates of underflow for the design of any seepage cutoff measure that may be selected.

APPENDIX D

ADDITIONAL GEOTECHNICAL ANALYSIS

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MEMORANDUM

City of Branson, Missouri
Compton WWTP Flood Study
Flood Wall Analysis

B&V Project Number 192653.0100
B&V File Number
10/27/2016

To: Charlie Sievert and Molly Pesce

From: Jacques Moraille

This memorandum summarizes the flood wall analysis to determine the cross section and flood wall length for wall stability during a 100-year flood event at the Compton WWTP. The wall evaluation was based on the location for a newly proposed flood wall using vertical steel sheet piling. The top of wall elevation used in the analysis was El. 720 with flood water EL. 718.5, which allows for 1.5 ft. of freeboard. The footprint and alignment of the new floodwall is shown on Figure 1.

Figure 1 – Permanent flood wall alignment (red line).

B&V Project 192653.0100
B&V File
10/27/2016

A number of cross sections were taken within the flood wall limits using a north-south and east-west orientation. The elevations along each of the cross sections are based on GIS data provided by the Client. The cross section labels and locations with respect to the wall alignment and footprint are shown in Figure 2.

Figure 2 – Multiple cross sections running north-south and east west of the WWTP.

The cross sections chosen for the analysis were selected by comparing the ground surface profiles with respect to the wall location, as well as flood and wall elevations with respect to the ground surface. Sections C, E and F were selected to represent the flood wall. Appendix A shows the selected cross sections in complete view and detailed view for sections C, E and F. Red dots indicate location of permanent flood wall.

The analysis is conceptual in nature. The subsurface profile information is based on available data from three test borings drilled by Anderson Engineering in 1991, and one test boring drilled by Olsson in 2008. Based on the unknown location of the Anderson Engineering borings, the analysis places heavier reliance on the Olson test boring. Given the limited geotechnical information, the length of the wall (2,200 ft.) and the potential subsurface variability along the length of the wall, additional geotechnical borings and laboratory testing along the flood wall alignment would be needed to provide the information for flood wall design.

The wall and water elevations, the ground surface profile along both sides of the flood wall, and the subsurface layers and geotechnical properties of each layer were all gathered to start the analysis. The computer program CWALSHT was used to run the wall analysis for the different cross sections considered. The model assumes all effects on the wall tend to cause counter-clockwise rotation in the case of a cantilever wall, which was the type of wall being analyzed. The program uses soil mechanics procedures to determine the required depth of penetration of a new wall or assess the factors of safety of an existing wall. A final design is reached when values of wall penetration produce a pressure distribution where the sum of moments about any point and the sum of all horizontal forces are equal to zero. The chosen penetration depth was increased 30% to provide additional factor of safety against overturning.

This design did not consider any groundwater seepage cutoff effects provided by embedding the sheeting to a given depth. Such evaluation would need to consider the effects of existing dewatering wells at the plant, updated hydraulic conductivity parameters for the subsurface, and other hydrogeologic boundary conditions that this sheet pile analysis is unable to evaluate. For more information on this topic, please refer to the document titled: ***Compton Wastewater Treatment Plant – Conceptual Level Groundwater Seepage Analysis***, prepared by Kris Hahn (B&V).

The calculations are included in Appendix B. Table 1 summarizes the CWALSHT results and the embedment requirements for each of the cross sections evaluated. The results are based on using a **Skyline Steel NZ19, Grade 60** sheet pile or equivalent.

CROSS SECTION	SECTION MODULUS (S_x; in^3/ft)		SHEAR AREA (in^2/ft)		Deflection Δ (in)	TIP ELEVATION
	REQUIRED	AVAILABLE	REQUIRED	AVAILABLE		REQUIRED (El.)
Section C	3.28	35.08	0.105	7.684	0.13	696
Section E	1.58	35.08	0.077	7.684	0.03	703
Section F	2.09	35.08	0.093	7.684	0.05	701

Table 1 – Summary of CWALSHT sheet pile runs.

B&V Project 192653.0100
B&V File
10/27/2016

In conclusion, the recommendation is to use a **Skyline Steel NZ19, Grade 60 sheet pile or equivalent with tip El. 696 (24 ft. long sheets) for the 2,200 ft. long wall**.

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APPENDIX A

Cross Sections Selected for Evaluation

N
W E
S

0 125
Feet

1 inch = 125 feet

LEGEND

- Permanent Flood Wall
- Temporary Flood Wall
- Additional Improvements

Permanent & Temporary Flood Wall Alignments

Flood Control for Compton WWTP

PN 192653

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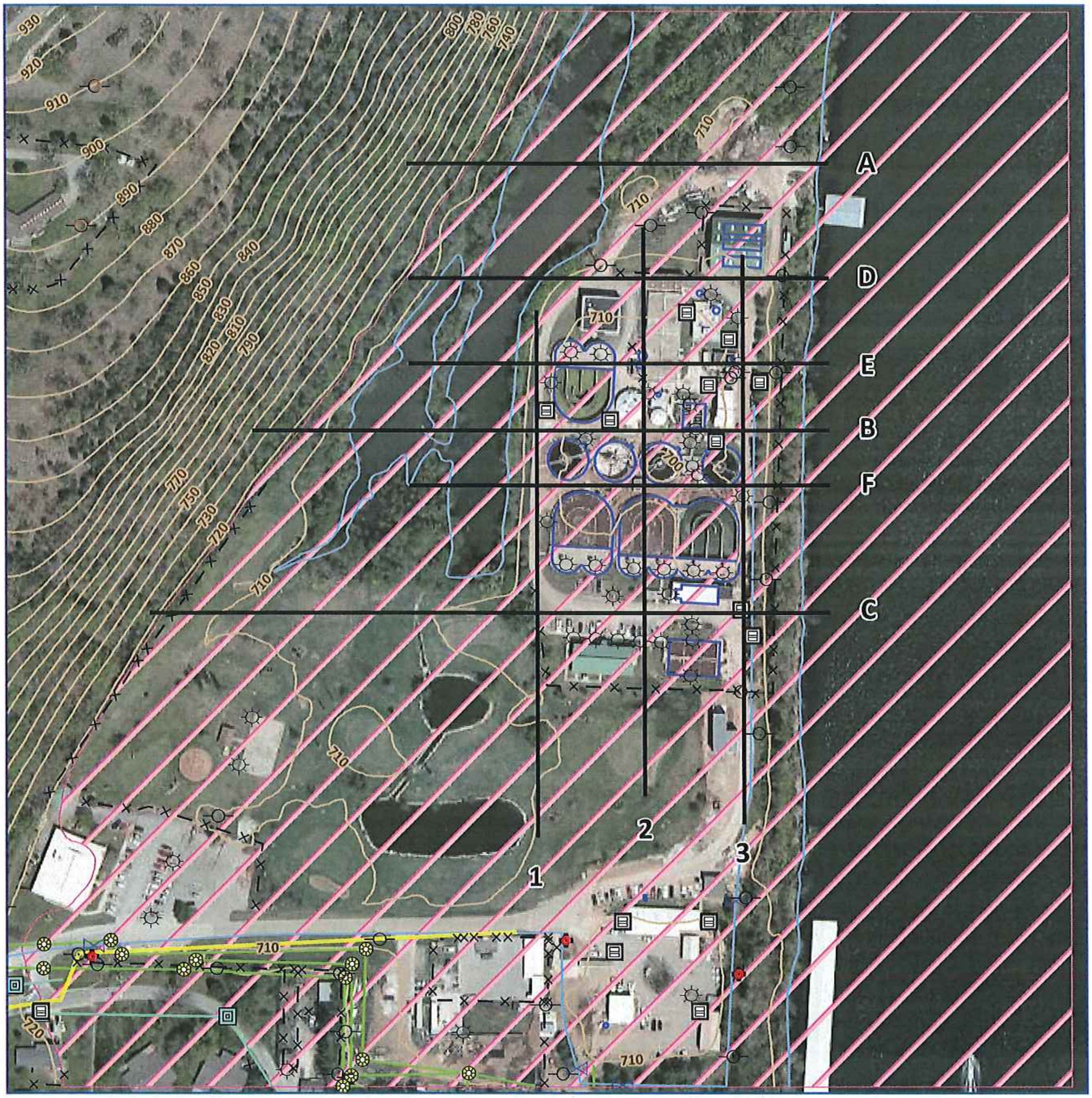
FloodWallAlignments October 07, 2016



N
W E
S

0 60 120
Feet

1 inch = 240 feet

LEGEND

Hydrants
Large Pole
Light Pole
Utility Pole
Utility Light Pole
Water Valve
Manholes
Catch Basin
Storm Inlet
Storm Outfall
Tanks
Fence
Natural Gas Lines
Sewer Mains
Storm Mains
Hydrology Lines
10 Ft Contours
Flood Event Inundation Zone

Inundation Limits for 100-Year Flood Event

Flood Control for Compton WWTP

192653

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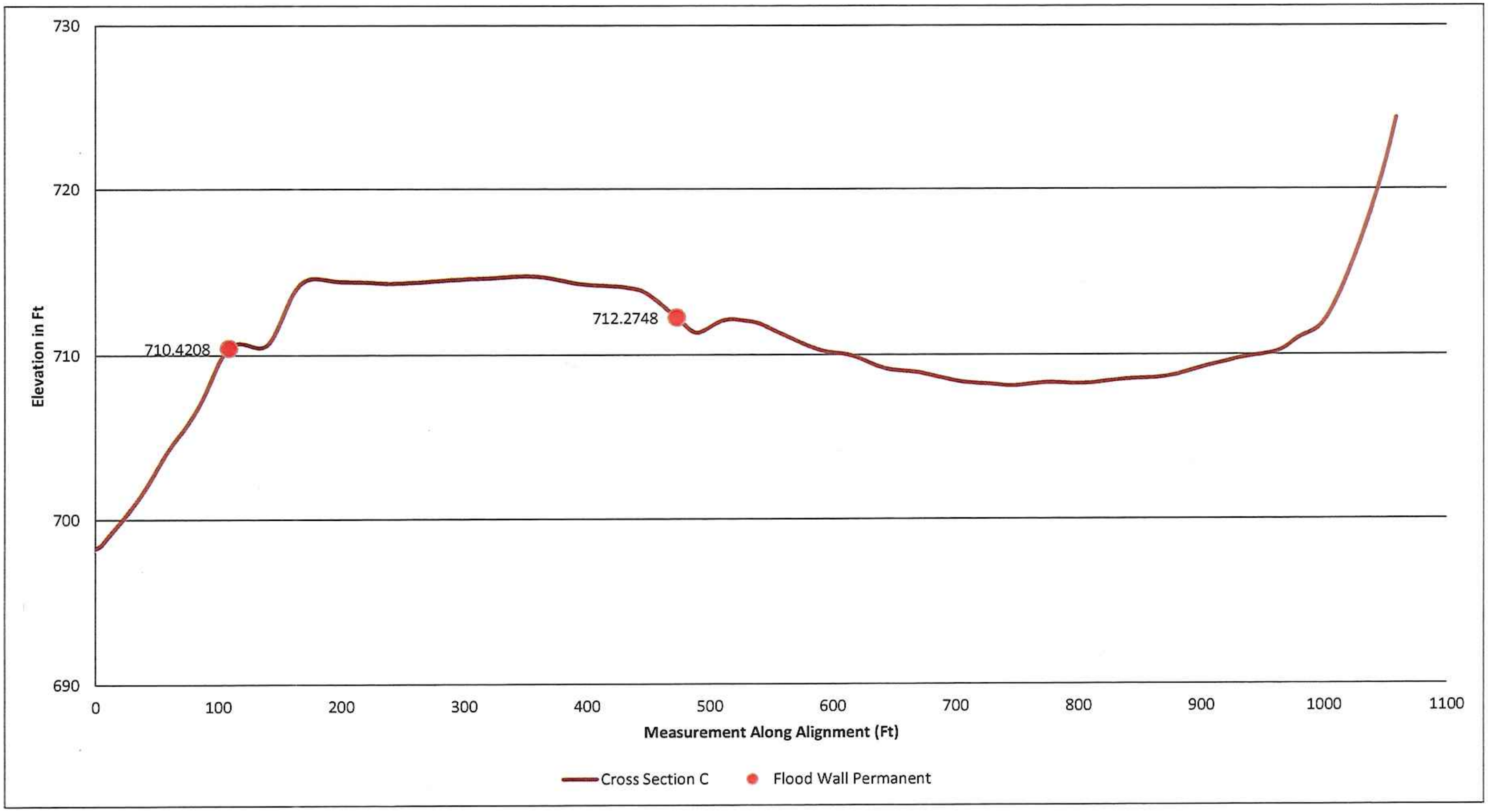
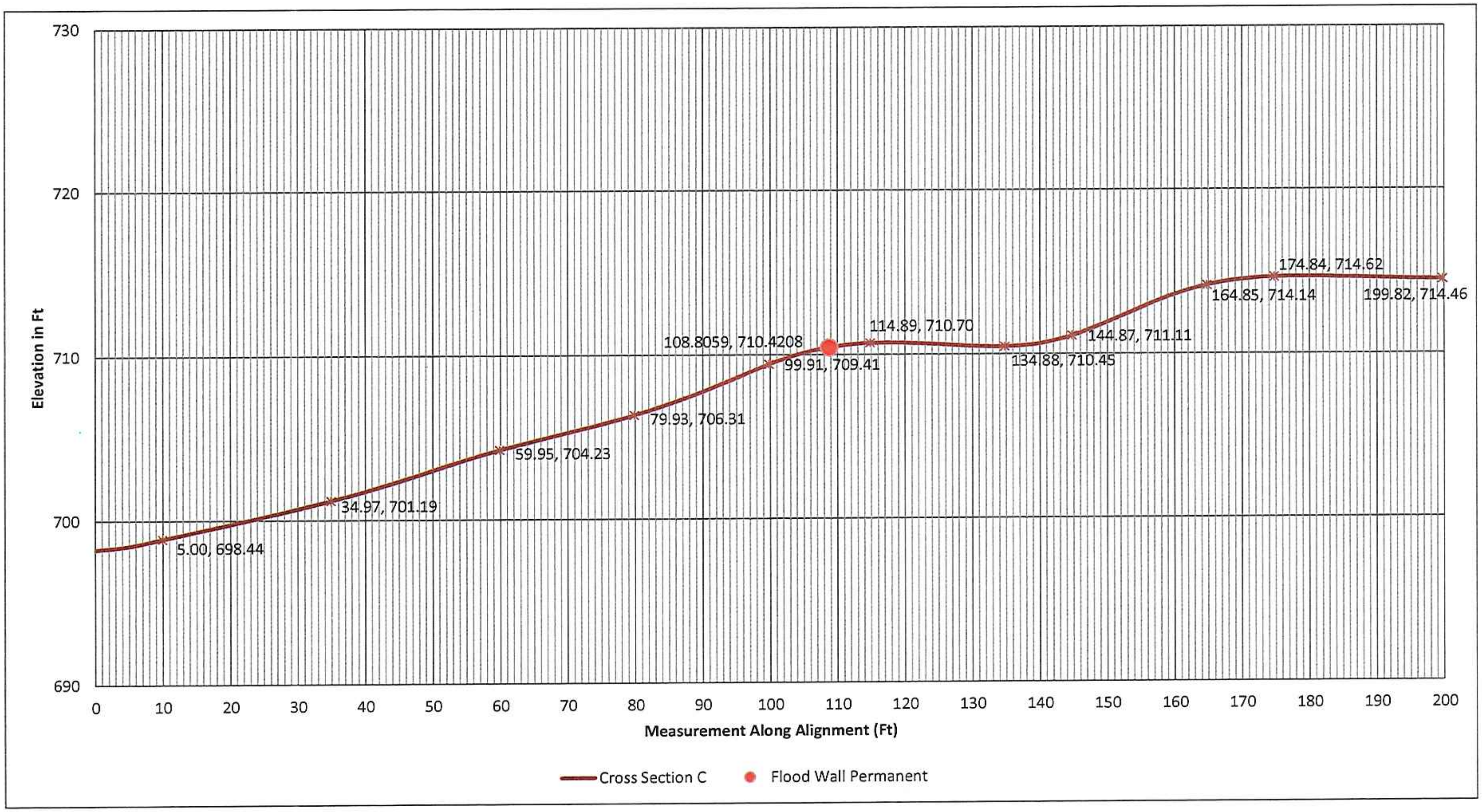
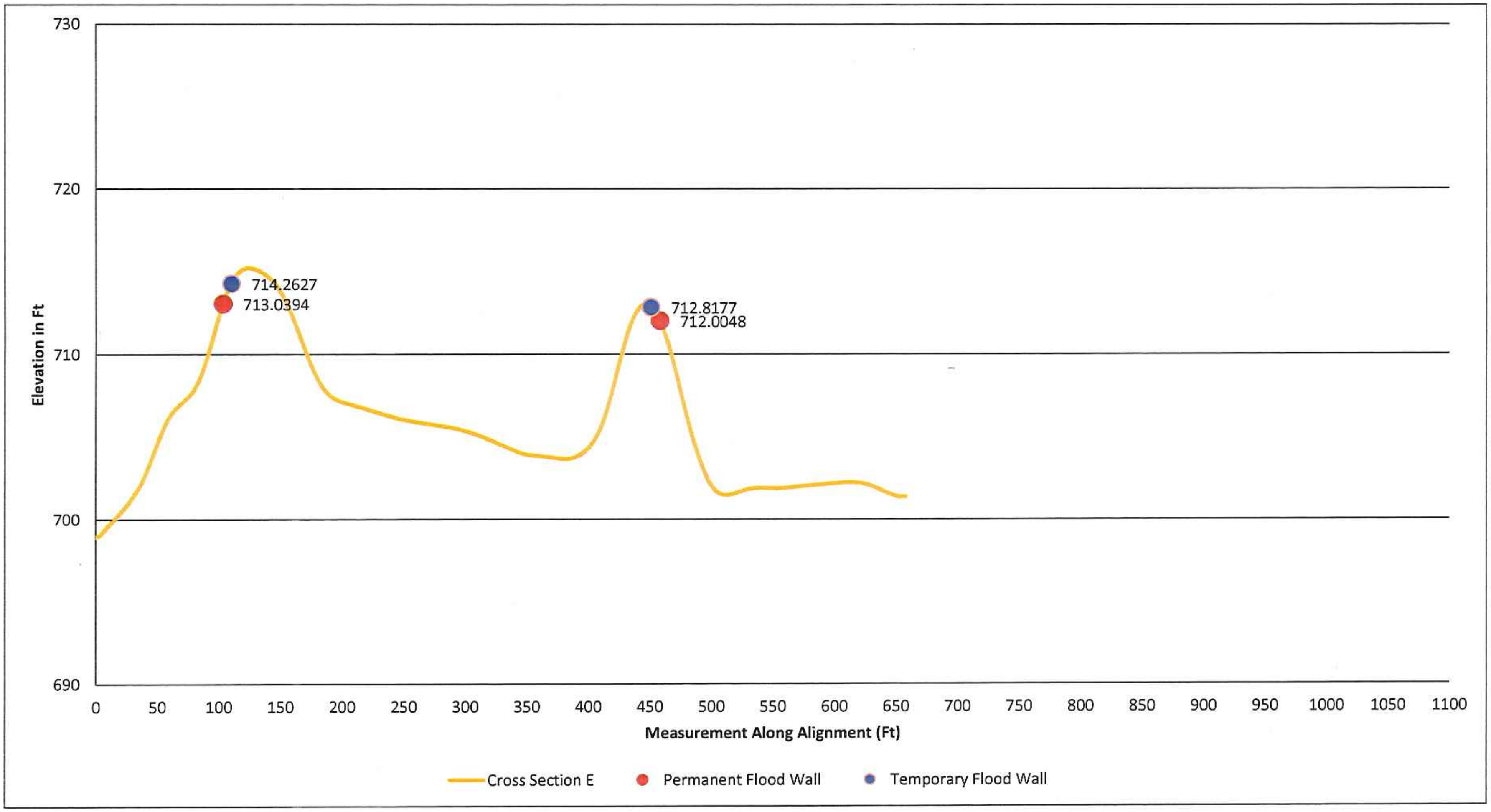
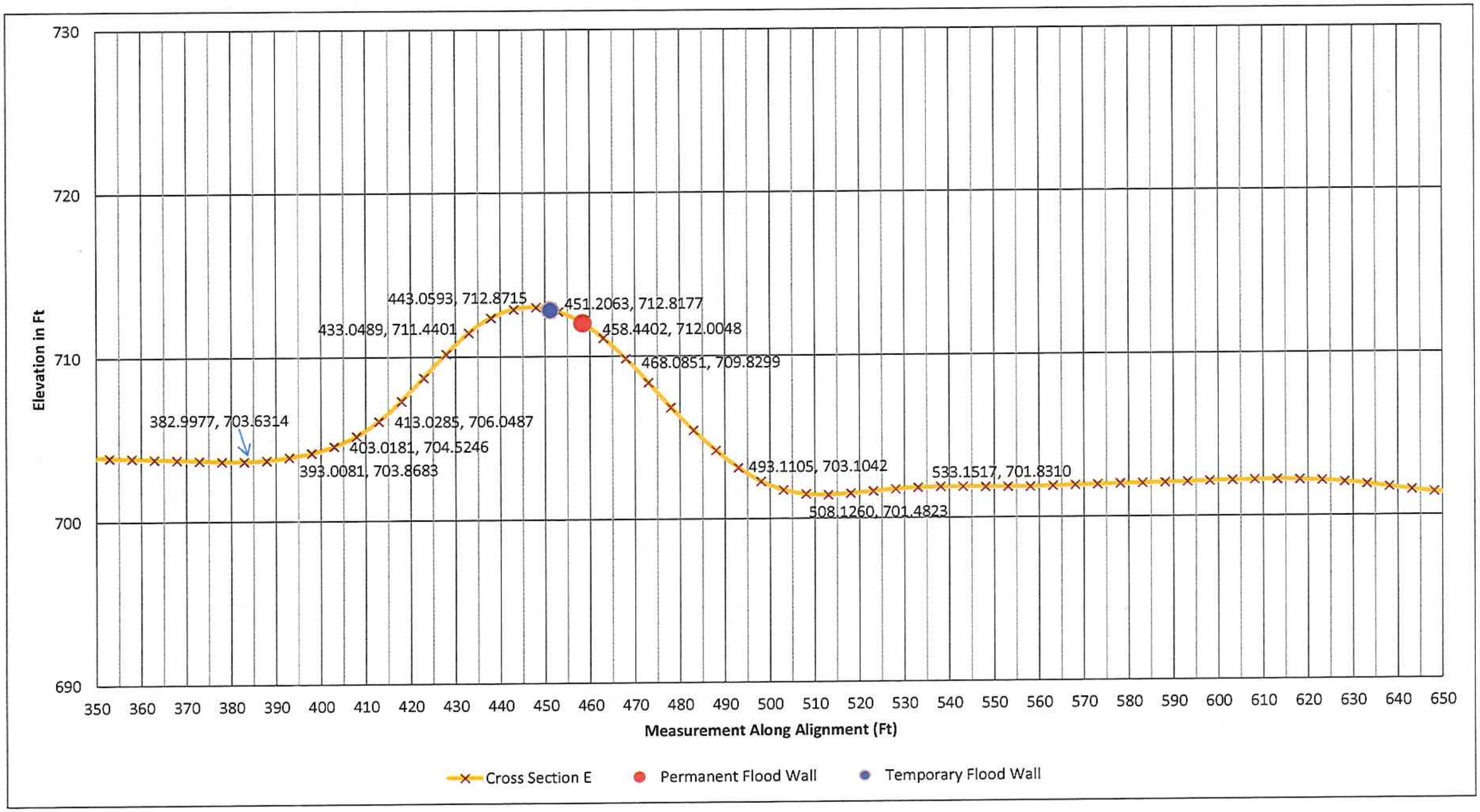
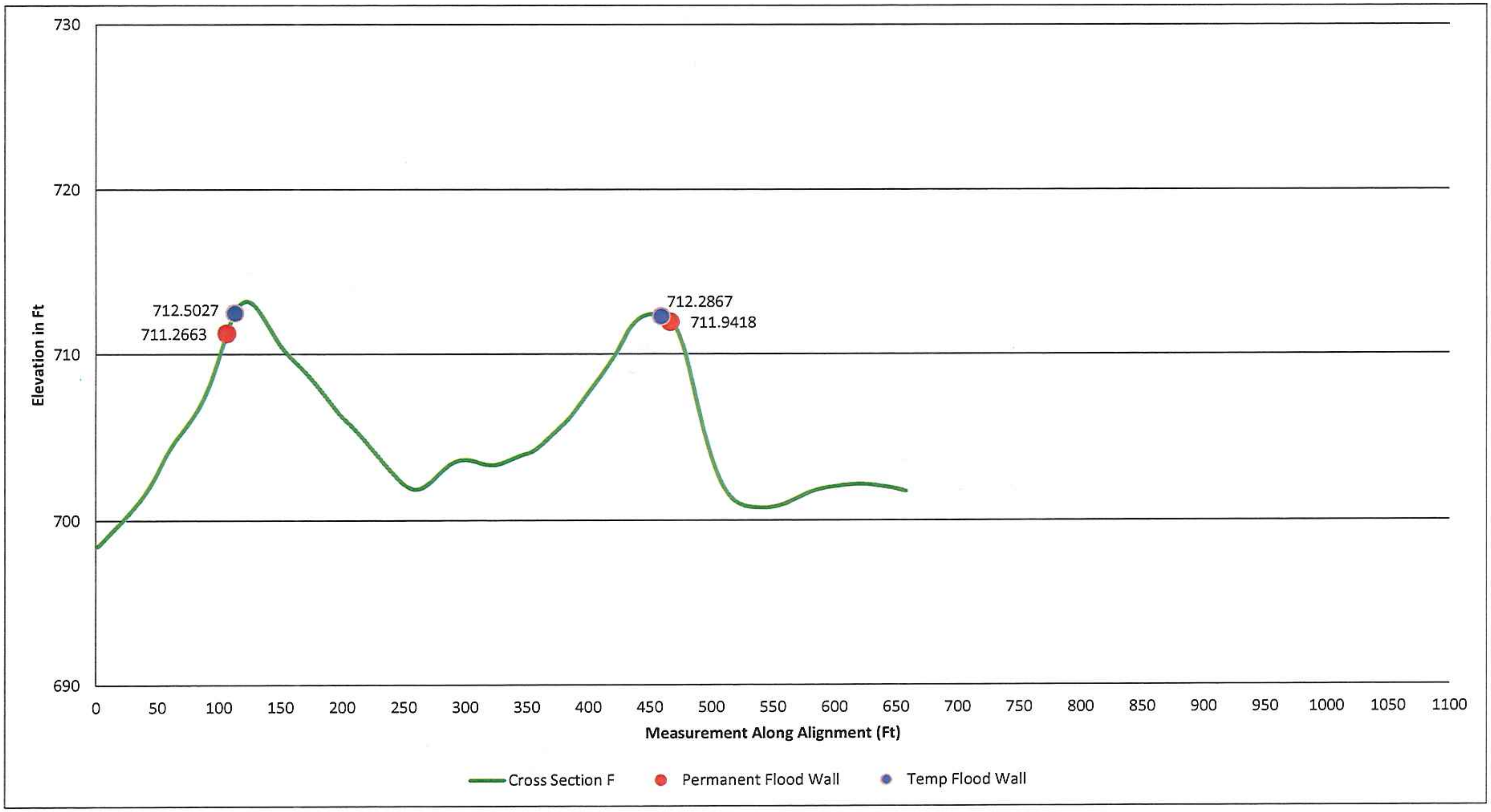

730
720
710
700
690
Elevation in Ft
0
100
200
300
400
500
600
700
800
900
1000
1100
Measurement Along Alignment (Ft)
710.4208
712.2748
Cross Section C
Flood Wall Permanent

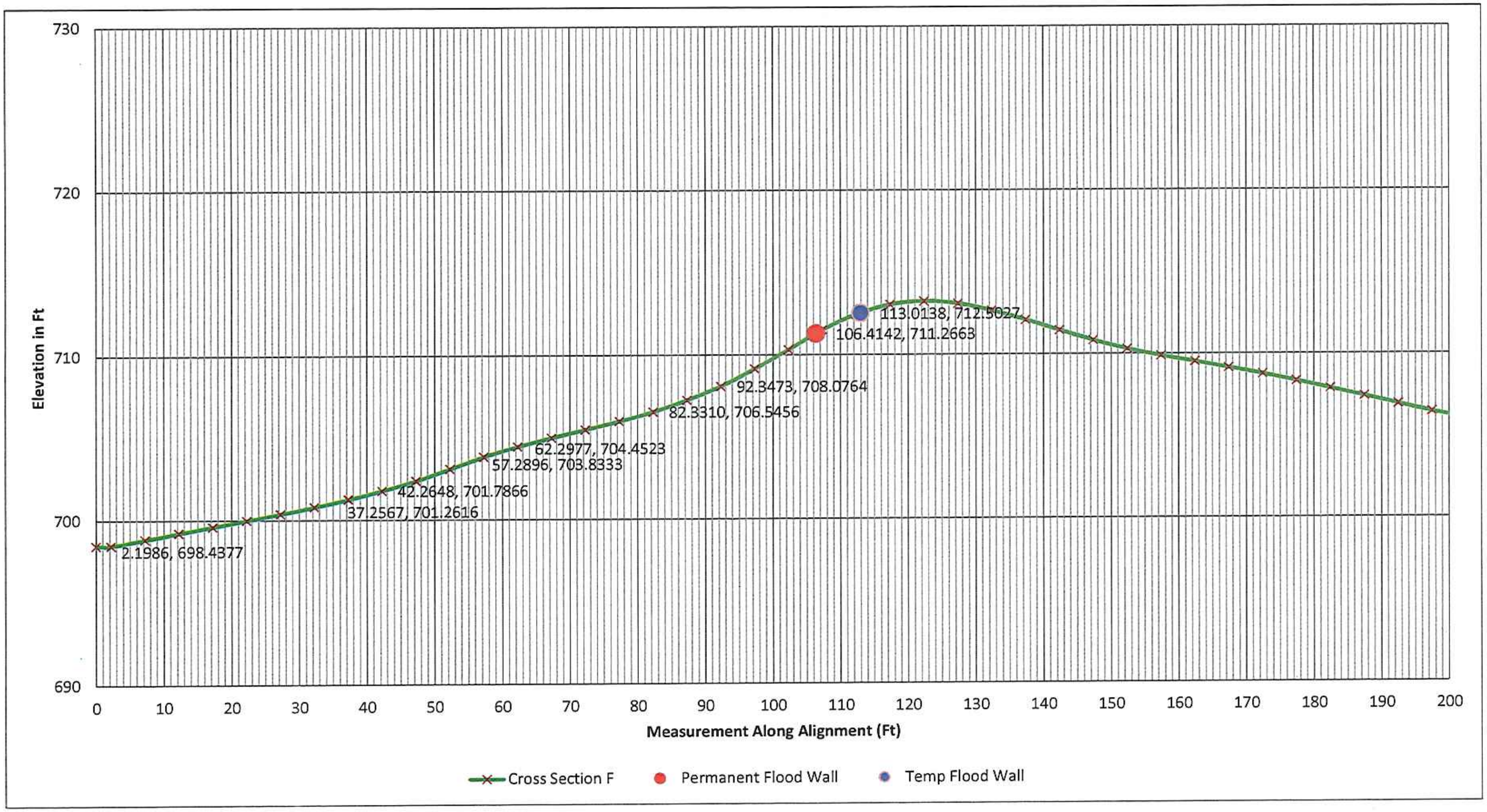

730
720
710
700
690
Elevation in Ft
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
Measurement Along Alignment (Ft)
5.00, 698.44
34.97, 701.19
59.95, 704.23
79.93, 706.31
99.91, 709.41
108.8059, 710.4208
114.89, 710.70
134.88, 710.45
144.87, 711.11
164.85, 714.14
174.84, 714.62
199.82, 714.46
Cross Section C
Flood Wall Permanent


730
720
710
700
690
Elevation in Ft
714.2627
713.0394
712.8177
712.0048
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
950
1000
1050
1100
Measurement Along Alignment (Ft)
Cross Section E
Permanent Flood Wall
Temporary Flood Wall



730
720
710
700
690
Elevation in Ft
350 360 370 380 390 400 410 420 430 440 450 460 470 480 490 500 510 520 530 540 550 560 570 580 590 600 610 620 630 640 650
Measurement Along Alignment (Ft)
382.9977, 703.6314
393.0081, 703.8683
403.0181, 704.5246
413.0285, 706.0487
433.0489, 711.4401
443.0593, 712.8715
451.2063, 712.8177
458.4402, 712.0048
468.0851, 709.8299
493.1105, 703.1042
508.1260, 701.4823
533.1517, 701.8310
Cross Section E
Permanent Flood Wall
Temporary Flood Wall


Elevation in Ft
730
720
710
700
690
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
950
1000
1050
1100
Measurement Along Alignment (Ft)
712.5027
711.2663
712.2867
711.9418
Cross Section F
Permanent Flood Wall
Temp Flood Wall



730
720
710
700
690
Elevation in Ft
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
Measurement Along Alignment (Ft)
2.1986, 698.4377
37.2567, 701.2616
42.2648, 701.7866
57.2896, 703.8333
62.2977, 704.4523
82.3310, 706.5456
92.3473, 708.0764
106.4142, 711.2663
113.0138, 712.5027
Cross Section F
Permanent Flood Wall
Temp Flood Wall

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APPENDIX B
Sheet Pile Calculations

DO NOT WRITE IN THIS SPACE

Objective: To determine sheet pile length and section required to provide flood control for the Compton WWTP.

Assumptions

- Top of wall EL. 720
- Use GIS survey data for cross sections along permanent Flood Wall
- Reviewed previous borings done in 1991. Used 2008 boring log by Olsson Associates as baseline for the profile
- Computed forces, deflections and bending moments using CWALSHT sofware from U.S. Army Corps of Engineers. Model assumes all effects on the wall tend to cause counter-clockwise rotation of a cantilever wall.
- Assume flood water El. 718.5, and WWTP area (interior of floodwall) dewatered to El. 703 using wells
- Chosen length was increased 30% to provide factor of safety against overturning
- Runs did not consider seepage cutoff effects on the existing or future dewatering pumps. This length may be evaluated further during detailed design based on additional geotechnical/hydrogeologic data and MODFlOW analysis.

Conclusion: Use NZ 19, Grade 60 sheeting down to tip EL. 696

Cantilever Wall Analysis (BASED ON SECTION C RUN)

Top of Wall Elevation =	720 ft
Leftside Elevation =	710.4 ft
Cantilever Height =	9.6 ft
Tip Elevation =	**696 ft**

Max. Bending Moment =	8.20E+03 lb*ft
Max. Scaled Deflection =	1.10E+09 $lb*in^3$
Max. Shear Force =	2087 lb

Calculate Minimum Section Modulus Required

$f_b = 0.5\ f_y =$ 30000 lb/in^2

$S_{min} = \dfrac{M_{max}}{f_b} =$ **3.28132** in^3/ft

TRY NZ 19, Grade 60

$S_x =$ 35.08 in^3/ft

Calculate Minimum Shear Area Per Foot of Wall Required

$f_v = 0.33\ f_y =$ 19800 lb/in^2

$A_{v,min} = \dfrac{V_{max}}{f_v} =$ **0.105404** in^2/ft

$A_v = t_w * (h/(w * 12))$ in^2/ft

$t_w =$ 0.375 in

h = 16.14 in

w = 27.56 in

$A_v =$ 7.684015 in^2/ft

Calculate Maximum Wall Deflection

E = 2.9E+07 lb/in^2

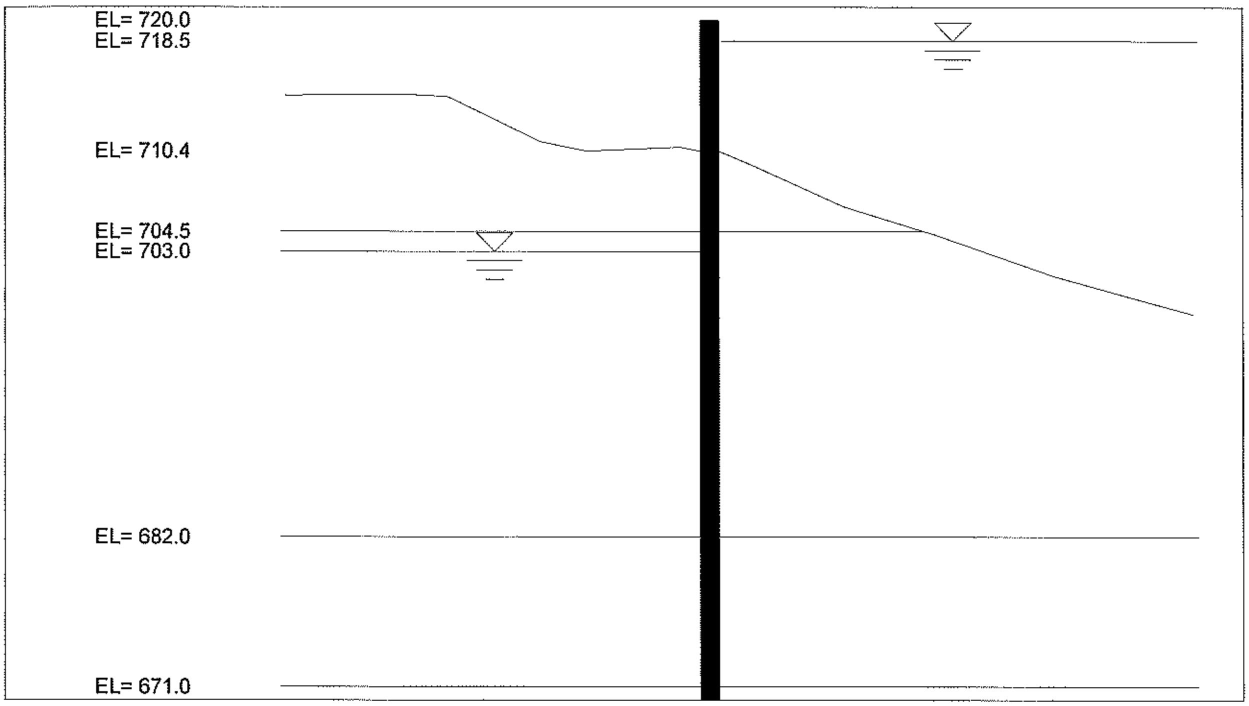
I (from chosen pile section) = 283.1 in^4

$\Delta = \dfrac{\text{Scaled deflection}}{(E * I)} =$ **1.3E-01** in

$\Delta =$ 0.13435 in

'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION

'SECTION C



Branson Sheet Pile Section C.out
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 26-OCTOBER-2016 TIME: 14:01:58

```
****************
*  INPUT DATA  *
****************

 I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'SECTION C

 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    = 720.00 FT.

 IV.--SURFACE POINT DATA

     IV.A.--RIGHTSIDE
        DIST. FROM     ELEVATION
        WALL (FT)        (FT)
            0.00        710.40
            8.90        709.40
           28.90        706.30
           48.90        704.20
           73.80        701.20
          103.80        698.40

     IV.B.--LEFTSIDE
        DIST. FROM     ELEVATION
        WALL (FT)        (FT)
            0.00        710.40
            6.10        710.70
           26.10        710.40
           36.10        711.10
           56.10        714.40
           66.00        714.60
           91.00        714.50

  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      0.00    700.00      0.00     500.00   704.50   0.00    DEF  DEF
110.00    108.00     29.00      0.00     14.00       0.00   682.00   0.00    DEF  DEF
120.00    118.00     33.00      0.00     16.00       0.00   671.00   0.00    DEF  DEF
150.00    145.00      0.00
                          500000.00      0.00     950.00                     DEF  DEF

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

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH- ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH- ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
122.00	120.00	0.00	700.00	0.00	500.00	704.50	0.00	DEF	DEF
110.00	108.00	29.00	0.00	14.00	0.00	682.00	0.00	DEF	DEF
120.00	118.00	33.00	0.00	16.00	0.00	671.00	0.00	DEF	DEF
150.00	145.00	0.00	500000.00	0.00	950.00			DEF	DEF

VI.--WATER DATA
UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 718.50 (FT)
LEFTSIDE ELEVATION = 703.00 (FT)
NO SEEPAGE

VII.--VERTICAL SURCHARGE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 26-OCTOBER-2016 TIME: 14:02:01

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**************************
*    SOIL PRESSURES FOR  *
* CANTILEVER WALL DESIGN *
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I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'SECTION C

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)
720.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
719.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
718.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
718.0	31.2	0.0	0.0	31.2	31.2	0.0	0.0
717.0	93.6	0.0	0.0	93.6	93.6	0.0	0.0
716.0	156.0	0.0	0.0	156.0	156.0	0.0	0.0
715.0	218.4	0.0	0.0	218.4	218.4	0.0	0.0
714.0	280.8	0.0	0.0	280.8	280.8	0.0	0.0
713.0	343.2	0.0	0.0	343.2	343.2	0.0	0.0

712.0	405.6	0.0	0.0	405.6	405.6	0.0	0.0
711.0	468.0	0.0	0.0	468.0	468.0	0.0	0.0
710.4+	505.4	0.0	0.0	505.4	505.4	0.0	0.0
710.4-	505.4	933.3	0.0	-427.9	1438.8	0.0	933.3
710.0	530.4	1336.9	0.0	-806.5	1638.2	0.0	1107.8
709.4	567.8	1413.6	0.0	-845.7	1706.1	0.0	1138.2
709.0	592.8	1464.7	0.0	-871.9	1751.4	0.0	1158.6
708.0	655.2	1592.4	0.0	-937.2	1864.6	0.0	1209.4
707.0	717.6	1720.2	0.0	-1002.6	1977.8	0.0	1260.2
706.0	780.0	1812.5	0.0	-1032.5	2091.0	0.0	1311.0
705.0	842.4	1889.6	0.0	-1047.2	2204.2	0.0	1361.8
704.5	873.6	3384.6	0.0	-2511.0	3134.4	0.0	2260.8
704.0	904.8	3942.1	0.0	-3037.3	3226.9	0.0	2322.1
703.0	967.2	2203.9	0.0	-1236.7	1638.3	0.0	671.1
702.0	967.2	2356.0	0.0	-1388.8	1437.8	0.0	470.6
701.0	967.2	2482.1	0.0	-1514.9	1485.4	0.0	518.2
700.0	967.2	2601.8	0.0	-1634.6	1994.4	0.0	1027.2
699.0	967.2	2706.0	0.0	-1738.8	2070.8	0.0	1103.6
698.0	967.2	2809.9	0.0	-1842.7	2017.8	0.0	1050.6
697.0	967.2	2924.3	0.0	-1957.1	2227.9	0.0	1260.7
696.0	967.2	3089.5	0.0	-2122.3	2317.0	0.0	1349.8
695.0	967.2	3289.9	0.0	-2322.7	2375.0	0.0	1407.8
694.0	967.2	3478.6	0.0	-2511.4	2494.3	0.0	1527.1
693.0	967.2	3645.1	0.0	-2677.9	2554.1	0.0	1586.9
692.0	967.2	3785.3	0.0	-2818.1	2684.8	0.0	1717.6
691.0	967.2	3925.6	0.0	-2958.4	2803.4	0.0	1836.2
690.0	967.2	4126.2	0.0	-3159.0	2890.9	0.0	1923.7
689.0	967.2	4268.7	0.0	-3301.5	2982.9	0.0	2015.7
688.0	967.2	4444.5	0.0	-3477.3	3080.3	0.0	2113.1
687.0	967.2	4749.8	15.0	-3782.6	3162.2	0.0	2210.0
686.0	967.2	4988.5	280.6	-4021.3	2993.6	0.0	2307.0
685.0	967.2	5153.0	537.7	-4108.0	2833.5	77.8	2404.0
684.0	967.2	5305.8	550.9	-4028.2	2917.3	310.5	2501.1
683.0	967.2	5487.9	564.1	-4050.0	3001.2	470.7	2598.1
682.0	967.2	7797.1	650.8	-6286.3	3664.7	543.6	3348.3
681.0	967.2	8561.6	611.2	-7082.3	4105.6	512.2	3749.7
680.0	967.2	7257.0	502.6	-5871.5	3972.3	418.3	3507.7

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

DATE: 26-OCTOBER-2016 TIME: 14:02:02

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****************************
*  SUMMARY OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
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I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'SECTION C

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)      :      701.91
      PENETRATION (FT)      :        8.49

MAX. BEND. MOMENT (LB-FT)   :  8.2033E+03
      AT ELEVATION (FT)     :      707.94

MAX. SCALED DEFL. (LB-IN^3):  1.1030E+09
      AT ELEVATION (FT)     :      720.00
```

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY 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: 26-OCTOBER-2016 TIME: 14:02:02

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****************************
*    COMPLETE RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
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I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'SECTION C

II.--RESULTS0. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
720.00	0.0000E+00	0.	1.1030E+09	0.00
719.00	-2.1828E-10	0.	1.0087E+09	0.00
718.50	-4.3656E-10	0.	9.6153E+08	0.00
718.00	1.3000E+00	8.	9.1438E+08	31.20
717.00	3.5100E+01	70.	8.2010E+08	93.60
716.00	1.6250E+02	195.	7.2589E+08	156.00
715.00	4.4590E+02	382.	6.3198E+08	218.40
714.00	9.4770E+02	632.	5.3888E+08	280.80
713.00	1.7303E+03	944.	4.4745E+08	343.20
712.00	2.8561E+03	1318.	3.5906E+08	405.60
711.00	4.3875E+03	1755.	2.7567E+08	468.00
710.40	5.5270E+03	2047.	2.2910E+08	505.44
710.40	5.5270E+03	2047.	2.2910E+08	-427.89
710.00	6.3015E+03	1800.	1.9992E+08	-806.51
709.40	7.2340E+03	1304.	1.5947E+08	-845.73
709.00	7.6875E+03	961.	1.3497E+08	-871.87
708.00	8.2016E+03	56.	8.3171E+07	-937.23
707.00	7.7785E+03	-914.	4.5413E+07	-1002.59
706.84	7.6205E+03	-1073.	4.0676E+07	-1007.34
706.00	6.4109E+03	-1745.	2.0956E+07	-589.93
705.00	4.4537E+03	-2087.	7.4912E+06	-93.56
704.50	3.4090E+03	-2071.	3.8632E+06	154.63
704.00	2.4029E+03	-1932.	1.7093E+06	402.81
703.00	7.5500E+02	-1281.	1.3758E+05	899.18

702.00	6.2493E+00	-134.	7.4856E+00	1395.55
701.91	0.0000E+00	0.	0.0000E+00	1442.33

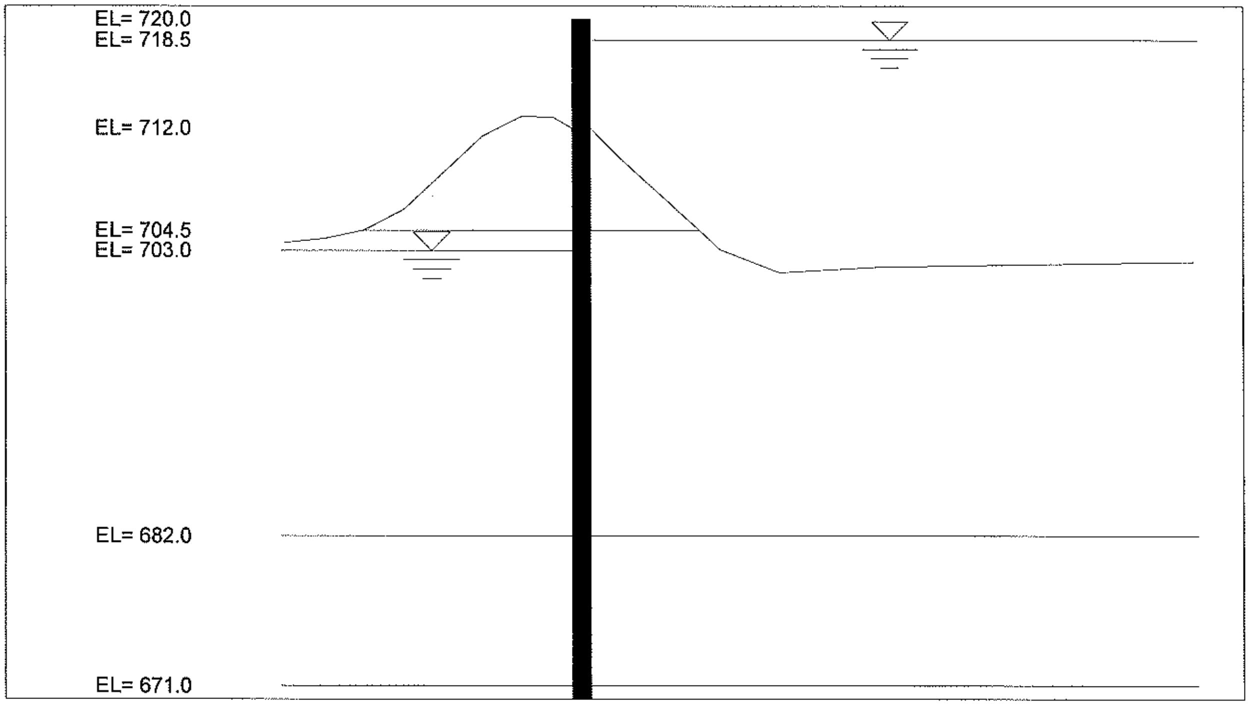
NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES------------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
720.00	0.	0.	0.	0.	0.
719.00	0.	0.	0.	0.	0.
718.50	0.	0.	0.	0.	0.
718.00	31.	0.	0.	0.	0.
717.00	94.	0.	0.	0.	0.
716.00	156.	0.	0.	0.	0.
715.00	218.	0.	0.	0.	0.
714.00	281.	0.	0.	0.	0.
713.00	343.	0.	0.	0.	0.
712.00	406.	0.	0.	0.	0.
711.00	468.	0.	0.	0.	0.
710.40+	505.	0.	0.	0.	0.
710.40+	505.	933.	0.	0.	933.
710.00	530.	1337.	0.	0.	1108.
709.40	568.	1414.	0.	0.	1138.
709.00	593.	1465.	0.	0.	1159.
708.00	655.	1592.	0.	0.	1209.
707.00	718.	1720.	0.	0.	1260.
706.84	780.	1735.	0.	0.	1268.
706.00	842.	1812.	0.	0.	1311.
705.00	874.	1890.	0.	0.	1362.
704.50	905.	3385.	0.	0.	2261.
704.00	967.	3942.	0.	0.	2322.
703.00	967.	2204.	0.	0.	671.
702.00	967.	2356.	0.	0.	471.
701.91	967.	2482.	0.	0.	518.
700.00	967.	2602.	0.	0.	1027.

