CPRA Contract No. 2503-11-63

FEDERAL PROJECT SPONSOR

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STATE OF LOUISIANA
COASTAL PROTECTION AND
RESTORATION AUTHORITY
COASTAL ENGINEERING DIVISION

RIVER REINTRODUCTION INTO MAUREPAS SWAMP

PROJECT No. PO-29
ST. JOHN THE BAPTIST PARISH, LOUISIANA

Pump Station
95% Structural Calculations Submittal

September 2013

RIVER REINTRODUCTION INTO MAUREPAS SWAMP: PUMP STATION STRUCTURAL CALCULATIONS

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SECTION 1

General Information

DESIGN CRITERIA

STRUCTURAL DESIGN CRITERIA

1. General

This design criteria includes a general description and definition of the basic structural design criteria controlling the design of the Pump Station (PS), control/operations buildings, and bridge pertaining to The Mississippi River Reintroduction into Maurepas Swamp Project. The design elements defined herein represent a 95-percent level conceptual design using the best available information. Analysis and design calculations for this phase can be found in Appendix C. Only the pump station is included in this report. The access bridge, parking platform and any other miscellaneous structures will be designed once they are more clearly defined in the next phase.

2. Codes and Standards

The following is a list of general Corps References and Industry codes and standards which are applicable to structural design. Local codes will govern in case of conflicting requirements. All of the general codes and standards listed below apply to design elements such as the PS, operations/control buildings and bridge, but are not necessarily limited to, the following:

- AASHTO, American Association of State Highway and Transportation Officials, LRFD 3rd Edition, 2004 with Interim Revisions excluding Section 6 of 2006.
- ACI 318-02 American Concrete Institute, Building Code Requirements for Structural Concrete.
- ACI 350R-89 American Concrete Institute, Concrete Sanitary Engineering Structures
- AISC, American Institute of Steel Construction, Inc., Manual of Steel Construction, 9th Edition
- ASCE 7-05 American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures
- ASTM, American Society for Testing and Materials
- AWS D1.1-02 American Welding Society, Structural Welding Code, latest edition
- COE EM 385-1-1 Safety and Health Requirements Manual, 03 Nov 03
- COE EM 1110-2-2000 Standard Practice for Concrete for Civil Works Structures
- COE EM 1110-2-2102, Water Stops and Other Preformed Joint Material for civil Works Structures

- COE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
- COE EM 1110-2-2105, Design of Hydraulic Steel Structures
- COE EM 1110-2-2502, Retaining and Flood Walls
- COE EM 1110-2-2906, Design of Pile Foundations
- COE EM 1110-2-3104, Structural and Architectural Design of Pumping Stations
- USACE (COE), Applicable U.S. Army Corps of Engineers Requirements (EMs/TMs)
- Aluminum Design manual

3. General Design Load Parameters

3.1.A. Load Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in ASCE 7-05. If load combinations are prescribed in an applicable engineering manual, this shall take precedence over ASCE 7-05.

Table 2
SERVICE LOAD COMBINATIONS
(SERVICEABILITY, DEFLECTION, CRACKING, PILE DESIGN)

Load Combinations		Strength Design U=Rf*Hf*(D+L+...)									
		Allowable Overstress Structure (%)	Dead (D)	Live (L)	Hydro-Static (H)	Uplift (U)	Wind (W)	Soil (S)	Settlement (ST)	Impact (I)	Notes:
Construction											
Construction condition	A1	1.166	1.0				1.0		1.0		
Operation											
Normal Operation Condition	B1	1	1.0	1.0	1.0	1.0		1.0		1.0	
Start-up Condition	B2	1	1.0	1.0	1.0	1.0		1.0		1.0	
High Head Condition	B3	1	1.0	1.0	1.0	1.0		1.0		1.0	
Blocked trash Rack Condition	B4	1.166	1.0	1.0	1.0	1.0		1.0		1.0	
Hurricane											
Hurricane Condition	C1	1.333	1.0	1.0	1.0	1.0	1.0	1.0		1.0	
Maintenance											
Maintenance Conditions	D1	1.166	1.0	1.0	1.0	1.0		1.0		1.0	

Table 3
STRENGTH LOAD COMBINATIONS
(CONCRETE DESIGN)

Load Combinations		Strength Design U=Rf*Hf*(D+L+...)										Notes:
		Reduction Factor (Rf)	Hydraulic Factor (Hf)	Dead (D)	Live (L)	Hydro-Static (H)	Uplift (U)	Wind (W)	Soil (S)	Settlement (ST)	Impact (I)	
Construction												
Construction condition	A1	0.86	1.3	1.7				1.7		1.7		
Operation												
Normal Operation Condition	B1	1	1.3	1.7	1.7	1.7	1.7		1.7		1.7	
Start-up Condition	B2	1	1.3	1.7	1.7	1.7	1.7		1.7		1.7	
High Head Condition	B3	1	1.3	1.7	1.7	1.7	1.7		1.7		1.7	
Blocked trash Rack Condition	B4	0.86	1.3	1.7	1.7	1.7	1.7		1.7		1.7	
Hurricane												
Hurricane Condition	C1	0.75	1.3	1.7	1.7	1.7	1.7	1.7	1.7		1.7	
Maintenance												
Maintenance Conditions	D1	0.86	1.3	1.7	1.7	1.7	1.7		1.7		1.7	

3.1.A. Hydraulic Stages

TABLE 1

Hydraulic Stages and Design Water Surface Elevations

Stage	Intake Side	Discharge Side
Normal	EL. 0.5	EL. 2.0
Max. Water	EL. 3.5	EL. 5.0
Min. Water	EL. 0.0	EL. 1.0

3.2. Load Cases

3.2.1. Dead Loads (D)

Dead loads shall be determined in accordance with applicable engineering manuals and ASCE 7-05, and shall include the self-weight of all permanent construction components

including foundations, slabs, walls, roofs, actual weights of equipment, overburden pressures, and all permanent non-removable stationary construction.

Table 4. - Unit Weights

Item	Weight [Pcf]	Notes
Water (Fresh)	62.4	
Semi-compacted Fill	110	
Fully Compacted Granular Fill, wet	120	
Fully Compacted Granular Fill, Effective	58	
Fully Compacted Clay Fill, wet	110	
Fully Compacted Clay Fill, Effective	48	
Riprap	130	
Silt	94	
Reinforced Concrete (Normal weight)	150	
Steel	490	

3.2.1.1 Equipment weight (D-E)

Equipment weight (65-percent level design) based on the best available information:

Engine: 1,711 lb (CAT C9, see attachment 1)
Gear Drive: 1,766 lb (DE'RAN Model M26A, see attachment 1)
FSI : 11,000 lb (approximate)
Pump: 11,300 lb (MWI Model N342x48, see attachment 1).

Note: Add weight of the water column for most pumps, with a 50% increase to account for dynamic effects.

3.2.2. Live Loads (L)

Live loads for building structures shall be determined in accordance with applicable engineering manuals and ASCE 7-05.

Roof Live Loads:

- Roof Live Loads: 60 psf

Floor Live Loads:

- Minimum unless noted otherwise: 100 psf
- Grating floors: 100 psf or a 300-pound concentrated load.
- Stairs and landings: 100 psf or a 300-pound concentrated load.
- Operating Floors: 250 psf .
- Equipment and Storage Rooms: 300 psf
- Control room: 125 psf
- Service Bridge: The worst condition of the following vehicles: 50 tons crane or AASHTO HS-20 truck.

3.2.2.1 Live Load Surcharge (LS)

A minimum vertical live load surcharge of 200 psf will be applied on floor slab and base slab during construction.

A minimum horizontal live load surcharge of 300 psf will be applied to all abutment walls and wing walls of hydraulic structures in addition to other live loads that may be applicable in accordance with AASHTO.

3.2.3. Soil Pressures (S)

Structures are designed for lateral and vertical soil pressures. Lateral pressures are determined using the at-rest coefficients, K_O obtained from the Geotechnical Report:

- Lateral Soils at-rest Pressure Coefficients:
  Ko = 0.8 for Clay.
  Ko = 0.48 for Granular Material.

3.2.4. Hydrostatic Loads (H)

Hydrostatic loads for which structures will be designed refer to the vertical and horizontal loads induced by a static water head and buoyant pressures, excluding uplift pressures. Dynamic Wave Load is neglected.

3.2.5. Uplift Loads (U)

Uplift loads for which structures will be designed are defined by two uplift conditions: Uplift Condition A, assumes the sheet pile cutoff wall is fully effective, and Uplift Condition B, assumes the sheet pile cutoff wall is ineffective (pressure assumed to be vary linearly across the base).

3.2.6. Wind Loads (W)

Structures are designed for wind loads established by ASCE No. 7, "Minimum Design Loads for Buildings and Other Structures," ***but in no case less than 50 psf***. The basic sustained wind speed is 120 miles per hour, and the exposure category is "C". Architectural roofs shall be designed for a 135 mile-per-hour sustained wind.

The importance factor used for all structures is to be 1.3. For flood control structures, a MRI of 200 years (Importance Factor = 1.3) will be used.

3.2.7. Impact Loads (I)

3.2.7.1. Operational Impact Loads

For elements supporting reciprocating or rotating equipment and cranes proper allowance, or as determined by analysis, shall be made for impact in addition to other loads. The following minimum impact loads shall be used.

- Traveling cranes and hoists: 25% of the lifted loads.
- Rotating equipment: 20% of the total machine weight.
- Reciprocating equipment: 50% of the total machine weight. Consideration will be given to the deflection of beams supporting reciprocating and rotating machines.

3.2.7.2. Pedestrian Railing Loads

- 200 lb (min.) concentrated load at top of railing in any direction and any location.
- 50 lbs/ft transverse and vertical simultaneously on all longitudinal members (rails).
- 50 lbs/ft x post spacing at height to center of top rail at each post.

3.2.7.3. Access Bridge Railing Impacts

Access bridge railings shall be designed per AASHTO for highway truck railing loadings.

3.2.8. Vibration Loads (V)

To help dampen vibration, equipment will be supported on concrete having a weight at least 3 times the total weight of the equipment or 15 times the rotating weight, whichever is greater. Vibration during the pumps operation shall include a dynamic factor of 1.3. A refined analytical approach shall be performed if required.

3.2.9. Settlement Loads (ST)

Structures are designed for forces generated by settlement (downdrag) in coordination with the Geotechnical Design. Downdrag forces are applied to sustained load cases (i.e., construction). The downdrag force exerted by settling soil adjacent to the Pump Station is applied to the perimeter of the structure. Downdrag forces are also included in the structural check of the piles. Downdrag loads are obtained from the geotechnical engineer on a case-by-case basis as applicable. How downdrag forces on piles are computed is explained in the geotechnical report.

4. Concrete Design Criteria

Concrete design shall utilize EM 1110-2-2104 and the ACI 350R Concrete Sanitary Engineering Structures and will comply with the ACI 318 latest edition strength design method, unless otherwise required. Typical design values are as follows unless otherwise noted:

- Structural Concrete: 4,000 psi @ 28 days with a maximum water/cement ratio = 0.40
- Steel reinforcement 60,000 psi (ASTM A615)

5. Steel Design Criteria

Steel design shall utilize the EM 1110-2-2105 and the 1989 AISC Steel Construction Manual, 9th edition. Load combinations shall be in accordance with ASCE 7-05. Typical design values are as follows unless otherwise noted:

(a)	Structural steel rolled shapes	ASTM 572, Grade 50 ASTM A992, Grade 50
(b)	Plates	ASTM A992, Grade 36
(c)	Bolts and nuts	ASTM A325, min. ¾" ASTM A490
(d)	Anchor Bolts	ASTM A449, (¾" dia. or greater)
(e)	Corrosion stainless steel	ASTM A304 (freshwater) ASTM A316 (saltwater)
(f)	Sheet Piles	ASTM A328, Grade 50 ASTM A572, Grade 50
(g)	Stainless Steel Embedded Anchors	ASTM A276 or UNS S21800

Normally, components that shall be exposed to the elements are either hot-dipped galvanized or primed, painted and sealed with coats of (10 mils min.) epoxy. Vertical lift gates and steel sheet pile structures shall be painted with an epoxy painting system.

6. Aluminum Design Criteria

General criteria for aluminum shall be in accordance with the Aluminum Design Manual. Aluminum alloy 6061-T6 will be used for the basic design of aluminum structures and members.

7. Pile Foundation Design Criteria

All forces applied to primary structures are resisted by the pile foundation. The Pump Station is supported independently and is not designed to transmit load to any adjoining structure. Pile designs are based on a soil structure interactive analysis, with the pile

supports input as springs in accordance with EM 1110-2-2906. The springs are based on P-Y and T-Z curves generated by geotechnical analysis. Group effect will be applied as required.

Piles may be steel pipe piles, steel H piles or square prestressed concrete piles. Pipe piles satisfy ASTM A252 with a minimum yield strength of 45 ksi. H-piles satisfy ASTM A36. Steel piles are designed structurally per AISC ASD, 9th Edition, as modified by EM 1110-2-2906. Concrete square piles have a strength equal to 5,000 psi at 28 days. Prestressed concrete piles (hollow or solid) are designed to satisfy both strength and serviceability requirements. Strength design follows the basic criteria set forth by ACI, except the strength reduction factor is 0.7 for all failure modes and the load factor is 1.9 for both dead and live loads. The prestressed concrete pile is designed for an axial strength limited to 80 percent of pure axial strength and a minimum eccentricity equal to 10 percent of the pile width. Control of cracking is achieved by limiting the concrete compressive stress to 0.4f'c and the tensile stress to zero. Combined axial and bending are considered when analyzing the stresses in the piles. Loads, deflections and stresses are presented for each design case.

CPGA will be the foundation analysis and design software used for 65% Design.

STRUCTURAL DESIGN

OF

PUMP STATION

Job: Maurepas Pump Station
Job No.: 10001663
Description: Introduction to the 65% Structural Design of the Maurepas Freshwater Diversion Pumping Station

Calculated By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

①

What follows is the structural design portion of the 95% design submittal for the Mississippi River Reintroduction into Maurepas Swamp Project pump station. Within this design all major structural elements are laid out, sized and detailed. Reinforcement has been designed for all major structural concrete elements and the foundation plan has been finalized. Also, the approach ramp and adjacent platform have been preliminarily designed. Loading values and load combinations are based on the recommendations of the documents listed in the Design Criteria.

a. General. The pump station is comprised of a pre-engineered metal building that will enclose the pumping equipment and control office and a concrete structure that acts as a foundation, a roadway, a trash rack support and housing for the intake equipment. The metal building measures 30 feet by 59 feet; the peak of the roof stands 36 feet (40 foot height used for wind analysis) above the floor slab and both sides of the roof have a slope of 1V:3H. The concrete structure is built off of a 2 foot 6 inch thick base slab with a 45 foot by 46 foot footprint. The top of this base slab is at elevation -7.0 feet NAVD. Four sump walls rise 18 feet 3 inches out of the slab to form three 12 foot 4 inch wide channels that guide water to the formed suction intakes at the base of the structure. The two outside sump walls and the head wall are 2 feet 6 inches thick while the two interior walls are 2 feet thick. Not only do these channels house the intake equipment, they also contain grooved tracks to hold the bulkheads and trash racks in place. The operating floor slab rests on top of the inlet walls at elevation 12.5 feet NAVD. This slab measures 51 feet 8 inches by 65 feet and extends 5 feet beyond the West side, 18 feet 10 inches beyond the North side and 14 feet beyond the East side of the base slab. The floor slab can be divided into three sections: a 1 foot 3 inch thick main operating floor within the metal building, a 14 foot wide roadway of the same thickness along the South side and a 9 inch thick walkway around the North and West sides of the building. Where the floor slab extends beyond the intake area, concrete piles rise above grade to support a network of 2'2" wide x 2' thick & 2'2" wide x 2'2" thick concrete floor beams connected to the slab.

The pile foundation is comprised of seventy-nine 14 inch by 14 inch prestressed precast concrete piles. Fifty-six of these piles are beneath the base slab while the remainders are supporting the floor beams as described above. All piles extend to elevation -80.0 feet NAVD.

Because the metal portion of the structure is a pre-engineered building, only a small amount of design of this section is presented in this report. Much analysis, however, has been done on the loads applied to the metal building and the impact of these loads on the concrete portion of the structure. A complete wind analysis is performed and a preliminary estimate of dead and live loads is generated so that the forces transferred by this structure can be accounted for in the rest of our design.

b. Modeling. Two SAP2000 models were used to design the concrete portion of the pump station. A 2-D frame model is used to find the base reactions of the pre-engineered steel structure and a 3-D model is used to design the concrete portion of the pump station. The 2-D model combines a number of wind, live, and dead loads to estimate the steel building's impact on the concrete structure. The model represents a cut in the building's short direction and is comprised of two 31 foot 3 inch columns and two roof beams on a 1 to 3 slope. The columns are fixed at their base to simulate the sections being embedded into the concrete structure. A 15 foot tributary length is used in the application of all area loads because this is the maximum column-to-column spacing for the structure. Pressures from wind analysis in the direction perpendicular to the roof ridge (in the short direction) are used in the model. Wind analysis is performed for both the directions parallel and perpendicular to the roof ridge. The wind forces found in the perpendicular direction (the building's short direction) are input into the model, except the windward load on the wall, which is set at 50 psf along the whole wall in accordance with the Design Criteria. Along with wind loads, a live roof load of 60 psf is applied as per the Design Criteria. The crane system that will be supported by corbels on the columns is represented with a downward force of 10 kips per column as well as a 6.67 kip-ft moment to account for the load's 8 inch offset from the column. Also, the weight of the insulated metal paneling on the walls and roof are modeled by 3 psf area loads.

Because the metal section of the pump station building will be a pre-engineered structure, an assumption needs to be made about the building's weight so that this can be accounted for in both the wind model and the 3-D concrete model. This is done by first creating the model with default frames, running all load combinations, and finding the maximum moments in the columns and the roof beams. These moments are then used to find the minimum section modulus needed to carry the moments and W-shapes are chosen accordingly. The frame elements in the model are then replaced with these W-shapes and the model is run again. This run produces two sets of reactions that are used in the design of the rest of the pump station, one with the worst case wind load reactions, and the other without any wind loads to simulate

The 3-D concrete model is mostly comprised of thick shell and frame elements. The shell elements are primarily comprised of 4000 psi concrete (frames for piles are assigned 5000 psi concrete) and are assigned the appropriate thicknesses. At all member intersections, stiffened concrete elements are used to represent the added rigidity found at these junctions. This procedure is a 3-dimensional approach to the rigid link method explained in Chapter 7 of ETL 1110-2-355. A length of one quarter of the thickness of the adjoining section is separated from the regular shell elements and the concrete therein is strengthened by multiplying its modulus of elasticity by 100. Concrete frame members are used to model the beams that run throughout the floor slab and also to model the piles that rise above ground elevation to meet the floor beams. The floor beams will be monolithically constructed with the floor slab, meaning only a small portion of these members will be protruding from the slab. Because the beam frames and the slab shell

elements are separate entities in the model, steps were taken to make sure the concrete volume at the tops of these beams is not accounted for twice. The weight of the beam's concrete is multiplied by the ratio of concrete within the slab to concrete beneath the slab, decreasing the weight of these beams to a more realistic level.

Pile-soil interaction is modeled with the use of soil spring constants. Vertical and horizontal load versus deflection curves provided by the geotechnical engineers are used to determine these spring values. For the axial spring, the slope of the linear part of the axial load versus displacement curve is used as the constant. The horizontal springs, however, need to be determined by an iterative procedure. First, a reasonable amount of deflection is chosen and the corresponding lateral load is found. In this instance, a starting point of 0.5 inches is used. The spring constant, which is the load divided by the deflection, is calculated and input into the model. The maximum lateral deflection of the model is recorded and, if this deflection is not close to that of the spring constant, a new spring constant is calculated the same way, this time with the resulting deflection as the basis for the new value. This procedure is repeated until the deflections match.

SECTION 2

Wind & Crane Load Analysis of Superstructure

A 2-D SAP2000 model is used to determine the member forces and base reactions that result from various combinations of wind loads. The model represents a cut in the building's short direction and is comprised of two 31 foot 3 inch columns and two roof beams on a 1 to 3 slope. The columns are fixed at their base to simulate the sections being embedded into the concrete structure. A 15 foot tributary length is used in the application of all area loads because this is the maximum column-to-column spacing for the structure. Pressures from wind analysis in the direction perpendicular to the roof ridge (in the short direction) are used in the model. Along with wind loads, a live roof load of 60 psf is applied as per the Design Criteria. The crane system that will be supported by corbels on the columns is represented with a downward force of 25 kips per column as well as a 16.67 kip-ft moment to account for the load's 8 inch offset from the column. Also, the weight of the insulated metal paneling on the walls and roof are modeled by 3 psf area loads.

Because the metal section of the pump station building will be a pre-engineered structure, an assumption needs to be made about the building's weight so that this can be accounted for in both the wind model and the 3-D concrete model. This is done by first creating the model with default frames, running all load combinations, and finding the maximum moments in the columns and the roof beams. These moments are then used to find the minimum section modulus needed to carry the moments and W-shapes are chosen accordingly. The frame elements in the model are then replaced with these W-shapes and the model is run again. This run produces two sets of reactions that are used in the design of the rest of the pump station, one with the worst case wind load reactions, and the other without any wind loads to simulate

Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations - perpendicular to roof ridge

Computed By: JY (6)
Date: 08/2010
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Date: 12/10

ASCE 7-05 Method 2 - Analytical Procedure for Enclosed/Partially Enclosed Structures

Input:

Basic Wind Speed (V):	**120** mph		(ASCE7-05 Fig. 6-1)
Wind Directionality Factor (K_d):	**0.85**		(ASCE7-05 Table 6-4)
Importance Factor (I):	**1.3**		(ASCE7-05 Table 6-1)
Topographic Factor (K_{zt}):	**1.0**		
Exposure Category:	**C**		(See ASCE7-05 C6.5.6)
Case 1 or 2 (see Table 6-3):	**2**		
Enclosure Classification:	**1**		(See ASCE7-05 6.2)
Enter 1 for Partially Enclosed			
Enter 2 for Enclosed			
Will Topographic Factor be calculated:	**2**		
Enter 1 for Yes			
Enter 2 for No			
Height of structure:	**40** ft		
Dimension Parallel to Wind, L:	**30** ft		
Dimension Perpendicular to Wind, B:	**59.5** ft		
Roof Slope	**18.49** deg	(3H:1V)	
If wind Parallel to ridge, enter 0 deg			

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Calculations:

1) Fundamental Frequency: Least length/width dimension: 30 ft

* Frequency = 1.33 - rigid structure

2) Velocity Pressure, q_z: $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (lb/ft2) (ASCE7-05 Eq. 6-15)

For $z < 15ft$: $K_z = 2.01(15/z_g)^{2/\alpha}$

For $15ft \le z \le z_g$: $K_z = 2.01(z/z_g)^{2/\alpha}$

z_g = 900
α = 9.5

Height (z) ft	K_z	q_z lb/ft^2	
10	0.849	34.6	
15	0.849	34.6	
20	0.902	36.7	
25	0.945	38.5	
30	0.982	40.0	
36	1.021	41.6	
40	1.044	42.5	** qh
50	1.094	44.6	

*** PLEASE ENTER q_h VALUE HERE:** **42.5** psf

3) Internal Pressure Coefficient, p_{int}: $p_{int} = q_i(GC_{pi})$ GC_{pi} = 0.55
-0.55

* $q_i(GC_{pi})$ = 23.4 psf (ACTING TOWARDS INTERNAL SURFACE)
-23.4 psf (ACTING AWAY FROM INTERNAL SURFACE)

4) External Wall Pressures, p_{ext}: $p_{ext} = q(GC_p)$ G = 0.85
L/B = 0.5

Windward Wall:	C_p =	0.80	GC_p =	0.680
Leeward Wall:	C_p =	-0.50	GC_p =	-0.425
Side Walls:	C_p =	-0.70	GC_p =	-0.595

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5) Design Wall Wind Pressure, p: $p = p_{ext} - p_{int} = qGC_p - q_i(GC_{pi})$ (lb/ft^2)

(ASCE7-05 Eq 6-17)

	Height (z) ft	q_z lb/ft^2	p_{ext} lb/ft^2	p_{int} (+/-) lb/ft^2	**p lb/ft^2**	**p lb/ft^2**
Windward Wall:	10	34.6	23.5138	23.4	**0.1**	**46.9**
	15	34.6	23.5138	23.4	**0.1**	**46.9**
	20	36.7	24.9819	23.4	**1.6**	**48.4**
	25	38.5	26.1835	23.4	**2.8**	**49.6**
	30	40.0	27.208	23.4	**3.8**	**50.6**
	36	41.6	28.2727	23.4	**4.9**	**51.6**
	40	42.5	28.9068	23.4	**5.5**	**52.3**
	50	44.6	30.2972	23.4	**6.9**	**53.7**

	q_h lb/ft^2	p_{ext} lb/ft^2	p_{int} (+/-) lb/ft^2	**p lb/ft^2**	**p lb/ft^2**
Leeward Wall:	42.5	-18.063	23.4	**-41.4**	**5.3**
Side Walls:	42.5	-25.288	23.4	**-48.7**	**-1.9**

6) External Roof Pressures, p_{ext}: $p_{ext} = q(GC_p)$

G = 0.85
h/L = 1.33
θ = 18.49 deg

* Where Roofs θ<10° or Wind applied Parallel to Ridge:

	C_p	GC_p
0 to h/2:	SEE θ ≥ 10 PROCEDURE	
h/2 to h:		
h to 2h:		
> 2h:		

* Where Roofs θ≥10°:

Windward Cp Table - Negative Pressures								
	θ							
h/L	10	15	20	25	30	35	40	45
0.25	-0.7	-0.5	-0.3	-0.2	-0.2	0	0	0
0.5	-0.9	-0.7	-0.4	-0.3	-0.2	-0.2	-0.1	0
1	-1.3	-1	-0.7	-0.5	-0.3	-0.2	-0.1	0

	15	20	
1	-1	-0.7	-0.79
1	-1	-0.7	-0.79

C_p = -0.79 GC_p = -0.67

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* Where Roofs θ≥10° (continued):

Windward Cp Table - Positive Pressures																	
	θ																
h/L	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90
0.25	-0.18	0	0.2	0.3	0.3	0.4	0.4	0.4	0.47	0.53	0.6	0.65	0.7	0.75	0.8	0.8	0.8
0.5	-0.18	-0.18	0	0.2	0.2	0.3	0.35	0.4	0.47	0.53	0.6	0.65	0.7	0.75	0.8	0.8	0.8
1	-0.18	-0.18	-0.18	0	0.2	0.2	0.25	0.3	0.4	0.5	0.6	0.65	0.7	0.75	0.8	0.8	0.8

	15	20	
1	-0.18	-0.18	-0.18
1	-0.18	-0.18	-0.18

C_p = -0.18 GC_p = -0.15 (The positive pressures have negative values because of building geometry)

Leeward Cp Table			
	θ		
h/L	10	15	20
0.25	-0.3	-0.5	-0.6
0.5	-0.5	-0.5	-0.6
1	-0.7	-0.6	-0.6

	15	20	
1	-0.6	-0.6	-0.60
1	-0.6	-0.6	-0.60

C_p = -0.60 GC_p = -0.51

7) Design Roof Wind Pressure, p: $p = p_{ext} - p_{int} = qGC_p - q_i(GC_{pi})$ (lb/ft^2)

* Where Roofs θ<10° or Wind applied Parallel to Ridge: p_{int} = 23.4 psf (+/-)

p_{ext} for 0 to h/2:	N/A	psf	**p = N/A**	**psf &**	**p = N/A**	**psf**
p_{ext} for h/2 to h:	N/A	psf	**p = N/A**	**psf &**	**p = N/A**	**psf**
p_{ext} for h to 2h:	N/A	psf	**p = N/A**	**psf &**	**p = N/A**	**psf**
p_{ext} for > 2h:	N/A	psf	**p = N/A**	**psf &**	**p = N/A**	**psf**

* Where Roofs θ≥10°:

Windward Roof Pressures:

Positive C_p		**Negative C_p**
p_{ext} = -6.503 psf	&	-28.6 psf

p_1_pos = -29.88 psf (wind pressure away from int. surface) **p_1_neg = -51.94 psf** (wind pressure towards internal surface)
p_2_pos = 16.87 psf **p_2_neg = -5.19 psf**

Leeward Roof Pressures:

p_{ext} = -21.675 psf

p_neg =	**-45.05 psf**	Negative	(wind pressure towards internal surface)
p_pos =	**1.70 psf**	Positive	(wind pressure away from internal surface)

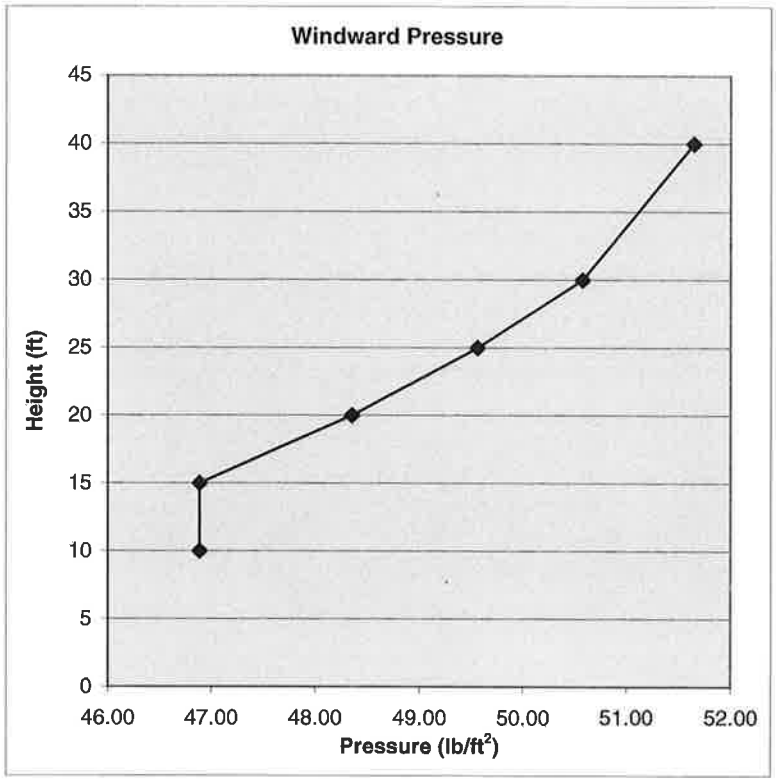
Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations perpendicular to roof ridge
Results

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

WALL PRESSURES

* Windward Wall Pressures:

Height (z) ft	p lb/ft^2	p lb/ft^2
10	0.14	46.89
15	0.14	46.89
20	1.61	48.36
25	2.81	49.56
30	3.83	50.58
40	4.90	51.65
50	6.92	53.67



* Leeward Wall Pressures:

p = -41.44 lb/ft^2
p = 5.31 lb/ft^2

* Side Wall Pressures:

p = -48.66 lb/ft^2
p = -1.91 lb/ft^2

Note: (+) pressures act towards surface, (-) pressures act away from surface

Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations perpendicular to roof ridge
Results

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

ROOF PRESSURES

* Pressures for Roofs with θ<10°:

p for 0ft to 20ft:	p =	N/A	psf &	p =	N/A	psf
p for 20ft to 40ft:	p =	N/A	psf &	p =	N/A	psf
p for 40ft to 80ft:	p =	N/A	psf &	p =	N/A	psf
p for > 80ft:	p =	N/A	psf &	p =	N/A	psf

* Pressures for Roofs with θ≥10°:

Windward Roof Pressures:

p =	-29.88 psf	-51.94 psf
	16.87 psf	-5.19 psf

Leeward Roof Pressure:

p =	-45.05 psf	&	1.70 psf

Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations parallel to roof ridge

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

ASCE 7-05 Method 2 - Analytical Procedure for Enclosed/Partially Enclosed Structures

Input:

Basic Wind Speed (V):	**120** mph	(ASCE7-05 Fig. 6-1)
Wind Directionality Factor (Kd):	**0.85**	(ASCE7-05 Table 6-4)
Importance Factor (I):	**1.3**	(ASCE7-05 Table 6-1)
Topographic Factor (K_{zt})	**1.0**	
Exposure Category:	**C**	(See ASCE7-05 C6.5.6)
Case 1 or 2 (see Table 6-3):	**2**	
Enclosure Classification:	**1**	(See ASCE7-05 6.2)
Enter 1 for Partially Enclosed		
Enter 2 for Enclosed		
Height of structure:	**40** ft	
Dimension Parallel to Wind, L:	**59.5** ft	
Dimension Perpendicular to Wind, B:	**30** ft	
Roof Slope	**0** deg	
If wind Parallel to ridge, enter 0 deg		

Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations parallel to roof ridge

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

Calculations:

1) Fundamental Frequency: Least length/width dimension: 30 ft

* Frequency = 1.33 - rigid structure

2) Velocity Pressure, q_z: $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (lb/ft2) (ASCE7-05 Eq. 6-15)

For $z < 15ft$: $K_z = 2.01(15/z_g)^{2/\alpha}$

For $15ft \le z \le z_g$: $K_z = 2.01(z/z_g)^{2/\alpha}$

z_g = 900

α = 9.5

Height (z) ft	K_z	q_z lb/ft^2	
10	0.849	34.6	
15	0.849	34.6	
20	0.902	36.7	
25	0.945	38.5	
30	0.982	40.0	
36	1.021	41.6	
40	1.044	42.5	** qh
50	1.094	44.6	

* PLEASE ENTER q_h VALUE HERE: **42.5** psf

3) Internal Pressure Coefficient, p_{int}: $p_{int} = q_i(GC_{pi})$ GC_{pi} = 0.55, -0.55

* $q_i(GC_{pi})$ = 23.4 psf, -23.4 psf

4) External Wall Pressures, p_{ext}: $p_{ext} = q(GC_p)$ G = 0.85, L/B = 2.0

Windward Wall:	C_p =	0.80	GC_p =	0.680
Leeward Wall:	C_p =	-0.30	GC_p =	-0.258
Side Walls:	C_p =	-0.70	GC_p =	-0.595

Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations parallel to roof ridge

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

5) Design Wall Wind Pressure, p $\quad p = p_{ext} - p_{int} = qGC_p - q_i(GC_{pi})$ (lb/ft^2)

(ASCE7-05 Eq 6-17)

	Height (z) ft	q_z lb/ft^2	p_{ext} lb/ft^2	p_{int} (+/-) lb/ft^2	**p lb/ft^2**	**p lb/ft^2**
Windward Wall:	10	34.6	23.5138	23.4	**0.1**	**46.9**
	15	34.6	23.5138	23.4	**0.1**	**46.9**
	20	36.7	24.9819	23.4	**1.6**	**48.4**
	25	38.5	26.1835	23.4	**2.8**	**49.6**
	30	40.0	27.208	23.4	**3.8**	**50.6**
	36	41.6	28.2727	23.4	**4.9**	**51.6**
	40	42.5	28.9068	23.4	**5.5**	**52.3**
	50	44.6	30.2972	23.4	**6.9**	**53.7**

	q_h lb/ft^2	p_{ext} lb/ft^2	p_{int} (+/-) lb/ft^2	**p lb/ft^2**	**p lb/ft^2**
Leeward Wall:	42.5	-10.96	23.4	**-34.3**	**12.4**
Side Walls:	42.5	-25.29	23.4	**-48.7**	**-1.9**

6) External Roof Pressures, p_{ext}: $\quad p_{ext} = q(GC_p)$

G = 0.85
h/L = 0.67
θ = 0 deg

* Where Roofs θ<10° or Wind applied Parallel to Ridge:

	C_p	GC_p
0 to h/2:	-1.038	-0.88
h/2 to h:	-0.831	-0.71
h to 2h:	-0.569	-0.48
> 2h:	-0.438	-0.37

* Where Roofs θ≥10°:

Windward Cp Table - Negative Pressures								
	θ							
h/L	10	15	20	25	30	35	40	45
0.25	-0.7	-0.5	-0.3	-0.2	-0.2	0	0	0
0.5	-0.9	-0.7	-0.4	-0.3	-0.2	-0.2	-0.1	0
1	-1.3	-1	-0.7	-0.5	-0.3	-0.2	-0.1	0

	10	10
0.5	-0.9	-0.9
1	-1.3	-1.3

C_p = N/A $\qquad GC_p$ = N/A

Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations parallel to roof ridge

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

* Where Roofs θ≥10° (continued):

Windward Cp Table - Positive Pressures																	
	θ																
h/L	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90
0.25	-0.18	0	0.2	0.3	0.3	0.4	0.4	0.4	0.47	0.53	0.6	0.65	0.7	0.75	0.8	0.8	0.8
0.5	-0.18	-0.18	0	0.2	0.2	0.3	0.35	0.4	0.47	0.53	0.6	0.65	0.7	0.75	0.8	0.8	0.8
1	-0.18	-0.18	-0.18	0	0.2	0.2	0.25	0.3	0.4	0.5	0.6	0.65	0.7	0.75	0.8	0.8	0.8

	10	10
0.5	-0.18	-0.18
1	-0.18	-0.18

C_p = N/A GC_p = N/A

Leeward Cp Table			
	θ		
h/L	10	15	20
0.25	-0.3	-0.5	-0.6
0.5	-0.5	-0.5	-0.6
1	-0.7	-0.6	-0.6

	10	10
0.5	-0.5	-0.5
1	-0.7	-0.7

C_p = N/A GC_p = N/A

7) Design Roof Wind Pressure, p: $p = p_{ext} - p_{int} = qGC_p - q_i(GC_{pi})$ (lb/ft^2)

* Where Roofs θ<10° or Wind applied Parallel to Ridge: p_{int} = 23.4 psf (+/-)

p_{ext} for 0 to h/2:	-37.49 psf	**p =**	**-60.87 psf &**	**p =**	**-14.12 psf**
p_{ext} for h/2 to h:	-30.02 psf	**p =**	**-53.40 psf &**	**p =**	**-6.65 psf**
p_{ext} for h to 2h:	-20.55 psf	**p =**	**-43.93 psf &**	**p =**	**2.82 psf**
p_{ext} for > 2h:	-15.82 psf	**p =**	**-39.19 psf &**	**p =**	**7.56 psf**

* Where Roofs θ≥10°:

Windward Roof Pressures:

p_{ext} = psf & psf

p = N/A psf N/A psf
N/A psf N/A psf

Leeward Roof Pressure:

p_{ext} = psf

p = N/A psf & N/A psf

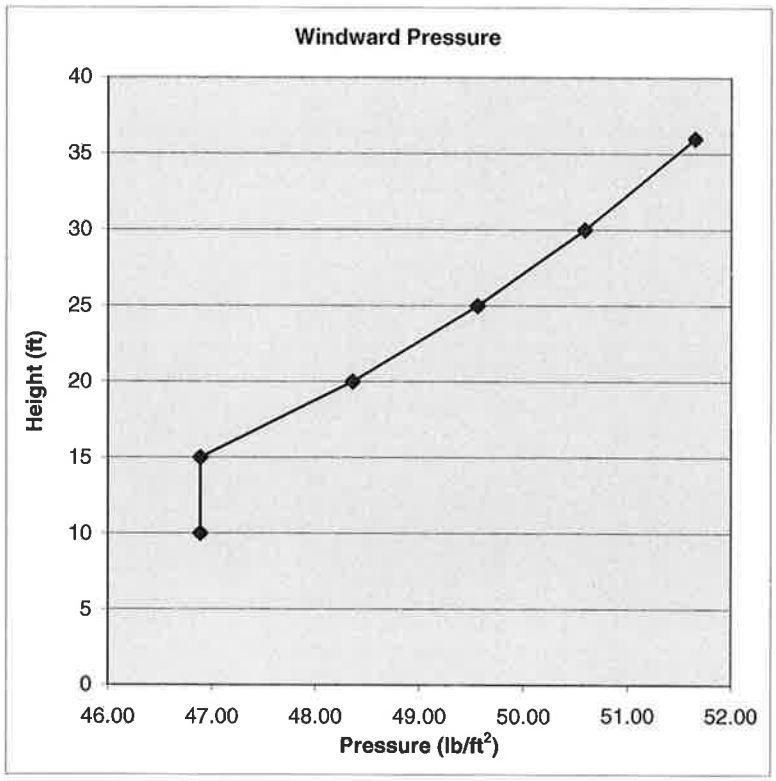
Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations parallel to roof ridge
Results

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

WALL PRESSURES

* Windward Wall Pressures:

Height (z) ft	p lb/ft^2	p lb/ft^2
10	0.14	46.89
15	0.14	46.89
20	1.61	48.36
25	2.81	49.56
30	3.83	50.58
36	4.90	51.65
40	5.53	52.28
50	6.92	53.67



* Leeward Wall Pressures:

p = -34.33 lb/ft^2
p = 12.42 lb/ft^2

* Side Wall Pressures:

p = -48.66 lb/ft^2
p = -1.91 lb/ft^2

Note: (+) pressures act towards surface, (-) pressures act away from surface

Job: Maurepas Pump Station
Project No: 10001663
Description: Design Wind Load Calculations parallel to roof ridge
Results

Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

ROOF PRESSURES

* Pressures for Roofs with θ<10°:

p for 0ft to 20ft:	p =	-60.87	psf &	p =	-14.12	psf
p for 20ft to 40ft:	p =	-53.40	psf &	p =	-6.65	psf
p for 40ft to 80ft:	p =	-43.93	psf &	p =	2.82	psf
p for > 80ft:	p =	-39.19	psf &	p =	7.56	psf

* Pressures for Roofs with θ≥10°:

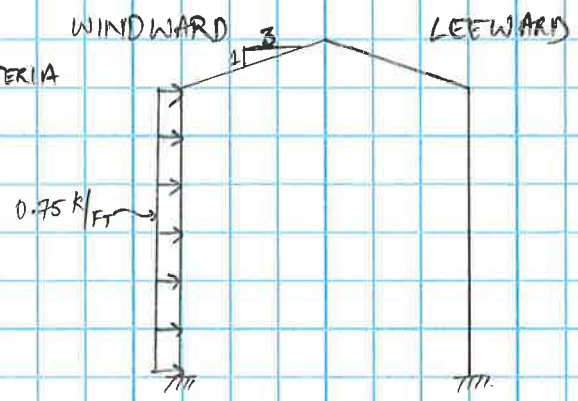
Windward Roof Pressures:

p =	N/A	psf	N/A	psf
	N/A	psf	N/A	psf

Leeward Roof Pressure:

p =	N/A	psf	&	N/A	psf

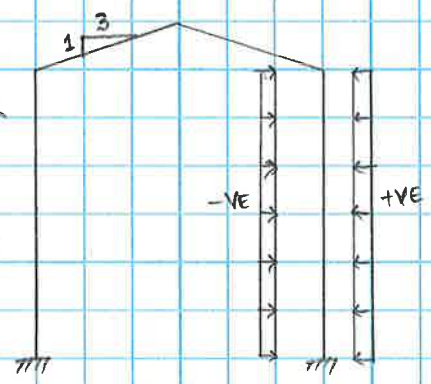
WIND WARD WALL:



DESIGN CRITERIA

Windward_Wall = 50 psf × 15'
= 750 lb/ft

LEEWARD WALL:



FROM THE WIND CALCULATIONS PERPENDICULAR TO RIDGE

Lee Wall_pos = 5.3 psf × 15' = 79.5 lb/ft

Lee Wall_neg = 41.4 psf × 15' = 621 lb/ft

WALL DEAD LOAD:

Wall_Dead = 3 psf × 15' = 45 lb/ft
ASCE 7-05

CRANE LOADS: (SVEDA 15 T CRANES - REFER WWW.SVEDA-CRANES.COM)

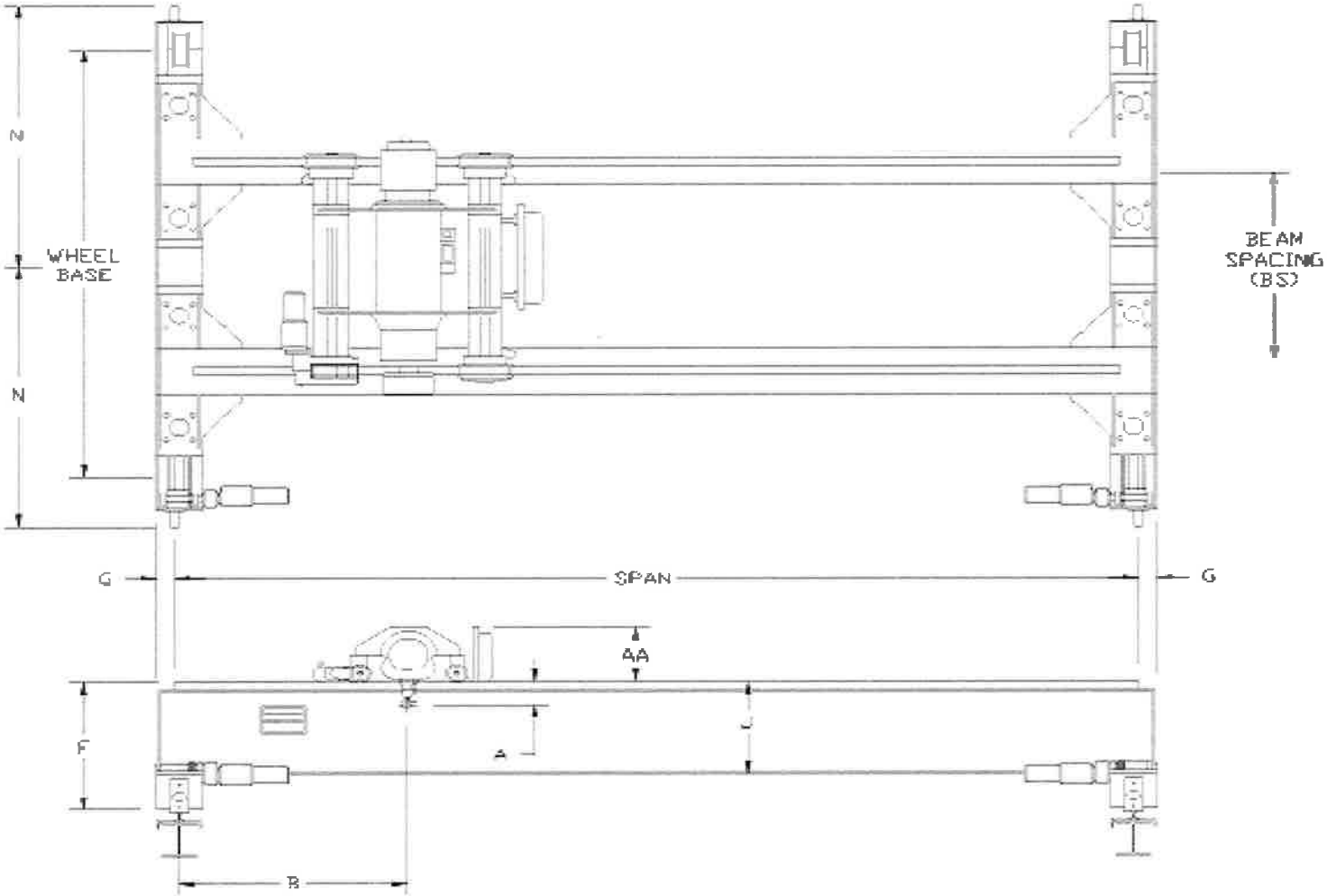
MAX. WHEEL LOAD OF A '15 T' CAPACITY CRANE WITH 48' MAX. SPACING
= 23,840 lbs ≃ 25 K

∴ VERTICAL CRANE LOAD REACTIONS AT BOTH ENDS OF THE FRAME = 25 K
WITH 8" ECCENTRICITY BENDING MOMENT DUE TO THIS LOAD = $25^k \times (8/12)'$
= 16.67 K-FT

LATERAL FORCE AT EACH = $20/100 \times (15 \times 2)^k/2 = 3^k$ [REFER IBC]

ROOF: Roof_Live = 60 psf × 15' = 900 lb/ft - (FROM DESIGN CRITERIA)
Roof_Dead = 3 psf × 15' = 45 lb/ft - (FROM ASCE 7-05)

SVEDA CRANE DATA

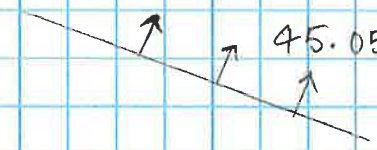


15 TONS									
Capacity in Tons	Max. Span ft.	Beam Spacing in. (BS)	J in.	F in.	HP for fpm 80	140	180	Weight lbs.	Wheel Loads lbs.
15	48	60 78	31-1/4	31-1/4	1	2	3	17,670	23,840
	52	60 78	37-1/8	31-1/4	1	2	3	18,830	24,140
	56	60 78	37-1/8	31-1/4	1	2	3	20,160	24,470
	60	60 78	43-1/8	31-1/4	1	2	3	21,880	24,900
	64	60 78	43-1/8	31-1/4	1	2	3	22,860	25,140
	68	60 78	48-1/8	36-1/4	1-1/2	2	3	25,010	25,680
	72	60 78	48-1/8	36-1/4	1-1/2	3	3	28,240	26,490
	76	60 78	48-1/4	36-1/4	1-1/2	3	3	30,060	26,940
	80	60 78	54-1/8	36-1/4	1-1/2	3	3	32,220	27,480
	84	60 78	54-1/4	36-1/4	1-1/2	3	4	35,580	28,320
	88	60 78	54-1/4	36-1/4	1-1/2	3	4	39,120	29,210
	92	60 78	54-3/8	36-1/4	1-1/2	3	4	42,390	30,020
	96	60 78	60-1/2	42-1/2	1-1/2	3	4	45,880	30,900
	100	60 78	60-1/2	42-1/2	1-1/2	3	4	47,470	31,240

Wheel Base	Beam Spacing (BS)	Wheel Tread Dia.	G	N
13'-0"	60	18	7-3/8	8'-0"
13'-0"	78	18	7-3/8	8'-0"
13'-0"	60	18	7-3/8	8'-0"
13'-0"	60	18	7-3/8	8'-0"
13'-0"	78	18	7-3/8	8'-0"
16'-0"	60	18	7-3/8	9'-6"
16'-0"	78	18	7-3/8	9'-6"
16'-0"	96	18	7-3/8	9'-6"
16'-0"	60	18	7-3/8	9'-6"
16'-0"	78	18	7-3/8	9'-6"
16'-0"	96	18	7-3/8	9'-6"
16'-0"	60	18	7-3/8	9'-7-1/2"
16'-0"	78	18	7-3/8	9'-7-1/2"
16'-0"	60	18	7-3/8	9'-7-1/2"
16'-0"	78	18	7-3/8	9'-7-1/2"

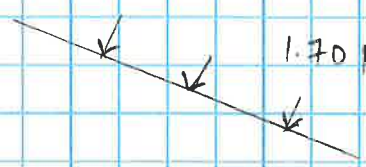
WIND LOADS ON ROOF

Lee Roof_neg:



45.05 psf x 15' = 675.8 lb/ft

Lee Roof_pos:



1.70 psf x 15' = 25.5 lb/ft

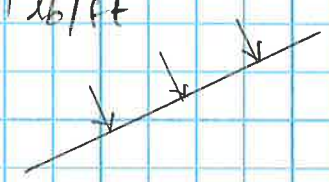
Wind Roof_pos1:

29.88 psf x 15' = 448.2 lb/ft

(NOTE: THE DIRECTION OF THE ARROWS REPRESENT NET EFFECT DUE TO WIND FLOW)

Wind Roof_pos2:

16.87 psf x 15' = 253.1 lb/ft

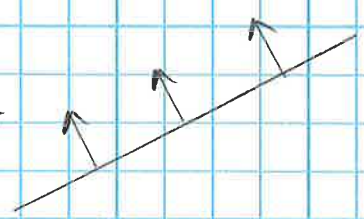


Wind Roof_neg1:

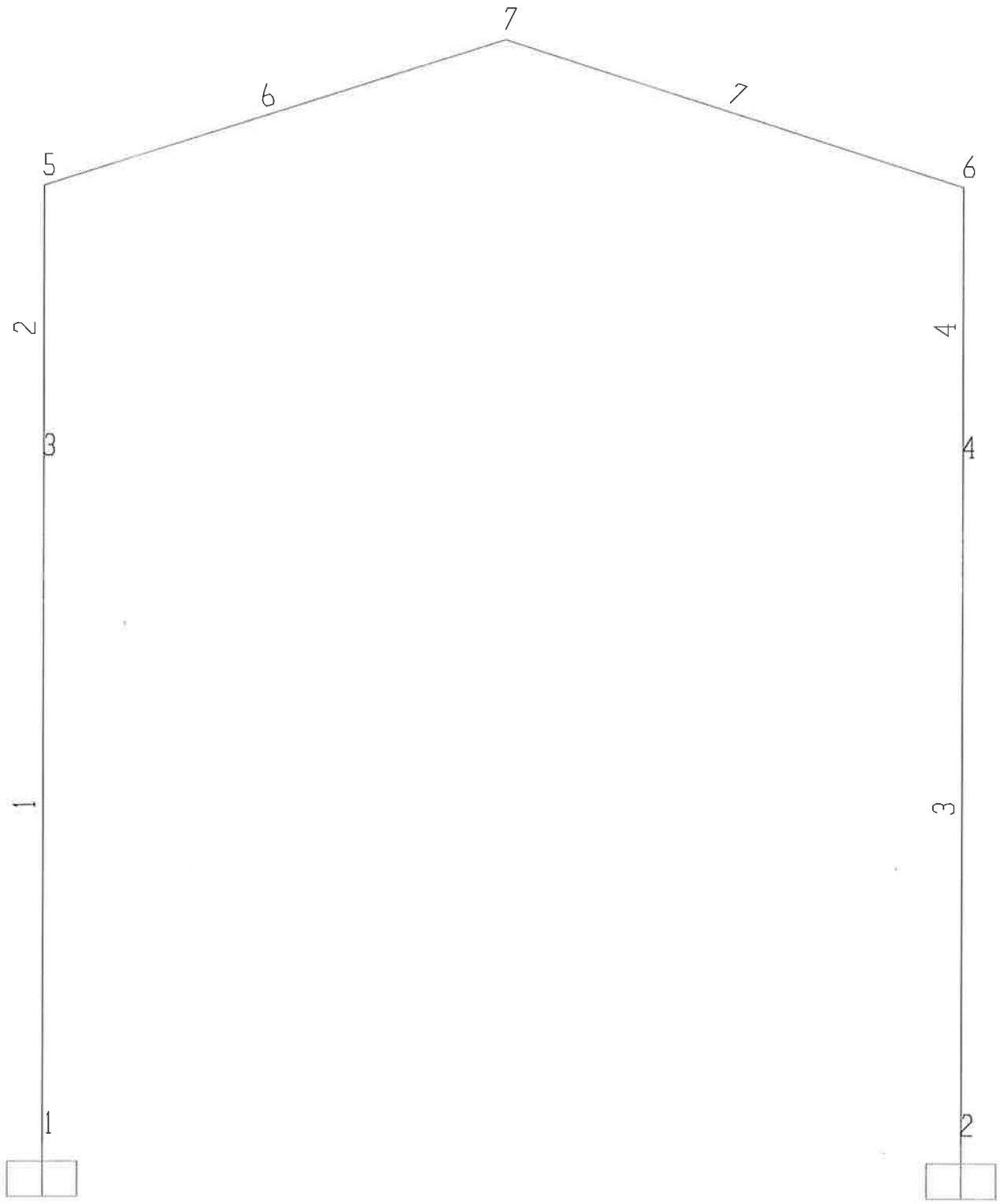
51.94 psf x 15' = 779.1 lb/ft

Wind Roof_neg2:

5.19 psf x 15' = 77.9 lb/ft



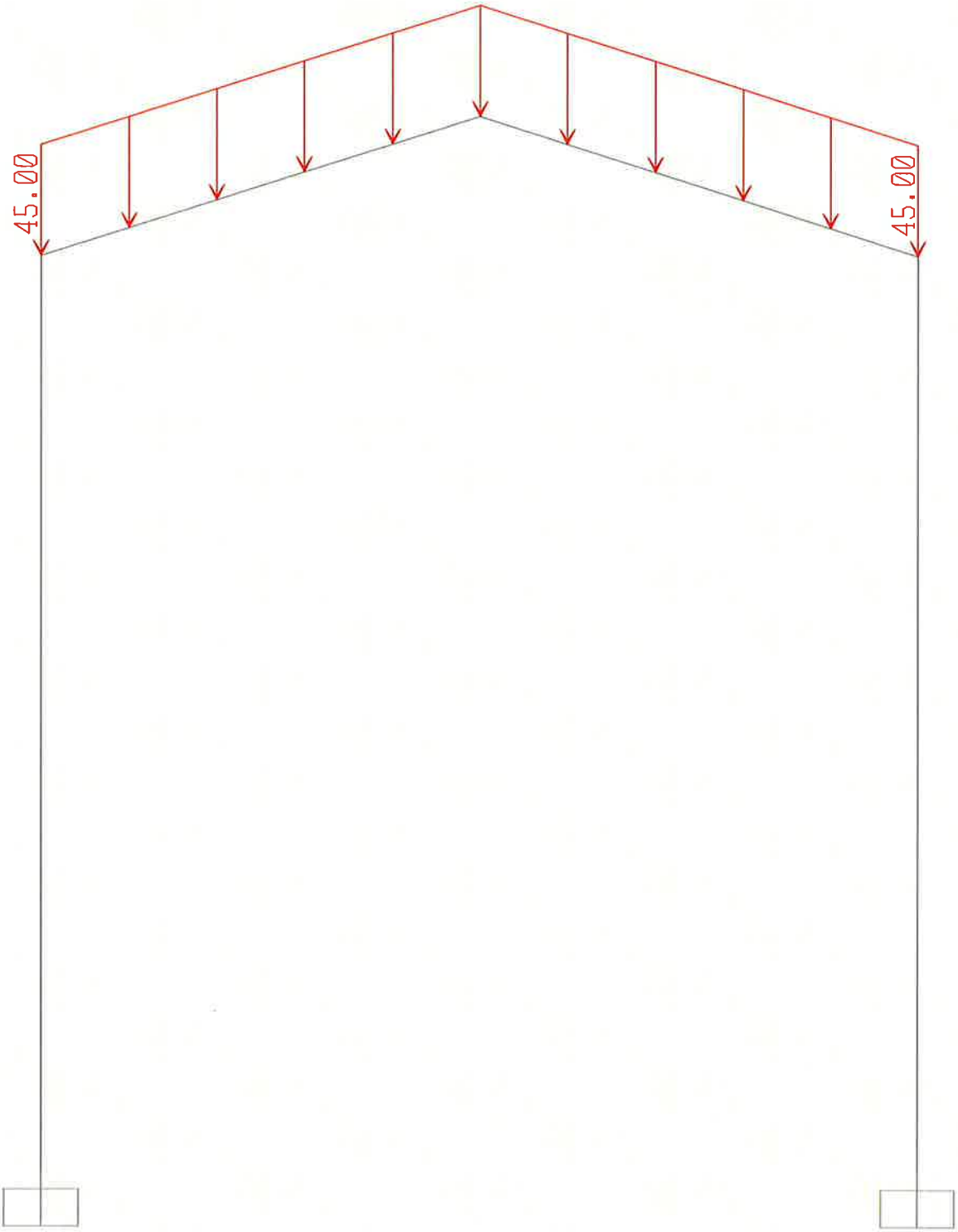
JT 08/10 (22)
EV 12/10
8/23/10 14:30:40



— FRAME ON PINNED CONNECTIONS.

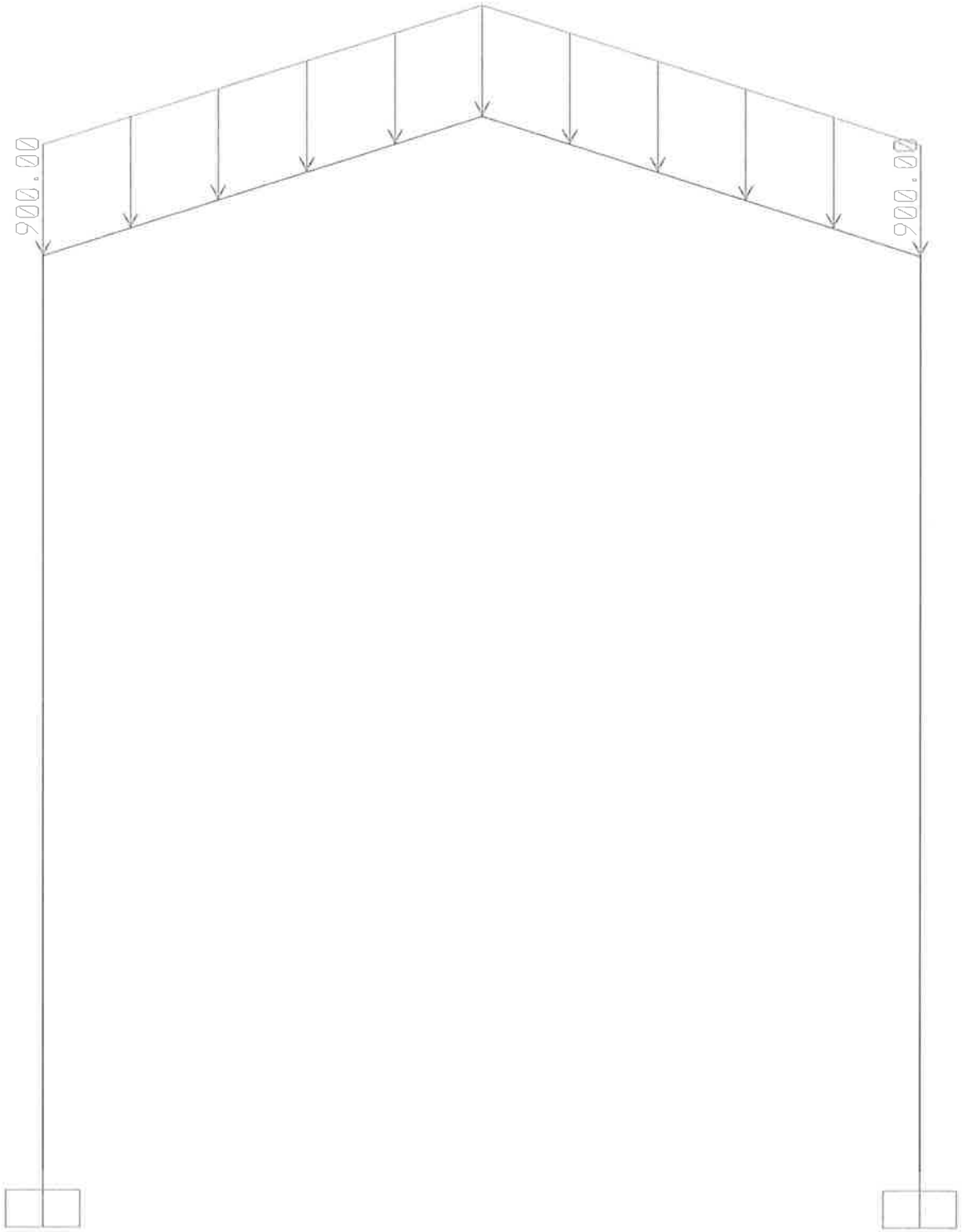
JY
EY 12/10 (23)

8/24/10 10:16:50

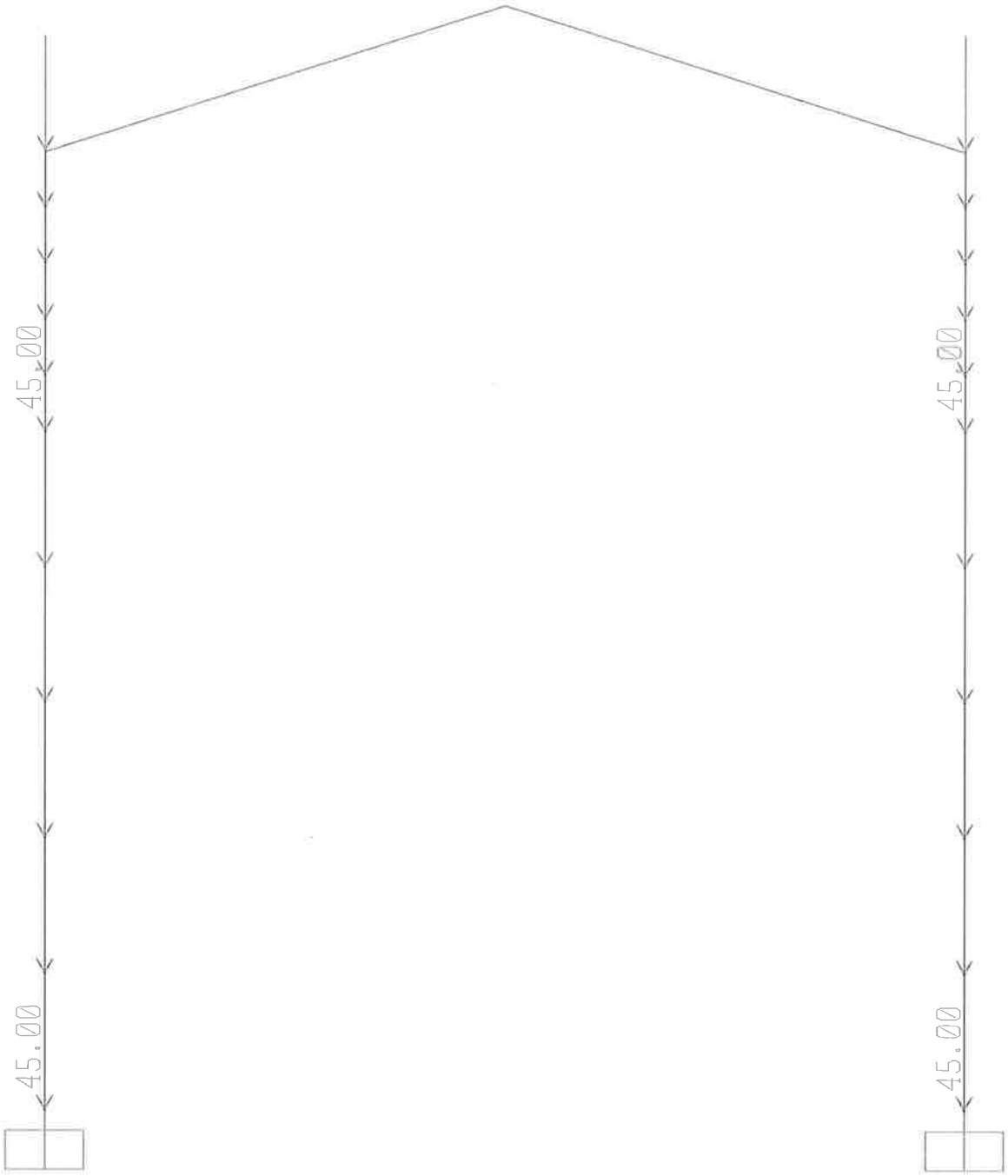


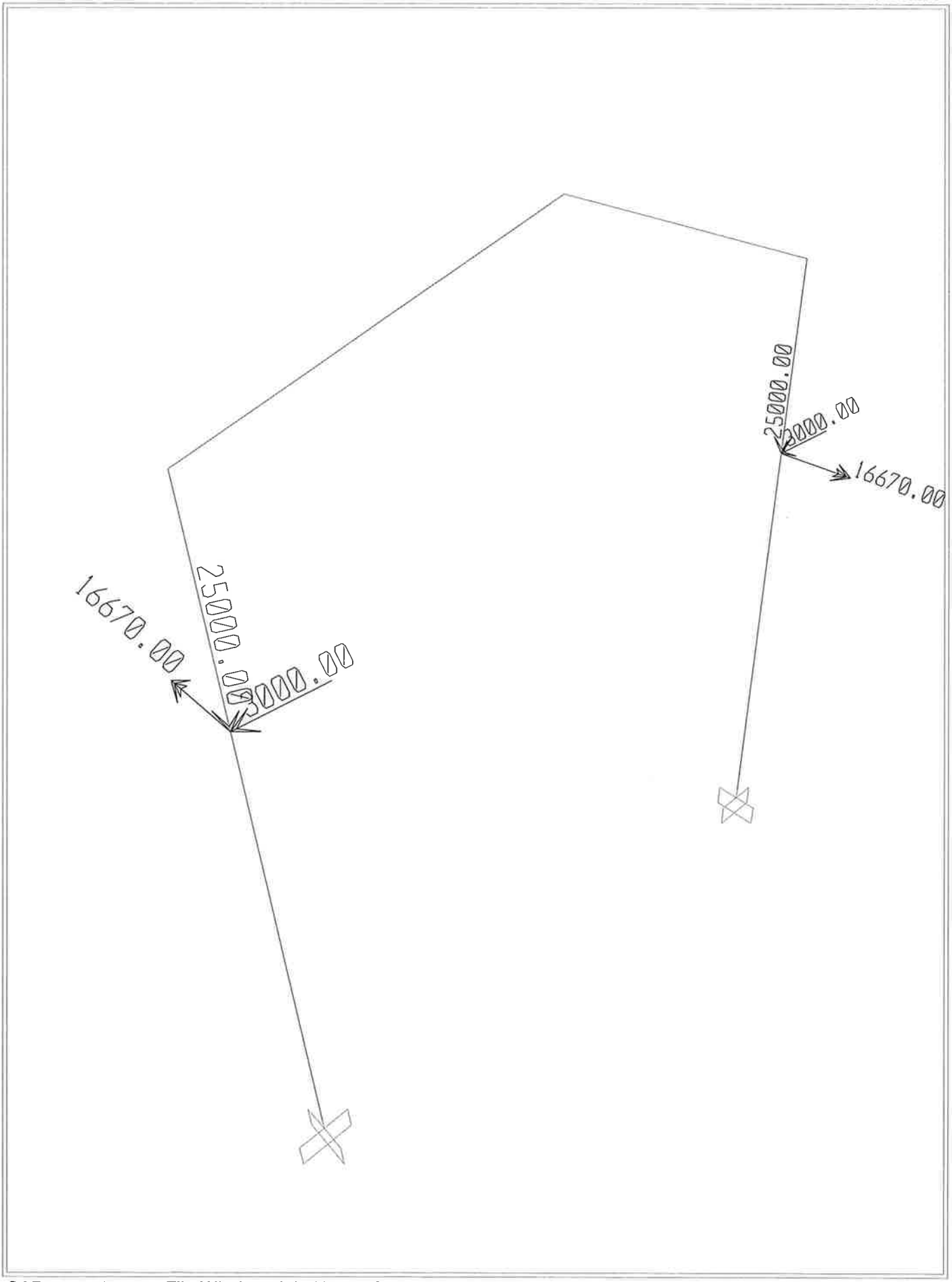
JY 12/10 (24)

