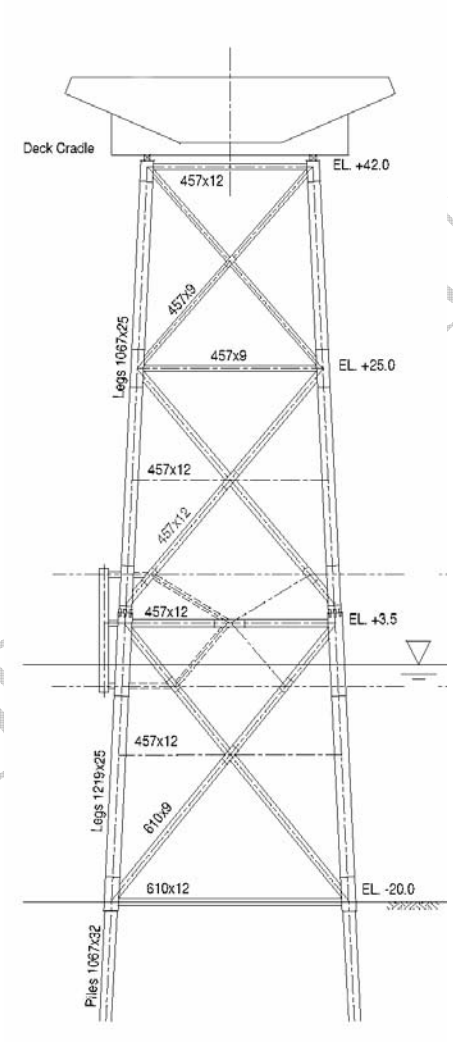
SFOBB – SAS

TEMPORARY TOWER DESIGN EXAMPLE



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T-Y-LIN INTERNATIONAL
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a joint venture

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Executive Summary

This Design Example is intended to provide a clarification of the design requirements indicated on the design drawings and special provisions, and serve as an example of the type of structures anticipated by the Engineer to have the sufficient strength and ductility for the intended purpose.

Two representative tower structures are included; these are: Tower C located at the tip of Yerba Buena Island and Tower G located within the shipping channel in 20m water depth.

Both towers are 3-dimensional steel cross-braced space frames, and are designed to be reliable and ductile. The basic approach used for Tower C was to design a tubular steel structure with bolted connections, which could be trucked to and assembled on site with the use of a crawler crane. The leg segments could be bolted together with splice flanges located near inflection points and have pre-installed gusset plates welded to them for the bracing members to be bolted onto. Tower C was analyzed as part of a larger structure comprised of Towers ABC, both east and west bound, all connected together, and as a single stand-alone tower. The benefit of each of these configurations should be further evaluated.

The basic approach used for Temporary Tower G was to design it as a tubular steel jacket with pin piles driven through the legs. Tower G is actually shown as a two-part tower whereby the base is first lifted into position on the seabed. After the piles are driven to final penetration, they are then welded to the tower legs utilizing shim plates and simple fillet welds in shear. Once the welding is complete, the upper section of the tower is stabbed into the pile tops and welded.

Both structures were analyzed linearly for the specified load combinations as well as with non-linear pushover analyses to verify ductility.

It should be noted that this Design Example is a first iteration in the design process and requires further refining in order to meet the full intent of the specified design requirements. As noted, there are several approaches to meet the requirements of the plans and special provisions.

DISCLAMER:
The key words for the Temporary Tower Design Examples are strictly that: Design Examples. They are presented in an attempt to show one example of how the drawings and special provisions can be applied, as well as the other codes and specifications referenced therein. These designs are <u>NOT FOR CONSTRUCTION</u>. The contractor shall perform a new and separate design to suit his construction preference as well as the final bridge erection sequence. The contractor is encouraged to use his experience in selecting the proper Temporary Tower structures and arrangement to suit his means and methods.

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Temporary Tower Design Example

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1. INTRODUCTION

This Design Example is intended to provide a clarification of the design requirements indicated on the design drawings and special provisions, and serve as an example of the type of structures anticipated by the Engineer to have the sufficient strength and ductility for the intended purpose.

The drawings and specifications were reviewed and two representative tower structures were selected, these are:

- Tower C located at the tip of Yerba Buena Island.
- Tower G located within the shipping channel in 20m water depth.

This Temporary Tower Design Example is intended to present a design of what the towers might look like, the procedure used to determine the design requirements, and the analysis required for design. The scope of this design example and the level of detail presented in this report does not mimic the actual effort required to provide a full set of design drawings required to construct these towers.

Assumptions/Limitations

Numerous assumptions have been made during the course of this Design Example as described within the appropriate sections of this report and will not be repeated herein.

- As noted on Construction Sequence No. 1 Drawing, the graphic representation of the Temporary Towers shown on the drawings is conceptual and merely an approximate indication of what they might look like; however, for the purpose of this Design Example, the tower locations and general configurations were not modified. It was also assumed that the deck lifts are to be performed in the order indicated and that their weights are final, and that the bearing locations and their final reactions shown on the drawings are also final.

- For purposes of this Design Example, a limited number of steps of the construction sequence were evaluated This Design Example attempted to locate the key stages that may govern overall tower design, including skew lifts during the initial deck erection. Other load conditions shall be investigated by the contractor, for their bid, including the condition where the entire deck is in-place and all cross beams are connected, prior to load transfer.

- Also, many of the details required to fabricate these towers have not been developed beyond a conceptual level and thus the sketches presented in this report should be considered conceptual and very preliminary.

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2. TOWER CONFIGURATIONS

Both Temporary Towers C and G consist of cross-braced space frames. In a cross-braced frame, the compression members offset and balance the tension members within a given bay. As the structure is displaced laterally, beyond the design limit, the compression members buckle and transfer load through the horizontal braces to the tension members, which then yield, predominately in an axial mode. The member end moments are typically insufficient in this type of framing system to form hinges prior to the compression members failing in Euler buckling. It should be noted that the horizontal framing members must therefore be capable of transferring the compression force over to the tension members without failing and that the connections must be designed to resist the tension yield capacity of the braces. This type of framing system thus provides a reliable and ductile structure.

Ductility is defined as the ratio of ultimate deformation to the deformation at yield. Ductile response of structural components is characterized by several cycles of inelastic deformation without significant degradation of strength or stiffness. The most desirable type of ductile response in bridge systems is sustained hysteric force-deformation cycles that dissipate energy. This type of response can be generated either internally, within the structural members, by the formation of flexural plastic hinges or externally with isolation bearings or external dampers.

2.1 Tower C

The approach used for Temporary Tower C was to design it as a tubular steel structure, which could be trucked and assembled on site with the use of a crawler or tower crane. It is assumed that all tower and truss joints are to be bolted. The leg segments could be bolted together with splice flanges located near inflection points and have pre-installed gusset plates welded to them for the bracing members to be bolted onto. It is intended that the gusset plates actually penetrate through the legs in the transverse direction in order to provide load continuity through the joints from the east to the west towers. The longitudinal gussets are thus welded only to the face of the leg perpendicular to the through plate. This type of detail with one through-plate allows significantly larger forces to be applied to the joint since the leg tubular cannot deform as a simple ring due to the presence of the through plate.

If the contractor elects to utilize the configuration indicated on the drawings, then the connecting truss chords and bracing could also be bolted together and to the towers. The truss could have a skid plate welded to its top chord in order to be able to slide the deck sections along it from offshore. Short in-fill skid plates will be required to be welded across the bolted top truss chord joints for sliding continuity. Once the deck sections are in place, there may be an advantage to disconnect all of the cross-connecting trusses in order to increase tower flexibility and thus reduce seismic response.

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The configuration for Tower C was based on the configuration depicted on the design drawings as well as the box girder weights and bearing footprint. A tower leg was located at or very close to each of the bearing locations. Tower C was initially configured as part of Towers A, B and C and later as a stand-alone tower (see Figures 2-1 and 2-2). The variable height of the westbound Towers A and B down the side slope has not been addressed in this Design Example; and may have an influence on the design of Tower C, when connected all together. The pairs of combined Towers A, B & C are also cross connected by means of a set of cross braced trusses spanning between the east and west bound structures. There may be an advantage of not connecting the pairs of towers, or each other. Typical bolted connection details are conceptually shown on Figure 2-3.

It is intended to set Tower C (as well as A & B) on a set of prepared concrete footings set into or on the YBI rock. Prior to erecting the tower, the concrete footings will be cast either onto the bedrock directly or into pits excavated into the hillside down to bedrock. A set of tie-downs may be required to develop shear friction between the footing and bedrock. A series of anchor bolts could be located around the legs, set into the footing. Tower C alone does develop small tension forces, which are easily resisted by the anchor bolts, but a small shear key could also be provided below the leg base plate.

Isolation concepts can significantly reduce seismic demands on the temporary tower structures. In this design example isolation was considered but not fully developed. One alternative was to supply a slip plane at the bearing locations such that the deck would only provide a small percentage of its mass to seismic excitation. A relatively simple bearing including a low friction material could be used. Keepers would be required similar to those shown at the Tower G cradle, to keep the deck structure from sliding off the support bearings; however, a yielding fuse element should also be incorporated at each bearing location to prevent significant impact loads to the deck.

2.2 Tower G

The basic approach used for Temporary Tower G was to design it as a tubular steel braced jacket with pin piles driven through the jacket legs (see Figure 2-4). It is assumed that the tower and piles will be fabricated in the Gulf Coast (Texas or Louisiana) or the Far East and barged to California either through the Panama Canal or across the Pacific. The structure has thus been sized to comply with Panamax requirements.

The layout for Tower G was developed as a 6-leg tubular jacket structure with a leg and pile through each leg at each of the bearing locations. Temporary Tower G is approximately 65m (213 ft) high, with a 14m x 30m (46 ft x 98 ft) bearing footprint at the top. The jacket top plan is thus 14m x 30m while the base plan dimensions were selected to provide approximately the same overturning resistance in both orthogonal directions.

Tower G has three principal bays between the mudline and the bearing deck, with a simple welded tower/pile connection occurring just above the waterline. This connection is comprised of a set of 6-8 shim plates fitted concentrically between the leg and pile.

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The leg has a series of open slots and the vertical edges of the shim plates and slots receive a simple field-performed vertical fillet weld. These welds provide both axial and moment transfer capability. If the tower were to be built in one piece, then the pin piles would be required to connect to the tower at the top with a similar leg to pile connection. This would require significantly more pile material; however, it may be possible to reduce the thickness of the legs above the impact zone, since the leg section will be predominately in tension.

The cross bracing configuration was chosen to provide significant lateral seismic capacity at an angle of about 45 degrees. The leg diameter was dictated by the pile diameter, which was based on an approximation of pile demand due to earthquake loads, assuming a pile penetration of about 52m (170ft). The bracing diameters were estimated based upon general engineering experience, using L/r ratios of about 75-90 and D/t ratios of about 48.

It is intended that a series of eight shim plates are to be welded to the inside of the tower legs at Elevations –20m and +3.5m in order to limit the amount of lateral movement between the tower leg and pile before the piles begin to resist the lateral loads. The shim to pile clearance should be as tight as possible taking both the leg and pile tubular out-of-roundness tolerances into consideration.

The box girder sections are intended to rest in a series of cradles, which in turn are supported on bearings. This cradle is conceptually shown on Figure 2-5.

The Design Example for Tower G also includes a simple fender system shown schematically on Figure 2-6, which is intended to deflect a runaway barge. The jacket structure has been designed for the full impact load. It is intended that the jacket leg above Elev. +3.5m be grouted within the Impact zone to preclude local leg collapse. The leg section below Elev. +3.5m is actually reinforced by the pile inside the leg; however these should also be grouted together within the Impact Zone to prevent local damage to the leg below the waterline. Additional local impact bracing has been added to the overall tower framing.

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3. DESIGN CRITERIA

3.1 Codes, Standards, Specifications

All references in the Special Provisions are from Section 10-1.41, TEMPORARY TOWERS, unless noted otherwise.

Applicable Codes/Specifications:
CalTrans Standard Specifications, 1999
CalTrans Special Provisions, Contract 04-0120F4
CalTrans Falsework Manual, Rev. 32, November 2001

AISC-LRFD, 1999 for all members and connections except tubular members
API RP2A-LRFD, July 1993 for tubular sections and connections (and pipe piles)
AASHTO-LRFD Bridge Construction Specification, 2nd Ed. (Concrete only)
ANSI/ASCE 7-95 (Wind loads only)

3.2 Independent Loads Cases

The independent load cases were assembled following conventional practice and include the following:

Dead Load

The Dead Load includes the weight of the Temporary Tower structure, its piles, plus the weight of the supported deck superstructure in the various stages of erection.

The Bearing locations are indicated on Sheet Temporary Structure No. 2 (see Figure 3.1), and the Bearing Loads are shown on Sheet Construction Load No. 1 (see Figure 3.2), indicated in Table 1 as an envelope of vertical forces. This envelope of loads is treated as Dead Load for this Design Example (see also Construction Loads/Stages), below.

Live Load

The Live Load is, as specified in Section 51-1.06, "Falsework" of the Standard Specifications, to be the anticipated Construction Loads or a uniform load of 960 N/m^2 over the entire deck area plus a perimeter load of 1100 N/m along deck edge. See Figure 3-3 for a typical deck section. Since it is not known what construction loads may be used, the deck uniform load plus perimeter loads have been used. This amounts to about 1500kN or 150 Tonnes for a 50m long by 27.9m wide deck section was used for the purpose of this Design Example. Actual equipment weights used during the erection of the bridge should be used, if they govern.

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Vessel Impact Load

The Impact Load for Tower G is provided in the Special Provisions as 7.6 MN from any direction and applied between Elev. +7.6m and –1.8m (see Figure 3-4). The resulting moment on Tower G when the Impact Load is applied at Elev. +3.5m is 179MN-m, for comparison with the other load cases.

Seismic Load

The seismic design response spectrum was provided on Sheet Construction Sequence No. 2 (see Figure 3-5). The spectrum peaks at 0.7g at a 0.25sec period, drops off to 0.2g at 1.0sec, then to 0.1g at 2.0sec, 0.06g at 3.0 sec and 0.05g at 4.0 sec and beyond.

The Earthquake analyses were performed from the two orthogonal directions and combined as specified in the Special Provisions as follows:

1.0 X-direction + 0.3 Y direction
0.3 X-direction + 1.0 Y direction

Additionally, two other cases were analyzed with a negative factor on the primary directions in order to compensate for the lack of directionality in a modal analysis, as follows:

-1.0 X-direction + 0.3 Y direction
0.3 X-direction - 1.0 Y direction

Other "loads" to be considered in combination with the appropriate load cases include added mass and member buoyancy, if applicable (for Tower G only). For the purpose of the seismic analysis, the added mass has been conservatively assumed to equal the displaced water of the structure below the waterline. Buoyancy was not applied since the members were assumed to be flooded.

The maximum seismic force and moment on Tower G was determined to be about 4.0MN and 196MN-m, respectively, for comparison to the other load cases.

Wind Load

ASCE 7-95 specifies an 85MPH Wind Gust @ +10m whereby Section 10-1.59 of the Special Provisions specifies a 77MPH 1-hr sustained Wind (100 MPH 3-sec gust) @ El. +50m. The 100MPH Wind Gust velocity stated in the Special Provisions thus govern the wind velocity determination.

The Wind Load was based on a wind gust velocity of 100 MPH and the wind force determination procedure given in ASCE 7-95, utilizing the exposure/height, the gust, and the shape factors. The 100MPH velocity at El. +50m was adjusted down to the +10m reference elevation; based on the square of the velocities and the ratio of height

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coefficients provided in Table 6-3 (ASCE 7-95). (Another way of determining the wind velocity pressure is to use the 100MPH wind velocity directly in the ASCE formula, but omit the exposure coefficient Kz since the reference elevation is already 50m at the deck level. However, the wind loads on the remaining portion of the structure would then require modification by adjusting the exposure coefficient downwards in proportion to its height below the new reference elevation of 50m.) The importance factor of 1.15 was used, as specified, along with the appropriate exposure coefficients associated with exposure D (open water). A gust factor of 0.85 was used in accordance with ASCE Section 6.6 while the shape factor used for the projected flat surface of the box girder was 1.5 (ASCE Table 6-8), and 0.8 for the tubular tower structures (ASCE Table 6-9).

The wind force thus generated for the 5.5m high box girder section based on projected area is 2.08kN/m^2, or 11.4kN/m of deck length. The wind force generated on the tubular tower structures vary with height from 1.44kN/m^2 at the top down to 0.72kN/m^2 at the waterline or groundline, as based on the Falsework criteria (see Figure 3-6).

It should be noted that the local wind load on the jacket structure itself is actually governed by the more stringent "Falsework" criteria of 30psf minimum, which was conservatively used for this Design Example; however, in order to generate a consistent set of wind loads on the entire structure, the lower values generated by ASCE could be used for both deck and tower, since this would generate a higher overall consistent loading condition as opposed to applying only the Falsework criteria, to both the deck and jacket structures. The more stringent of the two criteria were used for this Design Example.

Wave Load

A 2m high, 6sec wave was assumed, based on the limited fetch area to the site behind YBI and TI, and limited water depth in the vicinity of the bridge. Extensive analyses could be performed whereby a wind generated deeper-water wave height is statistically produced and transformed to the site location; however, since the total wave force is relatively low, the assumed wave height and period was conservatively used.

The wave force was generated using Linear wave theory, since the wavelength to water depth ratio is relatively high. The resulting wavelength is about 55m. The wave was stepped past a representative 1m-diameter tubular in order to combine the drag and inertia forces and thus determine the maximum total lateral force and overturning moment. This generally occurs at a wave phase angle of about 60-75 degrees. Based on drag and inertia coefficients of 0.65 and 1.6 (ref API RP2A), respectively, the maximum wave force and overturning moment on the representative tubular are 12.4kN and 164kN-m, respectively. This unit load was used along with the total projected area of the jacket structure in the water of about 350m^2 to determine the total wave force on the jacket. The maximum total wave force and moment on Tower G was thus determined to be 0.22MN and 2.9MN-m, respectively. Note that the area of members not normal to the direction of the wave force was adjusted to resolve the normal force component into the horizontal direction.

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Current Load

A 3-knot surface current was conservatively assumed based on the maximum possible Bay Area Tide and Current tables for the area (Note: The Vessel Collision Report indicates a uniform 2 knot design current is sufficient for Impact design). The surface current velocity profile with depth was developed utilizing a $1/7^{th}$ power distribution. The resulting current force and moment, on a representative 1m-diameter tubular, are 11.9kN and 137kN-m, respectively (similar to the wave force). Based on the total jacket area of about 350 m^2, the maximum total current force and moment on Tower G was thus determined to be 0.21MN and 2.4MN-m, respectively.

The wave and current forces for Tower G were generated using the Morrison equation provided in API RP2A. During final design, the wave and current kinematics should be combined prior to applying the Morrison equation in order to correctly account for their interaction.