'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION

'SECTION E



Branson Sheet Pile Section E.out
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 25-OCTOBER-2016 TIME: 16:22:17

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

I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'Section F

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 = 720.00 FT.

IV.--SURFACE POINT DATA

IV.A.--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
0.00	712.00
9.70	709.80
34.70	703.10
49.70	701.40
74.70	701.80
154.80	702.20

IV.B.--LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
0.00	712.00
7.20	712.80
15.30	712.90
25.40	711.40
45.40	706.00
55.40	704.50
65.40	703.90
75.40	703.60

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

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
122.00	120.00	0.00	700.00	0.00	500.00	704.50	0.00	DEF	DEF
110.00	108.00	29.00	0.00	14.00	0.00	682.00	0.00	DEF	DEF
120.00	118.00	33.00	0.00	16.00	0.00	671.00	0.00	DEF	DEF
150.00	145.00	0.00	500000.00	0.00	950.00			DEF	DEF

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

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH- ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH- ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
122.00	120.00	0.00	700.00	0.00	500.00	704.50	0.00	DEF	DEF
110.00	108.00	29.00	0.00	14.00	0.00	682.00	0.00	DEF	DEF
120.00	118.00	33.00	0.00	16.00	0.00	671.00	0.00	DEF	DEF
150.00	145.00	0.00							
			500000.00	0.00	950.00			DEF	DEF

VI.--WATER DATA
UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 718.50 (FT)
LEFTSIDE ELEVATION = 703.00 (FT)
NO SEEPAGE

VII.--VERTICAL SURCHARGE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 25-OCTOBER-2016 TIME: 16:29:11

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

I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'Section F

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)
720.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
719.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
718.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
718.0	31.2	0.0	0.0	31.2	31.2	0.0	0.0
717.0	93.6	0.0	0.0	93.6	93.6	0.0	0.0
716.0	156.0	0.0	0.0	156.0	156.0	0.0	0.0
715.0	218.4	0.0	0.0	218.4	218.4	0.0	0.0
714.0	280.8	0.0	0.0	280.8	280.8	0.0	0.0

713.0	343.2	0.0	0.0	343.2	343.2	0.0	0.0
712.0+	405.6	0.0	0.0	405.6	405.6	0.0	0.0
712.0-	405.6	933.3	0.0	-527.7	1338.9	0.0	933.3
711.0	468.0	1508.0	0.0	-1040.0	1476.1	0.0	1008.1
710.0	530.4	1645.6	0.0	-1115.2	1580.8	0.0	1050.4
709.0	592.8	1783.0	0.0	-1190.2	1685.5	0.0	1092.7
708.0	655.2	1918.8	0.0	-1263.6	1789.5	0.0	1134.3
707.0	717.6	2017.9	0.0	-1300.3	1892.5	0.0	1174.9
706.0	780.0	2083.7	0.0	-1303.7	1995.6	0.0	1215.6
705.0	842.4	2178.5	0.0	-1336.1	2077.6	0.0	1235.2
704.5	873.6	4349.7	0.0	-3476.1	3054.8	0.0	2181.2
704.0	904.8	5186.6	0.0	-4281.8	1375.6	0.0	470.8
703.0	967.2	2576.4	0.0	-1609.2	1485.6	0.0	518.4*
702.0	967.2	2621.9	0.0	-1654.7	1736.2	0.0	769.0*
701.0	967.2	2675.4	0.0	-1708.2	1809.9	0.0	842.7*
700.0	967.2	1204.8	0.0	-237.6	1897.4	0.0	930.2
699.0	967.2	1252.4	0.0	-285.2	1963.7	0.0	996.5
698.0	967.2	1948.4	0.0	-981.2	2038.7	0.0	1071.5
697.0	967.2	2051.7	0.0	-1084.5	2117.6	0.0	1150.4
696.0	967.2	2079.8	0.0	-1112.6	2200.6	0.0	1233.4
695.0	967.2	2106.5	0.0	-1139.3	2279.2	0.0	1312.0
694.0	967.2	2263.3	0.0	-1296.1	2358.6	0.0	1391.4
693.0	967.2	2345.8	0.0	-1378.6	2450.7	0.0	1483.5
692.0	967.2	2355.8	0.0	-1388.6	2531.5	0.0	1564.3
691.0	967.2	2467.5	0.0	-1500.3	2634.3	0.0	1667.1
690.0	967.2	2637.4	0.0	-1670.2	2716.7	0.0	1749.5
689.0	967.2	2662.0	0.0	-1694.8	2824.5	0.0	1857.3
688.0	967.2	2685.9	25.7	-1506.0	2890.9	212.7	1949.4
687.0	967.2	2847.6	337.7	-1482.5	2657.8	397.9	2028.3
686.0	967.2	2995.8	628.9	-1645.7	2505.0	382.9	2166.7
685.0	967.2	3026.3	639.0	-1590.0	2577.7	469.1	2249.5
684.0	967.2	3064.2	660.9	-1614.2	2637.4	482.8	2331.1
683.0	967.2	3213.3	704.5	-1807.6	2734.9	438.5	2472.2
682.0	967.2	4317.7	824.3	-2759.3	3386.1	591.3	3243.3
681.0	967.2	4863.1	771.0	-3356.7	3824.1	539.2	3627.9

* STANDARD WEDGE SOLUTION DOES NOT EXIST FOR INDICATED PRESSURE FOR THIS ELEVATION.

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

DATE: 25-OCTOBER-2016 TIME: 16:29:12

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****************************
*   SUMMARY OF RESULTS FOR  *
*   CANTILEVER WALL DESIGN  *
****************************
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I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'Section F

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

*****WARNING: STANDARD WEDGE SOLUTION DOES NOT EXIST
AT ALL ELEVATIONS. SEE COMPLETE OUTPUT.

```
WALL BOTTOM ELEV. (FT)      :     706.76
      PENETRATION (FT)      :       5.24

MAX. BEND. MOMENT (LB-FT)   :  3.9607E+03
      AT ELEVATION (FT)     :     710.50

MAX. SCALED DEFL. (LB-IN^3):  2.7807E+08
      AT ELEVATION (FT)     :     720.00
```

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELASTICITY 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: 25-OCTOBER-2016 TIME: 16:29:12

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

I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'Section F

II.--RESULTS0. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
720.00	0.0000E+00	0.	2.7807E+08	0.00
719.00	2.1828E-11	0.	2.4737E+08	0.00
718.50	-3.0559E-10	0.	2.3201E+08	0.00
718.00	1.3000E+00	8.	2.1666E+08	31.20
717.00	3.5100E+01	70.	1.8596E+08	93.60
716.00	1.6250E+02	195.	1.5534E+08	156.00
715.00	4.4590E+02	382.	1.2501E+08	218.40
714.00	9.4770E+02	632.	9.5494E+07	280.80
713.00	1.7303E+03	944.	6.7652E+07	343.20
712.00	2.8561E+03	1318.	4.2850E+07	405.60
712.00	2.8561E+03	1318.	4.2850E+07	-527.73
711.00	3.8251E+03	534.	2.2966E+07	-1039.98
710.00	3.8269E+03	-543.	9.5481E+06	-1115.16
709.54	3.4556E+03	-1066.	5.5988E+06	-1149.82
709.00	2.7442E+03	-1525.	2.5832E+06	-556.04
708.00	1.1248E+03	-1529.	2.8060E+05	547.69
707.00	5.3083E+01	-430.	4.5239E+02	1651.41
706.76	0.0000E+00	0.	0.0000E+00	1917.35

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELASTICITY IN PSI TIMES PILE MOMENT

OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

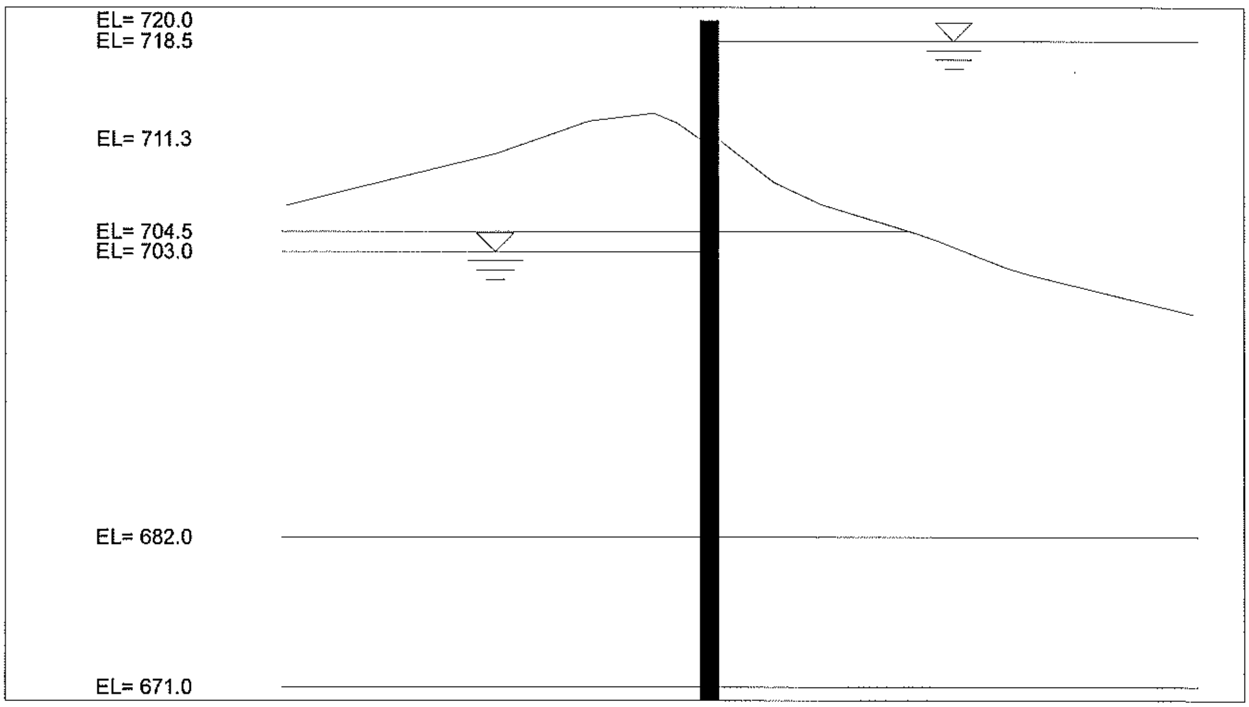
III.--WATER AND SOIL PRESSURES

		SOIL PRESSURES			
		LEFTSIDE		RIGHTSIDE	
ELEVATION (FT)	WATER PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
720.00	0.	0.	0.	0.	0.
719.00	0.	0.	0.	0.	0.
718.50	0.	0.	0.	0.	0.
718.00	31.	0.	0.	0.	0.
717.00	94.	0.	0.	0.	0.
716.00	156.	0.	0.	0.	0.
715.00	218.	0.	0.	0.	0.
714.00	281.	0.	0.	0.	0.
713.00	343.	0.	0.	0.	0.
712.00+	406.	0.	0.	0.	0.
712.00+	406.	933.	0.	0.	933.
711.00	468.	1508.	0.	0.	1008.
710.00	530.	1646.	0.	0.	1050.
709.54	593.	1709.	0.	0.	1070.
709.00	655.	1783.	0.	0.	1093.
708.00	718.	1919.	0.	0.	1134.
707.00	780.	2018.	0.	0.	1175.
706.76	842.	2084.	0.	0.	1216.
705.00	874.	2179.	0.	0.	1235.

* STANDARD WEDGE SOLUTION DOES NOT EXIST FOR INDICATED PRESSURE AT THIS ELEVATION.

'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION

'SECTION F



713.0	343.2	0.0	0.0	343.2	343.2	0.0	0.0
712.0	405.6	0.0	0.0	405.6	405.6	0.0	0.0
711.3+	449.3	0.0	0.0	449.3	449.3	0.0	0.0
711.3-	449.3	933.3	0.0	-484.1	1382.6	0.0	933.3
711.0	468.0	1501.3	0.0	-1033.3	1444.7	0.0	976.7
710.3	511.7	1604.2	0.0	-1092.5	1517.9	0.0	1006.2
710.0	530.4	1648.3	0.0	-1117.9	1549.3	0.0	1018.9
709.0	592.8	1793.6	0.0	-1200.8	1653.9	0.0	1061.1
708.0	655.2	1937.8	0.0	-1282.6	1758.4	0.0	1103.2
707.0	717.6	2082.5	0.0	-1364.9	1861.6	0.0	1144.0
706.0	780.0	2216.7	0.0	-1436.7	1964.3	0.0	1184.3
705.0	842.4	2335.0	0.0	-1492.6	2067.8	0.0	1225.4
704.5	873.6	4704.2	0.0	-3830.6	2988.0	0.0	2114.4
704.0	904.8	5572.5	0.0	-4667.7	3052.4	0.0	2147.6
703.0	967.2	2725.7	0.0	-1758.5	1456.8	0.0	489.6
702.0	967.2	2892.7	0.0	-1925.5	1491.5	0.0	524.3
701.0	967.2	3028.1	0.0	-2060.9	1539.1	0.0	571.9
700.0	967.2	3169.9	0.0	-2202.7	1866.4	0.0	899.2
699.0	967.2	3319.4	0.0	-2352.2	2049.6	0.0	1082.4
698.0	967.2	3381.2	0.0	-2414.0	2129.3	0.0	1162.1
697.0	967.2	3381.0	0.0	-2413.8	2142.2	0.0	1175.0
696.0	967.2	3434.8	0.0	-2467.6	2294.9	0.0	1327.7
695.0	967.2	3488.9	0.0	-2521.7	2389.7	0.0	1422.5
694.0	967.2	3540.8	0.0	-2573.6	2471.3	0.0	1504.1
693.0	967.2	3607.8	0.0	-2640.6	2576.6	0.0	1609.4
692.0	967.2	3630.0	0.0	-2662.8	2660.6	0.0	1693.4
691.0	967.2	3675.3	0.0	-2708.1	2766.7	0.0	1799.5
690.0	967.2	3765.6	0.0	-2798.4	2862.6	0.0	1895.4
689.0	967.2	3814.5	0.0	-2847.3	2951.9	0.0	1984.7
688.0	967.2	3820.7	0.0	-2853.5	3038.1	0.0	2070.9
687.0	967.2	3840.0	25.5	-2872.8	3103.9	0.0	2162.2
686.0	967.2	3855.5	331.1	-2822.7	2892.5	65.6	2256.3
685.0	967.2	3868.1	617.1	-2610.8	2700.6	290.1	2350.5
684.0	967.2	3811.8	632.2	-2407.4	2779.7	437.2	2444.7
683.0	967.2	3744.8	667.0	-2335.5	2839.2	442.1	2539.0
682.0	967.2	5117.3	794.8	-3609.9	3466.1	540.1	3293.7
681.0	967.2	5693.4	756.0	-4223.3	3914.3	502.9	3703.1

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

DATE: 26-OCTOBER-2016 TIME: 15:02:43

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

I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'SECTION F

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :      705.54
      PENETRATION (FT)     :        5.76

