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Section 3

Structural

3.1 C43PS-1

3.1.1 General Description

C43PS-1 will consist of a 1500 cfs pump station with intake located on the Townsend Canal and an over-the embankment arrangement for discharge to the Reservoir. The facility will house four main pumps. Two pumps will have 500 cfs capacity, and two will have 250 cfs capacity. The pump station building will consist of an operating floor for the placement of most pump station machinery, and a two-story structure housing the control room, break room and workshop. Truck access to the operating floor level will consist of a 14' x 30' truck bay recessed 4' below the operating floor level located on one end of the building. A concrete access ramp rising from exterior grade to an overhead door at the operating floor level will be provided on the opposite end of the building from the truck bay. Space will be provided at the operating floor level for equipment laydown for in-station maintenance. A 60-ton overhead bridge crane with a hook height approximately 25' above the operating floor will be provided to facilitate the maintenance of pump station equipment. The selection a 60-ton capacity crane is based on the anticipated weight of one 500 cfs pump. The bridge crane will not be used to remove the entire pump during maintenance operations. It is expected, however, that the bridge crane will lift the full weight of the pump while the pump is shored and sections are removed for maintenance.

A pipe gallery floor will be provided below the operating floor and will be used to route piping and conduits across the pump station. This level will also be used to house the auxiliary pump systems, water treatment systems and collection tanks for waste fuel and waste oil. Access for the installation and removal of equipment at the pipe gallery level will be provided by means of a 10' x 10' opening through the operating floor at one end of the building. A concrete access ramp from exterior grade down to a 10' x 10' overhead door at the pipe gallery level will be provided at the opposite end of the building. Personnel access to the pipe gallery level from the operating floor will be provided by means of stairs located at one end of the building. Ladders and access ways through the operating floor will be provided at other locations across the station.

3.1.2 Substructure

Based on the findings of the Preliminary Geotechnical Design Memo for the C-43 West Storage Reservoir, the in-situ soils in the area of the pump station consist of approximately 20 feet of relatively clean sands, clayey sands and/or silty sands over approximately 20 feet of clay on top of clayey sands and relatively clean to silty sands.

The substructure of the pump station will consist of cast-in-place reinforced concrete walls supported on mat foundations. The suction basin level of the pump station will bear on the medium to dense sands that underlie the clay layer. Portions of the substructure will bear above the existing clay layer. Recommendations in the Preliminary Geotechnical Design Memo include over-excavating the clays under these portions of the substructure and replacing with granular structural backfill.

The substructure will be constructed with a formed suction intake (FSI) for each main pump bay. The final dimensions of the FSI will be determined based on the dimensions of the pumps selected for use at the station.

The main pumps will be supported by the pipe gallery floor. The pump supports will be embedded in the concrete floor in accordance with details recommended by the pump vendor.

3.1.3 Superstructure

The superstructure is defined as the building elements enclosing the operating deck of the pumping station. The roof structure will consist of cast-in-place concrete slab on metal deck supported by intermediate structural steel roof purlins. The purlins will be supported by rigid steel frames consisting of steel trusses and wide flange steel columns. The roofing system will consist of fully-adhered EPDM roofing applied to the concrete roof deck. The exterior cladding system will consist of precast concrete wall panels. Steel girts spanning horizontally between the rigid steel frames will be used to provide lateral support for the precast wall panels.

The bridge crane runways will be supported by steel haunches at each rigid frame column. The height of the superstructure will provide adequate clearance below the roof trusses for the station bridge crane, utilizing a hook height requirement of 25 feet above the operating floor level.

3.1.4 Siphon Break and Saxophone Discharge Structures

To facilitate the over-the-embankment arrangement, four siphon break structures and four saxophone discharge structures will be constructed. One set of structures will be constructed for each discharge pipe. The cast-in-place concrete siphon break structures will be provided where the discharge pipes cross the top of the dam embankment, and will be designed to permit the passage of vehicular traffic over the discharge pipes. The structures will consist of base slabs, wing walls and vaults with access manholes. The structures will be designed to be water tight, and will permit the observation of all portions of the discharge pipes where they cross beneath the crest of the dam.

The saxophone discharge structures will consist of sheet pile cofferdams with tremie concrete seals to allow for dewatering the excavations required for the installation of the steel saxophone discharge pipes. Concrete thrust blocks will be poured around each discharge pipe

prior to backfilling the remaining excavation. A cast-in-place concrete cap will be constructed on top of the sheet pile on three sides to create a sharp crested weir. The fourth side will be enclosed by the dam embankment. The purpose of the sharp-crested weir structure is to ensure that the discharge pipes are submerged to the minimum elevation required by the hydraulic design of the pump station.

3.1.5 Service Bridge

An 18'-0" wide cast-in-place Service Bridge will be provided at the suction basin of the pump station. The bridge will be used to provide vehicular access from one side of the pump station to the other, and for positioning truck cranes during the installation and removal of stop logs. The bridge will be designed as a one-lane multiple span deck. The deck will utilize the suction basin walls as support bents. The deck will be fixed at an interior wall, and free to expand at other walls.

3.1.6 Trash Screen Bridge

Trash screens and rakes for the pump station will be located on a pile-supported bridge structure located approximately 200' upstream of the pump station. The trash screens will consist of stainless steel bars, and will span from the inlet channel bottom to the deck of the bridge structure. The design of the trash screen bridge will be based on the SFWMD G-S6, Standard Guidelines for Single Lane Bridge.

3.1.7 Fuel Oil Containment

Fuel oil supply and waste fuel and oil storage tanks will be located within a cast-in-place concrete containment structure at one end of the pump station. The containment will be designed to accommodate the volume of the largest fuel storage tank. The containment will include a sump pit and a discharge valve to permit the drainage of rainwater. The discharge valve will be closed under normal operating conditions.

3.1.8 Stoplogs

One set of stoplogs will be stored at the station to allow for dewatering one bay of the pump station for maintenance. The stoplogs will be designed as steel panels extending vertically from the bottom of the suction bay to the top of the suction bay walls. The stoplogs will slide down recesses cast into the suction basin walls and training piers. The panels will consist of steel plate with channel stiffeners, lifting eyes and a perimeter neoprene gasket. All steel panels will be hot-dipped galvanized.

3.1.9 Design Load Criteria

Live Loads
The following live loads will be the basis for design:

Occupancy or Use	Live Load
Roof	50 psf
Floors	
Operating Floor (non equipment areas)	250 psf
Operating Floor (equipment placement areas)	250 psf*
Control Room	100 psf
Restrooms	100 psf
Equipment Storage Rooms	200 psf
Maintenance Work Area	300 psf
Stairways	100 psf
Elevator Lift	200 psf
Deck Grating	250 psf
Service Bridge	HS-20**
Guardrails	50 lb/ft and 200 lb concentrated load

*- Equipment placement areas will be designed to support the weight of the heaviest piece of machinery anticipated.

** - Service bridge design will also consider current SFWMD truck crane equipment loadings.

Wind Loads
The design wind loading for the structure will be in accordance with the Florida Building Code, and ASCE 7:

Wind Speed: 120 mph (3 second gust)
Exposure C
$I_w = 1.3$

A mean recurrence interval (MRI) of 200 years will be used in place of the standard 50 year MRI. A factor of 1.14 will be applied to the code wind pressure values to upgrade to the 200 year MRI values.

Crane Loads
Bridge crane wheel loading will be treated as live load for the crane runway design. The design of the runway will consider impact loads equal to 18% of the crane load, longitudinal loads equal to 10% of the crane load, and lateral loads equal to 10% of the sum of the rated capacity of the crane and the weight of the hoist and trolley.

Hydrostatic Uplift Loads
Hydrostatic uplift forces will be determined in accordance with procedures described in U.S. Army Corps of Engineers, Engineering Manual (EM) 1110-2-3104, "Structural and Architectural Design of Pumping Stations." Hydrostatic uplift forces will be considered when evaluating the stability of the structure from overturning, sliding, uplift and foundation bearing. Uplift loading will be based on recommendations provided by the design team Geotechnical Consultant. The following recommendations were given for preliminary design (all elevations refer to NAVD 88):

Reservoir Level – EL 42.0
Toe of Dam – EL 20.0 (bench at dam toe was modeled, with a transition to EL 16.0 away from toe)

Lagoon Level – EL 0.0	
Location	Hydraulic Grade Line EL
Dam Side of PS-1	+8.3
Lagoon Side of PS-1	+8.0

Lagoon Level – EL 8.0	
Location	Hydraulic Grade Line EL
Dam Side of PS-1	+13.9
Lagoon Side of PS-1	+13.7

Groundwater levels near PS-1:
Groundwater Level @ Toe of Dam – el 12.9
Groundwater Level @ Intake – el 12.6
Groundwater Level of 13.0 was assumed for preliminary design purposes.

Soil Lateral Loading Criteria
All substructures and retaining walls will be designed to resist lateral soil loadings. The following table summarizes the design criteria presented in the Preliminary Geotechnical Memo for the design of sheet pile retaining walls at this pump station.

Depth (feet)	**Unit Weight (pcf)**		**ϕ Degrees**	**δ Degrees**	**C (psf)**	**Coefficient of Lateral Earth Pressure**			**k (pci)**
	Moist	**Buoyant**				**Ka**	**Ko**	**Kp**	
0-20	110	60	30	18	0	0.33	0.50	3.00	25
20-40	100	43	0	0	1000	1.00	1.00	1.00	500
40-100	120	65	34	20	0	0.28	0.44	3.54	125

Notes: ϕ indicates angle of internal friction (degrees)

δ indicates angle of wall friction (degrees)
C indicates cohesion
Ka indicates coefficient of active lateral earth pressure
Ko indicates coefficient of at-rest lateral earth pressure
Kp indicates coefficient of passive lateral earth pressure
k indicates modulus of horizontal subgrade reaction
Depth refers to existing land surface grade

Materials

Structural Steel:

W shapes: ASTM A992/A992M, Grade 50.

C, MC and S shapes: ASTM A36/A36M.

L shapes: ASTM A36/A36M.

Plates: ASTM A36/A36M.

Structural square and rectangular tubing: ASTM A500, Grade B.

Structural pipes: ASTM A53/A53M, Grade B.

Round HSS: ASTM A500, Grade B.

High-strength bolts: ASTM A325/A325M, Types 1 or 3.

Standard bolts: ASTM A307.

Anchor bolts: ASTM F1554, Grade 36.

Sheet Pile: ASTM A572, Grade 50.

Fill Concrete: f'c = 2,500 psi at 28 days.

Structural Concrete: f'c = 4,000 psi at 28 days.

Precast Concrete: f'c = 5,000 psi, minimum at 28 days.

Reinforcing Steel: ASTM A615/A615M, Grade 60.

Design Standards

Concrete:

ACI 318: Building Code Requirements for Reinforced Concrete, by the American Concrete Institute.

EM 1110-2-2104: Strength Design for Reinforced Concrete Hydraulic Structures.

Hydraulic factors will be applied to the overall load factor equations for structures or portions thereof that are classified as hydraulic in accordance with EM 1110-2-2104.

Structural Steel:

Manual of Steel Construction by the American Institute of Steel Construction Inc, Ninth Edition.

Masonry:

Building Code Requirements for Masonry Structures, ACI 530.

Sheet Pile Retaining Walls:

EM 1110-2-2504, Design of Sheet Pile Walls

Computations

Preliminary structural design computations are found in the following pages.

3.2 Pump Station C43PS-4

3.2.1 General Description

The C43PS-4 intake shall be at 90 degrees to the canal's centerline. Because of the significant difference in the Townsend Canal and Perimeter Canal NWL normal water elevation and the exterior proposed grade of the Reservoir, there is a need for several retaining walls to transition from the intake to the discharge canal side of the site. It was assumed the entire site would be graded to an elevation of approximately 23 feet NAVD to acquire borrow material for the Reservoir embankment. The access to the intake and trash rake area requires a 1 vertical to 10 horizontal ramp from the access road that approaches the site from the north. The future access road was assumed parallel to the Reservoir's toe of slope. The intake service area was selected as 10.0 feet NAVD allowing approximately 3 feet of freeboard above the maximum canal stage. The operating floor elevation was set at 19.0 feet NAVD based on the mechanical arrangement of the pumping system. The discharge side service bridge elevation of 30.9 feet NAVD was also based on the headroom required by the selected mechanical arrangement.

The access roads in the immediate vicinity of C43PS-4 station shall have a 9 inch compacted shell base to ensure a sufficient bearing capacity for the maintenance equipment as well as to stabilize the approach ramps. This shell rock is to be mined locally and processed for use in the various road works of the Reservoir. Access roads and ramps at the pump station shall have a maximum slope of 1 vertical to 10 horizontal.

3.2.2 Embankments

Embankment construction shall consist of cohesiveless backfill placed to the lines and grades shown on the site plan. Embankment slopes shall be 3 horizontal to 1 vertical. All vegetation, debris, or other rubble shall be removed from fill areas prior to backfilling. Embankment materials shall be placed in horizontal layers not exceeding 12 inches in loose thickness and compacted. Cohesionless materials shall be compacted to not less than 80 percent relative density determined in accordance with ASTM D4253-00 and D4254-00. Following completion of embankment placement and compaction, embankment slopes and adjacent transition areas shall be graded reasonably smooth and free from irregular surface changes. The degree of finish shall be that ordinarily obtained from blade grader or similar operations.

Roundings at bottom of slopes and other breaks in grade shall be provided. All slopes and areas disturbed during construction shall be grassed.

3.2.3 Discharge Canal

The C43PS-4 discharge canal to the Perimeter Canal shall be designed for a water surface elevation of 23.5 feet NAVD with three feet of freeboard. Maintenance berms 14 feet wide shall parallel both sides of the canal with a top elevation of 26.5 feet NAVD. The canal shall be trapezoidal in shape and shall have side slopes of 1 vertical to 2.5 horizontal providing approximately 266 square feet of flow area at design stage and a or a maximum velocity of 1.77 fps at the rated station capacity of 150 cfs. The side slopes as well as erosion protection shall be conservatively designed to ensure stable transitions from the discharge chamber to typical canal section. The discharge channel shall have a 100 feet transition from the structure discharge sill elevation of 20.0 feet to the proposed canal bottom elevation of 15.0 feet NAVD. The canal centerline shall parallel the Reservoir embankment toe of slope and shall connect to the Perimeter Canal.

3.2.4 Access Road

Access to the C43PS-4 station shall be from the north along a proposed access road paralleling the Reservoir embankment toe of slope. The road shall originate at the C43PS-1. Access shall be controlled at C43PS-1, therefore security fencing at the C43PS-4 station site is not considered necessary.

3.2.5 Approach Channel

Unlike the typical intake, the intake pipe of a "can intake" does not require any special consideration in regard to the approach flow asymmetry and unsteady flow conditions that are caused by the geometric layout of the approach channel and the intake. The intake structure provides a sound structural support for the inlet of the intake pipe but primarily provides the structural support for the trash rack and raking system desired by the District. Survey sections of the canal shall be taken for the 60 percent design phase to determine the transitions necessary for the intake structure.

3.2.6 Erosion Protection

Erosion protection for the approach and discharge channels shall be Type B stone as required in the District's standard specifications. The riprap and bedding stone thicknesses are 3 feet and 1 feet respectively and are typical for the low flow velocities expected at the maximum flow rates for the C43PS-4 structure.

3.2.7 Design Criteria

The following design codes, requirements, and standards shall apply:

- SFWMD Guidelines Type 2 Water Supply Pump Station;
- Florida Building Code;
- ACI 318-05 - Building Code Requirements for Reinforced Concrete ;
- ACI 350R - Concrete for Environmental Structures;
- ANSI/ASCE 7-02 Standard - Minimum Design Loads for Buildings and Other Structures;
- National Fire Protection Association - NFPA 101 - Life Safety Code;

- PCI MNL-120 - Design Handbook;
- Steel Construction Manual - AISC, Ninth Edition;
- Structural Welding Code – AWS;
- National Concrete Masonry Association Design Tables;
- Standard Specifications for Highway Bridges, American Association of State Highway Officials; and
- FDOT Standard Specifications for Road and Bridge Construction, 2000.

3.2.8 Functional and Technical Requirements

The purpose of the building is to provide a secure and weather tight facility for the pump equipment including the electrical control and communication equipment. The construction shall be Type 1, non-combustible, and shall include a bearing cast-in-place concrete frame and fully grouted concrete masonry units. The roof framing shall be cast-in-place concrete with precast concrete hollowcore roof slabs. The roof shall have an approximate 1/4 inch/feet slope. The operating room square footage is approximately (40 feet x 47.5 feet) 1900 sq. feet with a (16 feet x 28 feet) 448 sq. feet electrical room and a (16 feet x 12 feet) 192 sq. feet generator room.

The C43PS-4 will be un-manned and remotely or automatically operated. The following systems shall not be required:

- Restroom;
- Drinking water service;
- Security alarm system;
- Fire alarm system; and
- Telephone service.

Building Architectural Considerations: The building finishes shall be typical for a pump station as defined in the SFWMD guidelines.

- The exterior and interior shall be coated with concrete and masonry latex block filler with acrylic top coats;
- The interior finishes Class A with a flame spread of 0-24, smoke developed 0-450, therefore allowing the walls and floors to be painted;
- All interior areas of the control room shall meet the Florida Accessibility Code requirements;
- The pedestrian doors and frames shall be 10 gage hollow metal. Frames shall be fully grouted and the doors and frames painted;
- The roof system shall be a SBS modified bitumen roof system; and
- The insulation shall be 2.0 in. minimum thickness rigid insulation over both the electrical room and the operating room with tapered insulation to ensure roof drainage.

3.2.9 Life Safety Analysis

The Florida Building Code does not have an occupancy classification for this facility so the NFPA 101 Life Safety Code was used to determine occupancy classifications and means of egress requirements.

- Occupancy: Industrial - Special Purpose;
- Occupant Load: Maximum probable at any one time;
- Exit Width: All doors to be a minimum of 36 inches clear;
- Number of Exits: (5) provided;
- Arrangement of Exits: Exits are separated by more than one-half the diagonal distance apart;
- Dead End Paths: Limited to 50 feet;
- Common Path of Travel: Limited to 100 feet; and
- Travel Distance: Limited to 400 feet

3.2.10 Loads and Design Criteria

The following loads and design assumptions shall be used in the structural analysis and calculations:

- Roof L.L. = 50 psf;
- Reinforced concrete unit weight = 140 pcf;
- 10 inces thick hollowcore slab unit weight = 86 psf; and
- Roof covering with insulation unit weight = 9 psf

Wind loads shall be determined in accordance with the Florida Building Code and District guidelines for a Type 2, water supply station and shall utilize the following criteria:

- Wind Velocity: ~120 mph per wind map FBC;
- 200 yr mean recurrence interval;
- 50 yr from 200 yr recurrence adjustment factor = 1.14;
- Roof: 0 degrees;
- Mean Roof Height: 24 feet ;
- Importance Factor = 1.15, for water supply pump station;
- Exposure category C, adjustment factor ~ 1.35; and
- Building Dimensions: 46 feet x 56 feet ~ rectangular enclosed building, no windows, low-rise building.

The following load factors were utilized in accordance with ASCE-07 Standard "Minimum Design Loads for Buildings and Other Structures and ACI-318:

U = 1.4(D + F)	ACI Equation 9-1
U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)	ACI Equation 9-2
U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W)	ACI Equation 9-3
U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R)	ACI Equation 9-4
U = 1.2D + 1.0E + 1.0L + 1.6S	ACI Equation 9-5
U = 0.9D + 1.6W + 1.6H	ACI Equation 9-6
U = 0.9D + 1.0E + 1.6H	ACI Equation 9-7

Hydraulic structures are required by the SFWMD to satisfy both strength and serviceability requirements. ACI-350R requires an additional load factor of 1.3 be applied to the results of the above listed load equations. In addition, the factor for soil lateral loads is required to be increased to 1.7 as per ACI-350R.

As per the SFWMD guidelines for Type 2 pump stations, the C43PS-4 substructure shall be designed to resist the saturated soil pressure to grade with an added 500 psf for traffic surcharge. Service bridges and operating floors shall be designed to support AASHTO HS25 loading.

3.2.11 Stability

A stability analysis shall be made of the station's retaining walls and intake structure. Unfactored loads shall be used in the stability analysis as indicated in USACE EM 1110-2-2104. The walls shall be analyzed for failure due to sliding, rotation about the toe, and by failure of the foundation material. In accordance with the USACE Engineering Manual EC1110-2-291, a minimum safety factor of 2.0 and 1.5 shall be provided for overturning and sliding respectively.

3.2.12 Reinforced Concrete Design

The 28 day compressive strength for cast-in-place concrete elements shall be specified as 4000 psi. For precast elements, the strength shall be specified as 5000 psi. The minimum concrete cover shall be not less than 1.5 x the maximum aggregate or 2.5 x the bar diameter. The following concrete cover shall be assumed in the analysis:

- Cast against earth: 4 inches
- Elements with surfaces exposed to abrasion, channel side faces: 3 inches
- Slabs, walls, and beams typical: 1.5 inches

The reinforcement tensile strength shall be taken as 60 ksi. The minimum temperature steel for the abutments, piers, and walls shall be specified as #5 bars at 12 inches on centers. The wall thicknesses shall be sufficient to allow the proper placement and consolidation of concrete and shall be designed in multiples of 2 inches to simplify formwork. For typical design and detailing of the reinforcement, multiples of 6 inches shall be used in the spacing of the primary and secondary steel. The minimum bar spacing shall be taken as 1.5 x bar diameter or 1.5 x the maximum aggregate size. The minimum spacing between layers shall be 6 inches.

Construction joints shall be located to facilitate the construction, limit the horizontal depth of the placement and shall not impair the strength and water tightness of the structure. Horizontal joints shall be located at the base of walls and typically at the maximum spacing of 10 feet – 0 inches. Reinforcement shall be continuous through the construction joint. Waterstops shall be used in construction joints where there is a head differential and the possibility of water flow through the joint.

3.3 Water Control Structures

3.3.1 Culverts

Cast-in-place concrete box culverts C43S-7 (10 feet x 10 feet double barrel with slide and weir gates) and C43S-9 (6 feet x 6 feet double barrel with slide and weir gates) are located north of the storage Reservoir. C43S-14 (10 feet x 6 feet single barrel with slide gate), C43S-15 (10 feet x 8 feet double barrel with slide and weir gates), C43S-16 (10 feet x 6 feet single barrel without gate), and C43S-17 (6 feet x 6 feet double barrel with slide and weir gates), C43S-18 (6 feet x 6 feet double barrel with slide and weir gates) and C43S-19 (10 feet x 6 feet single barrel with slide gate) are located in the Header Canal east of Cell No. 2 of the Reservoir. The slide gates are upward opening, the weir gates are downward opening. Both

gate types are made of stainless steel. All culverts, except C43S-16, have a dewatering feature.

Calculations for the structures include structural stability checks for flotation and preliminary design for a typical culvert section. The preliminary design utilized the box culvert design program LRFD Box Culvert V2.21 as published by the Florida Department of Transportation. This program analyzes cast-in-place concrete box culverts for conformance to AASHTO LRFD bridge design specifications.

Anticipated settlement of the culvert structures is minimal. As per recommendation of the project Geotechnical Report, underseepage will be controlled by installing a 5' concrete cutoff wall.

Design references include the following:

- EM 1110-2-2902 Conduits, Culverts and Pipes, USACE 31 October, 1997;
- ETL 11102-307 Engineering and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987;
- Standard Specification for Highway Bridges, AASHTO; and
- Geotechnical Report by DE&T, dated 01/06/06.

3.3.2 Perimeter Canal Structures

C43S-3

S-3 is a double barrel reinforced concrete box culvert which will be used to regulate water flow around the Perimeter Canal. The structure has two (2) different types of gates: an upward opening stainless steel slide gate and a downward opening stainless steel weir gate.

Calculations for this structure include structural stability check against flotation (uplift) and quantity calculations.

Hydraulic design data based upon the 100 year flood event and Allowable Stability Factors recommended by the USACE are used for the stability checks. The Geotechnical Engineer's recommendation of a friction coefficient of 0.35 is used based upon the structure resting on limestone. Also, per geotechnical recommendations, a five foot tremie seal within a sheetpile cofferdam is assumed for construction and sheetpile cutoff walls embedded five feet into the clay layer are assumed at the upstream and downstream ends of the structure to reduce seepage flow underneath the structure. No longitudinal sheetpile cutoff wall is used since the perimeter dam includes a full depth soil-bentonite cutoff wall.

Design References

- EC 1110-2-6058 Stability Requirements, USACE 30 November, 2003;
- ETL 11102-307 Engineering and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987;
- Design of Small Dams, published by United State Department of Interior, Bureau of Reclamation, A Water Resources Technical Publication; and
- Geotechnical Report by DE&T, dated 01/06/06.

C43S-10

S-10 is a three (3) bay reinforced concrete spillway which will be used to regulate water flow around the Perimeter Canal. The structure has two (2) different types of gates: an upward opening stainless steel roller gate and downward opening stainless steel weir gates. S-10 has one (1) roller gate bay and two (2) weir gate bays.

Calculations for this structure include structural stability checks and quantity calculations. The stability checks include sliding, flotation (uplift) and bearing.

Hydraulic design data based upon the 100 year flood event and Allowable Stability Factors recommended by the USACE are used for the stability checks. The Geotechnical Engineer's recommendation of a 500 psf skin friction pressure is used based upon the structure resting on clay. Also, per geotechnical recommendations, a five foot tremie seal within a sheetpile cofferdam is assumed for construction and a sheetpile cutoff wall embedded five feet into the clay layer is assumed at the upstream end of the structure to reduce seepage flow underneath the structure. No longitudinal sheetpile cutoff wall is used since the perimeter dam includes a full depth soil-bentonite cutoff wall.

Design References

- EC 1110-2-6058 Stability Requirements, USACE 30 November, 2003;
- ETL 11102-307 Engineering and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987;
- Design of Small Dams, published by United State Department of Interior, Bureau of Reclamation, A Water Resources Technical Publication; and
- Geotechnical Report by DE&T, dated 01/06/06.

C43S-11

S-11 is a four (4) bay reinforced concrete spillway which will be used to regulate water flow around the Perimeter Canal. The structure has two (2) different types of gates: upward opening stainless steel roller gates and downward opening stainless steel weir gates. S-11 has two (2) roller gate bays and two (2) weir gate bays.

Calculations for this structure include structural stability checks and quantity calculations. The stability checks include sliding, flotation (uplift) and bearing.

Hydraulic design data based upon the 100 year flood event and Allowable Stability Factors recommended by the USACE are used for the stability checks. The Geotechnical Engineer's recommendation of a 500 psf skin friction pressure is used based upon the structure resting on clay. Also, per geotechnical recommendations, a five foot tremie seal within a sheetpile cofferdam is assumed for construction and a sheetpile cutoff wall embedded five feet into the clay layer is assumed at the upstream end of the structure to reduce seepage flow underneath the structure. No longitudinal sheetpile cutoff wall is used since the perimeter dam includes a full depth soil-bentonite cutoff wall.

Design References

- EC 1110-2-6058 Stability Requirements, USACE 30 November, 2003;
- ETL 11102-307 Engineering and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987;

- Design of Small Dams, published by United State Department of Interior, Bureau of Reclamation, A Water Resources Technical Publication; and
- Geotechnical Report by DE&T, dated 01/06/06.

3.3.3 Main Outlet Structures

Introduction

Each cell of the Reservoir has a gated outlet structure to provide operational flows to meet the needs of the project at S-79 in the Caloosahatchee River. The outlets also maintain necessary Reservoir levels. These outlets are located in the northwest corner of each cell and discharge to the Perimeter Canal. Further information on the hydraulic performance of the outlets is contained in Section 2 of this report. Supporting structural computations follow this report section and detailed quantities for the structures can be found in the Cost Estimate section of the report.

C43S-1 and C43S-8 are the main outlets for Cell 1 and Cell 2 respectively. The designs are very similar with the major difference being that C43S-8 has a two foot higher sill elevation and 1-foot shorter dam crest. Both outlets discharge into the north Pperimeter Canal with C43S-1 discharging four feet lower than C43S-8. Each outlet consists of the following outlet components, listed from upstream to downstream:

- Gated intake structure that allows inflows at upper or mid-level Reservoir depths for control of water quality and water temperature;
- Double barrel cast-in-place conduit though the base of the dam;
- Double gatewell to house service and emergency gates;
- Double-bay stilling basin for energy dissipation; and
- Transition channel to Perimeter Canal (shown in civil sheets).

Structural Design Criteria

Corps of Engineer Engineering Manuals referenced:

- EC 1110-2-6058 Stability Analysis of Concrete Structures
- ETL 1110-2-307 Flotation Stability Criteria for Concrete Hydraulic Structures
- EM1110-2-2400 Structural Design & Evaluation of Outlet Works
- EM1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures
- EM1110-2-2902 Conduits Culverts and Pipes
- EM1110-2-2502 Retaining & Flood Walls

General Criteria

- Use identical gates and similar structural layout and design for each cell to provide design, construction, and maintenance advantages;
- Provide redundant gates in each outlet to allow normal operation with one gate out of service or to allow Reservoir drawdown within 120 days;
- Provide expansion and construction joints that accommodate the expected differential settlement (12 to 16 inches at the centerline of the dam);
- Provide sufficient concrete cover over reinforcement in areas of major hydraulic activity;
- Provide proper access for maintenance, operation, and monitoring of components;

- Provide water tight connections at intersections with the dam's seepage controlling features;
- Provide means for capturing and handling large debris before it enters conduits; and
- Provide means for dewatering structural components for inspection and maintenance.

Stability Criteria

Each outlet component and element was evaluated and sized to provide a suitable margin of safety between the overall forces and combinations of loads that can cause unsatisfactory resistance to flotation, sliding or bearing failure. The required factors of safety for stability were designated for "critical" structures that are new and thus well defined (EC 1110-2-6058). A flotation safety factor of 1.30 was used for scheduled maintenance of all outlet components.

Strength Criteria

Preliminary structural analysis, design and detailing were performed for multiple structural elements for a typical outlet location. C43S-1 being a taller structure typically controlled the design requirements. Sections were analyzed for both shear and flexural strength based on Load Factor Design. The compressive strength of concrete was taken as 3,500 psi due to the lower stresses typical of hydraulic structures, and the steel reinforcement yield stress was chosen as 60,000 psi. This preliminary analysis only considered the load combination(s) that were regarded as critical to select the preliminary thickness and reinforcement for each element for the purposes of obtaining a reasonable cost estimate.

Load combinations included self weight, dry and saturated soil, internal and external water pressures, uplift, and construction vehicle loading. Loads were calculated in spreadsheets, based on selected geometry and layout. These loads were then entered into Staad.Pro (a finite element structural analysis software package) in order to determine factored moments and shears. These demands were then entered back into a spreadsheet for design of the various section components.

Serviceability Criteria

All of the outlet components were considered Hydraulic Structures and as such, factored loads were multiplied by an additional hydraulic factor H_f of 1.30 in lieu of performing additional serviceability analyses. Shrinkage and temperature reinforcement is taken as 0.2% of the gross section of the element. With regard to concrete cover protection over the steel reinforcement, hydraulic structures are required to have the following protection:

- Unformed surfaces in contact with foundation - 4 inches;
- Formed surfaces subject to abrasion - stilling basin slabs and baffles - 6 inches;
- Formed surfaces stilling basin walls, conduit slabs/walls 24" or >, - 4 inches;
- Formed surfaces stilling basin walls, conduit slabs/walls 12" to 23", - 3 inches; and

- Elements 12" or less in accordance with ACI code.

Please refer to the Civil section of this report for erosion protection details required upstream of the intake structure and downstream of the stilling basin. Civil sheets also show the access ramps, fencing, and general grading around the outlet structures.

In addition, Civil sheets show the geotechnical details for the intersection of the dam's soil-bentonite cut-off wall and the gatewell. Also shown are details of the intersection of the conduit with the drainage collection chimney and blanket drain on the downstream side. Additionally conduits are to be surrounded by a collar of drainage material for the downstream third of the conduit in order to collect any possible seepage that may occur thought the dam prism next to the conduit exterior surfaces.

The geotechnical report included in Appendix A details the soil column information under each of the structures and makes recommendations with regard to soil parameters, suitable foundation materials, dewatering methods, and estimates of expected settlement.

Description of Outlet Components

Intake Structure

The cast-in-place intake structure is box shaped with an open top. A center training wall is included to guide the water into each conduit. The top of the slab is set 1.0' to 1.5' feet higher than the surrounding existing grade to minimize the potential for sediment accumulation in the intake thus minimizing the migration of sediments to the Perimeter Canal.

Intake gates control the quality of the water being released through the outlet. Four upper slide gates are located on the side walls and four mid-level slide gates are located on the side wall and headwall. These gates can be remotely or manually operated to control water quality and temperature. Two lower-level slide gates are located in the headwall at the intake sill. These gates are manually operated during the rare event of a full Reservoir drawdown. All the slide gates are located on the inside of the walls and operate either in the fully open or fully closed position and are subject to unseated head conditions. On the exterior face of the wall opening, trash rakes are provided at two feet on centers in both directions to prevent large debris from entering the intake. A rounded entrance at each opening minimizes turbulence.

Sluice gates located at the entrance to the conduits allow for dewatering of the conduits upstream of the gatewell without having to drawdown the entire Reservoir. The sluice gates are manually operated, and also provide a second level of emergency protection for the gatewells. Sluice gates provide a better seal to minimize leakage. The gates work in either fully open or fully closed position and are subject to seated head conditions. The concrete surfaces at the entrances are bell-mouth shaped to minimize intake head losses. The gates are 8' wide to allow for a wider middle wall section to house the gate side frames.

Flotation stability was found to preclude the ability to dewater the large and relatively light intake structure with a full Reservoir pool. In order to safely dewater the intake structure, the surrounding Reservoir will have to be lowered to Elevation 24.0 or lower. It is anticipated that the Reservoir cells may be completely dry every 5 to 10 years at which time an intake inspection should be scheduled. The gates can all be removed from the top leaving few reasons to dewater the intake. The walls of the intake were designed as cantilevers to resist a nominal exterior (or interior) water load of 12 feet.

Overturning and sliding stability involves the calculation of dynamic load action due to Reservoir waves during a hurricane event. This computation will take place at the next level of design when the final intake configuration is approved.

Access to the top of the intake structure can be accomplished from the crest of the dam. The civil drawings show the access roads, fencing and the location of the control building. The top of the intake consists of heavy duty steel grating supported by concrete beams and girders. The grating panels at the gates are removable so that gates can be accessed, raised and serviced on site or removed off site. Ladder rungs are provided for access to the interior of the intake. An outside ladder is also provided for access from a boat or barge when the Reservoir is partially filled. An access road provides vehicular access to the bottom of the intake when the cell is empty.

Double Barrel Conduit

The cast-in-place double-barrel conduit is rectangular in shape and designed to resist hydrostatic loads due to a full pool during a dewatered condition. The conduits are also designed for a fully pressurized condition with moist soil conditions near the gatewell for a condition where the embankment upstream slope is faced with a fairly impervious layer, (a possible alternative being studied). The case with one full and one dewatered conduit was also analyzed.

Expansion joints are provided at the intersections of major elements in the outlet. Additional expansion joints have been introduced along the conduits to allow the structure to settle with the embankment. The maximum expected settlement is at the centerline of the dam and ranges from 12" to 16" (see Geotechnical Report). Settlement is negligible at the upstream and downstream toes. Camber is provided at the gatewell such that the conduits will have a final positive downward slope ranging from 2 to 3% after settlement has taken place.

Flotation stability was computed for the upstream section of the conduits. The conduits need to be overlaid by at least one foot of soil in order to be safely dewatered. The access ramp to the intake provides more than adequate soil support.

Gatewell Structure

The gatewell structure is a cast-in-place double-bay rectangular chamber that provides access to the service and emergency gates. The exterior walls were designed to resist surrounding water pressures from a full pool Reservoir condition while dewatered inside. The case with one bay full and the other dewatered was also analyzed.

The conduits are narrowed at the gatewell to allow for smaller gates to be used. Cast iron gates are proposed for the service and emergency gates due to the high head and requirement for full operating range (as opposed to fully open or closed). The service gates are operated remotely while the emergency gates are operated manually. The service gates operate under a seated head condition while the emergency gates operate under an unseated head condition.

The massive gatewells are not subject to major overturning or flotation instabilities. Bearing pressures were computed at the base of the gatewell for the critical condition when the concrete work is complete but the structure has not yet been backfilled. The allowable bearing pressure of 4,000 psf is well above the actual pressures exerted at the base.

Each bay of the gatewell has a ladder fitted with a safety cage and a retractable safety post at the top. The grating panels at the top are removable so that gates can be accessed, raised and serviced on site or removed off site.

Stilling Basin

The stilling basin is a cast-in-place double-bay rectangular conveyance channel that dissipates most of the discharged water's energy before it reaches the receiving channel; in this case the Perimeter Canal. The U-shaped channel walls were designed to resist at-rest lateral saturated soil loads and uplift pressures when dewatered.

At the downstream side, a slab connects the top of the walls to bridge over the structure and allow not only through traffic but also a location from which to install dewatering needles. A more detailed design of the bridge will be conducted once more vehicle load information is available in the 60 percent design phase. A sheet pile cut-off wall is provided to prevent scouring beneath the slab in the event of failure of the riprap system.

Flotation stability of the stilling basin was computed and found to require the Perimeter Canal water surface be set no higher than the NWL in order to safely dewater the stilling basin. The dewatered basin can be accessed from the gatewell and a dewatered conduit. A portable ladder can be lowered into the basin slab if there is a need to access the inside. Each conduit is independent from the other. Therefore inspection of a full conduit train is possible while the adjacent train is in operation.

Sliding stability of the basin was checked for water loading on the baffle blocks. A hydrostatic lateral pressure corresponding to the fluid energy grade line was used to compute the driving lateral force in lieu of a more detailed method of analysis. The basin was found to be safe against sliding instabilities.

Future Design Considerations

The following tasks are recommended to bring the structural design to the next level:

- Detailed design of the trash struts at intake;
- Details for gate frames cast in concrete;
- Constructability and cost of rounded surfaces;

- Update camber calculations on conduits if settlement computations are revised;
- Consider full range of load combinations for all the elements involved;
- Verify most convenient quantity and location for storage of dewatering needles;
- Develop a concrete mix design suitable for hydraulic structures;
- Evaluate the need for emergency gates at the gatewell if gates are provided at the entrance of the conduits; and
- Evaluate the need for permanent ladders to access the bottom of stilling basins.

3.3.4 Cell Balancing Structure

Introduction

The Cell Balancing Structure, C43S-12, is located between Cells 1 and 2 of the Reservoir. The primary function of the structure is to convey water between the cells and allow the cells to operate independently if needed. If one of the cells needs to be dewatered for maintenance C43S-12 may be closed to isolate the cells. This structure is located along the interior dam between cells. Further information on the hydraulic performance of this structure is contained in the Section 2 of this report. Supporting stability computations follow this report section and detailed quantities for the structures can be found in the Cost Estimate section of the report.

The general configuration of this structure is similar to the design of the main outlets C43S-1 and C43S-8 with the following major differences: C43S-12 is a triple conduit structure and water flows in the conduits in both directions depending on which cell is empty or filled. The dam crest elevation of the interior dam at the balancing structure matches that of Cell 1. The east and west stilling basins are identical in configuration except that the east basin is set one foot higher in grade to allow water to drain by gravity from Cell 2 to Cell 1 during a drawdown event. The top of the conduits at the ends are shaped upward to minimize head losses when water flows in the opposite direction. The balancing structure consists of the following components listed from west to east:

- Triple bay uncontrolled inlet/stilling basin structure;
- Triple barrel cast-in-place conduit along the base of the dam;
- Triple gatewell to house service and emergency gates; and
- Triple bay uncontrolled and un-gated inlet/stilling basin structure.

Structural Design Criteria

Corps of Engineer Engineering Manuals referenced:

- EC 1110-2-6058 Stability Analysis of Concrete Structures
- ETL 1110-2-307 Flotation Stability Criteria for Concrete Hydraulic Structures
- EM1110-2-2400 Structural Design & Evaluation of Outlet Works
- EM1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures
- EM1110-2-2902 Conduits Culverts and Pipes
- EM1110-2-2502 Retaining & Flood Walls

General Criteria

- Use identical gates and similar structural layout and design for each conduit to provide design, construction, and maintenance advantages;
- Provide one redundant conduit train to allow normal operation with one conduit out of service;

- Provide expansion and construction joints that accommodate the expected differential settlement (12 to 16 inches at the centerline of the dam);
- Provide sufficient concrete cover over reinforcement in areas of major hydraulic activity;
- Provide proper access for maintenance, operation, and monitoring of components; and
- Provide means for capturing and handling large debris before it enters conduits.

Stability Criteria

Each outlet component and element was evaluated and sized to provide a suitable margin of safety between the overall forces and combinations of loads that can cause unsatisfactory resistance to flotation, sliding or bearing failure. The required factors of safety for stability were designated for "critical" structures that are new and thus well defined (EC 1110-2-6058). A flotation safety factor of 1.30 was used for scheduled maintenance of all structure components.

Strength Criteria

Please refer to the Main Outlet Structural section for details on this section.

Serviceability Criteria

Please refer to the Main Outlet Structural section for details on this section.

Specifically, C43S-12 is located on the interior or cell separator dam which does not have special provisions for internal seepage cut-off. Instead, the dam's cross section is much wider to allow for safe embankment slope stability when one cell is empty. Conduits at C43S-12 need to be surrounded by a collar of drainage material at the downstream third of the conduit in order to collect any possible seepage that may occur through the dam prism next to the conduit exterior surfaces during times when one cell is empty.

Description of Structure Components

Inlet/Stilling Basin Structure

The cast-in-place inlet/stilling basin structure is formed by an excavated plunge pool that dissipates the energy from water releases at the start of filling a Reservoir cell. The top of the inlet wall of the east basin is set 1.0' higher than the surrounding grade to minimize sediment loads from entering Cell 1.

The structure is un-gated and uncontrolled, however trash rakes are provided at the entrances to prevent large debris from entering the conduit. A rounded entrance at each opening minimizes turbulence.

Flotation stability of the basin was found to preclude the ability to dewater the structure with a groundwater level to the top of the basin (same as the existing grade). In order to safely dewater the basin structure, the surrounding groundwater would have to be lowered a few feet to an elevation of 15.0 or lower. Weep holes and a drain system will be designed to allow basin dewatering. The basin walls were designed for a nominal exterior water load of 10' resisted as a cantilever wall.

Sliding was not considered because the excavated basin has adequate passive soil resistance behind the back wall.

An access road provides vehicular access to the bottom of the intake when the particular Reservoir cell is empty. A portable ladder can be lowered into the basin slab if there is a need to access the inside. Access and riprap protection around the basin is shown on the civil drawings.

Triple Barrel Conduit

Please refer to the Main Outlet Structural section for details on this section.

Flotation stability was not computed for the conduits because they can not be dewatered under a full pool Reservoir condition. Inspection of the conduits downstream of the gatewell can be performed when the respective cell is empty and vice versa. This inspection can take place every 5 or 10 years depending on cell use.

Gatewell Structure

The gatewell structure is a cast-in-place triple-bay rectangular chamber that provides access to the service and emergency gates. The exterior walls were designed to resist surrounding water pressures from a full pool Reservoir condition while dewatered inside. Please refer to the computations of the main outlets for specific information.

Normal operation of the structure is to balance the water levels at the two cells with all the gates fully open at all times. When one cell is to be dewatered, the gatewell gates operate in the same fashion as the service gates of the main outlets C43S-1 and C43S-8.

Both service and emergency gates have a dual and opposite role for reversed Reservoir operation. All 6 gates are thus furnished with controls to be operated remotely. The service gates operate under a seated head condition while the emergency gates operate under an unseated head condition. Cast iron gates are proposed for the service and emergency gates due to the high head and requirement for full operating range (as opposed to fully open or closed).

The massive gatewells are not subject to major overturning or flotation instabilities. Bearing pressures were computed at the base of the gatewell for the critical condition when the concrete work is complete but the structure has not yet been backfilled. The allowable bearing pressure of 4,000 psf is well above the actual pressures exerted at the base.

Each bay of the gatewell has a ladder fitted with a safety cage and a retractable safety post at the top. The grating panels at the top are removable so that gates can be accessed, raised and serviced on site or removed off site.

Future Design Considerations

The following tasks are recommended for the 60 percent design phase:

- Detailed design of the trash rakes at intake;
- Constructability and cost of rounded intake surfaces;
- Update camber calculations on conduits if settlement computations are revised;
- Consider full range of load combinations for all the elements involved; and
- Further evaluate dewatering systems for stilling basins.

3.3.5 Crest Spillway Structures

Introduction

Each cell of the Reservoir has an uncontrolled crest spillway structure to manage flood flows. C43CS-1 is located on the west side of Cell 1 and discharges to the Townsend Canal. C43CS-2 is located just east of C43S-8 in the northwest corner of Cell 2 and discharges to the Perimeter Canal. The design objective of the crest spillway structures is to minimize the surcharge storage of the Reservoir without exceeding the Allowable Discharge. Further information on the hydraulic performance of the outlets is contained in Section 2 of this report. Supporting structural computations follow this report section and detailed quantities for the structures can be found in the Cost Estimate section.

The design of the two crest spillways is very similar with the major difference being that C43CS-1 uses a deeper and different type of stilling basin design. The overflow weir and conduit for both spillways are set at the same elevation however the crest of the dam is set one foot higher at C43CS-1. Each spillway consists of the following components, listed from upstream to downstream:

- Uncontrolled/un-gated inlet weir allows overflows higher than the NFSL of 42.0.
- Single barrel cast-in-place conduit though dam at invert elevation 38.0.
- U-shaped spillway chute.
- Single-bay stilling basin for energy dissipation.
- Transition channel to Perimeter Canal (shown in civil sheets)

Structural Design Criteria

Corps of Engineer Engineering Manuals referenced:

- EC 1110-2-6058 Stability Analysis of Concrete Structures
- ETL 1110-2-307 Flotation Stability Criteria for Concrete Hydraulic Structures
- EM1110-2-2400 Structural Design & Evaluation of Outlet Works
- EM1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures
- EM1110-2-2902 Conduits Culverts and Pipes
- EM1110-2-2502 Retaining & Flood Walls

General Criteria

- Use similar structural layout and design for each spillway to provide design, construction, and maintenance advantages;
- Provide expansion and construction joints that accommodate the expected differential settlement;
- Provide sufficient concrete cover over reinforcement in areas of major hydraulic activity;
- Provide proper access for maintenance, operation, and monitoring of components;

- Provide water tight connections at intersections with the dam's seepage controlling features; and
- Provide means for dewatering structural components for inspection and maintenance.

Stability Criteria

Each spillway component and element was evaluated and sized to provide a suitable margin of safety between the overall forces and combinations of loads that can cause unsatisfactory resistance to flotation, sliding or bearing failure. The required factors of safety for stability were designated for "critical" structures that are new and thus well defined (EC 1110-2-6058). A flotation safety factor of 1.30 was used for scheduled maintenance of all spillway components.

Strength Criteria

Preliminary structural analysis, design and detailing were performed for multiple structural elements for a typical spillway location. C43CS-1 being a taller stilling basin structure typically controlled the design requirements. Sections were analyzed for both shear and flexural strength based on Load Factor Design. The compressive strength of concrete was taken as 3,500 psi due to the lower stresses typical of hydraulic structures, and the steel reinforcement yield stress was chosen as 60,000 psi. This preliminary analysis only considered the load combination(s) that were regarded as critical to select the preliminary thickness and reinforcement for each element for the purposes of obtaining a reasonable cost estimate.

Load combinations included self weight, dry and saturated soil, internal and external water pressures, uplift, and construction vehicle loading. Loads were calculated in spreadsheets, based on selected geometry and layout. These loads were then entered into Staad.Pro (a finite element structural analysis software package) in order to determine factored moments and shears. These demands were then entered back into a spreadsheet for design of the various section components.

Serviceability Criteria

All of the spillway components were considered Hydraulic Structures and as such, factored loads were multiplied by an additional hydraulic factor H_f of 1.30 in lieu of performing additional serviceability analyses. Shrinkage and temperature reinforcement is taken as 0.2% of the gross section of the element. With regard to concrete cover protection over the steel reinforcement, hydraulic structures are required to have the following protection:

- Unformed surfaces in contact with foundation - 4 inches;
- Formed surfaces subject to abrasion - stilling basin slabs and baffles - 6 inches;
- Formed surfaces stilling basin walls, conduit slabs/walls 24 inches or > - 4 inches;
- Formed surfaces stilling basin walls, conduit slabs/walls 12 inches to 23 inches - 3 inches; and
- Elements 12 inches or less in accordance with ACI code.

Please refer to the Civil section of this report for erosion protection details required upstream of the inlet weir structure and downstream of the stilling basin. Civil sheets also show the access ramps, fencing, and general grading around the spillway structures.

In addition, Civil sheets show the geotechnical details for the intersection of the dam's soil-bentonite cut-off wall and the base of the single barrel conduit. Also shown are details of the intersection of the stilling basin with the drainage collection chimney and blanket drain on the downstream side. It is anticipated that the single conduit will not need a downstream collar of drainage material. The conduit is located high enough and seepage is not expected at that height and downstream location.

The Geotechnical Report provided in the Civil section details the soil column information under each of the structures and makes recommendations with regard to soil parameters, suitable foundation materials, dewatering methods, and estimates of expected settlement.

Description of Spillway Components

Inlet Weir Structure

The cast-in-place inlet structure is basically a flat slab on grade supporting a concrete weir. The downstream end is joined by a vertical training wall to properly convey flows into the single conduit. The radius of the weir was selected based on USACE guidelines. However the upstream approach radius may be reduced to allow for less expensive construction. At the entrance to the conduit a small breast wall is provided to limit allowable discharges and to help deflect potential wave run up action.

Flotation stability criteria for hydraulic structures according to the 1987 ETL 1110-2-307 report a 1.50 factor of safety for normal operation, which is the case for this weir structure. However, flotation criteria according to the 2003 EC 1110-2-6058 (Stability Analysis of Concrete Structures), reflects a factor of safety of 1.3 for usual load conditions. The computed factor of safety is 1.40 for the proposed geometry. This is considered adequate given the massive weir shape.

Overturning and sliding stability were not of significant relevance to this structure.

Access to the inlet weir structure can be accomplished from the crest of the dam. A perimeter concrete pad is provided to allow maintenance vehicles to approach the area. The Reservoir can be lowered four feet temporarily to access this site. The civil drawings show the access roads, fencing and the location of the control building.

It is anticipated that this area will be surcharged and pre-loaded at the beginning of the construction of the project to accomplish most of the settlement. The construction of the weir will be done at the end of the project to the NFSL elevation plus a small amount recommended by the geotechnical engineer

corresponding to any calculated post-settlement. The settlement results from the current test cells project will provide a good measure to calibrate settlement.

Single Barrel Conduit

The cast-in-place single-barrel conduit is rectangular in shape and designed to resist soil loads to the crest of the dam. Construction vehicle loads were also analyzed.

Expansion joints are provided at the intersections of major elements in the spillway. The exact differential settlement at elevation 38.0 along the conduit will be determined in the 60 percent design phase in order to program the pre-loading conditions for these structures. After pre-loading, further settlement will be very small therefore the conduits can be constructed to a final positive downward slope of 2%. A small camber and an expansion joint in the center will be provided.

Spillway Chute

The spillway chute structure is a cast-in-place U-shaped single-bay rectangular chute that delivers flows to the stilling basin. The walls are outside the dam prism and avoid the need for fall protection.

The spillway chute is not subject to major overturning or flotation instabilities. However, sliding stability is of concern because of the relatively steep angle of the 3:1 slope. The calculated factor of safety is about 1.2 for sliding on random fill. The required factor of safety is 1.70. The downstream stilling basin, however are embedded deep and have ample capacity to resist and carry additional sliding forces from the chute.

Stilling Basin

The stilling basin is a cast-in-place single-bay rectangular conveyance channel that dissipates most of the discharged water's energy before it reaches the receiving channel. The U-shaped channel walls were designed to resist at-rest lateral saturated soil loads and uplift pressures when dewatered.

At the downstream side, a slab connects the top of the walls to bridge over the structure and allow not only through traffic but also a location from which to install dewatering needles. A more detailed design of the bridge will be conducted once more vehicle load information is available. C43CS-1 is fairly deep thus a similar slab was continued for a portion of the basin to reduce the moment and shear requirements at the base of the walls.

C43CS-2 is provided with a sheet pile cut-off wall to prevent scouring beneath the slab in the event of failure of the sloping riprap system. C43CS-1 is not provided with sheet pile protection because the downstream channel is flat. The concrete shear key is sufficient.

Flotation stability of the stilling basins was computed and found to require the receiving canal water surface be set no higher than the NWL in order to safely dewater the stilling basin. C43CS-1 is deeper and therefore has an increased ability to resist flotation with higher canal levels in the Townsend. The

dewatered basins can be accessed from the perimeter toe road. A portable ladder can be lowered into the basin slab if there is a need to access the inside.

Sliding stability of the basin was checked for water loading on the baffle blocks in the case of C43CS-2. A hydrostatic lateral pressure corresponding to the fluid energy grade line was used to compute the driving lateral force in lieu of a more detailed method of analysis. The basin were found to have ample resistance against sliding instabilities. C43CS-1 does not have baffle blocks.

Future Design Considerations

The following tasks are recommended in the 60 percent design phase:

- Evaluate the feasibility to discharge service spillway flows behind the gatewell and through the secondary conduit of the main outlets. This will avoid the construction of the crest spillway;
- Improve the shape of the weir inlet without compromising flotation;
- Evaluate the configuration of a dentated sill (as opposed to continuous) on C43CS1 for joint use with the needle sill;
- Constructability and cost of rounded surfaces;
- Update camber calculations on conduits when settlement computations are revised;
- Consider full range of load combinations for all the elements involved;
- Verify most convenient quantity and location for storage of dewatering needles; and
- Evaluate the need for permanent ladders to access the bottom of stilling basins.

C-43 West Storage Reservoir
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Uplift Summary

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C43PS1 - Floatation Check

Reference: ETL 1110-2-307, Floatation Stability Criteria for Concrete Hydraulic Structures

Description	Volume (ft^3)	Force (kips)
SUMMARY:		
FOUNDATION CONCRETE	125064	18760
SUCTION BASIN CONCRETE	42158	6324
INTAKE BAYS EL-16.0 TO EL-6.0 (to bottom of intermediate slab)	18243	2736
FILL CONCRETE (Intake pipe formation)	34771	5216
INTAKE BAYS EL-6.0 TO EL-3.5 (assumes slab thickness 2.5 ft)	16555	2483
INTAKE BAYS EL-3.5 TO EL+7.0 (to bottom of pipe gallery slab)	29337	4401
PIPE GALLERY SLAB EL+7.0 TO EL+10.0 (assumes slab thickness 3.0 ft)	66117	9918
PIPE GALLERY EL+10.0 TO EL+22.0 (to bottom of operating deck slab)	46042	6906
OPERATING DECK SLAB EL+22.0 TO EL+25.0 (assumes slab thickness 3.0 ft)	64412	9662
SERVICE BRIDGE TOC EL+16.0	11018	1653
SAND FILL	100705	11078
FILL ON FOUNDATION PROJECTIONS	45838	2750
WATER ON FOUNDATION PROJECTIONS	45838	2860
WATER IN SUCTION BASINS	0	0
UPLIFT (assumes water to grade, and suction bays dewatered)	900846	56213

Weight of the structure (not including superstructure) and soil above the strcuture	W_s =	81885 kips
Weight of water within structure controlled by a mechanical operator (i e pump)	W_c =	0 kips
Surcharge loads	S =	0 kips
Weight of surcharge water above the structure totally controlled by gravity flow	W_g =	2860 kips
Uplift forces acting on the base of the structure	U =	56213 kips
Floatation Safety Factor defined as: $\frac{W_s + W_c + S}{U - W_g}$	SF =	1.53 OK

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Description	Length (ft) - (in)	Width / Th (ft) - (in)	Depth / Ht (ft) - (in)	Number	Volume (ft³)	Volume (yd³)	γ (pcf)	Weight (short ton)
FOUNDATION CONCRETE								
Continuous base slab TOC EL -16 0	193 - 0 00	108 - 0 00	6 - 0 00	1	125064 0	4632 0	150	9,379 8
				SUBTOTALS	125064.0	4632.0		9,379.8
SUCTION BASIN CONCRETE								
Training Pier (EL-16.0 to EL+16 0)								
Overall dimension	6 - 0 00	3 - 0 00	32 - 0 00	12	6912 0	256.0	150	518 4
Subtract stoplog channel	1 - 2 00	0 - 3 50	32 - 0 00	-24	-261 3	-9 7	150	-19 6
East Wall (EL-16 0 to EL+16 0)								
Wall	48 - 0 00	4 - 0 00	32 - 0 00	1	6144.0	227.6	150	460.8
Subtract stoplog channel	1 - 2 00	0 - 3 50	32 - 0 00	-1	-10 9	-0 4	150	-0 8
Interior Divider Walls (EL-16 0 to EL+16 0)								
Wall	48 - 0 00	3 - 0 00	32 - 0 00	3	13824 0	512.0	150	1 036 8
Subtract stoplog channel	1 - 2 00	0 - 3 50	32 - 0 00	-6	-65 3	-2 4	150	-4 9
Auxillary Basin Walls (EL-16 0 to EL+16 0)								
Divider wall	48 - 0 00	3 - 0 00	32 - 0 00	1	4608.0	170.7	150	345.6
Subtract stoplog channel	1 - 2 00	0 - 3 50	32 - 0 00	-3	-32.7	-1 2	150	-2.5
North end wall	10 - 6 00	3 - 0 00	32 - 0 00	2	2016 0	74.7	150	151 2
Trash rake short walls	15 - 0 00	3 - 0 00	32 - 0 00	2	2880 0	106 7	150	216 0
West Wall (EL-16 0 to EL+16 0)								
Wall	48 - 0 00	4 - 0 00	32 - 0 00	1	6144 0	227 6	150	460 8
				SUBTOTALS	42157.8	1561.4		3,161.8
INTAKE BAYS EL-16.0 TO EL-6.0 (to bottom of intermediate slab)								
Exterior Walls								
North Wall (separates intake basin from main bldg)	177 - 0 00	3 - 0 00	10 - 0 00	1	5310.0	196 7	150	398 3
Subtract intake openings	24 - 0 00	3 - 0 00	10 - 0 00	-5	-3600.0	-133.3	150	-270 0
West Wall	56 - 0 00	4 - 0 00	10 - 0 00	1	2240 0	83.0	150	168 0
South Wall	177 - 0 00	3 - 0 00	10 - 0 00	1	5310 0	196.7	150	398 3
East Wall	56 - 0 00	4 - 0 00	10 - 0 00	1	2240 0	83 0	150	168 0
Interior Walls								
Pump basin dividers (N-S)	50 - 0 00	3 - 0 00	10 - 0 00	4	6000 0	222 2	150	450 0
Additional Interior Wall (either side of 250 cfs pipe)								
Wall (top of fill conc EL-9 641)	18 - 0 00	2 - 0 00	3 - 7 69	4	524 3	19 4	150	39 3
Additional Interior Wall (either side of 500 cfs pipe)								
Wall (top of fill conc EL-7 521)	18 - 0 00	2 - 0 00	1 - 6 25	4	219 0	8 1	150	16 4
				SUBTOTALS	18243.3	675.7		1,368.2
FILL CONCRETE (Intake pipe formation)								
250 CFS Bay								
Diffuser chamber total (EL-16 0 to EL-6 0)	10 - 9 56	33 - 0 00	10 - 0 00	2	7125.9	263 9	150	534 4
Subtract intake cone volume	11 - 0 00	12 - 0 00	4 - 6 00	-2	-1188 0	-44.0	150	-89.1
Add diffuser wall	11 - 0 00	0 - 6 00	4 - 6 00	2	49 5	1.8	150	3 7
Elbow chamber total (EL-16 0 to EL-9 641)	17 - 3 66	33 - 0 00	6 - 4 31	2	7263 1	269 0	150	544 7
Subtract elbow volume	8 - 0 00	8 - 0 00	4 - 6 00	-2	-432 0	-16 0	150	-32 4
Additional between North end and diffuser intake	12 - 0 00	4 - 0 00	10 - 0 00	4	1920 0	71 1	150	144 0
500 CFS Bay					0.0	0.0	150	0.0
Diffuser chamber total (EL-16 0 to EL-6 0)	17 - 4 78	33 - 0 00	10 - 0 00	2	11483 0	425 3	150	861 2
Subtract intake cone volume	13 - 0 00	18 - 0 00	5 - 9 00	-2	-2691.0	-99.7	150	-201.8
Add diffuser wall	13 - 0 00	1 - 0 00	5 - 9 00	2	149.5	5.5	150	11.2
Elbow chamber total (EL-16 0 to EL-7 521)	17 - 3 63	33 - 0 00	8 - 5 75	2	9682 7	358 6	150	726 2
Subtract elbow volume	12 - 0 00	8 - 0 00	6 - 0 00	-2	-1152 0	-42 7	150	-86 4
Additional between North end and diffuser intake	16 - 0 00	4 - 0 00	10 - 0 00	4	2560 0	94 8	150	192 0
				SUBTOTALS	34770.7	1287.8		2,607.8

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Description	Length (ft) - (in)	Width / Th (ft) - (in)	Depth / Ht (ft) - (in)	Number	Volume (ft^3)	Volume (yd^3)	γ(pcf)	Weight (short ton)
INTAKE BAYS EL-6.0 TO EL-3.5 (assumes slab thickness 2.5 ft)								
Exterior Walls								
North Wall	177 - 0 00	3 - 0 00	2 - 6 00	1	1327.5	49 2	150	99 6
West Wall	56 - 0 00	4 - 0 00	2 - 6 00	1	560.0	20 7	150	42 0
South Wall	177 - 0 00	3 - 0 00	2 - 6 00	1	1327.5	49 2	150	99 6
East Wall	56 - 0 00	4 - 0 00	2 - 6 00	1	560 0	20 7	150	42 0
Interior Walls								
Pump basin dividers	50 - 0 00	3 - 0 00	2 - 6 00	4	1500 0	55 6	150	112 5
Slab above intake								
Slab TOC EL-3 5	29 - 0 00	33 - 0 00	2 - 6 00	4	9570 0	354 4	150	717 8
Additional Interior Wall (either side of 250 cfs pumps)								
Wall	18 - 0 00	2 - 0 00	2 - 6 00	4	360 0	13 3	150	27 0
Additional Interior Wall (either side of 500 cfs pumps)								
Wall	18 - 0 00	2 - 0 00	2 - 6 00	4	360 0	13 3	150	27 0
Additional Divider Wall								
Wall	33 - 0 00	3 - 0 00	2 - 6 00	4	990 0	36 7	150	74 3
				SUBTOTALS	16555.0	613.1		1,241.6
INTAKE BAYS EL-3.5 TO EL+7.0 (to bottom of pipe gallery slab)								
Exterior Walls								
North Wall	177 - 0 00	3 - 0 00	10 - 6 00	1	5575 5	206.5	150	418 2
West Wall	56 - 0 00	4 - 0 00	10 - 6 00	1	2352 0	87.1	150	176 4
South Wall	177 - 0 00	3 - 0 00	10 - 6 00	1	5575 5	206.5	150	418 2
East Wall	56 - 0 00	4 - 0 00	10 - 6 00	1	2352 0	87 1	150	176 4
Interior Walls								
Pump basin dividers	50 - 0 00	3 - 0 00	10 - 6 00	4	6300 0	233 3	150	472 5
Additional Interior Wall (either side of 250 cfs pumps)								
Wall	18 - 0 00	2 - 0 00	10 - 6 00	4	1512 0	56 0	150	113 4
Additional Interior Wall (either side of 500 cfs pumps)								
Wall	18 - 0 00	2 - 0 00	10 - 6 00	4	1512 0	56 0	150	113 4
Additional Divider Wall								
Wall	33 - 0 00	3 - 0 00	10 - 6 00	4	4158 0	154 0	150	311 9
				SUBTOTALS	29337.0	1086.6		2,200.3
PIPE GALLERY SLAB EL+7.0 TO EL+10.0 (assumes slab thickness 3.0 ft)								
Continuous slab TOC EL +10 0	258 - 0 00	87 - 0 00	3 - 0 00	1	67338.0	2494.0	150	5 050.4
Subtract 250 cfs pipe penetrations (6 diam)	10 - 0 00	10 - 0 00	3 - 0 00	-2	-600 0	-22 2	150	-45 0
Subtract 500 cfs pipe penetrations (8 diam)	10 - 0 00	10 - 0 00	3 - 0 00	-2	-600.0	-22 2	150	-45.0
Subtract for auxillary pipe penetrations	1 - 0 00	1 - 0 00	3 - 0 00	-7	-21 0	-0 8	150	-1 6
				SUBTOTALS	66117.0	2448.8		4,958.8

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Description	Length (ft) - (in)	Width / Th (ft) - (in)	Depth / Ht (ft) - (in)	Number	Volume (ft³)	Volume (yd³)	γ (pcf)	Weight (short ton)
PIPE GALLERY EL+10.0 TO EL+22.0 (to bottom of operating deck slab)								
Exterior Walls								
North Wall	252 - 0 00	3 - 0 00	12 - 0 00	1	9072 0	336.0	150	680 4
Subtract access openings	3 - 0 00	3 - 0 00	12 - 0 00	-2	-216 0	-8.0	150	-16 2
West Wall	78 - 0 00	3 - 0 00	12 - 0 00	1	2808 0	104 0	150	210 6
Subtract access openings	12 - 0 00	3 - 0 00	12 - 0 00	-1	-432 0	-16 0	150	-32 4
South Wall	252 - 0 00	3 - 0 00	12 - 0 00	1	9072 0	336.0	150	680 4
Subtract 250 cfs pipe penetrations (6' diam)	6 - 0 00	3 - 0 00	6 - 0 00	-2	-216 0	-8 0	150	-16 2
Subtract 500 cfs pipe penetrations (8 diam)	8 - 0 00	3 - 0 00	8 - 0 00	-2	-384 0	-14 2	150	-28 8
East Wall	78 - 0 00	3 - 0 00	12 - 0 00	1	2808 0	104 0	150	210 6
Interior Walls								
Gallery entrance wall (south side)	13 - 8 00	2 - 0 00	12 - 0 00	1	328.0	12 1	150	24.6
West side of auxillary pumps	62 - 0 00	3 - 0 00	12 - 0 00	1	2232.0	82 7	150	167.4
South side of auxillary pumps	33 - 0 00	2 - 0 00	12 - 0 00	1	792.0	29.3	150	59.4
Column near starwell	2 - 0 00	2 - 0 00	12 - 0 00	1	48 0	1 8	150	3 6
Repeatable pump separation walls								
Main divider (N to S)	62 - 0 00	3 - 0 00	12 - 0 00	3	6696 0	248.0	150	502 2
Short wall (N to S)	18 - 0 00	2 - 0 00	12 - 0 00	4	1728 0	64.0	150	129 6
Hallway wall (N to S)	36 - 0 00	2 - 0 00	12 - 0 00	4	3456.0	128.0	150	259.2
Short wall (E to W)	7 - 6 00	2 - 0 00	12 - 0 00	4	720.0	26.7	150	54.0
Short wall (E to W)	25 - 6 00	2 - 0 00	12 - 0 00	8	4896 0	181 3	150	367 2
South East Corner Interior Walls								
Stairwell wall	15 - 6 00	2 - 0 00	12 - 0 00	1	372.0	13.8	150	27 9
Wall (E to W)	27 - 9 00	2 - 0 00	12 - 0 00	2	1332.0	49 3	150	99 9
Wall (N to S)	31 - 0 00	2 - 0 00	12 - 0 00	1	744 0	27.6	150	55 8
Stairwell dividing wall	15 - 6 00	1 - 0 00	12 - 0 00	1	186 0	6 9	150	14 0
				SUBTOTALS	46042.0	1705.3		3,453.2
OPERATING DECK SLAB EL+22.0 TO EL+25.0 (assumes slab thickness 3.0 ft)								
Continuous slab TOC EL +25.0	258 - 0 00	87 - 0 00	3 - 0 00	1	67338 0	2494.0	150	5,050.4
Subtract pump housing opening (14-2 diameter)	157 - 7 50	1 - 0 00	3 - 0 00	-4	-1891.5	-70 1	150	-141.9
Subtract pipe gallery equipment access	10 - 0 00	10 - 0 00	3 - 0 00	-1	-300 0	-11 1	150	-22 5
Subtract auxillary shaft opening	29 - 4 00	4 - 0 00	3 - 0 00	-1	-352 0	-13 0	150	-26 4
Subtract stairwell	17 - 0 00	7 - 6 00	3 - 0 00	-1	-382 5	-14 2	150	-28 7
				SUBTOTALS	64412.0	2385.8		4,830.9

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Description	Length (ft) - (in)	Width / Th (ft) - (in)	Depth / Ht. (ft) - (in)	Number	Volume (ft³)	Volume (yd³)	γ (pcf)	Weight (short ton)
SERVICE BRIDGE TOC EL+16.0								
Slabs								
Approach slab West	20 - 0.00	18 - 0.00	2 - 0.00	1	720.0	26.7	150	54.0
Approach slab East	20 - 0.00	28 - 0.00	2 - 0.00	1	1120.0	41.5	150	84.0
Bridge slabs TOC EL+16.0	18 - 0.00	36 - 0.00	2 - 0.00	5	6480.0	240.0	150	486.0
Trash collection pad	10 - 0.00	36 - 0.00	2 - 0.00	1	720.0	26.7	150	54.0
Guardrail								
Long walls	183 - 0.00	0 - 10.00	3 - 6.00	2	1067.5	39.5	150	80.1
Short wall at trash pad	9 - 2.00	0 - 10.00	3 - 6.00	1	26.7	1.0	150	2.0
Stair Platforms								
bottom platform	4 - 0.00	4 - 0.00	2 - 0.00	2	64.0	2.4	150	4.8
top platform	4 - 0.00	4 - 0.00	2 - 0.00	2	64.0	2.4	150	4.8
Training Pier Walkway								
Slab spanning intake bays TOC +16.0	36 - 0.00	3 - 6.00	2 - 0.00	4	1008.0	37.3	150	75.6
Subtract piers	3 - 0.00	3 - 6.00	2 - 0.00	-12	-252.0	-9.3	150	-18.9
				SUBTOTALS	11018.2	408.1		826.4
SAND FILL								
Cells above intake EL-3.5 to EL+7.0	29 - 0.00	33 - 0.00	10 - 6.00	4	40194.0	1488.7	110	2,210.7
Cells next to 250 cfs pipe EL-9.641 to EL+7.0	18 - 0.00	5 - 6.00	15 - 7.69	4	6193.8	229.4	110	340.7
Cells next to 500 cfs pipe EL-7.521 to EL+7.0	18 - 0.00	5 - 6.00	14 - 6.25	4	5750.3	213.0	110	316.3
From EL+10.0 to EL+22.0								
cells near main hallway	25 - 6.00	12 - 0.00	12 - 0.00	4	14688.0	544.0	110	807.8
cells near pipes	18 - 0.00	5 - 6.00	12 - 0.00	4	4752.0	176.0	110	261.4
cell near auxiliary pipes	33 - 0.00	26 - 0.00	12 - 0.00	1	10296.0	381.3	110	566.3
cell near auxiliary pipes	13 - 8.00	60 - 0.00	12 - 0.00	1	9840.0	364.4	110	541.2
cell near stairwell	27 - 9.00	11 - 0.00	12 - 0.00	1	3663.0	135.7	110	201.5
cell near stairwell	27 - 9.00	16 - 0.00	12 - 0.00	1	5328.0	197.3	110	293.0
				SUBTOTALS	100705.2	3729.8		5,538.8
FILL ON FOUNDATION PROJECTIONS								
Projection at EL -16.00	193 - 0.00	108 - 0.00	23 - 0.00	1	479412.0	17756.0	60	14,382.4
Subtract Volume Displaced by Wall Outline	185 - 0.00	104 - 0.00	23 - 0.00	-1	-442520.0	-16389.6	60	-13,275.6
Projection at EL 10.00	258 - 0.00	87 - 0.00	7 - 0.00	1	157122.0	5819.3	60	4,713.7
Subtract Volume Displaced by Wall Outline	252 - 0.00	84 - 0.00	7 - 0.00	-1	-148176.0	-5488.0	60	-4,445.3
				SUBTOTALS	45838.0	1697.7		1,375.1
WATER ON FOUNDATION PROJECTIONS								
Projection at EL -16.00	193 - 0.00	108 - 0.00	23 - 0.00	1	479412.0	17756.0	62.4	14,957.7
Subtract Volume Displaced by Wall Outline	185 - 0.00	104 - 0.00	23 - 0.00	-1	-442520.0	-16389.6	62.4	-13,806.6
Projection at EL 10.00	258 - 0.00	87 - 0.00	7 - 0.00	1	157122.0	5819.3	62.4	4,902.2
Subtract Volume Displaced by Wall Outline	252 - 0.00	84 - 0.00	7 - 0.00	-1	-148176.0	-5488.0	62.4	-4,623.1
				SUBTOTALS	45838.0	1697.7		1,430.1
WATER IN SUCTION BASINS								
Main Pump Bays	33 - 0.00	48 - 0.00	0 - 0.00	4	0.0	0.0	62.4	0.0
Subtract Training Piers	6 - 0.00	3 - 0.00	0 - 0.00	-12	0.0	0.0	62.4	0.0
Auxiliary Pump Bay	98 - 0.00	33 - 0.00	0 - 0.00	1	0.0	0.0	62.4	0.0
Subtract Wall Volume	60 - 0.00	3 - 0.00	0 - 0.00	-1	0.0	0.0	62.4	0.0
				SUBTOTALS	0.0	0.0		0.0
UPLIFT ON FOUNDATION								
Slab at EL -16.00	193 - 0.00	108 - 0.00	38 - 0.00	1	792072.0	29336.0	62.4	24,712.6
Slab at EL 10.00	258 - 0.00	87 - 0.00	9 - 0.00	1	202014.0	7482.0	62.4	6,302.8
Subtract Area Displaced by Wall Outline	185 - 0.00	56 - 0.00	9 - 0.00	-1	-93240.0	-3453.3	62.4	-2,909.1
				SUBTOTALS	900846.0	33364.7		28,106.4

Description	Length (ft) - (in)	Width / Th. (ft) - (in)	Depth / Ht (ft) - (in)	Number	Volume (ft^3)	Volume (yd^3)	γ (pcf)	Weight (short ton)
SUMMARY:								
FOUNDATION CONCRETE					125064.0	4632.0		9,379.8
SUCTION BASIN CONCRETE					42157.8	1561.4		3,161.8
INTAKE BAYS EL-16.0 TO EL-6.0 (to bottom of intermediate slab)					18243.3	675.7		1,368.2
FILL CONCRETE (Intake pipe formation)					34770.7	1287.8		2,607.8
INTAKE BAYS EL-6.0 TO EL-3.5 (assumes slab thickness 2.5 ft)					16555.0	613.1		1,241.6
INTAKE BAYS EL-3.5 TO EL+7.0 (to bottom of pipe gallery slab)					29337.0	1086.6		2,200.3
PIPE GALLERY SLAB EL+7.0 TO EL+10.0 (assumes slab thickness 3.0 ft)					66117.0	2448.8		4,958.8
PIPE GALLERY EL+10.0 TO EL+22.0 (to bottom of operating deck slab)					46042.0	1705.3		3,453.2
OPERATING DECK SLAB EL+22.0 TO EL+25.0 (assumes slab thickness 3.0 ft)					64412.0	2385.6		4,830.9
SERVICE BRIDGE TOC EL+16.0					11018.2	408.1		826.4
SAND FILL					100705.2	3729.8		5,538.8
FILL ON FOUNDATION PROJECTIONS					45838.0	1697.7		1,375.1
WATER ON FOUNDATION PROJECTIONS					45838.0	1697.7		1,430.1
WATER IN SUCTION BASINS					0.0	0.0		0.0
UPLIFT (assumes water to grade, and suction bays dewatered)					900846.0	33364.7		28,106.4

Preliminary Concrete Design

Description

Perform preliminary concrete design for the following components of C43PS-1:
1) Suction basin retaining walls
2) Suction Basin divider walls
3) Suction basin base slab
4) Suction basin training piers

References

Reference:K:\Technical_Programs\Structural\ST084 ACI 318-99 Mathcad Electronic Book.mcd

Reference:K:\Technical_Programs\Structural\Structural Computation Template.mcd

Preliminary structural plans and cross sections

Design Criteria

$EL_{basin} := -16ft$ Elevation of suction basin slab (TOC)

$EL_{top} := 16ft$ Elevation of grade adjacent to wall

$EL_{max\_water} := 8ft$ Maximum water elevation

$\gamma_{water} = 62.4\,pcf$ ASSUMES NO SEAWATER

$\gamma_{conc} = 150\,pcf$

$\gamma_{buoyant} := 60pcf$ buoyant unit weight of soil from Preliminary Geotechnical Design Memo ~~ACCEPT PER DRD~~ IN Geotech Attached

$\gamma_{soil} := \gamma_{buoyant} + \gamma_{water}$ $\gamma_{soil} = 122.4\,pcf$ saturated unit weight of soil

$K_o := 0.5$ at-rest pressure coefficient

$\gamma_{equiv} := (\gamma_{soil} - \gamma_{water})\,K_o + \gamma_{water}$ $\gamma_{equiv} = 92.4\,pcf$ equivalent lateral fluid pressure for retaining wall design

Load Factors

$LF_{LL} := 1.7$ $LF_{DL} := 1.4$

Hydraulic Factors

$HF_{flex} := 1.3$ $HF_{ten} := 1.65$ PER ACI 350

Retaining Wall Design

$H_{wall} := EL_{top} - EL_{basin}$ $\qquad H_{wall} = 32\,ft$

$P_u := LF_{LL}\,\gamma_{equiv}\,H_{wall}$ $\qquad P_u = 5.027\,ksf$

$M_u := 310ft\,\frac{k}{ft}$ See output from STAAD analysis "Wall1" Mx

ok ±2 $\times 1.3 = 403\,\frac{k\text{-}ft}{ft}$ Hf

$V_u := \frac{359psi\ 1ft}{\cancel{HF_{flex}}}$ $\qquad V_u = \cancel{39.766}\,kpf$ 51.7 STAAD model modified to uses 1 thick plate elements

$I_{slab} := 48\,in$

$$SlabWallInput := \begin{bmatrix} \text{"Subject bar location"} & \text{"outside"} \\ \text{"Outside bar size"} & 9 \\ \text{"Inside bar size"} & 9 \\ \text{"Subject bar moment (ft-k/ft)"} & \frac{M_u}{\left(\frac{ft\,k}{ft}\right)} \\ \text{"Shear (k/ft)"} & \frac{V_u}{kpf} \\ \text{"Bar spacing (in)"} & 6 \\ \text{"Outside bar concrete cover (in)"} & 4 \\ \text{"Tolerance (in)"} & 0.25 \\ \text{"Slab thickness (in)"} & \frac{I_{slab}}{in} \\ \text{"Crack control concrete cover to outside bar (in)"} & 1.5 \end{bmatrix}$$

$$SlabWallAnalysisResults\left(SlabWallInput, \frac{f_y}{psi}, \frac{f_c}{psi}\right) = \begin{pmatrix} \text{"Outside Reinforcing Analysis Results:"} & \text{"PASS"} \\ \text{"Slab thickness (inches):"} & 48 \\ \text{"Outside Reinforcing"} & \text{"\#9@6"} \\ \text{"Mu cap (ft-k/ft) / Ratio:"} & \text{"}\cancel{375.439 / 0.826}\text{"} \\ \text{"Vu cap (k) / Ratio:"} & \text{"}\cancel{55.719 / 0.714}\text{"} \\ \text{"Maximum crack control bar spacing, s (in):"} & 11.25 \\ \text{"Minimum bar cutoff extension (in):"} & 43.186 \end{pmatrix}$$

107%
93%

Use a 4'-0" thick wall — ok for Preliminary wall thickness — Need more reinforcing — #10 bars?

Divider Wall Design

$H_{water} := EL_{max\_water} - EL_{basin}$ $\qquad H_{water} = 24\,ft$

$P_u := LF_{LL}\ HF_{flex}\ \gamma_{water}\ H_{water}$ $\qquad P_u = 3.31\,ksf$ included Hydraulic Factor

$M_u := 204ft \cdot \frac{k}{ft}$ ok – High-Sea Rev 1 Model $\qquad$ See output from STAAD analysis "Wall2"

$V_u := \frac{236psi \cdot 1ft}{HF_{flex}}$ ok $\qquad V_u = 26.142\,kpf$ $\qquad$ STAAD model uses 1' thick plate elements

$T_{slab} := 36\,in$

$$SlabWallInput := \begin{bmatrix} \text{"Subject bar location"} & \text{"outside"} \\ \text{"Outside bar size"} & 9 \\ \text{"Inside bar size"} & 9 \\ \text{"Subject bar moment (ft-k/ft)"} & \frac{M_u}{\left(\frac{ft\,k}{ft}\right)} \\ \text{"Shear (k/ft)"} & \frac{V_u}{kpf} \\ \text{"Bar spacing (in)"} & 6 \\ \text{"Outside bar concrete cover (in)"} & 4 \\ \text{"Tolerance (in)"} & 0.25 \\ \text{"Slab thickness (in)"} & \frac{T_{slab}}{in} \\ \text{"Crack control concrete cover to outside bar (in)"} & 1.5 \end{bmatrix}$$

$$SlabWallAnalysisResults\left(SlabWallInput, \frac{f_y}{psi}, \frac{f'_c}{psi}\right) = \begin{pmatrix} \text{"Outside Reinforcing Analysis Results:"} & \text{"PASS"} \\ \text{"Slab thickness (inches):"} & 36 \\ \text{"Outside Reinforcing"} & \text{"\#9@6"} \\ \text{"Mu_cap (ft-k/ft) / Ratio:"} & \text{"267.439 / 0.763"} \\ \text{"Vu_cap (k) / Ratio:"} & \text{"40.237 / 0.65"} \\ \text{"Maximum crack control bar spacing, s (in):"} & 11.25 \\ \text{"Minimum bar cutoff extension (in):"} & 31.186 \end{pmatrix}$$

Use 3'-0" thick divider walls ok

ok

Base Slab Design

Design the suction basin base slab for the full hydrostatic force of water acting from the bottom of base slab to the ground surface. Analyze the slab as fixed at the face of the suction basin walls

$T_{slab} := 6ft$ Assumed base slab thickness

$H_{water} := EL_{top} - EL_{basin} + T_{slab}$ $H_{water} = 38\,ft$

$P_u := LF_{IL}\, HF_{flex}\, \gamma_{water}\, H_{water} - LF_{DL}\, T_{slab}\, \gamma_{conc}$ $P_u = 3.98\,ksf$

$M_u := 311ft\,\frac{k}{ft}$ See output from STAAD analysis "Slab1"

$V_u := \frac{330psi\,1ft}{HF_{flex}}$ $V_u = 36.554\,kpf$ STAAD model modified to use 6' thick plate elements

$$SlabWallInput := \begin{bmatrix} \text{"Subject bar location"} & \text{"outside"} \\ \text{"Outside bar size"} & 9 \\ \text{"Inside bar size"} & 9 \\ \text{"Subject bar moment (ft-k/ft)"} & \frac{M_u}{\left(\frac{ft\,k}{ft}\right)} \\ \text{"Shear (k/ft)"} & \frac{V_u}{kpf} \\ \text{"Bar spacing (in)"} & 6 \\ \text{"Outside bar concrete cover (in)"} & 4 \\ \text{"Tolerance (in)"} & 0.25 \\ \text{"Slab thickness (in)"} & \frac{T_{slab}}{in} \\ \text{"Crack control concrete cover to outside bar (in)"} & 1.5 \end{bmatrix}$$

$$SlabWallAnalysisResults\left(SlabWallInput, \frac{f_y}{psi}, \frac{f_c}{psi}\right) = \begin{pmatrix} \text{"Outside Reinforcing Analysis Results:"} & \text{"PASS"} \\ \text{"Slab thickness (inches):"} & 72 \\ \text{"Outside Reinforcing"} & \text{"\#9@6"} \\ \text{"Mu_cap (ft-k/ft) / Ratio:"} & \text{"591.439 / 0.526"} \\ \text{"Vu_cap (k) / Ratio:"} & \text{"86.685 / 0.422"} \\ \text{"Maximum crack control bar spacing, s (in):"} & 11.25 \\ \text{"Minimum bar cutoff extension (in):"} & 67.186 \end{pmatrix}$$

Use 6'-0" thick base slab — Conservative

Training Pier Design

The training piers will be equally spaced between the suction basin divider walls to allow the use of one standard stoplog width Vertically oriented stoplogs will be placed at mid-depth of the training pier

$Width_{pier} := 3ft$ assumed width of training pier

$Depth_{pier} := 6ft$ assumed depth of training pier

$H_{pier} := EL_{top} - EL_{basin}$ $H_{pier} = 32ft$

$Clearance := \frac{33ft - 3 \cdot Width_{pier}}{4}$ $Clearance = 6\,ft$ clear distance between training piers

$W_{trib\_pier} := Clearance + Width_{pier}$ $W_{trib\_pier} = 9\,ft$ tributary width of single pier subjected to hydrostatic loads

$H_{water} := EL_{max\_water} - EL_{basin}$ $H_{water} = 24\,ft$

$P_u := LF_{DL} \cdot Width_{pier} \cdot Depth_{pier} \cdot H_{pier} \cdot \gamma_{conc}$ $P_u = 121\,k$

$M_u := LF_{LL} \cdot \gamma_{water} \cdot W_{trib\_pier} \cdot \frac{H_{water}^3}{6}$ $M_u = 2200\,ft\,k$ < $\phi M_n := 5219\,ft \cdot k$ from PCACOL analysis

$V_u := LF_{LL} \cdot \gamma_{water} \cdot W_{trib\_pier} \cdot \frac{H_{water}^2}{2}$ $V_u = 275\,k$

$b := Width_{pier}$

$d := Depth_{pier}$ $barsz := 4$ $nlegs := 4$ $s := 12\,in$ $f_c := 4\,ksi$ $f_y := 60\,ksi$

$\phi V_{n\_rect\_beam}\left(\frac{b}{in}, \frac{d}{in}, barsz, nlegs, \frac{s}{in}, \frac{f_c}{psi}, \frac{f_y}{psi}\right) = (\text{"Factored Shear Capacity (k) = "}\ 461.899)$ > $V_u = 275\,k$ OK

Use 6' x 3' Training Piers

5/27/06

12a

PUMPING STATION STRUCTURE C43PS-1

Project Considerations

Pumping Station C43PS-1 will consist of a steel and reinforced concrete structure that will house the main pumps and pumping motors for the C-43 West Storage Reservoir Project. The structure will be located adjacent to the downstream side of the dam and will have plan dimensions of approximately 200 feet (parallel with the dam) by 100 feet (measured at the intake level) It will have a base slab elevation on the intake side of the structure of -16 feet with respect to the North American Vertical Datum of 1988 (NAVD). The structure bottom will step up to elevation +10 feet NAVD for a distance of about 30 feet in the area between the intake bay and the toe of the dam. The existing surrounding grades at the site of the proposed construction are approximately +16 to +17 feet NAVD

Below ground portion of the structure will likely be constructed within a dewatered cofferdam. The components of the cofferdam have not been finalized, but may include steel sheet piling and a concrete tremie seal

The average pressure imposed by the structure on the foundation soils is estimated to be 3 5 kips per square foot (ksf).

The proposed trash rake will be constructed as an independent structure away from the pumping station intake bay. It will be a pile supported bridge that spans the intake lagoon and utilizes driven precast concrete piles for support

Subsurface Conditions

Subsurface conditions in the project vicinity were explored by drilling four Standard Penetration Test (SPT) borings (CB-108 thru 111), each to a depth of 100 feet below surface grades The borings were drilled using a truck mounted Central Mine Equipment CME-55 drilling rig and mud rotary methodology Samples of the subsurface materials were obtained using a standard split barrel in accordance with procedures and nomenclature described in ASTM D 1586 The samples were obtained continuously in the upper 15 feet, at 2 5-foot centers between 15 and 75 feet deep, and at 5-foot centers thereafter to the terminal depth of exploration. The samples were placed in sealed glass jars, labeled and transported to the laboratory. Undisturbed samples of the clays were obtained using thin walled Shelby tube samplers Groundwater levels were measured in the borings and then the boreholes were sealed with cement grout in a bottom to top manner.

Results of the borings indicate that the proposed pumping station site is underlain by approximately 20 feet of relatively clean sands, clayey sands and/or silty sands over a moderately thick (i e 20 feet) layer of clay, and then clayey sands and relatively clean to silty sands to the maximum depth explored The SPT data indicates that the upper sands are loose to medium dense and the clays are firm to stiff The deeper sands are medium dense to very dense Subsurface profiles for the borings are provided on the following sheets

Groundwater levels were measured in the borings at depths of approximately 2 to 3 feet below surface grades.

One dimensional consolidation testing on undisturbed samples of the clays has shown that the clays are over-consolidated. The maximum past pressure was estimated to be approximately 5 to 6 ksf. Laboratory test data are provided in the Appendix to this report.

Geotechnical Overview

Based upon the results of the borings drilled for the proposed pumping station, it appears that the intake bay level will bottom on the medium dense to very dense sands that underlie the clays. These materials are suitable for support of the pumping station structure on a reinforced concrete mat foundation. The subsoils have sufficient strength to provide a factor of safety of at least 3 with respect to bearing capacity. Total settlement of the proposed structure should be less than 1 inch under the anticipated 3.5 ksf bearing pressure. The portion of the structure that bottoms at elevation +10 feet NAVD requires special consideration since this feature will bottom above the clay layer. In order to limit the potential for differential settlement between portions of the structure founded at different levels, we recommend the clays be over-excavated in this area and be replaced with tremied concrete and/or granular, structural backfill.

We expect that the sheet pile supported excavation (i.e. cofferdam) will include a tremie seal at its base. The cofferdam will likely require soil anchors at several levels in order to limit sheet pile deflections and moments to within tolerable levels. Design of the dewatering system should be by the contractor. However, for the purpose of this evaluation, we have assumed that the dewatering will likely consist of pumping from a rim ditch located around the perimeter of the excavation.

Recommended Geotechnical Design Parameters

The results of the field exploration and laboratory testing were used to estimate geotechnical parameters for preliminary design of the project components. These parameters include moist and buoyant unit weights, angle of internal friction, angle of wall friction, cohesion, adhesion, modulus of subgrade reaction, and active, passive and at-rest lateral earth pressure coefficients. Values for these parameters were selected based upon standard correlations available in the geotechnical literature that are dependent on SPT N-values. Specifically unit weights and friction angles were obtained from correlations with N-values provided by the Florida Department of Transportation (FDOT). Modulus of subgrade reaction values were estimated using technical documentation for the computer program COM624P by Shih-Tower Wang and Dr. Lymon C. Reese. A summary of the geotechnical design parameters is presented in the table that follows.