Construction Loads/Stages

The deck erection sequence and interconnection options can greatly affect the assumed load cases. The following stages, among others, of construction may govern the design of the Temporary Towers:

1. Individual deck lifts 1 thru 9 on the appropriate towers
2. In-fill / heavy lifts 1, 2, 3 & 4 on the appropriate towers
3. Connecting the deck sections together in the longitudinal direction
4. Connecting the cross beams in the transverse direction
5. Connecting the deck sections to W2 and E2 cap beams

Of the various stages of the erection sequence shown on the drawings, only the following independent load cases have been analyzed (for the corresponding tower) for the purposes of this Design Example:

Tower C
Temporary Tower (TT)
TT plus Deck Lift #1 (830 Tonnes) on 4 bearing pads
TT plus Deck Lift #4 (459 Tonnes) on 4 bearing pads
TT plus Deck Lift #4 plus half of Heavy Lift #1 (830/2 = 415 Tonnes) = total 875 T
Envelope of Vertical Forces (see Table 1 on Sheet Construction Load No. 1)

Since Tower C could actually be part of a larger Tower ABC combination, an additional load case of the entire configuration would be required where the EB and WB deck sections are supported but not connected to each other and then later connected to each other. The latter case with the two decks connected is believed to be less onerous on both

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the tower structure as well as the deck structure and was thus not checked for the purpose of this Design Example.

Tower G
Temporary Tower (TT)
TT plus Deck Lift #9 (892 Tonnes) on 6 bearing pads
TT plus Deck Lift #9 plus half of Heavy Lift #3 (1285/2 = 643 Tonnes) = total 1535 T
TT plus Deck Lift #9 plus half of Heavy Lifts #3 & #4 = 2177 T
Envelope of Vertical Forces (see Table 1 on Sheet Construction Load No. 1)

While not performed in this Design Example, the final tower configuration should also be checked as part of the overall bridge erection sequence whereby the entire bridge deck is in place, where the EB and WB deck sections are supported on all of the temporary towers, but not connected to each other; and then additionally where they are connected to each other.

3.3 Load Combinations

The above independent load cases were then combined per the load combinations dictated in the Special Provisions as follows:

1.4 DL
1.1 DL + 1.3 LL
1.0 DL + 1.0 LL + 0.5 Wind + 1.0 Current + 1.0 Impact (Vessel)
1.0 DL + 1.0 LL + 1.0 EQ
1.0 DL + 1.0 LL +1.3 Wind +1.3Wave + 1.3Current

It is good design practice to check the temporary towers using 0.9 DL in the load combination to capture possible tension loads in vertical members and to account for potential dead load fluctuation.

3.4 Other Criteria

A minimum yield stress of 345MPa was assumed for all steel with an expected ultimate strength of about 485MPa.

The Special Provisions specified that “All vertical members shall remain elastic; bracing members may yield and piles may yield to 2% strain” for pushover analyses purposes.

The elastic criteria are met by applying load factors to the load combinations analyzed and capacity reductions factors during the member and joint code checks. Response Spectrum Analysis was used for seismic load. The impact analyses performed was also a linear analysis. The in-elastic criteria was applied to the non-linear pushover analyses whereby the diagonal members may buckle and/or yield and the piles may exhibit

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inelastic properties when pushed laterally to 150% of the earthquake displacement levels. A pushover analyses was performed for Towers ABC, Tower C alone, and Tower G, with and without piles for this Design Example to verify the ductility of the Temporary Towers.

The Special Provisions also state that the design calculations shall demonstrate that the maximum design settlement shall not exceed 25mm at the mudline for the governing design load combination.

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4. DESIGN METHODOLOGY

4.1 Procedures

Sheets 967 thru 978 of 1204 of the Temporary Tower and Section 10-1.41, Page 227 thru 234 of the Special Provisions, including those items referenced specifically therein are the primary source for information used to establish the procedure and design methodology for the Temporary Towers. In addition, referenced items also used were: the Ship Collision Study, Feb. 14, 2000, the Geotechnical Foundation Report dated June 30, 2002, Section 51-1.06, "Falsework" of the Standard Specifications, and Section 10-1.59 "Steel Structures", subsection "Assembly" of the Special Provisions as well as all of the specifically listed Codes, Specifications and Recommended Practice.

Hand sketches of the tower configurations and conceptual details were developed in order to define the structural configuration and perform the preliminary hand calculations and spreadsheet analyses.

These hand calculations and spreadsheets were used to determine preliminary global forces and moments, foundation reactions, member sizes, pile diameter and thickness for Tower G, as well as a preliminary pile penetration.

Computer models for the various tower configurations were developed and analyzed using the SAP2000 computer suite (including the API code checks). Elastic beam elements were used to model the piles for Tower G since these best represent the overall soil pile behavior during the maximum load conditions. The pile elements were modeled down to the point of fixity; however, a linear vertical spring was also provided at each pile to represent the axial soil-pile behavior below that depth. This modeling is believed to be reasonable due to the soft Young Bay Mud not providing any significant lateral support within the depth of the dummy pile. The only thing missing is the very small lateral displacements and rotations at the assumed point of fixity that would be present in a non-linear analysis. P-Δ effects were included in all lateral analyses performed.

The stick models were defined in SAP and the independent load cases were set up. The Impact load was applied as a node load to one of the tower legs at Elev. +3.5m from three directions: longitudinal, transverse and diagonal. Hand and spreadsheet calculations were performed to check the Impact bracing and nodes at the other elevations between –1.8m and +7.6m. A modal analysis was performed extracting the first 15 modes using the specified spectrum, and a Response Spectrum Analysis was performed. The wind loads for the various heights were calculated using a spreadsheet, in accordance with ASCE 7-95 and falsework criteria, and conservatively applied to the deck and tower nodes individually. The wave loads were generated utilizing a linear wave theory and the current loads were calculated using a $1/7^{th}$ power distribution with depth. The wave and current loads were also applied to the applicable tower nodes separately.

A global analysis was performed and the independent load cases were then factored and combined per the specified load combinations. The results were then post-processed

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utilizing the automatic API-LRFD member and joint code check feature. It should be noted that the results of any automated feature of a computer program should be checked by hand calculations. For this Design Example, the API suite by SAP led to incorrect calculations of the unbraced lengths and joint type and had to be manually corrected.

A typical bolted joint for Tower C was hand calculated for the maximum demand, as an example, as well as a simplified leg base connection and a footing check. Member and joint checks for Tower G were also performed for the maximum demand by hand calculations as an example. It should be noted that in order to meet the pushover requirements, the joints are required to be designed for the tension yield capacity of the members framing into them. This should allow the joints to remain elastic.

4.2 Summary of Results

Vessel Impact generally governed Tower G, while Tower C is generally governed by the seismic demands.

Tower C

The Wind was assumed to act only in the broadside direction of the tower due to the orientation of Temporary Towers A, B & C. . It should also be noted that the prevailing winds are actually longitudinal, along the bridge structure, but have been conservatively assumed to act broadside to the supported deck structure and tower. The total wind force on the combined Towers ABC is about 1.5MN (EB or WB separately).

The first mode periods for the combined Towers ABC are just below 1.0 sec in both directions, which produces a lateral force equivalent to about 0.25g or about 12.5MN. Due to the overall footprint of the combination of the eastbound and westbound Towers ABC structures, the member forces and leg reactions are within a reasonable range for a structure of this height supporting this much weight.

The pushover capacity of Tower C as part of the combined Towers ABC is about two times the seismic displacement demand or about 125mm. Although this exceeds the specified criteria of 150% times the seismic demand, there is not much additional displacement capacity available. This may be due to the inherent complexity of the combined pairs of interconnected Towers ABC and their reduced periods and subsequent increased seismic demand. Isolation may reduce these forces but this was not done for this Design Example.

An analysis was performed on Tower C separately in order to determine if there was any merit in isolating it for this Design Example. It was thus determined that an individual tower designed to support the appropriate bearing loads is more flexible than the combined Towers AB&C and thus responds less to seismic excitation; however, the legs will resist higher axial forces. If this were to be implemented, the need for the

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interconnecting trusses between the east and west tower group could be eliminated. This finding, though based on a very brief analysis, warrants further consideration. It may also be possible to provide only temporary trusses between Towers A, B & C for the purpose of skidding deck sections from one end to the other. There may be an advantage to then removing the interconnecting trusses in order to reduce the tower group stiffness and thus reduce seismic excitation.

The first mode period of Tower C alone is about 1.6sec and 1.4sec in the longitudinal and transverse directions, respectively; which produces a lateral force equivalent to about 0.15g or about 7.5MN. The base reactions for the single tower, though slightly larger than the group of towers, are still similar due to the increase in period.

The pushover results indicate that Tower C alone can be designed to withstand all of the required load combinations and exhibit significant pushover capacity. Further study of the advantages and disadvantages of interconnecting these towers is thus warranted.

The members of both tower configurations were checked for conformance with API RP2A Section D, for actual load demands.

In order to meet the pushover demand capability, all connections are required to be stronger than the yield capacity of the diagonal braces. This is to ensure that after the compression braces buckle, the tension braces can yield axially. This requirement produced some very heavy connections with many bolts and thus it is recommended that some type of base isolation system be incorporated, for these land-based towers in order to reduce the seismic demand with its resulting member size reductions, pushover demand and joint connection capacity requirements.

The typical bolted connection utilizes gusset plates that penetrate the legs of the tower. The through plate stiffens the ring section and prevents significant out-of-plane bending in the ring itself. This also allows about twice the capacity in the gusset plates that are welded directly to the legs, perpendicular to the through plate.

For a typical 457mm x 9mm tubular brace with a cross sectional area of 127cm2 a slot is cut at each end and a 25mm plate is inserted into the slot and welded with a 750mm long partial penetration weld at four locations along the pipe. The plate should be flared in order to increase its cross sectional area as it approaches the first row of bolts to compensate for the loss of section at the bolt holes. The typical connection will require 24-32mm HS bolts in order to develop/resist the member yield capacity in tension.

Tower G

The Tower G computer model utilizes a foundation comprised of linear dummy piles in order for the analyses to remain elastic. A dummy pile is simply an extension of the pile section to a point of fixity below the mudline where the shear is zero and the in-ground

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moment is largest. The location/depth of the point of fixity varies with soil type and is typically in the range of 3-7 pile diameters; however, for the piles in Tower G, it occurs at about 9 pile diameters most likely due to the significant axial load in combination with the presence of a relatively soft clay layer (Young Bay Mud) overlying sands and stiff clay (Old Bay Clay). An axial spring was added at the point of fixity to approximate the axial soil/pile behavior below that point.

The dummy pile is connected to the pile segment within the tower leg, which in turn is rigidly connected to the leg joint at Elev. +3.5m. The connection of the pile/dummy pile to the leg at the mudline is by means of a master-slave link whereby the pile and leg are pinned together in the lateral direction while the pile is free to move axially within the leg in the vertical direction (see Figure 4-1). The clearance between the pile and leg at the mudline is significantly restricted by the shims welded to the inside of the leg. The pile modeling includes an intermediate node at mid water depth for monitoring the pile lateral displacement versus the leg lateral displacement to ensure that they do not touch. The governing Impact Analyses results were reviewed and it was found that the pile displaces about 44mm while the annular space between the pile and inside of the leg is 50mm. If they were to touch, the pile would be somewhat restrained from bending further by the tower leg.

For the purpose of the Vessel Impact and Wind analyses, the Wind, Wave and Current were assumed to act only in the broadside direction of the tower due to the orientation of Temporary Tower G relative to YB and Treasure Islands.

The Impact analyses were performed from the two orthogonal directions as well as the diagonal. The impact load was statically applied at the El. +3.5m level leg node and hand calculations were used to check the leg and fender framing at the upper and lower bound locations (EL. +7.5m and –1.8m). Due to the significant impact force, some type of fendering system is recommended, for example, either independent dolphins or local crumple zones. Additional bracing has been added to prevent significant leg damage and overall tower instability. Detailing of the fenders is beyond the scope of this Design Example.

The first mode periods for Tower G are 1.56sec and 1.33sec in the longitudinal and transverse direction, respectively, which produces a lateral accelerationof about 0.15g and 0.17g, respectively. The signs of the resulting member forces were also reversed when combined with the other load cases since a modal analysis loses the sign of the members force during modal combination.

The members and joints were checked for conformance with API RP2A Section D and E, respectively, and sample hand calculations are contained in the Appendices for actual load demands. The typical welded tubular joint is designed/checked to comply with API formulas for punching shear and thus required an additional 3-6mm of thickness to resist the applied loads. The piles were also been checked for compliance with API Section D.3.2.2 Piles for the maximum loads cases.

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The displacement capacity of Tower G, pinned at the base is greater than 1.0m in both directions. These analyses were performed to verify the ductility of the tower itself since the tower with the piles will meet the pushover criteria without the tower yielding at all. The non-linear P-Y soil springs in combination with the fact that Impact governs much of the tower structure causes this phenomenon. The pushover capacity of Tower G with piles and P-Y springs is about 0.5m at which point significant pile axial displacement begins to occur.

In order to meet the pushover demand capability, all tubular joints tower joints are required to be designed to resist the yield capacity of the diagonal braces. This is to ensure that after the compression braces buckle and the tension braces yield, the joints will remain intact. This requirement was met by increasing the nominal leg thickness by 6-9mm at the joint can locations,rounding up to the nearest 3mm to ensure that the joint can remains elastic.

For Tower G, a series of non-linear lateral pile analyses were also performed on several diameter pipe piles with varying wall thicknesses. The pile capacities were compared with the demands and a 1.067m diameter pile was chosen for its axial and lateral soil capacity and enhanced driveability. The initially assumed wall thickness of 38mm was reduced to 32mm, as a result of the preliminary analyses performed; however, an increased pile thickness will improve overall structure ductility.

The maximum tower pile forces were compared with individual non-linear pile analyses and found to generally match the maximum load condition.

The maximum axial pile demand from the various analyses performed is 14.0MN, which would require a pile penetration of about 52m. In order to meet the maximum settlement criteria, an additional few meters would be required. A pile driveability analysis should be performed at the east and west bound locations to verify that these pile can be successfully driven to this penetration. A few millimeters increase in pile wall thickness may be warranted to ensure pile driveability to the design penetration. This would also reduce axial pile settlement at the mudline.

Impact design

A simple fender has been conceptually shown on the tower sketch with a large diameter vertical tubular welded to a series of horizontal members which transmit the Vessel Impact load into the interior of the structure an beyond. The intent is to grout the annular space between the pile and the leg within the fender area. Additional joint cans are also required within the interior bracing where these members terminate to prevent the diagonal members from being overloaded during Vessel Impact. No reduction in Vessel Impact force has been assumed due to the bending and crumpling of the fender structure during Impact; however, it is believed that due to the inertia of the tower structure itself the piles and possibly the deck will not experience the resulting local forces caused by the very short duration vessel impact load. The structure and piles have, however, been checked to comply with this design criteria.

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The remaining Impact bracing has been checked for axial loads and joint punching shear. A slightly thicker joint can is required at these locations due to the significant Impact loads. Alternatively, these brace areas should be filled with concrete, locally.

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5. ANALYSES RESULTS

The following sections contain selected output graphics from the SAP2000 computer analyses as a representative summary of results required to fully present the response of the structure to the various load combinations specified. It also contains the results of the various pushover analyses performed. Most should be self-explanatory.

It should be noted that all possible load conditions have not been investigated with regard to deck interconnectivity, being beyond the scope of this Design Example.

5.1 Tower C

Tower C was analyzed both as a part of the group of combined Towers ABC, east and west bound, all connected together and also as a separate stand-alone tower .

Since the other towers restrain Tower C, it shares a lower modal period in both directions and thus has a higher seismic demand.

The pushover of the combined Towers ABC was not limited by events occurring in Tower C itself since Tower A controlled the response. Adjusting stiffness in the other towers would address this issue.

Tower C was also analyzed as a stand-alone structure, assuming no connectivity with any of the other structures in order to bound the behavior of the single tower as much as possible. It can be seen from the results that it has a higher modal period and thus lower seismic demands as compared with the group of towers.

Further refinement of the tower design should include investigating some type of base isolation system in order to improve ductility.

5.2 Tower G

Tower G was analyzed as part of the series of east bound towers along the bridge longitudinal axis, but not connected to the westbound side in order to limit the analyses conditions for this Design Example. Tower G was also checked as a single tower for several of the intermediate deck erection configurations; however, none of them governed overall design.

The Tower G pushover was performed both with and without piles since it can be seen that the soil and piles are providing most of the lateral flexibility. The soil and piles yield earlier than the bracing due to the impact demand causing larger bracing to be required as compared with seismic demands. Isolating the tower and pinning it at the base, indicated that the tubular braced structure has more than adequate ductility, by itself. Stiffening the piles and softening the diagonal bracing somewhat would provide a better balance between the two systems.

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Since this is only a first iteration in the design process, further refinement of the tower designs should lead to more ductile structures.

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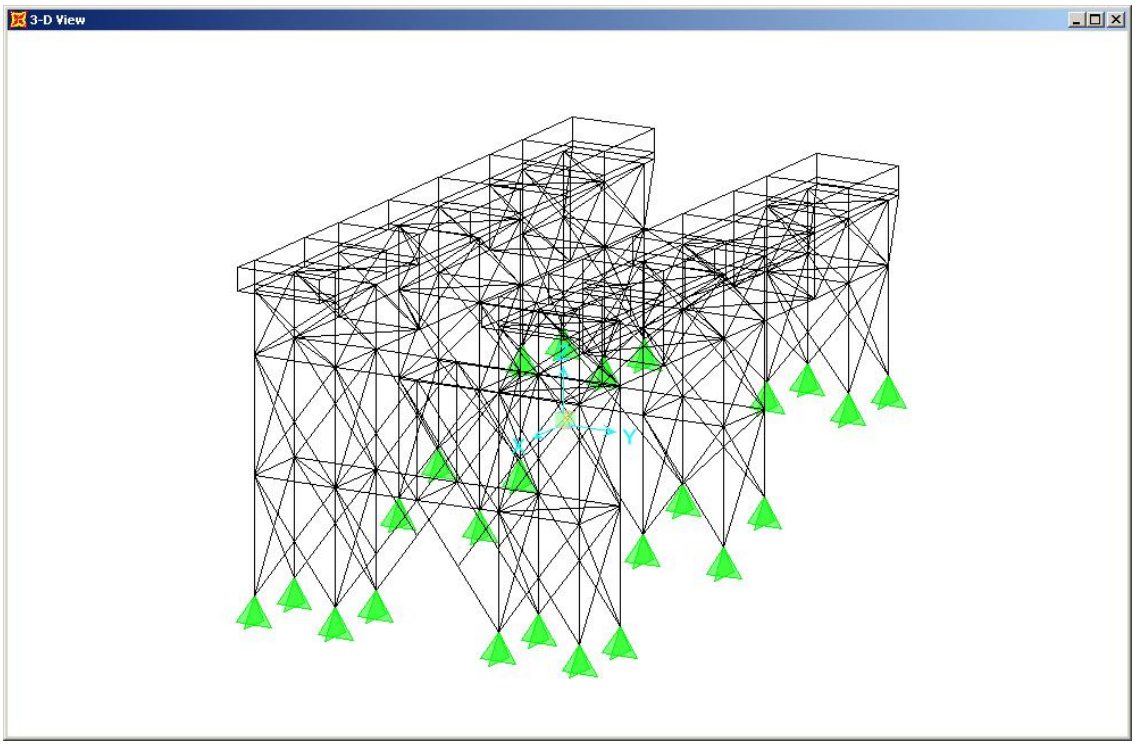
Towers ABC Modeled as a combined structure