MAX. BEND. MOMENT (LB-FT)  :  5.2399E+03
      AT ELEVATION (FT)    :      709.72

MAX. SCALED DEFL. (LB-IN^3):  4.4080E+08
      AT ELEVATION (FT)    :      720.00
```

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY 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: 26-OCTOBER-2016 TIME: 15:02:43

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

I.--HEADING
'BRANSON FLOOD WALL DESIGN - CANTILEVER SHEET PILE EVALUATION
'SECTION F

II.--RESULTSO. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
720.00	0.0000E+00	0.	4.4080E+08	0.00
719.00	4.3656E-11	0.	3.9588E+08	0.00
718.50	8.7311E-11	0.	3.7343E+08	0.00
718.00	1.3000E+00	8.	3.5097E+08	31.20
717.00	3.5100E+01	70.	3.0606E+08	93.60
716.00	1.6250E+02	195.	2.6123E+08	156.00
715.00	4.4590E+02	382.	2.1670E+08	218.40
714.00	9.4770E+02	632.	1.7297E+08	280.80
713.00	1.7303E+03	944.	1.3092E+08	343.20
712.00	2.8561E+03	1318.	9.1911E+07	405.60
711.30	3.8818E+03	1617.	6.7431E+07	449.28
711.30	3.8818E+03	1617.	6.7431E+07	-484.05
711.00	4.3370E+03	1390.	5.7893E+07	-1033.30
710.30	5.0518E+03	646.	3.8354E+07	-1092.54
710.00	5.1960E+03	314.	3.1258E+07	-1117.93
709.00	4.9374E+03	-845.	1.3440E+07	-1200.82
708.68	4.6047E+03	-1234.	9.5798E+06	-1227.00
708.00	3.5365E+03	-1830.	3.9843E+06	-526.39
707.00	1.6152E+03	-1841.	5.6347E+05	504.30
706.00	1.9821E+02	-821.	6.2600E+03	1535.00
705.54	0.0000E+00	0.	0.0000E+00	2012.25

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY 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)
720.00	0.	0.	0.	0.	0.
719.00	0.	0.	0.	0.	0.
718.50	0.	0.	0.	0.	0.
718.00	31.	0.	0.	0.	0.
717.00	94.	0.	0.	0.	0.
716.00	156.	0.	0.	0.	0.
715.00	218.	0.	0.	0.	0.
714.00	281.	0.	0.	0.	0.
713.00	343.	0.	0.	0.	0.
712.00	406.	0.	0.	0.	0.
711.30+	449.	0.	0.	0.	0.
711.30+	449.	933.	0.	0.	933.
711.00	468.	1501.	0.	0.	977.
710.30	512.	1604.	0.	0.	1006.
710.00	530.	1648.	0.	0.	1019.
709.00	593.	1794.	0.	0.	1061.
708.68	655.	1840.	0.	0.	1075.
708.00	718.	1938.	0.	0.	1103.
707.00	780.	2083.	0.	0.	1144.
706.00	842.	2217.	0.	0.	1184.
705.54	874.	2335.	0.	0.	1225.
704.50	905.	4704.	0.	0.	2114.

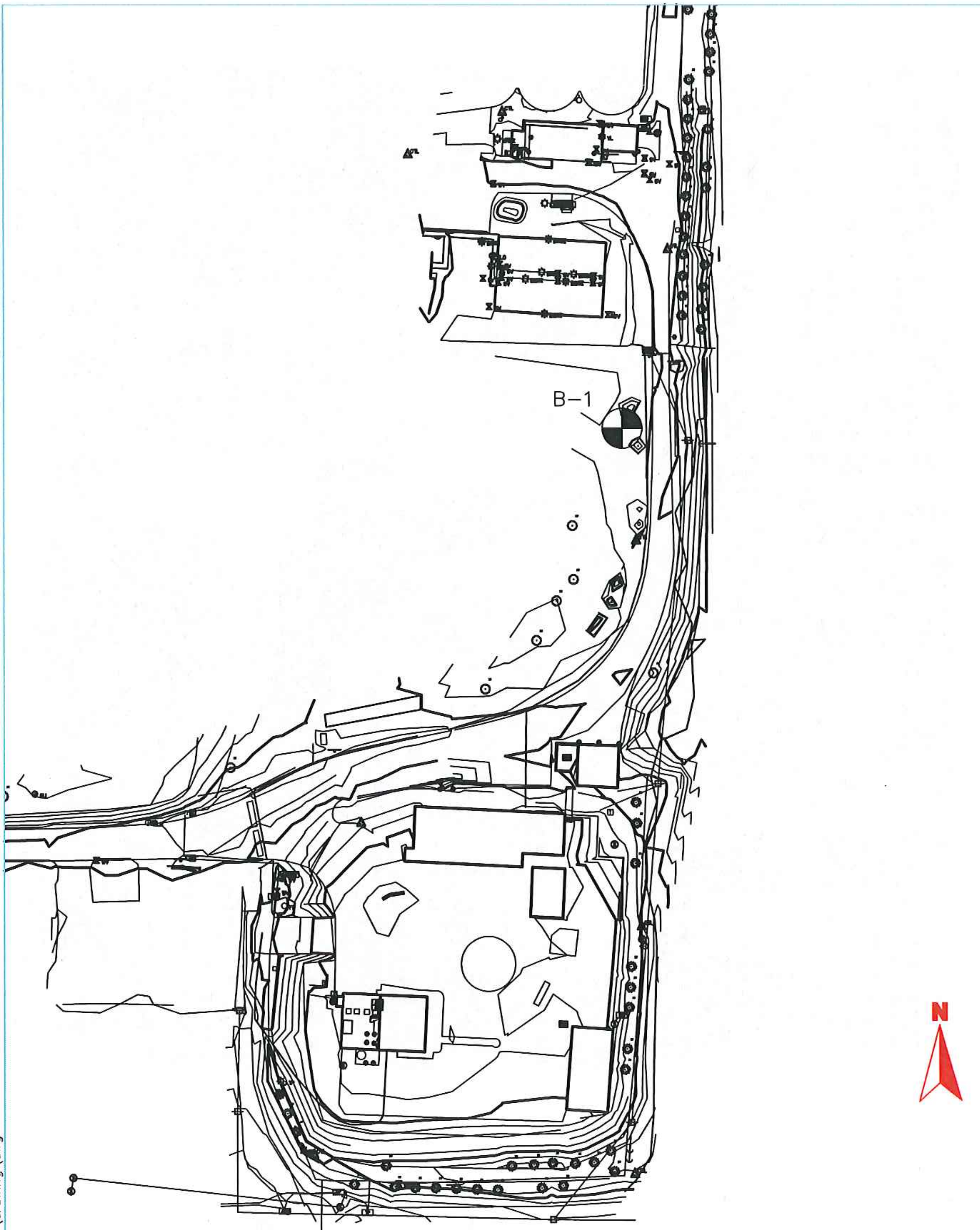
BLACK & VEATCH
Building a world of difference.

APPENDIX C
Boring Information

Figure No. 1
Site Location Plan

OLSSON ASSOCIATES

Scale: nts	**Branson Compton Drive WWTP-Bar Screen**
Project No. 008-1323	**Branson, Missouri**
Approved by: KRP	
Date: 1/9/2008	



LEGEND

BORING LOCATION

0 50 100 200
SCALE IN FEET

PROJECT: 008-1323

DRAWN BY:KRP REVISIONS: XXX

DATE: 9.8.08

BORING LOCATION MAP
BRANSON, MO

OLSSON ASSOCIATES

7301 West 133rd Street
Suite 200
Overland Park. KS 66213

TEL 913.381.1170
FAX 913.381.1174
www.oaconsulting.com

F:\Projects\008-1323\GTECH\drawing\dwg

SYMBOLS AND NOMENCLATURE

DRILLING NOTES

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon Sample
U:	Thin-walled Tube Sample
% Rec:	Percentage of Thin-walled Tube sample recovered
SPT Blow Counts:	Standard Penetration Test blows per 6" penetration
HSA:	Hollow Stem Auger
CFA:	Continuous Flight Auger
N.E.:	Not Encountered
N.A.:	Not Available

DRILLING PROCEDURES

Soil sampling and standard penetration testing performed in accordance with ASTM D 1586. The standard penetration resistance (SPT) N value is the number of blows of a 140 pound hammer falling 30 inches to drive a 2 inch O.D., 1.4 inch I.D. split-spoon sampler one foot. The thin-walled tube sampling procedure is described by ASTM specification D 1587.

WATER LEVEL MEASUREMENTS

Water levels indicated on the boring logs are levels measured in the borings at the times indicated. In relatively high permeable materials, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels is not possible with only short-term observations.

SOIL PROPERTIES & DESCRIPTIONS

Soil descriptions are based on the Unified Soil Classification System (USCS) as outlined in ASTM Designations D-2487 and D-2488. The USCS group symbol shown on the boring logs correspond to the group names listed below.

Group Symbol	Group Name	Group Symbol	Group Name
GW	Well Graded Gravel	CL	Lean Clay
GP	Poorly Graded Gravel	ML	Silt
GM	Silty Gravel	OL	Organic Clay or Silt
GC	Clayey Gravel	CH	Fat Clay
SW	Well Graded Sand	MH	Elastic Silt
SP	Poorly Graded Sand	OH	Organic Clay or Silt
SM	Silty Sand	PT	Peat
SC	Clayey Sand		

PARTICLE SIZE

Boulders	12 in. +	Coarse Sand	4.75mm-2.0mm	Silt	0.075mm-0.005mm
Cobbles	12 in.-3 in.	Medium Sand	2.0mm-0.425mm	Clay	<0.005mm
Gravel	3 in.-4.75mm	Fine Sand	0.425mm-0.075mm		

COHESIVE SOILS

Consistency	Unconfined Compressive Strength (Qu) (psf)
Very Soft	<500
Soft	500 - 1000
Firm	1001 - 2000
Stiff	2001 - 4000
Very Stiff	4001 - 8000
Hard	> 8000

COHESIONLESS SOILS

Relative Density	Angle Value
Very Loose	0 - 3
Loose	4 - 9
Medium Dense	10 - 29
Dense	30 - 49
Very Dense	≥ 50

OLSSON ASSOCIATES

TEST BORING REPORT

BORING NO. B-1

PROJECT: Branson Compton Drive WWTP
CLIENT: City of Branson, Missouri
DRILLING CONTRACTOR: Anderson Engineering
EQUIPMENT USED: ATV Rig

JOB NO. 008-1323
PAGE NO. 1 of 2
LOCATION: See Plans
ELEVATION: 714.0'
DATE START: 9/12/08
DATE FINISH: 9/12/08
DRILLER: GW
PREPARED BY: KP

GROUNDWATER		DEPTH TO:				CASING	SAMPLER	CORE BARREL
DATE	HRS AFTER COMP	WATER	BOTTOM OF CASING	BOTTOM OF HOLE	TYPE			
9/12/08	IAD	10.0'	----	43.5'	SIZE ID			
					HAMMER WT			
					HAMMER FALL			

DEPTH IN FEET	SAMPLER BLOWS PER 6 INCHES	SAMPLE NUMBER	SAMPLE DEPTH RANGE	MOISTURE (%)	FIELD CLASSIFICATION AND REMARKS
					0.5' -TOPSOIL-
		U-1	0.0' ---- 2.0'	---	-FILL-
	5 / 4 / 5	SS-2	2.0' ---- 3.5'	20.8	Stiff, brown mottled with gray and red, moist, lean to fat clay with few chert
5		U-3	3.5' ---- 5.5'	---	
					Driller's Note: Groundwater encountered @ 10.0' Percent Passing the #200 sieve @ 8.5' – 61.5%
10	0 / 3 / 3	SS-4	8.5' ---- 10.0'	35.6	Firm, brown mottled with gray and black, very moist, lean clay with sand 9.5'
	2 / 3 / 2	SS-5	10.0' ---- 11.5'	21.4	Loose, gray mottled with brown, wet
					-SITLY CLAYEY SAND-
15	0 / 0 / 0	SS-6	13.5' ---- 15.0'	26.3	Very loose, gray mottled with brown, wet
20	0 / 0 / 2	SS-7	18.5' ---- 20.0'	30.5	Very loose, gray mottled with brown, wet

BLOWS/FT	DENSITY	BLOWS/FT	CONSISTENCY	SAMPLE ID.		COMPONENT %		GROUNDWATER ABBREV
0-3	VERY LOOSE	0-1	VERY SOFT	SS	SPLIT SPOON	MOSTLY	50-100 %	WD - WHILE DRILLING
4-9	LOOSE	2-4	SOFT	U	TUBE	SOME	30-45 %	NE - NOT ENCOUNTERED
10-29	MEDIUM DENSE	5-8	FIRM	CA	CALIFORNIA	LITTLE	15-25 %	UR - NOT READ
30-49	DENSE	9-15	STIFF	G	GRAB SAMPLE	FEW	5-10 %	
>49	VERY DENSE	16-30	VERY STIFF	X	OTHER	TRACE	< 5 %	
		>30	HARD	NR	NO RECOVERY			BORING NO. B-1

OLSSON ASSOCIATES

TEST BORING REPORT

BORING NO. B-1

DEPTH IN FEET	SAMPLER BLOWS PER 6 INCHES	SAMPLE NUMBER	SAMPLE DEPTH RANGE	MOISTURE (%)	FIELD CLASSIFICATION AND REMARKS
					-SILTY CLAYEY SAND-
25	0 0 0	SS-8	23.5' ---- 25.0'	23.6	Very loose, gray mottled with brown, wet
					Percent Passing the #200 sieve @ 23.5' – 33.7%
30	0 0 5	SS-9	28.5' ---- 30.0'	28.5	Loose, gray mottled with brown, wet
					32.0'
					-POORLY GRADED SAND-
35	13 17 18	SS-10	35.0' ---- 36.5'	23.9	Dense, light brown to yellow, wet, little chert and gravel
40	5 4 6	SS-11	40.0' ---- 41.5'	20.4	Medium dense, light brown to yellow, wet, little chert and gravel
					43.0'
					-LIMESTONE-
					Base of boring @ 43.5'
45					

BLOWS/FT	DENSITY	BLOWS/FT	CONSISTENCY	SAMPLE ID.		COMPONENT %		GROUNDWATER ABBREV
0-3	VERY LOOSE	0-1	VERY SOFT	SS	SPLIT SPOON	MOSTLY	50-100 %	WD - WHILE DRILLING
4-9	LOOSE	2-4	SOFT	U	TUBE	SOME	30-45 %	NE - NOT ENCOUNTERED
10-29	MEDIUM DENSE	5-8	FIRM	CA	CALIFORNIA	LITTLE	15-25 %	UR - NOT READ
30-49	DENSE	9-15	STIFF	G	GRAB SAMPLE	FEW	5-10 %	
>49	VERY DENSE	16-30	VERY STIFF	X	OTHER	TRACE	< 5 %	
		>30	HARD	NR	NO RECOVERY			BORING NO. B-1

APPENDIX E

PRODUCT DATA OF TEMPORARY FLOOD PROTECTION OPTIONS

Floodgate and Building Modification Alternatives

Presray Flood Gates

Presray- Model CG3S

PRESRAY
Critical Containment Solutions
www.Presray.com

CG3S

Sliding Flood Panel With Compression Gasket

DESIGNED FOR

- Keeping flood water out of building openings or perimeter flood walls. Ideal for quick deployment requirements where a flush bottom sill is required.

PROTECTION TO

- Custom designed to match any size needs.

SEAL TYPE

- Compression, fully molded with molded corners
- Neoprene standard, viton available

SEAL AREA

- 3 Sides-Floor & Both Sides

UNIQUE FEATURES

- Compression gasket provides maximum protection with minimum maintenance
- No compressed air required
- Panel slides effortlessly into place when needed, stays hidden behind wall when not in use
- Sill trench covered by plate, no tripping hazard
- Frame is concealed by hinged cover plate

INSTALLATION

- Available for new or existing construction
- For existing openings, the frame is mounted to the face of the building using expansion anchors. A trench is cut into the existing concrete floor and the sill is cast in place. Once the frame and sill are secured, the storage side of the frame can be covered over using standard construction material.

PRESRAY
P.O. BOX 200, 32 NELSON HILL ROAD, WASSAIC, NY 12592
P: 845.373.9300 • F: 845.855.8034 • E: CONTACT@PRESRAY.COM

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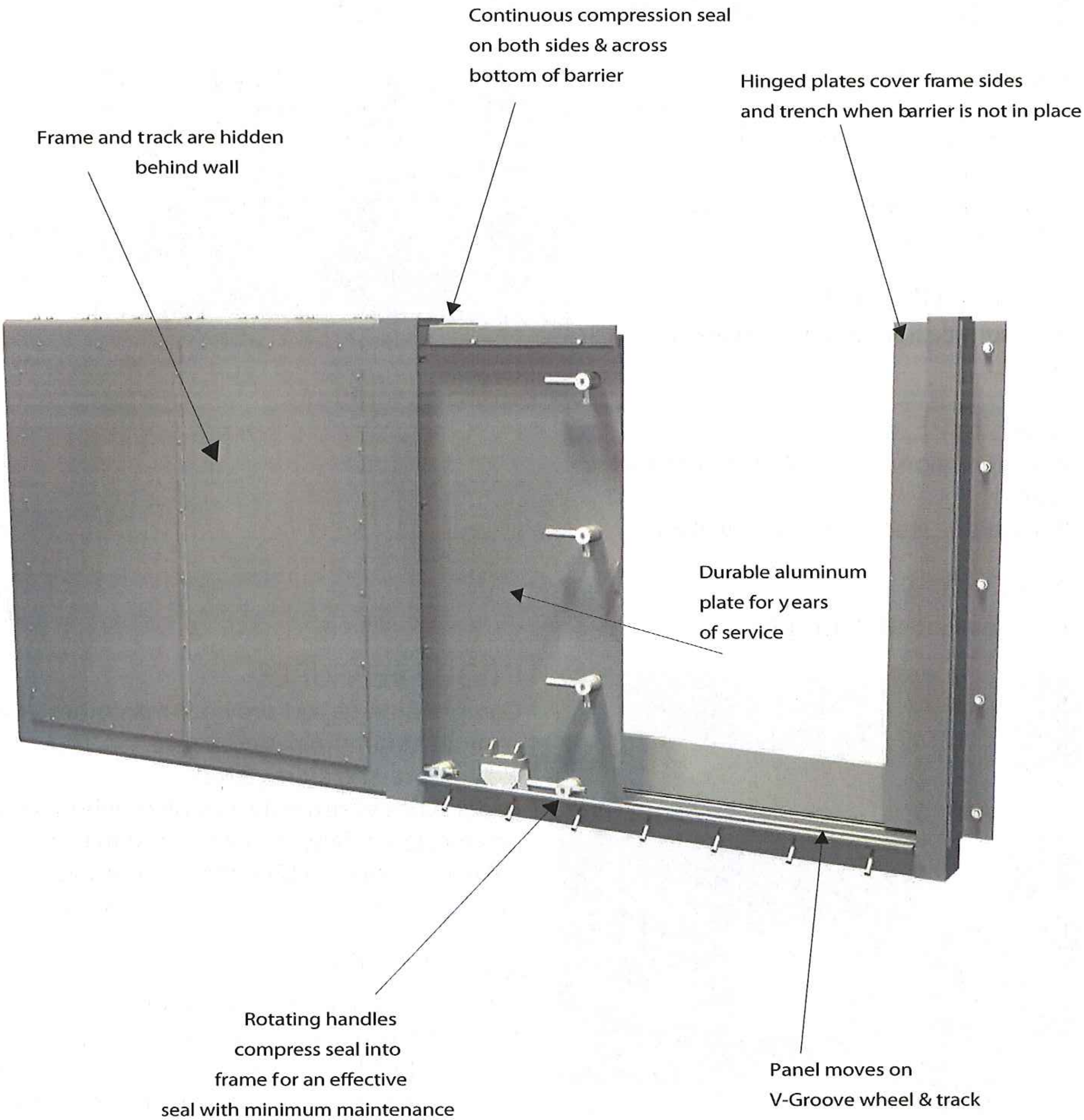
Critical Containment Solutions
www.Presray.com

CG3S Design



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Critical Containment Solutions
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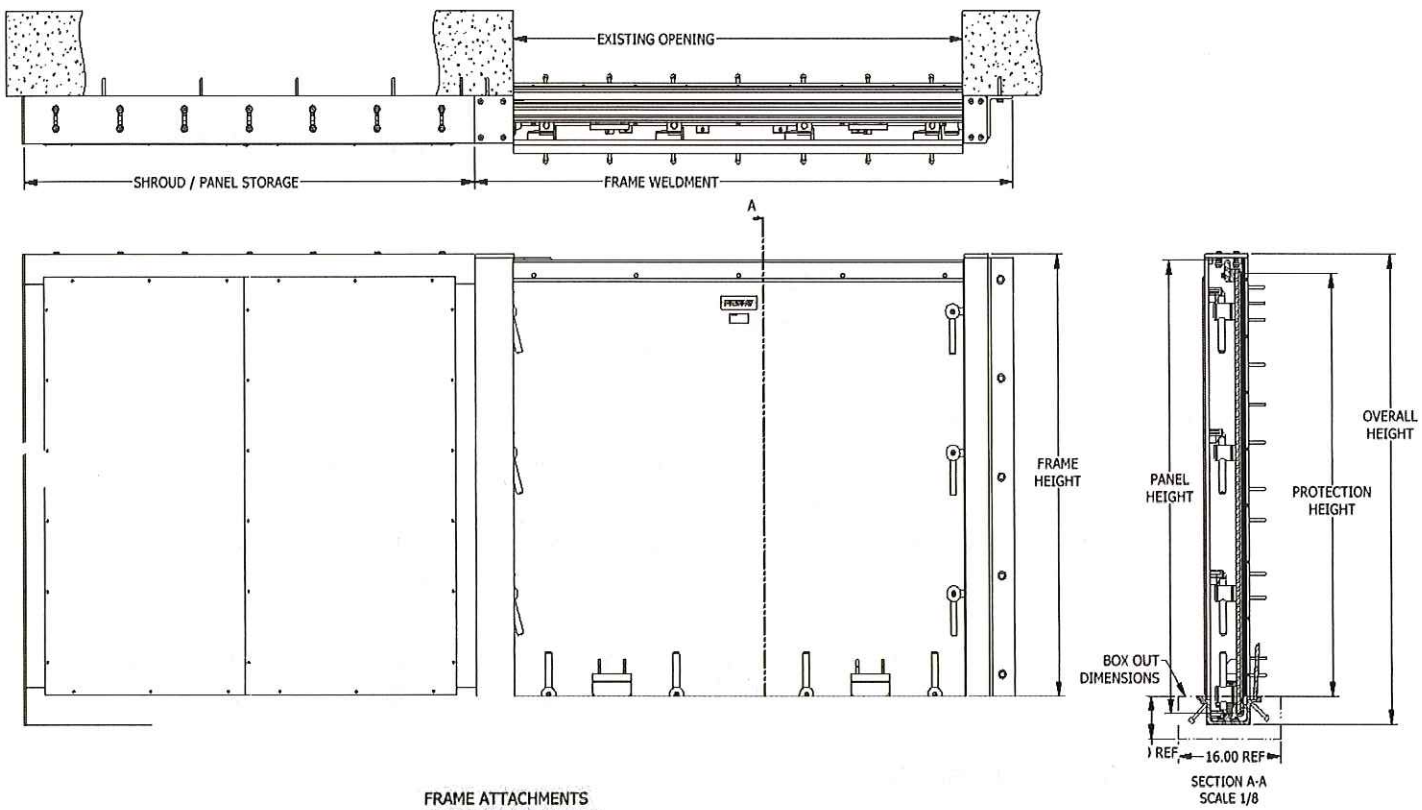
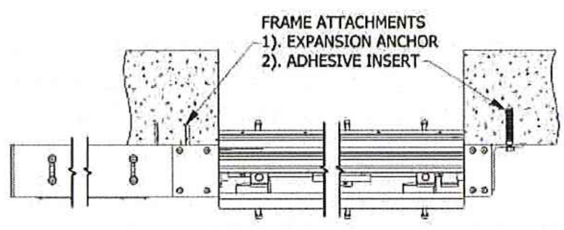
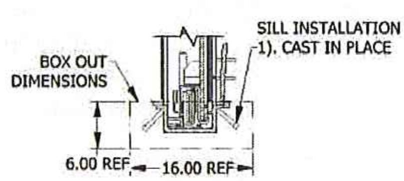
CG3S
Installation Detail





CG3S INSTALLATION



CG3S INSTALLATION

The unique design of the CG3S allows the barrier to stay at the opening, yet out of sight! When not in use the barrier panel sits behind the wall, with the jamb sides & sill hidden by a cover plate. When needed, the hinged sill & jamb cover plates are opened revealing the track, jamb & barrier panel. The panel is then rolled into position and secured using the quarter turn handles. Your opening is secured in under 1 minute with nothing to lift, no screws to remove, no compr essed air needed!
Models are available in all sizes for new or existing construction. For new construction, the sill is poured in place and the frame is bolted onto the wall using expansion anchors For existing openingsa trench is cut into the floor to receive the sill. Once the sill is in place, concrete is poured to secure it. After the sill is completed the fame is installed using expansion anchors.

PRESRAY
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P: 845.373.9300 • F: 845.855.8034 • E:CONTACT@PRESRAY.COM

PRESRAY
Critical Containment Solutions

Presray Corporation
Critical Containment
Solutions
P.O. Box 200
Wassaic, NY 12592

Phone: (845) 373-6700
Fax: (845) 855 – 8034
www.presray.com
Email: technicalsupport@presray.com

Suggested Specifications For Model CG3S Pocketed Sliding Door

Part 1 - General

- 1.01 Description
  - A. **Work Included:** Provide special door(s) factory assembled with frame(s) arid hardware in accordance with the contract documents.
- 1.02 Standards
  - A. Comply with the provisions of (as applicable).
    1. AWS Structural Welding Code.
    2. ASME Structural Welding Code Section IX.
- 1.03 Submittals
  - A. **Manufacturers Data:** Submit installation and maintenance instructions for flood barriers.
  - B. **Shop Drawings:** Submit shop drawings for flood barriers including dimensioned plans and elevations, sections, connections and anchorage, and parts list.
  - C. **Calculations *(Optional)*:** Submit calculations, approved by a qualified engineer, to verify the barrier's ability to withstand the design pressure loading.
- 1.04 Qualifications
  - A. **Experience:** The manufacturer of the flood barrier(s) shall present evidence attesting to at least 5 years of successful experience in the design and manufacture of both the flood barrier and flood barrier seal of the type specified.

Part 2 - Products

- 2.01 Watertight barrier shall be CG3S as manufactured by Presray Corporation.
- 2.02 Materials
- Materials
  - A. **Panel:** 6061 T6 Aluminum plate.
  - B. **Conversion Frame & Track:** Low carbon steel (stainless steel & aluminum optional).
  - C. **Finish:** Panel, bright aluminum finish. Conversion frame, brush-off blast clean per SSPC-SP7, primed with one coat rust inhibitive, lead free, red primer.

D. **Door Gasket:** Presray type 25 durometer neoprene, molded with fully molded corners, no mitered joints allowed. (Optional materials include Viton, consult Presray in cases of unusual environmental conditions).

E. **Hardware**

Shrouds: Hinged 6061 Aluminum (other materials optional)

F. **Compression Handles:** Presray Type handles with stainless steel rollers and provisions for adjusting seal compression after installation.

- 2.03 Design

A. Watertight barrier shall be designed with applicable safety factors in accordance with AISC specifications, and shall provide an effective seal against the design pressure.

B. The design of the door shall allow the pressure on the door to be transmitted to the frame and/or dogs.

C. Frame shall include suitable anchors for embedment in concrete (options available include strap anchors for mounting in new masonry block walls, gaskets, bolts and inserts for attachment to existing concrete or block, or the frame ready for welding to existing steel structure).

- 2.04 Fabrication

A. The coaming edge contacting the door gasket shall be machined, rather than as rolled, to maximize sealing.

- 2.05 Inspection and Test

A. All steel material welds in the potential "leak path" shall be liquid penetrant inspected in accordance with ASME Code of Section VIII Div.1 of Appendix 8.

B. Finished assembly, or assembly similar in design, shall be factory leak tested in accordance with ASTM E283.

Part 3 — Execution

- 3.01 Installation

A. Install special doors in accordance with manufacturer's instructions and approved shop drawings.

Part 4 — Warranty

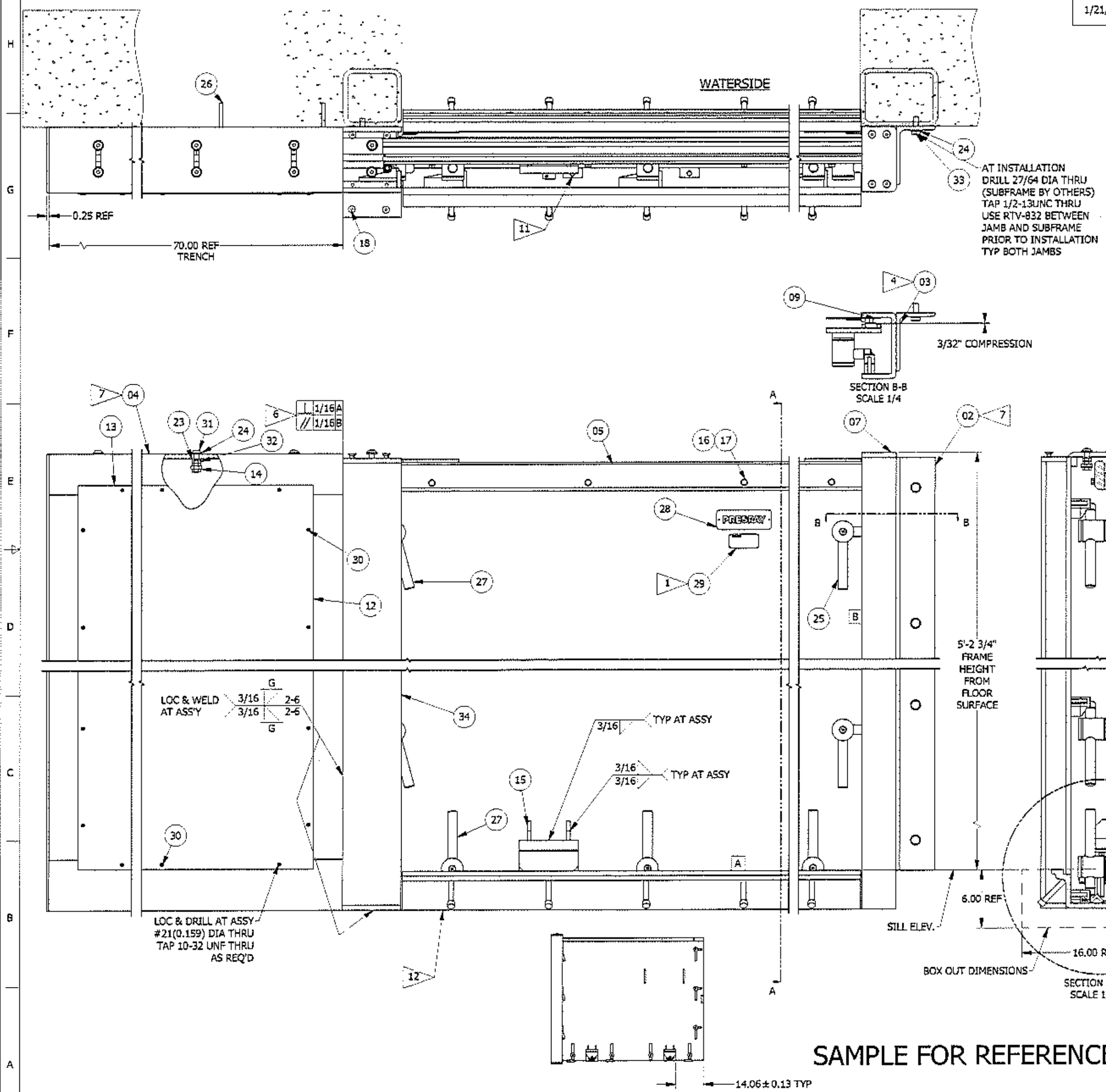
- 4.01 1-year limited against defects and workmanship from date of shipment.

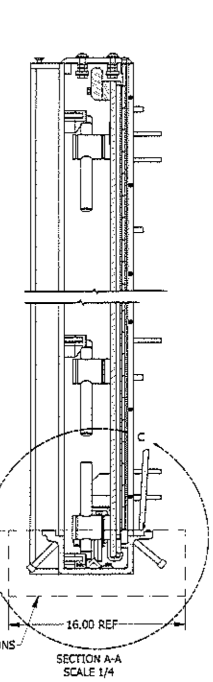
DATE	REV	SHT	DESCRIPTION	DR	CHK	APVD	QA
1/21/09	A	1,3	ADD DETAIL 34, WALL SUPPORT TO PREVENT DEFLECTION OF FRAME WALL BEHIND HANDLES WHEN SEAL IS COMPRESSED. EXTEND DET 06 TO COVER DET 34.	EH	JF	JF	BAH

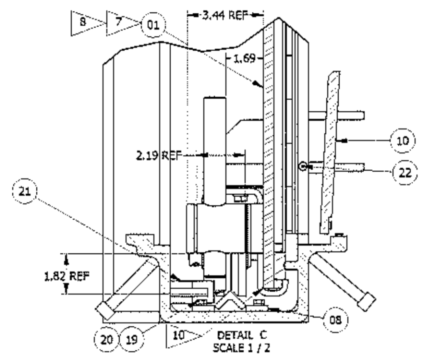
NOTES:

1. IMPRESSION STAMP DATE, SERIAL NO., ASSEMBLY NO. & OPENING NO.
2. APPROXIMATE WEIGHT OF PANEL: 170Lbs., ASSEMBLY: 800 LBS.
3. FOR INSTALLATION & OPERATION INSTRUCTION SEE PRESRAY MANUAL IMCG3S-10972-01.
4. SEAL MATERIAL TO BE MOLDED & VULCANIZED.
5. MASK SURFACE PRIOR TO PAINTING, CLEAN SURFACE AND BOND SEAL IN PLACE WITH RTV.
6. ALL TOLERANCES NON-ACCUMULATIVE.
7. BRUSH-OFF BLAST CLEAN PANEL AND FRAME PER SSPC-SP7.

   PRIME PANEL ONE COAT: INSLX-UNIVERSAL PRIMER GRAY #MP-9000.
   PAINT PANEL ONE COAT: INSLX-BRILLIANT ALUMINUM HIGH GLOSS #AL-2000.
   PAINT FRAME WITH INSL-X #EP5854 GREY EPOXY.
8. PANEL TO BE FLAT WITHIN 1/8 OVER ENTIRE LENGTH.
9. MACHINED COAMING MUST BE SMOOTH & UNINTERRUPTED BY STEPS GREATER THAN .015 & FREE OF CRACKS. FINISH LAY TO BE PARALLEL TO SEAL.
10. BEVEL EDGES OF PLATE AND RETAINER AS NECESSARY AT ASSY.
11. DRILL #7(0.201), TAP 1/4-20UNC X AR HOLES AT ASS'Y
12. DRAINAGE PROVISION REQUIRED FOR TRENCHES (BY OTHERS).







QTY	ITEM			MATERIAL	DESCRIPTION
1	35			SS	SHROUD TOOL
1	34			AL	WALL SUPPORT
11	33			SS	HEX HD SCREW
48	32			SS	JAMB NUT
