5.0 STRUCTURES

5.1 In General

This guidance applies to structures whose primary function is hurricane flood protection in the New Orleans area, which includes T, L & I-walls, sluice gates, fronting protection and flood gates. Sector gates and other navigable waterway structures shall have all design criteria approved prior to design.

The Corps of Engineers is governed by engineering regulations (ER's), engineering manuals (EM's), engineering technical letters (TL's) and engineering circulars (EC's). These Corps publications are available on line at the following web site: <u>http://publications.usace.army.mil/publications</u>. The designer is responsible for compliance with all civil works engineering regulations, circulars, technical letters and manuals (Corps publications). For convenience, this document highlights certain Corps publications that engineers should be aware of. Also, specific design criteria are identified in the following sections that may not agree with the Corps publications; in this case, the more conservative criteria shall be applied. Industry standards shall apply when Corps criteria is not applicable.

5.1.1 Sampling of References

USACE Publications

- EM 1110-2-2102, Waterstops and Other Preformed Joint Materials for Civil Works Structures, Sept 95
- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, June 92 (Including Change 1, Aug 03)
- EM 1110-2-2105, Design of Hydraulic Steel Structures (including Change 1), May 94
- EM 1110-2-2502, Retaining and Flood Walls, Sept. 89
- EM 1110-2-2906, Design of Pile Foundations, Jan. 91
- EM 1110-2-2503, Design of Sheet Pile Cellular Structures Cofferdams & Retaining Structures, Sept. 89
- EM 1110-2-2504, Design of Sheet Pile Walls, Mar. 94
- EM 1110-2-2705, Structural Design of Closure Structures for Local Flood Protection Projects, Mar. 94
- EM 1110-2-1901, Seepage Analysis and Control for Dams, Apr 93
- EM 1110-2-2100, Stability Analysis of Concrete Hydraulic Structures, Dec 05

Technical Publications

- American Concrete Institute, Building Code and Commentary, ACI 318-02
- American Institute of Steel construction, Manual of Steel Construction (9th Ed.)
- American Welding Society, AWS D1.1 (2006)
- American Welding Society, AWS D1.5 (2002)

• ASCE/SEI 7-05, Including Supplement No. 1, Minimum Design Loads for Buildings and Other Structures

Computer Software

- CE Pile Group Analysis Program, "CPGA"
- CE Structural Analysis Program, "C-Frame"
- CE Strength Analysis of Concrete Structural Elements, "CGSI"
- CE Sheet Pile Wall Design/Analysis Program, "CWALSHT"
- Structural Analysis and Design Software, "STAAD"
- Ensoft, "Group 7.0"
- Additional approved USACE programs

5.1.2 Survey Criteria

Surveys shall conform to "USACE New Orleans District Guide for Minimum Survey Standards" (see Section 9) and the following guidance at a minimum. A typical scope of services for surveys in support of structural designs is included in Section 9.4.

5.1.3 General Design Criteria

Walls shall be constructed using the latest datum from Permanent Benchmarks certified by NGS - NAVD88. A total of three Permanent Benchmarks are required, one for design/construction and two for verification.

The following is a summary of recommended wall heights:

- I-Walls 4 ft. maximum (Includes required 6 inches of overbuild. 6 inches of additional wall height is historically added to the top of I-walls to accommodate future subsidence in soft soils. This additional 6 inches is not added to T or L type walls since their deep pile foundations do not typically experience the same settlements as an I-wall.)
- T-Walls No height limit; Typically 4 ft. and greater
- L-Walls / Kicker Pile Walls 8 ft. maximum

The above permitted heights are measured on the protected side of the wall. The flood side height may be increased by 2 feet for both I-walls and L-Walls.

Structural Superiority – All new structures that are difficult to construct due to their nature, such as railroad and highway gate monoliths that require detours causing disruptions to traffic, pumping station fronting protection that require cofferdams within their discharge basins causing reductions to pumping capacity, sector gated structures causing disruptions to navigation, large utility crossings, etc., shall be designed with a minimum of 2 ft. of additional wall height. This additional height shall be included in all top of wall load cases. All variances shall be approved by the USACE engineer of record. Note that DIVR 1110-1-16 also provides guidance for structural superiority for major

hydraulic structures and is not applicable to hurricane protection floodwalls and fronting protection type structures designed utilizing the HSDRRS design criteria, whereas the height increase is similar, the HSDRRS is more applicable to coastal areas.

All concrete capped I-walls shall have 6 in. minimum overbuild. I-walls shall be symmetrical so not to create an unbalance concrete section.

T-walls are the recommended type of floodwalls where there is the potential for barge/boat impact loading or unbalanced forces predicted from a global stability analysis. Global stability, as it affects T-wall foundation design, is addressed in Section 3.4.3 T-Wall Design Procedure.

L-Walls may also be used where there is the potential for barge/boat impact loading; however, they shall **not** be used where an unbalanced force is predicted based on a global stability analysis.

Typically, I-walls shall **not** be used on navigable waterways or where there is the potential for barge/boat impact loading unless measures (such as berms for grounding vessels or separate pile fender systems) are taken to protect the wall. When placed atop a levee crown, walls under 2' (on both sides of the F/W) are permissible provided they are located on the protected side of the levee crown and armoring is provided on the protected side. However, I-walls are acceptable as tie-ins to levee embankments. Site and soil conditions will dictate their use in these applications.

Lengths of L-Wall or T-wall monoliths should generally be 40 to 60 feet between expansion joints. I-wall monoliths should generally be 30 to 40 feet.

Where walls form corners at Points of Intersection (PI), walls shall extend monolithically past the PI a minimum of 5 ft., but not less than 2 full sheet piling sections and at least one row of support piling, before terminating the monolith.

Fabricated sheet pile corners and connectors shall be bolted using 7/8" diameter high strength bolts meeting the requirements of ASTM A 325, Type 3, or ASTM A 490, Type 3. The bolts shall be spaced on 6 inch centers for the length of the section except for 2 feet at each end where they are spaced on 3 inch centers. Welding of the longitudinal sheet pile joint is **not** typically allowed and will only be permitted on seepage cut-off sheet piling driven in soft soils with no obstructions anticipated.

In lieu of fabricated sheet pile corners and connectors, one piece seamless extruded pile corners and connectors **shall** be allowed. The extruded pile corners and connectors shall be fabricated from the same grade and strength material as the adjoining sheet piling sections.

Geotechnical Engineers shall minimize the height of the wall system by designing the largest earthen section that is practical and stable for each individual project.

Seepage, global stability, heave, settlement and any other pertinent geotechnical analysis shall be performed in order to ensure that the overall stability of the system is designed to meet all Corps criteria.

Flood wall protection systems are dedicated single-purpose structures and shall not be dependent on or connected to other (non-Federal) structural or geotechnical features that affect their intended performance or stability.

5.2 T-wall & L-wall Design Criteria

T-walls, whose primary function in the New Orleans area is flood protection, are pile founded structures that consist of a reinforced concrete wall and base with steel sheet pile cut-off. Steel or prestressed concrete piles are battered towards the protected and flood sides and are the main components that support the concrete wall and base. The primary purpose of the steel sheet piling is to provide a seepage cutoff beneath the wall. T-wall foundation design procedures are included in Section 3.4.

T-walls exhibited the best overall performance during Katrina. Due to the transfer of all applied loadings to deeper soil strata, the T-wall, as a rule, is more resilient to overtopping and global instability than L-walls or I-walls. Additionally, because of the robust nature of the T-wall and its foundation, it is not as susceptible to catastrophic failure if impacted by debris or marine vessels (i.e. if struck by a barge, any failure would be localized to the impact point, versus I-wall, where progressive failure due to the erosion of passive resistance upon breaching would be likely) or there is a potential of foundation instability due to unbalanced loading.

Guidance on floodwalls subject to impact loads, is provided in Sections 5.2.1. Loading Conditions, 5.8 Impact Barriers (Dolphins, Pile Clusters, etc.) and 5.9 Loading Tables & Impact Maps.

L-walls are similar to T-walls except that the steel sheet pile replaces the flood side pile row.



Figure 5.1 Typical T-Wall and L-Wall Configuration

5.2.1 Loading Conditions

1) Load Cases. See Section "5.7 General Load Case Tables."

2) Impact Load Cases. See Section "5.9 Loading Tables & Maps" for additional guidance.

<u>BARGE IMPACT</u> - This guidance only addresses Hurricane event induced forces, it does not include any load case requirements for Marine Vessel navigation impacts which shall also be considered where applicable. The included Zone 1 load cases apply only to hurricane protection structures exposed to barge traffic. Impact barriers can be used to shelter hurricane protection structures; the loading would then be reduced to the Zone 3 Debris Loading. Impact barriers are covered in para 5.8. Based on IPET data, the single, light barge was selected as the design vessel. A full barge was also studied but the heavier vessel did not govern as the velocities were considerably less. For the Unusual load case, due to the instantaneous nature of the barge impact as compared to the much longer wave period, the barge impact and wave forces were considered to <u>NOT</u> occur simultaneous.