3D View of Towers ABC Model



Modes

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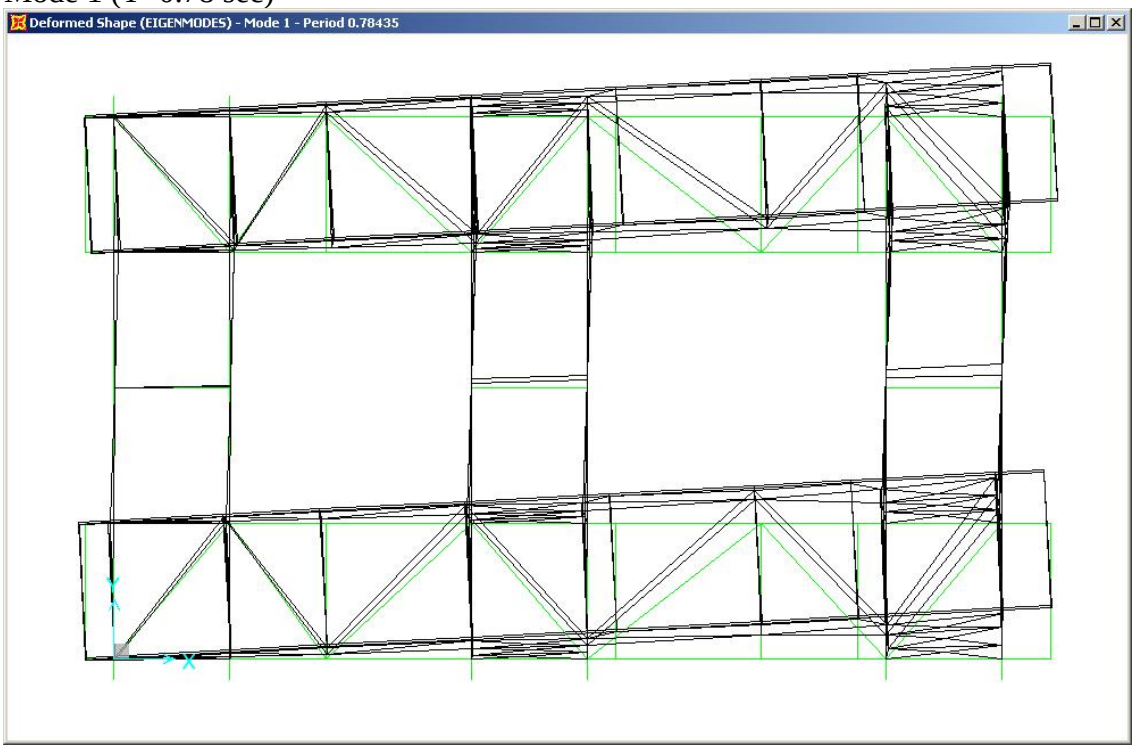
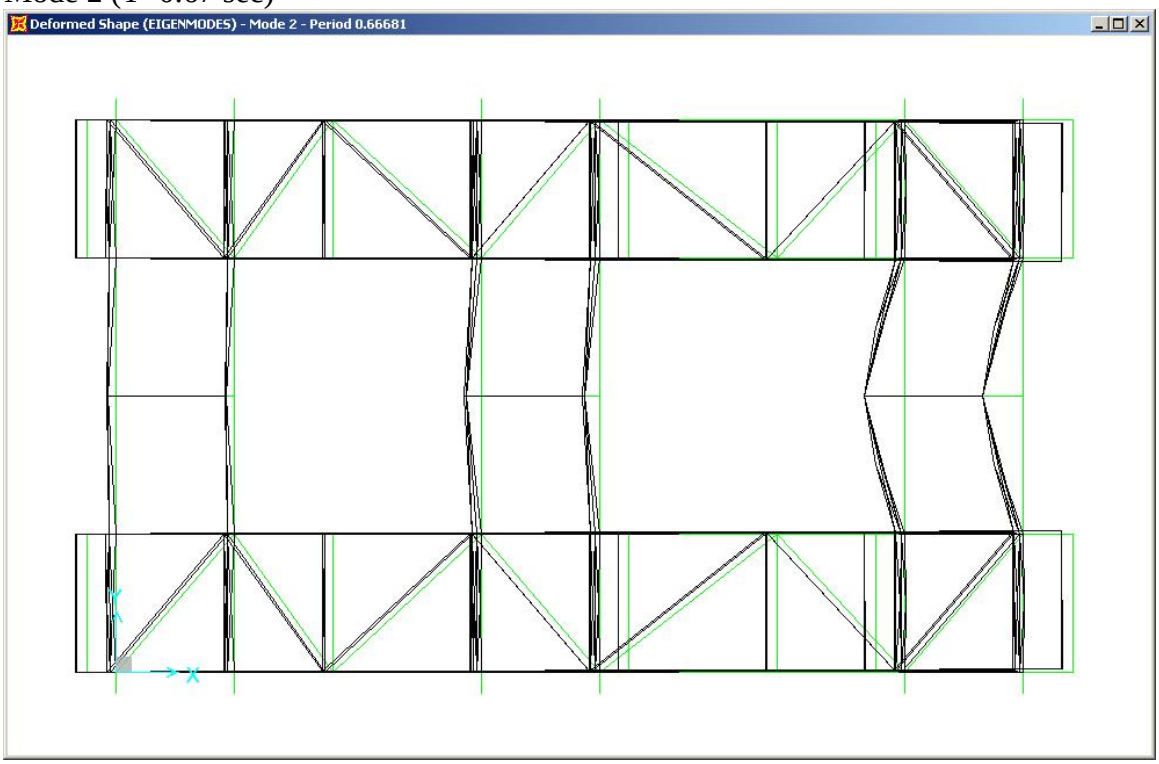
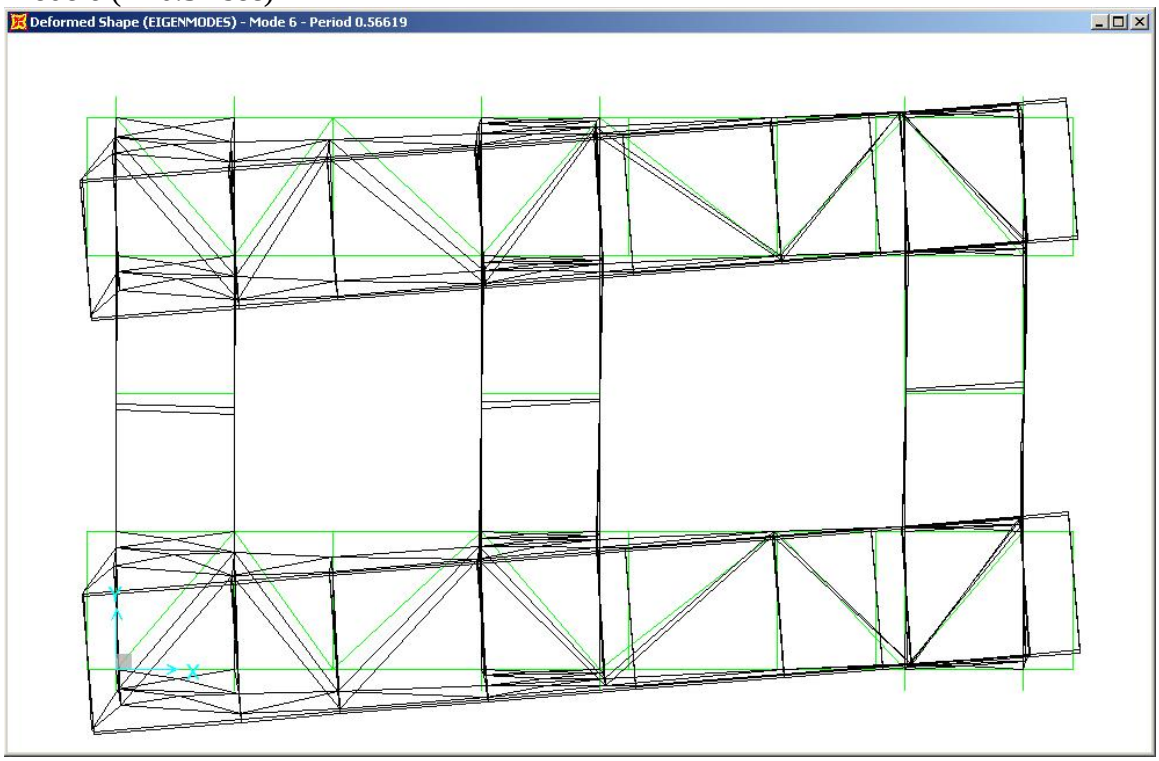


Mode 2 (T=0.67 sec)

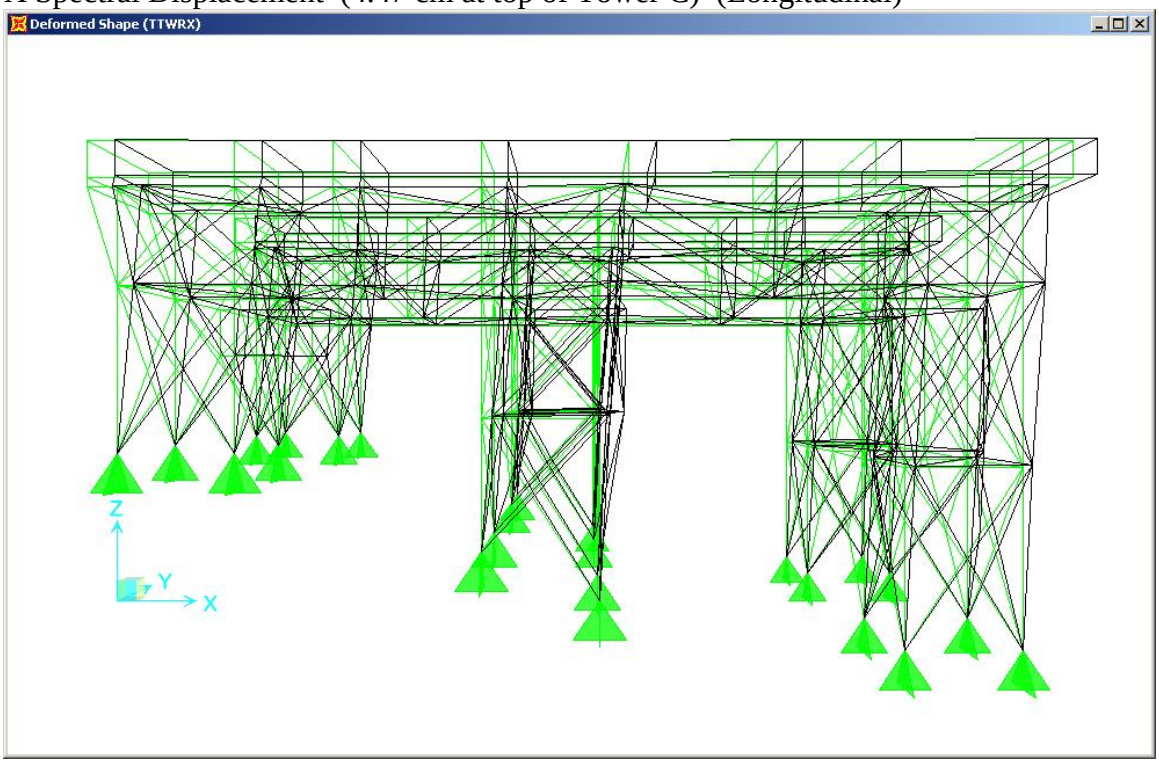


Mode 6 (T=0.57 sec)

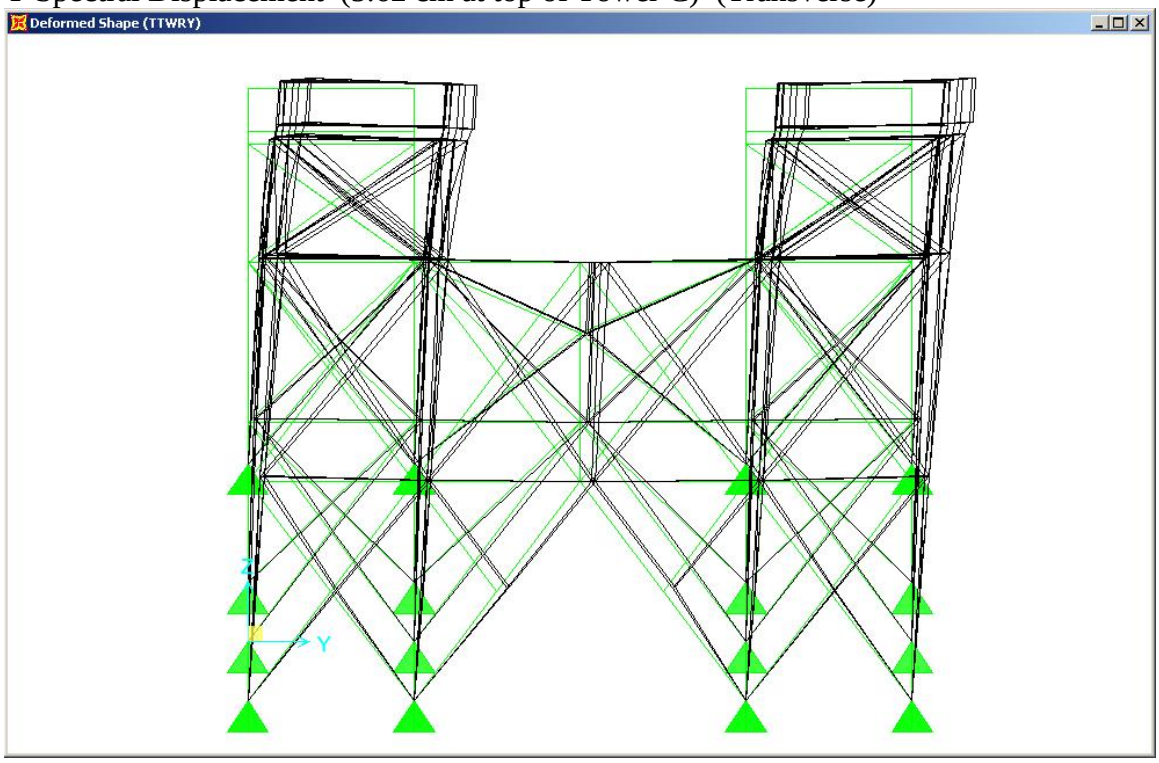


Displaced Shapes

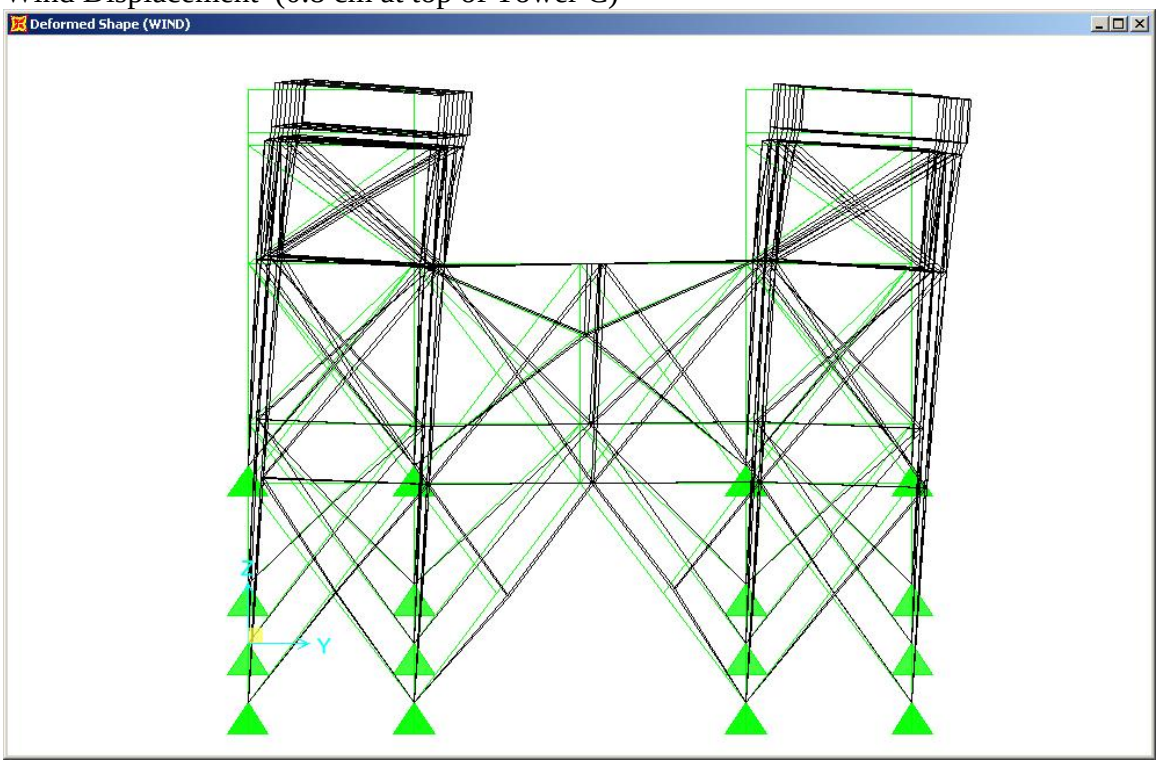
X Spectral Displacement (4.47 cm at top of Tower C) (Longitudinal)



Y Spectral Displacement (5.62 cm at top of Tower C) (Transverse)

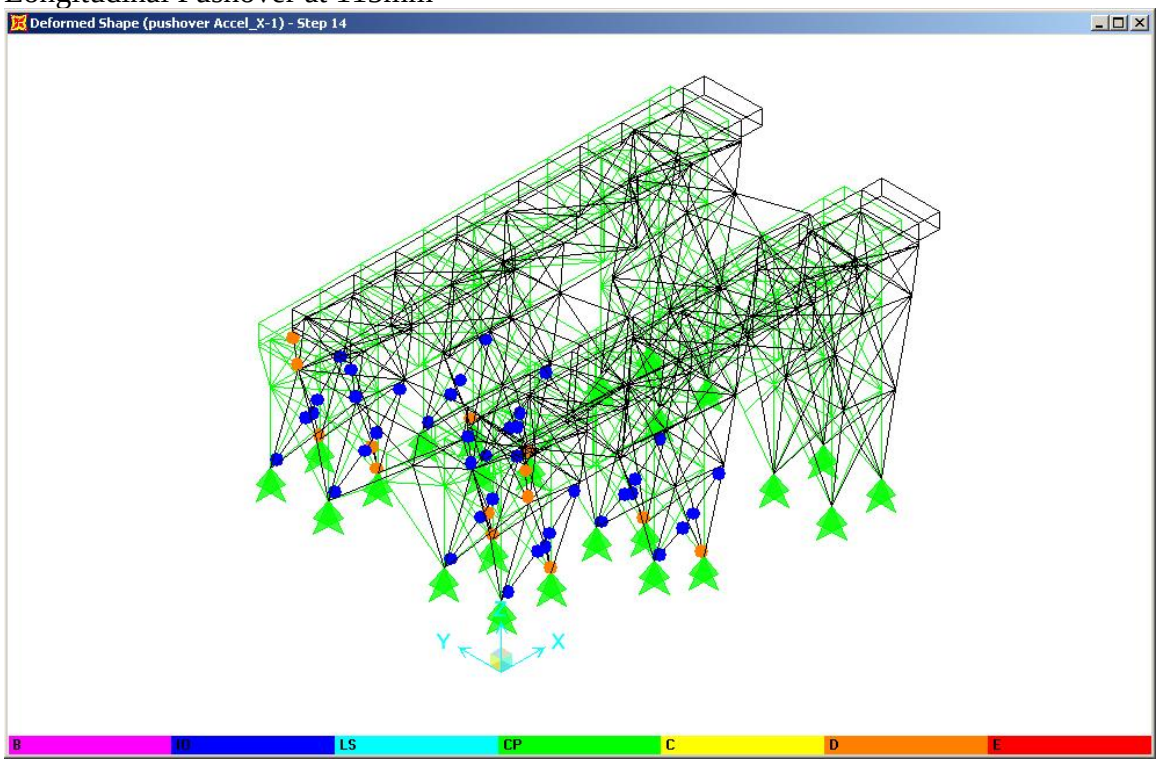


Wind Displacement (0.8 cm at top of Tower C)