8/24/10 10:18:56



JY (25)
EY 12/10
8/24/10 10:19:20




25000.00
3000.00
16670.00
25000.00
3000.00
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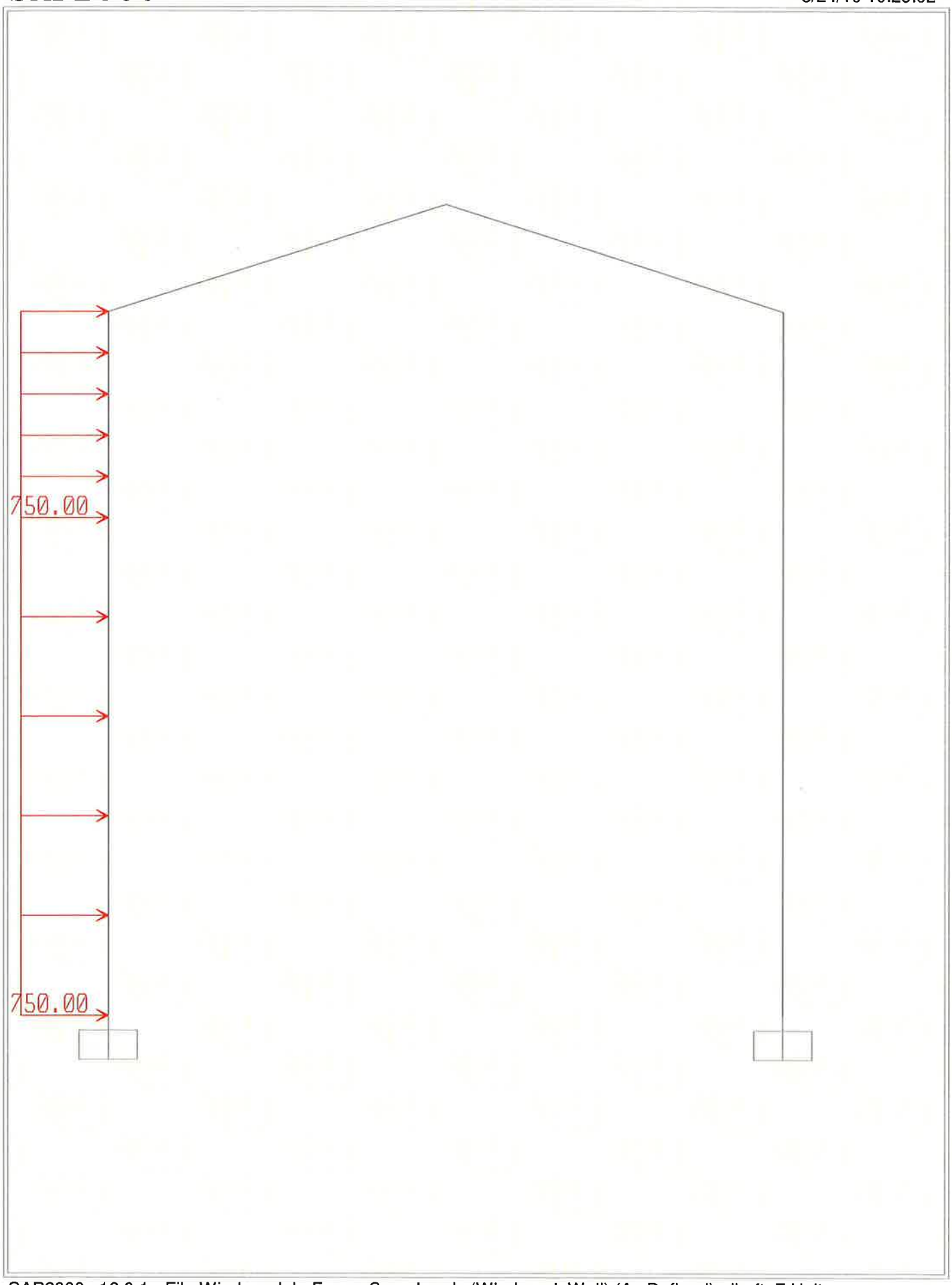
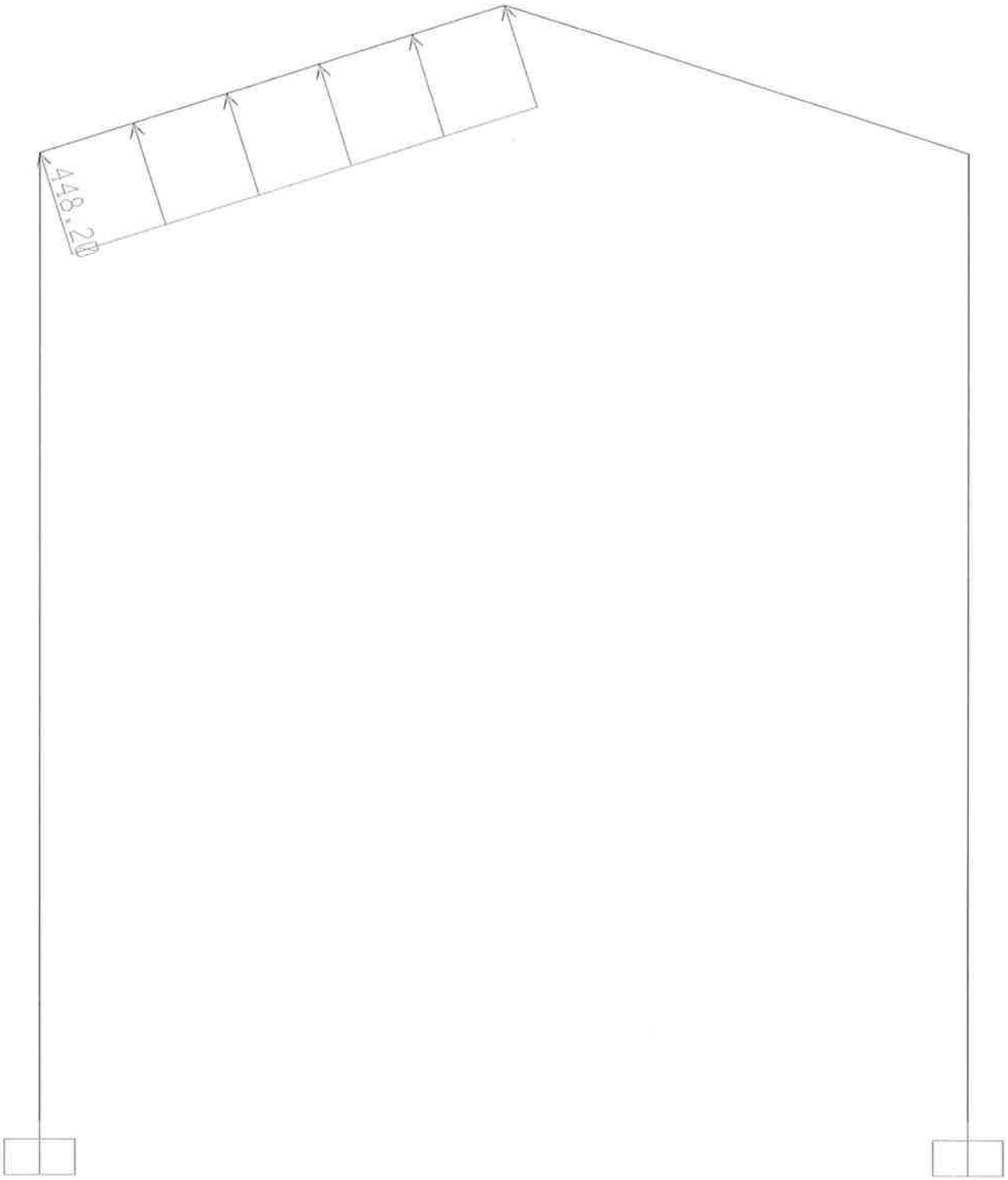
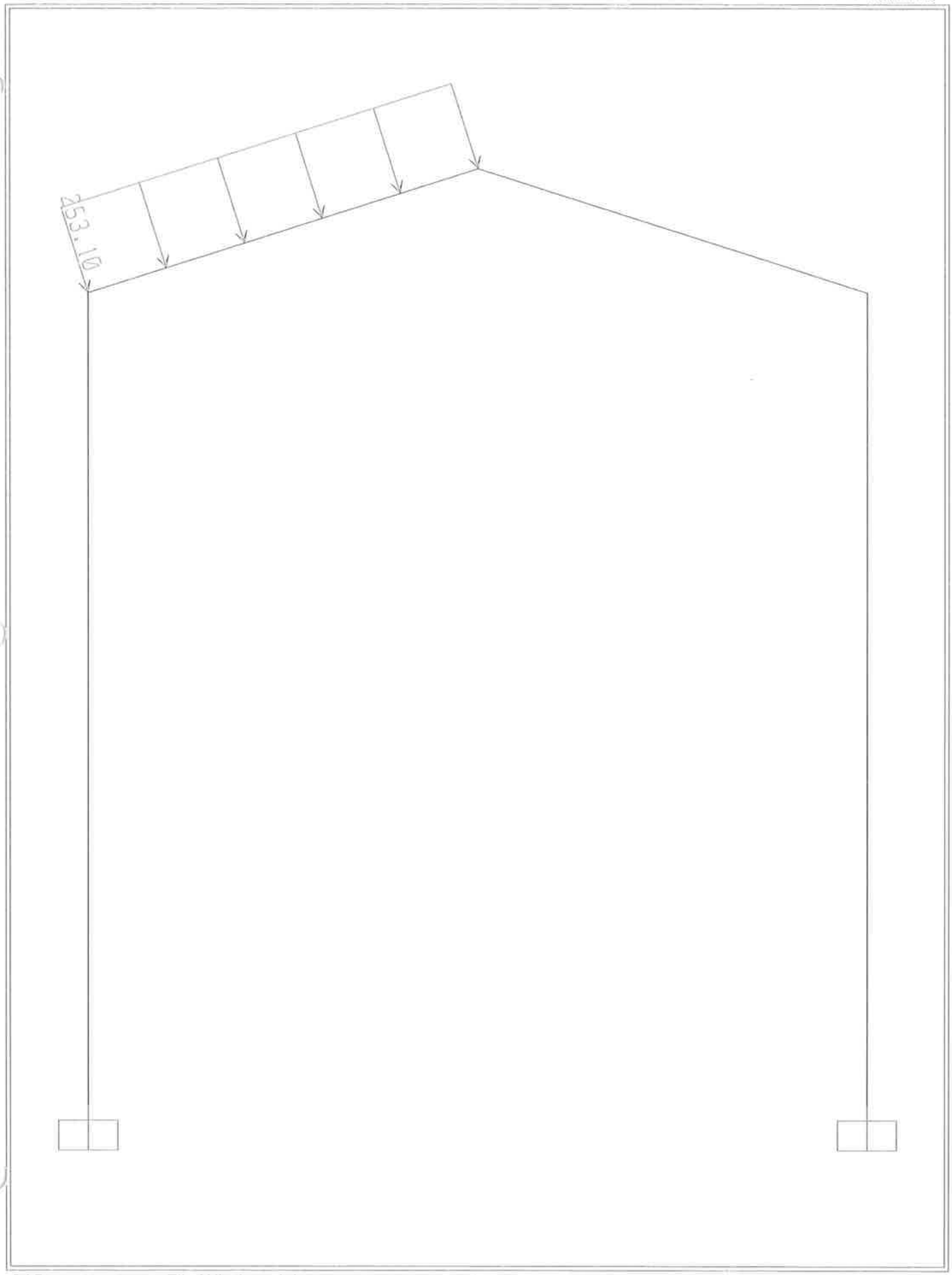
JY (24)
EY 12/10
8/24/10 10:29:02



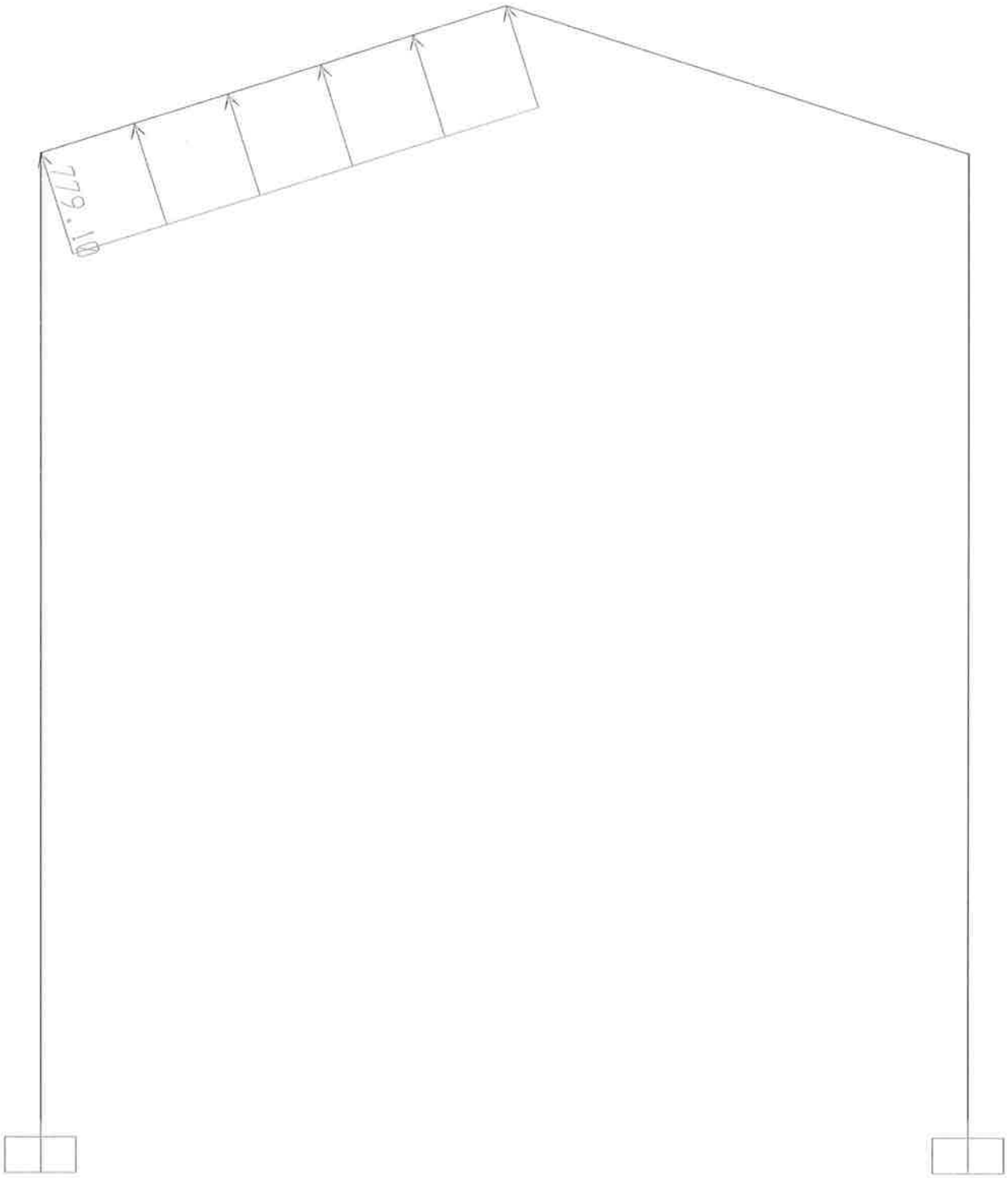
JY (28)
EY 12/10
8/24/10 10:21:49



JY (29)
EY 12/10
8/24/10 10:22:16

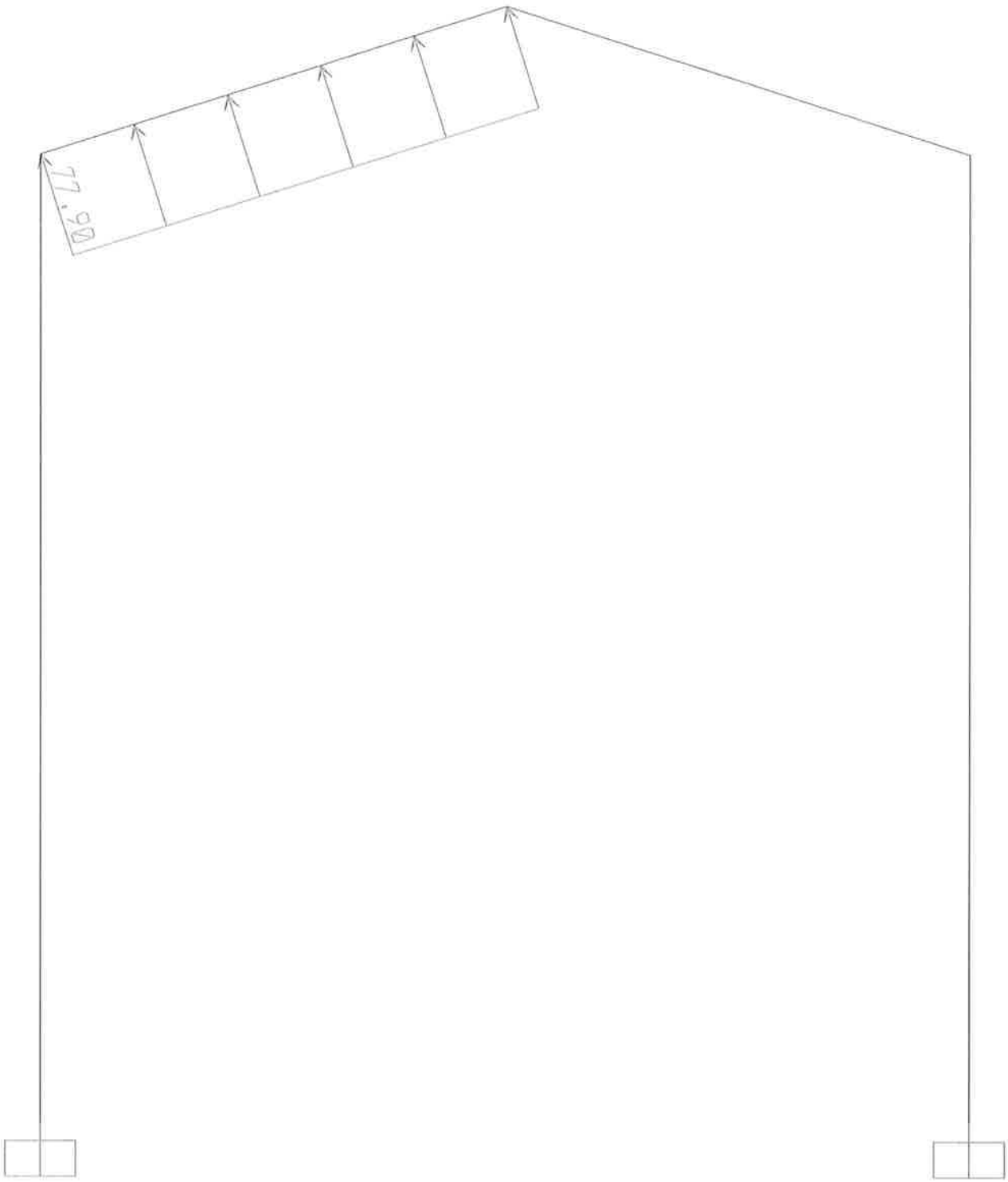


JY (30)
EY 12/10
8/24/10 10:22:29



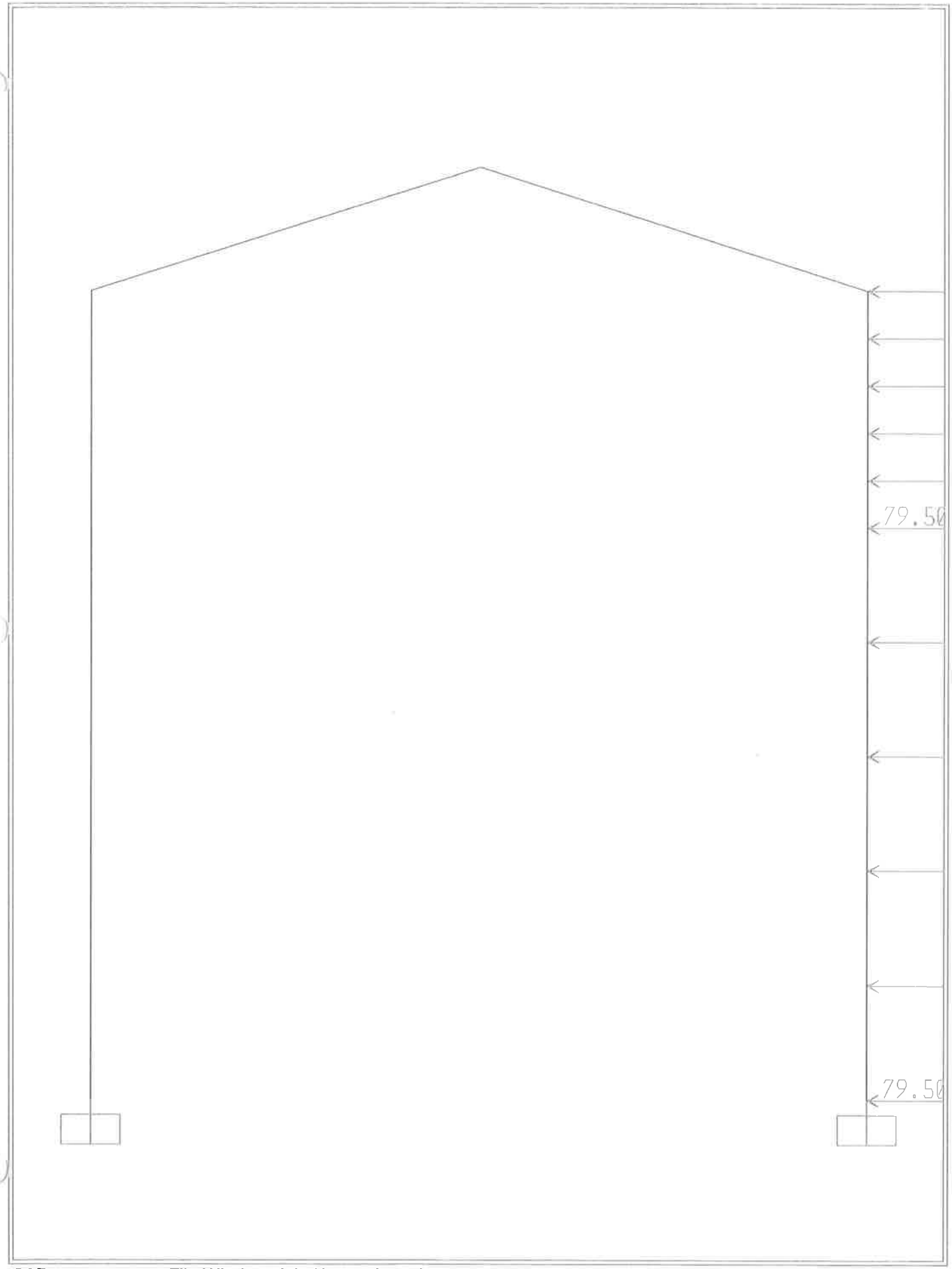
JY (81)
FY 12/10

8/24/10 10:22:53



JY (32)
EY 12/10

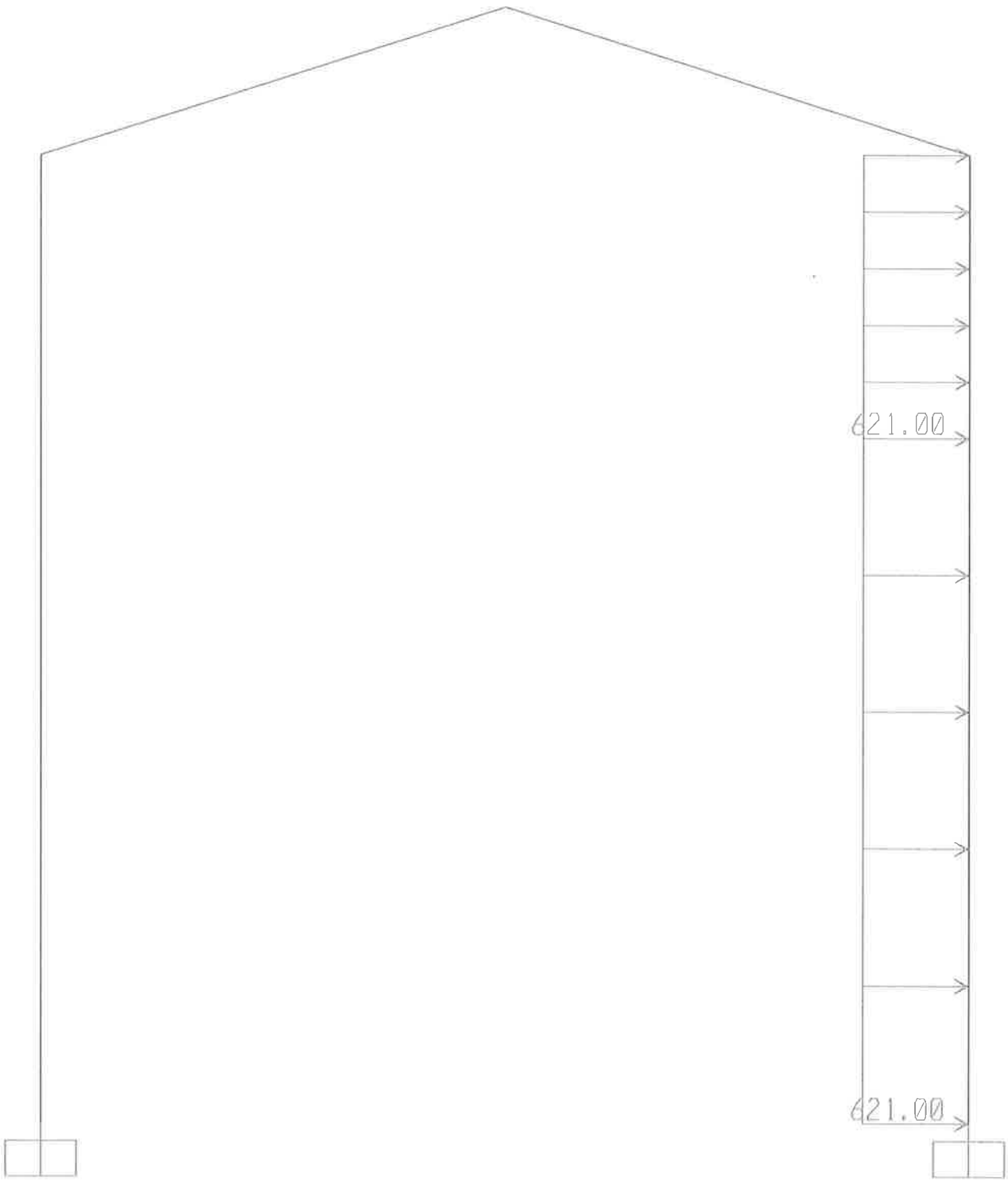
8/24/10 10:20:38



SAP2000 v12.0.1 - File:Wind model - Frame Span Loads (LeeWall_pos) (As Defined) - lb, ft, F Units

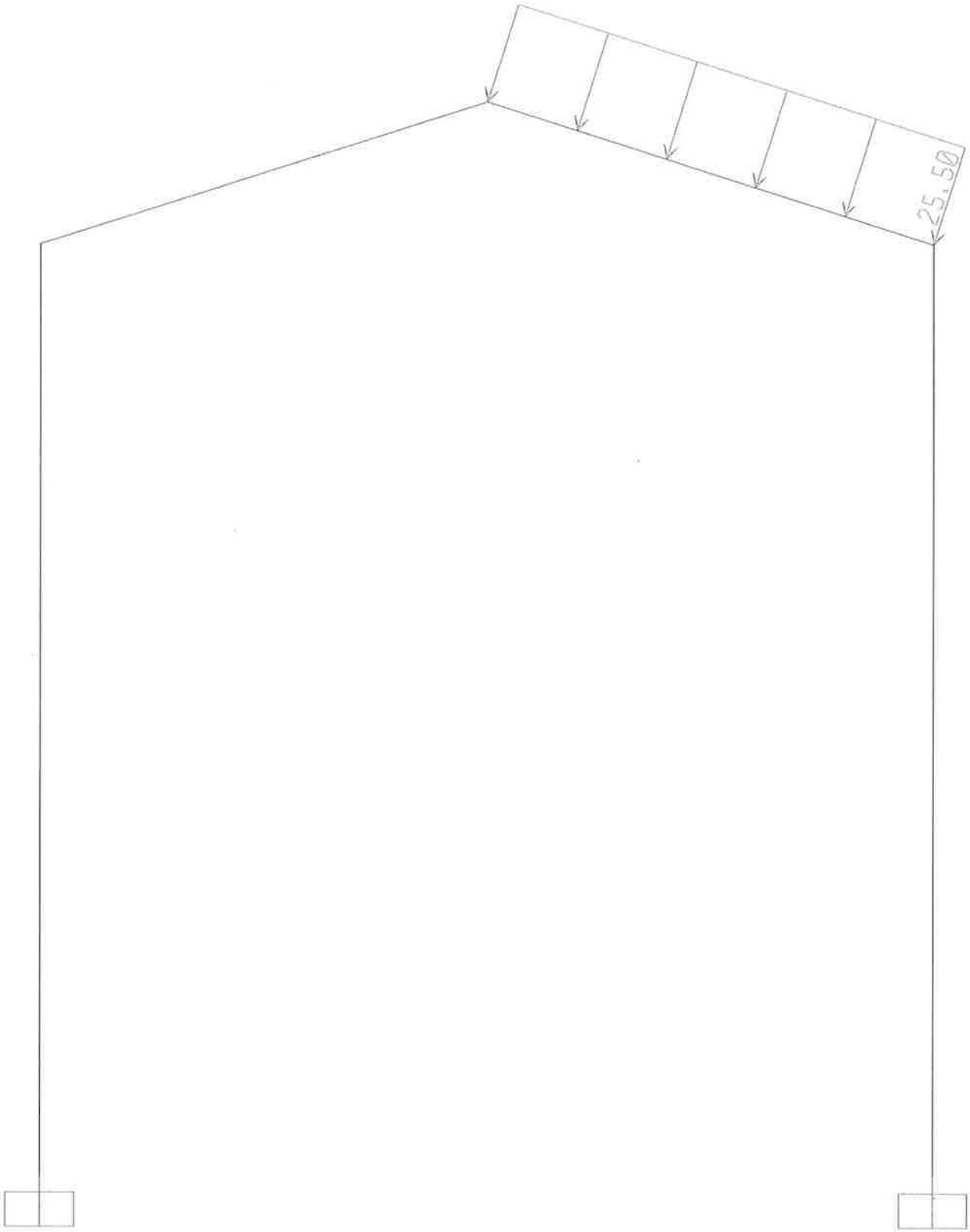
JY (33)
EY 12/10

8/24/10 10:20:52

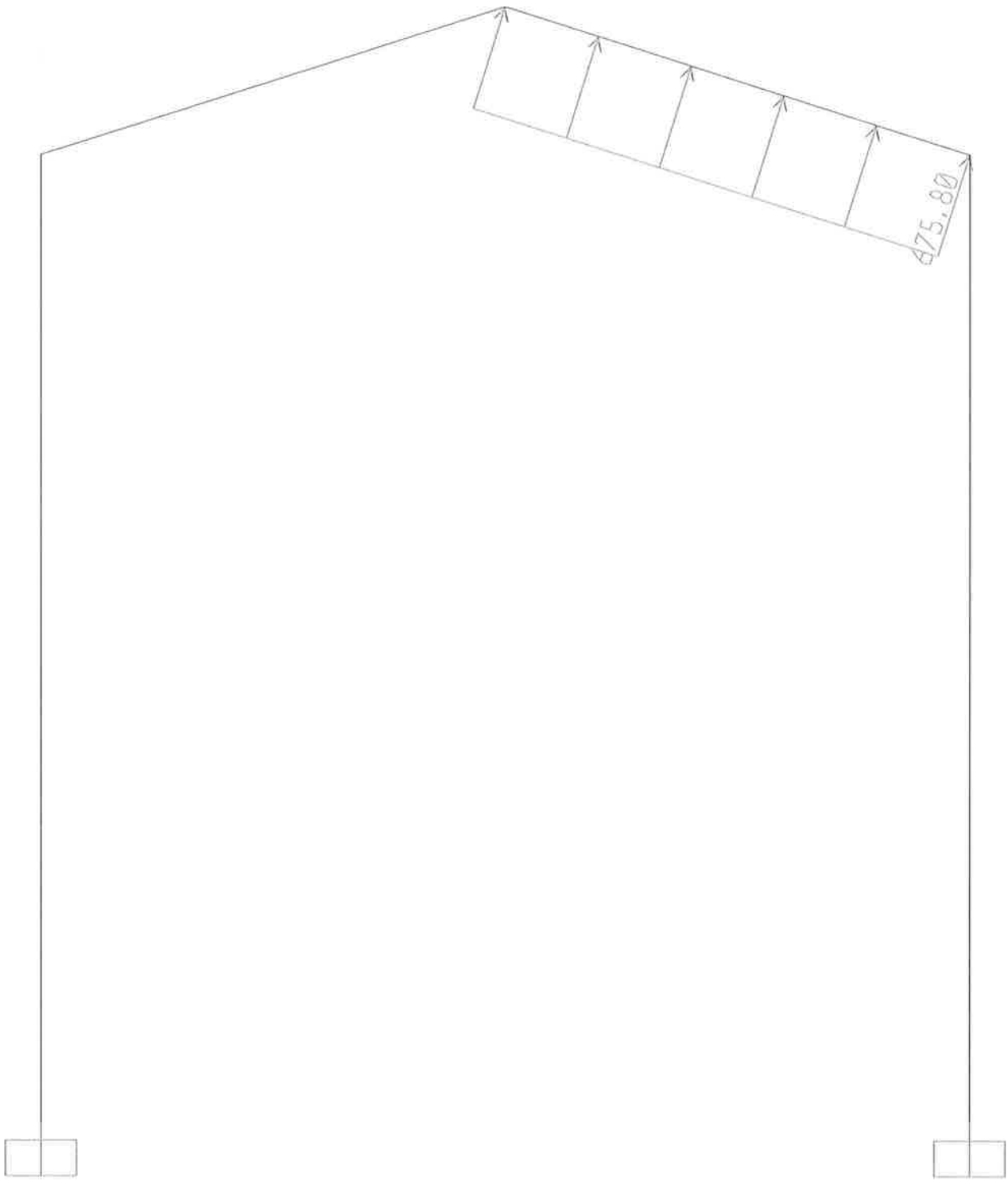


JY (34)
EY 12/10

8/24/10 10:21:10



JY (35)
EY 12/10
8/24/10 10:21:37



Job: Maurepas Pump Station
Description: Load Cases for
2-D Wind Design SAP model

Project No.: 10001663
Computed By: JY
Date: 08/2010
Checked By: CI
Date: 12/10

All combinations have the following loads:

- Roof_Live
- Roof_Dead
- Wall_Dead
- Windward_Wall
- Crane_Reactions
- DEAD

LC1:

- LeeWall_pos
- LeeRoof_neg
- WindRoof_pos1

LC2:

- LeeWall_pos
- LeeRoof_neg
- WindRoof_pos2

LC3:

- LeeWall_pos
- LeeRoof_neg
- WindRoof_neg1

LC4:

- LeeWall_pos
- LeeRoof_neg
- WindRoof_neg2

LC5:

- LeeWall_pos
- LeeRoof_pos
- WindRoof_pos1

LC6:

- LeeWall_pos
- LeeRoof_pos
- WindRoof_pos2

LC7:

- LeeWall_pos
- LeeRoof_pos
- WindRoof_neg1

LC8:

- LeeWall_pos
- LeeRoof_pos
- WindRoof_neg2

LC9:

- LeeWall_neg
- LeeRoof_neg
- WindRoof_pos1

LC10:

- LeeWall_neg
- LeeRoof_neg
- WindRoof_pos2

LC11:

- LeeWall_neg
- LeeRoof_neg
- WindRoof_neg1

LC12:

- LeeWall_neg
- LeeRoof_neg
- WindRoof_neg2

LC13:

- LeeWall_neg
- LeeRoof_pos
- WindRoof_pos1

LC14:

- LeeWall_neg
- LeeRoof_pos
- WindRoof_pos2

LC15:

- LeeWall_neg
- LeeRoof_pos
- WindRoof_neg1

LC16:

- LeeWall_neg
- LeeRoof_pos
- WindRoof_neg2

Job: Maurepas Pump Station
Description: Reactions from 2-D Wind Model

Project No.: 10001663
Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/2010

Max Reactions						
	Joint	F1 Kip	F3 Kip	Joint	F1 Kip	F3 Kip
Max Fx & Fz:	1	-19.6	39.7	2	-21.7	64.8

Reactions

Joint	OutputCase	F1 Kip	F3 Kip
1	LC1	-13.2	29.1
1	LC2	-14.2	33.3
1	LC3	-12.7	27.2
1	LC4	-13.7	31.3
1	LC5	-10.9	35.5
1	LC6	-11.9	39.7
1	LC7	-10.4	33.5
1	LC8	-11.4	37.7
1	LC9	-18.5	17.7
1	LC10	-19.6	21.9
1	LC11	-18.1	15.8
1	LC12	-19.1	19.9
1	LC13	-16.2	24.1
1	LC14	-17.3	28.2
1	LC15	-15.8	22.1
1	LC16	-16.8	26.3
2	LC1	-2.8	42.8
2	LC2	-5.1	49.2
2	LC3	-1.7	39.8
2	LC4	-4.1	46.2
2	LC5	-1.8	47.0
2	LC6	-4.1	53.4
2	LC7	-0.7	44.0
2	LC8	-3.0	50.3
2	LC9	-19.4	54.2
2	LC10	-21.7	60.6
2	LC11	-18.3	51.2
2	LC12	-20.6	57.6
2	LC13	-18.4	58.4
2	LC14	-20.7	64.8
2	LC15	-17.3	55.4
2	LC16	-19.6	61.8

Job: Maurepas Pump Station
Description: Reactions from 2-D Wind model

Project No.: 10001663
Computed By: JY
Date: 08/2010
Checked By: EY
Date: 12/10

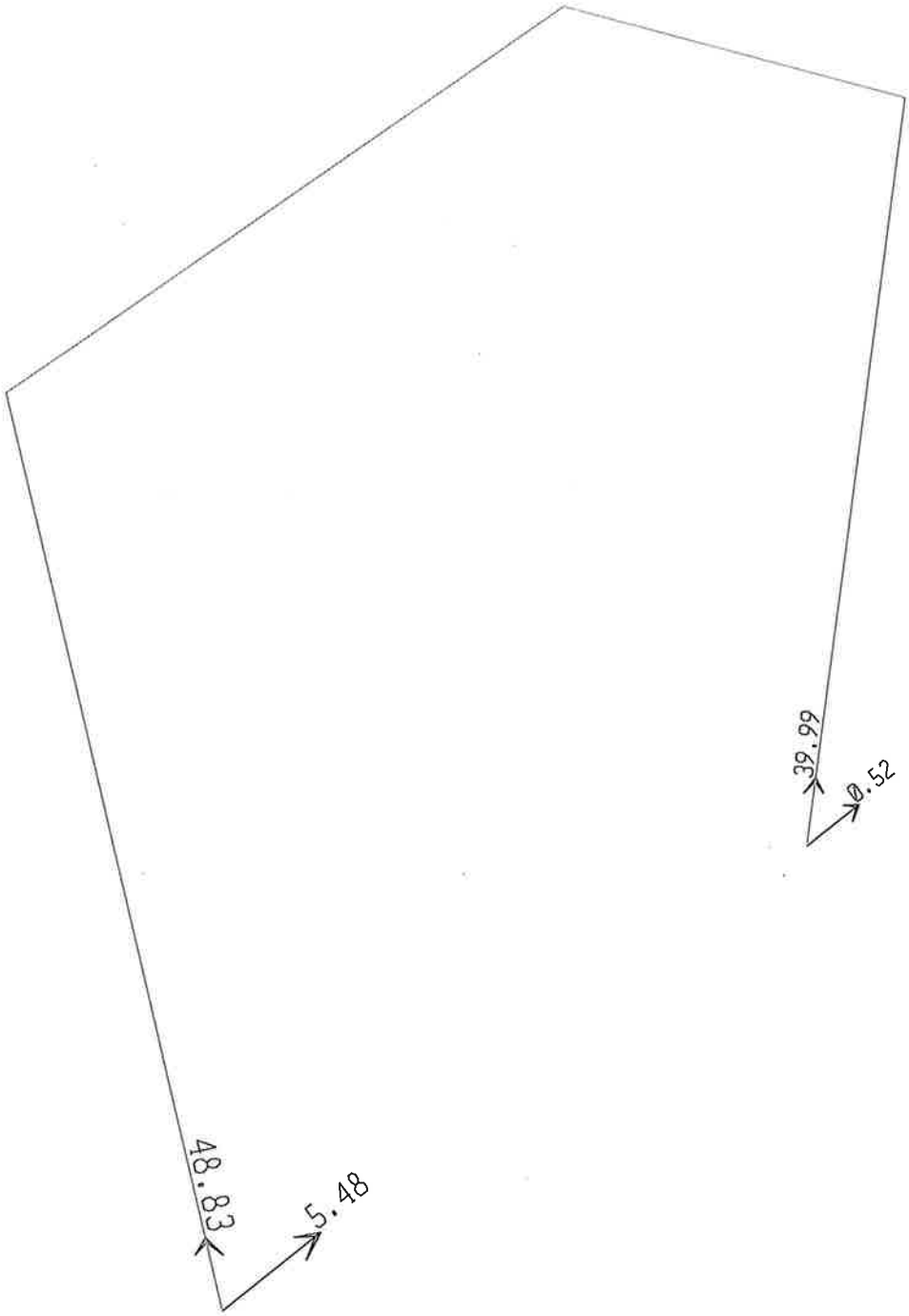
Reactions

Joint	OutputCase	F1	F3	Joint	F1	F3
		Kip	Kip		Kip	Kip
1	DEAD	0.1	3.1	2	-0.1	3.1
1	Roof_Live	1.8	14.2	2	-1.8	14.2
1	Roof_Dead	0.1	0.7	2	-0.1	0.7
1	Wall_Dead	0.0	1.4	2	0.0	1.4
1	Crane_Reactions	3.5	29.4	2	2.5	20.6
SUM:		**5.5**	**48.8**		**0.5**	**40.0**

* The sum of these reactions will be used in the 3-D model when wind is not prese

SAP2000

1/27/11 9:25:35



SAP2000 v14.2.3 - File:Wind model - Joint Reactions (DEAD for 3D) - Kip, in, F Units

SECTION 3

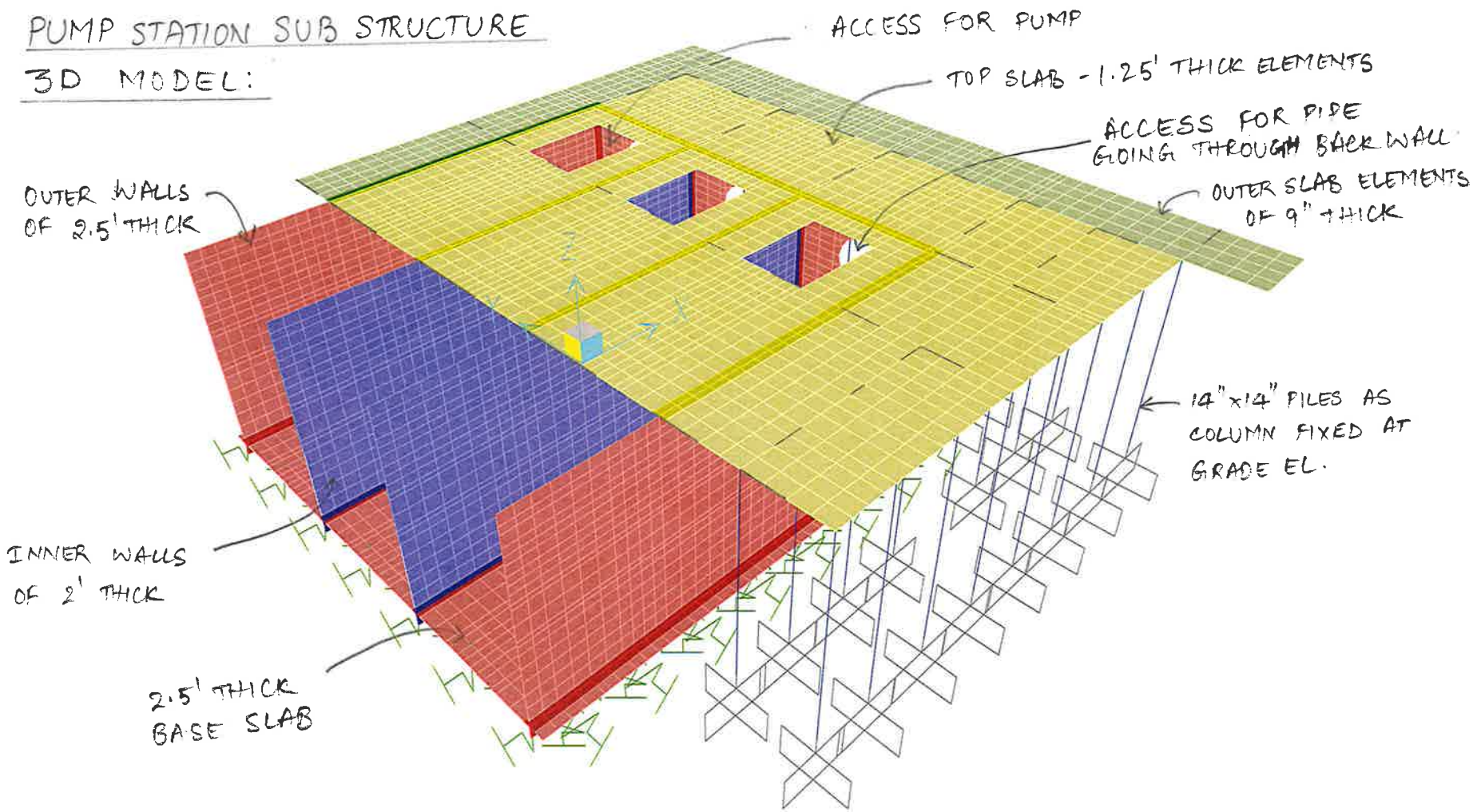
Substructure SAP Model Overview & Load Input

3.1

SAP Model Overview

PUMP STATION SUB STRUCTURE

3D MODEL:

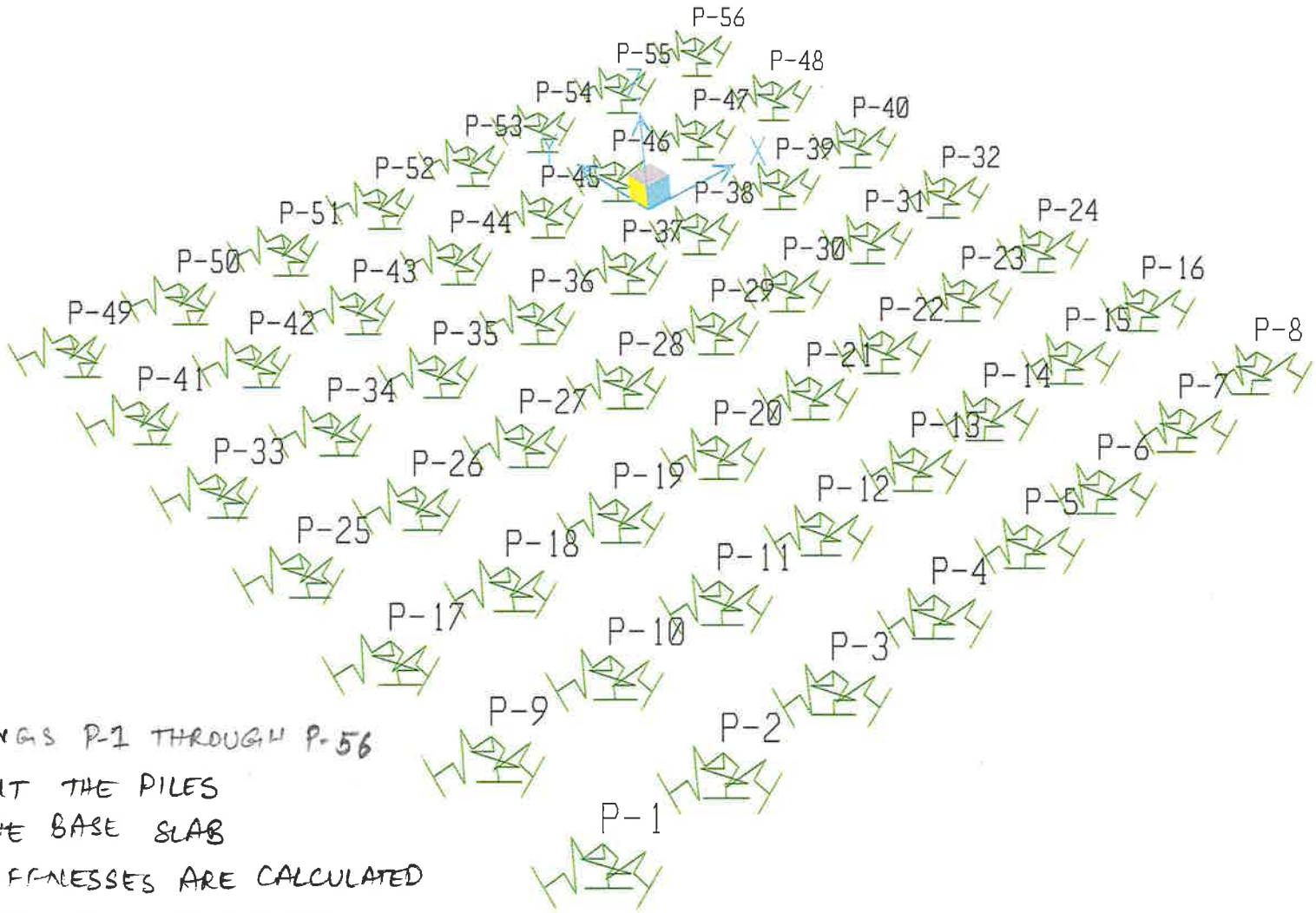


- ALL THE SLABS & WALLS ARE DIVIDED IN TO APPROX. 1'x1' ELEMENTS FOR 3-D FINITE ELEMENT ANALYSIS.
- THE COORDINATES OF THE MODEL REPRESENT THE TRUE ELEVATIONS OF THE ACTUAL STRUCTURE.

AY 9/21/10

42

PILES UNDER BASE SLAB AS SPRINGS!



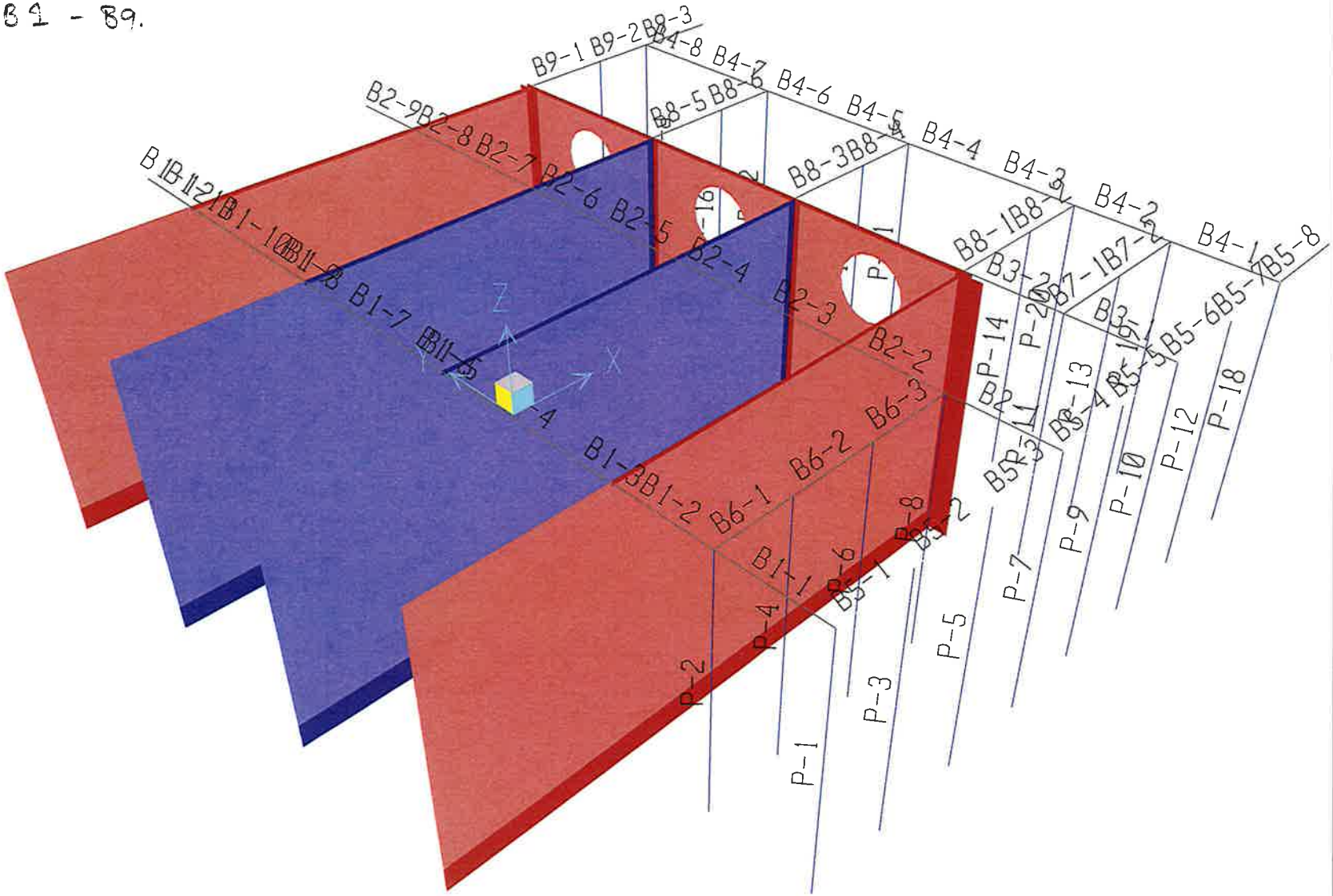
- THE SPRINGS P-1 THROUGH P-56 REPRESENT THE PILES UNDER THE BASE SLAB
- PILE STIFFNESSES ARE CALCULATED FROM THE SOIL PROPERTIES.

JY
BY 12/10

43

BEAM LAYOUT :

- FRAMES B1 THRU B9 REPRESENT BEAMS B1 - B9.



JP 9/21/10 44

3.2

Load Input

Job: Maurepas Pump Station
Job No.: 10001663
Description: Description of Load Cases

Calculated By: JY
Date: 08/2010
Checked By: EY
Date: 12/2010

DEAD LOADS:

D :	Dead load of structure
D-E_pump_gear:	Dead loads of the pumps and gear drives applied around the perimeter of the pump opening as a line load.
D-E_FSI:	Dead loads from formed suction intakes applied as area loads.
D-E_engine:	Dead loads from engines applied as area loads.

LIVE LOADS:

L_floor:	250psf live area load applied to operating floor.
LS_200:	200psf live surcharge area load applied to all floors for construction cases.
LS_300:	300psf live soil surcharge area load applied to abutment and wing walls for construction cases.
L_truck_m:	Point loads that correspond to an AASHTO HS-20 truck positioned above the centers of the intake bays. This location maximizes the moments that result from the truck's weight.
L_truck_s:	Point loads that correspond to an AASHTO HS-20 truck positioned slightly to the side of the intake bay walls. This location maximizes the shears that result from the truck's weight.
L_rack:	Line loads applied to top and bottom slab at the intake openings due to a blocked trash rack.

SOIL LOADS:

S_3.5:	Lateral soil pressures pushing inward for soil to EL 4.0 and water table at Max. EL 3.5 applied as surface pressures. (Uses joint patterns Soil_3.5_a and Soil_3.5_b)
S_0.5:	Lateral soil pressures pushing inward for soil to EL 4.0 and water table at Normal EL 0.5 applied as surface pressures. (Uses joint patterns Soil_0.5_a and Soil_0.5_b)
S_0.0:	Lateral soil pressures pushing inward for soil to EL 4.0 and water table at Min EL 0.0 applied as surface pressures. (Uses joint patterns Soil_0.0_a and Soil_0.0_b)

HYDRAULIC LOADS:

H_0.0_exterior:	Lateral water pressure pushing inward for water table at EL 0.0 applied as surface pressures. (Uses joint pattern water_0.0)
H_0.0:	Downward area load on base slab from water to EL 0.0
H_0.5:	Downward area load on base slab from water to EL 0.5
H_3.5:	Downward area load on base slab from water to EL 3.5
H_dewater:	Water pressure inside structure due to dewatering of the center bay.
U_0.0:	Uplift pressure acting on the base slab from the water table at EL 0.0
U_0.5:	Uplift pressure acting on the base slab from the water table at EL 0.5
U_3.5:	Uplift pressure acting on the base slab from the water table at EL 3.5

Job: Maurepas Pump Station
Job No.: 10001663
Description: Description of Load Cases

Calculated By: JY
Date: 08/2010
Checked By: EY
Date: 12/2010

LOADS FROM SUPER STRUCTURE:

R_PE: Maximum point forces generated by the Pre-Engineered building in the 2-D Wind Analysis model. The forces created by wind and other loads applied to the pump station pre-engineered building. These reactions are applied to ten points along the operating floor slab where the columns of the pre-engineered building will be located.

R_dead: Point forces created by all static elements of the 2-D wind model. Dead loads, roof live loads, and crane loads from the pre-engineered structure are accounted for. These reactions are used when wind is not included in a load combination.

WIND LOADS:

W_3.5: 50psf wind load applied at EL 3.5 and higher to the head wall of the pump station structure in the form of area loads. The wind direction correlates with the wind modeled in the 2-D model.

W_all: 50psf wind load applied as area loads to the entire head wall of the pump station structure.

Job: Maurepas Pump Station
Project No.: 10001663
Description: Soil and Hydrostatic Loads for Pump Station Model

Computed by: JY
Date: 08/2010
Checked By: EY
Date: 12/10

Soil and in-situ water Loads:

*Above water table: $P_{horizontal} = K_0*(q_{surcharge}+\gamma_{sat}*Z)$ (EM 1110-2-2502 Eq 3-39)

*Below water table: $P_{horizontal} = K_0*[q_{surcharge}+\gamma_{sat}*Z_{water} + \gamma' *(Z-Z_{water})]$ (EM 1110-2-2502 Eq 3-40)

*Water pressure: $P = (Z-Z_{water})*\gamma_{water}$ (EM 1110-2-2502 Sect 3-18)

* Live Load Surcharge on abutment and wing walls

$K_0 =$ 0.8

$q_{surcharge} =$ 300 psf

$P = K_0 * q_{surcharge} =$ 240 psf

* Hydrostatic & Clayey Soil Pressures for water EL 3.5

Top of Soil Elevation = 4 ft

Water Elevation = 3.5 ft

$\gamma_{water} =$ 62.4 pcf

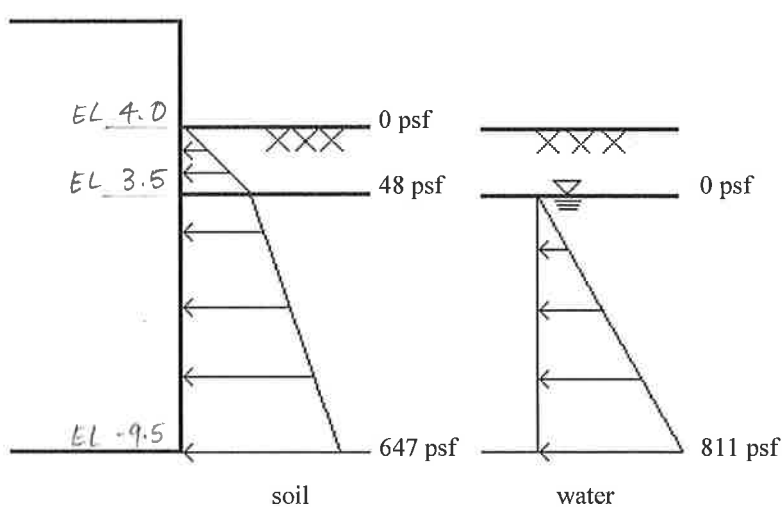
$\gamma_{sat} =$ 120 psf

$K_0 =$ 0.8 for clay

* Downward Pressure for Water EL 3.5 = 655.2 psf
(62.4 PCF × (3.5 − (−7))')
[∵ TOS EL. = 7.0]

* Uplift for Water EL 3.5 = 811.2 psf

Elevation (ft)	Water Pressure (psf)	Horizontal Soil Pressure (psf)
11.875	0.0	0.0
9.0416	0.0	0.0
5.2604	0.0	0.0
3.5	0.0	48.0
0.5	187.2	186.2
0	218.4	209.3
-2.2083	356.2	311.0
-4.9167	525.2	435.8
-7.625	694.2	560.6
-8.25	733.2	589.4
-9.5	811.2	647.0



Job: Maurepas Pump Station
Project No.: 10001663
Description: Soil and Hydrostatic Loads for Pump Station Model

Computed by: JY
Date: 08/2010
Checked By: EY
Date: 12/10

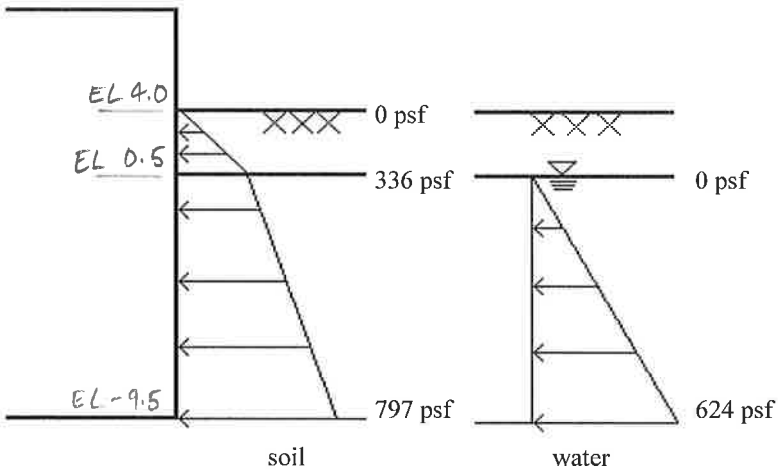
* Hydrostatic & Clayey Soil Pressures for water EL 0.5

Top of Soil Elevation = 4 ft
Water Elevation = 0.5 ft
γ_{water} = 62.4 pcf
γ_{sat} = 120 psf
K_0 = 0.8 for clay

* Downward Pressure for Water EL 0.5 = 468 psf

* Uplift for Water EL 0.5 = 624 psf

Elevation (ft)	Water Pressure (psf)	Horizontal Soil Pressure (psf)
11.875	0.0	0.0
9.0416	0.0	0.0
5.2604	0.0	0.0
3.2	0.0	76.8
2	0.0	192.0
0.5	0.0	336.0
-2.2083	169.0	460.8
-4.9167	338.0	585.6
-7.625	507.0	710.4
-8.25	546.0	739.2
-9.5	624.0	796.8



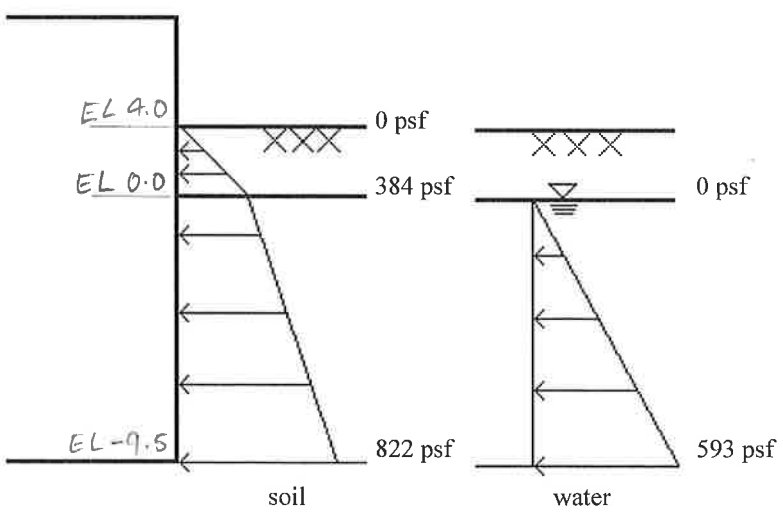
* Hydrostatic & Clayey Soil Pressures for water EL 0.0

Top of Soil Elevation = 4 ft
Water Elevation = 0 ft
γ_{water} = 62.4 pcf
γ_{sat} = 120 psf
K_0 = 0.8 for clay

* Downward Pressure for Water EL 0.0 = 436.8 psf

* Uplift for Water EL 0.0 = 592.8 psf

Elevation (ft)	Water Pressure (psf)	Horizontal Soil Pressure (psf)
11.875	0.0	0.0
9.0416	0.0	0.0
5.2604	0.0	0.0
3.5	0.0	48.0
0.5	0.0	336.0
0	0.0	384.0
-2.2083	137.8	485.8
-4.9167	306.8	610.6
-7.625	475.8	735.4
-8.25	514.8	764.2
-9.5	592.8	821.8



EQUATION FOR JOINT PATTERNS : $P = CZ + D$ (Z: JOINT COORDINATE IN SAP MODEL)

WATER AT EL 3.5 :

$$811 = -9.5C + D$$
$$0 = 3.5 \times C + D$$
$$811 = -13C \Rightarrow C = \frac{-811}{13} = -62.4$$

$$\therefore D = 811 + 9.5(-62.4) = 218.2$$

$$P = -0.0624Z + 0.2182$$
(Water_3.5)

SOIL LOADS :

SOIL WITH WATER @ EL 3.5

ABOVE WATER TABLE :

$$48 = 3.5C + D$$
$$0 = 4C + D$$
$$48 = -0.5C \Rightarrow C = \frac{-48}{0.5} = -96$$

$$D = 48 + 3.5 \times 96 = 384$$

$$\therefore P = 0.384 - 0.096Z \text{ (Soil_3.5_a)}$$

BELOW WATER TABLE :

$$647 = -9.5C + D$$
$$48 = 3.5C + D$$
$$599 = -13C \Rightarrow C = \frac{-599}{13} = -46.1$$

$$D = 647 + 9.5C$$
$$= 647 - 9.5 \times 46.1 = 209.1$$

$$\therefore P = 0.2091 - 0.0461Z \text{ (Soil_3.5_b)}$$

SOIL WITH WATER @ EL 0.5

ABOVE WATER TABLE:

$$336 = 0.5C + D$$
$$0 = 4C + D$$
$$336 = -3.5C \Rightarrow C = -336/3.5 = -96$$
$$\therefore D = 336 + 96 \times 0.5 = 384$$
$$\therefore P = 0.384 - 0.096z \quad (\text{Soil}\_0.5\_a)$$

BELOW WATER TABLE:

$$797 = -9.5 \times C + D$$
$$336 = 0.5 \times C + D$$
$$-461 = -10C \Rightarrow C = \frac{-461}{10} = -46.1$$
$$D = 797 - 9.5 \times 46.1 = 359.05$$
$$P = 0.3591 - 0.0461z \quad (\text{Soil}\_0.5\_b)$$

SOIL WITH WATER @ EL 0.0

ABOVE WATER TABLE:

$$384 = 0 \times C + D$$
$$0 = 4 \times C + D$$
$$384 = -4C \Rightarrow C = \frac{-384}{4} = -96$$
$$D = 384$$
$$P = 0.384 - 0.096z \quad (\text{Soil}\_0.0\_a)$$

BELOW WATER TABLE:

$$822 = -9.5C + D$$
$$384 = 0 \times C + D$$
$$438 = -9.5C \Rightarrow C = \frac{-438}{9.5} = -46.11$$
$$D = 822 - 9.5 \times 46.11 = 383.96$$
$$P = 0.38396 - 0.04611z \quad (\text{Soil}\_0.0\_b)$$

WATER AT EL 0.0:

$$593 = -9.5C + D$$
$$0 = 0.0C + D$$
$$593 = -9.5C \Rightarrow C = -593/9.5 = -62.42$$
$$D = 0$$
$$\therefore P = -0.06242z$$

- PUMP & GEAR DRIVE:

PUMP = 11300 LB } (AS PER DESIGN CRITERIA)
GEAR = 1766 LB }
TOTAL = 13066 LB = 13.07 K

THIS LOAD IS SUPPORTED BY AREA AROUND OPENINGS & MODELED BY TRANSFERING LOAD THROUGH FRAMES AROUND THE PERIMETER OF THE OPENING. FRAMES THEN TRANSFER LOAD TO FLOOR.

PERIMETER OF THE OPENING = (9'x2) + (6.5'x2) = 31'

LOAD = 13.07 K/31' = 0.422 K/FT

↑ LOAD PATTERN D-E-pumpgear

- FORMED SUCTION INTAKE (FSI): LOAD = 11000 LB = 11.0 K

THIS LOAD IS SPREAD ALONG BASE SLAB AREA WHERE 'FSI' WILL REST.

↑ COORDINATES FROM SAP MODEL

AREA UNDER FSI STRUCTURE = (14.5 - (-2.5))' x (17.583 - 11.083)'
= 110.5 FT²

FSI SURFACE PRESSURE = 11 K / 110.5 FT² = 0.1 K/FT²

↑ LOAD PATTERN D-E-FSI

- ENGINE: 12000 LB = 12.0 K - SPREAD ALONG FLOOR SLAB WHERE ENGINE SKIDS WILL REST

↑ CONSERVATIVELY

AREA UNDER SKIDS = 178" LONG x 70" WIDE = 14.83' x 5.83'

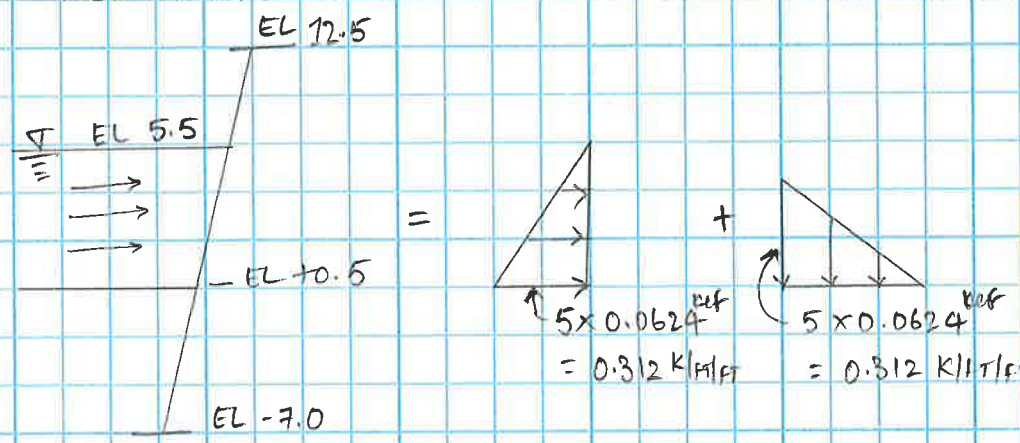
AREA UNDER ENGINE SKIDS = (30.708 - 19.875)' x (17.583 - 11.083)' = 70.415 FT²

ENGINE SURFACE PRESSURE = 12.0 K / 70.415 FT² = 0.17 K/FT²

BLOCKED TRASH RACK CONDITION:

- WATER @ EL 0.5. ASSUME THE DEBRIS LOAD AS 5' OF WATER HEAD ABOVE EL +0.5.

- THE TRASH RACK IS AT AN ANGLE OF 70° WITH HORIZONTAL.