Zone 1A - Barge Impact Design Load Cases (Southeasten Louisiana, Protected Waterways)

a. USUAL. Under usual conditions, the probability of barge impact is low and considered an unusual case; therefore, barge impact shall not be included as a usual load case.

b. UNUSUAL . A barge impact force of 200 kips shall be applied at the Top of Wall (TOW), including any Structural Superiority. The barge impact loading shall be combined with the hydrostatic pressure induced by the (100-yr) SWL, at 90% confidence, plus a wind load on any exposed portion of the wall above the (100-yr) SWL. The wind load, to be used in conjunction with the boat impact analysis, shall be computed in accordance with the following simplified formula, $F = .00256 (V^2)(I)(A)$, where the minimum wind velocity (V) shall be 140 mph, utilizing an Importance Factor (I) of 1.15, applied to the gross exposed area (A). The permitted pile capacity Factors of Safety and structural component overstresses are provided in Table 5.2 (b).

c. EXTREME - Case I. A barge impact force of 400 kips shall be applied at the Top of Wall (TOW), including any Structural Superiority. The barge impact loading shall be combined with the hydrostatic pressure induced by the (100-yr) SWL, at 90% confidence, plus a wind load on any exposed portion of the wall above the (100 yr) SWL. The wind load, to be used in conjunction with the boat impact analysis, shall be computed in accordance with the following simplified formula, $F = .00256 (V^2)(I)(A)$, where the minimum wind velocity (V) shall be 160 mph, utilizing an Importance Factor (I) of 1.15, applied to the gross exposed area (A).

d. EXTREME - Case II. A barge impact force of 200 kips shall be applied at the Top of Wall (TOW), including any Structural Superiority. The barge impact load shall be combined with the hydrostatic pressure induced by the (100-yr) SWL and the (100-yr) wave load, at 90% confidence.

The Extreme Load Cases shall not exceed the ultimate capacity of the structure as determine by a Push-Over analysis. An example of the push-over analysis is attached as an Appendix. In lieu of a push-over analysis, capacity can be determined by an elastic analysis of all structural components and limiting the pile capacity Factors of Safety and structural component overstresses and Load Factors to those designated in Table 5.2 (b).

Zone 1B - Barge Impact Design Load Cases (Southeastern Louisiana, Waterways Directly Exposed to Tidal Surge (500-yr) SWL)

a. USUAL. Under usual conditions, the probability of barge impact is considered low and shall not be included as a usual load case.

b. UNUSUAL. A barge impact force of 225 kips shall be applied at the lower of the Top of Wall (TOW), including any Structural Superiority, or the (100-yr) SWL plus 7 ft. The barge impact loading shall be combined with the hydrostatic pressure induced by the (100-yr) SWL, at 90% confidence, plus a wind load on any exposed portion of the wall above the (100-yr) SWL. The wind load, to be used in conjunction with the boat impact analysis, shall

be computed in accordance with the following simplified formula, $F = .00256 (V^2)(I)(A)$, where the minimum wind velocity (V) shall be 140 mph, utilizing an Importance Factor (I) of 1.15, applied to the gross exposed area (A). The permitted pile capacity Factors of Safety and structural component overstresses are provided in Table 5.2 (b).

c. EXTREME - Case I. A barge impact force of 450 kips shall be applied at the lower of the Top of Wall (TOW), including any Structural Superiority. or the (500 - yr) SWL plus 7 ft. The barge impact loading shall be combined with the hydrostatic pressure induced by the (500-yr) SWL, at 90% confidence, plus a wind load on any exposed portion of the wall above the (500-yr) SWL. The wind load, to be used in conjunction with the boat impact analysis, shall be computed in accordance with the following simplified formula, F = .00256 (V²)(I)(A), where the minimum wind velocity (V) shall be 160 mph, utilizing an Importance Factor (I) of 1.15, applied to the gross exposed area (A).

d. EXTREME - Case II. A barge impact force of 225 kips shall be applied at the lower of the Top of Wall (TOW), including any Structural Superiority, or the 100 yr SWL plus 7 ft. The barge impact load shall be combined with the hydrostatic pressure induced by the (100-yr) SWL and the wave load, at 90% confidence.

The Extreme loadings shall not exceed the ultimate capacity of the structure as determine by a Push-Over analysis. An example of the push-over analysis is attached as an Appendix. In lieu of a push-over analysis, capacity can be determined by an elastic analysis of all structural components and limiting the pile capacity Factor of Safety and structural component overstresses and Load Factors to those designated in Table 5.2 (b).

<u>BOAT IMPACT</u> – This guidance only addresses Hurricane event induced forces, it does not include any load case requirements for Marine Vessel navigation impacts. The load applies only to structures not exposed to barge traffic, such as pleasure craft and fishing boats.

Zone 2 - Boat Impact Design Load Case

a. USUAL. Under usual conditions, the probability of boat impact is considered low and shall not be included as a usual load case.

b. UNUSUAL (100-YR). A boat impact force of 50 kips shall be applied at the lower of the Top of Wall (TOW), including any Structural Superiority, or the (100-yr) SWL plus 7 ft. The boat impact loading shall be combined with the hydrostatic pressure induced by the (100-yr) SWL, at 90% confidence, plus a wind load on any exposed portion of the wall above the (100-yr) SWL. The wind load, to be used in conjunction with the boat impact analysis, shall be computed in accordance with the following simplified formula, F = .00256 $(V^2)(I)(A)$, where the minimum wind velocity (V) shall be 140 mph, utilizing an Importance Factor (I) of 1.15, applied to the gross exposed area (A). The permitted pile capacity Factors of Safety and structural component overstresses are provided in Table 5.2 (b).

c. Extreme load case is not applicable.

DEBRIS IMPACT

Zone 3 – Debris Impact Design Load Case

a. USUAL. Not Applicable.

b. UNUSUAL. All floodwalls outside locations of barge / boat impact zones, shall include a minimum debris impact loading of 0.5 kips/ft, applied at the TOW, but not to exceed the (500-yr) SWL.

c. EXTREME. Not Applicable.

5.2.2 Pile Foundations – Precast-Prestressed Concrete, Steel (H and Pipe) and Timber

5.2.2.1. Pile Design.

The factors of safety with no overstress for all MVN projects are:

	With Pile Load Test	W/O Pile Load Test
Q-Case	2.0*	3.0
S-Case	1.5	1.5

* FOS = 2.5 must be used with a PDA test for the Q-case (for compression piles only)

Q-Case = Unconsolidated - Undrained Shear Strength Test for soils

S-Case = Consolidated - Drained Shear Strength Test for soils

Spiral Welded Pipe (SWP) fabricated in accordance with ASTM A252 is only allowed in the design and construction of temporary retaining structures and work platforms. In permanent hydraulic structure foundations, SWP is permitted for use as service piles in flood protection foundations, but must comply with more stringent fabrication specifications, subject to the following:

- Use is limited to piles with outside diameters from 18 to 54 inches and wall thicknesses not greater than 1-1/8 inches. Negative wall thickness tolerances are not permitted
- The pile diameter to wall thickness ratio shall not exceed 55.
- Use is limited to pile group applications (not allowed for single pile supported structures).

- Use is confined to the Southeast Louisiana coastal area where soil conditions are comparable to those encountered at the test sites addressed in the report "Spiral Welded Pipe Piles for Coastal Structures" and cyclic loadings are not considered.
- The weld reinforcement (Bead Height) shall not be greater than 3/16 inch.
- The latest COE specifications are adhered to that include NDT and fabrication tolerances. SWP fabricated "only" in accordance with ASTM A 252 are not permitted.

Timber piles are **not** allowed in the design of hurricane flood protection structures where either tension or unbalanced loads exist.

To avoid the use of combining factored loads with actual service loads, the actual unfactored service loads shall be used when designing any pile foundation. See paragraphs 5.7 and 5.9 for further details on required Factors of Safety and overstress conditions.

When using pile analysis software, such as CPGA or Group 7, the unfactored soil properties shall be input, with the exception of the Subgrade Modulus (E_s) which may be reduced for group effects, if applicable.

Reductions for pile spacing and unstable soil wedges are included in Section 3.4.3.

For T-wall foundations, the designer may utilize either a pile stiffness based computer program, such as CPGA, or a computer program that models the nonlinear response of the soil, such as Group 7 (Ensoft, Inc).

CPGA is a simplified program for pile group analysis that accounts for the effects of pile locations and batters. It can represent linearly any type of pile-soil interaction and can represent either fixed or pinned interaction between the pile and the pile cap. The program does not account for the effects of pile cap flexibility, i.e. the pile cap is assumed rigid. The program does not account for the effects of non linear soil behavior.

Group 7 is also a program for analyzing the behavior of piles arranged in a group. Piles may be vertical or on a batter with the pile heads fixed, pinned or elastically restrained by the pile cap. The pile cap may settle, translate and/or rotate and is assumed to act as a rigid body. The program generates internally the nonlinear response of the soil and of each pile under combined loadings and assures compatibility of geometry and equilibrium of forces between the applied external loads and the reactions of each pile head. The program can internally compute the deflection, bending moment, shear, and soil resistance as a function of depth for each pile. For closely spaced piles, the pile-soilpile interaction can be taken into account by using reduction factors for each single pile.

When both analysis types are used, such as required when unbalanced loads are present, the more conservative pile tip elevation shall govern the final design.

The designer should use the referenced software. Other software may be suitable for the pile analysis; however, a formal request with all supporting documentation on the proposed software shall be submitted to the USACE for review and approval prior to proceeding with the design analysis.

Global stability in the form of unbalanced loads, and settlement effects must be considered in the design of pile foundations. The unbalanced load (UBL) is resisted by the bearing piles. The calculation of the UBL is described in Section 3.4.3. The design preference is to eliminate the UBL with stability berms and eliminate the effects of settlement with advance preloads.

