



**SAER-6027 STRUCTURAL CALCULATIONS FOR
THE STANDARD 10 METRE WIDE PRECAST
CONCRETE SUBSTATION BUILDING**

Prepared for the

Saudi Arabian Oil Company

**Saudi Aramco Report No. SAER-6027
ABS Consulting Report No. 1282017-R-01
Issue 3
February 2006**

DOCUMENT APPROVAL SHEET

Project N°: 1282017
Report N°: 1282017-R-01
Title: Structural Calculations for the Standard 10 Metre Wide Precast Concrete Substation Building
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ISSUE	DATE	PREPARED	REVIEWED	APPROVED
Issue 1	08/10/2004	Marc Lehmann (Principal Consultant)	Alistair Moat (Principal Consultant)	G. Graham (Principal Engineer)
Issue 2	20/12/2005	Andrew Nelson (Project Engineer)	Patrick Mann (Principal Engineer)	G. Graham (Principal Engineer)
Issue 3	01/02/2006	 Andrew Nelson (Project Engineer)	 Patrick Mann (Principal Engineer)	 Patrick Mann (Principal Engineer)

DOCUMENT REVISION HISTORY

ISSUE	DESCRIPTION
Issue 1	Final Issue
Issue 2	Second formal issue, incorporating additional design data in accordance with client comments
Issue 3	Third formal issue, incorporating missing Appendix info from Issue 2

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1282017-CALC-01

1 INTRODUCTION

The purpose of this report is to present the detailed design carried out solely by Saudconsult and Qanbar Dywidag Precast Concrete Ltd. for a 47.5m by 10m wide blast resistant precast concrete substation to accommodate the specific requirements of the Saudi Arabian Oil Company (Saudi Aramco).

This report factually presents the designers calculations and drawings in an ordered manner and ABS Consulting accept no responsibility or liability for the technical content or numerical accuracy contained therein.

The modular design has been based on the design calculations and construction drawings for two types of blast resistant substations at Saudi Aramco's Abu Ali Plant.

The dimensions of these structures, which have been designed in accordance with the relevant blast design codes and ARAMCO Standard SAES-M-009, are:

- a 10m-wide and 47.5m long substation with and an internal clearance to the underside of the beams of 6m.
- a 15m-wide and 40m long substation with and an internal clearance to the underside of the beams of 6m.

The following document provides initial design substantiation for modular design of a 10m wide precast concrete substation between 10m and 50m in length.

However note that the design calculations submitted only consider the Abu Ali 47.5m long substation and no substantiation has been provided for lengths other than a 47.5m long building.

Following receipt of Issue 1 of this report Saudi Aramco raised several queries (Reference 10). Issue 2 of this report addresses the additional design queries raised, specifically:

- The provision of substantiated P.I. diagram;
- the modification of many of the connection details in order to reduce the level of site welding;
- the inclusion of service attachments;
- the tabulation of the foundation loads.

The additional work was concluded and issued 3rd February 2005 (see appendix E) such that the construction drawings could be amended. The revised structural drawings, issued by Al-Osais are included within Appendix C of this report. It is highlighted, however, that the standard details for the Embedded Plates are currently being finalised. These will be included into Appendix C within Report Issue 3.

2 DESIGN REQUIREMENTS

2.1 Design Loads

The building is designed to resist blast loads with a peak side-on pressure of 2.0 psi and an impulse of 200 psi-ms. Where the actual design condition is different from the above, P-I diagrams have been produced to indicate the building performance under the varying load condition. Figure 2.1 illustrates the time history curve and shape for the blast loading.

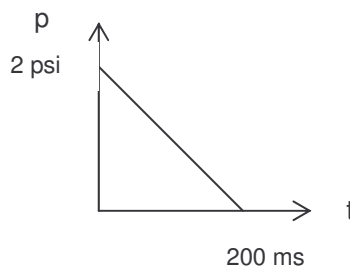


Fig 2.1: Blast Load Shape

As a blast wave strikes a structure facing the source, it creates an increase in the pressure applied at the interface. This is commonly known as a reflected blast pressure. The reflected pressure is a function of the angle at which the blast wave strikes the surface and the magnitude of the incident pressure. Normally reflected pressures are of the order of two to three times the side-on values for this pressure range. The reflected pressure predicted for this structure was 4.2 psi. Both the side-on and reflected blast loads were conservatively assumed to have a triangular pressure-time history.

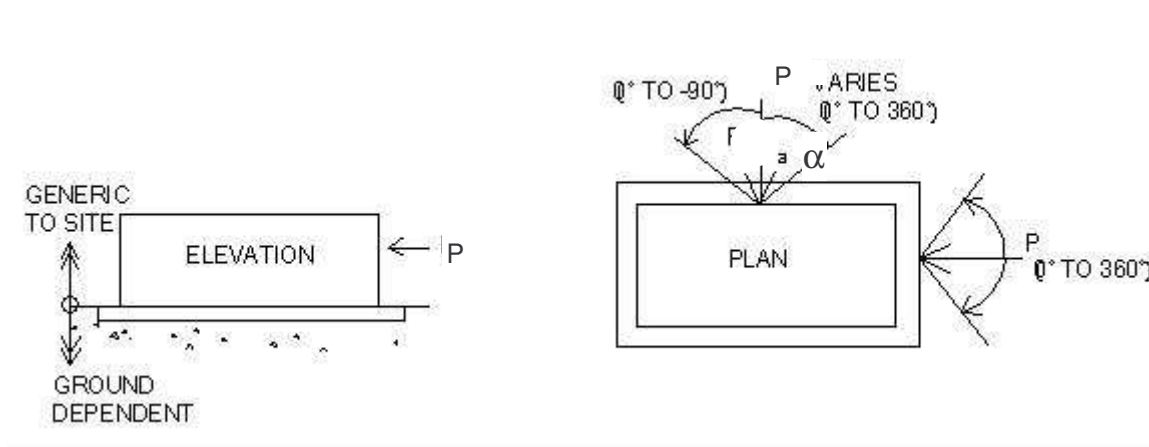


Fig 2.2: Plan and Elevation showing Blast Load striking the building

In a typical scenario, only walls facing the blast will receive reflected loading. However, the explosion loading is to be taken as potentially happening from any direction, which is conservative and consistent with a modular design concept.

The roof was assumed to be uniformly loaded by the incident or side-on pressures. If this design is used for a situation where the explosion source is elevated, the structural system should be reanalysed to determine if the roof can resist a reflected load.

2.2 Performance Criteria

An overall response criterion of "Low" was used as defined within Saudi Aramco SAES-M-009 dated 30th April 2002, Design Criteria for Blast Resistant Buildings, Reference 1.

This response criterion indicates localised building/component damage during the blast load. Following the blast the building can be used, however repairs are likely to be required to restore the integrity of the structure. The total cost of repairs is likely to be moderate. Also life-safety hazards due to collapse would be prevented.

2.3 Load Combinations

Dead loads were combined with blast loads to evaluate the effects of a decreased resistance of the roof members to an external load. Note a load factor of 1.0 was used for all blast load combinations and is consistent with ARAMCO requirements.

Live load from surfacing and cable trays was been taken as 1 kN/m^2 at roof level.

The following service load combinations were considered in the blast design:

- | | | |
|-------------------|-------|--------------------------------|
| a) (DL + LL) | where | DL = Dead Load |
| b) (DL + LL + WL) | | LL = Live Load |
| c) (DL + BL) | | WL = Wind Load |
| | | BL = time dependent Blast Load |

3 STRUCTURAL COMPONENTS

The principal structural components for modular construction of the substation are shown in Figure 3.1. The components are constructed from pre-cast wall panels and roof slabs with steel plate inserts that are connected together to form the integral structure.

3.1 Walls

The walls will be formed from insulated precast panels approximately 7.35m tall by 2.5m wide. The walls will have a sandwich construction consisting of an inner 250mm thick structural section, a 50mm insulated layer and a 60mm concrete outer skin. The panel will be thickened locally where there is a requirement to have a structural connection to the inside such as at the roof or foundation connection. These will be joined to adjacent panels by a bolted connection on the inner face of the wall. This is required to create a composite action between the panels in order to sustain the blast loading.

3.2 Roof

The roof panels will comprise of reinforced concrete double tee beams, 2.5m wide with a 150mm deep flange. The beam will initially rest on a reinforced concrete corbel during construction. However a bolted connection to the wall panel will be used to resist downward and rebound blast loading forces in the final condition. The possibility of using prestressed roof beams was investigated but it was found that the beams could not withstand the significant re-bound forces. This induces a significant hogging moment into the beams, which the prestressed design cannot withstand.

3.3 Foundations

The foundation comprises 950mm deep, 2500mm wide strip footings around the perimeter of the building and below all internal walls. The strip footings are each cast-in-situ integral with the 450mm deep ground slab.

3.4 Blast Doors

There are two types of blast doors personnel doors and equipment doors. The personnel doors are placed centrally within a panel on the long side of the building. The equipment doors are located centrally between two panels on the short side. All blast doors and anchored steel plate framing shall be designed in accordance with ARAMCO Standard SAES-M-009 and the relevant blast design codes.

3.5 Cable Tray Openings

Cable tray openings are positioned at approximately 4m above ground level along the front and rear elevations of the building to allow the intake of equipment ducts into the substation. These openings in the building envelope shall be designed to prevent entry of blast pressures in accordance with ARAMCO Standard SAES-M-009 and the relevant blast design codes.

Blast valves, blast attenuators, or other devices shall be used where required to limit blast pressure entry into the structure. The performance of the blast valve or attenuator shall be substantiated by test data and calculation.

3.6 HVAC

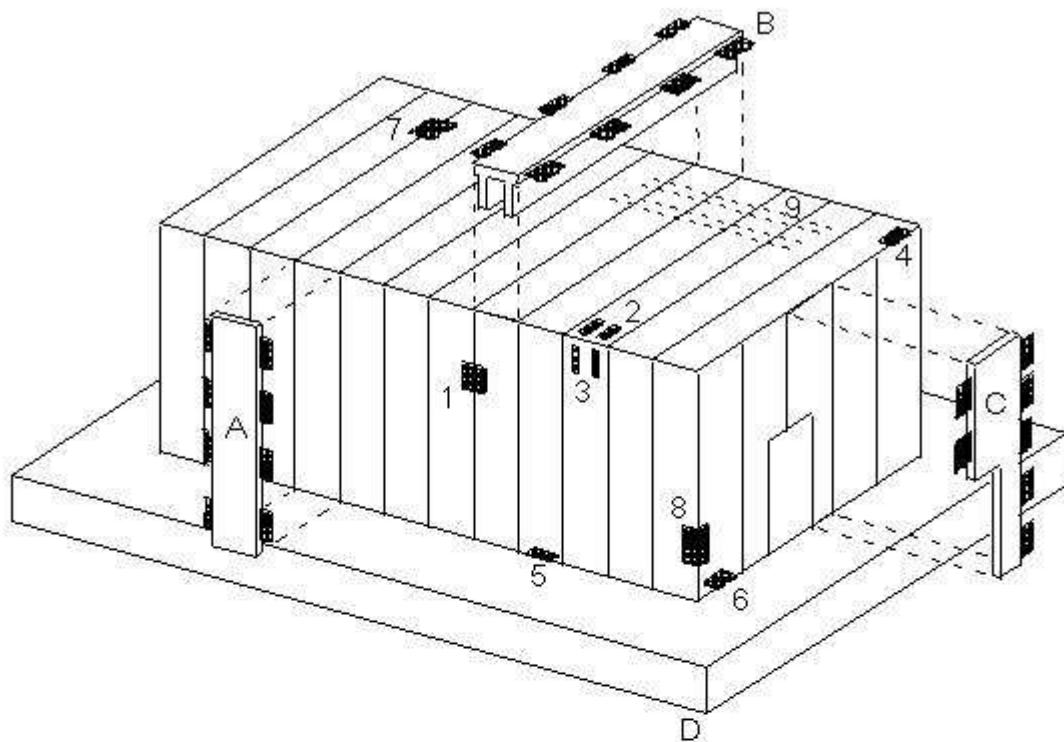
HVAC equipment shall be designed in accordance with ARAMCO Standard SAES-M-009 and the relevant blast design codes. Ductwork suspended from the roof inside the building shall be secured to structural framing members. Anchorage shall be designed to resist a

peak, statically applied force equal to the mass of the item times the maximum acceleration of the roof or 5 times the weight of the item, whichever is less.

3.7 Equipment and Internally Mounted Items

Equipment and Internally Mounted Items shall be designed in accordance with ARAMCO Standard SAES-M-009 and the relevant blast design codes. Instrumentation or electrical equipment shall not be mounted on the interior face of walls subjected to blast loads without prior written approval from the Manager, Consulting Services Department.

All fixed floor supported items, such as lockers, electrical cabinets, racks, etc. shall have a minimum clearance from exterior walls equal to the maximum calculated lateral blast load deflection. The maximum deflection shall consider both the overall building side sway plus deflection of any wall component(s) and shall be computed based on the maximum design blast load defined in Chapter 2.1. Supports and anchorage for such equipment shall be designed to resist a lateral force equal to 20% of the equipment weight.



MAIN PRECAST MEMBERS

- A: SOLID WALL PANEL
- B: DOUBLE-T ROOF PANEL
- C: WALL PANEL WITH DOOR OPENINGS
- D: FOUNDATION (DEPENDENT ON GROUND CONDITION)

TYPICAL STEEL PLATE CONNECTIONS

- 1: WALL PANEL TO WALL PANEL
- 2: ROOF PANEL TO WALL PANEL (FLANGE)
- 3: ROOF PANEL TO WALL PANEL (WEB)
- 4: END ROOF PANEL TO WALL PANEL
- 5: WALL PANEL TO FOUNDATION (LONG SIDE)
- 6: WALL PANEL TO FOUNDATION (SHORT SIDE)
- 7: ROOF PANEL TO ROOF PANEL
- 8: WALL PANEL CORNER CONNECTION
- 9: ROOF PANEL TENSION CHORDS

Figure 3.1: Overview – Structural Components of Modular Blast Resistant Substation

4 ANALYSIS METHODS

4.1 Conventional Loads

Conventional loads such as dead, live and wind loads have also been combined in the load combinations with blast and are presented in the final design calculations in Appendix A of this document.

4.2 Blast Loads

Horizontal blast loads are resisted by the wall panels acting in bending and shear to transfer the loads to the foundations (at the base of the panel) and to the roof plate (at the top of the panel). The welded double-tee pre-cast roof panels form the horizontal diaphragm that transmits these loads in bending to the end shear walls and down to the foundations. Figure 4.1 illustrates this concept and the expected structural load paths under blast loading.

Each member of the structure, including precast roof and wall elements, was designed for the blast loads applied to the structure as described in Chapter 2.1. The components were analysed by computing equivalent single-degree-of-freedom (SDOF) properties and determining dynamic response through the use of a numerical integration procedure. This method computes the time history response of a critical point on the member to the transient blast load. Response at this critical point is used to evaluate the adequacy of the structures through the use of response limits.

Maximum reactions at the end of each component were computed as a function of the maximum resistance of the member. Peak values for these reactions were used to design the connections rather than the peak value of the applied load. Using the peak value of the applied load could produce an unworkable connection design. The design philosophy for the connections is to provide sufficient resistance to develop the full flexural capacity of the member while preventing premature failure of the member at the connection.

The SDOF response of each of the components was determined using methods presented in the following documents: Saudi Aramco Engineering Standard Design Criteria for Blast Resistant Buildings (SAE-M-009), Design of Blast Resistant Buildings in Petrochemical Facilities (ASCE, 1998) and Structures to Resist the Effects of Accidental Explosions (TM5-1300). These documents are widely accepted as standard design guides for accidental explosions throughout the petrochemical industry.

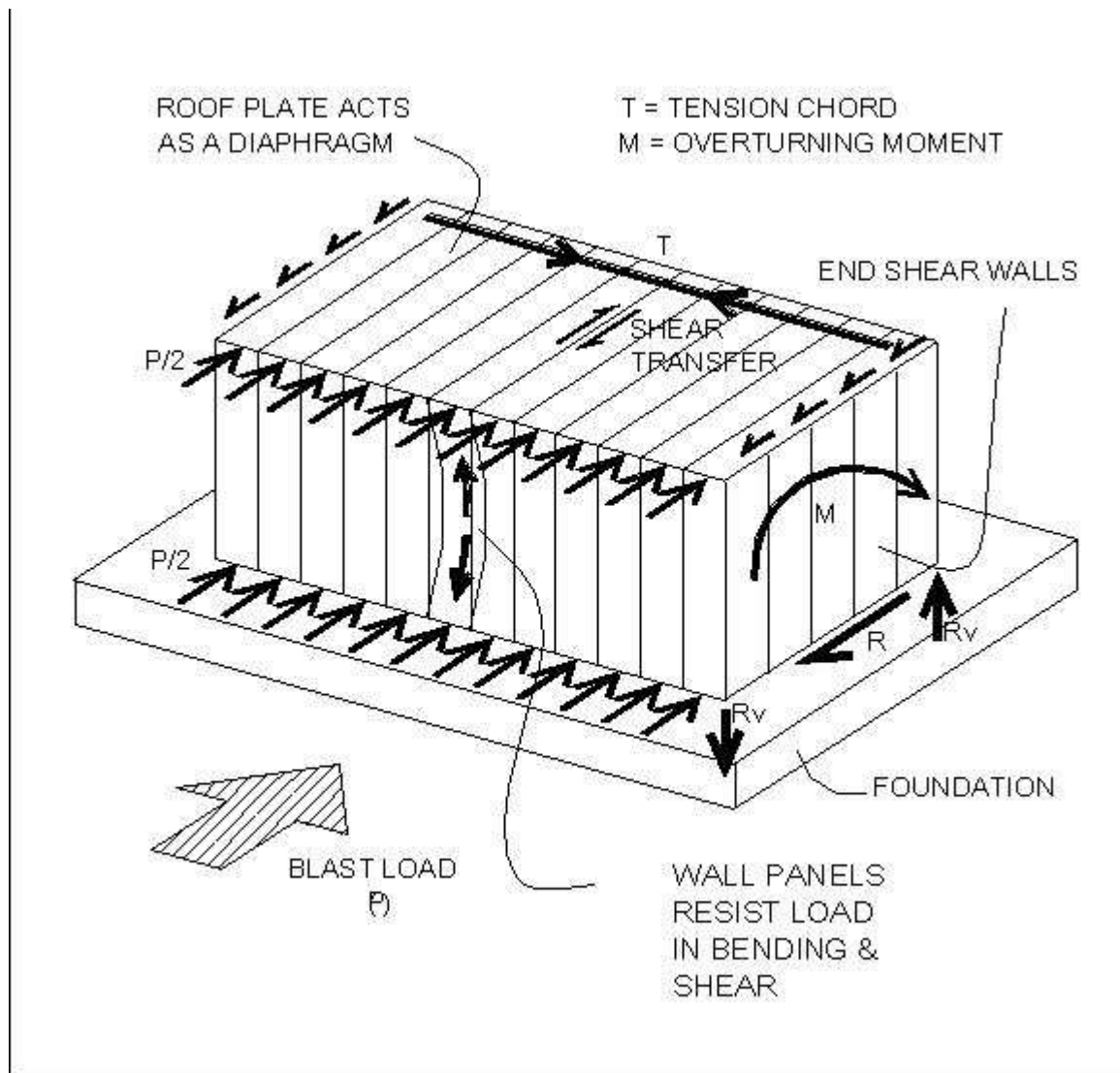


Figure 4.1: Structural Load Paths of Modular Blast Resistant Substation

5 MATERIALS

Design strengths for material properties were based on minimum specified values for commonly available materials used in this type of construction. These minimum values were used for computing member capacities for resistance to conventional static loads. However, elements subjected to blast loads are deformed or strained at a very rapid rate because of the short duration of the load. Because the material cannot respond as rapidly as the load is applied, it exhibits an apparent increase in yield strength or tensile strength. This strength increase occurs in both reinforced concrete and structural steel. The magnitude of the increase is dependent upon the type of stress being evaluated (flexure, shear, tension, or compression), and the magnitude of the load. The higher the applied load, the greater the strain rate effects on yield and ultimate strength. The ASCE guideline contains recommended values for dynamic increase factors for both concrete and steel as a function of load range and stress type. Values selected for use in the blast design are described in the sections below.

Material properties specified as minimum values in ASTM and other specifications are generally lower than the actual material properties produced by the manufacturer. The ASCE guideline recommends taking advantage of this fact by increasing the static yield strength of materials by approximately 10%. It recommends this strength increase on the yield strength of Grade 60 reinforcing bars and structural steel with a yield strength of 50 ksi or less. A dynamic design stress was used to compute member capacities by applying the dynamic increase factor and the strength increase factor to the minimum specified static strength for the material.

5.1 Concrete

A minimum 28-day compressive cylinder strength of 5,000 psi was used for all precast concrete elements. All cast-in-place concrete (i.e. foundations) was designed using a compressive strength of 4,000 psi. ASTM A615, Grade 60 reinforcing was used throughout the design. This material has a yield stress of 60,000 psi. The table below contains the dynamic increase factors, material strength increase factor and resulting dynamic design strengths for the precast concrete and reinforcing used in the design. The values for dynamic increase factors are based on recommendations given in the ASCE guideline.

Stress Type	Concrete		Reinforcing				
	DIF	F' _{dc} (psi)	DIF _{yield}	F _{dy} (psi)	DIF _{ult}	F _{du} (psi)	F _{ds} (psi)
Flexure	1.19	5,950	1.17	77,200	1.05	94,500	77,200
Diagonal Tension	1.0	5,000	1.0	66,000	--	--	66,000
Direct Shear	1.1	5,500	1.1	72,600	1.0	90,000	72,600
Bond	1.1	5,500	1.17	77,200	1.05	94,500	77,200
Compression	1.12	5,600	1.1	72,600	--	--	72,600

Table 5.1: Dynamic Properties for Precast Reinforced Concrete (f'_c=5000 psi, f_y=60000 psi)

Where

DIF = Dynamic Increase Factor,

F'_{dc} = dynamic compressive strength of concrete,

DIF_{yield} = Dynamic Increase Factor to yield strength of reinforcement,

F_{dy} = dynamic yield stress of reinforcement,

DIF_{ult} = Dynamic Increase Factor to ultimate strength of reinforcement,

F_{du} = dynamic ultimate stress of reinforcement,

F_{ds} = dynamic design stress of reinforcement.

5.2 Structural Steel Connections

The building is primarily a reinforced concrete design with structural steel plate inserts used for connections between different precast members and in the roof diaphragm to take the in-plane tension chord loads. For connections mild steel was used for all structural steel elements. Static properties for this material include a yield strength of 320MPa and an ultimate strength of 504MPa.

It is noted that the designer has utilised a strength increase factor of 1.1 however this is not considered strictly applicable on the steel connection checks since the yield stress used is a lower bound stress from the test results.

Dynamic increase factors were applied to the static properties to determine dynamic yield and ultimate strength design stresses. Since the assumed ductility ratio was less than 10 for this design, the dynamic design stress, F_{ds} was set equal to F_{dy} . The dynamic shear stress was taken to be equal to $0.55 * F_{ds}$. The table below contains the dynamic increase factors and dynamic design stresses for structural steel.

Element Type	Stress Type	DIF_{yield}	F_{dy}	DIF_{ult}	F_{du}	F_{ds}
Beam	Flexure	1.19	381	1.05	529	381
	Shear	1.19	381	1.05	529	381
	Tension/Compression	1.12	358	1.05	529	358

Table 5.2: Dynamic Properties for Structural Steel

Where DIF_{yield} = Dynamic Increase Factor yield strength
 F_{dy} = dynamic yield stress,
 DIF_{ult} = Dynamic Increase Factor ultimate strength
 F_{du} = dynamic ultimate stress
 F_{ds} = dynamic design stress

The blast design assumes the connections will carry only direct shear or tension loads i.e. no moment connections.

5.3 Response Criteria

Response limits for structural members subjected to blast loads were determined based on material type, element type, and element function as laid out in Reference 1. Values selected were based on "Low Response" criteria provided in the guideline. The tables below show the response limits used for design of steel and concrete elements. The limits are selected to avoid undue conservatism in limiting the response of the structure while still maintaining adequate protection against failure of the members and collapse of the building.

Element Type	Response Range ⁽¹⁾			
	Low		Medium	
	μ	θ	μ	θ
Beams, Girts, Purlins	3	2	10	6
Frame Members	1.5	1	2	1.5
Cold-Formed Panels	1.75	1.25	3	2
Open-Web Joists	1	1	2	1.5
Plates	5	3	10	6

Table 5.3: Deformation Limits for Structural Steel

(1) Response parameter: μ = ductility ratio, θ = support rotation (degrees)

Element Type	Controlling Stress	Ductility Ratio, μ	Support Rotation, θ (2)	
			Low	Medium
Beams	Flexure	N/A	1	2
	Shear: (1)			
	- Concrete Only	1.3		
	- Concrete + Stirrups	1.6		
	- Stirrups Only	3.0		
	Compression	1.3		
Slabs	Flexure	N/A	2	4
	Shear: (1)	1.3		
Beam-Columns	Flexure	N/A	1	2
	Compression	1.3		
	Tension	(3)		
	Shear (1)	1.3		
Shear Walls, Diaphragms	Flexure	3	1	1.5
	Shear (1)	1.5		

Table 5.4: Deformation Limits for Concrete

- Where
- (1) Shear controls when shear resistance is less than 120% of flexural resistance.
 - (2) Stirrups are required for support rotations greater than 2 degrees.
 - (3) Ductility ratio = $0.05 (\rho - \rho') < 10$, where ρ and ρ' are the tension and compression reinforcement ratios, respectively.)

6 MEMBER DESIGN

6.1 Precast Concrete Wall Design

The 2.5m wide precast reinforced concrete wall panels have been designed for reflected blast loads using ultimate strength methods. SDOF properties were determined for the walls based on section thickness, reinforcing, span, and boundary conditions. A time history response was then computed using numerical integration techniques. Results of this analysis included maximum deflection of the walls, maximum reactions, and shear capacity checks. The final design calculations and analysis are presented in the Appendices of this document.

To sustain the reflected blast loads the precast concrete walls have been designed as one-way members, simply supported at the base and roof. The walls have reinforcement on each face of the section. A 6.775m span for the walls was analysed and it was determined that T25 bars at 300mm centres would provide sufficient resistance to sustain the blast load. Maximum shear stress for the walls was determined to be less than the capacity of the walls as long as 5,000 psi concrete was used for the precast construction. Therefore, no shear reinforcing is required in the typical panel. Additionally, support rotations were held below 2° to meet the requirements of the ASCE guideline for concrete elements without stirrups. For higher loads, which necessitate a medium response, stirrups are required.

Three types of precast panel details have been developed that allowed for installation of doorways and transit openings. The first detail allows for a standard 625mm high by 1200mm wide cable tray opening at a height of 4m above ground. The second detail allows for a 2200mm high by 925mm wide Personnel doorway and the third a 2800mm high by 1380mm wide (1/2 door width) Equipment doorway. At each edge of the doorways, an in-wall beam-column has been designed to resist the blast loads from its own tributary width as well as one-half the width of the doorway.

The precast concrete walls have also been designed as shear walls to distribute loads from the roof diaphragm into the foundation (see Figure 2). The loads are transferred from the roof to the walls via bolted steel connection plates embedded between the wall and roof panels. To ensure the wall panels act together in shear, they are also attached together with inter-panel connections consisting of flat steel plates bolted to the embedded angles across the gap between the panels. In the case of the intersection of two panels at the corners of the building, a welded angle detail is substituted for the bolted flat plates.

Connections have been designed to provide the transfer of all loads from the walls into the roof diaphragm and foundations. Details of the precast wall connections are provided in the drawings shown in Appendix C, and calculations in Appendix B and E for the welded and bolted connections respectively. (Note: Appendix B includes designs for all connections, although most are now superseded by the details in Appendix E)

Depending on the direction of the blast load (i.e. the elevation parallel to the blast wave) and the chosen length of the building, the shear walls shall have the following minimum configurations:

Elevation	No. of Wall Panels	Building Length	Shear Wall Connection Detail (Drawing Ref Appendix C)
Side	4	$L \leq 10\text{m}$	979-BD-SB2-07-1/2, Detail 02 & 04
	4	$10\text{m} < L \leq 50\text{m}$	979-BD-SB2-07-1/2, Detail 01 & 05
Front	up to 6	$L \leq 15\text{m}$	979-BD-SB2-07-1/2, Detail 01 & 05
	7 to 19	$15\text{m} < L \leq 50\text{m}$	979-BD-SB2-07-1/2, Detail 02 & 04
Rear	up to 6	$L \leq 15\text{m}$	979-BD-SB2-07-1/2, Detail 01 & 05
	7 to 19	$15\text{m} < L \leq 50\text{m}$	979-BD-SB2-07-1/2, Detail 02 & 04

Table 6.1: Shear Wall Configurations

The precast concrete walls also serve to support the weight of the roof panels. Initially during construction this will be by use of a corbel onto which the roof panel will rest. Following installation, the web of the tee section will be bolted to the wall insert designed to carry the full downward and rebound forces. The wall panel is thickened locally to sustain this loading.

6.2 Precast Concrete Roof Design

The precast reinforced concrete roof panels have been designed for side-on blast loads using ultimate strength methods for prediction of its blast capacity. SDOF properties were determined for the slab based on section thickness, reinforcing, span, and boundary conditions. A time history response was then computed using numerical integration techniques. Results of this analysis included maximum deflection of the slab at mid span, maximum reactions, and shear capacity checks.

The roof panels have been designed as a double tee beam one-way spanning, pinned-pinned member. The flange is 150mm thick and 2500mm wide with the tee sections at 1250mm centreline spacing. The analysis confirmed that a 500mm deep (650mm overall) and 200mm wide beam web with 4 No. T25 bars would provide sufficient resistance to sustain the blast load.

The roof panels are connected together in the same way as the wall panels using flat steel plates bolted to embedded angles in the panels to allow the overall roof to act as a diaphragm to distribute the blast load reactions at the top of the walls back to the shear walls. Additional reinforcing steel plate is also provided along the longer edges of the roof to act as tensile chords for the moments induced in the diaphragm. This consists of three continuous steel strips welded to the embedded steel plates in the panels.

6.3 Steel Connection Design

Simple connections are provided at the intersection of the wall panels, roof panels, foundation and inter-panel joints. The use of the precast concrete roof panels as diaphragms and precast concrete wall panels as shear walls eliminated the need for moment resisting frames. An overview of these connections is shown in Figure 3.1.

Connection 1: Wall Panel to Wall Panel Connection (Long Wall)

This is a flat steel plate bolted to cast-in-place angle inserts at the outer edge of the wall panels. This is used to: -

- Ensure a rigid shear wall action by transferring in-plane shear forces between the wall panels

Connection 2: Roof Panel to Wall Panel Flange Detail

This is a flat steel plate bolted to cast-in-place inserts at the top flange at the end of the roof panels and a cast-in-place insert at the upper level of the wall panels. This is used to:

- Transfer out of plane reaction forces at the top of the wall panels into the roof diaphragm
- Transfer loads from the roof diaphragm into the end shear wall panels

Connection 3: Roof Panel to Wall Panel Web Detail

This is a flat steel plate bolted to cast-in-place inserts at the web of the roof panels tee beams and cast-in-place inserts at the upper level of the wall panels. This is used to: -

- Transfer out of plane reaction forces from the roof panel into the load bearing wall panel.

Connection 4: End Roof Panel to Wall Panel

These are 2 flat steel plates bolted to cast-in-place inserts at the top and bottom flange at the side of the roof panels and cast-in-place inserts at the upper level of the end wall panels. This is used to: -

- Transfer out of plane reaction forces from the wall panel back into the roof diaphragm.
- Transfer loads from the roof diaphragm into the end shear wall panels

Connection 5: Wall Panel to Foundation (Long Wall)

This is an angle section bolted to cast-in-place inserts at the bottom of the wall panels and bolted to the foundation using cast-in-place anchors. This is used to: -

- Transfer out of plane reaction forces from the wall panel into the foundation.
- Transfer in-plane shear wall forces to the foundation
- Transfer vertical uplift forces to the foundation

Connection 6: Wall Panel to Foundation (Short Wall)

These are two angle sections bolted to cast-in-place inserts at the bottom of the wall panels and cast-in-place inserts at the foundation. This is used to: -

- Transfer out of plane reaction forces from the wall panel into the foundation.
- Transfer in-plane shear wall forces to the foundation
- Transfer vertical uplift forces to the foundation

Connection 7: Roof Panel to Roof Panel Connection

This is a flat steel plate bolted to cast-in-place inserts at the top flange down the side of the roof panels. This is used to: -

- Ensure a rigid roof diaphragm by transferring in-plane shear forces between the roof panels

Connection 8: Wall Panel Corner Connection

This is a steel angle welded to cast-in-place angle inserts at the inner corner of the main wall panels. This is used to: -

- Ensure a rigid shear connection between the long and short walls at their intersection.

Connection 9: Roof Panel Tension Chord

These are 2.5m long flat steel embedded plates located along the edge of the roof plate and used to transfer the tension chord forces between roof panels and ensure the roof acts as a diaphragm. These are bolted together by a connecting splice plate.

6.4 Foundation Design

The foundation design has been undertaken for the arrangement highlighted within the contained design calculations and drawings. The design adopted an energy balance method which has shown the performance of both the SS#1 and SS#2 foundations to respond adequately. The assessment assumed that the:

- allowable bearing capacity exceeds 130kN/m^2 .
- foundation depth is 1m below ground level
- building base slab is a single unit with a slab depth of 450mm and 950mm at its perimeter
- suitable reinforcement is placed around the anchors such that their loading is transferred without the risk of edge bursting of the concrete.

Where the above assumptions are not valid or that the building floor level is raised due to the area classification, site specific foundation calculations shall be undertaken. The site specific design calculations shall use the loading provided in Table 2.

Substation		Maximum Vertical Loading	Maximum Loading Perpendicular to the Long Wall	Maximum Loading Parallel to the Long Wall	Maximum Anchor Loads
SS1	40m long x 15m wide	907kN	365kN	174kN	65kN
SS2	47.5m long x 10m wide	1732kN	606kN	338kN	65kN

Table 6.2: Foundation Design Loads

7 MODULAR DESIGN

The design basis dimension of the 10m wide, blast resistant substation is between 10m to 50m. The calculations for this are presented in Appendix A and B. The modular precast concrete designs for the walls and roof sections will allow for the lengthening and shortening of the overall building floor plans without requiring a change in the typical wall or roof design. Depending on the chosen length of the building the requirements for the shear wall connection details may vary and as a minimum should be in line with those given in Table 6.1. Since the roof sections are designed to span across the width of the building, these widths should be kept constant for the buildings. Otherwise a new roof T-beam section design would be required.

The typical width of the T-beam roof sections and precast walls is 2.5m. The coordination of the roof and wall sections will therefore allow for the overall length of the building to be lengthened or shortened in 2.5m sections. Changes in the length of the building will not affect the local blast response of any of the precast components. The global response of the building including diaphragm/shear wall connections and overturning/sliding response are the major technical concerns in changing length of the buildings.

The global response of the substation for lengths of the building ranging from 10m to 50m with a fixed width of 10m in 2.5m increments has been assessed. Floor plans falling within these limits can be implemented with minimal structural modifications. It was considered impractical to shorten the building beyond a square footprint and lengthening the building beyond 50m will likely require additional internal shear walls and strengthening of connections.

The following sketch shows the minimum and maximum lengths of the Substation floor plan modularization. Please note doorways are shown for illustrative purposes only and these locations may be altered to suit the client's needs.

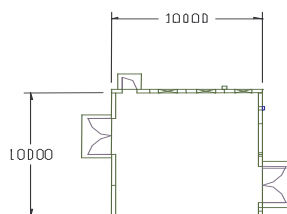


Figure 7.1: Minimum Floor Plan Size

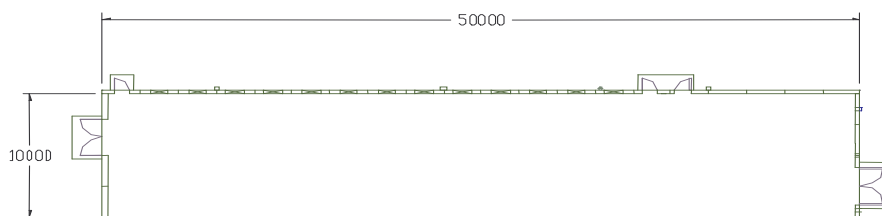


Figure 7.2: Maximum Floor Plan

8 PRESSURE-IMPULSE DIAGRAMS

P-I diagrams were developed for the Substation and are shown in Appendix E as part of the Dynamic Analysis. Individual diagrams for each major building component including roof T-beam sections, typical walls, walls with personnel doors, and walls with equipment doors were produced. An overall building P-I diagram was then constructed based on the controlling individual pressure and impulse values.

P-I diagrams were developed for response levels of Low and Medium as defined in SAES-M-009. For Low Response the following response criteria were used to develop the individual component diagrams:

Component	Rotation Limit (Degrees)
Roof T-Beam Sections	1
Precast Wall Sections	2

Table 8.1: Response Levels Used for P-I Diagram Development for Low Response

For Medium Response the following response criteria were used to develop the individual component diagrams:

Component	Rotation Limit (Degrees)
Roof T-Beam Sections	2
Precast Wall Sections	4*

* Requires stirrups

Table 8.2: Response Levels Used for P-I Diagram Development for Medium Response

Medium Response level for the precast wall sections was set at 4 degrees assuming that stirrups will be added to the designs. However, while, it is possible to include stirrups in precast concrete walls, in most cases it is not economical and can cause constructability problems. If stirrups are not provided, wall rotation limits shall not exceed 2 degrees, i.e. low response only.

The P-I diagrams developed for the substation can be used in conjunction with the standardised building sizes. Roof and diaphragms are designed to withstand the maximum plastic response of the roof and precast wall reactions. Lengthening the buildings will only affect the global response of the buildings. Therefore, these diagrams can be used to represent the substations in all of the floor plans within the limits described in the standardisation discussion.

9 CONCLUSIONS

The final design package for the 10m wide precast concrete, blast resistant substation building has been prepared for Saudi Aramco. These substations will be constructed initially at the Abu Ali Plant and then used as a modular design for multiple company sites. The defined blast loads on the standard buildings are specified as a peak side-on pressure of 2.0 psi with an impulse of 200psi-ms.

The standard substation buildings are to be a single storey, 10m wide and of variable length. The structure utilises an in-situ reinforced concrete foundation and precast reinforced concrete wall and roof panels. The walls are load bearing, which removes the requirement for beam and column construction and the roof beams are single spans removing any internal columns.

The structural performance goal for the design of the building is defined as “Low” for a Multi-Unit Substation and as “Medium” for a Single Unit Substation. The “Low” response criterion indicates localised building/component damage during the blast. Following the blast the building can be used, however repairs are likely to be required to restore the integrity of structure. The total cost of repairs is likely to be moderate. Pressure Impulse diagrams have been produced which can be used for the design of the buildings under varying pressure/impulse values for low response.

The design is modular in nature and analysis and can be constructed from a short “square” shape up to approx 50m in length.

However the design substantiation for building lengths other than the 47.5m length for Abu Ali have not been provided within this submission.

10 REFERENCES

1. Saudi Aramco SAES-M-009, Design Criteria for Blast Resistant Buildings, dated 30th April 2002.
2. Departments of the Army, the Navy, and the Air Force, "Structure to Resist the Effects of Accidental Explosions," Department of the Army Technical Manual TM 5-1300, Department of the Navy Publication NAVFAC P-397, Department of the Air Force Manual AFM 88-22, Revision 1, November 1990.
3. "Design of Blast Resistant Buildings in Petrochemical Facilities," Prepared by the Task Committee on Blast Resistant Design, Published by American Society of Civil Engineers, 1997.
4. "PCI Design Handbook", Precast / Prestressed Concrete Institute, 1992.
5. -ASCE Standard, "Minimum Design Loads for Buildings and other Structures", ANSI Approved November 1998.
6. ICBO, Uniform Building Code, International Conference of Building Officials, Whittier, CA, 1997.
7. AISC, Manual Of Steel Construction; Load Factor and Resistance Design LRFD (2nd Edition), American Institute of Steel Construction, Chicago, IL, 1994.
8. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary (ACI 318R-89)", American Concrete Institute, Detroit, MI, 1989.
9. ACI Committee 349, " Nuclear Safety Structures Code (ACI 349-01) and Commentary (ACI 349R-01)", American Concrete Institute, Detroit, MI, 2001.
10. E-mail from Perry Catmunan (Al-Osais) to Alistair Moat (ABS Consulting), dated 21/11/04.

APPENDIX A

Dynamic Analysis Calculations (incl. Pressure-Impulse Diagrams)

SAUDI ARABIAN OIL COMPANY

BI - 3039**REPLACE SUBSTATION 472, 321 & F07****ABU ALI****STRUCTURAL DESIGN CALCULATION****NEW SUBSTATION NO. 1 & 2****FINAL SUBMITTAL***Prepared by***Saudi Consulting Services
Al-Khobar, Saudi Arabia**

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ENG	MGG		
DESCRIPTION	ISSUED FOR 40% REVIEW		
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DESIGN CRITERIA

SAUDI ARABIAN OIL COMPANY

DESIGN CRITERIA**A. CODES AND STANDARDS**

The latest edition of the following codes & standards shall constitute the minimum requirement:

- | | | | |
|-----|------------|---|---|
| 1. | SAES-M-100 | - | Saudi Aramco Building Code |
| 2. | SAES-Q-001 | - | Criteria for Design & Construction of Concrete Structures |
| 3. | SAES-Q-005 | - | Concrete Foundation |
| 4. | SAES-M-001 | - | Structural Design Criteria for Non-Building Structures |
| 5. | SAES-M-009 | - | Design Criteria for Blast Resistant Building |
| 6. | SAES-A-112 | - | Meteorological and Seismic Design Data |
| 7. | SAES-B-014 | - | Safety Requirement for Plant & Operation Support Building |
| 8. | SAES-O-109 | - | Buildings Housing Sensitive or Vital Equipment |
| 9. | UBC | - | Uniform Building Code |
| 10. | ACI 318 | - | American Concrete Institute |
| 11. | AISC | - | American Institute of Steel Construction |
| 12. | ASCE 7 | - | American Society of Civil Engineers (Min. Design Loads for Buildings & Others Structures) |

MINIMUM DESIGN LOADS

1. Dead Load
 - a) Concrete = 24.0 kN/m³
 - b) Utilities (Refer to itemized loadings)
2. Live Load
 - a) Roof = 1.0 kN/m²
 - b) Battery Room = 6.0 kN/m²
 - c) Switchgear Room = 6.0 kN/m²
 - d) Mechanical Room = 7.2 kN/m²

C. DESIGN STRENGTH

1. Concrete
 - a) Structural Concrete ;
 - 1.) Precast Wall Panels ; $f'c = 34.5$ MPa
 - 2.) Precast Roof Double Tee Beam Panels ; $f'c = 34.5$ MPa
 - 3.) Foundation (Building and Equipment) ; $f'c = 27.5$ MPa

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2. Reinforcing Bars
 a) Deformed Bars ; $f_y = 420 \text{ MPa}$
3. Structural Steel ; $F_y = 248 \text{ MPa}$

ITEMIZED LOADING

1. Roof Loads (DL)
- | | |
|--------------------------------------|-------------------|
| Roofing Memberane | = 0.20 KPa |
| Gravel 50 mm thk Bedding | = 1.00 KPa |
| Rigid Roof Insulation | = 0.10 KPa |
| Insulating Concrete Screed 100mm thk | = 1.00 KPa |
| Airconditioning Utilities | = 0.20 KPa |
| Lighting Fixtures | = 0.05 KPa |
| Electrical Cable Trays | = <u>0.25 KPa</u> |

2. Ground Floor Loads (DL)

Floor Finishes	= 0.60 KPa
Equipment Weight	
AHU (39G1725)	= 2.30 KPa
AHU (39G1724)	= 1.98 KPa

WIND LOAD

Design Wind Pressure $\longrightarrow$ as per UBC-1997
 $P = C_e C_q q_s I_w \longrightarrow$ eq. 20-1 of UBC-1997

Where :

$C_e = 1.13$ (Exposure C) $\longrightarrow$ UBC Table 16-G
 $C_q = 1.3$ (Horizontal, any direction) $\longrightarrow$ UBC Table 16-H
 $q_s = 0.78 \text{ kPa}$
 $I_w = 1.15$ (Essential Facilities) $\longrightarrow$ UBC Table 16-K

$P = 1.13 (1.3) (0.78) (1.15)$
 $P = 1.32 \text{ KPa}$

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BLAST LOAD

Peak Side-On Overpressure = 13.8 KPa (2.0 psi)
 Front Wall Impulse = 1380 KPa -msec. (200 psi-msec)

SOIL PROPERTIES

Allowable Net Soil Bearing Capacity ; (Refer attached)

- 1.) Substation # 1 ; qa = 40 kPa
 2.) Substation # 2 ; qa = 90 kPa

Ultimate Net Soil Bearing Capacity ;

$$qa = qu / SF ; \quad qu = SF \cdot qa$$

$$SF = 3.0$$

- 3.) Substation # 1 ; qu = 120 kPa
 4.) Substation # 2 ; qu = 270 kPa

LOAD COMBINATIONS

1. Service Loads
 a) (DL+LL)
 b) (DL + LL+WL)
 c) (DL + BL)
2. Factored Loads
 a) (1.4 DL+ 1.7 LL)
 b) 0.75 (1.4 DL+ 1.7 LL+ 1.7 WL)

Where :

DL = Dead Load
 LL = Live Load
 WL = Wind Load
 BL = Blast Load

ALLOWABLE DEFLECTION

1. Vertical deflection – as per UBC Table 16-D

$$y = \frac{L}{360} \text{ - live load only}$$

$$y = \frac{L}{240} \text{ - dead load and live load}$$

2. Lateral drift

$$D_{\max} = \frac{h}{375}$$

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DESIGN CONCEPT OF STRUCTURAL ANALYSIS

The proposed substation buildings shall utilize combination of precast concrete panel (ie, Wall and Roof Double Tee Beam) and cast in situ concrete mat foundation. Each component of the structure were analyzed by computing the equivalent single-degree-of-freedom (SDOF) properties and determined dynamic response using numerical integration method.

Equivalent static design method was used for the foundation. Proposed new substations are analyzed as a Shear Wall System framing. Lateral loads (ie. Wind, Blast) shall be resisted by the diaphragm action of the roof to transfer loads at the end walls. The mat foundation was analyzed and designed using the finite element method from PCAMATS software where the loads applied are from the reactions of the STAAD analysis. In the equivalent static design method, the blast downward force from the overpressure on the roof is applied simultaneously with the horizontal force from the peak reflected pressure on the front wall. The negating effect on the rear wall is neglected.

The model in the STAAD analysis for foundation evaluation used the finite element model to model the precast wall panel. The roof tee beam are modeled as a beam, however a dummy model of a four node element was used to model the diaphragm effect of the tee beam flange. A dummy beam member was also used around the perimeter walls to take the uniform loads of the parapet. Only the applicable and appropriate properties are used on all dummy members otherwise near to zero for densities are inputted. Pinned model for the support is used.

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LOAD CALCULATION

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Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : STRUCTURAL FRAME LOADING CALCULATION

1. DEAD LOAD:

1.1 @ ROOF LEVEL:

MEMBER LOAD:

PARAPET WALL = $24 \times 0.36 \times 0.45$ = 3.888 KN/m

MEMB: 141 TO 184 UNIGY - 3.89 (SS# 1)

PARAPET WALL = $24 \times 0.36 \times 0.55$ = 4.752 KN/m

MEMB: 123 TO 168 UNIGY - 4.752 (SS# 2)

ELEMENT LOAD (AREA LOAD):

	<u>SS # 1</u>	<u>SS # 2</u>
ROOFING MEMBRANE	= 0.20 KN/m ²	0.20 KN/m ²
GRAVEL 50MM THK. BEDDING	= 1.00 KN/m ²	1.00 KN/m ²
75 mm RIGID ROOFING INSULATION	= 0.10 KN/m ²	0.10 KN/m ²
INSULATING CONC. SCREED @ 100mm THK.	= 1.00 KN/m ²	1.00 KN/m ²
PRECAST ROOF PANEL	= 5.67 KN/m ²	5.41 KN/m ²
AIR CONDITIONING DUCTING	= 0.20 KN/m ²	0.20 KN/m ²
LIGHTING FIXTURES	= 0.05 KN/m ²	0.05 KN/m ²
ELECTRICAL CABLE TRAYS	= 0.25 KN/m ²	0.25 KN/m ²
	<u>8.47 KN/m²</u>	<u>8.21 KN/m²</u>

ELEM: 45 TO 140 PRESSURE 8.47 (SS# 1)

ELEM: 47 TO 122 PRESSURE 8.21 (SS# 2)

1.2 @ FLOOR LEVEL:

- INTERIOR 200 MM THK PRECAST WALL PANEL PARTITION = $24 \times 0.2 \times 6.5$ = 31.20 KN/m
TO BE APPLIED ON FOUNDATION DESIGN AND ANALYSIS (PCAMATS)
- EXTERIOR 250 MM THK PRECAST WALL PANEL = $24 \times 0.25 \times 6.5$ = 39.00 KN/m
APPLIED AS SELFWEIGHT IN STAAD
- FLOOR FINISHES W/ 1" THK. SCREED = 0.60 KN/m²
TO BE APPLIED ON FOUNDATION DESIGN AND ANALYSIS (PCAMATS)
- AHU (39G1725) @ MECHANICAL ROOM = 2.30 KN/m² ; SUBSTATION # 1
AHU (39G1724) @ MECHANICAL ROOM = 1.98 KN/m² ; SUBSTATION # 2
- SWITCHGEAR & @BATTERY ROOM = 2.00 KN/m² ; SUBSTATION # 1 & 2

NOTE: THE PRECAST WALL PANEL SELFWEIGHT IS DIRECTLY CALCULATED
 BY STAAD PROGRAM THRU " SELFWEIGHT " COMMAND.

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		Check'd by :	SAK

2. LIVE LOAD:

2.1 @ ROOF LEVEL:

ELEMENT LOAD: (1.0 KN/m²)

ELEM: 45 TO 140 PRESSURE 1.0 (SS# 1)

ELEM: 47 TO 122 PRESSURE 1.0 (SS# 2)

2.2 @ FLOOR LEVEL:

ELEMENT LOAD:

@ MECHANICAL / AHU ROOM : (7.20 KN/m²)

@ SWITCHGEAR AND BATTERY ROOM : (6.0 KN/m²)

TO BE APPLIED ON FOUNDATION DESIGN AND ANALYSIS AS SURFACE LOAD(PCAMATS)

3. WIND LOAD (z-DIRECTION):

3.1 @ EL. 0.15 TO EL. 7.35

$$Pe = Ce Cq Qs Iw = 1.32 \text{ KN/m}^2$$

$$\text{Where: } Ce = 1.13$$

$$Cq = 1.30$$

$$Iw = 1.15$$

$$Qs = 0.78 \text{ KN/m}^2$$

ELEMENT LOAD:

ELEM: 17 TO 32 PRESSURE 1.32 (SS# 1)

ELEM: 20 TO 38 PRESSURE 1.32 (SS# 2)

NOTE:

THIS CALCULATION CONSIDER ONLY THE WIND AT Z-DIRECTION, OBVIOUSLY BECAUSE THIS IS THE WEAK AXIS OF THE BLDG. STRUCTURAL FRAME WHICH WILL PRODUCED THE MAX. MEMBER REACTIONS.

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Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject : BLAST LOADING CALCULATION SUBSTATION # 1

Sheet no. : of
Date : 13-Nov-03
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BLAST LOAD

DATA:

PEAK SIDE-ON OVERPRESSURE, P_{so} = 13.79 Kpa (2.00 psi)
POSITIVE PHASE IMPULSE, I_o = 1379 Kpa-msec. (200.0 psi-msec)
BLAST TIME DURATION, T_d = 200 msec.

BUILDING DATA:

BLDG. WIDTH NORMAL TO BLAST DIR., B = 40 m (131.2 ft)
BLDG. LENGTH ALONG BLAST DIRECTION, L = 15.72 m (51.58 ft)
BLDG. HEIGHT, H = 7.2 m (23.62 ft)

CALCULATION :

REFERENCE : ASCE REPORT " DESIGN OF BLAST RESISTANT BLDG."

SHOCK FRONT VELOCITY, $U = 1130 (1 + 0.058 P_{so})^{0.5}$ = 1193.7 ft/sec.
PEAK DYNAMIC WIND PRESSURE, $q_o = 0.022 P_{so}^2$ = 0.09 psi (0.6067 kPa)
CLEARANCE DISTANCE, $S = (\text{MIN. OF } H \text{ OR } B/2)$ = 23.623 ft
LENGTH OF PRESSURE WAVE, $L_w = U (T_d)$ = 238.75 ft

FRONT WALL DESIGN LOAD PRESSURE:

FRONT WALL DRAG COEFFICIENT, C_d = 1.00
FRONT STAGNATION PRESSURE, $P_s = P_{so} + C_d q_o$ = 2.09 psi (14.396 kPa)
REFLECTED OVERPRESSURE CLEARING TIME, $T_c = 3 (S / U)$ = 59.4 msec
REFLECTED OVERPRESSURE, $P_r = (2 + 0.05 P_{so}) P_{so}$ = 4.2 psi (28.958 kPa)
FRONT WALL IMPULSE, $I_w = 0.5 (P_r - P_s) T_c + 0.5 P_s T_d$ = 271.49 psi-msec (1871.9 Kpa-sec.)
EFFECTIVE TIME DURATION, $T_e = 2 I_w / P_r$ = 129.28 msec (0.129 sec.)

SIDE WALL DESIGN LOAD PRESSURE:

SIDE WALL DRAG COEFFICIENT, C_d = -0.40
EQUIVALENT LOAD COEFFICIENT, C_L = 1.00
SIDE WALL EQUIVALENT OVERPRESSURE, $P_s = C_L P_{so} + C_d q_o$ = 1.9648 psi (13.547 kPa)
Rise Time, $t_r = L_1 / U$, WHERE $L_1 = 1$ ft = 0.838 msec (0.001 sec.)
TOTAL POSITIVE PHASE DURATION, $T_o = T_d + T_r$ = 200.84 msec (0.201 sec.)

ROOF SLAB DESIGN LOAD PRESSURE:

ROOF SLAB DRAG COEFFICIENT, C_d = -0.40
EQUIVALENT LOAD COEFFICIENT, C_L = 1.00
ROOF SLAB EQUIVALENT OVERPRESSURE, $P_s = C_L P_{so} + C_d q_o$ = 1.9648 psi (13.547 kPa)
Rise Time, $t_r = L_1 / U$, WHERE $L_1 = 1$ ft = 0.838 msec (0.001 sec.)
TOTAL POSITIVE PHASE DURATION, $T_o = T_d + T_r$ = 200.84 msec (0.201 sec.)

REAR WALL DESIGN LOAD PRESSURE:

REAR WALL DRAG COEFFICIENT, C_d = -0.40
EQUIVALENT LOAD COEFFICIENT, C_L = 1.00
REAR WALL EQUIVALENT OVERPRESSURE, $P_s = C_L P_{so} + C_d q_o$ = 1.96 psi (13.547 kPa)
Arrival Time, $t_a = L_1 / U$, WHERE $L_1 = 51.577$ ft = 43.21 msec (0.043 sec.)
Rise Time, $t_r = S / U$, WHERE $S = 23.623$ ft = 19.79 msec (0.020 sec.)
TOTAL POSITIVE PHASE DURATION, $T_o = T_d + T_r$ = 219.79 msec (0.220 sec.)

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : BLAST LOADING CALCULATION SUBSTATION # 2

Sheet no. : of
 Date : 13-Nov-03
 Design by : MGG
 Check'd by : SAK

BLAST LOAD

DATA:

PEAK SIDE-ON OVERPRESSURE, P_{so} = 13.79 Kpa (2.00 psi)
 POSITIVE PHASE IMPULSE, I_o = 1379 Kpa-msec. (200.0 psi-msec)
 BLAST TIME DURATION, T_d = 200 msec.

BUILDING DATA:

BLDG. WIDTH NORMAL TO BLAST DIR., B = 47.5 m (155.8 ft)
 BLDG. LENGTH ALONG BLAST DIRECTION, L = 10.72 m (35.17 ft)
 BLDG. HEIGHT, H = 7.2 m (23.62 ft)

CALCULATION :

REFERENCE : ASCE REPORT " DESIGN OF BLAST RESISTANT BLDG."

SHOCK FRONT VELOCITY, $U = 1130 (1 + 0.058 P_{so})^{0.5}$ = 1193.7 ft/sec.
 PEAK DYNAMIC WIND PRESSURE, $q_o = 0.022 P_{so}^2$ = 0.09 psi (0.6067 kPa)
 CLEARANCE DISTANCE, $S = (\text{MIN. OF } H \text{ OR } B/2)$ = 23.623 ft
 LENGTH OF PRESSURE WAVE, $L_w = U (T_d)$ = 324.09 ft

FRONT WALL DESIGN LOAD PRESSURE:

FRONT WALL DRAG COEFFICIENT, C_d = 1.00
 FRONT STAGNATION PRESSURE, $P_s = P_{so} + C_d q_o$ = 2.09 psi (14.396 kPa)
 REFLECTED OVERPRESSURE CLEARING TIME, $T_c = 3 (S / U)$ = 59.4 msec
 REFLECTED OVERPRESSURE, $P_r = (2 + 0.05 P_{so}) P_{so}$ = 4.2 psi (28.958 kPa)
 FRONT WALL IMPULSE, $I_w = 0.5 (P_r - P_s) T_c + 0.5 P_s T_d$ = 271.49 psi-msec (1871.9 Kpa-sec.)
 EFFECTIVE TIME DURATION, $T_e = 2 I_w / P_r$ = 129.28 msec (0.129 sec.)

SIDE WALL DESIGN LOAD PRESSURE:

SIDE WALL DRAG COEFFICIENT, C_d = -0.40
 EQUIVALENT LOAD COEFFICIENT, C_L = 1.00
 SIDE WALL EQUIVALENT OVERPRESSURE, $P_s = C_L P_{so} + C_d q_o$ = 1.9648 psi (13.547 kPa)
 Rise Time, $t_r = L_1 / U$, WHERE $L_1 = 1$ ft = 0.838 msec (0.001 sec.)
 TOTAL POSITIVE PHASE DURATION, $T_o = T_d + T_r$ = 200.84 msec (0.201 sec.)

ROOF SLAB DESIGN LOAD PRESSURE:

ROOF SLAB DRAG COEFFICIENT, C_d = -0.40
 EQUIVALENT LOAD COEFFICIENT, C_L = 1.00
 ROOF SLAB EQUIVALENT OVERPRESSURE, $P_s = C_L P_{so} + C_d q_o$ = 1.9648 psi (13.547 kPa)
 Rise Time, $t_r = L_1 / U$, WHERE $L_1 = 1$ ft = 0.838 msec (0.001 sec.)
 TOTAL POSITIVE PHASE DURATION, $T_o = T_d + T_r$ = 200.84 msec (0.201 sec.)

REAR WALL DESIGN LOAD PRESSURE:

REAR WALL DRAG COEFFICIENT, C_d = -0.40
 EQUIVALENT LOAD COEFFICIENT, C_L = 1.00
 REAR WALL EQUIVALENT OVERPRESSURE, $P_s = C_L P_{so} + C_d q_o$ = 1.96 psi (13.547 kPa)
 Arrival Time, $t_a = L_1 / U$, WHERE $L_1 = 35.172$ ft = 29.46 msec (0.029 sec.)
 Rise Time, $t_r = S / U$, WHERE $S = 23.623$ ft = 19.79 msec (0.020 sec.)
 TOTAL POSITIVE PHASE DURATION, $T_o = T_d + T_r$ = 219.79 msec (0.220 sec.)

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

SUBSTATION # 1

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

STAAD III - ANALYSIS

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

PRECAST ROOF PANEL DOUBLE TEE BEAM CROSS SECTION ; SS # 1

Saudconsult

Job No. : BI-3039

Project : REPLACEMENT OF SUBSTATION 472, 321 & F07, ABU ALI

Subject : CROSS SECTION PROPERTIES, SUBSTATION # 1

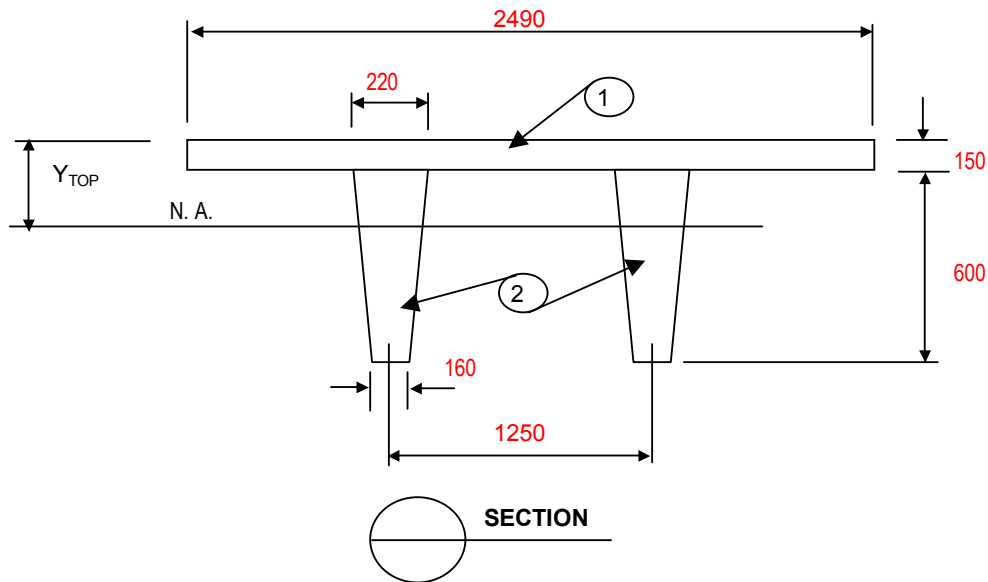
Sheet no. : of 3

Date : 18/11/03

Design by : MGG

Check'd by: SAK

PRECAST ROOF PANEL DOUBLE TEE BEAM CROSS SECTION



ELEMENT	AREA (A)	Y	A x Y	Io	D	A x D ²
1	373500	75.00	28012500	7.0E+08	136.160	6924479699
2	228000	434.21	99000000	6.8E+09	223.051	11343391085
TOTAL	601500		127012500	7.5E+09		18267870784

$$Y_{TOP} = \frac{127012500}{601500}$$

$$= 211.159601 \text{ MM}$$

$$I_x = 2.5751E+10 \text{ MM}^4$$

$$I_x = 0.0258 \text{ M}^4$$

$$\text{WEIGHT PER 2.5 M WIDTH PANEL} = \frac{(\text{TOTAL AREA} \times 23.56 \text{ KN/M}^3) / 2.5 \text{ M}}{2.5}$$

$$= \frac{0.6015 \times 23.56}{2.5}$$

$$= 5.669 \text{ Kpa}$$

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

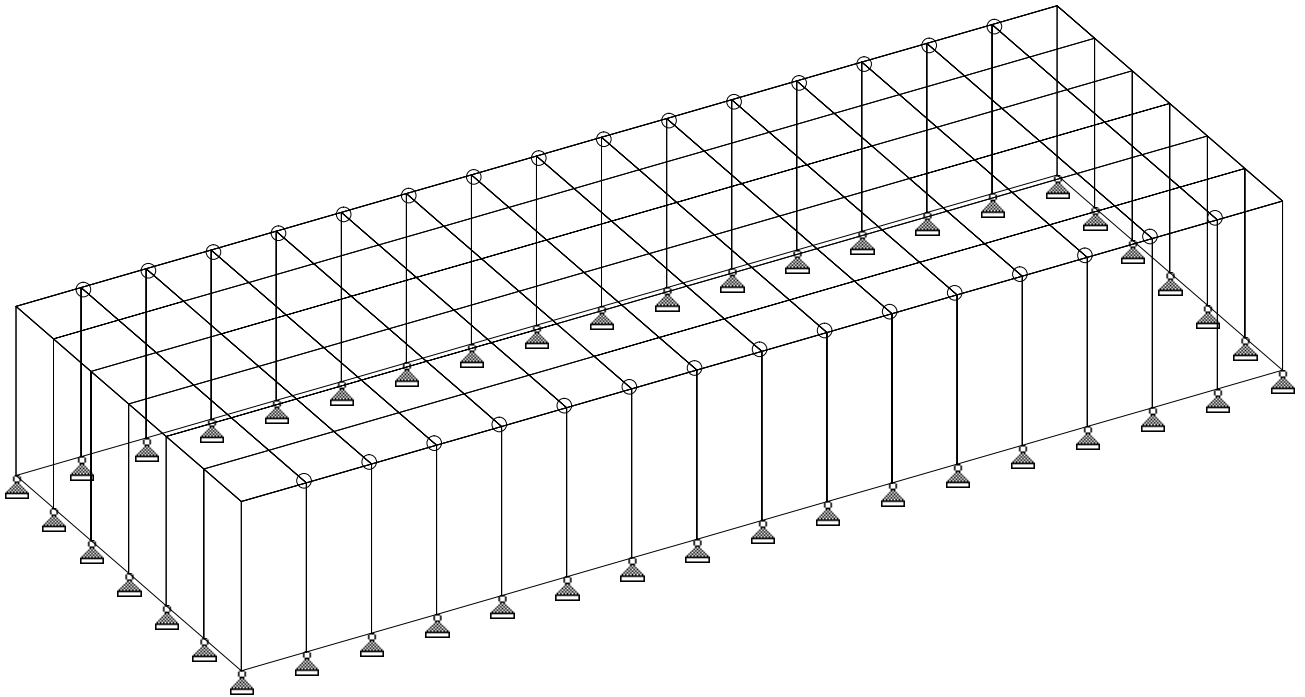
Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

STAAD III - MODEL



Software licensed to SC

Job No	Sheet No 1	Rev
Part		
Ref		
By	Date 20-Sep-03	Chd
Client	File SS1Af.std	Date/Time 27-Jan-2004 17:20



Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

STAAD III - OUTPUT

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*****
*                                     *
*          STAAD/Pro STAAD-III        *
*        Revision 3.1                *
*      Proprietary Program of        *
*    RESEARCH ENGINEERS, Inc.        *
*      Date=   JAN 27, 2004          *
*      Time=   17:20:14              *
*                                     *
*      USER ID: SC                   *
*****

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1. STAAD SPACE SUBSTATION # 1
2. START JOB INFORMATION
3. ENGINEER DATE 20-SEP-03
4. *****
5. *****          SUB-STATION NO. 1          *****
6. *****          BI-3039                    *****
7. *****          SAUDI CONSULTING SERVICES  *****
8. *****          SAUDCONSULT                *****
9. *****          CIVIL/STRUCTURAL DEPARTMENT *****
10. *****          DATE: JAN 27, 2004         *****
11. *****          REVISION 2                 *****
12. *****
13. ** THIS RUN IS CONSIDERED ONLY TO GET THE SUPPORT REACTION
14. ** FOR THE FOUNDATION DESIGN. PRECAST ROOF AND WALL PANELS
15. ** ARE BASE ON THE MANUFACTURERS DATA
16. END JOB INFORMATION
17. INPUT WIDTH 79
18. UNIT METER KN
19. JOINT COORDINATES
20. 1 0 0 0; 2 2.5 0 0; 3 5 0 0; 4 7.5 0 0; 5 10 0 0; 6 12.5 0 0; 7 15 0 0
21. 8 17.5 0 0; 9 20 0 0; 10 22.5 0 0; 11 25 0 0; 12 27.5 0 0; 13 30 0 0
22. 14 32.5 0 0; 15 35 0 0; 16 37.5 0 0; 17 40 0 0; 18 0 6.5 0; 19 2.5 6.5 0
23. 20 5 6.5 0; 21 7.5 6.5 0; 22 10 6.5 0; 23 12.5 6.5 0; 24 15 6.5 0
24. 25 17.5 6.5 0; 26 20 6.5 0; 27 22.5 6.5 0; 28 25 6.5 0; 29 27.5 6.5 0
25. 30 30 6.5 0; 31 32.5 6.5 0; 32 35 6.5 0; 33 37.5 6.5 0; 34 40 6.5 0; 35 0 0 15
26. 36 2.5 0 15; 37 5 0 15; 38 7.5 0 15; 39 10 0 15; 40 12.5 0 15; 41 15 0 15
27. 42 17.5 0 15; 43 20 0 15; 44 22.5 0 15; 45 25 0 15; 46 27.5 0 15; 47 30 0 15
28. 48 32.5 0 15; 49 35 0 15; 50 37.5 0 15; 51 40 0 15; 52 0 6.5 15; 53 2.5 6.5 15
29. 54 5 6.5 15; 55 7.5 6.5 15; 56 10 6.5 15; 57 12.5 6.5 15; 58 15 6.5 15
30. 59 17.5 6.5 15; 60 20 6.5 15; 61 22.5 6.5 15; 62 25 6.5 15; 63 27.5 6.5 15
31. 64 30 6.5 15; 65 32.5 6.5 15; 66 35 6.5 15; 67 37.5 6.5 15; 68 40 6.5 15
32. 69 0 0 2.5; 70 0 6.5 2.5; 71 0 0 5; 72 0 6.5 5; 73 0 0 7.5; 74 0 6.5 7.5
33. 75 0 0 10; 76 0 6.5 10; 77 0 0 12.5; 78 0 6.5 12.5; 79 40 0 2.5; 80 40 6.5 2.5
34. 81 40 0 5; 82 40 6.5 5; 83 40 0 7.5; 84 40 6.5 7.5; 85 40 0 10; 86 40 6.5 10
35. 87 40 0 12.5; 88 40 6.5 12.5; 89 2.5 6.5 2.5; 90 2.5 6.5 5; 91 2.5 6.5 7.5
36. 92 2.5 6.5 10; 93 2.5 6.5 12.5; 94 5 6.5 2.5; 95 5 6.5 5; 96 5 6.5 7.5
37. 97 5 6.5 10; 98 5 6.5 12.5; 99 7.5 6.5 2.5; 100 7.5 6.5 5; 101 7.5 6.5 7.5
38. 102 7.5 6.5 10; 103 7.5 6.5 12.5; 104 10 6.5 2.5; 105 10 6.5 5; 106 10 6.5 7.5
39. 107 10 6.5 10; 108 10 6.5 12.5; 109 12.5 6.5 2.5; 110 12.5 6.5 5
40. 111 12.5 6.5 7.5; 112 12.5 6.5 10; 113 12.5 6.5 12.5; 114 15 6.5 2.5
41. 115 15 6.5 5; 116 15 6.5 7.5; 117 15 6.5 10; 118 15 6.5 12.5; 119 17.5 6.5 2.5

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42. 120 17.5 6.5 5; 121 17.5 6.5 7.5; 122 17.5 6.5 10; 123 17.5 6.5 12.5
43. 124 20 6.5 2.5; 125 20 6.5 5; 126 20 6.5 7.5; 127 20 6.5 10; 128 20 6.5 12.5
44. 129 22.5 6.5 2.5; 130 22.5 6.5 5; 131 22.5 6.5 7.5; 132 22.5 6.5 10
45. 133 22.5 6.5 12.5; 134 25 6.5 2.5; 135 25 6.5 5; 136 25 6.5 7.5; 137 25 6.5 10
46. 138 25 6.5 12.5; 139 27.5 6.5 2.5; 140 27.5 6.5 5; 141 27.5 6.5 7.5
47. 142 27.5 6.5 10; 143 27.5 6.5 12.5; 144 30 6.5 2.5; 145 30 6.5 5
48. 146 30 6.5 7.5; 147 30 6.5 10; 148 30 6.5 12.5; 149 32.5 6.5 2.5
49. 150 32.5 6.5 5; 151 32.5 6.5 7.5; 152 32.5 6.5 10; 153 32.5 6.5 12.5
50. 154 35 6.5 2.5; 155 35 6.5 5; 156 35 6.5 7.5; 157 35 6.5 10; 158 35 6.5 12.5
51. 159 37.5 6.5 2.5; 160 37.5 6.5 5; 161 37.5 6.5 7.5; 162 37.5 6.5 10
52. 163 37.5 6.5 12.5
53. MEMBER INCIDENCES
54. 141 18 19; 142 19 20; 143 20 21; 144 21 22; 145 22 23; 146 23 24; 147 24 25
55. 148 25 26; 149 26 27; 150 27 28; 151 28 29; 152 29 30; 153 30 31; 154 31 32
56. 155 32 33; 156 33 34; 157 52 53; 158 53 54; 159 54 55; 160 55 56; 161 56 57
57. 162 57 58; 163 58 59; 164 59 60; 165 60 61; 166 61 62; 167 62 63; 168 63 64
58. 169 64 65; 170 65 66; 171 66 67; 172 67 68; 173 18 70; 174 70 72; 175 72 74
59. 176 74 76; 177 76 78; 178 78 82; 179 34 80; 180 80 82; 181 82 84; 182 84 86
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61. 190 93 94; 191 20 94; 192 94 95; 193 95 96; 194 96 97; 195 97 98; 196 98 99
62. 197 21 99; 198 99 100; 199 100 101; 200 101 102; 201 102 103; 202 103 104
63. 203 22 104; 204 104 105; 205 105 106; 206 106 107; 207 107 108; 208 108 109
64. 209 23 109; 210 109 110; 211 110 111; 212 111 112; 213 112 113; 214 113 114
65. 215 24 114; 216 114 115; 217 115 116; 218 116 117; 219 117 118; 220 118 119
66. 221 25 119; 222 119 120; 223 120 121; 224 121 122; 225 122 123; 226 123 124
67. 227 26 124; 228 124 125; 229 125 126; 230 126 127; 231 127 128; 232 128 129
68. 233 27 129; 234 129 130; 235 130 131; 236 131 132; 237 132 133; 238 133 134
69. 239 28 134; 240 134 135; 241 135 136; 242 136 137; 243 137 138; 244 138 139
70. 245 29 139; 246 139 140; 247 140 141; 248 141 142; 249 142 143; 250 143 144
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72. 257 31 149; 258 149 150; 259 150 151; 260 151 152; 261 152 153; 262 153 154
73. 263 32 154; 264 154 155; 265 155 156; 266 156 157; 267 157 158; 268 158 159
74. 269 33 159; 270 159 160; 271 160 161; 272 161 162; 273 162 163; 274 163 164
75. ELEMENT INCIDENCES SHELL
76. 1 1 18 19 2; 2 2 19 20 3; 3 3 20 21 4; 4 4 21 22 5; 5 5 22 23 6; 6 6 23 24 7
77. 7 7 24 25 8; 8 8 25 26 9; 9 9 26 27 10; 10 10 27 28 11; 11 11 28 29 12
78. 12 12 29 30 13; 13 13 30 31 14; 14 14 31 32 15; 15 15 32 33 16; 16 16 33 34 17
79. 17 35 52 53 36; 18 36 53 54 37; 19 37 54 55 38; 20 38 55 56 39; 21 39 56 57 40
80. 22 40 57 58 41; 23 41 58 59 42; 24 42 59 60 43; 25 43 60 61 44; 26 44 61 62 45
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83. 37 75 76 78 77; 38 77 78 82 35; 39 17 34 80 79; 40 79 80 82 81; 41 81 82 84 83
84. 42 83 84 86 85; 43 85 86 88 87; 44 87 88 68 51; 45 18 19 89 70; 46 19 20 94 89
85. 47 20 21 99 94; 48 21 22 104 99; 49 22 23 109 104; 50 23 24 114 109
86. 51 24 25 119 114; 52 25 26 124 119; 53 26 27 129 124; 54 27 28 134 129
87. 55 28 29 139 134; 56 29 30 144 139; 57 30 31 149 144; 58 31 32 154 149
88. 59 32 33 159 154; 60 33 34 80 159; 61 70 89 90 72; 62 89 94 95 90
89. 63 94 99 100 95; 64 99 104 105 100; 65 104 109 110 105; 66 109 114 115 110
90. 67 114 119 120 115; 68 119 124 125 120; 69 124 129 130 125; 70 129 134 135 130
91. 71 134 139 140 135; 72 139 144 145 140; 73 144 149 150 145; 74 149 154 155 150
92. 75 154 159 160 155; 76 159 80 82 160; 77 72 90 91 74; 78 90 95 96 91
93. 79 95 100 101 96; 80 100 105 106 101; 81 105 110 111 106; 82 110 115 116 111
94. 83 115 120 121 116; 84 120 125 126 121; 85 125 130 131 126; 86 130 135 136 131
95. 87 135 140 141 136; 88 140 145 146 141; 89 145 150 151 146; 90 150 155 156 151
96. 91 155 160 161 156; 92 160 82 84 161; 93 74 91 92 76; 94 91 96 97 92
97. 95 96 101 102 97; 96 101 106 107 102; 97 106 111 112 107; 98 111 116 117 112

98. 99 116 121 122 117; 100 121 126 127 122; 101 126 131 132 127
99. 102 131 136 137 132; 103 136 141 142 137; 104 141 146 147 142
100. 105 146 151 152 147; 106 151 156 157 152; 107 156 161 162 157
101. 108 161 84 86 162; 109 76 92 93 78; 110 92 97 98 93; 111 97 102 103 98
102. 112 102 107 108 103; 113 107 112 113 108; 114 112 117 118 113
103. 115 117 122 123 118; 116 122 127 128 123; 117 127 132 133 128
104. 118 132 137 138 133; 119 137 142 143 138; 120 142 147 148 143
105. 121 147 152 153 148; 122 152 157 158 153; 123 157 162 163 158
106. 124 162 86 88 163; 125 78 93 53 52; 126 93 98 54 53; 127 98 103 55 54
107. 128 103 108 56 55; 129 108 113 57 56; 130 113 118 58 57; 131 118 123 59 58
108. 132 123 128 60 59; 133 128 133 61 60; 134 133 138 62 61; 135 138 143 63 62
109. 136 143 148 64 63; 137 148 153 65 64; 138 153 158 66 65; 139 158 163 67 66
110. 140 163 88 68 67
111. MEMBER PROPERTY AMERICAN
112. 141 TO 184 PRIS AX 1E-005 IX 1E-005 IY 1E-005 IZ 1E-005
113. 185 TO 274 PRIS AX 0.228 IX 1E-005 IY 1E-005 IZ 0.0258
114. ELEMENT PROPERTY
115. 1 TO 44 THICKNESS 0.25
116. 45 TO 140 THICKNESS 0.15
117. CONSTANTS
118. E 2.5E+007 MEMB 1 TO 274
119. DENSITY CONCRETE MEMB 1 TO 44
120. DENSITY 1E-005 MEMB 45 TO 274
121. POISSON CONCRETE MEMB 1 TO 274
122. ALPHA CONCRETE MEMB 1 TO 274
123. SUPPORTS
124. 1 TO 17 35 TO 51 69 71 73 75 77 79 81 83 85 87 PINNED
125. MEMBER RELEASE
126. 185 191 197 203 209 215 221 227 233 239 245 251 257 263 269 START MZ
127. 190 196 202 208 214 220 226 232 238 244 250 256 262 268 274 END MZ
128. LOAD 1 DEAD LOAD (DL)
129. SELFWEIGHT Y -1
130. MEMBER LOAD
131. * PARAPET WALL WEIGHT
132. 141 TO 184 UNI GY -2.7
133. ELEMENT LOAD
134. 45 TO 140 PR 8.78
135. LOAD 2 LIVE LOAD (LL)
136. ELEMENT LOAD
137. 45 TO 140 PR 1
138. * THIS RUN CONSIDER ONLY LATERAL LOAD ALONG THE WEAK AXIS
139. LOAD 3 WIND LOAD (WL)
140. ELEMENT LOAD
141. 17 TO 32 PR 1.32
142. LOAD 4 BLAST LOAD (BL)
143. ELEMENT LOAD
144. *FRONT WALL BLAST LOAD
145. 17 TO 32 PR 28.99
146. 140 139 138 137 136 135 134 133 132 131 130 129 128 127 126 125 124 123 122 -
147. 121 120 119 117 116 115 114 112 111 110 109 108 107 105 104 102 101 100 99 97 -
148. 95 93 92 90 88 87 86 85 83 82 80 79 78 77 76 75 74 73 72 71 70 69 68 67 66 65 -
149. 64 63 62 61 60 59 58 57 56 55 54 53 52 51 50 49 48 47 46 45 PR GY -13.547
150. LOAD COMB 5 DL + LL
151. 1 1.0 2 1.0
152. LOAD COMB 6 DL + LL + WL
153. 1 1.0 2 1.0 3 1.0

154. LOAD COMB 7 0.9DL + WL
155. 1 0.9 3 1.0
156. *REFER TO SECT. 6.2.4 OF ASCE REPORT
157. LOAD COMB 8 DL + BL
158. 1 1.0 4 1.0
159. PERFORM ANALYSIS

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

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 163/ 274/ 44
ORIGINAL/FINAL BAND-WIDTH = 127/ 14
TOTAL PRIMARY LOAD CASES = 4, TOTAL DEGREES OF FREEDOM = 978
SIZE OF STIFFNESS MATRIX = 88020 DOUBLE PREC. WORDS
REQRD/AVAIL. DISK SPACE = 13.44/ 880.8 MB, EXMEM = 389.2 MB

++ Processing Element Stiffness Matrix.	17:20:15
++ Processing Global Stiffness Matrix.	17:20:15
++ Processing Triangular Factorization.	17:20:15
++ Calculating Joint Displacements.	17:20:15
++ Calculating Member Forces.	17:20:15

160. LOAD LIST 1 TO 4
161. PRINT SUPPORT REACTION

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	35.97	207.99	21.40	0.00	0.00	0.00
	2	2.50	11.58	0.90	0.00	0.00	0.00
	3	6.49	34.65	10.23	0.00	0.00	0.00
	4	173.89	907.14	236.61	0.00	0.00	0.00
2	1	34.77	184.87	-3.27	0.00	0.00	0.00
	2	4.08	9.21	-0.37	0.00	0.00	0.00
	3	2.23	12.19	-0.05	0.00	0.00	0.00
	4	97.31	382.98	-5.65	0.00	0.00	0.00
3	1	24.95	222.69	7.78	0.00	0.00	0.00
	2	2.67	13.79	0.89	0.00	0.00	0.00
	3	5.31	0.77	0.02	0.00	0.00	0.00
	4	146.47	185.41	10.31	0.00	0.00	0.00
4	1	12.61	252.12	11.83	0.00	0.00	0.00
	2	1.26	17.15	1.35	0.00	0.00	0.00
	3	6.57	-2.94	0.03	0.00	0.00	0.00
	4	156.99	140.94	15.39	0.00	0.00	0.00
5	1	4.16	265.35	12.61	0.00	0.00	0.00
	2	0.34	18.64	1.44	0.00	0.00	0.00
	3	6.07	-3.99	0.03	0.00	0.00	0.00
	4	136.33	133.14	16.28	0.00	0.00	0.00
6	1	0.40	268.99	12.43	0.00	0.00	0.00
	2	-0.05	19.04	1.42	0.00	0.00	0.00
	3	4.82	-4.29	0.03	0.00	0.00	0.00
	4	105.10	130.48	16.02	0.00	0.00	0.00
7	1	-0.60	269.27	12.16	0.00	0.00	0.00
	2	-0.13	19.07	1.38	0.00	0.00	0.00
	3	3.29	-4.39	0.04	0.00	0.00	0.00
	4	72.47	129.83	15.81	0.00	0.00	0.00
8	1	-0.46	268.97	12.04	0.00	0.00	0.00
	2	-0.08	19.03	1.37	0.00	0.00	0.00
	3	1.66	-4.42	0.04	0.00	0.00	0.00
	4	38.42	133.09	16.49	0.00	0.00	0.00
9	1	0.00	268.84	12.01	0.00	0.00	0.00
	2	0.00	19.01	1.37	0.00	0.00	0.00
	3	0.00	-4.43	0.04	0.00	0.00	0.00
	4	1.71	136.84	17.08	0.00	0.00	0.00
10	1	0.46	268.97	12.04	0.00	0.00	0.00
	2	0.08	19.03	1.37	0.00	0.00	0.00
	3	-1.66	-4.42	0.04	0.00	0.00	0.00
	4	-34.96	138.96	17.19	0.00	0.00	0.00
11	1	0.60	269.27	12.16	0.00	0.00	0.00
	2	0.13	19.07	1.38	0.00	0.00	0.00
	3	-3.29	-4.39	0.04	0.00	0.00	0.00
	4	-71.83	139.20	17.20	0.00	0.00	0.00
12	1	-0.40	268.99	12.43	0.00	0.00	0.00
	2	0.05	19.04	1.42	0.00	0.00	0.00
	3	-4.82	-4.29	0.03	0.00	0.00	0.00
	4	-107.68	138.32	17.06	0.00	0.00	0.00

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
13	1	-4.16	265.35	12.61	0.00	0.00	0.00
	2	-0.34	18.64	1.44	0.00	0.00	0.00
	3	-6.07	-3.99	0.03	0.00	0.00	0.00
	4	-139.12	136.63	16.74	0.00	0.00	0.00
14	1	-12.61	252.12	11.83	0.00	0.00	0.00
	2	-1.26	17.15	1.35	0.00	0.00	0.00
	3	-6.57	-2.94	0.03	0.00	0.00	0.00
	4	-159.75	139.45	15.62	0.00	0.00	0.00
15	1	-24.95	222.69	7.78	0.00	0.00	0.00
	2	-2.67	13.79	0.89	0.00	0.00	0.00
	3	-5.31	0.77	0.02	0.00	0.00	0.00
	4	-147.89	180.62	10.06	0.00	0.00	0.00
16	1	-34.77	184.87	-3.27	0.00	0.00	0.00
	2	-4.08	9.21	-0.37	0.00	0.00	0.00
	3	-2.23	12.19	-0.05	0.00	0.00	0.00
	4	-95.36	377.54	-5.44	0.00	0.00	0.00
17	1	-35.97	207.99	21.40	0.00	0.00	0.00
	2	-2.50	11.58	0.90	0.00	0.00	0.00
	3	-6.49	34.65	10.23	0.00	0.00	0.00
	4	-172.17	898.35	236.64	0.00	0.00	0.00
35	1	35.97	207.99	-21.40	0.00	0.00	0.00
	2	2.50	11.58	-0.90	0.00	0.00	0.00
	3	-6.42	-35.27	17.19	0.00	0.00	0.00
	4	-110.48	-630.92	365.26	0.00	0.00	0.00
36	1	34.77	184.87	3.27	0.00	0.00	0.00
	2	4.08	9.21	0.37	0.00	0.00	0.00
	3	-1.80	-13.31	10.69	0.00	0.00	0.00
	4	6.63	-180.56	239.01	0.00	0.00	0.00
37	1	24.95	222.69	-7.78	0.00	0.00	0.00
	2	2.67	13.79	-0.89	0.00	0.00	0.00
	3	-4.76	-0.99	10.74	0.00	0.00	0.00
	4	-76.98	138.80	226.30	0.00	0.00	0.00
38	1	12.61	252.12	-11.83	0.00	0.00	0.00
	2	1.26	17.15	-1.35	0.00	0.00	0.00
	3	-6.32	2.95	10.76	0.00	0.00	0.00
	4	-127.41	257.12	222.11	0.00	0.00	0.00
39	1	4.16	265.35	-12.61	0.00	0.00	0.00
	2	0.34	18.64	-1.44	0.00	0.00	0.00
	3	-5.93	3.95	10.76	0.00	0.00	0.00
	4	-126.64	294.66	221.22	0.00	0.00	0.00
40	1	0.40	268.99	-12.43	0.00	0.00	0.00
	2	-0.05	19.04	-1.42	0.00	0.00	0.00
	3	-4.70	4.18	10.76	0.00	0.00	0.00
	4	-101.84	305.56	221.40	0.00	0.00	0.00
41	1	-0.60	269.27	-12.16	0.00	0.00	0.00
	2	-0.13	19.07	-1.38	0.00	0.00	0.00
	3	-3.20	4.24	10.76	0.00	0.00	0.00
	4	-67.54	310.91	221.51	0.00	0.00	0.00

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
42	1	-0.46	268.97	-12.04	0.00	0.00	0.00
	2	-0.08	19.03	-1.37	0.00	0.00	0.00
	3	-1.61	4.26	10.76	0.00	0.00	0.00
	4	-33.32	317.43	220.92	0.00	0.00	0.00
43	1	0.00	268.84	-12.01	0.00	0.00	0.00
	2	0.00	19.01	-1.37	0.00	0.00	0.00
	3	0.00	4.27	10.77	0.00	0.00	0.00
	4	-0.16	319.01	220.46	0.00	0.00	0.00
44	1	0.46	268.97	-12.04	0.00	0.00	0.00
	2	0.08	19.03	-1.37	0.00	0.00	0.00
	3	1.61	4.26	10.76	0.00	0.00	0.00
	4	35.35	320.52	220.30	0.00	0.00	0.00
45	1	0.60	269.27	-12.16	0.00	0.00	0.00
	2	0.13	19.07	-1.38	0.00	0.00	0.00
	3	3.20	4.24	10.76	0.00	0.00	0.00
	4	70.77	318.94	220.23	0.00	0.00	0.00
46	1	-0.40	268.99	-12.43	0.00	0.00	0.00
	2	0.05	19.04	-1.42	0.00	0.00	0.00
	3	4.70	4.18	10.76	0.00	0.00	0.00
	4	103.42	314.54	220.44	0.00	0.00	0.00
47	1	-4.16	265.35	-12.61	0.00	0.00	0.00
	2	-0.34	18.64	-1.44	0.00	0.00	0.00
	3	5.93	3.95	10.76	0.00	0.00	0.00
	4	126.95	306.18	220.53	0.00	0.00	0.00
48	1	-12.61	252.12	-11.83	0.00	0.00	0.00
	2	-1.26	17.15	-1.35	0.00	0.00	0.00
	3	6.32	2.95	10.76	0.00	0.00	0.00
	4	124.69	269.55	221.21	0.00	0.00	0.00
49	1	-24.95	222.69	-7.78	0.00	0.00	0.00
	2	-2.67	13.79	-0.89	0.00	0.00	0.00
	3	4.76	-0.99	10.74	0.00	0.00	0.00
	4	72.78	144.37	226.15	0.00	0.00	0.00
50	1	-34.77	184.87	3.27	0.00	0.00	0.00
	2	-4.08	9.21	0.37	0.00	0.00	0.00
	3	1.80	-13.31	10.69	0.00	0.00	0.00
	4	-7.69	-180.95	239.18	0.00	0.00	0.00
51	1	-35.97	207.99	-21.40	0.00	0.00	0.00
	2	-2.50	11.58	-0.90	0.00	0.00	0.00
	3	6.42	-35.27	17.19	0.00	0.00	0.00
	4	110.83	-632.85	364.93	0.00	0.00	0.00
69	1	-3.95	143.57	16.21	0.00	0.00	0.00
	2	-0.45	4.53	2.10	0.00	0.00	0.00
	3	-0.05	14.19	10.45	0.00	0.00	0.00
	4	-6.52	370.67	254.13	0.00	0.00	0.00
71	1	6.08	149.15	9.46	0.00	0.00	0.00
	2	0.69	5.47	1.07	0.00	0.00	0.00
	3	-0.01	4.23	14.02	0.00	0.00	0.00
	4	8.16	163.26	320.41	0.00	0.00	0.00

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
73	1	8.70	153.51	0.00	0.00	0.00	0.00
	2	0.99	6.00	0.00	0.00	0.00	0.00
	3	0.00	0.86	15.69	0.00	0.00	0.00
	4	11.83	95.84	344.16	0.00	0.00	0.00
75	1	6.08	149.15	-9.46	0.00	0.00	0.00
	2	0.69	5.47	-1.07	0.00	0.00	0.00
	3	0.00	-2.68	14.14	0.00	0.00	0.00
	4	8.26	11.49	297.33	0.00	0.00	0.00
77	1	-3.95	143.57	-16.21	0.00	0.00	0.00
	2	-0.45	4.53	-2.10	0.00	0.00	0.00
	3	0.08	-14.10	9.11	0.00	0.00	0.00
	4	-3.51	-250.66	175.10	0.00	0.00	0.00
79	1	3.95	143.57	16.21	0.00	0.00	0.00
	2	0.45	4.53	2.10	0.00	0.00	0.00
	3	0.05	14.19	10.45	0.00	0.00	0.00
	4	5.94	369.01	252.77	0.00	0.00	0.00
81	1	-6.08	149.15	9.46	0.00	0.00	0.00
	2	-0.69	5.47	1.07	0.00	0.00	0.00
	3	0.01	4.23	14.02	0.00	0.00	0.00
	4	-7.68	161.79	319.98	0.00	0.00	0.00
83	1	-8.70	153.51	0.00	0.00	0.00	0.00
	2	-0.99	6.00	0.00	0.00	0.00	0.00
	3	0.00	0.86	15.69	0.00	0.00	0.00
	4	-11.16	94.37	344.30	0.00	0.00	0.00
85	1	-6.08	149.15	-9.46	0.00	0.00	0.00
	2	-0.69	5.47	-1.07	0.00	0.00	0.00
	3	0.00	-2.68	14.14	0.00	0.00	0.00
	4	-7.82	10.62	298.06	0.00	0.00	0.00
87	1	3.95	143.57	-16.21	0.00	0.00	0.00
	2	0.45	4.53	-2.10	0.00	0.00	0.00
	3	-0.08	-14.10	9.11	0.00	0.00	0.00
	4	3.22	-250.79	176.60	0.00	0.00	0.00

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

162. FINISH

***** END OF STAAD-III *****

**** DATE= JAN 27,2004 TIME= 17:20:18 ****

 * FOR QUESTIONS REGARDING THIS VERSION OF PROGRAM *
 * RESEARCH ENGINEERS, Inc at *
 * West Coast: Ph- (714) 974-2500 Fax- (714) 921-2543 *
 * East Coast: Ph- (978) 688-3636 Fax- (978) 685-7230 *

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

SUBSTATION # 2

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

STAAD III - ANALYSIS

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

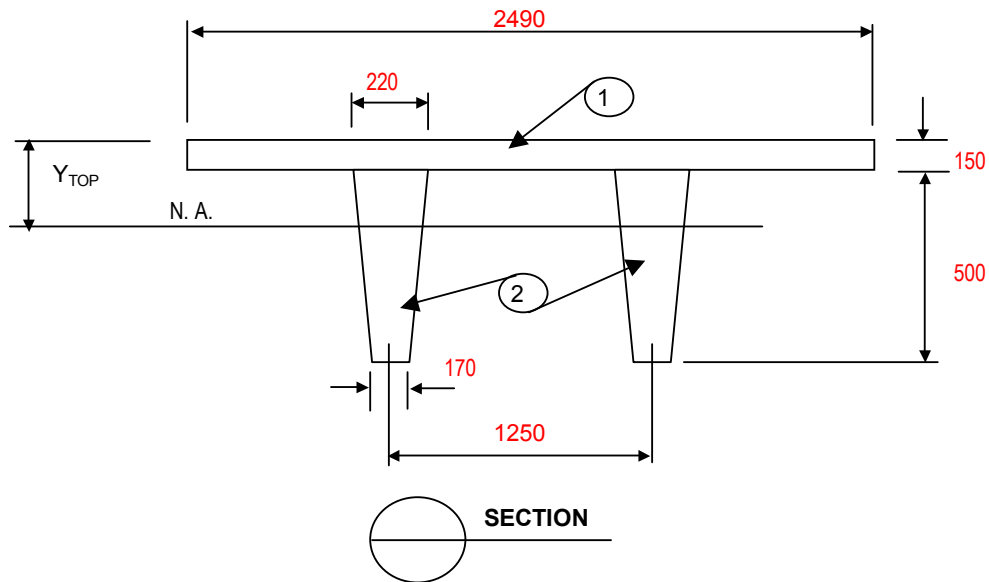
Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

PRECAST ROOF PANEL DOUBLE TEE BEAM CROSS SECTION ; SS # 2

Saudconsult

Job No. :	BI-3039	Sheet no. :	of 3
Project :	REPLACEMENT OF SUBSTATION 472, 321 & F07, ABU ALI	Date :	18/11/03
Subject :	CROSS SECTION PROPERTIES, SUBSTATION # 2	Design by :	MGG
		Check'd by:	SAK

PRECAST ROOF PANEL DOUBLE TEE BEAM CROSS SECTION



ELEMENT	AREA (A)	Y	A x Y	Io	D	A x D ²
1	373500	75.00	28012500	7.0E+08	107.813	4341427273
2	195000	389.32	75916666.67	4.0E+09	206.503	8315503007
TOTAL	568500		103929166.7	4.7E+09		12656930279

$$Y_{TOP} = \frac{103929166.7}{568500}$$

$$= 182.8129581 \text{ MM}$$

$$I_x = 1.7397E+10 \text{ MM}^4$$

$$I_x = 0.0174 \text{ M}^4$$

$$\text{WEIGHT PER 2.5 M WIDTH PANEL} = (\text{TOTAL AREA} \times 23.56 \text{ KN/M}^3) / 2.5 \text{ M}$$

$$= \frac{0.5685 \times 23.56}{2.5}$$

$$= 5.358 \text{ Kpa}$$

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

STAAD III - MODEL



Software licensed to SC

Job Title

Part

Ref

By

Date

Chd

File

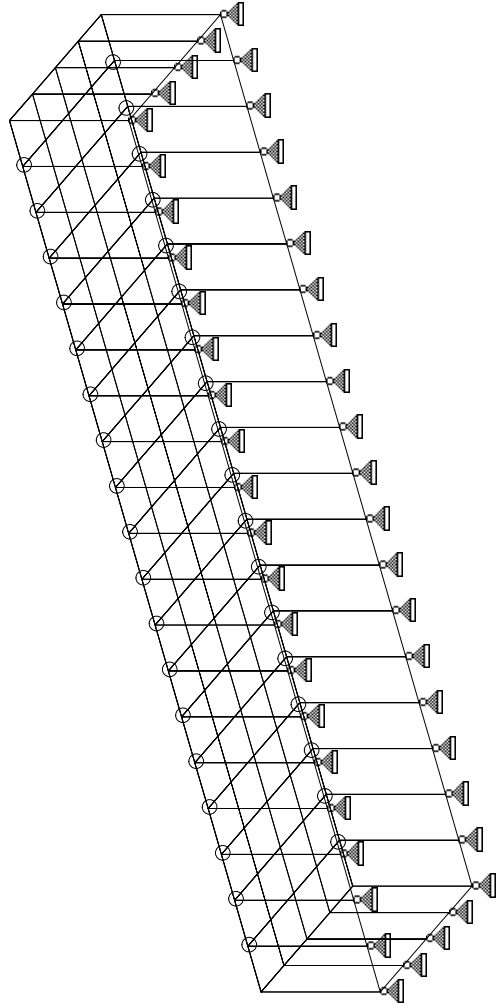
SS2Af.std

Date/Time

02-Feb-2004 15:31

Rev

1



Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

STAAD III - OUTPUT


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*****
*
*          STAAD/Pro STAAD-III
*          Revision 3.1
*          Proprietary Program of
*          RESEARCH ENGINEERS, Inc.
*          Date=    JUN  2, 2004
*          Time=    15:35:33
*
*          USER ID: SC
*****

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1. STAAD SPACE SUBSTATION # 2
2. START JOB INFORMATION
3. ENGINEER DATE 2-JUNE-04
4. *****
5. *****          SUB-STATION NO. 2          *****
6. *****          BI-3039                    *****
7. *****          SAUDI CONSULTING SERVICES  *****
8. *****          SAUDCONSULT                *****
9. *****          CIVIL/STRUCTURAL DEPARTMENT *****
10. *****          DATE: JUNE 2, 2004         *****
11. *****          REVISION 2A               *****
12. *****
13. ** THIS RUN IS CONSIDERED ONLY TO GET THE SUPPORT REACTION
14. ** FOR THE FOUNDATION DESIGN. PRECAST ROOF AND WALL PANELS
15. ** ARE BASE ON THE MANUFACTURERS DATA
16. END JOB INFORMATION
17. INPUT WIDTH 79
18. UNIT METER KN
19. JOINT COORDINATES
20. 1 0 0 0; 2 2.5 0 0; 3 5 0 0; 4 7.5 0 0; 5 10 0 0; 6 12.5 0 0; 7 15 0 0
21. 8 17.5 0 0; 9 20 0 0; 10 22.5 0 0; 11 25 0 0; 12 27.5 0 0; 13 30 0 0
22. 14 32.5 0 0; 15 35 0 0; 16 37.5 0 0; 17 40 0 0; 18 42.5 0 0; 19 45 0 0
23. 20 47.5 0 0; 21 0 0 10; 22 2.5 0 10; 23 5 0 10; 24 7.5 0 10; 25 10 0 10
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25. 32 27.5 0 10; 33 30 0 10; 34 32.5 0 10; 35 35 0 10; 36 37.5 0 10; 37 40 0 10
26. 38 42.5 0 10; 39 45 0 10; 40 47.5 0 10; 41 0 0 2.5; 42 47.5 0 2.5; 43 0 0 5
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35. 82 37.5 6.65 10; 83 40 6.65 10; 84 42.5 6.65 10; 85 45 6.65 10
36. 86 47.5 6.65 10; 87 0 6.65 2.5; 88 47.5 6.65 2.5; 89 0 6.65 5; 90 47.5 6.65 5
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41. 108 15 6.65 2.5; 109 15 6.65 5; 110 15 6.65 7.5; 111 17.5 6.65 2.5

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42. 112 17.5 6.65 5; 113 17.5 6.65 7.5; 114 20 6.65 2.5; 115 20 6.65 5
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51. MEMBER INCIDENCES
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71. ELEMENT INCIDENCES SHELL
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80. 42 91 67 21 45; 43 66 88 42 20; 44 88 90 44 42; 45 90 92 46 44; 46 92 86 40 46
81. 47 47 48 93 87; 48 48 49 96 93; 49 49 50 99 96; 50 50 51 102 99
82. 51 51 52 105 102; 52 52 53 108 105; 53 53 54 111 108; 54 54 55 114 111
83. 55 55 56 117 114; 56 56 57 120 117; 57 57 58 123 120; 58 58 59 126 123
84. 59 59 60 129 126; 60 60 61 132 129; 61 61 62 135 132; 62 62 63 138 135
85. 63 63 64 141 138; 64 64 65 144 141; 65 65 66 88 144; 66 87 93 94 89
86. 67 93 96 97 94; 68 96 99 100 97; 69 99 102 103 100; 70 102 105 106 103
87. 71 105 108 109 106; 72 108 111 112 109; 73 111 114 115 112; 74 114 117 118 115
88. 75 117 120 121 118; 76 120 123 124 121; 77 123 126 127 124; 78 126 129 130 127
89. 79 129 132 133 130; 80 132 135 136 133; 81 135 138 139 136; 82 138 141 142 139
90. 83 141 144 145 142; 84 144 88 90 145; 85 89 94 95 91; 86 94 97 98 95
91. 87 97 100 101 98; 88 100 103 104 101; 89 103 106 107 104; 90 106 109 110 107
92. 91 109 112 113 110; 92 112 115 116 113; 93 115 118 119 116; 94 118 121 122 119
93. 95 121 124 125 122; 96 124 127 128 125; 97 127 130 131 128; 98 130 133 134 131
94. 99 133 136 137 134; 100 136 139 140 137; 101 139 142 143 140
95. 102 142 145 146 143; 103 145 90 92 146; 104 91 95 68 67; 105 95 98 69 68
96. 106 98 101 70 69; 107 101 104 71 70; 108 104 107 72 71; 109 107 110 73 72
97. 110 110 113 74 73; 111 113 116 75 74; 112 116 119 76 75; 113 119 122 77 76

98. 114 122 125 78 77; 115 125 128 79 78; 116 128 131 80 79; 117 131 134 81 80
99. 118 134 137 82 81; 119 137 140 83 82; 120 140 143 84 83; 121 143 146 85 84
100. 122 146 92 86 85
101. MEMBER PROPERTY AMERICAN
102. 123 TO 168 PRIS AX 1E-005 IX 1E-005 IY 1E-005 IZ 1E-005
103. 169 TO 240 PRIS AX 0.195 IX 1E-005 IY 1E-005 IZ 0.0174
104. ELEMENT PROPERTY
105. 1 TO 46 THICKNESS 0.31
106. *250MM STRUCTURAL WALL PANEL + 60MM SKIN
107. 47 TO 122 THICKNESS 0.15
108. CONSTANTS
109. E 2.5E+007 MEMB 1 TO 240
110. DENSITY CONCRETE MEMB 1 TO 46
111. DENSITY 1E-005 MEMB 47 TO 240
112. POISSON CONCRETE MEMB 1 TO 240
113. ALPHA CONCRETE MEMB 1 TO 240
114. SUPPORTS
115. 1 TO 46 PINNED
116. MEMBER RELEASE
117. 237 233 229 225 221 217 213 209 205 201 197 193 189 185 181 177 173 -
118. 169 START MZ
119. 172 176 180 184 188 192 196 200 204 208 212 216 220 224 228 232 236 240 END MZ
120. LOAD 1 DEAD LOAD (DL)
121. SELFWEIGHT Y -1
122. MEMBER LOAD
123. * PARAPET WALL WEIGHT
124. 123 TO 168 UNI GY -4.752
125. ELEMENT LOAD
126. 47 TO 122 PR 8.21
127. LOAD 2 LIVE LOAD (LL)
128. ELEMENT LOAD
129. 47 TO 122 PR 1
130. * THIS RUN CONSIDER ONLY LATERAL LOAD ALONG THE WEAK AXIS
131. LOAD 3 WIND LOAD (WL)
132. ELEMENT LOAD
133. 20 TO 38 PR 1.32
134. LOAD 4 BLAST LOAD (BL)
135. ELEMENT LOAD
136. *FRONT WALL BLAST LOAD
137. 20 TO 38 PR 28.99
138. *ROOF SLAB BLAST LOAD
139. 47 TO 76 78 TO 93 95 TO 122 PR GY -13.457
140. LOAD COMB 5 DL + LL
141. 1 1.0 2 1.0
142. LOAD COMB 6 DL + LL + WL
143. 1 1.0 2 1.0 3 1.0
144. LOAD COMB 7 0.9DL + WL
145. 1 0.9 3 1.0
146. *REFER TO SECT. 6.2.4 OF ASCE REPORT
147. LOAD COMB 8 DL + BL
148. 1 1.0 4 1.0
149. PERFORM ANALYSIS

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

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 146/ 240/ 46
ORIGINAL/FINAL BAND-WIDTH = 86/ 10
TOTAL PRIMARY LOAD CASES = 4, TOTAL DEGREES OF FREEDOM = 876
SIZE OF STIFFNESS MATRIX = 57816 DOUBLE PREC. WORDS
REQRD/AVAIL. DISK SPACE = 13.15/ 3746.8 MB, EXMEM = 311.4 MB

++ Processing Element Stiffness Matrix.	15:35:34
++ Processing Global Stiffness Matrix.	15:35:34
++ Processing Triangular Factorization.	15:35:34
++ Calculating Joint Displacements.	15:35:34
++ Calculating Member Forces.	15:35:34

150. LOAD LIST 1 TO 4

151. PRINT SUPPORT REACTION

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	35.29	210.24	24.18	0.00	0.00	0.00
	2	1.99	8.73	0.82	0.00	0.00	0.00
	3	14.17	73.50	21.28	0.00	0.00	0.00
	4	337.97	1731.73	478.43	0.00	0.00	0.00
2	1	19.27	193.09	-1.77	0.00	0.00	0.00
	2	2.45	7.03	-0.21	0.00	0.00	0.00
	3	4.81	26.57	-0.18	0.00	0.00	0.00
	4	138.63	678.10	-6.68	0.00	0.00	0.00
3	1	16.16	212.87	3.54	0.00	0.00	0.00
	2	1.66	9.85	0.43	0.00	0.00	0.00
	3	11.28	2.39	0.05	0.00	0.00	0.00
	4	270.22	185.05	6.96	0.00	0.00	0.00
4	1	8.01	229.16	5.81	0.00	0.00	0.00
	2	0.66	11.86	0.71	0.00	0.00	0.00
	3	14.53	-5.23	0.08	0.00	0.00	0.00
	4	328.15	44.69	11.27	0.00	0.00	0.00
5	1	2.90	234.58	5.92	0.00	0.00	0.00
	2	0.11	12.49	0.72	0.00	0.00	0.00
	3	14.08	-7.03	0.08	0.00	0.00	0.00
	4	310.78	13.65	11.56	0.00	0.00	0.00
6	1	1.01	235.74	5.78	0.00	0.00	0.00
	2	-0.06	12.60	0.70	0.00	0.00	0.00
	3	12.05	-7.27	0.09	0.00	0.00	0.00
	4	263.69	9.77	11.48	0.00	0.00	0.00
7	1	0.35	235.99	5.68	0.00	0.00	0.00
	2	-0.09	12.61	0.69	0.00	0.00	0.00
	3	9.50	-7.22	0.10	0.00	0.00	0.00
	4	206.54	10.39	11.45	0.00	0.00	0.00
8	1	0.11	236.07	5.65	0.00	0.00	0.00
	2	-0.07	12.60	0.69	0.00	0.00	0.00
	3	6.80	-7.15	0.10	0.00	0.00	0.00
	4	146.49	9.31	11.20	0.00	0.00	0.00
9	1	0.02	236.10	5.64	0.00	0.00	0.00
	2	-0.05	12.60	0.69	0.00	0.00	0.00
	3	4.08	-7.11	0.10	0.00	0.00	0.00
	4	86.60	6.71	10.91	0.00	0.00	0.00
10	1	0.00	236.11	5.64	0.00	0.00	0.00
	2	-0.02	12.59	0.69	0.00	0.00	0.00
	3	1.36	-7.10	0.10	0.00	0.00	0.00
	4	26.66	2.49	10.57	0.00	0.00	0.00
11	1	0.00	236.11	5.64	0.00	0.00	0.00
	2	0.02	12.59	0.69	0.00	0.00	0.00
	3	-1.36	-7.10	0.10	0.00	0.00	0.00
	4	-31.70	-2.70	10.16	0.00	0.00	0.00
12	1	-0.02	236.10	5.64	0.00	0.00	0.00
	2	0.05	12.60	0.69	0.00	0.00	0.00
	3	-4.08	-7.11	0.10	0.00	0.00	0.00
	4	-88.24	-2.76	10.16	0.00	0.00	0.00

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
13	1	-0.11	236.07	5.65	0.00	0.00	0.00
	2	0.07	12.60	0.69	0.00	0.00	0.00
	3	-6.80	-7.15	0.10	0.00	0.00	0.00
	4	-146.31	-1.19	10.44	0.00	0.00	0.00
14	1	-0.35	235.99	5.68	0.00	0.00	0.00
	2	0.09	12.61	0.69	0.00	0.00	0.00
	3	-9.50	-7.22	0.10	0.00	0.00	0.00
	4	-204.28	0.29	10.64	0.00	0.00	0.00
15	1	-1.01	235.74	5.78	0.00	0.00	0.00
	2	0.06	12.60	0.70	0.00	0.00	0.00
	3	-12.05	-7.27	0.09	0.00	0.00	0.00
	4	-260.89	4.85	11.10	0.00	0.00	0.00
16	1	-2.90	234.58	5.92	0.00	0.00	0.00
	2	-0.11	12.49	0.72	0.00	0.00	0.00
	3	-14.08	-7.03	0.08	0.00	0.00	0.00
	4	-309.56	12.41	11.60	0.00	0.00	0.00
17	1	-8.01	229.16	5.81	0.00	0.00	0.00
	2	-0.66	11.86	0.71	0.00	0.00	0.00
	3	-14.53	-5.23	0.08	0.00	0.00	0.00
	4	-327.85	44.49	11.32	0.00	0.00	0.00
18	1	-16.16	212.87	3.54	0.00	0.00	0.00
	2	-1.66	9.85	0.43	0.00	0.00	0.00
	3	-11.28	2.39	0.05	0.00	0.00	0.00
	4	-270.19	185.06	6.98	0.00	0.00	0.00
19	1	-19.27	193.09	-1.77	0.00	0.00	0.00
	2	-2.45	7.03	-0.21	0.00	0.00	0.00
	3	-4.81	26.57	-0.18	0.00	0.00	0.00
	4	-138.65	678.20	-6.67	0.00	0.00	0.00
20	1	-35.29	210.24	24.18	0.00	0.00	0.00
	2	-1.99	8.73	0.82	0.00	0.00	0.00
	3	-14.17	73.50	21.28	0.00	0.00	0.00
	4	-338.01	1731.93	478.46	0.00	0.00	0.00
21	1	35.29	210.24	-24.18	0.00	0.00	0.00
	2	1.99	8.73	-0.82	0.00	0.00	0.00
	3	-14.07	-73.09	28.11	0.00	0.00	0.00
	4	-282.32	-1487.50	606.27	0.00	0.00	0.00
22	1	19.27	193.09	1.77	0.00	0.00	0.00
	2	2.45	7.03	0.21	0.00	0.00	0.00
	3	-4.41	-27.44	10.82	0.00	0.00	0.00
	4	-63.84	-508.07	240.44	0.00	0.00	0.00
23	1	16.16	212.87	-3.54	0.00	0.00	0.00
	2	1.66	9.85	-0.43	0.00	0.00	0.00
	3	-10.73	-2.64	11.01	0.00	0.00	0.00
	4	-213.31	74.72	236.05	0.00	0.00	0.00
24	1	8.01	229.16	-5.81	0.00	0.00	0.00
	2	0.66	11.86	-0.71	0.00	0.00	0.00
	3	-14.26	5.15	11.05	0.00	0.00	0.00
	4	-304.33	272.66	233.23	0.00	0.00	0.00

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
25	1	2.90	234.58	-5.92	0.00	0.00	0.00
	2	0.11	12.49	-0.72	0.00	0.00	0.00
	3	-13.91	6.93	11.06	0.00	0.00	0.00
	4	-304.02	320.24	233.14	0.00	0.00	0.00
26	1	1.01	235.74	-5.78	0.00	0.00	0.00
	2	-0.06	12.60	-0.70	0.00	0.00	0.00
	3	-11.90	7.15	11.06	0.00	0.00	0.00
	4	-262.45	326.52	233.46	0.00	0.00	0.00
27	1	0.35	235.99	-5.68	0.00	0.00	0.00
	2	-0.09	12.61	-0.69	0.00	0.00	0.00
	3	-9.37	7.10	11.07	0.00	0.00	0.00
	4	-208.23	324.45	233.75	0.00	0.00	0.00
28	1	0.11	236.07	-5.65	0.00	0.00	0.00
	2	-0.07	12.60	-0.69	0.00	0.00	0.00
	3	-6.71	7.05	11.07	0.00	0.00	0.00
	4	-151.37	319.42	234.29	0.00	0.00	0.00
29	1	0.02	236.10	-5.64	0.00	0.00	0.00
	2	-0.05	12.60	-0.69	0.00	0.00	0.00
	3	-4.03	7.02	11.08	0.00	0.00	0.00
	4	-92.18	312.85	234.81	0.00	0.00	0.00
30	1	0.00	236.11	-5.64	0.00	0.00	0.00
	2	-0.02	12.59	-0.69	0.00	0.00	0.00
	3	-1.34	7.01	11.08	0.00	0.00	0.00
	4	-31.78	309.74	235.11	0.00	0.00	0.00
31	1	0.00	236.11	-5.64	0.00	0.00	0.00
	2	0.02	12.59	-0.69	0.00	0.00	0.00
	3	1.34	7.01	11.08	0.00	0.00	0.00
	4	28.91	307.44	235.44	0.00	0.00	0.00
32	1	-0.02	236.10	-5.64	0.00	0.00	0.00
	2	0.05	12.60	-0.69	0.00	0.00	0.00
	3	4.03	7.02	11.08	0.00	0.00	0.00
	4	91.11	307.47	235.43	0.00	0.00	0.00
33	1	-0.11	236.07	-5.65	0.00	0.00	0.00
	2	0.07	12.60	-0.69	0.00	0.00	0.00
	3	6.71	7.05	11.07	0.00	0.00	0.00
	4	151.35	313.33	234.94	0.00	0.00	0.00
34	1	-0.35	235.99	-5.68	0.00	0.00	0.00
	2	0.09	12.61	-0.69	0.00	0.00	0.00
	3	9.37	7.10	11.07	0.00	0.00	0.00
	4	209.39	319.08	234.41	0.00	0.00	0.00
35	1	-1.01	235.74	-5.78	0.00	0.00	0.00
	2	0.06	12.60	-0.70	0.00	0.00	0.00
	3	11.90	7.15	11.06	0.00	0.00	0.00
	4	264.07	323.42	233.79	0.00	0.00	0.00
36	1	-2.90	234.58	-5.92	0.00	0.00	0.00
	2	-0.11	12.49	-0.72	0.00	0.00	0.00
	3	13.91	6.93	11.06	0.00	0.00	0.00
	4	304.84	319.40	233.12	0.00	0.00	0.00

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
37	1	-8.01	229.16	-5.81	0.00	0.00	0.00
	2	-0.66	11.86	-0.71	0.00	0.00	0.00
	3	14.26	5.15	11.05	0.00	0.00	0.00
	4	304.56	272.52	233.18	0.00	0.00	0.00
38	1	-16.16	212.87	-3.54	0.00	0.00	0.00
	2	-1.66	9.85	-0.43	0.00	0.00	0.00
	3	10.73	-2.64	11.01	0.00	0.00	0.00
	4	213.36	74.70	236.03	0.00	0.00	0.00
39	1	-19.27	193.09	1.77	0.00	0.00	0.00
	2	-2.45	7.03	0.21	0.00	0.00	0.00
	3	4.41	-27.44	10.82	0.00	0.00	0.00
	4	63.85	-508.06	240.44	0.00	0.00	0.00
40	1	-35.29	210.24	-24.18	0.00	0.00	0.00
	2	-1.99	8.73	-0.82	0.00	0.00	0.00
	3	14.07	-73.09	28.11	0.00	0.00	0.00
	4	282.33	-1487.51	606.28	0.00	0.00	0.00
41	1	-2.00	166.56	2.40	0.00	0.00	0.00
	2	-0.24	3.88	0.77	0.00	0.00	0.00
	3	-0.14	28.82	19.52	0.00	0.00	0.00
	4	-6.42	685.07	439.06	0.00	0.00	0.00
42	1	2.00	166.56	2.40	0.00	0.00	0.00
	2	0.24	3.88	0.77	0.00	0.00	0.00
	3	0.14	28.82	19.52	0.00	0.00	0.00
	4	6.43	685.14	439.05	0.00	0.00	0.00
43	1	0.85	162.98	0.00	0.00	0.00	0.00
	2	0.11	3.85	0.00	0.00	0.00	0.00
	3	-0.01	0.87	22.07	0.00	0.00	0.00
	4	1.19	70.96	484.64	0.00	0.00	0.00
44	1	-0.85	162.98	0.00	0.00	0.00	0.00
	2	-0.11	3.85	0.00	0.00	0.00	0.00
	3	0.01	0.87	22.07	0.00	0.00	0.00
	4	-1.19	70.95	484.66	0.00	0.00	0.00
45	1	-2.00	166.56	-2.40	0.00	0.00	0.00
	2	-0.24	3.88	-0.77	0.00	0.00	0.00
	3	0.19	-28.28	17.66	0.00	0.00	0.00
	4	0.78	-568.77	377.39	0.00	0.00	0.00
46	1	2.00	166.56	-2.40	0.00	0.00	0.00
	2	0.24	3.88	-0.77	0.00	0.00	0.00
	3	-0.19	-28.28	17.66	0.00	0.00	0.00
	4	-0.78	-568.77	377.43	0.00	0.00	0.00

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

152. FINISH

***** END OF STAAD-III *****

**** DATE= JUN 2,2004 TIME= 15:35:37 ****

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*****
*   FOR QUESTIONS REGARDING THIS VERSION OF PROGRAM   *
*               RESEARCH ENGINEERS, Inc at           *
* West Coast: Ph- (714) 974-2500 Fax- (714) 921-2543  *
* East Coast: Ph- (978) 688-3636 Fax- (978) 685-7230  *
*****
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Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

DYNAMIC ANALYSIS (NUMERICAL INTEGRATION)

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

REFLECTED WALL PANELS TYPICAL FOR SS # 1 AND SS # 2

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

TYPICAL SOLID WALL PANELS

Slenderness

Approximate Evaluation of Slenderness Effect

For Compression members braced against sidesway, the effect of slenderness may be neglected when,

$$\frac{kl_u}{r} < 34$$

M_{1b} = value of smaller end moment on compression members due to gravity loads

M_{2b} = value of larger end moment on compression members due to gravity loads

$$DL = \text{Roof DL} \times \text{unit width} \times 1/2 \text{ roof span} = 241.84 \text{ lbs}$$

$$e = 6.5 \text{ in}$$

$$\text{Moment} = DL \times e = 1571.94 \text{ lb-in}$$

where :

l_u = unsupported length of compression members

$$= 6630.000 \text{ mm}$$

$$= 261.024 \text{ in}$$

k_{LONG} = effective length factor

$$= 1$$

$$thk = 9.843 \text{ in}$$

$$w = 12 \text{ in}$$

$$I_g = 953.49600 \text{ in}^4$$

$$A_g = 118.11024 \text{ in}^2$$

r = radius of gyration

$$= \sqrt{I / A}$$

$$= 2.84129069$$

$$M_1 = 1571.94 \text{ lb-in}$$

$$M_2 = 1571.94 \text{ lb-in}$$

$$\frac{kl_u}{r} = 91.868$$

$$\frac{kl_u}{r} > 34$$

Therefore, consider slenderness effect.

Magnified design moment

$$M_{slender} = \delta M_u$$

where :

$$\delta_b \frac{C_m}{1 - \frac{P_u}{\phi P_c}} \leq 1.0$$

For members braced against sidesway and without transverse loads between supports, C_m may be taken as

$$C_m = 0.60 + 0.40 (M_{1b} / M_{2b})$$

but not less than 0.40

for this case use $C_m = 1.0$, for all other cases

then, $C_m = 1.00$

$$P_u = 1.0 \cdot P_D$$

$$P_u = 242 \text{ lb} \quad \text{factored axial load}$$

$$\phi = 0.700 \quad \text{strength reduction factor}$$

$$P_c = \frac{\pi^2 EI}{(k l_u)^2} \quad \text{Euler's buckling load}$$

$$E_c = 4032041 \text{ psi}$$

$$I_g = 953.496 \text{ in}^4$$

β_d = absolute value of ratio of maximum deadload moment to maximum total moment, always positive

$$\frac{M_{\max DL}}{M_{\max TOT}} = \frac{1571.94}{1572}$$

$$= 1.000$$

$$\text{Max. Mom} = 1572 \text{ KN-m}$$

$$EI = \frac{\frac{E_c I_g}{2.5}}{1 + \beta_d}$$

$$768906324.8$$

$$\text{then, } P_c = \frac{\pi^2 EI}{(k l_u)^2}$$

$$= 111381.6264$$

$$\delta_b \frac{C_m}{1 - \frac{P_u}{\phi P_c}} \geq 1. \quad 1.0031$$

$$\delta_{Mu} = 1576.828 \quad \text{lb-in}$$

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject : REFLECTED WALL PROPERTIES

Sheet no. : of
Date : 8-Jun-04
Design by : MGG
Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Roof Dead Load = 8.470 KN/m²
= 176.90 psf

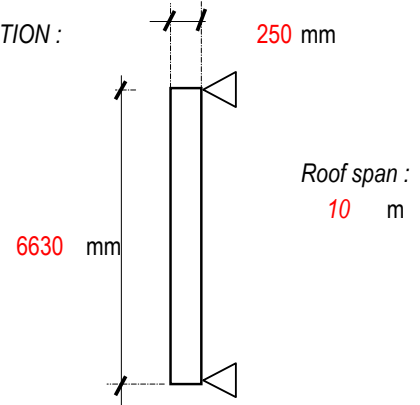
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
Reinf., f_y = 414 Mpa

PANEL DATA :

Panel Width, width = 2500 mm
Long Span, l_b = 6630 mm
Wall, t_{TRIAL} = 250 mm

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure,	P_s =	4.20	psi	(SEE LOADING CALCULATION)
Effective blast time duration,	T_e =	129	msec	
Consider 1ft. width slab:		1.0000	ft.	
Moment of Inertia,	I_g =	953.50	in ⁴	(396875000 mm ⁴)
Area of Bar ($\emptyset$), 25 mm $\emptyset$	A_b =	0.7609	in ²	(491 mm ²)
Rebar Spacing for top & bottom,	S =	12.0000	in	(305 mm)
Area of Rebar PER 1ft. unit width,	A_s =	0.7609	in ²	(491 mm ²)
Effective Slab Depth,	$d_c = d_s$ =	8.3425	in	(212 mm)
$f_{dc} = (SIF) \times (DIF) \times f'_m$ (for bending)		5954.3	psi	(41 Mpa.)
$f_{dc} = (SIF) \times (DIF) \times f'_m$ (for shear)		5504.0	psi	(38 Mpa.)
$f_{dy} = (SIF) \times (DIF) \times f_y$		77276	psi	(533 Mpa.)
Modulus of Elasticity,	$E_{dc} = 57000 \times (f'_c)^{1/2}$	4032041	psi	(27801 Mpa.)
Steel ratio,	$p = A_s / b \times d$	0.0076	>	$200 / f_{dy} = 0.003$
Stress block,	$a = A_s \times f_{dy} / 0.85 \times f_{dc} \times b$	0.9681	in	(25 mm)
Modular ratio,	$n = E_s / E_{dm}$	7.1940		

a.) BENDING RESISTANCE

Mgnified Moment = M (slender)		1576.828	lb-in	(178 KN-m)
Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$		462046.175	lb-in	(52202 KN-m)
Net Moment, = $M_p - M$ (slender)		460469.346	lb-in	(52023 KN-m)
Resistance at cracked moment,	$R_b = 8 \times M_p / L$	14112.73	lb	(62.77 KN)
Unit Resistance,	$r_b = R_b / L \times \text{unit width}$	4.5056	psi	(0.0311 Mpa.)

b.) SHEAR RESISTANCE

Shear,	$V_n = 2 \times b \times d \times (f_{dc})^{0.5}$	14854.1	lb	(66 KN)
At Support,	$R_s = V_n \times L / (0.5 \times L - t_w)$	31737	lb	(141 KN)
Unit Resistance,	$r_s = R_s / L \times \text{unit width}$	10.132	psi	(0.0699 Mpa.)

$\therefore$ Dynamic Resistance Controlled by Bending, R_u = 4.5056 psi > 4.20 $\therefore$ O.K!
14.1127 kips 13.16 kips

Cracked Moment of Inertia:

$C = -n \times A_s + [n \times A_s \times (n \times A_s + 2 \times b \times d)]^{0.5} / b$	$n \times A_s$ =	5.4736	in ²	(3531 mm ²)
$I_{cr} = b \times C^3 / 3 + n \times A_s \times (d - C)^2$		2.3401	in ²	(1510 mm ²)
		248.47	in ⁴	(103419712 mm ⁴)

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Average Moment of Inertia,	$I_a = (I_g + I_{cr}) / 2$	=	600.981	in ⁴	(250147356 mm ⁴)
Effective Stiffness,	$K = 384 \times E_{dc} \times I_a / 5 L^3$	=	10.4642	klps / in	
Yield Deflection,	$Y_e = R_u / K$	=	1.34866	in	
Wall Mass = $\gamma_c \times t_w \times \text{unit width} \times \text{span} / \text{gravity acc.}$		=	0.00693	kip-sec ² / in	
Roof Mass = Roof DL x unit width x 1/2 roof span / gravity acc.		=	0.00752	kip-sec ² / in	
Total Mass, M		=	0.01445	kip-sec ² / in	
Average Stiffness,	K_{LM}	=	0.72000	FROM ASCE TABLE 6.1	
Equivalent Mass,	$M_e = K_{LM} \times \text{Beam Mass}$	=	0.01040	kip-sec / in	
Period of Vibration,	$t_n = 2 \times \pi \times (M_e / K)^{0.5}$	=	0.19813	sec	
Time increment , $t_n/10$	Δt	=	0.01981	sec	; use $\Delta t = 0.00200$ sec
t_d / t_n		=	0.65252		
$R_u / P_s =$		=	1.07276		

NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	10.464 kips / in
Mass	M	=	0.0145 kip-sec ² / in
Tension resistance	Rut	=	14.11 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	13.16 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.1293 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load	
load (kip)	time (sec)
13.16	0
0	0.1293

Length	266.732 in
y _{max} =	1.608 in
r _{max} =	14.113 kips
μ =	1.193
θ =	0.691 degrees
Rut/Fo =	1.073
θ = < 2 degrees	
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.000	13.16	0.000	0.000	910.3373	0.00	1.51
0.002	12.95	0.002	1.805	894.9434	0.02	1.50
0.004	12.75	0.007	3.577	876.9587	0.08	1.50
0.006	12.55	0.016	5.311	856.4351	0.17	1.51
0.008	12.34	0.028	7.000	833.4321	0.30	1.53
0.010	12.14	0.044	8.642	808.0164	0.46	1.57
0.012	11.93	0.063	10.230	780.2614	0.66	1.63
0.014	11.73	0.085	11.761	750.2476	0.89	1.69
0.016	11.53	0.110	13.229	718.0618	1.15	1.77
0.018	11.32	0.138	14.631	683.7972	1.44	1.86
0.020	11.12	0.168	15.962	647.5530	1.76	1.96
0.022	10.92	0.202	17.219	609.4341	2.11	2.07
0.024	10.71	0.237	18.398	569.5509	2.48	2.19
0.026	10.51	0.275	19.496	528.0188	2.88	2.32
0.028	10.31	0.315	20.509	484.9582	3.30	2.45
0.030	10.10	0.357	21.434	440.4935	3.74	2.60
0.032	9.90	0.401	22.269	394.7537	4.19	2.75
0.034	9.70	0.446	23.012	347.8710	4.67	2.91
0.036	9.49	0.493	23.660	299.9812	5.16	3.08
0.038	9.29	0.541	24.211	251.2230	5.66	3.25
0.040	9.09	0.590	24.664	201.7375	6.17	3.42
0.042	8.88	0.639	25.017	151.6679	6.69	3.60
0.044	8.68	0.690	25.270	101.1593	7.22	3.78
0.046	8.47	0.740	25.422	50.3578	7.75	3.96
0.048	8.27	0.791	25.472	-0.5894	8.28	4.14
0.050	8.07	0.842	25.419	-51.5350	8.81	4.32
0.052	7.86	0.893	25.266	-102.3314	9.34	4.50
0.054	7.66	0.943	25.010	-152.8315	9.87	4.68
0.056	7.46	0.993	24.655	-202.8892	10.39	4.86
0.058	7.25	1.042	24.199	-252.3595	10.90	5.03
0.060	7.05	1.090	23.646	-301.0992	11.40	5.20
0.062	6.85	1.136	22.996	-348.9672	11.89	5.36

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.064	6.64	1.181	22.251	-395.8250	12.36	5.52
0.066	6.44	1.225	21.414	-441.5368	12.82	5.68
0.068	6.24	1.267	20.486	-485.9704	13.26	5.82
0.070	6.03	1.307	19.471	-528.9972	13.68	5.96
0.072	5.83	1.345	18.372	-570.4924	14.07	6.09
0.074	5.63	1.380	17.191	-587.3016	14.11	6.08
0.076	5.42	1.414	16.002	-601.3845	14.11	6.06
0.078	5.22	1.444	14.785	-615.4675	14.11	6.03
0.080	5.01	1.473	13.540	-629.5505	14.11	6.01
0.082	4.81	1.499	12.267	-643.6335	14.11	5.99
0.084	4.61	1.522	10.966	-657.7164	14.11	5.96
0.086	4.40	1.542	9.636	-671.7994	14.11	5.94
0.088	4.20	1.560	8.279	-685.8824	14.11	5.92
0.090	4.00	1.576	6.893	-699.9654	14.11	5.89
0.092	3.79	1.588	5.479	-714.0483	14.11	5.87
0.094	3.59	1.597	4.037	-728.1313	14.11	5.85
0.096	3.39	1.604	2.566	-742.2143	14.11	5.82
0.098	3.18	1.608	1.068	-756.2972	14.11	5.80
0.100	2.98	1.608	-0.459	-770.3802	14.11	5.78
0.102	2.78	1.606	-2.012	-782.6770	14.09	5.74
0.104	2.57	1.600	-3.587	-792.7080	14.03	5.70
0.106	2.37	1.591	-5.180	-800.4440	13.94	5.64
0.108	2.17	1.580	-6.787	-805.8628	13.81	5.57
0.110	1.96	1.564	-8.402	-808.9486	13.65	5.48
0.112	1.76	1.546	-10.020	-809.6924	13.46	5.38
0.114	1.56	1.524	-11.638	-808.0922	13.23	5.27
0.116	1.35	1.499	-13.250	-804.1526	12.97	5.15
0.118	1.15	1.471	-14.852	-797.8849	12.68	5.01
0.120	0.94	1.440	-16.439	-789.3074	12.35	4.86
0.122	0.74	1.406	-18.007	-778.4448	11.99	4.70
0.124	0.54	1.368	-19.551	-765.3286	11.60	4.53
0.126	0.33	1.327	-21.066	-749.9967	11.17	4.34
0.128	0.13	1.284	-22.549	-732.4936	10.72	4.14
0.130	0.00	1.237	-23.989	-707.8165	10.23	3.94
0.132	0.00	1.188	-25.369	-672.0675	9.71	3.74
0.134	0.00	1.136	-26.675	-634.3729	9.17	3.53
0.136	0.00	1.081	-27.905	-594.8418	8.60	3.31
0.138	0.00	1.024	-29.053	-553.5886	8.00	3.08
0.140	0.00	0.965	-30.117	-510.7327	7.38	2.84
0.142	0.00	0.904	-31.095	-466.3983	6.74	2.59
0.144	0.00	0.841	-31.982	-420.7136	6.08	2.34
0.146	0.00	0.776	-32.776	-373.8110	5.40	2.08
0.148	0.00	0.710	-33.476	-325.8261	4.71	1.81
0.150	0.00	0.642	-34.079	-276.8980	4.00	1.54
0.152	0.00	0.573	-34.583	-227.1683	3.28	1.26
0.154	0.00	0.504	-34.987	-176.7809	2.55	0.98
0.156	0.00	0.433	-35.289	-125.8818	1.82	0.70
0.158	0.00	0.363	-35.490	-74.6182	1.08	0.42
0.160	0.00	0.292	-35.588	-23.1386	0.33	0.13
0.162	0.00	0.220	-35.582	28.4080	-0.41	-0.16
0.164	0.00	0.149	-35.474	79.8723	-1.15	-0.44
0.166	0.00	0.079	-35.263	131.1054	-1.89	-0.73

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.168	0.00	0.008	-34.950	181.9590	-2.63	-1.01
0.170	0.00	-0.061	-34.536	232.2858	-3.36	-1.29
0.172	0.00	-0.130	-34.021	281.9401	-4.07	-1.57
0.174	0.00	-0.197	-33.409	330.7782	-4.78	-1.84
0.176	0.00	-0.263	-32.699	378.6587	-5.47	-2.11
0.178	0.00	-0.328	-31.895	425.4430	-6.15	-2.37
0.180	0.00	-0.391	-30.999	470.9956	-6.81	-2.62
0.182	0.00	-0.452	-30.013	515.1847	-7.45	-2.87
0.184	0.00	-0.511	-28.939	557.8823	-8.06	-3.10
0.186	0.00	-0.568	-27.783	598.9648	-8.66	-3.33
0.188	0.00	-0.622	-26.545	638.3134	-9.22	-3.55
0.190	0.00	-0.674	-25.231	675.8140	-9.77	-3.76
0.192	0.00	-0.723	-23.844	711.3581	-10.28	-3.96
0.194	0.00	-0.769	-22.388	744.8428	-10.76	-4.14
0.196	0.00	-0.812	-20.867	776.1712	-11.22	-4.32
0.198	0.00	-0.852	-19.285	805.2526	-11.64	-4.48
0.200	0.00	-0.889	-17.648	832.0027	-12.02	-4.63
0.202	0.00	-0.923	-15.960	856.3442	-12.38	-4.76
0.204	0.00	-0.953	-14.225	878.2066	-12.69	-4.89
0.206	0.00	-0.980	-12.450	897.5266	-12.97	-4.99
0.208	0.00	-1.003	-10.638	914.2482	-13.21	-5.09
0.210	0.00	-1.022	-8.795	928.3230	-13.42	-5.16
0.212	0.00	-1.038	-6.927	939.7104	-13.58	-5.23
0.214	0.00	-1.050	-5.039	948.3773	-13.71	-5.28
0.216	0.00	-1.058	-3.136	954.2986	-13.79	-5.31
0.218	0.00	-1.063	-1.225	957.4572	-13.84	-5.33
0.220	0.00	-1.063	0.691	957.8439	-13.84	-5.33
0.222	0.00	-1.060	2.604	955.4577	-13.81	-5.32
0.224	0.00	-1.053	4.510	950.3055	-13.73	-5.29
0.226	0.00	-1.042	6.402	942.4020	-13.62	-5.24
0.228	0.00	-1.027	8.277	931.7704	-13.47	-5.18
0.230	0.00	-1.009	10.127	918.4412	-13.27	-5.11
0.232	0.00	-0.987	11.948	902.4532	-13.04	-5.02
0.234	0.00	-0.961	13.734	883.8525	-12.77	-4.92
0.236	0.00	-0.932	15.481	862.6930	-12.47	-4.80
0.238	0.00	-0.899	17.182	839.0361	-12.13	-4.67
0.240	0.00	-0.863	18.834	812.9502	-11.75	-4.52
0.242	0.00	-0.824	20.432	784.5107	-11.34	-4.36
0.244	0.00	-0.781	21.970	753.8001	-10.89	-4.19
0.246	0.00	-0.736	23.445	720.9072	-10.42	-4.01
0.248	0.00	-0.688	24.852	685.9273	-9.91	-3.82
0.250	0.00	-0.637	26.186	648.9616	-9.38	-3.61
0.252	0.00	-0.583	27.446	610.1172	-8.82	-3.39
0.254	0.00	-0.527	28.625	569.5064	-8.23	-3.17
0.256	0.00	-0.468	29.722	527.2470	-7.62	-2.93
0.258	0.00	-0.408	30.733	483.4611	-6.99	-2.69
0.260	0.00	-0.346	31.654	438.2757	-6.33	-2.44
0.262	0.00	-0.281	32.484	391.8214	-5.66	-2.18
0.264	0.00	-0.216	33.221	344.2328	-4.97	-1.92
0.266	0.00	-0.149	33.860	295.6476	-4.27	-1.64
0.268	0.00	-0.080	34.402	246.2066	-3.56	-1.37
0.270	0.00	-0.011	34.845	196.0527	-2.83	-1.09

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.272	0.00	0.059	35.186	145.3313	-2.10	-0.81
0.274	0.00	0.130	35.425	94.1892	-1.36	-0.52
0.276	0.00	0.201	35.562	42.7744	-0.62	-0.24
0.278	0.00	0.272	35.596	-8.7643	0.13	0.05
0.280	0.00	0.343	35.527	-60.2776	0.87	0.34
0.282	0.00	0.414	35.355	-111.6164	1.61	0.62
0.284	0.00	0.484	35.081	-162.6320	2.35	0.90
0.286	0.00	0.554	34.705	-213.1769	3.08	1.19
0.288	0.00	0.623	34.229	-263.1045	3.80	1.46
0.290	0.00	0.691	33.654	-312.2705	4.51	1.74
0.292	0.00	0.758	32.981	-360.5325	5.21	2.01
0.294	0.00	0.823	32.213	-407.7507	5.89	2.27
0.296	0.00	0.886	31.351	-453.7885	6.56	2.52
0.298	0.00	0.948	30.399	-498.5125	7.20	2.77
0.300	0.00	1.008	29.359	-541.7934	7.83	3.01
0.302	0.00	1.065	28.233	-583.5057	8.43	3.25
0.304	0.00	1.121	27.026	-623.5288	9.01	3.47
0.306	0.00	1.174	25.741	-661.7468	9.56	3.68
0.308	0.00	1.224	24.381	-698.0490	10.09	3.88
0.310	0.00	1.271	22.951	-732.3304	10.58	4.07
0.312	0.00	1.315	21.454	-764.4916	11.05	4.25
0.314	0.00	1.357	19.895	-794.4397	11.48	4.42
0.316	0.00	1.395	18.278	-822.0878	11.88	4.57
0.318	0.00	1.430	16.609	-847.3560	12.25	4.71
0.320	0.00	1.461	14.891	-870.1710	12.58	4.84
0.322	0.00	1.489	13.131	-890.4670	12.87	4.95
0.324	0.00	1.514	11.332	-908.1850	13.12	5.05
0.326	0.00	1.535	9.501	-923.2738	13.34	5.14
0.328	0.00	1.552	7.642	-935.6897	13.52	5.21
0.330	0.00	1.565	5.761	-945.3968	13.66	5.26
0.332	0.00	1.575	3.863	-952.3669	13.76	5.30
0.334	0.00	1.581	1.954	-956.5799	13.82	5.32
0.336	0.00	1.583	0.039	-958.0237	13.84	5.33
0.338	0.00	1.581	-1.875	-956.6939	13.83	5.32
0.340	0.00	1.575	-3.785	-952.5945	13.77	5.30
0.342	0.00	1.566	-5.683	-945.7373	13.67	5.26
0.344	0.00	1.552	-7.565	-936.1422	13.53	5.21
0.346	0.00	1.535	-9.425	-923.8370	13.35	5.14
0.348	0.00	1.515	-11.258	-908.8573	13.13	5.06
0.350	0.00	1.490	-13.058	-891.2464	12.88	4.96
0.352	0.00	1.463	-14.820	-871.0553	12.59	4.85
0.354	0.00	1.431	-16.539	-848.3425	12.26	4.72
0.356	0.00	1.396	-18.211	-823.1738	11.90	4.58
0.358	0.00	1.358	-19.830	-795.6219	11.50	4.43
0.360	0.00	1.317	-21.391	-765.7667	11.07	4.26
0.362	0.00	1.273	-22.890	-733.6947	10.60	4.08
0.364	0.00	1.226	-24.324	-699.4985	10.11	3.89
0.366	0.00	1.176	-25.686	-663.2773	9.59	3.69
0.368	0.00	1.123	-26.975	-625.1360	9.03	3.48
0.370	0.00	1.068	-28.185	-585.1848	8.46	3.26
0.372	0.00	1.010	-29.314	-543.5395	7.85	3.02
0.374	0.00	0.951	-30.358	-500.3207	7.23	2.78

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.376	0.00	0.889	-31.314	-455.6534	6.58	2.54
0.378	0.00	0.825	-32.179	-409.6671	5.92	2.28
0.380	0.00	0.760	-32.951	-362.4947	5.24	2.02
0.382	0.00	0.694	-33.628	-314.2729	4.54	1.75
0.384	0.00	0.626	-34.207	-265.1413	3.83	1.48
0.386	0.00	0.557	-34.688	-215.2421	3.11	1.20
0.388	0.00	0.487	-35.068	-164.7197	2.38	0.92
0.390	0.00	0.417	-35.346	-113.7205	1.64	0.63
0.392	0.00	0.346	-35.522	-62.3921	0.90	0.35
0.394	0.00	0.275	-35.596	-10.8831	0.16	0.06
0.396	0.00	0.203	-35.566	40.6575	-0.59	-0.23
0.398	0.00	0.132	-35.433	92.0804	-1.33	-0.51
0.400	0.00	0.062	-35.198	143.2366	-2.07	-0.80
0.402	0.00	-0.008	-34.861	193.9782	-2.80	-1.08
0.404	0.00	-0.078	-34.422	244.1583	-3.53	-1.36
0.406	0.00	-0.146	-33.885	293.6315	-4.24	-1.63
0.408	0.00	-0.213	-33.249	342.2546	-4.95	-1.90
0.410	0.00	-0.279	-32.517	389.8869	-5.63	-2.17
0.412	0.00	-0.343	-31.690	436.3905	-6.31	-2.43
0.414	0.00	-0.406	-30.772	481.6307	-6.96	-2.68
0.416	0.00	-0.466	-29.765	525.4766	-7.59	-2.92
0.418	0.00	-0.525	-28.672	567.8012	-8.21	-3.16
0.420	0.00	-0.581	-27.496	608.4821	-8.79	-3.39
0.422	0.00	-0.634	-26.240	647.4013	-9.36	-3.60
0.424	0.00	-0.686	-24.908	684.4464	-9.89	-3.81
0.426	0.00	-0.734	-23.504	719.5100	-10.40	-4.00
0.428	0.00	-0.780	-22.032	752.4905	-10.87	-4.19
0.430	0.00	-0.822	-20.496	783.2926	-11.32	-4.36
0.432	0.00	-0.862	-18.901	811.8271	-11.73	-4.52
0.434	0.00	-0.898	-17.251	838.0113	-12.11	-4.66
0.436	0.00	-0.930	-15.551	861.7695	-12.45	-4.79
0.438	0.00	-0.960	-13.807	883.0328	-12.76	-4.91
0.440	0.00	-0.986	-12.022	901.7398	-13.03	-5.02
0.442	0.00	-1.008	-10.202	917.8362	-13.26	-5.11
0.444	0.00	-1.026	-8.353	931.2755	-13.46	-5.18
0.446	0.00	-1.041	-6.480	942.0187	-13.61	-5.24
0.448	0.00	-1.052	-4.588	950.0347	-13.73	-5.29
0.450	0.00	-1.060	-2.682	955.3004	-13.81	-5.32
0.452	0.00	-1.063	-0.769	957.8005	-13.84	-5.33
0.454	0.00	-1.063	1.146	957.5277	-13.84	-5.33
0.456	0.00	-1.059	3.058	954.4829	-13.79	-5.31
0.458	0.00	-1.050	4.961	948.6749	-13.71	-5.28
0.460	0.00	-1.039	6.850	940.1204	-13.59	-5.23
0.462	0.00	-1.023	8.719	928.8443	-13.42	-5.17
0.464	0.00	-1.004	10.563	914.8791	-13.22	-5.09
0.466	0.00	-0.981	12.376	898.2654	-12.98	-5.00
0.468	0.00	-0.954	14.153	879.0512	-12.70	-4.89
0.470	0.00	-0.924	15.889	857.2921	-12.39	-4.77
0.472	0.00	-0.891	17.580	833.0511	-12.04	-4.63
0.474	0.00	-0.854	19.219	806.3985	-11.65	-4.49
0.476	0.00	-0.814	20.803	777.4113	-11.23	-4.33
0.478	0.00	-0.771	22.327	746.1735	-10.78	-4.15
0.480	0.00	-0.725	23.786	712.7756	-10.30	-3.97
0.482	0.00	-0.676	25.176	677.3141	-9.79	-3.77
0.484	0.00	-0.624	26.493	639.8918	-9.25	-3.56
0.486	0.00	-0.570	27.733	600.6170	-8.68	-3.34
0.488	0.00	-0.513	28.894	559.6035	-8.09	-3.11
0.490	0.00	-0.454	29.970	516.9698	-7.47	-2.88
0.492	0.00	-0.393	30.960	472.8396	-6.83	-2.63
0.494	0.00	-0.331	31.860	427.3404	-6.18	-2.38