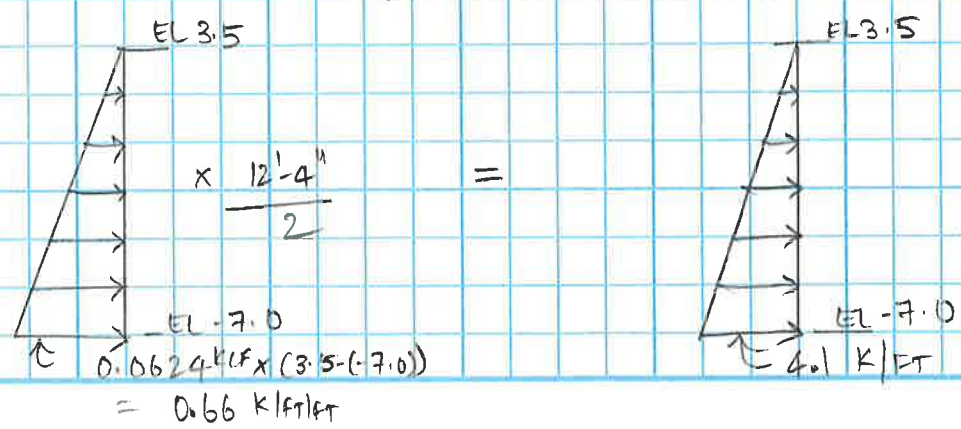


- USE SAP MODEL TO FIND THE REACTIONS ON THE TOP & BOTTOM SLABS PER FOOT. (SEE ATTACHED SHEETS 53 & 54)

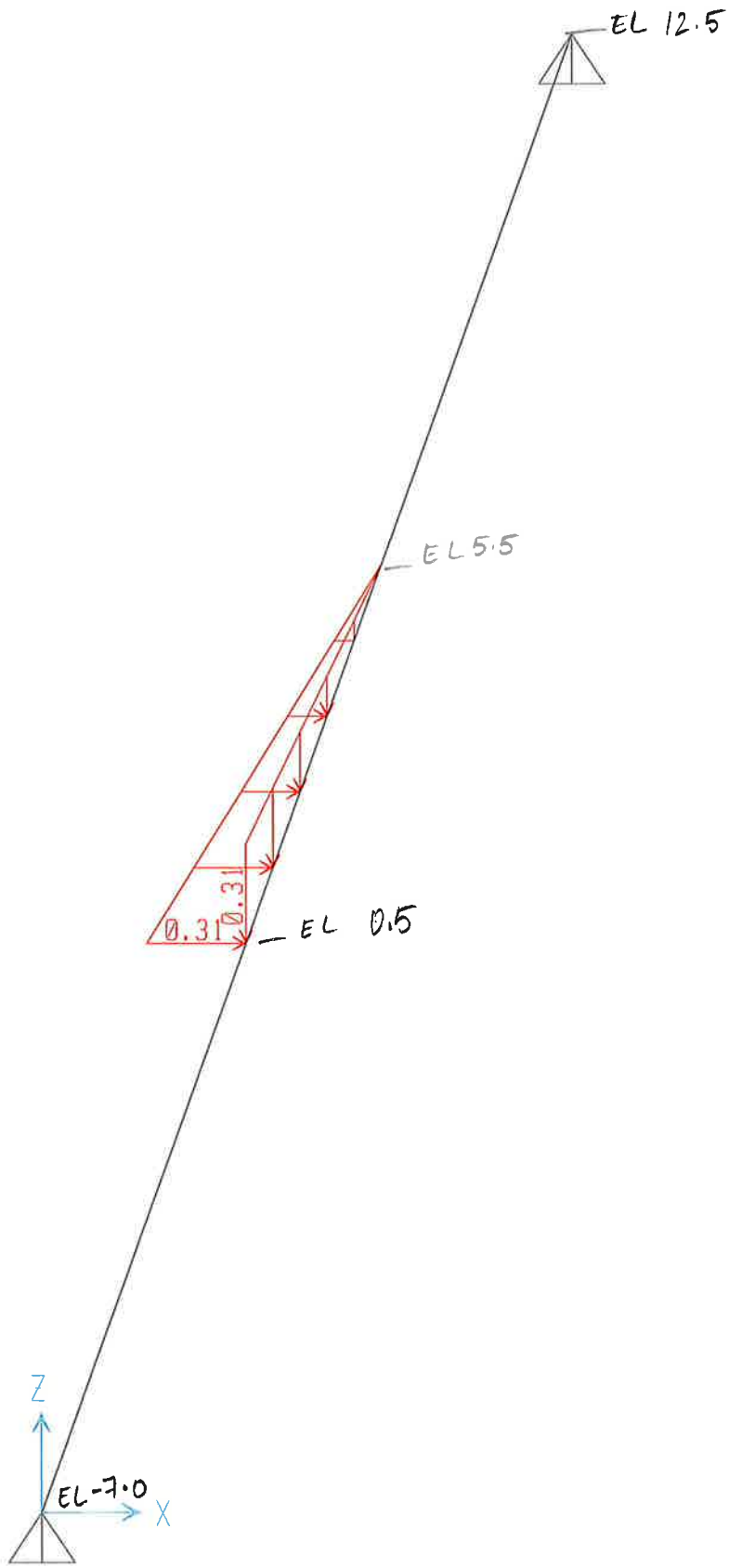
LOAD ON NEEDLE BEAMS SUPPORTED BY SIDE WALLS FOR DEWATERING LOAD CASE:

- NEEDLE BEAMS ARE VERTICAL BEAMS WHICH TAKE WATER LOAD SPANNING ALONG ONE OF THE BAYS (12'-4" LONG) & UP TO AN EL. 3.5

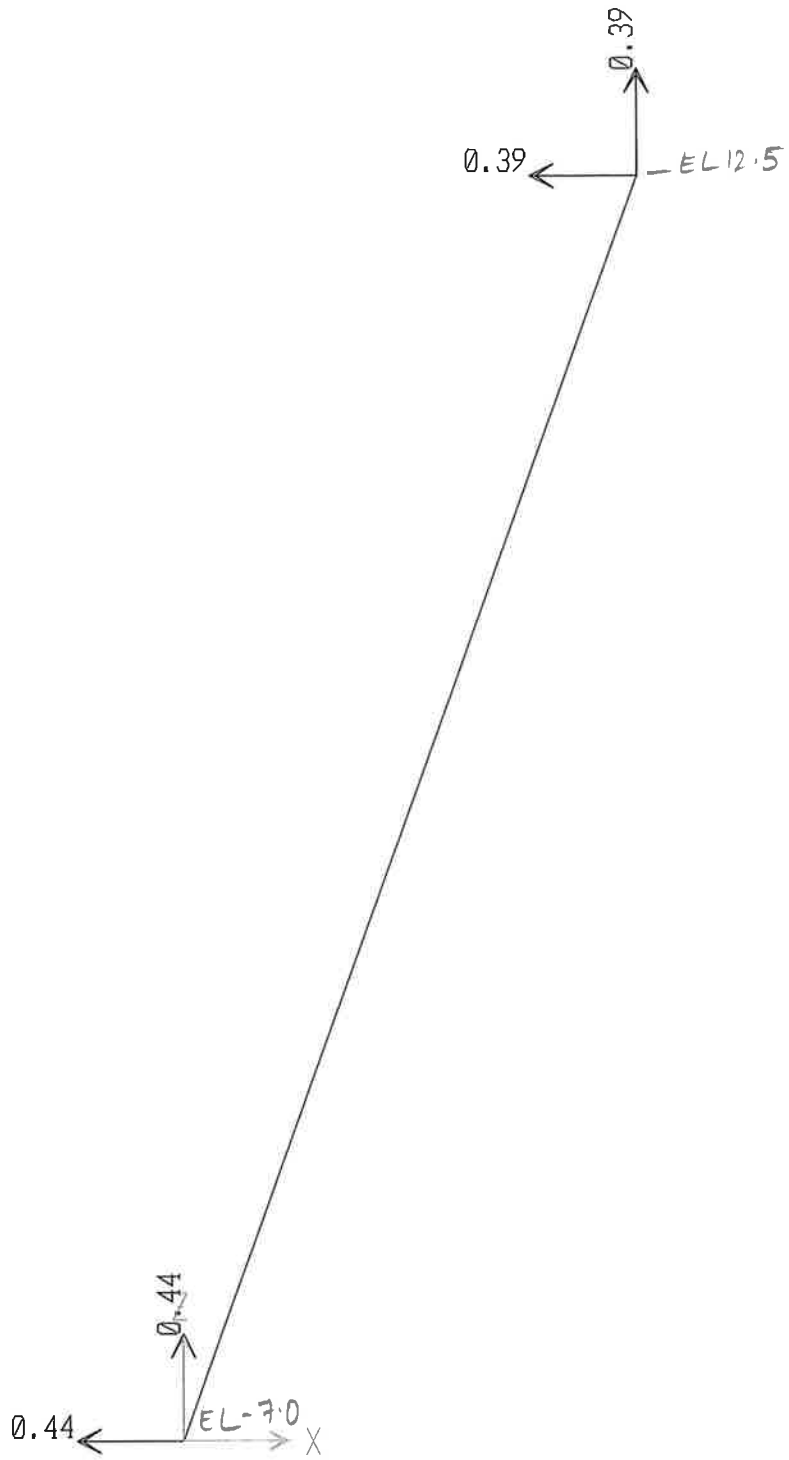
- A VERTICAL SLOT IS PROVIDED FOR EACH NEEDLE BEAM AT THE EDGE OF THE WALL. THEREFORE, LOAD ON EACH NEEDLE BEAM IS:

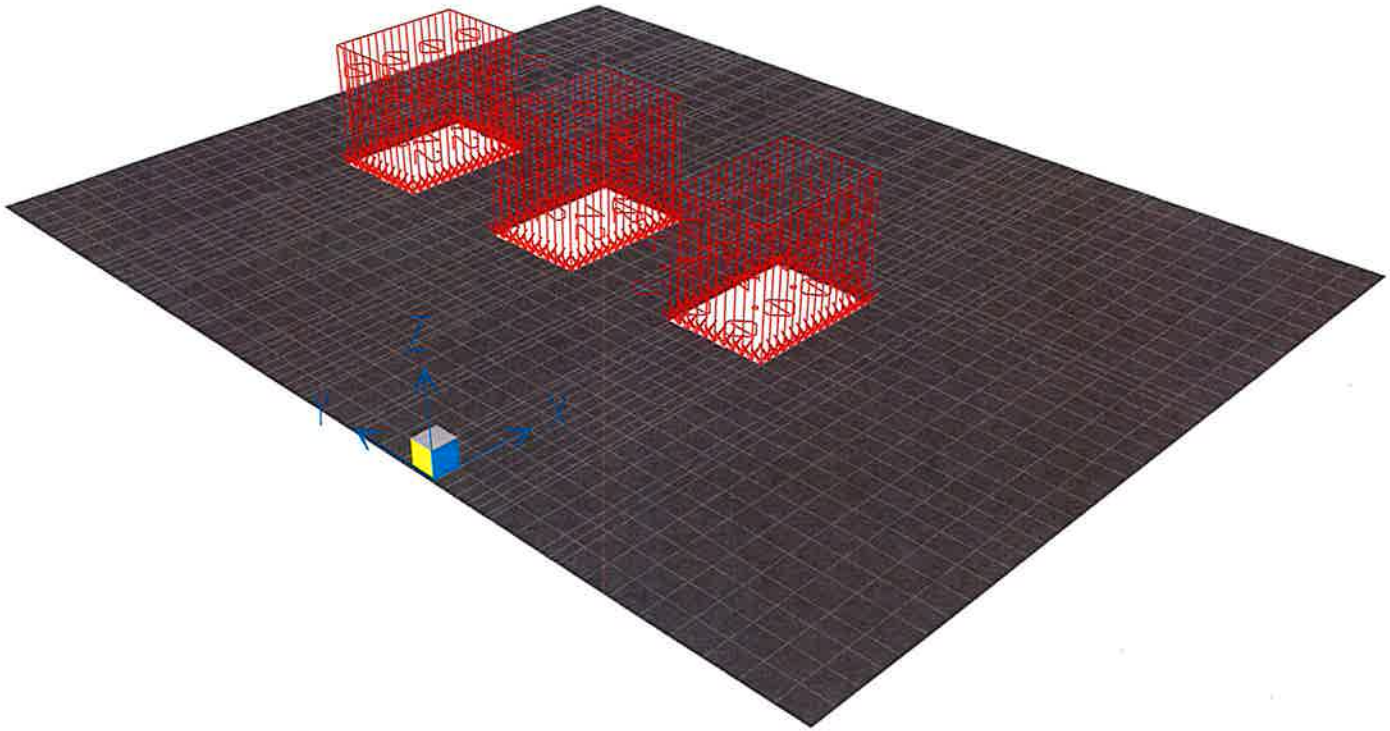


JY (53)
EY 12/10
9/28/10 9:09:53

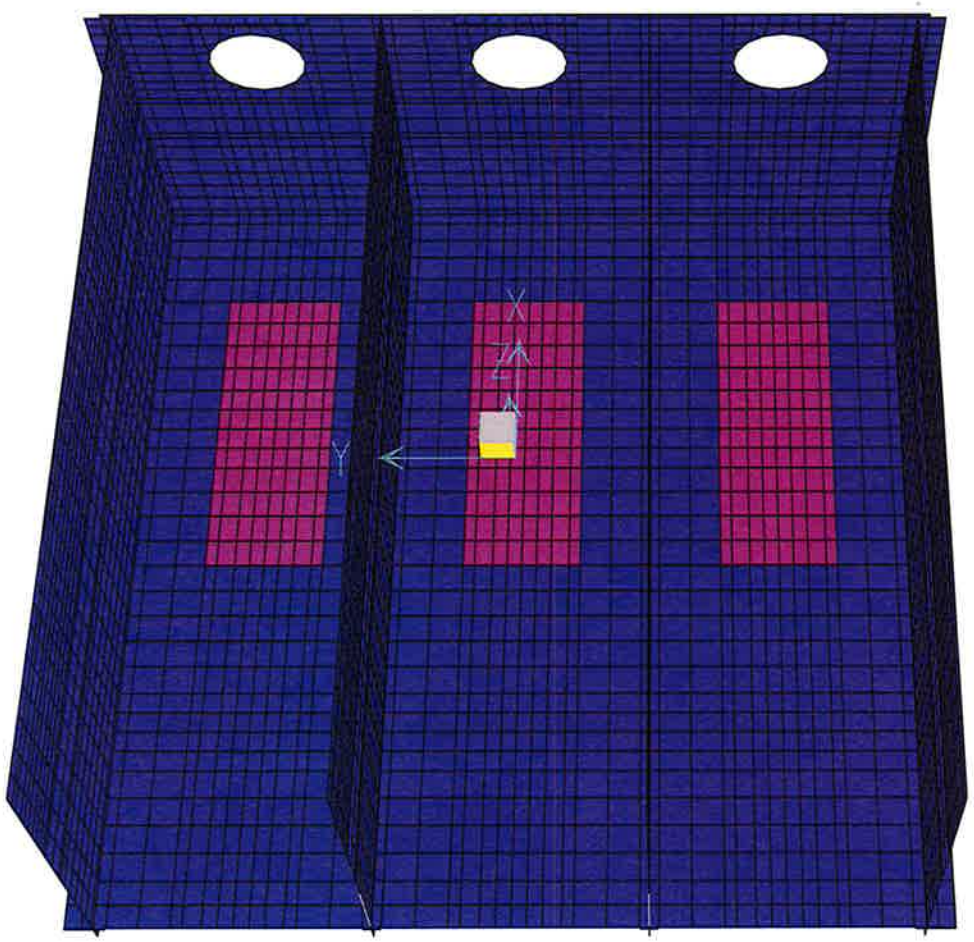


JY (54)
EY 12/10
9/28/10 9:14:05



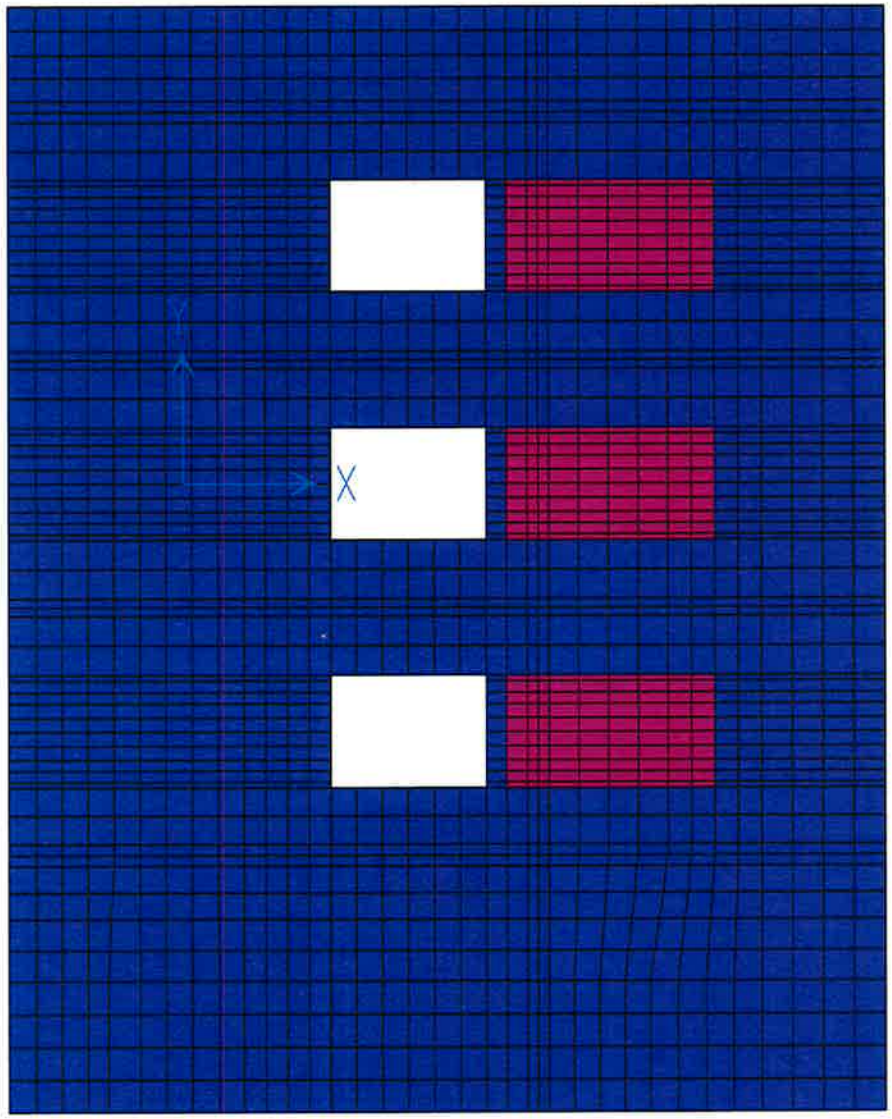


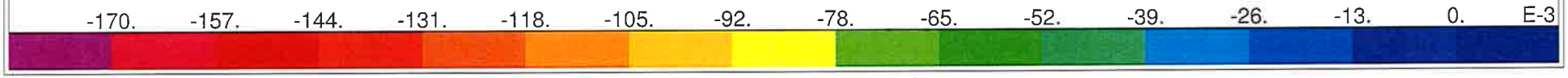
JY 55
EY 12/10



-100. -92. -85. -77. -69. -62. -54. -46. -38. -31. -23. -15. -8. 0. E-3

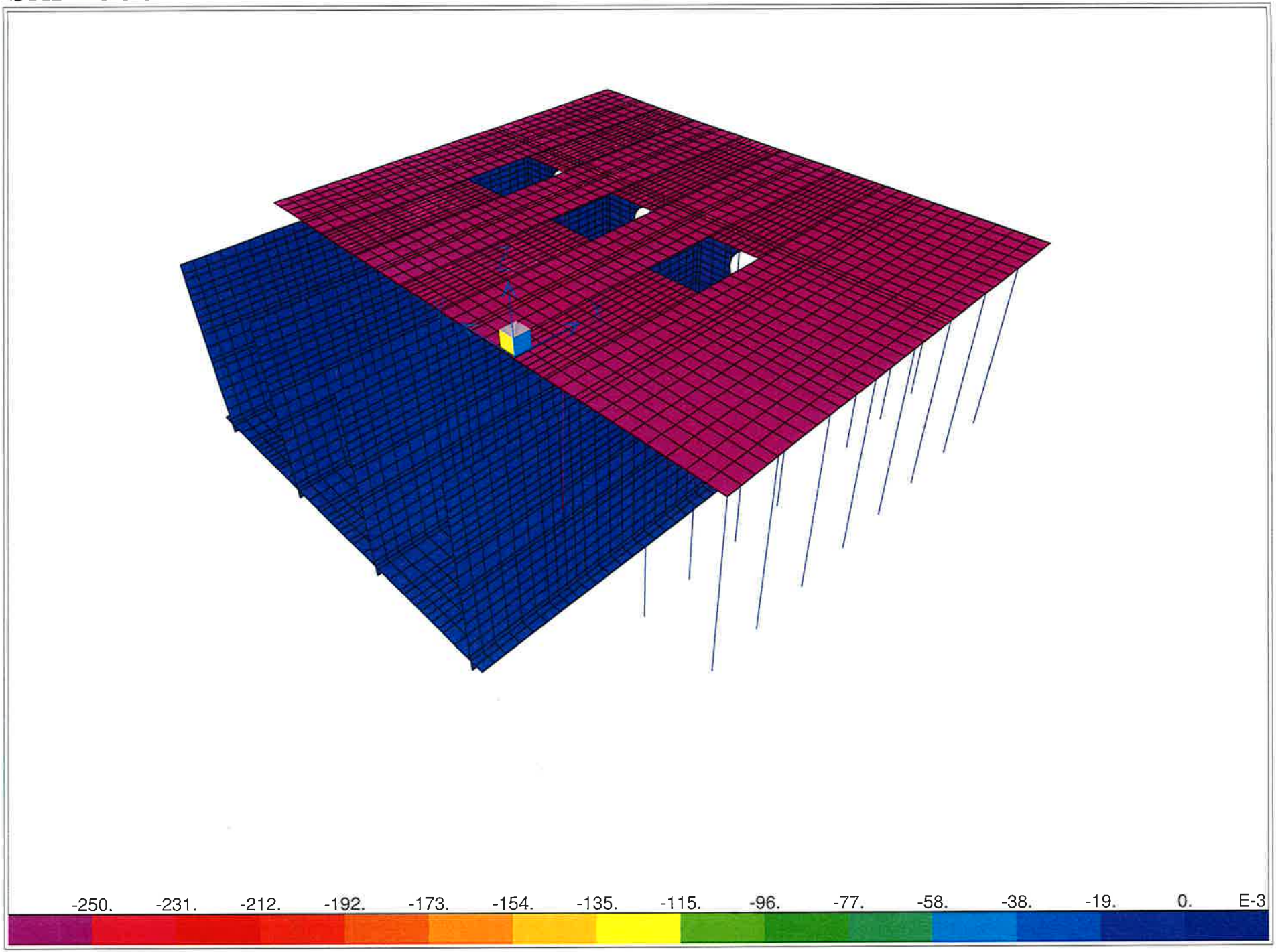
SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (D-E_FSI) (GLOBAL - Z) - Kip, ft, F Units





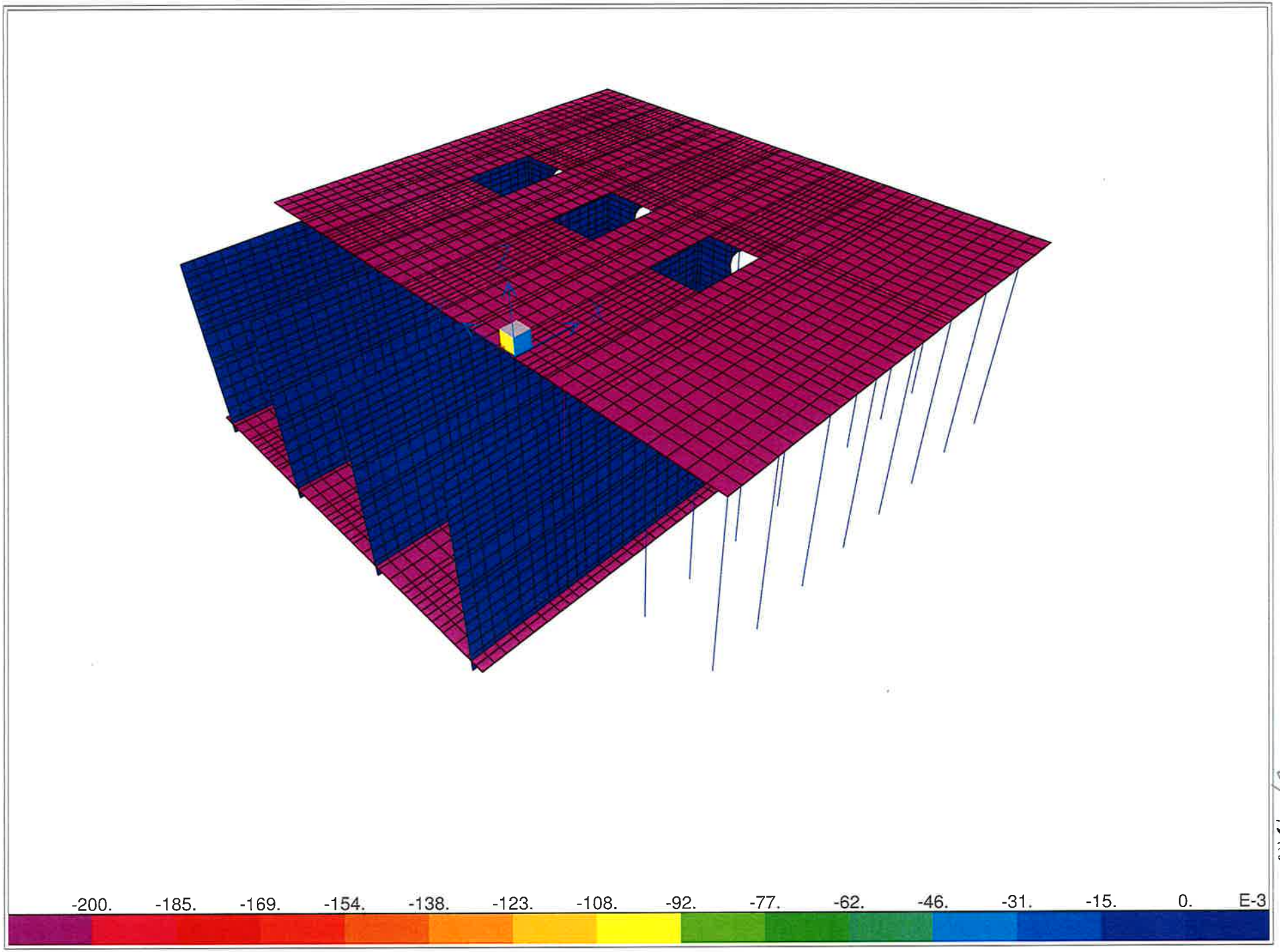
SAP2000 v14.2.3 - File:PS MODEL - Area Uniform (D-E_engine) (GLOBAL - Z) - Kip, ft, F Units

JY 63
EY 12/10



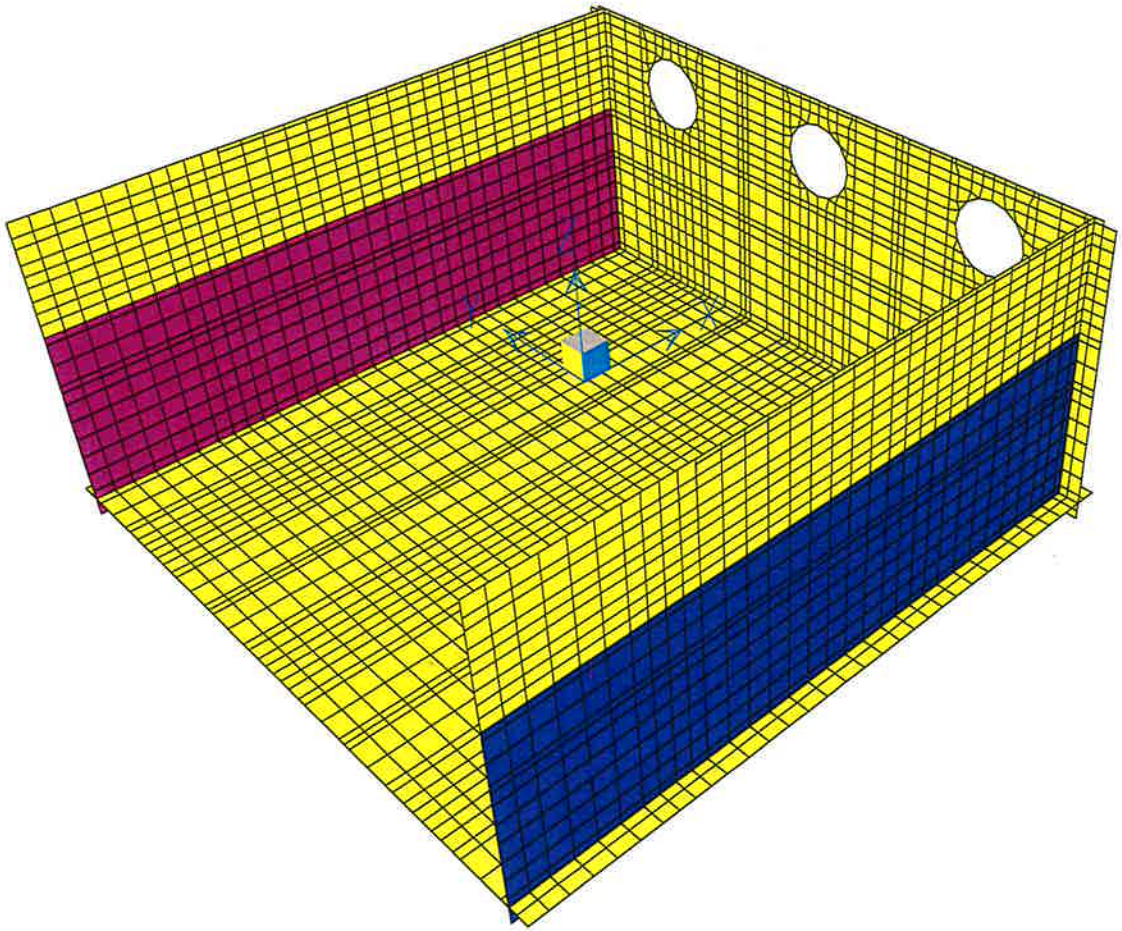
SAP2000 v14.2.3 - File:PS MODEL - Area Uniform (L_floor) (GLOBAL - Z) - Kip, ft, F Units

JY (58)
EY 10/10



SAP2000 v14.2.3 - File:PS MODEL - Area Uniform (LS_200) (GLOBAL - Z) - Kip, ft, F Units

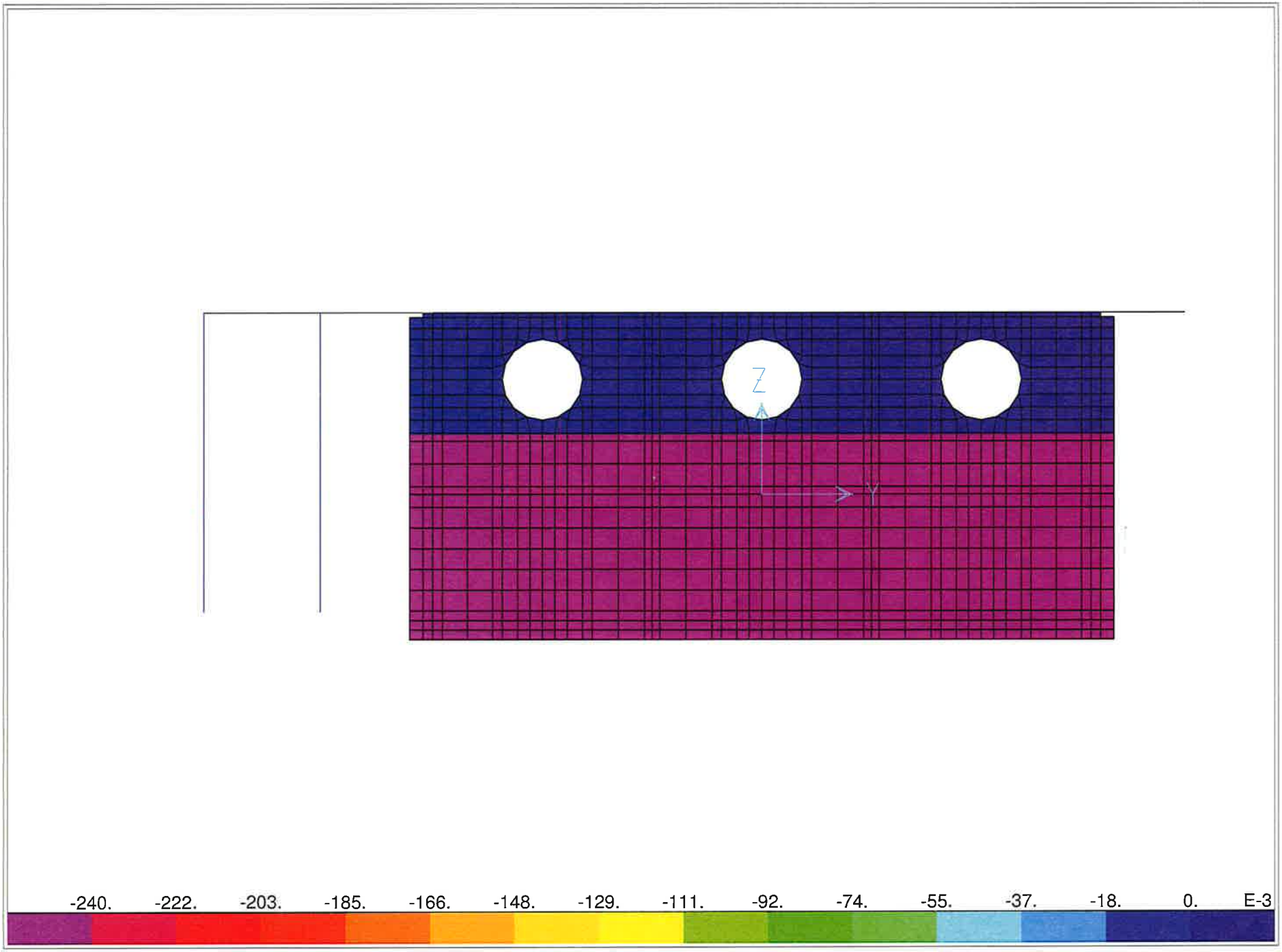
EY (59)
EY 12/10



-240. -203. -166. -129. -92. -55. -18. 18. 55. 92. 129. 166. 203. 240. E-3

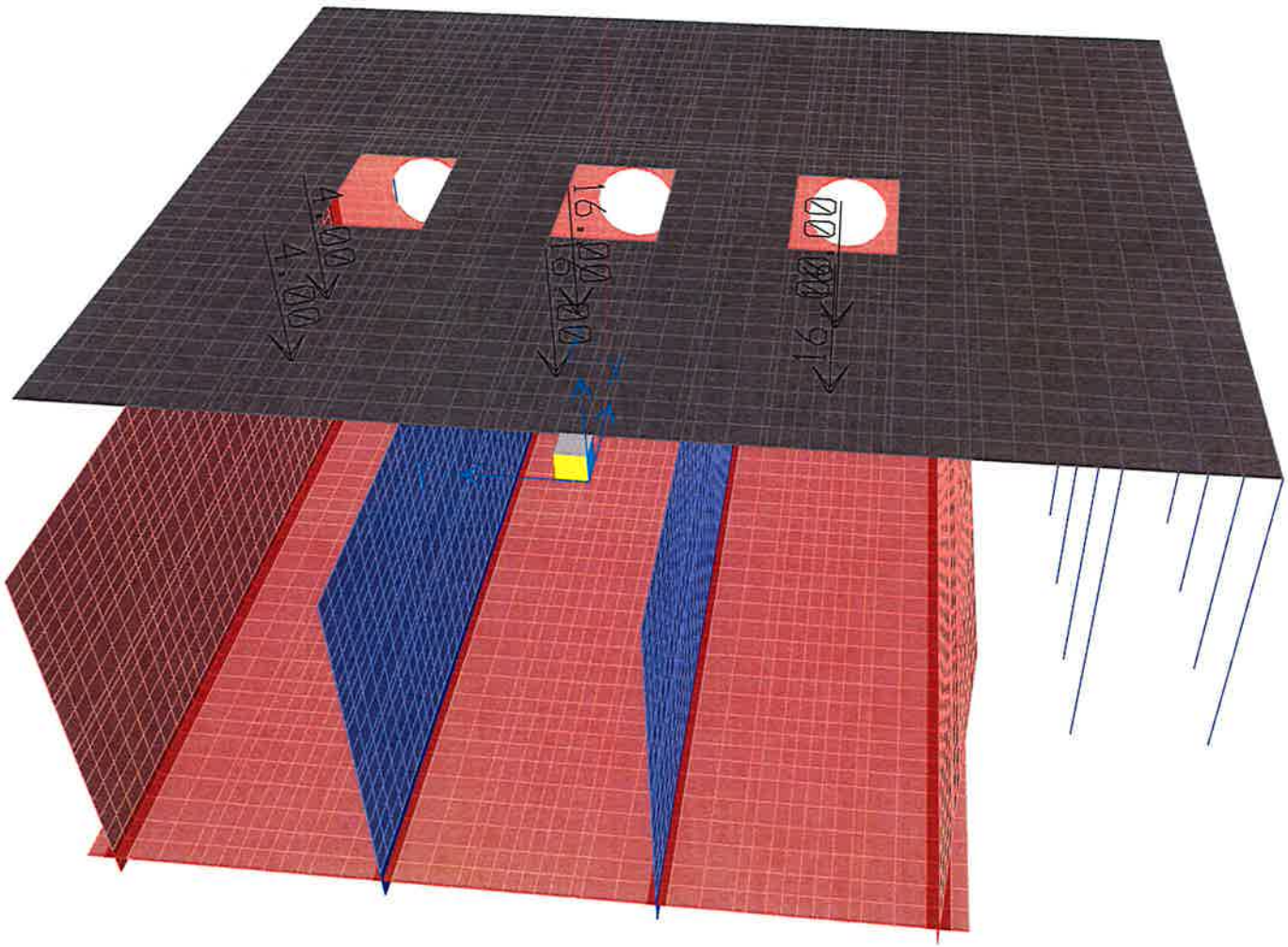
SAP2000 v14.2.3 - File:PS MODEL - Area Uniform (LS_300) (GLOBAL - Y) - Kip, ft, F Units

EY (60)
0/12/10



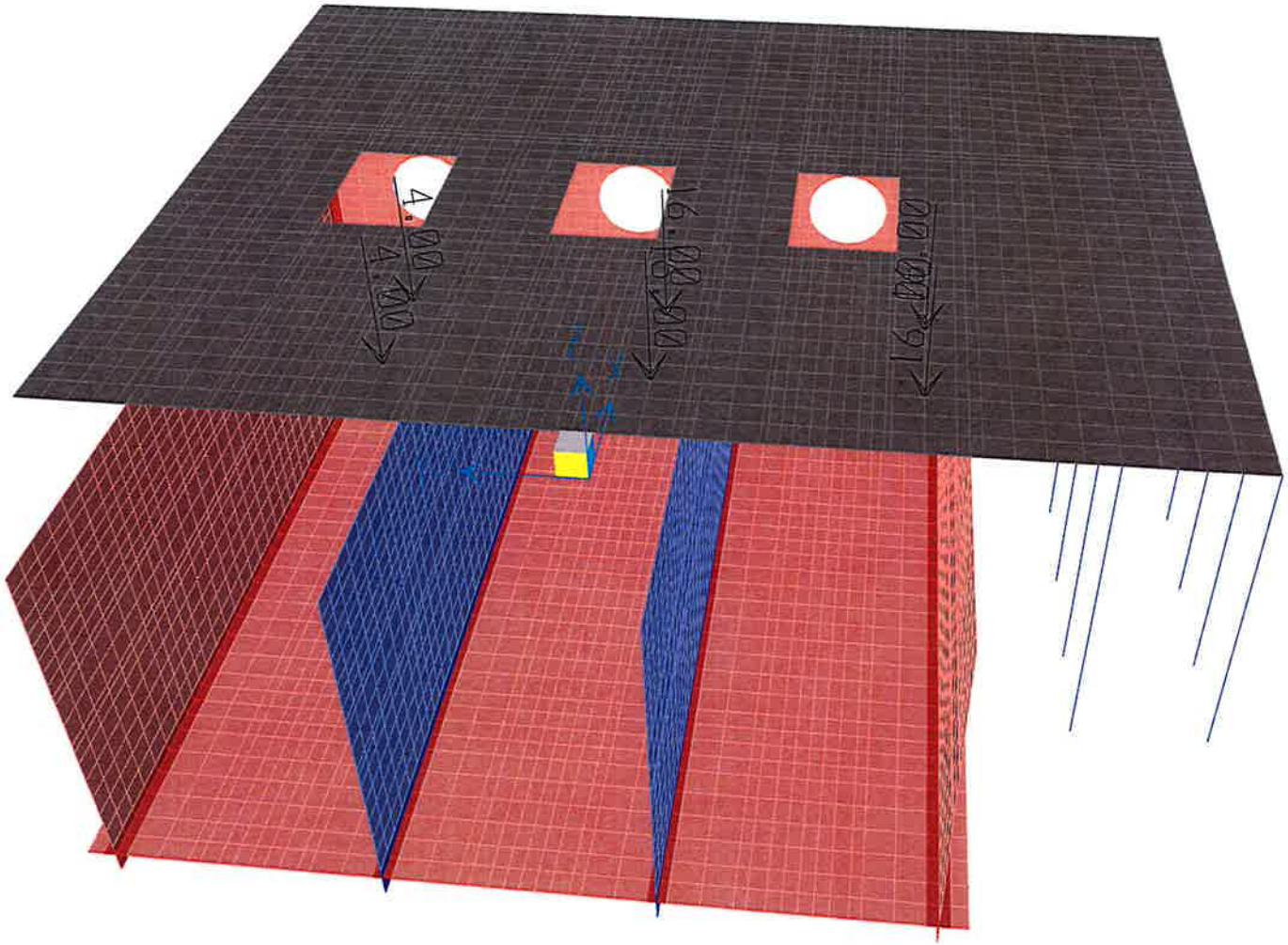
SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (LS_300) (GLOBAL - X) - Kip, ft, F Units

JY 9/21/10 (61)



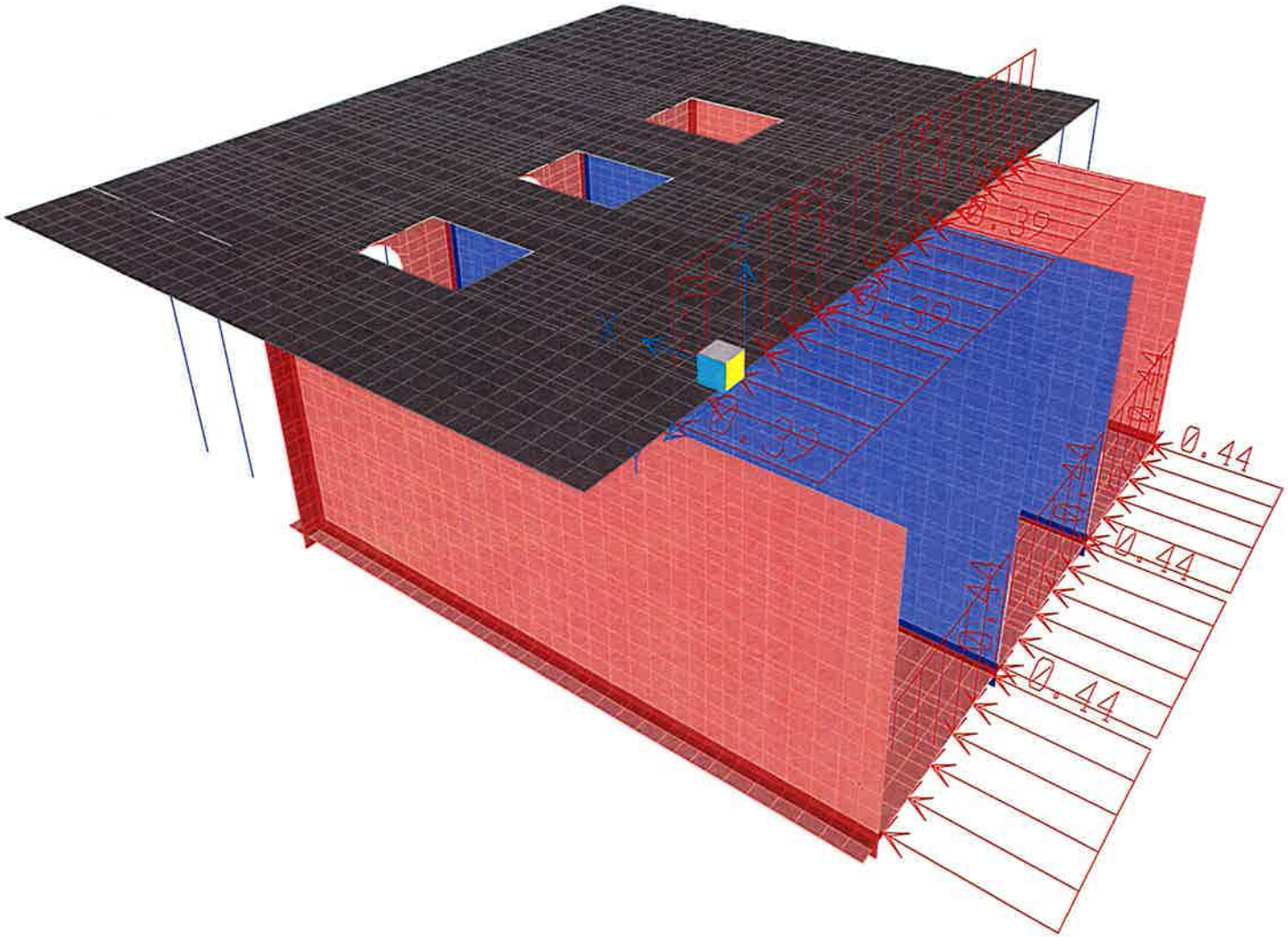
SAP2000 v14.2.3 - File:PS MODEL - Joint Loads (L_truck_m) (As Defined) - Kip, ft, F Units

JY 69
EY 12/10



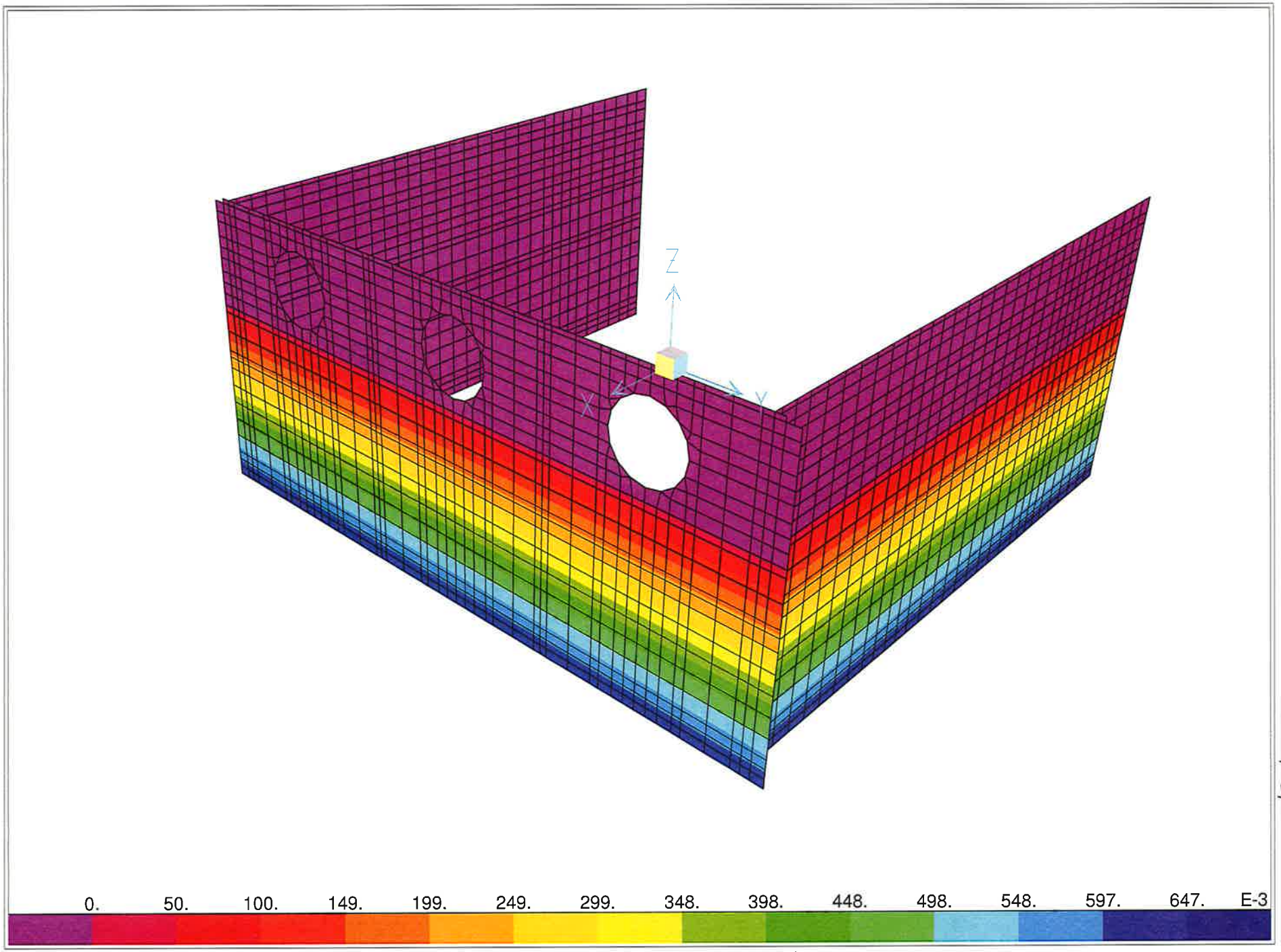
JY 63
EY 12/10

SAP2000 v14.2.3 - File:PS MODEL - Joint Loads (L_truck_s) (As Defined) - Kip, ft, F Units



SAP2000 v14.2.3 - File:PS MODEL - Frame Span Loads (L_rack) (As Defined) - Kip, ft, F Units

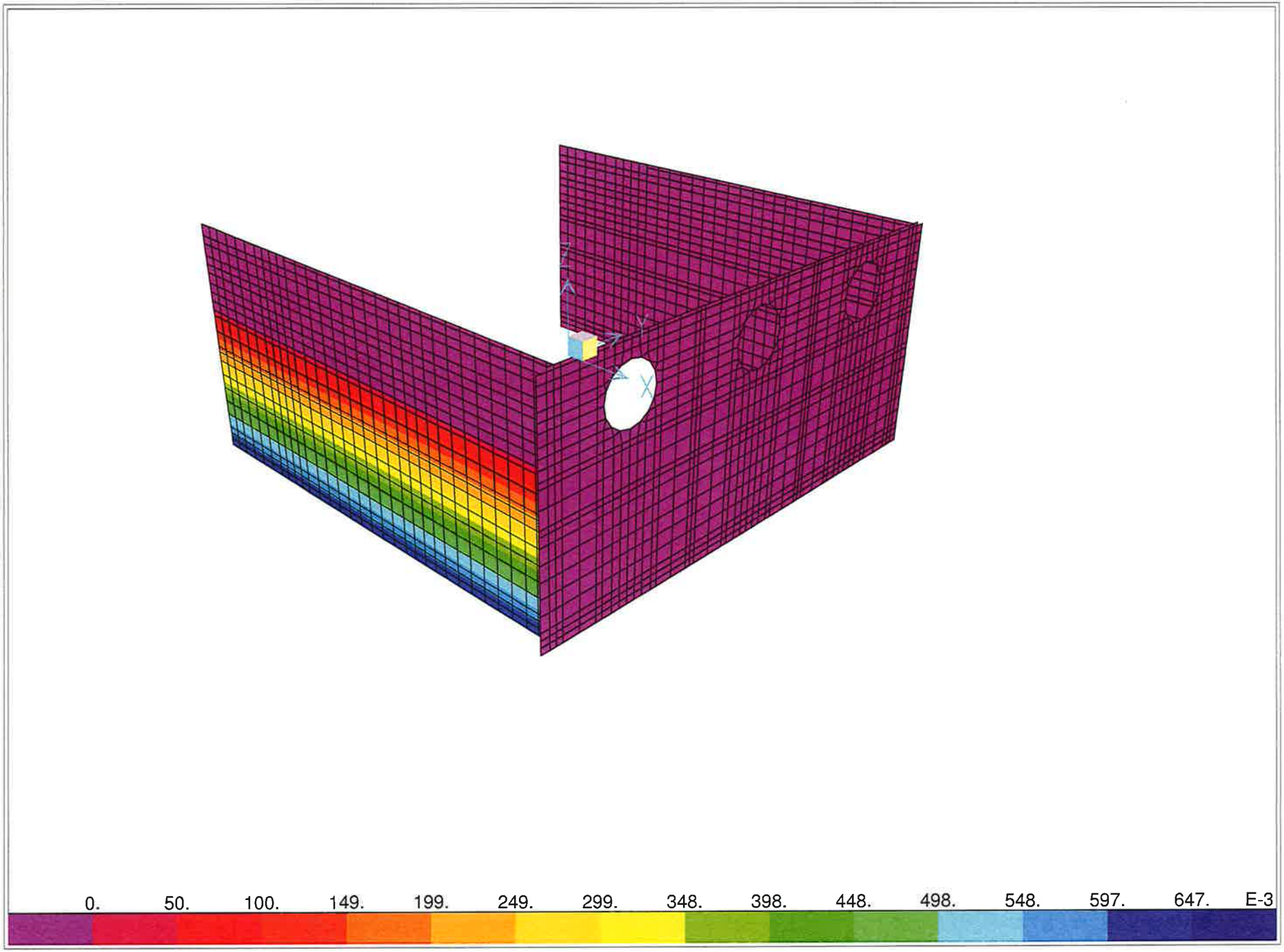
JY (64)
EY 12/10



SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Bottom (S_3.5) - Kip, ft, F Units

JY
EY 12/10

65

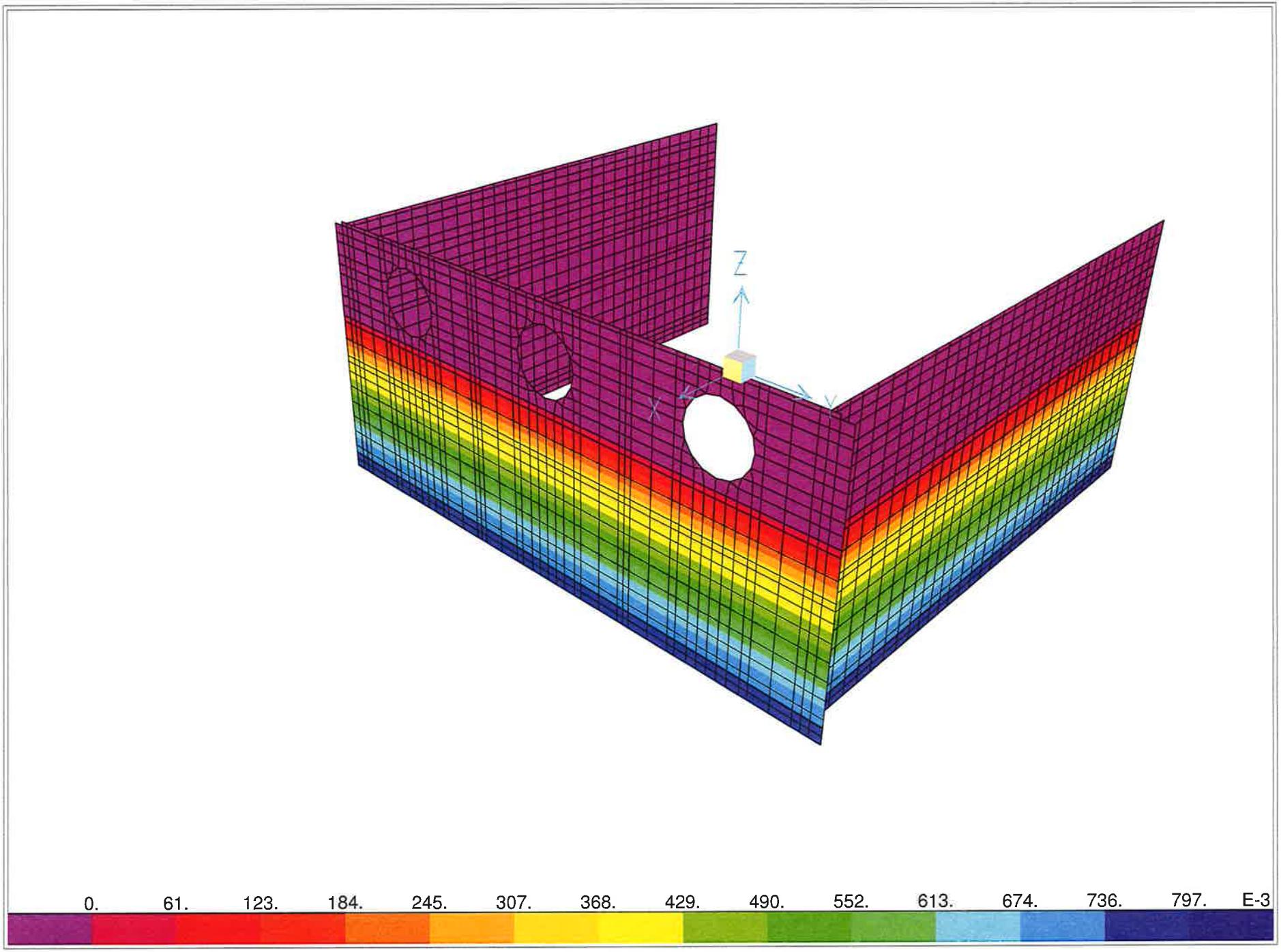


SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Top (S_3.5) - Kip, ft, F Units

EY 01/12/10 (66)

SAP2000

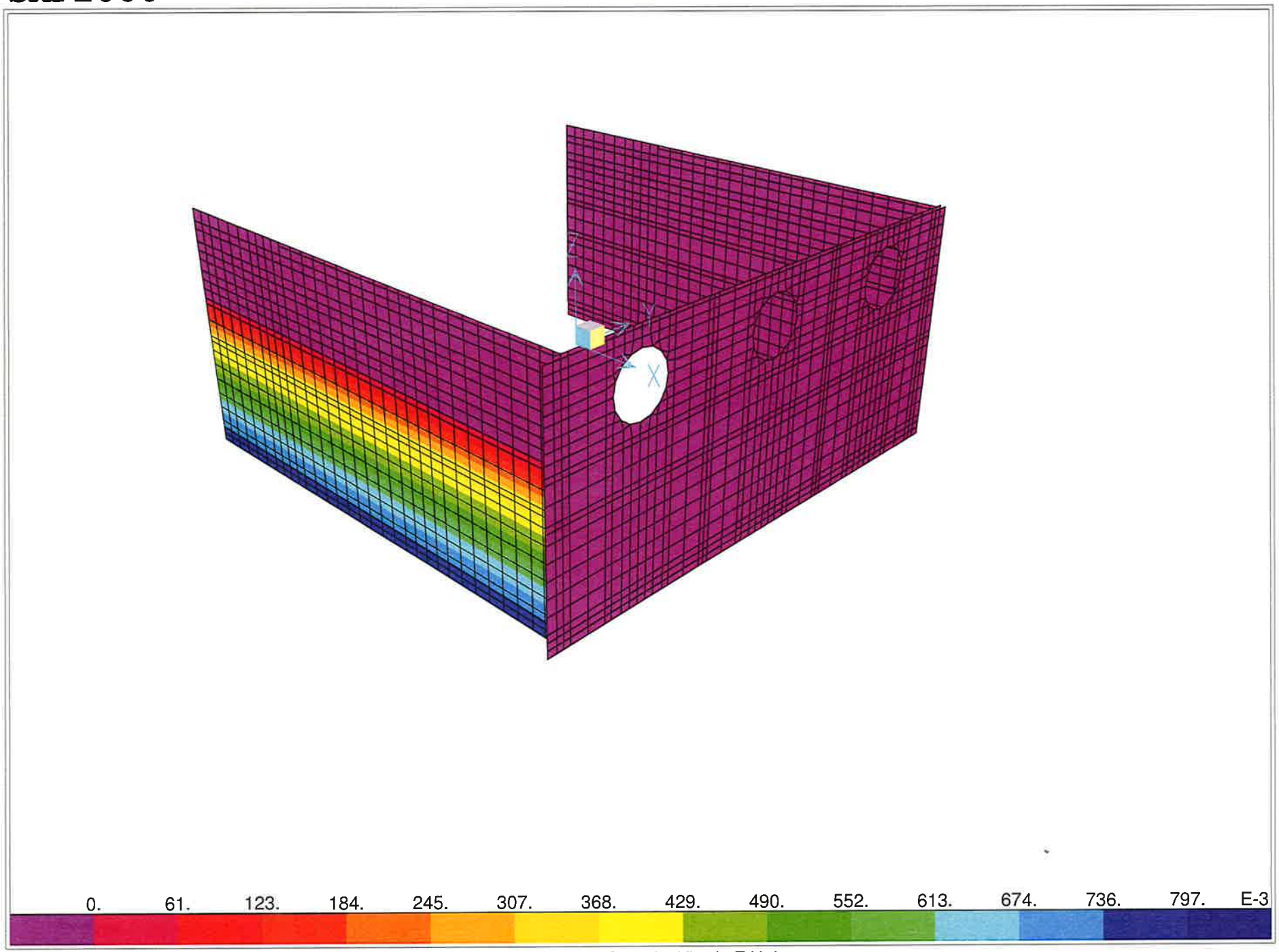
9/22/10 15:45:58



SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Bottom (S_0.5) - Kip, ft, F Units

JY
9/12/10

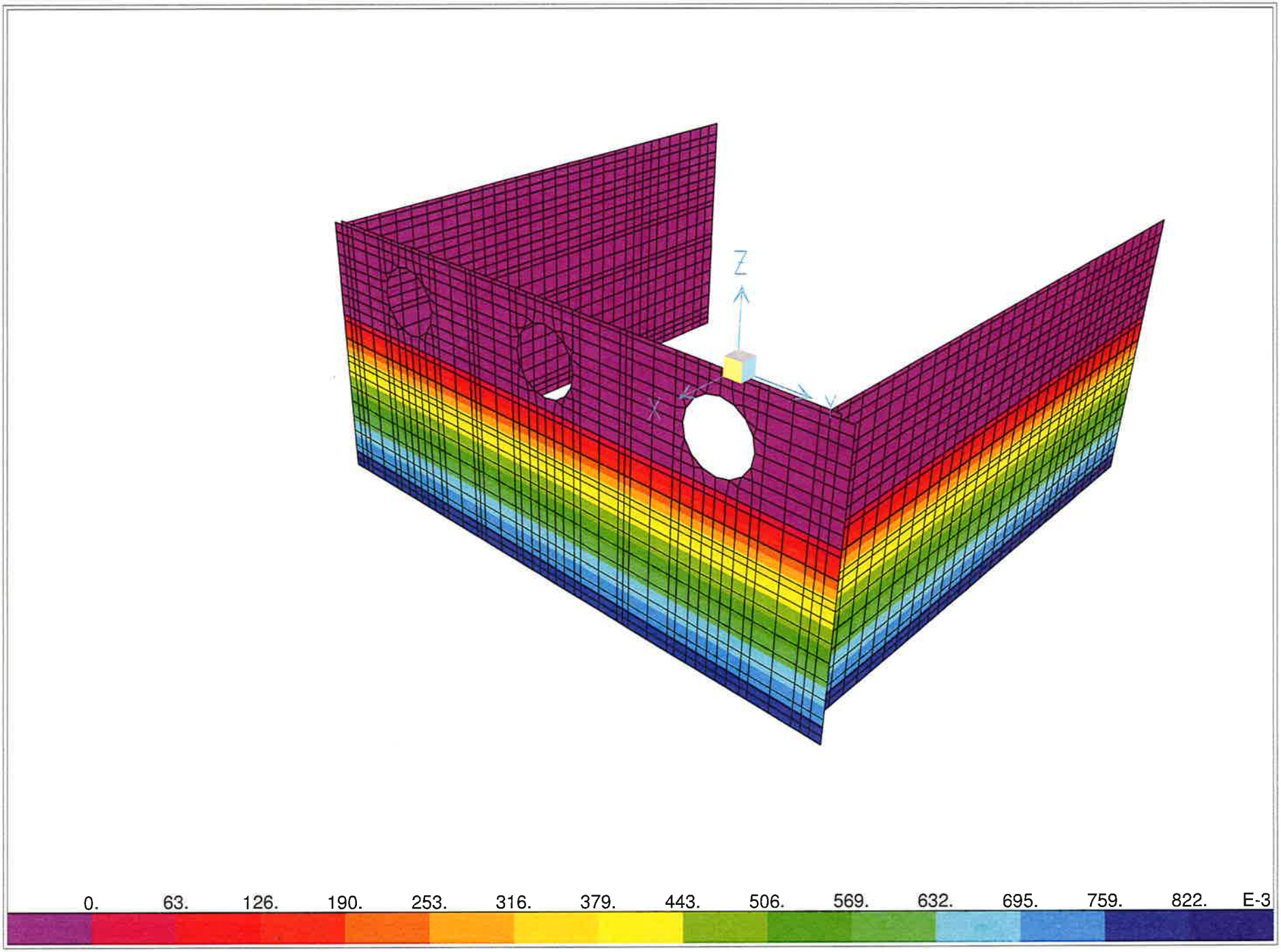
11



SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Top (S_0.5) - Kip, ft, F Units

JY
CY 12/10

68



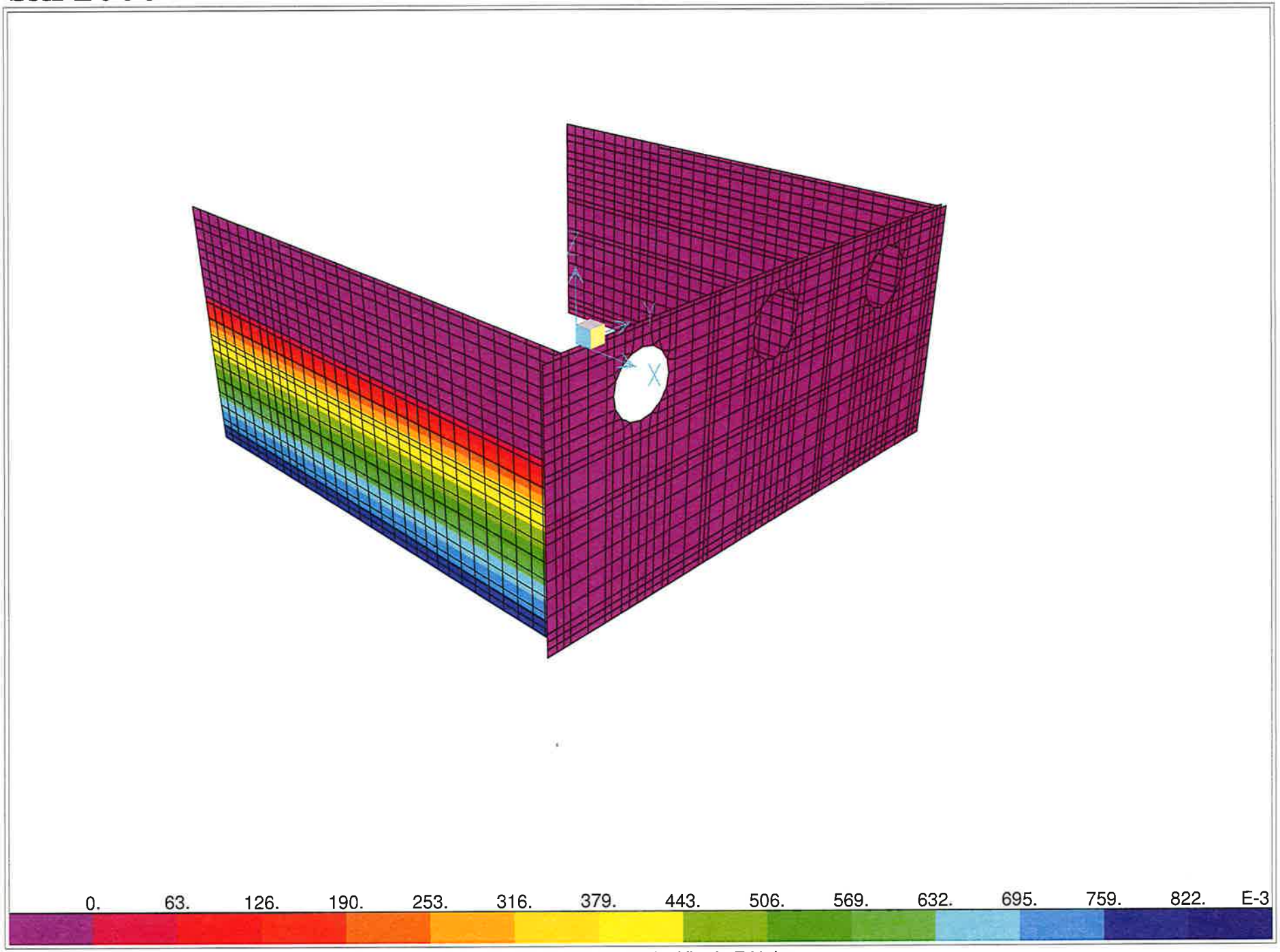
SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Bottom (S_0.0) - Kip, ft, F Units

JY 9/12/10

69

SAP2000

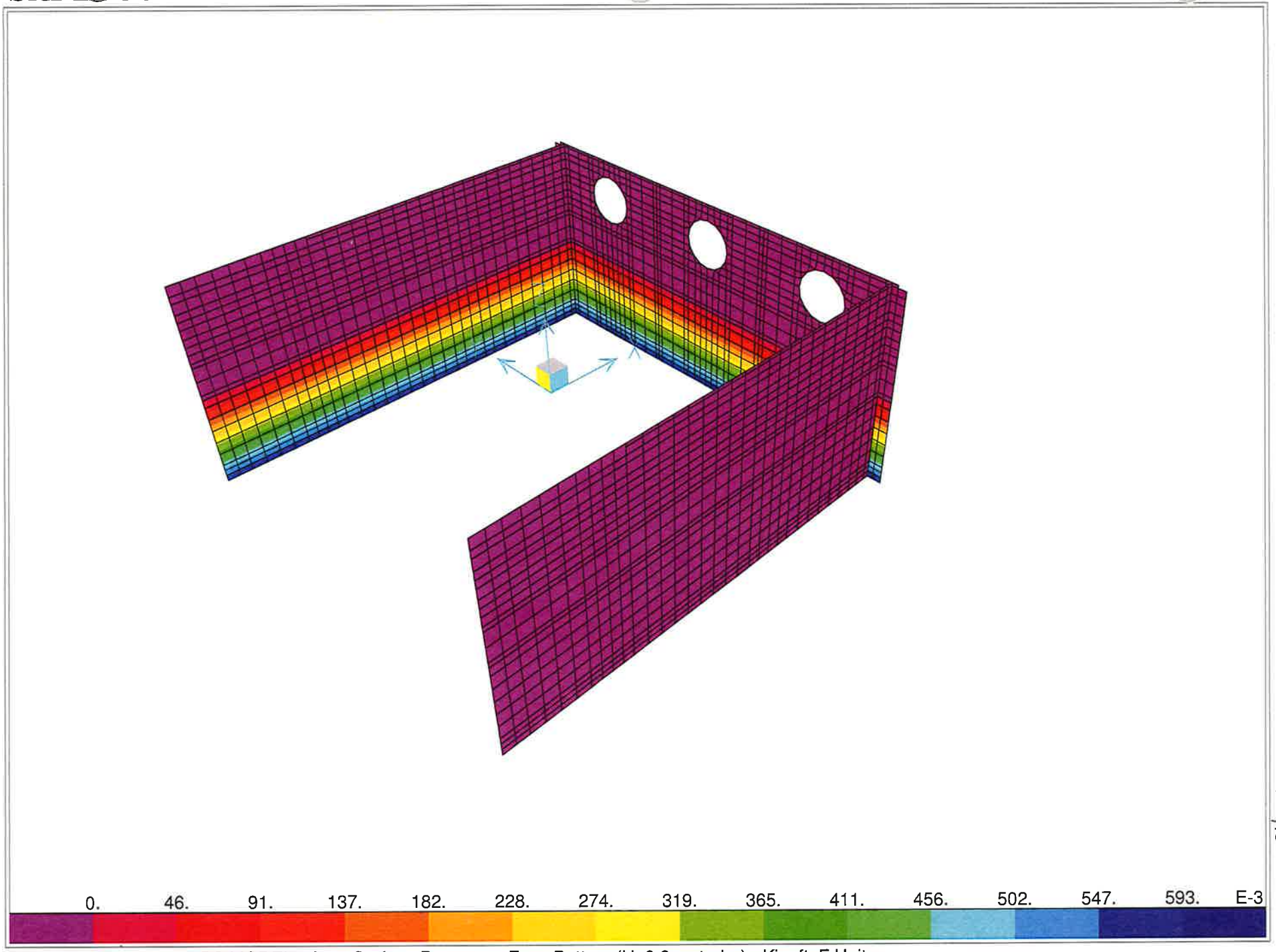
9/22/10 15:44:54



SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Top (S_0.0) - Kip, ft, F Units

JY 9/12/10

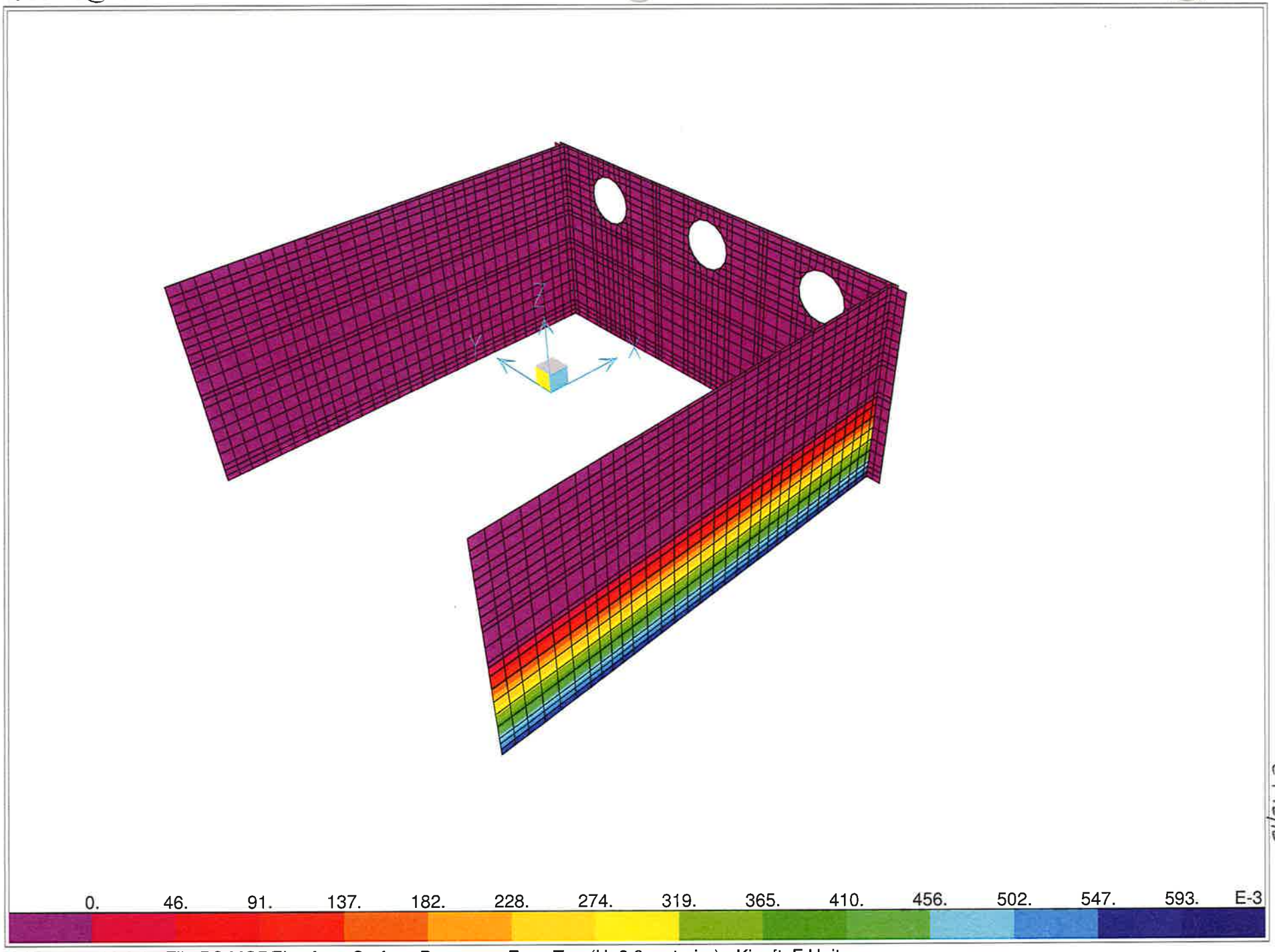
(70)



SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Bottom (H_0.0_exterior) - Kip, ft, F Units

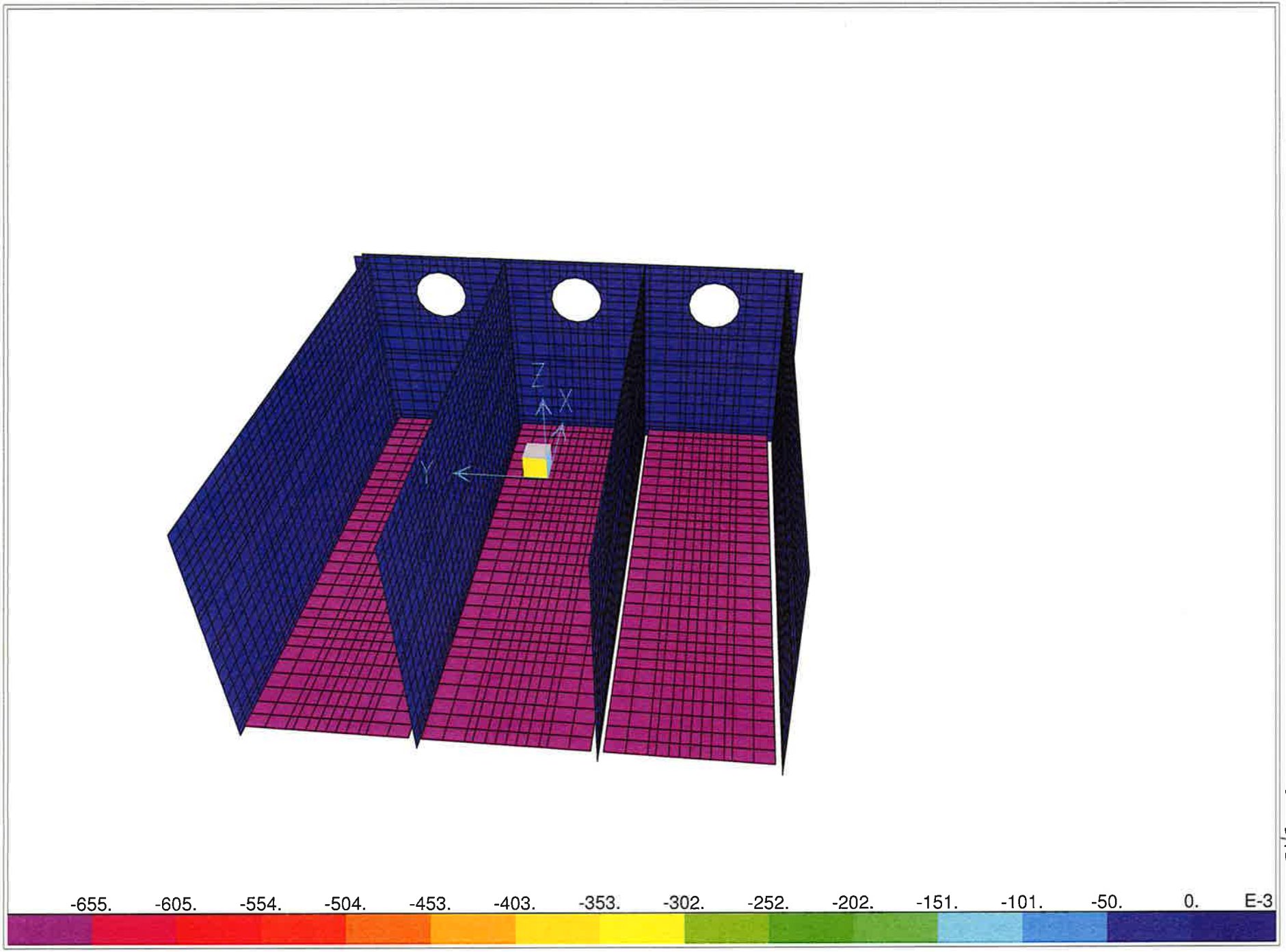
JY 01/12/10

71



SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Top (H_0.0_exterior) - Kip, ft, F Units