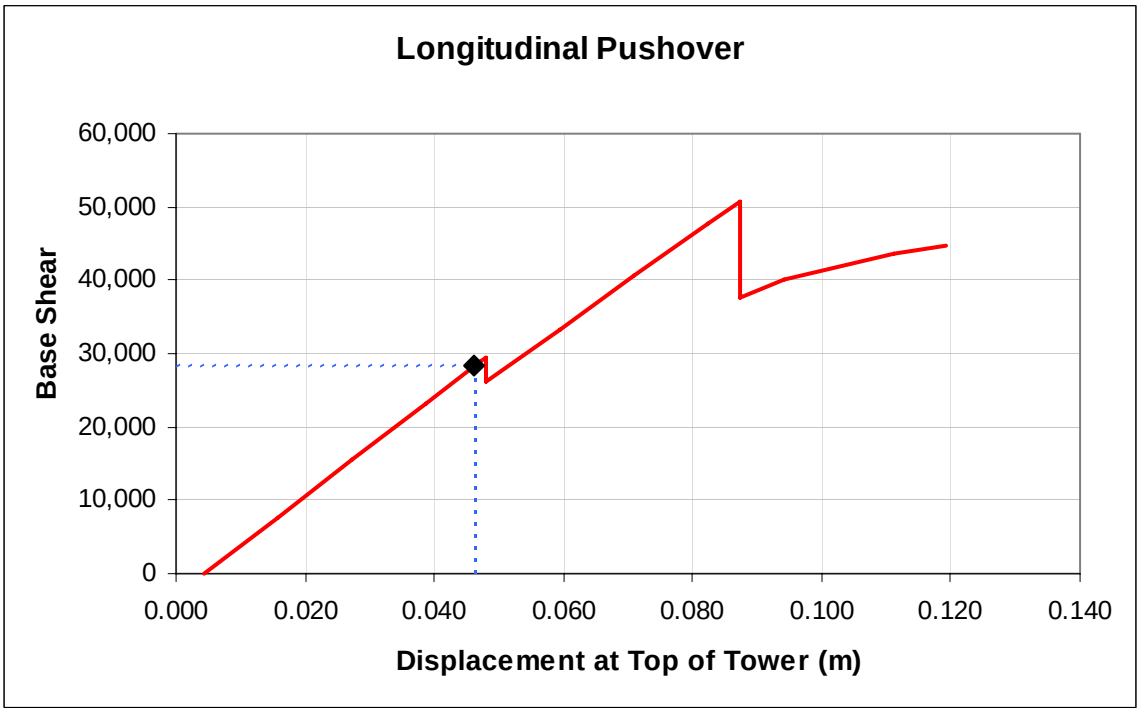


Pushover

Longitudinal Pushover at 115mm

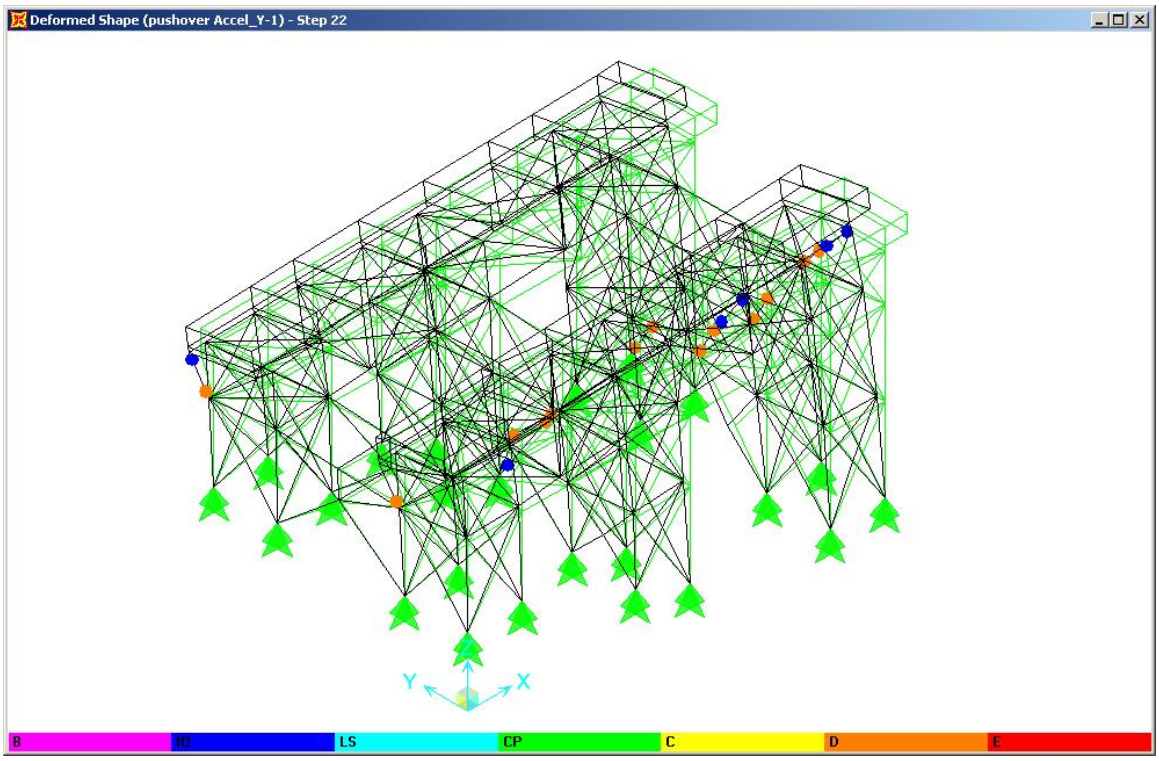


Longitudinal Pushover to 115mm

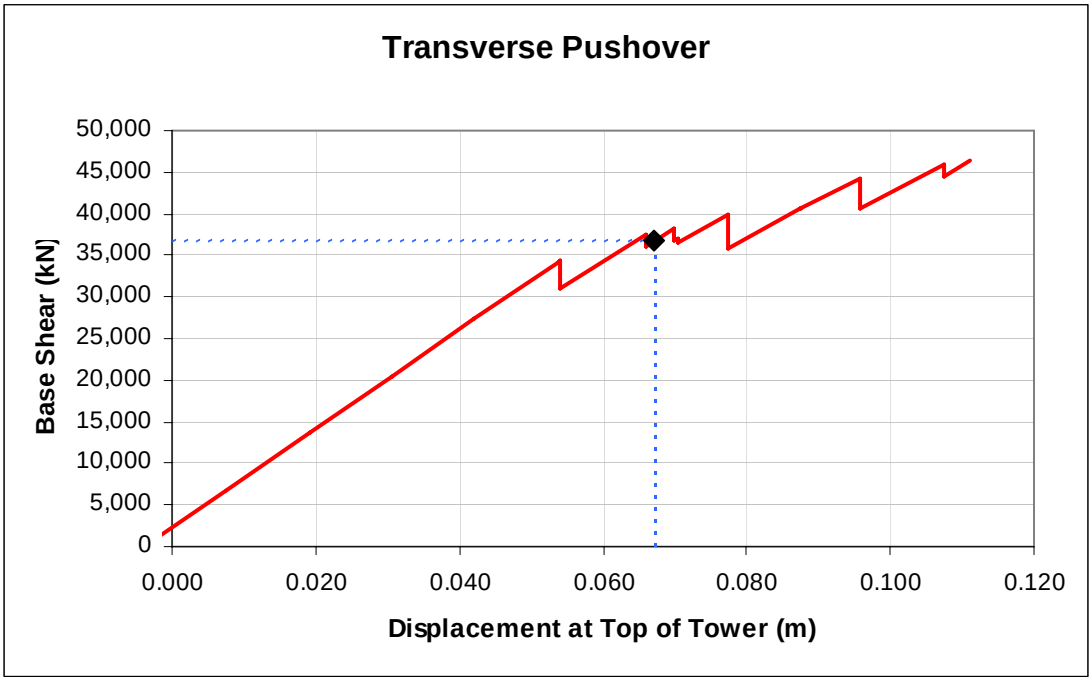


Shear reductions are due to compression member buckling (tension members take over)

Transverse Pushover at 115mm



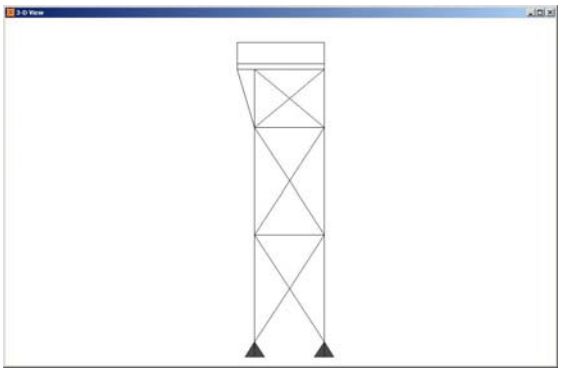
Transverse Pushover to 115mm



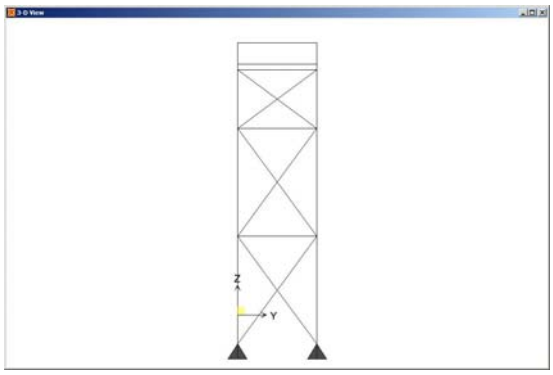
Shear reductions are due to compression member buckling (tension members take over)

Computer Model – Tower C Only - as a separated structure

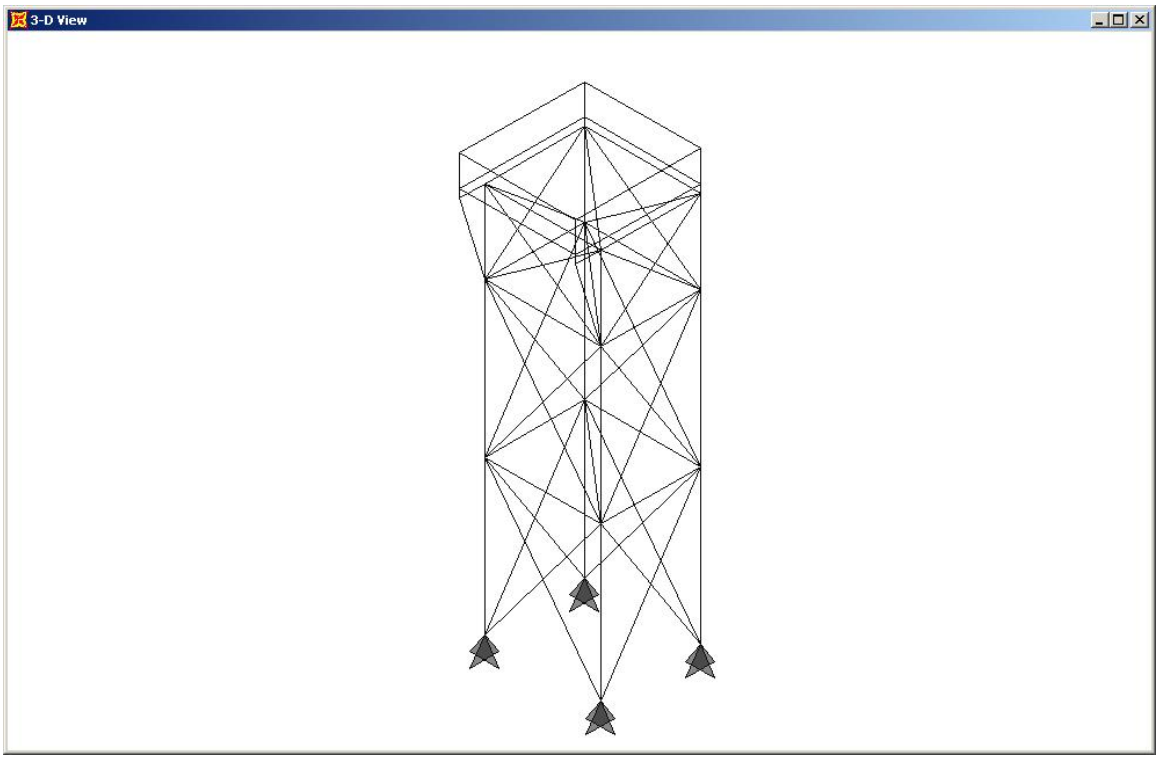
Side View of Tower C Model



End View of Tower



3D View of Tower C Model



Modes

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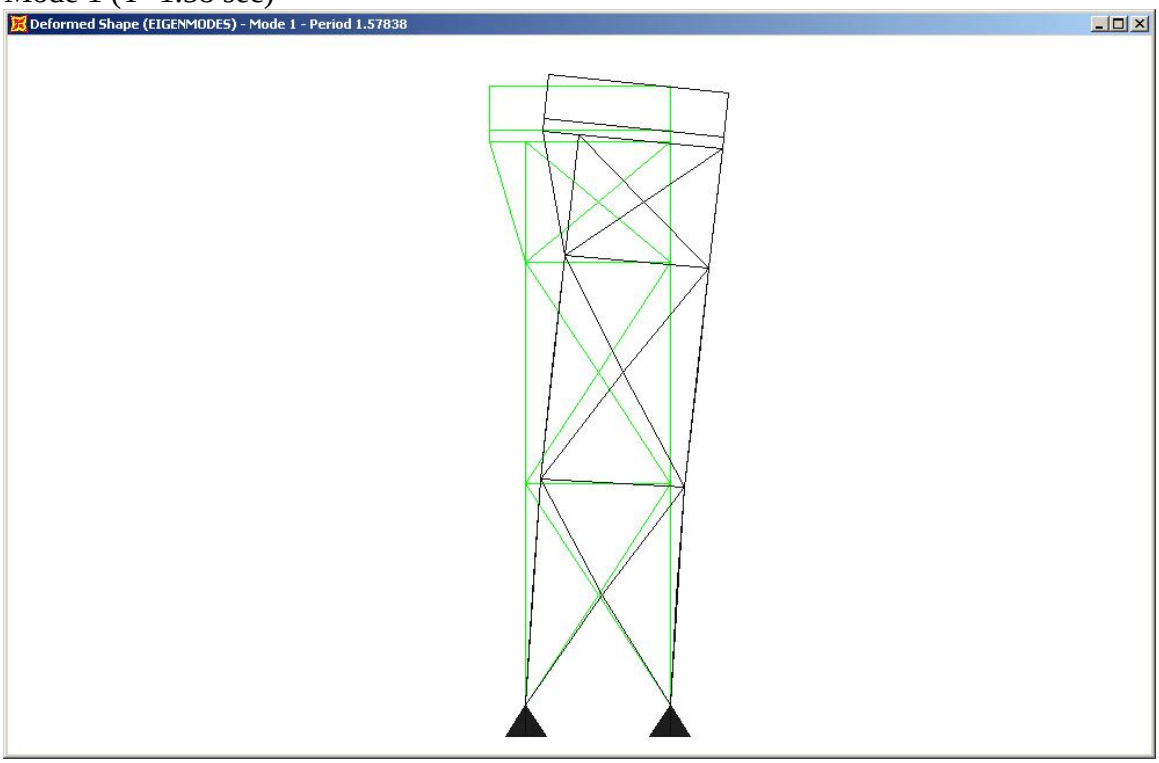
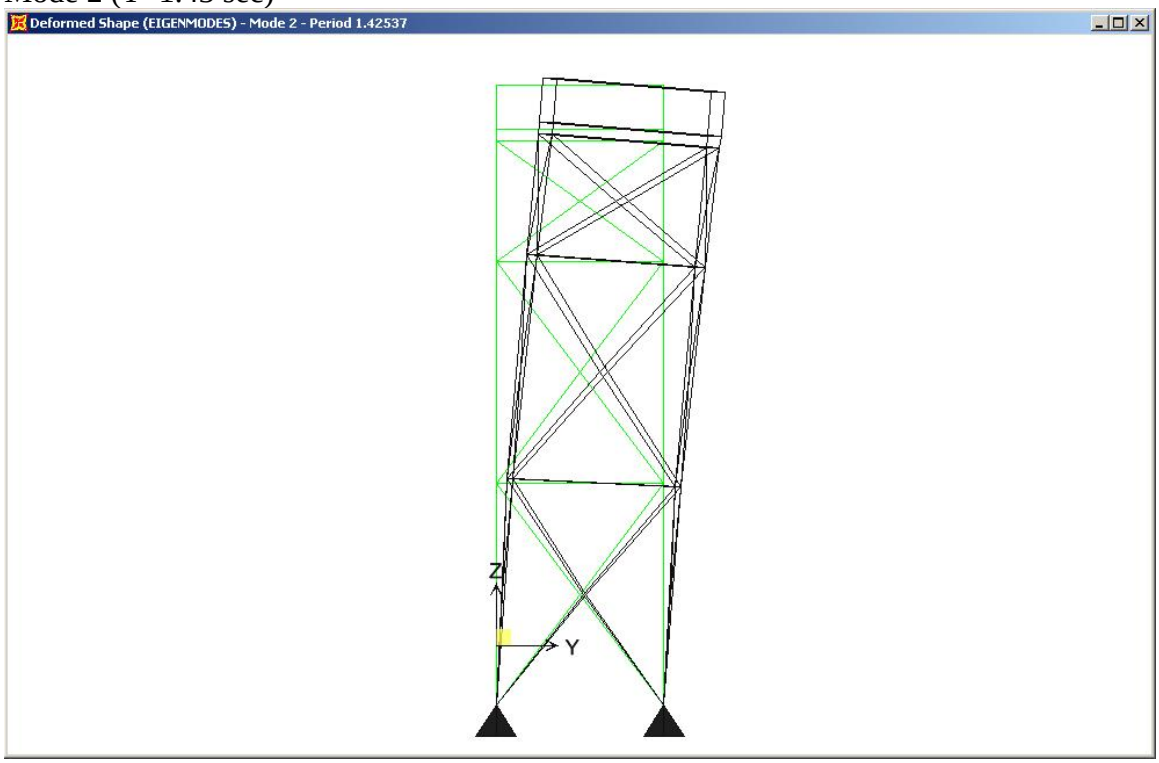
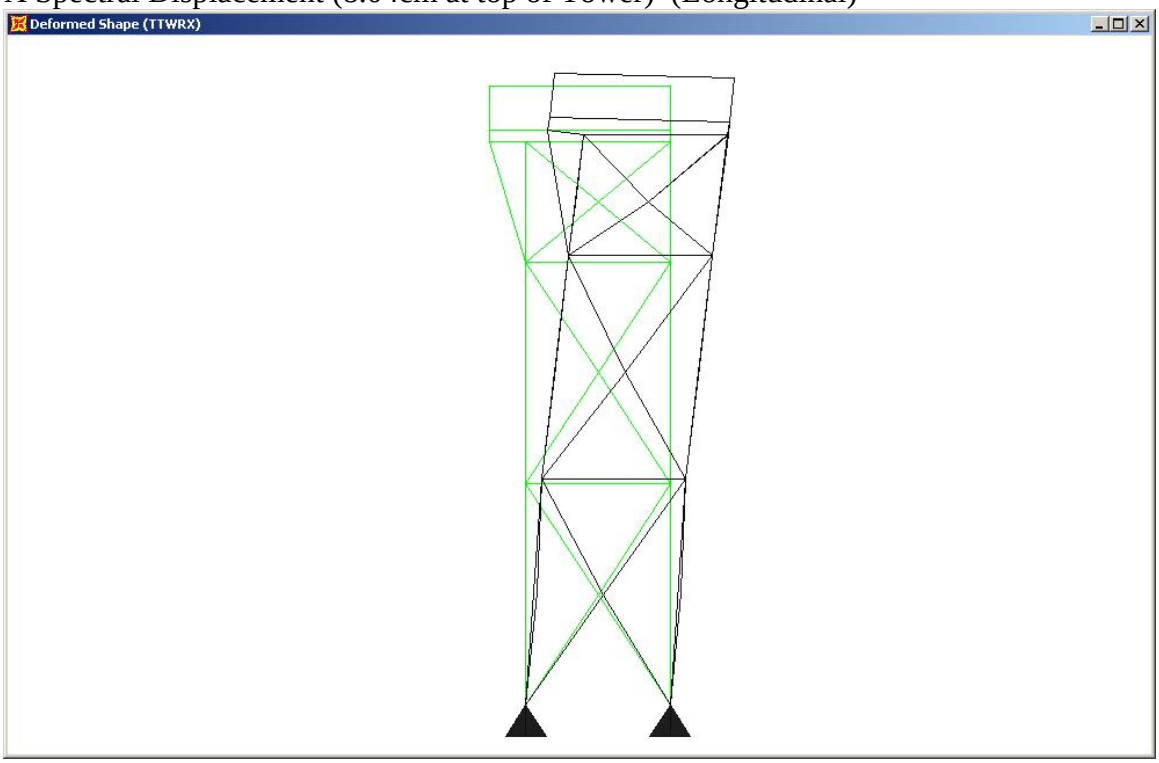