Depth (feet)	Unit Weight (pcf)		ϕ Degrees	δ Degrees	C (psf)	Coefficient of Lateral Earth Pressure			k (pci)
	Moist	Buoyant				Ka	Ko	Kp	
0-20	110	60	30	18	0	0.33	0.50	3.00	25
20-40	100	43	0	0	1000	1.00	1.00	1 00	500
40-100	120	65	34	20	0	0.28	0.44	3.54	125

Notes: ϕ indicates angle of internal friction (degrees)
δ indicates angle of wall friction (degrees)
C indicates cohesion
Ka indicates coefficient of active lateral earth pressure
Ko indicates coefficient of at-rest lateral earth pressure
Kp indicates coefficient of passive lateral earth pressure
k indicates modulus of horizontal subgrade reaction
Depth refers to existing land surface grade

The global stability of the proposed steel sheet pile wall was evaluated using the computer program CWALSHT developed by the U S. Army Corps of Engineers (USACE) The lateral load resistance of driven piling for the trash rake was estimated using the computer program COM624P developed by Shih-Tower Wang and Dr Lymon C Reese Variables in the analysis beside soil and rock properties include pile length and butt width, pile stiffness, lateral load, and unsupported pile length Axial pile capacity of the dock piles was estimated using the *Static Pile Bearing Analysis Program - SPT97* published by the Structure Design Office of the Florida Department of Transportation. The program uses a methodology based on empirical correlations between cone penetrometer tests and Standard Penetration Tests for typical Florida soils and rock The SPT97 allowable pile capacity is equal to the pile's ultimate side friction resistance plus one-third of its mobilized end-bearing capacity, all divided by 2 (factor of safety)

Excavation Support

The pumping station excavation was modeled as a steel sheet pile supported excavation made in two stages as follows:

1. The excavation was made to elevation -30 feet NAVD. The groundwater elevation on both side of the sheeting was assumed to be +12 feet NAVD. The top of ground elevation on the exterior side of the excavation was assumed to be +16 feet NAVD

2. A 14-foot thick concrete seal was placed in the excavation bottom up to elevation -16 feet NAVD. Groundwater levels were then lowered to elevations 0 feet and -16 feet NAVD on the exterior and interior sides of the excavation.

A PZ40 steel sheet pile section was used to provide lateral support for the excavation sidewalls. The sheet pile has a moment of inertia (I) of 490.8 inch^4/foot and a section modulus (S) of 60.7 inch^3/foot. Soil anchors were assumed to act at elevations +8 feet, 0 feet and -8 feet NAVD. The results of the external stability analysis are presented in the following table.

Result	Stage 1	Stage 2
Wall Bottom Elevation (feet-NAVD)	-50	-17
Maximum Moment (foot-pounds)	72,650	24,800
Elevation of Maximum Moment (feet-NAVD)	-24.2	-5.2
Maximum Stress (ksi)	14.4	4.9
Maximum Deflection (inches)	1.00	0.2
Elevation of Maximum Deflection (feet-NAVD)	-50	-5
Anchor Pull (kips/foot)		
At Elevation +8	3.6	5.9
At Elevation 0	3.0	3.0
At Elevation -8	3.0	3.0

The 14-foot thick concrete seal was evaluated for the case when the excavation is dewatered to elevation -16 feet NAVD and assuming groundwater levels are maintained by pumping from the rim ditch no higher than elevation 0 feet NAVD. Based upon these assumptions, a factor of safety of 1.23 is estimated against buoyant rise of the seal.

Structural Backfill

Backfill placed adjacent to the structure walls and/or used to replace the over-excavated volume should consist of reasonably well-graded sands (ASTM D 2487) having a maximum size of 4 inches. It should be placed in 12-inch (loose measure) horizontal lifts and be compacted to at least 95 percent of the maximum dry density determined in accordance with the Standard Proctor (ASTM D 698).

Lateral earth pressures associated with the structural backfill should be calculated based upon moist and buoyant unit weights of 115 and 60 pounds per cubic foot, respectively, and active, at-rest and passive earth pressure coefficients of 0.30, 0.47 and 3.30, respectively. The resulting equivalent fluid pressures should be factored to account for allowable wall strain.

Pumping Station Foundation Support

The proposed pumping station structure may be supported on a reinforced concrete mat foundation designed for an allowable bearing pressure of 3.5 ksf This contact pressure is sufficiently modest so as to not create a concern for shearing failure in the materials that are to support the substructure. Design of the mat foundation should seek to limit the contact pressure imposed on the subgrade materials under wind overturning effects to 125 percent of that induced by the dead and reduced live loads.

Considering the presence of the relatively dense sand stratum found in the borings just below the proposed structure bottom level, and the proposed concrete tremie seal, we estimate that the total settlement beneath the center of the structure will be approximately 2 inches. Edge settlements of the mat in the transverse direction are anticipated to be about 75 percent of the center settlement, and those in the longitudinal dimension about 60 percent.

Foundation settlements should occur virtually as rapidly as the loads are applied to the mat owing to the granular nature of the profile components that will be stressed by the imposed structure loads

Lateral resistance of the foundation to wind shear will be provided by earth pressure mobilized on the vertical faces of substructure located orthogonal to and leeward of the direction of thrust and shearing forces acting at the footing–subgrade interface Calculations for lateral resistance should be based upon the geotechnical design parameters provided earlier in this report, and a friction factor of 0 40 for mass concrete in contact with granular soils. Use of these values assumes that the substructures can safely sustain a horizontal translation on the order of three eighth to one half inch. Horizontal resistance determined in accordance with the above recommended values should be factored for safety. We suggest that the safety factor be not less than 1 5.

Proposed Trash Rake

The proposed trash rake is to be separated from the pumping station structure We understand that the preliminary design calls for a pile supported structure We estimate that 18-inch square precast concrete piles, driven to a tip elevation of -50 feet NAVD will be capable of supporting 100 tons in axial compression. Pile tip elevations and corresponding axial compressive capacities are presented on Sheets _____

However, considering the anticipated lagoon bottom elevation of -16 feet NAVD, we expect that the unsupported pile lengths will be of the order of 30 feet or somewhat more The following table provides the results of the lateral load analysis for the piling used to support the trash rake The pile heads were assumed to be fixed against rotation within a rigid pile cap

Lateral Load (kips)	Pile Head Deflection (inches)		Maximum Moment (inch-kips)	
	18-Inch	24-Inch	18-Inch	24-Inch
2	0.71	0.22	525	489
4	1.49	0.45	1070	993
6	2.34	0.70	1660	1510
8	3.24	0.96	2210	2030
10	4.21	1.24	2810	2550

The deflections presented in the table are considered more than normal, largely owing to the relatively long unsupported pile lengths The flexural stresses in the pile section that results from these deflections should be checked and compared with the pile's allowable stress The minimum pile length required for lateral stability is 50 feet (i e 18 feet of embedment). Lateral load-deflection and lateral load-moment graphs are provided on Sheets _____

Prestressed concrete piles should be driven to the minimum tip elevations required for lateral stability and to the depth required to achieve the design axial compressive capacity. Eighteen-inch square prestressed concrete piles (PCP) driven to a tip elevation of -50 feet NAVD should be capable of achieving an allowable axial compressive capacity of 100 tons Piling driven to a tip elevation of -34 feet NAVD (or below) should be considered stable from a lateral load standpoint Twenty-four inch square PCPs driven to a similar tip elevation should be capable of supporting 125 tons of vertical compressive load Uplift resistance of such piling may be taken as one-third of the allowable load in compression

We recommend that the piles be installed in accordance with the FDOT's *Supplemental Specification Section A455 Structures Foundations - Piling* The piles should be driven from the bottom of the intake basin to the minimum tip elevation or to the depth required to achieve the required axial capacity, whichever is deeper. They should be driven with a steam, air, or diesel hammer developing not less than 45,000 foot-pounds of energy per blow Driving criteria for production piles should be established as part of a test pile installation and monitoring program We recommend that two test piles be installed for the trash rake at locations coincident with those of production piles

The test pile installation should be monitored by recording blows per foot over the full length of the pile's penetration Data from the test pile monitoring should then be input to the computer program Wave Equation Analysis of Piles (WEAP) for selection of the finalized driving criteria and for ordering production pile lengths

PROGRAM ST071 - LOADED PLATE STAADPro FINITE ELEMENT MODELER

For User Notes see notebook tab labeled "User's Notes"

Copy Staad Input To Clipboard | Print Staad Input | Print Spreadsheet

Notes placed in STAAD3 file:

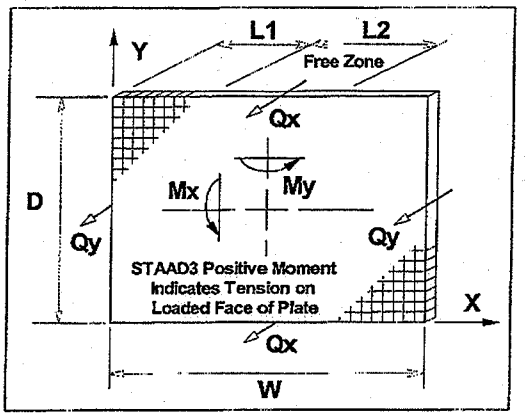

Figure 1. - STAAD3 Finite Element Plate Model and Stress Notations

INPUT: Model Data (see Figure 1.)			
		Dimension W(feet):	48.00
		Dimension D (feet):	33.00
		Top edge fixed (FX), pinnned (PN) or free (FR):	PN
Top edge free zone (no supports in L2 zone); Valid if top edge pinned or fixed; Enter "N" if not applicable:			
L1 (feet):	4.00	L2 (feet):	44.00
		Bottom edge fixed (FX), pinnned (PN) or free (FR):	FX
		Left edge fixed (FX), pinnned (PN) or free (FR):	FR
		Right edge fixed (FX), pinnned (PN) or free (FR):	FX
		Maximum element dimension (feet):	2.000
		STAAD3 filename (8 characters.std):	Wall1.std
Input Check:	O.K.		
	O.K.		

OUTPUT: Model Statistics	
Number of Joints:	450
Number of Elements:	408
Rows of Elements:	17
Columns of Elements:	24
Element X dimension (feet):	2.000
Element Y dimension (feet):	1.941

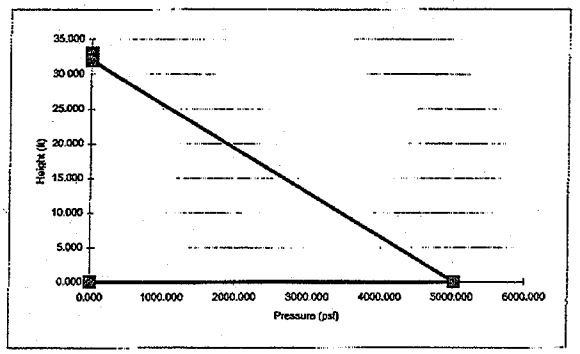

Figure 2. - Pressures Acting on Plate Specified in Table Below

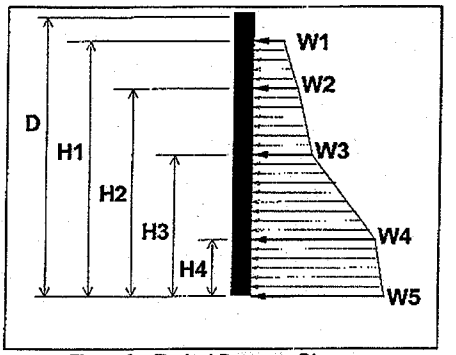

Figure 3. - Typical Pressure Diagragm Applied Normal to Plate

D = 33.00 Feet

INPUT: Load Data (see Figure 2. and 3.)			
Notes: At a minimum W1 W5 H1 and H5 must have values Enter 'N' for intermediate W and H values not used No required order of input for intermediate values	Position	W# (psf)	H# (ft)
	1	0.00	32.00
	2	N	N
	3	N	N
	4	N	N
	5	5027.00	0.00
Load Factor:	1.00		
Input Check:	O.K.		

14

LWS 5/23/06

```
STAAD SPACE - PROJECT 17300 09 02 - C43PS-1 - Suction B
* FILE CREATED BY:D  Dreyer
* FILENAME = Wall1 std
*
*
*
*
* ######################
* FOLLOWING IS INPUT DATA ECHOED FROM ST071"
* WIDTH = 48 00 ft
* DEPTH = 33 00 ft
* TOP EDGE = PN
* FREE EDGE BETWEEN  L1 = 4 00 FT  AND L2 = 44 00 FT
* BOT EDGE = FX
* LEFT EDGE = FR
* RIGHT EDGE = FX
* MAX ELEMENT DIM = 24 000 INCHES
*
* LOADING
* W1 = 0 00 PSF    H1 = 32 00 FT
* W2 = 0 PSF    H2 = 0 FT
* W3 = 0 PSF    H3 = 0 FT
* W4 = 0 PSF    H4 = 0 FT
* W5 = 5,027.00 PSF    H5 = 0 00 FT
* LOAD FACTOR = 1 00
* ######################
UNITS KIP FEET
INPUT WIDTH 72
INPUT NODESIGN
*************************************************
JOINT COORDINATES
1 0.000 0 000 0.000 25 48 000 0 000 0 000
REPEAT 17 0.000 1.941 0.000
*************************************************
ELEMENT INCIDENCES
1 1 26 27 2 TO 24
REPEAT 16 24 25
*************************************************
SUPPORTS
*CORNERS
1 FIXED BUT MY
25 FIXED
426 PINNED
450 FIXED BUT MX
*BOTTOM EDGE
2 TO 24 FIXED
* TOP EDGE
427 TO 428 PINNED
*LEFT EDGE
*
*RIGHT EDGE
50 TO 425 BY 25 FIXED
UNIT INCH
*************************************************
ELEMENT PROPERTY
1 TO 408 THICKNESS 12.00
*************************************************
UNIT FEET
*************************************************
CONSTANTS
E CONCRETE ALL
*************************************************
LOAD 1
ELEMENT LOAD
1 TO 24 PR GZ 4 8745
25 TO 48 PR GZ 4 5696
49 TO 72 PR GZ 4 2646
73 TO 96 PR GZ 3.9597
97 TO 120 PR GZ 3.6547
121 TO 144 PR GZ 3 3498
145 TO 168 PR GZ 3 0448
169 TO 192 PR GZ 2 7399
193 TO 216 PR GZ 2 4350
217 TO 240 PR GZ 2 1300
241 TO 264 PR GZ 1 8251
265 TO 288 PR GZ 1 5201
```

1 ft thick

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289 TO 312 PR GZ 1 2152
313 TO 336 PR GZ 0 9102
337 TO 360 PR GZ 0 6053
361 TO 384 PR GZ 0 3003
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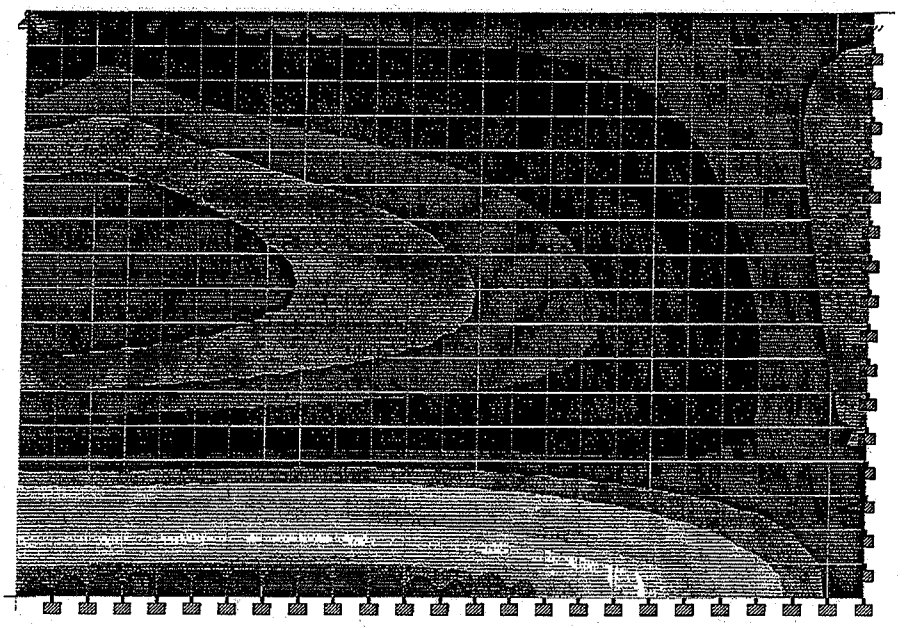
************************************************************
PERFORM ANALYSIS
*
PRINT SUPPORT REACTIONS ALL
FINISH
```

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Wall1.std	Date/Time 17-May-2006 19:15	

5/25/06

MX
Kip-ft/ft

- <= -157
- -128
- -99 1
- -69 9
- -40 7
- -11 5
- 17 7
- 46 9
- 76 1
- 105
- 134
- 164
- 193
- 222
- 251
- 280
- >= 310



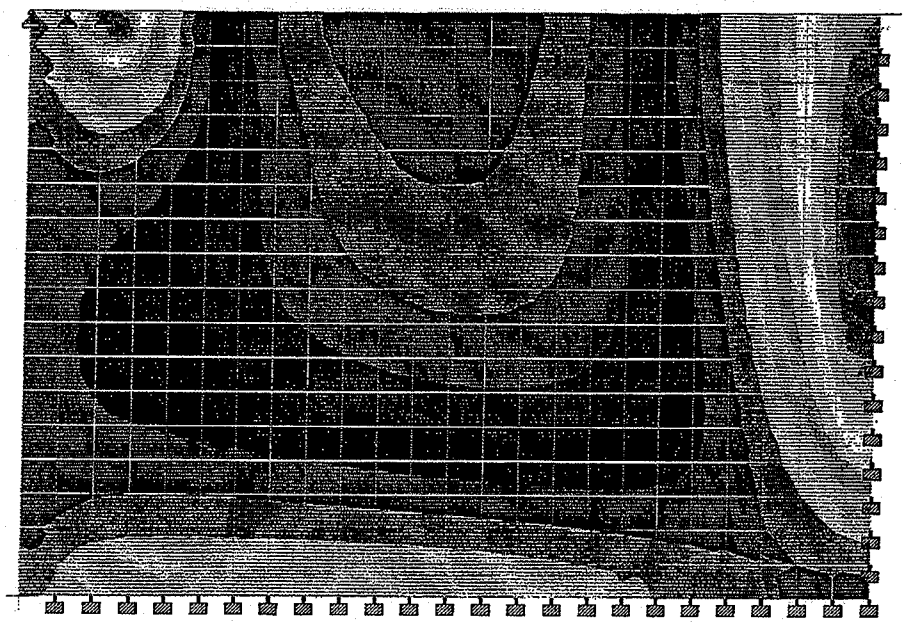
Load 1

17

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
Job Title C43PS-1	Part		
	Ref		
	By DRD	Date 06-May-06	Chd Sandman
Client SFWMD	File Wall1.std	Date/Time 17-May-2006 19:15	

5/23/06

MY
Kip-ft/ft
<= -102
-83 5
-64 7
-46
-27 2
-8 43
10 3
29 1
47 9
66 6
85 4
104
123
142
160
179
>= 198



Load 1

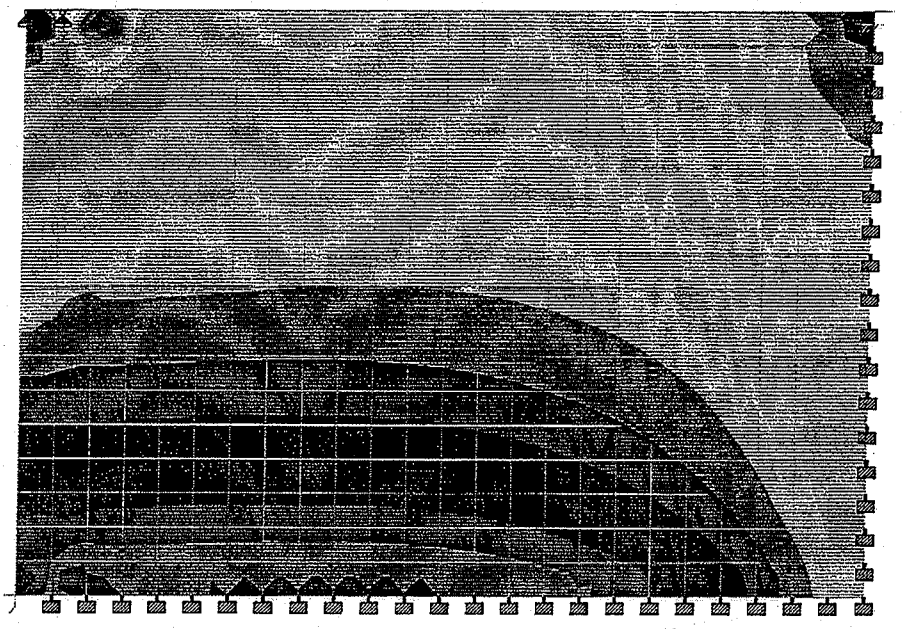
18

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Wall1.std	Date/Time 17-May-2006 19:15	

5/23/06

SQX
ksi

- <= -0 496
- -0 427
- -0 359
- -0 291
- -0 223
- -0 154
- -0 086
- -0 018
- 0 050
- 0 119
- 0 187
- 0 255
- 0 324
- 0 392
- 0 460
- 0 528
- >= 0 597

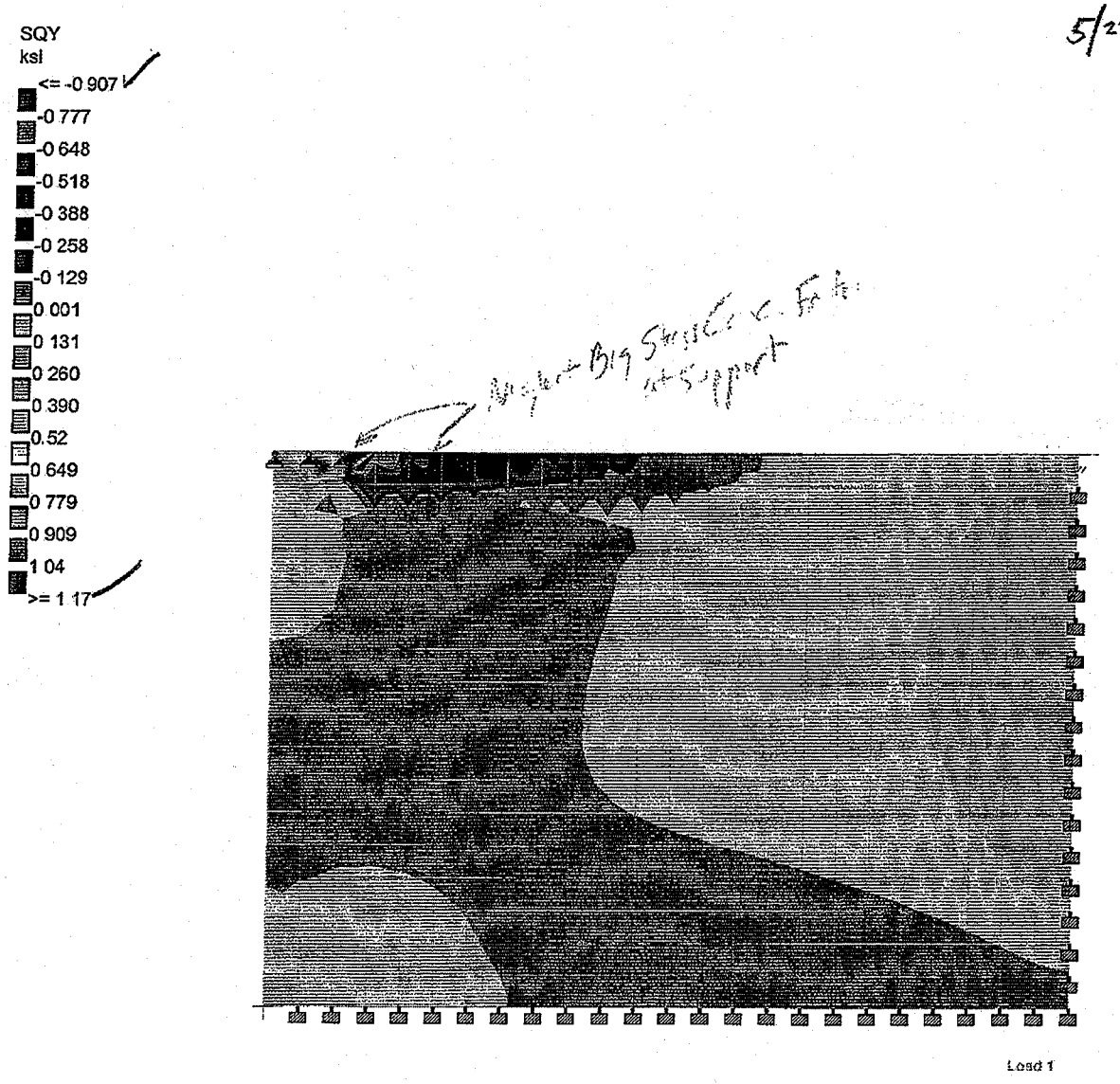


Load 1

19

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd Sandhan
Client SFWMD	File Wall1.std	Date/Time 17-May-2006 19:15	

5/23/06



STANLEY CONSULTANTS

Computed by: D. Dreyer — Date 17-May-06
Checked by: Sandhgar — Date 5/23/06
Approved by: — Date

Project 17300.09.02 — Page No 20
Subject Structural
C43PS-1
Suction Basin Divider Walls
Sheet No ____ of ____

PROGRAM ST071 - LOADED PLATE STAADPro FINITE ELEMENT MODELER

For User Notes see notebook tab labeled User's Notes"

Copy Staad Input To Clipboard | Print Staad Input | Print Spreadsheet

Notes placed in STAAD3 file:

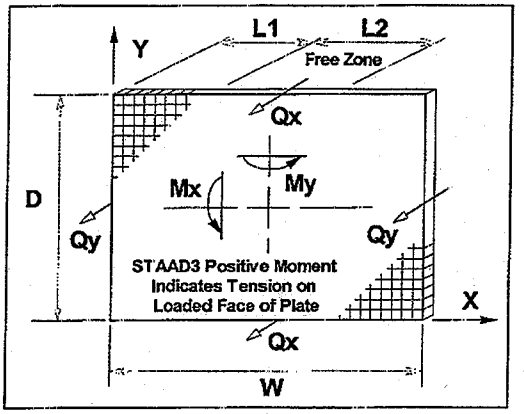


Figure 1. - STAAD3 Finite Element Plate Model and Stress Notations

INPUT: Model Data (see Figure 1.)	
Dimension W(feet):	48.00
Dimension D (feet):	33.00
Top edge fixed (FX), pinnned (PN) or free (FR):	PN
Top edge free zone (no supports in L2 zone); Valid if top edge pinned or fixed; Enter "N" if not applicable:	
L1 (feet): 4.00	L2 (feet): 44.00
Bottom edge fixed (FX), pinnned (PN) or free (FR):	FX
Left edge fixed (FX), pinnned (PN) or free (FR):	FR
Right edge fixed (FX), pinnned (PN) or free (FR):	FX
Maximum element dimension (feet):	2.000
STAAD3 filename (8 characters.std):	Slab1.std
Input Check:	O.K.
	O.K.

OUTPUT: Model Statistics	
Number of Joints:	450
Number of Elements:	408
Rows of Elements:	17
Columns of Elements:	24
Element X dimension (feet):	2.000
Element Y dimension (feet):	1.941

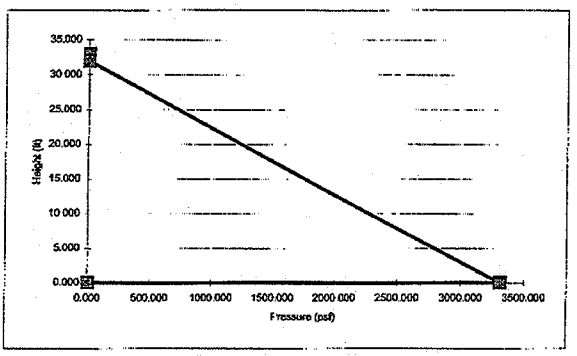


Figure 2. - Pressures Acting on Plate Specified in Table Below

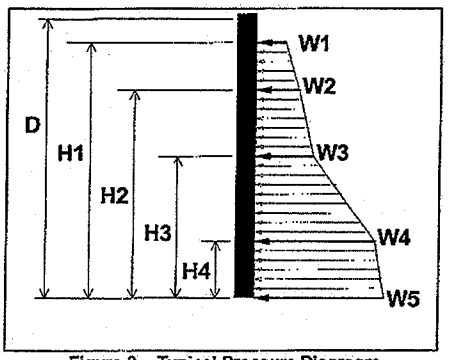
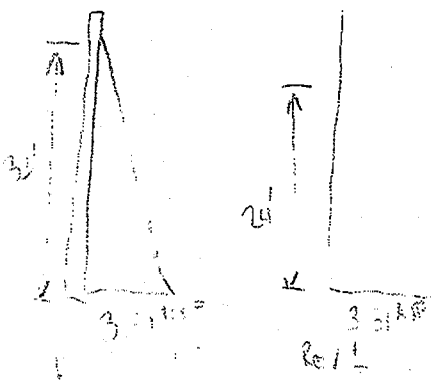


Figure 3. - Typical Pressure Diagragm Applied Normal to Plate

D = 33.00 Feet

INPUT: Load Data (see Figure 2. and 3.)			
Notes: At a minimum W1 W5 H1 and H5 must have values Enter 'N' for intermediate W and H values not used No required order of input for intermediate values	Position	W# (psf)	H# (ft)
	1	0.00	[illegible]
	2	N	N
	3	N	N
	4	N	N
	5	3310.00	0.00
Load Factor:	1.00		
Input Check:	O.K.		

24' Ht of water



21

LLS
5/23/06

```
STAAD SPACE - PROJECT 17300 09 02 - C43PS-1 - Suction B
* FILE CREATED BY:D  Dreyer
* FILENAME = Slab1 std
*
*
*
*
* ######################
* FOLLOWING IS INPUT DATA ECHOED FROM ST071'
* WIDTH = 48 00 ft
* DEPTH = 33 00 ft
* TOP EDGE = PN
* FREE EDGE BETWEEN  L1 = 4 00 FT  AND L2 = 44 00 FT
* BOT EDGE = FX
* LEFT EDGE = FR
* RIGHT EDGE = FX
* MAX ELEMENT DIM = 24 000 INCHES
*
* LOADING
* W1 = 0 00 PSF    H1 = 32.00 FT
* W2 = 0 PSF    H2 = 0 FT
* W3 = 0 PSF    H3 = 0 FT
* W4 = 0 PSF    H4 = 0 FT
* W5 = 3,310.00 PSF    H5 = 0 00 FT
* LOAD FACTOR = 1 00
* ######################
UNITS KIP FEET
INPUT WIDTH 72
INPUT NODESIGN
*****************************************************
JOINT COORDINATES
1 0.000 0 000 0 000 25 48 000 0 000 0 000
REPEAT 17 0.000 1.941 0.000
*****************************************************
ELEMENT INCIDENCES
1 1 26 27 2 TO 24
REPEAT 16 24 25
*****************************************************
SUPPORTS
*CORNERS
1 FIXED BUT MY
25 FIXED
426 PINNED
450 FIXED BUT MX
*BOTTOM EDGE
2 TO 24 FIXED
* TOP EDGE
427 TO 428 PINNED
*LEFT EDGE
*
*RIGHT EDGE
50 TO 425 BY 25 FIXED
UNIT INCH
*****************************************************
ELEMENT PROPERTY
1 TO 408 THICKNESS 12.00
*****************************************************
UNIT FEET
*****************************************************
CONSTANTS
E CONCRETE ALL
*****************************************************
LOAD 1
ELEMENT LOAD
1 TO 24 PR GZ 3.2096
25 TO 48 PR GZ 3 0088
49 TO 72 PR GZ 2 8080
73 TO 96 PR GZ 2.6072
97 TO 120 PR GZ 2.4064
121 TO 144 PR GZ 2 2057
145 TO 168 PR GZ 2 0049
169 TO 192 PR GZ 1 8041
193 TO 216 PR GZ 1 6033
217 TO 240 PR GZ 1 4025
241 TO 264 PR GZ 1 2017
265 TO 288 PR GZ 1 0009
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24' - See Rev 1 Stand Model

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289 TO 312 PR GZ 0 8001
313 TO 336 PR GZ 0 5993
337 TO 360 PR GZ 0 3985
361 TO 384 PR GZ 0 1977
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PERFORM ANALYSIS
*
PRINT SUPPORT REACTIONS ALL
FINISH
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23

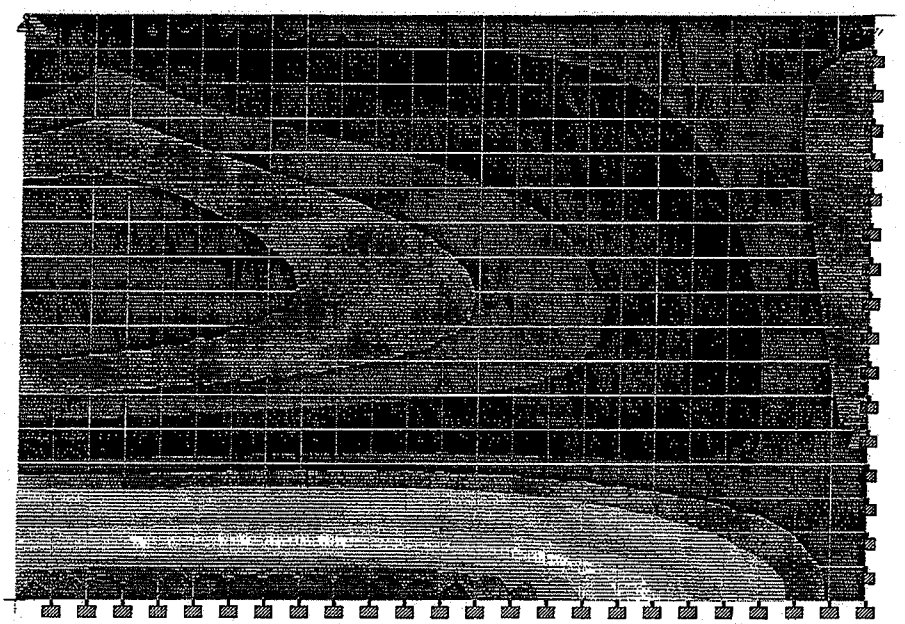
Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.00	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd Sandlman
Client SFWMD	File Wall2.std	Date/Time 17-May-2006 19:05	

5/23/06

MX
Kip-ft/ft
<= -104
-84 5
-65 2
-46
-26 8
-7 58
11 6
30 9
50 1
69 3
88 5
108
127
146
165
185
>= 204 VS

135 k-ft

for Rev 1
Model



Load 4

24

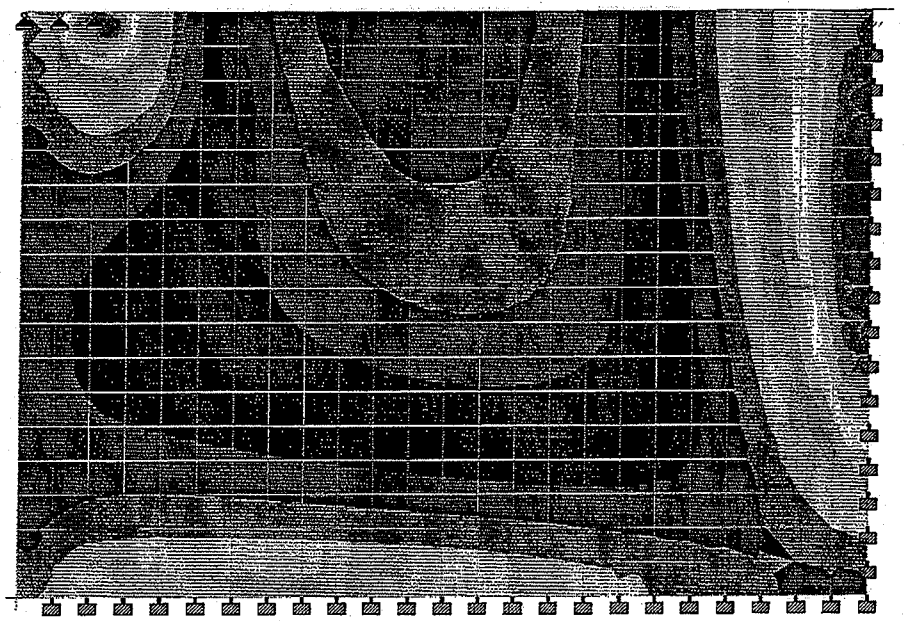
Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.00	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd LWS
Client SFWMD	File Wall2.std	Date/Time 17-May-2006 19:05	

5/23/06

MY
Kip-ft/ft

- <= -67 3
- -55
- -42 6
- -30 3
- -17 9
- -5 55
- 6 8
- 19 2
- 31 5
- 43 9
- 56 2
- 68 6
- 80.9
- 93 3
- 106
- 118
- >= 130

VS
69.8 k-ft/ft
Rev 1 Model



Load 1

25

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.00	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Wall2.std	Date/Time 17-May-2006 19:05	

5/23/06

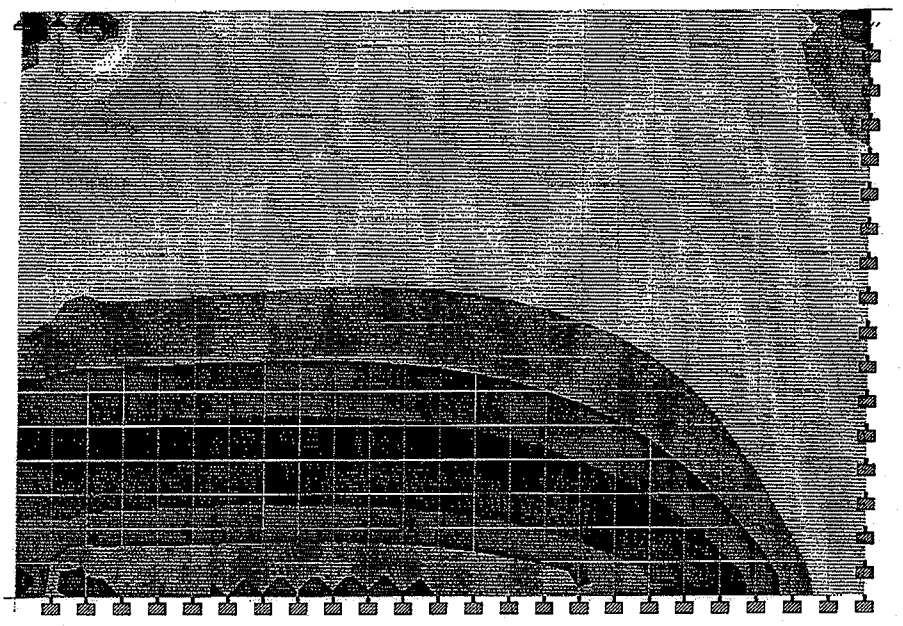
SQX
ksi

- <= -0 326
- -0 281
- -0 236
- -0 192
- -0 147
- -0 102
- -0 057
- -0 012
- 0.033
- 0 078
- 0 123
- 0 168
- 0 213
- 0 258
- 0 303
- 0 348
- >= 0.393

vs 163 psi

Rev 1

Staad

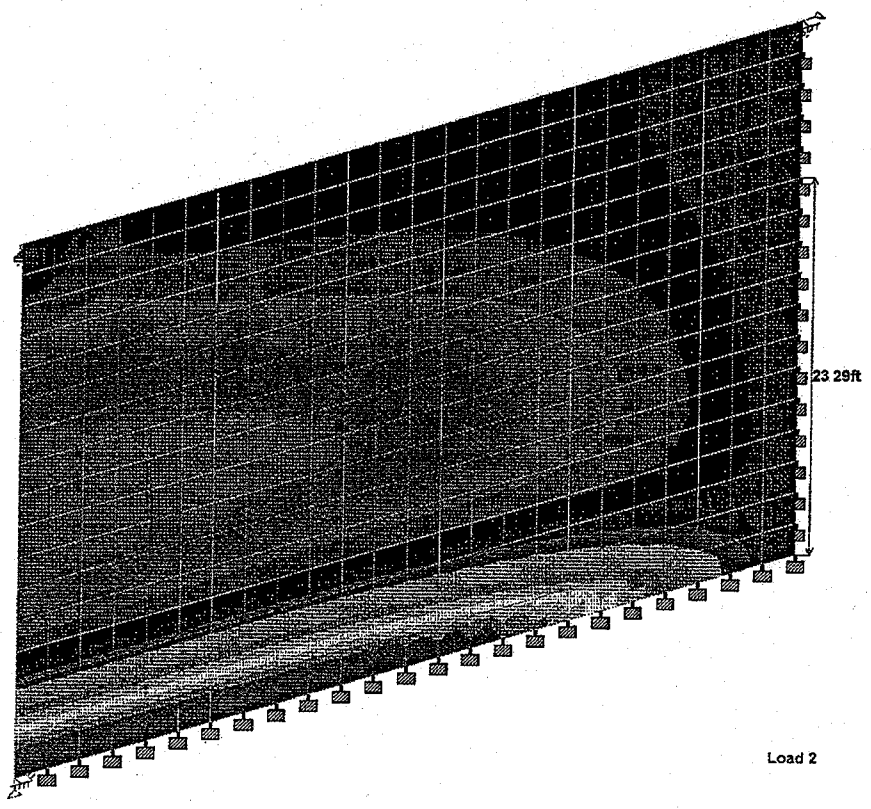


Load 1

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.00	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Wall2_rev1.std	Date/Time 23-May-2006 15:31	

MX
lb-in/in

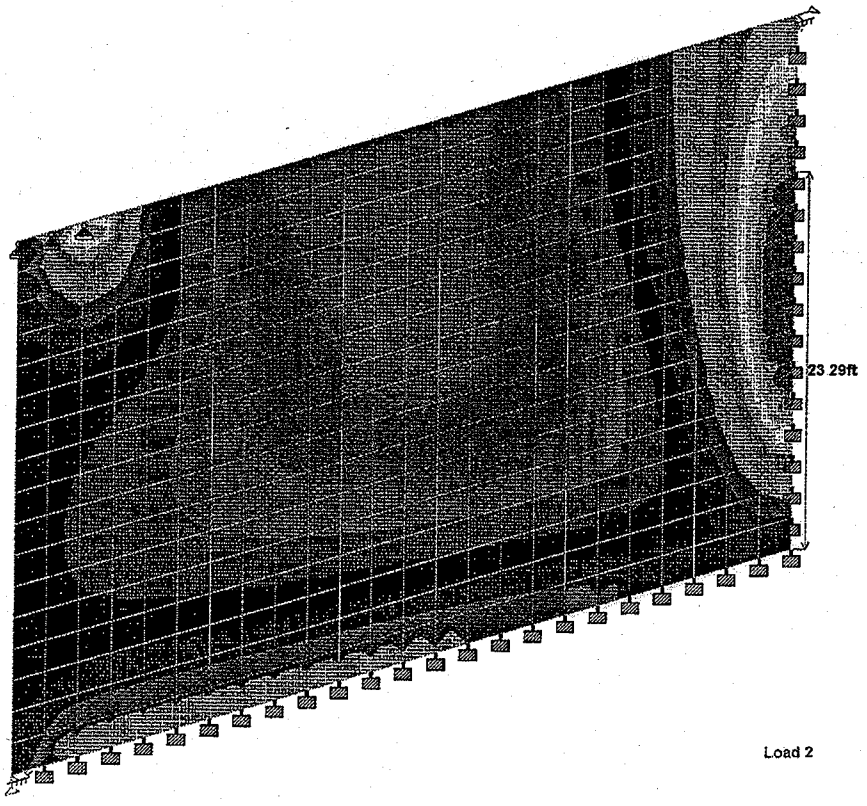
- <= -61 E3
- -48 7 E3
- -36 5 E3
- -24 3 E3
- -12 1 E3
- 127
- 12 3 E3
- 24 6 E3
- 36 8 E3
- 49 E3
- 61 2 E3
- 73 4 E3
- 85 6 E3
- 97 9 E3
- 110 E3
- 122 E3
- >= 135 E3



Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.00	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Wall2_rev1.std	Date/Time 23-May-2006 15:31	

MY
lb-in/in

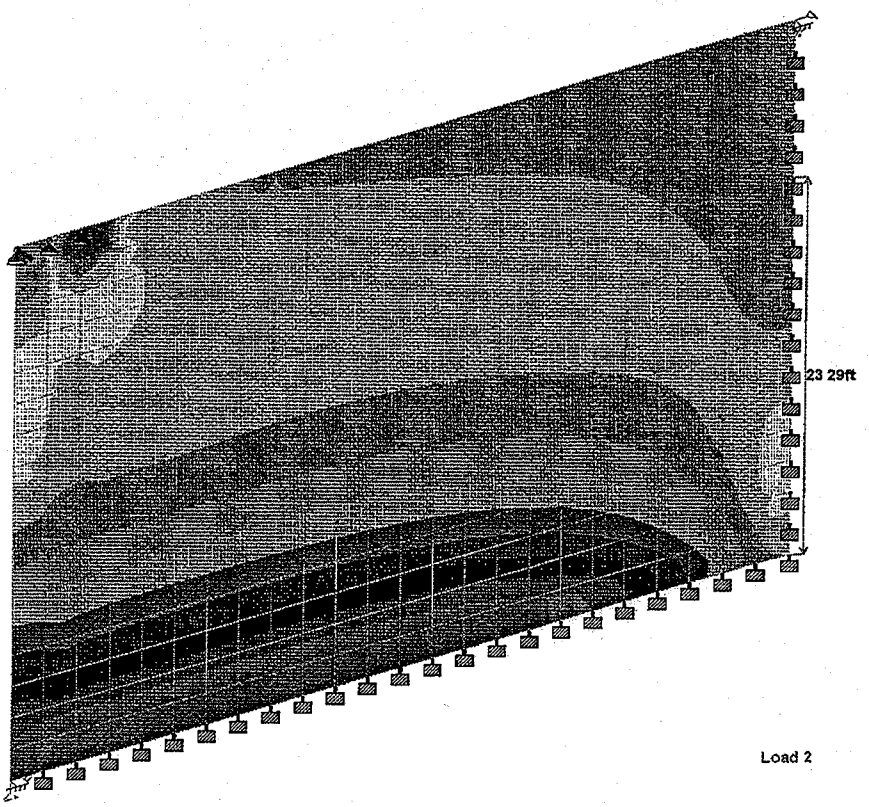
- <= -27 3 E3
- -21 3 E3
- -15 2 E3
- -9133
- -3061
- 3011
- 9083
- 15 2 E3
- 21 2 E3
- 27 3 E3
- 33 4 E3
- 39 4 E3
- 45 5 E3
- 51 6 E3
- 57 7 E3
- 63.7 E3
- >= 69.8 E3



Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.00	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Wall2_rev1.std	Date/Time 23-May-2006 15:31	

SQX
psi

- <= -247
- -221
- -196
- -170
- -144
- -119
- -93 1
- -67 4
- -41 8
- -16 1
- 9 55
- 35 2
- 60 9
- 86 5
- 112
- 138
- >= 163



STANLEY CONSULTANTS

Computed by: D. Dreyer — Date 17-May-06
Checked by: [illegible] — Date 5/23/06
Approved by: — Date

Project 17300.09.02 — Page No 29
Subject Structural
C43PS-1
Suction Basin Base Slab
Sheet No ____ of ____

PROGRAM ST071 - LOADED PLATE STAADPro FINITE ELEMENT MODELER

For User Notes see notebook tab labeled 'User's Notes'

Copy Staad Input To Clipboard | Print Staad Input | Print Spreadsheet

Notes placed in STAAD3 file:

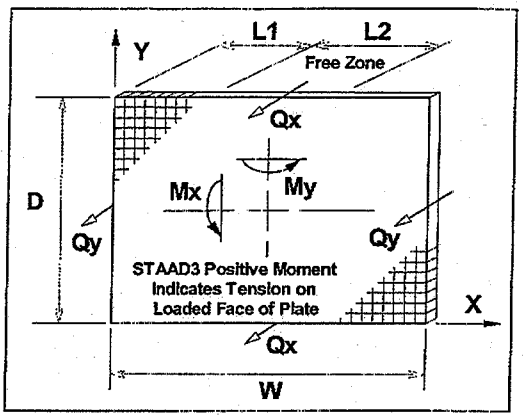


Figure 1. - STAAD3 Finite Element Plate Model and Stress Notations

INPUT: Model Data (see Figure 1.)	
Dimension W(feet):	33.00
Dimension D (feet):	48.00
Top edge fixed (FX), pinnned (PN) or free (FR):	FX
Top edge free zone (no supports in L2 zone); Valid if top edge pinned or fixed; Enter "N" if not applicable:	
L1 (feet): N	L2 (feet): N
Bottom edge fixed (FX), pinnned (PN) or free (FR):	FR
Left edge fixed (FX), pinnned (PN) or free (FR):	FX
Right edge fixed (FX), pinnned (PN) or free (FR):	FX
Maximum element dimension (feet):	2.000
STAAD3 filename (8 characters.std):	Slab1.std
Input Check:	O.K.
	O.K.

OUTPUT: Model Statistics	
Number of Joints:	450
Number of Elements:	408
Rows of Elements:	24
Columns of Elements:	17
Element X dimension (feet):	1.941
Element Y dimension (feet):	2.000

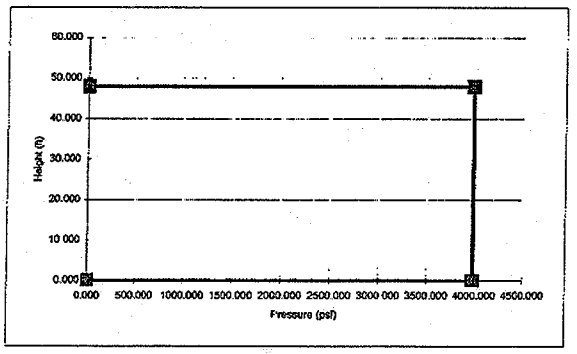


Figure 2. - Pressures Acting on Plate Specified In Table Below

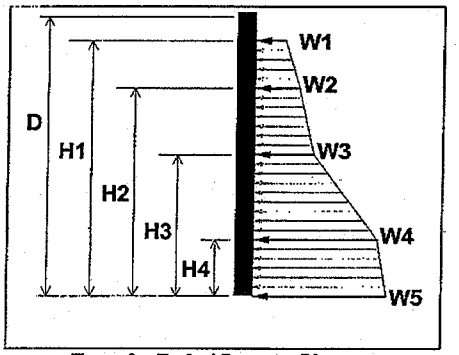


Figure 3. - Typical Pressure Diagragm Applied Normal to Plate

D = 48 00 Feet

INPUT: Load Data (see Figure 2. and 3.)			
Notes: At a minimum W1 W5 H1 and H5 must have values Enter "N" for intermediate W and H values not used No required order of input for intermediate values	Position	W# (psf)	H# (ft)
	1	3980.00	48.00
	2	N	N
	3	N	N
	4	N	N
	5	3980.00	0.00
Load Factor:	1.00		
Input Check:	O.K.		

30

/

LJS

5/23/06

```
STAAD SPACE - PROJECT 17300 09 02 - C43PS-1 - Suction B
* FILE CREATED BY:D  Dreyer
* FILENAME = Slab1 std
*
*
*
*
* ######################
* FOLLOWING IS INPUT DATA ECHOED FROM ST071"
* WIDTH = 33 00 ft
* DEPTH = 48 00 ft
* TOP EDGE = FX
* FREE EDGE BETWEEN  L1 = 0 FT  AND L2 = 0 FT
* BOT EDGE = FR
* LEFT EDGE = FX
* RIGHT EDGE = FX
* MAX ELEMENT DIM = 24 000 INCHES
*
* LOADING
* W1 = 3 980.00 PSF    H1 = 48 00 FT
* W2 = 0 PSF    H2 = 0 FT
* W3 = 0 PSF    H3 = 0 FT
* W4 = 0 PSF    H4 = 0 FT
* W5 = 3,980.00 PSF    H5 = 0 00 FT
* LOAD FACTOR = 1 00
* ######################
UNITS KIP FEET
INPUT WIDTH 72
INPUT NODESIGN
*************************************************************
JOINT COORDINATES
1 0.000 0.000 0 000 18 33 000 0 000 0.000
REPEAT 24 0.000 2.000 0.000
*************************************************************
ELEMENT INCIDENCES
1 1 19 20 2 TO 17
REPEAT 23 17 18
*************************************************************
SUPPORTS
*CORNERS
1 FIXED BUT MX
18 FIXED BUT MX
433 FIXED
450 FIXED
*BOTTOM EDGE
*
* TOP EDGE
434 TO 449 FIXED
*LEFT EDGE
19 TO 415 BY 18 FIXED
*RIGHT EDGE
36 TO 432 BY 18 FIXED
UNIT INCH
*************************************************************
ELEMENT PROPERTY
1 TO 408 THICKNESS 12.00
*************************************************************
UNIT FEET
*************************************************************
CONSTANTS
E CONCRETE ALL
*************************************************************
LOAD 1
ELEMENT LOAD
1 TO 17 PR GZ 3.9800
18 TO 34 PR GZ 3 9800
35 TO 51 PR GZ 3 9800
52 TO 68 PR GZ 3 9800
69 TO 85 PR GZ 3.9800
86 TO 102 PR GZ 3.9800
103 TO 119 PR GZ 3 9800
120 TO 136 PR GZ 3 9800
137 TO 153 PR GZ 3 9800
154 TO 170 PR GZ 3 9800
171 TO 187 PR GZ 3 9800
188 TO 204 PR GZ 3 9800
```

31

```
205 TO 221 PR GZ 3 9800
222 TO 238 PR GZ 3 9800
239 TO 255 PR GZ 3 9800
256 TO 272 PR GZ 3 9800
273 TO 289 PR GZ 3 9800
290 TO 306 PR GZ 3 9800
307 TO 323 PR GZ 3 9800
324 TO 340 PR GZ 3 9800
341 TO 357 PR GZ 3 9800
358 TO 374 PR GZ 3 9800
375 TO 391 PR GZ 3 9800
392 TO 408 PR GZ 3 9800
*
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*
*
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*
*
*
*
****************************************************************
PERFORM ANALYSIS
*
PRINT SUPPORT REACTIONS ALL
FINISH
```
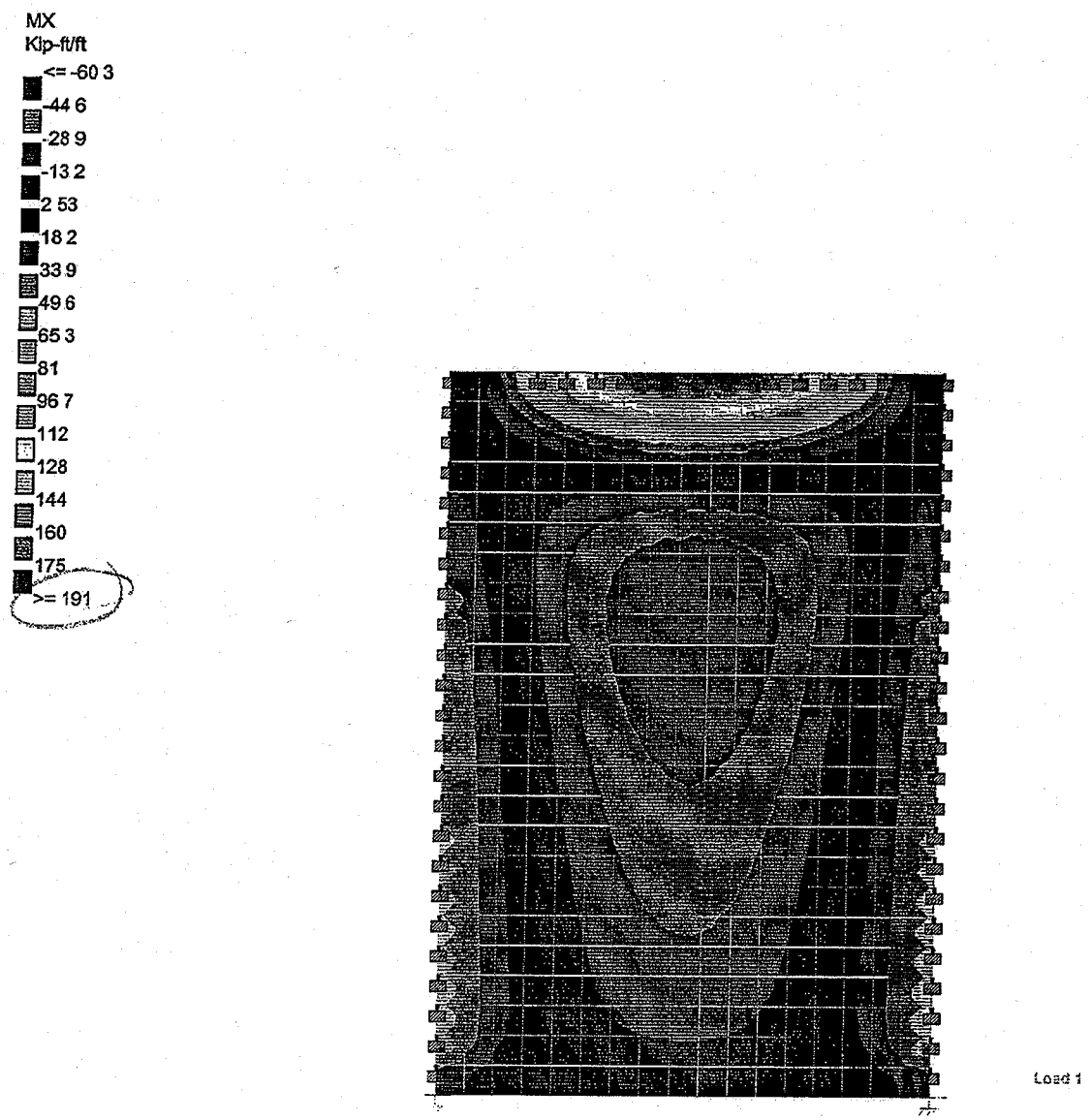

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd Sandhaus
Client SFWMD	File Slab1.std	Date/Time 17-May-2006 19:24	

5/23/06



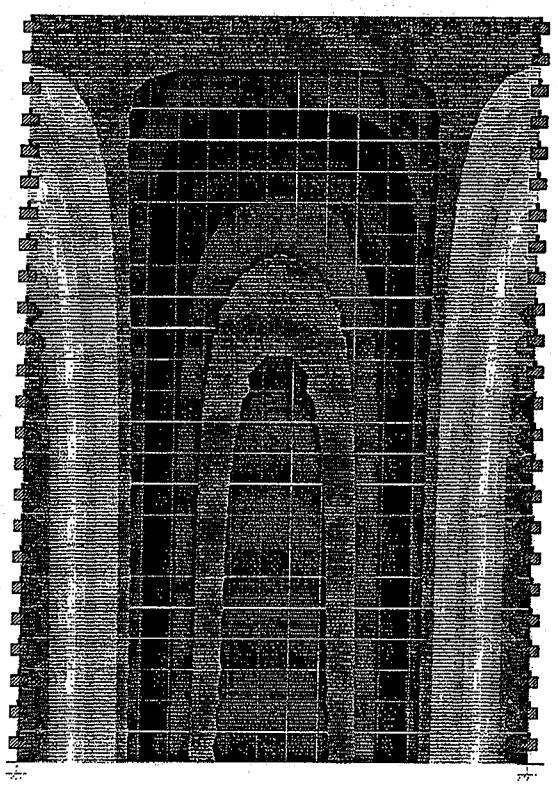
33

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd Sandhman
Client SFWMD	File Slab1.std	Date/Time 17-May-2006 19:24	

5/23/06

MY
Kip-ft/ft

- <= -186
- -155
- -124
- -92 8
- -61 7
- -30.7
- 0 370
- 31 4
- 62 5
- 93 5
- 125
- 156
- 187
- 218
- 249
- 280
- >= 311



Load 1

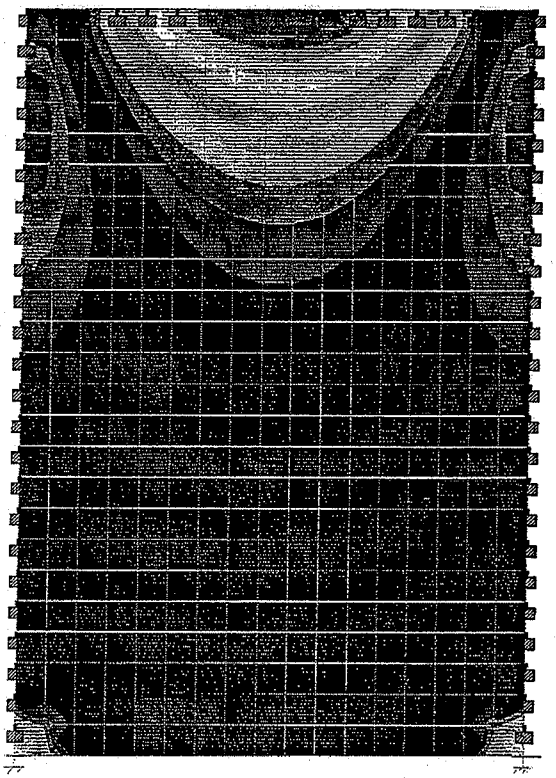
34

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Slab1.std	Date/Time 17-May-2006 19:24	

5/23/06

SQX
ksi

- <= -0 128
- -0 096
- -0 065
- -0.033
- -0 002
- 0 030
- 0 061
- 0 092
- 0 124
- 0 155
- 0 187
- 0 218
- 0 250
- 0 281
- 0 313
- 0 344
- >= 0 376



Load 1

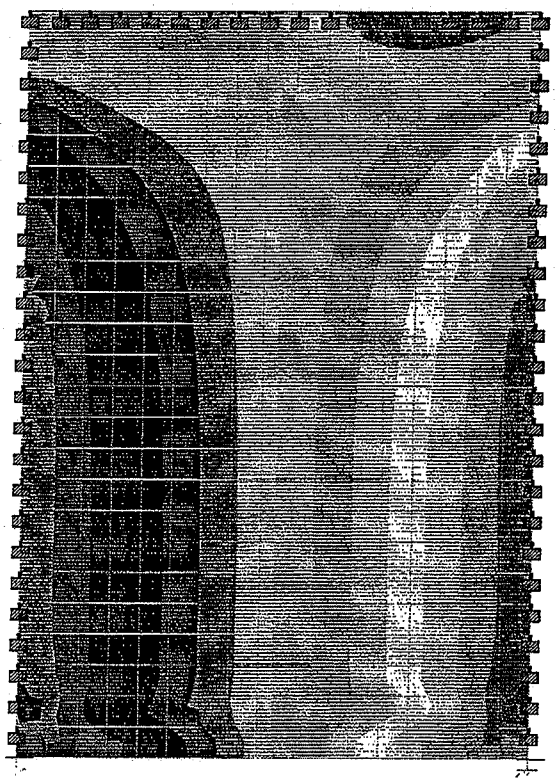
35

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS-1	Ref		
	By DRD	Date 06-May-06	Chd
Client SFWMD	File Slab1.std	Date/Time 17-May-2006 19:24	

5/23/06

SQY
ksi

- <= -0.528
- -0.462
- -0.396
- -0.330
- -0.264
- -0.198
- -0.132
- -0.066
- 0
- 0.066
- 0.132
- 0.198
- 0.264
- 0.330
- 0.396
- 0.462
- >= 0.528



Load 1

LWS
5/23/06

36

```
0000000    00000    00000    00000    00000    00
00    00   00  00   00  00   00  00   00  00   00
00    00   00  00   00  00   00  00   00  00   00
00    00   00       00  00   00       00  00   00
00    00   00       0000000  00       00  00   00
0000000    00  00   00  00   00  00   00  00   00
00         00  00   00  00   00  00   00  00   00
00          00000   00  00    00000    00000    00000  (TM)
```

==========
Computer program for the Strength Design of Reinforced Concrete Sections
==========

Licensee stated above acknowledges that Portland Cement Association (PCA) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the PCACOL(tm) computer program Furthermore, PCA neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the PCACOL(tm) program Although PCA has endeavored to produce PCACOL(tm) error free, the program is not and can't be certified infallible The final and only responsibility for analysis, design and engineering documents is the licensees Accordingly, PCA disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the PCACOL(tm) program

LWS
5/23/06

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General Information:
==========

File Name: Z:\T_PIER COL
Project: C43PS-1
Column: TrainingPier
Code: ACI 318-95

Engineer: DRD
Units: English

Run Option: Investigation
Run Axis: X-axis

Slenderness: Not considered
Column Type: Structural

Material Properties:
==========

f'c = 4 ksi
Ec = 3605 ksi
fc = 3 4 ksi
Ultimate strain = 0 003 in/in
Beta1 = 0 85

fy = 60 ksi
Es = 29000 ksi
Rupture strain = Infinity

Section:
==========

Rectangular: Width = 36 in
Depth = 72 in

Gross section area, Ag = 2592 in^2
Ix = 1 11974e+006 in^4
Xo = 0 in

Iy = 279936 in^4
Yo = 0 in

Reinforcement:
==========

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0 11	# 4	0.50	0.20	# 5	0.63	0 31
# 6	0 75	0.44	# 7	0 88	0 60	# 8	1 00	0.79
# 9	1 13	1 00	# 10	1 27	1 27	# 11	1 41	1.56
# 14	1 69	2.25	# 18	2 26	4.00			

Confinement: Tied; #4 ties with #10 bars, #4 with larger bars
phi(a) = 0 8, phi(b) = 0 9, phi(c) = 0 7

Pattern: Irregular

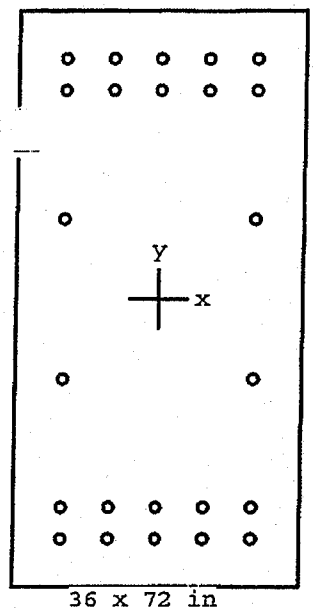
Total steel area, As = 37.44 in^2 at 1 44%

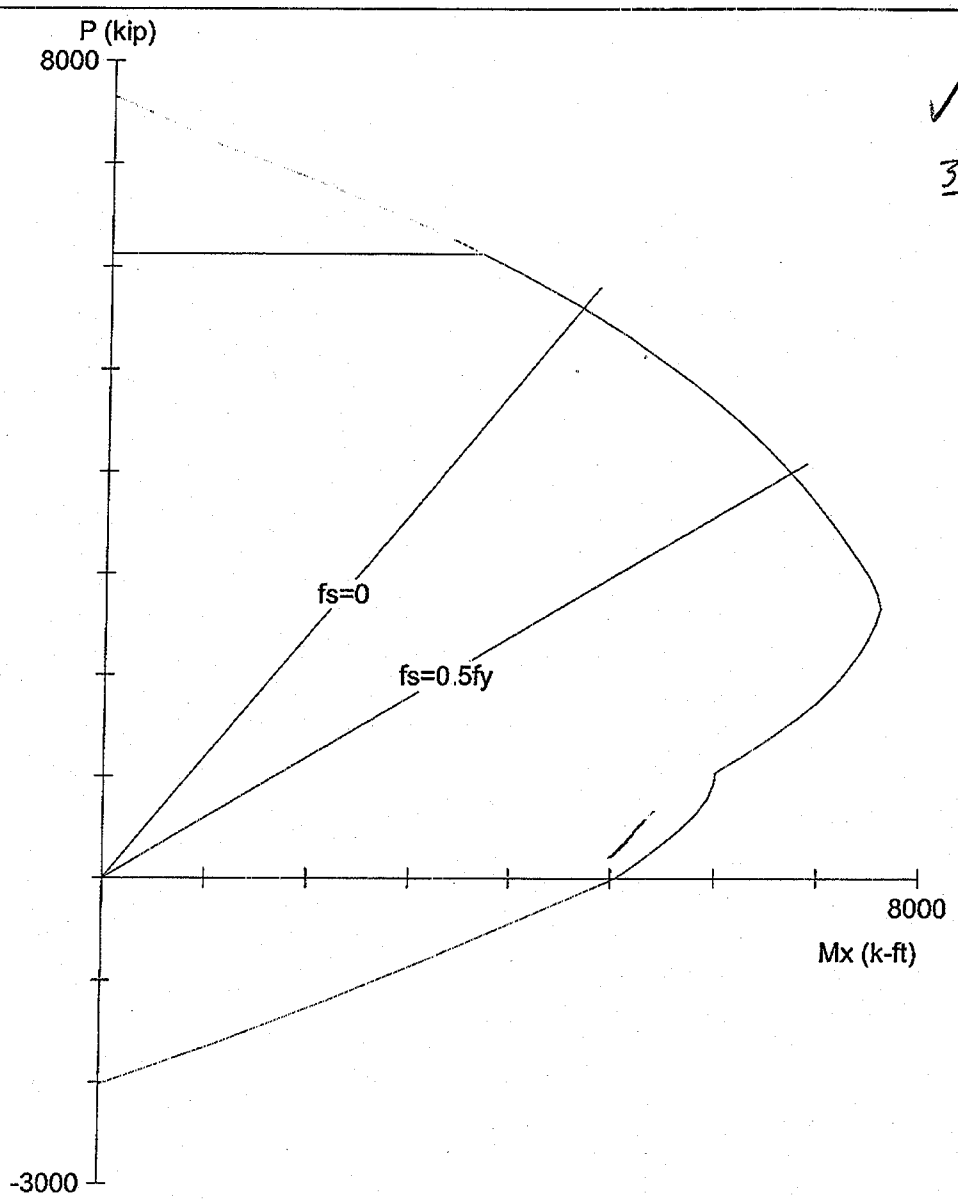
Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)
1 56	12 0	30 0	1 56	6 0	30 0	1 56	0 0	30 0
1.56	-6 0	30 0	1 56	-12 0	30 0	1 56	-12.0	26 0
1 56	-6 0	26 0	1 56	0 0	26 0	1.56	6 0	26 0
1 56	12 0	26 0	1.56	12 0	-30 0	1.56	6 0	-30 0
1.56	0 0	-30 0	1 56	-6 0	-30 0	1 56	-12 0	-30 0
1 56	-12 0	-26 0	1 56	-6 0	-26 0	1 56	0 0	-26 0
1 56	6 0	-26 0	1.56	12 0	-26 0	1 56	12 0	10 0
1 56	12 0	-10 0	1 56	-12 0	-10 0	1 56	-12 0	10 0

Axial Load and Corresponding Moment Capacities: (see user's manual for notation)
==========

Load No.	fPn kip	fMnx k-ft	N A depth in
1	121 0	5219 4	10 93

*** Program completed as requested! ***





38

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

ıderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 05/17/06

Time: 19:46:43

PCACOL V3 00 (PCA 1999) - Licensed to: Stanley Consultants, Muscatine, IA

File: Z:\T_PIER COL

Project: C43PS-1

Column: TrainingPier

Engineer: DRD

f'c = 4 ksi	fy = 60 ksi	Ag = 2592 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37 44 in^2	Rho = 1 44%
= 3 4 ksi	e_rup = Infinity	Xo = 0 00 in	Ix = 1 11974e+006 in^4
e_u = 0 003 in/in		Yo = 0 00 in	Iy = 279936 in^4
Beta1 = 0 85		Clear spacing = 2 59 in	Clear cover = 5 30 in
Confinement: Tied	phi(a) = 0 8, phi(b) = 0 9, phi(c) = 0 7		

C43 Pumpstation Bridge Crane Girder

Description

Design crane runway girder for 60 ton crane

References

1. *Manual of Steel Construction: Allowable Stress Design Ninth Edition, Vol I and II*
2. *"Crane Runway Systems" by John C Rowswell University of Toronto Department of Civil Engineering - August 1987*
3. *EM 1110-2-3104* Structural and Architectural Design of Pumping Stations
4. *SOUTH FLORIDA WATER MANAGEMENT DISTRICT MAJOR PUMPING STATION ENGINEERING GUIDELINES*
5. *Product Information for: American Crane & Equipment Corporation - 60ton capacity Class D Top Running Double Girder*

Design Information

$k := 1000\ lbf$ $\quad kpf := \frac{k}{ft}$

$Capacity := 120000lbf$

$Wt_{crane} := 72900lbf\ 1\ 1$

Increase crane weight by 10% to allow for different manufacturers

$Wt_{trolley} := 10000lbf\ 1\ 1$

$Crane_{span} := 80ft$ Center to Center distance of rails

$Clearance_{load} := 3\ 83ft$ Closest distance from lifted load to rail

$\Delta V_{max}(span) := \frac{span}{1000}$ Maximum allowable vertical deflection - Ref 2

$\Delta H_{max}(span) := \frac{span}{600}$ Maximum allowable horizontal deflection - Ref 2

$S_{wheel} := 13ft$ Two wheels on each rail

Cranes should be simple span to minimize fatigue from stress reversal

$Girder_{span} := 18ft$ distance between columns

$Girder_{span} = 18\ ft$

$$R_{wheel} := \frac{\frac{Capacity\left(Crane_{span} - Clearance_{load}\right)}{Crane_{span}}}{2} + \frac{Wt_{crane}}{4} + \frac{\frac{Wt_{trolley}\left(Crane_{span} - Clearance_{load}\right)}{Crane_{span}}}{2}$$

$R_{wheel} = 82\ 412\ k$ maximum vertical wheel load $\quad R_{wheel} := 85\ 6k \cdot 1\ 10$ per Ref 5

Vertical Loads

$I_{vert} := 1\ 18$ Impact factor of 18% of maximum vertical wheel loads Ref 4

$F_{vert} := R_{wheel}\, I_{vert}$ $F_{vert} = 111\ 109\,k$ per wheel

$Wt_{rail} := 100\,\frac{lbf}{yd}$ $Wt_{rail} = 0\ 033\,kpf$ Assume 100 lb/yd Rail - per Ref 5

Longitudinal Forces: 10% of maximum vertical wheel loads (applied at top the rail)

$F_{longit} := 0\ 10\, R_{wheel}$ $F_{longit} = 9\ 416\,k$ per wheel

$Bm_{depth} := 24in$

$Height_{rail} := 6in$

$\frac{Bm_{depth}}{2} + Height_{rail} = 1\ 5\,ft$ Height above rail for load application

Lateral Forces: The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 10% of the sum of the rated capacity of the crane and the weight of the hoist and trolley The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with 75% on either side - per Ref 3

$F_{lateral} := \left[0\ 10\left(Wt_{trolley} + Capacity\right)\right]\frac{0.75}{2}$ $F_{lateral} = 4\ 913\,k$ per wheel

$\frac{Bm_{depth}}{2} + Height_{rail} = 1\ 5\,ft$ Height above rail for load application

Load Placement

See Ref 1, page 2-310: case 41

$\left(2 - \sqrt{2}\right) Girder_{span} = 10\ 544\,ft$ < $S_{wheel} = 13\,ft$

Max moment will occur with one wheel at center of span, and the other wheel on the adjacent span

Max reaction will occur with one load at the support, and one 13ft away from support

Use STAAD to design Crane beam Torsion must be included due to the eccentric loads

41

```
****************************************************
*                                                  *
*          STAAD Pro                               *
*          Version  2005    Bld 1001 US            *
*          Proprietary Program of                  *
*          Research Engineers,  Intl               *
*          Date=    MAY 20, 2006                   *
*          Time=     7:43:25                       *
*                                                  *
*     USER ID: STANLEY CONSULTANTS INC             *
****************************************************
```

C43 P51

```
     1  SIAAD SPACE [redacted] CRANE GIRDER
INPUT FILE: Crane_Girder SID
     2  SIART JOB INFORMATION
     3  JOB NAME C43 PUMPSIAIION
     4  JOB CLIENI SFWMD
     5  JOB NO 17300 09 02
     6  JOB PARI CRANE RUNWAY GIRDER
     7  ENGINEER NAME K  HALL
     8  ENGINEER DATE 5/20/06
     9  END JOB INFORMAIION
    10  INPUI WIDIH 79
    11  UNIT FEET KIP
    12. JOINI COORDINAIES
    13  1 0 0 0; 2 18 0 0
    14  MEMBER INCIDENCES
    15  1 1 2
    16  DEFINE MAIERIAL SIARI
    17  ISOTROPIC STEEL
    18  E 4 176E+006
    19. POISSON 0 3
    20  DENSITY 0 489024
    21  ALPHA 6 5E-006
    22  DAMP 0 03
    23  END DEFINE MAIERIAL
    24  CONSIANIS
    25  MAIERIAL SIEEL MEMB 1
    26  MEMBER PROPERIY AMERICAN
    27. 1 IABIE ST W24X162
    28  SUPPORIS
    29  1 FIXED BUT MY MZ
    30  2 FIXED BUT FX MY MZ
    31  LOAD 1 LOADIYPE NONE  IIIIE CRANE WHEEL 1
    32  MEMBER LOAD
    33  *SEE MAIHCAD WORKSHEET FOR LOAD CALCULAIIONS
    34  *VERIICAL WHEEL LOADS
    35  1 CON GY -111 1 9 0
    36. *LONGIIUDINAI IOADS APPLIED AI IOP OF RAIL
    37  1 CON GX 9 4 9 1 5
    38  *IATERAL LOADS APPLIED AI IOP OF RAIL
    39  1 CON GZ 4 9 9 1 5
    40  LOAD 2 LOADIYPE NONE  IITIE CRANE WHEEL 2
```

✓ DRD
5/25/06

42

```
41. MEMBER LOAD
42. *SEE MAIHCAD WORKSHEET FOR LOAD CALCULAIIONS
43. *VERTICAL WHEEL LOADS
44. 1 CON GY -111 1 0 01 0
45. 1 CON GY -111 1 13 0
46. *LONGITUDINAL LOADS APPLIED AI IOP OF RAIL
47. 1 CON GX 9 4 0 01 1.5
48. 1 CON GX 9 4 13 1 5
49. *LAIERAI LOADS APPLIED AI IOP OF RAIL
50. 1 CON GZ 4 9 0 01 1 5
51. 1 CON GZ 4 9 13 1.5
52. LOAD 3 DEAD IOAD
53. SEIFWEIGHT Y -1
54. MEMBER IOAD
55, *RAIL WEIGHI 175 LB/YD
56. 1 IIN Y -0 035 -0 035
57. LOAD COMB 4 DEAD+CRANE1
58. 1 1 0 3 1 0
59. LOAD COMB 5 DEAD+CRANE2
60. 2 1 0 3 1 0
61. PERFORM ANALYSIS
```

P R O B L E M S T A T I S T I C S

```
NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS =     2/     1/     2
ORIGINAL/FINAL BAND-WIDTH=     1/     1/      5 DOF
TOTAL PRIMARY LOAD CASES =     3, TOTAL DEGREES OF FREEDOM =      5
SIZE OF STIFFNESS MATRIX =        1 DOUBLE  KILO-WORDS
REQRD/AVAIL. DISK SPACE  =     12 0/ 309155 5 MB
```

```
62. IOAD LIST 1 IO 5
63. PRINI MAXFORCE ENVELOPE
```

43

MEMBER FORCE ENVELOPE

ALL UNITS ARE KIP FEET

MAX AND MIN FORCE VALUES AMONGST ALL SECTION LOCATIONS

MEMB	FY/ FZ	DIST	LD LD	MZ/ MY	DIST DIST	LD LD	FX	DIST	LD
1 MAX	143.67	0.00	5	0.00	0.00	4			
	3.54	13.50	2	0.00	0.00	2	0.00	0.00	3
MIN	-82.07	18.00	5	-507.93	9.00	4			
	-6.26	0.00	5	-22.05	9.00	4	18.80 T	0.00	5

********** END OF FORCE ENVELOPE FROM INTERNAL STORAGE **********

64. PRINT SUPPORT REACTION

44

SUPPORT REACTIONS -UNIT KIP FEET STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	-9.40	55.55	-2.45	-3.68	0.00	0.00
	2	-18.80	141.90	-6.26	-9.39	0.00	0.00
	3	0.00	1.77	0.00	0.00	0.00	0.00
	4	-9.40	57.32	-2.45	-3.68	0.00	0.00
	5	-18.80	143.67	-6.26	-9.39	0.00	0.00
2	1	0.00	55.55	-2.45	-3.68	0.00	0.00
	2	0.00	80.30	-3.54	-5.31	0.00	0.00
	3	0.00	1.77	0.00	0.00	0.00	0.00
	4	0.00	57.32	-2.45	-3.68	0.00	0.00
	5	0.00	82.07	-3.54	-5.31	0.00	0.00

********** END OF LATEST ANALYSIS RESULT **********

```
65. LOAD LIST 4 5
66  PARAMETER
67  CODE AISC
68  TORSION 1 ALL
69  FYLD 7200 ALL
70. RATIO 1 ALL
71  TRACK 2 ALL
72  DFF 1000 ALL
73  CHECK CODE ALL
```

STAAD PRO CODE CHECKING - (AISC 9TH EDITION)
****

```
|-----------------------------------------------------------------------|
|                                                  Y        PROPERIIES   |
|*************                                     |        IN INCH UNIT |
|            *  |=============================|   ===|===     ------------ |
|MEMBER   1  *  |  AISC SECTIONS              |      |        AX =  47 70  |
|            *  | SI  W24X162                 |      |   --Z  AY =  15 66  |
|DESIGN CODE *  |                             |      |        AZ =  21 18  |
| AISC-1989  *  ==============================   ===|===     SY =  68 39  |
|            *                                                SZ = 413 60  |
|            *  |<---LENGTH (FT)=   18 00 --->|                RY =   3 05  |
|*************                                                RZ =  10 41  |
|                                                                         |
|                507 9 (KIP-FEET)                                         |
|PARAMETER        |                  L4                      STRESSES     |
|IN KIP  INCH     |              L4      L4                  IN KIP  INCH |
|---------------  +                          L5              -------------|
| KI/R-Y=  70 88  |                                          FA  =  30 00 |
| KL/R-Z=  20 75  +          L4                              fa  =   0 20 |
| UNL   = 216.00  |                              L5          FCZ =  30 00 |
| CB    =   1 00  +      L4                                  FIZ =  30 00 |
| CMY   =   0 85  |  L4                              I5      FCY =  37.50 |
| CMZ   =   0 85  +                                          FIY =  37 50 |
| FYID  =  50 00  |L0                                   L0   fbz =   0 00 |
| NSF   =   1 00  +---+---+---+---+---+---+---+---+---+---|   fby =   0 00 |
| DFF   =1000 00 -28 2                                       Fey =  29 73 |
| dff   =1215.35              ABSOIUIE MZ ENVEIOPE           Fez = 346 91 |
|                                (WIIH LOAD NO )             FV  =  20 00 |
|                                                            fv  =   4 54 |
|                                                                         |
|                    MAX FORCE/ MOMENI SUMMARY  (KIP-FEET)                |
|                    -------------------------                            |
|                                                                         |
|               AXIAL      SHEAR-Y    SHEAR-Z    MOMENI-Y    MOMENI-Z     |
|                                                                         |
|      VALUE    -18 8       143 7       6 3        22 1       507 9       |
|   LOCATION      0 0         0 0       0 0         9 0         9 0       |
|    LOADING        5           5         5           4           4       |
|                                                                         |
|*************************************************************************|
|*                                                                       *|
|*                       DESIGN SUMMARY  (KIP-FEET)                      *|
|*                       --------------                                  *|
|*                                                                       *|
|*       RESULI/     CRIIICAL COND/     RATIO/         LOADING/          *|
|         FX              MY              MZ           LOCATION           |
|        ==================================================              |
|          PASS        DEFLECIION        0 823             4              |
|          9 40 T          0 00          0 00            9 00             |
|*                                                                       *|
|*************************************************************************|
|                                                                         |
|-----------------------------------------------------------------------|
```

ALL UNITS ARE - KIP FEET (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION

```
| MEMB      1 : Stresses caused by Iorsion (MX = -3 675E+00)          |
| UNITS - KIP   FEEI                                                  |
| C G  of Flange :                                                    |
|      Shear Stress due to warping                : 1 189E+01         |
|      Shear Stress due to pure torsion           : 2 001E+02         |
|      Normal Stress (along axis) due to warping : 0.000E+00          |
| Junction of Flange _Web :                                           |
|      Shear Stress due to warping                : 0 000E+00         |
|      Shear Stress due to pure torsion           : 0 000E+00         |
|      Normal Stress (along axis) due to warping : 0 000E+00          |
| C G  of Web :                                                       |
|      Shear Stress due to warping                : 0 000E+00         |
|      Shear Stress due to pure torsion           : 1 156E+02         |
|      Normal Stress (along axis) due to warping : 0 000E+00          |
```

74 FINISH

********** END OF IHE SIAAD Pro RUN **********

**** DATE= MAY 20,2006 IIME= 7:43:26 ****

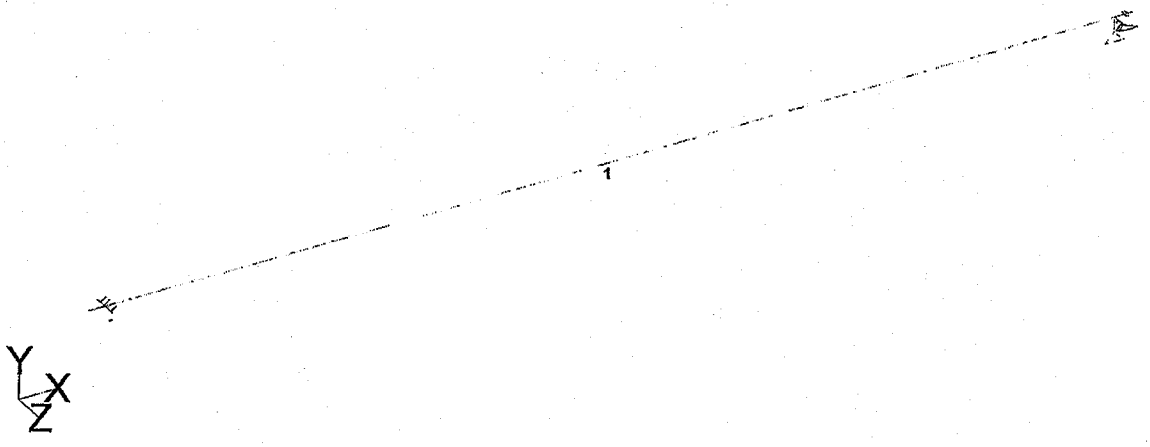
```
************************************************************
*        For questions on SIAAD Pro, please contact        *
*   Research Engineers Offices at the following locations  *
*                                                          *
*              Telephone                Email              *
*  USA:      +1 (714)974-2500       support@reiusa com      *
*  CANADA    +1 (905)632-4771       detech@odandetech com   *
*  CANADA    +1 (604)629 6087       staad@dowco com         *
*  UK        +44(1454)207-000       support@reel co uk      *
*  FRANCE    +33(0)1 64551084       support@reel co uk      *
*  GERMANY   +49/931/40468-71       info@reig de            *
*  NORWAY    +47 67 57 21 30        staad@edr no            *
*  SINGAPORE +65 6225-6015/16       support@reiasia net     *
*  INDIA     +91(033)2357-3575 support@calcutta reiusa com *
*  JAPAN     +81(03)5952-6500       eng-eye@crc co jp       *
*  CHINA     +86(411)363-1983       support@reiasia net     *
*                                                          *
*  North America                   support@reiusa com       *
*  Europe                          support@reel co uk       *
*  Asia                            support@reiasia net      *
************************************************************
```

48

Information about the key files in the current distribution

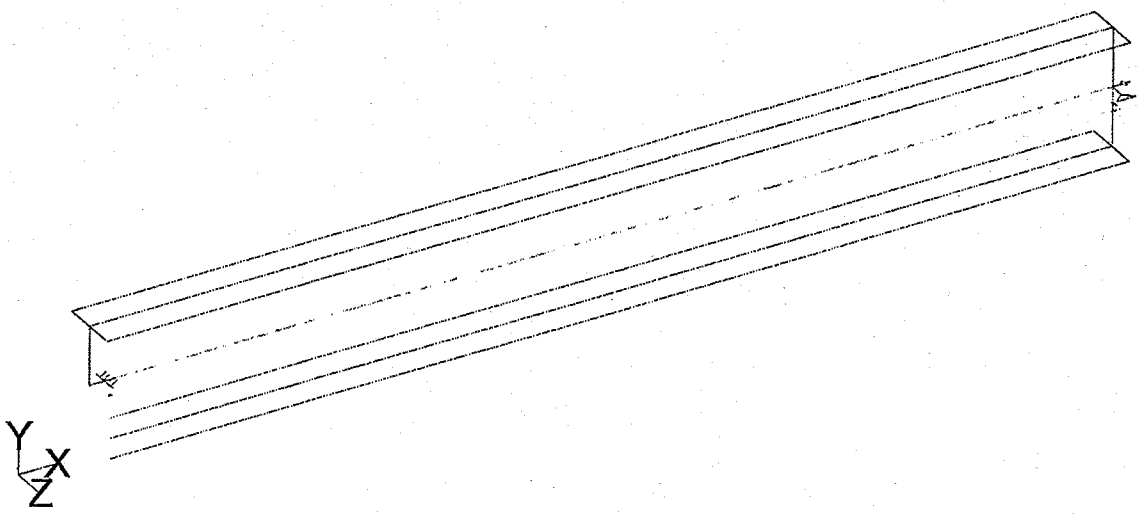
Modfication Date	CRC	Size (Bytes)	File Name
07/12/2005	0xaf01	11636736	SProStaad exe
12/17/2004	0xca81	04558848	SProStaadStl exe
09/19/2003	0x2fc0	00081970	CMesh dll
07/11/2005	0xc7c0	02396160	dbsectioninterface dll
01/24/2001	0x9b40	00073728	IoadGen dll
09/25/2003	0x6340	00704512	MeshEngine dll
09/22/2003	0xce00	00069632	QuadPlateEngine dll
04/16/2004	0x91c1	00139264	SurfMesh dll
06/23/2005	0x4c01	00493568	aiscsections mdb
01/05/2005	0x79c1	00319488	aiscsections_all_editions mdb
01/05/2005	0x4b81	01810432	aiscsteeljoists mdb
01/05/2005	0xcac1	03651584	aitctimbersections mdb
01/27/2005	0xeb01	00552960	aluminumsections mdb
01/05/2005	0xcd01	00163840	australiansections mdb
01/05/2005	0x6a41	00229376	britishsections mdb
07/08/2005	0x9d41	00434176	bscoldformedsections mdb
06/28/2005	0x8201	00327680	butlercoldformedsections mdb
01/05/2005	0xabc0	00262144	canadiansections mdb
05/31/2005	0x9e81	00450560	canadiantimbersections mdb
05/05/2005	0x7f80	00409600	chinesesections mdb
01/05/2005	0xd6c0	00600064	dutchsections mdb
01/05/2005	0x1a00	00354304	europeansections mdb
01/05/2005	0xd301	00202752	frenchsections mdb
01/05/2005	0x11c1	00233472	germansections mdb
01/05/2005	0x3c40	00264192	indiansections mdb
01/05/2005	0xd540	00180224	iscoldformedsections mdb
2/03/2005	0x9c01	00202752	japanesesections mdb
01/05/2005	0xb740	00174080	koreansections mdb
02/03/2005	0xda00	00096256	lysaghtcoldformedsections mdb
02/07/2005	0x9a00	00243712	mexicansteeltables mdb
02/03/2005	0x9b40	00307200	russiansections mdb
01/05/2005	0x9081	00206848	southafricansections mdb
01/06/2005	0x9341	00194560	spanishsections mdb
06/23/2005	0x2800	00204800	uscoldformedsections mdb
01/05/2005	0xbac0	00149504	usersectionstemplate mdb
01/05/2005	0x9d41	00141312	venezuelansections mdb

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part Crane Runway Girder		
Job Title C43 Pumpstation	Ref		
	By K. Hall	Date 24-Sep-03	Chd
Client SFWMD	File Crane_Girder.std	Date/Time 20-May-2006 07:40	

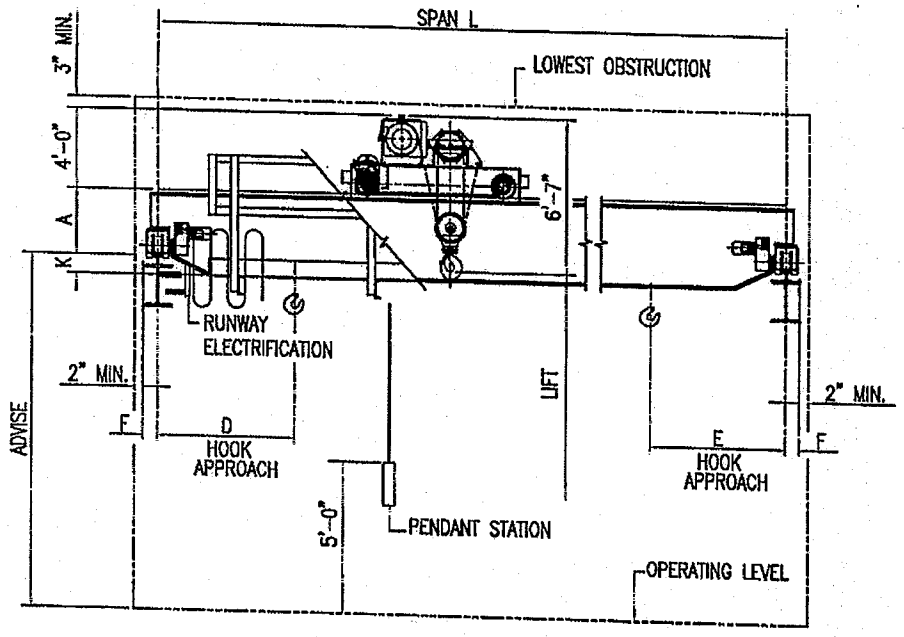


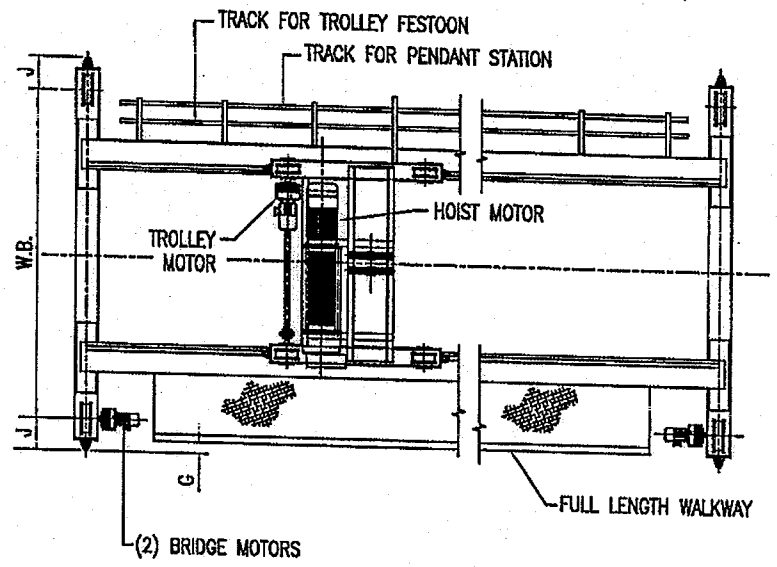
Load 5

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part Crane Runway Girder		
Job Title C43 Pumpstation	Ref		
	By K. Hall	Date 24-Sep-03	Chd
Client SFWMD	File Crane_Girder.std	Date/Time 20-May-2006 07:40	



Load 5





CAP. TONS	SPAN L	A	D	E	F	G	J MIN.	K	WHEEL DIA.	W.B.	RUNWAY RAIL SIZE	MAX. WHEEL LOAD LBS.	BRIDGE WEIGHT LBS.
60	40'-0"	3'-6"	4'-6"	3'-10"	9"	2"	1'-9"	9"	24"	10'-0"	100#	67200	24800
	50'-0"	3'-9"	4'-6"	3'-10"	9'	2"	1'-9"	1'-1"	24"	10'-0"	100#	72500	33100
	60'-0"	4'-0"	4'-6"	3'-10"	10"	0"	1'-11"	10"	27"	11'-0"	100#	76000	41900
	70'-0"	4'-3"	4'-6"	3'-10"	10"	0"	1'-11"	8"	27"	12'-0"	100#	80100	54200
	80'-0"	4'-6"	4'-6"	3'-10"	10"	0"	1'-11"	7"	27"	13'-0"	100#	85600	72900
	90'-0"	4'-9"	4'-6"	3'-10"	10"	0"	1'-11"	9"	27"	14'-0"	135#	90900	91000
	100'-0"	5'-0"	4'-6"	3'-10"	10"	-1"	1'-11"	11"	27"	15'-0"	135#	96800	111200

	TROLLEY MODEL		
	3161D-2826	3161D-4826	3161D-5826
LIFT	25'-10"	34'-0"	38'-11"
TROLLEY WEIGHT	8400 LBS.	9330 LBS.	9460 LBS.