16	31			SS	BUTT HD CAP SCREW
30	30			SS	BUTT HD SCREW
1	29			AL	DATA PLATE
1	28			AL	NAME PLATE
7	27			STL	HANDLE ASSY
13	26			ZPS	3/8" HEX HD EXPANSION ANCHOR
3	25			STL	HANDLE ASSY
27	24			SS	FLAT WASHER
16	23			SS	LOCK WASHER
34	22			SS	FLAT HD SCREW
10	21			AL	WEDGE ASSY
24	20			SS	LOCK WASHER
24	19			SS	HEX HD SCREW
8	18			SS	FLAT HD SCREW
5	17			SS	HEX HD SCREW
5	16			SS	LOCK WASHER
2	15			SS	WHEEL ASSEMBLY
16	14			SS	ROLLER
1	13	7		AL	SHROUD COVER
1	12	7		AL	SHROUD COVER
1	11	6		AL	TRENCH COVER HINGE
1	10	6		AL	TRENCH COVER
2	09	6		AL	JAMB SHROUD
2	08	5		SS	TRACK WELDMENT
1	07	5		AL	SMALL CAP
1	06	6		AL	LARGE COVER
1	05	5		AL	TRACK BAR
1	04	4		AL	STORAGE WELDMENT
1	03	4		EPDM	COMPRESSION SEAL
1	02	3		AL	FRAME WELDMENT
1	01	2		AL	PANEL WELDMENT

SAMPLE FOR REFERENCE - CG3S

THE PRESRAY CORP.
WASSAIC, NEW YORK

NAME: CG3S

MATERIAL

PAINT/FINISH

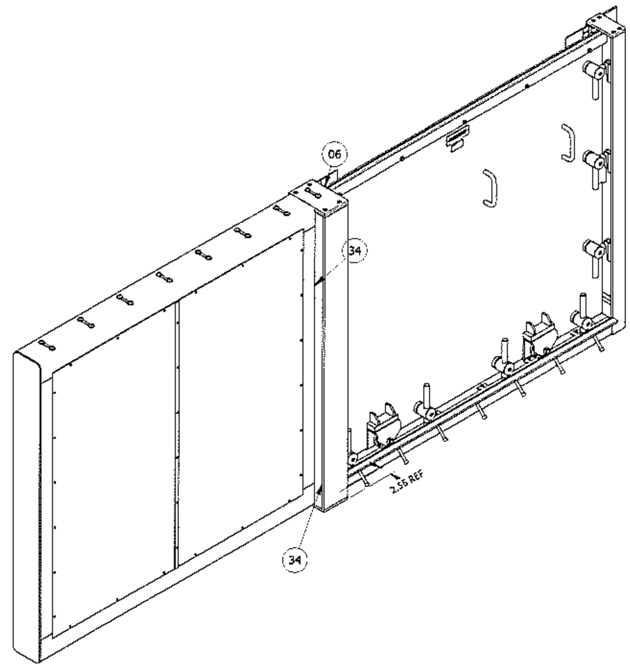
QC REVIEW OK

9/29/2008 EH

CG3S-10972-01 A

SHEET 1 OF 2

19428



THE PRESRAY CORP.
WASSAIC, NEW YORK

MATERIAL
PAINT/FINISH
TOLERANCE UNLESS NOTED
QC REVIEW OK
9/29/2000
CG3S-10972-01
SHEET 2 OF 2

Presray- Model FB44

PRESRAY

Critical Containment Solutions

www.Presray.com

FB44

Side Hinged Aluminum Panel With Inflatable Seals

DESIGNED FOR

- Keeping flood water out of building openings or perimeter flood walls. Ideal for quick deployment requirements where a flush bottom sill is required.

PROTECTION TO

- Custom designed to match any size needs.

SEAL TYPE

- Dual inflatable for redundant protection

SEAL AREA

- 3 Sides, sill & both sides

UNIQUE FEATURES

- 3/8"thick sill can be recessed to prevent tripping hazzard
- Hinged panel glides effortlessly into place
- Dual seals provide redundant protection
- Seals can be inflated by a hand pump, compressed air tank, or air compressor
- Slide latches secure panel when in place

INSTALLATION

- Available for new or existing construction
- For existing openings, frame is mounted to the opening using expansion anchors or epoxy type anchors
- For new construction the frame can be poured in place or anchors can be used similar to existing

PRESRAY

P.O. BOX 200, 32 NELSON HILL ROAD, WASSAIC, NY 12592

P: 845.373.9300 • F: 845.855.8034 • E:CONTACT@PRESRAY.COM

PRESRAY

Critical Containment Solutions

www.Presray.com

FB44 Design

Air connection ports for dual seals. Fill with compressed air from compressor, portable tank or hand pump

Presray designed 6 way adjustable hinge. Low friction with oil impregnated bronze bushing provide effortless motion

Slide latch locks barrier securely in opening

Panel is 6061-T6 aluminum for years of maintenance free use. Can be left natural, or painted to your specifications

Conversion frame is low carbon steel (stainless steel available), and available in face mount or, jamb mount

Dual inflatable seals provide redundant protection while ensuring a complete seal

PRESRAY
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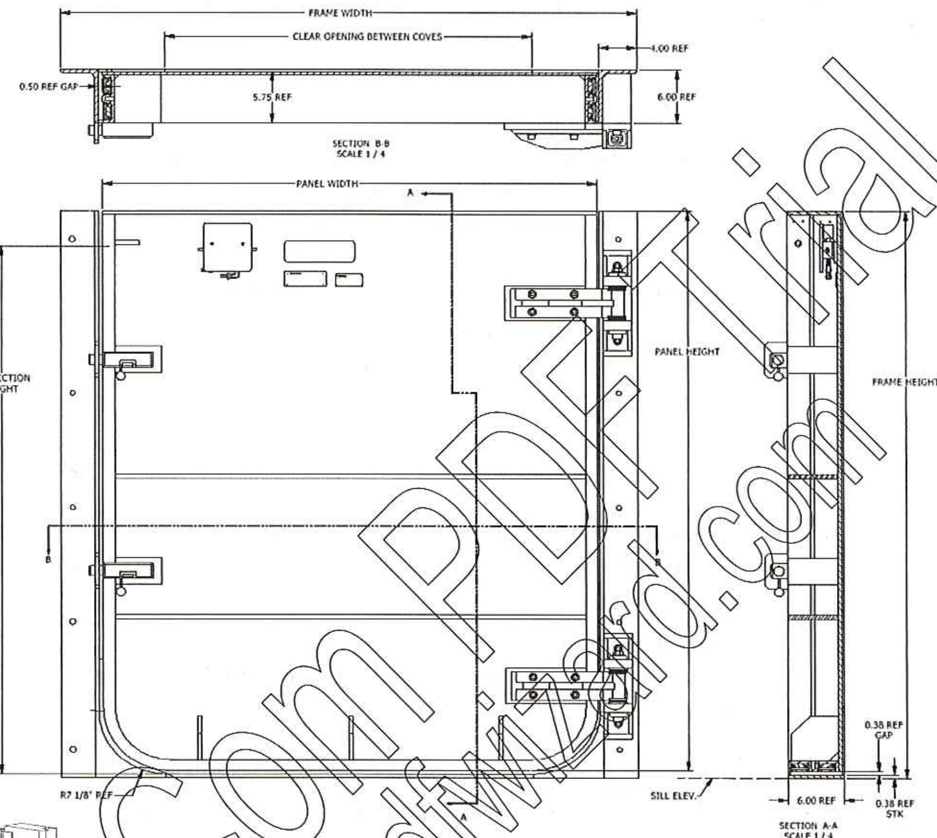
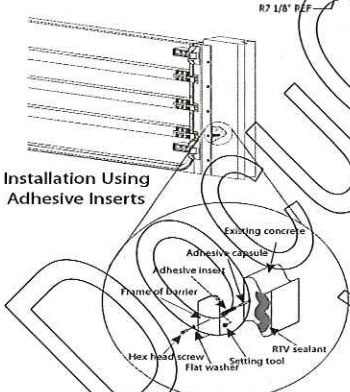
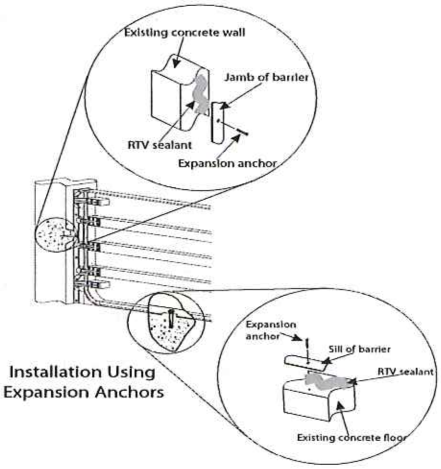
Critical Containment Solutions
www.Presray.com

FB44 Installation Detail

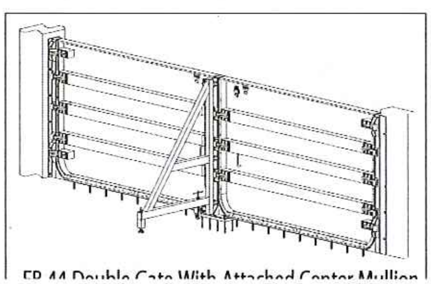




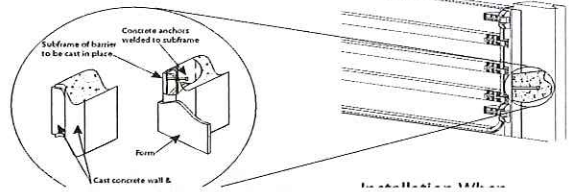


The FB44 Hinged Flood Barrier provides maximum protection by simply closing a gate! The barrier is always in place, always ready to go! In the event of a flood condition, simply close the gate, lock the latch and inflate the seals. The dual redundant seals provide excellent protection.

For large width openings, dual FB44's with attached center mullion provide fast protection. Simply close the first gate (with center mullion attached) and seal and secure the mullion to the ground. After the center mullion is secured the second gate is closed and latched. Then inflate the seals. That's it



FB 44 Double Gate With Attached Center Mullion



Installation When

PRESRAY
Critical Containment Solutions

Presray Corporation
Critical Containment
P.O. Box 200
Wassaic, NY 12592

Phone: (845) 373-6700
Fax: (845) 855 – 8034
www.presray.com
contact@presray.com

Suggested Specifications for Model FB44 Side Hinged Flood Gate with Inflatable Gaskets

Part 1 - General

- 1.01 Description

  A. **Work Included:** Provide flood barrier(s) factory assembled with frame(s) and hardware in accordance with the contract documents.

- 1.02 Standards

  A. Comply with the provisions of (as applicable).

  1. AWS Structural Welding Code.
  2. ASME Structural Welding Code Section IX.

- 1.03 Submittals

  A. **Manufacturers Data:** Submit installation and maintenance instructions for flood barriers.

  B. **Shop Drawings:** Submit shop drawings for flood barriers including dimensioned plans and elevations, sections, connections and anchorage, and parts list.

  C. **Calculations *(Optional)*:** Submit calculations, approved by a qualified engineer, to verify the barrier's ability to withstand the design pressure loading.

- 1.04 Qualifications

  A. **Experience:** The manufacturer of the flood barrier(s) shall present evidence attesting to at least 5 years of successful experience in the design and manufacture of both the flood barrier and flood barrier seal of the type specified.

Part 2 - Products

- 2.01 Flood barrier shall be Model FB44 as manufactured by Presray Corporation.

- 2.02 Materials

  A. **Panel:** Aluminum plate.

  B. **Conversion Frame:** Low carbon steel (stainless steel optional).

  C. **Finish:** Panel, bright aluminum finish. Conversion frame, brush-off blast clean per SSPC-SP7, primed with one coat rust inhibitive, lead free, red primer.

  D. **Seals:** Dual Presray type Pneuma-Seal® inflatable gaskets. Each seal shall have an automotive type air inflation stem and independent 0-60 PSI pressure gauge.

E. **Hinges:** Presray six way adjustable hinges. Low friction hinges with oil impregnated bronze bushing for radial and thrust loads.

F. **Latches:** Slide bolts to hold panel in closed position.

G. **Optional:** *For use where facility air is not available*
Air Source: Control Panel with manifold and air tanks or hand pump (portable compressors also available for multiple flood barrier installations)
Nose Wheels on leading edge for extra wide openings.
Multi-Panel Systems with removable mullions between panels for openings too wide to be accommodated by single panel.

- 2.03 Design

A. Flood barrier(s) shall be designed with a minimum 2:1 factor of safety based on material yield strength, and shall provide an effective seal against the design flood level.

B. Panel and conversion frame shall have lower corners radiused to optimize sealing.

C. Conversion frame shall have mounting holes for expansion anchors and bolts (Options available include epoxy anchors for block walls, and studs for embedment in concrete).

- 2.04 Fabrication

A. Sealing surfaces shall be finished to 63 microinch to maximize sealing, uninterrupted by steps greater than .015, free from cracks, and with finish lay parallel to seal.

B. Frame to be straight within l/8" over entire length.

- 2.05 Inspection and Test

A. Proof test and leak test inflatable seals per Presray standard practice.

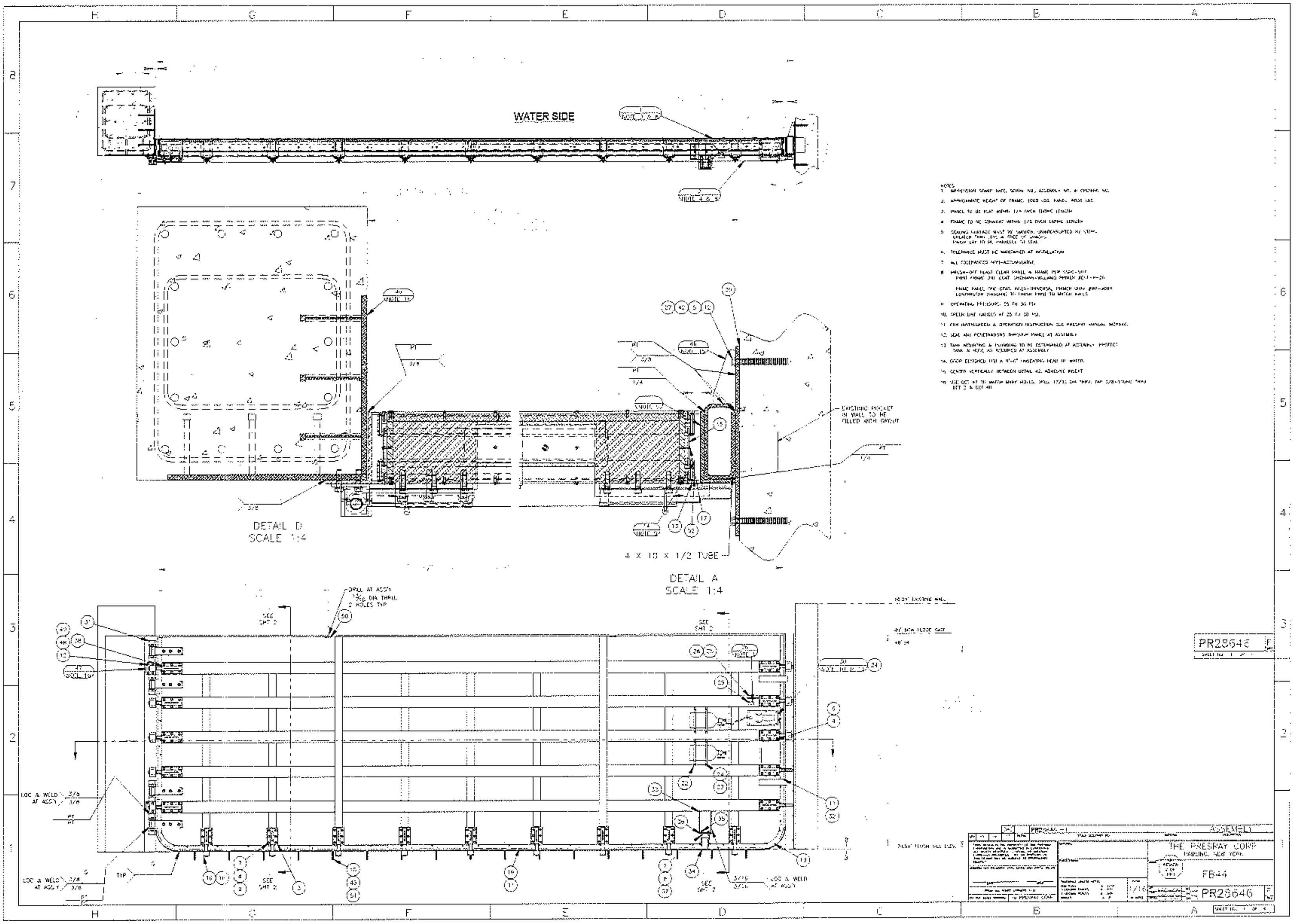
Part 3 — Execution

- 3.01 Installation

A. Install flood barriers in accordance with manufacturer's instructions and approved shop drawings.

Part 4 — Warranty

- 4.01 1-year limited against defects and workmanship from date of shipment.


WATER SIDE
DETAIL D
SCALE 1:4
DETAIL A
SCALE 1:4
4 X 10 X 1/2 TUBE
EXISTING POCKET IN WALL TO BE FILLED WITH GROUT
DRILL AT ASSY
2 HOLES TYP
SEE SHT 2
LOC & WELD AT ASSY
TYP
THE PRESRAY CORP
PAWLING, NEW YORK
FB44
PR28646
ASSEMBLY

FloodBreak Floodgate Alternative and Building Modifications Alternative

PRODUCT DATASHEET

100% PASSIVE

FULLY AUTOMATIC

24/7 FLOOD PROTECTION

PREVENTION THROUGH INNOVATION

Vehicle Gate

The FloodBreak Vehicle Gate is a fully-engineered system that will automatically block entrances from street-level flooding. Using FloodBreak's passive flood mitigation technology, these vehicular gates provide worry-free flood protection 24/7 while allowing full access to your facility.

Driveways * Loading Docks * Garage Ramps * Equipment Bays

Revolutionary Flood Control

Like the rest of FloodBreak's passive flood mitigation product line, the Vehicle Gate is fully automatic and does not depend on people or power to deploy. It is the only practical, truly passive flood control solution - the preferred method according to FEMA.

The Smart Choice

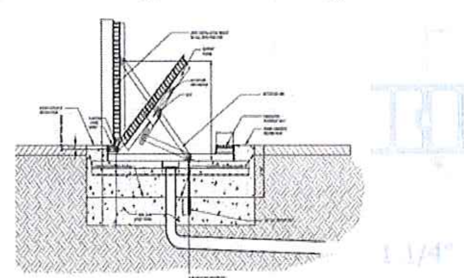
The FloodBreak Vehicle Gate has been protecting customers since 2002. Major hospitals, governments and commercial facilities all rely on FloodBreak's 24/7 flood protection. In the past two years alone, there have been 12 identified flood saving deployments.

How It Works

The FloodBreak system uses hydrostatic pressure created by the rising flood waters to automatically activate the gate. When the flood recedes, the gate automatically returns to its hidden position underneath the ground allowing full access to the facility.

Features:

- **Passive flood mitigation preferred by FEMA**
- **Manufactured to exact size requirements**
- **Weather resistant materials & durable rubber gaskets**
- **Minimal maintenance**
- **No training required**
- **Easy to install**

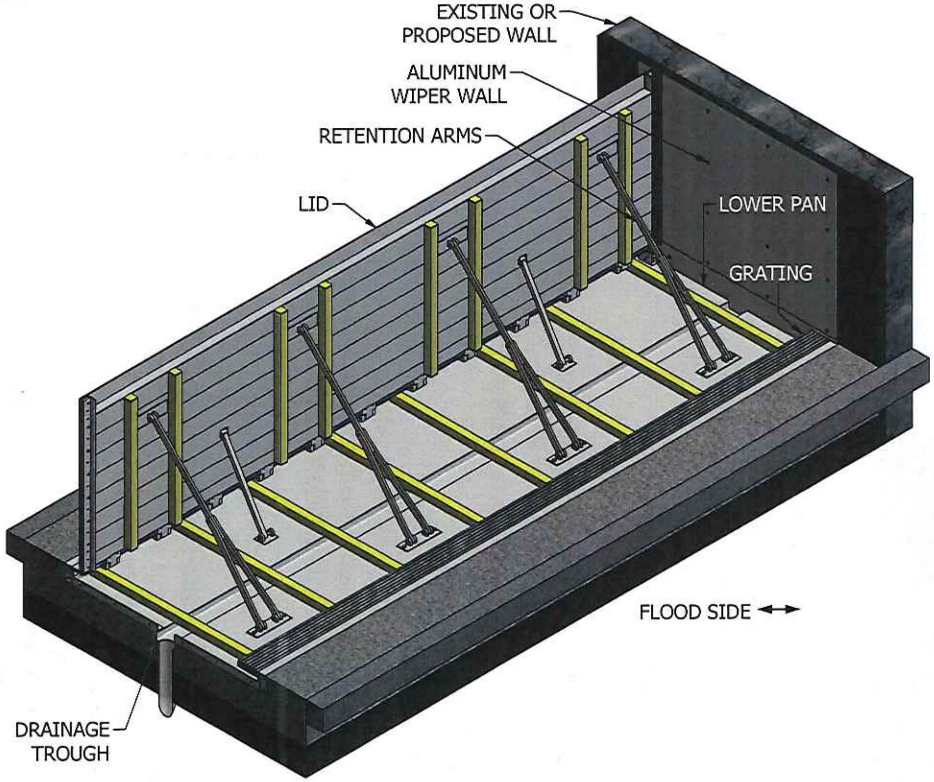


1.713.980.6610
floodbreak.com

NO PEOPLE, NO POWER

24/7 CONFIDENCE

FloodBreak
REVOLUTIONARY FLOOD CONTROL



STRUCTURAL SPECIFICATIONS:

1. FLOODGATE MATERIAL TO BE ALUMINUM AS FOLLOWS:
   LID - 5" x 2 1/2" x 1/8" ALUM EXTRUSIONS - GRADE 6005-T5 MIN. F_Y=35 KSI
   LID AND PAN - 2" x 2" x 1/4" ALUM TUBING - GRADE 6061 MIN. F_Y=40 KSI
   PAN - 1/4" SMOOTH ALUM PLATE - GRADE 5052 MIN. F_Y=30 KSI
   ALUM FLAT BARS, STRUCTURAL ANGLES, HINGES GRADE 6061-T6 MIN. F_Y=40 KSI
   ALUM CHANNELS - 4" x 2" x 1/4" VERTICAL & 6" x 2" x 1/4" HORIZONTAL.
2. HINGE BOLTS, PINS, AND MACHINE SCREWS TO BE STAINLESS STEEL - GRADE 304, MIN. F_Y=90 KSI.
3. RETENTION ARM ANCHOR BOLTS SHALL BE STAINLESS STEEL STANDARD THREAD BOLTS SET IN VINYLESTER BASED ADHESIVE CONTAINED IN A GLASS CAPSULE, INSTALLED PER SIMPSON STRONG TIE SPECIFICATIONS.
4. ALUMINUM TO BE WELDED WITH ALUMINUM WIRE - PER 4043 AWS A5.10 3/64.
5. GROUT TO BE COMMERCIAL GRADE NON-SHRINKING GROUT.
6. ALL WELDS REQUIRED FOR STRUCTURAL STRENGTH OF THE LID OR PAN ARE CALLED OUT ON THESE DRAWINGS. ALL OTHER WELDING, NOT SHOWN OR CALLED OUT ON THESE DRAWINGS, ARE ESSENTIALLY NON-STRUCTURAL WELDS OR WELDS WITH NEGLIGIBLE LOADS AND RESULTING STRESSES. EXAMPLES OF SUCH WELDS ARE AT SEAMS, SIDES, PAN TROUGH, AND LID TRIM ANGLES. THESE WELDS ARE TO BE SIZED BY THE FABRICATOR, TAKING INTO CONSIDERATION ASSEMBLY, TRANSPORT LIFT AND CONTINUITY REQUIREMENTS. THEY MUST BE APPROVED BY FLOODBREAK.
7. ALL CONCRETE FOUNDATION POURS AND THEIR TIE-DOWNS TO EXISTING FOUNDATIONS SHOWN IN THESE DRAWINGS ARE FOR ILLUSTRATIVE PURPOSES ONLY. DESIGN OF THE CONCRETE FOUNDATION SLABS IS BY OTHERS. DESIGN AND SUPERVISION OF INSTALLATION OF RETENTION ARMS, ANCHOR BOLTS, AND GATE ANCHORS ARE BY FLOODBREAK. ALL CONCRETE TO BE 4000 PSI MINIMUM 28 DAY STRENGTH. REINFORCED IN EACH DIRECTION WITH ASTM - A615 MIN. F_Y=60 KSI. SPECIAL ATTENTION SHALL BE PAID TO PROPER SUPPORT OF RETENTION ARM ANCHOR BOLTS INTO THE SUPPORTING CONCRETE.
8. ALL GASKET MATERIAL TO BE EDPM RUBBER.

TYPICAL VEHICULAR GATE

NOTE: LAYOUT, SIZES AND DETAIL ARE GATE-SPECIFIC. THIS VIEW SHOWN IS SECTIONED IN HALF.

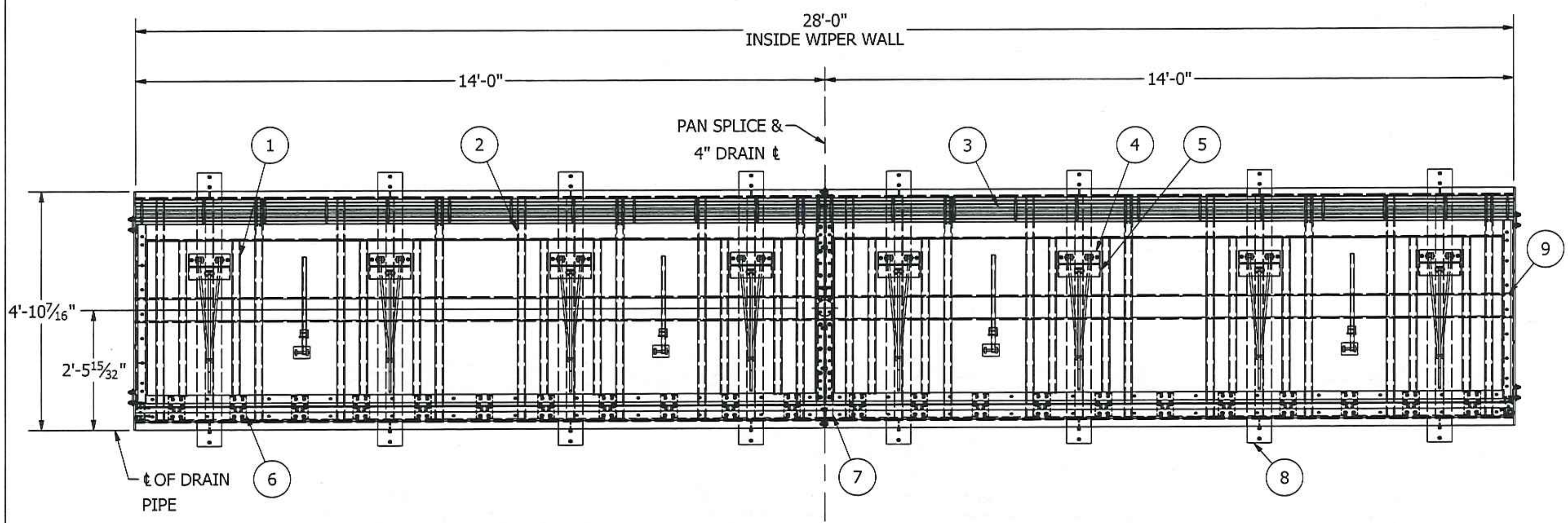
GENERAL ISOMETRIC LAYOUT

FloodBreak
REVOLUTIONARY FLOOD CONTROL

EXAMPLE VEHICULAR GATE
28'-0" x 3'-8"

VG #SAMPLE

SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 1 OF 8	



TYP LID AND PAN LAYOUT

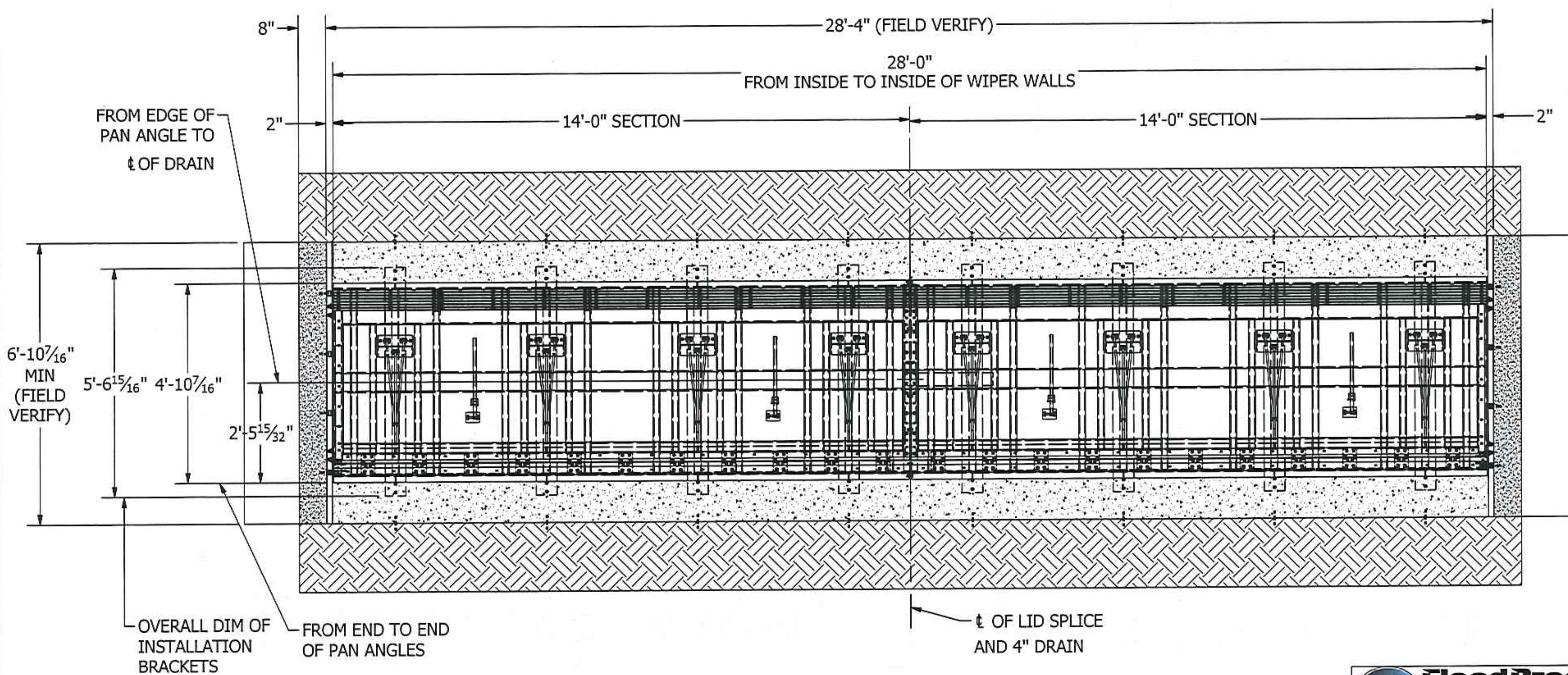
LEGEND	
ITEM	DESCRIPTION
1	TYPICAL 2"x2"x1/4 LID STIFFENER TUBE
2	TYPICAL 2"x2"x1/4 PAN STIFFENER TUBE
3	TYPICAL GRATING
4	TYPICAL PAN ANCHOR PLATE
5	TYPICAL LID ANCHOR PLATE AND RETENTION ARMS
6	TYPICAL HINGE DETAIL
7	TYPICAL PAN SPLICE JOINT DETAIL
8	TYPICAL 4" VERTICAL AND 6" HORIZONTAL INSTALL BRACKETS
9	6"x2" CONTINUOUS DRAIN TROUGH

FloodBreak
REVOLUTIONARY FLOOD CONTROL

EXAMPLE VEHICULAR GATE
28'-0" x 3'-8"

VG #SAMPLE

SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 2 OF 8	



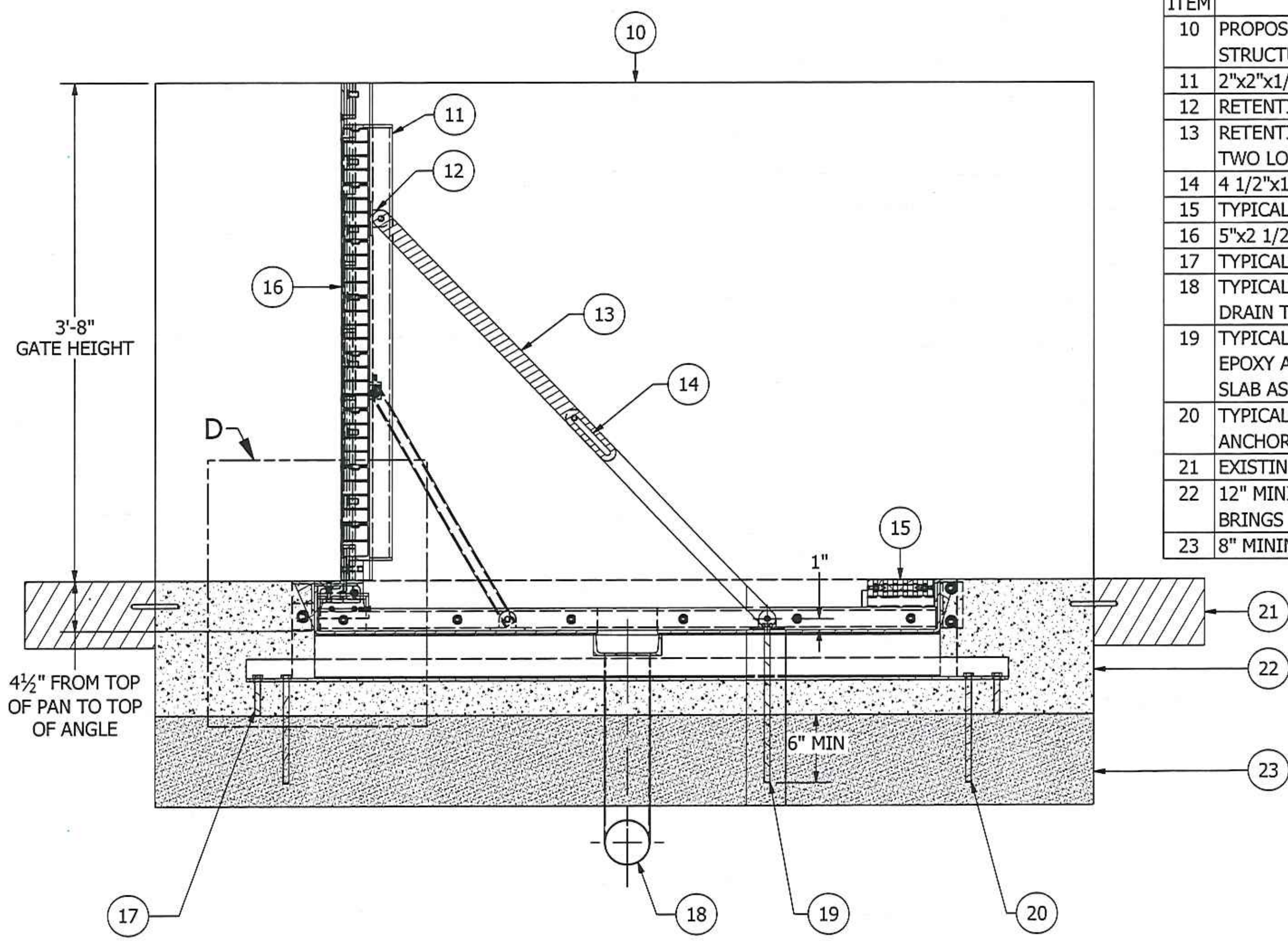
LID AND PAN INSTALLATION VIEW
(WITH CONCRETE SET-DOWN)

GENERAL FORMULA FOR SET-DOWN:
GATE HEIGHT + 38.5"± 1" = WIDTH
GATE LENGTH + 4" = LENGTH
12" TOPPING SLAB = DEPTH

FloodBreak
REVOLUTIONARY FLOOD CONTROL

EXAMPLE VEHICULAR GATE 28'-0" x 3'-8"	
VG #SAMPLE	
SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 3 OF 8	

TYPICAL CROSS-SECTION OF GATE ASSEMBLY



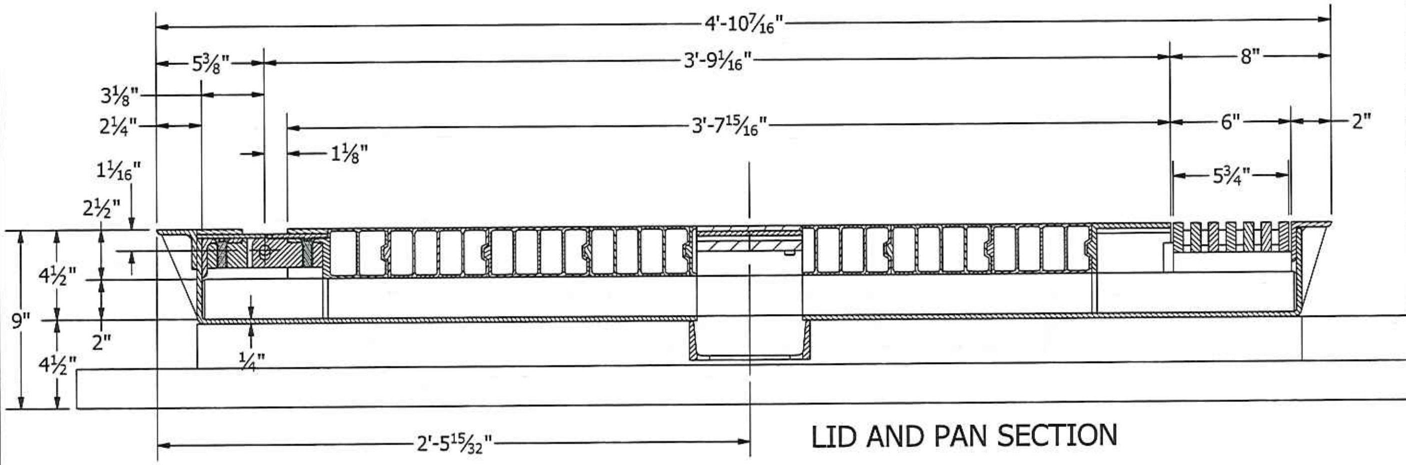
LEGEND	
ITEM	DESCRIPTION
10	PROPOSED OR EXISTING WIPER WALL SUPPORT STRUCTURE (BY OTHERS)
11	2"x2"x1/4" LID STIFFENER TUBE
12	RETENTION ARM ANCHOR PLATE
13	RETENTION ARMS: 1 1/2"x1/2" FLAT STOCK, ONE UPPER, TWO LOWER.
14	4 1/2"x1/2" SLOT W/ 1/2" STAINLESS STEEL PIN
15	TYPICAL GRATING
16	5"x2 1/2"x1/8" EXTRUDED RIBBED PANELS
17	TYPICAL 1/2" LEVELING BOLTS
18	TYPICAL 4" DRAIN INSTALLED IN 6"x2" CONTINUOUS DRAIN TROUGH
19	TYPICAL 1/2" STAINLESS STEEL ANCHOR BOLTS SET IN EPOXY AND ANCHORED INTO CONCRETE STRUCTURAL SLAB AS SHOWN
20	TYPICAL 1/2" STAINLESS STEEL INSTALLATION BOLTS ANCHORED INTO CONCRETE STRUCTURAL SLAB
21	EXISTING SURFACE
22	12" MINIMUM TOPPING SLAB ENVELOPES PAN AND BRINGS IT UP TO GRADE (FIELD VERIFY SET-DOWN)
23	8" MINIMUM CONCRETE STRUCTURAL SLAB (BY OTHERS)

FloodBreak
REVOLUTIONARY FLOOD CONTROL

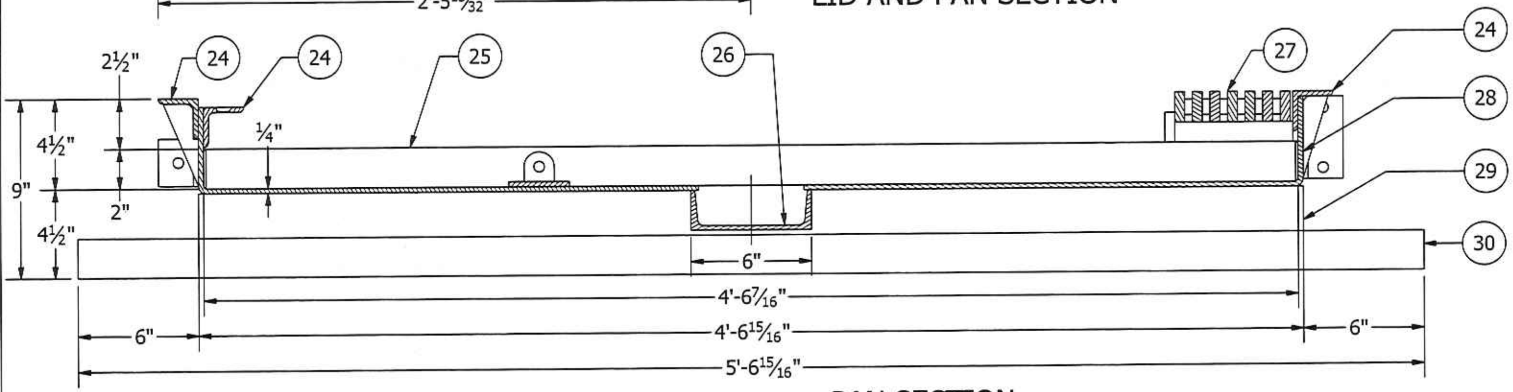
EXAMPLE VEHICULAR GATE
28'-0" x 3'-8"

VG #SAMPLE

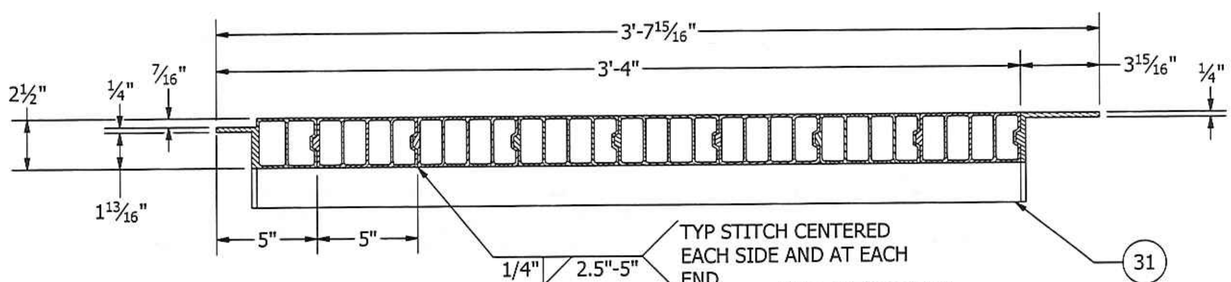
SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 4 OF 8	



LID AND PAN SECTION



PAN SECTION



LID SECTION

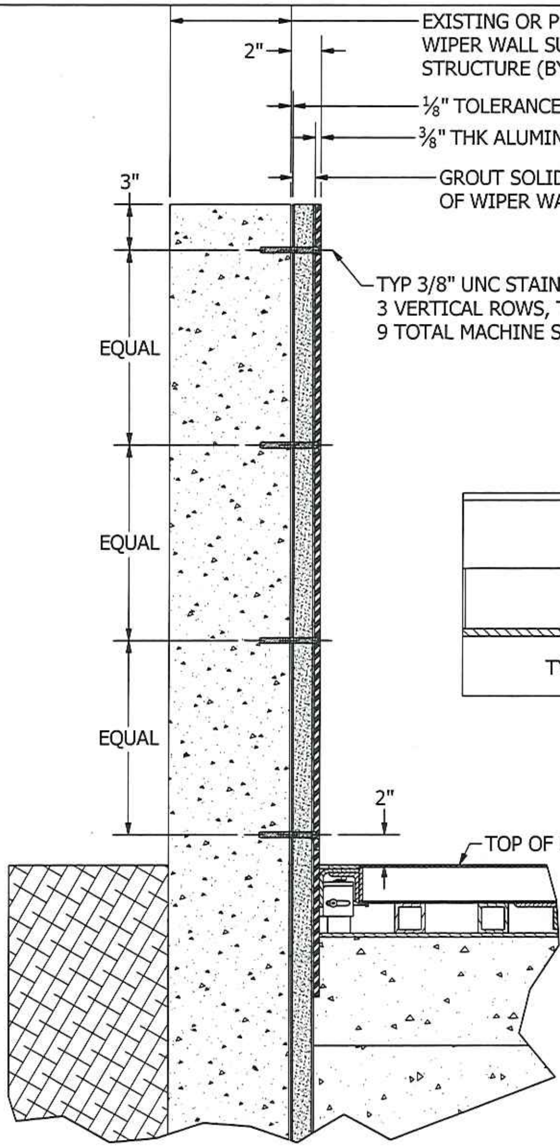
LEGEND	
ITEM	DESCRIPTION
24	TYPICAL 2"x2"x1/4" ANGLE
25	TYPICAL 2"x2"x1/4 PAN STIFFENER TUBE
26	6"x2" CONTINUOUS DRAIN TROUGH
27	TYPICAL GRATING
28	PAN
29	4"x2"x1/4" VERTICAL CHANNEL INSTALL BRACKET
30	6"x2"x1/4" HORIZONTAL CHANNEL INSTALL BRACKET
31	TYPICAL 2"x2"x1/4 LID STIFFENER TUBE

FloodBreak
REVOLUTIONARY FLOOD CONTROL

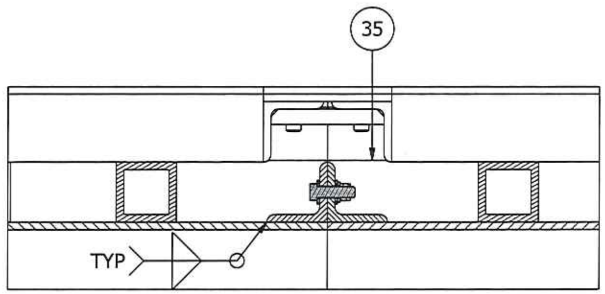
EXAMPLE VEHICULAR GATE
28'-0" x 3'-8"

VG #SAMPLE

SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 5 OF 8	

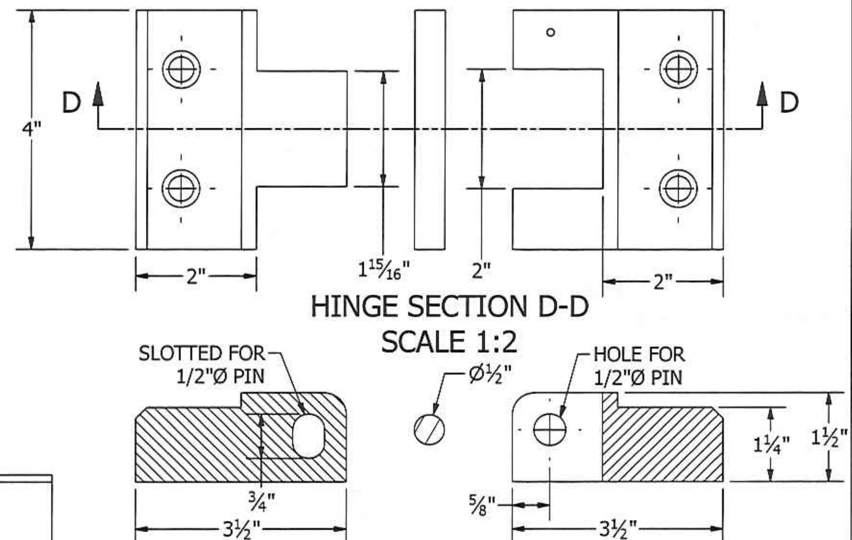


TYP WIPER WALL SECTION FRONT ELEVATION



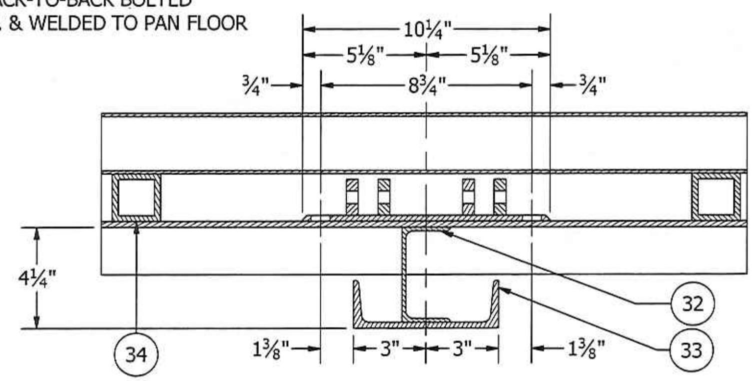
TYP PAN SPLICE JOINT DETAIL

NOTE: 2"x2"x1/4" ALUMINUM ANGLE BACK-TO-BACK BOLTED AT 6" O.C. & WELDED TO PAN FLOOR



HINGE SECTION D-D
SCALE 1:2

LEGEND	
ITEM	DESCRIPTION
32	4"x2"x1/4" VERTICAL INSTALL CHANNEL
33	6"x2"x1/4" HORIZONTAL INSTALL CHANNEL
34	TYPICAL 2"x2"x1/4" PAN STIFFENER TUBE
35	TYPICAL 2"x2"x1/4" ANGLE



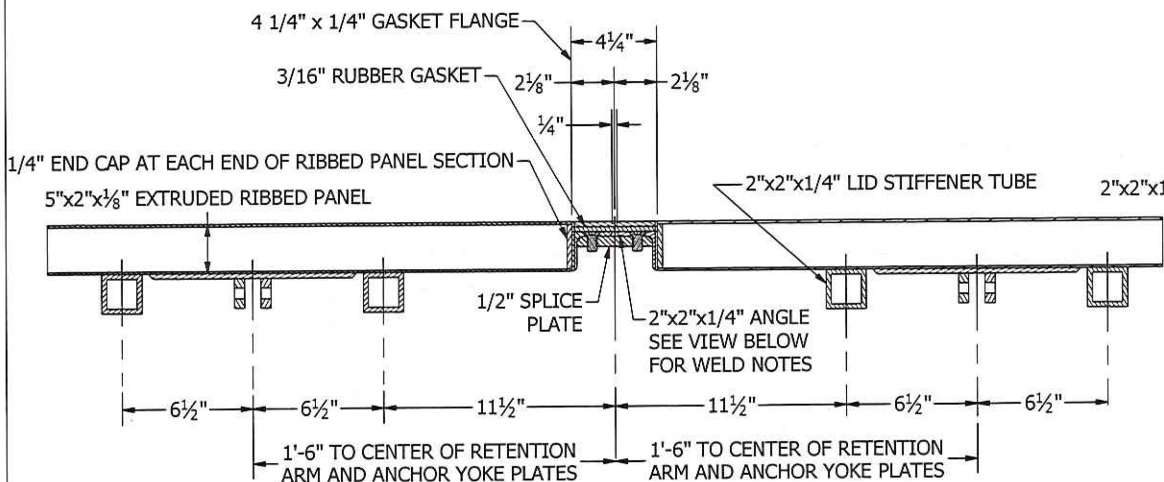
TYP PAN COMPONENT ASSY

FloodBreak
REVOLUTIONARY FLOOD CONTROL

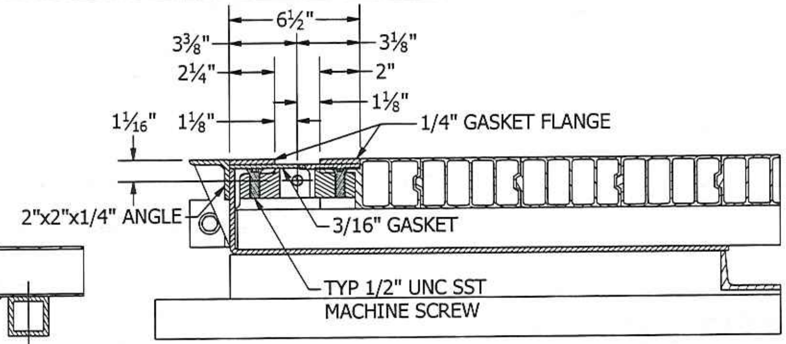
EXAMPLE VEHICULAR GATE
28'-0" x 3'-8"

VG #SAMPLE

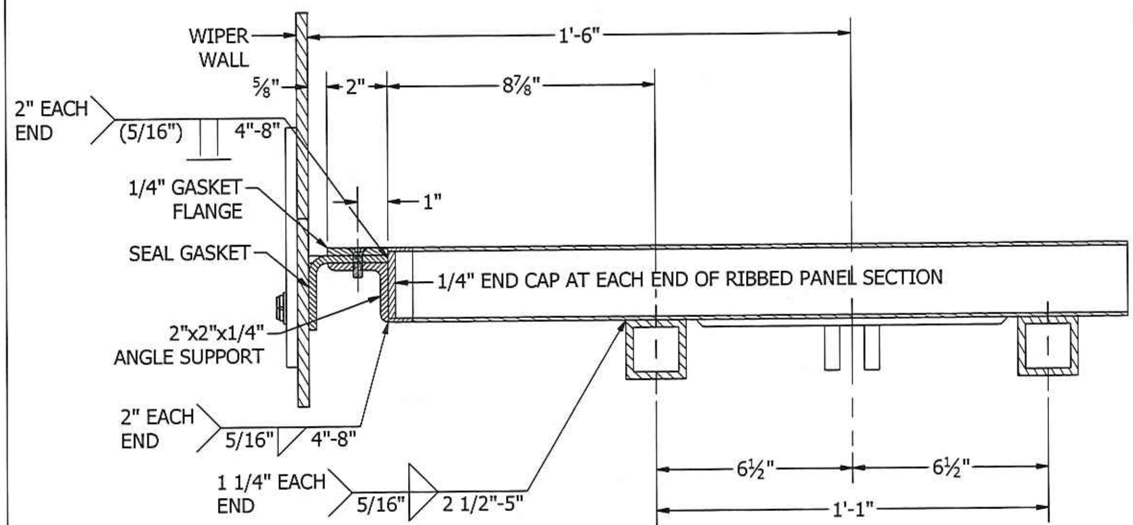
SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 6 OF 8	



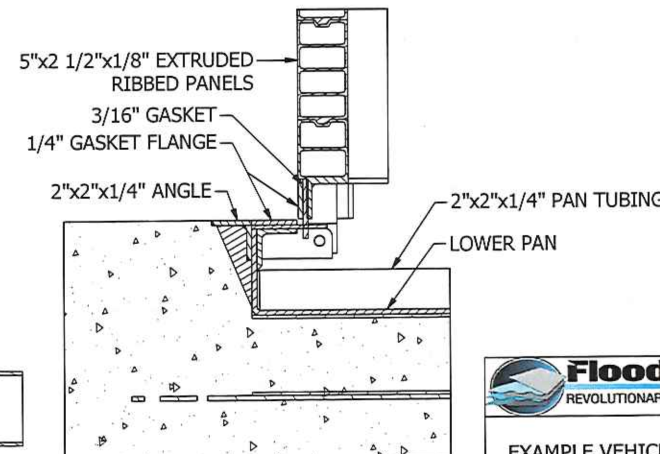
TYPICAL LID AT SPLICE



HINGE DETAIL CLOSED



TYPICAL LID AT WIPER WALL



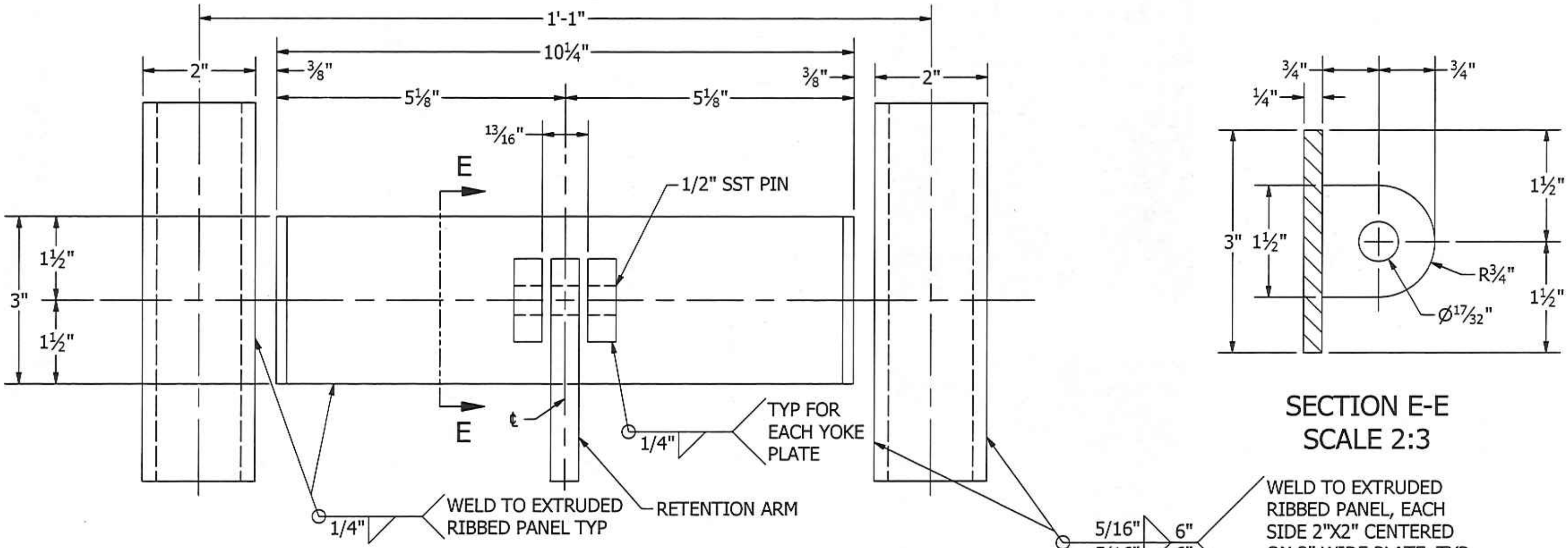
DETAIL D
HINGE DETAIL OPEN

FloodBreak
REVOLUTIONARY FLOOD CONTROL

EXAMPLE VEHICULAR GATE
28'-0" x 3'-8"

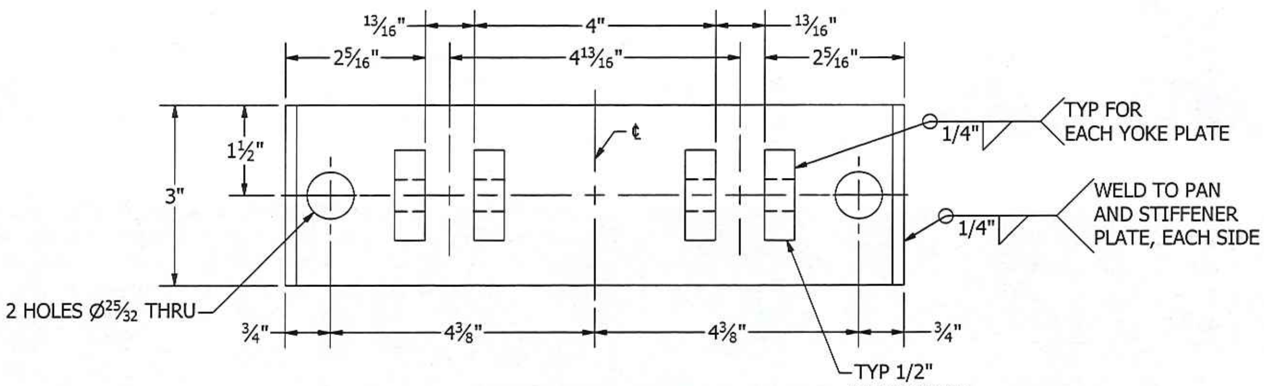
VG #SAMPLE

SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 7 OF 8	



TYPICAL LID ANCHOR YOKE PLATE DETAIL
SCALE 1:2

SECTION E-E
SCALE 2:3



TYPICAL PAN ANCHOR PLATE DETAIL
SCALE 1:2

FloodBreak
REVOLUTIONARY FLOOD CONTROL

EXAMPLE VEHICULAR GATE
28'-0" x 3'-8"

VG #SAMPLE

SCALE	VARIES
DRAWN	A. JOLLY
CHECKED	
DATE	11/5/2013
SHEET 8 OF 8	

PRODUCT DATASHEET

Pedestrian Gate

The Floodbreak Pedestrian Gate is a fully-engineered system that will automatically protect pedestrian entrances from street-level flooding. When not deployed, the gate remains hidden underground and with zero impact on pedestrian traffic. The system can be covered with a variety of materials, including carpet, pavers, or tile.

Doorways * Entrance Foyers * Stairwells * Elevator Lobbies

Revolutionary Flood Control

Like the rest of FloodBreak's passive flood mitigation product line, the Pedestrian Gate is fully automatic and does not depend on people or power to deploy. It is the only practical, truly passive flood control solution - the preferred method according to FEMA.

The Smart Choice

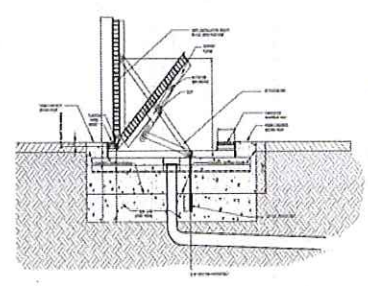
FloodBreak has been protecting customers since 2002. Major hospitals, governments, and commercial facilities all rely on FloodBreak's 24/7 flood protection. In the past two years alone, there have been 12 identified flood saving deployments.

How It Works

The FloodBreak system uses hydrostatic pressure created by the rising flood waters to automatically activate the gate. When the flood recedes, the gate automatically returns to its hidden position beneath the ground allowing pedestrian traffic to resume.

Features:

- **Passive flood mitigation preferred by FEMA**
- **Manufactured to exact size requirements**
- **Weather resistant materials & durable rubber gaskets**
- **Minimal maintenance**
- **No training required**
- **Easy to install**



100% PASSIVE

FULLY AUTOMATIC

24/7 FLOOD PROTECTION

PREVENTION THROUGH INNOVATION

1.713.980.6610

floodbreak.com

NO PEOPLE, NO POWER

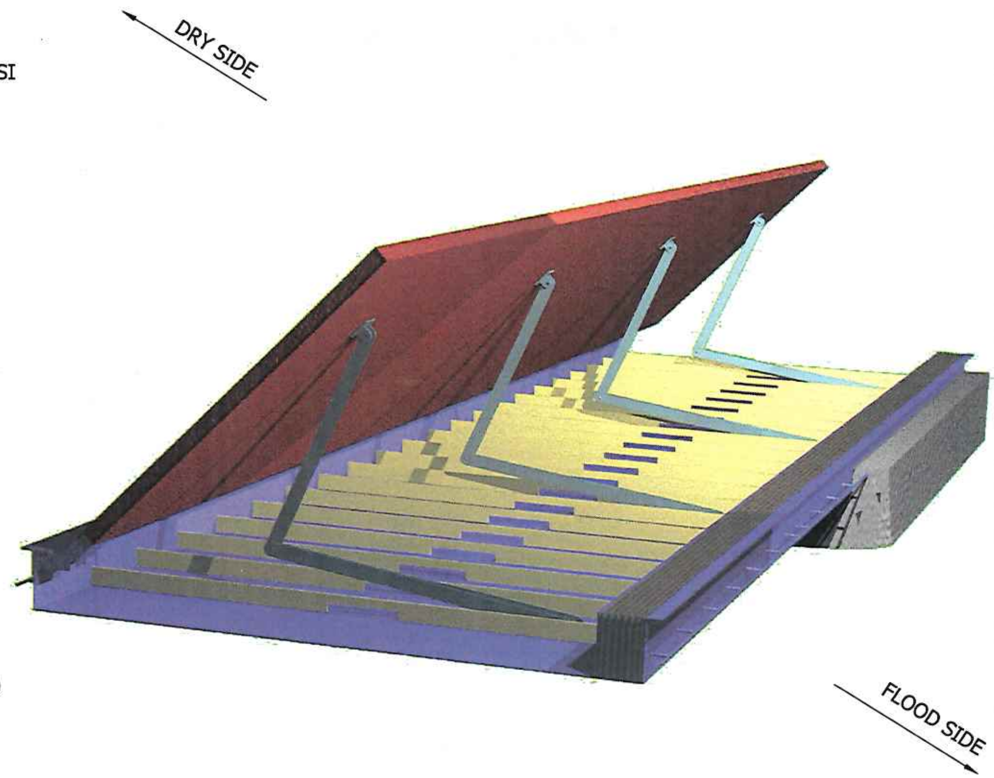
24/7 CONFIDENCE

FloodBreak
REVOLUTIONARY FLOOD CONTROL

NOTES:

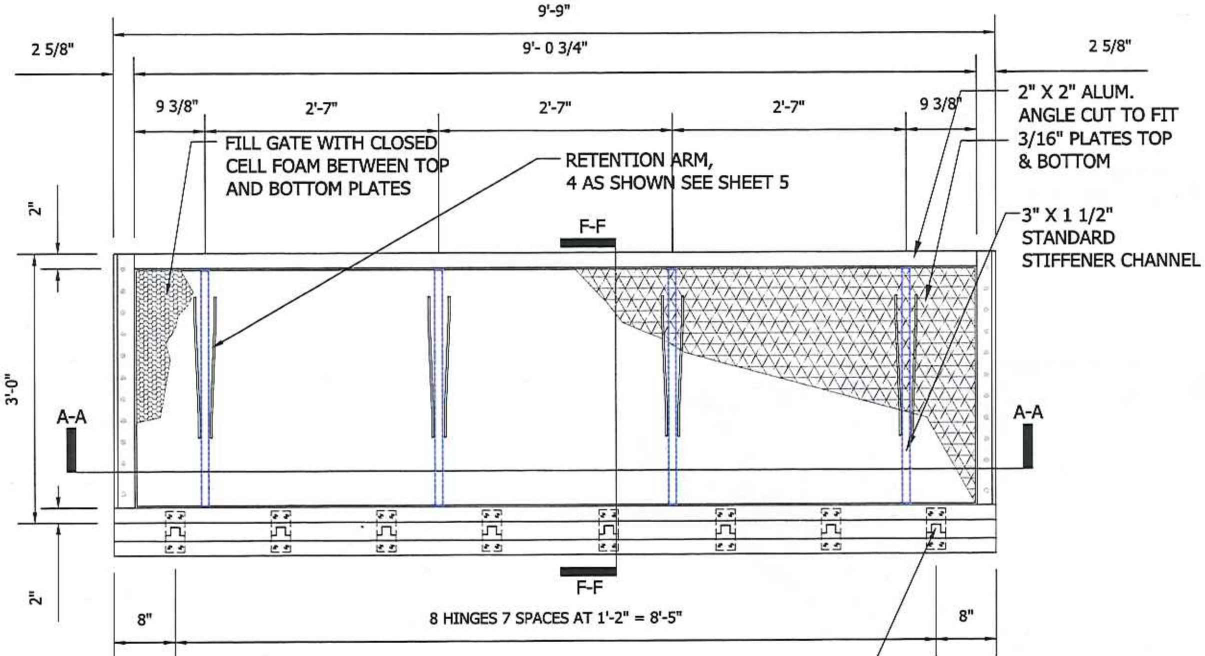
1. FLOOD GATE MATERIAL TO BE ALUMINUM AS FOLLOWS
   - -GATE TOP AND BOTTOM 3/16" SMOOTH PLATE - GRADE 5052 MIN F_Y = 40 KSI
   - -GATE CHANNELS ARE 3" X 1 1/2" STANDARD CHANNEL, 1.135 LBS/FT. - GRADE 6061 - T6, MIN F_Y = 40 KSI
   - -PAN 1/4" SMOOTH PLATE - GRADE 5052 MIN F_Y = 40 KSI
   - -FLAT BARS, ROUND BARS, STRUCTURAL ANGLES, HINGES - GRADE 6061-T6 MIN F_Y = 40 KSI
2. HINGE BOLTS TO BE STAINLESS STEEL - GRADE 304, MIN F_Y = 90 KSI
3. ANCHOR BOLTS SHALL BE STAINLESS STEEL STANDARD THREAD BOLTS SET IN VINYLESTER BASED ADHESIVE CONTAINED IN A GLASS CAPSULE, INSTALLED PER SIMPSON STRONG TIE SPECIFICATIONS.
4. ALUMINUM TO BE WELDED WITH ALUMINUM WIRE - ER 4043 AWS A5.10 3/32
5. GROUT AT WIPER WALLS TO BE NON-SHRINK GROUT.
6. GATE IS FILLED WITH FROTH PAK 180 POLYURETHANE HIGH DENSITY CLOSED CELL FOAM AT 1.75 PCF.
7. ALL WELDS REQUIRED FOR STRUCTURAL STRENGTH OF THE GATE OR PAN ARE CALLED OUT ON THESE DRAWINGS. ALL OTHER WELDING, NOT SHOWN OR CALLED OUT ON THESE DRAWINGS, ARE ESSENTIALLY NON-STRUCTURAL WELDS OR WELDS WITH NEGLIGIBLE LOADS AND RESULTING STRESSES. EXAMPLES OF SUCH WELDS ARE AT SEAMS, SIDES, STIFFENERS AND TROUGH OF THE PAN, AND TRIM ANGLES OF THE GATE. THESE WELDS ARE TO BE SELECTED BY THE FABRICATOR, TAKING INTO CONSIDERATION ASSEMBLY, TRANSPORT AND CONTINUITY REQUIREMENTS, AND MUST BE APPROVED BY FLOODBREAK.
8. ALL CONCRETE FOUNDATION POURS AND THEIR TIE DOWNS TO EXISTING FOUNDATIONS SHOWN IN THESE DRAWINGS ARE FOR ILLUSTRATIVE PURPOSES ONLY. DESIGN OF CONCRETE FOUNDATION SLABS IS BY OTHERS. DESIGN AND SUPERVISION OF INSTALLATION OF RETENTION ARMS TIE DOWN BOLTS AND GATE ANCHORS ARE BY FLOODBREAK. CONCRETE TO BE 4000 PSI MINIMUM 28 DAY STRENGTH. REINFORCED IN EACH DIRECTION WITH ASTM - A615, MIN F_Y = 60 KSI. SPECIAL ATTENTION SHALL BE PAID TO PROPER ANCHORING OF RETENTION ARMS INTO THE SUPPORTING CONCRETE.

NOTE:
WIPER WALL EACH END NOT SHOWN FOR CLARITY PURPOSES



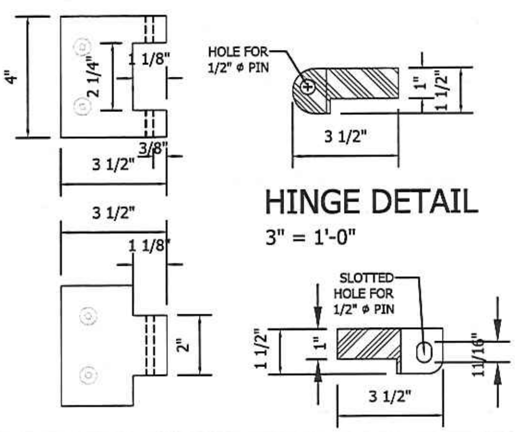
NOTE: ISOMETRIC VIEW MAY NOT BE TYPICAL OF ACTUAL GATE

FloodBreak — Revolutionary Flood Control — Automatic Floodgates - no people, no power!		SAMPLE 9'-9" X 3'-0" PEDESTRIAN GATE	
		FLOODBREAK SERIAL #	
SCALE:	AS NOTED	GENERAL DETAILS	
DRAWN BY:	CWJJ	DATE:	06-10-10
CHECKED BY:	JK-OSI	SHEET:	1 0F 7



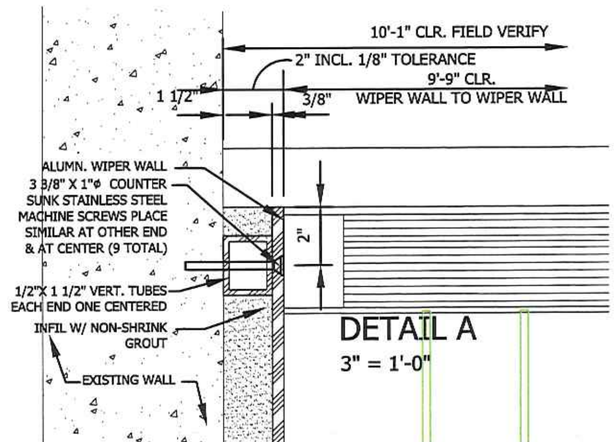
PLAN VIEW AT GATE

SCALE 3/4" = 1'-0"



HINGE DETAIL

3" = 1'-0"



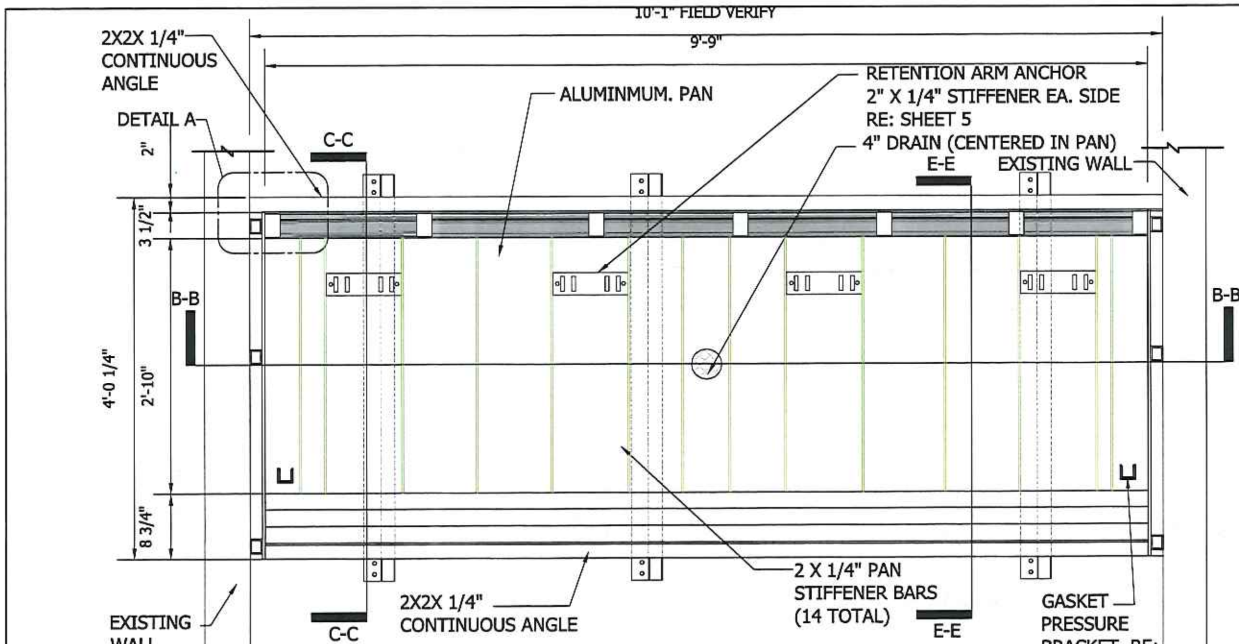
DETAIL A

3" = 1'-0"

NOTES: GATE

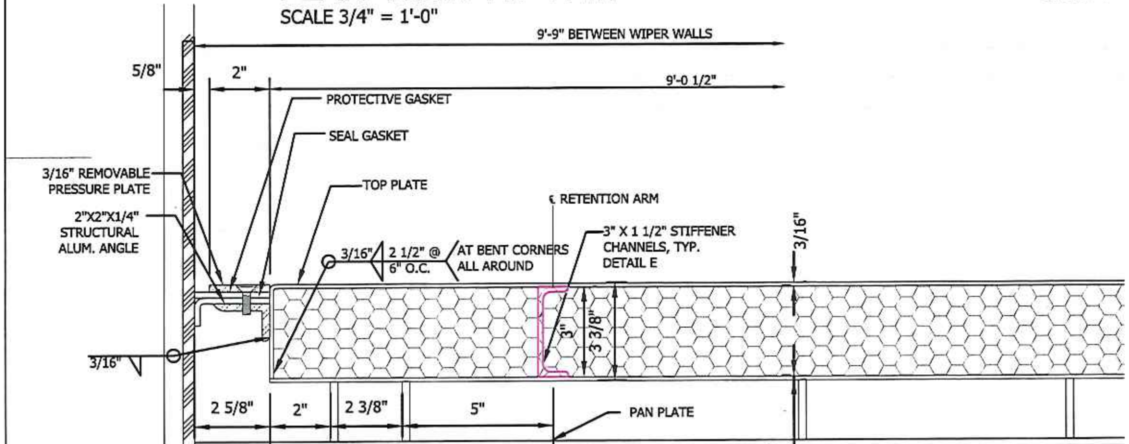
1. TOP PLATE BENT DOWN ON SHORT SIDES, BOTTOM PLATE BENT UP ON LONG SIDES.

FloodBreak Revolutionary Flood Control *Automatic Floodgates - no people, no power!*		SAMPLE 9'-9" X 3'-0" PEDESTRIAN GATE
		FLOODBREAK SERIAL #
SCALE:	AS NOTED	GENERAL DETAILS
DRAWN BY:	CWJJ	DATE: 06-10-10
CHECKED BY:	JK-OSI	SHEET: 2 0F 7



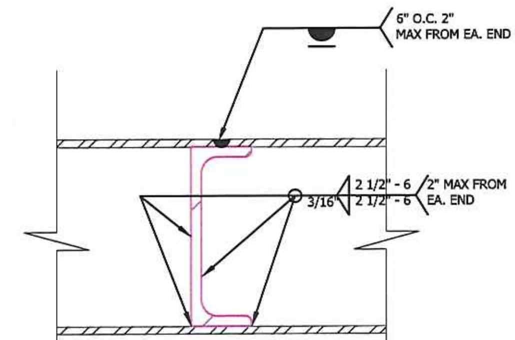
PLAN VIEW AT PAN

SCALE 3/4" = 1'-0"



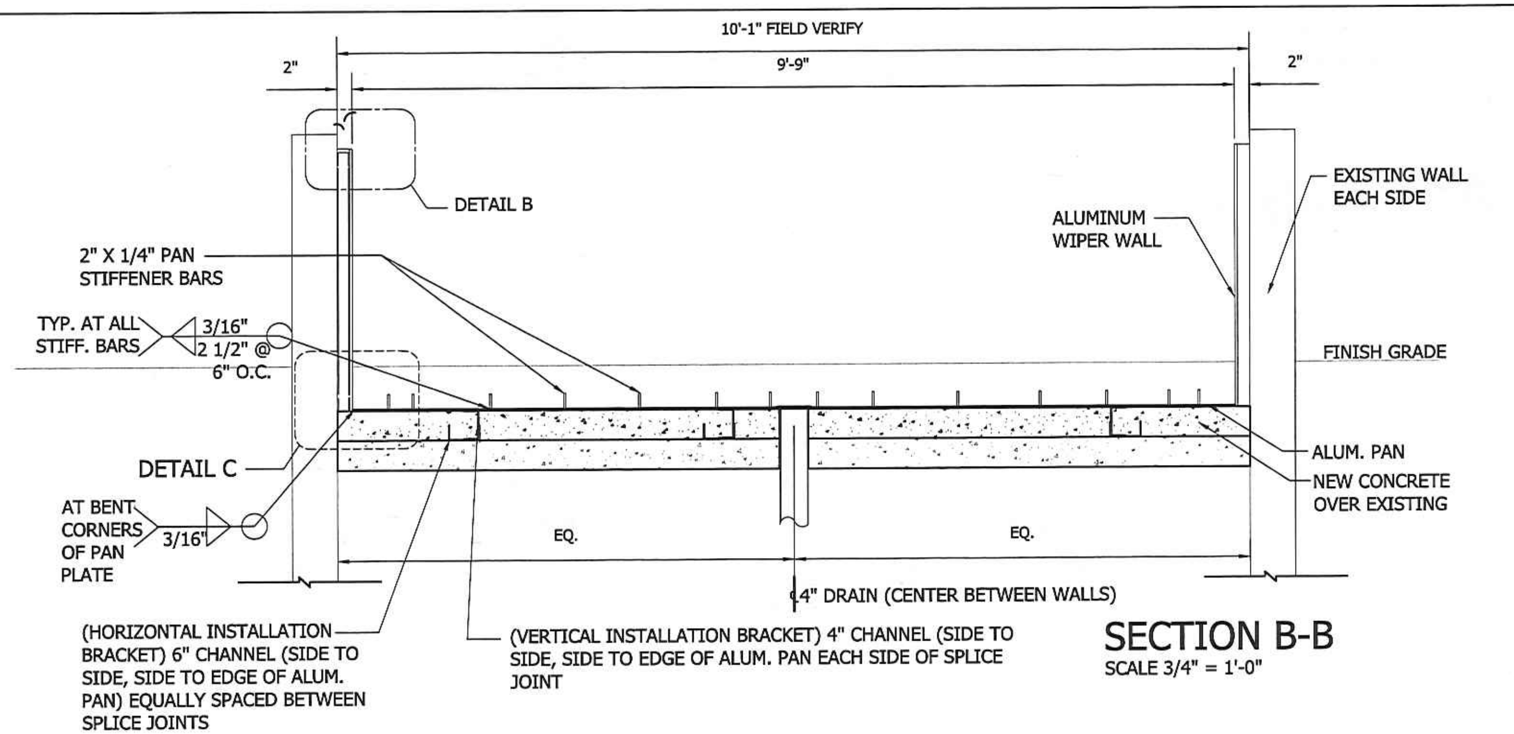
SECTION F-F

SCALE 3" = 1'

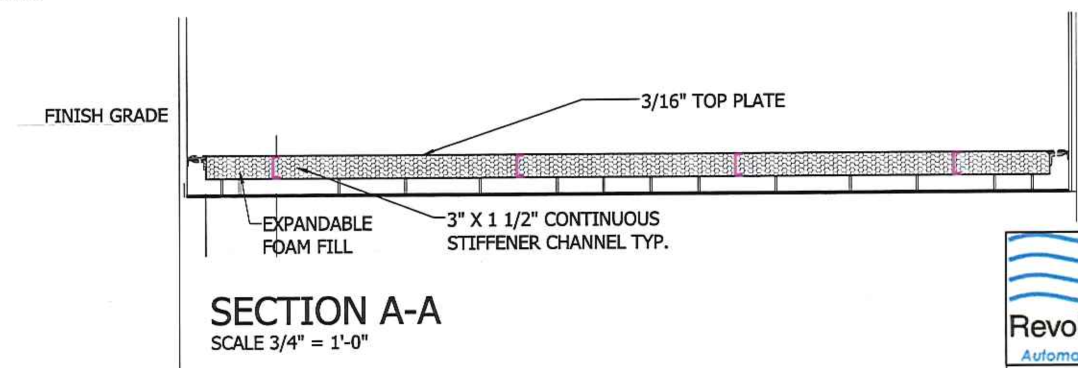


DETAIL E

FloodBreak Revolutionary Flood Control *Automatic Floodgates - no people, no power!*		SAMPLE 9'-9" X 3'-0" PEDESTRIAN GATE	
		FLOODBREAK SERIAL #	
SCALE:	AS NOTED	GENERAL DETAILS	
DRAWN BY:	CWJJ	DATE:	06-10-10
CHECKED BY:	JK-OSI	SHEET:	3 OF 7

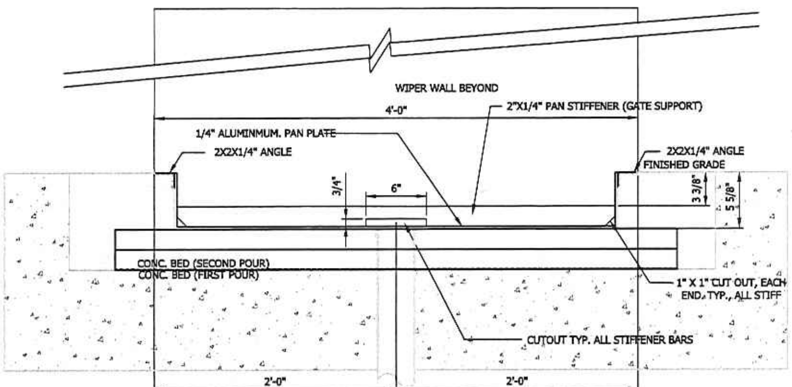


SECTION B-B
SCALE 3/4" = 1'-0"

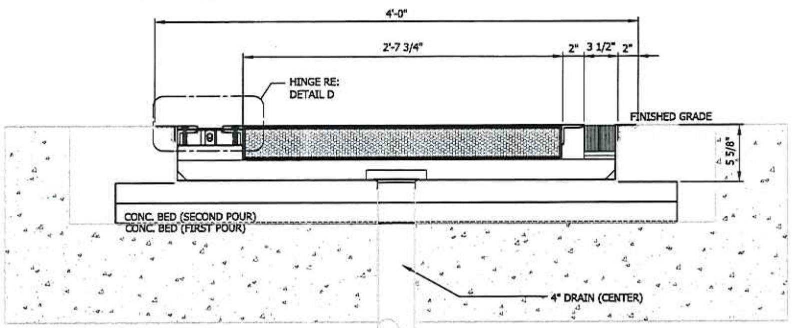


SECTION A-A
SCALE 3/4" = 1'-0"

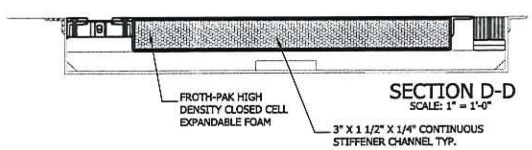
FloodBreak Revolutionary Flood Control *Automatic Floodgates - no people, no power!*		SAMPLE 9'-9" X 3'-0" PEDESTRIAN GATE	
		FLOODBREAK SERIAL #	
SCALE:	AS NOTED	GENERAL DETAILS	
DRAWN BY:	CWJJ	DATE:	06-10-10
CHECKED BY:	JK-OSI	SHEET:	4 0F 7



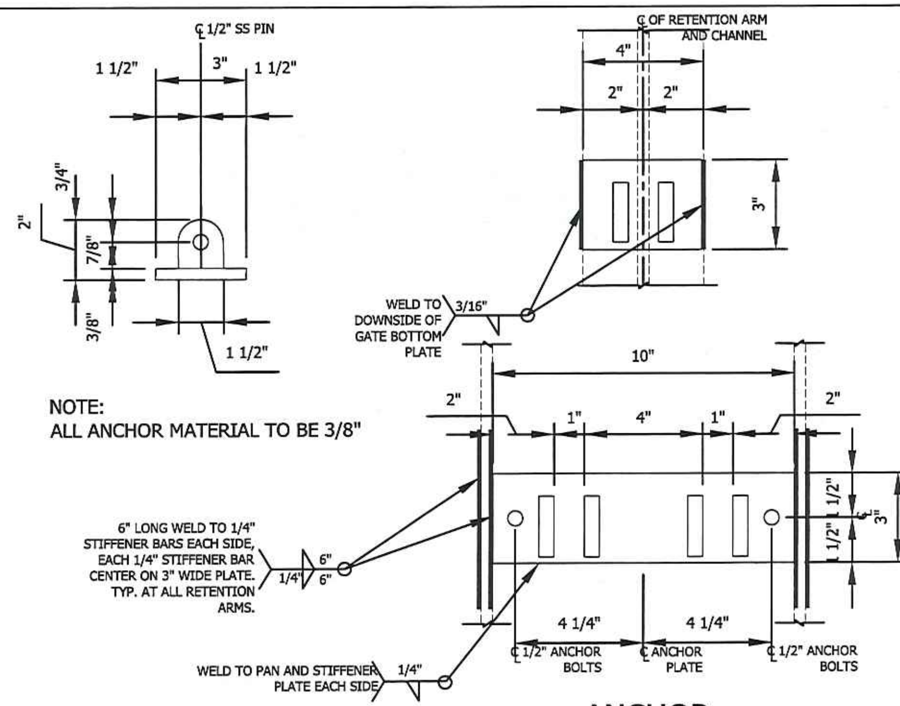
SECTION C-C (PAN ONLY)
SCALE: 1" = 1'-0"



SECTION E-E (WITH GATE IN CLOSED POSITION)
SCALE: 1" = 1'-0"



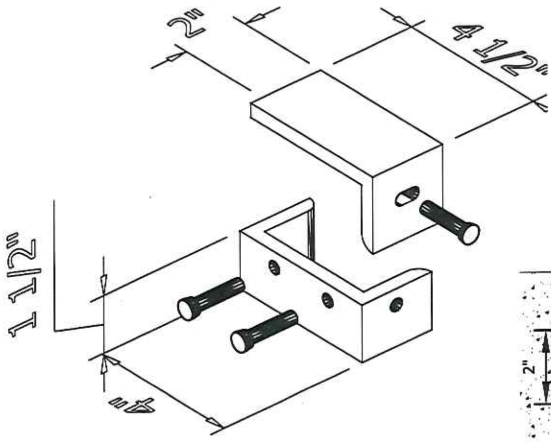
SECTION D-D
SCALE: 1" = 1'-0"

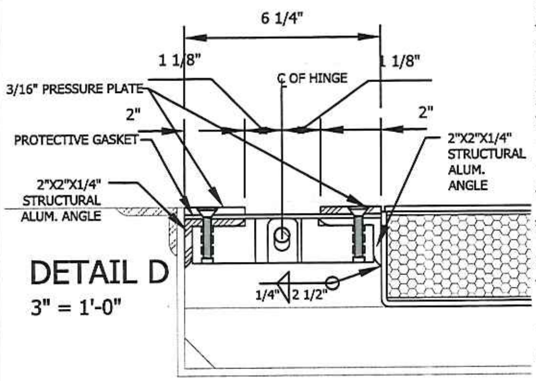


NOTE:
ALL ANCHOR MATERIAL TO BE 3/8"

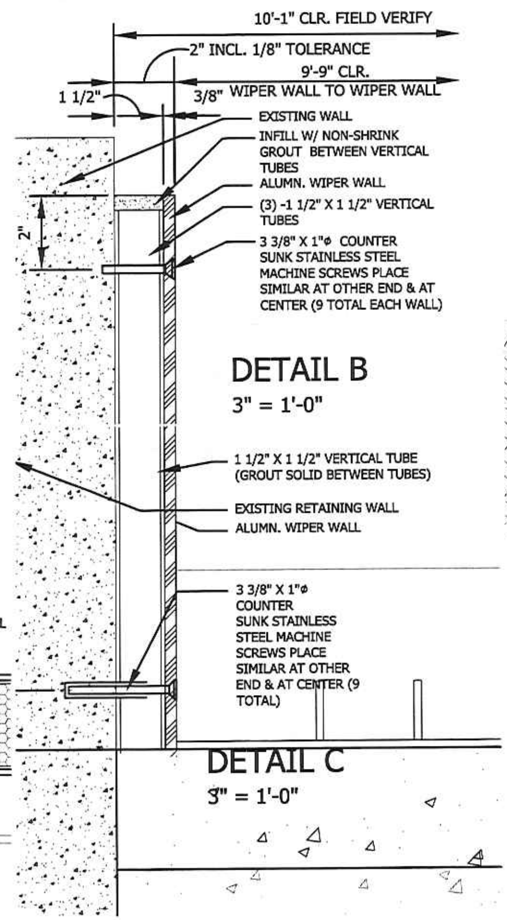
ANCHOR
3" = 1'-0"

FloodBreak Revolutionary Flood Control — Automatic Floodgates - no people, no power!		SAMPLE 9'-9" X 3'-0" PEDESTRIAN GATE	
		FLOODBREAK SERIAL #	
SCALE:	AS NOTED	GENERAL DETAILS	
DRAWN BY:	CWJJ	DATE:	06-10-10
CHECKED BY:	JK-OSI	SHEET:	5 0F 7



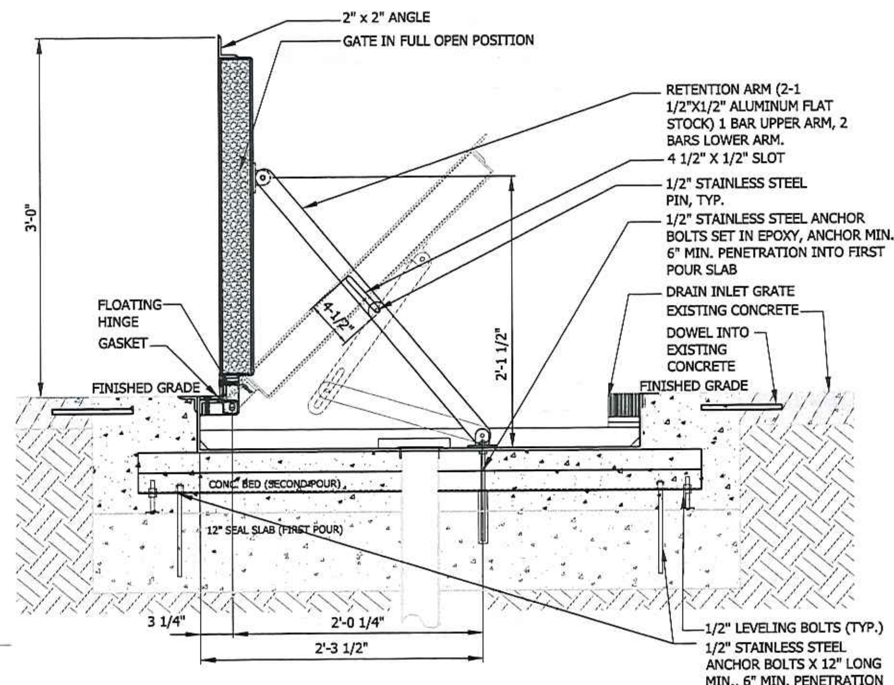


DETAIL D
3" = 1'-0"



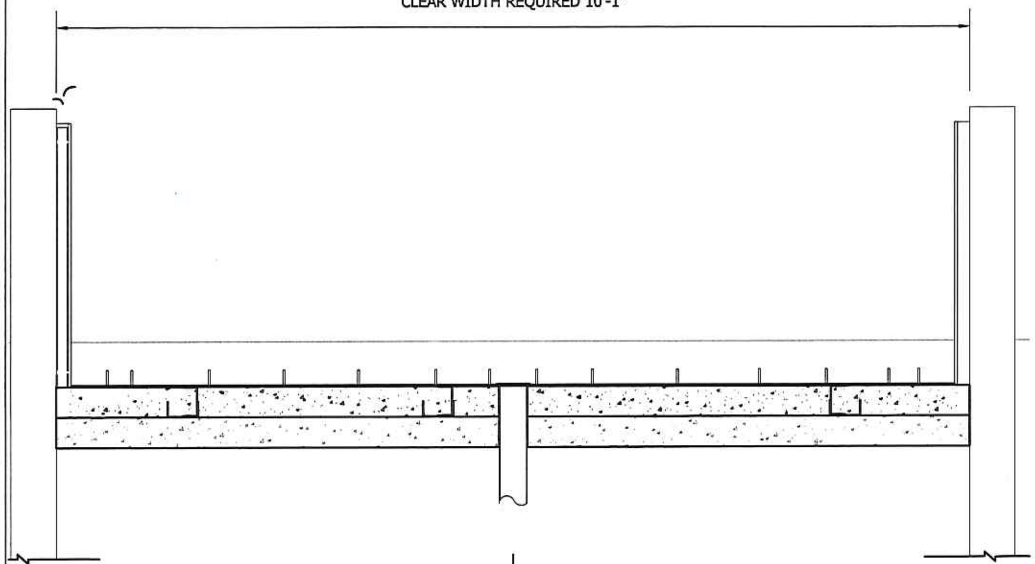
DETAIL B
3" = 1'-0"

DETAIL C
3" = 1'-0"

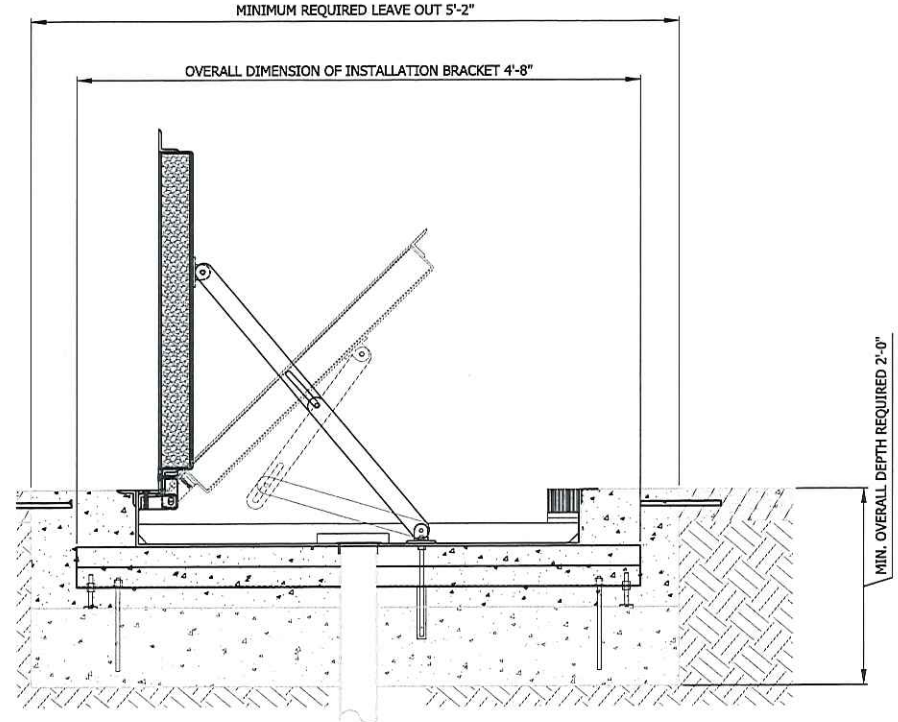


SECTION F-F (OPEN POSITION)
SCALE: 1" = 1'-0"

FloodBreak Revolutionary Flood Control *Automatic Floodgates - no people, no power!*	SAMPLE 9'-9" X 3'-0" PEDESTRIAN GATE
	FLOODBREAK SERIAL #
SCALE: AS NOTED	GENERAL DETAILS
DRAWN BY: CWJJ	DATE: 06-10-10
CHECKED BY: JK-OSI	SHEET: 6 0F 7



INSTALLATION DIMENSIONS
SCALE 3/4" = 1'-0"



INSTALLATION DIMENSIONS
SCALE: 1" = 1'-0"

FloodBreak Revolutionary Flood Control *Automatic Floodgates - no people, no power!*		SAMPLE 9'-9" X 3'-0" PEDESTRIAN GATE	
		FLOODBREAK SERIAL #	
SCALE:	AS NOTED	INSTALLATION DIMENSIONS	
DRAWN BY:	CWJJ	DATE:	06-10-10
CHECKED BY:	JK-OSI	SHEET:	7 0F 7

FLOODBREAK PHOTOS

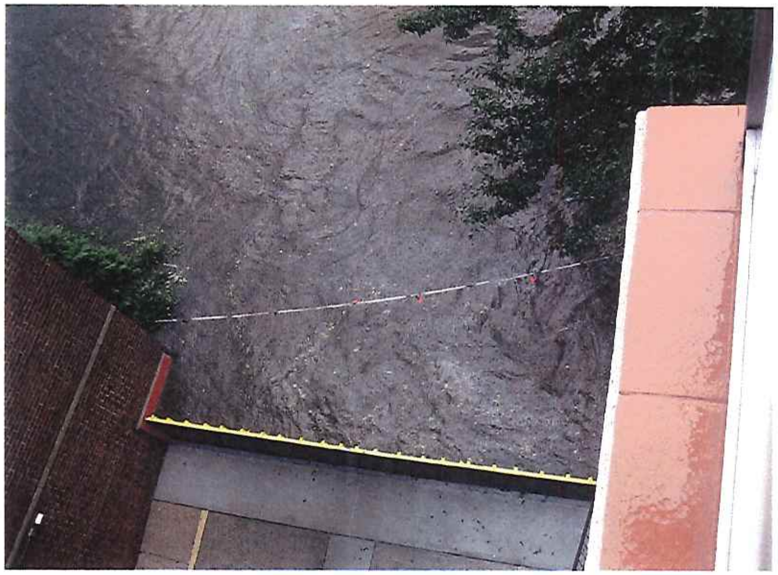
FloodBreak Vehicular Flood Gate

Floodway Bridge Crossing, US 83 East Levee

Heavy Highway Traffic on FloodBreak Roadway Gate

Heavy traffic on FloodBreak Roadway Gate, Hidalgo County

Jackson Road , Hidalgo County

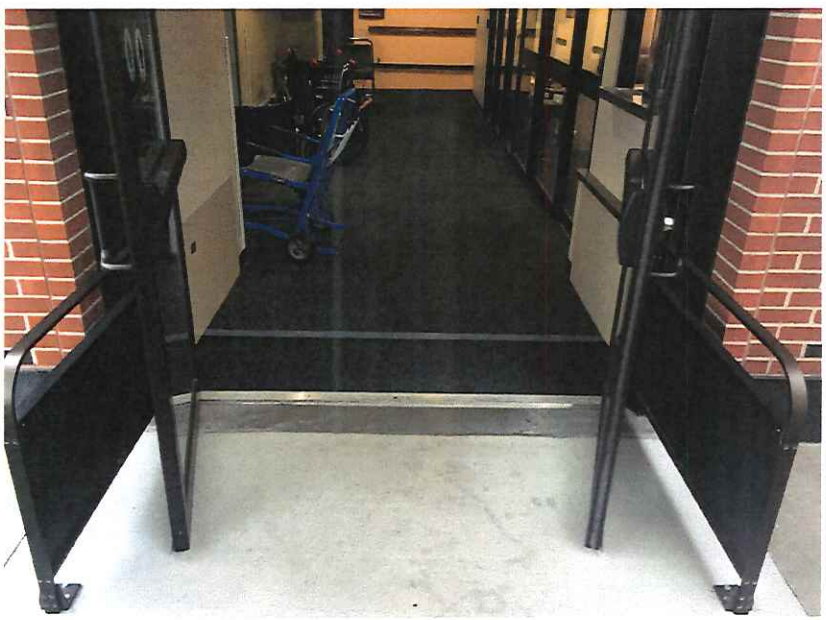


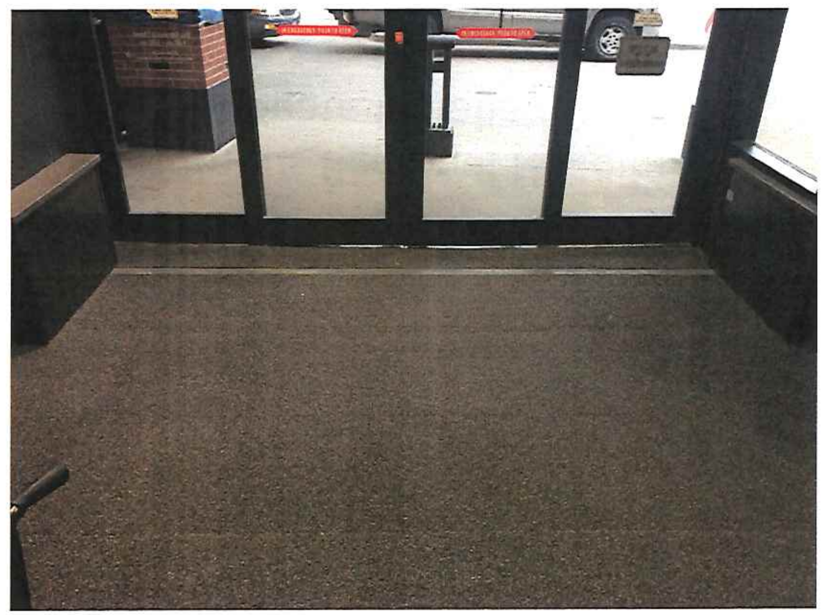
Great neck FloodBreak Vehicle Gate (Birdseye View)

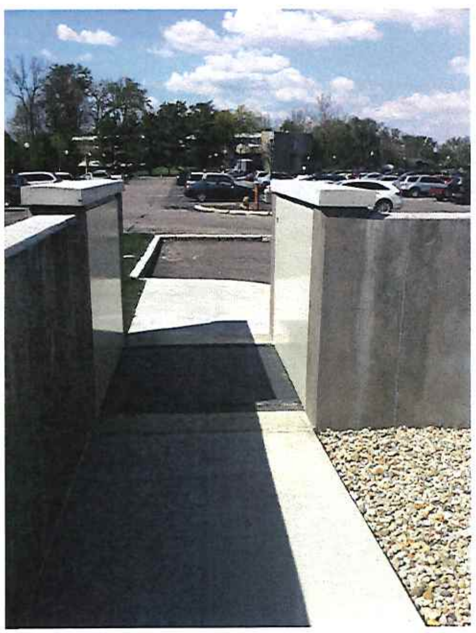
Lourdes Aerial

FloodBreak Pedestrian Flood Gate

Floodgate installation in Lobby



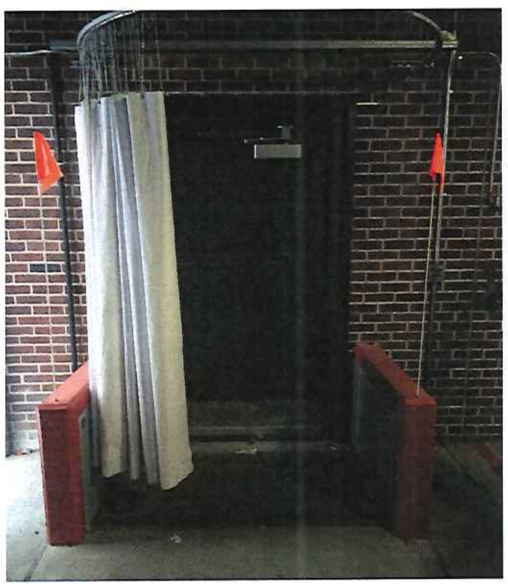




Columbus Regional Hospital
Floodgate Installations

Entrance Door Floodgate Installations

Entrance Door Floodgate
Installations



SPECIFICATIONS
SECTION ________

FLOOD CONTROL GATES

PART 1 GENERAL

1.1 SECTION INCLUDES

A. Flood Gates.

1.2 RELATED SECTIONS

A. N/A.

1.3 REFERENCES

A. ASTM C 39 - Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.

B. ASTM A 240 / 240M - Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications

C. AWS - American Welding Society.

D. FEMA - Federal Emergency Management Agency.

1.4 DEFINITIONS

A. Mitigation Height: The height of flood waters based on the local FEMA five-hundred (500) year flood plain plus one (1) inch.

1.5 SUBMITTALS

A. Submit under provisions of Section _________.

B. Product Data: Manufacturer's data sheets on each product to be used, including:
1. Preparation instructions and recommendations.
2. Storage and handling requirements and recommendations.
3. Installation methods.

C. Shop Drawings: Submit plan, section, elevation and perspective drawings as necessary to depict proper placement, installation and operation methods for each gate to be installed.

1.6 QUALITY ASSURANCE

A. Manufacturer Qualifications: All primary products specified in this section will be supplied by a single manufacturer with a minimum of 5 years experience in design and manufacturer of passive flood barrier systems and evidence of a minimum of 25 projects.

B. Installer Qualifications: All Work listed in this section is to be installed by a contractor approved by FloodBreak.. Floodbreak representative must be on-site during gate installation to provide advisory services.

C. Mock-Up: Provide a mock-up for evaluation of surface preparation techniques and application workmanship.

1. Finish areas designated by Architect.
2. Do not proceed with remaining work until workmanship, color, and sheen are approved by Architect.
3. Refinish mock-up area as required to produce acceptable work.

1.7 DELIVERY, STORAGE, AND HANDLING

A. Store products in manufacturer's unopened packaging until ready for installation.

B. Store and dispose of hazardous materials, and materials contaminated by hazardous materials, in accordance with requirements of local authorities having jurisdiction.

1.8 PROJECT CONDITIONS

A. Maintain environmental conditions (temperature, humidity, and ventilation) within limits recommended by manufacturer for optimum results. Do not install products under environmental conditions outside manufacturer's absolute limits.

1.9 WARRANTY

A. At project closeout, provide to Owner or Owners Representative an executed copy of the manufacturer's standard limited warranty against manufacturing defect, outlining its terms, conditions, and exclusions from coverage.

PART 2 PRODUCTS

2.1 MANUFACTURERS

A. Acceptable Manufacturer: FloodBreak Automatic Floodgates, which is located at: 2800 Post Oak Blvd. Suite 5850 ; Houston, TX 77056; Tel: 713-980-6610; Fax: 713-629-9936; Email: info@floodbreak.com; Web: www.floodbreak.com

B. Substitutions: Substitutions are allowed so long as all other requirements of the specification are met by the substitute bidder.

C. Requests for substitutions will be considered in accordance with provisions of Section 01600.

2.2 APPLICATIONS/SCOPE

A. Provide a means of passively protecting human and property assets subject to damage during a flood caused by external forces. Passive shall mean that the gate functions without human intervention or power to make the gate deploy and drain.

2.3 DESIGN REQUIREMENTS

A. Design gate height based on the Mitigation Height at the location of the gate as determined by the Federal Emergency Management Agency (FEMA) or equivalent entity.

B. Design the gate to allow safe passage of vehicular and human traffic while in its dry or "Closed" position.

C. Design the gate to hinder the passage of floodwater and resist hydrostatic pressures while in its operating or "Open" position.

D. Design the gate to exclude the use of any electric or mechanical powered support equipment or pumps, for any operation of the gate to its open or closed position in passive mode.

E. Design the system to include the ability to actively power the gate into operating position using a pushbutton-activated powered lift system. The active power system shall be fully decoupled so that at no time will it interfere with or be required for the fully passive operation of the gate, regardless of power availability.

F. Design that the actual gate installation "set-down" below surface grade is a maximum of 12 inches for pedestrian openings and 24" for vehicular or roadway applications.. Gate shall anchor into structural foundation.

G. Design the gate system using only aluminum and stainless steel components to resist corrosion and EPDM rubber for gasketing.

2.4 COMPONENT

A. Concrete: ASTM C 39 concrete; Compressive strength as recommended by project engineer.

B. Pan Inlet Grate:
   1. Vehicular Grates: 3/8 by 1 inch (10mm x 25mm) flat aluminum bar spaced 3/8 inch (10mm).
   2. Pedestrian Grates: 1/8 by 1 inch (3mm x 25mm) flat aluminum bar spaced 1/8

inch (3mm)

C. Gaskets: 3/16 inch (4.8mm) EPDM rubber.

D. Gate Support Tubing:
   1. Material: 3/16 inch (4.8mm) structural 2 inch by 2 inch (51mm x 51mm) square extrusions - Grade 6063 aluminum.
   2. Minimum Yield (Fy): 40 KSI.

E. Hardware:
   1. Concrete Anchor Bolts:
      a. Material: 1/2 inch (13mm) diameter ASTM A 240/240M Grade 304 Stainless Steel.
      b. Minimum Yield (Fy): 90 KSI.
   2. Hinge Pins:
      a. Material: 1/2 inch (13mm) diameter ASTM A 240/240M Grade 304 Stainless Steel.
      b. Minimum Yield (Fy): 90 KSI.
   3. Bolts:
      a. Material: Countersunk ASTM A 240/240M Grade 304 Stainless Steel bolts. Bolt diameter as noted on the contract drawings.
      b. Minimum Yield (Fy): 90 KSI.
   4. Retention Arm Anchors:
      a. Material: 3/8 inch (13mm) ASTM A 240/240M Grade 304 Stainless Steel.
      b. Minimum Yield (Fy): 90 KSI.
   5. Welding Wire: Aluminum Wire - ER 4043 AWS A5.10 3/32

F. Pan Support Tubing:
   1. Material: 1/4 inch (6mm) structural 2 inch by 2 inch (51mm x 51mm) square extrusions - Grade 6063 Aluminum.
   2. Minimum Yield (Fy): 40 KSI.

G. Pan:
   1. Material: 1/4 inch (6mm) smooth plate - Grade 5052 Aluminum.
   2. Minimum Yield (Fy): 30 KSI.

H. Gasket Flanges:
   1. Material: 1/4 inch (6mm) 6061-T6 aluminum.
   2. Minimum Yield (Fy): 40 KSI.

I. Retention Arm:
   1. Material: 1/2 inch by 1/2 inch (13mm x 13mm) 6061-T6 Aluminum flat stock.
   2. Minimum Yield (Fy): 40 KSI.

J. Structural Angles:
   1. Material: 1/4 inch (6mm) structural 2 inch by 2 inch (51mm x 51mm) angles - 6061-T6 aluminum.
   2. Minimum Yield (Fy): 40 KSI.

2.5 FABRICATION

A. General Requirements:
   1. Fabricate all components and elements following the standards, tolerances and guidelines noted in the contract drawings.
   2. All welding to be performed by a certified welder in accordance with AWS standards and guidelines.

3. Tighten all bolts to torque specifications determined by the manufacturer and Engineer of record.

B. Concrete: Encapsulate pan and extending bars in a monolithic concrete pour with a depth of no less than 11 inches (280mm) and extending a lateral distance from the pan no less than 12 inches (305mm) in any direction.

C. Pan:
 1. Fabricate pan to include a drainage trough running parallel to and for the entire length of the gate at the approximate centerline of the pan. Trough will have a depth of 2 inches (51mm) and a width of 6 inches (152mm).

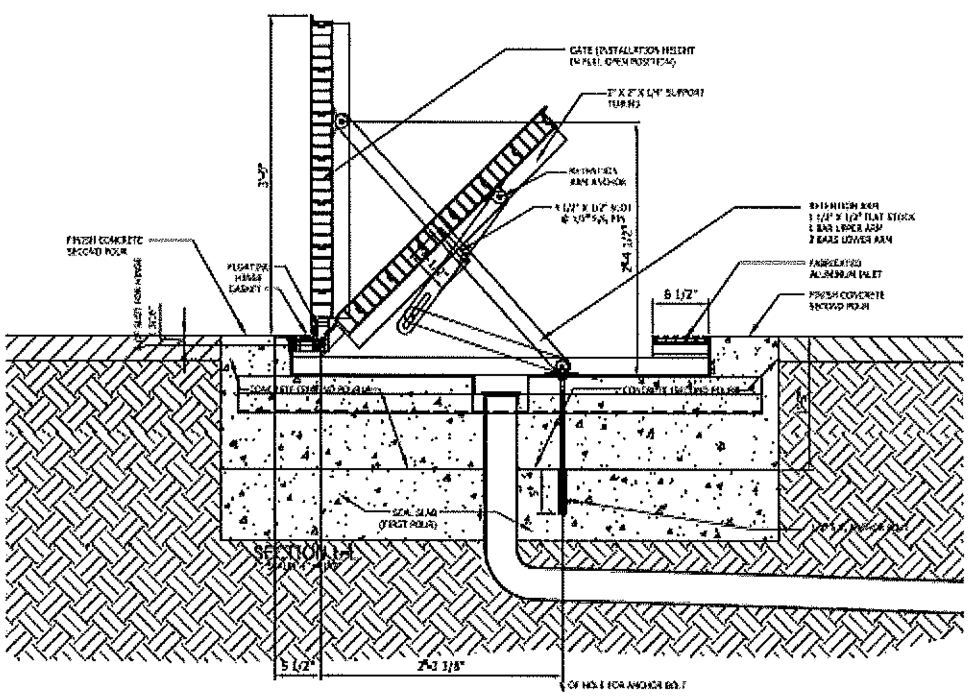