Mode 2 (T=1.43 sec)

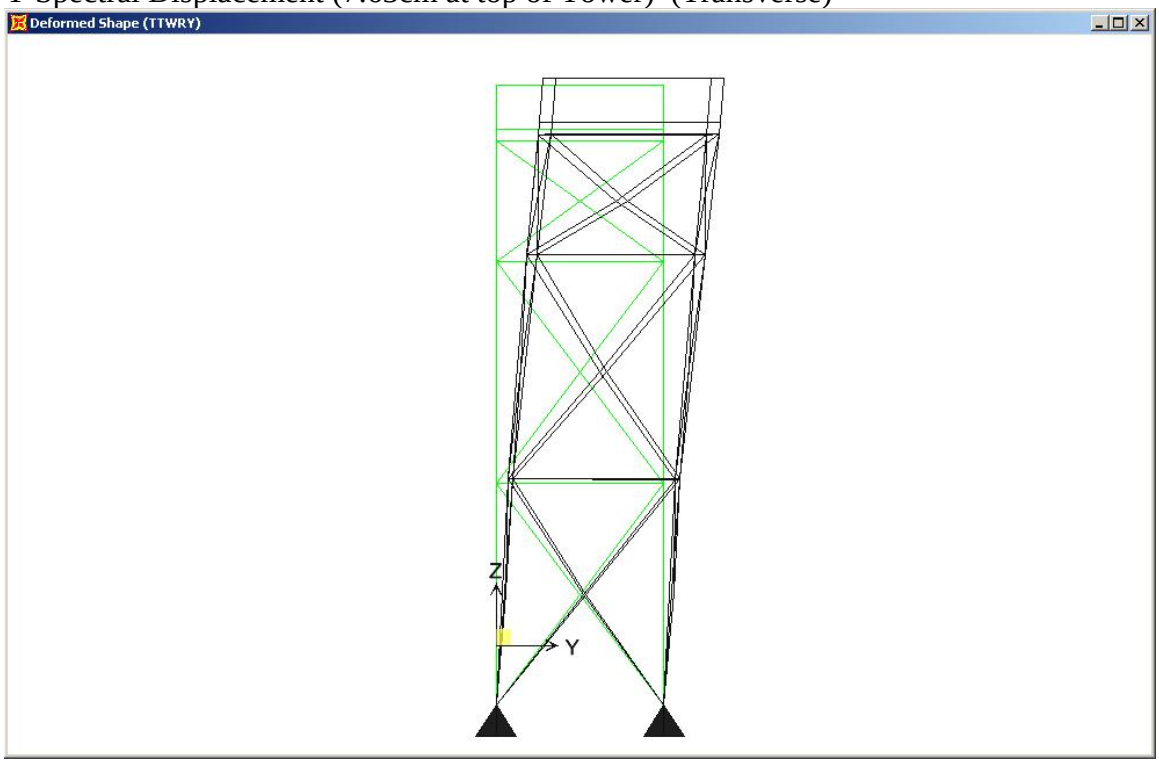


Displaced Shapes

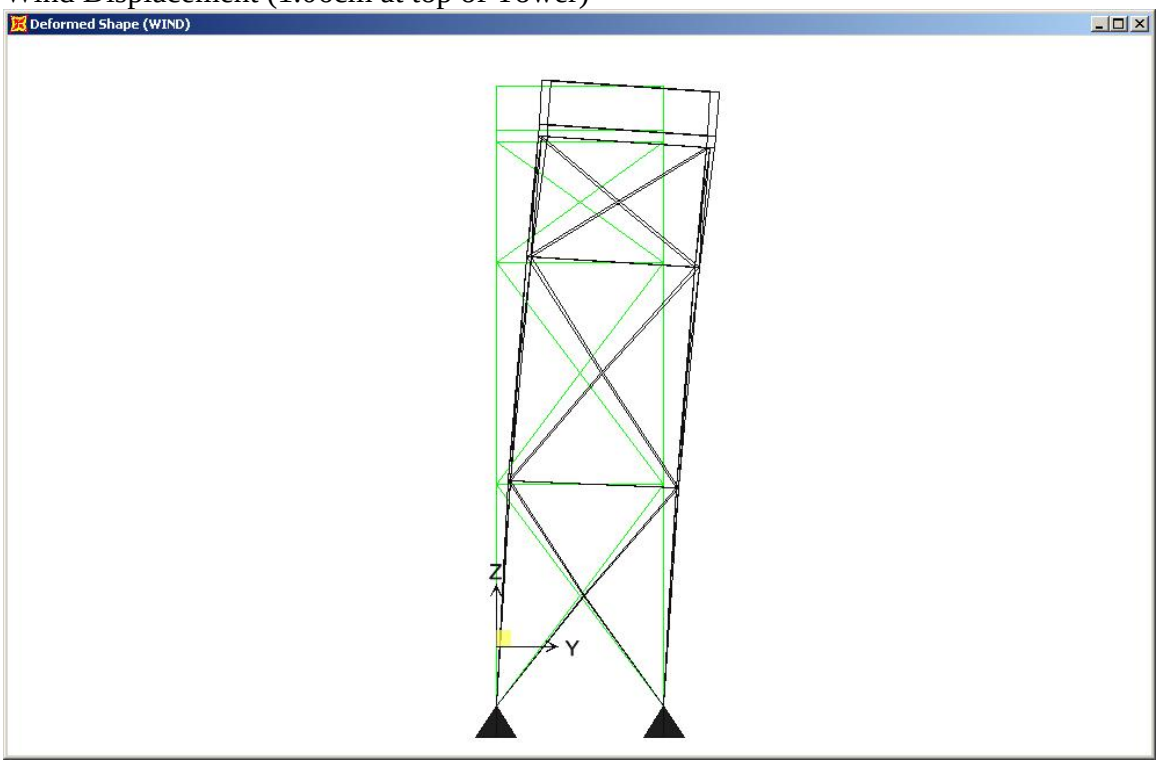
X Spectral Displacement (8.04cm at top of Tower) (Longitudinal)



Y Spectral Displacement (7.63cm at top of Tower) (Transverse)

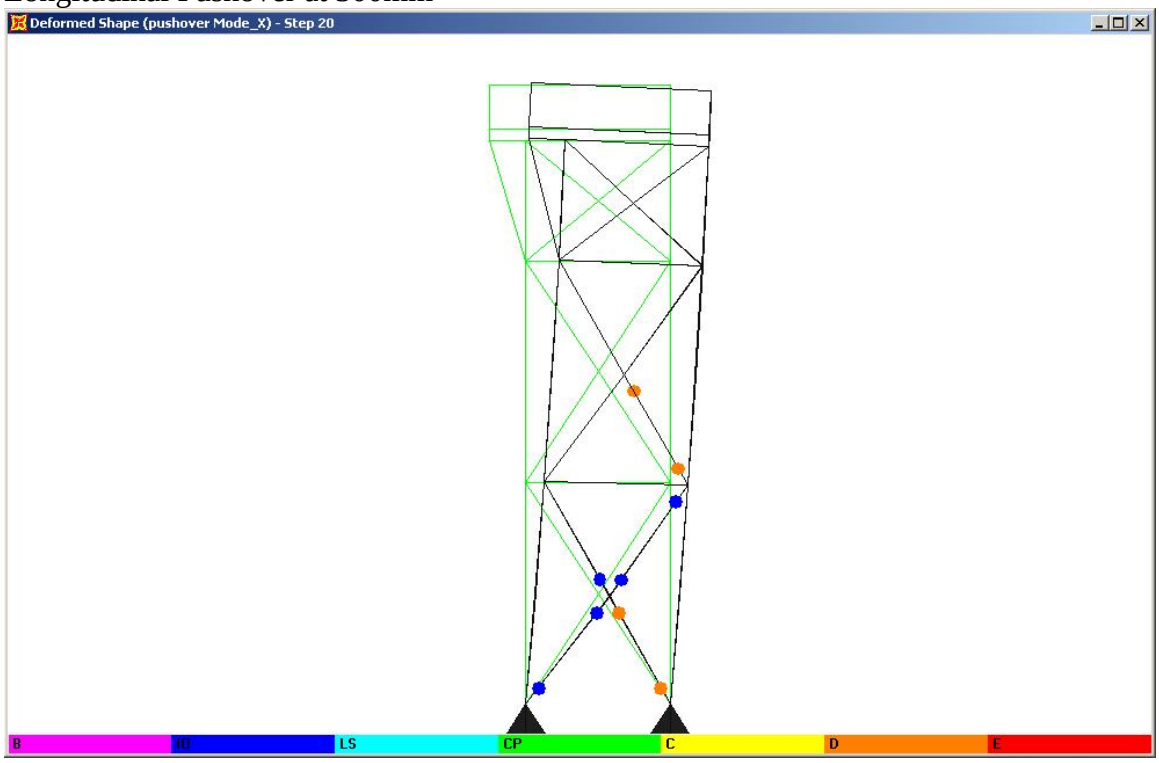


Wind Displacement (1.06cm at top of Tower)

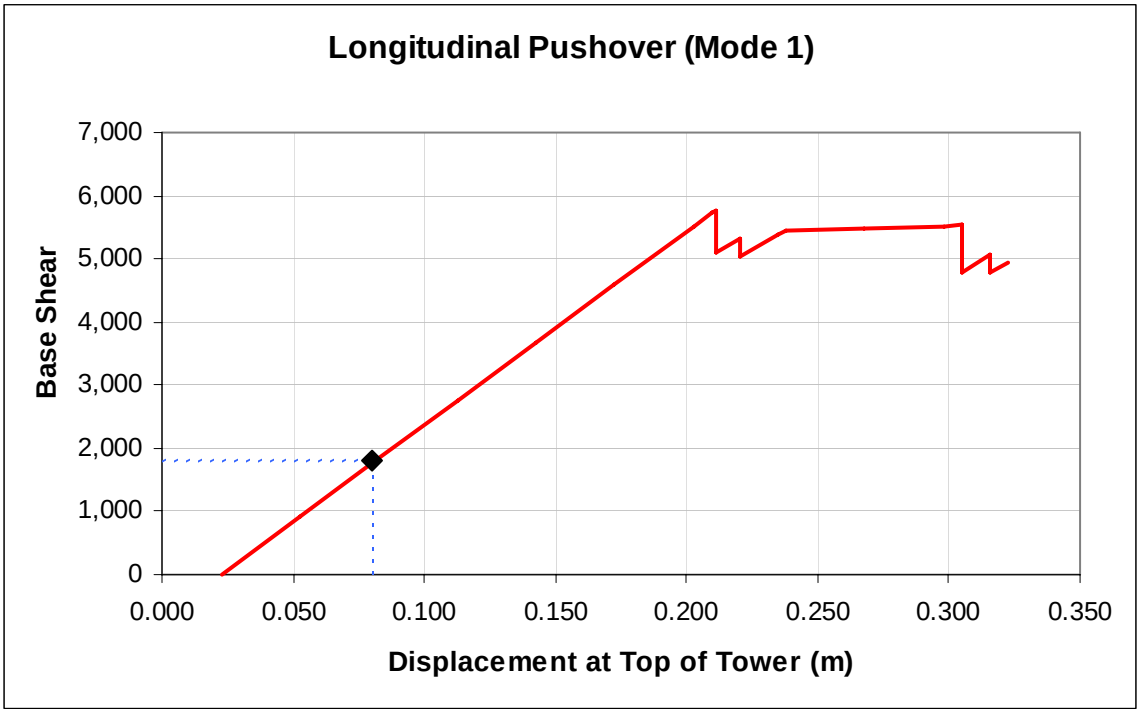


Pushover Results

Longitudinal Pushover at 300mm

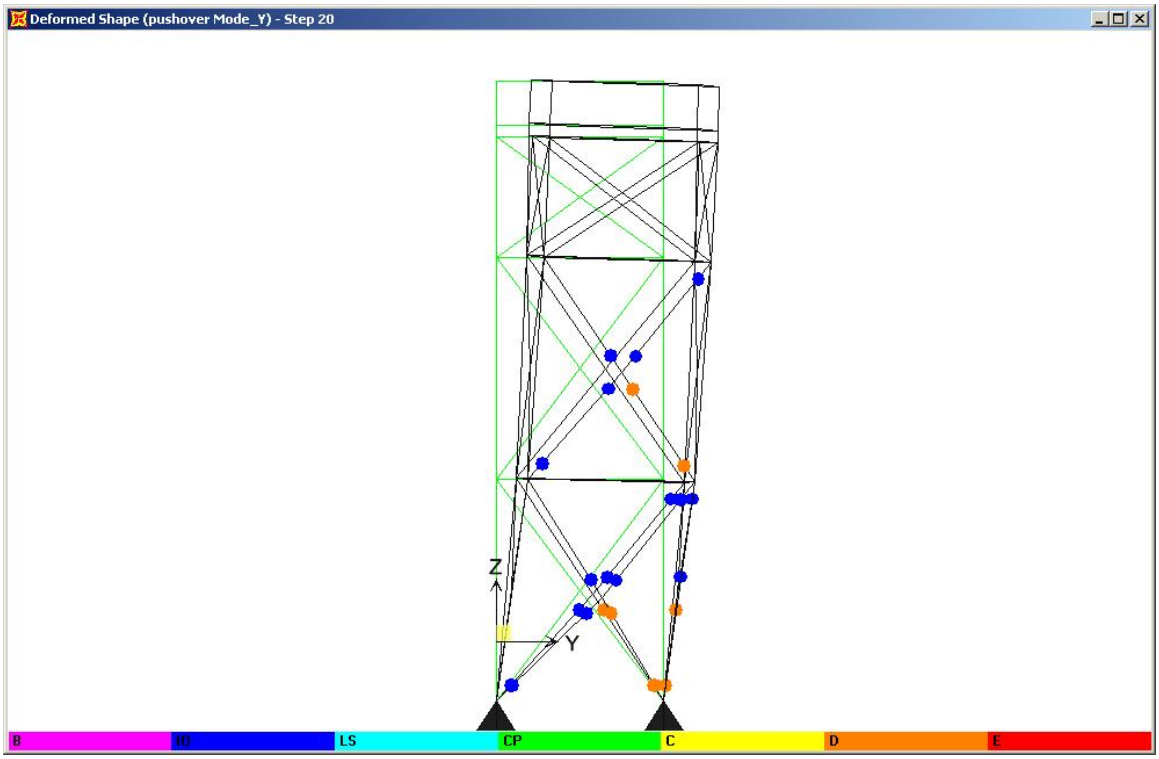


Longitudinal Pushover to 300mm, Force-Displacement Curve (80mm Seismic Demand)

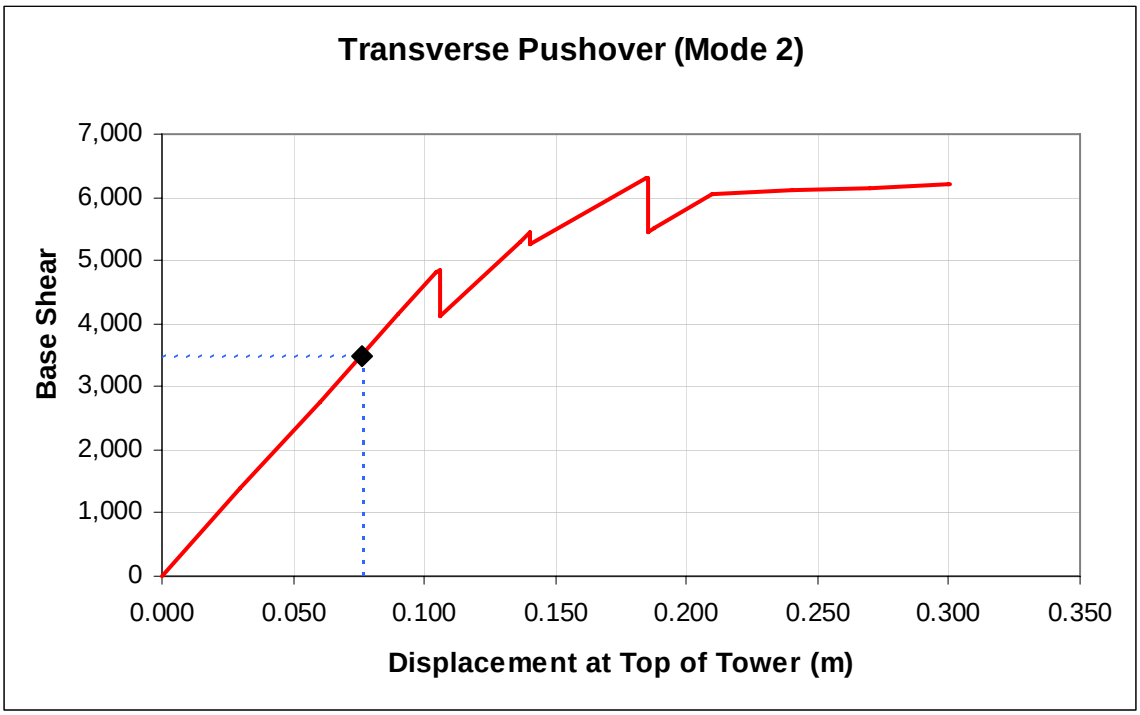


Shear reductions are due to compression member buckling (tension members take over)