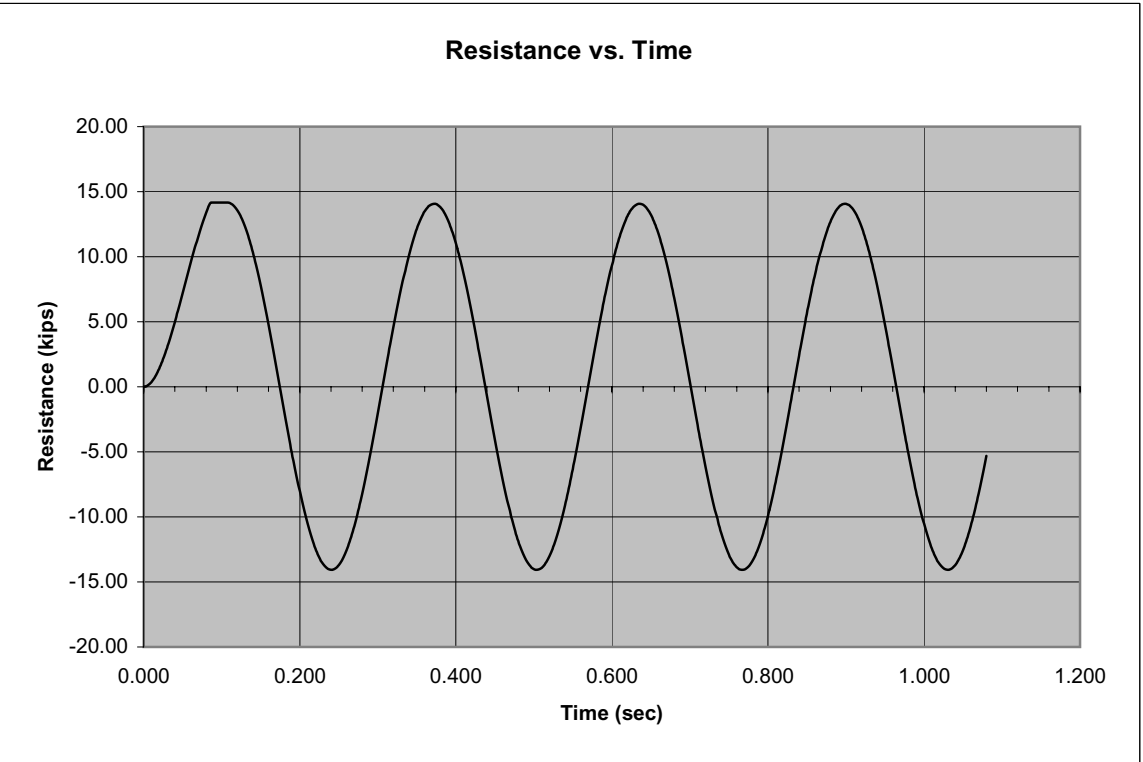
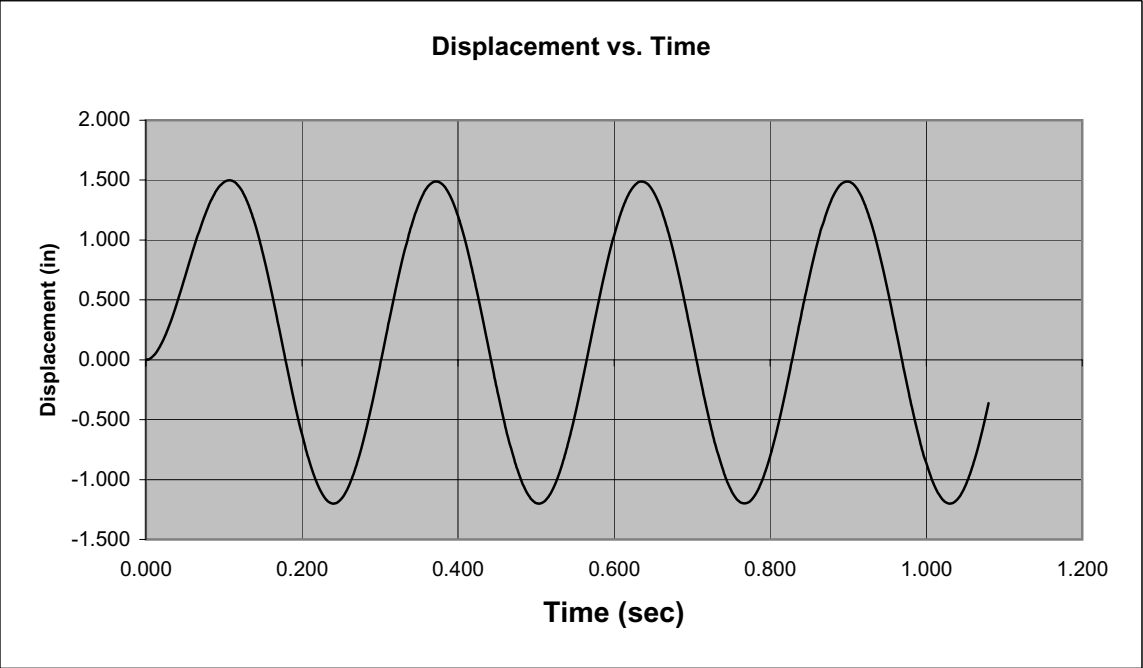
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.496	0.00	-0.266	32.668	380.6041	-5.50	-2.12
0.498	0.00	-0.200	33.381	332.7660	-4.81	-1.85
0.500	0.00	-0.133	33.998	283.9645	-4.10	-1.58
0.502	0.00	-0.064	34.516	234.3409	-3.39	-1.30
0.504	0.00	0.005	34.935	184.0389	-2.66	-1.02
0.506	0.00	0.076	35.252	133.2040	-1.92	-0.74
0.508	0.00	0.146	35.467	81.9836	-1.18	-0.46
0.510	0.00	0.217	35.580	30.5258	-0.44	-0.17
0.512	0.00	0.289	35.589	-21.0203	0.30	0.12
0.514	0.00	0.360	35.496	-72.5056	1.05	0.40
0.516	0.00	0.431	35.300	-123.7810	1.79	0.69
0.518	0.00	0.501	35.001	-174.6980	2.52	0.97
0.520	0.00	0.571	34.601	-225.1093	3.25	1.25
0.522	0.00	0.639	34.101	-274.8689	3.97	1.53
0.524	0.00	0.707	33.503	-323.8328	4.68	1.80
0.526	0.00	0.773	32.807	-371.8591	5.37	2.07
0.528	0.00	0.838	32.016	-418.8089	6.05	2.33
0.530	0.00	0.901	31.133	-464.5463	6.71	2.58
0.532	0.00	0.962	30.159	-508.9388	7.35	2.83
0.534	0.00	1.022	29.099	-551.8579	7.98	3.07
0.536	0.00	1.079	27.954	-593.1794	8.57	3.30
0.538	0.00	1.134	26.728	-632.7836	9.14	3.52
0.540	0.00	1.186	25.424	-670.5559	9.69	3.73
0.542	0.00	1.235	24.047	-706.3869	10.21	3.93
0.544	0.00	1.282	22.601	-740.1730	10.70	4.12
0.546	0.00	1.326	21.089	-771.8162	11.15	4.29
0.548	0.00	1.366	19.516	-801.2250	11.58	4.46
0.550	0.00	1.404	17.886	-828.3142	11.97	4.61
0.552	0.00	1.438	16.205	-853.0055	12.33	4.75
0.554	0.00	1.468	14.477	-875.2273	12.65	4.87
0.556	0.00	1.496	12.706	-894.9154	12.93	4.98
0.558	0.00	1.519	10.900	-912.0126	13.18	5.07
0.560	0.00	1.539	9.061	-926.4695	13.39	5.15
0.562	0.00	1.555	7.196	-938.2443	13.56	5.22
0.564	0.00	1.568	5.311	-947.3029	13.69	5.27
0.566	0.00	1.577	3.410	-953.6191	13.78	5.31
0.568	0.00	1.582	1.499	-957.1745	13.83	5.33
0.570	0.00	1.583	-0.416	-957.9588	13.84	5.33
0.572	0.00	1.580	-2.330	-955.9699	13.82	5.32
0.574	0.00	1.573	-4.237	-951.2134	13.75	5.29
0.576	0.00	1.563	-6.132	-943.7031	13.64	5.25
0.578	0.00	1.549	-8.009	-933.4609	13.49	5.19
0.580	0.00	1.531	-9.863	-920.5162	13.30	5.12
0.582	0.00	1.509	-11.689	-904.9066	13.08	5.03
0.584	0.00	1.484	-13.480	-886.6773	12.81	4.93
0.586	0.00	1.455	-15.233	-865.8811	12.51	4.82
0.588	0.00	1.423	-16.941	-842.5782	12.18	4.69
0.590	0.00	1.388	-18.601	-816.8359	11.80	4.54
0.592	0.00	1.349	-20.206	-788.7290	11.40	4.39
0.594	0.00	1.307	-21.753	-758.3386	10.96	4.22
0.596	0.00	1.262	-23.237	-725.7528	10.49	4.04
0.598	0.00	1.214	-24.654	-691.0660	9.99	3.84
0.600	0.00	1.163	-26.000	-654.3786	9.46	3.64
0.602	0.00	1.110	-27.270	-615.7967	8.90	3.43
0.604	0.00	1.054	-28.461	-575.4320	8.32	3.20
0.606	0.00	0.996	-29.570	-533.4015	7.71	2.97
0.608	0.00	0.936	-30.593	-489.8268	7.08	2.73
0.610	0.00	0.874	-31.528	-444.8340	6.43	2.47
0.612	0.00	0.810	-32.371	-398.5535	5.76	2.22
0.614	0.00	0.745	-33.121	-351.1191	5.07	1.95
0.616	0.00	0.678	-33.775	-302.6682	4.37	1.68

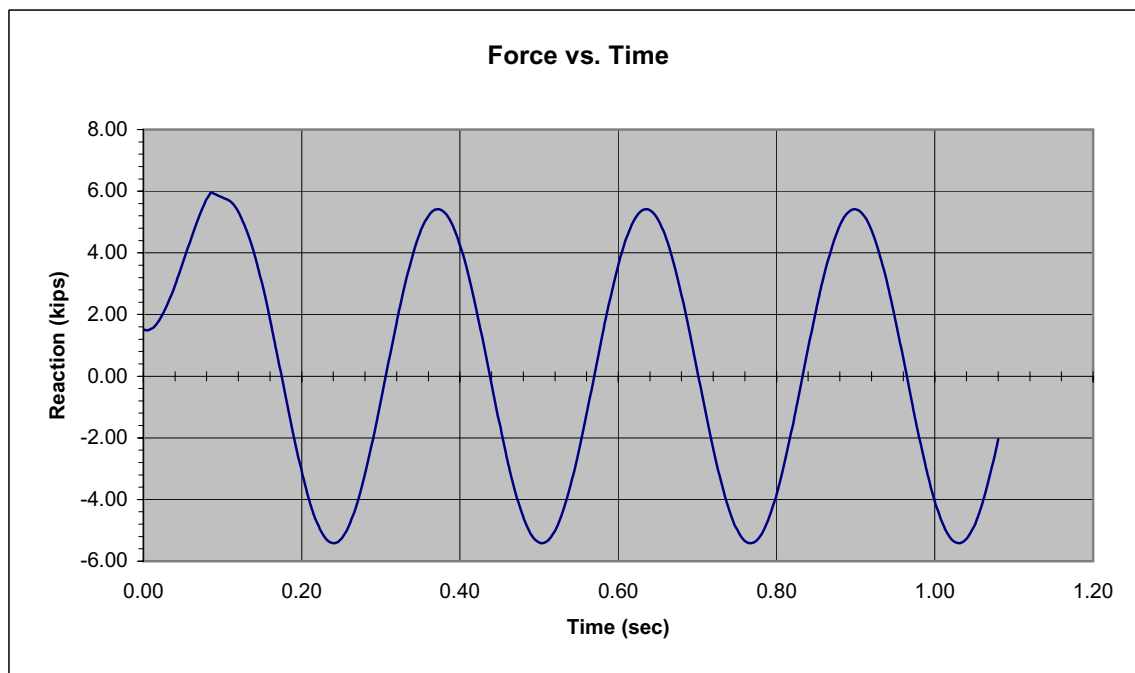
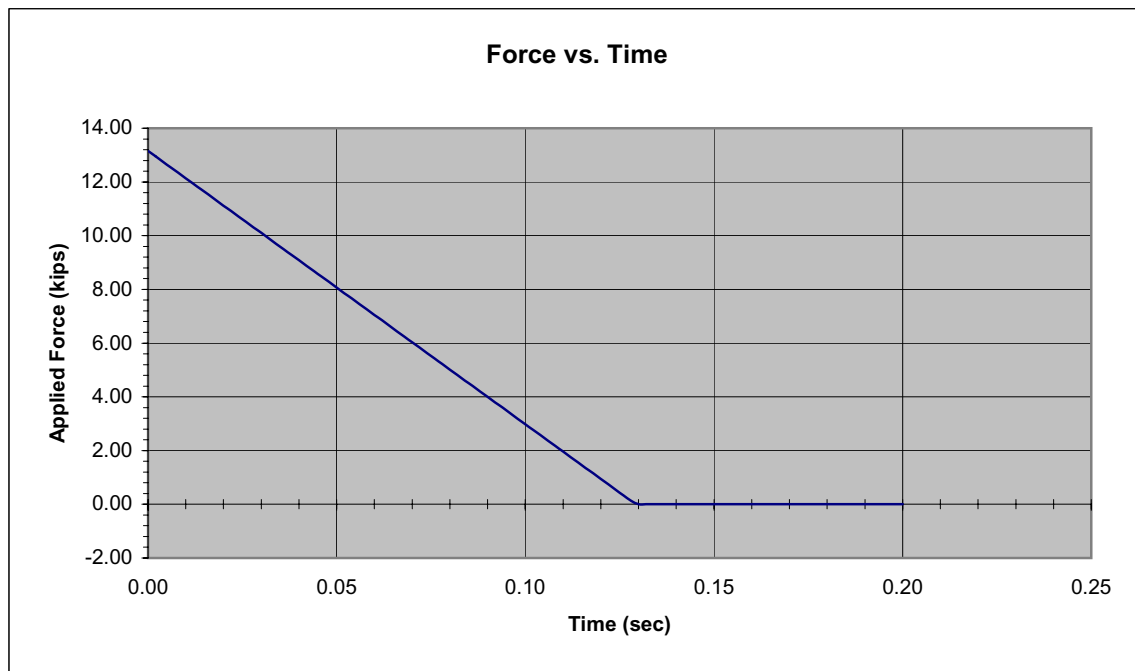
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.618	0.00	0.610	-34.331	-253.3411	3.66	1.41
0.620	0.00	0.540	-34.787	-203.2806	2.94	1.13
0.622	0.00	0.470	-35.143	-152.6315	2.21	0.85
0.624	0.00	0.400	-35.397	-101.5406	1.47	0.56
0.626	0.00	0.329	-35.549	-50.1558	0.72	0.28
0.628	0.00	0.258	-35.598	1.3743	-0.02	-0.01
0.630	0.00	0.187	-35.544	52.9004	-0.76	-0.29
0.632	0.00	0.116	-35.386	104.2733	-1.51	-0.58
0.634	0.00	0.045	-35.127	155.3444	-2.24	-0.86
0.636	0.00	-0.025	-34.765	205.9657	-2.98	-1.15
0.638	0.00	-0.094	-34.304	255.9908	-3.70	-1.42
0.640	0.00	-0.162	-33.742	305.2748	-4.41	-1.70
0.642	0.00	-0.229	-33.083	353.6750	-5.11	-1.97
0.644	0.00	-0.294	-32.329	401.0513	-5.80	-2.23
0.646	0.00	-0.358	-31.480	447.2665	-6.46	-2.49
0.648	0.00	-0.420	-30.541	492.1869	-7.11	-2.74
0.650	0.00	-0.480	-29.513	535.6825	-7.74	-2.98
0.652	0.00	-0.538	-28.400	577.6272	-8.35	-3.21
0.654	0.00	-0.594	-27.204	617.8997	-8.93	-3.44
0.656	0.00	-0.647	-25.930	656.3834	-9.49	-3.65
0.658	0.00	-0.697	-24.580	692.9668	-10.01	-3.86
0.660	0.00	-0.745	-23.160	727.5441	-10.51	-4.05
0.662	0.00	-0.790	-21.672	760.0151	-10.98	-4.23
0.664	0.00	-0.832	-20.122	790.2859	-11.42	-4.40
0.666	0.00	-0.870	-18.514	818.2688	-11.83	-4.55
0.668	0.00	-0.906	-16.851	843.8828	-12.20	-4.70
0.670	0.00	-0.938	-15.140	867.0537	-12.53	-4.82
0.672	0.00	-0.966	-13.386	887.7145	-12.83	-4.94
0.674	0.00	-0.991	-11.592	905.8054	-13.09	-5.04
0.676	0.00	-1.013	-9.765	921.2740	-13.31	-5.13
0.678	0.00	-1.030	-7.910	934.0754	-13.50	-5.20
0.680	0.00	-1.044	-6.031	944.1727	-13.64	-5.25
0.682	0.00	-1.054	-4.136	951.5366	-13.75	-5.29
0.684	0.00	-1.061	-2.228	956.1458	-13.82	-5.32
0.686	0.00	-1.063	-0.314	957.9870	-13.84	-5.33
0.688	0.00	-1.062	1.601	957.0548	-13.83	-5.32
0.690	0.00	-1.057	3.511	953.3519	-13.78	-5.30
0.692	0.00	-1.048	5.412	946.8890	-13.68	-5.27
0.694	0.00	-1.035	7.296	937.6848	-13.55	-5.22
0.696	0.00	-1.019	9.160	925.7661	-13.38	-5.15
0.698	0.00	-0.999	10.997	911.1673	-13.17	-5.07
0.700	0.00	-0.975	12.802	893.9306	-12.92	-4.97
0.702	0.00	-0.948	14.570	874.1060	-12.63	-4.86
0.704	0.00	-0.917	16.296	851.7508	-12.31	-4.74
0.706	0.00	-0.882	17.974	826.9298	-11.95	-4.60
0.708	0.00	-0.845	19.601	799.7149	-11.56	-4.45
0.710	0.00	-0.804	21.171	770.1847	-11.13	-4.29
0.712	0.00	-0.760	22.680	738.4249	-10.67	-4.11
0.714	0.00	-0.713	24.122	704.5273	-10.18	-3.92
0.716	0.00	-0.664	25.496	668.5901	-9.66	-3.72
0.718	0.00	-0.611	26.795	630.7173	-9.11	-3.51
0.720	0.00	-0.557	28.017	591.0186	-8.54	-3.29
0.722	0.00	-0.499	29.157	549.6089	-7.94	-3.06
0.724	0.00	-0.440	30.213	506.6080	-7.32	-2.82
0.726	0.00	-0.379	31.182	462.1406	-6.68	-2.57
0.728	0.00	-0.315	32.061	416.3352	-6.02	-2.32
0.730	0.00	-0.250	32.846	369.3245	-5.34	-2.05
0.732	0.00	-0.184	33.537	321.2447	-4.64	-1.79
0.734	0.00	-0.116	34.130	272.2348	-3.93	-1.51
0.736	0.00	-0.048	34.625	222.4368	-3.21	-1.24
0.738	0.00	0.022	35.020	171.9948	-2.49	-0.96

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.740	0.00	0.092	35.313	121.0550	-1.75	-0.67
0.742	0.00	0.163	35.503	69.7646	-1.01	-0.39
0.744	0.00	0.234	35.591	18.2723	-0.26	-0.10
0.746	0.00	0.306	35.576	-33.2729	0.48	0.19
0.748	0.00	0.377	35.458	-84.7218	1.22	0.47
0.750	0.00	0.447	35.238	-135.9254	1.96	0.76
0.752	0.00	0.518	34.915	-186.7355	2.70	1.04
0.754	0.00	0.587	34.491	-237.0050	3.43	1.32
0.756	0.00	0.655	33.968	-286.5883	4.14	1.59
0.758	0.00	0.723	33.346	-335.3420	4.85	1.87
0.760	0.00	0.789	32.627	-383.1249	5.54	2.13
0.762	0.00	0.853	31.814	-429.7986	6.21	2.39
0.764	0.00	0.916	30.909	-475.2281	6.87	2.64
0.766	0.00	0.977	29.915	-519.2818	7.50	2.89
0.768	0.00	1.036	28.834	-561.8321	8.12	3.13
0.770	0.00	1.092	27.669	-602.7559	8.71	3.35
0.772	0.00	1.146	26.425	-641.9348	9.28	3.57
0.774	0.00	1.198	25.103	-679.2552	9.82	3.78
0.776	0.00	1.247	23.709	-714.6092	10.33	3.98
0.778	0.00	1.292	22.247	-747.8944	10.81	4.16
0.780	0.00	1.335	20.720	-779.0144	11.26	4.33
0.782	0.00	1.375	19.133	-807.8791	11.67	4.49
0.784	0.00	1.412	17.491	-834.4051	12.06	4.64
0.786	0.00	1.445	15.798	-858.5154	12.41	4.78
0.788	0.00	1.475	14.059	-880.1403	12.72	4.90
0.790	0.00	1.501	12.280	-899.2172	12.99	5.00
0.792	0.00	1.524	10.465	-915.6909	13.23	5.09
0.794	0.00	1.543	8.620	-929.5136	13.43	5.17
0.796	0.00	1.559	6.750	-940.6454	13.59	5.23
0.798	0.00	1.570	4.860	-949.0540	13.72	5.28
0.800	0.00	1.578	2.956	-954.7151	13.80	5.31
0.802	0.00	1.582	1.044	-957.6123	13.84	5.33
0.804	0.00	1.582	-0.871	-957.7371	13.84	5.33
0.806	0.00	1.579	-2.784	-955.0893	13.80	5.31
0.808	0.00	1.571	-4.689	-949.6766	13.72	5.28
0.810	0.00	1.560	-6.580	-941.5145	13.61	5.24
0.812	0.00	1.545	-8.452	-930.6267	13.45	5.18
0.814	0.00	1.526	-10.300	-917.0447	13.25	5.10
0.816	0.00	1.504	-12.118	-900.8079	13.02	5.01
0.818	0.00	1.478	-13.901	-881.9632	12.75	4.91
0.820	0.00	1.448	-15.643	-860.5652	12.44	4.79
0.822	0.00	1.415	-17.340	-836.6759	12.09	4.66
0.824	0.00	1.379	-18.988	-810.3644	11.71	4.51
0.826	0.00	1.339	-20.580	-781.7069	11.30	4.35
0.828	0.00	1.296	-22.112	-750.7863	10.85	4.18
0.830	0.00	1.251	-23.581	-717.6922	10.37	3.99
0.832	0.00	1.202	-24.981	-682.5204	9.86	3.80
0.834	0.00	1.151	-26.309	-645.3727	9.33	3.59
0.836	0.00	1.097	-27.560	-606.3566	8.76	3.37
0.838	0.00	1.041	-28.732	-565.5851	8.17	3.15
0.840	0.00	0.982	-29.821	-523.1762	7.56	2.91
0.842	0.00	0.921	-30.824	-479.2527	6.93	2.67
0.844	0.00	0.859	-31.737	-433.9418	6.27	2.41
0.846	0.00	0.795	-32.558	-387.3746	5.60	2.16
0.848	0.00	0.729	-33.285	-339.6860	4.91	1.89
0.850	0.00	0.662	-33.916	-291.0140	4.21	1.62
0.852	0.00	0.593	-34.448	-241.4995	3.49	1.34
0.854	0.00	0.524	-34.881	-191.2858	2.76	1.06
0.856	0.00	0.454	-35.213	-140.5184	2.03	0.78
0.858	0.00	0.383	-35.443	-89.3441	1.29	0.50
0.860	0.00	0.312	-35.570	-37.9112	0.55	0.21

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.862	0.00	0.241	-35.594	13.6314	-0.20	-0.08
0.864	0.00	0.170	-35.516	65.1346	-0.94	-0.36
0.866	0.00	0.099	-35.334	116.4492	-1.68	-0.65
0.868	0.00	0.028	-35.050	167.4267	-2.42	-0.93
0.870	0.00	-0.041	-34.665	217.9195	-3.15	-1.21
0.872	0.00	-0.110	-34.179	267.7814	-3.87	-1.49
0.874	0.00	-0.178	-33.594	316.8681	-4.58	-1.76
0.876	0.00	-0.244	-32.912	365.0374	-5.28	-2.03
0.878	0.00	-0.310	-32.135	412.1500	-5.96	-2.29
0.880	0.00	-0.373	-31.265	458.0694	-6.62	-2.55
0.882	0.00	-0.435	-30.304	502.6626	-7.26	-2.80
0.884	0.00	-0.494	-29.256	545.8007	-7.89	-3.04
0.886	0.00	-0.552	-28.123	587.3586	-8.49	-3.27
0.888	0.00	-0.607	-26.908	627.2162	-9.06	-3.49
0.890	0.00	-0.659	-25.616	665.2579	-9.61	-3.70
0.892	0.00	-0.709	-24.249	701.3737	-10.14	-3.90
0.894	0.00	-0.756	-22.812	735.4591	-10.63	-4.09
0.896	0.00	-0.800	-21.309	767.4152	-11.09	-4.27
0.898	0.00	-0.841	-19.745	797.1497	-11.52	-4.44
0.900	0.00	-0.879	-18.123	824.5765	-11.92	-4.59
0.902	0.00	-0.914	-16.449	849.6160	-12.28	-4.73
0.904	0.00	-0.945	-14.727	872.1960	-12.60	-4.85
0.906	0.00	-0.973	-12.963	892.2509	-12.89	-4.96
0.908	0.00	-0.997	-11.161	909.7227	-13.15	-5.06
0.910	0.00	-1.017	-9.326	924.5609	-13.36	-5.14
0.912	0.00	-1.034	-7.465	936.7225	-13.54	-5.21
0.914	0.00	-1.047	-5.582	946.1722	-13.67	-5.26
0.916	0.00	-1.056	-3.683	952.8828	-13.77	-5.30
0.918	0.00	-1.062	-1.773	956.8348	-13.83	-5.32
0.920	0.00	-1.063	0.141	958.0167	-13.84	-5.33
0.922	0.00	-1.061	2.056	956.4251	-13.82	-5.32
0.924	0.00	-1.055	3.964	952.0647	-13.76	-5.30
0.926	0.00	-1.045	5.861	944.9481	-13.66	-5.26
0.928	0.00	-1.032	7.741	935.0958	-13.51	-5.20
0.930	0.00	-1.014	9.599	922.5364	-13.33	-5.13
0.932	0.00	-0.993	11.429	907.3063	-13.11	-5.05
0.934	0.00	-0.969	13.226	889.4494	-12.85	-4.95
0.936	0.00	-0.940	14.984	869.0177	-12.56	-4.84
0.938	0.00	-0.909	16.699	846.0701	-12.23	-4.71
0.940	0.00	-0.874	18.366	820.6731	-11.86	-4.57
0.942	0.00	-0.835	19.980	792.9003	-11.46	-4.41
0.944	0.00	-0.794	21.535	762.8320	-11.02	-4.24
0.946	0.00	-0.749	23.029	730.5553	-10.56	-4.06
0.948	0.00	-0.702	24.455	696.1637	-10.06	-3.87
0.950	0.00	-0.651	25.811	659.7566	-9.53	-3.67
0.952	0.00	-0.599	27.093	621.4395	-8.98	-3.46
0.954	0.00	-0.543	28.295	581.3234	-8.40	-3.23
0.956	0.00	-0.485	29.416	539.5243	-7.80	-3.00
0.958	0.00	-0.426	30.452	496.1633	-7.17	-2.76
0.960	0.00	-0.364	31.399	451.3659	-6.52	-2.51
0.962	0.00	-0.300	32.256	405.2618	-5.86	-2.25
0.964	0.00	-0.235	33.019	357.9845	-5.17	-1.99
0.966	0.00	-0.168	33.687	309.6708	-4.48	-1.72
0.968	0.00	-0.100	34.257	260.4606	-3.76	-1.45
0.970	0.00	-0.031	34.728	210.4963	-3.04	-1.17
0.972	0.00	0.039	35.098	159.9227	-2.31	-0.89
0.974	0.00	0.109	35.367	108.8861	-1.57	-0.61
0.976	0.00	0.180	35.534	57.5342	-0.83	-0.32
0.978	0.00	0.251	35.597	6.0158	-0.09	-0.03
0.980	0.00	0.323	35.558	-45.5200	0.66	0.25
0.982	0.00	0.393	35.415	-96.9240	1.40	0.54

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.984	0.00	0.464	35.170	-148.0475	2.14	0.82
0.986	0.00	0.534	34.823	-198.7423	2.87	1.11
0.988	0.00	0.603	34.376	-248.8618	3.60	1.38
0.990	0.00	0.672	33.829	-298.2608	4.31	1.66
0.992	0.00	0.739	33.184	-346.7964	5.01	1.93
0.994	0.00	0.804	32.443	-394.3280	5.70	2.19
0.996	0.00	0.868	31.608	-440.7180	6.37	2.45
0.998	0.00	0.931	30.681	-485.8321	7.02	2.70
1.000	0.00	0.991	29.666	-529.5397	7.65	2.95
1.002	0.00	1.049	28.564	-571.7143	8.26	3.18
1.004	0.00	1.105	27.380	-612.2338	8.85	3.41
1.006	0.00	1.159	26.117	-650.9808	9.41	3.62
1.008	0.00	1.210	24.778	-687.8433	9.94	3.83
1.010	0.00	1.258	23.368	-722.7144	10.44	4.02
1.012	0.00	1.303	21.890	-755.4933	10.92	4.20
1.014	0.00	1.345	20.348	-786.0850	11.36	4.37
1.016	0.00	1.384	18.748	-814.4010	11.77	4.53
1.018	0.00	1.420	17.093	-840.3593	12.14	4.68
1.020	0.00	1.453	15.389	-863.8848	12.48	4.81
1.022	0.00	1.482	13.640	-884.9093	12.79	4.92
1.024	0.00	1.507	11.851	-903.3719	13.05	5.03
1.026	0.00	1.529	10.029	-919.2193	13.28	5.11
1.028	0.00	1.547	8.177	-932.4056	13.47	5.19
1.030	0.00	1.562	6.302	-942.8925	13.63	5.25
1.032	0.00	1.573	4.408	-950.6498	13.74	5.29
1.034	0.00	1.579	2.502	-955.6548	13.81	5.32
1.036	0.00	1.583	0.589	-957.8933	13.84	5.33
1.038	0.00	1.582	-1.327	-957.3587	13.84	5.33
1.040	0.00	1.577	-3.238	-954.0525	13.79	5.31
1.042	0.00	1.569	-5.140	-947.9843	13.70	5.27
1.044	0.00	1.557	-7.027	-939.1716	13.57	5.23
1.046	0.00	1.541	-8.894	-927.6401	13.41	5.16
1.048	0.00	1.521	-10.735	-913.4231	13.20	5.08
1.050	0.00	1.498	-12.545	-896.5616	12.96	4.99
1.052	0.00	1.471	-14.319	-877.1046	12.68	4.88
1.054	0.00	1.441	-16.051	-855.1084	12.36	4.76
1.056	0.00	1.407	-17.737	-830.6367	12.00	4.62
1.058	0.00	1.370	-19.371	-803.7602	11.62	4.47
1.060	0.00	1.329	-20.950	-774.5568	11.19	4.31
1.062	0.00	1.286	-22.467	-743.1111	10.74	4.13
1.064	0.00	1.239	-23.920	-709.5141	10.25	3.95
1.066	0.00	1.190	-25.303	-673.8630	9.74	3.75
1.068	0.00	1.138	-26.613	-636.2611	9.19	3.54
1.070	0.00	1.084	-27.846	-596.8172	8.62	3.32
1.072	0.00	1.027	-28.999	-555.6455	8.03	3.09
1.074	0.00	0.968	-30.067	-512.8652	7.41	2.85
1.076	0.00	0.907	-31.049	-468.6002	6.77	2.61
1.078	0.00	0.844	-31.940	-422.9786	6.11	2.35
1.080	0.00	0.779	-32.740	-376.1324	5.44	2.09





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Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

WALL PANELS W/ CABLE TRAY OPENING

Slenderness

Approximate Evaluation of Slenderness Effect

For Compression members braced against sidesway, the effect of slenderness may be neglected when,

$$\frac{kl_u}{r} < 34$$

M_{1b} = value of smaller end moment on compression members due to gravity loads

M_{2b} = value of larger end moment on compression members due to gravity loads

$$DL = \text{Roof DL} \times \text{unit width} \times 1/2 \text{ roof span} = 241.84 \text{ lbs}$$

$$e = 6.5 \text{ in}$$

$$\text{Moment} = DL \times e = 1571.94 \text{ lb-in}$$

where :

l_u = unsupported length of compression members

$$= 6630.000 \text{ mm}$$

$$= 261.024 \text{ in}$$

k_{LONG} = effective length factor

$$= 1$$

$$thk = 9.843 \text{ in}$$

$$w = 51.181 \text{ in}$$

$$I_g = 4066.74804 \text{ in}^4$$

$$A_g = 503.75101 \text{ in}^2$$

r = radius of gyration

$$= \sqrt{I / A}$$

$$= 2.84129069$$

$$M_1 = 1571.94 \text{ lb-in}$$

$$M_2 = 1571.94 \text{ lb-in}$$

$$\frac{kl_u}{r} = 91.868$$

$$\frac{kl_u}{r} > 34$$

Therefore, consider slenderness effect.

Magnified design moment

$$M_{slender} = \delta M_u$$

where :

$$\delta_b \frac{C_m}{1 - \frac{P_u}{\phi P_c}} \leq 1.0$$

For members braced against sidesway and without transverse loads between supports, C_m may be taken as

$$C_m = 0.60 + 0.40 (M_{1b} / M_{2b})$$

but not less than 0.40

for this case use $C_m = 1.0$, for all other cases

then, $C_m = 1.00$

$$P_u = 1.0 \cdot P_D$$

$$P_u = 242 \text{ lb} \quad \text{factored axial load}$$

$$\phi = 0.700 \quad \text{strength reduction factor}$$

$$P_c = \frac{\pi^2 EI}{(k l_u)^2} \quad \text{Euler's buckling load}$$

$$E_c = 4032041 \text{ psi}$$

$$I_g = 4066.75 \text{ in}^4$$

$\beta_d = \text{absolute value of ratio of maximum deadload moment to maximum total moment, always positive}$

$$\frac{M_{\max DL}}{M_{\max TOT}} = \frac{1571.94}{1572}$$

$$= 1.000$$

$$\text{Max. Mom} = 1572 \text{ KN-m}$$

$$EI = \frac{\frac{E_c I_g}{2.5}}{1 + \beta_d}$$

$$3279456110$$

$$\text{then, } P_c = \frac{\pi^2 EI}{(k l_u)^2}$$

$$= 475052.8684$$

$$\delta_b \frac{C_m}{1 - \frac{P_u}{\phi P_c}} \geq 1. \quad 1.0007$$

$$\delta_{Mu} = 1573.081 \quad \text{lb-in}$$

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Job No. : BI - 3039
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 Design by : MGG
 Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Roof Dead Load = 8.470 KN/m²
 = 176.90 psf

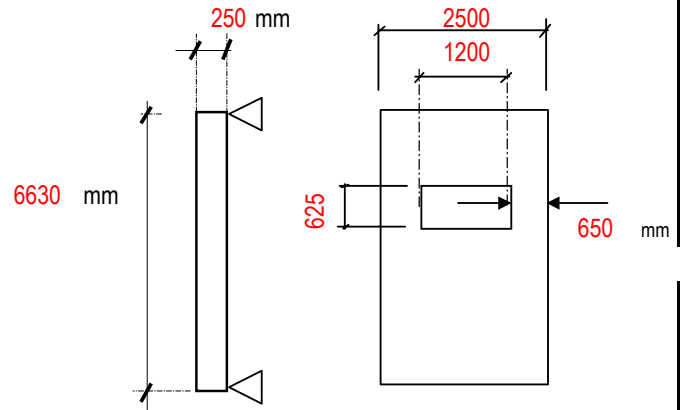
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
 Reinf., f_y = 414 Mpa

PANEL DATA :

Panel Width, width = 2500 mm
 Wall Span, lb = 6630 mm
 Wall, t_{TRIAL} = 250 mm
 (Panel Width - Opening) ; b = 1300 mm
 Roof span : = 10 m

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure, P_s = 4.20 psi (SEE LOADING CALCULATION)
 Effective blast time duration, T_e = 129 msec

Moment of Inertia, I_g = 4066.75 in⁴ (1692708333 mm⁴)
 Area of Bar ($\emptyset$), 25 mm $\emptyset$ A_b = 0.7609 in² (491 mm²)
 Number of Bars per side of opening N = 10.0000 in
 Area of Rebar PER side, A_s = 7.6086 in² (4909 mm²)
 Effective Depth, $d_c = d_s$ = 8.3425 in (212 mm)
 $f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for bending) = 5954.3 psi (41 Mpa.)
 $f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for shear) = 5504.0 psi (38 Mpa.)
 $f_{dy} = (SIF) \times (DIF) \times f_y$ = 77276 psi (533 Mpa.)
 Modulus of Elasticity, $E_{dc} = 57000 \times (f'_c)^{1/2}$ = 4032041 psi (27801 Mpa.)
 Steel ratio, $p = A_s / b \times d$ = 0.0178 > 200 / f_{dy} = 0.003
 Stress block, $a = A_s \times f_{dy} / 0.85 \times f'_{dc} \times b$ = 2.2698 in (58 mm)
 Modular ratio, $n = E_s / E_{dm}$ = 7.1940

a.) BENDING RESISTANCE

Mgnified Moment = M (slender) = 1573.081 lb-in (178 KN-m)
 Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$ = 4237786.57 lb-in (478782 KN-m)
 Net Moment, = $M_p - M$ (slender) = 4236213.49 lb-in (478604 KN-m)
 Resistance at cracked moment, $R_b = 8 \times M_p / L$ = 129882.09 lb (577.72 KN)
 Unit Resistance, $r_b = R_b / (L \times \text{panel width})$ = 5.0555 psi (0.0349 Mpa.)

b.) SHEAR RESISTANCE

Shear, $V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$ = 63354.2 lb (282 KN)
 At Support, $R_s = V_n \times L / (0.5 \times L - t_w)$ = 135361 lb (602 KN)
 Unit Resistance, $r_s = R_s / (L \times \text{panel width})$ = 5.269 psi (0.0363 Mpa.)

∴ Consider 1 ft strip

∴ Dynamic Resistance Controlled by Bending, R_u = 5.0555 psi > 4.20 ∴ O.K!
 15.8352 kips 13.16 kips

Cracked Moment of Inertia:

$n \times A_s$ = 54.7360 in² (35313 mm²)
 $C = -n \times A_s + [n \times A_s * (n \times A_s + 2 \times b \times d)]^{0.5} / b$ = 3.2880 in² (2121 mm²)

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$$I_{cr} = b \times C^3 / 3 + n \times A_s \times (d - C)^2 = 2004.84 \text{ in}^4 \quad (834477277 \text{ mm}^4)$$

Average Moment of Inertia,	$I_a = (I_g + I_{cr}) / 2$	=	3035.794	in ⁴	(1263592805.1 mm ⁴)
Effective Stiffness,	$K = 384 \times E_{dc} \times I_a / 5 L^3$	=	6.4446	kips / in	(per 1 ft strip)
Yield Deflection,	$Y_e = R_u / K$	=	2.45714	in	
Wall Mass = wall total weight / panel width / gravity acc.		=	0.00662	kip-sec ² / in	
Roof Mass = Roof DL x unit width x 1/2 roof span / gravity acc.		=	0.00752	kip-sec ² / in	
Total Mass, M		=	0.01414	kip-sec ² / in	
Average Stiffness,	K_{LM}	=	0.72000	FROM ASCE TABLE 6.1	
Equivalent Mass,	$M_e = K_{LM} \times \text{Beam Mass}$	=	0.01018	kip-sec ² / in	
Period of Vibration,	$t_n = 2 \times \pi \times (M_e / K)^{0.5}$	=	0.24971	sec	
Time increment, $t_n/10$	Δt	=	0.02497	sec	; use $\Delta t = 0.00200$ sec
t_d / t_n		=	0.51773		
R_u / P_s		=	1.20369		

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NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	10.464 kips / in
Mass	M	=	0.0145 kip-sec ² / in
Tension resistance	Rut	=	14.11 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	13.16 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.1293 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load	
load (kip)	time (sec)
13.16	0
0	0.1293

Length	266.732 in
y _{max} =	1.608 in
r _{max} =	14.113 kips
μ =	1.193
θ =	0.691 degrees
Rut/Fo =	1.073
θ = < 2 degrees	
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.000	13.16	0.000	0.000	910.3373	0.00	1.51
0.002	12.95	0.002	1.805	894.9434	0.02	1.50
0.004	12.75	0.007	3.577	876.9587	0.08	1.50
0.006	12.55	0.016	5.311	856.4351	0.17	1.51
0.008	12.34	0.028	7.000	833.4321	0.30	1.53
0.010	12.14	0.044	8.642	808.0164	0.46	1.57
0.012	11.93	0.063	10.230	780.2614	0.66	1.63
0.014	11.73	0.085	11.761	750.2476	0.89	1.69
0.016	11.53	0.110	13.229	718.0618	1.15	1.77
0.018	11.32	0.138	14.631	683.7972	1.44	1.86
0.020	11.12	0.168	15.962	647.5530	1.76	1.96
0.022	10.92	0.202	17.219	609.4341	2.11	2.07
0.024	10.71	0.237	18.398	569.5509	2.48	2.19
0.026	10.51	0.275	19.496	528.0188	2.88	2.32
0.028	10.31	0.315	20.509	484.9582	3.30	2.45
0.030	10.10	0.357	21.434	440.4935	3.74	2.60
0.032	9.90	0.401	22.269	394.7537	4.19	2.75
0.034	9.70	0.446	23.012	347.8710	4.67	2.91
0.036	9.49	0.493	23.660	299.9812	5.16	3.08
0.038	9.29	0.541	24.211	251.2230	5.66	3.25
0.040	9.09	0.590	24.664	201.7375	6.17	3.42
0.042	8.88	0.639	25.017	151.6679	6.69	3.60
0.044	8.68	0.690	25.270	101.1593	7.22	3.78
0.046	8.47	0.740	25.422	50.3578	7.75	3.96
0.048	8.27	0.791	25.472	-0.5894	8.28	4.14
0.050	8.07	0.842	25.419	-51.5350	8.81	4.32
0.052	7.86	0.893	25.266	-102.3314	9.34	4.50
0.054	7.66	0.943	25.010	-152.8315	9.87	4.68
0.056	7.46	0.993	24.655	-202.8892	10.39	4.86
0.058	7.25	1.042	24.199	-252.3595	10.90	5.03
0.060	7.05	1.090	23.646	-301.0992	11.40	5.20
0.062	6.85	1.136	22.996	-348.9672	11.89	5.36

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.064	6.64	1.181	22.251	-395.8250	12.36	5.52
0.066	6.44	1.225	21.414	-441.5368	12.82	5.68
0.068	6.24	1.267	20.486	-485.9704	13.26	5.82
0.070	6.03	1.307	19.471	-528.9972	13.68	5.96
0.072	5.83	1.345	18.372	-570.4924	14.07	6.09
0.074	5.63	1.380	17.191	-587.3016	14.11	6.08
0.076	5.42	1.414	16.002	-601.3845	14.11	6.06
0.078	5.22	1.444	14.785	-615.4675	14.11	6.03
0.080	5.01	1.473	13.540	-629.5505	14.11	6.01
0.082	4.81	1.499	12.267	-643.6335	14.11	5.99
0.084	4.61	1.522	10.966	-657.7164	14.11	5.96
0.086	4.40	1.542	9.636	-671.7994	14.11	5.94
0.088	4.20	1.560	8.279	-685.8824	14.11	5.92
0.090	4.00	1.576	6.893	-699.9654	14.11	5.89
0.092	3.79	1.588	5.479	-714.0483	14.11	5.87
0.094	3.59	1.597	4.037	-728.1313	14.11	5.85
0.096	3.39	1.604	2.566	-742.2143	14.11	5.82
0.098	3.18	1.608	1.068	-756.2972	14.11	5.80
0.100	2.98	1.608	-0.459	-770.3802	14.11	5.78
0.102	2.78	1.606	-2.012	-782.6770	14.09	5.74
0.104	2.57	1.600	-3.587	-792.7080	14.03	5.70
0.106	2.37	1.591	-5.180	-800.4440	13.94	5.64
0.108	2.17	1.580	-6.787	-805.8628	13.81	5.57
0.110	1.96	1.564	-8.402	-808.9486	13.65	5.48
0.112	1.76	1.546	-10.020	-809.6924	13.46	5.38
0.114	1.56	1.524	-11.638	-808.0922	13.23	5.27
0.116	1.35	1.499	-13.250	-804.1526	12.97	5.15
0.118	1.15	1.471	-14.852	-797.8849	12.68	5.01
0.120	0.94	1.440	-16.439	-789.3074	12.35	4.86
0.122	0.74	1.406	-18.007	-778.4448	11.99	4.70
0.124	0.54	1.368	-19.551	-765.3286	11.60	4.53
0.126	0.33	1.327	-21.066	-749.9967	11.17	4.34
0.128	0.13	1.284	-22.549	-732.4936	10.72	4.14
0.130	0.00	1.237	-23.989	-707.8165	10.23	3.94
0.132	0.00	1.188	-25.369	-672.0675	9.71	3.74
0.134	0.00	1.136	-26.675	-634.3729	9.17	3.53
0.136	0.00	1.081	-27.905	-594.8418	8.60	3.31
0.138	0.00	1.024	-29.053	-553.5886	8.00	3.08
0.140	0.00	0.965	-30.117	-510.7327	7.38	2.84
0.142	0.00	0.904	-31.095	-466.3983	6.74	2.59
0.144	0.00	0.841	-31.982	-420.7136	6.08	2.34
0.146	0.00	0.776	-32.776	-373.8110	5.40	2.08
0.148	0.00	0.710	-33.476	-325.8261	4.71	1.81
0.150	0.00	0.642	-34.079	-276.8980	4.00	1.54
0.152	0.00	0.573	-34.583	-227.1683	3.28	1.26
0.154	0.00	0.504	-34.987	-176.7809	2.55	0.98
0.156	0.00	0.433	-35.289	-125.8818	1.82	0.70
0.158	0.00	0.363	-35.490	-74.6182	1.08	0.42
0.160	0.00	0.292	-35.588	-23.1386	0.33	0.13
0.162	0.00	0.220	-35.582	28.4080	-0.41	-0.16
0.164	0.00	0.149	-35.474	79.8723	-1.15	-0.44
0.166	0.00	0.079	-35.263	131.1054	-1.89	-0.73

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.168	0.00	0.008	-34.950	181.9590	-2.63	-1.01
0.170	0.00	-0.061	-34.536	232.2858	-3.36	-1.29
0.172	0.00	-0.130	-34.021	281.9401	-4.07	-1.57
0.174	0.00	-0.197	-33.409	330.7782	-4.78	-1.84
0.176	0.00	-0.263	-32.699	378.6587	-5.47	-2.11
0.178	0.00	-0.328	-31.895	425.4430	-6.15	-2.37
0.180	0.00	-0.391	-30.999	470.9956	-6.81	-2.62
0.182	0.00	-0.452	-30.013	515.1847	-7.45	-2.87
0.184	0.00	-0.511	-28.939	557.8823	-8.06	-3.10
0.186	0.00	-0.568	-27.783	598.9648	-8.66	-3.33
0.188	0.00	-0.622	-26.545	638.3134	-9.22	-3.55
0.190	0.00	-0.674	-25.231	675.8140	-9.77	-3.76
0.192	0.00	-0.723	-23.844	711.3581	-10.28	-3.96
0.194	0.00	-0.769	-22.388	744.8428	-10.76	-4.14
0.196	0.00	-0.812	-20.867	776.1712	-11.22	-4.32
0.198	0.00	-0.852	-19.285	805.2526	-11.64	-4.48
0.200	0.00	-0.889	-17.648	832.0027	-12.02	-4.63
0.202	0.00	-0.923	-15.960	856.3442	-12.38	-4.76
0.204	0.00	-0.953	-14.225	878.2066	-12.69	-4.89
0.206	0.00	-0.980	-12.450	897.5266	-12.97	-4.99
0.208	0.00	-1.003	-10.638	914.2482	-13.21	-5.09
0.210	0.00	-1.022	-8.795	928.3230	-13.42	-5.16
0.212	0.00	-1.038	-6.927	939.7104	-13.58	-5.23
0.214	0.00	-1.050	-5.039	948.3773	-13.71	-5.28
0.216	0.00	-1.058	-3.136	954.2986	-13.79	-5.31
0.218	0.00	-1.063	-1.225	957.4572	-13.84	-5.33
0.220	0.00	-1.063	0.691	957.8439	-13.84	-5.33
0.222	0.00	-1.060	2.604	955.4577	-13.81	-5.32
0.224	0.00	-1.053	4.510	950.3055	-13.73	-5.29
0.226	0.00	-1.042	6.402	942.4020	-13.62	-5.24
0.228	0.00	-1.027	8.277	931.7704	-13.47	-5.18
0.230	0.00	-1.009	10.127	918.4412	-13.27	-5.11
0.232	0.00	-0.987	11.948	902.4532	-13.04	-5.02
0.234	0.00	-0.961	13.734	883.8525	-12.77	-4.92
0.236	0.00	-0.932	15.481	862.6930	-12.47	-4.80
0.238	0.00	-0.899	17.182	839.0361	-12.13	-4.67
0.240	0.00	-0.863	18.834	812.9502	-11.75	-4.52
0.242	0.00	-0.824	20.432	784.5107	-11.34	-4.36
0.244	0.00	-0.781	21.970	753.8001	-10.89	-4.19
0.246	0.00	-0.736	23.445	720.9072	-10.42	-4.01
0.248	0.00	-0.688	24.852	685.9273	-9.91	-3.82
0.250	0.00	-0.637	26.186	648.9616	-9.38	-3.61
0.252	0.00	-0.583	27.446	610.1172	-8.82	-3.39
0.254	0.00	-0.527	28.625	569.5064	-8.23	-3.17
0.256	0.00	-0.468	29.722	527.2470	-7.62	-2.93
0.258	0.00	-0.408	30.733	483.4611	-6.99	-2.69
0.260	0.00	-0.346	31.654	438.2757	-6.33	-2.44
0.262	0.00	-0.281	32.484	391.8214	-5.66	-2.18
0.264	0.00	-0.216	33.221	344.2328	-4.97	-1.92
0.266	0.00	-0.149	33.860	295.6476	-4.27	-1.64
0.268	0.00	-0.080	34.402	246.2066	-3.56	-1.37
0.270	0.00	-0.011	34.845	196.0527	-2.83	-1.09

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.272	0.00	0.059	35.186	145.3313	-2.10	-0.81
0.274	0.00	0.130	35.425	94.1892	-1.36	-0.52
0.276	0.00	0.201	35.562	42.7744	-0.62	-0.24
0.278	0.00	0.272	35.596	-8.7643	0.13	0.05
0.280	0.00	0.343	35.527	-60.2776	0.87	0.34
0.282	0.00	0.414	35.355	-111.6164	1.61	0.62
0.284	0.00	0.484	35.081	-162.6320	2.35	0.90
0.286	0.00	0.554	34.705	-213.1769	3.08	1.19
0.288	0.00	0.623	34.229	-263.1045	3.80	1.46
0.290	0.00	0.691	33.654	-312.2705	4.51	1.74
0.292	0.00	0.758	32.981	-360.5325	5.21	2.01
0.294	0.00	0.823	32.213	-407.7507	5.89	2.27
0.296	0.00	0.886	31.351	-453.7885	6.56	2.52
0.298	0.00	0.948	30.399	-498.5125	7.20	2.77
0.300	0.00	1.008	29.359	-541.7934	7.83	3.01
0.302	0.00	1.065	28.233	-583.5057	8.43	3.25
0.304	0.00	1.121	27.026	-623.5288	9.01	3.47
0.306	0.00	1.174	25.741	-661.7468	9.56	3.68
0.308	0.00	1.224	24.381	-698.0490	10.09	3.88
0.310	0.00	1.271	22.951	-732.3304	10.58	4.07
0.312	0.00	1.315	21.454	-764.4916	11.05	4.25
0.314	0.00	1.357	19.895	-794.4397	11.48	4.42
0.316	0.00	1.395	18.278	-822.0878	11.88	4.57
0.318	0.00	1.430	16.609	-847.3560	12.25	4.71
0.320	0.00	1.461	14.891	-870.1710	12.58	4.84
0.322	0.00	1.489	13.131	-890.4670	12.87	4.95
0.324	0.00	1.514	11.332	-908.1850	13.12	5.05
0.326	0.00	1.535	9.501	-923.2738	13.34	5.14
0.328	0.00	1.552	7.642	-935.6897	13.52	5.21
0.330	0.00	1.565	5.761	-945.3968	13.66	5.26
0.332	0.00	1.575	3.863	-952.3669	13.76	5.30
0.334	0.00	1.581	1.954	-956.5799	13.82	5.32
0.336	0.00	1.583	0.039	-958.0237	13.84	5.33
0.338	0.00	1.581	-1.875	-956.6939	13.83	5.32
0.340	0.00	1.575	-3.785	-952.5945	13.77	5.30
0.342	0.00	1.566	-5.683	-945.7373	13.67	5.26
0.344	0.00	1.552	-7.565	-936.1422	13.53	5.21
0.346	0.00	1.535	-9.425	-923.8370	13.35	5.14
0.348	0.00	1.515	-11.258	-908.8573	13.13	5.06
0.350	0.00	1.490	-13.058	-891.2464	12.88	4.96
0.352	0.00	1.463	-14.820	-871.0553	12.59	4.85
0.354	0.00	1.431	-16.539	-848.3425	12.26	4.72
0.356	0.00	1.396	-18.211	-823.1738	11.90	4.58
0.358	0.00	1.358	-19.830	-795.6219	11.50	4.43
0.360	0.00	1.317	-21.391	-765.7667	11.07	4.26
0.362	0.00	1.273	-22.890	-733.6947	10.60	4.08
0.364	0.00	1.226	-24.324	-699.4985	10.11	3.89
0.366	0.00	1.176	-25.686	-663.2773	9.59	3.69
0.368	0.00	1.123	-26.975	-625.1360	9.03	3.48
0.370	0.00	1.068	-28.185	-585.1848	8.46	3.26
0.372	0.00	1.010	-29.314	-543.5395	7.85	3.02
0.374	0.00	0.951	-30.358	-500.3207	7.23	2.78

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.376	0.00	0.889	-31.314	-455.6534	6.58	2.54
0.378	0.00	0.825	-32.179	-409.6671	5.92	2.28
0.380	0.00	0.760	-32.951	-362.4947	5.24	2.02
0.382	0.00	0.694	-33.628	-314.2729	4.54	1.75
0.384	0.00	0.626	-34.207	-265.1413	3.83	1.48
0.386	0.00	0.557	-34.688	-215.2421	3.11	1.20
0.388	0.00	0.487	-35.068	-164.7197	2.38	0.92
0.390	0.00	0.417	-35.346	-113.7205	1.64	0.63
0.392	0.00	0.346	-35.522	-62.3921	0.90	0.35
0.394	0.00	0.275	-35.596	-10.8831	0.16	0.06
0.396	0.00	0.203	-35.566	40.6575	-0.59	-0.23
0.398	0.00	0.132	-35.433	92.0804	-1.33	-0.51
0.400	0.00	0.062	-35.198	143.2366	-2.07	-0.80
0.402	0.00	-0.008	-34.861	193.9782	-2.80	-1.08
0.404	0.00	-0.078	-34.422	244.1583	-3.53	-1.36
0.406	0.00	-0.146	-33.885	293.6315	-4.24	-1.63
0.408	0.00	-0.213	-33.249	342.2546	-4.95	-1.90
0.410	0.00	-0.279	-32.517	389.8869	-5.63	-2.17
0.412	0.00	-0.343	-31.690	436.3905	-6.31	-2.43
0.414	0.00	-0.406	-30.772	481.6307	-6.96	-2.68
0.416	0.00	-0.466	-29.765	525.4766	-7.59	-2.92
0.418	0.00	-0.525	-28.672	567.8012	-8.21	-3.16
0.420	0.00	-0.581	-27.496	608.4821	-8.79	-3.39
0.422	0.00	-0.634	-26.240	647.4013	-9.36	-3.60
0.424	0.00	-0.686	-24.908	684.4464	-9.89	-3.81
0.426	0.00	-0.734	-23.504	719.5100	-10.40	-4.00
0.428	0.00	-0.780	-22.032	752.4905	-10.87	-4.19
0.430	0.00	-0.822	-20.496	783.2926	-11.32	-4.36
0.432	0.00	-0.862	-18.901	811.8271	-11.73	-4.52
0.434	0.00	-0.898	-17.251	838.0113	-12.11	-4.66
0.436	0.00	-0.930	-15.551	861.7695	-12.45	-4.79
0.438	0.00	-0.960	-13.807	883.0328	-12.76	-4.91
0.440	0.00	-0.986	-12.022	901.7398	-13.03	-5.02
0.442	0.00	-1.008	-10.202	917.8362	-13.26	-5.11
0.444	0.00	-1.026	-8.353	931.2755	-13.46	-5.18
0.446	0.00	-1.041	-6.480	942.0187	-13.61	-5.24
0.448	0.00	-1.052	-4.588	950.0347	-13.73	-5.29
0.450	0.00	-1.060	-2.682	955.3004	-13.81	-5.32
0.452	0.00	-1.063	-0.769	957.8005	-13.84	-5.33
0.454	0.00	-1.063	1.146	957.5277	-13.84	-5.33
0.456	0.00	-1.059	3.058	954.4829	-13.79	-5.31
0.458	0.00	-1.050	4.961	948.6749	-13.71	-5.28
0.460	0.00	-1.039	6.850	940.1204	-13.59	-5.23
0.462	0.00	-1.023	8.719	928.8443	-13.42	-5.17
0.464	0.00	-1.004	10.563	914.8791	-13.22	-5.09
0.466	0.00	-0.981	12.376	898.2654	-12.98	-5.00
0.468	0.00	-0.954	14.153	879.0512	-12.70	-4.89
0.470	0.00	-0.924	15.889	857.2921	-12.39	-4.77
0.472	0.00	-0.891	17.580	833.0511	-12.04	-4.63
0.474	0.00	-0.854	19.219	806.3985	-11.65	-4.49
0.476	0.00	-0.814	20.803	777.4113	-11.23	-4.33
0.478	0.00	-0.771	22.327	746.1735	-10.78	-4.15
0.480	0.00	-0.725	23.786	712.7756	-10.30	-3.97
0.482	0.00	-0.676	25.176	677.3141	-9.79	-3.77
0.484	0.00	-0.624	26.493	639.8918	-9.25	-3.56
0.486	0.00	-0.570	27.733	600.6170	-8.68	-3.34
0.488	0.00	-0.513	28.894	559.6035	-8.09	-3.11
0.490	0.00	-0.454	29.970	516.9698	-7.47	-2.88
0.492	0.00	-0.393	30.960	472.8396	-6.83	-2.63
0.494	0.00	-0.331	31.860	427.3404	-6.18	-2.38

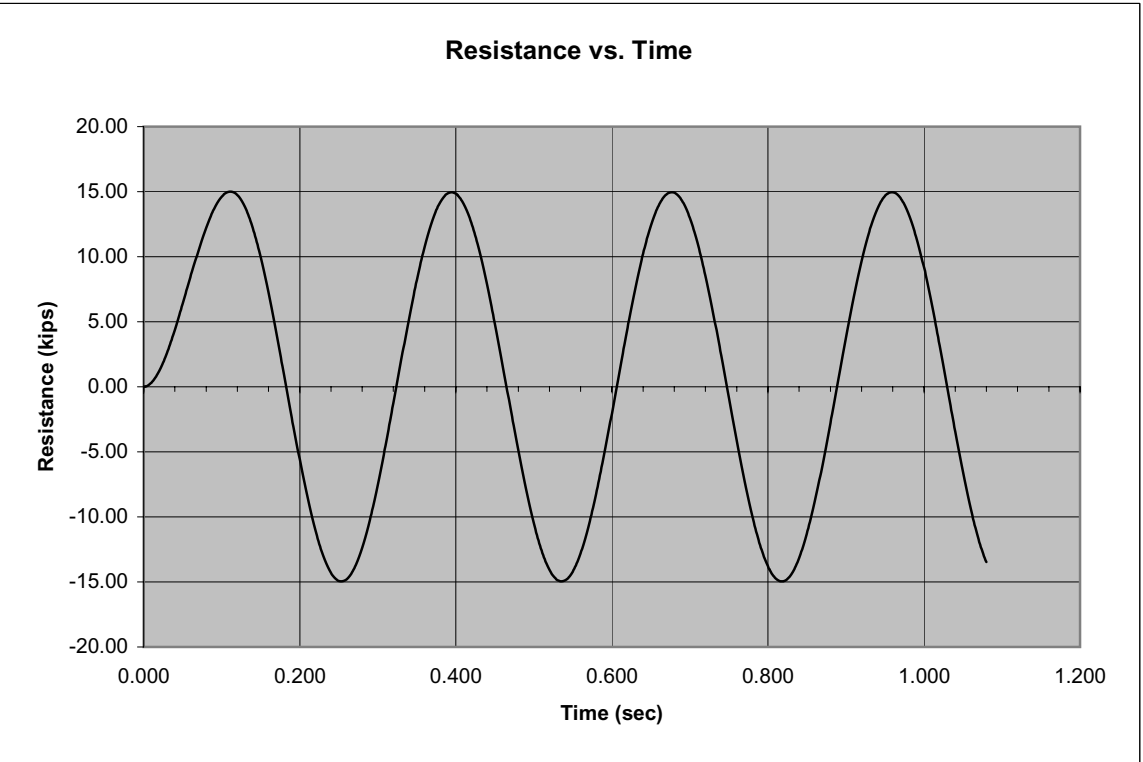
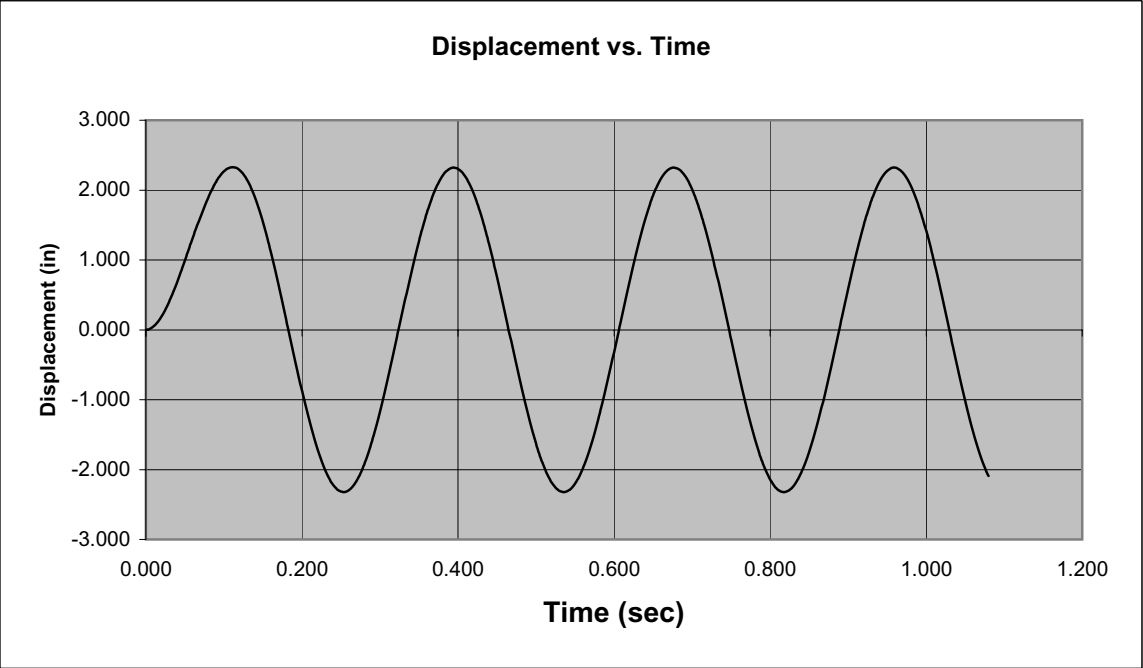
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.496	0.00	-0.266	32.668	380.6041	-5.50	-2.12
0.498	0.00	-0.200	33.381	332.7660	-4.81	-1.85
0.500	0.00	-0.133	33.998	283.9645	-4.10	-1.58
0.502	0.00	-0.064	34.516	234.3409	-3.39	-1.30
0.504	0.00	0.005	34.935	184.0389	-2.66	-1.02
0.506	0.00	0.076	35.252	133.2040	-1.92	-0.74
0.508	0.00	0.146	35.467	81.9836	-1.18	-0.46
0.510	0.00	0.217	35.580	30.5258	-0.44	-0.17
0.512	0.00	0.289	35.589	-21.0203	0.30	0.12
0.514	0.00	0.360	35.496	-72.5056	1.05	0.40
0.516	0.00	0.431	35.300	-123.7810	1.79	0.69
0.518	0.00	0.501	35.001	-174.6980	2.52	0.97
0.520	0.00	0.571	34.601	-225.1093	3.25	1.25
0.522	0.00	0.639	34.101	-274.8689	3.97	1.53
0.524	0.00	0.707	33.503	-323.8328	4.68	1.80
0.526	0.00	0.773	32.807	-371.8591	5.37	2.07
0.528	0.00	0.838	32.016	-418.8089	6.05	2.33
0.530	0.00	0.901	31.133	-464.5463	6.71	2.58
0.532	0.00	0.962	30.159	-508.9388	7.35	2.83
0.534	0.00	1.022	29.099	-551.8579	7.98	3.07
0.536	0.00	1.079	27.954	-593.1794	8.57	3.30
0.538	0.00	1.134	26.728	-632.7836	9.14	3.52
0.540	0.00	1.186	25.424	-670.5559	9.69	3.73
0.542	0.00	1.235	24.047	-706.3869	10.21	3.93
0.544	0.00	1.282	22.601	-740.1730	10.70	4.12
0.546	0.00	1.326	21.089	-771.8162	11.15	4.29
0.548	0.00	1.366	19.516	-801.2250	11.58	4.46
0.550	0.00	1.404	17.886	-828.3142	11.97	4.61
0.552	0.00	1.438	16.205	-853.0055	12.33	4.75
0.554	0.00	1.468	14.477	-875.2273	12.65	4.87
0.556	0.00	1.496	12.706	-894.9154	12.93	4.98
0.558	0.00	1.519	10.900	-912.0126	13.18	5.07
0.560	0.00	1.539	9.061	-926.4695	13.39	5.15
0.562	0.00	1.555	7.196	-938.2443	13.56	5.22
0.564	0.00	1.568	5.311	-947.3029	13.69	5.27
0.566	0.00	1.577	3.410	-953.6191	13.78	5.31
0.568	0.00	1.582	1.499	-957.1745	13.83	5.33
0.570	0.00	1.583	-0.416	-957.9588	13.84	5.33
0.572	0.00	1.580	-2.330	-955.9699	13.82	5.32
0.574	0.00	1.573	-4.237	-951.2134	13.75	5.29
0.576	0.00	1.563	-6.132	-943.7031	13.64	5.25
0.578	0.00	1.549	-8.009	-933.4609	13.49	5.19
0.580	0.00	1.531	-9.863	-920.5162	13.30	5.12
0.582	0.00	1.509	-11.689	-904.9066	13.08	5.03
0.584	0.00	1.484	-13.480	-886.6773	12.81	4.93
0.586	0.00	1.455	-15.233	-865.8811	12.51	4.82
0.588	0.00	1.423	-16.941	-842.5782	12.18	4.69
0.590	0.00	1.388	-18.601	-816.8359	11.80	4.54
0.592	0.00	1.349	-20.206	-788.7290	11.40	4.39
0.594	0.00	1.307	-21.753	-758.3386	10.96	4.22
0.596	0.00	1.262	-23.237	-725.7528	10.49	4.04
0.598	0.00	1.214	-24.654	-691.0660	9.99	3.84
0.600	0.00	1.163	-26.000	-654.3786	9.46	3.64
0.602	0.00	1.110	-27.270	-615.7967	8.90	3.43
0.604	0.00	1.054	-28.461	-575.4320	8.32	3.20
0.606	0.00	0.996	-29.570	-533.4015	7.71	2.97
0.608	0.00	0.936	-30.593	-489.8268	7.08	2.73
0.610	0.00	0.874	-31.528	-444.8340	6.43	2.47
0.612	0.00	0.810	-32.371	-398.5535	5.76	2.22
0.614	0.00	0.745	-33.121	-351.1191	5.07	1.95
0.616	0.00	0.678	-33.775	-302.6682	4.37	1.68

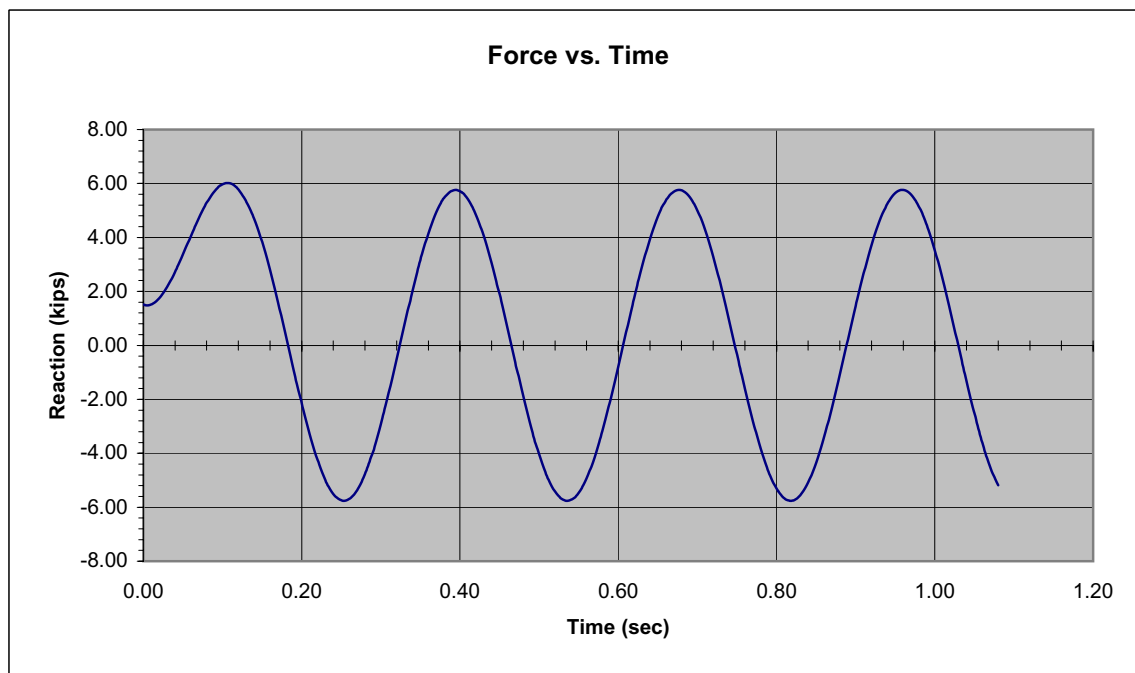
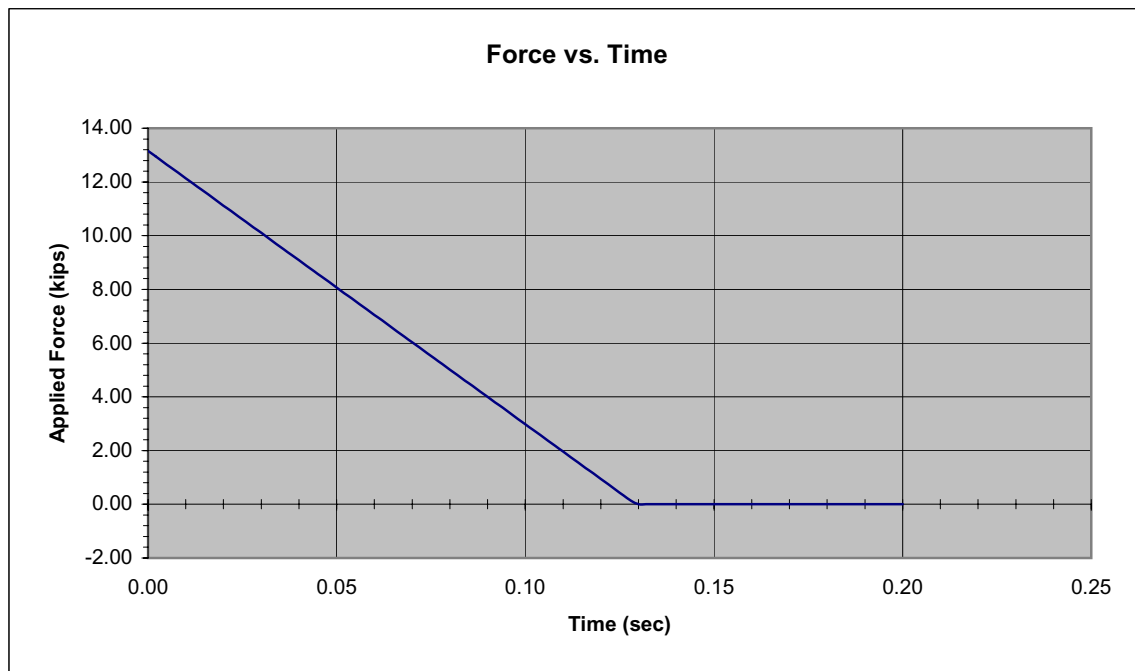
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.618	0.00	0.610	-34.331	-253.3411	3.66	1.41
0.620	0.00	0.540	-34.787	-203.2806	2.94	1.13
0.622	0.00	0.470	-35.143	-152.6315	2.21	0.85
0.624	0.00	0.400	-35.397	-101.5406	1.47	0.56
0.626	0.00	0.329	-35.549	-50.1558	0.72	0.28
0.628	0.00	0.258	-35.598	1.3743	-0.02	-0.01
0.630	0.00	0.187	-35.544	52.9004	-0.76	-0.29
0.632	0.00	0.116	-35.386	104.2733	-1.51	-0.58
0.634	0.00	0.045	-35.127	155.3444	-2.24	-0.86
0.636	0.00	-0.025	-34.765	205.9657	-2.98	-1.15
0.638	0.00	-0.094	-34.304	255.9908	-3.70	-1.42
0.640	0.00	-0.162	-33.742	305.2748	-4.41	-1.70
0.642	0.00	-0.229	-33.083	353.6750	-5.11	-1.97
0.644	0.00	-0.294	-32.329	401.0513	-5.80	-2.23
0.646	0.00	-0.358	-31.480	447.2665	-6.46	-2.49
0.648	0.00	-0.420	-30.541	492.1869	-7.11	-2.74
0.650	0.00	-0.480	-29.513	535.6825	-7.74	-2.98
0.652	0.00	-0.538	-28.400	577.6272	-8.35	-3.21
0.654	0.00	-0.594	-27.204	617.8997	-8.93	-3.44
0.656	0.00	-0.647	-25.930	656.3834	-9.49	-3.65
0.658	0.00	-0.697	-24.580	692.9668	-10.01	-3.86
0.660	0.00	-0.745	-23.160	727.5441	-10.51	-4.05
0.662	0.00	-0.790	-21.672	760.0151	-10.98	-4.23
0.664	0.00	-0.832	-20.122	790.2859	-11.42	-4.40
0.666	0.00	-0.870	-18.514	818.2688	-11.83	-4.55
0.668	0.00	-0.906	-16.851	843.8828	-12.20	-4.70
0.670	0.00	-0.938	-15.140	867.0537	-12.53	-4.82
0.672	0.00	-0.966	-13.386	887.7145	-12.83	-4.94
0.674	0.00	-0.991	-11.592	905.8054	-13.09	-5.04
0.676	0.00	-1.013	-9.765	921.2740	-13.31	-5.13
0.678	0.00	-1.030	-7.910	934.0754	-13.50	-5.20
0.680	0.00	-1.044	-6.031	944.1727	-13.64	-5.25
0.682	0.00	-1.054	-4.136	951.5366	-13.75	-5.29
0.684	0.00	-1.061	-2.228	956.1458	-13.82	-5.32
0.686	0.00	-1.063	-0.314	957.9870	-13.84	-5.33
0.688	0.00	-1.062	1.601	957.0548	-13.83	-5.32
0.690	0.00	-1.057	3.511	953.3519	-13.78	-5.30
0.692	0.00	-1.048	5.412	946.8890	-13.68	-5.27
0.694	0.00	-1.035	7.296	937.6848	-13.55	-5.22
0.696	0.00	-1.019	9.160	925.7661	-13.38	-5.15
0.698	0.00	-0.999	10.997	911.1673	-13.17	-5.07
0.700	0.00	-0.975	12.802	893.9306	-12.92	-4.97
0.702	0.00	-0.948	14.570	874.1060	-12.63	-4.86
0.704	0.00	-0.917	16.296	851.7508	-12.31	-4.74
0.706	0.00	-0.882	17.974	826.9298	-11.95	-4.60
0.708	0.00	-0.845	19.601	799.7149	-11.56	-4.45
0.710	0.00	-0.804	21.171	770.1847	-11.13	-4.29
0.712	0.00	-0.760	22.680	738.4249	-10.67	-4.11
0.714	0.00	-0.713	24.122	704.5273	-10.18	-3.92
0.716	0.00	-0.664	25.496	668.5901	-9.66	-3.72
0.718	0.00	-0.611	26.795	630.7173	-9.11	-3.51
0.720	0.00	-0.557	28.017	591.0186	-8.54	-3.29
0.722	0.00	-0.499	29.157	549.6089	-7.94	-3.06
0.724	0.00	-0.440	30.213	506.6080	-7.32	-2.82
0.726	0.00	-0.379	31.182	462.1406	-6.68	-2.57
0.728	0.00	-0.315	32.061	416.3352	-6.02	-2.32
0.730	0.00	-0.250	32.846	369.3245	-5.34	-2.05
0.732	0.00	-0.184	33.537	321.2447	-4.64	-1.79
0.734	0.00	-0.116	34.130	272.2348	-3.93	-1.51
0.736	0.00	-0.048	34.625	222.4368	-3.21	-1.24
0.738	0.00	0.022	35.020	171.9948	-2.49	-0.96

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.740	0.00	0.092	35.313	121.0550	-1.75	-0.67
0.742	0.00	0.163	35.503	69.7646	-1.01	-0.39
0.744	0.00	0.234	35.591	18.2723	-0.26	-0.10
0.746	0.00	0.306	35.576	-33.2729	0.48	0.19
0.748	0.00	0.377	35.458	-84.7218	1.22	0.47
0.750	0.00	0.447	35.238	-135.9254	1.96	0.76
0.752	0.00	0.518	34.915	-186.7355	2.70	1.04
0.754	0.00	0.587	34.491	-237.0050	3.43	1.32
0.756	0.00	0.655	33.968	-286.5883	4.14	1.59
0.758	0.00	0.723	33.346	-335.3420	4.85	1.87
0.760	0.00	0.789	32.627	-383.1249	5.54	2.13
0.762	0.00	0.853	31.814	-429.7986	6.21	2.39
0.764	0.00	0.916	30.909	-475.2281	6.87	2.64
0.766	0.00	0.977	29.915	-519.2818	7.50	2.89
0.768	0.00	1.036	28.834	-561.8321	8.12	3.13
0.770	0.00	1.092	27.669	-602.7559	8.71	3.35
0.772	0.00	1.146	26.425	-641.9348	9.28	3.57
0.774	0.00	1.198	25.103	-679.2552	9.82	3.78
0.776	0.00	1.247	23.709	-714.6092	10.33	3.98
0.778	0.00	1.292	22.247	-747.8944	10.81	4.16
0.780	0.00	1.335	20.720	-779.0144	11.26	4.33
0.782	0.00	1.375	19.133	-807.8791	11.67	4.49
0.784	0.00	1.412	17.491	-834.4051	12.06	4.64
0.786	0.00	1.445	15.798	-858.5154	12.41	4.78
0.788	0.00	1.475	14.059	-880.1403	12.72	4.90
0.790	0.00	1.501	12.280	-899.2172	12.99	5.00
0.792	0.00	1.524	10.465	-915.6909	13.23	5.09
0.794	0.00	1.543	8.620	-929.5136	13.43	5.17
0.796	0.00	1.559	6.750	-940.6454	13.59	5.23
0.798	0.00	1.570	4.860	-949.0540	13.72	5.28
0.800	0.00	1.578	2.956	-954.7151	13.80	5.31
0.802	0.00	1.582	1.044	-957.6123	13.84	5.33
0.804	0.00	1.582	-0.871	-957.7371	13.84	5.33
0.806	0.00	1.579	-2.784	-955.0893	13.80	5.31
0.808	0.00	1.571	-4.689	-949.6766	13.72	5.28
0.810	0.00	1.560	-6.580	-941.5145	13.61	5.24
0.812	0.00	1.545	-8.452	-930.6267	13.45	5.18
0.814	0.00	1.526	-10.300	-917.0447	13.25	5.10
0.816	0.00	1.504	-12.118	-900.8079	13.02	5.01
0.818	0.00	1.478	-13.901	-881.9632	12.75	4.91
0.820	0.00	1.448	-15.643	-860.5652	12.44	4.79
0.822	0.00	1.415	-17.340	-836.6759	12.09	4.66
0.824	0.00	1.379	-18.988	-810.3644	11.71	4.51
0.826	0.00	1.339	-20.580	-781.7069	11.30	4.35
0.828	0.00	1.296	-22.112	-750.7863	10.85	4.18
0.830	0.00	1.251	-23.581	-717.6922	10.37	3.99
0.832	0.00	1.202	-24.981	-682.5204	9.86	3.80
0.834	0.00	1.151	-26.309	-645.3727	9.33	3.59
0.836	0.00	1.097	-27.560	-606.3566	8.76	3.37
0.838	0.00	1.041	-28.732	-565.5851	8.17	3.15
0.840	0.00	0.982	-29.821	-523.1762	7.56	2.91
0.842	0.00	0.921	-30.824	-479.2527	6.93	2.67
0.844	0.00	0.859	-31.737	-433.9418	6.27	2.41
0.846	0.00	0.795	-32.558	-387.3746	5.60	2.16
0.848	0.00	0.729	-33.285	-339.6860	4.91	1.89
0.850	0.00	0.662	-33.916	-291.0140	4.21	1.62
0.852	0.00	0.593	-34.448	-241.4995	3.49	1.34
0.854	0.00	0.524	-34.881	-191.2858	2.76	1.06
0.856	0.00	0.454	-35.213	-140.5184	2.03	0.78
0.858	0.00	0.383	-35.443	-89.3441	1.29	0.50
0.860	0.00	0.312	-35.570	-37.9112	0.55	0.21

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.862	0.00	0.241	-35.594	13.6314	-0.20	-0.08
0.864	0.00	0.170	-35.516	65.1346	-0.94	-0.36
0.866	0.00	0.099	-35.334	116.4492	-1.68	-0.65
0.868	0.00	0.028	-35.050	167.4267	-2.42	-0.93
0.870	0.00	-0.041	-34.665	217.9195	-3.15	-1.21
0.872	0.00	-0.110	-34.179	267.7814	-3.87	-1.49
0.874	0.00	-0.178	-33.594	316.8681	-4.58	-1.76
0.876	0.00	-0.244	-32.912	365.0374	-5.28	-2.03
0.878	0.00	-0.310	-32.135	412.1500	-5.96	-2.29
0.880	0.00	-0.373	-31.265	458.0694	-6.62	-2.55
0.882	0.00	-0.435	-30.304	502.6626	-7.26	-2.80
0.884	0.00	-0.494	-29.256	545.8007	-7.89	-3.04
0.886	0.00	-0.552	-28.123	587.3586	-8.49	-3.27
0.888	0.00	-0.607	-26.908	627.2162	-9.06	-3.49
0.890	0.00	-0.659	-25.616	665.2579	-9.61	-3.70
0.892	0.00	-0.709	-24.249	701.3737	-10.14	-3.90
0.894	0.00	-0.756	-22.812	735.4591	-10.63	-4.09
0.896	0.00	-0.800	-21.309	767.4152	-11.09	-4.27
0.898	0.00	-0.841	-19.745	797.1497	-11.52	-4.44
0.900	0.00	-0.879	-18.123	824.5765	-11.92	-4.59
0.902	0.00	-0.914	-16.449	849.6160	-12.28	-4.73
0.904	0.00	-0.945	-14.727	872.1960	-12.60	-4.85
0.906	0.00	-0.973	-12.963	892.2509	-12.89	-4.96
0.908	0.00	-0.997	-11.161	909.7227	-13.15	-5.06
0.910	0.00	-1.017	-9.326	924.5609	-13.36	-5.14
0.912	0.00	-1.034	-7.465	936.7225	-13.54	-5.21
0.914	0.00	-1.047	-5.582	946.1722	-13.67	-5.26
0.916	0.00	-1.056	-3.683	952.8828	-13.77	-5.30
0.918	0.00	-1.062	-1.773	956.8348	-13.83	-5.32
0.920	0.00	-1.063	0.141	958.0167	-13.84	-5.33
0.922	0.00	-1.061	2.056	956.4251	-13.82	-5.32
0.924	0.00	-1.055	3.964	952.0647	-13.76	-5.30
0.926	0.00	-1.045	5.861	944.9481	-13.66	-5.26
0.928	0.00	-1.032	7.741	935.0958	-13.51	-5.20
0.930	0.00	-1.014	9.599	922.5364	-13.33	-5.13
0.932	0.00	-0.993	11.429	907.3063	-13.11	-5.05
0.934	0.00	-0.969	13.226	889.4494	-12.85	-4.95
0.936	0.00	-0.940	14.984	869.0177	-12.56	-4.84
0.938	0.00	-0.909	16.699	846.0701	-12.23	-4.71
0.940	0.00	-0.874	18.366	820.6731	-11.86	-4.57
0.942	0.00	-0.835	19.980	792.9003	-11.46	-4.41
0.944	0.00	-0.794	21.535	762.8320	-11.02	-4.24
0.946	0.00	-0.749	23.029	730.5553	-10.56	-4.06
0.948	0.00	-0.702	24.455	696.1637	-10.06	-3.87
0.950	0.00	-0.651	25.811	659.7566	-9.53	-3.67
0.952	0.00	-0.599	27.093	621.4395	-8.98	-3.46
0.954	0.00	-0.543	28.295	581.3234	-8.40	-3.23
0.956	0.00	-0.485	29.416	539.5243	-7.80	-3.00
0.958	0.00	-0.426	30.452	496.1633	-7.17	-2.76
0.960	0.00	-0.364	31.399	451.3659	-6.52	-2.51
0.962	0.00	-0.300	32.256	405.2618	-5.86	-2.25
0.964	0.00	-0.235	33.019	357.9845	-5.17	-1.99
0.966	0.00	-0.168	33.687	309.6708	-4.48	-1.72
0.968	0.00	-0.100	34.257	260.4606	-3.76	-1.45
0.970	0.00	-0.031	34.728	210.4963	-3.04	-1.17
0.972	0.00	0.039	35.098	159.9227	-2.31	-0.89
0.974	0.00	0.109	35.367	108.8861	-1.57	-0.61
0.976	0.00	0.180	35.534	57.5342	-0.83	-0.32
0.978	0.00	0.251	35.597	6.0158	-0.09	-0.03
0.980	0.00	0.323	35.558	-45.5200	0.66	0.25
0.982	0.00	0.393	35.415	-96.9240	1.40	0.54

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.984	0.00	0.464	35.170	-148.0475	2.14	0.82
0.986	0.00	0.534	34.823	-198.7423	2.87	1.11
0.988	0.00	0.603	34.376	-248.8618	3.60	1.38
0.990	0.00	0.672	33.829	-298.2608	4.31	1.66
0.992	0.00	0.739	33.184	-346.7964	5.01	1.93
0.994	0.00	0.804	32.443	-394.3280	5.70	2.19
0.996	0.00	0.868	31.608	-440.7180	6.37	2.45
0.998	0.00	0.931	30.681	-485.8321	7.02	2.70
1.000	0.00	0.991	29.666	-529.5397	7.65	2.95
1.002	0.00	1.049	28.564	-571.7143	8.26	3.18
1.004	0.00	1.105	27.380	-612.2338	8.85	3.41
1.006	0.00	1.159	26.117	-650.9808	9.41	3.62
1.008	0.00	1.210	24.778	-687.8433	9.94	3.83
1.010	0.00	1.258	23.368	-722.7144	10.44	4.02
1.012	0.00	1.303	21.890	-755.4933	10.92	4.20
1.014	0.00	1.345	20.348	-786.0850	11.36	4.37
1.016	0.00	1.384	18.748	-814.4010	11.77	4.53
1.018	0.00	1.420	17.093	-840.3593	12.14	4.68
1.020	0.00	1.453	15.389	-863.8848	12.48	4.81
1.022	0.00	1.482	13.640	-884.9093	12.79	4.92
1.024	0.00	1.507	11.851	-903.3719	13.05	5.03
1.026	0.00	1.529	10.029	-919.2193	13.28	5.11
1.028	0.00	1.547	8.177	-932.4056	13.47	5.19
1.030	0.00	1.562	6.302	-942.8925	13.63	5.25
1.032	0.00	1.573	4.408	-950.6498	13.74	5.29
1.034	0.00	1.579	2.502	-955.6548	13.81	5.32
1.036	0.00	1.583	0.589	-957.8933	13.84	5.33
1.038	0.00	1.582	-1.327	-957.3587	13.84	5.33
1.040	0.00	1.577	-3.238	-954.0525	13.79	5.31
1.042	0.00	1.569	-5.140	-947.9843	13.70	5.27
1.044	0.00	1.557	-7.027	-939.1716	13.57	5.23
1.046	0.00	1.541	-8.894	-927.6401	13.41	5.16
1.048	0.00	1.521	-10.735	-913.4231	13.20	5.08
1.050	0.00	1.498	-12.545	-896.5616	12.96	4.99
1.052	0.00	1.471	-14.319	-877.1046	12.68	4.88
1.054	0.00	1.441	-16.051	-855.1084	12.36	4.76
1.056	0.00	1.407	-17.737	-830.6367	12.00	4.62
1.058	0.00	1.370	-19.371	-803.7602	11.62	4.47
1.060	0.00	1.329	-20.950	-774.5568	11.19	4.31
1.062	0.00	1.286	-22.467	-743.1111	10.74	4.13
1.064	0.00	1.239	-23.920	-709.5141	10.25	3.95
1.066	0.00	1.190	-25.303	-673.8630	9.74	3.75
1.068	0.00	1.138	-26.613	-636.2611	9.19	3.54
1.070	0.00	1.084	-27.846	-596.8172	8.62	3.32
1.072	0.00	1.027	-28.999	-555.6455	8.03	3.09
1.074	0.00	0.968	-30.067	-512.8652	7.41	2.85
1.076	0.00	0.907	-31.049	-468.6002	6.77	2.61
1.078	0.00	0.844	-31.940	-422.9786	6.11	2.35
1.080	0.00	0.779	-32.740	-376.1324	5.44	2.09





Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

WALL PANELS W/ PERSONNEL DOOR OPENING

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : WALL PROPERTIES WITH PERSONNEL DOOR

Sheet no. : of
 Date : 6-Jun-04
 Design by : MGG
 Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Roof Dead Load = 8.470 KN/m²
 = 176.90 psf

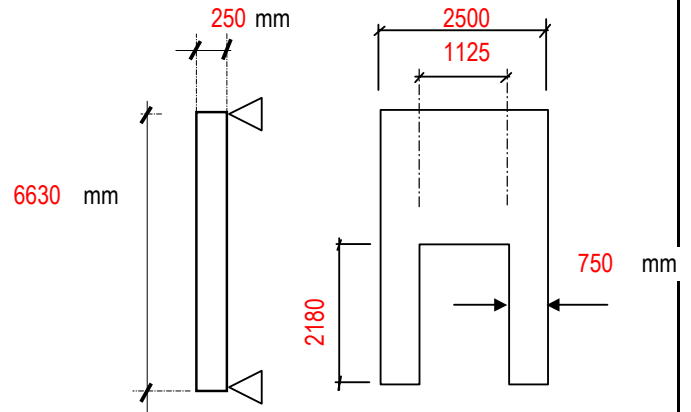
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
 Reinf., f_y = 414 Mpa

PANEL DATA :

Panel Width, width = 2500 mm
 Wall Span, lb = 6630 mm
 Wall, t_{TRIAL} = 250 mm
 (Panel Width - Opening) ; b = 1500 mm
 Roof span : = 15.22 m

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure, P_s = 4.20 psi (SEE LOADING CALCULATION)
 Effective blast time duration, T_e = 129 msec

Moment of Inertia, I_g = 4692.40 in⁴ (1953125000 mm⁴)
 Area of Bar ($\emptyset$), 25 mm $\emptyset$ A_b = 0.7609 in² (491 mm²)
 Number of Bars per side of opening N = 10.0000 in
 Area of Rebar PER side, A_s = 7.6086 in² (4909 mm²)
 Effective Depth, $d_c = d_s$ = 8.3425 in (212 mm)
 $f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for bending) = 5954.3 psi (41 Mpa.)
 $f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for shear) = 5504.0 psi (38 Mpa.)
 $f'_{dy} = (SIF) \times (DIF) \times f_y$ = 77276 psi (533 Mpa.)
 Modulus of Elasticity, $E_{dc} = 57000 \times (f'_c)^{1/2}$ = 4032041 psi (27801 Mpa.)
 Steel ratio, $p = A_s / b \times d$ = 0.0154 > 200 / f'_{dy} = 0.003
 Stress block, $a = A_s \times f_{dy} / 0.85 \times f'_{dc} \times b$ = 1.9672 in (50 mm)
 Modular ratio, $n = E_s / E_{dm}$ = 7.1940

a.) BENDING RESISTANCE

Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$ = 4326756.36 lb-in (488833 KN-m)
 Resistance at cracked moment, $R_b = 8 \times M_p / L$ = 132608.88 lb (589.84 KN)
 Unit Resistance, $r_b = R_b / (L \times \text{panel width})$ = 5.1616 psi (0.0356 Mpa.)

b.) SHEAR RESISTANCE

Shear, $V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$ = 73101.0 lb (325 KN)
 At Support, $R_s = V_n \times L / (0.5 \times L - t_w)$ = 156186 lb (695 KN)
 Unit Resistance, $r_s = R_s / (L \times \text{panel width})$ = 6.079 psi (0.0419 Mpa.)

∴ Consider 1 ft strip

∴ Dynamic Resistance Controlled by Bending, R_u = 5.1616 psi > 4.20 ∴ O.K!
 16.1677 kips 13.16 kips

Cracked Moment of Inertia:

$n \times A_s$ = 54.7360 in² (35313 mm²)
 $C = -n \times A_s + [n \times A_s * (n \times A_s + 2 \times b \times d)]^{0.5} / b$ = 3.1134 in² (2009 mm²)
 $I_{cr} = b \times C^3 / 3 + n \times A_s * (d - C)^2$ = 2090.76 in⁴ (870239940 mm⁴)

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject : WALL PROPERTIES WITH PERSONNEL DOOR

Sheet no. : of
Date : 6-Jun-04
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Check'd by : SAK

Average Moment of Inertia, $I_a = (I_g + I_{cr}) / 2 = 3391.581 \text{ in}^4$ (1411682470.0 mm⁴)
Effective Stiffness, $K = 384 \times E_{dc} \times I_a / 5 L^3 = 7.1999 \text{ kips / in}$ (per 1 ft strip)
Yield Deflection, $Y_e = R_u / K = 2.24556 \text{ in}$
Wall Mass = wall total weight / panel width / gravity acc. = 0.00591 kip-sec² / in
Roof Mass = Roof DL x unit width x 1/2 roof span / gravity acc. = 0.01144 kip-sec² / in
Total Mass, M = 0.01735 kip-sec² / in
Average Stiffness, $K_{LM} = 0.72000$ FROM ASCE TABLE 6.1
Equivalent Mass, $M_e = K_{LM} \times \text{Beam Mass} = 0.01249 \text{ kip-sec}^2 / \text{in}$
Period of Vibration, $t_n = 2 \times \pi \times (M_e / K)^{0.5} = 0.26172 \text{ sec}$
Time increment, $t_n/10$ $\Delta t = 0.02617 \text{ sec}$; use $\Delta t = 0.00200 \text{ sec}$
 $t_d / t_n = 0.49397$
 $R_u / P_s = 1.22896$

NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	7.200 kips / in
Mass	M	=	0.0174 kip-sec ² / in
Tension resistance	Rut	=	16.17 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	13.16 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.1293 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load	
load (kip)	time (sec)
13.16	0
0	0.1293

Length	266.732 in
y _{max} =	1.978 in
r _{max} =	14.242 kips
μ =	0.881
θ =	0.850 degrees
Rut/Fo =	1.229
θ = < 2 degrees	
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.000	13.16	0.000	0.000	758.2470	0.00	1.51
0.002	12.95	0.002	1.504	745.8910	0.01	1.49
0.004	12.75	0.006	2.982	732.2972	0.04	1.48
0.006	12.55	0.013	4.432	717.4882	0.10	1.48
0.008	12.34	0.024	5.851	701.4886	0.17	1.48
0.010	12.14	0.037	7.237	684.3249	0.26	1.50
0.012	11.93	0.053	8.587	666.0256	0.38	1.52
0.014	11.73	0.071	9.900	646.6210	0.51	1.55
0.016	11.53	0.092	11.173	626.1434	0.66	1.58
0.018	11.32	0.116	12.403	604.6268	0.83	1.62
0.020	11.12	0.142	13.590	582.1068	1.02	1.67
0.022	10.92	0.170	14.731	558.6208	1.22	1.73
0.024	10.71	0.201	15.824	534.2078	1.44	1.79
0.026	10.51	0.233	16.867	508.9084	1.68	1.86
0.028	10.31	0.268	17.859	482.7644	1.93	1.93
0.030	10.10	0.305	18.797	455.8193	2.19	2.01
0.032	9.90	0.343	19.681	428.1178	2.47	2.09
0.034	9.70	0.383	20.509	399.7059	2.76	2.18
0.036	9.49	0.425	21.279	370.6306	3.06	2.27
0.038	9.29	0.469	21.991	340.9404	3.37	2.37
0.040	9.09	0.513	22.642	310.6843	3.69	2.47
0.042	8.88	0.559	23.233	279.9127	4.03	2.57
0.044	8.68	0.606	23.762	248.6766	4.36	2.68
0.046	8.47	0.654	24.227	217.0278	4.71	2.79
0.048	8.27	0.703	24.629	185.0189	5.06	2.90
0.050	8.07	0.753	24.967	152.7029	5.42	3.01
0.052	7.86	0.803	25.240	120.1335	5.78	3.13
0.054	7.66	0.853	25.447	87.3648	6.14	3.25
0.056	7.46	0.905	25.589	54.4511	6.51	3.36
0.058	7.25	0.956	25.665	21.4471	6.88	3.48
0.060	7.05	1.007	25.675	-11.5926	7.25	3.60
0.062	6.85	1.058	25.619	-44.6130	7.62	3.72

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.064	6.64	1.110	25.497	-77.5593	7.99	3.84
0.066	6.44	1.160	25.309	-110.3770	8.35	3.96
0.068	6.24	1.211	25.055	-143.0115	8.72	4.07
0.070	6.03	1.261	24.737	-175.4087	9.08	4.19
0.072	5.83	1.310	24.354	-207.5148	9.43	4.30
0.074	5.63	1.358	23.907	-239.2765	9.78	4.41
0.076	5.42	1.405	23.397	-270.6412	10.12	4.52
0.078	5.22	1.451	22.825	-301.5568	10.45	4.62
0.080	5.01	1.497	22.191	-331.9719	10.77	4.72
0.082	4.81	1.540	21.498	-361.8361	11.09	4.82
0.084	4.61	1.582	20.745	-391.0999	11.39	4.92
0.086	4.40	1.623	19.934	-419.7147	11.69	5.01
0.088	4.20	1.662	19.067	-447.6330	11.97	5.09
0.090	4.00	1.699	18.144	-474.8085	12.24	5.17
0.092	3.79	1.735	17.168	-501.1960	12.49	5.24
0.094	3.59	1.768	16.140	-526.7519	12.73	5.31
0.096	3.39	1.799	15.062	-551.4336	12.95	5.38
0.098	3.18	1.828	13.935	-575.2002	13.16	5.43
0.100	2.98	1.855	12.762	-598.0124	13.36	5.48
0.102	2.78	1.879	11.544	-619.8321	13.53	5.53
0.104	2.57	1.901	10.284	-640.6233	13.69	5.57
0.106	2.37	1.920	8.983	-660.3514	13.83	5.60
0.108	2.17	1.937	7.644	-678.9837	13.95	5.62
0.110	1.96	1.951	6.268	-696.4892	14.05	5.63
0.112	1.76	1.962	4.859	-712.8390	14.13	5.64
0.114	1.56	1.970	3.418	-728.0058	14.19	5.64
0.116	1.35	1.976	1.948	-741.9645	14.22	5.63
0.118	1.15	1.978	0.451	-754.6920	14.24	5.62
0.120	0.94	1.977	-1.070	-766.1671	14.24	5.59
0.122	0.74	1.974	-2.612	-776.3708	14.21	5.56
0.124	0.54	1.967	-4.174	-785.2862	14.16	5.51
0.126	0.33	1.957	-5.752	-792.8984	14.09	5.46
0.128	0.13	1.944	-7.344	-799.1948	14.00	5.40
0.130	0.00	1.928	-8.943	-799.9550	13.88	5.34
0.132	0.00	1.908	-10.535	-791.8709	13.74	5.29
0.134	0.00	1.886	-12.109	-782.4726	13.58	5.23
0.136	0.00	1.860	-13.664	-771.7759	13.39	5.16
0.138	0.00	1.831	-15.195	-759.7985	13.18	5.08
0.140	0.00	1.799	-16.702	-746.5602	12.95	4.99
0.142	0.00	1.764	-18.180	-732.0831	12.70	4.89
0.144	0.00	1.726	-19.629	-716.3911	12.43	4.79
0.146	0.00	1.686	-21.045	-699.5102	12.14	4.67
0.148	0.00	1.642	-22.426	-681.4686	11.82	4.55
0.150	0.00	1.596	-23.769	-662.2961	11.49	4.42
0.152	0.00	1.547	-25.074	-642.0246	11.14	4.29
0.154	0.00	1.496	-26.336	-620.6877	10.77	4.15
0.156	0.00	1.442	-27.555	-598.3207	10.38	4.00
0.158	0.00	1.386	-28.729	-574.9609	9.98	3.84
0.160	0.00	1.327	-29.854	-550.6469	9.55	3.68
0.162	0.00	1.266	-30.930	-525.4192	9.12	3.51
0.164	0.00	1.203	-31.955	-499.3196	8.66	3.34
0.166	0.00	1.138	-32.927	-472.3914	8.20	3.16

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.168	0.00	1.072	-33.844	-444.6793	7.72	2.97
0.170	0.00	1.003	-34.705	-416.2292	7.22	2.78
0.172	0.00	0.933	-35.508	-387.0885	6.72	2.59
0.174	0.00	0.861	-36.252	-357.3054	6.20	2.39
0.176	0.00	0.788	-36.937	-326.9293	5.67	2.18
0.178	0.00	0.713	-37.560	-296.0107	5.14	1.98
0.180	0.00	0.638	-38.120	-264.6010	4.59	1.77
0.182	0.00	0.561	-38.618	-232.7521	4.04	1.55
0.184	0.00	0.483	-39.051	-200.5170	3.48	1.34
0.186	0.00	0.405	-39.419	-167.9491	2.91	1.12
0.188	0.00	0.326	-39.722	-135.1026	2.34	0.90
0.190	0.00	0.246	-39.960	-102.0318	1.77	0.68
0.192	0.00	0.166	-40.130	-68.7917	1.19	0.46
0.194	0.00	0.085	-40.235	-35.4375	0.61	0.24
0.196	0.00	0.005	-40.272	-2.0244	0.04	0.01
0.198	0.00	-0.076	-40.243	31.3919	-0.54	-0.21
0.200	0.00	-0.156	-40.147	64.7563	-1.12	-0.43
0.202	0.00	-0.236	-39.984	98.0131	-1.70	-0.65
0.204	0.00	-0.316	-39.755	131.1073	-2.27	-0.88
0.206	0.00	-0.395	-39.460	163.9839	-2.85	-1.10
0.208	0.00	-0.474	-39.099	196.5884	-3.41	-1.31
0.210	0.00	-0.552	-38.674	228.8667	-3.97	-1.53
0.212	0.00	-0.628	-38.184	260.7652	-4.52	-1.74
0.214	0.00	-0.704	-37.631	292.2310	-5.07	-1.95
0.216	0.00	-0.779	-37.015	323.2118	-5.61	-2.16
0.218	0.00	-0.852	-36.339	353.6563	-6.14	-2.36
0.220	0.00	-0.924	-35.601	383.5138	-6.65	-2.56
0.222	0.00	-0.995	-34.805	412.7350	-7.16	-2.76
0.224	0.00	-1.063	-33.951	441.2713	-7.66	-2.95
0.226	0.00	-1.130	-33.041	469.0753	-8.14	-3.13
0.228	0.00	-1.195	-32.076	496.1008	-8.61	-3.31
0.230	0.00	-1.259	-31.057	522.3032	-9.06	-3.49
0.232	0.00	-1.320	-29.987	547.6388	-9.50	-3.66
0.234	0.00	-1.379	-28.868	572.0656	-9.93	-3.82
0.236	0.00	-1.435	-27.700	595.5431	-10.33	-3.98
0.238	0.00	-1.489	-26.486	618.0323	-10.72	-4.13
0.240	0.00	-1.541	-25.229	639.4959	-11.10	-4.27
0.242	0.00	-1.590	-23.929	659.8983	-11.45	-4.41
0.244	0.00	-1.637	-22.590	679.2057	-11.78	-4.54
0.246	0.00	-1.681	-21.214	697.3859	-12.10	-4.66
0.248	0.00	-1.722	-19.802	714.4089	-12.39	-4.77
0.250	0.00	-1.760	-18.357	730.2463	-12.67	-4.88
0.252	0.00	-1.795	-16.882	744.8720	-12.92	-4.98
0.254	0.00	-1.827	-15.379	758.2615	-13.16	-5.06
0.256	0.00	-1.856	-13.850	770.3928	-13.37	-5.15
0.258	0.00	-1.883	-12.299	781.2456	-13.55	-5.22
0.260	0.00	-1.906	-10.727	790.8020	-13.72	-5.28
0.262	0.00	-1.926	-9.137	799.0461	-13.86	-5.34
0.264	0.00	-1.942	-7.532	805.9643	-13.98	-5.38
0.266	0.00	-1.956	-5.914	811.5449	-14.08	-5.42
0.268	0.00	-1.966	-4.287	815.7789	-14.15	-5.45
0.270	0.00	-1.973	-2.653	818.6591	-14.20	-5.47

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.272	0.00	-1.976	-1.014	820.1808	-14.23	-5.48
0.274	0.00	-1.977	0.627	820.3414	-14.23	-5.48
0.276	0.00	-1.974	2.266	819.1407	-14.21	-5.47
0.278	0.00	-1.968	3.902	816.5807	-14.17	-5.45
0.280	0.00	-1.958	5.531	812.6656	-14.10	-5.43
0.282	0.00	-1.946	7.151	807.4020	-14.01	-5.39
0.284	0.00	-1.930	8.759	800.7985	-13.89	-5.35
0.286	0.00	-1.911	10.353	792.8661	-13.76	-5.30
0.288	0.00	-1.888	11.930	783.6180	-13.60	-5.23
0.290	0.00	-1.863	13.486	773.0695	-13.41	-5.16
0.292	0.00	-1.834	15.021	761.2381	-13.21	-5.08
0.294	0.00	-1.803	16.530	748.1436	-12.98	-5.00
0.296	0.00	-1.768	18.012	733.8075	-12.73	-4.90
0.298	0.00	-1.731	19.464	718.2537	-12.46	-4.80
0.300	0.00	-1.690	20.884	701.5079	-12.17	-4.69
0.302	0.00	-1.647	22.269	683.5981	-11.86	-4.57
0.304	0.00	-1.601	23.617	664.5539	-11.53	-4.44
0.306	0.00	-1.553	24.926	644.4069	-11.18	-4.30
0.308	0.00	-1.502	26.194	623.1905	-10.81	-4.16
0.310	0.00	-1.448	27.418	600.9399	-10.43	-4.01
0.312	0.00	-1.392	28.596	577.6922	-10.02	-3.86
0.314	0.00	-1.334	29.728	553.4858	-9.60	-3.70
0.316	0.00	-1.273	30.809	528.3609	-9.17	-3.53
0.318	0.00	-1.211	31.840	502.3592	-8.72	-3.36
0.320	0.00	-1.146	32.818	475.5239	-8.25	-3.18
0.322	0.00	-1.079	33.741	447.8994	-7.77	-2.99
0.324	0.00	-1.011	34.609	419.5317	-7.28	-2.80
0.326	0.00	-0.941	35.419	390.4678	-6.77	-2.61
0.328	0.00	-0.869	36.170	360.7560	-6.26	-2.41
0.330	0.00	-0.796	36.861	330.4455	-5.73	-2.21
0.332	0.00	-0.722	37.491	299.5866	-5.20	-2.00
0.334	0.00	-0.646	38.059	268.2306	-4.65	-1.79
0.336	0.00	-0.570	38.564	236.4295	-4.10	-1.58
0.338	0.00	-0.492	39.004	204.2360	-3.54	-1.36
0.340	0.00	-0.414	39.380	171.7036	-2.98	-1.15
0.342	0.00	-0.335	39.691	138.8863	-2.41	-0.93
0.344	0.00	-0.255	39.936	105.8385	-1.84	-0.71
0.346	0.00	-0.175	40.114	72.6150	-1.26	-0.49
0.348	0.00	-0.095	40.226	39.2711	-0.68	-0.26
0.350	0.00	-0.014	40.271	5.8620	-0.10	-0.04
0.352	0.00	0.066	40.249	-27.5568	0.48	0.18
0.354	0.00	0.147	40.161	-60.9299	1.06	0.41
0.356	0.00	0.227	40.006	-94.2019	1.63	0.63
0.358	0.00	0.307	39.784	-127.3176	2.21	0.85
0.360	0.00	0.386	39.497	-160.2220	2.78	1.07
0.362	0.00	0.465	39.144	-192.8605	3.35	1.29
0.364	0.00	0.543	38.726	-225.1790	3.91	1.50
0.366	0.00	0.620	38.243	-257.1238	4.46	1.72
0.368	0.00	0.696	37.698	-288.6419	5.01	1.93
0.370	0.00	0.770	37.089	-319.6810	5.55	2.14
0.372	0.00	0.844	36.419	-350.1896	6.08	2.34
0.374	0.00	0.916	35.689	-380.1172	6.60	2.54

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.376	0.00	0.987	34.900	-409.4139	7.10	2.73
0.378	0.00	1.056	34.052	-438.0312	7.60	2.93
0.380	0.00	1.123	33.148	-465.9216	8.08	3.11
0.382	0.00	1.188	32.189	-493.0389	8.55	3.29
0.384	0.00	1.251	31.177	-519.3380	9.01	3.47
0.386	0.00	1.313	30.113	-544.7752	9.45	3.64
0.388	0.00	1.372	28.999	-569.3085	9.88	3.80
0.390	0.00	1.429	27.836	-592.8970	10.29	3.96
0.392	0.00	1.483	26.628	-615.5016	10.68	4.11
0.394	0.00	1.535	25.375	-637.0848	11.05	4.26
0.396	0.00	1.585	24.081	-657.6109	11.41	4.39
0.398	0.00	1.632	22.746	-677.0456	11.75	4.52
0.400	0.00	1.676	21.374	-695.3568	12.06	4.64
0.402	0.00	1.717	19.966	-712.5141	12.36	4.76
0.404	0.00	1.755	18.525	-728.4891	12.64	4.87
0.406	0.00	1.791	17.053	-743.2551	12.90	4.96
0.408	0.00	1.824	15.553	-756.7878	13.13	5.06
0.410	0.00	1.853	14.027	-769.0646	13.34	5.14
0.412	0.00	1.880	12.478	-780.0651	13.53	5.21
0.414	0.00	1.903	10.908	-789.7712	13.70	5.28
0.416	0.00	1.923	9.320	-798.1667	13.85	5.33
0.418	0.00	1.940	7.717	-805.2377	13.97	5.38
0.420	0.00	1.954	6.101	-810.9725	14.07	5.42
0.422	0.00	1.965	4.474	-815.3614	14.15	5.45
0.424	0.00	1.972	2.841	-818.3973	14.20	5.47
0.426	0.00	1.976	1.202	-820.0752	14.23	5.48
0.428	0.00	1.977	-0.438	-820.3921	14.23	5.48
0.430	0.00	1.974	-2.078	-819.3477	14.22	5.47
0.432	0.00	1.969	-3.714	-816.9436	14.17	5.46
0.434	0.00	1.960	-5.345	-813.1838	14.11	5.43
0.436	0.00	1.947	-6.966	-808.0746	14.02	5.40
0.438	0.00	1.932	-8.576	-801.6244	13.91	5.35
0.440	0.00	1.913	-10.171	-793.8440	13.77	5.30
0.442	0.00	1.891	-11.750	-784.7462	13.62	5.24
0.444	0.00	1.866	-13.309	-774.3462	13.43	5.17
0.446	0.00	1.838	-14.846	-762.6611	13.23	5.09
0.448	0.00	1.807	-16.358	-749.7105	13.01	5.01
0.450	0.00	1.772	-17.843	-735.5158	12.76	4.91
0.452	0.00	1.735	-19.299	-720.1005	12.49	4.81
0.454	0.00	1.695	-20.722	-703.4903	12.21	4.70
0.456	0.00	1.652	-22.112	-685.7127	11.90	4.58
0.458	0.00	1.607	-23.464	-666.7971	11.57	4.45
0.460	0.00	1.559	-24.778	-646.7750	11.22	4.32
0.462	0.00	1.508	-26.050	-625.6797	10.86	4.18
0.464	0.00	1.454	-27.279	-603.5460	10.47	4.03
0.466	0.00	1.399	-28.463	-580.4108	10.07	3.88
0.468	0.00	1.341	-29.600	-556.3125	9.65	3.72
0.470	0.00	1.280	-30.688	-531.2909	9.22	3.55
0.472	0.00	1.218	-31.724	-505.3878	8.77	3.38
0.474	0.00	1.153	-32.708	-478.6459	8.30	3.20
0.476	0.00	1.087	-33.638	-451.1098	7.83	3.01
0.478	0.00	1.019	-34.512	-422.8250	7.34	2.82
0.480	0.00	0.949	-35.329	-393.8387	6.83	2.63
0.482	0.00	0.878	-36.087	-364.1987	6.32	2.43
0.484	0.00	0.805	-36.785	-333.9544	5.79	2.23
0.486	0.00	0.731	-37.422	-303.1559	5.26	2.03
0.488	0.00	0.655	-37.997	-271.8543	4.72	1.82
0.490	0.00	0.579	-38.509	-240.1016	4.17	1.60
0.492	0.00	0.501	-38.957	-207.9505	3.61	1.39
0.494	0.00	0.423	-39.341	-175.4543	3.04	1.17

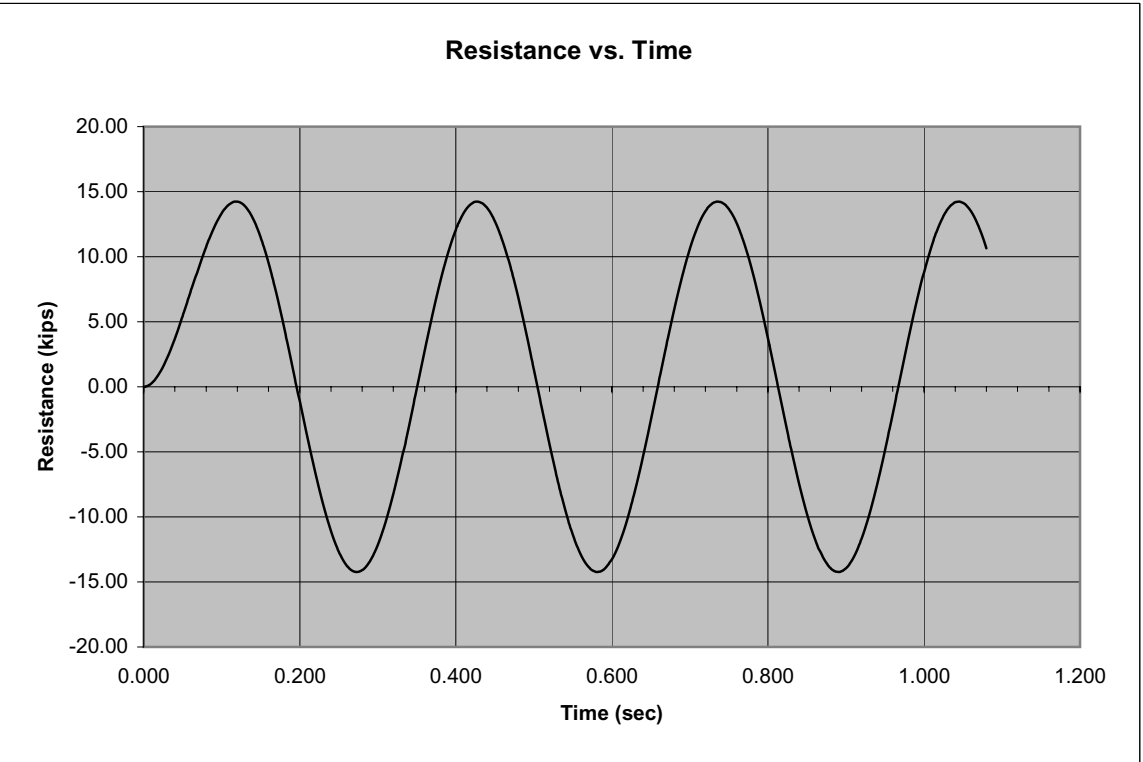
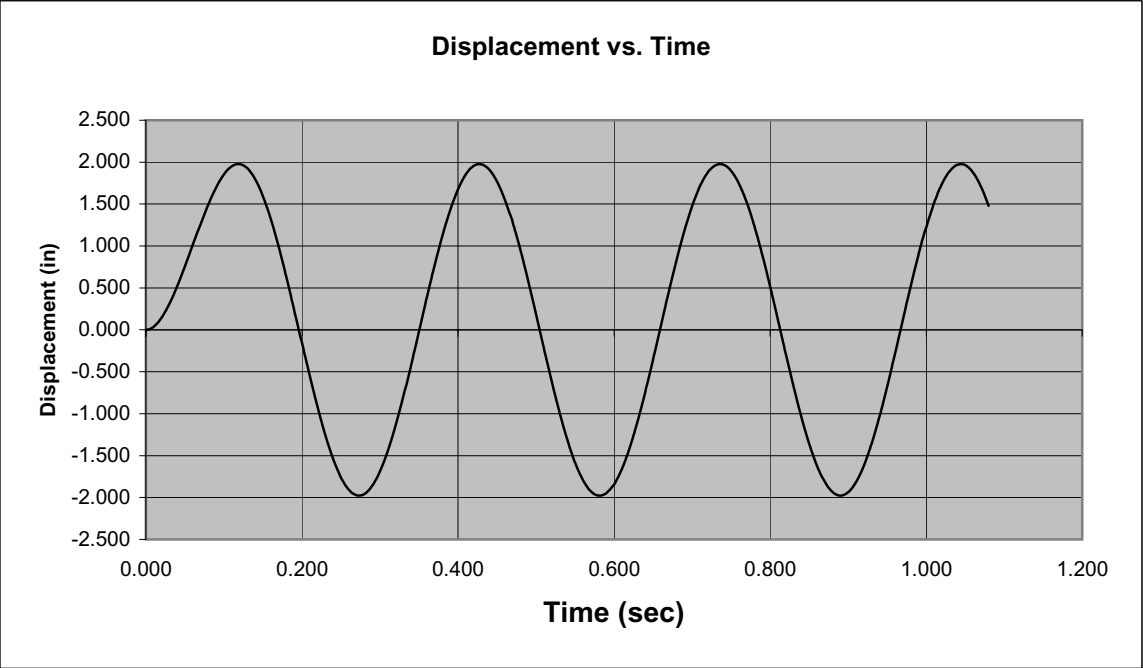
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.496	0.00	0.344	-39.659	-142.6669	2.48	0.95
0.498	0.00	0.264	-39.911	-109.6428	1.90	0.73
0.500	0.00	0.184	-40.097	-76.4367	1.33	0.51
0.502	0.00	0.104	-40.217	-43.1038	0.75	0.29
0.504	0.00	0.023	-40.269	-9.6994	0.17	0.06
0.506	0.00	-0.057	-40.255	23.7211	-0.41	-0.16
0.508	0.00	-0.138	-40.175	57.1023	-0.99	-0.38
0.510	0.00	-0.218	-40.027	90.3887	-1.57	-0.60
0.512	0.00	-0.298	-39.813	123.5251	-2.14	-0.83
0.514	0.00	-0.377	-39.533	156.4566	-2.71	-1.05
0.516	0.00	-0.456	-39.188	189.1284	-3.28	-1.26
0.518	0.00	-0.534	-38.777	221.4863	-3.84	-1.48
0.520	0.00	-0.611	-38.302	253.4767	-4.40	-1.69
0.522	0.00	-0.687	-37.763	285.0465	-4.95	-1.90
0.524	0.00	-0.762	-37.162	316.1432	-5.49	-2.11
0.526	0.00	-0.836	-36.499	346.7154	-6.02	-2.32
0.528	0.00	-0.908	-35.776	376.7122	-6.54	-2.52
0.530	0.00	-0.979	-34.993	406.0838	-7.05	-2.71
0.532	0.00	-1.048	-34.152	434.7816	-7.54	-2.90
0.534	0.00	-1.115	-33.255	462.7578	-8.03	-3.09
0.536	0.00	-1.181	-32.302	489.9662	-8.50	-3.27
0.538	0.00	-1.244	-31.296	516.3614	-8.96	-3.45
0.540	0.00	-1.306	-30.237	541.8998	-9.40	-3.62
0.542	0.00	-1.365	-29.129	566.5390	-9.83	-3.78
0.544	0.00	-1.422	-27.972	590.2380	-10.24	-3.94
0.546	0.00	-1.477	-26.769	612.9575	-10.63	-4.09
0.548	0.00	-1.529	-25.521	634.6598	-11.01	-4.24
0.550	0.00	-1.579	-24.231	655.3090	-11.37	-4.38
0.552	0.00	-1.626	-22.901	674.8707	-11.71	-4.51
0.554	0.00	-1.671	-21.533	693.3125	-12.03	-4.63
0.556	0.00	-1.712	-20.129	710.6038	-12.33	-4.75
0.558	0.00	-1.751	-18.692	726.7159	-12.61	-4.85
0.560	0.00	-1.787	-17.224	741.6220	-12.87	-4.95
0.562	0.00	-1.820	-15.727	755.2974	-13.10	-5.05
0.564	0.00	-1.850	-14.204	767.7195	-13.32	-5.13
0.566	0.00	-1.877	-12.657	778.8676	-13.51	-5.20
0.568	0.00	-1.901	-11.089	788.7231	-13.68	-5.27
0.570	0.00	-1.921	-9.503	797.2699	-13.83	-5.33
0.572	0.00	-1.939	-7.902	804.4936	-13.96	-5.37
0.574	0.00	-1.953	-6.287	810.3823	-14.06	-5.41
0.576	0.00	-1.964	-4.661	814.9261	-14.14	-5.44
0.578	0.00	-1.971	-3.028	818.1177	-14.19	-5.46
0.580	0.00	-1.976	-1.390	819.9517	-14.23	-5.48
0.582	0.00	-1.977	0.250	820.4249	-14.23	-5.48
0.584	0.00	-1.975	1.890	819.5368	-14.22	-5.47
0.586	0.00	-1.969	3.527	817.2886	-14.18	-5.46
0.588	0.00	-1.961	5.158	813.6842	-14.12	-5.44
0.590	0.00	-1.949	6.780	808.7295	-14.03	-5.40
0.592	0.00	-1.934	8.391	802.4328	-13.92	-5.36
0.594	0.00	-1.915	9.989	794.8045	-13.79	-5.31
0.596	0.00	-1.894	11.569	785.8572	-13.63	-5.25
0.598	0.00	-1.869	13.131	775.6059	-13.46	-5.18
0.600	0.00	-1.841	14.670	764.0675	-13.26	-5.10
0.602	0.00	-1.810	16.186	751.2611	-13.03	-5.02
0.604	0.00	-1.777	17.674	737.2081	-12.79	-4.92
0.606	0.00	-1.740	19.133	721.9317	-12.53	-4.82
0.608	0.00	-1.700	20.561	705.4573	-12.24	-4.71
0.610	0.00	-1.657	21.954	687.8122	-11.93	-4.59
0.612	0.00	-1.612	23.311	669.0257	-11.61	-4.47
0.614	0.00	-1.564	24.629	649.1291	-11.26	-4.34
0.616	0.00	-1.514	25.906	628.1552	-10.90	-4.20

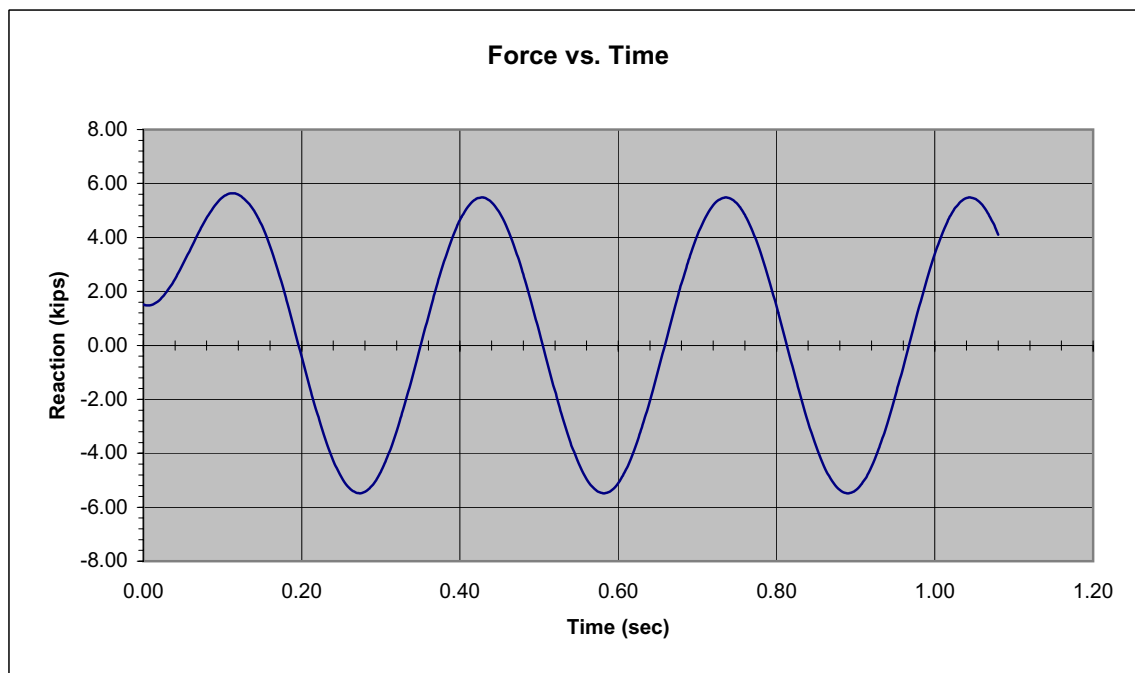
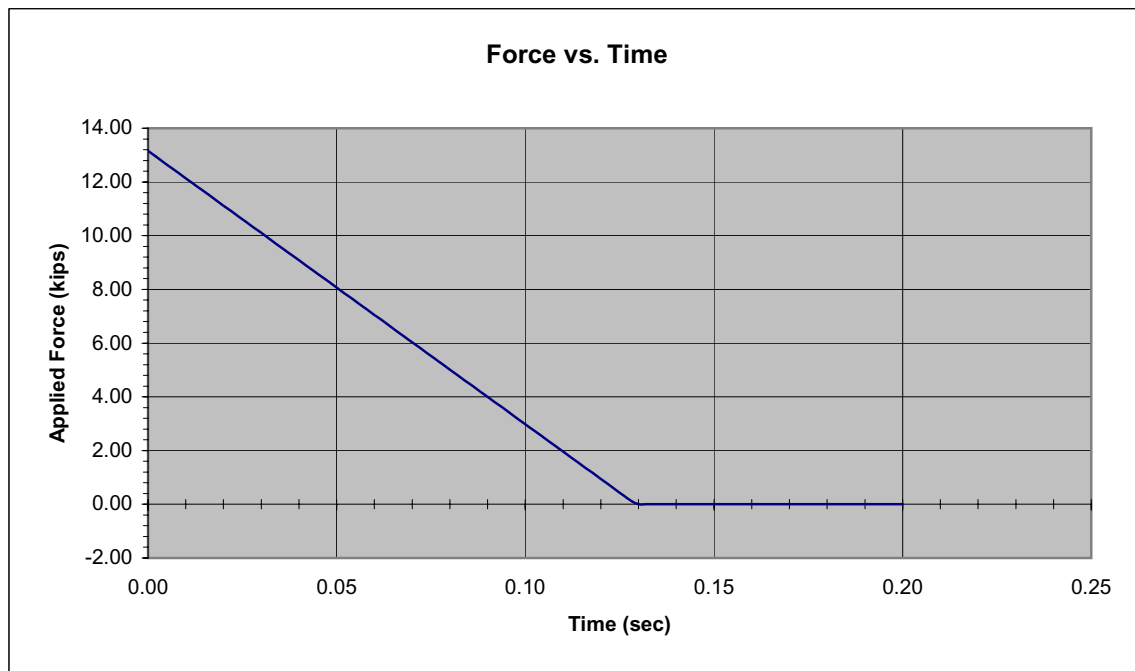
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.618	0.00	-1.461	27.141	606.1389	-10.52	-4.05
0.620	0.00	-1.405	28.330	583.1168	-10.12	-3.90
0.622	0.00	-1.347	29.472	559.1270	-9.70	-3.73
0.624	0.00	-1.287	30.565	534.2094	-9.27	-3.57
0.626	0.00	-1.225	31.608	508.4053	-8.82	-3.40
0.628	0.00	-1.161	32.598	481.7575	-8.36	-3.22
0.630	0.00	-1.095	33.534	454.3102	-7.88	-3.03
0.632	0.00	-1.027	34.415	426.1091	-7.39	-2.85
0.634	0.00	-0.957	35.238	397.2009	-6.89	-2.65
0.636	0.00	-0.886	36.003	367.6335	-6.38	-2.46
0.638	0.00	-0.813	36.708	337.4560	-5.85	-2.25
0.640	0.00	-0.739	37.352	306.7186	-5.32	-2.05
0.642	0.00	-0.664	37.934	275.4721	-4.78	-1.84
0.644	0.00	-0.587	38.454	243.7686	-4.23	-1.63
0.646	0.00	-0.510	38.909	211.6605	-3.67	-1.41
0.648	0.00	-0.432	39.300	179.2012	-3.11	-1.20
0.650	0.00	-0.353	39.625	146.4445	-2.54	-0.98
0.652	0.00	-0.273	39.885	113.4448	-1.97	-0.76
0.654	0.00	-0.193	40.079	80.2568	-1.39	-0.54
0.656	0.00	-0.113	40.206	46.9356	-0.81	-0.31
0.658	0.00	-0.033	40.267	13.5366	-0.23	-0.09
0.660	0.00	0.048	40.260	-19.8849	0.35	0.13
0.662	0.00	0.128	40.187	-53.2734	0.92	0.36
0.664	0.00	0.209	40.047	-86.5735	1.50	0.58
0.666	0.00	0.289	39.841	-119.7300	2.08	0.80
0.668	0.00	0.368	39.569	-152.6877	2.65	1.02
0.670	0.00	0.447	39.231	-185.3921	3.22	1.24
0.672	0.00	0.525	38.827	-217.7888	3.78	1.45
0.674	0.00	0.602	38.360	-249.8241	4.33	1.67
0.676	0.00	0.678	37.828	-281.4449	4.88	1.88
0.678	0.00	0.753	37.234	-312.5986	5.42	2.09
0.680	0.00	0.827	36.579	-343.2335	5.96	2.29
0.682	0.00	0.900	35.862	-373.2989	6.48	2.49
0.684	0.00	0.971	35.086	-402.7448	6.99	2.69
0.686	0.00	1.040	34.252	-431.5224	7.49	2.88
0.688	0.00	1.107	33.361	-459.5839	7.97	3.07
0.690	0.00	1.173	32.414	-486.8827	8.45	3.25
0.692	0.00	1.237	31.414	-513.3736	8.91	3.43
0.694	0.00	1.299	30.362	-539.0125	9.35	3.60
0.696	0.00	1.359	29.259	-563.7570	9.78	3.77
0.698	0.00	1.416	28.107	-587.5660	10.19	3.92
0.700	0.00	1.471	26.909	-610.3999	10.59	4.08
0.702	0.00	1.524	25.667	-632.2209	10.97	4.22
0.704	0.00	1.574	24.382	-652.9928	11.33	4.36
0.706	0.00	1.621	23.056	-672.6810	11.67	4.49
0.708	0.00	1.666	21.692	-691.2530	11.99	4.62
0.710	0.00	1.708	20.292	-708.6779	12.30	4.73
0.712	0.00	1.747	18.859	-724.9267	12.58	4.84
0.714	0.00	1.783	17.394	-739.9726	12.84	4.94
0.716	0.00	1.816	15.900	-753.7906	13.08	5.04
0.718	0.00	1.847	14.380	-766.3576	13.30	5.12
0.720	0.00	1.874	12.836	-777.6529	13.49	5.19
0.722	0.00	1.898	11.270	-787.6578	13.67	5.26
0.724	0.00	1.919	9.686	-796.3556	13.82	5.32
0.726	0.00	1.937	8.086	-803.7318	13.94	5.37
0.728	0.00	1.951	6.473	-809.7743	14.05	5.41
0.730	0.00	1.963	4.849	-814.4730	14.13	5.44
0.732	0.00	1.971	3.216	-817.8202	14.19	5.46
0.734	0.00	1.976	1.579	-819.8102	14.22	5.48
0.736	0.00	1.977	-0.062	-820.4398	14.23	5.48
0.738	0.00	1.975	-1.702	-819.7079	14.22	5.48

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.740	0.00	1.970	-3.339	-817.6157	14.19	5.46
0.742	0.00	1.962	-4.971	-814.1668	14.13	5.44
0.744	0.00	1.950	-6.594	-809.3668	14.04	5.41
0.746	0.00	1.936	-8.207	-803.2236	13.94	5.37
0.748	0.00	1.918	-9.806	-795.7476	13.81	5.32
0.750	0.00	1.896	-11.389	-786.9511	13.65	5.26
0.752	0.00	1.872	-12.953	-776.8486	13.48	5.19
0.754	0.00	1.845	-14.495	-765.4571	13.28	5.11
0.756	0.00	1.814	-16.013	-752.7952	13.06	5.03
0.758	0.00	1.781	-17.505	-738.8842	12.82	4.94
0.760	0.00	1.744	-18.967	-723.7470	12.56	4.83
0.762	0.00	1.705	-20.399	-707.4088	12.27	4.73
0.764	0.00	1.662	-21.796	-689.8967	11.97	4.61
0.766	0.00	1.618	-23.157	-671.2397	11.65	4.48
0.768	0.00	1.570	-24.480	-651.4689	11.30	4.35
0.770	0.00	1.520	-25.762	-630.6169	10.94	4.21
0.772	0.00	1.467	-27.001	-608.7185	10.56	4.07
0.774	0.00	1.412	-28.196	-585.8100	10.16	3.91
0.776	0.00	1.354	-29.343	-561.9293	9.75	3.75
0.778	0.00	1.294	-30.442	-537.1161	9.32	3.59
0.780	0.00	1.232	-31.491	-511.4117	8.87	3.42
0.782	0.00	1.168	-32.487	-484.8585	8.41	3.24
0.784	0.00	1.102	-33.430	-457.5008	7.94	3.06
0.786	0.00	1.035	-34.316	-429.3838	7.45	2.87
0.788	0.00	0.965	-35.146	-400.5544	6.95	2.68
0.790	0.00	0.894	-35.918	-371.0602	6.44	2.48
0.792	0.00	0.822	-36.630	-340.9502	5.92	2.28
0.794	0.00	0.748	-37.281	-310.2745	5.38	2.07
0.796	0.00	0.673	-37.871	-279.0839	4.84	1.86
0.798	0.00	0.596	-38.397	-247.4302	4.29	1.65
0.800	0.00	0.519	-38.860	-215.3658	3.74	1.44
0.802	0.00	0.441	-39.258	-182.9441	3.17	1.22
0.804	0.00	0.362	-39.591	-150.2188	2.61	1.00
0.806	0.00	0.283	-39.859	-117.2442	2.03	0.78
0.808	0.00	0.203	-40.060	-84.0751	1.46	0.56
0.810	0.00	0.122	-40.195	-50.7664	0.88	0.34
0.812	0.00	0.042	-40.263	-17.3735	0.30	0.12
0.814	0.00	-0.039	-40.264	16.0483	-0.28	-0.11
0.816	0.00	-0.119	-40.199	49.4434	-0.86	-0.33
0.818	0.00	-0.199	-40.067	82.7564	-1.44	-0.55
0.820	0.00	-0.279	-39.868	115.9322	-2.01	-0.77
0.822	0.00	-0.359	-39.603	148.9155	-2.58	-0.99
0.824	0.00	-0.438	-39.273	181.6517	-3.15	-1.21
0.826	0.00	-0.516	-38.877	214.0865	-3.71	-1.43
0.828	0.00	-0.593	-38.417	246.1661	-4.27	-1.64
0.830	0.00	-0.670	-37.893	277.8371	-4.82	-1.86
0.832	0.00	-0.745	-37.306	309.0470	-5.36	-2.06
0.834	0.00	-0.819	-36.657	339.7442	-5.89	-2.27
0.836	0.00	-0.891	-35.947	369.8775	-6.42	-2.47
0.838	0.00	-0.962	-35.178	399.3970	-6.93	-2.67
0.840	0.00	-1.032	-34.350	428.2538	-7.43	-2.86
0.842	0.00	-1.100	-33.466	456.3999	-7.92	-3.05
0.844	0.00	-1.166	-32.526	483.7886	-8.39	-3.23
0.846	0.00	-1.230	-31.531	510.3745	-8.86	-3.41
0.848	0.00	-1.292	-30.485	536.1135	-9.30	-3.58
0.850	0.00	-1.352	-29.388	560.9628	-9.73	-3.75
0.852	0.00	-1.409	-28.242	584.8812	-10.15	-3.91
0.854	0.00	-1.465	-27.049	607.8290	-10.55	-4.06
0.856	0.00	-1.518	-25.812	629.7682	-10.93	-4.21
0.858	0.00	-1.568	-24.531	650.6623	-11.29	-4.35
0.860	0.00	-1.616	-23.210	670.4767	-11.63	-4.48

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.862	0.00	-1.661	-21.851	689.1784	-11.96	-4.60
0.864	0.00	-1.703	-20.455	706.7365	-12.26	-4.72
0.866	0.00	-1.743	-19.025	723.1218	-12.55	-4.83
0.868	0.00	-1.779	-17.563	738.3071	-12.81	-4.93
0.870	0.00	-1.813	-16.073	752.2672	-13.05	-5.02
0.872	0.00	-1.843	-14.555	764.9790	-13.27	-5.11
0.874	0.00	-1.871	-13.014	776.4213	-13.47	-5.19
0.876	0.00	-1.895	-11.451	786.5752	-13.65	-5.25
0.878	0.00	-1.917	-9.869	795.4238	-13.80	-5.31
0.880	0.00	-1.935	-8.271	802.9525	-13.93	-5.36
0.882	0.00	-1.950	-6.659	809.1487	-14.04	-5.40
0.884	0.00	-1.962	-5.035	814.0021	-14.12	-5.44
0.886	0.00	-1.970	-3.404	817.5048	-14.18	-5.46
0.888	0.00	-1.975	-1.767	819.6508	-14.22	-5.48
0.890	0.00	-1.977	-0.127	820.4367	-14.23	-5.48
0.892	0.00	-1.976	1.514	819.8611	-14.22	-5.48
0.894	0.00	-1.971	3.151	817.9250	-14.19	-5.46
0.896	0.00	-1.963	4.784	814.6316	-14.13	-5.44
0.898	0.00	-1.952	6.409	809.9863	-14.05	-5.41
0.900	0.00	-1.937	8.023	803.9969	-13.95	-5.37
0.902	0.00	-1.920	9.623	796.6733	-13.82	-5.32
0.904	0.00	-1.899	11.208	788.0277	-13.67	-5.26
0.906	0.00	-1.875	12.774	778.0744	-13.50	-5.20
0.908	0.00	-1.848	14.319	766.8299	-13.30	-5.12
0.910	0.00	-1.818	15.840	754.3129	-13.09	-5.04
0.912	0.00	-1.785	17.335	740.5442	-12.85	-4.95
0.914	0.00	-1.748	18.801	725.5465	-12.59	-4.85
0.916	0.00	-1.709	20.236	709.3449	-12.31	-4.74
0.918	0.00	-1.667	21.637	691.9661	-12.01	-4.62
0.920	0.00	-1.623	23.003	673.4390	-11.68	-4.50
0.922	0.00	-1.575	24.330	653.7944	-11.34	-4.37
0.924	0.00	-1.526	25.617	633.0649	-10.98	-4.23
0.926	0.00	-1.473	26.861	611.2848	-10.61	-4.08
0.928	0.00	-1.418	28.061	588.4904	-10.21	-3.93
0.930	0.00	-1.361	29.214	564.7193	-9.80	-3.77
0.932	0.00	-1.301	30.319	540.0112	-9.37	-3.61
0.934	0.00	-1.240	31.373	514.4069	-8.92	-3.44
0.936	0.00	-1.176	32.376	487.9490	-8.47	-3.26
0.938	0.00	-1.110	33.324	460.6813	-7.99	-3.08
0.940	0.00	-1.043	34.217	432.6492	-7.51	-2.89
0.942	0.00	-0.973	35.054	403.8991	-7.01	-2.70
0.944	0.00	-0.902	35.832	374.4788	-6.50	-2.50
0.946	0.00	-0.830	36.551	344.4370	-5.98	-2.30
0.948	0.00	-0.756	37.210	313.8237	-5.44	-2.10
0.950	0.00	-0.681	37.806	282.6896	-4.90	-1.89
0.952	0.00	-0.605	38.340	251.0864	-4.36	-1.68
0.954	0.00	-0.528	38.810	219.0665	-3.80	-1.46
0.956	0.00	-0.450	39.216	186.6831	-3.24	-1.25
0.958	0.00	-0.371	39.556	153.9899	-2.67	-1.03
0.960	0.00	-0.292	39.831	121.0411	-2.10	-0.81
0.962	0.00	-0.212	40.040	87.8915	-1.52	-0.59
0.964	0.00	-0.132	40.183	54.5961	-0.95	-0.36
0.966	0.00	-0.051	40.259	21.2100	-0.37	-0.14
0.968	0.00	0.029	40.268	-12.2112	0.21	0.08
0.970	0.00	0.110	40.210	-45.6122	0.79	0.30
0.972	0.00	0.190	40.085	-78.9375	1.37	0.53
0.974	0.00	0.270	39.894	-112.1318	1.95	0.75
0.976	0.00	0.350	39.637	-145.1401	2.52	0.97
0.978	0.00	0.429	39.314	-177.9074	3.09	1.19
0.980	0.00	0.507	38.926	-210.3796	3.65	1.41
0.982	0.00	0.584	38.473	-242.5026	4.21	1.62

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.984	0.00	0.661	37.956	-274.2232	4.76	1.83
0.986	0.00	0.736	37.376	-305.4888	5.30	2.04
0.988	0.00	0.810	36.735	-336.2474	5.83	2.25
0.990	0.00	0.883	36.032	-366.4480	6.36	2.45
0.992	0.00	0.954	35.269	-396.0405	6.87	2.65
0.994	0.00	1.024	34.448	-424.9758	7.37	2.84
0.996	0.00	1.092	33.570	-453.2059	7.86	3.03
0.998	0.00	1.158	32.636	-480.6839	8.34	3.21
1.000	0.00	1.223	31.648	-507.3643	8.80	3.39
1.002	0.00	1.285	30.608	-533.2027	9.25	3.56
1.004	0.00	1.345	29.516	-558.1562	9.68	3.73
1.006	0.00	1.403	28.376	-582.1836	10.10	3.89
1.008	0.00	1.459	27.189	-605.2448	10.50	4.04
1.010	0.00	1.512	25.956	-627.3017	10.88	4.19
1.012	0.00	1.562	24.680	-648.3176	11.25	4.33
1.014	0.00	1.610	23.364	-668.2576	11.59	4.46
1.016	0.00	1.656	22.009	-687.0887	11.92	4.59
1.018	0.00	1.698	20.617	-704.7796	12.23	4.71
1.020	0.00	1.738	19.191	-721.3010	12.51	4.82
1.022	0.00	1.775	17.733	-736.6254	12.78	4.92
1.024	0.00	1.809	16.245	-750.7274	13.03	5.01
1.026	0.00	1.840	14.731	-763.5836	13.25	5.10
1.028	0.00	1.868	13.192	-775.1727	13.45	5.18
1.030	0.00	1.893	11.632	-785.4754	13.63	5.25
1.032	0.00	1.915	10.052	-794.4747	13.78	5.31
1.034	0.00	1.933	8.455	-802.1556	13.92	5.36
1.036	0.00	1.948	6.844	-808.5053	14.03	5.40
1.038	0.00	1.960	5.222	-813.5134	14.11	5.43
1.040	0.00	1.969	3.592	-817.1715	14.18	5.46
1.042	0.00	1.975	1.955	-819.4735	14.22	5.47
1.044	0.00	1.977	0.315	-820.4156	14.23	5.48
1.046	0.00	1.976	-1.325	-819.9964	14.23	5.48
1.048	0.00	1.972	-2.964	-818.2163	14.20	5.47
1.050	0.00	1.964	-4.597	-815.0785	14.14	5.44
1.052	0.00	1.953	-6.222	-810.5881	14.06	5.41
1.054	0.00	1.939	-7.838	-804.7526	13.96	5.38
1.056	0.00	1.922	-9.440	-797.5816	13.84	5.33
1.058	0.00	1.902	-11.027	-789.0871	13.69	5.27
1.060	0.00	1.878	-12.595	-779.2831	13.52	5.21
1.062	0.00	1.851	-14.143	-768.1860	13.33	5.13
1.064	0.00	1.821	-15.667	-755.8141	13.11	5.05
1.066	0.00	1.789	-17.165	-742.1879	12.88	4.96
1.068	0.00	1.753	-18.634	-727.3301	12.62	4.86
1.070	0.00	1.714	-20.073	-711.2654	12.34	4.75
1.072	0.00	1.672	-21.478	-694.0203	12.04	4.64
1.074	0.00	1.628	-22.848	-675.6236	11.72	4.51
1.076	0.00	1.581	-24.179	-656.1057	11.38	4.38
1.078	0.00	1.531	-25.471	-635.4990	11.03	4.24
1.080	0.00	1.479	-26.720	-613.8377	10.65	4.10





Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

WALL PANELS W/ EQUIPMENT DOOR OPENING

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject : WALL PROPERTIES WITH EQUIPMENT DOOR

Sheet no. : of
Date : 6-Jun-04
Design by : MGG
Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Roof Dead Load = 8.470 KN/m²
= 176.90 psf

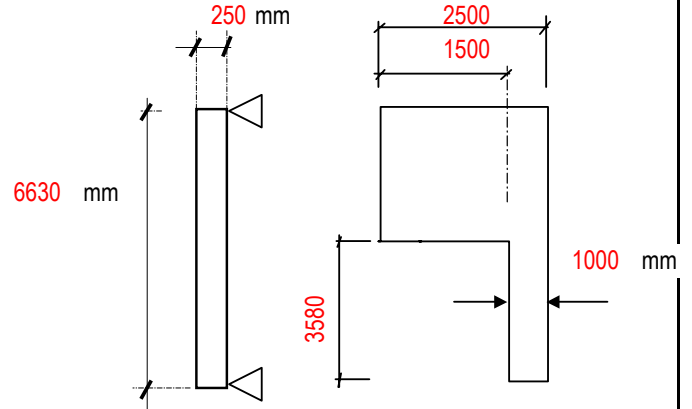
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
Reinf., f_y = 414 Mpa

PANEL DATA :

Panel Width, width = 2500 mm
Wall Span, lb = 6630 mm
Wall, t_{TRIAL} = 250 mm
(Panel Width - Opening) ; b = 1000 mm
Roof span : = 15.22 m

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure, P_s = 4.20 psi (SEE LOADING CALCULATION)
Effective blast time duration, T_e = 129 msec

Moment of Inertia, I_g = 3128.27 in⁴ (1302083333 mm⁴)
Area of Bar ($\emptyset$), 25 mm $\emptyset$ A_b = 0.7609 in² (491 mm²)
Number of Bars per side of opening N = 9.0000
Area of Rebar PER side, A_s = 6.8477 in² (4418 mm²)
Effective Depth, $d_c = d_s$ = 8.3425 in (212 mm)
 $f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for bending) = 5954.3 psi (41 Mpa.)
 $f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for shear) = 5504.0 psi (38 Mpa.)
 $f_{dy} = (SIF) \times (DIF) \times f_y$ = 77276 psi (533 Mpa.)
Modulus of Elasticity, $E_{dc} = 57000 \times (f'_c)^{1/2}$ = 4032041 psi (27801 Mpa.)
Steel ratio, $p = A_s / b \times d$ = 0.0208 > 200 / f_{dy} = 0.003
Stress block, $a = A_s \times f_{dy} / 0.85 \times f'_{dc} \times b$ = 2.6557 in (67 mm)
Modular ratio, $n = E_s / E_{dm}$ = 7.1940

a.) BENDING RESISTANCE

Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$ = 3711915.08 lb-in (419369 KN-m)
Resistance at cracked moment, $R_b = 8 \times M_p / L$ = 113764.88 lb (506.03 KN)
Unit Resistance, $r_b = R_b / (L \times \text{panel width})$ = 4.4281 psi (0.0305 Mpa.)

b.) SHEAR RESISTANCE

Shear, $V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$ = 48734.0 lb (217 KN)
At Support, $R_s = V_n \times L / (0.5 \times L - t_w)$ = 104124 lb (463 KN)
Unit Resistance, $r_s = R_s / (L \times \text{panel width})$ = 4.053 psi (0.0279 Mpa.)