JY 12/10 #2

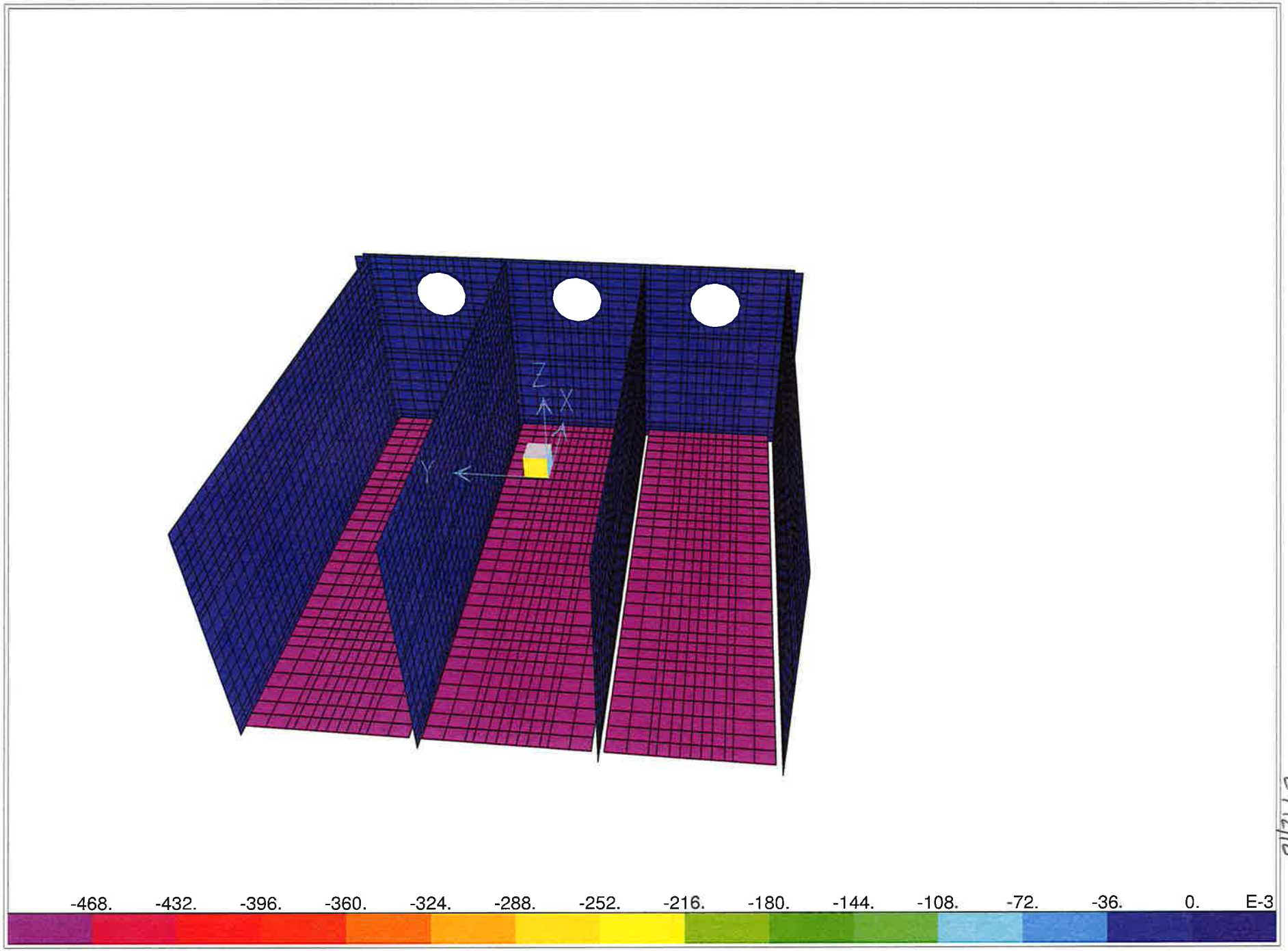


SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (H_3.5) (GLOBAL - Z) - Kip, ft, F Units

JY 9/12/10 (73)

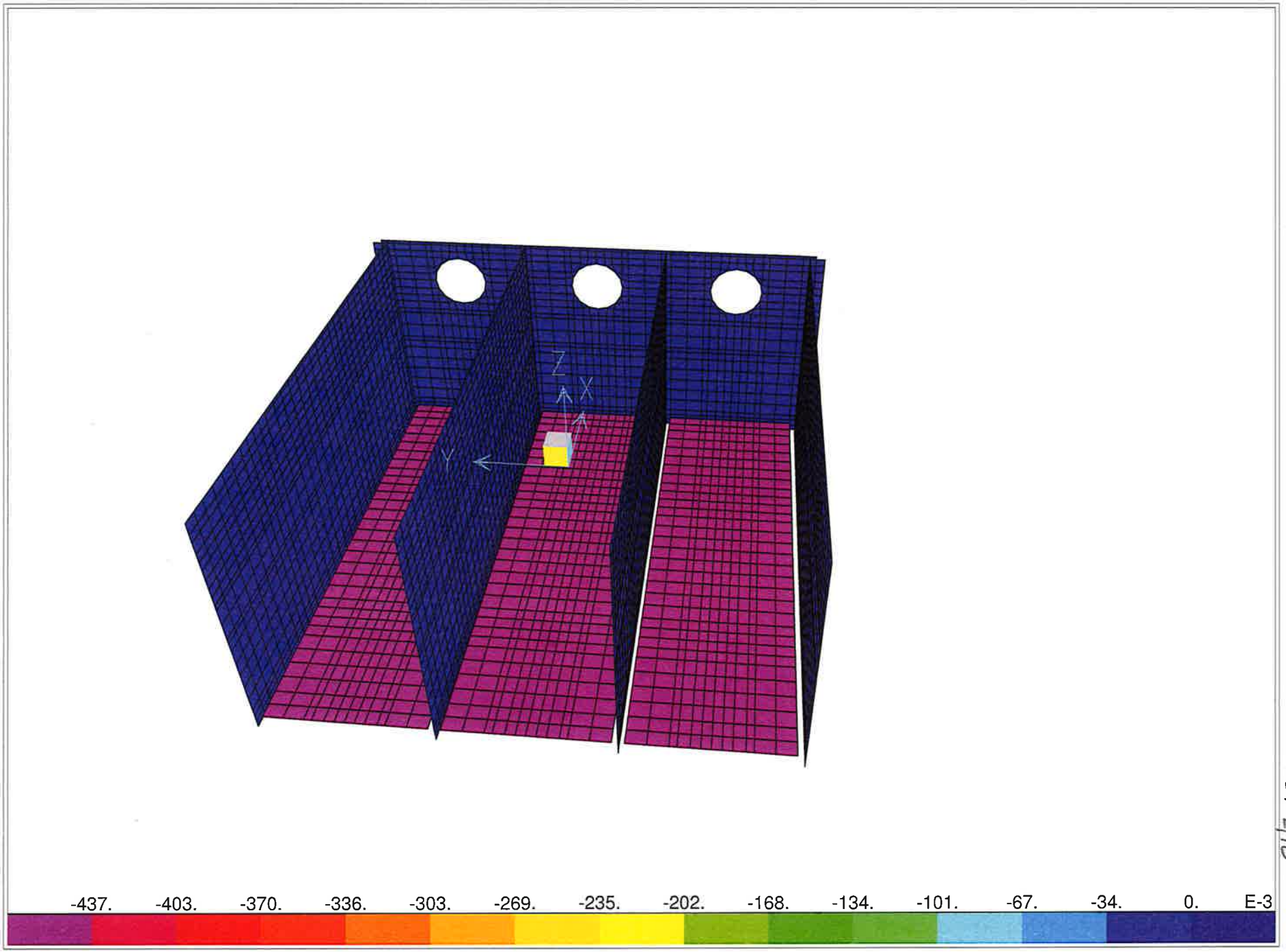
SAP2000

9/22/10 15:40:25



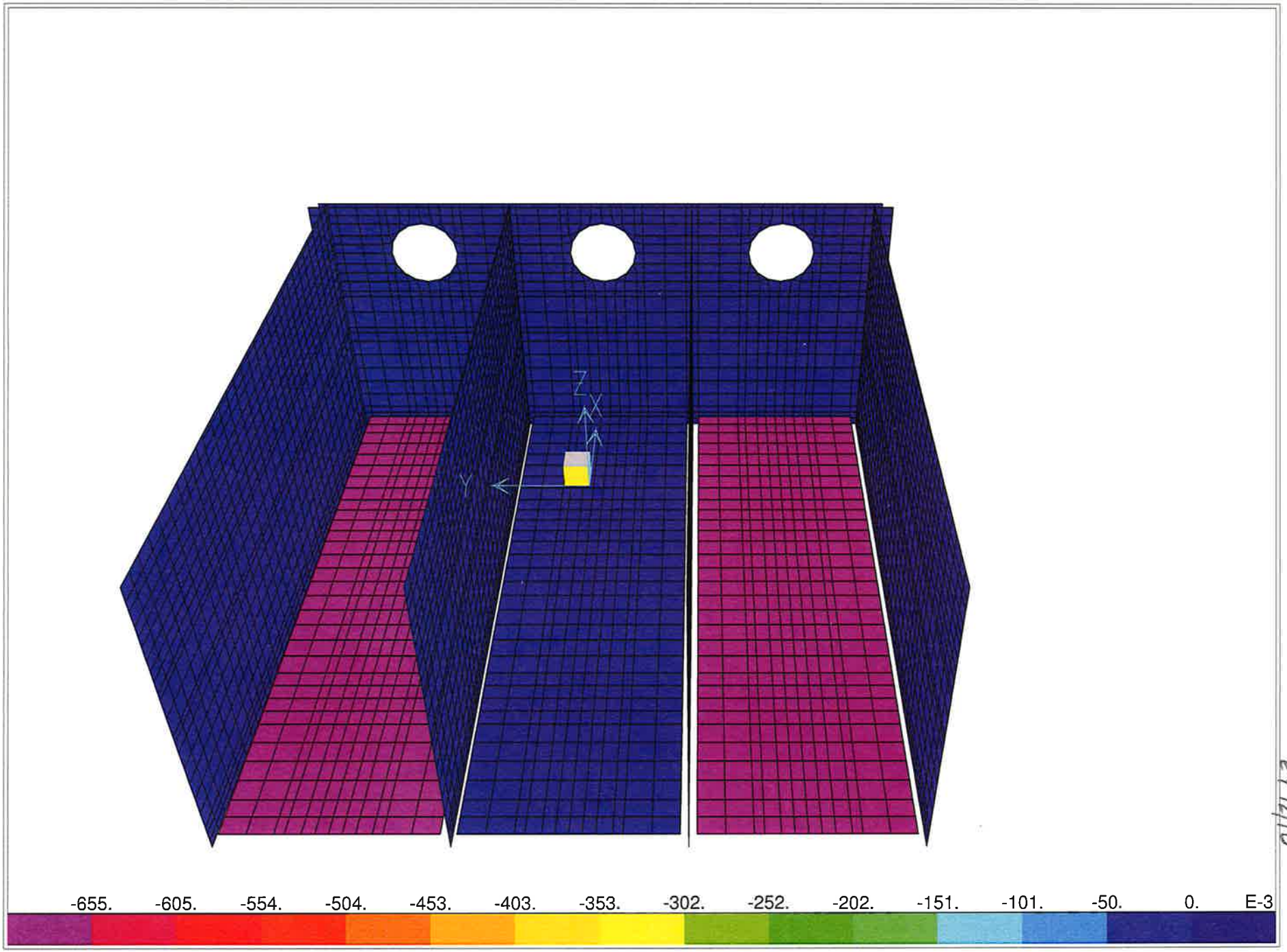
SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (H_0.5) (GLOBAL - Z) - Kip, ft, F Units

JY
E/12/10
(44)



SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (H_0.0) (GLOBAL - Z) - Kip, ft, F Units

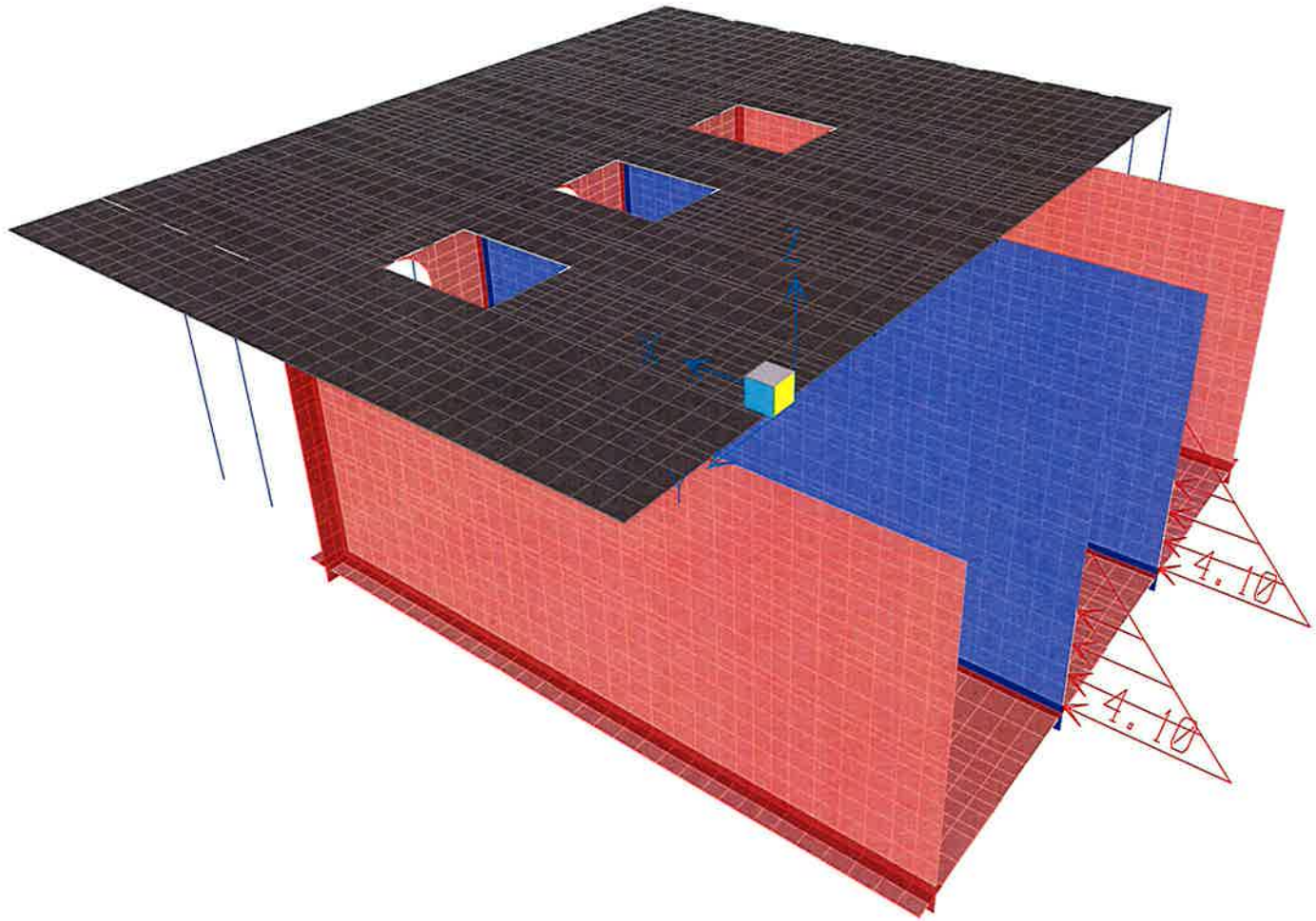
JY 9/12/10 (75)



SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (H_dewater) (GLOBAL - Z) - Kip, ft, F Units

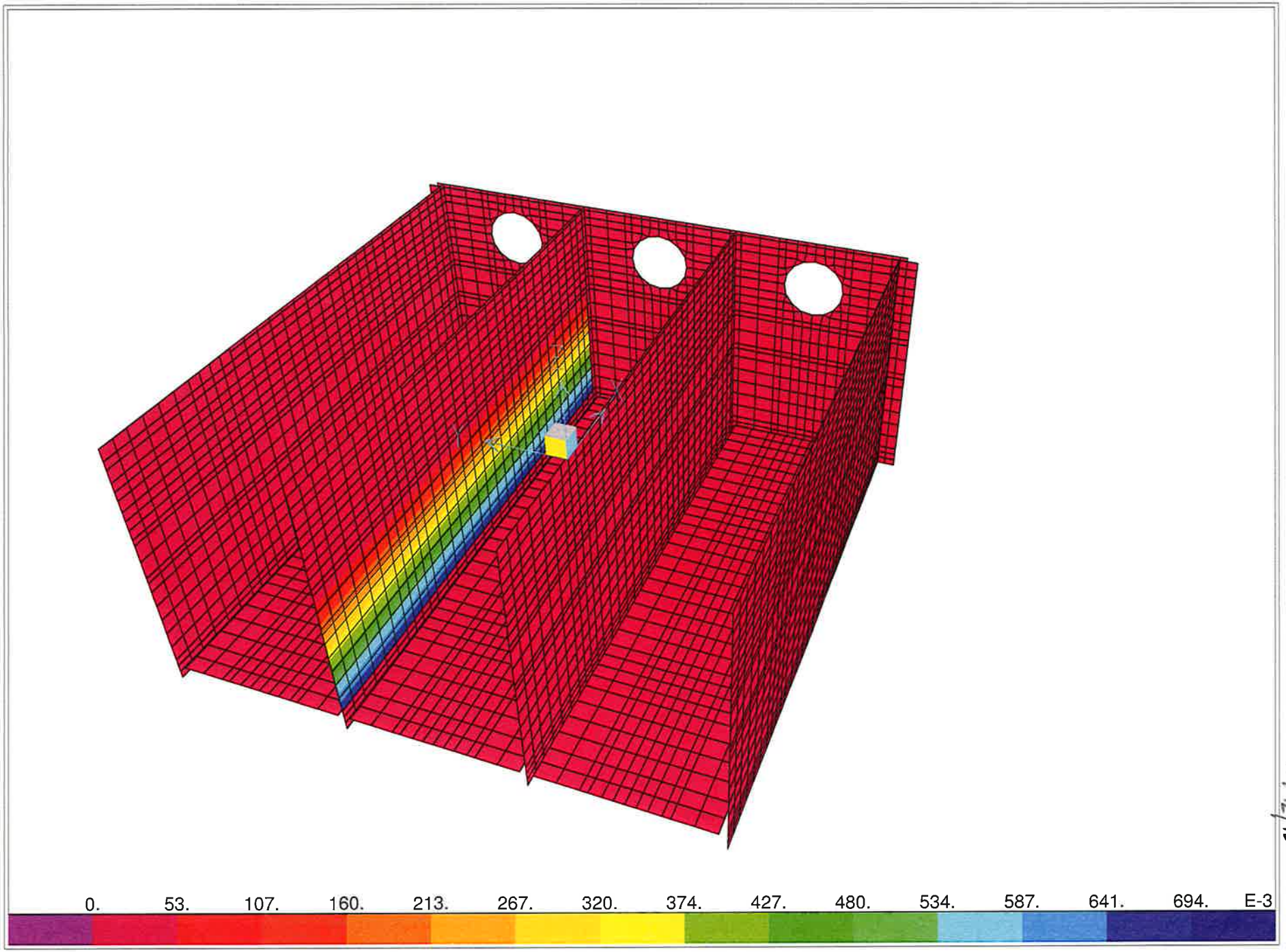
JY 9/21/10

(26)



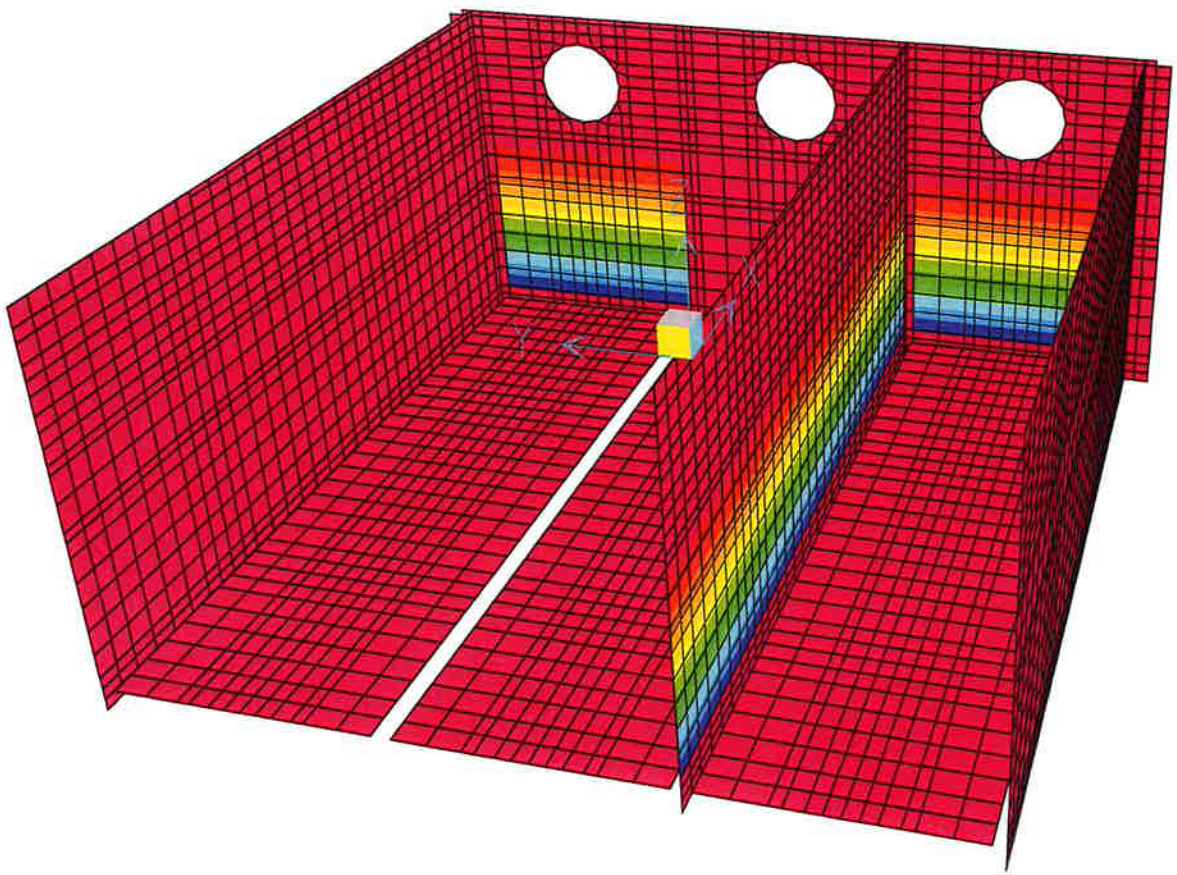
SAP2000 v14.2.3 - File:PS MODEL - Frame Span Loads (H_dewater) (As Defined) - Kip, ft, F Units

EY 1/1/10
5.1 (23)



SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Bottom (H_dewater) - Kip, ft, F Units

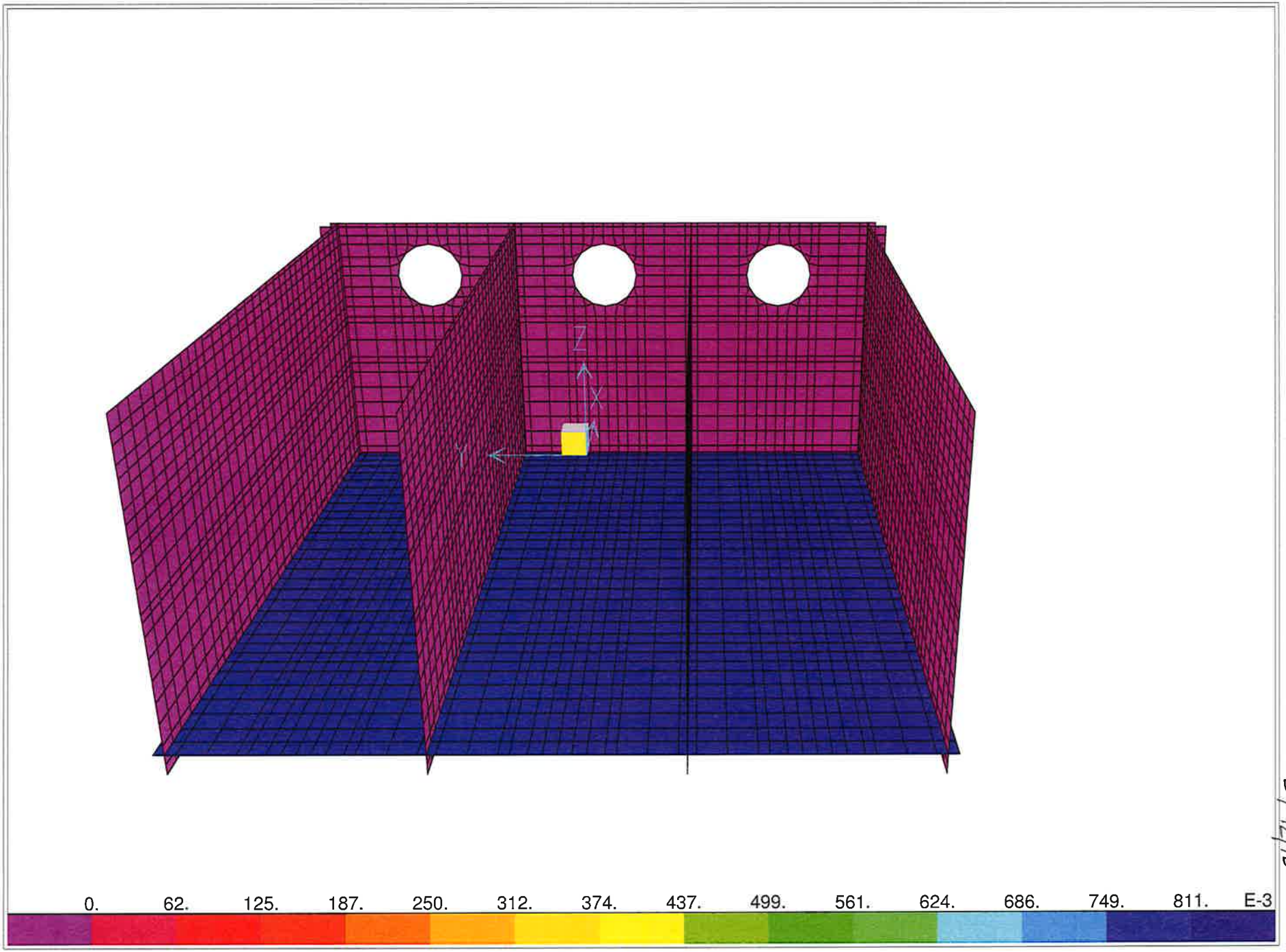
47 EY 12/10 48



0. 53. 107. 160. 213. 267. 320. 374. 427. 480. 534. 587. 641. 694. E-3

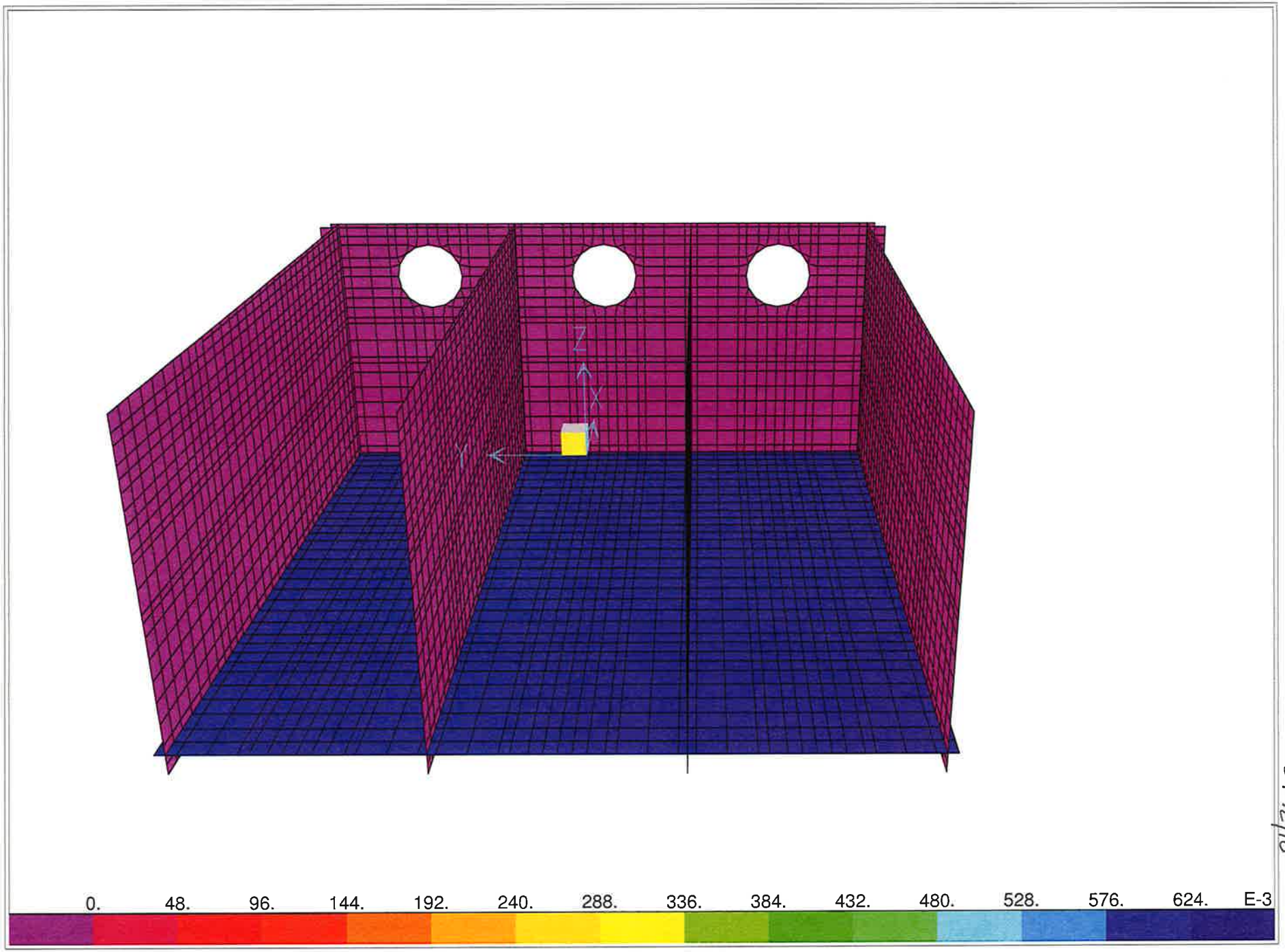
SAP2000 v12.0.1 - File:PS MODEL - Area Surface Pressure - Face Top (H_dewater) - Kip, ft, F Units

BY 2/17/10 (49)



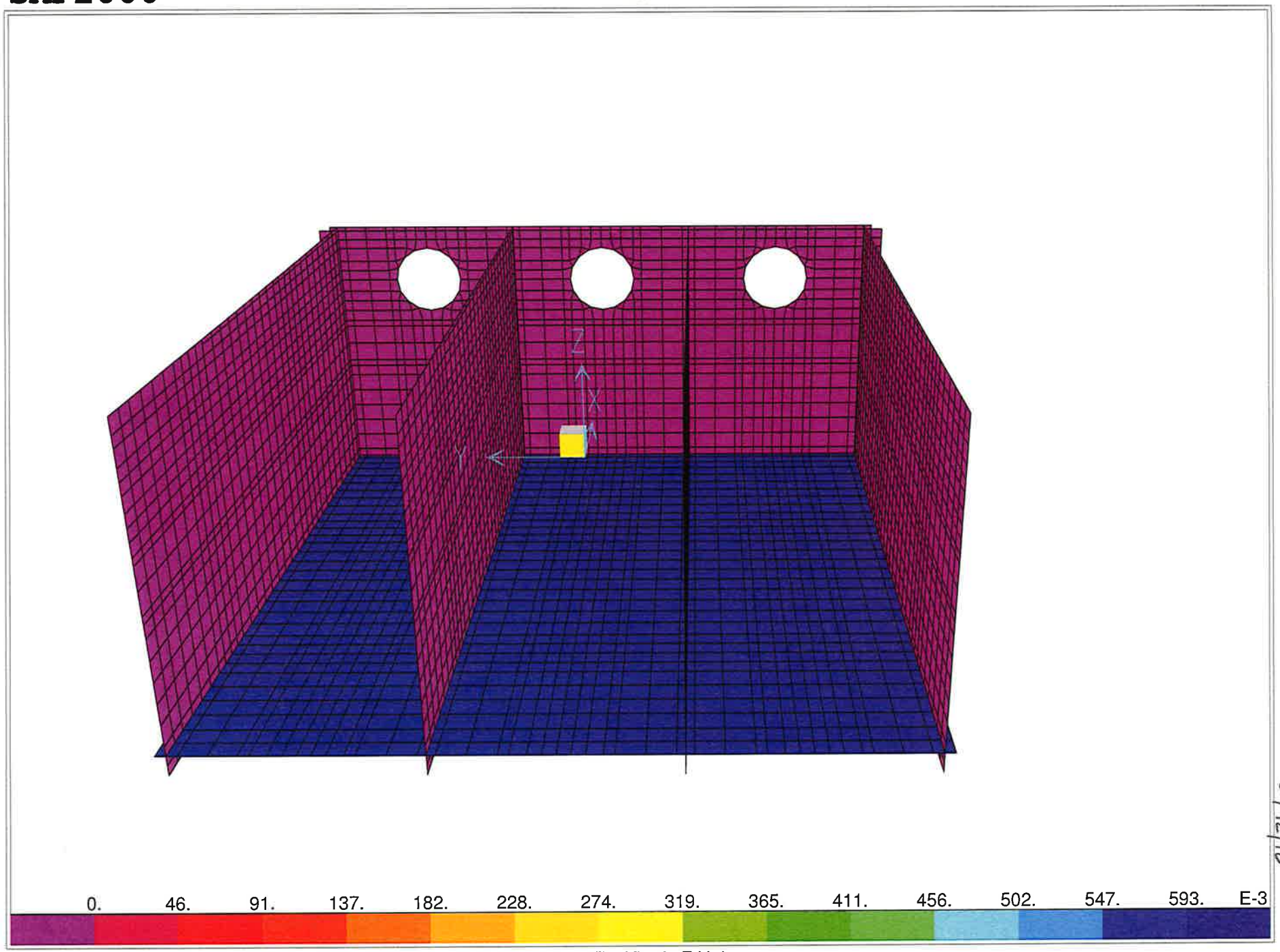
SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (U_3.5) (GLOBAL - Z) - Kip, ft, F Units

JA
EY 12/10
80



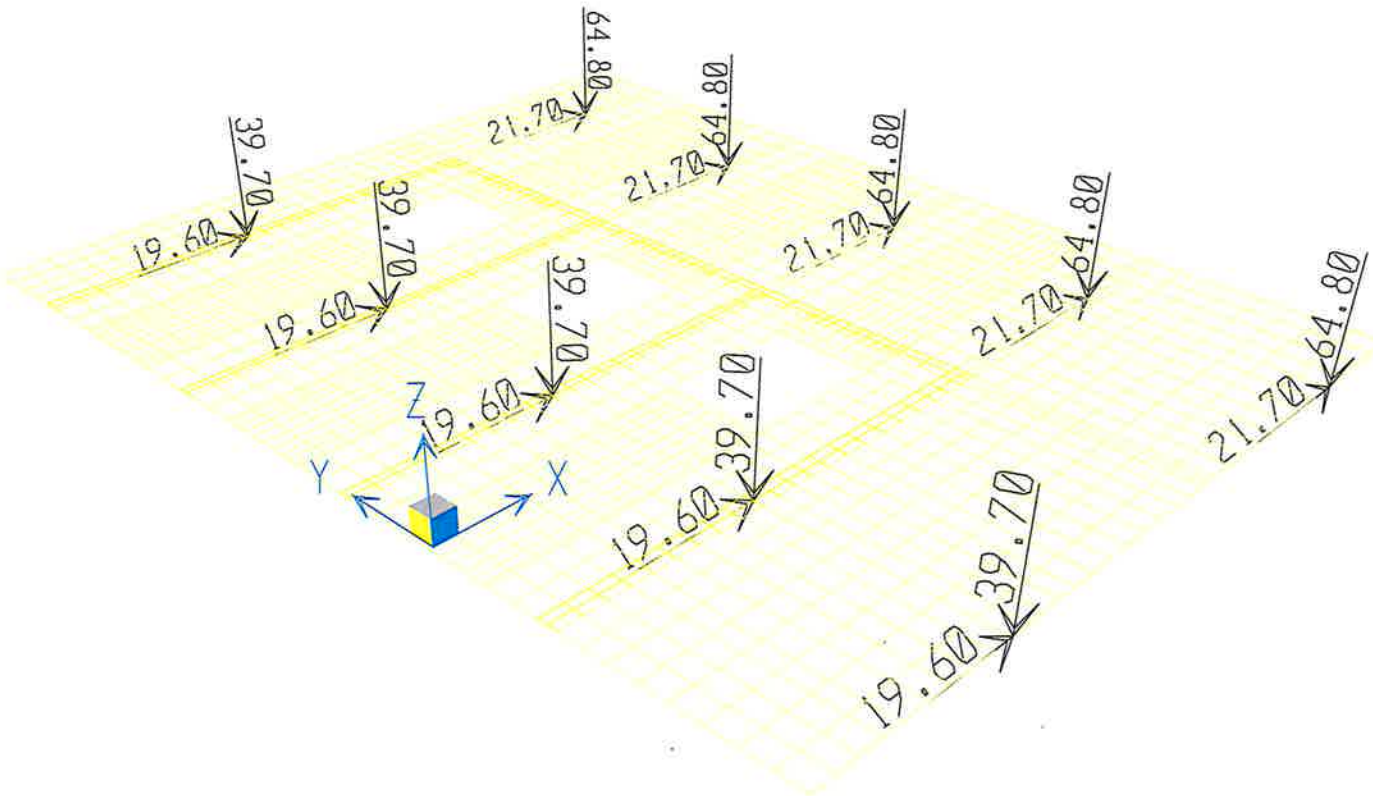
SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (U_0.5) (GLOBAL - Z) - Kip, ft, F Units

JY 9/21/10 (18)

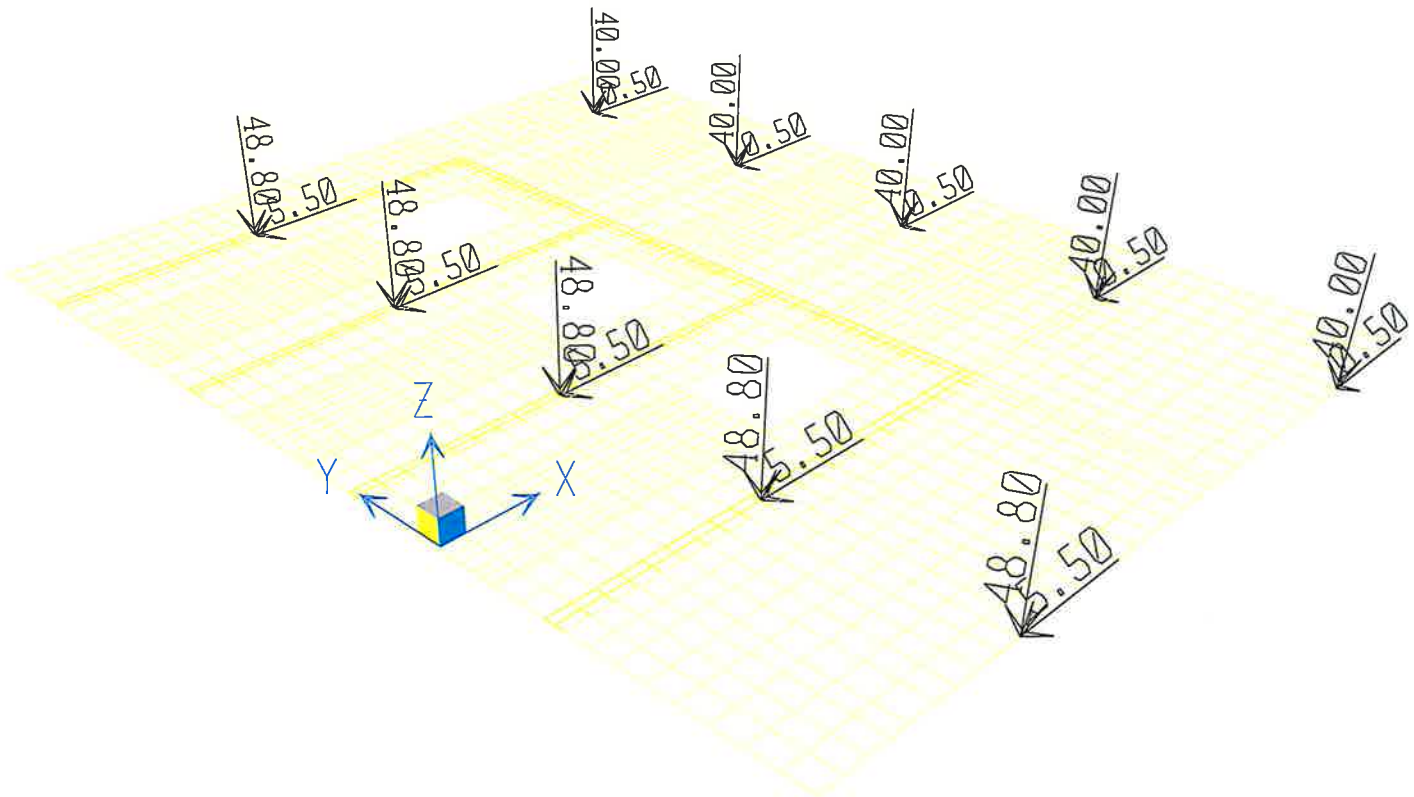


SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (U_0.0) (GLOBAL - Z) - Kip, ft, F Units

JY 9/12/10 (82)


39.70
19.60
39.70
19.60
39.70
19.60
64.80
21.70
64.80
21.70
64.80
21.70
64.80
21.70
64.80
21.70
39.70
19.60
39.70
19.60
Z
Y
X

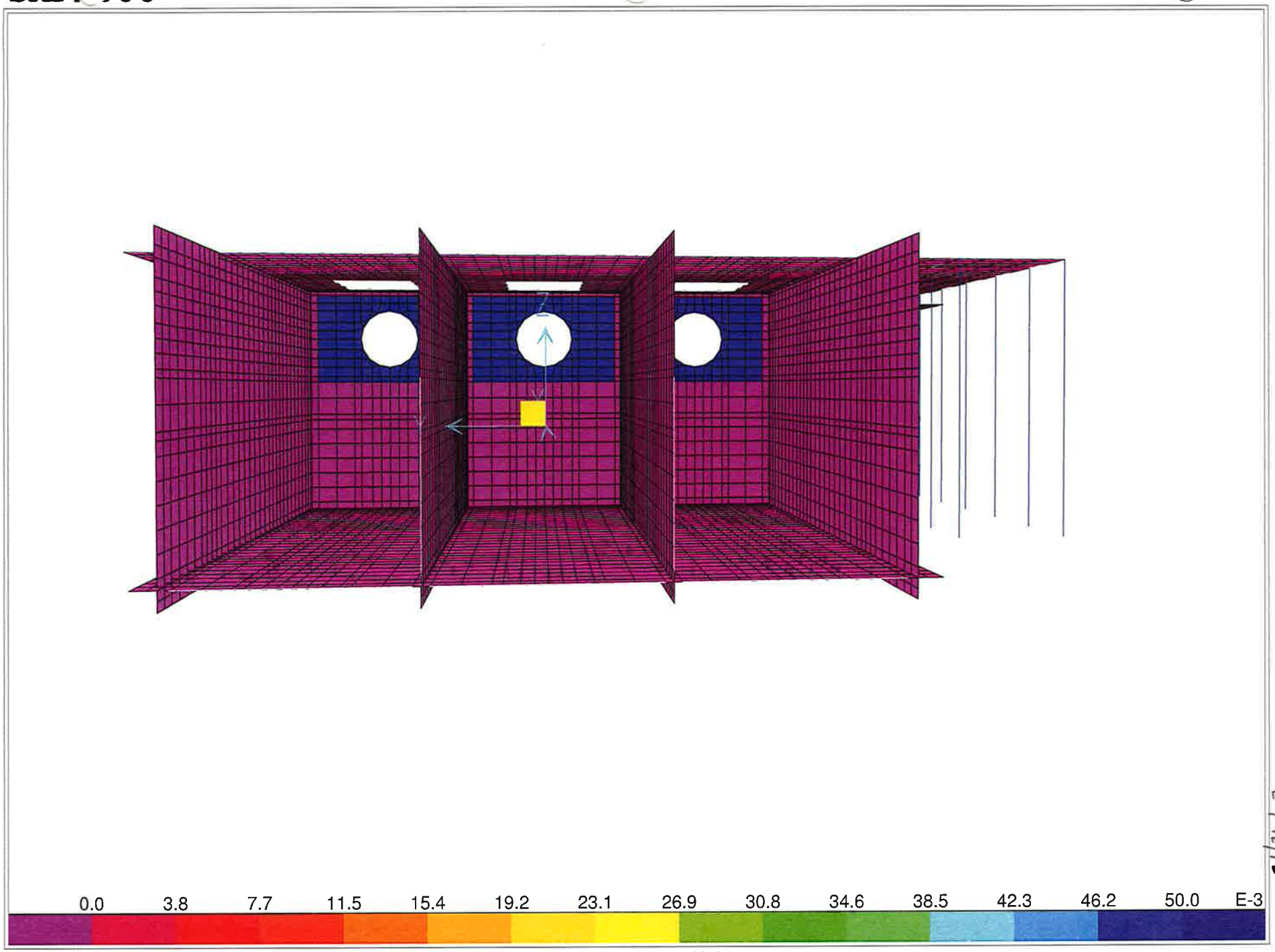
JY 1/6/10 (83)



(84) NOT USED

LY (85)

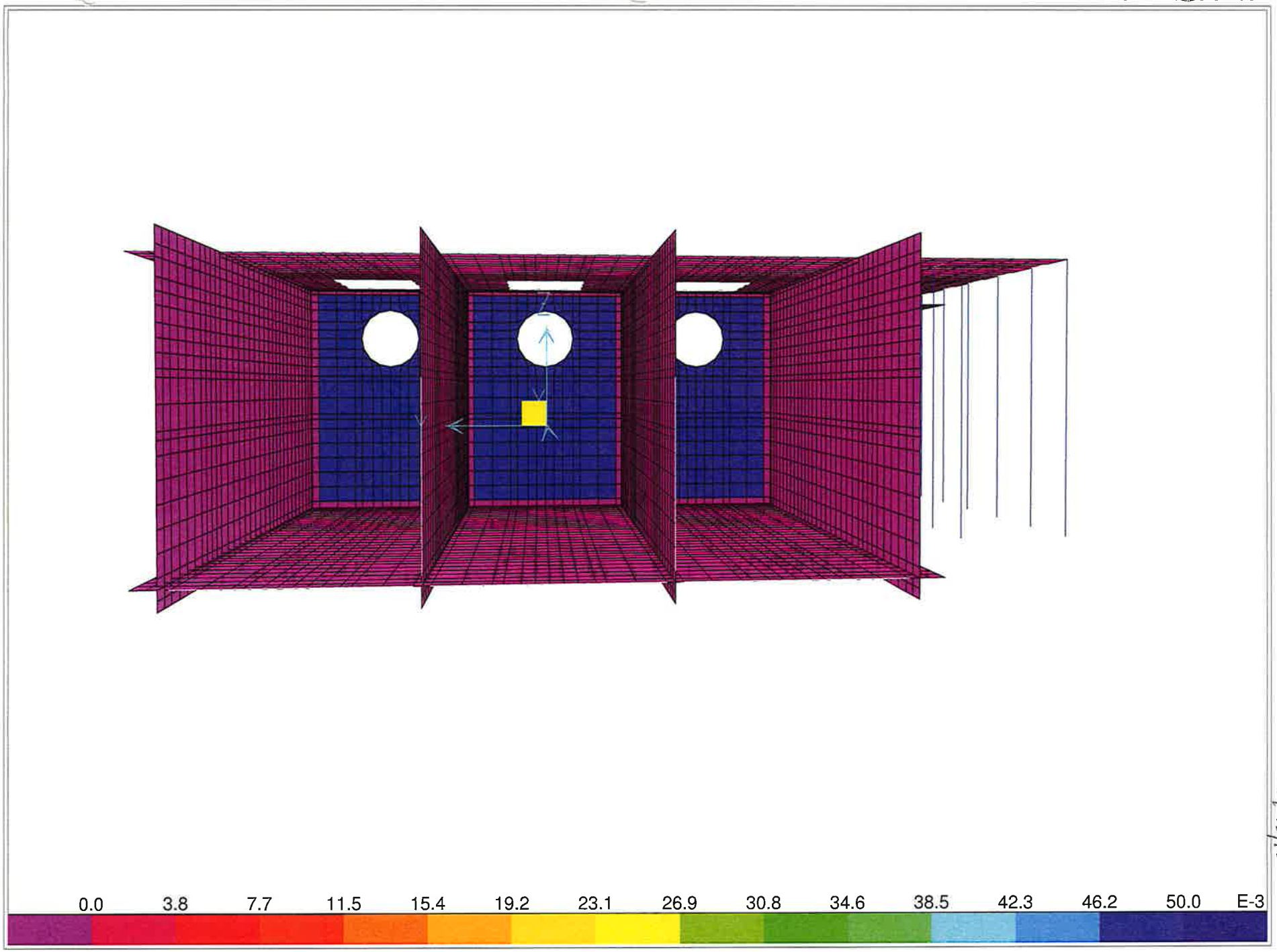
EY 12/10



SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (W_3.5) (GLOBAL - X) - Kip, ft, F Units

BY 12/10

LY (86)



SAP2000 v12.0.1 - File:PS MODEL - Area Uniform (W_all) (GLOBAL - X) - Kip, ft, F Units

JY
ey 12/10
R7

Job: Maurepas Pump Station
Job No.: 10001663
Description: Description of Load Case Combinations

Calculated By: JY
Date: 08/2010
Checked By: EY
Date: 12/2010

A1 (Construction Condition): (1.167)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- LS_200
- LS_300
- W_all
- R_PE

B1_s (Normal Operation Condition) (1.00)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_s
- S_0.5
- U_0.5
- H_0.5
- R_dead

B1_m (Normal Operation Condition) (1.00)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_m
- S_0.5
- U_0.5
- H_0.5
- R_dead

B3_s (High Head Condition): (1.00)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_s
- S_3.5
- U_3.5
- H_3.5
- R_dead

B3_m (High Head Condition): (1.00)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_m
- S_3.5
- U_3.5
- H_3.5
- R_dead

B4_s (Blocked Trash Rack Condition): (1.167)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_s
- L_rack
- S_0.5
- U_0.5
- H_0.5
- R_dead

B4_m (Blocked Trash Rack Condition): (1.167)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_m
- L_rack
- S_0.5
- U_0.5
- H_0.5
- R_dead

C1 (Hurricane Condition): (1.33)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- S_3.5
- U_3.5
- H_3.5
- R_PE
- W_3.5

Job: Maurepas Pump Station
Job No.: 10001663
Description: Description of Load Case Combinations

Calculated By: JY
Date: 08/2010
Checked By: EY
Date: 12/2010

D1_s (Maintenance Condition): (1.167)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_s
- S_0.0
- U_0.0
- H_0.0_exterior
- R_dead

D1_m (Maintenance Condition): (1.167)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_m
- S_0.0
- U_0.0
- H_0.0_exterior
- R_dead

D2_s (Dewatered Condition): (1.167)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_s
- S_3.5
- U_3.5
- H_dewater
- R_dead

D2_m (Dewatered Condition): (1.167)

- D
- D-E_pump_gear
- D-E_FSI
- D-E_engine
- L_floor
- L_truck_m
- S_3.5
- U_3.5
- H_dewater
- R_dead

Job: Maurepas Pump Station
Project No.: 10001663
Description: Load Combination Max/Min Envelope Definition

Computed by: JY
Date: 09/2010
Checked By: EY
Date: 12/10

TABLE: Combination Definitions					
ComboName	**ComboType**	**CaseType**	**CaseName**	**Overstress (OS)**	**ScaleFactor = (1.7x1.3)/OS**
Text	Text	Text	Text	Unitless	Unitless
COMB1	Envelope	Linear Static	A1	1.167	1.89
COMB1		Linear Static	B1_m	1	2.21
COMB1		Linear Static	B1_s	1	2.21
COMB1		Linear Static	B3_m	1	2.21
COMB1		Linear Static	B3_s	1	2.21
COMB1		Linear Static	B4_m	1.167	1.89
COMB1		Linear Static	B4_s	1.167	1.89
COMB1		Linear Static	C1	1.33	1.66
COMB1		Linear Static	D1_m	1.167	1.89
COMB1		Linear Static	D1_s	1.167	1.89
COMB1		Linear Static	D2_m	1.167	1.89
COMB1		Linear Static	D2_s	1.167	1.89

3.3

SAP Model Calibration

PROJECT # 10001663 JY 09/10 EY 12/10 (91)

TABLE: Base Reactions				
OutputCase	CaseType	GlobalFX	GlobalFY	GlobalFZ
Text	Text	Kip	Kip	Kip
D	LinStatic	0.0	0.0	2919.75

CHECK AGAINST ACTUAL DEAD LOAD :

Operating Floor - Thickness = 1.25 ft
Length = 65 ft
Width = 50.6 ft

Weight = 616.69 K

Beams -

	Width (ft)	Depth (ft)	Length (ft)
B1	2.167	0.75	56
B2	2.167	0.75	61
B3	2.167	0.75	12.167
B4	2.167	0.75	56
B5	3	1.147	51.66
B6	2.167	0.75	17.083
B7	2.167	0.75	10.33
B8	2.167	0.75	10.33
B9	2.167	0.75	49.5

Weight = 98.11 K

Notes: - Beam dimensions modified due to beam to Slab intersections.
- There are 3 B8 beams.

Walls -

	Width (ft)	Depth (ft)	Height (ft)
Outer Sump Wall	42.5	2.5	18.25
Inner Sump Wall	42.5	2	18.25
Head Wall	46	2.5	18.25

Weight = 1361.91 K

Note: There are 2 inner sump walls and 2 outer sump walls.

Base Slab - Thickness = 2.5 ft
Length = 46 ft
Width = 45 ft

Weight = 776.25 K

Piles above soil - Thickness = 1.167 ft
Length = 1.167 ft
Width = 7.25 ft

Weight = 34.04 K (23 Piles)

Therefore, Total Dead Load = 2887.00 K approx. = 2919.75 K, OK

Job: Maurepas Pump Station
Project No.: 10001663
Description: SAP Model check for Dead Load

Computed by: JY
Date: 10/2010
Checked By: EY
Date: 12/10

TABLE: Joint Reactions due to Dead Load case - D

Joint	F3	Joint	F3
Text	Kip	Text	Kip
P-1	34.35	P-41	42.06
P-2	34.30	P-42	42.44
P-3	34.24	P-43	42.81
P-4	34.18	P-44	43.17
P-5	34.11	P-45	43.49
P-6	34.07	P-46	43.87
P-7	34.05	P-47	44.57
P-8	34.09	P-48	45.68
P-9	35.23	P-49	45.64
P-10	35.31	P-50	45.99
P-11	35.37	P-51	46.33
P-12	35.41	P-52	46.67
P-13	35.42	P-53	47.00
P-14	35.50	P-54	47.30
P-15	35.78	P-55	47.69
P-16	36.39	P-56	48.14
P-17	37.42	P-57	8.98
P-18	37.58	P-58	9.76
P-19	37.74	P-59	11.02
P-20	37.91	P-60	12.72
P-21	38.07	P-61	13.61
P-22	38.23	P-62	12.92
P-23	38.48	P-63	10.93
P-24	38.76	P-64	9.81
P-25	38.26	P-65	33.23
P-26	38.53	P-66	33.13
P-27	38.80	P-67	34.23
P-28	39.06	P-68	36.94
P-29	39.28	P-69	36.60
P-30	39.57	P-70	28.32
P-31	40.13	P-71	22.57
P-32	41.04	P-72	41.90
P-33	40.59	P-73	33.27
P-34	40.91	P-74	50.46
P-35	41.26	P-75	41.72
P-36	41.62	P-76	54.99
P-37	42.00	P-77	43.33
P-38	42.36	P-78	59.59
P-39	42.85	P-79	45.29
P-40	43.35		

Total F3 = 2919.74 K

↑ EQUAL TO DEAD LOAD CALCULATED ON PG. 91.
∴ OK

(93) NOT USED

Job: Maurepas Pump Station
Project No.: 10001663
Description: SAP Model check for Live Load

Computed by: JY
Date: 10/2010
Checked By: EY
Date: 12/10

TABLE: Base Reactions				
OutputCase	CaseType	GlobalFX	GlobalFY	GlobalFZ
Text	Text	Kip	Kip	Kip
A1_a	LinStatic	-92.4	0.0	4580.8

TABLE: Joint Reactions					
Joint	OutputCase	CaseType	F1	F2	F3
Text	Text	Text	Kip	Kip	Kip
P-1	A1	LinStatic	-0.314	-0.132	38.844
P-2	A1	LinStatic	-0.314	-0.122	41.217
P-3	A1	LinStatic	-0.314	-0.113	43.584
P-4	A1	LinStatic	-0.314	-0.103	45.938
P-5	A1	LinStatic	-0.314	-0.094	48.278
P-6	A1	LinStatic	-0.314	-0.086	50.239
P-7	A1	LinStatic	-0.314	-0.076	52.64
P-8	A1	LinStatic	-0.314	-0.066	55.127
P-9	A1	LinStatic	-0.325	-0.131	41.104
P-10	A1	LinStatic	-0.325	-0.121	43.573
P-11	A1	LinStatic	-0.325	-0.112	46.009
P-12	A1	LinStatic	-0.325	-0.103	48.416
P-13	A1	LinStatic	-0.325	-0.093	50.753
P-14	A1	LinStatic	-0.325	-0.085	52.771
P-15	A1	LinStatic	-0.325	-0.076	55.435
P-16	A1	LinStatic	-0.325	-0.066	58.685
P-17	A1	LinStatic	-0.336	-0.13	44.251
P-18	A1	LinStatic	-0.336	-0.121	46.873
P-19	A1	LinStatic	-0.336	-0.111	49.496
P-20	A1	LinStatic	-0.336	-0.102	52.098
P-21	A1	LinStatic	-0.336	-0.093	54.678
P-22	A1	LinStatic	-0.336	-0.085	56.849
P-23	A1	LinStatic	-0.336	-0.076	59.523
P-24	A1	LinStatic	-0.336	-0.066	62.276
P-25	A1	LinStatic	-0.347	-0.129	45.727
P-26	A1	LinStatic	-0.347	-0.12	48.486
P-27	A1	LinStatic	-0.347	-0.111	51.223
P-28	A1	LinStatic	-0.347	-0.101	53.94
P-29	A1	LinStatic	-0.347	-0.092	56.597
P-30	A1	LinStatic	-0.347	-0.085	58.912
P-31	A1	LinStatic	-0.347	-0.075	62
P-32	A1	LinStatic	-0.347	-0.066	65.705
P-33	A1	LinStatic	-0.357	-0.128	48.732
P-34	A1	LinStatic	-0.357	-0.119	51.651
P-35	A1	LinStatic	-0.357	-0.11	54.583
P-36	A1	LinStatic	-0.357	-0.101	57.514
P-37	A1	LinStatic	-0.357	-0.092	60.453
P-38	A1	LinStatic	-0.358	-0.084	62.949
P-39	A1	LinStatic	-0.358	-0.075	66.033
P-40	A1	LinStatic	-0.358	-0.066	69.145
P-41	A1	LinStatic	-0.368	-0.127	50.801
P-42	A1	LinStatic	-0.368	-0.118	53.792
P-43	A1	LinStatic	-0.368	-0.109	56.759
P-44	A1	LinStatic	-0.369	-0.1	59.71

Job: Maurepas Pump Station
Project No.: 10001663
Description: SAP Model check for Live Load

Computed by: JY
Date: 10/2010
Checked By: EY
Date: 12/10

P-45	A1	LinStatic	-0.369	-0.091	62.604
P-46	A1	LinStatic	-0.369	-0.084	65.132
P-47	A1	LinStatic	-0.369	-0.075	68.515
P-48	A1	LinStatic	-0.369	-0.066	72.575
P-49	A1	LinStatic	-0.379	-0.127	54.959
P-50	A1	LinStatic	-0.379	-0.117	58.035
P-51	A1	LinStatic	-0.379	-0.108	61.112
P-52	A1	LinStatic	-0.379	-0.099	64.175
P-53	A1	LinStatic	-0.38	-0.09	67.227
P-54	A1	LinStatic	-0.38	-0.083	69.794
P-55	A1	LinStatic	-0.38	-0.075	72.936
P-56	A1	LinStatic	-0.38	-0.066	76.131
P-57	A1	LinStatic	-2.485	0.375	6.772
P-58	A1	LinStatic	-2.677	0.215	17.17
P-59	A1	LinStatic	-3.282	0.17	26.648
P-60	A1	LinStatic	-4.435	0.907	26.584
P-61	A1	LinStatic	-4.406	1.365	23.396
P-62	A1	LinStatic	-3.745	1.474	22.977
P-63	A1	LinStatic	-1.902	0.636	32.092
P-64	A1	LinStatic	-0.579	-0.221	56.967
P-65	A1	LinStatic	-3.112	1.242	39.415
P-66	A1	LinStatic	-3.446	0.874	45.733
P-67	A1	LinStatic	-3.504	0.762	51.622
P-68	A1	LinStatic	-3.6	1.13	57.626
P-69	A1	LinStatic	-4.563	2.189	58.571
P-70	A1	LinStatic	-3.801	1.058	56.507
P-71	A1	LinStatic	-1.415	0.349	70.021
P-72	A1	LinStatic	-4.281	0.412	82.729
P-73	A1	LinStatic	-1.663	0.967	94.256
P-74	A1	LinStatic	-3.685	-1.225	100.376
P-75	A1	LinStatic	-1.293	-1.179	117.825
P-76	A1	LinStatic	-4.599	-1.361	107.621
P-77	A1	LinStatic	-2.045	-1.551	121.869
P-78	A1	LinStatic	-5.718	-1.432	113.927
P-79	A1	LinStatic	-2.78	-1.703	123.579
TOTAL			**-92.4**	**0.0**	**4580.8**

Job: Maurepas Pump Station
Project No.: 10001663
Description: SAP Model check for Live Load

Computed by: JY
Date: 10/2010
Checked By: EY
Date: 12/10

Load Case A1_a:		FX	FY	FZ
D				2919.7
D-E_pump_gear				
Frame load of 0.42 k/ft applied over the openings				
= 3x0.42k/ftx(2x(6.5+9))'				39.1
D-E_FSI				
= 3x11k	(See Pg 51)			33.0
D-E_engine				
3 engines of 12000 lb each 3x12k	(See Pg 51)			36.0
LS_200				
0.2ksf x (areas of Floor slab & Base slab)				
= 0.2x(50.58x64-3x6.5x9+7.08x6.91+45x46)				1036.1
LS_300				
300psf surcharge load applied to the side walls and back wall. The loads on side walls balance out.				
Load on back wall				
= 0.24ksfx(4.0-(-9.5))'x46'		-149.0		
W_all				
50psf wind load applied on back wall in between the side walls.				
Load on back wall				
= 0.05ksfx(253.5+256+253.5)sft		38.2		
R_PE				
Reactions from pre-engineered building				
X direction force =	5x(19.6+21.7)k	206.5		
Z direction force =	5x(39.7+64.8)k			522.5
TOTAL (in Kips)		**95.6**	**0**	**4586.4**

SECTION 4

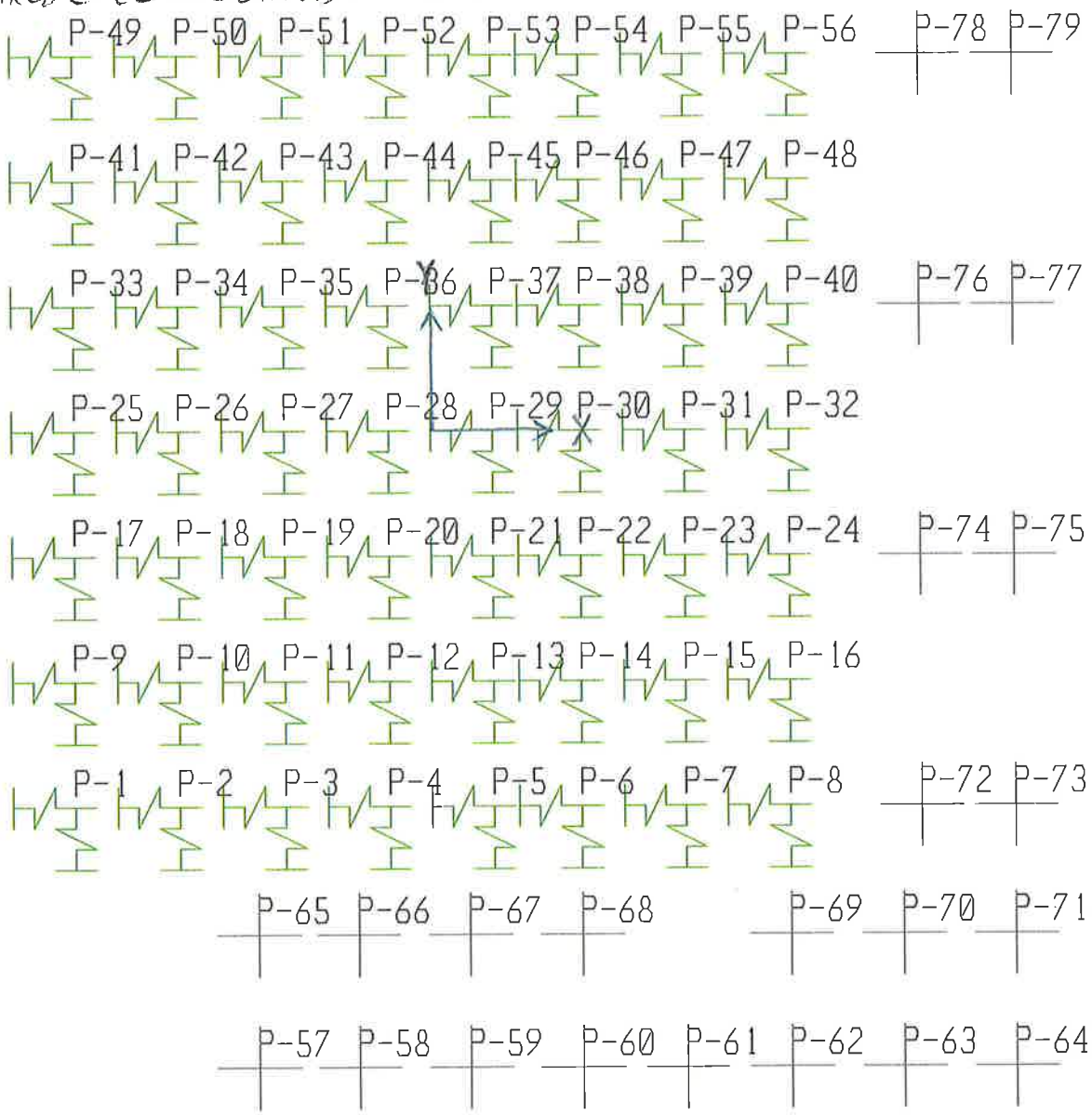
Pile Foundation Analysis & Design

JY (98)

EY 12/10

PUMP STATION PILE LAYOUT & PILE LABELS IN THE SAP MODEL:

- PILES P-1 THROUGH P-56 SHOWN AS SPRINGS ARE UNDER THE BASE SLAB @ EL -9.0.
- PILES P-57 THROUG P-79 ARE PILE COLUMNS OUTSIDE THE BASE SLAB FIXED @ EL 14.0 (GRADE). HENCE, THESE ARE SHOWN AS FIXED SUPPORTS.



- PILE LOCAL AXES: AXIS 1 PARALLEL TO X-AXIS
  AXIS 2 PARALLEL TO Y-AXIS
  AXIS 3 PARALLEL TO Z-AXIS (OUT OF PAGE)

CHECK FOR SPRING STIFFNESSES IN THE SAP MODEL:

- THE SPRING CONSTANTS FOR THE INITIAL RUN OF THE SAP MODEL:

  VERTICAL CONSTANT – BASED ON THE LINEAR ELASTIC SECTION OF THE T-Z CURVE FROM GEOTECHNICAL REPORT

  HORIZONTAL CONSTANT – BASED ON AN INITIAL ESTIMATE OF 0.5" TOTAL DEFLECTION.

→ $K_{VERTICAL} = 421.34$ K/IN

$K_{HORIZONTAL} = 2.95^{k}/_{0.5''} = 5.9$ K/IN (REFER ATTACHED SPREADSHEETS)

→ 1ST RUN: ACTUAL DISPLACEMENTS FROM THE MODEL WITH 'K' VALUES CALCULATED ABOVE

VERTICAL $\Delta V_{MAX} = 0.17''$ – VERY LOW OK KEEP $K_V = 421.34$ K/IN

HORIZONTAL $\Delta H_{MAX} = 0.64'' > 0.5''$ ASSUMED DEFLECTION

∴ FIND NEW 'K_H' FOR $\Delta H_{MAX} = 0.64''$

NEW 'K_H' FROM GEOTECHNICAL DATA $= 3.4^{K}/_{0.64''} = 5.3$ K/IN

(SEE ATTACHED SPREADSHEETS)

'ΔH_{MAX}' WITH NEW 'K_H' = 0.65" VERY CLOSE TO 0.64" ∴ OK

(SEE ATTACHED SPREADSHEET)

∴ SPRING STIFFNESS CONSTANTS USED ARE 'K_H' = 5.3 K/IN

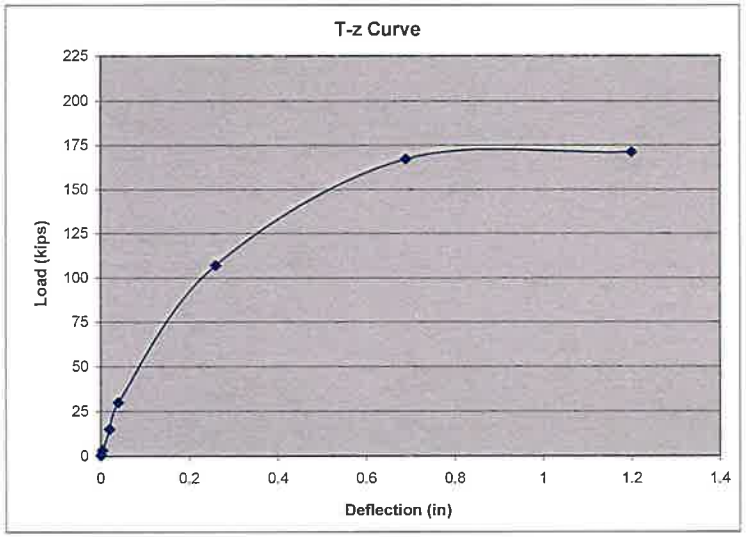
& 'K_V' = 421.34 K/IN

Job: Maurepas Pump Station
Project No.: 10001663
Description: P-y and T-z curves for Maurepas Pump Station Location

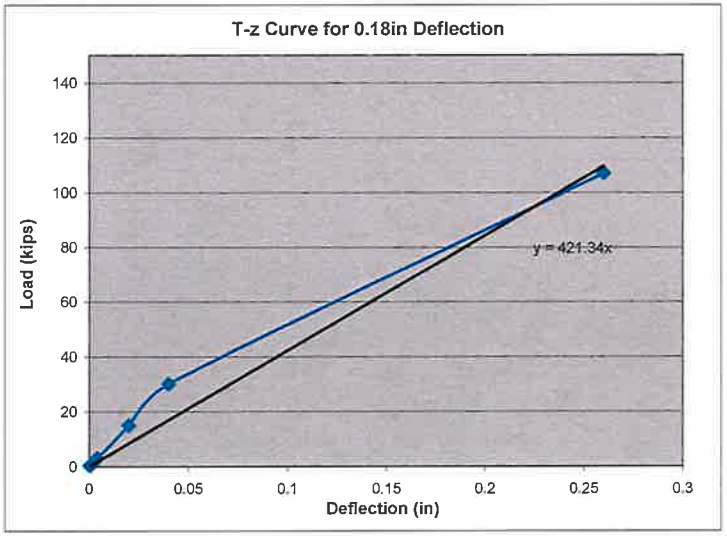
Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

Relationship Between Applied Axial Load and Vertical Pile Butt Deflection for 14 Inch Precast Concrete Piles (from geotechnical report Sep 19th, 2008)

Applied Axial Load (kips)	Axial Butt Deflection (in)
0.3	0.0004
3	0.004
15	0.02
30	0.04
107	0.26
167	0.69
171	1.2



Slope of elastic section of T-z curve:

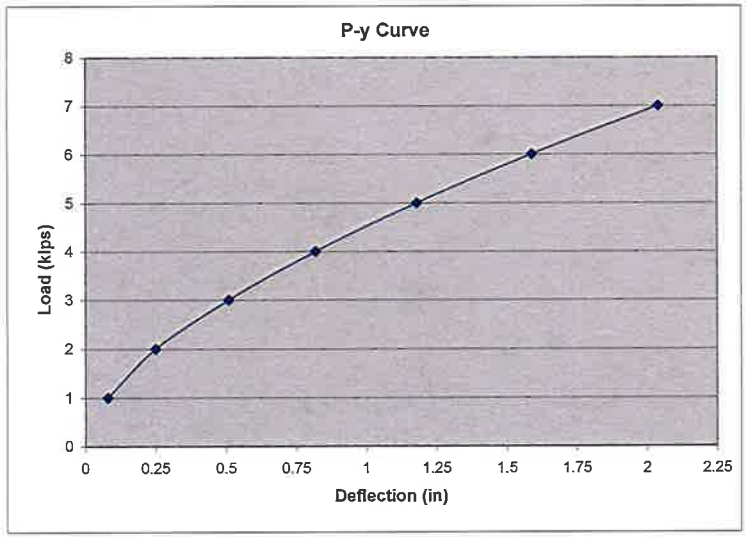


Job: Maurepas Pump Station
Project No.: 10001663
Description: P-y and T-z curves for Maurepas Pump Station Location

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

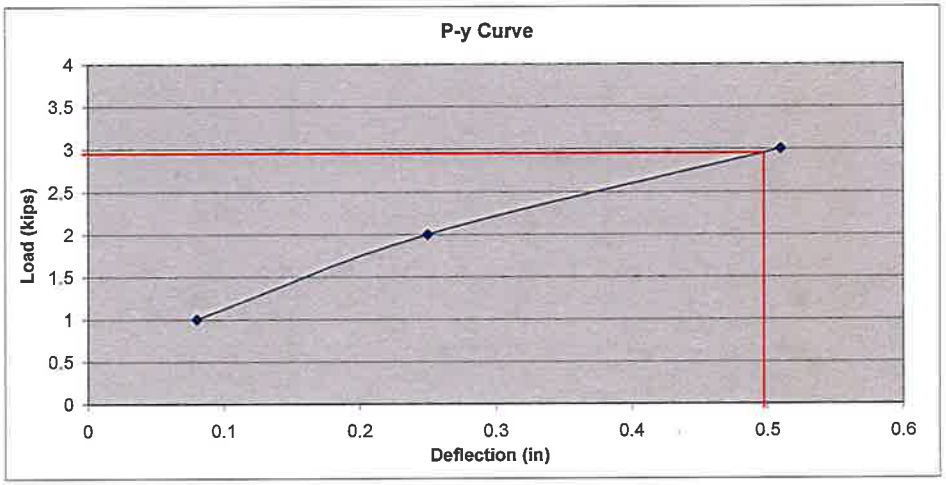
Relationship Between Applied Lateral Load and Horizontal Pile Butt Deflection for 14 Inch Precast Concrete Piles 24 - 79 Free-head condition (from geotechnical report Sep 19th, 2008):

Applied Lateral Load (kips)	Lateral Butt Deflection (in)
1	0.08
2	0.25
3	0.51
4	0.82
5	1.18
6	1.59
7	2.04



For a horizontal deflection of 0.5in:

* 0.5 inch deflection yeilds 2.95 kip lateral load

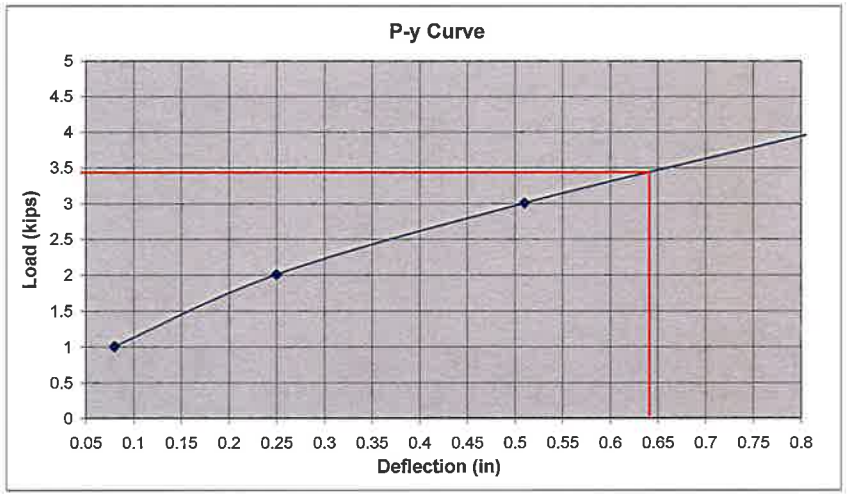


Job: Maurepas Pump Station
Project No.: 10001663
Description: P-y and T-z curves for Maurepas Pump Station Location

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

For a horizontal deflection of 0.64 in:

* 0.64 inch deflection yeilds 3.4 kip lateral load



Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Pile Displacements from SAP model

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

SUMMARY:

U1 Maximum = 0.0992 in
U1 Minimum = -0.6484 in
U2 Maximum = 0.0534 in
U2 Minimum = -0.2714 in
U3 Maximum = -0.0148 in
U3 Minimum = -0.1697 in

** Maximum Lateral Displacement = 0.65 inches
** Maximum Vertical Displacement = 0.17 inches

	Data					
OutputCase	**Max of U1**	**Min of U1**	**Max of U2**	**Min of U2**	**Max of U3**	**Min of U3**
A1_a	0.0992	0.0766	0.0313	0.0093	-0.0950	-0.1680
A1_b	0.0805	0.0658	0.0245	0.0099	-0.0956	-0.1697
B1_m	-0.2111	-0.4425	0.0403	-0.1810	-0.0577	-0.1610
B1_s	-0.2109	-0.4424	0.0412	-0.1802	-0.0591	-0.1601
B3_m	-0.1572	-0.3263	0.0319	-0.1298	-0.0609	-0.1527
B3_s	-0.1570	-0.3262	0.0329	-0.1290	-0.0623	-0.1517
B4_m	-0.1879	-0.3935	0.0370	-0.1598	-0.0624	-0.1589
B4_s	-0.1877	-0.3934	0.0379	-0.1590	-0.0639	-0.1580
C1_a	0.0234	-0.0115	0.0142	-0.0197	-0.0527	-0.1432
C1_b	0.0126	-0.0302	0.0149	-0.0265	-0.0532	-0.1449
D1_m	-0.3099	-0.6484	0.0524	-0.2714	-0.0148	-0.1487
D1_s	-0.3097	-0.6483	0.0534	-0.2706	-0.0163	-0.1477
D2_m	-0.0601	-0.1133	0.0156	-0.0359	-0.0550	-0.1326
D2_s	-0.0599	-0.1132	0.0165	-0.0351	-0.0561	-0.1316

JY 09/10
EY 12/10

SUMMARY:

Maximum F1 =	26.086	kips
Minimum F1 =	-4.901	kips
Maximum F2 =	7.747	kips
Minimum F2 =	-8.160	kips
Maximum F3 =	105.922	kips
Minimum F3 =	4.473	kips
Maximum M1 =	80.620	kip-in
Minimum M1 =	-77.671	kip-in
Maximum M2 =	266.343	kip-in
Minimum M2 =	-47.678	kip-in
Maximum M3 =	0.108	kip-in
Minimum M3 =	-1.910	kip-in

** Maximum Lateral Reaction = 26.09 kips
** Maximum Vertical Reaction = 105.92 kips
** Maximum Bending Moment = 22.2 kip-ft

REACTIONS OUTPUT FROM SAP MODEL:

Output case		Overstress	Force W/OS
A1			
Max of F1	-0.314	0.1667	-0.27
Min of F1	-5.718	0.1667	-4.90
Max of F2	2.189	0.1667	1.88
Min of F2	-1.703	0.1667	-1.46
Max of F3	123.579	0.1667	105.92
Min of F3	6.772	0.1667	5.80
Max of M1	17.004	0.1667	14.57
Min of M1	-6.6192	0.1667	-5.67
Max of M2	0	0.1667	0.00
Min of M2	-55.6261	0.1667	-47.68
Max of M3	0.1257	0.1667	0.11
Min of M3	0	0.1667	0.00

CONTINUED. . . .