D. Drainage: Connect 4 inch (102mm) diameter drain to the drainage trough centered within the pan in all directions.

E. Gate:
 1. At panel joints, stitch weld every 5 inches (127mm) on center with a 3/16 inch fillet weld 3 inches (76mm) long.
 2. At panel splices, place splice flanges within 12 inches (305mm) of adjacent retention arms.

F. Hinges and Anchors:
 1. Seam-weld retention arm brackets to gate and pan. Include stiffener plates on each side.
 2. Attach retention arm anchors through pan and into concrete with 1/2 inch (13mm) diameter anchor bolts.

G. Wiper Wall: Manufacturer to provide 3/8 inch (10mm) aluminum wiper wall to maintain contact with gate seal and protective gaskets at all points of operation.

2.6 DRAWING



PART 3 EXECUTION

3.1 EXAMINATION

A. Do not begin installation until substrates have been properly prepared.

B. If substrate preparation is the responsibility of another installer, notify Architect of unsatisfactory preparation before proceeding.

3.2 PREPARATION

A. Clean surfaces thoroughly prior to installation.

B. Prepare surfaces using the methods recommended by the manufacturer for achieving the best result for the substrate under the project conditions.

3.3 INSTALLATION

A. Install in accordance with manufacturer's instructions.

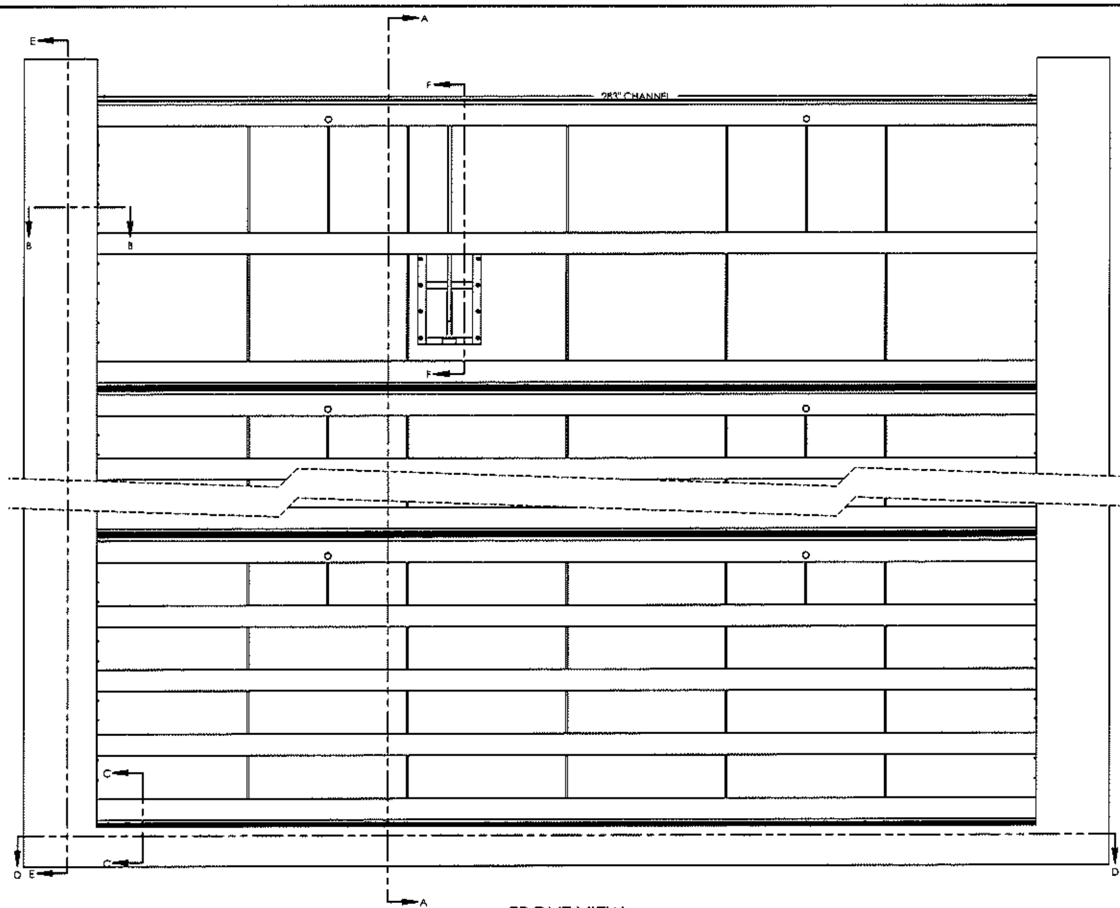
3.4 PROTECTION

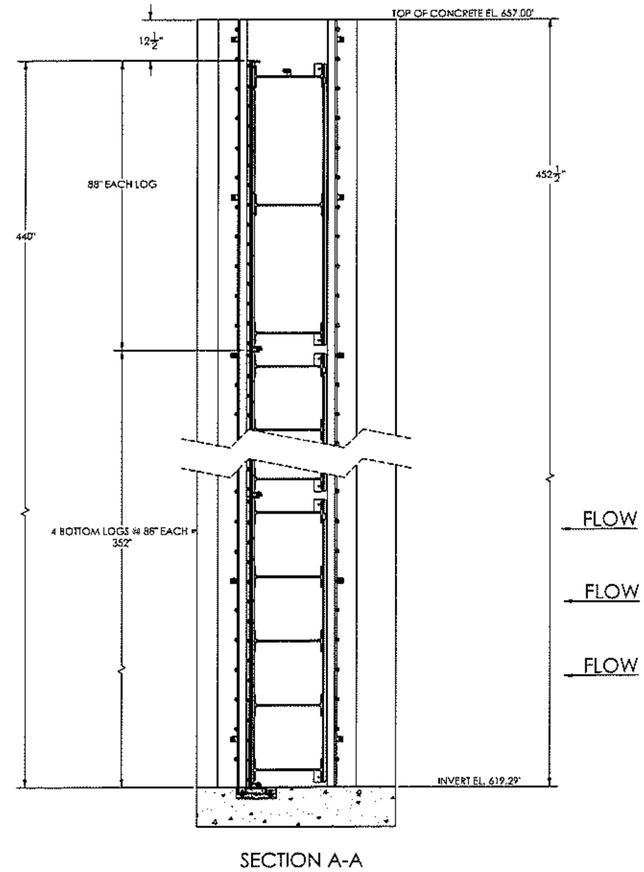
A. Protect installed products until completion of project.

B. Touch-up, repair or replace damaged products before Substantial Completion.

END OF SECTION

HydroGate Stop Logs
Flood Gate





NOTES:

1. DO NOT SCALE DRAWINGS
2. CONTRACTOR TO VERIFY ALL DIMENSIONS AND ELEVATIONS
3. MATERIAL SPECIFICATIONS AND COATINGS PER DRAWING: 1560172500
4. TAG: 283X456 STOP LOG
5. LOCATION: ENTRANCE BADGER PLANT
6. REFERENCE SPECIFICATIONS SECTION: AISC AND USACE EM 1110-2-2105
7. REFERENCE PLAN DRAWINGS: 4112-ST-2614-4
8. APPROXIMATE WEIGHTS: BOTTOM LOGS 11300 LB EACH, TOP LOG 9,300 LB, EMBEDED WALL SLOTS 2400 LB, INVERT PLATE 250 LB
9. GUIDES SHALL BE INSTALLED PARALLEL AND PLUMB WHILE MAINTAINING INSIDE OF GUIDE TO INSIDE OF GUIDE DIMENSION.
10. BOTTOM LOGS RATED FOR 33.00 FEET OF HEAD MEASURED FROM CENTERLINE TOP LOG RATED FOR 3.66 FEET OF HEAD MEASURED FROM CENTERLINE

QUANTITIES:

(2) STOP LOG GUIDE SETS EACH INCLUDING TWO EMBEDED WALL SLOTS AND ONE INVERT PLATE.

(4) BOTTOM STOP LOGS

(1) TOP LOG WITH HANDPULL GATE

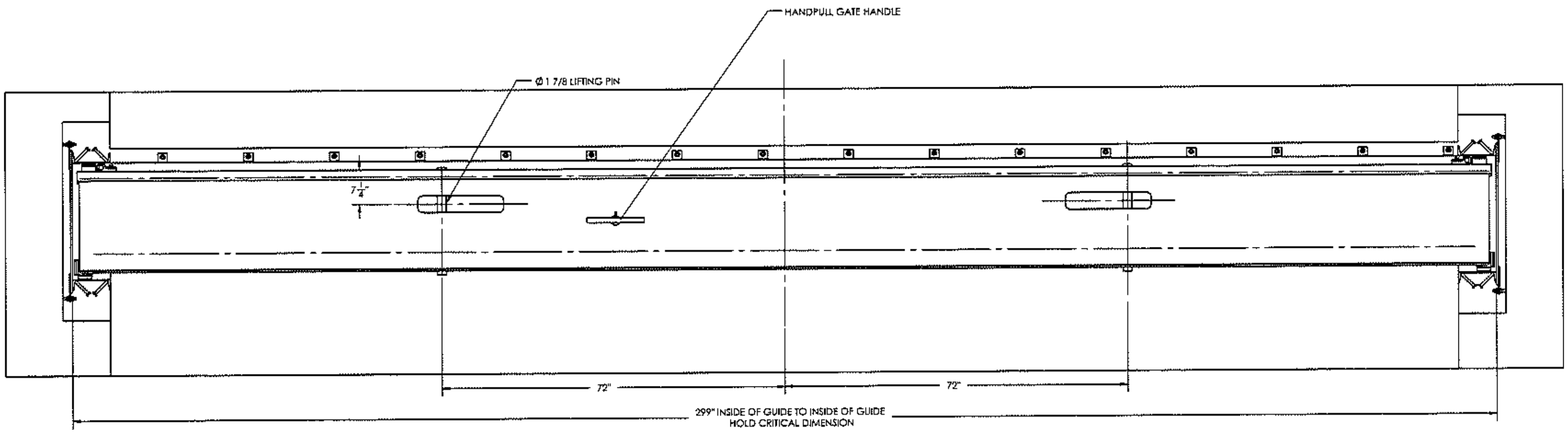
Hydro Gate

283" X 453" STOP LOGS

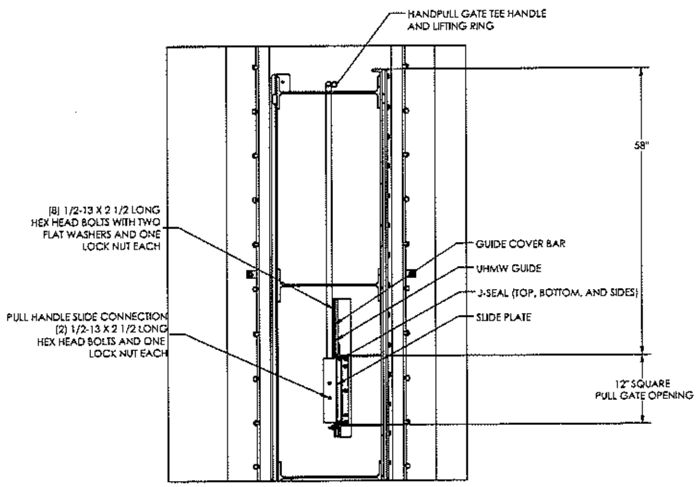
QTY: SEE NOTE | MATERIAL: CARBON STEEL | SCALE: 1"=5/16"

REV	DATE	INITIALS
A	12/06/12	AMM

DRAWN BY	CUSTOMER NO.	SALES
AMM	LOI	1560172
CHECKED BY	DATE	DRAWING NO.
AMM	08/22/2012	1560172-01 SHEET 1



TOP VIEW



SECTION F-F
HANDPULL GATE DETAIL
(TOP LOG ONLY)

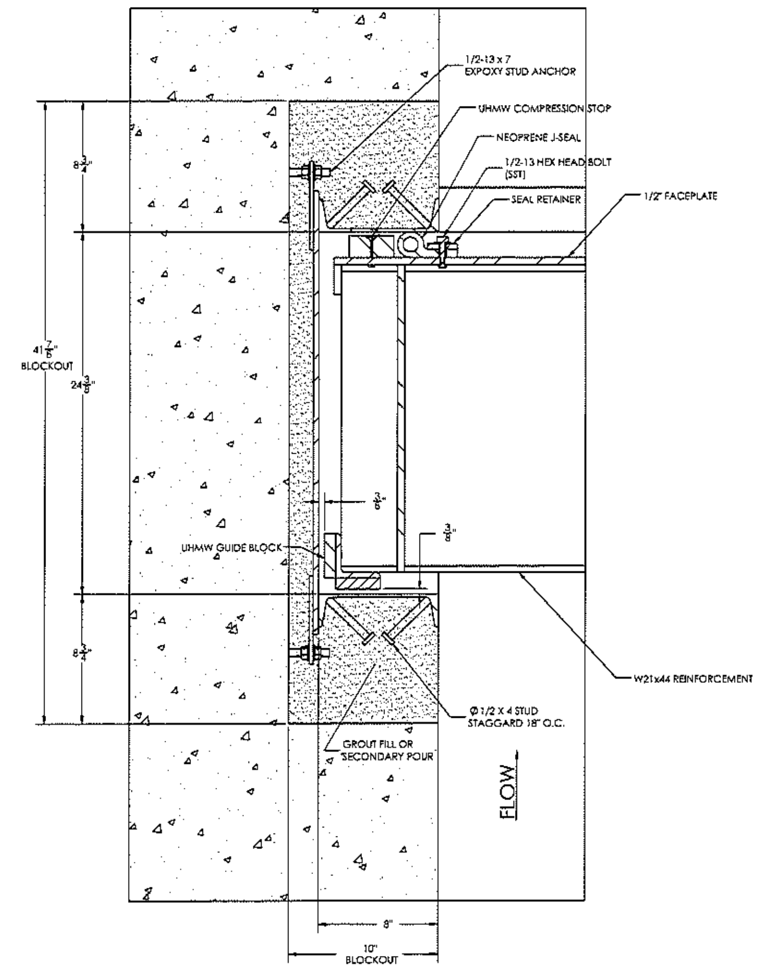
Hydro Gate

283" X 453" STOP LOGS

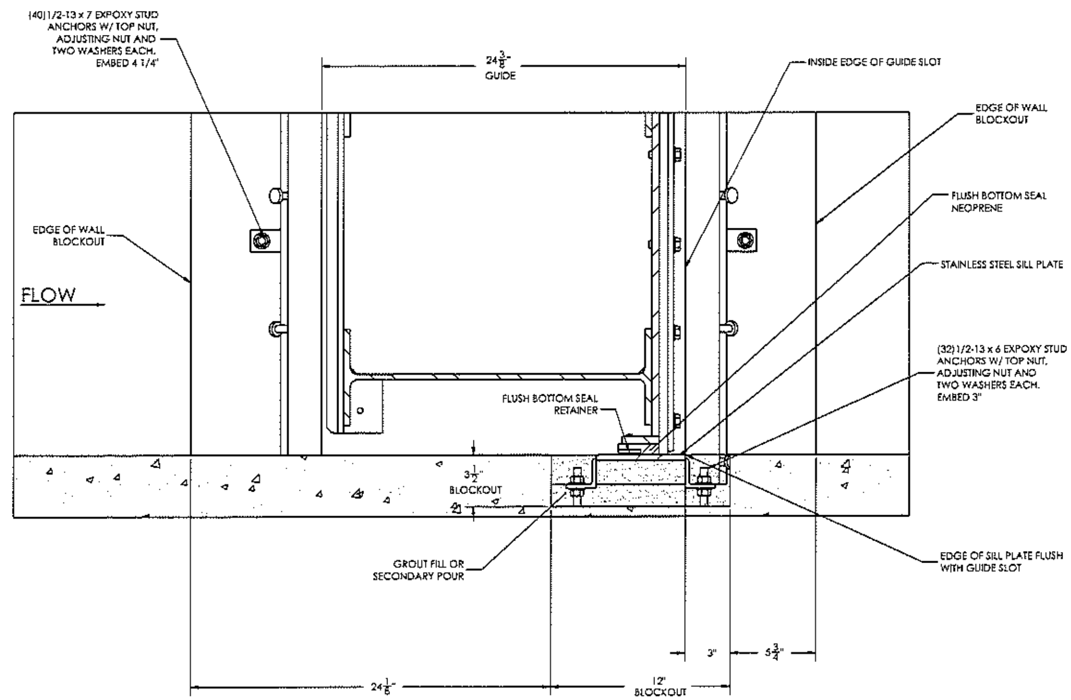
QTY: SEE NOTE | MATERIAL: CARBON STEEL | SCALE: 1"=1/2"

REV	DATE	INITIALS
A	12/06/12	AMM

DRAWN BY	CUSTOMER NO.	SALES
AMM	LOI	1560172
CHECKED BY	DATE	DRAWING NO.
AMM	08/22/2012	1560172-01 SHEET 2



SECTION B-B
TYPICAL SIDE SECTION



SECTION C-C
INVERT SECTION

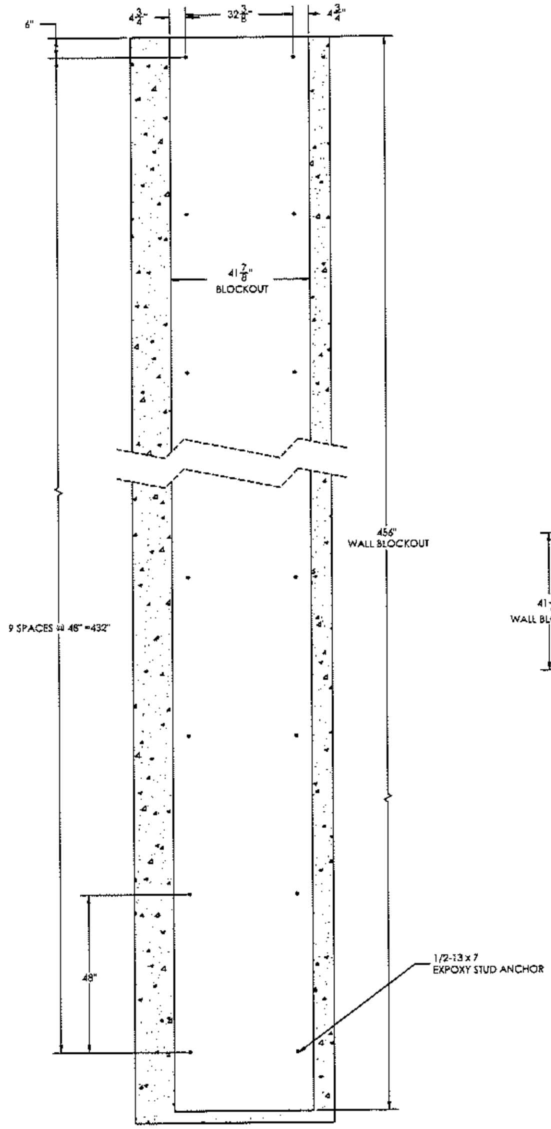
Hydro Gate

283" X 453" STOP LOGS

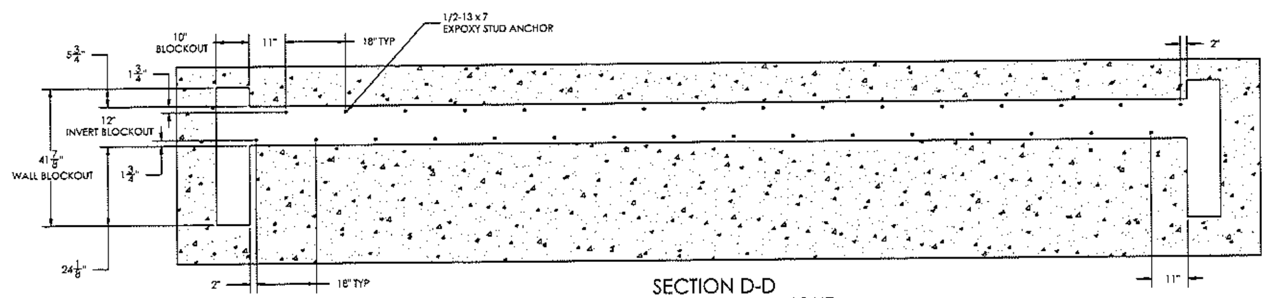
QTY: SEE NOTE | MATERIAL: CARBON STEEL | SCALE: 1"=1 1/2'

DRAWN BY	CUSTOMER NO.	SALES
AMM	LOI	1560172
CHECKED BY	DATE	DRAWING NO.
AMM	08/22/2012	1560172-01 SHEET 3

REV	DATE	INITIALS
A	12/06/12	AMM



SECTION E-E
WALL ANCHOR LAYOUT



SECTION D-D
INVERT ANCHOR LAYOUT

Hydro Gate

283" X 453" STOP LOGS

QTY: SEE NOTE | MATERIAL: CARBON STEEL | SCALE: 1"=5/16"

THIS IS A PROPRIETARY DESIGN OF HYDRO GATE, LLC. THE DESIGN, DATA AND INFORMATION RELATING THERETO IS NOT TO BE USED, DISSEMINATED OR REPRODUCED IN WHOLE OR IN PART WITHOUT THE EXPRESS WRITTEN CONSENT OF HYDRO GATE, LLC.

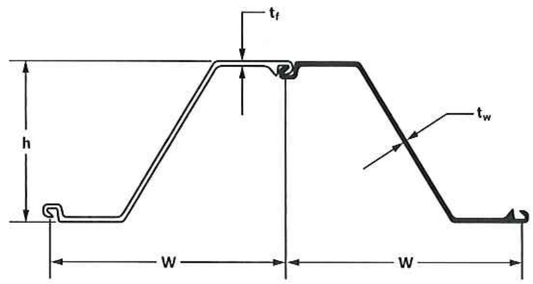
REV	DATE	INITIALS
A	12/06/12	AMM

DRAWN BY	CUSTOMER NO.	SALES
AMM	LOI	1560172
CHECKED BY	DATE	DRAWING NO.
AMM	08/22/2012	1560172-01 SHEET 4

Permanent Sheet Piles Alternative Construction of Material

NZ

NZ Hot Rolled Steel Sheet Pile



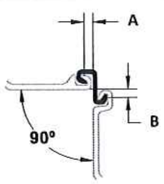
SECTION	Width (w) in (mm)	Height (h) in (mm)	THICKNESS Flange (t_f) in (mm)	Web (t_w) in (mm)	Cross Sectional Area in^2/ft (cm^2/m)	WEIGHT Pile lb/ft (kg/m)	Wall lb/ft^2 (kg/m^2)	SECTION MODULUS Elastic in^3/ft (cm^3/m)	Plastic in^3/ft (cm^3/m)	Moment of Inertia in^4/ft (cm^4/m)	COATING AREA Both Sides ft^2/ft of single (m^2/m)	Wall Surface ft^2/ft^2 (m^2/m^2)
NZ 19	27.56 700	16.14 410.0	0.375 9.5	0.375 9.5	7.04 148.9	55 81.85	23.95 116.93	35.08 1886	41.33 2222	283.1 38659	6.18 1.88	1.35 1.35
NZ 20	27.56 700	16.16 410.5	0.394 10.0	0.394 10.0	7.29 154.4	57 84.83	24.82 121.18	36.24 1948	42.80 2301	292.8 39984	6.18 1.88	1.35 1.35
NZ 21	27.56 700	16.20 411.5	0.433 11.0	0.433 11.0	7.80 165.2	61 90.78	26.56 129.68	38.69 2080	45.85 2465	313.4 42797	6.18 1.88	1.35 1.35
NZ 26	27.56 700	17.32 440.0	0.500 12.7	0.500 12.7	9.08 192.3	71 105.66	30.92 150.94	48.50 2608	57.01 3065	419.9 57340	6.49 1.98	1.41 1.41
NZ 28	27.56 700	17.38 441	0.560 14.2	0.560 14.2	9.98 211.2	78 116.08	33.96 165.82	52.62 2829	62.16 3342	457.4 62461	6.49 1.98	1.41 1.41

NZ

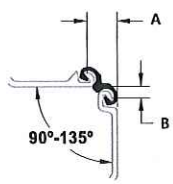
NZ Hot Rolled Steel Sheet Pile

Available Steel Grades		
NZ		
ASTM	YIELD STRENGTH	
	(ksi)	(MPa)
A 328	39	270
A 572 Gr. 50	50	345
A 572 Gr. 60	60	415
A 588	50	345
A 690	50	345

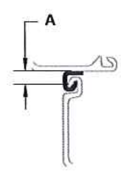
Corner Piles



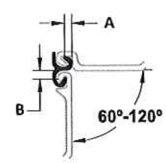
C 14
Gr: S 355 GP
Wt: 9.68 lb/ft (14.4 kg/m)
A: ~0.98" (25 mm)
B: ~0.98" (25 mm)



Omega 18
Gr: S 430 GP
Wt: 12.10 lb/ft (18.0 kg/m)
A: ~2.76" ~(70 mm)
B: ~1.18" ~(30 mm)

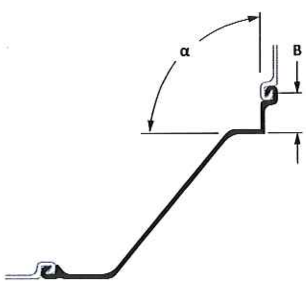


E 22
Gr: S 355 GP
Wt: 6.87 lb/ft (10.2 kg/m)
A: ~1.18" (~30 mm)

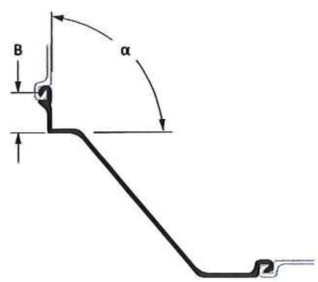


Delta 13
Gr: S 355 GP
Wt: 8.73 lb/ft (13.0 kg/m)
A: ~0.59" (~15 mm)
B: ~0.79" (~20 mm)

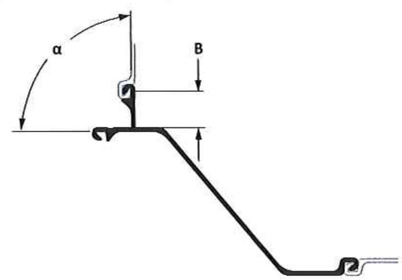
Fabricated Corner Piles



Type 1 - α
α: Angle varies
B: 3"–6" (76.2mm– 152.4mm)



Type 2 - α
α: Angle varies
B: 3"–6" (76.2mm– 152.4mm)



T Pile
α: Angle varies
B: 3"–6" (76.2mm– 152.4mm)

Delivery Conditions & Tolerances

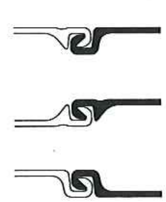
	ASTM A 6	
Mass	± 2.5%	
Length	+ 5 inches	– 0 inches

Maximum Rolled Lengths*

NZ	105.0 feet	(32.0 m)

* Longer lengths may be possible upon request.

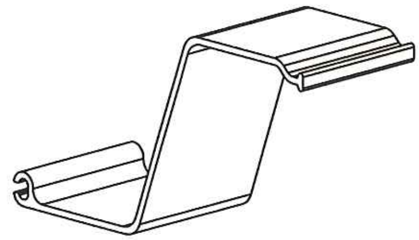
Interlock Combinations



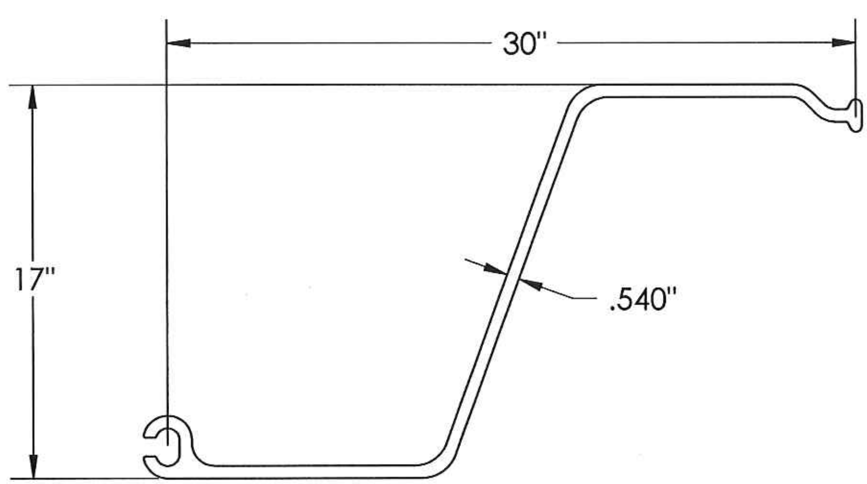
Crane Materials International

UltraComposite
Sheet Piling and Piles

Updated November 1, 2014

UC-95	

Allowable Moment (M)	48,750 ft-lb/ft	216.84 kN-m/m
Section Modulus (Z)	58.5 in^3/ft	3,145 cm^3/m
Moment of Inertia (I)	497 in^4/ft	67,870 cm^4/m
Thickness (t)	0.540 in	13.7 mm
Section Depth	17 in	432 mm
Section Width	30 in	762 mm
Material	Structural FRP Composite	
Standard Colors	Charcoal	
Profile/Patented Features	Z Profile	

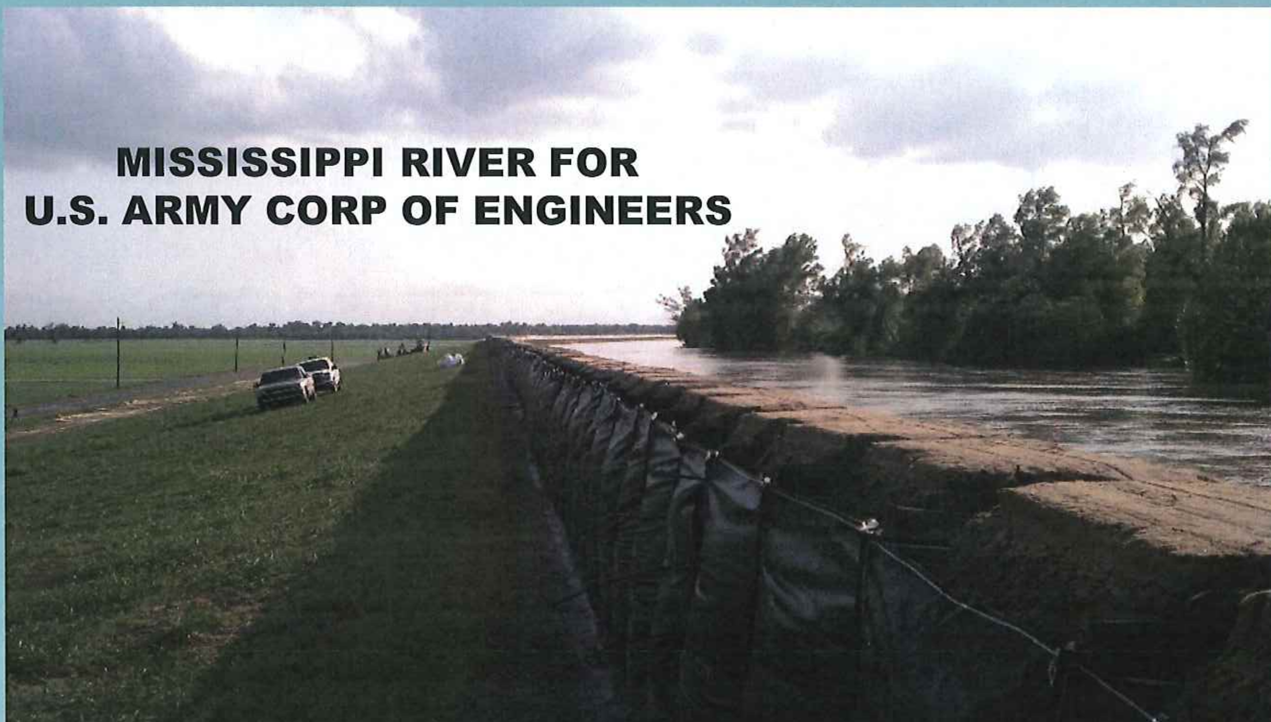


Temporary Flood Protection Alternatives

TrapBag Temporary Flood Protection Alternative

TrapBag®

FLOOD & EROSION PROTECTION



You may not always be able to predict however you can always prepare

Advantage Points

TrapBag®
FLOOD & EROSION PROTECTION

- Low cost
- Easy to transport
- Easy to store
- Low installation cost
- Less manpower
- Quick & easy installation
- Flexible installation
- 35% less fill material
- Attractive print design
- Long life
- Great Support
- Double Fabric Layers Option (DL)
- Endless Structures
- Solid Immovable Structures
- High stability and load capacity
- Structures of multiple heights
- Stackable Structures
- Permanent when filled with concrete

TrapBag®

TrapBag Applications

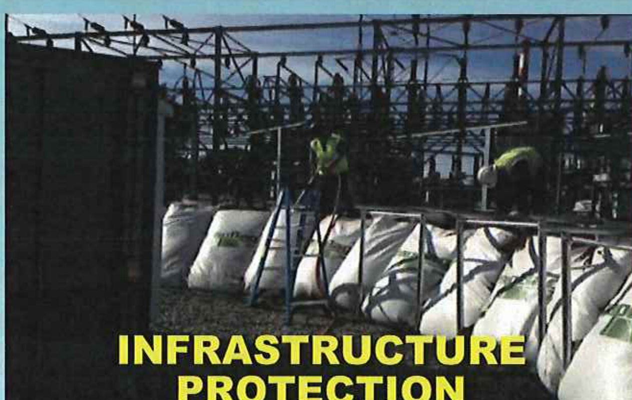
TrapBag®
FLOOD & EROSION PROTECTION

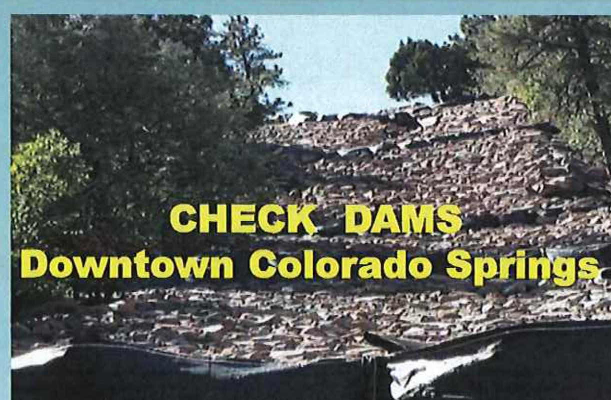
TrapBag®

Product Specifications

TrapBag®
FLOOD & EROSION PROTECTION

DESCRIPTIONS		TBR60	TBR120	TBS /TBSD200
	Ft.	2 ft.	4. ft.	6 ft.
Width of each cell	In	40	40	40
Depth of each cell	In	30	55	96
Volume of each cell	Cu. Yds.	.60	2.2	5.9
Cells per set		15	15	15
Filling Volume per set	Cu. Yds.	9	33	88
Filling Weight per set	Tons	12 +/-	44.5	120
Partition wall		RIGID	RIGID	SOFT
Cover (Option)		YES	YES	NO
Double Fabric Layer (DL)		No	No	No
Installation Kit Required		NO	NO	YES





TRAPBAG.COM **TRAPBAG DISTRIBUTOR**

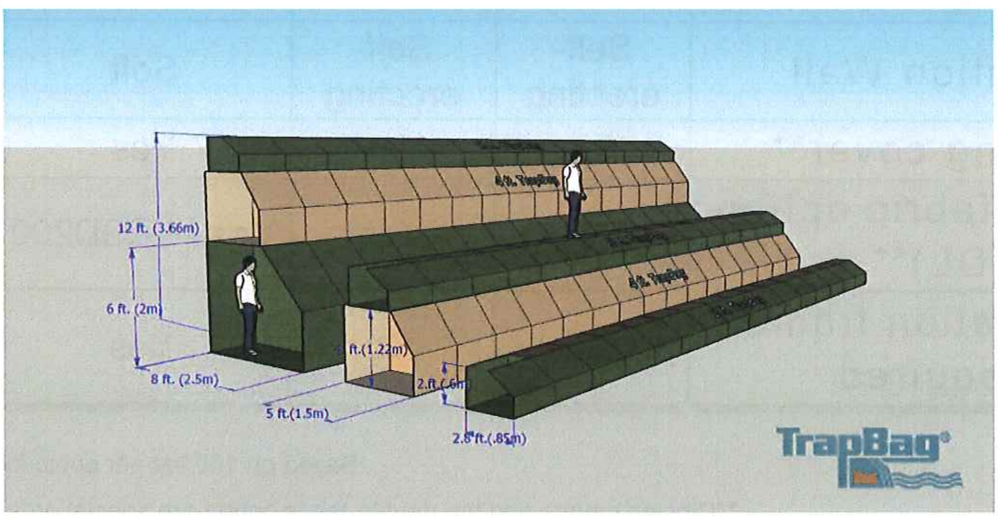
Everett Waid

President, Owner
Sentinel Barriers, LLC
15465 Pine Ridge Rd.,
Fort Myers, FL 33908
(239)229-5285

TrapBag®
FLOOD & EROSION PROTECTION

SPECIFICATIONS

TrapBag® barrier bags come in three sizes: 2ft., 4ft., and 6ft. heights, can be filled with various materials for semi-permanent or permanent installation, and can be stacked on top of each other to provide even larger barriers.



All of our TrapBag models have the option of a double front fabric layer (DL) that can be topped with a closing cover.

The double layer option will ensure a longer life to our bags in more severe conditions where the barrier is exposed to excessive abrasion and UV rays for extended periods of time.

With the closing cover option the fill material is not eroded from the compacted barrier when wave action or overtopping is encountered.

Specifications

Model	TBR60 (2ft.)	TBR120 (4ft.)	TBS/TBSD200 (6ft.)
Width/depth/height ea. cell	40 in./ 30 in/ 2 ft.	40 in./ 55 in./ 4 ft.	40 in./ 96 in./ 6 ft.
Volume of each cell	0.6 yd^3 +/-	2.2 yd^3 +/-	5.9 yd^3 +/-
Cells per 50 ft.	15	15	15
Filling volume per 50 ft.	9 yd^3 +/-	33 yd^3 +/-	88 yd^3 +/-
Filling weight per 50 ft.*	12 tons +/-	44.5 tons +/-	120 tons +/-
Partition Wall	Self-erecting	Self-erecting	Soft
Closing cover**	Yes	Yes	Yes
Double fabric option (DL)**	Yes	Yes	Yes - TBSD200
Installation frame Required	No	No	Yes

*Based on 100 lbs per cubic foot

**Closing covers and the double fabric option are special orders

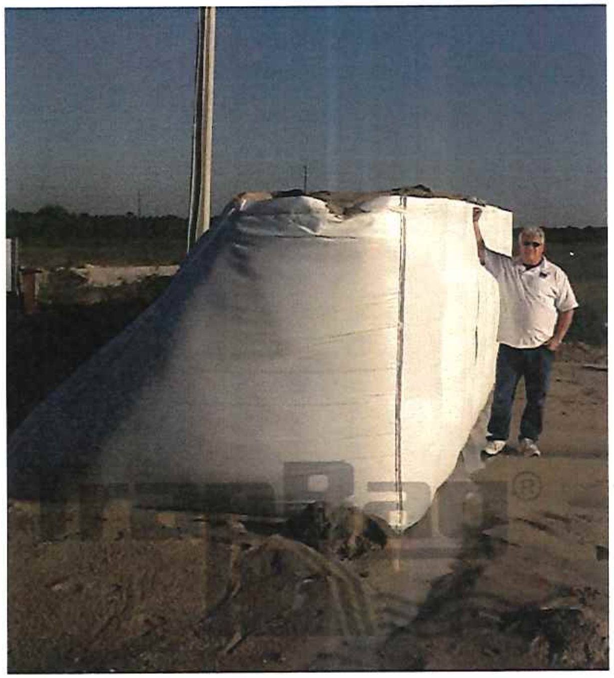
Each of our barrier sizes (2 ft.,4 ft., & 6 ft.) individually have a minimum mass-to-hydraulic-load (safety) ratio of 3.5 to 1 at overtopping or greater. This is based on 100 lb. per cubic ft. fill material. That safety ratio is exponentially greater as the water level declines from overtopping or heaver fill is used.

The above does not include stacking: stacking is used to create 2 ft. of freeboard when the 2 ft. is stacked on top of the 4 ft. barrier or the 4ft. is stacked on the 6 ft. This provides a 3+ to 1 safety ratio.

The TrapBag Barrier TBR models

Because our 2 ft. and 4 ft. TrapBags are built with a rigid partition wall between the cells, installation time is drastically reduced since these bags can be set up without using the installation frames.

TRAPBAG
PHOTOS



6 ft tall TrapBag

4 ft tall TrapBag

Filling a 4 ft tall TrapBag

TrapBag Pallet

Stacking 2 ft tall TrapBag on top of 4 ft tall TrapBag.

Filling a 4 ft tall TrapBag

TrapBag with UV guard

Installation of the 6 ft tall TrapBags

Muscle Wall Temporary Flood Protection Alternative

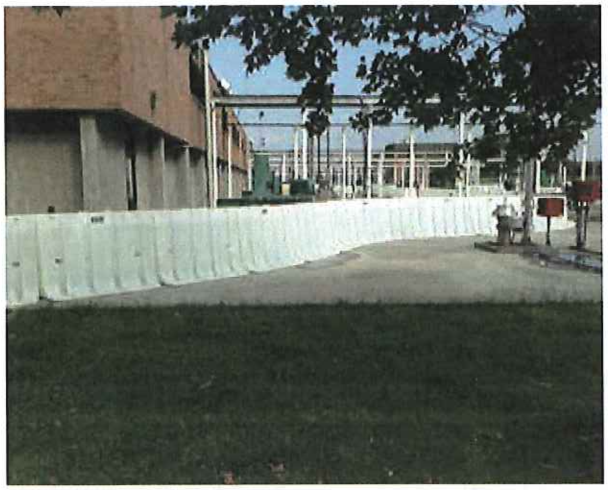
MUSCLE WALL®

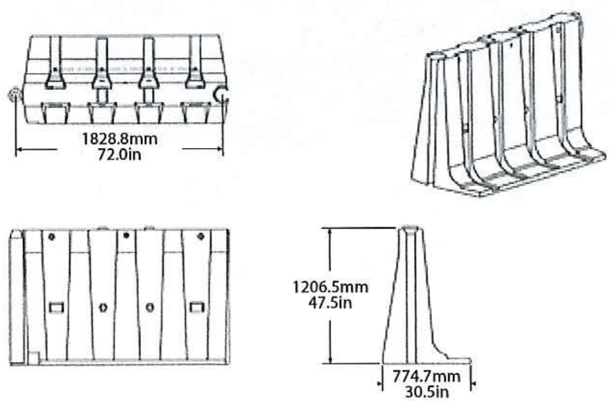
Specifications:

- Material
  - Low density polyethylene
  - Elongation to yield: 20%
  - Impact strength: 190 ft-lb
  - Tensile strength at yield: 2600 psi
- All Season Compatible
  - Temperature range: -40˚ F to 180˚ F
  - 10 year UV rated
- Portable
  - Weight per unit (empty): 110lbs
  - Weight per unit (filled): 1400lbs
  - Units nest together for transportation
- Ground Pressure
  - Empty: 0.0527 psi
  - Filled: 0.6705 psi
- Dimensions
  - Minimum polyethylene thickness: 0.25"
  - Footprint on ground: 14.5 ft^2
  - 6 ft. wide x 2.54 ft. deep x 4 ft. high
  - Installed in 6 ft. sections
  - Fit 80 units on one 48 ft. flatbed trailer

4 Foot System

MUSCLE WALL®





Features:

- Walls interconnect
- Connection acts like a hinge allowing for 22° of motion
- Corner piece allows 90° turns
- Ratchet straps restrain adjacent panels
- Tongue and groove panel interface for easy staking
- Patent Protected
  - US 8.313.265 B2
  - USD 631977
  - US 634443

Contact us:

Please contact us with any questions you have regarding Muscle Wall and how we can help you with your water management needs.

Email – info@musclewall.com
Toll-free – 1.800. 801.8739
Fax – 435.514.6707
Mail – 675 North 600 West Suite 1
Logan, UT 84321
Web – www.musclewall.com

MUSCLE WALL®

Standard Operating Procedures
4ft Muscle Wall

Flood & Stormwater Management

Muscle Wall Holdings, LLC
1.800.801.8739
info@musclewall.com
www.musclewall.com

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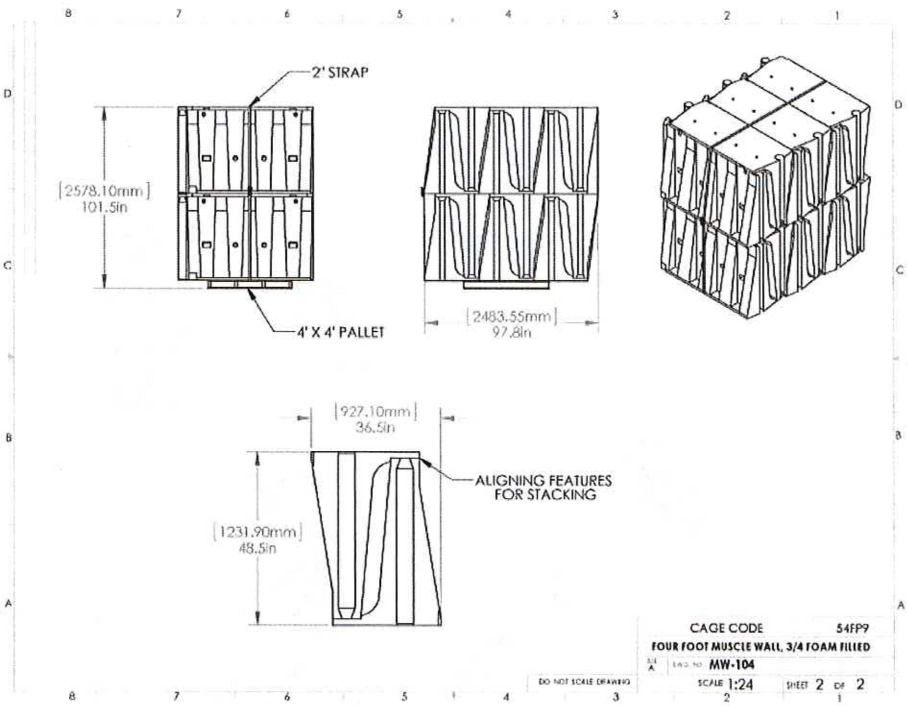
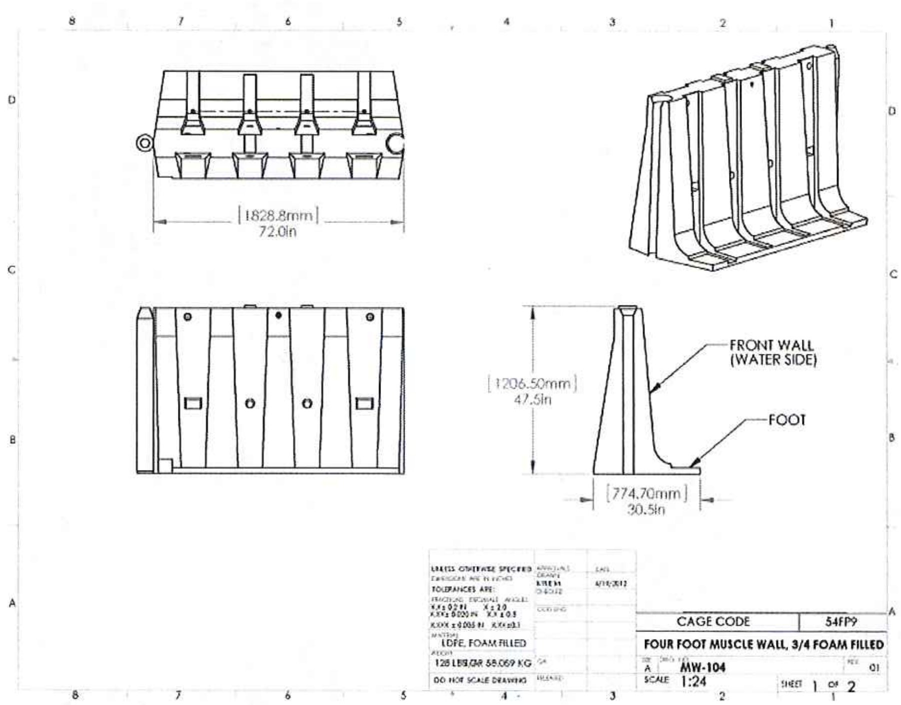
Muscle Wall Holdings, LLC
1.800.801.8739
info@musclewall.com
www.musclewall.com

Specifications





Muscle Wall Holdings, LLC
1.800.801.8739
info@musclewall.com
www.musclewall.com

Items Needed

- Shovel
- 200ft Measuring Tape
- Marking Paint
- 2 Sledge Hammers
- 10 Sand Bags for every Section of Muscle Wall
- Gloves
- 2 Razor Knives
- 500-1000ft of String
- Trash Pump
- Fork Lift
- Lifting Dolly
- Gorilla Tape
- Pressure Washer
- "Great Stuff" Foam Sealant
- Trencher











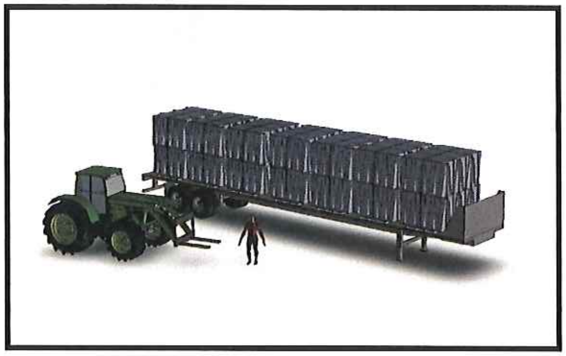


Stage 1

Muscle Wall Setup

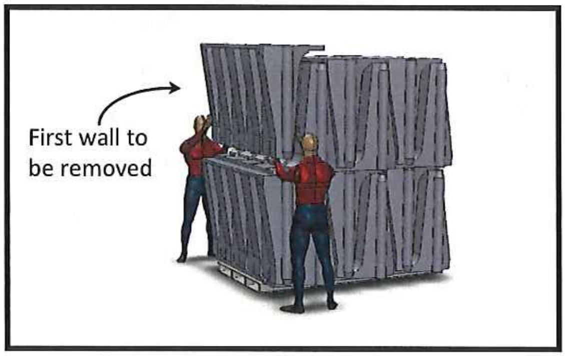
Step 1

With forklift unload trailer and strategically place bundles throughout area for deployment.



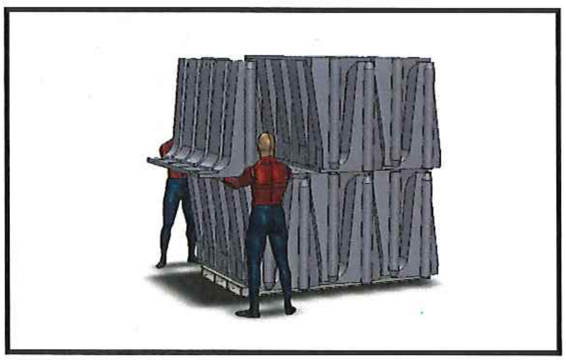
Step 2

Always remove the upside-down wall first. One person on each side lifts the wall up, freeing the securing pegs, then lowers the wall to a comfortable carrying position.



Step 3

When removing the right-side up wall one person stands on each side, slides the wall to the edge, then lowers the wall to a comfortable carrying position. Dropping the Muscle Wall could cause damage and/or personal injury. Handle with care.



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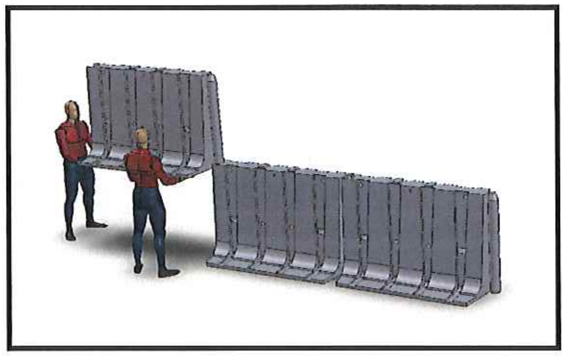
Step 4

One person on each side of the Muscle Wall raises it and rests it on top of the connecting wall until ready to slide into place. Be sure to exercise proper lifting techniques and to keep hands free of the joint while the Muscle Wall is sliding into place.

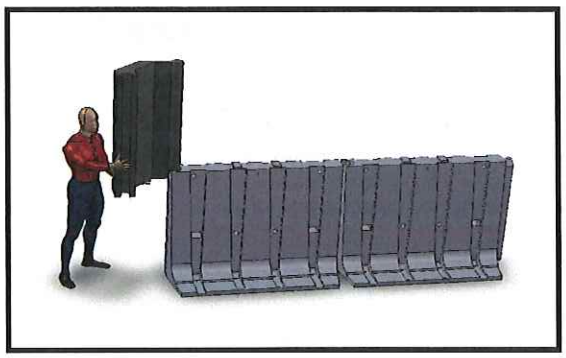


Step 5

Be sure that the toe of the Muscle Wall is facing the water.

Step 6

If corners are being used, one or two people raise and slide the corner piece into the connecting wall. When using corners two straps per connection are required.



Step 7

Once walls are set in place begin filling walls with water using the trash pump. In most situations filling walls half way is all that is necessary.

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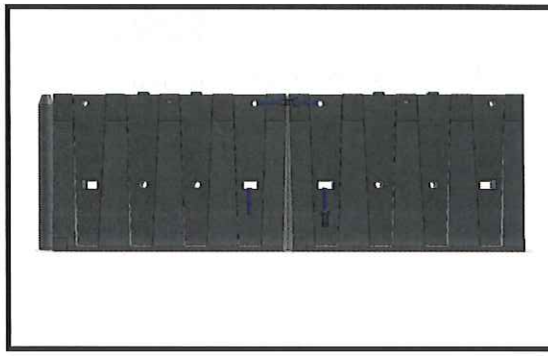
Step 8

Put rachet straps through the Muscle Wall in the closest holes to the joint. Tighten only the top strap for now. The straps need to be fed into the Muscle Wall from the side without the toe.



Stage 2

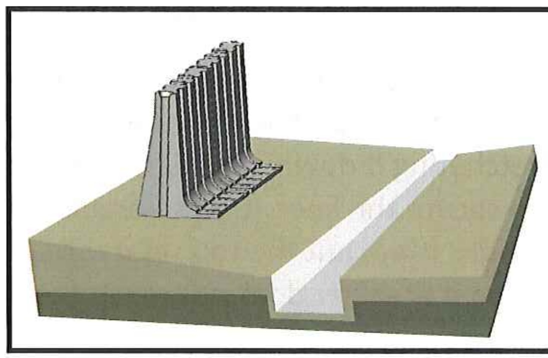
Liner Deployment

Deploying on Soil

Trench Option

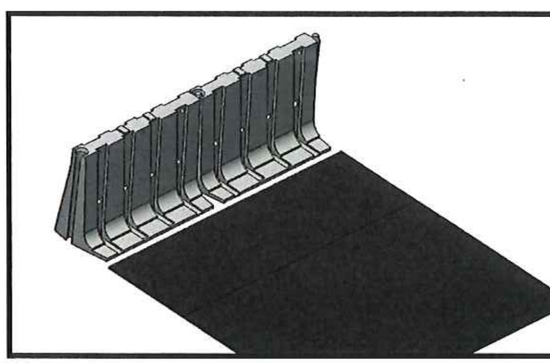
Step 1

About 3 feet away from the toe of the Muscle Wall dig a trench approximately 16 inches deep that spans the entire wall of Muscle Wall.



Step 2

Roll out and unfold the liner in front of the toe of the Muscle Wall, placing about 16-20 inches of the edge of the liner in the trench, and fill the trench back in with soil. Compact the soil as much as possible.