∴ Consider 1 ft strip

∴ Dynamic Resistance Controlled by Bending, R_u = 4.0529 psi < 4.20 ∴
12.6948 kips 13.16 kips **USE 13.16**

Cracked Moment of Inertia:

$n \times A_s$ = 49.2624 in² (31782 mm²)
 $C = -n \times A_s + [n \times A_s \times (n \times A_s + 2 \times b \times d)]^{0.5} / b$ = 3.4861 in² (2249 mm²)
 $I_{cr} = b \times C^3 / 3 + n \times A_s \times (d - C)^2$ = 1717.83 in⁴ (715015635 mm⁴)

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : WALL PROPERTIES WITH EQUIPMENT DOOR

Sheet no. : of
 Date : 6-Jun-04
 Design by : MGG
 Check'd by : SAK

Average Moment of Inertia, $I_a = (I_g + I_{cr}) / 2 = 2423.050 \text{ in}^4$ (1008549484.1 mm⁴)
 Effective Stiffness, $K = 384 \times E_{dc} \times I_a / 5 L^3 = 5.1438 \text{ kips / in}$ (per 1 ft strip)
 Yield Deflection, $Y_e = R_u / K = 2.46798 \text{ in}$
 Wall Mass = wall total weight / panel width / gravity acc. = 0.00469 kip-sec² / in
 Roof Mass = Roof DL x unit width x 1/2 roof span / gravity acc. = 0.01144 kip-sec² / in
 Total Mass, M = 0.01613 kip-sec² / in
 Average Stiffness, $K_{LM} = 0.72000$ FROM ASCE TABLE 6.1
 Equivalent Mass, $M_e = K_{LM} \times \text{Beam Mass} = 0.01161 \text{ kip-sec}^2 / \text{in}$
 Period of Vibration, $t_n = 2 \times \pi \times (M_e / K)^{0.5} = 0.29855 \text{ sec}$
 Time increment, $t_n/10$ $\Delta t = 0.02985 \text{ sec}$; use $\Delta t = 0.00200 \text{ sec}$
 $t_d / t_n = 0.43303$
 $R_u / P_s = 0.96497$

NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	5.144 kips / in
Mass	M	=	0.0116 kip-sec ² / in
Tension resistance	Rut	=	13.16 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	13.16 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.1293 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load	
load (kip)	time (sec)
13.16	0
0	0.1293

Length	266.732 in
y _{max} =	2.840 in
r _{max} =	13.156 kips
μ =	1.110
θ =	1.220 degrees
Rut/Fo =	1.000
θ = < 2 degrees	
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.000	13.16	0.000	0.000	1132.7988	0.00	1.51
0.002	12.95	0.002	2.247	1114.2763	0.01	1.49
0.004	12.75	0.009	4.455	1093.7803	0.05	1.48
0.006	12.55	0.020	6.620	1071.3470	0.10	1.48
0.008	12.34	0.035	8.739	1047.0162	0.18	1.49
0.010	12.14	0.055	10.806	1020.8309	0.28	1.50
0.012	11.93	0.079	12.820	992.8376	0.40	1.53
0.014	11.73	0.106	14.776	963.0858	0.55	1.56
0.016	11.53	0.138	16.671	931.6282	0.71	1.60
0.018	11.32	0.173	18.501	898.5206	0.89	1.64
0.020	11.12	0.212	20.263	863.8215	1.09	1.70
0.022	10.92	0.254	21.955	827.5925	1.31	1.76
0.024	10.71	0.299	23.572	789.8976	1.54	1.82
0.026	10.51	0.348	25.113	750.8037	1.79	1.90
0.028	10.31	0.400	26.574	710.3801	2.06	1.98
0.030	10.10	0.454	27.953	668.6982	2.34	2.06
0.032	9.90	0.512	29.248	625.8320	2.63	2.15
0.034	9.70	0.571	30.455	581.8573	2.94	2.25
0.036	9.49	0.633	31.574	536.8520	3.26	2.35
0.038	9.29	0.698	32.602	490.8959	3.59	2.45
0.040	9.09	0.764	33.537	444.0704	3.93	2.56
0.042	8.88	0.832	34.377	396.4583	4.28	2.67
0.044	8.68	0.901	35.122	348.1440	4.64	2.78
0.046	8.47	0.972	35.769	299.2132	5.00	2.90
0.048	8.27	1.044	36.318	249.7523	5.37	3.02
0.050	8.07	1.117	36.768	199.8491	5.75	3.14
0.052	7.86	1.191	37.117	149.5920	6.13	3.26
0.054	7.66	1.266	37.366	99.0698	6.51	3.39
0.056	7.46	1.341	37.513	48.3723	6.90	3.51
0.058	7.25	1.416	37.559	-2.4110	7.28	3.64
0.060	7.05	1.491	37.504	-53.1900	7.67	3.76
0.062	6.85	1.566	37.347	-103.8748	8.05	3.89

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.064	6.64	1.640	37.088	-154.3756	8.44	4.01
0.066	6.44	1.714	36.729	-204.6029	8.82	4.13
0.068	6.24	1.787	36.270	-254.4679	9.19	4.26
0.070	6.03	1.859	35.712	-303.8822	9.56	4.37
0.072	5.83	1.930	35.055	-352.7583	9.93	4.49
0.074	5.63	1.999	34.302	-401.0095	10.28	4.61
0.076	5.42	2.067	33.452	-448.5506	10.63	4.72
0.078	5.22	2.133	32.508	-495.2971	10.97	4.82
0.080	5.01	2.197	31.472	-541.1664	11.30	4.93
0.082	4.81	2.259	30.344	-586.0772	11.62	5.03
0.084	4.61	2.318	29.128	-629.9500	11.92	5.12
0.086	4.40	2.375	27.826	-672.7071	12.22	5.21
0.088	4.20	2.429	26.439	-714.2726	12.50	5.29
0.090	4.00	2.481	24.970	-754.5731	12.76	5.37
0.092	3.79	2.529	23.422	-793.5371	13.01	5.44
0.094	3.59	2.574	21.797	-823.6498	13.16	5.48
0.096	3.39	2.616	20.132	-841.1743	13.16	5.45
0.098	3.18	2.655	18.433	-858.6988	13.16	5.43
0.100	2.98	2.690	16.698	-876.2232	13.16	5.41
0.102	2.78	2.722	14.928	-893.7477	13.16	5.38
0.104	2.57	2.750	13.123	-911.2721	13.16	5.36
0.106	2.37	2.774	11.283	-928.7966	13.16	5.34
0.108	2.17	2.795	9.407	-946.3211	13.16	5.31
0.110	1.96	2.812	7.497	-963.8455	13.16	5.29
0.112	1.76	2.825	5.552	-981.3700	13.16	5.27
0.114	1.56	2.834	3.572	-998.8945	13.16	5.24
0.116	1.35	2.839	1.556	-1016.4189	13.16	5.22
0.118	1.15	2.840	-0.494	-1033.9434	13.16	5.20
0.120	0.94	2.837	-2.578	-1050.1097	13.14	5.17
0.122	0.74	2.830	-4.692	-1064.4161	13.10	5.13
0.124	0.54	2.818	-6.834	-1076.8372	13.04	5.08
0.126	0.33	2.802	-8.998	-1087.3510	12.96	5.03
0.128	0.13	2.782	-11.181	-1095.9390	12.86	4.97
0.130	0.00	2.758	-13.373	-1096.2964	12.73	4.90
0.132	0.00	2.729	-15.553	-1083.4823	12.58	4.84
0.134	0.00	2.696	-17.705	-1068.7492	12.41	4.78
0.136	0.00	2.658	-19.826	-1052.1232	12.22	4.70
0.138	0.00	2.616	-21.912	-1033.6336	12.00	4.62
0.140	0.00	2.570	-23.959	-1013.3134	11.77	4.53
0.142	0.00	2.520	-25.964	-991.1983	11.51	4.43
0.144	0.00	2.467	-27.922	-967.3278	11.23	4.33
0.146	0.00	2.409	-29.831	-941.7439	10.94	4.21
0.148	0.00	2.347	-31.687	-914.4920	10.62	4.09
0.150	0.00	2.282	-33.487	-885.6205	10.29	3.96
0.152	0.00	2.213	-35.228	-855.1803	9.93	3.82
0.154	0.00	2.141	-36.907	-823.2255	9.56	3.68
0.156	0.00	2.066	-38.520	-789.8127	9.17	3.53
0.158	0.00	1.987	-40.065	-755.0009	8.77	3.38
0.160	0.00	1.906	-41.538	-718.8519	8.35	3.21
0.162	0.00	1.821	-42.939	-681.4297	7.91	3.05
0.164	0.00	1.734	-44.263	-642.8006	7.47	2.87
0.166	0.00	1.644	-45.509	-603.0330	7.00	2.70

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.168	0.00	1.552	-46.674	-562.1973	6.53	2.51
0.170	0.00	1.457	-47.757	-520.3659	6.04	2.33
0.172	0.00	1.361	-48.754	-477.6128	5.55	2.14
0.174	0.00	1.262	-49.666	-434.0138	5.04	1.94
0.176	0.00	1.162	-50.490	-389.6461	4.53	1.74
0.178	0.00	1.061	-51.224	-344.5882	4.00	1.54
0.180	0.00	0.957	-51.868	-298.9201	3.47	1.34
0.182	0.00	0.853	-52.419	-252.7225	2.93	1.13
0.184	0.00	0.748	-52.878	-206.0773	2.39	0.92
0.186	0.00	0.642	-53.243	-159.0671	1.85	0.71
0.188	0.00	0.535	-53.514	-111.7751	1.30	0.50
0.190	0.00	0.428	-53.690	-64.2852	0.75	0.29
0.192	0.00	0.320	-53.771	-16.6814	0.19	0.07
0.194	0.00	0.213	-53.757	30.9519	-0.36	-0.14
0.196	0.00	0.105	-53.647	78.5304	-0.91	-0.35
0.198	0.00	-0.002	-53.443	125.9698	-1.46	-0.56
0.200	0.00	-0.108	-53.144	173.1861	-2.01	-0.77
0.202	0.00	-0.214	-52.750	220.0956	-2.56	-0.98
0.204	0.00	-0.319	-52.264	266.6154	-3.10	-1.19
0.206	0.00	-0.423	-51.684	312.6629	-3.63	-1.40
0.208	0.00	-0.526	-51.013	358.1566	-4.16	-1.60
0.210	0.00	-0.627	-50.252	403.0160	-4.68	-1.80
0.212	0.00	-0.727	-49.402	447.1616	-5.19	-2.00
0.214	0.00	-0.825	-48.464	490.5151	-5.70	-2.19
0.216	0.00	-0.921	-47.441	532.9999	-6.19	-2.38
0.218	0.00	-1.015	-46.333	574.5407	-6.67	-2.57
0.220	0.00	-1.106	-45.144	615.0639	-7.14	-2.75
0.222	0.00	-1.195	-43.874	654.4977	-7.60	-2.93
0.224	0.00	-1.282	-42.527	692.7722	-8.05	-3.10
0.226	0.00	-1.365	-41.104	729.8198	-8.48	-3.26
0.228	0.00	-1.446	-39.609	765.5747	-8.89	-3.42
0.230	0.00	-1.524	-38.043	799.9737	-9.29	-3.58
0.232	0.00	-1.598	-36.410	832.9558	-9.67	-3.72
0.234	0.00	-1.669	-34.713	864.4625	-10.04	-3.87
0.236	0.00	-1.737	-32.954	894.4382	-10.39	-4.00
0.238	0.00	-1.801	-31.137	922.8297	-10.72	-4.13
0.240	0.00	-1.861	-29.264	949.5867	-11.03	-4.25
0.242	0.00	-1.918	-27.340	974.6619	-11.32	-4.36
0.244	0.00	-1.971	-25.368	998.0107	-11.59	-4.46
0.246	0.00	-2.019	-23.350	1019.5919	-11.84	-4.56
0.248	0.00	-2.064	-21.291	1039.3673	-12.07	-4.65
0.250	0.00	-2.105	-19.194	1057.3017	-12.28	-4.73
0.252	0.00	-2.141	-17.064	1073.3636	-12.47	-4.80
0.254	0.00	-2.173	-14.903	1087.5243	-12.63	-4.86
0.256	0.00	-2.200	-12.716	1099.7588	-12.77	-4.92
0.258	0.00	-2.224	-10.506	1110.0455	-12.89	-4.96
0.260	0.00	-2.242	-8.277	1118.3661	-12.99	-5.00
0.262	0.00	-2.257	-6.034	1124.7059	-13.06	-5.03
0.264	0.00	-2.267	-3.780	1129.0537	-13.11	-5.05
0.266	0.00	-2.272	-1.520	1131.4017	-13.14	-5.06
0.268	0.00	-2.273	0.743	1131.7458	-13.14	-5.06
0.270	0.00	-2.269	3.005	1130.0855	-13.12	-5.05

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.272	0.00	-2.261	5.261	1126.4235	-13.08	-5.04
0.274	0.00	-2.248	7.509	1120.7665	-13.02	-5.01
0.276	0.00	-2.231	9.743	1113.1244	-12.93	-4.98
0.278	0.00	-2.209	11.959	1103.5108	-12.82	-4.93
0.280	0.00	-2.183	14.155	1091.9428	-12.68	-4.88
0.282	0.00	-2.152	16.325	1078.4407	-12.52	-4.82
0.284	0.00	-2.117	18.467	1063.0285	-12.35	-4.75
0.286	0.00	-2.078	20.575	1045.7335	-12.14	-4.68
0.288	0.00	-2.035	22.648	1026.5863	-11.92	-4.59
0.290	0.00	-1.988	24.680	1005.6209	-11.68	-4.50
0.292	0.00	-1.937	26.668	982.8744	-11.41	-4.39
0.294	0.00	-1.881	28.610	958.3871	-11.13	-4.29
0.296	0.00	-1.822	30.500	932.2023	-10.83	-4.17
0.298	0.00	-1.759	32.337	904.3664	-10.50	-4.04
0.300	0.00	-1.693	34.116	874.9287	-10.16	-3.91
0.302	0.00	-1.623	35.835	843.9414	-9.80	-3.77
0.304	0.00	-1.550	37.490	811.4594	-9.42	-3.63
0.306	0.00	-1.473	39.079	777.5401	-9.03	-3.48
0.308	0.00	-1.393	40.599	742.2436	-8.62	-3.32
0.310	0.00	-1.311	42.047	705.6326	-8.19	-3.15
0.312	0.00	-1.225	43.420	667.7717	-7.76	-2.99
0.314	0.00	-1.137	44.717	628.7281	-7.30	-2.81
0.316	0.00	-1.046	45.934	588.5709	-6.84	-2.63
0.318	0.00	-0.953	47.070	547.3713	-6.36	-2.45
0.320	0.00	-0.858	48.123	505.2022	-5.87	-2.26
0.322	0.00	-0.761	49.090	462.1383	-5.37	-2.07
0.324	0.00	-0.662	49.970	418.2559	-4.86	-1.87
0.326	0.00	-0.561	50.762	373.6327	-4.34	-1.67
0.328	0.00	-0.459	51.464	328.3477	-3.81	-1.47
0.330	0.00	-0.355	52.075	282.4811	-3.28	-1.26
0.332	0.00	-0.251	52.594	236.1143	-2.74	-1.06
0.334	0.00	-0.145	53.019	189.3292	-2.20	-0.85
0.336	0.00	-0.039	53.351	142.2088	-1.65	-0.64
0.338	0.00	0.068	53.588	94.8366	-1.10	-0.42
0.340	0.00	0.176	53.730	47.2963	-0.55	-0.21
0.342	0.00	0.283	53.777	-0.3277	0.00	0.00
0.344	0.00	0.391	53.729	-47.9511	0.56	0.21
0.346	0.00	0.498	53.585	-95.4896	1.11	0.43
0.348	0.00	0.605	53.347	-142.8590	1.66	0.64
0.350	0.00	0.711	53.014	-189.9753	2.21	0.85
0.352	0.00	0.817	52.587	-236.7552	2.75	1.06
0.354	0.00	0.922	52.067	-283.1157	3.29	1.27
0.356	0.00	1.025	51.455	-328.9748	3.82	1.47
0.358	0.00	1.128	50.752	-374.2512	4.35	1.67
0.360	0.00	1.228	49.959	-418.8648	4.86	1.87
0.362	0.00	1.327	49.077	-462.7365	5.37	2.07
0.364	0.00	1.424	48.109	-505.7886	5.87	2.26
0.366	0.00	1.520	47.055	-547.9449	6.36	2.45
0.368	0.00	1.613	45.918	-589.1306	6.84	2.63
0.370	0.00	1.703	44.700	-629.2730	7.31	2.81
0.372	0.00	1.791	43.402	-668.3007	7.76	2.99
0.374	0.00	1.877	42.028	-706.1449	8.20	3.16

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.376	0.00	1.959	40.579	-742.7383	8.63	3.32
0.378	0.00	2.039	39.058	-778.0162	9.04	3.48
0.380	0.00	2.116	37.468	-811.9161	9.43	3.63
0.382	0.00	2.189	35.812	-844.3780	9.81	3.78
0.384	0.00	2.259	34.092	-875.3443	10.17	3.91
0.386	0.00	2.325	32.312	-904.7603	10.51	4.05
0.388	0.00	2.388	30.474	-932.5738	10.83	4.17
0.390	0.00	2.447	28.583	-958.7356	11.13	4.29
0.392	0.00	2.502	26.641	-983.1993	11.42	4.40
0.394	0.00	2.554	24.652	-1005.9215	11.68	4.50
0.396	0.00	2.601	22.619	-1026.8622	11.93	4.59
0.398	0.00	2.644	20.547	-1045.9840	12.15	4.68
0.400	0.00	2.683	18.437	-1063.2533	12.35	4.75
0.402	0.00	2.718	16.295	-1078.6394	12.53	4.82
0.404	0.00	2.748	14.125	-1092.1151	12.68	4.88
0.406	0.00	2.774	11.929	-1103.6564	12.82	4.93
0.408	0.00	2.796	9.712	-1113.2430	12.93	4.98
0.410	0.00	2.813	7.478	-1120.8578	13.02	5.01
0.412	0.00	2.826	5.230	-1126.4875	13.08	5.04
0.414	0.00	2.834	2.974	-1130.1219	13.12	5.05
0.416	0.00	2.838	0.712	-1131.7547	13.14	5.06
0.418	0.00	2.837	-1.551	-1131.3830	13.14	5.06
0.420	0.00	2.832	-3.812	-1129.0074	13.11	5.05
0.422	0.00	2.822	-6.065	-1124.6322	13.06	5.03
0.424	0.00	2.807	-8.308	-1118.2650	12.99	5.00
0.426	0.00	2.788	-10.536	-1109.9173	12.89	4.96
0.428	0.00	2.765	-12.746	-1099.6037	12.77	4.92
0.430	0.00	2.737	-14.933	-1087.3425	12.63	4.86
0.432	0.00	2.705	-17.093	-1073.1554	12.46	4.80
0.434	0.00	2.669	-19.223	-1057.0677	12.28	4.73
0.436	0.00	2.629	-21.320	-1039.1076	12.07	4.65
0.438	0.00	2.584	-23.378	-1019.3072	11.84	4.56
0.440	0.00	2.535	-25.395	-997.7014	11.59	4.46
0.442	0.00	2.482	-27.367	-974.3285	11.32	4.36
0.444	0.00	2.426	-29.291	-949.2299	11.02	4.24
0.446	0.00	2.365	-31.162	-922.4501	10.71	4.12
0.448	0.00	2.301	-32.979	-894.0365	10.38	4.00
0.450	0.00	2.233	-34.737	-864.0394	10.03	3.86
0.452	0.00	2.162	-36.433	-832.5119	9.67	3.72
0.454	0.00	2.088	-38.065	-799.5099	9.28	3.57
0.456	0.00	2.010	-39.630	-765.0919	8.89	3.42
0.458	0.00	1.929	-41.124	-729.3187	8.47	3.26
0.460	0.00	1.845	-42.546	-692.2539	8.04	3.10
0.462	0.00	1.759	-43.892	-653.9629	7.59	2.92
0.464	0.00	1.670	-45.161	-614.5136	7.14	2.75
0.466	0.00	1.578	-46.349	-573.9760	6.67	2.57
0.468	0.00	1.485	-47.456	-532.4217	6.18	2.38
0.470	0.00	1.389	-48.478	-489.9244	5.69	2.19
0.472	0.00	1.291	-49.414	-446.5594	5.19	2.00
0.474	0.00	1.191	-50.263	-402.4035	4.67	1.80
0.476	0.00	1.090	-51.023	-357.5349	4.15	1.60
0.478	0.00	0.987	-51.693	-312.0330	3.62	1.40
0.480	0.00	0.883	-52.271	-265.9784	3.09	1.19
0.482	0.00	0.778	-52.756	-219.4528	2.55	0.98
0.484	0.00	0.672	-53.148	-172.5384	2.00	0.77
0.486	0.00	0.565	-53.446	-125.3185	1.46	0.56
0.488	0.00	0.458	-53.649	-77.8766	0.90	0.35
0.490	0.00	0.351	-53.758	-30.2968	0.35	0.14
0.492	0.00	0.243	-53.771	17.3367	-0.20	-0.08
0.494	0.00	0.136	-53.688	64.9395	-0.75	-0.29

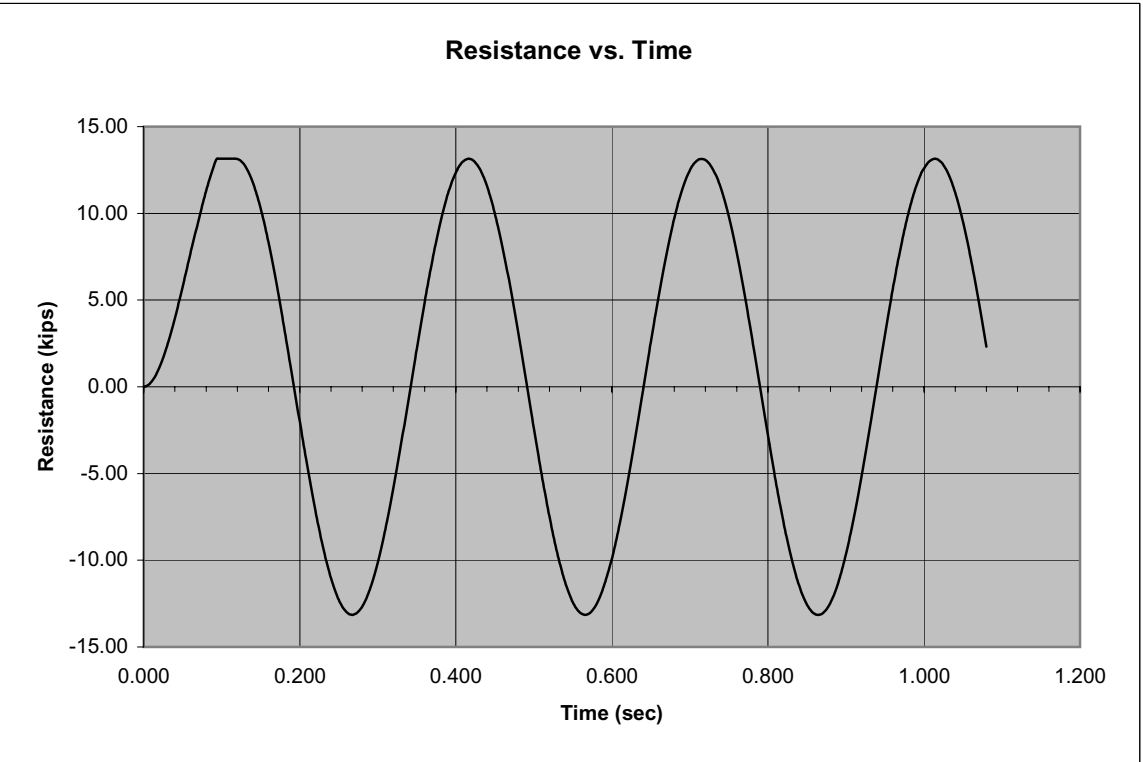
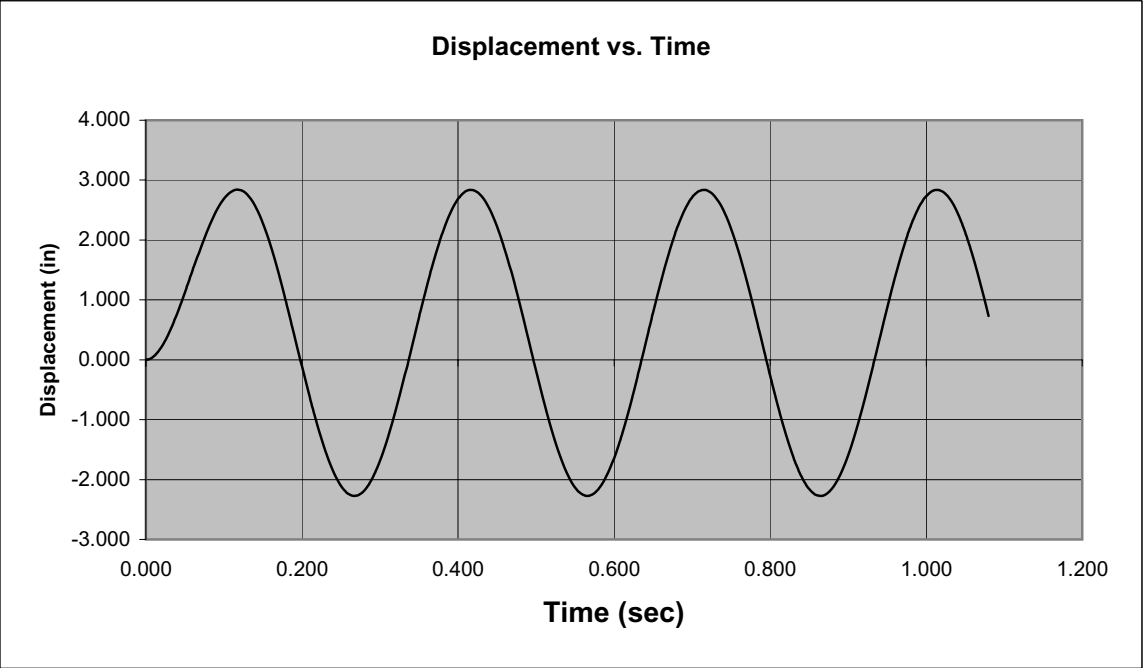
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.496	0.00	0.029	-53.511	112.4273	-1.31	-0.50
0.498	0.00	-0.078	-53.239	159.7159	-1.85	-0.71
0.500	0.00	-0.184	-52.872	206.7216	-2.40	-0.92
0.502	0.00	-0.289	-52.412	253.3613	-2.94	-1.13
0.504	0.00	-0.394	-51.859	299.5521	-3.48	-1.34
0.506	0.00	-0.497	-51.215	345.2124	-4.01	-1.54
0.508	0.00	-0.599	-50.479	390.2613	-4.53	-1.74
0.510	0.00	-0.699	-49.654	434.6190	-5.05	-1.94
0.512	0.00	-0.797	-48.741	478.2069	-5.55	-2.14
0.514	0.00	-0.894	-47.742	520.9478	-6.05	-2.33
0.516	0.00	-0.988	-46.658	562.7660	-6.54	-2.52
0.518	0.00	-1.080	-45.492	603.5875	-7.01	-2.70
0.520	0.00	-1.170	-44.245	643.3399	-7.47	-2.88
0.522	0.00	-1.257	-42.920	681.9529	-7.92	-3.05
0.524	0.00	-1.342	-41.519	719.3580	-8.35	-3.22
0.526	0.00	-1.423	-40.044	755.4890	-8.77	-3.38
0.528	0.00	-1.502	-38.498	790.2820	-9.18	-3.53
0.530	0.00	-1.577	-36.884	823.6752	-9.57	-3.68
0.532	0.00	-1.649	-35.205	855.6095	-9.94	-3.83
0.534	0.00	-1.718	-33.463	886.0284	-10.29	-3.96
0.536	0.00	-1.783	-31.662	914.8781	-10.62	-4.09
0.538	0.00	-1.844	-29.805	942.1073	-10.94	-4.21
0.540	0.00	-1.902	-27.895	967.6679	-11.24	-4.33
0.542	0.00	-1.956	-25.936	991.5146	-11.51	-4.43
0.544	0.00	-2.006	-23.931	1013.6052	-11.77	-4.53
0.546	0.00	-2.052	-21.884	1033.9005	-12.01	-4.62
0.548	0.00	-2.093	-19.797	1052.3646	-12.22	-4.71
0.550	0.00	-2.131	-17.676	1068.9648	-12.41	-4.78
0.552	0.00	-2.164	-15.523	1083.6717	-12.59	-4.85
0.554	0.00	-2.193	-13.343	1096.4592	-12.73	-4.90
0.556	0.00	-2.217	-11.139	1107.3048	-12.86	-4.95
0.558	0.00	-2.238	-8.916	1116.1891	-12.96	-4.99
0.560	0.00	-2.253	-6.677	1123.0964	-13.04	-5.02
0.562	0.00	-2.264	-4.426	1128.0146	-13.10	-5.04
0.564	0.00	-2.271	-2.167	1130.9349	-13.13	-5.06
0.566	0.00	-2.273	0.096	1131.8521	-13.14	-5.06
0.568	0.00	-2.270	2.359	1130.7646	-13.13	-5.06
0.570	0.00	-2.263	4.617	1127.6744	-13.10	-5.04
0.572	0.00	-2.252	6.867	1122.5868	-13.04	-5.02
0.574	0.00	-2.236	9.106	1115.5110	-12.95	-4.99
0.576	0.00	-2.216	11.328	1106.4594	-12.85	-4.95
0.578	0.00	-2.191	13.529	1095.4481	-12.72	-4.90
0.580	0.00	-2.161	15.707	1082.4966	-12.57	-4.84
0.582	0.00	-2.128	17.857	1067.6278	-12.40	-4.77
0.584	0.00	-2.090	19.976	1050.8681	-12.20	-4.70
0.586	0.00	-2.048	22.059	1032.2471	-11.99	-4.62
0.588	0.00	-2.002	24.103	1011.7979	-11.75	-4.52
0.590	0.00	-1.952	26.105	989.5565	-11.49	-4.42
0.592	0.00	-1.897	28.060	965.5626	-11.21	-4.32
0.594	0.00	-1.839	29.965	939.8584	-10.91	-4.20
0.596	0.00	-1.778	31.817	912.4896	-10.60	-4.08
0.598	0.00	-1.712	33.613	883.5047	-10.26	-3.95
0.600	0.00	-1.643	35.350	852.9549	-9.91	-3.81
0.602	0.00	-1.571	37.024	820.8944	-9.53	-3.67
0.604	0.00	-1.495	38.632	787.3799	-9.14	-3.52
0.606	0.00	-1.416	40.172	752.4709	-8.74	-3.36
0.608	0.00	-1.335	41.641	716.2291	-8.32	-3.20
0.610	0.00	-1.250	43.035	678.7188	-7.88	-3.03
0.612	0.00	-1.162	44.354	640.0063	-7.43	-2.86
0.614	0.00	-1.072	45.594	600.1603	-6.97	-2.68
0.616	0.00	-0.980	46.754	559.2514	-6.49	-2.50

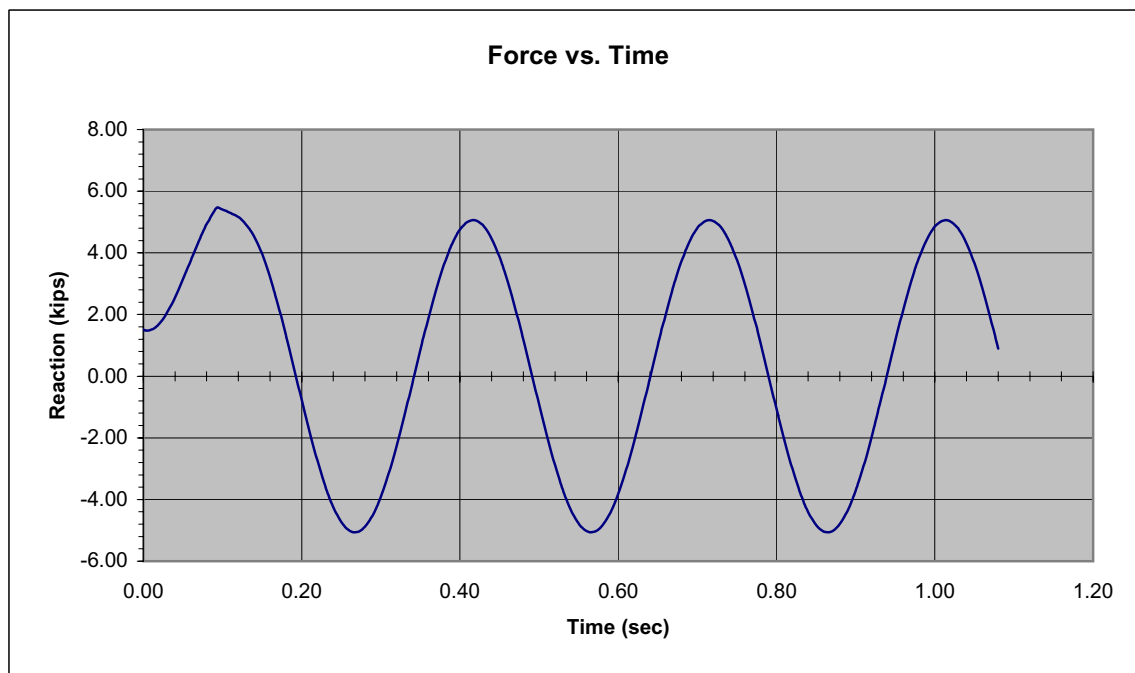
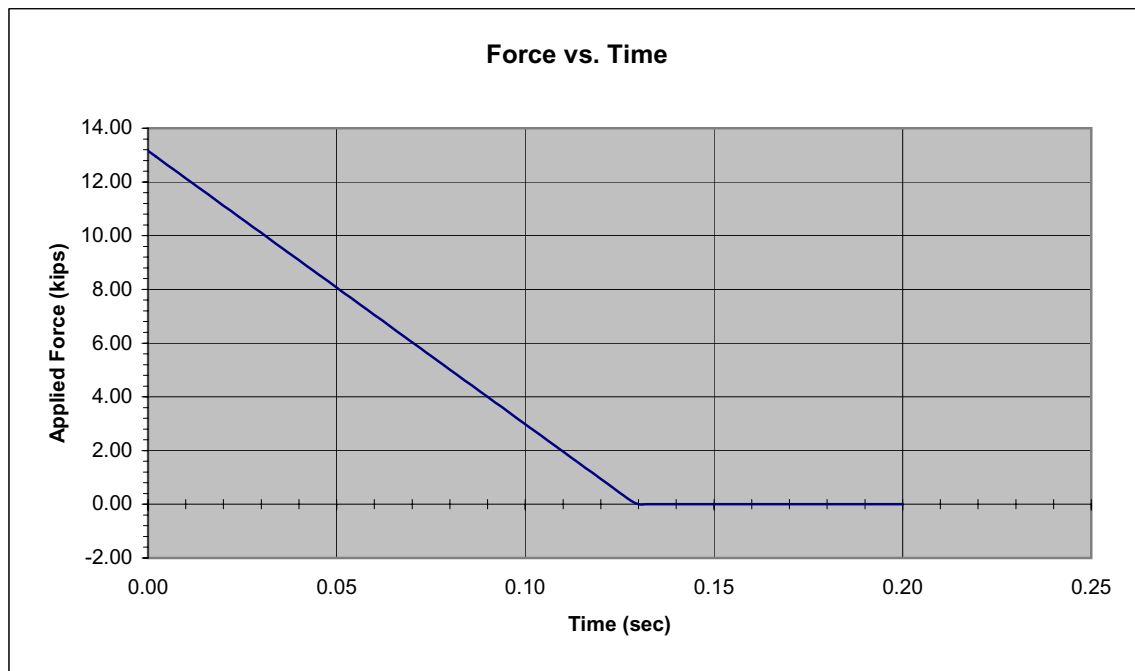
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.618	0.00	-0.885	47.830	517.3519	-6.01	-2.31
0.620	0.00	-0.789	48.822	474.5360	-5.51	-2.12
0.622	0.00	-0.690	49.728	430.8797	-5.00	-1.93
0.624	0.00	-0.590	50.545	386.4603	-4.49	-1.73
0.626	0.00	-0.488	51.273	341.3563	-3.96	-1.53
0.628	0.00	-0.385	51.910	295.6478	-3.43	-1.32
0.630	0.00	-0.281	52.455	249.4156	-2.90	-1.12
0.632	0.00	-0.175	52.907	202.7417	-2.35	-0.91
0.634	0.00	-0.069	53.266	155.7086	-1.81	-0.70
0.636	0.00	0.038	53.530	108.3998	-1.26	-0.48
0.638	0.00	0.145	53.699	60.8990	-0.71	-0.27
0.640	0.00	0.253	53.773	13.2904	-0.15	-0.06
0.642	0.00	0.360	53.752	-34.3418	0.40	0.15
0.644	0.00	0.467	53.636	-81.9132	0.95	0.37
0.646	0.00	0.575	53.425	-129.3395	1.50	0.58
0.648	0.00	0.681	53.119	-176.5367	2.05	0.79
0.650	0.00	0.787	52.719	-223.4213	2.59	1.00
0.652	0.00	0.892	52.225	-269.9101	3.13	1.21
0.654	0.00	0.996	51.640	-315.9209	3.67	1.41
0.656	0.00	1.098	50.962	-361.3721	4.20	1.62
0.658	0.00	1.200	50.195	-406.1833	4.72	1.82
0.660	0.00	1.299	49.338	-450.2750	5.23	2.01
0.662	0.00	1.397	48.394	-493.5693	5.73	2.21
0.664	0.00	1.493	47.365	-535.9894	6.22	2.40
0.666	0.00	1.586	46.251	-577.4601	6.71	2.58
0.668	0.00	1.678	45.056	-617.9081	7.18	2.76
0.670	0.00	1.766	43.781	-657.2616	7.63	2.94
0.672	0.00	1.853	42.428	-695.4510	8.08	3.11
0.674	0.00	1.936	41.000	-732.4087	8.51	3.27
0.676	0.00	2.017	39.500	-768.0692	8.92	3.43
0.678	0.00	2.094	37.929	-802.3692	9.32	3.59
0.680	0.00	2.168	36.292	-835.2482	9.70	3.73
0.682	0.00	2.239	34.590	-866.6478	10.06	3.87
0.684	0.00	2.307	32.827	-896.5124	10.41	4.01
0.686	0.00	2.370	31.005	-924.7892	10.74	4.13
0.688	0.00	2.431	29.129	-951.4280	11.05	4.25
0.690	0.00	2.487	27.201	-976.3817	11.34	4.37
0.692	0.00	2.539	25.225	-999.6060	11.61	4.47
0.694	0.00	2.588	23.205	-1021.0599	11.86	4.57
0.696	0.00	2.632	21.143	-1040.7053	12.09	4.65
0.698	0.00	2.672	19.044	-1058.5075	12.29	4.73
0.700	0.00	2.708	16.911	-1074.4348	12.48	4.80
0.702	0.00	2.740	14.748	-1088.4592	12.64	4.87
0.704	0.00	2.767	12.559	-1100.5557	12.78	4.92
0.706	0.00	2.790	10.348	-1110.7030	12.90	4.97
0.708	0.00	2.809	8.118	-1118.8831	12.99	5.00
0.710	0.00	2.823	5.874	-1125.0814	13.07	5.03
0.712	0.00	2.832	3.620	-1129.2870	13.11	5.05
0.714	0.00	2.837	1.359	-1131.4925	13.14	5.06
0.716	0.00	2.838	-0.904	-1131.6939	13.14	5.06
0.718	0.00	2.834	-3.166	-1129.8909	13.12	5.05
0.720	0.00	2.825	-5.422	-1126.0867	13.08	5.03
0.722	0.00	2.812	-7.668	-1120.2880	13.01	5.01
0.724	0.00	2.794	-9.901	-1112.5050	12.92	4.97
0.726	0.00	2.772	-12.116	-1102.7517	12.81	4.93
0.728	0.00	2.746	-14.310	-1091.0452	12.67	4.88
0.730	0.00	2.715	-16.478	-1077.4063	12.51	4.82
0.732	0.00	2.680	-18.618	-1061.8591	12.33	4.75
0.734	0.00	2.641	-20.724	-1044.4312	12.13	4.67
0.736	0.00	2.597	-22.794	-1025.1535	11.91	4.58
0.738	0.00	2.549	-24.823	-1004.0600	11.66	4.49

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.740	0.00	2.498	-26.808	-981.1882	11.39	4.39
0.742	0.00	2.442	-28.746	-956.5785	11.11	4.28
0.744	0.00	2.383	-30.633	-930.2746	10.80	4.16
0.746	0.00	2.320	-32.465	-902.3231	10.48	4.03
0.748	0.00	2.253	-34.240	-872.7733	10.14	3.90
0.750	0.00	2.183	-35.955	-841.6777	9.77	3.76
0.752	0.00	2.109	-37.606	-809.0914	9.40	3.62
0.754	0.00	2.032	-39.190	-775.0721	9.00	3.47
0.756	0.00	1.953	-40.705	-739.6800	8.59	3.31
0.758	0.00	1.870	-42.147	-702.9778	8.16	3.14
0.760	0.00	1.784	-43.515	-665.0305	7.72	2.97
0.762	0.00	1.696	-44.806	-625.9053	7.27	2.80
0.764	0.00	1.605	-46.018	-585.6715	6.80	2.62
0.766	0.00	1.512	-47.148	-544.4005	6.32	2.43
0.768	0.00	1.416	-48.194	-502.1652	5.83	2.25
0.770	0.00	1.319	-49.156	-459.0404	5.33	2.05
0.772	0.00	1.220	-50.030	-415.1027	4.82	1.86
0.774	0.00	1.119	-50.815	-370.4297	4.30	1.66
0.776	0.00	1.017	-51.511	-325.1007	3.78	1.45
0.778	0.00	0.913	-52.115	-279.1958	3.24	1.25
0.780	0.00	0.808	-52.627	-232.7965	2.70	1.04
0.782	0.00	0.702	-53.046	-185.9848	2.16	0.83
0.784	0.00	0.596	-53.371	-138.8437	1.61	0.62
0.786	0.00	0.489	-53.601	-91.4567	1.06	0.41
0.788	0.00	0.382	-53.736	-43.9077	0.51	0.20
0.790	0.00	0.274	-53.777	3.7190	-0.04	-0.02
0.792	0.00	0.167	-53.721	51.3392	-0.60	-0.23
0.794	0.00	0.059	-53.571	98.8684	-1.15	-0.44
0.796	0.00	-0.048	-53.326	146.2226	-1.70	-0.65
0.798	0.00	-0.154	-52.987	193.3177	-2.25	-0.86
0.800	0.00	-0.259	-52.553	240.0705	-2.79	-1.07
0.802	0.00	-0.364	-52.027	286.3980	-3.33	-1.28
0.804	0.00	-0.468	-51.408	332.2183	-3.86	-1.49
0.806	0.00	-0.570	-50.698	377.4502	-4.38	-1.69
0.808	0.00	-0.670	-49.899	422.0135	-4.90	-1.89
0.810	0.00	-0.769	-49.011	465.8294	-5.41	-2.08
0.812	0.00	-0.866	-48.037	508.8202	-5.91	-2.28
0.814	0.00	-0.961	-46.977	550.9099	-6.40	-2.46
0.816	0.00	-1.054	-45.834	592.0237	-6.88	-2.65
0.818	0.00	-1.145	-44.610	632.0891	-7.34	-2.83
0.820	0.00	-1.232	-43.307	671.0348	-7.79	-3.00
0.822	0.00	-1.318	-41.927	708.7921	-8.23	-3.17
0.824	0.00	-1.400	-40.473	745.2940	-8.66	-3.33
0.826	0.00	-1.480	-38.947	780.4758	-9.06	-3.49
0.828	0.00	-1.556	-37.352	814.2753	-9.46	-3.64
0.830	0.00	-1.629	-35.691	846.6326	-9.83	-3.79
0.832	0.00	-1.699	-33.967	877.4904	-10.19	-3.92
0.834	0.00	-1.765	-32.183	906.7940	-10.53	-4.05
0.836	0.00	-1.827	-30.342	934.4915	-10.85	-4.18
0.838	0.00	-1.886	-28.447	960.5338	-11.16	-4.29
0.840	0.00	-1.941	-26.501	984.8749	-11.44	-4.40
0.842	0.00	-1.992	-24.509	1007.4717	-11.70	-4.50
0.844	0.00	-2.039	-22.473	1028.2840	-11.94	-4.60
0.846	0.00	-2.082	-20.398	1047.2751	-12.16	-4.68
0.848	0.00	-2.121	-18.286	1064.4113	-12.36	-4.76
0.850	0.00	-2.155	-16.142	1079.6622	-12.54	-4.83
0.852	0.00	-2.185	-13.969	1093.0009	-12.69	-4.89
0.854	0.00	-2.211	-11.772	1104.4037	-12.83	-4.94
0.856	0.00	-2.232	-9.553	1113.8505	-12.94	-4.98
0.858	0.00	-2.249	-7.318	1121.3244	-13.02	-5.01
0.860	0.00	-2.262	-5.070	1126.8122	-13.09	-5.04

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.862	0.00	-2.269	-2.813	1130.3044	-13.13	-5.05
0.864	0.00	-2.273	-0.551	1131.7945	-13.14	-5.06
0.866	0.00	-2.272	1.712	1131.2801	-13.14	-5.06
0.868	0.00	-2.266	3.972	1128.7620	-13.11	-5.05
0.870	0.00	-2.256	6.225	1124.2446	-13.06	-5.03
0.872	0.00	-2.241	8.467	1117.7361	-12.98	-5.00
0.874	0.00	-2.222	10.694	1109.2478	-12.88	-4.96
0.876	0.00	-2.198	12.902	1098.7949	-12.76	-4.91
0.878	0.00	-2.170	15.087	1086.3959	-12.62	-4.86
0.880	0.00	-2.138	17.246	1072.0726	-12.45	-4.79
0.882	0.00	-2.101	19.374	1055.8506	-12.26	-4.72
0.884	0.00	-2.060	21.467	1037.7585	-12.05	-4.64
0.886	0.00	-2.015	23.523	1017.8283	-11.82	-4.55
0.888	0.00	-1.966	25.537	996.0954	-11.57	-4.45
0.890	0.00	-1.913	27.506	972.5983	-11.30	-4.35
0.892	0.00	-1.856	29.426	947.3785	-11.00	-4.24
0.894	0.00	-1.796	31.293	920.4808	-10.69	-4.12
0.896	0.00	-1.731	33.106	891.9527	-10.36	-3.99
0.898	0.00	-1.663	34.860	861.8449	-10.01	-3.85
0.900	0.00	-1.592	36.552	830.2105	-9.64	-3.71
0.902	0.00	-1.517	38.179	797.1058	-9.26	-3.56
0.904	0.00	-1.439	39.739	762.5892	-8.86	-3.41
0.906	0.00	-1.358	41.228	726.7220	-8.44	-3.25
0.908	0.00	-1.274	42.644	689.5676	-8.01	-3.08
0.910	0.00	-1.188	43.985	651.1919	-7.56	-2.91
0.912	0.00	-1.098	45.248	611.6629	-7.10	-2.73
0.914	0.00	-1.007	46.431	571.0504	-6.63	-2.55
0.916	0.00	-0.913	47.531	529.4266	-6.15	-2.37
0.918	0.00	-0.817	48.547	486.8650	-5.65	-2.18
0.920	0.00	-0.719	49.478	443.4412	-5.15	-1.98
0.922	0.00	-0.619	50.320	399.2319	-4.64	-1.79
0.924	0.00	-0.517	51.074	354.3155	-4.11	-1.58
0.926	0.00	-0.415	51.737	308.7716	-3.59	-1.38
0.928	0.00	-0.311	52.309	262.6808	-3.05	-1.17
0.930	0.00	-0.205	52.787	216.1248	-2.51	-0.97
0.932	0.00	-0.099	53.173	169.1859	-1.96	-0.76
0.934	0.00	0.007	53.464	121.9474	-1.42	-0.55
0.936	0.00	0.114	53.660	74.4929	-0.87	-0.33
0.938	0.00	0.222	53.762	26.9065	-0.31	-0.12
0.940	0.00	0.329	53.768	-20.7276	0.24	0.09
0.942	0.00	0.437	53.679	-68.3250	0.79	0.31
0.944	0.00	0.544	53.495	-115.8013	1.34	0.52
0.946	0.00	0.651	53.216	-163.0726	1.89	0.73
0.948	0.00	0.757	52.843	-210.0550	2.44	0.94
0.950	0.00	0.862	52.376	-256.6654	2.98	1.15
0.952	0.00	0.966	51.816	-302.8212	3.52	1.35
0.954	0.00	1.069	51.165	-348.4407	4.05	1.56
0.956	0.00	1.171	50.423	-393.4430	4.57	1.76
0.958	0.00	1.271	49.592	-437.7484	5.08	1.96
0.960	0.00	1.369	48.673	-481.2786	5.59	2.15
0.962	0.00	1.466	47.668	-523.9563	6.08	2.34
0.964	0.00	1.560	46.578	-565.7060	6.57	2.53
0.966	0.00	1.652	45.406	-606.4537	7.04	2.71
0.968	0.00	1.741	44.153	-646.1273	7.50	2.89
0.970	0.00	1.828	42.823	-684.6565	7.95	3.06
0.972	0.00	1.913	41.416	-721.9731	8.38	3.23
0.974	0.00	1.994	39.936	-758.0109	8.80	3.39
0.976	0.00	2.072	38.385	-792.7062	9.21	3.54
0.978	0.00	2.147	36.767	-825.9975	9.59	3.69
0.980	0.00	2.219	35.083	-857.8258	9.96	3.84
0.982	0.00	2.288	33.337	-888.1348	10.31	3.97

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.984	0.00	2.353	31.532	-916.8707	10.65	4.10
0.986	0.00	2.414	29.671	-943.9827	10.96	4.22
0.988	0.00	2.471	27.758	-969.4227	11.26	4.33
0.990	0.00	2.525	25.795	-993.1458	11.53	4.44
0.992	0.00	2.574	23.787	-1015.1098	11.79	4.54
0.994	0.00	2.620	21.736	-1035.2759	12.02	4.63
0.996	0.00	2.661	19.647	-1053.6084	12.24	4.71
0.998	0.00	2.698	17.524	-1070.0747	12.43	4.78
1.000	0.00	2.731	15.369	-1084.6458	12.60	4.85
1.002	0.00	2.760	13.187	-1097.2958	12.74	4.91
1.004	0.00	2.784	10.982	-1108.0023	12.87	4.95
1.006	0.00	2.804	8.757	-1116.7463	12.97	4.99
1.008	0.00	2.819	6.517	-1123.5124	13.05	5.02
1.010	0.00	2.830	4.265	-1128.2886	13.10	5.04
1.012	0.00	2.836	2.006	-1131.0664	13.14	5.06
1.014	0.00	2.838	-0.257	-1131.8410	13.14	5.06
1.016	0.00	2.835	-2.520	-1130.6108	13.13	5.06
1.018	0.00	2.828	-4.778	-1127.3781	13.09	5.04
1.020	0.00	2.816	-7.027	-1122.1487	13.03	5.02
1.022	0.00	2.800	-9.264	-1114.9318	12.95	4.99
1.024	0.00	2.779	-11.485	-1105.7401	12.84	4.94
1.026	0.00	2.754	-13.685	-1094.5900	12.71	4.89
1.028	0.00	2.724	-15.861	-1081.5012	12.56	4.84
1.030	0.00	2.690	-18.009	-1066.4969	12.39	4.77
1.032	0.00	2.652	-20.126	-1049.6036	12.19	4.69
1.034	0.00	2.610	-22.206	-1030.8514	11.97	4.61
1.036	0.00	2.563	-24.247	-1010.2733	11.73	4.52
1.038	0.00	2.513	-26.245	-987.9059	11.47	4.42
1.040	0.00	2.459	-28.197	-963.7887	11.19	4.31
1.042	0.00	2.400	-30.099	-937.9645	10.89	4.19
1.044	0.00	2.338	-31.947	-910.4790	10.57	4.07
1.046	0.00	2.272	-33.739	-881.3809	10.24	3.94
1.048	0.00	2.203	-35.471	-850.7218	9.88	3.80
1.050	0.00	2.131	-37.140	-818.5558	9.51	3.66
1.052	0.00	2.055	-38.744	-784.9401	9.12	3.51
1.054	0.00	1.976	-40.279	-749.9341	8.71	3.35
1.056	0.00	1.894	-41.742	-713.5999	8.29	3.19
1.058	0.00	1.809	-43.132	-676.0018	7.85	3.02
1.060	0.00	1.721	-44.445	-637.2063	7.40	2.85
1.062	0.00	1.631	-45.680	-597.2823	6.94	2.67
1.064	0.00	1.539	-46.833	-556.3004	6.46	2.49
1.066	0.00	1.444	-47.904	-514.3332	5.97	2.30
1.068	0.00	1.347	-48.890	-471.4550	5.48	2.11
1.070	0.00	1.248	-49.789	-427.7418	4.97	1.91
1.072	0.00	1.148	-50.600	-383.2710	4.45	1.71
1.074	0.00	1.046	-51.321	-338.1213	3.93	1.51
1.076	0.00	0.943	-51.952	-292.3728	3.40	1.31
1.078	0.00	0.838	-52.490	-246.1065	2.86	1.10
1.080	0.00	0.733	-52.936	-199.4042	2.32	0.89





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Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

REAR WALL PANELS SS # 1 & SS # 2

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

REAR WALL PANELS ; SS # 1

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject : REAR WALL PROPERTIES SS # 1

Sheet no. : of
Date : 2-Mar-04
Design by : MGG
Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Blast Load (Ps) = 8.470 KN/m²
Dead Load (DL) = 176.90 psf

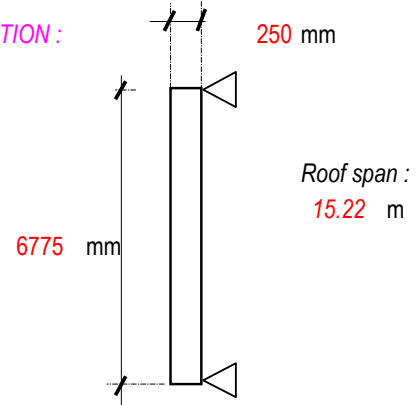
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
Reinf., f_y = 414 Mpa

PANEL DATA :

Panel Width, width = 2500 mm
Long Span, l_b = 6775 mm
Wall, t_{TRIAL} = 250 mm

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure,	P_s	=	1.96	psi	(SEE LOADING CALCULATION)
Blast time duration,	T_d	=	200	msec	
Consider 1ft. width slab:		=	1.0000	ft.	
Moment of Inertia,	I_g	=	953.50	in ⁴	(33072917 mm ⁴)
Area of Bar ($\emptyset$), 25 mm $\emptyset$	A_b	=	0.7609	in ²	(491 mm ²)
Rebar Spacing for top & bottom,	S	=	12.0000	in	(305 mm)
Area of Rebar PER 1ft. unit width,	A_s	=	0.7609	in ²	(13 mm ²)
Effective Slab Depth,	$d_c = d_s$	=	8.3425	in	(212 mm)
$f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for bending)		=	5954.3	psi	(41 Mpa.)
$f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for shear)		=	5504.0	psi	(38 Mpa.)
$f_{dy} = (SIF) \times (DIF) \times f_y$		=	77276	psi	(533 Mpa.)
Modulus of Elasticity,	$E_{dc} = 57000 \times (f'_c)^{1/2}$	=	4032041	psi	(27801 Mpa.)
Steel ratio,	$p = A_s / b \times d$	=	0.0076	>	$200 / f_{dy} = 0.003$
Stress block,	$a = A_s \times f_{dy} / 0.85 \times f'_{dc} \times b$	=	0.9681	in	(25 mm)
Modular ratio,	$n = E_s / E_{dm}$	=	7.1940		

a.) BENDING RESISTANCE

Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$	=	462046.175	lb-in	(52202 KN-m)
Resistance at cracked moment, $R_b = 8 \times M_p / L$	=	13857.98	lb	(61.64 KN)
Unit Resistance, $r_b = R_b / L \times \text{unit width}$	=	4.3296	psi	(0.0299 Mpa.)

b.) SHEAR RESISTANCE

Shear,	$V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$	=	14854.1	lb	(66 KN)
At Support,	$R_s = V_n \times L / (0.5 \times L - t_w)$	=	31691	lb	(141 KN)
Unit Resistance,	$r_s = R_s / L \times \text{unit width}$	=	9.901	psi	(0.0683 Mpa.)

$\therefore$ Dynamic Resistance Controlled by Bending, R_u = 4.3296 psi > 1.96 $\therefore$ O.K!
13.8580 kips 6.29 kips

Cracked Moment of Inertia:

$n \times A_s$	=	5.4736	in ²	(3531 mm ²)
$C = -n \times A_s + [n \times A_s \times (n \times A_s + 2 \times b \times d)]^{0.5} / b$	=	2.3401	in ²	(1510 mm ²)
$I_{cr} = b \times C^3 / 3 + n \times A_s \times (d - C)^2$	=	248.47	in ⁴	(103419712 mm ⁴)

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$$\begin{aligned}
 \text{Average Moment of Inertia, } I_a &= (I_g + I_{cr}) / 2 = 600.981 \text{ in}^4 \quad (250147356 \text{ mm}^4) \\
 \text{Effective Stiffness, } K &= 384 \times E_{dc} \times I_a / 5 L^3 = 9.8066 \text{ kips / in} \\
 \text{Yield Deflection, } Y_e &= R_u / K = 1.41312 \text{ in} \\
 \text{Beam Mass} &= \gamma_c \times t_w \times \text{unit width} \times \text{span} / \text{gravity acc.} = 0.00708 \text{ kip-sec}^2 / \text{in} \\
 \text{Roof Mass} &= \text{Roof DL} \times \text{unit width} \times 1/2 \text{ roof span} / \text{gravity acc.} = 0.01144 \text{ kip-sec}^2 / \text{in} \\
 \text{Total Mass, } M &= 0.01853 \text{ kip-sec}^2 / \text{in} \\
 \text{Average Stiffness, } K_{LM} &= 0.72000 \text{ FROM ASCE TABLE 6.1} \\
 \text{Equivalent Mass, } M_e &= K_{LM} \times \text{Beam Mass} = 0.01334 \text{ kip-sec}^2 / \text{in} \\
 \text{Period of Vibration, } t_n &= 2 \times \pi \times (M_e / K)^{0.5} = 0.23174 \text{ sec} \\
 \text{Time increment, } t_n/10 &= 0.02317 \text{ sec} ; \quad \text{use } \Delta t = 0.00200 \text{ sec} \\
 t_d / t_n &= 0.86305 \\
 R_u / P_s &= 2.20356 \\
 \\
 \text{Arrival time ; } t_a &= 0.04321 \text{ sec} \\
 \text{Rise time ; } t_r &= 0.01979 \text{ sec} \\
 \text{Time at Peak pressure; } t_a + t_r &= 0.06300 \text{ sec} \\
 \text{Time Duration; } T_d &= 0.20000 \text{ sec} \\
 \text{Time end ; } T_e = t_a + T_d &= 0.24321 \text{ sec}
 \end{aligned}$$

NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	9.80664453 kips / in
Mass	M	=	0.013339788 kip-sec ² / in
Tension resistance	Rut	=	13.8579753 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	0.00 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.243 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load
load (kips)
6.29
0

Length	256.000
y _{max} =	0.981
r _{max} =	9.623
μ =	0.694
θ =	0.439
θ =	< 2 degrees
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.04321	0.00	0.000	0.000	0.0000	0.00	0.00
0.04400	0.25	0.000	0.019	18.8103	0.00	0.03
0.046	0.89	0.000	0.104	66.3759	0.00	0.10
0.048	1.52	0.000	0.284	113.7464	0.00	0.18
0.050	2.16	0.001	0.559	160.7826	0.01	0.25
0.052	2.79	0.003	0.927	207.3463	0.03	0.33
0.054	3.43	0.005	1.387	253.3005	0.05	0.41
0.056	4.06	0.008	1.939	298.5103	0.08	0.50
0.058	4.70	0.013	2.581	342.8427	0.13	0.59
0.060	5.34	0.019	3.310	386.1674	0.18	0.68
0.062	5.97	0.026	4.124	428.3572	0.26	0.79
0.064	6.25	0.035	4.995	442.8626	0.35	0.85
0.066	6.18	0.046	5.868	429.6412	0.45	0.89
0.068	6.11	0.059	6.713	415.1570	0.58	0.93
0.070	6.04	0.073	7.527	399.4525	0.72	0.97
0.072	5.97	0.089	8.309	382.5741	0.87	1.02
0.074	5.91	0.106	9.056	364.5712	1.04	1.08
0.076	5.84	0.125	9.766	345.4968	1.23	1.14
0.078	5.77	0.145	10.437	325.4069	1.42	1.21
0.080	5.70	0.167	11.067	304.3606	1.64	1.28
0.082	5.63	0.190	11.654	282.4197	1.86	1.36
0.084	5.56	0.213	12.196	259.6488	2.09	1.44
0.086	5.49	0.238	12.692	236.1148	2.34	1.53
0.088	5.42	0.264	13.140	211.8867	2.59	1.62
0.090	5.35	0.291	13.539	187.0359	2.85	1.71
0.092	5.28	0.318	13.887	161.6354	3.12	1.81
0.094	5.21	0.346	14.185	135.7598	3.40	1.91
0.096	5.14	0.375	14.430	109.4852	3.68	2.01
0.098	5.07	0.404	14.622	82.8888	3.96	2.11
0.100	5.00	0.433	14.761	56.0488	4.25	2.21
0.102	4.93	0.463	14.846	29.0441	4.54	2.31
0.104	4.86	0.493	14.877	1.9540	4.83	2.42

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.106	4.79	0.522	14.854	-25.1419	5.12	2.52
0.108	4.72	0.552	14.777	-52.1638	5.41	2.63
0.110	4.65	0.582	14.646	-79.0325	5.70	2.73
0.112	4.58	0.611	14.461	-105.6688	5.99	2.83
0.114	4.51	0.639	14.223	-131.9946	6.27	2.93
0.116	4.44	0.668	13.933	-157.9324	6.55	3.03
0.118	4.37	0.695	13.592	-183.4061	6.82	3.13
0.120	4.30	0.722	13.200	-208.3407	7.08	3.22
0.122	4.23	0.748	12.759	-232.6629	7.33	3.31
0.124	4.16	0.773	12.270	-256.3014	7.58	3.40
0.126	4.09	0.797	11.735	-279.1865	7.81	3.48
0.128	4.02	0.820	11.154	-301.2510	8.04	3.56
0.130	3.95	0.841	10.531	-322.4302	8.25	3.63
0.132	3.88	0.862	9.866	-342.6617	8.45	3.70
0.134	3.81	0.881	9.161	-361.8860	8.64	3.76
0.136	3.74	0.898	8.419	-380.0467	8.81	3.82
0.138	3.67	0.915	7.642	-397.0904	8.97	3.88
0.140	3.60	0.929	6.832	-412.9671	9.11	3.92
0.142	3.53	0.942	5.991	-427.6299	9.24	3.96
0.144	3.46	0.953	5.123	-441.0359	9.35	4.00
0.146	3.39	0.962	4.229	-453.1456	9.44	4.02
0.148	3.32	0.970	3.311	-463.9235	9.51	4.04
0.150	3.25	0.976	2.374	-473.3379	9.57	4.06
0.152	3.18	0.979	1.420	-481.3610	9.60	4.06
0.154	3.11	0.981	0.450	-487.9694	9.62	4.06
0.156	3.04	0.981	-0.531	-493.1435	9.62	4.05
0.158	2.97	0.979	-1.521	-496.8683	9.60	4.04
0.160	2.90	0.975	-2.517	-499.1326	9.56	4.02
0.162	2.83	0.969	-3.516	-499.9300	9.50	3.98
0.164	2.76	0.961	-4.515	-499.2580	9.42	3.95
0.166	2.69	0.951	-5.512	-497.1186	9.33	3.90
0.168	2.62	0.939	-6.502	-493.5182	9.21	3.85
0.170	2.55	0.925	-7.484	-488.4672	9.07	3.79
0.172	2.48	0.909	-8.455	-481.9805	8.91	3.72
0.174	2.42	0.891	-9.411	-474.0773	8.74	3.64
0.176	2.35	0.871	-10.350	-464.7807	8.55	3.56
0.178	2.28	0.850	-11.268	-454.1180	8.33	3.47
0.180	2.21	0.826	-12.165	-442.1206	8.10	3.37
0.182	2.14	0.801	-13.036	-428.8237	7.86	3.27
0.184	2.07	0.774	-13.879	-414.2665	7.59	3.16
0.186	2.00	0.746	-14.691	-398.4917	7.31	3.04
0.188	1.93	0.715	-15.472	-381.5457	7.02	2.92
0.190	1.86	0.684	-16.217	-363.4783	6.71	2.80
0.192	1.79	0.651	-16.924	-344.3426	6.38	2.66
0.194	1.72	0.616	-17.593	-324.1948	6.04	2.52
0.196	1.65	0.580	-18.220	-303.0941	5.69	2.38

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.198	1.58	0.543	-18.804	-281.1026	5.33	2.23
0.200	1.51	0.505	-19.344	-258.2849	4.95	2.08
0.202	1.44	0.466	-19.837	-234.7081	4.57	1.92
0.204	1.37	0.426	-20.282	-210.4415	4.18	1.76
0.206	1.30	0.385	-20.678	-185.5563	3.77	1.60
0.208	1.23	0.343	-21.024	-160.1258	3.36	1.44
0.210	1.16	0.301	-21.318	-134.2246	2.95	1.27
0.212	1.09	0.258	-21.560	-107.9289	2.53	1.10
0.214	1.02	0.215	-21.749	-81.3160	2.10	0.93
0.216	0.95	0.171	-21.885	-54.4641	1.68	0.75
0.218	0.88	0.127	-21.967	-27.4522	1.25	0.58
0.220	0.81	0.083	-21.995	-0.3595	0.81	0.41
0.222	0.74	0.039	-21.968	26.7342	0.38	0.23
0.224	0.67	-0.005	-21.888	53.7493	-0.05	0.06
0.226	0.60	-0.048	-21.754	80.6065	-0.47	-0.11
0.228	0.53	-0.092	-21.566	107.2267	-0.90	-0.29
0.230	0.46	-0.135	-21.325	133.5318	-1.32	-0.46
0.232	0.39	-0.177	-21.032	159.4444	-1.74	-0.62
0.234	0.32	-0.219	-20.688	184.8884	-2.15	-0.79
0.236	0.25	-0.260	-20.293	209.7890	-2.55	-0.95
0.238	0.18	-0.300	-19.849	234.0730	-2.94	-1.11
0.240	0.11	-0.339	-19.357	257.6690	-3.33	-1.27
0.242	0.04	-0.377	-18.819	280.5077	-3.70	-1.42
0.244	0.00	-0.414	-18.234	304.5971	-4.06	-1.56
0.246	0.00	-0.450	-17.599	330.9457	-4.41	-1.70
0.248	0.00	-0.485	-16.911	356.3217	-4.75	-1.83
0.250	0.00	-0.518	-16.174	380.6504	-5.08	-1.95
0.252	0.00	-0.549	-15.390	403.8603	-5.39	-2.07
0.254	0.00	-0.579	-14.560	425.8832	-5.68	-2.19
0.256	0.00	-0.608	-13.688	446.6544	-5.96	-2.29
0.258	0.00	-0.634	-12.775	466.1128	-6.22	-2.39
0.260	0.00	-0.659	-11.825	484.2012	-6.46	-2.49
0.262	0.00	-0.681	-10.839	500.8665	-6.68	-2.57
0.264	0.00	-0.702	-9.823	516.0597	-6.88	-2.65
0.266	0.00	-0.721	-8.777	529.7362	-7.07	-2.72
0.268	0.00	-0.737	-7.705	541.8556	-7.23	-2.78
0.270	0.00	-0.751	-6.611	552.3825	-7.37	-2.84
0.272	0.00	-0.764	-5.497	561.2859	-7.49	-2.88
0.274	0.00	-0.773	-4.367	568.5395	-7.58	-2.92
0.276	0.00	-0.781	-3.225	574.1222	-7.66	-2.95
0.278	0.00	-0.786	-2.073	578.0174	-7.71	-2.97
0.280	0.00	-0.789	-0.914	580.2138	-7.74	-2.98
0.282	0.00	-0.790	0.247	580.7049	-7.75	-2.98
0.284	0.00	-0.788	1.407	579.4891	-7.73	-2.98
0.286	0.00	-0.784	2.563	576.5702	-7.69	-2.96
0.288	0.00	-0.778	3.711	571.9567	-7.63	-2.94
0.290	0.00	-0.769	4.849	565.6621	-7.55	-2.91
0.292	0.00	-0.759	5.972	557.7050	-7.44	-2.86
0.294	0.00	-0.746	7.078	548.1087	-7.31	-2.81
0.296	0.00	-0.730	8.163	536.9014	-7.16	-2.76
0.298	0.00	-0.713	9.224	524.1161	-6.99	-2.69
0.300	0.00	-0.693	10.258	509.7904	-6.80	-2.62

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.302	0.00	-0.672	11.262	493.9663	-6.59	-2.54
0.304	0.00	-0.648	12.232	476.6904	-6.36	-2.45
0.306	0.00	-0.623	13.167	458.0135	-6.11	-2.35
0.308	0.00	-0.596	14.063	437.9903	-5.84	-2.25
0.310	0.00	-0.567	14.918	416.6799	-5.56	-2.14
0.312	0.00	-0.536	15.729	394.1448	-5.26	-2.02
0.314	0.00	-0.504	16.493	370.4513	-4.94	-1.90
0.316	0.00	-0.470	17.209	345.6689	-4.61	-1.78
0.318	0.00	-0.435	17.875	319.8706	-4.27	-1.64
0.320	0.00	-0.399	18.488	293.1321	-3.91	-1.51
0.322	0.00	-0.361	19.047	265.5321	-3.54	-1.36
0.324	0.00	-0.323	19.549	237.1517	-3.16	-1.22
0.326	0.00	-0.283	19.995	208.0742	-2.78	-1.07
0.328	0.00	-0.243	20.381	178.3852	-2.38	-0.92
0.330	0.00	-0.202	20.708	148.1719	-1.98	-0.76
0.332	0.00	-0.160	20.973	117.5230	-1.57	-0.60
0.334	0.00	-0.118	21.177	86.5288	-1.15	-0.44
0.336	0.00	-0.075	21.319	55.2803	-0.74	-0.28
0.338	0.00	-0.032	21.398	23.8692	-0.32	-0.12
0.340	0.00	0.010	21.415	-7.6119	0.10	0.04
0.342	0.00	0.053	21.368	-39.0708	0.52	0.20
0.344	0.00	0.096	21.258	-70.4147	0.94	0.36
0.346	0.00	0.138	21.086	-101.5517	1.35	0.52
0.348	0.00	0.180	20.852	-132.3903	1.77	0.68
0.350	0.00	0.222	20.557	-162.8397	2.17	0.84
0.352	0.00	0.262	20.202	-192.8105	2.57	0.99
0.354	0.00	0.302	19.787	-222.2147	2.96	1.14
0.356	0.00	0.341	19.313	-250.9657	3.35	1.29
0.358	0.00	0.379	18.783	-278.9791	3.72	1.43
0.360	0.00	0.416	18.198	-306.1725	4.08	1.57
0.362	0.00	0.452	17.560	-332.4661	4.44	1.71
0.364	0.00	0.487	16.869	-357.7825	4.77	1.84
0.366	0.00	0.520	16.130	-382.0473	5.10	1.96
0.368	0.00	0.551	15.342	-405.1892	5.41	2.08
0.370	0.00	0.581	14.510	-427.1403	5.70	2.19
0.372	0.00	0.609	13.635	-447.8359	5.97	2.30
0.374	0.00	0.636	12.720	-467.2152	6.23	2.40
0.376	0.00	0.660	11.767	-485.2214	6.47	2.49
0.378	0.00	0.683	10.780	-501.8014	6.69	2.58
0.380	0.00	0.703	9.762	-516.9066	6.90	2.65
0.382	0.00	0.722	8.714	-530.4925	7.08	2.72
0.384	0.00	0.738	7.641	-542.5192	7.24	2.79
0.386	0.00	0.752	6.546	-552.9514	7.38	2.84
0.388	0.00	0.764	5.431	-561.7584	7.49	2.89
0.390	0.00	0.774	4.301	-568.9144	7.59	2.92
0.392	0.00	0.781	3.157	-574.3982	7.66	2.95
0.394	0.00	0.787	2.005	-578.1937	7.71	2.97
0.396	0.00	0.789	0.846	-580.2899	7.74	2.98
0.398	0.00	0.790	-0.315	-580.6806	7.75	2.98
0.400	0.00	0.788	-1.475	-579.3645	7.73	2.98
0.402	0.00	0.784	-2.631	-576.3456	7.69	2.96
0.404	0.00	0.778	-3.779	-571.6328	7.63	2.94

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.406	0.00	0.769	-4.915	-565.2399	7.54	2.90
0.408	0.00	0.758	-6.038	-557.1856	7.43	2.86
0.410	0.00	0.745	-7.143	-547.4937	7.30	2.81
0.412	0.00	0.729	-8.226	-536.1927	7.15	2.75
0.414	0.00	0.712	-9.286	-523.3157	6.98	2.69
0.416	0.00	0.692	-10.318	-508.9006	6.79	2.61
0.418	0.00	0.671	-11.320	-492.9898	6.58	2.53
0.420	0.00	0.647	-12.288	-475.6301	6.34	2.44
0.422	0.00	0.621	-13.221	-456.8724	6.09	2.35
0.424	0.00	0.594	-14.115	-436.7718	5.83	2.24
0.426	0.00	0.565	-14.967	-415.3876	5.54	2.13
0.428	0.00	0.534	-15.775	-392.7825	5.24	2.02
0.430	0.00	0.502	-16.537	-369.0229	4.92	1.90
0.432	0.00	0.468	-17.250	-344.1788	4.59	1.77
0.434	0.00	0.433	-17.912	-318.3230	4.25	1.63
0.436	0.00	0.397	-18.522	-291.5317	3.89	1.50
0.438	0.00	0.359	-19.078	-263.8835	3.52	1.36
0.440	0.00	0.320	-19.577	-235.4597	3.14	1.21
0.442	0.00	0.281	-20.019	-206.3439	2.75	1.06
0.444	0.00	0.240	-20.402	-176.6216	2.36	0.91
0.446	0.00	0.199	-20.725	-146.3802	1.95	0.75
0.448	0.00	0.157	-20.987	-115.7086	1.54	0.59
0.450	0.00	0.115	-21.187	-84.6968	1.13	0.43
0.452	0.00	0.073	-21.325	-53.4362	0.71	0.27
0.454	0.00	0.030	-21.401	-22.0185	0.29	0.11
0.456	0.00	-0.013	-21.413	9.4640	-0.13	-0.05
0.458	0.00	-0.056	-21.363	40.9186	-0.55	-0.21
0.460	0.00	-0.098	-21.250	72.2529	-0.96	-0.37
0.462	0.00	-0.141	-21.074	103.3749	-1.38	-0.53
0.464	0.00	-0.183	-20.837	134.1931	-1.79	-0.69
0.466	0.00	-0.224	-20.538	164.6168	-2.20	-0.85
0.468	0.00	-0.265	-20.179	194.5567	-2.60	-1.00
0.470	0.00	-0.305	-19.760	223.9248	-2.99	-1.15
0.472	0.00	-0.344	-19.284	252.6347	-3.37	-1.30
0.474	0.00	-0.382	-18.750	280.6022	-3.74	-1.44
0.476	0.00	-0.419	-18.162	307.7448	-4.11	-1.58
0.478	0.00	-0.454	-17.520	333.9830	-4.46	-1.72
0.480	0.00	-0.489	-16.827	359.2396	-4.79	-1.84
0.482	0.00	-0.522	-16.085	383.4403	-5.12	-1.97
0.484	0.00	-0.553	-15.295	406.5140	-5.42	-2.09
0.486	0.00	-0.583	-14.460	428.3930	-5.71	-2.20
0.488	0.00	-0.611	-13.582	449.0128	-5.99	-2.31
0.490	0.00	-0.637	-12.665	468.3130	-6.25	-2.41
0.492	0.00	-0.661	-11.710	486.2366	-6.49	-2.50
0.494	0.00	-0.684	-10.721	502.7312	-6.71	-2.58
0.496	0.00	-0.704	-9.701	517.7482	-6.91	-2.66
0.498	0.00	-0.723	-8.652	531.2435	-7.09	-2.73
0.500	0.00	-0.739	-7.578	543.1774	-7.25	-2.79
0.502	0.00	-0.753	-6.481	553.5148	-7.38	-2.84
0.504	0.00	-0.765	-5.365	562.2253	-7.50	-2.89
0.506	0.00	-0.774	-4.234	569.2834	-7.59	-2.92
0.508	0.00	-0.782	-3.090	574.6683	-7.67	-2.95
0.510	0.00	-0.787	-1.937	578.3642	-7.72	-2.97
0.512	0.00	-0.789	-0.778	580.3602	-7.74	-2.98
0.514	0.00	-0.790	0.383	580.6504	-7.75	-2.98
0.516	0.00	-0.788	1.543	579.2340	-7.73	-2.97
0.518	0.00	-0.784	2.698	576.1152	-7.69	-2.96
0.520	0.00	-0.777	3.846	571.3031	-7.62	-2.93
0.522	0.00	-0.768	4.982	564.8119	-7.53	-2.90

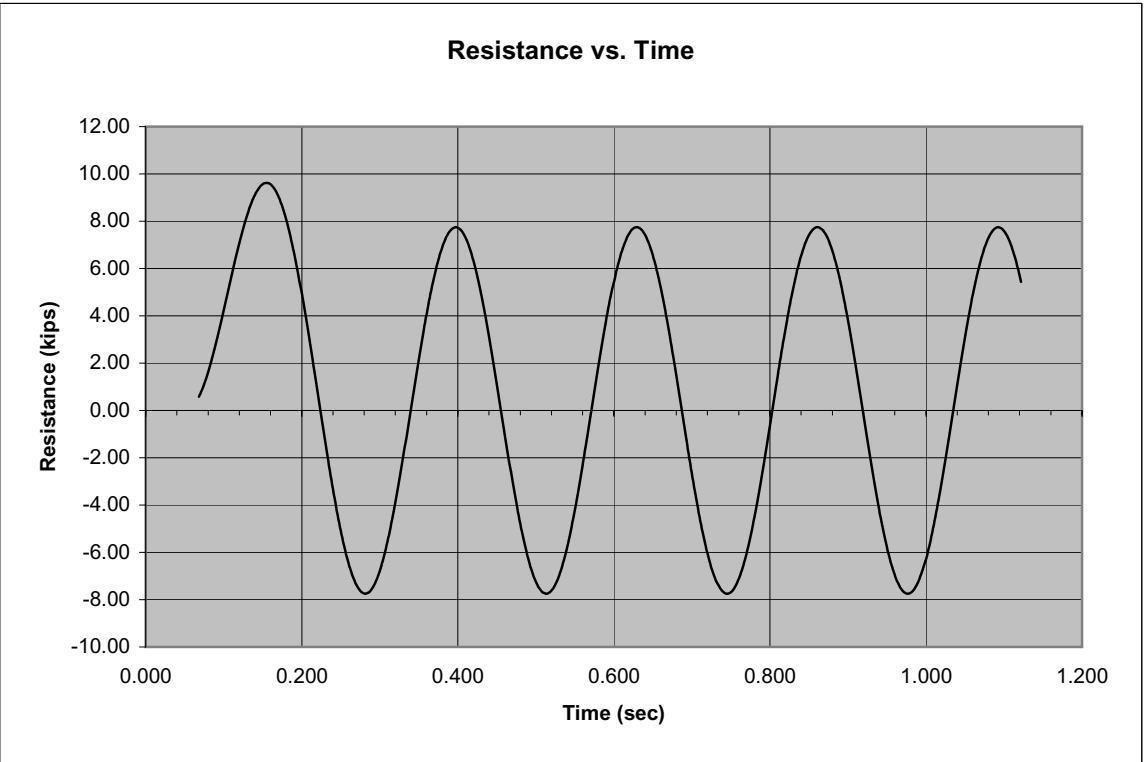
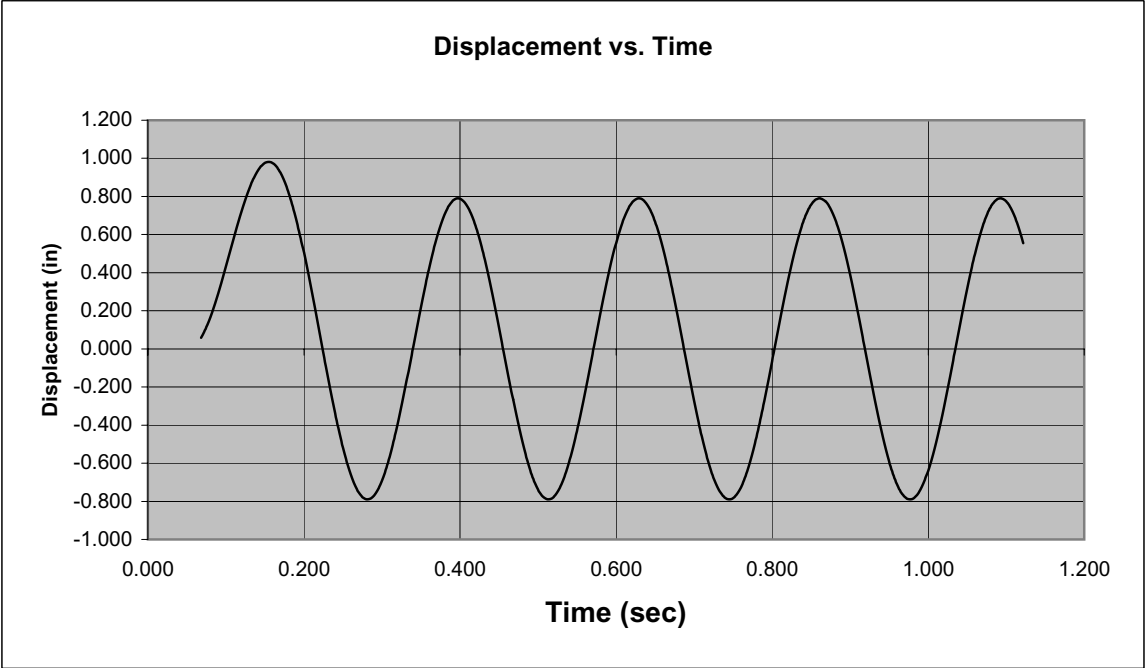
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.524	0.00	-0.757	6.103	556.6606	-7.43	-2.86
0.526	0.00	-0.744	7.207	546.8732	-7.30	-2.81
0.528	0.00	-0.728	8.289	535.4785	-7.14	-2.75
0.530	0.00	-0.711	9.347	522.5100	-6.97	-2.68
0.532	0.00	-0.691	10.378	508.0057	-6.78	-2.61
0.534	0.00	-0.669	11.378	492.0083	-6.56	-2.53
0.536	0.00	-0.646	12.344	474.5649	-6.33	-2.44
0.538	0.00	-0.620	13.275	455.7266	-6.08	-2.34
0.540	0.00	-0.592	14.166	435.5489	-5.81	-2.24
0.542	0.00	-0.563	15.016	414.0911	-5.52	-2.13
0.544	0.00	-0.532	15.821	391.4162	-5.22	-2.01
0.546	0.00	-0.500	16.580	367.5909	-4.90	-1.89
0.548	0.00	-0.466	17.290	342.6851	-4.57	-1.76
0.550	0.00	-0.431	17.950	316.7722	-4.23	-1.63
0.552	0.00	-0.394	18.557	289.9283	-3.87	-1.49
0.554	0.00	-0.357	19.109	262.2322	-3.50	-1.35
0.556	0.00	-0.318	19.605	233.7654	-3.12	-1.20
0.558	0.00	-0.278	20.043	204.6115	-2.73	-1.05
0.560	0.00	-0.238	20.423	174.8562	-2.33	-0.90
0.562	0.00	-0.197	20.742	144.5870	-1.93	-0.74
0.564	0.00	-0.155	21.000	113.8929	-1.52	-0.58
0.566	0.00	-0.113	21.197	82.8640	-1.11	-0.43
0.568	0.00	-0.070	21.332	51.5916	-0.69	-0.26
0.570	0.00	-0.027	21.403	20.1675	-0.27	-0.10
0.572	0.00	0.015	21.412	-11.3159	0.15	0.06
0.574	0.00	0.058	21.358	-42.7660	0.57	0.22
0.576	0.00	0.101	21.241	-74.0904	0.99	0.38
0.578	0.00	0.143	21.062	-105.1970	1.40	0.54
0.580	0.00	0.185	20.821	-135.9945	1.81	0.70
0.582	0.00	0.226	20.518	-166.3922	2.22	0.85
0.584	0.00	0.267	20.156	-196.3009	2.62	1.01
0.586	0.00	0.307	19.734	-225.6326	3.01	1.16
0.588	0.00	0.346	19.254	-254.3012	3.39	1.31
0.590	0.00	0.384	18.717	-282.2224	3.76	1.45
0.592	0.00	0.421	18.126	-309.3140	4.13	1.59
0.594	0.00	0.456	17.481	-335.4966	4.48	1.72
0.596	0.00	0.491	16.785	-360.6931	4.81	1.85
0.598	0.00	0.523	16.039	-384.8294	5.13	1.98
0.600	0.00	0.555	15.247	-407.8347	5.44	2.09
0.602	0.00	0.584	14.409	-429.6414	5.73	2.21
0.604	0.00	0.612	13.529	-450.1852	6.01	2.31
0.606	0.00	0.639	12.610	-469.4059	6.26	2.41
0.608	0.00	0.663	11.653	-487.2470	6.50	2.50
0.610	0.00	0.685	10.662	-503.6559	6.72	2.59
0.612	0.00	0.705	9.640	-518.5846	6.92	2.66
0.614	0.00	0.724	8.589	-531.9891	7.10	2.73
0.616	0.00	0.740	7.514	-543.8299	7.25	2.79
0.618	0.00	0.754	6.416	-554.0724	7.39	2.85
0.620	0.00	0.765	5.299	-562.6864	7.51	2.89
0.622	0.00	0.775	4.167	-569.6466	7.60	2.93
0.624	0.00	0.782	3.022	-574.9326	7.67	2.95
0.626	0.00	0.787	1.869	-578.5287	7.72	2.97
0.628	0.00	0.790	0.710	-580.4245	7.74	2.98
0.630	0.00	0.790	-0.451	-580.6143	7.75	2.98
0.632	0.00	0.788	-1.611	-579.0976	7.73	2.97
0.634	0.00	0.783	-2.766	-575.8789	7.68	2.96
0.636	0.00	0.777	-3.913	-570.9676	7.62	2.93
0.638	0.00	0.768	-5.048	-564.3781	7.53	2.90
0.640	0.00	0.756	-6.169	-556.1299	7.42	2.86
0.642	0.00	0.743	-7.271	-546.2471	7.29	2.81
0.644	0.00	0.727	-8.352	-534.7589	7.13	2.75

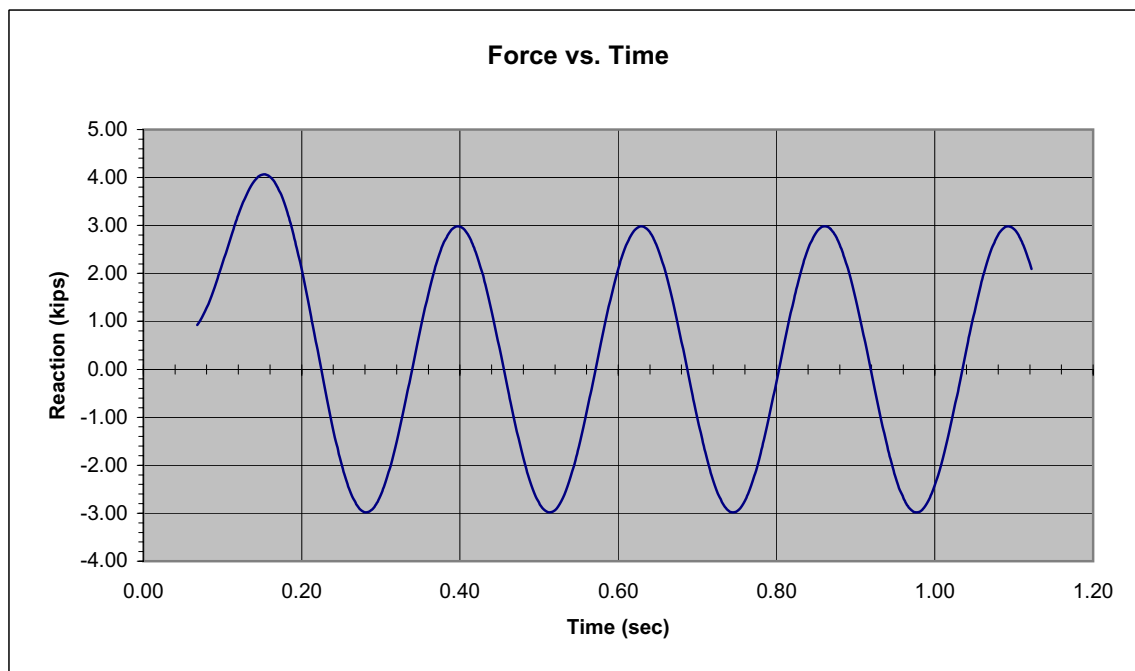
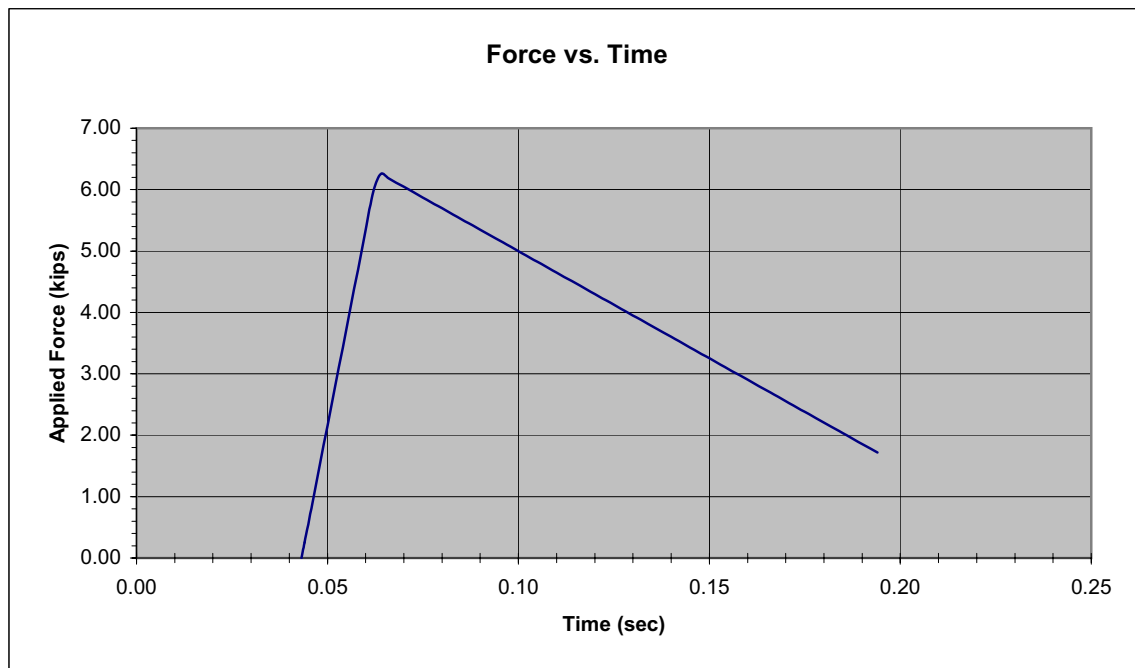
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.646	0.00	0.710	-9.409	-521.6989	6.96	2.68
0.648	0.00	0.690	-10.437	-507.1056	6.76	2.60
0.650	0.00	0.668	-11.436	-491.0218	6.55	2.52
0.652	0.00	0.644	-12.400	-473.4948	6.32	2.43
0.654	0.00	0.618	-13.328	-454.5762	6.06	2.33
0.656	0.00	0.591	-14.217	-434.3215	5.79	2.23
0.658	0.00	0.562	-15.064	-412.7903	5.51	2.12
0.660	0.00	0.531	-15.867	-390.0459	5.20	2.00
0.662	0.00	0.498	-16.623	-366.1551	4.88	1.88
0.664	0.00	0.464	-17.331	-341.1880	4.55	1.75
0.666	0.00	0.429	-17.987	-315.2182	4.20	1.62
0.668	0.00	0.392	-18.591	-288.3219	3.85	1.48
0.670	0.00	0.354	-19.139	-260.5782	3.48	1.34
0.672	0.00	0.316	-19.632	-232.0687	3.10	1.19
0.674	0.00	0.276	-20.067	-202.8770	2.71	1.04
0.676	0.00	0.235	-20.443	-173.0891	2.31	0.89
0.678	0.00	0.194	-20.759	-142.7924	1.90	0.73
0.680	0.00	0.152	-21.014	-112.0761	1.50	0.58
0.682	0.00	0.110	-21.207	-81.0303	1.08	0.42
0.684	0.00	0.068	-21.338	-49.7464	0.66	0.26
0.686	0.00	0.025	-21.406	-18.3163	0.24	0.09
0.688	0.00	-0.018	-21.411	13.1677	-0.18	-0.07
0.690	0.00	-0.061	-21.353	44.6129	-0.60	-0.23
0.692	0.00	-0.103	-21.233	75.9271	-1.01	-0.39
0.694	0.00	-0.146	-21.050	107.0180	-1.43	-0.55
0.696	0.00	-0.187	-20.805	137.7945	-1.84	-0.71
0.698	0.00	-0.229	-20.499	168.1659	-2.24	-0.86
0.700	0.00	-0.269	-20.133	198.0431	-2.64	-1.02
0.702	0.00	-0.309	-19.707	227.3382	-3.03	-1.17
0.704	0.00	-0.348	-19.224	255.9651	-3.41	-1.31
0.706	0.00	-0.386	-18.684	283.8397	-3.79	-1.46
0.708	0.00	-0.423	-18.089	310.8801	-4.15	-1.60
0.710	0.00	-0.458	-17.441	337.0067	-4.50	-1.73
0.712	0.00	-0.493	-16.742	362.1429	-4.83	-1.86
0.714	0.00	-0.525	-15.994	386.2147	-5.15	-1.98
0.716	0.00	-0.557	-15.199	409.1513	-5.46	-2.10
0.718	0.00	-0.586	-14.359	430.8854	-5.75	-2.21
0.720	0.00	-0.614	-13.476	451.3530	-6.02	-2.32
0.722	0.00	-0.640	-12.554	470.4941	-6.28	-2.42
0.724	0.00	-0.664	-11.596	488.2523	-6.51	-2.51
0.726	0.00	-0.686	-10.603	504.5755	-6.73	-2.59
0.728	0.00	-0.707	-9.579	519.4157	-6.93	-2.67
0.730	0.00	-0.725	-8.527	532.7292	-7.11	-2.74
0.732	0.00	-0.741	-7.450	544.4770	-7.26	-2.80
0.734	0.00	-0.754	-6.350	554.6245	-7.40	-2.85
0.736	0.00	-0.766	-5.233	563.1419	-7.51	-2.89
0.738	0.00	-0.775	-4.100	570.0041	-7.60	-2.93
0.740	0.00	-0.782	-2.954	575.1910	-7.67	-2.95
0.742	0.00	-0.787	-1.800	578.6874	-7.72	-2.97
0.744	0.00	-0.790	-0.641	580.4829	-7.74	-2.98
0.746	0.00	-0.790	0.520	580.5723	-7.74	-2.98
0.748	0.00	-0.788	1.679	578.9553	-7.72	-2.97
0.750	0.00	-0.783	2.834	575.6367	-7.68	-2.96
0.752	0.00	-0.776	3.980	570.6263	-7.61	-2.93
0.754	0.00	-0.767	5.115	563.9387	-7.52	-2.90
0.756	0.00	-0.756	6.234	555.5936	-7.41	-2.85
0.758	0.00	-0.742	7.335	545.6155	-7.28	-2.80
0.760	0.00	-0.726	8.415	534.0338	-7.12	-2.74
0.762	0.00	-0.709	9.470	520.8825	-6.95	-2.68
0.764	0.00	-0.689	10.497	506.2003	-6.75	-2.60
0.766	0.00	-0.667	11.493	490.0303	-6.54	-2.52

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.768	0.00	-0.643	12.456	472.4200	-6.30	-2.43
0.770	0.00	-0.617	13.382	453.4212	-6.05	-2.33
0.772	0.00	-0.589	14.268	433.0898	-5.78	-2.22
0.774	0.00	-0.560	15.113	411.4854	-5.49	-2.11
0.776	0.00	-0.529	15.913	388.6716	-5.18	-2.00
0.778	0.00	-0.496	16.666	364.7155	-4.87	-1.87
0.780	0.00	-0.462	17.371	339.6875	-4.53	-1.74
0.782	0.00	-0.427	18.024	313.6610	-4.18	-1.61
0.784	0.00	-0.390	18.624	286.7127	-3.82	-1.47
0.786	0.00	-0.352	19.170	258.9216	-3.45	-1.33
0.788	0.00	-0.313	19.659	230.3696	-3.07	-1.18
0.790	0.00	-0.274	20.091	201.1405	-2.68	-1.03
0.792	0.00	-0.233	20.463	171.3202	-2.29	-0.88
0.794	0.00	-0.192	20.776	140.9964	-1.88	-0.72
0.796	0.00	-0.150	21.027	110.2581	-1.47	-0.57
0.798	0.00	-0.108	21.216	79.1958	-1.06	-0.41
0.800	0.00	-0.065	21.343	47.9008	-0.64	-0.25
0.802	0.00	-0.022	21.408	16.4649	-0.22	-0.08
0.804	0.00	0.020	21.409	-15.0193	0.20	0.08
0.806	0.00	0.063	21.348	-46.4594	0.62	0.24
0.808	0.00	0.106	21.223	-77.7630	1.04	0.40
0.810	0.00	0.148	21.037	-108.8380	1.45	0.56
0.812	0.00	0.190	20.788	-139.5931	1.86	0.72
0.814	0.00	0.231	20.479	-169.9379	2.27	0.87
0.816	0.00	0.272	20.109	-199.7833	2.67	1.03
0.818	0.00	0.312	19.680	-229.0414	3.06	1.18
0.820	0.00	0.350	19.194	-257.6264	3.44	1.32
0.822	0.00	0.388	18.651	-285.4542	3.81	1.47
0.824	0.00	0.425	18.053	-312.4430	4.17	1.60
0.826	0.00	0.460	17.402	-338.5135	4.52	1.74
0.828	0.00	0.495	16.700	-363.5890	4.85	1.87
0.830	0.00	0.527	15.948	-387.5959	5.17	1.99
0.832	0.00	0.558	15.150	-410.4637	5.48	2.11
0.834	0.00	0.588	14.308	-432.1250	5.76	2.22
0.836	0.00	0.616	13.423	-452.5162	6.04	2.32
0.838	0.00	0.641	12.499	-471.5775	6.29	2.42
0.840	0.00	0.666	11.538	-489.2527	6.53	2.51
0.842	0.00	0.688	10.544	-505.4899	6.74	2.60
0.844	0.00	0.708	9.518	-520.2415	6.94	2.67
0.846	0.00	0.726	8.464	-533.4639	7.12	2.74
0.848	0.00	0.742	7.386	-545.1185	7.27	2.80
0.850	0.00	0.755	6.285	-555.1709	7.41	2.85
0.852	0.00	0.767	5.166	-563.5916	7.52	2.89
0.854	0.00	0.776	4.033	-570.3558	7.61	2.93
0.856	0.00	0.783	2.887	-575.4436	7.68	2.96
0.858	0.00	0.787	1.732	-578.8402	7.72	2.97
0.860	0.00	0.790	0.573	-580.5354	7.74	2.98
0.862	0.00	0.790	-0.588	-580.5244	7.74	2.98
0.864	0.00	0.787	-1.747	-578.8072	7.72	2.97
0.866	0.00	0.783	-2.902	-575.3887	7.68	2.96
0.868	0.00	0.776	-4.047	-570.2792	7.61	2.93
0.870	0.00	0.767	-5.181	-563.4935	7.52	2.89
0.872	0.00	0.755	-6.300	-555.0516	7.40	2.85
0.874	0.00	0.741	-7.400	-544.9783	7.27	2.80
0.876	0.00	0.725	-8.478	-533.3033	7.11	2.74
0.878	0.00	0.707	-9.531	-520.0609	6.94	2.67
0.880	0.00	0.687	-10.557	-505.2899	6.74	2.60
0.882	0.00	0.665	-11.551	-489.0338	6.52	2.51
0.884	0.00	0.641	-12.511	-471.3403	6.29	2.42
0.886	0.00	0.615	-13.435	-452.2616	6.03	2.32
0.888	0.00	0.587	-14.319	-431.8536	5.76	2.22

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.890	0.00	0.558	-15.161	-410.1763	5.47	2.11
0.892	0.00	0.527	-15.958	-387.2934	5.17	1.99
0.894	0.00	0.494	-16.709	-363.2723	4.85	1.87
0.896	0.00	0.460	-17.410	-338.1834	4.51	1.74
0.898	0.00	0.425	-18.061	-312.1006	4.16	1.60
0.900	0.00	0.388	-18.658	-285.1005	3.80	1.46
0.902	0.00	0.350	-19.200	-257.2624	3.43	1.32
0.904	0.00	0.311	-19.686	-228.6682	3.05	1.17
0.906	0.00	0.271	-20.114	-199.4019	2.66	1.02
0.908	0.00	0.231	-20.483	-169.5495	2.26	0.87
0.910	0.00	0.189	-20.792	-139.1989	1.86	0.71
0.912	0.00	0.148	-21.040	-108.4390	1.45	0.56
0.914	0.00	0.105	-21.225	-77.3605	1.03	0.40
0.916	0.00	0.063	-21.349	-46.0546	0.61	0.24
0.918	0.00	0.020	-21.410	-14.6134	0.19	0.08
0.920	0.00	-0.023	-21.407	16.8708	-0.23	-0.09
0.922	0.00	-0.066	-21.342	48.3055	-0.64	-0.25
0.924	0.00	-0.108	-21.214	79.5981	-1.06	-0.41
0.926	0.00	-0.151	-21.024	110.6568	-1.48	-0.57
0.928	0.00	-0.192	-20.772	141.3903	-1.89	-0.73
0.930	0.00	-0.234	-20.459	171.7082	-2.29	-0.88
0.932	0.00	-0.274	-20.086	201.5214	-2.69	-1.03
0.934	0.00	-0.314	-19.653	230.7423	-3.08	-1.19
0.936	0.00	-0.353	-19.163	259.2851	-3.46	-1.33
0.938	0.00	-0.390	-18.617	287.0657	-3.83	-1.47
0.940	0.00	-0.427	-18.016	314.0027	-4.19	-1.61
0.942	0.00	-0.463	-17.362	340.0168	-4.54	-1.75
0.944	0.00	-0.497	-16.657	365.0315	-4.87	-1.87
0.946	0.00	-0.529	-15.903	388.9733	-5.19	-2.00
0.948	0.00	-0.560	-15.102	411.7719	-5.49	-2.11
0.950	0.00	-0.589	-14.257	433.3602	-5.78	-2.23
0.952	0.00	-0.617	-13.370	453.6748	-6.05	-2.33
0.954	0.00	-0.643	-12.444	472.6561	-6.31	-2.43
0.956	0.00	-0.667	-11.481	490.2481	-6.54	-2.52
0.958	0.00	-0.689	-10.484	506.3992	-6.76	-2.60
0.960	0.00	-0.709	-9.457	521.0620	-6.95	-2.68
0.962	0.00	-0.727	-8.401	534.1933	-7.13	-2.74
0.964	0.00	-0.742	-7.321	545.7545	-7.28	-2.80
0.966	0.00	-0.756	-6.220	555.7116	-7.41	-2.85
0.968	0.00	-0.767	-5.100	564.0355	-7.52	-2.90
0.970	0.00	-0.776	-3.965	570.7016	-7.61	-2.93
0.972	0.00	-0.783	-2.819	575.6903	-7.68	-2.96
0.974	0.00	-0.788	-1.664	578.9870	-7.72	-2.97
0.976	0.00	-0.790	-0.505	580.5820	-7.74	-2.98
0.978	0.00	-0.790	0.656	580.4706	-7.74	-2.98
0.980	0.00	-0.787	1.815	578.6531	-7.72	-2.97
0.982	0.00	-0.782	2.969	575.1349	-7.67	-2.95
0.984	0.00	-0.775	4.114	569.9262	-7.60	-2.93
0.986	0.00	-0.766	5.247	563.0425	-7.51	-2.89
0.988	0.00	-0.754	6.365	554.5039	-7.40	-2.85
0.990	0.00	-0.740	7.464	544.3356	-7.26	-2.80
0.992	0.00	-0.724	8.541	532.5674	-7.10	-2.74
0.994	0.00	-0.706	9.592	519.2339	-6.93	-2.67
0.996	0.00	-0.686	10.616	504.3743	-6.73	-2.59
0.998	0.00	-0.664	11.608	488.0323	-6.51	-2.51
1.000	0.00	-0.640	12.567	470.2559	-6.27	-2.42
1.002	0.00	-0.614	13.488	451.0974	-6.02	-2.32
1.004	0.00	-0.586	14.370	430.6130	-5.74	-2.21
1.006	0.00	-0.556	15.209	408.8630	-5.45	-2.10
1.008	0.00	-0.525	16.004	385.9113	-5.15	-1.98
1.010	0.00	-0.492	16.752	361.8253	-4.83	-1.86

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
1.012	0.00	-0.458	17.450	336.6759	-4.49	-1.73
1.014	0.00	-0.422	18.097	310.5370	-4.14	-1.59
1.016	0.00	-0.386	18.691	283.4854	-3.78	-1.46
1.018	0.00	-0.348	19.230	255.6005	-3.41	-1.31
1.020	0.00	-0.309	19.713	226.9644	-3.03	-1.17
1.022	0.00	-0.269	20.138	197.6613	-2.64	-1.02
1.024	0.00	-0.228	20.503	167.7772	-2.24	-0.86
1.026	0.00	-0.187	20.808	137.3999	-1.83	-0.71
1.028	0.00	-0.145	21.052	106.6189	-1.42	-0.55
1.030	0.00	-0.103	21.234	75.5244	-1.01	-0.39
1.032	0.00	-0.060	21.354	44.2080	-0.59	-0.23
1.034	0.00	-0.017	21.411	12.7617	-0.17	-0.07
1.036	0.00	0.025	21.405	-18.7222	0.25	0.10
1.038	0.00	0.068	21.336	-50.1510	0.67	0.26
1.040	0.00	0.111	21.205	-81.4324	1.09	0.42
1.042	0.00	0.153	21.011	-112.4745	1.50	0.58
1.044	0.00	0.195	20.755	-143.1860	1.91	0.74
1.046	0.00	0.236	20.439	-173.4767	2.31	0.89
1.048	0.00	0.276	20.062	-203.2575	2.71	1.04
1.050	0.00	0.316	19.626	-232.4409	3.10	1.19
1.052	0.00	0.355	19.133	-260.9411	3.48	1.34
1.054	0.00	0.393	18.583	-288.6744	3.85	1.48
1.056	0.00	0.429	17.979	-315.5592	4.21	1.62
1.058	0.00	0.465	17.322	-341.5166	4.56	1.75
1.060	0.00	0.499	16.614	-366.4702	4.89	1.88
1.062	0.00	0.531	15.857	-390.3467	5.21	2.00
1.064	0.00	0.562	15.054	-413.0759	5.51	2.12
1.066	0.00	0.591	14.206	-434.5910	5.80	2.23
1.068	0.00	0.619	13.316	-454.8288	6.07	2.34
1.070	0.00	0.644	12.388	-473.7299	6.32	2.43
1.072	0.00	0.668	11.423	-491.2385	6.55	2.52
1.074	0.00	0.690	10.424	-507.3034	6.77	2.61
1.076	0.00	0.710	9.395	-521.8772	6.96	2.68
1.078	0.00	0.728	8.338	-534.9171	7.14	2.75
1.080	0.00	0.743	7.257	-546.3849	7.29	2.81
1.082	0.00	0.757	6.154	-556.2468	7.42	2.86
1.084	0.00	0.768	5.034	-564.4737	7.53	2.90
1.086	0.00	0.777	3.898	-571.0417	7.62	2.93
1.088	0.00	0.783	2.751	-575.9312	7.68	2.96
1.090	0.00	0.788	1.596	-579.1280	7.73	2.97
1.092	0.00	0.790	0.436	-580.6227	7.75	2.98
1.094	0.00	0.790	-0.725	-580.4109	7.74	2.98
1.096	0.00	0.787	-1.883	-578.4931	7.72	2.97
1.098	0.00	0.782	-3.037	-574.8751	7.67	2.95
1.100	0.00	0.775	-4.181	-569.5675	7.60	2.93
1.102	0.00	0.765	-5.313	-562.5858	7.50	2.89
1.104	0.00	0.754	-6.430	-553.9506	7.39	2.84
1.106	0.00	0.740	-7.528	-543.6873	7.25	2.79
1.108	0.00	0.723	-8.603	-531.8260	7.09	2.73
1.110	0.00	0.705	-9.653	-518.4017	6.92	2.66
1.112	0.00	0.685	-10.675	-503.4536	6.72	2.59
1.114	0.00	0.662	-11.666	-487.0259	6.50	2.50
1.116	0.00	0.638	-12.622	-469.1667	6.26	2.41
1.118	0.00	0.612	-13.541	-449.9286	6.00	2.31
1.120	0.00	0.584	-14.420	-429.3680	5.73	2.21
1.122	0.00	0.554	-15.257	-407.5455	5.44	2.09





Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

REAR WALL PANELS ; SS # 2

Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject : REAR WALL PROPERTIES SS # 2

Sheet no. : of
Date : 2-Mar-04
Design by : MGG
Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Blast Load (Ps) = 8.210 KN/m²
Dead Load (DL) = 171.47 psf

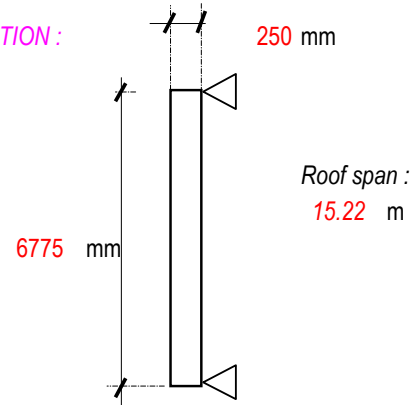
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
Reinf., f_y = 414 Mpa

PANEL DATA :

Panel Width, width = 2500 mm
Long Span, l_b = 6775 mm
Wall, t_{TRIAL} = 250 mm

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure,	P_s	=	1.96	psi	(SEE LOADING CALCULATION)
Blast time duration,	T_d	=	200	msec	
Consider 1ft. width slab:		=	1.0000	ft.	
Moment of Inertia,	I_g	=	953.50	in ⁴	(33072917 mm ⁴)
Area of Bar ($\emptyset$), 25 mm $\emptyset$	A_b	=	0.7609	in ²	(491 mm ²)
Rebar Spacing for top & bottom,	S	=	12.0000	in	(305 mm)
Area of Rebar PER 1ft. unit width,	A_s	=	0.7609	in ²	(13 mm ²)
Effective Slab Depth,	$d_c = d_s$	=	8.3425	in	(212 mm)
$f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for bending)		=	5954.3	psi	(41 Mpa.)
$f'_{dc} = (SIF) \times (DIF) \times f'_m$ (for shear)		=	5504.0	psi	(38 Mpa.)
$f_{dy} = (SIF) \times (DIF) \times f_y$		=	77276	psi	(533 Mpa.)
Modulus of Elasticity,	$E_{dc} = 57000 \times (f'_c)^{1/2}$	=	4032041	psi	(27801 Mpa.)
Steel ratio,	$p = A_s / b \times d$	=	0.0076	>	$200 / f_{dy} = 0.003$
Stress block,	$a = A_s \times f_{dy} / 0.85 \times f'_{dc} \times b$	=	0.9681	in	(25 mm)
Modular ratio,	$n = E_s / E_{dm}$	=	7.1940		

a.) BENDING RESISTANCE

Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$	=	462046.175	lb-in	(52202 KN-m)
Resistance at cracked moment, $R_b = 8 \times M_p / L$	=	13857.98	lb	(61.64 KN)
Unit Resistance, $r_b = R_b / L \times \text{unit width}$	=	4.3296	psi	(0.0299 Mpa.)

b.) SHEAR RESISTANCE

Shear,	$V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$	=	14854.1	lb	(66 KN)
At Support,	$R_s = V_n \times L / (0.5 \times L - t_w)$	=	31691	lb	(141 KN)
Unit Resistance,	$r_s = R_s / L \times \text{unit width}$	=	9.901	psi	(0.0683 Mpa.)

$\therefore$ Dynamic Resistance Controlled by Bending, R_u = 4.3296 psi > 1.96 $\therefore$ O.K!
13.8580 kips 6.29 kips

Cracked Moment of Inertia:

$n \times A_s$	=	5.4736	in ²	(3531 mm ²)
$C = -n \times A_s + [n \times A_s \times (n \times A_s + 2 \times b \times d)]^{0.5} / b$	=	2.3401	in ²	(1510 mm ²)
$I_{cr} = b \times C^3 / 3 + n \times A_s \times (d - C)^2$	=	248.47	in ⁴	(103419712 mm ⁴)

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : REAR WALL PROPERTIES SS # 2

Sheet no. : of
 Date : 2-Mar-04
 Design by : MGG
 Check'd by : SAK

$$\begin{aligned}
 \text{Average Moment of Inertia, } I_a &= (I_g + I_{cr}) / 2 = 600.981 \text{ in}^4 \quad (250147356 \text{ mm}^4) \\
 \text{Effective Stiffness, } K &= 384 \times E_{dc} \times I_a / 5 L^3 = 9.8066 \text{ kips / in} \\
 \text{Yield Deflection, } Y_e &= R_u / K = 1.41312 \text{ in} \\
 \text{Beam Mass} &= \gamma_c \times t_w \times \text{unit width} \times \text{span} / \text{gravity acc.} = 0.00708 \text{ kip-sec}^2 / \text{in} \\
 \text{Roof Mass} &= \text{Roof DL} \times \text{unit width} \times 1/2 \text{ roof span} / \text{gravity acc.} = 0.01109 \text{ kip-sec}^2 / \text{in} \\
 \text{Total Mass, } M &= 0.01818 \text{ kip-sec}^2 / \text{in} \\
 \text{Average Stiffness, } K_{LM} &= 0.72000 \text{ FROM ASCE TABLE 6.1} \\
 \text{Equivalent Mass, } M_e &= K_{LM} \times \text{Beam Mass} = 0.01309 \text{ kip-sec}^2 / \text{in} \\
 \text{Period of Vibration, } t_n &= 2 \times \pi \times (M_e / K)^{0.5} = 0.22953 \text{ sec} \\
 \text{Time increment, } t_n / 10 &= \Delta t = 0.02295 \text{ sec} ; \quad \text{use } \Delta t = 0.00200 \text{ sec} \\
 t_d / t_n &= 0.87135 \\
 R_u / P_s &= 2.20356 \\
 \\
 \text{Arrival time ; } t_a &= 0.02946 \text{ sec} \\
 \text{Rise time ; } t_r &= 0.01979 \text{ sec} \\
 \text{Time at Peak pressure; } t_a + t_r &= 0.04925 \text{ sec} \\
 \text{Time Duration; } T_d &= 0.20000 \text{ sec} \\
 \text{Time end ; } T_e = t_a + T_d &= 0.02946 \text{ sec}
 \end{aligned}$$

NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	9.80664453 kips / in
Mass	M	=	0.013086885 kip-sec ² / in
Tension resistance	Rut	=	13.8579753 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	0.00 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.229 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load
load (kips)
6.29
0

Length	256.000
y _{max} =	0.990
r _{max} =	9.706
μ =	0.700
θ =	0.443
θ =	< 2 degrees
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.02946	0.00	0.000	0.000	0.0000	0.00	0.00
0.04400	2.73	0.000	0.208	208.2204	0.00	0.31
0.046	3.10	0.001	0.653	236.2377	0.01	0.36
0.048	3.48	0.003	1.152	263.5472	0.03	0.41
0.050	3.85	0.006	1.706	290.0671	0.06	0.46
0.052	4.23	0.010	2.312	315.7180	0.09	0.52
0.054	4.60	0.015	2.968	340.4231	0.15	0.59
0.056	4.98	0.022	3.673	364.1083	0.21	0.65
0.058	5.35	0.030	4.423	386.7026	0.29	0.73
0.060	5.73	0.039	5.218	408.1385	0.39	0.81
0.062	6.10	0.051	6.055	428.3516	0.50	0.89
0.064	6.25	0.064	6.913	430.0755	0.62	0.96
0.066	6.18	0.078	7.756	413.3050	0.77	1.01
0.068	6.10	0.095	8.565	395.2963	0.93	1.06
0.070	6.02	0.112	9.336	376.1034	1.10	1.12
0.072	5.95	0.132	10.068	355.7837	1.29	1.18
0.074	5.87	0.153	10.759	334.3981	1.50	1.25
0.076	5.80	0.175	11.405	312.0106	1.71	1.33
0.078	5.72	0.198	12.006	288.6885	1.94	1.41
0.080	5.65	0.223	12.559	264.5014	2.19	1.49
0.082	5.57	0.248	13.063	239.5219	2.44	1.58
0.084	5.50	0.275	13.516	213.8248	2.70	1.67
0.086	5.42	0.302	13.918	187.4872	2.97	1.77
0.088	5.34	0.331	14.266	160.5878	3.24	1.86
0.090	5.27	0.360	14.559	133.2074	3.53	1.96
0.092	5.19	0.389	14.798	105.4278	3.81	2.07
0.094	5.12	0.419	14.981	77.3324	4.11	2.17
0.096	5.04	0.449	15.107	49.0054	4.40	2.27
0.098	4.97	0.479	15.177	20.5315	4.70	2.38
0.100	4.89	0.509	15.189	-8.0039	5.00	2.49
0.102	4.82	0.540	15.145	-36.5153	5.29	2.59
0.104	4.74	0.570	15.043	-64.9174	5.59	2.70

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.106	4.66	0.600	14.885	-93.1249	5.88	2.80
0.108	4.59	0.629	14.671	-121.0534	6.17	2.90
0.110	4.51	0.659	14.401	-148.6193	6.46	3.01
0.112	4.44	0.687	14.077	-175.7399	6.74	3.10
0.114	4.36	0.715	13.699	-202.3341	7.01	3.20
0.116	4.29	0.742	13.268	-228.3220	7.27	3.29
0.118	4.21	0.768	12.786	-253.6259	7.53	3.38
0.120	4.14	0.793	12.255	-278.1700	7.78	3.47
0.122	4.06	0.817	11.674	-301.8808	8.01	3.55
0.124	3.98	0.840	11.048	-324.6871	8.23	3.63
0.126	3.91	0.861	10.377	-346.5207	8.44	3.70
0.128	3.83	0.881	9.663	-367.3161	8.64	3.77
0.130	3.76	0.900	8.909	-387.0111	8.82	3.83
0.132	3.68	0.917	8.116	-405.5467	8.99	3.88
0.134	3.61	0.932	7.288	-422.8672	9.14	3.93
0.136	3.53	0.946	6.426	-438.9210	9.28	3.98
0.138	3.46	0.958	5.533	-453.6597	9.39	4.01
0.140	3.38	0.968	4.612	-467.0393	9.49	4.04
0.142	3.30	0.976	3.666	-479.0198	9.57	4.07
0.144	3.23	0.983	2.698	-489.5651	9.64	4.08
0.146	3.15	0.987	1.710	-498.6437	9.68	4.09
0.148	3.08	0.989	0.705	-506.2285	9.70	4.09
0.150	3.00	0.990	-0.314	-512.2966	9.71	4.08
0.152	2.93	0.988	-1.343	-516.8300	9.69	4.07
0.154	2.85	0.984	-2.380	-519.8150	9.65	4.04
0.156	2.78	0.979	-3.421	-521.2427	9.60	4.01
0.158	2.70	0.971	-4.463	-521.1087	9.52	3.98
0.160	2.62	0.961	-5.503	-519.4136	9.42	3.93
0.162	2.55	0.949	-6.539	-516.1624	9.30	3.88
0.164	2.47	0.935	-7.567	-511.3648	9.17	3.81
0.166	2.40	0.918	-8.583	-505.0353	9.01	3.74
0.168	2.32	0.900	-9.585	-497.1926	8.83	3.67
0.170	2.25	0.880	-10.570	-487.8605	8.63	3.58
0.172	2.17	0.858	-11.535	-477.0667	8.41	3.49
0.174	2.10	0.834	-12.477	-464.8438	8.18	3.39
0.176	2.02	0.808	-13.393	-451.2281	7.93	3.28
0.178	1.94	0.780	-14.281	-436.2607	7.65	3.17
0.180	1.87	0.751	-15.137	-419.9863	7.37	3.05
0.182	1.79	0.720	-15.959	-402.4536	7.06	2.92
0.184	1.72	0.687	-16.745	-383.7152	6.74	2.79
0.186	1.64	0.653	-17.493	-363.8273	6.40	2.65
0.188	1.57	0.617	-18.200	-342.8493	6.05	2.51
0.190	1.49	0.580	-18.863	-320.8442	5.69	2.36
0.192	1.42	0.542	-19.482	-297.8779	5.31	2.21
0.194	1.34	0.502	-20.054	-274.0192	4.93	2.05
0.196	1.26	0.462	-20.577	-249.3396	4.53	1.89

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.198	1.19	0.420	-21.051	-223.9129	4.12	1.72
0.200	1.11	0.377	-21.472	-197.8154	3.70	1.55
0.202	1.04	0.334	-21.841	-171.1253	3.28	1.38
0.204	0.96	0.290	-22.156	-143.9226	2.85	1.21
0.206	0.89	0.246	-22.417	-116.2886	2.41	1.03
0.208	0.81	0.201	-22.621	-88.3063	1.97	0.85
0.210	0.74	0.155	-22.770	-60.0594	1.52	0.67
0.212	0.66	0.109	-22.861	-31.6325	1.07	0.49
0.214	0.58	0.064	-22.896	-3.1109	0.62	0.31
0.216	0.51	0.018	-22.874	25.4200	0.18	0.13
0.218	0.43	-0.028	-22.794	53.8747	-0.27	-0.05
0.220	0.36	-0.073	-22.658	82.1681	-0.72	-0.24
0.222	0.28	-0.118	-22.466	110.2153	-1.16	-0.41
0.224	0.21	-0.163	-22.218	137.9323	-1.60	-0.59
0.226	0.13	-0.207	-21.915	165.2361	-2.03	-0.77
0.228	0.06	-0.251	-21.557	192.0448	-2.46	-0.94
0.230	0.00	-0.293	-21.145	219.8250	-2.88	-1.11
0.232	0.00	-0.335	-20.674	251.1706	-3.29	-1.27
0.234	0.00	-0.376	-20.142	281.7636	-3.69	-1.42
0.236	0.00	-0.416	-19.548	311.5126	-4.08	-1.57
0.238	0.00	-0.454	-18.896	340.3283	-4.45	-1.71
0.240	0.00	-0.491	-18.188	368.1244	-4.82	-1.85
0.242	0.00	-0.527	-17.425	394.8176	-5.17	-1.99
0.244	0.00	-0.561	-16.610	420.3280	-5.50	-2.12
0.246	0.00	-0.593	-15.745	444.5791	-5.82	-2.24
0.248	0.00	-0.624	-14.833	467.4983	-6.12	-2.36
0.250	0.00	-0.653	-13.876	489.0170	-6.40	-2.46
0.252	0.00	-0.679	-12.878	509.0706	-6.66	-2.56
0.254	0.00	-0.704	-11.842	527.5991	-6.90	-2.66
0.256	0.00	-0.727	-10.769	544.5469	-7.13	-2.74
0.258	0.00	-0.747	-9.665	559.8633	-7.33	-2.82
0.260	0.00	-0.765	-8.532	573.5025	-7.51	-2.89
0.262	0.00	-0.781	-7.373	585.4235	-7.66	-2.95
0.264	0.00	-0.795	-6.192	595.5906	-7.79	-3.00
0.266	0.00	-0.806	-4.992	603.9734	-7.90	-3.04
0.268	0.00	-0.815	-3.778	610.5467	-7.99	-3.08
0.270	0.00	-0.821	-2.552	615.2909	-8.05	-3.10
0.272	0.00	-0.825	-1.318	618.1918	-8.09	-3.11
0.274	0.00	-0.826	-0.081	619.2406	-8.10	-3.12
0.276	0.00	-0.825	1.157	618.4342	-8.09	-3.12
0.278	0.00	-0.822	2.391	615.7751	-8.06	-3.10
0.280	0.00	-0.816	3.618	611.2711	-8.00	-3.08
0.282	0.00	-0.807	4.834	604.9359	-7.92	-3.05
0.284	0.00	-0.796	6.036	596.7883	-7.81	-3.01
0.286	0.00	-0.783	7.220	586.8528	-7.68	-2.96
0.288	0.00	-0.768	8.382	575.1592	-7.53	-2.90
0.290	0.00	-0.750	9.518	561.7424	-7.35	-2.83
0.292	0.00	-0.729	10.627	546.6428	-7.15	-2.75
0.294	0.00	-0.707	11.703	529.9054	-6.93	-2.67
0.296	0.00	-0.683	12.745	511.5805	-6.69	-2.58
0.298	0.00	-0.656	13.748	491.7229	-6.44	-2.48
0.300	0.00	-0.628	14.710	470.3922	-6.16	-2.37

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.302	0.00	-0.597	15.628	447.6523	-5.86	-2.26
0.304	0.00	-0.565	16.500	423.5712	-5.54	-2.13
0.306	0.00	-0.531	17.321	398.2211	-5.21	-2.01
0.308	0.00	-0.496	18.091	371.6780	-4.86	-1.87
0.310	0.00	-0.459	18.807	344.0215	-4.50	-1.73
0.312	0.00	-0.421	19.466	315.3342	-4.13	-1.59
0.314	0.00	-0.381	20.067	285.7023	-3.74	-1.44
0.316	0.00	-0.341	20.608	255.2144	-3.34	-1.29
0.318	0.00	-0.299	21.087	223.9619	-2.93	-1.13
0.320	0.00	-0.256	21.503	192.0384	-2.51	-0.97
0.322	0.00	-0.213	21.855	159.5396	-2.09	-0.80
0.324	0.00	-0.169	22.141	126.5629	-1.66	-0.64
0.326	0.00	-0.124	22.361	93.2070	-1.22	-0.47
0.328	0.00	-0.079	22.514	59.5718	-0.78	-0.30
0.330	0.00	-0.034	22.599	25.7582	-0.34	-0.13
0.332	0.00	0.011	22.617	-8.1326	0.11	0.04
0.334	0.00	0.056	22.567	-41.9990	0.55	0.21
0.336	0.00	0.101	22.449	-75.7397	0.99	0.38
0.338	0.00	0.146	22.264	-109.2534	1.43	0.55
0.340	0.00	0.190	22.012	-142.4397	1.86	0.72
0.342	0.00	0.234	21.694	-175.1994	2.29	0.88
0.344	0.00	0.277	21.312	-207.4342	2.71	1.05
0.346	0.00	0.319	20.865	-239.0475	3.13	1.20
0.348	0.00	0.360	20.356	-269.9446	3.53	1.36
0.350	0.00	0.400	19.786	-300.0331	3.93	1.51
0.352	0.00	0.439	19.157	-329.2226	4.31	1.66
0.354	0.00	0.477	18.470	-357.4259	4.68	1.80
0.356	0.00	0.513	17.728	-384.5583	5.03	1.94
0.358	0.00	0.548	16.933	-410.5386	5.37	2.07
0.360	0.00	0.581	16.088	-435.2890	5.70	2.19
0.362	0.00	0.612	15.194	-458.7354	6.00	2.31
0.364	0.00	0.642	14.254	-480.8074	6.29	2.42
0.366	0.00	0.669	13.272	-501.4389	6.56	2.53
0.368	0.00	0.695	12.250	-520.5682	6.81	2.62
0.370	0.00	0.718	11.191	-538.1379	7.04	2.71
0.372	0.00	0.739	10.099	-554.0954	7.25	2.79
0.374	0.00	0.759	8.976	-568.3929	7.44	2.86
0.376	0.00	0.775	7.827	-580.9875	7.60	2.93
0.378	0.00	0.790	6.654	-591.8416	7.75	2.98
0.380	0.00	0.802	5.461	-600.9226	7.86	3.03
0.382	0.00	0.812	4.252	-608.2032	7.96	3.06
0.384	0.00	0.819	3.030	-613.6618	8.03	3.09
0.386	0.00	0.824	1.799	-617.2819	8.08	3.11
0.388	0.00	0.826	0.563	-619.0526	8.10	3.12
0.390	0.00	0.826	-0.675	-618.9688	8.10	3.12
0.392	0.00	0.823	-1.911	-617.0305	8.08	3.11
0.394	0.00	0.818	-3.141	-613.2438	8.03	3.09
0.396	0.00	0.811	-4.362	-607.6198	7.95	3.06
0.398	0.00	0.801	-5.570	-600.1754	7.85	3.02
0.400	0.00	0.789	-6.761	-590.9330	7.73	2.98
0.402	0.00	0.774	-7.932	-579.9201	7.59	2.92
0.404	0.00	0.757	-9.079	-567.1699	7.42	2.86

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.406	0.00	0.738	-10.199	-552.7206	7.23	2.78
0.408	0.00	0.716	-11.288	-536.6153	7.02	2.70
0.410	0.00	0.692	-12.344	-518.9024	6.79	2.61
0.412	0.00	0.667	-13.362	-499.6349	6.54	2.52
0.414	0.00	0.639	-14.341	-478.8705	6.27	2.41
0.416	0.00	0.609	-15.276	-456.6715	5.98	2.30
0.418	0.00	0.578	-16.166	-433.1044	5.67	2.18
0.420	0.00	0.545	-17.007	-408.2397	5.34	2.06
0.422	0.00	0.510	-17.798	-382.1520	5.00	1.93
0.424	0.00	0.474	-18.535	-354.9194	4.64	1.79
0.426	0.00	0.436	-19.216	-326.6235	4.27	1.65
0.428	0.00	0.397	-19.840	-297.3490	3.89	1.50
0.430	0.00	0.357	-20.405	-267.1837	3.50	1.35
0.432	0.00	0.315	-20.908	-236.2180	3.09	1.19
0.434	0.00	0.273	-21.349	-204.5446	2.68	1.03
0.436	0.00	0.230	-21.726	-172.2584	2.25	0.87
0.438	0.00	0.186	-22.038	-139.4561	1.83	0.70
0.440	0.00	0.142	-22.283	-106.2360	1.39	0.54
0.442	0.00	0.097	-22.462	-72.6977	0.95	0.37
0.444	0.00	0.052	-22.574	-38.9415	0.51	0.20
0.446	0.00	0.007	-22.618	-5.0687	0.07	0.03
0.448	0.00	-0.038	-22.594	28.8193	-0.38	-0.15
0.450	0.00	-0.084	-22.503	62.6209	-0.82	-0.32
0.452	0.00	-0.128	-22.344	96.2350	-1.26	-0.48
0.454	0.00	-0.173	-22.118	129.5607	-1.70	-0.65
0.456	0.00	-0.217	-21.826	162.4983	-2.13	-0.82
0.458	0.00	-0.260	-21.469	194.9491	-2.55	-0.98
0.460	0.00	-0.303	-21.047	226.8158	-2.97	-1.14
0.462	0.00	-0.344	-20.562	258.0030	-3.38	-1.30
0.464	0.00	-0.385	-20.015	288.4172	-3.77	-1.45
0.466	0.00	-0.424	-19.409	317.9674	-4.16	-1.60
0.468	0.00	-0.462	-18.745	346.5650	-4.54	-1.75
0.470	0.00	-0.499	-18.024	374.1243	-4.90	-1.89
0.472	0.00	-0.535	-17.249	400.5627	-5.24	-2.02
0.474	0.00	-0.568	-16.423	425.8011	-5.57	-2.15
0.476	0.00	-0.600	-15.547	449.7639	-5.89	-2.27
0.478	0.00	-0.630	-14.625	472.3792	-6.18	-2.38
0.480	0.00	-0.659	-13.659	493.5793	-6.46	-2.49
0.482	0.00	-0.685	-12.652	513.3007	-6.72	-2.59
0.484	0.00	-0.709	-11.608	531.4843	-6.96	-2.68
0.486	0.00	-0.731	-10.528	548.0756	-7.17	-2.76
0.488	0.00	-0.751	-9.417	563.0250	-7.37	-2.84
0.490	0.00	-0.769	-8.278	576.2876	-7.54	-2.90
0.492	0.00	-0.784	-7.113	587.8237	-7.69	-2.96
0.494	0.00	-0.797	-5.928	597.5987	-7.82	-3.01
0.496	0.00	-0.808	-4.725	605.5834	-7.93	-3.05
0.498	0.00	-0.816	-3.507	611.7538	-8.01	-3.08
0.500	0.00	-0.822	-2.280	616.0914	-8.06	-3.10
0.502	0.00	-0.825	-1.045	618.5833	-8.10	-3.12
0.504	0.00	-0.826	0.193	619.2220	-8.10	-3.12
0.506	0.00	-0.825	1.430	618.0056	-8.09	-3.11
0.508	0.00	-0.821	2.663	614.9377	-8.05	-3.10
0.510	0.00	-0.814	3.888	610.0275	-7.98	-3.07
0.512	0.00	-0.805	5.101	603.2897	-7.90	-3.04
0.514	0.00	-0.794	6.299	594.7445	-7.78	-3.00
0.516	0.00	-0.780	7.478	584.4175	-7.65	-2.94
0.518	0.00	-0.764	8.635	572.3397	-7.49	-2.88
0.520	0.00	-0.745	9.766	558.5472	-7.31	-2.81
0.522	0.00	-0.725	10.868	543.0813	-7.11	-2.74

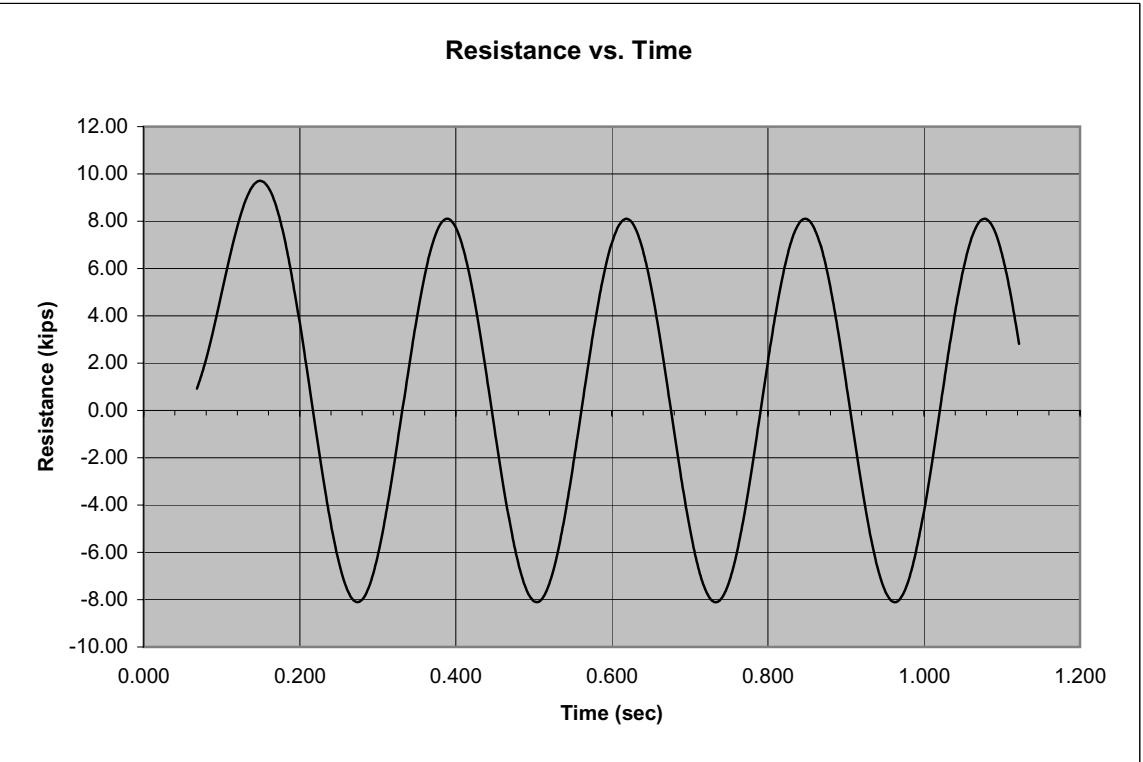
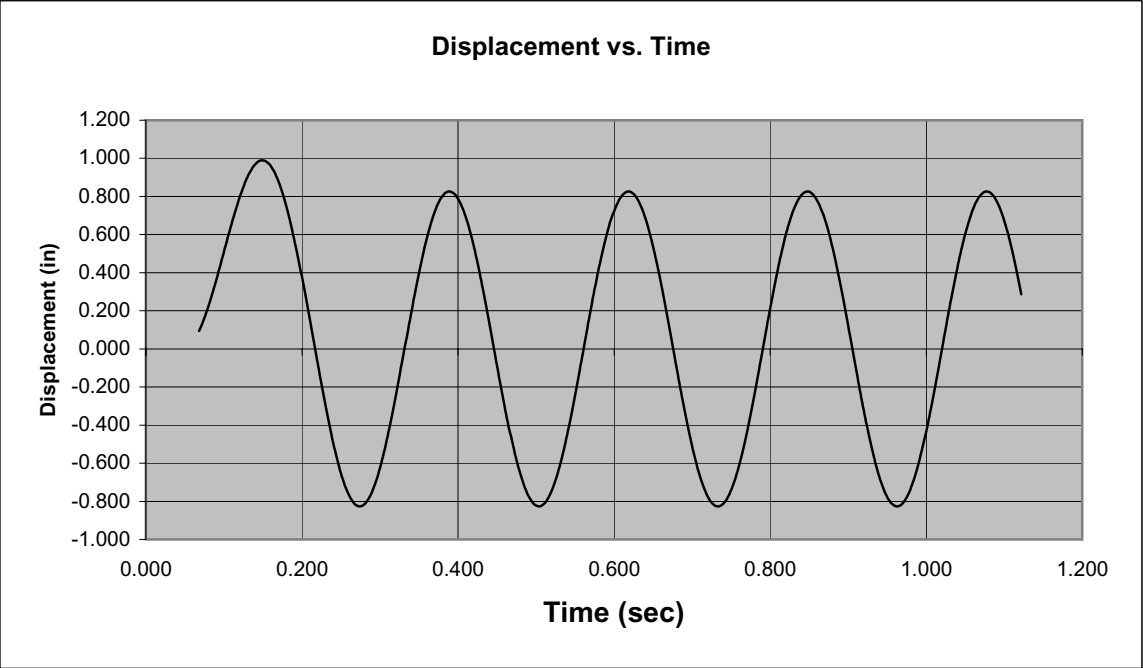
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.524	0.00	-0.702	11.937	525.9885	-6.88	-2.65
0.526	0.00	-0.677	12.970	507.3198	-6.64	-2.56
0.528	0.00	-0.650	13.965	487.1312	-6.38	-2.45
0.530	0.00	-0.621	14.917	465.4833	-6.09	-2.35
0.532	0.00	-0.590	15.825	442.4408	-5.79	-2.23
0.534	0.00	-0.558	16.686	418.0728	-5.47	-2.11
0.536	0.00	-0.524	17.496	392.4523	-5.14	-1.98
0.538	0.00	-0.488	18.254	365.6560	-4.79	-1.84
0.540	0.00	-0.451	18.958	337.7643	-4.42	-1.70
0.542	0.00	-0.412	19.604	308.8606	-4.04	-1.56
0.544	0.00	-0.372	20.192	279.0317	-3.65	-1.41
0.546	0.00	-0.331	20.720	248.3668	-3.25	-1.25
0.548	0.00	-0.290	21.185	216.9578	-2.84	-1.09
0.550	0.00	-0.247	21.587	184.8988	-2.42	-0.93
0.552	0.00	-0.203	21.924	152.2859	-1.99	-0.77
0.554	0.00	-0.159	22.195	119.2168	-1.56	-0.60
0.556	0.00	-0.114	22.400	85.7905	-1.12	-0.43
0.558	0.00	-0.070	22.538	52.1072	-0.68	-0.26
0.560	0.00	-0.024	22.609	18.2678	-0.24	-0.09
0.562	0.00	0.021	22.611	-15.6264	0.20	0.08
0.564	0.00	0.066	22.546	-49.4737	0.65	0.25
0.566	0.00	0.111	22.414	-83.1729	1.09	0.42
0.568	0.00	0.156	22.214	-116.6228	1.53	0.59
0.570	0.00	0.200	21.948	-149.7234	1.96	0.75
0.572	0.00	0.243	21.615	-182.3753	2.39	0.92
0.574	0.00	0.286	21.219	-214.4810	2.81	1.08
0.576	0.00	0.328	20.758	-245.9440	3.22	1.24
0.578	0.00	0.369	20.236	-276.6703	3.62	1.39
0.580	0.00	0.409	19.652	-306.5676	4.01	1.54
0.582	0.00	0.448	19.010	-335.5465	4.39	1.69
0.584	0.00	0.485	18.311	-363.5202	4.76	1.83
0.586	0.00	0.521	17.557	-390.4047	5.11	1.97
0.588	0.00	0.555	16.751	-416.1197	5.45	2.10
0.590	0.00	0.588	15.894	-440.5880	5.77	2.22
0.592	0.00	0.619	14.990	-463.7364	6.07	2.34
0.594	0.00	0.648	14.040	-485.4954	6.35	2.45
0.596	0.00	0.675	13.049	-505.8000	6.62	2.55
0.598	0.00	0.700	12.019	-524.5892	6.87	2.64
0.600	0.00	0.723	10.952	-541.8068	7.09	2.73
0.602	0.00	0.744	9.853	-557.4012	7.29	2.81
0.604	0.00	0.762	8.724	-571.3257	7.48	2.88
0.606	0.00	0.779	7.570	-583.5386	7.64	2.94
0.608	0.00	0.793	6.392	-594.0032	7.77	2.99
0.610	0.00	0.804	5.195	-602.6883	7.89	3.04
0.612	0.00	0.813	3.983	-609.5677	7.98	3.07
0.614	0.00	0.820	2.759	-614.6210	8.04	3.10
0.616	0.00	0.824	1.526	-617.8329	8.09	3.11
0.618	0.00	0.826	0.289	-619.1939	8.10	3.12
0.620	0.00	0.826	-0.949	-618.6998	8.10	3.12
0.622	0.00	0.823	-2.184	-616.3521	8.07	3.11
0.624	0.00	0.817	-3.412	-612.1579	8.01	3.08
0.626	0.00	0.809	-4.630	-606.1298	7.93	3.05
0.628	0.00	0.798	-5.835	-598.2858	7.83	3.01
0.630	0.00	0.786	-7.022	-588.6493	7.70	2.97
0.632	0.00	0.770	-8.188	-577.2493	7.55	2.91
0.634	0.00	0.753	-9.329	-564.1200	7.38	2.84
0.636	0.00	0.733	-10.442	-549.3005	7.19	2.77
0.638	0.00	0.711	-11.525	-532.8355	6.97	2.68
0.640	0.00	0.687	-12.572	-514.7741	6.74	2.59
0.642	0.00	0.661	-13.582	-495.1705	6.48	2.49
0.644	0.00	0.633	-14.551	-474.0834	6.20	2.39

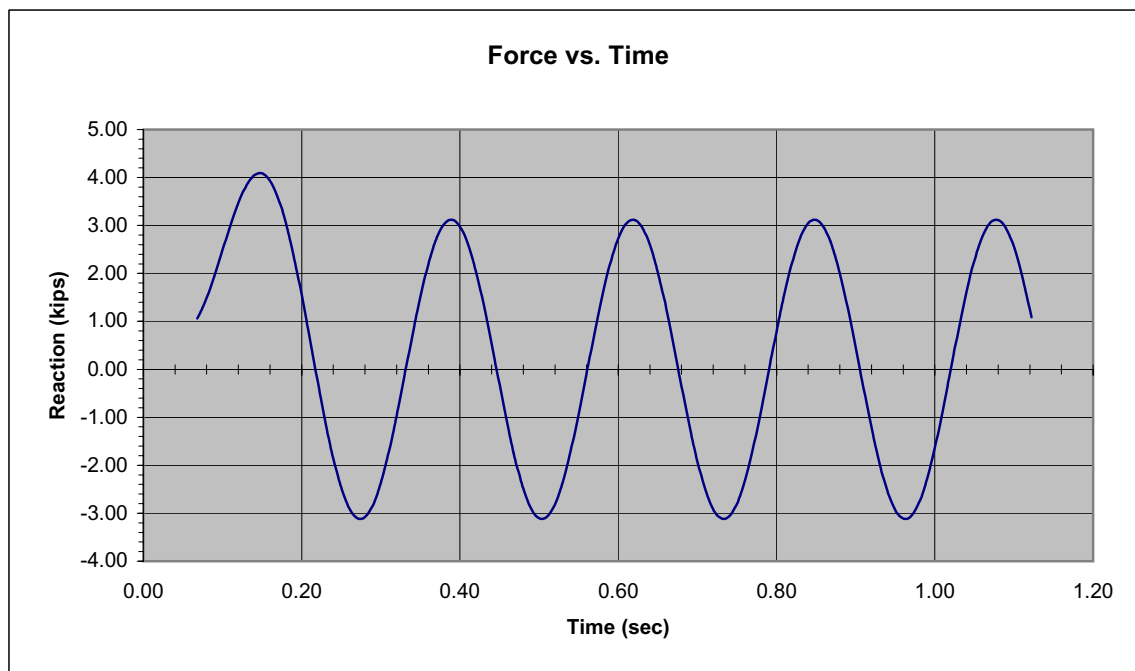
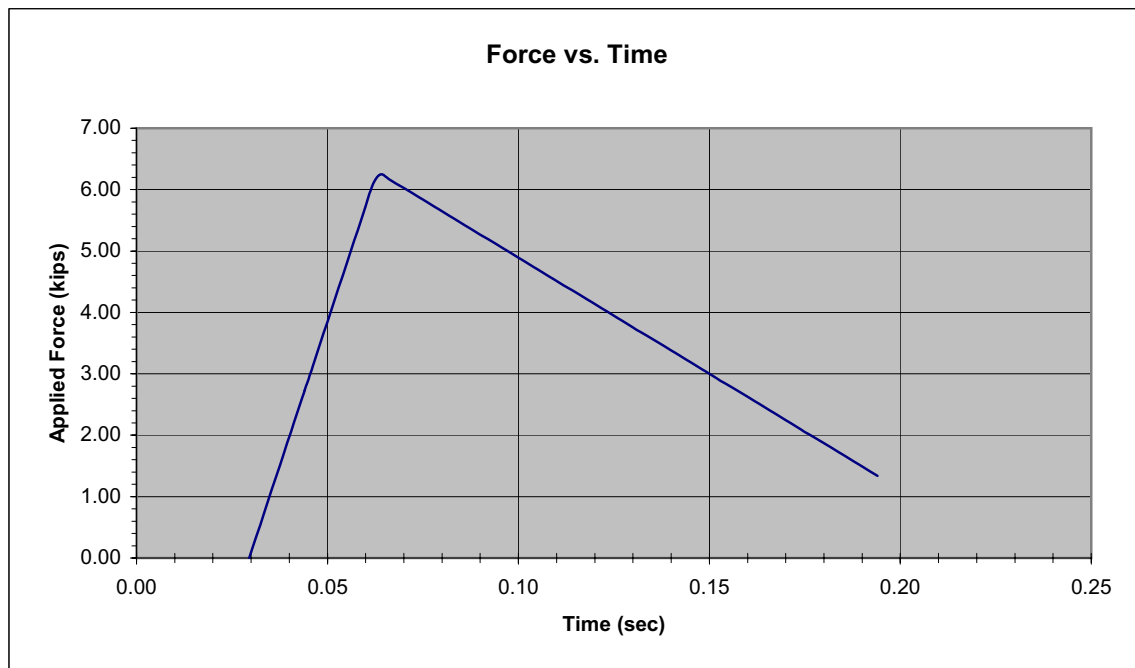
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.646	0.00	0.603	-15.477	-451.5761	5.91	2.28
0.648	0.00	0.571	-16.356	-427.7158	5.60	2.16
0.650	0.00	0.537	-17.187	-402.5742	5.27	2.03
0.652	0.00	0.502	-17.965	-376.2264	4.92	1.90
0.654	0.00	0.465	-18.690	-348.7516	4.56	1.76
0.656	0.00	0.427	-19.359	-320.2319	4.19	1.61
0.658	0.00	0.388	-19.970	-290.7528	3.81	1.46
0.660	0.00	0.348	-20.522	-260.4027	3.41	1.31
0.662	0.00	0.306	-21.011	-229.2724	3.00	1.16
0.664	0.00	0.264	-21.438	-197.4553	2.58	0.99
0.666	0.00	0.220	-21.800	-165.0466	2.16	0.83
0.668	0.00	0.176	-22.098	-132.1434	1.73	0.67
0.670	0.00	0.132	-22.329	-98.8443	1.29	0.50
0.672	0.00	0.087	-22.493	-65.2492	0.85	0.33
0.674	0.00	0.042	-22.589	-31.4585	0.41	0.16
0.676	0.00	-0.003	-22.618	2.4264	-0.03	-0.01
0.678	0.00	-0.048	-22.580	36.3041	-0.48	-0.18
0.680	0.00	-0.094	-22.473	70.0729	-0.92	-0.35
0.682	0.00	-0.138	-22.300	103.6319	-1.36	-0.52
0.684	0.00	-0.183	-22.059	136.8804	-1.79	-0.69
0.686	0.00	-0.226	-21.753	169.7188	-2.22	-0.86
0.688	0.00	-0.270	-21.381	202.0487	-2.64	-1.02
0.690	0.00	-0.312	-20.945	233.7733	-3.06	-1.18
0.692	0.00	-0.353	-20.446	264.7976	-3.47	-1.33
0.694	0.00	-0.394	-19.887	295.0285	-3.86	-1.49
0.696	0.00	-0.433	-19.267	324.3756	-4.25	-1.63
0.698	0.00	-0.471	-18.590	352.7509	-4.62	-1.78
0.700	0.00	-0.507	-17.857	380.0693	-4.97	-1.91
0.702	0.00	-0.542	-17.071	406.2492	-5.32	-2.05
0.704	0.00	-0.575	-16.233	431.2119	-5.64	-2.17
0.706	0.00	-0.607	-15.347	454.8828	-5.95	-2.29
0.708	0.00	-0.637	-14.415	477.1909	-6.24	-2.40
0.710	0.00	-0.665	-13.440	498.0693	-6.52	-2.51
0.712	0.00	-0.691	-12.424	517.4556	-6.77	-2.61
0.714	0.00	-0.714	-11.372	535.2917	-7.01	-2.70
0.716	0.00	-0.736	-10.285	551.5241	-7.22	-2.78
0.718	0.00	-0.755	-9.167	566.1042	-7.41	-2.85
0.720	0.00	-0.773	-8.022	578.9882	-7.58	-2.92
0.722	0.00	-0.788	-6.853	590.1377	-7.72	-2.97
0.724	0.00	-0.800	-5.663	599.5192	-7.85	-3.02
0.726	0.00	-0.810	-4.457	607.1046	-7.95	-3.06
0.728	0.00	-0.818	-3.237	612.8712	-8.02	-3.09
0.730	0.00	-0.823	-2.007	616.8017	-8.07	-3.11
0.732	0.00	-0.826	-0.771	618.8843	-8.10	-3.12
0.734	0.00	-0.826	0.467	619.1128	-8.10	-3.12
0.736	0.00	-0.824	1.703	617.4864	-8.08	-3.11
0.738	0.00	-0.819	2.935	614.0102	-8.04	-3.09
0.740	0.00	-0.812	4.157	608.6944	-7.97	-3.07
0.742	0.00	-0.803	5.368	601.5551	-7.87	-3.03
0.744	0.00	-0.791	6.562	592.6136	-7.76	-2.99
0.746	0.00	-0.777	7.736	581.8966	-7.62	-2.93
0.748	0.00	-0.760	8.888	569.4363	-7.45	-2.87
0.750	0.00	-0.741	10.012	555.2701	-7.27	-2.80
0.752	0.00	-0.720	11.107	539.4404	-7.06	-2.72
0.754	0.00	-0.697	12.168	521.9945	-6.83	-2.63
0.756	0.00	-0.671	13.193	502.9848	-6.58	-2.53
0.758	0.00	-0.644	14.179	482.4682	-6.31	-2.43
0.760	0.00	-0.615	15.122	460.5061	-6.03	-2.32
0.762	0.00	-0.583	16.020	437.1645	-5.72	-2.20
0.764	0.00	-0.550	16.869	412.5131	-5.40	-2.08
0.766	0.00	-0.516	17.668	386.6259	-5.06	-1.95

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.768	0.00	-0.480	18.415	359.5804	-4.71	-1.81
0.770	0.00	-0.442	19.106	331.4576	-4.34	-1.67
0.772	0.00	-0.403	19.739	302.3418	-3.96	-1.52
0.774	0.00	-0.363	20.314	272.3202	-3.56	-1.37
0.776	0.00	-0.322	20.828	241.4828	-3.16	-1.22
0.778	0.00	-0.280	21.279	209.9220	-2.75	-1.06
0.780	0.00	-0.237	21.667	177.7322	-2.33	-0.90
0.782	0.00	-0.194	21.990	145.0099	-1.90	-0.73
0.784	0.00	-0.149	22.247	111.8533	-1.46	-0.56
0.786	0.00	-0.105	22.437	78.3615	-1.03	-0.39
0.788	0.00	-0.060	22.560	44.6349	-0.58	-0.22
0.790	0.00	-0.014	22.615	10.7747	-0.14	-0.05
0.792	0.00	0.031	22.603	-23.1179	0.30	0.12
0.794	0.00	0.076	22.523	-56.9412	0.75	0.29
0.796	0.00	0.121	22.375	-90.5939	1.19	0.46
0.798	0.00	0.165	22.161	-123.9752	1.62	0.62
0.800	0.00	0.209	21.880	-156.9850	2.05	0.79
0.802	0.00	0.253	21.533	-189.5246	2.48	0.95
0.804	0.00	0.296	21.122	-221.4964	2.90	1.12
0.806	0.00	0.337	20.648	-252.8045	3.31	1.27
0.808	0.00	0.378	20.112	-283.3553	3.71	1.43
0.810	0.00	0.418	19.515	-313.0572	4.10	1.58
0.812	0.00	0.456	18.860	-341.8213	4.47	1.72
0.814	0.00	0.493	18.149	-369.5612	4.84	1.86
0.816	0.00	0.529	17.383	-396.1940	5.18	2.00
0.818	0.00	0.563	16.565	-421.6398	5.52	2.12
0.820	0.00	0.595	15.698	-445.8224	5.83	2.25
0.822	0.00	0.625	14.784	-468.6694	6.13	2.36
0.824	0.00	0.654	13.825	-490.1124	6.41	2.47
0.826	0.00	0.681	12.825	-510.0869	6.68	2.57
0.828	0.00	0.705	11.786	-528.5333	6.92	2.66
0.830	0.00	0.728	10.712	-545.3963	7.14	2.75
0.832	0.00	0.748	9.606	-560.6254	7.34	2.82
0.834	0.00	0.766	8.471	-574.1748	7.51	2.89
0.836	0.00	0.782	7.311	-586.0041	7.67	2.95
0.838	0.00	0.795	6.129	-596.0778	7.80	3.00
0.840	0.00	0.807	4.928	-604.3656	7.91	3.05
0.842	0.00	0.815	3.713	-610.8429	7.99	3.08
0.844	0.00	0.821	2.487	-615.4901	8.05	3.10
0.846	0.00	0.825	1.253	-618.2934	8.09	3.12
0.848	0.00	0.826	0.016	-619.2444	8.10	3.12
0.850	0.00	0.825	-1.222	-618.3401	8.09	3.12
0.852	0.00	0.821	-2.456	-615.5834	8.06	3.10
0.854	0.00	0.815	-3.682	-610.9825	8.00	3.08
0.856	0.00	0.807	-4.898	-604.5511	7.91	3.05
0.858	0.00	0.796	-6.099	-596.3085	7.80	3.00
0.860	0.00	0.782	-7.281	-586.2794	7.67	2.95
0.862	0.00	0.767	-8.442	-574.4939	7.52	2.89
0.864	0.00	0.749	-9.578	-560.9873	7.34	2.83
0.866	0.00	0.728	-10.685	-545.8001	7.14	2.75
0.868	0.00	0.706	-11.759	-528.9776	6.92	2.67
0.870	0.00	0.681	-12.799	-510.5704	6.68	2.57
0.872	0.00	0.655	-13.800	-490.6336	6.42	2.47
0.874	0.00	0.626	-14.760	-469.2269	6.14	2.36
0.876	0.00	0.596	-15.676	-446.4144	5.84	2.25
0.878	0.00	0.564	-16.544	-422.2646	5.53	2.13
0.880	0.00	0.530	-17.363	-396.8496	5.19	2.00
0.882	0.00	0.494	-18.130	-370.2458	4.85	1.87
0.884	0.00	0.457	-18.843	-342.5327	4.48	1.73
0.886	0.00	0.419	-19.500	-313.7934	4.11	1.58
0.888	0.00	0.379	-20.097	-284.1140	3.72	1.43

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.890	0.00	0.338	-20.635	-253.5835	3.32	1.28
0.892	0.00	0.297	-21.111	-222.2932	2.91	1.12
0.894	0.00	0.254	-21.524	-190.3370	2.49	0.96
0.896	0.00	0.211	-21.872	-157.8106	2.07	0.80
0.898	0.00	0.167	-22.154	-124.8113	1.63	0.63
0.900	0.00	0.122	-22.371	-91.4382	1.20	0.46
0.902	0.00	0.077	-22.520	-57.7911	0.76	0.29
0.904	0.00	0.032	-22.602	-23.9708	0.31	0.12
0.906	0.00	-0.013	-22.616	9.9212	-0.13	-0.05
0.908	0.00	-0.058	-22.562	43.7835	-0.57	-0.22
0.910	0.00	-0.103	-22.441	77.5147	-1.01	-0.39
0.912	0.00	-0.148	-22.252	111.0136	-1.45	-0.56
0.914	0.00	-0.192	-21.997	144.1800	-1.89	-0.73
0.916	0.00	-0.236	-21.676	176.9144	-2.32	-0.89
0.918	0.00	-0.279	-21.290	209.1187	-2.74	-1.05
0.920	0.00	-0.321	-20.840	240.6966	-3.15	-1.21
0.922	0.00	-0.362	-20.328	271.5534	-3.55	-1.37
0.924	0.00	-0.402	-19.755	301.5966	-3.95	-1.52
0.926	0.00	-0.441	-19.122	330.7363	-4.33	-1.67
0.928	0.00	-0.479	-18.433	358.8851	-4.70	-1.81
0.930	0.00	-0.515	-17.688	385.9588	-5.05	-1.94
0.932	0.00	-0.550	-16.890	411.8761	-5.39	-2.08
0.934	0.00	-0.583	-16.042	436.5595	-5.71	-2.20
0.936	0.00	-0.614	-15.145	459.9350	-6.02	-2.32
0.938	0.00	-0.643	-14.203	481.9326	-6.31	-2.43
0.940	0.00	-0.671	-13.219	502.4864	-6.58	-2.53
0.942	0.00	-0.696	-12.195	521.5348	-6.83	-2.63
0.944	0.00	-0.719	-11.134	539.0207	-7.05	-2.72
0.946	0.00	-0.740	-10.040	554.8917	-7.26	-2.80
0.948	0.00	-0.759	-8.916	569.1004	-7.45	-2.87
0.950	0.00	-0.776	-7.766	581.6041	-7.61	-2.93
0.952	0.00	-0.791	-6.592	592.3654	-7.75	-2.98
0.954	0.00	-0.802	-5.398	601.3520	-7.87	-3.03
0.956	0.00	-0.812	-4.188	608.5370	-7.96	-3.07
0.958	0.00	-0.819	-2.966	613.8989	-8.03	-3.09
0.960	0.00	-0.824	-1.734	617.4216	-8.08	-3.11
0.962	0.00	-0.826	-0.498	619.0946	-8.10	-3.12
0.964	0.00	-0.826	0.740	618.9128	-8.10	-3.12
0.966	0.00	-0.823	1.976	616.8768	-8.07	-3.11
0.968	0.00	-0.818	3.206	612.9928	-8.02	-3.09
0.970	0.00	-0.810	4.426	607.2722	-7.95	-3.06
0.972	0.00	-0.800	5.633	599.7324	-7.85	-3.02
0.974	0.00	-0.788	6.823	590.3958	-7.73	-2.97
0.976	0.00	-0.773	7.993	579.2904	-7.58	-2.92
0.978	0.00	-0.756	9.139	566.4496	-7.41	-2.85
0.980	0.00	-0.737	10.257	551.9117	-7.22	-2.78
0.982	0.00	-0.715	11.345	535.7203	-7.01	-2.70
0.984	0.00	-0.691	12.398	517.9240	-6.78	-2.61
0.986	0.00	-0.665	13.415	498.5761	-6.52	-2.51
0.988	0.00	-0.638	14.391	477.7344	-6.25	-2.41
0.990	0.00	-0.608	15.324	455.4615	-5.96	-2.29
0.992	0.00	-0.576	16.212	431.8241	-5.65	-2.18
0.994	0.00	-0.543	17.050	406.8930	-5.32	-2.05
0.996	0.00	-0.508	17.838	380.7429	-4.98	-1.92
0.998	0.00	-0.472	18.572	353.4521	-4.63	-1.78
1.000	0.00	-0.434	19.251	325.1024	-4.25	-1.64
1.002	0.00	-0.395	19.872	295.7787	-3.87	-1.49
1.004	0.00	-0.354	20.433	265.5689	-3.48	-1.34
1.006	0.00	-0.313	20.933	234.5635	-3.07	-1.18
1.008	0.00	-0.271	21.371	202.8554	-2.65	-1.02
1.010	0.00	-0.228	21.744	170.5395	-2.23	-0.86

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
1.012	0.00	-0.184	22.052	137.7127	-1.80	-0.69
1.014	0.00	-0.139	22.294	104.4733	-1.37	-0.53
1.016	0.00	-0.095	22.470	70.9210	-0.93	-0.36
1.018	0.00	-0.050	22.578	37.1561	-0.49	-0.19
1.020	0.00	-0.004	22.618	3.2800	-0.04	-0.02
1.022	0.00	0.041	22.591	-30.6060	0.40	0.15
1.024	0.00	0.086	22.496	-64.4003	0.84	0.32
1.026	0.00	0.131	22.334	-98.0016	1.28	0.49
1.028	0.00	0.175	22.104	-131.3094	1.72	0.66
1.030	0.00	0.219	21.809	-164.2237	2.15	0.83
1.032	0.00	0.262	21.448	-196.6461	2.57	0.99
1.034	0.00	0.305	21.023	-228.4793	2.99	1.15
1.036	0.00	0.346	20.535	-259.6280	3.40	1.31
1.038	0.00	0.387	19.985	-289.9989	3.80	1.46
1.040	0.00	0.426	19.375	-319.5010	4.18	1.61
1.042	0.00	0.464	18.708	-348.0459	4.55	1.75
1.044	0.00	0.501	17.984	-375.5481	4.91	1.89
1.046	0.00	0.536	17.207	-401.9252	5.26	2.03
1.048	0.00	0.570	16.378	-427.0982	5.59	2.15
1.050	0.00	0.602	15.500	-450.9916	5.90	2.27
1.052	0.00	0.632	14.575	-473.5339	6.20	2.39
1.054	0.00	0.660	13.607	-494.6575	6.47	2.49
1.056	0.00	0.686	12.598	-514.2992	6.73	2.59
1.058	0.00	0.710	11.551	-532.4001	6.97	2.68
1.060	0.00	0.733	10.470	-548.9060	7.18	2.77
1.062	0.00	0.752	9.357	-563.7674	7.38	2.84
1.064	0.00	0.770	8.217	-576.9398	7.55	2.91
1.066	0.00	0.785	7.051	-588.3838	7.70	2.96
1.068	0.00	0.798	5.865	-598.0650	7.83	3.01
1.070	0.00	0.809	4.661	-605.9545	7.93	3.05
1.072	0.00	0.817	3.443	-612.0286	8.01	3.08
1.074	0.00	0.822	2.215	-616.2691	8.07	3.11
1.076	0.00	0.826	0.980	-618.6634	8.10	3.12
1.078	0.00	0.826	-0.258	-619.2042	8.10	3.12
1.080	0.00	0.825	-1.495	-617.8899	8.09	3.11
1.082	0.00	0.820	-2.728	-614.7245	8.04	3.10
1.084	0.00	0.814	-3.952	-609.7174	7.98	3.07
1.086	0.00	0.805	-5.165	-602.8837	7.89	3.04
1.088	0.00	0.793	-6.362	-594.2438	7.78	2.99
1.090	0.00	0.779	-7.540	-583.8237	7.64	2.94
1.092	0.00	0.763	-8.696	-571.6544	7.48	2.88
1.094	0.00	0.744	-9.825	-557.7725	7.30	2.81
1.096	0.00	0.724	-10.925	-542.2196	7.10	2.73
1.098	0.00	0.701	-11.992	-525.0422	6.87	2.65
1.100	0.00	0.676	-13.024	-506.2919	6.63	2.55
1.102	0.00	0.649	-14.016	-486.0248	6.36	2.45
1.104	0.00	0.620	-14.966	-464.3016	6.08	2.34
1.106	0.00	0.589	-15.872	-441.1874	5.77	2.22
1.108	0.00	0.556	-16.730	-416.7514	5.45	2.10
1.110	0.00	0.522	-17.538	-391.0669	5.12	1.97
1.112	0.00	0.486	-18.293	-364.2108	4.77	1.84
1.114	0.00	0.449	-18.993	-336.2636	4.40	1.69
1.116	0.00	0.410	-19.637	-307.3089	4.02	1.55
1.118	0.00	0.370	-20.222	-277.4336	3.63	1.40
1.120	0.00	0.329	-20.746	-246.7271	3.23	1.24
1.122	0.00	0.287	-21.208	-215.2815	2.82	1.08





Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

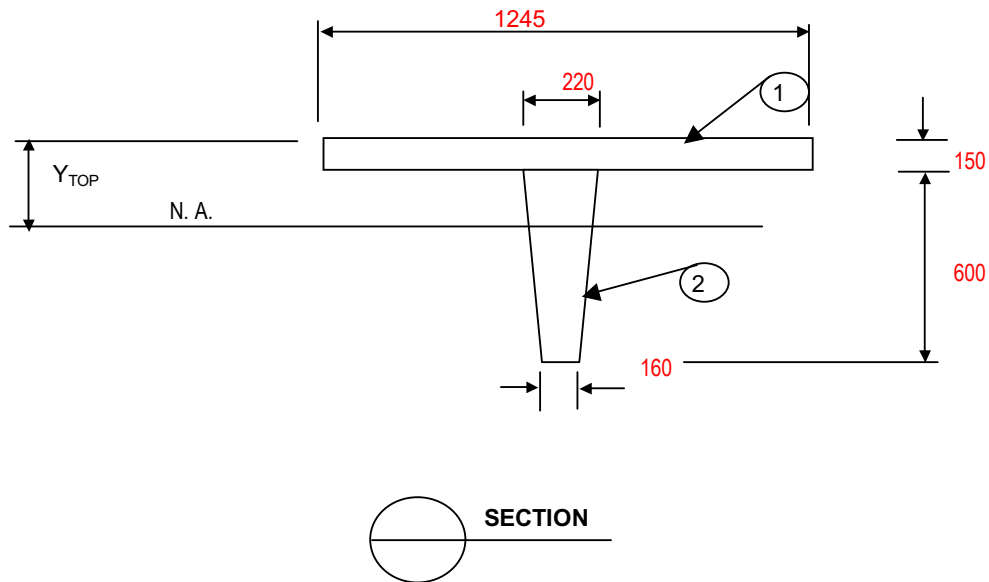
Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

ROOF DOUBLE TEE BEAM ; SS # 1

Saudconsult

Job No. :	BI-3039	Sheet no. :	of 3
Project :	REPLACEMENT OF SUBSTATION 472, 321 & F07, ABU ALI	Date :	18/11/2003
Subject :	CROSS SECTION PROPERTIES, SUBSTATION # 1	Design by :	MGG
		Check'd by:	SAK

PRECAST ROOF PANEL DOUBLE TEE BEAM CROSS SECTION



ELEMENT	AREA (A)	Y	A x Y	Io	D	A x D ²
1	186750	75.00	14006250	3.5E+08	136.160	3462239849
2	114000	434.21	49500000	3.4E+09	223.051	5671695543
TOTAL	300750		63506250	3.7E+09		9133935392

$$Y_{TOP} = \frac{63506250}{300750}$$

$$= 211.159601 \text{ MM}$$

$$I_x = 1.2876E+10 \text{ MM}^4$$

$$I_x = 0.0129 \text{ M}^4$$

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : DOUBLE ROOF TEE BEAM PROPERTIES, SS #1

Sheet no. : of
 Date : 27-Jan-03
 Design by : MGG
 Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Roof Dead Load = 8.470 KN/m²
 = 176.90 psf

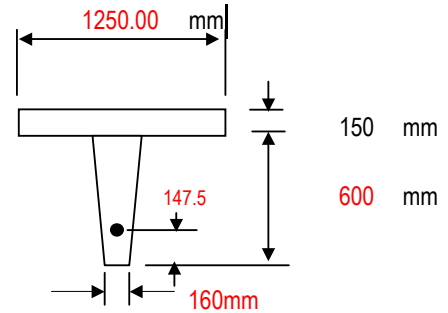
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
 Reinf., f_y = 414 Mpa

PANEL DATA : (CONSIDER HALF WIDTH)

Panel Width, width = 1250 mm (49.21 in)
 Flange Thickness = 150 mm (5.91 in)
 Web Depth = 600 mm (23.62 in)
 Total Depth, d_t = 750 mm (29.53 in)
 Panel Span = 15220 mm (599.21 in)

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure, P_s = 1.96 psi (13.547 kPa)
 Blast time duration, T_d = 200 msec

$f'_{dc} = (SIF) \times (DIF) \times f'_c$ (for bending) = 5954.3 psi (41 Mpa.)
 $f'_{dc} = (SIF) \times (DIF) \times f'_c$ (for shear) = 5504.0 psi (38 Mpa.)
 $f_{dy} = (SIF) \times (DIF) \times f_y$ = 77276 psi (533 Mpa.)
 Modulus of Elasticity, $E_{dc} = 57000 \times (f'_c)^{1/2}$ = 4032041 psi (27801 Mpa.)