JY 09/10
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Output case		Overstress	Force W/OS
B1_m			
Max of F1	20.989	0	20.99
Min of F1	1.129	0	1.13
Max of F2	6.968	0	6.97
Min of F2	-7.187	0	-7.19
Max of F3	100.744	0	100.74
Min of F3	22.212	0	22.21
Max of M1	71.2619	0	71.26
Min of M1	-63.8767	0	-63.88
Max of M2	213.5715	0	213.57
Min of M2	0	0	0.00
Max of M3	0	0	0.00
Min of M3	-1.5078	0	-1.51
B1_s			
Max of F1	21.004	0	21.00
Min of F1	1.128	0	1.13
Max of F2	7.003	0	7.00
Min of F2	-7.194	0	-7.19
Max of F3	100.385	0	100.39
Min of F3	22.232	0	22.23
Max of M1	71.3133	0	71.31
Min of M1	-64.0177	0	-64.02
Max of M2	213.6055	0	213.61
Min of M2	0	0	0.00
Max of M3	0	0	0.00
Min of M3	-1.5052	0	-1.51
B3_m			
Max of F1	15.956	0	15.96
Min of F1	0.845	0	0.85
Max of F2	5.293	0	5.29
Min of F2	-5.62	0	-5.62
Max of F3	98.137	0	98.14
Min of F3	22.789	0	22.79
Max of M1	55.8047	0	55.80
Min of M1	-46.2999	0	-46.30
Max of M2	159.4778	0	159.48
Min of M2	0	0	0.00
Max of M3	0	0	0.00
Min of M3	-1.0931	0	-1.09
B3_s			
Max of F1	15.971	0	15.97
Min of F1	0.844	0	0.84
Max of F2	5.328	0	5.33
Min of F2	-5.627	0	-5.63
Max of F3	97.777	0	97.78
Min of F3	22.851	0	22.85
Max of M1	55.8562	0	55.86
Min of M1	-46.4409	0	-46.44
Max of M2	159.5118	0	159.51
Min of M2	0	0	0.00
Max of M3	0	0	0.00
Min of M3	-1.0906	0	-1.09

CONTINUED....

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B4_m			
Max of F1	18.883	0.167	16.18
Min of F1	1.007	0.167	0.86
Max of F2	6.352	0.167	5.44
Min of F2	-6.565	0.167	-5.63
Max of F3	100.088	0.167	85.77
Min of F3	22.715	0.167	19.46
Max of M1	65.1379	0.167	55.82
Min of M1	-56.966	0.167	-48.81
Max of M2	191.1335	0.167	163.78
Min of M2	0	0.167	0.00
Max of M3	0	0.167	0.00
Min of M3	-1.3353	0.167	-1.14
B4_s			
Max of F1	18.898	0.167	16.19
Min of F1	1.006	0.167	0.86
Max of F2	6.387	0.167	5.47
Min of F2	-6.572	0.167	-5.63
Max of F3	99.728	0.167	85.46
Min of F3	22.735	0.167	19.48
Max of M1	65.1893	0.167	55.86
Min of M1	-57.107	0.167	-48.93
Max of M2	191.1676	0.167	163.81
Min of M2	0	0.167	0.00
Max of M3	0	0.167	0.00
Min of M3	-1.3328	0.167	-1.14
C1			
Max of F1	3.38	0.333	2.54
Min of F1	-3.479	0.333	-2.61
Max of F2	1.878	0.333	1.41
Min of F2	-2.615	0.333	-1.96
Max of F3	121.069	0.333	90.82
Min of F3	10.208	0.333	7.66
Max of M1	26.1552	0.333	19.62
Min of M1	-10.4862	0.333	-7.87
Max of M2	21.5044	0.333	16.13
Min of M2	-32.8751	0.333	-24.66
Max of M3	0	0.333	0.00
Min of M3	-0.2867	0.333	-0.22

CONTINUED...

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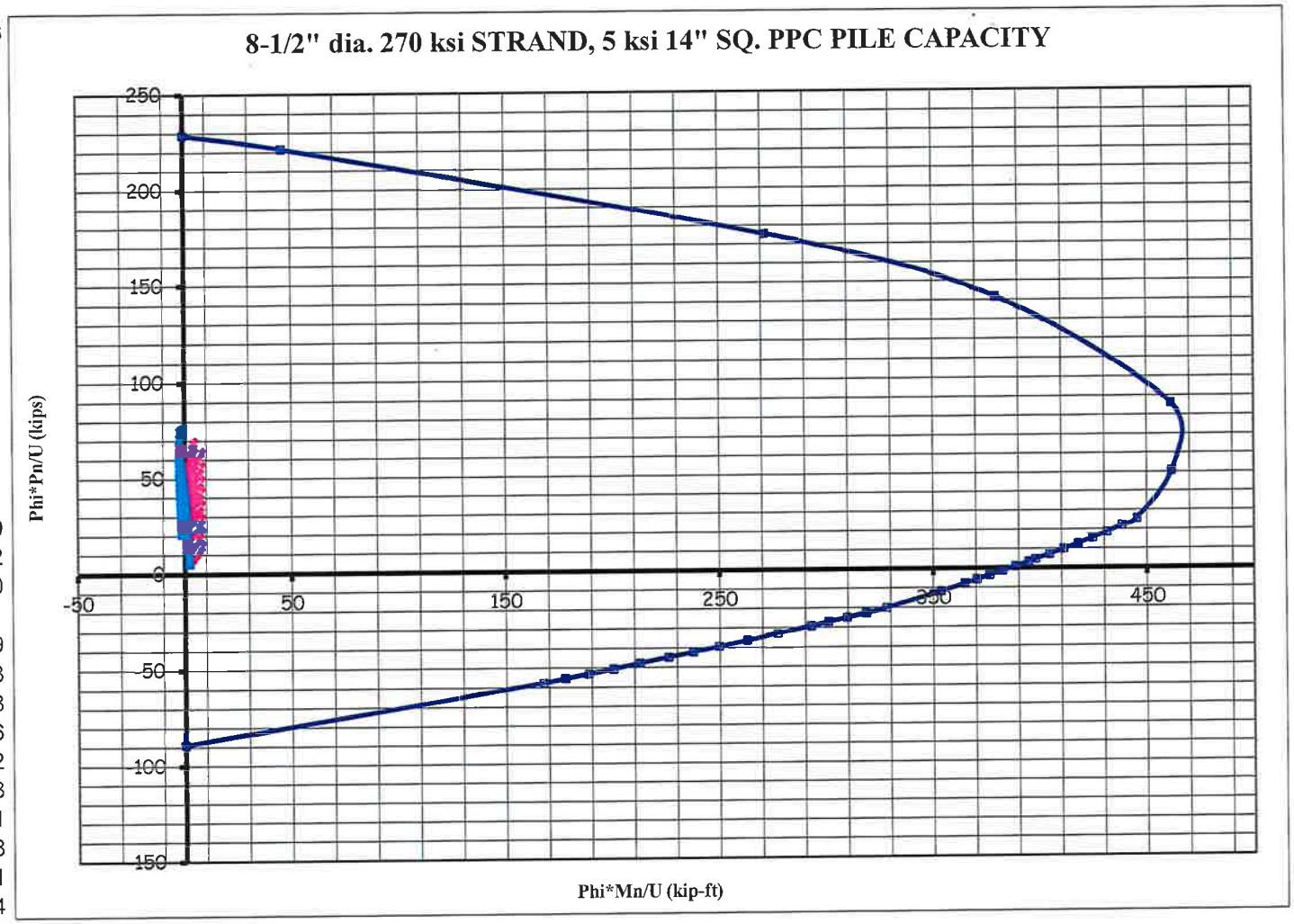
D1_m			
Max of F1	30.422	0.167	26.07
Min of F1	1.652	0.167	1.42
Max of F2	9.006	0.167	7.72
Min of F2	-9.516	0.167	-8.15
Max of F3	95.439	0.167	81.78
Min of F3	5.22	0.167	4.47
Max of M1	94.0325	0.167	80.58
Min of M1	-90.5017	0.167	-77.55
Max of M2	310.7546	0.167	266.29
Min of M2	0	0.167	0.00
Max of M3	0	0.167	0.00
Min of M3	-2.2286	0.167	-1.91
D1_s			
Max of F1	30.442	0.167	26.09
Min of F1	1.651	0.167	1.41
Max of F2	9.041	0.167	7.75
Min of F2	-9.523	0.167	-8.16
Max of F3	95.079	0.167	81.47
Min of F3	5.822	0.167	4.99
Max of M1	94.0839	0.167	80.62
Min of M1	-90.6426	0.167	-77.67
Max of M2	310.8227	0.167	266.34
Min of M2	0	0.167	0.00
Max of M3	0	0.167	0.00
Min of M3	-2.2261	0.167	-1.91
D2_m			
Max of F1	6.645	0.167	5.69
Min of F1	0.332	0.167	0.28
Max of F2	1.87	0.167	1.60
Min of F2	-2.624	0.167	-2.25
Max of F3	98.624	0.167	84.51
Min of F3	16.138	0.167	13.83
Max of M1	26.2057	0.167	22.46
Min of M1	-11.9017	0.167	-10.20
Max of M2	58.7168	0.167	50.31
Min of M2	0	0.167	0.00
Max of M3	0	0.167	0.00
Min of M3	-0.326	0.167	-0.28
D2_s			
Max of F1	6.66	0.167	5.71
Min of F1	0.331	0.167	0.28
Max of F2	1.905	0.167	1.63
Min of F2	-2.631	0.167	-2.25
Max of F3	98.494	0.167	84.40
Min of F3	16.197	0.167	13.88
Max of M1	26.2608	0.167	22.50
Min of M1	-12.0426	0.167	-10.32
Max of M2	58.7508	0.167	50.34
Min of M2	0	0.167	0.00
Max of M3	0	0.167	0.00
Min of M3	-0.3239	0.167	-0.28

Job: Maurepas Pump Station
Description: Pile interaction diagram with Moments vs. Axial forces graphed.

Project No.: 1[illegible]01663
Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

PHI=0.7
U=1.7*1.3

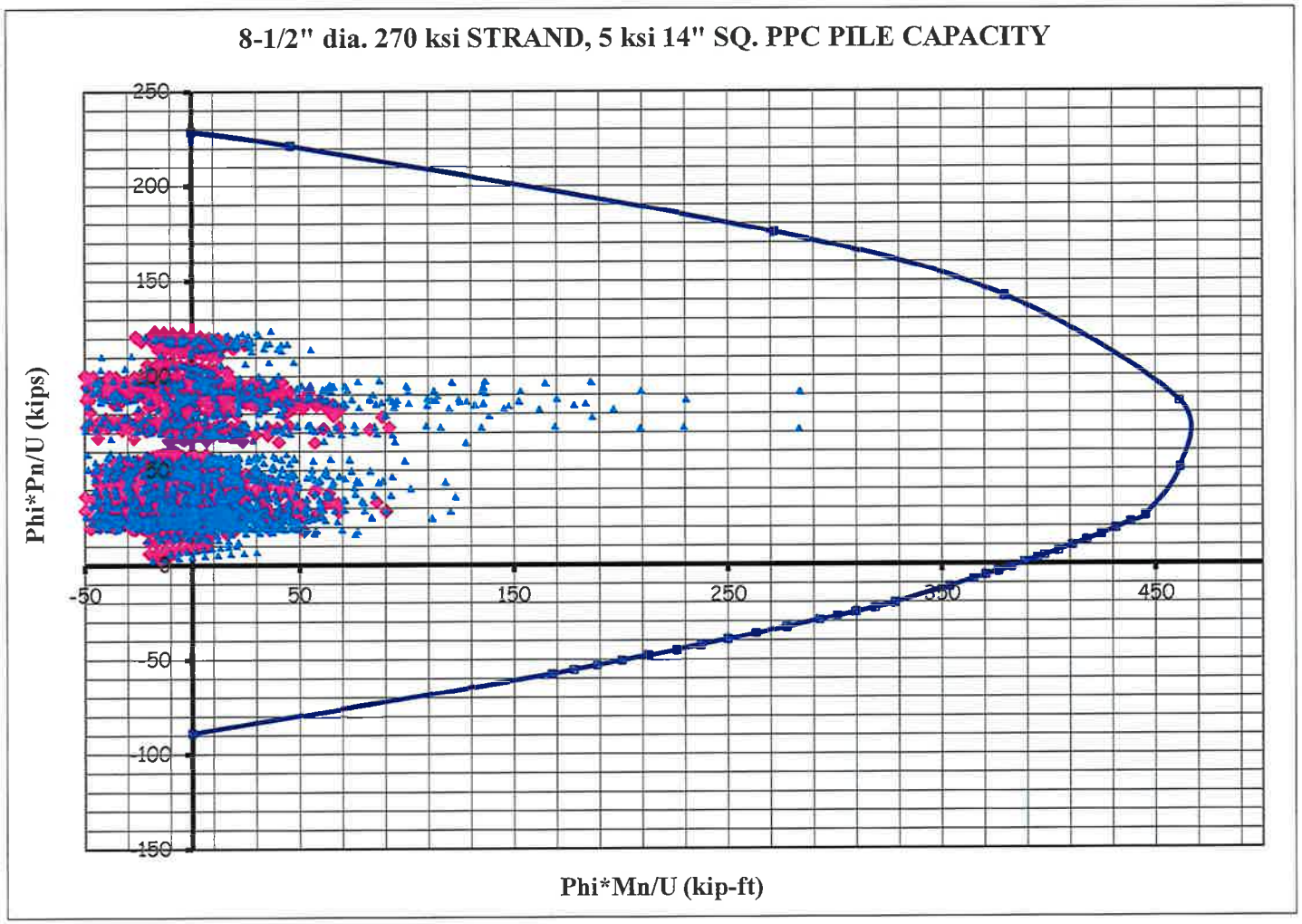
PHI*Mn/U	PHI*Pn/U
0.00	228.67
46.04	221.30
271.83	175.17
379.56	141.48
461.21	85.87
461.57	50.70
445.23	25.38
438.31	21.88
431.41	18.58
424.55	15.44
417.77	12.47
411.09	9.64
404.51	6.95
398.05	4.37
394.86	3.13
388.59	0.72
382.44	-1.59
376.43	-3.81
370.56	-5.94
364.81	-8.00
353.71	-11.90
328.13	-20.32
318.69	-22.80
309.68	-25.11
301.07	-27.29
292.82	-29.33
277.33	-33.08
263.05	-36.46
249.82	-39.52
237.52	-42.33
226.03	-44.91
212.66	-47.88
200.26	-50.61
188.68	-53.14
177.83	-55.50
167.61	-57.71
0.00	-88.98



* Graphed are all moments found in piles within the soil

Job: Maurepas Pump Station
Description: Pile interaction diagram with Moments vs. Axial forces graphed.

Project No.: 10001663
Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10



* Graphed are all moments found in piles above the soil (between EL 4.0 and EL 11.25)

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Calculation of Pile Tip Elevation from SAP Pile Reactions:

Maximum Axial load among all piles = 105.92 kips

From Table 1 in geotechnical report, Allowable compression capacity at El. -80.0 = 105 kips

Required pile capacity is greater than pile capacity at El. -80.0 by 0.92 kips for 1 pile only.

Since, FOS of 2.0 is applied for pile capacity calculation it is OK to use pile tip EL Of **-80.0**

(See attached Geotechnical report)

SECTION 5

Base Slab Design

(112-125 NOT USED)

URS

Job MAUREPAS PUMP STATION | Project No. 10001663 | Sheet ____ of ____

Description DESIGN OF BASE SLAB FOR PUMP STATION SUB STRUCTURE | Computed by JY | Date 09/10

Checked by EY | Date 12/10

Reference

SHEAR CAPACITY OF BASE SLAB:

DEPTH OF BASE SLAB = 2'6" = 30"

EFFECTIVE DEPTH OF THE BASE SLAB = 30" − 4" (clc) − 0.5×1" (1" ϕ ASSUME) = 25.5"

∴ SHEAR CAPACITY $\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = 0.85 \times 2\sqrt{4000} \times (12'') \times 25.5''$ (12" = 1' STRIP)

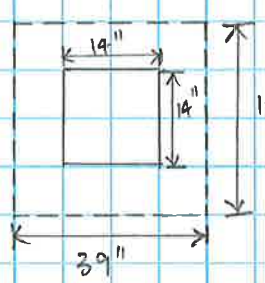
$= 32.9\ K/FT$

THE MAXIMUM SHEAR IN THE BASE SLAB IS (V_{23} MAX)

$= 27.7\ K/FT$ CALCULATED AT THE EDGE OF THE PILE IN SAP MODEL. SEE ATTACHED SHEAR DRAWINGS IN THE BASE SLAB.

SINCE, SHEAR CAPACITY = 32.9 K/FT > MAX. SHEAR = 27.7 K/FT, THE 2.5' THICK BASE SLAB IS SAFE IN SHEAR.

CHECK FOR PUNCHING SHEAR CAPACITY: (REFER ACI 318 - SEC. 11.12.2)



$14'' + d = 14'' + (30'' - 5'') = 39''$ (APPROX.) ∴ $b_o = 39'' \times 4 = 156''$

PUNCHING SHEAR CAPACITY V'_c

$= \left(2 + \frac{4}{\beta}\right)\sqrt{f'_c}\, b_o d$, $\beta = \frac{14''}{14''} = 1$ (11-33)

$= \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$, $\alpha_s = 20$ (11-34)

$= 4\sqrt{f'_c}\, b_o d$ (11-35)

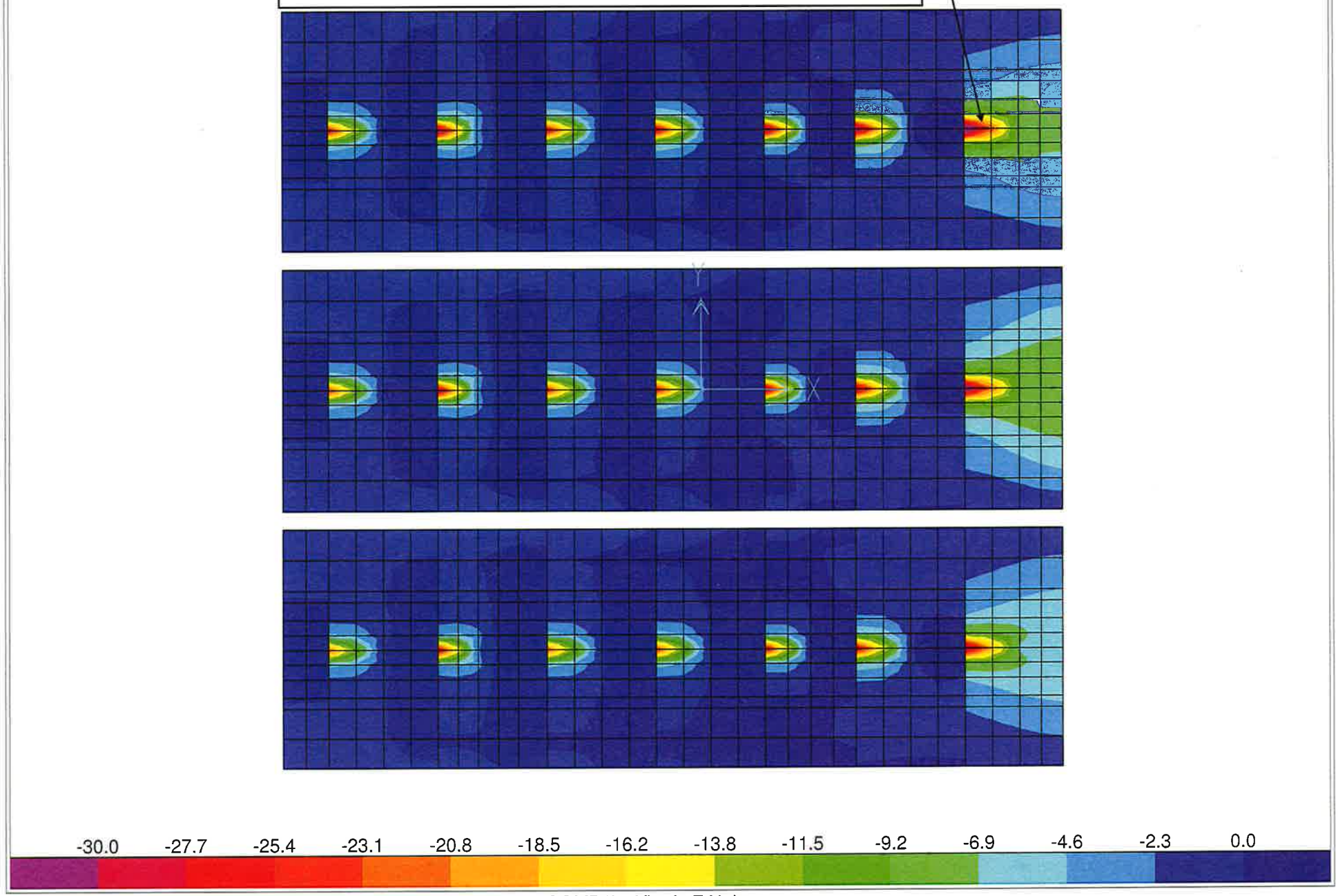
$\left(2 + \frac{4}{\beta}\right) = 2 + 4 = 6$; $\frac{\alpha_s d}{b_o} + 2 = \frac{20 \times 25.5}{156} + 2 = 5.21$

∴ CONTROLLING FACTOR = 4 ⇒ $V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{4000} \times 156 \times 2.5 = 98.6\ K \gg 27.7\ K/FT$

∴ 2.5' THICK SLAB IS SAFE IN PUNCHING SHEAR.

BASE SLAB

MIN V13 AT THE EDGE OF THE PILE AVERAGED AMONG AREA ELEMENTS BETWEEN THE PILES = 25.4 k/ft



SAP2000 v12.0.1 - File:PS MODEL - Resultant V13 Diagram (COMB1) - Kip, ft, F Units

Job - Maurepas Pumpstation
Project # 10001663

JY 10/10
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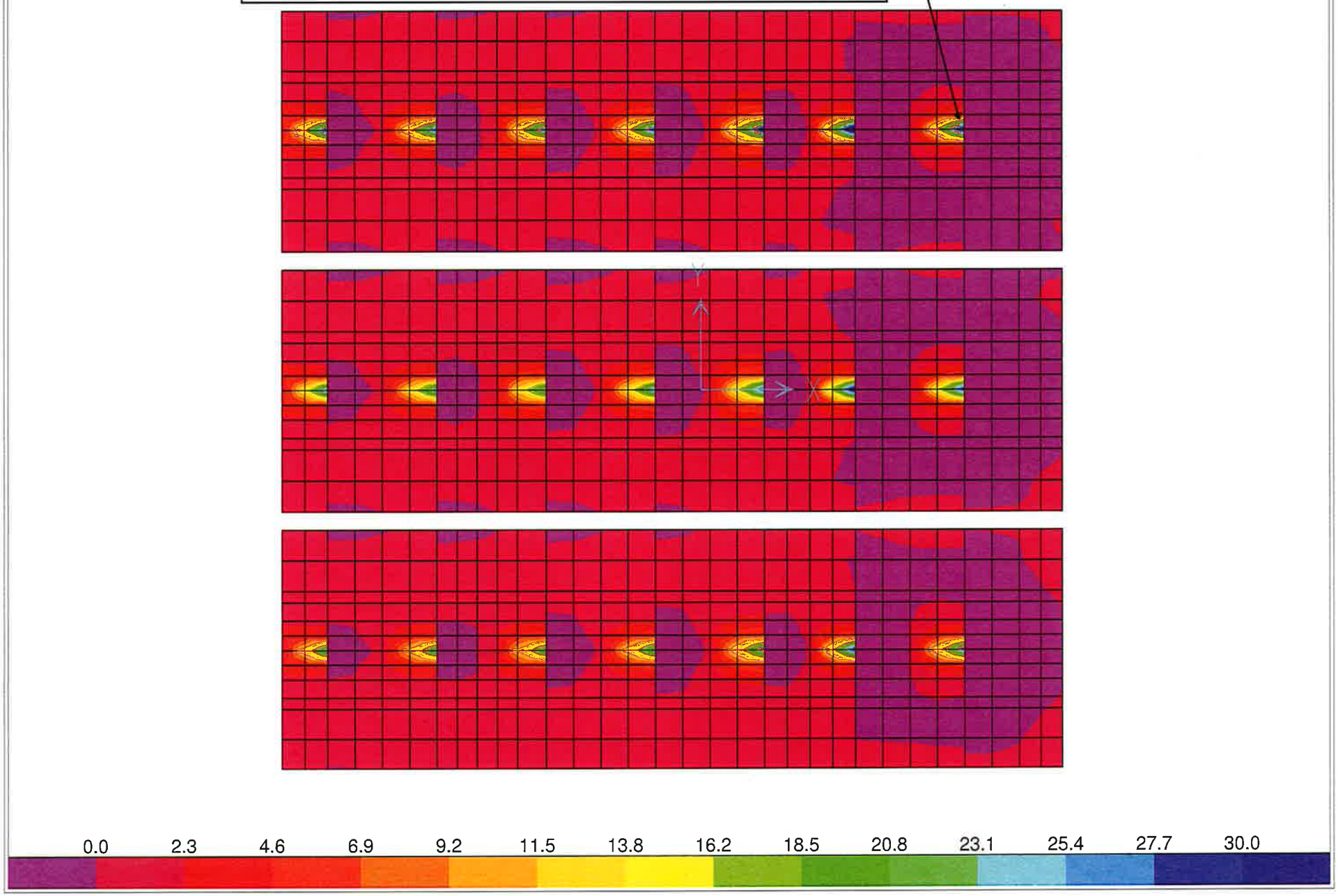
BASE SLAB

MAX V13 AT THE EDGE OF THE PILE AVERAGED AMONG AREA ELEMENTS BETWEEN THE PILES = 23.1 k/ft



SAP2000 v12.0.1 - File:PS MODEL - Resultant V13 Diagram (COMB1) - Kip, ft, F Units

Job - Maurepas Pumpstation
Project # 10001663

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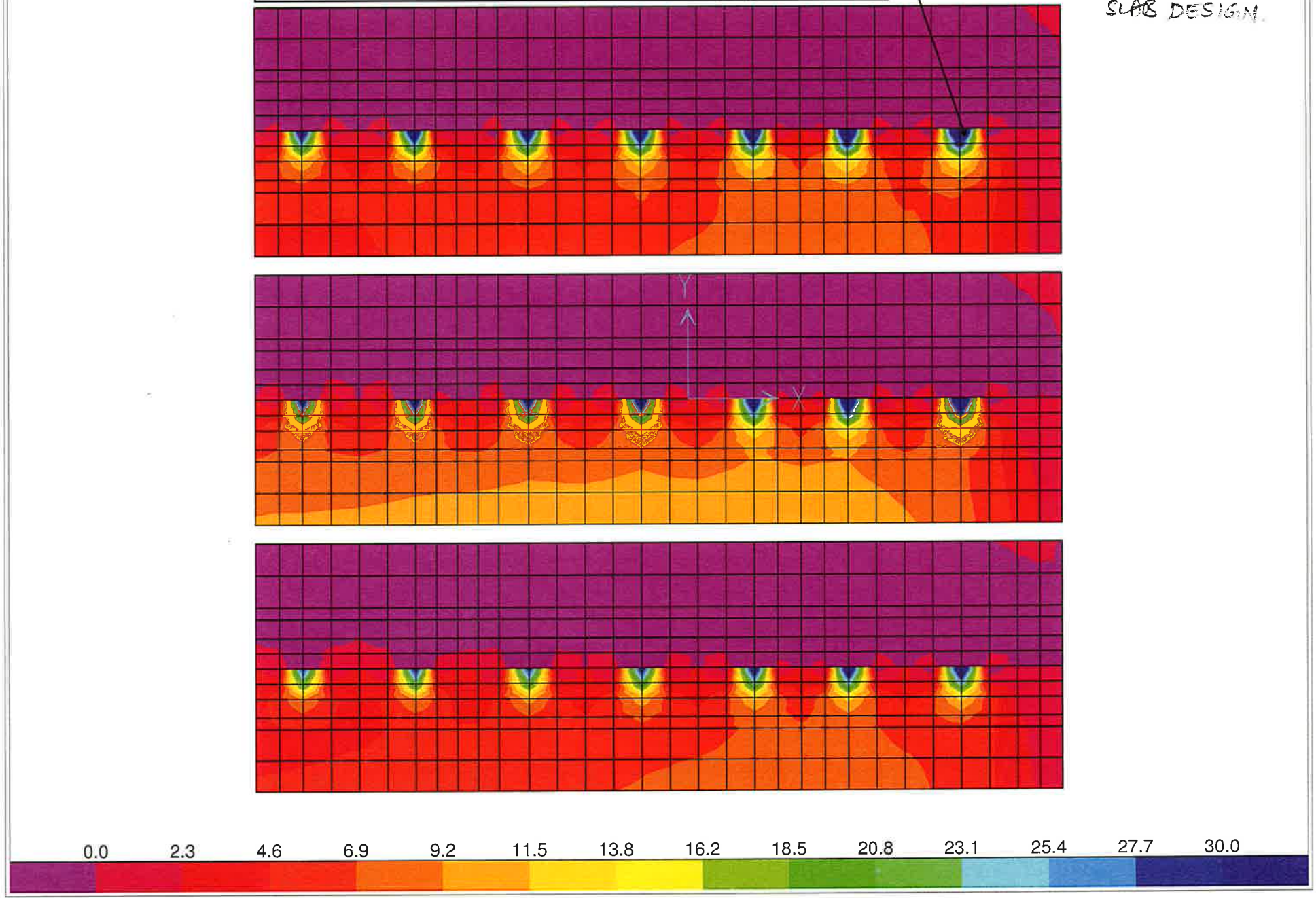
Job - Maurepas Pumpstation
Project # 10001663

BASE SLAB

MAX V23 AT THE EDGE OF THE PILE AVERAGED AMONG AREA ELEMENTS BETWEEN THE PILES = 27.7 k/ft

CONTROLS THE BASE SLAB DESIGN.



SAP2000 v12.0.1 - File:PS MODEL - Resultant V23 Diagram (COMB1) - Kip, ft, F Units

JY 10/10
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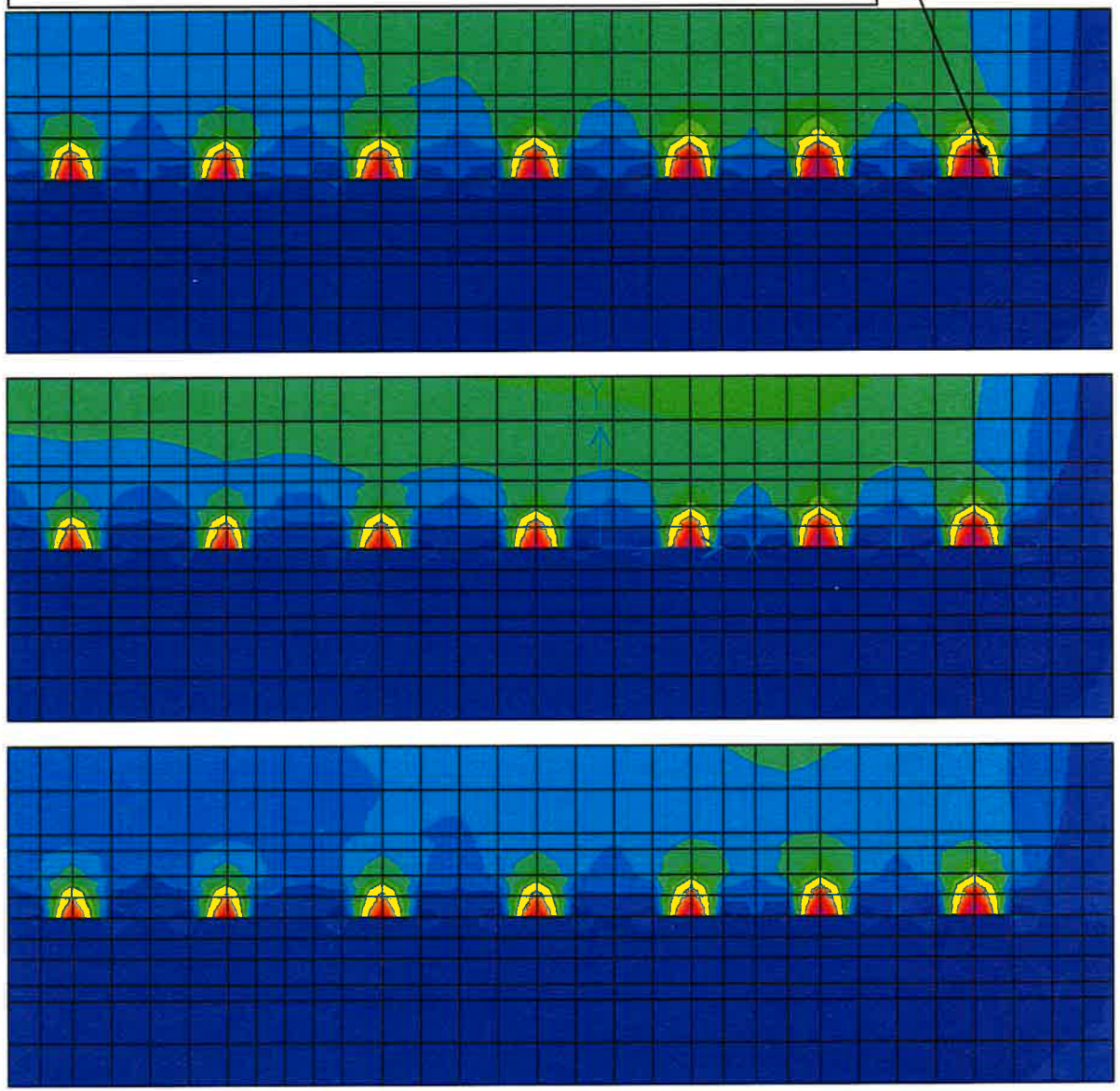
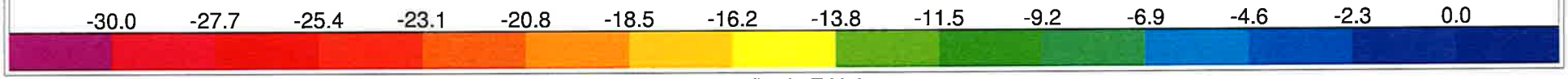
Job - Maurepas Pumpstation
Project # 10001663

BASE SLAB

MIN V23 AT THE EDGE OF THE PILE AVERAGED AMONG AREA ELEMENTS BETWEEN THE PILES = 23.1 k/ft





SAP2000 v12.0.1 - File:PS MODEL - Resultant V23 Diagram (COMB1) - Kip, ft, F Units

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REINFORCEMENT REQUIRED IN BASE SLAB:

CONTROLLING MOMENT FOR BASE SLAB DESIGN = MIN. M22

= 44.1 K-FT/FT

(SEE ATTACHED SPREADSHEET & MOMENT DRAWINGS)

WIDTH OF SLAB = 1' = 12" (DESIGNING PER FT.)

DEPTH OF SLAB = 2.5' = 30"

EFFECTIVE DEPTH OF THE SLAB = 30" − 4" − 0.5×1" = 25.5"

(CLR. COVER; ASSUME 1" ØBAR)

f_y STEEL = 60 ksi ; f'_c CONCRETE = 4 ksi

ACCORDING TO ACI 318-02 SEC. 7.12.2.1, MIN. R/F REQUIRED

$$\text{MIN. } A_s = \rho_{MIN} \cdot b \cdot d = 0.0018 \times b \times d \text{ PER FACE}$$

(FOR 60 ksi STEEL)

$$= 0.0018 \times 12'' \times 25.5'' = 0.551 \text{ in}^2/\text{FT}$$

$$\text{CAPACITY OF MIN. R/F} = \phi \times A_s \times f_y \times \left(d - \frac{1}{2}\left(\frac{A_s \cdot f_y}{0.85 \times f'_c \times b}\right)\right)$$

$$= 0.9 \times 0.551 \text{ in}^2 \times 60 \text{ ksi} \times \left(25.5 - \frac{1}{2} \times \frac{0.551 \text{ in}^2 \times 60 \text{ ksi}}{0.85 \times 4 \text{ ksi} \times 12''}\right)$$

$$= 746.7 \text{ K-in/FT} = 62.22 \text{ K-FT/FT}$$

> 44.1 K-FT/FT CONTROLLING MOM.

∴ PROVIDE MIN. R/F = 0.551 in²/FT ⇒ #7 @ 12" TOP & BOTTOM

OR #5 @ 6" TOP & BOTTOM

= 2 × 0.31 IN² = 0.62 IN²

PROVIDED R/F IN SLAB = #5 @ 6" PER FACE

= 0.62 in^2/FT

MAX. R/F THAT CAN BE PROVIDED

$$\rho_{MAX} = 0.375\,\rho_{BAL}$$

WHERE $\rho_{BAL} = 0.0285$ (EM 1110-2-2104, CHAPTER 3-5)

$$\therefore \rho_{MAX} = 0.375 \times 0.0285 = 0.01069$$

$$\text{MAX. } A_s = \rho_{MAX} \times b \times d = 0.01069 \times 12'' \times 25.5''$$

$$= 3.27 \text{ in}^2/\text{FT}$$

$$> 0.62 \text{ in}^2/\text{FT} \quad \underline{\underline{OK}}$$

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments in Base Slab

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

BASE SLAB MOMENTS SUMMARY:

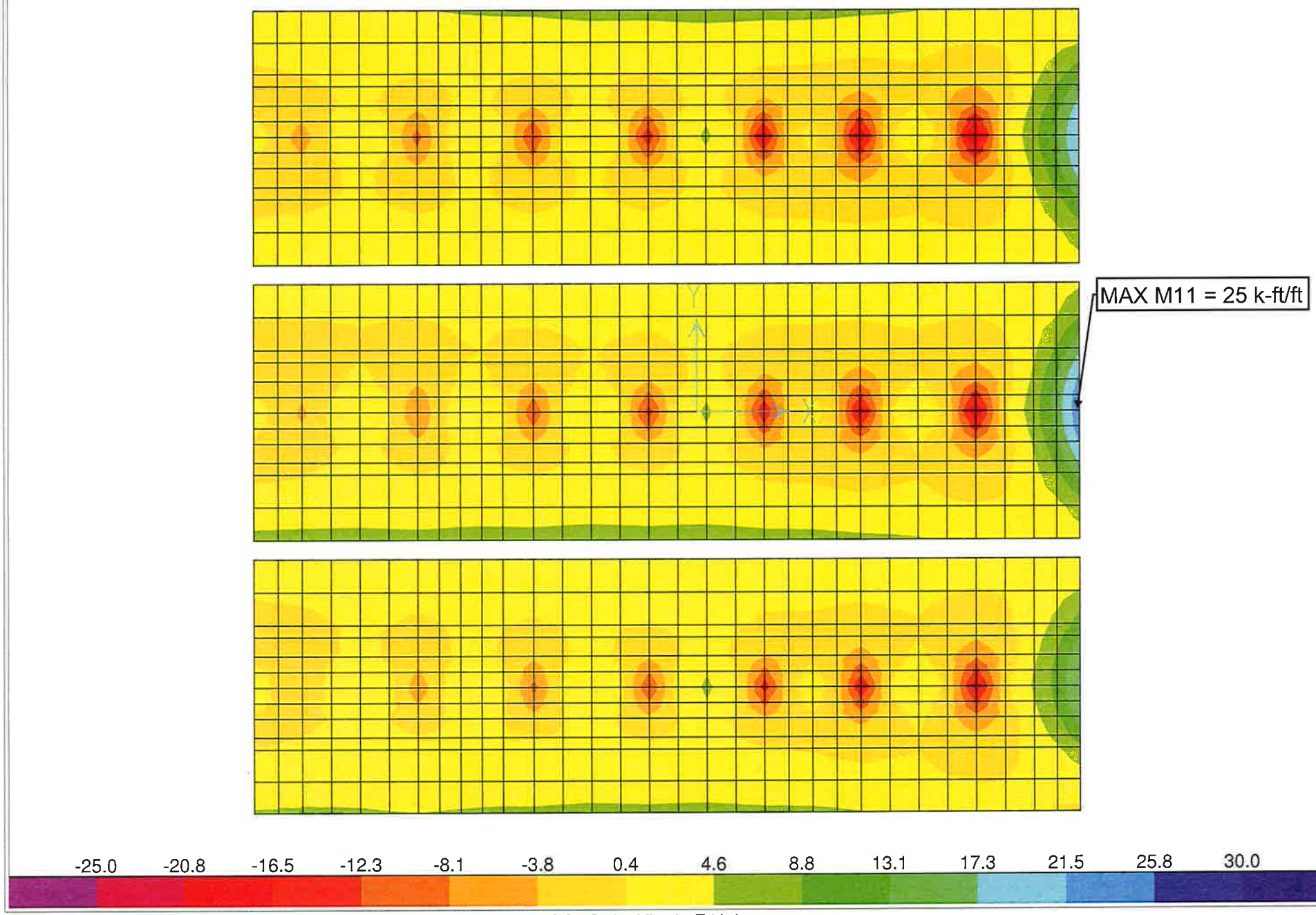
Maximum M11 =	25.054	kip-ft/ft
Minimum M11 =	-36.531	kip-ft/ft
Maximum M22 =	32.541	kip-ft/ft
Minimum M22 =	-44.096	kip-ft/ft

Base Slab Element Results

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
A1_a	Max of M11	6.603	0.167	5.659	12.507
	Min of M11	-16.616	0.167	-14.241	-31.474
	Max of M22	10.598	0.167	9.084	20.075
	Min of M22	-20.397	0.167	-17.483	-38.636
A1_b	Max of M11	6.737	0.167	5.774	12.761
	Min of M11	-16.798	0.167	-14.398	-31.820
	Max of M22	10.698	0.167	9.169	20.264
	Min of M22	-20.641	0.167	-17.692	-39.098
B1_m	Max of M11	8.916	0.000	8.916	19.705
	Min of M11	-16.530	0.000	-16.530	-36.531
	Max of M22	12.084	0.000	12.084	26.705
	Min of M22	-19.953	0.000	-19.953	-44.096
B1_s	Max of M11	8.909	0.000	8.909	19.690
	Min of M11	-16.430	0.000	-16.430	-36.311
	Max of M22	12.047	0.000	12.047	26.623
	Min of M22	-19.845	0.000	-19.845	-43.858
B3_m	Max of M11	8.476	0.000	8.476	18.732
	Min of M11	-15.747	0.000	-15.747	-34.802
	Max of M22	11.155	0.000	11.155	24.653
	Min of M22	-19.281	0.000	-19.281	-42.611
B3_s	Max of M11	8.469	0.000	8.469	18.717
	Min of M11	-15.648	0.000	-15.648	-34.581
	Max of M22	11.203	0.000	11.203	24.758
	Min of M22	-19.173	0.000	-19.173	-42.372
B4_m	Max of M11	8.874	0.167	7.606	16.810
	Min of M11	-16.383	0.167	-14.042	-31.034
	Max of M22	12.039	0.167	10.319	22.804
	Min of M22	-19.943	0.167	-17.094	-37.777
B4_s	Max of M11	8.868	0.167	7.600	16.797
	Min of M11	-16.284	0.167	-13.957	-30.845
	Max of M22	12.002	0.167	10.287	22.735
	Min of M22	-19.835	0.167	-17.001	-37.573
C1_a	Max of M11	7.578	1.330	3.252	7.188
	Min of M11	-14.632	1.330	-6.280	-13.879
	Max of M22	9.852	1.330	4.228	9.344
	Min of M22	-17.660	1.330	-7.579	-16.750
C1_b	Max of M11	7.712	1.330	3.310	7.315
	Min of M11	-14.815	1.330	-6.358	-14.052
	Max of M22	10.001	1.330	4.292	9.486
	Min of M22	-17.904	1.330	-7.684	-16.981
D1_m	Max of M11	11.476	0.167	9.836	21.738
	Min of M11	-16.051	0.167	-13.758	-30.404
	Max of M22	16.730	0.167	14.340	31.691
	Min of M22	-19.179	0.167	-16.439	-36.330
D1_s	Max of M11	11.469	0.167	9.830	21.725
	Min of M11	-15.951	0.167	-13.672	-30.215
	Max of M22	16.693	0.167	14.308	31.621
	Min of M22	-19.072	0.167	-16.347	-36.126
D2_m	Max of M11	13.182	0.167	11.298	24.969
	Min of M11	-14.439	0.167	-12.376	-27.351
	Max of M22	17.132	0.167	14.684	32.451
	Min of M22	-19.814	0.167	-16.983	-37.533
D2_s	Max of M11	13.226	0.167	11.337	25.054
	Min of M11	-14.410	0.167	-12.351	-27.296
	Max of M22	17.179	0.167	14.724	32.541
	Min of M22	-19.823	0.167	-16.990	-37.549

Job - Maurepas Pumpstation
Project # 10001663

BASE SLAB



SAP2000 v12.0.1 - File:PS MODEL - Resultant M11 Diagram (COMB1) - Kip, ft, F Units

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BY 12/10
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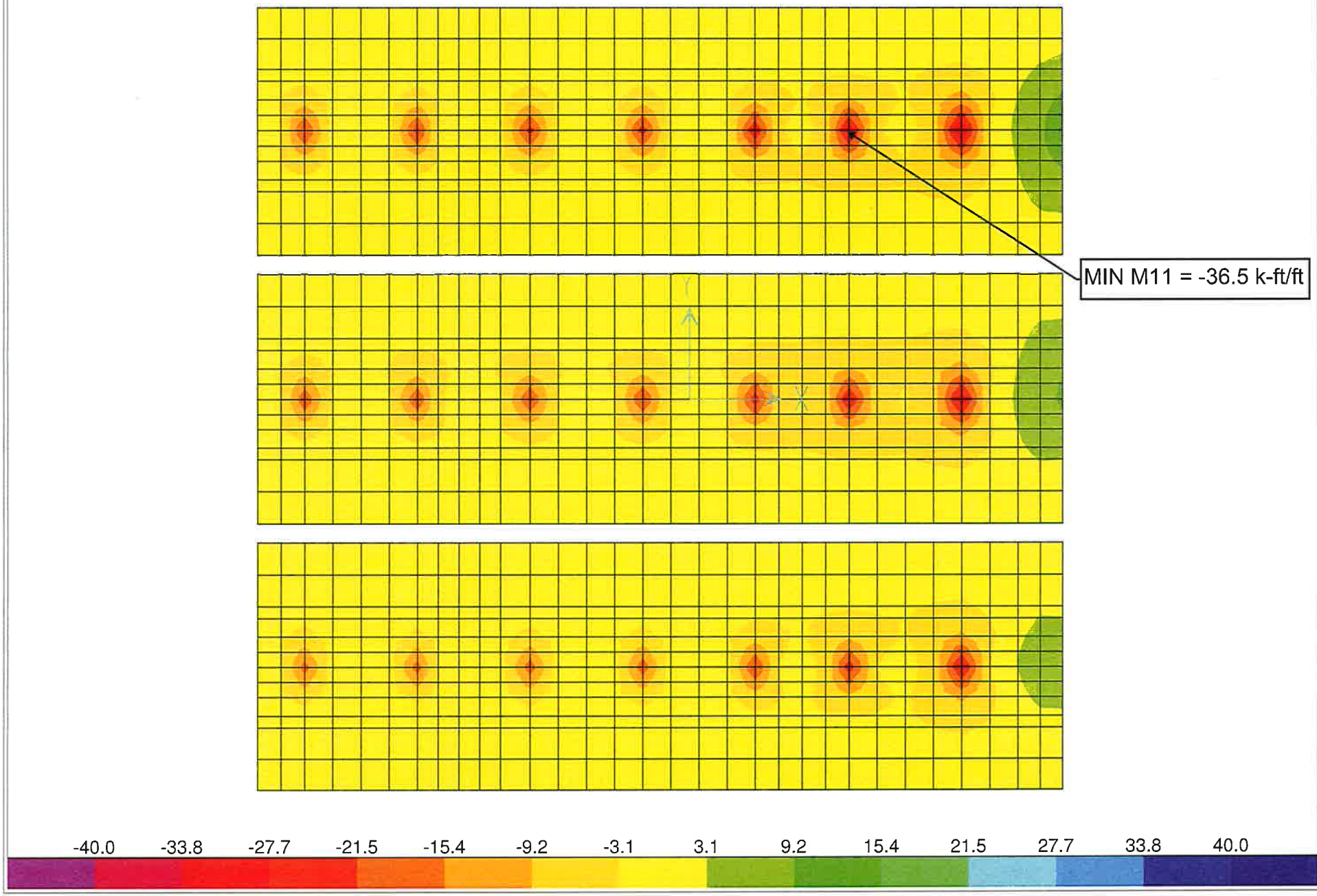
Job - Maurepas Pumpstation
Project # 10001663

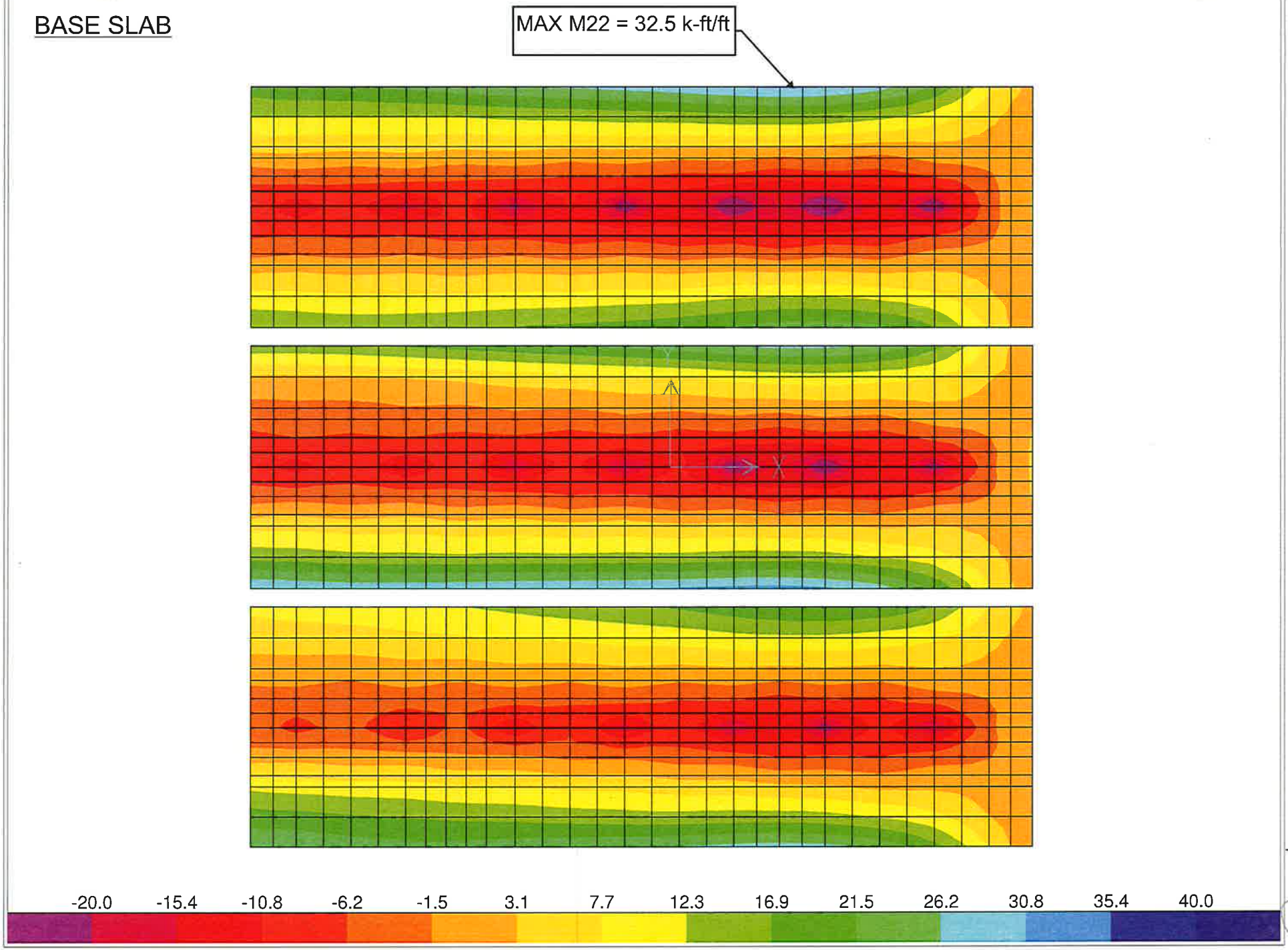
BASE SLAB



SAP2000 v12.0.1 - File:PS MODEL - Resultant M11 Diagram (COMB1) - Kip, ft, F Units

JY 09/10
EY 12/10
(135)

BASE SLAB



SAP2000 v12.0.1 - File:PS MODEL - Resultant M22 Diagram (COMB1) - Kip, ft, F Units

Job - Maurepas Pumpstation
Project # 10001663

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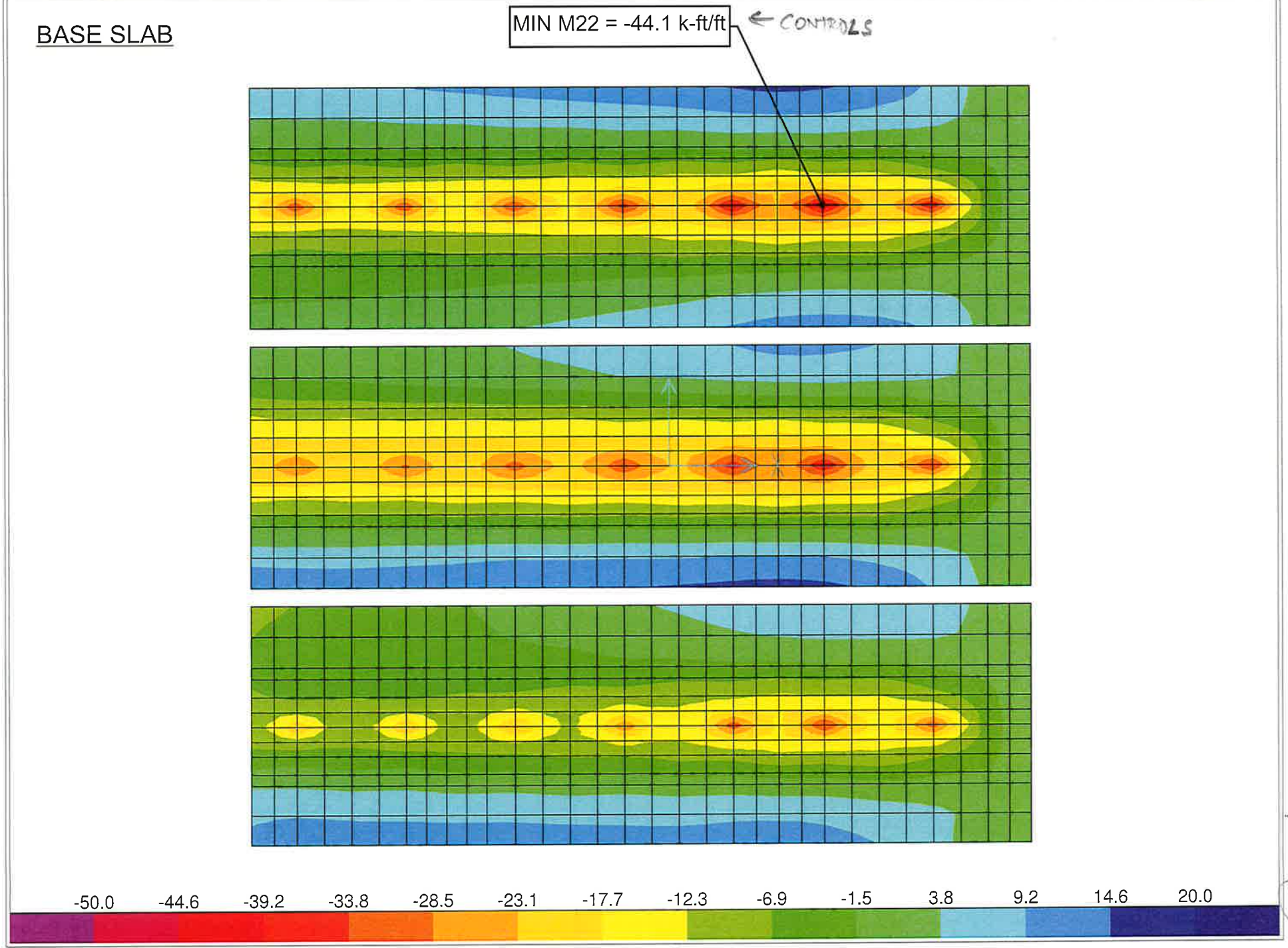
BASE SLAB



SAP2000 v12.0.1 - File:PS MODEL - Resultant M22 Diagram (COMB1) - Kip, ft, F Units

SECTION 6

Exterior Wall Design

CHECK FOR SHEAR IN OUTER WALLS:

CONTROLLING SHEAR FOR DESIGN OF OUTER WALLS = 11.161 K/FT (SEE ATTACHED SPREADSHEETS & DRAWINGS)

DEPTH OF THE WALL SECTION = 2'6" = 30"

SHEAR CAPACITY OF 30" DEPTH SECTION AS CALCULATED FOR BASE SLAB DESIGN = 32.9 K/FT > 11.161 K/FT ∴ EXTERIOR WALLS ARE SAFE IN SHEAR.

DESIGN OF OUTERWALLS FOR MAX. MOMENT:

CONTROLLING MOMENT FOR DESIGN = 61.46 K-FT/FT (SEE ATTACHED SHEETS)

EFFECTIVE DEPTH AVAILABLE FOR 2'6" SECTION = 25.5"

MIN. EFF. DEPTH REQUIRED 'd_d' = $\sqrt{\frac{2.4956\, M_n}{b}}$ — (EM 1110-2-2104, TABLE D-1)

$$= \sqrt{\frac{2.4956}{b} \times \frac{M_u}{\phi}} = \sqrt{\frac{2.4956}{12''} \times \frac{61.46 \times 12}{0.9}}$$

$$= 13.05'' < 25.5'' \text{ OK}$$

SINCE THE CAPACITY OF MIN. R/F FOR 2'6" SECTION = 62.22 K-FT/FT > 61.46 K-FT/FT

PROVIDE MIN. R/F i.e., #5 @ 6" EACH FACE. AS CALCULATED FOR BASE SLAB

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Outer Side Wall 1

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

OUTER SIDE WALL 1 MOMENT & SHEAR SUMMARY:

Maximum M11 =	14.645	kip-ft/ft
Minimum M11 =	-22.742	kip-ft/ft
Maximum M22 =	8.784	kip-ft/ft
Minimum M22 =	-61.459	kip-ft/ft
Maximum V13 =	10.588	kip-ft/ft
Minimum V13 =	-3.872	kip-ft/ft
Maximum V23 =	11.161	kip-ft/ft
Minimum V23 =	-10.039	kip-ft/ft

CONTROLLING MOMENT & SHEAR VALUES FOR DESIGN.