HOISTING SPEED	10/1.7 FPM
BRIDGE SPEED	100 FPM (VARIABLE)
TROLLEY SPEED	80 FPM (VARIABLE)
POWER SUPPLY	460/3/60

NOTES:
1. HOIST CLASSIFICATION: HMI H4 / FEM 3M
2. LONGER LIFTS ARE AVAILABLE.
3. SPEEDS SHOWN ARE STANDARD, OTHER SPEEDS ARE AVAILABLE.
4. AUXILIARY HOIST AVAILABLE.
5. VARIABLE FREQUENCY DRIVE AVAILABLE FOR HOIST

ACECO
AMERICAN CRANE & EQUIPMENT CORPORATION
DOUGLASSVILLE, PA. 19518 U.S.A.
PHONE: 610-385-6061
FAX: 610-385-3191

CLASS D TOP RUNNING DOUBLE GIRDER CRANE (WITH MODULAR HOIST)

60 TON CAPACITY

PRODUCT INFORMATION

DWG. NO: A-20418-317

DATE: 10/8/04 REVISION: 0

Dreyer, Daniel

From: Jiang, Feng
Sent: Thursday, April 13, 2006 9:39 AM
To: Dreyer, Daniel; Roeth, Brad
Subject: FW: WFLT-15675 / Stanley / SFWMD
Attachments: hyd-FSI xls

Dan,
Flowserve helped me to refresh my memory. They are right. The pump need to be lifted entirely first. They will support the second low section of the pump to dissemble the first section for removal.
Please find attached weight info for Fairbanks and Flowserve. I also confirmed with Flowserve and ITT yesterday, all the weight of the pump will be loaded on the sole plate, which is 10 ft slab for our pump station design. FSI will be connected with pump utilizing a bell/spigot connection. Typically, loading is not considered at the connection.
Jeff

From: Bob Cornman [mailto:BCornman@flowserve.com]
Sent: Thursday, April 13, 2006 6:56 AM
To: Jiang, Feng
Cc: Greg Poska; Jondrejack@flowserve.com
Subject: RE: WFLT-15675 / Stanley / SFWMD

Jeff,

See answers below

Regards,

Bob Cornman
Director, Technical Services NA & LA
Flowserve Pump Division
942 Memorial Parkway
Phillipsburg, NJ USA

Tel: 908-859-7256
Fax: 908-859-7590

NOTICE: The information contained in this e-mail, and attachment(s) thereto, is confidential and may contain attorney - client privileged communications. If you are not the intended recipient, you are hereby notified that any dissemination, distribution, or copying of this communication is strictly prohibited. If you have received this communication in error, please notify the sender immediately and delete the e-mail from your computer system without retaining any copies. Thank you.

"Jiang, Feng" <JiangFeng@stanleygroup.com>

04/12/2006 04:12 PM

To: "Bob Cornman" <BCornman@flowserve.com>
cc: "Greg Poska" <gposka@flowserve.com>, <Jondrejack@flowserve.com>
Subject: RE: WFLT-15675 / Stanley / SFWMD

Stanley Consultants INC

A Stanley Group Company
Engineering, Environmental and Construction Services Worldwide

TABLE 3
SOUTH FLORIDA WATER MANAGEMETN DISTRICT
ENGINEERING AND CONSTRUCTION DEPARTMENT
C-43 STORAGE RESERVOIR
PUMP STATION 1
PUMP INFORMATION

Parameters		Fairbanks Morse		Flowserve		ITT	
		500 cfs	250 cfs	500 cfs	250 cfs	500 cfs	250 cfs
Longest Component	(feet)	18		20			
Assembly Weight	(lbs)	112000	56000	99000	54500		
Weight of Heaviest Component	(lbs)						
Discharge	(inch)	96	72	84	60	84	60

C43PS-1 Preliminary Superstructure Design

Description

Perform the preliminary design of a typical interior moment frame for the C-43 Pump Station PS-1. A typical frame will consist of Wide Flange column sections supporting a steel truss at the top and bottom chords. At each panel point, the trusses will support roof purlins supporting a 6" concrete and metal form deck

Frames will be spaced at 18'-0" on center

Columns will be designed for a fixed-base condition

References

1. *Manual of Steel Construction: Allowable Stress Design Ninth Edition, Vol. I and II*
2. *EM 1110-2-3104*: Structural and Architectural Design of Pumping Stations
3. *SOUTH FLORIDA WATER MANAGEMENT DISTRICT MAJOR PUMPING STATION ENGINEERING GUIDELINES*

Reference:K:\Technical_Programs\Structural\ST061 ASCE 7-02 Wind Loading.mcd

Units

$k := 1000\ lbf$ $kpf := \frac{k}{ft}$ $ksf := \frac{k}{ft^2}$ $ksi := \frac{k}{in^2}$ $kcf := \frac{k}{ft^3}$ $ppf := \frac{lbf}{ft}$ $psf := \frac{lbf}{ft^2}$ $psi := \frac{lbf}{in^2}$ $pcf := \frac{lbf}{ft^3}$

$\gamma_{soil} := 110\ pcf$ $\gamma_{water} := 62.4\ pcf$ $\gamma_{conc} := 150\ pcf$ $f_c := 4\ ksi$ $f_y := 60\ ksi$ $F_y := 50\ ksi$ $E := 29000\ ksi$

$rpm := \frac{2\pi\ rad}{min}$ $cps := \frac{2\pi\ rad}{sec}$ $MPa := 10^6 \frac{newton}{m^2}$

Design Loads

$S_{bay} := 18ft$ typical bay spacing

$S_{verticals} := 6.84ft$ spacing of truss verticals

Dead Loads

$DL_{roof\_slab} := 6in\ \gamma_{conc}\ S_{bay}\ S_{verticals}$

$DL_{roof\_slab} = 9.2k$

$DL_{parapet} := 2ft\ 3.5ft\ S_{bay}\ \gamma_{conc}$

$DL_{parapet} = 18.9k$

$DL_{purlins} := 26ppf\ S_{bay} + 0.5k$ add 500 lb per panel point for misc. bracing, etc

$DL_{purlins} = 1\ k$

$DL_{crane\_girder} := 1.77k\ 2$ DL reaction from STAAD analysis of crane beam. Multiply by 2 for 2 beams framing into column

$DL_{crane\_girder} = 3.5k$

Live Loads

$LL_{roof} := 50psf\ S_{verticals}\ S_{bay}$

$LL_{roof} = 6.2\ k$

$LL_{crane} := 141.9k$ LL reaction from STAAD analysis of crane beam.

$LL_{crane} = 141.9k$

Wind Loads

Program ST061 - ASCE 7-02 Wind Loading to Buildings

General Notes

1. Analysis is per ASCE 7-02, Method 2 [Section 6.5]

2. Main wind force resisting system forces will be based on optional use of "All Heights" criteria of Section 6.5.12.2.1 and Figure 6-6 (i.e., "Low-rise" analysis will not be used for buildings with h <= 60'). This is permitted by code

3. Building meets requirements of Section 6.5.1

4. Roof has slope less than 7 degrees. Roof overhangs (Figure 6-11 B) will not be considered in this program

5. K_d = 0.85 per Table 6-4, page 76; as noted in code, if wind forces are used for design not involving ASCE load combinations, then wind forces should be factored by 1/0.85 = 1.18; e.g., design of metal deck, siding and their fasteners

6. Building is assumed to be located on flat terrain with topographic factor K_{zt} = 1.0 for all heights above grade. If building is located on an escarpment, ridge, or hill, the windward pressures calculated by the program must be modified in accordance with paragraph 6.5.7.2 and Figure 6-4

7. Structure is a "rigid" structure as defined in Section 2 (fundamental frequency > 1 Hz)

8. Reduction factor for large volume is not utilized [Section 6.5.11.1.1); safe under certain conditions

9. Building height <= 500 feet

Inserts (input and output)

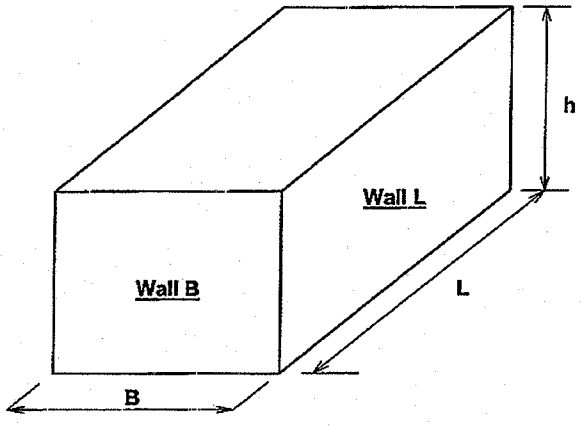


FIGURE 1. - BUILDING DIMENSIONS

SFWMD standards require a mean recurrence interval (MRI) of 200 years for wind loading When applying code values, multiply the code wind values by 1 14 to upgrade to 200 year MRI value In addition, SFWMD requires an Importance Factor value of 1 3

$MRI_{factor} := 1\ 14$ Conversion factor to upgrade to 200 year MRI

$I := 1\ 3$ Importance Factor for Wind Loads

Input:

"Basic wind speed [Figure 6-1, page 37] (mph):"	120
"Exposure [Sect 6 5 6, page 28]:"	"C"
"Importance factor (I) [Table 6-1, page 73]:"	I
"this entry not used"	"-"
"Gust effect factor (G) [Sect 6 5 8, page 30]:"	0 85
"Enclosure - enclosed (E), partial (P) or open(O) [Sect 6 2, page 23]:"	"P"
"Building height h (feet):"	46 33
"Building length L (feet):"	252
"Building width B (feet):"	84
"Height to highest wall opng for partial enclosed bldg [Sect 6 5 12 2 1] (ft):"	46 33

Input :=

Last entry is used to calculate internal positive pressure from windward wind for partial enclosure This can conservatively be taken as h (disregarding location of openings)

Output:

A := MainBldgSystemWindForces(Input)

A =

"MAIN FRAME WIND LOAD"	"VALUES"
"External leeward neg pressure on Wall B (psf):"	9.317
"External leeward neg pressure on Wall L (psf):"	18.634
"External sidewall neg pressure on Walls B & L (psf):"	26.088
"Internal positive pressure (psf):"	24.115
"Internal negative pressure (psf):"	24.115
"External roof uplift pressure - Zone L1 (psf):"	35.079
"External roof uplift pressure - Zone L2 (psf):"	32.773
"External roof uplift pressure - Zone L3 (psf):"	19.403
"External roof uplift pressure - Zone L4 (psf):"	"NA"
"External roof uplift pressure - Zone B1 (psf):"	33.542
"External roof uplift pressure - Zone B2 (psf):"	33.542
"External roof uplift pressure - Zone B3 (psf):"	18.634
"External roof uplift pressure - Zone B4 (psf):"	11.181
"External Windward Wall Forces"	""
"Height z (feet)"	"Pressure (psf)"
15	23.514
20	24.982
25	26.183
30	27.208
40	28.907
46.33	29.815

$P_{leeward} := MRI_{factor}\ A_{3,2}\ psf$ ✓ $P_{leeward} = 21.2\ psf$

$P_{windward} := MRI_{factor}\ A_{22,2}\ psf$ ✓ $P_{windward} = 34\ psf$

average of Zone L1 L2 and L3 pressures)

$$P_{uplift} := MRI_{factor}\left[A_{5,2} + \frac{\left(A_{7,2} + A_{8,2} + A_{9,2}\right)}{3}\right] psf$$ ✓ $P_{uplift} = 60.6\ psf$

Assume wall cladding will transfer wind load to 2 points on frame, one at crane beam level, and the other at the top chord of the roof truss

WIND 1 - WIND RIGHT TO LEFT

$$WL_{ww\_1_1} := \left(\frac{23.58ft + 17.75ft}{2}\right) P_{windward}\, S_{bay}$$

$$WL_{ww\_1_2} := \left(46\ 33ft - 41\ 33ft + \frac{17.75ft}{2}\right) P_{windward}\, S_{bay}$$

$$WL_{ww\_1} = \begin{pmatrix} 12.6 \\ 8.5 \end{pmatrix} k$$

$$WL_{lw\_1_1} := \left(\frac{23.58ft + 21.25ft}{2}\right) P_{leeward}\, S_{bay}$$

$$WL_{lw\_1_2} := \left(46\ 33ft - 44\ 83ft + \frac{21.25ft}{2}\right) P_{leeward}\, S_{bay}$$

$$WL_{lw\_1} = \begin{pmatrix} 8.6 \\ 4.6 \end{pmatrix} k$$

$$WL_{uplift} := P_{uplift}\, S_{bay}\, S_{verticals}$$

$$WL_{uplift} = 7.5k$$

WIND 2 - WIND LEFT TO RIGHT

$$WL_{ww\_2_1} := \left(\frac{23.58ft + 21.25ft}{2}\right) P_{windward}\, S_{bay}$$

$$WL_{ww\_2_2} := \left(46\ 33ft - 44\ 83ft + \frac{21.25ft}{2}\right) P_{windward}\, S_{bay}$$

$$WL_{ww\_2} = \begin{pmatrix} 13.7 \\ 7.4 \end{pmatrix} k$$

$$WL_{lw\_2_1} := \left(\frac{23.58ft + 17.75ft}{2}\right) P_{leeward}\, S_{bay}$$

$$WL_{lw\_2_2} := \left(46\ 33ft - 41\ 33ft + \frac{17.75ft}{2}\right) P_{leeward}\, S_{bay}$$

$$WL_{lw\_2} = \begin{pmatrix} 7.9 \\ 5.3 \end{pmatrix} k$$

$$WL_{uplift} = 7.5k$$

57

```
**************************************************
*                                                *
*           STAAD Pro                            *
*           Version  2005    Bld 1001.US         *
*           Proprietary Program of               *
*           Research Engineers,  Intl.           *
*           Date=    MAY 30, 2006                *
*           Time=    18:58: 0                    *
*                                                *
*      USER ID: STANLEY CONSULTANTS INC          *
**************************************************
```

```
     1. STAAD SPACE
INPUT FILE: C43 Superstructure.STD
     2. START JOB INFORMATION
     3. JOB NAME C43PS1
     4. JOB CLIENT SFWMD
     5. JOB NO 17300.09.02
     6. ENGINEER NAME KBH
     7. ENGINEER DATE 19-MAY-06
     8. END JOB INFORMATION
     9. INPUT WIDTH 79
    10. UNIT FEET KIP
    11. JOINT COORDINATES
    12. 1 0 0 0; 2 0 23.583 0; 3 0 25.583 0; 4 0 34.333 0; 5 0 41.333 0; 6 82 0 0
    13. 7 82 23.583 0; 8 82 25.583 0; 9 82 37.83 0; 10 82 44.83 0
    14. 11 6.83333 34.6244 0; 12 13.6667 34.9158 0; 13 20.5 35.2072 0
    15. 14 27.3333 35.4987 0; 15 34.1667 35.7901 0; 16 41 36.0815 0
    16. 17 47.8333 36.3729 0; 18 54.6667 36.6643 0; 19 61.5 36.9557 0
    17. 20 68.3333 37.2472 0; 21 75.1667 37.5386 0; 22 6.83333 41.6244 0
    18. 23 13.6667 41.9158 0; 24 20.5 42.2072 0; 25 27.3333 42.4987 0
    19. 26 34.1667 42.7901 0; 27 41.43.0815 0; 28 47.8333 43.3729 0
    20. 29 54.6667 43.6643 0; 30 61.5 43.9557 0; 31 68.3333 44.2472 0
    21. 32 75.1667 44.5386 0; 33 2 23.583 0; 34 80 23.583 0
    22. MEMBER INCIDENCES
    23. 1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 6 7; 6 7 8; 7 8 9; 8 9 10; 9 5 22; 10 4 11
    24. 11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17; 17 17 18; 18 18 19
    25. 19 19 20; 20 20 21; 21 21 9; 22 22 23; 23 23 24; 24 24 25; 25 25 26; 26 26 27
    26. 27 27 28; 28 28 29; 29 29 30; 30 30 31; 31 31 32; 32 32 10; 33 11 22; 34 12 23
    27. 35 13 24; 36 14 25; 37 15 26; 38 16 27; 39 17 28; 40 18 29; 41 19 30; 42 20 31
    28. 43 21 32; 44 5 11; 45 22 12; 46 23 13; 47 24 14; 48 25 15; 49 26 16; 50 16 28
    29. 51 17 29; 52 18 30; 53 19 31; 54 20 32; 55 21 10; 56 2 33; 57 7 34
    30. DEFINE MATERIAL START
    31. ISOTROPIC STEEL
    32. E 4.176E+006
    33. POISSON 0.3
    34. DENSITY 0.489024
    35. ALPHA 6.5E-006
    36. DAMP 0.03
    37. END DEFINE MATERIAL
    38. MEMBER PROPERTY AMERICAN
    39. 1 TO 8 56 57 TABLE ST W14X145
    40. 9 22 TO 32 TABLE I W18X97
```

J. Dreyer
5/30/06

58

```
41. 10 TO 21 TABLE I W18X97
42  33 TO 55 TABLE LD L40408
43  CONSTANTS
44  BETA 180 MEMB 10 TO 21
45  MATERIAL STEEL MEMB 1 TO 57
46  SUPPORTS
47  1 6 FIXED
48  MEMBER RELEASE
49  33 TO 55 START MY MZ
50  33 TO 55 END MY MZ
51  9 10 START MY MZ
52  21 32 END MY MZ
53  LOAD 1 LOADTYPE NONE  TITLE DEAD
54  SELFWEIGHT Y -1
55  JOINT LOAD
56  22 TO 32 FY -9.2
57  22 TO 32 FY -1.
58  5 10 FY -4.6
59  5 10 FY -18.9
60  33 34 FY -3.5
61  LOAD 2 LOADTYPE NONE  TITLE LIVE
62  JOINT LOAD
63  33 34 FY -141.9
64  22 TO 32 FY -6.2
65. 5 10 FY -3.1
66  LOAD 3 LOADTYPE NONE  TITLE WIND 1
67  JOINT LOAD
68. 2 FX 12.6
69  5 FX 8.5
70  10 FX 4.6
71  7 FX 8.6
72  5 10 22 TO 32 FY 7.5
73  LOAD 4 LOADTYPE NONE  TITLE WIND 2
74  JOINT LOAD
75  2 FX -7.9
76  5 FX -5.3
77  10 FX -7.4
78  7 FX -13.7
79  5 10 22 TO 32 FY 7.5
80. LOAD COMB 5 DEAD + LIVE
81  1 1.0 2 1.0
82  LOAD COMB 6 DEAD + 0.75 LIVE + 0.75 WIND 1
83  1 1.0 2 0.75 3 0.75
84  LOAD COMB 7 DEAD + 0.75 LIVE + 0.75 WIND 2
85  1 1.0 2 0.75 4 0.75
86  LOAD COMB 8 DEAD + WIND 1
87  1 1.0 3 1.0
88  LOAD COMB 9 DEAD + WIND 2
89  1 1.0 4 1.0
90. LOAD COMB 10 0.6 DEAD + WIND 1
91  1 0.6 3 1.0
92  LOAD COMB 11 0.6 DEAD + WIND 2
93  1 0.6 4 1.0
94  PERFORM ANALYSIS
```

59

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 34/ 57/ 2
ORIGINAL/FINAL BAND-WIDTH= 31/ 3/ 24 DOF
TOTAL PRIMARY LOAD CASES = 4, TOTAL DEGREES OF FREEDOM = 192
SIZE OF STIFFNESS MATRIX = 5 DOUBLE KILO-WORDS
REQRD/AVAIL. DISK SPACE = 12.1/ 0.0 MB

**** WARNING : AVAILABLE HARD DISK SPACE MAY NOT BE ENOUGH TO COMPLETE EXECUTION. IF YOUR AVAILABLE HARD DISK SPACE ON THE ANALYSIS DRIVE IS GREATER THAN 3GB THIS MESSAGE MAY BE ERRONEOUS

```
 95. LOAD LIST 5 TO 11
 96. PARAMETER
 97. CODE AISC
 98. FYLD 5184 MEMB 33 TO 55
 99. FYLD 7200 MEMB 1 TO 32 56 57
100. LZ 20.52 MEMB 10 TO 21
101. UNB 20 52 MEMB 10 TO 21
102. CHECK CODE MEMB 1 TO 55
```

60

STAAD.PRO CODE CHECKING - (AISC 9TH EDITION)
****

ALL UNITS ARE - KIP FEET (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 761	7
		197.90 C	0 00	-342 93	0 00
2 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-3	0.287	6
		75 97 C	0.00	-144 36	0 00
3 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-3	0 432	7
		82 71 C	0.00	230.07	8 75
4 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-3	0 428	7
		80 43 C	0.00	230 07	0 00
5 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-1	0.747	6
		198 09 C	0.00	334 09	0 00
6 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-3	0 282	7
		76 16 C	0 00	140 76	0.00
7 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-3	0 442	6
		82 39 C	0.00	-234 50	12 25
8 SI	W14X145		(AISC SECIIONS)		
		PASS	AISC- H1-3	0.439	6
		84.45 C	0 00	-234 50	0 00
9 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 219	5
		73 90 C	0 00	-1.62	6 84
10 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 239	7
		51.93 C	0.00	-0 68	6.84
11 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 168	11
		36 11 C	0.00	-0 71	0 00
12 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H2-1	0 337	5
		132 43 I	0 00	3.31	3 99
13 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H2-1	0 480	5
		191 60 I	0 00	3 91	5 70
14 T	W18X97		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.581	5
		233 65 T	0 00	4 19	3.42

ALL UNITS ARE - KIP FEET (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
15 I	W18X97		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.647	5
		258.80 T	0 00	5 16	6 84
16 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H2-1	0.645	5
		257 83 I	0 00	5.16	0 00
17 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H2-1	0 577	5
		231 74 I	0 00	4 27	2 85
18 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H2-1	0.474	5
		188 73 I	0 00	3.98	1 14
19 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H2-1	0 328	5
		128.59 T	0 00	3 39	2 28
20 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H2-1	0 145	5
		51 79 I	0 00	2.94	0 00
21 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 231	6
		50 38 C	0.00	-0 56	0 00
22 I	W18X97		(AISC SECTIONS)		
		PASS	AISC- H1-1	0 439	5
		149 73 C	0 00	-2 49	5 70
23 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 612	5
		208 90 C	0 00	-3 30	5 70
24 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 736	5
		250 95 C	0 00	-3 95	5 13
25 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 809	5
		276.11 C	0 00	-4 15	3 42
26 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 836	5
		283 86 C	0.00	-4 86	6 27
27 I	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 836	5
		283 86 C	0 00	-4 87	0 57
28 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0 806	5
		275 14 C	0 00	-4 22	3.42
29 T	W18X97		(AISC SECIIONS)		
		PASS	AISC- H1-1	0.731	5
		249.04 C	0 00	-3 98	1 71

ALL UNITS ARE - KIP FEET (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
30 I	W18X97		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.604	5
		206.03 C	0.00	-3.32	0.57
31 T	W18X97		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.428	5
		145.90 C	0.00	-2.48	0.57
32 T	W18X97		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.205	5
		69.08 C	0.00	-1.59	0.00
33 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.762	5
		94.54 C	0.00	0.00	0.00
34 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.626	5
		77.58 C	0.00	0.00	0.00
35 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.484	5
		60.04 C	0.00	0.00	0.00
36 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.344	5
		42.68 C	0.00	0.00	0.00
37 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.203	5
		25.12 C	0.00	0.00	0.00
38 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-3	0.134	5
		16.63 C	0.00	0.00	0.00
39 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.210	5
		26.10 C	0.00	0.00	0.00
40 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.352	5
		43.65 C	0.00	0.00	0.00
41 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.492	5
		61.03 C	0.00	0.00	0.00
42 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.634	5
		78.57 C	0.00	0.00	0.00
43 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H1-1	0.771	5
		95.55 C	0.00	0.00	0.00
44 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.833	5
		130.62 T	0.00	-0.21	4.79

ALL UNITS ARE - KIP FEET (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
45 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.682	5
		106.18 I	0.00	-0.21	4.79
46 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.538	5
		82.84 I	0.00	-0.21	4.79
47 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.390	5
		58.87 I	0.00	-0.21	4.79
48 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.244	5
		35.22 I	0.00	-0.21	4.79
49 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.100	7
		11.84 I	0.00	-0.21	4.79
50 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.111	6
		13.54 I	0.00	-0.22	5.00
51 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.263	5
		38.12 I	0.00	-0.22	5.00
52 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.416	5
		62.84 I	0.00	-0.22	5.00
53 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.570	5
		87.86 I	0.00	-0.22	5.00
54 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.721	5
		112.24 I	0.00	-0.22	5.00
55 LD	L40408		(AISC SECTIONS)		
		PASS	AISC- H2-1	0.878	5
		137.75 I	0.00	-0.22	5.00

103 PARAMETER
104 CODE AISC
105 STEEL TAKE OFF ALL

STEEL TAKE-OFF

PROFILE	LENGTH(FEET)	WEIGHT(KIP)
ST W14X145	90.16	13.074
I W18X97	164.15	7.944
LD L40408	194.41	4.952
PRISMATIC STEEL	0.00	0.000
	TOTAL =	25.970

********** END OF DATA FROM INTERNAL STORAGE **********

106. FINISH

********** END OF THE STAAD.Pro RUN **********

**** DATE= MAY 30,2006 TIME= 18:58: 1 ****

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*************************************************************
*         For questions on SIAAD Pro, please contact        *
*   Research Engineers Offices at the following locations   *
*                                                           *
*                Telephone                Email              *
*  USA:       +1 (714)974-2500       support@reiusa com      *
*  CANADA     +1 (905)632-4771       detech@odandetech com   *
*  CANADA     +1 (604)629 6087       staad@dowco com         *
*  UK         +44(1454)207-000       support@reel co uk      *
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*  NORWAY     +47 67 57 21 30        staad@edr no            *
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*  INDIA      +91(033)2357-3575 support@calcutta reiusa com *
*  JAPAN      +81(03)5952-6500       eng-eye@crc co jp       *
*  CHINA      +86(411)363-1983       support@reiasia net     *
*                                                           *
*  North America                    support@reiusa com       *
*  Europe                           support@reel co uk       *
*  Asia                             support@reiasia net      *
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66

ormation about the key files in the current distribution

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07/12/2005	0xaf01	11636736	SProStaad exe
12/17/2004	0xca81	04558848	SProStaadStl exe
09/19/2003	0x2fc0	00081970	CMesh dll
07/11/2005	0xc7c0	02396160	dbsectioninterface dll
01/24/2001	0x9b40	00073728	LoadGen dll
09/25/2003	0x6340	00704512	MeshEngine dll
09/22/2003	0xce00	00069632	QuadPlateEngine dll
04/16/2004	0x91c1	00139264	SurfMesh.dll
06/23/2005	0x4c01	00493568	aiscsections mdb
01/05/2005	0x79c1	00319488	aiscsections_all_editions mdb
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Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
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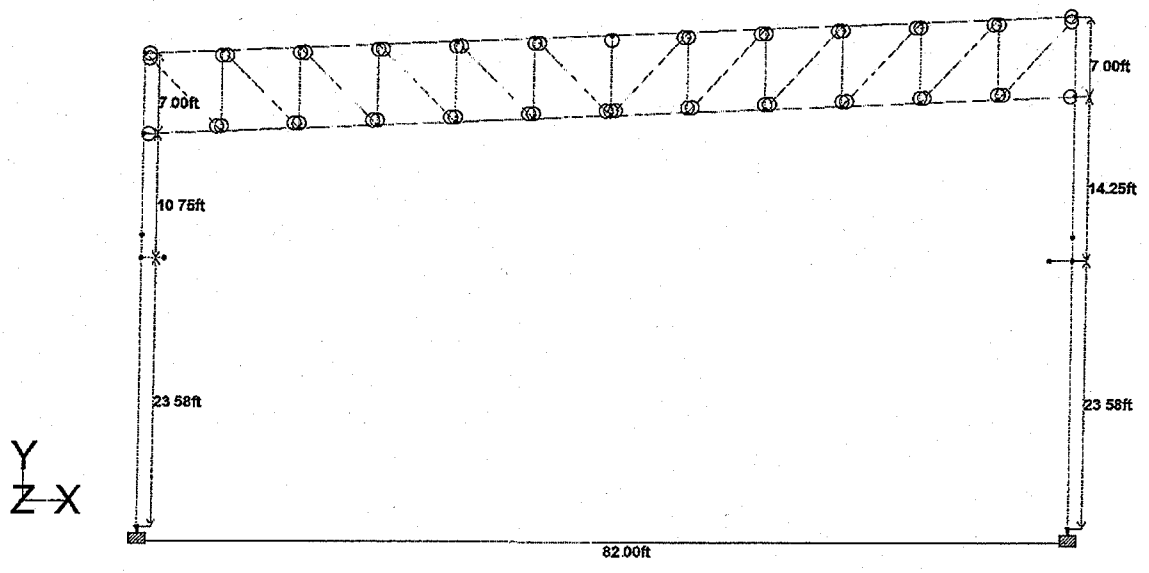
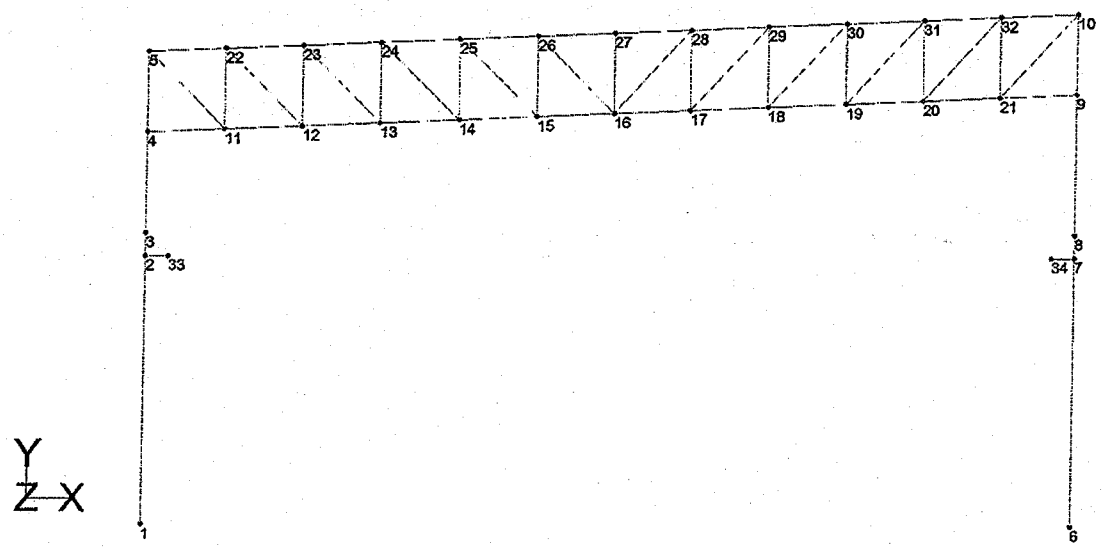
	By KBH	Date 19-May-06	Chd
Client SFWMD	File C43 Superstructure.std	Date/Time 30-May-2006 18:57	



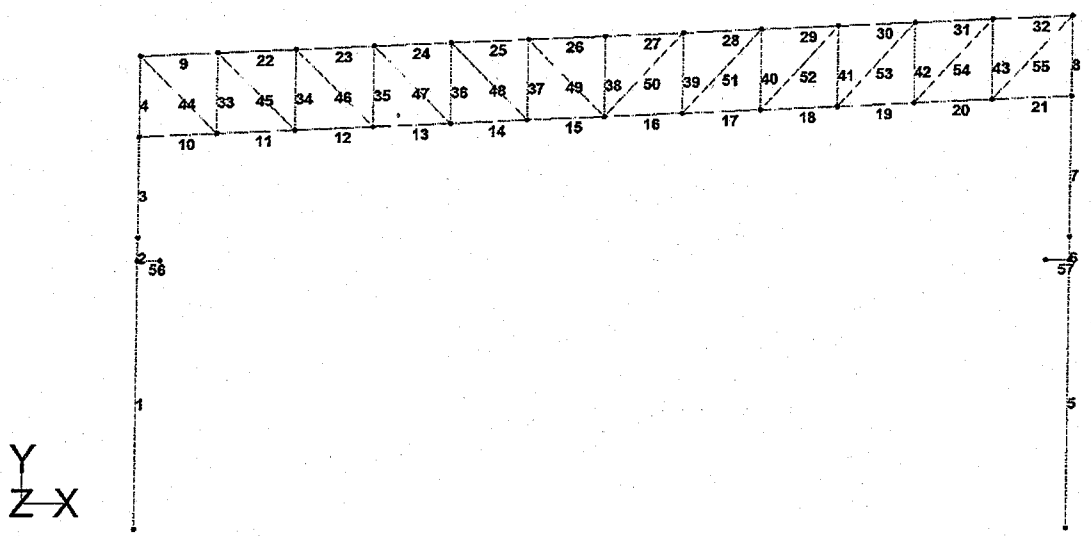
Load 4

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS1	Ref		
	By KBH	Date 19-May-06	Chd
Client SFWMD	File C43 Superstructure.std	Date/Time 30-May-2006 18:57	



Load 4

Software licensed to STANLEY CONSULTANTS INC	Job No 17300.09.02	Sheet No 1	Rev
	Part		
Job Title C43PS1	Ref		
	By KBH	Date 19-May-06	Chd
Client SFWMD	File C43 Superstructure.std	Date/Time 30-May-2006 18:57	



Load 4

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Frame Structural Design

NOTE: DIMENSIONS AND REINFORCEMENT DENOTED ON DRAWINGS IS PRELIMINARY AND SHALL BE MODIFIED IN FUTURE DESIGN PHASES.

PURPOSE: Provide preliminary loading for building reinforced concrete frame.

REFERENCES:

1. Florida Building Code.
2. Building Code Requirements for Reinforced Concrete – ACI 318-05.
3. Concrete for Environmental Structures – ACI 350R.
4. ASCE standard "Minimum Design Loads for Buildings and Other Structures: ANSI/ASCE 7-02.
5. SFWMD Guidelines Type 2 Pump Station.

DESIGN CRITERIA:

1. F'_c = 4000 psi.
2. Fy = 60 ksi.
3. The structure's dead loads shall be estimated with the assumption of a concrete unit weight of 140 pcf.
4. The structure's dead loads shall be estimated with the assumption of the unit weight of a fully grouted 12 in. wide CMU = 135 pcf.
5. The structure's dead load shall be assumed with a unit weight for a 10" thick hollowcore slab = 86 psf.
6. The roofing D.L. shall be taken as 9 psf.
7. Roof loads shall be carried by the center roof beams or the east or west beams at el. 39.0 ft.
8. For the main wind resisting system the design pressure on the walls shall be taken as:
   - Interior area of walls: +32.6 psf -27.2 psf
   - Wall edges: +45.0 psf -35.2 psf
10. The net wind pressures calculated for components and cladding:
    - Roof Interior +23 psf -64 psf
    - Masonry infill panels: +57 psf -64 psf
    - Precast roof panels: +23 psf -64 psf
11. Load Factors, ref. 2
    - U = 1.4(D + F) ACI Equation 9-1
    - U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) ACI Equation 9-2
    - U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) ACI Equation 9-3
    - U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) ACI Equation 9-4
    - U = 1.2D + 1.0E + 1.0L + 1.6S ACI Equation 9-5
    - U = 0.9D + 1.6W + 1.6H ACI Equation 9-6
    - U = 0.9D + 1.0E + 1.6H ACI Equation 9-7

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Frame Structural Design

ASSUMPTIONS:

1. The transfer of the lateral loads of the masonry infill panels is assumed to be vertically to the floor slabs and beams.
2. The transfer of the roof loads shall be to the east and west walls and to the center roof beams.
3. The beam end loads shall be to transferred to columns which will in turn transfer their loads to the floor slab.
4. The rain load shall consider the condition of complete closure of all westerly scuppers and flow over the ridge to the east side. Therefore:

   R.L.= ((43.3 - 42.6)/2) x 62.4 = 21.8 psf.

   The westerly part of the roof shall carry 2/3 of this load or:

   (21.8 x 26) x 2/3 = 378 lbs/lf along the westerly roof edge

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Frame Structural Design

CALCULATIONS: BEAM B-1 TYPICAL LOAD CASE:

PRELIMINARY DIMENSIONS 24"d x 24"w beam at el. 29.0 ft.

Uniform Lateral Wind Load = (5 + 4 +2) x -64 psf = 704 lbs/lf

Lateral Wind Load

U = 1.4(D + F) = 0.0 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 0.0 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.563 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 1.126 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0.0 k/lf

U = 0.9D + 1.6W + 1.7H = 1.126 k/lf

U = 0.9D + 1.0E + 1.7H = 0.0 k/lf

Serviceability factor = 1.3, ref. 3

1.126 x 1.3 = 1.46 k/lf

Beam D.L. = (4 x 140) = 0.56 k/lf

Masonry Load D.L.= (8) x 135 psf = 1.08 k/lf

Total D.L. = 0.56 + 1.08 = 1.64 k/lf

U = 1.4(D + F) = 2.296

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 1.968 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 1.968 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 1.968 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 1.968 k/lf

U = 0.9D + 1.6W + 1.7H = 1.476 k/lf

U = 0.9D + 1.0E + 1.7H = 1.476 k/lf

Serviceability factor = 1.3, ref. 3

2.296 x 1.3 = 2.98 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Frame Structural Design

BEAM B-2 - LOAD CASE: WEST BEAM - TAKEN AS TYPICAL FOR FULL PERIMETER

PRELIMINARY DIMENSIONS: 30"d x 24"w at el. 39.0 ft.

NOTE: West Beam considered to have greatest loading and shall be used for design of full perimeter beam.

w = Uniform Lateral Wind Load = (2.67 + 4 +3.83) x -64 psf = 672 lbs/lf

Lateral Wind Load

U = 1.4(D + F) = 0.0 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 0.0 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.538 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 1.075 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0.0 k/lf

U = 0.9D + 1.6W + 1.7H = 1.075 k/lf

U = 0.9D + 1.0E + 1.7H = 0.0 k/lf

Serviceability factor = 1.3, ref. 3

1.075 x 1.3 = 1.40 k/lf

Beam D.L. = (2.5 x 2 x 140) = 0.70 k/lf

Masonry D.L. (Parapet) = (4) x 135 psf = 0.54 k/lf

Roof L.L. ref. 5 = 50 psf x 10 = 0.50 k/lf

Rain Load= (21.8 x 26) x 2/3 = 0.378 k/lf

Roofing D.L. = 9 x 10 = 0.090 k/lf

Precast Slab = 86 x 10 = 0.86 k/lf

Compression wind load = 23 x 10 = 0.23 k/lf

Total D.L. = 0.70 + 0.54 + 0.090 + 0.86 = 2.19 k/lf

U = 1.4(D + F) = 3.07

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 2.88 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 3.43 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 3.25 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 2.63 k/lf

U = 0.9D + 1.6W + 1.7H = 2.34 k/lf

U = 0.9D + 1.0E + 1.7H = 1.97 k/lf

Serviceability factor = 1.3, ref. 3

3.43 x 1.3 = 4.46 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Frame Structural Design

BEAM B-3 - LOAD CASE: N-W BEAM - TAKEN AS TYPICAL FOR ALL ROOF BEAMS

PRELIMINARY DIMENSIONS: 36"d x 24"w at el. 39.0 ft.

NOTE: N-W Beam considered to have greatest loading and shall be used for design of all roof beams.

Beam D.L. = (3.0 x 2 x 140) = 0.840 k/lf

Roof Slab D.L. = 6 x 140 psf = 0.840 k/lf

Roof L.L. ref. 5 = 50 psf x (10 + 6) = 0.80 k/lf

Rain Load= (21.8 x 26) x 1/3 = 0.187 k/lf

Roofing D.L. = 9 x 16 = 0.144 k/lf

Precast Slab = 86 x 10 = 0.86 k/lf

Compression wind load = 23 x 16 = 0.368 k/lf

Total Load D.L. = 0.84 + 0.84 + 0.144 + 0.86 = 1.8 k/lf

U = 1.4(D + F) = 2.52 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 2.56 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 3.73 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 3.15 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 2.16 k/lf

U = 0.9D + 1.6W + 1.7H = 2.21 k/lf

U = 0.9D + 1.0E + 1.7H = 1.62 k/lf

Serviceability factor = 1.3, ref. 3

3.35 x 1.3 = 4.36 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Frame Structural Design

BEAM B-4 - LOAD CASE: TYPICAL

PRELIMINARY DIMENSIONS: 24"d x 24"w at el. 29.0 ft.

Beam B-4 D.L. = (2 x 2 x 140) = 0.560 k/lf

Floor Slab D.L.: self-supporting

Beam B-2 D.L. = 2.19 k/lf

Masonry D.L. = 8 x 135 psf = 1.08 k/lf

Beam B-2 Roof L.L. = 0.50 k/lf

Beam B-2 Rain Load = 0.378 k/lf

Beam B-2 Compression wind load = 0.23 k/lf

Total D.L. = 0.56 + 2.19 + 1.08 = 3.83 k/lf

U = 1.4(D + F) = 5.362 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 4.846 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 5.396 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 5.214 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 4.596 k/lf

U = 0.9D + 1.6W + 1.7H = 3.815 k/lf

U = 0.9D + 1.0E + 1.7H = 3.443 k/lf

Serviceability factor = 1.3, ref. 3

5.396 x 1.3 = 7.01 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Frame Structural Design

CALCULATIONS: COLUMN C-1 INTERIOR WALL LOAD CASE - WEST WALL CONSIDERED WORSE CASE LOADING CONDITIONS

PRELIMINARY DIMENSIONS 24" x 24" from el.19.0 ft. to el. 39.0 ft.

Beam B-1 lateral wind load (factored) = 1.46 x 10 = 14.6 k at el. 29.0 ft.

Beam B-2 lateral wind load (factored) = 1.40 x 10 = 14.0 k at el. 39.0 ft.

Column face wind load = 2 x 26.5 x -64 = 3.4 k x 1.6 x 1.3 = 7.1 acting at el. 32.25

Beam B-1 vertical loads (factored) = 2.98 x 10 = 29.8 k at el. 29.0 ft.

Beam B-2 vertical loads (factored) = 4.46 x 10 = 44.6 k at el. 39.0 ft.

CALCULATIONS: COLUMN C-2 LOAD CASE - NW COLUMN CONSIDERED WORSE CASE LOADING CONDITIONS

PRELIMINARY DIMENSIONS 24" x 24" from el.19.0 ft. to el. 39.0 ft.

Beam B-3 vertical loads (factored) = 4.36 X 46/2 = 100.3 k

CALCULATIONS: COLUMN C-3 INTERIOR WALL CONSIDERED WORSE CASE LOADING CONDITIONS

PRELIMINARY DIMENSIONS 24" x 24" from el. 31.0 ft. to el. 39.0 ft.

Beam B-2 lateral wind load (factored) = 1.40 x 10 = 14.0 k at el. 39.0 ft.

Column face wind load = 2 x 14.5 x -64 = 1.9 k x 1.6 x 1.3 = 3.9 acting at el. 38.25

Beam B-2 vertical loads (factored) = 4.46 x 10 = 44.6 k at el. 39.0 ft.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Service Bridge and Operating Floor Slab Structural Design

NOTE: SLAB DIMENSIONS AND REINFORCEMENT CURRENTLY SHOWN ON DRAWINGS IS CONCEPTUAL AND SHALL BE MODIFIED IN ACCORDANCE WITH THE FUTURE PHASES OF THE DETAILED DESIGN.

PURPOSE: Provide PRELIMINARY structural design operating level floor slabs

REFERENCES:

1. Florida Building Code.
2. Building Code Requirements for Reinforced Concrete – ACI 318-05.
3. Concrete for Environmental Structures – ACI 350R.
4. ASCE standard "Minimum Design Loads for Buildings and Other Structures: ANSI/ASCE 7-02
5. Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, (AASHTO).
6. SFWMD Guidelines Type 1 Pump Station.

DESIGN CRITERIA:

1. $F'_c = 4000$ psi.
2. $F_y = 60$ ksi.
3. AASHTO HS25 loading shall be considered for all floor slabs. ref. 6.
4. The structure's dead loads shall be estimated with the assumption of the concrete unit weight of 140 pcf.
5. The slabs shall be designed for a 100 psf live load loading. ref 6.
6. Load Factors, ref. 2

- $U = 1.4(D + F)$ — ACI Equation 9-1
- $U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ — ACI Equation 9-2
- $U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)$ — ACI Equation 9-3
- $U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$ — ACI Equation 9-4
- $U = 1.2D + 1.0E + 1.0L + 1.6S$ — ACI Equation 9-5
- $U = 0.9D + 1.6W + 1.6H$ — ACI Equation 9-6
- $U = 0.9D + 1.0E + 1.6H$ — ACI Equation 9-7

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Service Bridge and Operating Floor Slab Structural Design

ASSUMPTIONS:

1. SLAB S-1: Electrical room floor slab, S-1 with a T.O.S. of el. 31.0 ft. shall be considered as a one-way slab supported by the piers or abutments with a L.L. of 100 psf as per ref. 6. No additional loading shall be considered.
2. SLAB S-2: The roof slab spans between roof beams and shall be considered as a one-way slab.
3. SLAB S-3: The operating room slab shall be a slab on grade and shall support AASHTO HS25 loading as well as a 100 psf L.L. as per ref.6. AASHTO HS25 loading shall be taken as a 10 k front axle, 40 k drive axle and a 40 k trailer axle with the concentrated wheel load of 20k acting on a 4 ft. wide strip of floor slab.
4. SLAB S-4: The discharge chamber slab S-4 at elevation 30.9 shall be considered as a one-way slab supported by the piers or abutments with a L.L. of 100 psf and AASHTO HS25 loading as per ref. 6.
5. SLAB S-5: The discharge chamber floor slab shall be a slab on grade and shall support a 100 psf L.L. as per ref.6. and a water load to elevation 27.4 ft.

CALCULATIONS SLAB S-1, Electrical and Generator Rooms:
PRELIMINARY: Thickness 12"

Floor D. L. for 12 in. slab width = 140 x 1.0 = 0.14 k/lf

L.L. for 12 in. slab width = 100 psf = 0.1 k/lf

U = 1.4(D + F) = 0.196 k/lf

U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) = 0.328 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.218 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0.218 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0.218 k/lf

U = 0.9D + 1.6W + 1.6H = 0.126 k/lf

U = 0.9D + 1.0E + 1.6H = 0.126 k/lf

Serviceability factor = 1.3, ref. 3

w = 1.3 x 0.328 = 0.426 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Service Bridge and Operating Floor Slab Structural Design

CALCULATIONS SLAB S-2, Roof Slab:
PRELIMINARY: Thickness 12"

Slab D. L. for 12 in. slab width = 140 x 1.0 = 0.14 k/lf

Roofing D.L. = 9 psf = 0.009 k/lf

Compression Wind Load = 23 psf = 0.023 k/lf

Uplift Wind Load = -64 psf = - 0.064 k/lf

L.L. for 12 in. slab width = 50 psf = 0.05 k/lf

For maximum vertical loading;

U = 1.4(D + F) = 0.209 k/lf
U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) = 0.259 k/lf
U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.204 k/lf
U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0.241 k/lf
U = 1.2D + 1.0E + 1.0L + 1.6S = 0.204 k/lf
U = 0.9D + 1.6W + 1.6H = 0.171 k/lf
U = 0.9D + 1.0E + 1.6H = 0.134 k/lf
Serviceability factor = 1.3, ref. 3
w = 1.3 x 0.259 = 0.337 k/lf

For maximum uplift loading;

U = 1.4(D + F) = 0.209 k/lf
U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) = 0.259 k/lf
U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.204 k/lf
U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0.101 k/lf
U = 1.2D + 1.0E + 1.0L + 1.6S = 0.204 k/lf
U = 0.9D + 1.6W + 1.6H = 0.032 k/lf
U = 0.9D + 1.0E + 1.6H = 0.134 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Service Bridge and Operating Floor Slab Structural Design

CALCULATIONS SLAB S-3, Operating Room Floor:
PRELIMINARY: Thickness 36"

Floor D. L. for 12 in. slab width = 140 x 3.0 = 0.420 k/lf; for L = 10 ft. D.L. = 4.2 k

L.L. for 12 in. slab width = 100 psf = 0.1 k/lf: for L = 10 ft. D.L. = 1.0 k

AASHTO HS25 Loading: AASHTO HS25 loading shall be taken as a 10 k front axle, 40 k drive axle and a 40 k trailer axle with the concentrated wheel load of 20k acting on a 4 ft. wide strip of floor slab.

L.L. = 20k/4 = 5k

U = 1.4(D + F) = 5.88 k

U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) = 14.64 k

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 8.04 k/

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 8.04 k

U = 1.2D + 1.0E + 1.0L + 1.6S = 8.04 k

U = 0.9D + 1.6W + 1.6H = 3.78 k

U = 0.9D + 1.0E + 1.6H = 3.78 k

Serviceability factor = 1.3, ref. 3

P = 1.3 x 14.64 = 19.03 k

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Service Bridge and Operating Floor Slab Structural Design

CALCULATIONS SLAB S-4, Discharge Chamber Service Bridge:
PRELIMINARY: Thickness 12"

Floor D. L. for 12 in. slab width = 140 x 1.0 = 0.140 k/lf; for L = 10 ft.: D.L. = 1.4k

L.L. for 12 in. slab width = 100 psf = 0.1 k/lf; for L = 10 ft.: L.L. = 1.0 k

AASHTO HS25 Loading: AASHTO HS25 loading shall be taken as a 10 k front axle, 40 k drive axle and a 40 k trailer axle with the concentrated wheel load of 20k acting on a 4 ft. wide strip of floor slab.

L.L. = 20k/4 = 5k

U = 1.4(D + F) = 1.96 k

U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) = 11.28 k

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 4.68 k

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 4.68 k

U = 1.2D + 1.0E + 1.0L + 1.6S = 4.68 k

U = 0.9D + 1.6W + 1.6H = 1.26 k

U = 0.9D + 1.0E + 1.6H = 1.26 k

Serviceability factor = 1.3, ref. 3

P = 1.3 x 11.28 = 14.66 k

CALCULATIONS SLAB S-5, Discharge Chamber Base Slab:
PRELIMINARY: Thickness 36"

Floor D. L. for 12 in. slab width = 140 x 3.0 = 0.42 k/lf

L.L. for 12 in. slab width = 100 psf = 0.1 k/lf

Water load = 8.4 x 62.4 = 0.524 k/lf

U = 1.4(D + F) = 1.322 k/lf

U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) = 1.293 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.554 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0.554 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0.554 k/lf

U = 0.9D + 1.6W + 1.6H = 0.378 k/lf

U = 0.9D + 1.0E + 1.6H = 0.378 k/lf

Serviceability factor = 1.3, ref. 3

w = 1.3 x 1.293 = 1.68 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 4-16-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Intake Structural Design

PURPOSE: Provide preliminary structural design of buttressed portion of intake structure

REFERENCES:

1. Florida Building Code.
2. Building Code Requirements for Reinforced Concrete – ACI 318-05.
3. Concrete for Environmental Structures – ACI 350R.
4. ASCE standard "Minimum Design Loads for Buildings and Other Structures: ANSI/ASCE 7-02.
5. SFWMD Guidelines Type 2 Pump Station.
6. Tables and Formulas for Fixed End Moments, Rogers.

DESIGN CRITERIA:

1. F'_c = 4000 psi.
2. Fy = 60 ksi.
3. The lateral soil and water load was taken as an equivalent fluid pressure 92.4 pcf.
4. The structure's dead loads shall be estimated with the assumption of the concrete unit weight of 140 pcf.
5. The wall shall be analyzed for the condition of the soil saturated to grade, ref. 5. for the reinforced concrete design.
6. A 500 psf surcharge shall be considered as per ref 5. for the reinforced concrete design.

ASSUMPTIONS:

1. The transfer of the lateral loads is assumed to be horizontally to the abutment and pier walls.
2. Case 1 Backwall: The wall above the intake pipe at the crest elevation of el. -6.4 was analyzed as a beam which spans from wall to wall with the most severe loading condition for the one foot strip above this elevation. 70% of the load either side of the pipe is considered to be distributed to the pier, abutment and base slab with the remaining 30% evenly distributed to this one foot strip of wall above.
3. Case 2 Abutment and Piers: Pier and abutment thicknesses 2.0 ft. as per requirements of ref. 5. The walls structural function is for the transfer of overturning moment and shear to the base slab. The shear load shall be shared with the effective length of the backwall. The bays will not be dewatered.
4. Case 3 Base Slab: The slab shall be assumed to be a rigid mat with a equivalent bearing pressure of 2230 psf as calculated in the stability analysis. Analyze 1 ft. strip of base spanning between the piers and abutments.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 4-16-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Intake Structural Design

CALCULATIONS - CASE 1 - BACKWALL:

w = Uniform soil load at el. -12.0 = 22 x 92.4 = 2032 lbs/lf

w = Uniform soil load at el. -6.9 = 17 x 92.4 = 1571 lbs/lf

load on wall between el. -12.0 and -6.9 ft. ~ 8998 x 6 = 54k

30% attributed to wall above = 0.3 x 54k/10 = 1620 lbs/lf

total w (for one ft. strip) at elevation -6.9 ft. = 1571 + 1620 = 3191 lbs/lf

surcharge = 500 psf

Load Factors, ref. 2

U = 1.4(D + F)	ACI Equation 9-1
U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)	ACI Equation 9-2
U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W)	ACI Equation 9-3
U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R)	ACI Equation 9-4
U = 1.2D + 1.0E + 1.0L + 1.6S	ACI Equation 9-5
U = 0.9D + 1.6W + 1.6H	ACI Equation 9-6
U = 0.9D + 1.0E + 1.6H	ACI Equation 9-7

Increase lateral soil load factor to 1.7, ref. 3.

U = 1.4(D + F) = 0.7 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 6.02 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.6 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0.6 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0.6 k/lf

U = 0.9D + 1.6W + 1.7H = 5.87 k/lf

U = 0.9D + 1.0E + 1.7H = 5.87 k/lf

Serviceability factor = 1.3, ref. 3

6.02 x 1.3 = 7.83 k/lf

Vu = 7.83 x 10/2 = 19.58 k/lf at the buttress

ϕVc = (0.75)(2 x (4000^1/2)(12)(x) = 19.58 k

x = 17.2"

Analyze as a beam fixed at both ends with uniform load:

M_{max} at ends = (7.83k) x 10^2)/12

M_{max} @ ends = 65.25 k-ft. = 783,000 lb-in.

M @ center = (7.83k) x 10^2)/24

M @ center = 32.63 k-ft.

A_s (#6 @ in) = 0.884 sq. in.

b = 12 in.

d = 21.5 in.

h = 24.0 in.

RESULTS:

Computed design data for bending moment specified:

- required A_s = 0.691 sq. in.
- rho = 0.0027
- phi = 0.900
- phi(M_n) = 0.900 x 870324 = 783,292 lb-in

For steel provided:

- rho = 0.003
- phi = 0.900
- phi(M_n) = 0.900 x 1,105,884 = 995,295.6 lb-in.

Provide #6 @ 6 in. horizontal each face w/ #6 @ 6 in. vertical each face.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 4-16-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Intake Structural Design

CALCULATIONS - CASE 2 - PIERS:

Backwall lateral soil load = 22.4 k/lf x 12 = 268.8 k

Backwall lateral surcharge load = 11.0k x 12 = 132 k

Total length wall and pier resisting shear = (18 x 2) + (12 x 2) = 60 sq. ft.

Total lateral load = 268.8k + 132k = 400.8k

Load Factors, ref. 2

U = 1.4(D + F)	ACI Equation 9-1
U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)	ACI Equation 9-2
U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W)	ACI Equation 9-3
U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R)	ACI Equation 9-4
U = 1.2D + 1.0E + 1.0L + 1.6S	ACI Equation 9-5
U = 0.9D + 1.6W + 1.6H	ACI Equation 9-6
U = 0.9D + 1.0E + 1.6H	ACI Equation 9-7

Increase lateral soil load factor to 1.7, ref. 3.

U = 1.4(D + F) = 184.8 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 615.4 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 158.4 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 158.4 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 158.4 k/lf

U = 0.9D + 1.6W + 1.7H = 575.8 k/lf

U = 0.9D + 1.0E + 1.7H = 575.8 k/lf

Serviceability factor = 1.3, ref. 3

615.4 x 1.3 = 800 k

Vu = 800k/30 = 24k/lf

ϕV_c = (0.75)(2 x (4000^1/2)(12)(x) = 24 k

x = 21.08"

Piers have no vertical or lateral loads.

The moment induced factored compressive = 800k/(18) = 22.2k/lf, consider an axial loaded 1 ft. diagonal strip of pier from el. 10.0 ft. to the base slab.

Minimum A_s = 2.88 sq in as per ACI 10.9.1

For the forces applied:

A_s = 0.001
phi = 0.900
0.900 x P_n = 22,200 lbs
0.900 x M_n = 260921 in-lbs

RESULTS:

Provide 2 ft. wide pier w/ # 8 @ 6 in. vertical and horizontal, each face.

For the axial force and steel area specified:

A_s = 3.140
phi = 0.900
0.900 x P_n = 22,166 lbs
0.900 x M_n = 3,540,225 in-lbs

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 4-16-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Intake Structural Design

CALCULATIONS - CASE 3 - BASE SLAB:

w = equivalent uniform bearing pressure from lateral soil loads = 2230 lbs/lf

D.L. base slab = 135 x 3 = 0.405 k/lf

D.L. pier = (135 x 22 x 2)/12 = 0.495 k/lf

Total w = 3.13 k/lf

Load Factors, ref. 2

U = 1.4(D + F)	ACI Equation 9-1
U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)	ACI Equation 9-2
U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W)	ACI Equation 9-3
U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R)	ACI Equation 9-4
U = 1.2D + 1.0E + 1.0L + 1.6S	ACI Equation 9-5
U = 0.9D + 1.6W + 1.6H	ACI Equation 9-6
U = 0.9D + 1.0E + 1.6H	ACI Equation 9-7

Increase lateral soil load factor to 1.7, ref. 3.

U = 1.4(D + F) = 0 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 5.321 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0 k/lf

U = 0.9D + 1.6W + 1.7H = 5.321 k/lf

U = 0.9D + 1.0E + 1.7H = 5.321 k/lf

Serviceability factor = 1.3, ref. 3

5.321 x 1.3 = 6.92 k/lf

maximum V_u at piers = 0.600wL = 0.6 x 6.92 x 10 = 41.52k

ϕVc = (0.75)(2 x (4000^1/2)(12)(x) = 41.52 k

x = 36.48"

Analyze as a slab as continuous beam fixed at both ends with uniform load:

M_{pier} = -0.100 x w x L^2 = 0.100 x 6.92 x 100 = 69.2 k-ft. = 830,400 in-lbs

$M_{center\ span}$ = +0.080 x w L^2 = 0.08 x 6.92 x 100 = 55.36 k-ft = 664,320 in-lbs

As = 1.570 sq in

b = 12 in.

d = 31.5 in.

t = 36 in.

For the negative bending moment specified:

As = 0.494 sq in
rho = 0.0013
phi = 0.900
0.900 x Mn = 830,604 lb-in

For steel provided:

rho = 0.004
phi = 0.900
0.900 x (2,858,554) = 2,572,699 lb-in

For the positive bending moment specified:

As = 0.395 sq in
rho = 0.0010
phi = 0.900
0.900 x Mn = 665,670 lb-in

For steel provided:

rho = 0.004
phi = 0.900
0.900 x (2,858,554) = 2,572,699 lb-in

RESULTS:

Provide #8 @ 6 in. top and bottom in the N-S direction, #8 @ 12 in. top and bottom in the E-W direction.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Masonry Design

PURPOSE: Provide PRELIMINARY structural design of masonry of building infill panels.

REFERENCES:

1. Florida Building Code.
2. Building Code Requirements for Reinforced Concrete – ACI 318-05.
3. National Concrete Masonry Association design tables
4. ASCE standard "Minimum Design Loads for Buildings and Other Structures: ANSI/ASCE 7-02.
5. SFWMD guidelines for Type 1 Pump Stations:

DESIGN CRITERIA:

1. Fy = 60 ksi.
2. Wind loads calculated for 200 yr. recurrence for components and cladding, ref. 5 = +57 psf, -64 psf, typical for masonry infill panels.
3. The masonry wall shall be fully grouted w/ a 12 in. x 16 in. x 8 in. two cell concrete block.
4. The masonry unit shall have fm = 2000 psi.
5. The grout shall have a compressive strength of 2500 psi.
6. The mortar shall have a compressive strength of 2500 psi.
7. Load Factors, ref. 2
   - U = 1.4(D + F) ACI Equation 9-1
   - U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) ACI Equation 9-2
   - U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) ACI Equation 9-3
   - U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) ACI Equation 9-4
   - U = 1.2D + 1.0E + 1.0L + 1.6S ACI Equation 9-5
   - U = 0.9D + 1.6W + 1.6H ACI Equation 9-6
   - U = 0.9D + 1.0E + 1.6H ACI Equation 9-7

ASSUMPTIONS:

1. The masonry wall loading is uniformly distributed.
2. The masonry wall is not axially loaded.
3. The masonry wall shall span vertically from horizontal frame component to frame component.
4. The masonry wall is tied to the adjacent columns by horizontal bond beams.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-20-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Masonry Design

CALCULATIONS - TYPICAL 10 FT. X 10 FT. INFILL PANEL:

Wind lateral load = 0.064 k/lf

U = 1.4(D + F) = 0

U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) = 0

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0.051 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0.1029 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0

U = 0.9D + 1.6W + 1.6H = 0.1029 k/lf

U = 0.9D + 1.0E + 1.6H = 0

Serviceability factor = 1.3, ref. 3

w = 1.3 x 0.1029 = 0.1338 k/lf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-11-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Retaining Walls Structural Design

PURPOSE: Provide preliminary structural design of retaining walls.

REFERENCES:

1. Florida Building Code.
2. Building Code Requirements for Reinforced Concrete – ACI 318-05.
3. Concrete for Environmental Structures – ACI 350R.
4. ASCE standard "Minimum Design Loads for Buildings and Other Structures: ANSI/ASCE 7-02.
5. SFWMD Guidelines Type 2 Pump Station.

DESIGN CRITERIA:

1. F'_c = 4000 psi.
2. Fy = 60 ksi.
3. The lateral soil and water load was taken as an equivalent fluid pressure 92.4 pcf.
4. The structure's dead loads shall be estimated with the assumption of the concrete unit weight of 140 pcf.
5. The wall shall be analyzed for the condition of the soil saturated to grade, ref. 5. for the reinforced concrete design.
6. A 500 psf surcharge shall be considered as per ref 5. for the reinforced concrete design.
7. Backfill angle = 0 degrees.
8. Load Factors, ref. 2
   - U = 1.4(D + F) — ACI Equation 9-1
   - U = 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R) — ACI Equation 9-2
   - U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) — ACI Equation 9-3
   - U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) — ACI Equation 9-4
   - U = 1.2D + 1.0E + 1.0L + 1.6S — ACI Equation 9-5
   - U = 0.9D + 1.6W + 1.6H — ACI Equation 9-6
   - U = 0.9D + 1.0E + 1.6H — ACI Equation 9-7
9. Increase lateral soil load factor to 1.7, ref. 3.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-11-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Retaining Walls Structural Design

ASSUMPTIONS:

1. Backfill behind the walls and foundation assumed to be cohesiveless structural fill.
2. Active pressure was assumed for the fill behind the walls.
3. Case 1: Intake Structure Retaining Walls with Top Elevation 10.0 ft. NAVD: The fill in front of the wall at the abutment is elevation -1.0 ft. for both walls. The side slope goes upward at 2h:1v at this outside face therefore lessening the differential soil load to an insignificant amount proceeding away from the intake. The walls and their base slab shall be tied to the intake, therefore a one foot width of wall shall be analyzed at a distance 8 ft. from the abutment face. The channel side soil elevation shall be taken as el. 3.0 at this point.
4. Case 2: West Retaining Wall with Top Elevation of 19 ft. NAVD: The fill in front of the wall shall be taken as elevation 10.0 ft. for the wall as the worse case scenario for the analysis. The fill behind the wall shall be taken as 19.0, The top of the base slab at elevation 8.0 ft. The wall and its base slab shall be tied to the building but the analysis shall assume an independent structure. The wall at the structure is further buttressed by the access step structure.
5. Case 3A East Retaining Walls with Top Elevation 31.0 ft. NAVD, Discharge Side: The wall and base slab shall be tied to the abutment therefore adding support in the vicinity of the structure. Therefore a one foot strip of the wall shall be analyzed at a distance 10 ft. from the structure. The side slope goes upward at 2.5h:1v at this outside face from the discharge sill therefore lessening the differential soil load to an insignificant amount proceeding away from the structure. The fill in front of the wall shall be taken as elevation 24.0 ft. for both walls as the worse case scenario for the analysis. The inside face soil elevation also slopes down at a 10h:1v slope, therefore further lessening the differential soil load on the wall. The soil elevation shall be taken as 30.0 ft. NAVD for the inside face.
6. Case 3B East Retaining Walls with Top Elevation 31.0 ft. NAVD, Intake Side: The wall and base slab shall be tied to the building wall therefore adding support in the vicinity of the structure. Therefore a one foot strip of the wall shall be analyzed at a distance 10 ft. from the structure. The side slope goes upward at 3h:1v at this outside face from the discharge sill therefore lessening the differential soil load to an insignificant amount proceeding away from the structure. The fill in front of the wall shall be taken as elevation 23.0 ft. for both walls as the worse case scenario for the analysis. The inside face soil elevation also slopes down at a 10h:1v slope, as well as from the roadway to the wall, (a distance of approximately 10 ft.) at a 3h:1v slope therefore further lessening the differential soil load on the wall. The soil elevation shall be taken as 28.0 ft. NAVD for the inside face.

CASE 1- CALCULATIONS - REINFORCED CONCRETE STEM:

Top of wall (stem) = el. 10.0 ft. NAVD w/ 2 ft. thickness.

Top of base slab = el. -12.0 f. NAVD w/ 4.5 ft. thickness.

Toe extension length = 10.0 ft.

Heel extension length = 6.5 ft.

Lateral soil load = (22^2)/2 x 92.4 = 22,361 lbs/lf) - (15^2)/2 x 92.4 = 10,395 lbs/lf) = 11.97 k/lf

surcharge = 22.0 x 500 psf = 11.0 lbs/lf

Total lateral load = 25.0 k/lf

Increase lateral soil load factor to 1.7, ref. 3.

U = 1.4(D + F) = 0 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 42.5 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0 k/lf

U = 0.9D + 1.6W + 1.7H = 42.5 k/lf

U = 0.9D + 1.0E + 1.7H = 42.5 k/lf

Serviceability factor = 1.3, ref. 3

42.5 x 1.3 = 55.25 k/lf

V_u = 24.5 k/lf at base of stem

ϕV_c = (0.75)(2 x (4000^1/2)(12)(x) = 55.25 k

x = 48.55"

Analyze stem as cantilever w/o channel face fill:

22.4k x 1.7 x 1.3 = 49.4 k/lf at el. -4.67

500 x 22.0 x 1.7 x 1.3 = 24.3 k/lf at el. -1.0

M_{max} at base of stem = (49.4k x 7.33) + (24.3k x 11) = 629.6 k-ft/lf = 7,554,800 in-lbs/lf

A_s (#11 @ 6 in) = 3.128 sq. in.

b = 12 in.

d = 50.0 in.

h = 54.0 in.

RESULTS:

For steel provided:

- rho = 0.052
- phi = 0.900
- phi(M_n) = 0.900 x 8,941,439 = 8,047,295 lbs-in.

Provide #11 @ 6 in. vertical tension face.

CASE 2- CALCULATIONS - REINFORCED CONCRETE STEM:

Top of wall (stem) = el. 19.0 ft. NAVD w/ 1.5 ft. thickness.

Top of base slab = el. 8.0 f. NAVD w/ 2 ft. thickness.

Toe extension length = 4.0 ft.

Heel extension length = 2.0 ft.

Lateral soil load = (11^2)/2 x 92.4 = 5590 lbs/lf) = 5.6 k/lf

surcharge = 11.0 x 500 psf = 5.5 lbs/lf

Total lateral load = 11.1 k/lf

Increase lateral soil load factor to 1.7, ref. 3.

U = 1.4(D + F) = 0 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 18.87 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0 k/lf

U = 0.9D + 1.6W + 1.7H = 18.87 k/lf

U = 0.9D + 1.0E + 1.7H = 18.87 k/lf

Serviceability factor = 1.3, ref. 3

18.187 x 1.3 = 24.5 k/lf

V_u = 24.5 k/lf at base of stem

ϕV_c = (0.75)(2 x (4000^1/2)(12)(x) = 24.5 k

x = 21.55"

Analyze stem as cantilever:

5.6k x 1.7 x 1.3 = 12.4 k/lf at el. 11.67

500 x 11.0 x 1.7 x 1.3 = 12.2 k/lf at el. 13.5

M_{max} at base of stem = (12.4k x 3.67) + (12.2k x 5.5) = 112.6 k-ft/lf = 1,351,296 in-lbs/lf

A_s (#8 @ 6 in) = 1.570 sq. in.

b = 12 in.

d = 21.5.0 in.

h = 24.0 in.

RESULTS:

For steel provided:

- rho = 0.0061
- phi = 0.900
- phi(M_n) = 0.900 x 1,916,554 = 1,724,899 lbs-in.

Provide #8 @ 6 in. vertical tension face.

CASE 3A- CALCULATIONS - REINFORCED CONCRETE STEM:

Top of wall (stem) = el. 31.0 ft. NAVD w/ 1.5 ft. thickness.

Top of base slab = el. 18.0 f. NAVD w/ 2 ft. thickness.

Toe extension length = 2.0 ft.

Heel extension length = 4.0 ft.

Lateral soil load = (12^2)/2 x 92.4 = 6653 lbs/lf) - (6.5^2)/2 x 92.4 = 1,952 lbs/lf) = 4.7 k/lf

surcharge = 12.0 x 500 psf = 6.0 lbs/lf

Total lateral load = 10.7 k/lf

Load Factors, ref. 2

Increase lateral soil load factor to 1.7, ref. 3.

U = 1.4(D + F) = 0 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 18.19 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0 k/lf

U = 0.9D + 1.6W + 1.7H = 18.19 k/lf

U = 0.9D + 1.0E + 1.7H = 18.19 k/lf

Serviceability factor = 1.3, ref. 3

18.19 x 1.3 = 23.65 k/lf

V_u = 23.65 k/lf at base of stem

ϕV_c = (0.75)(2 x (4000^1/2)(12)(x) = 23.65 k

x = 20.8"

Analyze stem as cantilever, ignore soil side fill:

6.65k x 1.7 x 1.3 = 14.7 k/lf at el. 22.0

500 x 12.0 x 1.7 x 1.3 = 13.3 k/lf at el. 24.0

M_{max} at base of stem = (14.7k x 4) + (13.3k x 6) = 138.6 k-ft/lf = 1,663,200 in-lbs/lf

A_s (#8 @ 6 in) = 1.570 sq. in.

b = 12 in.

d = 21.5.0 in.

h = 24.0 in.

RESULTS:

For steel provided:

- rho = 0.0061
- phi = 0.900
- phi(M_n) = 0.900 x 1,916,554 = 1,724,899 lbs-in.

Provide #8 @ 6 in. vertical tension face.

CASE 3B- CALCULATIONS - REINFORCED CONCRETE STEM:

Top of wall (stem) = el. 31.0 ft. NAVD w/ 1.5 ft. thickness.

Top of base slab = el. 18.0 f. NAVD w/ 2 ft. thickness.

Toe extension length = 2.0 ft.

Heel extension length = 4.0 ft.

Lateral soil load = (10^2)/2 x 92.4 = 4620 lbs/lf) - (4.33^2)/2 x 92.4 = 866 lbs/lf) = 3.75 k/lf

surcharge = 10.0 x 500 psf = 5.0 lbs/lf

Total lateral load = 8.75 k/lf

Load Factors, ref. 2

Increase lateral soil load factor to 1.7, ref. 3.

U = 1.4(D + F) = 0 k/lf

U = 1.2(D + F + T) + 1.7(L + H) + 0.5(Lr or S or R) = 14.9 k/lf

U = 1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W) = 0 k/lf

U = 1.2D + 1.6W + 1.0L + 0.5(Lr or S or R) = 0 k/lf

U = 1.2D + 1.0E + 1.0L + 1.6S = 0 k/lf

U = 0.9D + 1.6W + 1.7H = 14.9 k/lf

U = 0.9D + 1.0E + 1.7H = 14.9 k/lf

Serviceability factor = 1.3, ref. 3

14.9 x 1.3 = 19.33 k/lf

V_u = 19.33 k/lf at base of stem

ϕV_c = (0.75)(2 x (4000^1/2)(12)(x) = 19.33 k

x = 17.0"

Analyze stem as cantilever, ignore soil side fill:

4.62k x 1.7 x 1.3 = 10.2 k/lf at el. 21.33

500 x 10.0 x 1.7 x 1.3 = 11.05 k/lf at el. 23.0

M_{max} at base of stem = (10.2k x 3.33) + (11.05k x 5) = 89.2 k-ft/lf = 1,070,592 in-lbs/lf

A_s (#8 @ 6 in) = 1.570 sq. in.

b = 12 in.

d = 21.5.0 in.

h = 24.0 in.

RESULTS:

For steel provided:

- rho = 0.0061
- phi = 0.900
- phi(M_n) = 0.900 x 1,916,554 = 1,724,899 lbs-in.

Provide #8 @ 6 in. vertical tension face.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-06-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Stability Analysis

PURPOSE: Provide preliminary stability analysis of retaining wall structures. A more detailed analysis shall performed in the next detailed design phase by geotechnical engineer based on addition site investigations.

REFERENCES:

1. Soil Mechanics in Engineering Practice, Terzaghi, Peck.
2. Foundation Design, Coduto.
3. Foundation Engineering, Peck.

DESIGN CRITERIA:

1. The analysis was performed using the Coulomb method.
2. Backfill angle = 0 degrees.
3. Foundation soil: friction angle = 35 deg., cohesion = 0, unit wt. = 120 pcf, footing friction = 30 deg.
4. Soil behind wall: friction angle = 35 deg., cohesion = 0, unit wt. = 114 pcf, wall friction = 26 deg.

ASSUMPTIONS:

1. Backfill behind the walls and foundation assumed to be cohesiveless structural fill.
2. Active pressure was assumed for the fill behind the walls.
3. Case 1: Intake structure.
4. Case 2: Intake structure cantilever retaining walls with top elevation of 10 ft. NAVD and no fill placed on canal side of wall with no fill placed on canal side face of wall.
5. Case 3: Retaining walls with top elevation of 31.0 ft. NAVD with no fill placed on outside side face of wall.
6. Case 4: Retaining wall with top elevation of 19.0 ft. NAVD.
7. Case 5: Building foundation wall during construction with backfill to elevation 16.0 ft. NAVD prior to placement of base slab which is to tie to top of wall.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-06-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Stability Analysis

CASE 1: CALCULATIONS - STABILITY ANALYSIS INTAKE:

Top of wall (stem) = el. 10.0 ft. NAVD

Stem thickness = 2.0 ft.

Top of base slab = el. -12.0 f. NAVD

Base slab thickness = 3.0 ft.

Toe extension length = 18.0 ft.

Heel extension length = 8.0 ft.

Key offset = 0.0 ft.

Key depth = 4.0 ft.

Key thickness = 2.0 ft.

Front cover = 3.0 ft.

Center of overturning = 2.0 ft. from face of toe

RESULTS -STABILITY ANALYSIS:

Sliding FS = 2.77

Overturning FS = 8.39

Middle third: yes

Toe bearing pressure = 449 psf

Heel bearing pressure = 2921 psf

Equivalent bearing pressure = 2230 psf

Va/Pf = 0.14

Eccentricity = -3.42 ft.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-06-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Stability Analysis

CASE 2: CALCULATIONS - STABILITY ANALYSIS INTAKE RETAINING WALLS:

Top of wall (stem) = el. 10.0 ft. NAVD

Stem thickness = 2.0 ft.

Top of base slab = el. -12.0 f. NAVD

Base slab thickness = 3.0 ft.

Toe extension length = 10.0 ft.

Heel extension length = 8.0 ft.

Key - none provided

Front cover = 3.0 ft.

Center of over turning at face of toe.

RESULTS -STABILITY ANALYSIS:

Sliding FS = 2.49

Overturning FS = 5.70

Middle third: yes

Toe bearing pressure = 914 psf

Heel bearing pressure = 3152 psf

Equivalent bearing pressure = 2489 psf

Va/Pf = 0.12

Eccentricity = -1.84 ft.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-06-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Stability Analysis

CASE 3: CALCULATIONS - STABILITY ANALYSIS RETAINING WALLS WITH TOP ELEVATION OF 31.0 FT.:

Top of wall (stem) = el. 31.0 ft. NAVD

Stem thickness at top = 1.5 ft.

Stem thickness at bottom = 2.0 ft.

Top of base slab = el. 18.0 ft. NAVD

Base slab thickness = 2.0 ft.

Toe extension length = 2.5 ft.

Heel extension length = 4.0 ft.

Key - none provided

Front cover = 3.0 ft.

Center of over turning at face of toe.

RESULTS -STABILITY ANALYSIS:

Sliding FS = 2.70

Overturning FS = 3.17

Middle third: yes

Toe bearing pressure = 2395 psf

Heel bearing pressure = 1049 psf

Equivalent bearing pressure = 1980 psf

Va/Pf = 0.13

Eccentricity = 0.52 ft.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-06-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Stability Analysis

CASE 4: CALCULATIONS - STABILITY ANALYSIS RETAINING WALL WITH TOP ELEVATION OF 19.0 FT.:

Top of wall (stem) = el. 19.0 ft. NAVD

Stem thickness at top = 1.5 ft.

Stem thickness at bottom = 2.0 ft.

Top of base slab = el. 9.0 ft. NAVD

Base slab thickness = 2.0 ft.

Toe extension length = 4.0 ft.

Heel extension length = 2.0 ft.

Key - none provided

Front cover = 4.0 ft.

Center of over turning at face of toe.

RESULTS -STABILITY ANALYSIS:

Sliding FS = 3.39

Overturning FS = 3.78

Middle third: yes

Toe bearing pressure = 1309 psf

Heel bearing pressure = 1217 psf

Equivalent bearing pressure = 1279 psf

Va/Pf = 0.13

Eccentricity = 0.05 ft.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-06-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Stability Analysis

CASE 5: CALCULATIONS - STABILITY ANALYSIS PUMP HOUSE FOUNDATION WALL DURING CONSTRUCTION BACKFILLING:

Top of wall (stem) = el. 16.0 ft. NAVD

Stem thickness at top = 2.0 ft.

Stem thickness at bottom = 2.0 ft.

Top of base slab = el. 9.0 ft. NAVD

Base slab thickness = 3.0 ft.

Toe extension length = 3.0 ft.

Heel extension length = 3.0 ft.

Key - none provided

Front cover = 4.0 ft.

Center of over turning at face of toe.

RESULTS -STABILITY ANALYSIS:

Sliding FS = 5.43

Overturning FS = 6.92

Middle third: yes

Toe bearing pressure = 994 psf

Heel bearing pressure = 1319 psf

Equivalent bearing pressure = 1213 psf

Va/Pf = 0.09

Eccentricity = -0.19 ft.

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 5-06-06

Project: C43 Reservoir Pump Station PS4
Subject: PRELIMINARY Building Wall Footing Bearing Capacity Analysis

PURPOSE: Provide preliminary estimate of average wall footing bearing pressure and allowable capacity for station walls. The future design phase shall provide a more detailed analysis by the geotechnical engineer.

REFERENCES:

1. Soil Mechanics in Engineering Practice, Terzaghi, Peck.
2. Foundation Design, Coduto.
3. Foundation Engineering, Peck.

DESIGN CRITERIA:

1. The allowable bearing pressures were determined by three methods: Terzaghi, Meyerhof, and Brinch-Hansen
2. Backfill angle = 0 degrees.
3. F.S. = 3.0

ASSUMPTIONS:

1. The wind lateral loads shall be transferred to the base slabs. The wall footings will address only the vertical dead loads of the walls and roof that the wall supports.
2. Soil properties: cohesion = 0; soil friction = 30 degrees; soil unit weight = 114 pcf
3. Depth to groundwater 3.0 ft.

CALCULATIONS:

Footing width = 8.0 ft.

Footing depth = 4.0 ft.

Base inclination = 0

D.L. = wall (7.5 k/lf) + average roof load (1.9 k/lf) = 9.4 k/lf

Estimated bearing pressure = 9.4/8 = 1.2 k/lf

RESULTS:

Allowable Bearing Capacity (psf)

	Terzaghi	Meyerhof	Brinch Hansen
gross	4330	3795	3799
net	4178	3043	3647

Allowable Wall Load (ksf)

	33.4	29.1	27.2

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 4-26-06

Project: C43 Reservoir Pump Station PS4
Subject: Wind Load Estimation

PURPOSE: wind load analysis

REFERENCES:

1. Florida Building Code, Section 1606 - Wind Loads
2. SFWMD Guidelines for Type 2 Water Supply Pump Station.
3. ASCE 7-02, Minimum Design Loads for Buildings and Other Structures.

DESIGN CRITERIA:

1. Wind Velocity: ~120 mph per wind map, ref. 1.
2. 200 yr mean recurrence interval, ref. 2.
3. 50 yr. from 200 yr. recurrence adjustment factor = 1.14, ref. 4.
4. Roof: 0 degrees
5. Mean Roof Height: 24 ft.
6. Importance Factor = 1.15, for water supply pump station, ref. 2.
7. Exposure category C, adjustment factor ~ 1.35, ref. 2
8. Building Dimensions: 46' x 56' ~ rectangular.
9. Enclosed building, no windows, low-rise building, ref.1.

ASSUMPTIONS:

1. Since the pump house is a rectangular box, practically square, there is insignificant difference in wall pressure for a change in wind direction. Therefore the walls were assumed to have similar pressures.

CALCULATIONS:

50 yr. mean recurrence wind velocity (mph) converted to 200 yr. recurrence wind velocity:

120 (ref. 1) x 1.14 = 136.8 mph

Internal pressure coefficient ref. 4:

$K_{zt} = 1.0$

Conversion of basic wind speed to 3 sec. gusts as per fig. 1606.1.6.1, ref. 1:

137 mph ~ 157 mph

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 4-26-06

Project: C43 Reservoir Pump Station PS4
Subject: Wind Load Estimation

Height and exposure adjustments for exposure "C" as per fig. 1606.2D, ref.1 ~ 1.35

Wind pressure table 1606 -2A Main Wind Force Resisting System Wind Loads, ref. 1.

	150 mph conversion to 157 mph		x ~ 1.35 exposure C adjustment	x 1.15 importance factor
Zone 1E:	+27.0	+29.0	=	+45.0
Zone 4E:	- 20.9	- 22.0	=	- 34.2
Zone 1:	+19.9	+21.0	=	+32.6
Zone 4:	- 16.1	- 17.5	=	- 27.2

Edge strip as per ref.1 = 10% x 46 ft. = 4.6 ft.

End zone as per ref.1 = 2 x 4.6 (edge strip) = 9.2ft.

Wind pressure table 1606 -2B Components and Cladding Wind Loads, ref. 1.

	150 mph conversion to 157 mph				x 1.35 exposure C adjustment	x 1.15 importance factor	
Wind area 10 sf							
Zone 4:	+40.5	-43.9	+46.0	-49.0		+71.4	-76.1
Zone 5:	+40.5	-54.2	+45.0	-59.0		+69.9	-91.6
wind area 50 sf							
Zone 1:	+14.1	-38.1	+16.0	-42.0		+24.8	-65.2
Zone 2:	+14.1	-51.1	+16.0	-56.0		+24.8	-86.9
Zone 3:	+14.1	-61.5	+16.0	-66.0		+24.8	-102.5
Wind area 100 sf							
Zone 1	+13.0	-37.0	+15.0	-41.0		+23.3	-63.7
Zone 2:	+13.0	-43.9	+15.0	-49.0		+23.3	-68.2
Zone 4:	+34.4	-37.8	+37.0	-41.0		+57.4	-63.7
Zone 5:	+34.4	-42.1	+37.0	-45.0		+57.4	-69.9

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 4-26-06

Project: C43 Reservoir Pump Station PS4
Subject: Wind Load Estimation

RESULTS:

For the main wind resisting system the design pressure on the walls shall be taken as:

- Interior area of walls: +32.6 psf -27.2 psf
- Wall edges: +45.0 psf -35.2 psf

The net wind pressures calculated for components and cladding:

- Roof Interior +23 -64 psf
- Single doors: +70 psf -92 psf
- Double doors: +71 psf -76 psf
- Roof hatch: +23 psf -64 psf
- Overhead door: +57 psf -70 psf
- Masonry infill panels: +57 psf -64 psf
- Precast roof panels: +23 psf -64 psf
- 6 ft. x 8 ft. louvers +57 psf -70 psf
- 3 ft. x 3 ft. hoods +71 psf -76 psf

POWELL KUGLER INC.

Designer: z.kugler
Date: revised 6-07-06

Project: C43 Reservoir Pump Station PS4
Subject: Stability Analysis - Flotation of Intake Can

PURPOSE: Provide stability analysis in regard to the flotation of the pump's can intake.

REFERENCES:

1. Floway Pump Company Intake Can Design.
2. DET Preliminary Geotechnical Design Memorandum, dated. 5-9-06.

DESIGN CRITERIA:

1. Weight of steel = 490 pcf.
2. Required safety factor = 2.0

ASSUMPTIONS:

1. Normal canal water level 0.0 ft. NAVD, assume water elevation in can.
2. Pump and motor removed from can intake.
3. Assumed groundwater elevation = +17.0 ft. NAVD. ref. 2
4. Intake wall thickness = 0.5 inches
5. Ignore hold down capacity of 48 in. diameter intake pipe.
6. Ignore skin friction of can with soil.
7. Ignore skin friction with base slab.

CALCULATIONS:

Area = 0.785 (d_1^2 - d_2^2) = 0.785 x (25 - 24.59) = 0.33 sf

Can height = 40' - 3" ref. 1

Weight of can = 40.25 x 0.33 x 490 = 6,508 lbs.

Add for base plate = 0.785 (6^2 - 5^2) = 8.635 x 2/12 x 490 = 705 lbs

Add for 48" diameter tee connection ~ 400 lbs

Add for flow vanes ~ 200 lbs

Total can weight = 6508 + 705 + 400 + 200 = 7,813 lbs

Weight of water displaced = (3.14 x 2.5^2) x 17 x 62.4 = 20,818 lbs

20,818 - 7,813 = 13,005 lbs x 2.0 sf = 26,010 lbs anchorage required.