Positive :

Area of Bar ($\emptyset$), 25 mm $\emptyset$ A_b = 0.7609 in² (491 mm²)
 No. of Web Main Bar = 8.0000 in (203 mm)
 Total Area of Web Rebar PER panel, A_s = 6.0868 in² (3927 mm²)
 Effective Depth, $d_e = d_t - 147.5$ = 23.7205 in (603 mm)
 Steel ratio, $p = A_s / b \times d$ = 0.0052 > $200 / f_{dy} = 0.003$
 Stress block, $a = A_s \times f_{dy} / 0.85 \times f'_{dc} \times b$ = 1.8885 in (48 mm)
 Modular ratio, $n = E_s / E_{dc}$ = 7.1940
 Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$ = 10713196.1 lb-in (1210368 KN-m)
 Shear, $V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$ = 173208.2 lb (770 KN)
 Resistance at cracked moment, $R_u = 8 \times M_p / L$ = 143030 lb (636.20 KN)
 143.030 kips > 57.94 $\therefore$ O.K!

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : DOUBLE ROOF TEE BEAM PROPERTIES, SS #1

Sheet no. : of
 Date : 27-Jan-03
 Design by : MGG
 Check'd by : SAK

Reversal :

Area of Bar ($\emptyset$), 20 mm $\emptyset$ $Ab = 0.4869$ in² (314 mm²)
 No. of Flange Rebar = 5.0000
 Total Area of Flange Rebar PER panel, = 2.4347 in² (13 mm²)
 Effective Depth, $de = dt - tf/2$ = 26.5748 in (675 mm)
 Steel ratio, $p = As / b \times d$ = 0.0145 > 200 / $f_{dy} = 0.003$
 Stress block, $a = As \times f_{dy} / 0.85 \times f'_{dc} \times b$ = 5.9015 in (150 mm)
 Modular ratio, $n = E_s / E_{dc}$ = 7.1940
 Max. Moment, $M_p = M_{pc} = M_{ps} = As \times f_{dy} [d - a / 2]$ = 4444795.3 lb-in (502169 KN-m)
 Shear, $V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$ = 24838.5 lb (110 KN)
 Resistance at cracked moment, $R_u = 8 \times M_p / L$ = 59342 lb (263.95 KN)
 59.342 kips

Cracked Moment of Inertia:

$n \times As = 43.7888$ in² (28251 mm²)
 $C = -n \times As + [n \times As \times (n \times As + 2 \times b \times d)]^{0.5} / b = 5.6680$ in² (3657 mm²)
 $I_{cr} = b \times C^3 / 3 + n \times As \times (d - C)^2 = 17257.48$ in⁴ (7183105801 mm⁴)

Moment of Inertia, $I_g = 30933.92$ in⁴ (12875670453 mm⁴)
 Effective Moment of Inertia, $0.5 I_g = 15466.96$ in⁴ (6437835227 mm⁴)
 Average Moment of Inertia, $I_a = (I_g + I_{cr}) / 2 = 16362.221$ in⁴ (6810470514 mm⁴)
 Effective Stiffness, $K = 384 \times E_{dc} \times I_a / 5 L^3 = 23.5$ kips / in
 Yield Deflection, $Y_e = R_u / K = 6.07355$ in
 Total Roof Mass = Roof DL x width x roof span / gravity acc. = 0.09385 kip-sec² / in
 Additional Mass = 0.00000 kip-sec² / in
 Total Mass, M = 0.09385 kip-sec² / in
 Average Stiffness, mass factor $K_{LM} = 0.72000$ FROM ASCE TABLE 6.1
 Equivalent Mass, $Me = K_{LM} \times \text{Beam Mass} = 0.06757$ kip-sec² / in
 Period of Vibration, $tn = 2 \times \pi \times (Me / K)^{0.5} = 0.33657$ sec
 Time increment, $tn/10$ $\Delta t = 0.03366$ sec ; use $\Delta t = 0.00200$ sec
 $td / tn = 0.59424$

Initial Load and Deflection based on Gravity loads:

$R_{ini} = \text{Roof DL} \times \text{Width} \times \text{Span}$ $R_{ini} = 36.2261$ kips (161.13 KN)
 Deflection = R_{ini} / K = 1.5383 in (39 mm)
 Rotation ; θ = 0.2533 degrees

Total Initial Net Loads on member

$F(t) = P_s + R_{ini}$ $F(t) = 94.1657$ kips (418.85 KN)

NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	23.550 kips / in
Mass	M	=	0.0939 kip-sec ² / in
Tension resistance	Rut	=	143.03 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	94.17 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.200 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load History	
load (kips)	time (sec)
94.17	0
36.226107	0.2000

Length	599.213 in
y _{max} =	2.956 in
r _{max} =	105.832 kips
μ =	0.487
θ =	0.565 degrees
Rut/Fo =	1.519
θ = < 2 degrees	
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.000	94.17	0.000	0.000	617.3639	36.23	24.78
0.002	93.59	0.001	1.228	610.8815	36.26	24.72
0.004	93.01	0.005	2.443	603.7861	36.34	24.69
0.006	92.43	0.011	3.643	596.0847	36.48	24.68
0.008	91.85	0.019	4.827	587.7852	36.68	24.69
0.010	91.27	0.030	5.993	578.8957	36.94	24.72
0.012	90.69	0.043	7.142	569.4253	37.25	24.77
0.014	90.11	0.059	8.270	559.3835	37.61	24.84
0.016	89.53	0.076	9.379	548.7803	38.03	24.94
0.018	88.95	0.096	10.465	537.6264	38.49	25.05
0.020	88.37	0.118	11.529	525.9329	39.01	25.18
0.022	87.79	0.142	12.568	513.7117	39.58	25.33
0.024	87.21	0.169	13.583	500.9749	40.20	25.51
0.026	86.63	0.197	14.572	487.7353	40.86	25.69
0.028	86.05	0.227	15.533	474.0063	41.57	25.90
0.030	85.47	0.259	16.467	459.8016	42.32	26.12
0.032	84.90	0.293	17.372	445.1355	43.12	26.36
0.034	84.32	0.328	18.247	430.0226	43.96	26.62
0.036	83.74	0.366	19.092	414.4782	44.84	26.89
0.038	83.16	0.405	19.905	398.5179	45.76	27.18
0.040	82.58	0.445	20.685	382.1576	46.71	27.48
0.042	82.00	0.487	21.433	365.4138	47.70	27.80
0.044	81.42	0.531	22.147	348.3033	48.73	28.12
0.046	80.84	0.576	22.826	330.8432	49.79	28.47
0.048	80.26	0.622	23.470	313.0512	50.88	28.82
0.050	79.68	0.670	24.078	294.9449	52.00	29.18
0.052	79.10	0.719	24.649	276.5427	53.15	29.56
0.054	78.52	0.768	25.184	257.8630	54.32	29.94
0.056	77.94	0.819	25.680	238.9245	55.52	30.34
0.058	77.36	0.871	26.139	219.7462	56.74	30.74
0.060	76.78	0.924	26.559	200.3474	57.98	31.15
0.062	76.20	0.977	26.940	180.7475	59.24	31.57

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.064	75.63	1.032	27.282	160.9662	60.52	32.00
0.066	75.05	1.086	27.584	141.0234	61.81	32.43
0.068	74.47	1.142	27.846	120.9391	63.12	32.86
0.070	73.89	1.198	28.068	100.7334	64.43	33.30
0.072	73.31	1.254	28.249	80.4266	65.76	33.75
0.074	72.73	1.311	28.389	60.0391	67.09	34.19
0.076	72.15	1.368	28.489	39.5914	68.43	34.64
0.078	71.57	1.425	28.548	19.1039	69.78	35.09
0.080	70.99	1.482	28.565	-1.4028	71.12	35.55
0.082	70.41	1.539	28.542	-21.9080	72.47	36.00
0.084	69.83	1.596	28.478	-42.3913	73.81	36.45
0.086	69.25	1.653	28.372	-62.8320	75.15	36.90
0.088	68.67	1.709	28.226	-83.2097	76.48	37.34
0.090	68.09	1.766	28.040	-103.5038	77.81	37.79
0.092	67.51	1.822	27.813	-123.6941	79.12	38.23
0.094	66.93	1.877	27.545	-143.7603	80.43	38.66
0.096	66.35	1.932	27.238	-163.6821	81.72	39.09
0.098	65.78	1.986	26.891	-183.4398	82.99	39.52
0.100	65.20	2.039	26.504	-203.0133	84.25	39.93
0.102	64.62	2.092	26.079	-222.3831	85.49	40.34
0.104	64.04	2.143	25.615	-241.5297	86.70	40.75
0.106	63.46	2.194	25.113	-260.4339	87.90	41.14
0.108	62.88	2.244	24.573	-279.0768	89.07	41.52
0.110	62.30	2.292	23.997	-297.4396	90.21	41.90
0.112	61.72	2.340	23.384	-315.5039	91.33	42.26
0.114	61.14	2.386	22.735	-333.2516	92.42	42.61
0.116	60.56	2.431	22.051	-350.6649	93.47	42.95
0.118	59.98	2.474	21.333	-367.7262	94.49	43.28
0.120	59.40	2.516	20.581	-384.4186	95.48	43.59
0.122	58.82	2.556	19.795	-400.7251	96.43	43.89
0.124	58.24	2.595	18.978	-416.6295	97.34	44.18
0.126	57.66	2.632	18.129	-432.1158	98.22	44.45
0.128	57.08	2.668	17.250	-447.1684	99.05	44.70
0.130	56.50	2.701	16.341	-461.7723	99.84	44.94
0.132	55.93	2.733	15.403	-475.9127	100.59	45.16
0.134	55.35	2.763	14.438	-489.5756	101.29	45.36
0.136	54.77	2.791	13.446	-502.7471	101.95	45.55
0.138	54.19	2.817	12.427	-515.4142	102.56	45.72
0.140	53.61	2.841	11.384	-527.5639	103.12	45.87
0.142	53.03	2.862	10.318	-539.1843	103.63	46.00
0.144	52.45	2.882	9.228	-550.2635	104.09	46.11
0.146	51.87	2.899	8.117	-560.7905	104.50	46.20
0.148	51.29	2.914	6.986	-570.7548	104.86	46.27
0.150	50.71	2.927	5.835	-580.1462	105.16	46.32
0.152	50.13	2.938	4.666	-588.9555	105.41	46.35
0.154	49.55	2.946	3.480	-597.1737	105.60	46.35
0.156	48.97	2.951	2.278	-604.7926	105.73	46.34
0.158	48.39	2.955	1.061	-611.8046	105.81	46.30
0.160	47.81	2.956	-0.169	-618.2026	105.83	46.24
0.162	47.23	2.954	-1.411	-623.9802	105.80	46.16
0.164	46.66	2.950	-2.664	-629.1316	105.70	46.06
0.166	46.08	2.943	-3.927	-633.6517	105.54	45.93

*

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.168	45.50	2.934	-5.198	-637.5358	105.33	45.78
0.170	44.92	2.923	-6.477	-640.7802	105.05	45.61
0.172	44.34	2.908	-7.761	-643.3815	104.72	45.42
0.174	43.76	2.892	-9.049	-645.3371	104.32	45.20
0.176	43.18	2.872	-10.341	-646.6451	103.87	44.95
0.178	42.60	2.850	-11.635	-647.3042	103.35	44.69
0.180	42.02	2.826	-12.930	-647.3136	102.77	44.40
0.182	41.44	2.799	-14.224	-646.6735	102.13	44.09
0.184	40.86	2.769	-15.516	-645.3843	101.43	43.75
0.186	40.28	2.736	-16.805	-643.4476	100.67	43.39
0.188	39.70	2.702	-18.089	-640.8650	99.85	43.01
0.190	39.12	2.664	-19.368	-637.6394	98.97	42.60
0.192	38.54	2.624	-20.639	-633.7738	98.02	42.17
0.194	37.96	2.582	-21.902	-629.2722	97.02	41.72
0.196	37.38	2.537	-23.156	-624.1391	95.96	41.24
0.198	36.81	2.489	-24.398	-618.3797	94.84	40.75
0.200	36.23	2.439	-25.628	-611.9996	93.66	40.23
0.202	36.23	2.386	-26.839	-598.8328	92.43	39.75
0.204	36.23	2.332	-28.023	-585.0651	91.13	39.25
0.206	36.23	2.274	-29.179	-570.7102	89.79	38.73
0.208	36.23	2.215	-30.305	-555.7825	88.39	38.19
0.210	36.23	2.153	-31.402	-540.2972	86.93	37.64
0.212	36.23	2.089	-32.466	-524.2695	85.43	37.06
0.214	36.23	2.023	-33.498	-507.7158	83.88	36.46
0.216	36.23	1.955	-34.496	-490.6526	82.27	35.84
0.218	36.23	1.885	-35.460	-473.0969	80.63	35.21
0.220	36.23	1.814	-36.388	-455.0665	78.93	34.56
0.222	36.23	1.740	-37.280	-436.5794	77.20	33.89
0.224	36.23	1.664	-38.134	-417.6542	75.42	33.20
0.226	36.23	1.587	-38.950	-398.3098	73.61	32.50
0.228	36.23	1.509	-39.727	-378.5657	71.75	31.79
0.230	36.23	1.428	-40.464	-358.4417	69.87	31.06
0.232	36.23	1.347	-41.161	-337.9580	67.94	30.32
0.234	36.23	1.264	-41.816	-317.1351	65.99	29.57
0.236	36.23	1.180	-42.429	-295.9940	64.01	28.81
0.238	36.23	1.094	-42.999	-274.5558	61.99	28.03
0.240	36.23	1.008	-43.527	-252.8421	59.96	27.25
0.242	36.23	0.920	-44.010	-230.8746	57.89	26.46
0.244	36.23	0.832	-44.450	-208.6755	55.81	25.65
0.246	36.23	0.742	-44.845	-186.2669	53.71	24.84
0.248	36.23	0.652	-45.195	-163.6714	51.59	24.03
0.250	36.23	0.562	-45.499	-140.9117	49.45	23.20
0.252	36.23	0.470	-45.758	-118.0105	47.30	22.38
0.254	36.23	0.379	-45.971	-94.9910	45.14	21.55
0.256	36.23	0.286	-46.138	-71.8761	42.97	20.71
0.258	36.23	0.194	-46.259	-48.6890	40.80	19.87
0.260	36.23	0.101	-46.333	-25.4531	38.61	19.03
0.262	36.23	0.009	-46.361	-2.1917	36.43	18.19
0.264	36.23	-0.084	-46.342	21.0720	34.25	17.35
0.266	36.23	-0.177	-46.276	44.3145	32.07	16.51
0.268	36.23	-0.269	-46.164	67.5125	29.89	15.67
0.270	36.23	-0.361	-46.006	90.6428	27.72	14.84

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.272	36.23	-0.453	-45.802	113.6821	25.56	14.01
0.274	36.23	-0.544	-45.552	136.6073	23.41	13.18
0.276	36.23	-0.635	-45.256	159.3954	21.27	12.35
0.278	36.23	-0.725	-44.914	182.0236	19.14	11.54
0.280	36.23	-0.815	-44.528	204.4691	17.04	10.73
0.282	36.23	-0.903	-44.097	226.7094	14.95	9.92
0.284	36.23	-0.991	-43.621	248.7222	12.88	9.13
0.286	36.23	-1.078	-43.102	270.4854	10.84	8.34
0.288	36.23	-1.164	-42.539	291.9771	8.82	7.56
0.290	36.23	-1.248	-41.934	313.1759	6.83	6.80
0.292	36.23	-1.331	-41.287	334.0603	4.87	6.04
0.294	36.23	-1.413	-40.598	354.6095	2.95	5.30
0.296	36.23	-1.494	-39.869	374.8028	1.05	4.57
0.298	36.23	-1.573	-39.100	394.6200	-0.81	3.85
0.300	36.23	-1.650	-38.291	414.0412	-2.63	3.15
0.302	36.23	-1.726	-37.444	433.0468	-4.42	2.47
0.304	36.23	-1.800	-36.559	451.6179	-6.16	1.80
0.306	36.23	-1.872	-35.638	469.7358	-7.86	1.14
0.308	36.23	-1.942	-34.681	487.3822	-9.51	0.50
0.310	36.23	-2.011	-33.689	504.5396	-11.12	-0.12
0.312	36.23	-2.077	-32.663	521.1906	-12.69	-0.72
0.314	36.23	-2.141	-31.605	537.3186	-14.20	-1.30
0.316	36.23	-2.203	-30.514	552.9073	-15.66	-1.86
0.318	36.23	-2.263	-29.393	567.9412	-17.08	-2.41
0.320	36.23	-2.321	-28.243	582.4051	-18.43	-2.93
0.322	36.23	-2.376	-27.064	596.2846	-19.74	-3.43
0.324	36.23	-2.429	-25.859	609.5656	-20.98	-3.91
0.326	36.23	-2.480	-24.627	622.2349	-22.17	-4.37
0.328	36.23	-2.528	-23.370	634.2798	-23.30	-4.80
0.330	36.23	-2.573	-22.090	645.6882	-24.37	-5.22
0.332	36.23	-2.616	-20.788	656.4485	-25.38	-5.61
0.334	36.23	-2.656	-19.465	666.5501	-26.33	-5.97
0.336	36.23	-2.694	-18.123	675.9828	-27.21	-6.31
0.338	36.23	-2.729	-16.762	684.7371	-28.04	-6.63
0.340	36.23	-2.761	-15.384	692.8042	-28.79	-6.92
0.342	36.23	-2.790	-13.991	700.1761	-29.49	-7.19
0.344	36.23	-2.817	-12.584	706.8453	-30.11	-7.43
0.346	36.23	-2.841	-11.165	712.8051	-30.67	-7.64
0.348	36.23	-2.862	-9.734	718.0496	-31.16	-7.83
0.350	36.23	-2.880	-8.293	722.5735	-31.59	-8.00
0.352	36.23	-2.895	-6.844	726.3723	-31.94	-8.13
0.354	36.23	-2.907	-5.388	729.4421	-32.23	-8.24
0.356	36.23	-2.916	-3.927	731.7799	-32.45	-8.33
0.358	36.23	-2.923	-2.462	733.3833	-32.60	-8.39
0.360	36.23	-2.926	-0.994	734.2508	-32.68	-8.42
0.362	36.23	-2.927	0.474	734.3813	-32.70	-8.42
0.364	36.23	-2.924	1.942	733.7749	-32.64	-8.40
0.366	36.23	-2.919	3.409	732.4321	-32.51	-8.35
0.368	36.23	-2.911	4.871	730.3543	-32.32	-8.28
0.370	36.23	-2.899	6.329	727.5435	-32.05	-8.17
0.372	36.23	-2.885	7.781	724.0026	-31.72	-8.05
0.374	36.23	-2.868	9.224	719.7352	-31.32	-7.89

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.376	36.23	-2.848	10.659	714.7454	-30.85	-7.71
0.378	36.23	-2.826	12.083	709.0384	-30.32	-7.51
0.380	36.23	-2.800	13.494	702.6198	-29.71	-7.27
0.382	36.23	-2.772	14.893	695.4961	-29.05	-7.02
0.384	36.23	-2.741	16.276	687.6744	-28.31	-6.73
0.386	36.23	-2.707	17.643	679.1626	-27.51	-6.43
0.388	36.23	-2.670	18.992	669.9693	-26.65	-6.09
0.390	36.23	-2.631	20.322	660.1035	-25.72	-5.74
0.392	36.23	-2.589	21.631	649.5754	-24.74	-5.36
0.394	36.23	-2.544	22.919	638.3954	-23.69	-4.95
0.396	36.23	-2.497	24.184	626.5747	-22.58	-4.53
0.398	36.23	-2.447	25.425	614.1252	-21.41	-4.08
0.400	36.23	-2.395	26.640	601.0594	-20.18	-3.60
0.402	36.23	-2.341	27.829	587.3904	-18.90	-3.11
0.404	36.23	-2.284	28.989	573.1319	-17.56	-2.60
0.406	36.23	-2.225	30.121	558.2983	-16.17	-2.06
0.408	36.23	-2.164	31.222	542.9044	-14.73	-1.50
0.410	36.23	-2.100	32.292	526.9656	-13.23	-0.93
0.412	36.23	-2.034	33.329	510.4981	-11.68	-0.33
0.414	36.23	-1.967	34.333	493.5182	-10.09	0.28
0.416	36.23	-1.897	35.303	476.0430	-8.45	0.91
0.418	36.23	-1.826	36.237	458.0901	-6.77	1.56
0.420	36.23	-1.752	37.135	439.6775	-5.04	2.23
0.422	36.23	-1.677	37.995	420.8237	-3.27	2.91
0.424	36.23	-1.600	38.818	401.5475	-1.46	3.60
0.426	36.23	-1.522	39.601	381.8684	0.39	4.32
0.428	36.23	-1.442	40.345	361.8060	2.27	5.04
0.430	36.23	-1.360	41.048	341.3806	4.19	5.78
0.432	36.23	-1.278	41.710	320.6125	6.14	6.53
0.434	36.23	-1.194	42.330	299.5227	8.12	7.29
0.436	36.23	-1.108	42.908	278.1324	10.12	8.06
0.438	36.23	-1.022	43.442	256.4629	12.16	8.85
0.440	36.23	-0.935	43.933	234.5360	14.21	9.64
0.442	36.23	-0.846	44.380	212.3738	16.29	10.44
0.444	36.23	-0.757	44.782	189.9984	18.39	11.25
0.446	36.23	-0.667	45.140	167.4324	20.51	12.06
0.448	36.23	-0.577	45.452	144.6983	22.65	12.88
0.450	36.23	-0.485	45.719	121.8190	24.79	13.71
0.452	36.23	-0.394	45.939	98.8175	26.95	14.54
0.454	36.23	-0.302	46.114	75.7168	29.12	15.38
0.456	36.23	-0.209	46.242	52.5401	31.30	16.21
0.458	36.23	-0.117	46.324	29.3107	33.48	17.05
0.460	36.23	-0.024	46.359	6.0519	35.66	17.89
0.462	36.23	0.069	46.348	-17.2130	37.84	18.73
0.464	36.23	0.161	46.290	-40.4606	40.02	19.57
0.466	36.23	0.254	46.186	-63.6676	42.20	20.41
0.468	36.23	0.346	46.036	-86.8108	44.37	21.25
0.470	36.23	0.438	45.839	-109.8668	46.54	22.08
0.472	36.23	0.529	45.596	-132.8125	48.69	22.91
0.474	36.23	0.620	45.308	-155.6250	50.83	23.74
0.476	36.23	0.710	44.974	-178.2813	52.96	24.55
0.478	36.23	0.800	44.595	-200.7586	55.07	25.37
0.480	36.23	0.889	44.171	-223.0345	57.16	26.17
0.482	36.23	0.977	43.703	-245.0866	59.23	26.97
0.484	36.23	1.064	43.191	-266.8927	61.27	27.76
0.486	36.23	1.149	42.636	-288.4310	63.30	28.53
0.488	36.23	1.234	42.038	-309.6798	65.29	29.30
0.490	36.23	1.318	41.397	-330.6179	67.25	30.06
0.492	36.23	1.400	40.716	-351.2241	69.19	30.80
0.494	36.23	1.480	39.993	-371.4779	71.09	31.54

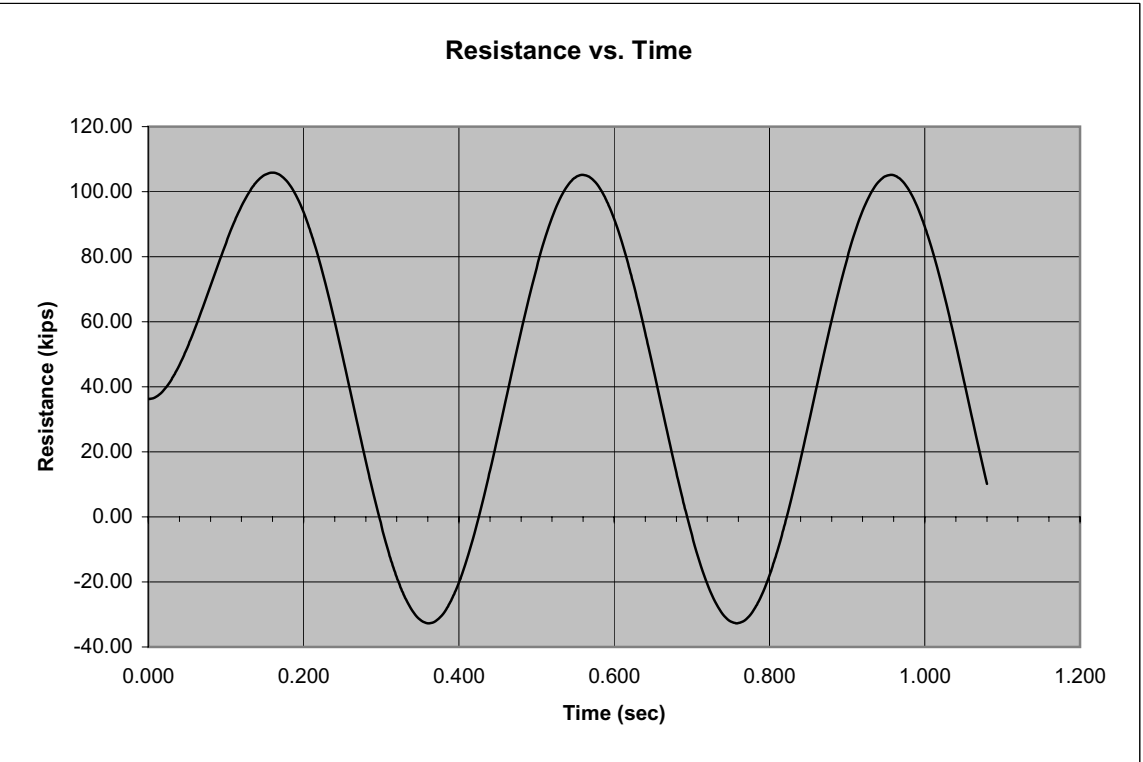
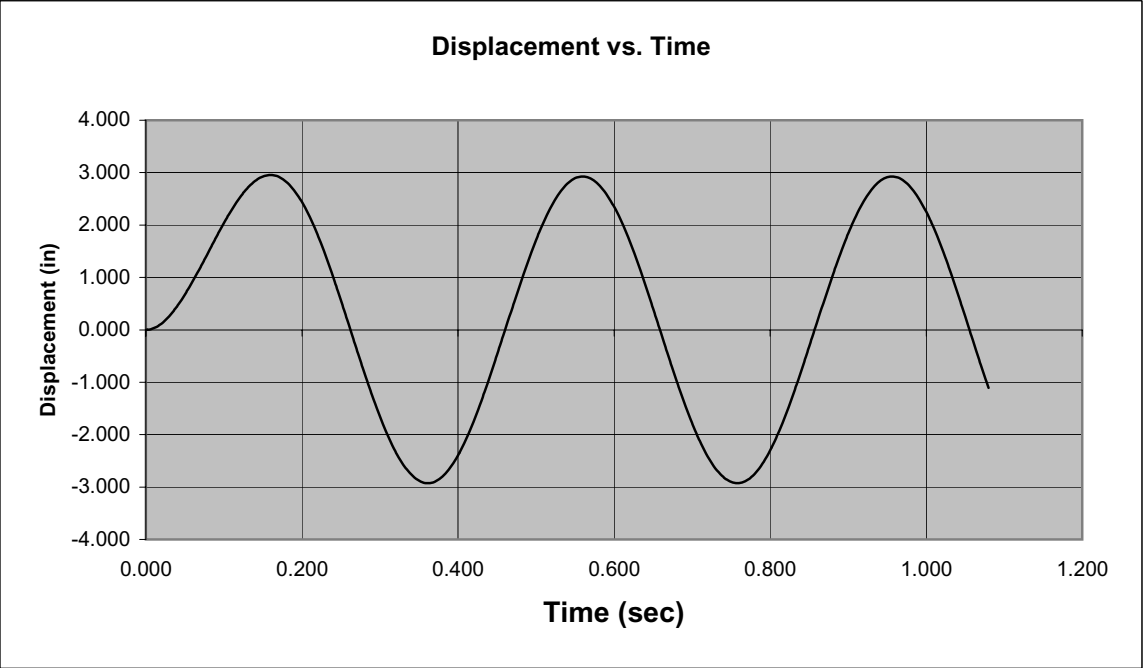
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.496	36.23	1.560	39.230	-391.3589	72.96	32.25
0.498	36.23	1.637	38.428	-410.8471	74.78	32.96
0.500	36.23	1.713	37.587	-429.9231	76.57	33.65
0.502	36.23	1.788	36.709	-448.5676	78.32	34.32
0.504	36.23	1.860	35.793	-466.7619	80.03	34.98
0.506	36.23	1.931	34.842	-484.4878	81.70	35.62
0.508	36.23	1.999	33.856	-501.7275	83.31	36.24
0.510	36.23	2.066	32.836	-518.4637	84.88	36.85
0.512	36.23	2.131	31.782	-534.6796	86.41	37.43
0.514	36.23	2.193	30.697	-550.3589	87.88	38.00
0.516	36.23	2.254	29.582	-565.4859	89.30	38.55
0.518	36.23	2.312	28.436	-580.0454	90.66	39.07
0.520	36.23	2.367	27.262	-594.0228	91.98	39.58
0.522	36.23	2.421	26.060	-607.4041	93.23	40.06
0.524	36.23	2.472	24.833	-620.1758	94.43	40.52
0.526	36.23	2.520	23.580	-632.3251	95.57	40.96
0.528	36.23	2.566	22.304	-643.8399	96.65	41.38
0.530	36.23	2.609	21.006	-654.7085	97.67	41.77
0.532	36.23	2.650	19.686	-664.9201	98.63	42.14
0.534	36.23	2.688	18.347	-674.4644	99.52	42.48
0.536	36.23	2.723	16.989	-683.3319	100.36	42.80
0.538	36.23	2.756	15.614	-691.5136	101.12	43.10
0.540	36.23	2.786	14.224	-699.0014	101.83	43.37
0.542	36.23	2.813	12.819	-705.7876	102.46	43.61
0.544	36.23	2.837	11.401	-711.8656	103.03	43.83
0.546	36.23	2.858	9.972	-717.2292	103.54	44.03
0.548	36.23	2.877	8.533	-721.8730	103.97	44.20
0.550	36.23	2.892	7.085	-725.7924	104.34	44.34
0.552	36.23	2.905	5.630	-728.9834	104.64	44.45
0.554	36.23	2.915	4.170	-731.4428	104.87	44.54
0.556	36.23	2.922	2.705	-733.1682	105.03	44.60
0.558	36.23	2.926	1.238	-734.1578	105.13	44.64
0.560	36.23	2.927	-0.230	-734.4107	105.15	44.65
0.562	36.23	2.925	-1.699	-733.9265	105.11	44.63
0.564	36.23	2.920	-3.165	-732.7058	104.99	44.59
0.566	36.23	2.912	-4.629	-730.7498	104.81	44.52
0.568	36.23	2.901	-6.088	-728.0605	104.55	44.42
0.570	36.23	2.888	-7.540	-724.6405	104.23	44.30
0.572	36.23	2.871	-8.986	-720.4933	103.84	44.15
0.574	36.23	2.852	-10.422	-715.6231	103.39	43.97
0.576	36.23	2.830	-11.847	-710.0347	102.86	43.77
0.578	36.23	2.805	-13.261	-703.7337	102.27	43.54
0.580	36.23	2.777	-14.662	-696.7265	101.61	43.29
0.582	36.23	2.746	-16.047	-689.0201	100.89	43.01
0.584	36.23	2.712	-17.417	-680.6223	100.10	42.71
0.586	36.23	2.676	-18.769	-671.5414	99.25	42.38
0.588	36.23	2.637	-20.102	-661.7865	98.33	42.02
0.590	36.23	2.596	-21.416	-651.3676	97.36	41.65
0.592	36.23	2.552	-22.707	-640.2949	96.32	41.25
0.594	36.23	2.505	-23.976	-628.5797	95.22	40.83
0.596	36.23	2.456	-25.221	-616.2337	94.06	40.38
0.598	36.23	2.404	-26.440	-603.2693	92.84	39.91
0.600	36.23	2.350	-27.633	-589.6995	91.57	39.42
0.602	36.23	2.294	-28.799	-575.5378	90.24	38.91
0.604	36.23	2.235	-29.935	-560.7986	88.86	38.38
0.606	36.23	2.174	-31.041	-545.4966	87.42	37.82
0.608	36.23	2.111	-32.116	-529.6472	85.93	37.25
0.610	36.23	2.045	-33.159	-513.2662	84.40	36.66
0.612	36.23	1.978	-34.169	-496.3701	82.81	36.05
0.614	36.23	1.909	-35.144	-478.9760	81.18	35.42
0.616	36.23	1.838	-36.084	-461.1011	79.50	34.77

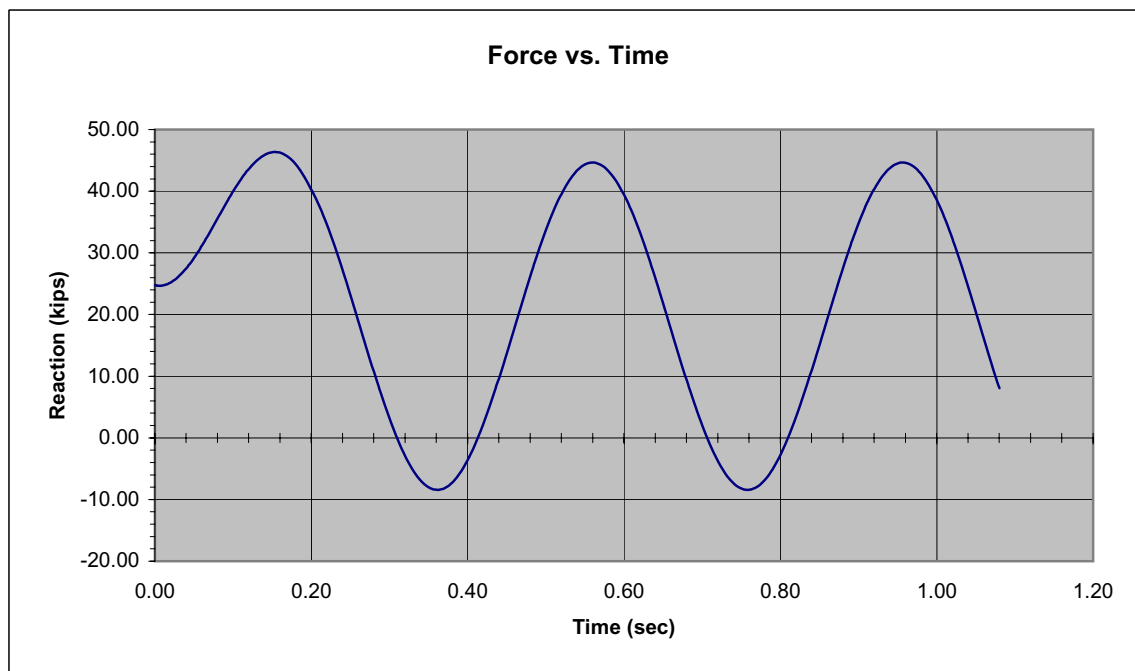
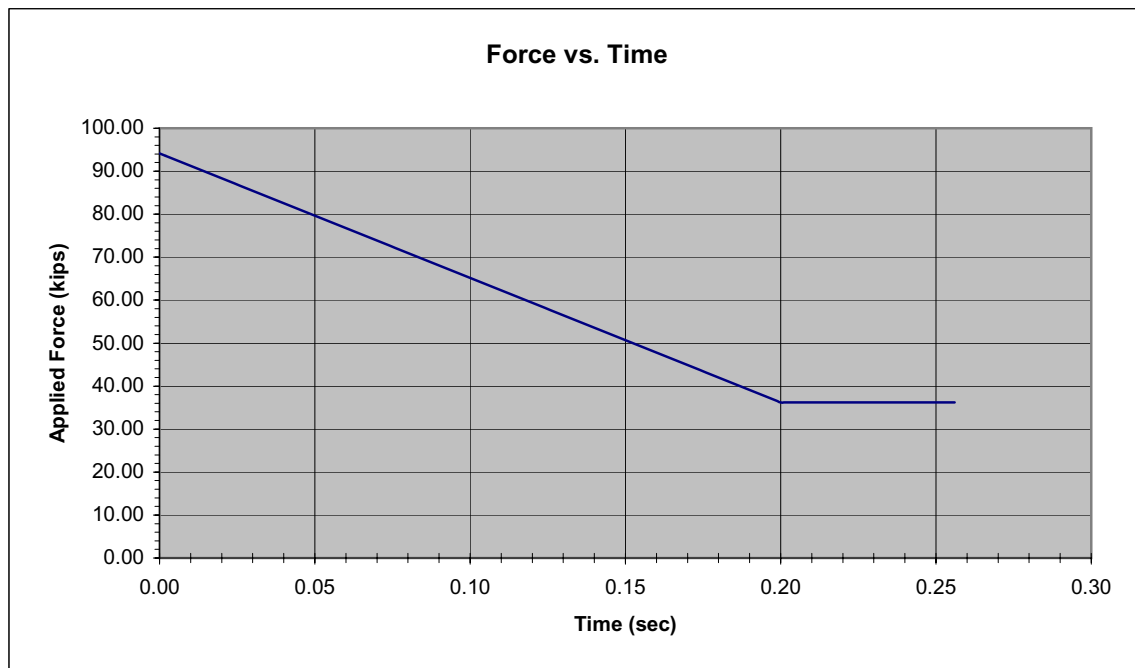
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.618	36.23	1.764	-36.988	-442.7635	77.78	34.11
0.620	36.23	1.690	-37.855	-423.9816	76.02	33.43
0.622	36.23	1.613	-38.684	-404.7741	74.21	32.74
0.624	36.23	1.535	-39.474	-385.1605	72.37	32.03
0.626	36.23	1.455	-40.224	-365.1604	70.50	31.31
0.628	36.23	1.374	-40.934	-344.7937	68.59	30.57
0.630	36.23	1.292	-41.603	-324.0811	66.64	29.82
0.632	36.23	1.208	-42.230	-303.0432	64.67	29.06
0.634	36.23	1.123	-42.815	-281.7013	62.66	28.29
0.636	36.23	1.036	-43.356	-260.0766	60.63	27.51
0.638	36.23	0.949	-43.855	-238.1909	58.58	26.72
0.640	36.23	0.861	-44.309	-216.0662	56.50	25.92
0.642	36.23	0.772	-44.719	-193.7246	54.41	25.11
0.644	36.23	0.682	-45.084	-171.1887	52.29	24.30
0.646	36.23	0.592	-45.403	-148.4809	50.16	23.48
0.648	36.23	0.501	-45.677	-125.6241	48.02	22.65
0.650	36.23	0.409	-45.906	-102.6413	45.86	21.82
0.652	36.23	0.317	-46.088	-79.5555	43.69	20.99
0.654	36.23	0.225	-46.224	-56.3898	41.52	20.15
0.656	36.23	0.132	-46.313	-33.1675	39.34	19.31
0.658	36.23	0.040	-46.357	-9.9120	37.16	18.47
0.660	36.23	-0.053	-46.353	13.3535	34.97	17.63
0.662	36.23	-0.146	-46.303	36.6056	32.79	16.79
0.664	36.23	-0.238	-46.207	59.8210	30.61	15.95
0.666	36.23	-0.331	-46.064	82.9763	28.44	15.11
0.668	36.23	-0.423	-45.875	106.0484	26.27	14.28
0.670	36.23	-0.514	-45.640	129.0140	24.12	13.45
0.672	36.23	-0.605	-45.359	151.8502	21.97	12.63
0.674	36.23	-0.696	-45.033	174.5340	19.85	11.81
0.676	36.23	-0.785	-44.661	197.0426	17.73	10.99
0.678	36.23	-0.874	-44.245	219.3535	15.64	10.19
0.680	36.23	-0.962	-43.784	241.4442	13.57	9.39
0.682	36.23	-1.049	-43.279	263.2927	11.52	8.60
0.684	36.23	-1.135	-42.731	284.8769	9.49	7.82
0.686	36.23	-1.220	-42.140	306.1752	7.49	7.05
0.688	36.23	-1.304	-41.507	327.1663	5.52	6.29
0.690	36.23	-1.386	-40.832	347.8291	3.58	5.55
0.692	36.23	-1.467	-40.116	368.1427	1.68	4.81
0.694	36.23	-1.547	-39.359	388.0870	-0.20	4.09
0.696	36.23	-1.625	-38.564	407.6417	-2.03	3.38
0.698	36.23	-1.701	-37.729	426.7874	-3.83	2.69
0.700	36.23	-1.775	-36.857	445.5048	-5.58	2.02
0.702	36.23	-1.848	-35.948	463.7751	-7.30	1.36
0.704	36.23	-1.919	-35.002	481.5799	-8.97	0.71
0.706	36.23	-1.988	-34.022	498.9015	-10.60	0.09
0.708	36.23	-2.055	-33.007	515.7224	-12.17	-0.52
0.710	36.23	-2.120	-31.959	532.0258	-13.70	-1.11
0.712	36.23	-2.183	-30.880	547.7952	-15.18	-1.68
0.714	36.23	-2.244	-29.769	563.0149	-16.61	-2.23
0.716	36.23	-2.302	-28.628	577.6696	-17.99	-2.76
0.718	36.23	-2.358	-27.459	591.7446	-19.31	-3.27
0.720	36.23	-2.412	-26.262	605.2257	-20.57	-3.76
0.722	36.23	-2.463	-25.038	618.0995	-21.78	-4.22
0.724	36.23	-2.512	-23.790	630.3529	-22.93	-4.66
0.726	36.23	-2.558	-22.518	641.9738	-24.02	-5.08
0.728	36.23	-2.602	-21.223	652.9504	-25.05	-5.48
0.730	36.23	-2.643	-19.906	663.2718	-26.02	-5.85
0.732	36.23	-2.682	-18.570	672.9275	-26.93	-6.20
0.734	36.23	-2.718	-17.215	681.9079	-27.77	-6.53
0.736	36.23	-2.751	-15.843	690.2040	-28.55	-6.83
0.738	36.23	-2.781	-14.455	697.8074	-29.26	-7.10

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.740	36.23	-2.808	-13.053	704.7105	-29.91	-7.35
0.742	36.23	-2.833	-11.637	710.9065	-30.49	-7.57
0.744	36.23	-2.855	-10.210	716.3890	-31.01	-7.77
0.746	36.23	-2.874	-8.772	721.1525	-31.45	-7.94
0.748	36.23	-2.890	-7.326	725.1924	-31.83	-8.09
0.750	36.23	-2.903	-5.872	728.5045	-32.14	-8.21
0.752	36.23	-2.914	-4.413	731.0855	-32.39	-8.30
0.754	36.23	-2.921	-2.949	732.9328	-32.56	-8.37
0.756	36.23	-2.925	-1.482	734.0446	-32.66	-8.41
0.758	36.23	-2.927	-0.013	734.4197	-32.70	-8.42
0.760	36.23	-2.925	1.455	734.0578	-32.67	-8.41
0.762	36.23	-2.921	2.922	732.9593	-32.56	-8.37
0.764	36.23	-2.914	4.386	731.1252	-32.39	-8.30
0.766	36.23	-2.903	5.846	728.5573	-32.15	-8.21
0.768	36.23	-2.890	7.300	725.2584	-31.84	-8.09
0.770	36.23	-2.874	8.746	721.2316	-31.46	-7.95
0.772	36.23	-2.855	10.184	716.4810	-31.02	-7.78
0.774	36.23	-2.834	11.612	711.0113	-30.50	-7.58
0.776	36.23	-2.809	13.027	704.8282	-29.92	-7.35
0.778	36.23	-2.781	14.430	697.9377	-29.28	-7.11
0.780	36.23	-2.751	15.818	690.3468	-28.56	-6.83
0.782	36.23	-2.718	17.191	682.0631	-27.79	-6.53
0.784	36.23	-2.682	18.546	673.0949	-26.94	-6.21
0.786	36.23	-2.644	19.883	663.4512	-26.04	-5.86
0.788	36.23	-2.603	21.199	653.1418	-25.07	-5.49
0.790	36.23	-2.559	22.494	642.1768	-24.04	-5.09
0.792	36.23	-2.513	23.767	630.5675	-22.95	-4.67
0.794	36.23	-2.464	25.016	618.3253	-21.80	-4.23
0.796	36.23	-2.413	26.240	605.4626	-20.60	-3.76
0.798	36.23	-2.359	27.437	591.9922	-19.33	-3.28
0.800	36.23	-2.303	28.607	577.9278	-18.01	-2.77
0.802	36.23	-2.245	29.748	563.2834	-16.64	-2.24
0.804	36.23	-2.184	30.860	548.0737	-15.21	-1.69
0.806	36.23	-2.121	31.940	532.3140	-13.73	-1.12
0.808	36.23	-2.056	32.989	516.0201	-12.20	-0.53
0.810	36.23	-1.989	34.004	499.2084	-10.62	0.08
0.812	36.23	-1.920	34.985	481.8957	-9.00	0.70
0.814	36.23	-1.850	35.931	464.0993	-7.33	1.34
0.816	36.23	-1.777	36.841	445.8372	-5.62	2.00
0.818	36.23	-1.702	37.714	427.1277	-3.86	2.68
0.820	36.23	-1.626	38.549	407.9896	-2.06	3.37
0.822	36.23	-1.548	39.345	388.4420	-0.23	4.08
0.824	36.23	-1.469	40.102	368.5046	1.64	4.80
0.826	36.23	-1.388	40.819	348.1974	3.55	5.53
0.828	36.23	-1.305	41.495	327.5407	5.49	6.28
0.830	36.23	-1.222	42.129	306.5554	7.46	7.04
0.832	36.23	-1.137	42.721	285.2624	9.45	7.81
0.834	36.23	-1.051	43.270	263.6831	11.48	8.59
0.836	36.23	-0.964	43.775	241.8392	13.53	9.37
0.838	36.23	-0.876	44.237	219.7526	15.60	10.17
0.840	36.23	-0.787	44.654	197.4455	17.70	10.98
0.842	36.23	-0.697	45.026	174.9402	19.81	11.79
0.844	36.23	-0.607	45.353	152.2594	21.94	12.61
0.846	36.23	-0.516	45.635	129.4258	24.08	13.44
0.848	36.23	-0.424	45.871	106.4623	26.23	14.27
0.850	36.23	-0.332	46.061	83.3919	28.40	15.10
0.852	36.23	-0.240	46.205	60.2379	30.57	15.94
0.854	36.23	-0.148	46.302	37.0234	32.75	16.78
0.856	36.23	-0.055	46.353	13.7717	34.93	17.62
0.858	36.23	0.038	46.357	-9.4937	37.12	18.46
0.860	36.23	0.131	46.315	-32.7497	39.30	19.30

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.862	36.23	0.223	46.226	-55.9727	41.48	20.14
0.864	36.23	0.315	46.091	-79.1396	43.65	20.97
0.866	36.23	0.407	45.909	-102.2271	45.82	21.81
0.868	36.23	0.499	45.682	-125.2120	47.98	22.64
0.870	36.23	0.590	45.409	-148.0712	50.12	23.46
0.872	36.23	0.681	45.090	-170.7819	52.25	24.28
0.874	36.23	0.770	44.726	-193.3211	54.37	25.10
0.876	36.23	0.859	44.317	-215.6664	56.47	25.91
0.878	36.23	0.948	43.863	-237.7952	58.54	26.71
0.880	36.23	1.035	43.366	-259.6854	60.60	27.50
0.882	36.23	1.121	42.825	-281.3149	62.63	28.28
0.884	36.23	1.206	42.241	-302.6622	64.63	29.05
0.886	36.23	1.290	41.614	-323.7057	66.61	29.81
0.888	36.23	1.373	40.946	-344.4244	68.55	30.56
0.890	36.23	1.454	40.237	-364.7974	70.46	31.29
0.892	36.23	1.534	39.488	-384.8043	72.34	32.02
0.894	36.23	1.612	38.698	-404.4251	74.18	32.73
0.896	36.23	1.688	37.870	-423.6400	75.98	33.42
0.898	36.23	1.763	37.004	-442.4297	77.75	34.10
0.900	36.23	1.836	36.101	-460.7755	79.47	34.76
0.902	36.23	1.908	35.162	-478.6588	81.15	35.41
0.904	36.23	1.977	34.187	-496.0618	82.78	36.04
0.906	36.23	2.044	33.178	-512.9669	84.37	36.65
0.908	36.23	2.110	32.135	-529.3573	85.91	37.24
0.910	36.23	2.173	31.061	-545.2164	87.39	37.81
0.912	36.23	2.234	29.955	-560.5284	88.83	38.37
0.914	36.23	2.293	28.819	-575.2779	90.22	38.90
0.916	36.23	2.349	27.655	-589.4500	91.55	39.41
0.918	36.23	2.403	26.462	-603.0306	92.82	39.90
0.920	36.23	2.455	25.243	-616.0061	94.04	40.37
0.922	36.23	2.504	23.999	-628.3633	95.20	40.82
0.924	36.23	2.551	22.730	-640.0900	96.30	41.24
0.926	36.23	2.595	21.439	-651.1743	97.34	41.64
0.928	36.23	2.637	20.126	-661.6051	98.32	42.02
0.930	36.23	2.676	18.793	-671.3719	99.23	42.37
0.932	36.23	2.712	17.441	-680.4650	100.09	42.70
0.934	36.23	2.745	16.072	-688.8752	100.88	43.00
0.936	36.23	2.776	14.687	-696.5941	101.60	43.28
0.938	36.23	2.804	13.286	-703.6139	102.26	43.54
0.940	36.23	2.829	11.873	-709.9277	102.85	43.76
0.942	36.23	2.852	10.447	-715.5289	103.38	43.97
0.944	36.23	2.871	9.011	-720.4121	103.84	44.14
0.946	36.23	2.888	7.566	-724.5724	104.23	44.29
0.948	36.23	2.901	6.114	-728.0055	104.55	44.42
0.950	36.23	2.912	4.655	-730.7080	104.80	44.52
0.952	36.23	2.920	3.192	-732.6772	104.99	44.59
0.954	36.23	2.925	1.725	-733.9111	105.10	44.63
0.956	36.23	2.927	0.257	-734.4085	105.15	44.65
0.958	36.23	2.926	-1.212	-734.1689	105.13	44.64
0.960	36.23	2.922	-2.679	-733.1925	105.04	44.60
0.962	36.23	2.915	-4.144	-731.4803	104.88	44.54
0.964	36.23	2.905	-5.604	-729.0341	104.65	44.45
0.966	36.23	2.893	-7.059	-725.8562	104.35	44.34
0.968	36.23	2.877	-8.507	-721.9499	103.98	44.20
0.970	36.23	2.859	-9.946	-717.3190	103.55	44.03
0.972	36.23	2.837	-11.375	-711.9684	103.04	43.84
0.974	36.23	2.813	-12.793	-705.9032	102.48	43.62
0.976	36.23	2.786	-14.198	-699.1296	101.84	43.37
0.978	36.23	2.756	-15.589	-691.6544	101.14	43.10
0.980	36.23	2.724	-16.964	-683.4851	100.37	42.81
0.982	36.23	2.689	-18.322	-674.6299	99.54	42.49

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.984	36.23	2.651	-19.662	-665.0976	98.65	42.14
0.986	36.23	2.610	-20.982	-654.8979	97.69	41.78
0.988	36.23	2.567	-22.281	-644.0410	96.67	41.38
0.990	36.23	2.521	-23.558	-632.5377	95.59	40.97
0.992	36.23	2.472	-24.811	-620.3997	94.45	40.53
0.994	36.23	2.422	-26.039	-607.6391	93.25	40.07
0.996	36.23	2.368	-27.241	-594.2687	92.00	39.59
0.998	36.23	2.313	-28.415	-580.3018	90.69	39.08
1.000	36.23	2.255	-29.561	-565.7527	89.32	38.55
1.002	36.23	2.194	-30.678	-550.6357	87.90	38.01
1.004	36.23	2.132	-31.763	-534.9662	86.43	37.44
1.006	36.23	2.067	-32.817	-518.7598	84.91	36.86
1.008	36.23	2.001	-33.838	-502.0329	83.34	36.25
1.010	36.23	1.932	-34.825	-484.8021	81.72	35.63
1.012	36.23	1.861	-35.776	-467.0847	80.06	34.99
1.014	36.23	1.789	-36.692	-448.8987	78.36	34.33
1.016	36.23	1.715	-37.572	-430.2621	76.61	33.66
1.018	36.23	1.639	-38.413	-411.1938	74.82	32.97
1.020	36.23	1.561	-39.216	-391.7128	72.99	32.27
1.022	36.23	1.482	-39.979	-371.8387	71.12	31.55
1.024	36.23	1.401	-40.703	-351.5914	69.22	30.82
1.026	36.23	1.319	-41.385	-330.9913	67.29	30.07
1.028	36.23	1.236	-42.027	-310.0590	65.33	29.32
1.030	36.23	1.151	-42.625	-288.8156	63.33	28.55
1.032	36.23	1.065	-43.182	-267.2824	61.31	27.77
1.034	36.23	0.978	-43.694	-245.4809	59.26	26.98
1.036	36.23	0.890	-44.163	-223.4330	57.20	26.19
1.038	36.23	0.802	-44.588	-201.1609	55.11	25.38
1.040	36.23	0.712	-44.968	-178.6870	53.00	24.57
1.042	36.23	0.622	-45.302	-156.0337	50.87	23.75
1.044	36.23	0.531	-45.592	-133.2239	48.73	22.93
1.046	36.23	0.439	-45.835	-110.2803	46.58	22.10
1.048	36.23	0.348	-46.033	-87.2261	44.41	21.26
1.050	36.23	0.255	-46.184	-64.0843	42.24	20.43
1.052	36.23	0.163	-46.289	-40.8783	40.06	19.59
1.054	36.23	0.070	-46.347	-17.6312	37.88	18.75
1.056	36.23	-0.022	-46.359	5.6336	35.70	17.91
1.058	36.23	-0.115	-46.325	28.8928	33.51	17.07
1.060	36.23	-0.208	-46.244	52.1229	31.33	16.23
1.062	36.23	-0.300	-46.116	75.3007	29.16	15.39
1.064	36.23	-0.392	-45.943	98.4030	26.99	14.56
1.066	36.23	-0.484	-45.723	121.4065	24.83	13.73
1.068	36.23	-0.575	-45.457	144.2882	22.68	12.90
1.070	36.23	-0.666	-45.146	167.0251	20.55	12.08
1.072	36.23	-0.756	-44.789	189.5943	18.43	11.26
1.074	36.23	-0.845	-44.388	211.9733	16.33	10.45
1.076	36.23	-0.933	-43.942	234.1396	14.25	9.65
1.078	36.23	-1.020	-43.451	256.0709	12.19	8.86
1.080	36.23	-1.107	-42.918	277.7452	10.16	8.08





Saudconsult

Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

PRECAST ROOF PANEL DOUBLE TEE BEAM CROSS SECTION ; SS # 2

Saudconsult

Job No. : BI-3039

Project : REPLACEMENT OF SUBSTATION 472, 321 & F07, ABU ALI

Subject : CROSS SECTION PROPERTIES, SUBSTATION # 2

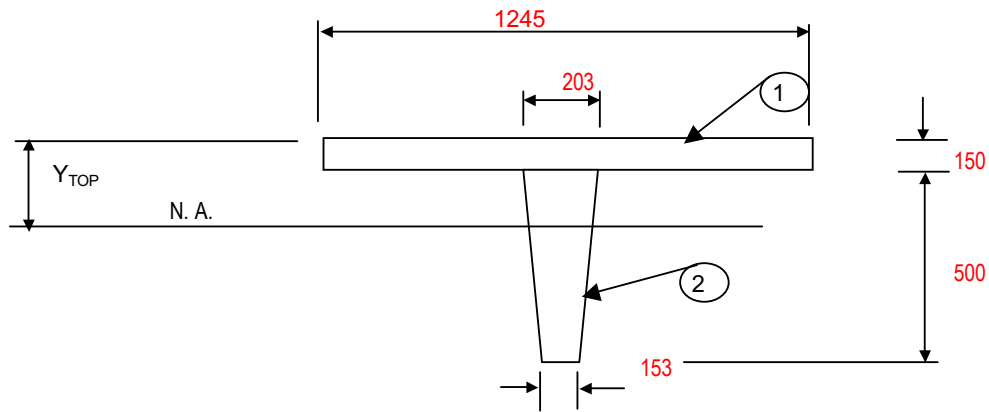
Sheet no. : of 3

Date : 6/6/2004

Design by : MGG

Check'd by: SAK

PRECAST ROOF PANEL DOUBLE TEE BEAM CROSS SECTION



ELEMENT	AREA (A)	Y	A x Y	Io	D	A x D ²
1	186750	75.00	14006250	3.5E+08	101.118	1909496864
2	89000	388.30	34558333.33	1.8E+09	212.178	4006725161
TOTAL	275750		48564583.33	2.2E+09		5916222025

$$Y_{TOP} = \frac{48564583.33}{275750}$$

$$= 176.1181626 \text{ MM}$$

$$I_x = 8.1084E+09 \text{ MM}^4$$

$$I_x = 0.0081 \text{ M}^4$$

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : DOUBLE ROOF TEE BEAM PROPERTIES, SS #2

Sheet no. : of
 Date : 3-Jun-04
 Design by : MGG
 Check'd by : SAK

DESIGN LOADS (UNFACTORED) :

Roof Dead Load = 8.210 KN/m²
 = 171.47 psf

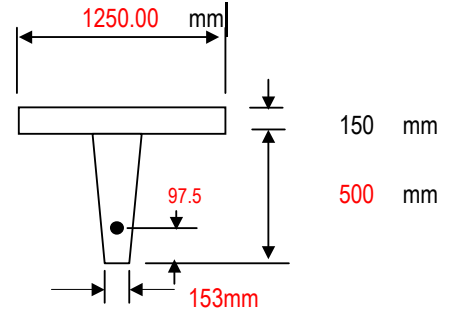
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
 Reinf., f_y = 414 Mpa

PANEL DATA : (CONSIDER HALF WIDTH)

Panel Width, width = 1250 mm (49.21 in)
 Flange Thickness = 150 mm (5.91 in)
 Web Depth = 500 mm (19.69 in)
 Total Depth, d_t = 650 mm (25.59 in)
 Panel Span = 10220 mm (402.36 in)

CONFIGURATION :



CHECK DYNAMIC REQUIRED RESISTANCE:

Design overpressure, P_s = 1.96 psi (13.547 kPa)
 Blast time duration, T_d = 200 msec

$f'_{dc} = (SIF) \times (DIF) \times f'_c$ (for bending) = 5954.3 psi (41 Mpa.)
 $f'_{dc} = (SIF) \times (DIF) \times f'_c$ (for shear) = 5504.0 psi (38 Mpa.)
 $f_{dy} = (SIF) \times (DIF) \times f_y$ = 77276 psi (533 Mpa.)
 Modulus of Elasticity, $E_{dc} = 57000 \times (f'_c)^{1/2}$ = 4032041 psi (27801 Mpa.)

Positive :

Area of Bar ($\emptyset$), 25 mm $\emptyset$ A_b = 0.7609 in² (491 mm²)
 No. of Web Main Bar = 4.0000 in (102 mm)
 Total Area of Web Rebar PER panel, A_s = 3.0434 in² (1963 mm²)
 Effective Depth, $d_e = d_t - 97.5$ = 21.7520 in (553 mm)
 Steel ratio, $p = A_s / b \times d$ = 0.0028 > $200 / f_{dy} = 0.003$
 Stress block, $a = A_s \times f_{dy} / 0.85 \times f'_{dc} \times b$ = 0.9442 in (24 mm)
 Modular ratio, $n = E_s / E_{dc}$ = 7.1940
 Max. Moment, $M_p = M_{pc} = M_{ps} = A_s \times f_{dy} [d - a / 2]$ = 5004672.4 lb-in (565424 KN-m)
 Shear, $V_n = 2 \times b \times d \times (f'_{dc})^{0.5}$ = 158834.1 lb (706 KN)
 Resistance at cracked moment, $R_u = 8 \times M_p / L$ = 99506 lb (442.60 KN)
 99.506 kips > 38.91 $\therefore$ O.K!

Saudconsult

Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : DOUBLE ROOF TEE BEAM PROPERTIES, SS #2

Sheet no. : of
 Date : 3-Jun-04
 Design by : MGG
 Check'd by : SAK

Reversal :

Area of Bar (Ø),	20 mm Ø	$Ab =$	0.4869 in ²	(314 mm ²)
No. of Flange Rebar		$=$	5.0000	
Total Area of Flange Rebar PER panel,		$=$	2.4347 in ²	(13 mm ²)
Effective Depth, $de = dt - tf/2$		$=$	22.6378 in	(575 mm)
Steel ratio,	$p = As / b \times d$	$=$	0.0179	> 200 / fdy = 0.003
Stress block,	$a = As \times fdy / 0.85 \times fdc \times b$	$=$	6.1715 in	(157 mm)
Modular ratio,	$n = Es / Edc$	$=$	7.1940	
Max. Moment, $Mp = Mpc = Mps = As \times fdy [d - a / 2]$		$=$	3678659.5 lb-in	(415612 KN-m)
Shear,	$Vn = 2 \times b \times d \times (fdc)^{0.5}$	$=$	20233.0 lb	(90 KN)
Resistance at cracked moment,	$Ru = 8 \times Mp / L$	$=$	73141 lb	(325.33 KN)
			73.141 kips	

Cracked Moment of Inertia:

	$n \times As =$	21.8944 in ²	(14125 mm ²)
$C = -n \times As + [n \times As \times (n \times As + 2 \times b \times d)]^{0.5} / b$	$=$	3.9769 in ²	(2566 mm ²)
$Icr = b \times C^3 / 3 + n \times As \times (d - C)^2$	$=$	7949.39 in ⁴	(3308786286 mm ⁴)

Moment of Inertia,	$Ig =$	19480.40 in ⁴	(8108352998 mm ⁴)
Effective Moment of Inertia,	$0.5 Ig =$	9740.20 in ⁴	(4054176499 mm ⁴)
Average Moment of Inertia,	$Ia = (Ig + Icr) / 2 =$	8844.794 in ⁴	(3681481393 mm ⁴)
Effective Stiffness,	$K = 384 \times Edc \times Ia / 5 L^3 =$	42.0 kips / in	
Yield Deflection,	$Ye = Ru / K =$	2.36661 in	
Total Roof Mass = Roof DL x width x roof span / gravity acc.	$=$	0.06108 kip-sec ² / in	
Additional Mass =	$=$	0.00000 kip-sec ² / in	
Total Mass, M	$=$	0.06108 kip-sec ² / in	
Average Stiffness, mass factor	$K_{LM} =$	0.72000	FROM ASCE TABLE 6.1
Equivalent Mass,	$Me = K_{LM} \times Beam Mass =$	0.04398	kip-sec ² / in
Period of Vibration,	$tn = 2 \times \pi \times (Me / K)^{0.5} =$	0.20321 sec	
Time increment , $tn/10$	$\Delta t =$	0.02032 sec	; use $\Delta t =$ 0.00200 sec
td / tn	$=$	0.98419	

Initial Load and Deflection based on Gravity loads:

$R_{ini} = \text{Roof DL} \times \text{Width} \times \text{Span}$	$R_{ini} =$	23.5786 kips	(104.88 KN)
Deflection = R_{ini} / K	$=$	0.5608 in	(14 mm)
Rotation ; θ	$=$	0.2370	degrees

Total Initial Net Loads on member

$F(t) = Ps + R_{ini}$	$F(t) =$	62.4842 kips	(277.93 KN)
-----------------------	----------	--------------	---------------

NUMERICAL INTEGRATION METHOD

INPUT

Stiffness	K	=	42.046 kips / in
Mass	M	=	0.0440 kip-sec ² / in
Tension resistance	Rut	=	99.51 kips
Compression resistance	Ruc	=	0 kips
Damping co-efficient	C	=	0
Forcing function	F(t)	=	62.48 kips
Time start	Ts	=	0 sec
Time end	Te	=	0.200 sec
Time increment	Δt	=	0.002 sec
Reaction resistance (R) coeff.	a	=	0.385
Reaction force (f) coeff.	b	=	0.115
Key ₀	Key ₀	=	0

RESULTS

Blast Load History	
load (kips)	time (sec)
62.48	0
23.57858	0.2000

Length	402.362 in
y _{max}	1.428 in
r _{max}	83.629 kips
μ	0.603
θ	0.407 degrees
Rut/Fo	1.592
θ = < 2 degrees	
low response	

Time (sec)	force (kips)	y (in)	v (in/sec)	a (in/sec ²)	resistance (kips)	reaction (kips)
0.000	62.48	0.000	0.000	884.6042	23.58	16.26
0.002	62.10	0.002	1.759	874.0735	23.65	16.25
0.004	61.71	0.007	3.493	860.2024	23.87	16.29
0.006	61.32	0.016	5.196	843.0441	24.24	16.38
0.008	60.93	0.028	6.862	822.6639	24.75	16.53
0.010	60.54	0.043	8.484	799.1399	25.39	16.74
0.012	60.15	0.062	10.055	772.5619	26.17	16.99
0.014	59.76	0.083	11.571	743.0316	27.08	17.30
0.016	59.37	0.108	13.025	710.6616	28.12	17.65
0.018	58.98	0.135	14.411	675.5758	29.27	18.05
0.020	58.59	0.166	15.724	637.9083	30.54	18.50
0.022	58.20	0.198	16.960	597.8029	31.91	18.98
0.024	57.82	0.233	18.113	555.4130	33.39	19.50
0.026	57.43	0.271	19.180	510.9005	34.96	20.06
0.028	57.04	0.310	20.155	464.4356	36.61	20.65
0.030	56.65	0.351	21.036	416.1958	38.34	21.28
0.032	56.26	0.394	21.818	366.3655	40.15	21.93
0.034	55.87	0.438	22.500	315.1350	42.01	22.60
0.036	55.48	0.484	23.078	262.7003	43.93	23.29
0.038	55.09	0.531	23.549	209.2616	45.89	24.00
0.040	54.70	0.578	23.914	155.0233	47.88	24.73
0.042	54.31	0.626	24.169	100.1925	49.91	25.46
0.044	53.92	0.675	24.314	44.9788	51.95	26.20
0.046	53.54	0.723	24.349	-10.4068	53.99	26.94
0.048	53.15	0.772	24.273	-65.7526	56.04	27.69
0.050	52.76	0.820	24.086	-120.8472	58.07	28.43
0.052	52.37	0.868	23.790	-175.4799	60.09	29.16
0.054	51.98	0.915	23.385	-229.4420	62.07	29.87
0.056	51.59	0.962	22.873	-282.5273	64.02	30.58
0.058	51.20	1.007	22.256	-334.5328	65.91	31.27
0.060	50.81	1.051	21.536	-385.2600	67.76	31.93
0.062	50.42	1.093	20.716	-434.5148	69.53	32.57

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.064	50.03	1.134	19.800	-482.1091	71.24	33.18
0.066	49.65	1.172	18.790	-527.8610	72.86	33.76
0.068	49.26	1.209	17.690	-571.5956	74.40	34.31
0.070	48.87	1.243	16.505	-613.1458	75.83	34.82
0.072	48.48	1.275	15.240	-652.3528	77.17	35.29
0.074	48.09	1.304	13.898	-689.0669	78.39	35.71
0.076	47.70	1.330	12.486	-723.1475	79.50	36.09
0.078	47.31	1.354	11.009	-754.4647	80.49	36.43
0.080	46.92	1.374	9.471	-782.8986	81.35	36.72
0.082	46.53	1.391	7.880	-808.3405	82.08	36.95
0.084	46.14	1.406	6.241	-830.6934	82.68	37.14
0.086	45.75	1.416	4.560	-849.8716	83.13	37.27
0.088	45.37	1.424	2.845	-865.8021	83.44	37.34
0.090	44.98	1.428	1.100	-878.4238	83.61	37.36
0.092	44.59	1.428	-0.666	-887.6885	83.63	37.32
0.094	44.20	1.425	-2.447	-893.5608	83.50	37.23
0.096	43.81	1.418	-4.236	-896.0184	83.22	37.08
0.098	43.42	1.408	-6.028	-895.0517	82.79	36.87
0.100	43.03	1.394	-7.813	-890.6646	82.20	36.60
0.102	42.64	1.377	-9.587	-882.8736	81.47	36.27
0.104	42.25	1.356	-11.341	-871.7088	80.59	35.89
0.106	41.86	1.332	-13.070	-857.2126	79.57	35.45
0.108	41.48	1.304	-14.767	-839.4405	78.39	34.95
0.110	41.09	1.273	-16.425	-818.4604	77.08	34.40
0.112	40.70	1.238	-18.038	-794.3525	75.63	33.80
0.114	40.31	1.200	-19.599	-767.2089	74.05	33.14
0.116	39.92	1.160	-21.104	-737.1334	72.34	32.44
0.118	39.53	1.116	-22.545	-704.2409	70.50	31.69
0.120	39.14	1.070	-23.918	-668.6570	68.55	30.89
0.122	38.75	1.020	-25.217	-630.5178	66.48	30.05
0.124	38.36	0.969	-26.437	-589.9691	64.31	29.17
0.126	37.97	0.915	-27.575	-547.1657	62.04	28.25
0.128	37.58	0.858	-28.624	-502.2713	59.67	27.30
0.130	37.20	0.800	-29.582	-455.4575	57.23	26.31
0.132	36.81	0.740	-30.444	-406.9030	54.70	25.29
0.134	36.42	0.679	-31.208	-356.7936	52.11	24.25
0.136	36.03	0.615	-31.870	-305.3206	49.46	23.18
0.138	35.64	0.551	-32.428	-252.6809	46.75	22.10
0.140	35.25	0.486	-32.880	-199.0755	44.01	21.00
0.142	34.86	0.420	-33.223	-144.7093	41.23	19.88
0.144	34.47	0.353	-33.458	-89.7901	38.42	18.76
0.146	34.08	0.286	-33.582	-34.5277	35.60	17.63
0.148	33.69	0.219	-33.596	20.8666	32.78	16.49
0.150	33.30	0.152	-33.499	76.1811	29.95	15.36
0.152	32.92	0.085	-33.292	131.2045	27.15	14.24
0.154	32.53	0.019	-32.975	185.7265	24.36	13.12
0.156	32.14	-0.047	-32.549	239.5388	21.60	12.01
0.158	31.75	-0.112	-32.017	292.4356	18.89	10.92
0.160	31.36	-0.175	-31.381	344.2149	16.22	9.85
0.162	30.97	-0.237	-30.642	394.6787	13.61	8.80
0.164	30.58	-0.297	-29.804	443.6342	11.07	7.78
0.166	30.19	-0.356	-28.869	490.8944	8.60	6.78

*

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.168	29.80	-0.413	-27.842	536.2785	6.22	5.82
0.170	29.41	-0.467	-26.726	579.6132	3.92	4.89
0.172	29.03	-0.520	-25.526	620.7329	1.73	4.00
0.174	28.64	-0.570	-24.245	659.4804	-0.37	3.15
0.176	28.25	-0.617	-22.890	695.7076	-2.35	2.34
0.178	27.86	-0.661	-21.465	729.2762	-4.22	1.58
0.180	27.47	-0.703	-19.976	760.0577	-5.96	0.86
0.182	27.08	-0.741	-18.428	787.9347	-7.57	0.20
0.184	26.69	-0.776	-16.827	812.8005	-9.06	-0.42
0.186	26.30	-0.808	-15.180	834.5601	-10.40	-0.98
0.188	25.91	-0.837	-13.492	853.1303	-11.61	-1.49
0.190	25.52	-0.862	-11.771	868.4403	-12.67	-1.94
0.192	25.13	-0.884	-10.022	880.4314	-13.59	-2.34
0.194	24.75	-0.902	-8.252	889.0579	-14.36	-2.68
0.196	24.36	-0.917	-6.469	894.2869	-14.97	-2.96
0.198	23.97	-0.928	-4.678	896.0982	-15.44	-3.19
0.200	23.58	-0.936	-2.888	894.4850	-15.76	-3.36
0.202	23.58	-0.940	-1.095	898.2938	-15.93	-3.42
0.204	23.58	-0.940	0.702	898.6698	-15.95	-3.43
0.206	23.58	-0.937	2.496	895.6115	-15.81	-3.38
0.208	23.58	-0.930	4.281	889.1304	-15.53	-3.27
0.210	23.58	-0.920	6.049	879.2515	-15.09	-3.10
0.212	23.58	-0.906	7.795	866.0125	-14.51	-2.87
0.214	23.58	-0.889	9.510	849.4639	-13.78	-2.59
0.216	23.58	-0.868	11.189	829.6691	-12.91	-2.26
0.218	23.58	-0.844	12.826	806.7036	-11.90	-1.87
0.220	23.58	-0.817	14.413	780.6552	-10.76	-1.43
0.222	23.58	-0.786	15.945	751.6235	-9.48	-0.94
0.224	23.58	-0.753	17.417	719.7194	-8.08	-0.40
0.226	23.58	-0.717	18.821	685.0648	-6.55	0.19
0.228	23.58	-0.678	20.154	647.7922	-4.91	0.82
0.230	23.58	-0.636	21.410	608.0440	-3.16	1.49
0.232	23.58	-0.592	22.584	565.9722	-1.31	2.21
0.234	23.58	-0.546	23.672	521.7374	0.63	2.95
0.236	23.58	-0.497	24.669	475.5087	2.67	3.74
0.238	23.58	-0.447	25.572	427.4629	4.78	4.55
0.240	23.58	-0.395	26.377	377.7835	6.96	5.39
0.242	23.58	-0.342	27.082	326.6603	9.21	6.26
0.244	23.58	-0.287	27.683	274.2888	11.52	7.14
0.246	23.58	-0.231	28.178	220.8691	13.86	8.05
0.248	23.58	-0.174	28.565	166.6053	16.25	8.97
0.250	23.58	-0.117	28.844	111.7048	18.67	9.90
0.252	23.58	-0.059	29.012	56.3775	21.10	10.83
0.254	23.58	-0.001	29.069	0.8346	23.54	11.78
0.256	23.58	0.057	29.015	-54.7114	25.98	12.72
0.258	23.58	0.115	28.850	-110.0483	28.42	13.65
0.260	23.58	0.173	28.575	-164.9647	30.83	14.58
0.262	23.58	0.229	28.191	-219.2506	33.22	15.50
0.264	23.58	0.285	27.699	-272.6987	35.57	16.41
0.266	23.58	0.340	27.101	-325.1046	37.88	17.29
0.268	23.58	0.394	26.400	-376.2681	40.13	18.16
0.270	23.58	0.446	25.598	-425.9937	42.31	19.00

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.272	23.58	0.496	24.697	-474.0913	44.43	19.82
0.274	23.58	0.544	23.703	-520.3772	46.47	20.60
0.276	23.58	0.591	22.618	-564.6743	48.41	21.35
0.278	23.58	0.635	21.446	-606.8135	50.27	22.06
0.280	23.58	0.676	20.193	-646.6338	52.02	22.74
0.282	23.58	0.715	18.862	-683.9828	53.66	23.37
0.284	23.58	0.752	17.460	-718.7180	55.19	23.96
0.286	23.58	0.785	15.990	-750.7066	56.60	24.50
0.288	23.58	0.816	14.460	-779.8262	57.88	24.99
0.290	23.58	0.843	12.874	-805.9657	59.03	25.44
0.292	23.58	0.867	11.239	-829.0251	60.04	25.83
0.294	23.58	0.888	9.561	-848.9164	60.91	26.16
0.296	23.58	0.905	7.847	-865.5634	61.65	26.45
0.298	23.58	0.919	6.102	-878.9026	62.23	26.67
0.300	23.58	0.930	4.334	-888.8831	62.67	26.84
0.302	23.58	0.937	2.550	-895.4666	62.96	26.95
0.304	23.58	0.940	0.756	-898.6280	63.10	27.01
0.306	23.58	0.940	-1.041	-898.3552	63.09	27.00
0.308	23.58	0.936	-2.834	-894.6493	62.93	26.94
0.310	23.58	0.928	-4.616	-887.5244	62.61	26.82
0.312	23.58	0.917	-6.381	-877.0077	62.15	26.64
0.314	23.58	0.903	-8.121	-863.1396	61.54	26.40
0.316	23.58	0.885	-9.830	-845.9728	60.79	26.11
0.318	23.58	0.864	-11.502	-825.5731	59.89	25.77
0.320	23.58	0.839	-13.129	-802.0184	58.85	25.37
0.322	23.58	0.811	-14.707	-775.3988	57.68	24.92
0.324	23.58	0.780	-16.228	-745.8159	56.38	24.42
0.326	23.58	0.746	-17.687	-713.3828	54.95	23.87
0.328	23.58	0.709	-19.079	-678.2234	53.41	23.27
0.330	23.58	0.670	-20.397	-640.4722	51.75	22.63
0.332	23.58	0.628	-21.638	-600.2733	49.98	21.95
0.334	23.58	0.583	-22.796	-557.7805	48.11	21.23
0.336	23.58	0.537	-23.867	-513.1560	46.15	20.48
0.338	23.58	0.488	-24.847	-466.5705	44.10	19.69
0.340	23.58	0.437	-25.732	-418.2020	41.97	18.87
0.342	23.58	0.385	-26.518	-368.2352	39.77	18.02
0.344	23.58	0.331	-27.203	-316.8613	37.51	17.15
0.346	23.58	0.276	-27.784	-264.2764	35.20	16.26
0.348	23.58	0.220	-28.259	-210.6815	32.84	15.36
0.350	23.58	0.163	-28.626	-156.2816	30.45	14.44
0.352	23.58	0.106	-28.884	-101.2843	28.03	13.50
0.354	23.58	0.048	-29.031	-45.9001	25.60	12.57
0.356	23.58	-0.010	-29.067	9.6596	23.15	11.63
0.358	23.58	-0.068	-28.992	65.1824	20.71	10.69
0.360	23.58	-0.126	-28.807	120.4561	18.28	9.75
0.362	23.58	-0.183	-28.511	175.2694	15.87	8.82
0.364	23.58	-0.240	-28.106	229.4130	13.49	7.90
0.366	23.58	-0.296	-27.594	282.6798	11.15	7.00
0.368	23.58	-0.350	-26.977	334.8663	8.85	6.12
0.370	23.58	-0.404	-26.256	385.7732	6.61	5.26
0.372	23.58	-0.455	-25.435	435.2057	4.44	4.42
0.374	23.58	-0.505	-24.517	482.9751	2.34	3.61

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.376	23.58	-0.553	-23.505	528.8988	0.32	2.83
0.378	23.58	-0.599	-22.403	572.8012	-1.61	2.09
0.380	23.58	-0.643	-21.216	614.5146	-3.45	1.38
0.382	23.58	-0.684	-19.948	653.8796	-5.18	0.72
0.384	23.58	-0.723	-18.603	690.7458	-6.80	0.09
0.386	23.58	-0.758	-17.187	724.9722	-8.31	-0.49
0.388	23.58	-0.791	-15.706	756.4281	-9.69	-1.02
0.390	23.58	-0.821	-14.164	784.9932	-10.95	-1.50
0.392	23.58	-0.848	-12.569	810.5584	-12.07	-1.94
0.394	23.58	-0.871	-10.925	833.0261	-13.06	-2.32
0.396	23.58	-0.892	-9.240	852.3102	-13.91	-2.64
0.398	23.58	-0.908	-7.519	868.3371	-14.61	-2.91
0.400	23.58	-0.922	-5.770	881.0457	-15.17	-3.13
0.402	23.58	-0.931	-3.998	890.3872	-15.58	-3.29
0.404	23.58	-0.938	-2.212	896.3261	-15.84	-3.39
0.406	23.58	-0.940	-0.417	898.8396	-15.95	-3.43
0.408	23.58	-0.939	1.380	897.9181	-15.91	-3.41
0.410	23.58	-0.935	3.172	893.5651	-15.72	-3.34
0.412	23.58	-0.927	4.951	885.7974	-15.38	-3.21
0.414	23.58	-0.915	6.711	874.6444	-14.89	-3.02
0.416	23.58	-0.900	8.446	860.1490	-14.25	-2.78
0.418	23.58	-0.881	10.149	842.3664	-13.47	-2.47
0.420	23.58	-0.859	11.813	821.3646	-12.55	-2.12
0.422	23.58	-0.834	13.431	797.2240	-11.48	-1.71
0.424	23.58	-0.805	14.998	770.0367	-10.29	-1.25
0.426	23.58	-0.774	16.508	739.9066	-8.96	-0.74
0.428	23.58	-0.739	17.955	706.9489	-7.51	-0.18
0.430	23.58	-0.702	19.333	671.2896	-5.95	0.42
0.432	23.58	-0.662	20.638	633.0648	-4.26	1.07
0.434	23.58	-0.620	21.863	592.4208	-2.48	1.76
0.436	23.58	-0.575	23.005	549.5128	-0.59	2.48
0.438	23.58	-0.528	24.059	504.5048	1.39	3.25
0.440	23.58	-0.479	25.021	457.5687	3.45	4.04
0.442	23.58	-0.428	25.888	408.8841	5.60	4.87
0.444	23.58	-0.375	26.655	358.6368	7.81	5.72
0.446	23.58	-0.321	27.321	307.0190	10.08	6.59
0.448	23.58	-0.266	27.882	254.2279	12.40	7.48
0.450	23.58	-0.210	28.337	200.4653	14.76	8.39
0.452	23.58	-0.153	28.683	145.9365	17.16	9.32
0.454	23.58	-0.095	28.920	90.8501	19.58	10.25
0.456	23.58	-0.037	29.046	35.4164	22.02	11.19
0.458	23.58	0.021	29.062	-20.1526	24.46	12.13
0.460	23.58	0.079	28.966	-75.6446	26.91	13.07
0.462	23.58	0.137	28.759	-130.8475	29.33	14.00
0.464	23.58	0.194	28.443	-185.5503	31.74	14.93
0.466	23.58	0.251	28.018	-239.5441	34.11	15.85
0.468	23.58	0.306	27.486	-292.6224	36.45	16.74
0.470	23.58	0.360	26.848	-344.5824	38.73	17.62
0.472	23.58	0.413	26.109	-395.2256	40.96	18.48
0.474	23.58	0.465	25.269	-444.3584	43.12	19.31
0.476	23.58	0.514	24.333	-491.7931	45.21	20.12
0.478	23.58	0.562	23.304	-537.3483	47.21	20.89
0.480	23.58	0.608	22.186	-580.8500	49.12	21.62
0.482	23.58	0.651	20.983	-622.1320	50.94	22.32
0.484	23.58	0.691	19.699	-661.0364	52.65	22.98
0.486	23.58	0.730	18.341	-697.4146	54.25	23.60
0.488	23.58	0.765	16.912	-731.1276	55.73	24.17
0.490	23.58	0.797	15.419	-762.0465	57.09	24.69
0.492	23.58	0.826	13.867	-790.0533	58.33	25.17
0.494	23.58	0.853	12.262	-815.0407	59.42	25.59

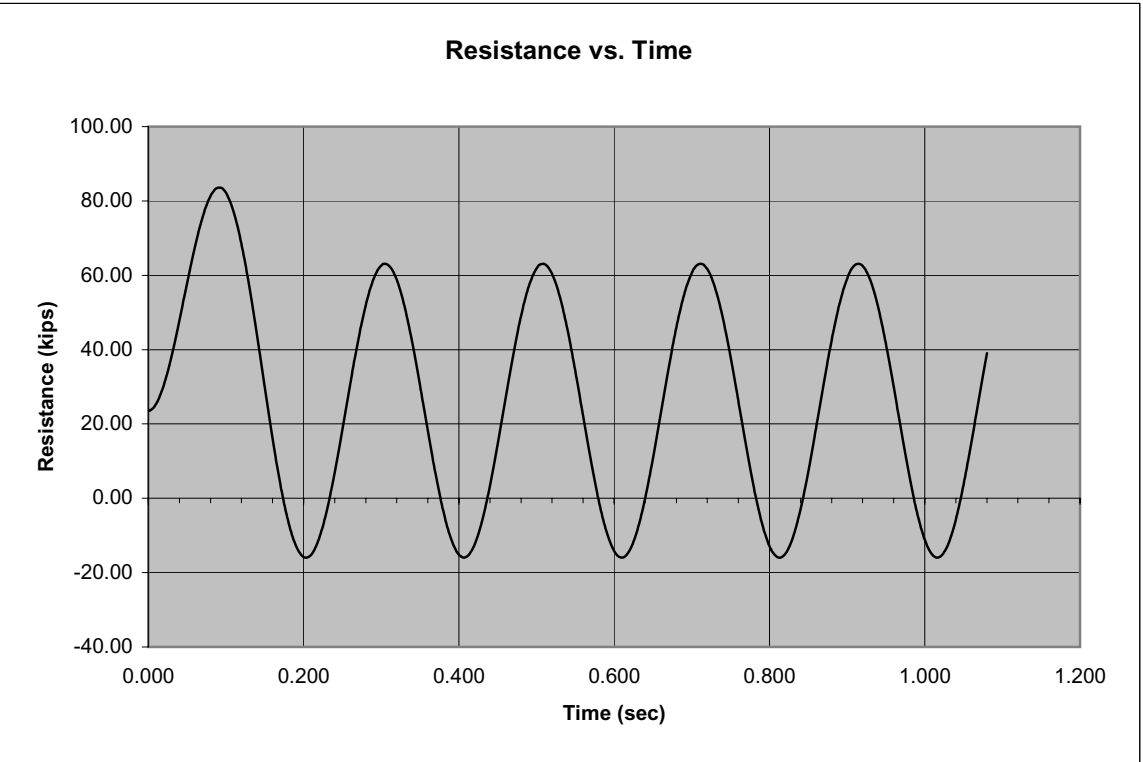
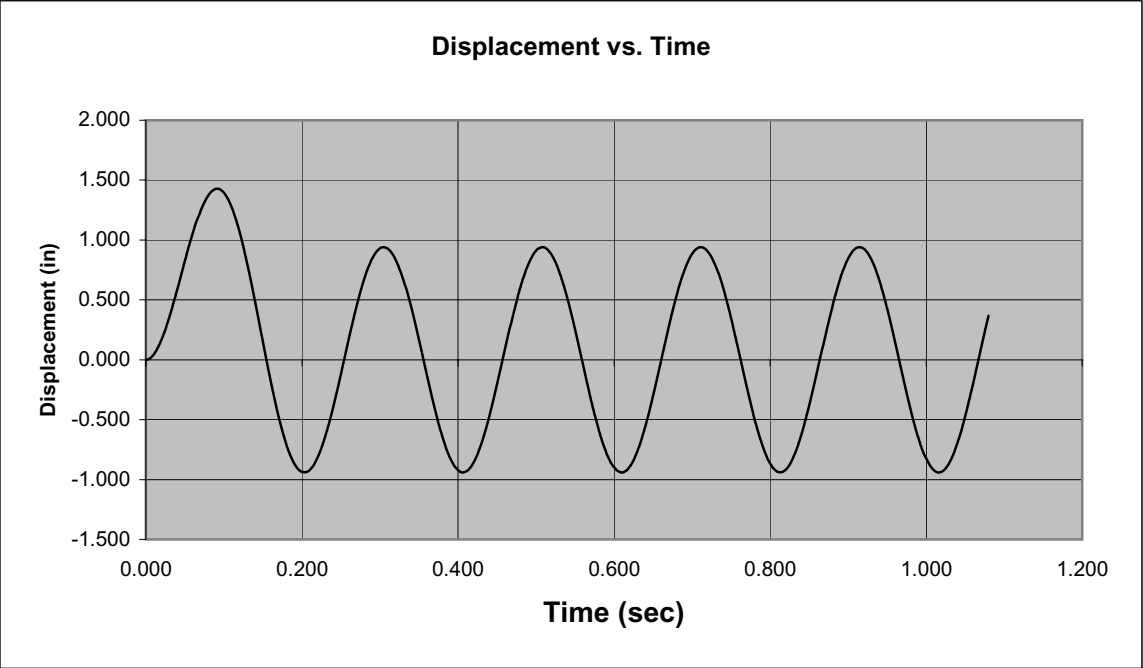
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.496	23.58	0.875	10.610	-836.9134	60.39	25.96
0.498	23.58	0.895	8.918	-855.5878	61.21	26.28
0.500	23.58	0.911	7.191	-870.9925	61.89	26.54
0.502	23.58	0.924	5.437	-883.0686	62.42	26.74
0.504	23.58	0.933	3.662	-891.7701	62.80	26.89
0.506	23.58	0.938	1.873	-897.0635	63.03	26.98
0.508	23.58	0.940	0.077	-898.9287	63.11	27.01
0.510	23.58	0.939	-1.719	-897.3587	63.05	26.98
0.512	23.58	0.933	-3.509	-892.3592	62.83	26.90
0.514	23.58	0.925	-5.285	-883.9496	62.46	26.76
0.516	23.58	0.912	-7.041	-872.1619	61.94	26.56
0.518	23.58	0.896	-8.770	-857.0412	61.27	26.30
0.520	23.58	0.877	-10.466	-838.6452	60.46	25.99
0.522	23.58	0.855	-12.122	-817.0442	59.51	25.62
0.524	23.58	0.829	-13.731	-792.3209	58.43	25.21
0.526	23.58	0.800	-15.288	-764.5696	57.20	24.74
0.528	23.58	0.768	-16.786	-733.8965	55.86	24.22
0.530	23.58	0.733	-18.221	-700.4187	54.38	23.65
0.532	23.58	0.695	-19.585	-664.2642	52.79	23.04
0.534	23.58	0.654	-20.875	-625.5712	51.09	22.38
0.536	23.58	0.611	-22.085	-584.4875	49.28	21.69
0.538	23.58	0.566	-23.211	-541.1702	47.38	20.95
0.540	23.58	0.519	-24.248	-495.7847	45.38	20.18
0.542	23.58	0.469	-25.192	-448.5046	43.30	19.38
0.544	23.58	0.418	-26.040	-399.5104	41.15	18.55
0.546	23.58	0.365	-26.789	-348.9895	38.93	17.70
0.548	23.58	0.311	-27.435	-297.1349	36.65	16.82
0.550	23.58	0.255	-27.976	-244.1448	34.32	15.92
0.552	23.58	0.199	-28.411	-190.2217	31.94	15.01
0.554	23.58	0.142	-28.736	-135.5716	29.54	14.08
0.556	23.58	0.084	-28.952	-80.4034	27.11	13.15
0.558	23.58	0.026	-29.058	-24.9279	24.67	12.21
0.560	23.58	-0.032	-29.052	30.6428	22.23	11.27
0.562	23.58	-0.090	-28.935	86.0964	19.79	10.33
0.564	23.58	-0.148	-28.708	141.2210	17.37	9.40
0.566	23.58	-0.205	-28.371	195.8059	14.97	8.47
0.568	23.58	-0.261	-27.925	249.6425	12.60	7.56
0.570	23.58	-0.316	-27.373	302.5251	10.27	6.67
0.572	23.58	-0.371	-26.716	354.2515	8.00	5.79
0.574	23.58	-0.423	-25.958	404.6242	5.78	4.94
0.576	23.58	-0.474	-25.100	453.4506	3.64	4.11
0.578	23.58	-0.524	-24.146	500.5440	1.56	3.31
0.580	23.58	-0.571	-23.099	545.7246	-0.42	2.55
0.582	23.58	-0.616	-21.965	588.8197	-2.32	1.82
0.584	23.58	-0.659	-20.746	629.6645	-4.11	1.13
0.586	23.58	-0.699	-19.448	668.1031	-5.81	0.48
0.588	23.58	-0.736	-18.076	703.9884	-7.38	-0.13
0.590	23.58	-0.771	-16.635	737.1834	-8.84	-0.69
0.592	23.58	-0.803	-15.130	767.5611	-10.18	-1.21
0.594	23.58	-0.832	-13.568	795.0056	-11.39	-1.67
0.596	23.58	-0.857	-11.953	819.4119	-12.46	-2.09
0.598	23.58	-0.879	-10.293	840.6868	-13.40	-2.45
0.600	23.58	-0.898	-8.594	858.7489	-14.19	-2.75
0.602	23.58	-0.914	-6.862	873.5292	-14.84	-3.00
0.604	23.58	-0.926	-5.103	884.9713	-15.34	-3.20
0.606	23.58	-0.934	-3.325	893.0313	-15.70	-3.33
0.608	23.58	-0.939	-1.534	897.6786	-15.90	-3.41
0.610	23.58	-0.940	0.262	898.8954	-15.96	-3.43
0.612	23.58	-0.938	2.058	896.6769	-15.86	-3.39
0.614	23.58	-0.932	3.845	891.0317	-15.61	-3.30
0.616	23.58	-0.923	5.618	881.9814	-15.21	-3.14

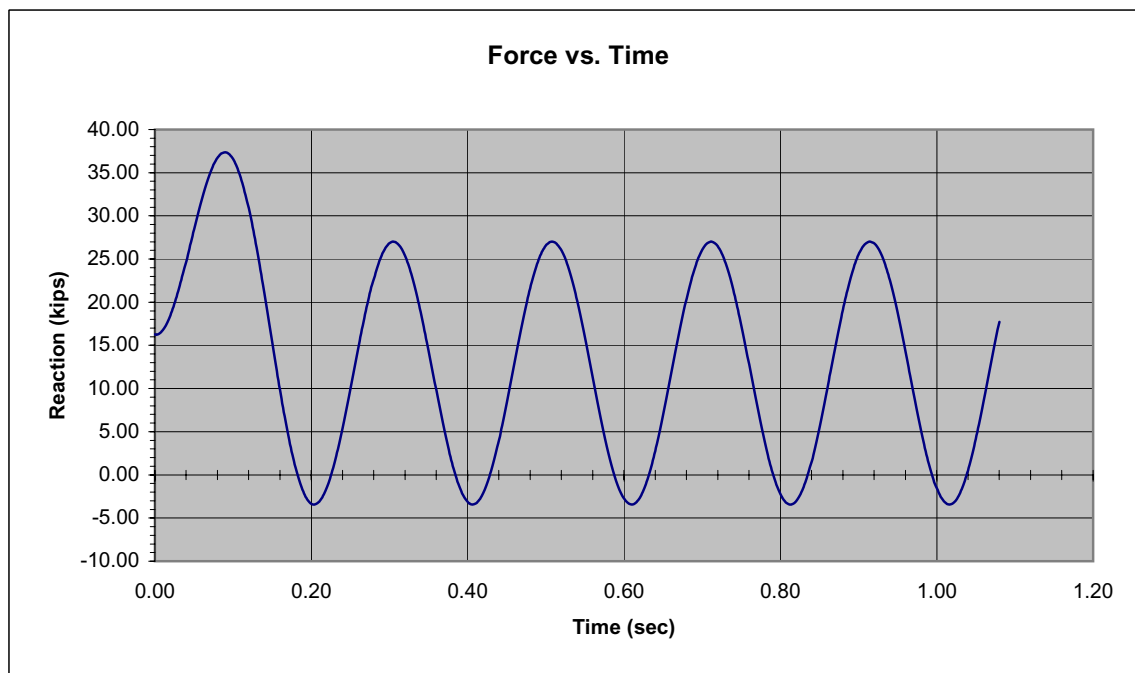
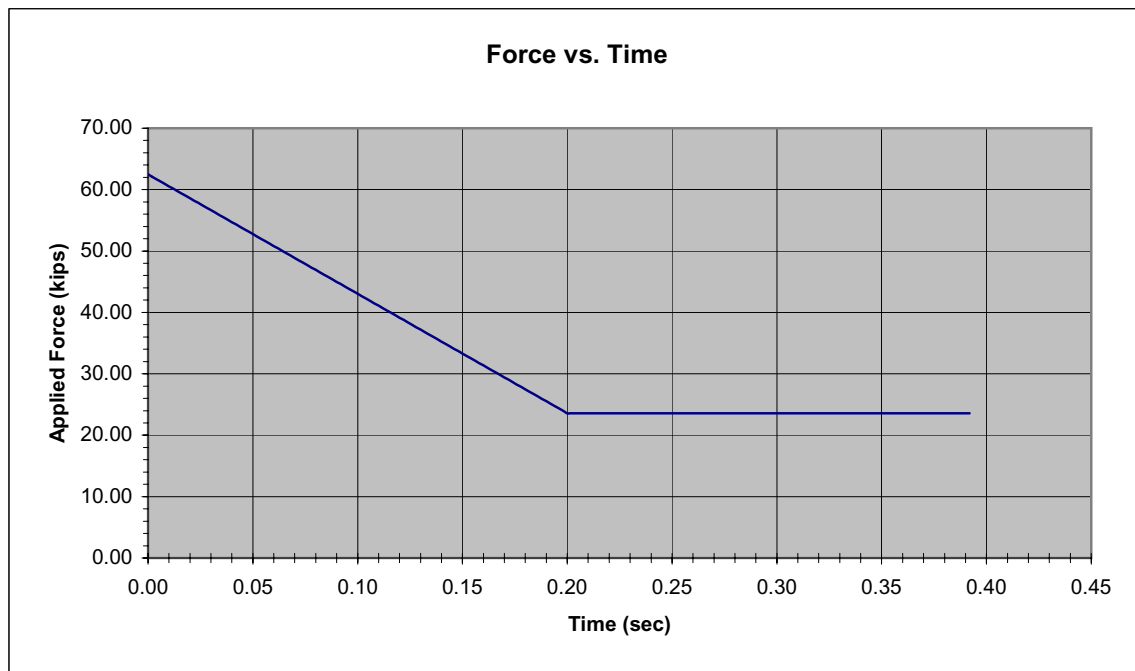
Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.618	23.58	-0.910	7.370	869.5605	-14.67	-2.93
0.620	23.58	-0.893	9.093	853.8165	-13.97	-2.67
0.622	23.58	-0.873	10.782	834.8096	-13.14	-2.35
0.624	23.58	-0.850	12.429	812.6124	-12.16	-1.97
0.626	23.58	-0.824	14.029	787.3098	-11.05	-1.54
0.628	23.58	-0.794	15.576	758.9983	-9.80	-1.06
0.630	23.58	-0.761	17.062	727.7863	-8.43	-0.53
0.632	23.58	-0.726	18.484	693.7930	-6.93	0.04
0.634	23.58	-0.687	19.835	657.1484	-5.32	0.66
0.636	23.58	-0.646	21.110	617.9923	-3.60	1.33
0.638	23.58	-0.603	22.305	576.4746	-1.78	2.03
0.640	23.58	-0.557	23.414	532.7539	0.15	2.77
0.642	23.58	-0.509	24.434	486.9971	2.16	3.54
0.644	23.58	-0.460	25.360	439.3793	4.25	4.35
0.646	23.58	-0.408	26.189	390.0823	6.42	5.18
0.648	23.58	-0.355	26.919	339.2947	8.66	6.04
0.650	23.58	-0.300	27.545	287.2103	10.95	6.93
0.652	23.58	-0.245	28.066	234.0284	13.29	7.83
0.654	23.58	-0.188	28.480	179.9521	15.66	8.74
0.656	23.58	-0.131	28.786	125.1882	18.07	9.67
0.658	23.58	-0.073	28.981	69.9458	20.50	10.60
0.660	23.58	-0.015	29.065	14.4361	22.94	11.54
0.662	23.58	0.043	29.038	-41.1288	25.39	12.49
0.664	23.58	0.101	28.901	-96.5365	27.82	13.42
0.666	23.58	0.159	28.653	-151.5753	30.24	14.36
0.668	23.58	0.216	28.295	-206.0348	32.64	15.28
0.670	23.58	0.272	27.829	-259.7069	35.00	16.19
0.672	23.58	0.327	27.257	-312.3865	37.32	17.08
0.674	23.58	0.381	26.581	-363.8724	39.58	17.95
0.676	23.58	0.433	25.803	-413.9676	41.79	18.80
0.678	23.58	0.484	24.927	-462.4809	43.92	19.62
0.680	23.58	0.533	23.955	-509.2267	45.97	20.41
0.682	23.58	0.580	22.892	-554.0265	47.95	21.17
0.684	23.58	0.624	21.741	-596.7091	49.82	21.89
0.686	23.58	0.666	20.507	-637.1113	51.60	22.58
0.688	23.58	0.706	19.195	-675.0787	53.27	23.22
0.690	23.58	0.743	17.809	-710.4662	54.83	23.82
0.692	23.58	0.777	16.356	-743.1386	56.26	24.37
0.694	23.58	0.809	14.840	-772.9711	57.57	24.88
0.696	23.58	0.837	13.267	-799.8496	58.76	25.33
0.698	23.58	0.862	11.643	-823.6714	59.80	25.74
0.700	23.58	0.883	9.975	-844.3455	60.71	26.09
0.702	23.58	0.901	8.269	-861.7929	61.48	26.38
0.704	23.58	0.916	6.531	-875.9468	62.10	26.62
0.706	23.58	0.928	4.769	-886.7533	62.58	26.80
0.708	23.58	0.935	2.988	-894.1709	62.90	26.93
0.710	23.58	0.940	1.195	-898.1714	63.08	27.00
0.712	23.58	0.940	-0.601	-898.7395	63.11	27.01
0.714	23.58	0.937	-2.396	-895.8730	62.98	26.96
0.716	23.58	0.931	-4.182	-889.5828	62.70	26.85
0.718	23.58	0.920	-5.951	-879.8930	62.28	26.69
0.720	23.58	0.907	-7.698	-866.8406	61.70	26.47
0.722	23.58	0.890	-9.415	-850.4755	60.98	26.19
0.724	23.58	0.869	-11.096	-830.8603	60.12	25.86
0.726	23.58	0.845	-12.735	-808.0699	59.12	25.47
0.728	23.58	0.818	-14.326	-782.1914	57.98	25.03
0.730	23.58	0.788	-15.861	-753.3236	56.71	24.55
0.732	23.58	0.755	-17.336	-721.5770	55.31	24.01
0.734	23.58	0.719	-18.745	-687.0728	53.80	23.42
0.736	23.58	0.680	-20.082	-649.9429	52.16	22.79
0.738	23.58	0.638	-21.342	-610.3292	50.42	22.12

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.740	23.58	0.595	-22.521	-568.3831	48.58	21.41
0.742	23.58	0.548	-23.613	-524.2649	46.64	20.67
0.744	23.58	0.500	-24.616	-478.1431	44.61	19.89
0.746	23.58	0.450	-25.524	-430.1941	42.50	19.07
0.748	23.58	0.398	-26.335	-380.6011	40.32	18.23
0.750	23.58	0.345	-27.045	-329.5536	38.07	17.37
0.752	23.58	0.290	-27.652	-277.2466	35.77	16.48
0.754	23.58	0.234	-28.153	-223.8801	33.43	15.58
0.756	23.58	0.177	-28.546	-169.6581	31.04	14.66
0.758	23.58	0.120	-28.831	-114.7877	28.63	13.73
0.760	23.58	0.062	-29.005	-59.4786	26.19	12.80
0.762	23.58	0.004	-29.069	-3.9422	23.75	11.86
0.764	23.58	-0.054	-29.021	51.6092	21.31	10.92
0.766	23.58	-0.112	-28.862	106.9634	18.87	9.98
0.768	23.58	-0.169	-28.593	161.9089	16.46	9.05
0.770	23.58	-0.226	-28.215	216.2356	14.07	8.13
0.772	23.58	-0.282	-27.729	269.7359	11.72	7.22
0.774	23.58	-0.337	-27.137	322.2054	9.41	6.33
0.776	23.58	-0.391	-26.442	373.4436	7.15	5.47
0.778	23.58	-0.443	-25.645	423.2547	4.96	4.62
0.780	23.58	-0.493	-24.750	471.4482	2.84	3.81
0.782	23.58	-0.542	-23.761	517.8401	0.80	3.02
0.784	23.58	-0.588	-22.681	562.2530	-1.15	2.27
0.786	23.58	-0.632	-21.514	604.5172	-3.01	1.55
0.788	23.58	-0.674	-20.265	644.4712	-4.77	0.88
0.790	23.58	-0.713	-18.939	681.9623	-6.41	0.24
0.792	23.58	-0.750	-17.540	716.8472	-7.95	-0.35
0.794	23.58	-0.783	-16.074	748.9926	-9.36	-0.89
0.796	23.58	-0.814	-14.547	778.2757	-10.65	-1.39
0.798	23.58	-0.842	-12.964	804.5846	-11.81	-1.83
0.800	23.58	-0.866	-11.332	827.8187	-12.83	-2.23
0.802	23.58	-0.887	-9.656	847.8892	-13.71	-2.57
0.804	23.58	-0.905	-7.943	864.7194	-14.45	-2.85
0.806	23.58	-0.919	-6.200	878.2450	-15.05	-3.08
0.808	23.58	-0.929	-4.434	888.4144	-15.49	-3.25
0.810	23.58	-0.936	-2.650	895.1886	-15.79	-3.37
0.812	23.58	-0.940	-0.856	898.5418	-15.94	-3.43
0.814	23.58	-0.940	0.941	898.4611	-15.94	-3.42
0.816	23.58	-0.936	2.734	894.9469	-15.78	-3.36
0.818	23.58	-0.929	4.517	888.0126	-15.48	-3.25
0.820	23.58	-0.918	6.283	877.6846	-15.02	-3.07
0.822	23.58	-0.904	8.024	864.0026	-14.42	-2.84
0.824	23.58	-0.886	9.735	847.0187	-13.67	-2.55
0.826	23.58	-0.865	11.409	826.7978	-12.78	-2.21
0.828	23.58	-0.840	13.039	803.4172	-11.76	-1.81
0.830	23.58	-0.813	14.620	776.9664	-10.59	-1.37
0.832	23.58	-0.782	16.144	747.5463	-9.30	-0.87
0.834	23.58	-0.748	17.607	715.2693	-7.88	-0.32
0.836	23.58	-0.712	19.003	680.2590	-6.34	0.27
0.838	23.58	-0.672	20.326	642.6489	-4.69	0.91
0.840	23.58	-0.630	21.571	602.5830	-2.92	1.59
0.842	23.58	-0.586	22.734	560.2142	-1.06	2.30
0.844	23.58	-0.539	23.810	515.7045	0.90	3.06
0.846	23.58	-0.491	24.795	469.2240	2.94	3.84
0.848	23.58	-0.440	25.685	420.9503	5.06	4.66
0.850	23.58	-0.388	26.477	371.0680	7.26	5.51
0.852	23.58	-0.334	27.168	319.7675	9.51	6.37
0.854	23.58	-0.280	27.755	267.2451	11.82	7.26
0.856	23.58	-0.224	28.235	213.7013	14.18	8.17
0.858	23.58	-0.167	28.609	159.3409	16.57	9.09
0.860	23.58	-0.109	28.872	104.3716	18.99	10.02

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.862	23.58	-0.051	29.026	49.0034	21.42	10.96
0.864	23.58	0.007	29.068	-6.5521	23.87	11.90
0.866	23.58	0.065	28.999	-62.0826	26.31	12.84
0.868	23.58	0.123	28.820	-117.3758	28.74	13.78
0.870	23.58	0.180	28.530	-172.2204	31.15	14.71
0.872	23.58	0.237	28.132	-226.4069	33.54	15.62
0.874	23.58	0.293	27.626	-279.7282	35.88	16.53
0.876	23.58	0.347	27.014	-331.9804	38.18	17.41
0.878	23.58	0.401	26.299	-382.9640	40.42	18.27
0.880	23.58	0.452	25.484	-432.4840	42.60	19.11
0.882	23.58	0.502	24.571	-480.3512	44.70	19.92
0.884	23.58	0.551	23.564	-526.3828	46.73	20.70
0.886	23.58	0.597	22.467	-570.4028	48.67	21.45
0.888	23.58	0.640	21.285	-612.2429	50.51	22.16
0.890	23.58	0.682	20.021	-651.7432	52.24	22.82
0.892	23.58	0.720	18.680	-688.7529	53.87	23.45
0.894	23.58	0.756	17.268	-723.1305	55.38	24.03
0.896	23.58	0.789	15.790	-754.7445	56.77	24.57
0.898	23.58	0.820	14.252	-783.4743	58.04	25.06
0.900	23.58	0.846	12.659	-809.2099	59.17	25.49
0.902	23.58	0.870	11.018	-831.8531	60.16	25.87
0.904	23.58	0.890	9.335	-851.3173	61.02	26.20
0.906	23.58	0.907	7.616	-867.5281	61.73	26.48
0.908	23.58	0.921	5.868	-880.4236	62.30	26.70
0.910	23.58	0.931	4.098	-889.9545	62.72	26.86
0.912	23.58	0.937	2.312	-896.0843	62.99	26.96
0.914	23.58	0.940	0.517	-898.7897	63.11	27.01
0.916	23.58	0.939	-1.280	-898.0603	63.08	27.00
0.918	23.58	0.935	-3.072	-893.8989	62.89	26.93
0.920	23.58	0.927	-4.852	-886.3214	62.56	26.80
0.922	23.58	0.916	-6.614	-875.3567	62.08	26.61
0.924	23.58	0.901	-8.350	-861.0468	61.45	26.37
0.926	23.58	0.882	-10.055	-843.4463	60.67	26.07
0.928	23.58	0.860	-11.721	-822.6226	59.76	25.72
0.930	23.58	0.835	-13.342	-798.6551	58.70	25.31
0.932	23.58	0.807	-14.912	-771.6355	57.52	24.86
0.934	23.58	0.776	-16.425	-741.6670	56.20	24.35
0.936	23.58	0.741	-17.876	-708.8642	54.75	23.79
0.938	23.58	0.704	-19.258	-673.3524	53.19	23.19
0.940	23.58	0.665	-20.567	-635.2673	51.52	22.55
0.942	23.58	0.622	-21.797	-594.7546	49.74	21.86
0.944	23.58	0.577	-22.944	-551.9689	47.85	21.14
0.946	23.58	0.530	-24.003	-507.0738	45.88	20.38
0.948	23.58	0.481	-24.970	-460.2409	43.82	19.58
0.950	23.58	0.431	-25.842	-411.6492	41.68	18.76
0.952	23.58	0.378	-26.615	-361.4843	39.48	17.91
0.954	23.58	0.324	-27.286	-309.9379	37.21	17.04
0.956	23.58	0.269	-27.854	-257.2072	34.89	16.14
0.958	23.58	0.213	-28.314	-203.4934	32.53	15.23
0.960	23.58	0.156	-28.667	-149.0020	30.13	14.31
0.962	23.58	0.098	-28.910	-93.9412	27.71	13.38
0.964	23.58	0.040	-29.042	-38.5214	25.27	12.44
0.966	23.58	-0.018	-29.064	17.0456	22.83	11.50
0.968	23.58	-0.076	-28.974	72.5475	20.39	10.56
0.970	23.58	-0.134	-28.774	127.7721	17.96	9.63
0.972	23.58	-0.191	-28.463	182.5085	15.55	8.70
0.974	23.58	-0.247	-28.044	236.5474	13.18	7.78
0.976	23.58	-0.303	-27.518	289.6823	10.84	6.88
0.978	23.58	-0.357	-26.887	341.7101	8.55	6.00
0.980	23.58	-0.410	-26.153	392.4321	6.32	5.14
0.982	23.58	-0.462	-25.319	441.6544	4.15	4.31

Time	force	y	v	a	resistance	reaction
(sec)	(kips)	(in)	(in/sec)	(in/sec2)	(kips)	(kips)
0.984	23.58	-0.512	-24.388	489.1888	2.06	3.51
0.986	23.58	-0.559	-23.364	534.8538	0.06	2.73
0.988	23.58	-0.605	-22.250	578.4748	-1.86	1.99
0.990	23.58	-0.648	-21.052	619.8851	-3.68	1.29
0.992	23.58	-0.689	-19.773	658.9265	-5.40	0.63
0.994	23.58	-0.727	-18.419	695.4497	-7.01	0.01
0.996	23.58	-0.763	-16.994	729.3152	-8.50	-0.56
0.998	23.58	-0.795	-15.504	760.3936	-9.86	-1.09
1.000	23.58	-0.825	-13.955	788.5661	-11.10	-1.56
1.002	23.58	-0.851	-12.353	813.7250	-12.21	-1.99
1.004	23.58	-0.874	-10.704	835.7742	-13.18	-2.36
1.006	23.58	-0.894	-9.013	854.6294	-14.01	-2.68
1.008	23.58	-0.910	-7.288	870.2186	-14.69	-2.95
1.010	23.58	-0.923	-5.536	882.4821	-15.23	-3.15
1.012	23.58	-0.932	-3.762	891.3732	-15.62	-3.30
1.014	23.58	-0.938	-1.973	896.8579	-15.87	-3.40
1.016	23.58	-0.940	-0.178	898.9151	-15.96	-3.43
1.018	23.58	-0.939	1.619	897.5371	-15.90	-3.41
1.020	23.58	-0.934	3.409	892.7290	-15.68	-3.33
1.022	23.58	-0.925	5.186	884.5093	-15.32	-3.19
1.024	23.58	-0.913	6.944	872.9094	-14.81	-2.99
1.026	23.58	-0.897	8.675	857.9736	-14.16	-2.74
1.028	23.58	-0.878	10.372	839.7590	-13.35	-2.43
1.030	23.58	-0.856	12.030	818.3352	-12.41	-2.07
1.032	23.58	-0.830	13.642	793.7841	-11.33	-1.65
1.034	23.58	-0.801	15.202	766.1994	-10.12	-1.18
1.036	23.58	-0.770	16.704	735.6867	-8.78	-0.67
1.038	23.58	-0.735	18.142	702.3624	-7.31	-0.10
1.040	23.58	-0.697	19.511	666.3541	-5.73	0.51
1.042	23.58	-0.657	20.805	627.7992	-4.03	1.16
1.044	23.58	-0.614	22.020	586.8451	-2.23	1.85
1.046	23.58	-0.569	23.150	543.6484	-0.33	2.58
1.048	23.58	-0.521	24.192	498.3740	1.66	3.35
1.050	23.58	-0.472	25.142	451.1951	3.73	4.15
1.052	23.58	-0.421	25.995	402.2919	5.89	4.98
1.054	23.58	-0.368	26.750	351.8513	8.10	5.83
1.056	23.58	-0.314	27.402	300.0661	10.38	6.71
1.058	23.58	-0.259	27.949	247.1342	12.71	7.60
1.060	23.58	-0.202	28.389	193.2578	15.08	8.52
1.062	23.58	-0.145	28.721	138.6428	17.48	9.44
1.064	23.58	-0.087	28.943	83.4981	19.91	10.38
1.066	23.58	-0.029	29.055	28.0342	22.35	11.31
1.068	23.58	0.029	29.055	-27.5368	24.79	12.26
1.070	23.58	0.087	28.945	-83.0025	27.23	13.19
1.072	23.58	0.145	28.724	-138.1511	29.65	14.13
1.074	23.58	0.202	28.393	-192.7717	32.06	15.05
1.076	23.58	0.258	27.953	-246.6556	34.43	15.97
1.078	23.58	0.313	27.407	-299.5969	36.76	16.86
1.080	23.58	0.368	26.756	-351.3933	39.03	17.74





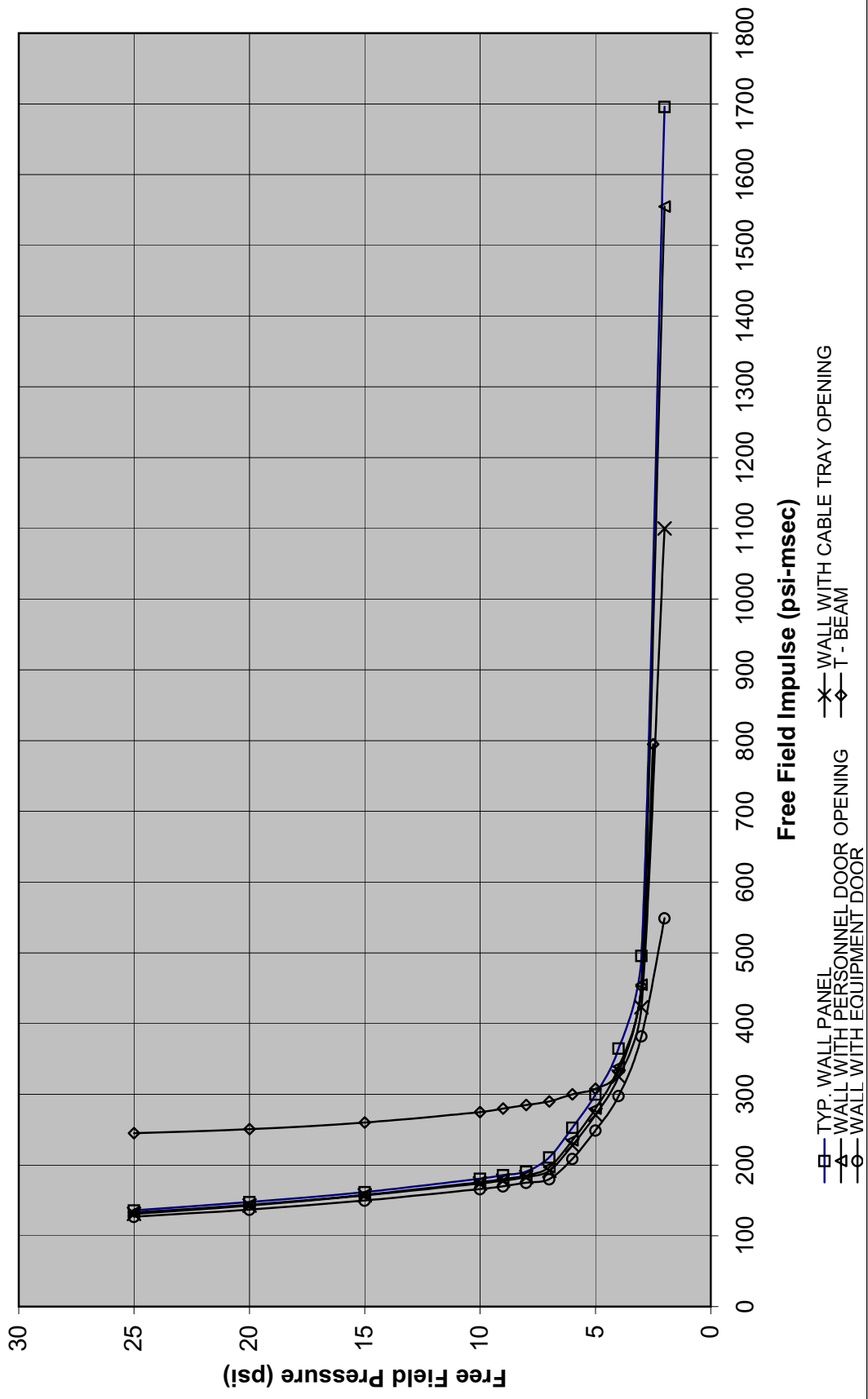
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Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

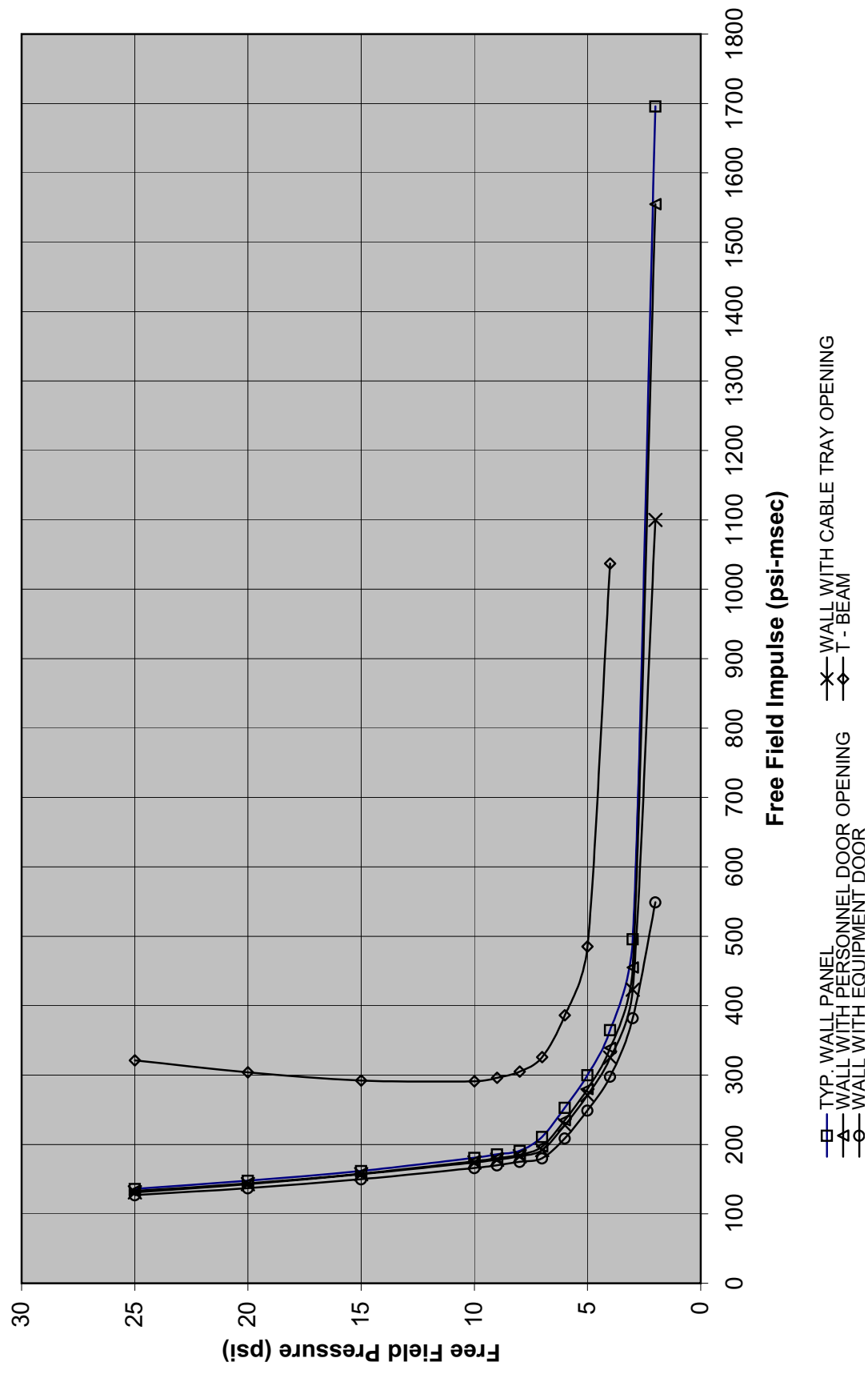
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Check'd by: SAK

PRESSURE - IMPULSE (PI) DIAGRAM

SUBSTATION 1 LOW RESPONSE P-I Diagram



SUBSTATION 2 LOW RESPONSE P-I Diagram



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 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : TENSION CHORD AND SHEARWALL LOAD SS # 2

Sheet no. : of
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 Design by : MGG
 Check'd by : SAK

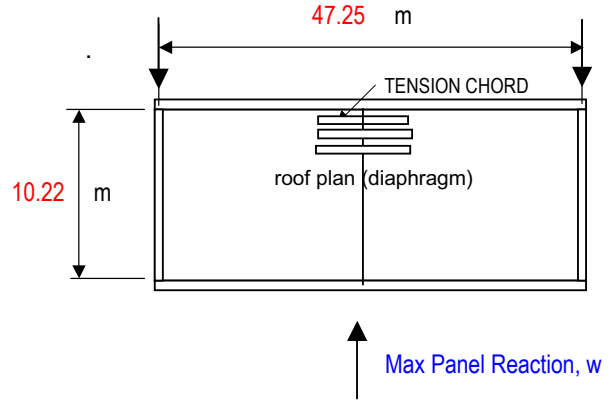
DESIGN STRESSES :

Conc., f_c' = 34.5 Mpa
 Reinf., f_y = 414 Mpa
 Structural Steel, GR 50, f_y = 355 Mpa

DATA :

Roof Diaphragm :
 Long Span, l_b = 47250 mm
 Diaphragm depth = 10220 mm
 Slab, thickness = 150 mm
 Wall Height ; h_t = 6775 mm

CONFIGURATION :



Diaphragm Loads : (Rear Wall Reaction to be neglected)

Front Wall Maximum Reaction :

	w	unit	(kN/m)	no. of walls
Typical Wall	5.85	k / ft	(85.59)	3 walls
Wall with Cable Tray opening	5.26	k / ft	(76.96)	13 walls
Wall with Personnel Door	5.59	k / ft	(81.78)	3 walls

Average Wall Reaction ; w = 5.41 k / ft (79.08 Kn/m)

Front Wall Reaction w = 5.41 k / ft (79.08 Kn/m)

Bending Moment in Diaphragm, $Ma = wL^2 / 8$ Ma = 22069.02 Kn - m

Distance of tension Chord from diaphragm edge ; d_{chord} = 800.00 mm

Effective Diaphragm Depth, $d_{ep} = d_p - d_{chord}$ = 9420.00 mm

Tension ; $T = Ma / d_{ep}$ T = 2342.78 Kn

$f'_{dc} = (SIF) \times (DIF) \times f'_c$ (for bending) = 5954.3 psi (41 Mpa.)

$f'_{dc} = (SIF) \times (DIF) \times f'_c$ (for shear) = 5504.0 psi (38 Mpa.)