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
A1_a	Max of M11	7.7314	0.167	6.627	14.645
	Min of M11	-8.4809	0.167	-7.269	-16.065
	Max of M22	1.5361	0.167	1.317	2.910
	Min of M22	-31.4462	0.167	-26.953	-59.566
	Max of V13	4.562	0.167	3.910	8.641
	Min of V13	-1.67	0.167	-1.431	-3.163
	Max of V23	4.718	0.167	4.044	8.937
	Min of V23	-3.496	0.167	-2.996	-6.622
A1_b	Max of M11	7.7233	0.167	6.620	14.630
	Min of M11	-8.4862	0.167	-7.274	-16.075
	Max of M22	1.525	0.167	1.307	2.889
	Min of M22	-31.4449	0.167	-26.952	-59.564
	Max of V13	4.565	0.167	3.913	8.647
	Min of V13	-1.672	0.167	-1.433	-3.167
	Max of V23	4.722	0.167	4.047	8.945
	Min of V23	-3.497	0.167	-2.997	-6.624
B1_m	Max of M11	3.4228	0.000	3.423	7.564
	Min of M11	-10.2907	0.000	-10.291	-22.742
	Max of M22	3.3383	0.000	3.338	7.378
	Min of M22	-27.8096	0.000	-27.810	-61.459
	Max of V13	4.791	0.000	4.791	10.588
	Min of V13	-1.752	0.000	-1.752	-3.872
	Max of V23	5.05	0.000	5.050	11.161
	Min of V23	-3.581	0.000	-3.581	-7.914
B1_s	Max of M11	3.5798	0.000	3.580	7.911
	Min of M11	-9.8451	0.000	-9.845	-21.758
	Max of M22	3.1881	0.000	3.188	7.046
	Min of M22	-27.1214	0.000	-27.121	-59.938
	Max of V13	4.687	0.000	4.687	10.358
	Min of V13	-1.703	0.000	-1.703	-3.764
	Max of V23	4.939	0.000	4.939	10.915
	Min of V23	-3.597	0.000	-3.597	-7.949
B3_m	Max of M11	5.6859	0.000	5.686	12.566
	Min of M11	-6.9273	0.000	-6.927	-15.309
	Max of M22	1.8666	0.000	1.867	4.125
	Min of M22	-24.7967	0.000	-24.797	-54.801
	Max of V13	3.656	0.000	3.656	8.080
	Min of V13	-1.359	0.000	-1.359	-3.003
	Max of V23	3.795	0.000	3.795	8.387
	Min of V23	-2.783	0.000	-2.783	-6.150
B3_s	Max of M11	5.843	0.000	5.843	12.913
	Min of M11	-6.571	0.000	-6.571	-14.522
	Max of M22	1.7709	0.000	1.771	3.914
	Min of M22	-24.1178	0.000	-24.118	-53.300
	Max of V13	3.552	0.000	3.552	7.850
	Min of V13	-1.31	0.000	-1.310	-2.895
	Max of V23	3.684	0.000	3.684	8.142
	Min of V23	-2.708	0.000	-2.708	-5.985

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Outer Side Wall 1

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
B4_m	Max of M11	4.015	0.167	3.441	7.605
	Min of M11	-10.3171	0.167	-8.843	-19.543
	Max of M22	3.2783	0.167	2.810	6.210
	Min of M22	-28.9692	0.167	-24.830	-54.874
	Max of V13	4.871	0.167	4.175	9.227
	Min of V13	-1.781	0.167	-1.527	-3.374
	Max of V23	5.121	0.167	4.389	9.700
	Min of V23	-3.567	0.167	-3.057	-6.757
B4_s	Max of M11	4.172	0.167	3.576	7.903
	Min of M11	-9.8716	0.167	-8.461	-18.699
	Max of M22	3.1391	0.167	2.691	5.946
	Min of M22	-28.281	0.167	-24.240	-53.571
	Max of V13	4.767	0.167	4.086	9.030
	Min of V13	-1.732	0.167	-1.485	-3.281
	Max of V23	5.01	0.167	4.294	9.490
	Min of V23	-3.583	0.167	-3.071	-6.787
C1_a	Max of M11	5.0693	1.330	2.176	4.808
	Min of M11	-5.7788	1.330	-2.480	-5.481
	Max of M22	1.8427	1.330	0.791	1.748
	Min of M22	-21.0147	1.330	-9.019	-19.932
	Max of V13	3.181	1.330	1.365	3.017
	Min of V13	-1.146	1.330	-0.492	-1.087
	Max of V23	3.3	1.330	1.416	3.130
	Min of V23	-2.518	1.330	-1.081	-2.388
C1_b	Max of M11	5.0611	1.330	2.172	4.800
	Min of M11	-5.7841	1.330	-2.482	-5.486
	Max of M22	1.8339	1.330	0.787	1.739
	Min of M22	-21.0134	1.330	-9.019	-19.931
	Max of V13	3.184	1.330	1.367	3.020
	Min of V13	-1.148	1.330	-0.493	-1.089
	Max of V23	3.304	1.330	1.418	3.134
	Min of V23	-2.518	1.330	-1.081	-2.388
D1_m	Max of M11	1.9481	0.167	1.670	3.690
	Min of M11	-10.8701	0.167	-9.317	-20.590
	Max of M22	4.6373	0.167	3.975	8.784
	Min of M22	-19.9195	0.167	-17.073	-37.732
	Max of V13	4.511	0.167	3.866	8.545
	Min of V13	-1.583	0.167	-1.357	-2.999
	Max of V23	4.864	0.167	4.169	9.214
	Min of V23	-5.284	0.167	-4.529	-10.009
D1_s	Max of M11	1.9326	0.167	1.656	3.661
	Min of M11	-10.4246	0.167	-8.935	-19.747
	Max of M22	4.477	0.167	3.837	8.480
	Min of M22	-19.2313	0.167	-16.484	-36.429
	Max of V13	4.407	0.167	3.777	8.348
	Min of V13	-1.534	0.167	-1.315	-2.906
	Max of V23	4.752	0.167	4.073	9.001
	Min of V23	-5.3	0.167	-4.543	-10.039
D2_m	Max of M11	5.5136	0.167	4.726	10.444
	Min of M11	-4.7081	0.167	-4.035	-8.918
	Max of M22	1.3809	0.167	1.184	2.616
	Min of M22	-19.0396	0.167	-16.319	-36.065
	Max of V13	2.853	0.167	2.445	5.404
	Min of V13	-0.996	0.167	-0.854	-1.887
	Max of V23	2.945	0.167	2.524	5.579
	Min of V23	-2.573	0.167	-2.205	-4.874
D2_s	Max of M11	5.6706	0.167	4.860	10.741
	Min of M11	-4.3518	0.167	-3.730	-8.243
	Max of M22	1.2867	0.167	1.103	2.437
	Min of M22	-18.3606	0.167	-15.737	-34.779
	Max of V13	2.749	0.167	2.356	5.207
	Min of V13	-0.948	0.167	-0.813	-1.796
	Max of V23	2.833	0.167	2.428	5.366
	Min of V23	-2.589	0.167	-2.219	-4.904

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Outer Side Wall 2

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

OUTER SIDE WALL 2 MOMENT & SHEAR SUMMARY:

Maximum M11 =	21.041	kip-ft/ft
Minimum M11 =	-4.228	kip-ft/ft
Maximum M22 =	33.096	kip-ft/ft
Minimum M22 =	-10.938	kip-ft/ft
Maximum V13 =	2.325	kip-ft/ft
Minimum V13 =	-6.774	kip-ft/ft
Maximum V23 =	10.331	kip-ft/ft
Minimum V23 =	-7.536	kip-ft/ft

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
A1_a	Max of M11	9.3773	0.167	8.037	17.763
	Min of M11	-1.332	0.167	-1.142	-2.523
	Max of M22	9.4819	0.167	8.127	17.961
	Min of M22	-3.9312	0.167	-3.370	-7.447
	Max of V13	1.023	0.167	0.877	1.938
	Min of V13	-2.899	0.167	-2.485	-5.491
	Max of V23	2.094	0.167	1.795	3.967
	Min of V23	-3.221	0.167	-2.761	-6.101
A1_b	Max of M11	9.3316	0.167	7.998	17.676
	Min of M11	-1.3279	0.167	-1.138	-2.515
	Max of M22	9.4303	0.167	8.083	17.863
	Min of M22	-3.9104	0.167	-3.352	-7.407
	Max of V13	1.018	0.167	0.873	1.928
	Min of V13	-2.887	0.167	-2.475	-5.469
	Max of V23	2.096	0.167	1.797	3.970
	Min of V23	-3.207	0.167	-2.749	-6.075
B1_m	Max of M11	9.337	0.000	9.337	20.635
	Min of M11	-1.8294	0.000	-1.829	-4.043
	Max of M22	12.6796	0.000	12.680	28.022
	Min of M22	-4.8628	0.000	-4.863	-10.747
	Max of V13	1.045	0.000	1.045	2.309
	Min of V13	-3.051	0.000	-3.051	-6.743
	Max of V23	3.807	0.000	3.807	8.413
	Min of V23	-3.392	0.000	-3.392	-7.496
B1_s	Max of M11	9.5208	0.000	9.521	21.041
	Min of M11	-1.8349	0.000	-1.835	-4.055
	Max of M22	12.5738	0.000	12.574	27.788
	Min of M22	-4.9491	0.000	-4.949	-10.938
	Max of V13	1.052	0.000	1.052	2.325
	Min of V13	-3.065	0.000	-3.065	-6.774
	Max of V23	3.805	0.000	3.805	8.409
	Min of V23	-3.41	0.000	-3.410	-7.536
B3_m	Max of M11	6.3682	0.000	6.368	14.074
	Min of M11	-1.1609	0.000	-1.161	-2.566
	Max of M22	8.4697	0.000	8.470	18.718
	Min of M22	-2.9875	0.000	-2.988	-6.602
	Max of V13	0.729	0.000	0.729	1.611
	Min of V13	-2.085	0.000	-2.085	-4.608
	Max of V23	2.651	0.000	2.651	5.859
	Min of V23	-2.302	0.000	-2.302	-5.087
B3_s	Max of M11	6.5521	0.000	6.552	14.480
	Min of M11	-1.1664	0.000	-1.166	-2.578
	Max of M22	8.364	0.000	8.364	18.484
	Min of M22	-3.0738	0.000	-3.074	-6.793
	Max of V13	0.736	0.000	0.736	1.627
	Min of V13	-2.099	0.000	-2.099	-4.639
	Max of V23	2.649	0.000	2.649	5.854
	Min of V23	-2.321	0.000	-2.321	-5.129

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Outer Side Wall 2

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
B4_m	Max of M11	9.4955	0.167	8.139	17.987
	Min of M11	-1.8309	0.167	-1.569	-3.468
	Max of M22	12.5826	0.167	10.785	23.834
	Min of M22	-4.866	0.167	-4.171	-9.217
	Max of V13	1.072	0.167	0.919	2.031
	Min of V13	-3.125	0.167	-2.678	-5.919
	Max of V23	3.793	0.167	3.251	7.185
	Min of V23	-3.463	0.167	-2.968	-6.560
B4_s	Max of M11	9.6794	0.167	8.296	18.335
	Min of M11	-1.8365	0.167	-1.574	-3.479
	Max of M22	12.4768	0.167	10.694	23.634
	Min of M22	-4.9523	0.167	-4.245	-9.381
	Max of V13	1.079	0.167	0.925	2.044
	Min of V13	-3.139	0.167	-2.690	-5.946
	Max of V23	3.791	0.167	3.249	7.181
	Min of V23	-3.482	0.167	-2.984	-6.596
C1_a	Max of M11	6.7123	1.330	2.881	6.367
	Min of M11	-1.1931	1.330	-0.512	-1.132
	Max of M22	8.245	1.330	3.539	7.820
	Min of M22	-3.2097	1.330	-1.378	-3.044
	Max of V13	0.75	1.330	0.322	0.711
	Min of V13	-2.144	1.330	-0.920	-2.034
	Max of V23	2.642	1.330	1.134	2.506
	Min of V23	-2.373	1.330	-1.018	-2.251
C1_b	Max of M11	6.6667	1.330	2.861	6.323
	Min of M11	-1.189	1.330	-0.510	-1.128
	Max of M22	8.2733	1.330	3.551	7.847
	Min of M22	-3.1864	1.330	-1.368	-3.022
	Max of V13	0.745	1.330	0.320	0.707
	Min of V13	-2.131	1.330	-0.915	-2.021
	Max of V23	2.644	1.330	1.135	2.508
	Min of V23	-2.359	1.330	-1.012	-2.238
D1_m	Max of M11	9.3828	0.167	8.042	17.773
	Min of M11	-2.2263	0.167	-1.908	-4.217
	Max of M22	17.4721	0.167	14.976	33.096
	Min of M22	-5.4566	0.167	-4.677	-10.336
	Max of V13	1.043	0.167	0.894	1.976
	Min of V13	-3.201	0.167	-2.744	-6.063
	Max of V23	5.454	0.167	4.675	10.331
	Min of V23	-3.575	0.167	-3.064	-6.772
D1_s	Max of M11	9.5667	0.167	8.200	18.122
	Min of M11	-2.2319	0.167	-1.913	-4.228
	Max of M22	17.3664	0.167	14.885	32.896
	Min of M22	-5.5429	0.167	-4.751	-10.500
	Max of V13	1.051	0.167	0.901	1.991
	Min of V13	-3.215	0.167	-2.756	-6.090
	Max of V23	5.452	0.167	4.673	10.327
	Min of V23	-3.594	0.167	-3.080	-6.808
D2_m	Max of M11	5.0215	0.167	4.304	9.512
	Min of M11	-1.3603	0.167	-1.166	-2.577
	Max of M22	9.2139	0.167	7.897	17.453
	Min of M22	-2.5918	0.167	-2.221	-4.909
	Max of V13	0.563	0.167	0.483	1.066
	Min of V13	-1.744	0.167	-1.495	-3.304
	Max of V23	2.709	0.167	2.322	5.131
	Min of V23	-1.927	0.167	-1.652	-3.650
D2_s	Max of M11	5.2054	0.167	4.462	9.860
	Min of M11	-1.4739	0.167	-1.263	-2.792
	Max of M22	9.1082	0.167	7.807	17.253
	Min of M22	-2.6781	0.167	-2.295	-5.073
	Max of V13	0.57	0.167	0.489	1.080
	Min of V13	-1.758	0.167	-1.507	-3.330
	Max of V23	2.706	0.167	2.319	5.126
	Min of V23	-1.945	0.167	-1.667	-3.684

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Back Wall

Computed By: JY
Date: 09/2010
Checked By: EJ
Date: 12/10

BACK WALL MOMENT & SHEAR SUMMARY:

Maximum M11 =	10.027	kip-ft/ft
Minimum M11 =	-4.091	kip-ft/ft
Maximum M22 =	12.284	kip-ft/ft
Minimum M22 =	-3.837	kip-ft/ft
Maximum V13 =	4.325	kip-ft/ft
Minimum V13 =	-4.355	kip-ft/ft
Maximum V23 =	5.810	kip-ft/ft
Minimum V23 =	-1.797	kip-ft/ft

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
A1_a	Max of M11	2.189	0.167	1.876	4.146
	Min of M11	-1.6939	0.167	-1.452	-3.209
	Max of M22	4.0702	0.167	3.489	7.710
	Min of M22	-0.539	0.167	-0.462	-1.021
	Max of V13	0.758	0.167	0.650	1.436
	Min of V13	-0.731	0.167	-0.627	-1.385
	Max of V23	0.714	0.167	0.612	1.352
	Min of V23	-0.721	0.167	-0.618	-1.366
A1_b	Max of M11	2.1905	0.167	1.878	4.149
	Min of M11	-1.6566	0.167	-1.420	-3.138
	Max of M22	3.8942	0.167	3.338	7.377
	Min of M22	-0.5271	0.167	-0.452	-0.998
	Max of V13	0.759	0.167	0.651	1.438
	Min of V13	-0.732	0.167	-0.627	-1.387
	Max of V23	0.717	0.167	0.615	1.358
	Min of V23	-0.705	0.167	-0.604	-1.335
B1_m	Max of M11	4.0284	0.000	4.028	8.903
	Min of M11	-1.6973	0.000	-1.697	-3.751
	Max of M22	4.479	0.000	4.479	9.899
	Min of M22	-1.4004	0.000	-1.400	-3.095
	Max of V13	1.635	0.000	1.635	3.613
	Min of V13	-1.635	0.000	-1.635	-3.613
	Max of V23	2.038	0.000	2.038	4.504
	Min of V23	-0.805	0.000	-0.805	-1.779
B1_s	Max of M11	4.0325	0.000	4.033	8.912
	Min of M11	-1.6973	0.000	-1.697	-3.751
	Max of M22	4.4794	0.000	4.479	9.899
	Min of M22	-1.4017	0.000	-1.402	-3.098
	Max of V13	1.637	0.000	1.637	3.618
	Min of V13	-1.636	0.000	-1.636	-3.616
	Max of V23	2.039	0.000	2.039	4.506
	Min of V23	-0.813	0.000	-0.813	-1.797
B3_m	Max of M11	2.6915	0.000	2.692	5.948
	Min of M11	-1.0933	0.000	-1.093	-2.416
	Max of M22	3.2041	0.000	3.204	7.081
	Min of M22	-0.8373	0.000	-0.837	-1.850
	Max of V13	1.131	0.000	1.131	2.500
	Min of V13	-1.13	0.000	-1.130	-2.497
	Max of V23	1.452	0.000	1.452	3.209
	Min of V23	-0.557	0.000	-0.557	-1.231
B3_s	Max of M11	2.6928	0.000	2.693	5.951
	Min of M11	-1.0933	0.000	-1.093	-2.416
	Max of M22	3.2045	0.000	3.205	7.082
	Min of M22	-0.8395	0.000	-0.840	-1.855
	Max of V13	1.132	0.000	1.132	2.502
	Min of V13	-1.131	0.000	-1.131	-2.500
	Max of V23	1.452	0.000	1.452	3.209
	Min of V23	-0.565	0.000	-0.565	-1.249

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Back Wall

Computed By: JY
Date: 09/2010
Checked By: EJ
Date: 12/10

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
B4_m	Max of M11	4.023	0.167	3.448	7.620
	Min of M11	-1.6925	0.167	-1.451	-3.206
	Max of M22	4.4828	0.167	3.842	8.491
	Min of M22	-1.3931	0.167	-1.194	-2.639
	Max of V13	1.635	0.167	1.401	3.097
	Min of V13	-1.632	0.167	-1.399	-3.091
	Max of V23	2.038	0.167	1.747	3.860
	Min of V23	-0.787	0.167	-0.675	-1.491
B4_s	Max of M11	4.0271	0.167	3.452	7.628
	Min of M11	-1.6925	0.167	-1.451	-3.206
	Max of M22	4.4831	0.167	3.843	8.492
	Min of M22	-1.3943	0.167	-1.195	-2.641
	Max of V13	1.636	0.167	1.402	3.099
	Min of V13	-1.633	0.167	-1.400	-3.093
	Max of V23	2.038	0.167	1.747	3.860
	Min of V23	-0.795	0.167	-0.681	-1.506
C1_a	Max of M11	2.7404	1.330	1.176	2.599
	Min of M11	-1.2232	1.330	-0.525	-1.160
	Max of M22	3.0102	1.330	1.292	2.855
	Min of M22	-0.9711	1.330	-0.417	-0.921
	Max of V13	1.126	1.330	0.483	1.068
	Min of V13	-1.115	1.330	-0.479	-1.058
	Max of V23	1.414	1.330	0.607	1.341
	Min of V23	-0.56	1.330	-0.240	-0.531
C1_b	Max of M11	2.7393	1.330	1.176	2.598
	Min of M11	-1.1859	1.330	-0.509	-1.125
	Max of M22	2.9645	1.330	1.272	2.812
	Min of M22	-0.959	1.330	-0.412	-0.910
	Max of V13	1.127	1.330	0.484	1.069
	Min of V13	-1.116	1.330	-0.479	-1.059
	Max of V23	1.417	1.330	0.608	1.344
	Min of V23	-0.544	1.330	-0.233	-0.516
D1_m	Max of M11	5.2935	0.167	4.537	10.027
	Min of M11	-2.1598	0.167	-1.851	-4.091
	Max of M22	6.4845	0.167	5.558	12.283
	Min of M22	-2.0257	0.167	-1.736	-3.837
	Max of V13	2.282	0.167	1.956	4.323
	Min of V13	-2.297	0.167	-1.969	-4.351
	Max of V23	3.067	0.167	2.629	5.810
	Min of V23	-0.912	0.167	-0.782	-1.728
D1_s	Max of M11	5.2882	0.167	4.533	10.017
	Min of M11	-2.1597	0.167	-1.851	-4.091
	Max of M22	6.4849	0.167	5.558	12.284
	Min of M22	-2.0178	0.167	-1.729	-3.822
	Max of V13	2.283	0.167	1.957	4.325
	Min of V13	-2.299	0.167	-1.971	-4.355
	Max of V23	3.067	0.167	2.629	5.810
	Min of V23	-0.92	0.167	-0.789	-1.743
D2_m	Max of M11	2.7943	0.167	2.395	5.293
	Min of M11	-1.5878	0.167	-1.361	-3.008
	Max of M22	3.4624	0.167	2.968	6.559
	Min of M22	-0.5988	0.167	-0.513	-1.134
	Max of V13	1.114	0.167	0.955	2.110
	Min of V13	-1.129	0.167	-0.968	-2.139
	Max of V23	1.441	0.167	1.235	2.730
	Min of V23	-0.429	0.167	-0.368	-0.813
D2_s	Max of M11	2.807	0.167	2.406	5.317
	Min of M11	-1.7233	0.167	-1.477	-3.264
	Max of M22	3.4628	0.167	2.968	6.559
	Min of M22	-0.6006	0.167	-0.515	-1.138
	Max of V13	1.113	0.167	0.954	2.108
	Min of V13	-1.13	0.167	-0.969	-2.140
	Max of V23	1.442	0.167	1.236	2.731
	Min of V23	-0.438	0.167	-0.375	-0.830

Job - Maurepas Pumpstation
Project # 10001663

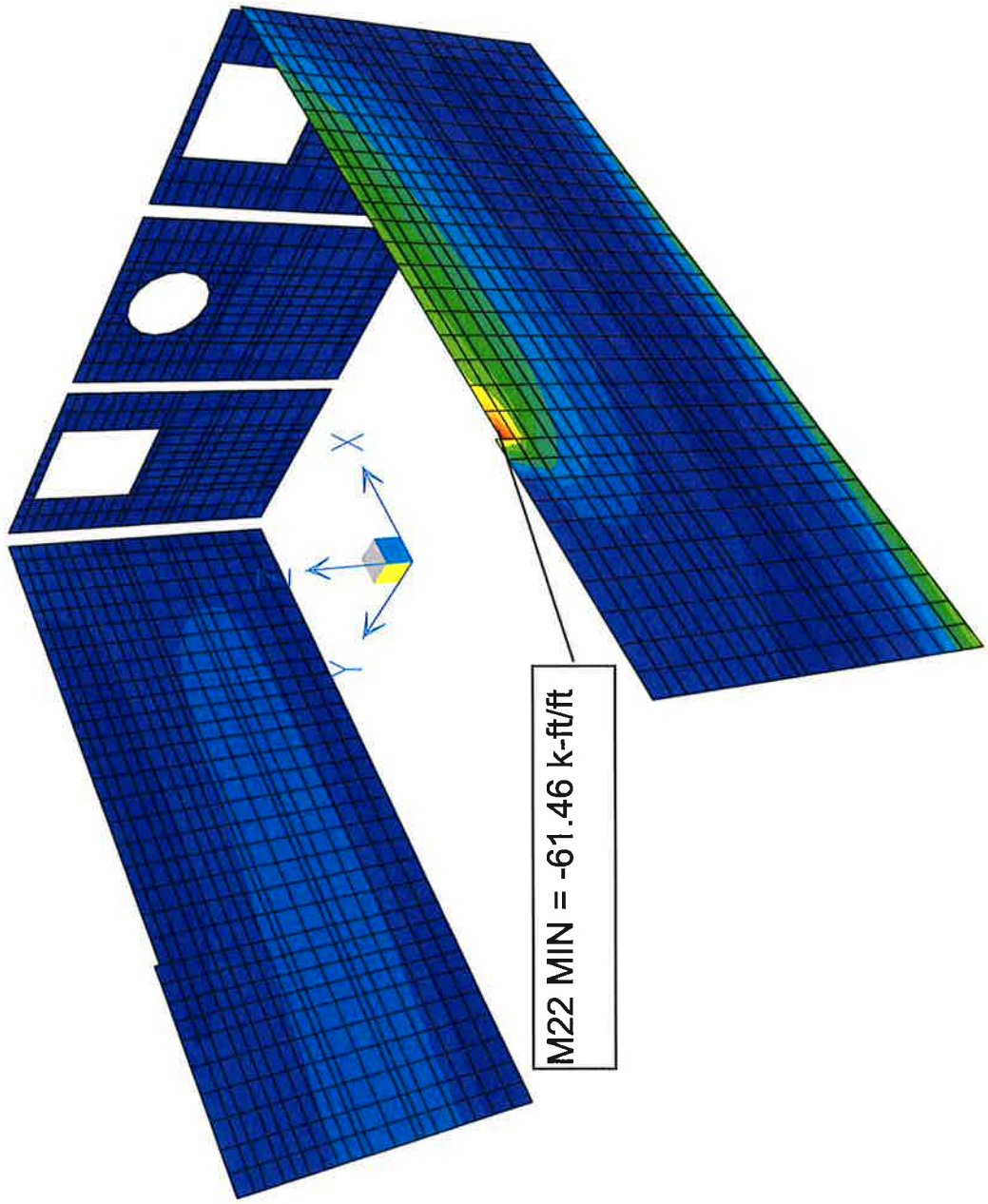
Date 10/10
EF 12/10

(145)

10/26/10 5:42

SAP2000

OUTER WALLS



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SAP2000 v12.0.1 - File:PS MODEL - Resultant M22 Diagram (COMB1) - Kip, ft, F Units

Job - Maurepas Pumpstation
Project # 10001663

Date 10/10
EY 12/10

(146)

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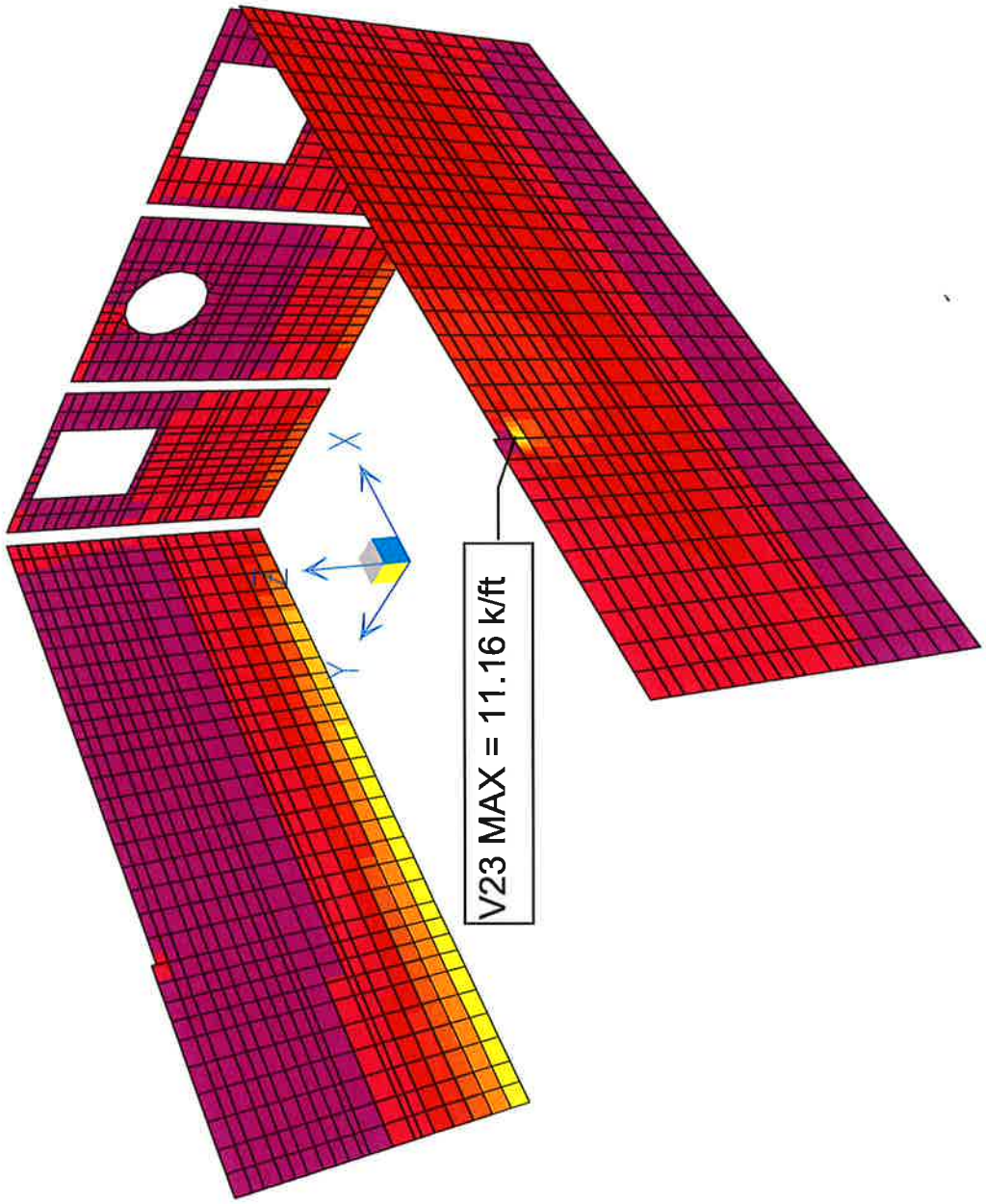
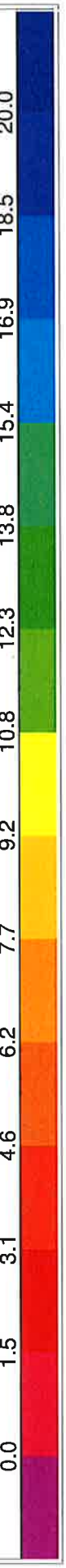
SAP200

OUTER WALLS





SAP2000 v12.0.1 - File:PS MODEL - Resultant V23 Diagram (COMB1) - Kip, ft, F Units

SECTION 7

Interior Wall Design

CHECK FOR SHEAR IN INNER WALLS:

CONTROLLING SHEAR VALUE = 6.027 k/FT (SEE ATTACHED SHEETS)

DEPTH OF WALL SECTION = 2' = 24" (ASSUMED)
EFF. DEPTH WITH 4" CLEAR COVER & 1" Ø BAR = 24" − 4" − 0.5" = 19.5"

∴ SHEAR CAPACITY = $\phi V_c = \phi\, 2\sqrt{f'_c}\, b_w d = 0.85 \times 2 \times \sqrt{4000} \times 12'' \times 19.5'$ (1' STRIP)

= 25.2 k/FT

> 6.027 k/FT CONTROLLING SHEAR
OK

DESIGN OF INNER WALLS FOR MAX. MOMENT:
CONTROLLING MOMENT FOR DESIGN = 22.85 k-FT/FT (SEE ATTACHED SHEETS)

MIN $A'_s = \rho_{MIN} \cdot b_w \cdot d = 0.0014 \times 12'' \times 19.5''$ [REFER ACI-02 SEC. 7.12.2.1]
$= 0.3276\ in^2/FT$

CAPACITY OF MIN $A'_s = \phi \times A_s \times f_y \times \left(d - \frac{1}{2} \times \frac{A_s \cdot f_y}{0.85 \times f'_c \times b}\right)$

$= 0.9 \times 0.3276 \times 60 \times \left(19.5 - \frac{1}{2} \times \frac{0.3276 \times 60}{0.85 \times 4 \times 12}\right)$

$= 340.70\ k\text{-}in/FT = 28.39\ k\text{-}FT/FT$

> 22.85 k-FT/FT CONTROLLING MOM.

∴ PROVIDE MIN. R/F = 0.3276 in²/FT = #5 @ 6" EACH FACE
i.e., 0.62 in²/FT

MAX A'_s = 0.01069 (SEE BASE SLAB CALCS.) × 12" × 19.5' = 2.50 in²/FT > 0.62 in²/FT PROVIDED OK

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Inner Side Wall 1

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

INNER SIDE WALL 1 MOMENT & SHEAR SUMMARY:

Maximum M11 =	6.288	kip-ft/ft
Minimum M11 =	-6.489	kip-ft/ft
Maximum M22 =	13.767	kip-ft/ft
Minimum M22 =	-22.850	kip-ft/ft
Maximum V13 =	2.745	kip-ft/ft
Minimum V13 =	-2.250	kip-ft/ft
Maximum V23 =	2.334	kip-ft/ft
Minimum V23 =	-6.067	kip-ft/ft

CONTROLLING MOMENT & SHEAR FOR DESIGN.

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
A1_a	Max of M11	3.3193	0.167	2.845	6.288
	Min of M11	-0.5627	0.167	-0.482	-1.066
	Max of M22	7.2679	0.167	6.229	13.767
	Min of M22	-2.8414	0.167	-2.435	-5.382
	Max of V13	0.542	0.167	0.465	1.027
	Min of V13	-1.188	0.167	-1.018	-2.250
	Max of V23	1.232	0.167	1.056	2.334
	Min of V23	-1.202	0.167	-1.030	-2.277
A1_b	Max of M11	3.3117	0.167	2.839	6.273
	Min of M11	-0.5618	0.167	-0.482	-1.064
	Max of M22	7.2586	0.167	6.221	13.749
	Min of M22	-2.8385	0.167	-2.433	-5.377
	Max of V13	0.541	0.167	0.464	1.025
	Min of V13	-1.187	0.167	-1.017	-2.248
	Max of V23	1.23	0.167	1.054	2.330
	Min of V23	-1.201	0.167	-1.029	-2.275
B1_m	Max of M11	1.8748	0.000	1.875	4.143
	Min of M11	-0.4454	0.000	-0.445	-0.984
	Max of M22	4.0492	0.000	4.049	8.949
	Min of M22	-1.7099	0.000	-1.710	-3.779
	Max of V13	0.303	0.000	0.303	0.670
	Min of V13	-0.682	0.000	-0.682	-1.507
	Max of V23	0.686	0.000	0.686	1.516
	Min of V23	-0.69	0.000	-0.690	-1.525
B1_s	Max of M11	2.0203	0.000	2.020	4.465
	Min of M11	-0.3785	0.000	-0.379	-0.836
	Max of M22	3.5194	0.000	3.519	7.778
	Min of M22	-1.7776	0.000	-1.778	-3.928
	Max of V13	0.289	0.000	0.289	0.639
	Min of V13	-0.634	0.000	-0.634	-1.401
	Max of V23	0.588	0.000	0.588	1.299
	Min of V23	-0.651	0.000	-0.651	-1.439
B3_m	Max of M11	2.0164	0.000	2.016	4.456
	Min of M11	-0.4008	0.000	-0.401	-0.886
	Max of M22	4.4633	0.000	4.463	9.864
	Min of M22	-1.7925	0.000	-1.793	-3.961
	Max of V13	0.331	0.000	0.331	0.732
	Min of V13	-0.736	0.000	-0.736	-1.627
	Max of V23	0.756	0.000	0.756	1.671
	Min of V23	-0.744	0.000	-0.744	-1.644
B3_s	Max of M11	2.1619	0.000	2.162	4.778
	Min of M11	-0.3584	0.000	-0.358	-0.792
	Max of M22	3.9308	0.000	3.931	8.687
	Min of M22	-1.8597	0.000	-1.860	-4.110
	Max of V13	0.316	0.000	0.316	0.698
	Min of V13	-0.689	0.000	-0.689	-1.523
	Max of V23	0.658	0.000	0.658	1.454
	Min of V23	-0.706	0.000	-0.706	-1.560

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Inner Side Wall 1

Computed By: JY
Date: 09/2010
Checked By: CY
Date: 12/10

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
B4_m	Max of M11	1.9494	0.167	1.671	3.693
	Min of M11	-0.4331	0.167	-0.371	-0.820
	Max of M22	4.2177	0.167	3.615	7.989
	Min of M22	-1.7639	0.167	-1.512	-3.341
	Max of V13	0.316	0.167	0.271	0.599
	Min of V13	-0.707	0.167	-0.606	-1.339
	Max of V23	0.714	0.167	0.612	1.352
	Min of V23	-0.716	0.167	-0.614	-1.356
B4_s	Max of M11	2.0948	0.167	1.795	3.968
	Min of M11	-0.3662	0.167	-0.314	-0.694
	Max of M22	3.6864	0.167	3.160	6.983
	Min of M22	-1.8316	0.167	-1.570	-3.469
	Max of V13	0.301	0.167	0.258	0.570
	Min of V13	-0.66	0.167	-0.566	-1.250
	Max of V23	0.617	0.167	0.529	1.169
	Min of V23	-0.677	0.167	-0.580	-1.282
C1_a	Max of M11	2.4648	1.330	1.058	2.338
	Min of M11	-0.4145	1.330	-0.178	-0.393
	Max of M22	5.2679	1.330	2.261	4.997
	Min of M22	-2.1206	1.330	-0.910	-2.011
	Max of V13	0.398	1.330	0.171	0.378
	Min of V13	-0.871	1.330	-0.374	-0.826
	Max of V23	0.892	1.330	0.383	0.846
	Min of V23	-0.883	1.330	-0.379	-0.838
C1_b	Max of M11	2.4572	1.330	1.055	2.331
	Min of M11	-0.4136	1.330	-0.178	-0.392
	Max of M22	5.2587	1.330	2.257	4.988
	Min of M22	-2.1177	1.330	-0.909	-2.009
	Max of V13	0.397	1.330	0.170	0.377
	Min of V13	-0.87	1.330	-0.373	-0.825
	Max of V23	0.891	1.330	0.382	0.845
	Min of V23	-0.882	1.330	-0.379	-0.837
D1_m	Max of M11	1.4592	0.167	1.251	2.764
	Min of M11	-0.366	0.167	-0.314	-0.693
	Max of M22	2.8945	0.167	2.481	5.483
	Min of M22	-1.4274	0.167	-1.223	-2.704
	Max of V13	0.229	0.167	0.196	0.434
	Min of V13	-0.521	0.167	-0.447	-0.987
	Max of V23	0.489	0.167	0.419	0.926
	Min of V23	-0.532	0.167	-0.456	-1.008
D1_s	Max of M11	1.6046	0.167	1.375	3.039
	Min of M11	-0.3858	0.167	-0.331	-0.731
	Max of M22	2.3789	0.167	2.039	4.506
	Min of M22	-1.5007	0.167	-1.286	-2.843
	Max of V13	0.214	0.167	0.183	0.405
	Min of V13	-0.473	0.167	-0.405	-0.896
	Max of V23	0.391	0.167	0.335	0.741
	Min of V23	-0.493	0.167	-0.423	-0.934
D2_m	Max of M11	1.0914	0.167	0.935	2.067
	Min of M11	-3.4254	0.167	-2.936	-6.489
	Max of M22	3.0195	0.167	2.588	5.720
	Min of M22	-11.9191	0.167	-10.216	-22.578
	Max of V13	1.449	0.167	1.242	2.745
	Min of V13	-0.21	0.167	-0.180	-0.398
	Max of V23	0.643	0.167	0.551	1.218
	Min of V23	-3.203	0.167	-2.745	-6.067
D2_s	Max of M11	1.0866	0.167	0.931	2.058
	Min of M11	-3.1545	0.167	-2.704	-5.975
	Max of M22	2.487	0.167	2.132	4.711
	Min of M22	-12.0631	0.167	-10.340	-22.850
	Max of V13	1.439	0.167	1.233	2.726
	Min of V13	-0.219	0.167	-0.188	-0.415
	Max of V23	0.677	0.167	0.580	1.282
	Min of V23	-3.197	0.167	-2.740	-6.056

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Inner Side Wall 2

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

INNER SIDE WALL 2 MOMENT & SHEAR SUMMARY:

Maximum M11 =	10.836	kip-ft/ft
Minimum M11 =	-2.254	kip-ft/ft
Maximum M22 =	17.390	kip-ft/ft
Minimum M22 =	-5.635	kip-ft/ft
Maximum V13 =	1.498	kip-ft/ft
Minimum V13 =	-4.048	kip-ft/ft
Maximum V23 =	5.620	kip-ft/ft
Minimum V23 =	-4.342	kip-ft/ft

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
A1_a	Max of M11	2.8566	0.167	2.448	5.411
	Min of M11	-0.5006	0.167	-0.429	-0.948
	Max of M22	5.8214	0.167	4.990	11.027
	Min of M22	-2.4978	0.167	-2.141	-4.731
	Max of V13	0.448	0.167	0.384	0.849
	Min of V13	-0.999	0.167	-0.856	-1.892
	Max of V23	0.987	0.167	0.846	1.870
	Min of V23	-1.016	0.167	-0.871	-1.925
A1_b	Max of M11	2.8478	0.167	2.441	5.394
	Min of M11	-0.4991	0.167	-0.428	-0.945
	Max of M22	5.7999	0.167	4.971	10.986
	Min of M22	-2.4916	0.167	-2.136	-4.720
	Max of V13	0.447	0.167	0.383	0.847
	Min of V13	-0.996	0.167	-0.854	-1.887
	Max of V23	0.984	0.167	0.843	1.864
	Min of V23	-1.013	0.167	-0.868	-1.919
B1_m	Max of M11	1.9823	0.000	1.982	4.381
	Min of M11	-0.5166	0.000	-0.517	-1.142
	Max of M22	5.2706	0.000	5.271	11.648
	Min of M22	-1.8134	0.000	-1.813	-4.008
	Max of V13	0.368	0.000	0.368	0.813
	Min of V13	-0.824	0.000	-0.824	-1.821
	Max of V23	0.899	0.000	0.899	1.987
	Min of V23	-0.825	0.000	-0.825	-1.823
B1_s	Max of M11	1.978	0.000	1.978	4.371
	Min of M11	-0.3423	0.000	-0.342	-0.756
	Max of M22	4.2502	0.000	4.250	9.393
	Min of M22	-1.7187	0.000	-1.719	-3.798
	Max of V13	0.321	0.000	0.321	0.709
	Min of V13	-0.709	0.000	-0.709	-1.567
	Max of V23	0.72	0.000	0.720	1.591
	Min of V23	-0.719	0.000	-0.719	-1.589
B3_m	Max of M11	2.0877	0.000	2.088	4.614
	Min of M11	-0.4248	0.000	-0.425	-0.939
	Max of M22	5.3176	0.000	5.318	11.752
	Min of M22	-1.8985	0.000	-1.899	-4.196
	Max of V13	0.377	0.000	0.377	0.833
	Min of V13	-0.844	0.000	-0.844	-1.865
	Max of V23	0.907	0.000	0.907	2.004
	Min of V23	-0.847	0.000	-0.847	-1.872
B3_s	Max of M11	2.0835	0.000	2.084	4.605
	Min of M11	-0.3601	0.000	-0.360	-0.796
	Max of M22	4.2971	0.000	4.297	9.497
	Min of M22	-1.8105	0.000	-1.811	-4.001
	Max of V13	0.329	0.000	0.329	0.727
	Min of V13	-0.729	0.000	-0.729	-1.611
	Max of V23	0.728	0.000	0.728	1.609
	Min of V23	-0.741	0.000	-0.741	-1.638

Job: Maurepas Pump Station
Project No.: 10001663
Description: Summary of Moments Shears in Inner Side Wall 2

Computed By: JY
Date: 09/2010
Checked By: EY
Date: 12/10

OutputCase	Data	Total	Allowable O.S	Forces w/Overstress	Forces*1.3*1.7
B4_m	Max of M11	2.0685	0.167	1.773	3.918
	Min of M11	-0.5077	0.167	-0.435	-0.962
	Max of M22	5.4364	0.167	4.660	10.298
	Min of M22	-1.8861	0.167	-1.617	-3.573
	Max of V13	0.381	0.167	0.327	0.722
	Min of V13	-0.853	0.167	-0.731	-1.616
	Max of V23	0.927	0.167	0.795	1.756
	Min of V23	-0.854	0.167	-0.732	-1.618
B4_s	Max of M11	2.0642	0.167	1.769	3.910
	Min of M11	-0.3567	0.167	-0.306	-0.676
	Max of M22	4.4159	0.167	3.785	8.365
	Min of M22	-1.7934	0.167	-1.537	-3.397
	Max of V13	0.334	0.167	0.286	0.633
	Min of V13	-0.738	0.167	-0.633	-1.398
	Max of V23	0.748	0.167	0.641	1.417
	Min of V23	-0.748	0.167	-0.641	-1.417
C1_a	Max of M11	2.0382	1.330	0.875	1.933
	Min of M11	-0.355	1.330	-0.152	-0.337
	Max of M22	4.2039	1.330	1.804	3.987
	Min of M22	-1.7646	1.330	-0.757	-1.674
	Max of V13	0.322	1.330	0.138	0.305
	Min of V13	-0.715	1.330	-0.307	-0.678
	Max of V23	0.713	1.330	0.306	0.676
	Min of V23	-0.726	1.330	-0.312	-0.689
C1_b	Max of M11	2.0294	1.330	0.871	1.925
	Min of M11	-0.3535	1.330	-0.152	-0.335
	Max of M22	4.1825	1.330	1.795	3.967
	Min of M22	-1.7584	1.330	-0.755	-1.668
	Max of V13	0.32	1.330	0.137	0.304
	Min of V13	-0.711	1.330	-0.305	-0.674
	Max of V23	0.71	1.330	0.305	0.673
	Min of V23	-0.723	1.330	-0.310	-0.686
D1_m	Max of M11	0.8786	0.167	0.753	1.664
	Min of M11	-0.6425	0.167	-0.551	-1.217
	Max of M22	3.0912	0.167	2.650	5.855
	Min of M22	-0.804	0.167	-0.689	-1.523
	Max of V13	0.193	0.167	0.165	0.366
	Min of V13	-0.433	0.167	-0.371	-0.820
	Max of V23	0.53	0.167	0.454	1.004
	Min of V23	-0.424	0.167	-0.363	-0.803
D1_s	Max of M11	0.8368	0.167	0.717	1.585
	Min of M11	-0.3068	0.167	-0.263	-0.581
	Max of M22	2.0707	0.167	1.775	3.922
	Min of M22	-0.685	0.167	-0.587	-1.298
	Max of V13	0.145	0.167	0.124	0.275
	Min of V13	-0.318	0.167	-0.273	-0.602
	Max of V23	0.351	0.167	0.301	0.665
	Min of V23	-0.318	0.167	-0.273	-0.602
D2_m	Max of M11	5.7206	0.167	4.903	10.836
	Min of M11	-1.1893	0.167	-1.019	-2.253
	Max of M22	9.1807	0.167	7.869	17.390
	Min of M22	-2.8769	0.167	-2.466	-5.450
	Max of V13	0.791	0.167	0.678	1.498
	Min of V13	-2.137	0.167	-1.832	-4.048
	Max of V23	2.953	0.167	2.531	5.594
	Min of V23	-2.292	0.167	-1.965	-4.342
D2_s	Max of M11	5.7164	0.167	4.900	10.828
	Min of M11	-1.19	0.167	-1.020	-2.254
	Max of M22	9.1302	0.167	7.826	17.295
	Min of M22	-2.9747	0.167	-2.550	-5.635
	Max of V13	0.744	0.167	0.638	1.409
	Min of V13	-2.022	0.167	-1.733	-3.830
	Max of V23	2.967	0.167	2.543	5.620
	Min of V23	-2.192	0.167	-1.879	-4.152

Job - Maurepas Pumpstation
Project # 10001663

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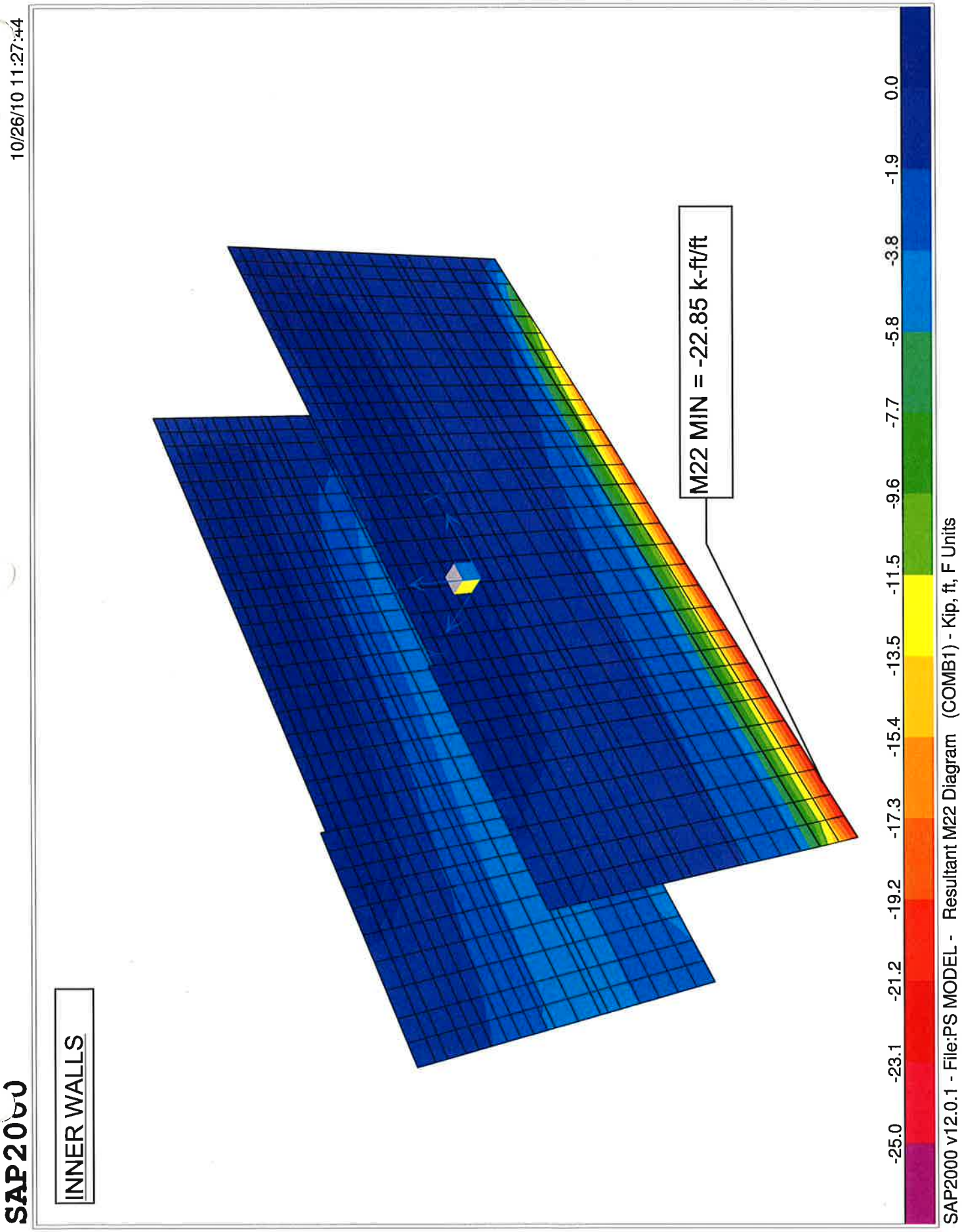
Date 10/10
EY 12/10



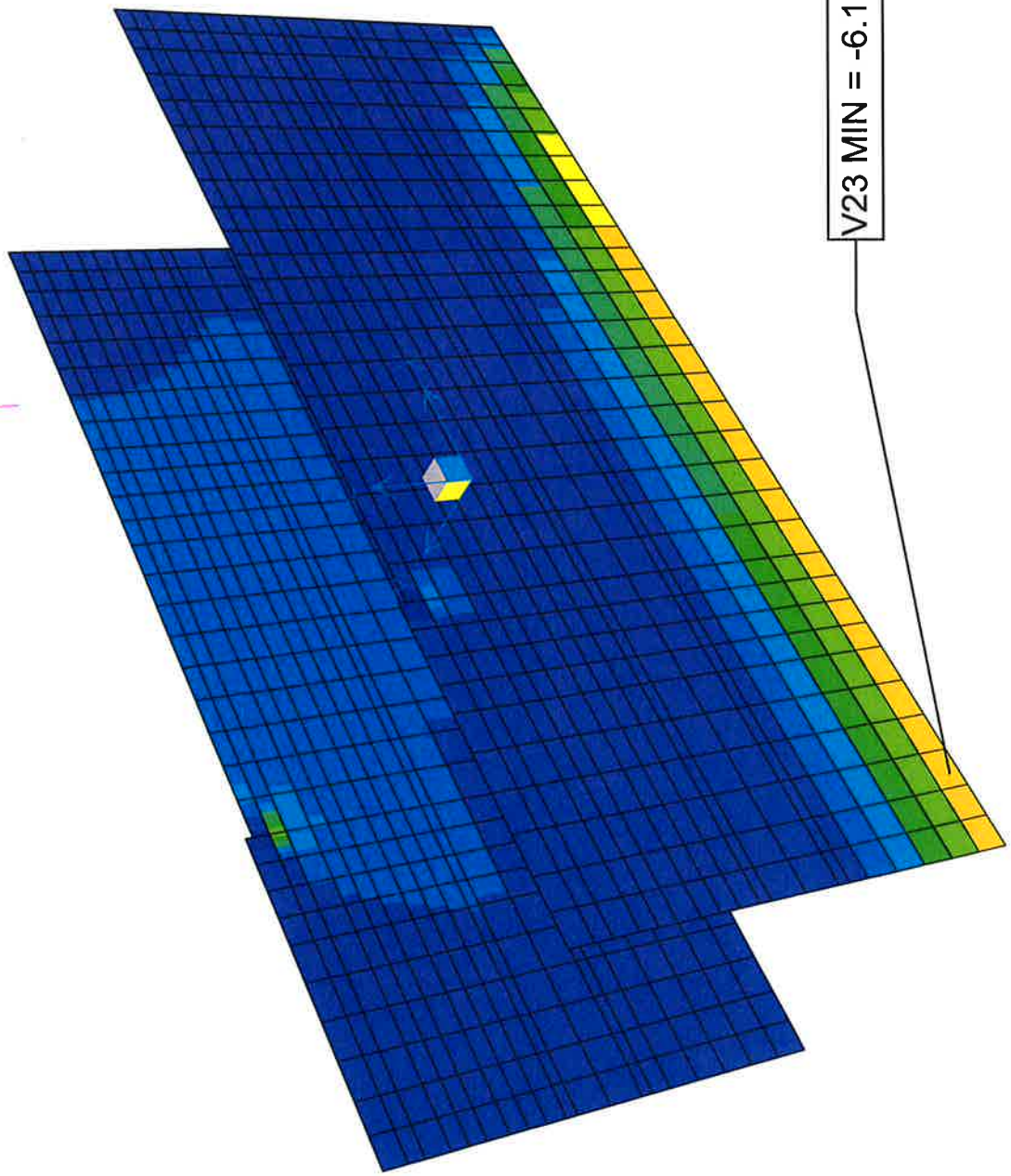
Job - Maurepas Pumpstation
Project # 10001663

Date 10/10
EY 12/10

10/26/10 11:29:32

SAP2000

INNER WALLS



-10.0 -9.2 -8.5 -7.7 -6.9 -6.2 -5.4 -4.6 -3.8 -3.1 -2.3 -1.5 -0.8 0.0

SAP2000 v12.0.1 - File:PS MODEL - Resultant V23 Diagram (COMB1) - Kip, ft, F Units

SECTION 8

Top Slab Design

Job: Maurepas Pump Station
Project No.: 10001663
Description: Design of 1'3" slab

Calculated By: JY
Date: 10/2010
Checked By: EJ
Date: 12/10

Design of 1.25' Slab:

b =	12 in	
d =	12.69 in	(2" clear cover and #5 bar)
h =	15 in	
fy =	60000 psi	
f'c =	4000 psi	

Shear Design:

Controlling Shear value = 13.8 k/ft (see attached drawings)

* Shear Capacity:

$\varphi Vc = \varphi*2*\sqrt{(f'c)}*b*d$ $\varphi = 0.85$ (ACI 318-02 Eq. 11-3)

$\varphi V_c =$ 16366.3 lbs/ft

$\varphi V_c =$ 16.37 kips/ft > 13.8 k/ft OK

Reinforcement Design:

Controlling mom. in Top & Bot. Faces Mu = 19.2 k-ft/ft (see attached drawings)

Capacity of Minimum Reinforcement for 1.25' thick slab:

Capacity of Minimum Temperature & Shrinkage Reinforcement:

$A_{s,\,min} = \rho_{min}*b*d$ $\rho_{min} = 0.0018$ (ACI 318-02 Ch. 7.12.2.1)

$A_{s,\,min} =$ 0.274 in^2/ft per face

$M_u = \varphi M_n = \varphi*A_s*f_y*(d-a/2)$ $\varphi = 0.9$ where $a = (A_s*f_y)/(0.85*f'_c*b)$

$M_u =$ 184704 lb-in/ft

= 15.39 k-ft/ft < 19.2 k-ft/ft **NOT OK**

Capacity of Minimum Reinforcement required for flexural members:

$A_{s,\,min} = (200/f_y)*b*d$ (ACI 318-02 Ch. 10.5.1)

$A_{s,\,min} =$ 0.507 in^2/ft per face

$M_u = \varphi M_n = \varphi*A_s*f_y*(d-a/2)$ $\varphi = 0.9$ where $a = (A_s*f_y)/(0.85*f'_c*b)$

$M_u =$ 342044 lb-in/ft

= 28.50 k-ft/ft > 19.2 k-ft/ft **OK, PROVIDE MIN. R/F**

Therefore, provide 1.00 in^2/ft per face i.e., #5 @ 3" (1.24 in^2/ft).

Minimum reinforcement = 0.507 in^2/ft

Provide #5 @ 6" O.C.=0.62 in^2/ft at Top and Bottom faces of the slab.

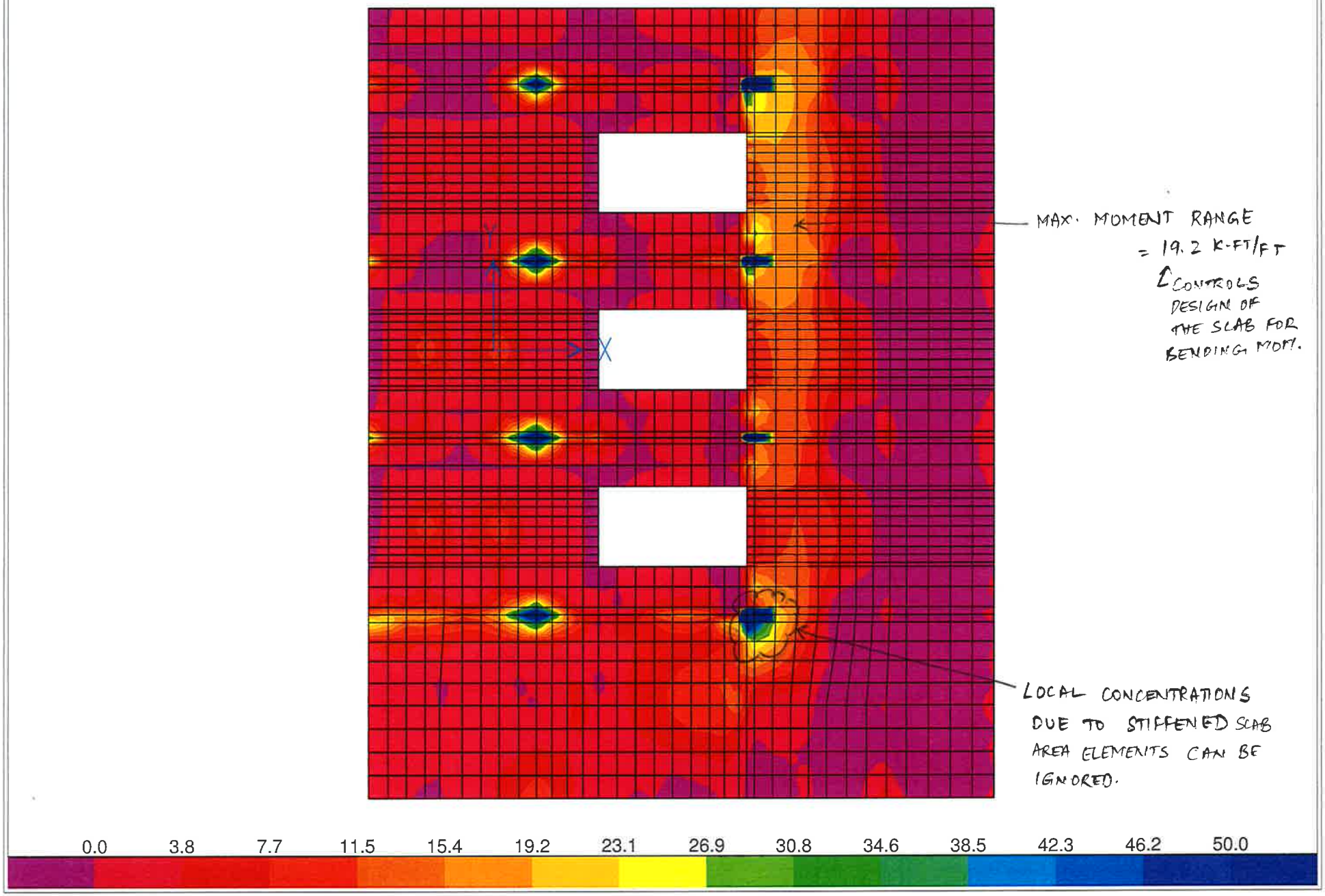
* Maximum Reinforcement:

$\rho_{max} = 0.375*\rho_{bal}$ $\rho_{bal} = 0.0285$ (ACI R.10.3.5)

$\rho_{max} =$ 0.01069 per face

$A_{s,\,max} =$ 1.924 in^2/ft > 0.62 in^2/ft **OK**

BENDING MOMENT CONTOUR DIAGRAM @ BOTTOM OF THE SLAB (+VE MOMENT)

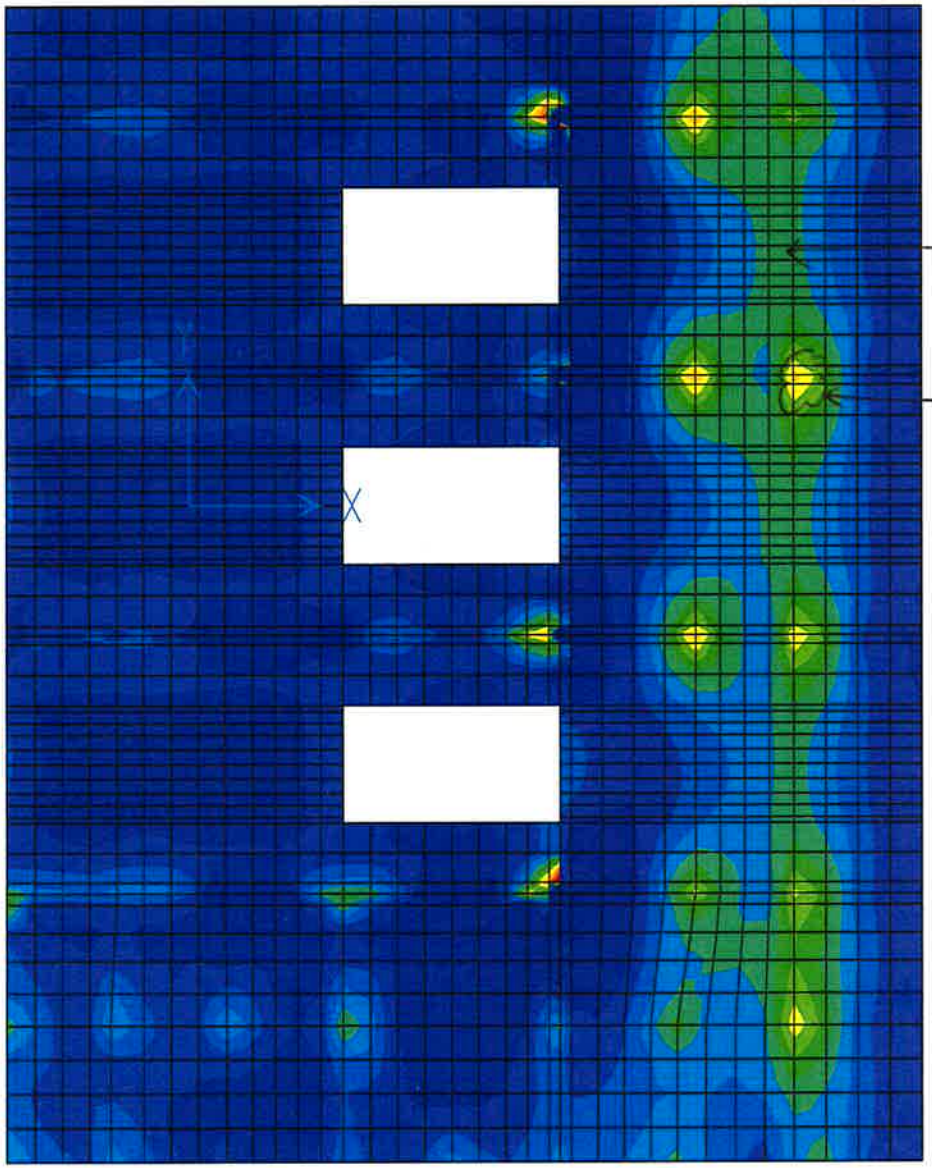


MAX. MOMENT RANGE = 19.2 K-FT/FT
CONTROLS DESIGN OF THE SLAB FOR BENDING MOM.

LOCAL CONCENTRATIONS DUE TO STIFFENED SLAB AREA ELEMENTS CAN BE IGNORED.

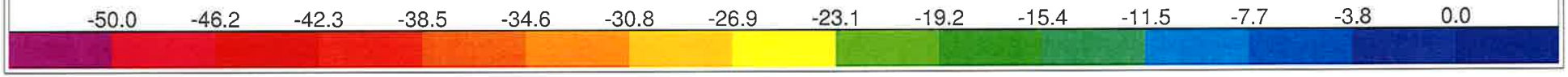
SAP2000 v14.2.3 - File:PS MODEL - Resultant M11 Diagram (COMB1 - Max) - Kip, ft, F Units

BENDING MOMENT CONTOUR DIAGRAM @ TOP FACE OF THE SLAB (-VE MOMENT)



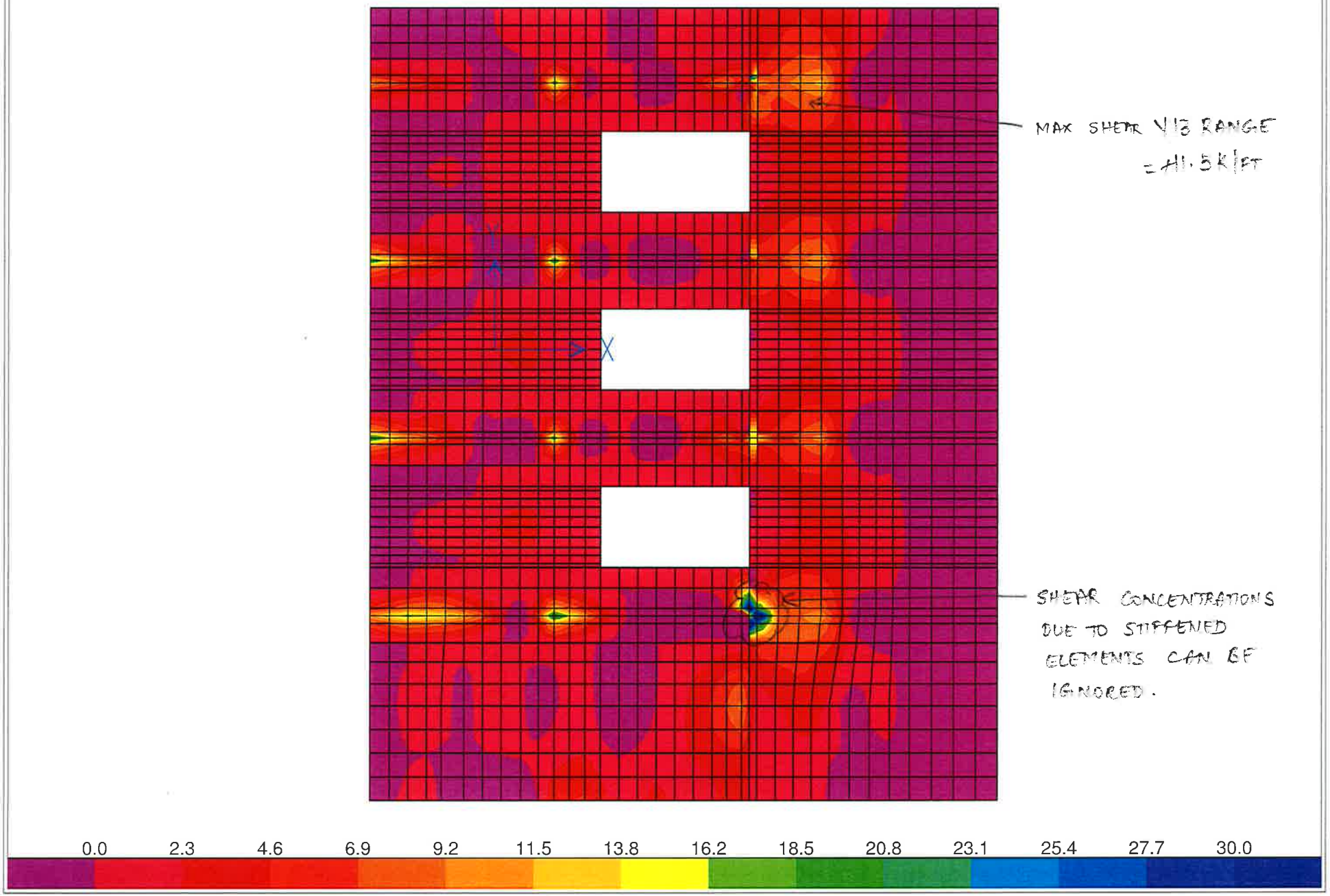
HIGHEST MOMENT RANGE = -15.4 K-FT/FT

PILE CONCENTRATED MOMENTS CAN BE IGNORED.



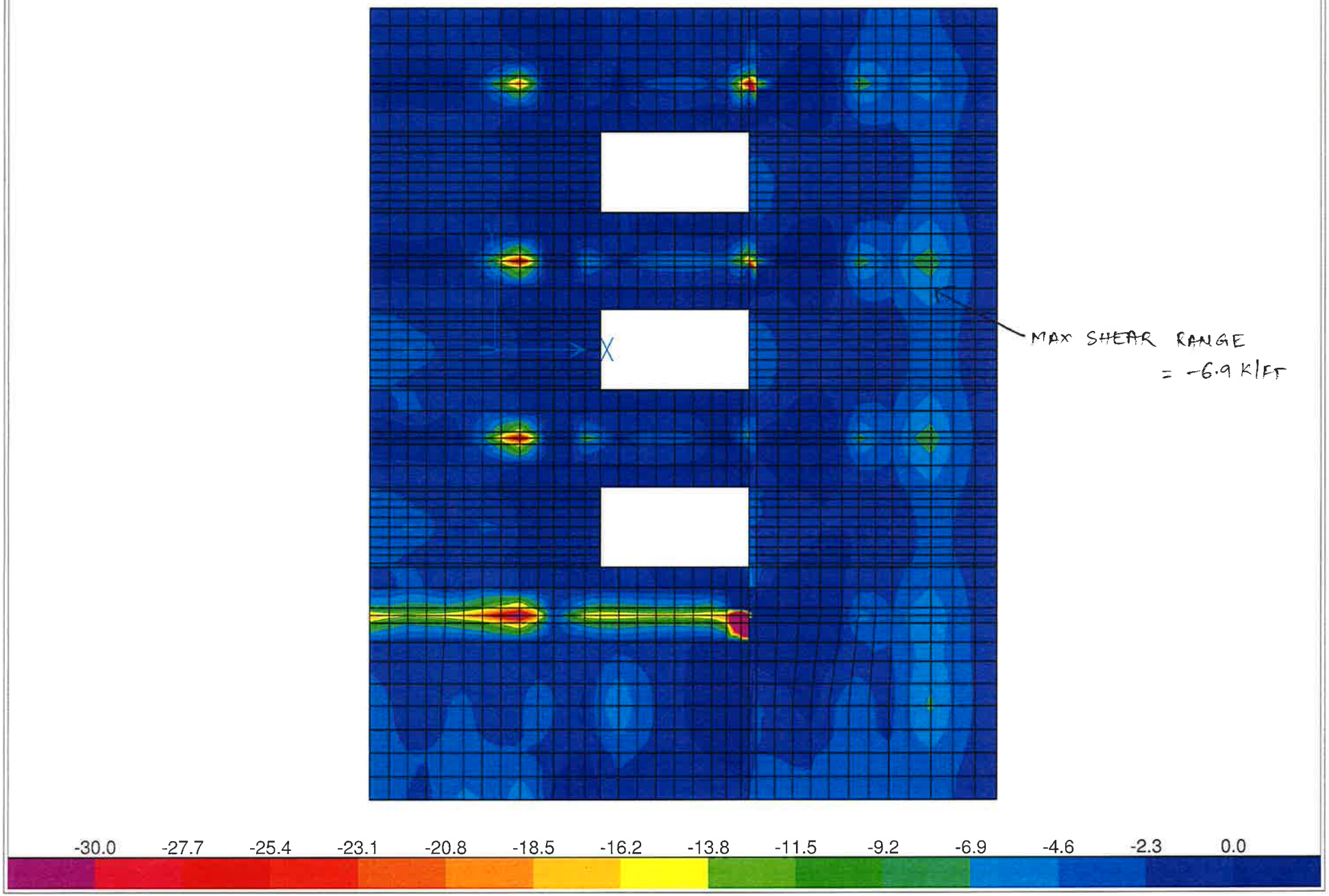
SAP2000 v14.2.3 - File:PS MODEL - Resultant M11 Diagram (COMB1 - Min) - Kip, ft, F Units

POSITIVE SHEAR CONTOUR IN SLAB (+V13):



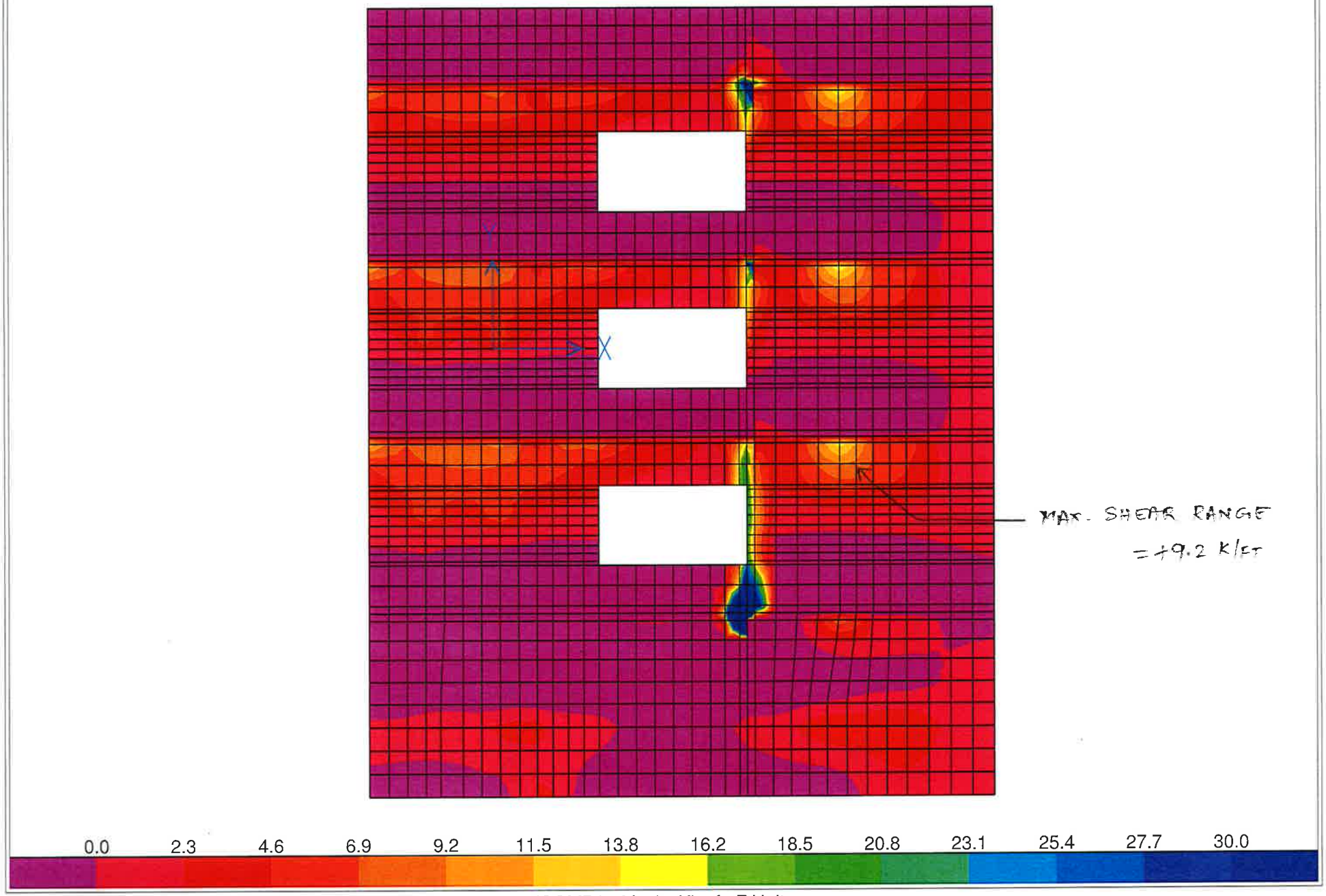
SAP2000 v14.2.3 - File:PS MODEL - Resultant V13 Diagram (COMB1 - Max) - Kip, ft, F Units

NEGATIVE SHEAR CONTOUR IN SLAB (-V13):



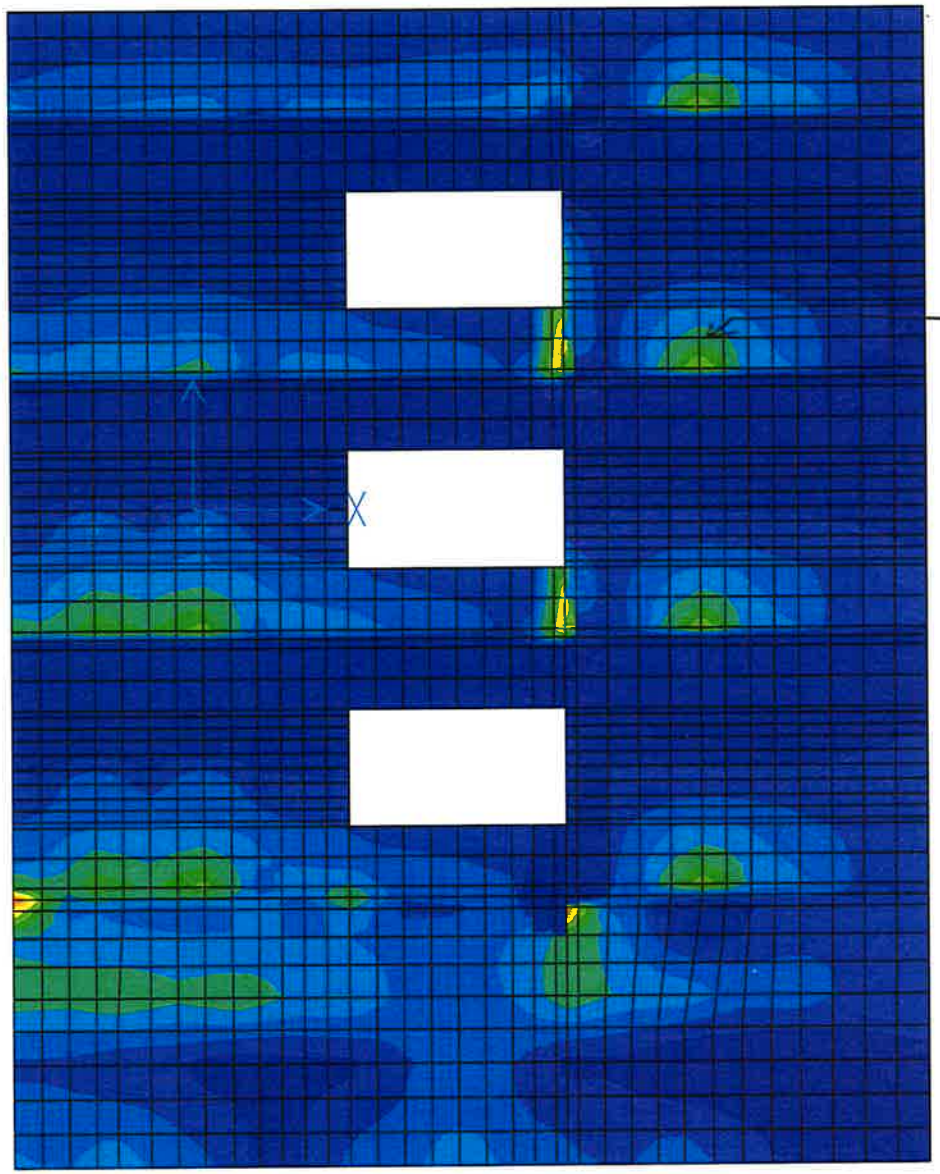
SAP2000 v14.2.3 - File:PS MODEL - Resultant V13 Diagram (COMB1 - Min) - Kip, ft, F Units

POSITIVE SHEAR CONTOUR IN SLAB (+V23):

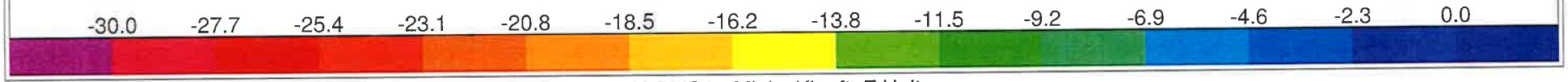


SAP2000 v14.2.3 - File:PS MODEL - Resultant V23 Diagram (COMB1 - Max) - Kip, ft, F Units

NEGATIVE SHEAR CONTOUR IN SLAB (-V23) :



MAX. SHEAR RANGE
= -13.8 K/FT
CONTROLS SHEAR DESIGN OF THE SLAB.