Weight of concrete required = 26,010/135 = 7.13 cy

RESULTS:

Provide anchorage of can to tremie concrete base. Tremie concrete base shall be a minimum of 10 cy.

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S19.dat"

Box Culvert Program:
Box & Headwall Load Cases

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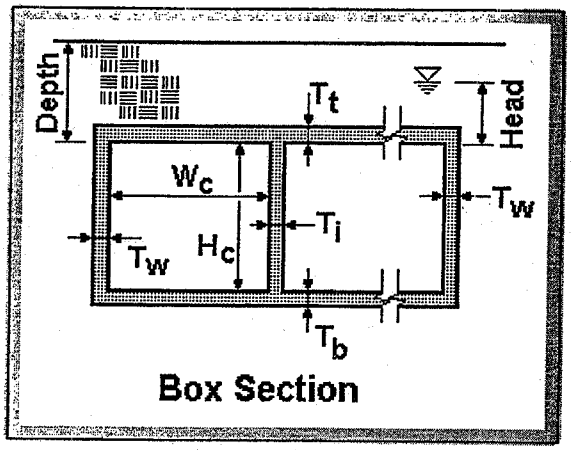
Project = "C43S7"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 10.7$ ft — *opening width of cell*

$H_c = 10$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 164$ ft — *length of culvert along centerline*

NoOfCells = 2 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 3 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$NoOfCells

HydraulicOpening = 214 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 110 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 30$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$BarSize_{long} = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix}$ $S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in$

Change Bar Group (color) values or change individual Bars (white)

$n := BarSize_{long} = XX$

$newS_{long} = XX \cdot in$

$BarSizeD1_{09} := XX \; SD1_{09} := XX \cdot in$

$BarSizeD1_{10} := XX \; SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX \; SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX \; SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX \; SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp.box} := (\text{"ok"} \; \text{"ok"} \; \text{"ok"} \; \text{"ok"} \; \text{"ok"})^T$

$Summary(CheckAs_{temp.box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S19.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs}$ $\quad DataOut_{33} := \frac{S_{slabs}}{in}$ $\quad DataOut_{34} := BarSize_{walls}$ $\quad DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners}$ $\quad DataOut_{37} := \frac{S_{corners}}{in}$ $\quad DataOut_{38} := BarSize_{long}$ $\quad DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$

$DataOut_{90} := ReinfBox$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) =$ "NoGood"

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) =$ "OK"

$CheckAs_{min\,box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min\,box}) =$ "NoGood"

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) =$ "OK"

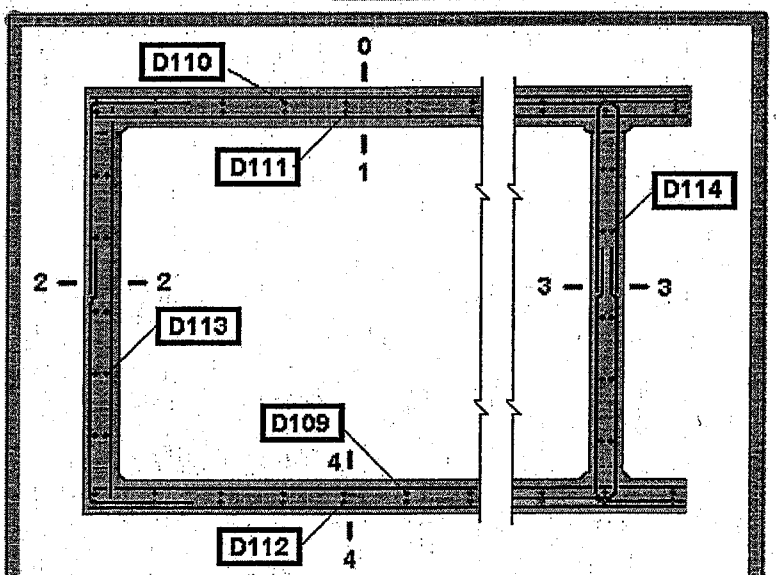
$Summary(CheckAll_{box}) =$ "NoGood"

3. Design and Check Box Longitudinal Reinforcing

CurrentDataFile := "C43S19.dat"

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0..4$ *index for longitudinal sections*



Box Cross Section
Longitudinal Reinf. and Analysis Locations

Material Properties

Env = 3 *Environmental Class* *1-slightly aggressive* *2-moderate* *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

$ConWheel1 = 6.74\,kip$ $ConWheel2 = 7.87\,kip$ $ConWheel3 = 7.87\,kip$ *construction wheel loads*

$ConAxleSpacing1 = 16.4\,ft$ *space between axles 1 and 2*

$ConAxleSpacing2 = 16.4\,ft$ *space between axles 2 and 3*

2. Design and Check Main Reinforcing

[redacted]

$it := 0..20$ *index for transverse sections*

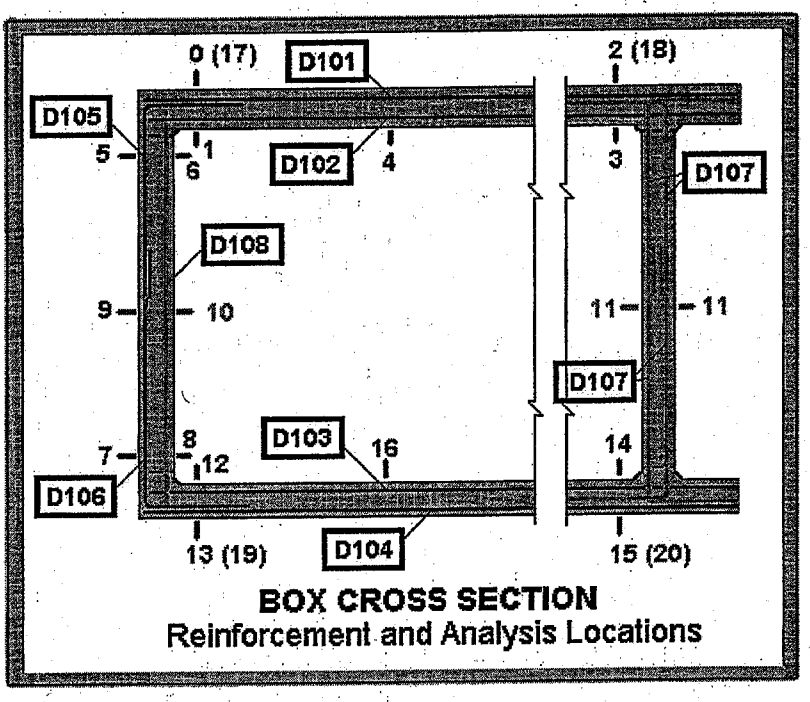
$BarSize_{box_{it}} := 0$ $S_{box_{it}} := 12 \cdot in$

Cross Section Notes:
1. Bars shown with the same color are combined as Bar Groups.
2. Colored numbers indicate moment analysis locations.
3. Colored numbers in parenthesis indicate shear analysis locations.
4. Black numbers refer to bar designations.

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number) *Bar Sizes & Spacings used in analysis* *Change Bar Group values (color) or change individual Bars (white)*

D101(2)

D102(1,3,4)

D103(12,14,16)

D104(15)

$BarSize_{slabs} = \begin{pmatrix} 7 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 12 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

[redacted]

D105(9,0,5)

D106(9,7,13)

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

[redacted]

D107(11)

D108(6,8,10)

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

[redacted]

$BarSizeD1_{01} := XX$ $SD1_{01} := XX \cdot in$

$BarSizeD1_{02} := XX$ $SD1_{02} := XX \cdot in$

$BarSizeD1_{03} := XX$ $SD1_{03} := XX \cdot in$

$BarSizeD1_{04} := XX$ $SD1_{04} := XX \cdot in$

$BarSizeD1_{05} := XX$ $SD1_{05} := XX \cdot in$

$BarSizeD1_{06} := XX$ $SD1_{06} := XX \cdot in$

$BarSizeD1_{07} := XX$ $SD1_{07} := XX \cdot in$

$BarSizeD1_{08} := XX$ $SD1_{08} := XX \cdot in$

Box Culvert Program:
Box Reinforcement Design

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Project = "C43S7"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile = "C43S19.dat"

Geometry and Box Dimensions

$W_c = 10.7$ ft *opening width of cell*

$H_c = 10$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

$L_c = 164$ ft *length of culvert along centerline*

Depth = 3 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

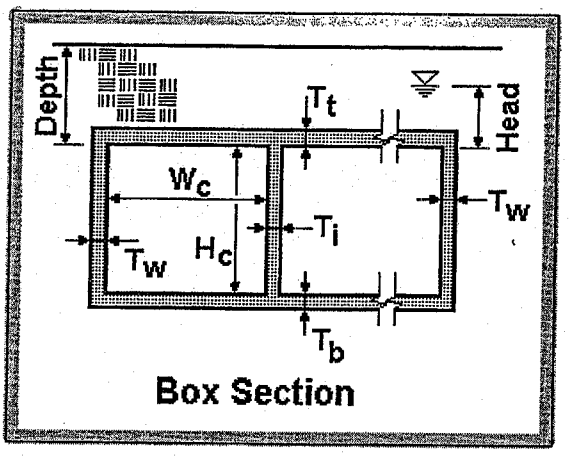
NoOfCells = 2 *number of cells*

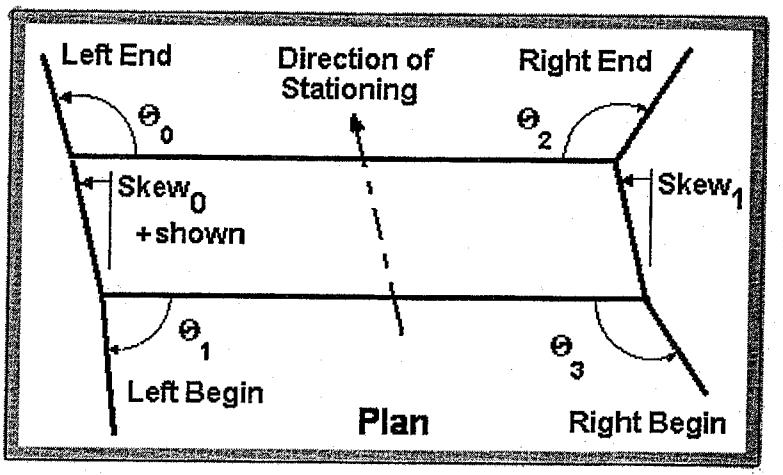
Head = 0 ft *water head at top of box (typically = 0.0)*

$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 214 ft^2





Soil Properties

$\gamma = 110 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 30$ deg *soil friction angle*

$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$

$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$

$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$

$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$

$Strength_{hw} = \begin{pmatrix} 37.17 & 25.33 & 68.18 & 72.59 & 45.3 \\ 37.17 & 25.33 & 68.18 & 72.59 & 45.3 \end{pmatrix}$

$Service_{hw} = \begin{pmatrix} 22.5 & 16.58 & 43.18 & 43.8 & 27.99 \\ 22.5 & 16.58 & 43.18 & 43.8 & 27.99 \end{pmatrix}$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S19.dat"

DataOut := DataIn

$DataOut_{31} := \frac{L_0}{ft}$

$DataOut_{85} := Strength_{box}$

$DataOut_{86} := Service_{box}$

$DataOut_{87} := Strength_{hw}$

$DataOut_{88} := Service_{hw}$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

$iw = \begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$ $iw = \begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $H_{hw} = \begin{pmatrix} 42 \\ 42 \end{pmatrix} in$ $Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix} deg$ *left* *right*

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ *Additional dead load if a barrier is located on top of the headwall*

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0..(nc-1)$	*index for cells*	$NoOfCells = 2$	*number of cells*	$f_c = 5.5 ksi$	
$W_c = 10.7 ft$	*opening width of cell*	$H_c = 10 ft$	*opening height of cell*	$T_i = 12 in$	*interior wall thickness*
$T_t = 12 in$	*top slab thickness*	$T_w = 12 in$	*exterior wall thickness*	$Env = 3$	*Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*
$E = 4200 ksi$	*concrete modulus of elasticity*	$\gamma = 110 \frac{lbf}{ft^3}$	*soil density*		
$SoilHeight = 2 ft$	*soil height above top slab*	$\gamma_w = 62.4 \frac{lbf}{ft^3}$	*weight of water*	$n_{mod} = 9$	*ratio of steel and concrete modulus of elasticity*
$\gamma_{conc} = 150 \frac{lbf}{ft^3}$	*concrete unit weight*	$F_y = 60 ksi$			

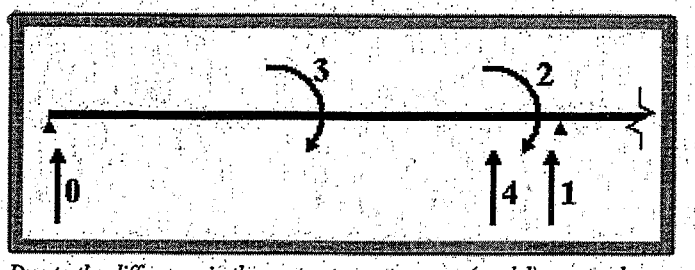
Calculate load combinations

Strength

$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$

Service

$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$Strength_{hw} = \begin{pmatrix} 37.17 & 25.33 & 76.63 & 76.98 & 45.3 \\ 37.17 & 25.33 & 76.63 & 76.98 & 45.3 \end{pmatrix}$ $Service_{hw} = \begin{pmatrix} 22.5 & 16.58 & 48.71 & 46.45 & 27.99 \\ 22.5 & 16.58 & 48.71 & 46.45 & 27.99 \end{pmatrix}$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8
0	12.3	2.77	23.81	-1.49	21.7	10.95	3.06	18.09	-2.83
1	7.8	1.45	9.33	0.61	0	5.96	0	-5.21	0
2	0.67	0	0	0	0	7.8	1.45	14.02	2.8

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.45	0.93	14.89	0	14.77	6.57	1.18	11.73	0	8.82
1	4.92	1.3	5.96	0.91	0	3.97	0	-3.69	0	0
2	0.38	0	0	0	0	4.92	1.3	8.94	4.07	8.57

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	12.3	2.77	23.81	-1.49	21.7	10.95	3.06	18.09	-2.83	13.48

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.45	0.93	14.89	0	14.77	6.57	1.18	11.73	0	8.82

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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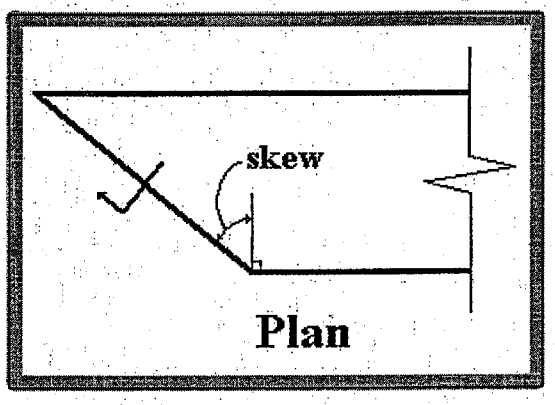
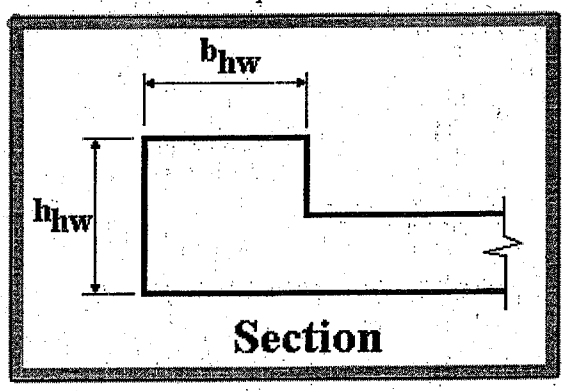
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad iwend := if(Extension \neq 1, 1, 0)$ $\quad iw := iwbeg..iwend$ *index for walls*

WO No. C-C10401P-WO No. 9
C-43 West Storage Reservoir

Project No.: _____ Page ___ of ___
Calculated By: WRK Date: 5/06
Checked: [illegible] Date: 5/06

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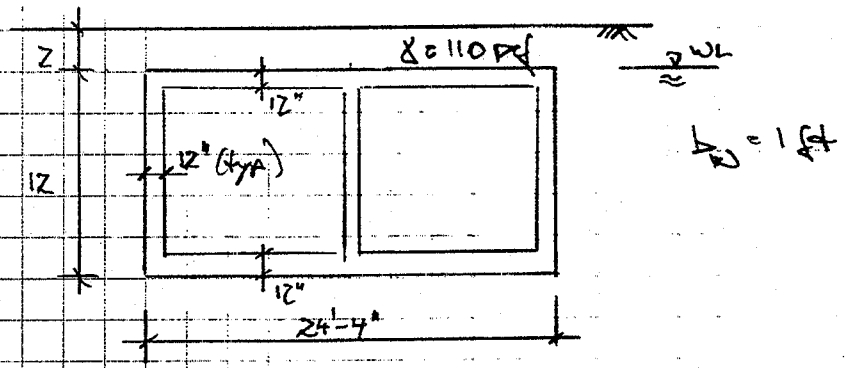
Flotation check C43S7



Weight of structure and surcharge:

$W_{st} = ((24.33 \times 12 - 2(10.67 \times 10))\,150 = 11784$ lbs

$W_{sur} = 24.33 \times 2 \times 110 = 5353$ lbs

$W_{tot} = 17137$ lbs

Uplift:

$U = 24.33 \times 12 \times 62.4 = 18218$ lbs

Flotation Safety Factor:

$SF_{fl} < 1.1$

Verify at 60% design whether water level assumed at top of structure for extreme maintenance condition is justified or can be lowered. Structure geometry may change.

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$$BarSize_{long} = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix} \qquad S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} in$$

Change Bar Group (color) values or change individual Bars (white)

$BarSizeD1_{09} := XX$ $SD1_{09} := XX \cdot in$

[redacted]

$BarSizeD1_{10} := XX$ $SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX$ $SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX$ $SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX$ $SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp\ box}{}^{T} = (\text{"ok"} \quad \text{"ok"} \quad \text{"ok"} \quad \text{"ok"} \quad \text{"ok"})$

$Summary(CheckAs_{temp\ box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S19.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs}$ $\quad DataOut_{33} := \frac{S_{slabs}}{in}$ $\quad DataOut_{34} := BarSize_{walls}$ $\quad DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners}$ $\quad DataOut_{37} := \frac{S_{corners}}{in}$ $\quad DataOut_{38} := BarSize_{long}$ $\quad DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$

$DataOut_{90} := ReinfBox$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) = \text{"NoGood"}$

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) = \text{"OK"}$

$CheckAs_{min.box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min\,box}) = \text{"NoGood"}$

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) = \text{"OK"}$

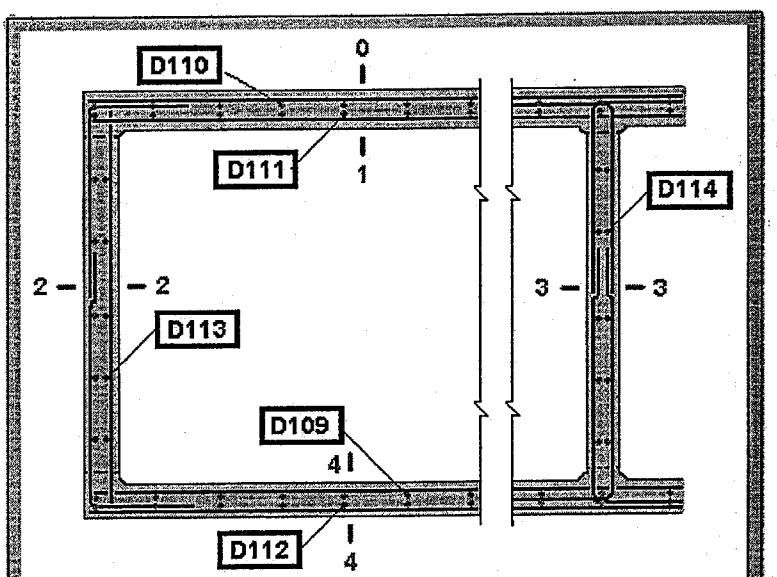
$Summary(CheckAll_{box}) = \text{"NoGood"}$

3. Design and Check Box Longitudinal Reinforcing

$CurrentDataFile := \text{"C43S19.dat"}$

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0..4$ *index for longitudinal sections*



Box Cross Section
Longitudinal Reinf. and Analysis Locations

Material Properties

Env = 3 *Environmental Class* *1-slightly aggressive* *2-moderate* *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

ConWheel1 = 6.74 kip ConWheel2 = 7.87 kip ConWheel3 = 7.87 kip *construction wheel loads*

ConAxleSpacing1 = 16.4 ft *space between axles 1 and 2*

ConAxleSpacing2 = 16.4 ft *space between axles 2 and 3*

2. Design and Check Main Reinforcing

CurrentDataFile := "C43S19.dat"

it := 0 .. 20 *index for transverse sections*

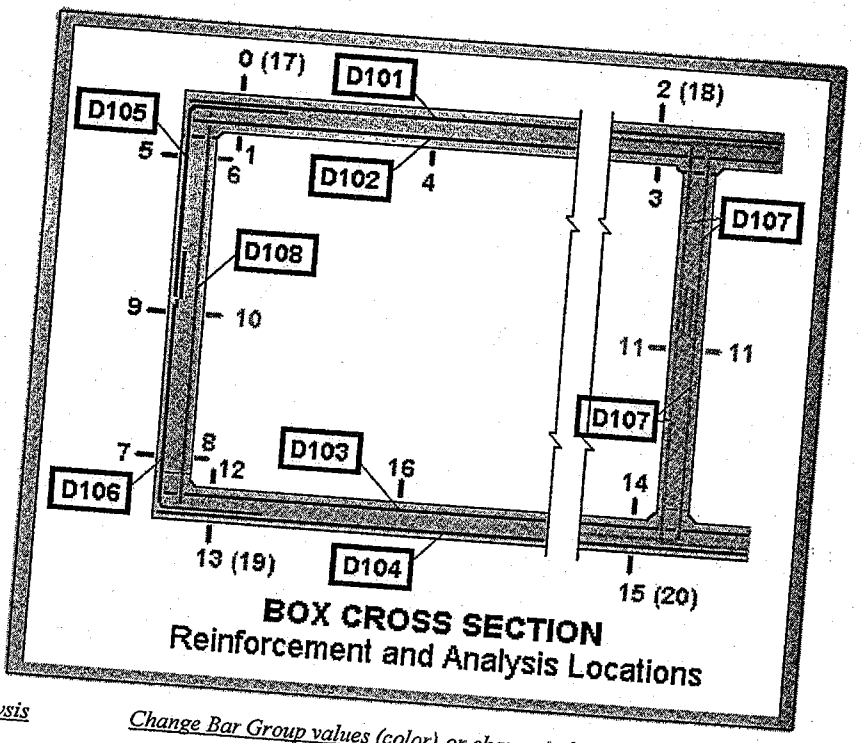
$BarSize_{box_{it}} := 0$ $S_{box_{it}} := 12 \cdot in$

Cross Section Notes:
1. *Bars shown with the same color are combined as Bar Groups.*
2. *Colored numbers indicate moment analysis locations.*
3. *Colored numbers in parenthesis indicate shear analysis locations.*
4. *Black numbers refer to bar designations.*

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number)

D101(2)

D102(1,3,4)

D103(12,14,16)

D104(15)

D105(9,0,5)

D106(9,7,13)

D107(11)

D108(6,8,10)

Bar Sizes & Spacings used in analysis

$BarSize_{slabs} = \begin{pmatrix} 7 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 12 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

Change Bar Group values (color) or change individual Bars (white)

$newBarSize_{slabs} := XX$

$newS_{slabs} := XX \cdot in$

$newBarSize_{corners} := XX$

$newS_{corners} := XX \cdot in$

$newBarSize_{walls} := XX$

$newS_{walls} := XX \cdot in$

$BarSizeD1_{01} := XX$ $SD1_{01} := XX \cdot in$

$BarSizeD1_{02} := XX$ $SD1_{02} := XX \cdot in$

$BarSizeD1_{03} := XX$ $SD1_{03} := XX \cdot in$

$BarSizeD1_{04} := XX$ $SD1_{04} := XX \cdot in$

$BarSizeD1_{05} := XX$ $SD1_{05} := XX \cdot in$

$BarSizeD1_{06} := XX$ $SD1_{06} := XX \cdot in$

$BarSizeD1_{07} := XX$ $SD1_{07} := XX \cdot in$

$BarSizeD1_{08} := XX$ $SD1_{08} := XX \cdot in$

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

Box Culvert Program:
Box Reinforcement Design

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Project = "C43S9"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

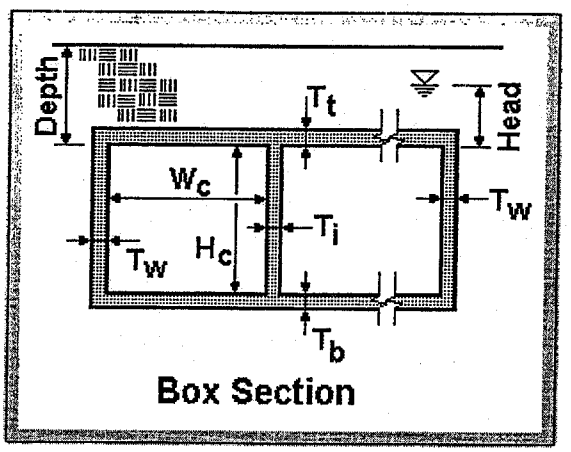
Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile = "C43S19.dat"

Geometry and Box Dimensions



$W_c = 6.7$ ft *opening width of cell*

$H_c = 6$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

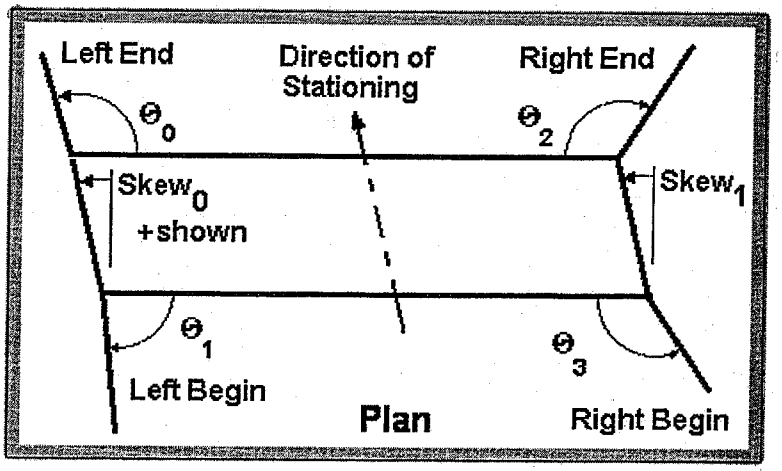
$L_c = 62$ ft *length of culvert along centerline*

Depth = 3 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

NoOfCells = 2 *number of cells*

Head = 0 ft *water head at top of box (typically = 0.0)*



$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg

$HydraulicOpening := W_c \cdot H_c \cdot NoOfCells$

$HydraulicOpening = 80.4\ ft^2$

Soil Properties

$\gamma = 105 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 29$ deg *soil friction angle*

$$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Strength_{hw} = \begin{pmatrix} 29.41 & 25.14 & 27.15 & 40.85 & 36.48 \\ 29.41 & 25.14 & 27.15 & 40.85 & 36.48 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 17.48 & 15.49 & 16.87 & 24.13 & 21.97 \\ 17.48 & 15.49 & 16.87 & 24.13 & 21.97 \end{pmatrix}$$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S19.dat"

DataOut := DataIn

$$DataOut_{31} := \frac{L_0}{ft}$$

$$DataOut_{85} := Strength_{box}$$

$$DataOut_{86} := Service_{box}$$

$$DataOut_{87} := Strength_{hw}$$

$$DataOut_{88} := Service_{hw}$$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

iw = $\begin{pmatrix} 0 \\ 1 \end{pmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix}$ in

iw = $\begin{pmatrix} 0 \\ 1 \end{pmatrix}$ $H_{hw} = \begin{pmatrix} 36 \\ 36 \end{pmatrix}$ in

$Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg *left* *right*

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ *Additional dead load if a barrier is located on top of the headwall*

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0..(nc-1)$ *index for cells*

$NoOfCells = 2$ *number of cells*

$f_c = 5.5\,ksi$

$W_c = 6.7\,ft$ *opening width of cell*

$H_c = 6\,ft$ *opening height of cell*

$T_i = 12\,in$ *interior wall thickness*

$T_t = 12\,in$ *top slab thickness*

$T_w = 12\,in$ *exterior wall thickness*

$Env = 3$ *Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$\gamma = 105 \frac{lbf}{ft^3}$ *soil density*

$SoilHeight = 2\,ft$ *soil height above top slab*

$\gamma_w = 62.4 \frac{lbf}{ft^3}$ *weight of water*

$n_{mod} = 9$ *ratio of steel and concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ *concrete unit weight*

$F_y = 60\,ksi$

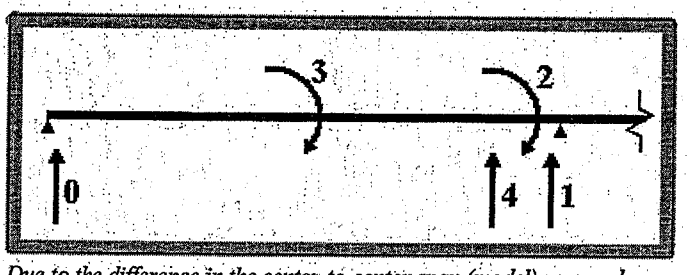
Calculate load combinations

Strength

$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$

Service

$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$Strength_{hw} = \begin{pmatrix} 29.41 & 25.14 & 35.53 & 44.72 & 36.48 \\ 29.41 & 25.14 & 35.53 & 44.72 & 36.48 \end{pmatrix}$

$Service_{hw} = \begin{pmatrix} 17.48 & 15.49 & 22.04 & 26.42 & 21.97 \\ 17.48 & 15.49 & 22.04 & 26.42 & 21.97 \end{pmatrix}$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	6.12	2.82	13.74	-0.97	12.77	5.39	2.97	11.25	-0.61	7.77
1	6.93	6.46	5.93	0.49	0	3.49	0	-1.36	0	0
2	0.78	0	0	0	0	6.93	6.46	10	4.08	9.67

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	3.57	1.36	8.37	0	7.98	3.12	1.49	6.95	0	4.8
1	4.24	3.98	3.81	0.68	0	2.21	0	-1.12	0	0
2	0.45	0	0	0	0	4.24	3.98	6.29	2.92	6.12

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	6.12	2.82	13.74	-0.97	12.77	5.39	2.97	11.25	-0.61	7.77

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	3.57	1.36	8.37	0	7.98	3.12	1.49	6.95	0	4.8

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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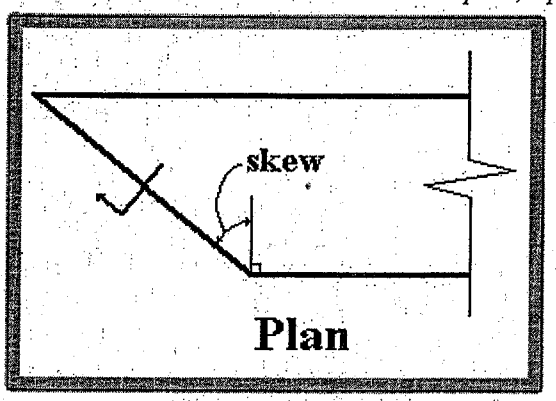
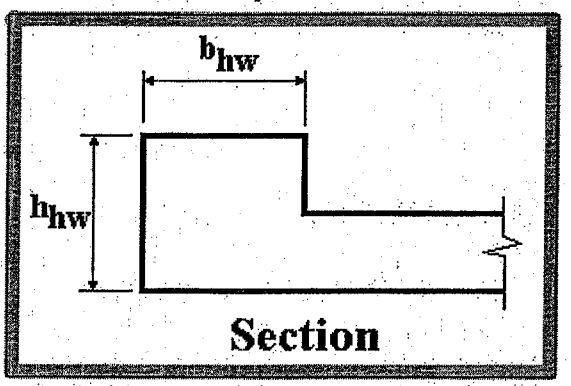
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad iwend := if(Extension \neq 1, 1, 0)$ $\quad iw := iwbeg .. iwend$ $\quad$ *index for walls*

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S19.dat"

Box Culvert Program:
Box & Headwall Load Cases

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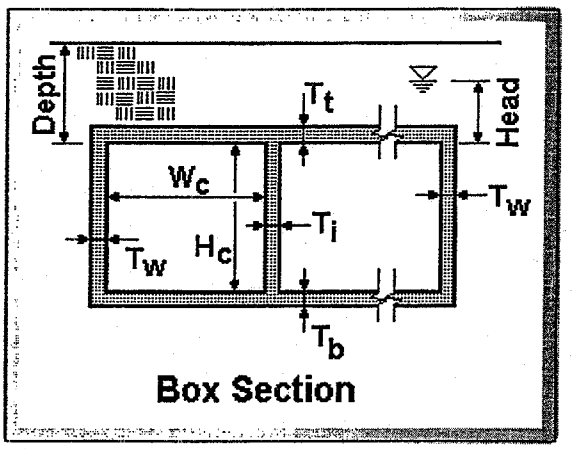
Project = "C43S9"
DesignedBy = "WBK"
CheckedBy = "JB"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 6.7$ ft — *opening width of cell*

$H_c = 6$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 62$ ft — *length of culvert along centerline*

NoOfCells = 2 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 3 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$NoOfCells

HydraulicOpening = 80.4 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 105 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 29$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

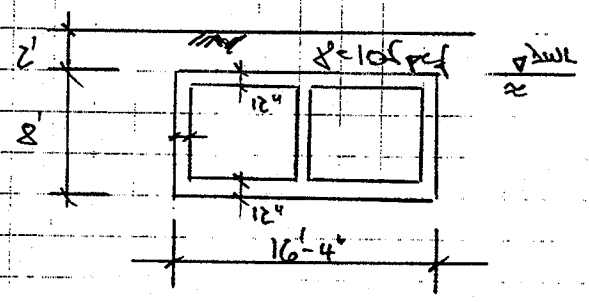
$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

R.C.T. ENGINEERING, INC.
CONSULTING ENGINEERS • PROGRAM MANAGERS

WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir
Project No.: _____ Page ___ of ___
Calculated By: WSK Date: 5/06
Checked: [illegible] Date: 5/06

Flotation check C4359:



$h_w = 1$ ft

Weight of structure and surcharge:

$W_{st} = ((16.33 \times 8 - 2(6.67 \times 6))150 = 7594$ lbs

$W_{surf} = 16.33 \times 2 \times 105 = 3429$ lbs

$W_{tot} = 11023$ lbs

Uplift:

$U = 16.33 \times 8 \times 62.4 = 8152$ lbs

Flotation Safety Factor:

$SF_H = \frac{11023}{8152} = 1.35 > 1.1$ for extreme maintenance case

701 Northpoint Parkway, Suite 310, West Palm Beach, FL 33407 Phone: 561 684 7534 Fax: 561 684 2801 RCT-WPB@RCTEngineering.com www.RCTEngineering.com

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$$BarSize_{long} = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix} \qquad S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} in$$

Change Bar Group (color) values or change individual Bars (white)

[redacted]

$BarSizeD1_{09} := XX \quad SD1_{09} := XX \cdot in$

$BarSizeD1_{10} := XX \quad SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX \quad SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX \quad SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX \quad SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp.box}^{T} = (\text{"ok"} \ \text{"ok"} \ \text{"ok"} \ \text{"ok"} \ \text{"ok"})$

$Summary(CheckAs_{temp.box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S19.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs}$ $\quad DataOut_{33} := \frac{S_{slabs}}{in}$ $\quad DataOut_{34} := BarSize_{walls}$ $\quad DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners}$ $\quad DataOut_{37} := \frac{S_{corners}}{in}$ $\quad DataOut_{38} := BarSize_{long}$ $\quad DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$

$DataOut_{90} := ReinfBox$

$WRITEPRN(CurrentDataFile) := DataOut$

$WRITEPRN(NewDataFile) := DataOut$

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) = "NoGood"$

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) = "OK"$

$CheckAs_{min.box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min.box}) = "NoGood"$

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) = "OK"$

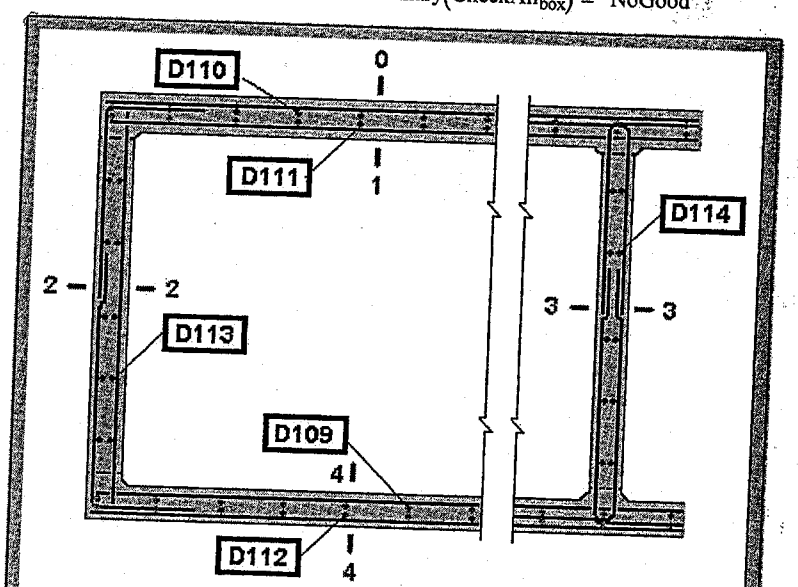
$Summary(CheckAll_{box}) = "NoGood"$

3. Design and Check Box Longitudinal Reinforcing

CurrentDataFile := "C43S19.dat"

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0..4$ *index for longitudinal sections*



Box Cross Section
Longitudinal Reinf. and Analysis Locations

Material Properties

Env = 3 *Environmental Class* *1-slightly aggressive* *2-moderate* *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

ConWheel1 = 6.74 kip ConWheel2 = 7.87 kip ConWheel3 = 7.87 kip *construction wheel loads*

ConAxleSpacing1 = 16.4 ft *space between axles 1 and 2*

ConAxleSpacing2 = 16.4 ft *space between axles 2 and 3*

2. Design and Check Main Reinforcing

CurrentDataFile := "C43St9.dat"

it := 0 .. 20 *index for transverse sections*

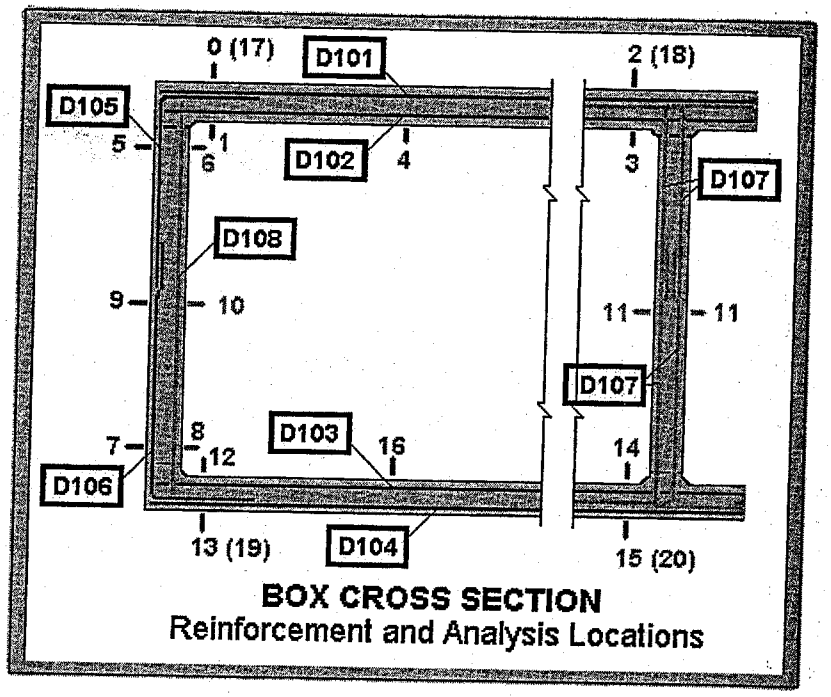
$BarSize_{box_{it}} := 0$ $S_{box_{it}} := 12 \cdot in$

Cross Section Notes:
1. Bars shown with the same color are combined as Bar Groups.
2. Colored numbers indicate moment analysis locations.
3. Colored numbers in parenthesis indicate shear analysis locations.
4. Black numbers refer to bar designations.

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number)

D101(2)
D102(1,3,4)
D103(12,14,16)
D104(15)
D105(9,0,5)
D106(9,7,13)
D107(11)
D108(6,8,10)

Bar Sizes & Spacings used in analysis

$BarSize_{slabs} = \begin{pmatrix} 7 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 12 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

Change Bar Group values (color) or change individual Bars (white)

$newBarSize_{slabs} := XX$

$newS_{slabs} := XX \cdot in$

$newBarSize_{corners} := XX$

$newS_{corners} := XX \cdot in$

$newBarSize_{walls} := XX$

$newS_{walls} := XX \cdot in$

$BarSizeD1_{01} := XX$ $SD1_{01} := XX \cdot in$

$BarSizeD1_{02} := XX$ $SD1_{02} := XX \cdot in$

$BarSizeD1_{03} := XX$ $SD1_{03} := XX \cdot in$

$BarSizeD1_{04} := XX$ $SD1_{04} := XX \cdot in$

$BarSizeD1_{05} := XX$ $SD1_{05} := XX \cdot in$

$BarSizeD1_{06} := XX$ $SD1_{06} := XX \cdot in$

$BarSizeD1_{07} := XX$ $SD1_{07} := XX \cdot in$

$BarSizeD1_{08} := XX$ $SD1_{08} := XX \cdot in$

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

Box Culvert Program:
Box Reinforcement Design

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Project = "C43S14"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

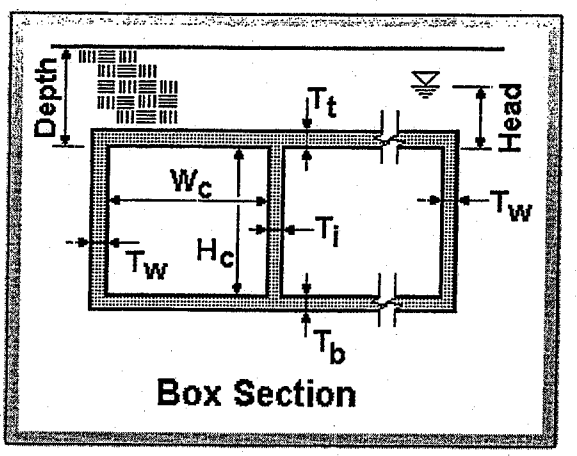
Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile := "C43S19.dat"

Geometry and Box Dimensions



$W_c = 10$ ft *opening width of cell*

$H_c = 6$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

$L_c = 39$ ft *length of culvert along centerline*

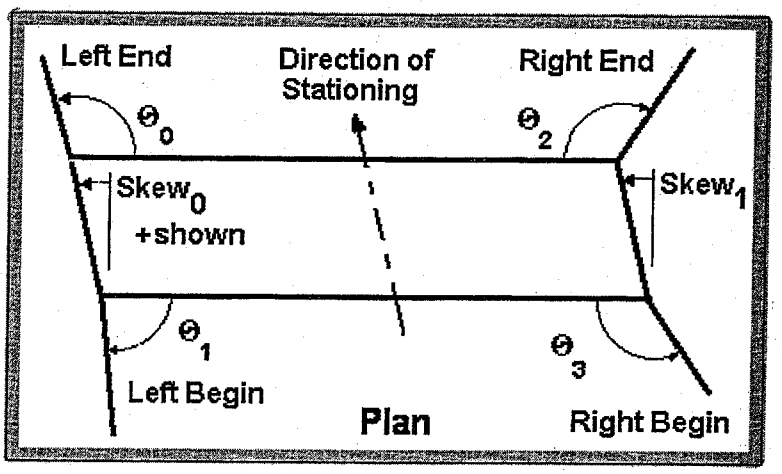
Depth = 7 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

NoOfCells = 1 *number of cells*

Head = 0 ft *water head at top of box (typically = 0.0)*

$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg



HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 60 ft^2

Soil Properties

$\gamma = 110 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 30$ deg *soil friction angle*

$$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Strength_{hw} = \begin{pmatrix} 46.95 & 40.82 & -13.61 & 105.56 & 46.95 \\ 46.95 & 40.82 & -13.61 & 105.56 & 46.95 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 29.8 & 26.3 & -8.77 & 68.01 & 29.8 \\ 29.8 & 26.3 & -8.77 & 68.01 & 29.8 \end{pmatrix}$$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S19.dat"

DataOut := DataIn

$$DataOut_{31} := \frac{L_0}{ft}$$

$$DataOut_{85} := Strength_{box}$$

$$DataOut_{86} := Service_{box}$$

$$DataOut_{87} := Strength_{hw}$$

$$DataOut_{88} := Service_{hw}$$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix}$ in

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $H_{hw} = \begin{pmatrix} 90 \\ 90 \end{pmatrix}$ in

$Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg *left* *right*

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ ***Additional dead load if a barrier is located on top of the headwall***

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0..(nc-1)$	*index for cells*	$NoOfCells = 1$	*number of cells*	$f_c = 5.5\,ksi$	
$W_c = 10\,ft$	*opening width of cell*	$H_c = 6\,ft$	*opening height of cell*	$T_i = 12\,in$	*interior wall thickness*
$T_t = 12\,in$	*top slab thickness*	$T_w = 12\,in$	*exterior wall thickness*	$Env = 3$	*Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*
$E = 4200\,ksi$	*concrete modulus of elasticity*	$\gamma = 110 \frac{lbf}{ft^3}$	*soil density*		
$SoilHeight = 6\,ft$	*soil height above top slab*	$\gamma_w = 62.4 \frac{lbf}{ft^3}$	*weight of water*	$n_{mod} = 9$	*ratio of steel and concrete modulus of elasticity*
$\gamma_{conc} = 150 \frac{lbf}{ft^3}$	*concrete unit weight*	$F_y = 60\,ksi$			

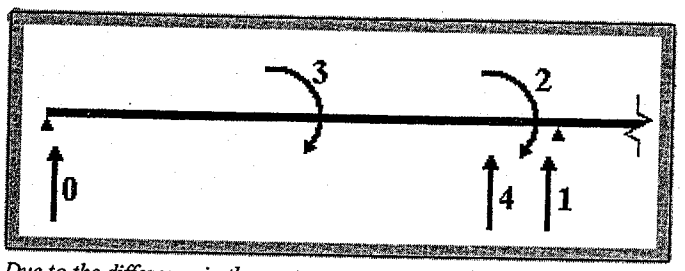
Calculate load combinations

Strength

$$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$

Service

$$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$$Strength_{hw} = \begin{pmatrix} 46.95 & 40.82 & 2.91 \times 10^{-14} & 112.37 & 46.95 \\ 46.95 & 40.82 & 2.91 \times 10^{-14} & 112.37 & 46.95 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 29.8 & 26.3 & 1.76 \times 10^{-14} & 72.39 & 29.8 \\ 29.8 & 26.3 & 1.76 \times 10^{-14} & 72.39 & 29.8 \end{pmatrix}$$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	9.82	-0.8	9.82	-0.8	15.96	10.4	-0.8	13.24	-2.76	12.62
1	10.59	1.04	10.59	1.04	0	3.9	0	-4.22	0	0
2	0.06	0	0.06	0	0	10.59	1.04	11.91	2.34	11.91

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	6.61	0	6.61	0	10.97	7.17	0	9.21	0	8.27
1	7.2	0.82	7.2	0.82	0	2.57	0	-2.85	0	0
2	0.03	0	0.03	0	0	7.2	0.82	8.25	1.88	8.25

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	9.82	-0.8	9.82	-0.8	15.96	10.4	-0.8	13.24	-2.76	12.62

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	6.61	0	6.61	0	10.97	7.17	0	9.21	0	8.27

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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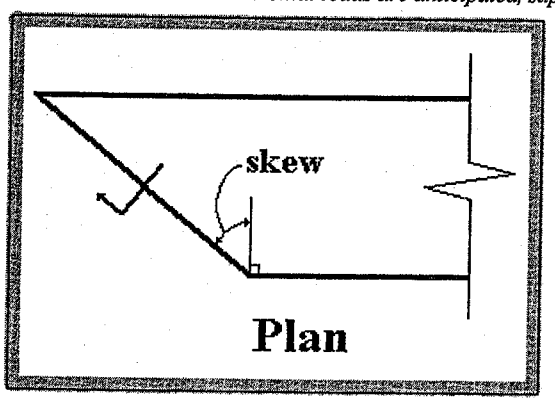
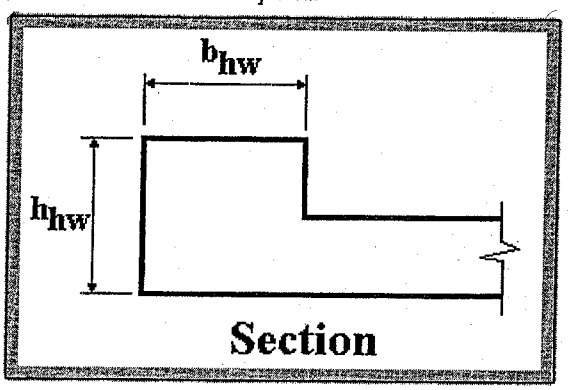
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad iwend := if(Extension \neq 1, 1, 0)$ $\quad iw := iwbeg .. iwend$ $\quad$ *index for walls*

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S19.dat"

Box Culvert Program:
Box & Headwall Load Cases

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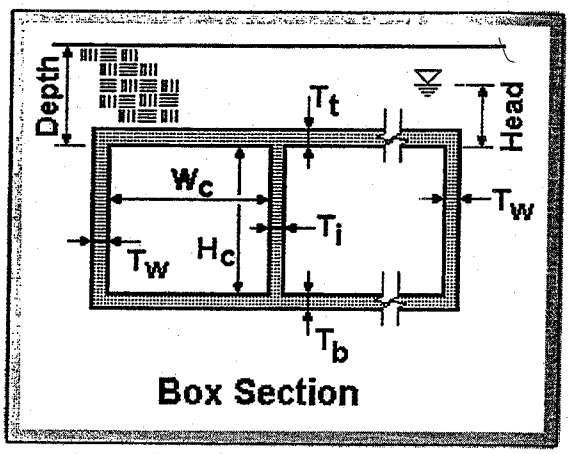
Project = "C43S14"
DesignedBy = "WBK"
CheckedBy = "JB"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 10$ ft — *opening width of cell*

$H_c = 6$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 39$ ft — *length of culvert along centerline*

NoOfCells = 1 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 7 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$NoOfCells

HydraulicOpening = 60 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 110 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 30$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

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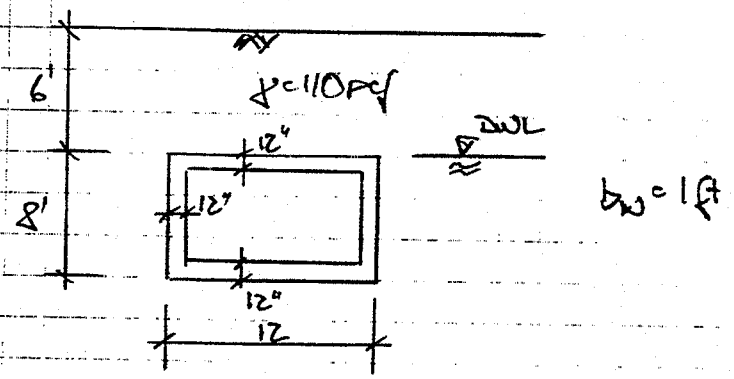
Flotation check C43S14:



$b_w = 1\,ft$

Weight of structure and surcharge:

$W_{cc} = (12 \times 8 - 10 \times 6)\,150 = 5400$ lbs

$W_{sur} = 12 \times 6 \times 110 = 7920$ lbs

$W_{tot} = 13320$ lbs

Uplift:

$U = 12 \times 8 \times 62.4 = 5990$ lbs

Flotation Safety Factor:

$SF_{Fl} = \frac{13320}{5990} = 2.2 \gg 1.1$ Extr. Maint. Case

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$$BarSize_{long} = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix} \qquad S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} in$$

Change Bar Group (color) values or change individual Bars (white)

$BarSizeD1_{09} := XX$ $SD1_{09} := XX \cdot in$

$BarSizeD1_{10} := XX$ $SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX$ $SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX$ $SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX$ $SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp.box}^{T} = (\text{"ok"}\ \text{"ok"}\ \text{"ok"}\ \text{"ok"}\ \text{"ok"})$

$Summary(CheckAs_{temp.box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S19.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs}$ $\quad DataOut_{33} := \frac{S_{slabs}}{in}$ $\quad DataOut_{34} := BarSize_{walls}$ $\quad DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners}$ $\quad DataOut_{37} := \frac{S_{corners}}{in}$ $\quad DataOut_{38} := BarSize_{long}$ $\quad DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$

$DataOut_{90} := ReinfBox$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) = "NoGood"$

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) = "OK"$

$CheckAs_{min\,box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min\,box}) = "NoGood"$

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) = "OK"$

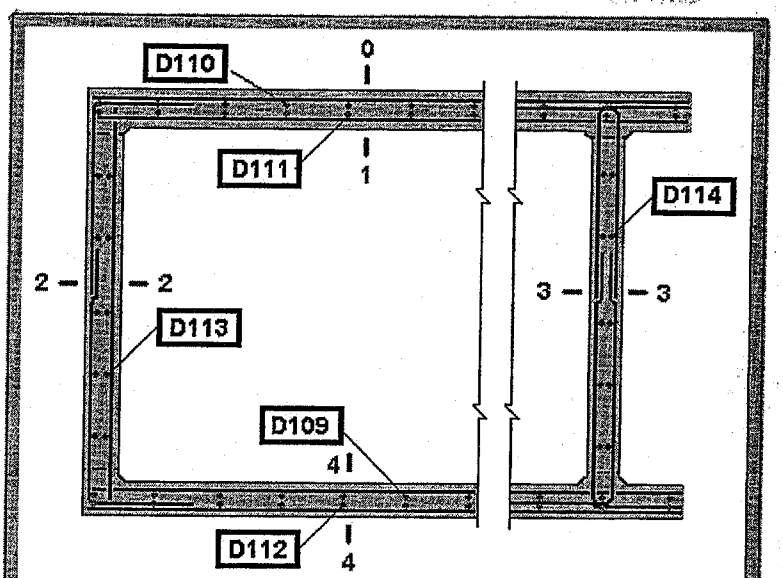
$Summary(CheckAll_{box}) = "NoGood"$

3. Design and Check Box Longitudinal Reinforcing

CurrentDataFile = "C43S19.dat"

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0 .. 4$ *index for longitudinal sections*



Box Cross Section
Longitudinal Reinf. and Analysis Locations

Material Properties

Env = 3 — *Environmental Class*: *1-slightly aggressive*, *2-moderate*, *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

ConWheel1 = 6.74 kip ConWheel2 = 7.87 kip ConWheel3 = 7.87 kip *construction wheel loads*

ConAxleSpacing1 = 16.4 ft *space between axles 1 and 2*

ConAxleSpacing2 = 16.4 ft *space between axles 2 and 3*

2. Design and Check Main Reinforcing

CurrentDataFile := "C43S19.dat"

$it := 0..20$ *index for transverse sections*

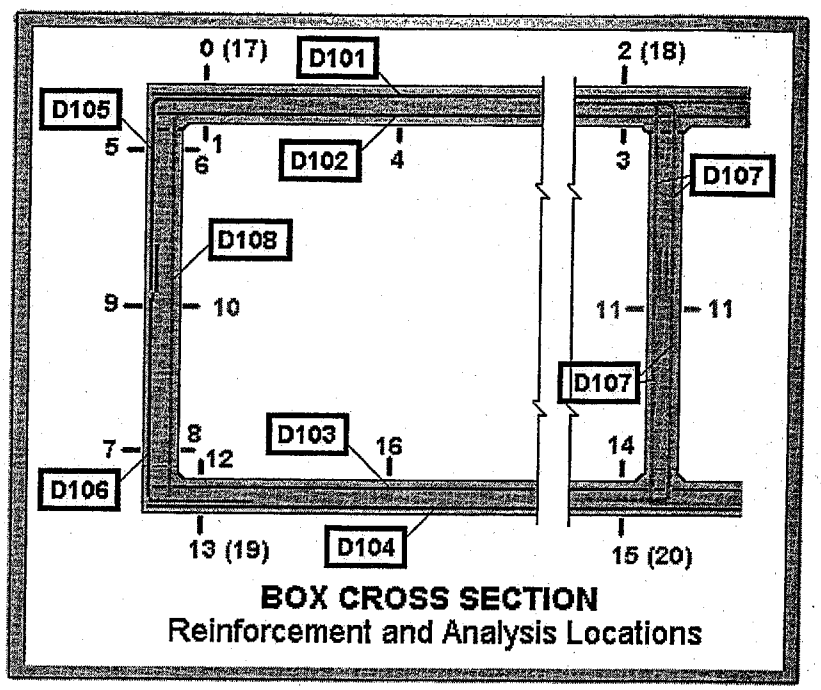
$BarSize_{box_{it}} := 0 \quad S_{box_{it}} := 12 \cdot in$

Cross Section Notes:
1. Bars shown with the same color are combined as Bar Groups.
2. Colored numbers indicate moment analysis locations.
3. Colored numbers in parenthesis indicate shear analysis locations.
4. Black numbers refer to bar designations.

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number) — *Bar Sizes & Spacings used in analysis* — *Change Bar Group values (color) or change individual Bars (white)*

D101(2)
D102(1,3,4)
D103(12,14,16)
D104(15)

$BarSize_{slabs} = \begin{pmatrix} 7 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 12 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

$newBarSize_{slabs} := XX$

$newS_{slabs} := XX \cdot in$

D105(9,0,5)
D106(9,7,13)
D107(11)

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

$newBarSize_{corners} := XX$

$newS_{corners} := XX \cdot in$

D108(6,8,10)

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

$newBarSize_{walls} := XX$

$newS_{walls} := XX \cdot in$

$BarSizeD1_{01} := XX \quad SD1_{01} := XX \cdot in$

$BarSizeD1_{02} := XX \quad SD1_{02} := XX \cdot in$

$BarSizeD1_{03} := XX \quad SD1_{03} := XX \cdot in$

$BarSizeD1_{04} := XX \quad SD1_{04} := XX \cdot in$

$BarSizeD1_{05} := XX \quad SD1_{05} := XX \cdot in$

$BarSizeD1_{06} := XX \quad SD1_{06} := XX \cdot in$

$BarSizeD1_{07} := XX \quad SD1_{07} := XX \cdot in$

$BarSizeD1_{08} := XX \quad SD1_{08} := XX \cdot in$

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir

Box Culvert Program:
Box Reinforcement Design

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Project = "C43S15"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile = "C43S19.dat"

Geometry and Box Dimensions

$W_c = 10.7$ ft *opening width of cell*

$H_c = 8$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

$L_c = 45$ ft *length of culvert along centerline*

Depth = 4 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

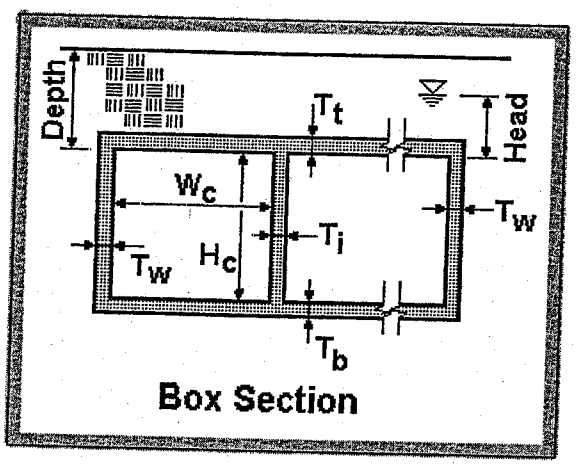
NoOfCells = 2 *number of cells*

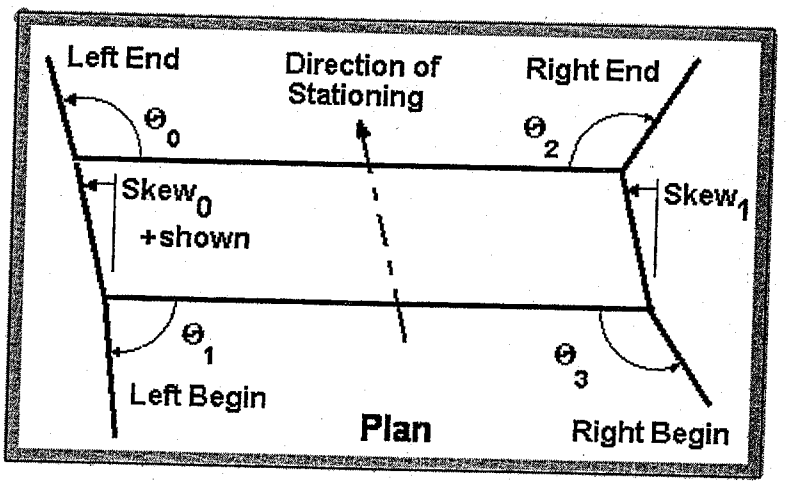
Head = 0 ft *water head at top of box (typically = 0.0)*

$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 171.2 ft^2





Soil Properties

$\gamma = 110 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 30$ deg *soil friction angle*

$$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Strength_{hw} = \begin{pmatrix} 38.4 & 27.39 & 72.31 & 74.86 & 47.36 \\ 38.4 & 27.39 & 72.31 & 74.86 & 47.36 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 23.49 & 18.22 & 46.48 & 45.62 & 29.64 \\ 23.49 & 18.22 & 46.48 & 45.62 & 29.64 \end{pmatrix}$$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S19.dat"

DataOut := DataIn

$$DataOut_{31} := \frac{L_0}{ft}$$

$$DataOut_{85} := Strength_{box}$$

$$DataOut_{86} := Service_{box}$$

$$DataOut_{87} := Strength_{hw}$$

$$DataOut_{88} := Service_{hw}$$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix}$ in

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $H_{hw} = \begin{pmatrix} 54 \\ 54 \end{pmatrix}$ in

$Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg *left* *right*

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ *Additional dead load if a barrier is located on top of the headwall*

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0..(nc-1)$	*index for cells*	$NoOfCells = 2$	*number of cells*	$f_c = 5.5\,ksi$	
$W_c = 10.7\,ft$	*opening width of cell*	$H_c = 8\,ft$	*opening height of cell*	$T_i = 12\,in$	*interior wall thickness*
$T_t = 12\,in$	*top slab thickness*	$T_w = 12\,in$	*exterior wall thickness*	$Env = 3$	*Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*
$E = 4200\,ksi$	*concrete modulus of elasticity*	$\gamma = 110 \frac{lbf}{ft^3}$	*soil density*		
$SoilHeight = 3\,ft$	*soil height above top slab*	$\gamma_w = 62.4 \frac{lbf}{ft^3}$	*weight of water*	$n_{mod} = 9$	*ratio of steel and concrete modulus of elasticity*
$\gamma_{conc} = 150 \frac{lbf}{ft^3}$	*concrete unit weight*	$F_y = 60\,ksi$			

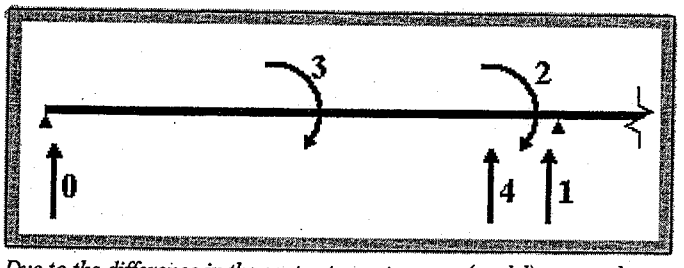
Calculate load combinations

Strength

$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$

Service

$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$Strength_{hw} = \begin{pmatrix} 38.4 & 27.39 & 81.44 & 79.38 & 47.36 \\ 38.4 & 27.39 & 81.44 & 79.38 & 47.36 \end{pmatrix}$

$Service_{hw} = \begin{pmatrix} 23.49 & 18.22 & 52.56 & 48.37 & 29.64 \\ 23.49 & 18.22 & 52.56 & 48.37 & 29.64 \end{pmatrix}$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	11.57	1.48	24.02	-2.04	19.44	10.79	1.74	15.19	-2.68	12.7
1	8.61	0.42	10.18	0.79	0	4.91	0	-5.15	0	0
2	0.72	0	0	0	0	8.61	0.42	12.92	2.53	10.73

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.01	0	15.29	0	12.91	6.54	0	9.94	0	8.13
1	5.5	2.15	6.62	1.01	0	3.2	0	-3.51	0	0
2	0.41	0	0	0	0	5.5	2.15	8.29	4.44	7.12

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	11.57	1.48	24.02	-2.04	19.44	10.79	1.74	15.19	-2.68	12.7

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.01	0	15.29	0	12.91	6.54	0	9.94	0	8.13

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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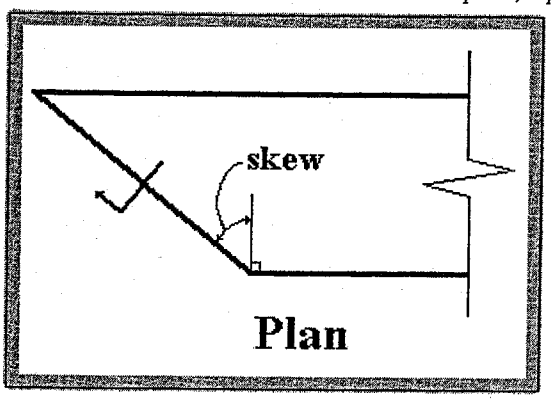
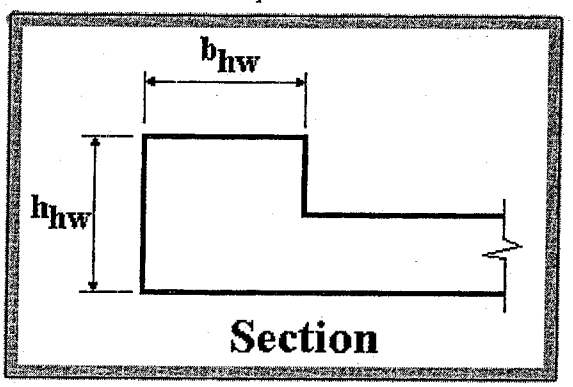
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad iwend := if(Extension \neq 1, 1, 0)$ $\quad iw := iwbeg .. iwend$ $\quad$ *index for walls*

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S19.dat"

Box Culvert Program:
Box & Headwall Load Cases

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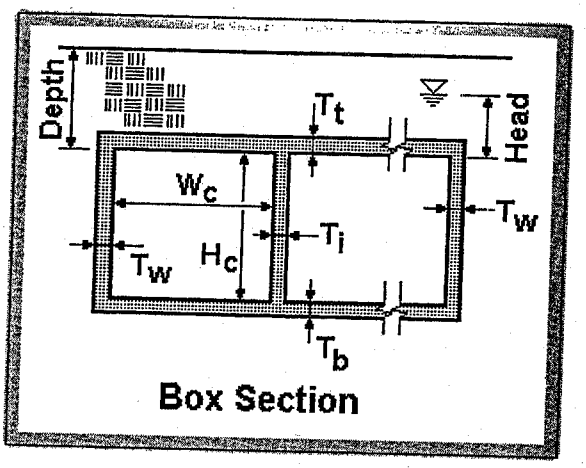
Project = "C43S15"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 10.7$ ft — *opening width of cell*

$H_c = 8$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 45$ ft — *length of culvert along centerline*

NoOfCells = 2 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 4 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 171.2 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 110 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 30$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

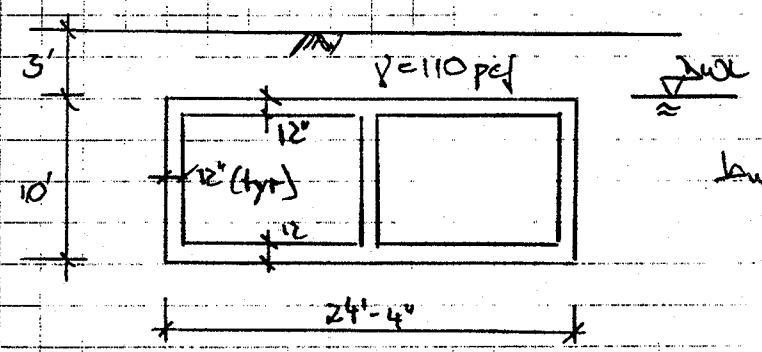
WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir

Project No.: _____ Page ___ of ___
Calculated By: WJK Date: 5/06
Checked: [illegible] Date: 5/06

R.C.T. ENGINEERING, INC.

CONSULTING ENGINEERS • PROGRAM MANAGERS

Flotation check C43S15



$b_w = 1$ ft

Weight of structure and surcharge:

$W_{st} = ((24.33 \times 10) - 2(10.67 \times 8))\,150 = 10887$ lbs

$W_{sur} = 24.33 \times 3 \times 110 = 8029$ lbs

$W_{tot} = 18916$ lbs

Uplift:

$U = 24.33 \times 10 \times 62.4 = 15182$ lbs

Flotation Safety Factor:

$SF_H = \frac{18916}{15182} = 1.25 > 1.1$ Extr. Maint. Case

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$BarSize_{long} = \begin{pmatrix} 3 \\ 5 \\ 3 \\ 5 \\ 3 \end{pmatrix}$ $S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in$

Change Bar Group (color) values or change individual Bars (white)

$NewBarSize_{long} := XX$

$NewS_{long} := XX \cdot in$

$BarSizeD1_{09} := XX$ $SD1_{09} := XX \cdot in$

$BarSizeD1_{10} := XX$ $SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX$ $SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX$ $SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX$ $SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp\,box} := (\text{"ok"}\ \text{"ok"}\ \text{"ok"}\ \text{"ok"}\ \text{"ok"})^T$

$Summary(CheckAs_{temp\,box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S14.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs}$ $DataOut_{33} := \frac{S_{slabs}}{in}$ $DataOut_{34} := BarSize_{walls}$ $DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners}$ $DataOut_{37} := \frac{S_{corners}}{in}$ $DataOut_{38} := BarSize_{long}$ $DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$

$DataOut_{90} := ReinfBox$

$WRITEPRN(CurrentDataFile) := DataOut$

$WRITEPRN(NewDataFile) := DataOut$

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) = "NoGood"$

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) = "OK"$

$CheckAs_{min.box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min.box}) = "NoGood"$

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) = "OK"$

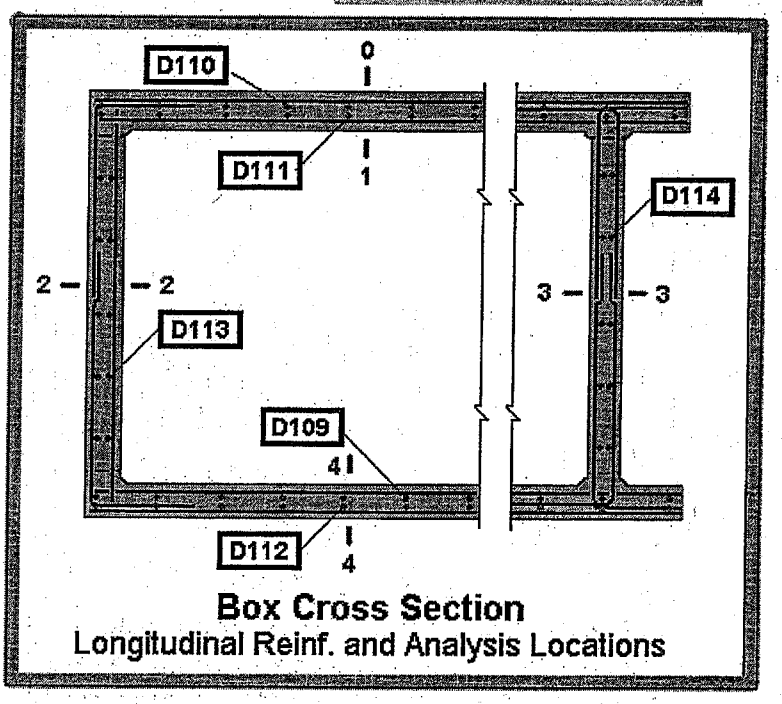
$Summary(CheckAll_{box}) = "NoGood"$

3. Design and Check Box Longitudinal Reinforcing

$CurrentDataFile := "C43S14.dat"$

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0..4$ *index for longitudinal sections*



Material Properties

Env = 3 *Environmental Class* *1-slightly aggressive* *2-moderate* *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

ConWheel1 = 6.74 kip ConWheel2 = 7.87 kip ConWheel3 = 7.87 kip *construction wheel loads*

ConAxleSpacing1 = 16.4 ft *space between axles 1 and 2*

ConAxleSpacing2 = 16.4 ft *space between axles 2 and 3*

2. Design and Check Main Reinforcing

[redacted]

$it := 0..20$ *index for transverse sections*

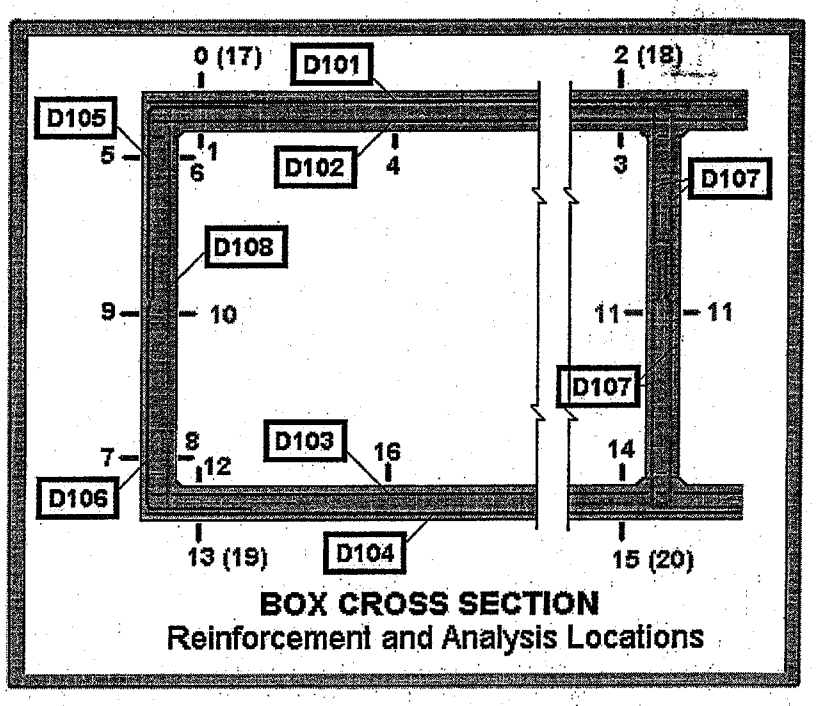
$BarSize_{box_{it}} := 0$ $S_{box_{it}} := 12 \cdot in$

Cross Section Notes:
1. Bars shown with the same color are combined as Bar Groups.
2. Colored numbers indicate moment analysis locations.
3. Colored numbers in parenthesis indicate shear analysis locations.
4. Black numbers refer to bar designations.

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number) *Bar Sizes & Spacings used in analysis* *Change Bar Group values (color) or change individual Bars (white)*

D101(2)

D102(1,3,4)

D103(12,14,16)

D104(15)

$BarSize_{slabs} = \begin{pmatrix} 7 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 12 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

[redacted]

D105(9,0,5)

D106(9,7,13)

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

[redacted]

D107(11)

D108(6,8,10)

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

[redacted]

$BarSizeD1_{01} := XX$ $SD1_{01} := XX \cdot in$

$BarSizeD1_{02} := XX$ $SD1_{02} := XX \cdot in$

$BarSizeD1_{03} := XX$ $SD1_{03} := XX \cdot in$

$BarSizeD1_{04} := XX$ $SD1_{04} := XX \cdot in$

$BarSizeD1_{05} := XX$ $SD1_{05} := XX \cdot in$

$BarSizeD1_{06} := XX$ $SD1_{06} := XX \cdot in$

$BarSizeD1_{07} := XX$ $SD1_{07} := XX \cdot in$

$BarSizeD1_{08} := XX$ $SD1_{08} := XX \cdot in$

Box Culvert Program:
Box Reinforcement Design

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Project = "C43S16"

DesignedBy = "WBK"

CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile := "C43S14.dat"

Geometry and Box Dimensions

$W_c = 10$ ft *opening width of cell*

$H_c = 6$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

$L_c = 29$ ft *length of culvert along centerline*

Depth = 3.5 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

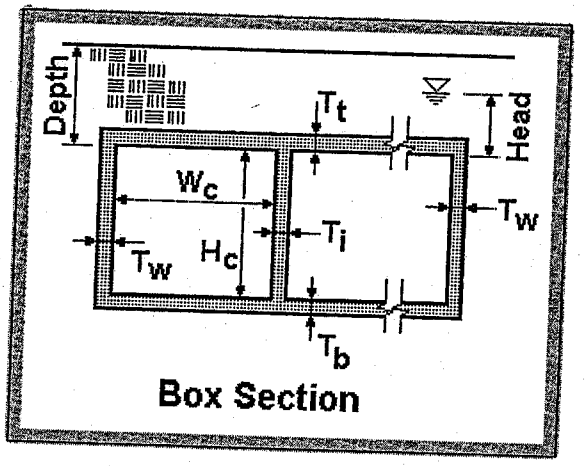
NoOfCells = 1 *number of cells*

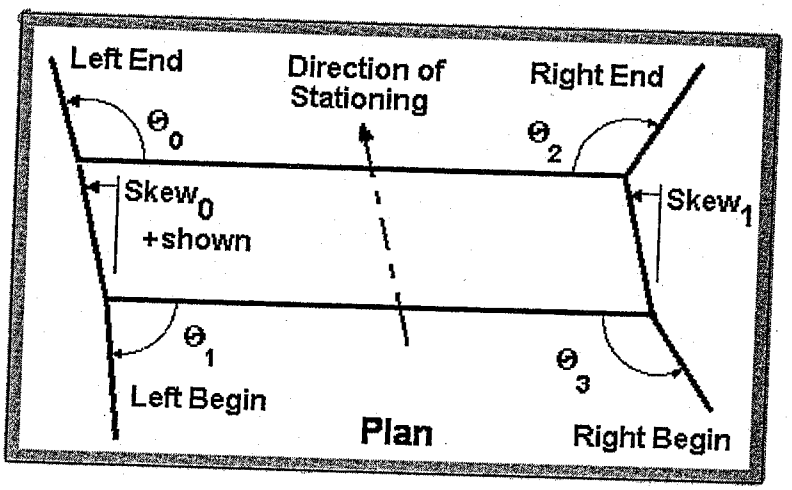
Head = 0 ft *water head at top of box (typically = 0.0)*

$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 60 ft^2





Soil Properties

$\gamma = 110 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 30$ deg *soil friction angle*

$$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Strength_{hw} = \begin{pmatrix} 41.53 & 21.1 & -7.03 & 91.57 & 41.53 \\ 41.53 & 21.1 & -7.03 & 91.57 & 41.53 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 25.47 & 13.8 & -4.6 & 56.82 & 25.47 \\ 25.47 & 13.8 & -4.6 & 56.82 & 25.47 \end{pmatrix}$$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S14.dat"

DataOut := DataIn

$$DataOut_{31} := \frac{L_0}{ft}$$

$$DataOut_{85} := Strength_{box}$$

$$DataOut_{86} := Service_{box}$$

$$DataOut_{87} := Strength_{hw}$$

$$DataOut_{88} := Service_{hw}$$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

iw = $\begin{pmatrix} 0 \\ 1 \end{pmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix}$ in

iw = $\begin{pmatrix} 0 \\ 1 \end{pmatrix}$ $H_{hw} = \begin{pmatrix} 48 \\ 48 \end{pmatrix}$ in

$Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg *left* *right*

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ *Additional dead load if a barrier is located on top of the headwall*

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0..(nc-1)$	*index for cells*	$NoOfCells = 1$	*number of cells*	$f_c = 5.5\,ksi$	
$W_c = 10\,ft$	*opening width of cell*	$H_c = 6\,ft$	*opening height of cell*	$T_i = 12\,in$	*interior wall thickness*
$T_t = 12\,in$	*top slab thickness*	$T_w = 12\,in$	*exterior wall thickness*	$Env = 3$	*Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*
$E = 4200\,ksi$	*concrete modulus of elasticity*	$\gamma = 110 \frac{lbf}{ft^3}$	*soil density*		
$SoilHeight = 2.5\,ft$	*soil height above top slab*	$\gamma_w = 62.4 \frac{lbf}{ft^3}$	*weight of water*	$n_{mod} = 9$	*ratio of steel and concrete modulus of elasticity*
$\gamma_{conc} = 150 \frac{lbf}{ft^3}$	*concrete unit weight*	$F_y = 60\,ksi$			

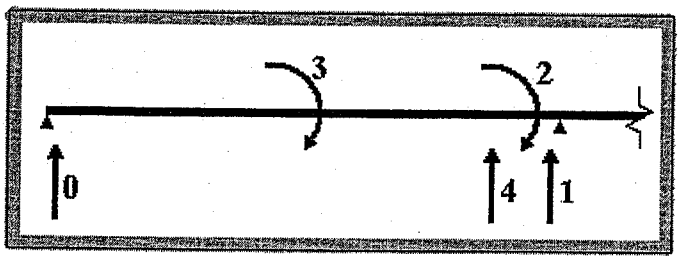
Calculate load combinations

Strength

$$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$

Service

$$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$$Strength_{hw} = \begin{pmatrix} 41.53 & 21.1 & 2.11 \times 10^{-14} & 97.48 & 41.53 \\ 41.53 & 21.1 & 2.11 \times 10^{-14} & 97.48 & 41.53 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 25.47 & 13.8 & 1.26 \times 10^{-14} & 60.48 & 25.47 \\ 25.47 & 13.8 & 1.26 \times 10^{-14} & 60.48 & 25.47 \end{pmatrix}$$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8
0	11.72	-0.82	11.72	-0.82	19.25	11.49	-0.82	13.89	-2.78
1	11.61	1.04	11.61	1.04	0	3.31	0	-3.56	0
2	0.39	0	0.39	0	0	11.61	1.04	14.42	2.34

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.21	0	7.21	0	12.04	7.16	0	8.97	0	9.48
1	7.23	0.82	7.23	0.82	0	2.14	0	-2.38	0	0
2	0.22	0	0.22	0	0	7.23	0.82	9.13	1.88	8.45

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	11.72	-0.82	11.72	-0.82	19.25	11.49	-0.82	13.89	-2.78	15.19

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.21	0	7.21	0	12.04	7.16	0	8.97	0	9.48

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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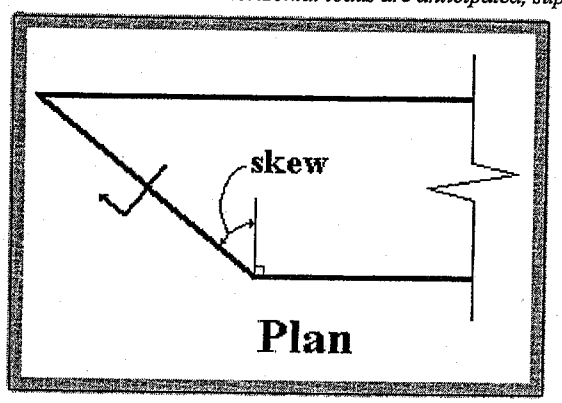
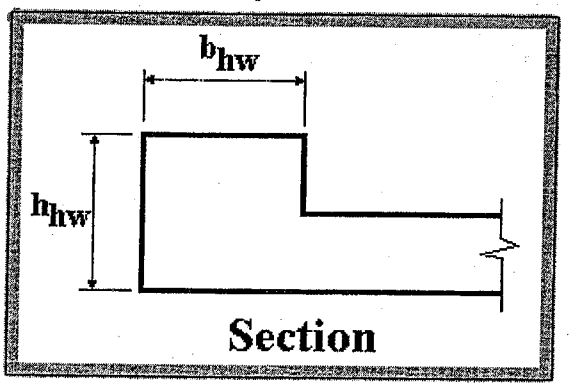
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad iwend := if(Extension \neq 1, 1, 0)$ $\quad iw := iwbeg .. iwend$ $\quad$ *index for walls*

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S14.dat"

Box Culvert Program:
Box & Headwall Load Cases

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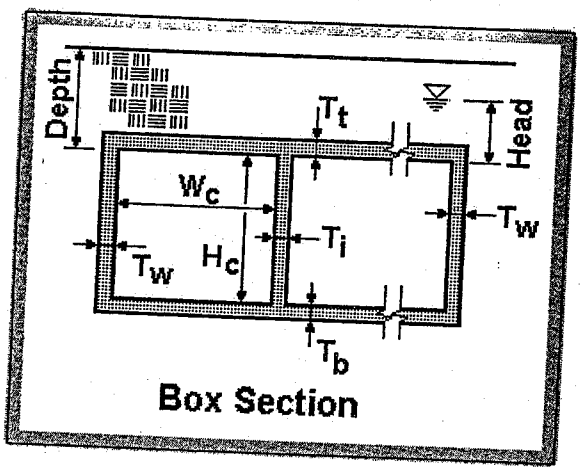
Project = "C43S16"

DesignedBy = "WBK"

CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 10$ ft — *opening width of cell*

$H_c = 6$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 29$ ft — *length of culvert along centerline*

NoOfCells = 1 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 3.5 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 60 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 110 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 30$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

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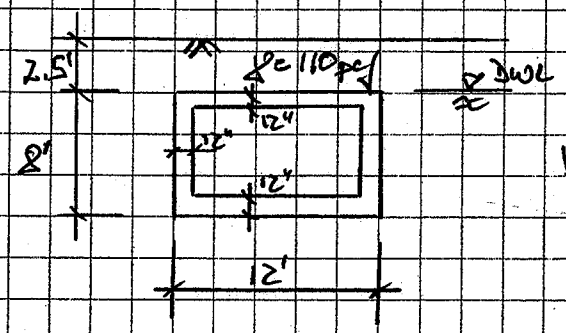
WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir
Project No.: _____ Page ____ of ____
Calculated By: WRK Date: 5/06
Checked: JB. Date: 5/06

Flotation check C43316:



$b_w = 1$ ft

Weight of structure and surcharge:

$W_{st} = (12 \times 8 - 10 \times 6)\,150 = 5400$ lks

$W_{sur} = 12 \times 2.5 \times 110 = 3300$ lks

$W_{tot} = 8700$ lks

Uplift:

$U = 12 \times 8 \times 62.4 = 5990$ lks

Flotation Safety Factor:

$SF_H = \dfrac{8700}{5990} = 1.45 > 1.1$ Ext. Maint. Case

WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$BarSize_{long} = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix}$ $S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} in$

Change Bar Group (color) values or change individual Bars (white)

$NewBarSize_{long} := XX$

$newS_{long} := XX \cdot in$

$BarSizeD1_{09} := XX \quad SD1_{09} := XX \cdot in$

$BarSizeD1_{10} := XX \quad SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX \quad SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX \quad SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX \quad SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp.box} := (\text{"ok"}\ \text{"ok"}\ \text{"ok"}\ \text{"ok"}\ \text{"ok"})^T$

$Summary(CheckAs_{temp.box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S19.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs}$ $\quad DataOut_{33} := \frac{S_{slabs}}{in}$ $\quad DataOut_{34} := BarSize_{walls}$ $\quad DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners}$ $\quad DataOut_{37} := \frac{S_{corners}}{in}$ $\quad DataOut_{38} := BarSize_{long}$ $\quad DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$ $\quad DataOut_{90} := ReinfBox$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) = "NoGood"$

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) = "NoGood"$

$CheckAs_{min.box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min.box}) = "NoGood"$

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) = "OK"$

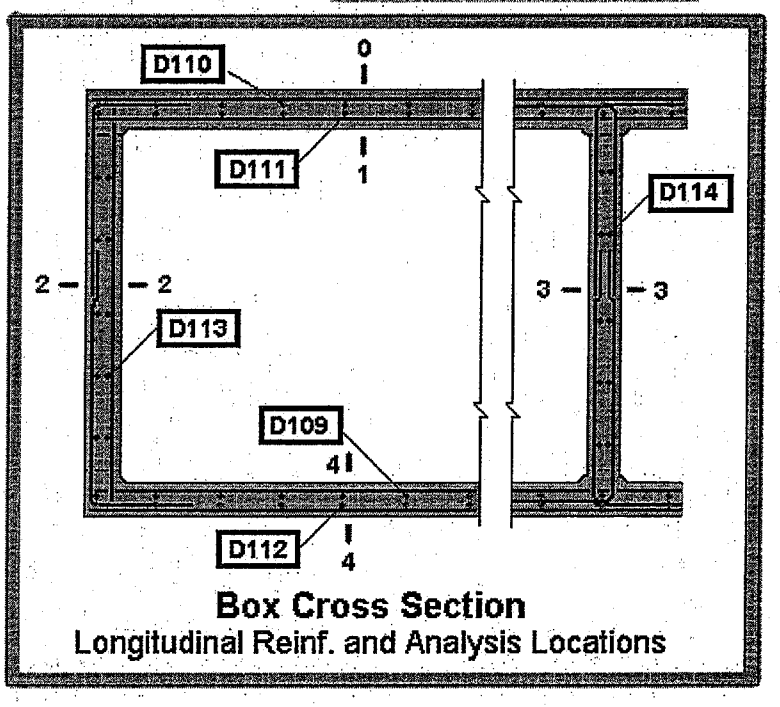
$Summary(CheckAll_{box}) = "NoGood"$

3. Design and Check Box Longitudinal Reinforcing

$CurrentDataFile = "C43S19.dat"$

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0..4$ *index for longitudinal sections*



Material Properties

Env = 3 *Environmental Class* *1-slightly aggressive* *2-moderate* *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

ConWheel1 = 6.74 kip ConWheel2 = 7.87 kip ConWheel3 = 7.87 kip *construction wheel loads*

ConAxleSpacing1 = 16.4 ft *space between axles 1 and 2*

ConAxleSpacing2 = 16.4 ft *space between axles 2 and 3*

2. Design and Check Main Reinforcing

Current Location = C43S1Loc1

$it := 0..20$ *index for transverse sections*

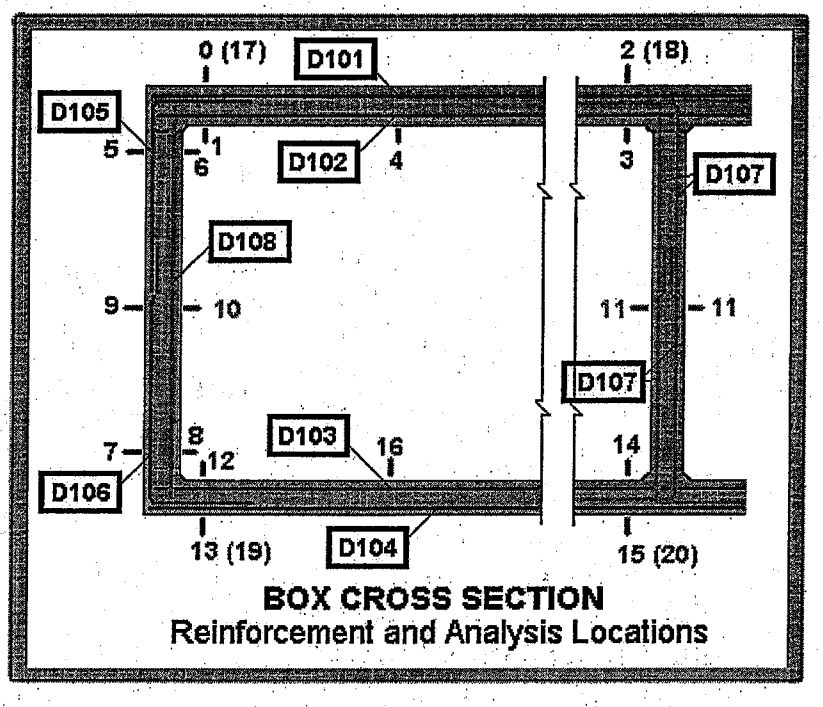
$BarSize_{box_{it}} := 0$ $S_{box_{it}} := 12 \cdot in$

Cross Section Notes:
1. Bars shown with the same color are combined as Bar Groups.
2. Colored numbers indicate moment analysis locations.
3. Colored numbers in parenthesis indicate shear analysis locations.
4. Black numbers refer to bar designations.