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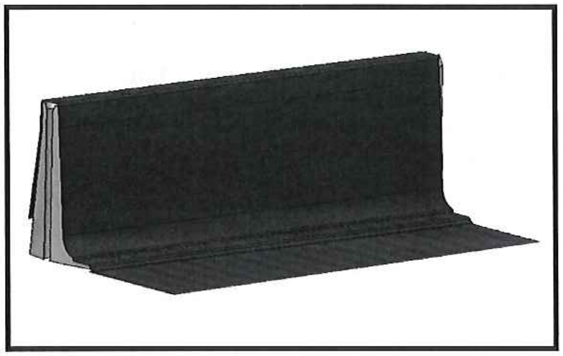
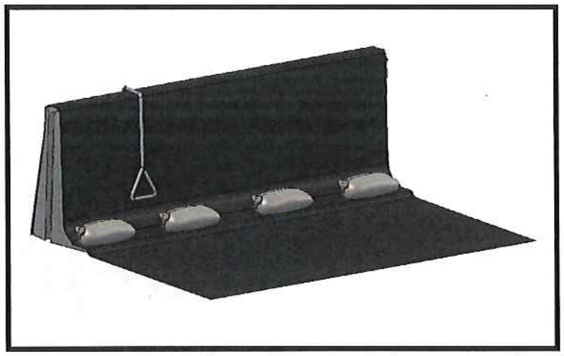
Step 3

Pull liner up and over the wall. Do not pull liner too tight as this may cause tenting which may lead to rips or tears.



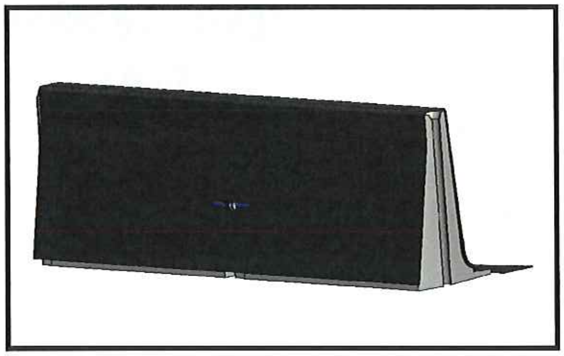
Step 4

Ensure that the liner is flat against the wall and place sandbags on the toe to stop the wind from picking up the liner. Place a steel clip every fourth wall to secure the liner to the Muscle Wall.



Step 4

Secure the liner by cutting a small horizontal slit in the liner and pulling the safety strap through and ratcheting it down. Only one of the straps is needed to secure the liner. It is preferable to use the bottom safety strap, but the top safety strap may be also be used if the liner does not reach the bottom one.



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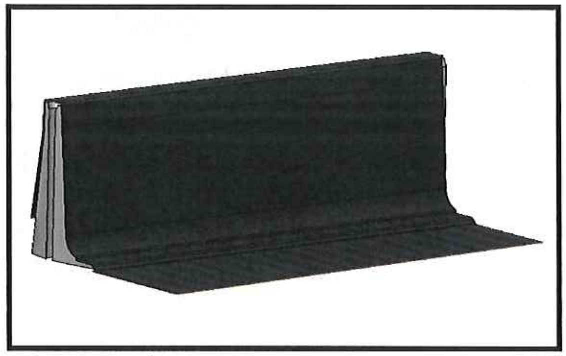
Liner Deployment

Deploying on Asphalt

Foam Option

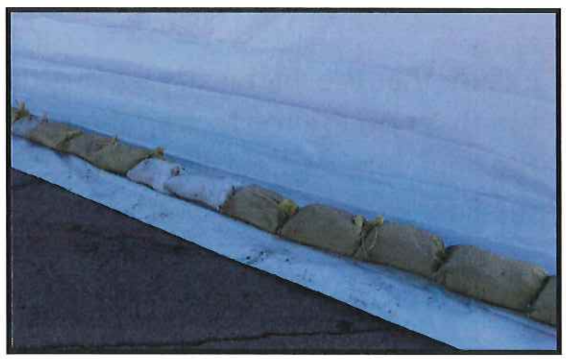
Step 1

Roll out and unfold the liner. Pull it up and over the Muscle Wall with approximately 3 feet of the liner flat on the ground in front of the toe of the Muscle Wall.



Step 2

Lay a line of sandbags back to back on top of the liner approximately 1 foot away from the edge of the liner.



Step 3

Lift the edge of the liner up and spray a liberal amount of foam on the ground under the liner. Ensure that there is enough foam to bubble out from underneath the liner when you lay the liner flat on the ground.

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Step 4

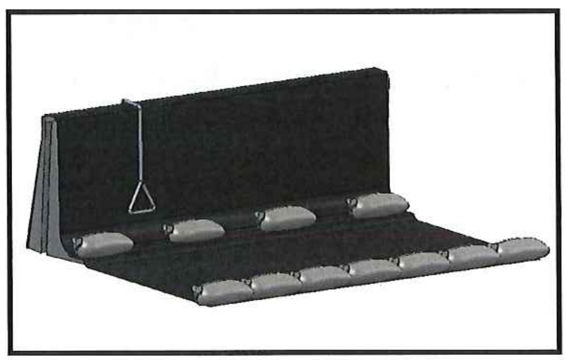
Once a 10 foot section of foam has been applied lay the liner on top of the foam and roll the sandbags over so they are on top of the liner right over where the foam is underneath. Ensure that the sandbags are hanging over the edge of the liner by a 1-2 inches. Walk on the sandbags to compress them down and to strengthen the seal of the foam.

Step 5

Start right where you left off and spray another 10 foot section of foam, lay the liner on top of the foam, and then lay the sandbags on the edge of the liner. Continue across the whole wall.

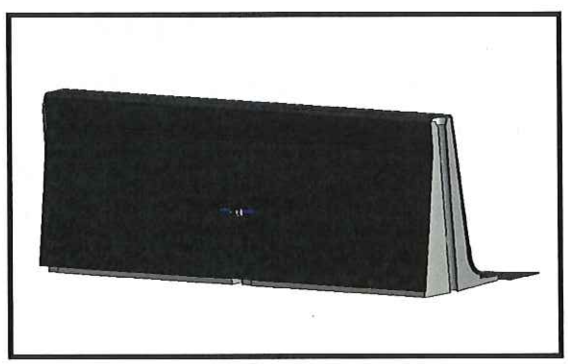
Step 6

Ensure that the liner is flat against the wall and place sandbags on the toe to stop the wind from picking up the liner. Place a steel clip every fourth wall to secure the liner to the Muscle Wall.



Step 7

Secure the liner by cutting a small horizontal slit in the liner and pulling the safety strap through and ratcheting it down. Only one of the straps is needed to secure the liner. It is preferable to use the bottom safety strap, but the top safety strap may be also be used if the liner does not reach the bottom one.



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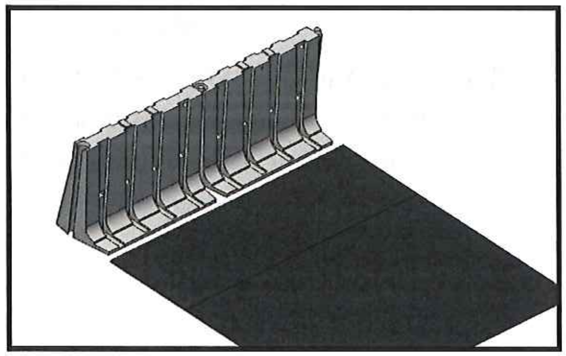
Liner Deployment
Deploying on Asphalt

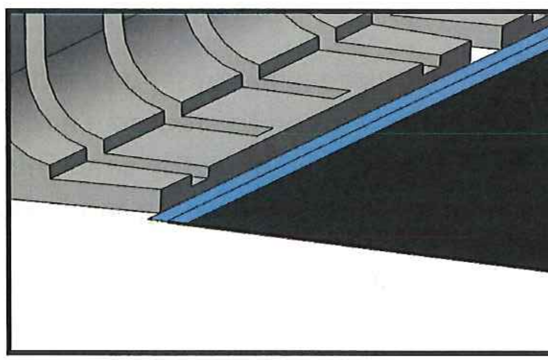
Sandbag & Sand Option
Step 1

Unfold the liner in front of the toe of the wall. Fold the edge of the liner closest to the toe back on top of itself approximately 2 ft. Then move the edge of the folded liner approximately 4 inches away from the toe of the wall.



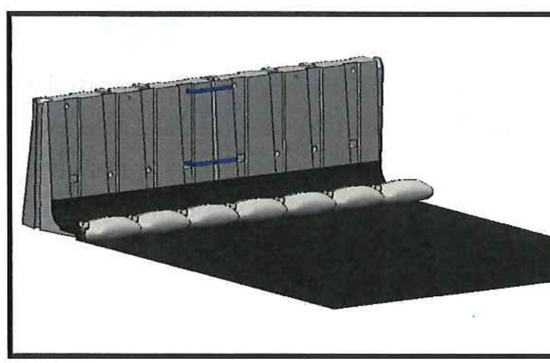
Step 2

Optional: Tape the edge of the liner down to the ground next to the toe of the wall. Once tape is down, walk along the tape to strengthen the seal.



Step 3 Option A

Pull back the 2 ft. of liner that was folded over earlier and set one row of sandbags tightly along the toe. If using tape make sure the sand bags are placed directly on top of the line of tape.



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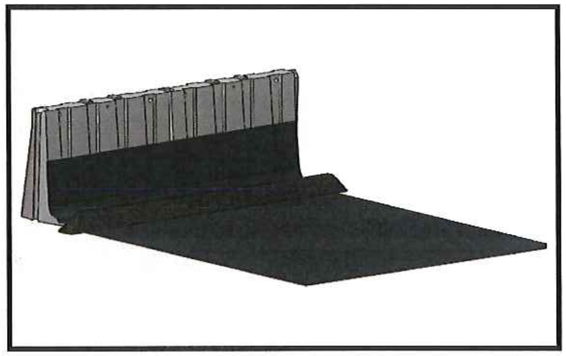
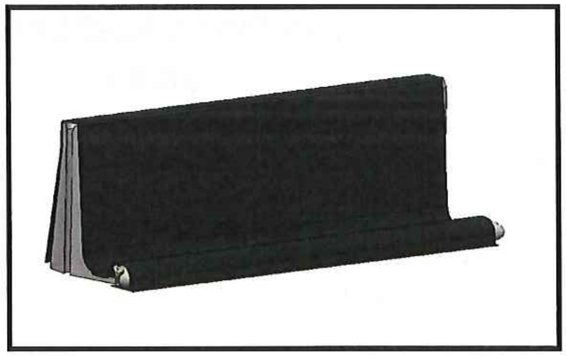
Step 3 Option B

If sandbags are not available or desired, sand may be placed between the liner fold. If using tape make sure the sand is placed directly on top of the line of tape.



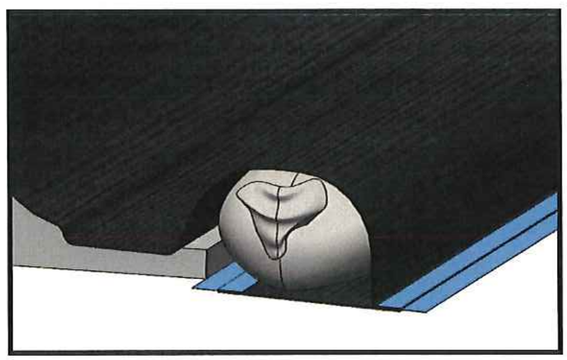
Step 4

Pull liner up and over the wall. Do not pull liner too tight as this may cause tenting which may lead to rips or tears. Once liner is over, walk on the sand or sand-bags to compress them down.



Step 5

Optional: Place a line of tape sealing the front of the liner to the ground to create an additional seal. Walk on the tape to enhance the seal. This same method can be used if you are using sand instead of sand-bags.



Step 6

Ensure that the liner is flat against the wall and place sandbags on the toe to stop the wind from picking up the liner. Place a steel clip every fourth wall to secure the liner to the Muscle Wall.

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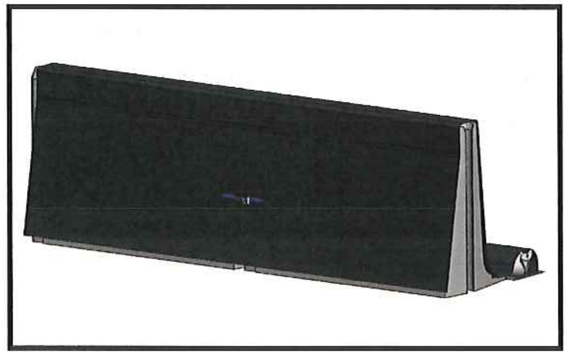
Step 7

Secure the liner by cutting a small horizontal slit in the liner and pulling the safety strap through and ratcheting it down. Only one of the straps is needed to secure the liner. It is preferable to use the bottom safety strap, but the top safety strap may be also be used if the liner does not reach the bottom one.

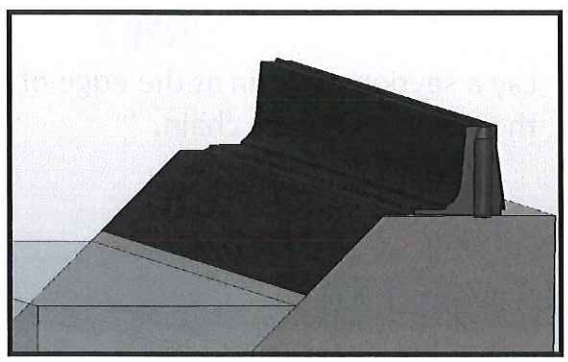


Liner Deployment

Deploying on a Levee

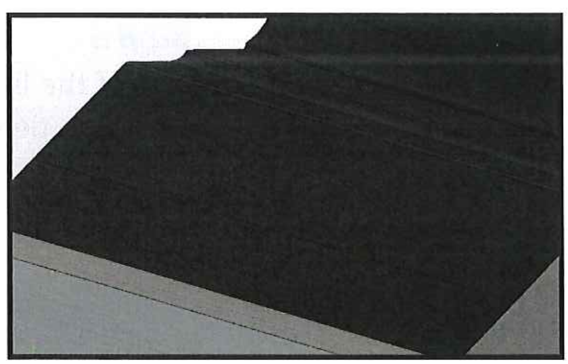
Stake Option
Step 1

Roll out, unfold, and place the liner over the Muscle Wall. Ensure that at least 3 feet of the liner is on the downward slope of the levee.



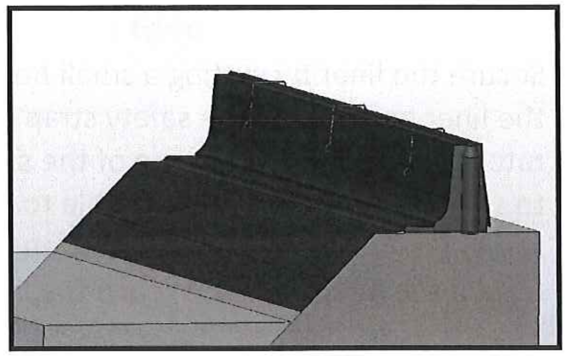
Step 2

Place a row of lawn stakes approximately 6in-1ft apart at the edge of the liner. Place another row of lawn stakes 1 foot higher, approximately 2-3 feet apart.



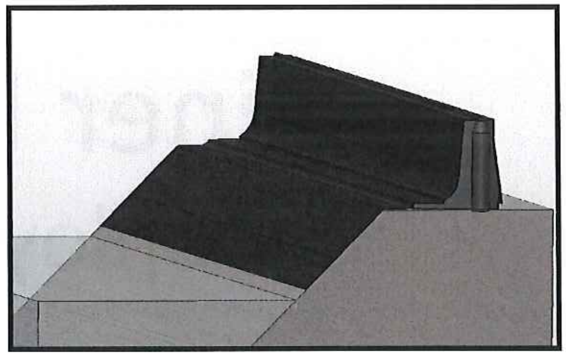
Step 3

Place 1 liner clip on every unit of Muscle Wall to secure the liner to the wall.



Chain Option
Step 1

Roll out, unfold, and place the liner over the Muscle Wall. Ensure that at least 3 feet of the liner is on the downward slope of the levee.



Step 2

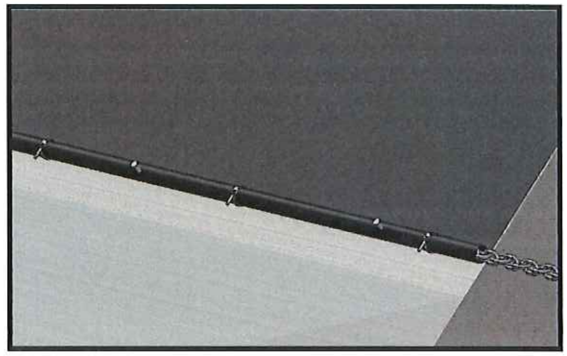
Lay a section of chain at the edge of the liner and role the liner around the chain.

Step 3

Put a small hole in the edge of the liner approximately every foot and zip tie the liner around the chain.

Step 4

Drive a stake through a link in the chain and into the ground approximately every 1 foot to insure that the chain stays in place.

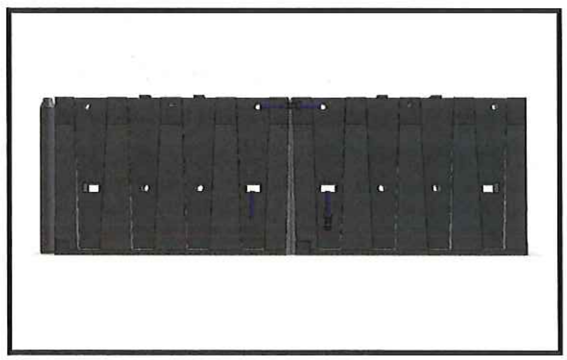


Stage 3

Takedown & Consolidation

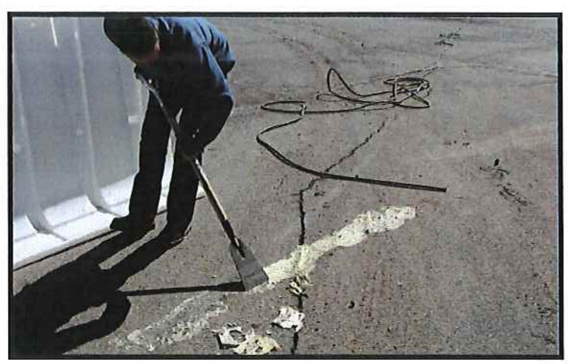
Step 1

Disconnect straps, roll up, and place in a storage container. Remove liner from the wall and fold for future installations. Properly dispose of any contaminated sand and/or sand bags.



Step 2

If foam was used to secure the liner to asphalt, scrape up as much of the foam as you can, sweep up, and dispose of.



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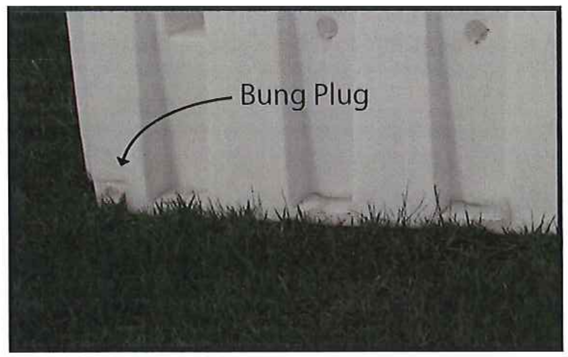
Step 3

If foam was used to secure the liner to asphalt, use a pressure washer to clean up the rest of the residue from the foam.

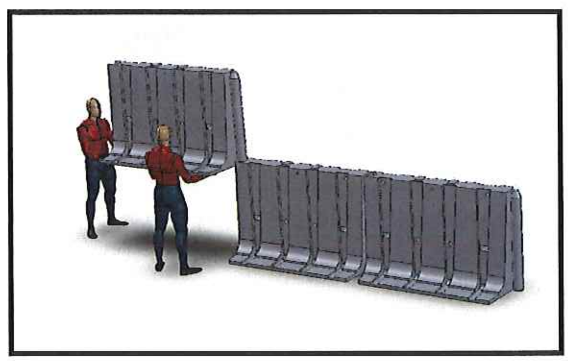
Step 4

Empty the Muscle Wall by using the bung wrench to unscrew the bung plugs on the backside of each wall. Place bung plugs in a secure location where they wont be lost or broken. The trash pump can also be used to pump the water out of the walls.



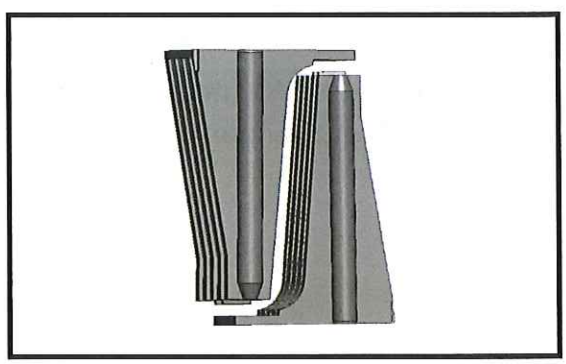
Step 5

Disconnect each Muscle Wall. One person on each side lifts the wall up until the wall is free of its connection.



Step 6

Stack the walls together in the same fashion as they were shipped. Turn one of the walls upside down and lay it on another wall ensuring that the pegs on top of each Muscle Wall are secured into the holes on the toe of the other wall.



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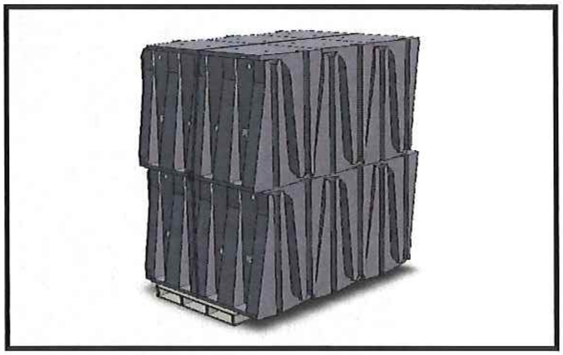
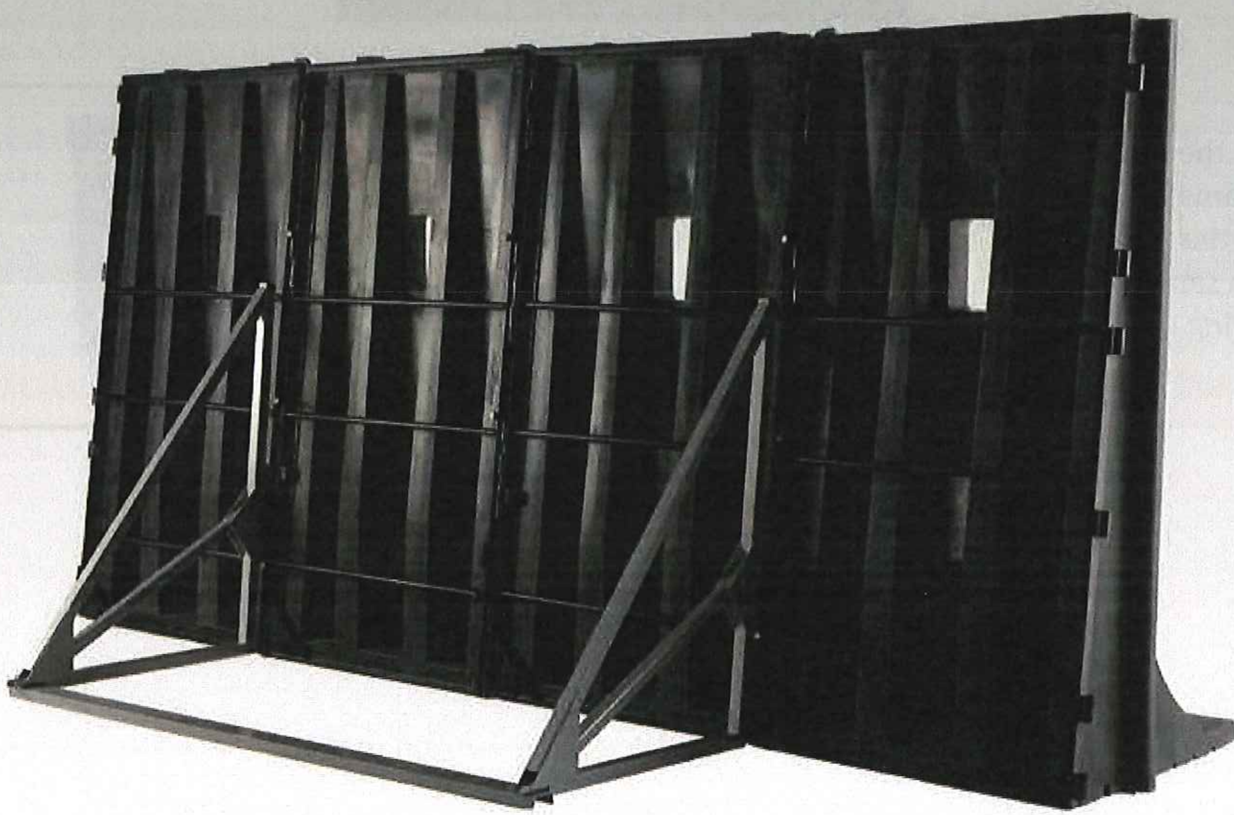
Step 7

Stack the Muscle Wall back on their original pallets in the same fashion as they were shipped. Ensure that all of the male ends of the walls are on the same side. Run a safety strap all the way around the bundle securing it to the pallet. Transport to storage location.





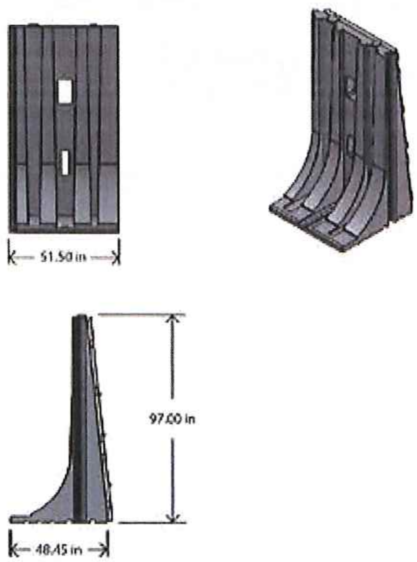
MUSCLE WALL®

Specifications:

- Material
  - Low density polyethylene
  - Elongation to yield: 20%
  - Impact strength: 190 ft-lb
  - Tensile strength at yield: 2600 psi
  - Polyurethane foam core density: 3.0lb/ft^3
  - Thermal insulation value: R=38.18(hr-ft^2-°F/
- All Season Compatible
  - Temperature range: -40° F to 180° F
  - 10 year UV rated
- Ground Pressure: 0.159 psi
- Portable
  - Weight per unit: 350lbs
  - Units nest together for transportation
- Dimensions
  - Minimum polyethylene thickness: 0.25"
  - Footprint on ground: 15.286 ft^2
  - 4 ft. wide x 4 ft. deep x 8 ft. high
  - Installed in 4 ft. sections
  - Fit 8 units for every 8 feet of trailer

8 Foot System

MUSCLE WALL®



Features:

- Tongue and groove panel interface
- Rugged 1" diameter steel pins at segment interface
- Corner piece allows 90° turns
- Units are easily stacked together for storage or shipping
- Powder coated, steel reinforced frame and support system
- Patent Protected
  - US 8.313.265 B2
  - USD 631977
  - US 634443

Contact us:

Please contact us with any questions you have regarding Muscle Wall and how we can help you with your water management needs.

Email – info@musclewall.com
Toll-free – 1.800. 801.8739
Fax – 435.514.6707
Mail – 675 North 600 West Suite 1
Logan, UT 84321
Web – www.musclewall.com

MUSCLE WALL®

Standard Operating Procedures
8ft Muscle Wall

Flood & Stormwater Management

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www.musclewall.com

Table of Contents

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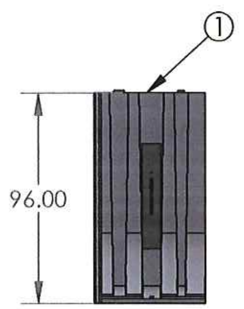
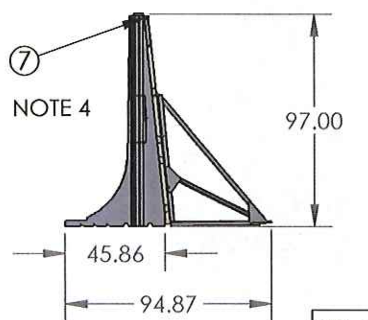
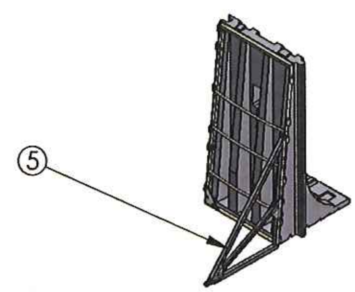
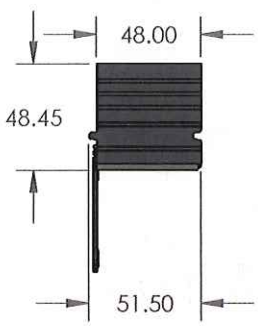
Specifications

EIGHT FOOT WALL DIMENSIONS









ITEM NO.	PART NUMBER	DESCRIPTION
1	MW-848	EIGHT FOOT STRAIGHT WALL
2	i04328_01.8FT_BCK_BRC	EIGHT FOOT INSIDE CORNER
3	i04394_01.TI_BR	EIGHT FOOT OUTSIDE CORNER
4	MW-860	BACK BRACE ASSEMBLY
5	MW-860.01	BACK BRACE A-FRAME
6	MW-860.02	BACK BRACE CROSS SUPPORT
7	MW-860.04	CONNECTOR PIN
8	MW-870	LINER CLIP

NOTES:

1. INTERPRET PER ASME Y14.41-2003. THIS DRAWING SHALL BE USED WITH MODEL OF THE SAME NAME FOR COMPLETE PRODUCT DEFINITION.
2. EACH LAYOUT CONTAINS TWO WALL SEGMENTS.
3. RECOMMENDED LINER WIDTH FOR ALL EIGHT FOOT WALL LAYOUTS IS 24 FEET.
4. PIN IS IN ALL CONNECTIONS (4 PER WALL CONNECTION)

PROPRIETARY

UNLESS OTHERWISE SPECIFIED DIMENSIONS ARE IN INCHES
TOLERANCES ARE:
FRACTIONS DECIMALS ANGLES
X.X± 0.2 IN X ± 2.0
X.XX± 0.020 IN X.X ± 0.5
X.XXX ± 0.005 IN X.XX±0.1

MATERIAL N/A
FINISH N/A
DO NOT SCALE DRAWING

APPROVALS	DATE
DRAWN KYLE M	11/18/2013
CHECKED	
COG ENG	
QA	
RELEASED	

MUSCLE WALL®
FLOOD AND CONTAINMENT SOLUTIONS

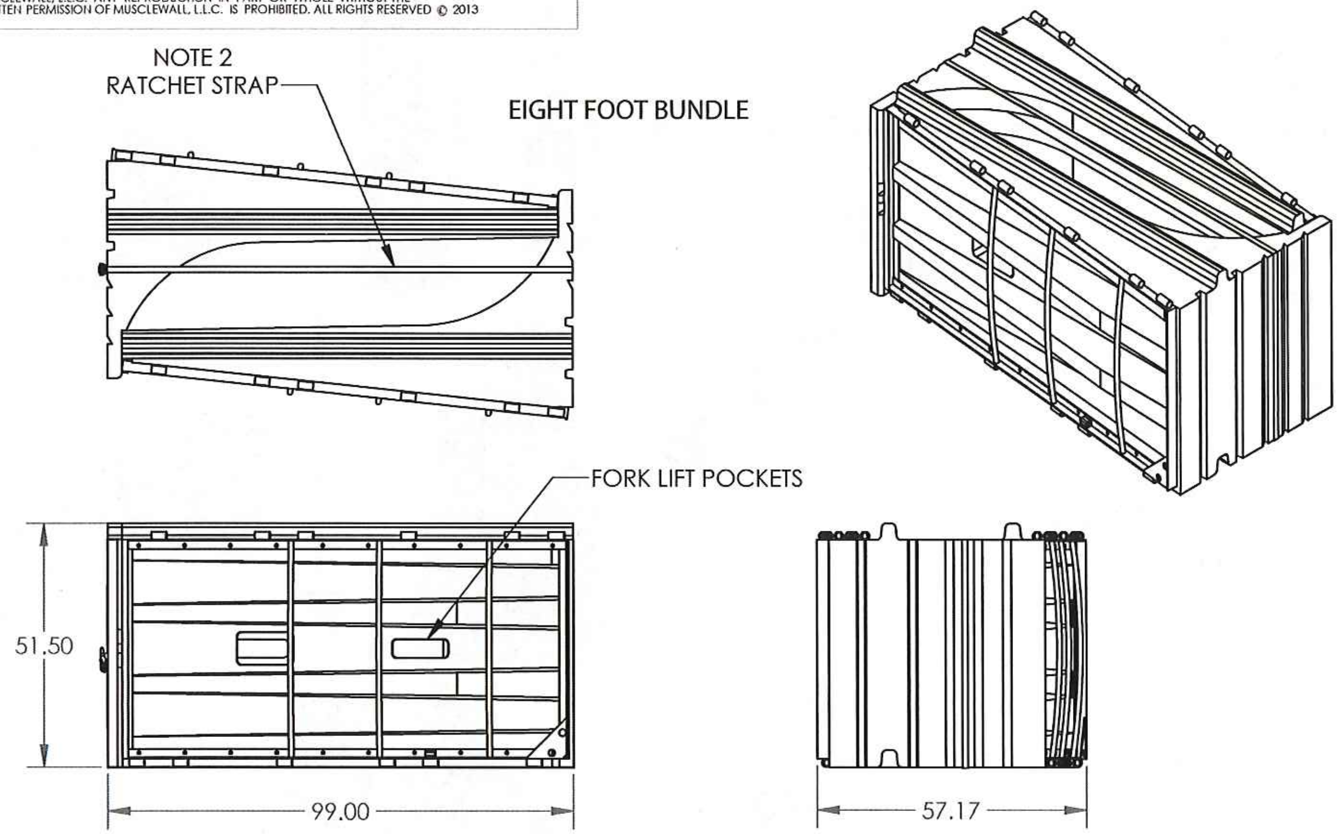
CAGE CODE 54FP9

LAYOUTS, EIGHT FOOT WALL

SIZE A DWG. NO. MWD-00003_R03

SCALE 1:64 SHEET 1 OF 13

Specifications



NOTES:

1. EIGHT FOOT BUNDLE CONSISTS OF 2 WALLS.
2. RATCHET STRAP SHOULD WRAP AROUND BOTH WALLS LENGTHWISE.
3. NO PALLET USED IN THIS BUNDLE.
4. EACH BUNDLE WEIGHS 700 LBS.

PROPRIETARY

DO NOT SCALE DRAWING

MUSCLE WALL®
FLOOD AND CONTAINMENT SOLUTIONS

CAGE CODE	54FP9
8 FT TRAILER LOADING	
SIZE A	DWG. NO. MWD-00017_R01
SCALE 1:30	SHEET 2 OF 3

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Items Needed

- Shovel
- 200ft Measuring Tape
- Marking Paint
- 2 Sledge Hammers
- 10 Sand Bags for every Section of Muscle Wall
- Gloves
- 2 Razor Knives
- 500-1000ft of String
- Trash Pump
- Fork Lift
- Lifting Dolly
- Gorilla Tape
- Pressure Washer
- "Great Stuff" Foam Sealant
- Trencher












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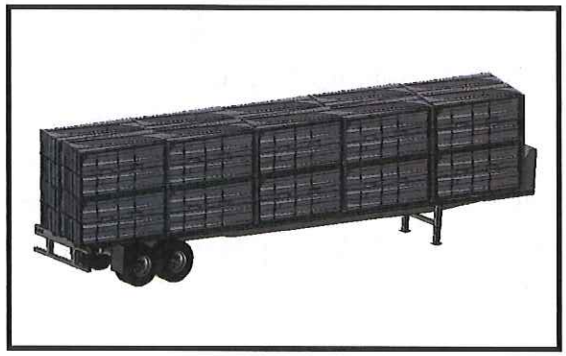
Stage 1

Muscle Wall Deployment

Step 1
Use the forklift to unload the Muscle Wall from the trailer. Strategically place the bundles around the location of deployment for a quicker deployment.



Step 2
Once the bundles are off of the trailer, take off the safety strap and separate the walls to enable the hand truck to get under each individual wall.

Step 3
Transport the Muscle Wall using the hand truck as pictured to the right. One person will operate the hand truck and another will help stabilize the wall while moving. Exercise extreme caution while moving Muscle Wall because each wall weighs approximately 350lbs.

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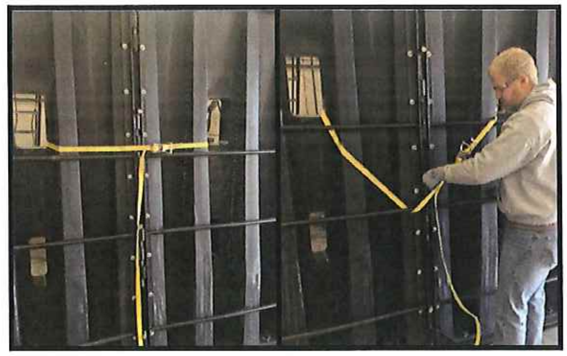
Step 4

Once each wall is placed, secure the connection by pushing the wall tight against its connecting wall so that the tongue and grove are connected, and then installing the included metal pins into the hinges of the metal frames. The corners are installed in the same fashion.

Step 5

When installing the pins it may be necessary to push or pivot the wall to insert them if the ground is uneven. The safety straps that were used to secure the bundles may also be used, as pictured to the right, to help line up the hinges so the pins will go through.



Step 6

Make sure that all of the top pins are facing sideways, perpendicular with the wall. If the top pins are facing forward or backward they may tear the liner.

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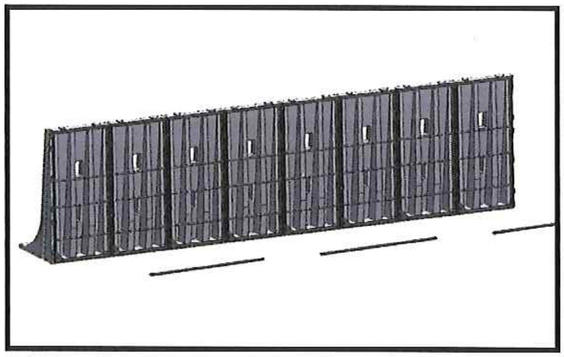
Stage 2

Support Frame Deployment

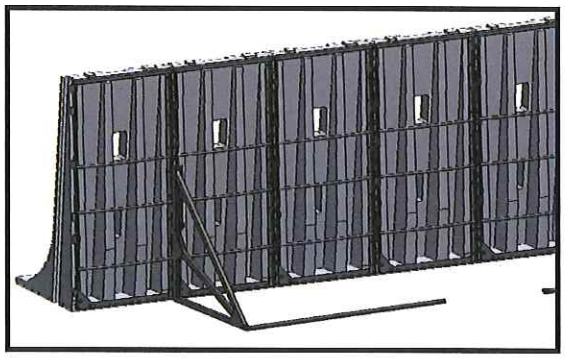
Step 1

Beginning with the second wall from a corner, place an 8 foot channel on the ground parallel to the wall about 4 feet from the wall. Skip one wall and then lay another 8 foot channel. Continue doing so along the whole system.



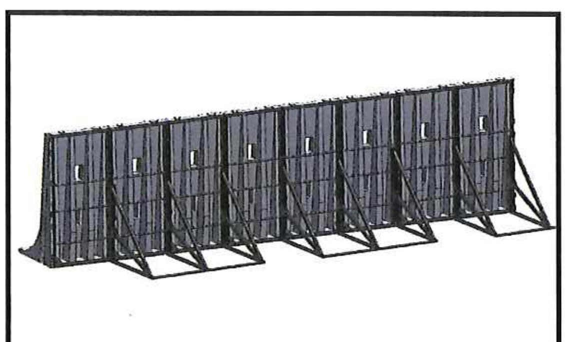
Step 2

Connect the angle brace to the containment by attaching the hinge system of the brace to the hinge on the right side of the back of the Muscle Wall. Connect the brace to the 8ft channel by inserting the peg on the channel into the hole of the brace and inserting the 1 inch pin through the peg.



Step 3

Connect all of the remaining braces along the whole system.



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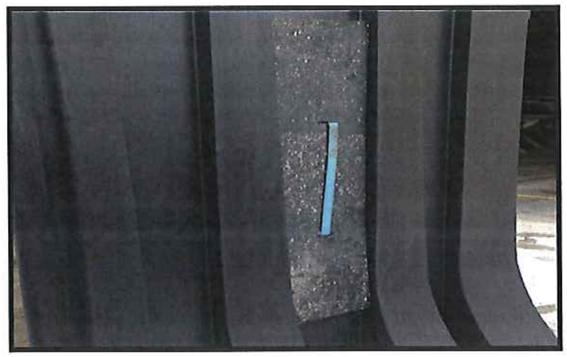
Step 4

Install the forklift hole shields by placing them over the forklift holes on the side with the toe and then attaching and tightening the strap on the back of the wall.



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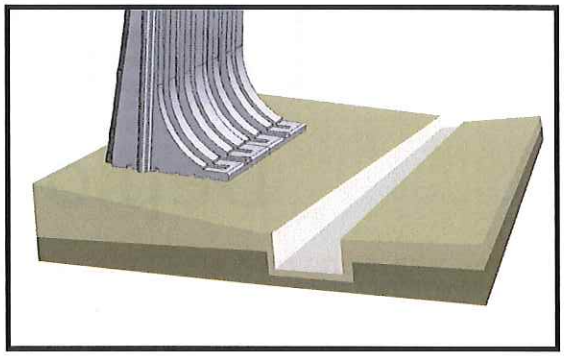
Stage 3

Liner Deployment

Deploying on Soil

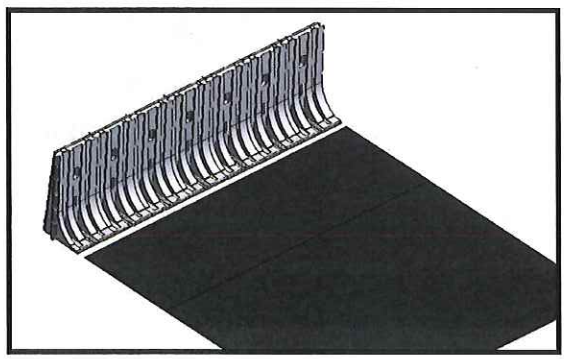
Trench Option
Step 1

About 3 feet away from the toe of the Muscle Wall dig a trench approximately 16 inches deep that spans the entire wall of Muscle Wall.



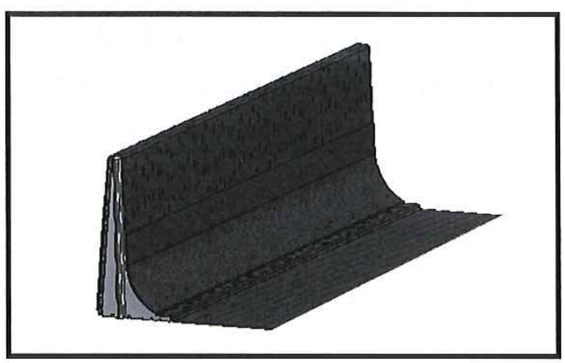
Step 2

Roll out and unfold the liner in front of the toe of the Muscle Wall, placing about 16-20 inches of the edge of the liner in the trench, and fill the trench back in with soil. Compact the soil as much as possible.



Step 3

Pull liner up and over the wall. Do not pull liner too tight as this may cause tenting which may lead to rips or tears.



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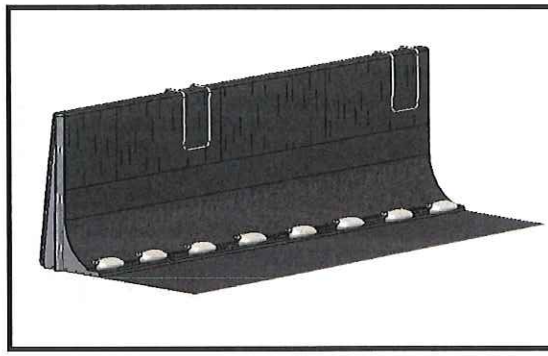
Step 4

Ensure that the liner is flat against the wall and place sandbags on the toe to stop the wind from picking up the liner. Place a steel clip every fourth wall to secure the liner to the Muscle Wall.



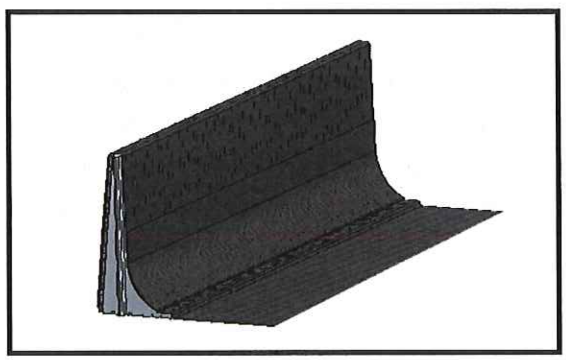
Liner Deployment

Deploying on Asphalt