Additional flexural stresses are induced by settlement of soil above the piles. Vertical settlement induces a load normal to the longitudinal axis of the pile. Preloading the foundation to eliminate excess settlement should be first course of action when possible. Consolidation time can be significantly reduced with the inclusion of wick drains. When settlement is not eliminated, bending stresses shall be calculated in accordance with the L-Pile Method as prepared by the COE and VA Tech, and included in Chapter 3 of this Design Guidance. Settlement induced stresses are residual and shall be included in all load cases. A normal operating case must be included from which long term settlement is calculated. These settlement induced stresses are added as a separate component of the interaction equation for combined axial and bending stresses. Allowable stresses for load cases that include the UBL are listed in Tables 5.2 (a & b). The settlement induced stress component of the interactive equation shall be included as a separate factor as "fbd/Fbd". Combined allowable stresses for Load Cases that include settlement induced bending. The interactive equation is as follows:

 $(fa / Fah) + (fb / Fbh) + (fbd / Fbd) \le 1.0$

Fah - allowable axial stress (hurricane loading)

Fbh - allowable bending stress (hurricane loading)

Fbd - allowable bending stress (settlement induced bending)

Notes:

a. Piles are considered laterally supported throughout the embedded length of pile regardless of the presence of settlement and unbalanced loads.

b. The pile design sectional properties shall **not** consider the sacrificial steel added to combat corrosion.

c. Referencing Table 5.2 (b) for Service Load factors, settlement overstresses for Fbd are as follows:

BLC I and II - settlement effects not considered in Construction Cases

NORMAL OPERATING - 17% (Long term case after construction) BLC III and VII - 17%

BLC IV, V, VI, VIII, IX and DRC I - 40%

All other Design Resiliency Checks (DRC) shall use the Load Case overstresses listed in Table 5.2 (b).

Prestressed Concrete Piles are **NOT** recommended for use where settlement is significant. In existing monoliths that include prestressed concrete piles the allowable Fbd factor can be determined by utilizing the overstress factors listed above or use increased allowables as follows:

NORMAL OPERATING, BLC III and VII

Fc = 0.45 f'c - fpe (compression) Ft = $3\sqrt{f'c}$ (tension)

BLC IV, V, VI, VIII, IX

Fc = 0.60 f'c - fpe (compression) Ft = $6\sqrt{f'c}$ (tension)

All other DRC cases

Fc = 0.85 f'c - fpe (compression) Ft = $6\sqrt{f'c}$ (tension)

Example: LC V - (100 yr) SWL + Wave

HSDRRS - Pile OS allowable - 33%, Table 5.2 (a) Fbd OS - 40%

Interaction equation:

fa / (Fah)1.33 + fb /(Fbh)1.33 + fb /(Fbd)1.40 \leq 1.0

EM 1110-2-2906, Design of Pile Foundations, provides specific guidance on pile stresses. Size selection for a particular hammer must consider the pile's anticipated driving resistance, ultimate capacity, pile stresses expected during driving, and pile setup. Final equipment approval will be based on a review of the pile stresses, produced in

a Wave Equation Analysis, that is required to be performed by the contractor. The hammer type and size used for production should always match that used in the test program. Diesel hammers shall "not" be allowed for driving precast prestressed piling.

Maximum Driving Stresses as determined by PDA or Wave Equation Analysis shall not exceed:

Steel Piles	-	0.85 fy
Prestressed Concrete	-	0.85 f'c - fpe (compression)
	-	$3\sqrt{f'c} + fpe$ (tension)

Limiting penetration rates are generally established by the Government based upon the results of a Wave Equation Analysis. Typically the following maximum blow counts apply. Timber (3-4 blows per inch), Concrete (10 blows per inch), Steel Pipe (10 - 20 blows per inch), Steel H (10 - 20 blows per inch).

Unless considered in the pile load test, the increased friction capacity due to the added length of a battered pile versus the vertical component shall be ignored.

Piles battered at a slope steeper than 1H on 8V shall be analyzed as vertical piles.

Weight of piles may be neglected in pile design.

Pre-stressed Concrete Piles shall have a strand pattern that is symmetric about both axes or placed in an evenly distributed pattern.

Maximum structural foundation deflections at top of pile (Basic Load Cases, Excluding Settlement):

Normal case, no overstress allowed Vertical – 0.50" or less Horizontal – 0.75" or less Case with 16^{2/3} % overstress allowed (Construction) Vertical – 0.583" or less Horizontal – 0.875" or less Case with 33^{1/3} % overstress allowed (Wind, wave and impact) Vertical – 0.67" or less Horizontal – 1.0" or less

These vertical and horizontal displacements are those normally generated from the short and long term load cases indicated. Adherence to these values typically ensures proper operation and integrity of hydraulic type structures.

Larger deflections may be allowed for "Design Resiliency Checks" and load cases involving boat / barge impacts if stresses in the structure and piles are not excessive. Larger deflections are limited to values that remain in the elastic state of the soil.

The design process requires consideration of water-tightness and potential differential movements between monoliths and at tie-ins. Water-tightness and differential movements shall be accounted for in details such as waterstops and sheet pile connections. Standard details that address these issues are provided in the Typical Drawings.

The lateral behavior of the foundation is sensitive to pile fixity. A minimum pile embedment of 9" is required. The connection may be assumed to be pinned if the embedment is between 9" and 12". A pile embedment length equal to or greater than twice the pile depth or diameter is required to develop full fixity for a pile embedded in the base of the structure. Should a fixed condition be assumed, the full effective depth of the moment resisting foundation base slab shall be reinforced. If tremie concrete is used to provide composite action it shall be reinforced. All reinforced – Concrete Hydraulic Structures). This reinforcement criteria shall be met regardless of the stress levels in the base slab.

Any embedment depth between these two options must be researched to determine the applicable connection. CERL Technical Report M-339, dated Feb 1984 and entitled "Fixity of Members Embedded in Concrete," is a recommended information source. The embedded portion of a pile consists of the solid concrete or steel section and does not included the tension hooks, see Figure 5.2.

The moment from the piles transferred into the base slab must be considered when designing the concrete reinforcement. Care must be taken to ensure proper moment orientation. A pile moment which is beneficial to the design shall be neglected.

5.2.2.2 Pile Tension Connectors

Pre-stressed Concrete. Pre-stressed concrete pile tension capacity is limited to the development length of the rebar and strand at the top of pile. Development lengths and hooks shall comply with the applicable provisions of ACI 318, Chapter 12. All potential failure modes listed in ACI 318-08, Appendix D, should be checked for concrete pullout capacity.

Steel H and Pipe. Pile tension connectors shall meet the requirements of ASTM A 706, deformed rebar for any open-ended tension hooks. Smooth round bar meeting the requirements of ASTM A 572 or flat bar meeting the requirements of ASTM A 572 will be allowed in closed-loop applications where the bars are connected to both sides of the foundation pile flanges. A joint specific Procedure Qualification Record, Welding Procedure Specification, and the Welder Qualifications shall be submitted for all types of welds used. 100% of all welds shall be visually inspected and 25% of all welds shall be tested using the appropriate non-destructive testing required for the weld type used per

the requirements of AWS D1.4 for rebar/round bar and AWS D1.1 for flat bar. Rebar shall be welded to the plate using double-flare-bevel-groove welds using an indirect butt joint procedure.

AWS D1.4/D1.4M:2005



The maximum load from CPGA analysis (load case with appropriate allowable overstress

factor applied, if applicable) will be used in all design checks.

The following failure modes shall be investigated for all tension connector hooks.

- Check allowable stress in each anchor bar using AISC 9th edition, ASD criteria (Chapter D, Specification and Commentary). The allowable stress should be reduced by 5/6 of the computed capacity. For deformed rebar, the development length shall be checked using the requirements of ACI-318.
- Design the weld connecting the plate to the flange of the foundation pile using AISC J2 (5-67) criteria. The shear resistance of the weld should be reduced by 5/6 of the computed capacity.
- Design the weld connecting the hooks to the plate using AISC J2 (5-67) criteria. The shear resistance of the weld should be reduced by 5/6 of the computed capacity.
- Check plate shear using AISC F4 (5-49) criteria. The shear resistance of the plate should be reduced by 5/6 of the computed capacity.



Figure 5.2 Depth of pile embedment

- Check capacity of foundation pile flange using AISC J4 (5-77) criteria. The shear resistance of the flange should be reduced by 5/6 of the computed capacity.
 - Check shear in the flange based on yielding and rupture.
 - Check block shear between plate welds.
 - Check block shear from far fillet weld to edge of flange.
- All potential failure modes listed in ACI 318-08, Appendix D, should be checked for concrete pullout capacity.

All tension piles, regardless of load case or magnitude, shall have tension connectors. Tension connectors are not required on compression piles unless any load case for a particular pile induces a compressive load of 10 kips of compression or less. To assure resiliency, a minimum of 2 pile rows shall have tension connectors.

5.2.2.3 Pile Splices.

Pre-stressed Concrete – Pre-stressed concrete piles shall be delivered to the site full length, pile splices are not allowed.

Steel Piles -

• Generally, splices shall be located in the middle 1/3 of the pile. To avoid soil disturbance, splices in the lower 20 ft. of the pile shall **not** be permitted. Splices outside the middle 1/3 of the pile will be permitted on a case-by-case basis, particularly where overhead obstructions demand an increase in splicing. In these limited cases, the standard full penetration weld with cover plates shall be used; commercial splices shall **not** be allowed. Additionally, 100% of the weld length shall be non-destructively tested using both VT and UT.

• Driving stresses must be checked with a Wave (WEAP) analysis for the type of hammer and soil profile or from pile load test PDA results. Maximum Driving Stresses – Compression or Tension shall be less than 0.85 Fy.