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number) *Bar Sizes & Spacings used in analysis* *Change Bar Group values (color) or change individual Bars (white)*

D101(2)
D102(1,3,4)
D103(12,14,16)
D104(15)

$BarSize_{slabs} = \begin{pmatrix} 8 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 10 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

$newBarSize_{slabs} := XX$

$newS_{slabs} := XX \cdot in$

D105(9,0,5)
D106(9,7,13)
D107(11)
D108(6,8,10)

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

$newBarSize_{corners} := XX$

$newS_{corners} := XX \cdot in$

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

$newBarSize_{walls} := XX$

$newS_{walls} := XX \cdot in$

$BarSizeD1_{01} := 8$ $SD1_{01} := 10 \cdot in$

$BarSizeD1_{02} := XX$ $SD1_{02} := XX \cdot in$

$BarSizeD1_{03} := XX$ $SD1_{03} := XX \cdot in$

$BarSizeD1_{04} := XX$ $SD1_{04} := XX \cdot in$

$BarSizeD1_{05} := XX$ $SD1_{05} := XX \cdot in$

$BarSizeD1_{06} := XX$ $SD1_{06} := XX \cdot in$

$BarSizeD1_{07} := XX$ $SD1_{07} := XX \cdot in$

$BarSizeD1_{08} := XX$ $SD1_{08} := XX \cdot in$

WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

Box Culvert Program:
Box Reinforcement Design

Project = "C43S17"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

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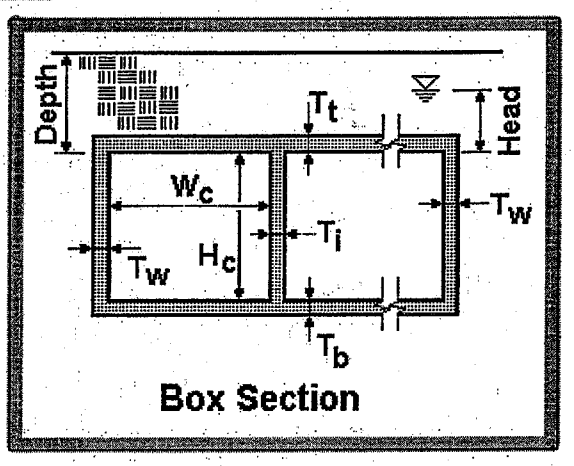
Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile = "C43S19.dat"

Geometry and Box Dimensions



$W_c = 6.7$ ft *opening width of cell*

$H_c = 6$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

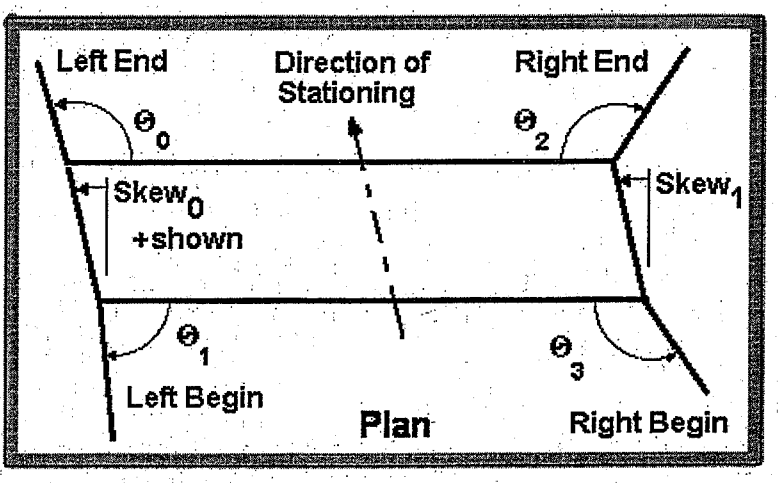
$L_c = 27$ ft *length of culvert along centerline*

Depth = 3.5 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

NoOfCells = 2 *number of cells*

Head = 0 ft *water head at top of box (typically = 0.0)*



$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg

$HydraulicOpening := W_c \cdot H_c \cdot NoOfCells$

$HydraulicOpening = 80.4\ ft^2$

Soil Properties

$\gamma = 110 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 30$ deg *soil friction angle*

$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$

$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$

$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$

$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$

$Strength_{hw} = \begin{pmatrix} 30.22 & 26.5 & 28.78 & 41.8 & 37.83 \\ 30.22 & 26.5 & 28.78 & 41.8 & 37.83 \end{pmatrix}$

$Service_{hw} = \begin{pmatrix} 18.13 & 16.58 & 18.18 & 24.89 & 23.05 \\ 18.13 & 16.58 & 18.18 & 24.89 & 23.05 \end{pmatrix}$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S19.dat"

DataOut := DataIn

$DataOut_{31} := \frac{L_0}{ft}$

$DataOut_{85} := Strength_{box}$

$DataOut_{86} := Service_{box}$

$DataOut_{87} := Strength_{hw}$

$DataOut_{88} := Service_{hw}$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

iw = $\begin{pmatrix} 0 \\ 1 \end{pmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix}$ in

iw = $\begin{pmatrix} 0 \\ 1 \end{pmatrix}$ $H_{hw} = \begin{pmatrix} 48 \\ 48 \end{pmatrix}$ in

$Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg *left* *right*

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ *Additional dead load if a barrier is located on top of the headwall*

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0 .. (nc - 1)$	*index for cells*	$NoOfCells = 2$	*number of cells*	$f_c = 5.5\,ksi$	
$W_c = 6.7\,ft$	*opening width of cell*	$H_c = 6\,ft$	*opening height of cell*	$T_i = 12\,in$	*interior wall thickness*
$T_t = 12\,in$	*top slab thickness*	$T_w = 12\,in$	*exterior wall thickness*	$Env = 3$	*Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*
$E = 4200\,ksi$	*concrete modulus of elasticity*	$\gamma = 110 \frac{lbf}{ft^3}$	*soil density*		
$SoilHeight = 2.5\,ft$	*soil height above top slab*	$\gamma_w = 62.4 \frac{lbf}{ft^3}$	*weight of water*	$n_{mod} = 9$	*ratio of steel and concrete modulus of elasticity*
$\gamma_{conc} = 150 \frac{lbf}{ft^3}$	*concrete unit weight*	$F_y = 60\,ksi$			

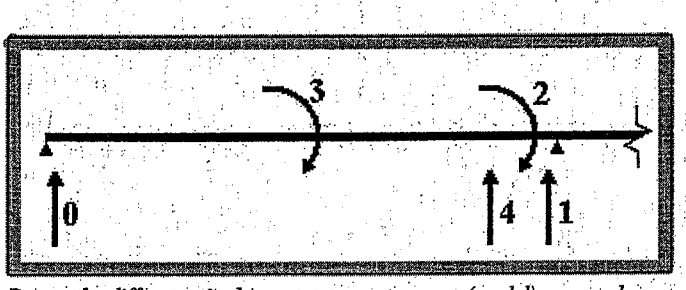
Calculate load combinations

Strength

$$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$

Service

$$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$$Strength_{hw} = \begin{pmatrix} 30.22 & 26.5 & 37.61 & 45.76 & 37.83 \\ 30.22 & 26.5 & 37.61 & 45.76 & 37.83 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 18.13 & 16.58 & 23.7 & 27.25 & 23.05 \\ 18.13 & 16.58 & 23.7 & 27.25 & 23.05 \end{pmatrix}$$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	5.78	1.69	11.69	-0.97	10.53	5.1	1.85	8.84	-1.12	6.62
1	6.59	3.8	6.54	0.49	0	3.59	0	-2.51	0	0
2	0.71	0	0	0	0	6.59	3.8	9.42	1.93	9.07

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	3.4	0.64	7.25	0	6.76	2.99	0.77	5.6	0	4.14
1	4.1	2.52	4.21	0.68	0	2.29	0	-1.79	0	0
2	0.41	0	0	0	0	4.1	2.52	6.01	3.15	5.83

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	5.78	1.69	11.69	-0.97	10.53	5.1	1.85	8.84	-1.12	6.62

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	3.4	0.64	7.25	0	6.76	2.99	0.77	5.6	0	4.14

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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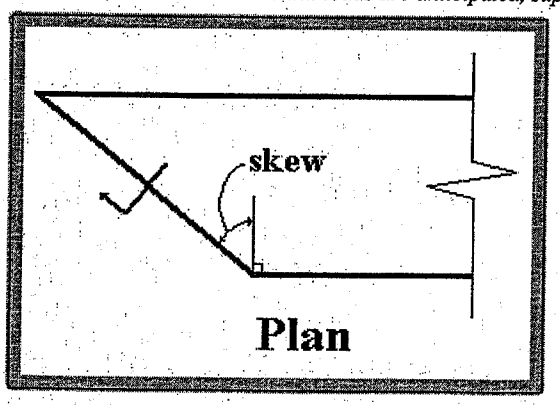
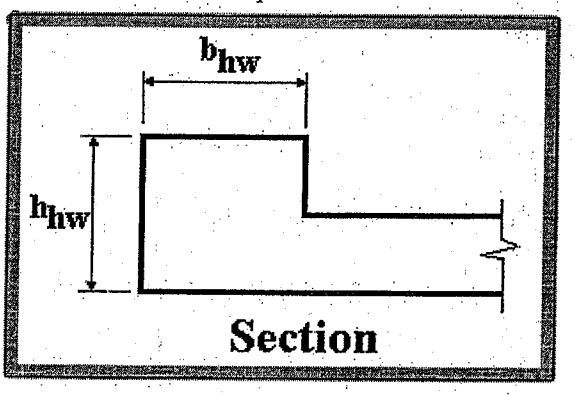
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad iwend := if(Extension \neq 1, 1, 0)$ $\quad iw := iwbeg..iwend$ $\quad$ *index for walls*

WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S19.dat"

Box Culvert Program:
Box & Headwall Load Cases

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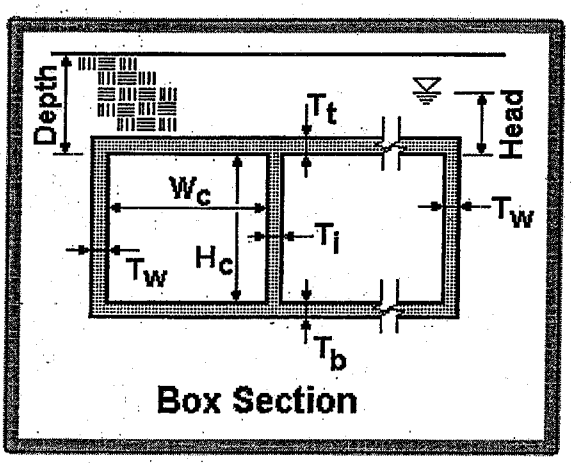
Project = "C43S17"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 6.7$ ft — *opening width of cell*

$H_c = 6$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 27$ ft — *length of culvert along centerline*

NoOfCells = 2 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 3.5 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 80.4 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 110 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 30$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir

Project No.: _____ Page ____ of ____
Calculated By: WJBK Date: 5/06
Checked: LB. Date: 5/06

R.C.T. ENGINEERING, INC.

CONSULTING ENGINEERS • PROGRAM MANAGERS

Flotation check C43S17:

See check for C43S9

701 Northpoint Parkway, Suite 310, West Palm Beach, FL 33407 Phone: 561.684.7534 Fax: 561.684.2801 RCT-WPB@RCTEngineering.com www.RCTEngineering.com

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$$BarSize_{long} = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix} \qquad S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} in$$

Change Bar Group (color) values or change individual Bars (white)

$BarSizeD1_{09} := XX \quad SD1_{09} := XX \cdot in$

$BarSizeD1_{10} := XX \quad SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX \quad SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX \quad SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX \quad SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp.box}^{T} = (\text{"ok"} \ \text{"ok"} \ \text{"ok"} \ \text{"ok"} \ \text{"ok"})$

$Summary(CheckAs_{temp.box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S19.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs} \qquad DataOut_{33} := \frac{S_{slabs}}{in} \qquad DataOut_{34} := BarSize_{walls} \qquad DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners} \qquad DataOut_{37} := \frac{S_{corners}}{in} \qquad DataOut_{38} := BarSize_{long} \qquad DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$

$DataOut_{90} := ReinfBox$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) = "NoGood"$

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) = "OK"$

$CheckAs_{min\ box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min\ box}) = "OK"$

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) = "OK"$

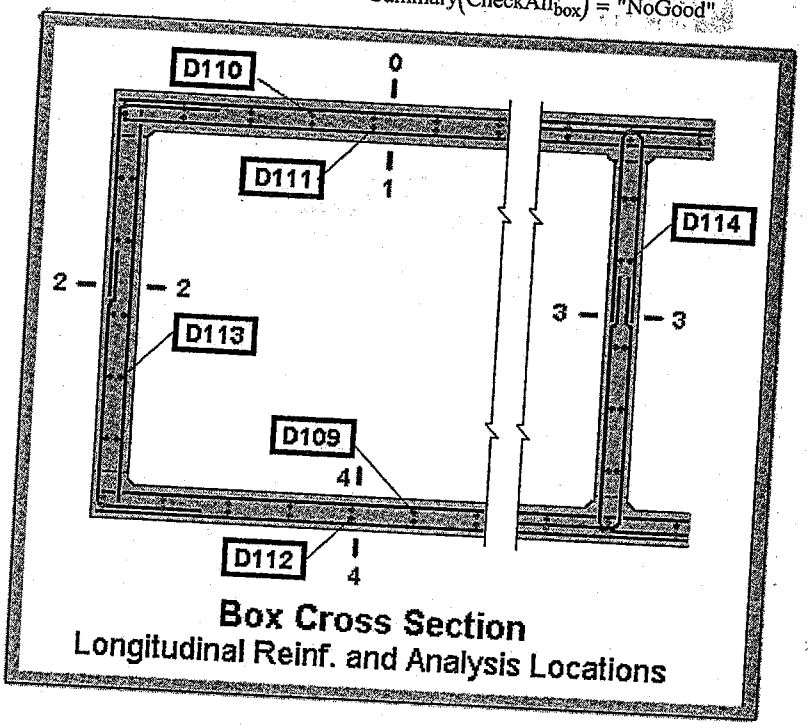
$Summary(CheckAll_{box}) = "NoGood"$

3. Design and Check Box Longitudinal Reinforcing

$CurrentDataFile = "C43S19.dat"$

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0..4$ *index for longitudinal sections*



Box Cross Section
Longitudinal Reinf. and Analysis Locations

Material Properties

Env = 3 *Environmental Class* *1-slightly aggressive* *2-moderate* *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

ConWheel1 = 6.74 kip ConWheel2 = 7.87 kip ConWheel3 = 7.87 kip *construction wheel loads*

ConAxleSpacing1 = 16.4 ft *space between axles 1 and 2*

ConAxleSpacing2 = 16.4 ft *space between axles 2 and 3*

2. Design and Check Main Reinforcing

CurrentDataFile := "C43S19.dat"

$it := 0..20$ *index for transverse sections*

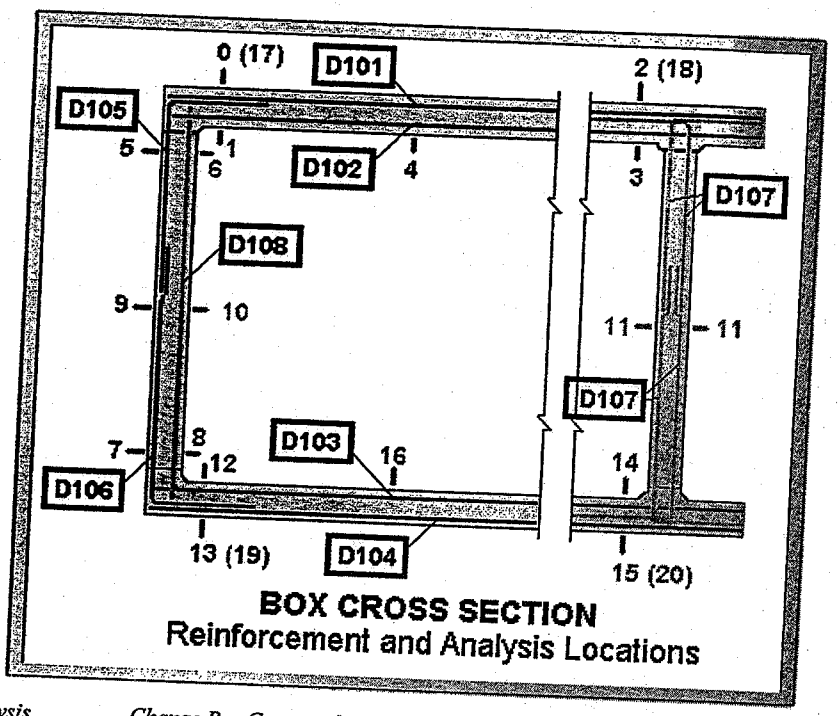
$BarSize_{box_{it}} := 0$ $S_{box_{it}} := 12 \cdot in$

Cross Section Notes:
1. Bars shown with the same color are combined as Bar Groups.
2. Colored numbers indicate moment analysis locations.
3. Colored numbers in parenthesis indicate shear analysis locations.
4. Black numbers refer to bar designations.

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number) *Bar Sizes & Spacings used in analysis* *Change Bar Group values (color) or change individual Bars (white)*

D101(2)

D102(1,3,4)

D103(12,14,16)

D104(15)

$BarSize_{slabs} = \begin{pmatrix} 7 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 12 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

$newBarSize_{slabs} := XX$

$newS_{slabs} := XX \cdot in$

D105(9,0,5)

D106(9,7,13)

D107(11)

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

$newBarSize_{corners} := XX$

$newS_{corners} := XX \cdot in$

D108(6,8,10)

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

$newBarSize_{walls} := XX$

$newS_{walls} := XX \cdot in$

$BarSizeD1_{01} := XX$ $SD1_{01} := XX \cdot in$

$BarSizeD1_{02} := XX$ $SD1_{02} := XX \cdot in$

$BarSizeD1_{03} := XX$ $SD1_{03} := XX \cdot in$

$BarSizeD1_{04} := XX$ $SD1_{04} := XX \cdot in$

$BarSizeD1_{05} := XX$ $SD1_{05} := XX \cdot in$

$BarSizeD1_{06} := XX$ $SD1_{06} := XX \cdot in$

$BarSizeD1_{07} := XX$ $SD1_{07} := XX \cdot in$

$BarSizeD1_{08} := XX$ $SD1_{08} := XX \cdot in$

Box Culvert Program:
Box Reinforcement Design

Project = "C43S18"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

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Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile = "C43S19.dat"

Geometry and Box Dimensions

$W_c = 6.7$ ft *opening width of cell*

$H_c = 6$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

$L_c = 25$ ft *length of culvert along centerline*

Depth = 3 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

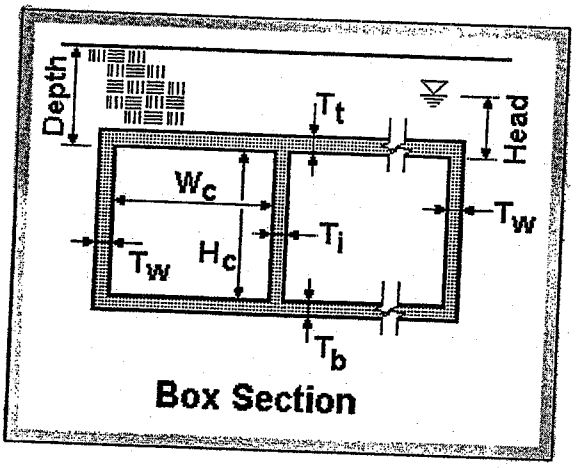
NoOfCells = 2 *number of cells*

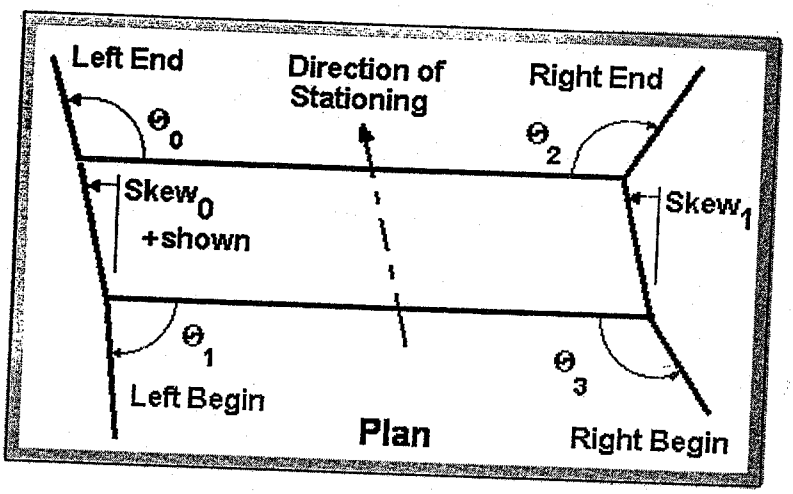
Head = 0 ft *water head at top of box (typically = 0.0)*

$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg

$HydraulicOpening := W_c \cdot H_c \cdot NoOfCells$

$HydraulicOpening = 80.4\ ft^2$





Soil Properties

$\gamma = 110 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 30$ deg *soil friction angle*

$$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Strength_{hw} = \begin{pmatrix} 29.81 & 25.82 & 27.96 & 41.32 & 37.15 \\ 29.81 & 25.82 & 27.96 & 41.32 & 37.15 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 17.81 & 16.04 & 17.53 & 24.51 & 22.51 \\ 17.81 & 16.04 & 17.53 & 24.51 & 22.51 \end{pmatrix}$$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S19.dat"

DataOut := DataIn

$$DataOut_{31} := \frac{L_0}{ft}$$

$$DataOut_{85} := Strength_{box}$$

$$DataOut_{86} := Service_{box}$$

$$DataOut_{87} := Strength_{hw}$$

$$DataOut_{88} := Service_{hw}$$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix}$ in

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $H_{hw} = \begin{pmatrix} 42 \\ 42 \end{pmatrix}$ in

$Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg *left* right

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ *Additional dead load if a barrier is located on top of the headwall*

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0..(nc - 1)$ *index for cells*

$NoOfCells = 2$ *number of cells*

$f_c = 5.5\,ksi$

$W_c = 6.7\,ft$ *opening width of cell*

$H_c = 6\,ft$ *opening height of cell*

$T_t = 12\,in$ *top slab thickness*

$T_w = 12\,in$ *exterior wall thickness*

$T_i = 12\,in$ *interior wall thickness*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$\gamma = 110 \frac{lbf}{ft^3}$ *soil density*

$Env = 3$ *Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*

$SoilHeight = 2\,ft$ *soil height above top slab*

$\gamma_w = 62.4 \frac{lbf}{ft^3}$ *weight of water*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ *concrete unit weight*

$F_y = 60\,ksi$

$n_{mod} = 9$ *ratio of steel and concrete modulus of elasticity*

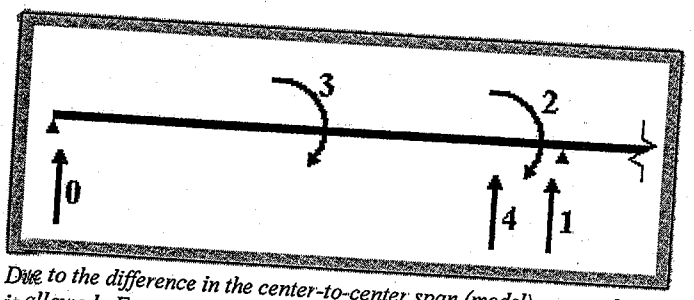
Calculate load combinations

Strength

$$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$

Service

$$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$$Strength_{hw} = \begin{pmatrix} 29.81 & 25.82 & 36.57 & 45.24 & 37.15 \\ 29.81 & 25.82 & 36.57 & 45.24 & 37.15 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 17.81 & 16.04 & 22.87 & 26.84 & 22.51 \\ 17.81 & 16.04 & 22.87 & 26.84 & 22.51 \end{pmatrix}$$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	6.16	2.78	13.8	-0.96	12.83	5.43	2.94	11.28	-0.65	7.79
1	6.98	6.5	5.99	0.49	0	3.54	0	-1.41	0	0
2	0.78	0	0	0	0	6.98	6.5	10.06	4.13	9.72

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	3.59	1.33	8.42	0	8.02	3.15	1.46	6.97	0	4.81
1	4.28	4.02	3.85	0.67	0	2.25	0	-1.15	0	0
2	0.45	0	0	0	0	4.28	4.02	6.33	2.96	6.15

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	6.16	2.78	13.8	-0.96	12.83	5.43	2.94	11.28	-0.65	7.79

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	3.59	1.33	8.42	0	8.02	3.15	1.46	6.97	0	4.81

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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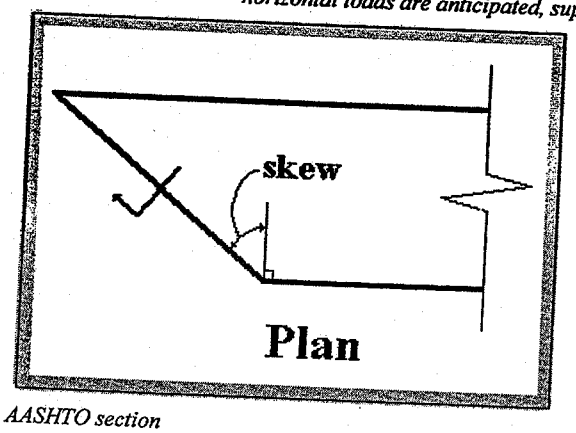
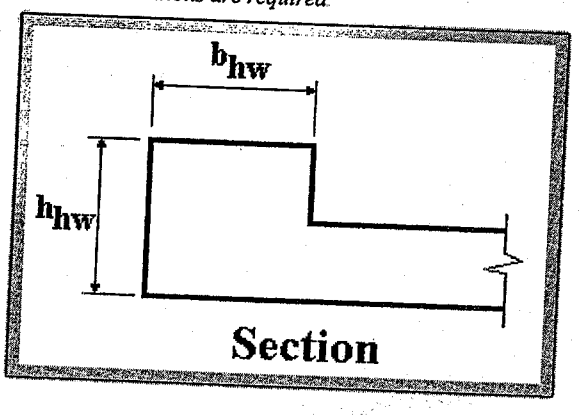
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad iwend := if(Extension \neq 1, 1, 0)$ $\quad iw := iwbeg .. iwend$ *index for walls*

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S19.dat"

Box Culvert Program:
Box & Headwall Load Cases

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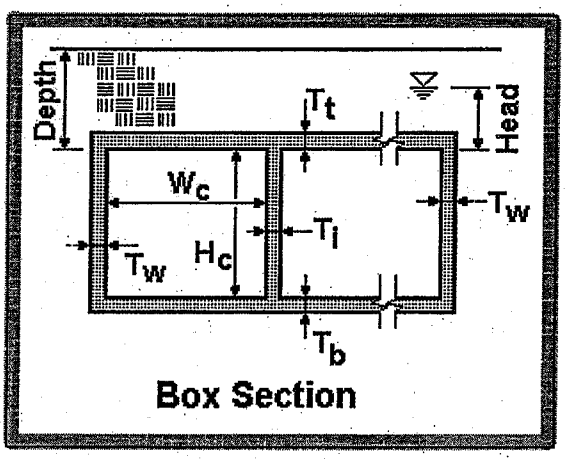
Project = "C43S18"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 6.7$ ft — *opening width of cell*

$H_c = 6$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 25$ ft — *length of culvert along centerline*

NoOfCells = 2 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 3 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 80.4 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 110 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 30$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

WO No. C-C10401P-WO
C-43 West Storage Reservoir

Project No.: _____ Page ____ of ____
Calculated By: WJK Date: 5/06
Checked: PJR Date: 5/06

R.C.T. ENGINEERING, INC.

CONSULTING ENGINEERS • PROGRAM MANAGERS

Flotation check C43S18:

See check for C43S9

Enter Box Longitudinal Reinforcing

Bar Sizes & Spacings used in analysis

$$BarSize_{long} = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix} \qquad S_{long} = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} in$$

Change Bar Group (color) values or change individual Bars (white)

$newBarSize_{long} := XX$

$newS_{long} := XX \cdot in$

$BarSizeD1_{09} := XX \quad SD1_{09} := XX \cdot in$

$BarSizeD1_{10} := XX \quad SD1_{10} := XX \cdot in$

$BarSizeD1_{11} := XX \quad SD1_{11} := XX \cdot in$

$BarSizeD1_{12} := XX \quad SD1_{12} := XX \cdot in$

$BarSizeD1_{13} := XX \quad SD1_{13} := XX \cdot in$

Section 3 - Box Longitudinal Reinforcement, 3 pages

$CheckAs_{temp\ box}^{T} = (\text{"ok"} \ \text{"ok"} \ \text{"ok"} \ \text{"ok"} \ \text{"ok"})$

$Summary(CheckAs_{temp.box}) = \text{"OK"}$

$Summary(CheckAll_{box}) = \text{"NoGood"}$

Write Box Design Data to CurrentDataFile = "C43S19.dat" file

Assign the data values read in to the ones to be read out, then change only the ones modified in this file

$DataOut := DataIn$

$DataOut_{32} := BarSize_{slabs}$ $\quad DataOut_{33} := \frac{S_{slabs}}{in}$ $\quad DataOut_{34} := BarSize_{walls}$ $\quad DataOut_{35} := \frac{S_{walls}}{in}$

$DataOut_{36} := BarSize_{corners}$ $\quad DataOut_{37} := \frac{S_{corners}}{in}$ $\quad DataOut_{38} := BarSize_{long}$ $\quad DataOut_{39} := \frac{S_{long}}{in}$

$DataOut_{55} := str2vec(Summary(CheckAll_{box}))$ $\quad DataOut_{90} := ReinfBox$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

$CheckCracking_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckCracking_{box}) = "NoGood"$

$CheckM_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckM_{box}) = "OK"$

$CheckAs_{min.box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckAs_{min.box}) = "NoGood"$

$CheckShear_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"	"ok"

$Summary(CheckShear_{box}) = "OK"$

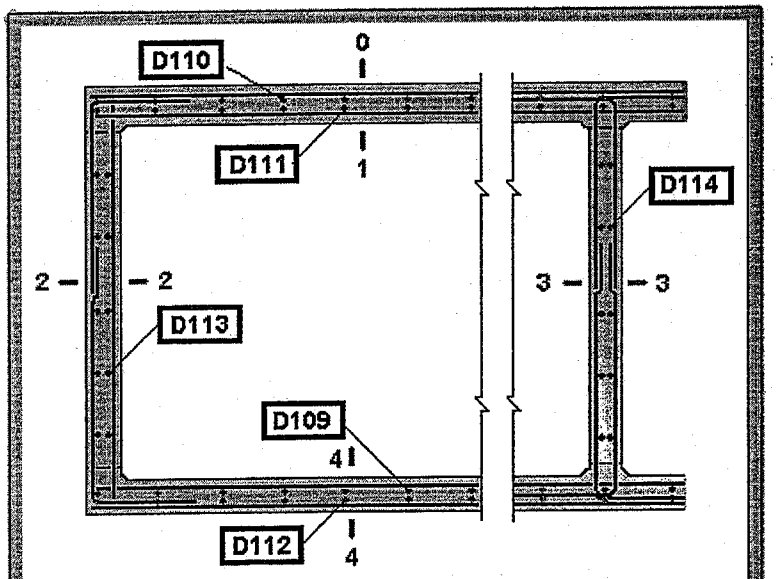
$Summary(CheckAll_{box}) = "NoGood"$

3. Design and Check Box Longitudinal Reinforcing

$CurrentDataFile = "C43S19.dat"$

To meet LRFD temperature and shrinkage requirements, reinforcement spacing should not exceed 12 inches.

$il := 0 .. 4$ *index for longitudinal sections*



Box Cross Section
Longitudinal Reinf. and Analysis Locations

Material Properties

Env = 3 — *Environmental Class*: *1-slightly aggressive*, *2-moderate*, *3-extreme*

$n_{mod} = 9$ *modular ratio*

$f_c = 5.5\,ksi$ *concrete strength*

$\gamma_{conc} = 150\,\frac{lbf}{ft^3}$ *density of concrete*

$E = 4200\,ksi$ *concrete modulus of elasticity*

$F_y = 60\,ksi$ *reinforcing strength*

Construction Vehicle Loads (if required)

ConWheel1 = 6.74 kip ConWheel2 = 7.87 kip ConWheel3 = 7.87 kip *construction wheel loads*

ConAxleSpacing1 = 16.4 ft *space between axles 1 and 2*

ConAxleSpacing2 = 16.4 ft *space between axles 2 and 3*

2. Design and Check Main Reinforcing

CurrentBarFile := "C43S19.dat"

$it := 0..20$ *index for transverse sections*

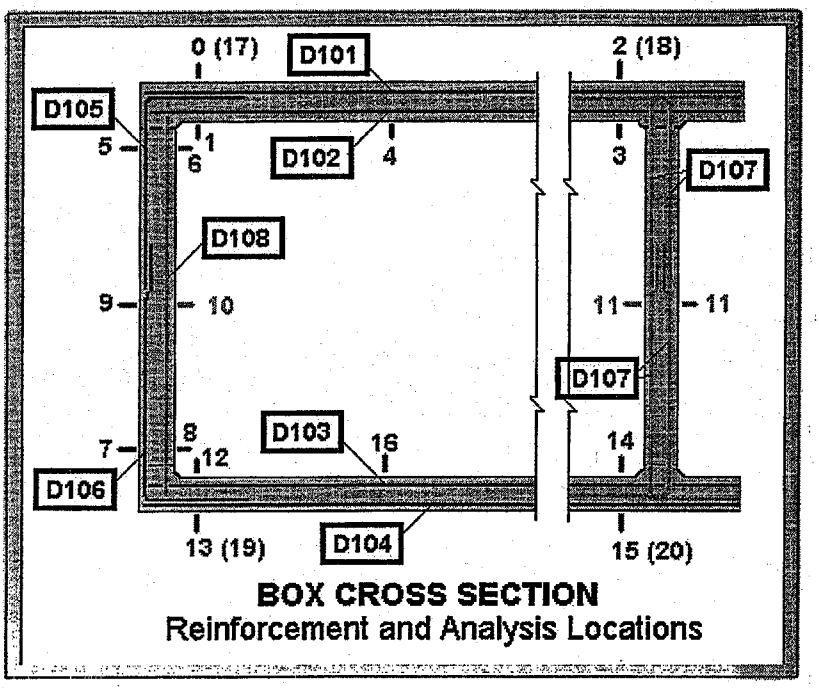
$BarSize_{box_{it}} := 0$ $S_{box_{it}} := 12\cdot in$

Cross Section Notes:
1. Bars shown with the same color are combined as Bar Groups.
2. Colored numbers indicate moment analysis locations.
3. Colored numbers in parenthesis indicate shear analysis locations.
4. Black numbers refer to bar designations.

Enter box reinforcing size and spacing

Generally, reinforcement should be at least a #3 bar and spacing should not exceed 12 inches.

Note: Shear generally controls slab and wall thicknesses and cracking generally controls reinforcement areas



Bar Designation (section number) — *Bar Sizes & Spacings used in analysis* — *Change Bar Group values (color) or change individual Bars (white)*

D101(2)
D102(1,3,4)
D103(12,14,16)
D104(15)

$BarSize_{slabs} = \begin{pmatrix} 7 \\ 8 \\ 6 \\ 5 \end{pmatrix}$ $S_{slabs} = \begin{pmatrix} 12 \\ 10 \\ 10 \\ 10 \end{pmatrix} in$

newBarSize$_{slabs}$:= XX
newS$_{slabs}$:= XX·in

D105(9,0,5)
D106(9,7,13)

$BarSize_{corners} = \begin{pmatrix} 8 \\ 6 \end{pmatrix}$ $S_{corners} = \begin{pmatrix} 10 \\ 10 \end{pmatrix} in$

newBarSize$_{corners}$:= XX
newS$_{corners}$:= XX·in

D107(11)
D108(6,8,10)

$BarSize_{walls} = \begin{pmatrix} 3 \\ 3 \end{pmatrix}$ $S_{walls} = \begin{pmatrix} 18 \\ 18 \end{pmatrix} in$

newBarSize$_{walls}$:= XX
newS$_{walls}$:= XX·in

$BarSizeD1_{01} := XX$ $SD1_{01} := XX\cdot in$
$BarSizeD1_{02} := XX$ $SD1_{02} := XX\cdot in$
$BarSizeD1_{03} := XX$ $SD1_{03} := XX\cdot in$
$BarSizeD1_{04} := XX$ $SD1_{04} := XX\cdot in$
$BarSizeD1_{05} := XX$ $SD1_{05} := XX\cdot in$
$BarSizeD1_{06} := XX$ $SD1_{06} := XX\cdot in$
$BarSizeD1_{07} := XX$ $SD1_{07} := XX\cdot in$
$BarSizeD1_{08} := XX$ $SD1_{08} := 12\cdot in$

Box Culvert Program:
Box Reinforcement Design

Project = "C43S19"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

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Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

$b_w = 1$ ft *This program analyzes a one foot wide cross section*

1. Design Parameters

CurrentDataFile := "C43S19.dat"

Geometry and Box Dimensions

$W_c = 10$ ft *opening width of cell*

$H_c = 6$ ft *opening height of cell*

$T_t = 12$ in *top slab thickness, (8 inch min.)*

$T_b = 12$ in *bottom slab thickness, (8 inch min.)*

$T_w = 12$ in *exterior wall thickness, (8 inch min.)*

$T_i = 12$ in *interior wall thickness, (8 inch min.) (Ti=Tw for one cell boxs)*

$L_c = 39$ ft *length of culvert along centerline*

Depth = 3.5 ft *distance from top of opening to surface*

Extension = 0 *extension type*
0 new box, no extension
1 left extension
2 right extension

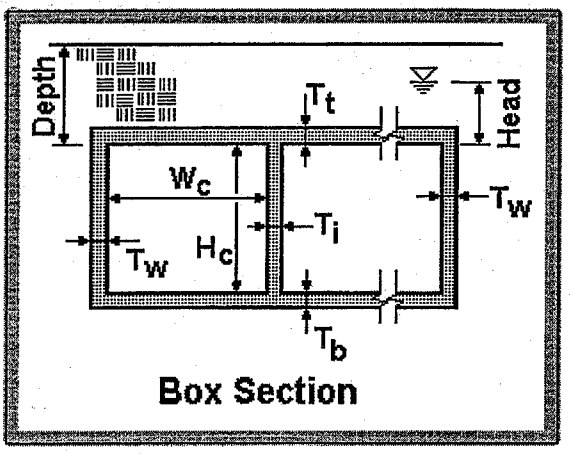
NoOfCells = 1 *number of cells*

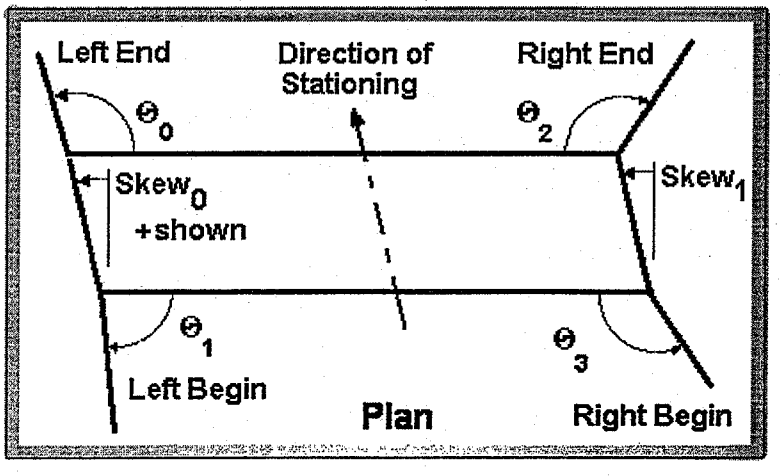
Head = 0 ft *water head at top of box (typically = 0.0)*

$Skew_{box} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg

HydraulicOpening := $W_c \cdot H_c \cdot$ NoOfCells

HydraulicOpening = 60 ft^2





Soil Properties

$\gamma = 110 \frac{lbf}{ft^3}$ *density of soil*

$k_s = 100000 \frac{lbf}{ft^3}$ *modulus of subgrade reaction*

$\phi = 30$ deg *soil friction angle*

$$Strength_{hw_{iw,2}} := Strength_{hw_{iw,2}} - \frac{1}{3} \cdot Strength_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,2}} := Service_{hw_{iw,2}} - \frac{1}{3} \cdot Service_{hw_{iw,1}} \cdot \frac{T_w}{ft}$$

$$Strength_{hw_{iw,3}} := Strength_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Strength_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Service_{hw_{iw,3}} := Service_{hw_{iw,3}} - \frac{1}{6} \cdot \left(\frac{4 \cdot Service_{hw_{iw,3}}}{L_{hw_{iw,0}}} \right) \cdot \frac{T_w}{ft}$$

$$Strength_{hw} = \begin{pmatrix} 41.53 & 21.1 & -7.03 & 91.57 & 41.53 \\ 41.53 & 21.1 & -7.03 & 91.57 & 41.53 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 25.47 & 13.8 & -4.6 & 56.82 & 25.47 \\ 25.47 & 13.8 & -4.6 & 56.82 & 25.47 \end{pmatrix}$$

Write box and headwall load CurrentDataFile

CurrentDataFile = "C43S19.dat"

DataOut := DataIn

$$DataOut_{31} := \frac{L_0}{ft}$$

$$DataOut_{85} := Strength_{box}$$

$$DataOut_{86} := Service_{box}$$

$$DataOut_{87} := Strength_{hw}$$

$$DataOut_{88} := Service_{hw}$$

WRITEPRN(CurrentDataFile) := DataOut

WRITEPRN(NewDataFile) := DataOut

Design parameters

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $B_{hw} = \begin{pmatrix} 18 \\ 18 \end{pmatrix}$ in

iw = $\begin{bmatrix} 0 \\ 1 \end{bmatrix}$ $H_{hw} = \begin{pmatrix} 48 \\ 48 \end{pmatrix}$ in

$Skew = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ deg *left* *right*

$BarrierDL_{hw} = 0 \frac{kip}{ft}$ *Additional dead load if a barrier is located on top of the headwall*

$ConsiderLL_{hw} = 1$ *This variable sets whether a line of truck wheels is considered as a loading. A 1 value means it is considered and a 0 means it is not*

$ic := 0..(nc-1)$	*index for cells*	$NoOfCells = 1$	*number of cells*	$f_c = 5.5\,ksi$	
$W_c = 10\,ft$	*opening width of cell*	$H_c = 6\,ft$	*opening height of cell*	$T_i = 12\,in$	*interior wall thickness*
$T_t = 12\,in$	*top slab thickness*	$T_w = 12\,in$	*exterior wall thickness*	$Env = 3$	*Environmental Class 1 is slightly aggressive 2 is moderate 3 is extreme*
$E = 4200\,ksi$	*concrete modulus of elasticity*	$\gamma = 110 \frac{lbf}{ft^3}$	*soil density*		
$SoilHeight = 2.5\,ft$	*soil height above top slab*	$\gamma_w = 62.4 \frac{lbf}{ft^3}$	*weight of water*	$n_{mod} = 9$	*ratio of steel and concrete modulus of elasticity*
$\gamma_{conc} = 150 \frac{lbf}{ft^3}$	*concrete unit weight*	$F_y = 60\,ksi$			

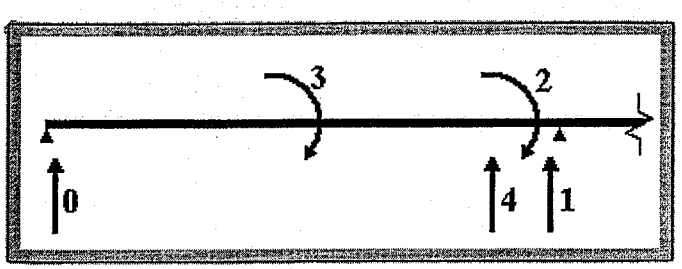
Calculate load combinations

Strength

$Strength_{hw} := 1.25 \cdot Force_{sw} + 1.35 \cdot Force_{ew} + 1.75 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.75 \cdot Force_{lane} \cdot ConsiderLL_{hw}$

Service

$Service_{hw} := 1.0 \cdot Force_{sw} + 1.0 \cdot Force_{ew} + 1.0 \cdot Force_{ll} \cdot ConsiderLL_{hw} + 1.0 \cdot Force_{lane} \cdot ConsiderLL_{hw}$



*Due to the difference in the center-to-center span (model) versus clear span (actual) a reduction in the moment forces is allowed. For end moments, a reduction of 1/3*Vend*t is used and for midspan moments 1/6*Vend*t is used.*

$$Strength_{hw} = \begin{pmatrix} 41.53 & 21.1 & 2.11 \times 10^{-14} & 97.48 & 41.53 \\ 41.53 & 21.1 & 2.11 \times 10^{-14} & 97.48 & 41.53 \end{pmatrix}$$

$$Service_{hw} = \begin{pmatrix} 25.47 & 13.8 & 1.26 \times 10^{-14} & 60.48 & 25.47 \\ 25.47 & 13.8 & 1.26 \times 10^{-14} & 60.48 & 25.47 \end{pmatrix}$$

$Strength_{box}^{T} =$

	0	1	2	3	4	5	6	7	8
0	11.72	-0.82	11.72	-0.82	19.25	11.49	-0.82	13.89	-2.78
1	11.61	1.04	11.61	1.04	0	3.31	0	-3.56	0
2	0.39	0	0.39	0	0	11.61	1.04	14.42	2.34

$Service_{box}^{T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.21	0	7.21	0	12.04	7.16	0	8.97	0	9.48
1	7.23	0.82	7.23	0.82	0	2.14	0	-2.38	0	0
2	0.22	0	0.22	0	0	7.23	0.82	9.13	1.88	8.45

$\left(Strength_{box}^{\langle 0 \rangle T}\right) =$

	0	1	2	3	4	5	6	7	8	9
0	11.72	-0.82	11.72	-0.82	19.25	11.49	-0.82	13.89	-2.78	15.19

$Service_{box}^{\langle 0 \rangle T} =$

	0	1	2	3	4	5	6	7	8	9
0	7.21	0	7.21	0	12.04	7.16	0	8.97	0	9.48

Write box load data to DataOut variable

$DataOut_{85} := Strength_{box}$ $\qquad$ $DataOut_{86} := Service_{box}$

Box Culvert Design: Headwall Design

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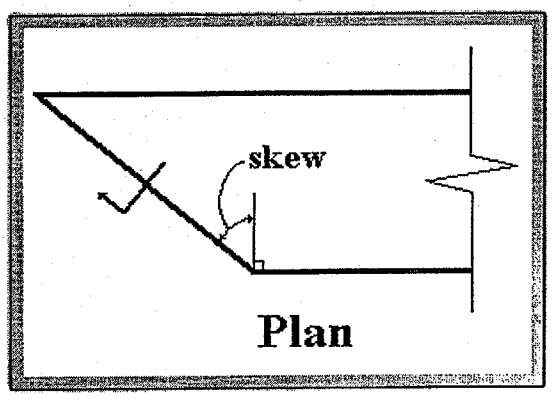
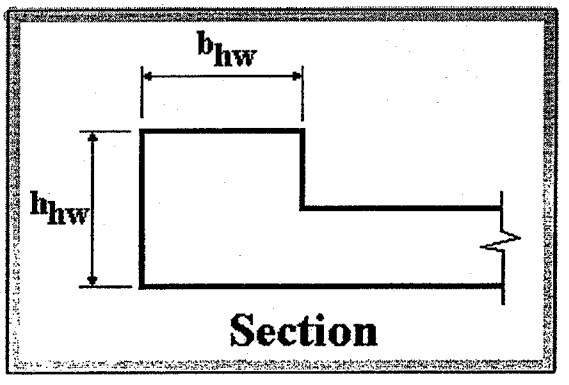
Design Headwall

Note: No lateral load analysis is performed on the headwall. If significant horizontal loads are anticipated, supplemental calculations are required.





Note: AASHTO section references are shown at right margin in bold-italic font.

Skew.mcd

Tributary Area for headwall design is defined in this file

$nc := NoOfCells$

$iwbeg := if(Extension \neq 2, 0, 1)$ $\quad$ $iwend := if(Extension \neq 1, 1, 0)$ $\quad$ $iw := iwbeg .. iwend$ $\quad$ *index for walls*

WO No. C-C10401P-WO No.9
C-43 West Storage Reservoir

Reference:C:\fdot_str\programs\LRFDBoxCulvertV2.21\ReadData.mcd(R)

CurrentDataFile = "C43S19.dat"

Box Culvert Program:
Box & Headwall Load Cases

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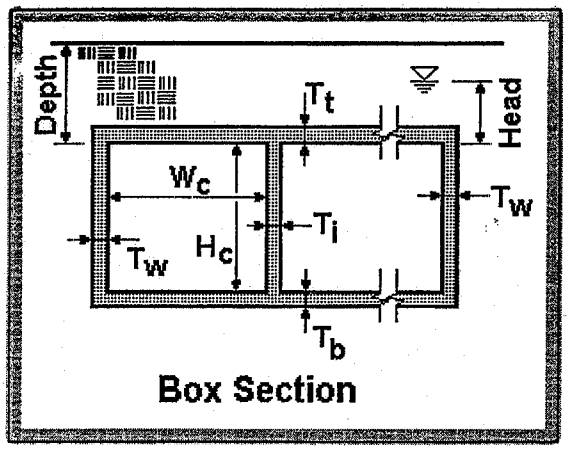
Project = "C43S19"
DesignedBy = "WBK"
CheckedBy = "[illegible]"

Note: AASHTO section references are shown at right margin, where appropriate, in bold-italic font

Design Parameters



Geometry and box dimensions from CurrentDataFile

$W_c = 10$ ft — *opening width of cell*

$H_c = 6$ ft — *opening height of cell*

$T_t = 12$ in — *top slab thickness, (8 inch min.)*

$T_b = 12$ in — *bottom slab thickness, (8 inch min.)*

$T_i = 12$ in — *interior wall thickness, (8 inch min.)*

$T_w = 12$ in — *exterior wall thickness, (8 inch min.)*

$L_c = 39$ ft — *length of culvert along centerline*

NoOfCells = 1 — *number of cells*

Extension = 0 — *extension type*
0 new box no extension,
1 left extension,
2 right extension

Head = 0 ft — *water head at top of box (typically = 0.0)*

Depth = 3.5 ft — *distance from top of opening to surface*

HydraulicOpening := $W_c \cdot H_c \cdot$NoOfCells

HydraulicOpening = 60 ft^2

$b_w = 1$ ft — *This program analyses a one foot wide cross section*

Soil properties

$\gamma = 110 \frac{lbf}{ft^3}$ — *density of soil*

$\phi = 30$ deg — *soil friction angle*

$k_s = 100000 \frac{lbf}{ft^3}$ — *modulus of subgrade reaction*

$q_{nom} = 5000\ ft \frac{lbf}{ft^3}$

Material properties

Env = 3 — *Environmental Class* *1-slightly aggresive* *2-moderate* *3-extreme*

$n_{mod} = 9$ — *modular ratio*

$f_c = 5.5$ ksi — *concrete strength preset for FDOT work*

E = 4200 ksi — *concrete modulus of elasticity*

$\gamma_{conc} = 150 \frac{lbf}{ft^3}$ — *density of concrete*

$F_y = 60$ ksi — *reinforcing strength*

Construction vehicle loads (if required)

ConWheel1 = 6.74 kip

ConWheel2 = 7.87 kip

ConWheel3 = 7.87 kip — *construction wheel loads*

ConAxleSpacing1 = 16.4 ft — *axle spacing between Wheels 1 and 2*

ConAxleSpacing2 = 16.4 ft — *axle spacing between Wheels 2 and 3*

Headwall Loads

$ConsiderLL_{hw} = 1$

$BarrierDL_{hw} = 0 \frac{kip}{ft}$

WO No. C-C10401P-WO N
C-43 West Storage Reservoi

Project No.: _____ Page ____ of ____
Calculated By: WRK Date: 5/06
Checked: JB. Date: 5/06

R.C.T. ENGINEERING, INC.

CONSULTING ENGINEERS • PROGRAM MANAGERS

Flotation Check C43S19:

See check for C43S16

... Phone 561 684 7524 Fax 561 684 2901 RCTWPB@RCTEngineering.com www.RCTEngineering.com

Culvert S-3: Stability Check

Units:

$k := 1000 \cdot lbf$ $kpf := \frac{kip}{ft}$ $ksf := \frac{kip}{ft^2}$ $ksi := \frac{kip}{in^2}$ $kcf := \frac{kip}{ft^3}$ $ppf := \frac{lbf}{ft}$ $psf := \frac{lbf}{ft^2}$ $psi := \frac{lbf}{in^2}$

$pcf := \frac{lbf}{ft^3}$ $\gamma_{water} := 62.4 \cdot pcf$ $\gamma_{conc} := 150 \cdot pcf$ $f_c := 4 \cdot ksi$ $f_y := 60 \cdot ksi$ $F_y := 50 \cdot ksi$ $E := 29000 \cdot ksi$

Description

Reinforced Concrete Culvert Structure with one (1) Sluice Gate (i.e., Roller Gate) and one (1) Weir Gate (i.e., Vertical Double-Slide Aluminum Gate) to regulate water flow around Perimeter Canal.

References

EC 1110-2-6058 Stability Requirements, USACE 30 November 2003
ETL 11102-307 Engineerin and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987
Design of Small Dams, published by United State Department of the Interior, Bureau of Reclamation; A Water Resources Technical Publication.
Geotechnical Report by DE&T, dated 1/06/06

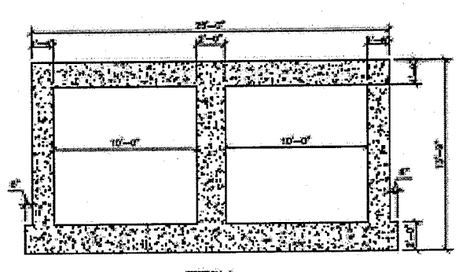
$SF_{Floating\_USACE} := 1.2$

Elevation and Design Input

Water Elevation, in ft:	$EL_{water} := 23.5 \cdot ft$	Maximum High Water 100 years event
Ground Line Elevation, in ft:	$EL_{GL} := 27.0 \cdot ft$	Ground line at CL of Levee Assume ground water to surface - saturated soil
Flow line Elevation, in ft:	$EL_{FL} := 15.00 \cdot ft$	
Saturated Soil Unit Weight,in pcf:	$\gamma_{soil} := 125 \cdot pcf$	

Structure Input

Culvert Clear Span, in ft:	$W := 10.0 \cdot ft$
Culvert Clear Height, in ft:	$H := 10.0 \cdot ft$
Culvert Wall Thickness, in ft:	$T_{wall} := 1.5 ft$
Culvert Mid_Wall Thickness, in ft:	$T_{mid\_wall} := 2 ft$
Top Slab Thickness, in ft:	$T_{top} := 1.75 \cdot ft$
Bot Slab Thickness, in ft:	$T_{bot} := 2.0 \cdot ft$
Face of Wall to Edge of Bot Slab, in ft:	$OH := 0.5 \cdot ft$
Channel Slope in Bottom Slab:	$Slope := 0$



Culvert S-3: Stability Check

Units:

$k := 1000 \cdot lbf$ $kpf := \frac{kip}{ft}$ $ksf := \frac{kip}{ft^2}$ $ksi := \frac{kip}{in^2}$ $kcf := \frac{kip}{ft^3}$ $ppf := \frac{lbf}{ft}$ $psf := \frac{lbf}{ft^2}$ $psi := \frac{lbf}{in^2}$

$pcf := \frac{lbf}{ft^3}$ $\gamma_{water} := 62.4 \cdot pcf$ $\gamma_{conc} := 150 \cdot pcf$ $f_c := 4 \cdot ksi$ $f_y := 60 \cdot ksi$ $F_y := 50 \cdot ksi$ $E := 29000 \cdot ksi$

Description

Reinforced Concrete Culvert Structure with one (1) Sluice Gate (i.e., Roller Gate) and one (1) Weir Gate (i.e., Vertical Double-Slide Aluminum Gate) to regulate water flow around Perimeter Canal.

References

EC 1110-2-6058 Stability Requirements, USACE 30 November 2003
ETL 11102-307 Engineerin and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987
Design of Small Dams, published by United State Department of the Interior, Bureau of Reclamation, A Water Resources Technical Publication.
Geotechnical Report by DE&T, dated 1/06/06

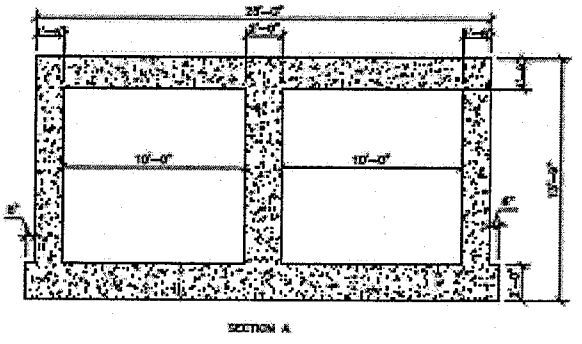
$SF_{Floating\_USACE} := 1.2$

Elevation and Design Input

Water Elevation, in ft:	$EL_{water} := 23.5 \cdot ft$	Maximum High Water 100 years event
Ground Line Elevation, in ft:	$EL_{GL} := 27.0 \cdot ft$	Ground line at CL of Levee Assume ground water to surface - saturated soil
Flow line Elevation, in ft:	$EL_{FL} := 15.00 \cdot ft$	
Saturated Soil Unit Weight,in pcf:	$\gamma_{soil} := 125 \cdot pcf$	

Structure Input

Culvert Clear Span, in ft:	$W := 10.0 \cdot ft$
Culvert Clear Height, in ft:	$H := 10.0 \cdot ft$
Culvert Wall Thickness, in ft:	$T_{wall} := 1.5 \cdot ft$
Culvert Mid_Wall Thickness, in ft:	$T_{mid\_wall} := 2 \cdot ft$
Top Slab Thickness, in ft:	$T_{top} := 1.75 \cdot ft$
Bot Slab Thickness, in ft:	$T_{bot} := 2.0 \cdot ft$
Face of Wall to Edge of Bot Slab, in ft:	$OH := 0.5 \cdot ft$
Channel Slope in Bottom Slab:	$Slope := 0$



Computed Values

$W_{TS} := 2W + 2 \cdot T_{wall} + T_{mid\_wall}$ $\quad W_{TS} = 25\ ft$ width of top slab

$W_{BS} := W_{TS} + 2 \cdot OH$ $\quad W_{BS} = 26\ ft$ width of bottom slab

$H_{slope} := \frac{W}{2} \cdot Slope$ $\quad H_{slope} = 0$ height of box slope

$H_{wall} := H - H_{slope}$ $\quad H_{wall} = 10\ ft$ ht of wall

$EL_{TS} := EL_{FL} + H + T_{top}$ $\quad EL_{TS} = 26.75\ ft$ Top of Top Slab Elevation, in ft:

$EL_{BS} := EL_{FL} - T_{bot}$ $\quad EL_{BS} = 13\ ft$ Bot of Bot Slab Elevation, in ft:

$EL_{TBS} := EL_{FL} + H_{slope}$ $\quad EL_{TBS} = 15\ ft$ EL top of bottom slab

$\gamma_{buoy} := \gamma_{soil} - \gamma_{water}$ $\quad \gamma_{buoy} = 62.6\ pcf$ buoyant wt of soils

ht of water above structure

$$H_{water\_top} := \begin{cases} EL_{water} - EL_{TS} & \text{if } EL_{water} \geq EL_{TS} \\ 0 \cdot ft & \text{otherwise} \end{cases} \qquad H_{water\_bot} := \begin{cases} EL_{water} - EL_{TBS} & \text{if } EL_{water} \geq EL_{TBS} \\ 0 \cdot ft & \text{otherwise} \end{cases}$$

$H_{water\_top} = 0\ ft$ $\qquad H_{water\_bot} = 8.5\ ft$

ht of soil above structure

$$H_{soil\_top} := \begin{cases} EL_{GL} - EL_{TS} & \text{if } EL_{GL} \geq EL_{TS} \\ 0 \cdot ft & \text{otherwise} \end{cases} \qquad H_{soil\_bot} := \begin{cases} EL_{GL} - EL_{TBS} & \text{if } EL_{GL} \geq EL_{TBS} \\ 0 \cdot ft & \text{otherwise} \end{cases}$$

$H_{soil\_top} = 0.25\ ft$ $\qquad H_{soil\_bot} = 12\ ft$

ht of buoyant soil above structure

$$H_{buoy\_top} := \begin{cases} H_{soil\_top} & \text{if } EL_{water} \geq EL_{GL} \\ EL_{water} - EL_{TS} & \text{if } EL_{GL} > EL_{water} \geq EL_{TS} \\ 0 \cdot ft & \text{otherwise} \end{cases}$$

$H_{soil\_top} = 0.25\ ft$

$$H_{buoy\_bot} := \begin{cases} H_{soil\_bot} & \text{if } EL_{water} \geq EL_{GL} \\ EL_{water} - EL_{TBS} & \text{if } EL_{GL} > EL_{water} \geq EL_{TBS} \\ 0 \cdot ft & \text{otherwise} \end{cases}$$

$H_{soil\_bot} = 12\ ft$

ht of saturated soil above structure

$$H_{sat\_top} := \begin{cases} H_{soil\_top} - H_{buoy\_top} & \text{if } H_{soil\_top} \geq H_{buoy\_top} \\ 0\,ft & \text{otherwise} \end{cases}$$

$H_{sat\_top} = 0.25\ ft$

$$H_{sat\_bot} := \begin{cases} H_{soil\_bot} - H_{buoy\_bot} & \text{if } H_{soil\_bot} \geq H_{buoy\_bot} \\ 0\,ft & \text{otherwise} \end{cases}$$

$H_{sat\_bot} = 3.5\ ft$

Weight of Structure

$TopSlab := W_{TS} \cdot T_{top} \cdot \gamma_{conc}$ $\quad TopSlab = 6.562 \frac{kip}{ft}$

$Walls := 2H_{wall} \cdot T_{wall} \cdot \gamma_{conc}$ $\quad Walls = 4.5 \frac{kip}{ft}$

$BotSlab := \left[W_{BS} \cdot (T_{bot} + H_{slope}) - H_{slope} \cdot \frac{W}{2}\right] \cdot \gamma_{conc}$ $\quad BotSlab = 7.8 \frac{kip}{ft}$

$Wt_{struct} := TopSlab + Walls + BotSlab$

$Wt_{struct} = 18.863 \frac{k}{ft}$

Weight of Soil Above Structure

$$Wt_{soil} := H_{buoy\_top} \cdot W_{TS} \cdot \gamma_{buoy} + H_{sat\_top} \cdot W_{TS} \cdot \gamma_{soil} + H_{buoy\_bot} \cdot 2 \cdot OH \cdot \gamma_{buoy} + H_{sat\_bot} \cdot 2 \cdot OH \cdot \gamma_{soil}$$

$H_{buoy\_top} \cdot W_{TS} \cdot \gamma_{buoy} = 0 \frac{k}{ft}$ $\quad H_{buoy\_bot} \cdot 2 \cdot OH \cdot \gamma_{buoy} = 0.532 \frac{k}{ft}$

$H_{sat\_top} \cdot W_{TS} \cdot \gamma_{soil} = 0.781 \frac{k}{ft}$ $\quad H_{sat\_bot} \cdot 2 \cdot OH \cdot \gamma_{soil} = 0.438 \frac{k}{ft}$

$Wt_{soil} = 1.751 \frac{k}{ft}$

Uplift Forces: force of water on the bottom of the structure

$water_{press} := EL_{water} - EL_{BS}$ $\quad EL_{water} = 23.5\ ft$ $\quad EL_{BS} = 13\ ft$

$water_{press} = 10.5\ ft$

$U := W_{BS} \cdot water_{press} \cdot \gamma_{water}$ $\quad W_{BS} = 26\ ft$

$U = 17.035 \frac{k}{ft}$

Weight of Water on Top of Structure

$H_{water\_top} = 0$ ft

$H_{water\_bot} = 8.5$ ft

$W_{TS} = 25$ ft

$OH = 0.5$ ft

$W_G := (H_{water\_top} \cdot W_{TS} + H_{water\_bot} \cdot 2 \cdot OH) \cdot \gamma_{water}$

$W_G = 0.53 \frac{k}{ft}$

Computing Factor of Safety Against Uplift force:

$Wt_{struct} = 18.863 \frac{k}{ft}$ $\quad Wt_{soil} = 1.751 \frac{k}{ft}$

$W_S := Wt_{struct} + Wt_{soil}$

$W_S = 20.613 \frac{k}{ft}$ $\quad U = 17.035 \frac{k}{ft}$

$SF_{Floating} := \frac{W_S}{U - W_G}$

$SF_{Floating} = 1.249$ Compare to EC 1110-2-6058 Table 3-4 reqd FS for flotation
FS = 1.3 Usual Event, FS = 1.2 Unusual Event, FS = 1.1 Extreme Event

$Check\_Floating := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$

Check_Floating = "O.K."

Quantity take-off for Structure S-3

Item	Quantity	Length	Width	Thickness/ Height	Linear	Areas	Volume	Volume	Weight
		ft	ft	ft	ft	sft	cft	CY	tons
Spread Footing:									
Formwork									
Concrete									
Reiforcement									
Finishes									
SOG:									
Bottom Slab at Culvert:									
Formwork	2	20.00	26	2		184.00			
Concrete	1	20.00	26	2		520.00	1040.00	38.52	
Reiforcement	0.015						15.60		3.82
Finishes						624.00			
Roadway Approach Slab:									
Formwork	4	10	13	1		92.00			
Concrete	2	10	13	1		260.00	260.00	9.63	
Reiforcement	0.015						3.90		0.96
Finishes						260.00			
Apron Slab:									
Formwork	2	48.46	56	1.5		313.38			
Concrete	1	28.44	28	1.5		796.25	1194.38	44.24	
Concrete	1	20.02	28	1.5		560.56	840.88	31.14	
Reiforcement	0.0214						43.51		10.66
Finishes						1356.83			
Walls:									
Abutment (upstream):									
Formwork	4	10.00		1.50		1349.00			
Concrete	2	10.00		1.50		655.00	982.50	36.39	
Reiforcement	0.015						14.74		3.61
Finishes						1381.90			
Abutment (downstream):									
Formwork	4	18.42		1.50		1358.20			
Concrete	2	18.42		1.50		921.65	1382.47	51.20	
Reiforcement	0.015						20.74		5.08
Finishes						1412.18			
Pier:									
Formwork	4	8.50		17.50		630.00			
Concrete	2	8.50		17.50		297.5	297.50	11.02	
Reiforcement	0.015						4.46		1.09
Finishes						647.00			
Exterior Walls:									
Formwork	4	20.00		10.00		830.00			
Concrete	2	20.00		10.00		400	600.00	22.22	
Reiforcement	0.015						9.00		2.21
Finishes						830.00			
Interior Wall:									
Formwork	2	20.00		10.00		440.00			
Concrete	1	20.00		10.00		200	400.00	14.81	
Reiforcement	0.015						6.00		1.47
Finishes						440.00			
Head Wall:									
Formwork	4	28	1.5	6.75		796.50			
Concrete	2	28	1.5	6.75		378.00	567	21.00	
Reiforcement	0.015						8.51		2.08
Finishes						880.50			
Columns (Supporting Platform):									
At Abutments:									
Formwork									
Concrete									
Reiforcement									
Finishes									
At Piers:									
Formwork									
Concrete									
Reiforcement									
Finishes									
Weir (under roller gate):									
Formwork									
Concrete									
Reiforcement									
Finishes									

Quantity take-off for Structure S-3

Item	Quantity	Length ft	Width ft	Thickness/ Height ft	Linear ft	Areas sft	Volume cft	Volume CY	Weight tons
Sills :									
End Sill:									
Formwork	1	26		1		26			
Concrete	1	26		1		1.84	47.71	1.77	
Reiforcement	0.008						0.38		0.09
Finishes						102.83			
Weir Sill									
Formwork	1	10	1.67	7		163.33			
Concrete	1	10	2	7			140	5.19	
Reiforcement	0.008						1.12		0.27
Finishes						180.00			
For Dewatering Needles:									
Formwork	2	26		0.5		26			
Concrete	2	26		0.5		0.63	32.5	1.20	
Reiforcement	0.008						0.26		0.06
Finishes						114.77			
Baffles:									
Formwork	15	3.50		1.67		95.83			
Concrete	15			1.50		1.94	43.75	1.62	
Reiforcement	0.008						0.35		0.09
Finishes						136.26			
Elevated Slabs:									
Top Slab at Culvert:									
Formwork	2	20.00	25	1.75		657.50			
Concrete	1	20.00	25	1.75		500.00	875.00	32.41	
Reiforcement	0.025						21.88		5.36
Finishes						87.50			
Operating Platform:									
Formwork	2	25.00	2.5	0.5		90.00			
Concrete	1	25.00	2.5	0.5		62.50	31.25	1.16	
Reiforcement	0.025						0.78		0.19
Finishes						2.50			
Roadway:									
Formwork	2	17	25.00			46			
Concrete (wearing surface & sidewalk)	1	17	25.00				195.31	7.23	
Reiforcement	0.015						2.93		0.72
Finishes						477.88			
Breast Wall:									
Formwork	2	24	1.50	1.50		216			
Concrete	2	24	1.50	1.50			108	4.00	
Reiforcement	0.015						1.62		0.40
Finishes						288			
Sheet Piling:									
Sheetpiles:									
Peripheric locking sheet piles	0	65.46		9.00		0.00			
Peripheric locking sheet piles (tremie)	2	77.46		5.00		774.58			
Peripheric locking sheet piles (up/downstream cutoff wall)	2	28.00		12.50		700.00			
Peripheric locking sheet piles (tremie)	2	39.00		5.00		390.00			
Upstream wingwalls	2	40.00		14.50		1740.00			
Downstream wingwalls	2	25.00		16.00		1200.00			
Pile Cap:									
Formwork	4	20.00	2	2.00		480.00			
Concrete	4	20.00	2.00	2.00			320.00	11.85	
Reiforcement	0.008						2.56		0.63
Finishes						640.00			
Seal Concrete:									
Concrete (poor strength)	1	77.46	39.00	5.00			15104.38	559.42	
Control Building:									
Precast Control Bldg.	1								
Excavation:									
Excavation	1	77.46	39.00	26.90			90324.16	3345.34	
Erosion Control System:									
Erosion Control Fabric	4	20	37.70			5255.69			
Bedding Rock	1			0.5			2627.85	97.33	
Rip-rap	1			1.5			7883.54	291.98	

Spillway S-10: Load Case 1A (Normal Pool Condition with tail water at weir crest elevation of structure downstream)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	As_1	2		40.7		15.7	2.50	3194.0	479.1			43.4	20810.8	
Abutment 2 (height varies)	As_2	2		23.1		15.7	2.50	1812.9	271.9			11.5	3140.0	
Breast Wall	Bw_3	1		20.0		6.3	1.00	125.0	18.8			53.3	1000.2	
Bridge	Bs	1		45.0		19.5	1.33	1170.0	175.5			39.5	6937.8	
Column-Support Platform	Cs_1	2		11.5	20.8			478.3	71.7			48.5	3482.7	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			48.5	3081.2	
Platform	Pl3	1		45.0	9.8			440.2	66.0			48.5	3205.0	
Spread Foundation (Apron Slab)	S_1	1		54.0		63.8	3.50	12054.7	1808.2			31.9	57665.0	
Pier	P_1	2		37.2		15.7	3.00	3503.1	525.5			41.7	21905.2	
Weir	Ws2			14.0	38.8			542.9	81.4			49.0	3987.8	
Abutment Triangular Section at Base	Ac_1				7.1	63.8		449.7	67.4			31.9	2151.0	
Roller Gate	G_1	1		15.6	168.9	10.8			11.0			49.9	548.3	
Weir Gate Sill	Sl_w	2		10.0	15.7	4.7			23.5			49.9	1173.3	
Baffles	B_3	26		1.5	6.0			232.2	34.8			17.5	608.5	
Water on top of Heel (upstream side)	W_c_heel	1		34.0		10.4	13.70	4861.8	303.4			58.6	17766.5	
Water on top of Crest (upstream side)	W_c_crest	1		34.0		2.5	9.00	774.6	48.3			52.1	2517.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		34.0		13.6	2.34	1083.2	67.6			29.9	2021.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		40.0		23.1	2.34	2161.6	134.9			11.5	1557.5	
Water above End Sill (Surcharge Water)	W_g	1		40.0		23.1	6.66	6152.2	383.9			11.5	4432.9	
Head Water Active Pressure (upstream side)	W_c_head water										415.4	5.7		2381.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										72.8	2.4	174.7	
Uplift Water Pressure	U_1	1		54.0		63.78	7.20			1547.4		42.5		65797.4
Uplift Water Pressure	U_{d1}	1		54.0		63.78	10.00			1074.8		31.9		34269.5
Silt Passive Pressure (downstream side)-neglected														
Total									4636.5	2622.0	342.6		158166.6	102448.3

Overturning neglecting Uplift (for bearing check only): 2381.4

W_c	554.2
W_g	383.9
W_c	3698.4
	4636.5

Safety Factors Check:	
S.F. against Overturning:	1.54
S.F. against Sliding:	4.74
S.F. agaist Floatation:	1.90

Elevations (ft):	Upstream	Downstream
Normal Water	19.00	9.00
Maximum Water		
Top of Slab	5.30	5.30
Bottom of Slab	1.80	1.80

Baffles:	yes	
Roller Gate Weir Elev.:	10.00	ft
Weir Gate Sill Elev.:	13.50	ft
Roller Gate Weir Width.:	13.14	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	40.69	ft
Abutment Length:	23.09	ft
Pier Width:	3.00	ft
Pier Length:	37.19	ft
Pier Elev.:	21.00	ft
Pier Elev. (end):	21.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	34.00	ft
Top of Service Bridge	21.00	ft
Breast Wall Top Elev.:	25.00	ft
Breast Wall Bottom Elev.:	20.00	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	19.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	63.78	ft
End Sill Height:	2.34	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Friction Coefficient:	0.35	
No. of Roller Gates	1.00	
No. of Weir Gates	2.00	

Lane width:	13 ft
Sidewalk:	5.67 ft
Curb:	0.83 ft
Bridge width:	19.50 ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:		
area	6.25	4.53125
neutral axis		0.725 ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	C.L. of Baffles from toe:
1.50		17.47 ft
1.50		
14.00	10.00	
10	16	

7

Spillway S-10: Load Case 1B (Normal Pool Condition with tail water provided in Structure Operations Summary)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	A_{s1}	2		40.7		15.7	2.50	3194.0	479.1			43.4	20810.8	
Abutment 2 (height varies)	A_{s2}	2		23.1		15.7	2.50	1812.9	271.9			11.5	3140.0	
Breast Wall	B_{ws}	1		20.0		6.3	1.00	125.0	18.8			53.3	1000.2	
Bridge	B_s	1		45.0		19.5	1.33	1170.0	175.5			39.5	6937.8	
Column-Support Platform	Cs_1	2		11.5	20.8			478.3	71.7			48.5	3482.7	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			48.5	3081.2	
Platform	PLs	1		45.0	9.8			440.2	66.0			48.5	3205.0	
Spread Foundation (Apron Slab)	S_s	1		54.0		63.8	3.50	12054.7	1808.2			31.9	57665.0	
Pier	P_r	2		37.2		15.7	3.00	3503.1	525.5			41.7	21805.2	
Weir	Ws2			14.0	38.8			542.9	81.4			49.0	3987.8	
Abutment Triangular Section at Base	A_{s3}				7.1	63.8		449.7	67.4			31.9	2151.0	
Roller Gate	G_s	1		15.6	168.9	10.8			11.0			49.9	548.3	
Weir Gate Sill	SL_g	2		10.0	15.7	4.7			23.5			49.9	1173.3	
Baffles	B_d	26		1.5	6.0			232.2	34.8			17.5	608.5	
Water on top of Heel (upstream side)	W_c_heel	1		34.0		10.4	13.70	4861.8	303.4			58.6	17766.5	
Water on top of Crest (upstream side)	W_c_crest	1		34.0		2.5	9.00	774.6	48.3			52.1	2517.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		34.0		13.6	2.34	1083.2	67.6			29.9	2021.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		40.0		23.1	2.34	2161.6	134.9			11.5	1557.5	
Water above End Sill (Surcharge Water)	W_g	1		40.0		23.1	12.66	11694.8	729.8			11.5	8426.5	
Head Water Active Pressure (upstream side)	W_c_head water										415.4	5.7		2381.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										244.6	4.4	1076.4	
Uplift Water Pressure	U_d	1		54.0		63.78	13.20			2836.9		42.5		120628.7
Uplift Water Pressure	U_{ut}	1		54.0		63.78	4.00			429.8		31.9		13707.8
Silt Passive Pressure (downstream side)-neglected														
Total									4982.3	3266.8	170.7		163061.9	136717.8

Overturning neglecting Uplift (for bearing check only): 2381.4

W_c	554.2
W_g	729.8
W_c	3698.4
	4982.3

Safety Factors Check:	
S.F. against Overturning:	1.19
S.F. against Sliding:	10.21
S.F. agaist Floatation:	1.68

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:		
area	6.25	4.53125
neutral axis		0.725 ft

Lane width: 13 ft
Sidewalk: 5.67 ft
Curb: 0.83 ft
Bridge width: 19.50 ft

Elevations (ft):	Upstream	Downstream
Normal Water	19.00	15.00
Maximum Water		
Top of Slab	5.30	5.30
Bottom of Slab	1.80	1.80

Baffles:	yes	
Roller Gate Weir Elev.:	10.00	ft
Weir Gate Sill Elev.:	13.50	ft
Roller Gate Weir Width.:	13.14	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	40.69	ft
Abutment Length:	23.09	ft
Pier Width:	3.00	ft
Pier Length:	37.19	ft
Pier Elev.:	21.00	ft
Pier Elev. (end):	21.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	34.00	ft
Top of Service Bridge	21.00	ft
Breast Wall Top Elev.:	25.00	ft
Breast Wall Bottom Elev.:	20.00	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	19.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	63.78	ft
End Sill Height:	2.34	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Friction Coefficient:	0.35	
No. of Roller Gates	1.00	
No. of Weir Gates	2.00	

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	C.L. of Baffles from toe:
1.50		17.47 ft
1.50		
14.00	10.00	
10	16	

00

Spillway S-10: Load Case 2A (Maximum Pool Condition with tail water at weir crest elevation of structure downstream)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	A_{s1}	2		40.7		15.7	2.50	3194.0	479.1			43.4	20810.8	
Abutment 2 (height varies)	A_{s2}	2		23.1		15.7	2.50	1812.9	271.9			11.5	3140.0	
Breast Wall	B_{ws}	1		20.0		6.3	1.00	125.0	18.8			53.3	1000.2	
Bridge	Bs	1		45.0		19.5	1.33	1170.0	175.5			39.5	6937.8	
Column-Support Platform	Cs_1	2		11.5	20.8			478.3	71.7			48.5	3482.7	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			48.5	3081.2	
Platform	PLs	1		45.0	9.8			440.2	66.0			48.5	3205.0	
Spread Foundation (Apron Slab)	S_s	1		54.0		63.8	3.50	12054.7	1808.2			31.9	57685.0	
Pier	P_s	2		37.2		15.7	3.00	3503.1	525.5			41.7	21905.2	
Weir	Ws2			14.0	38.8			542.9	81.4			49.0	3987.8	
Abutment Triangular Section at Base	A_{s3}				7.1	63.8		449.7	67.4			31.9	2151.0	
Roller Gate	G_s	1		15.6	168.9	10.8			11.0			49.9	548.3	
Weir Gate Sill	Sl_{wg}	2		10.0	15.7	4.7			23.5			49.9	1173.3	
Baffles	B_{rr}	26		1.5	6.0			232.2	34.8			17.5	608.5	
Water on top of Heel (upstream side)	W_c_heel	1		34.0		10.4	14.70	5216.7	325.5			58.6	19063.3	
Water on top of Crest (upstream side)	W_c_crest	1		34.0		2.5	10.00	860.6	53.7			52.1	2796.8	
Water contained on top of Apron (downstream side)	W_c_apron	1		34.0		13.6	2.34	1083.2	67.6			29.9	2021.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		40.0		23.1	2.34	2161.6	134.9			11.5	1557.5	
Water above End Sill (Surcharge Water)	W_g	1		40.0		23.1	6.66	6152.2	383.9			11.5	4432.9	
Head Water Active Pressure (upstream side)	W_c_head water										465.1	6.1		2821.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										72.8	2.4	174.7	
Uplift Water Pressure	U_d	1		54.0		63.78	7.20			1547.4		42.5		65797.4
Uplift Water Pressure	U_{ut}	1		54.0		63.78	11.00			1182.0		31.9		37696.5
Silt Passive Pressure (downstream side)-neglected														
Total									4664.0	2729.5	392.3		159743.1	106315.3

Overturning neglecting Uplift (for bearing check only): 2821.4

W_c:	581.7
W_g:	383.9
W_r:	3698.4
	4664.0

Safety Factors Check	
Against Overturning:	1.50
Against Sliding:	4.16
S.F. against Floatation:	1.82

Elevations (ft):	Upstream	Downstream
Normal Water		
Maximum Water	20.00	9.00
Top of Slab	5.30	5.30
Bottom of Slab	1.80	1.80

Baffles:	yes	
Roller Gate Weir Elev.:	10.00	ft
Weir Gate Sill Elev.:	13.50	ft
Roller Gate Weir Width.:	13.14	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	40.69	ft
Abutment Length:	23.09	ft
Pier Width:	3.00	ft
Pier Length:	37.19	ft
Pier Elev.:	21.00	ft
Pier Elev. (end):	21.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	34.00	ft
Top of Service Bridge	21.00	ft
Breast Wall Top Elev.:	25.00	ft
Breast Wall Bottom Elev.:	20.00	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	19.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	63.78	ft
End Sill Height:	2.34	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Friction Coefficient:	0.35	
No. of Roller Gates	1.00	
No. of Weir Gates	2.00	

Lane width:	13 ft
Sidewalk:	5.67 ft
Curb:	0.83 ft
Bridge width:	19.50 ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:			
area	6.25	4.53125	
neutral axis		0.725	ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	C.L. of Baffles from toe:
1.50		17.47 ft
1.50		
14.00	10.00	
10	16	

Spillway S-10: Load Case 2B (Maximum Pool Condition with tail water provided in Structure Operations Summary)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	A_{s1}	2		40.7		15.7	2.50	3194.0	479.1			43.4	20810.8	
Abutment 2 (height varies)	A_{s2}	2		23.1		15.7	2.50	1812.9	271.9			11.5	3140.0	
Breast Wall	B_{w2}	1		20.0		6.3	1.00	125.0	18.8			53.3	1000.2	
Bridge	B_d	1		45.0		19.5	1.33	1170.0	175.5			39.5	6937.8	
Column-Support Platform	Cs_1	2		11.5	20.8			478.3	71.7			48.5	3482.7	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			48.5	3081.2	
Platform	PL_s	1		45.0	9.8			440.2	66.0			48.5	3205.0	
Spread Foundation (Apron Slab)	S_s	1		54.0		63.8	3.50	12054.7	1808.2			31.8	57665.0	
Pier	P_s	2		37.2		15.7	3.00	3503.1	525.5			41.7	21905.2	
Weir	Ws2			14.0	38.8			542.9	81.4			49.0	3987.8	
Abutment Triangular Section at Base	A_{s3}				7.1	63.8		449.7	67.4			31.9	2151.0	
Roller Gate	G_s	1		15.6	168.9	10.8			11.0			49.8	548.3	
Weir Gate Sill	Sl_{ws}	2		10.0	15.7	4.7			23.5			49.9	1173.3	
Baffles	B_{rf}	28		1.5	8.0			232.2	34.8			17.5	608.5	
Water on top of Heel (upstream side)	W_c_heel	1		34.0		10.4	14.70	5216.7	325.5			58.6	19063.3	
Water on top of Crest (upstream side)	W_c_crest	1		34.0		2.5	10.00	860.6	53.7			52.1	2796.8	
Water contained on top of Apron (downstream side)	W_c_apron	1		34.0		13.6	2.34	1083.2	67.6			29.9	2021.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		40.0		23.1	2.34	2161.6	134.9			11.5	1557.5	
Water above End Sill (Surcharge Water)	W_g	1		40.0		23.1	14.16	13080.4	816.2			11.5	9424.9	
Head Water Active Pressure (upstream side)	W_c_head water										465.1	6.1		2821.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										303.4	4.9	1486.6	
Uplift Water Pressure	U_d	1		54.0		63.78	14.70			3159.3		42.5		134336.5
Uplift Water Pressure	U_{ut}	1		54.0		63.78	3.50			376.1		31.9		11994.3
Silt Passive Pressure (downstream side)-neglected														
Total									5096.3	3535.4	161.7		166047.0	149152.2

Overturning neglecting Uplift (for bearing check only): 2821.4

W_c:	581.7
W_g:	816.2
W_s:	3698.4
	5096.3

Safety Factors Check	
Against Overturning:	1.11
Against Sliding:	11.03
S.F. against Floatation:	1.57

Lane width:	13	ft
Sidewalk:	5.67	ft
Curb:	0.83	ft
Bridge width:	19.50	ft

Elevations (ft):	Upstream	Downstream
Normal Water		
Maximum Water	20.00	16.50
Top of Slab	5.30	5.30
Bottom of Slab	1.80	1.80

Baffles:	yes	
Roller Gate Weir Elev.:	10.00	ft
Weir Gate Sill Elev.:	13.50	ft
Roller Gate Weir Width.:	13.14	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	40.69	ft
Abutment Length:	23.09	ft
Pier Width:	3.00	ft
Pier Length:	37.19	ft
Pier Elev.:	21.00	ft
Pier Elev. (end):	21.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	34.00	ft
Top of Service Bridge	21.00	ft
Breast Wall Top Elev.:	25.00	ft
Breast Wall Bottom Elev.:	20.00	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	19.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	63.78	ft
End Sill Height:	2.34	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Friction Coefficient:	0.35	
No. of Roller Gates	1.00	
No. of Weir Gates	2.00	

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:		
area	6.25	4.53125
neutral axis		0.725 ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	C.L. of Baffles from toe:
1.50		17.47 ft
1.50		
14.00	10.00	
10	16	

Spillway S-10: Load Case 3A (De-Watered Condition with tail water at weir crest elevation of structure downstream)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft kip)
Abutment 1	A_{S1}	2		40.7		15.7	2.50	3194.0	479.1			43.4	20810.8	
Abutment 2 (height varies)	A_{S2}	2		23.1		15.7	2.50	1812.9	271.9			11.5	3140.0	
Breast Wall	B_{WS}	1		20.0		6.3	1.00	125.0	18.8			53.3	1000.2	
Bridge	B_S	1		45.0		19.5	1.33	1170.0	175.5			39.5	6937.8	
Column-Support Platform	CS_1	2		11.5	20.8			478.3	71.7			48.6	3462.7	
Column-Support Platform	CS_2	2		11.5	18.4			423.1	63.6			48.5	3081.2	
Platform	PLS	1		45.0	9.8			440.2	66.0			48.6	3205.0	
Spread Foundation (Apron Slab)	S_1	1		54.0		63.8	3.50	12054.7	1808.2			31.9	57665.0	
Pier	P_S	2		37.2		15.7	3.00	3503.1	525.5			41.7	21905.2	
Weir	Ws2			14.0	38.8			542.9	81.4			49.0	3987.8	
Abutment Triangular Section at Base	A_{CS}				7.1	63.8		449.7	67.4			31.9	2151.0	
Roller Gate	G_L	1		15.6	168.9	10.8			11.0			49.6	548.3	
Weir Gate Sill	Sl_{wg}	2		10.0	15.7	4.7			23.5			49.9	1173.3	
Baffles	B_{af}	26		1.5	6.0			232.2	34.8			17.5	608.5	
Water on top of Heel (upstream side)	W_c_heel	1		34.0		10.4	13.70	4861.8	303.4			58.6	17766.5	
Water on top of Crest (upstream side)	W_c_crest	1		34.0		2.5	9.00	774.6	48.3			52.1	2517.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		34.0		13.6	2.34	1083.2	67.6			29.9	2021.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		40.0		23.1	2.34	2161.6	134.9			11.5	1557.5	
Water above End Sill (Surcharge Water)	W_g	1		40.0		23.1	6.66	6152.2	383.9			11.5	4432.9	
Head Water Active Pressure (upstream side)	W_c_head water										415.4	5.7		2381.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										72.8	2.4	174.7	
Uplift Water Pressure	U_{ai}	1		54.0		63.78	7.20			1547.4		42.5		65797.4
Uplift Water Pressure	U_{ai}	1		54.0		63.78	10.00			1074.6		31.9		34269.6
Silt Passive Pressure (downstream side)-neglected														
Total									4636.5	2622.0	342.6		158168.6	102448.3

Overturning neglecting Uplift (for bearing check only): 2381.4

W_c:	554.2
W_g:	383.9
W_s:	3698.4
	4636.5

Safety Factors Check:	
S.F. against Overturning:	1.54
S.F. against Sliding:	4.74
S.F. against Floatation:	1.90

Elevations (ft):	Upstream	Downstream
Normal Water	19.00	9.00
Maximum Water		
Top of Slab	5.30	5.30
Bottom of Slab	1.80	1.80

Baffles:	yes	
Roller Gate Weir Elev.:	10.00	ft
Weir Gate Sill Elev.:	13.50	ft
Roller Gate Weir Width.:	13.14	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	40.69	ft
Abutment Length:	23.09	ft
Pier Width:	3.00	ft
Pier Length:	37.19	ft
Pier Elev.:	21.00	ft
Pier Elev. (end):	21.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	34.00	ft
Top of Service Bridge	21.00	ft
Breast Wall Top Elev.:	25.00	ft
Breast Wall Bottom Elev.:	20.00	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	19.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	63.78	ft
End Sill Height:	2.34	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Friction Coefficient:	0.35	
No. of Roller Gates	1.00	
No. of Weir Gates	2.00	

Lane width:	13 ft
Sidewalk:	5.67 ft
Curb:	0.83 ft
Bridge width:	19.50 ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:		
area	6.25	4.53125
neutral axis		0.725 ft

Considerations:

1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	C.L. of Baffles from toe:
1.50		17.47 ft
1.50		
14.00	10.00	
10	16	

Spillway S-10: Load Case 3B (De-Watered Condition with tail water provided in Structure Operations Summary)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{st} (ft-kip)
Abutment 1	Ab_1	2		40.7		15.7	2.50	3194.0	479.1			43.4	20810.8	
Abutment 2 (height varies)	Ab_2	2		23.1		15.7	2.50	1812.9	271.9			11.5	3140.0	
Breast Wall	B_{w2}	1		20.0		6.3	1.00	125.0	18.8			53.3	1000.2	
Bridge	B_d	1		45.0		19.5	1.33	1170.0	175.5			39.5	6937.8	
Column-Support Platform	Cs_1	2		11.5	20.8			478.3	71.7			48.5	3462.7	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			48.5	3081.2	
Platform	PLs	1		45.0	9.8			440.2	66.0			48.5	3205.0	
Spread Foundation (Apron Slab)	S_f	1		54.0		63.8	3.50	12054.7	1808.2			31.9	57685.0	
Pier	P_s	2		37.2		15.7	3.00	3503.1	525.5			41.7	21905.2	
Weir	Ws2			14.0	38.8			542.9	81.4			49.0	3987.8	
Abutment Triangular Section at Base	A_3				7.1	63.8		449.7	67.4			31.9	2151.0	
Roller Gate	G_s	1		15.6	168.9	10.8			11.0			49.9	548.3	
Weir Gate Sill	Sl_{wg}	2		10.0	15.7	4.7			23.5			49.9	1173.3	
Baffles	B_d	26		1.5	6.0			232.2	34.8			17.5	608.5	
Water on top of Heel (upstream side)	W_c_heel	1		34.0		10.4	13.70	4861.8	303.4			58.6	17766.5	
Water on top of Crest (upstream side)	W_c_crest	1		34.0		2.5	9.00	774.6	48.3			52.1	2517.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		34.0		13.6	2.34	1083.2	67.6			29.9	2021.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		40.0		23.1	2.34	2161.6	134.9			11.5	1557.5	
Water above End Sill (Surcharge Water)	W_s	1		40.0		23.1	12.66	11694.8	729.8			11.5	8428.5	
Head Water Active Pressure (upstream side)	W_c_head water										415.4	5.7		2381.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										244.6	4.4	1076.4	
Uplift Water Pressure	U_d	1		54.0		63.78	13.20			2836.9		42.5		120628.7
Uplift Water Pressure	U_s	1		54.0		63.78	4.00			429.8		31.8		13707.8
Silt Passive Pressure (downstream side)-neglected														
Total									4982.3	3266.8	170.7		163061.9	136717.8

Overturning neglecting Uplift (for bearing check only): 2381.4

W_c	554.2
W_g	729.8
W_s	3698.4
	4982.3

Safety Factors Check:	
S.F. against Overturning:	1.19
S.F. against Sliding:	10.21
S.F. against Floatation:	1.68

Elevations (ft):	Upstream	Downstream
Normal Water	19.00	15.00
Maximum Water		
Top of Slab	5.30	5.30
Bottom of Slab	1.80	1.80

Baffles:	yes	
Roller Gate Weir Elev.:	10.00	ft
Weir Gate Sill Elev.:	13.50	ft
Roller Gate Weir Width.:	13.14	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	40.69	ft
Abutment Length:	23.09	ft
Pier Width:	3.00	ft
Pier Length:	37.19	ft
Pier Elev.:	21.00	ft
Pier Elev. (end):	21.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	34.00	ft
Top of Service Bridge	21.00	ft
Breast Wall Top Elev.:	25.00	ft
Breast Wall Bottom Elev.:	20.00	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	19.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	63.78	ft
End Sill Height:	2.34	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Friction Coefficient:	0.35	
No. of Roller Gates	1.00	
No. of Weir Gates	2.00	

Lane width:	13	ft
Sidewalk:	5.67	ft
Curb:	0.83	ft
Bridge width:	19.50	ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:		
area	6.25	4.53125
neutral axis		0.725 ft

Considerations:
1. Weir area obtaninded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamartes comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	C.L. of Baffles from toe:
1.50		
1.50		
		17.47 ft
14.00	10.00	
10	16	

Spillway S-10: Stability Check

Reference Files

References

EC 1110-2-6058 Stability Requirements, USACE 30 November 2003
ETL 11102-307 Engineerin and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987
Design of Small Dams, published by United State Department of the Interior, Bureau of Reclamation; A Water Resources Technical Publication.
Geotechnical Report by DE&T, dated 1/06/06

Units:

$psi := \frac{lbf}{in^2}$ $psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $kip := 1000lbf$ $ksi := \frac{kip}{in^2}$ $ksf := \frac{kip}{ft^2}$ $kcf := \frac{kip}{ft^3}$

Constants

$\gamma_c := 150 \cdot pcf$ $\gamma_{sat} := 110pcf$ $C := 500psf$ (assumed)

$\gamma_w := 62.4pcf$ $\gamma_{steel} := 490pcf$ $\phi := 30deg$ (assumed)

Structure Description:

Reinforced Concrete Spillway Structure with one (1) Sluice Gates (i.e., Roller Gates) and two (2) Weir Gates (i.e., Vertical Double-Slide Aluminum Gates) to regulate water flow around Perimeter Canal.