SAP2000 v14.2.3 - File:PS MODEL - Resultant V23 Diagram (COMB1 - Min) - Kip, ft, F Units

Job: Maurepas Pump Station
Project No.: 10001663
Description: Design of reinforcement for pump openings

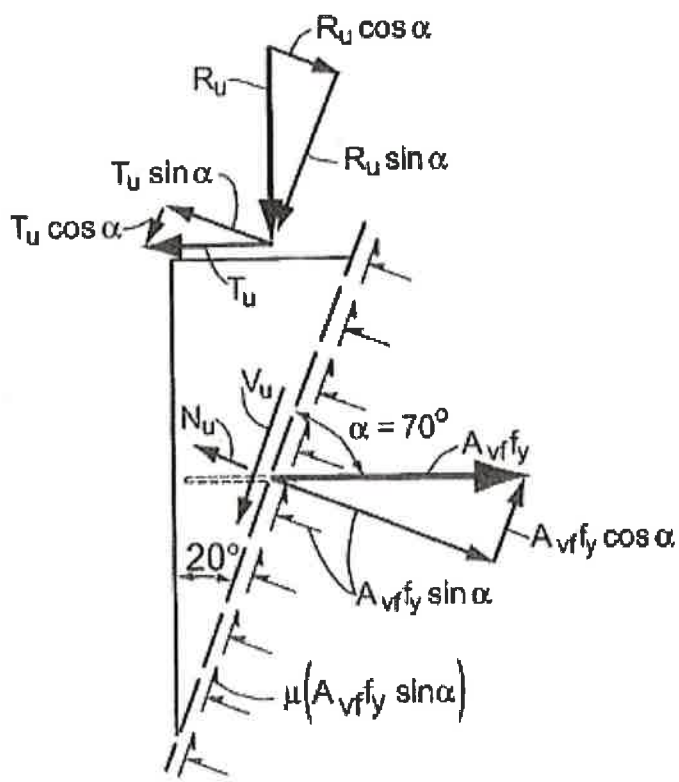
Calculated By: JY 02/11
Checked By: LBL 8/2013

References:

* Notes on ACI 318-05 Building Code Requirements for Structural Concrete. Portland Cement Association. 2005
* ACI 318-02

Symbols:

R_u	Pump Reaction
T_u	Shrinkage, Temperature & Vibration Effects (estimated as 10% total force)
V_u	Direct Shear Transfer Force
N_u	Net Tension Across Shear Plane
A_{vf}	Shear-friction Reinforcement for Direct Shear Transfer
A_n	Net Tension Reinforcement
A_s	Total Area of Reinforcement
V_{nmax}	Maximum Shear-Transfer Strength
A_c	Area of Concrete
α	Angle of Shear Plane, 20 degrees



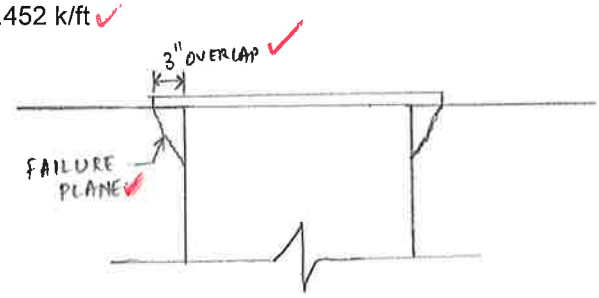
Job: Maurepas Pump Station
Project No.: 10001663
Description: Design of reinforcement for pump openings

Calculated By: JY 02/11
Checked By: LBL 8/2013

Given Information:

Opening size for the Pump & Gear Drive = 8' x 6'-6" ; Plate Size = 8'-6" x 7'

Load due to the Pump and Gear Drive = 13.1 k (Calculated earlier)
Distributing this load over the perimeter of 8' x 2 + 6'-6" x 2 = 29 ft
Therefore, Load of the concrete edges = 13.1k / 29' = 0.452 k/ft



$R_u := 0.452 \cdot kip$

$T_u := R_u \cdot 0.1$

$T_u = 0.045 \cdot kip$

$\alpha := 20 \cdot °$

$\lambda := 1$

$\mu := 1.4 \cdot \lambda = 1.4$ MONOLITHIC CONCRETE

$\Phi := .85$

$f_y := 60 \cdot ksi$

Calculations:

$V_u := R_u \cdot \sin(\alpha) + T_u \cdot \cos(\alpha)$

$V_u = 0.197 \cdot kip$

$N_u := T_u \cdot \sin(\alpha) - R_u \cdot \cos(\alpha)$ * negative value means net compression

$N_u = -0.409 \cdot kip$

$$A_{vf} := \frac{V_u}{\Phi \cdot f_y \cdot (\mu \cdot \sin(\alpha) + \cos(\alpha))}$$

$A_{vf} = 2.724 \times 10^{-3} \cdot in^2$

$$A_n := \frac{N_u}{\Phi \cdot f_y \cdot \sin(\alpha)}$$

* Because area is in net compression, no tension reinforcement needed.

$A_n = -0.023 \cdot in^2$

$A_s := A_{vf}$

$A_s = 2.724 \times 10^{-3} \cdot in^2$

** Minimal reinforcement required to support pump. Current slab reinforcement adequate.

(160c)

SECTION 9

Beam Designs

Job: Maurepas Pump Station
Project No.: 10001663
Description: Design of Beams

Calculated By: JY
Date: 10/2010
Checked By: EY
Date: 12/2010

Design of 2'2"x2' Beams:

b =	26 in	
d =	19.5 in	(4" clear cover and #8 bar)
h =	24 in	
fy =	60000 psi	
f'c =	4000 psi	

Shear Design:

Controlling Shear value = 41.76 k/ft (see attached drawings-Beam B8)

* Shear Capacity:

$\varphi Vc = \varphi*2*\sqrt{(f'c)}*b*d$ $\varphi = 0.85$ (ACI 318-02 Eq. 11-3)

$\varphi V_c =$ 54511.3 lbs/ft

$\varphi V_c =$ 54.51 kips/ft > 41.76 k/ft OK

Reinforcement Design:

Controlling mom. in Top & Bot. Faces Mu = 296.5 k-ft/ft (see attached drawings-Beam B3)

Capacity of Minimum Reinforcement for 2'2' x 2' beam:

Capacity of Minimum Temperature & Shrinkage Reinforcement:

$A_{s,\,min} = \rho_{min}*b*d$ $\rho_{min} = 0.0018$ (ACI 318-02 Ch. 7.12.2.1)

$A_{s,\,min} =$ 0.913 in^2/ft per face

$M_u = \varphi M_n = \varphi*A_s*f_y*(d-a/2)$ $\varphi = 0.9$ where $a = (A_s*f_y)/(0.85*f'_c*b)$

$M_u =$ 945705.37 lb-in/ft

= 78.81 k-ft/ft < 296.5 k-ft/ft **NOT OK**

Capacity of Minimum Reinforcement required for flexural members:

$A_{s,\,min} = (200/f_y)*b*d$ (ACI 318-02 Ch. 10.5.1)

$A_{s,\,min} =$ 1.690 in^2/ft per face

$M_u = \varphi M_n = \varphi*A_s*f_y*(d-a/2)$ $\varphi = 0.9$ where $a = (A_s*f_y)/(0.85*f'_c*b)$

$M_u =$ 1751306.24 lb-in/ft

= 145.94 k-ft/ft < 296.5 k-ft/ft **NOT OK**

Area of Steel required ($A_{s,\,reqd}$) :

$A_{s,\,reqd} = (0.85*f'_c/f_y*b*d)[1-\sqrt{(1-(2*Mu/(0.85*f'_c*b*d^2*\varphi))}]$ $\varphi = 0.9$

$A_{s,\,reqd} =$ 3.61 in^2/ft per face

Therefore, provide 3.9 in^2 at each face conservatively i.e., 5 - #8 bars = 5x0.79 = 3.95 in^2 T & B.

* Maximum Reinforcement:

$\rho_{max} = 0.375*\rho_{bal}$ $\rho_{bal} = 0.0285$ (ACI R.10.3.5)

$\rho_{max} =$ 0.01069 per face

$A_{s,\,max} =$ 6.669 in^2/ft > 3.90 in^2/ft **OK**

Job: Maurepas Pump Station
Project No.: 10001663
Description: Design of Beams

Calculated By: JY
Date: 10/2010
Checked By: EY
Date: 12/2010

Design of 2'2"x2'2" Beams:

b =	26 in	
d =	21.5 in	(4" clear cover and #8 bar)
h =	26 in	
fy =	60000 psi	
f'c =	4000 psi	

Shear Design:

Controlling Shear value = 55.6 k/ft (see attached drawings-Beam B9)

* Shear Capacity:

$\varphi Vc = \varphi*2*\sqrt{(f'c)}*b*d$ $\varphi = 0.85$ (ACI 318-02 Eq. 11-3)

$\varphi V_c =$ 60102.2 lbs/ft

$\varphi V_c =$ 60.10 kips/ft > 55.6 k/ft OK

Reinforcement Design:

Controlling mom. in Top & Bot. Faces Mu = 278.2 k-ft/ft (see attached drawings-Beam B9)

Capacity of Minimum Reinforcement for 2'2' x 2'2" beam:

Capacity of Minimum Temperature & Shrinkage Reinforcement:

$A_{s,\,min} = \rho_{min}*b*d$ $\rho_{min} = 0.0018$ (ACI 318-02 Ch. 7.12.2.1)

$A_{s,\,min} =$ 1.006 in^2/ft per face

$M_u = \varphi M_n = \varphi*A_s*f_y*(d-a/2)$ $\varphi = 0.9$ where $a = (A_s*f_y)/(0.85*f'_c*b)$

$M_u =$ 1149644.5 lb-in/ft

= 95.80 k-ft/ft < 278.2 k-ft/ft **NOT OK**

Capacity of Minimum Reinforcement required for flexural members:

$A_{s,\,min} = (200/f_y)*b*d$ (ACI 318-02 Ch. 10.5.1)

$A_{s,\,min} =$ 1.863 in^2/ft per face

$M_u = \varphi M_n = \varphi*A_s*f_y*(d-a/2)$ $\varphi = 0.9$ where $a = (A_s*f_y)/(0.85*f'_c*b)$

$M_u =$ 2128971.23 lb-in/ft

= 177.41 k-ft/ft < 278.2 k-ft/ft **NOT OK**

Area of Steel required ($A_{s,\,reqd}$) :

$A_{s,\,reqd} = (0.85*f'_c/f_y*b*d)[1-\sqrt{(1-(2*Mu/(0.85*f'_c*b*d^2*\varphi))}]$ $\varphi = 0.9$

$A_{s,\,reqd} =$ 3.02 in^2/ft per face

Therefore, provide 3.16 in^2 at each face conservatively i.e., 4 - #8 bars = 4x0.79 = 3.16 in^2 T & B.

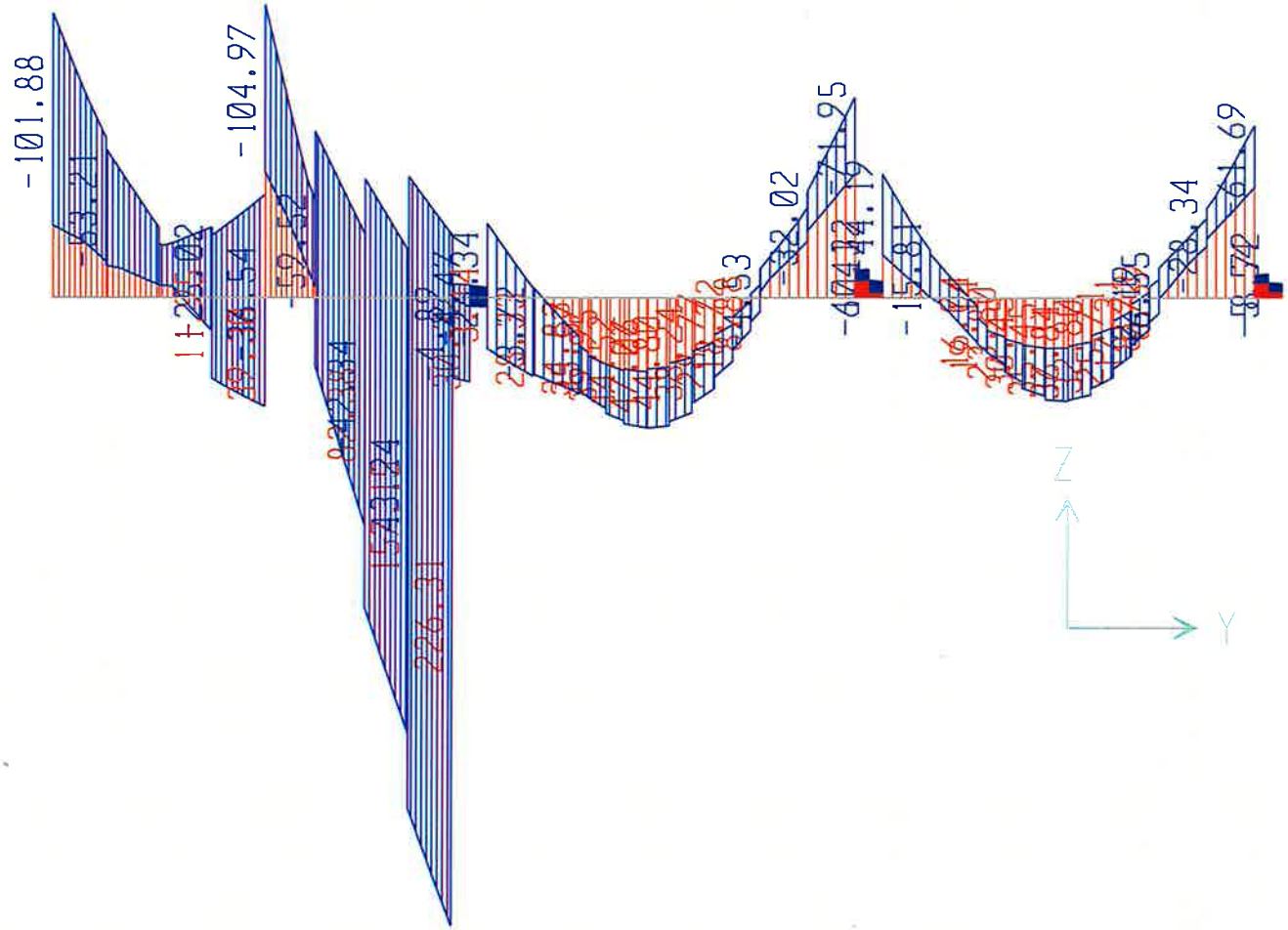
* Maximum Reinforcement:

$\rho_{max} = 0.375*\rho_{bal}$ $\rho_{bal} = 0.0285$ (ACI R.10.3.5)

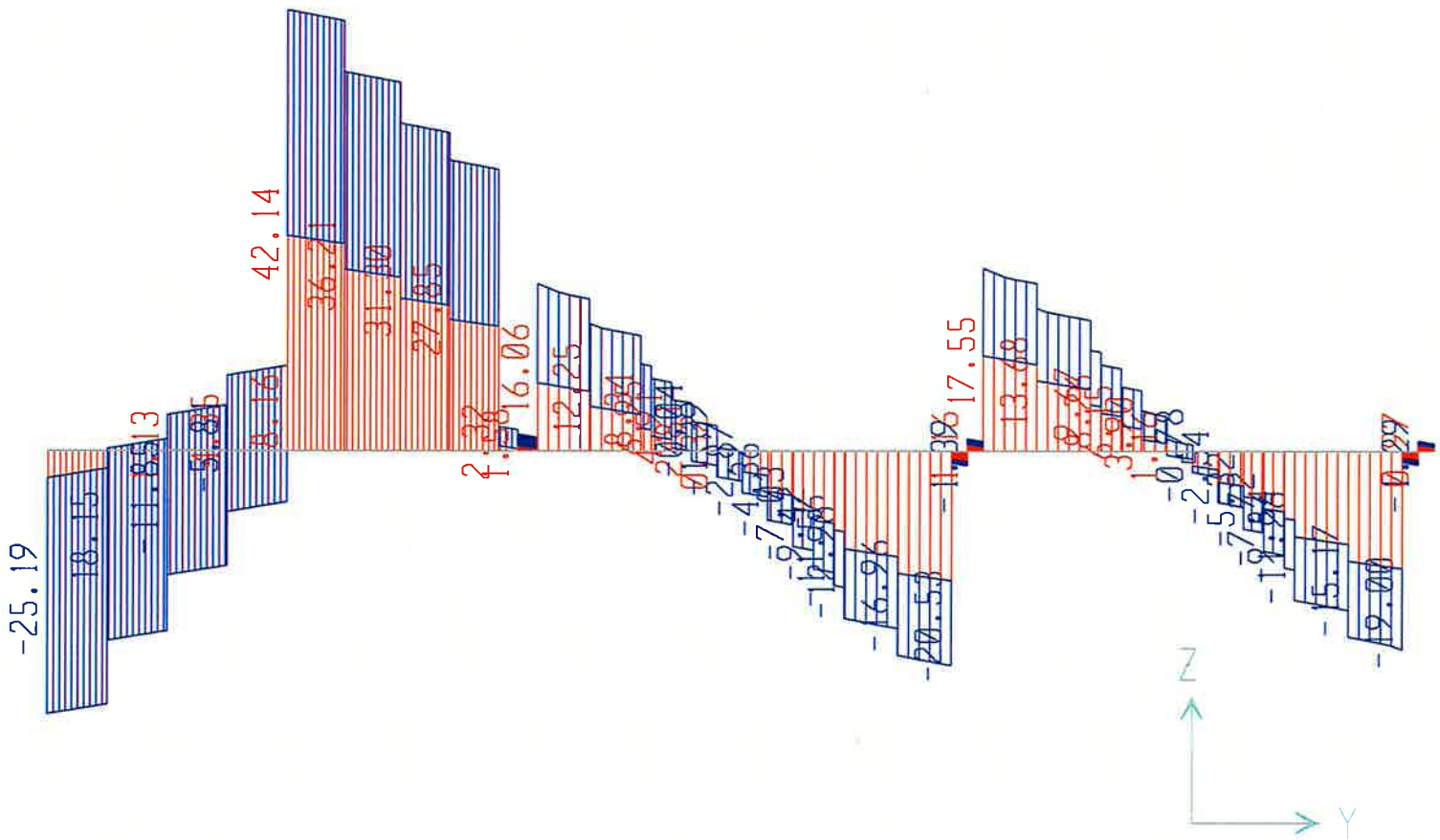
$\rho_{max} =$ 0.01069 per face

$A_{s,\,max} =$ 7.225 in^2/ft > 3.16 in^2/ft **OK**

BEAM B1 - BENDING MOMENT:



BEAM B1 - SHEAR :



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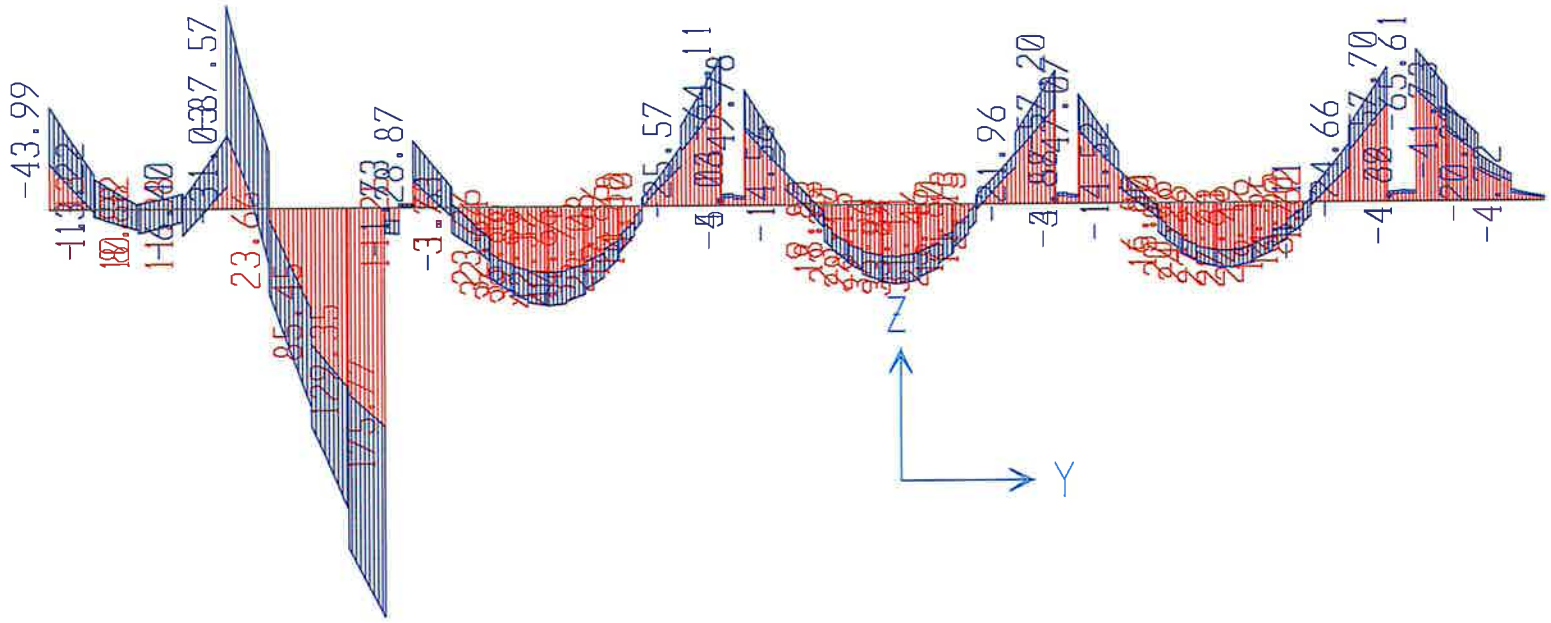
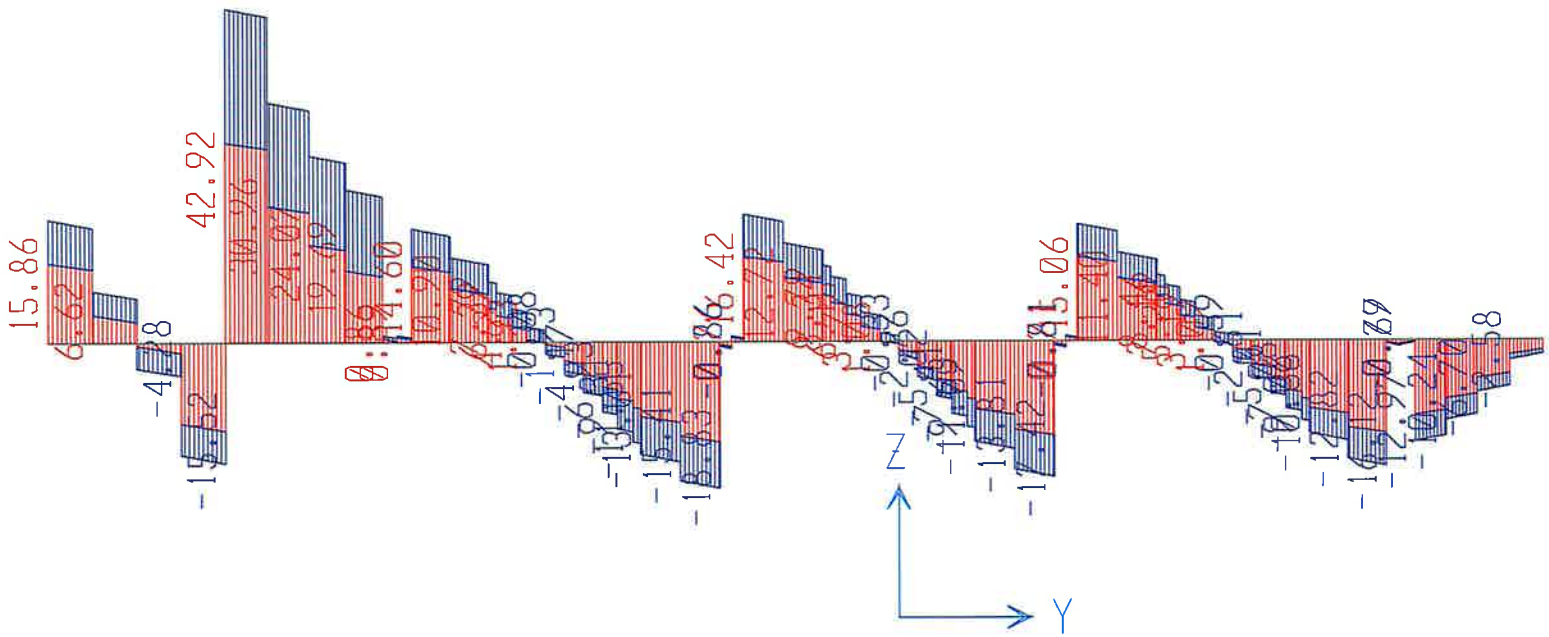
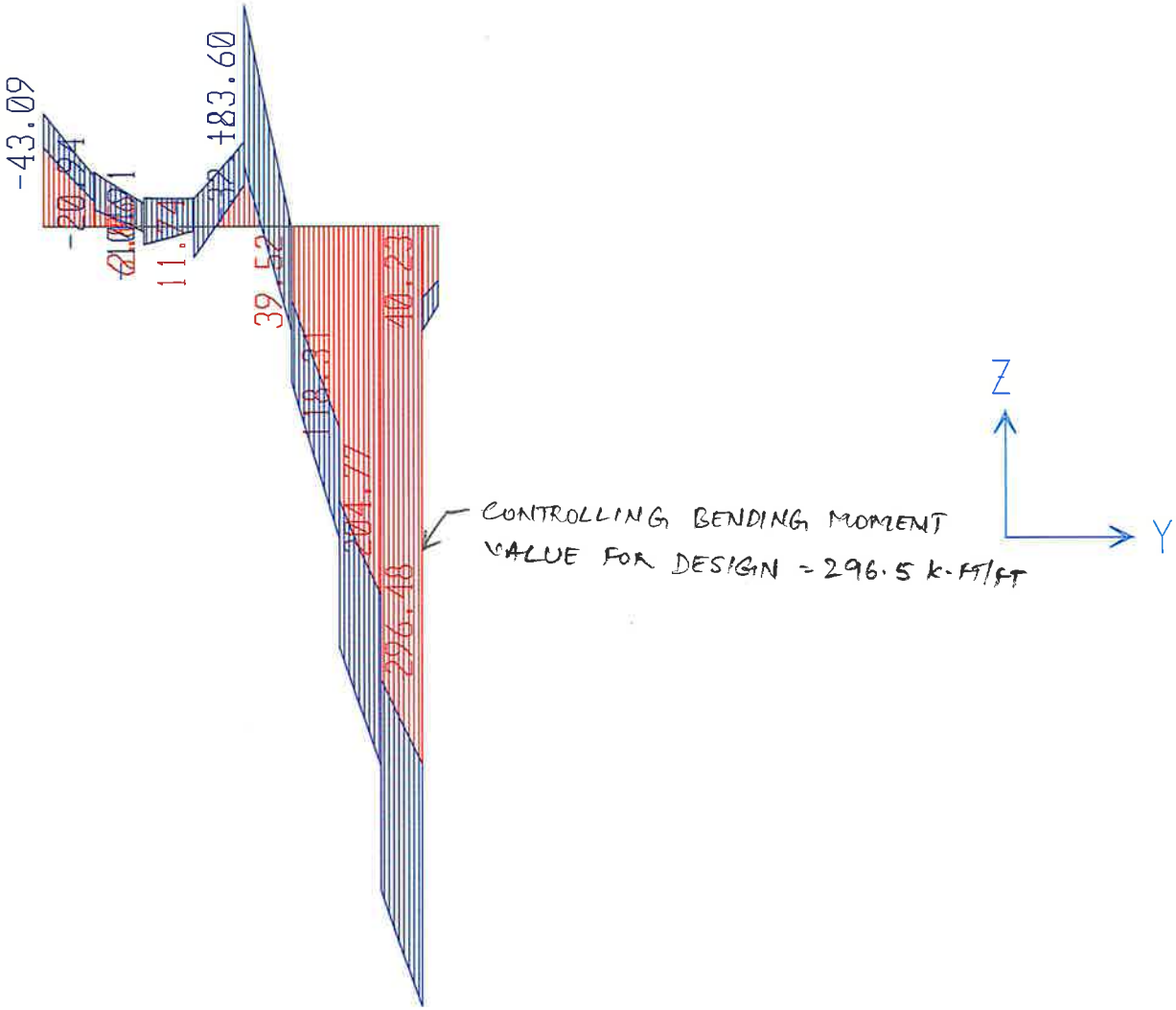
BEAM B2- BENDING MOMENT!



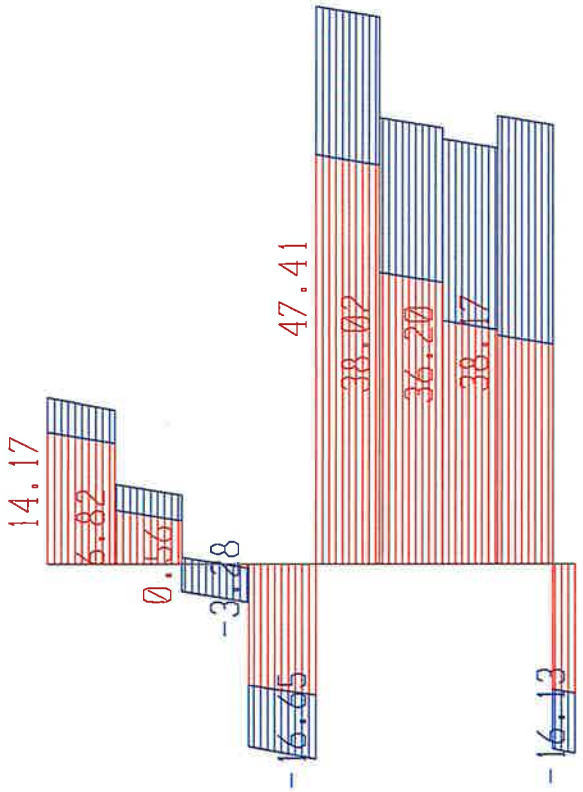
BEAM B2- SHEAR:

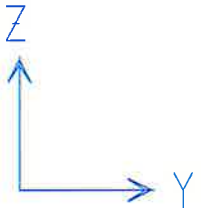


BEAM B3- BENDING MOMENT:

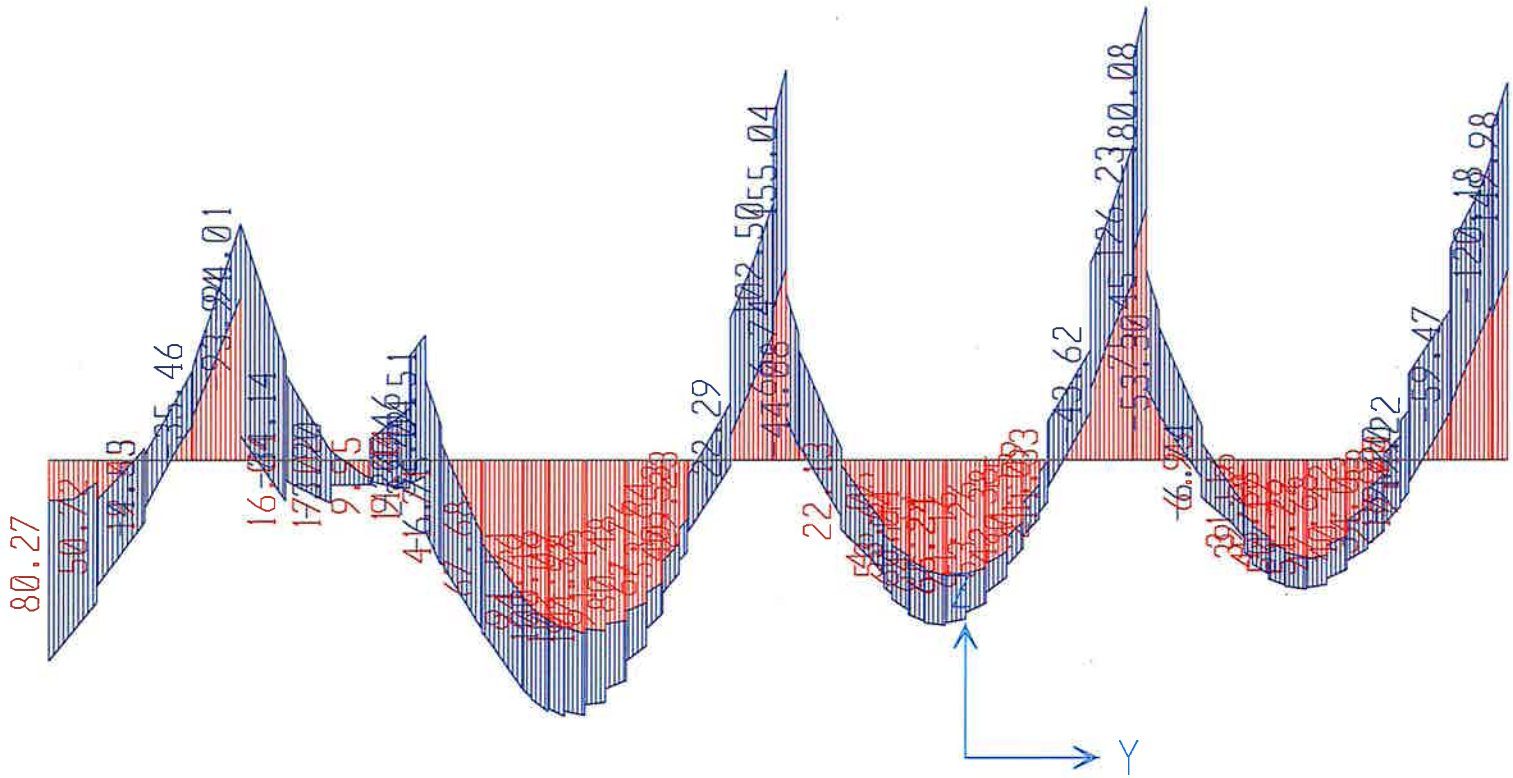


BEAM B3- SHEAR:

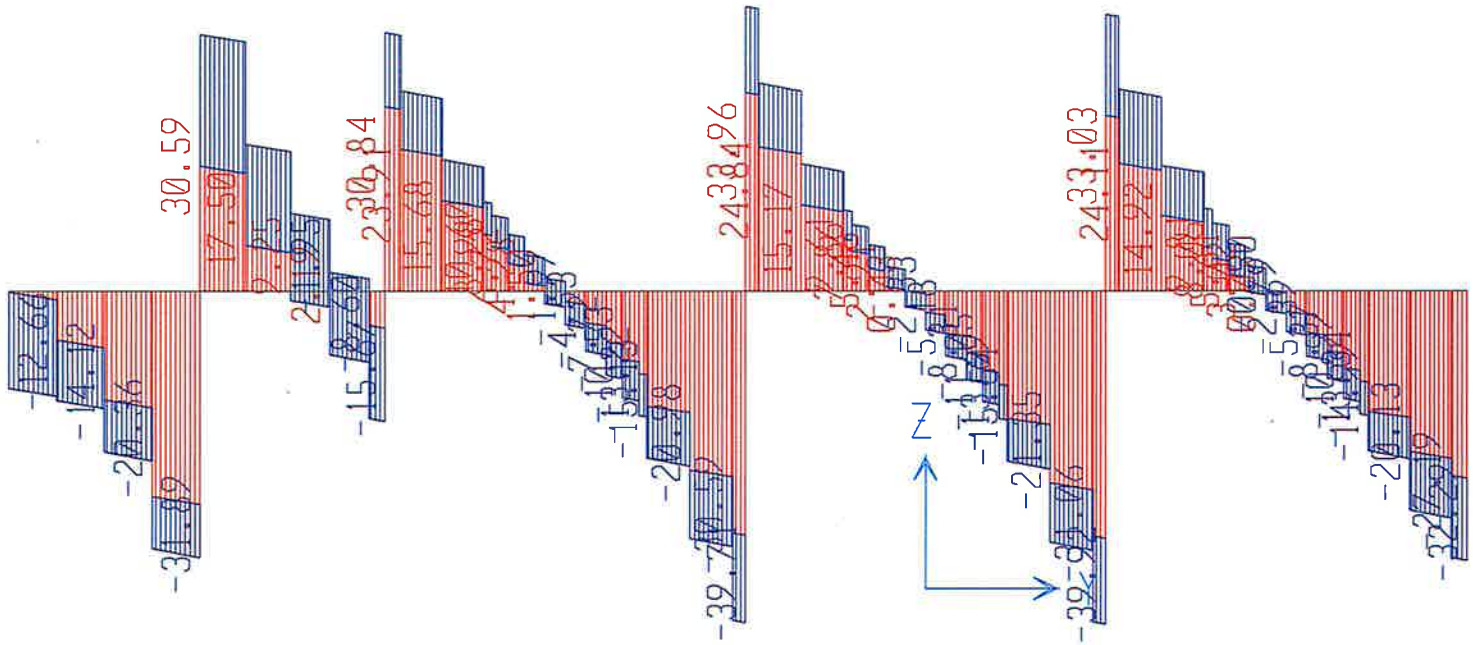




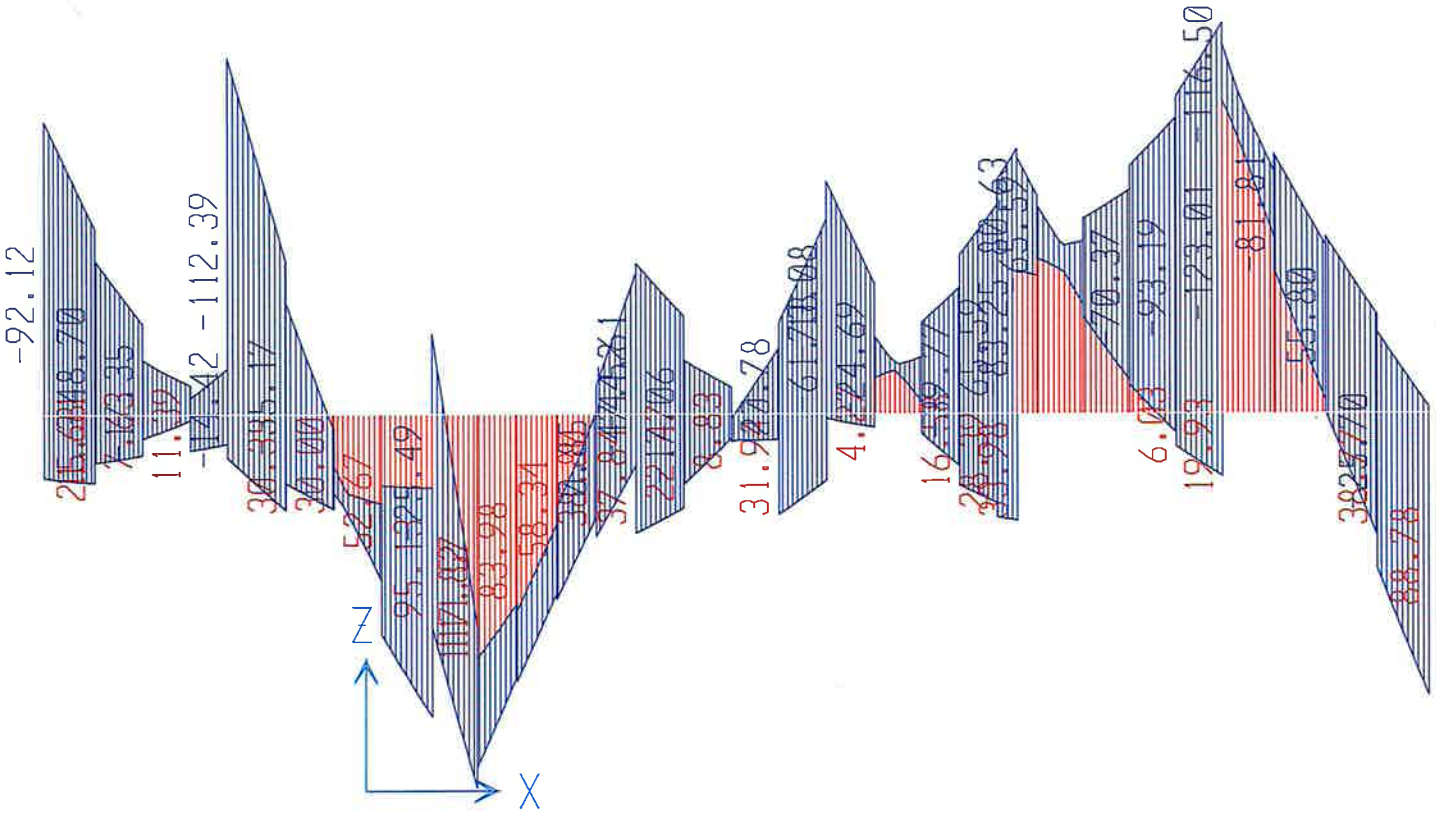
BEAM B4 - BENDING MOMENT:



BEAM B4- SHEAR:



BEAM B5- BENDING MOMENT!



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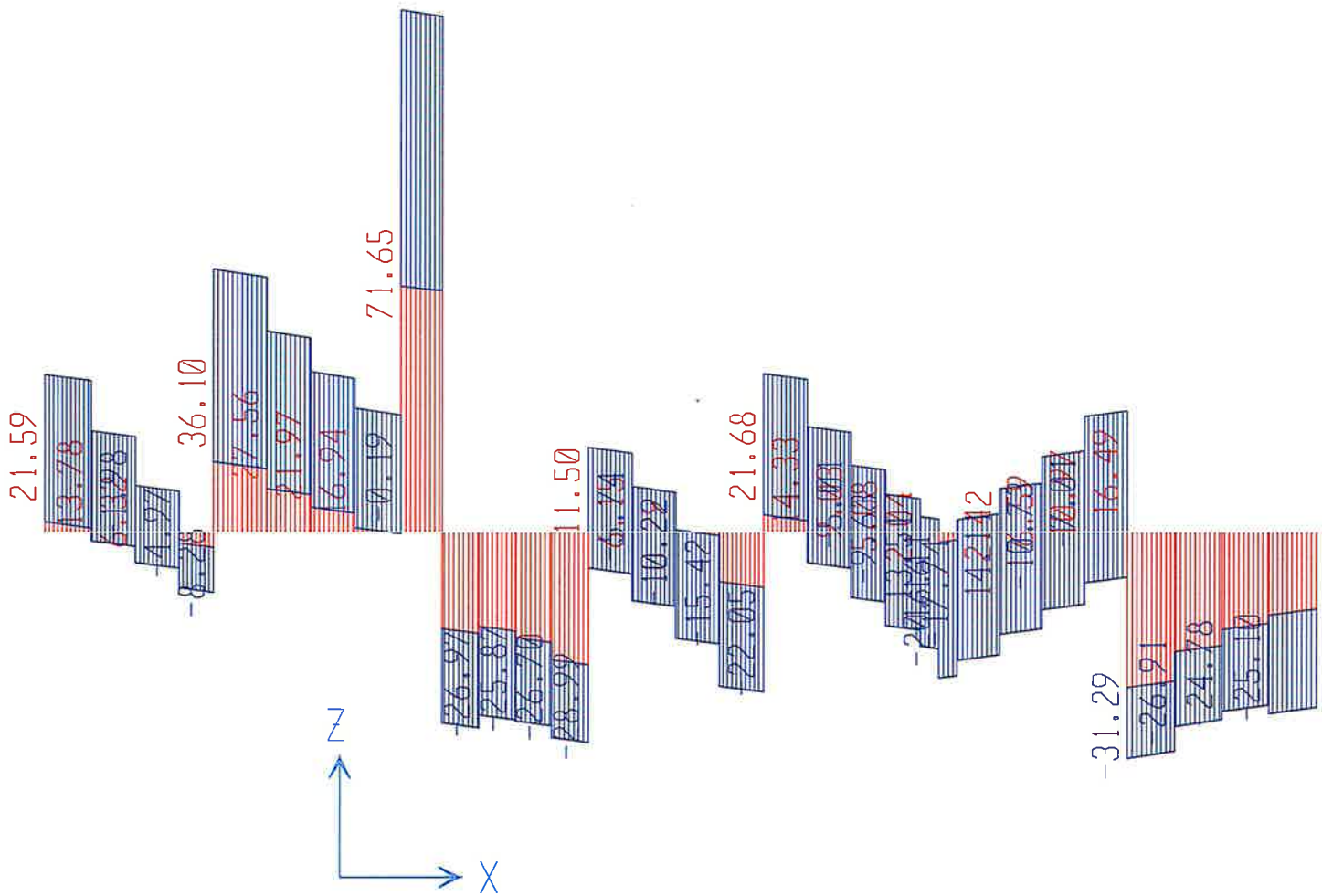
BEAM B5 - SHEAR:



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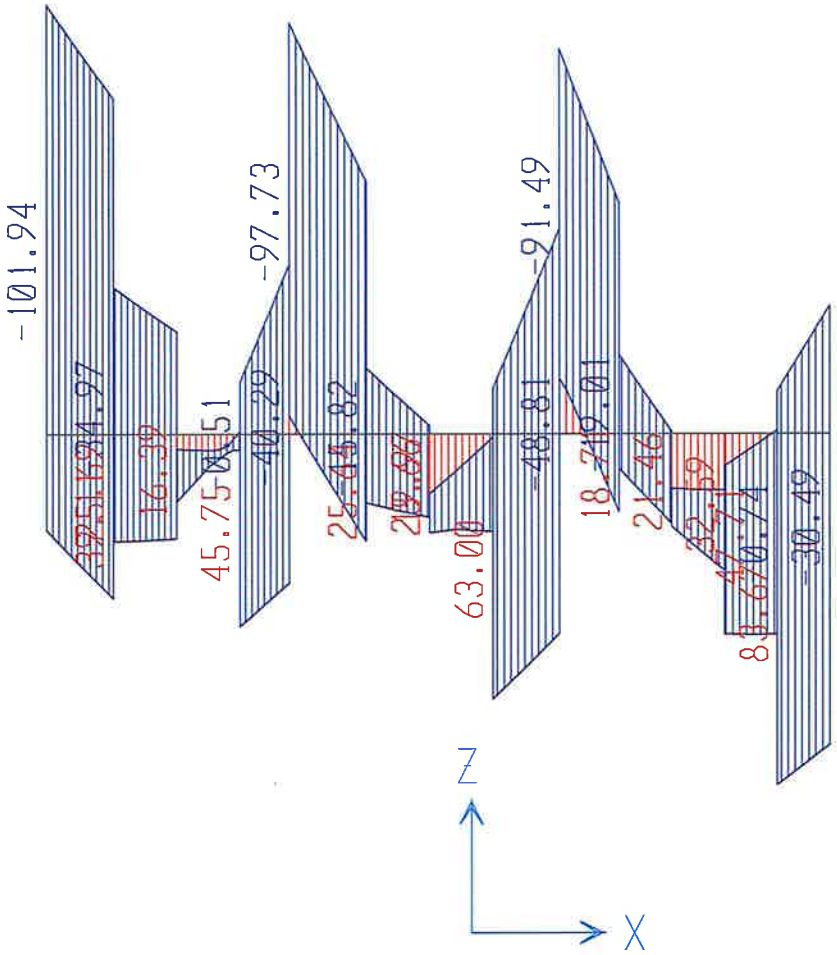
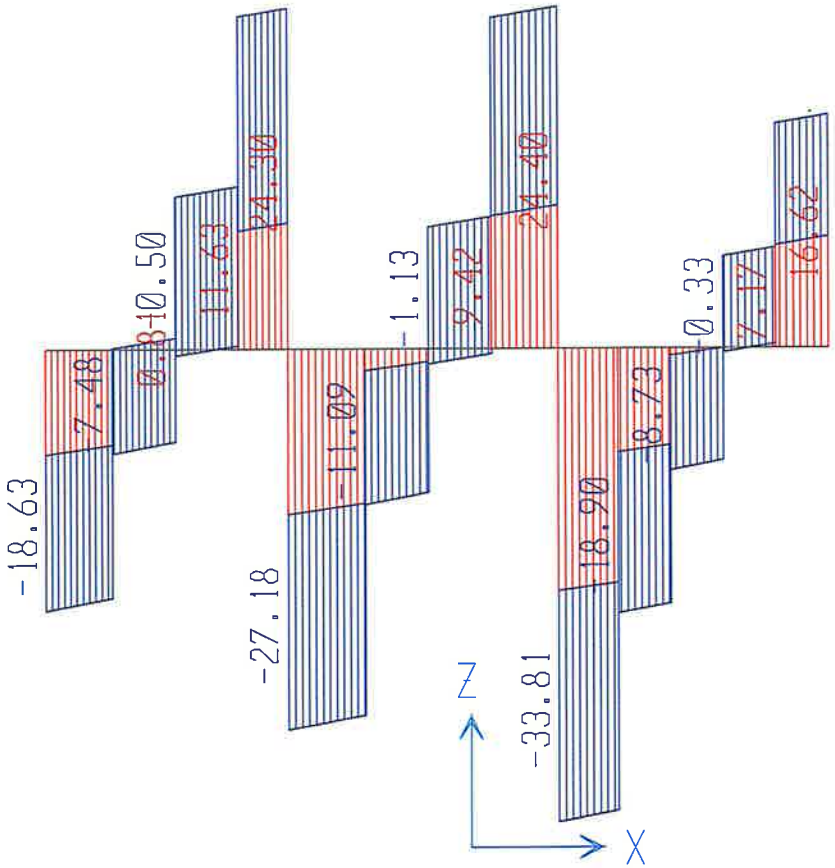
BEAM B6- BENDING MOMENT:

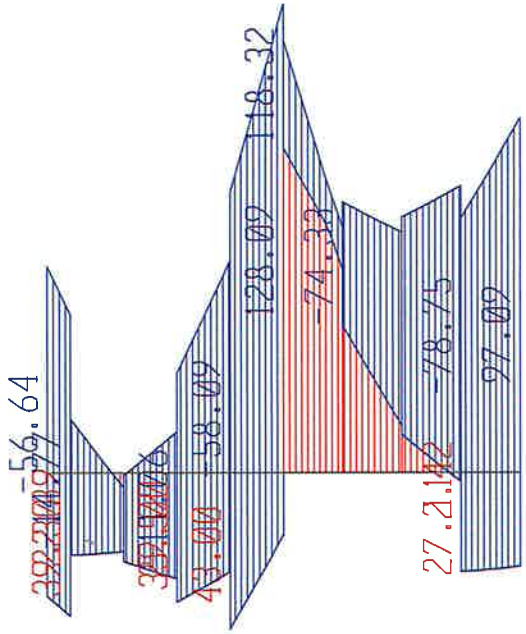


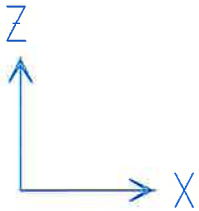
173

BEAM B6 - SHEAR!

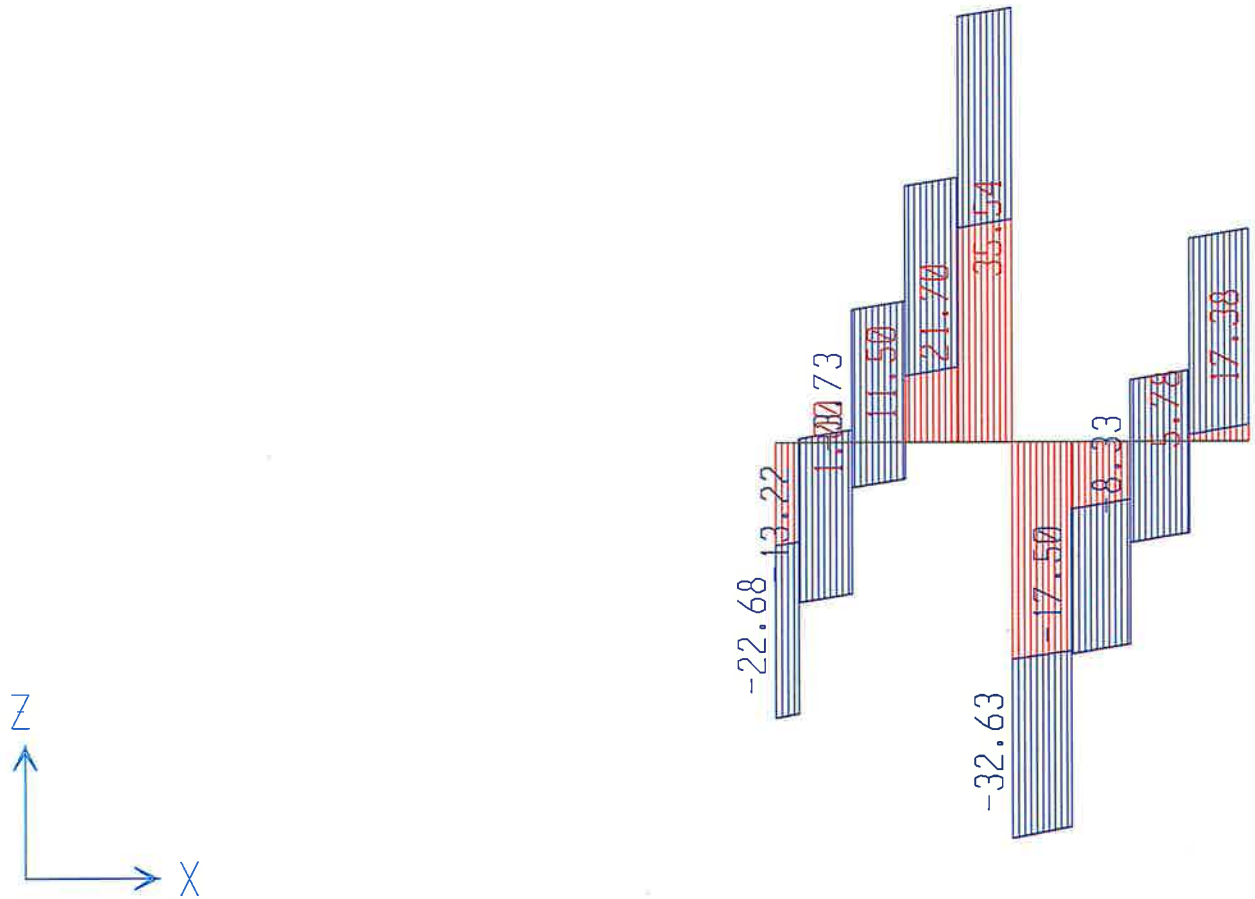


BEAM B7 - BENDING MOMENT:

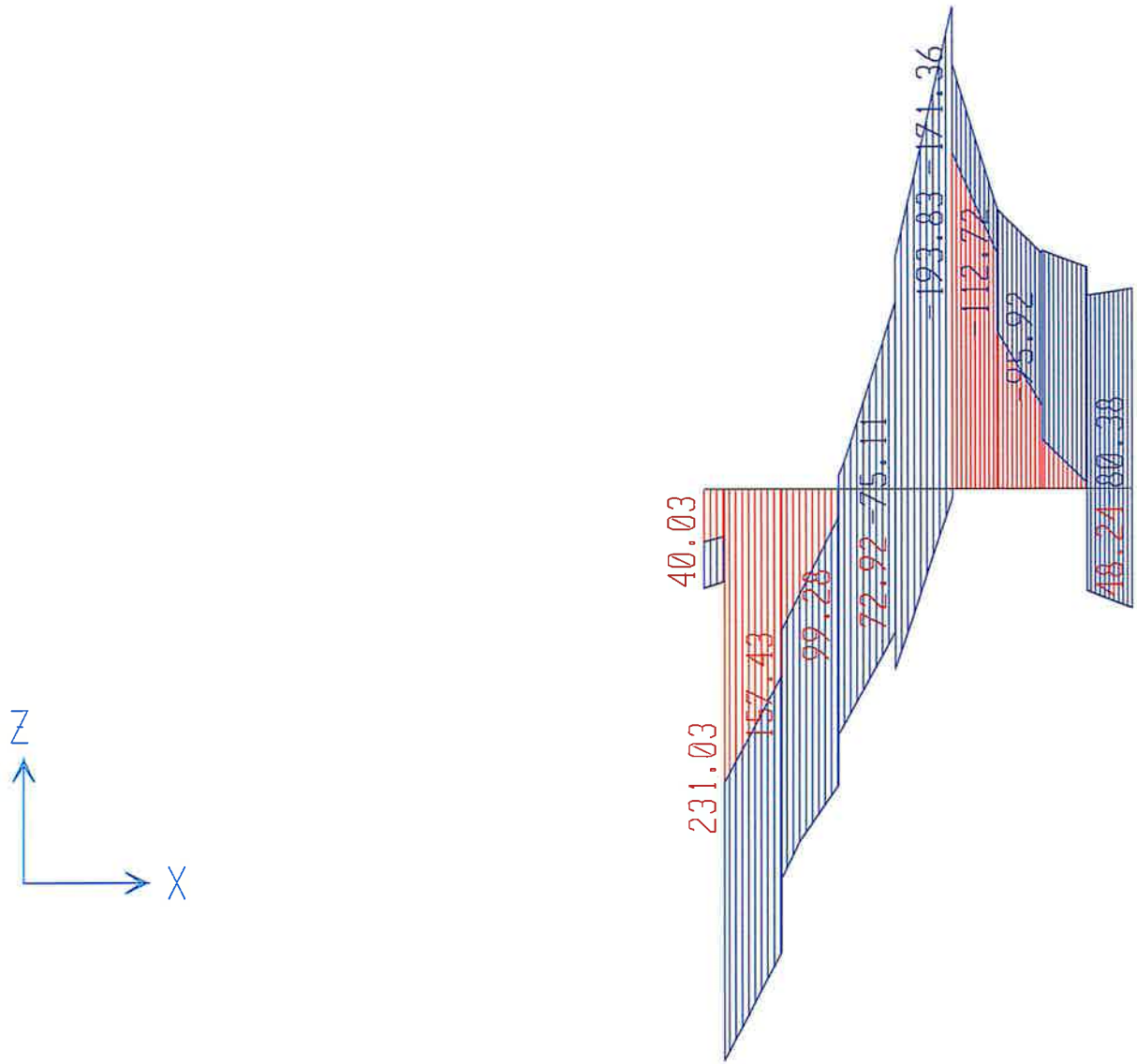




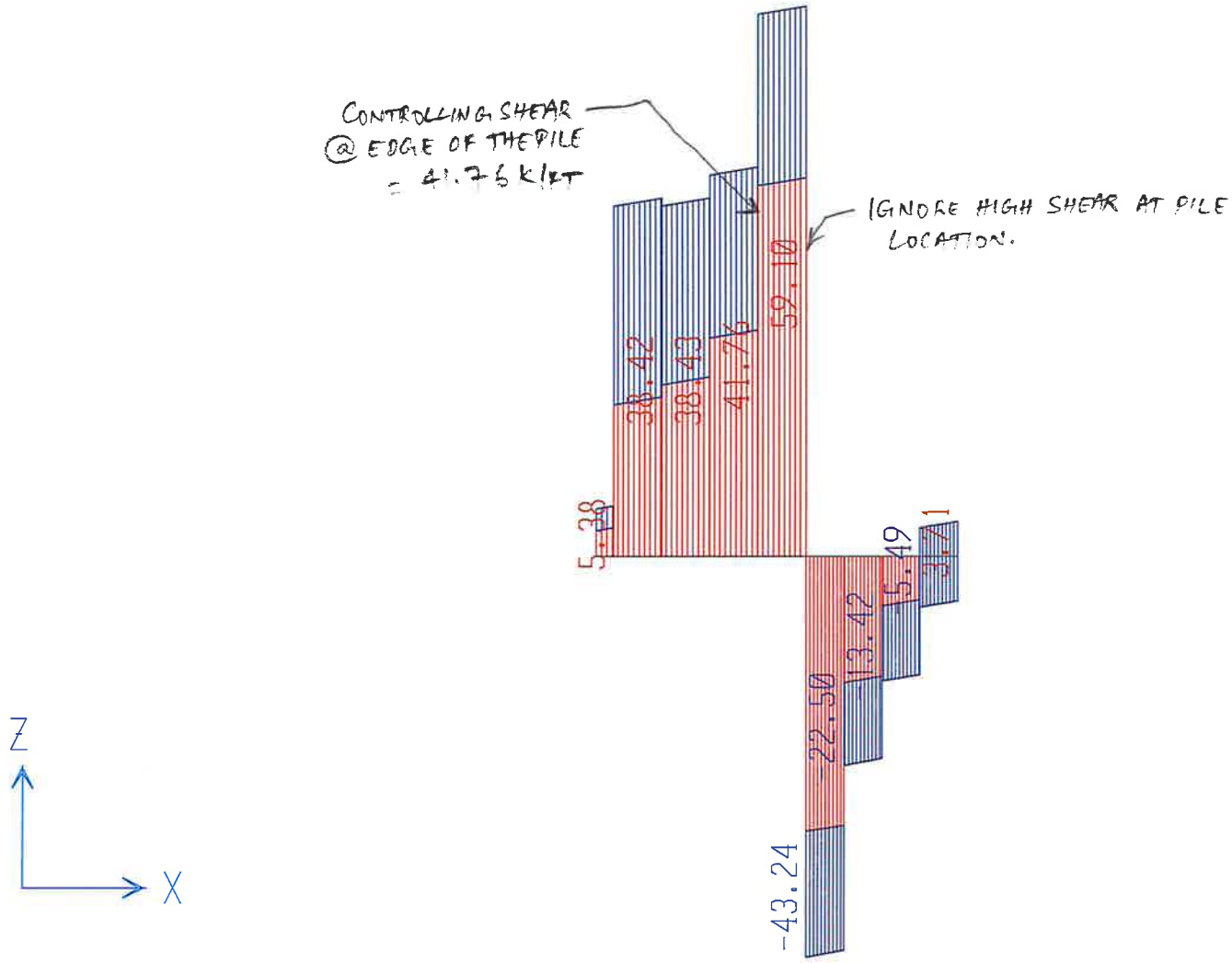
BEAM B7-SHEAR!



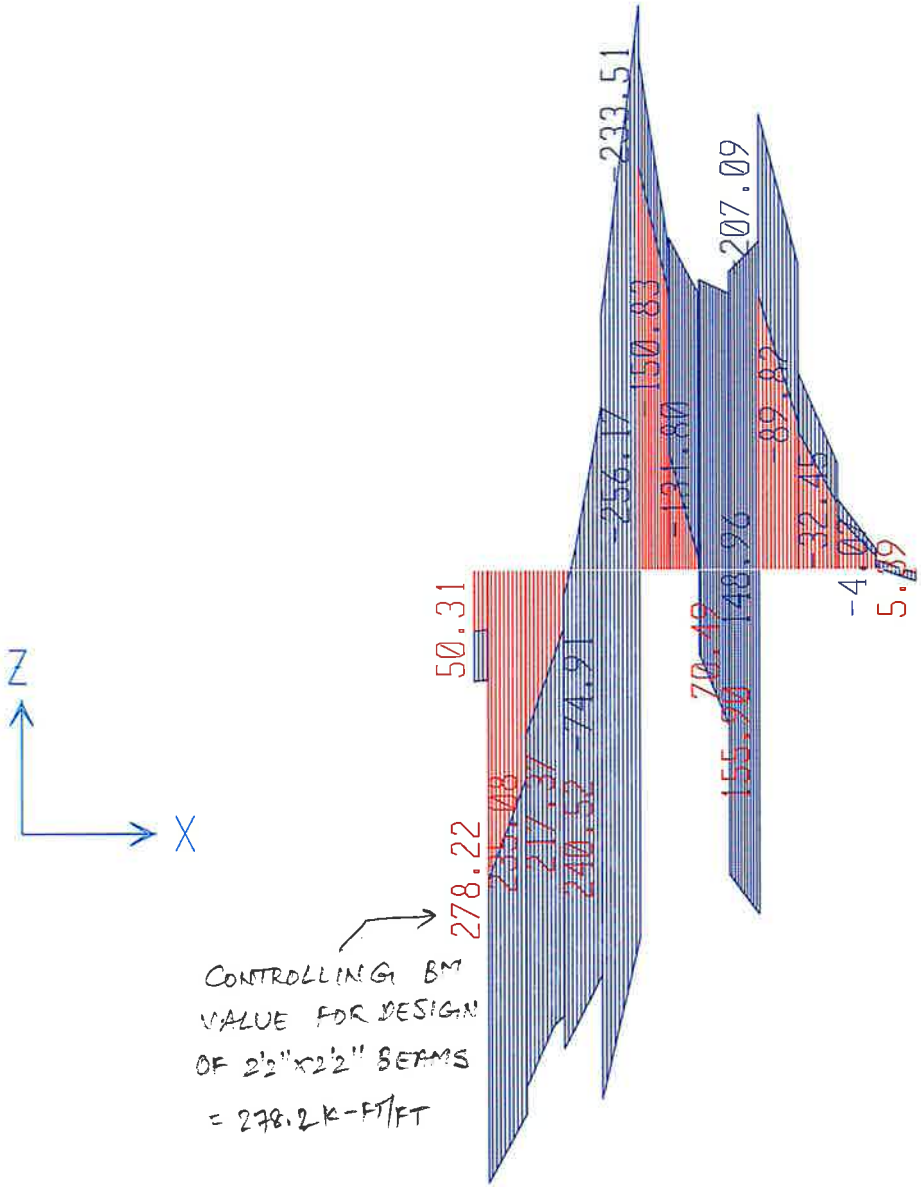
BEAM B8 - BENDING MOMENT:



BEAM B8 - SHEAR!

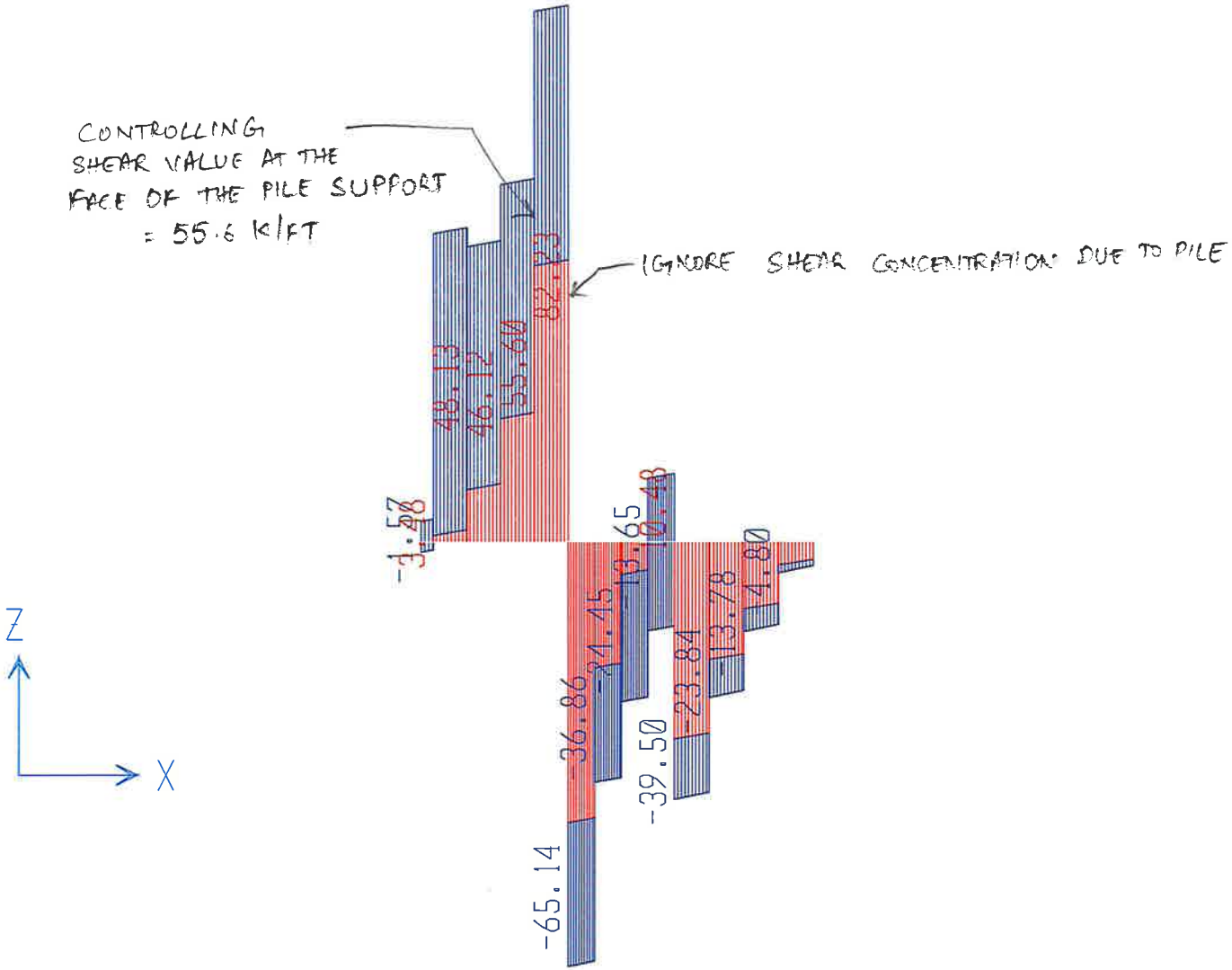


BEAM B9 - BENDING MOMENT:



CONTROLLING BM VALUE FOR DESIGN OF 2'2"x2'2" BEAMS = 278.2K-FT/FT

BEAM B9- SHEAR:



SECTION 10

Miscellaneous Detail Designs & Checks

(180b)

10.1

Bulkhead Design Check

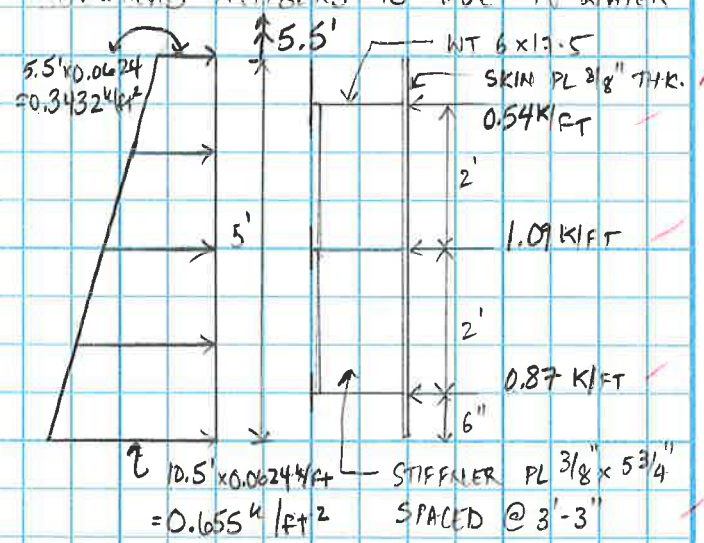
DESIGN OF BULK HEAD:

- THE BULKHEAD IS 5' TALL WITH THE BOTTOM ELEVATION AT -7.0

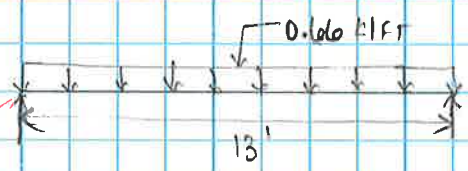
- LENGTH OF BULKHEAD BETWEEN THE SLOTS = 13'-0"

- THE MAXIMUM LOAD ON THE BULKHEAD MEMBERS IS DUE TO WATER FORCE ACTING TO THE TOP.

SHOWN IS THE BOTTOM BULKHEAD W/ WATER TO MAX. EL 3.5:



MAX. REACTION IS ON THE BOTTOM BEAM = 0.66 k/FT



MAX. BM = $0.66^{k/FT} \times \frac{13^2}{8} = 13.94$ K-FT

MAX. REACTION ON THE SLOTS = $\frac{0.66^{k/FT} \times 13^{FT}}{2} = 4.3$ K

STIFFNERS:

MAX. BM IN THE STIFFNER PLATES BETWEEN 'T' SECTIONS = 0.16 K-FT/FT

TOTAL BM = 3.25' x 0.16 K-FT/FT
= 0.52 K-FT

SKIN PLATE DESIGN:

INPUT

$b := 24$ in, length of stiffner plate between intercostals = 2'-0"

$F_y := 36$ ksi

$F_b := (0.75 \cdot F_y)$

- **Load:** Water to the top of Bulkhead

$p := 0.66$ ksf, the pressure at the bottom taken conservatively.

$p = 0.66$ ksf

$p := \frac{p}{144}$ $p = 0.00458$ ksi

$M := \frac{p \cdot b^2}{12}$

$M = 0.22$ k-in

$F_b := (0.75 \cdot F_y)$

$F_b = 27$ ksi

$E := 29000$ ksi

$t_{minstress} := \left(\frac{p \cdot b^2}{2 \cdot F_b}\right)^{\frac{1}{2}}$

$t_{minstress} = 0.221$ in

$t_{mindeflection} := \left(\frac{p \cdot b^4}{12.8 \cdot E}\right)^{\frac{1}{4}}$ (limiting the skin plate deflection to 1/4")

$t_{mindeflection} = 0.253$ in

Provided Skin Plate thickness = 3/8" = 0.375", OK.

INTERCOSTAL DESIGN

The intercostals are designed as simple beams spanning between girders.

Material: A36 Steel F_y = 36 ksi

Skin Plate and Stiffener treated as a T-section

	t_f =	0.375 in	Skin plate thickness
	b =2*(t*95/(Fy)^0.5) =	11.88 in	Effective width of skin plate, (EM 1110-2-2105 pg. B-4)
TRY	t_w =	0.375 in	
	$d_{max} = t_w (127/\sqrt{F_y})$ =	7.94 in	d/t = 127/(Fy)^0.5 (Non-Compact Section, AISC)
TRY	d' =	5.75 in	Depth of stem
	Check if Trial Section Is Noncompact	d/t= 16.33	**NONCOMPACT**

TRY SECTION

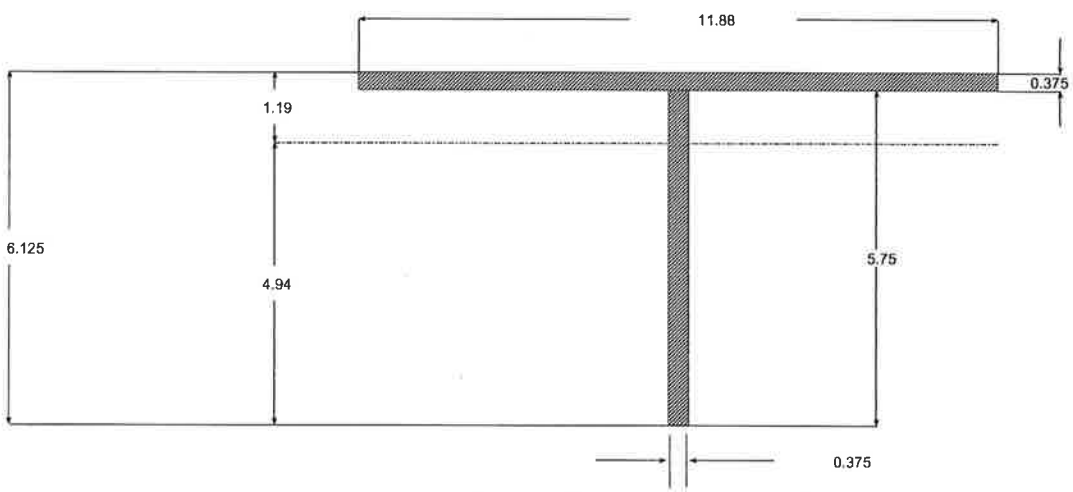


Plate	A	y	Ay	Ix	Ad^2	Iy	J (1/3*b*t^3)
Flange	4.45	0.19	0.83	0.05	4.45	52.33	0.21
Web	2.16	3.25	7.01	5.94	9.18	0.20	0.10

ΣA 6.61 ΣAy 7.8 in^2 ΣJ = 0.31 in^4

y = 1.19 in

Ix = 19.6 in^4 Iy = 52.5 in^4

S_T = (section modulus) 16.53 in^3

S_C = (Section Modulus) 3.97 in^3

From AISC, for non-compact shapes, Fb = 0.6 Fy

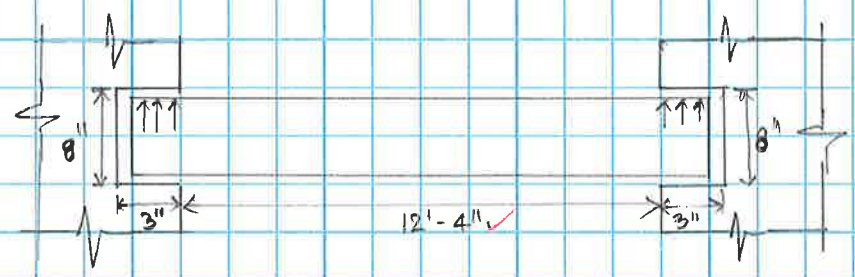
Fb = 0.60*Fy = 21.6 ksi

Mmax = 0.52 k-ft
fb= Mmax / Sc = 1.57 ksi < Fb, OK.