Transverse Pushover at 300mm



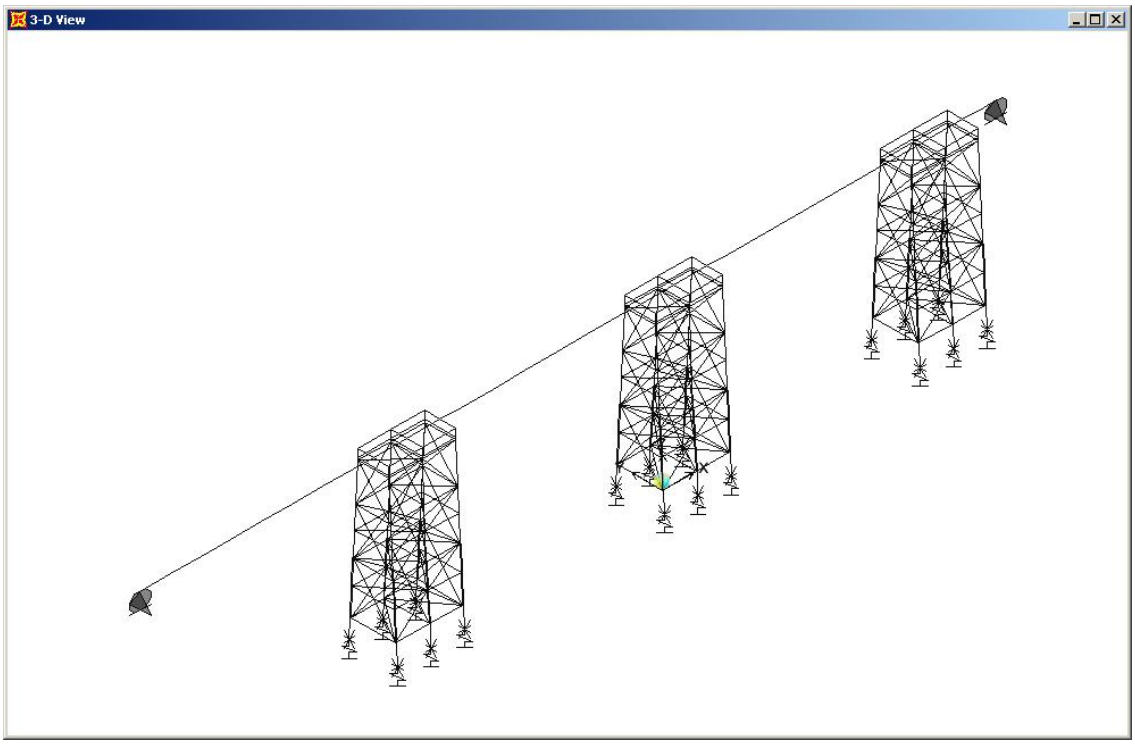
Transverse Pushover to 300mm, Force-Displacement Curve (76mm Seismic Demand)



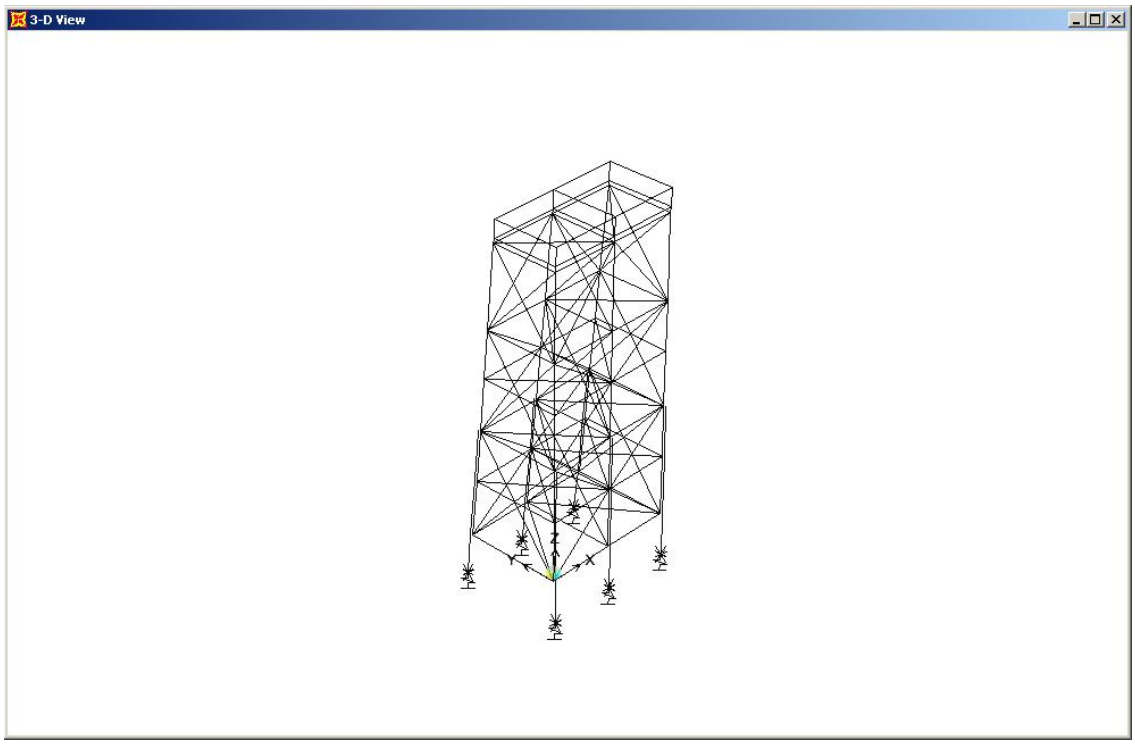
Shear reductions are due to compression member buckling (tension members take over)

Tower G Computer Model

3D View of Tower G Model



3D View of Tower



Modes

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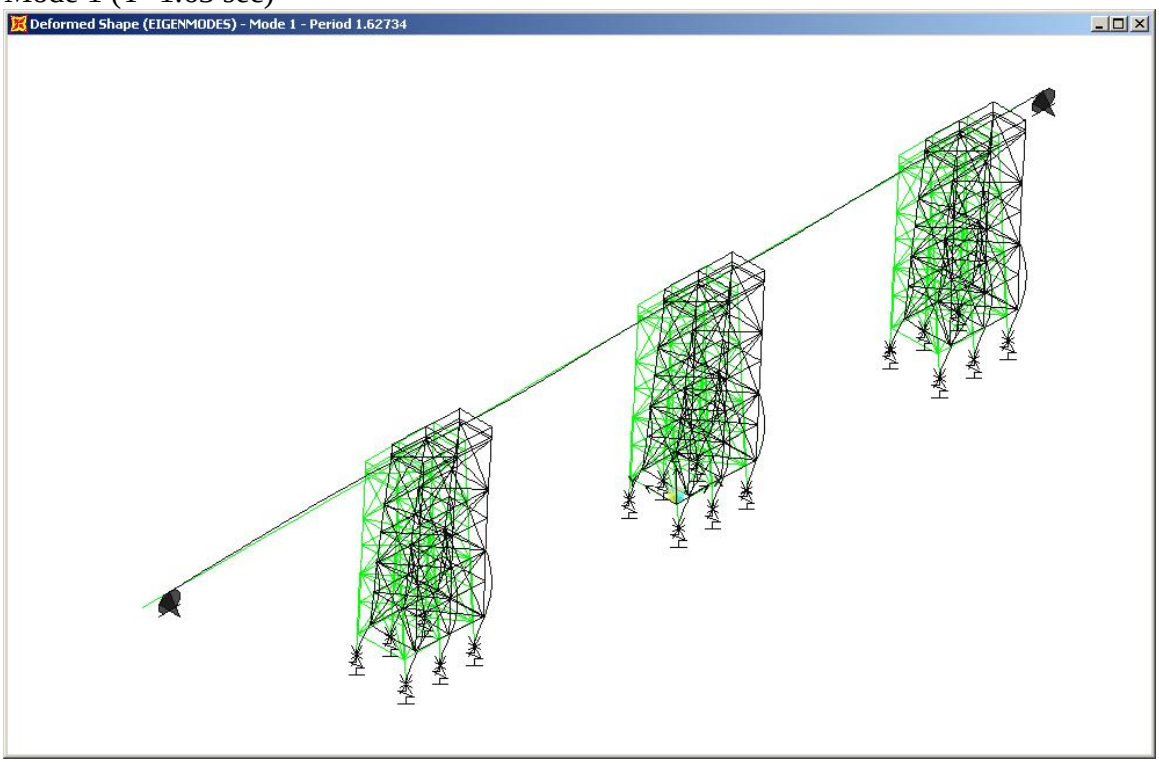
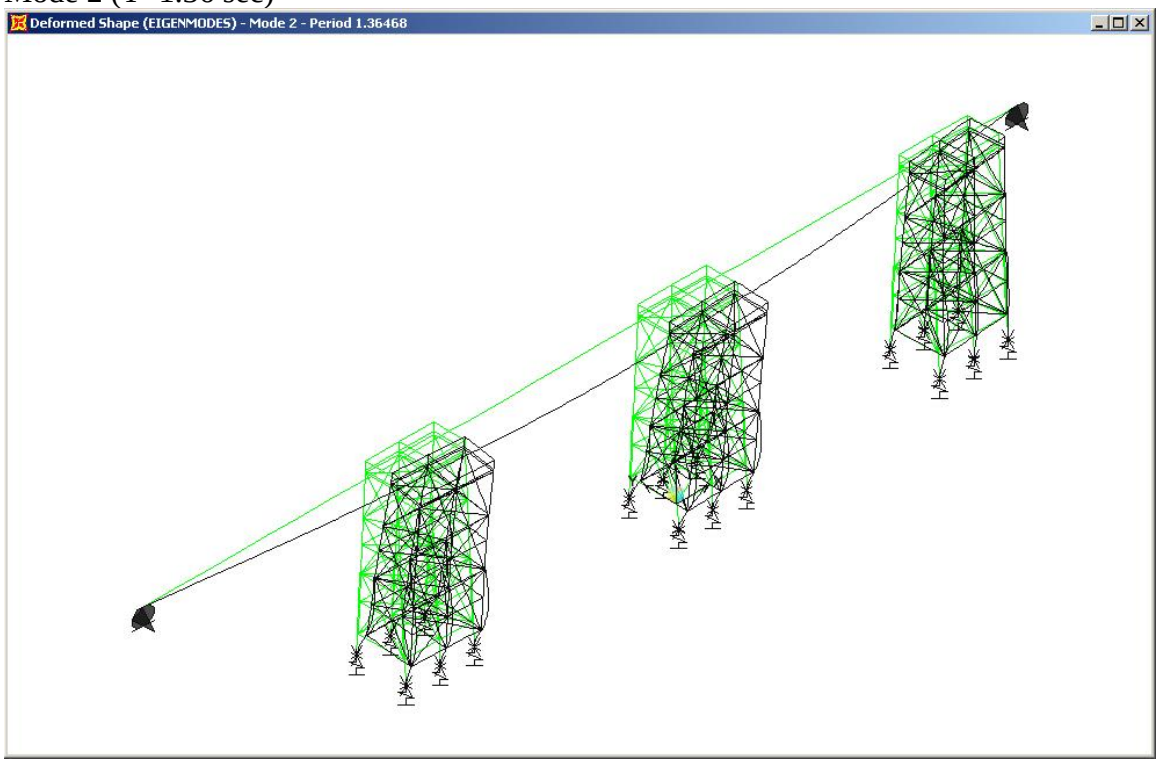
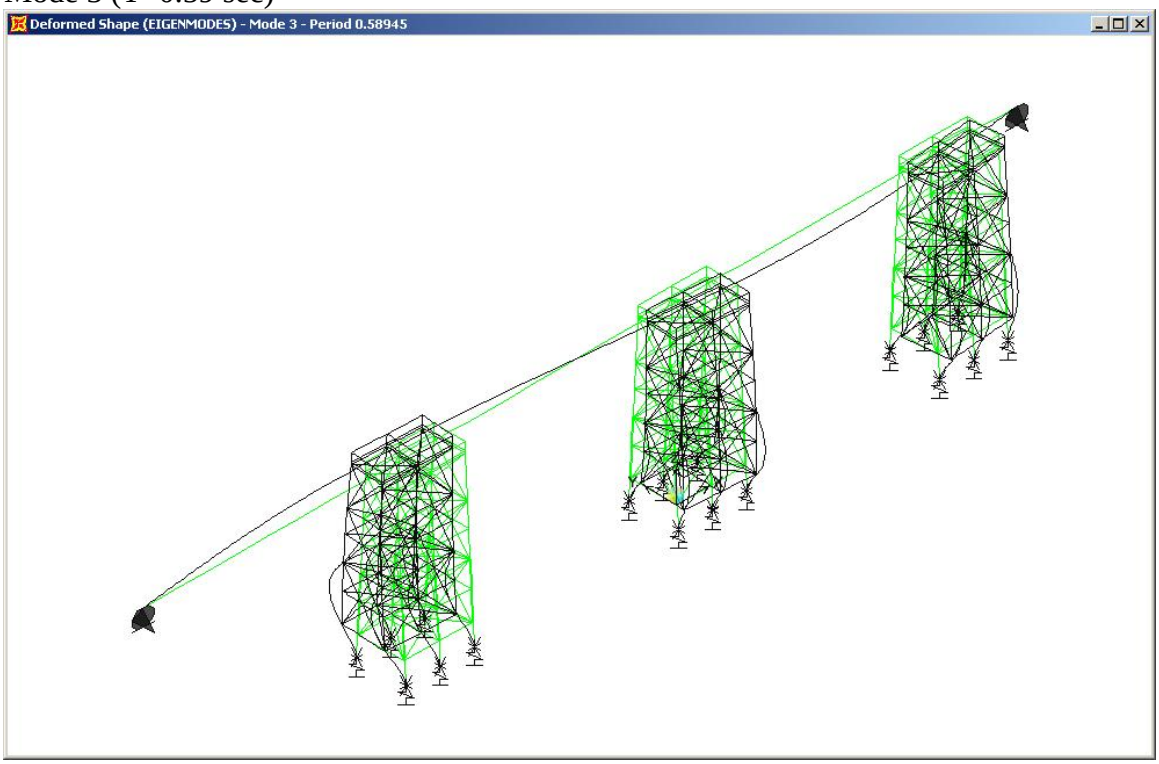


Mode 2 (T=1.36 sec)

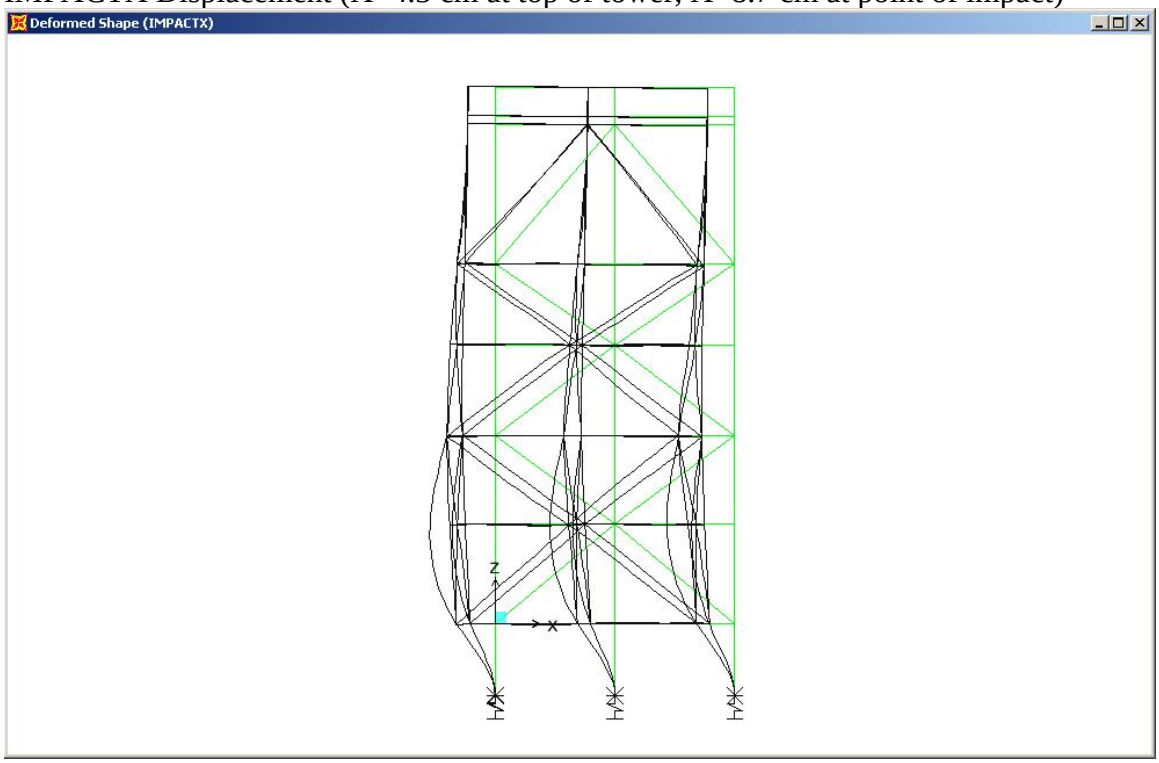


Mode 3 (T=0.59 sec)