When deploying on asphalt it is required for the back of the bracing system to butt up against a curb, retaining wall, or something of the sort.

Foam Option
Step 1

Roll out and unfold the liner. Pull it up and over the Muscle Wall with approximately 3 feet of the liner flat on the ground in front of the toe of the Muscle Wall.



Step 2

Lay a line of sandbags back to back on top of the liner approximately 1 foot away from the edge of the liner.

Step 3

Lift the edge of the liner up and spray a liberal amount of foam on the ground under the liner. Ensure that there is enough foam to bubble out from underneath the liner when you lay the liner flat on the ground.

Step 4

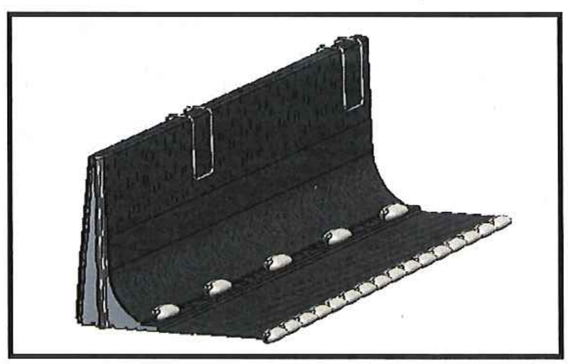
Once a 10 foot section of foam has been applied lay the liner on top of the foam and roll the sandbags over so they are on top of the liner right over where the foam is underneath. Ensure that the sandbags are hanging over the edge of the liner by a 1-2 inches. Walk on the sandbags to compress them down and to strengthen the seal of the foam.

Step 5

Start right where you left off and spray another 10 foot section of foam, lay the liner on top of the foam, and then lay the sandbags on the edge of the liner. Continue across the whole wall.

Step 6

Ensure that the liner is flat against the wall and place sandbags on the toe to stop the wind from picking up the liner. Place a steel clip every fourth wall to secure the liner to the Muscle Wall.



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Liner Deployment

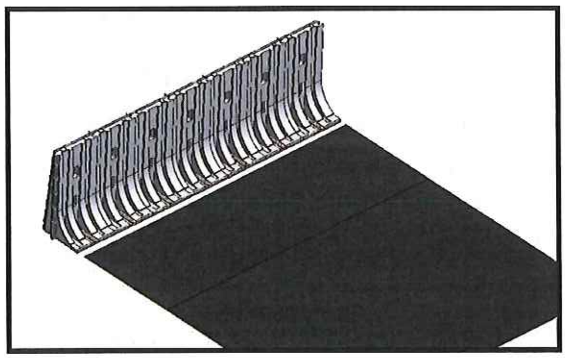
Deploying on Asphalt

When deploying on asphalt it is required for the back of the bracing system to butt up against a curb, retaining wall, or something of the sort.

Sandbag & Sand Option

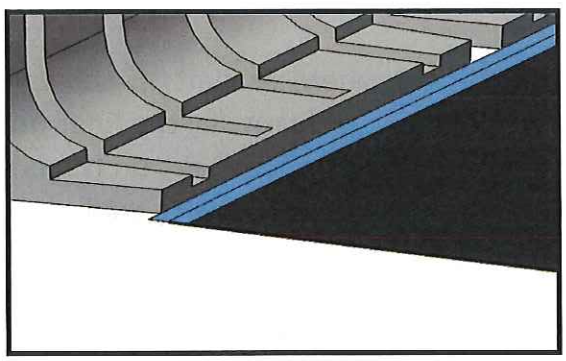
Step 1

Unfold the liner in front of the toe of the wall. Fold the edge of the liner closest to the toe back on top of itself approximately 2 ft. Then move the edge of the folded liner approximately 4 inches away from the toe of the wall.



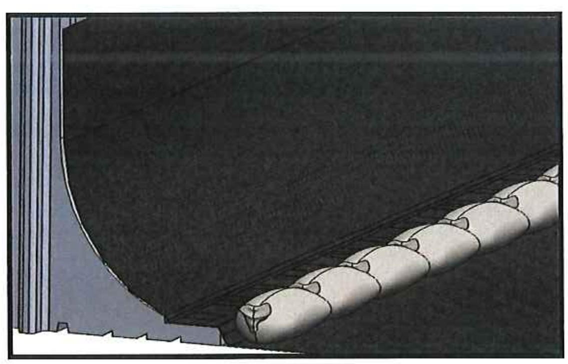
Step 2

Optional: Tape the edge of the liner down to the ground next to the toe of the wall. Once tape is down, walk along the tape to strengthen the seal.



Step 3A

Pull back the 2 ft. of liner that was folded over earlier and set one row of sandbags tightly along the toe. If using tape make sure the sand bags are placed directly on top of the line of tape.



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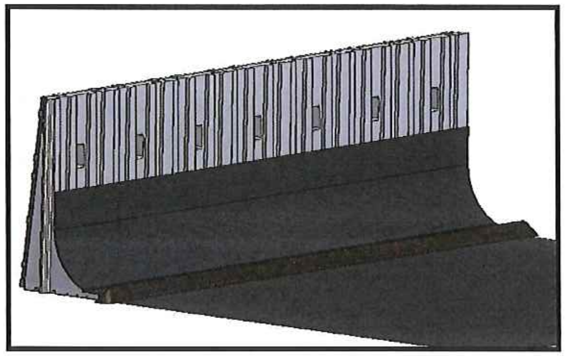
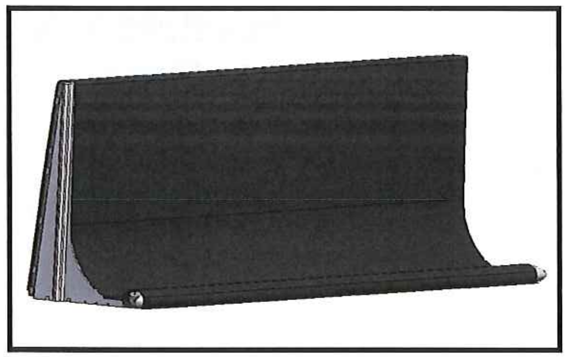
Step 3B

If sandbags are not available or desired, sand may be placed between the liner fold. If using tape make sure the sand is placed directly on top of the line of tape.



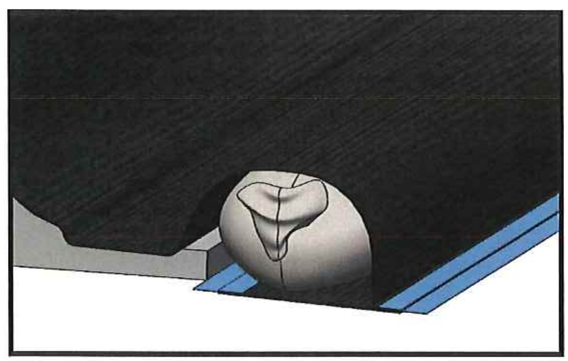
Step 4

Pull liner up and over the wall. Do not pull liner too tight as this may cause tenting which may lead to rips or tears. Once liner is over, walk on the sand or sand-bags to compress them down.



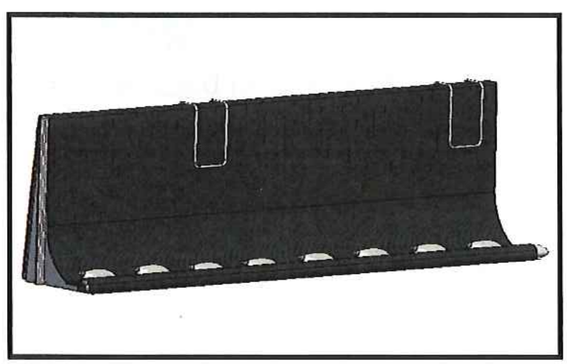
Step 5

Optional: Place a line of tape sealing the front of the liner to the ground to create an additional seal. Walk on the tape to enhance the seal. This same method can be used if you are using sand instead of sandbags.



Step 6

Place additional sand bags every few feet on top of the liner to prevent wind getting under the liner. Install a steel clip over each fourth Muscle Wall to secure the liner to the wall.



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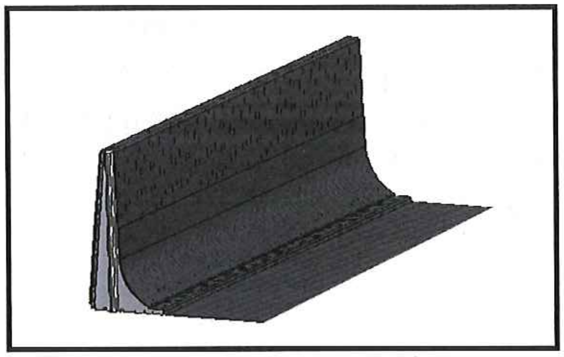
Liner Deployment

Deploying on Grass

Lawn Stake Option

Step 1

Roll out and unfold the liner. Pull it up and over the Muscle Wall with approximately 3-4 feet of the liner flat on the ground in front of the toe of the Muscle Wall.

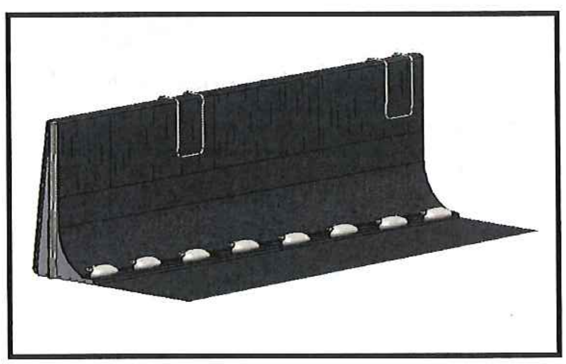


Step 2

Approximately once every linear foot secure the liner down to the ground by placing a yard staple.

Step 3

Place additional sand bags every few feet on top of the liner to prevent wind getting under the liner. Install a steel clip over each fourth Muscle Wall to secure the liner to the wall.

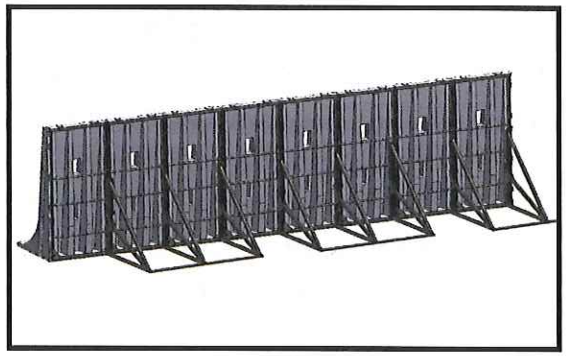


Stage 4

Takedown & Consolidation

Step 1

Remove steel clips and properly dispose of any contaminated sand and/or sand bags. Remove liner from the wall and fold for future installations.



Step 2

If foam was used to secure the liner to asphalt, scrape up as much of the foam as you can, sweep up, and dispose of.

Step 3

If foam was used to secure the liner to asphalt, use a pressure washer to clean up the rest of the residue from the foam.

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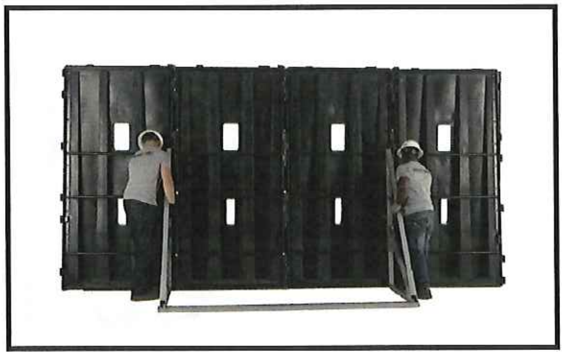
Step 4

Disconnect the support frame from the Muscle Wall system. Remove all of the pins connecting the walls together. Place all of the pins from the Muscle Wall, as well as from the support frame, in a secure container to prevent loosing any.



Step 5

Stack the walls together in the same fashion as they were shipped. Lay both walls on their side and slide them together so the pegs on the top of the wall fit into place on the toe of the connecting wall. Secure the walls together by running the strap around the bundle and tightening it.

Step 6

Use a forklift to transport the bundles back to a trailer or to a storage location.

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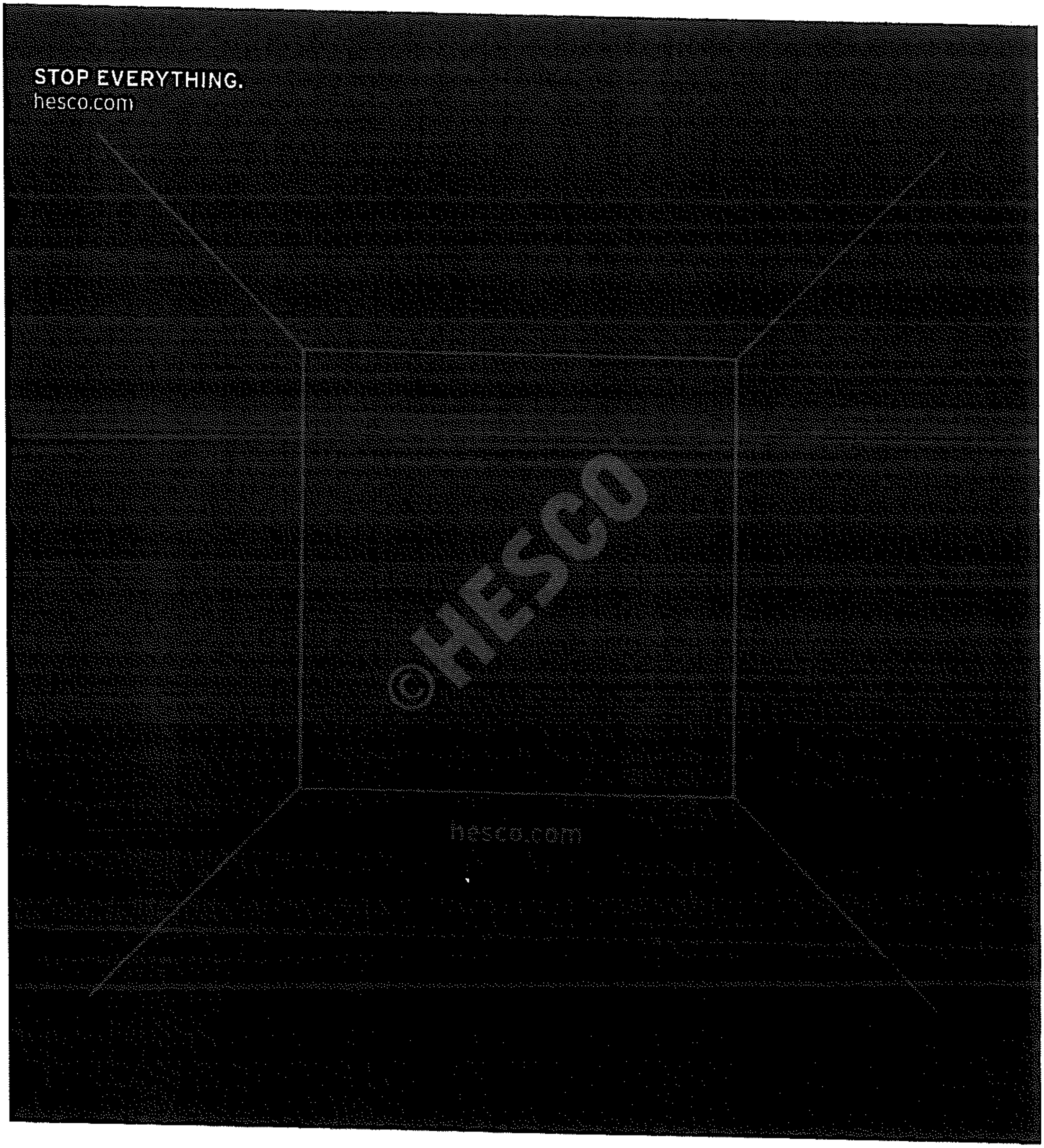
HESCO Barriers Temporary Flood Protection Alternative

RAPIDLY
DEPLOYABLE
FLOOD BARRIERS

HESCO


STOP EVERYTHING.
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HESCO Flood and Environmental Barriers are proven to protect against some of the harshest environments. When faced with tropical storms and seasonal flooding they will defend critical infrastructure, protect homes and secure assets.

RAPID RESPONSE FOR EMERGENCY FLOOD DEFENSE

Whether the increase in water level is gradual or due to a sudden surge caused by storms, HESCO Flood Barriers are rapidly deployable, quick and easy to construct without the need for specialized skills or equipment. They are a flexible and efficient solution for virtually any emergency flood defense

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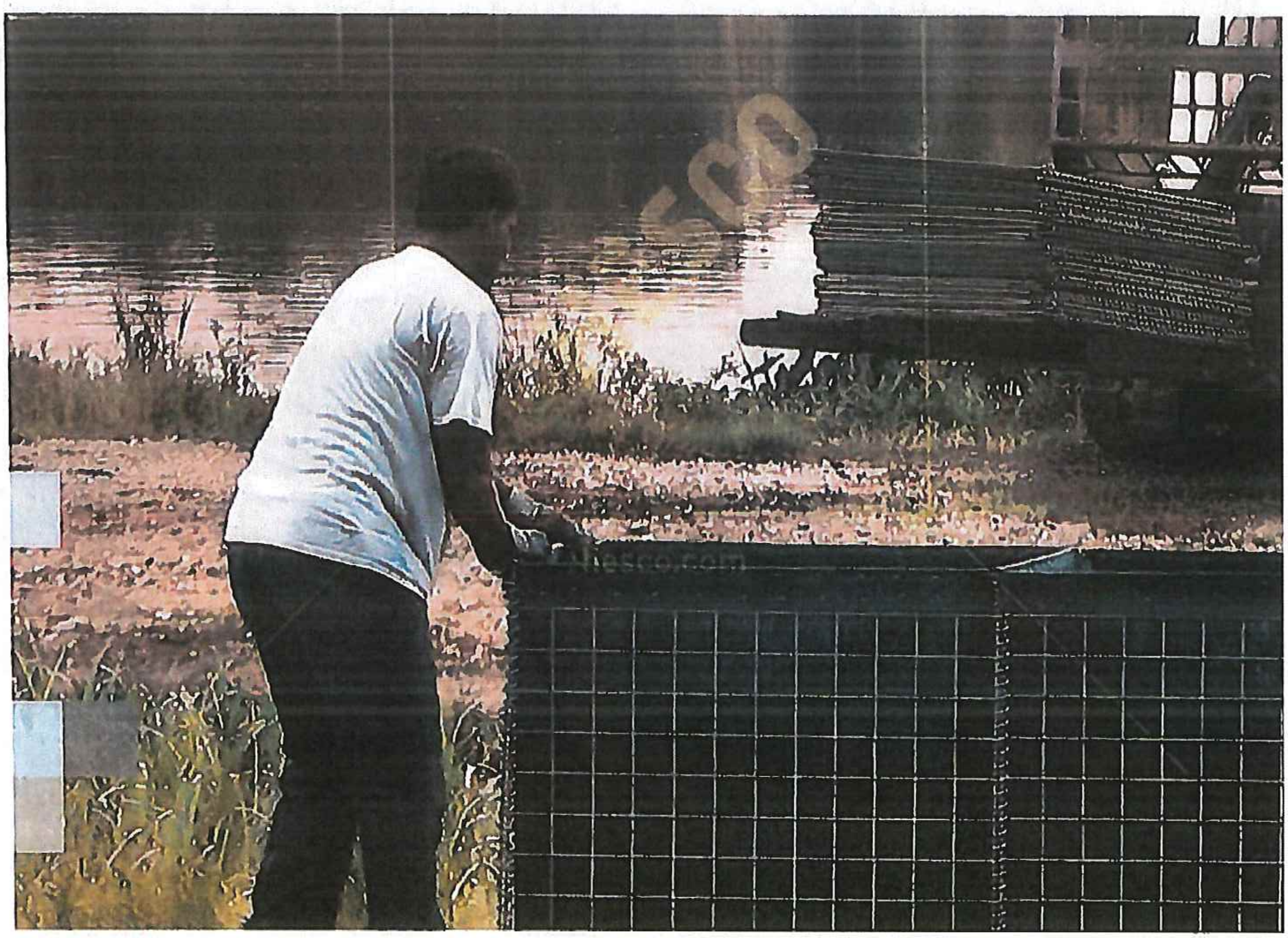
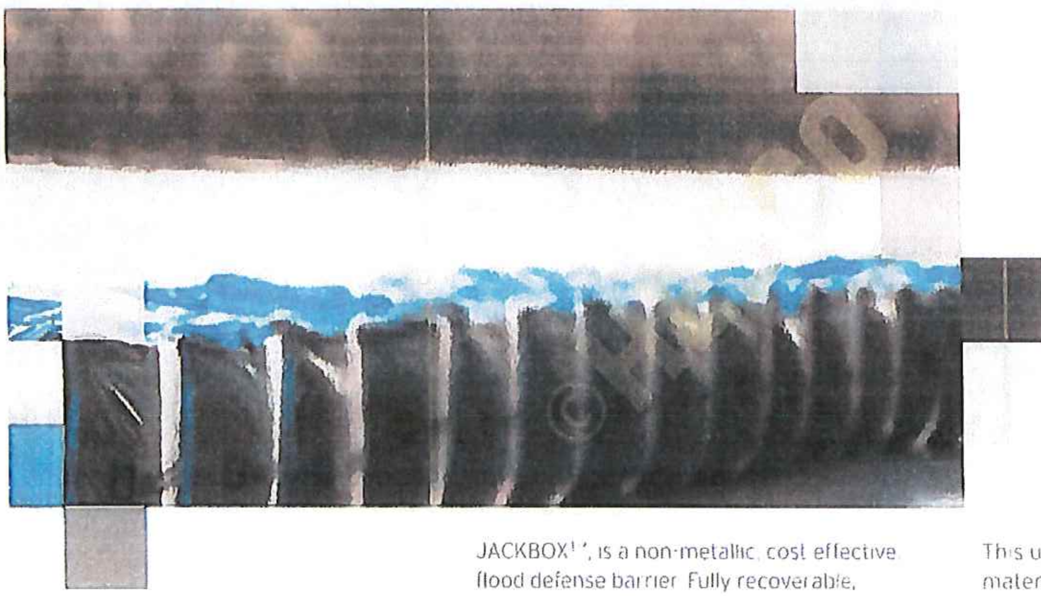
BAM Nuttall Construction, 2016




JACKBOX
JACKBOX
JACKBOX
JACKBOX

INTRODUCING JACKBOX™ THE LIGHTWEIGHT, RECOVERABLE FLOOD BARRIER

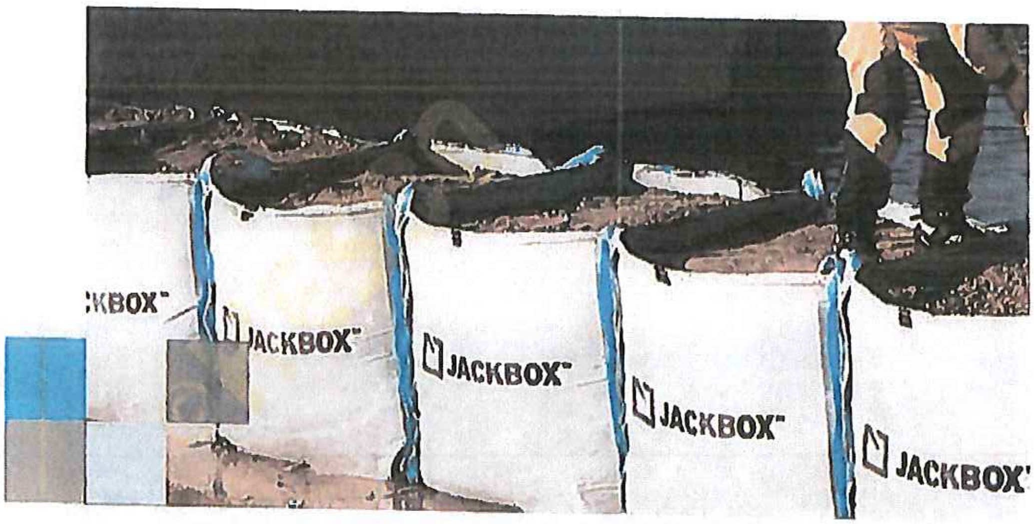


JACKBOX™, is a non-metallic, cost effective flood defense barrier. Fully recoverable, recyclable and lightweight, JACKBOX is engineered for ease of use, to hold back rising flood water.

The integrated base allows filled cells to be lifted away individually by mechanical handling equipment, allowing rapid recovery and minimal impact on the surroundings.

This unique, patented combination of materials and design makes JACKBOX the future of surface mounted, rapidly deployable and rapidly recoverable barrier systems.

WE ENGINEER PRODUCTS THAT INSPIRE CONFIDENCE



Tested at the US Corps of Engineers Research and Development Center, Vicksburg, Mississippi, USA. JACKBOX™ units have successfully undergone test and simulation against rising flood and floating debris.

JACKBOX™ moves beyond traditional gabion technology of wire mesh to provide a self supporting, light and flexible barrier; creating earth-filled units that protect against flood and storm events.

JACKBOX can be deployed easily by one person in under one minute, ready for fill material.

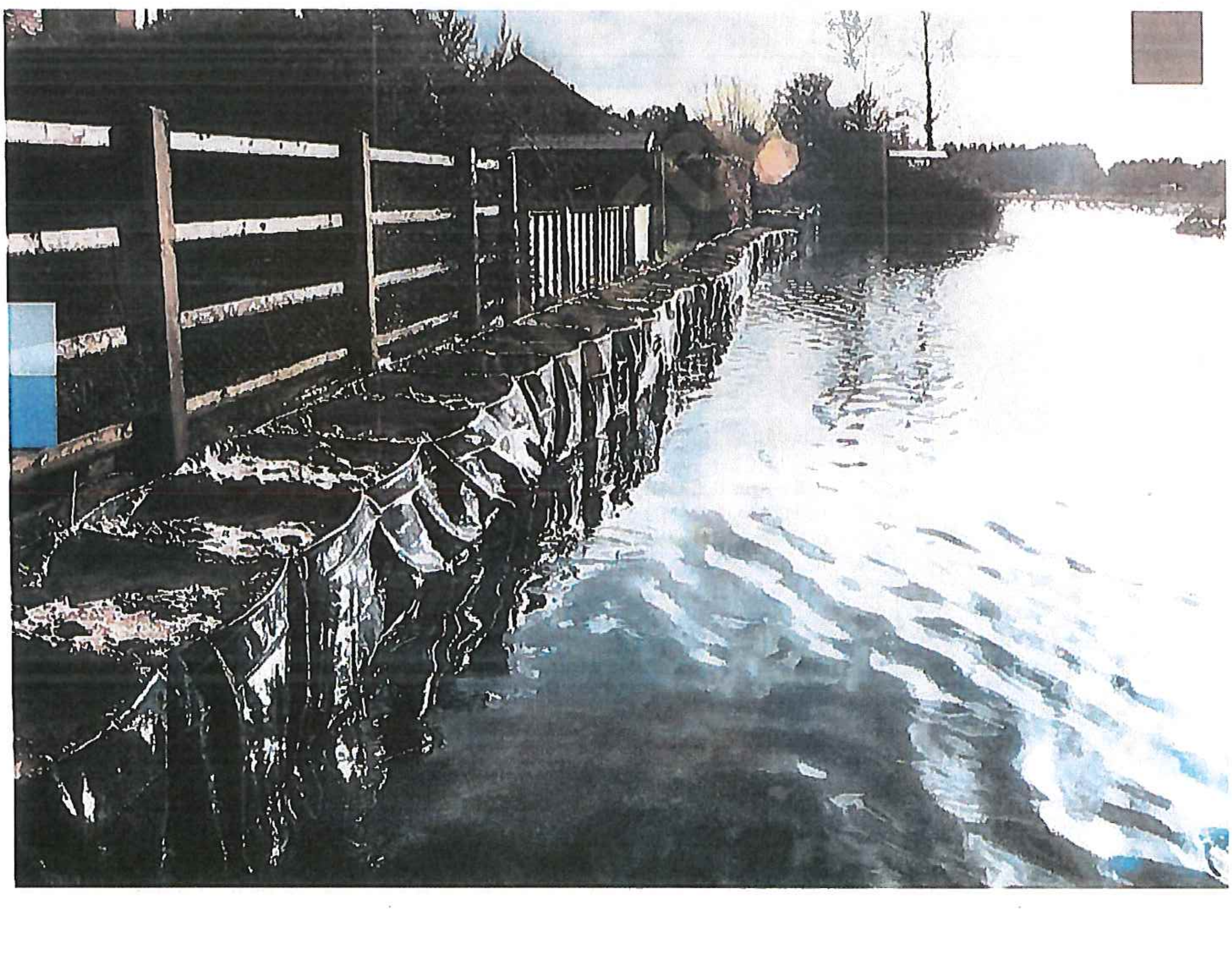
Engineered to be separated as individual cells during clean up after the flood event, JACKBOX enables the fill material to be disposed of with ease leaving no mess behind, and the plastic barrier to be recycled.

READY WHEN YOU ARE, RAPIDLY DEPLOYABLE

The design of HESCO units provides a multi cellular wall system that can be filled with almost any locally available soil or granular fill material

Individual cells are pre-assembled to create standard-length units that pack flat, to be unfolded on site and deployed easily by hand Units can then be joined on site to make structures of varying length to suit a wide range of profiles

The wire mesh and permeable fabric creates a cost effective, rapidly deployable and long lasting structure with minimal manpower and equipment

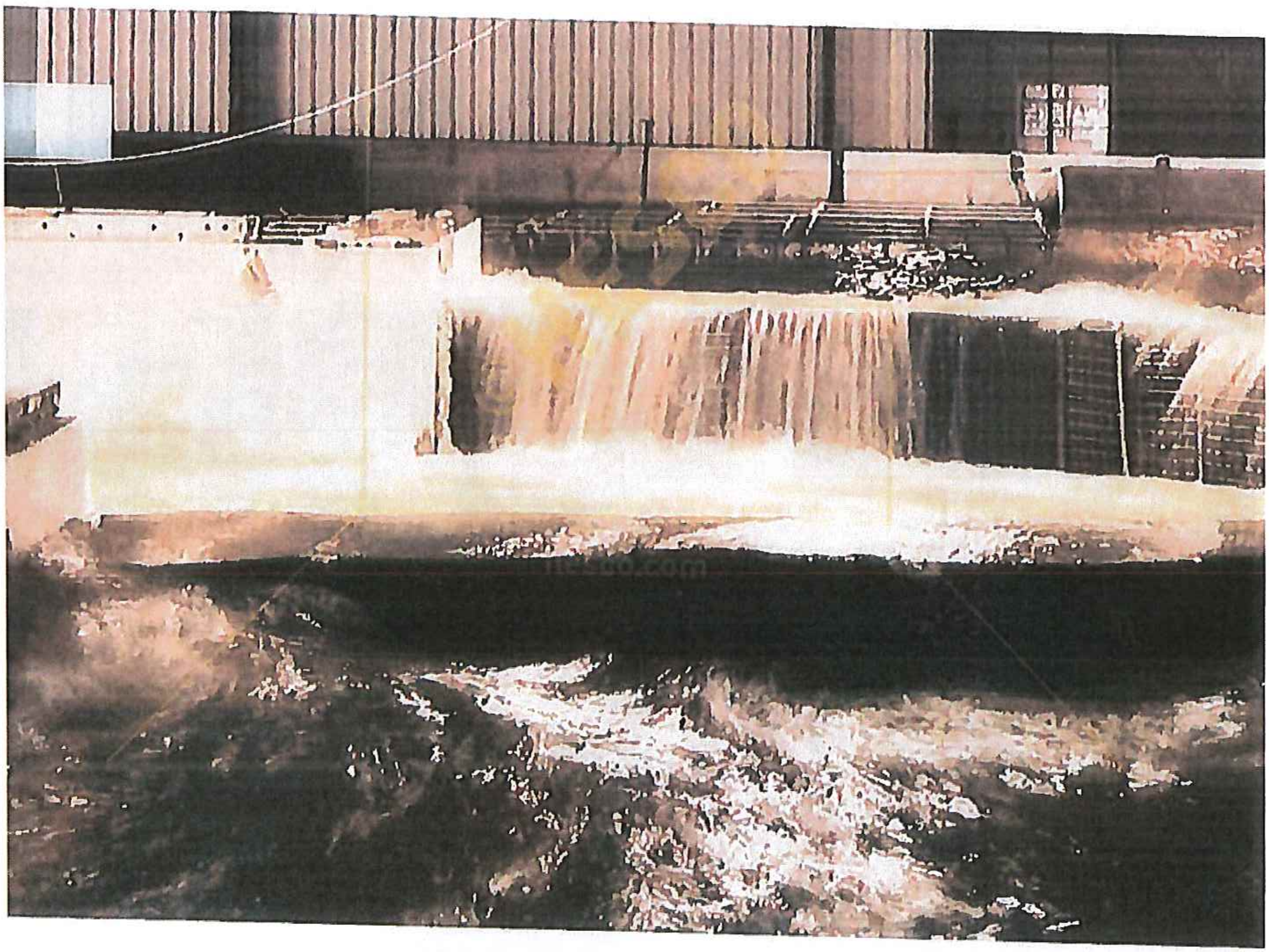


THE FORCES OF NATURE

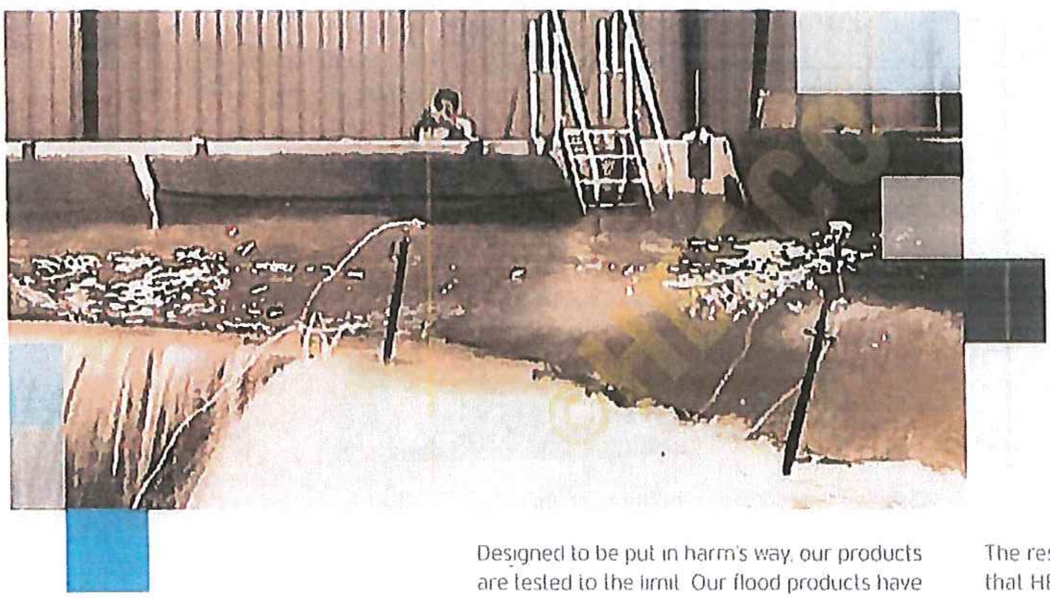
The design, strength and versatility of our flood barriers allow for corners, curves and angles to be constructed easily. There is no limit to the height or length of a HESCO protective wall, provided that space is available for sufficient base width.

Structures formed using HESCO products have significant advantages over traditional sandbags, not only by minimizing seepage and required footprint, but also reducing cost, time, and the manpower needed to deploy.

When a more permanent structure is required, such as on a river or canal bank, we have engineered units that can be planted with suitable grasses and other vegetation, forming a natural landscape, as well as a protective structure.



SETTING NEW STANDARDS TESTED TO THE LIMIT



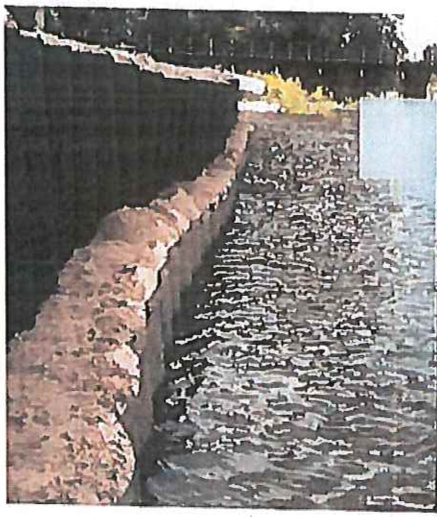
Designed to be put in harm's way, our products are tested to the limit. Our flood products have undergone an intensive series of laboratory and field tests and have been assessed by leading authorities in the US and worldwide.

Tests in the US include those undertaken as part of a research program into temporary flood-fighting structures. The research was conducted by the US Army Engineer Research and Development Center (ERDC).

The resulting report concluded that HESCO Concertainer units had significant advantages over traditional sandbags in terms of cost, time and labor requirement for installation, and also outperformed alternative systems in many areas tested.

Hesco continue to test new products to the same high standards and continue to test existing products against new threats and in new configurations.

PROVEN AROUND THE WORLD



Hesco's products are unrivaled through their use in the protection of communities, infrastructure, such as airports and hospitals, business premises, large and small, and vulnerable natural habitats, at risk from climatic events.

Hesco's wide product range defend against man-made as well as natural threats, to protect embassies, government buildings and public events from hostile attack.

"Very interesting to see how Hesco [illegible] rapidly and efficiently. I can see [illegible] length of [illegible]"

Joe Flores, JF Engineering
LA River deployment, 2016


HESCO

FASTER CLEAN UP WITH RECOVERABLE BARRIER UNITS

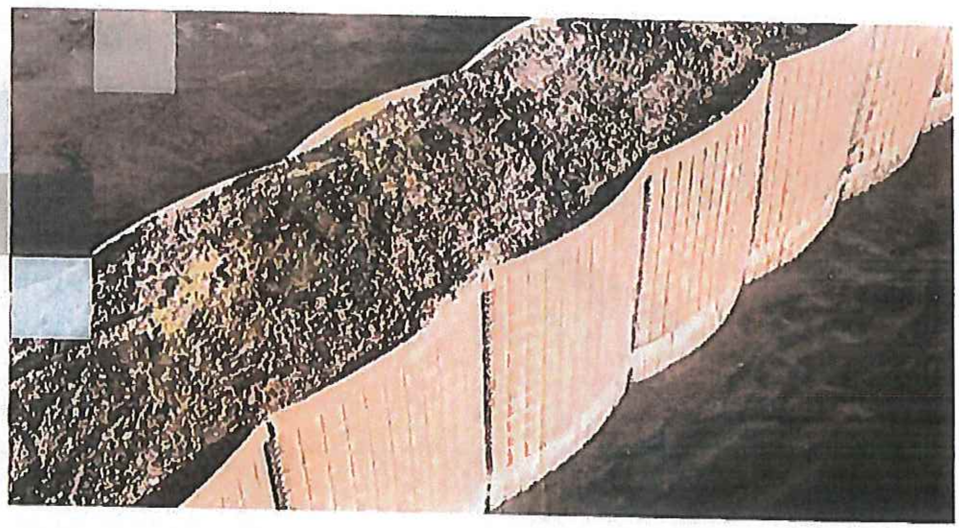
Responding to environmental responsibility and the rising costs of recovering material both during and at the end of an emergency, many of our units are now available in recoverable formats.

Recoverable units are deployed in exactly the same way as standard products. Once the requirement for flood and storm defenses is over, efficient recovery for disposal can begin.

The units can be recovered, flat packed and transported for recycling or disposal, providing substantial reduction in logistical and environmental impact.

Saving time, space and money, the volume of recoverable units occupies only twice its original volume when removed. It's as easy as pulling a pin, removing the fill material, and then folding up the barrier to be transported away.

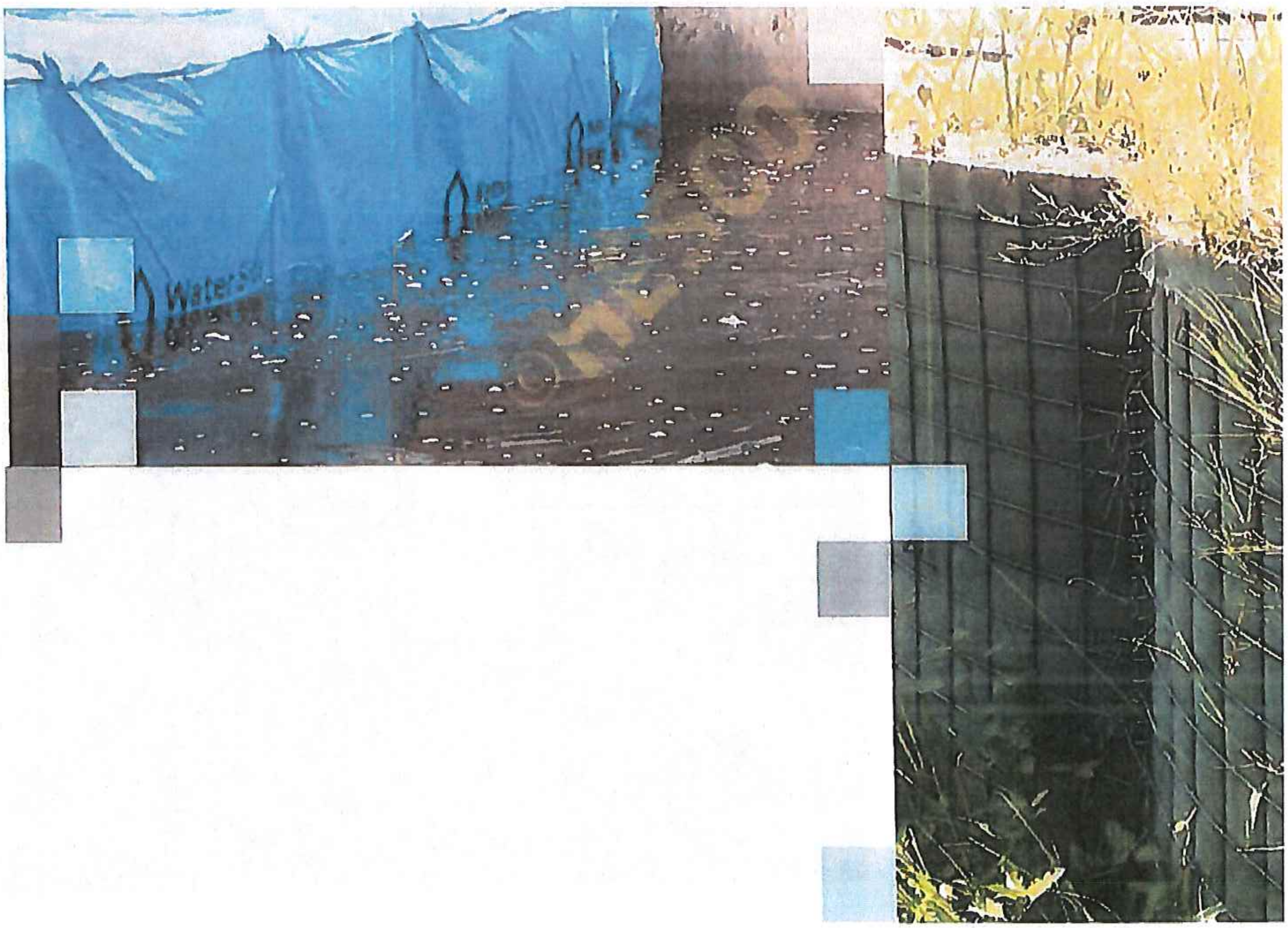
COME RAIN OR SHINE
AN INTEGRATED PRODUCT RANGE



Since its first deployment in the military field of operations, HESCO products have been adapted to meet a diverse range of requirements, in the military, security and environmental sectors

Flood and storm events are a constant threat in many parts of the world. Since Hurricane Katrina Hesco has developed unrivaled experience in dealing with these climatic events, in rural and urban landscapes. Working with local and national authorities has enhanced our knowledge and our ability to provide response on an unprecedented scale, across the United States and beyond

New challenges emerge and Hesco continues to develop new products using new approaches, techniques and materials to provide the world with a comprehensive range of products to defend against threats both natural and man-made

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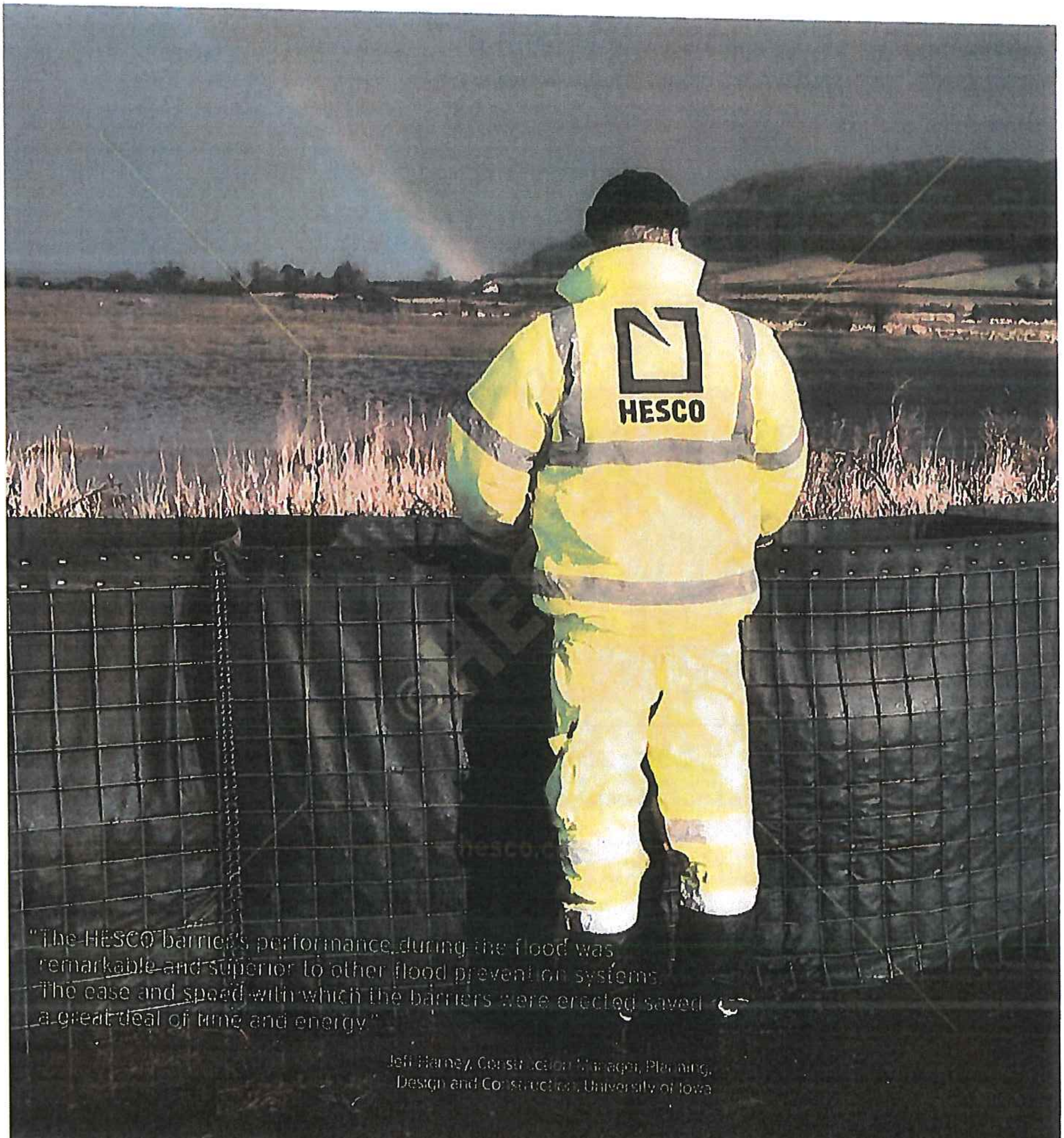
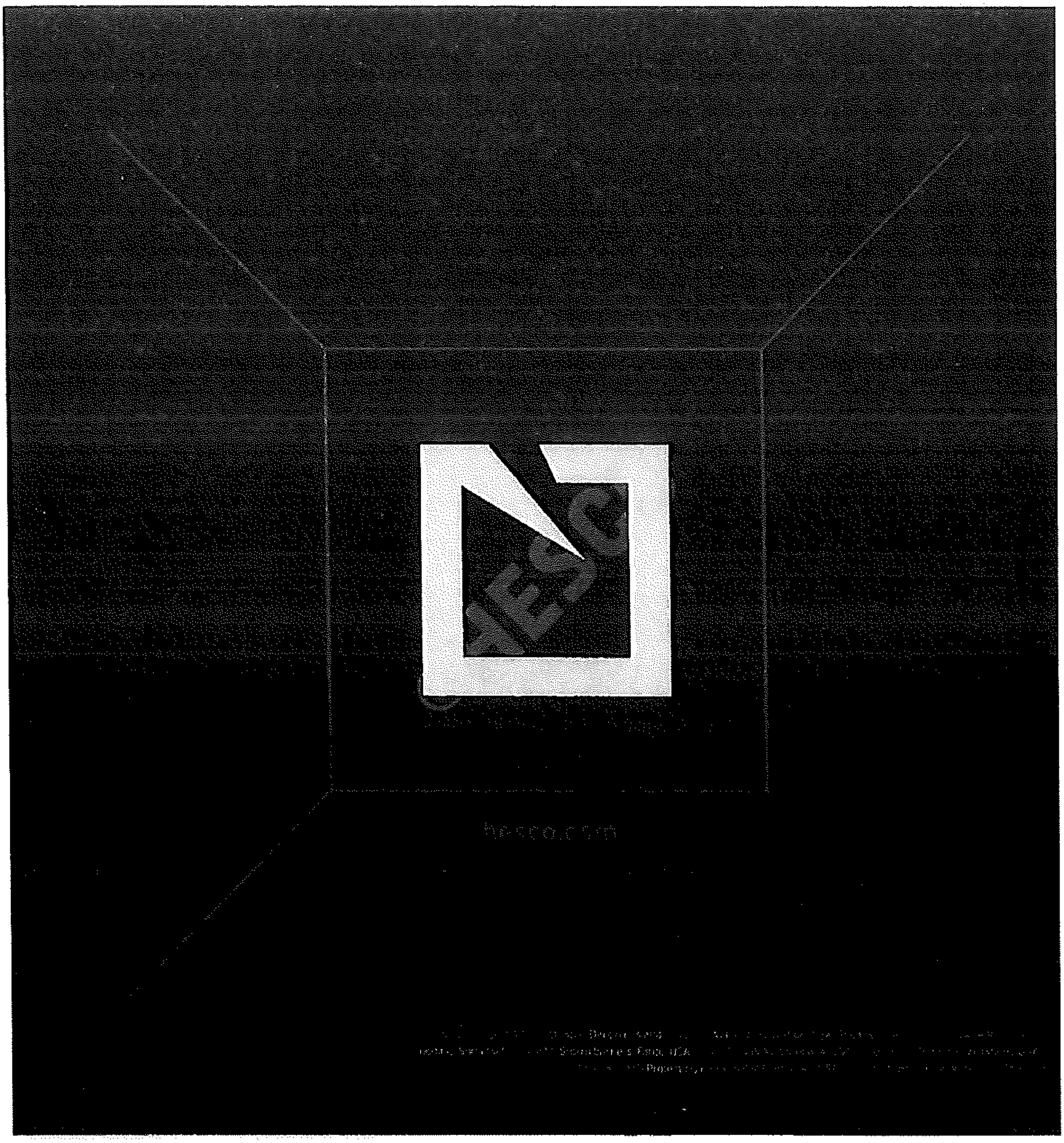
TRAINING AND CONSULTATION
COMPLETE ONSITE SOLUTIONS

Hesco doesn't just supply products, but also the know-how to use them effectively, through hands-on training and tailored project consulting. The use of our products on a huge scale in conflicts and emergency situations around the world has made the provision of experts in the best use of our products an essential part of our service.

The complexity of any situation is heightened in emergency, that's why our technical teams are on hand to train and to help orchestrate response, to ensure the optimum result for our customers, whatever the threat.


HESCO
"The HESCO barrier's performance during the flood was remarkable and superior to other flood prevention systems. The ease and speed with which the barriers were erected saved us a great deal of time and energy."
Jeff Harney, Construction Manager, Planning, Design and Construction, University of Iowa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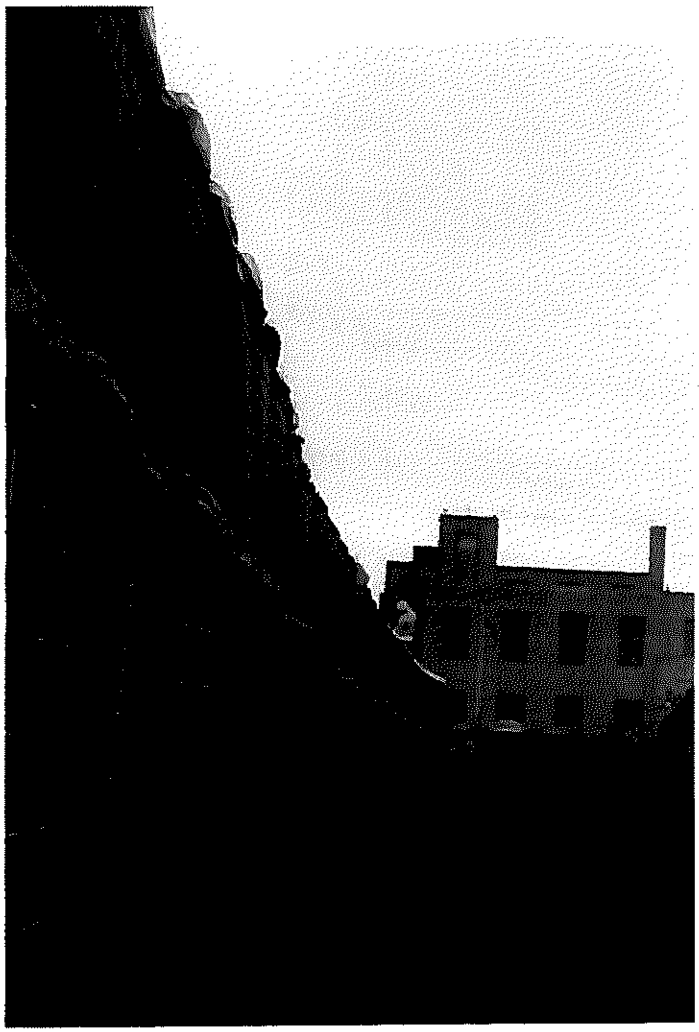
Quality Standards
[illegible]

HESCO Barrier
PHOTOS

General Photo of HESCO Barrier Block Installation

General Photo of HESCO Barrier Block Installation with Flood



HESCO Barrier Block Installation
at Council Bluffs, IA

TECHNICAL SPECIFICATIONS
DEFENSIVE BARRIERS

MIL™ 1 UNIT

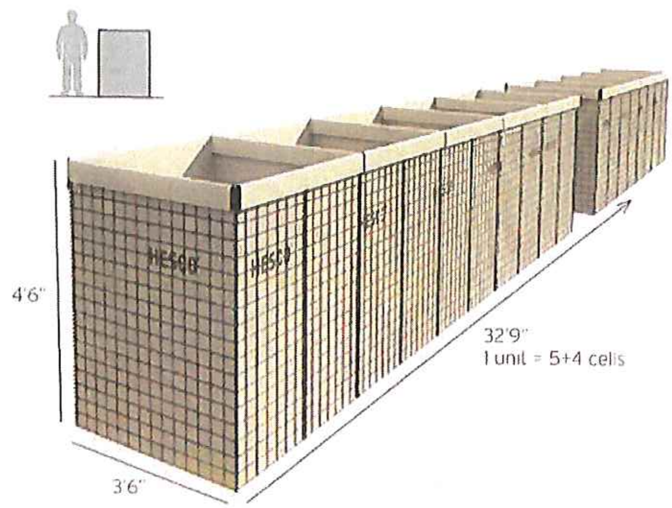
UNIT	HEIGHT	WIDTH	LENGTH	NSN
MIL1 (Beige)	4'6" (1.37m)	3'6" (1.06m)	32'9" (10m)	5680-99-835-7866
MIL1 (Green)	4'6" (1.37m)	3'6" (1.06m)	32'9" (10m)	5680-99-001-9396

A geotextile-lined unit for general use as an earth filled barrier. The units are suitable for filling with earth, sand, gravel, crushed rock and other granular materials. The unit fulfills a wide range of uses, including the construction of protective walls and barriers.

GENERAL SPECIFICATIONS

Geotextile-lined welded mesh barrier coated to ASTM A 856. All wires conform to BS EN 10218 2 2012. Zinc-Aluminum coatings are to BS EN 10244-2.2009 where appropriate. The geotextile is a heavy duty, non-woven, permeable, polypropylene fabric, available in either beige or green color.

All dimensions and weights are nominal. Diagrams and product images are for illustrative purposes only.



PACKAGING AND TRANSPORT INFORMATION

FLAT-PACKED INDIVIDUAL UNIT DIMENSIONS & WEIGHTS

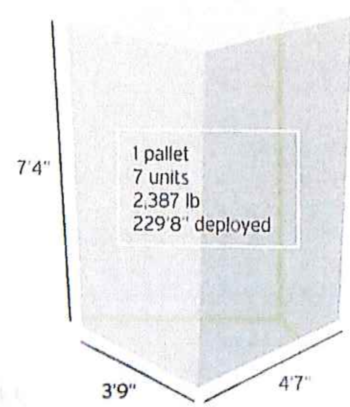
HEIGHT	WIDTH	LENGTH	WEIGHT
10" (0.25m)	3'10" (1.17m)	4'6" (1.37m)	329 lb (149kg)

PALLET INFORMATION

PER PALLET	HEIGHT	WIDTH	LENGTH	WEIGHT
7 units	7'4" (2.24m)	3'9" (1.14m)	4'7" (1.40m)	2,387 lb (1,083kg)

TRANSPORT INFORMATION

TYPE	PALLETS	UNITS	DEPLOYED LENGTH
13.5 Trailer	18	126	4,133' (1,260m)
20' Container	8	56	1,837' (560m)
40' Container	16	112	3,674' (1,120m)



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Quality Standards
Hesco operates a quality management system certified to ISO 9001 2008
Approved Certificate Number LRQ 0910654