Design Criteria

Allowable Stability Factors by USACE; Construction Materials in accordance with ASTM Standards; Allowable Stress Design. Design for a 100 years event (S. Gaitan correspondence 4/21/06). A friction coefficient of 0.35 will be assumed based on Geotechnical Engineer's recommendations. Hydraulic data based on Structure Operations Summaries by C. Zamaites dated 5/4/06.

$SF_{Sliding\_USACE} := 1.3$ $SF_{Floating\_USACE} := 1.2$

$q_{max\_allow} := 6900psf$

Load Case 1A (Normal Pool with tail water at weir crest elev. of structure downstream):

Load Case 1B (Normal Pool with tail water provided in Structure Oper. Summaries):

Elevations (ft):			
	Upstream	Downstream	
		1A	1B
Normal Water	19.00	9.00	15.00
Top of Slab	5.30	5.30	5.30
Bottom of Slab	1.80	1.80	1.80
Baffles:	yes		
Roller Gate Weir Elev.:	10.00	ft	
Weir Gate Sill Elev.:	13.50	ft	
Roller Gate Weir Width.:	13.14	ft	
Roller Gate Weir Length:	14.00	ft	
Weir Gate Sill Length:	10.00	ft	
Weir Gate Sill Width:	1.67	ft	
Abutment Width:	2.50	ft	
Abutment Length:	40.69	ft	
Abutment Length:	23.09	ft	
Pier Width:	3.00	ft	
Pier Length:	37.19	ft	
Pier Elev.:	21.00	ft	
Pier Elev. (end):	21.00	ft	
Platform Thickness:	1.50	ft	
Platform Width:	8.00	ft	
Heel Length:	10.44	ft	
Apron's Overhang	4.50	ft	
Top of Operating Platform	34.00	ft	
Top of Service Bridge	21.00	ft	
Breast Wall Top Elev.:	25.00	ft	
Breast Wall Bottom Elev.:	20.00	ft	
Breast Wall Thickness:	1.00	ft	
Bridge Width:	19.50	ft	
Bridge Thickness:	1.33	ft	
Apron Slab (overall length):	63.78	ft	
End Sill Height:	2.34	ft	
Apron Slab Thickness:	3.50	ft	
Skin Friction Pressure:	500.00	psf	
Friction Coefficient	0.35		
No. of Roller Gates	1.00		
No. of Weir Gates	2.00		

Sliding Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$\mu := 0.35$ friction coefficient for sand

$W1A := 4636.5\,kip$ $W1B := 4982.3\,kip$ weight of the structure

$Apron\_length := 63.78\,ft$

$Apron\_width := 54.0\,ft$

$Active\_Force1A := 342.6\,kip$ $Active\_Force1B := 170.7\,kip$

$A_{apron\_slab} := Apron\_length \cdot Apron\_width$

$SF_{Sliding1A} := \dfrac{\mu \cdot W1A}{Active\_Force1A}$ $SF_{Sliding1A} = 4.737$

$Check\_Sliding1A := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding1A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding1A = \text{"O.K."}$

$SF_{Sliding1B} := \dfrac{\mu \cdot W1B}{Active\_Force1B}$ $SF_{Sliding1B} = 10.216$

$Check\_Sliding1B := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding1A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding1B = \text{"O.K."}$

Floating (Uplift) Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$W_s := 3698.4kip$ weight of the structure

$W_c := 554.2kip$ weight of the water contained within the structur

$U1A := 2622.0kip$ $U1B := 3266.8kip$ uplift force acting on the base of the structure

$S := 0$ surcharge loads

$W_{g1A} := 383.9kip$ $W_{g1B} := 729.8kip$ weight of the surcharge water

$$SF_{Floating1A} := \frac{W_s + W_c + S}{U1A - W_{g1A}} \qquad SF_{Floating1A} = 1.9$$

$$Check\_Floating1A := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating1A} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating1A = \text{"O.K."}$$

$$SF_{Floating1B} := \frac{W_s + W_c + S}{U1B - W_{g1B}} \qquad SF_{Floating1B} = 1.676$$

$$Check\_Floating1B := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating1B} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating1B = \text{"O.K."}$$

Bearing Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$\Sigma M_{R1A} := 158166.6\,ft\cdot kip$ $\Sigma M_{R1B} := 163061.9\,ft\cdot kip$

$\Sigma M_{OT} := 2381.4\,ft\cdot kip$

$\Sigma W := W_c + W_s$ $\Sigma W = 4252.6\ kip$

$e1A := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R1A} - \Sigma M_{OT}}{\Sigma W} \right|$ $e1A = 4.743\ ft$

$Check\_eccentricity1A := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e1A \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$

Check_eccentricity1A = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$q_{max1A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e1A}{\frac{A_{apron\_slab}}{ft}} \right)$ $q_{max1A} = 1.245\ ksf$

$Check\_q_{max1A} := \begin{cases} \text{"O.K."} & \text{if } q_{max1A} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases}$ $Check\_q_{max1A} = \text{"O.K."}$

$q_{min1A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e1A}{\frac{A_{apron\_slab}}{ft}} \right)$ $q_{min1A} = 1.225\ ksf$

$Check\_q_{min1A} := \begin{cases} \text{"O.K."} & \text{if } q_{min1A} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases}$ $Check\_q_{min1A} = \text{"O.K."}$

$$e1B := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R1B} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e1B = 5.894\ ft$$

$$Check\_eccentricity1B := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e1B \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity1B = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max1B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e1B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max1B} = 1.247\ ksf$$

$$Check\_q_{max1B} := \begin{cases} \text{"O.K."} & \text{if } q_{max1B} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max1B} = \text{"O.K."}$$

$$q_{min1B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e1B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min1B} = 1.222\ ksf$$

$$Check\_q_{min1B} := \begin{cases} \text{"O.K."} & \text{if } q_{min1B} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min1B} = \text{"O.K."}$$

Load Case 2A (Max. Pool with tail water at weir crest elev. of structure downstream):
Load Case 2B (Max. Pool with tail water provided in Structure Oper. Summaries):

Elevations (ft):			
	Upstream	Downstream	
		2A	2B
Maximum Water	20.00	9.00	16.50
Top of Slab	5.30	5.30	5.30
Bottom of Slab	1.80	1.80	1.80
Baffles:	yes		
Roller Gate Weir Elev.:	10.00	ft	
Weir Gate Sill Elev.:	13.50	ft	
Roller Gate Weir Width.:	13.14	ft	
Roller Gate Weir Length:	14.00	ft	
Weir Gate Sill Length:	10.00	ft	
Weir Gate Sill Width:	1.67	ft	
Abutment Width:	2.50	ft	
Abutment Length:	40.69	ft	
Abutment Length:	23.09	ft	
Pier Width:	3.00	ft	
Pier Length:	37.19	ft	
Pier Elev.:	21.00	ft	
Pier Elev. (end):	21.00	ft	
Platform Thickness:	1.50	ft	
Platform Width:	8.00	ft	
Heel Length:	10.44	ft	
Apron's Overhang	4.50	ft	
Top of Operating Platform	34.00	ft	
Top of Service Bridge	21.00	ft	
Breast Wall Top Elev.:	25.00	ft	
Breast Wall Bottom Elev.:	20.00	ft	
Breast Wall Thickness:	1.00	ft	
Bridge Width:	19.50	ft	
Bridge Thickness:	1.33	ft	
Apron Slab (overall length):	63.78	ft	
End Sill Height:	2.34	ft	
Apron Slab Thickness:	3.50	ft	
Skin Friction Pressure:	500.00	psf	
Friction Coefficient	0.35		
No. of Roller Gates	1.00		
No. of Weir Gates	2.00		

Sliding Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$\mu := 0.35$ friction coefficient for sand

$W2A := 4664.0\,kip$ $W2B := 5096.3\,kip$ weight of the structure

$Apron\_length := 63.78\,ft$

$Apron\_width := 54.0\,ft$

$Active\_Force2A := 392.3\,kip$ $Active\_Force2B := 161.7\,kip$

$A_{apron\_slab} := Apron\_length \cdot Apron\_width$

$SF_{Sliding2A} := \dfrac{\mu \cdot W2A}{Active\_Force2A}$ $SF_{Sliding2A} = 4.161$

$Check\_Sliding2A := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding2A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding2A = \text{"O.K."}$

$SF_{Sliding2B} := \dfrac{\mu \cdot W2B}{Active\_Force2B}$ $SF_{Sliding2B} = 11.031$

$Check\_Sliding2B := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding2B} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding2B = \text{"O.K."}$

Floating (Uplift) Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$W_s := 3698.4kip$ — weight of the structure

$W_c := 581.7kip$ — weight of the water contained within the structu

$U2A := 2729.5kip$ $U2B := 3535.4kip$ — uplift force acting on the base of the structure

$S := 0$ — surcharge loads

$W_{g2A} := 383.9kip$ $W_{g2B} := 816.2kip$ — weight of the surcharge water

$$SF_{Floating2A} := \frac{W_s + W_c + S}{U2A - W_{g2A}} \qquad SF_{Floating2A} = 1.825$$

$$Check\_Floating2A := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating2A} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating2A = \text{"O.K."}$$

$$SF_{Floating2B} := \frac{W_s + W_c + S}{U2B - W_{g2B}} \qquad SF_{Floating2B} = 1.574$$

$$Check\_Floating2B := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating2B} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating2B = \text{"O.K."}$$

Bearing Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$\Sigma M_{R2A} := 159743.1\ ft \cdot kip$ $\Sigma M_{R2B} := 166047.0\ ft \cdot kip$

$\Sigma M_{OT} := 2821.4\ ft \cdot kip$

$\Sigma W := W_c + W_s$ $\Sigma W = 4280.1\ kip$

$$e2A := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R2A} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e2A = 4.773\ ft$$

$$Check\_eccentricity2A := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e2A \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity2A = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max2A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e2A}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max2A} = 1.253\ ksf$$

$$Check\_q_{max2A} := \begin{cases} \text{"O.K."} & \text{if } q_{max2A} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max2A} = \text{"O.K."}$$

$$q_{min2A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e2A}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min2A} = 1.232\ ksf$$

$$Check\_q_{min2A} := \begin{cases} \text{"O.K."} & \text{if } q_{min1A} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min2A} = \text{"O.K."}$$

$$e2B := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R2B} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e2B = 6.246\ ft$$

$$Check\_eccentricity2B := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e2B \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity2B = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max2B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e2B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max2B} = 1.256\ ksf$$

$$Check\_q_{max2B} := \begin{cases} \text{"O.K."} & \text{if } q_{max2B} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max2B} = \text{"O.K."}$$

$$q_{min2B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e2B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min2B} = 1.229\ ksf$$

$$Check\_q_{min2B} := \begin{cases} \text{"O.K."} & \text{if } q_{min2B} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min2B} = \text{"O.K."}$$

Load Case 3A (De-Watered with tail water at weir crest elev. of structure downstream):
Load Case 3B (De-Watered with tail water provided in Structure Oper. Summaries):

Elevations (ft):			
	Upstream	Downstream	
		3A	3B
Maximum Water	19.00	9.00	15.00
Top of Slab	5.30	5.30	5.30
Bottom of Slab	1.80	1.80	1.80
Baffles:	yes		
Roller Gate Weir Elev.:	10.00	ft	
Weir Gate Sill Elev.:	13.50	ft	
Roller Gate Weir Width.:	13.14	ft	
Roller Gate Weir Length:	14.00	ft	
Weir Gate Sill Length:	10.00	ft	
Weir Gate Sill Width:	1.67	ft	
Abutment Width:	2.50	ft	
Abutment Length:	40.69	ft	
Abutment Length:	23.09	ft	
Pier Width:	3.00	ft	
Pier Length:	37.19	ft	
Pier Elev.:	21.00	ft	
Pier Elev. (end):	21.00	ft	
Platform Thickness:	1.50	ft	
Platform Width:	8.00	ft	
Heel Length:	10.44	ft	
Apron's Overhang	4.50	ft	
Top of Operating Platform	34.00	ft	
Top of Service Bridge	21.00	ft	
Breast Wall Top Elev.:	25.00	ft	
Breast Wall Bottom Elev.:	20.00	ft	
Breast Wall Thickness:	1.00	ft	
Bridge Width:	19.50	ft	
Bridge Thickness:	1.33	ft	
Apron Slab (overall length):	63.78	ft	
End Sill Height:	2.34	ft	
Apron Slab Thickness:	3.50	ft	
Skin Friction Pressure:	500.00	psf	
Friction Coefficient	0.35		
No. of Roller Gates	1.00		
No. of Weir Gates	2.00		

Sliding Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$\mu := 0.35$ friction coefficient for sand

$W3A := 4636.5kip$ $W3B := 4982.3kip$ weight of the structure

$Apron\_length := 63.78ft$

$Apron\_width := 54.0ft$

$Active\_Force3A := 342.6kip$ $Active\_Force3B := 170.7kip$

$A_{apron\_slab} := Apron\_length \cdot Apron\_width$

$SF_{Sliding3A} := \dfrac{\mu \cdot W3A}{Active\_Force3A}$ $SF_{Sliding3A} = 4.737$

$Check\_Sliding3A := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding3A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding3A = \text{"O.K."}$

$SF_{Sliding3B} := \dfrac{\mu \cdot W3B}{Active\_Force3B}$ $SF_{Sliding3B} = 10.216$

$Check\_Sliding3B := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding3B} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding3B = \text{"O.K."}$

Floating (Uplift) Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$W_s := 3698.4kip$ — weight of the structure

$W_c := 554.2kip$ — weight of the water contained within the structu

$U3A := 2622.0kip$ $U3B := 3266.8kip$ — uplift force acting on the base of the structure

$S := 0$ — surcharge loads

$W_{g3A} := 383.9kip$ $W_{g3B} := 729.8kip$ — weight of the surcharge water

$$SF_{Floating3A} := \frac{W_s + W_c + S}{U3A - W_{g3A}} \qquad SF_{Floating3A} = 1.9$$

$$Check\_Floating3A := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating3A} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating3A = \text{"O.K."}$$

$$SF_{Floating3B} := \frac{W_s + W_c + S}{U3B - W_{g3B}} \qquad SF_{Floating3B} = 1.676$$

$$Check\_Floating3B := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating3B} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating3B = \text{"O.K."}$$

Bearing Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-10":

$\Sigma M_{R3A} := 158166.6\,ft\cdot kip$ $\Sigma M_{R3B} := 163061.9\,ft\cdot kip$

$\Sigma M_{OT} := 2381.4\,ft\cdot kip$

$\Sigma W := W_c + W_s$ $\Sigma W = 4252.6\ kip$

$e3A := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R3A} - \Sigma M_{OT}}{\Sigma W} \right|$ $e3A = 4.743\ ft$

$Check\_eccentricity3A := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e3A \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$

Check_eccentricity2A = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$q_{max3A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e3A}{\frac{A_{apron\_slab}}{ft}} \right)$ $q_{max3A} = 1.245\ ksf$

$Check\_q_{max3A} := \begin{cases} \text{"O.K."} & \text{if } q_{max3A} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases}$ $Check\_q_{max3A} = \text{"O.K."}$

$q_{min3A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e3A}{\frac{A_{apron\_slab}}{ft}} \right)$ $q_{min3A} = 1.225\ ksf$

$Check\_q_{min3A} := \begin{cases} \text{"O.K."} & \text{if } q_{min3A} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases}$ $Check\_q_{min3A} = \text{"O.K."}$

$$e3B := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R3B} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e3B = 5.894\ ft$$

$$Check\_eccentricity3B := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e3B \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity3B = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max3B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e3B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max3B} = 1.247\ ksf$$

$$Check\_q_{max3B} := \begin{cases} \text{"O.K."} & \text{if } q_{max3B} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max3B} = \text{"O.K."}$$

$$q_{min3B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e3B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min3B} = 1.222\ ksf$$

$$Check\_q_{min3B} := \begin{cases} \text{"O.K."} & \text{if } q_{min3B} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min3B} = \text{"O.K."}$$

Summary:

Sliding Check:

Check_Sliding1A = "O.K."

Check_Sliding2A = "O.K."

Check_Sliding3A = "O.K."

Check_Sliding1B = "O.K."

Check_Sliding2B = "O.K."

Check_Sliding3B = "O.K."

Floating (uplift) Check:

Check_Floating1A = "O.K."

Check_Floating2A = "O.K."

Check_Floating3A = "O.K."

Check_Floating1B = "O.K."

Check_Floating2B = "O.K."

Check_Floating3B = "O.K."

Bearing Pressures (q_{max}, q_{min}):

Check_eccentricity1A = "O.K. The entire Foundation is in Compression"

Check_eccentricity1B = "O.K. The entire Foundation is in Compression"

Check_eccentricity2A = "O.K. The entire Foundation is in Compression"

Check_eccentricity2B = "O.K. The entire Foundation is in Compression"

Check_eccentricity3A = "O.K. The entire Foundation is in Compression"

Check_eccentricity3B = "O.K. The entire Foundation is in Compression"

Check_q_{max1A} = "O.K."

Check_q_{min1A} = "O.K."

Check_q_{max2A} = "O.K."

Check_q_{min2A} = "O.K."

Check_q_{min3A} = "O.K."

Check_q_{max1B} = "O.K."

Check_q_{min1B} = "O.K."

Check_q_{max2B} = "O.K."

Check_q_{min2B} = "O.K."

Check_q_{min3B} = "O.K."

Stanley Consultants, Inc.		Page No._____.
Computed by: AZI	Comp Date: 4/14/06	Project No.17300
Checked by: RJV	Print Date: 5/30/2006	Berry Groves C-43 Reservoir Spillway S-10
Approved by: _____	Print Time: 6:00 PM	Sheet 18 of 18

Filename: Stability Check Spillway S-10_30% Design_1gate & 2weir gates_5-4-06.xmcd

Quantity take-off for Structure S-10

Item	Quantity	Length	Width	Thickness/ Height	Linear	Areas	Volume	Volume	Weight
		ft	ft	ft	ft	sft	cft	CY	tons
Spread Footing:									
Formwork									
Concrete									
Reiforcement									
Finishes									
SOG:									
Bridge Approach Slab:									
Formwork	4	10	13	1		92.00			
Concrete	2	10	13	1		260.00	260.00	9.63	
Reiforcement	0.015						3.90		0.96
Finishes						260.00			
Ladder Platform:									
Formwork	2	6	8	0.5		14.00			
Concrete	1	6	8	0.5		48.00	24.00	0.89	
Reiforcement	0.008						0.19		0.05
Finishes						48.00			
Apron Slab:									
Formwork	2	63.78	54.00	3.50		824.47			
Concrete	1			3.50		3444.20	12054.70	446.47	
Reiforcement	0.0214						257.73		63.14
Finishes		63.78	54.00			3289.37			
Walls:									
Abutment:									
Formwork	4	66.28		15.70		4162.48			
Concrete	2	63.78	2.50	15.70			5006.85	185.44	
Reiforcement	0.015						75.10		18.40
Finishes						4481.3857			
Pier:									
Formwork	6	40.19		15.70		3785.6625			
Concrete	3	37.19	3.00	15.70			5254.59	194.61	
Reiforcement	0.015						78.82		19.31
Finishes						4120.35			
Columns (Supporting Platform):									
At Abutments:									
Formwork	4	10.50		11.50		483.00			
Concrete	1						423.14	15.67	
Reiforcement	0.025						10.58		2.59
Finishes						483.00			
At Piers:									
Formwork	4	11.00		11.50		506.00			
Concrete	1						478.29	17.71	
Reiforcement	0.025						11.96		2.93
Finishes						506.00			
Weir (under roller gate):									
Formwork	1	14		4.70		65.8			
Concrete							542.93	20.11	
Reiforcement	0.015						8.14		2.00
Finishes						281.11			

Quantity take-off for Structure S-10

Stanley Consultants, Inc.
Computed by:
Checked by:
Approved by:
File name:Q:\17300\07-Design\03-CostEst\Structures 30% Submittal Quantity Take-off\Quantity Take off for S-3, S-10 & S-11.xls

Comp date: 5/16/06
Print date: 5/30/2006
Print time: 5:37 PM

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C-43 West Storage Reservoir

Item		Quantity	Length ft	Width ft	Thickness/ Height ft	Linear ft	Areas sft	Volume cft	Volume CY	Weight tons
Sill (under weir gate):										
	Formwork	2	20		4.70		188			
	Concrete							156.67	5.80	
	Reiforcement	0.025						3.92		0.96
	Finishes						221.33			
Baffles:										
	Formwork	26	9.76		1.50		380.68			
	Concrete	26			1.50		5.95	232.24	8.60	
	Reiforcement	0.008						1.86		0.46
	Finishes						535.51			
Elevated Slabs:										
	Service bridge slab:									
	Formwork	1	45	19.5			877.5			
	Edge	2	45		1.83		165.00			
	Flat									
	Concrete	1	45	19.5	1.33		877.50	1316.25	48.75	
	Reiforcement	0.025						32.91		8.06
	Finishes		38	51	1.33		1920.00			
	Operating Platform:									
	Formwork	1	45	8.00			360			
	Edge	2	45		1.50		135.00			
	Flat									
	Concrete	1	45	8.00	1.50		360.00	652.50	24.17	
	Reiforcement	0.025						16.31		4.00
	Finishes		38	51	1.33		990.00			
Breast Wall:										
	Formwork	2	20	1.00	6.25		270			
	Concrete	1						125	4.63	
	Reiforcement	0.015						1.88		0.46
	Finishes						290			
Sheet Piling:										
	Sheetpiles:									
	Peripheric locking sheet piles	0	63.78		6.00		0.00			
	Peripheric locking sheet piles (tremie)	2	75.78		5.00		757.82			
	Peripheric locking sheet piles (up/downstream cutoff wall)	1	54.00		6.00		324.00			
	Peripheric locking sheet piles (tremie)	2	65.00		5.00		650.00			
	Upstream wingwalls	2	30.00		27.20		2448.00			
	Downstream wingwalls	2	38.50		24.20		2795.10			
	Pile Cap:									
	Formwork	4	30.00	2	2.00		720.00			
	Concrete	4	30.00	2.00	2.00			480.00	17.78	
	Reiforcement	0.008						3.84		0.94
	Finishes						960.00			
	Seal Concrete:									
	Concrete (poor strength)	1	75.78	65	5.00			24628.99	912.18	
Control Building:										
	Precast Control Bldg.	1								
Excavation:										
	Excavation	1	75.78	65.00	29.90			147281.35	5454.86	
Erosion Control System:										
	Erosion Control Fabric	4	20	51.70			6295.81			
	Bedding Rock	1			0.5			3147.90	116.59	
	Rip-rap	1			1.5			9443.71	349.77	

Spillway S-11: Load Case 1A (Normal Pool Condition with tail water at weir crest elevation of structure downstream)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	As_1	2		52.3		21.4	2.50	5590.8	838.6			50.9	42725.6	
Abutment 2 (height varies)	As_2	2		24.8		21.4	2.50	2656.1	398.4			12.4	4944.8	
Breast Wall	B_{ws}	1		20.0		5.8	1.00	115.0	17.3			66.6	1149.5	
Bridge	Bs	1		62.0		30.5	1.33	2521.3	378.2			46.8	17684.8	
Column-Support Platform	Cs_1	3		11.5	20.8			717.4	107.6			61.8	6654.4	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			61.8	3924.8	
Platform	PLs	1		62.0	9.8			606.4	91.0			61.8	5624.9	
Spread Foundation (Apron Slab)	S_t	1		71.0		77.1	3.50	19152.6	2872.9			38.5	110711.1	
Pier	P_1	3		49.3		21.4	3.00	9485.6	1422.8			49.4	70356.1	
Weir	Ws2			28.0	156.7			4388.5	658.3			60.3	39722.3	
Abutment Triangular Section at Base	As_3				20.1	77.1		1549.2	232.4			38.5	8954.9	
Roller Gate	G_r	2		15.6	197.3	6.3			12.8			63.2	810.6	
Weir Gate Sill	Sl_{wg}	2		10.0	46.3	13.9			69.5			63.2	4393.7	
Baffles	B_a	0		1.5	6.0			0.0	0.0			19.2	0.0	
Water on top of Heel (upstream side)	W_c_heel	1		48.0		10.4	19.40	9719.4	606.5			71.9	43578.9	
Water on top of Crest (upstream side)	W_c_crest	1		48.0		2.5	6.00	729.0	45.5			65.4	2973.6	
Water contained on top of Apron (downstream side)	W_c_apron	1		48.0		19.9	2.61	2497.6	155.9			34.8	5422.3	
Water contained on top of Apron (downstream side)	W_c_apron	1		57.0		24.8	2.61	3692.9	230.4			12.4	2860.1	
Water above End Sill (Surcharge Water)	W_s	1		57.0		24.8	0.00	0.0	0.0			12.4	0.0	
Head Water Active Pressure (upstream side)	W_c_head water										1014.4	7.6		7743.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										204.0	3.4	698.5	
Uplift Water Pressure	U_d	1		71.0		77.07	10.27			3506.8		51.4		-180187.9
Uplift Water Pressure	U_{up}	1		71.0		77.07	12.63			2156.3		38.5		83097.9
Silt Passive Pressure (downstream side)-neglected														
Total									8201.5	5663.2	810.4		373190.8	271029.2

Overturning neglecting Uplift (for bearing check only): 7743.4

W_c	1038.3
W_g	0.0
W_r	7163.2
	8201.5

Safety Factors Check:	
S.F. against Overturning:	1.38
S.F. against Sliding:	2.25
S.F. agaist Floatation:	1.45

Elevations (ft):	Upstream	Downstream
Normal Water	15.00	2.37
Maximum Water		
Top of Slab	-4.40	-4.40
Bottom of Slab	-7.90	-7.90

Baffles:	no	
Roller Gate Weir Elev.:	9.00	ft
Weir Gate Sill Elev.:	9.50	ft
Roller Gate Weir Width.:	18.88	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	52.25	ft
Abutment Length:	24.82	ft
Pier Width:	3.00	ft
Pier Length:	49.25	ft
Pier Elev.:	17.00	ft
Pier Elev. (end):	17.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	30.00	ft
Top of Service Bridge	17.00	ft
Breast Wall Top Elev.:	19.00	ft
Breast Wall Bottom Elev.:	14.50	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	30.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	77.07	ft
End Sill Height:	2.61	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Skin Friction Reduction Factor:	0.67	
No. of Roller Gates	2.00	
No. of Weir Gates	2.00	

Lane width:	24 ft
Sidewalk:	5.67 ft
Curb:	0.83 ft
Bridge width:	30.5 ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:			
area	5.75	4.28125	
neutral axis		0.744565217	ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	
1.50		
1.50		C.L. of Baffles from toe:
		19.20 ft
14.00	10.00	
20	16	

Spillway S-11: Load Case 1B (Normal Pool Condition with tail water provided in Structure Operations Summary)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	A_{S1}	2		52.3		21.4	2.50	5590.8	838.6			50.9	42725.6	
Abutment 2 (height varies)	A_{S2}	2		24.6		21.4	2.50	2656.1	398.4			12.4	4944.8	
Breast Wall	B_{WS}	1		20.0		5.8	1.00	115.0	17.3			66.6	1149.5	
Bridge	B_S	1		62.0		30.5	1.33	2521.3	378.2			46.8	17684.8	
Column-Support Platform	Ce_1	3		11.5	20.6			717.4	107.6			61.8	6654.4	
Column-Support Platform	Ce_2	2		11.5	16.4			423.1	63.5			61.8	3924.8	
Platform	PL_S	1		62.0	9.8			606.4	91.0			61.8	5624.9	
Spread Foundation (Apron Slab)	S_A	1		71.0		77.1	3.50	19152.6	2872.9			38.5	110711.1	
Pier	P_1	3		49.3		21.4	3.00	9485.6	1422.8			49.4	70356.1	
Weir	Ws2			28.0	156.7			4388.5	658.3			60.3	39722.3	
Abutment Triangular Section at Base	A_{C3}				20.1	77.1		1549.2	232.4			38.5	8954.9	
Roller Gate	G_1	2		15.6	197.3	6.3			112.8			63.2	610.6	
Weir Gate Sill	Sl_{wg}	2		10.0	46.3	13.9			69.5			63.2	4393.7	
Baffles	B_d	0		1.5	8.0			0.0	0.0			0.0	0.0	
Water on top of Heel (upstream side)	W_c_heel	1		48.0		10.4	19.40	9719.4	606.5			71.9	43578.9	
Water on top of Crest (upstream side)	W_c_crest	1		48.0		2.5	6.00	729.0	45.5			65.4	2973.6	
Water contained on top of Apron (downstream side)	W_c_apron	1		48.0		19.9	2.61	2497.6	155.9			34.8	5422.3	
Water contained on top of Apron (downstream side)	W_c_apron	1		57.0		24.8	2.61	3692.9	230.4			12.4	2860.1	
Water above End Sill (Surcharge Water)	W_s	1		57.0		24.8	1.89	2674.2	166.9			12.4	2071.1	
Head Water Active Pressure (upstream side)	W_c_head water										1014.4	7.6		7743.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										297.4	4.1	1229.4	
Uplift Water Pressure	U_1	1		71.0		77.07	12.40			4234.2		51.4		217558.9
Uplift Water Pressure	U_{1a}	1		71.0		77.07	10.50			1792.7		38.5		69083.7
Silt Passive Pressure (downstream side)-neglected														
Total									8368.4	6026.8	717.0		375792.8	294386.1

Overturning neglecting Uplift (for bearing check only): 7743.4

W_c	1038.3
W_g	166.9
W_c	7163.2
	8368.4

Safety Factors Check:	
S.F. against Overturning:	1.28
S.F. against Sliding:	2.54
S.F. agaist Floatation:	1.40

Elevations (ft):	Upstream	Downstream
Normal Water	15.00	4.50
Maximum Water		
Top of Slab	-4.40	-4.40
Bottom of Slab	-7.90	-7.90

Baffles:	no	
Roller Gate Weir Elev.:	9.00	ft
Weir Gate Sill Elev.:	9.50	ft
Roller Gate Weir Width.:	18.88	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	52.25	ft
Abutment Length:	24.82	ft
Pier Width:	3.00	ft
Pier Length:	49.25	ft
Pier Elev.:	17.00	ft
Pier Elev. (end):	17.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	30.00	ft
Top of Service Bridge	17.00	ft
Breast Wall Top Elev.:	19.00	ft
Breast Wall Bottom Elev.:	14.50	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	30.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	77.07	ft
End Sill Height:	2.61	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Skin Friction Reduction Factor:	0.67	
No. of Roller Gates	2.00	
No. of Weir Gates	2.00	

Lane width:	24 ft
Sidewalk:	5.67 ft
Curb:	0.83 ft
Bridge width:	30.5 ft

Breast Wall:		
area	5.75	4.28125
neutral axis		0.744565217 ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
1.50	1.50	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	
1.50		ft
1.50		C.L. of Baffles from toe:
		19.20
14.00	10.00	
20	16	

Spillway S-11: Load Case 2A (Maximum Pool Condition with tail water at weir crest elevation of structure downstream)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	A_{b1}	2		52.3		21.4	2.50	5590.8	838.6			50.9	42725.6	
Abutment 2 (height varies)	A_{b2}	2		24.8		21.4	2.50	2656.1	398.4			12.4	4944.8	
Breast Wall	B_{ws}	1		20.0		5.8	1.00	115.0	17.3			66.6	1149.5	
Bridge	B_s	1		62.0		30.5	1.33	2521.3	378.2			46.8	17684.8	
Column-Support Platform	Cs_1	3		11.5	20.8			717.4	107.6			61.6	6954.4	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			61.6	3924.8	
Platform	PLs	1		62.0	9.8			606.4	91.0			61.6	5624.9	
Spread Foundation (Apron Slab)	S_a	1		71.0		77.1	3.50	19152.8	2872.9			38.5	110711.1	
Pier	P_s	3		49.3		21.4	3.00	9485.6	1422.8			49.4	70356.1	
Weir	Ws2			28.0	156.7			4388.5	658.3			60.3	39722.3	
Abutment Triangular Section at Base	A_{c1}				20.1	77.1		1549.2	232.4			38.5	8954.9	
Roller Gate	G_s	2		15.6	197.3	6.3			12.8			63.2	810.6	
Weir Gate Sill	Sl_{wg}	2		10.0	46.3	13.9			69.5			63.2	4393.7	
Baffles	B_d	0		1.5	6.0			0.0	0.0			0.0	0.0	
Water on top of Heel (upstream side)	W_c_heel	1		48.0		10.4	20.90	10470.9	653.4			71.9	46948.4	
Water on top of Crest (upstream side)	W_c_crest	1		48.0		2.5	7.50	911.3	56.9			65.4	3717.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		48.0		19.9	2.61	2497.6	155.9			34.8	5422.3	
Water contained on top of Apron (downstream side)	W_c_apron	1		57.0		24.8	2.61	3692.9	230.4			12.4	2860.1	
Water above End Sill (Surcharge Water)	W_g	1		57.0		24.8	0.00	0.0	0.0			12.4	0.0	
Head Water Active Pressure (upstream side)	W_c_head water										1151.7	8.1		9366.9
Tail Water Passive Pressure (downstream side)	W_c_tail water										204.0	3.4	698.5	
Uplift Water Pressure	U_{cl}	1		71.0		77.07	10.27			3506.8		51.4		180187.9
Uplift Water Pressure	U_{cd}	1		71.0		77.07	14.13			2412.4		38.5		92967.0
Silt Passive Pressure (downstream side)-neglected														
Total									8259.8	5919.3	947.6		377303.7	282521.8

Overturning neglecting Uplift (for bearing check only): 9366.9

W_c:	1096.5
W_g:	0.0
W_s:	7163.2
	8259.8

Safety Factors Check	
Against Overturning:	1.34
Against Sliding:	1.92
S.F. against Floatation:	1.40

Elevations (ft):	Upstream	Downstream
Normal Water		
Maximum Water	16.50	2.37
Top of Slab	-4.40	-4.40
Bottom of Slab	-7.90	-7.90

Baffles:	no	
Roller Gate Weir Elev.:	9.00	ft
Weir Gate Sill Elev.:	9.50	ft
Roller Gate Weir Width.:	18.88	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	52.25	ft
Abutment Length:	24.82	ft
Pier Width:	3.00	ft
Pier Length:	49.25	ft
Pier Elev.:	17.00	ft
Pier Elev. (end):	17.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	30.00	ft
Top of Service Bridge	17.00	ft
Breast Wall Top Elev.:	19.00	ft
Breast Wall Bottom Elev.:	14.50	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	30.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	77.07	ft
End Sill Height:	2.61	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Skin Friction Reduction Factor:	0.67	
No. of Roller Gates	2.00	
No. of Weir Gates	2.00	

Lane width:	24	ft
Sidewalk:	5.67	ft
Curb:	0.83	ft
Bridge width:	30.5	ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:			
area	5.75	4.28125	
neutral axis		0.744565217	ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
0.00	0.00	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	
1.50		ft
1.50		C.L. of Baffles from toe:
		19.20
12.50	8.50	
20	16	

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Spillway S-11: Load Case 2B (Maximum Pool Condition with tail water provided in Structure Operations Summary)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	A_{b1}	2		52.3		21.4	2.50	5590.8	838.6			50.9	42725.8	
Abutment 2 (height varies)	A_{b2}	2		24.8		21.4	2.50	2656.1	398.4			12.4	4944.8	
Breast Wall	B_{ws}	1		20.0		5.8	1.00	115.0	17.3			66.6	1149.5	
Bridge	B_d	1		62.0		30.5	1.33	2521.3	378.2			46.8	17684.8	
Column-Support Platform	Cs_1	3		11.5	20.8			717.4	107.6			61.8	6654.4	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			61.8	3924.8	
Platform	PLs	1		62.0	9.8			606.4	91.0			61.8	5624.9	
Spread Foundation (Apron Slab)	S_t	1		71.0		77.1	3.50	19152.9	2872.9			38.5	110711.1	
Pier	P_s	3		49.3		21.4	3.00	9485.6	1422.8			49.4	70356.1	
Weir	$Ws2$			28.0	156.7			4386.5	658.3			60.3	39722.3	
Abutment Triangular Section at Base	A_{c1}				20.1	77.1		1549.2	232.4			38.5	8954.9	
Roller Gate	G_t	2		15.6	197.3	6.3			12.8			63.2	810.6	
Weir Gate Sill	Sl_{wg}	2		10.0	46.3	13.9			69.5			63.2	4393.7	
Baffles	B_d	0		1.5	6.0			0.0	0.0			0.0	0.0	
Water on top of Heel (upstream side)	W_c_heel	1		48.0		10.4	20.90	10470.9	653.4			71.9	46948.4	
Water on top of Crest (upstream side)	W_c_crest	1		48.0		2.5	7.50	911.3	56.9			65.4	3717.1	
Water contained on top of Apron (downstream side)	W_c_apron	1		48.0		19.9	2.61	2497.6	155.9			34.8	5422.3	
Water contained on top of Apron (downstream side)	W_c_apron	1		57.0		24.8	2.61	3692.9	230.4			12.4	2860.1	
Water above End Sill (Surcharge Water)	W_s	1		57.0		24.8	1.89	2674.2	166.9			12.4	2071.1	
Head Water Active Pressure (upstream side)	W_c_head water										1151.7	8.1		9366.9
Tail Water Passive Pressure (downstream side)	W_c_tail water										297.4	4.1	1229.4	
Uplift Water Pressure	U_H	1		71.0		77.07	12.40			4234.2		51.4		217558.9
Uplift Water Pressure	U_{tw}	1		71.0		77.07	12.00			2048.8		38.5		78952.8
Silt Passive Pressure (downstream side)-neglected														
Total									8426.6	6282.9	854.2		379905.7	305878.7

Overturning neglecting Uplift (for bearing check only): 9366.9

W_{tc}:	1096.5
W_{gc}:	166.9
W_{sc}:	7163.2
	8426.6

Safety Factors Check	
Against Overturning:	1.24
Against Sliding:	2.14
S.F. against Floatation:	1.35

Elevations (ft):	Upstream	Downstream
Normal Water		
Maximum Water	16.50	4.50
Top of Slab	-4.40	-4.40
Bottom of Slab	-7.90	-7.90

Baffles:	no	
Roller Gate Weir Elev.:	9.00	ft
Weir Gate Sill Elev.:	9.50	ft
Roller Gate Weir Width.:	18.88	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	52.25	ft
Abutment Length:	24.82	ft
Pier Width:	3.00	ft
Pier Length:	49.25	ft
Pier Elev.:	17.00	ft
Pier Elev. (end):	17.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	30.00	ft
Top of Service Bridge	17.00	ft
Breast Wall Top Elev.:	19.00	ft
Breast Wall Bottom Elev.:	14.50	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	30.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	77.07	ft
End Sill Height:	2.61	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Skin Friction Reduction Factor:	0.67	
No. of Roller Gates	2.00	
No. of Weir Gates	2.00	

Lane width:	24 ft
Sidewalk:	5.67 ft
Curb:	0.83 ft
Bridge width:	30.5 ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:		
area	5.75	4.28125
neutral axis		0.744565217 ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
0.00	0.00	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	
1.50		ft
1.50		C.L. of Baffles from toe:
		19.20
12.50	8.50	
20	16	

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Spillway S-11: Load Case 3A (De-Watered Condition with tail water at weir crest elevation of structure downstream)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	A_{s1}	2		52.3		21.4	2.50	5590.8	838.6			50.9	42725.6	
Abutment 2 (height varies)	A_{s2}	2		24.8		21.4	2.50	2650.1	398.4			12.4	4944.8	
Breast Wall	B_{W3}	1		20.0		5.8	1.00	115.0	17.3			66.8	1149.5	
Bridge	B_s	1		62.0		30.5	1.33	2521.3	378.2			46.8	17684.8	
Column-Support Platform	Cs_1	3		11.5	20.8			717.4	107.6			61.8	6654.4	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			61.8	3924.8	
Platform	PL_s	1		62.0	9.8			606.4	91.0			61.8	5624.9	
Spread Foundation (Apron Slab)	S_s	1		71.0		77.1	3.50	19152.6	2872.9			38.5	110711.1	
Pier	P_s	3		49.3		21.4	3.00	9485.6	1422.8			49.4	70356.1	
Weir	Ws2			28.0	156.7			4386.5	658.3			60.3	39722.3	
Abutment Triangular Section at Base	Ac_3				20.1	77.1		1549.2	232.4			38.5	8954.9	
Roller Gate	G_s	2		16.6	197.3	6.3			12.8			63.2	810.6	
Weir Gate Sill	Sl_{wg}	2		10.0	48.3	13.9			69.5			63.2	4393.7	
Baffles	B_{cr}	0		1.5	6.0			0.0	0.0			0.0	0.0	
Water on top of Heel (upstream side)	W_c_heel	1		48.0		10.4	19.40	9719.4	606.5			71.9	43578.9	
Water on top of Crest (upstream side)	W_c_crest	1		48.0		2.5	6.00	729.0	45.5			65.4	2973.6	
Water contained on top of Apron (downstream side)	W_c_apron	1		48.0		19.9	2.61	2497.6	155.9			34.8	5422.3	
Water contained on top of Apron (downstream side)	W_c_apron	1		57.0		24.8	2.61	3692.9	230.4			12.4	2860.1	
Water above End Sill (Surcharge Water)	W_g	1		57.0		24.8	0.00	0.0	0.0			12.4	0.0	
Head Water Active Pressure (upstream side)	W_c_head water										1014.4	7.6		7743.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										204.0	3.4	698.5	
Uplift Water Pressure	U_u	1		71.0		77.07	10.27			3506.8		51.4		180187.0
Uplift Water Pressure	U_{ut}	1		71.0		77.07	12.63			2156.3		38.5		83097.9
Silt Passive Pressure (downstream side)-neglected														
Total									8201.5	5663.2	810.4		373190.8	271029.2

Overturning neglecting Uplift (for bearing check only): 7743.4

W_c:	1038.3
W_g:	0.0
W_c:	7163.2
	8201.5

Safety Factors Check:	
S.F. against Overturning:	1.38
S.F. against Sliding:	2.25
S.F. against Floatation:	1.45

Elevations (ft):	Upstream	Downstream
Normal Water	15.00	2.37
Maximum Water		
Top of Slab	-4.40	-4.40
Bottom of Slab	-7.90	-7.90

Baffles:	no	
Roller Gate Weir Elev.:	9.00	ft
Weir Gate Sill Elev.:	9.50	ft
Roller Gate Weir Width.:	18.88	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	52.25	ft
Abutment Length:	24.82	ft
Pier Width:	3.00	ft
Pier Length:	49.25	ft
Pier Elev.:	17.00	ft
Pier Elev. (end):	17.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	30.00	ft
Top of Service Bridge	17.00	ft
Breast Wall Top Elev.:	19.00	ft
Breast Wall Bottom Elev.:	14.50	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	30.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	77.07	ft
End Sill Height:	2.61	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Skin Friction Reduction Factor:	0.67	
No. of Roller Gates	2.00	
No. of Weir Gates	2.00	

Lane width:	24	ft
Sidewalk:	5.67	ft
Curb:	0.83	ft
Bridge width:	30.5	ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:		
area	5.75	4.28125
neutral axis		0.744565217 ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
0.00	0.00	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	
1.50		ft
1.50		C.L. of Baffles from toe:
		19.20
12.50	8.50	
20	16	

Spillway S-11: Load Case 3B (De-Watered Condition with tail water provided in Structure Operations Summary)

Item	Label	Qty	Area (sf)	Length (ft)	Area (sf)	Width/Height (ft)	Thickness (ft)	Volume (ft3)	Weight (kip)	Uplift (kip)	Active Force (kip)	Lever Arm (ft)	M_R (ft-kip)	M_{ot} (ft-kip)
Abutment 1	As_1	2		52.3		21.4	2.50	5590.8	838.6			50.9	42725.6	
Abutment 2 (height varies)	As_2	2		24.8		21.4	2.50	2656.1	398.4			12.4	4944.8	
Breast Wall	B_{WB}	1		20.0		5.8	1.00	115.0	17.3			66.6	1149.5	
Bridge	Bs	1		62.0		30.5	1.33	2521.3	378.2			46.8	17684.6	
Column-Support Platform	Cs_1	3		11.5	20.8			717.4	107.6			61.8	6654.4	
Column-Support Platform	Cs_2	2		11.5	18.4			423.1	63.5			61.8	3924.8	
Platform	PLs	1		62.0	9.8			606.4	91.0			61.8	5624.9	
Spread Foundation (Apron Slab)	S_s	1		71.0		77.1	3.50	19152.6	2872.9			38.5	110711.1	
Pier	P_1	3		49.3		21.4	3.00	9485.6	1422.8			49.4	70358.1	
Weir	Ws2			28.0	156.7			4388.5	658.3			60.3	39722.3	
Abutment Triangular Section at Base	As_3				20.1	77.1		1549.2	232.4			38.5	8954.9	
Roller Gate	G_1	2		15.6	197.3	6.3			12.6			63.2	810.6	
Weir Gate Sill	Sl_{wg}	2		10.0	46.3	13.9			69.5			63.2	4393.7	
Baffles	B_u	0		1.5	6.0			0.0	0.0			0.0	0.0	
Water on top of Heel (upstream side)	W_c_heel	1		48.0		10.4	19.40	9719.4	606.5			71.9	43578.9	
Water on top of Crest (upstream side)	W_c_crest	1		48.0		2.5	6.00	729.0	45.5			65.4	2973.6	
Water contained on top of Apron (downstream side)	W_c_apron	1		48.0		19.9	2.61	2497.6	155.9			34.8	5422.3	
Water contained on top of Apron (downstream side)	W_c_apron	1		57.0		24.8	2.61	3692.9	230.4			12.4	2860.1	
Water above End Sill (Surcharge Water)	W_a	1		57.0		24.8	1.89	2674.2	166.9			12.4	2071.1	
Head Water Active Pressure (upstream side)	W_c_head water										1014.4	7.6		7743.4
Tail Water Passive Pressure (downstream side)	W_c_tail water										297.4	4.1	1229.4	
Uplift Water Pressure	U_H	1		71.0		77.07	12.40			4234.2		51.4		217558.9
Uplift Water Pressure	U_{dl}	1		71.0		77.07	10.50			1792.7		38.5		69083.7
Silt Passive Pressure (downstream side)-neglected														
Total									8368.4	6026.8	717.0		375792.8	294386.1

Overturning neglecting Uplift (for bearing check only): 7743.4

W_c	1038.3
W_g	166.9
W_c	7163.2
	8368.4

Safety Factors Check	
S.F. against Overturning:	1.28
S.F. against Sliding:	2.54
S.F. against Floatation:	1.40

Elevations (ft):	Upstream	Downstream
Normal Water	15.00	4.50
Maximum Water		
Top of Slab	-4.40	-4.40
Bottom of Slab	-7.90	-7.90

Baffles:	no	
Roller Gate Weir Elev.:	9.00	ft
Weir Gate Sill Elev.:	9.50	ft
Roller Gate Weir Width.:	18.88	ft
Roller Gate Weir Length:	14.00	ft
Weir Gate Sill Length:	10.00	ft
Weir Gate Sill Width:	1.67	ft
Abutment Width:	2.50	ft
Abutment Length:	52.25	ft
Abutment Length:	24.82	ft
Pier Width:	3.00	ft
Pier Length:	49.25	ft
Pier Elev.:	17.00	ft
Pier Elev. (end):	17.00	ft
Platform Thickness:	1.50	ft
Platform Width:	8.00	ft
Heel Length:	10.44	ft
Apron's Overhang	4.50	ft
Top of Operating Platform	30.00	ft
Top of Service Bridge	17.00	ft
Breast Wall Top Elev.:	19.00	ft
Breast Wall Bottom Elev.:	14.50	ft
Breast Wall Thickness:	1.00	ft
Bridge Width:	30.50	ft
Bridge Thickness:	1.33	ft
Apron Slab (overall length):	77.07	ft
End Sill Height:	2.61	ft
Apron Slab Thickness:	3.50	ft
Skin Friction Pressure:	500.00	psf
Skin Friction Reduction Factor:	0.67	
No. of Roller Gates	2.00	
No. of Weir Gates	2.00	

Lane width:	24	ft
Sidewalk:	5.67	ft
Curb:	0.83	ft
Bridge width:	30.5	ft

Item	Density (kcf)	Weight (lbf/sf)
RC Structure	0.150	
Roller Gates (Steel)		65
Weir Gate (Aluminum)		34

Breast Wall:			
area	5.75	4.28125	
neutral axis		0.744565217	ft

Considerations:
1. Weir area obtanainded from Mathcad file "Spillway Geometry_30% Design" by RES
2. Silt Passive Pressure (downstream side) was neglected due to the possibility of scouring
3. Silt Passive Pressure (upstream side) was also neglected due to the possibility of scouring and in case not, the resisting moment due to silt vertical component is much larger than the overturning moment.
4. Spread Foundation (Apron Slab) width to be confirmed by Peter deGolian with DHI
5. Spread Foundation (Apron Slab) length to be assumed from STD Dwg S103 by USACE
6. Overturning due to Uplift Water Pressure neglected in checking bearing capacity failure, in accordance with USACE
7. Based on Structure Operations Summaries by Chris F. Zamaites comps dated 5/4/06 10:49 am.

No. Baffles/14'	No. Baffles/10'	
0.50	0.50	
0.00	0.00	Area Baffle
1.50	1.50	5.95 sf
1.50	1.50	
1.50	1.50	Baffle Length
1.50	1.50	1.50 ft
1.50	1.50	
1.50	0.50	
1.50		ft
1.50		C.L. of Baffles from toe:
		19.20
12.50	8.50	
20	16	

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Spillway S-11: Stability Check

Reference Files

References

EC 1110-2-6058 Stability Requirements, USACE 30 November 2003
ETL 11102-307 Engineerin and Design Flotation Stability Criteria for Concrete Hydraulic Structures, USACE 20 August 1987
Design of Small Dams, published by United State Department of the Interior, Bureau of Reclamation; A Water Resources Technical Publication.
Geotechnical Report by DE&T, dated 1/06/06

Units:

$psi := \frac{lbf}{in^2}$ $psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $kip := 1000lbf$ $ksi := \frac{kip}{in^2}$ $ksf := \frac{kip}{ft^2}$ $kcf := \frac{kip}{ft^3}$

Constants

$\gamma_c := 150\,pcf$ $\gamma_{sat} := 110pcf$ $C := 500psf$ (assumed)

$\gamma_w := 62.4pcf$ $\gamma_{steel} := 490pcf$ $\phi := 30deg$ (assumed)

Structure Description:

Reinforced Concrete Spillway Structure with two (2) Sluice Gates (i.e., Roller Gates) and two (2) Weir Gates (i.e., Vertical Double-Slide Aluminum Gates) to regulate water flow around Perimeter Canal.

Design Criteria

Allowable Stability Factors by USACE; Construction Materials in accordance with ASTM Standards; Allowable Stress Design. Design for a 100 years event (S. Gaitan correspondence 4/21/06). A skin friction pressure of 500 psf will be assumed based on Geotechnical Engineer's recommendations. Hydraulic data based on Structure Operations Summaries by C. Zamaites dated 5/4/06.

$SF_{Sliding\_USACE} := 1.3$ $SF_{Floating\_USACE} := 1.2$

$q_{max\_allow} := 6900psf$

Load Case 1A (Normal Pool with tail water at weir crest elev. of structure downstream):

Load Case 1B (Normal Pool with tail water provided in Structure Oper. Summaries):

Elevations (ft):			
	Upstream	Downstream	
		1A	1B
Normal Water	15.00	2.37	4.50
Top of Slab	-4.40	-4.40	-4.40
Bottom of Slab	-7.90	-7.90	-7.90
Baffles:	no		
Roller Gate Weir Elev.:	9.00	ft	
Weir Gate Sill Elev.:	9.50	ft	
Roller Gate Weir Width.:	18.88	ft	
Roller Gate Weir Length:	14.00	ft	
Weir Gate Sill Length:	10.00	ft	
Weir Gate Sill Width:	1.67	ft	
Abutment Width:	2.50	ft	
Abutment Length:	52.25	ft	
Abutment Length:	24.82	ft	
Pier Width:	3.00	ft	
Pier Length:	49.25	ft	
Pier Elev.:	17.00	ft	
Pier Elev. (end):	17.00	ft	
Platform Thickness:	1.50	ft	
Platform Width:	8.00	ft	
Heel Length:	10.44	ft	
Apron's Overhang	4.50	ft	
Top of Operating Platform	30.00	ft	
Top of Service Bridge	17.00	ft	
Breast Wall Top Elev.:	19.00	ft	
Breast Wall Bottom Elev.:	14.50	ft	
Breast Wall Thickness:	1.00	ft	
Bridge Width:	30.50	ft	
Bridge Thickness:	1.33	ft	
Apron Slab (overall length):	77.07	ft	
End Sill Height:	2.61	ft	
Apron Slab Thickness:	3.50	ft	
Skin Friction Pressure:	500.00	psf	
Skin Friction Reduction Factor:	0.67		
No. of Roller Gates	2.00		
No. of Weir Gates	2.00		

Sliding Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$k := \frac{2}{3}$ reduction factor for cohesive strength

$c := 500psf$ cohesive strength of the foundation material under the structural wedge

$Apron\_length := 77.07ft$

$Apron\_width := 71.0ft$

$Active\_Force1A := 810.4kip$ $Active\_Force1B := 717.0kip$

$A_{apron\_slab} := Apron\_length \cdot Apron\_width$

$SF_{Sliding1A} := \frac{k \cdot c \cdot A_{apron\_slab}}{Active\_Force1A}$ $SF_{Sliding1A} = 2.251$

$Check\_Sliding1A := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding1A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding1A = \text{"O.K."}$

$SF_{Sliding1B} := \frac{k \cdot c \cdot A_{apron\_slab}}{Active\_Force1B}$ $SF_{Sliding1B} = 2.544$

$Check\_Sliding1B := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding1A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding1B = \text{"O.K."}$

Floating (Uplift) Check:

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from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$W_s := 7163.2kip$		weight of the structure
$W_c := 1038.3kip$		weight of the water contained within the structure
$U1A := 5663.2kip$	$U1B := 6026.8kip$	uplift force acting on the base of the structure
$S := 0$		surcharge loads
$W_{g1A} := 0kip$	$W_{g1B} := 166.9kip$	weight of the surcharge water

---

$$SF_{Floating1A} := \frac{W_s + W_c + S}{U1A - W_{g1A}} \qquad SF_{Floating1A} = 1.448$$

$$Check\_Floating1A := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating1A} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating1A = \text{"O.K."}$$

$$SF_{Floating1B} := \frac{W_s + W_c + S}{U1B - W_{g1B}} \qquad SF_{Floating1B} = 1.4$$

$$Check\_Floating1B := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating1B} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating1B = \text{"O.K."}$$

Bearing Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$\Sigma M_{R1A} := 373190.8\,ft\cdot kip$ $\Sigma M_{R1B} := 375792.8\,ft\cdot kip$

$\Sigma M_{OT} := 7743.4\,ft\cdot kip$

$\Sigma W := W_c + W_s$ $\Sigma W = 8201.5\ kip$

$$e1A := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R1A} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e1A = 6.024\ ft$$

$$Check\_eccentricity1A := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e1A \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity1A = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max1A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e1A}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max1A} = 1.509\ ksf$$

$$Check\_q_{max1A} := \begin{cases} \text{"O.K."} & \text{if } q_{max1A} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max1A} = \text{"O.K."}$$

$$q_{min1A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e1A}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min1A} = 1.489\ ksf$$

$$Check\_q_{min1A} := \begin{cases} \text{"O.K."} & \text{if } q_{min1A} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min1A} = \text{"O.K."}$$

$$e1B := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R1B} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e1B = 6.341\ ft$$

$$Check\_eccentricity1B := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e1B \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity1B = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max1B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e1B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max1B} = 1.509\ ksf$$

$$Check\_q_{max1B} := \begin{cases} \text{"O.K."} & \text{if } q_{max1B} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max1B} = \text{"O.K."}$$

$$q_{min1B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e1B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min1B} = 1.488\ ksf$$

$$Check\_q_{min1B} := \begin{cases} \text{"O.K."} & \text{if } q_{min1B} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min1B} = \text{"O.K."}$$

Load Case 2A (Max. Pool with tail water at weir crest elev. of structure downstream):
Load Case 2B (Max. Pool with tail water provided in Structure Oper. Summaries):

Elevations (ft):			
	Upstream	Downstream	
		2A	2B
Maximum Water	16.50	2.37	4.50
Top of Slab	-4.40	-4.40	-4.40
Bottom of Slab	-7.90	-7.90	-7.90
Baffles:	no		
Roller Gate Weir Elev.:	9.00	ft	
Weir Gate Sill Elev.:	9.50	ft	
Roller Gate Weir Width.:	18.88	ft	
Roller Gate Weir Length:	14.00	ft	
Weir Gate Sill Length:	10.00	ft	
Weir Gate Sill Width:	1.67	ft	
Abutment Width:	2.50	ft	
Abutment Length:	52.25	ft	
Abutment Length:	24.82	ft	
Pier Width:	3.00	ft	
Pier Length:	49.25	ft	
Pier Elev.:	17.00	ft	
Pier Elev. (end):	17.00	ft	
Platform Thickness:	1.50	ft	
Platform Width:	8.00	ft	
Heel Length:	10.44	ft	
Apron's Overhang	4.50	ft	
Top of Operating Platform	30.00	ft	
Top of Service Bridge	17.00	ft	
Breast Wall Top Elev.:	19.00	ft	
Breast Wall Bottom Elev.:	14.50	ft	
Breast Wall Thickness:	1.00	ft	
Bridge Width:	30.50	ft	
Bridge Thickness:	1.33	ft	
Apron Slab (overall length):	77.07	ft	
End Sill Height:	2.61	ft	
Apron Slab Thickness:	3.50	ft	
Skin Friction Pressure:	500.00	psf	
Skin Friction Reduction Factor:	0.67		
No. of Roller Gates	2.00		
No. of Weir Gates	2.00		

Sliding Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$k := \frac{2}{3}$ reduction factor for cohesive strength

$c := 500psf$ cohesive strength of the foundation material under the structural wedge

$Apron\_length := 77.07ft$

$Apron\_width := 71.0ft$

$Active\_Force2A := 947.6kip$ $Active\_Force2B := 854.2kip$

$A_{apron\_slab} := Apron\_length \cdot Apron\_width$

$SF_{Sliding2A} := \frac{k \cdot c \cdot A_{apron\_slab}}{Active\_Force2A}$ $SF_{Sliding2A} = 1.925$

$Check\_Sliding2A := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding2A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding2A = \text{"O.K."}$

$SF_{Sliding2B} := \frac{k \cdot c \cdot A_{apron\_slab}}{Active\_Force2B}$ $SF_{Sliding2B} = 2.135$

$Check\_Sliding2B := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding2B} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding2B = \text{"O.K."}$

Floating (Uplift) Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$W_s := 7163.2kip$ — weight of the structure

$W_c := 1096.5kip$ — weight of the water contained within the structur

$U2A := 5919.3kip$ $U2B := 6282.9kip$ — uplift force acting on the base of the structure

$S := 0$ — surcharge loads

$W_{g2A} := 0kip$ $W_{g2B} := 166.9kip$ — weight of the surcharge water

$$SF_{Floating2A} := \frac{W_s + W_c + S}{U2A - W_{g2A}} \qquad SF_{Floating2A} = 1.395$$

$$Check\_Floating2A := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating2A} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating2A = \text{"O.K."}$$

$$SF_{Floating2B} := \frac{W_s + W_c + S}{U2B - W_{g2B}} \qquad SF_{Floating2B} = 1.351$$

$$Check\_Floating2B := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating2B} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating2B = \text{"O.K."}$$

Bearing Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$\Sigma M_{R2A} := 377303.7\,ft\cdot kip$ $\Sigma M_{R2B} := 379905.7\,ft\cdot kip$

$\Sigma M_{OT} := 9366.9\,ft\cdot kip$

$\Sigma W := W_c + W_s$ $\Sigma W = 8259.7\,kip$

$$e2A := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R2A} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e2A = 6.011\ ft$$

$$Check\_eccentricity2A := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e2A \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity2A = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max2A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e2A}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max2A} = 1.519\ ksf$$

$$Check\_q_{max2A} := \begin{cases} \text{"O.K."} & \text{if } q_{max2A} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max2A} = \text{"O.K."}$$

$$q_{min2A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e2A}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min2A} = 1.5\ ksf$$

$$Check\_q_{min2A} := \begin{cases} \text{"O.K."} & \text{if } q_{min1A} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min2A} = \text{"O.K."}$$

$$e2B := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R2B} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e2B = 6.326\ ft$$

$$Check\_eccentricity2B := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e2B \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity2B = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max2B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e2B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max2B} = 1.52\ ksf$$

$$Check\_q_{max2B} := \begin{cases} \text{"O.K."} & \text{if } q_{max2B} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max2B} = \text{"O.K."}$$

$$q_{min2B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e2B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min2B} = 1.499\ ksf$$

$$Check\_q_{min2B} := \begin{cases} \text{"O.K."} & \text{if } q_{min2B} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min2B} = \text{"O.K."}$$

Load Case 3A (De-Watered with tail water at weir crest elev. of structure downstream):
Load Case 3B (De-Watered with tail water provided in Structure Oper. Summaries):

Elevations (ft):			
	Upstream	Downstream	
		3A	3B
Normal Water	15.00	2.37	4.50
Top of Slab	-4.40	-4.40	-4.40
Bottom of Slab	-7.90	-7.90	-7.90
Baffles:	no		
Roller Gate Weir Elev.:	9.00	ft	
Weir Gate Sill Elev.:	9.50	ft	
Roller Gate Weir Width.:	18.88	ft	
Roller Gate Weir Length:	14.00	ft	
Weir Gate Sill Length:	10.00	ft	
Weir Gate Sill Width:	1.67	ft	
Abutment Width:	2.50	ft	
Abutment Length:	52.25	ft	
Abutment Length:	24.82	ft	
Pier Width:	3.00	ft	
Pier Length:	49.25	ft	
Pier Elev.:	17.00	ft	
Pier Elev. (end):	17.00	ft	
Platform Thickness:	1.50	ft	
Platform Width:	8.00	ft	
Heel Length:	10.44	ft	
Apron's Overhang	4.50	ft	
Top of Operating Platform	30.00	ft	
Top of Service Bridge	17.00	ft	
Breast Wall Top Elev.:	19.00	ft	
Breast Wall Bottom Elev.:	14.50	ft	
Breast Wall Thickness:	1.00	ft	
Bridge Width:	30.50	ft	
Bridge Thickness:	1.33	ft	
Apron Slab (overall length):	77.07	ft	
End Sill Height:	2.61	ft	
Apron Slab Thickness:	3.50	ft	
Skin Friction Pressure:	500.00	psf	
Skin Friction Reduction Factor:	0.67		
No. of Roller Gates	2.00		
No. of Weir Gates	2.00		

Sliding Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$k := \frac{2}{3}$ reduction factor for cohesive strength

$c := 500psf$ cohesive strength of the foundation material under the structural wedge

$Apron\_length := 77.07ft$

$Apron\_width := 71.0ft$

$Active\_Force3A := 810.4kip$ $Active\_Force3B := 717.0kip$

$A_{apron\_slab} := Apron\_length \cdot Apron\_width$

$SF_{Sliding3A} := \frac{k \cdot c \cdot A_{apron\_slab}}{Active\_Force3A}$ $SF_{Sliding3A} = 2.251$

$Check\_Sliding3A := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding3A} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding3A = \text{"O.K."}$

$SF_{Sliding3B} := \frac{k \cdot c \cdot A_{apron\_slab}}{Active\_Force3B}$ $SF_{Sliding3B} = 2.544$

$Check\_Sliding3B := \begin{cases} \text{"O.K."} & \text{if } SF_{Sliding3B} \geq SF_{Sliding\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases}$ $Check\_Sliding3B = \text{"O.K."}$

Floating (Uplift) Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$W_s := 7163.2kip$ — weight of the structure

$W_c := 1038.3kip$ — weight of the water contained within the structur

$U3A := 5663.2kip$ $U3B := 6026.8kip$ — uplift force acting on the base of the structure

$S := 0$ — surcharge loads

$W_{g3A} := 0kip$ $W_{g3B} := 166.9kip$ — weight of the surcharge water

$$SF_{Floating3A} := \frac{W_s + W_c + S}{U3A - W_{g3A}} \qquad SF_{Floating3A} = 1.448$$

$$Check\_Floating3A := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating3A} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating3A = \text{"O.K."}$$

$$SF_{Floating3B} := \frac{W_s + W_c + S}{U3B - W_{g3B}} \qquad SF_{Floating3B} = 1.4$$

$$Check\_Floating3B := \begin{cases} \text{"O.K."} & \text{if } SF_{Floating3B} \geq SF_{Floating\_USACE} \\ \text{"Re-design"} & \text{otherwise} \end{cases} \qquad Check\_Floating3B = \text{"O.K."}$$

Bearing Check:

from Excel worksheet attached "Input Forces and Moments Stability Check Spillway S-11":

$\Sigma M_{R3A} := 373190.8\,ft\cdot kip$ $\Sigma M_{R3B} := 375792.8\,ft\cdot kip$

$\Sigma M_{OT} := 7743.4\,ft\cdot kip$

$\Sigma W := W_c + W_s$ $\Sigma W = 8201.5\ kip$

$e3A := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R3A} - \Sigma M_{OT}}{\Sigma W} \right|$ $e3A = 6.024\ ft$

$Check\_eccentricity3A := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e3A \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$

Check_eccentricity2A = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$q_{max3A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e3A}{\frac{A_{apron\_slab}}{ft}} \right)$ $q_{max3A} = 1.509\ ksf$

$Check\_q_{max3A} := \begin{cases} \text{"O.K."} & \text{if } q_{max3A} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases}$ $Check\_q_{max3A} = \text{"O.K."}$

$q_{min3A} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e3A}{\frac{A_{apron\_slab}}{ft}} \right)$ $q_{min3A} = 1.489\ ksf$

$Check\_q_{min3A} := \begin{cases} \text{"O.K."} & \text{if } q_{min3A} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases}$ $Check\_q_{min3A} = \text{"O.K."}$

$$e3B := \left| \frac{Apron\_length}{2} - \frac{\Sigma M_{R3B} - \Sigma M_{OT}}{\Sigma W} \right| \qquad e3B = 6.341\ ft$$

$$Check\_eccentricity3B := \begin{cases} \text{"O.K. The entire Foundation is in Compression"} & \text{if } e3B \le \frac{Apron\_length}{6} \\ \text{"Re-Design. Resultant outside of kern."} & \text{otherwise} \end{cases}$$

Check_eccentricity3B = "O.K. The entire Foundation is in Compression"

Bearing Pressures (q_{max}, q_{min}):

$$q_{max3B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 + \frac{6 \cdot e3B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{max3B} = 1.509\ ksf$$

$$Check\_q_{max3B} := \begin{cases} \text{"O.K."} & \text{if } q_{max3B} \le q_{max\_allow} \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{max3B} = \text{"O.K."}$$

$$q_{min3B} := \frac{\Sigma W}{A_{apron\_slab}} \cdot \left(1 - \frac{6 \cdot e3B}{\frac{A_{apron\_slab}}{ft}} \right) \qquad q_{min3B} = 1.488\ ksf$$

$$Check\_q_{min3B} := \begin{cases} \text{"O.K."} & \text{if } q_{min3B} \ge 0 \\ \text{"Re-Design"} & \text{otherwise} \end{cases} \qquad Check\_q_{min3B} = \text{"O.K."}$$

Summary:

Sliding Check:

Check_Sliding1A = "O.K."

Check_Sliding1B = "O.K."

Check_Sliding2A = "O.K."

Check_Sliding2B = "O.K."

Check_Sliding3A = "O.K."

Check_Sliding3B = "O.K."

Floating (uplift) Check:

Check_Floating1A = "O.K."

Check_Floating1B = "O.K."

Check_Floating2A = "O.K."

Check_Floating2B = "O.K."

Check_Floating3A = "O.K."

Check_Floating3B = "O.K."

Bearing Pressures (q_{max}, q_{min}):

Check_eccentricity1A = "O.K. The entire Foundation is in Compression"

Check_eccentricity1B = "O.K. The entire Foundation is in Compression"

Check_eccentricity2A = "O.K. The entire Foundation is in Compression"

Check_eccentricity2B = "O.K. The entire Foundation is in Compression"

Check_eccentricity3A = "O.K. The entire Foundation is in Compression"

Check_eccentricity3B = "O.K. The entire Foundation is in Compression"

Check_q_{max1A} = "O.K."

Check_q_{max1B} = "O.K."

Check_q_{min1A} = "O.K."

Check_q_{min1B} = "O.K."

Check_q_{max2A} = "O.K."

Check_q_{max2B} = "O.K."

Check_q_{min2A} = "O.K."

Check_q_{min2B} = "O.K."

Check_q_{min3A} = "O.K."

Check_q_{min3B} = "O.K."

Stanley Consultants, Inc.		Page No._____.
Computed by: AZI	Comp Date: 4/17/06	Project No.17300
Checked by: RJV	Print Date: 5/30/2006	Berry Groves C-43 Reservoir Spillway S-11
Approved by: _____	Print Time: 6:01 PM	Sheet 18 of 18
Filename: Stability Check Spillway S-11_30% Design_2gates & 2weir gates_5-4-06.xmcd		

Quantity take-off for Structure S-11

Item	Quantity	Length	Width	Thickness/ Height	Linear	Areas	Volume	Volume	Weight
		ft	ft	ft	ft	sft	cft	CY	tons
Spread Footing:									
Formwork									
Concrete									
Reiforcement									
Finishes									
SOG:									
Bridge Approach Slab:									
Formwork	4	10	13	1		92.00			
Concrete	2	10	13	1		260.00	260.00	9.63	
Reiforcement	0.015						3.90		0.96
Finishes						260.00			
Ladder Platform:									
Formwork	2	6	8	0.5		14.00			
Concrete	1	6	8	0.5		48.00	24.00	0.89	
Reiforcement	0.004						0.10		0.02
Finishes						48.00			
Apron Slab:									
Formwork	2	77.07	71.00	3.50		1036.51			
Concrete	1			3.50		5472.18	19152.62	709.36	
Reiforcement	0.0214						409.48		100.32
Finishes		77.07	71.00			4081.70			
Walls:									
Abutment:									
Formwork	4	79.57		21.40		6811.44			
Concrete	2	77.07	2.50	21.40			8246.80	305.44	
Reiforcement	0.015						123.70		30.31
Finishes						7196.80625			
Pier:									
Formwork	6	52.25		21.40		6708.9			
Concrete	3	49.25	3.00	21.40			9485.55	351.32	
Reiforcement	0.015						142.28		34.86
Finishes						7152.15			
Columns (Supporting Platform):									
At Abutments:									
Formwork	4	10.50		11.50		483.00			
Concrete	1						423.14	15.67	
Reiforcement	0.025						10.58		2.59
Finishes						483.00			
At Piers:									
Formwork	4	11.00		11.50		506.00			
Concrete	1						717.43	26.57	
Reiforcement	0.025						17.94		4.39
Finishes						506.00			
Weir (under roller gate):									
Formwork	1	28		13.40		375.2			
Concrete							4388.47	162.54	
Reiforcement	0.015						65.83		16.13
Finishes						1023.36			

Stanley Consultants, Inc.
Computed by:
Checked by:
Approved by:
File name:Q:\17300\07-Design\03-CostEst\Structures 30% Submittal Quantity Take-off\Quantity Take off for S-3, S-10 & S-11.xls

Comp date: 5/16/06
Print date: 5/30/2006
Print time: 5:37 PM

Page No.____
Project No. 17300
C-43 West Storage Reservoir

Quantity take-off for Structure S-11

Item	Quantity	Length ft	Width ft	Thickness/ Height ft	Linear ft	Areas sft	Volume cft	Volume CY	Weight tons
Sill (under weir gate):									
Formwork	2	20		13.90		556			
Concrete							463.33	17.16	
Reiforcement	0.025						11.58		2.84
Finishes						589.33			
Baffles:									
Formwork	0	9.76		1.50		0.00			
Concrete	0			1.50		5.95	0.00	0.00	
Reiforcement	0.008						0.00		0.00
Finishes						0.00			
Elevated Slabs:									
Service bridge slab:									
Formwork	1	62	30.5			1891			
Edge	2	62		1.83		227.33			
Flat									
Concrete	1	62	30.5	1.33		1891.00	2722.83	100.85	
Reiforcement	0.025						68.07		16.68
Finishes		38	51	1.33		4009.33			
Operating Platform:									
Formwork	1	62	8.00			496			
Edge	2	62		1.50		186.00			
Flat									
Concrete	1	62	8.00	1.50		496.00	899.00	33.30	
Reiforcement	0.025						22.48		5.51
Finishes		38	51	1.33		1364.00			
Breast Wall:									
Formwork	2	20	1.00	5.75		250			
Concrete	1						115	4.26	
Reiforcement	0.015						1.73		0.42
Finishes						270			
Sheet Piling:									
Sheetpiles:									
Peripheric locking sheet piles (tremie)	2	89.07		5.00		890.73			
Peripheric locking sheet piles (up/downstream cutoff wall)	1	71.00		4.00		284.00			
Peripheric locking sheet piles (tremie)	2	82.00		5.00		820.00			
Upstream wingwalls	2	62.00		32.90		6119.40			
Downstream wingwalls	2	74.00		29.90		6637.80			
Pile Cap:									
Formwork	4	30.00	2	2.00		720.00			
Concrete	4	30.00	2.00	2.00			480.00	17.78	
Reiforcement	0.008						3.84		0.94
Finishes						960.00			
Seal Concrete:									
Concrete (poor strength)	1	89.07	82	5.00			36519.90	1352.59	
Control Building:									
Precast Control Bldg.	1								
Excavation:									
Excavation	1	89.07	82.00	29.90			218388.98	8088.48	
Erosion Control System:									
Erosion Control Fabric	4	20	67.05			8203.62			
Bedding Rock	1			0.5			4101.81	151.92	
Rip-rap	1			1.5			12305.44	455.76	

58

Flotation Safety Factor: $SF_f = \frac{Ws + Wc + S}{U - Wg}$ (1)

(1) Department of the Army
ETL 1110-2-307
20 August 1987

Where: SF_f = Flotation Safety Factor chosen as 1.30 - Scheduled Maintenance
Ws = Weight of structure including weight of concrete and weight of soil on top of the structure
Wc = Weight of the water contained in the structure (Zero for dewatering purposes)
S = Surcharge loads (Conservatively assumed zero)
U = Uplift forces acting on the base of the structure.
Wg = Weight of water above the structure controlled by gravity
Displaced water weight = U - Wg

Material Unit Weight Assumptions

Concrete	0.1500 kcf
Water	0.0624 kcf
Soil	0.1200 kcf

Main Outlet Stilling Basins

Location	Elevations (ft) Top of Wall	Invert	Discharge Canal Water Surface	Structure dimensions (ft) Exterior Walls No.	Thick	Interior Walls No.	Thick	Slabs Floor Slab	Top Elevated Slab	Interior Bays No.	Width	Out-to-out width (ft)	Wall Height (ft)	Sub-mergd Depth (ft)		Conc area (ft2)	Water Displ (ft2)	Ws = Concrete wght (kips)	U - Wg = Displaced water wght (kips)	Minimum Flotation FOS
S1	24.00	5.00	15.00	2	1.50	1	1.50	3.00	1.25	2	10.00	24.50	17.75	13.00		184.0	318.5	27.6	19.9	1.39
S8	27.00	9.00	19.00	2	1.50	1	1.50	3.00	1.25	2	10.00	24.50	16.75	13.00		179.5	318.5	26.9	19.9	1.35

S1 Perimeter Canal W.S.E. to be lowered a minimum elev. of 15 - 15.00 equals **0.00** ft to safely dewater stilling basin.
S8 Perimeter Canal W.S.E. to be lowered a minimum elev. of 19 - 19.00 equals **0.00** ft to safely dewater stilling basin.

Main Outlet Intake Structures

Location	Elevations (ft) Top of Wall	Invert	Reservoir Water Surface	Structure dimensions (ft) Exterior Walls No.	Thick	Interior Walls No.	Thick	Slabs Floor Slab	Top Elevated Slab	Interior Bays No.	Width	Out-to-out width (ft)	Wall Height (ft)	Sub-mergd Depth (ft)	Openings (ft2)	Conc area (ft2)	Water Displ (ft2)	Ws = Concrete wght (kips)	U - Wg = Displaced water wght (kips)	Minimum Flotation FOS
S1	44.00	18.00	24.25	2	2.00	1	1.50	3.00		2	21.75	49.00	26.00	9.25	-45.00	245.0	453.0	36.8	28.3	1.30
S8	44.00	21.00	26.62	2	2.00	1	1.50	3.00		2	21.75	49.00	23.00	8.62	-45.00	228.5	422.5	34.3	26.4	1.30

Opening Dimensions
Gate 6 ft
Interior wall 14 ft

Reservoir W.S.E. to be lowered to a minimum elevation of 24.25' to safely dewater intake structures.
Conduit entrance gates will be required to dewater the conduit upstream of the gatewell without having to lower the reservoir.

Main Outlet Conduits (Just Downstream of Intake)

	Elevations (ft)			Structure dimensions (ft)																
				Exterior Walls		Interior Walls		Slabs		Interior Bays										
Location	Top of Conduit	invert	Water Surface	No.	Thick	No.	Thick	Floor Slab	Top Elevated Slab	No.	Width	Out-to-out width (ft)	Wall Height (ft)	Sub-mergd Depth (ft)		Conc area (ft2)	Water Displ (ft2)	Conc wght (kips)	U - Wg = Displaced water wght (kips)	Computed Flotation FOS
S1	26.00	18.00	42.00	2	1.50	1	1.50	1.83	2.00	2	10.00	24.50	6.00	9.83		120.8	240.8	18.1	15.0	1.21
S8	29.00	21.00	42.00	2	1.50	1	1.50	1.83	2.00	2	10.00	24.50	6.00	9.83		120.8	240.8	18.1	15.0	1.21

	Soil Characteristics				
Location	Soil Depth (ft)	Soil Area (ft2)	Submerged Soil Wght (kips)	Ws = Total Weight (kips)	Minimum Flotation FOS
S1	1.00	24.50	1.41	19.54	**1.30**
S8	1.00	24.50	1.41	19.54	**1.30**

A minimum of 1' of soil is required over the top slab of the conduits to safely protect against flotation
Sheet 85 shows soil over conduits up to elevation 44.0.

Main Outlet Gatewells (S1 Controls)

		Structure dimensions							
Location	Water Surface El. (ft)	Length (ft)	Width (ft)	Height Submerged (ft)	Water Volume Displaced (cf)	Concrete Weight (Reference: Quantity Computations)		U - Wg = Displ water wght (kips)	Flotation FOS
S1 - Conduit at Gatewell	42.00	40.00	25.50	9.00	9180.0	Conduit Base Concrete	2710 cu ft	870.9	**1.90**
S1 - Gatewell	42.00	13.00	24.50	15.00	4777.5	Conduit Top Slab	1948 cu ft		
						Conduit Walls	2670 cu ft		
						Conduit_to_Gatewell Wall Transition	285 cu ft		
						Gatewell Walls (⊥ to flow)	2058 cu ft		
						Gatewell Walls (ll to flow)	1260 cu ft		
						Air Vent Casings	103 cu ft		
						Total Volume of Concrete	11034 cu ft		
						Ws = Weight of Concrete	1655 kips		

Gatewell Scheduled Maintenance can proceed at normal reservoir water levels.
Soil loading not included in calculations (conservative).

Factor of Safety for Sliding: $FSs = \frac{N \tan\Phi + cL}{T}$ (1)

(1) Department of the Army
ETL 1110-2-6058
30 November 2003

Where: FSs = Factor of Safety for Sliding: Chosen as 1.70 - Well Defined Critical Structure with Usual Load Conditions
N = Force acting normal to the sliding failure plane under the structure
Φ = Angle of internal friction between the foundation material and the structure. For a sand_concrete shear plane, $\Phi_{soil\text{-}str} = (2/3)\Phi_{soil}$
c = Adhesive strength of the foundation material under the structure. (Adhesion = 2/3 Cohesion)
L = Length of the structure in contact with the foundation material
T = Shear force acting parallel to the base of the structure

Material Unit Weight Assumptions
Concrete 0.1500 kcf
Water 0.0624 kcf
Constants
Gravity 32.2 ft/s2

Stilling Basins (Sliding due to impact of water on baffle blocks)

		Structure dimensions (ft)																			
		Exterior Walls			Interior Walls			Base Slab		Top Slab								Baffle Blocks			
Location	Soil Material	No	Height	Thick	No	Height	Thick	Width	Thick	Width	Thick	Length of Structure (with bridge)	Concrete Area (ft2)	Total Concrete Wght (kips)	Max Water Velocity (ft/s)	Equivalent Static Head (ft)	Water Depth (ft)	No	Width	Area Subject to Water Pressure (ft2)	Force of Water (kips)
S1	Silt	2	17.75	1.50	1	17.75	1.50	28.00	3.00	24.50	1.25	24.00	4668.0	700.20	41.1	26.2	2.0	8	1.25	20.00	16.29
S8	Sand	2	16.75	1.50	1	16.75	1.50	28.00	3.00	24.50	1.25	24.00	4560.0	684.00	40.1	25.0	2.0	8	1.25	20.00	15.51

Location	Soil Material	N (kips)	T (kips)	Φ_{soil} (degrees)	$\Phi_{soil\text{-}str.}$ (degrees)	c (kips/ft2)	L (ft)	FOS
S1	Silt	700.20	16.29	0	0	0.2	28.00	**8.25**
S8	Sand	684.00	15.51	32	21.3	0.0	28.00	**17.23**

Reference Geotechnical Design Memo for soil types and soil characteristics.
Weight of water in stilling basins can be neglected as a conservative approach.
Stilling Basins are safe against sliding for their respective soil conditions.

Intake Structures (Sliding due to dynamic wave impact)

Sliding stability needs to be computed at next level of design (60%) once the geometry has been approved.

S1 Gatewell Foundation Bearing Pressure Computations		
Base Length	40	ft
Base Width	26	ft
Base Area	**1040**	**sq ft**
Conduit Base Concrete	2710	cu ft
Conduit Top Slab	1948	cu ft
Conduit Walls	2670	cu ft
Conduit_to_Gatewell Wall Transition	285	cu ft
Gatewell Walls (⊥ to flow)	2058	cu ft
Gatewell Walls (ll to flow)	1260	cu ft
Air Vent Casings	103	cu ft
Total Concrete	**11034**	**cu ft**
Weight of Concrete @ 150 lbs / cu ft	1655055	lbs
Bearing Pressure	**1591**	**psf**

S8 Gatewell Foundation Bearing Pressure Computations		
Base Length	40	ft
Base Width	26	ft
Base Area	**1040**	**sq ft**
Conduit Base Concrete	2710	cu ft
Conduit Top Slab	1948	cu ft
Conduit Walls	2670	cu ft
Conduit_to_Gatewell Wall Transition	285	cu ft
Gatewell Walls (⊥ to flow)	1764	cu ft
Gatewell Walls (ll to flow)	1080	cu ft
Air Vent Casings	87	cu ft
Total Concrete	**10544**	**cu ft**
Weight of Concrete @ 150 lbs / cu ft	1581672	lbs
Bearing Pressure	**1521**	**psf**

6'x10' Outlet Conduits, 10'x10' Vertical Gatewell, Layouts and Loading

GEOMETRY

Elevations - values taken from Dam x-sect sketch prv'd by S Gaitan

Outlet Conduits & Gatewell

Top of dry soil	56.0 ft (at top of dam)
Top of wet soil	46.0 ft (at typ conduit sect just upstream of gates)
Water surface	46.0 ft (at typ conduit sect just upstream of gates)
Water surface	30.0 ft (for pressurized case only)
Conduit invert	18.0 ft
Gatewell	28.0 ft (lower controlling section)

Conduit Geometry

			Thick (ft)
Clear Width	10.0 ft (for 1 cell, 2 cells total)	Top slab	2.00
Clear Height	6.0 ft	Ext walls	1.50
		Int wall	1.50
Overall W	24.5 ft (2 cells + 3 walls)	Floor slab	2.00
Overall H	10.0 ft (cell H + slabs)		

Gatewell Geometry

NOTE: width measured normal to conduit water flow direction, height measured parrallel to flow

Vertical flare dimension from top of conduit roof slab to edge of gatewell: 2.0 ft

			Thick (ft)	
Clear Width	10.0 ft (2 cells)	Top slab	1.67	
Clear Height	10.0 ft	Ext walls	1.50	
		Int walls	1.50	(1 vert, 1 horiz)
Overall W	24.5 ft (2 cells + 3 walls)	Floor slab	1.67	
Overall H	13.33 ft (1 cell + 2 slabs)			

MATERIAL PROPERTIES

Concrete unit weight	0.150 kcf	
Conc f'c	504 kcf	3.50 ksi
Conc Ec =	485592 kcf	3372 ksi
Modulus of Rupture, fr =	64 kcf	0.444 ksi

Conc reinf cover requirements:
Prv'd by S. Gaitan

3.0	in for formed sections
4.0	in poured against soil
6.0	in surfaces at pressurized flow

Water unit weight	0.0624 kcf
Head for pressurized case, Elev	42.0 ft

Soil

Soil unit weight, dry	0.120 kcf (assumed)
Ko for at rest pressure	0.50 (assumed)
At-rest EFP =	0.060 kcf
At-rest EFP, saturated =	0.029 kcf
Vert modulus of subgrade reaction	150 kcf (assumed)

Vert soil spring constant:		Trib Dimension (ft) Longit	Trib Dimension (ft) Transv	Spring K (k/ft)
Conduit	Exterior springs	1.0	2.0	300.0
	Interior springs	1.0	2.0	300.0

LOADING

Conduit - Non-Pressurized

Loading	Submerged Soil Height (ft)	Submerged Soil Load (k/ft)	External Water Height (ft)	External Water Load (k/ft)	Internal Water Height (ft)	Internal Water Load (k/ft)
Vertical						
Top of conduit	20.0	1.15	20.0	1.25	n/a	n/a
Bot of conduit	n/a	n/a	n/a	n/a	6.0	0.37
Horizontal						
Top of conduit	20.0	0.58	20.0	1.25	0.0	0.0
Bot of conduit	30.0	0.86	30.0	1.87	6.0	0.37

Conduit - Pressurized

Loading	Soil Height (ft)	Soil Load (k/ft)	Submerged Soil Height (ft)	Submerged Soil Load (k/ft)	External Water Height (ft)	External Water Load (k/ft)	Pressurized Int Water Height (ft)	Pressurized Int Water Load (k/ft)
Vertical								
Top of conduit	16.3	1.96	3.7	0.21	3.7	0.23	18.0	1.12
Bot of conduit					n/a	n/a	24.0	1.50
Horizontal								
Top of conduit	16.3	0.98	3.7	0.11	3.7	0.23	18.0	1.12
Bot of conduit	16.3	0.98	13.7	0.39	13.7	0.85	24.0	1.50

Gatewells

Section taken just above conduits

	Downstream Side						Upstream Side					
	Soil		Soil + Submrgd Soil		External Water		Submerged Soil		External Water		Internal Water	
	Height (ft)	Load (k/ft)	Height (ft)	Load (k/ft)	Height (ft)	Load (k/ft)	Height (ft)	Load (k/ft)	Height (ft)	Load (k/ft)	Height (ft)	Load (k/ft)
Gatewell	14.0	0.84	14.00	1.24	14.00	0.87	18.0	0.52	18.0	1.12	18.0	1.12
Vert along wall	(0'-14')		(14'-28')									

ACI 318-02 Load Factors & Mods

Eqn No.	9-1		9-2	
	1.4	D	1.2	D (dead)
	1.4	F	1.2	F (fluid)
			1.6	H (soil)

By Arm Corp Engr Strength Design for Reinf-Conc Strs - EM 1110-2-2104 3-3.c :
Use 1.7 for F instead of 1.4 in 9-1
F : 1.7
Use Hf (hydralic factor) for all load combos
Hf : 1.3

Then for Staad Combo Load Cases, modified fators are:

Mod 9-1		Mod 9-2	
1.82	D	1.56	D (dead)
2.21	F	1.56	F (fluid)
		2.08	H (soil)

Load Combos:
1 & 2. 9-1 & 9-2 w/both cells non-pressurized
3 & 4 9-1 & 9-2 w/LT cell non-pressurized
5. 9-1 w/both cells empty
6 & 7. 9-1 & 9-2 w/both cells pressurized
8 & 9. 9-1 & 9-2 w/LT cell pressurized
10 9-2 w/both cells empty

Vehicle Cst Loading

From P. McRae typical heaviest cst truck on site = Caterpiller 740. Check for min soil cover req'd.