Commercial Steel Splices -

- The Champion H-pile Splicer HP-3000, with minimum plates of 3/8 inch and material steel grade of GR50 yield, or equal, is an approved H-Pile splice alternative. The Manufacturer must provide test data for the Splicer (x-x) bending capacity along with material mill certificates.
- Welding and Testing Requirements:

Flanges - shall be 100% Complete Joint Penetration (CJP) single bevel weld.

Web - 5/16 minimum fillet weld for the splicer to H-Pile. Minimum weld length shall be 2 ½ inches on each splicer flange to HP flange and then down the flange and across the entire H-pile web (HP14x73 and HP14x89), but not less than the manufacturers recommendations. The splicer web fillet weld (size) shall develop the axial tension equivalent to Fa = 0.5Fy X Pile web area (net).

Provide WPS for each weld type.

Testing shall be 100 % visual testing of fillet welds and 25% UT for the flange welds.



5.2.2.4 Pile Handling

Handling holes are permitted in H-piles subject to the following criteria:

- A Request for Information (RFI) shall be required from Construction Division.
- Holes are prohibited when driving stresses exceed 85% Fy.
- Burning of holes shall not permitted, all holes must be drilled. Holes shall not exceed 1-1/2" in diameter.
- Holes shall be located below the upper 1/3 of the pile. At this location plug welding of holes is not required.
- Upon approval of the "Designer of Record", holes may be permitted in the upper 1/3 of the pile but shall be plug welded. The plug weld will include a 3/8 inch backing plate welded to the interior face of the flange. Coating touch up will be required where applicable.
- Typically, when the unbraced length of H-piling exceeds 80 ft., the pile self weight alone may cause the pile to become overstressed during lifting and therefore should be checked. An overstress of 50% is permitted for this short term loading condition.

5.2.3 T-wall Sheet Piling Section

The primary purpose of the steel sheet piling is a pile acting to control seepage. Piping and Seepage Analysis methods are described in Section 3.4.2.5.

A minimum PZ-22 hot rolled sheet piling shall be utilized for seepage cut-off.

The sheet pile shall be adequately anchored into the base slab to resist pull out. This is particularly important when downdrag is present. This can be achieved by passing U-bars through existing handling holes or burning holes in the sheet pile, if necessary.

5.2.4 L-wall Sheet Piling Section

The steel sheet piling is a pile acting to control seepage and provide support to the structure.

The sheet pile shall be designed to take the tension loads resulting from an inverted T-Wall analysis (CPGA) for the listed loading conditions. In addition, the sheet pile shall be designed as a compression member for the dead load case.

The minimum sheet piling section shall be a hot rolled PZ–27.

Due to the embedment of the sheet pile, approximately 2.75 to 3.0 feet into the base slab, the sheet pile should be assumed to be a fixed pile in the CPGA program.

The sheet pile properties should be assumed to be the summation of the pile properties for the kicker pile spacing.

The sheet pile shall be adequately anchored into the base slab to resist tension loads. This can be achieved by the use of welded studs or welded tension connectors.

5.2.5 Sheet Piling Tip Penetration

See the Geotechnical Section of this document for sheet pile tip penetration requirements for T-walls & L-walls.

5.3 I-wall Design Criteria

5.3.1 Loading Conditions

(1) Load Cases. See Section "5.7 General Load Case Tables."

(2) Impact Cases. See Section "5.9 Boat/Barge Impact Loading Tables & Maps."

5.3.2 I-wall Sheet Piling Section

The steel sheet piling is a pile acting to control seepage and provide support to the structure.

Design the steel sheet piling using the moments and shears developed by the factored soil properties in the geotechnical design for tip penetration.



Figure 5.3 Typical I-wall Configuration

The minimum sheet piling type shall be hot rolled PZ–27. However, I-walls within the levee tie-ins may have as a minimum a hot rolled PZ-22.

The sheet pile shall be adequately anchored into the concrete stem to resist pull out. A minimum embedment of 2'-9" shall be used on PZ-35 or smaller sheet pile. Bond development shall be checked for larger sheets. The projected area of the sheet piling shall be sufficiently embedded to develop bond between the piling and concrete cap adequate to resist the moment couple force. Additionally, U-bars shall be passed through existing handling holes or by burning holes in the sheet pile.

I-wall sheet pile shall be designed such that settlement is limited to an acceptable amount and differential settlement is negligible. Settlement of the cap should be less than 6 inches. Deviations shall be approved in advance by the USACE engineer of record. Concrete capping of walls shall be delayed in levees with anticipated settlement until movement has subsided. In the interim, the sheet piling shall be extended to the project Design Grade.

I-walls have been limited to 4 ft. of stick-up. This limitation negates the need to address horizontal displacements except as required for tension cracks addressed in the Geotechnical Section 3.

5.3.3 I-wall Sheet Piling Tip Penetration

See the Geotechnical Section of this document for sheet pile tip penetration requirements for I-walls.

5.3.4 Reinforced Concrete Section

It is recommended that all I-walls shall be at least 2 ft. thick. There shall be a minimum 6" of concrete clear cover beyond the sheet piling section.

5.4 Temporary Retaining Structure (TRS) Design Criteria

A TRS is used for braced excavation construction purposes. The TRS design is the responsibility of the contractor but shall be submitted for approval. Where applicable, construction live loads shall be considered in the TRS design; a common minimum is 200 pounds per square foot. Actual equipment loads shall be verified and used. For braced excavations constructed in water, only hot-rolled piling shall be permitted. Boat impact shall be applied where applicable unless protective marine fenders are included in the TRS design.

5.4.1 General Notes (Flood Protection)

TRS walls that serve as interim flood protection must comply with interim design guidelines dated 20 April 2006 and supplemented with Phase 1 design criteria dated 7 Feb 2007.

Areas below the required flood protection elevation will be considered breaches in the protection. Contractors will be permitted to allow an area in the existing flood protection to fall below the required elevation provided that area can be closed with steel sheet piling in a maximum of forty-eight (48) continuous hours. The length of the breech shall not exceed 300'. The interim protection shall be built to the lesser of the height of the adjacent levee/floodwalls or the 100 year (2011) Still Water Level (mean surge).

The sheet pile materials for closing such breaches shall be stockpiled at the site. Plans for closing breaches in the floodwall shall be updated periodically to reflect the status of construction progress.

The Contractor shall develop and submit for approval, plans, including methods, equipment, materials and actions to close breaches in the event that an impending storm or high water event threatens the area. Prior to removing any existing flood protection, the Contractor shall have the plan of interim protection approved.

The option or requirement to flood an excavation during a potential flood event may be used.

5.4.2 Sheet Piling Section (for Non-Flood Protection)

Design the steel sheet piling, using the moments and shears obtained from the geotechnical design for tip penetration, with allowable steel stresses, $F_b = 0.65 F_y$ and $F_v = 0.40 F_y$.

If archweb "U" piles are used, then the design shall account for and include calculations for shear transfer across their interlocks. Arch web piles or piles with interlocks at or near their center of gravity tend to slip under loading when the shear transfer cannot be achieved across their interlocks. Arch web piles shall be designed in accordance with the recommendations set forth in the standard CUR 166 published in 1993 in Holland by the Center for Execution, Investigations and Standardization in Civil Engineering (CUR), available from New Orleans District, Corps of Engineers, ED-T. Anti-slipping connections such as welding or crimping of the interlocks can be employed to help prevent displacement of the interlocks. The design calculations shall include all assumptions and shall consider the type(s) of soil, the effects of water, type of wall (i.e. cantilevered versus braced and shall include the location and number of wales, struts, etc), whether the piles are driven singly, in pairs, triple, etc., effects of phased excavation, treatment of the interlocks (i.e. how shear transfer is accomplished through welding or crimping), references cited, and any other considerations.

5.4.3 General Notes (for Non-Flood Protection)

Design steel struts, tie rods and steel wales using the maximum forces obtained from the unfactored geotechnical design and the latest AISC industry standards.

Design the anchors and deadmen, using the maximum anchor forces obtained from the factored geotechnical design and the latest AISC and ACI industry standards.

5.4.4 References

- "Steel Sheet Piling Design Manual", United States Steel Corporation
- "Steel Sheet Pile Design Manual", Pile Buck Inc.
- "Engineering Manual for Sheet Pile Walls", Virginia Tech Department of Civil Engineering
- "Design of Sheet Pile Walls", USACE Engineering Manual EM 1110-2-2504
- "CUR 166", published in 1993 in Holland by the Center for Execution, Investigations and Standardization in Civil Engineering (CUR) ('Dammwandconstructies' Civieltechnisch Centrum Uitvoering Research en Regelgeving, Holland

5.5 Reinforced Concrete Design Criteria

5.5.1 Structural Concrete

 $f_c' = 4000$ psi minimum – 28 day compressive strength (except concrete piles) or 90 days if pozzolans are used to replace cement. (3000 psi may be used for incidental structures or if heat control is required).

 f_c ' = 5000 psi minimum (prestressed concrete).

Thermal considerations: Slab and wall components that are greater than 4 feet thick shall require a thermal analysis. A simplified Level 1 analysis, as specified in ETL 1110-2-542 (dated 30 May 97), will suffice. A low-heat mix shall be included in the project specifications when analysis proves thermal stresses are elevated. A low-heat mix can be achieved by replacing the chirt aggregate with limestone; the larger the aggregate size the better. Additionally, replace the cement content with as much pozzalan as possible. Not all flyash and slags reduce heat. The most benefical are Class F flyash and Grade 120 ground granulated blast-furnace slag.