10.2

Trash Rack & Bulkhead Slot Design Checks

TRASH RACK SLOTS:



MAX. FORCE ON SLOT IS WHEN WATER IS AT HIGHEST EL. 3.5 & OCCURS @ EL -7.0

WATER PRESSURE = $(3.5-(-7.0)) \times 0.0624^{KCF} = 0.7$ K/FT/FT OVER 13'

$= 13' \times 0.7^{K/FT/FT} = 9.1$ K/FT

INCLUDING THE HYDRAULIC & LOAD FACTORS FOR CONCRETE DESIGN

MAX. FORCE $= 9.1^{K/FT} \times 1.3 \times 1.7 = 20.1^{K}$

FORCE @ EACH SLOT $= 20.1^{K}/2 = 10.05^{K}$

∴ BEARING PRESSURE ON 3" WIDE SLOT

$= \frac{10.05^{K}}{3''} = 3.35$ K/in

- SEE NEXT PAGE FOR DESIGN CHECK OF THE SLOT.

- THE SAME DESIGN APPLIES FOR THE BULKHEAD SLOTS.

* BULKHEAD SLOTS ARE SAME OVERALL WIDTH (13') BUT HAVE LARGER BRACKETS (4'12" BEARING WIDTH INSTEAD OF 3"). THEREFORE, IF TRASH RACK SLOTS ARE OK FOR WATER TO EL 3.5, THEN BULKHEAD SLOTS ALSO OK.

Job: Maurepas Pump Station
Project No.: 10001663
Description: Design of reinforcement for trash rack slots
+bulkhead slots

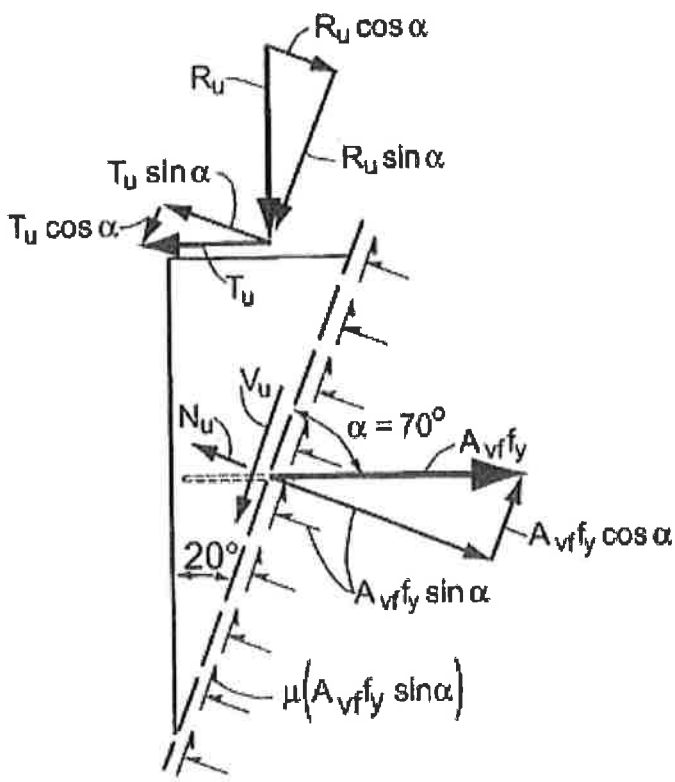
Calculated By: JY 02/11
Checked By: LBA 8/13

References:

* Notes on ACI 318-05 Building Code Requirements for Structural Concrete. Portland Cement Association. 2005
* ACI 318-02

Symbols:

R_u	Needle Beam Reaction
T_u	Shrinkage and Temperature Effects
V_u	Direct Shear Transfer Force
N_u	Net Tension Across Shear Plane
A_{vf}	Shear-friction Reinforcement for Direct Shear Transfer
A_n	Net Tension Reinforcement
A_s	Total Area of Reinforcement
V_{nmax}	Maximum Shear-Transfer Strength
A_c	Area of Concrete
α	Angle of Shear Plane, 20 degrees



Job: Maurepas Pump Station
Project No.: 10001663
Description: Design of reinforcement for trash rack slots + bulkhead slots

Calculated By: JY 02/11
Checked By: LBR 8/2013

Given Information:

$R_u := 10.1 \cdot kip$ (see hand calculations)

$T_u := 0 \cdot kip$

$\alpha := 20 \cdot °$

$\lambda := 1$

$\mu := 1.4 \cdot \lambda = 1.4$ MONOLITHIC CONCRETE

$\Phi := .85$

$f_y := 60 \cdot ksi$

Calculations:

$V_u := R_u \cdot \sin(\alpha) + T_u \cdot \cos(\alpha)$

$V_u = 3.454 \cdot kip$

$N_u := T_u \cdot \sin(\alpha) - R_u \cdot \cos(\alpha)$

$N_u = -9.491 \cdot kip$ * negative value means net compression

$$A_{vf} := \frac{V_u}{\Phi \cdot f_y \cdot (\mu \cdot \sin(\alpha) + \cos(\alpha))}$$

$A_{vf} = 0.048 \cdot in^2$

$$A_n := \frac{N_u}{\Phi \cdot f_y \cdot \sin(\alpha)}$$

$A_n = -0.544 \cdot in^2$ * Because area is in net compression, no tension reinforcement needed.

$A_s := A_{vf}$

$A_s = 0.048 \cdot in^2$

** 2"x 3/8" x 1'-3" anchor bars @ 12" adequate for both the needle beam and trash rack slots.

10.3

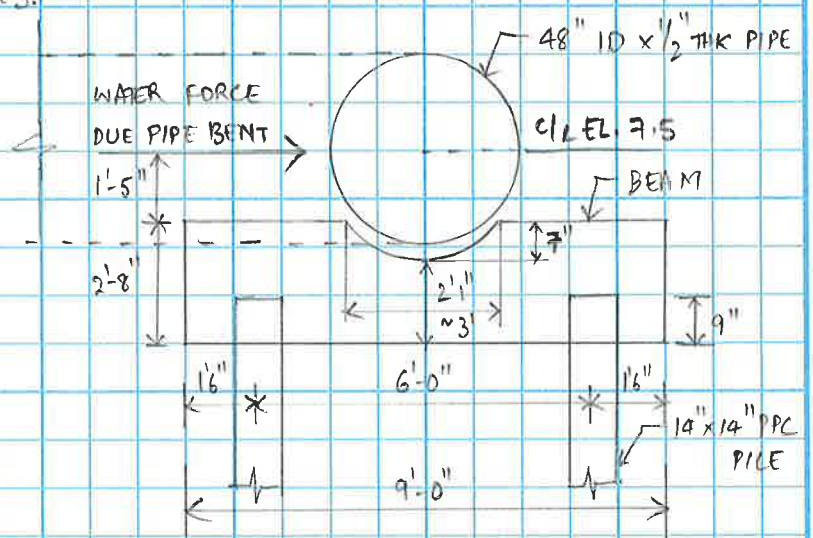
Discharge Pipe Support Designs

DESIGN OF PIPE SUPPORTS:

THERE ARE TWO TYPES OF PIPE SUPPORTS IN THIS PROJECT. THE FIRST TYPE CARRIES ONLY ONE PIPE WHILE SECOND TYPE CARRIES 3 PIPES. (SEE DRAWINGS FOR THE PIPE LAYOUT)

DESIGN OF ONE PIPE SUPPORT:

THIS PIPE SUPPORT IS MADE 2'-4" x 2'-8" x 9'-0" LONG CONCRETE BEAM PLACED ON 2 PSC PILES.



DESIGN OF THE PILE CAP BEAM:

PIPE & WATER LOADS: THE LENGTH OF THE PIPE SUPPORTED BY THIS STRUCTURE IS TAKEN 30' CONSERVATIVELY. ✓

48" IS O.D.

WEIGHT OF PIPE $= \frac{\pi}{4} \times (48.5^2 - 48^2) \times 30' \times 12 \times \frac{499\ LB/FT^3}{12 \times 12 \times 12} = 3940\ LB$ OK

WEIGHT DUE TO WATER WHEN COMPLETELY FILLED WITH WATER

$= \frac{\pi}{4} \times \frac{48^2}{144} \times 30 \times 62.4\ LB/FT^3 = 23524\ LB$ OK

TOTAL VERTICAL LOAD $= 23524 + 3940 = 27464\ LB$ ✓

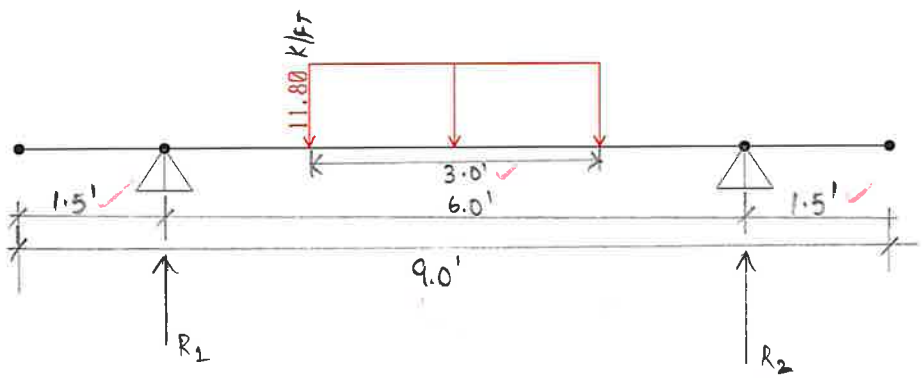
$\approx 27.5\ K$ ✓

THIS LOAD IS DISTRIBUTED OVER WIDTH OF 2'-4" ✓ AND ON A LENGTH OF 3' APPROX. ✓ ∴ DISTRIBUTED FORCE $= \frac{27.5^k}{2.33'} = 11.8\ K/FT$ ✓

– SEE FOLLOWING SHEETS FOR THE DESIGN

PIPE LOAD ON PIPE SUPPORT STRUCTURE:

- FOR DESIGN PURPOSES THE PILE SUPPORTS ARE CONSIDERED AS PINNED SUPPORTS.



- REACTIONS $R_1 = R_2 = \frac{11.8^{K/FT} \times 3'}{2} + \frac{9.0' \times 2.33' \times 2.67' \times 0.15^{K/F}}{2} = 21.9^{K}$

↑ D.L

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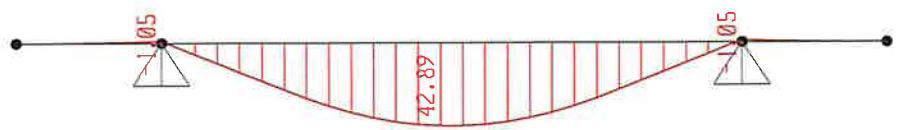
MAXIMUM BENDING MOMENT DESIGN:

- THE LOAD CASE ANALYZED INCLUDES DEAD WEIGHT OF THE BEAM ALSO.



- FROM THE CAD ANALYSIS $M_{MAX} = 42.9$ K-FT ; INCLUDING HYDRAULIC & LOAD FACTORS $M_{MAX} = 42.9^{k\text{-}ft} \times 1.3 \times 1.7 \simeq 95$ k-FT
- THE DESIGN IS DONE FOR THE MINIMUM SECTION AVAILABLE $= 2'4' \times 2'1'' = 28'' \times 25''$; $b = 28''$, $d = 25'' - 4''_{c.c} - 0.5''_{\text{ASSUME } 1''\phi \text{ BAR}} = 20.5''$
- $A_{S\,REQ'D} = \dfrac{0.85 f'_c bd}{f_y}\left[1 - \sqrt{1 - \dfrac{2M_u}{0.85 f'_c b d^2 \phi}}\right] = \dfrac{0.85 \times 4000 \times 28 \times 20.5}{60000}\left[1 - \sqrt{1 - \dfrac{2 \times 95 \times 12000}{(0.85 \times 4000 \times 28 \times 20.5^2 \times 0.9)}}\right] = 1.05 \text{ in}^2$
- MIN. R/F REQ'D IN FLEXURAL MEMBERS $= \dfrac{200}{f_y} bd = \dfrac{200}{60000} \times 28 \times 20.5 = 1.91 \text{ in}^2 > 1.05 \text{ in}^2$ ∴ PROVIDE MIN R/F.

HENCE, PROVIDE 5 - #6 BARS AT BOTH FACES CONSERVATIVELY.

SAP2000 v14.2.3 - File:Pipe Structure Beam - Moment 3-3 Diagram (PIPE LOAD) - Kip, ft, F Units

CAPACITY OF 4-#6 BARS: $a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{(1.76 \text{ in}^2)(60 \text{ KSI})}{(0.85)(4 \text{ KSI})(28'')} = 1.109''$

$\phi M_n = \phi A_s f_y (d - a/2)$
$= (0.9)(1.76 \text{ in}^2)(60 \text{ KSI})\left(20.5'' - \dfrac{1.109''}{2}\right) = \underline{157.97^{k\cdot ft}}$

$\phi M_n = 158^{k\cdot ft} > M_u \times 1.33 = 95^{k\cdot ft} \times 1.33 = 126.7^{k\cdot ft}$ - CAN USE LESS THAN FLEX. MIN. (ACI 318 SECT 10.5.3)

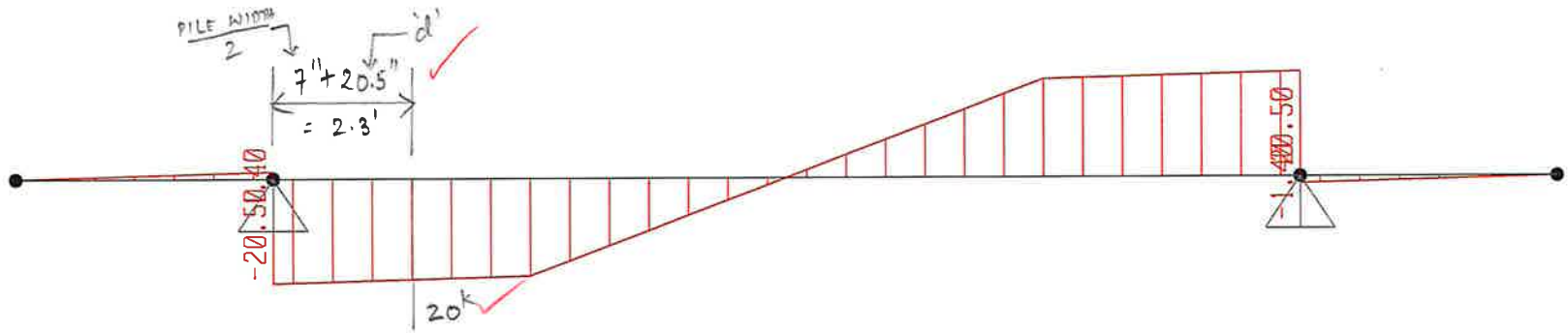
LBK 01/11

CHECK FOR SHEAR CAPACITY:

- SHEAR CAPACITY OF THE BEAM $\phi V_c = \phi \times 2 \times \sqrt{f'_c} \times b \times d$

$= 0.85 \times 2 \times \sqrt{4000} \times 28'' \times 20.5''$

$= 61.7^k$



- SHEAR IN THE BEAM AT A DISTANCE 'd' FROM THE EDGE OF THE PILE SUPPORT = 20^k

SHEAR INCLUDING FACTORS 1.3 & 1.7 = $20^k \times 1.3 \times 1.7$

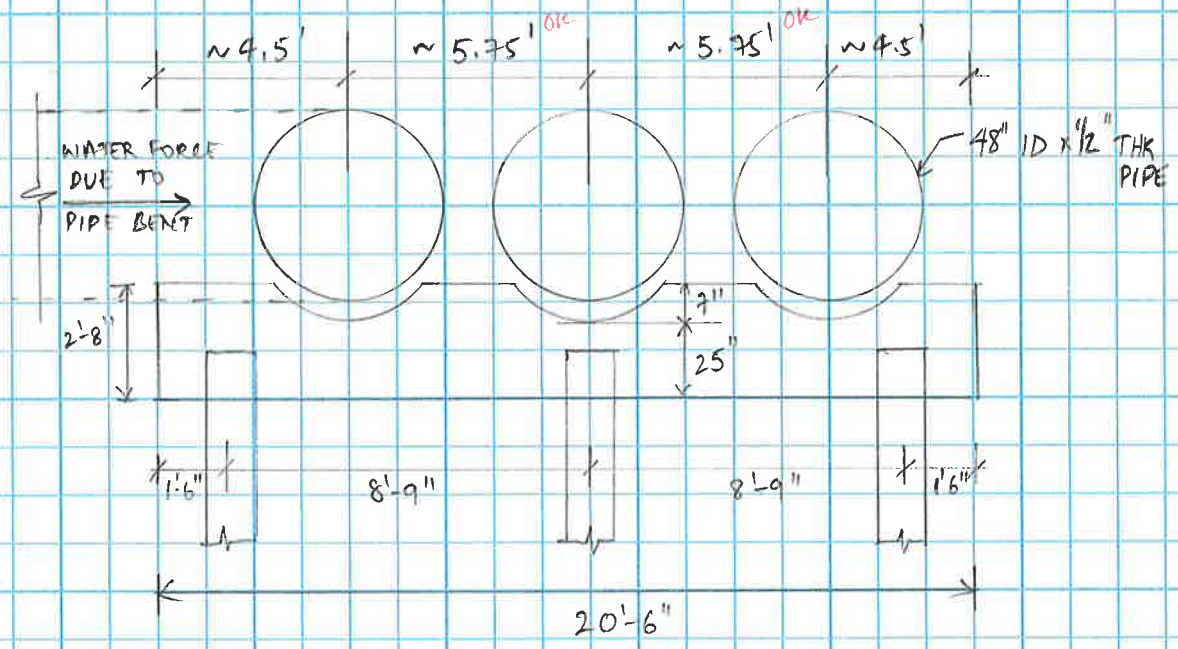
$= 44.2^k$

$< 61.7^k$ OK

LBP 5/11

DESIGN OF THREE PIPE SUPPORT STRUCTURE:

THIS PIPE SUPPORT CONSISTS OF 2'-4" x 2'-8" x 20'-6" LONG CONCRETE BEAM ON 3 PPC PILES SPACED AS SHOWN BELOW



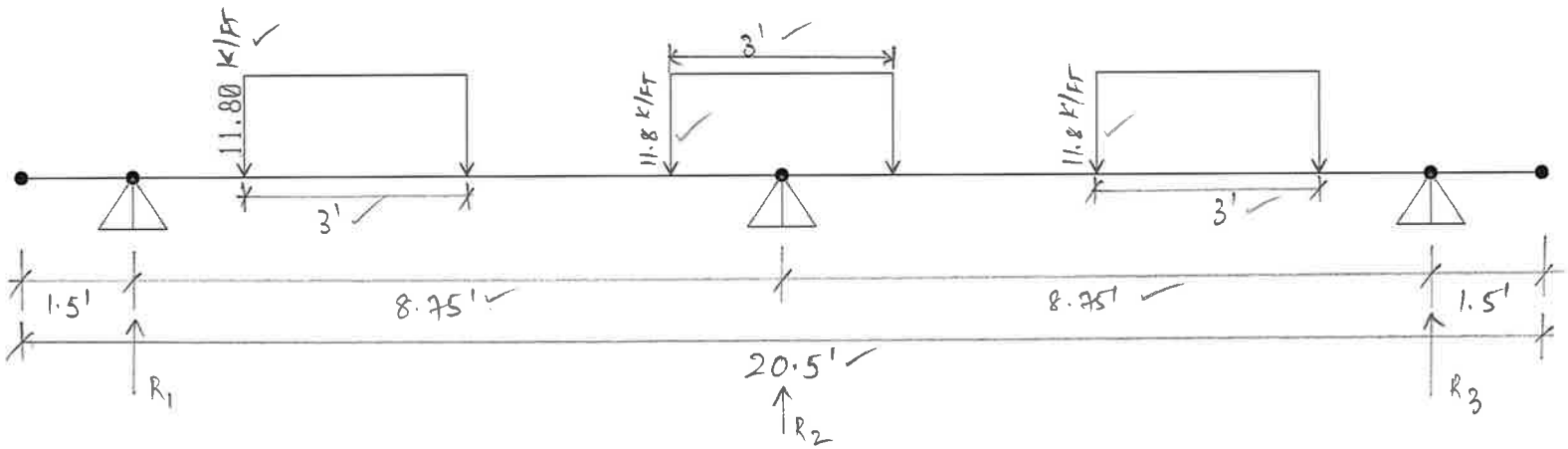
- THE VERTICAL FORCE DUE TO PIPE & WATER ARE SAME AS CALCULATED EARLIER FOR ONE PIPE SUPPORT.

- THE HORIZONTAL WATER FORCE EFFECT DUE TO 90° PIPE BENT IS CONSIDERED ONLY AT ONE PIPE, SINCE THE OTHER TWO PIPES HAVE INDIVIDUAL SINGLE PIPE SUPPORTS. THE VERTICAL COMPRESSION FORCE DUE TO THIS MOMENT IS ADDED TO THE MIDDLE PILE.

- SEE ATTACHED SHEETS FOR DESIGN.

PIPE LOADS ON 3 PIPE STRUCTURE:

- PIPES TAKEN AS PINNED SUPPORTS.



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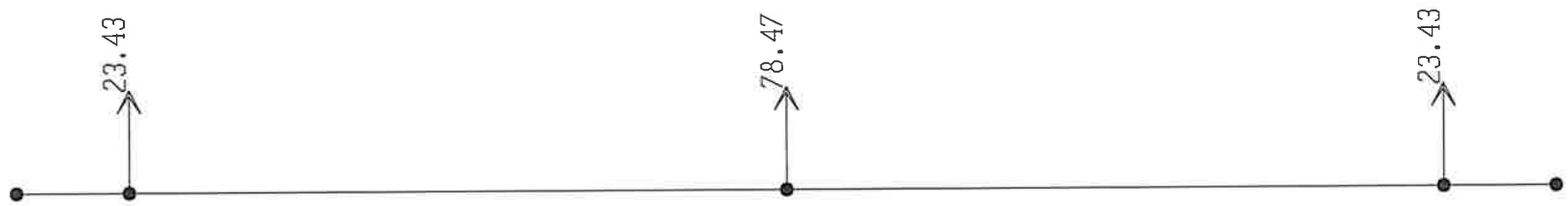
PILE REACTIONS IN 3 PIPE STRUCTURE:



JY 02/11
LBR 5/2011

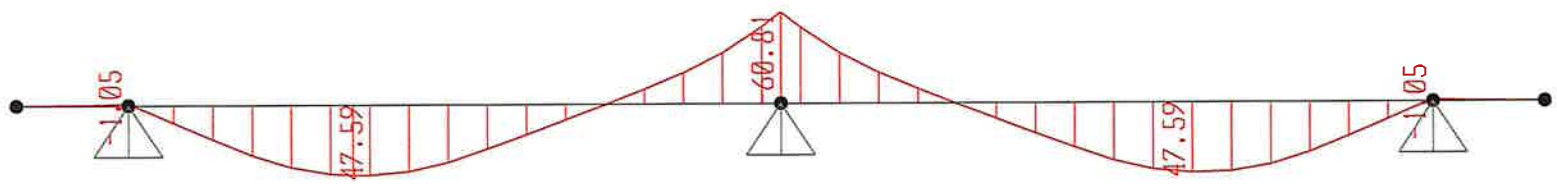
MAXIMUM BENDING MOMENT DESIGN:

- THE LOAD CASE ANALYZED INCLUDES DEAD LOAD DUE TO BEAM WEIGHT.

- FROM THE SAP ANALYSIS, M_{MAX} (TOP) = 60.81 k-ft (↑ CONTROLS) & M_{MAX} (BOT.) = 47.59 k-ft [SEE DWG. BELOW]

- MIN. SECTION AVAILABLE = 28" (↑ b) × 20.5" (↑ d)

- DESIGN MOMENT = 60.81 k-ft × 1.3 × 1.7 = 134.4 k-ft



$$\therefore A_{s\,REQ'D} = \frac{0.85 f_c' b d}{f_y}\left[1-\sqrt{1-\frac{2M_u}{0.85 f_c' b d^2 \phi}}\right]$$

$$= \frac{0.85\times4000\times28\times20.5}{60000}\left[1-\sqrt{1-\frac{2\times134.4\times12000}{(0.85\times4000\times28\times20.5^2\times0.9)}}\right] = 1.5\ in^2$$

- $A_{s,MIN}$ = 1.91 in² > 1.5 in², PROVIDE MIN. A_s i.e., 5-#6 BARS.

- PROVIDE 5-#6 BARS AT BOTH FACES CONSERVATIVELY.

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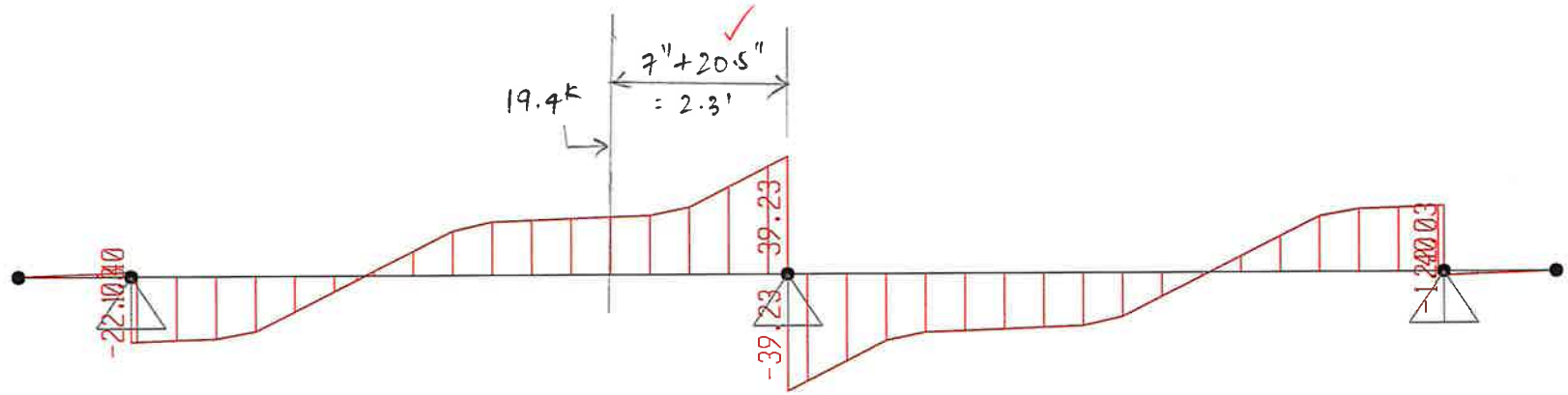
CHECK FOR SHEAR CAPACITY:

- SHEAR CAPACITY = 61.7^k



- SHEAR AT DISTANCE 'd' FROM PILE SUPPORT EDGE = 19.4^k

WITH 'H_f' & LOAD FACTOR S.S = 19.4^k x 1.3x1.2 = 42.9^k

< 61.7^k OK

- THE PIPE SUPPORT IS ALSO DESIGNED FOR HORIZONTAL FORCE DUE TO 90° PIPE BENT
- HORIZONTAL FORCE ON THE PILE CAP BEAM DURING THE OPERATION OF PUMP AT FULL CAPACITY :

MAXIMUM CAPACITY OF THE PUMP 'Q' = 125 CUSECS

∴ VELOCITY OF WATER 'V' = Q/A

$= (125\ FT^3/S) / (\frac{\pi}{4} \times 4^2) FT^2$

$= \cancel{98.2\ FT/S}$ 9.95 FT/S

MAX. HORIZONTAL FORCE 'F' $= \frac{V \times Q \times W}{g}$ WHERE W = UNIT WT OF WATER

$= 62.4\ LB/FT^3$

g = ACCELERATION DUE TO GRAVITY

$= 32.2\ FT/S^2$

$= \frac{98.2\ FT/S \times 125\ FT^3/S \times 62.4\ LB/FT^3}{32.2\ FT/S^2}$

$= 23787.6\ LB \simeq 24^K$ CONSERVATIVE - OK

- THE FORCE 'F' IS APPLIED AT THE CENTER LINE OF THE PIPE.

∴ THE MOMENT AT THE TOP OF THE PILES $= 24^K \times 4.083'$

THIS MOMENT IS TRANSFERRED IN TO PILES AS AXIAL FORCES

$= \frac{24^K \times 4.083'}{9'} \simeq 11^K$

THE OUTER PILE IS SUBJECTED TO A COMPRESSION OF '11^K' & INNER PILE TO A TENSION OF '11^K'

NOTE: THOUGH THE WATER FORCE IN THE PIPE GETS DIVIDED IN TO FORCE COMPONENTS IN DIFFERENT DIRECTIONS AT THE PIPE BENT, THE PIPE SUPPORT IS DESIGNED TO TAKE THE COMPLETE WATER FORCE IN HORIZONTAL DIRECTION.

CHECK 14" x 14" PPC PILES :

- MAXIMUM PILE REACTION = 78.47^k (CENTER PILE OF 3-PIPE SUPPORT)

- PILE AXIAL FORCE DUE HORIZONTAL FORCE EFFECT OF WATER FLOW = 11^k

∴ TOTAL AXIAL FORCE IN PILE = 78.47^k + 11^k ≃ 90^k

- THE GEOTECHNICAL CAPACITY REQ'D WITH F.O.S OF 2 = 2×90^k = 180^k

∴ PROVIDE A PILE TIP EL. OF -71.0. (SEE ATTACHED PILE CURVES)

TIP EL. REQ'D FOR PIPE STRUCTURES = -71.0

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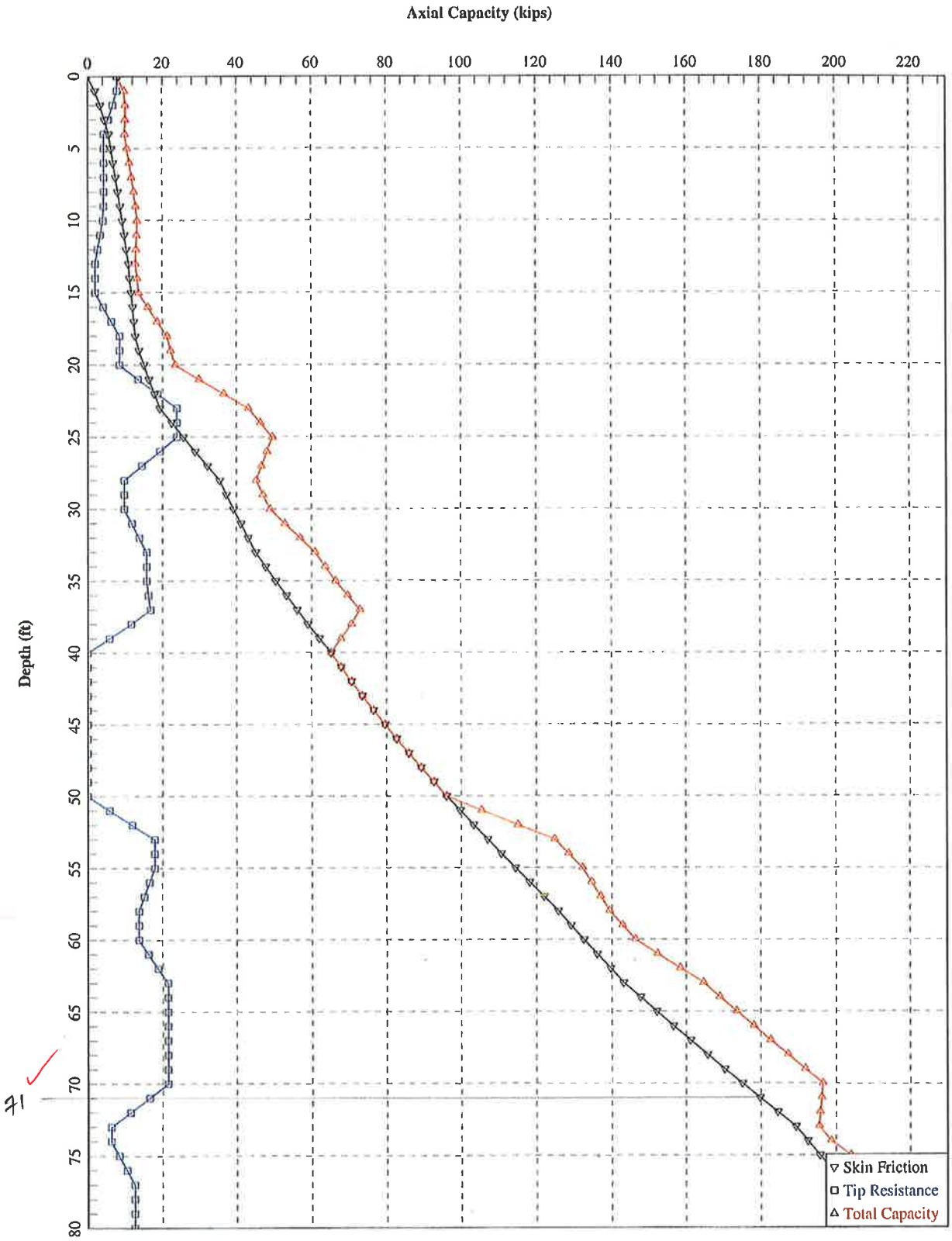
CBR 5/11

200



B-7 14 INCH CONCRETE

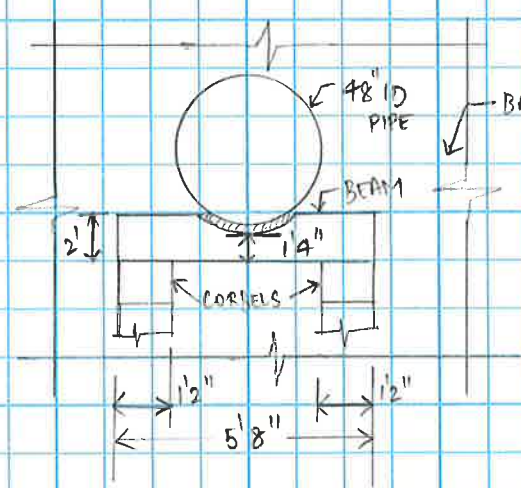
10.4

Pipe Support Corbel Design

DESIGN OF PIPE SUPPORT:

- PIPE SUPPORT CAN BE DESCRIBED AS A BEAM SUPPORTED BY TWO CORBELS. THE PIPE COMING OUT OF THE PUMP STATION SUBSTRUCTURE SITS ON THE BEAM.

- FOR DESIGN PURPOSE THE BEAM ON THE CORBELS IS CONSIDERED TO BE SIMPLY SUPPORTED.



PIPE SUPPORT ELEVATION

LOADS ON THE SUPPORT:

- PIPE LOAD DUE TO THE MAX. UNSUPPORTED LENGTH:

THE MAXIMUM UNSUPPORTED LENGTH IS FOR THE FIRST PIPE = 15' APPROX. (ASSUMED CONSERVATIVELY) (SEE DWGS.)

PIPE OD = 48"

PIPE WALL THICKNESS = 1/2"

$$\therefore \text{PIPE WEIGHT} = \frac{\pi}{4}(48^2 - 47^2) \times 15' \times 12 \times \frac{499 \text{ LB/FT}^3}{(12 \times 12 \times 12)}$$

$$= 3878 \text{ LB}$$

$$\text{WATER WEIGHT} = \frac{\pi}{4} \times \left(\frac{47}{12}\right)^2 \times 15' \times 62.4 \text{ LB/FT}^3 = 11277 \text{ LB/FT}$$

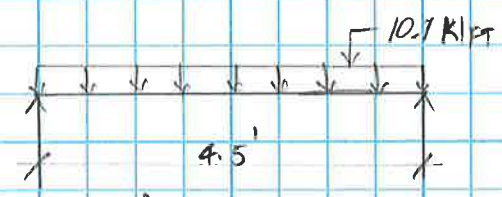
$$\text{TOTAL WEIGHT} = 3878 + 11277 = 15155 \text{ LB}$$

DESIGN OF BEAM: WIDTH OF PIPE CRADLE AREA = 1.5'

∴ UNIFORMLY DISTRIBUTED LOAD ON LENGTH OF THE BEAM $= \frac{15155 \text{ LB}}{1.5 \text{ FT}} = 10{,}109$ LB/FT

$= 10.1$ K/FT

C/C DISTANCE BETWEEN THE CORBELS = 5'8" − 1'2" = 4'6" = 4.5'



MAX. BM IN BEAM $= 10.1^{K/FT} \times \frac{(4.5')^2}{8} = 25.6$ K-FT

MAX. SHEAR IN BEAM $= 10.1^{K/FT} \times \frac{4.5'}{2} = 22.7$ K

MAX. BM DUE TO DEAD LOAD OF THE BEAM $= 0.15^{KCF} \times 1.5' \times 2' \times \frac{4.5^2}{8} = 1.14$ K-FT

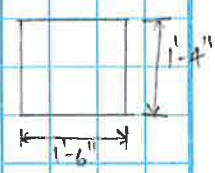
MAX. SHEAR DUE TO DEAD LOAD OF THE BEAM $= 0.15^{KCF} \times 1.5' \times 2' \times \frac{4.5}{2} = 1.01$ K

TOTAL DESIGN B.M $= 25.6^{K\text{-}FT} + 1.14^{K\text{-}FT} = 26.74$ K-FT

SHEAR $= 22.7^{K} + 1.01^{K} = 23.71$ K

DESIGN BEAM FOR THE SMALLEST SECTION:

THE SMALLEST SECTION IS AT THE CENTER = 1'-4"



EFFECTIVE DEPTH $= 1'4'' - \overset{C.C}{3''} - 0.5 \times \overset{\#4\ BAR}{0.5''} = 12.75''$

SHEAR STRENGTH $\phi V_c = \phi 2\sqrt{f'_c}\, bd = 0.85 \times 2\sqrt{4000} \times 18'' \times 12.75''$

$= 24.7\text{ K} > 23.71\text{ K}$ OK

'A_s' REQ'D FOR 'M_u' = 26.74 K-FT = 0.4 in²/'b' DIMENSION.

∴ PROVIDE 2 - #4 BARS

Job: Maurepas Pump Station
Project No.: 10001663
Description: Corbel Pipe Support Design

Calculated By: JY 02/11
Checked By: RBJ 9/13

References:

* Notes on ACI 318-05 Building Code Requirements for Structural Concrete. Portland Cement Association. 2005
* ACI 318-02

Symbols:

f_y	Yeild strength of reinforcing steel
f_c	Compressive strength of concrete
V_u	Shear Force
V_n	Shear-transfer strength
a_v	Shear span
d	Depth of corbel
h	Height of corbel
b	Width of bearing area
N_{uc}	Horizontal tensile force (estimated as 20% of shear force)
A_{vf}	Shear-friction reinforcement
A_n	Direct tension reinforcement
M_u	Moment
A_f	Flexural reinforcement
A_s	Primary tension reinforcement
A_h	Shear reinforcement (closed stirrups or ties)

Job: Maurepas Pump Station
Project No.: 10001663
Description: Corbel Pipe Support Design

Calculated By: JY 02/11
Checked By: RBJ 9/13

Given Information

$f_y := 60000$ psi

$f_c := 4000$ psi

Weight of the Pipe = 15155 lb

Weight of the Beam $Wb := 150 \cdot \frac{18}{12} \cdot \frac{24}{12} \cdot 5.67 = 2.551 \times 10^3$ lb

Weight of each corbel - length = 5'8", width = 1'2", depth(avg.) = 4' approx.

$Wc := 150 \cdot 4 \cdot 1.17 \cdot 5.67 = 3.98 \times 10^3$ lb,approx.(see dwg. for corbel dims.)

Design shear in Corbel :

$$V_u := \frac{15155 + 2551}{2} + 3980 = 1.283 \times 10^4 \text{ lbs}$$

$\lambda := 1$

$\mu := 1.4 \cdot \lambda = 1.4$

$b := 14$ in

$h := 4.583 \cdot 12 = 54.996$ in

$\Phi := 0.85$

$j_u := 0.9$

Calculations:

* Determine shear span: $a_v := \left[\left(\frac{2}{3}\right) \cdot 1.5 + 3.5\right] \cdot 12$

$a_v = 54$ in

* Determine required depth of corbel:

*Vn is the lesser of:

$V_{n1} := 800 \cdot b \cdot d$

$V_{n2} := 0.2 \cdot f_c \cdot b \cdot d$ where $0.2 \cdot f_c = 800$ psi

* Therefore, both Vn are equal and d equals:

$$d := \frac{V_u}{(\Phi \cdot 0.2 \cdot f_c \cdot b)} = 1.348 \text{ in}$$

* required d is much smaller than actual d → current dimensions OK

* Actual height is 4'-7"

$d := 4.5833 - .25 = 4.333$ ft, with 3" clearance

$d := d \cdot 12 = 52$ in

* Determine shear-friction reinforcement: $A_{vf} := \frac{V_u}{\Phi \cdot f_y \cdot \mu} = 0.18$ in^2

Job: Maurepas Pump Station
Project No.: 10001663
Description: Corbel Pipe Support Design

Calculated By: JY 02/11
Checked By: RBJ 9/13

* Determine direct tension reinforcement:

$$N_{uc} := V_u \cdot 0.2 = 2.567 \times 10^3 \text{ lb}$$

$$A_n := \frac{N_{uc}}{\Phi \cdot f_y} = 0.05 \quad \text{in}^2$$

* Determine flexural reinforcement:

$$M_u := \left[V_u \cdot a_v + N_{uc} \cdot (h - d)\right]$$

$$M_u = 7.007 \times 10^5 \text{ lb-in}$$

$$A_f := \frac{M_u}{\Phi \cdot f_y \cdot j_u \cdot d} = 0.294 \quad \text{in}^2$$

* Determine primary tension reinforcement:

$$A_{vf} = 0.18 \quad \text{in}^2 \qquad \left(\frac{3}{2}\right) \cdot A_f = 0.44 \quad \text{in}^2$$

$$A_s := \text{if}\left[A_{vf} > 3 \cdot \frac{A_f}{2}, \left(2 \cdot \frac{A_{vf}}{3}\right) + A_n, A_f + A_n\right]$$

$A_s = 0.344$ in^2, provide 2-# 5 bars = 2x0.31 = 0.62 in^2, OK

* Determine shear reinforcement:

$$A_h := \text{if}\left(A_{vf} > 3 \cdot \frac{A_f}{2}, \frac{A_{vf}}{3}, \frac{A_f}{2}\right)$$

$A_h = 0.147$ in^2/ft, provide at least 3-# 4 bars in top 2/3rd of d = 2'-8" depth of corbel at both faces = 2x3x0.2 = 1.2 in^2, OK

10.5

Temporary Resisting System (TRS) Design

URS

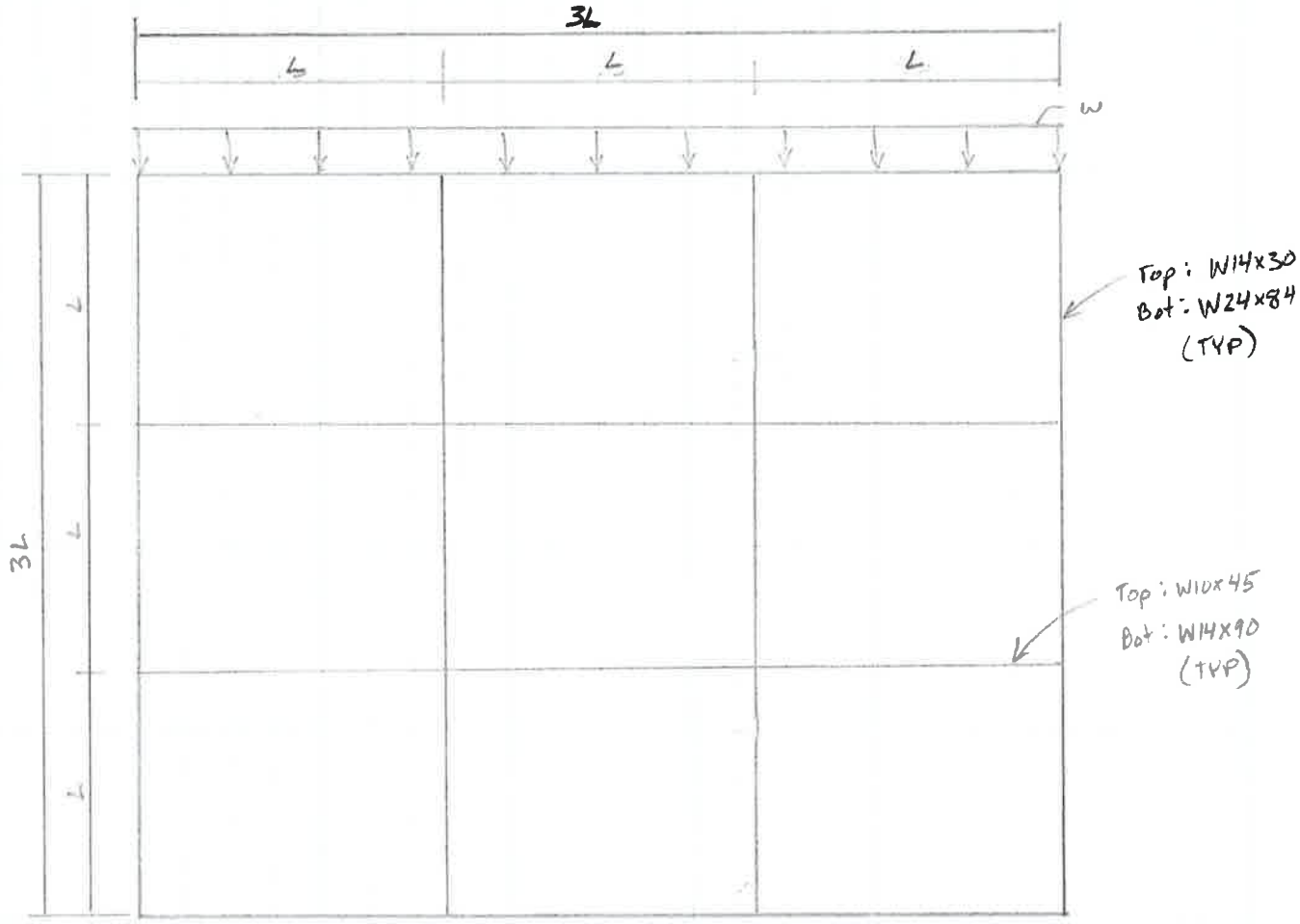
Page 208 of ____

Job Maurepas | Project No. 10001801 | Sheet ____ of ____

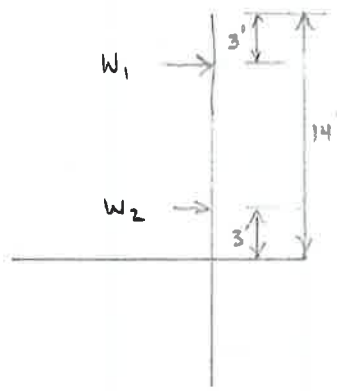
Description Pump Station TRS | Computed by BCB | Date 9/12/13

Walker & Strut Design | Checked by AB | Date 9/12/13

Reference



$3L_{max} = 62'$ so $L = 20.667'$



$w_1 = 2.64$ k/ft $\quad w_2 = 12.26$ k/ft

(SEE ATTACH SPW911 CALCULATION)

Using AISC Beam Diagrams (Table 3-23 #39):

$M_{max} = -0.1\,wL^2$

$\therefore M_{max_1} = -0.1(2.64)(20.67)^2 = -112.79$ kft

$M_{max_2} = 0.1(12.26)(20.67)^2 = -523.81$ kft

$S_{x_1} = \dfrac{M_{max_1}}{0.66F_y} = \dfrac{112.79(12)}{0.66(50)} = 41.01\ in^3 \quad \therefore$ W14x30

$S_{x_2} = \dfrac{523.81(12)}{0.66(50)} = 190.48\ in^3$ say W24x84

Using AISC Beam Diagrams (Table 3-23 #39):

$$R_B = R_C = 1.10\,wL$$

$$\therefore R_1 = 1.10(2.64)(20.67) = 60.03\ k$$
$$R_2 = 1.10(12.26)(20.67) = 278.76\ k$$

Minor Axis Length = 20.67'
Major Axis Length = 62'

Beam$_1$ = W10x33 => $r_x/r_y = 2.16$
Beam$_2$ = W14x74 => $r_x/r_y = 2.44$

Equivalent Effective Length For X-X axis:

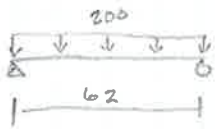
$$\frac{62}{2.16} = 28.7 = KL_1 \Rightarrow \text{use W10x45} \checkmark$$

$$\frac{62}{2.44} = 25.41 = KL_2 \Rightarrow \text{use W14x90} \checkmark$$

Combined Axial & Bending:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$$

Beam 1 => $f_a = \frac{P}{A} = \frac{60.03}{13.3} = 4.51$ ksi



$$f_b = \frac{M}{S_x} = \frac{\frac{wL^2}{8}(12)}{49.1} = 23.49 \text{ ksi}$$

$$\frac{4.51}{0.6(50)} + \frac{23.49}{0.66(50)} = 0.862 \le 1.0 \checkmark$$

Beam 2 => $f_a = \frac{278.76}{26.5} = 10.52$ ksi

$$f_b = \frac{1153.2}{143} = 8.06 \text{ ksi}$$

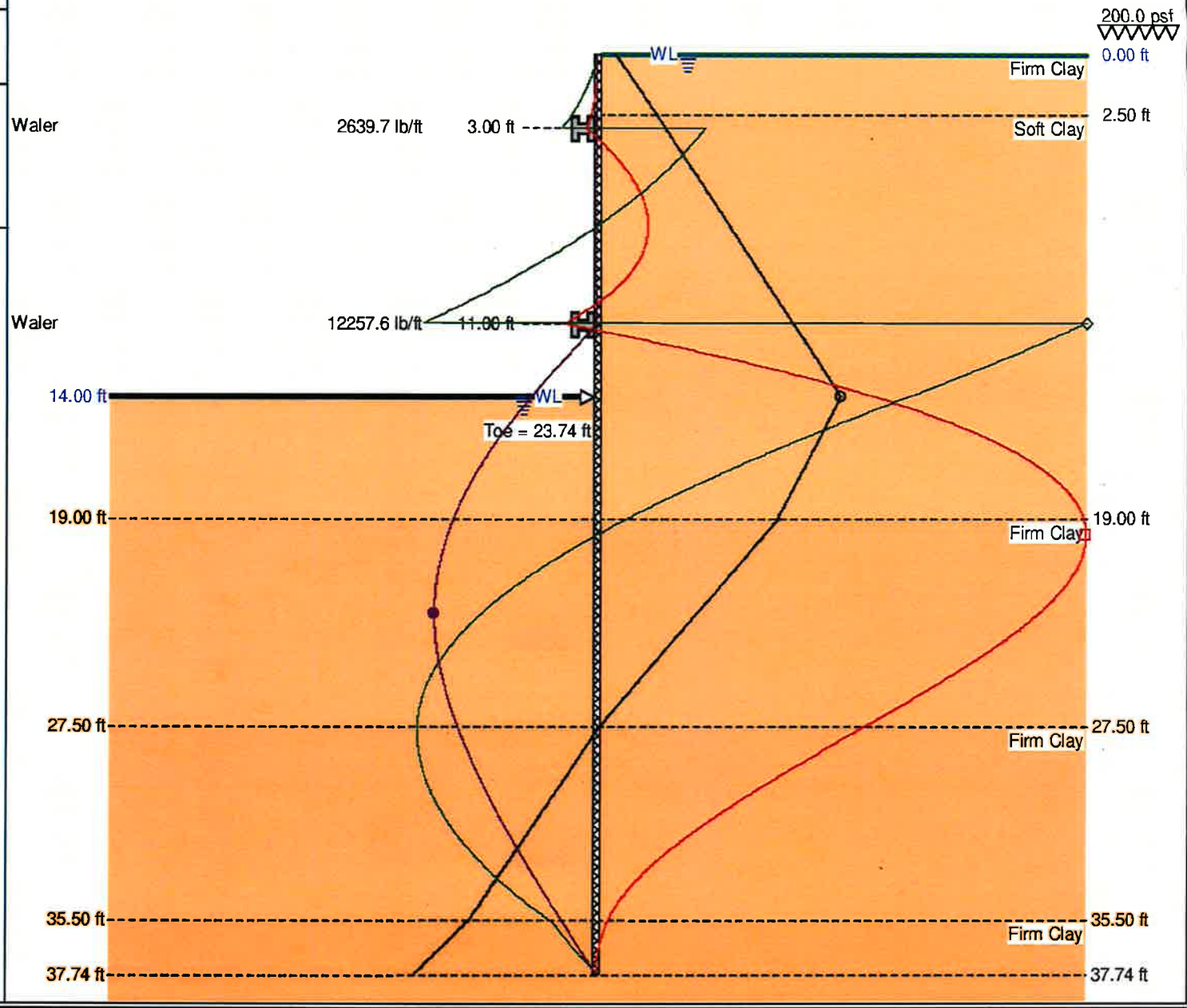
$$\frac{10.52}{0.6(50)} + \frac{8.06}{0.66(50)} = 0.59 \le 1.0 \checkmark$$

Client: Lake Maurepas Pump Station
Page: 1
Date: 9.9.13

Sheet: PZ27 50Ksi
Pressure: Coulomb
FOS: 1.3

Maximum	d (ft)
1204.0 psf	14.00
35123.6 ftlb/ft	19.62
9022.2 lb/ft	11.00
0.7 in	22.83

TRS Structure
Section A - S-Case



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SPW911, v2.39

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SECTION 11

Pump Station Geotechnical Information

URS

September 16, 2008

URS Corporation
3500 N. Causeway Blvd.
Suite 900
Metairie, LA 70002

Attn: Mr. Richard Murley, PE

Re: Geotechnical Investigation and Report (Final)
LDNR Pump Station
Gramercy, Louisiana
URS File No. 10001572

Dear Mr. Murley:

We have completed our final geotechnical investigation and report for the referenced project. This report supersedes our preliminary geotechnical report dated December 11, 2007, which was prepared for general project design and budgeting purposes for the pump station. This final report and recommendations presented are based upon the updated project design information provided by your office on August 21, 2008.

This final report addresses the following geotechnical issues for the pump station:

- Pile capacity (vertical and lateral)
- Settlement of pile
- Slope stability
- Sheet pile wall as intake basin wall
- Site preparation

The field investigation information and laboratory test results presented in our preliminary report were used to develop the geotechnical recommendations outlined herein.

FURNISHED INFORMATION

Based on the provided project drawings (Appendix A), we understand the proposed pump station will be constructed north of Airline Highway as part of the Maurepas Division Canal Project. The station will be located in Gramercy, LA at the convergence of the Hope and Bourgeois Canals. The 200-feet wide conveyance channel, which will convey approximately 2,000 cfs of river water into the Maurepas Swamp, is located immediately to the west of the pump station (Figure 1).

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For this project, the proposed structure accommodating the pump facilities will be supported by a total of 79 precast concrete piles 14 inches square as shown in drawing Sheet 8 in the Appendix. Provided structural loading information includes maximum compressive loads for each pile of 92.6 kips. No lateral load or tensile loading information was provided.

FIELD INVESTIGATION

The subsurface investigation for this project consisted of drilling and sampling one (1) geotechnical exploratory soil boring using a track-mounted rotary drill rig to a depth of 100 feet below existing grade. In addition, one (1) Cone Penetration Test (CPT) was performed to a depth of 75 feet. The boring and sounding were performed at the approximate locations shown on the attached Test Location Plan in the Appendix B.

Relatively undisturbed 3-inch diameter tube samples were generally obtained in cohesive, fine-grained soils (ASTM D-1587), and disturbed 2-inch diameter split-spoon samples (ASTM D-1586) were obtained in coarse grained soils. Standard Penetration Tests (SPT) were performed during the split-spoon sampling. This test consists of dropping a 140-pound hammer 30 inches and recording the number of blows required to drive the sampler in three 6-inch increments. The number of blows in the final 12 inches is recorded on the boring log under the "Sampling Resistance" column. The depths at which the driven split-spoon samples were obtained are indicated as cross-hatched square symbol in the "Sample" column on the boring logs. The depth between which tube samples were obtained is shown as shaded symbols under the "Sample" column of the boring logs. For tubes in which samples were not recovered, split-spoon samples were taken immediately beneath the lost samples for visual field classification.

The CPT Log Sheet is included in the Appendix B graphically showing the cone tip resistance, friction, pore pressure, soil behavior type, and equivalent SPT N values. We highly recommend the engineer be thoroughly familiar with and fully understand the capabilities of the CPT test before using this information for design purposes in order to achieve reliable geotechnical parameters.

Ground water. Ground water was encountered at a depth of about 6.5 feet at the time of our investigation. The depth to water should be verified prior to the initiation of foundation construction, trenching or other operations adversely affected by ground water.

LABORATORY TESTING

Soil mechanics laboratory tests were performed on selected samples from the boring for use in foundation analyses. The results of all tests are shown on the boring log in Appendix B. Unconsolidated undrained tests were performed on selected samples to evaluate soil parameters for use in calculating shear strength. Consolidation tests were performed to evaluate soil past stress history. Atterberg limits determinations, water content determination tests and grain size distribution tests were also performed to classify the subsurface soils more accurately than attainable by field methods.

SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site may be generally described as very soft to medium clays extending from ground surface to a depth of about 30 feet. This layer is underlain by medium silty clays and sandy clays extending to a depth of about 48 feet. Alternating layers of very soft to stiff clays and silty clays, then follow extending to the maximum explored depth of 100 feet. A detailed description of the materials encountered at the site is shown on the Soil Boring Log in the Appendix B.

PILE FOUNDATION RECOMMENDATIONS

Based upon the provided foundation drawings, we understand that 14-inch square precast concrete piles will be used to support the structural components of this project. The piles on this project will derive their compressive capacities primarily from skin resistance. The capacities presented are based upon soil-pile interaction and do not consider the structural aspects of the pile. Piles should be advanced to full penetration to develop the shown capacities. The designer should confirm the pile structural capacity for combined axial and lateral loading.

Driven Piles. Ultimate compressive capacities for single piles were computed based on 14 inches square pre-cast concrete piles. A factor of safety of 2.0 has been applied to develop the allowable capacities as presented in Table 1 below.

The piles were divided into three groups based on the provided criteria of cut off elevation shown in drawing Sheet 8 of Appendix A. The first pile group (No. 1 – 15) has a cut off elevation at 11 feet, the second group (No. 16-23) at elevation 10 feet, and the third group (No. 24-79) at elevation -9.83 feet. For determination of capacity, all pile configurations were assumed to have an embedment beginning at elevation -9.83 feet. Pile lengths should be adjusted so as to maintain proper "tip elevation" as pile butt (or pile head) elevations are adjusted.

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TABLE 1

14' SQUARE PRECAST CONCRETE PILES
SINGLE ALLOWABLE PILE COMPRESSIVE CAPACITY, KIPS

Pile Tip Elev. (ft)	Pile No.		
	1-15	16-23	24-79
-40	30	30	30
-50	44	44	44
-60	59	59	59
-70	78	78	78
-80	105	105	105

Uplift capacity for concrete piles may be taken as 50 percent of the allowable axial capacity. The recommended compression capacities may be increased by 30 percent for temporary loads such as wind or seismic loads.

Negative Skin Friction. Since it appears that little or no fill is to be added in the area of the foundation piles, the predicted pile capacities assume no allowance for negative skin friction. However, if the piles are installed immediately after placement of fill, the soil will settle downward relative to the pile rather than supporting it, imparting a downward load to the pile through shear transfer at the pile-soil interface. This phenomenon, called negative skin friction, requires a reduction in allowable single pile capacity of a given length. Therefore, in all cases, fill should be placed as far in advance of any pile driving operations. If this time frame is not achievable and piles are installed immediately after placement of fill, then the design/builder's geotechnical consultant should be notified to estimate the reduction in pile capacity.

Probe Piles and Pile Test Program. We recommend a pile load test program be performed to verify the pile capacity, drivability, and finalize production pile lengths. Because results of this testing program will have an impact on the final design, it is recommended that these pile load tests be performed in the design phase, rather than immediately prior to construction. The results of these tests may determine if it is possible to reduce the total number of piles used for the project. Representative pile type, pile diameter, driving hammer, and installation procedures should be selected based on the expected production piles. **For driven piles, we recommend a dynamic load test program (PDA) in addition to static load tests to verify compressive capacity.** The dynamic load test program should be performed in general accordance with ASTM D4945. The static pile load test program should include compression, uplift, and lateral

load tests performed in general accordance with ASTM standards: compression – ASTM D1143, uplift – ASTM D3689, and lateral – ASTM D3966. For the static load tests, all piles should be allowed to "set" a minimum of 14 days prior to being loaded.

Lateral Load vs. Horizontal Deflection. Lateral load analyses were performed for individual piles using the computer program LPILE by Ensoft, Inc. Our analyses were performed to estimate the pile head deflection (or butt deflection) for various lateral loading configurations with maximum moment and its location. The piles were divided into three groups based on the provided cut off elevations as shown in drawing Sheet 8 of Appendix A. The first pile group (No. 1 – 15) has cut off elevation at 11 feet, the second group (No. 16-23) at elevation 10 feet, and the third group (No. 24-79) at elevation -9.83 feet. As shown in the drawing Sheet 8, the first and second pile groups have a considerable free-standing portion of the pile. The results of our analyses for 14 inches square precast concrete pile are provided in Tables 2 thru 7. The analyses were performed assuming free and fixed head conditions, respectively. As expected, lateral deflection of the first and second pile groups is greater than that of the third group pile due to the effect of free-standing, unsupported area of the pile.

Table 2

Pile 1-15 with Free-head
Pile Cut-Off Elev. 11 Feet

14-Inch Square Precast Concrete Pile			
Applied Lateral Load (kips)	**Lateral Butt Deflection (in)**	**Maximum Moment (in-kip)**	**Location of Max. Moment below Pile Butt (in)**
0.1	0.25	47	288
0.2	0.67	108	288
0.3	1.23	181	300
0.4	1.92	263	300
0.5	2.74	357	300
0.6	3.70	462	324
0.7	4.81	579	324

Notes:
(1) Pile lengths and tip depths are measured from elevation 11.0 feet. Pile lengths should be adjusted as to maintain proper tip depth as pile butt (cutoff) elevations are adjusted
(2) Pile length 91 ft.

Table 3

Pile 1-15 with Fixed-head
Pile Cut-Off Elev. 11 Feet

14-Inch Square Precast Concrete Pile			
Applied Lateral Load (kips)	Lateral Butt Deflection (in)	Maximum Moment (in-kip)	Location of Max. Moment below Pile Butt (in)
0.1	0.03	18	288
0.2	0.07	37	300
0.3	0.12	57	300
0.4	0.18	78	312
0.5	0.23	100	312
0.6	0.30	121	324
0.7	0.36	144	324

Notes:
(1) Pile lengths and tip depths are measured from elevation 11.0 feet. Pile lengths should be adjusted as to maintain proper tip depth as pile butt (cutoff) elevations are adjusted
(2) Pile length 91 ft.

Table 4

Pile 16-23 with Free-head
Pile Cut-Off Elev. 10 Feet

14-Inch Square Precast Concrete Pile			
Applied Lateral Load (kips)	Lateral Butt Deflection (in)	Maximum Moment (in-kip)	Location of Max. Moment below Pile Butt (in)
0.1	0.23	45	288
0.2	0.59	101	288
0.3	1.07	167	300
0.4	1.65	240	300
0.5	2.32	322	312
0.6	3.08	411	312
0.7	3.94	507	312

Notes:
(1) Pile lengths and tip depths are measured from elevation 10.0 feet. Pile lengths should be adjusted as to maintain proper tip depth as pile butt (cutoff) elevations are adjusted
(2) Pile length 90 ft.

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Table 5

Pile 16-23 with Fixed-head
Pile Cut-Off Elev. 10 Feet

14-Inch Square Precast Concrete Pile			
Applied Lateral Load (kips)	Lateral Butt Deflection (in)	Maximum Moment (in-kip)	Location of Max. Moment below Pile Butt (in)
0.1	0.03	17	288
0.2	0.07	36	288
0.3	0.12	56	300
0.4	0.17	76	300
0.5	0.22	97	312
0.6	0.28	119	312
0.7	0.37	141	312

Notes:
(1) Pile lengths and tip depths are measured from elevation 10.0 feet. Pile lengths should be adjusted as to maintain proper tip depth as pile butt (cutoff) elevations are adjusted
(2) Pile length 90 ft.

Table 6

Pile 24-79 with Free-head
Pile Cut-Off Elev. -9.83 Feet

14-Inch Square Precast Concrete Pile			
Applied Lateral Load (kips)	Lateral Butt Deflection (in)	Maximum Moment (in-kip)	Location of Max. Moment below Pile Butt (in)
1.0	0.08	57	120
2.0	0.25	147	156
3.0	0.51	262	192
4.0	0.82	396	192
5.0	1.18	542	204
6.0	1.59	694	204
7.0	2.04	852	204

Notes:
(1) Pile lengths and tip depths are measured from elevation -9.83 feet. Pile lengths should be adjusted as to maintain proper tip depth as pile butt (cutoff) elevations are adjusted
(2) Pile length 70 ft.

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Table 7

Pile 24-79 with Fixed-head
Pile Cut-Off Elev. -9.83 Feet

14-Inch Square Precast Concrete Pile			
Applied Lateral Load (kips)	**Lateral Butt Deflection (in)**	**Maximum Moment (in-kip)**	**Location of Max. Moment below Pile Butt (in)**
1.0	0.02	53	144
2.0	0.06	129	162
3.0	0.11	214	162
4.0	0.18	306	168
5.0	0.25	402	174
6.0	0.33	501	180
7.0	0.42	604	180

Notes:
(1) Pile lengths and tip depths are measured from elevation -9.83 feet. Pile lengths should be adjusted as to maintain proper tip depth as pile butt (cutoff) elevations are adjusted
(2) Pile length 70 ft.

PILE SETTLEMENT

We estimate settlement of all piles due to provided sustained structural loads (maximum axial load, 92.6 kips) to be on the order ¼ inch. These estimates were for short term deformation and does not consider elastic deformation of the piles. We estimated the short term deformation by using Load Transfer curves, sometimes refer to 't-z' curves for settlement in pile shaft interface and Q-z curves for settlement below pile tip. The values presented in Table 8 are the total short term deformation at pile butt from the Load Transfer curves. Load transfer curves for skin friction and plots between axial load and butt deflection are also provided in Appendix C. These curves and plots are for all three pile groups based on the provided criteria of cut off elevation mentioned in previous sections.

Table 8

14-Inch Square Precast Concrete Pile (1,2)	
Applied Axial Load (kips)	**Axial Butt Deflection (in)**
0.3	0.0004
3.0	0.004
15	0.02
30.0	0.04
107.0	0.26
167.0	0.69
171.0	1.20

SLOPE STABILITY

The slope stability analyses were performed using SLOPE/W software program by Geo-Slope International, Ltd. Four (4) slopes in section A of Sheet 6 of Appendix A were evaluated in our analyses. These analyses were based on a steady state seepage condition. The steady state seepage condition is for the condition when the water level reaches its non-flooding elevations and only slight water level change is expected. The analyzed slope geometry varies depending on each slope and ranges from 5:1 to 3:1. Based on provided information, the water level and elevation of canal's bottom are at elevation 3 feet and -3 feet, respectively. The calculated minimum safety factor from the analyses ranges from 4.2 to 2.4, meeting typical safety requirements for slope stability. Again, this analyses result was based on steady state seepage condition representing a non-hurricane condition. The results of our analyses are presented graphically in Appendix D.

SHEET PILE ANALYSIS FOR INTAKE BASIN WALL

As requested, steel sheet piles were analyzed for incorporation as the intake basin walls for the suction approach channel. Our analysis was based upon a top of wall elevation of 4.0 feet, and a maximum channel bottom elevation of -8.0 feet, and rapid drawdown conditions. The intake basin wall configuration is shown in section B of drawing Sheet 6 in the Appendix. For the analysis, the SPW 11 (Pile Buck) software program was used. An AZ36 steel sheet pile was considered having a section modulus of 67.0 in^3/ft, and a moment of inertia of 606.3 in^4/ft.

A cantilever type wall was analyzed which resulted in significant top of wall deflections. Cantilever sheet piling is usually driven to a sufficient depth into the ground to become fixed as a

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vertical cantilever for resisting the lateral active earth pressure. Due to the excessive top of wall deflections, we consider a cantilever sheet pile system impractical for this project. Analyses were also performed for a tie back (anchored) sheet pile wall system. In the design of a tie back wall, the tie back is assumed to prevent any lateral deflection at the anchor elevation. For the analysis, the anchor was assumed to be located at a distance of 1.0 feet from the top of the sheet pile wall. Our analyses results are summarized in Table 9 below. The complete analysis results are presented in Appendix E.

Table 9

Anchored Sheet Pile Wall

Arbed AZ36 I= 606.30 in^4/ft Z = 67.00 in^3/ft				
Top of Wall Elev. (feet)	**Bottom of Wall Elev. (feet)**	**Tie Back Elev. (feet)**	**Anchor Force**	**Maximum Bending Moment**
4.0	-30.0	3.0	7.4 kips/ft	68.4 kip ft/ft

As an alternative to the tie backs, struts may also be considered spanning between the sheet piles on the opposite sides of the excavation. The struts should be designed considering the same loading conditions as the tie backs. However, the loadings for struts will be in compression rather than tension for tie backs.

Global Slope Stability. To check the global stability of the soil adjacent to a sheet pile wall, a conventional slope stability analyses was performed for Section B in drawing Sheet 6 of Appendix A. The slope stability analyses were performed also using SLOPE/W marked by Geo-Slope International, Ltd. The program is capable of searching for a minimum safety factor with an easy to use interface. The calculated minimum safety factor for this condition is 1.9, meeting typical safety requirements for the slope stability. The range of safety factors typically vary from 1.3 to 1.5 depending on design purpose. The results of the analyses are presented in Appendix E.

Bottom Heave. To check the stability of the excavated bottom from heave, Terzaghi method (1943) was used in our analyses. The estimated embedded depth of sheet pile from Table 9 is 22 feet, being acceptable for heaving stability with excavation depth of 12 feet from the ground surface.

Location of Tie Back Anchors Behind Wall. To obtain maximum efficiency, the tie back anchor locations should be outside of the active and passive soil zone. Based on the location of zero bending moment on the wall and the Rankin definition for the active and passive soil zone, the tie back anchors should be located a horizontal distance at least 34.4 feet behind the wall.

SITE PREPARATION

The sites should be stripped of any subsurface obstructions and topsoil materials, particularly loose or water-softened surface materials, general fill, vegetation, wood, roots, etc. All shallow excavations should be properly backfilled as recommended below.

URS recommends that the upper 8 inches of the subgrade be recompacted at a moisture content within 3 percent of optimum moisture content to a minimum of 95 percent of the maximum dry density obtained in the Standard Proctor Compaction Test (ASTM D698). Moisture contents to achieve recommended compaction will vary depending on material type and should be recommended by a qualified geotechnical engineer. Silty soils tend to lose strength with increasing compaction moisture content. Silty soils, if wet, when being compacted are subject to "pumping." This pumping can often be counter-acted with the addition of three to five percent cement or fly ash by volume.

Since the bearing soils are sensitive to moisture changes, the time an excavation remains open, prior to pouring the concrete foundation should be kept to a minimum. Any open excavation should be protected to prevent entry of rainwater. Also it may be necessary to install a lean concrete "mud" bottom for use as a working platform to facilitate construction during periods of inclement weather.

Structural Fill. Structural fill shall consist of a clean, select, non-expansive fill, free from excess silt, clay balls, or other deleterious matter, and having a plasticity index between 10 and 25 and a maximum liquid limit of 45. Generally, soils meeting these plasticity requirements and classified as lean clays (CL), sandy clays (CL), or clayey sands (SC) by the Unified Soil Classification System exhibit the characteristics of a desirable structural fill.

Other materials not meeting these criteria may be locally available, that satisfy the intent of the earthwork requirements. In all cases, because of possible material variations from the borrow pit and changing site conditions, we recommend that all proposed fill materials be approved by a geotechnical engineer prior to use. The fill should be placed at appropriate moisture contents and lift thickness as determined by a qualified geotechnical engineer. Each lift should be compacted to a minimum of 95 percent of the maximum dry density obtained by a Standard

Proctor Compaction Test (ASTM D698). The compaction and soil materials for each lift should be inspected and approved by an engineering technician supervised by a geotechnical engineer before another lift is added. Backfill of utility and plumbing trenches should also comply with these recommendations.

Drainage. Drainage should be maintained away from foundations both during and after construction. Foundation excavations should be left open for the shortest possible duration to minimize moisture changes in exposed bearing soils. All exposed foundation excavations should be kept free of loose or water-softened soils. Such materials should be removed prior to concrete placement.

Seepage. Seepage of water into the excavations can be handled by a system of sumps and pumps but the groundwater level should be re-confirmed prior to excavation. The slope of open cuts should meet the minimum OSHA guidelines in a dry environment. However, such slopes without sheet pile are subject to degradation and sloughing from rainfall and fluctuations in the groundwater level. Care should be exercised during shored excavations to reduce the potential for excess hydrostatic pressure to build up behind the sheet pile in the event of a heavy rainfall. Any cracks that form between the soils and sheetpile and in the soil itself should be backfilled to reduce infiltration of water behind the sheet pile.

Areal Settlement. Estimated settlements were calculated for varying fill heights to achieve an increase in site elevation. Estimated fill induced settlements are summarized in Table 9. In our settlement analysis, soil boring information and laboratory tests results from the LDNR Diversion Canal project was considered.

Table 9

Fill Height (ft)	Estimated Settlements (inches)
2	7.5
4	11.0
6	15.5

LIMITATIONS

Professional judgments and recommendations are presented in this geotechnical investigation. They are based partly on the information provided, partly on evaluations of technical information gathered, and partly on our general experience with subsurface conditions in the area. We do not

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guarantee the performance of the project in any respect other than our engineering work and the judgments rendered meet the standards and care of our profession as practiced in the State of Louisiana in light of the level of investigative effort and design information available. It should be noted that the widely spaced borings may not represent potentially unfavorable subsurface conditions between borings. If such conditions become evident, additional borings should be performed to characterize these conditions for design review. The recommendations presented in this report are applicable only to this specific site and should not be used for other purposes. Attached at the end of this report is a document entitled "Important Information about your Geotechnical Engineering Report," which is published by ASFE, The Association of Engineering Firms Practicing in the Geosciences, Appendix F. This document should be considered as part of the report and should be furnished to all persons who receive part or all of this report.

We appreciate the opportunity to be of service to you on this project and will be happy to discuss any questions you may have concerning this report. We look forward to serving you again on future projects.

Very truly yours,

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Attachments:

- Figure 1 – Site Location Map
- Appendix A – Pump Station Layouts
- Appendix B – Borings Log and CPT Sounding
- Appendix C – Load Transfer Curves and Axial Load – Deflection Plots
- Appendix D – Slope Stabilities
- Appendix E – Sheet Pile Wall
- Appendix F – Important Information About Your Geotechnical Engineering Report