$f_{dy} = (SIF) \times (DIF) \times f_y$ = 437 Mpa

Steel Area Required; $As = T / f_{dy}$ = 5356.6 mm²

Plate Width x Thickness; 200 mm x 12 mm As = 2400.0000 mm²

Provide 3 - 200 mm x 12 mm As = 7200.0000 mm²

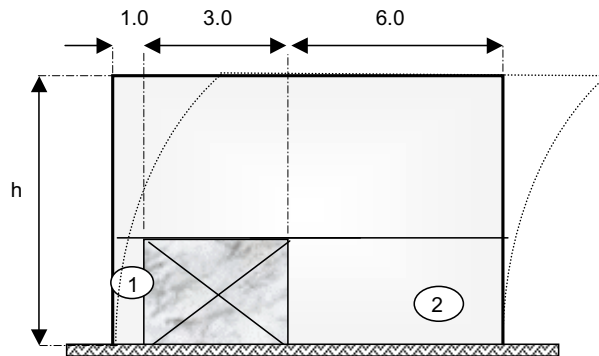
Shearwall Loads :

Shear Force ; $V_u = w \times \text{Bldg Length} / 2$ V_u = 1868.28 Kn

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Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : CHECK SHEAR WALL (10 M) as CANTILEVER

Sheet no. : of
 Date : 17-Apr-04
 Design by : MGG
 Check'd by : SAK



Total Weight, Nu = 562.46 Kn
 PIER 1 = 122.47 Kn
 PIER 2 = 439.99 Kn

Input Data:

f'_c	34.5	MPa
F_y	414	MPa
t	250	mm
V	1868.3	kN
Vert bar	25	mm
Hori bar	12	mm
$V_{(final)}$	1868.3	kN

P	1000	kN
E_c	24870	MPa
Length of Wall	10.0	m

DISTRIBUTION OF FORCES TO EACH PIER

$$\Delta_F = P[4(h/d)^3 + 3(h/d)]/Ect$$

$$R_F = 1/\Delta_F$$

$$M = V \cdot h$$

$$\% \text{ lateral} = R_F / \Sigma R_F \cdot 100$$

$$V = \frac{\% \text{ lateral} \times (V_{final})}{100}$$

Pier No.	h (m)	lw (m)	h/lw	Δ_F	R_F	% lateral force	Vu (kN)	Mu	Nu (kN)
1	7.500	1.000	7.50	0.2750	3.636	0.672	12.55	94.11	122.47
2	7.500	6.000	1.25	0.0019	537.730	99.328	1855.75	13918	439.99
					541.37	100.00	1868.30	14012.3	562.46

DIAGONAL SHEAR CRACKING CAPACITY

Shear strength V_n shall not be taken greater than ;

$$V_n = 0.83 A_{cv} \sqrt{f'_c}$$

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Job No. : BI - 3039
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Shear strength V_c shall be the minimum of :

$$V_{c1} = 0.274 \sqrt{f_c} d t + \frac{N u d}{4 l w}$$

$$V_{c2} = 0.05 \sqrt{f_c} + \frac{l w (0.104 f_c + 0.2 N u / (l w t))}{M u / V u - 0.5 l w} ; \text{ this will not apply if less than zero}$$

Shear strength V_s shall be the minimum of :

$$V_s = \frac{2 A_h f_{yds} d}{s} \quad f_{yds} = 1.1 \times f_y$$

REINFORCEMENT PROVIDED :

USING $\rho_{min} = 0.0025$ (for horizontal and vertical shear reinforcement)

Vertical Reinforcement reqrd:

Horizontal Reinforcement:

PROVIDED FOR PIER 1 (CRITICAL)

$$A_o = 490.874$$

$$\text{no of 25 mm} = 18$$

$$p_v = N_o \times A_o / (t \times d)$$

$$p_v = 0.0059 > \rho_{min} \text{ Ok!}$$

$$12 \text{ DIA. @ , 300}$$

$$A_o = 113.097$$

$$p_n = 2 A_o / (t \times \text{spacing})$$

$$p_n = 0.0030 > \rho_{min} \text{ Ok!}$$

Pier	$l w \times t \text{ (mm}^2\text{)}$	$d = 0.8 l w$	V_{c1}	V_{c2}	V_s	$V_n = V_c + V_s$	$V u / \phi$	REMARKS
1	250000	0.8	346.37	78.99	274.69	353.68	14.76	ok
2	1500000	4.8	2019.26	1423.66	1648.14	3667.41	2183.24	ok
Σ	1750000					4021 kN	2198 kN	

$$V_n = 0.83 A_{cv} \sqrt{f_c} = 8531.51 \text{ KN}$$

SHEAR FRICTION CAPACITY

Check shear friction at sections along the height of the opening

$$V_n = A_{vf} f_{yds} u ; \text{ but should be less than } 0.2 f_{cds} A_c \text{ nor } 5.53 A_c$$

A_{vf} = Area of shear friction reinforcement

u = shear friction coefficient

$$f_{yds} = 1.1 \times 1.1 \times f_y$$

$$f_{cds} = 1.0 \times 1.1 \times f_c$$

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Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : CHECK SHEAR WALL (10 M) as CANTILEVER

Sheet no. : of
 Date : 17-Apr-04
 Design by : MGG
 Check'd by : SAK

Pier No.	Ac	A _{vf}	V _{n1} (Kn)	0.2f _{cds} Ac	5.53 Ac	V _n (Kn)	V _u /φ	
1	250000	8835.660	4426.14	1897.50	1382.50	1382.50	14.76	OK
2	1500000	26506.980	13278.41	11385.00	8295.00	8295.00	2183.24	OK

REQUIRED REBARS IN FLEXURE

$R_u = M_u / \phi b d^2$ where:

$\phi = 0.9$ for bending

$R_u = f'_{cds} \omega (1 - 0.59 \omega)$

$d_{eff} = 0.8d$

$b = t$

$f_{yds} = 1.1 \times 1.17 \times f_y$

$f_{cds} = 1.0 \times 1.19 \times f_c$

$\rho = \omega f'_{cds} / f_{yds}$

$= 250 \text{ mm}$

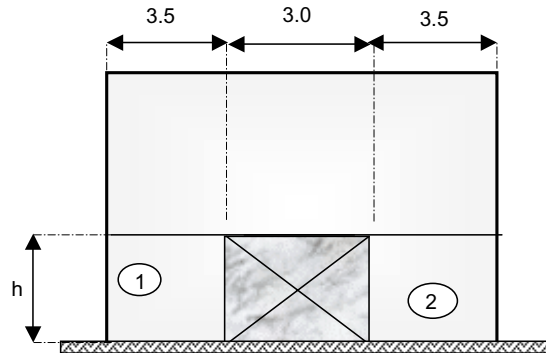
REQUIRED REINFORCEMENT

Pier	M _u	d _{eff}	R _u	ω	ρ	As	No of -25mmφ required
1	94.11	0.8	0.65	0.0161	0.00124	248	0.50
2	13918.14	4.8	2.68	0.0681	0.00525	6300	12.83

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Job No. : BI - 3039
 Project : REPLACE SUBSTATIONS 472, 321 & F07
 Subject : CHECK SHEAR WALL

Sheet no. : of
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 Design by : MGG
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Total Weight, Nu = 562.46 Kn
 PIER 1 & 2 = 281.23 Kn

Input Data:

f'_c	34.5	MPa
F_y	414	MPa
t	250	mm
V	1868.3	kN
Vert bar	25	mm
Hori bar	12	mm
$V_{(final)}$	1868.3	kN

P	1000	kN
E_c	24870	MPa
Length of Wall	10.0	m

DISTRIBUTION OF FORCES TO EACH PIER

$$\Delta_F = P[(h/d)^3 + 3(h/d)]/Ect$$

$$R_F = 1/\Delta_F$$

$$M = V \cdot h/2$$

$$\% \text{ lateral} = R_F / \sum R_F \cdot 100$$

$$V = \frac{\% \text{ lateral} \times (V_{final})}{100}$$

Pier No.	h (m)	lw (m)	h/lw	Δ_F	R_F	% lateral force	Vu (kN)	Mu	Nu (kN)
1	2.800	3.500	0.80	0.0005	2135.130	50.000	934.15	1307.81	281.23
2	2.800	3.500	0.80	0.0005	2135.130	50.000	934.15	1307.81	281.23
					4270.26	100.00	1868.30	2615.62	562.46

DIAGONAL SHEAR CRACKING CAPACITY

Shear strength V_n shall not be taken greater than ;

$$V_n = 0.83 A_{cv} \sqrt{f'_c}$$

Shear strength V_c shall be the minimum of :

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$$V_{c1} = 0.274 \sqrt{f_c} d t + \frac{N_u d}{4 l_w}$$

$$V_{c2} = 0.05 \sqrt{f_c} + \frac{l_w (0.104 f_c + 0.2 N_u / (l_w t))}{M_u / V_u - 0.5 l_w} ; \text{ this will not apply if less than zero}$$

Shear strength Vs shall be the minimum of :

$$V_s = \frac{2 A_h f_{yds} d}{s} \quad f_{yds} = 1.1 \times f_y$$

REINFORCEMENT PROVIDED :

USING $p_{min} = 0.0025$ (for horizontal and vertical shear reinforcement)

Vertical Reinforcement reqrd:

Horizontal Reinforcement:

PROVIDED FOR PIER 2 & 3

12 DIA. @ , 300

$$A_o = 490.874$$

$$A_o = 113.097$$

$$\text{no of 25 mm} = 36$$

$$p_n = 2 A_o / (t \times \text{spacing})$$

$$p_v = N_o \times A_o / (t \times d)$$

$$p_n = 0.0030 > p_{min} \text{ Ok!}$$

$$p_v = 0.0202 > p_{min} \text{ Ok!}$$

Pier	$l_w \times t \text{ (mm}^2\text{)}$	$d = 0.8 l_w$	V_{c1}	V_{c2}	V_s	$V_n = V_c + V_s$	V_u / ϕ	REMARKS
1	875000	2.8	1182.82	(4520.42)	961.42	2144.23	1099.00	ok
2	875000	2.8	1182.82	(4520.42)	961.42	2144.23	1099.00	ok
Σ	1750000					4288 kN	2198 kN	

$$V_n = 0.83 A_{cv} \sqrt{f_c} = 8531.51 \text{ KN}$$

SHEAR FRICTION CAPACITY

Check shear friction at sections along the height of the opening

$$V_n = A_{vf} f_{yds} u ; \text{ but should be less than } 0.2 f_{cds} A_c \text{ nor } 5.53 A_c$$

A_{vf} = Area of shear friction reinforcement

u = shear friction coefficient

$$f_{yds} = 1.1 \times 1.1 \times f_y$$

$$f_{cds} = 1.0 \times 1.1 \times f_c$$

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Sheet no. : of
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Pier No.	Ac	A _{vf}	V _{n1} (Kn)	0.2f _{cds} Ac	5.53 Ac	V _n (Kn)	V _u /φ
1	875000	8835.660	4426.14	6641.25	4838.75	4426.14	1099.00
2	875000	8835.660	4426.14	6641.25	4838.75	4426.14	1099.00

REQUIRED REBARS IN FLEXURE

$R_u = M_u / \phi b d^2$ where:

$\phi = 0.9$ for bending

$R_u = f'_{cds} \omega (1 - 0.59 \omega)$

$\rho = \omega f'_{cds} / f_{yds}$

$d_{eff} = 0.8d$

$b = t$

$= 250 \text{ mm}$

$f_{yds} = 1.1 \times 1.17 \times f_y$

$f_{cds} = 1.0 \times 1.19 \times f_c$

REQUIRED REINFORCEMENT

Pier	M _u	d _{eff}	R _u	ω	ρ	As	No of -25mmφ required
1	1307.81	2.8	0.74	0.0183	0.00141	985	2.01
2	1307.81	2.8	0.74	0.0183	0.00141	985	2.01

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Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/10/2004
Design by : MGG
Check'd by: SAK

CORBEL DESIGN

Job No. BI -3039

Date Jan 14, 2003

Project REPLACE SUBSTATIONS 472, 321 & F07 - ABU ALI

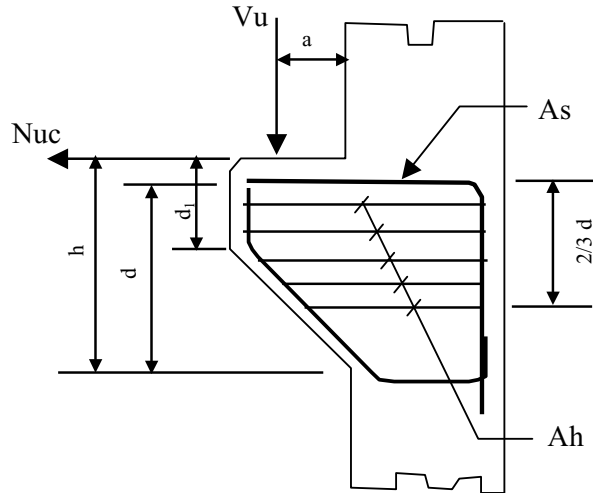
By MGG

Subject DESIGN OF CORBEL

Ch'k by EY

I. DATA :

$$\begin{aligned} V_u &= 209.06 \text{ kN} \\ N_{uc} &= 45.26 \text{ kN} \\ d &= 252.00 \text{ mm} \\ h &= 310.00 \text{ mm} \\ a &= 75.00 \text{ mm} \\ d_1 &= 310.00 \text{ mm} \\ b_w &= 310.00 \text{ mm} \\ f'_c &= 28.00 \text{ Mpa} \\ f_y &= 420.00 \text{ Mpa} \\ \phi &= 0.75 \end{aligned}$$



$$\begin{aligned} A_s \text{ bar } \phi &= 16 \text{ mm } \phi \quad (\text{primary reinforcement}) \\ A_h \text{ bar } \phi &= 10 \text{ mm } \phi \quad (\text{closed stirrups}) \end{aligned}$$

II. SPAN TO DEPTH RATIO ; a/d

ACI (11.9.1)

$$\begin{aligned} \frac{a}{d} &= 0.30 < 1.0 ; \text{ ok} \\ V_u &> N_{uc} ; \text{ ok} \end{aligned}$$

III. DEPTH AT OUTSIDE EDGE OF BEARING AREA

ACI (11.9.2)

$$d_1 > (0.5d = 126.00) ; \text{ ok}$$

IV. SHEAR FRICTION REINFORCEMENT TO RESIST V_u ; A_v_f

ACI (11.9.3.2)

CHECK NOMINAL SHEAR ; V_n (NORMAL CONCRETE WT); ACI (11.9.3.2.1)

$$\begin{aligned} V_n &= \frac{V_u}{\phi} \\ &= 278.75 \text{ kN} ; \text{ but should be} \\ &< 0.2 f'_c b_w d = 437.47 \text{ kN } \text{ ok} \\ &< 5.52 b_w d = 431.2 \text{ kN } \text{ ok} \end{aligned}$$

$$\begin{aligned} A_{v_f} &= \frac{V_u}{\phi f_y u} \quad \text{where } u = 1\lambda \quad \text{cast on hardened concrete intentionally roughened} \\ \lambda &= 1.0 \quad \text{ACI (11.7.4.1)} \end{aligned}$$

$$A_{v_f} = 663.69 \text{ mm}^2$$

Job No.	BI -3039	Date	Jan 14, 2003
Project	REPLACE SUBSTATIONS 472, 321 & F07 - ABU ALI	By	MGG
Subject	DESIGN OF CORBEL	Ch'k by	EY

V. REINFORCEMENT TO RESIST MOMENT MU ; Af ACI (11.9.3.3)

$$\begin{aligned} M_u &= V_u (a) + N_{uc} (h - d) \\ &= 0.02 \text{ kN} \cdot \text{m} \end{aligned}$$

$$R_n = \frac{M_u}{\phi b d^2} = 0.00124 \quad ; \quad m = \frac{f_y}{0.85 f'_c} = 17.647$$

$$p = \frac{1}{m} \left[1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right] = 0.000003$$

$$\begin{aligned} p_{\min} &= 1.4 / f_y \\ &= 0.003 \quad \text{GOVERNS} \end{aligned}$$

$$A_f = p b_w d = 260.40 \text{ mm}^2$$

VI. REINFORCEMENT TO RESIST TENSILE FORCE Nuc ; An ACI (11.9.3.4)

$$\begin{aligned} N_{uc} &= \phi A_n f_y \quad \text{but } N_{uc} \text{ shall not be taken less than } 0.2V_u \\ 0.2 V_u &= 41.81 \text{ kN} \\ \text{use } N_{uc} &= 45.26 \text{ kN} \end{aligned}$$

$$A_n = \frac{N_{uc}}{\phi f_y} = 143.69 \text{ mm}^2$$

VII. PRIMARY REINFORCEMENT; As ACI (11.9.3.5)

$$\begin{aligned} A_s &= A_n + A_f = 404.09 \text{ mm}^2 \\ &= 2/3 A_{v_f} + A_n = 586.15 \text{ mm}^2 \end{aligned}$$

$$A_{s_{\min}} = 0.04 f'_c / f_y b_w d = 208.32 \text{ mm}^2 \quad \text{ACI (11.9.5)}$$

$$\begin{aligned} \text{Governing } A_s &= 586.15 \text{ mm}^2 \\ &; \text{ provide } 3 \text{ - } 16 \text{ mm } \phi \quad ; A_s = 603.19 \text{ mm}^2 \end{aligned}$$

VIII. CLOSE TIES PARALLEL TO As ; Ah ACI (11.9.4)

$$\begin{aligned} A_h &= 0.5 (A_s - A_n) \\ &= 229.75 \text{ mm}^2 \\ &; \text{ provide } 3 \text{ - } 10 \text{ mm } \phi \quad ; A_h = 235.62 \text{ mm}^2 \\ &\text{and shall be distributed within } 2/3 \text{ of effective depth; } d \end{aligned}$$

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Job No. : BI - 3039
Project : REPLACE SUBSTATIONS 472, 321 & F07
Subject :

Sheet no. : of
Date : 2/17/2004
Design by : MGG
Check'd by: SAK

DYNAMIC SLIDING AND OVERTURNING

Saudconsult

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Job No. : BI 3039
 Project : ABU ALI SUBSTATION 1 & 2
 Subject : DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 1)

Date : 1/18/2004
 By : MGG
 Chck'd by: SAK

INPUT PARAMETERS

Physical Constants:

Acceleration of gravity	$g =$	386.4	in/sec ²
Co-efficient of sliding friction	$\mu d =$	0.35	
Unit weight of concrete	$\gamma_{conc} =$	150	lbs/ft ³
Blast wave reflection coefficient	$Cra =$	2.1	

Physical Parameters of the Building:

Length of target surface on the building	$L_b =$	131	ft	1575	in
Height of target surface on the building	$H_b =$	24.1	ft	289	in
Width of the building	$B_b =$	49.2	ft	590	in
Weight of roofing	$Roofwt =$	1.23	psi	8.47	kpa
Weight of wall	$Wallwt =$	1.08	psi	7.44	kpa

Operating weight of the building

$$W_b = Roofwt * L_b * B_b + Wallwt * (2 * L_b + 2 * B_b) * H_b = 2496255.05 \text{ lbs}$$

Height of Center of gravity above the base

of the building module (estimated) ; $H_b / 4 = H_{bcg} = 72.3 \text{ in}$

Physical Parameters of the Foundation:

Foundation length ; $L_b + 1.5 (2) \text{ m}$	$L_f =$	1693	in
Foundation depth ; 0.8 m.	$H_f =$	2.63	ft 31.6 in
Foundation width ; $B_b + 1.5 (2) \text{ m}$	$B_f =$	709	in
Other foundation dimensions ;	$bf =$	157	in
	$e =$	0	in
	$ts =$	17.8	in

Design Loads for the Building

Reflected pressure	$Par =$	4.2	lbs/in ²
Reflected impulse per unit urea	$iar =$	0.42	lbs-sec/in ²
Target surface area ; A	$H_b * L_b =$	455455.296	in ²
Reflected force on target surface ; Far	$Par * A =$	1912912.24	lbs
Reflected impulse on target surface ; lar	$iar * A =$	191291.224	lbs-sec

CALCULATED PARAMETERS

Building Properties

Mass of building ;	$W_b/g =$	$M_b =$	6460.29	lbs-sec/in ²
Center of gravity location with respect to the origin ;	$B_f / 2 =$	$X_{cg} =$	354.258	in
	$H_{bcg} + H_f =$	$Y_{cg} =$	103.86	in

Job No. :	BI 3039	Date	1/18/2004
Project :	ABU ALI SUBSTATION 1 & 2	By :	MGG
Subject :	DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 1)	Chck'd by:	SAK

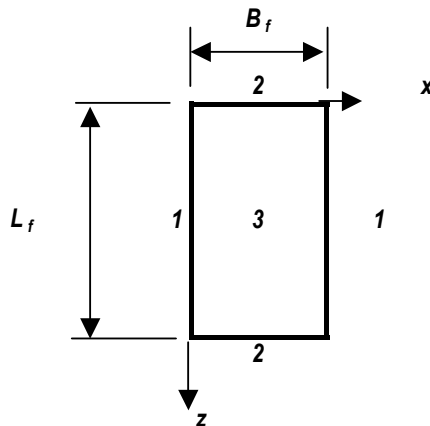
Rotation arm about Point A ;

$$db = \sqrt{(B_f - X_{cg})^2 + Y_{cg}^2} = 369.17 \text{ in}$$

Rotary inertia of the building about Point A ;

$$M_b * d_b^2 = I_{Aab} = 8.804E+08 \text{ lbs-sec-in}$$

Foundation Properties



Average width of edge beams (components nos. 1 and 2)

$$b_{avg} = 0.5 * (b_f + (b_f + e)) = 157.49 \text{ in}$$

Depth of edge beams (Component Nos. 1 and 2)

$$db = H_f - t_s = 13.76 \text{ in}$$

Length of edge beams

$$L1 = L_f = 1693.00 \text{ in}$$

$$L2 = Bf - 2 * b_{avg} = 393.54 \text{ in}$$

Weights of each component

$$Wf1 = \gamma_{conc} * (2 * (L_1 * b_{avg} * d_b)) / 1728 = 636941.21 \text{ lbs}$$

$$Wf2 = \gamma_{conc} * (2 * (L_2 * b_{avg} * d_b)) / 1728 = 148058.14 \text{ lbs}$$

$$Wf3 = \gamma_{conc} * (L_f * Bf * t_s) / 1728 = 1853416.89 \text{ lbs}$$

Total foundation weight ;

$$Wf = W_{f1} + W_{f2} + W_{f3} = 2638416.25 \text{ lbs}$$

Total foundation mass

$$Mf = W_f / g = 6828.20 \text{ lbs-sec/in}^2$$

Moment arm from Pont A to Center of gravity

$$a1 = \sqrt{(d_b/2)^2 + (B_f - b_{avg}/2)^2} = 629.81 \text{ in}$$

$$a2 = \sqrt{(d_b/2)^2 + (B_f/2)^2} = 354.32 \text{ in}$$

$$a3 = \sqrt{(d_b + (t_s/2))^2 + (Bf/2)^2} = 354.98 \text{ in}$$

Rotary inertia about pivot at point A for each component treating each component as a rectangular prism rotating about a corner ;

$$IAf1 = W_{f1} * a_1^2 / g = 653853957 \text{ lbs-sec/in}^2$$

$$IAf2 = 2 * 4/3 * W_{f2} / g * a_2^2 = 128282383 \text{ lbs-sec/in}^2$$

$$IAf3 = 4/3 * W_{f3} / g * a_3^2 = 805911497 \text{ lbs-sec/in}^2$$

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Job No. : BI 3039

Date 1/18/2004

Project : ABU ALI SUBSTATION 1 & 2

By : MGG

Subject : DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 1)

Chck'd by: SAK

Total rotary inertia of the foundation

$$IA_{af} = IA_{f1} + IA_{f2} + IA_{f3} = 1588047837 \text{ lbs-sec/in}^2$$

Combined System Properties

$$\text{Total weight ; } W = W_b + W_f = 5134671.30 \text{ lbs}$$

$$\text{Total Mass ; } M = M_b + M_f = 13288.49 \text{ lbs-sec/in}^2$$

$$\text{Total rotary inertia } IA_{\alpha} = I_{Aab} + I_{Aaf} = 2.468.E+09 \text{ lbs-sec/in}^2$$

SLIDING ANALYSIS

Work-Energy Equation:

Let,

$$\begin{aligned} \text{Sliding distance ; } &= \Delta \\ \text{Velocity ; } &= v \\ \text{Applied Force ; } &= F \\ \text{Load duration ; } &= t_d \\ \text{Mass, ; } W/g &= M \\ \text{Kinetic energy ; } KE &= 1/2 * W/g * V^2 \end{aligned}$$

Net work done for a body initially at rest

$$PE = W * \mu_d * \Delta$$

$$\text{Applied impulse, } I_a = F * t_d$$

Initial velocity of a body given this impulse ;

$$v = I_a / W * g$$

Resulting kinetic energy ;

$$(\text{substituting } v = i * g / W) ; KE = (I_a^2 / 2 * W) * g$$

Substituting KE = PE ;

$$W * \mu_d * \Delta = (I_a^2 / 2 * W) * g$$

$$W * \mu_d * \Delta = I_a^2 / 2 * M$$

Sliding displacement ;

$$\Delta = I_a^2 / (2 * W * M * \mu_d)$$

$$\Delta = 0.766 \text{ in}$$

Overtuning Analysis

Let,

$$\begin{aligned} \text{Angular rotation} &= \theta \\ \text{Angular velocity} &= \omega \\ \text{Weight} &= W \\ \text{Load duration} &= t_d \\ \text{Rotary inertia} &= I_{\alpha} \\ \text{Uplift at c.g} &= y_o \\ \text{Rotation} &= \theta \end{aligned}$$

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Job No. : BI 3039

Date 1/18/2004

Project : ABU ALI SUBSTATION 1 & 2

By : MGG

Subject : DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 1)

Chck'd by: SAK

$$\begin{aligned} \text{Applied Force} &= F \\ \text{Kinetic Energy ; } KE &= 1/2 * I_{aa} * \omega^2 \\ \text{Uplift at c.g } y_o &= y_o = (B_f / 2) * \tan\theta \end{aligned}$$

Net work done for a body initially at rest

$$\begin{aligned} PE &= W * y_o \\ PE &= W * (B_f / 2) * \tan\theta \\ \text{Applied moment ; } M &= F * H_b / 2 \\ \text{Applied angular impulse ; } I_{aa} &= M * t_d \end{aligned}$$

Initial rotational velocity of a body given this impulse ;

$$\omega = I_{aa} / I_a$$

Resulting kinetic energy

$$\text{(substituting } \omega = I_{aa} / I_a); KE = I_{aa}^2 / (2 * I_a)$$

Substituting KE = PE ;

$$W * B_f / 2 * \tan\theta = I_{aa}^2 / (2 * I_a)$$

Solve for Sliding displacement,

$$\begin{aligned} \tan\theta &= I_{aa}^2 / (2 * I_a) * 2 / (W * B_f) \\ \theta &= \tan^{-1} (I_{aa}^2 / (I_a) * 1 / (W * B_f)) \end{aligned}$$

Reflected overturning impulse ;

$$\begin{aligned} I_{ara} &= I_{ar} * H_b / 2 \\ &= 27660711 \text{ lbs-sec/in} \end{aligned}$$

Rotation angle

$$\begin{aligned} \theta &= \tan^{-1} (I_{aa}^2 / (I_a) * 1 / (W * B_f)) \\ \theta &= 0.000085 \text{ radians} \end{aligned}$$

Uplift at far edge of foundation ;

$$\begin{aligned} y_{max} &= B_f * \tan\theta \\ &= 0.060 \text{ inches} < 25 \text{ inches ; ok} \end{aligned}$$

Saudconsult

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Job No. : BI 3039
 Project : ABU ALI SUBSTATION 1 & 2
 Subject : DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 2)

Date : 1/18/2004
 By : MGG
 Chck'd by: SAK

INPUT PARAMETERS

Physical Constants:

Acceleration of gravity	$g =$	386.4	in/sec ²
Co-efficient of sliding friction	$\mu d =$	0.35	
Unit weight of concrete	$\gamma_{conc} =$	150	lbs/ft ³
Blast wave reflection coefficient	$Cra =$	2.1	

Physical Parameters of the Building:

Length of target surface on the building	$L_b =$	156	ft	1870	in
Height of target surface on the building	$H_b =$	24.1	ft	289	in
Width of the building	$B_b =$	32.8	ft	394	in
Weight of roofing	$Roofwt =$	1.2	psi	8.21	kpa
Weight of wall	$Wallwt =$	1.08	psi	7.44	kpa

Operating weight of the building

$$W_b = Roofwt * L_b * B_b + Wallwt * (2 * L_b + 2 * B_b) * H_b = 2297776.69 \text{ lbs}$$

Height of Center of gravity above the base

of the building module (estimated) ; $H_b / 4 = H_{bcg} = 72.3 \text{ in}$

Physical Parameters of the Foundation:

Foundation length ; L_b	$L_f =$	1870	in
Foundation depth ; 0.9 m.	$H_f =$	29.5	ft 354 in
Foundation width ; B_b	$B_f =$	394	in
Other foundation dimensions ;	$bf =$	98.4	in
	$e =$	0	in
	$ts =$	17.7	in

Design Loads for the Building

Reflected pressure	$Par =$	4.2	lbs/in ²
Reflected impulse per unit urea	$iar =$	0.42	lbs-sec/in ²
Target surface area ; A	$H_b * L_b =$	540853.164	in ²
Reflected force on target surface ; Far	$Par * A =$	2271583.29	lbs
Reflected impulse on target surface ; lar	$iar * A =$	227158.329	lbs-sec

CALCULATED PARAMETERS

Building Properties

Mass of building ;	$W_b/g =$	$M_b =$	5946.63	lbs-sec/in ²
Center of gravity location with respect to the origin ;	$B_f / 2 =$	$X_{cg} =$	196.86	in
	$H_{bcg} + H_f =$	$Y_{cg} =$	426.648	in

Job No. : BI 3039

Date : 1/18/2004

Project : ABU ALI SUBSTATION 1 & 2

By : MGG

Subject : DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 2)

Chck'd by: SAK

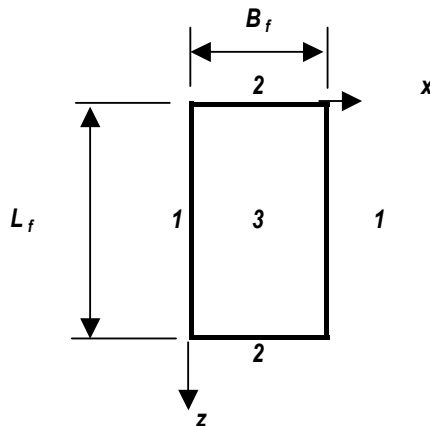
Rotation arm about Point A ;

$$db = \sqrt{(B_f - X_{cg})^2 + Y_{cg}^2} = 469.87 \text{ in}$$

Rotary inertia of the building about Point A ;

$$M_b * d_b^2 = I_{Aab} = 1.313E+09 \text{ lbs-sec-in}$$

Foundation Properties



Average width of edge beams (components nos. 1 and 2)

$$b_{avg} = 0.5 * (b_f + (b_f + e)) = 98.43 \text{ in}$$

Depth of edge beams (Component Nos. 1 and 2)

$$db = H_f - t_s = 336.63 \text{ in}$$

Length of edge beams

$$L1 = L_f = 1870.17 \text{ in}$$

$$L2 = Bf - 2 * b_{avg} = 196.86 \text{ in}$$

Weights of each component

$$Wf1 = \gamma_{conc} * (2 * (L_1 * b_{avg} * d_b)) / 1728 = 10758229.2 \text{ lbs}$$

$$Wf2 = \gamma_{conc} * (2 * (L_2 * b_{avg} * d_b)) / 1728 = 1132445.18 \text{ lbs}$$

$$Wf3 = \gamma_{conc} * (L_f * Bf * t_s) / 1728 = 1132387.01 \text{ lbs}$$

Total foundation weight ;

$$Wf = W_{f1} + W_{f2} + W_{f3} = 13023061.4 \text{ lbs}$$

Total foundation mass

$$Mf = W_f / g = 33703.58 \text{ lbs-sec/in}^2$$

Moment arm from Pont A to Center of gravity

$$a1 = \sqrt{(d_b/2)^2 + (B_f - b_{avg}/2)^2} = 383.42 \text{ in}$$

$$a2 = \sqrt{(d_b/2)^2 + (B_f/2)^2} = 259.01 \text{ in}$$

$$a3 = \sqrt{(d_b + (t_s/2))^2 + (Bf/2)^2} = 397.64 \text{ in}$$

Rotary inertia about pivot at point A for each component treating each component as a rectangular prism rotating about a corner ;

$$IAf1 = W_{f1} * a_1^2 / g = 4.09E+09 \text{ lbs-sec/in}^2$$

$$IAf2 = 2 * 4/3 * W_{f2} / g * a_2^2 = 524285742 \text{ lbs-sec/in}^2$$

$$IAf3 = 4/3 * W_{f3} / g * a_3^2 = 617838697 \text{ lbs-sec/in}^2$$

Saudconsult

Sheet _____ of _____ 4

Job No. : BI 3039

Date 1/18/2004

Project : ABU ALI SUBSTATION 1 & 2

By : MGG

Subject : DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 2)

Chck'd by: SAK

Total rotary inertia of the foundation

$$IA_{af} = IA_{f1} + IA_{f2} + IA_{f3} = 5.235E+09 \text{ lbs-sec/in}^2$$

Combined System Properties

$$\text{Total weight ; } W = W_b + W_f = 1.53E+07 \text{ lbs}$$

$$\text{Total Mass ; } M = M_b + M_f = 39650.20 \text{ lbs-sec/in}^2$$

$$\text{Total rotary inertia } IA_a = I_{Aab} + I_{Aaf} = 6.548.E+09 \text{ lbs-sec/in}^2$$

SLIDING ANALYSIS

Work-Energy Equation:

Let,

$$\text{Sliding distance ; } = \Delta$$

$$\text{Velocity ; } = v$$

$$\text{Applied Force ; } = F$$

$$\text{Load duration ; } = t_d$$

$$\text{Mass, ; } W/g = M$$

$$\text{Kinetic energy ; } KE = 1/2 * W/g * V^2$$

Net work done for a body initially at rest

$$PE = W * \mu_d * \Delta$$

$$\text{Applied impulse, } I_a = F * t_d$$

Initial velocity of a body given this impulse ;

$$v = I_a / W * g$$

Resulting kinetic energy ;

$$(\text{substituting } v = i * g / W) ; KE = (I_a^2 / 2 * W) * g$$

Substituting KE = PE ;

$$W * \mu_d * \Delta = (I_a^2 / 2 * W) * g$$

$$W * \mu_d * \Delta = I_a^2 / 2 * M$$

Sliding displacement ;

$$\Delta = I_a^2 / (2 * W * M * \mu_d)$$

$$\Delta = 0.121 \text{ in}$$

Overtuning Analysis

Let,

$$\text{Angular rotation } = \theta$$

$$\text{Angular velocity } = \omega$$

$$\text{Weight } = W$$

$$\text{Load duration } = t_d$$

$$\text{Rotary inertia } = I_a$$

$$\text{Uplift at c.g } = y_o$$

$$\text{Rotation } = \theta$$

Saudconsult

Sheet _____ of 4

Job No. : BI 3039

Date 1/18/2004

Project : ABU ALI SUBSTATION 1 & 2

By : MGG

Subject : DYNAMIC SLIDING AND OVERTURNING ANALYSIS (S/S # 2)

Chck'd by: SAK

$$\begin{aligned} \text{Applied Force} &= F \\ \text{Kinetic Energy ; } KE &= 1/2 * I_{aa} * \omega^2 \\ \text{Uplift at c.g } y_o &= y_o = (B_f / 2) * \tan\theta \end{aligned}$$

Net work done for a body initially at rest

$$\begin{aligned} PE &= W * y_o \\ PE &= W * (B_f / 2) * \tan\theta \\ \text{Applied moment ; } M &= F * H_b / 2 \\ \text{Applied angular impulse ; } I_{aa} &= M * t_d \end{aligned}$$

Initial rotational velocity of a body given this impulse ;

$$\omega = I_{aa} / I_a$$

Resulting kinetic energy

$$\text{(substituting } \omega = I_{aa} / I_a); KE = I_{aa}^2 / (2 * I_a)$$

Substituting KE = PE ;

$$W * B_f / 2 * \tan\theta = I_{aa}^2 / (2 * I_a)$$

Solve for Sliding displacement,

$$\begin{aligned} \tan\theta &= I_{aa}^2 / (2 * I_a) * 2 / (W * B_f) \\ \theta &= \tan^{-1} (I_{aa}^2 / (I_a) * 1 / (W * B_f)) \end{aligned}$$

Reflected overturning impulse ;

$$\begin{aligned} I_{ara} &= I_{ar} * H_b / 2 \\ &= 32847094.4 \text{ lbs-sec/in} \end{aligned}$$

Rotation angle

$$\begin{aligned} \theta &= \tan^{-1} (I_{aa}^2 / (I_a) * 1 / (W * B_f)) \\ \theta &= 0.000027 \text{ radians} \end{aligned}$$

Uplift at far edge of foundation ;

$$\begin{aligned} y_{max} &= B_f * \tan\theta \\ &= 0.011 \text{ inches} < 25 \text{ inches ; ok} \end{aligned}$$

APPENDIX B

Connection Design Calculations

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "1 " OF BASIC DRAWING OF 979-BD-SB1/SB2-07

STEEL PLATES CONNECTOR OF SHEAR WALL INSULATED PANELS

CONSIDER E50 (EMBEDDED PLATE , SEE FILE OF ACCESSORIES SHOP DRAWINGS)

APPLIED UPWARD SHEAR FORCE PER FIXING = $644 / 5 = 128.8$ KN

DUE TO THE DOOR OPENING IN SHEARWALL SIDE, ADDITIONAL LOAD ON FIXING IS
ADDED DUE TO ABSENCE OF 6 PCS. FIXINGS DUE TO DOOR OPENINGS

ADDITIONAL UPWARD FORCE AT CORNER FIXING OF SHEARWALL & NON-SHEARWALL:
 $= 6 (128.8) (5..35) / 10.64 / 14 = 28$ KN FOR EACH FIXING (MINIMAL)

USING GRADE 60 WELDING ELECTRODE

E60XX $f_t = 60$ ksi (414 MPa)

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR. (SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3$ (ft)

$f_s = 124$ MPa

$f_{sd} = f_s (SIF)(DIF) = (124)(1.1)(1.19) = 162.32$ Mpa

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = (320)(1.1)(1.19) = 418.90$ MPa

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128$ MPa

$f_{vd} = 128 (1.1)(1.19) = 167.60$ MPa

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE STRESS = 420 MPa

$f_{ybd} = 420 (1.1)(1.19) = 549.78$ MPa

ALLOWABLE SHEAR STRESS OF BOLT = $420 (.4) = 168$ MPa

$f_{vbd} = 168 (1.1) (1.19) = 219.91$ MPa

USING 6- DIA. 12MM ANCHOR STUDS & 8MM WELD SIZE

CHECK FOR SHEAR CAPACITY OF WELD:

Weld size = 8mm , $t = \text{throat}$ $t = \cos 45 (8) = 5.65$ mm,

WELD SHEAR CAP. = $5.65 \times 2 \times 3.1416 \times 6 \times 162.32 \times 1 / 1000 = 207.48$ KN > 128.8 KN
THEREFORE OK.

SHEAR CAP. OF DIA 12MM ANCHOR STUDS 6 NOS.

STUD SHEAR CAP. = $3.1416 (12)^{2(1/4)}(6)(219.91)(1/1000) = 149.23 \text{ KN} > 184 \text{ KN}$
THEREFORE OK.

DETAIL # 1 OF BASIC DRAWING 979-BD-SB1/SB2-07

SHEARWALL STEEL PLATE CONNECTOR

CONSIDER E50A (BY MAIN CONTRACTOR) (EMBEDDED IN CIP SLAB SEE BASIC DRAWING)

UPWARD FORCE OF 644 KN PER PANEL.

TENSILE CAPACITY OF 12MM DIA. ANCHOR STUDS

TENSILE FORCE ON 4- DIA. 12MM STUDS = $644/5 \times 125 \times 1/1.1 = 161 \text{ KN}$

TENSILE CAP. OF 4- DIA. 12MM STUD = $113 (549.78)(1/1000)(4) = 248.5 \text{ KN} > 161 \text{ KN}$
THEREFORE OK

USING 10MM WELD SIZE , THROAT $t = \cos 45 (10) = 7.07 \text{ MM}$

SHEAR CAP. OF 10MM WELD = $2 \times 3.1416 \times 6 \times 7.07 \times 4 \times 162.32 \times 1/1000 = 173.1 \text{ KN} > 161 \text{ KN}$
THEREFORE OK.

DETAIL # 1 OF BASIC DRAWING 979-BD-SB1/SB2-07

APPLIED HORIZONTAL SHEAR FORCE PER FIXING = $428.81/5 = 85.76 \text{ KN}$

USING 8- DIA 12MM ANCHOR STUDS & 10MM WELD SIZE

CHECK FOR SHEAR CAPACITY OF WELD

WELD SIZE = 10mm , $t = \text{throat}$ $t = \cos 45 (10) = 7.07 \text{ mm}$:

WELD SHEAR CAP. = $7.07 \times 2 \times 3.1416 \times 6 \times 8 \times 162.32 \times 1/1000 = 345.58 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF DIA. 12MM ANCHOR STUDS 8 NOS

STUD SHEAR CAP. = $3.1416 \times 12^2 \times 1/4 \times 8 \times 219.91 \times 1/1000 = 199 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK

SHEAR CAPACITY OF ACC-01 (FIXING ACCESSORY FOR DETAIL 1)

WELD SIZE OF BENT PLATE ACC-01 = 8MM , $t = \cos 45 (8) = 5.65 \text{ MM}$

L = LENGTH OF WELD FOR BENT PLATE ACC-01 125X125X12MM THK. = 450 MM

SHEAR CAP. OF WELD = $5.65 \times 450 \times 162.32 \times 1/1000 = 413 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK.

HOR. SHEAR CAP. OF ACC-01 (125X125X12MM THK. X 200MM LONG)

SHEAR CAP. OF BENT PLATE = $12 \times 200 \times 167.60 \times 1/1000 = 402.24 \text{ KN} > 85.76 \text{ KN}$

SHEAR CAP. OF BENT PLATE = $12 \times 200 \times 167.60 \times 1/1000 = 402.24 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK.

DETAIL # 1 OF 979-BD-SB1/SB2-07

CONSIDER ACC-01 125X125X12MM THK. X 200MM LONG ACCESSORY

UPWARD FORCE = $644 / 5 = 128.8 \text{ KN PER FIXING}$

UPWARD SHEAR CAP. OF 8MM SIZE WELD , $t = \cos 45 (8) = 5.65 \text{ MM}$

I of weld of ACC-O1 = $200 \times 5.65^3 / 12 (2) + 200 \times 5.65 \times 62.5^2 (2) + 2(5.65 \times 125^3 / 12)$
 $= 10.673 \times 10^6 \text{ MM}^4$

S (section modulus) = $10.673 \times 10^6 / 62.5 = 170773 \text{ MM}^3$

M = MOMENT INDUCED DUE TO ECCENTRIC LOADING
 $= 128.8 (.0625) = 8.05 \text{ KN.M}$

SHEAR CAP. OF 8MM WELD ON ACC-01 ACCESSORY
 $= 128.8 (1000) / 5.65 (450) + M/S$
 $= 128.8 (1000) / 5.65 (450) + 8.05 \times 10^6 / 170773$
 $= 97.8 \text{ MPa} < 162.32 \text{ MPa}$, therefore OK.

DETAIL # 1 OF 979-BD-SB1/SB2-07

CHECK FOR THE UPWARD SHEAR FORCE WALL LOAD = 644 KN PER PANEL

FOR ACC-01 UPWARD SHEAR FORCE PER FIXING = $644 / 5 = 128.8 \text{ KN}$

SHEAR CAPACITY OF 8MM WELD , $t = \cos 45 (8) = 5.65 \text{ MM}$

L = LENGTH OF WELD , $L = 105 \times 2 + 200 = 410 \text{ MM}$

SHEAR CAP. OF 8MM WELD = $5.65 (410) (162.32) (1/1000) = 376.30 \text{ KN} > 128.8 \text{ KN}$
THEREFORE OK

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "14 " OF BASIC DRAWING OF 979-BD-SB2-07 2/2

TENSILE STEEL PLATES CONNECTOR AT TOP OF DOUBLE TEE SLAB

CONSIDER E64 EMBEDS , SUBSTATION 2 (SEE SHOP DRAWINGS)

APPLIED HORIZONTAL TENSILE FORCE PERPENDICULAR TO END OF DT = 2342.78KN
(SEE CALCULATION DATA SHEET ATTACHED)

USING GRADE 60 WELDING ELECTRODE

E60XX $f_t = 60 \text{ ksi}$ (414 MPa)

SIF = 1.00 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (REF. SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3 f_t$

$f_s = 124 \text{ MPa}$

$f_{sd} = f_s (SIF)(DIF) = (124)(1.0)(1.19) = 147.60 \text{ MPa}$

STEEL PLATE :

$f_u = 504 \text{ MPa}$ (f_u = STRUCTURAL STEEL PLATE ULTIMATE STRESS)

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = 320 (1.0)(1.12) = 358.40 \text{ MPa}$

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128 \text{ MPa}$

$f_{vd} = 128(1.0)(1.19) = 152.32 \text{ MPa}$

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE (YIELD) STRESS = 420 MPa

$f_{ybd} = 420(1.0)(1.12) = 470.40 \text{ MPa}$

ALLOWABLE SHEAR STRESS = $420 (.4) = 168 \text{ MPa}$

$f_{vbd} = 168 (1.0)(1.19) = 199.92 \text{ MPa}$

USING 3 PCS. 200 X 12MM THK. STEEL PLATES (E64) at SB2

TOTAL APPLIED TENSILE FORCE ON THREE LINE OF STEEL PLATES = 2342.78 KN
(SEE CALCULATION DATA SHEET ATTACHED)

TENSILE CAPACITY OF E64 STEEL PLATES

TENSILE CAP. = $12 \times 200 \times 358.40 \times 1/1000 \times 3 = 2580.48 \text{ KN} > 2342.78 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF WELD OF 12 - 20mm dia. STUDS FOR E64 (10MM WELD SIZE).

Weld size = 10mm , $t = \text{throat} = \cos 45 (10) = 7.1 \text{ mm}$,
WELD SHEAR CAP. = $7.1 \times 3.1416 \times 2 \times 10 \times 12 \times 147.60 \times 3 \times 1/1000 = 2370.43 \text{ KN} > 2342.78 \text{ KN}$
THEREFORE OK.

DETAIL # 14 OF BASIC DRAWING 979-BD-SB2-07 2/2

TENSILE PLATE CONNECTOR AT TOP OF DT

CONSIDER E-64 EMBEDS WITH 12- 20mm DIA. STUD ANCHOR

HOR. TENSILE FORCE = 2342.78 KN

CHECK SHEAR CAPACITY OF 12 PCS.DIA. 20 MM STUD OF E64 EMBEDS

SHEAR CAPACITY OF 12- 20 MM DIA. ANCHOR STUDS

SHEAR CAP. OF 12- 20MM DIA. STUDS ~~314~~ $(219.91) 12(3)(1/1000)$

$= 2259.90 \text{ KN} < 2342.78 \text{ KN}$, THEREFORE NOT OK

The above calculation check for shear in anchor studs is failing, however it is safe if we use SIF = 1.1 as it was used in ABS calculations in steel precast connection design. See ABS calculations attached.

USING 10M WELD SIZE ALL AROUND ACC-09 STEEL PLATES OF SB2

CHECK FOR SHEAR CAPACITY OF 10MM WELD SIZE

THROAT $t = \cos 45 (10) = 7.1 \text{ MM}$, $L = 167 + 298 \times 2 = 763 \text{ MM}$

SHEAR CAPACITY OF 10MM WELD SIZE OF ACC-09. = $7.1 (763)(147.60)(1/1000)(3)$
 $= 2399.00 \text{ KN} > 2342.78 \text{ KN}$
THEREFORE OK

TENSILE CAPACITY OF PLATE ACC-09 OF SB2 (16MM X 200MM STEEL PLATE)

TENSILE PLATE CAPACITY = $16 \times 200 \times 358.40 \times 1/1000 \times 3 = 3440.64 \text{ KN} > 2342.78 \text{ KN}$
THEREFORE OK

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "14 " OF BASIC DRAWING OF 979-BD-SB2-07 2/2

TENSILE STEEL PLATES CONNECTOR AT TOP OF DOUBLE TEE SLAB

CONSIDER E64 EMBEDS , SUBSTATION 2 (SEE SHOP DRAWINGS)

APPLIED HORIZONTAL TENSILE FORCE PERPENDICULAR TO END OF DT = 2342.78KN
(SEE CALCULATION DATA SHEET ATTACHED)

USING GRADE 60 WELDING ELECTRODE

E60XX $f_t = 60 \text{ ksi } (414 \text{ MPa })$

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (REF. SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3 f_t$

$f_s = 124 \text{ MPa}$

$f_{sd} = f_s (SIF)(DIF) = (124)(1.1)(1.19) = 162.32 \text{ MPa}$

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = 320 (1.1)(1.19) = 418.90 \text{ MPa}$

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128 \text{ MPa}$

$f_{vd} = 128(1.1)(1.19) = 167.60 \text{ MPa}$

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE (YIELD) STRESS = 420 MPa

$f_{ybd} = 420(1.1)(1.19) = 549.78 \text{ MPa}$

ALLOWABLE SHEAR STRESS = $420 (.4) = 168 \text{ MPa}$

$f_{vbd} = 168 (1.1)(1.19) = 219.91 \text{ MPa}$

USING 3 PCS. 200 X 12MM THK. STEEL PLATES (E64) at SB2

TOTAL APPLIED TENSILE FORCE ON THREE LINE OF STEEL PLATES = 2342.78 KN
(SEE CALCULATION DATA SHEET ATTACHED)

TENSILE CAPACITY OF E64 STEEL PLATES

TENSILE CAP. = $12 \times 200 \times 418.90 \times 1/1000 \times 3 = 3016.10 \text{ KN} > 2342.78 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF WELD OF 12 - 20mm dia. STUDS FOR E64 (10MM WELD SIZE).

Weld size = 10mm , $t = \text{throat} \quad t = \cos 45 (10) = 7.1 \text{ mm},$
WELD SHEAR CAP. = $7.1 \times 3.1416 \times 2 \times 10 \times 12 \times 162.32 \times 3 \times 1/1000 = 2606.83 \text{ KN} > 2342.78 \text{ KN} ,$
THEREFORE OK.

DETAIL # 14 OF BASIC DRAWING 979-BD-SB2-07 2/2

TENSILE PLATE CONNECTOR AT TOP OF DT

CONSIDER E-64 EMBEDS WITH 12- 20mm DIA. STUD ANCHOR

HOR. TENSILE FORCE = 2342.78 KN

CHECK SHEAR CAPACITY OF 12 PCS.DIA. 20 MM STUD OF E64 EMBEDS

SHEAR CAPACITY OF 12- 20 MM DIA. ANCHOR STUDS

SHEAR CAP. OF 12- 20MM DIA. STUDS = $314 (219.91) 12(3)(1/1000)$
= 2487.8 KN > 2342.78 KN , THEREFORE OK

USING 10M WELD SIZE ALL AROUND ACC-09 STEEL PLATES OF SB2

USING 10M WELD SIZE ALL AROUND ACC-09 STEEL PLATES OF SB2

CHECK FOR SHEAR CAPACITY OF 10MM WELD SIZE

THROAT $t = \cos 45 (10) = 7.1 \text{ MM}$, $L = 160 + 290 \times 2 = 740 \text{ MM}$

SHEAR CAPACITY OF 10MM WELD SIZE OF ACC-09. $= 7.1 (740)(162.32)(1/1000)(3)$
 $= 2558.50 \text{ KN} > 2342.78 \text{ KN}$
THEREFORE OK

TENSILE CAPACITY OF PLATE ACC-09 OF SB2 (16MM X 200MM STEEL PLATE)

TENSILE PLATE CAPACITY $= 16 \times 200 \times 324.63 \times 1/1000 \times 3 = 3118.84 \text{ KN} > 2342.78 \text{ KN}$
THEREFORE OK

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "13 " OF BASIC DRAWING OF 979-BD-SB1/SB2-07 2/2

STEEL PLATES CONNECTOR END OF DOUBLE TEE TO INSULATED PANEL

CONSIDER E61 EMBEDS , (SEE SHOP DRAWINGS)

APPLIED HORIZONTAL SHEAR FORCE PERPENDICULAR TO PANEL = 181 KN/ Panel
USING GRADE 60 WELDING ELECTRODE

E60XX ft = 60 ksi (414 MPa)

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (REF. SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3 f_t$

$f_s = 124 \text{ MPa}$

$f_{sd} = f_s (SIF)(DIF) = (124)(1.1)(1.19) = 162.32 \text{ MPa}$

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = 320 (1.1)(1.19) = 418.90 \text{ MPa}$

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128 \text{ MPa}$

$f_{vd} = 128(1.1)(1.19) = 167.60 \text{ MPa}$

ALLOWABLE BENDING STRESS OF STEEL PLATE

$f_{bd} = 320(0.6)(1.1)(1.19) = 251.33 \text{ MPa}$

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE STRESS = 420 MPa

$f_{ybd} = 420(1.1)(1.19) = 549.78 \text{ MPa}$

ALLOWABLE SHEAR STRESS = $420 (.4) = 168 \text{ MPa}$

$f_{vbd} = 168 (1.1)(1.19) = 219.91 \text{ MPa}$

USING 50 X 150 X 6MM THK. STEEL BENT PLATE (E61)

APPLIED SHEAR PER FXING AT 500MM C/C = $181/5 = 36.20 \text{ KN}$

SHEAR CAPACITY OF E61 STEEL BENT PLATE ANCHOR STUDS 4- 12mm DIA.

SHEAR CAP. OF 12MM DIA. STUDS = $3.1416(12^2)(1/4)219.91(1/1000)(4) = 99.5 \text{ KN} > 32.6 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF WELD OF STUDS FOR E61 (6MM WELD SIZE)

Weld size = 6mm , $t = \text{throat}$ $t = \cos 45 (6) = 4.24 \text{ mm}$,

WELD SHEAR CAP. = $4.24 \times 3.1416 \times 2 \times 6 \times 4 \times 162.32 \times 1/1000 = 103.86 \text{ KN} > 32.6 \text{ KN}$
THEREFORE OK.

DETAIL # 13 OF BASIC DRAWING 979-BD-SB1/SB2-07 2/2

END OF DOUBLE TEE TO INSULATED PANEL CONNECTION

USING 50 X 150 X 6MM THK. STEEL BENT PLATE (E61)
APPLIED SHEAR PER FIXING AT 500MM C/C = $181/5 = 36.20$ KN

SHEAR CAPACITY OF E61 STEEL BENT PLATE ANCHOR STUDS 4- 12mm DIA.
SHEAR CAP. OF 12MM DIA. STUDS = $3.1416(12^2)(1/4)219.91(1/1000)(4) = 99.5$ KN > 32.6KN
THEREFORE OK.

SHEAR CAPACITY OF WELD OF STUDS FOR E61 (6MM WELD SIZE)

Weld size = 6mm , t = throat t = $\cos 45 (6) = 4.24$ mm,

WELD SHEAR CAP. = $4.24 \times 3.1416 \times 2 \times 6 \times 4 \times 162.32 \times 1/1000 = 103.86$ KN > 32.6 KN
THEREFORE OK.

DETAIL # 13 OF BASIC DRAWING 979-BD-SB1/SB2-07 2/2

END OF DOUBLE TEE TO INSULATED PANEL CONNECTION

CONSIDER E57 EMBEDS

HOR. SHEAR FORCE OF 181 KN PER PANEL = 36.2 KN PER FIXING.

CHECK SHEAR CAPACITY OF 4 PCS.DIA. 12 MM STUD OF E57 EMBEDS

USING 6MM WELD SIZE , THROAT, t = $\cos 45 (6) = 4.24$ MM

SHEAR CAPACITY OF WELD OF 4- DIA ANCHOR STUDS:

SHEAR CAP. OF 6MM WELD = $2 \times 3.1416 \times 6 \times 4.24 \times 4 \times 162.32 \times 1/1000 = 103.86$ KN > 36.2KN
THEREFORE OK.

USING 4- DIA. 12MM ANCHOR STUDS, CHECK FOR TENSILE CAPACITY

CHECK FOR TENSILE CAPACITY OF 4 -12MM DIA. ANCHOR STUDS:

TENSILE CAPACITY OF 12MM DIA. = $3.1416(12^2)(1/4)4(549.78)(1/1000)$
= 248.70 KN > 36.2 KN

THEREFORE OK

CHECK FOR THE BENDING OF PLATE E57

$M = \{(.09-.01)/.09(36)(.01)\}-.02 = 0.12$ KN.M

S = SECTION MODULUS

$S = 175 (6)^2/6 = 1050$ MM ^3

$F_b = M/S$

$F_b = 0.12 \times 10^6 / 1050 = 114$ MPa < 251.33 MPa

THEREFORE OK

CHECK FOR THE BENDING OF PLATE E57

$$M = \{(.09-.01)/.09(36)(.01)\}-.02 = 0.12 \text{ KN.M}$$

S = SECTION MODULUS

$$S = 175 (6)^{2/6} = 1050 \text{ MM}^3$$

$$F_b = M/S$$

$$F_b = 0.12 \times 10^6 / 1050 = 114 \text{ MPa} < 251.33 \text{ MPa}$$

THEREFORE OK

CHECK FOR THE SHEAR CAPACITY OF CONCRETE AT E57 FIXING

$$V_c = .85 F'_c{}^{1/2} (b)d (1/6) (1/1000)$$

$$F'_c = 35 \text{ MPa}$$

$$F'_{cd} = F'_c (SIF)(DIF)$$

$$F'_{cd} = 35 (1.1)(1.19) = 45.82 \text{ MPa}$$

$$b = 120(2) + 100(2) = 440 \text{ mm}$$

$$d = 160 \text{ MM}$$

$$V_c = .85 (45.82^{1/2})(440)(160) (1/6)(1/1000)$$

$$V_c = 67.51 \text{ KN} > 36 \text{ KN}$$

THEREFORE OK

DET. "13" , CONSIDER E62/ E62A EMBEDS

END OF DOUBLE TEE TO PANEL CONNECTION

APPLIED HORIZONTAL SHEAR PER WEB = 45.26 KN

SHEAR CAPACITY OF 6MM WELD OF OF PLATE EMBEDS E 62/ E62A

$$\text{WELD SIZE} = 6\text{MM} \quad , \quad \text{THROAT } t = \cos 45^\circ 6 = 4.24 \text{ MM}$$

SHEAR CAP. OF WELD OF 3+3 PCS. 12MM DIA. STUDS

$$= 4.24 \times 2 \times 3.1416 \times 6 \times 6 \times 162.32 \times 1/1000$$

$$= 155.68 \text{ KN} > 45.26 \text{ KN} \quad , \text{ THEREFORE OK}$$

SHEAR CAPACITY OF 3+3- 12MM DIA. STUDS FOR E62 /E62A

$$= 113 (219.91) (6) (1/1000) = 149.21 \text{ KN} > 45.26 \text{ KN} \quad , \text{ THEREFORE OK}$$

SHEAR CAP. OF WELD OF 3+3 PCS. 12MM DIA. STUDS
= $4.24 \times 2 \times 3.1416 \times 6 \times 162.32 \times 1/1000$
= 155.68 KN > 45.26 KN , THEREFORE OK

SHEAR CAPACITY OF 3+3- 12MM DIA. STUDS FOR E62 /E62A

= $113 (219.91) (6) (1/1000) = 149.21 \text{ KN} > 45.26 \text{ KN}$, THEREFORE OK

END OF DOUBLE TEE TO PANEL CONNECTION

APPLIED HORIZONTAL SHEAR FORCE PER WEB = 45.26 KN

SHEAR CAPACITY OF 6MM WELD OF 4- 12MM DIA STUDS OF E58 EMBEDS

WELD SIZE = 6 MM THROAT $t = \cos 45 (6) = 4.24 \text{ MM}$

SHEAR CAP. OF WELD = $2 \times 3.1416 \times 6 \times 4.24 \times 4 \times 162.32 \times 1/1000 = 103.86 \text{ KN} > 45.26 \text{ KN}$
THEREFORE OK.

TENSILE CAPACITY OF 4- 12MM DIA. STUDS OF E58 EMBEDS

TENSILE CAP. OF STUDS = $113 (549.78) (4) (1/1000) = 248.69 \text{ KN} > 45.26 \text{ KN}$
THEREFORE OK

TENSILE CAPACITY OF ACC-08 (100x150x6MM THK. STEEL PLATE)

TENSILE CAPACITY OF PLATE = $150 (6) 418.90 (1/1000) = 377.01 \text{ KN} > 45.26 \text{ KN}$, OK

SHEAR CAPACITY OF WELD CONNECTING E58 & ACC-08, WELD SIZE=6MM , $t = 4.24$
SHEAR CAP. = $150 (4.24) (162.32) (1/1000) = 103.31 \text{ KN} > 45.26 \text{ KN}$, OK

CHECK FOR THE BENDING CAPACITY OF E58 EMBED PLATE

$M = 45.26 / 2 \{ (200-190) / 2 / 1000 \} = 0.113 \text{ KN.M}$

FLEXURAL STRESS $F_b = M / S$

$S = 200 (6^2) / 6 = 1200 \text{ MM}^3$

$F_b = 0.113 \times 10^6 / 1200 = 94.2 \text{ MPa} < 251.33 \text{ MPa}$
THEREFORE , OK

$$M = 45.26/2 \{(200-190)/2/1000\} = 0.113 \text{ KN.M}$$

$$\text{FLEXURAL STRESS } F_b = M/S$$

$$S = 200 (6^2)/6 = 1200 \text{ MM}^3$$

$$F_b = 0.113 \times 10^6 / 1200 = 94.2 \text{ MPa} < 251.33 \text{ MPa}$$

THEREFORE , OK

CHECK FOR THE CONCRETE PULL OUT SHEAR OF CONCRETE

$$V_c = \text{CONCRETE SHEAR CAPACITY}$$

$$V_c = .85 (F'_{cd}) b d (1/6) (1/1000)$$

$$b = 130(2) + 200 (2) = 660 \text{ mm}$$

$$d = 150 \text{ MM}$$

$$V_c = .85 (45.82)^{1/2} (660) (150) (1/6) (1/1000)$$
$$= 94.94 \text{ KN} > 45.26 \text{ KN}, \text{ THEREFORE OK}$$

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "12 " OF BASIC DRAWING OF 979-BD-SB1/SB2-07 2/2

STEEL PLATES CONNECTOR OF DOUBLE TEE TO DOUBLE TEE FLANGE.

CONSIDER ACC-07 ACCESSORIES , (SEE SHOP DRAWINGS)

APPLIED HORIZONTAL SHEAR FORCE PER METER = 130 KN/M
USING GRADE 60 WELDING ELECTRODE

E60XX $f_t = 60 \text{ ksi}$ (414 MPa)

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (REF. SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3 f_t$

$f_s = 124 \text{ MPa}$

$f_{sd} = f_s (SIF) (DIF) = 124(1.10)(1.19) = 162.32 \text{ MPa}$

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = 320 (1.10)(1.19) = 418.90 \text{ MPa}$

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128 \text{ MPa}$

$f_{vd} = 128 (1.1)(1.19) = 167.60 \text{ MPa}$

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE STRESS = 420 MPa

$f_{ybd} = 420 (1.1)(1.19) = 549.78 \text{ MPa}$

ALLOWABLE SHEAR STRESS = $420 (0.4) = 168 \text{ MPa}$

$f_{vbd} = 168 (1.1)(1.19) = 219.91 \text{ MPa}$

USING 100 X 150 X 6MM THK. STEEL PLATE (ACC-07)

APPLIED SHEAR PER FIXING AT 500MM C/C = $130/2 = 65.0 \text{ KN}$

SHEAR CAPACITY OF ACC-07 STEEL PLATE ACCESSORY :

SHEAR CAP. OF 6MM STEEL PLATE = $6 (150) (167.60)(1/1000) = 150.84 \text{ KN} > 65 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF ACC-07 WELD (6MM WELD SIZE)

Weld size = 6mm , t = throat t = $\cos 45 (6) = 4.24$ mm,

L= LENGTH OF WELD = $40 \times 2 + 150 = 230$ MM

WELD SHEAR CAP. = $4.24 \times 230 \times 162.32 \times 1/1000 = 158.41$ KN > 65 KN
THEREFORE OK.

DETAIL # 12 OF BASIC DRAWING 979-BD-SB1/SB2-07 2/2

DOUBLE TEE TO DOUBLE TEE FLANGE CONNECTION

CONSIDER E60 EMBEDS

HOR. SHEAR FORCE OF 130 KN PER METER = 65 KN PER FIXING.

CHECK SHEAR CAPACITY OF 3 PCS.DIA. 12 MM STUD OF E60 EMBEDS

USING 6MM WELD SIZE , THROAT, t = $\cos 45 (6) = 4.24$ MM

SHEAR CAPACITY OF WELD OF 3 DIA ANCHOR STUDS:

SHEAR CAP. OF 6MM WELD = $2 \times 3.1416 \times 6 \times 4.24 \times 3 \times 162.32 \times 1/1000 = 77.91$ KN > 65KN
THEREFORE OK.

USING 3- DIA. 12MM ANCHOR STUDS WELD AT E60 BENT PLATE

CHECK FOR SHEAR CAPACITY OF 3 -12MM DIA. ANCHOR STUDS:

SHEAR CAPACITY OF 12MM DIA. = $3.1416 (12^2) (1/4) 3 (219.91) 1/1000$
= 74.61 KN > 65.0 KN

THEREFORE OK

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "11 " OF BASIC DRAWING OF 979-BD-SB1/SB2-07

STEEL PLATES CONNECTOR OF SHEARWALL INSULATED PANELS TO DOUBLE TEE

CONSIDER ACC-05 ACCESSORIES , (SEE SHOP DRAWINGS)

APPLIED HORIZONTAL SHEAR FORCE PER PANEL = 428 KN/M
USING GRADE 60 WELDING ELECTRODE

E60XX $f_t = 60 \text{ ksi}$ (414 MPa)

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3 f_t$

$f_s = 124 \text{ MPa}$

$f_{sd} = f_s (SIF)(DIF) = 124 (1.1)(1.19) = 162.32 \text{ MPa}$

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = 320 (1.1)(1.19) = 418.90 \text{ MPa}$

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128 \text{ MPa}$

$f_{vd} = 128 (1.1)(1.19) = 167.60 \text{ MPa}$

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE STRESS = 420 MPa

$f_{ybd} = 420 (1.1)(1.19) = 549.78 \text{ MPa}$

ALLOWABLE SHEAR STRESS = $420 (0.4) = 168 \text{ MPa}$

$f_{vbd} = 168 (1.1)(1.19) = 219.91 \text{ MPa}$

USING 125 X 125 X 8MM THK. STEEL PLATE (ACC-05)

APPLIED SHEAR PER FIXING AT 500MM C/C = $428/5 = 85.6 \text{ KN}$

SHEAR CAPACITY OF BOTH SIDES ACC-05 STEEL PLATE ACCESSORY

SHEAR CAP. OF 8MM STEEL PLATE = $8 (125) (167.60)(1/1000) (2) = 335.20 \text{ KN} > 85.6 \text{ KN}$

THEREFORE OK.

SHEAR CAPACITY OF ACC-05 WELD (8MM WELD SIZE)

Weld size = 8mm , $t = \text{throat}$ $t = \cos 45 (8) = 5.65 \text{ mm}$,

L= LENGTH OF WELD = $125 (2) = 250 \text{ MM}$

WELD SHEAR CAP. = $5.65 \times 250 \times 162.32 \times 1/1000 = 229.45 \text{ KN} > 85.6 \text{ KN}$

THEREFORE OK.

DETAIL # 11 OF BASIC DRAWING 979-BD-SB1/SB2-07

SHEAR CAP. OF 8MM STEEL PLATE = $8 (125) (167.60)(1/1000) (2) = 335.20 \text{ KN} > 85.6 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF ACC-05 WELD (8MM WELD SIZE)

Weld size = 8mm , t = throat t = $\cos 45 (8) = 5.65 \text{ mm}$,
L= LENGTH OF WELD = $125 (2) = 250 \text{ MM}$

WELD SHEAR CAP. = $5.65 \times 250 \times 162.32 \times 1/1000 = 229.45 \text{ KN} > 85.6 \text{ KN}$
THEREFORE OK.

DETAIL # 11 OF BASIC DRAWING 979-BD-SB1/SB2-07

INSULATED SHEARWALL PANEL TO DOUBLE TEE CONNECTION

CONSIDER E59 ASSEMBLY EMBEDDED IN DOUBLE TEE

HOR. SHEAR FORCE OF 428 KN PER PANEL = 85.6 KN PER FIXING.

CHECK SHEAR CAPACITY OF 4 PCS.DIA. 12 MM STUD OF E59 EMBEDS

USING 8MM WELD SIZE , THROAT, t = $\cos 45 (8) = 5.65 \text{ MM}$

SHEAR CAPACITY OF WELD OF 3 DIA ANCHOR STUDS:

SHEAR CAP. OF 8MM WELD = $2 \times 3.1416 \times 6 \times 5.65 \times 8 \times 162.32 \times 1/1000 = 276.8 \text{ KN} > 85.6 \text{ KN}$
THEREFORE OK.

USING 4- DIA. 12MM ANCHOR STUDS WELD AT E59 PLATE ASSEMBLY EMBEDS

CHECK FOR SHEAR CAPACITY OF 4 -12MM DIA. ANCHOR STUDS:

SHEAR CAPACITY OF 12MM DIA. = $3.1416 (12^2)(1/4) 8 (219.91) 1/1000$
= $199.12 \text{ KN} > 85.6 \text{ KN}$

THEREFORE OK

CONSIDER E56 STEEL PLATE EMBEDS 250X175X8MM THK. PLATE

SHEAR APPLIED FORCE = $428 / 5 = 85.6 \text{ KN}$

SHEAR CAPACITY OF WELD OF 6- 12MM DIA. ANCHOR STUDS

SHEAR CAPACITY OF 8MM WELD SIZE , t = $\cos 45 (8) = 5.65 \text{ MM}$

SHEAR CAPACITY OF WELD OF 6- 12MM DIA. ANCHOR STUDS

SHEAR CAPACITY OF 8MM WELD SIZE , $t = \cos 45 (8) = 5.65 \text{ MM}$

SHEAR CAP. OF WELD = $2 \times 3.1416 \times 6 \times 5.65 \times 6 \times 162.32 \times 1/1000 = 207.64 \text{ KN} > 85.6 \text{ KN}$
THEREFORE OK

SHEAR CAPACITY OF 6- 12MM DIA. ANCHOR STUDS

STUDS SHEAR CAP. = $3.1416(12^2)(1/4)(1/1000)(219.91)(6) = 149.34 \text{ KN} > 85.6$
THEREFORE OK

TENSILE CAPACITY OF 6- 12MM DIA. ANCHOR STUDS OF E56 EMBEDS

TENSILE CAP. OF STUDS = $113 (549.78) (1/1000)6 = 373.04 \text{ KN} > 181/5 = 36.2 \text{ KN}$, OK

CONCRETE SHEAR CHECK AT THE ROOF FLANGE OF DTS

HORIZONTAL FORCE = 181 PER PANEL

LOAD PER FIXING = 36.2 KN

V_c = SHEAR CAPACITY OF CONCRETE

$$V_c = .85 (F'_c)^{1/2} (b) (d) / 6$$

F'_c = COMPRESSIVE STRENGTH OF CONCRETE AT 28 DAYS = 48 MPa

SIF = 1.1 AVERAGE MATERIAL STRENGTH INCREASE (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (SAES 009)

$$F'_{cd} = F'_c (SIF) (DIF) = 48 (1.1)(1.19) = 62.83 \text{ MPa}$$

$$b = \text{THE PERIMETER OF PUNCHING AREA} = 125 (2) + 175 = 425 \text{ MM}$$

$$d = \text{DEPTH OF FLANGE} = 150 (.8) = 120 \text{ MM}$$

$$V = \text{APPLIED HORIZONTAL SHEAR} = 36.2 \text{ KN}$$

$V = \text{APPLIED HORIZONTAL SHEAR} = 36.2 \text{ KN}$

$V_c = .85 (62.83)^{1/2} (425) (120) / 6 (1/1000)$

$V_c = 65.5 \text{ KN} > 36.2 \text{ KN}$ THEREFORE OK.

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "5 " OF BASIC DRAWING OF 979-BD-SB1/SB2-07

STEEL PLATES CONNECTOR OF SHEARWALL INSULATED PANELS

CONSIDER ACC-05 ACCESSORIES , (SEE SHOP DRAWINGS)

APPLIED UPWARD SHEAR FORCE PER METER = 261.6 KN/M
USING GRADE 60 WELDING ELECTRODE

E60XX ft = 60 ksi (414 MPa)

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (REF. SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3 f_t$

$f_s = 124 \text{ MPa}$

$f_{sd} = f_s (SIF)(DIF) = 124 (1.1)(1.19) = 162.32 \text{ MPa}$

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = 320 (1.1)(1.19) = 418.90 \text{ MPa}$

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128 \text{ MPa}$

$f_{vd} = 128 (1.1)(1.19) = 167.6 \text{ MPa}$

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE STRESS = 420 MPa

$f_{ybd} = 420(1.1)(1.19) = 549.78 \text{ MPa}$

ALLOWABLE SHEAR STRESS = $420 (.4) = 168 \text{ MPa}$

$f_{vbd} = 168 (1.1)(1.19) = 219.91 \text{ MPa}$

USING 125 X 125 X 8MM THK. STEEL PLATE (ACC-05)

APPLIED SHEAR PER FIXING AT 500MM C/C = $261.6/2 = 130.8 \text{ KN}$

SHEAR CAPACITY OF BOTH SIDE ACC-05 STEEL PLATE ACCESSORY:

SHEAR CAPACITY OF BOTH SIDE ACC-05 STEEL PLATE ACCESSORY:

SHEAR CAP. OF 8MM STEEL PLATE = $8 (125) (167.60) (1/1000) (2) = 335.20 \text{ KN} > 130.8 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF ACC-05 WELD (8MM WELD SIZE)

Weld size = 8mm , $t = \text{throat} \quad t = \cos 45 (8) = 5.65 \text{ mm},$

$L = \text{LENGTH OF WELD} = 50 \times 2 + 125 = 225 \text{MM}$

WELD SHEAR CAP. = $5.65 \times 225 \times 162.32 \times 1/1000 = 206.50 \text{ KN} > 130.8 \text{ KN}$
THEREFORE OK.

DETAIL # 5 OF BASIC DRAWING 979-BD-SB1/SB2-07

INSULATED SHEARWALL PANEL TO PANEL CONNECTION

CONSIDER E66 (EMBEDDED INSULATED SHEARWALL PANEL)

UPWARD FORCE OF 261.6 KN PER METER = 130.8 KN PER FIXING.

CHECK SHEAR CAPACITY OF 3 PCS.DIA. 12 MM STUD OF E66 EMBEDS

USING 8MM WELD SIZE , THROAT, $t = \cos 45 (8) = 5.65 \text{ MM}$

SHEAR CAPACITY OF WELD OF 3 DIA ANCHOR STUDS:

SHEAR CAP. OF 8MM WELD = $2 \times 3.1416 \times 6 \times 5.65 \times 3 \times 162.32 \times 1/1000 (2) = 207.64 \text{ KN} > 130.8 \text{ KN}$
THEREFORE OK.

USING 3- DIA. 12MM ANCHOR STUDS WELD AT E66 PLATE EMBEDS

CHECK FOR SHEAR CAPACITY OF 12MM DIA. ANCHOR STUDS:

SHEAR CAPACITY OF 12MM DIA. = $3.1416 (12^2) (1/4) (3 (219.91) 1/1000) (2)$
= 149.34 KN > 130.8 KN

THEREFORE OK

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "4 " OF BASIC DRAWING OF 979-BD-SB1/SB2-07

STEEL PLATES CONNECTOR OF LONGITUDINAL INSULATED PANELS

CONSIDER ACC-03 ACCESSORIES , (SEE SHOP DRAWINGS)

APPLIED UPWARD SHEAR FORCE PER METER = 20.5 KN/M
USING GRADE 60 WELDING ELECTRODE

E60XX $f_t = 60 \text{ ksi}$ (414 MPa)

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR (REF SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3 f_t$

$f_s = 124 \text{ MPa}$

$f_{sd} = f_s (SIF)(DIF) = (124)(1.1)(1.19) = 162.32 \text{ MPa}$

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = 320 (1.1)(1.19) = 418.90 \text{ MPa}$

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128 \text{ MPa}$

$f_{vd} = 128 (1.1)(1.19) = 167.60 \text{ MPa}$

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE STRESS = 420 MPa

$f_{ybd} = 420(1.1)(1.19) = 549.78 \text{ MPa}$

ALLOWABLE SHEAR STRESS = $420 (.4) = 168 \text{ MPa}$

$f_{vbd} = 168 (1.1)(1.19) = 219.91 \text{ MPa}$

USING 100 X 125 X 6MM THK. STEEL PLATE (ACC-03)

APPLIED SHEAR PER STUD AT 500MM C/C = 20.5 KN / 100 MM C/C = 0.205 KN

USING 100 X 125 X 6MM THK. STEEL PLATE (ACC-03)

APPLIED SHEAR PER FIXING AT 500MM C/C = $20.5/2 = 10.25$ KN

SHEAR CAP. OF 6MM PLATE (ACC-03) = $125 \times 6 \times 167.60 (1/1000) = 125.70$ KN > 10.25 KN
THEREFORE OK

CHECK FOR WELD SHEAR CAPACITY OF ACC-03 :

Weld size = 5mm , t = throat t = $\cos 45 (5) = 3.54$ mm,

L= LENGTH OF WELD = $40 \times 2 \times 125 = 205$ MM

WELD SHEAR CAP. = $3.54 \times 205 \times 162.32(1/1000) = 117.90$ KN > 10.25 KN
THEREFORE OK.

DETAIL # 4 OF BASIC DRAWING 979-BD-SB1/SB2-07

INSULATED PANEL WALL TO WALL CONNECTION

CONSIDER E52 (EMBEDDED INSULATED PANEL)

UPWARD FORCE OF 20.5 KN PER METER = 10.25 KN PER FIXING.

CHECK SHEAR CAPACITY OF 2 PCS.DIA. 12 MM STUD OF E52 EMBEDS

USING 6MM WELD SIZE , THROAT, t = $\cos 45 (6) = 4.24$ MM

SHEAR CAPACITY OF WELD:

SHEAR CAP. OF 6MM WELD = $2 \times 3.1416 \times 6 \times 4.24 \times 2 \times 162.32 \times 1/1000 = 51.90$ KN > 10.25 KN
THEREFORE OK.

USING 2- DIA. 12MM ANCHOR STUDS WELD AT E52 PLATE EMBEDS

CHECK FOR SHEAR CAPACITY OF 12MM DIA. ANCHOR STUDS:

SHEAR CAPACITY OF 12MM DIA. = $3.1416(12^2)(1/4)2(219.91)(1/1000) = 49.78$ KN > 10.25KN
THEREFORE OK

PROJECT : PRECAST EXTERNAL INSULATED CLADDINGS FOR ARAMCO REPLACEMENT
SUBSTATION 1 & 2 (ABU ALI)

SUBJECT : DESIGN CALCULATION OF STEEL PLATE PRECAST CONCRETE ELEMENTS
CONNECTION.

DETAIL "1 " OF BASIC DRAWING OF 979-BD-SB1/SB2-07

STEEL PLATES CONNECTOR OF SHEAR WALL INSULATED PANELS

CONSIDER E50 (EMBEDDED PLATE , SEE FILE OF ACCESSORIES SHOP DRAWINGS)

APPLIED UPWARD SHEAR FORCE PER FIXING = $644 / 5 = 128.8$ KN

DUE TO THE DOOR OPENING IN SHEARWALL SIDE, ADDITIONAL LOAD ON FIXING IS
ADDED DUE TO ABSENCE OF 6 PCS. FIXINGS DUE TO DOOR OPENINGS

ADDITIONAL UPWARD FORCE AT CORNER FIXING OF SHEARWALL & NON-SHEARWALL:
 $= 6 (128.8) (5..35) / 10.64 / 14 = 28$ KN FOR EACH FIXING (MINIMAL)

USING GRADE 60 WELDING ELECTRODE

E60XX ft = 60 ksi (414 MPa)

SIF = 1.10 AVERAGE MATERIAL STRENGTH INCREASE FACTOR (REF. SAES 009)

DIF = 1.19 DYNAMIC INCREASE FACTOR. (SAES 009)

ALLOWABLE SHEARING STRESS OF WELDING ELECTRODE

$f_s = 0.3$ (ft)

$f_s = 124$ MPa

$f_{sd} = f_s (SIF)(DIF) = (124)(1.1)(1.19) = 162.32$ Mpa

STEEL PLATE :

ALLOWABLE TENSILE (YIELD) STRESS = 320 MPa

$f_{yd} = (320)(1.1)(1.19) = 418.90$ MPa

ALLOWABLE SHEAR STRESS OF PLATE = $320 \times 0.4 = 128$ MPa

$f_{vd} = 128 (1.1)(1.19) = 167.60$ MPa

ANCHOR STUD BAR GRADE 60

ALLOWABLE TENSILE STRESS = 420 MPa

$f_{ybd} = 420 (1.1)(1.19) = 549.78$ MPa

ALLOWABLE SHEAR STRESS OF BOLT = $420 (.4) = 168$ MPa

$f_{vbd} = 168 (1.1) (1.19) = 219.91$ MPa

USING 6- DIA. 12MM ANCHOR STUDS & 8MM WELD SIZE

CHECK FOR SHEAR CAPACITY OF WELD:

CHECK FOR SHEAR CAPACITY OF WELD:

Weld size = 8mm , t = throat t = $\cos 45 (8) = 5.65 \text{ mm}$,

WELD SHEAR CAP. = $5.65 \times 2 \times 3.1416 \times 6 \times 6 \times 162.32 \times 1 / 1000 = 207.48 \text{ KN} > 128.8 \text{ KN}$
THEREFORE OK.

SHEAR CAP. OF DIA 12MM ANCHOR STUDS 6 NOS.

STUD SHEAR CAP. = $3.1416 (12)^2 (1/4) (6) (219.91) (1/1000) = 149.23 \text{ KN} > 184 \text{ KN}$
THEREFORE OK.

DETAIL # 1 OF BASIC DRAWING 979-BD-SB1/SB2-07

SHEARWALL STEEL PLATE CONNECTOR

CONSIDER E50A (BY MAIN CONTRACTOR) (EMBEDDED IN CIP SLAB SEE BASIC DRAWING)

UPWARD FORCE OF 644 KN PER PANEL.

TENSILE CAPACITY OF 12MM DIA. ANCHOR STUDS

TENSILE FORCE ON 4- DIA. 12MM STUDS = $644 / 5 \times 125 \times 1 / .1 = 161 \text{ KN}$

TENSILE CAP. OF 4- DIA. 12MM STUD = $113 (549.78) (1/1000) (4) = 248.5 \text{ KN} > 161 \text{ KN}$
THEREFORE OK

USING 10MM WELD SIZE , THROAT t = $\cos 45 (10) = 7.07 \text{ MM}$

SHEAR CAP. OF 10MM WELD = $2 \times 3.1416 \times 6 \times 7.07 \times 4 \times 162.32 \times 1 / 1000 = 173.1 \text{ KN} > 161 \text{ KN}$
THEREFORE OK.

DETAIL # 1 OF BASIC DRAWING 979-BD-SB1/SB2-07

APPLIED HORIZONTAL SHEAR FORCE PER FIXING = $428.81 / 5 = 85.76 \text{ KN}$

USING 8- DIA 12MM ANCHOR STUDS & 10MM WELD SIZE

CHECK FOR SHEAR CAPACITY OF WELD

WELD SIZE = 10mm , t = throat t = $\cos 45 (10) = 7.07 \text{ mm}$:

WELD SHEAR CAP. = $7.07 \times 2 \times 3.1416 \times 6 \times 8 \times 162.32 \times 1 / 1000 = 345.58 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK.

WELD SHEAR CAP. = $7.07 \times 2 \times 3.1416 \times 6 \times 8 \times 162.32 \times 1/1000 = 345.58 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK.

SHEAR CAPACITY OF DIA. 12MM ANCHOR STUDS 8 NOS

STUD SHEAR CAP. = $3.1416 \times 12^2 \times 1/4 \times 8 \times 219.91 \times 1/1000 = 199 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK

SHEAR CAPACITY OF ACC-01 (FIXING ACCESSORY FOR DETAIL 1)

WELD SIZE OF BENT PLATE ACC-01 = 8MM , $t = \cos 45 (8) = 5.65 \text{ MM}$

L = LENGTH OF WELD FOR BENT PLATE ACC-01 125X125X12MM THK. = 450 MM

SHEAR CAP. OF WELD = $5.65 \times 450 \times 162.32 \times 1/1000 = 413 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK.

HOR. SHEAR CAP. OF ACC-01 (125X125X12MM THK. X 200MM LONG)

SHEAR CAP. OF BENT PLATE = $12 \times 200 \times 167.60 \times 1/1000 = 402.24 \text{ KN} > 85.76 \text{ KN}$
THEREFORE OK.

DETAIL # 1 OF 979-BD-SB1/SB2-07

CONSIDER ACC-01 125X125X12MM THK. X 200MM LONG ACCESSORY

UPWARD FORCE = $644 / 5 = 128.8 \text{ KN PER FIXING}$

UPWARD SHEAR CAP. OF 8MM SIZE WELD , $t = \cos 45 (8) = 5.65 \text{ MM}$

I of weld of ACC-O1 = $200 \times 5.65^3 / 12 (2) + 200 \times 5.65 \times 62.5^2 (2) + 2 (5.65 \times 125^3 / 12)$
 $= 10.673 \times 10^6 \text{ MM}^4$

S (section modulus) = $10.673 \times 10^6 / 62.5 = 170773 \text{ MM}^3$

M = MOMENT INDUCED DUE TO ECCENTRIC LOADING
 $= 128.8 (.0625) = 8.05 \text{ KN.M}$

SHEAR CAP. OF 8MM WELD ON ACC-01 ACCESSORY
 $= 128.8 (1000) / 5.65 (450) + M/S$
 $= 128.8 (1000) / 5.65 (450) + 8.05 \times 10^6 / 170773$
 $= 97.8 \text{ MPa} < 162.32 \text{ MPa}$, therefore OK.

DETAIL # 1 OF 979-BD-SB1/SB2-07

CHECK FOR THE UPWARD SHEAR FORCE WALL LOAD = 644 KN PER PANEL

FOR ACC-01 UPWARD SHEAR FORCE PER FIXING = $644 / 5 = 128.8 \text{ KN}$

FOR ACC-01 UPWARD SHEAR FORCE PER FIXING = $644 / 5 = 128.8$ KN

SHEAR CAPACITY OF 8MM WELD , $t = \cos 45 (8) = 5.65$ MM

L = LENGTH OF WELD , $L = 105 \times 2 + 200 = 410$ MM

SHEAR CAP. OF 8MM WELD = $5.65 (410) (162.32)(1/1000) = 376.30$ KN > 128.8KN
THEREFFORE OK

CHECK TENSILE CAPACITY OF STEEL BENT PLATE ACC-01 12MM THK. 125X125

TENSILE CAP. = $12 \times 200 \times 418.90 (1/1000) = 1005.40$ KN > 128.8 KN
THEREFORE OK

PULL OUT SHEAR CAPACITY OF CONCRETE AT E50A EMBEDS(BY MAIN CONTRACTOR)
IN CIP SLAB IS NOT QDC'S SCOPE OF WORK.

APPENDIX C

Construction Drawings

- **General Layout**
 - **Wall Panels**
 - **Reinforcement**
 - **Roof Slab**
 - **Connection Details**
 - **Standard Embedment Details**
-

APPENDIX C.1

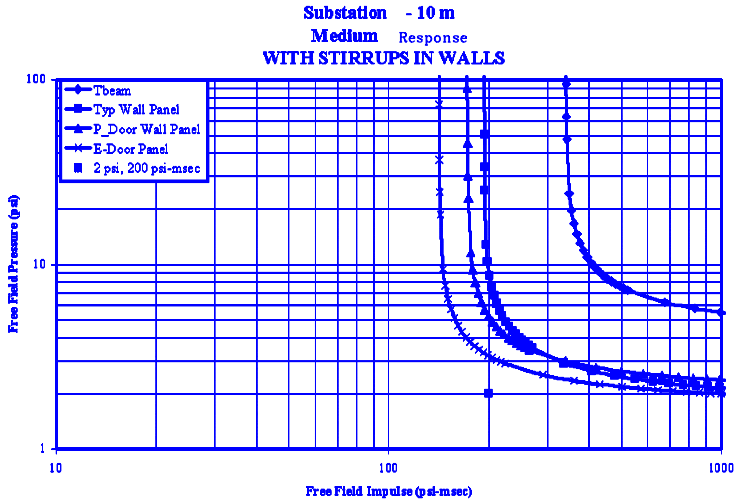
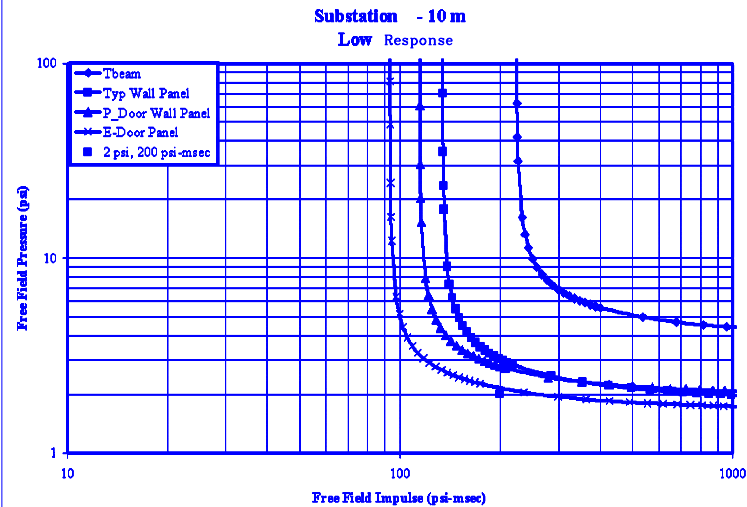
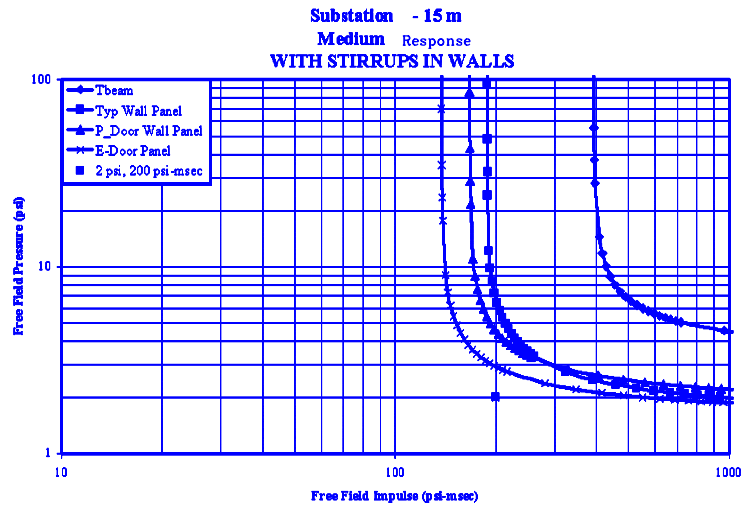
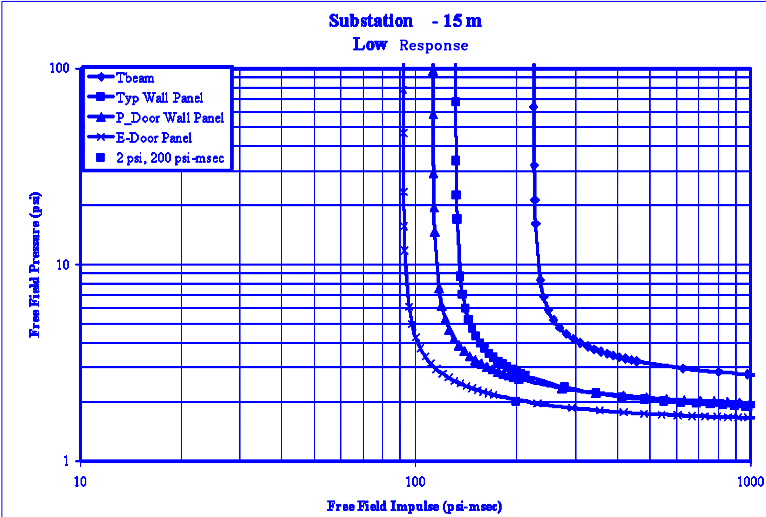
General Layout Drawings

12. DETERMINE IF THE BUILDING MUST BE DESIGNED AS A BLAST RESISTANT BUILDING PER THE BUILDING RISK ASSESSMENT STUDY AND SAES-B-014. IF THE BUILDING IS DESIGNATED AS BLAST RESISTANT BUILDING, DETERMINE IF IT IS TO BE DESIGNED TO "LOW" OR "MEDIUM" RESPONSE PER SAES-M-009. IF THE BUILDING IS DESIGNATED TO HAVE MEDIUM RESPONSE, THE WALL PANELS WILL HAVE TO BE MODIFIED TO INCLUDE STIRRUPS.
13. VERIFY THAT THE PRESSURE AND IMPULSE VALUE DETERMINED IN THE BUILDING RISK ASSESSMENT DOES NOT EXCEED THE APPROPRIATE P-I CURVE SHOWN ON DRAWING DA-950152 SH 001. IF THE BUILDING BLAST CAPACITY IS ADEQUATE PER THE P-DIAGRAM AND THE BUILDING LENGTH DOES NOT EXCEED THE 50 METER MAXIMUM LIMIT, THEN NO FURTHER STRUCTURAL DESIGN IS NECESSARY FOR THE BUILDING SHELL.
14. PREPARE FOUNDATION DESIGN CALCULATIONS IF THE ALLOWABLE SOIL BEARING CAPACITY AT THE SELECTED BUILDING SITE IS LESS THAN 130kN/M2 OR IF THE BUILDING FLOOR MUST BE ELEVATED DUE TO THE AREA CLASSIFICATION. IF THE FOUNDATION RING BEAM THICKNESS AND SLAB IS GREATER THAN 900MM AND 450MM RESPECTIVELY, THE PANEL FIXING DETAILS SHALL BE CHECKED.
15. FLOOR SLAB LEVELNESS SHALL NOT EXCEED 3mm GAP AT ANY POINT BETWEEN A 3.0M STRAIGHTEDGE AND THE FLOOR SLAB SHALL MEET THE "VERY FLAT" CRITERIA AS PER ASTM E115-87 AND ACI 117. FLATNESS UNDER THE EQUIPMENT SHALL MEET THE CRITERIA ABOVE USING GROUT. IF THIS REQUIREMENT IS MET LEVELING CHANNELS ARE NOT REQUIRED. IF THE FLATNESS CRITERIA CAN NOT BE MET, LEVELING CHANNELS ARE REQUIRED IN LOCATIONS SPECIFIED BY THE EQUIPMENT MANUFACTURER. NOTE THAT SAES-P-119 DOES NOT ALLOW ELECTRICAL CABLE/CONDUIT PENETRATIONS IN THE FLOOR SLAB.
16. FLOOR HARDENER SHALL BE APPLIED PER THE MANUFACTURER'S RECOMMENDATIONS A MINIMUM OF ONE METER IN FRONT OF ALL SWITCHGEAR.
17. THE INTERIOR WALL SEPARATING THE MECHANICAL/ BATTERY ROOMS FROM THE ELECTRICAL EQUIPMENT SHALL BE LOCATED BASED UPON FLOOR SPACE REQUIREMENTS FOR THE AIR HANDLING UNITS.
18. NO ELECTRICAL EQUIPMENT SHALL BE MOUNTED ON THE INTERIOR SURFACES OF EXTERIOR BUILDING WALLS (SEE SAES-M-009).
19. REFER TO SAES-P-103 FOR ADDITIONAL BATTERY ROOM REQUIREMENTS.
REFER TO SAES-P-119 FOR ADDITIONAL REQUIREMENTS.
20. EMBEDDED HALFEN CHANNELS IN THE ROOF DOUBLE-TEE ARE LIMITED TO A SAFE WORKING LOAD OF 2.6kN (600 LBS) AT ANY SINGLE ANCHORAGE POINT. THE MINIMUM SPACING BETWEEN ANCHORAGE POINTS IS 250mm (10in).
21. BLAST RESISTANT DOORS SHALL BE DESIGNED TO WITHSTAND THE REFLECTED BLAST PRESSURE AND IMPULSE THAT WOULD CORRELATE TO THE APPROPRIATE P-DIAGRAM SHOWN ON DWG DA-950152 SH 001.

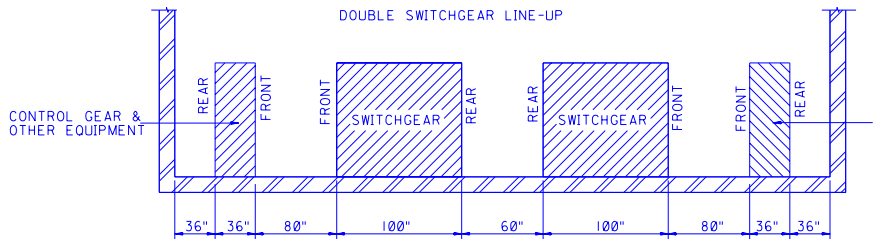
VECTOR CADD-3

STANDARD SUBSTATION BASIS AND LIMITATIONS

1. THE CONCEPT BEHIND THE STANDARD SUBSTATION BUILDINGS IS TO PROVIDE A BUILDING WITH A PRE-DEFINED HEIGHT AND WIDTH THAT CAN BE STRETCHED LENGTHWISE TO ACCOMMODATE THE EQUIPMENT LAYOUT.
2. THE HEIGHT WAS BASED ON CABLING CONSTRAINTS ASSOCIATED WITH TOP EQUIPMENT ENTRY. SEE CONCEPTUAL SKETCHES (SECTION/ELEVATION) ON THIS DRAWING.
3. THE DOUBLE-TEE ROOF SECTIONS ARE PROVIDED WITH HALFEN CHANNEL EMBEDDED TO ALLOW HANGERS TO BE ATTACHED AT ANY POINT ALONG THE BOTTOM FLANGE OF THE DOUBLE-TEE.
4. THESE BUILDINGS ARE BASED ON TOP ENTRY OF CABLE INTO ELECTRICAL EQUIPMENT AND NO CABLE/CONDUIT PENETRATIONS IN THE FLOOR SLAB.
5. THE BUILDING LENGTH CAN BE STRETCHED IN EVEN 2.5 M INCREMENTS TO A MAXIMUM OF 50 M BETWEEN SHEAR WALLS.
6. THE BUILDING IS DESIGNED FOR A MODERATE LEVEL OF BLAST RESISTANCE AND THE STANDARD DESIGN CAN BE USED WITHOUT ADDITIONAL CALCULATIONS PROVIDED THAT THE CALCULATED BLAST PRESSURE AND IMPULSE AT THE PROPOSED BUILDINGS LOCATION DOES NOT EXCEED THE P-I CAPACITY CURVE SHOWN ON THIS DRAWING. THE LENGTH LIMIT FOR THESE BUILDINGS IS 50 METER BETWEEN SHEAR WALLS WITHOUT ADDITIONAL STRUCTURAL CALCULATIONS.
7. THE DOUBLE-TEE PRECAST ROOF SECTIONS ARE NOT PRESTRESSED CONCRETE AND MUST BE CONVENTIONALLY REINFORCED TO RESIST BLAST LOADS AND REBOUND EFFECTS.
8. STRUCTURAL CALCULATIONS ARE AVAILABLE FROM THE TECHNICAL INFORMATION CENTER (SAER-6027-10M BLDG, SAER-6028-15M BLDG)
9. FOUNDATIONS CAN BE ELEVATED BY BUILDING UP COMPACTED FILL TO THE DESIRED HEIGHT FOR BUILDINGS THAT ARE LOCATED WHERE THE AREA CLASSIFICATION REQUIRES AN ELEVATED BUILDING.
10. THE FOUNDATION SHOWN ON THIS DRAWING IS BASED ON AN ALLOWABLE SOIL BEARING PRESSURE OF 130 KN/M2. AN APPROVED GEOTECHNICAL REPORT SHOULD BE USED TO DETERMINE IF THE ALLOWABLE BEARING PRESSURE AT THE SELECTED SITE EXCEEDS THE DESIGN BASIS. IF A SITE IS ENCOUNTERED WITH SOIL BEARING PRESSURE THAT IS LESS THAN THE DESIGN BASIS, A NEW FOUNDATION DESIGN SHOULD BE PREPARED FOR THE SITE CONDITIONS.
11. AREA BETWEEN TOP OF ELECTRICAL EQUIPMENT AND BOTTOM DOUBLE-TEE ROOF SECTIONS IS RESERVED FOR ELECTRICAL CABLING SYSTEM, LIGHTING AND VERTICAL HVAC DROPS.
12. MAIN HVAC DUCT IS INSTALLED IN THE UPPER EDGE OF THE BUILDING INTERIOR ON THE TRANSFORMER SIDE OF THE BUILDING. HVAC LATERALS SHALL BE INSTALLED BETWEEN THE DOUBLE-TEE ROOF SECTIONS AND NOT PROTRUDE BELOW THE BOTTOM FLANGE OF THE ROOF SECTIONS.
13. EMBEDDED HALFEN CHANNELS IN THE ROOF DOUBLE-TEE ARE LIMITED TO A SAFE WORKING LOAD OF 2.6kN (600 LBS) AT ANY SINGLE ANCHORAGE POINT. THE MINIMUM SPACING BETWEEN ANCHORAGE POINTS IS 205MM (10 IN).



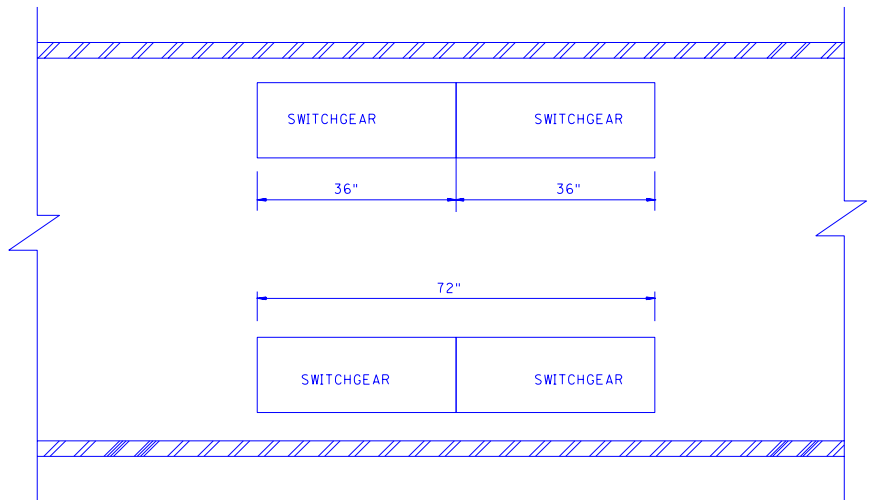
INTERIOR WIDTH:



SECTION/ELEVATION

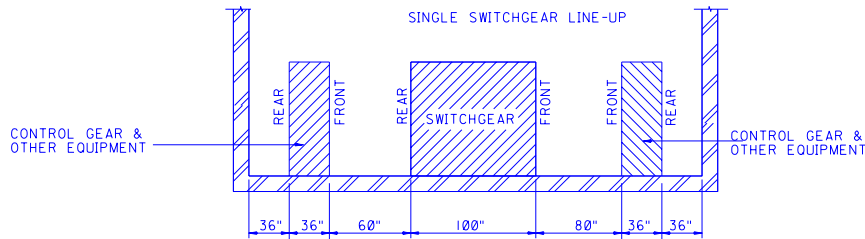
WIDTH : 564" (14.3M)
USE 15M

WALL PANEL CABLE OPENING



PLAN VIEW

INTERIOR WIDTH:



SECTION/ELEVATION

WIDTH : 384" (9.75M)
USE 10M

ASSUMPTIONS :

- TWO BREAKER HIGH SWITCHGEAR.
- BREAKERS 2000A RATED.
- EACH BREAKER REQUIRES 4-1000MCM.
- CABLES PER PHASE TO FIELD (10: 12 CABLES)
- THEREFORE : - 24 CABLES PER SWITCHGEAR UNIT
- 96 (4x24) CABLES EXITING BUILDING IN 72INCHES OF LINEAR WALL SPACE
- 1000MCM CABLE HAS OD OF 1.5 INCHES
- 1xOD SPACE BETWEEN CABLES.
- 36" WIDE CABLE WALL OPENING.

$$\frac{36" \text{ OPENING}}{3" \text{ PER CABLE}} = 12 \text{ CABLES PER ROW}$$

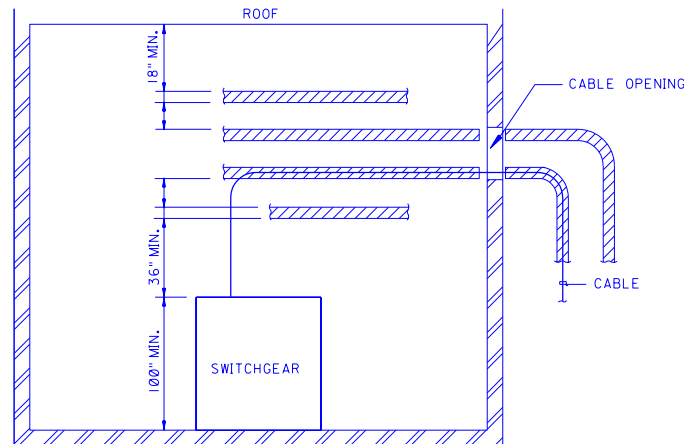
$$\frac{96" \text{ CABLES}}{12 \text{ CABLES PER ROW}} = 8 \text{ ROWS OF CABLE}$$

$$8 \text{ ROWS} \times 3" \text{ PER CABLE} = 24" \text{ OPENING HEIGHT}$$

CONCLUSION :

FOR 72" OF LINEAR FLOOR SPACE NEED 36" WIDE x 24" HIGH CABLE OPENING. OPENING WIDTH IS 50% OF WALL PANEL WIDTH)

INTERIOR HEIGHT:



SECTION/ELEVATION

ASSUMPTION : 6" CABLE TRAY RAIL HEIGHT

CONCLUSION:

HEIGHT TO BOTTOM OF CABLE OPENING:
 $100" + 36" + 6" + 12" = 154" (3.91M)$
USE 4 METERS (157 INCHES)

HEIGHT TO CEILING:
 $157" + 24" + 12" + 6" + 18" = 217" (5.5M)$
USE 6 METERS

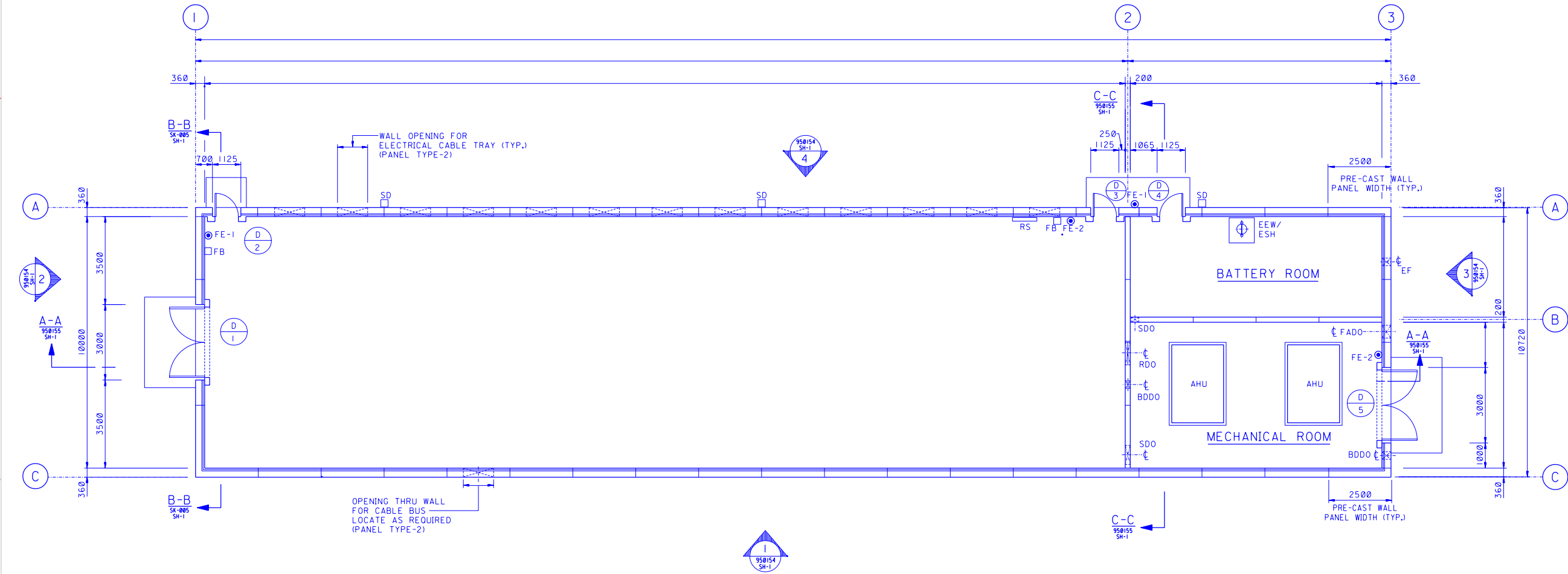
NOTES:

1. ALL CONSTRUCTION AND INSTALLATIONS TO COMPLY WITH SAUDI ARAMCO ENGINEERING STANDARDS (SAES) AND CONSTRUCTION SPECIFICATIONS.

SCALE: 1" = 5'

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY			CHECKED BY			
DATE STARTED			DATE COMPLETED			
DISCIPLINE ENGINEER			PROJECT ENGINEER			
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.			CERTIFIED BY:		NAME: DATE	
SIGNATURE			SIGNATURE		DATE	
SUBSTATION BASIS & LIMITATIONS						DRAWING TYPE
						ARC
STANDARD SUBSTATION - 10M & 15M WIDE						
LIBRARY DRAWING			SAUDI ARABIA			
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950152	001	00		

00	001	00153	DA-950153	R	M88
REV. NO.	SHT. NO.	INDEX	DRAWING NUMBER	PLANT NO.	REV. NO.



FLOOR PLAN - A
N T S

NOTE:- THIS DRAWING SHOWS STANDARD ORIENTATION A, ORIENTATION B,C AND D MUST BE DEVELOPED BASED ON THIS DRAWING WHEN REQUIRED.

NOTES:

- ALL DIMENSIONS ARE IN MILLIMETERS AND ALL ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.
- ALL CONSTRUCTION AND INSTALLATIONS TO COMPLY WITH SAUDI ARAMCO ENGINEERING STANDARDS (SAES) AND CONSTRUCTION SPECIFICATIONS.
- CONTRACTOR SHALL PROVIDE AND INSTALL CAGE LADDER FIXED TO THE WALL OF THE BUILDING FOR ROOF ACCESS.CAGE LADDER TO BE FABRICATED AND INSTALLED IN ACCORDANCE WITH STD. DWG. PIP STF05501 AND I2-SAMSS-007.

LEGEND

EEW/ESH	EMERGENCY EYE WASH/SHOWER
FB	FIRE BLANKET
FE-1	FIRE EXTINGUISHER, DRY CHEMICAL (25 LBS)
FE-2	FIRE EXTINGUISHER, (10 LBS, CO ₂)
EL	ELEVATION
FFL	FINISH FLOOR LEVEL
TBD	TO BE DETERMINED (FROM VENDOR)
SD	SCUPPER DRAIN
RDO	RETURN DUCT OPENING
SDO	SUPPLY DUCT OPENING
FADO	FRESH AIR DUCT OPENING
EF	EXHAUST FAN
BDDO	BACK DRAFT DAMPER OPENING
AHU	AIR HANDLING UNIT
ACCU	AIR COOLED CONDENSING UNIT
RS	RESCUE STRETCHER (BASKET TYPE)

REFERENCE DRAWINGS

GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
SUBSTATION BASIS AND LIMITATIONS	DA-950152.001
BUILDING ELEVATIONS	DA-950154.001
BUILDING SECTIONS	DA-950155.001
ROOF PLAN, INTERIOR FINISH SCHEDULE	DA-950156.001
FOUNDATION PLAN/SECTION	DA-950161.001

SCALE: 1 N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____ CHECKED BY _____						
DATE STARTED _____ DATE COMPLETED _____						
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.						CERTIFIED BY: _____ SIGNATURE _____ DATE _____
FLOOR PLAN						DRAWING TYPE ARC
STANDARD SUBSTATION - 10M WIDE						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950153	001	00		

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS AND ALL ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.
2. GRADE ELEVATION HAS BEEN ASSUMED AT 0.00, FINAL GRADE ELEVATION TO BE CONFIRMED BY SURVEY.
3. CONTRACTOR SHALL PROVIDE AND INSTALL CAGE LADDER FIXED TO THE WALL OF THE BUILDING FOR ROOF ACCESS. CAGE LADDER TO BE FABRICATED AND INSTALLED IN ACCORDANCE WITH STD. DWG. PIP STF05501 AND I2-SAMSS-007.

LEGEND

EEW/ESH	EMERGENCY EYE WASH/SHOWER
FB	FIRE BLANKET
FE-1	FIRE EXTINGUISHER, DRY CHEMICAL (25 LBS)
FE-2	FIRE EXTINGUISHER, (10 LBS, CO ₂)
EL	ELEVATION
FFL	FINISH FLOOR LEVEL
TBD	TO BE DETERMINED (FROM VENDOR)
SD	SCUPPER DRAIN
RDO	RETURN DUCT OPENING
SDO	SUPPLY DUCT OPENING
FADO	FRESH AIR DUCT OPENING
EF	EXHAUST FAN
BDDO	BACK DRAFT DAMPER OPENING
RS	RESCUE STRETCHER

REFERENCE DRAWINGS	
GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
SUBSTATION BASIS AND LIMITATIONS	DA-950152.001
BUILDING ELEVATIONS	DA-950154.002
BUILDING SECTIONS	DA-950155.002
ROOF PLAN, INTERIOR FINISH SCHEDULE	DA-950156.002
FOUNDATION PLAN/SECTION	DA-950162.001-002

SCALE: NTS

FLOOR PLAN-A
N T S

NOTE:- THIS DRAWING SHOWS STANDARD OREINTATION A,
OREINTATION B,C AND D MUST BE DEVELOPED
BASED ON THIS DRAWING WHEN REQUIRED.

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPROV.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____			CHECKED BY _____			
DATE STARTED _____			DATE COMPLETED _____			
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						

THIS DRAWING IS NOT TO BE
USED FOR CONSTRUCTION
UNTIL CERTIFIED AND DATED.

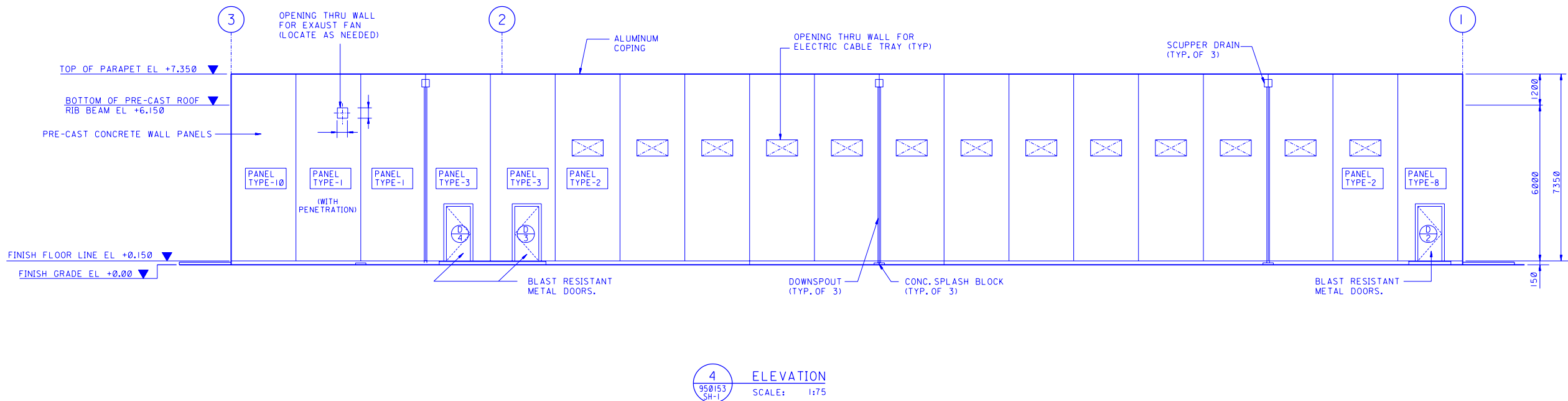
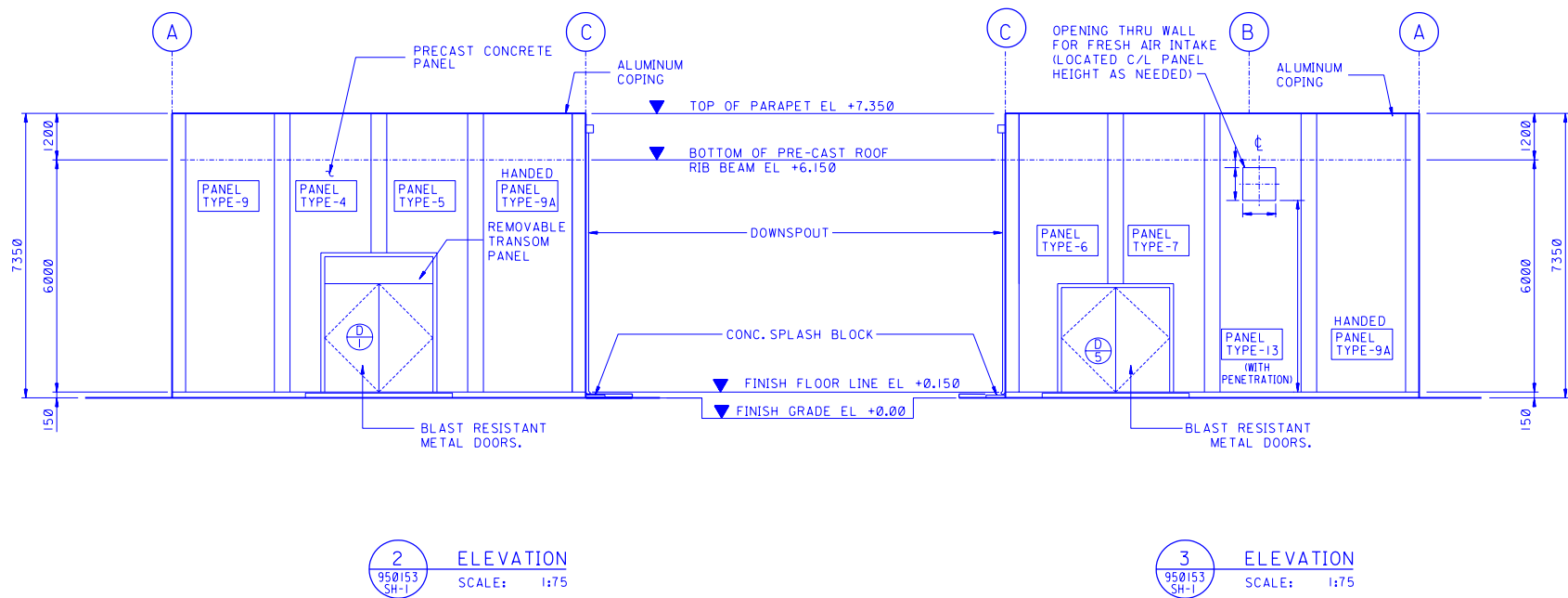
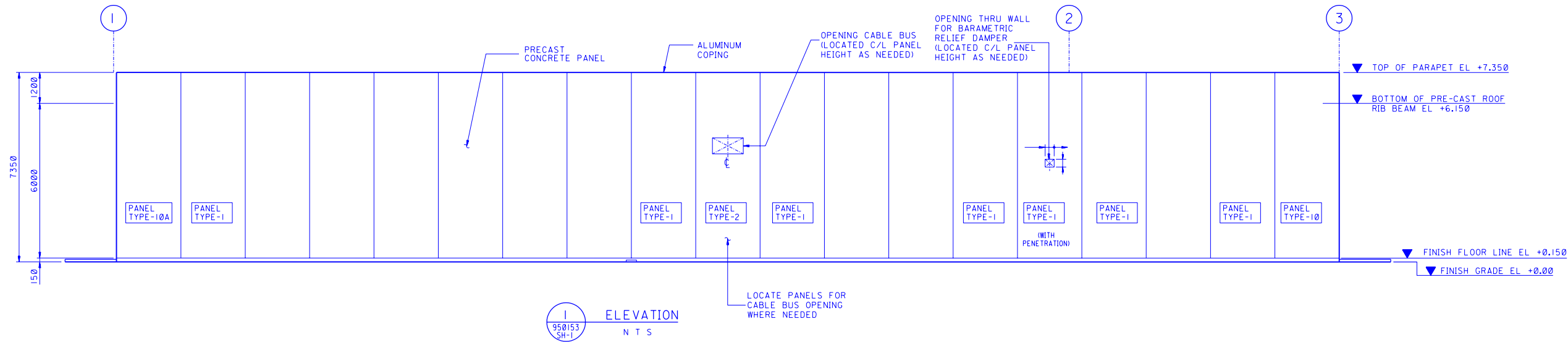
CERTIFIED BY: _____
(NAME)

SIGNATURE _____ DATE _____

FLOOR PLAN	DRAWING TYPE ARC
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STANDARD SUBSTATION - 15M WIDE			
LIBRARY DRAWING		SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO., REV. NO.
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00	001	4	DA-950154	R	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	



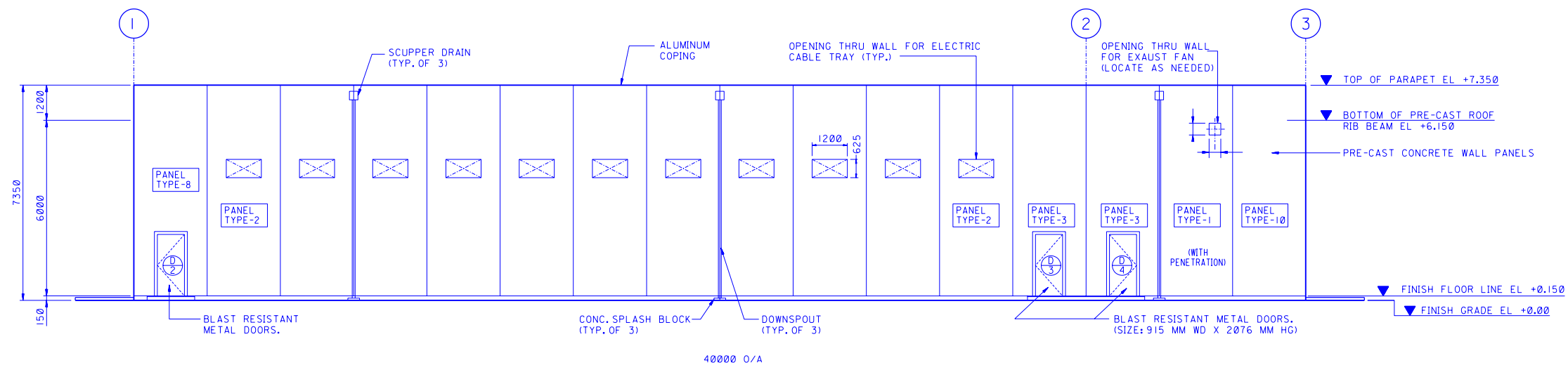
- NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETERS AND ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.
 2. ALL EXTERNAL DOORS TO BE BLAST RESISTANT METAL DOORS.

LEGEND	
EL	ELEVATION

REFERENCE DRAWINGS	
GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
SUBSTATION BASIS AND LIMITATIONS	DA-950152.001
FLOOR PLAN	DA-950153.001
BUILDING SECTIONS	DA-950155.001
ROOF PLAN, INTERIOR FINISH SCHEDULE	DA-950156.001
TYP. PRECAST PANEL DETAILS	DA-950158.01-13
FOUNDATION PLAN/SECTION	DA-950161.001

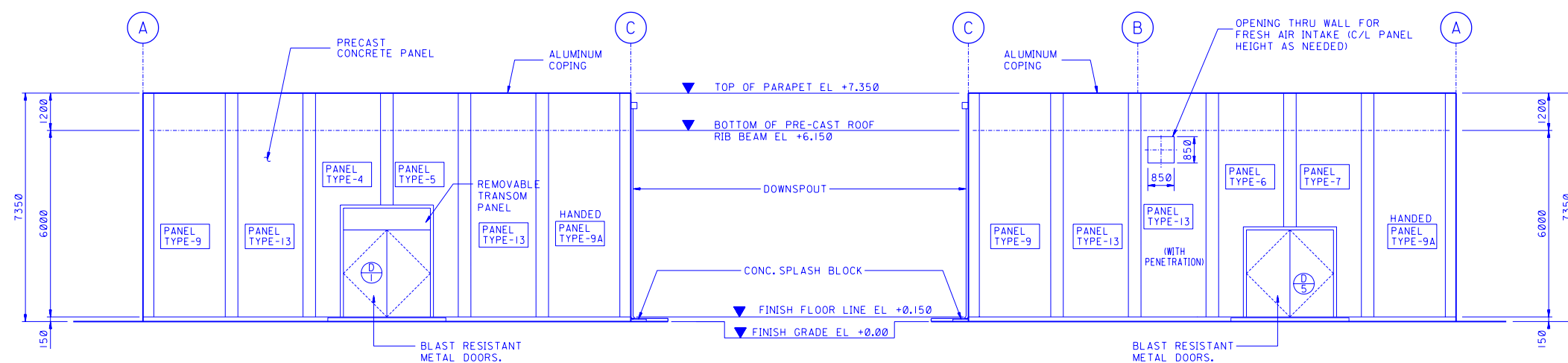
REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY			CHECKED BY			
DATE STARTED			DATE COMPLETED			
DISCIPLINE ENGINEER			PROJECT ENGINEER			
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.			CERTIFIED BY: NAME: DATE: SIGNATURE: DATE:			
ELEVATION			DRAWING TYPE: ARC			
STANDARD SUBSTATION - 10M WIDE						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950154	001	00		

M88	R	DA-950154	002	00
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.



1
950153
SH-2

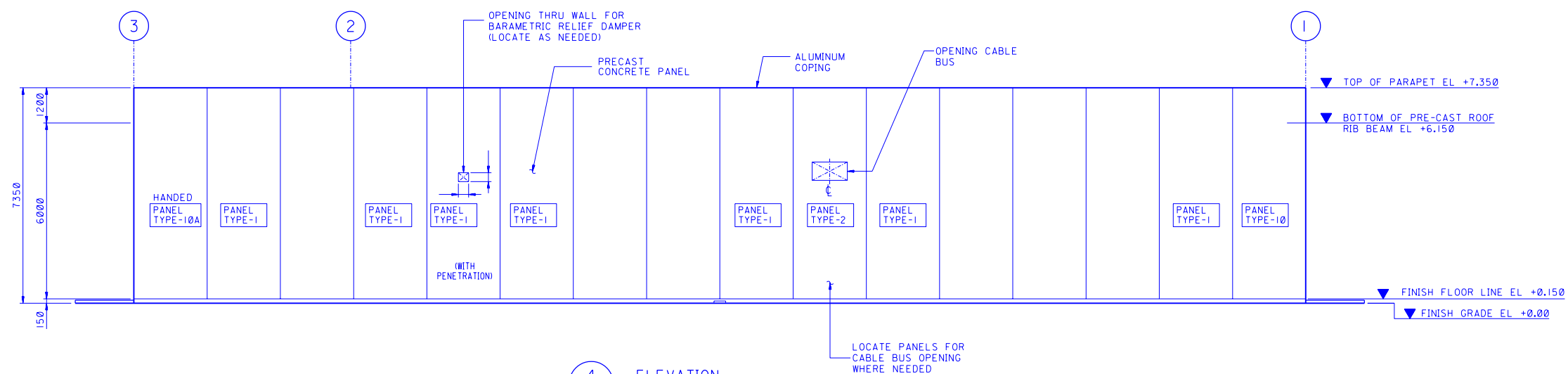
ELEVATION
N T S



2
950153
SH-2

ELEVATION
N T S

3 ELEVATION
950153 N T S
SH-2



4 ELEVATION
950153
SH-2 N T S

1. ALL DIMENSIONS ARE IN MILLIMETERS AND ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.
2. ALL EXTERNAL DOORS TO BE BLAST RESISTANT METAL DOORS.

LEGEND

EL	ELEVATION
----	-----------

REFERENCE DRAWINGS

GENERAL NOTES AND INSTRUCTIONS	DA-950151.00
SUBSTANTIATION BASIS AND LIMITATIONS	DA-950152.00
FLOOR PLAN	DA-950153.00
BUILDING SECTIONS	DA-950155.00
ROOF PLAN, INTERIOR FINISH SCHEDULE	DA-950156.00
TYP. PRECAST PANEL DETAILS	DA-950158.01
FOUNDATION PLAN/SECTION	DA-950162.00
FOUNDATION LAYOUT	DA-950162.00

SCALE: N T S

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: _____
	SIGNATURE _____ (NAME) DATE _____

ELEVATION

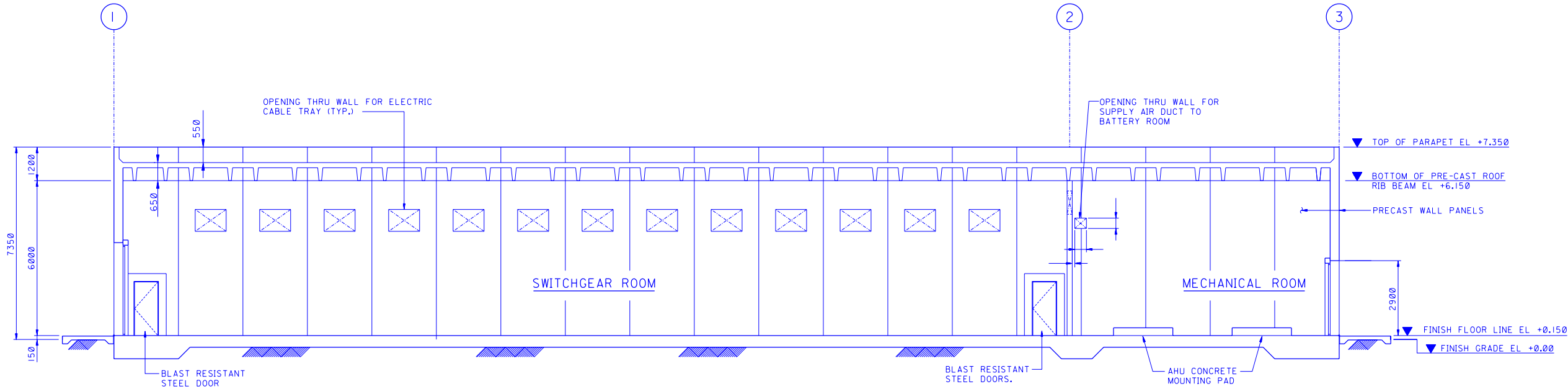
DRAWING TYPE	ARC
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STANDARD SUBSTATION - 15M WIDE

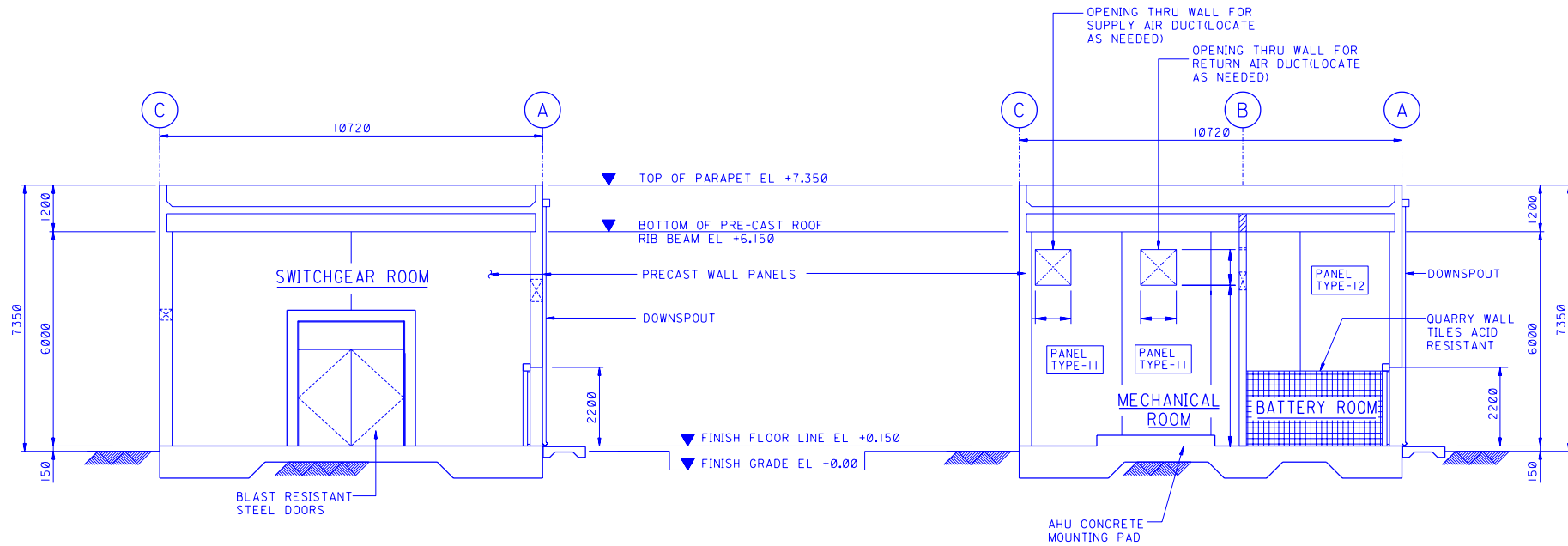
LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950154	002	00

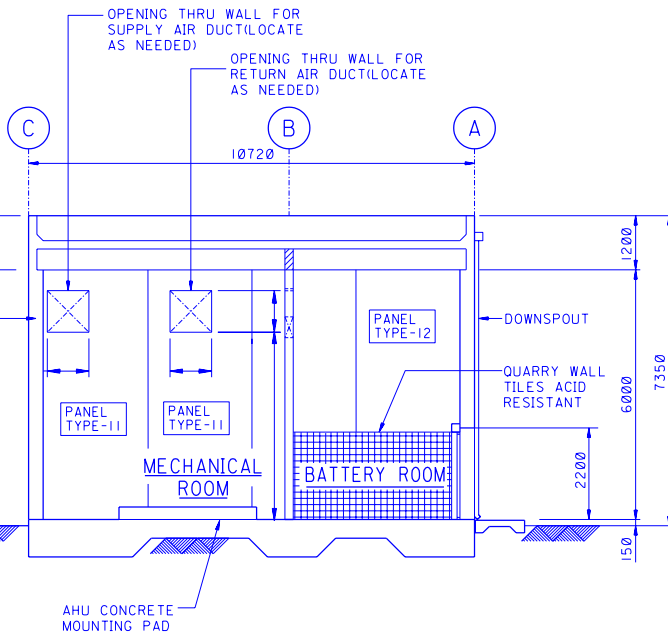
00	001	DA-950155	R	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.



A-A SECTION
950153 SH-1
N T S



B-B SECTION
950153 SH-1
N T S



C-C SECTION
950153 SH-1
N T S

NOTES:

- ALL DIMENSIONS ARE IN MILLIMETERS AND ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.
- ALL EXTERNAL DOORS TO BE BLAST RESISTANT STEEL DOORS.
- WALL PANEL OPENING SHALL BE AS PER FINAL CABLEBUS ARRANGEMENT.

LEGEND

EL ELEVATION

REFERENCE DRAWINGS

GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
SUBSTATION BASIS AND LIMITATIONS	DA-950152.001
FLOOR PLAN	DA-950153.001
BUILDING ELEVATIONS	DA-950155.001
ROOF PLAN, INTERIOR FINISH SCHEDULE	DA-950156.001
TYP. PRECAST PANEL DETAILS	DA-950158.01-13
FOUNDATION PLAN/SECTION	DA-950161.001

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY: _____ CHECKED BY: _____
DATE STARTED: _____ DATE COMPLETED: _____
DISCIPLINE ENGINEER: _____
PROJECT ENGINEER: _____

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.
CERTIFIED BY: _____ NAME: _____
SIGNATURE: _____ DATE: _____

SECTIONS

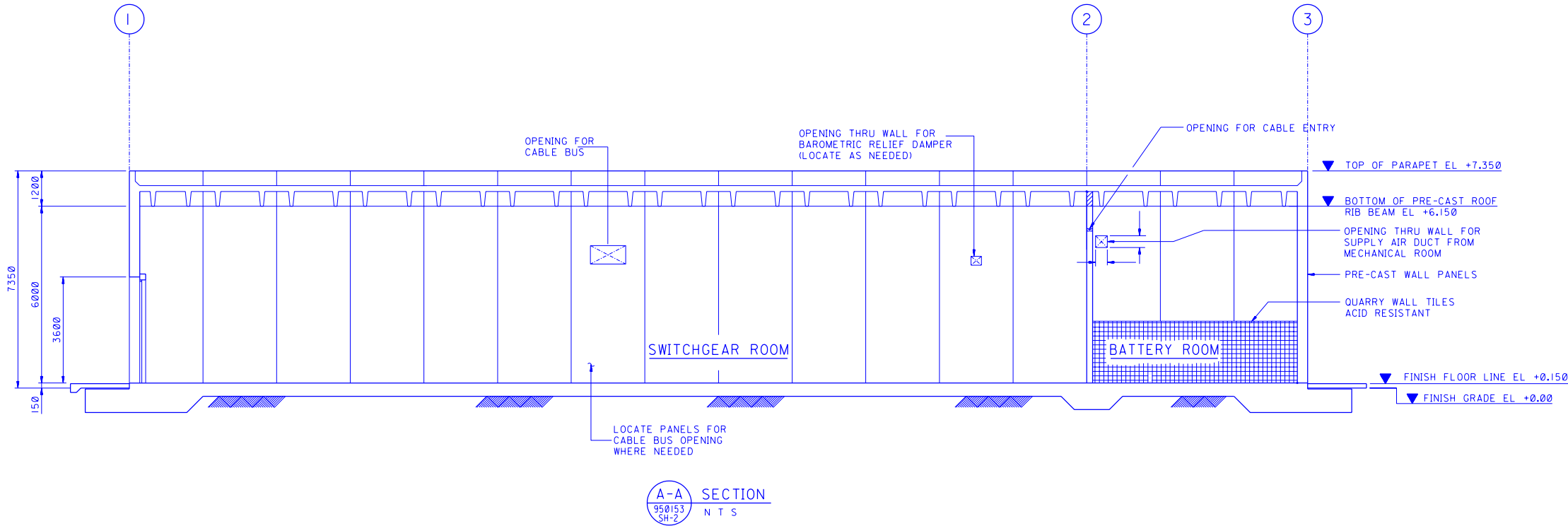
DRAWING TYPE

STANDARD SUBSTATION - 10M WIDE

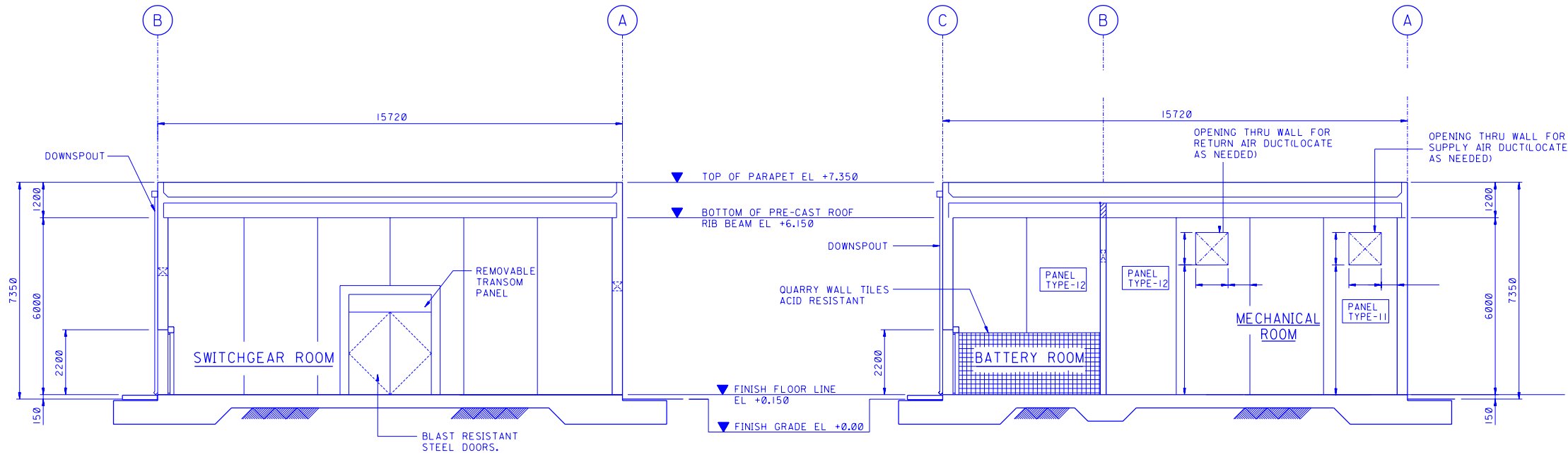
LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950155	001	00

00	002	DA-950155	R	M88
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.



A-A SECTION
950153 SH-2
N T S



B-B SECTION
950153 SH-2
N T S

C-C SECTION
950153 SH-2
N T S

NOTES:

- ALL DIMENSIONS ARE IN MILLIMETERS AND ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.
- ALL EXTERNAL DOORS TO BE BLAST RESISTANT STEEL DOORS.

LEGEND

EL ELEVATION
TYP. TYPICAL

REFERENCE DRAWINGS

GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
SUBSTATION BASIS AND LIMITATIONS	DA-950152.001
FLOOR PLAN	DA-950153.002
BUILDING ELEVATIONS	DA-950154.002
ROOF PLAN, INTERIOR FINISH SCHEDULE	DA-950156.002
TYP. PRECAST PANEL DETAILS	DA-950158.01-13
FOUNDATION PLAN/SECTION	DA-950162.001
FOUNDATION PLAN/SECTION	DA-950162.002

SCALE: _____

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.
CERTIFIED BY: _____ NAME: _____
SIGNATURE _____ DATE _____

SECTION

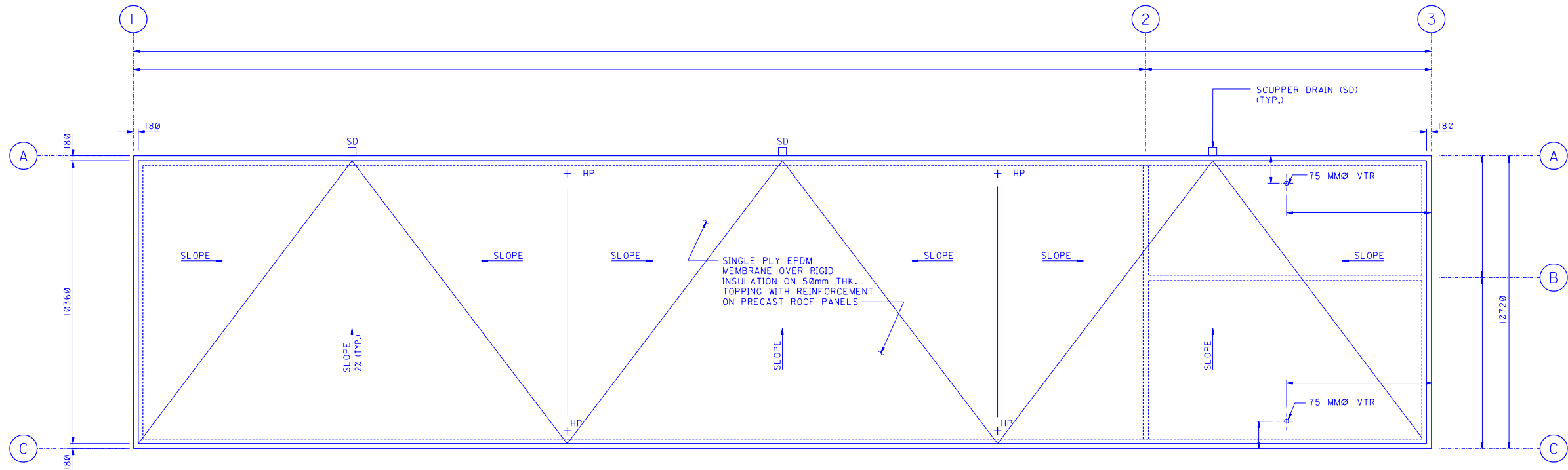
DRAWING TYPE
ARC

STANDARD SUBSTATION - 15M WIDE

LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950155	002	00

M88	R	DA-950156	001	00
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.



ROOF PLAN
N T S

ROOM FINISH SCHEDULE																								
FINISH SCHEDULE		FLOOR			WALL BASE			WALLS												CEILING			REMARKS	
								A			B			C			D							
ROOM NAME	MATERIAL SPECIFICATIONS	CONCRETE FLOOR W/ FLOOR HARDENER	QUARRY TILES (ACID RESISTANT)	EPOXY PAINT (GRAY)	QUARRY TILES (ACID RESISTANT)			EXPOSED CONCRETE PRECAST PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED CONCRETE PRECAST PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED CONCRETE PRECAST PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED PRE-CAST PANELS PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED PRECAST DOUBLE TEE SUB PAINTED FINISH				
		1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3		
SWITCHGEAR ROOM		●		●				●			●			●			●			●				
MECHANICAL ROOM		●		●				●			●			●			●			●				
BATTERY ROOM			●		●			●	●		●	●		●	●		●	●		●		PAINT (ACID RESISTANT) WHITE		

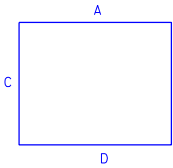
NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE NOTED.

LEGEND & ABBREVIATIONS

THK	THICK
FFL	FINISH FLOOR LEVEL
HP	HIGH POINT
SD	SCUPPER DRAIN
TBD	TO BE DETERMINED (FROM VENDOR)

KEY TO ROOM FINISH



REFERENCE DRAWINGS

FLOOR PLAN	DA-950153.001
BUILDING ELEVATION	DA-950154.001
BUILDING SECTIONS	DA-950155.001

SCALE: NTS

REV.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPROV

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

THIS DRAWING IS NOT TO BE
USED FOR CONSTRUCTION
UNTIL CERTIFIED AND DATED.

CERTIFIED BY: _____ (NAME)
SIGNATURE _____ DATE _____

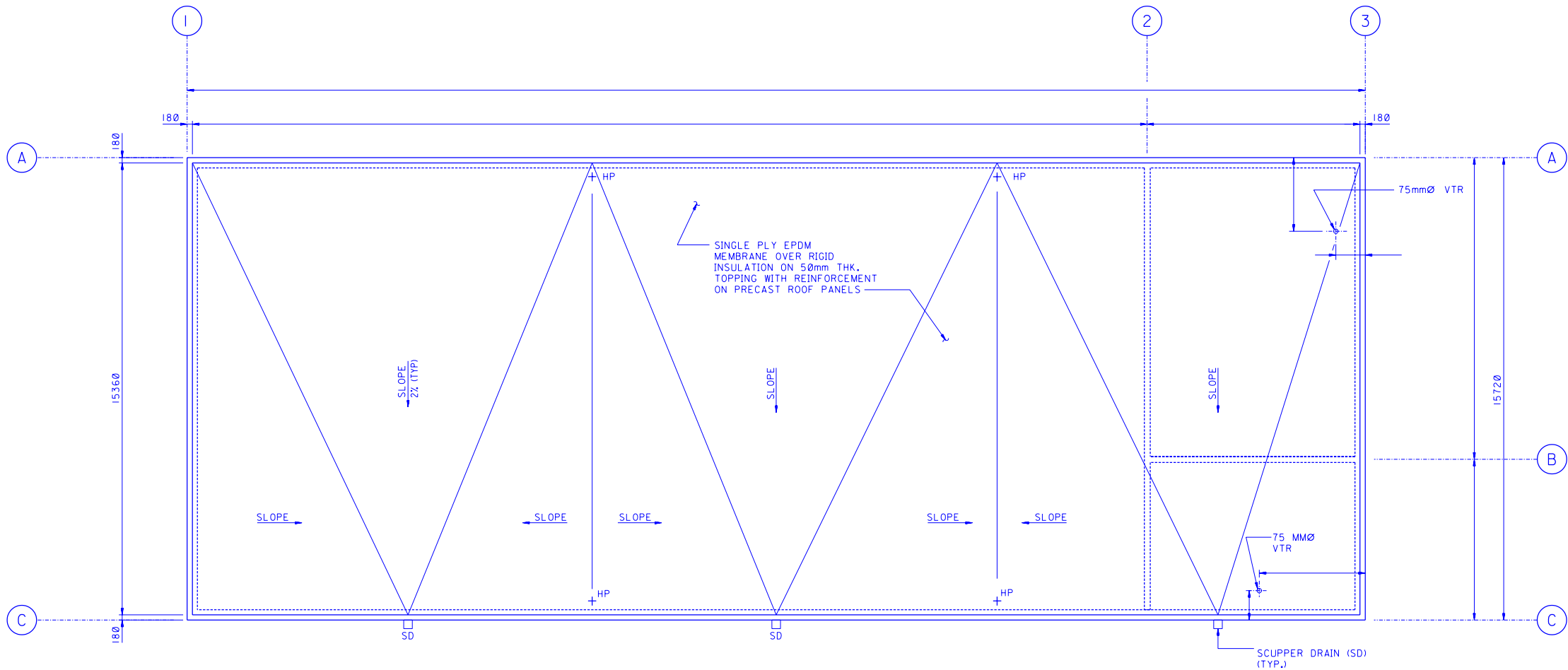
ROOF PLAN & FINISH SCHEDULE

STANDARD SUBSTATION - 10M WIDE

LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950156	001	00

00	002	DA-950156	R	M88
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.



ROOF PLAN
N T S

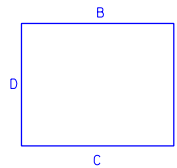
ROOM FINISH SCHEDULE																							
FINISH SCHEDULE		FLOOR			WALL BASE			WALLS												CEILING			REMARKS
								A			B			C			D						
MATERIAL SPECIFICATIONS	ROOM NAME	CONCRETE FLOOR W/ FLOOR HARDENER	QUARRY TILES (ACID RESISTANT)	EPOXY PAINT (GRAY)	QUARRY TILES (ACID RESISTANT)			EXPOSED CONCRETE PRECAST PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED CONCRETE PRECAST PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED CONCRETE PRECAST PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED PRE-CAST CONCRETE PAINTED FINISH	QUARRY WALL TILES (ACID RESISTANT)		EXPOSED PRECAST DOUBLE TEE SLAB PAINTED FINISH			
		1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	
	SWITCHGEAR ROOM	●		●				●			●			●			●			●			
	MECHANICAL ROOM	●		●				●			●			●			●			●			
	BATTERY ROOM		●		●			●	●		●	●		●	●		●	●		●			Paint (Acid Resistant) White

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE NOTED.

LEGEND & ABBREVIATIONS

THK	THICK
FFL	FINISH FLOOR LEVEL
HP	HIGH POINT
SD	SCUPPER DRAIN
TBD	TO BE DETERMINED (FROM VENDOR)

KEY TO ROOM FINISH



REFERENCE DRAWINGS

FLOOR PLAN	DA-950153.002
BUILDING ELEVATION	DA-950154.002
BUILDING SECTION	DA-950155.002

SCALE: _____

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY	CHECKED BY
DATE STARTED	DATE COMPLETED
DISCIPLINE ENGINEER	
PROJECT ENGINEER	

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: _____ SIGNATURE _____ DATE _____
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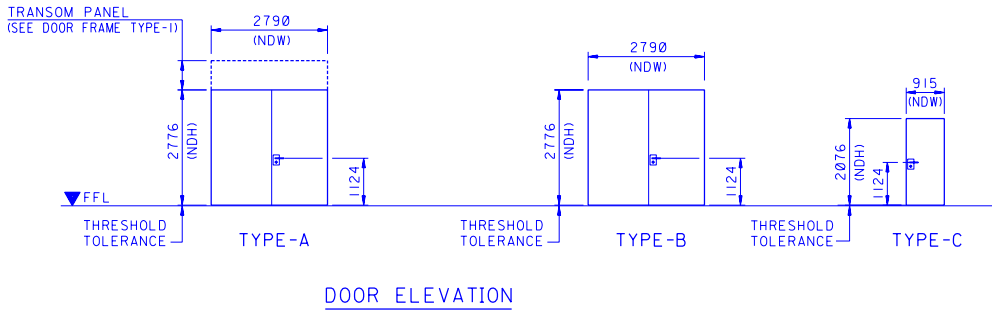
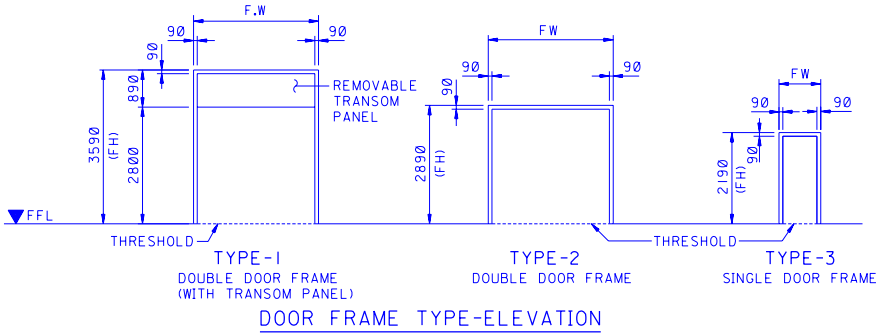
ROOF PLAN & FINISH SCHEDULE

STANDARD SUBSTATION - 15M WIDE

LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950156	002	00

00	001	DA-950157	R	M88	PLANT NO.
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	XREF	



DOOR SCHEDULE																
DOOR NUMBER	DOOR DESCRIPTION				DOOR FRAME TYPE		DOOR LEAF TYPE		WEATHER STRIPPING	HARDWARE PER SSD-8			FIRE RATING	DOOR FUNCTION	REMARKS	
	ROUGH OPENING		SIZE		DOOR THK.	TYPE	MATL.	TYPE		MATL.	LOCK SET	DOOR CLOSER				PANIC BAR
	WD	HT	WD	HT												
D-1	2800	3500	2790	2776		I	STL	A	STL BLAST RESISTANT	●	●	●	●	3 HR	DOUBLE	WITH REMOVABLE PANEL (SEE NOTES 2, 3 & 4)
D-2	925	2100	915	2076		3	STL	C	STL BLAST RESISTANT	●	●	●	●	3 HR	SINGLE	SEE NOTES 2, 3, 4 & 5
D-3	925	2100	915	2076		3	STL	C	STL BLAST RESISTANT	●	●	●	●	3 HR	SINGLE	SEE NOTES 2, 3, 4 & 5
D-4	925	2100	915	2076		3	STL	C	STL BLAST RESISTANT	●	●	●	●	3 HR	SINGLE	SEE NOTES 2, 3, 4 & 5
D-5	2800	2800	2790	2776		2	STL	B	STL BLAST RESISTANT	●	●	●	●	3 HR	DOUBLE	SEE NOTES 2, 3, 4 & 5

- NOTES:
- ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE NOTED.
 - DOOR FRAME & DOOR LEAF PAINT COATING IN ACCORDANCE WITH COATING SYSTEM OF SAES-H-101, APCS-6.
 - ALL STEEL DOOR TO BE BLAST RESISTANT DOORS.
 - ALL DOORS SHALL BE AS PER THE REQUIREMENTS OF SSD-7 & 8
 - LABELS TO BE PROVIDED ON DOORS AS FIRE RATED TESTED DOOR.

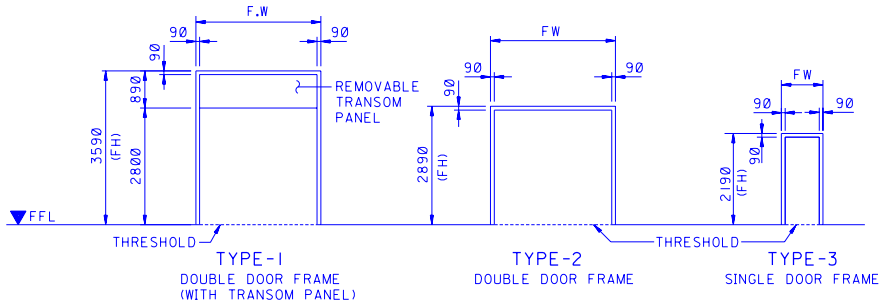
LEGEND & ABBREVIATIONS	
STL	STEEL
FW	FRAME WIDTH
WD	WIDTH
HT	HEIGHT
FH	FRAME HEIGHT
NDW	NET DOOR WIDTH
NDH	NET DOOR HEIGHT

REFERENCE DRAWINGS	
FLOOR PLAN	DA-950153.001
BUILDING ELEVATION	DA-950154.001
BUILDING SECTION	DA-950155.001

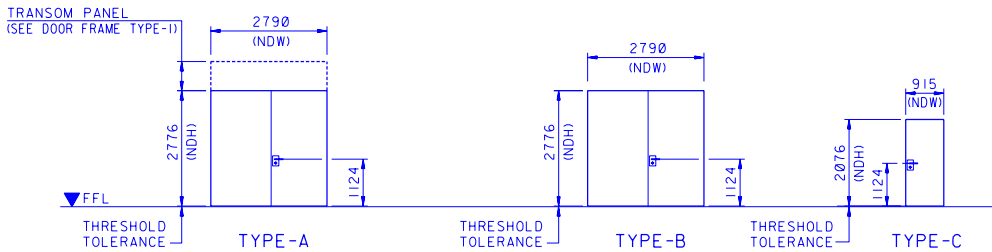
SCALE: _____

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____			CHECKED BY _____			
DATE STARTED _____			DATE COMPLETED _____			
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.			CERTIFIED BY: _____		NAME: _____	
			SIGNATURE _____		DATE _____	
DOOR SCHEDULE						DRAWING TYPE
						SCH
STANDARD SUBSTATION - 10M WIDE						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER		SHT. NO.	REV. NO.	
M88	R	DA-950157		001	00	

00	002	DA-950157	R	M88
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.