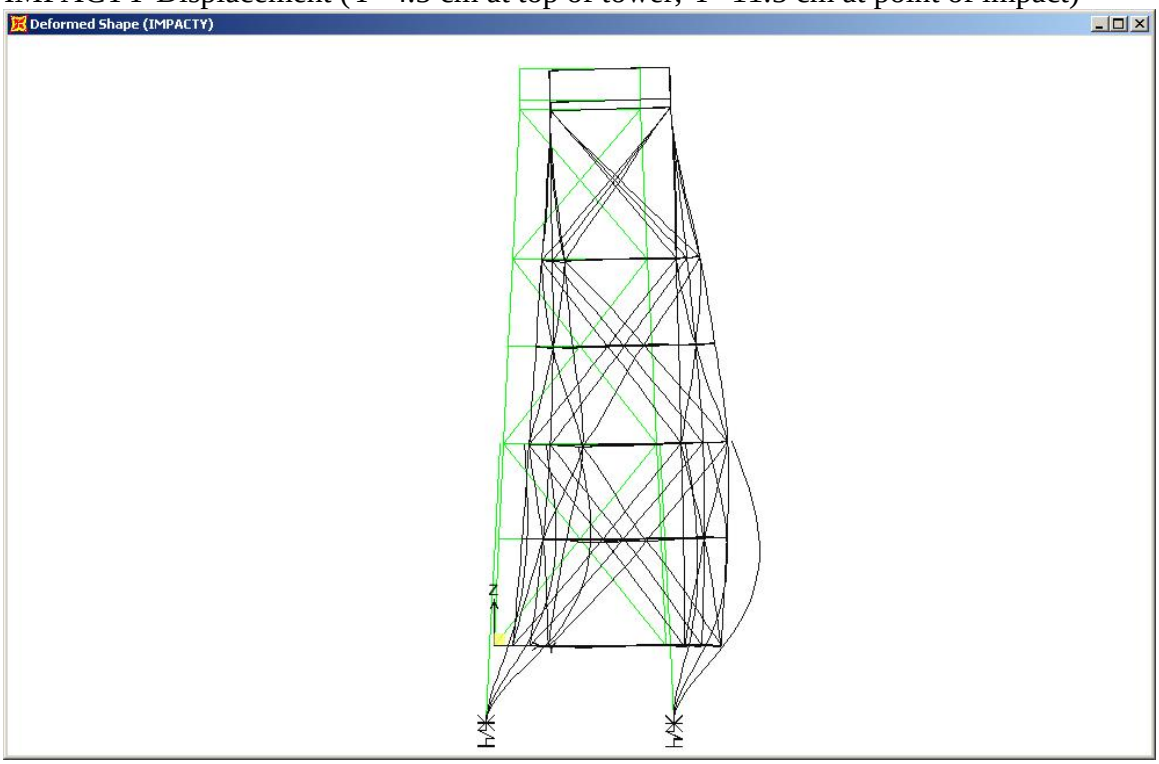


Displaced Shapes

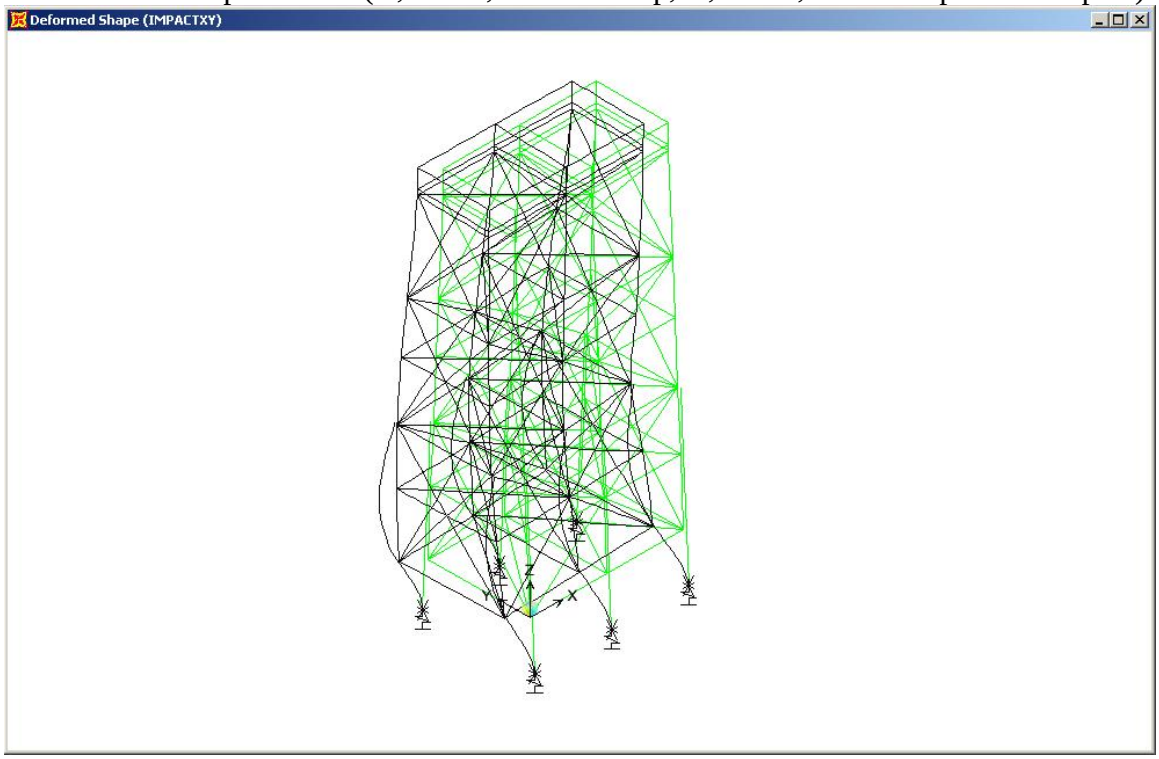
IMPACTX Displacement (X=4.3 cm at top of tower, X=8.7 cm at point of impact)



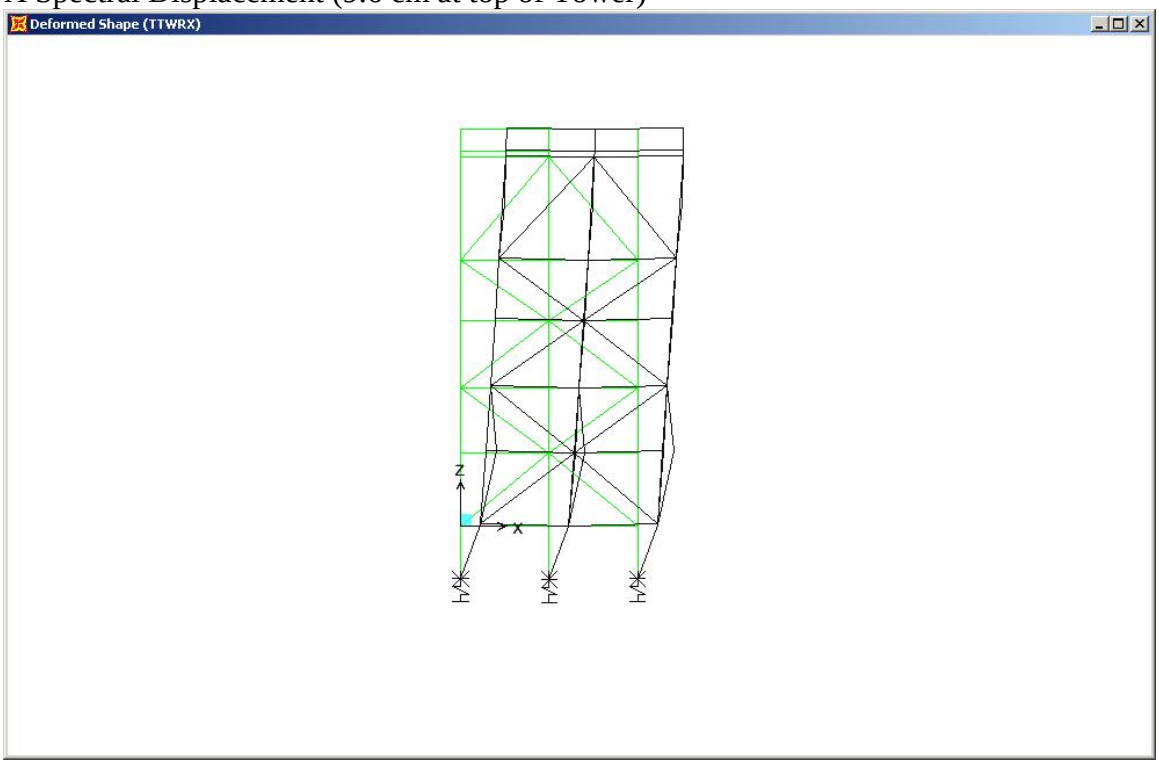
IMPACTY Displacement (Y=4.5 cm at top of tower, Y=11.5 cm at point of impact)



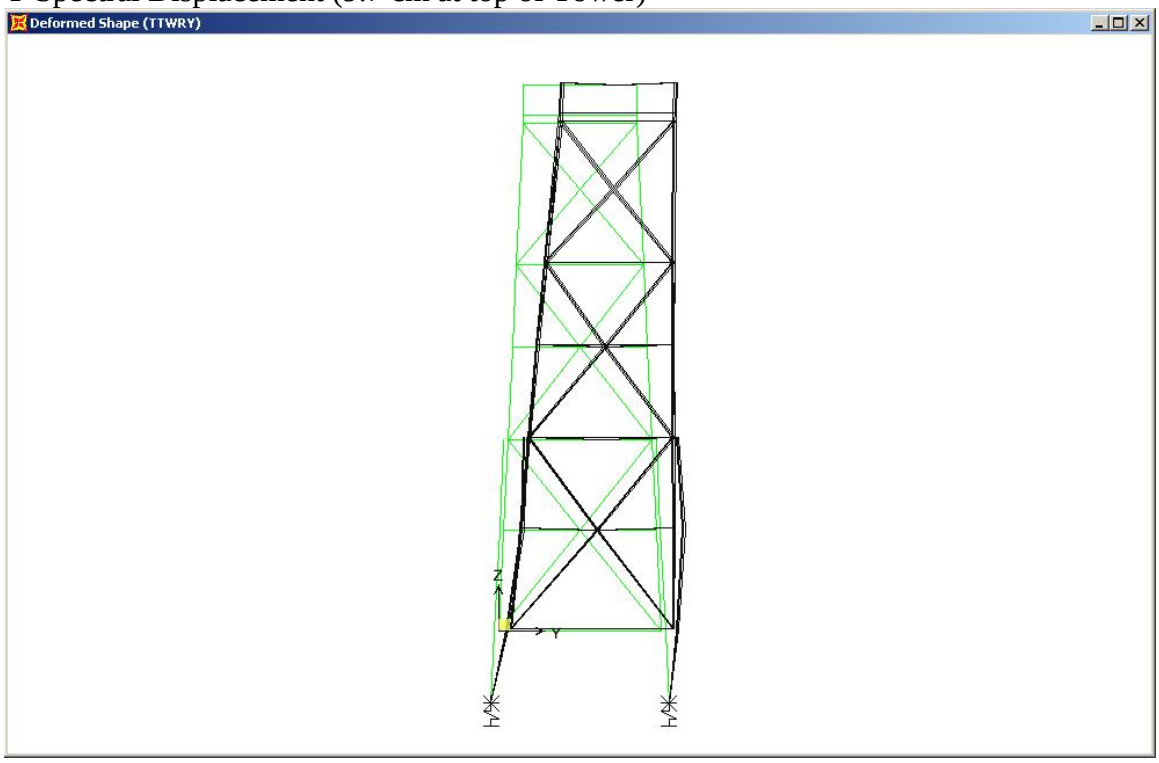
IMPACTXY Displacement (X,Y=3.1, 3.3 cm at top, X,Y=4.8, 6.7 cm at point of impact)



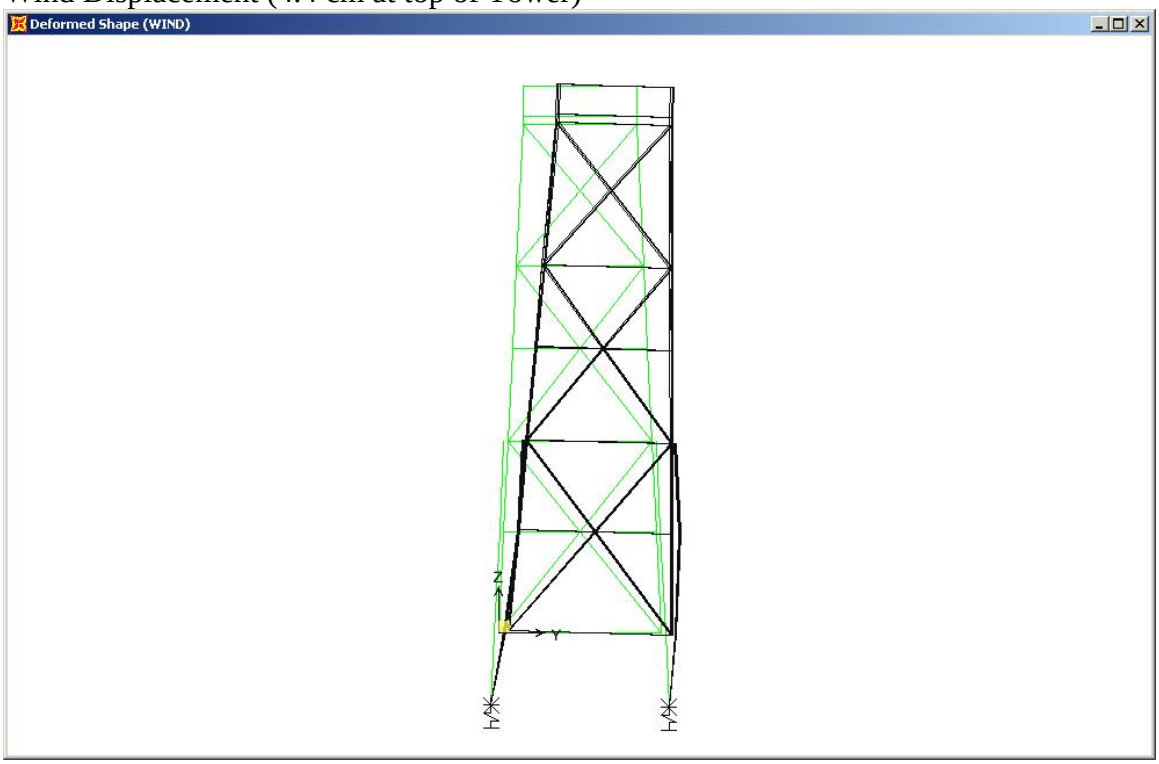
X Spectral Displacement (9.6 cm at top of Tower)



Y Spectral Displacement (9.7 cm at top of Tower)

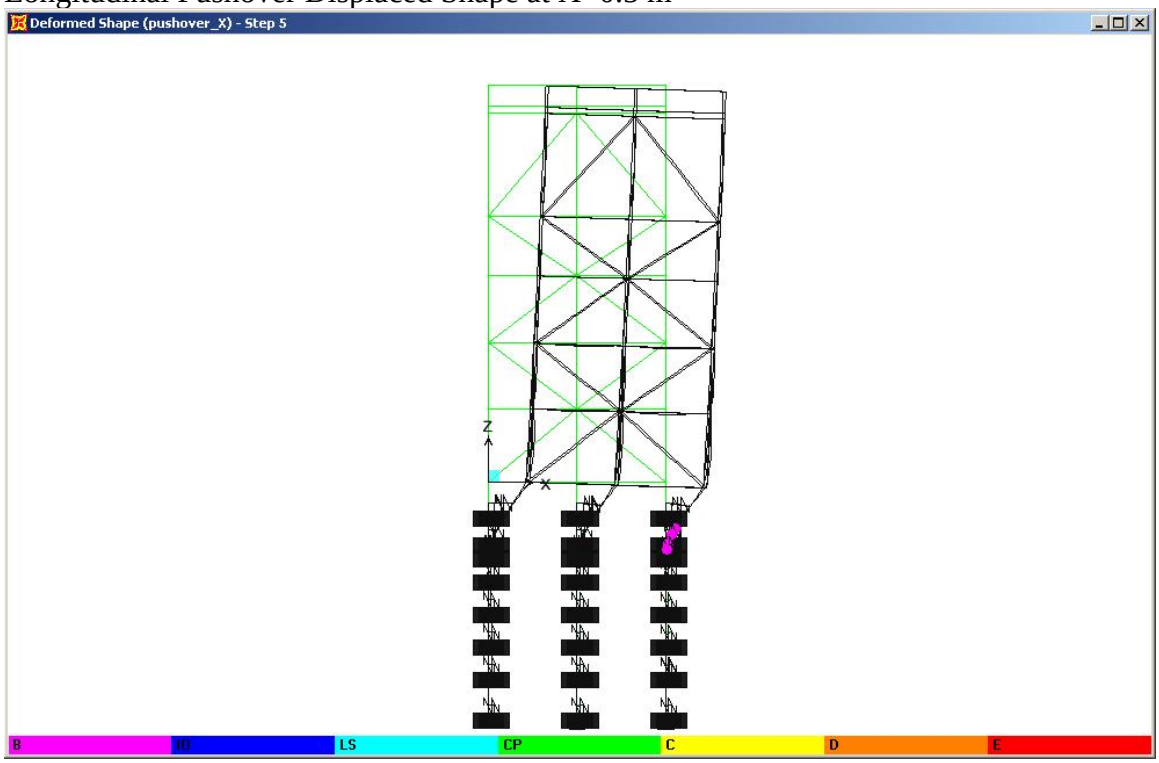


Wind Displacement (4.4 cm at top of Tower)

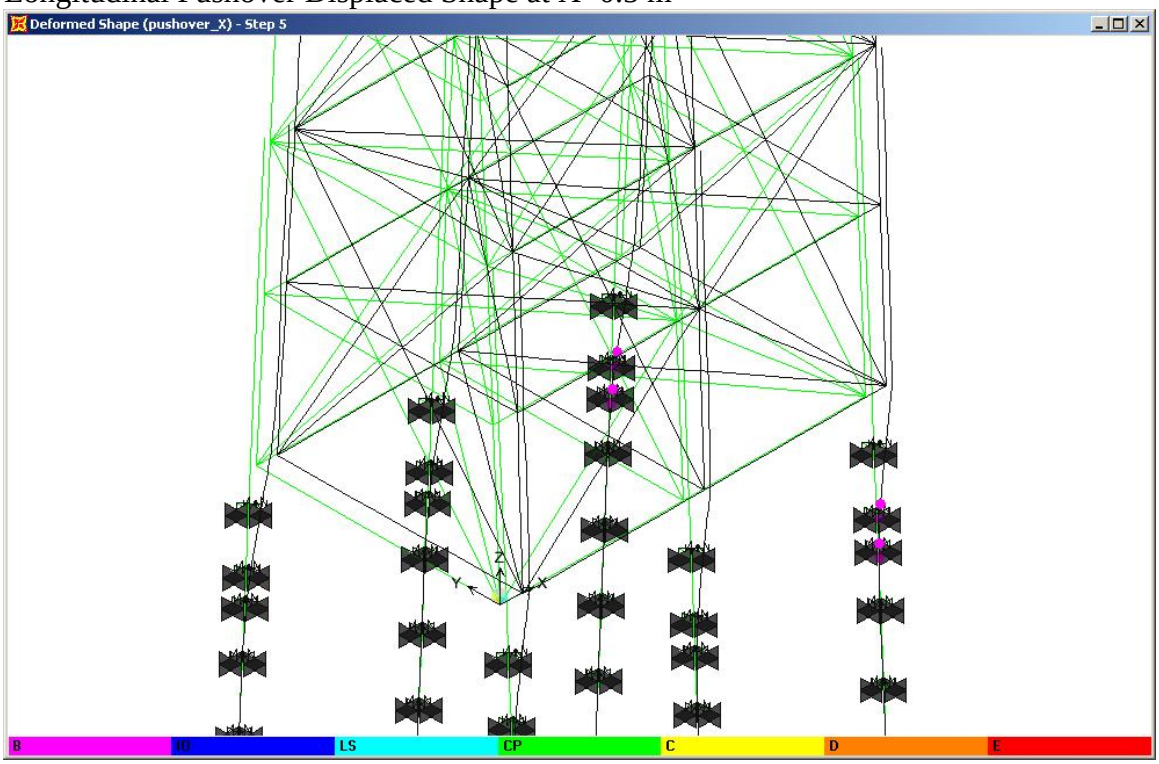


Pushover Results

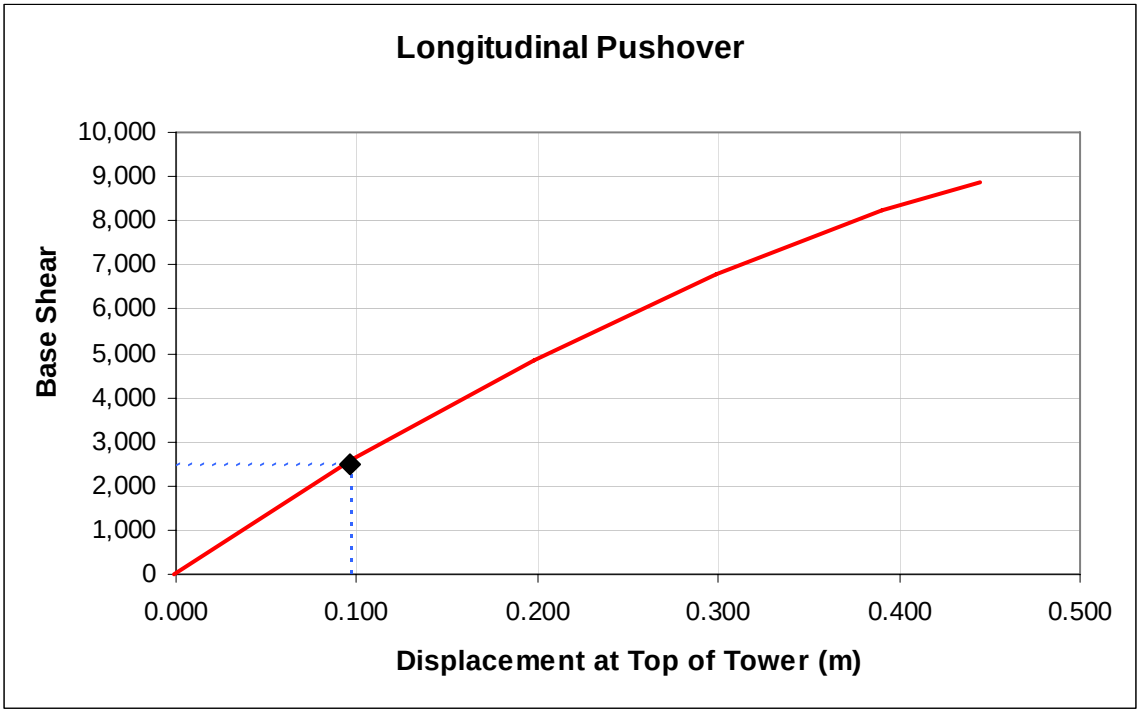
Longitudinal Pushover Displaced Shape at X=0.5 m