Slab design width	1.0	ft
Soil cover	3.0	ft
Dist factor	1.75	from AASHTO 6.4.1 for soil cover >=2'-0 over box culverts
Tire length	1.20	ft (assumed, std HS20 = 10")
Tire width	2.40	ft (from 740 cut sheet)
Dist length =	6.45	ft (for 2 longit sides, for each tire, if overlap use outside perimeter with all contrib loads)
Dist width =	7.65	ft (for 2 transv sides, for each tire, if overlap use outside perimeter with all contrib loads)
Dist area =	49.3	ft2

From figure below, almost no overlap for load distribution, ignore minor overlap.

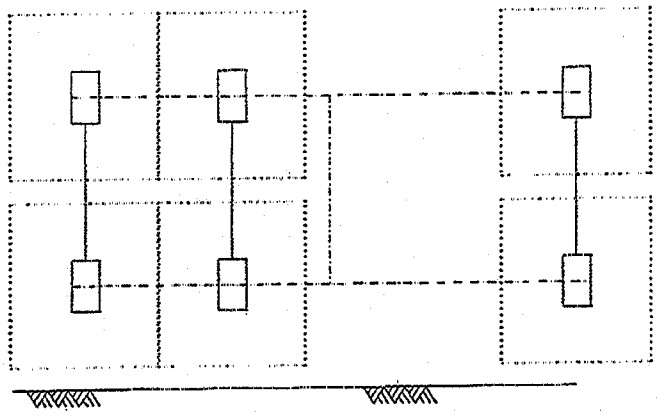
	Front	Center	Rear	
Loaded truck, Axle	47,270	58,250	56,570	lbs (from cut sheet)
Wheels per axle	2	2	2	
Loaded truck, wheel load =	23.6	29.1	28.3	kips
Dist load =	0.48	0.59	0.57	ksf
Dist load per design width =	0.48	0.59	0.57	k/ft

Aside:
Loaded truck, wght = 81.0 tons

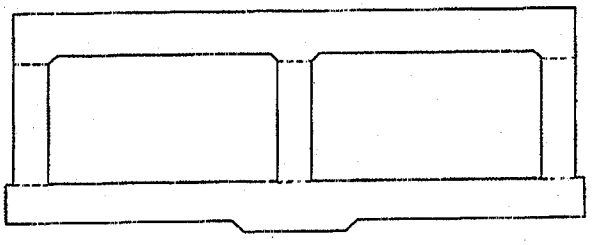
Cst Soil load: Vert = 0.36 k/ft
Horiz = 0.78 k/ft at base

Load Factors: ACI 9-2
1.2 D
1.69 L+I (1.3 for Impact & 1.3 for L (< 1.6 typ, since temp cst load))
1.6 H

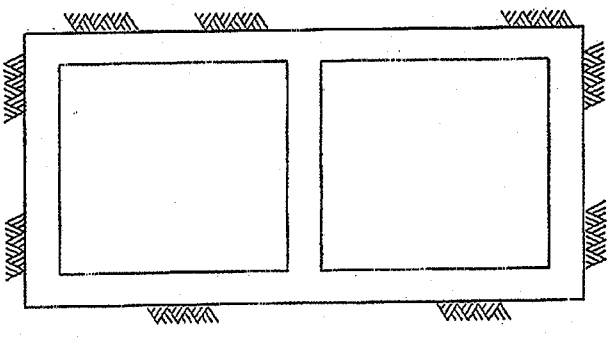
By inspection of Staad model demand results, Fctrd Cst Loading << Factr Load Envelope, therefor 3'-0 cover OK
(True at Crest Spillway also, although Cst Load Demands are closer to Factr Load Envelope, 3'-0 also ok)



Cat 740 Wheel & Axle layout
with wheel distr pattern



Typical Conduit Section



Typical vertical Gatewell section

Job Information

	Engineer	Checked	Approved
Name:	D. Beck	J. Lange	
Date:	May-06	May-06	

Comments

Conduit location just upstream of gates

Structure Type	PLANE FRAME

Number of Nodes	16	Highest Node	16
Number of Elements	17	Highest Beam	17

Number of Basic Load Cases	10
Number of Combination Load Cases	16

Included in this printout are data for:

Beams	1 to 6, 9 11 12 13 17

Included in this printout are results for load cases:

Type	L/C	Name
Combination	11	SERV BRG PRESSURE
Combination	12	SERV BRG PR W/INT PRESSURE
Combination	13	9-1 BOTH CELLS FULL
Combination	14	9-1 LT CELL FULL
Combination	15	9-1 BOTH CELLS EMPTY
Combination	16	9-1 BOTH CELLS FULL PR
Combination	17	9-1 LT CELL FULL PR
Combination	18	9-2 BOTH CELLS FULL
Combination	19	9-2 LT CELL FULL
Combination	20	9-2 BOTH CELLS EMPTY
Combination	21	9-2 BOTH CELLS FULL PR
Combination	22	9-2 LT CELL FULL PR
Combination	23	P INT WATER 2 CELLS NO SOIL
Combination	24	P EXT WATER LT CELL NO SOIL
Combination	25	9-2 CST LOADING PSTN1
Combination	26	9-2 CST LOADING PSTN2

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	Rect 24.00x12.00	288.000	3.46E 3	13.8E 3	9.49E 3	CONCRETE

Basic Load Cases

Number	Name
1	CONC SLF WGHT
2	SOIL
3	EXT WATER
4	INT WATER 2 CELLS
5	INT WATER LT CELL
6	P WATER 2 CELLS
7	P WATER LT CELL
8	CST SOIL
9	CST CAT 740 TRUCK PSTN1
10	CST CAT 740 TRUCK PSTN1

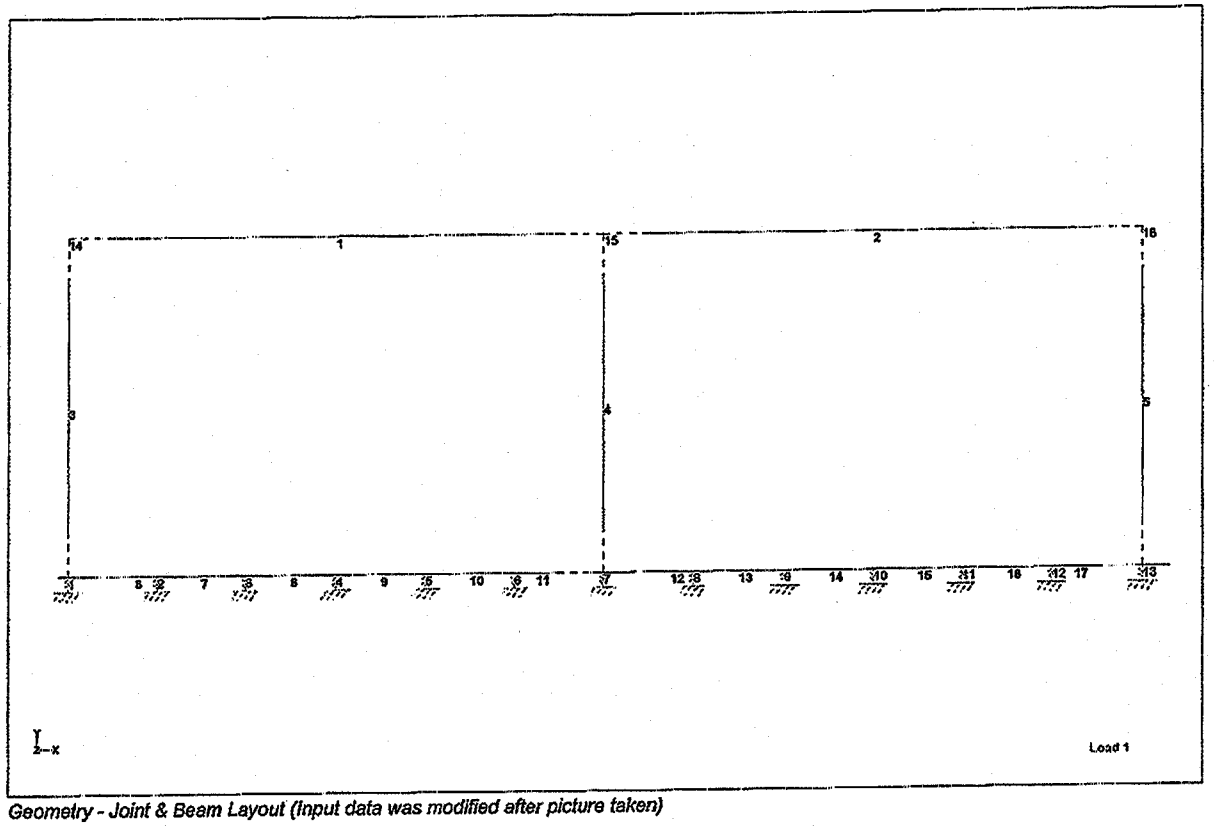
Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
11	SERV BRG PRESSURE	1	CONC SLF WGHT	1.00
		2	SOIL	1.00
		3	EXT WATER	1.00
		4	INT WATER 2 CELLS	1.00
12	SERV BRG PR W/INT PRESSURE	1	CONC SLF WGHT	1.00
		2	SOIL	1.00
		3	EXT WATER	1.00
		6	P WATER 2 CELLS	1.00
13	9-1 BOTH CELLS FULL	1	CONC SLF WGHT	1.82
		3	EXT WATER	2.21
		4	INT WATER 2 CELLS	2.21
		2	SOIL	1.00
14	9-1 LT CELL FULL	1	CONC SLF WGHT	1.82
		3	EXT WATER	2.21
		5	INT WATER LT CELL	2.21
		2	SOIL	1.00
15	9-1 BOTH CELLS EMPTY	1	CONC SLF WGHT	1.82
		3	EXT WATER	2.21
		2	SOIL	1.00
16	9-1 BOTH CELLS FULL PR	1	CONC SLF WGHT	1.82
		3	EXT WATER	2.21
		6	P WATER 2 CELLS	2.21
		2	SOIL	1.00
17	9-1 LT CELL FULL PR	1	CONC SLF WGHT	1.82
		3	EXT WATER	2.21
		7	P WATER LT CELL	2.21
		2	SOIL	1.00
18	9-2 BOTH CELLS FULL	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		4	INT WATER 2 CELLS	1.56

Combination Load Cases Cont...

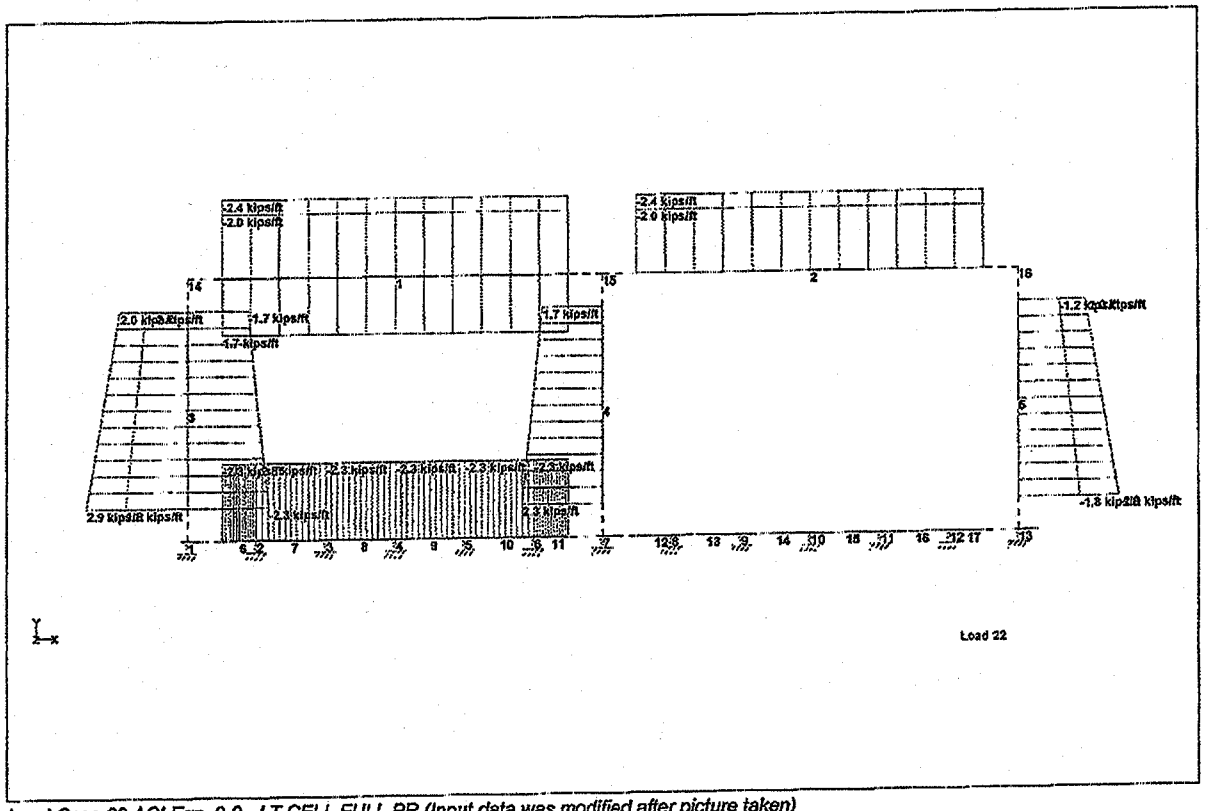
Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
		2	SOIL	2.08
19	9-2 LT CELL FULL	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		5	INT WATER LT CELL	1.56
		2	SOIL	2.08
20	9-2 BOTH CELLS EMPTY	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		2	SOIL	2.08
21	9-2 BOTH CELLS FULL PR	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		6	P WATER 2 CELLS	1.56
		2	SOIL	2.08
22	9-2 LT CELL FULL PR	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		7	P WATER LT CELL	1.56
		2	SOIL	2.08
23	P INT WATER 2 CELLS NO SOIL	1	CONC SLF WGHT	1.82
		6	P WATER 2 CELLS	2.21
24	P EXT WATER LT CELL NO SOIL	1	CONC SLF WGHT	1.82
		7	P WATER LT CELL	2.21
25	9-2 CST LOADING PSTN1	1	CONC SLF WGHT	1.20
		9	CST CAT 740 TRUCK PSTN1	1.69
		8	CST SOIL	1.60
26	9-2 CST LOADING PSTN2	1	CONC SLF WGHT	1.20
		10	CST CAT 740 TRUCK PSTN1	1.69
		8	CST SOIL	1.60

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 4 12	Rev
	Part Conduit		
Job Title C43	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Conduit_Model1.std	Date/Time 12-May-2006 13:12	



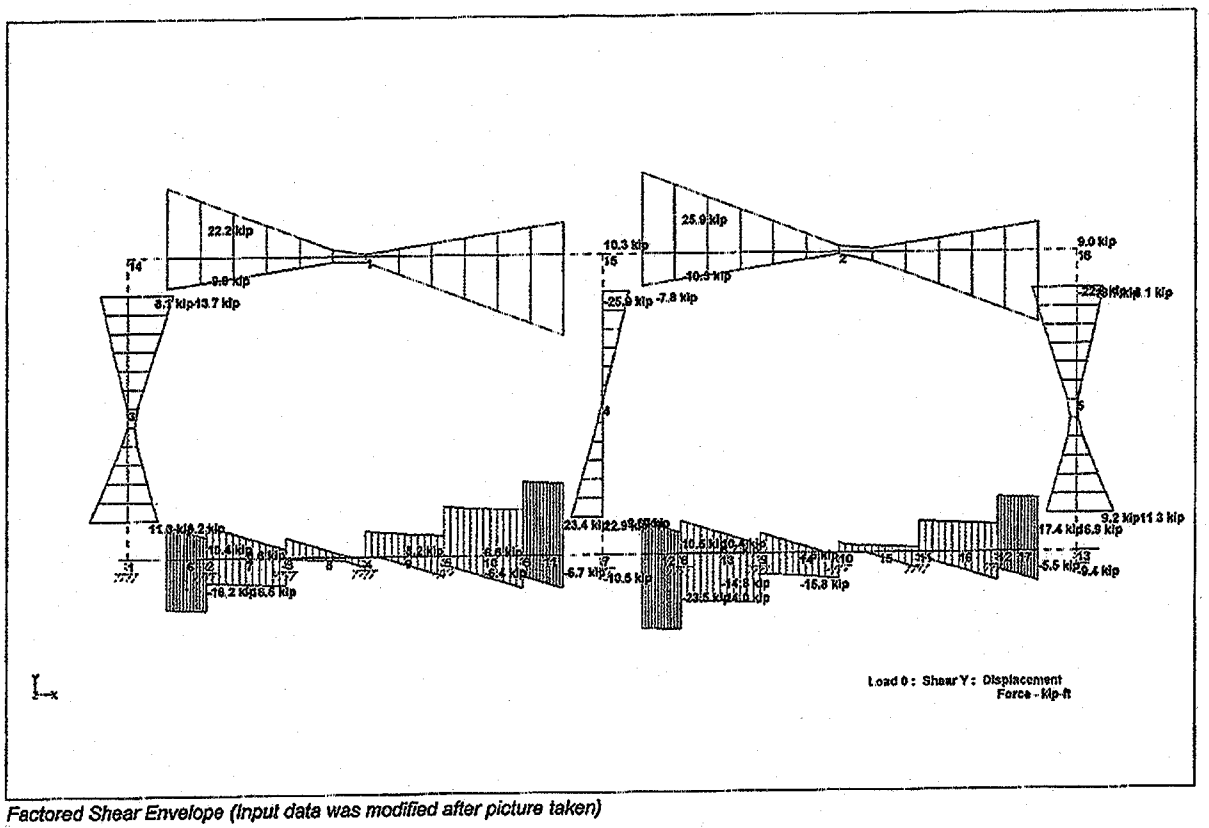
Geometry - Joint & Beam Layout (input data was modified after picture taken)

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 5/13	Rev
	Part Conduit		
Job Title C43	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Conduit_Model1.std	Date/Time 12-May-2006 13:12	



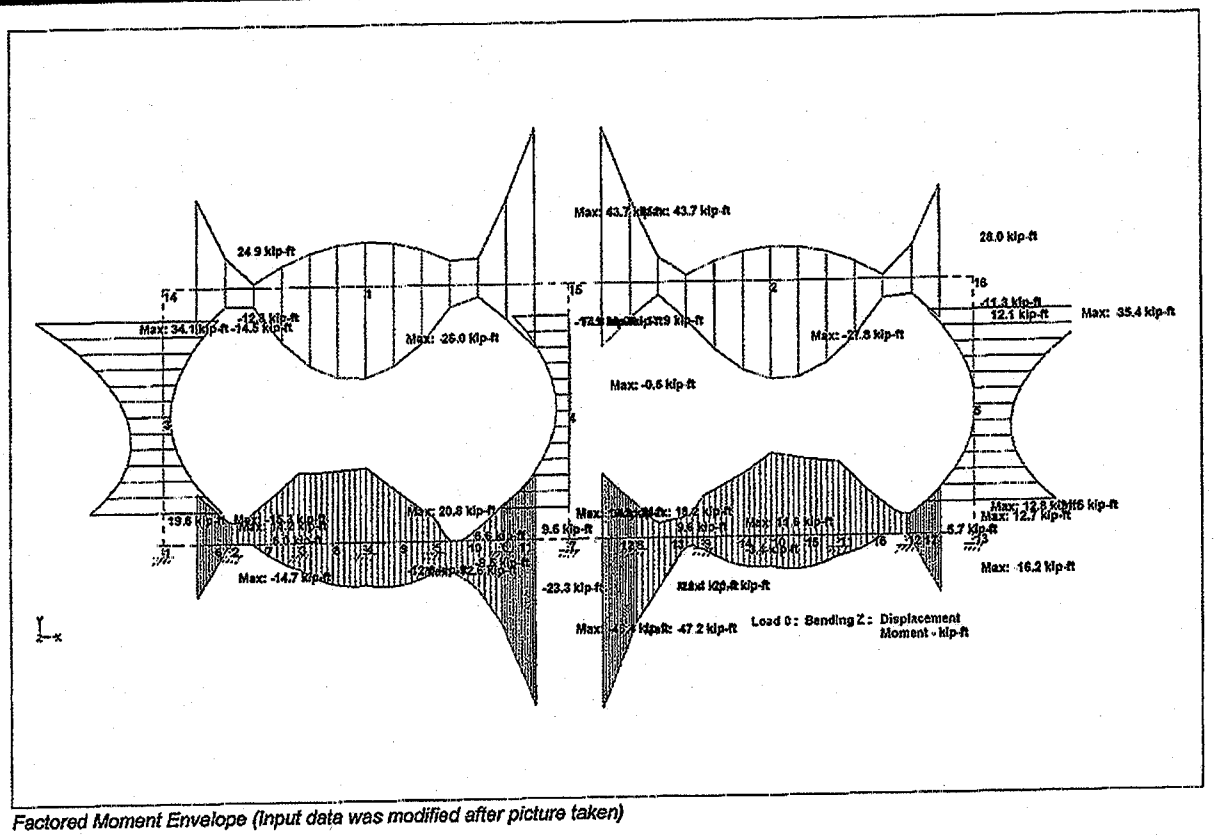
Load Case 22 ACI Eqn. 9-2 - LT CELL FULL PR (Input data was modified after picture taken)

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No ~~6~~ 14	Rev
	Part Conduit		
Job Title C43	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Conduit_Model1.std	Date/Time 12-May-2006 13:12	



Factored Shear Envelope (Input data was modified after picture taken)

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 15	Rev
	Part Conduit		
Job Title C43	Ref		
	By D. Beck	Date May 06	Chd J. Lange
Client US Army Core of Engr	File Conduit_Model1 std	Date/Time 12-May-2006 13:12	



Factored Moment Envelope (Input data was modified after picture taken)

Beam End Forces Envelope

Sign convention is as the action of the joint on the beam.

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip ft)	Mz (kip-ft)
1	14	+ve	13.7	22.2	0.0	0.0	0.0	24.9
			15:9-1 BC	18:9-2 BC	-	-	-	18:9-2 BC
			-8.1	-9.9	0.0	0.0	0.0	-12.8
			23:P INT 1	24:P EXT	-	-	-	24:P EXT
	15	-ve	13.7	10.3	0.0	0.0	0.0	43.7
			15:9-1 BC	23:P INT 1	-	-	-	20:9-2 BC
			-8.1	-25.9	0.0	0.0	0.0	-17.9
			23:P INT 1	20:9-2 BC	-	-	-	23:P INT 1
2	15	+ve	13.7	25.9	0.0	0.0	0.0	43.7
			15:9-1 BC	20:9-2 BC	-	-	-	20:9-2 BC
			-8.1	-10.3	0.0	0.0	0.0	-17.9
			23:P INT 1	23:P INT 1	-	-	-	23:P INT 1
	16	-ve	13.7	9.0	0.0	0.0	0.0	26.0
			15:9-1 BC	23:P INT 1	-	-	-	22:9-2 LT
			-8.1	-22.8	0.0	0.0	0.0	-11.3
			23:P INT 1	22:9-2 LT	-	-	-	23:P INT 1

Beam End Forces Envelope Cont...

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip*ft)	My (kip*ft)	Mz (kip*ft)
2	15	+ve	13.7	25.9	0.0	0.0	0.0	43.7
			15:9-1 BC	20:9-2 BC	-	-	-	20:9-2 BC
			-8.1	-10.3	0.0	0.0	0.0	-17.9
			23:P INT \	23:P INT \	-	-	-	23:P INT \
	16	-ve	13.7	9.0	0.0	0.0	0.0	26.0
			15:9-1 BC	23:P INT \	-	-	-	22:9-2 LT
			-8.1	-22.8	0.0	0.0	0.0	-11.3
			23:P INT \	22:9-2 LT	-	-	-	23:P INT \
3	1	+ve	25.0	11.3	0.0	0.0	0.0	19.6
			18:9-2 BC	15:9-1 BC	-	-	-	20:9-2 BC
			-6.6	-9.2	0.0	0.0	0.0	-15.3
			24:P EXT	24:P EXT	-	-	-	24:P EXT
	14	-ve	22.2	8.1	0.0	0.0	0.0	34.1
			18:9-2 BC	23:P INT \	-	-	-	18:9-2 BC
			-9.9	-13.7	0.0	0.0	0.0	-14.5
			24:P EXT	15:9-1 BC	-	-	-	24:P EXT
4	7	+ve	54.7	9.6	0.0	0.0	0.0	17.8
			20:9-2 BC	17:9-1 LT	-	-	-	17:9-1 LT
			-17.4	-0.0	0.0	0.0	0.0	-0.6
			23:P INT \	26:9-2 CS	-	-	-	25:9-2 CS
	15	-ve	51.9	0.4	0.0	0.0	0.0	14.8
			20:9-2 BC	25:9-2 CS	-	-	-	17:9-1 LT
			20.6	-7.8	0.0	0.0	0.0	-2.9
			23:P INT \	17:9-1 LT	-	-	-	25:9-2 CS
5	13	+ve	25.6	9.2	0.0	0.0	0.0	12.8
			22:9-2 LT	23:P INT \	-	-	-	23:P INT \
			-5.7	-11.3	0.0	0.0	0.0	-21.5
			23:P INT \	14:9-1 LT	-	-	-	17:9-1 LT
	16	-ve	22.8	13.7	0.0	0.0	0.0	12.1
			22:9-2 LT	15:9-1 BC	-	-	-	23:P INT \
			-9.0	-8.1	0.0	0.0	0.0	-35.4
			23:P INT \	23:P INT \	-	-	-	22:9-2 LT
6	1	+ve	0.0	10.4	0.0	0.0	0.0	14.2
			-	24:P EXT	-	-	-	24:P EXT
			0.0	-16.2	0.0	0.0	0.0	-14.7
			-	20:9-2 BC	-	-	-	15:9-1 BC
	2	-ve	0.0	6.6	0.0	0.0	0.0	6.0
			-	24:P EXT	-	-	-	22:9-2 LT
			0.0	-16.6	0.0	0.0	0.0	0.0
			-	20:9-2 BC	-	-	-	-

Beam End Forces Envelope Cont...

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip'ft)	My (kip'ft)	Mz (kip'ft)
9	4	+ve	0.0	8.2	0.0	0.0	0.0	20.8
			-	18:9-2 BC	-	-	-	20:9-2 BC
			0.0	0.0	0.0	0.0	0.0	12.0
			-	-	-	-	-	24:P EXT
	5	-ve	0.0	6.6	0.0	0.0	0.0	6.6
			-	20:9-2 BC	-	-	-	20:9-2 BC
			0.0	-6.4	0.0	0.0	0.0	-8.5
			-	23:P INT 1	-	-	-	24:P EXT
11	6	+ve	0.0	23.4	0.0	0.0	0.0	9.6
			-	18:9-2 BC	-	-	-	23:P INT 1
			0.0	-6.7	0.0	0.0	0.0	-23.3
			-	23:P INT 1	-	-	-	20:9-2 BC
	7	-ve	0.0	22.9	0.0	0.0	0.0	18.2
			-	20:9-2 BC	-	-	-	23:P INT 1
			0.0	-10.5	0.0	0.0	0.0	-46.4
			-	23:P INT 1	-	-	-	20:9-2 BC
12	7	+ve	0.0	10.5	0.0	0.0	0.0	18.2
			-	23:P INT 1	-	-	-	23:P INT 1
			0.0	-23.5	0.0	0.0	0.0	-47.2
			-	19:9-2 LT	-	-	-	19:9-2 LT
	8	-ve	0.0	6.7	0.0	0.0	0.0	9.6
			-	23:P INT 1	-	-	-	23:P INT 1
			0.0	-24.0	0.0	0.0	0.0	-23.4
			-	19:9-2 LT	-	-	-	19:9-2 LT
13	8	+ve	0.0	10.4	0.0	0.0	0.0	9.6
			-	23:P INT 1	-	-	-	23:P INT 1
			0.0	-14.8	0.0	0.0	0.0	-23.4
			-	19:9-2 LT	-	-	-	19:9-2 LT
	9	-ve	0.0	2.6	0.0	0.0	0.0	11.6
			-	23:P INT 1	-	-	-	17:9-1 LT
			0.0	-15.8	0.0	0.0	0.0	-3.4
			-	18:9-2 BC	-	-	-	23:P INT 1
17	12	+ve	0.0	17.4	0.0	0.0	0.0	5.7
			-	22:9-2 LT	-	-	-	21:9-2 BC
			0.0	-5.5	0.0	0.0	0.0	0.0
			-	23:P INT 1	-	-	-	-
	13	-ve	0.0	16.9	0.0	0.0	0.0	12.7
			-	22:9-2 LT	-	-	-	23:P INT 1
			0.0	-9.4	0.0	0.0	0.0	-16.2
			-	23:P INT 1	-	-	-	17:9-1 LT

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
Max Fx	4	7	20:9-2 BOTH C	54.7	-0.0	0.0	0.0	0.0	-0.0
Min Fx	4	15	23:P INT WATI	-20.6	-0.0	0.0	0.0	0.0	0.0
Max Fy	2	15	20:9-2 BOTH C	13.4	25.9	0.0	0.0	0.0	43.7
Min Fy	1	15	20:9-2 BOTH C	13.4	-25.9	0.0	0.0	0.0	43.7
Max Fz	1	14	11:SERV BRG	7.5	12.5	0.0	0.0	0.0	14.1
Min Fz	1	14	11:SERV BRG	7.5	12.5	0.0	0.0	0.0	14.1
Max Mx	1	14	11:SERV BRG	7.5	12.5	0.0	0.0	0.0	14.1
Min Mx	1	14	11:SERV BRG	7.5	12.5	0.0	0.0	0.0	14.1
Max My	1	14	11:SERV BRG	7.5	12.5	0.0	0.0	0.0	14.1
Min My	1	14	11:SERV BRG	7.5	12.5	0.0	0.0	0.0	14.1
Max Mz	1	15	20:9-2 BOTH C	13.4	-25.9	0.0	0.0	0.0	43.7
Min Mz	12	7	19:9-2 LT CELI	0.0	-23.5	0.0	0.0	0.0	-47.2

STAAD CONDUIT
MODEL INPUT FILE

C:\BECK\Jobs\Florida\17300_C43\Staad\Conduit_Model1.std 05/12/06 13:12:48

```
STAAD PLANE
START JOB INFORMATION
JOB NAME C43
JOB CLIENT US Army Core of Engr
JOB NO 17300
JOB PART Conduit*
JOB COMMENT Conduit location just upstream of gates
ENGINEER NAME D. Beck
CHECKER NAME J. Lange
ENGINEER DATE May-06
CHECKER DATE May-06
END JOB INFORMATION
INPUT WIDTH 79
*
*
*
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 2 0 0; 3 4 0 0; 4 6 0 0; 5 8 0 0; 6 10 0 0; 7 12 0 0; 8 14 0 0;
9 16 0 0; 10 18 0 0; 11 20 0 0; 12 22 0 0; 13 24 0 0; 14 0 8 0; 15 12 8 0;
16 24 8 0;
MEMBER INCIDENCES
1 14 15; 2 15 16; 3 1 14; 4 7 15; 5 13 16; 6 1 2; 7 2 3; 8 3 4; 9 4 5; 10 5 6;
11 6 7; 12 7 8; 13 8 9; 14 9 10; 15 10 11; 16 11 12; 17 12 13;
*
*
*
DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 485600
POISSON 0.17
DENSITY 0.15
ALPHA 5.5e-006
DAMP 0.05
END DEFINE MATERIAL
CONSTANTS
MATERIAL CONCRETE MEMB 1 TO 17
MEMBER PROPERTY AMERICAN
1 TO 17 PRIS YD 2 ZD 1
*
*
*
MEMBER OFFSET
1 2 6 12 START 1 0 0
1 2 11 17 END -1 0 0
3 TO 5 START 0 1 0
3 TO 5 END 0 -1 0
*
*
*
SUPPORTS
1 TO 13 FIXED BUT MZ KFY 300
*
*
*
LOAD 1 LOADTYPE Dead  TITLE CONC SLF WGHT
SELFWEIGHT Y -1
LOAD 2 LOADTYPE Soil  TITLE SOIL
MEMBER LOAD
3 TRAP GX 0.86 0.58
5 TRAP GX -0.86 -0.58
```

```
LOAD 3 LOADTYPE Fluids  TITLE EXT WATER
MEMBER LOAD
3 TRAP GX 1.87 1.25
5 TRAP GX -1.87 -1.25
1 2 UNI GY -1.25
LOAD 4 LOADTYPE Fluids  TITLE INT WATER 2 CELLS
MEMBER LOAD
6 TO 17 UNI GY -0.37
4 5 TRAP GX 0.37 0
3 4 TRAP GX -0.37 0
LOAD 5 LOADTYPE Fluids  TITLE INT WATER LT CELL
MEMBER LOAD
6 TO 11 UNI GY -0.37
4 TRAP GX 0.37 0
3 TRAP GX -0.37 0
*
*
*
*P indicates presurrized internal water
LOAD 6 LOADTYPE Fluids  TITLE P WATER 2 CELLS
MEMBER LOAD
1 2 UNI GY 1.12
6 TO 17 UNI GY -1.5
4 5 TRAP GX 1.5 1.12
3 4 TRAP GX -1.5 -1.12
LOAD 7 LOADTYPE Fluids  TITLE P WATER LT CELL
MEMBER LOAD
1 UNI GY 1.12
6 TO 11 UNI GY -1.5
4 TRAP GX 1.5 1.12
3 TRAP GX -1.5 -1.12
*
*
*Construction Loading, Cat 740 truck w/3' of soil cover
LOAD 8 LOADTYPE Soil  TITLE CST SOIL
MEMBER LOAD
1 2 UNI GY -0.36
3 TRAP GX 0.78 0 0 6
5 TRAP GX -0.78 0 0 6
LOAD 9 LOADTYPE Live  TITLE CST CAT 740 TRUCK PSTN1
MEMBER LOAD
1 UNI GY -0.57 0 3.55
1 UNI GY -0.59 3.55 10
2 UNI GY -0.48 3.55 10
LOAD 10 LOADTYPE Live  TITLE CST CAT 740 TRUCK PSTN1
MEMBER LOAD
1 UNI GY -0.57 3.55 10
2 UNI GY -0.59 0 6.45
*
*
*
*Start Load combinations
*
*Service loads for brg pressure only
LOAD COMB 11 SERV BRG PRESSURE
1 1.0 2 1.0 3 1.0 4 1.0
LOAD COMB 12 SERV BRG PR W/INT PRESSURE
1 1.0 2 1.0 3 1.0 6 1.0
*
*
*
```

21


```
LOAD COMB 13 9-1 BOTH CELLS FULL
1 1.82 3 2.21 4 2.21 2 1.0
LOAD COMB 14 9-1 LT CELL FULL
1 1.82 3 2.21 5 2.21 2 1.0
LOAD COMB 15 9-1 BOTH CELLS EMPTY
1 1.82 3 2.21 2 1.0
*Pressurized below
LOAD COMB 16 9-1 BOTH CELLS FULL PR
1 1.82 3 2.21 6 2.21 2 1.0
LOAD COMB 17 9-1 LT CELL FULL PR
1 1.82 3 2.21 7 2.21 2 1.0
*
*
*
LOAD COMB 18 9-2 BOTH CELLS FULL
1 1.56 3 1.56 4 1.56 2 2.08
LOAD COMB 19 9-2 LT CELL FULL
1 1.56 3 1.56 5 1.56 2 2.08
LOAD COMB 20 9-2 BOTH CELLS EMPTY
1 1.56 3 1.56 2 2.08
*Pressurized below
LOAD COMB 21 9-2 BOTH CELLS FULL PR
1 1.56 3 1.56 6 1.56 2 2.08
LOAD COMB 22 9-2 LT CELL FULL PR
1 1.56 3 1.56 7 1.56 2 2.08
*
*
*Add'l Load combos neglecting soil
*(pressurized int water w/ no soil resistance unlikely)
LOAD COMB 23 P INT WATER 2 CELLS NO SOIL
1 1.82 6 2.21
LOAD COMB 24 P EXT WATER LT CELL NO SOIL
1 1.82 7 2.21
*
*
*Cst Loading
LOAD COMB 25 9-2 CST LOADING PSTN1
1 1.2 9 1.69 8 1.6
LOAD COMB 26 9-2 CST LOADING PSTN2
1 1.2 10 1.69 8 1.6
PERFORM ANALYSIS
FINISH
```

Conduit Roof Slab Design

Slab Details

GEOMETRY

Slab depth =	24.0	in
Design width =	12.0	in
Top clr =	3.0	in
Bot clr =	6.0	in

MATERIAL PROPERTIES

Concrete			Steel Reinf		
f'_c =	3500	psi	f_y =	60000	psi
E_c =	3.41E+06	psi	E_s =	2.90E+07	psi
f_r =	444	psi			
n =	8.51				

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#6		#6	
Bar spa =	10.0	in	8.0	in
Bar dia =	0.75	in	0.75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\ prv'd}$ =	0.53	in2	0.66	in2
No. bars =	1.20		1.50	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 18.86 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot Mat	Top Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	27.80	43.70	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	17.63	in
A_s, prv'd =	0.53	in2
ρ =	0.00250	
k =	0.1860	
j =	0.9380	

Comp. Stress Concrete, fc =	37 psi	1400 psi Allowable	OK	Add'l Prv'd = 37.93
Tensile Stress Reinf., fs =	1375 psi	24000 psi Allowable	OK	Add'l Prv'd = 17.46

Top Reinf Mat: Allowable stress = 100%

d = 20.63 in

A_s, prv'd = 0.66 in2
ρ = 0.00267
k = 0.1915
j = 0 9362

Comp. Stress Concrete, fc =	26 psi	1400 psi Allowable	OK	Add'l Prv'd = 53.39
Tensile Stress Reinf., fs =	942 psi	24000 psi Allowable	OK	Add'l Prv'd = 25.49

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	27 8 ft-lb	I_g =	13824 in4
$4/3M_u$ =	37.1 ft-lb	y_t =	12 0 in
$1.2M_{cr}$ =	51 1 ft-lb		
M_{design} =	37.1 ft-lb		

a = 0.887 in
ϕM_n = 40.8 ft-lb OK Add'l Prv'd = 1.1

Top Reinf Mat:

Demand Moments:

M_u =	43.7 ft-lb
$4/3M_u$ =	58.3 ft-lb
$1.2M_{cr}$ =	51.1 ft-lb
M_{design} =	51.1 ft-lb

a = 1.109 in
ϕM_n = 59.6 ft-lb OK Add'l Prv'd = 1.17

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	5 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1

OK Add'l Prv'd = 11.89

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 18 86 kips
ϕV_c = 21.27 kips OK Add'l Prv'd = 1 13

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Conduit Floor Slab Design

Slab Details

GEOMETRY

Slab depth =	28.0 in
Design width =	12.0 in
Top clr =	6.0 in
Bot clr =	4.0 in

MATERIAL PROPERTIES

Concrete

f_c =	3500 psi
E_c =	3.41E+06 psi
f_r =	444 psi
n =	8.51

Steel Reinf

f_y =	60000 psi
E_s =	2.90E+07 psi

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#6		#5	
Bar spa =	8.0	in	10.0	in
Bar dia =	0.75	in	0.625	in
Bar area =	0.44	in2	0.31	in2
$A_{s\ prv'd}$ =	0.66	in2	0.37	in2
No. bars =	1.50		1.20	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 24.00 kips

Note: shear taken distance d from face of support

	Bot Mat	Top Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	47.20	23.30	ft-kips

NOTE: ASD design NOT used, ignore all input and results

Sensitive to Kv spring value, at 150kcf = 24kips at 300kcf = 21.9 kips (26" req'd)

-Shear also drops off quickly away from center wall, could thicken sect at center wall only. Only 17 kips at ext walls (22" thick req'd)

FLEXURE DESIGN

Working Stress

Bot Reinf Mat:

Allowable stress = 100%

d =	23.63 in			
A_s, prv'd =	0.66 in2			
ρ =	0.00233			
k =	0.1802			
j =	0.9399			
Comp. Stress Concrete, fc =	21 psi	1400 psi Allowable	OK	Add'l Prv'd = 66.18
Tensile Stress Reinf., fs =	819 psi	24000 psi Allowable	OK	Add'l Prv'd = 29.31

Top Reinf Mat:

Allowable stress = 100%

d = 21.69 in

A_s, prv'd = 0.37 in2
ρ = 0.00143
k = 0.1443
j = 0.9519

Comp. Stress Concrete, fc =	31 psi	1400 psi Allowable	OK	Add'l Prv'd = 45.21
Tensile Stress Reinf., fs =	1563 psi	24000 psi Allowable	OK	Add'l Prv'd = 15.36

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	47.2 ft-lb	I_g =	21952 in4
$4/3M_u$ =	62.9 ft-lb	y_t =	14.0 in
$1.2M_{cr}$ =	69.6 ft-lb		
M_{design} =	62.9 ft-lb		
a =	1.109 in		
ϕM_n =	68.5 ft-lb	OK	Add'l Prv'd = 1.09

Top Reinf Mat:

Demand Moments:

M_u =	23.3 ft-lb		
$4/3M_u$ =	31.1 ft-lb		
$1.2M_{cr}$ =	69.6 ft-lb		
M_{design} =	31.1 ft-lb		
a =	0.625 in		
ϕM_n =	35.8 ft-lb	OK	Add'l Prv'd = 1.15

SHEAR DESIGN

Working Stress

Allowable stress =	100%		
Shear stress, v =	4 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)	
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1	
		OK	Add'l Prv'd = 14.63

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u =	24.00 kips		
ϕV_c =	26.17 kips	OK	Add'l Prv'd = 1.09

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Conduit Exterior Wall Design

Slab Details

GEOMETRY

Slab depth =	18.0 in
Design width =	12.0 in
Top clr =	3.0 in (ext face)
Bot clr =	6.0 in (int face)

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot (int) mat:		Top (ext) mat:	
Bar size =	#6		#6	
Bar spa =	12.0	in	8.0	in
Bar dia =	0.75	in	0.75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\,prv'd}$ =	0.44	in2	0.66	in2
No. bars =	1.00		1.50	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips
Factored Shear, V_u = 10.40 kips
Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	15.30	35.40	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat:

Allowable stress = 100%

d =	11.63 in			
A_S, prv'd =	0.44 in2			
ρ =	0.00315			
k =	0.2064			
j =	0.9312			
Comp Stress Concrete, fc =	77 psi	1400 psi Allowable	OK	Add'l Prv'd = 18.18
Tensile Stress Reinf., fs =	2519 psi	24000 psi Allowable	OK	Add'l Prv'd = 9.53

Top Reinf Mat:

Allowable stress = 100%

d =	14.63 in
A_s, prv'd =	0.66 in2
ρ =	0.00376
k =	0.2230
j =	0.9257

Comp Stress Concrete, fc =	45 psi	1400 psi Allowable	OK	Add'l Prv'd = 30.9
Tensile Stress Reinf., fs =	1343 psi	24000 psi Allowable	OK	Add'l Prv'd = 17.87

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	15.3 ft-lb	I_g =	5832 in4
$4/3M_u$ =	20.4 ft-lb	y_t =	9.0 in
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	20.4 ft-lb		

a = 0.739 in
ϕM_n = 22.3 ft-lb OK Add'l Prv'd = 1.09

Top Reinf Mat:

Demand Moments:

M_u =	35.4 ft-lb
$4/3M_u$ =	47.2 ft-lb
$1.2M_{cr}$ =	28.8 ft-lb
M_{design} =	35.4 ft-lb

a = 1.109 in
ϕM_n = 41.8 ft-lb OK Add'l Prv'd = 1.18

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	7 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 7.84

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 10.40 kips
ϕV_c = 14.03 kips OK Add'l Prv'd = 1.35

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Conduit Interior Wall Design

Slab Details

GEOMETRY

Slab depth =	18.0 in
Design width =	12.0 in
Top clr =	6.0 in
Bot clr =	6.0 in

MATERIAL PROPERTIES

Concrete

f'_c =	3500	psi
E_c =	3.41E+06	psi
f_r =	444	psi
n =	8.51	

Steel Reinf

f_y =	60000	psi
E_s =	2.90E+07	psi

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#6		#6	
Bar spa =	10.0	in	10.0	in
Bar dia =	0.75	in	0.75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\ prv'd}$ =	0.53	in2	0.53	in2
No bars =	1.20		1.20	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 6.50 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	17.80	17.80	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat:

Allowable stress = 100%

d =	11.63	in
A_S, prv'd =	0.53	in2
ρ =	0.00378	
k =	0.2236	
j =	0.9255	

Comp. Stress Concrete, fc =	72 psi	1400 psi Allowable	OK	Add'l Prv'd = 19.58
Tensile Stress Reinf, fs =	2112 psi	24000 psi Allowable	OK	Add'l Prv'd = 11.36

Top Reinf Mat:

Allowable stress = 100%

d = 11.63 in

A_s, prv'd = 0.53 in2
ρ = 0.00378
k = 0.2236
j = 0.9255

Comp. Stress Concrete, fc =	72 psi	1400 psi Allowable	OK	Add'l Prv'd = 19.58
Tensile Stress Reinf., fs =	2112 psi	24000 psi Allowable	OK	Add'l Prv'd = 11.36

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	17.8 ft-lb	I_g =	5832 in4
$4/3M_u$ =	23.7 ft-lb	y_t =	9.0 in
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	23.7 ft-lb		

a = 0.887 in
ϕM_n = 26.6 ft-lb OK Add'l Prv'd = 1.12

Top Reinf Mat:

Demand Moments:

M_u =	17.8 ft-lb
$4/3M_u$ =	23.7 ft-lb
$1.2M_{cr}$ =	28.8 ft-lb
M_{design} =	23.7 ft-lb

a = 0.887 in
ϕM_n = 26.6 ft-lb OK Add'l Prv'd = 1.12

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	7 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 7.84

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 6.50 kips
ϕV_c = 14.03 kips OK Add'l Prv'd = 2.16

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Quantitiy Estimates

Conduit

Concrete (cy)

124.6 ft3/ft (x-area measured in Cadd)
4.61 cy/ft

Steel Reinforcing (lbs)

#5 90 deg hook 0.83 ft
#6 90 deg hook 1.00 ft

Size	Weight (lb/ft)	No. Req'd	Location	Epoxy	Type	Length	Total ft	No. Splices	Splice Length	A ft	B ft	C ft	D ft	E ft	F ft	G ft	H ft	WT lb
Conduit																		
			Roof Slab															
#6	1.502	1.5	Transv top mat			23.83	36	0	0.00	23.83								54
#6	1.502	1.2	Transv bot mat			23.83	29	0	0.00	23.83								43
#5	1.043	25	Longit top mat			1.00	25	0	0.00	1.00								26
#5	1.043	25	Bot transv mat			1.00	25	0	0.00	1.00								26
			Floor Slab															
#5	1.043	1.2	Transv top mat			24.50	29	0	0.00	24.50								31
#6	1.502	1.5	Transv bot mat			24.50	37	0	0.00	24.50								55
#5	1.043	26	Longit top mat			1.00	26	0	0.00	1.00								27
#5	1.043	26	Bot transv mat			1.00	26	0	0.00	1.00								27
			Thickened Floor Slab															
#5	1.043	0	Transv			0.00	0	0	0.00	0.00								0
#5	1.043	4	Longit			1.00	4	0	0.00	1.00								4
			Exterior Walls															
			No. Ext Walls =	2														
#6	1.502	1.5	Vert ext face (2 bars w/lap & hooks)			14.17	21	0	0.00	1.00	4.50	7.67	1.00					32
#6	1.502	1	Vert int face (2 bars w/lap & hooks)			13.33	13	0	0.00	0.83	4.00	7.67	0.83					20
#5	1.043	7	Longit ext face			1.00	7	0	0.00	1.00								7
#5	1.043	7	Longit int face			1.00	7	0	0.00	1.00								7
																Total for 2nd wall =		67
			Interior Wall															
#6	1.502	1.2	Vert Lt face (2 bars w/lap & hooks)			15.50	19	0	0.00	1.83	5.00	7.67	1.00					28
#6	1.502	1.2	Vert Rt face (2 bars w/lap & hooks)			15.50	19	0	0.00	1.83	5.00	7.67	1.00					28
#5	1.043	7	Longit ext face			1.00	7	0	0.00	1.00								7
#5	1.043	7	Longit int face			1.00	7	0	0.00	1.00								7
																	Total all =	500 lb/ft

Job Information

	Engineer	Checked	Approved
Name:	D. Beck	J. Lange	
Date:	May-06	May-06	

Comments

Gatewell location just above conduit
flared section

Structure Type	PLANE FRAME

Number of Nodes	30	Highest Node	30
Number of Elements	31	Highest Beam	31

Number of Basic Load Cases	7
Number of Combination Load Cases	5

Included in this printout are data for:

Beams	3 17 19 27 25 5 1 9 11 6

Included in this printout are results for load cases:

Type	L/C	Name
Combination	8	INT WATER 2 CELLS
Combination	9	INT WATER LT CELL
Combination	10	ALL SOIL & EXT WATER - EMPTY CELL!
Combination	11	ALL SOIL & EXT WATER - BOTH CELLS
Combination	12	ALL SOIL & EXT WATER - LT CELL

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	Rect 18.00x12.00	216.000	2.59E 3	5.83E 3	6.09E 3	CONCRETE

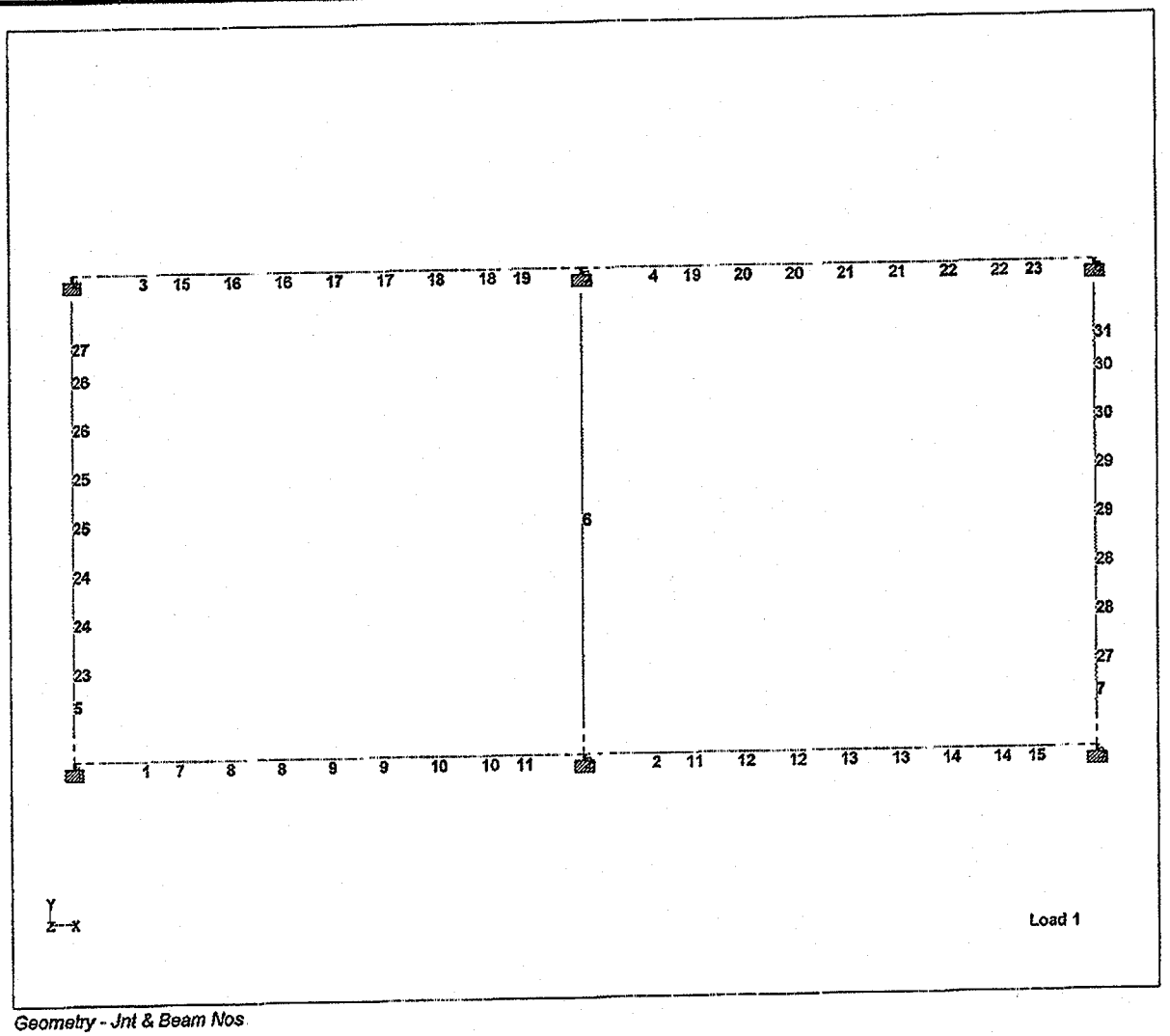
Basic Load Cases

Number	Name
1	SOIL DWNSTRM
2	SUBM SOIL DWNSTRM
3	SUBM SOIL UPSTRM
4	EXT WATER DWNSTRM
5	EXT WATER UPSTRM
6	INT WATER 2 CELLS
7	INT WATER LT CELL

Combination Load Cases

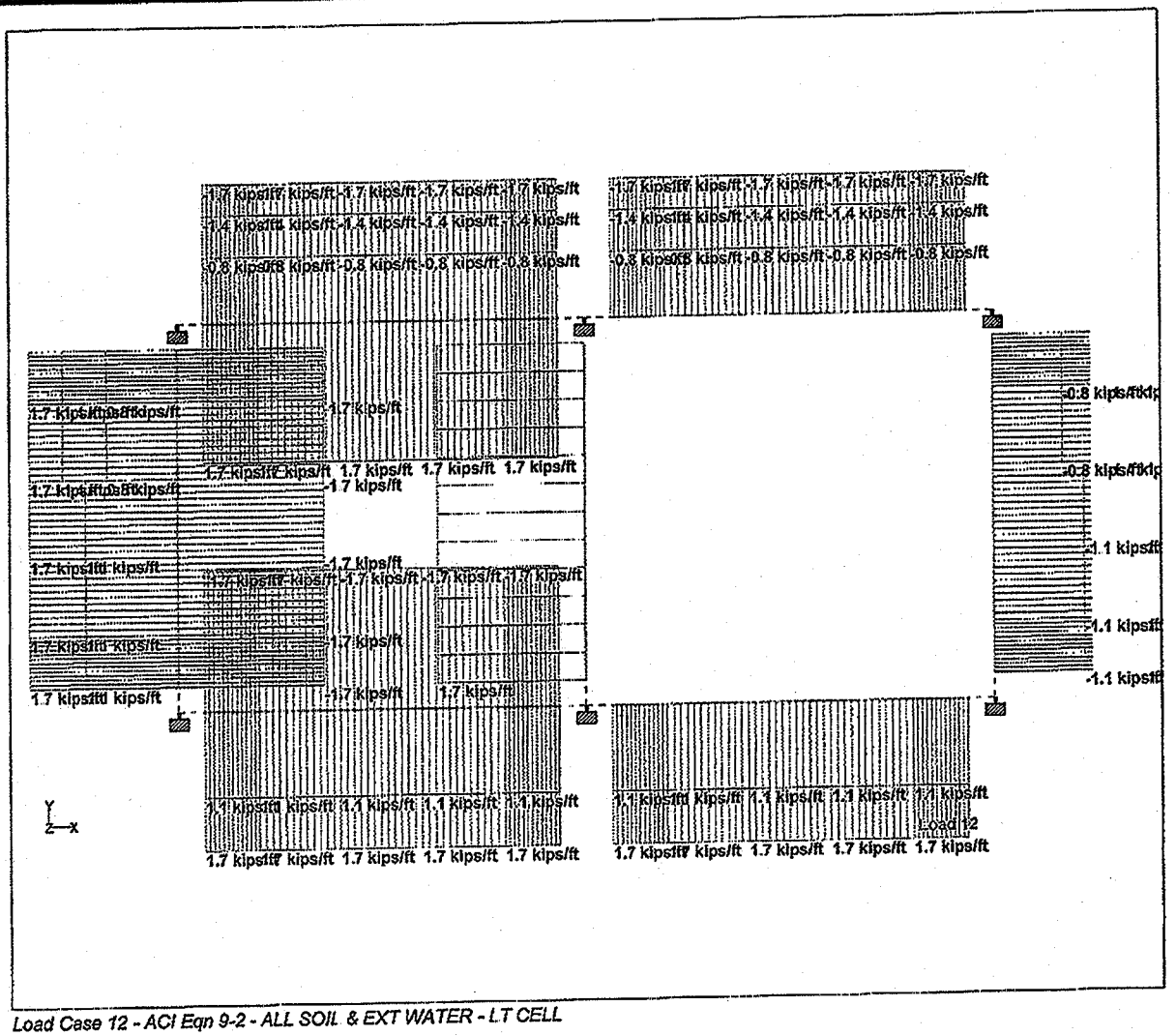
Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
8	INT WATER 2 CELLS	6	INT WATER 2 CELLS	2.21
9	INT WATER LT CELL	7	INT WATER LT CELL	2.21
10	ALL SOIL & EXT WATER - EMPTY CELL!	1	SOIL DWNSTRM	2.08
		2	SUBM SOIL DWNSTRM	2.08
		3	SUBM SOIL UPSTRM	2.08
		4	EXT WATER DWNSTRM	1.56
		5	EXT WATER UPSTRM	1.56
11	ALL SOIL & EXT WATER - BOTH CELLS	1	SOIL DWNSTRM	2.08
		2	SUBM SOIL DWNSTRM	2.08
		3	SUBM SOIL UPSTRM	2.08
		4	EXT WATER DWNSTRM	1.56
		5	EXT WATER UPSTRM	1.56
		6	INT WATER 2 CELLS	1.56
12	ALL SOIL & EXT WATER - LT CELL	1	SOIL DWNSTRM	2.08
		2	SUBM SOIL DWNSTRM	2.08
		3	SUBM SOIL UPSTRM	2.08
		4	EXT WATER DWNSTRM	1.56
		5	EXT WATER UPSTRM	1.56
		7	INT WATER LT CELL	1.56

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 33	Rev
	Part Gatewell		
Job Title C43 West Storage Reservoir	Ref		
	By D. Beck	Date May-06	Chd J Lange
Client US Army Core of Engr	File Gatewell_Model1.std	Date/Time 12-May-2006 13:15	



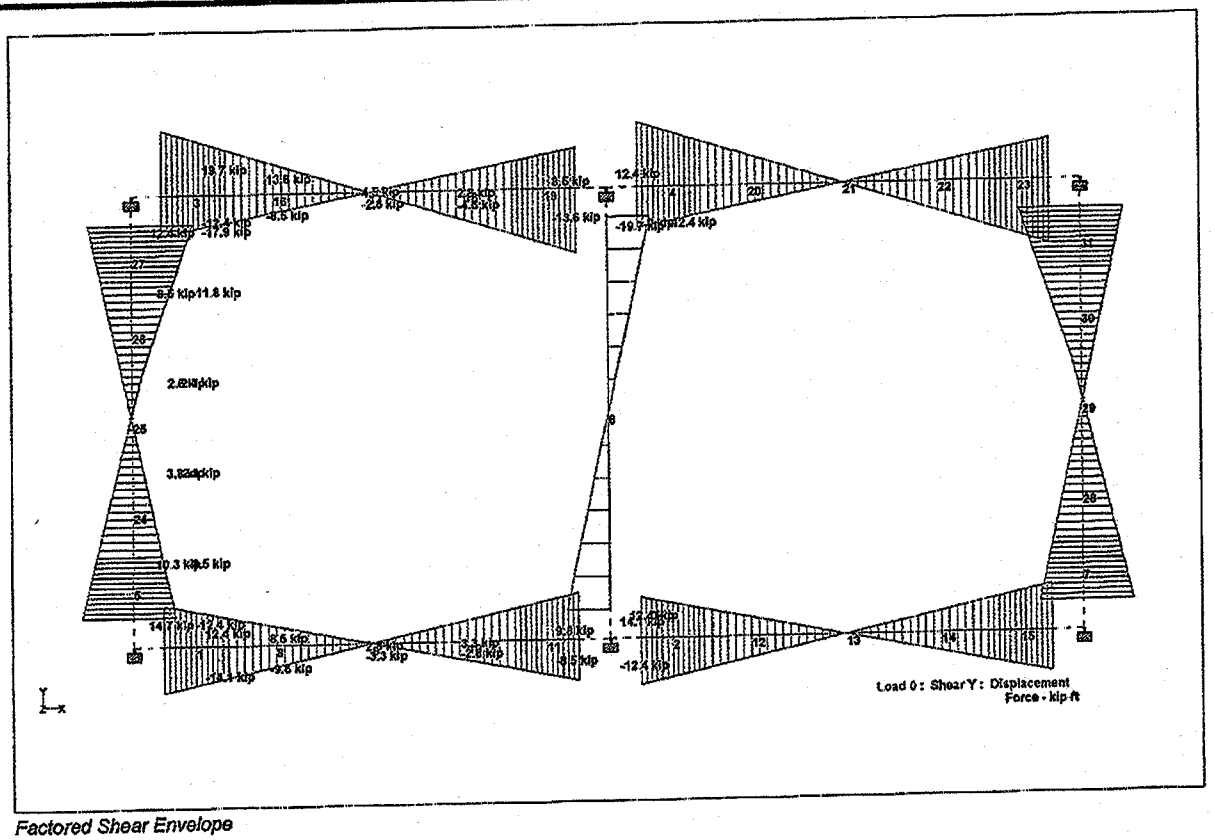
Geometry - Jnt & Beam Nos

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No #34	Rev
	Part Gatewell		
Job Title C43 West Storage Reservoir	Ref		
	By D Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Gatewell_Model1.std	Date/Time 12-May-2006 13:15	

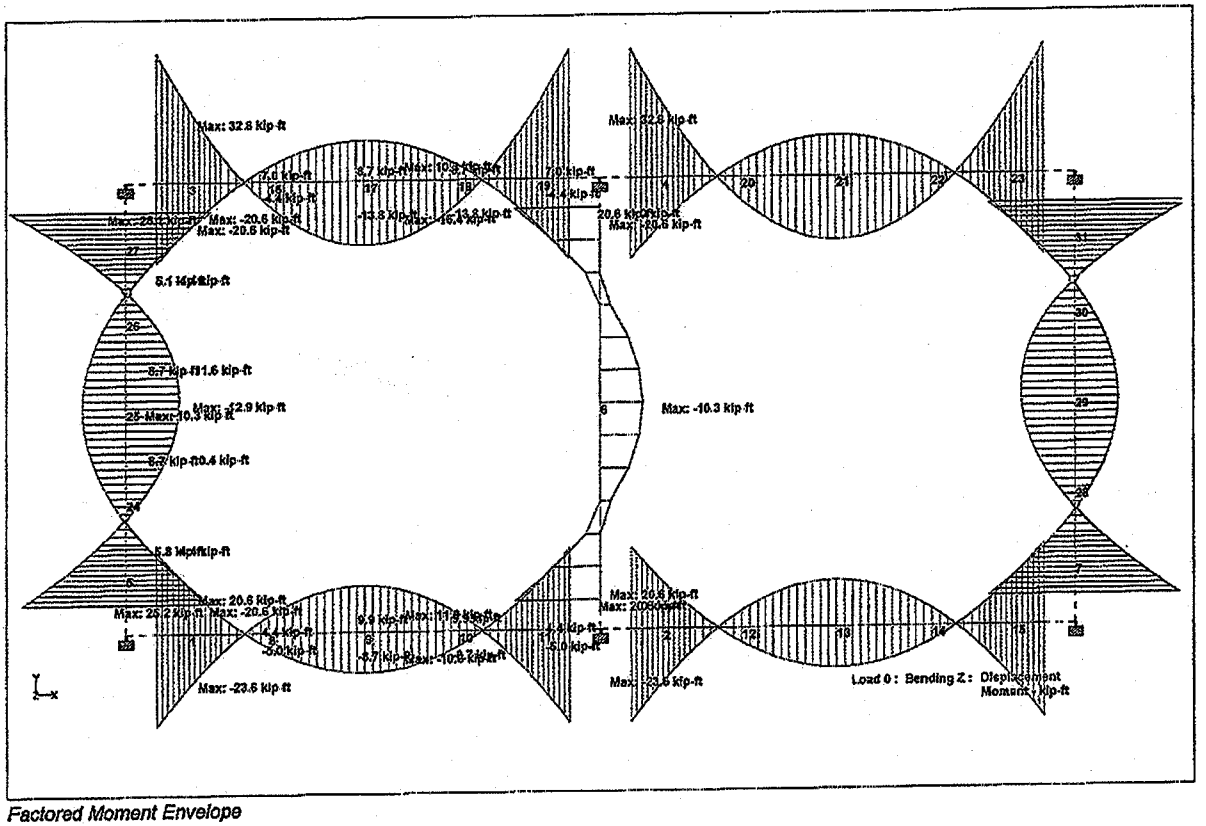


Load Case 12 - ACI Eqn 9-2 - ALL SOIL & EXT WATER - LT CELL

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 35	Rev
	Part Gatewell		
Job Title C43 West Storage Reservoir	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Gatewell_Model1.std	Date/Time 12-May-2006 13:15	



Factored Shear Envelope



Factored Moment Envelope

Beam End Forces Envelope

Sign convention is as the action of the joint on the beam.

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1	1	+ve	0.0	12.4	0.0	0.0	0.0	20.6
			-	8:INT WA	-	-	-	8:INT WA
			0.0	-14.1	0.0	0.0	0.0	-23.6
			-	10:ALL S(	-	-	-	10:ALL S(
	7	-ve	0.0	8.5	0.0	0.0	0.0	4.4
			-	8:INT WA	-	-	-	8:INT WA
			0.0	-9.8	0.0	0.0	0.0	-5.0
			-	10:ALL S(	-	-	-	10:ALL S(
3	4	+ve	0.0	19.7	0.0	0.0	0.0	32.8
			-	10:ALL S(	-	-	-	10:ALL S(
			0.0	-12.4	0.0	0.0	0.0	-20.6
			-	8:INT WA	-	-	-	8:INT WA
	15	-ve	0.0	13.6	0.0	0.0	0.0	7.0
			-	10:ALL S(	-	-	-	10:ALL S(
			0.0	-8.5	0.0	0.0	0.0	-4.4
			-	8:INT WA	-	-	-	8:INT WA

Beam End Forces Envelope Cont...

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip ft)	My (kip ft)	Mz (kip ft)
3	4	+ve	0.0	19.7	0.0	0.0	0.0	32.8
			-	10:ALL S(	-	-	-	10:ALL S(
			0.0	-12.4	0.0	0.0	0.0	-20.6
			-	8:INT WA	-	-	-	8:INT WA
	15	-ve	0.0	13.6	0.0	0.0	0.0	7.0
			-	10:ALL S(	-	-	-	10:ALL S(
			0.0	-8.5	0.0	0.0	0.0	-4.4
			-	8:INT WA	-	-	-	8:INT WA
5	1	+ve	0.0	14.7	0.0	0.0	0.0	25.2
			-	10:ALL S(	-	-	-	10:ALL S(
			0.0	-12.4	0.0	0.0	0.0	-20.6
			-	8:INT WA	-	-	-	8:INT WA
	23	-ve	0.0	10.3	0.0	0.0	0.0	5.8
			-	10:ALL S(	-	-	-	10:ALL S(
			0.0	-8.5	0.0	0.0	0.0	-4.4
			-	8:INT WA	-	-	-	8:INT WA
6	2	+ve	0.0	12.4	0.0	0.0	0.0	20.6
			-	9:INT WA	-	-	-	9:INT WA
			0.0	-0.0	0.0	0.0	0.0	-0.0
			-	8:INT WA	-	-	-	8:INT WA
	5	-ve	0.0	0.0	0.0	0.0	0.0	20.6
			-	8:INT WA	-	-	-	9:INT WA
			0.0	-12.4	0.0	0.0	0.0	-0.0
			-	9:INT WA	-	-	-	8:INT WA
9	8	+ve	0.0	2.8	0.0	0.0	0.0	9.9
			-	8:INT WA	-	-	-	10:ALL S(
			0.0	-3.3	0.0	0.0	0.0	-8.7
			-	10:ALL S(	-	-	-	8:INT WA
	9	-ve	0.0	3.3	0.0	0.0	0.0	9.9
			-	10:ALL S(	-	-	-	10:ALL S(
			0.0	-2.8	0.0	0.0	0.0	-8.7
			-	8:INT WA	-	-	-	8:INT WA
11	10	+ve	0.0	9.8	0.0	0.0	0.0	4.4
			-	10:ALL S(	-	-	-	8:INT WA
			0.0	-8.5	0.0	0.0	0.0	-5.0
			-	8:INT WA	-	-	-	10:ALL S(
	2	-ve	0.0	14.1	0.0	0.0	0.0	20.6
			-	10:ALL S(	-	-	-	8:INT WA
			0.0	-12.4	0.0	0.0	0.0	-23.6
			-	8:INT WA	-	-	-	10:ALL S(

Beam End Forces Envelope Cont...

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip ft)	My (kip ft)	Mz (kip ft)
17	16	+ve	0.0	4.5	0.0	0.0	0.0	8.7
			-	10:ALL S(	-	-	-	8:INT WA
			0.0	-2.8	0.0	0.0	0.0	-13.8
			-	8:INT WA	-	-	-	10:ALL S(
	17	-ve	0.0	2.8	0.0	0.0	0.0	8.7
			-	8:INT WA	-	-	-	8:INT WA
			0.0	-4.5	0.0	0.0	0.0	-13.8
			-	10:ALL S(	-	-	-	10:ALL S(
19	18	+ve	0.0	8.5	0.0	0.0	0.0	7.0
			-	8:INT WA	-	-	-	10:ALL S(
			0.0	-13.6	0.0	0.0	0.0	-4.4
			-	10:ALL S(	-	-	-	8:INT WA
	5	-ve	0.0	12.4	0.0	0.0	0.0	32.8
			-	8:INT WA	-	-	-	10:ALL S(
			0.0	-19.7	0.0	0.0	0.0	-20.6
			-	10:ALL S(	-	-	-	8:INT WA
25	24	+ve	0.0	3.8	0.0	0.0	0.0	8.7
			-	10:ALL S(	-	-	-	8:INT WA
			0.0	-2.8	0.0	0.0	0.0	-10.4
			-	8:INT WA	-	-	-	10:ALL S(
	25	-ve	0.0	2.8	0.0	0.0	0.0	8.7
			-	8:INT WA	-	-	-	8:INT WA
			0.0	-2.7	0.0	0.0	0.0	-11.6
			-	10:ALL S(	-	-	-	10:ALL S(
27	26	+ve	0.0	8.5	0.0	0.0	0.0	5.1
			-	8:INT WA	-	-	-	10:ALL S(
			0.0	-11.8	0.0	0.0	0.0	-4.4
			-	10:ALL S(	-	-	-	8:INT WA
	4	-ve	0.0	12.4	0.0	0.0	0.0	28.1
			-	8:INT WA	-	-	-	10:ALL S(
			0.0	-17.9	0.0	0.0	0.0	-20.6
			-	10:ALL S(	-	-	-	8:INT WA

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip ft)	My (kip ft)	Mz (kip ft)
Max Fx	1	1	8:INT WATER	0.0	12.4	0.0	0.0	0.0	20.6
Min Fx	1	1	8:INT WATER	0.0	12.4	0.0	0.0	0.0	20.6
Max Fy	3	4	10:ALL SOIL &	0.0	19.7	0.0	0.0	0.0	32.8
Min Fy	19	5	10:ALL SOIL &	0.0	-19.7	0.0	0.0	0.0	32.8
Max Fz	1	1	8:INT WATER	0.0	12.4	0.0	0.0	0.0	20.6
Min Fz	1	1	8:INT WATER	0.0	12.4	0.0	0.0	0.0	20.6

```
STAAD PLANE
START JOB INFORMATION
JOB NAME C43 West Storage Reservoir
JOB CLIENT US Army Core of Engr
JOB NO 17300
JOB PART Gatewell
JOB COMMENT Gatewell location just above conduit
JOB COMMENT flared section
ENGINEER NAME D. Beck
CHECKER NAME J. Lange
ENGINEER DATE May-06
CHECKER DATE May-06
END JOB INFORMATION
INPUT WIDTH 79
*
*
*
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 11.5 0 0; 3 23 0 0; 4 0 11.5 0; 5 11.5 11.5 0; 6 23 11.5 0;
7 2.3 0 0; 8 4.6 0 0; 9 6.9 0 0; 10 9.2 0 0; 11 13.8 0 0; 12 16.1 0 0;
13 18.4 0 0; 14 20.7 0 0; 15 2.3 11.5 0; 16 4.6 11.5 0; 17 6.9 11.5 0;
18 9.2 11.5 0; 19 13.8 11.5 0; 20 16.1 11.5 0; 21 18.4 11.5 0; 22 20.7 11.5 0;
23 0 2.3 0; 24 0 4.6 0; 25 0 6.9 0; 26 0 9.2 0; 27 23 2.3 0; 28 23 4.6 0;
29 23 6.9 0; 30 23 9.2 0;
MEMBER INCIDENCES
1 1 7; 2 2 11; 3 4 15; 4 5 19; 5 1 23; 6 2 5; 7 3 27; 8 7 8; 9 8 9; 10 9 10;
11 10 2; 12 11 12; 13 12 13; 14 13 14; 15 14 3; 16 15 16; 17 16 17; 18 17 18;
19 18 5; 20 19 20; 21 20 21; 22 21 22; 23 22 6; 24 23 24; 25 24 25; 26 25 26;
27 26 4; 28 27 28; 29 28 29; 30 29 30; 31 30 6;
*
*
*
DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 485600
POISSON 0.17
DENSITY 0.15
ALPHA 5.5e-006
DAMP 0.05
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
1 TO 31 PRIS YD 1.5 ZD 1
CONSTANTS
MATERIAL CONCRETE MEMB 1 TO 31
*
*
*
*
MEMBER OFFSET
1 TO 7 START LOCAL 0.75 0 0
6 11 15 19 23 27 31 END LOCAL -0.75 0 0
*
*
*
*
*
*
SUPPORTS
1 TO 6 FIXED
*
```

STAAD GATEWELL
MODEL INPUT FILE

40


```
*
*No self wght as model is oout of vertical plane
*self wght causes axial demand only
LOAD 1 LOADTYPE Soil  TITLE SOIL DWNSTRM
MEMBER LOAD
3 4 16 TO 23 UNI GY -0.84
26 27 UNI GX 0.84
30 31 UNI GX -0.84
LOAD 2 LOADTYPE Soil  TITLE SUBM SOIL DWNSTRM
MEMBER LOAD
3 4 16 TO 23 UNI GY -0.4
26 27 UNI GX 0.4
30 31 UNI GX -0.4
LOAD 3 LOADTYPE Soil  TITLE SUBM SOIL UPSTRM
MEMBER LOAD
1 2 8 TO 15 UNI GY 0.52
5 24 25 UNI GX 0.52
7 28 29 UNI GX -0.52
LOAD 4 LOADTYPE Fluids  TITLE EXT WATER DWNSTRM
MEMBER LOAD
3 4 16 TO 23 UNI GY -0.87
26 27 UNI GX 0.87
30 31 UNI GX -0.87
LOAD 5 LOADTYPE Fluids  TITLE EXT WATER UPSTRM
MEMBER LOAD
1 2 8 TO 15 UNI GY 1.12
5 24 25 UNI GX 1.12
7 28 29 UNI GX -1.12
LOAD 6 LOADTYPE Fluids  TITLE INT WATER 2 CELLS
MEMBER LOAD
3 4 16 TO 23 UNI GY 1.12
1 2 8 TO 15 UNI GY -1.12
6 7 28 TO 31 UNI GX 1.12
5 6 24 TO 27 UNI GX -1.12
LOAD 7 LOADTYPE Fluids  TITLE INT WATER LT CELL
MEMBER LOAD
3 16 TO 19 UNI GY 1.12
1 8 TO 11 UNI GY -1.12
6 UNI GX 1.12
5 24 TO 27 UNI GX -1.12
LOAD COMB 8 INT WATER 2 CELLS
6 2.21
LOAD COMB 9 INT WATER LT CELL
7 2.21
LOAD COMB 10 ALL SOIL & EXT WATER - EMPTY CELLS
1 2.08 2 2.08 3 2.08 4 1.56 5 1.56
LOAD COMB 11 ALL SOIL & EXT WATER - BOTH CELLS
1 2.08 2 2.08 3 2.08 4 1.56 5 1.56 6 1.56
LOAD COMB 12 ALL SOIL & EXT WATER - LT CELL
1 2.08 2 2.08 3 2.08 4 1.56 5 1.56 7 1.56
PERFORM ANALYSIS
FINISH
```

Gatewell Long Side Ext Wall

Slab Details

GEOMETRY

Slab depth =	18.0 in
Design width =	12.0 in
Top clr =	3.0 in
Bot clr =	3.0 in

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#6		#6	
Bar spa =	10.0	in	10.0	in
Bar dia =	0.75	in	0.75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\,prvd}$ =	0.53	in2	0.53	in2
No bars =	1.20		1.20	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 14.90 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	32.80	32.80	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	14.63 in			
A_s, prv'd =	0.53 in2			
ρ =	0.00301			
k =	0.2021			
j =	0.9326			
Comp. Stress Concrete, fc =	50 psi	1400 psi Allowable	OK	Add'l Prv'd = 28.22
Tensile Stress Reinf., fs =	1666 psi	24000 psi Allowable	OK	Add'l Prv'd = 14.4

Top Reinf Mat: Allowable stress = 100%

d = 14.63 in

A_s, prv'd = 0.53 in2
ρ = 0.00301
k = 0.2021
j = 0.9326

Comp. Stress Concrete, fc =	50 psi	1400 psi Allowable	OK	Add'l Prv'd = 28.22
Tensile Stress Reinf., fs =	1666 psi	24000 psi Allowable	OK	Add'l Prv'd = 14.4

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	32.8 ft-lb	I_g =	5832 in4
$4/3M_u$ =	43.7 ft-lb	y_t =	9.0 in
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	32.8 ft-lb		

a = 0.887 in
ϕM_n = 33.7 ft-lb OK Add'l Prv'd = 1.03

Top Reinf Mat:

Demand Moments:

M_u =	32.8 ft-lb
$4/3M_u$ =	43.7 ft-lb
$1.2M_{cr}$ =	28.8 ft-lb
M_{design} =	32.8 ft-lb

a = 0.887 in
ϕM_n = 33.7 ft-lb OK Add'l Prv'd = 1.03

SHEAR DESIGN

Working Stress

Allowable stress = 100%
Shear stress, v = 6 psi — v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c = 56 psi — $v_c = 0.95*f_c^{1/2}$, AASHTO 8.15.5.2.1
OK Add'l Prv'd = 9.86

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 14.90 kips
ϕV_c = 17.65 kips OK Add'l Prv'd = 1.18

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Gatewell Short Side Ext Wall

Slab Details

GEOMETRY

Slab depth =	18.0 in
Design width =	12.0 in
Top clr =	3.0 in
Bot clr =	3.0 in

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#6		#6	
Bar spa =	12.0	in	10.0	in
Bar dia =	0.75	in	0.75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\ prv'd}$ =	0.44	in2	0.53	in2
No. bars =	1.00		1.20	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 13.10 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	20.60	28.10	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat:

Allowable stress = 100%

d =	14.63 in			
A_s, prv'd =	0.44 in2			
ρ =	0.00251			
k =	0.1863			
j =	0.9379			
Comp. Stress Concrete, fc =	54 psi	1400 psi Allowable	OK	Add'l Prv'd = 26.16
Tensile Stress Reinf., fs =	1988 psi	24000 psi Allowable	OK	Add'l Prv'd = 12.07

Top Reinf Mat:

Allowable stress = 100%

d = 14.63 in

A_s, prv'd = 0.53 in2
ρ = 0.00301
k = 0.2021
j = 0.9326

Comp. Stress Concrete, fc =	50 psi	1400 psi Allowable	OK	Add'l Prv'd = 28.22
Tensile Stress Reinf., fs =	1666 psi	24000 psi Allowable	OK	Add'l Prv'd = 14.4

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	20.6 ft-lb	I_g =	5832 in4
$4/3M_u$ =	27.5 ft-lb	y_t =	9.0 in
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	27.5 ft-lb		

a = 0.739 in
ϕM_n = 28.2 ft-lb OK Add'l Prv'd = 1.03

Top Reinf Mat:

Demand Moments:

M_u =	28.1 ft-lb
$4/3M_u$ =	37.5 ft-lb
$1.2M_{cr}$ =	28.8 ft-lb
M_{design} =	28.8 ft-lb

a = 0.887 in
ϕM_n = 33.7 ft-lb OK Add'l Prv'd = 1.17

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	6 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 9.86

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 13.10 kips
ϕV_c = 17.65 kips OK Add'l Prv'd = 1.35

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Gatewell Int Wall

Slab Details

GEOMETRY

Slab depth =	18.0 in
Design width =	12.0 in
Top clr =	3.0 in
Bot clr =	3.0 in

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#6		#6	
Bar spa =	12.0	in	12.0	in
Bar dia =	0.75	in	0.75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\ prv'd}$ =	0.44	in2	0.44	in2
No. bars =	1.00		1.00	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 9.40 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	20.60	20.60	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	14.63 in			
A_s, prv'd =	0.44 in2			
ρ =	0.00251			
k =	0.1863			
j =	0.9379			
Comp Stress Concrete, fc =	54 psi	1400 psi Allowable	OK	Add'l Prv'd = 26.16
Tensile Stress Reinf., fs =	1988 psi	24000 psi Allowable	OK	Add'l Prv'd = 12.07

Top Reinf Mat: Allowable stress = 100%

d = 14.63 in

A_s, prv'd = 0.44 in2
p = 0.00251
k = 0.1863
j = 0.9379

Comp. Stress Concrete, fc =	54 psi	1400 psi Allowable	OK	Add'l Prv'd = 26.16
Tensile Stress Reinf., fs =	1988 psi	24000 psi Allowable	OK	Add'l Prv'd = 12 07

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	20 6 ft-lb	I_g =	5832 in4
$4/3M_u$ =	27.5 ft-lb	y_t =	9 0 in
$1.2M_{cr}$ =	28 8 ft-lb		
M_{design} =	27 5 ft-lb		
a =	0.739 in		
ϕM_n =	28.2 ft-lb	OK	Add'l Prv'd = 1 03

Top Reinf Mat:

Demand Moments:

M_u =	20.6 ft-lb		
$4/3M_u$ =	27.5 ft-lb		
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	27.5 ft-lb		
a =	0.739 in		
ϕM_n =	28.2 ft-lb	OK	Add'l Prv'd = 1.03

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	6 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 9.86

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u =	9.40 kips		
ϕV_c =	17 65 kips	OK	Add'l Prv'd = 1.88

Working Stress Design Formulas

$k = [2np+(np)^2]^{1/2} - np$
$j = 1-k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$

$M_{cr} = f_r I_g / y_t$

Computed By: D. Beck
Checked By: J. Lange

Stanley Consultants Inc.

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Gatewell

Concrete (cy)

126.7 ft3/ft (13'-4x24'-6 less 2x(10'x10'))
4.69 cy/ft

Steel Reinforcing (lbs)

Size	Weight (lb/ft)	No. Req'd	Location	Epoxy	Type	Length	Total ft	No. Splices	Splice Length	A ft	B ft	C ft	D ft	E ft	F ft	G ft	H ft	WT lb
Gatewell																		
			Long Side Exterior Walls															
			No. Ext Walls = 2															
#6	1.502	1.2	Ext Face			24.00	29	0	0.00	24.00								43
#6	1.502	1.2	Int Face			24.00	29	0	0.00	24.00								43
#5	1.043	25	Longit ext face			1.00	25	0	0.00	1.00								26
#5	1.043	25	Longit int face			1.00	25	0	0.00	1.00								26
															Total for 2nd wall =			139
			Short Side Exterior Walls															
			No. Ext Walls = 2															
#6	1.502	1.2	Ext Face w/hooks			18.83	23	0	0.00	12.83	3.00	3.00						34
#6	1.502	1	Int Face			12.83	13	0	0.00	12.83								19
#5	1.043	11	Longit ext face			1.00	11	0	0.00	1.00								11
#5	1.043	11	Longit int face			1.00	11	0	0.00	1.00								11
															Total for 2nd wall =			76
			Interior Wall															
#6	1.502	1	Vert Lt face w/hooks			18.83	19	0	0.00	12.83	3.00	3.00						28
#6	1.502	1	Vert Rt face w/hooks			18.83	19	0	0.00	12.83	3.00	3.00						28
#5	1.043	11	Longit LT face			1.00	11	0	0.00	1.00								11
#5	1.043	11	Longit Rt face			1.00	11	0	0.00	1.00								11

Total all = 510 lb/ft

Computed By: D Beck
Checked By: J Lange

Stanley Consultants Inc.