DOOR FRAME TYPE-ELEVATION



DOOR ELEVATION

DOOR SCHEDULE																
DOOR NUMBER	DOOR DESCRIPTION					DOOR FRAME TYPE		DOOR LEAF TYPE		WEATHER STRIPPING	HARDWARE PER SSD-8			FIRE RATING	DOOR FUNCTION	REMARKS
	ROUGH OPENING		SIZE		DOOR THK.	TYPE	MATL.	TYPE	MATL.		LOCK SET	DOOR CLOSER	PANIC BAR			
	WD	HT	WD	HT												
D-1	2800	3500	2790	2776		1	STL	A	STL BLAST RESISTANT	●	●	●	●	3 HR	DOUBLE	WITH REMOVABLE PANEL (SEE NOTES 2,3 & 4)
D-2	925	2100	915	2076		3	STL	C	STL BLAST RESISTANT	●	●	●	●	3 HR	SINGLE	SEE NOTES 2, 3, 4 & 5
D-3	925	2100	915	2076		3	STL	C	STL BLAST RESISTANT	●	●	●	●	3 HR	SINGLE	SEE NOTES 2, 3, 4 & 5
D-4	925	2100	915	2076		3	STL	C	STL BLAST RESISTANT	●	●	●	●	3 HR	SINGLE	SEE NOTES 2, 3, 4 & 5
D-5	2800	2800	2790	2776		2	STL	B	STL BLAST RESISTANT	●	●	●	●	3 HR	DOUBLE	SEE NOTES 2, 3, 4 & 5

- NOTES:
- ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE NOTED.
 - DOOR FRAME & DOOR LEAF PAINT COATING IN ACCORDANCE WITH COATING SYSTEM OF SAES-H-101, APCS-6.
 - ALL STEEL DOOR TO BE BLAST RESISTANT DOORS.
 - ALL DOORS SHALL BE AS PER THE REQUIREMENTS OF SSD-7 & 8
 - LABELS TO BE PROVIDED ON DOORS AS FIRE RATED TESTED DOOR.

LEGEND & ABBREVIATIONS	
STL	STEEL
FW	FRAME WIDTH
WD	WIDTH
HT	HEIGHT
FH	FRAME HEIGHT
NDW	NET DOOR WIDTH
NDH	NET DOOR HEIGHT

REFERENCE DRAWINGS	
FLOOR PLAN	DA-950153.002
BUILDING ELEVATION	DA-950154.002
BUILDING SECTION	DA-950155.002

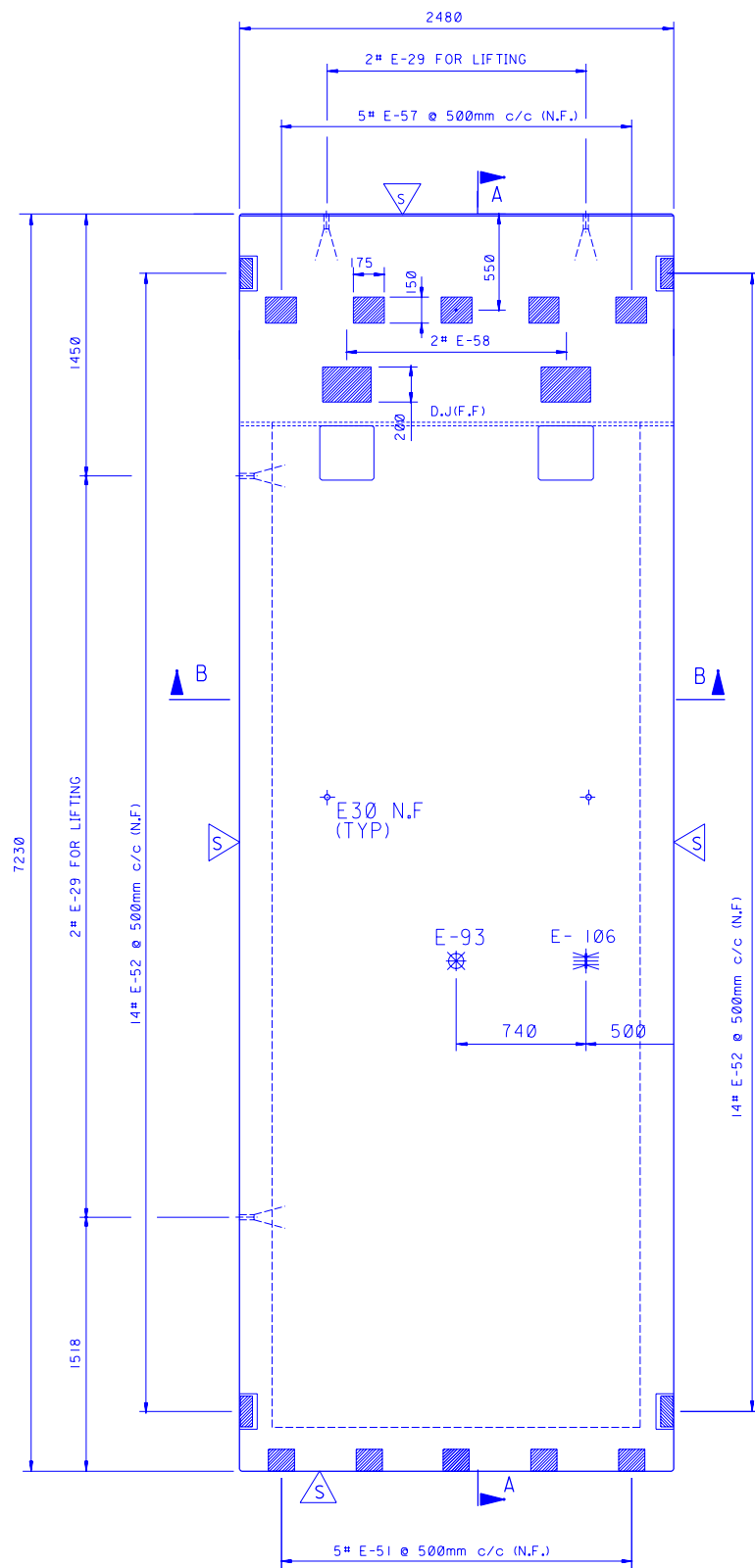
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REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____ CHECKED BY _____						
DATE STARTED _____ DATE COMPLETED _____						
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.				CERTIFIED BY: _____ NAME: _____ DATE: _____		
DOOR SCHEDULE						DRAWING TYPE
STANDARD SUBSTATION - 15M WIDE						SCH
LIBRARY DRAWING						SAUDI ARABIA
PLANT NO.	INDEX	DRAWING NUMBER		SHT. NO.	REV. NO.	
M88	R	DA-950157		002	00	

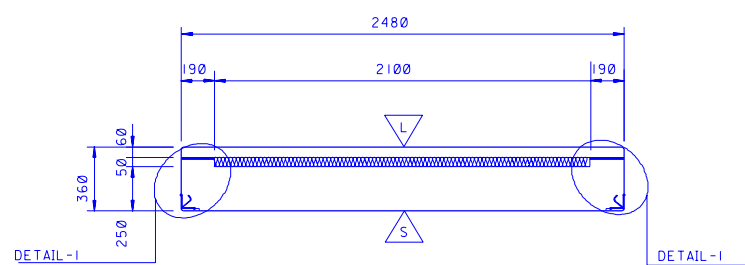
APPENDIX C.2

Wall Panel Drawings

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950158	001	00

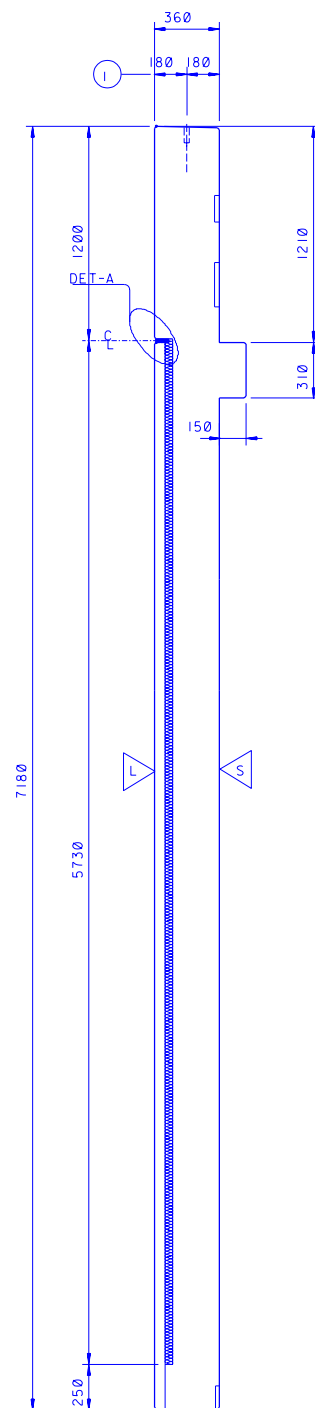


ELEVATION
N T S



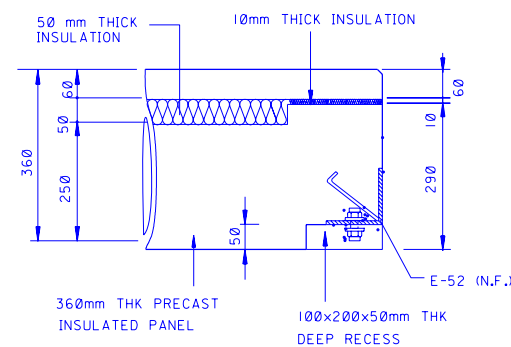
SECTION-B

N T S

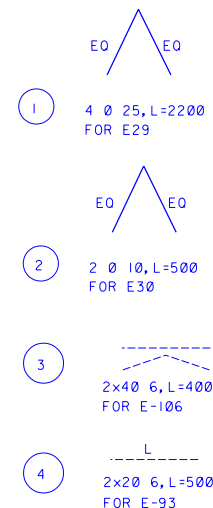


SECTION-A

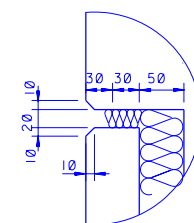
N T S



DETAIL -
N T S



ITEM No.		AS SHOWN				
QTY.	No.	6				
VOL.	m3	5.96				
WEIGHT	to	14.90				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					
CONCRETE COVER 30mm MIN. EXT. / 20mm. MIN. INTERNAL						
MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE						
	DESCRIPTION				QTY FOR 1 UNIT	TOTAL QTY
E29	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T				4	
E30	M16 FOR ERECTION SOCKET				2	
E93	DEHA SLEEVE ANCHOR 5300-150-127				1	
E106	DEHA FLAT ANCHOR 5301-150-160				1	
E51	150x250x12mm THK STEEL PLATE (GALVANIZED)				5	
E52	75x75x5mm THK (L=175) BENT PLATE (GALVANIZED)				28	
E57	150x175x6mm THK STEEL PLATE (GALVANIZED)				5	
E58	350x200x6mm THK STEEL PLATE (GALVANIZED)				2	



DETAIL-A

N T S

REF. DETAIL-1 ON
DA-950159
SH 001 & 002

NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
4. WHITE CEMENT & KHURAI'S AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

	SMOOTH TROWEL FINISH	
	HEAVY SANDBLASTED FINISH	
	LIGHT SANDBLASTED FINISH	
	BROOM FINISH	
	TYP. CHAMFER	10x10

ABBREVIATIONS

	MOULD FACE
	ERECTION DIRECTION
N.F.	NEAR FACE
F.F.	FAR FACE
	BLOCKOUT
	RECESS

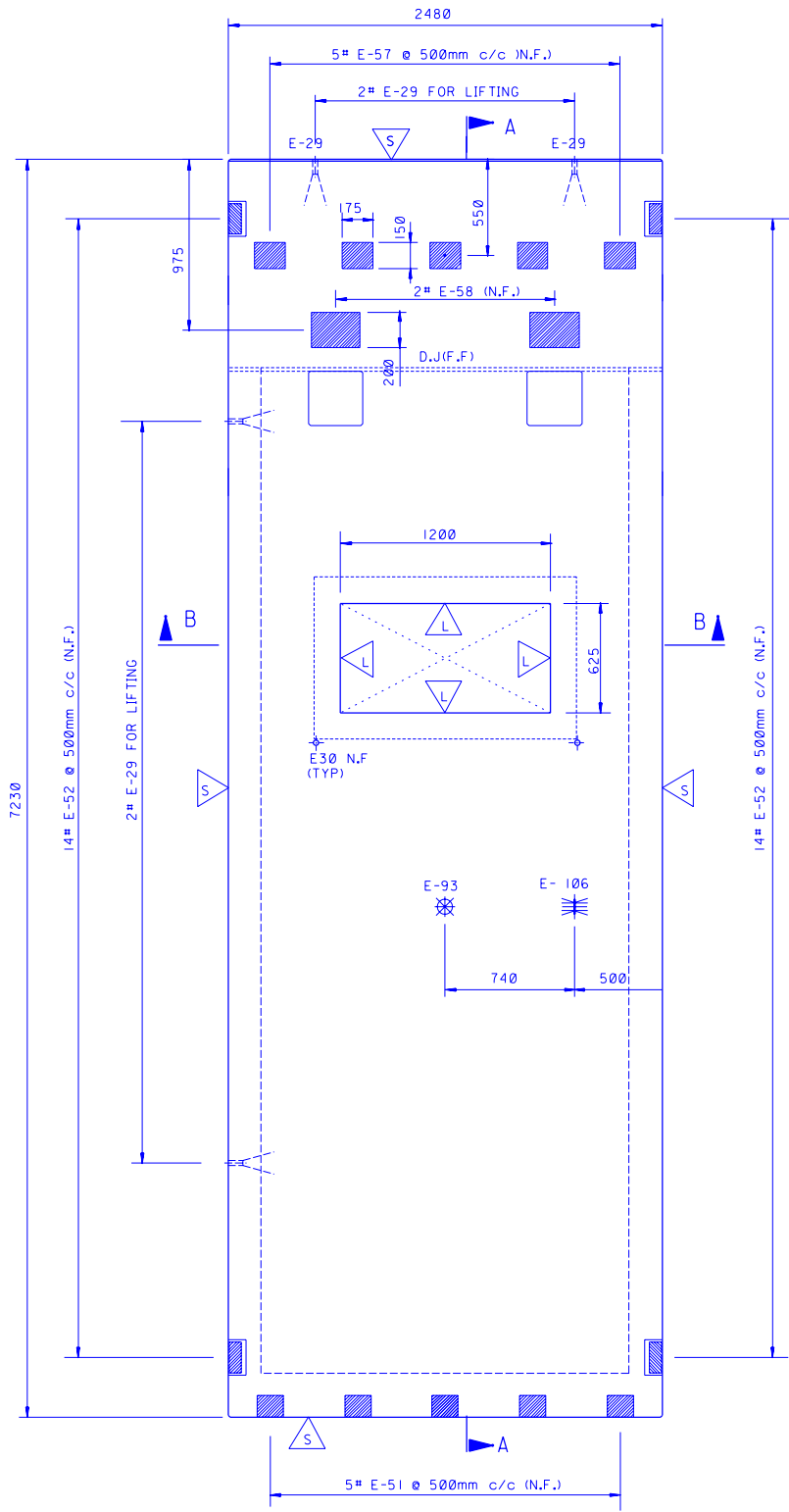
REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

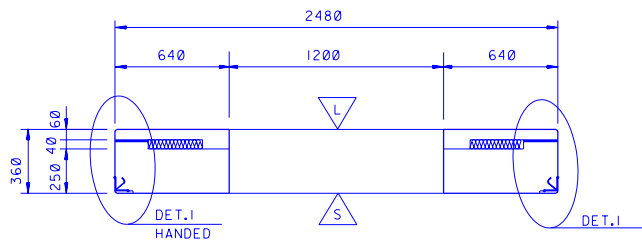
SCALE: N T S

REV.	NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPROV.
SAUDI ARABIAN OIL COMPANY							
DRAWN BY _____				CHECKED BY _____			
DATE STARTED _____				DATE COMPLETED _____			
DISCIPLINE ENGINEER _____							
PROJECT ENGINEER _____							
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNLESS CERTIFIED AND DATED.				CERTIFIED BY: _____ SIGNATURE _____ INCHES DATE _____			
TYP. PRECAST INSULATED PANEL - TYPE I							DRAWING TYPE ARC
STANDARD SUBSTATION							
LIBRARY DRAWING						SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER			SHT. NO.	REV. NO.	
M88	R	DA-950158			001	00	

00	002	002	DA-950158	R	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	

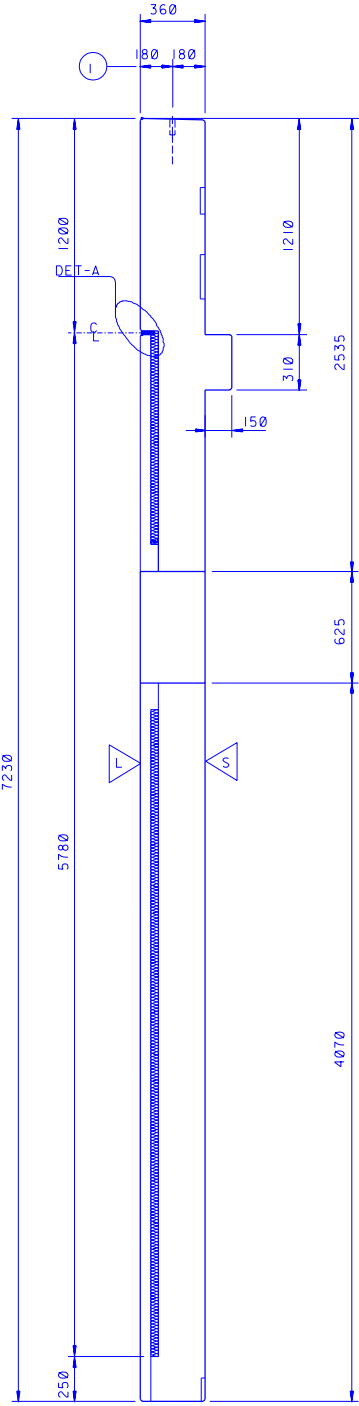


ELEVATION
N T S

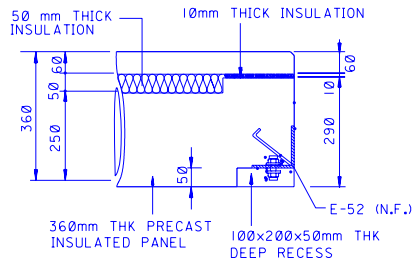


SECTION-B
N T S

WALL PANEL TYPE-2 (STD. PENETRATION)
NOT TO SCALE



SECTION-A
N T S



DETAIL-I
N T S

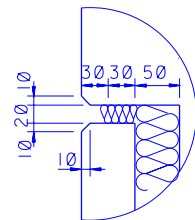
ITEM No. AS SHOWN

QTY.	No.	8				
VOL.	m3	5.75				
WEIGHT	to	14.38				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					

CONCRETE COVER 30mm MIN.EXT. / 20mm. MIN.INTERNAL

MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE

	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY
E29	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T	4	
E30	M16 FOR ERECTION SOCKET	2	
E-93	DEHA SLEEVE ANCHOR 5300-150-127	1	
E-106	DEHA FLAT ANCHOR 5301-150-160	1	
E51	150x250x12mm THK STEEL PLATE (GALVANIZED)	5	
E52	75x75x5mm THK (L=175) BENT PLATE (GALVANIZED)	28	
E57	150x175x6mm THK STEEL PLATE (GALVANIZED)	5	
E58	350X200x6mm THK STEEL PLATE (GALVANIZED)	2	



DETAIL-A
N T S

REF. DETAIL-1 ON
DA-950159
SH 003

EQ
EQ
4 Ø 25, L=2200
FOR E29

EQ
EQ
2 Ø 10, L=500
FOR E30

2x40 6, L=400
FOR E-106

L
2x20 6, L=500
FOR E-93

NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
4. WHITE CEMENT & KHURALS AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- S SMOOTH TROWEL FINISH
- B HEAVY SANDBLASTED FINISH
- L LIGHT SANDBLASTED FINISH
- R BROOM FINISH
- TYP. CHAMFER 10x10

ABBREVIATIONS

- M MOULD FACE
- ERECTION DIRECTION
- N.F. NEAR FACE
- F.F. FAR FACE
- BLOCKOUT
- RECESS

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPRD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

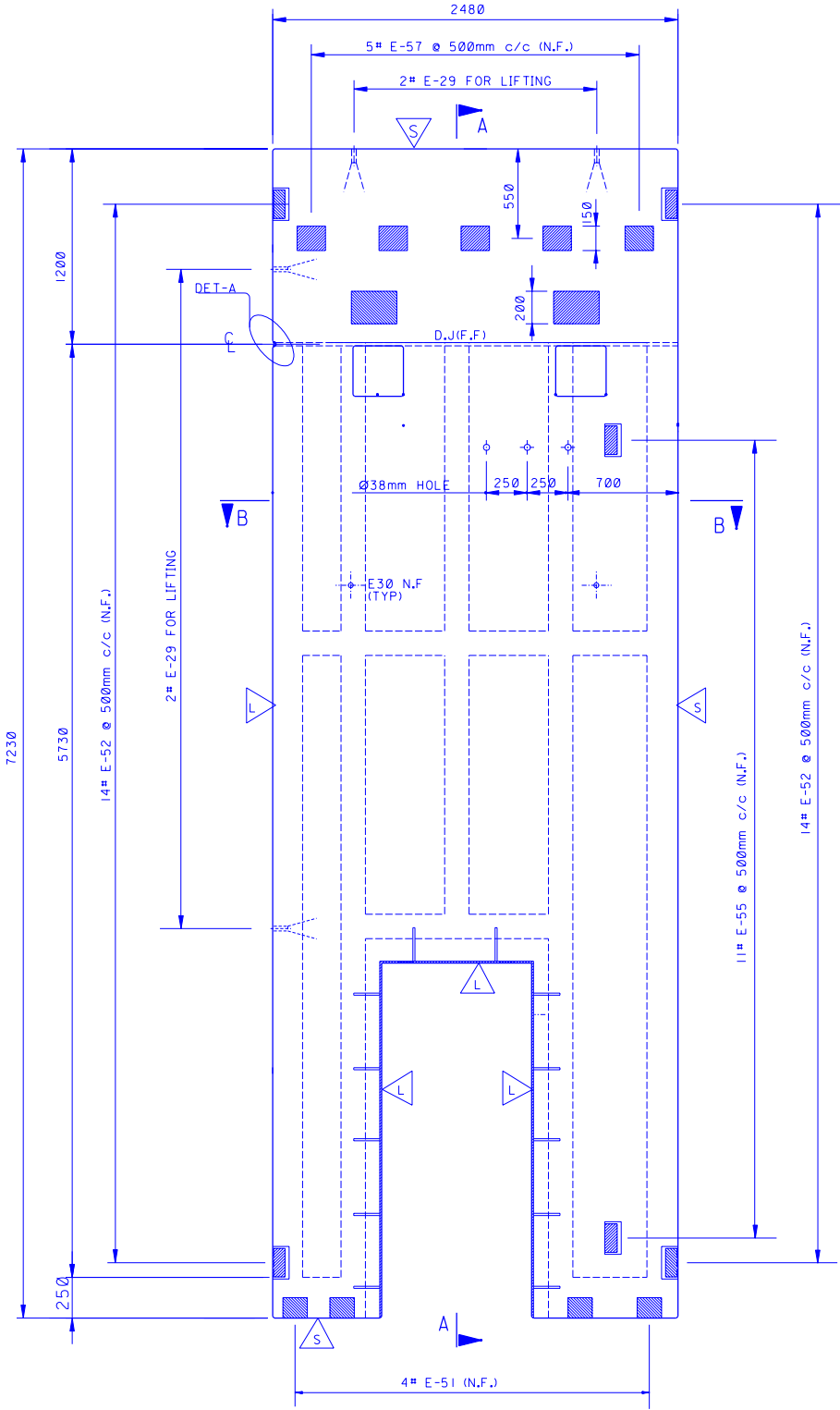
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.
CERTIFIED BY: _____ NAME: _____
SIGNATURE _____ DATE _____

TYP. PRECAST INSULATED PANEL-TYPE-2
ARC

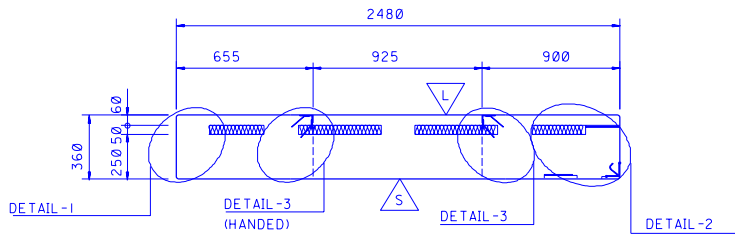
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950158	002	00

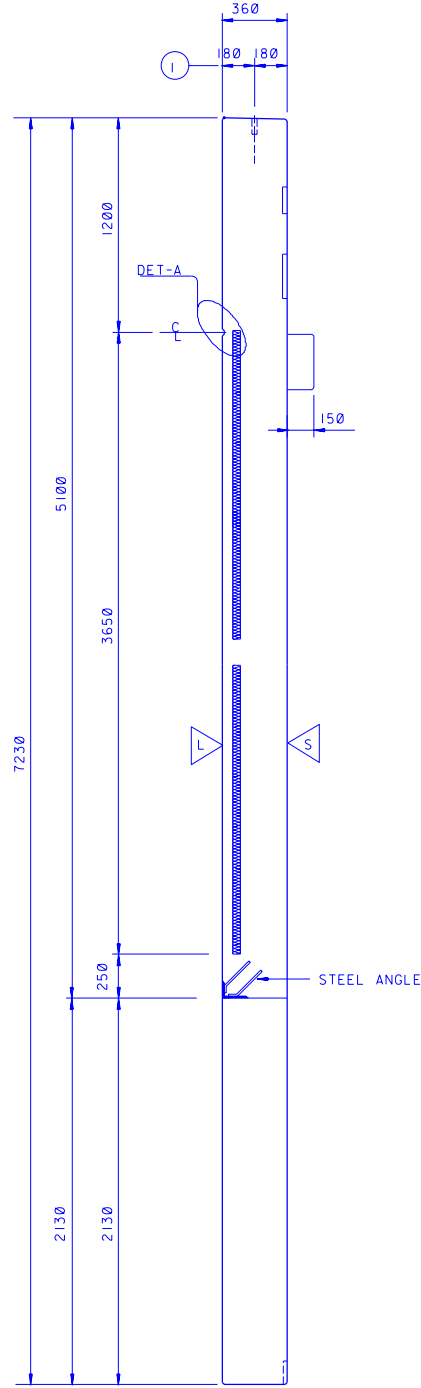
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REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	



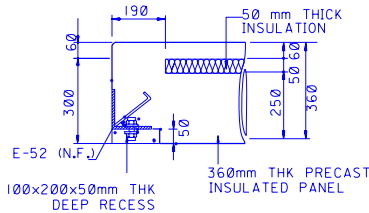
ELEVATION
N T S



SECTION-B
N T S



SECTION-A
N T S

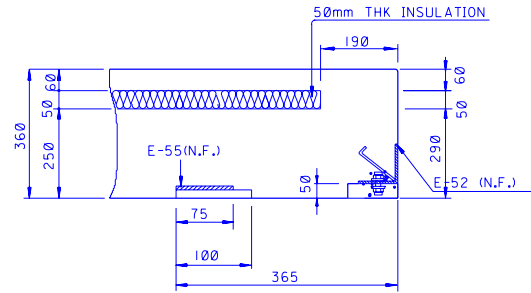


DETAIL-1
N T S

WALL PANEL-TYPE -3 (PERSONNEL DOOR)

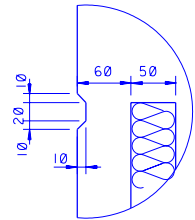
NOT TO SCALE

ITEM No.	AS SHOWN				
QTY.	No.	I			
VOL.	m3	5.20			
WEIGHT	to	13.00			
REBARS	kg				
WIRE MESH	kg				
REINFORCEMENT	kg				
CONCRETE COVER		30mm MIN. EXT. / 20mm. MIN. INTERNAL			
MIX DESIGN		35Mpa N/mm2, WHITE & GREY CONCRETE			
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY		
E29	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T	4			
E30	M16 FOR ERECTION SOCKET	2			
E51	150x250x12mm THK STEEL PLATE (GALVANIZED)	4			
E52	75x75x5mm THK (L=175) BENT PLATE (GALVANIZED)	28			
E57	150x175x6mm THK STEEL PLATE (GALVANIZED)	5			
E58	350x200x6mm THK STEEL PLATE (GALVANIZED)	2			
E55	180x180x10mm THK STEEL PLATE (GALVANIZED)	11			
E67	150x90x10mm THK STEEL ANGLE (GALVANIZED)	1			



DETAIL-2
N T S

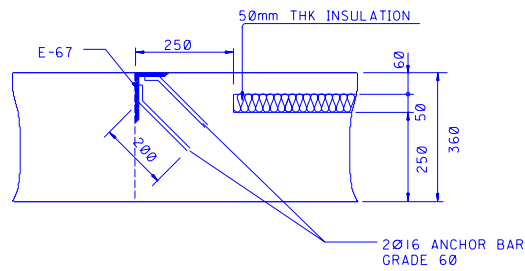
REF. DETAIL-E
ON DA-950159
SH 001 & 002



DETAIL-A
N T S

EO
4 Ø 25, L=2200
FOR E29

EO
2 Ø 10, L=500
FOR E30



DETAIL-3
N T S

REF. DETAIL-P
ON DA-950159
SH 003

NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
4. WHITE CEMENT & KHURALS AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- SMOOTH TROWEL FINISH
- HEAVY SANDBLASTED FINISH
- LIGHT SANDBLASTED FINISH
- BROOM FINISH
- TYP. CHAMFER 10x10

ABBREVIATIONS

- MOULD FACE
- ERECTION DIRECTION
- N.F.
- F.F.
- BLOCKOUT
- RECESS

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
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SAUDI ARABIAN OIL COMPANY

DRAWN BY	CHECKED BY
DATE STARTED	DATE COMPLETED
DISCIPLINE ENGINEER	
PROJECT ENGINEER	

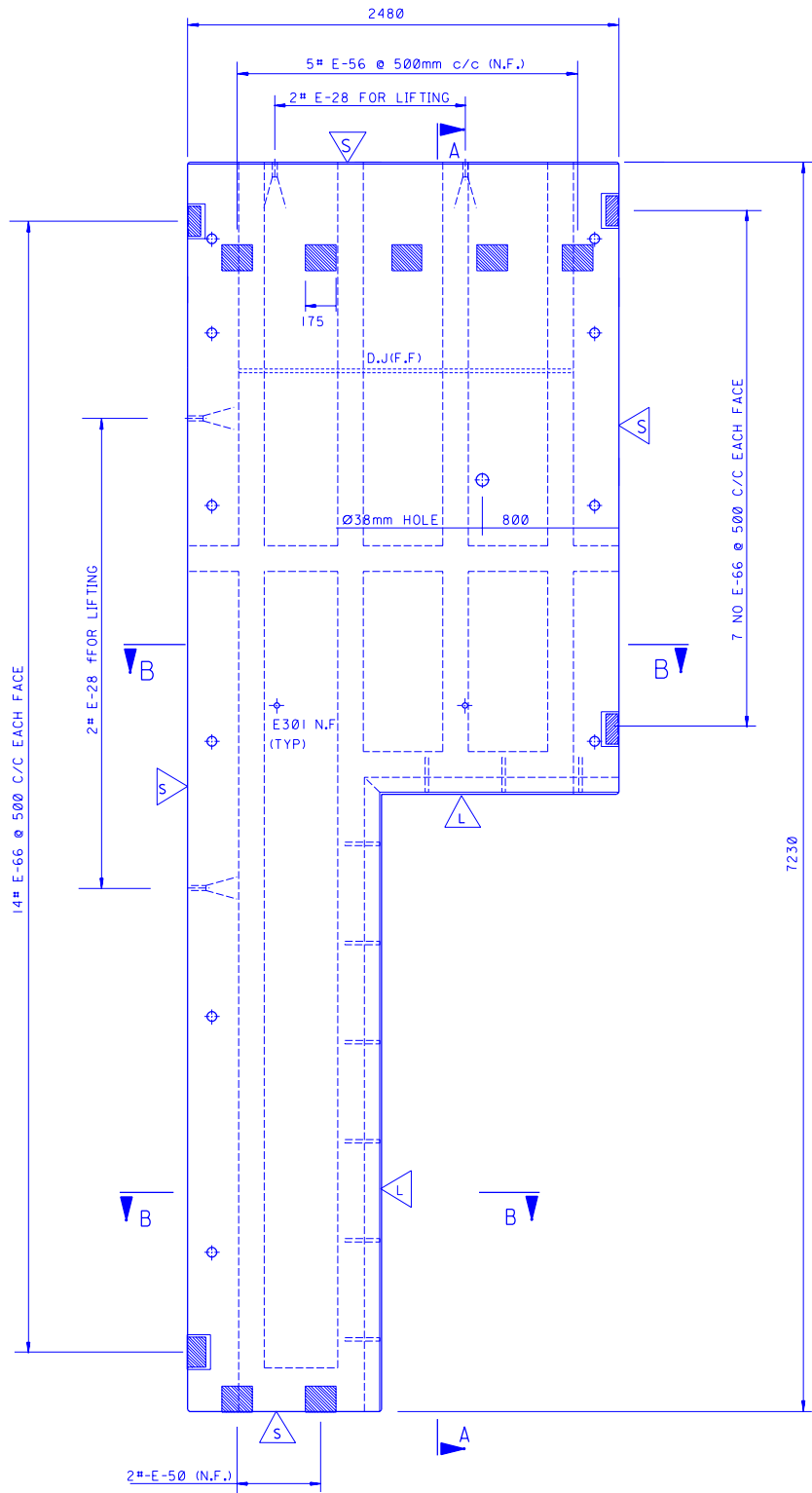
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TYP. PRECAST INSULATED PANEL -TYPE 3	DRAWING TYPE
ARC	

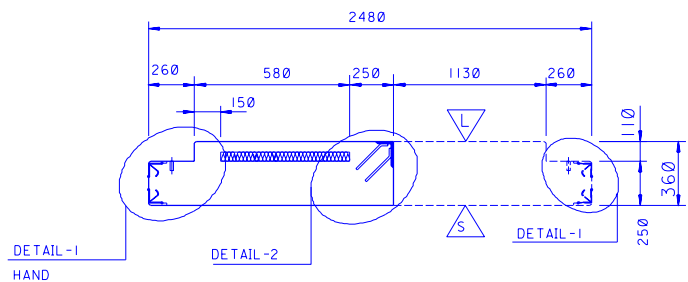
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950158	003	00

00	004	008	DA-950158	R	M88
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.	

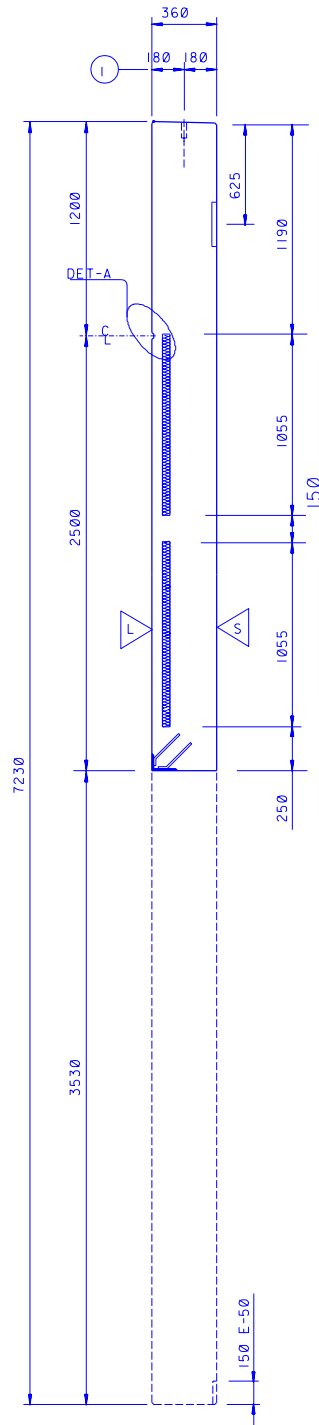


ELEVATION
N T S



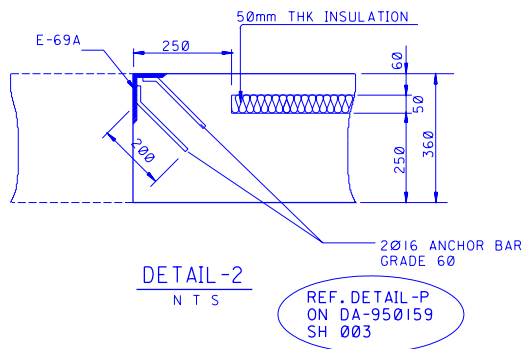
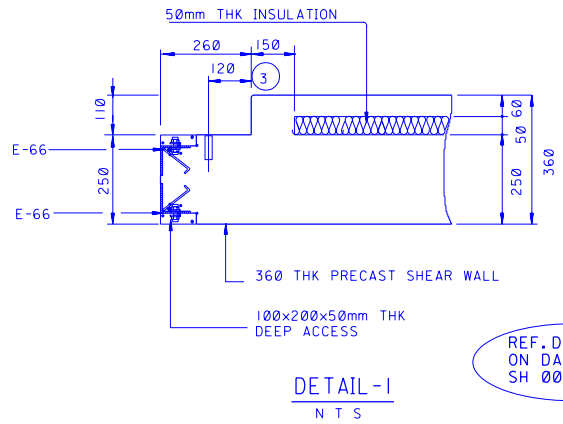
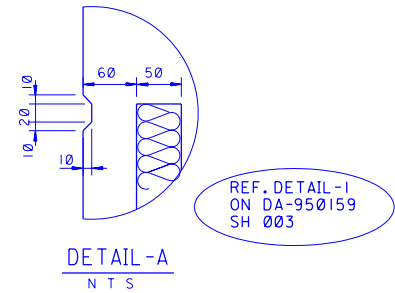
SECTION-B
N T S

SHEAR WALL PANEL - TYPE 4 (DOUBLE DOOR W/TRANSOM)
NOT TO SCALE



SECTION-A
N T S

ITEM No.	AS SHOWN					
QTY.	No.	I				
VOL.	m3	3.86				
WEIGHT	to	9.65				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					
CONCRETE COVER 30mm MIN. EXT. / 20mm. MIN. INTERNAL						
MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE						
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY			
E28	PFEIFER LIFTING SOCKET (Rd 52) I2.0 T	4				
E30	M16 FOR ERECTION SOCKET	2				
E50	150x250x12mm THK STEEL PLATE (GALVANIZED)	2				
E66	75x75x8mm THK (L=180) BENT PLATE (GALVANIZED)	42				
E56	250x175x8mm THK STEEL PLATE (GALVANIZED)	5				
E22	PFEIFER FIXING SOCKET (Rd 16) I.2 T	10				
E69A	150x90x10mm THK STEEL ANGLE (GALVANIZED)	1				



- EQ EQ
- 1 4 Ø 25, L=2200 FOR E28
- EQ EQ
- 2 2 Ø 10, L=500 FOR E30
- 3 10 Ø 10, L=800 FOR E22,FOR FIXING

- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. FINISHING AS PER APPROVED SAMPLE
 3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
 4. WHITE CEMENT & KHURAISS AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- SMOOTH TROWEL FINISH
- HEAVY SANDBLASTED FINISH
- LIGHT SANDBLASTED FINISH
- BROOM FINISH
- TYP. CHAMFER 10x10
- ABBREVIATIONS
- MOULD FACE
- ERECTION DIRECTION
- N.F. NEAR FACE
- F.F. FAR FACE
- BLOCKOUT
- RECESS

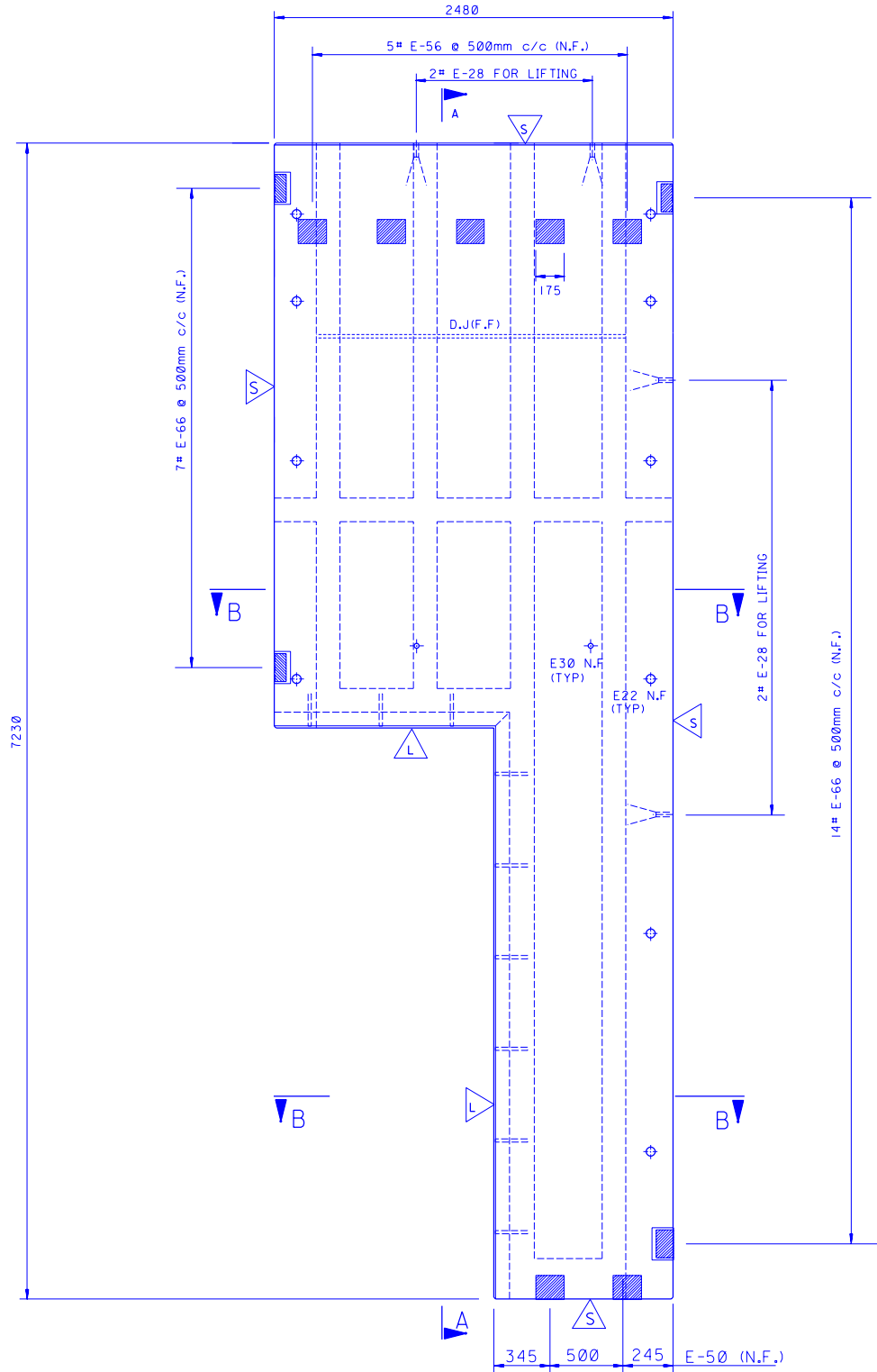
REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

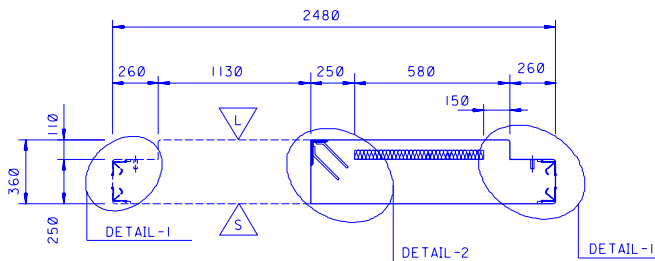
SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____ CHECKED BY _____						
DATE STARTED _____ DATE COMPLETED _____						
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.						
CERTIFIED BY: _____ NAME: _____ DATE: _____						
TYP.PRECAST INSULATED PANEL -TYPE 4						
DRAWING TYPE ARC						
STANDARD SUBSTATION						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
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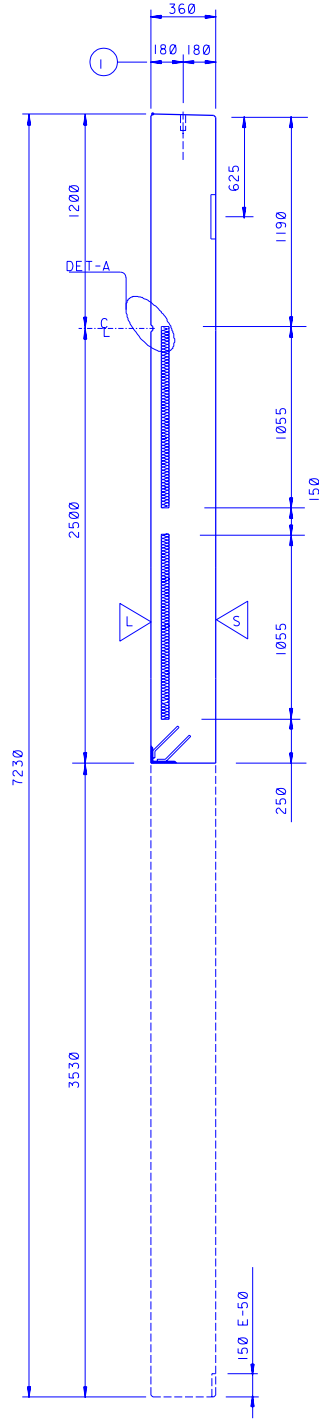
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PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.	



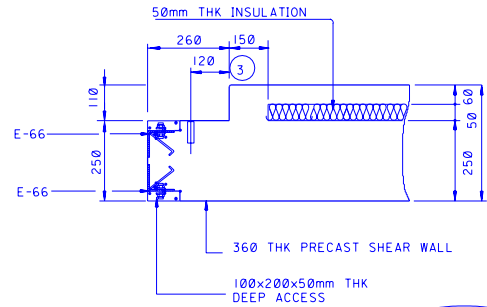
ELEVATION
N T S



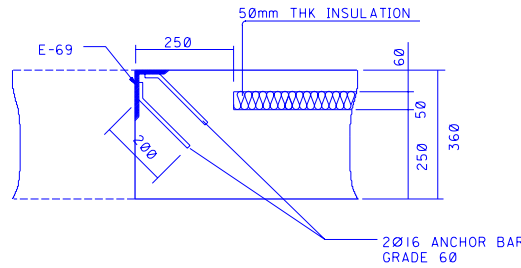
SECTION-B
N T S



SECTION-A
N T S

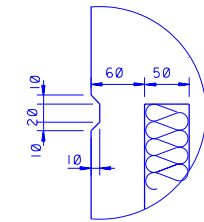


DETAIL-1
N T S



DETAIL-2
N T S

ITEM No.	AS SHOWN					
QTY.	No.	I				
VOL.	m3	3.86				
WEIGHT	to	9.65				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					
CONCRETE COVER 30mm MIN. EXT. / 20mm, MIN. INTERNAL						
MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE						
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY			
E28	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T	4				
E30	M16 FOR ERECTION SOCKET	2				
E50	150x250x12mm THK STEEL PLATE (GALVANIZED)	2				
E66	75x75x8mm THK (L=180) BENT PLATE (GALVANIZED)	42				
E56	250x175x8mm THK STEEL PLATE (GALVANIZED)	5				
E22	PFEIFER FIXING SOCKET (Rd 16) 1.2 T	10				
E69	150x90x10mm THK STEEL ANGLE (GALVANIZED)	1				



DETAIL-A
N T S

REF. DETAIL-1
ON DA-950159
SH 003

REF. DETAIL-D
ON DA-950159
SH 001 & 002

REF. DETAIL-P
ON DA-950159
SH 003

- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. FINISHING AS PER APPROVED SAMPLE
 3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
 4. WHITE CEMENT & KHURAI AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- SMOOTH TROWEL FINISH
- HEAVY SANDBLASTED FINISH
- LIGHT SANDBLASTED FINISH
- BROOM FINISH
- TYP. CHAMFER 10x10

ABBREVIATIONS

- MOULD FACE
- ERECTION DIRECTION
- N.F. NEAR FACE
- F.F. FAR FACE
- BLOCKOUT
- RECESS

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
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SAUDI ARABIAN OIL COMPANY

DRAWN BY	CHECKED BY
DATE STARTED	DATE COMPLETED
DISCIPLINE ENGINEER	
PROJECT ENGINEER	

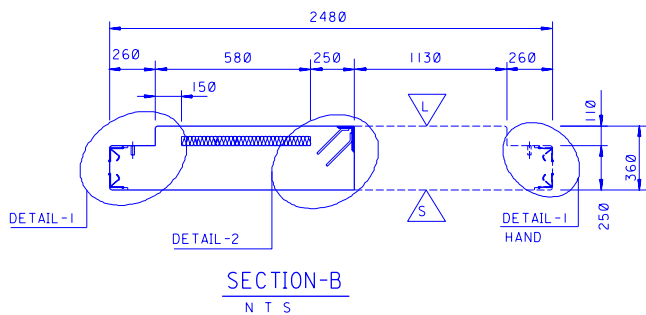
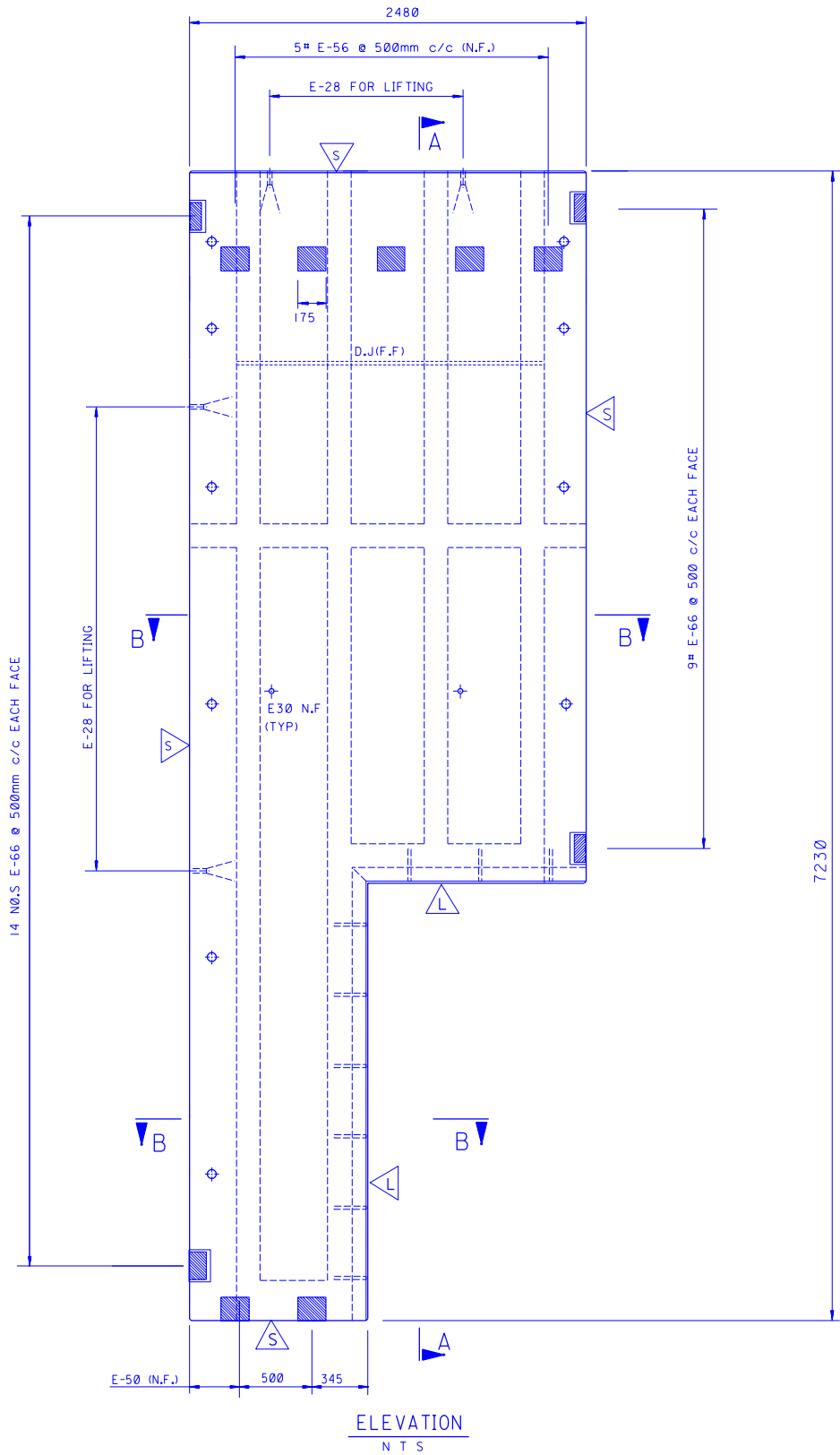
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY:	SIGNATURE	NAME	DATE
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TYP. PRECAST INSULATED PANEL-TYPE 5	DRAWING TYPE
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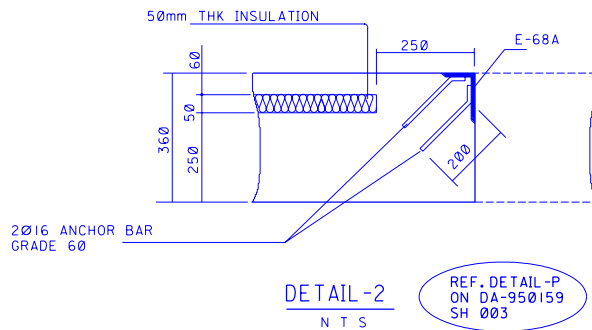
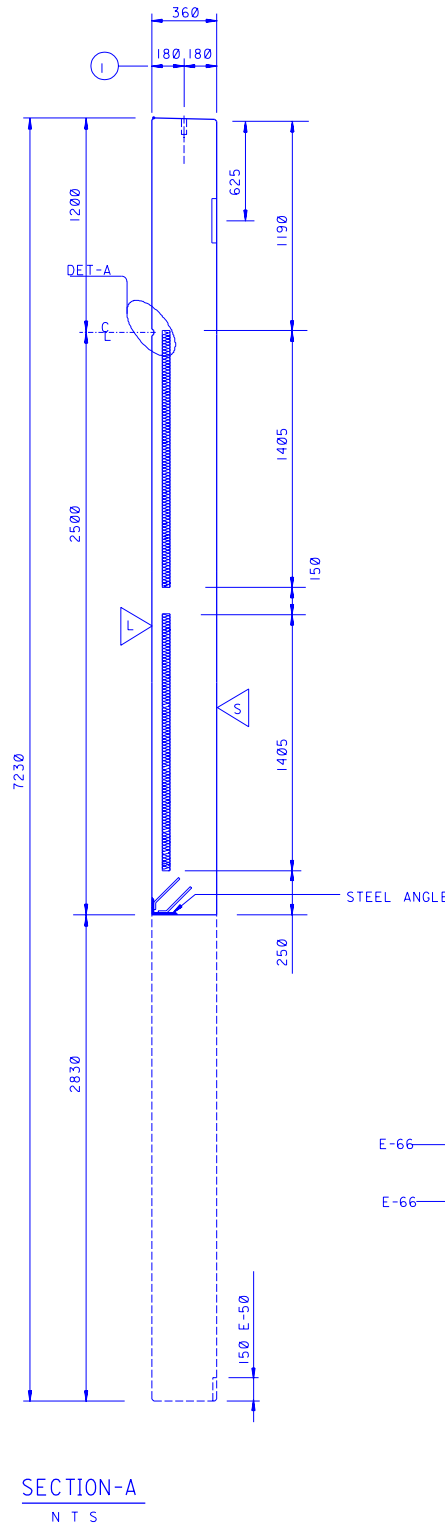
STANDARD SUBSTATION

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950158	005	00

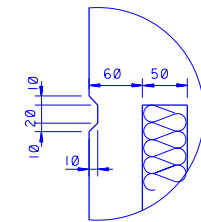
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PLANT NO.	INDEX	XREF	DRAWING NUMBER	SHT. NO.	REV. NO.



SHEAR WALL PANEL-TYPE 6 (DOUBLE DOOR)
NOT TO SCALE

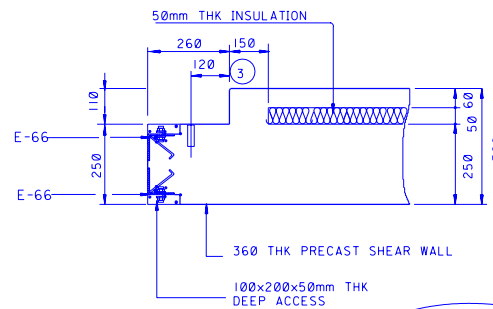


ITEM No.		AS SHOWN					
QTY.	No.	I					
VOL.	m3	4.21					
WEIGHT	to	10.53					
REBARS	kg						
WIRE MESH	kg						
REINFORCEMENT	kg						
CONCRETE COVER		30mm MIN. EXT. / 20mm. MIN. INTERNAL					
MIX DESIGN		35Mpa. N/mm2, WHITE & GREY CONCRETE					
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY				
E28	PFEIFER LIFTING SOCKET (Rd 42) 8.0 T	4					
E30	M16 FOR ERECTION SOCKET	2					
E50	150x250x12mm THK STEEL PLATE (GALVANIZED)	2					
E66	75x75x8mm THK (L=180) BENT PLATE (GALVANIZED)	46					
E56	250x175x8mm THK STEEL PLATE (GALVANIZED)	5					
E22	PFEIFER FIXING SOCKET (Rd 16) 1.2 T	10					
E68A	150x90x10mm THK STEEL ANGLE (GALVANIZED)	1					



DETAIL-A
N T S

REF. DETAIL-I
ON DA-950159
SH 003



DETAIL-I
N T S

REF. DETAIL-D
ON DA-950159
SH 001 & 002

- EQ EQ
- 1 4 Ø 25, L=2200 FOR E28
- EQ EQ
- 2 2 Ø 10, L=500 FOR E30
- 3 10 Ø 10, L=800 FOR E22, FOR FIXING

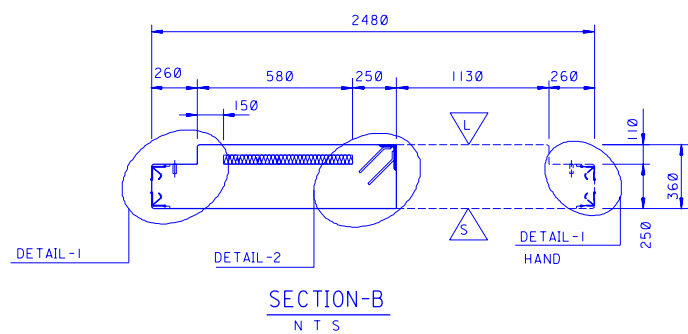
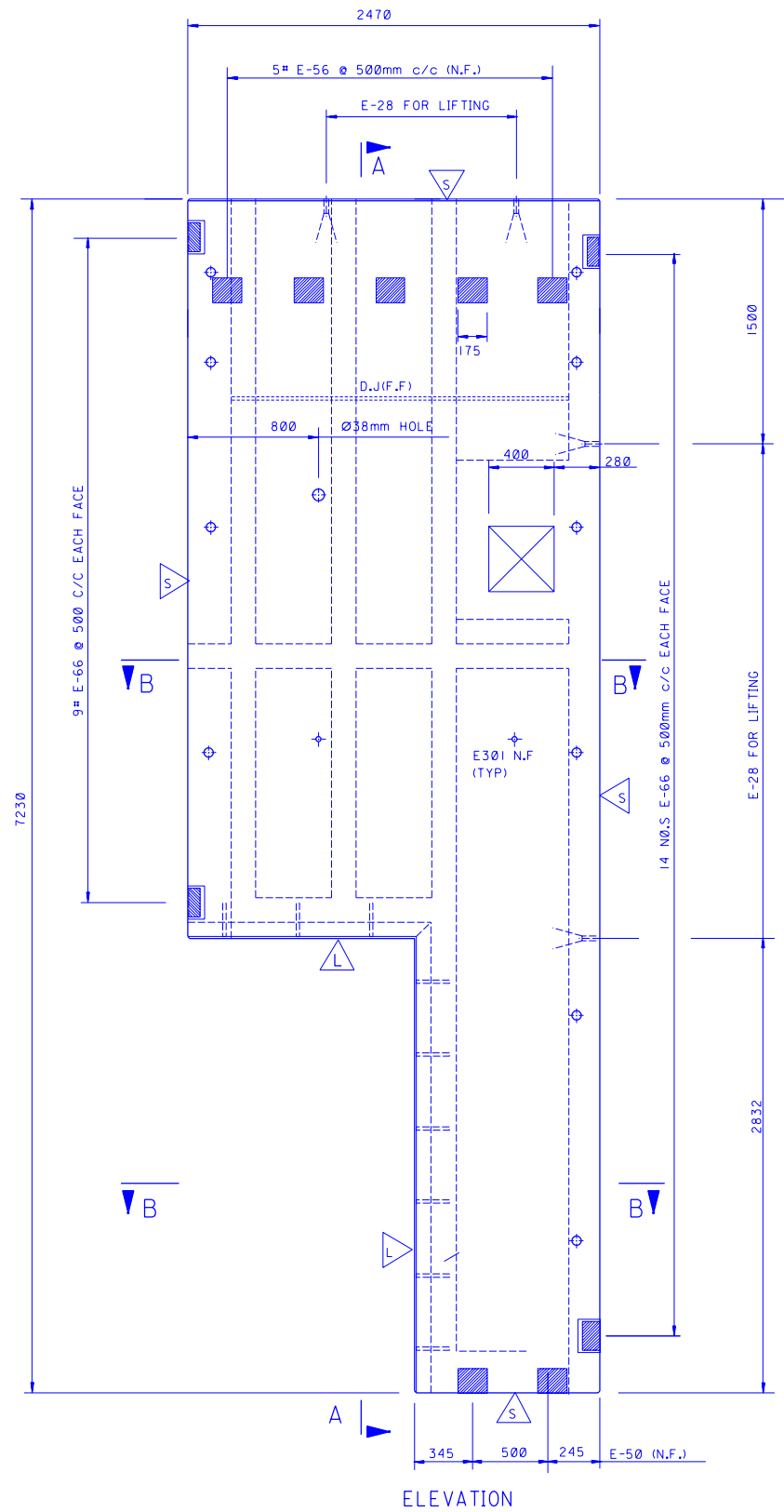
- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. FINISHING AS PER APPROVED SAMPLE
 3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
 4. WHITE CEMENT & KHURAI AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND	
	SMOOTH TROWEL FINISH
	HEAVY SANDBLASTED FINISH
	LIGHT SANDBLASTED FINISH
	BROOM FINISH
	TYP. CHAMFER 10x10
ABBREVIATIONS	
	MOULD FACE
	ERECTION DIRECTION
	NEAR FACE
	FAR FACE
	BLOCKOUT
	RECESS

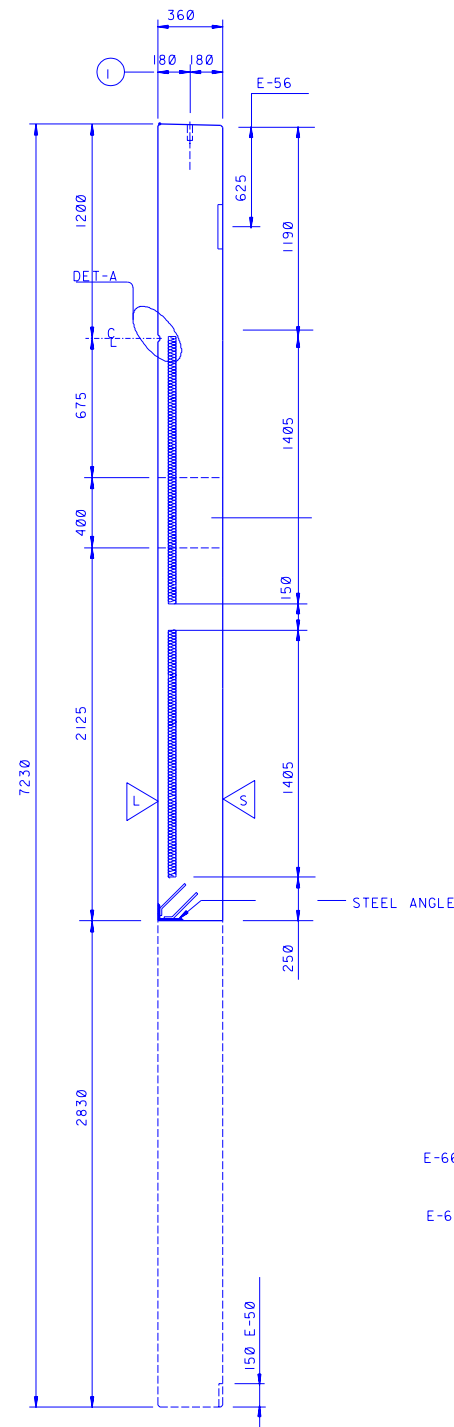
REFERENCE DRAWINGS	
GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPV.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____ CHECKED BY _____						
DATE STARTED _____ DATE COMPLETED _____						
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.						
CERTIFIED BY: _____ NAME: _____ DATE: _____						
TYP. PRECAST INSULATED PANEL-TYPE 6						
STANDARD SUBSTATION						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
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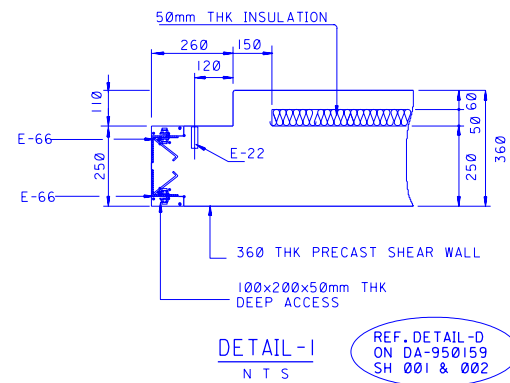
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M88	R	DA-950158	007	00



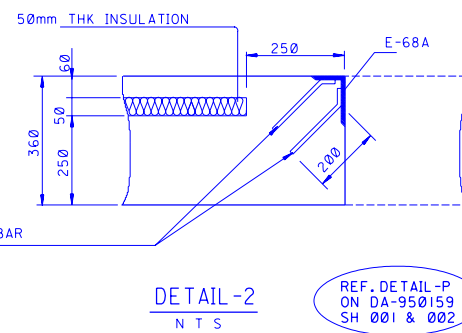
WALL PANEL-TYPE 7 (DOUBLE DOOR)
NOT TO SCALE



SECTION-A

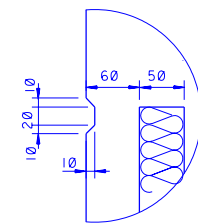


DETAIL -
N T S



DETAIL-2
N T S

ITEM No.		AS SHOWN				
QTY.	No.	1				
VOL.	m3	4.33				
WEIGHT	to	10.83				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					
CONCRETE COVER 30mm MIN.EXT. / 20mm. MIN.INTERNAL						
MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE						
	DESCRIPTION		QTY FOR 1 UNIT	TOTAL QTY		
E28	PFEIFER LIFTING SOCKET (Rd 42) 8.0 T		4			
E30	M16 FOR ERECTION SOCKET		2			
E50	150x250x12mm THK STEEL PLATE (GALVANIZED)		2			
E66	75x75x8mm THK (L=180) BENT PLATE (GALVANIZED)		18			
E54	75x75x10mm THK (L=180) BENT PLATE (GALVANIZED)		14			
E56	250x175x8mm THK STEEL PLATE (GALVANIZED)		5			
E22	PFEIFER FIXING SOCKET (Rd 16) 1.2 T		4			
E68	150x90x10mm THK STEEL ANGLE (GALVANIZED)		1			



DETAIL - A

N T S

REF. DETAIL-1
ON DA-950159
SH 003

NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
4. WHITE CEMENT & KHURAI'S AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND		
	SMOOTH TROWEL FINISH	
	HEAVY SANDBLASTED FINISH	
	LIGHT SANDBLASTED FINISH	
	BROOM FINISH	
	TYP. CHAMFER	10x10
<u>ABBREVIATIONS</u>		
	MOULD FACE	
	ERECTION DIRECTION	
N.F.	NEAR FACE	
F.F.	FAR FACE	
	BLOCKOUT	
	RECESS	

REFERENCE DRAWINGS	
GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: NTS

1

EO  EO

4 Ø 25, L=2200
FOR E28

EO  EO

2

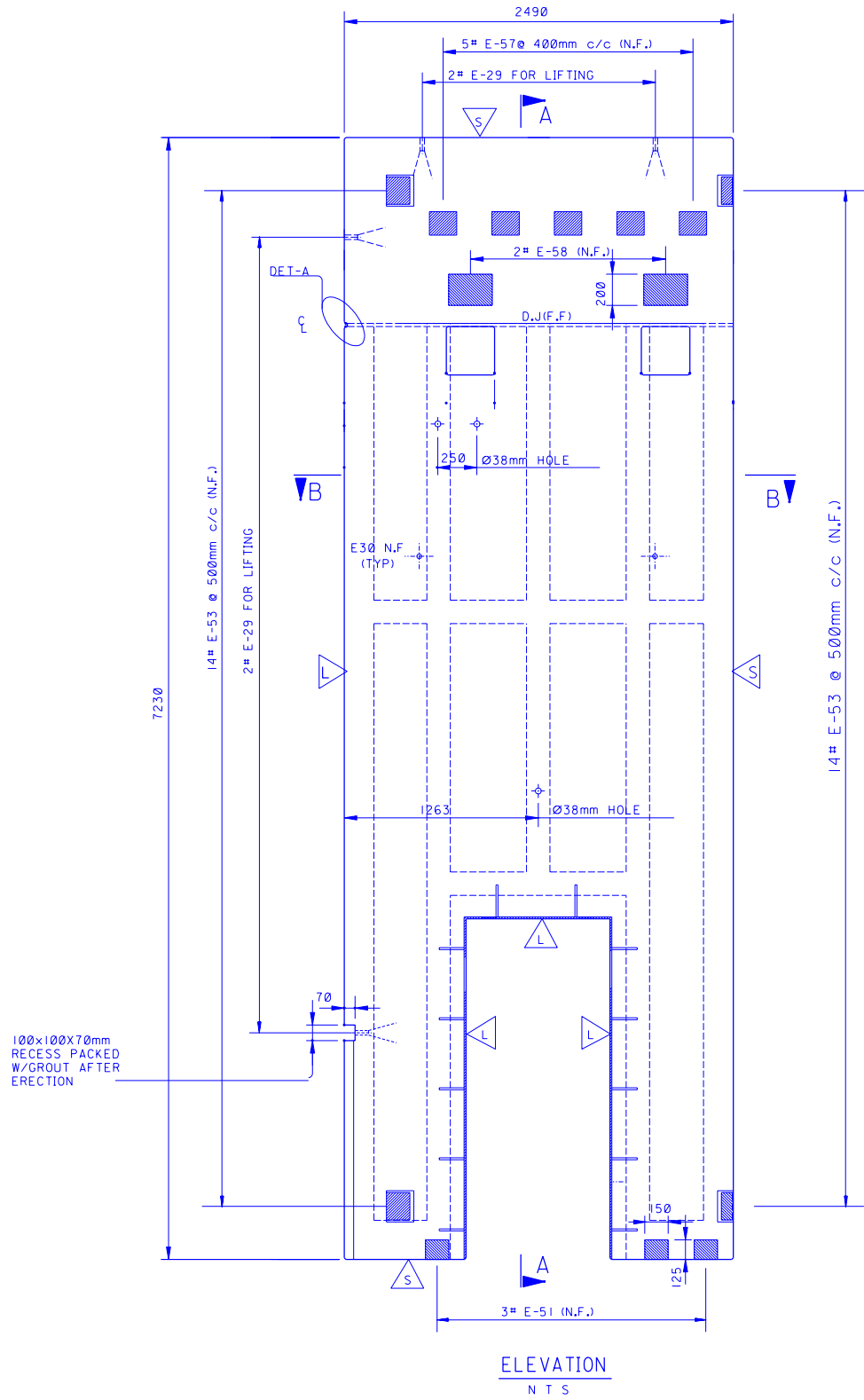
2 Ø 10, L=500
FOR E30

3

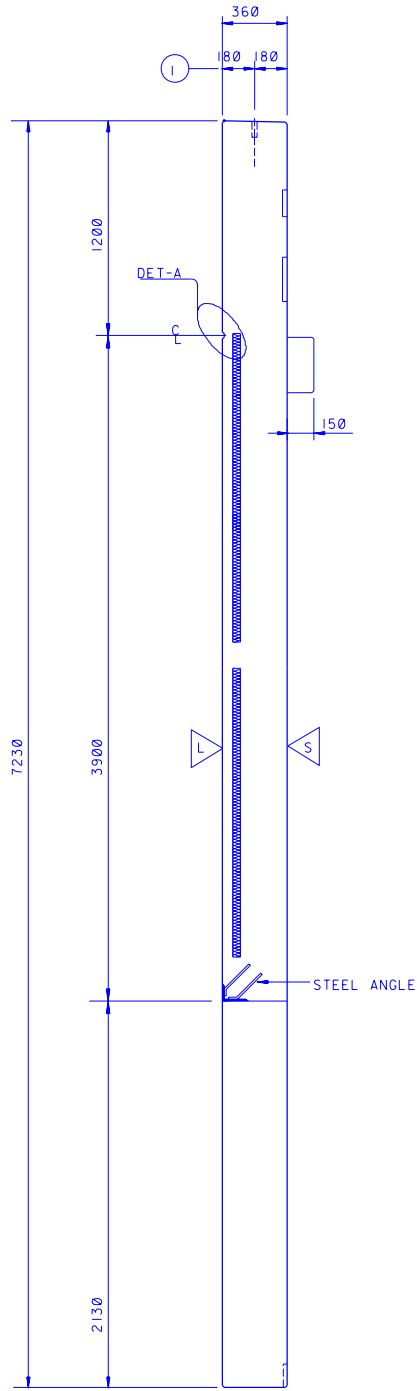
10 Ø 10, L=800
FOR E22, FOR FIXING

NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPROV.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____			CHECKED BY _____			
DATE STARTED _____			DATE COMPLETED _____			
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.			CERTIFIED BY: _____ SIGNATURE _____		NUMBER _____ DATE _____	
TYP. PRECAST INSULATED PANEL - TYPE 7					DRAWING TYPE ARC	
STANDARD SUBSTATION						
LIBRARY DRAWING				SAUDI ARABIA		
PLANT NO.	INDEX	DRAWING NUMBER		SHT. NO.	REV. NO.	
M88	R	DA-950158		007	00	

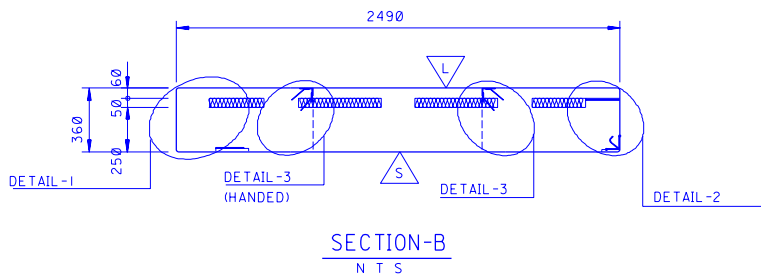
00	008	8	DA-950158	R	M88
REV. NO.	SHT. NO.	INDEX	DRAWING NUMBER	PLANT NO.	INDEX



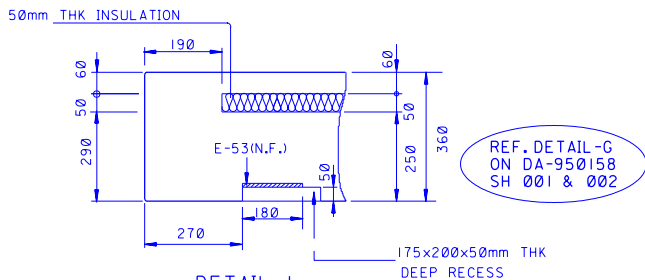
100x100x70mm
RECESS PACKED
W/GROUT AFTER
ERECTION



SECTION-A
N T S



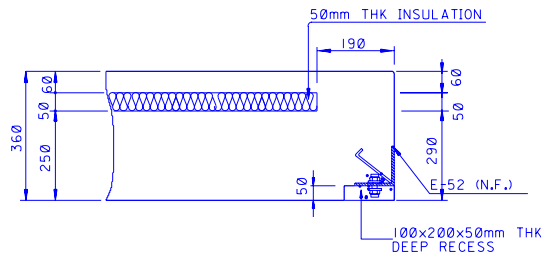
SECTION-B
N T S



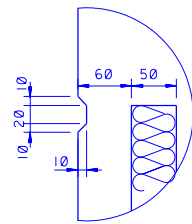
DETAIL-1
N T S

WALL PANEL TYPE-8(CORNER DOOR PANEL)
NOT TO SCALE

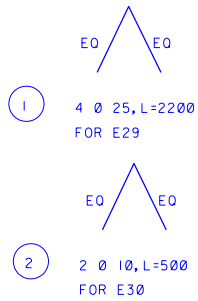
ITEM No.	AS SHOWN				
QTY.	No.	I			
VOL.	m3	5.25			
WEIGHT	to	13.13			
REBARS	kg				
WIRE MESH	kg				
REINFORCEMENT	kg				
CONCRETE COVER		30mm MIN. EXT. / 20mm MIN. INTERNAL			
MIX DESIGN		35Mpa N/mm2, WHITE & GREY CONCRETE			
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY		
E29	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T	4			
E30	M16 FOR ERECTION SOCKET	2			
E51	150x250x12mm THK STEEL PLATE (GALVANIZED)	3			
E52	75x75x5mm THK (L=175) BENT PLATE (GALVANIZED)	14			
E57	150x175x6mm THK STEEL PLATE (GALVANIZED)	5			
E58	350x200x6mm THK STEEL PLATE (GALVANIZED)	2			
E53	180x180x10mm THK STEEL PLATE (GALVANIZED)	14			
E67	150x90x10mm THK STEEL ANGLE (GALVANIZED)	1			



DETAIL-2
N T S



DETAIL-A
N T S



DETAIL-3
N T S

- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. FINISHING AS PER APPROVED SAMPLE
 3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
 4. WHITE CEMENT & KHURAI AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- SMOOTH TROWEL FINISH
- HEAVY SANDBLASTED FINISH
- LIGHT SANDBLASTED FINISH
- BROOM FINISH
- TYP. CHAMFER 10x10

ABBREVIATIONS

- MOULD FACE
- ERECTION DIRECTION
- N.F.
- F.F.
- BLOCKOUT
- RECESS

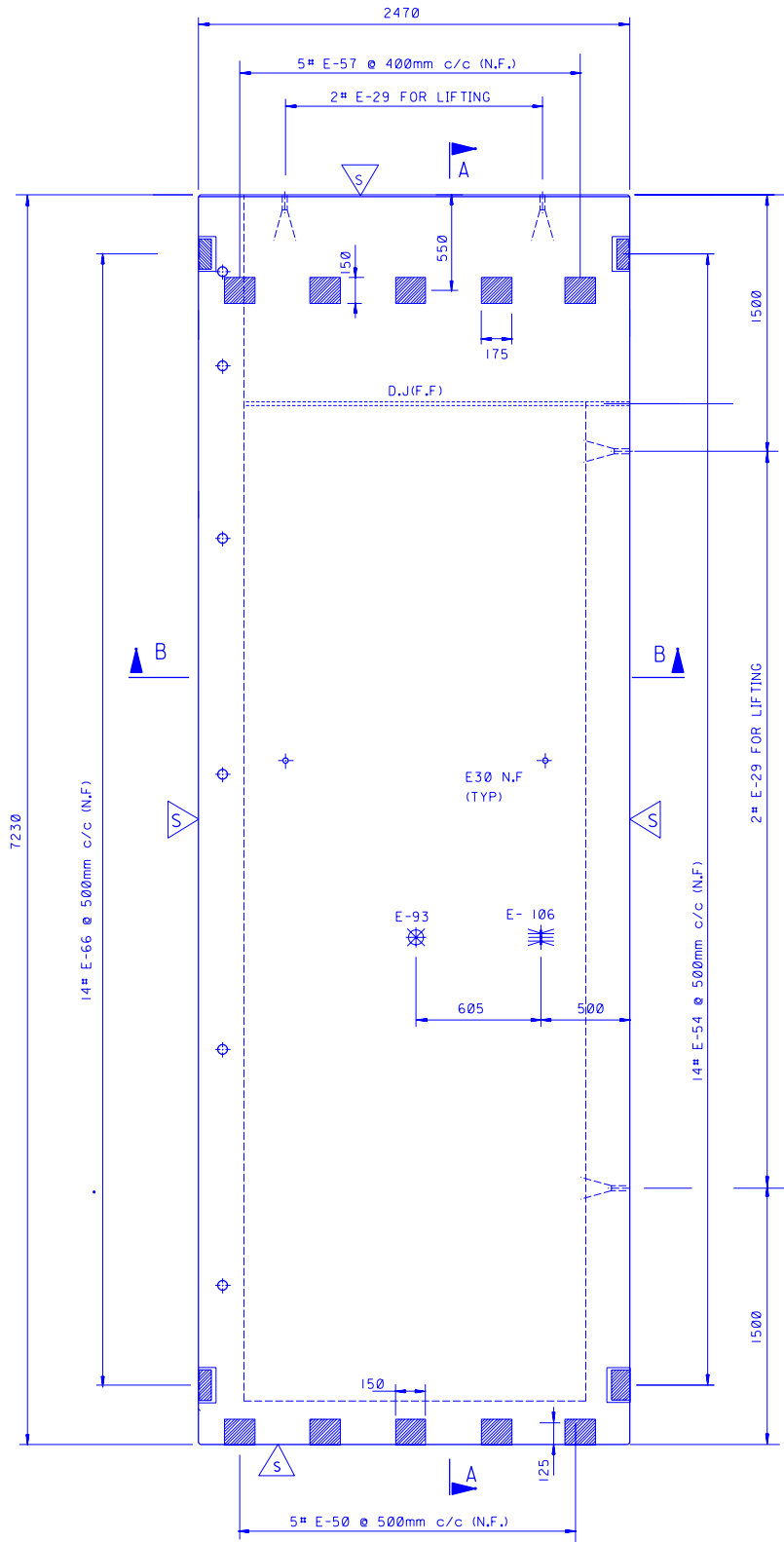
REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

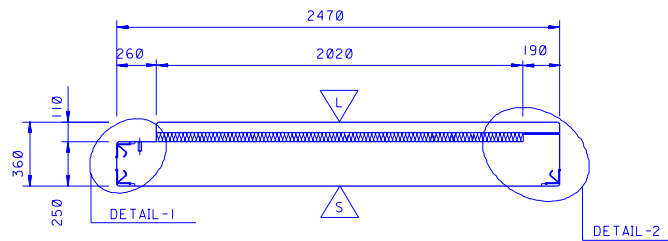
SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPV.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY: _____ CHECKED BY: _____						
DATE STARTED: _____ DATE COMPLETED: _____						
DISCIPLINE ENGINEER: _____ PROJECT ENGINEER: _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.						
CERTIFIED BY: _____ NAME: _____ DATE: _____						
TYP. PRECAST INSULATED PANEL - TYPE 8						
STANDARD SUBSTATION						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950158	008	00		

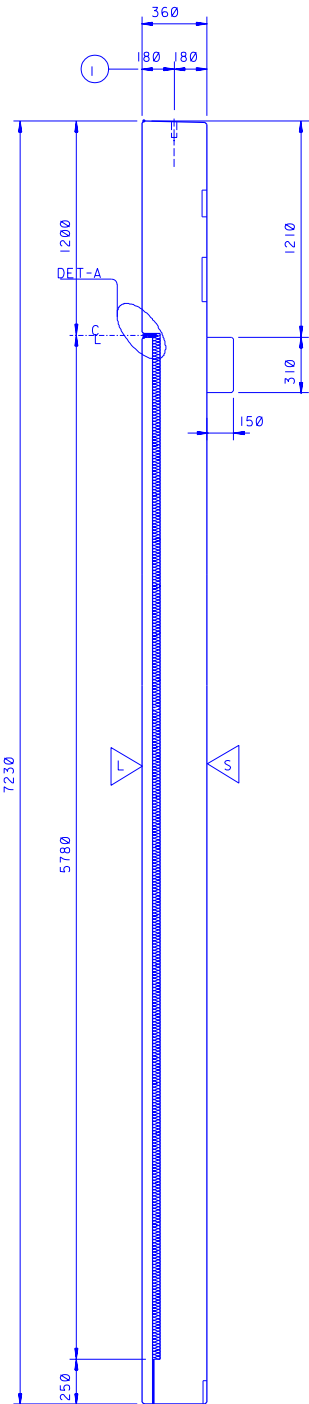
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PLANT NO.	INDEX	XREF	DRAWING NUMBER	REV. NO.	SHT. NO.



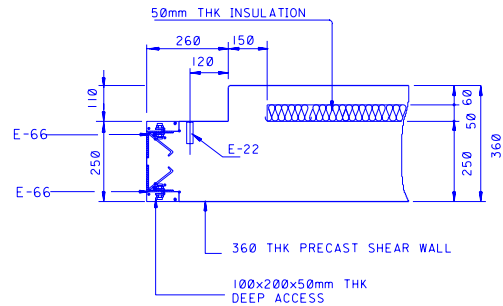
ELEVATION
N T S



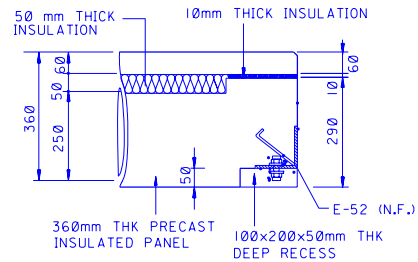
SECTION-B
N T S



SECTION-A
N T S

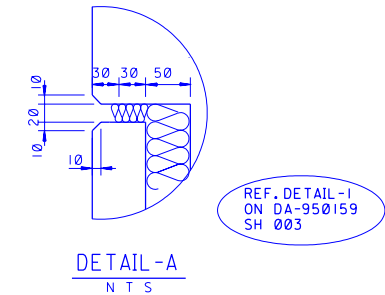


DETAIL-A
N T S



DETAIL-2
N T S

ITEM No.	AS SHOWN				
QTY.	No.	I			
VOL.	m3	5.60			
WEIGHT	to	14.00			
REBARS	kg				
WIRE MESH	kg				
REINFORCEMENT	kg				
CONCRETE COVER	30mm MIN. EXT. / 20mm. MIN. INTERNAL				
MIX DESIGN	35Mpa N/mm2, WHITE & GREY CONCRETE				
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY		
E29	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T	4			
E30	M16 FOR ERECTION SOCKET	2			
E-50	150x250x12mm THK STEEL PLATE (GALVANIZED)	5			
E-66	75x75x8mm THK (L=180) BENT PLATE (GALVANIZED)	28			
E54	75x75x10mm THK (L=180) BENT PLATE (GALVANIZED)	14			
E56	250x175x8mm THK STEEL PLATE (GALVANIZED)	5			
E22	PFEIFER FIXING SOCKET (Rd 16) 1.2 T	5			
E93	DEHA SLEEVE ANCHOR 5300-150-127	1			
E106	DEHA FLAT ANCHOR 5301-150-160	1			



DETAIL-A
N T S

DETAIL-1
N T S

- 1 4 Ø 25, L=2200 FOR E29
- 2 2 Ø 10, L=500 FOR E30
- 3 6 Ø 10, L=800 FOR E22, FOR FIXING
- 4 2x40 6, L=400 FOR E-106
- 5 2x20 6, L=500 FOR E-93

- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. FINISHING AS PER APPROVED SAMPLE
 3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
 4. WHITE CEMENT & KHURALS AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- △ S SMOOTH TROWEL FINISH
- △ B HEAVY SANDBLASTED FINISH
- △ L LIGHT SANDBLASTED FINISH
- △ R BROOM FINISH
- △ TYP. CHAMFER 10x10

ABBREVIATIONS

- △ M MOULD FACE
- △ ERECTION DIRECTION
- N.F. NEAR FACE
- F.F. FAR FACE
- ⊗ BLOCKOUT
- ⊞ RECESS

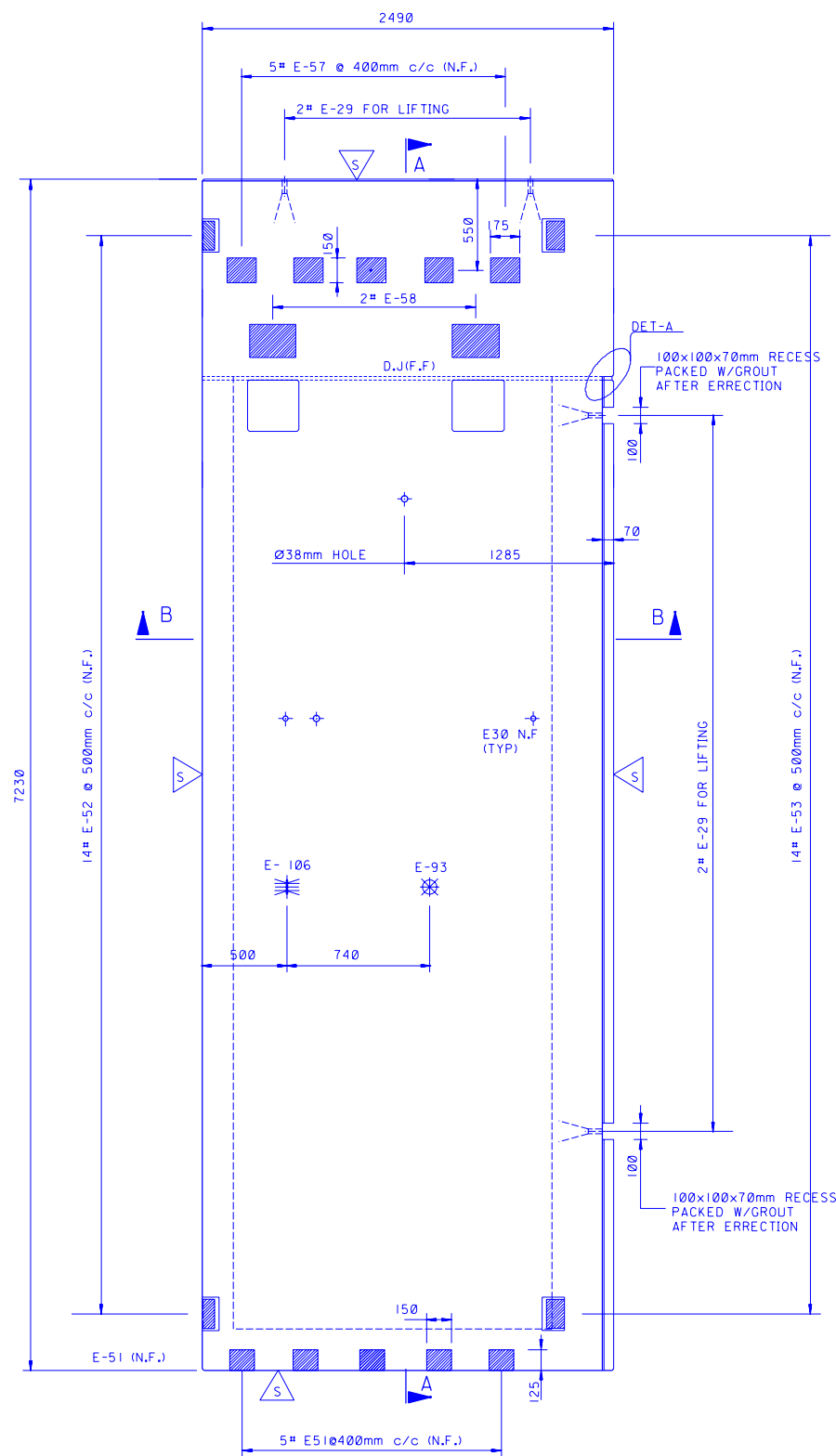
REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

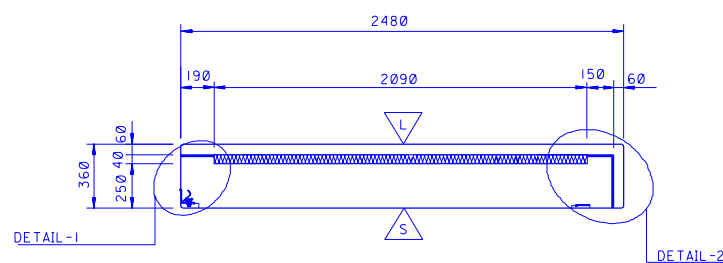
SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPROV.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY			CHECKED BY			
DATE STARTED			DATE COMPLETED			
DISCIPLINE ENGINEER			PROJECT ENGINEER			
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.						
SIGNATURE			NAME DATE			
TYP. PRECAST INSULATED PANEL-TYPE 9 & 9A (HANDED)						
STANDARD SUBSTATION						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950158	009	00		

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950158	010	00

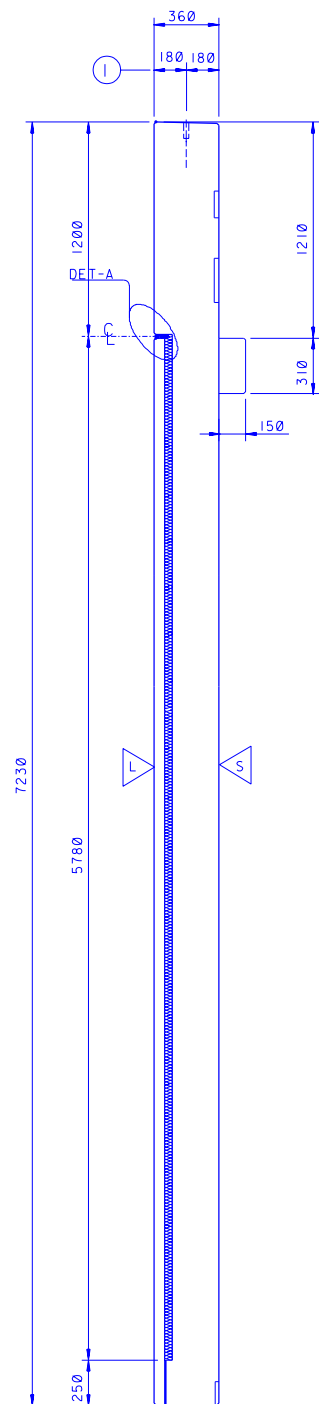


ELEVATION
N T S



SECTION-B

N T S



SECTION-A

N T S

WALL PANEL - TYPE 10 (PLAIN CORNER PANEL)
NOT TO SCALE

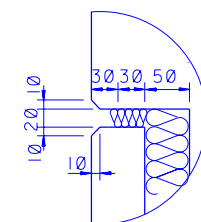
ITEM No. AS SHOWN

QTY.	No.	2				
VOL.	m3	6.01				
WEIGHT	to	15.03				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					

CONCRETE COVER 30mm MIN. EXT. / 20mm. MIN. INTERNAL

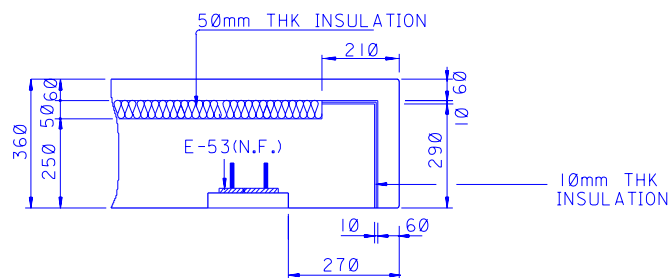
MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE

	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY
E29	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T	4	
E30	M16 FOR ERECTION SOCKET	2	
E-93	DEHA SLEEVE ANCHOR 5300-150-127	1	
E-106	DEHA FLAT ANCHOR 5301-150-160	1	
E51	150x250x12mm THK STEEL PLATE (GALVANIZED)	5	
E52	75x75x5mm THK (L=175) BENT PLATE (GALVANIZED)	14	
E57	150x175x6mm THK STEEL PLATE (GALVANIZED)	5	
E58	350x200x6mm THK STEEL PLATE (GALVANIZED)	2	
E53	180x180x10mm THK STEEL PLATE (GALVANIZED)	14	



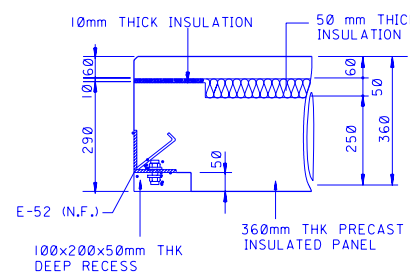
DETAIL - A

N T S



DETAIL-2

N T S



DETAIL - I
N T S

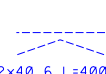
REF. DETAIL - G
ON DA-950159
SH 001 & 002



```
4 0 25, L=2200
FOR E29
```



```
2 0 10, L=500
FOR E30
```



1

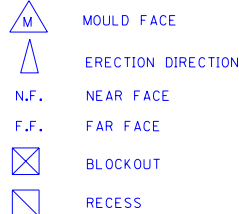
2x20 6, L=500
FOR E-93

1. ALL DIMENSIONS ARE IN mm.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
4. WHITE CEMENT & KHURAS AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND



ABBREVIATIONS



REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS		DA-950155.001
FLOOR PLAN-STD, SUBSTN, 10M WIDE		DA-950155.001
FLOOR PLAN-STD, SUBSTN, 15M WIDE		DA-950155.002
ELEVATION-STD, SUBSTN, 10M WIDE		DA-950154.001
ELEVATION-STD, SUBSTN, 15M WIDE		DA-950154.002
SECTION-STD, SUBSTN, 10M WIDE		DA-950155.001
SECTION-STD, SUBSTN, 15M WIDE		DA-950155.002
TYP, CONNECTION DETAILS		DA-950159.001
TYP, CONNECTION DETAILS		DA-950159.002
TYP, CONNECTION DETAILS		DA-950159.003

SCALE: NTS

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.

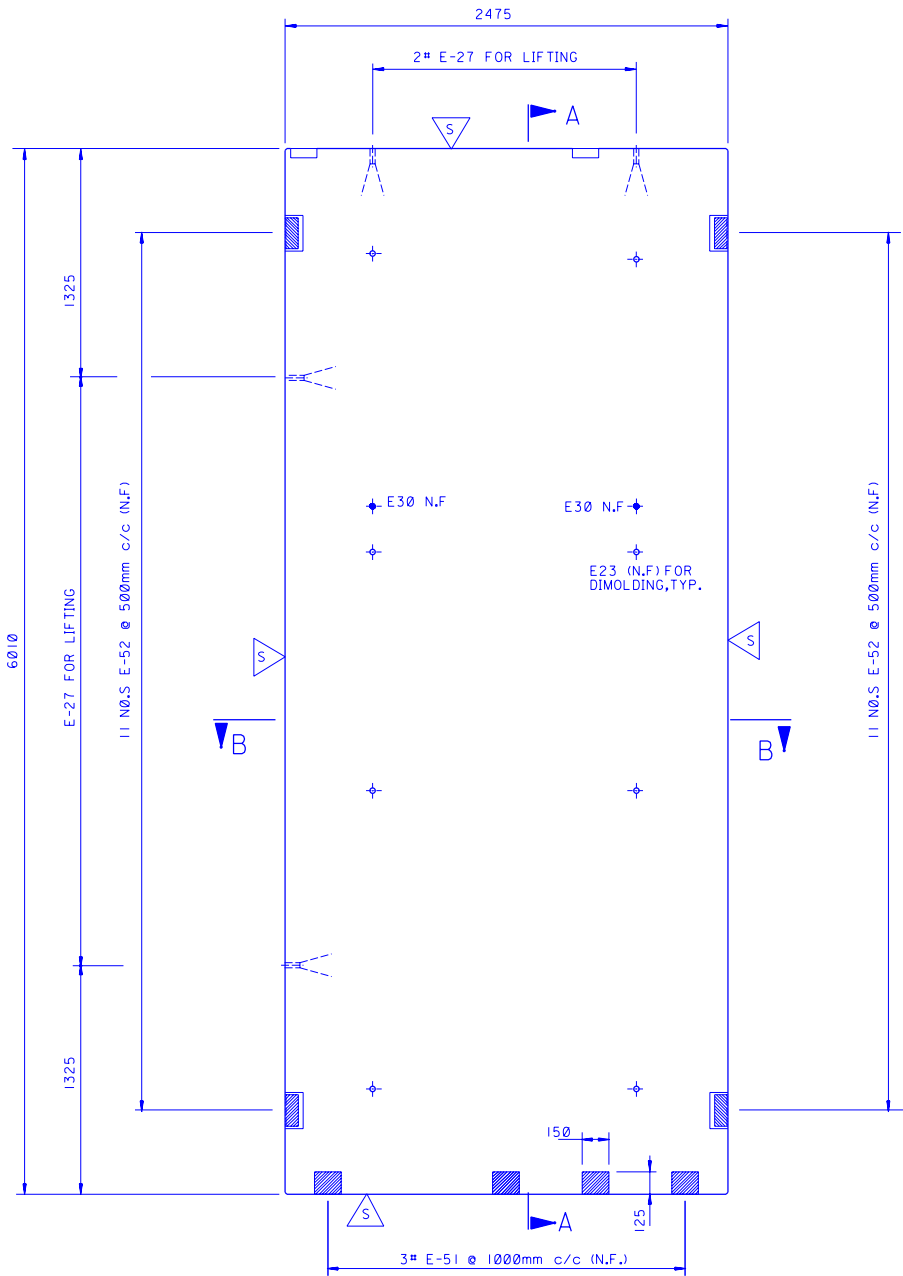
CERTIFIED BY: _____ (NAME)
SIGNATURE _____ DATE _____

TYP.PRECAST INSULATED PANEL-TYPE 10

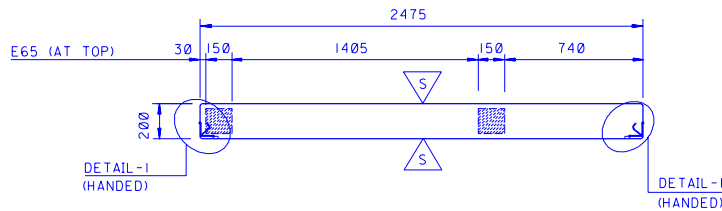
STANDARD SUBSTATION

LIBRARY DRAWING		SAUDI ARABIA		
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950158	010	00

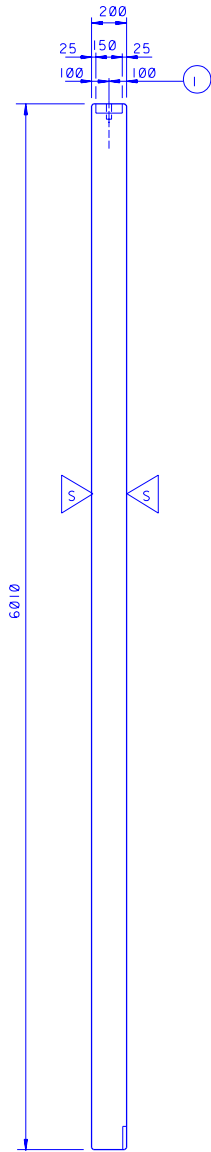
00	01	08	DA-950158	R	M88
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.	



ELEVATION
N T S



SECTION-B
N T S



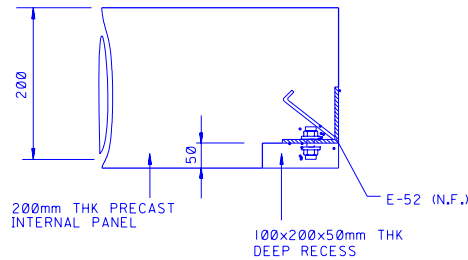
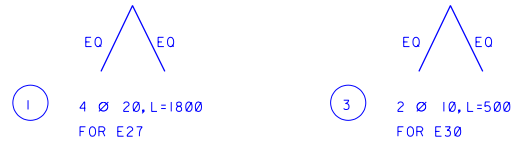
SECTION-A
N T S

ITEM No. AS SHOWN

QTY.	No.	I				
VOL.	m3	2.95				
WEIGHT	to	7.37				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					

CONCRETE COVER	30mm MIN. EXT. / 20mm, MIN. INTERNAL
MIX DESIGN	35Mpa N/mm2, WHITE & GREY CONCRETE

	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY
E27	PFEIFER LIFTING SOCKET (Rd 36) 6.3 T	4	
E23	PFEIFER DIMOLDING SOCKET (Rd 18) 1.6 T	8	
E30	M16 FOR ERECTION SOCKET	2	
E51	150x250x12mm THK STEEL PLATE (GALVANIZED)	3	
E52	75x75x5mm THK (L=175) BENT PLATE (GALVANIZED)	22	
E65	150x150x8mm THK STEEL PLATE (GALVANIZED)	2	



DETAIL-1
N T S

REF. DETAIL-F, I & H
TO SUIT, ON
DA-950159
SH 001 & 002

WALL PANEL-TYPE II (INTERNAL PLAIN PANEL)
NOT TO SCALE

NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
4. WHITE CEMENT & KHURAI'S AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- △ S SMOOTH TROWEL FINISH
- △ B HEAVY SANDBLASTED FINISH
- △ L LIGHT SANDBLASTED FINISH
- △ R BROOM FINISH
- ▽ TYP. CHAMFER 10x10

ABBREVIATIONS

- △ M MOULD FACE
- △ ERECTION DIRECTION
- N.F. NEAR FACE
- F.F. FAR FACE
- ⊗ BLOCKOUT
- RECESS

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.
CERTIFIED BY: _____ NAME: _____
SIGNATURE _____ DATE _____

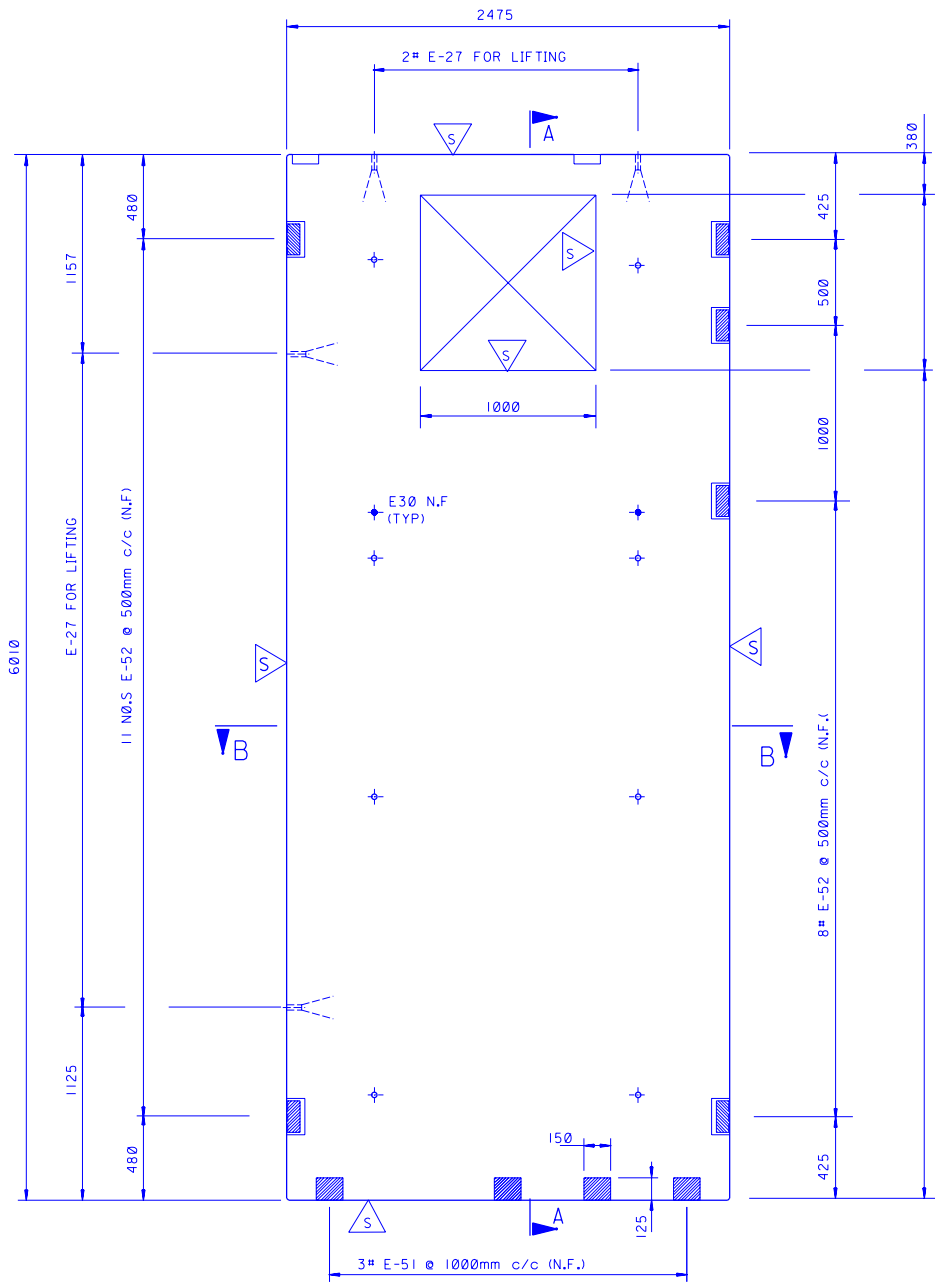
TYP. PRECAST INTERNAL PANEL - TYPE II DRAWING TYPE
ARC

STANDARD SUBSTATION

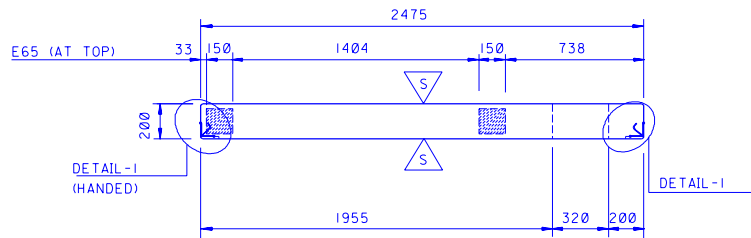
LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950158	011	00

00	012	058	DA-950158	R	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	INDEX



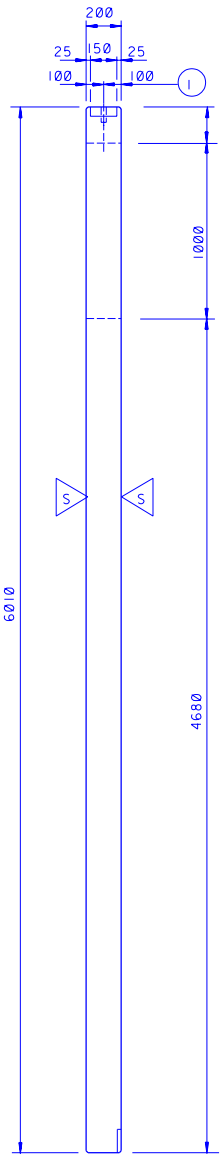
ELEVATION



SECTION-B

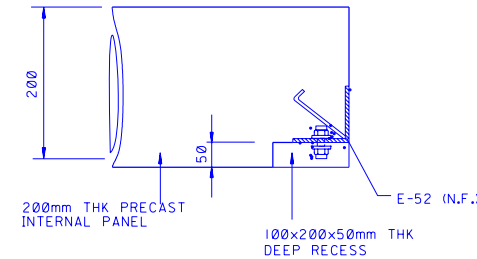
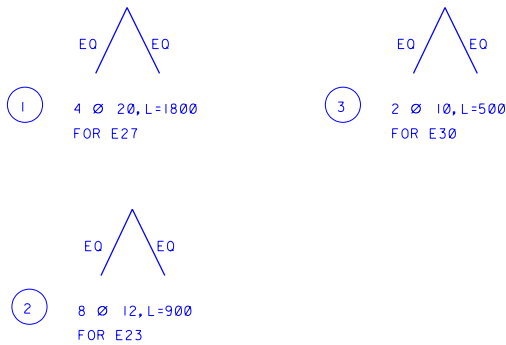
WALL PANEL - TYPE 12 (STD. PENETRATION INTERNAL)

NOT TO SCALE



SECTION-A
N T S

ITEM No.	AS SHOWN					
QTY.	No.	I				
VOL.	m3	2.92				
WEIGHT	to	7.30				
REBARS	kg					
WIRE MESH	kg					
REINFORCEMENT	kg					
CONCRETE COVER 30mm MIN. EXT. / 20mm. MIN. INTERNAL						
MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE						
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY			
E27	PFEIFER LIFTING SOCKET (Rd 36) 6.3 T	4				
E23	PFEIFER DIMOLDING SOCKET (Rd 18) 1.6 T	8				
E30	M16 FOR ERECTION SOCKET	2				
E51	150x250x12mm THK STEEL PLATE (GALVANIZED)	3				
E52	75x75x5mm THK (L=175) BENT PLATE (GALVANIZED)	21				
E65	150x150x8mm THK STEEL PLATE (GALVANIZED)	2				



DETAIL-1
N T S

REF. DETAIL-F, I & H
TO SUIT, ON
SK-009 SH 001

NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
4. WHITE CEMENT & KHURAI'S AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- SMOOTH TROWEL FINISH
- HEAVY SANDBLASTED FINISH
- LIGHT SANDBLASTED FINISH
- BROOM FINISH
- TYP. CHAMFER 10x10

ABBREVIATIONS

- MOULD FACE
- ERECTION DIRECTION
- N.F. NEAR FACE
- F.F. FAR FACE
- BLOCKOUT
- RECESS

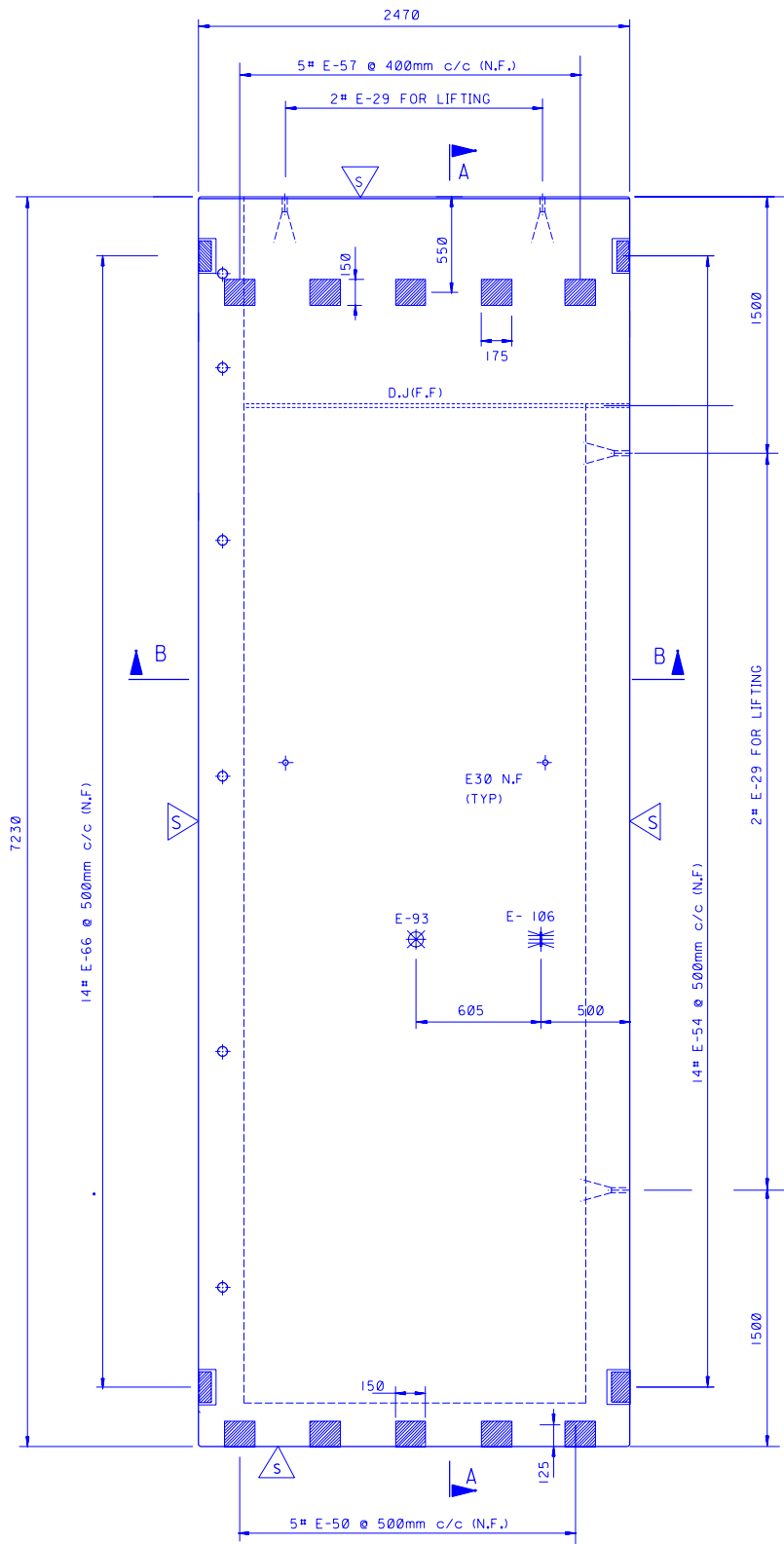
REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

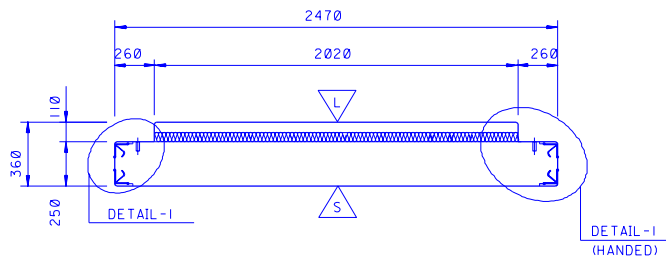
SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPV.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____			CHECKED BY _____			
DATE STARTED _____			DATE COMPLETED _____			
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.			CERTIFIED BY: _____		NAME: _____	
			SIGNATURE _____		DATE _____	
TYP.PRECAST INTERNAL PANEL-TYPE 12					DRAWING TYPE	
					ARC	
STANDARD SUBSTATION						
LIBRARY DRAWING			SAUDI ARABIA			
PLANT NO.	INDEX	DRAWING NUMBER		SHT. NO.	REV. NO.	
M88	R	DA-950158		012	00	

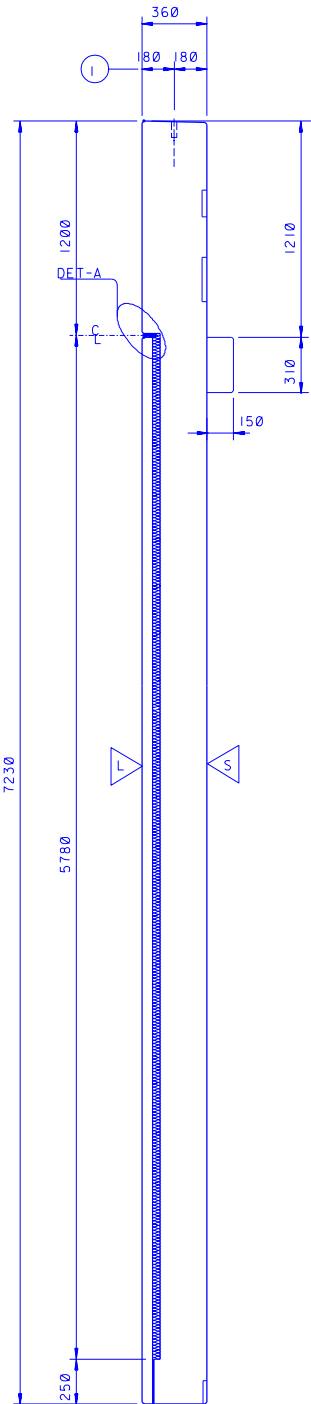
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PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.	



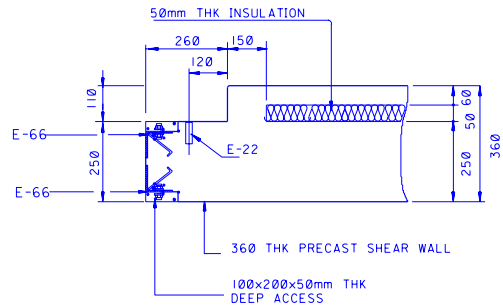
ELEVATION
N T S



SECTION-B
N T S

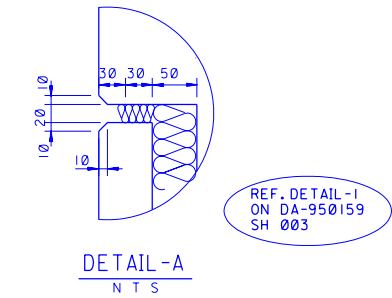


SECTION-A
N T S



DETAIL-I
N T S

REF. DETAIL-D
ON DA-950159
SH 001 & 002



DETAIL-A
N T S

ITEM No.	AS SHOWN				
QTY.	No.	I			
VOL.	m3	5.60			
WEIGHT	to	14.00			
REBARS	kg				
WIRE MESH	kg				
REINFORCEMENT	kg				
CONCRETE COVER 30mm MIN. EXT. / 20mm MIN. INTERNAL					
MIX DESIGN 35Mpa N/mm2, WHITE & GREY CONCRETE					
	DESCRIPTION	QTY FOR 1 UNIT	TOTAL QTY		
E29	PFEIFER LIFTING SOCKET (Rd 52) 12.0 T	4			
E30	M16 FOR ERECTION SOCKET	2			
E-50	150x250x12mm THK STEEL PLATE (GALVANIZED)	5			
E-66	75x75x8mm THK (L=180) BENT PLATE (GALVANIZED)	28			
E54	75x75x10mm THK (L=180) BENT PLATE (GALVANIZED)	14			
E56	250x175x8mm THK STEEL PLATE (GALVANIZED)	5			
E22	PFEIFER FIXING SOCKET (Rd 16) 1.2 T	5			
E93	DEHA SLEEVE ANCHOR 5300-150-127	1			
E106	DEHA FLAT ANCHOR 5301-150-160	1			

- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. FINISHING AS PER APPROVED SAMPLE
 3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING).
 4. WHITE CEMENT & KHURALS AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.

LEGEND

- SMOOTH TROWEL FINISH
HEAVY SANDBLASTED FINISH
LIGHT SANDBLASTED FINISH
BROOM FINISH
TYP. CHAMFER 10x10

ABBREVIATIONS

- MOULD FACE
ERECTION DIRECTION
N.F. NEAR FACE
F.F. FAR FACE
BLOCKOUT
RECESS

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

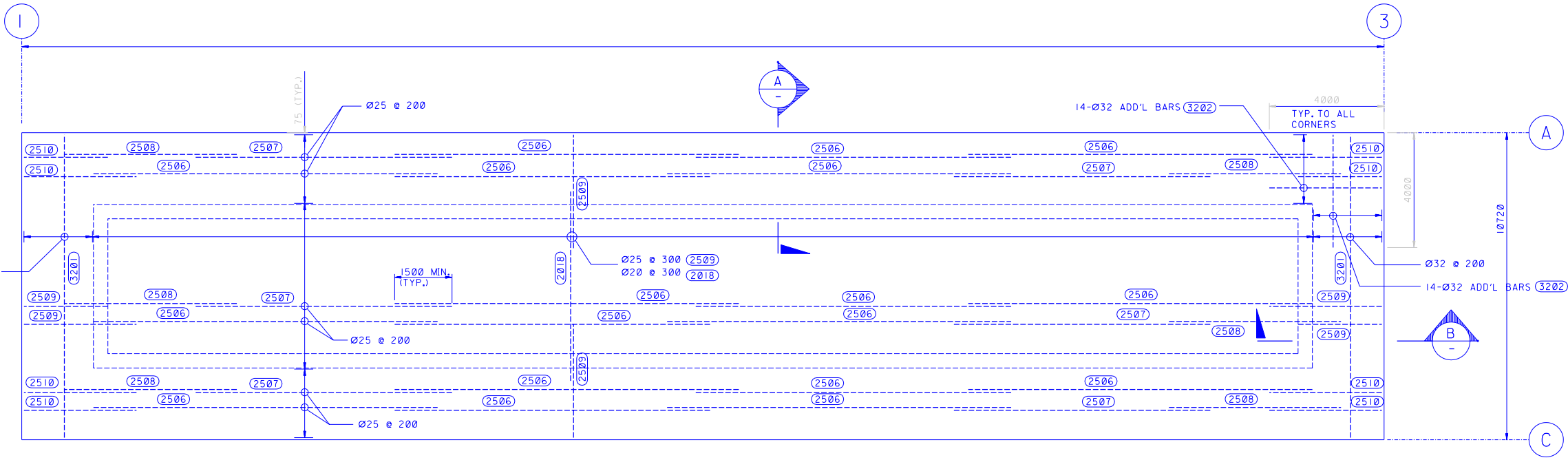
- 1 4 Ø 25, L=2200 FOR E29
- 2 2 Ø 10, L=500 FOR E30
- 3 6 Ø 10, L=800 FOR E22, FOR FIXING
- 4 2x40 6, L=400 FOR E-106
- 5 2x20 6, L=500 FOR E-93

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPRD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____ CHECKED BY _____						
DATE STARTED _____ DATE COMPLETED _____						
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.						
CERTIFIED BY: _____ NAME: _____ DATE: _____						
TYP. PRECAST INSULATED PANEL-TYPE 13						
STANDARD SUBSTATION						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950158	013	00		

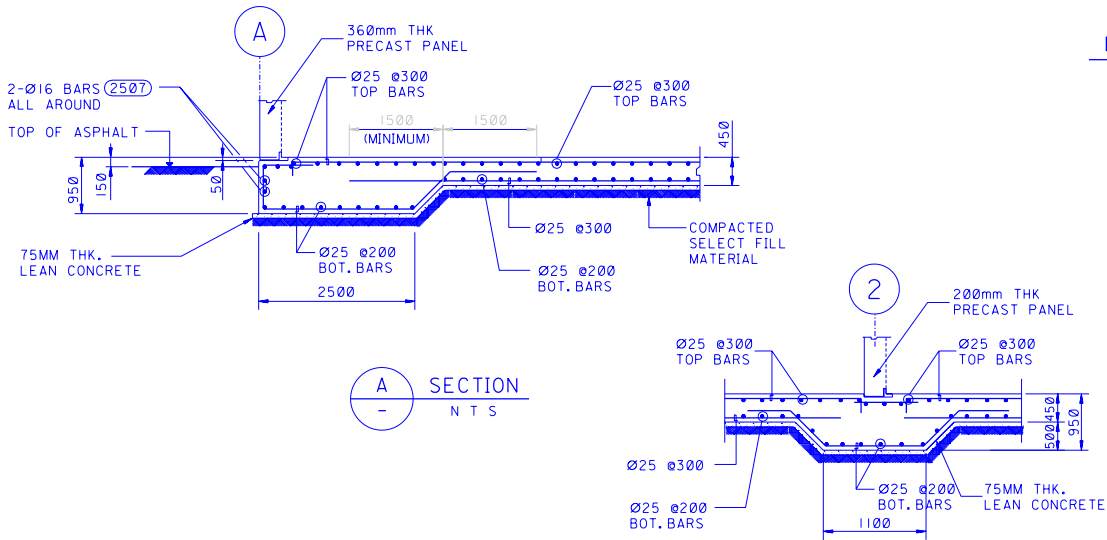
APPENDIX C.3

Reinforcement Drawings

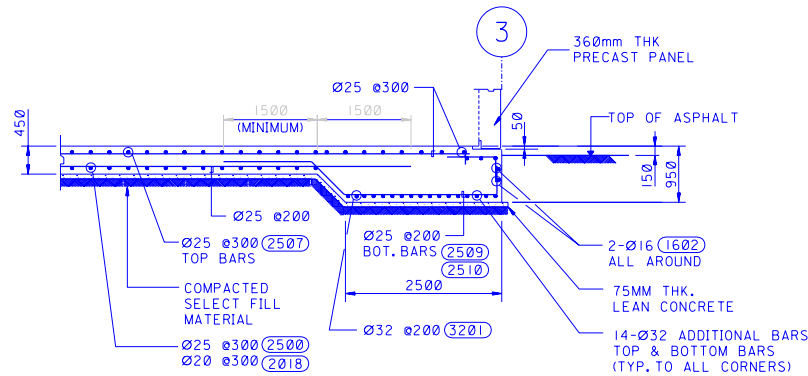
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PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.	



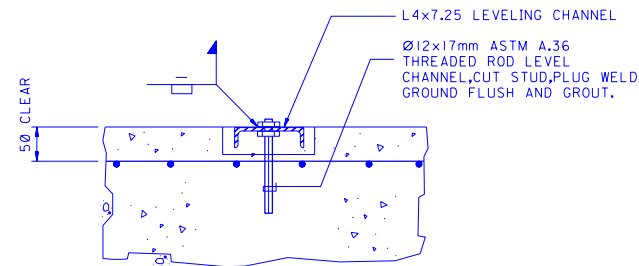
FOUNDATION PLAN (SHOWING BOTTOM REINFORCEMENT)
N T S



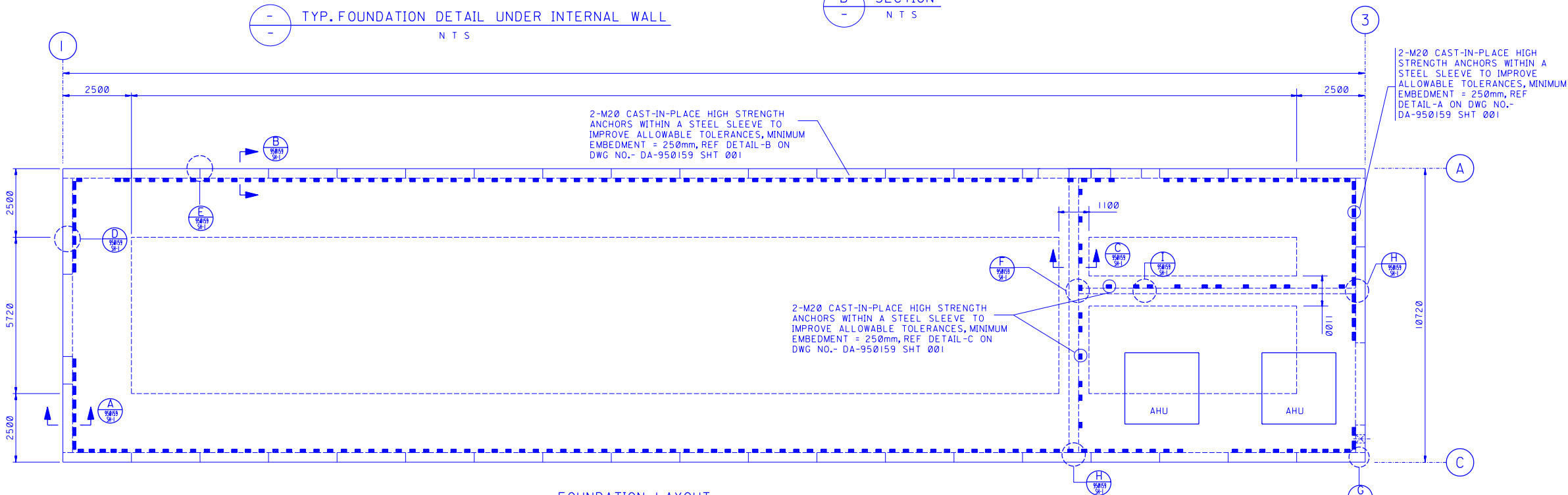
TYP. FOUNDATION DETAIL UNDER INTERNAL WALL
N T S



SECTION
N T S



OPTIONAL LEVELING CHANNEL DETAIL
N T S
(SEE INSTRUCTION NOTE 15, ON DA-950151 SH 001)



FOUNDATION LAYOUT
N T S

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. ALL REINFORCING STEEL BARS SHALL BE DEROCED HIGH GRADE WITH A MINIMUM $F_y=422$ Mpa AND SHALL BE EPOXY COATED PER 09-SAMSS-106.
3. ALL CAST IN SITU CONCRETE SHALL CONFORM TO SAUDI ARAMCO STANDARD SAES-0-001 AND SHALL A MINIMUM COMPRESSIVE STRENGTH OF 4000 PSI AT 28 DAYS. IT SHALL BE MIXED AND PLACED IN ACCORDANCE WITH 09-SAMSS-097 AND SHALL BE REPLACED TO CONFORM TO SAES-0-001.
4. UNSUITABLE SOIL AT THE BOTTOM OF THE EXCAVATION SHALL BE REMOVED TO AN ADEQUATE STRATUM, AS DIRECTED BY COMPANY ENGINEER AND REPLACED WITH SELECT FILL MATERIAL IN ACCORDANCE WITH SAES-A-114.
5. COMPACTION SHALL BE ACCORDING TO SAUDI ARAMCO ENGINEERING STANDARD SAES-A-114.
6. FOR WALL TO BASE CONNECTION DETAILS, REFER TO DWG NO. SK-006 SHT.001.
7. LEVELING SHIMS (20mm THK) SHALL BE PROVIDED BETWEEN WALL PANEL AND SLAB SURFACE.
8. SLAB LEVELNESS SHALL NOT EXCEED $\frac{1}{8}$ INCH (3mm) GAP AT ANY POINT BETWEEN 10FT STRAIGHTEDGE AND THE SLAB FLOOR AS PER ASTM E115-87 AND AS PER ACI 117 SECTION 4.5.7 "VERY FLAT" CRITERIA.
9. FOR MCC'S AND OTHER CABINET, FLOOR HARDENER SHALL BE PROVIDED AT THE BASE AT LEAST 72 HRS AFTER FOUNDATION SLAB PLACEMENT.
10. LAYOUT OF STEEL PLATE SHALL MATCH WITH VENDORS WALL PANELS EMBEDDED PLATE AT BASE.

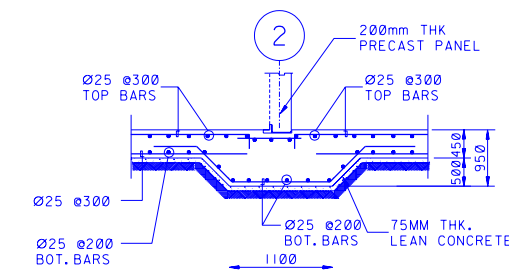
REFERENCE DRAWINGS

GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
FLOOR PLAN	DA-950153.001
TYPICAL CONNECTION DETAILS	DA-950159.001
TYPICAL CONNECTION DETAILS	DA-950159.003

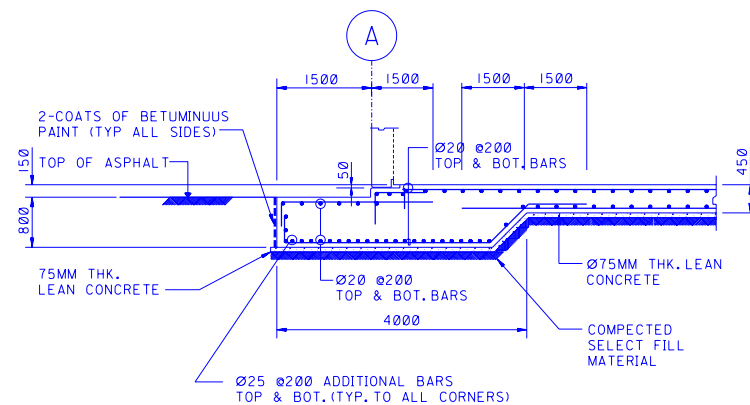
SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY			CHECKED BY			
DATE STARTED			DATE COMPLETED			
DISCIPLINE ENGINEER						
PROJECT ENGINEER						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.						
CERTIFIED BY:			NAME:			
SIGNATURE			DATE			
FOUNDATION PLAN/SECTION						DRAWING TYPE
						FDN
STANDARD SUBSTATION - 10M WIDE						
LIBRARY DRAWING			SAUDI ARABIA			
PLANT NO.	INDEX	DRAWING NUMBER		SHT. NO.	REV. NO.	
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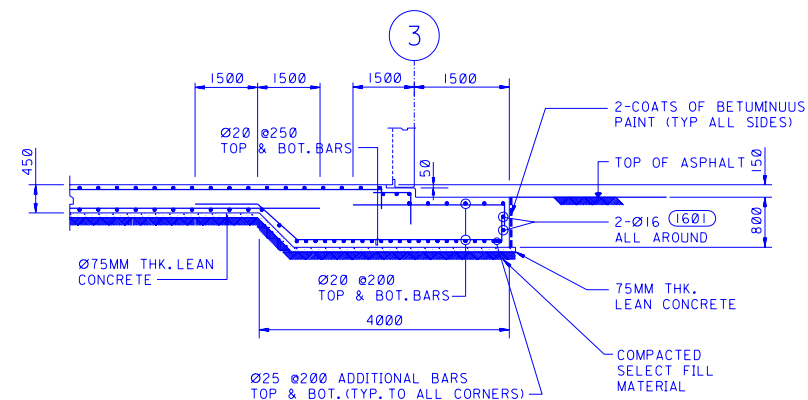
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	0	DA-950162	001	00



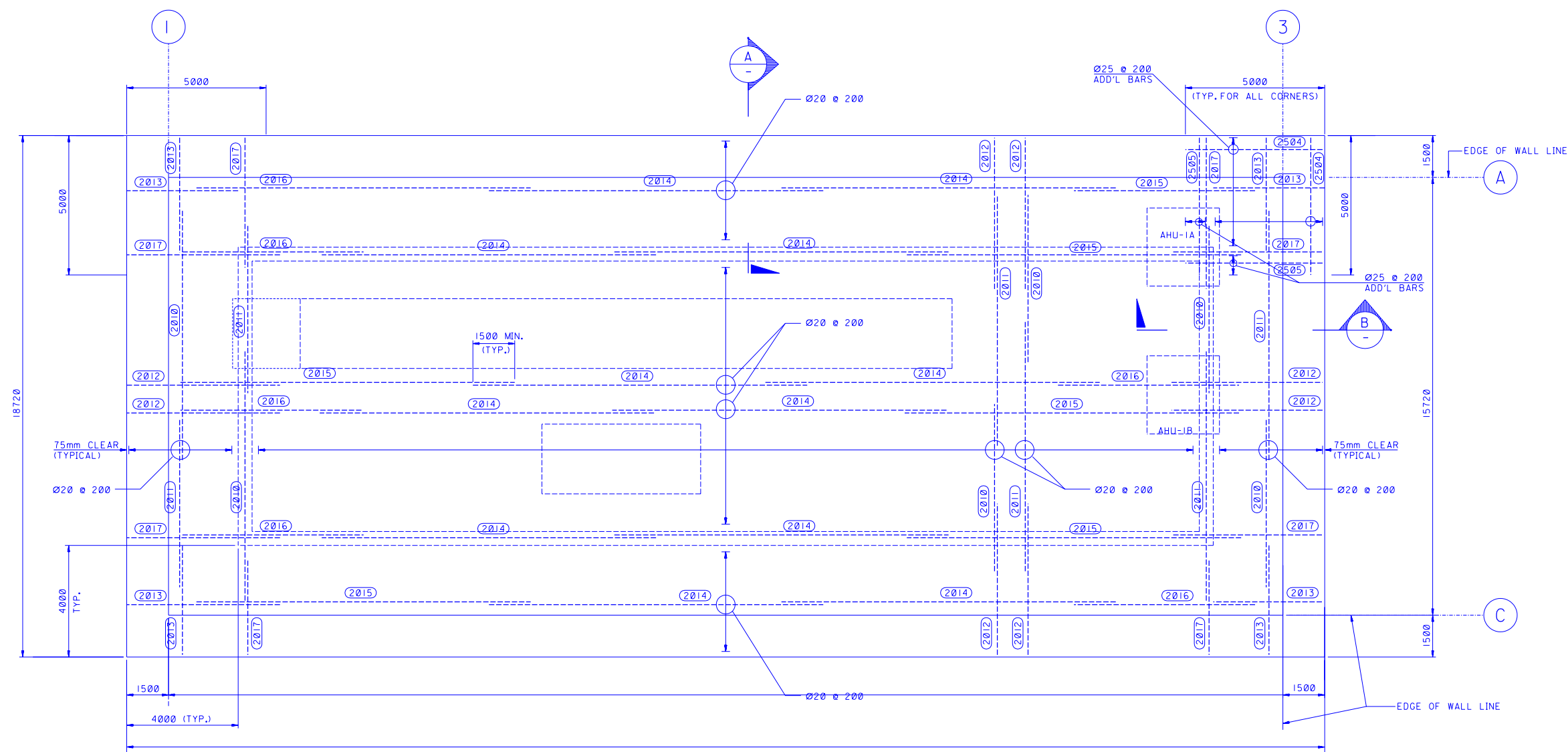
 TYP. FOUNDATION DETAIL UNDER INTERNAL WALL
 N T S



A SECTION
- N T S



B SECTION
- N T S



FOUNDATION PLAN (SHOWING BOTTOM REINFORCEMENT)

- NOTES:**
1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE INDICATED.
 2. ALL REINFORCING STEEL BARS SHALL BE DEFORMED HIGH GRADE WITH A MINIMUM $F_y=422$ Mpa AND SHALL BE EPOXY COATED PER 09-SAMSS-106.
 3. ALL CAST IN SITU CONCRETE SHALL CONFORM TO SAUDI ARAMCO STANDARD SAES-0-001 AND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 4000 PSI AT 28 DAYS.
IT SHALL BE MIXED AND PLACED IN ACCORDANCE WITH 09-SAMSS-097 AND SHALL BE REPLACED TO CONFORM TO SAES-0-001.
 4. UNSUITABLE SOIL AT THE BOTTOM OF THE EXCAVATION SHALL BE REMOVED TO AN ADEQUATE STRATUM, AS DIRECTED BY COMPANY ENGINEER AND REPLACED WITH SELECT FILL MATERIAL IN ACCORDANCE WITH SAES-A-114.
 5. COMPACTION SHALL BE ACCORDING TO SAUDI ARAMCO ENGINEERING STANDARD SAES-A-114.
 6. FOR WALL TO BASE CONNECTION DETLS., REFER TO DWG NO. SK-006 SHT.001.
 7. LEVELING SHIMS (20mm THK) SHALL BE PROVIDED BETWEEN WALL PANEL AND SLAB SURFACE.
 8. SLAB LEVELNESS SHALL NOT EXCEED $\frac{1}{8}$ INCH (3mm) GAP AT ANY POINT BETWEEN 10FT STRAIGHTEDGE AND THE SLAB FLOOR AS PER ASTM E115-87 AND AS PER ACI 117 SECTION 4.5,7 "VERY FLAT" CRITERIA.

REFERENCE DRAWINGS	
GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
FLOOR PLAN	DA-950153.002
TYPICAL CONNECTION DETAILS	DA-950159.002
TYPICAL CONNECTION DETAILS	DA-950159.003

SCALE: NTS

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
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SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.

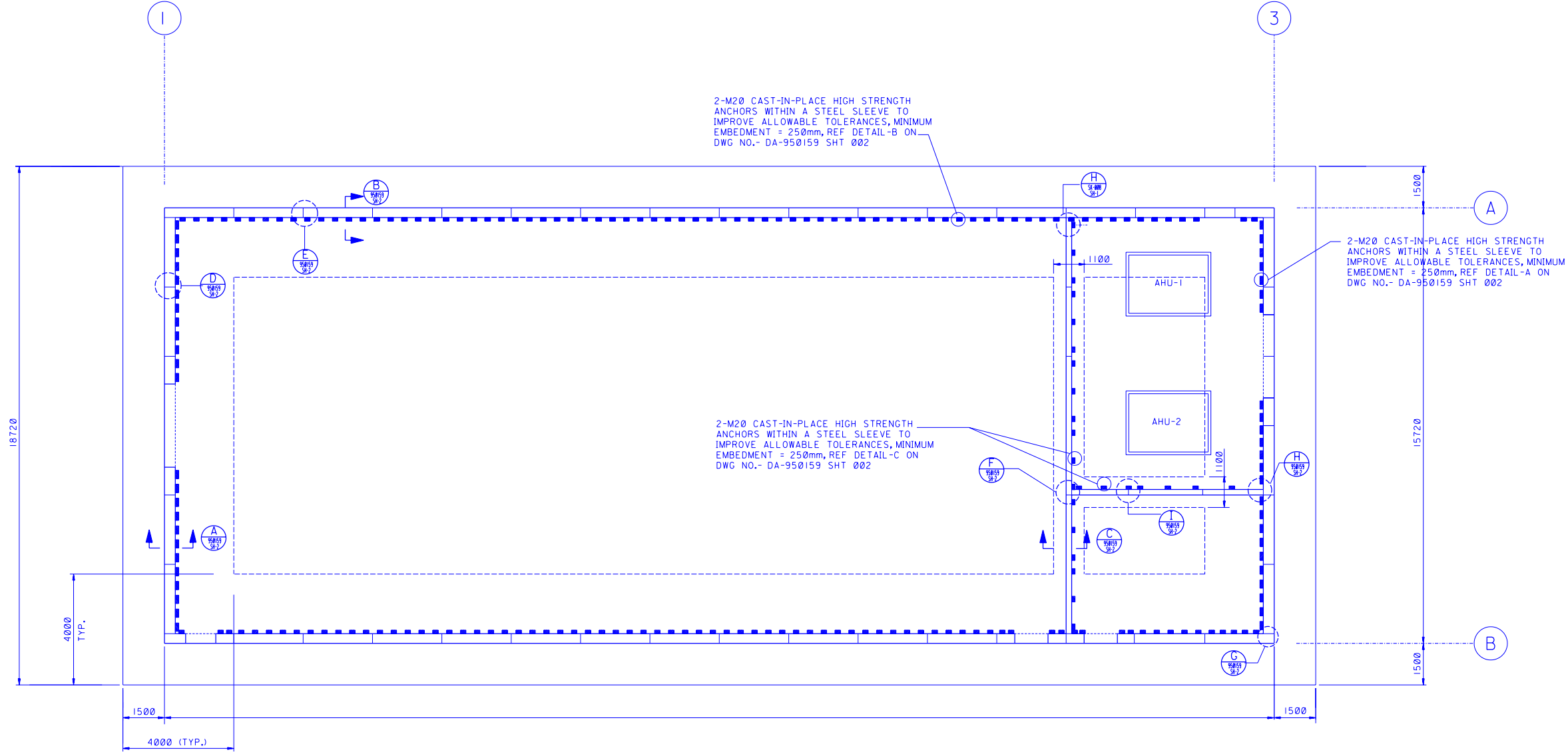
CERTIFIED BY: _____ (NAME)
SIGNATURE _____ DATE _____

FOUNDATION PLAN/SECTION DRAWING TYPE
FDN

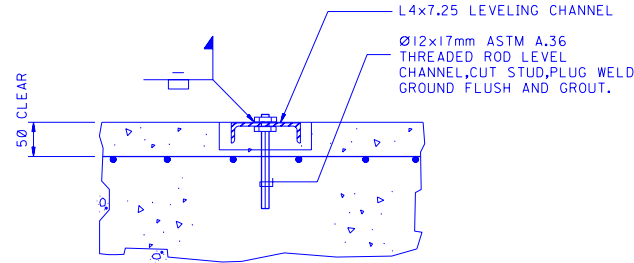
STANDARD SUBSTATION - 15M WIDE

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	Q	DA-950162	001	00

00	002	02	DA-950162	0	M88
PLANT NO.	INDEX	SHEET NO.	DRAWING NUMBER	REV. NO.	REV. NO.



FOUNDATION PLAN (SHOWING BOTTOM REINFORCEMENT)
N T S



OPTIONAL LEVELING CHANNEL DETAIL
N T S
(SEE INSTRUCTION NOTE 15, ON DA-950151 SH 001)

NOTES:

- ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE INDICATED.
- ALL REINFORCING STEEL BARS SHALL BE DEFORMED HIGH GRADE WITH A MINIMUM $F_y=422$ Mpa AND SHALL BE EPOXY COATED PER 09-SAMSS-106.
- ALL CAST IN SITU CONCRETE SHALL CONFORM TO SAUDI ARAMCO STANDARD SAES-0-001 AND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 4000 PSI AT 28 DAYS. IT SHALL BE MIXED AND PLACED IN ACCORDANCE WITH 09-SAMSS-097 AND SHALL BE REPLACED TO CONFORM TO SAES-0-001.
- UNSUITABLE SOIL AT THE BOTTOM OF THE EXCAVATION SHALL BE REMOVED TO AN ADEQUATE STRATUM, AS DIRECTED BY COMPANY ENGINEER AND REPLACED WITH SELECT FILL MATERIAL IN ACCORDANCE WITH SAES-A-114.
- COMPACTION SHALL BE ACCORDING TO SAUDI ARAMCO ENGINEERING STANDARD SAES-A-114.
- FOR WALL TO BASE CONNECTION DETILS, REFER TO DWG NO. SK-008 SHT.001.
- LEVELING SHIMS (20mm THK) SHALL BE PROVIDED BETWEEN WALL PANEL AND SLAB SURFACE.
- SLAB LEVELNESS SHALL NOT EXCEED $\frac{1}{8}$ INCH (3mm) GAP AT ANY POINT BETWEEN 10FT STRAIGHTEDGE AND THE SLAB FLOOR AS PER ASTM E115-87 AND AS PER ACI 117 SECTION 4.5.7 "VERY FLAT" CRITERIA.