5.5.2 Steel Reinforcment

Steel reinforcing shall be ASTM A615 Gr. 60 with $f_y = 60$ ksi (Designs utilizing $f_y > 60$ ksi are not allowed). Reinforcement shall comply to ASTM A706 when welding is required.

Steel reinforcing for prestress concrete shall be Grade 270 strands (270,000 psi).

5.5.3 Load Factors

Reinforced concrete hydraulic structures must follow Corps criteria (EM 1110-2-2104). EM 1110-2-2104 procedures are referenced to the load factors and strength reduction factors found in ACI 318-1999.

Single Load Factor of 1.7 for dead and live loads shall be used in addition to a Hydraulic Factor.

Hydraulic Factor of 1.3 shall be applied to both shear and moment. The hydraulic factor is used to improve crack control in hydraulic concrete structures by increasing reinforcement requirements, thus reducing steel stresses.

Hydraulic Factor of 1.65 shall be used for member in direct tension. This includes base slab sections which have a net tensile stress resulting from load and pile reactions.

Strength reduction factor for bending shall be 0.9

Strength reduction factor for shear shall be 0.85

In accordance with paragraph 3-3 of EM 1110-2-2104, the capacity needed to resist diagonal shear is as follows:

$$V_{uh} = V_u \cdot H_f = V_u \cdot 1.3$$

For Concrete Shear Strength only: $\Phi \cdot V_c \ge V_{uh}$

For Concrete with Stirrups: If $\Phi \cdot V_c \leq V_{uh}$ then $\Phi \cdot V_s \geq V_{uh} - 1.3 \cdot \Phi \cdot V_c$

 V_u = ultimate factored shear force V_{uh} = ultimate factored shear force for hydraulic structures H_f = Hydraulic Factor V_c = nominal shear strength provided by concrete Φ = strength reduction factor V_s = nominal shear strength provided by shear reinforcement

This effectively reduces the stirrup reinforcement load factor in comparison to unreinforced concrete. This same method is used in ACI 350, paragraph 9.2.8.3. The reasoning is that the H_f is added for durability. Increasing the concrete section reduces the cracking thus minimizing rebar exposure to corrosion. Once the concrete is cracked, the stirrups are exposed thus the load factors revert back to those used in ACI 318, excluding the H_f . MVN typically does not include this reduction and will accept designs that provide shear reinforcement as:

$$\Phi \cdot V_{s} \geq V_{uh} - \Phi \cdot V_{c}$$

5.5.4 Steel Requirements

Maximum Flexural Reinforcement

 $\begin{array}{l} 0.25 \ \rho_b \ (Recommended) \\ 0.375 \ \rho_b \ (Permitted \ w/o \ special \ studies) \\ \rho_b = balanced \ steel \ ratio \end{array}$

Minimum Flexural Reinforcement ACI Code

Temperature Reinforcement 0.0028Ag (1/2 in each face)

5.5.5 Concrete Requirements

Clear Cover (except for channel lining) (Also see Section 12.0 – Typical Drawings):

- 2" min. for concrete sections equal to or less than 12" in thickness.
- 3" min. for concrete sections greater than 12" and less than 24" in thickness.
- 4" min. for concrete sections equal to or greater than 24" in thickness and when concrete is placed directly in contact with the ground.

Minimum Wall Thickness:

- T-walls = 18" minimum (for impact loads less than 50 kips)
- T-walls = 24" minimum (for impact loads 50 kips or greater)
- L-walls and I-walls = the width of the sheet piling plus 12"

Tapered walls of varying thickness were developed to save in concrete volume. In the construction of these types of walls, it is much more labor intensive; therefore, if the depth of the wall varies less than 18 inches from top to bottom, the wall thickness should remain constant.

5.5.6 Lap Splices

See typical drawings and details in Section 12.0 for Lap Splice charts and notes.

Splices shall be staggered whenever possible. Otherwise, the ACI code shall be adhered to.

Mechanical Splices

- 1) Mechanical Connectors
- 2) Thermit Welding (Cadweld) (Only use when necessary)
- 3) Welding (Never to be used)

When using mechanical splicers, do not add the coupling device to a short bar (usually equal to the lap length) that in turn laps to a long length. This creates two lap splices at the same location. Lap splices should be held to a minimum. When staggered in accordance with Class A requirements, mechanical splices shall develop 1.25Fy of the rebar, when not staggered the splice shall provide 1.50Fy.

5.5.7 Prestress Concrete

Prestress structural concrete (except piles) shall be approved in advance by the USACE engineer of record.

Prestress concrete piles are permitted in foundations resisting an unbalanced load provided the leading pile can resist 100% of the combined stresses, including those from the unbalanced load.

The piles combined axial and bending allowables for all unbalanced load cases are increased to $F_{\rm C} = 0.45 \cdot F_{\rm C}^{\prime}$ and $F_{\rm T} = 3 \cdot \sqrt{F_{\rm C}^{\prime}}$; For all other loading cases, the allowables are $F_{\rm C} = 0.40 \cdot F_{\rm C}^{\prime}$ and $F_{\rm T} = 0$

5.5.8 General Notes

In a base slab where 3 or more pile rows are present, it is recommended that primary and secondary reinforcing steel be placed above piles when possible.

When primary steel is placed above embedded piles, temperature steel shall be placed in the thickness of concrete below the primary steel (typically 12 inches). The temperature steel requirement is based on the depth of concrete below the primary steel, not the total depth of concrete.

5.6 Miscellaneous

5.6.1 Material Unit Weights

MATERIAL	UNIT WT (lb/ft ³)
Water	(62.4 to 64.0)
Reinforced Concrete	150
Steel	490
Rip rap	132
Semi-Compacted Granular Fill	120
Fully-Compacted Granular Fill, Wet	120
Fully-Compacted Granular Fill, Effec	tive 58
90% -Compacted Clay Fill, Wet	110
90% -Compacted Clay Fill, Effective	48

The unit weight of water shall be dependent on the salinity content at the project site.

The unit weights of soils are the minimum required based on many years of soil classification and testing in the region.

5.6.2 Loading Considerations

1) Concrete

- Unit weight of monolith
- Neglect weight of stabilization and tremie slab when beneficial to the foundation loading (i.e. uplift)

2) Water and Waves.

- SWL Elev. (Hydrostatic pressure)
- Wave Loading (exclude the water weight due to the wave weight above the SWL when designing the foundation)
- In designing the foundation for wave loading, the total resultant force, at its point of application, shall be utilized as provided by the hydraulic engineer.
- Due to the empirical nature of the formulas used in deriving the resultant wave force, it may not produce the same resultant if one was to use the derived pressure diagram. These differences are typically negligible; however, the design engineer shall use sound engineering judgment when applying these criteria. For a definition sketch of the GODA formulation for computing wave forces, see Figure 1.5 in Chapter 1.0 HYDRAULICS.

3) Soil

- Vertical Use Unit Weight
- Horizontal Use Unit Weight and K_o (at-rest lateral earth pressure coefficients) $K_o = 0.8$ for clay $K_o = 0.5$ for granular materials $K_o = 0.5$ for rip rap

4) Wind

- The wind load shall be computed in accordance with the following simplified formula, $F = .00256 (V^2)(I)(A)$, where (V) is the minimum wind velocity, (I) an Importance Factor of 1.15, and (A) the gross area exposed to the wind. See paragraphs 5.2.1 and 5.7 for additional guidance.
- Minimum design wind pressure for hydraulic structures shall be 50 psf.
- Wind Load shall be combined with other load cases so that it produces the most unfavorable effect.



Examples of Uplift Cases.

5) Uplift

Uplift pressure is comprised of position pressure and seepage pressure. Position pressure is based on the hydrostatic head due to the difference in water elevation and the base of the structure. Seepage pressure is a function of the equipotential flowline caused by the difference in water elevation between flood side and protected side, including the effectiveness of the sheet pile cut-off.

- Impervious sheet pile cut-off, 100% effective
- Pervious sheet pile cut-off, slopes uniformly along base from flood side uplift at flood side edge of base to protected side uplift at protected side edge of base

5.6.3 Structural Steel Design

Minimum steel thickness = 5/16" (For gate skin plates, 1/16" of thickness is added for corrosion, the 5/16" total thickness represents a minimum design thickness of 1/4" + 1/16" sacrificial thickness for corrosion control.) See EM 110-2-2105, Design of Hydraulic Steel Structures, for additional guidance. The 5/16" minimum is applicable regardless of corrosion requirements and is applicable to both stainless steel and aluminum.

Allowable stress = 5/6 of AISC allowable stress

The ASD method shall be used. The LRFD design method may **not** be used for structural steel design.

The American Welding Society, AWS D1.5 (2002) code shall be used for fracture critical members.

Welded structures should be welded all around (seal welded). Welds shall be designed and not simply made full penetration as the cost and residual stresses imparted by unequal cooling are detrimental. Weld inspection and NDT shall be made part of the contract requirements.

5.6.4 Steel Sheet Pile Design

$$\begin{split} F_b &= 0.5 \ f_y \\ F_v &= 0.33 \ f_y \\ F_a &= 5/6 \ AISC \ allowable \end{split}$$

Non-flood protection TRS allowables can be found in Section 5.4.2.

Thickness = 0.375 in. minimum

Only hot-rolled steel sheet piling sections are allowed.