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Stilling Basin Inlet Structure
Layout and Loading

GEOMETRY

Elevations - values taken from Dam x-sect sketch prv'd by S. Gaitan -

	S1	S8	
Top of walls	24.00	27.0	ft
Invert Elev	5.06	8.98	ft (varies - min shown)
Max water El	16.5	20.5	

Inlet Str:	S1 No.	S1 Thick (ft)	S8 No.	S8 Thick (ft)
Ext Walls	2	1.50	2	1.50
Int Wall	1	1.50	1	1.50
Floor Slab	1	2.00	1	2.00
		Width (ft)		Width (ft)
Int Bay	2	10.00	2	10.00

	S1	S8
Out-out width =	24.50 ft	24.50 ft
Max height, from invert =	18.94 ft	18.02 ft
Total conc x-area =	134.2 ft2	130.1 ft2
Water suface Elev w/dewatering	13.15	16.75 ft (max 16.5' at S1, 20.5' at S8 w/o dewatering)
Water height, from invert =	8.1 ft	7.8 ft
Sumberged depth =	10.09 ft	9.77 ft
Submerged area =	247.2 ft2	239.4 ft2

MATERIAL PROPERTIES

Concrete unit weight	0.150 kcf	
Conc f'c	504 kcf	3.50 ksi
Conc Ec =	485592 kcf	3372 ksi
Modulus of Rupture, fr =	64 kcf	0.444 ksi
Water unit weight	0.0624 kcf	

Soil

Soil unit weight, dry	0.120 kcf (assumed)
Ko for at rest pressure	0.50 (assumed)
At-rest EFP =	0.060 kcf
At-rest EFP, saturated =	0.029 kcf
Vert modulus of subgrade reaction	150 kcf (assumed)

Vert soil spring constant:

	Trib Dimension (ft) Longit	Trib Dimension (ft) Transv	Spring K (k/ft)
Exterior springs	1.0	1.9	285.0
Interior springs	1.0	2.3	345.0

LOADING

Loading	Soil Height (ft)	Soil Load (k/ft)	Soil + Submrgd Soil Height (ft)	Soil + Submrgd Soil Load (k/ft)	External Water Height (ft)	External Water Load (k/ft)	Internal Water Height (ft)	Internal Water Load (k/ft)
S1	7.50	0.45	11.50	0.78	11.50	0.72	11.50	0.72
Vert along wall	(0-7.5)		(7.5-19)					

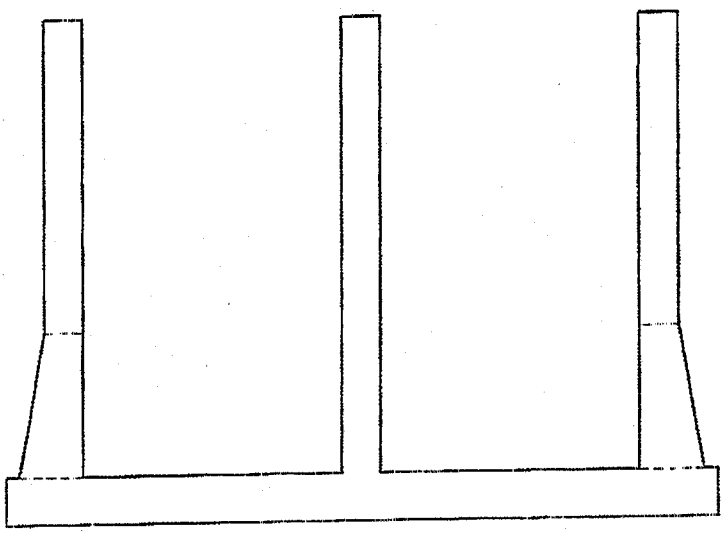
ACI 318-02 Load Factors & Mods

Eqn No.	9-1		9-2	
	1.4	D	1.2	D (dead)
	1.4	F	1.2	F (fluid)
			1.6	H (soil)

By Arm Corp Engr Strength Design for Reinf-Conc Strs - EM 1110-2-2104 3-3.c :
Use 1.7 for F instead of 1.4 in 9-1
F : 1.7
Use Hf (hydralic factor) for all load combos
Hf : 1.3

Then for Staad Combo Load Cases, modified fators are:

Mod 9-1		Mod 9-2	
1.82	D	1.56	D (dead)
2.21	F	1.56	F (fluid)
		2.08	H (soil)



Open section, wall height >12'-0 and <= 19'-0

Job Information

	Engineer	Checked	Approved
Name:	D. Beck	J. Lange	
Date:	May-06	May-06	

Comments

Stilling Well Basin
S1 & S8 worse case

Structure Type	PLANE FRAME

Number of Nodes	22	Highest Node	22
Number of Elements	21	Highest Beam	21

Number of Basic Load Cases	5
Number of Combination Load Cases	7

Included in this printout are data for:

Beams	19 14 16 13

Included in this printout are results for load cases:

Type	L/C	Name
Combination	8	9-1 LT CELL INT WATER
Combination	9	9-1 BOTH CELL INT WATER
Combination	10	9-2 DL+SOIL+EXT WATER
Combination	11	9-2 ALL LOADS LT CELL
Combination	12	9-2 ALL LOADS BOTH CELLS

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	Rect 18.00x12.00	216.000	2.59E 3	5.83E 3	6.09E 3	CONCRETE
2	Rect 30.00x12.00	360.000	4.32E 3	27E 3	12.9E 3	CONCRETE

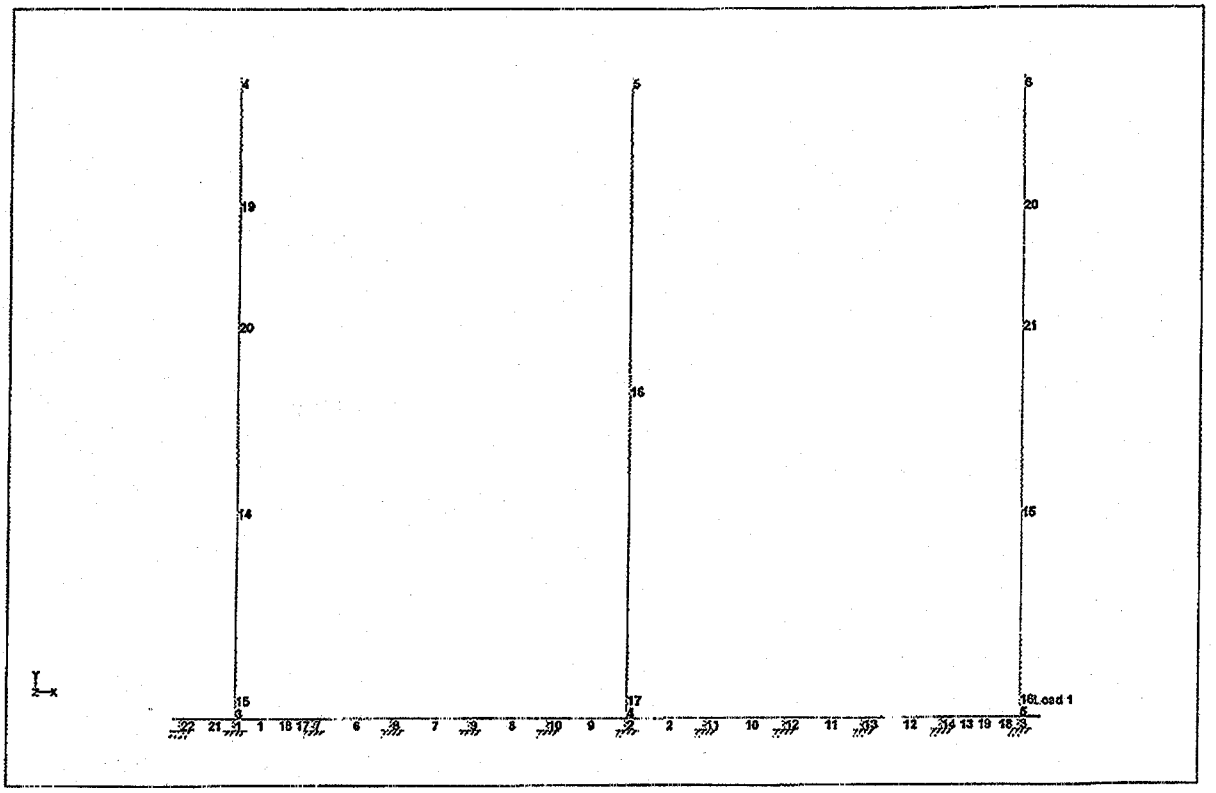
Basic Load Cases

Number	Name
1	CONC SLF WGHT
2	DRY+SUBMRGD SOIL
3	EXT WATER
4	WATER LT CELL
5	WATER BOTH CELLS

Combination Load Cases

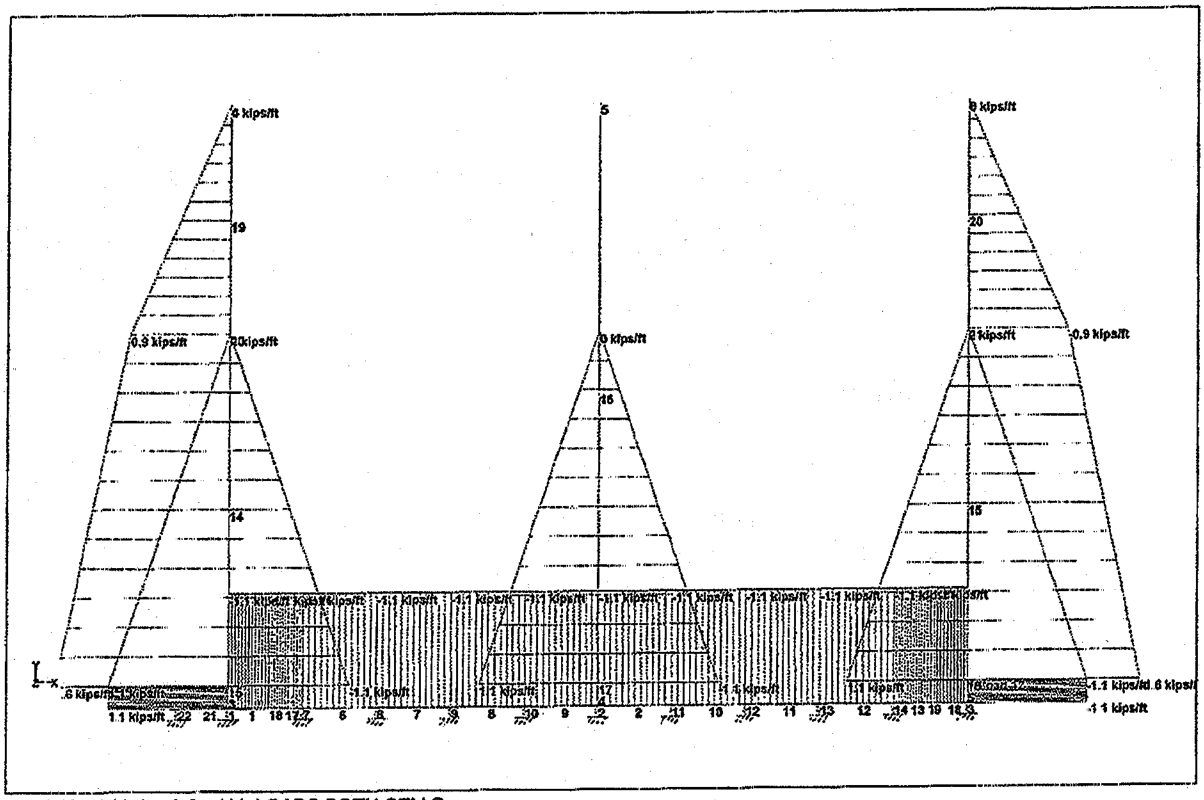
Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
6	SERV BRG PRESSURE1	1	CONC SLF WGHT	1.00
		2	DRY+SUBMRGD SOIL	1.00
		3	EXT WATER	1.00
7	SERV BRG PRESSURE 2	1	CONC SLF WGHT	1.00
		2	DRY+SUBMRGD SOIL	1.00
		3	EXT WATER	1.00
		5	WATER BOTH CELLS	1.00
8	9-1 LT CELL INT WATER	1	CONC SLF WGHT	1.82
		4	WATER LT CELL	2.21
9	9-1 BOTH CELL INT WATER	1	CONC SLF WGHT	1.82
		5	WATER BOTH CELLS	2.21
10	9-2 DL+SOIL+EXT WATER	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		2	DRY+SUBMRGD SOIL	2.08
11	9-2 ALL LOADS LT CELL	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		4	WATER LT CELL	1.56
		2	DRY+SUBMRGD SOIL	2.08
12	9-2 ALL LOADS BOTH CELLS	1	CONC SLF WGHT	1.56
		3	EXT WATER	1.56
		5	WATER BOTH CELLS	1.56
		2	DRY+SUBMRGD SOIL	2.08

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 52	Rev
	Part Stilling Basin		
Job Title C43 West Storage Reservoir	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Stilling_Basin_S1&8.std	Date/Time 12-May-2006 13:16	



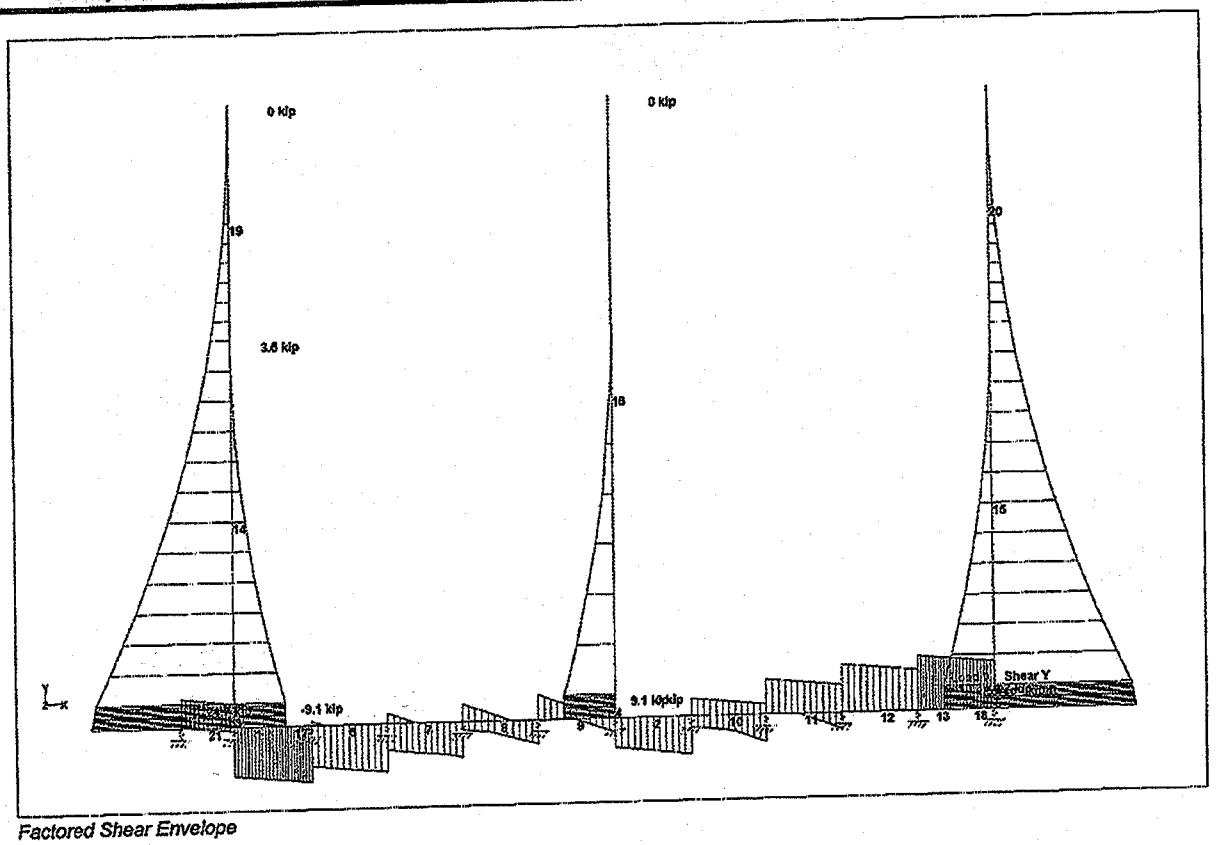
Geometry - Jnt & Beam Nos.

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No #53	Rev
	Part Stilling Basin		
Job Title C43 West Storage Reservoir	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Stilling_Basin_S1&8.std	Date/Time 12-May-2006 13:16	

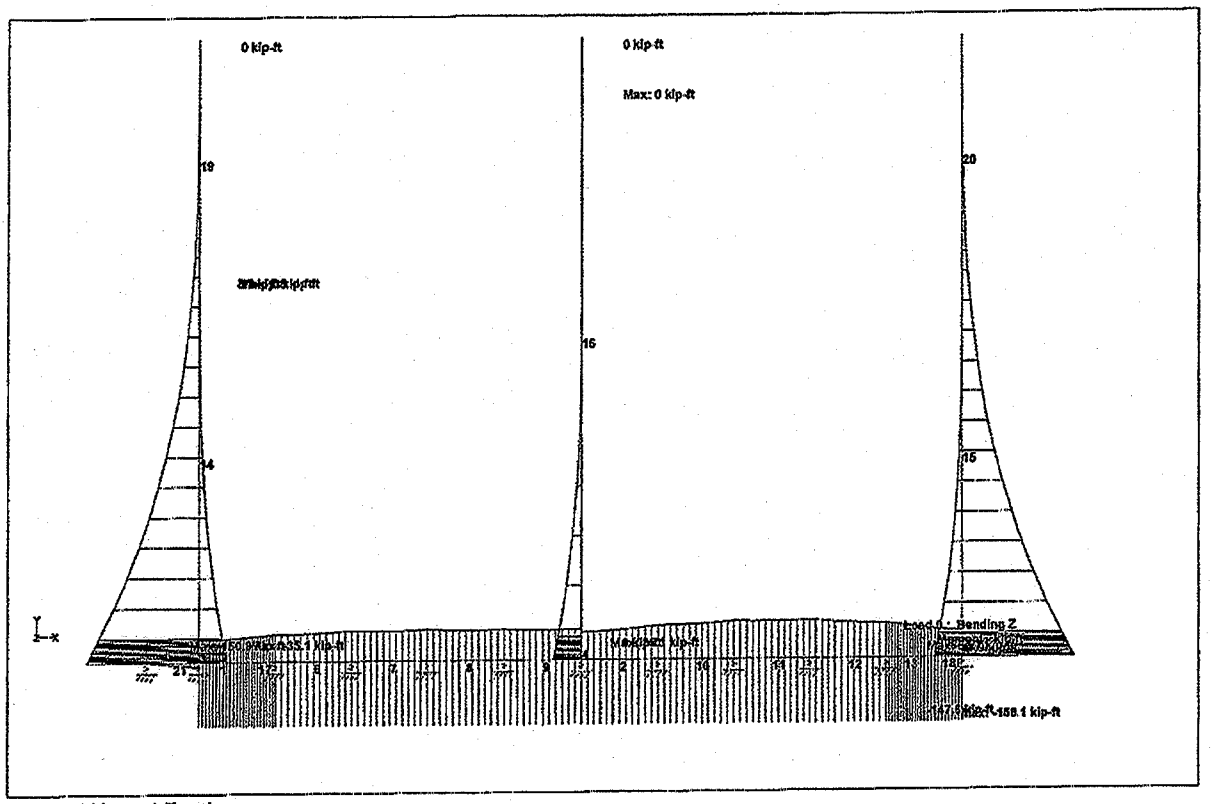


Load 12 - ACI Eqn 9-2 - ALL LOADS BOTH CELLS

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 5/54	Rev
	Part Stilling Basin		
Job Title C43 West Storage Reservoir	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Stilling_Basin_S1&8.std	Date/Time 12-May-2006 13:16	



Factored Shear Envelope



Factored Moment Envelope

Beam End Forces Envelope

Sign convention is as the action of the joint on the beam.

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip ft)	My (kip ft)	Mz (kip ft)
13	14	+ve	0.0	10.3	0.0	0.0	0.0	55.7
			-	10:9-2 DL	-	-	-	9:9-1 BOT
			0.0	0.0	0.0	0.0	0.0	-147.6
			-	-	-	-	-	11:9-2 AL
	19	-ve	0.0	9.7	0.0	0.0	0.0	47.9
			-	10:9-2 DL	-	-	-	9:9-1 BOT
			0.0	0.0	0.0	0.0	0.0	-158.1
			-	-	-	-	-	11:9-2 AL
14	15	+ve	10.9	24.7	0.0	0.0	0.0	150.9
			8:9-1 LT (	10:9-2 DL	-	-	-	10:9-2 DL
			0.0	-9.1	0.0	0.0	0.0	-35.1
			-	8:9-1 LT (	-	-	-	8:9-1 LT (
	20	-ve	3.1	3.5	0.0	0.0	0.0	8.8
			8:9-1 LT (	10:9-2 DL	-	-	-	10:9-2 DL
			0.0	-0.0	0.0	0.0	0.0	-0.0
			-	8:9-1 LT (	-	-	-	8:9-1 LT (

Beam End Forces Envelope Cont...

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip'ft)	My (kip'ft)	Mz (kip'ft)
14	15	+ve	10.9	24.7	0.0	0.0	0.0	150.9
			8:9-1 LT C	10:9-2 DL	-	-	-	10:9-2 DL
			0.0	-9.1	0.0	0.0	0.0	-35.1
			-	8:9-1 LT C	-	-	-	8:9-1 LT C
	20	-ve	3.1	3.5	0.0	0.0	0.0	8.8
			8:9-1 LT C	10:9-2 DL	-	-	-	10:9-2 DL
			0.0	-0.0	0.0	0.0	0.0	-0.0
				8:9-1 LT C	-	-	-	8:9-1 LT C
16	17	+ve	7.8	9.1	0.0	0.0	0.0	35.1
			8:9-1 LT C	8:9-1 LT C	-	-	-	8:9-1 LT C
			0.0	-0.0	0.0	0.0	0.0	-0.0
			-	10:9-2 DL	-	-	-	10:9-2 DL
	5	-ve	0.0	0.0	0.0	0.0	0.0	0.0
			10:9-2 DL	-	-	-	-	9:9-1 BOT
			0.0	0.0	0.0	0.0	0.0	0.0
			-	9:9-1 BOT	-	-	-	-
19	20	+ve	3.1	3.5	0.0	0.0	0.0	8.8
			8:9-1 LT C	10:9-2 DL	-	-	-	10:9-2 DL
			0.0	-0.0	0.0	0.0	0.0	-0.0
			-	8:9-1 LT C	-	-	-	8:9-1 LT C
	4	-ve	0.0	0.0	0.0	0.0	0.0	0.0
			-	-	-	-	-	11:9-2 AL
			-0.0	-0.0	0.0	0.0	0.0	0.0
			10:9-2 DL	11:9-2 AL	-	-	-	-

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip'ft)	My (kip'ft)	Mz (kip'ft)
Max Fx	14	15	8:9-1 LT CELL	10.9	-9.1	0.0	0.0	0.0	-35.1
Min Fx	19	4	10:9-2 DL+SOI	-0.0	-0.0	0.0	0.0	0.0	0.0
Max Fy	14	15	10:9-2 DL+SOI	9.4	24.7	0.0	0.0	0.0	150.9
Min Fy	14	15	8:9-1 LT CELL	10.9	-9.1	0.0	0.0	0.0	-35.1
Max Fz	13	14	8:9-1 LT CELL	0.0	8.6	0.0	0.0	0.0	17.9
Min Fz	13	14	8:9-1 LT CELL	0.0	8.6	0.0	0.0	0.0	17.9
Max Mx	13	14	8:9-1 LT CELL	0.0	8.6	0.0	0.0	0.0	17.9
Min Mx	13	14	8:9-1 LT CELL	0.0	8.6	0.0	0.0	0.0	17.9
Max My	13	14	8:9-1 LT CELL	0.0	8.6	0.0	0.0	0.0	17.9
Min My	13	14	8:9-1 LT CELL	0.0	8.6	0.0	0.0	0.0	17.9
Max Mz	14	15	10:9-2 DL+SOI	9.4	24.7	0.0	0.0	0.0	150.9
Min Mz	13	19	11:9-2 ALL LO	0.0	9.7	0.0	0.0	0.0	-158.1

STAAD STILLING BASIN
MODEL INPUT FILE

57


```
STAAD PLANE
START JOB INFORMATION
JOB NAME C43 West Storage Reservoir
JOB CLIENT US Army Core of Engr
JOB NO 17300
JOB PART Stilling Basin*
JOB COMMENT Stilling Well Basin
JOB COMMENT S1 & S8 worse case
ENGINEER NAME D. Beck
CHECKER NAME J. Lange
ENGINEER DATE May-06
CHECKER DATE May-06
END JOB INFORMATION
INPUT WIDTH 79
*
*
*
*
*
*
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 11.5 0 0; 3 23 0 0; 4 0 19.75 0; 5 11.5 19.75 0; 6 23 19.75 0;
7 2.3 0 0; 8 4.6 0 0; 9 6.9 0 0; 10 9.2 0 0; 11 13.8 0 0; 12 16.1 0 0;
13 18.4 0 0; 14 20.7 0 0; 15 0 0.75 0; 16 23 0.75 0; 17 11.5 0.75 0;
18 1.25 0 0; 19 21.75 0 0; 20 0 12.25 0; 21 23 12.25 0; 22 -1.58 0 0;
MEMBER INCIDENCES
1 1 18; 2 2 11; 3 1 15; 4 2 17; 5 3 16; 6 7 8; 7 8 9; 8 9 10; 9 10 2; 10 11 12;
11 12 13; 12 13 14; 13 14 19; 14 15 20; 15 16 21; 16 17 5; 17 18 7; 18 19 3;
19 20 4; 20 21 6; 21 22 1;
*
*
*
DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 485600
POISSON 0.17
DENSITY 0.15
ALPHA 5.5e-006
DAMP 0.05
END DEFINE MATERIAL
*
*
*
MEMBER PROPERTY AMERICAN
4 16 19 20 PRIS YD 1.5 ZD 1
MEMBER PROPERTY AMERICAN
1 TO 3 5 TO 15 17 18 21 PRIS YD 2.5 ZD 1
CONSTANTS
MATERIAL CONCRETE MEMB 1 TO 21
*
*
*
*
SUPPORTS
1 3 FIXED BUT MZ KFY 285
2 7 TO 14 FIXED BUT MZ KFY 345
22 FIXED BUT MZ KFY 235
*
*
*
```

```
*
LOAD 1 LOADTYPE Dead  TITLE CONC SLF WGHT
SELFWEIGHT Y -1
LOAD 2 LOADTYPE Soil  TITLE DRY+SUBMRGD SOIL
MEMBER LOAD
19 TRAP GX 0.45 0 0 7.5
14 TRAP GX 0.78 0.45 0 11.5
20 TRAP GX -0.45 0 0 7.5
15 TRAP GX -0.78 -0.45 0 11.5
LOAD 3 LOADTYPE Fluids  TITLE EXT WATER
MEMBER LOAD
14 TRAP GX 0.72 0 0 11.5
3 UNI GX 0.72
15 TRAP GX -0.72 0 0 11.5
5 UNI GX -0.72
LOAD 4 LOADTYPE Fluids  TITLE WATER LT CELL
MEMBER LOAD
14 TRAP GX -0.72 0 0 11.5
16 TRAP GX 0.72 0 0 11.5
1 6 TO 9 17 UNI GY -0.72
LOAD 5 LOADTYPE Fluids  TITLE WATER BOTH CELLS
MEMBER LOAD
14 TRAP GX -0.72 0 0 11.5
16 TRAP GX -0.72 0 0 11.5
16 TRAP GX 0.72 0 0 11.5
15 TRAP GX 0.72 0 0 11.5
1 2 6 TO 13 17 18 UNI GY -0.72
*
*
LOAD COMB 6 SERV BRG PRESSURE1
1 1.0 2 1.0 3 1.0
LOAD COMB 7 SERV BRG PRESSURE 2
1 1.0 2 1.0 3 1.0 5 1.0
*
*
LOAD COMB 8 9-1 LT CELL INT WATER
1 1.82 4 2.21
LOAD COMB 9 9-1 BOTH CELL INT WATER
1 1.82 5 2.21
LOAD COMB 10 9-2 DL+SOIL+EXT WATER
1 1.56 3 1.56 2 2.08
LOAD COMB 11 9-2 ALL LOADS LT CELL
1 1.56 3 1.56 4 1.56 2 2.08
LOAD COMB 12 9-2 ALL LOADS BOTH CELLS
1 1.56 3 1.56 5 1.56 2 2.08
PERFORM ANALYSIS
FINISH
```

Stilling Basin Inlet Floor Slab Design

Slab Details Design at 3'-6 out from BF of Ext wall, where hooks end

GEOMETRY

Slab depth =	24.0 in
Design width =	12.0 in
Top clr =	6.0 in
Bot clr =	4.0 in

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#8		#6	
Bar spa =	6.0	in	6.0	in
Bar dia =	1	in	0.75	in
Bar area =	0.79	in2	0.44	in2
$A_{s\ prv'd}$ =	1.58	in2	0.88	in2
No. bars =	2.00		2.00	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 11.00 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot Mat	Top Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	129.00	61.00	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	19.50 in			
$A_{S,\ prv'd}$ =	1.58 in2			
ρ =	0.00675			
k =	0.2863			
j =	0.9046			
Comp. Stress Concrete, fc =	20 psi	1400 psi Allowable	OK	Add'l Prv'd = 68.94
Tensile Stress Reinf., fs =	431 psi	24000 psi Allowable	OK	Add'l Prv'd = 55.74

Top Reinf Mat: Allowable stress = 100%

d = 17.63 in

A_s, prv'd = 0.88 in2
ρ = 0.00416
k = 0 2330
j = 0.9223

Comp. Stress Concrete, fc =	30 psi	1400 psi Allowable	OK	Add'l Prv'd = 46.73
Tensile Stress Reinf., fs =	839 psi	24000 psi Allowable	OK	Add'l Prv'd = 28 61

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	129.0 ft-lb	I_g =	13824 in4
$4/3M_u$ =	172.0 ft-lb	y_t =	12 0 in
$1.2M_{cr}$ =	51.1 ft-lb		
M_{design} =	129.0 ft-lb		

a = 2.655 in
ϕM_n = 129.2 ft-lb OK Add'l Prv'd = 1

Top Reinf Mat:

Demand Moments:

M_u =	61.0 ft-lb
$4/3M_u$ =	81.3 ft-lb
$1.2M_{cr}$ =	51.1 ft-lb
M_{design} =	61.0 ft-lb

a = 1.479 in
ϕM_n = 66.9 ft-lb OK Add'l Prv'd = 1 1

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	5 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 11.89

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 11.00 kips
ϕV_c = 21.27 kips OK Add'l Prv'd = 1.93

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Stilling Basin Inlet Floor Slab Design

Slab Details Design at Floor & Ext Wall Corners (max M)

GEOMETRY

Slab depth =	24.0 in
Design width =	12.0 in
Top clr =	6.0 in
Bot clr =	4.0 in

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot mat:		Top mat:		
Bar size =	#8		#6		
Bar spa =	3.0	in *	6.0	in	*(SEE Add'l steel note below for Bot mat)
Bar dia =	1	in	0.75	in	
Bar area =	0.79	in2	0.44	in2	
$A_{s\ prv'd}$ =	3.16	in2	0.88	in2	
No. bars =	4.00		2.00	(within design width)	

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 11.00 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot Mat	Top Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	157.00	61.00	ft-kips

Add'l Steel Note:

Reinf at corners prv'd by ext face of ext wall reinf hooks:	1.58 in2
Total As prv'd typ floor bot mat:	1.58 in2
Total for both =	3.16 in2
Eqiv spa for both =	3.0 in

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	19.50 in
A_S, prv'd =	3.16 in2
ρ =	0.01350
k =	0.3780
j =	0.8740

Comp. Stress Concrete, fc =	16 psi	1400 psi Allowable	OK	Add'l Prv'd = 87.94
Tensile Stress Reinf., fs =	223 psi	24000 psi Allowable	OK	Add'l Prv'd = 107.71

Top Reinf Mat: Allowable stress = 100%

d = 17.63 in

A_s, prv'd = 0.88 in2
ρ = 0.00416
k = 0.2330
j = 0.9223

Comp Stress Concrete, fc =	30 psi	1400 psi Allowable	OK	Add'l Prv'd = 46.73
Tensile Stress Reinf., fs =	839 psi	24000 psi Allowable	OK	Add'l Prv'd = 28.61

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	157.0 ft-lb	I_g =	13824 in4
$4/3M_u$ =	209.3 ft-lb	y_t =	12.0 in
$1.2M_{cr}$ =	51.1 ft-lb		
M_{design} =	157.0 ft-lb		
a =	5.311 in		
ϕM_n =	239.5 ft-lb	OK	Add'l Prv'd = 1.53

Top Reinf Mat:

Demand Moments:

M_u =	61.0 ft-lb		
$4/3M_u$ =	81.3 ft-lb		
$1.2M_{cr}$ =	51.1 ft-lb		
M_{design} =	61.0 ft-lb		
a =	1.479 in		
ϕM_n =	66.9 ft-lb	OK	Add'l Prv'd = 1.1

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	5 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 11.89

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u =	11.00 kips		
ϕV_c =	21.27 kips	OK	Add'l Prv'd = 1.93

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Stilling Basin Inlet Ext Wall Design

Slab Details Design at wall base (max M)

GEOMETRY

Slab depth =	30.0 in
Design width =	12.0 in
Top clr =	3.0 in (ext face)
Bot clr =	3.0 in (int face)

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot (int) mat:		Top (ext) mat:	
Bar size =	#6		#8	
Bar spa =	12.0	in	6.0	in
Bar dia =	0.75	in	1	in
Bar area =	0.44	in2	0.79	in2
$A_{s\,prv'd}$ =	0.44	in2	1.58	in2
No. bars =	1.00		2.00	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 24.70 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	36.00	151.00	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	26.63 in			
A_s, prv'd =	0.44 in2			
ρ =	0.00138			
k =	0.1418			
j =	0.9527			
Comp. Stress Concrete, fc =	21 psi	1400 psi Allowable	OK	Add'l Prv'd = 67.04
Tensile Stress Reinf, fs =	1075 psi	24000 psi Allowable	OK	Add'l Prv'd = 22.32

Top Reinf Mat: Allowable stress = 100%

d =	26.50 in
A_s, prv'd =	1.58 in2
ρ =	0.00497
k =	0.2515
j =	0.9162

Comp. Stress Concrete, fc =	12 psi	1400 psi Allowable	OK	Add'l Prv'd = 113.28
Tensile Stress Reinf., fs =	313 psi	24000 psi Allowable	OK	Add'l Prv'd = 76.72

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	36.0 ft-lb	I_g =	27000 in4
$4/3M_u$ =	48.0 ft-lb	y_t =	15.0 in
$1.2M_{cr}$ =	79.9 ft-lb		
M_{design} =	48.0 ft-lb		

a = 0.739 in
ϕM_n = 52.0 ft-lb OK Add'l Prv'd = 1.08

Top Reinf Mat:

Demand Moments:

M_u =	151.0 ft-lb
$4/3M_u$ =	201.3 ft-lb
$1.2M_{cr}$ =	79.9 ft-lb
M_{design} =	151.0 ft-lb

a = 2.655 in
ϕM_n = 179.0 ft-lb OK Add'l Prv'd = 1.19

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	3 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 17.87

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 24.70 kips
ϕV_c = 31.98 kips OK Add'l Prv'd = 1.29

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Stilling Basin Inlet Ext Wall Design

Slab Details Design at 6'-0 up from floor slab

GEOMETRY

Slab depth =	18 0 in
Design width =	12.0 in
Top clr =	3 0 in (ext face)
Bot clr =	3 0 in (int face)

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot (int) mat:		Top (ext) mat:	
Bar size =	#6		#6	
Bar spa =	12.0	in	6.0	in
Bar dia =	0.75	in	0 75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\,prv'd}$ =	0.44	in2	0.88	in2
No. bars =	1.00		2.00	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 11.00 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	4.00	46.60	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	14.63 in			
A_S, prv'd =	0 44 in2			
ρ =	0 00251			
k =	0.1863			
j =	0.9379			
Comp Stress Concrete, fc =	54 psi	1400 psi Allowable	OK	Add'l Prv'd = 26.16
Tensile Stress Reinf., fs =	1988 psi	24000 psi Allowable	OK	Add'l Prv'd = 12 07

Top Reinf Mat: Allowable stress = 100%

d = 14.63 in
A_s, prv'd = 0.88 in2
ρ = 0.00501
k = 0.2525
j = 0.9158

Comp. Stress Concrete, fc =	40 psi	1400 psi Allowable	OK	Add'l Prv'd = 34.63
Tensile Stress Reinf., fs =	1018 psi	24000 psi Allowable	OK	Add'l Prv'd = 23.57

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	4.0 ft-lb	I_g =	5832 in4
$4/3M_u$ =	5.3 ft-lb	y_t =	9.0 in
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	5.3 ft-lb		

a = 0.739 in
ϕM_n = 28.2 ft-lb OK Add'l Prv'd = 5.29

Top Reinf Mat:

Demand Moments:

M_u =	46.6 ft-lb
$4/3M_u$ =	62.1 ft-lb
$1.2M_{cr}$ =	28.8 ft-lb
M_{design} =	46.6 ft-lb

a = 1.479 in
ϕM_n = 55.0 ft-lb OK Add'l Prv'd = 1.18

SHEAR DESIGN

Working Stress

Allowable stress = 100%
Shear stress, v = 6 psi — v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c = 56 psi — $v_c = 0.95*f_c^{1/2}$, AASHTO 8.15.5.2.1
OK Add'l Prv'd = 9.86

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 11.00 kips
ϕV_c = 17.65 kips OK Add'l Prv'd = 1.6

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Stilling Basin Inlet Int Wall Design

Slab Details Design at 6'-0 up from floor slab

GEOMETRY

Slab depth =	18.0 in
Design width =	12.0 in
Top clr =	3.0 in (ext face)
Bot clr =	3.0 in (int face)

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot (int) mat:		Top (ext) mat:	
Bar size =	#6		#6	
Bar spa =	8.0	in	8.0	in
Bar dia =	0.75	in	0.75	in
Bar area =	0.44	in2	0.44	in2
$A_{s\,prv'd}$ =	0.66	in2	0.66	in2
No. bars =	1.50		1.50	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 9.20 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	35.10	35.10	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	14.63 in
A_S, prv'd =	0.66 in2
ρ =	0.00376
k =	0.2230
j =	0.9257

Comp. Stress Concrete, fc =	45 psi	1400 psi Allowable	OK	Add'l Prv'd = 30.9
Tensile Stress Reinf., fs =	1343 psi	24000 psi Allowable	OK	Add'l Prv'd = 17.87

Top Reinf Mat: Allowable stress = 100%

d =	14.63 in			
A_s, prv'd =	0.66 in2			
ρ =	0 00376			
k =	0 2230			
j =	0 9257			
Comp. Stress Concrete, fc =	45 psi	1400 psi Allowable	OK	Add'l Prv'd = 30.9
Tensile Stress Reinf , fs =	1343 psi	24000 psi Allowable	OK	Add'l Prv'd = 17.87

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0 90

Bot Reinf Mat:

Demand Moments:	M_u =	35 1 ft-lb	I_g =	5832 in4
	$4/3M_u$ =	46 8 ft-lb	y_t =	9.0 in
	$1.2M_{cr}$ =	28.8 ft-lb		
	M_{design} =	35.1 ft-lb		
	a =	1 109 in		
	ϕM_n =	41 8 ft-lb	OK	Add'l Prv'd = 1.19

Top Reinf Mat:

Demand Moments:	M_u =	35.1 ft-lb		
	$4/3M_u$ =	46 8 ft-lb		
	$1.2M_{cr}$ =	28 8 ft-lb		
	M_{design} =	35.1 ft-lb		
	a =	1.109 in		
	ϕM_n =	41.8 ft-lb	OK	Add'l Prv'd = 1.19

SHEAR DESIGN

Working Stress

Allowable stress =	100%		
Shear stress, v =	6 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)	
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8 15.5 2.1	
		OK	Add'l Prv'd = 9 86

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0 85

V_u =	9.20 kips		
ϕV_c =	17.65 kips	OK	Add'l Prv'd = 1.92

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$

$M_{cr} = f_r I_g / y_t$

Stilling Basin Inlet

Concrete (cy)

x-area of walls only for 19' H section: 91.5 ft2
for walls <19'-0 use ratio for x-area

Location	Length (ft)	Area (ft2)	Vol (yd3)	
S1				
At Bridge	24.0	171.5	152	
Constant height sect ahd/bk bridge	12.0	146.5	65	
Varying height section	24.0		111	based on avg
Max H (19-0)		146.5	-	
Min H (10'-0)		103.2	-	
Total all =			329 yd3	
S8				
At Bridge	24.0	171.5	152	
Constant height sect ahd/bk bridge	12.0	146.5	65	
Varying height section	27.0		122	based on avg
Max H (18'-0)		141.7	-	
Min H (10'-0)		103.2	-	
Total all =			340 yd3	

Steel Reinforcing (lbs)

Taken at max H = 19'-0

Size	Weight (lb/ft)	No. Req'd	Location	Epoxy	Type	Length	Total ft	No. Splices	Splice Length	A ft	B ft	C ft	D ft	E ft	F ft	G ft	H ft	WT lb
Stilling Basin Inlet																		
			Floor Slab															
#8	2.67	2	Ext Face			26.83	54	0	0.00	26.83								143
#6	1.502	2	Int Face			26.83	54	0	0.00	26.83								81
#5	1.043	28	Longit ext face			1.00	28	0	0.00	1.00								29
#5	1.043	28	Longit int face			1.00	28	0	0.00	1.00								29
			Exterior Walls															
			No. Ext Walls =	2														
#8	2.67	2	Ext Face w/hooks at bot			16.00	32	0	0.00	10.00	6.00							86
#6	1.502	2	Ext face strght at top			14.75	30	0	0.00	14.75								44
#6	1.502	1	Int Face			20.25	20	0	0.00	20.25								30
#5	1.043	20	Longit ext face			1.00	20	0	0.00	1.00								21
#5	1.043	20	Longit int face			1.00	20	0	0.00	1.00								21
															Total for 2nd wall =			202

Computed By: D. Beck
Checked By: J. Lange

Stanley Consultants Inc.

Project: C-43 West Storage Reservoir
WO No. C-C10401P-WO No. 09
Date: March 06
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			Interior Wall																
#6	1.502	1.5	Vert Lt face w/hooks			20.50	31	0	0.00	18.50	2.00								46
#6	1.502	1.5	Vert Rt face w/hooks			20.50	31	0	0.00	18.50	2.00								46
#5	1.043	20	Longit LT face			1.00	20	0	0.00	1.00									21
#5	1.043	20	Longit Rt face			1.00	20	0	0.00	1.00									21

Total all = 830 lb/ft

Total reinf = 830 lb/ft
X-area = 5.43 yd3/ft
Reinf density = 153 lb/ft (rounded up to nearest 1)

Based on reinf density, rounded up to 500:
Total for S1 = 50,500 lbs
Total for S8 = 52,500 lbs

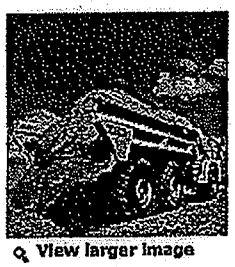
Machines
» Articulated Trucks
» Backhoe Loaders
» Cold Planers
» Compactors
» Feller Bunchers
» Forest Machines
» Forwarders
» Front Shovels
» Harvesters
» Hydraulic Excavators
» Knuckleboom Loaders
» Material Handlers
» Motor Graders
» Multi Terrain Loaders
» Off Highway Tractors
» Off Highway Trucks
» Paving Equipment
» Pipelayers
» Road Reclaimers
» Scrapers
» Skid Steer Loaders
» Skidders
» Soil Stabilizers
» Telehandlers
» Track Loaders
» Track-Type Tractors
» Underground Mining
» Wheel Dozers
» Wheel Excavators
» Wheel Loaders

740 EJECTOR ARTICULATED TRUCKS

Overview & Specifications | Benefits & Features | Standard Equipment | Optional Equipment

Print Version

740 EJECTOR OVERVIEW & SPECIFICATIONS



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740 EJECTOR

The 740 Ejector truck offers proven reliability and durability, high performance, low fuel consumption, operator comfort and low operating costs.

NEXT STEPS
Use the resources below to help you make the right decision.
- Find Your Dealer
- Financing
- Rentals
- Insurance
- Training
- Maintenance & Support
- Build & Quote

RELATED INDUSTRIES
- Governmental

740 EJECTOR ARTICULATED TRUCKS
Download printable, graphical specifications in PDF format.
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740 EJECTOR DETAILED SPECIFICATIONS

Engine	
Engine Model	Cat® C15 ACERT™
Gross Power - SAE J1995	457 hp / 341 kW
Net Power - SAE J1349	436 hp / 325 kW
Net Power - ISO 9249	438 hp / 327 kW
Net Power - EEC 80/1269	438 hp / 327 kW
Flywheel Power	436 hp / 325 kW
Bore	5.4 in / 137 mm
Stroke	6.75 in / 171.5 mm
Displacement	926 in3 / 15.2 L
Weights	
Rated Payload	42 tons / 38 tonnes
Body Capacities	
Heaped SAE 2:1	30.2 yd3 / 23.1 m3
Struck	23.3 yd3 / 17.8 m3
Heaped SAE 1:1	37.3 yd3 / 28.5 m3
Transmission	
Forward 1	5.5 mph / 8.85 km/h
Forward 2	7.52 mph / 12.1 km/h
Forward 3	10.2 mph / 16.41 km/h
Forward 4	13.7 mph / 22.04 km/h
Forward 5	18.58 mph / 29.9 km/h
Forward 6	25.06 mph / 40.32 km/h
Forward 7	33.99 mph / 54.69 km/h
Reverse 1	5.19 mph / 8.35 km/h
Reverse 2	7.21 mph / 11.6 km/h

Operating Weights	
Front Axle - Empty	43651 lb / 19800 kg
Center Axle - Empty	18166 lb / 8240 kg
Rear Axle - Empty	16700 lb / 7570 kg
Total - Empty	78507 lb / 35610 kg
Front Axle - Rated Load	3516 lb / 1640 kg
Center Axle - Rated Load	40080 lb / 18180 kg
Rear Axle - Rated Load	40080 lb / 18180 kg
Total - Rated Load	83756 lb / 38000 kg
Front Axle - Loaded	47267 lb / 21440 kg
Center Axle - Loaded	58246 lb / 26420 kg
Rear Axle - Loaded	56569 lb / 25750 kg
Total - Loaded	162262 lb / 73610 kg
Body Plate Thickness	
Front	.24 in / 6 mm
Side	.24 in / 6 mm
Base	.39 in / 10 mm
Service Refill Capacities	
Fuel Tank	148 gal / 560 L
Cooling System	25.4 gal / 96 L
Hydraulic System	86.6 gal / 328 L
Engine Crankcase	10 gal / 38 L
Transmission	19 gal / 72 L
Final Drives/Differential	21.1 gal / 80 L
Output Transfer Gear Box	5.6 gal / 21 L
Ejector Cycle Times	
Retract Time (in Seconds)	24 Seconds / 24 Seconds
Sound Levels	
Interior Cab	79 dB(A) / 79 dB(A)
Body Hoist	
Raise time	14 Seconds / 14 Seconds
Standards	
Brakes	SAE J/ISO 3450-1998
Cab/FOPS	SAE J231 JAN81 and ISO 3449-1992 Level II
Cab/ROPS	SAE J1040 May 1994 and ISO 3471-1994

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740 Ejector Articulated Truck

Turning Circle

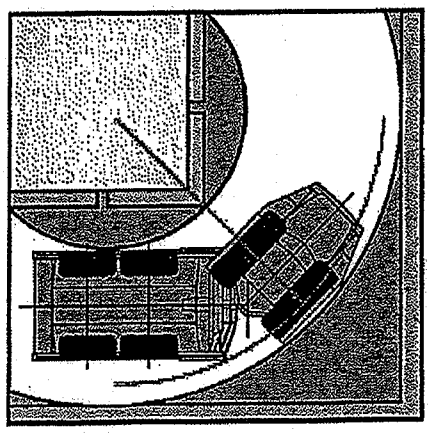
Dimensions are for machines equipped with 29.5R25 tires.

Turning dimensions

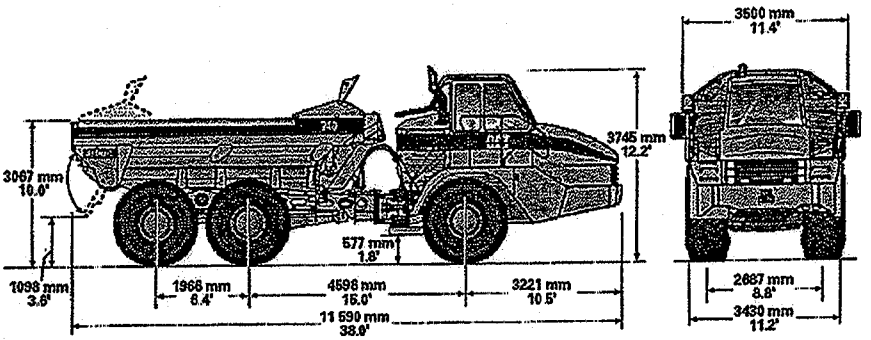
Steer angle — left/right	45°	
SAE turning radius	8640 mm	340.2 in
Clearance radius	9090 mm	357.9 in
Inside radius	4470 mm	176.0 in
Aisle width	5930 mm	233.5 in

Steering

Lock to Lock		5 seconds
Flow Rate	190 L/min	50 gal/min



Dimensions



Optimal Loader/Truck Pass Matching

Hydraulic Excavators	385B	365B II	345B II
Loader Capacity (Tonnes) – 50 min hr	954-1193	750-1100	665-805
Loader Capacity (Tons) – 50 min hr	1049-1314	825-1210	735-885
Passes	4	5	6-7

Wheel Loaders	988G	980G II	972G II	966G II
Loader Capacity (Tonnes) – 50 min hr	565-790	590-650	490-565	400-535
Loader Capacity (Tons) – 50 min hr	625-870	650-717	540-625	440-590
Passes	3-4	4	5	6

An optimum system match gives you a major productivity advantage. The 740 Ejector is an excellent match for the Cat 365B II and 385B Hydraulic Excavators; and 972G II, 980G II and 988G Wheel Loaders. This results in increased production and lower system costs per unit of volume moved.

Flotation Safety Factor: $SF_f = \frac{Ws + Wc + S}{U - Wg}$ (1)

(1) Department of the Army
ETL 1110-2-307
20 August 1987

Where: SF_f = Flotation Safety Factor chosen as 1.30 - Scheduled Maintenance
Ws = Weight of structure including weight of concrete and weight of soil on top of the structure
Wc = Weight of the water contained in the structure (Zero for dewatering purposes)
S = Surcharge loads (Conservatively assumed zero)
U = Uplift forces acting on the base of the structure.
Wg = Weight of water above the structure controlled by gravity
Displaced water weight = U - Wg

Material Unit Weight Assumptions

Concrete	0.1500 kcf
Water	0.0624 kcf
Soil	0.1200 kcf

S12 Cell Balancing Structure Stilling Basins

	Elevations (ft)			Structure dimensions (ft)															
				Exterior Walls		Interior Walls		Slabs		Interior Bays									
Location	Top of Wall	Invert	Water Surface	No.	Thick	No.	Thick	Floor Slab	Top Elevated Slab	No.	Width	Out-to-out width (ft)	Wall Height (ft)	Sub-mergd Depth (ft)	Conc area (ft2)	Water Displ (ft2)	Ws = Conc wght (kips)	U - Wg = Displaced water wght (kips)	Minimum Flotation FOS
S12	21.00	12.00	15.09	2	1.50			2.00		1	33.00	36.00	9.00	5.09	99.0	183.1	14.85	11.42	1.30

Reservoir W.S.E. to be lowered to a minimum elevation of 15.09' to safely dewater cell balancing structure stilling basins.
Provide weeps and drain system to dewater surrounding groundwater to elevation 15.09'.

S12 Cell Balancing Structure Conduit

The S12 cell balancing conduit structure will be safe against flotation as it cannot be dewatered with a fully watered cell.

S12 Cell Balancing Structure Gatewell

By inspection of S1/S8 gatewell computations, the S12 cell balancing gatewell structure will be safe against flotation.

S12 Gatewell Foundation Bearing Pressure Computations		
Base Length	40	ft
Base Width	37.5	ft
Base Area	**1500**	**sq ft**
Conduit Base Concrete	3909	cu ft
Conduit Top Slab	2862	cu ft
Conduit Walls	3905	cu ft
Conduit_to_Gatewell Wall Transition	413	cu ft
Gatewell Walls (⊥ to flow)	2970	cu ft
Gatewell Walls (ll to flow)	1650	cu ft
Air Vent Casings	151	cu ft
Total Concrete	**15860**	**cu ft**
Weight of Concrete @ 150 lbs / cu ft	2378935	lbs
Bearing Pressure	**1586**	**psf**

S12 Cell Balancing Structure - Structural Analysis & Design

Outlet Conduits Analysis (STAAD.Pro)
*See C43-S1/S2 Main Outlet Structural Computations for similar 6'x10' Conduit Analysis

Outlet Conduits Design
*See C43-S1/S2 Main Outlet Structural Computations for similar 6'x10' Conduit Design

Gatewell Analysis (STAAD.Pro)
*See C43-S1/S2 Main Outlet Structural Computations for similar 10'x10' Gatewell Analysis

Gatewell Design
*See C43-S1/S2 Main Outlet Structural Computations for similar 10'x10' Gatewell Design

Stilling Basin Structure Analysis (STAAD.Pro)
*See C43-S1/S2 Main Outlet Structural Computations for similar Stilling Basin Analysis

Stilling Basin Structure Design
*See C43-S1/S2 Main Outlet Structural Computations for similar Stilling Basin Design

Construction Heavy Equipment Information
*See C43-S1/S2 Main Outlet Structural Computations for Heavy Equipment Information

Flotation Safety Factor: $SF_f = \frac{Ws + Wc + S}{U - Wg}$ (1)

(1) Department of the Army
ETL 1110-2-307
20 August 1987

Where: SF_f = Flotation Safety Factor chosen as 1.30 - Scheduled Maintenance
Ws = Weight of structure including weight of concrete and weight of soil on top of the structure
Wc = Weight of the water contained in the structure (Zero for dewatering purposes)
S = Surcharge loads (Conservatively assumed zero)
U = Uplift forces acting on the base of the structure.
Wg = Weight of water above the structure controlled by gravity
Displaced water weight = U - Wg

Material Unit Weight Assumptions

Concrete	0.1500 kcf
Water	0.0624 kcf
Soil	0.1200 kcf

CS1 Crest Spillway Inlet Weir Structure

	Elevations (ft)			Structure dimensions (ft)																
				Exterior Walls		Interior Walls		Slabs		Interior Bays										
Location	Top of Wall	Invert	Water Surface	No.	Thick	No.	Thick	Floor Slab	Elevated Slab	No.	Width	Out-to-out width (ft)	Wall Height (ft)	Sub-mergd Depth (ft)	Additional Concrete Slab (ft2)	Conc area (ft2)	Water Displ (ft2)	Ws = Conc wght (kips)	U - Wg = Displaced water wght (kips)	Flotation FOS
CS1	42.00	38.00	42.00			2	5.00	1.25		1	18.00	28.00	4.00	5.25	25.0	100.0	172.0	15.0	10.7	1.40

The CS1 Crest Spillway Inlet Weir is safe against flotation.

Crest Spillway Conduit

The crest spillway conduit structure will be safe against flotation as it cannot be dewatered.

Crest Spillway Stilling Basins

	Elevations (ft)			Structure dimensions (ft)																
				Exterior Walls		Interior Walls		Slabs		Interior Bays										
Location	Top of Wall	Invert	Discharge Canal Water Surface	No.	Thick	No.	Thick	Floor Slab	Top Elevated Slab	No.	Width	Out-to-out width (ft)	Wall Height (ft)	Sub-mergd Depth (ft)		Conc area (ft2)	Water Displ (ft2)	Ws = Conc wght (kips)	U - Wg = Displaced water wght (kips)	Flotation FOS
CS1	20.00	-7.00	2.10	2	1.50			3.00	1.25	1	10.00	13.00	25.75	12.10		132.5	157.3	19.9	9.8	2.02
CS2	27.00	7.00	19.00	2	1.50			3.00	1.25	1	10.00	13.00	18.75	15.00		111.5	195.0	16.7	12.2	1.37

Scheduled Maintenance can proceed at normal tailwater elevation levels of the discharge canals.

Factor of Safety for Sliding: $FSs = \frac{N \tan\Phi + cL}{T}$ (1)

(1) Department of the Army
ETL 1110-2-6058
30 November 2003

Where: FSs = Factor of Safety for Sliding: Chosen as 1.70 - Well Defined Critical Structure with Usual Load Conditions
N = Force acting normal to the sliding failure plane under the structure
Φ = Angle of internal friction between the foundation material and the structure. For a sand_concrete shear plane, $\Phi_{soil\text{-}str} = (2/3)\Phi_{soil}$
c = Adhesive strength of the foundation material under the structure. (Adhesion = 2/3 Cohesion)
L = Length of the structure in contact with the foundation material
T = Shear force acting parallel to the base of the structure

Material Unit Weight Assumptions

Concrete	0.1500 kcf
Water	0.0624 kcf

Constants

Gravity	32.2 ft/s2

Crest Spillway U-Shaped Conduits on 3:1 Slope

		Structure dimensions (ft)																	
		Exterior Walls			Base Slab				Slope										
Location	Soil Material	No	Height	Thick	Width	Thick	Concrete Area (ft2)	Concrete Wght (kips)	Vertical	Horizon-tal	Angle of Incline (degrees)	N (kips)	T (kips)	Φ_{soil} (degrees)	$\Phi_{soil\text{-}str}$ (degrees)	c (kips/ft2)	L (ft)	FOS
CS1,CS2	Random Fill	2	5.00	1.00	13.00	1.50	29.50	4.43	1.00	3.00	18.43	4.20	1.40	32	21.3	0.0	13.00	1.17

The soil make-up of the dam is shown in the drawings as random fill. Random fill will be sand with an angle of internal friction of 30 degrees.
Reference Geotechnical Design Memo for soil types and soil characteristics.
Weight of water can be neglected as it does not alter sliding stability FOS with c=0.
U-Shaped Conduit does not slide in the random fill soil condition on a 3:1 surface. Stilling basin computations show that the stilling basins have additional sliding resistance to bring the Factor of Safety from 1.17 to 1.70.

Stilling Basins (Sliding due to impact of water on baffle blocks)

		Structure dimensions (ft)																		
		Exterior Walls			Interior Walls			Base Slab		Top Slab								Baffle Blocks		
Location	Soil Material	No	Height	Thick	No	Height	Thick	Width	Thick	Width	Thick	Length of Structure (with bridge)	Concrete Area (ft2)	Total Concrete Wght (kips)	Max Water Velocity (ft/s)	Equivalent Static Head (ft)	Water Depth (ft)	No	Width	Area Subject to Water Pressure (ft2)	Force of Water (kips)
CS1	Clay	2	25.75	1.50				16.50	3.00	13.00	1.25	46.00	6578.0	986.70	59.5	55.0	1.3	0	1.25	0.00	0.00
CS2	Clay	2	18.75	1.50				16.50	3.00	13.00	1.25	24.00	2928.0	439.20	49.5	38.0	1.5	4	1.25	7.50	8.86
CS2	Sand	2	18.75	1.50				16.50	3.00	13.00	1.25	24.00	2928.0	439.20	49.5	38.0	1.5	4	1.25	7.50	8.86

Location	Soil Material	N (kips)	T (kips)	Φ_{soil} (degrees)	$\Phi_{soil\text{-}str}$ (degrees)	c (kips/ft2)	L (ft)	FOS
CS1	Clay	986.70	0.00	0	0	0.6	16.50	*
CS2	Clay	439.20	8.86	0	0	0.6	16.50	26.81
CS2	Sand	439.20	8.86	32	21.3	0.0	16.50	19.36

*Not subject to shear force (T = 0) because of zero baffle blocks.

Reference Geotechnical Design Memo for soil types and characteristics.
Weight of water in stilling basins can be neglected as a conservative approach.
Stilling Basins are safe against sliding for their respective soil conditions.

Layouts and Loading

GEOMETRY

Elevations - values taken from Dam x-sect sketch prv'd by S Gaitan

Crest Spillway

Top of dam	55.0 ft
Bot roof slab	43.0 ft
Top floor slab	37 0 ft

MATERIAL PROPERTIES

Concrete unit weight	0.150 kcf	
Conc f'c	504 kcf	3 50 ksi
Conc Ec =	485592 kcf	3372 ksi
Modulus of Rupture, fr =	64 kcf	0.444 ksi

Conc reinf cover requirements: 3.0 in for formed sections
Prv'd by S. Gaitan 4 0 in poured against soil
6 0 in surfaces at pressurized flow

Water unit weight	0 0624 kcf
Head for pressurized case, Elev	42.0 ft

Soil

Soil unit weight, dry	0 120 kcf (assumed)
Ko for at rest pressure	0 50 (assumed)
At-rest EFP =	0 060 kcf
At-rest EFP, saturated =	0 029 kcf
Vert modulus of subgrade reaction	150 kcf (assumed)

LOADING

Crest Spillway

	Submerged Soil		External Water		Internal Water	
Loading	Height (ft)	Load (k/ft)	Height (ft)	Load (k/ft)	Height (ft)	Load (k/ft)
Vertical						
Top of conduit	10.5	0.60	10 5	0.66	n/a	n/a
Bot of conduit	n/a	n/a	n/a	n/a	6.0	0.37
Horizontal						
Top of conduit	10.5	0.30	10.5	0.66	0.0	0.0
Bot of conduit	19.5	0.56	19 5	1.22	6 0	0.37

Job Information

	Engineer	Checked	Approved
Name:	D. Beck	J. Lange	
Date:	May-06	May-06	

Comments

6'x10' Single Box

Structure Type	PLANE FRAME

Number of Nodes	17	Highest Node	17
Number of Elements	17	Highest Beam	20

Number of Basic Load Cases	7
Number of Combination Load Cases	6

Included in this printout are data for:

Beams	5 8 13 15

Included in this printout are results for load cases:

Type	L/C	Name
Combination	8	SERV BRG PRESSURE
Combination	9	9-1 EXT WATER
Combination	10	9-1 INT WATER
Combination	11	9-2 EXT WATER
Combination	12	9-2 INT ALL WATER
Combination	13	CST LOADING

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	Rect 18.00x12.00	216.000	2.59E 3	5.83E 3	6.09E 3	CONCRETE
2	Rect 12.00x12.00	144.000	1.73E 3	1.73E 3	2.92E 3	CONCRETE

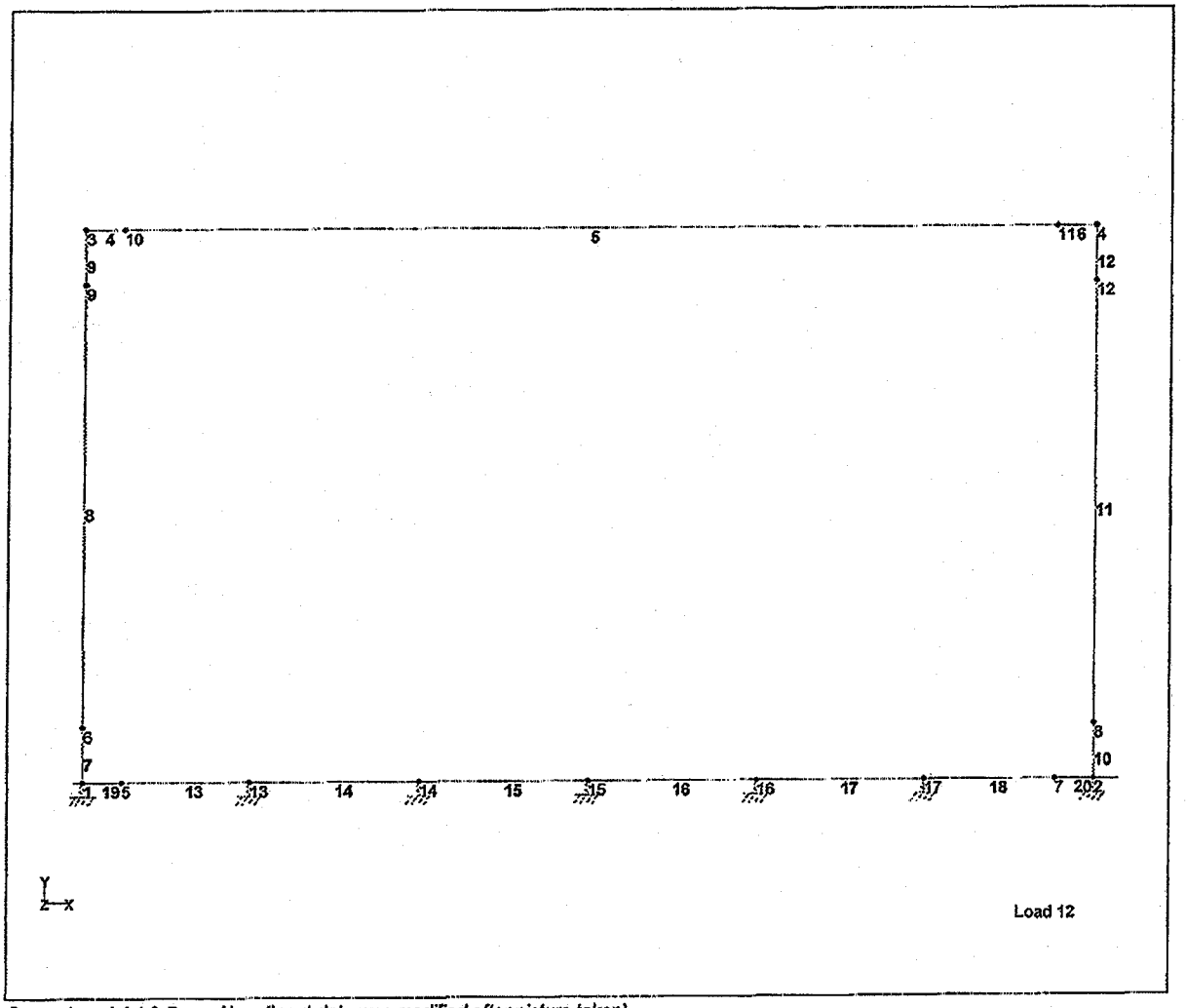
Basic Load Cases

Number	Name
1	CONC SLF WGHT
2	SUB SOIL VERT
3	SUBMRGD SOIL HOIZ
4	EXT WATER
5	INT WATER
6	CST SOIL
7	CST TRUCK

Combination Load Cases

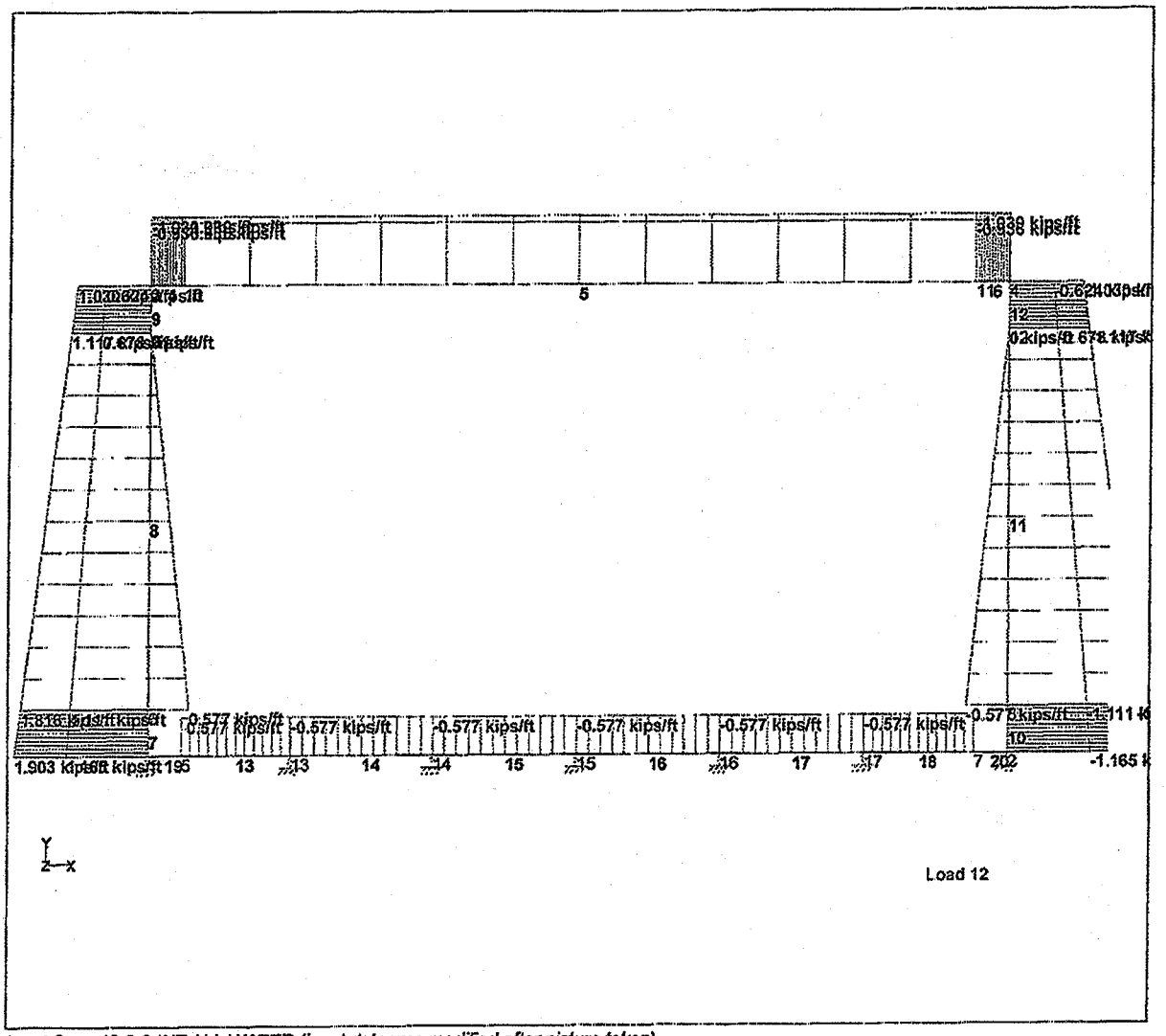
Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
8	SERV BRG PRESSURE	1	CONC SLF WGHT	1.00
		2	SUB SOIL VERT	1.00
		3	SUBMRGD SOIL HOIZ	1.00
		4	EXT WATER	1.00
		5	INT WATER	1.00
9	9-1 EXT WATER	1	CONC SLF WGHT	1.82
		4	EXT WATER	2.21
10	9-1 INT WATER	1	CONC SLF WGHT	1.82
		5	INT WATER	2.21
11	9-2 EXT WATER	1	CONC SLF WGHT	1.56
		2	SUB SOIL VERT	1.56
		3	SUBMRGD SOIL HOIZ	2.08
		4	EXT WATER	1.56
12	9-2 INT ALL WATER	1	CONC SLF WGHT	1.56
		2	SUB SOIL VERT	1.56
		4	EXT WATER	1.56
		5	INT WATER	1.56
		3	SUBMRGD SOIL HOIZ	2.08
13	CST LOADING	1	CONC SLF WGHT	1.20
		6	CST SOIL	1.60
		7	CST TRUCK	1.69

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 36	Rev
	Part Crest Spillway Design		
Job Title C-43 W. STORAGE RESERVOIR	Ref		
	By D Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Crest_Spillway.std	Date/Time 12-May-2006 13:11	



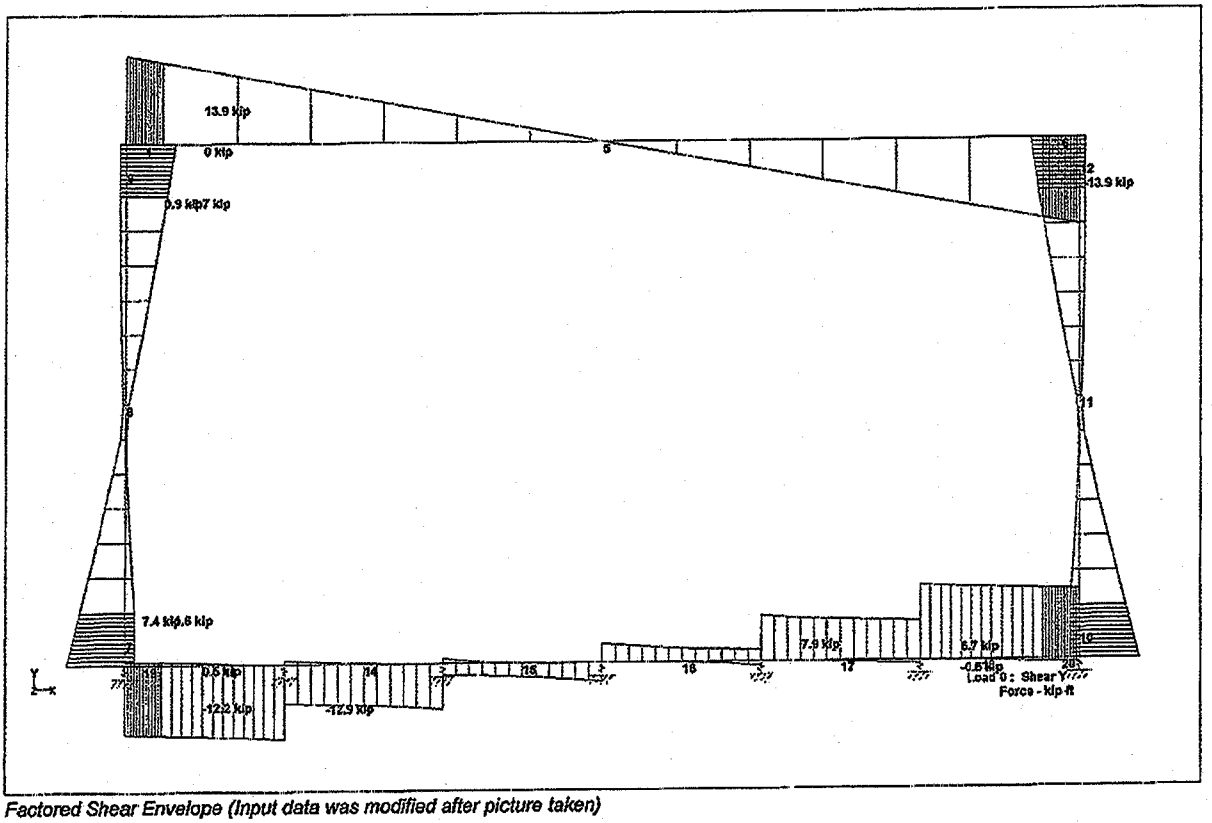
Geometry - Joint & Beam Nos. (Input data was modified after picture taken)

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No #7	Rev
	Part Crest Spillway Design		
Job Title C-43 W. STORAGE RESERVOIR	Ref		
	By D. Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Crest_Spillway.std	Date/Time 12-May-2006 13:11	

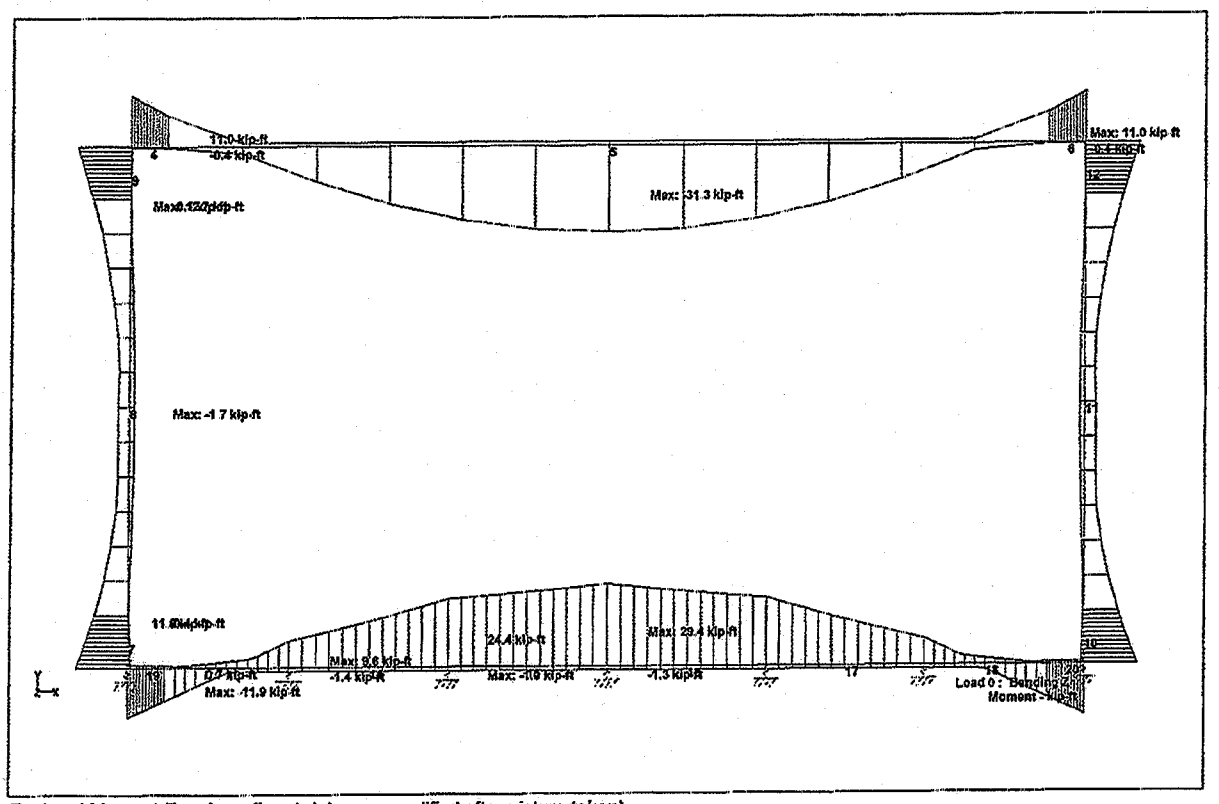


Load Case 12 9-2 INT ALL WATER (Input data was modified after picture taken)

Software licensed to STANLEY CONSULTANTS INC	Job No 17300	Sheet No 5 8	Rev
	Part Crest Spillway Design		
Job Title C-43 W. STORAGE RESERVOIR	Ref		
	By D Beck	Date May-06	Chd J. Lange
Client US Army Core of Engr	File Crest_Spillway.std	Date/Time 12-May-2006 13:11	



Factored Shear Envelope (Input data was modified after picture taken)



Factored Moment Envelope (Input data was modified after picture taken)

Beam End Forces Envelope

Sign convention is as the action of the joint on the beam.

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
5	10	+ve	8.0	13.9	0.0	0.0	0.0	11.0
			11:9-2 EX	11:9-2 EX	-	-	-	11:9-2 EX
			-0.9	0.0	0.0	0.0	0.0	-0.4
			10:9-1 INT	-	-	-	-	10:9-1 INT
	11	-ve	8.0	0.0	0.0	0.0	0.0	11.0
			11:9-2 EX	-	-	-	-	11:9-2 EX
			-0.9	-13.9	0.0	0.0	0.0	-0.4
			10:9-1 INT	11:9-2 EX	-	-	-	10:9-1 INT
8	6	+ve	16.6	7.4	0.0	0.0	0.0	11.5
			11:9-2 EX	11:9-2 EX	-	-	-	11:9-2 EX
			0.0	-1.6	0.0	0.0	0.0	0.0
			-	10:9-1 INT	-	-	-	-
	9	-ve	15.2	0.9	0.0	0.0	0.0	12.7
			11:9-2 EX	10:9-1 INT	-	-	-	11:9-2 EX
			0.0	-6.7	0.0	0.0	0.0	0.0
			-	11:9-2 EX	-	-	-	-

Beam End Forces Envelope Cont...

Beam	Node	Envelope	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip'ft)	My (kip'ft)	Mz (kip'ft)
8	6	+ve	16.6	7.4	0.0	0.0	0.0	11.5
			11:9-2 EX	11:9-2 EX	-	-	-	11:9-2 EX
			0.0	-1.6	0.0	0.0	0.0	0.0
			-	10:9-1 INT	-	-	-	-
	9	-ve	15.2	0.9	0.0	0.0	0.0	12.7
			11:9-2 EX	10:9-1 INT	-	-	-	11:9-2 EX
			0.0	-6.7	0.0	0.0	0.0	0.0
			-	11:9-2 EX	-	-	-	-
13	5	+ve	0.0	0.0	0.0	0.0	0.0	0.1
			-	-	-	-	-	10:9-1 INT
			0.0	-12.2	0.0	0.0	0.0	-11.9
			-	11:9-2 EX	-	-	-	11:9-2 EX
	13	-ve	0.0	0.0	0.0	0.0	0.0	10.5
			-	-	-	-	-	13:CST L(
			0.0	-12.9	0.0	0.0	0.0	0.0
			-	12:9-2 INT	-	-	-	-
15	14	+ve	0.0	0.8	0.0	0.0	0.0	24.4
			-	10:9-1 INT	-	-	-	12:9-2 INT
			0.0	-1.9	0.0	0.0	0.0	0.0
			-	11:9-2 EX	-	-	-	-
	15	-ve	0.0	0.0	0.0	0.0	0.0	29.4
			-	-	-	-	-	11:9-2 EX
			0.0	-3.2	0.0	0.0	0.0	0.0
			-	12:9-2 INT	-	-	-	-

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip'ft)	My (kip'ft)	Mz (kip'ft)
Max Fx	8	6	11:9-2 EXT W/	16.6	7.4	0.0	0.0	0.0	11.5
Min Fx	5	10	10:9-1 INT WA	-0.9	2.5	0.0	0.0	0.0	-0.4
Max Fy	5	10	11:9-2 EXT W/	8.0	13.9	0.0	0.0	0.0	11.0
Min Fy	5	11	11:9-2 EXT W/	8.0	-13.9	0.0	0.0	0.0	11.0
Max Fz	5	10	8:SERV BRG F	4.3	8.9	0.0	0.0	0.0	6.3
Min Fz	5	10	8:SERV BRG F	4.3	8.9	0.0	0.0	0.0	6.3
Max Mx	5	10	8:SERV BRG F	4.3	8.9	0.0	0.0	0.0	6.3
Min Mx	5	10	8:SERV BRG F	4.3	8.9	0.0	0.0	0.0	6.3
Max My	5	10	8:SERV BRG F	4.3	8.9	0.0	0.0	0.0	6.3
Min My	5	10	8:SERV BRG F	4.3	8.9	0.0	0.0	0.0	6.3
Max Mz	15	15	11:9-2 EXT W/	0.0	-2.7	0.0	0.0	0.0	29.4
Min Mz	13	5	11:9-2 EXT W/	0.0	-12.2	0.0	0.0	0.0	-11.9

STAAD CREST SPILLWAY
MODEL INPUT FILE

11


```
STAAD PLANE
START JOB INFORMATION
JOB NAME C-43 W. STORAGE RESERVOIR
JOB CLIENT US Army Core of Engr
JOB NO 17300
JOB PART Crest Spillway Design
JOB COMMENT 6'x10' Single Box
ENGINEER NAME D. Beck
CHECKER NAME J. Lange
ENGINEER DATE May-06
CHECKER DATE May-06
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 13 0 0; 3 0 7.5 0; 4 13 7.5 0; 5 0.5 0 0; 6 0 0.75 0; 7 12.5 0 0;
8 13 0.75 0; 9 0 6.75 0; 10 0.5 7.5 0; 11 12.5 7.5 0; 12 13 6.75 0;
13 2.16667 0 0; 14 4.33333 0 0; 15 6.5 0 0; 16 8.66667 0 0; 17 10.8333 0 0;
MEMBER INCIDENCES
4 3 10; 5 10 11; 6 11 4; 7 1 6; 8 6 9; 9 9 3; 10 2 8; 11 8 12; 12 12 4;
13 5 13; 14 13 14; 15 14 15; 16 15 16; 17 16 17; 18 17 7; 19 1 5; 20 7 2;
*
*
DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 453600
POISSON 0.17
DENSITY 0.14999
ALPHA 5.5e-006
DAMP 0.05
END DEFINE MATERIAL
*
*
MEMBER PROPERTY AMERICAN
4 TO 6 13 TO 20 PRIS YD 1.5 ZD 1
7 TO 12 PRIS YD 1 ZD 1
CONSTANTS
MATERIAL CONCRETE MEMB 4 TO 20
*
*
SUPPORTS
13 TO 17 FIXED BUT MZ KFY 325
1 2 FIXED BUT MZ KFY 238
*
*
*
*
LOAD 1 LOADTYPE Dead  TITLE CONC SLF WGHT
SELFWEIGHT Y -1
LOAD 2 LOADTYPE Soil  TITLE SUB SOIL VERT
MEMBER LOAD
4 TO 6 UNI GY -0.6
LOAD 3 LOADTYPE Soil  TITLE SUBMRGD SOIL HOIZ
MEMBER LOAD
7 TRAP GX 0.56 0.534
8 TRAP GX 0.534 0.326
9 TRAP GX 0.326 0.3
10 TRAP GX -0.56 -0.534
11 TRAP GX -0.534 -0.326
12 TRAP GX -0.326 -0.3
LOAD 4 LOADTYPE Fluids  TITLE EXT WATER
```

12


```
4 TO 6 UNI GY -0.66
7 TRAP GX 1.22 1.164
8 TRAP GX 1.164 0.716
9 TRAP GX 0.716 0.66
10 TRAP GX -1.22 -1.164
11 TRAP GX -1.164 -0.716
12 TRAP GX -0.716 -0.66
LOAD 5 LOADTYPE Fluids  TITLE INT WATER
MEMBER LOAD
8 LIN Y 0.37 0
11 LIN Y -0.37 0
13 TO 18 UNI GY -0.37
LOAD 6 LOADTYPE Soil  TITLE CST SOIL
MEMBER LOAD
4 TO 6 UNI GY -0.36
7 TRAP GX 0.78 0.72
8 TRAP GX 0.72 0.24
9 TRAP GX 0.24 0.18
10 TRAP GX -0.78 -0.72
11 TRAP GX -0.72 -0.24
12 TRAP GX -0.24 -0.18
LOAD 7 LOADTYPE Live  TITLE CST TRUCK
MEMBER LOAD
4 TO 6 UNI GY -0.59
*
*
*
LOAD COMB 8 SERV BRG PRESSURE
1 1.0 2 1.0 3 1.0 4 1.0 5 1.0
LOAD COMB 9 9-1 EXT WATER
1 1.82 4 2.21
LOAD COMB 10 9-1 INT WATER
1 1.82 5 2.21
LOAD COMB 11 9-2 EXT WATER
1 1.56 2 1.56 3 2.08 4 1.56
LOAD COMB 12 9-2 INT ALL WATER
1 1.56 2 1.56 4 1.56 5 1.56 3 2.08
LOAD COMB 13 CST LOADING
1 1.2 6 1.6 7 1.69
PERFORM ANALYSIS
PRINT MEMBER FORCES ALL
PRINT SUPPORT REACTION ALL
FINISH
```

Crest Spillway Roof Slab Design

Slab Details

GEOMETRY

Slab depth =	18.0 in
Design width =	12.0 in
Top clr =	3.0 in
Bot clr =	3.0 in

MATERIAL PROPERTIES

Concrete			Steel Reinf		
f'_c =	3500	psi	f_y =	60000	psi
E_c =	3.41E+06	psi	E_s =	2.90E+07	psi
f_r =	444	psi			
n =	8.51				

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#6		#5	
Bar spa =	10.0	in	12.0	in
Bar dia =	0.75	in	0.625	in
Bar area =	0.44	in2	0.31	in2
$A_{s\,prv'd}$ =	0.53	in2	0.31	in2
No. bars =	1.20		1.00	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 11.10 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot Mat	Top Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	31.30	11.00	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	14.63 in			
A_S, prv'd =	0.53 in2			
ρ =	0.00301			
k =	0.2021			
j =	0.9326			
Comp. Stress Concrete, fc =	50 psi	1400 psi Allowable	OK	Add'l Prv'd = 28.22
Tensile Stress Reinf., fs =	1666 psi	24000 psi Allowable	OK	Add'l Prv'd = 14.4

Top Reinf Mat: Allowable stress = 100%

d = 14.69 in

A_s, prv'd = 0.31 in2
ρ = 0.00176
k = 0 1587
j = 0 9471

Comp. Stress Concrete, fc =	62 psi	1400 psi Allowable	OK	Add'l Prv'd = 22 69
Tensile Stress Reinf., fs =	2783 psi	24000 psi Allowable	OK	Add'l Prv'd = 8 62

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	31.3 ft-lb	I_g =	5832 in4
$4/3M_u$ =	41.7 ft-lb	y_t =	9.0 in
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	31 3 ft-lb		

a = 0.887 in
ϕM_n = 33.7 ft-lb OK Add'l Prv'd = 1.08

Top Reinf Mat:

Demand Moments:

M_u =	11.0 ft-lb
$4/3M_u$ =	14.7 ft-lb
$1.2M_{cr}$ =	28.8 ft-lb
M_{design} =	14.7 ft-lb

a = 0.521 in
ϕM_n = 20.1 ft-lb OK Add'l Prv'd = 1.37

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	6 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 9.86

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u = 11.10 kips
ϕV_c = 17.65 kips OK Add'l Prv'd = 1.59

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

Crest Spillway Floor Slab Design

Slab Details

GEOMETRY

Slab depth =	18.0	in
Design width =	12.0	in
Top clr =	3.0	in
Bot clr =	4.0	in

MATERIAL PROPERTIES

Concrete			Steel Reinf		
f'_c =	3500	psi	f_y =	60000	psi
E_c =	3.41E+06	psi	E_s =	2.90E+07	psi
f_r =	444	psi			
n =	8.51				

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#5		#6	
Bar spa =	12.0	in	10.0	in
Bar dia =	0.625	in	0.75	in
Bar area =	0.31	in2	0.44	in2
$A_{s\ prv'd}$ =	0.31	in2	0.53	in2
No. bars =	1.00		1.20	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 12.90 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot Mat	Top Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	11.90	29.40	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	13.69	in
A_S, prv'd =	0.31	in2
ρ =	0.00189	
k =	0.1639	
j =	0.9454	

Comp. Stress Concrete, fc =	69 psi	1400 psi Allowable	OK	Add'l Prv'd = 20.32
Tensile Stress Reinf., fs =	2992 psi	24000 psi Allowable	OK	Add'l Prv'd = 8.02

Top Reinf Mat: Allowable stress = 100%

d = 14.63 in

A_s, prv'd = 0.53 in2
ρ = 0.00301
k = 0.2021
j = 0.9326

Comp. Stress Concrete, fc =	50 psi	1400 psi Allowable	OK	Add'l Prv'd = 28.22
Tensile Stress Reinf., fs =	1666 psi	24000 psi Allowable	OK	Add'l Prv'd = 14.4

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	11.9 ft-lb	I_g =	5832 in4
$4/3M_u$ =	15.9 ft-lb	y_t =	9.0 in
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	15.9 ft-lb		
a =	0.521 in		
ϕM_n =	18.7 ft-lb	OK	Add'l Prv'd = 1.18

Top Reinf Mat:

Demand Moments:

M_u =	29.4 ft-lb		
$4/3M_u$ =	39.2 ft-lb		
$1.2M_{cr}$ =	28.8 ft-lb		
M_{design} =	29.4 ft-lb		
a =	0.887 in		
ϕM_n =	33.7 ft-lb	OK	Add'l Prv'd = 1.15

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	6 psi	v = V/bd AASHTO Eqn 8.3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 9.23

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u =	12.90 kips		
ϕV_c =	16.52 kips	OK	Add'l Prv'd = 1.28

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$

$M_{cr} = f_r I_g / y_t$

Crest Spillway Wall Design

Slab Details

GEOMETRY

Slab depth =	12.0 in
Design width =	12.0 in
Top clr =	3.0 in
Bot clr =	3.0 in

MATERIAL PROPERTIES

Concrete		Steel Reinf	
f'_c =	3500 psi	f_y =	60000 psi
E_c =	3.41E+06 psi	E_s =	2.90E+07 psi
f_r =	444 psi		
n =	8.51		

REINFORCING

	Bot mat:		Top mat:	
Bar size =	#5		#5	
Bar spa =	12.0	in	10.0	in
Bar dia =	0.625	in	0.625	in
Bar area =	0.31	in2	0.31	in2
$A_{s\ prv'd}$ =	0.31	in2	0.37	in2
No. bars =	1.00		1.20	(within design width)

LOADING

Total Service Shear, V_{serv} = 1 kips

Factored Shear, V_u = 5.40 kips

Note: shear taken distance d from face of support

NOTE: ASD design NOT used, ignore all input and results

	Bot (int) Mat	Top (ext) Mat	
Total Service Moment, M_{serv} =	1	1	ft-kips
Factored Moment, M_u =	1.80	12.70	ft-kips

FLEXURE DESIGN

Working Stress

Bot Reinf Mat: Allowable stress = 100%

d =	8.69 in
A_s, prv'd =	0.31 in2
ρ =	0.00297
k =	0.2011
j =	0.9330

Comp. Stress Concrete, fc =	141 psi	1400 psi Allowable	OK	Add'l Prv'd = 9.91
Tensile Stress Reinf , fs =	4776 psi	24000 psi Allowable	OK	Add'l Prv'd = 5.03

Top Reinf Mat: Allowable stress = 100%

d = 8.69 in

A_s, prv'd = 0 37 in2
ρ = 0.00357
k = 0.2179
j = 0 9274

Comp. Stress Concrete, fc =	131 psi	1400 psi Allowable	OK	Add'l Prv'd = 10.68
Tensile Stress Reinf., fs =	4004 psi	24000 psi Allowable	OK	Add'l Prv'd = 5 99

Load Factor Design $\phi M_n > M_u$ For Flexure ϕ = 0.90

Bot Reinf Mat:

Demand Moments:

M_u =	1.8 ft-lb	I_g =	1728 in4
$4/3M_u$ =	2 4 ft-lb	y_t =	6.0 in
$1.2M_{cr}$ =	12.8 ft-lb		
M_{design} =	2.4 ft-lb		
a =	0.521 in		
ϕM_n =	11.8 ft-lb	OK	Add'l Prv'd = 4.9

Top Reinf Mat:

Demand Moments:

M_u =	12.7 ft-lb		
$4/3M_u$ =	16 9 ft-lb		
$1.2M_{cr}$ =	12.8 ft lb		
M_{design} =	12 8 ft-lb		
a =	0.625 in		
ϕM_n =	14 0 ft-lb	OK	Add'l Prv'd = 1.1

SHEAR DESIGN

Working Stress

Allowable stress =	100%	
Shear stress, v =	10 psi	v = V/bd AASHTO Eqn 8-3 (using min d value)
Conc shear stress, v_c =	56 psi	$v_c = 0.95*f'_c^{1/2}$, AASHTO 8.15.5.2.1
		OK Add'l Prv'd = 5 86

Load Factor Design $\phi V_c > V_u$ For shear ϕ = 0.85

V_u =	5.40 kips		
ϕV_c =	10 48 kips	OK	Add'l Prv'd = 1.94

Working Stress Design Formulas

$k = [2n\rho + (n\rho)^2]^{1/2} - n\rho$
$j = 1 - k/3$
$f_c = 2M/(kjbd^2)$
$f_s = M/(A_s jd)$
$f_{vc} = V/bd$

Load Factor Design Formulas

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad M_n = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$\phi V_c = \phi 2(f'_c)^{1/2} bd$
$M_{cr} = f_r I_g / y_t$

S12 Cell Balancing Structure - Structural Analysis & Design

Stilling Basin Structure Analysis (STAAD.Pro)
*See C43-S1/S2 Main Outlet Structural Computations for similar Stilling Basin Analysis

Stilling Basin Structure Design
*See C43-S1/S2 Main Outlet Structural Computations for similar Stilling Basin Design

Construction Heavy Equipment Information
*See C43-S1/S2 Main Outlet Structural Computations for Heavy Equipment Information