REFERENCE DRAWINGS

GENERAL NOTES AND INSTRUCTIONS	DA-950151.001
FLOOR PLAN	DA-950153.002
TYPICAL CONNECTION DETAILS	DA-950159.002
TYPICAL CONNECTION DETAILS	DA-950159.003

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
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SAUDI ARABIAN OIL COMPANY

DRAWN BY	CHECKED BY
DATE STARTED	DATE COMPLETED
DISCIPLINE ENGINEER	
PROJECT ENGINEER	

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: NAME: DATE: SIGNATURE: DATE:
--	--

FOUNDATION LAYOUT

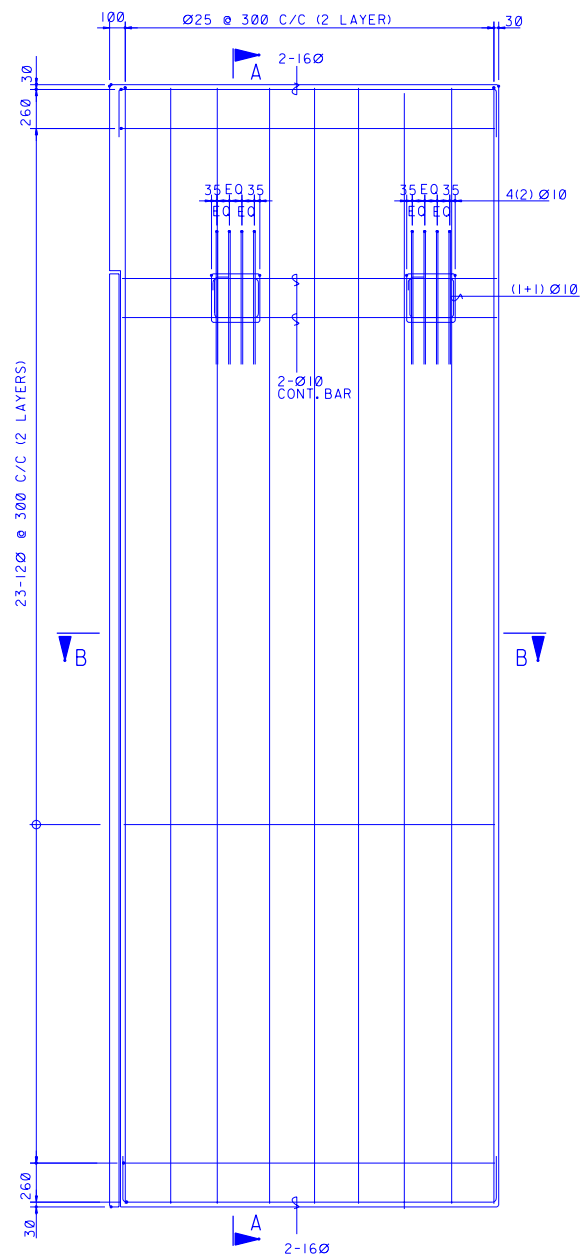
DRAWING TYPE

STANDARD SUBSTATION - 15M WIDE

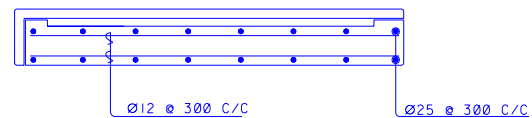
LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	0	DA-950162	002	00

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	0	DA-950163	001	00



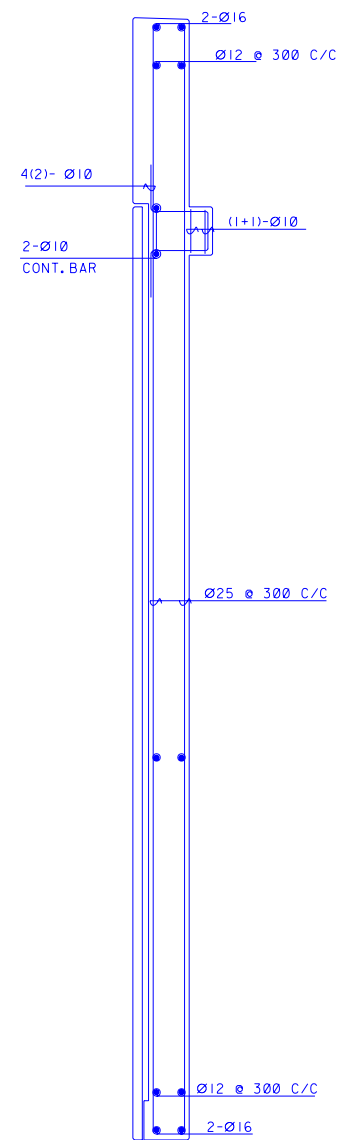
ELEVATION
N T S



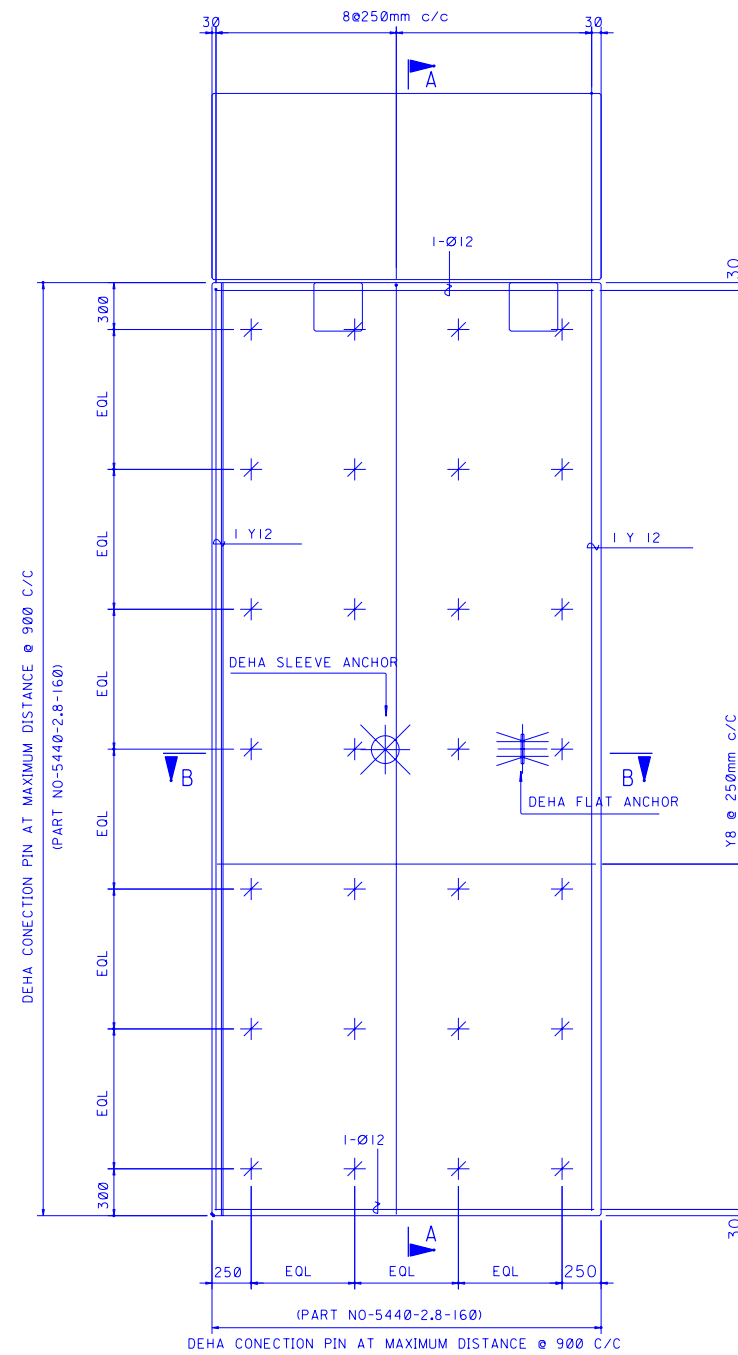
SECTION-B

N T S

INNERSKIN REINFORCEMENT (PANEL TYPE-I)



SECTION-A
N T S



ELEVATION
N T S



SECTION-B

N T S

OUTERSKIN REINFORCEMENT (PANEL TYPE-I)

1. ALL DIMENSIONS ARE IN mm.
2. TYPICAL REINFORCEMENT DETAIL.
3. REINFORCEMENT, YIELD STRENGTH OF REBAR
fy=420 MPa, ASTM 615, GRADE 60 BLACK STEEL
(EPOXY COATED).
4. 30mm MIN. EXTERNAL / 20mm MIN. INTERNAL.

REFERENCE DRAWINGS	
GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD SUBSTN 10M WIDE	DA-950154.001
ELEVATION-STD SUBSTN 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN 15M WIDE	DA-950155.002
TYP. PRECAST PANEL DETAILS	DA-950158.001-013
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: NTS

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: _____ (NAME) SIGNATURE _____ DATE _____
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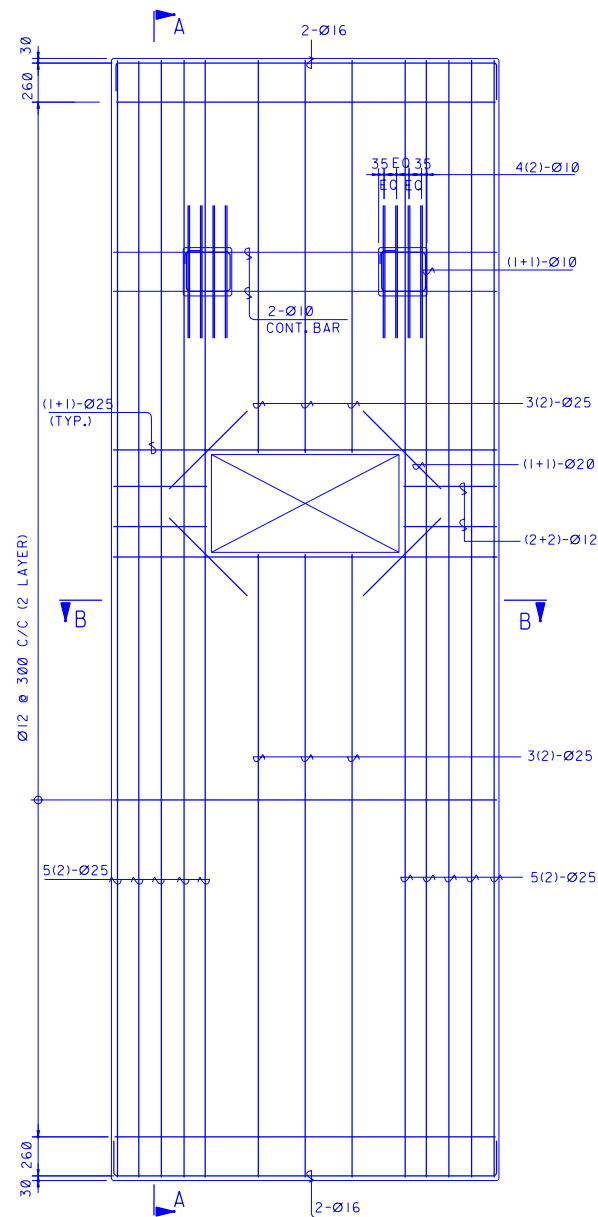
TYP.PRECAST STRUCTURAL DETAIL	DRAWING TYPE SDT
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STANDARD SUBSTATION

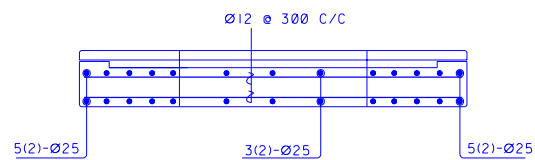
LIBRARY DRAWING SAUDI ARABIA

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	Q	DA-950163	001	00

M88	0	DA-950163	002	00
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.

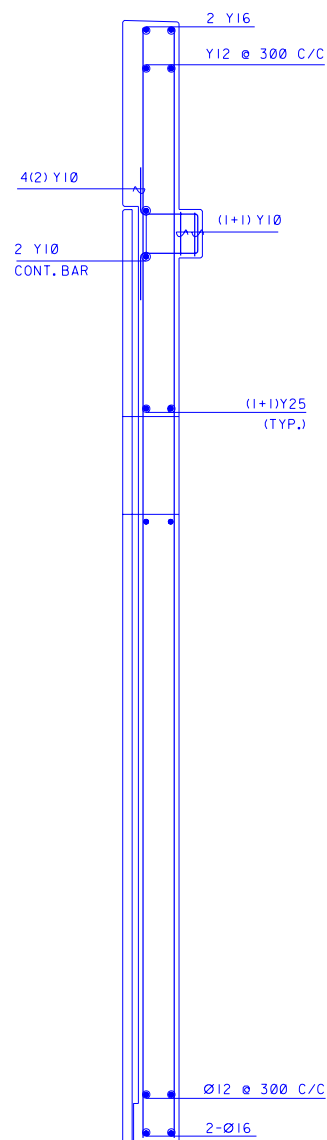


ELEVATION
N T S

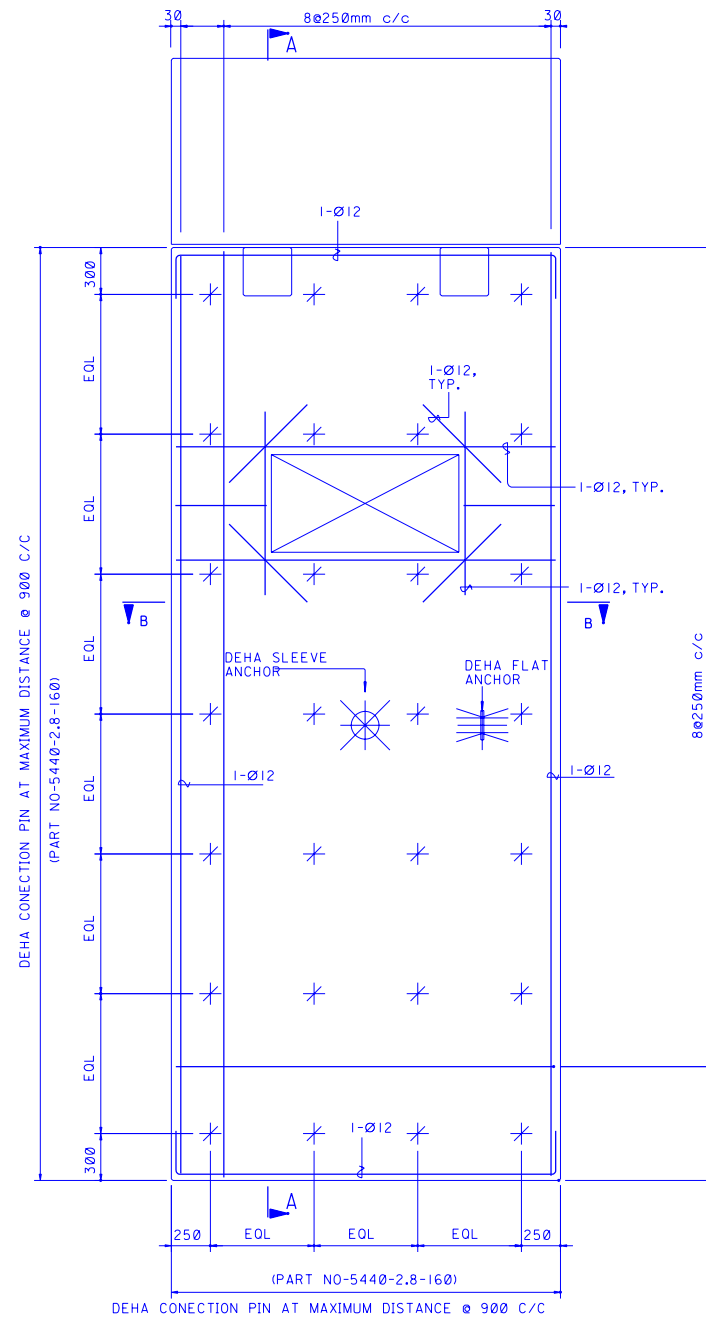


SECTION-B
N T S

INNERSKIN REINFORCEMENT (PANEL TYPE-2)



SECTION-A
N T S

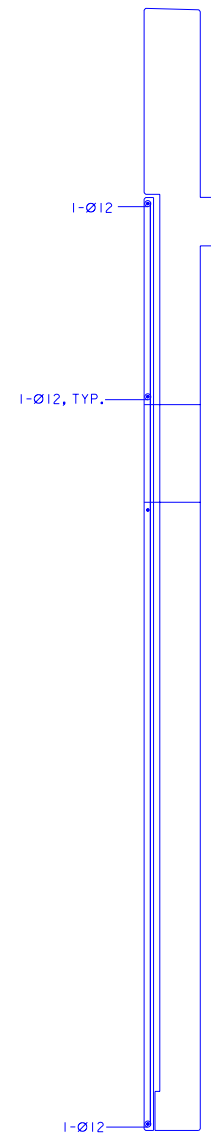


ELEVATION
N T S



SECTION-B
N T S

OUTERSKIN REINFORCEMENT (PANEL TYPE-2)



SECTION-A
N T S

1. ALL DIMENSIONS ARE IN mm.
2. TYPICAL REINFORCEMENT DETAIL.
3. REINFORCEMENT, YIELD STRENGTH OF REBAR
fy=420 Mpa, ASTM 615, GRADE 60 BLACK STEEL
(EPOXY COATED).
4. 30mm MIN. EXTERNAL / 20mm MIN. INTERNAL.

REFERENCE DRAWINGS	
GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD SUBSTN 10M WIDE	DA-950154.001
ELEVATION-STD SUBSTN 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN 15M WIDE	DA-950155.002
TYP. PRECAST PANEL DETAILS	DA-950158.001-013
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

SAUDI ARABIAN OIL COMPANY

DRAWN BY _____ CHECKED BY _____
DATE STARTED _____ DATE COMPLETED _____
DISCIPLINE ENGINEER _____
PROJECT ENGINEER _____

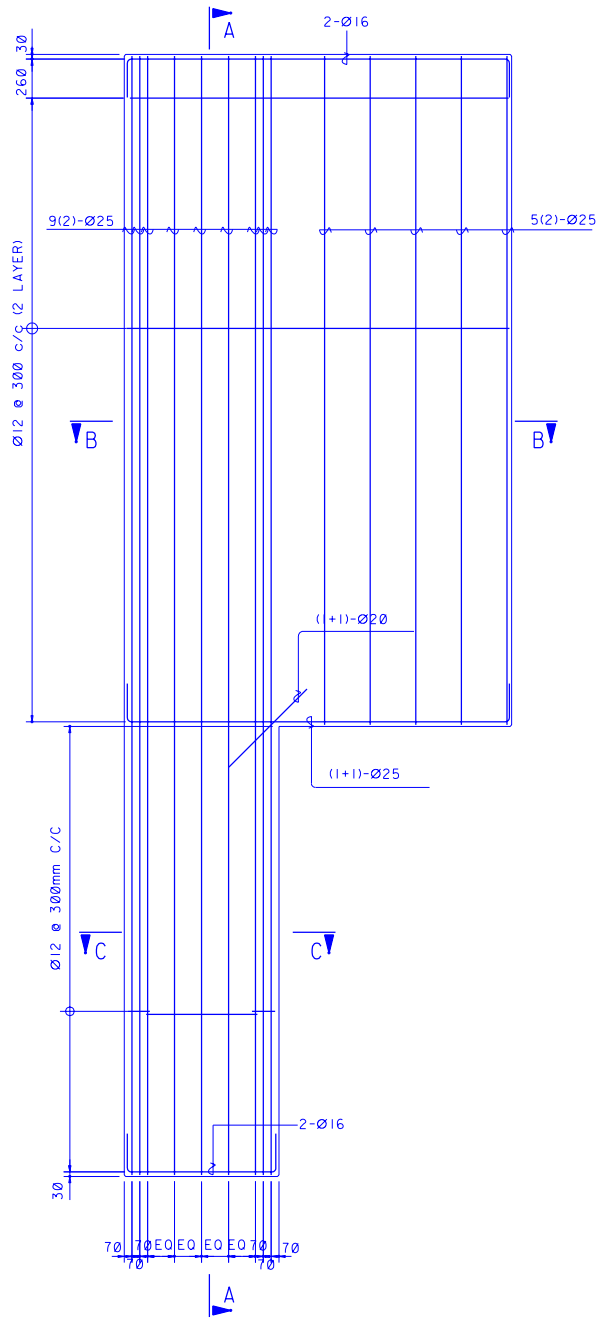
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: _____
	_____ (NAME) SIGNATURE _____ DATE _____

TYP.PRECAST STRUCTURAL DETAIL DRAWING TYPE
SDT

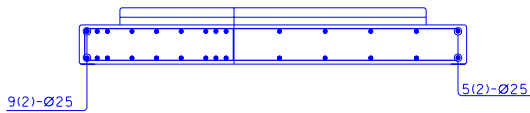
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	Q	DA-950163	002	00

00	003	9163	DA-950163	0	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	

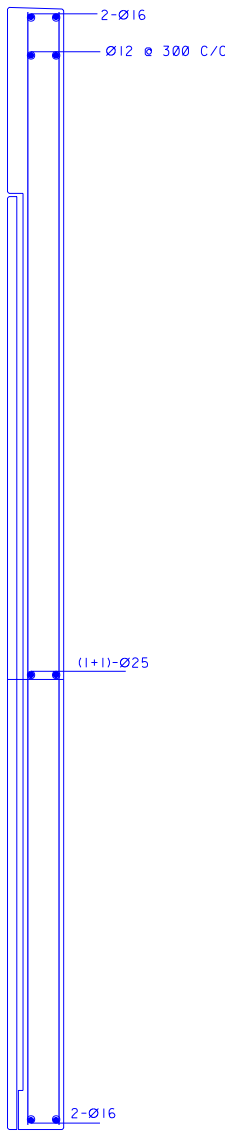


ELEVATION
N T S

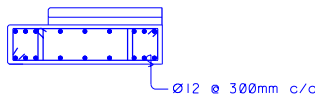


SECTION-B
N T S

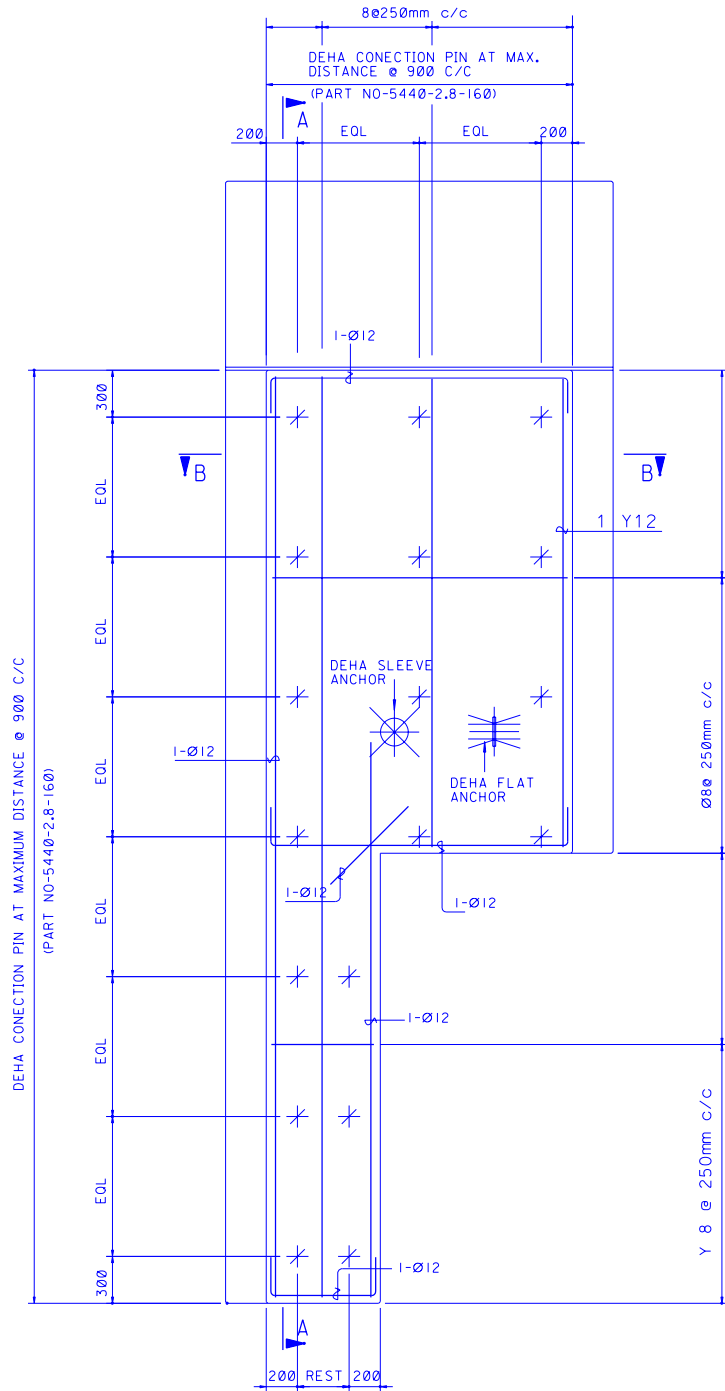
INNERSKIN REINFORCEMENT (PANEL TYPE-7)



SECTION-A
N T S



SECTION-C
N T S

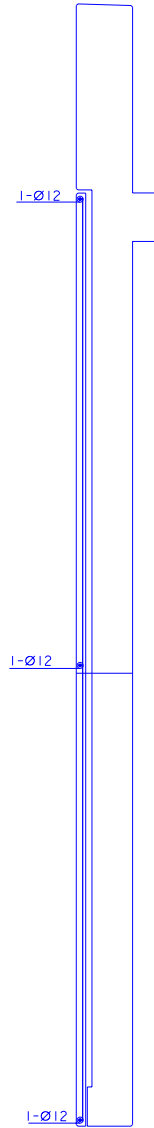


ELEVATION
N T S



SECTION-B
N T S

OUTERSKIN REINFORCEMENT (PANEL TYPE-7)



SECTION-A
N T S

NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. TYPICAL REINFORCEMENT DETAIL.
3. REINFORCEMENT, YIELD STRENGTH OF REBAR $f_y=420$ MPa, ASTM 615, GRADE 60 BLACK STEEL (EPOXY COATED).
4. 30mm MIN. EXTERNAL / 20mm MIN. INTERNAL.

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. PRECAST PANEL DETAILS	DA-950158.001-013
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY	CHECKED BY
DATE STARTED	DATE COMPLETED
DISCIPLINE ENGINEER	
PROJECT ENGINEER	

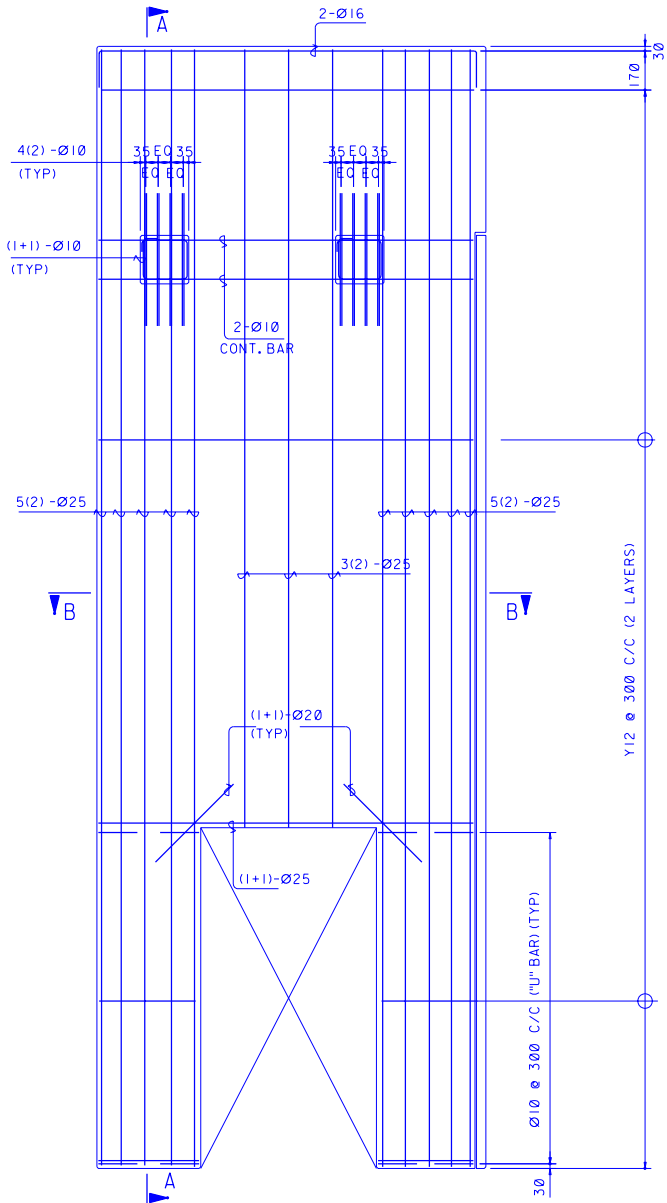
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: NAME: DATE: SIGNATURE: DATE:
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TYP. PRECAST STRUCTURAL DETAIL	DRAWING TYPE SDT
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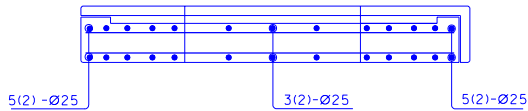
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	Q	DA-950163	003	00

00	004	8510158	DA-950156	0	M88
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.	

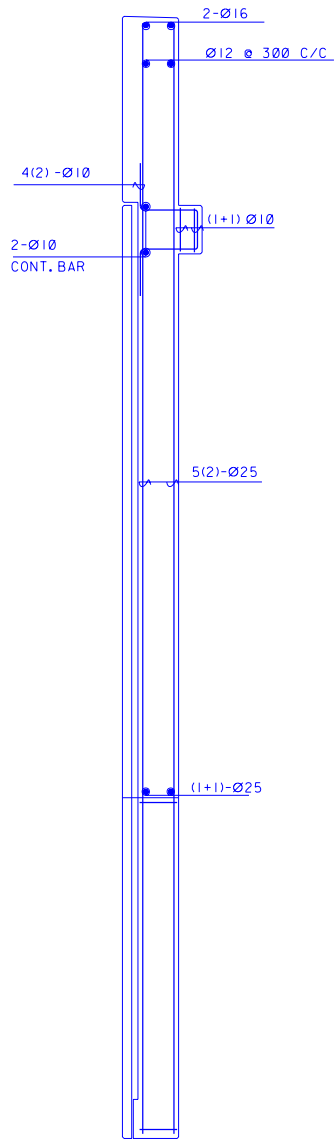


ELEVATION
N T S

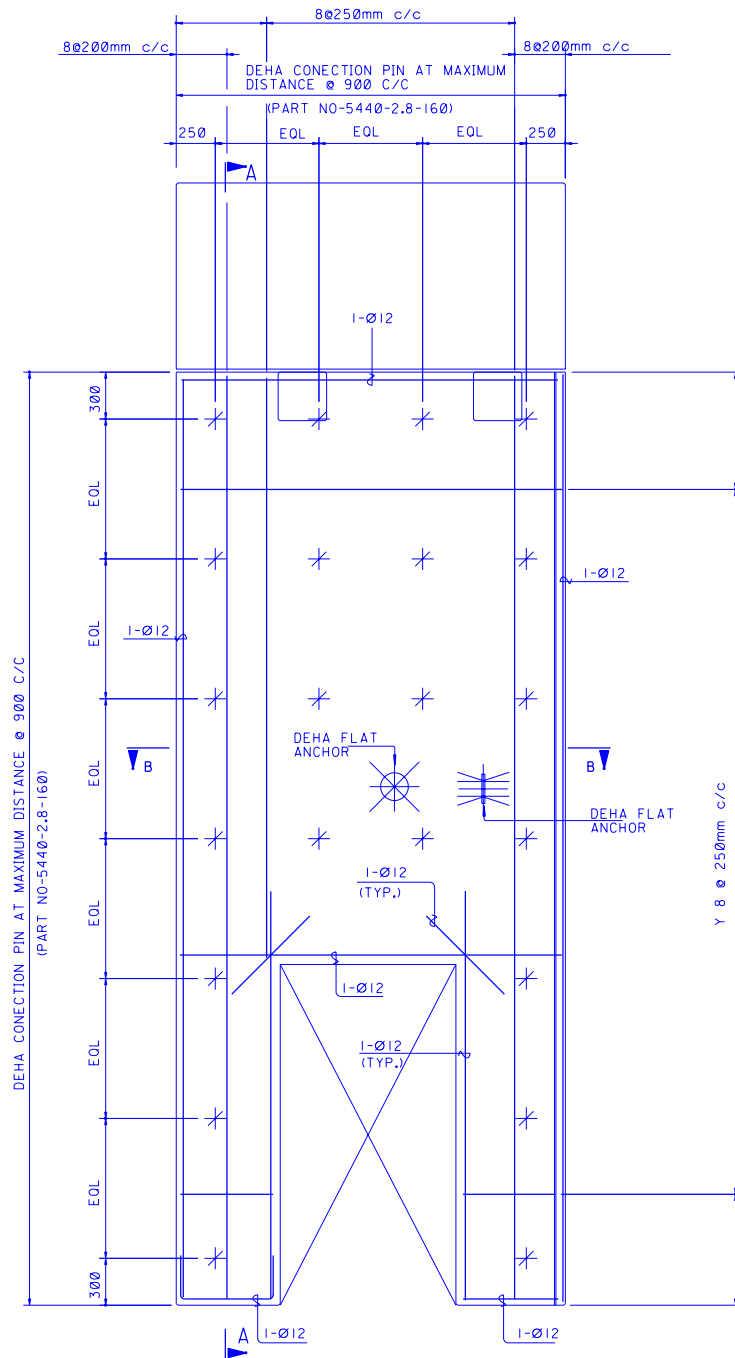


SECTION-B
N T S

INNERSKIN REINFORCEMENT (PANEL TYPE-8)



SECTION-A
N T S

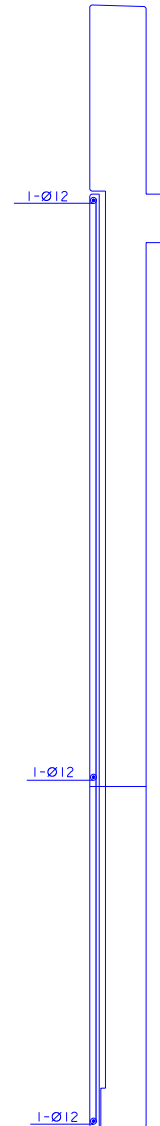


ELEVATION
N T S



SECTION-B
N T S

OUTERSKIN REINFORCEMENT (PANEL TYPE-8)



SECTION-A
N T S

- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. TYPICAL REINFORCEMENT DETAIL.
 3. REINFORCEMENT, YIELD STRENGTH OF REBAR $f_y=420$ MPa, ASTM 615, GRADE 60 BLACK STEEL (EPOXY COATED).
 4. 30mm MIN. EXTERNAL / 20mm MIN. INTERNAL.

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. PRECAST PANEL DETAILS	DA-950158.001-013
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: 1 N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY	CHECKED BY
DATE STARTED	DATE COMPLETED
DISCIPLINE ENGINEER	
PROJECT ENGINEER	

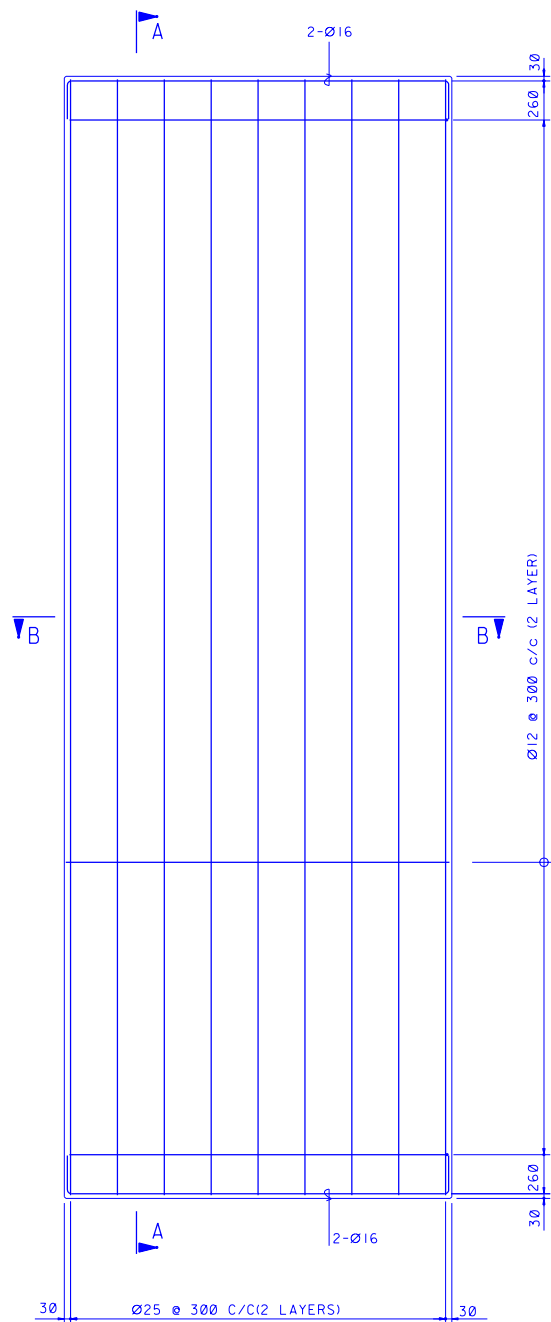
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: _____	SIGNATURE	NAME	DATE
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TYP. PRECAST STRUCTURAL DETAIL	DRAWING TYPE
	SDT

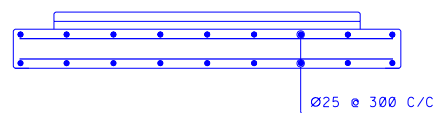
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	Q	DA-950163	004	00

PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	0	DA-950163	005	00



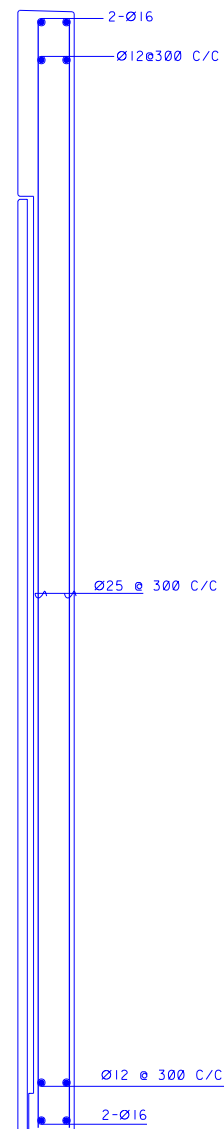
ELEVATION
N T S



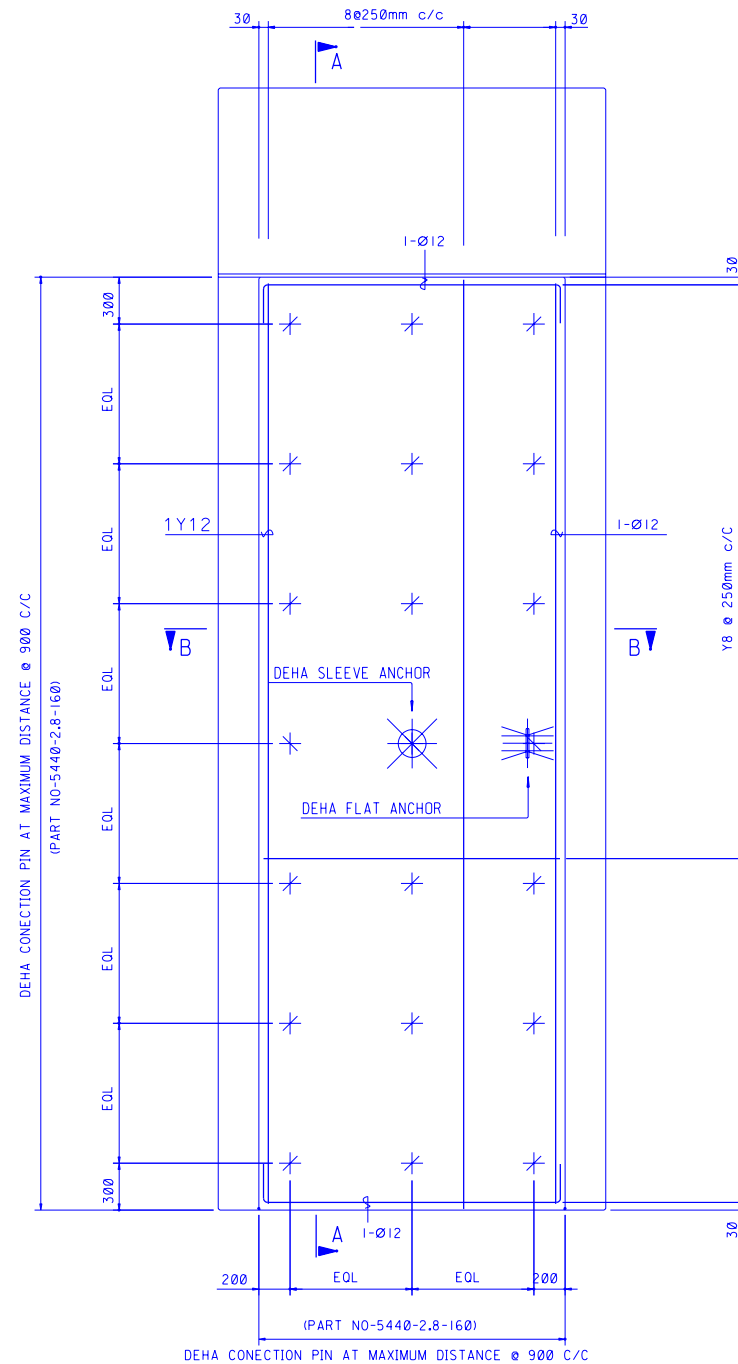
SECTION-B

N T S

INNERSKIN REINFORCEMENT (PANEL TYPE-9)



SECTION-A
N T S



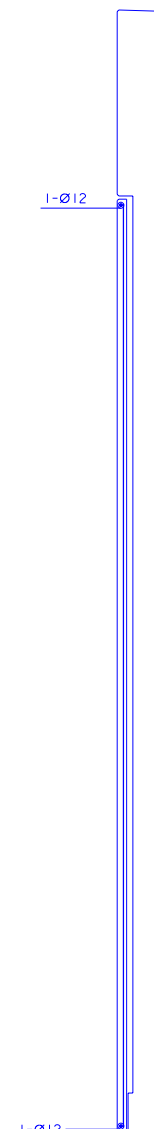
ELEVATION
N T S



SECTION-B

N T S

OUTERSKIN REINFORCEMENT (PANEL TYPE-9)



SECTION-A
N T S

1. ALL DIMENSIONS ARE IN mm.
2. TYPICAL REINFORCEMENT DETAIL.
3. REINFORCEMENT, YIELD STRENGTH OF REBAR $f_y=420$ Mpa, ASTM 615, GRADE 60 BLACK STEEL (EPOXY COATED).
4. 30mm MIN. EXTERNAL/20mm MIN. INTERNAL.

REFERENCE DRAWINGS	
GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. PRECAST PANEL DETAILS	DA-950158.001-013
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: NTS

SAUDI ARABIAN OIL COMPANY	
DRAWN BY _____	CHECKED BY _____
DATE STARTED _____	DATE COMPLETED _____
DISCIPLINE ENGINEER _____	
PROJECT ENGINEER _____	

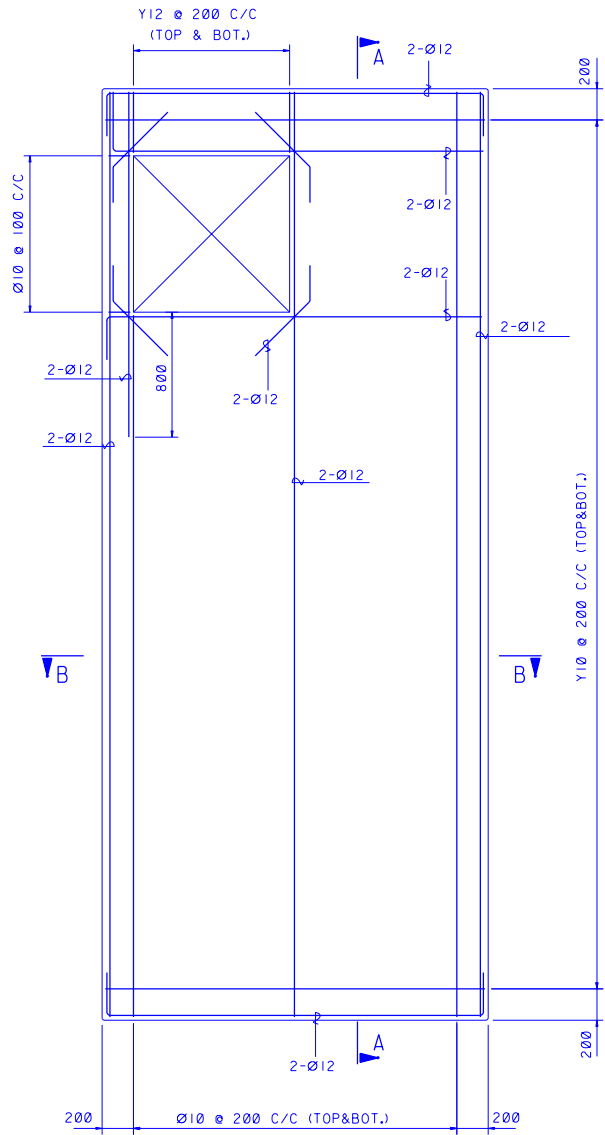
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: _____
	SIGNATURE _____ (NAME) DATE _____

TYP.PRECAST STRUCTURAL DETAIL	DRAWING TYPE SDT
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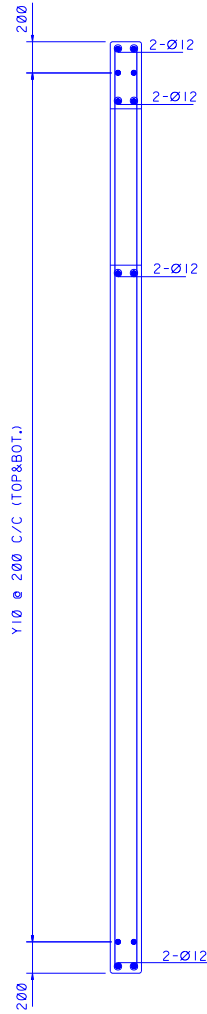
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	Q	DA-950163	005	00

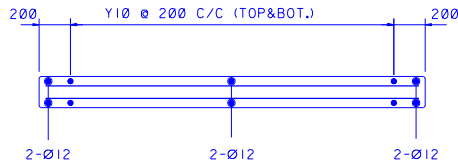
00	006	3	DA-950163	0	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	



ELEVATION
N T S

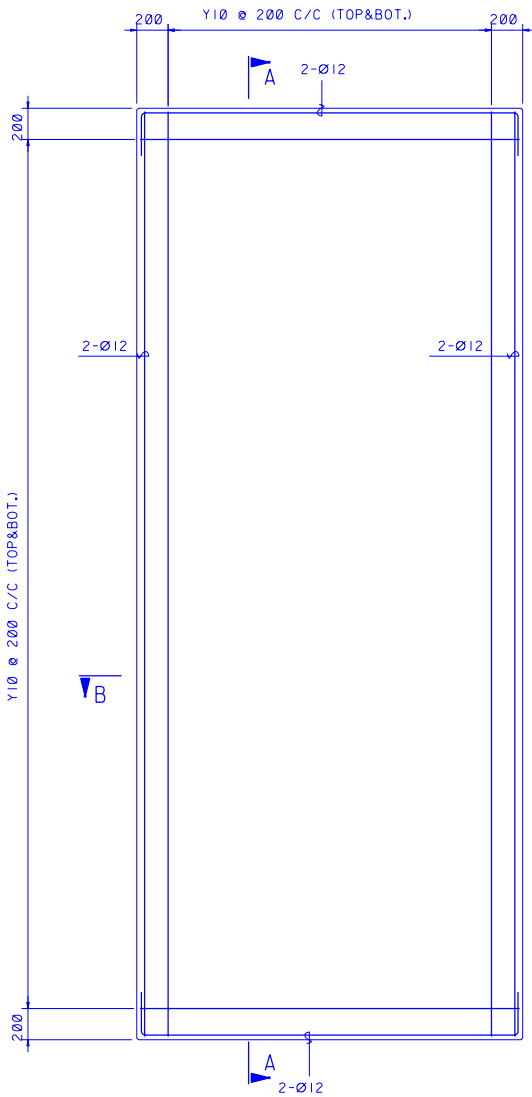


SECTION-A
N T S

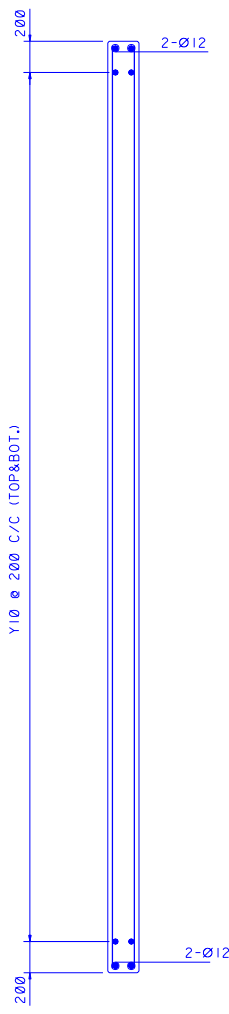


SECTION-B
N T S

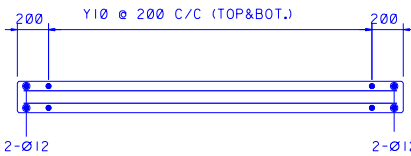
INTERNAL PLAIN PANEL REINFORCEMENT (PANEL-TYPE 12)
NOT TO SCALE



ELEVATION
N T S

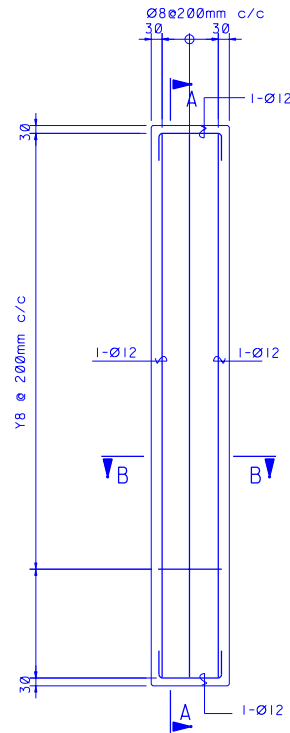


SECTION-A
N T S



SECTION-B
N T S

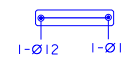
INTERNAL PLAIN PANEL REINFORCEMENT (PANEL-TYPE 11)
NOT TO SCALE



ELEVATION
N T S



SECTION-A
N T S



SECTION-B
N T S

- NOTES:
1. ALL DIMENSIONS ARE IN mm.
 2. TYPICAL REINFORCEMENT DETAIL.
 3. REINFORCEMENT, YIELD STRENGTH OF REBAR $f_y=420$ MPa, ASTM 615, GRADE 60 BLACK STEEL (EPOXY COATED).
 4. 30mm MIN. EXTERNAL / 20mm MIN. INTERNAL.

REFERENCE DRAWINGS

GENERAL NOTES & INSTRUCTIONS	DA-950151.001
FLOOR PLAN-STD. SUBSTN. 10M WIDE	DA-950153.001
FLOOR PLAN-STD. SUBSTN. 15M WIDE	DA-950153.002
ELEVATION-STD. SUBSTN. 10M WIDE	DA-950154.001
ELEVATION-STD. SUBSTN. 15M WIDE	DA-950154.002
SECTION-STD. SUBSTN. 10M WIDE	DA-950155.001
SECTION-STD. SUBSTN. 15M WIDE	DA-950155.002
TYP. PRECAST PANEL DETAILS	DA-950158.001-013
TYP. CONNECTION DETAILS	DA-950159.001
TYP. CONNECTION DETAILS	DA-950159.002
TYP. CONNECTION DETAILS	DA-950159.003

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.

SAUDI ARABIAN OIL COMPANY

DRAWN BY	CHECKED BY
DATE STARTED	DATE COMPLETED
DISCIPLINE ENGINEER	
PROJECT ENGINEER	

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.	CERTIFIED BY: _____	NAME: _____	DATE: _____
	SIGNATURE		

TYP. PRECAST STRUCTURAL DETAIL	DRAWING TYPE
	SDT

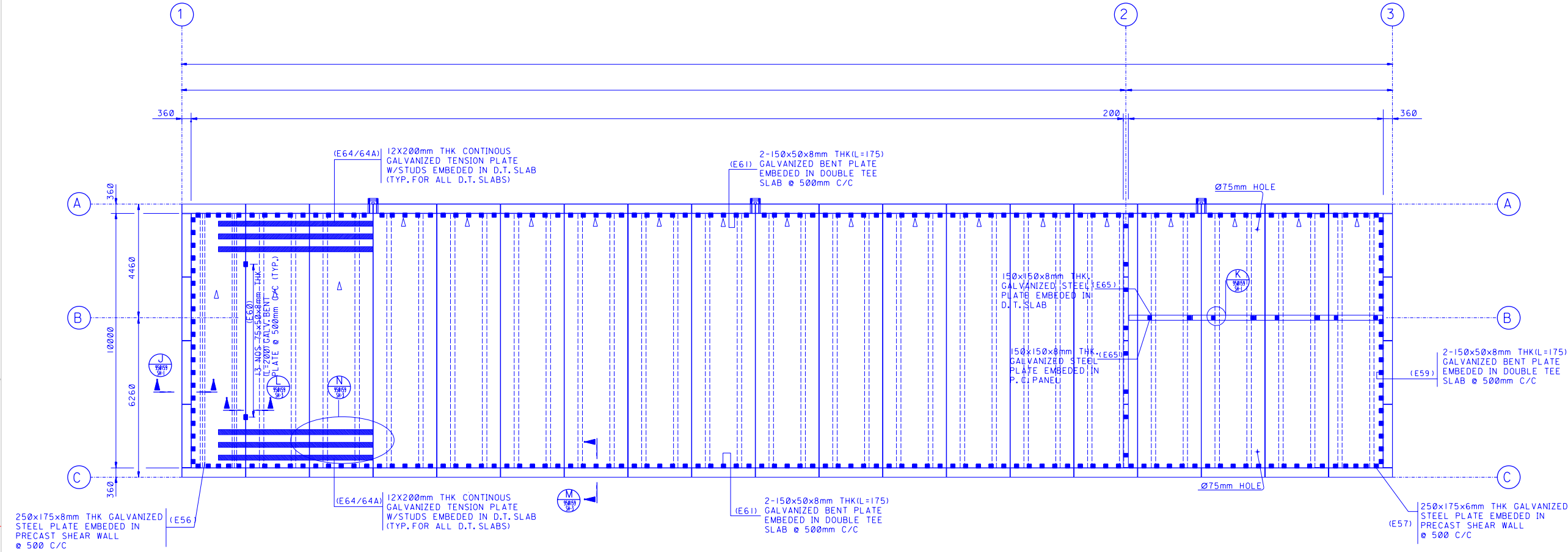
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	Q	DA-950163	006	00

APPENDIX C.4

Roof Slab Drawings

00	001	00	0A-950160	R	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	



ROOF DOUBLE TEE SLAB LAYOUT
N T S

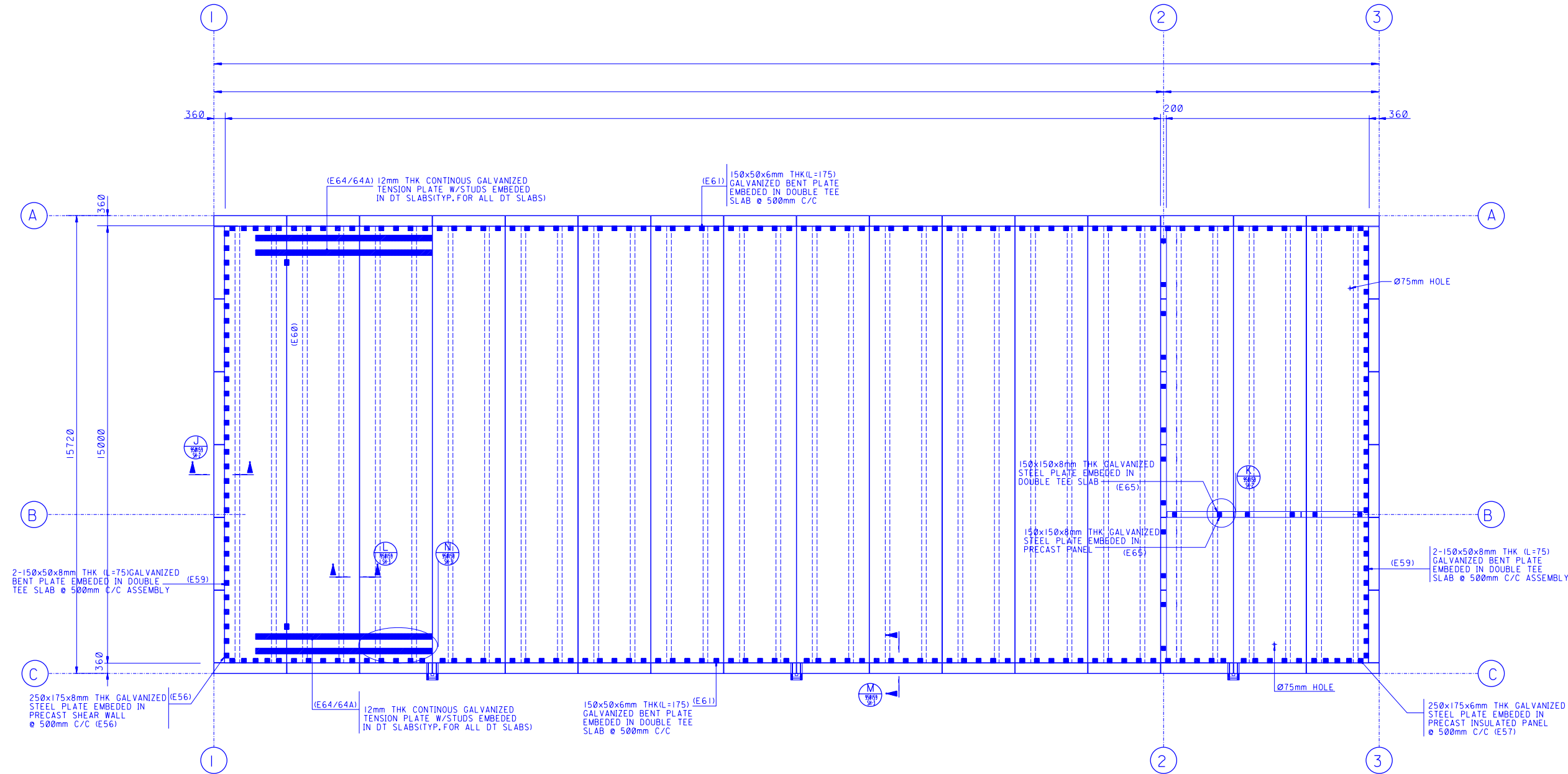
NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETERS AND ALL ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.

REFERENCE DRAWINGS	
FLOOR PLAN	DA-950153.001
ROOF PLAN	DA-950156.001
TYPICAL CONNECTION DETAILS	DA-950159.001
TYPICAL CONNECTION DETAILS	DA-950159.003

SCALE: 1:100

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY		CHECKED BY				
DATE STARTED		DATE COMPLETED				
DISCIPLINE ENGINEER						
PROJECT ENGINEER						
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.		CERTIFIED BY:		NAME: DATE:		
SIGNATURE		DATE		DRAWING TYPE		
ROOF DOUBLE TEE SLAB LAYOUT		ARC				
STANDARD SUBSTATION - 10M WIDE						
LIBRARY DRAWING		SAUDI ARABIA				
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950160	001	00		

00	002	00	DA-950160	R	M88
REV. NO.	SHT. NO.	DRAWING NUMBER	INDEX	PLANT NO.	



NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS AND ALL ELEVATIONS ARE IN METERS UNLESS OTHERWISE NOTED.

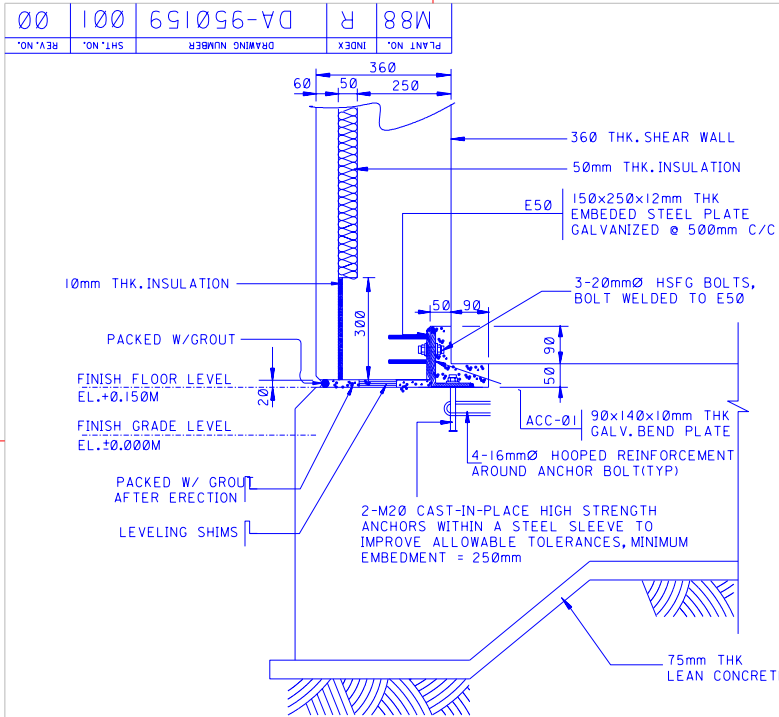
REFERENCE DRAWINGS	
FLOOR PLAN	DA-950153.002
ROOF PLAN	DA-950156.002
TYPICAL CONNECTION DETAILS	DA-950159.002
TYPICAL CONNECTION DETAILS	DA-950159.003

SCALE: _____

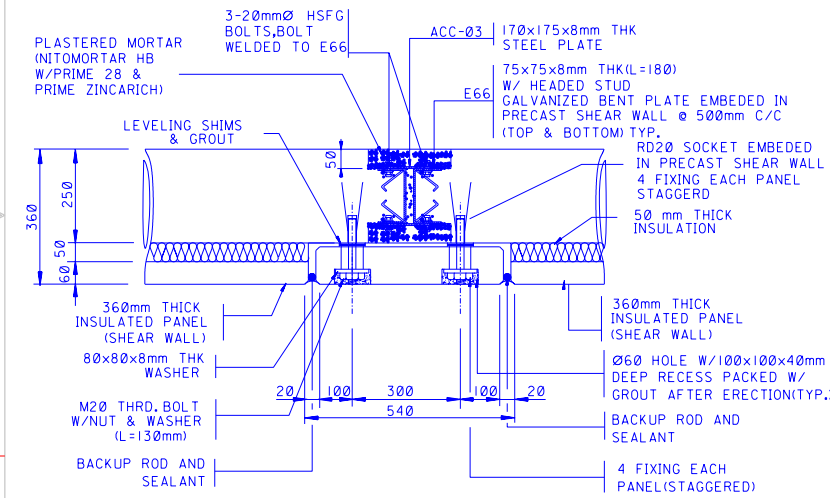
REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPVD.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____			CHECKED BY _____			
DATE STARTED _____			DATE COMPLETED _____			
DISCIPLINE ENGINEER _____			PROJECT ENGINEER _____			
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.			CERTIFIED BY: _____		NAME: _____ DATE: _____	
SIGNATURE _____			DATE _____		DRAWING TYPE _____	
ROOF DOUBLE TEE SLAB LAYOUT			ARC			
STANDARD SUBSTATION - 15M WIDE						
LIBRARY DRAWING SAUDI ARABIA						
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950160	002	00		

APPENDIX C.5

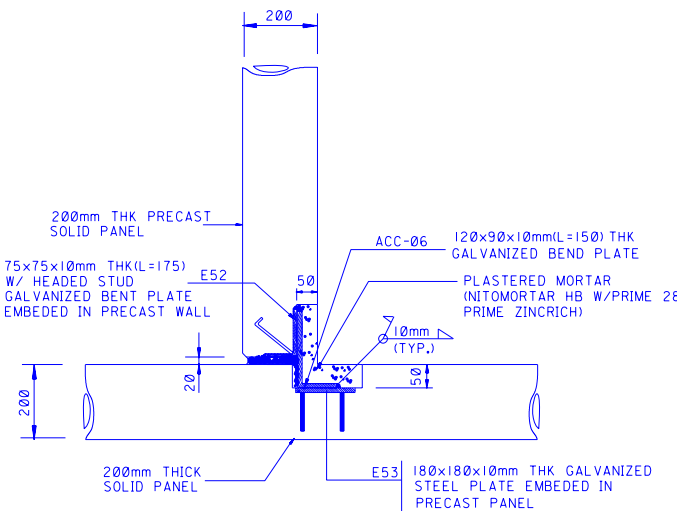
Connection Details



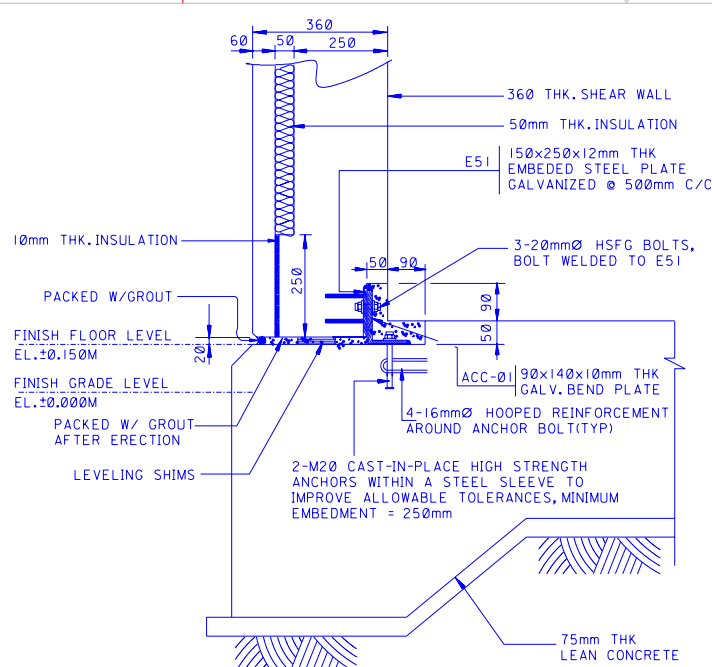
A SHEAR WALL TO FOUNDATION SLAB
N T S



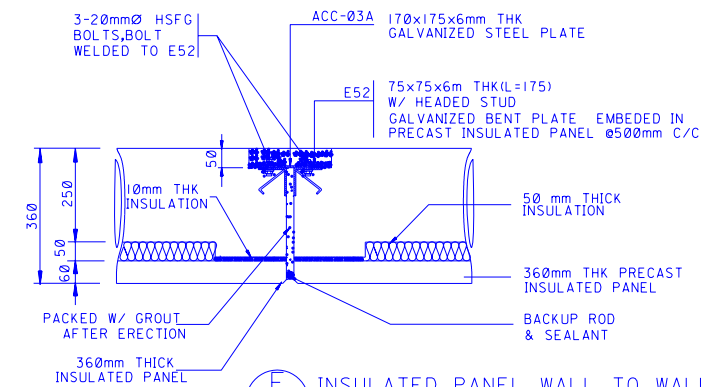
D SHEAR WALL, WALL TO WALL
N T S



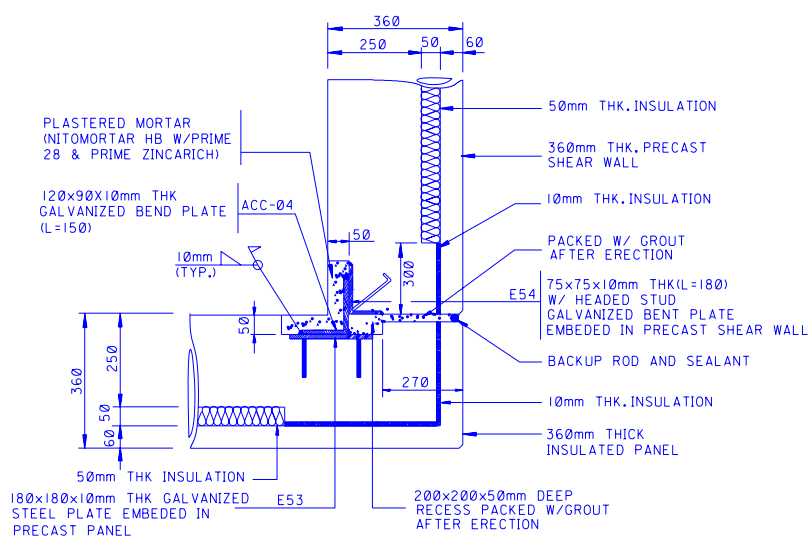
F SOLID PANEL WALL TO WALL
N T S



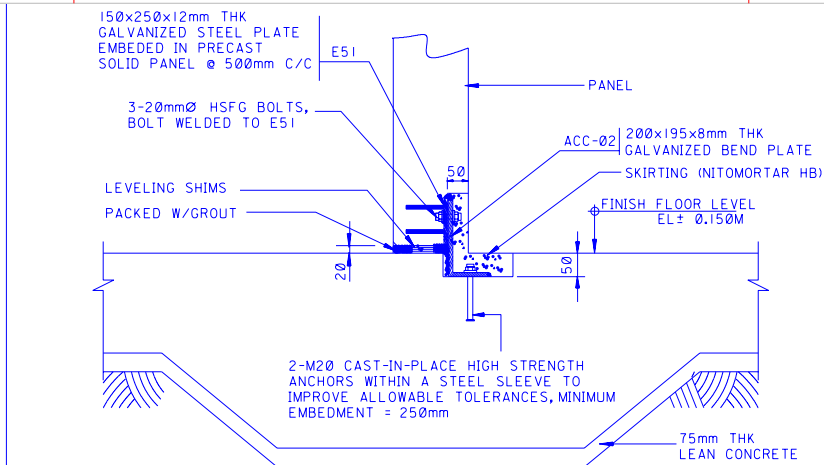
B INSULATED PANEL EXTERIOR WALL TO SLAB
N T S



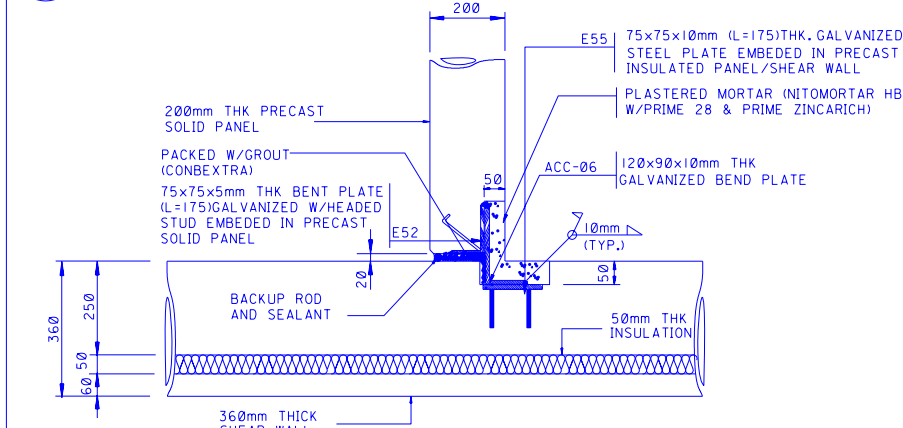
E INSULATED PANEL WALL TO WALL
N T S



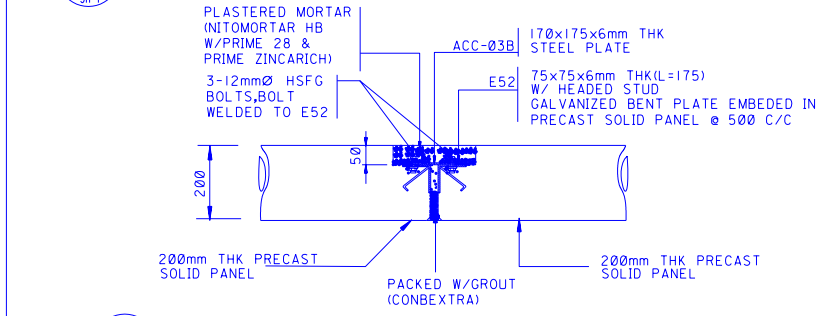
G SHEAR WALL TO INSULATED EXTERIOR WALL
N T S



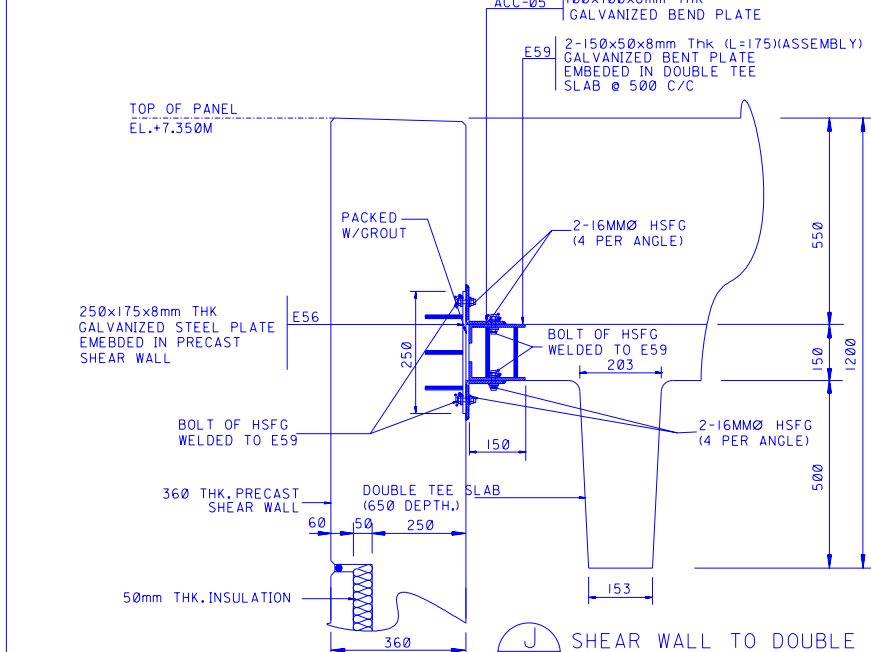
C SOLID PANEL INTERIOR WALL TO SLAB
N T S



H SHEAR WALL/INSULATED PANEL TO INTERIOR SOLID PANEL
N T S



I SOLID PANEL, INTERIOR WALL TO WALL
N T S

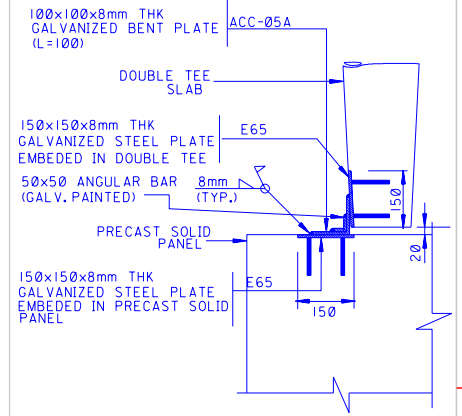


J SHEAR WALL TO DOUBLE TEE
N T S

- NOTES:**
1. ALL DIMENSIONS ARE IN MILLIMETERS AND ALL LEVELS ARE IN METERS UNLESS NOTED OTHERWISE.
 2. FINISHING AS PER APPROVED SAMPLE
 3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING)
 4. WHITE CEMENT & KHURAI AGGREGATE ON EXTERNAL FACE, (60mm THICK) BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.
 5. PRIOR APPROVAL SHALL BE OBTAINED FOR ALL AND ANY CHANGES MADE TO THE STEEL EMBEDMENT AND OR FIXING ANGLE PLATES.
 6. HOLES FOR FIXING IN ACCESSORIES TO BE SLOTTED AND OR DRILLED ON SITE TO SUIT EMBEDMENTS.
 7. SINCE ACCESS TO THE HEAD OF THE HSGF BOLTS WILL NOT BE POSSIBLE DURING FABRICATION, THE HEADS OF ALL OF THE HSGF BOLTS SHALL BE INSERTED AND RESTRAINED AGAINST ROTATION USING EITHER SHIMS OR WELD AND PROTECTED FROM THE CONCRETE.

LEGEND	
F. F. L.	- FINISH FLOOR LEVEL
T. O. C.	- TOP OF CONCRETE
T. O. P.	- TOP OF PANEL
▷	- ERECTION MARK
HSGF	- HIGH STRENGTH FRICTION GRIP

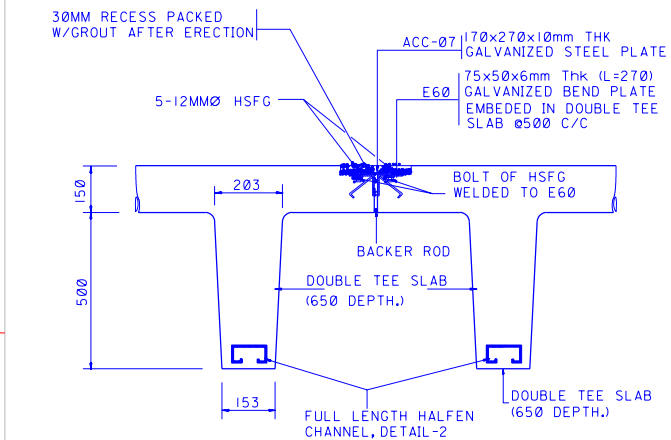
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TYP. PRECAST PANEL DETAILS	DA-950158.001,013
ROOF DOUBLE TEE SLAB LAYOUT	DA-950160.001
ROOF DOUBLE TEE SLAB LAYOUT	DA-950160.002
FOUNDATION DETAILS	DA-950161.001
FOUNDATION PLAN	DA-950162.001
FOUNDATION LAYOUT	DA-950162.002



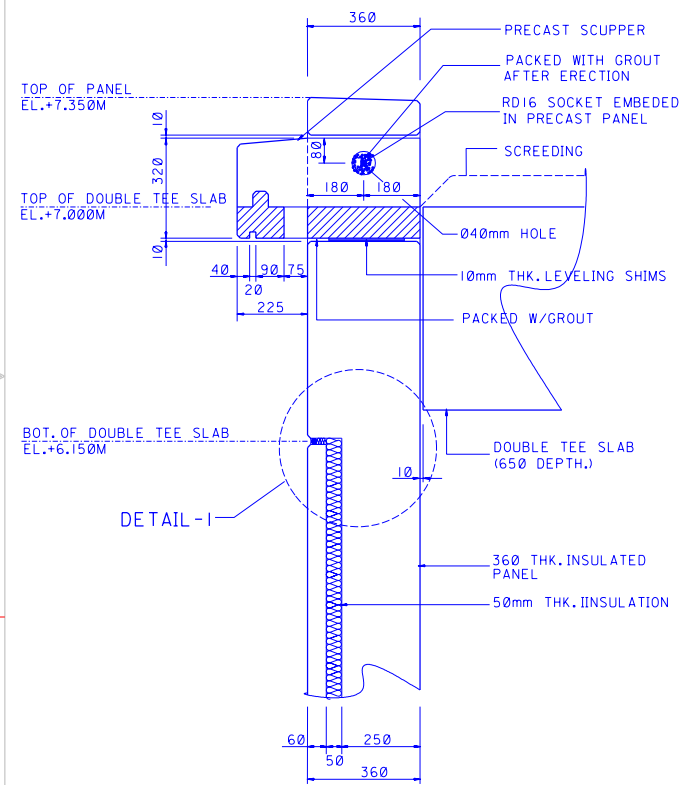
K DOUBLE TEE TO SOLID PANEL
N T S

SCALE: <u> N T S </u>								

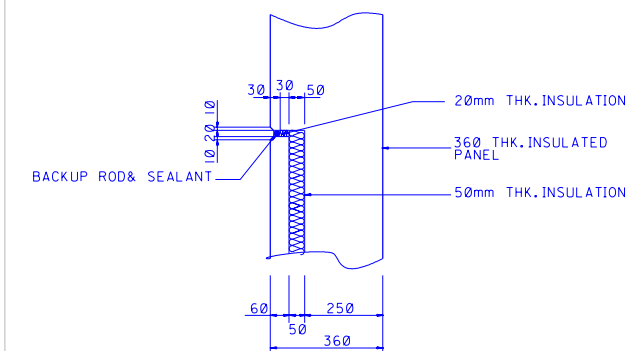
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M88	R	DA-950159	003	00



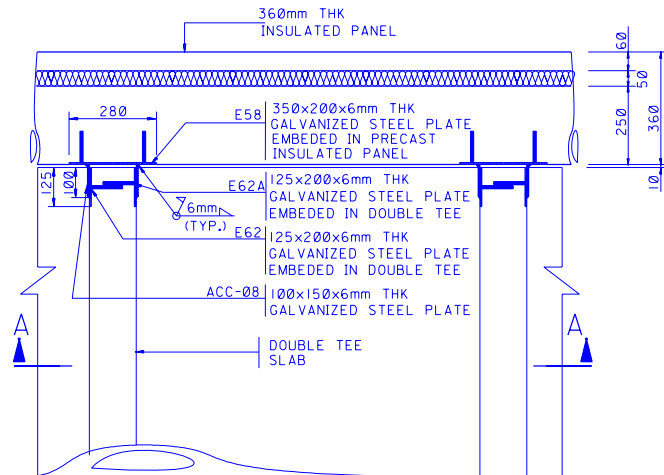
DOUBLE TEE TO DOUBLE TEE
N T S



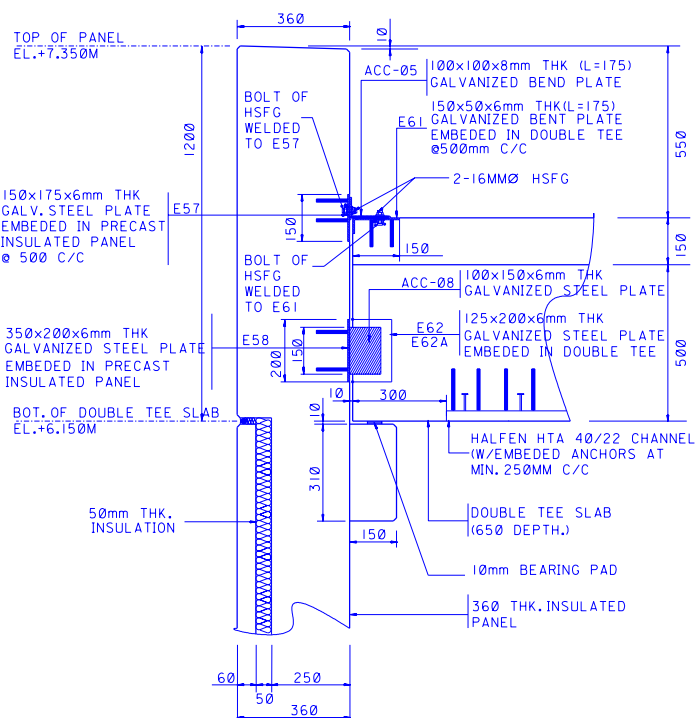
TYPICAL SCUPPER DETAIL
N T S



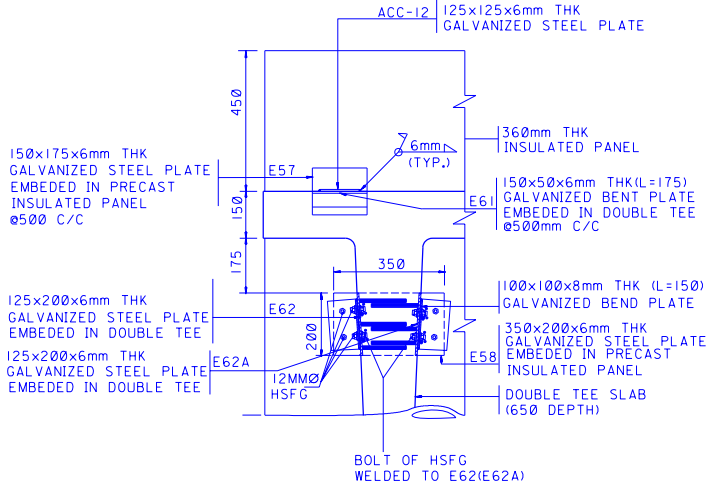
TYPICAL DETAIL
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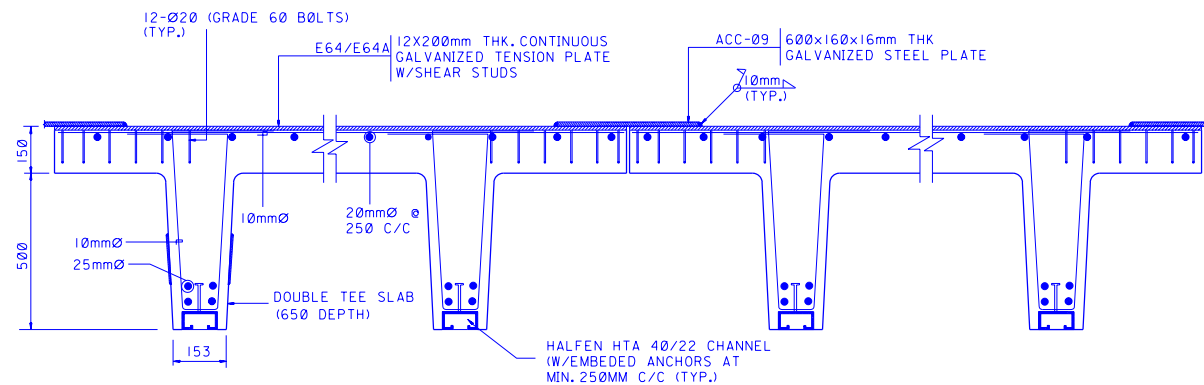
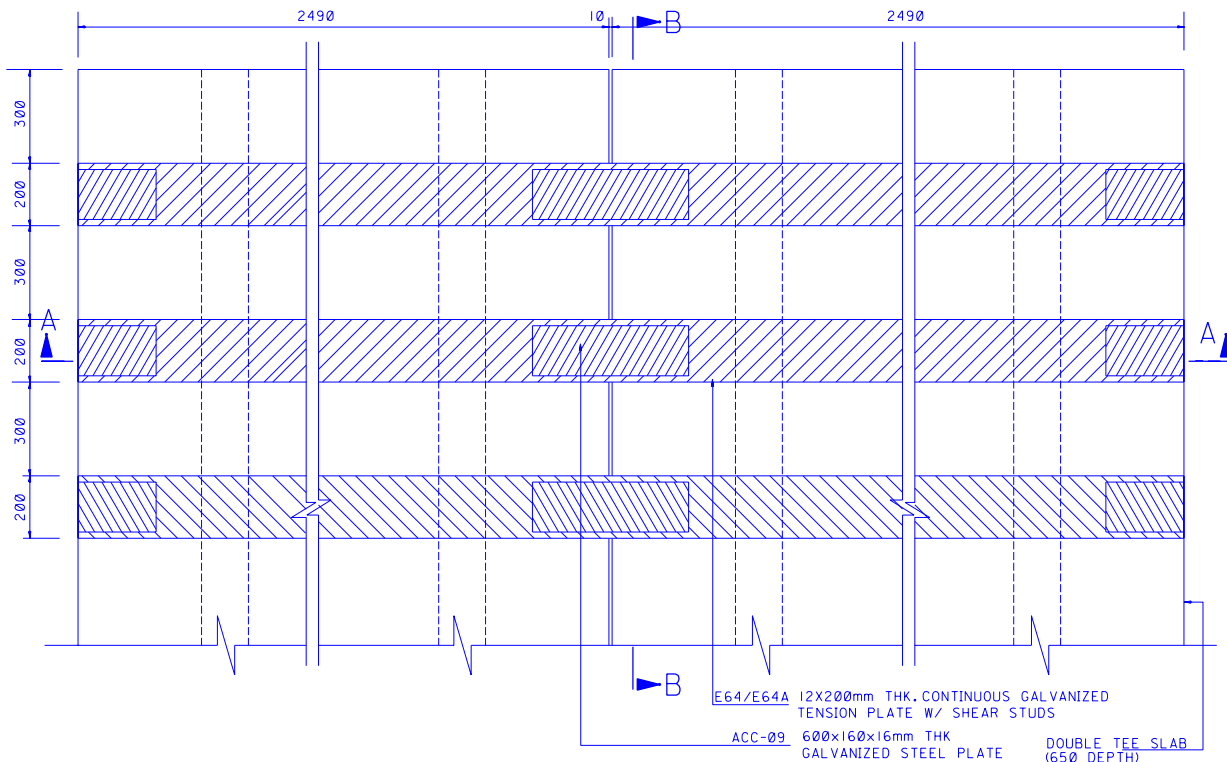
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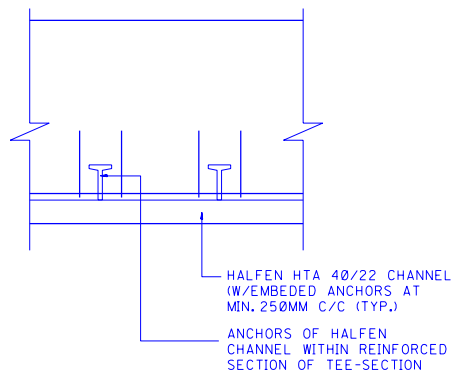
ELEVATION
N T S



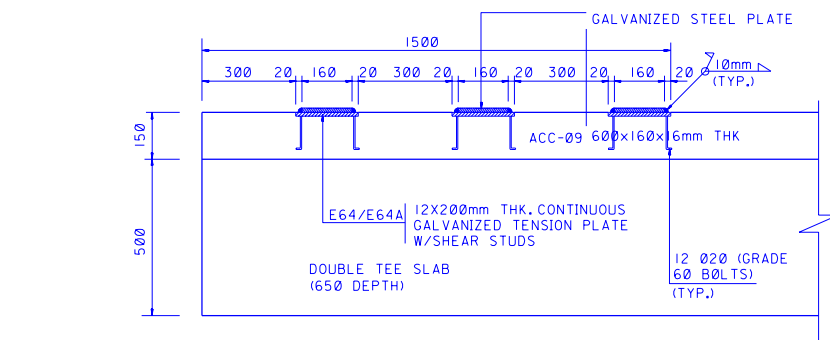
SECTION-A
(INSULATED PANEL TO DOUBLE TEE)



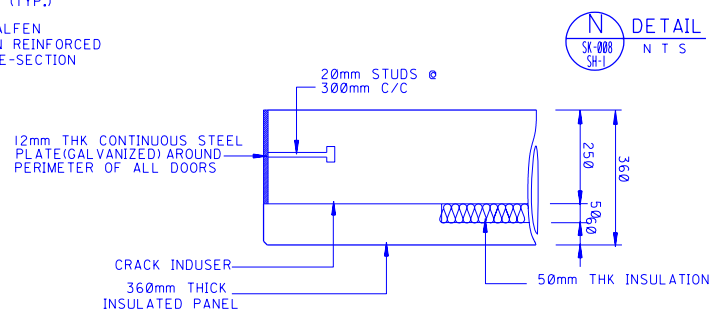
SECTION-A
N T S



2 TYPICAL DETAIL
— N T S



SECTION-B




 TYPICAL DOOR PLATE EMBEDMENT DETAIL
 N T S

1. ALL DIMENSIONS ARE IN MILLIMETERS AND ALL LEVELS ARE IN METERS UNLESS NOTED OTHERWISE.
2. FINISHING AS PER APPROVED SAMPLE
3. REINFORCEMENT- BLACK STEEL, ASTM A 615, GRADE 60 BLACK STEEL (WITH EPOXY COATING)
4. WHITE CEMENT & KHURAISS AGGREGATE ON EXTERNAL FACE, (60mm THICK)
BACK FACE GREY CEMENT & ABU HADRIYAH AGGREGATES.
5. PRIOR APPROVAL SHALL BE OBTAINED FOR ALL AND ANY CHANGES MADE TO THE STEEL EMBEDMENT AND OR FIXING ANGLE PLATES.
6. HOLES FOR FIXING IN ACCESSORIES TO BE SLOTTED AND OR DRILLED ON SITE TO SUIT EMBEDMENTS.
7. SINCE ACCESS TO THE HEAD OF THE HSFG BOLTS WILL NOT BE POSSIBLE DURING FABRICATION, THE HEADS OF ALL OF THE HSFG BOLTS SHALL BE INSERTED AND RESTRAINED AGAINST ROTATION USING EITHER SHIMS OR WELD AND PROTECTED FROM THE CONCRETE.

REFERENCE DRAWINGS	
TYP. PRECAST PANEL DETAILS	DA-950158.001.013
ROOF DOUBLE TEE SLAB LAYOUT	DA-950160.001
ROOF DOUBLE TEE SLAB LAYOUT	DA-950160.002
FOUNDATION DETAILS	DA-950161.001
FOUNDATION PLAN	DA-950162.001
FOUNDATION LAYOUT	DA-950162.002

SCALE: N T S

REV. NO.	DATE	BY	DESCRIPTION	CHKD.	CERT.	APPV.
SAUDI ARABIAN OIL COMPANY						
DRAWN BY _____			CHECKED BY _____			
DATE STARTED _____			DATE COMPLETED _____			
DISCIPLINE ENGINEER _____						
PROJECT ENGINEER _____						

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.

CERTIFIED BY: _____
SIGNATURE _____

TITLE DATE _____

TYPICAL CONNECTION DETAILS	DRAWING TYPE ARC
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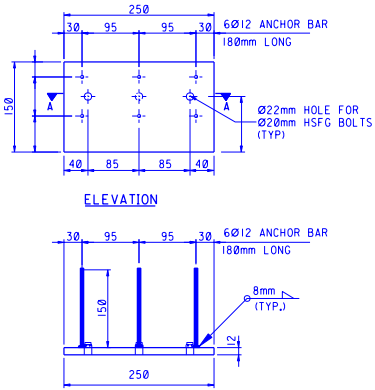
STANDARD SUBSTATION

LIBRARY DRAWING			SAUDI ARABIA	
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950159	003	00

APPENDIX C.6

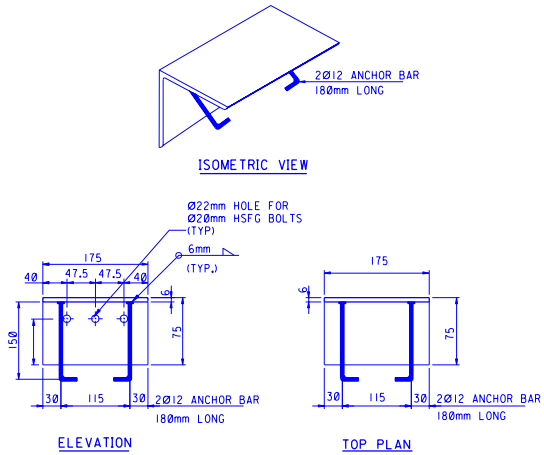
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00	004	00	DA-950159	R	M88
PLANT NO.	INDEX	REV. NO.	DRAWING NUMBER	SHT. NO.	REV. NO.



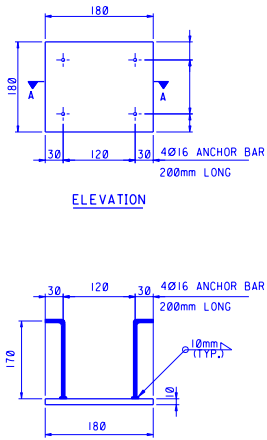
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STEEL PLATE GALVANIZED
150x250x12mm THK.

EMBEDS (E-50 & E51)



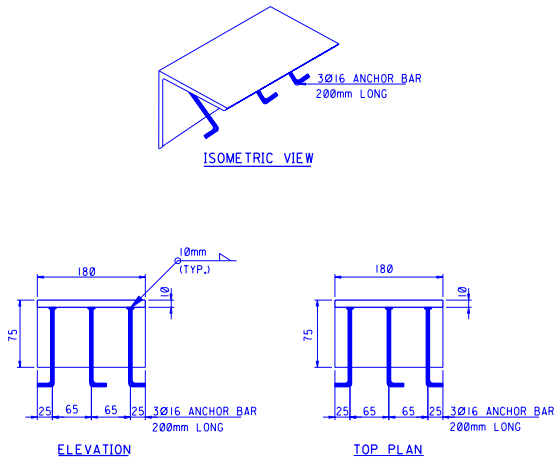
BENT PLATE GALVANIZED
75x75x5mm THK
(L=175)

EMBEDS (E-52)



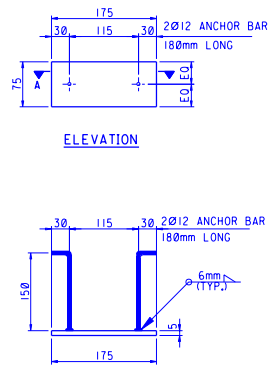
SECTION-A
STEEL PLATE GALVANIZED
180x180x10mm THK

EMBEDS (E-53)



BENT PLATE GALVANIZED
75x75x10mm THK
(L=180)

EMBEDS (E-54)



SECTION-A
STEEL PLATE
75x175x5mmTHK

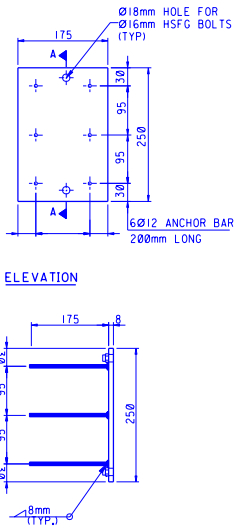
EMBEDS (E-55)

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS AND UNLESS NOTED OTHERWISE.
2. PRIOR APPROVAL SHALL BE OBTAINED FOR ALL AND ANY CHANGES MADE TO THE STEEL EMBEDMENT AND OR FIXING ANGLE PLATES.
3. HOLES FOR FIXING IN ACCESSORIES TO BE SLOTTED AND OR DRILLED ON SITE TO SUIT EMBEDMENTS.

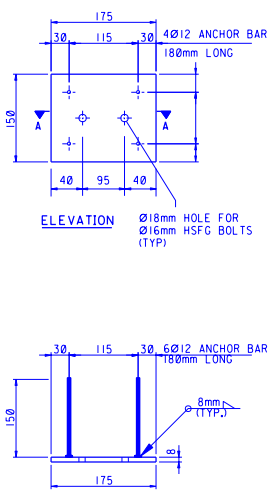
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ROOF DOUBLE TEE SLAB LAYOUT	DA-950160.002
FOUNDATION DETAILS	DA-950161.001
FOUNDATION PLAN	DA-950162.001
FOUNDATION LAYOUT	DA-950162.002



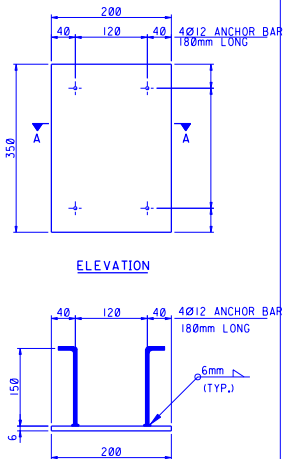
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STEEL PLATE GALVANIZED
250x175x8mmTHK

EMBEDS (E-56)



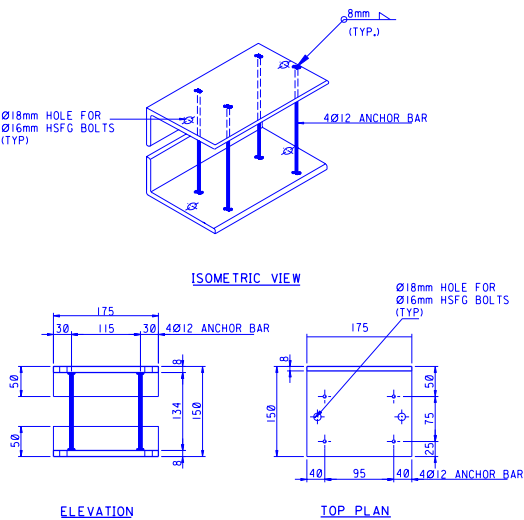
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STEEL PLATE GALVANIZED
150x175x8mmTHK

EMBEDS (E-57)



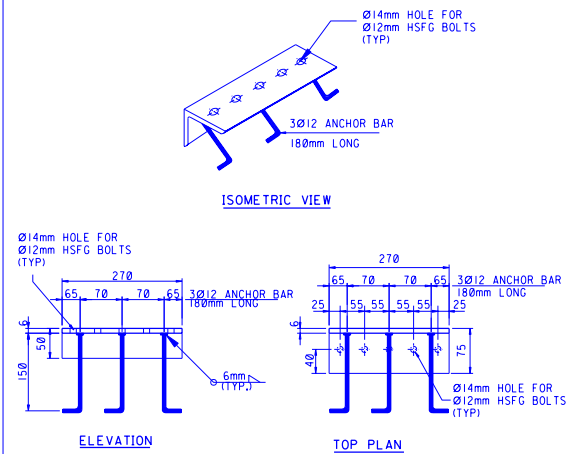
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STEEL PLATE GALVNIZED
350x200x6mm

EMBEDS (E-58)



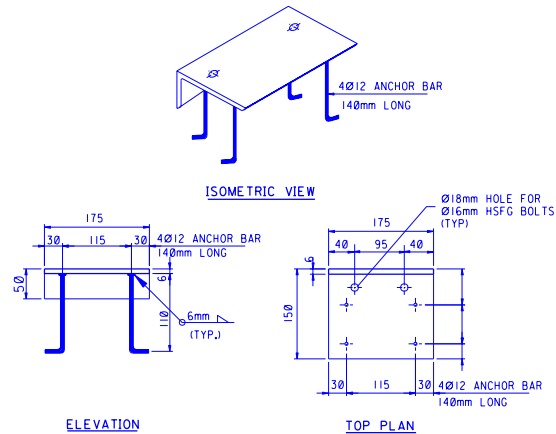
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2-150x50x8mm THK
(L=175)

EMBEDS (E-59)



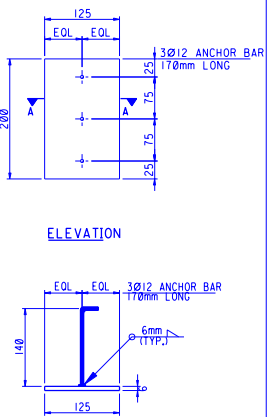
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75x50x6mm THK
(L=270)

EMBEDS (E-60)



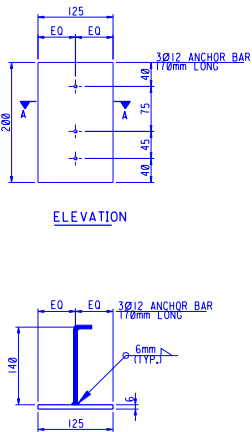
BENT PLATE GALVANIZED
150x50x6mm THK
(L=175)

EMBEDS (E-61)



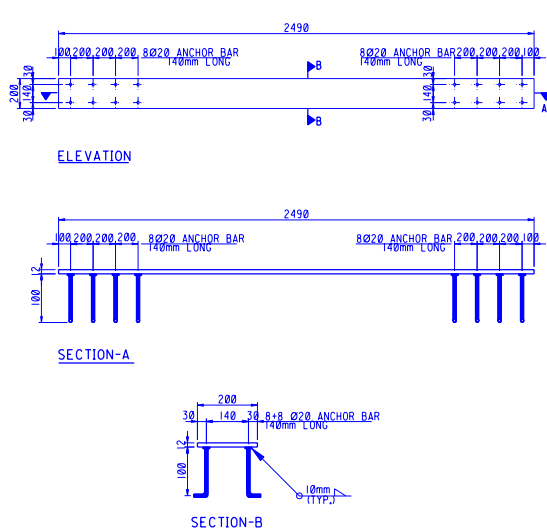
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STEEL PLATE GALVANIZED
125x200x6mm THK

EMBEDS (E-62)



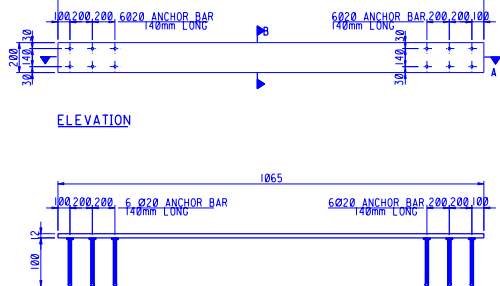
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STEEL PLATE
125x200x6mm THK

EMBEDS (E-62A)



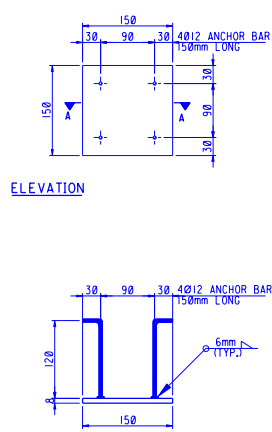
TENSION PLATE W/ SHEAR STUDS GALVANIZED
2490x200x12mm THK

EMBEDS (E-64)



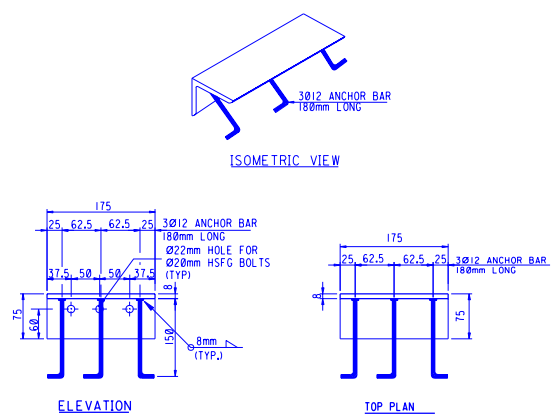
TENSION PLATE W/ SHEAR STUDS GALVANIZED
1065x200x12mm THK

EMBEDS (E-64A)



SECTION-A
STEEL PLATE GALVANIZED
150x150x8mmTHK

EMBEDS (E-65)



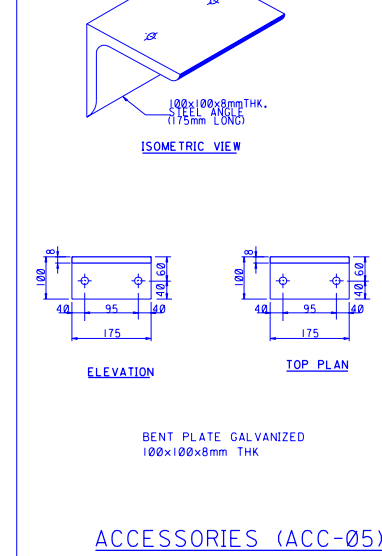
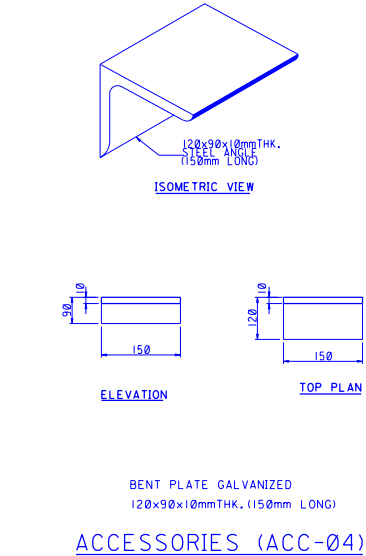
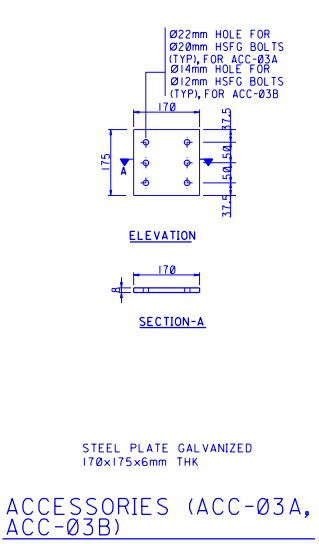
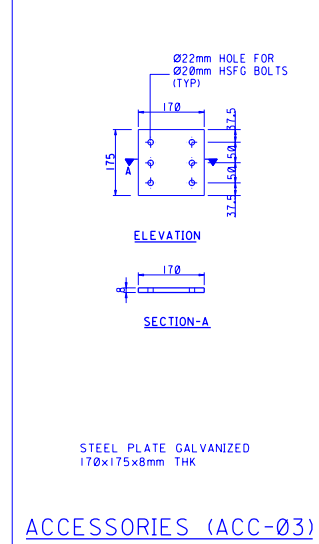
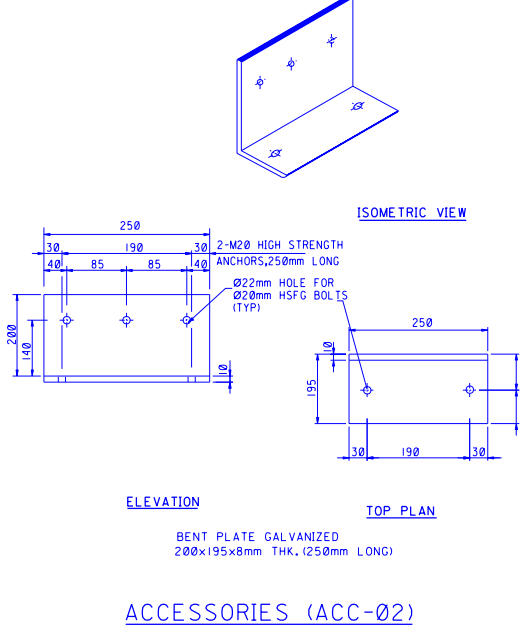
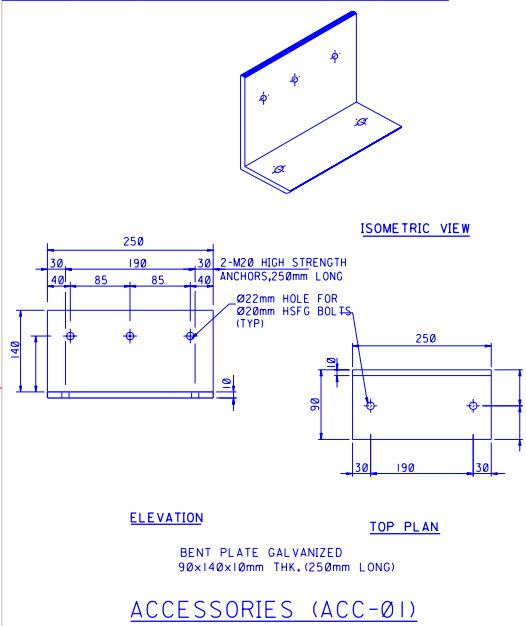
BENT PLATE
75x75x8mm THK
(L=175)

EMBEDS (E-66)

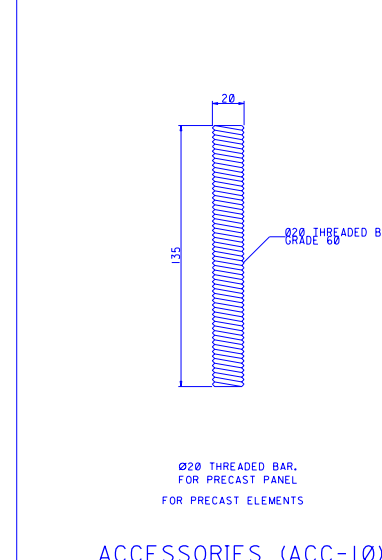
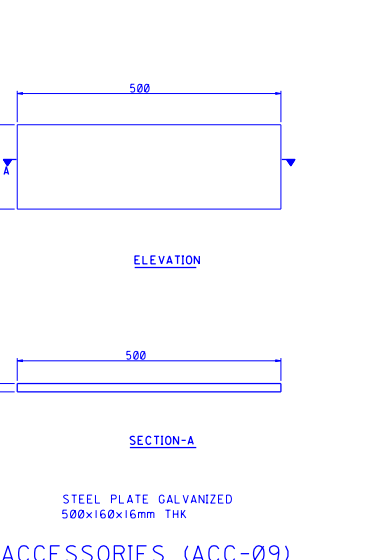
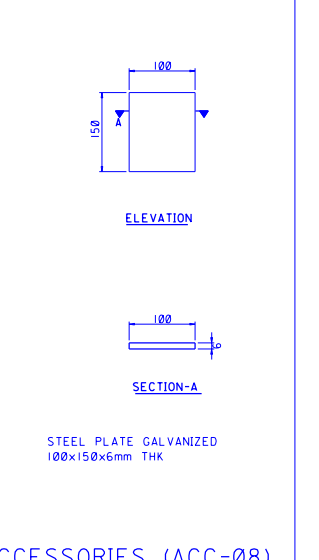
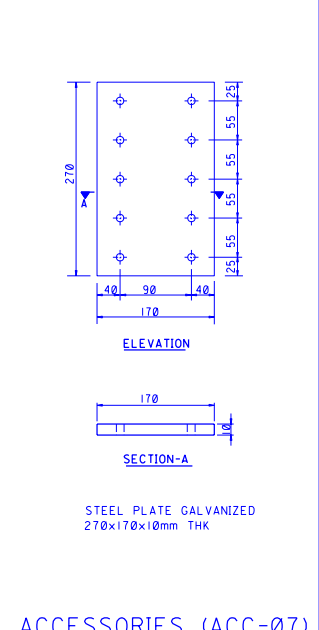
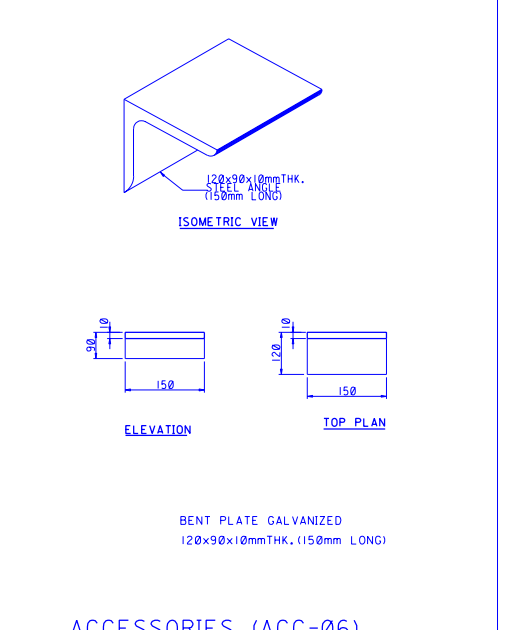
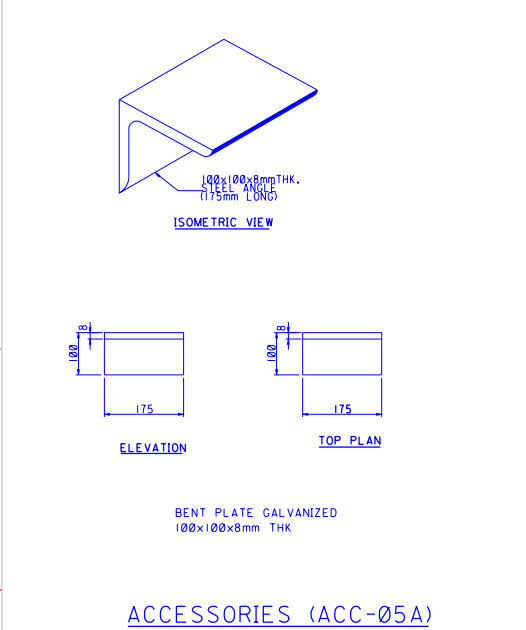
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DRAWN BY			CHECKED BY			
DATE STARTED			DATE COMPLETED			
DISCIPLINE ENGINEER			PROJECT ENGINEER			
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.			CERTIFIED BY: NAME: DATE: SIGNATURE: DATE:			
TYPICAL STEEL PLATE EMBEDS DETAILS			DRAWING TYPE			
STANDARD SUBSTATION			ARC			
LIBRARY DRAWING			SAUDI ARABIA			
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.		
M88	R	DA-950159	004	00		

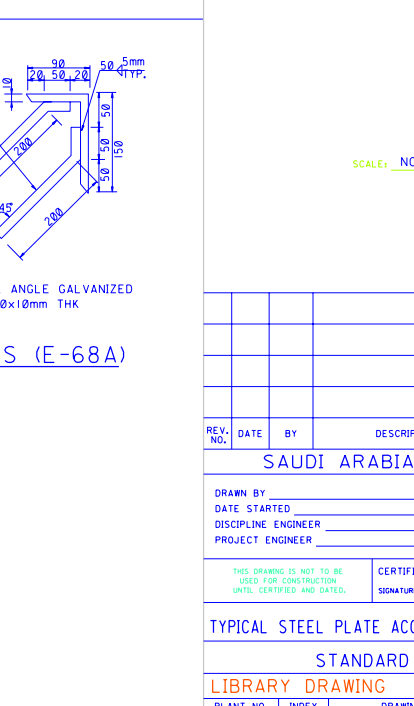
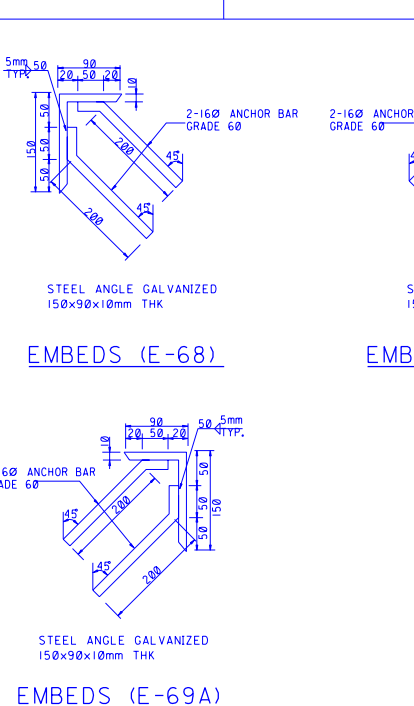
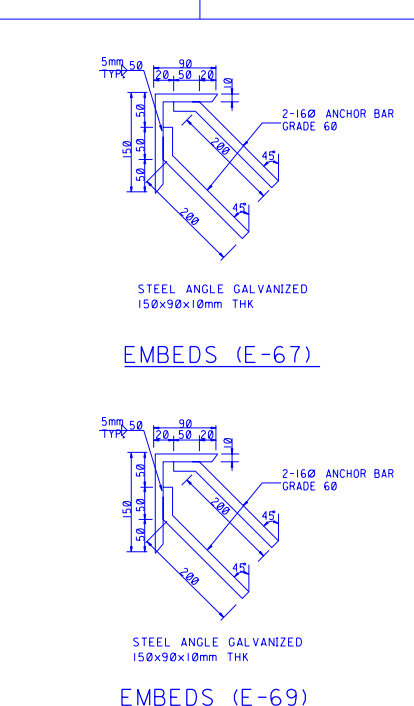
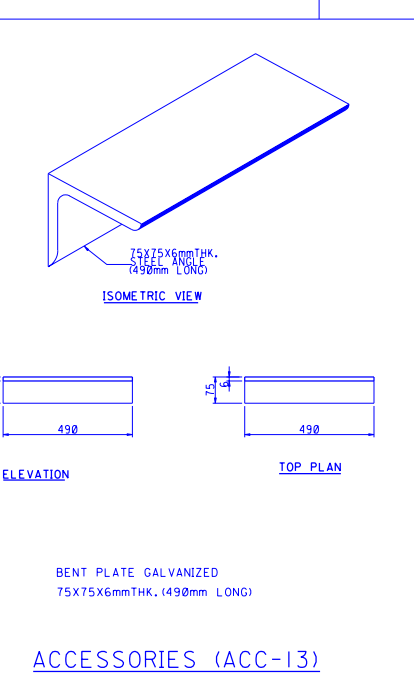
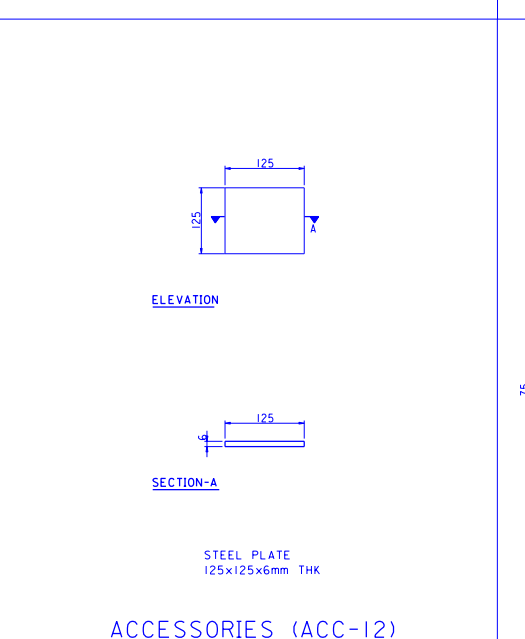
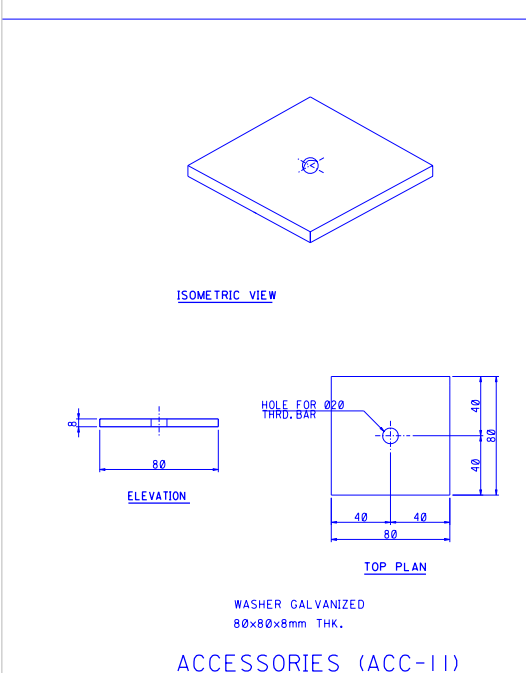
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PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.



- NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETERS AND UNLESS NOTED OTHERWISE.
 2. PRIOR APPROVAL SHALL BE OBTAINED FOR ALL AND ANY CHANGES MADE TO THE STEEL EMBEDMENT AND OR FIXING ANGLE PLATES.
 3. HOLES FOR FIXING IN ACCESSORIES TO BE SLOTTED AND OR DRILLED ON SITE TO SUIT EMBEDMENTS.



REFERENCE DRAWINGS	
TYP. PRECAST PANEL DETAILS	DA-950158.001,013
ROOF DOUBLE TEE SLAB LAYOUT	DA-950160.001
ROOF DOUBLE TEE SLAB LAYOUT	DA-950160.002
FOUNDATION DETAILS	DA-950161.001
FOUNDATION PLAN	DA-950162.001
FOUNDATION LAYOUT	DA-950162.002



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REV. NO.	DATE	BY	DESCRIPTION	CHKD. CERT. APPV.
SAUDI ARABIAN OIL COMPANY				
DRAWN BY		CHECKED BY		
DATE STARTED		DATE COMPLETED		
DISCIPLINE ENGINEER				
PROJECT ENGINEER				
THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL CERTIFIED AND DATED.		CERTIFIED BY: NAME: DATE: SIGNATURE: DATE:		
TYPICAL STEEL PLATE ACCESSORIES DETAILS				DRAWING TYPE
STANDARD SUBSTATION				ARC
LIBRARY DRAWING SAUDI ARABIA				
PLANT NO.	INDEX	DRAWING NUMBER	SHT. NO.	REV. NO.
M88	R	DA-950159	005	00

APPENDIX D

SAES-M-09 Appendix A, Blast Design Requirements (BRD) Data Sheet

Blast Design Requirements (BDR) Data Sheet

Page 1 of 3

Facility Name/Location: <u>ABU ALI SUBSTATION</u>		Job Number: <u>BI – 3039</u>	
Unit: _____		Building Number: _____	
Building Name: <u>SUBSTATION 2</u>		Building Number: _____	

A-1. Blast Resistant Building Construction

A-1.1 Building Response Range: ☒ Low ☐ Medium ☐ High

A-1.2 Building Structural System:
(Check construction type from each column below – refer to Tables 1 & 2. Provide details on under Special Requirements if necessary.)

Frame	Wall	Roof
☐ Cast-In-Place Concrete	☐ Cast-In-Place Concrete	☐ Cast-In-Place Concrete
☐ Hot-Rolled Steel	☒ Precast Concrete	☒ Precast Concrete
☐ Pre-Engineered Metal	☐ Reinforced Masonry / CMU	☐ Single Sheet Metal Panel
☒ Load Bearing Wall	☐ Single Sheet Metal Panel	☐ Insulated Metal Sandwich Panel
	☐ Insulated Metal Sandwich Panel	☐ Composite Concrete Deck
☐ Other: _____	☐ Other: _____	☒ Other: _____

A-1.3. Fragment Resistant Required: ☐ Yes ☒ No

A-2. Blast Load Design Requirements

For each load case, the blast on each surface shall be determined using procedures provided in the ASCE (1997) (Chapter 3), based on the peak side-on pressure and corresponding durations or impulses as follows:
(See Figure A1 on following page.)

Blast Load Parameters (1, 2, 3)

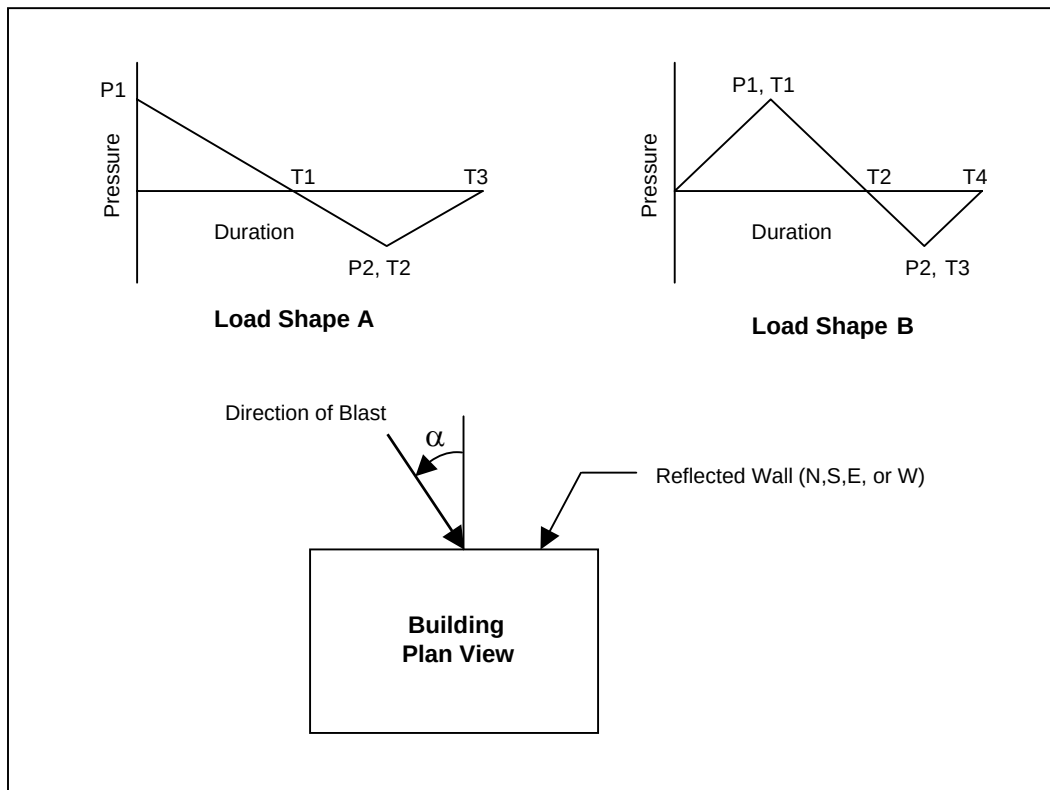
(4) Load Case	Description	Reflected Wall (4) (N, S, E, or W)	Load Shape and Angle		Peak Side-On Pressure (6)		Duration msec			
					unit:	Psi	T1	T2	T3	T4
			A, B	$\alpha^{(5)}$	P1	P2				
1		N,S,E & W	A	0	2		200			

Notes:

- (1) Blast loads shall be computed at the wall nearest the blast center.
- (2) Roof shall be considered loaded by the peak side-on pressure.
- (3) Depending on the load shape, some of the parameters may not be applicable.
- (4) Each load case may see a different reflected wall. Indicate the wall (N, S, E, or W) facing the blast for the current load case.
- (5) Indicate angle of blast source (in degrees) measured from the line normal to the reflected wall.
- (6) Choose pressure unit from list (psi, MPa, millibar) or enter special unit name.

No.	Date	Description	By	Check.	App.

Figure A1 - Load Shapes and Building Orientation



A-3. Blast Door Requirements

The table below summarizes the design requirements for each blast door located in the exterior walls of blast resistant buildings.

Blast Door ID or No.	Building Face Door Location (N, S, E, W)	Performance Category (see Table 11) (I, II)
D/6	W	II
D/7	N	II
D/8	N	II
D/9	N	II
D/10	E	II

A-4. Geotechnical Requirements (Refer to SAES-A-113)

A-4.1. Foundation Type (Select one and provide data in the appropriate section.):

☒ Mat Foundation ☐ Spread Footing ☐ Pile Supported

A-4.2. Mat Foundation

Item	Value	Unit (*) / Special Unit	
Ultimate Net Bearing Capacity	270	Kpa	kN/m ² (psf)
At Depth	1.0	M	M (ft)
Dynamic Modulus of Subgrade Reaction	6136	kN/m ³	kN/m ³ (pcf)
Sliding Friction Coefficient	0.4		
Passive Pressure Coefficient	3.25		

A-4.3. Spread Footing Foundation

Ultimate Net Bearing Capacity			kN/m ² (psf)
At Depth			m (ft)
Sliding Friction Coefficient			
Passive Pressure Coefficient			

A-4.4. Pile-Supported Foundation

Pile Description			
Maximum Vertical Pile Capacity			kN (kips)
Maximum Horizontal Pile Capacity			kN (kips)
Vertical Pile Spring Constant			kN/m (kips/in)
Horizontal Pile Spring Constant			kN/m (kips/in)

(*) Note: Choose unit from list or enter user-defined name under Special Unit.

A-5. Other Special Requirements:

☐ Blast Valves ☐ Blast Attenuators

Special Requirements:

Building is a precast reinforced concrete building with shearwalls. It is constructed from precast panels with cast-inserts welded together to form integral structure. It does not conform to Table 2 of SAES-M-009

Structural Precast Concrete Wall dimensions are 360mm thick (including 50mm thk insulation and 60mm thick concrete outer skin) and Precast Concrete Roof Panels are 150 mm thick

APPENDIX E

Supplementary Design Calculations for Aramco Comments

**CALCULATION
APPROVAL COVER SHEET**

PROJECT NO:

1282017

CALCULATION NO:

1282017-CALC-01

TITLE:

Further Substantiation of Blast Resistant Modular
Precast Concrete Substation Design Report

CLIENT:

Saudi Arabian Oil Company

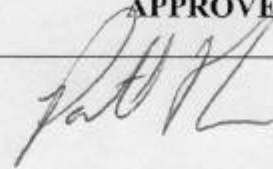
REVISION

DATE

APPROVED

0

3/2/05



COMMENTS

REV.	BY	DATE	CHECKED	DATE	CALCULATION NO:	PAGE
0	AJN	26/01/2005	PRM	03/02/2005	1282017-CALC-01	
					Further Substantiation of Blast Resistant Modular Precast Concrete Substation Design Report	ABS Consulting

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2 SCOPE	0
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1 INTRODUCTION

Saudi Arabian Oil Company (Saudi Aramco) are in the process of commissioning a number of blast resistant precast concrete substations, one of which is 47.5m by 10m wide and the other is 40m by 15m wide.

As part of the commissioning process ABS Consulting (ABSC) prepared a report for each substation (References 1 and 2), presenting the detailed design carried out solely by Saudconsult and Qanbar Dywidag Precast Concrete Ltd for these two buildings.

Following the Saudi Aramco design review, several issues were raised (see Reference 3) that required addressing before they could accept the design. Furthermore, it was requested that ABS Consulting (ABSC) undertake all of the additional assessment activities needed to address these issues, opposed to the designers.

The purpose of this calculation is to document further work carried out by ABSC to address the Saudi Aramco comments, as outlined in Reference 4.

2 SCOPE

The scope of this work is to undertake the additional assessment activities needed to address the Saudi Aramco comments (Reference 3) on the detailed design of the blast resistant precast concrete substations, as outlined in Reference 4. The issues addressed within this calculation are summarised below.

- Full Justification of Pressure-Impulse Diagrams: No substantiated Pressure-Impulse diagrams were provided in the previous design calculations. ABSC are to develop building specific Pressure-Impulse curves, with fully supporting calculations.
- Review of Connections to Minimise Site Welding: Saudi Aramco have concluded that the amount of site welding is unacceptable. ABSC are to review the proposed connection details, and where possible and judged cost effective to change, provide an alternative connection detail.
- Foundation Design Loads: Saudi Aramco require foundation loading data to be extracted from the Saudconsult and Qanbar Dywidag Precast Concrete Ltd design calculations, such that they can be tabulated for use by future foundation designers.
- Cast-In Inserts for Support of Services: Within the scope of the original design there was a requirement for an adequate fastening into the roof slab to allow for the hanging of building services off. The original design proposed a cast in insert through the web of the roof tee-section to allow hanging of the building services. Saudi Aramco have decided (Reference 5) that it is preferable to provide a cast-in insert into the underside of the roof tee-section, as this allows greater flexibility for the hanging of building services. ABSC will provide details of the preferred arrangement.

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3 RESULTS

3.1 Pressure-Impulse Diagrams

Figures 1 to 4 show low and medium response curves for Substation 1 and 2 (15m and 10m wide respectively). It is highlighted the medium response curves in Figures 3 and 4 are only attainable if a suitable stirrup arrangement is included within the wall design.

3.1.1 Overview of Changes Since Preliminary Design

The Pressure-Impulse diagrams presented in References 1 and 2 were based on calculations performed for the preliminary design. The Pressure-Impulse diagrams are based on the final structural details. Full supporting calculations are presented in Appendix A of this calculation.

A number of changes since the preliminary design have caused changes in the previous Pressure-Impulse diagrams. These changes include:-

- The concrete used in the roof tee-beam sections (7500psi) has a higher strength than in the original blast design,
- The reinforcement and cross-section of the roof tee-beam sections has been increased since the original blast design,
- The roof tee-beam sections are stronger than necessary for the design basis loading (SS1 stronger than the original design by 33%).
- Additional P-delta loads from the stronger roof generates slight reduction in wall capacity.

3.1.2 Limitations of Pressure-Impulse Diagrams

The Pressure-Impulse diagrams for both Substations 1 and 2 are representative of building lengths between 10 and 50m.

The reflected walls with equipment doors control the capacity of the buildings.

Only low damage response is acceptable if no stirrups are provided in the wall sections. Medium damage response is acceptable if a suitable stirrup arrangement is included within the wall sections. No high damage response is provided since the walls are load bearing and would incur very high P-delta forces, which would induce additional loading on the connections.

3.2 Simplification of Connections

To avoid spending undue design time on all the connections it was initially decided to perform a review of all the connections to determine which connections were suitable for a bolted connection, and the number of each connection per building. This study was only performed for Substation 1 (47.5m by 10m wide), the results of which are shown in Table 1.

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As can be seen in Table 1, it was decided to concentrate design efforts on the following types of connection;

- Wall Panel to Wall Panel Connection (Short Wall)
- Wall Panel to Wall Panel Connection (Long Wall)
- Wall Panel to Foundation Connection
- Roof Panel to Roof Panel Connection
- Roof Panel (Tee-section) to Wall Panel Connection (Short Wall)
- Roof Panel (Flange of Tee-section) to Wall Panel Connection (Long Wall)
- Roof Panel (Web of Tee-section) to Wall Panel Connection (Long Wall)

The bolted connections have been designed in accordance with the requirements of ACI 318 (Reference 6). The applied loading to the connections has been taken from the original Saudconsult welded connection design calculations; no further analysis has been performed.

The bolted connections have been designed to minimise modification from the original design, yet produce effective and functional connections. High Strength Friction Grip (HSFG) bolts have been adopted such that slotted holes can be used to suit construction tolerances. It is highlighted that all embedded rods shall be located within the reinforcement such that edge bursting is precluded.

Sketches of the proposed bolted connections for these seven types of connection are shown in Figures 5 to 11. The justification calculations for these bolted connections are included within Appendix B of this calculation.

3.3 Foundation Design Loads

The maximum loading on the foundation for both buildings has been extracted from Saudconsult and Qanbar Dywidag Precast Concrete Ltd design calculations.

It is highlighted that the static dead plus blast reaction loads for the SS#2 foundation is 787kN/m. This equates to almost 1600kN/m² for a 0.5m wide foundation. As such it is uneconomical to design the foundations statically for the blast loading. Instead the foundations have been checked using an energy balance method, which has shown the performance of both the SS#1 and SS#2 foundations to respond adequately.

The building design shall therefore adopt the foundation loads provided in Table 2, which are based on the Dead plus Live plus Wind load case. This predicts that the foundations allowable bearing capacity must exceed 130kN/m (260kN/m² assuming 0.5m wide wall foundation). However, in order use these loads it is essential that the following criteria is achieved, to be consistent with the energy balance calculation;

- The wall foundation must have a depth of at least 1.0m and a width not less than 0.5m.
- The ground slab and wall foundations must be dowelled together in order to obtain load transfer.

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- The ground slab shall be a single membrane over the full area of the building, with a edge thickness not less than 450mm.

3.4 Cast-In Inserts for Support of Services

To provide a suitable anchorage point for building services, it is proposed to use a Halfen channel cast-in-place within the underside of each of the roof double tee-sections, continuous along the centreline. This detail is shown in Figure 12.

Given the width of the double tee-sections at the underside of the web (143mm), the choice of Halfen channels is limited. This choice is further limited by the need for an insert suitable for dynamic loading. It is therefore proposed to use a Halfen HTA 40/22 channel (with embedded anchors at 250mm centres), and use a Halfen M16 T-head bolts. This type of Halfen channel is “hot rolled”, and suitable for dynamic loads.

The maximum loading on this type of channel, from the design information, for a single anchorage point is 8kN, with a minimum spacing between anchorage points of 250mm.

To allow for blast loading, it is considered appropriate to reduce this allowable load by a factor of approximately three. Therefore it is recommended that the safe working load for a single anchorage point on the cast-in-place Halfen channels is limited to 2.6kN, with a minimum spacing between anchorage points of 250mm.

4 CONCLUSIONS AND RECOMMENDATIONS

This calculation has presented the further work carried out by ABS Consulting to address the client comments received from Saudi Aramco (Reference 3) on the reports of the design of the 47.5m by 10m wide and 40m by 15m wide blast resistant precast concrete substations (References 1 and 2).

Low and medium response curves, with a suitable stirrup arrangements in the walls, have been developed for both Substation 1 and 2, and are presented in Figures 1 to 4.

Alternative bolted connections, to the original welded connections, for the following connections are presented within this calculation;

- Wall Panel to Wall Panel Connection (Short Wall) – See Figure 5
- Wall Panel to Wall Panel Connection (Long Wall) – See Figure 6
- Wall Panel to Foundation Connection – See Figure 7
- Roof Panel to Roof Panel Connection – See Figure 8
- Roof Panel (Tee-section) to Wall Panel Connection (Short Wall) – See Figure 9
- Roof Panel (Flange of Tee-section) to Wall Panel Connection (Long Wall) – See Figure 10

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- Roof Panel (Web of Tee-section) to Wall Panel Connection (Long Wall) – See Figure 11

These revised connection details reduce the level of site connection welding by approximately 95%.

The foundation design loads have been extracted from the Saudconsult and Qanbar Dywidag Precast Concrete Ltd calculations and are included in Table 2 of this calculation.

A revised services support insert detail is also shown in Figure 12. This has a capacity of 2.6kN for bolts spaced at a minimum of 250mm centres.

5 REFERENCES

1. ABS Consulting Report No. 1282017-R-01, 10m Blast Resistant Modular Precast Concrete Substation, Issue 1, October 2004.
2. ABS Consulting Report No. 1282017-R-02, 15m Blast Resistant Modular Precast Concrete Substation, Issue 1, October 2004.
3. E-mail from Perry Catmunan (Al-Osais) to Alistair Moat (ABS Consulting), dated 21/11/04.
4. ABS Proposal 2004/1175-201/AM, Additional Blast Resistant Substation Design Work, Issue 1, January 2005.
5. E-mail from Chuck Baldwin (Saudi Aramco) to Patrick Mann (ABS Consulting), dated 26/01/05, ABS Consulting Reference 1336545-I-004.
6. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-02) and Commentary (ACI 318R-02)", American Concrete Institute, Detroit, MI, 2002.