5.6.5 Gate Design

5.6.5.1 Concrete Monolith

For the foundation design of most of the gate monoliths in our flood protection system, standard practice for the pile layout is to use battered piles to resist the horizontal loads at the columns and use vertical piles to resist vehicular and railway loads in the center of the monolith. Engineering judgment shall be used to determine the zone of influence to resist the horizontal loads in respect to battered pile placement. Where unbalanced loads are present in the foundation design, battered piles may also be required in the center. Low unbalanced loads may also be transferred to the end walls where battered piles are concentrated.

5.6.5.2 Steel Gates

Gates 12 feet tall or less may utilize a two girder system. The gates are considered low head and need not comply with Fracture Critical Requirements. Girder splices are not recommended, but when approved the splice shall be NDT tested along 100% of the length. Stress levels and deflections shall limit the girder capacity. Stress levels about the major axis shall be kept below 0.5 Fy and combined stresses about both axis maintained below 90% of unity.

Gates taller than 12 ft. \leq to 16 ft. may also utilize a two girder system, but must meet all fracture critical criteria for a hydraulic steel structure. Fracture critical requirements are specified in ER 1110-2-8157. Non-redundant tension members shall comply with AWS D1.5 and 100% of welded tension connections shall be NDT tested, including all plates and stiffeners welded to the tension flange of both girders. Splices in the critical horizontal girders are prohibited.

Gates taller than 16 ft. shall utilize three girders. At the hinge column, the third girder shall transfer the lateral load to the column through an additional hinge. For welded connections, AWS D1.1 is adequate; however, splices in the critical horizontal girders are prohibited.

Roller, swing or miter type floodgates, placed in "dry" conditions, subject to atmospheric surroundings, shall be painted with a vinyl paint system (V-766e). Floodgates, such as sluice type gates, placed in "wet" conditions, subject to tidal and/or splash zones shall be painted with a coal tar epoxy paint system (C 200a).

<u>Roller Type Gates</u>. Consideration should be given to the design of the gate in respect to rolling the gate into placement. New gates may be very large and will pose concerns when the gate is moved into position. Roller gates shall be used when the clearance requirements within the closure swing cannot be guaranteed. Typically, if the skin plate is placed on the flood side of the girders, gate uplift can become a issue. The location of the bottom seal is critical in reducing flotation. Flotation Factors of Safety shall comply with EM 1110-2-2100, Table 3-4.

<u>Swing Type Gates</u>. The use of three hinges or extension of columns and tension supports should be considered for gates that are very large in height. The top hinge tends to bind when moving gates that are very heavy. Adjustable bottom seals shall be added where slight variations in sill height could occur (i.e. road pavement topping improvements).

<u>Overhead Roller Type Gates</u>. The use of this type of gate shall be of last resort as it creates an overhead obstruction. If there are no problems with swing tolerances, then we recommend using a swing gate.

<u>Miter Type Gates</u>. The latching of the gates after placed into the closed position is very critical for the proper function of the miter gate. A latching system should be investigated if miter gates are being considered. The latch shall resist all applicable design hurricane protection design cases.

<u>Sector Gates</u>. Sector gates should be investigated for a full range of possible hydraulic and operational loads, as are typically determined by hydraulic analysis. Effects of "Reverse Head" loading should be included in the design, where applicable.

<u>Sluice Gates.</u> Cast iron is the preferred material for sluice gates used in hurricane flood protection projects; however, stainless steel may also be utilized. If stainless steel is used in the design of a sluice gate, the gates shall be furnished in one piece. All members of the gate shall have a minimum thickness of 5/16" (including wall embedments) and no intermittent welds shall be allowed. Depending on the type of stainless steel used, appropriate reductions in the yield stress of the base metal in the heat affected zone (HAZ) should be taken per AWS D1 and other applicable industry standards.

<u>Slide Gates.</u> Slide gates are typically used in light duty applications involving the control of channel flow and are not recommended for use in hurricane flood protection applications.

5.6.6 General Design Considerations

Where levees will be raised or new embankment constructed, the adverse effects of foundation consolidation must be considered which includes drag forces on both the sheet pile cut-off and support piles. In addition, these drag forces must be considered in settlement calculations.

Where non-displacement piles are required and corrosion is not a controlling factor, consider H-piles or pipe piles; otherwise, investigate the use of prestressed concrete piles which are typically more cost effective.

5.6.7 Utility Crossings

For a structural alternative on utility crossings, the utility shall only be allowed to pass through a pile founded L-Wall or T-Wall. Utilities should pass through a properly sealed pipe sleeve in the cut-off sheet piling. See Section 12.0 for typical examples and utility clearances.

On case-by-case bases, utilities may pass through the concrete wall and in general, shall not be attached, sleeves are mandatory. Only metal (steel or iron) sleeves and carrier pipes shall be permitted to penetrate the wall; no plastic or PVC. Shut off valves are required on all gravity flow pipelines and shall be placed on the protected side. See Section 12.0 for typical examples and utility clearances.

All Utility Crossings shall approved by the USACE engineer of record. See Section 3.6 for other utility crossing options and pipe support requirements.

5.6.8 Corrosion Protection

Steel pile lengths driven in undisturbed soils, where they are not exposed to air or moisture, <u>or</u> are located beneath the low water table stages, do <u>not</u> require corrosion protection, since the rate of corrosion decreases rapidly without the presence of oxygen.

Steel sheet, H and pipe piling that will be installed in new fill, disturbed materials, exposed to air or fluctuating water tables (splash zone), embedded in undisturbed but pervious coarse-grained soils above the low water table, <u>or</u> beneath slabs where soil subsidence can occur exposing the piling to air and moisture, shall be protected from corrosion using one of the following methods:

- 1. Painting with an approved coal tar epoxy system.
 - This is the preferred method and should be considered first unless there are cogent reasons for using the sacrificial thickness method below. The steel piling shall be painted 3 inches above the base slab and to a 5 ft. minimum below new fill material, disturbed soil or the lowest elevation of the fluctuating water table. Piles exposed in tidal zones and/or splash zones, such as steel piling in breakwaters or dolphins shall be coated the full length exposed to the tidal zone plus an additional 5 ft. of length. Only coal tar epoxy (C 200a) is approved for use at this time.
- 2. Addition of sacrificial thickness.
 - a. Steel piling exposed to atmospheric conditions or new/disturbed soil.
 - In lieu of painting, a thicker piling section may be provided to accommodate the future loss of material due to corrosion.
 - Steel pile thicknesses shall be increased by a minimum of 0.075 inches, for piling driven into new fill or disturbed materials, which will not be exposed to air or moisture due to soil subsidence beneath the base slab.
 - Steel pile thicknesses shall be increased from 0.10 inches to 0.150 inches, for piling that will be exposed to air or moisture, due to soil subsidence beneath the base slab. The minimum 0.075 inches is acceptable when soil conditions are found by testing to be not conducive to corrosion. For more aggressive exposure conditions, steel pile thicknesses ranging from 0.150 inches to 0.300 inches may be more appropriate. In choosing the required sacrificial thickness, factors such as the position of the piling or base slab with respect to the water table, the presence of oxygen and moisture, salinity levels and stress levels shall be considered.
 - b. Steel piling exposed to tidal conditions and/or splash zones.
 - In <u>addition</u> to painting, use of marine grade steel (ASTM A690) or a thicker piling section may be provided to accommodate the future loss of material due to corrosion in tidal and splash zones

- For piling exposed to freshwater, steel pile thicknesses shall be increased by a minimum 0.100 inches. If a concrete core is provided, the sacrificial thickness may be reduced to 0.050 inches.
- For piling exposed to brackish water (chlorides <2000 ppm), steel pile thicknesses shall be increased by a minimum of 0.150 inches. If a concrete core is provided, the sacrificial thickness may be reduced to 0.075 inches.
- For piling exposed to salt water (chlorides >2000 ppm), the increased pile thickness method is not allowed, marine grade steel is recommended along with the specified coal tar coating.
- c. Typical H-piling Thicknesses

HP 14x73 (0.505 inches) HP 14x89 (0.615 inches) HP 14x102 (0.705 inches) HP 14x117 (0.805 inches)

d. Typical Pipe Piling Thicknesses

Typical wall thicknesses include (0.250", 0.375", 0.500" and 0.625")

e. Typical Sheet Piling Thicknesses

PZ 22 (0.375 inches) PZ 27 (0.375 inches) PZ 35 (0.500 inches) PZC 14 (0.420 inches) PZC 19 (0.420 inches) PZC 28 (0.570 inches)

- Sheet piling used as seepage cut-off has no structural requirement. The minimum thickness after a 50 year design life shall not be less than 5/16 inch provided the section has sufficient shear capacity to meet any drag down requirements due to soil subsidence beneath the base slab.
- 3. Addition of cathodic protection.
 - Although cathodic protection may be an effective method of corrosion control, it is not recommended for use as the primary corrosion inhibitor in floodwall applications. It should be noted that these systems are only partially effective in tidal zones and offer no protection in the splash zone.

- 4. Soil testing and salinity levels.
 - Corrosion rates shall be derived from existing steel structures located in the same vicinity and exposed to a similar environment. In the absence of historical corrosion data, soil testing shall be performed to determine the level of the corrosive properties of the soil. Common corrosion indicators are salinity, and resistivity. Typically, greater sacrificial thicknesses are required when soil salinity levels (chlorides) exceed 2000 ppm and soil resistivity is less than 2000 ohm-cm. When testing is performed, salinity levels in soils shall be sampled at intervals not to exceed 1000 ft.

5.6.9 Levee Tie-ins, Transitions and Scour Protection

Typical scour protection details can be found in Appendix C.