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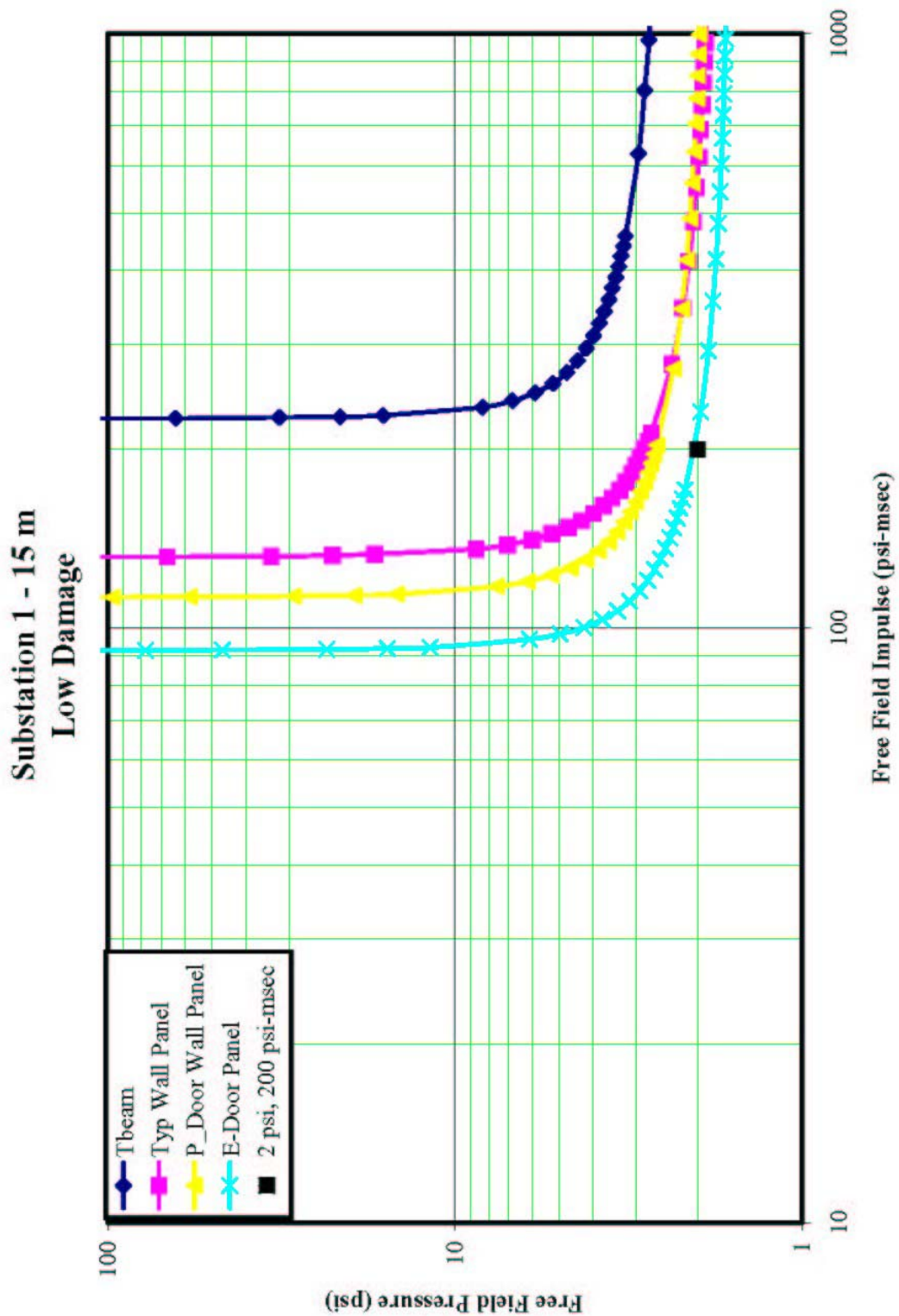


Figure 1 – Low Response Curve for Substation 1 (15m wide)

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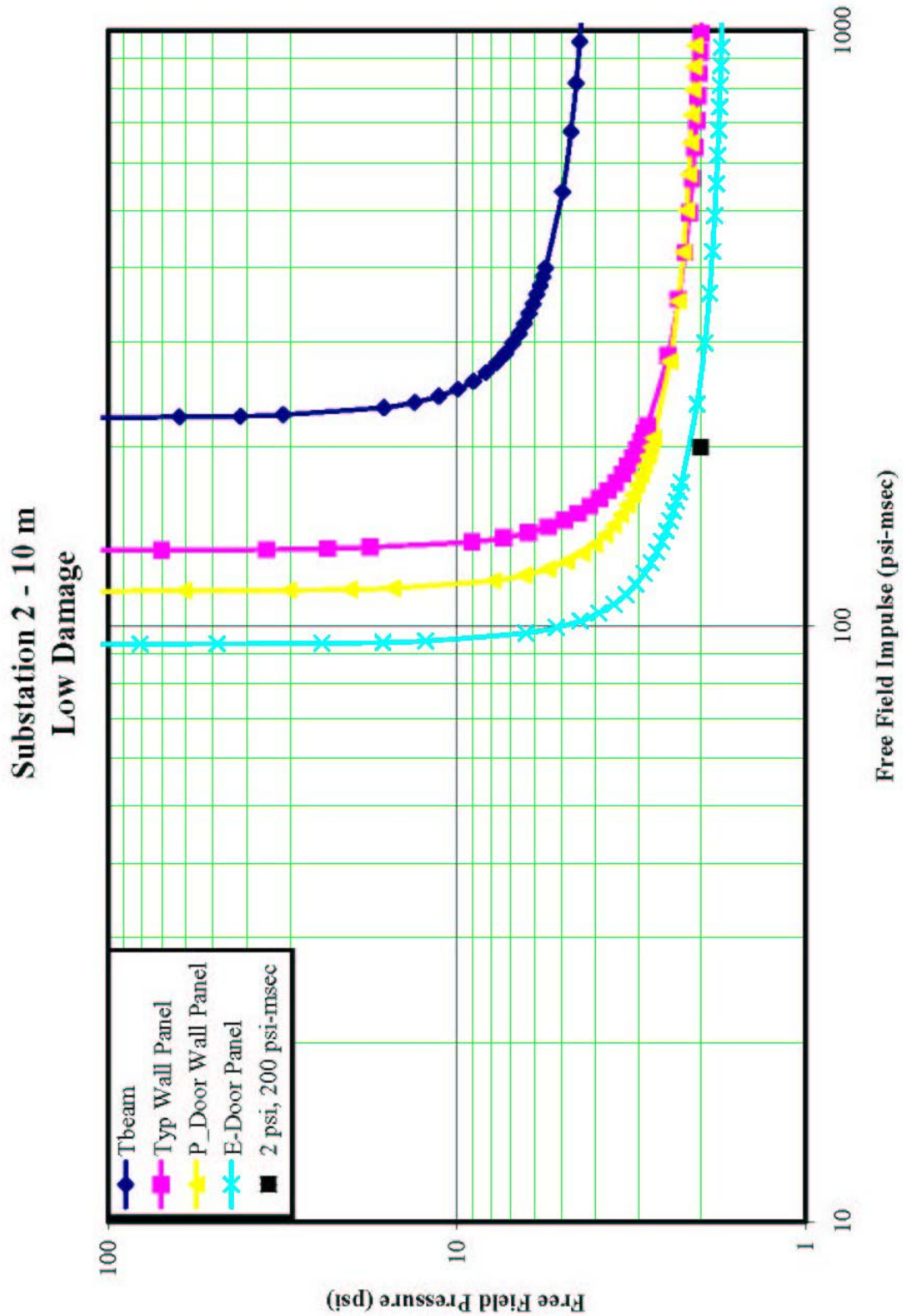


Figure 2 – Low Response Curve for the Substation 2 (10m wide)

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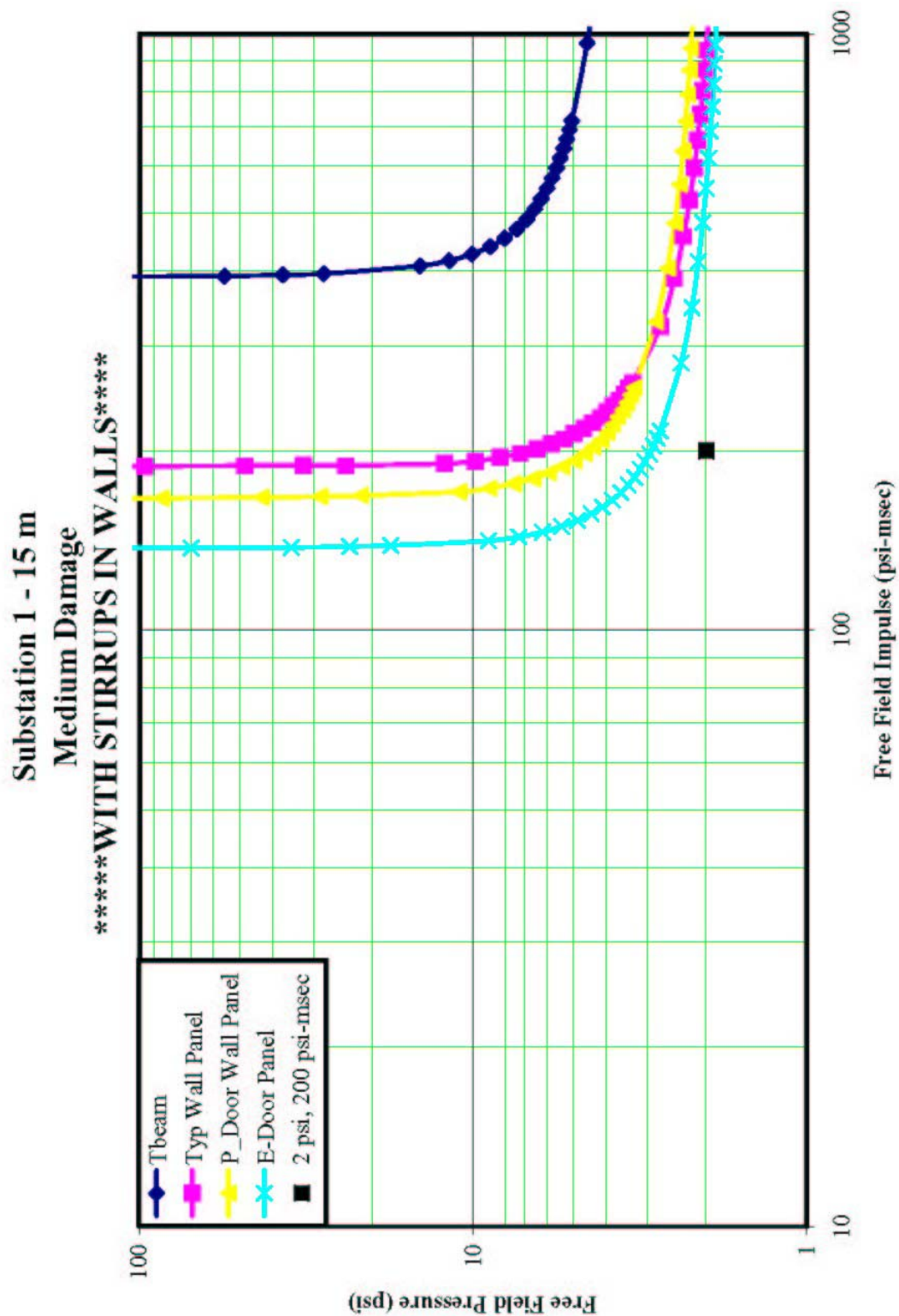


Figure 3 – Medium Response Curve for Substation 1 (15m wide) (with Stirrups in Wall Panels)

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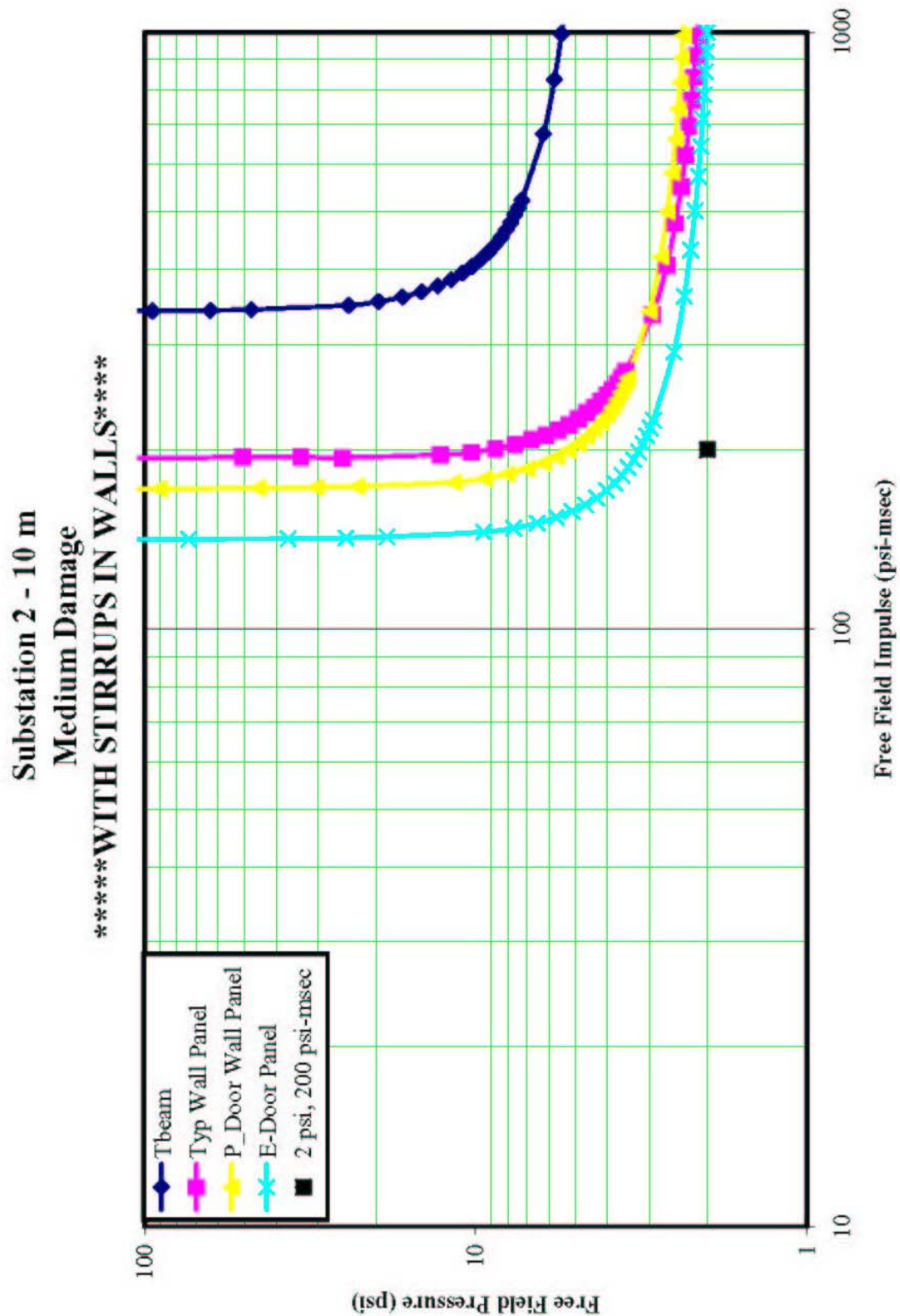


Figure 4 – Medium Response Curve for Substation 2 (10m wide) (with Stirrups in Wall Panels)

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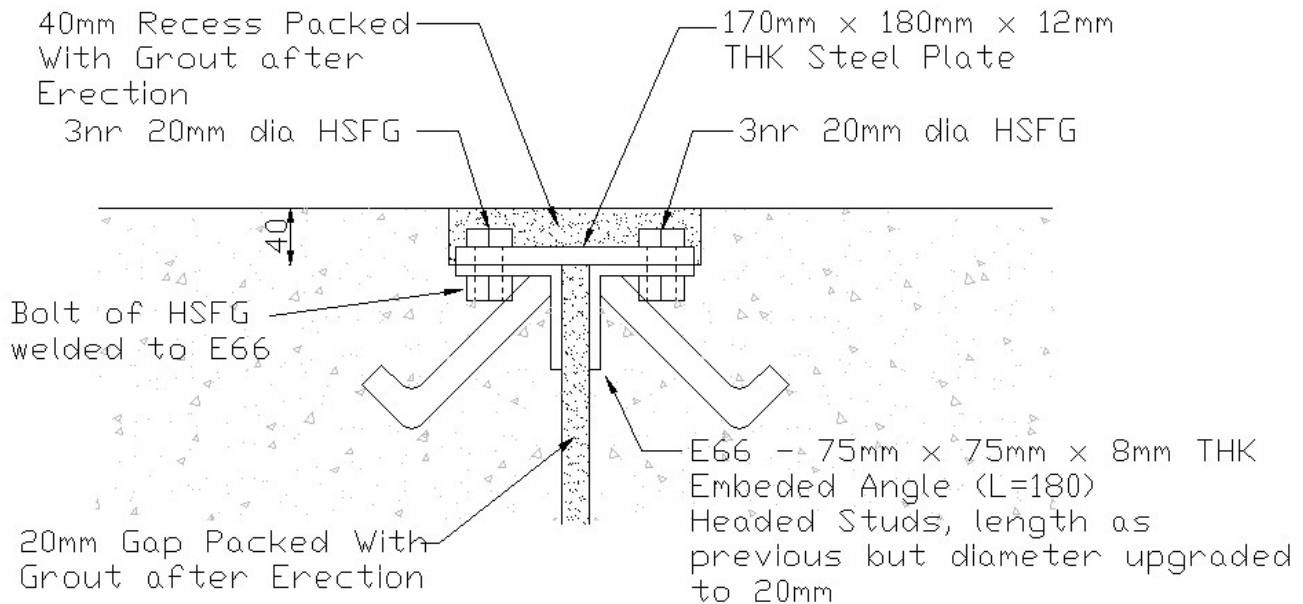


Figure 5 – Sketch of Proposed Wall Panel to Wall Panel Bolted Connection (Short Wall)

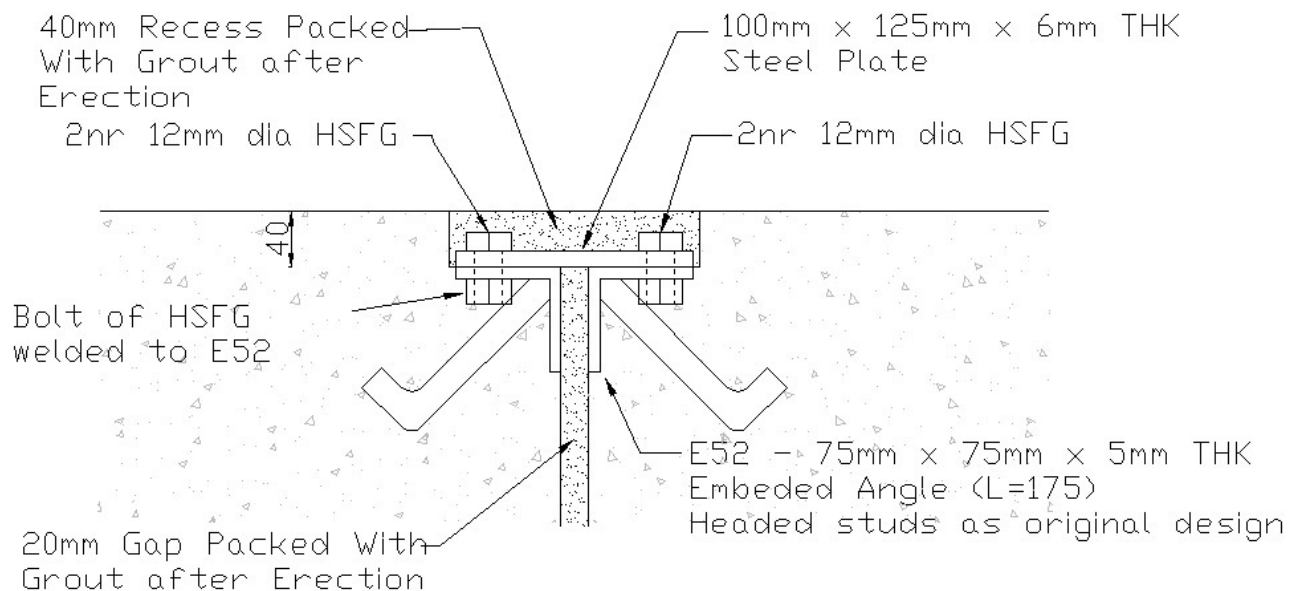


Figure 6 – Sketch of Proposed Wall Panel to Wall Panel Bolted Connection (Long Wall)

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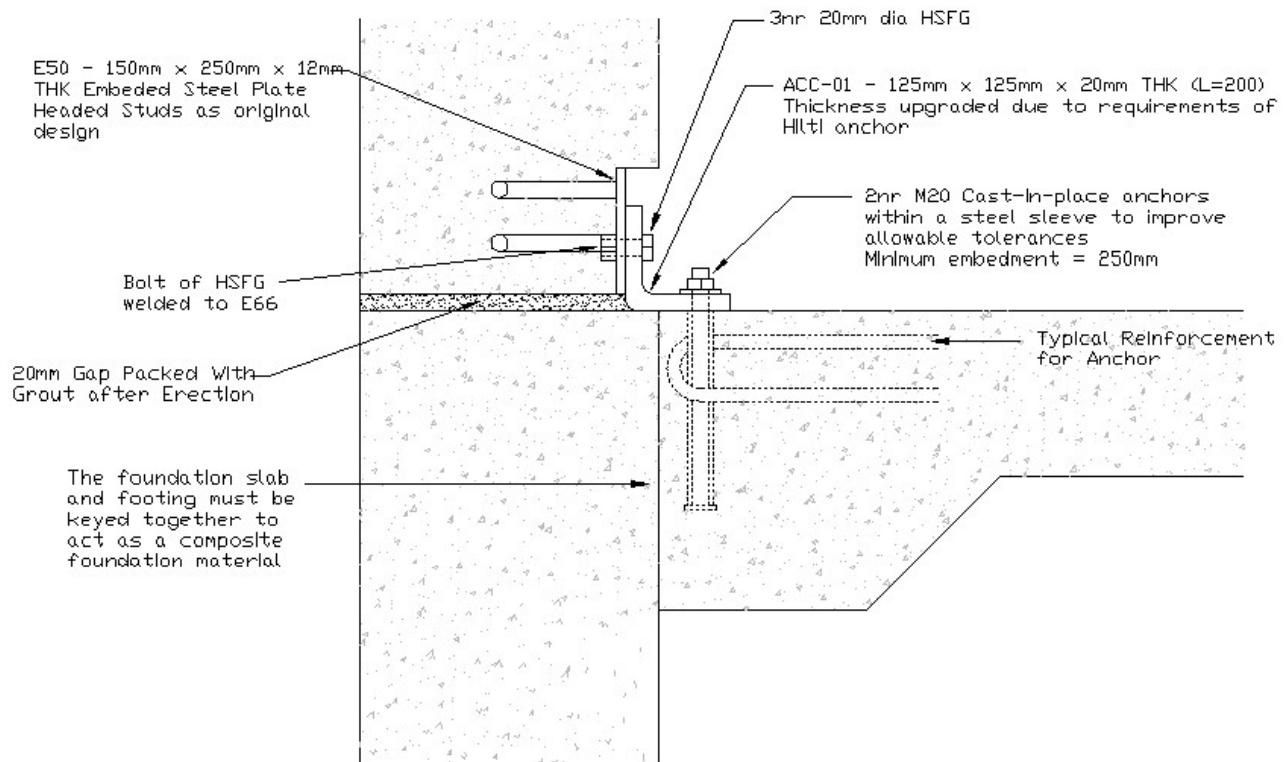


Figure 7 – Sketch of Proposed Wall Panel to Foundation Bolted Connection

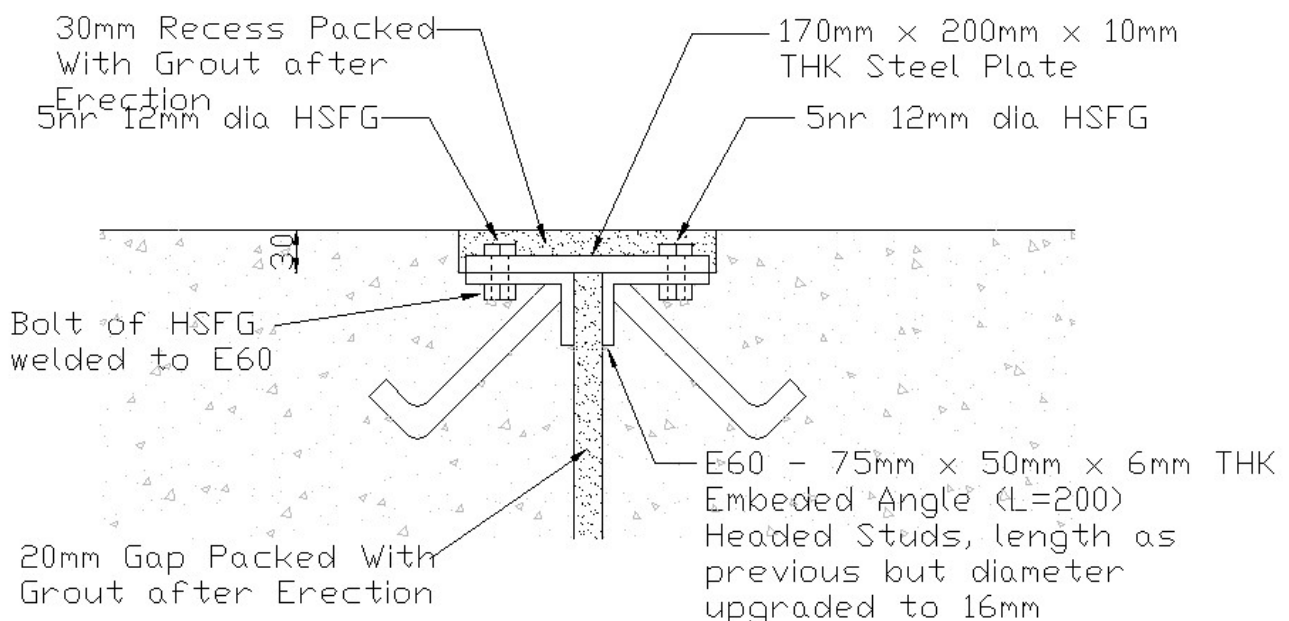


Figure 8 – Sketch of Proposed Roof Panel to Roof Panel Bolted Connection

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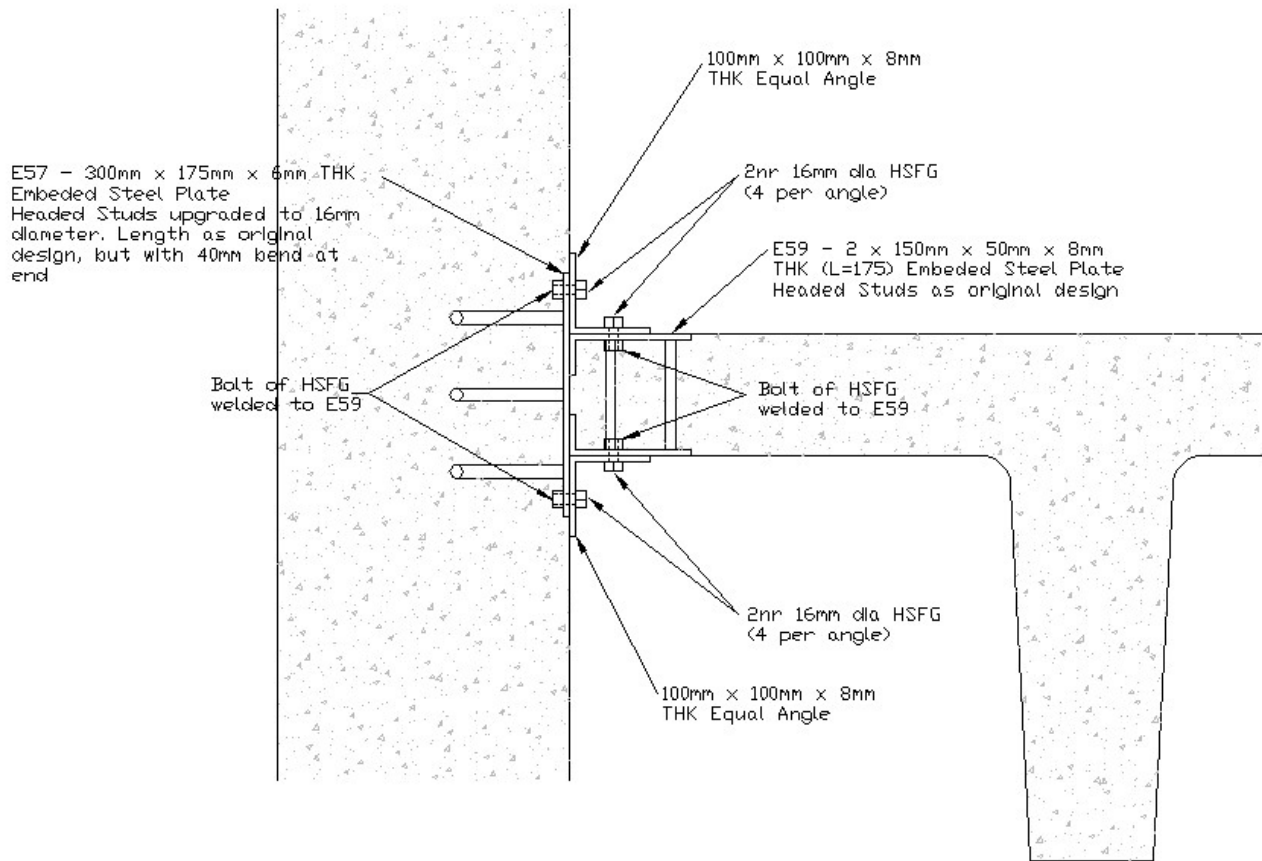


Figure 9 – Sketch of Proposed Roof Panel (Tee-section) to Wall Panel Connection (Short Wall)

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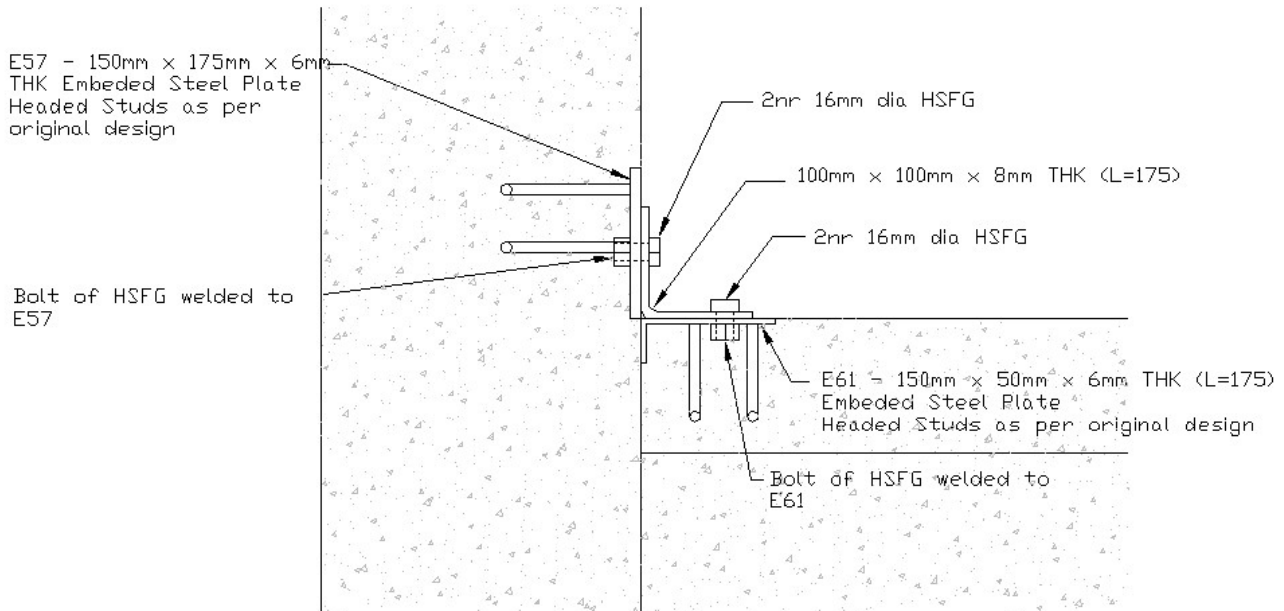


Figure 10 – Sketch of Proposed Roof Panel (Flange of Tee-section) to Wall Panel Connection (Long Wall)

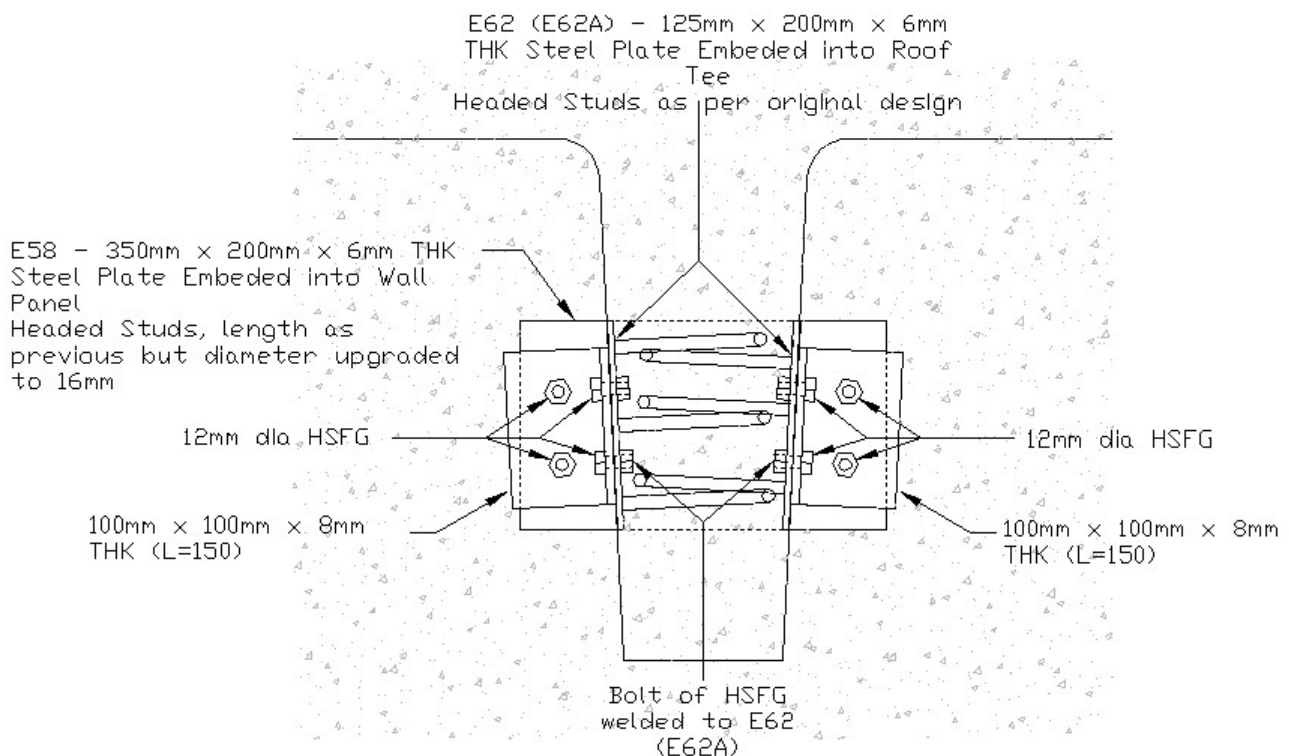


Figure 11 – Sketch of Proposed Roof Panel (Web of Tee-section) to Wall Panel Connection (Long Wall)

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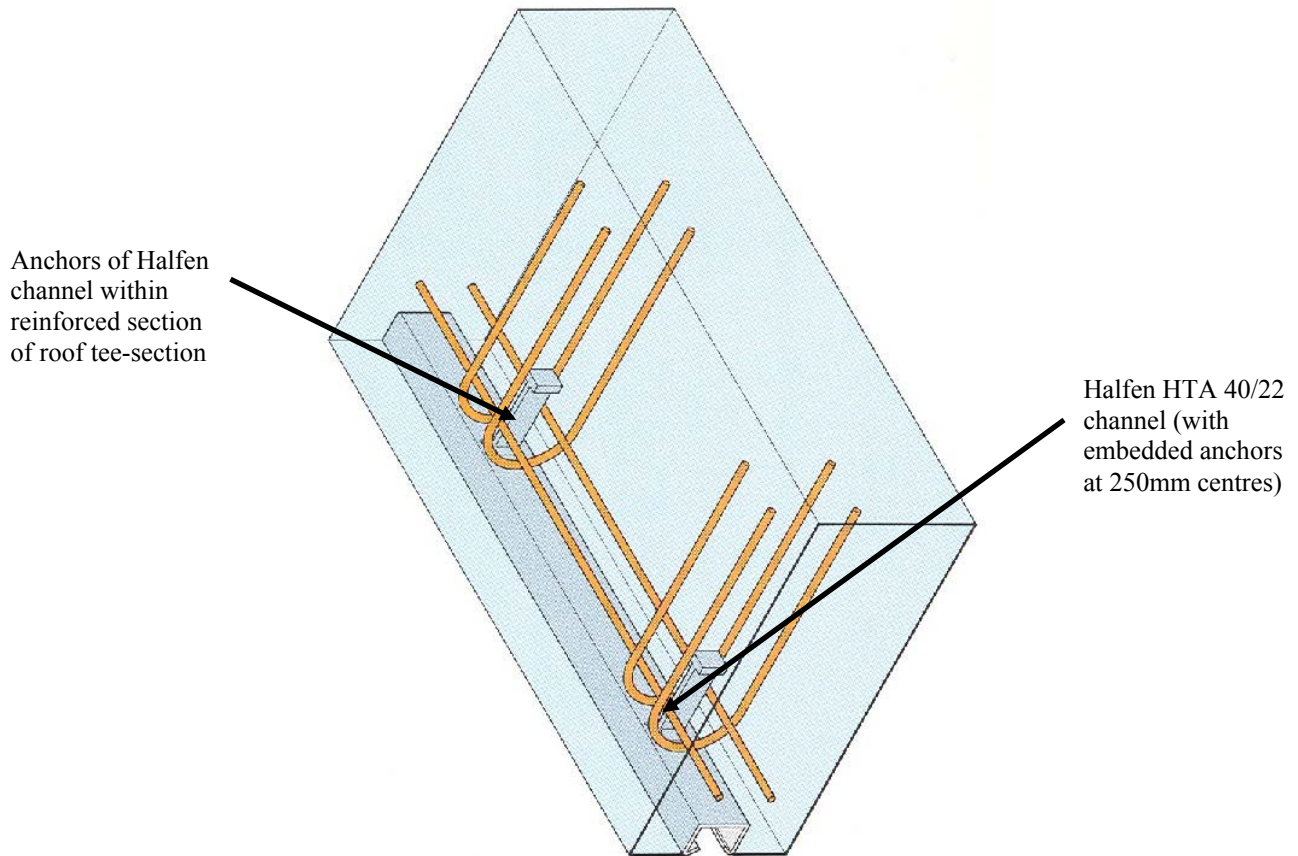


Figure 12 – Typical Halfen Channel Cast-In-Place Detail

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Connection Location	Existing Connection Description	Total Number of Connections	Comments
Wall Panel to Wall Panel (Long Wall)	This is a flat steel plate welded to cast-in-place angle inserts at the outer edge of the wall panels. This is used to: - · Ensure a rigid shear wall action by transferring in-plane shear forces between the wall panels	504	An alternative bolted connection will be examined. See Figure 6.
Wall Panel to Wall Panel (Short Wall)	This is a flat steel plate welded to cast-in-place angle inserts at the outer edge of the wall panels. This is used to: - · Ensure a rigid shear wall action by transferring in-plane shear forces between the wall panels	84	An alternative bolted connection will be examined. See Figure 5.
Roof Panel Flange Detail to Wall Panel	This is a flat steel plate welded to cast-in-place inserts at the top flange at the end of the roof panels and a cast-in-place insert at the upper level of the wall panels. This is used to: · Transfer out of plane reaction forces at the top of the wall panels into the roof diaphragm · Transfer loads from the roof diaphragm into the end shear wall panels	190	An alternative bolted connection will be examined. See Figure 10.
Roof Panel Web Detail to Wall Panel	This is a flat steel plate welded to cast-in-place inserts at the web of the roof panels tee beams and cast-in-place inserts at the upper level of the wall panels. This is used to: - · Transfer out of plane reaction forces from the roof panel into the load bearing wall panel.	152	An alternative bolted connection will be examined. See Figure 11
End Roof Panel to Wall Panel	These are 2 flat steel plates welded to cast-in-place inserts at the top and bottom flange at the side of the roof panels and cast-in-place inserts at the upper level of the end wall panels. This is used to: - · Transfer out of plane reaction forces from the wall panel back into the roof diaphragm. · Transfer loads from the roof diaphragm into the end shear wall panels	40	An alternative bolted connection will be examined, since the detail is not dissimilar to the Roof Panel Flange Detail to Wall Panel. See Figure 9.
Wall Panel to Foundation (Long Wall)	This is an angle section welded to cast-in-place inserts at the bottom of the wall panels and cast-in-place inserts at the foundation. This is used to: - · Transfer out of plane reaction forces from the wall panel into the foundation. · Transfer in-plane shear wall forces to the foundation · Transfer vertical uplift forces to the foundation	81	An alternative bolted connection will be examined. The Qanbar Dywidag Precast Concrete Ltd connection calculations do not contain design calculations for this connection. However, the loading is substantially lower than the long wall foundation connection, therefore this detail will be adopted for both connections. See Figure 7.

Table 1 – Review of Existing Welded Connections (15m wide Substation) (Sheet 1 of 2)

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Connection Location	Connection Description	Total Number of Connections	Comments
Wall Panel to Foundation (Short Wall)	These are two angle sections welded to cast-in-place inserts at the bottom of the wall panels and cast-in-place inserts at the foundation. This is used to: - · Transfer out of plane reaction forces from the wall panel into the foundation. · Transfer in-plane shear wall forces to the foundation · Transfer vertical uplift forces to the foundation	28	An alternative bolted connection will be examined, since the detail is not dissimilar to the long wall foundation connection. See Figure 7.
Roof Panel to Roof Panel Connection	This is a flat steel plate welded to cast-in-place inserts at the top flange down the side of the roof panels. This is used to: - · Ensure a rigid roof diaphragm by transferring in-plane shear forces between the roof panels	234	An alternative bolted connection will be examined. See Figure 8.
Wall Panel Corner Connection	This is a steel angle welded to cast-in-place angle inserts at the inner corner of the main wall panels. This is used to: - · Ensure a rigid shear connection between the long and short walls at their intersection	56	No alternative bolted connection will be examined. Connection not considered to lead to an excessive amount of welding.

Table 1 – Review of Existing Welded Connections (15m wide Substation) (Sheet 2 of 2)

Substation		Maximum Vertical Loading	Maximum Anchor Loads
SS1	40m long x 15m wide	130kN/m	65kN
SS2	47.5m long x 10m wide	130kN/m	65kN

Table 2 – Foundation Design Loads

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APPENDIX A

Full Justification of Pressure-Impulse Diagrams

Concrete T-Beam SDOF Properties

Substation 2 - Application

$L := 32.8 \cdot \text{ft}$	Span Length
$b := 41.8 \cdot \text{in}$	Actual Width
$b_{\text{eff}} := 36.4 \cdot \text{in}$	Effective Flange Width
$t_{\text{flange}} := 5.9 \cdot \text{in}$	Thickness of the flange (slab)
$b_1 := 47.2 \cdot \text{in}$	Stem Spacing
$b_{\text{stem}} := 6.02 \cdot \text{in}$	Stem width
$H := 25.6 \cdot \text{in}$	Stem Depth
$w_2 := 5 \cdot \text{psf}$	Weight of Supported Structure
$f_c := 7500 \cdot \text{psi}$	Compressive Strength of Concrete
$f_y := 60000 \cdot \text{psi}$	Yield Strength of Standard Reinforcing
$A_{s_t} := 4 \cdot 0.761 \cdot \text{in}^2$	Area of Reinforcing Steel
$A_{s_reb} := 0.974 \cdot \text{in}^2$	
$d := 22.44 \cdot \text{in}$	Depth to positive reinforcing
$d_{\text{reb}} := 23.5 \cdot \text{in}$	Depth to rebound reinforcing
$I_g := 17657 \cdot \text{in}^4$	Gross Moment of Inertia of Beam from ShapeBuilder
$E := 57000 \cdot \sqrt{f_c} \cdot \text{psi}$	Modulus of Elasticity of Concrete
$E = 4.936 \times 10^6 \text{ psi}$	

$$\rho_{\text{conc}} := 150 \cdot \frac{\text{lb}}{\text{ft}^3}$$

Calculate reinforcing ratios:

$$\rho := \frac{A_{s_t}}{b \cdot d} \quad \rho = 0.00325$$

$$\rho_{\text{reb}} := \frac{A_{s_reb}}{b_{\text{stem}} \cdot d_{\text{reb}}} \quad \rho_{\text{reb}} = 0.00688$$

Calculated Parameters:

Determine the equivalent whitney stress block

$$a := \frac{(A_{s_t} \cdot f_y)}{0.85 \cdot f_c \cdot b} \quad a = 0.69 \text{ in}$$

Determine the Moment Capacity of the Member:

$$M_p := A_{s_t} \cdot f_y \cdot \left(d - \frac{a}{2} \right)$$

$$M_p = 4.0359 \times 10^6 \text{ in} \cdot \text{lb} \quad M_p = 4.56 \times 10^5 \text{ N} \cdot \text{m}$$

Effective Moment of Inertia

$$I_{\text{eff}} := \frac{I_g}{2} \quad I_{\text{eff}} = 8829 \text{ in}^4$$

Weight of the Member and Supported Structure

$$W_{\text{stem}} := b_{\text{stem}} \cdot H \cdot L \cdot \rho_{\text{conc}} \quad W_{\text{stem}} = 5265.49 \text{ lb}$$

$$W_{\text{flange}} := b \cdot t_{\text{flange}} \cdot L \cdot \rho_{\text{conc}} \quad W_{\text{flange}} = 2.71 \times 10^5 \text{ lb ft sec}^{-2}$$

Total

$$W := W_{\text{stem}} + W_{\text{flange}} + w_2 \cdot b \cdot L$$

$$W = 14262.94 \text{ lbf}$$

SDOF Properties of the Double Tee System for Design**Mass:**

$$\text{Mass} := \frac{W}{g \cdot b \cdot L}$$

$$\text{Mass} = 2245.385 \frac{\text{psi} \cdot \text{msec}^2}{\text{in}}$$

**Load Mass Factors per ASCE Design Manual
For Simply supported member**

$$k_{lme} = 0.78$$

$$k_{lmp} = 0.66$$

Stiffness:

$$k_e := \frac{384 \cdot E \cdot I_{\text{eff}}}{5 \cdot b \cdot L^4}$$

$$k_e = 3.34 \frac{\text{psi}}{\text{in}}$$

$$k_p := 0$$

Resistance:

$$r_u := 8 \cdot \frac{M_p}{b \cdot L^2}$$

$$r_u = 4.99 \text{ psi}$$

Find the rebound resistance of the member

$$a_{\text{reb}} := \frac{(A_{s_reb} \cdot f_y)}{0.85 \cdot f_c \cdot b_{\text{stem}}} \quad a_{\text{reb}} = 1.52 \text{ in}$$

Determine the Moment Capacity of the Member:

$$M_{\text{reb}} := -A_{s_reb} \cdot f_y \cdot \left(d_{\text{reb}} - \frac{a_{\text{reb}}}{2} \right)$$
$$M_{\text{reb}} = -1.3288 \times 10^6 \text{ in} \cdot \text{lbf} \quad M_{\text{reb}} = -1.5 \times 10^5 \text{ N} \cdot \text{m}$$

Rebound Resistance:

$$r_{\text{reb}} := 8 \cdot \frac{M_{\text{reb}}}{b \cdot L^2} \quad r_{\text{reb}} = -1.64 \text{ psi}$$

Determine the initial deflection based on the initial loads:

$$\text{load}_1 := \frac{W}{b \cdot L} \quad \text{load}_1 = 0.87 \text{ psi} \quad \textbf{Gravity Loads}$$

$$\text{load}_2 := 0 \cdot \text{psf} \quad \text{load}_2 = 0 \text{ psi} \quad \textbf{Other loads}$$

$$\text{load}_{\text{init}} := \text{load}_1 + \text{load}_2 \quad \text{load}_{\text{init}} = 0.87 \text{ psi}$$

$$X_1 := \frac{\text{load}_{\text{init}}}{k_e} \quad X_1 = 0.26 \text{ in}$$

Blast Loads as Specified by the client:

Side-on loads:

$$p_s := 2 \cdot \text{psi}$$

$$i := 200 \cdot \text{psi} \cdot \text{msec}$$

$$t_d := \frac{2 \cdot i}{p_s} \qquad t_d = 200 \text{ msec}$$

Total net loading on the member including initial loads:

$$p_1 := p_s + \text{load}_{\text{init}}$$

$$t_1 := 0 \cdot \text{msec}$$

$$p_2 := \text{load}_{\text{init}}$$

$$t_2 := t_d$$

$$p_3 := \text{load}_{\text{init}}$$

$$t_3 := t_d + 1000 \cdot \text{msec}$$

$$p_1 = 2.87 \text{ psi}$$

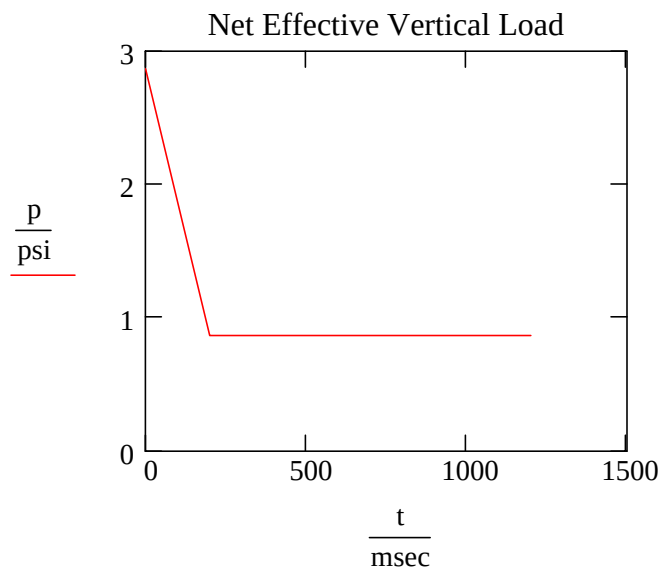
$$t_1 = 0 \text{ msec}$$

$$p_2 = 0.87 \text{ psi}$$

$$t_2 = 200 \text{ msec}$$

$$p_3 = 0.87 \text{ psi}$$

$$t_3 = 1200 \text{ msec}$$



Determine the maximum dynamic shear load on the member:

Per SAES M-009 v_{max} must be greater than 120% the flexural capacity.

$$V_{\max} := \frac{r_u \cdot b \cdot L}{2} \cdot 120\%$$

$$V_{\max} = 49.22 \text{ kip}$$

$$V_{\max} = 2.19 \times 10^5 \text{ N}$$

Check shear@ d from support

$$V_d := \frac{V_{\max} \cdot \left(\frac{L}{2} - d \right)}{\left(\frac{L}{2} \right)}$$

$$V_d = 43.61 \text{ kip}$$

Check the concrete stem shear capacity

$$V_c := 2 \cdot \sqrt{f_c \cdot \psi_i} \cdot b_{\text{stem}} \cdot d$$

$$V_c = 23.4 \text{ kip}$$

$$\text{if}(V_c > V_d, \text{"OK"}, \text{"Stirrups Required"}) = \text{"Stirrups Required"}$$

Determine the stirrups requirements: (Design V_{max} instead of V_d for conservatism)

Load to be carried by the stirrups

$$V_{s_req} := V_{\max} - V_c$$

$$V_{s_req} = 25.82 \text{ kip}$$

Assuming No. 3 stirrups, find the spacing required to meet the shear demand:

$$s_{\max} := \min\left(\frac{d}{2}, 24 \cdot \text{in}\right) \quad s_{\max} = 11.22 \text{ in}$$

$$A_v := 2 \cdot (0.12 \cdot \text{in}^2) \quad A_v = 0.24 \text{ in}^2$$

$$s := \frac{A_v \cdot f_y \cdot d}{V_{s_req}} \quad s = 12.52 \text{ in}$$

spacing must not exceed s

Use single loop No. 3 stirrups @ 11.8 inches on center horizontally in the beam

Maximum spacing for this member is 12 in.

Minimum area of steel is as follows:

$$s := 8.18 \cdot \text{in} \quad \text{actual stirrup/bar spacing used}$$

$$A_{v_min} := \frac{(50 \cdot \text{psi}) \cdot b_{\text{stem}} \cdot s}{f_y} \quad A_{v_min} = 0.04 \text{ in}^2$$

Shear Reinf Capacity Supplied

$$V_{s_supplied} := \frac{A_v \cdot f_y \cdot d}{s} \quad V_{s_supplied} = 39.5 \text{ kip}$$

$$\text{if}(V_{s_supplied} > V_{s_req}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$$

$$V_{s_check} := 4 \cdot \sqrt{f_c \cdot \text{psi}} \cdot b_{\text{stem}} \cdot d \quad V_{s_check} = 46.8 \text{ kip}$$

d/2 spacing is OK

shear must be 120% higher capacity than flexural capacity in rebound

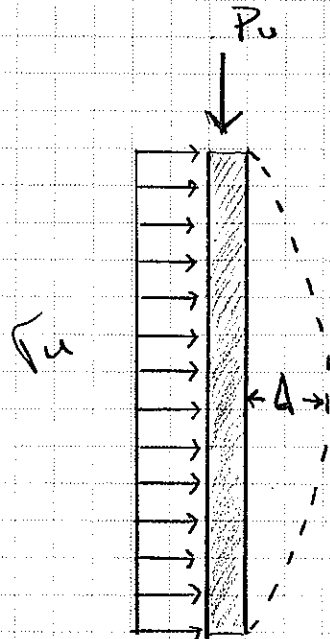
$$v_{\text{reb}} := \frac{r_{\text{reb}} \cdot b \cdot L \cdot 1.2}{2} \quad v_{\text{reb}} = -16.21 \text{ kip}$$
$$v_{\text{reb}} = -72083.74 \text{ N}$$

$$\theta_{\text{low}} := 1 \cdot \text{deg} \quad \theta_{\text{med}} := 2 \cdot \text{deg} \quad \theta_{\text{high}} := 4 \cdot \text{deg}$$

$$x_{\text{low}} := \frac{L}{2} \cdot \tan(\theta_{\text{low}}) \quad x_{\text{low}} = 3.44 \text{ in}$$

$$x_{\text{med}} := \frac{L}{2} \cdot \tan(\theta_{\text{med}}) \quad x_{\text{med}} = 6.87 \text{ in}$$

$$x_{\text{high}} := \frac{L}{2} \cdot \tan(\theta_{\text{high}}) \quad x_{\text{high}} = 13.76 \text{ in}$$

JOB NO. _____ JOB SAUDI ARABIAN STANDARD BUILDING BY BFIH DATE _____
 CALC. NO. _____ SUBJECT ABU ALI LBW A LIMIT CHK'D _____ DATE _____

 FROM CIBAD USER
 GUIDE:

M_u : Ultimate Moment Capacity
 M_a : Allowable Moment Capacity
 P_u : Ultimate Axial Capacity
 P_e : Axial Load

$$\frac{M_a}{M_u} = 1 - \frac{P_e}{P_u}$$

$$M_a = e P_u$$

$$\therefore e = M_u \left(\frac{1}{P_e} - \frac{1}{P_u} \right)$$

$$P_u = 0.85 f_c' (A_g - A_{st}) + f_y A_{st}$$

JOB NO. _____ JOB SAUDI ARAMCO STANDARD BUILDING BY BFT DATE _____
 CALC. NO. _____ SUBJECT ABU ALI LBW A LIMIT CHK'D _____ DATE _____

IN GENERAL, THE PRECAST WALLS CONTAIN

 $\overset{0.984\text{ in}}{\#25\phi \text{ Bar @ } 300\text{ mm c-to-c e.f.}}$

$$A_s = 2 \cdot \frac{\pi}{4} (0.984\text{ in})^2 \cdot \frac{12\text{ in}}{\text{ft}} \cdot \frac{1}{11.8\text{ in}}$$

 WHERE $25\text{ mm} = 0.984\text{ in}$
 $300\text{ mm} = 11.8\text{ in}$

$$A_s = 1.52\text{ in}^2 \cdot \frac{12\text{ in}}{\text{ft}} \cdot \frac{1}{11.8\text{ in}}$$

$$A_s = 1.547\text{ in}^2/\text{ft}$$

$$A_g = 9.84\text{ in} = 250\text{ mm} \cdot 12\text{ in}$$

$$A_g = 11.808\text{ in}^2$$

$$f_c = 5,000\text{ psi}$$

$$\alpha = 1.0$$

$$C = 1.12 \quad \text{--- Compression}$$

$$P_u = 0.85(f_{ck})(A_g - A_{st}) + f_{dy} A_{st}$$

$$= 0.85(5600)(11.808 - 1.547) + 72600(1.547)$$

$$P_u = 161.2\text{ k}/\text{ft}$$

$$f_{dy} = 5600\text{ psi}$$

$$f_{dy} = 60,000\text{ psi}$$

$$\alpha = 1.1$$

$$C = 1.1 \quad \text{--- Compression}$$

$$f_{dy} = 72,600\text{ psi}$$

FROM EXCEL

$$M_u = 465.6\text{ kip}\cdot\text{in}/\text{ft}$$

$$P_u = 16.8\text{ kip}/\text{ft}$$

$$e_{\max} = 465.6\text{ kip}\cdot\text{in}/\text{ft} \cdot \left(\frac{\text{ft}}{16.8\text{ kip}} - \frac{\text{ft}}{161.2\text{ k}} \right)$$

$$\underline{e_{\max} = 24.8\text{ in}}$$

CHECK

$$\frac{P_u}{P_n} + \frac{M_u}{M_n} \leq 1$$

$$\frac{16.8}{161.2} + \frac{(16.8)(24.8)}{465.6} = 1.0 \quad \checkmark$$

X_{nigh} (8deg) < 24 in, OK ✓



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Bar spacing adjusted for equivalent steel area

Component:

Global Input Parameters		
L	256	Span (in)
P-P		Boundary condition
Uniform		Loading condition
Tributary Dead Load Computation:		
Deck	30	psf
Insulation	0	psf
Other	0	psf
Total tributary wt.	153	psf (including beam weight)
100%	153	psf (portion to include as mass)
Associated mass	0	psf (not included in weight)
Tot. additional mass	153	psf
Axial Load Calculations		
Yes		Include P-Delta?
P	14400	lbs (total applied axial force)
Delta	0	in (eccentricity used for p-delta calcs.)
P-Delta	0	in-lbs (reduction in moment for p-delta)

Calculate Structural Properties		
	NonLoad	Load Side
β_1	0.80	0.80 factor in ACI, 10.2.7.3
A_s	0.77	0.77 area of steel per foot (in ²)
a (depth of stress blk)	0.9602	0.9602 ($A_s f_{dy}) / (b(0.85 f_{dc}))$
M_{nom} (nom moment)	465654	465654 $0.85 f_{dc} ab(d-a/2)$ (lb-in)
M_{net} (net moment)	465654	465654 (including P-delta) (lb-in)
V_n	154	diagonal tension shear capacity (psi)
Mass	2749.74	total mass (psi-ms ² /in)
Calculate SDOF Properties		
r_{ult}	4.74	Input Ultimate resistance (psi)
K_e	3.61	Effective stiffness (psi/in)
K_{LM}	0.72	Average load-mass factor ^{A, B and C}
M_e	1980	Equivalent mass = $M * K_{LM}$
T_n	147.1	Nat. period (ms) = $2\pi * (M_e / K_e)^{1/2}$
X_{elst}	1.31	Max deflection from dead load (in)
Rxn Coeff.	R	F Dyn rxn factors ^{A and B}
	0.385	0.115

Specify Member Parameters		
Concrete Properties		
t_{conc}	9.84	Component thickness (in)
Wt_c	150	Concrete weight (pcf)
f'_c	5000	concrete strength (psi)
SIF	1.00	Avg. increase (Table 5.A.1 ^A)
DIF ₁	1.19	Dynamic Increase - Bending (Table 5.A.3 ^A)
DIF ₂	1.10	Dynamic Increase - Shear (Table 5.A.3 ^A)
$f'_{dc,b}$	5950	Dynamic yield - Bending (psi) = SIF * DIF * Fb
$f'_{dc,v}$	5500	Dynamic yield - Shear (psi) = SIF * DIF * Fv
w	123	element weight (psf) = $t_c * Wt_c / 12$
E_c	4030509	modulus of elasticity = $57000 (f'_c)^{1/2}$ (psi)
f_r	530.3	modulus of rupture = $7.5 (f'_c)^{1/2}$ (psi)
Reinforcing Steel Properties		
	NonLoad Side	Load side
t_{cover}	1.5	1.5 clear cover to center of steel (in)
d	8.34	8.34 depth to steel (in)
b_{steel}	12.2	12.2 reinforcement spacing (in)
8		8 select a bar size
A_{steel}	0.780	0.780 Area of steel per bar (in ²)
ρ	0.0077	0.0077 reinforcing ratio = A_s/bd
ρ_{min}	0.0018	0.0018 min. reinforcing ratio ACI, 7.12 & 10.5.3
ρ_{max}	0.0252	0.0252 $0.75 * ((0.85 \beta_1 f'_c) / F_y) * (87000 / (87000 + F_y))$
F_y	60000	yield stress (psi)
SIF	1.10	Avg. increase (Table 5.A.1 ^A)
DIF	1.17	Dynamic increase - Bending (Table 5.A.3 ^A)
F_{dy}	77220	Dynamic yield - reinforcing (psi) = SIF * DIF * Fb
Es	29000000	Young's modulus (psi)
Calculate Moment of Inertia		
I_g	968.64	gross moment of inertia (in ⁴)
n	7.20	modulus of elasticity ratio = E_s / E_c
nA_s	5.61	transformed area of steel (in ²)
c	2.35	transformed sect. neut. axis (singly reinforced sect.) = $(-nA_s + (nA_s + 2bd)^{1/2}) / b$
I_{cr}	254.14	cracked moment of inertia (singly reinforced section) = $(bc^3) / 3 + nA_s(d-c)^2$
I_a	611.39	average moment of inertia (in ⁴) = $(I_g + I_{cr}) / 2$

Response Limits		Slabs
Low		
θ_{max}	2	Max allowable rotation (degrees) (Table 5.B.2 ^A)
μ_{max}	NA	Max allowable ductility (Table 5.B.2 ^A)
X_{max}	4.47	Max allowable deflection (minimum of criteria)



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Component:

Global Input Parameters		
L	256	Span (in)
B	49	Spacing (in)
P-P		Boundary condition
Uniform		Loading condition

Tributary Dead Load Computation:		
Deck	30	psf
Insulation	0	psf
Other	0	psf
Total tributary wt.	107.1	psf (including beam weight)
100%	107.1	psf (portion to include as mass)
Associated mass	0	psf (not included in weight)
Tot. additional mass	107.1	psf

Axial Load Calculations		
No		Include P-Delta?
P	70560	lbs (total applied axial force)
Delta	0	in (eccentricity used for p-delta calcs.)
P-Delta	0	in-lbs (reduction in moment for p-delta)

Calculate Structural Properties		
	NonLoad	Load Side
β_1	0.80	0.80 factor in ACI, 10.2.7.3
A_s	3.90	3.90 area of steel (in ²)
a (depth of stress blk)	1.94	1.94 ($A_s f_{dy} / (b(0.85 f_{dc}))$)
M_{nom} (nom moment)	2219589	2219589 $0.85 f_{dc} ab(d-a/2)$ (lb-in)
M_{net} (net moment)	2219589	2219589 (including P-delta) (lb-in)
V_n	172	diagonal tension shear capacity (psi)
Mass	1924.2	total mass (psi-ms ² /in)

Calculate SDOF Properties		
r_{ult}	5.53	Input Ultimate resistance (psi)
K_e	2.58	Effective stiffness (psi/in)
K_{LM}	0.72	Average load-mass factor ^{A, B and C}
M_e	1385	Equivalent mass = $M * K_{LM}$
T_n	145.5	Nat. period (ms) = $2\pi * (M_e / K_e)^{1/2}$
X_{elst}	2.14	Max deflection from dead load (in)
Rxn Coeff.	R	E Dyn rxn factors ^{A and B}
	0.385	0.115

Specify Member Parameters		
Concrete Properties		
L_1	9.84	Component thickness (in)
L_2	30.7	Component width (in)
Wt_c	150	Concrete weight (pcf)
f'_c	5000	concrete strength (psi)
SIF	1.00	Avg. increase (Table 5.A.1 ^A)
DIF_1	1.19	Dynamic increase - Bending (Table 5.A.3 ^A)
DIF_2	1.10	Dynamic increase - Shear (Table 5.A.3 ^A)
$f'_{dc,b}$	5950	Dynamic yield - Bending (psi) = $SIF * DIF * F_b$
$f'_{dc,v}$	5500	Dynamic yield - Shear (psi) = $SIF * DIF * F_v$
w	77.1	element weight (psf) = $L_1 * L_2 * Wt_c / B$
E_c	4030509	modulus of elasticity = $57000 (f'_c)^{1/2}$ (psi)
f_r	530.3	modulus of rupture = $7.5 (f'_c)^{1/2}$ (psi)

Reinforcing Steel Properties		
NonLoad Side		Load side
t_{cover}	1.5	1.5 clear cover to center of steel (in)
d	8.34	8.34 depth to steel (in)
Bar Size	Number of Bars	Bar Size Number of Bars
8	5	8 5 select bar size and number
A_{steel}	3.90	3.90 Area of steel per bar (in ²)
p	0.0152	0.0152 reinforcing ratio = A_s / bd
p_{min}	0.0018	0.0018 min. reinforcing ratio ACI, 7.12 & 10.5.3
p_{max}	0.0252	0.0252 $0.75 * ((0.85 \beta_1 f'_c) / F_y) * (87000 / (87000 + F_y))$
F_y	60000	yield stress (psi)
SIF	1.10	Avg. increase (Table 5.A.1 ^A)
DIF	1.17	Dynamic increase - Bending (Table 5.A.3 ^A)
F_{dy}	77220	Dynamic yield - reinforcing (psi) = $SIF * DIF * F_b$
Es	29000000	Young's modulus (psi)

Calculate Moment of Inertia		
I_g	2437.49	gross moment of inertia (in ⁴)
n	7.20	modulus of elasticity ratio = E_s / E_c
nA_s	28.06	transformed area of steel (in ²)
c	3.10	transformed sect. neut. axis (singly reinforced sect.) = $(-nA_s + (nA_s^2 + 2bd))^{1/2} / b$
I_{cr}	1075.35	cracked moment of inertia (singly reinforced section) = $(bc^3) / 3 + nA_s(d-c)^2$
I_a	1756.42	average moment of inertia (in ⁴) = $(I_g + I_{cr}) / 2$

Response Limits		Slabs
θ_{max}	2	Max allowable rotation (degrees) (Table 5.B.2 ^A)
μ_{max}	NA	Max allowable ductility (Table 5.B.2 ^A)
X_{max}	4.47	Max allowable deflection (minimum of criteria)



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Component:

Global Input Parameters		
L	256	Span (in)
B	97.6	Spacing (in)
P-P		Boundary condition
Uniform		Loading condition

Tributary Dead Load Computation:		
Deck	30	psf
Insulation	0	psf
Other	0	psf
Total tributary wt.	84.1	psf (including beam weight)
100%	84.1	psf (portion to include as mass)
Associated mass	0	psf (not included in weight)
Tot. additional mass	84.1	psf

Axial Load Calculations		
No		Include P-Delta?
P		lbs (total applied axial force)
Delta		in (eccentricity used for p-delta calcs.)
P-Delta	0	in-lbs (reduction in moment for p-delta)

Calculate Structural Properties		
	NonLoad	Load Side
β_1	0.80	0.80 factor in ACI, 10.2.7.3
A_s	7.02	7.02 area of steel (in ²)
a (depth of stress blk)	2.50	2.50 ($A_s f_{dy} / (b(0.85 f_{dc}))$)
M_{nom} (nom moment)	3979315	3979315 $0.85 f_{dc} ab(d-a/2)$ (lb-in)
M_{net} (net moment)	3979315	3979315 (including P-delta) (lb-in)
V_n	182	diagonal tension shear capacity (psi)
Mass	1510.8	total mass (psi-ms ² /in)

Calculate SDOF Properties		
r_{ult}	4.98	Input Ultimate resistance (psi)
K_e	1.97	Effective stiffness (psi/in)
K_{LM}	0.72	Average load-mass factor ^{A, B and C}
M_e	1088	Equivalent mass = $M * K_{LM}$
T_n	147.8	Nat. period (ms) = $2\pi * (M_e / K_e)^{1/2}$
X_{elst}	2.53	Max deflection from dead load (in)
Rxn Coeff.	R	E Dyn rxn factors ^{A and B}
	0.385	0.115

Specify Member Parameters		
Concrete Properties		
L_1	9.84	Component thickness (in)
L_2	42.9	Component width (in)
Wt_c	150	Concrete weight (pcf)
f'_c	5000	concrete strength (psi)
SIF	1.00	Avg. increase (Table 5.A.1 ^A)
DIF_1	1.19	Dynamic increase - Bending (Table 5.A.3 ^A)
DIF_2	1.10	Dynamic increase - Shear (Table 5.A.3 ^A)
$f'_{dc,b}$	5950	Dynamic yield - Bending (psi) = SIF * DIF * Fb
$f'_{dc,v}$	5500	Dynamic yield - Shear (psi) = SIF * DIF * Fv
w	54.1	element weight (psf) = $L_1 * L_2 * Wt_c / B$
E_c	4030509	modulus of elasticity = $57000 (f'_c)^{1/2}$ (psi)
f_r	530.3	modulus of rupture = $7.5 (f'_c)^{1/2}$ (psi)

Reinforcing Steel Properties		
NonLoad Side		Load side
t_{cover}	1.25	1.25 clear cover to center of steel (in)
d	8.59	8.59 depth to steel (in)
Bar Size	Number of Bars	Bar Size Number of Bars
8	9	8 9 select bar size and number
A_{steel}	7.02	7.02 Area of steel per bar (in ²)
p	0.0190	0.0190 reinforcing ratio = A_s / bd
p_{min}	0.0018	0.0018 min. reinforcing ratio ACI, 7.12 & 10.5.3
p_{max}	0.0252	0.0252 $0.75 * ((0.85 \beta_1 f'_c) / F_y) * (87000 / (87000 + F_y))$
F_y	60000	yield stress (psi)
SIF	1.10	Avg. increase (Table 5.A.1 ^A)
DIF	1.17	Dynamic increase - Bending (Table 5.A.3 ^A)
F_{dy}	77220	Dynamic yield - reinforcing (psi) = SIF * DIF * Fb
Es	29000000	Young's modulus (psi)

Calculate Moment of Inertia		
I_g	3406.13	gross moment of inertia (in ⁴)
n	7.20	modulus of elasticity ratio = E_s / E_c
nA_s	50.51	transformed area of steel (in ²)
c	3.47	transformed sect. neut. axis (singly reinforced sect.) = $(-nA_s + (nA_s^2 + 2bd))^{1/2} / b$
I_{cr}	1921.56	cracked moment of inertia (singly reinforced section) = $(bc^3) / 3 + nA_s(d-c)^2$
I_a	2663.85	average moment of inertia (in ⁴) = $(I_g + I_{cr}) / 2$

Response Limits		Slabs
θ_{max}	2	Max allowable rotation (degrees) (Table 5.B.2 ^A)
μ_{max}	NA	Max allowable ductility (Table 5.B.2 ^A)
X_{max}	4.47	Max allowable deflection (minimum of criteria)

Roof Data

$$r_{u_roof} := 4.99 \cdot \text{psi}$$

$$L_{roof} := 9980 \cdot \text{mm}$$

$$P_{roof} := \frac{L_{roof}}{2} \cdot r_{u_roof} \qquad P_{roof} = 11.76 \frac{\text{kip}}{\text{ft}}$$

Reflected Wall

$$L := 256 \cdot \text{in}$$

$$\text{mass} := 2749.74 \cdot \frac{\text{psi} \cdot \text{msec}^2}{\text{in}}$$

$$r_u := 4.74 \cdot \text{psi}$$

$$k_e := 3.61 \cdot \frac{\text{psi}}{\text{in}}$$

$$x_{low} := \frac{L}{2} \cdot \tan(2 \cdot \text{deg}) \qquad x_{med} := \frac{L}{2} \cdot \tan(4 \cdot \text{deg}) \qquad x_{high} := \min \left(\frac{12 \cdot \text{in}}{\frac{L}{2} \cdot \tan(6 \cdot \text{deg})} \right)$$

$$x_{low} = 4.47 \text{ in}$$

$$x_{med} = 8.95 \text{ in}$$

$$x_{high} = 12 \text{ in}$$

$$r_{up\Delta} := r_u - \frac{8P_{roof} \cdot \frac{x_{low}}{2}}{L^2} \qquad r_{up\Delta M} := r_u - \frac{8P_{roof} \cdot \frac{x_{med}}{2}}{L^2} \qquad r_{up\Delta H} := r_u - \frac{8P_{roof} \cdot \frac{x_{high}}{2}}{L^2}$$

$$r_{up\Delta} = 4.47 \text{ psi}$$

$$r_{up\Delta M} = 4.2 \text{ psi}$$

$$r_{up\Delta H} = 4.02 \text{ psi}$$

Walls With Personnel Doors

$$L := 256 \cdot \text{in}$$

$$b := 49.2$$

$$\text{mass} := 1924 \cdot \frac{\text{psi} \cdot \text{msec}^2}{\text{in}}$$

$$r_u := 5.53 \cdot \text{psi}$$

$$k_e := 2.58 \cdot \frac{\text{psi}}{\text{in}}$$

$$x_{\text{low}} := \frac{L}{2} \cdot \tan(2 \cdot \text{deg})$$

$$x_{\text{med}} := \frac{L}{2} \cdot \tan(4 \cdot \text{deg})$$

$$x_{\text{high}} := \min \left(\frac{12 \cdot \text{in}}{\frac{L}{2} \cdot \tan(6 \cdot \text{deg})} \right)$$

$$x_{\text{low}} = 4.47 \text{ in}$$

$$x_{\text{med}} = 8.95 \text{ in}$$

$$x_{\text{high}} = 12 \text{ in}$$

$$r_{\text{up}\Delta} := r_u - \frac{8P_{\text{roof}} \cdot \frac{x_{\text{low}}}{2}}{L^2}$$

$$r_{\text{up}\Delta M} := r_u - \frac{8P_{\text{roof}} \cdot \frac{x_{\text{med}}}{2}}{L^2}$$

$$r_{\text{up}\Delta H} := r_u - \frac{8P_{\text{roof}} \cdot \frac{x_{\text{high}}}{2}}{L^2}$$

$$r_{\text{up}\Delta} = 5.26 \text{ psi}$$

$$r_{\text{up}\Delta M} = 4.99 \text{ psi}$$

$$r_{\text{up}\Delta H} = 4.81 \text{ psi}$$

Walls With Equipment Doors

$$L := 256 \cdot \text{in}$$

$$b := 97.6$$

$$\text{mass} := 1511 \cdot \frac{\text{psi} \cdot \text{msec}^2}{\text{in}}$$

$$r_u := 4.98 \cdot \text{psi}$$

$$k_e := 1.97 \cdot \frac{\text{psi}}{\text{in}}$$

$$x_{\text{low}} := \frac{L}{2} \cdot \tan(2 \cdot \text{deg})$$

$$x_{\text{med}} := \frac{L}{2} \cdot \tan(4 \cdot \text{deg})$$

$$x_{\text{high}} := \min \left(\frac{12 \cdot \text{in}}{\frac{L}{2} \cdot \tan(6 \cdot \text{deg})} \right)$$

$$x_{\text{low}} = 4.47 \text{ in}$$

$$x_{\text{med}} = 8.95 \text{ in}$$

$$x_{\text{high}} = 12 \text{ in}$$

$$r_{\text{up}\Delta} := r_u - \frac{8P_{\text{roof}} \cdot \frac{x_{\text{low}}}{2}}{L^2}$$

$$r_{\text{up}\Delta M} := r_u - \frac{8P_{\text{roof}} \cdot \frac{x_{\text{med}}}{2}}{L^2}$$

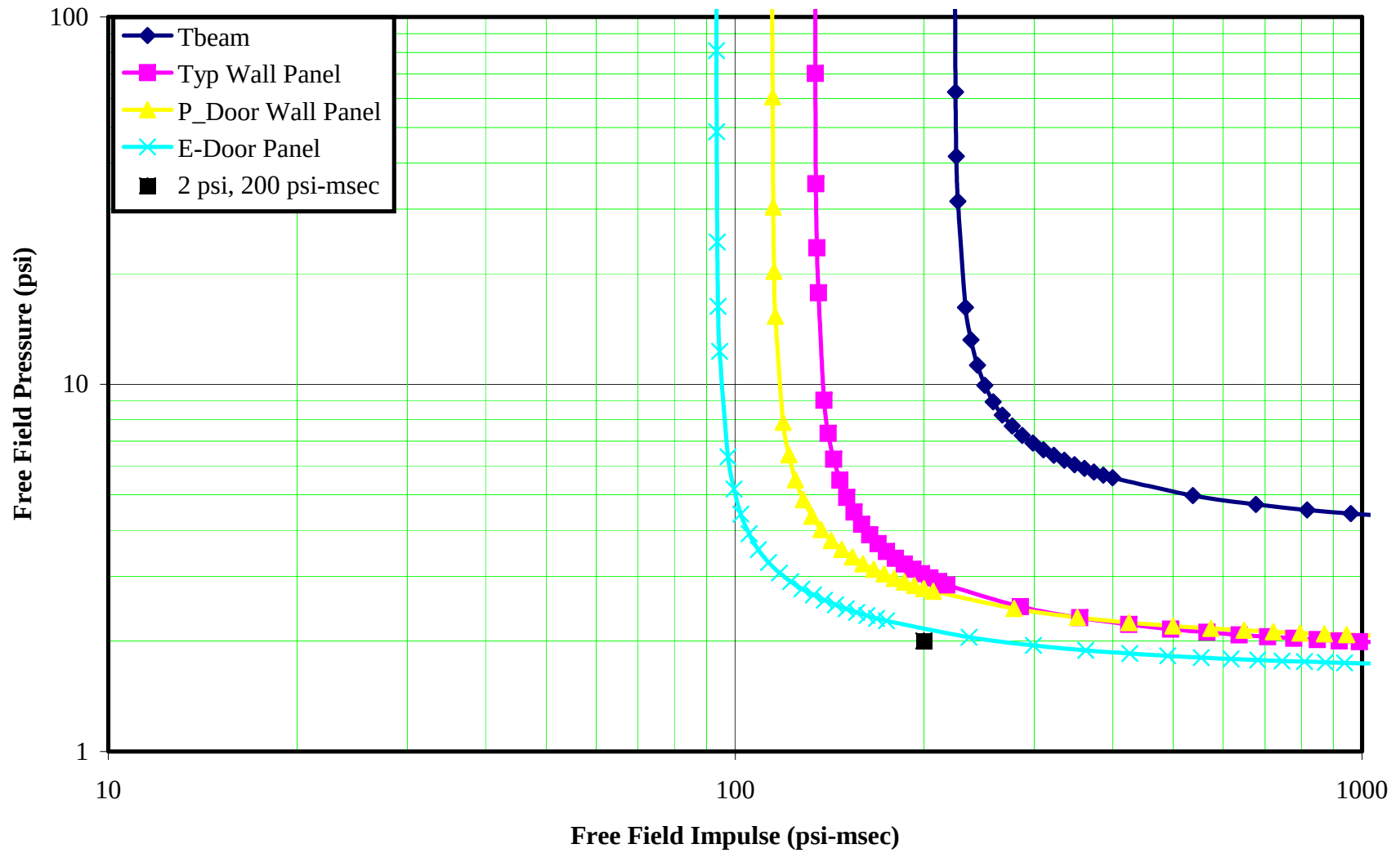
$$r_{\text{up}\Delta H} := r_u - \frac{8P_{\text{roof}} \cdot \frac{x_{\text{high}}}{2}}{L^2}$$

$$r_{\text{up}\Delta} = 4.71 \text{ psi}$$

$$r_{\text{up}\Delta M} = 4.44 \text{ psi}$$

$$r_{\text{up}\Delta H} = 4.26 \text{ psi}$$

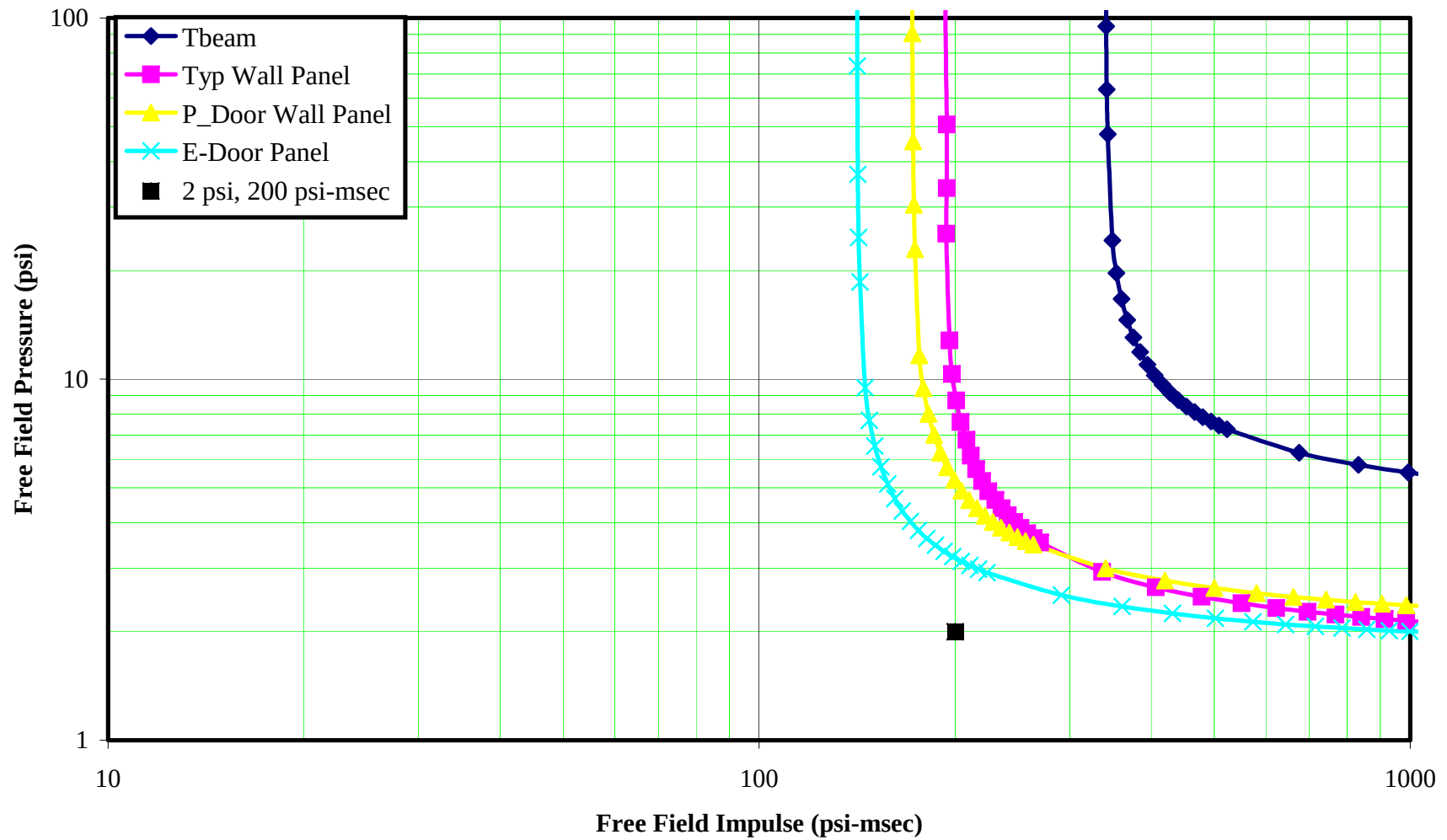
Substation 2 - 10 m
Low Damage



Substation 2 - 10 m

Medium Damage

*****WITH STIRRUPS IN WALLS*****



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APPENDIX B

Bolted Connections Calculations

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Introduction

Saudi Aramco have decided that the amount of site welding required for the connection details within the original design is not acceptable.

Table 1, within the main body of this calculation, was produced to avoid spending undue design time on all the connections. This study was performed for the 47.5m by 10m wide building only, but the results of which are assumed to be typical of the 40m by 15m wide building also. It was therefore decided to concentrate design efforts on the following types of connection;

- Wall Panel to Wall Panel Connection (Short Wall)
- Wall Panel to Wall Panel Connection (Long Wall)
- Wall Panel to Foundation Connection
- Roof Panel to Roof Panel Connection
- Roof Panel (Tee-section) to Wall Panel Connection (Short Wall)
- Roof Panel (Flange of Tee-section) to Wall Panel Connection (Long Wall)
- Roof Panel (Web of Tee-section) to Wall Panel Connection (Long Wall)

The bolted connections have been designed in accordance with the requirements of ACI 318-02.

The applied loading to the connections has been taken from the original Saudconsult welded connection design calculations; no further analysis has been performed. The Saudconsult results are the worst case loading for the two substations, and therefore the welded connections are suitable for both structures. Similarly, given the loads used within the ABSC calculations are taken from the Saudconsult results, the bolted connections are suitable for both substations.

Calculations

The design calculations for the revised bolted connections are included on the following pages. These calculations include sketches of the proposed bolted connections.

Within these calculations no material strength increase factors and/or dynamic increase factors have been included.

Conclusion

Saudi Aramco have decided that the amount of site welding required for the connection details within the original design of the blast resistant precast concrete substations is unacceptable. Within this appendix, justification calculations are presented for the design of alternative bolted connections for the precast concrete substations.

The applied loading used within these justification calculations is taken from the original Saudconsult welded connection design calculations; no further analysis has been performed.

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SAUDI ARAMCO - Abu Ali Plant

Bolted Connections

Introduction

This calculation presents the detailed re-design of the connections of the modular blast resistant precast concrete substation for the Saudi Arabian Oil Company (Saudi Aramco). The building design utilises precast sections rather than in-situ concrete to improve concrete quality control.

The original design of the connections utilised a high number of welded connections. It has been decided that the connections need readdressing to, if possible, reduce the number of welds required, by providing alternative bolted connections.

Original Connection Capacities

The original welded connection capacities have been taken from *ABS Report 1282017-R-01, Issue 1, Appendix B Connection Design Calculations*.

Wall to Wall Connection (Short Wall)

- Applied Upward Shear Force Per Meter $\text{Load}_{\text{short,perm}} := 261.6 \cdot \text{kN} \cdot \text{m}^{-1}$
- Applied Upward Shear Force Per Fixing (500mm c/c) $\text{Load}_{\text{short}} := 0.5 \cdot \text{m} \cdot \text{Load}_{\text{short,perm}}$ $\text{Load}_{\text{short}} = 130.8 \text{ kN}$

Wall to Wall Connection (Long Wall)

- Applied Upward Shear Force Per Meter $\text{Load}_{\text{long,perm}} := 20.5 \cdot \text{kN} \cdot \text{m}^{-1}$
- Applied Upward Shear Force Per Fixing (500mm c/c) $\text{Load}_{\text{long}} := 0.5 \cdot \text{m} \cdot \text{Load}_{\text{long,perm}}$ $\text{Load}_{\text{long}} = 10.25 \text{ kN}$

Shear Wall to Foundation Slab Connection (Short Wall)

- Applied Upward Shear Force Per Fixing $\text{Load}_{\text{found}} := 128.8 \cdot \text{kN}$

Roof Panel to Roof Panel Connection

- Applied Horizontal Shear Force Per Meter $\text{Load}_{\text{roof,perm}} := 130 \cdot \text{kN} \cdot \text{m}^{-1}$
- Applied Horizontal Shear Force Per Fixing (500mm c/c) $\text{Load}_{\text{roof}} := 0.5 \cdot \text{m} \cdot \text{Load}_{\text{roof,perm}}$ $\text{Load}_{\text{roof}} = 65 \text{ kN}$

Roof Panel to Shear Wall Connection (T-section to short wall)

- Applied Horizontal Shear Force Per Panel $\text{Load}_{\text{roof2,panel}} := 428 \cdot \text{kN}$
- Applied Horizontal Shear Force Per Fixing (500mm c/c) $\text{Load}_{\text{roof2}} := \frac{\text{Load}_{\text{roof2,panel}}}{5}$ $\text{Load}_{\text{roof2}} = 85.6 \text{ kN}$

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Roof Panel to Shear Wall Connection (Flange to long wall)

- Applied Horizontal Shear Force Per Panel $\text{Load}_{\text{roof3,panel}} := 181 \cdot \text{kN}$

- Applied Horizontal Shear Force Per Fixing (500mm c/c) $\text{Load}_{\text{roof3}} := \frac{\text{Load}_{\text{roof3,panel}}}{5}$ $\text{Load}_{\text{roof3}} = 36.2 \text{ kN}$

Roof Panel to Shear Wall Connection (Web to long wall)

- Applied Horizontal Shear Force Per Web $\text{Load}_{\text{roof4}} := 45.26 \cdot \text{kN}$

Wall to Wall Connection (Short Wall)

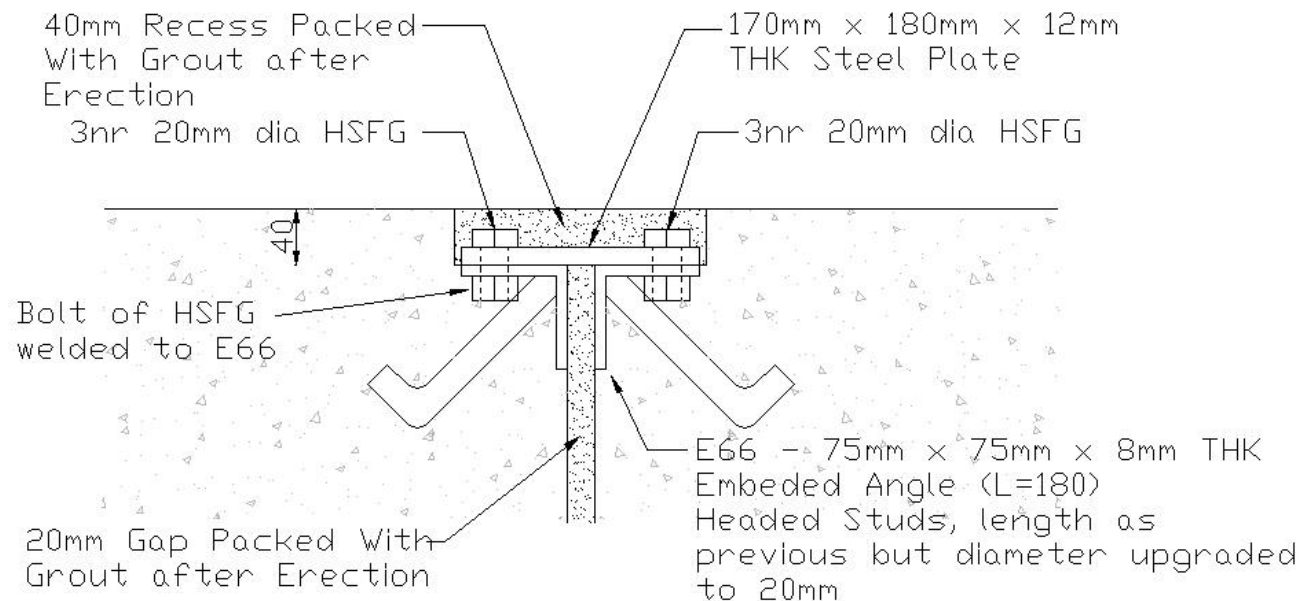
The proposed arrangement of the wall to wall (short wall) bolted connection is shown below. As per the original design, it is proposed to use these fixings at 500mm centres.

The proposed modified design, utilises the use of the originally embedded 75mm x 75mm x 8mm equal angle with headed studs (member E66). However, the headed studs have been upgraded to 3nr 20mm dia anchor studs per member.

It is also proposed that a recess at the side of the precast concrete wall panel be introduced, to allow the bolting connection to be offset from the face of the panel. Following erection of the shear wall it is recommended that the 40mm recess is packed with grout, to give a smooth external face.

Each of the embedded equal angles will be bolted to the same 170mm x 180mm x 12mm steel plate using 3nr 20mm dia High Strength Friction Grip Bolts (HSFG) on each embedded equal angle.

As shown below, the bolts of the HSFG anchors will be welded to the inside of the angle, to stop rotation of the bolt following installation.



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Shear Capacity of the Proposed High Strength Friction Grip Bolts

The shear capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 20mm dia
High Strength Friction Grip Bolt

$$V_{\text{cap.bolt.wall}} := 71.3 \cdot \text{kN}$$

Shear Capacity of the Bolted Connection

$$V_{\text{cap.wall}} := 3 \cdot V_{\text{cap.bolt.wall}}$$

$$V_{\text{cap.wall}} = 213.9 \text{ kN}$$

Factor of Safety against Shear Failure of the
High Strength Friction Grip Bolt

$$\text{FOS}_{\text{short.bolt}} := \frac{V_{\text{cap.wall}}}{\text{Load}_{\text{short}}}$$

$$\text{FOS}_{\text{short.bolt}} = 1.635$$

Shear Capacity of the Anchorage of the Studs into the Precast Concrete Wall Panels

The capacity of the anchorage of headed stud into the wall panel will be calculated using the methodology in ACI 318-02 "Building Code Requirements for Structural Concrete", Appendix D. The studs will be checked for;

- 1) steel strength of the anchor in shear,
- 2) concrete breakout strength of anchor in shear,
- 3) concrete pryout strength of anchor in shear.

Shear Strength of Anchor in Steel (cl. D.6.1)

Effective cross-sectional area of an anchor

$$A_{\text{se}} := 245 \cdot \text{mm}^2$$

Specified tensile strength of anchor steel

$$f_{\text{ut}} := 620 \cdot \text{MPa}$$

Specified yield strength of anchor steel

$$f_y := 420 \cdot \text{MPa}$$

Limitations on tensile strength of anchor steel

$$f_{\text{ut}} := \min(f_{\text{ut}}, 1.9 \cdot f_y, 125000 \cdot \text{psi})$$

$$f_{\text{ut}} = 620 \text{ MPa}$$

Nominal Strength of Headed Studs in Shear

$$V_s := 3 \cdot A_{\text{se}} \cdot f_{\text{ut}}$$

$$V_s = 455.7 \text{ kN}$$

Factor of Safety against Shear Strength Failure
of the Studs

$$\text{FOS}_{\text{short.steel}} := \frac{V_s}{\text{Load}_{\text{short}}}$$

$$\text{FOS}_{\text{short.steel}} = 3.484$$

Shear Strength of Concrete in Breakout (cl. D.6.2)

The anchorage arrangement is reinforced, which will prevent concrete breakout since the rebar will effectively attract the shear loading.

It is therefore considered inappropriate to use the methodology defined within ACI 318-02, Clause D.6.2 for the determination of concrete breakout strength, due to the reinforcement in the wall slabs.

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Shear Concrete Pryout Strength of Anchor (cl. D.6.3)

Effective anchor embedment depth $h_{ef} := 150 \cdot \text{mm}$ $h_{ef} = 5.906 \text{ in}$

$h_{ef} > 2.5 \text{ in}$, therefore, $k_{cp} := 2$

Given the reinforced anchorage arrangement, it is considered appropriate to define the Nominal concrete breakout strength as being equal to the Basic concrete breakout strength (i.e. $N_{cb} = N_b$).

ACI 318-02 constant, for cast-in-anchors $k := 24$

Compressive Strength of Concrete $f_c := 35 \cdot \text{N} \cdot \text{mm}^{-2}$

Basic concrete breakout strength (cl. D.5.2.2) $N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$ $N_b = 109.159 \text{ kN}$

Nominal concrete breakout strength (cl. D.5.2.1) $N_{cb} := N_b$ $N_{cb} = 109.159 \text{ kN}$

Nominal Pryout Strength $V_{cp} := k_{cp} \cdot N_{cb}$ $V_{cp} = 218.317 \text{ kN}$

Factor of Safety against Shear Concrete Pryout Strength Failure of the Studs $\text{FOS}_{\text{short.pry}} := \frac{V_{cp}}{\text{Load}_{\text{short}}}$ $\text{FOS}_{\text{short.pry}} = 1.669$

Summary of Bolted Precast Concrete Wall Panel to Wall Panel Connection (Short Wall)

Minimum Factor of Safety for Bolted Connection $\text{FOS}_{\text{short.wall}} := \min(\text{FOS}_{\text{short.bolt}}, \text{FOS}_{\text{short.steel}}, \text{FOS}_{\text{short.pry}})$
 $\text{FOS}_{\text{short.wall}} = 1.635$

This calculation has demonstrated that the proposed bolted connection of the precast concrete wall panel to wall panel (short wall) has adequate capacity to withstand the imposed loading.

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Wall to Wall Connection (Long Wall)

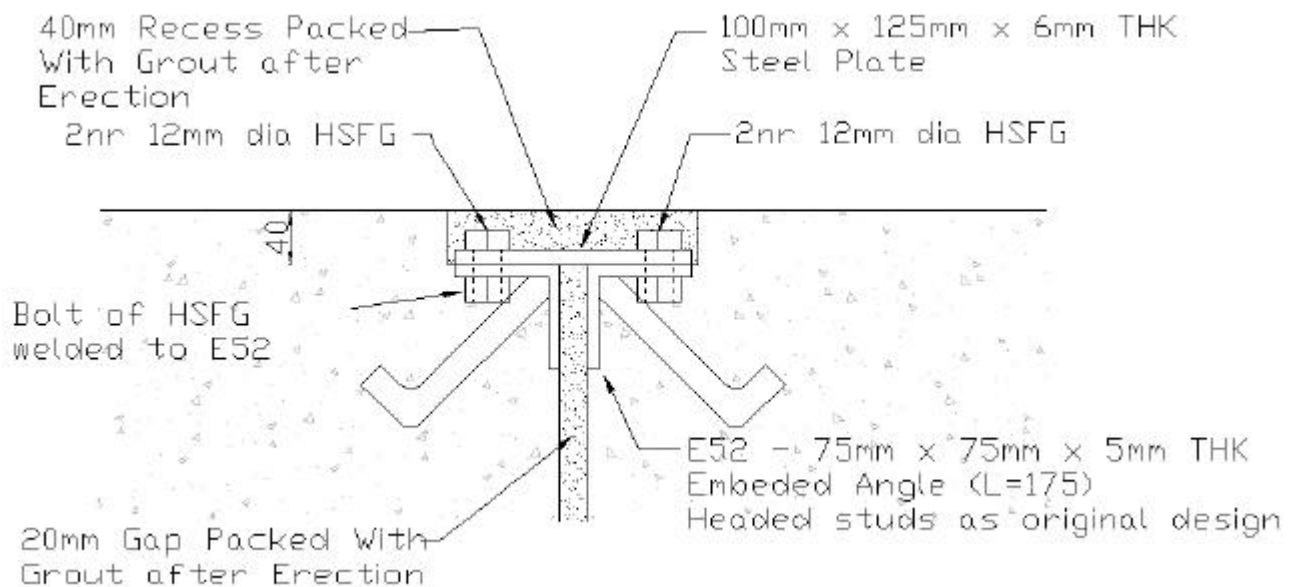
The proposed arrangement of the wall to wall (long wall) bolted connection is shown below. As per the original design, it is proposed to use these fixings at 500mm centres.

The proposed modified design, utilises the use of the originally embedded 75mm x 75mm x 5mm equal angle with headed studs (member E52).

It is also proposed that a recess at the side of the precast concrete wall panel be introduced, to allow the bolting connection to be offset from the face of the panel. Following erection of the shear wall it is recommended that the 40mm recess is packed with grout, to give a smooth external face.

Each of the embedded equal angles will be bolted to the same 100mm x 125mm x 5mm steel plate using 2nr 12mm dia High Strength Friction Grip Bolts (HSFG) on each embedded equal angle.

As shown below, the bolts of the HSFG anchors will be welded to the inside of the angle, to stop rotation of the bolt following installation.



Shear Capacity of the Proposed High Strength Friction Grip Bolts

The shear capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 12mm dia High Strength Friction Grip Bolt

$$V_{cap.bolt.wall} := 24.5 \cdot kN$$

Shear Capacity of the Bolted Connection

$$V_{cap.wall} := 2 \cdot V_{cap.bolt.wall}$$

$$V_{cap.wall} = 49 \cdot kN$$

Factor of Safety against Shear Failure of the High Strength Friction Grip Bolt

$$FOS_{long.bolt} := \frac{V_{cap.wall}}{Load_{long}}$$

$$FOS_{long.bolt} = 4.78$$

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Shear Capacity of the Anchorage of the Studs into the Precast Concrete Wall Panels

The capacity of the anchorage of headed stud into the wall panel will be calculated using the methodology in ACI 318-02 "Building Code Requirements for Structural Concrete", Appendix D. The studs will be checked for;

- 1) steel strength of the anchor in shear,
- 2) concrete breakout strength of anchor in shear,
- 3) concrete pryout strength of anchor in shear.

Shear Strength of Anchor in Steel (cl. D.6.1)

Effective cross-sectional area of an anchor	$A_{se} := 84.3 \cdot \text{mm}^2$	
Specified tensile strength of anchor steel	$f_{ut} := 620 \cdot \text{MPa}$	
Specified yield strength of anchor steel	$f_y := 420 \cdot \text{MPa}$	
Limitations on tensile strength of anchor steel	$f_{ut} := \min(f_{ut}, 1.9 \cdot f_y, 125000 \cdot \text{psi})$	$f_{ut} = 620 \text{ MPa}$
Nominal Strength of Headed Studs in Shear	$V_s := 2 \cdot A_{se} \cdot f_{ut}$	$V_s = 104.532 \text{ kN}$
Factor of Safety against Shear Strength Failure of the Studs	$\text{FOS}_{\text{long, steel}} := \frac{V_s}{\text{Load}_{\text{long}}}$	$\text{FOS}_{\text{long, steel}} = 10.198$

Shear Strength of Concrete in Breakout (cl. D.6.2)

The anchorage arrangement is reinforced, which will prevent concrete breakout since the rebar will effectively attract the shear loading.

It is therefore considered inappropriate to use the methodology defined within ACI 318-02, Clause D.6.2 for the determination of concrete breakout strength, due to the reinforcement in the wall slabs.

Shear Concrete Pryout Strength of Anchor (cl. D.6.3)

Effective anchor embedment depth	$h_{ef} := 150 \cdot \text{mm}$	$h_{ef} = 5.906 \text{ in}$
$h_{ef} > 2.5 \text{ in}$, therefore,	$k_{cp} := 2$	
Given the reinforced anchorage arrangement, it is considered appropriate to define the Nominal concrete breakout strength as being equal to the Basic concrete breakout strength (i.e. $N_{cb} = N_b$).		
ACI 318-02 constant, for cast-in-anchors	$k := 24$	
Compressive Strength of Concrete	$f_c := 35 \cdot \text{N} \cdot \text{mm}^{-2}$	
Basic concrete breakout strength (cl. D.5.2.2)	$N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$	$N_b = 109.159 \text{ kN}$
Nominal concrete breakout strength (cl. D.5.2.1)	$N_{cb} := N_b$	$N_{cb} = 109.159 \text{ kN}$

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Nominal Pryout Strength

$$V_{cp} := k_{cp} \cdot N_{cb}$$

$$V_{cp} = 218.317 \text{ kN}$$

Factor of Safety against Shear Concrete Pryout
Strength Failure of the Studs

$$FOS_{long.pry} := \frac{V_{cp}}{Load_{long}}$$

$$FOS_{long.pry} = 21.299$$

Summary of Bolted Precast Concrete Wall Panel to Wall Panel Connection (Short Wall)

Minimum Factor of Safety for Bolted Connection

$$FOS_{long.wall} := \min(FOS_{long.bolt}, FOS_{long.steel}, FOS_{long.pry})$$

$$FOS_{long.wall} = 4.78$$

This calculation has demonstrated that the proposed bolted connection of the precast concrete wall panel to wall panel (long wall) has adequate capacity to withstand the imposed loading.

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Roof Panel to Roof Panel Connection

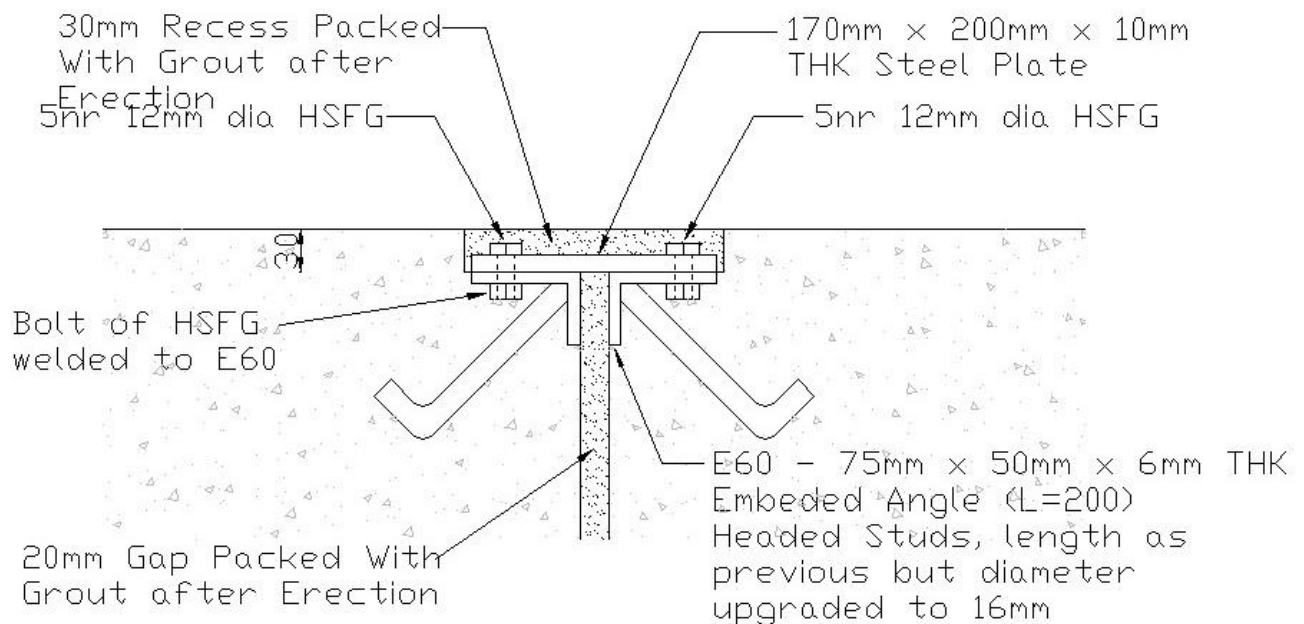
The proposed arrangement of the roof panel to roof panel bolted connection is shown below. As per the original design, it is proposed to use these fixings at 500mm centres.

The proposed modified design, utilises the use of the originally embedded 75mm x 50mm x 6mm angle section with headed studs (member E60). However, the headed studs have been upgraded to 3nr 16mm dia anchor studs per member.

It is also proposed that a recess at the side of the precast concrete roof panel be introduced, to allow the bolting connection to be offset from the face of the panel. Following erection of the shear wall it is recommended that the 30mm recess is packed with grout, to give a smooth external face.

Each of the embedded equal angles will be bolted to the same 170mm x 200mm x 10mm steel plate using 5nr 12mm dia High Strength Friction Grip Bolts (HSFG) on each embedded equal angle.

As shown below, the bolts of the HSFG anchors will be welded to the inside of the angle, to stop rotation of the bolt following installation.



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Shear Capacity of the Proposed High Strength Friction Grip Bolts

The shear capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 12mm dia High Strength Friction Grip Bolt

$$V_{\text{cap.bolt.roof}} := 24.5 \text{ kN}$$

Shear Capacity of the Bolted Connection

$$V_{\text{cap.roof}} := 5 \cdot V_{\text{cap.bolt.roof}}$$

$$V_{\text{cap.roof}} = 122.5 \text{ kN}$$

Factor of Safety against Shear Failure of the High Strength Friction Grip Bolt

$$\text{FOS}_{\text{roof.bolt}} := \frac{V_{\text{cap.roof}}}{\text{Load}_{\text{roof}}}$$

$$\text{FOS}_{\text{roof.bolt}} = 1.885$$

Shear Capacity of the Anchorage of the Studs into the Precast Concrete Roof Panels

As for the precast concrete wall panels, the capacity of the anchorage of headed stud into the roof panel will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in shear,
- 2) concrete breakout strength of anchor in shear,
- 3) concrete pryout strength of anchor in shear.

Shear Strength of Anchor in Steel (cl. D.6.1)

Effective cross-sectional area of an anchor

$$A_{\text{se}} := 157 \cdot \text{mm}^2$$

Specified tensile strength of anchor steel

$$f_{\text{ut}} := 620 \cdot \text{MPa}$$

Specified yield strength of anchor steel

$$f_y := 420 \cdot \text{MPa}$$

Limitations on tensile strength of anchor steel

$$f_{\text{ut}} := \min(f_{\text{ut}}, 1.9 \cdot f_y, 125000 \cdot \text{psi})$$

$$f_{\text{ut}} = 6.2 \times 10^8 \text{ Pa}$$

Nominal Strength of Headed Studs in Shear

$$V_s := 3 \cdot A_{\text{se}} \cdot f_{\text{ut}}$$

$$V_s = 292.02 \text{ kN}$$

Factor of Safety against Shear Strength Failure of the Studs

$$\text{FOS}_{\text{roof.steel}} := \frac{V_s}{\text{Load}_{\text{roof}}}$$

$$\text{FOS}_{\text{roof.steel}} = 4.493$$

Shear Strength of Concrete in Breakout (cl. D.6.2)

The anchorage arrangement is reinforced, which will prevent concrete breakout since the rebar will effectively attract the shear loading.

It is therefore considered inappropriate to use the methodology defined within ACI 318-02, Clause D.6.2 for the determination of concrete breakout strength, due to the reinforcement in the roof slabs.

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Shear Concrete Pryout Strength of Anchor (cl. D.6.3)

Effective anchor embedment depth $h_{ef} := 150 \cdot \text{mm}$ $h_{ef} = 5.906 \text{ in}$

$h_{ef} > 2.5 \text{ in}$, therefore, $k_{cp} := 2$

Given the reinforced anchorage arrangement, it is considered appropriate to define the Nominal concrete breakout strength as being equal to the Basic concrete breakout strength (i.e. $N_{cb} = N_b$).

ACI 318-02 constant, for cast-in-anchors $k := 24$

Compressive Strength of Concrete $f_c := 35 \cdot \text{N} \cdot \text{mm}^{-2}$

Basic concrete breakout strength (cl. D.5.2.2) $N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$ $N_b = 109.159 \text{ kN}$

Nominal concrete breakout strength (cl. D.5.2.1) $N_{cb} := N_b$ $N_{cb} = 109.159 \text{ kN}$

Nominal Pryout Strength $V_{cp} := k_{cp} \cdot N_{cb}$ $V_{cp} = 218.317 \text{ kN}$

Factor of Safety against Shear Concrete Pryout Strength Failure of the Studs $\text{FOS}_{\text{roof.pry}} := \frac{V_{cp}}{\text{Load}_{\text{roof}}}$ $\text{FOS}_{\text{roof.pry}} = 3.359$

Summary of Bolted Precast Concrete Roof Slab to Roof Slab Connection

Minimum Factor of Safety for Bolted Connection $\text{FOS}_{\text{roof}} := \min(\text{FOS}_{\text{roof.bolt}}, \text{FOS}_{\text{roof.steel}}, \text{FOS}_{\text{roof.pry}})$
 $\text{FOS}_{\text{roof}} = 1.885$

This calculation has demonstrated that the proposed bolted connection of the precast concrete roof panel to roof panel has adequate capacity to withstand the imposed loading.

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Wall Panel to Foundation Connection

The proposed arrangement of the wall panel to foundation slab bolted connection is shown below. As per the original design, it is proposed to use these fixings at 400mm centres.

It is proposed to modify the design of the equal angle (member ACC-01) used in this connection detail. As per the original design the a 125mm x 125mm equal angle is specified, however, this will be made from 20mm thick plate.

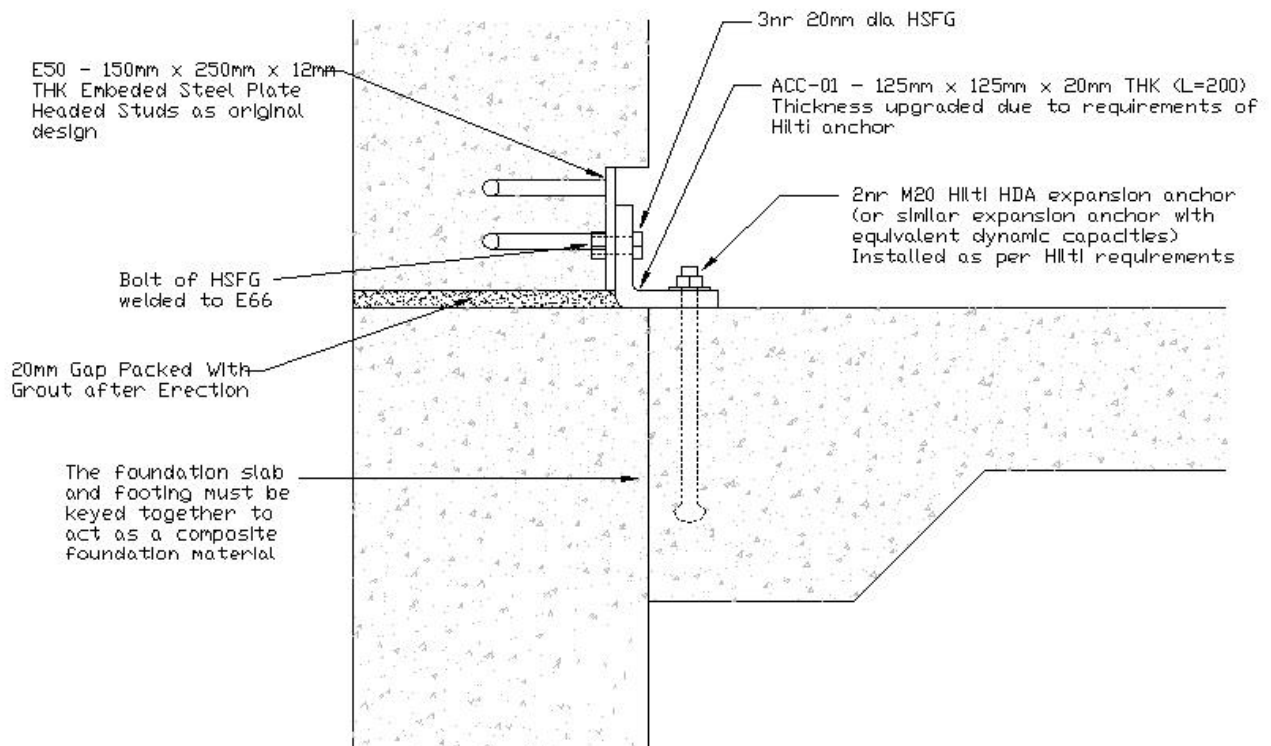
The proposed modified design, utilises the originally embedded 150mm x 250mm x 12mm thick steel plate with headed studs (member E50) into the base of the wall panel.

The new connection will be anchored to the foundation by 2nr M20 diameter Hilti HDA expansion anchors (or a similar expansion anchor with equivalent dynamic capacities).

To provide sufficient anchorage capacity of the expansion anchors into the foundation slab it is required that the slab and wall footing are keyed together to act as a composite foundation material.

The equal angles will be bolted to the same 170mm x 200mm x 10mm steel plate using 5nr 12mm dia High Strength Friction Grip Bolts (HSFG) on each embedded equal angle.

As shown below, the bolts of the HSFG anchors will be welded to the inside of the angle, to stop rotation of the bolt following installation.



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Pullout Capacity of the Expansion Anchors (into Foundation Slab)

The tensile capacity of the anchorage of the Hilti post-installed anchors into the foundation slab will be calculated using the methodology in *Hilti Fastening Technology Manual, Issue 2003*. The anchors will be checked for;

- 1) pullout resistance for cracked concrete,
- 2) concrete cone resistance,
- 3) steel design tensile resistance.

Pullout Resistance for Cracked Concrete

Design pullout resistance

$$N_{0,Rd,p} := 63.4 \text{ kN}$$

Influence of concrete strength

$$f_B := \frac{\left(\frac{f_c}{0.8} \right)}{\sqrt{25 \cdot \text{N} \cdot \text{mm}^{-2}}} \quad f_B = 1.323$$

Cracked concrete design resistance

$$N_{Rd,p} := N_{0,Rd,p} \cdot f_B \quad N_{Rd,p} = 83.87 \text{ kN}$$

Concrete Cone Resistance

Design concrete cone resistance

$$N_{0,Rd,c} := 109.4 \text{ kN}$$

Spacing of anchors

$$s_{anc} := 250 \text{ mm}$$

Effective anchorage depth

$$h_{ef} := 250 \text{ mm}$$

Influence of anchor spacing

$$f_{A,N} := 0.5 + \frac{s_{anc}}{6 \cdot h_{ef}} \quad f_{A,N} = 0.667$$

It is recommended that the foundation slab and footing are keyed together to act as a composite foundation material for the anchorage, therefore, no edge distance concerns exist.

Influence of edge distance

$$f_{R,N} := 1$$

Concrete cone design resistance

$$N_{Rd,c} := N_{0,Rd,c} \cdot f_B \cdot f_{A,N} \cdot f_{R,N} \quad N_{Rd,c} = 96.482 \text{ kN}$$

Steel Design Tensile Resistance

Steel design tensile resistance

$$N_{Rd,s} := 130.7 \text{ kN}$$

Tensile Capacity Check of HDA Anchor

Tensile resistance of a single M20 HDA Anchor

$$N_{Rd} := \min(N_{Rd,p}, N_{Rd,c}, N_{Rd,s}) \quad N_{Rd} = 83.87 \text{ kN}$$

Factor of Safety against Tensile Failure of Hilti HDA Anchors

$$\text{FOS}_{\text{found.hilti}} := \frac{2 \cdot N_{Rd}}{\text{Load}_{\text{found}}} \quad \text{FOS}_{\text{found.hilti}} = 1.302$$

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Shear Capacity of the Proposed High Strength Friction Grip Bolts (Angle to Wall Connection)

The shear capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 20mm dia
High Strength Friction Grip Bolt

$$V_{\text{cap.bolt.found}} := 71.3 \cdot \text{kN}$$

Shear Capacity of the Bolted Connection

$$V_{\text{cap.found}} := 3 \cdot V_{\text{cap.bolt.found}}$$

$$V_{\text{cap.found}} = 213.9 \cdot \text{kN}$$

Factor of Safety against Shear Failure of the
High Strength Friction Grip Bolt

$$\text{FOS}_{\text{found.bolt}} := \frac{V_{\text{cap.found}}}{\text{Load}_{\text{found}}}$$

$$\text{FOS}_{\text{found.bolt}} = 1.661$$

Shear Capacity of the Anchorage of the Studs into the Precast Concrete Wall Panels

As previously for the precast concrete wall panel and roof panel connections, the capacity of the anchorage of headed stud into the base of the wall panels for the connections to the foundations will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in shear,
- 2) concrete breakout strength of anchor in shear,
- 3) concrete pryout strength of anchor in shear.

Shear Strength of Anchor in Steel (cl. D.6.1)

Effective cross-sectional area of an anchor

$$A_{\text{se}} := 84.3 \cdot \text{mm}^2$$

Specified tensile strength of anchor steel

$$f_{\text{ut}} := 620 \cdot \text{MPa}$$

Specified yield strength of anchor steel

$$f_y := 420 \cdot \text{MPa}$$

Limitations on tensile strength of anchor steel

$$f_{\text{ut}} := \min(f_{\text{ut}}, 1.9 \cdot f_y, 125000 \cdot \text{psi})$$

$$f_{\text{ut}} = 620 \cdot \text{MPa}$$

Nominal Strength of Headed Studs in Shear

$$V_s := 6 \cdot A_{\text{se}} \cdot f_{\text{ut}}$$

$$V_s = 313.596 \cdot \text{kN}$$

Factor of Safety against Shear Strength Failure
of the Studs

$$\text{FOS}_{\text{found.steel}} := \frac{V_s}{\text{Load}_{\text{found}}}$$

$$\text{FOS}_{\text{found.steel}} = 2.435$$

Shear Strength of Concrete in Breakout (cl. D.6.2)

The anchorage arrangement is reinforced, which will prevent concrete breakout since the rebar will effectively attract the shear loading.

It is therefore considered inappropriate to use the methodology defined within ACI 318-02, Clause D.6.2 for the determination of concrete breakout strength, due to the reinforcement in the wall slabs.

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Shear Concrete Pryout Strength of Anchor (cl. D.6.3)

Effective anchor embedment depth $h_{ef} := 150 \cdot \text{mm}$ $h_{ef} = 5.906 \text{ in}$

$h_{ef} > 2.5 \text{ in}$, therefore, $k_{cp} := 2$

Given the reinforced anchorage arrangement, it is considered appropriate to define the Nominal concrete breakout strength as being equal to the Basic concrete breakout strength (i.e. $N_{cb} = N_b$).

ACI 318-02 constant, for cast-in-anchors $k := 24$

Compressive Strength of Concrete $f_c := 35 \cdot \text{N} \cdot \text{mm}^{-2}$

Basic concrete breakout strength (cl. D.5.2.2) $N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$ $N_b = 109.159 \text{ kN}$

Nominal concrete breakout strength (cl. D.5.2.1) $N_{cb} := N_b$ $N_{cb} = 109.159 \text{ kN}$

Nominal Pryout Strength $V_{cp} := k_{cp} \cdot N_{cb}$ $V_{cp} = 218.317 \text{ kN}$

Factor of Safety against Shear Concrete Pryout Strength Failure of the Studs $\text{FOS}_{\text{found.pry}} := \frac{V_{cp}}{\text{Load}_{\text{found}}}$ $\text{FOS}_{\text{found.pry}} = 1.695$

Summary of Bolted Precast Concrete Wall Panel to Foundation Connection

Minimum Factor of Safety for Bolted Connection

$\text{FOS}_{\text{found}} := \min(\text{FOS}_{\text{found.hilti}}, \text{FOS}_{\text{found.bolt}}, \text{FOS}_{\text{found.steel}}, \text{FOS}_{\text{found.pry}})$ $\text{FOS}_{\text{found}} = 1.302$

This calculation has demonstrated that the proposed bolted connection of the precast concrete wall panel to foundation has adequate capacity to withstand the imposed loading.

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Roof Panel to Shear Wall Connection (T-section to Short wall)

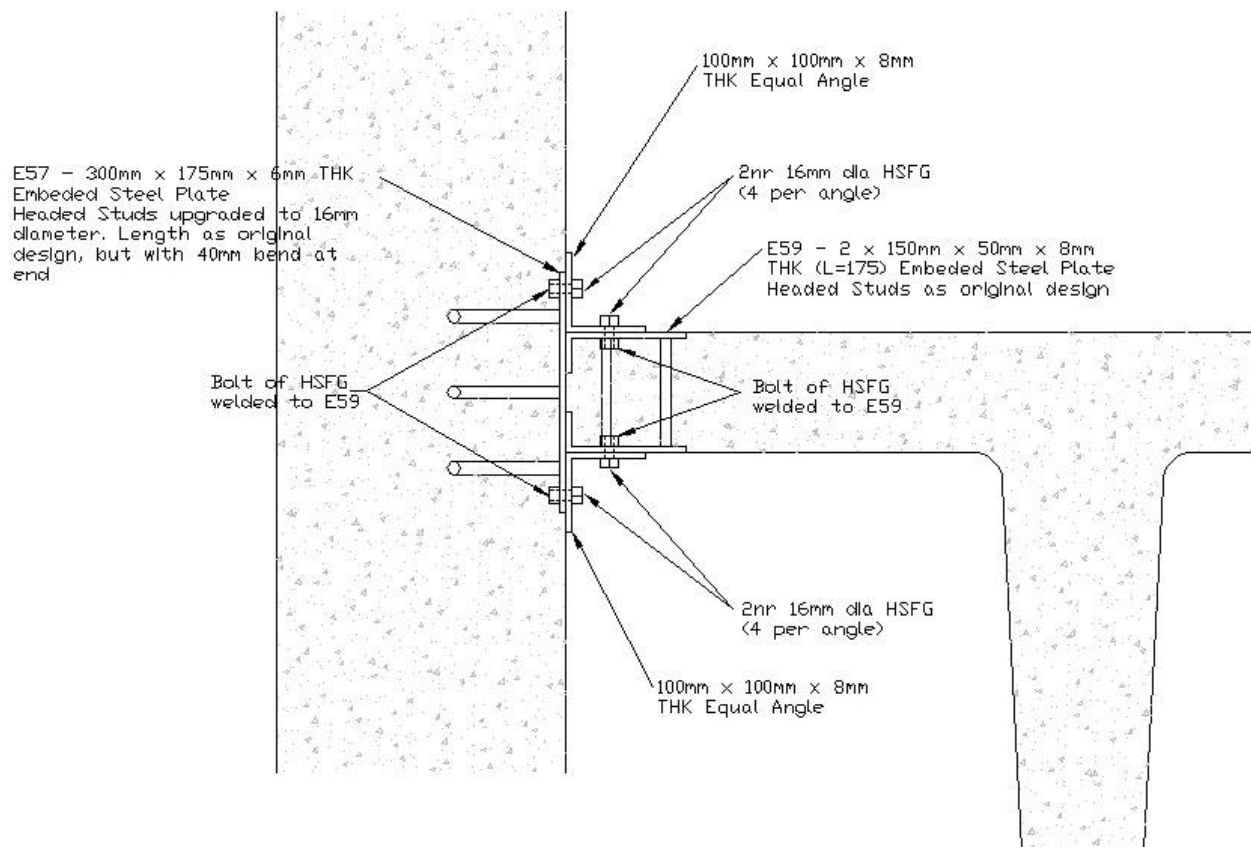
The proposed arrangement of the roof panel to shear wall bolted connection (T-section to short wall) is shown below. As per the original design, it is proposed to use these fixings at 500mm centres.

It is proposed to use a 100mm x 100mm x 8mm equal angle, above and below the roof flange section, bolted to the steel sections embedded into the precast concrete roof and wall panels. The angles will be bolted to the embedded steel sections using 16mm dia High Strength Friction Grip Bolts (HSFG).

As shown below, the bolts of the HSFG anchors will be welded to the inside of the embedded steel sections, to stop rotation of the bolt following installation.

It is proposed to modify the design of the embedded steel plate into the wall section (member E56) used in this connection detail. The size of the embedded plate has been upgraded to 300mm x 175mm x 8mm thick plate and the anchorage has been upgraded to 16mm dia anchor studs, with a 40mm bend at the end.

No modifications are proposed to the embedded member (E59) in the roof section.



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Shear Capacity of the Proposed High Strength Friction Grip Bolts (into Roof Slab)

The shear capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 16mm dia
High Strength Friction Grip Bolt

$$V_{cap.bolt.roof2} := 45.6 \text{ kN}$$

Shear Capacity of the Bolted Connection

$$V_{cap.roof2} := 2 \cdot (2 \cdot V_{cap.bolt.roof2}) \quad V_{cap.roof2} = 182.4 \text{ kN}$$

Factor of Safety against Shear Failure of the
High Strength Friction Grip Bolt

$$FOS_{roof.bolt2} := \frac{V_{cap.roof2}}{Load_{roof2}} \quad FOS_{roof.bolt2} = 2.131$$

Shear Capacity of the Anchorage of the Studs into the Precast Concrete Roof Panels

As previously, the capacity of the anchorage of headed stud into the roof panel will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in shear,
- 2) concrete breakout strength of anchor in shear,
- 3) concrete pryout strength of anchor in shear.

Shear Strength of Anchor in Steel (cl. D.6.1)

Effective cross-sectional area of an anchor

$$A_{se} := 84.3 \text{ mm}^2$$

Specified tensile strength of anchor steel

$$f_{ut} := 620 \text{ MPa}$$

Specified yield strength of anchor steel

$$f_y := 420 \text{ MPa}$$

Limitations on tensile strength of anchor steel

$$f_{ut} := \min(f_{ut}, 1.9 \cdot f_y, 125000 \text{ psi}) \quad f_{ut} = 6.2 \times 10^8 \text{ Pa}$$

Nominal Strength of Headed Studs in Shear

$$V_s := 4 \cdot A_{se} \cdot f_{ut} \quad V_s = 209.064 \text{ kN}$$

Factor of Safety against Shear Strength Failure
of the Studs

$$FOS_{roof.steel.shear} := \frac{V_s}{Load_{roof2}} \quad FOS_{roof.steel.shear} = 2.442$$

Shear Strength of Concrete in Breakout (cl. D.6.2)

The anchorage arrangement is reinforced, which will prevent concrete breakout since the rebar will effectively attract the shear loading.

It is therefore considered inappropriate to use the methodology defined within ACI 318-02, Clause D.6.2 for the determination of concrete breakout strength, due to the reinforcement in the roof slabs.

Shear Concrete Pryout Strength of Anchor (cl. D.6.3)

Shear concrete pryout is not considered a potential failure mechanism, as the anchorage studs pass from a steel plate on one side of the roof slab to a steel plate on the otherside of the roof slab.

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Pullout Capacity of the Proposed High Strength Friction Grip Bolts (into Shear Wall)

The pullout capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 16mm dia
High Strength Friction Grip Bolt

$$P_{\text{cap.bolt.roof}} := 82.9 \cdot \text{kN}$$

Shear Capacity of the Bolted Connection

$$P_{\text{cap.roof}} := 2 \cdot (2 \cdot P_{\text{cap.bolt.roof}})$$

$$P_{\text{cap.roof}} = 331.6 \cdot \text{kN}$$

Factor of Safety against Shear Failure of the
High Strength Friction Grip Bolt

$$\text{FOS}_{\text{roof.pull}} := \frac{P_{\text{cap.roof}}}{\text{Load}_{\text{roof2}}}$$

$$\text{FOS}_{\text{roof.pull}} = 3.874$$

Pullout Capacity of the Anchorage of the Studs into the Precast Concrete Wall Panels

The pullout capacity of the anchorage of headed stud into the wall panel will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in tension,
- 2) concrete breakout strength of anchor in tension,
- 3) pullout strength of the anchor in tension.

Tensile Strength of Anchor in Steel (cl. D.5.1)

Effective cross-sectional area of an anchor

$$A_{\text{se}} := 157 \cdot \text{mm}^2$$

Specified tensile strength of anchor steel

$$f_{\text{ut}} := 620 \cdot \text{MPa}$$

Specified yield strength of anchor steel

$$f_y := 420 \cdot \text{MPa}$$

Limitations on tensile strength of anchor steel

$$f_{\text{ut}} := \min(f_{\text{ut}}, 1.9 \cdot f_y, 125000 \cdot \text{psi})$$

$$f_{\text{ut}} = 6.2 \times 10^8 \cdot \text{Pa}$$

Nominal Strength of Headed Studs in Tension

$$N_s := 6 \cdot A_{\text{se}} \cdot f_{\text{ut}}$$

$$N_s = 584.04 \cdot \text{kN}$$

Factor of Safety against Shear Strength Failure
of the Studs

$$\text{FOS}_{\text{roof.steel.tensile}} := \frac{N_s}{\text{Load}_{\text{roof2}}}$$

$$\text{FOS}_{\text{roof.steel.tensile}} = 6.823$$

Tensile Strength of Concrete in Breakout (cl. D.5.2)

Effective anchor embedment depth

$$h_{\text{ef}} := 140 \cdot \text{mm}$$

Projected concrete failure area of group of anchors

$$A_N := (2 \cdot 1.5 \cdot h_{\text{ef}} + 115 \cdot \text{mm}) \cdot \left(\frac{1.5 \cdot h_{\text{ef}} + 95 \cdot \text{mm} \dots}{+ 95 \cdot \text{mm} + 200 \cdot \text{mm}} \right)$$

$$A_N = 0.321 \cdot \text{m}^2$$

Projected concrete failure area a single anchor,
when not limited by edge distance or spacing

$$A_{N0} := 9 \cdot h_{\text{ef}}^2$$

$$A_{N0} = 0.176 \cdot \text{m}^2$$

Minimum edge distance

$$c := 200 \cdot \text{mm}$$

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Modification factor for edge effects

$$\psi_2 := 0.7 + 0.3 \cdot \frac{c}{1.5 \cdot h_{ef}}$$

$$\psi_2 = 0.986$$

Modification factor for cracking

$$\psi_3 := 1.0$$

(assume cracked concrete)

ACI 318-02 constant, for cast-in-anchors

$$k = 24$$

Compressive Strength of Concrete

$$f_c = 35 \text{ N} \cdot \text{mm}^{-2}$$

Basic concrete breakout strength

$$N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$$

$$N_b = 98.427 \text{ kN}$$

Nominal concrete breakout strength in tension

$$N_{cb} := \frac{A_N}{A_{No}} \cdot \psi_2 \cdot \psi_3 \cdot N_b$$

$$N_{cb} = 176.551 \text{ kN}$$

Factor of Safety against Tensile Concrete Breakout of the Anchors

$$\text{FOS}_{\text{roof.break}} := \frac{N_{cb}}{\text{Load}_{\text{roof2}}}$$

$$\text{FOS}_{\text{roof.break}} = 2.063$$

Tensile Strength of Anchor in Pullout (cl. D.5.3)

Modification factor for cracking

$$\psi_4 := 1.0$$

(assume cracked concrete)

Distance from inner surface of the bolt shaft to outer tip of J-bolt

$$e_h := 40 \cdot \text{mm}$$

Shaft diameter of J-bolt

$$d_o := 16 \cdot \text{mm}$$

Pullout strength of a single hooked bolt

$$N_p := 0.9 \cdot f_c \cdot e_h \cdot d_o$$

$$N_p = 20.16 \text{ kN}$$

Nominal pullout strength of an anchor in tension

$$N_{pn} := \psi_4 \cdot N_p$$

$$N_{pn} = 20.16 \text{ kN}$$

Factor of Safety against Tensile Failure of Anchor in Pullout

$$\text{FOS}_{\text{roof.pullout}} := \frac{6 \cdot N_{pn}}{\text{Load}_{\text{roof2}}}$$

$$\text{FOS}_{\text{roof.pullout}} = 1.413$$

Summary of Bolted Precast Concrete Roof Panel to Shear Wall Connection (T-section to Short wall)

Minimum Factor of Safety for Bolted Connection

$$\text{FOS}_{\text{roof2}} := \min(\text{FOS}_{\text{roof.bolt2}}, \text{FOS}_{\text{roof.steel.shear}}, \text{FOS}_{\text{roof.pull}}, \text{FOS}_{\text{roof.steel.tensile}}, \text{FOS}_{\text{roof.break}}, \text{FOS}_{\text{roof.pullout}})$$

$$\text{FOS}_{\text{roof2}} = 1.413$$

This calculation has demonstrated that the proposed bolted connection of the precast concrete wall panel to roof panel (Tee-section to short wall) has adequate capacity to withstand the imposed loading.

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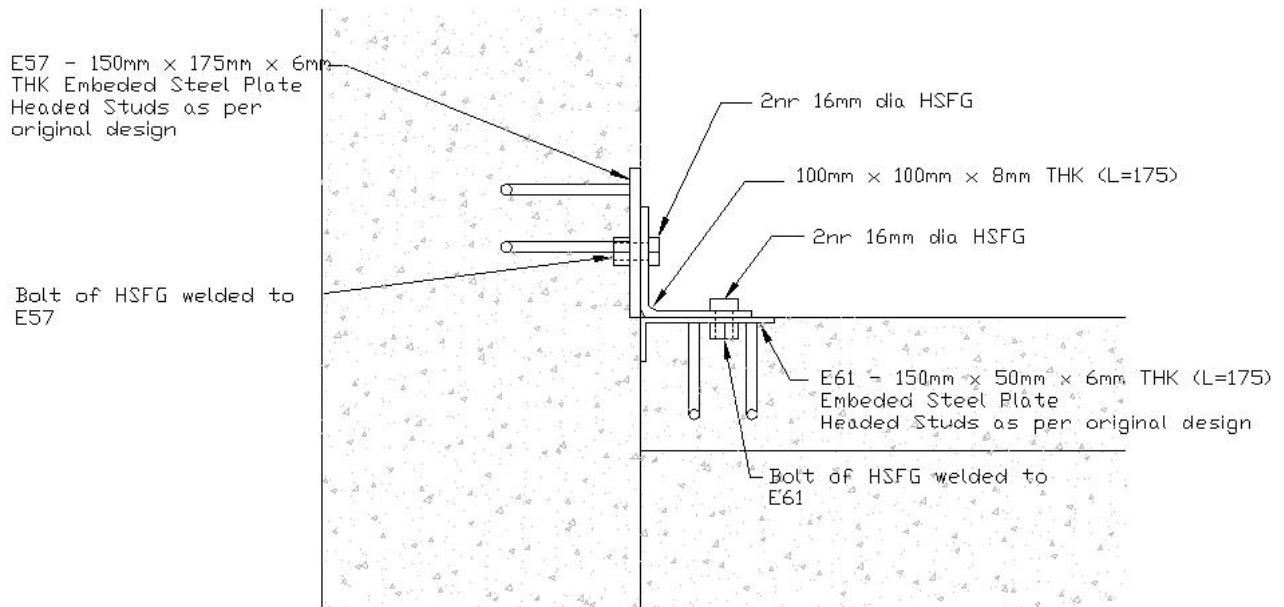
Roof Panel to Shear Wall Connection (Flange to Long wall)

The proposed arrangement of the roof panel to shear wall bolted connection (Flange to long wall) is shown below. As per the original design, it is proposed to use these fixings at 500mm centres.

It is proposed to use a 100mm x 100mm x 8mm equal angle bolted to the steel sections embedded into the precast concrete roof and wall panels. The angles will be bolted to the embedded steel sections using 16mm dia High Strength Friction Grip Bolts (HSFG).

As shown below, the bolts of the HSFG anchors will be welded to the inside of the embedded steel sections, to stop rotation of the bolt following installation.

Both the embedded steel section into the roof (member E61) and the wall (member E57) are as for the original welded connections, however, member E57 is located 50mm higher in the wall.



Shear Capacity of the Proposed High Strength Friction Grip Bolts (into Roof Slab)

The shear capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 16mm dia High Strength Friction Grip Bolt

$$V_{cap.bolt.roof3} := 45.6 \text{ kN}$$

Shear Capacity of the Bolted Connection

$$V_{cap.roof3} := 2 \cdot V_{cap.bolt.roof3}$$

$$V_{cap.roof3} = 91.2 \text{ kN}$$

Factor of Safety against Shear Failure of the High Strength Friction Grip Bolt

$$FOS_{roof.bolt3} := \frac{V_{cap.roof3}}{Load_{roof3}}$$

$$FOS_{roof.bolt3} = 2.519$$

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Shear Capacity of the Anchorage of the Studs into the Precast Concrete Roof Panels

As previously, the capacity of the anchorage of headed stud into the roof panel will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in shear,
- 2) concrete breakout strength of anchor in shear,
- 3) concrete pryout strength of anchor in shear.

Shear Strength of Anchor in Steel (cl. D.6.1)

Effective cross-sectional area of an anchor	$A_{se} := 84.3 \cdot \text{mm}^2$	
Specified tensile strength of anchor steel	$f_{ut} := 620 \cdot \text{MPa}$	
Specified yield strength of anchor steel	$f_y := 420 \cdot \text{MPa}$	
Limitations on tensile strength of anchor steel	$f_{ut} := \min(f_{ut}, 1.9 \cdot f_y, 125000 \cdot \text{psi})$	$f_{ut} = 620 \cdot \text{MPa}$
Nominal Strength of Headed Studs in Shear	$V_s := 4 \cdot A_{se} \cdot f_{ut}$	$V_s = 209.064 \cdot \text{kN}$
Factor of Safety against Shear Strength Failure of the Studs	$\text{FOS}_{\text{roof.steel.shear2}} := \frac{V_s}{\text{Load}_{\text{roof3}}}$	$\text{FOS}_{\text{roof.steel.shear2}} = 5.775$

Shear Strength of Concrete in Breakout (cl. D.6.2)

The anchorage arrangement is reinforced, which will prevent concrete breakout since the rebar will effectively attract the shear loading.

It is therefore considered inappropriate to use the methodology defined within ACI 318-02, Clause D.6.2 for the determination of concrete breakout strength, due to the reinforcement in the roof slabs.

Shear Concrete Pryout Strength of Anchor (cl. D.6.3)

Effective anchor embedment depth	$h_{ef} := 150 \cdot \text{mm}$	$h_{ef} = 5.906 \cdot \text{in}$
$h_{ef} > 2.5 \cdot \text{in}$, therefore,	$k_{cp} := 2$	
Given the reinforced anchorage arrangement, it is considered appropriate to define the Nominal concrete breakout strength as being equal to the Basic concrete breakout strength (i.e. $N_{cb} = N_b$).		
ACI 318-02 constant, for cast-in-anchors	$k := 24$	
Compressive Strength of Concrete	$f_c := 35 \cdot \text{N} \cdot \text{mm}^{-2}$	
Basic concrete breakout strength (cl. D.5.2.2)	$N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$	$N_b = 109.159 \cdot \text{kN}$
Nominal concrete breakout strength (cl. D.5.2.1)	$N_{cb} := N_b$	$N_{cb} = 109.159 \cdot \text{kN}$

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Nominal Pryout Strength

$$V_{cp} := k_{cp} \cdot N_{cb}$$

$$V_{cp} = 218.317 \text{ kN}$$

Factor of Safety against Shear Concrete Pryout Strength Failure of the Studs

$$FOS_{\text{roof.pry2}} := \frac{V_{cp}}{\text{Load}_{\text{roof3}}}$$

$$FOS_{\text{roof.pry2}} = 6.031$$

Pullout Capacity of the Proposed High Strength Friction Grip Bolts (into Shear Wall)

The pullout capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 16mm dia High Strength Friction Grip Bolt

$$P_{\text{cap.bolt.roof}} := 82.9 \text{ kN}$$

Shear Capacity of the Bolted Connection

$$P_{\text{cap.roof}} := 2 \cdot P_{\text{cap.bolt.roof}}$$

$$P_{\text{cap.roof}} = 165.8 \text{ kN}$$

Factor of Safety against Shear Failure of the High Strength Friction Grip Bolt

$$FOS_{\text{roof.pull2}} := \frac{P_{\text{cap.roof}}}{\text{Load}_{\text{roof3}}}$$

$$FOS_{\text{roof.pull2}} = 4.58$$

Pullout Capacity of the Anchorage of the Studs into the Precast Concrete Wall Panels

The pullout capacity of the anchorage of headed stud into the wall panel will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in tension,
- 2) concrete breakout strength of anchor in tension,
- 3) pullout strength of the anchor in tension.

Tensile Strength of Anchor in Steel (cl. D.5.1)

Effective cross-sectional area of an anchor

$$A_{se} := 84.3 \text{ mm}^2$$

Specified tensile strength of anchor steel

$$f_{ut} := 620 \text{ MPa}$$

Specified yield strength of anchor steel

$$f_y := 420 \text{ MPa}$$

Limitations on tensile strength of anchor steel

$$f_{ut} := \min(f_{ut}, 1.9 \cdot f_y, 125000 \text{ psi})$$

$$f_{ut} = 6.2 \times 10^8 \text{ Pa}$$

Nominal Strength of Headed Studs in Tension

$$N_s := 4 \cdot A_{se} \cdot f_{ut}$$

$$N_s = 209.064 \text{ kN}$$

Factor of Safety against Shear Strength Failure of the Studs

$$FOS_{\text{roof.steel.tensile2}} := \frac{N_s}{\text{Load}_{\text{roof2}}}$$

$$FOS_{\text{roof.steel.tensile2}} = 2.442$$

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Tensile Strength of Concrete in Breakout (cl. D.5.2)

Effective anchor embedment depth	$h_{ef} := 150 \cdot \text{mm}$	
Projected concrete failure area of group of anchors	$A_N := (2 \cdot 1.5 \cdot h_{ef} + 90 \cdot \text{mm}) \cdot (2 \cdot 1.5 \cdot h_{ef} + 115 \cdot \text{mm})$	$A_N = 0.305 \text{ m}^2$
Projected concrete failure area a single anchor, when not limited be edge distance or spacing	$A_{No} := 9 \cdot h_{ef}^2$	$A_{No} = 0.202 \text{ m}^2$
Minimum edge distance	$c := 200 \cdot \text{mm}$	
Modification factor for edge effects	$\psi_2 := 0.7 + 0.3 \cdot \frac{c}{1.5 \cdot h_{ef}}$	$\psi_2 = 0.967$
Modification factor for cracking	$\psi_3 := 1.0$	(assume cracked concrete)
ACI 318-02 constant, for cast-in-anchors	$k = 24$	
Compressive Strength of Concrete	$f_c = 35 \text{ N} \cdot \text{mm}^{-2}$	
Basic concrete breakout strength	$N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$	$N_b = 109.159 \text{ kN}$
Nominal concrete breakout strength in tension	$N_{cb} := \frac{A_N}{A_{No}} \cdot \psi_2 \cdot \psi_3 \cdot N_b$	$N_{cb} = 158.984 \text{ kN}$
Factor of Safety against Tensile Concrete Breakout of the Anchors	$\text{FOS}_{\text{roof.break2}} := \frac{N_{cb}}{\text{Load}_{\text{roof3}}}$	$\text{FOS}_{\text{roof.break2}} = 4.392$

Tensile Strength of Anchor in Pullout (cl. D.5.3)

Modification factor for cracking	$\psi_4 := 1.0$	(assume cracked concrete)
Distance from inner surface of the bolt shaft to outer tip of J-bolt	$e_h := 30 \cdot \text{mm}$	
Shaft diameter of J-bolt	$d_o := 12 \cdot \text{mm}$	
Pullout strength of a single hooked bolt	$N_p := 0.9 \cdot f_c \cdot e_h \cdot d_o$	$N_p = 11.34 \text{ kN}$
Nominal pullout strength of an anchor in tension	$N_{pn} := \psi_4 \cdot N_p$	$N_{pn} = 11.34 \text{ kN}$
Factor of Safety against Tensile Failure of Anchor in Pullout	$\text{FOS}_{\text{roof.pullout2}} := \frac{4 \cdot N_{pn}}{\text{Load}_{\text{roof3}}}$	$\text{FOS}_{\text{roof.pullout2}} = 1.253$

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Summary of Bolted Precast Concrete Roof Panel to Shear Wall Connection (Flange to Long wall)

Minimum Factor of Safety for Bolted Connection

$$FOS_{roof3} := \min(FOS_{roof.bolt3}, FOS_{roof.steel.shear2}, FOS_{roof.pry2}, FOS_{roof.pull2}, FOS_{roof.steel.tensile2}, FOS_{roof.break2}, FOS_{roof.f})$$

$$FOS_{roof3} = 1.253$$

This calculation has demonstrated that the proposed bolted connection of the precast concrete wall panel to roof panel (flange to long wall) has adequate capacity to withstand the imposed loading.

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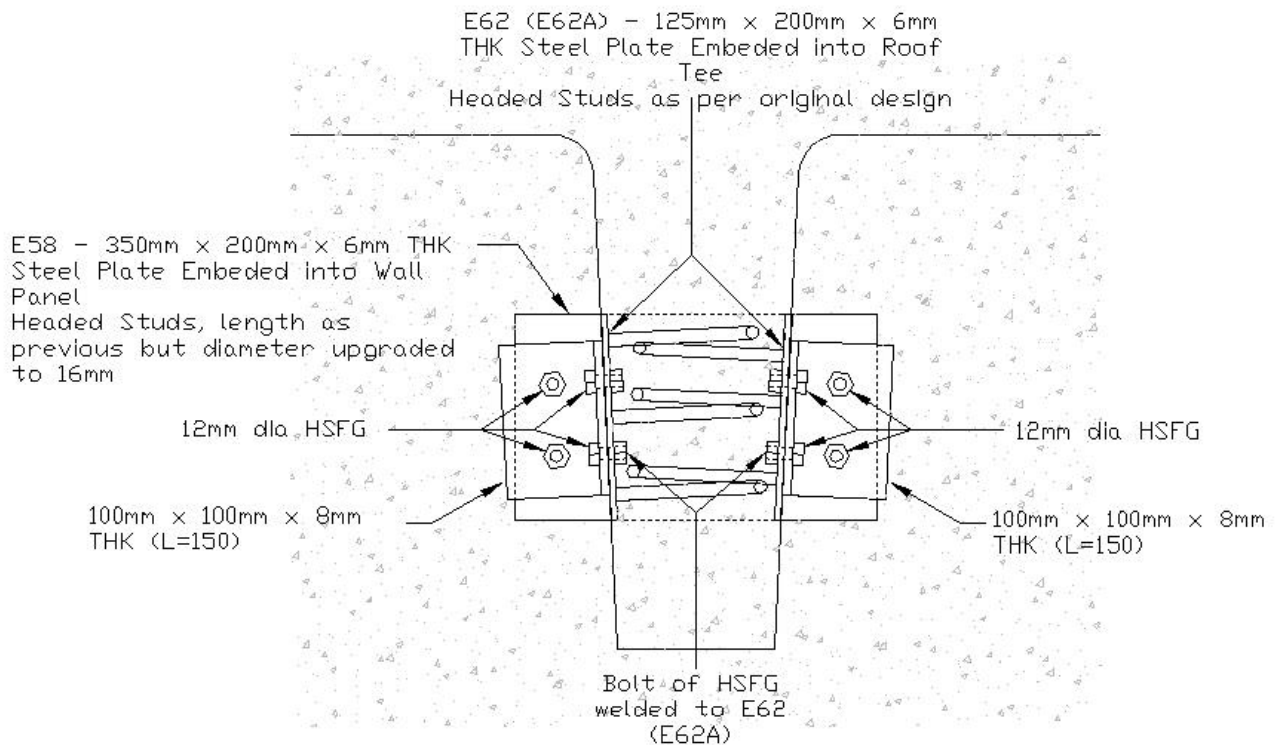
Roof Panel to Shear Wall Connection (Web to Long wall)

The proposed arrangement of the roof panel to shear wall bolted connection (Web to long wall) is shown below. As per the original design, it is proposed to use these fixings either side of the roof t-slab web.

It is proposed to use a 100mm x 100mm x 8mm equal angle, on each side of the roof web, bolted to the steel sections embedded into the precast concrete roof web and wall panels. The angles will be bolted to the embedded steel sections using 12mm dia High Strength Friction Grip Bolts (HSFG).

As shown below, the bolts of the HSFG anchors will be welded to the inside of the embedded steel sections, to stop rotation of the bolt following installation.

The embedded steel sections into the roof (members E62 & E62A) are as for the original welded connections. However, the embedded steel sections into the wall (member E58) will be extended to 350mm long, and the anchorage has been upgraded to 16mm dia anchor studs.



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Shear Capacity of the Proposed High Strength Friction Grip Bolts (into Roof Slab)

The shear capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 12mm dia
High Strength Friction Grip Bolt

$$V_{\text{cap.bolt.roof}} := 24.5 \text{ kN}$$

Shear Capacity of the Bolted Connection

$$V_{\text{cap.roof}} := 2 \cdot V_{\text{cap.bolt.roof}}$$

$$V_{\text{cap.roof}} = 49 \text{ kN}$$

Factor of Safety against Shear Failure of the
High Strength Friction Grip Bolt

$$\text{FOS}_{\text{roof.bolt4}} := \frac{V_{\text{cap.roof}}}{0.5 \cdot \text{Load}_{\text{roof4}}}$$

$$\text{FOS}_{\text{roof.bolt4}} = 2.165$$

Shear Capacity of the Anchorage of the Studs into the Precast Concrete Roof Panels

As previously, the capacity of the anchorage of headed stud into the roof panel will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in shear,
- 2) concrete breakout strength of anchor in shear,
- 3) concrete pryout strength of anchor in shear.

Shear Strength of Anchor in Steel (cl. D.6.1)

Effective cross-sectional area of an anchor

$$A_{\text{se}} := 84.3 \cdot \text{mm}^2$$

Specified tensile strength of anchor steel

$$f_{\text{ut}} := 620 \text{ MPa}$$

Specified yield strength of anchor steel

$$f_y := 420 \text{ MPa}$$

Limitations on tensile strength of anchor steel

$$f_{\text{ut}} := \min(f_{\text{ut}}, 1.9 \cdot f_y, 125000 \cdot \text{psi}) \quad f_{\text{ut}} = 620 \text{ MPa}$$

Nominal Strength of Headed Studs in Shear

$$V_s := 3 \cdot A_{\text{se}} \cdot f_{\text{ut}} \quad V_s = 156.798 \text{ kN}$$

Factor of Safety against Shear Strength Failure
of the Studs

$$\text{FOS}_{\text{roof.steel.shear3}} := \frac{V_s}{0.5 \cdot \text{Load}_{\text{roof4}}} \quad \text{FOS}_{\text{roof.steel.shear3}} = 6.929$$

Shear Strength of Concrete in Breakout (cl. D.6.2)

The anchorage arrangement is reinforced, which will prevent concrete breakout since the rebar will effectively attract the shear loading.

It is therefore considered inappropriate to use the methodology defined within ACI 318-02, Clause D.6.2 for the determination of concrete breakout strength, due to the reinforcement in the roof slabs.

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Shear Concrete Pryout Strength of Anchor (cl. D.6.3)

Effective anchor embedment depth $h_{ef} := 140 \cdot \text{mm}$ $h_{ef} = 5.512 \text{ in}$

$h_{ef} > 2.5 \text{ in}$, therefore, $k_{cp} := 2$

Given the reinforced anchorage arrangement, it is considered appropriate to define the Nominal concrete breakout strength as being equal to the Basic concrete breakout strength (i.e. $N_{cb} = N_b$).

ACI 318-02 constant, for cast-in-anchors $k := 24$

Compressive Strength of Concrete $f_c := 35 \cdot \text{N} \cdot \text{mm}^{-2}$

Basic concrete breakout strength (cl. D.5.2.2) $N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$ $N_b = 98.427 \text{ kN}$

Nominal concrete breakout strength (cl. D.5.2.1) $N_{cb} := N_b$ $N_{cb} = 98.427 \text{ kN}$

Nominal Pryout Strength $V_{cp} := k_{cp} \cdot N_{cb}$ $V_{cp} = 196.854 \text{ kN}$

Factor of Safety against Shear Concrete Pryout Strength Failure of the Studs $\text{FOS}_{\text{roof.pry3}} := \frac{V_{cp}}{0.5 \cdot \text{Load}_{\text{roof4}}}$ $\text{FOS}_{\text{roof.pry3}} = 8.699$

Pullout Capacity of the Proposed High Strength Friction Grip Bolts (into Shear Wall)

The pullout capacity of the proposed high strength friction grip bolts will be calculated using the allowable loads given in BS5950.

Single Shear Capacity of a 12mm dia High Strength Friction Grip Bolt $P_{\text{cap.bolt.roof}} := 44.5 \cdot \text{kN}$

Shear Capacity of the Bolted Connection $P_{\text{cap.roof}} := 2 \cdot P_{\text{cap.bolt.roof}}$ $P_{\text{cap.roof}} = 89 \text{ kN}$

Factor of Safety against Shear Failure of the High Strength Friction Grip Bolt $\text{FOS}_{\text{roof.pull3}} := \frac{P_{\text{cap.roof}}}{0.5 \cdot \text{Load}_{\text{roof4}}}$ $\text{FOS}_{\text{roof.pull3}} = 3.933$

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Pullout Capacity of the Anchorage of the Studs into the Precast Concrete Wall Panels

The pullout capacity of the anchorage of headed stud into the wall panel will be calculated using the methodology in ACI 318-02. The studs will be checked for;

- 1) steel strength of the anchor in tension,
- 2) concrete breakout strength of anchor in tension,
- 3) pullout strength of the anchor in tension.

Tensile Strength of Anchor in Steel (cl. D.5.1)

Effective cross-sectional area of an anchor	$A_{se} := 157 \cdot \text{mm}^2$	
Specified tensile strength of anchor steel	$f_{ut} := 620 \cdot \text{MPa}$	
Specified yield strength of anchor steel	$f_y := 420 \cdot \text{MPa}$	
Limitations on tensile strength of anchor steel	$f_{ut} := \min(f_{ut}, 1.9 \cdot f_y, 125000 \cdot \text{psi})$	$f_{ut} = 6.2 \times 10^8 \text{ Pa}$
Nominal Strength of Headed Studs in Tension	$N_s := 4 \cdot A_{se} \cdot f_{ut}$	$N_s = 389.36 \text{ kN}$
Factor of Safety against Shear Strength Failure of the Studs	$\text{FOS}_{\text{roof.steel.tensile3}} := \frac{N_s}{\text{Load}_{\text{roof4}}}$	$\text{FOS}_{\text{roof.steel.tensile3}} = 8.603$

Tensile Strength of Concrete in Breakout (cl. D.5.2)

Effective anchor embedment depth	$h_{ef} := 140 \cdot \text{mm}$	
Projected concrete failure area of group of anchors	$A_N := (2 \cdot 1.5 \cdot h_{ef} + 200 \cdot \text{mm}) \cdot (2 \cdot 1.5 \cdot h_{ef} + 120 \cdot \text{mm})$	$A_N = 0.335 \text{ m}^2$
Projected concrete failure area a single anchor, when not limited be edge distance or spacing	$A_{No} := 9 \cdot h_{ef}^2$	$A_{No} = 0.176 \text{ m}^2$
Minimum edge distance	$c := 200 \cdot \text{mm}$	
Modification factor for edge effects	$\psi_2 := 0.7 + 0.3 \cdot \frac{c}{1.5 \cdot h_{ef}}$	$\psi_2 = 0.986$
Modification factor for cracking	$\psi_3 := 1.0$	(assume cracked concrete)
ACI 318-02 constant, for cast-in-anchors	$k = 24$	
Compressive Strength of Concrete	$f_c = 35 \text{ N} \cdot \text{mm}^{-2}$	
Basic concrete breakout strength	$N_b := k \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \left(\frac{h_{ef}}{\text{in}} \right)^{1.5} \cdot \text{in}^2$	$N_b = 98.427 \text{ kN}$
Nominal concrete brekout strength in tension	$N_{cb} := \frac{A_N}{A_{No}} \cdot \psi_2 \cdot \psi_3 \cdot N_b$	$N_{cb} = 184.141 \text{ kN}$
Factor of Safety against Tensile Concrete Breakout of the Anchors	$\text{FOS}_{\text{roof.break3}} := \frac{N_{cb}}{\text{Load}_{\text{roof4}}}$	$\text{FOS}_{\text{roof.break3}} = 4.069$

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Tensile Strength of Anchor in Pullout (cl. D.5.3)

Modification factor for cracking	$\psi_4 := 1.0$	(assume cracked concrete)
Distance from inner surface of the bolt shaft to outer tip of J-bolt	$e_h := 30 \text{ mm}$	
Shaft diameter of J-bolt	$d_o := 16 \text{ mm}$	
Pullout strength of a single hooked bolt	$N_p := 0.9 \cdot f_c \cdot e_h \cdot d_o$	$N_p = 15.12 \text{ kN}$
Nominal pullout strength of an anchor in tension	$N_{pn} := \psi_4 \cdot N_p$	$N_{pn} = 15.12 \text{ kN}$
Factor of Safety against Tensile Failure of Anchor in Pullout	$FOS_{\text{roof, pullout3}} := \frac{4 \cdot N_{pn}}{\text{Load}_{\text{roof4}}}$	$FOS_{\text{roof, pullout3}} = 1.336$

Summary of Bolted Precast Concrete Roof Panel to Shear Wall Connection (Web to Long wall)

Minimum Factor of Safety for Bolted Connection

$$FOS_{\text{roof4}} := \min(FOS_{\text{roof, bolt4}}, FOS_{\text{roof, steel, shear3}}, FOS_{\text{roof, pull3}}, FOS_{\text{roof, steel, tensile3}}, FOS_{\text{roof, break3}}, FOS_{\text{roof, pullout3}})$$

$$FOS_{\text{roof4}} = 1.336$$

This calculation has demonstrated that the proposed bolted connection of the precast concrete wall panel to roof panel (web to long wall) has adequate capacity to withstand the imposed loading.

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Conclusion

This calculation has presented the justification for the re-design of some of the connection details of the modular blast resistant precast concrete substation for the Saudi Arabian Oil Company (Saudi Aramco).

The original design of the connections utilised a high number of welded connections. It was decided that, if possible, the number of welded connections need to be reduced by providing alternative bolted connections. This calculation has presented the justification calculations for these redesigned bolted connections.

The minimum factors of safety for the three connection details considered are listed below.

Precast Concrete Wall Panel to Wall Panel (Short Wall) Connection	$FOS_{\text{short.wall}} = 1.635$
Precast Concrete Wall Panel to Wall Panel (Long Wall) Connection	$FOS_{\text{long.wall}} = 4.78$
Precast Concrete Roof Slab to Roof Slab Connection	$FOS_{\text{roof}} = 1.885$
Precast Concrete Wall Panel to Foundation Connection	$FOS_{\text{found}} = 1.302$
Precast Concrete Roof Slab to Wall Panel (Short Wall) Connection	$FOS_{\text{roof2}} = 1.413$
Precast Concrete Roof Slab (T-section) to Wall Panel (Short wall) Connection	$FOS_{\text{roof3}} = 1.253$
Precast Concrete Roof Slab (Web section) to Wall Panel (Long wall) Connection	$FOS_{\text{roof4}} = 1.336$