ERDC Overtopping Protection can be found in Appendix D. It shall be used to determine whether the minimums set in Appendix C are adequate.

Proper engineering judgment and settlement considerations shall be used to determine the proper level of scour protection. Scour protection materials and details should be properly engineered and suitable for the specific site location. Scour protection on the flood side should be considered on a case-by-case basis, especially if hurricane wave loading exists.

95% compaction of the scour protection sub-base shall be considered to minimize settlement. The structural backfill shall be fully compacted, fertilized and seeded where concrete armor is not applied.

Scour protection is required on the protected side of all I-walls and L-Walls. Scour protection is also required on the protected side of T-walls that include a stability berm. Scour protection shall transition a minimum of 10' into any adjacent T-wall sections then curve inward at a radius equal to that of the protection width.

Proper earthen cover and scour protection are mandatory. Future settlement should be accounted for in detailing scour protection over the sheeting piling.

Typical MVN details should be used for transitions from L-Wall or T-wall to T-wall, L-Wall or T-wall to I-wall and L-Wall or T-wall to uncapped sheet piling (slip joint). See Section 12.0 for typical drawings.

The tie-in details for T-Walls, L-Walls and I-walls that terminate into a levee section must follow the latest guidance. As a minimum, the uncapped cut-off sheet piling must extend horizontally 30 feet into the full levee section for erosion and seepage control. In order to avoid seepage problems, the sheet pile tip elevations shall remain at the same tip elevation as the adjacent walls, unless seepage calculations determine that a lesser tip elevation is adequate. Seepage analysis shall be performed by either the Harr, Line-of Creep Methods, or flownets.

A minimum hot rolled PZ-22 or equivalent shall be used at all levee tie-ins.

5.6.10 Expansion Joints.

To prevent concrete from crushing from movement caused by expansion, expansion joints are placed between abutting concrete monoliths. Typically L-wall or T-wall monoliths should be 40 to 60 feet in length between expansion joints, where I-walls are typically 30 to 40 feet. At these lengths, a 1/2 inch joint should be adequate for expansion. Expansion joints are also placed to isolate structural elements from each other such as concrete slope pavement. Expansion joints shall be made the full depth of the concrete. Dowels may be used across expansion joints to resist undesirable lateral or vertical movement of concrete elements where necessary. For additional guidance, see EM 1110-2-2102, Waterstops and Other Preformed Joint Materials for Civil Works Structures.

5.6.11 Waterstops.

Waterstops are primarily embedded in the monolith joints of hydraulic concrete structures such as floodwalls and control structures, to stop the passage of water through the joint. Waterstops shall be nonmetallic. Nonmetallic waterstops are manufactured in a variety of shapes; however, a 9 inch, 3 bulb PVC waterstop is typically used in the joints for floodwalls. The center bulb shall be hollow which allows for a wider range of movement in both the tranverse and lateral directions. For additional guidance, see EM 1110-2-2102, Waterstops and Other Preformed Joint Materials for Civil Works Structures.

5.7 Basic Load Case Tables

Loadings covered by the following table represent those typically encountered during construction and hurricane storm events. Fire, blasts and other similar accidental loadings are considered extreme loads not directly associated with hurricane events and as such are not included. However, designers should consult the designer of record for guidance on the applicability of these extreme loadings to a given structure based on its criticality to the system and its exposure/vulnerability to such loadings. In most cases, conventional methods such as increased safety and/or security measures of critical components can be implemented more feasibly than adjusting designs for the increased loading. It is important to note that these tables are not inclusive of all possible load case combinations. The designer of record shall assess and document any additional load cases in the Design Documentation Report (DDR).

LC	Overs Allo	stress wed	Load Case	Description
No.	Struc.	Fndn.	Name	
BLC 1a	16⅔ %	16⅔ %	Construction	Dead load 200 psf equipment surcharge No uplift No wind
BLC 1b	33⅓ %	33⅓ %	Construction plus Wind	Dead load No unbalanced load No uplift Wind from protected side
BLC 2a	0	0	Water to SWL (Impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind ² No boat/barge impact
BLC 2b	0	0	Water to SWL (Pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off No wind ² No boat/barge impact
BLC 2c	331⁄3%	33⅓%	Water to SWL, plus Wind, (Impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off Wind load
BLC 2d	33⅓%	33⅓%	Water to SWL, plus Wind, (Pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off Wind load
BLC 3a	331⁄3%	33⅓%	Water to SWL, plus Wave Load (Impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind Wave load applied
BLC 3b	331⁄3%	331⁄3%	Water at SWL plus Wave Load (Pervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind Wave load applied

LC	Overs Allo	stress wed	Load Case	Description
No.	Struc.	Fndn.	Name	
BLC 4a	50%	33⅓%	Water to SWL plus Wind Load plus Unusual Barge Impact (Impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off Wind load applied See "Boat/Barge Impact Loading Tables & Maps"
BLC 4b	50%	33⅓%	Water to SWL, plus Wind Load, plus Unusual Barge Impact (Pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off Wind load applied See "Boat/Barge Impact Loading Tables & Maps"
BLC 5a	0%	0%	Water to Reverse Head, (Pervious or Impervious)	Dead load Unbalanced load (if present) Pervious or Impervious sheet pile cut-off No boat/barge impact
BLC 5b	331⁄3%	33⅓%	Water to Reverse Head, plus Wind (Pervious or Impervious)	Dead load Unbalanced load (if present) Pervious or Impervious sheet pile cut-off No boat/barge impact Wind load
DRC A	33⅓%	33⅓%	Water to Top of Wall (IA) or 500 YR SWL Plus Wave (IB)(Pervious or Impervious)	Dead load No unbalanced load Pervious or impervious sheet pile cut-off No wave load No wind load No boat/barge impact load
DRC B	50%	50%	Water to Top of Wall (IA) or 500 YR SWL Plus Wave (IB)Pervious or Impervious)	Dead load With unbalanced load Pervious or impervious sheet pile cut-off No wave load No wind load No boat/barge impact load

LC	Overs Allo	stress wed	Load Case	Description		
No.	Struc.	Fndn.	Name			
DRC C (Zone IA)	Note 4.	Note 4.	EXTREME - Case I Barge Impact, plus hydrostatic loading to SWL, plus wind, (Impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wave load Wind load See "Boat/Barge Impact Loading Tables & Maps"		
DRC D (Zone IA)	Note 4.	Note 4.	EXTREME – Case II Barge Impact, plus hydrostatic load to SWL, plus wave, plus wind, (Pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off Wave load Wind load See "Boat/Barge Impact Loading Tables & Maps"		

NOTES:

1. In cases where a dolphin or other structure will resist barge impact, only the minimum debris impact load of 0.5 kips per foot shall be applied to the wall design.

2. If wind load is applied, a 33¹/₃% overstress is allowed for SWL and RH cases.

3. In designing the pile foundation, the vessel impact load shall be located in the center of the monolith. Under this concentric loading, stress levels shall comply with Tables 5.1 and 5.2. As a design resiliency check, the foundation shall be analyzed with the vessel impact occurring 5 ft. from the vertical monolith joint. On unsymmetrical monoliths, the loading shall be checked at both monolith joints. Under this extreme load case condition, the factor of safety shall be equal to or greater than unit.

4. For applicable LRFD load factor combinations see Table 5.2.



TYPICAL LOADING – BASIC LOAD CASE 4b (No Unbalanced Load)

5.8 Impact Barriers (Dolphins, Pile Clusters, etc.)

Floodwalls, of any type, should be avoided along major navigation routes. Where unavoidable, the floodwall shall be designed using the appropriate load conditions included in this document. Where floodwalls cannot be designed, for the required design load, impact barriers such as sheet pile dolphins or pile clusters, shall be constructed in front of the floodwall to provide protection.

Impact barriers shall be stand alone structures whose sole function is to provide protection from impact, with no capacity to transfer load to adjacent structures. The minimum top of impact barrier shall be constructed to the greater of:

- 7 ft. above the 100-yr SWL
- The Top of Structure based on the 1 % wave overtopping (excluding Structural Superiority). Where the protected structure Top of Wall (TOW) exceeds the 500-yr SWL. The impact barrier shall be constructed no higher than 7ft. above the 500-yr SWL.

Earthen impact barriers may be used, but shall be constructed to the Still Water Level, at a minimum when located in Zone 1A.

Impact Barrier - Design load Cases:

a. USUAL. A minimum 100 kip will be applied at the top of the barrier. No overstress is permitted. The water stage shall be the mean annual high water level. Impact shall occur at the lower of one foot below the dolphin, or 7 ft. above the mean annual stage.

b. UNUSUAL. A 160 kip impact force shall be applied at the lower of one foot below the top of the impact barrier, or 7 ft. above the SWL. This load case shall include the force of the wind on the exposed portion of the impact barrier above the SWL. The wind load, to be used in conjunction with the impact analysis, shall be computed in accordance with the following simplified formula, $F = .00256 (V^2)(I)(A)$, where the minimum wind velocity (V) shall be 140 mph, utilizing an Importance Factor (I) of 1.15, applied to the gross exposed area (A). The permitted pile capacity Factors of Safety and structural component overstresses are provided in Table 5.2 (a).

c. EXTREME. A 300 kip impact force shall be applied at the top of the impact barrier. The load case shall include the force of the wind on the exposed portion of the dolphin above the SWL. The wind load, to be used in conjunction with the boat impact analysis, shall be computed in accordance with the following simplified formula, F = .00256 (V²)(I)(A), where the minimum wind velocity (V) shall be 160 mph, utilizing an Importance Factor (I) of 1.15, applied to the gross exposed area (A). The permitted pile capacity Factors of Safety and structural component overstresses are provided in Table 5.2 (a).

The impact barrier capacity shall be based on a push-over analysis. In lieu of a push-over analysis, capacity can be determined by an elastic analysis of all structural components and limiting the pile capacity Factor of Safety and structural components Load Factors to those designated in Table 5.2 (b) An example of the push-over analysis is attached as an Appendix.

Where floodwalls are protected by significant structures such as pile founded wharves and warehouses or significant tree lines, impact barriers will not be required. An inspection report of the facility or site, to determine its integrity, shall be submitted along with a waiver request to the Chief of Structures at Division (MVD) for approval. Any structure or tree line used as justification for not designing for impact shall be inspected on an annual basis.

5.9 Loading Tables & Impact Maps

Impact loads for barges, boats and debris shall be considered as shown in the following tables and maps.

The maps show the <u>minimum</u> impact load to be applied to wall designs along the various reaches. If the impact loads expected are higher than shown, research into the appropriate impact loading for the each design should be performed.

The barge impact load is considered to act concentrically on the structure. The foundation is checked for an eccentric load as stated in Note 3 of Table 5.1. There is sufficient lateral reinforcement to distribute the impact over the monoliths. Any impact that would occur immediately at the monolith joint would create localized concrete damage near the top of wall, as witnessed in Katrina, and not lead to catastrophic failure.

<u>Table 5.2 (a)</u>

HSDRRS – IMPACT BARRIER BASIC LOAD CASES (BLC)								
	ALLOWABLE	OVERSTRESS	PILE LOAD - FACTORS OF SAFETY (FOR Q-CASE)					
LOAD CASE	STRUCTURE	FOUNDATION	STATIC LOAD TEST		PDA LOAD TEST		NO LOAD TEST	
		PILES	С	Т	С	Т	С	Т
I. BARGE IMPACT (USUAL)	0 %	0 %	2.00	2.00	2.50	3.00	3.00	3.00
II. BARGE IMPACT (UNUSUAL) + WIND	50 %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
III. BARGE IMPACT (EXTREME) + WIND	**	**	1.15	1.15	1.40	1.40	1.70	1.70

** Both concrete and steel designs shall utilize the LRFD methods of analyses. The strength reduction factor Φ shall comply with ACI and AISC codes. The hydraulic factor (H_f) shall equal 1.0. The applicable load factor combination is:

EXTREME CASE = 1.0 (DL + WIND) + 1.0 (BARGE IMPACT)

<u> Table 5.2 (b)</u>

HSDRRS - BASIC LOAD CASES (BLC)								
	ALLOWABLE OVERSTRESS		PILE LOAD - FACTORS OF SAFETY (FOR Q-CASE)					
LOAD CASE	STRUCTURE	FOUNDATION	STATIC LOAD TEST		PDA LOAD TEST		NO LOAD TEST	
	& GATES	PILES	С	Т	С	Т	С	Т
I. CONSTRUCTION	16⅔ %	16⅔ %	1.70	1.70	2.15	2.60	2.60	2.60
II. CONSTRUCTION + WIND	331⁄3 %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
* III. STILL WATER LEVEL (SWL)	0	0	2.00	2.00	2.50	3.00	3.00	3.00
* IV. SWL + WIND	331⁄3 %	33¼ %	1.50	1.50	1.90	2.25	2.25	2.25
* V. SWL + WAVE	33⅓ %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
* VI. SWL + WIND + (UNUSUAL) BARGE, BOAT or DEBRIS IMPACT	50 %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
* VII. REVERSE HEAD (RH)	0	0	2.00	2.00	2.50	3.00	3.00	3.00
* VIII. RH + WIND	331⁄3 %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
* IX. RH + WAVE	331⁄3 %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
* X. RH + WIND + (UNUSUAL) BARGE, BOAT or DEBRIS) IMPACT	50 %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
* Note: If unbalanced load is pre	esent for SWL (10	00-yr) or RH load (cases, it sh	all be inclu	uded in all	SWL load	case comb	inations.

HSDRRS - DESIGN RESILIENCY CHECKS (DRC)								
	ALLOWABLE	OVERSTRESS	PILE LOAD - FACTORS OF SAFETY (FOR Q-CASE)					
	STRUCTURE	FOUNDATION	STATIC LOAD TEST		PDA LOAD TEST		NO LOAD TEST	
	& GATES	PILES	С	Т	С	Т	С	Т
I. WATER to TOW, W/O UNBALANCED LOAD + NO WAVE LOAD	33⅓ %	33⅓ %	1.50	1.50	1.90	2.25	2.25	2.25
II. WATER to TOW, W/ UNBALANCED LOAD + NO WAVE LOAD	50 %	50 %	1.33	1.33	1.67	2.00	2.00	2.00
III. SWL (500-yr) + WAVE LOAD, W/O UNBALANCED LOAD	50 %	50 %	1.33	1.33	1.67	2.00	2.00	2.00
IV. SWL (500-yr) + WAVE LOAD, W/ UNBALANCED LOAD	67 %	67 %	1.20	1.20	1.50	1.80	1.80	1.80
V. EXTREME – CASE I (Zone 1A) BARGE IMPACT + SWL (100-yr) + WIND	**	**	1.15	1.15	1.40	1.40	1.70	1.70
VI. EXTREME – CASE I (Zone 1B) BARGE IMPACT + SWL (500-yr) + WIND	**	**	1.15	1.15	1.40	1.40	1.70	1.70
VII. EXTREME – CASE II (Zone 1A & 1B) BARGE IMPACT + SWL (100-yr) + WAVE	**	**	1.15	1.15	1.40	1.40	1.70	1.70

NOTE: LC III and IV are applicable only where the SWL (500-yr) is below the TOW.

** Both reinforced concrete and structural steel designs shall utilize the LRFD methods of analyses. The strength reduction						
factor Φ shall comply with ACI and AISC codes. The hydraulic factor (H _f) shall equal 1.0. The applicable load factor combinations are:						
Zone 1A						
EXTREME - CASE I = 1.0 (DL + H_s + WIND) + 1.0 (BARGE IMPACT)	H_{S} = (100-yr) SWL Hydrostatic Force					
EXTREME - CASE II = 1.0 (DL + H_S + BARGE IMPACT) + 1.2 (WAVE)	WAVE = (100-yr) Wave Force					
Zone 1B						
EXTREME - CASE I = 1.0 (DL + H_S + WIND) + 1.0 (BARGE IMPACT)	H_{S} = (500-yr) SWL Hydrostatic Force					
EXTREME - CASE II = 1.0 (DL + H_S + BARGE IMPACT) + 1.2(WAVE)	WAVE = (100-yr) Wave Force					

GENERAL NOTES:

1. Actual "unfactored" service loads shall be used in any pile analysis program.

2. An increase in allowable deflections will be allowed for overstress conditions. Sound engineering judgment shall be utilized in deciding the appropriate overstress. Deviation from deflections and overstress guidance shall be approved by the USACE engineer of record.

3. The Reverse Head load case is not a typical floodwall load case, but is typical for structures which contain sector or sluice type gates and/or are submerged.

4. Design Resiliency Checks (DRC) are not typical HSDRRS design load cases. These cases, where water is checked to the top of wall or an extreme SWL load case combination, were developed for design checks for the survivability of a structure.

NOTES ON IMPACT LOADS:

1. All walls outside locations of barge / boat impact zones, shall include a minimum debris impact loading of 0.5 kips/lf

2. Designs shall assume the appropriate barge impact load (See paragraph 5.2.1 Loading Conditions) where barge impact can occur now or in the future. Use a 50 kip load for other vessels, such as pleasure craft or work boats. Current obstructions that are marginal and have a high probability of not lasting the project life shall be assumed non-existent.

3. Wall load distribution. Boat/Barge impact loads shall be distributed over a 5 foot width plus the width gained along 45-degree angles in the vertical direction thus increasing the distribution downward towards the base. Minimum debris impact loads shall be applied across the entire wall/monolith length. As a design check, the boat/barge impact loadings shall be applied 5 feet from the edge of the monolith with a minimum Factor of Safety of 1.0 (no overstress) and submitted to the USACE engineer of record for review.

4. Foundation load distribution. Impact loads (Barge, Boat, Debris), shall be distributed over the full width of the monolith foundation at the appropriate elevation. As a design check, the boat/barge impact loadings shall be applied 5 feet from the edge of the monolith with a minimum Factor of Safety of 1.0 (no overstress) and submitted to the USACE engineer of record for review.

5. Gate load distribution. Boat/Barge impact loads shall be distributed over a 5 foot width on the upper girder. No load is assumed on the lower girder(s). Minimum debris impact loads shall be applied across the entire upper girder length.

NOTES ON STILL WATER LEVELS (SWL)

1. For BASIC LOAD CASE (BLC) COMBINATIONS, use the 1% (100-yr) SWL at a 90% confidence and the Unusual (100-yr) impact loads.

2. For DESIGN RESILIENCY CHECKS (DRS), use the 0.2% (500 yr) SWL, at a 90% confidence, if this elevation is "less" than the Top of Wall (TOW) elevation.



ST. CHARLES - JEFFERSON Figure 0.4 Boat/Barge Impact Map St. Charles – Jefferson Parish

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NEW ORLEANS Figure 5.5 Boat/Barge Impact Map New Orleans Area

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Figure 5.6 Boat/Barge Impact Map Plaquemines Parish

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WESTBANK Figure 5.7 Boat/Barge Impact West Bank

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