Longitudinal Pushover Displaced Shape at X=0.5 m

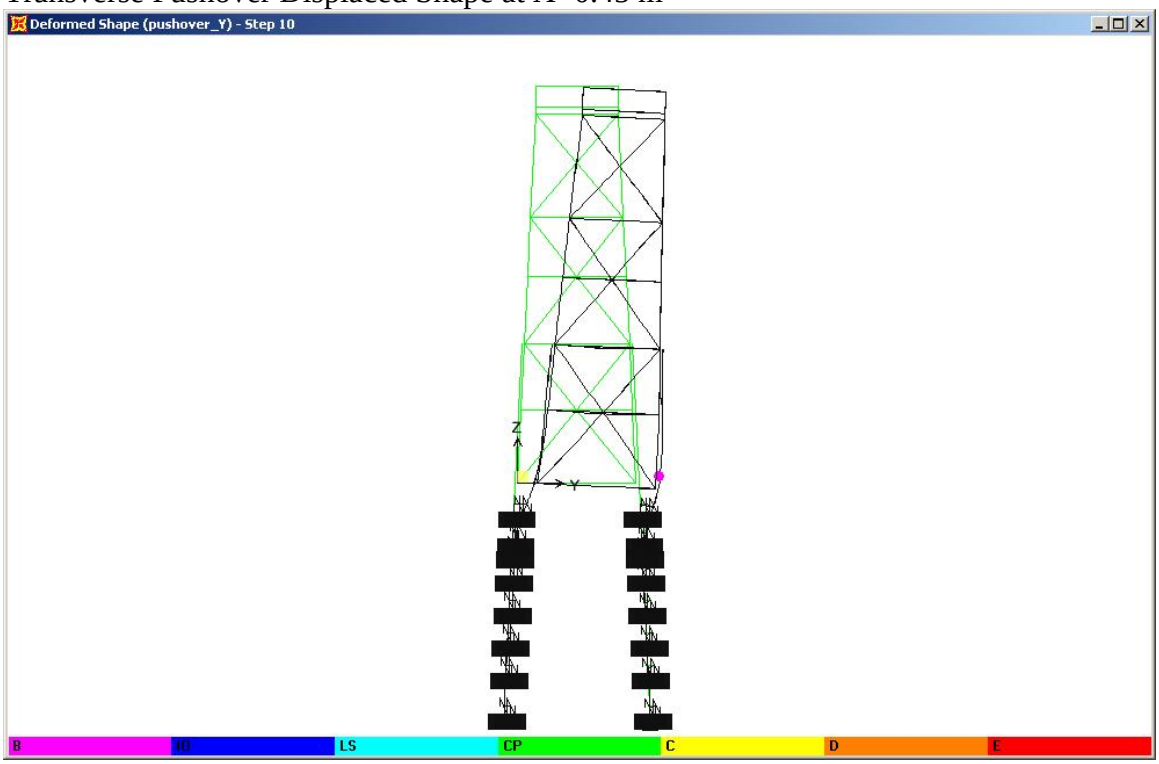


Longitudinal Pushover Curve to 0.5 m

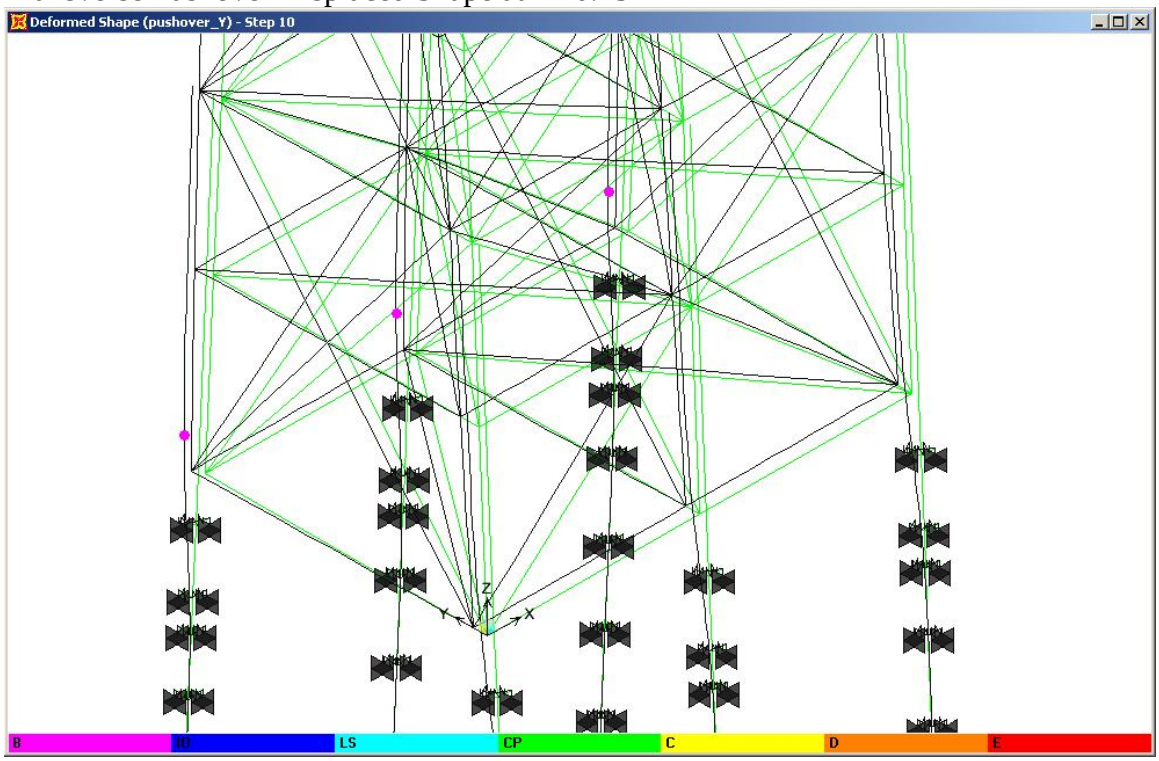


No member buckling present at this load level – piles govern capacity

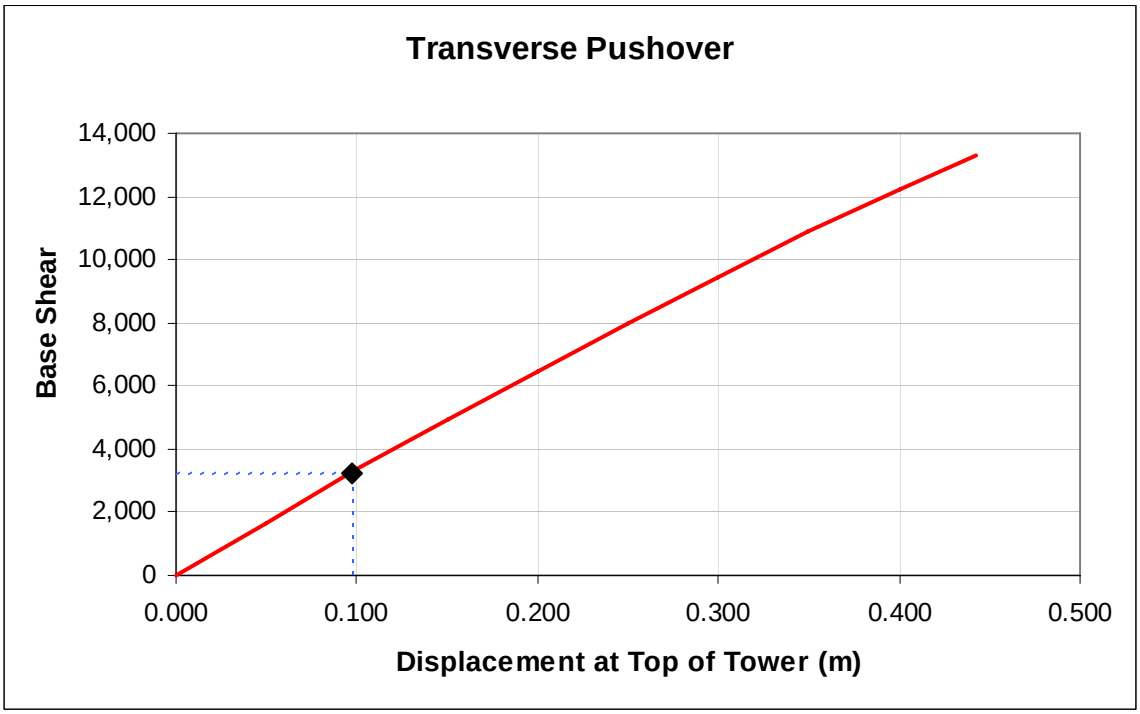
Transverse Pushover Displaced Shape at X=0.45 m



Transverse Pushover Displaced Shape at X=0.45 m

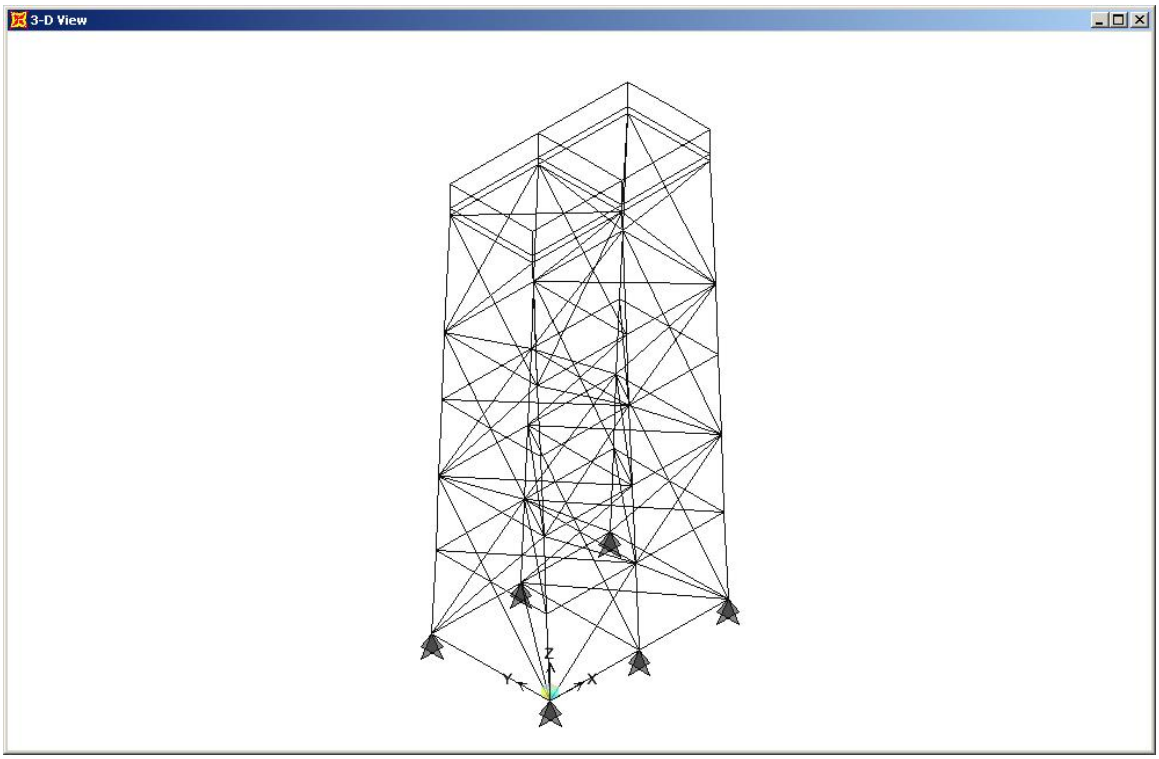


Transverse Pushover Curve to 0.45 m

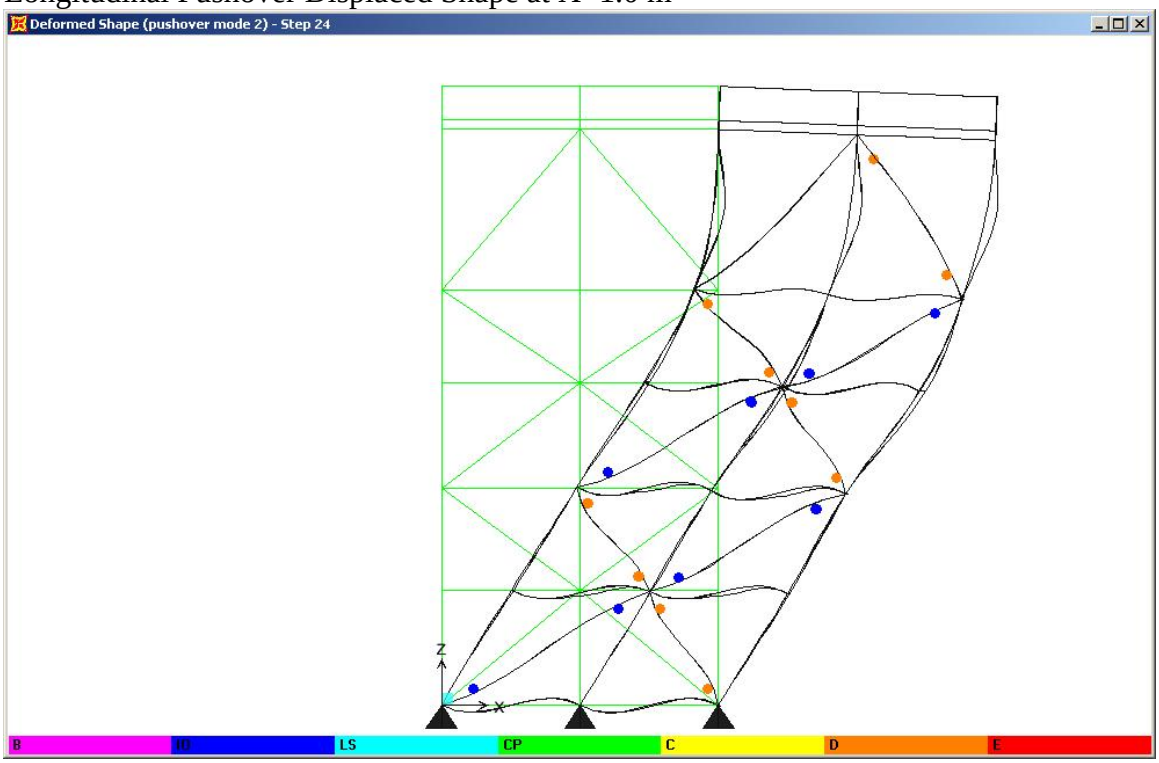


No member buckling present at this load level – piles govern capacity

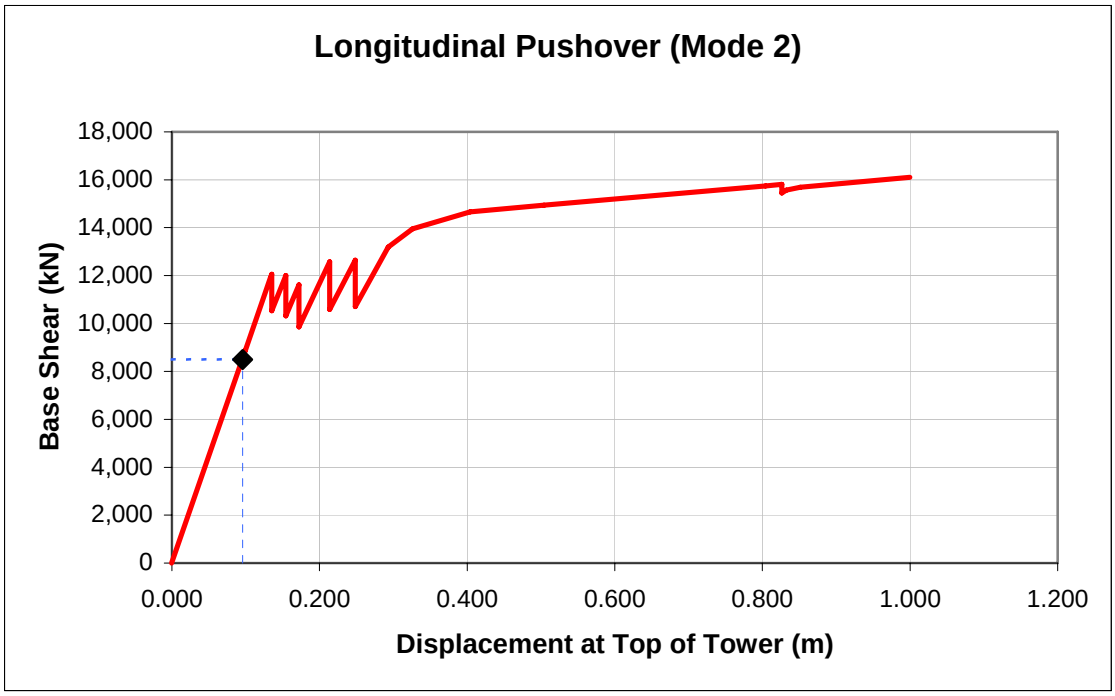
3D View of Tower G Model Pinned at the Base



Longitudinal Pushover Displaced Shape at X=1.0 m

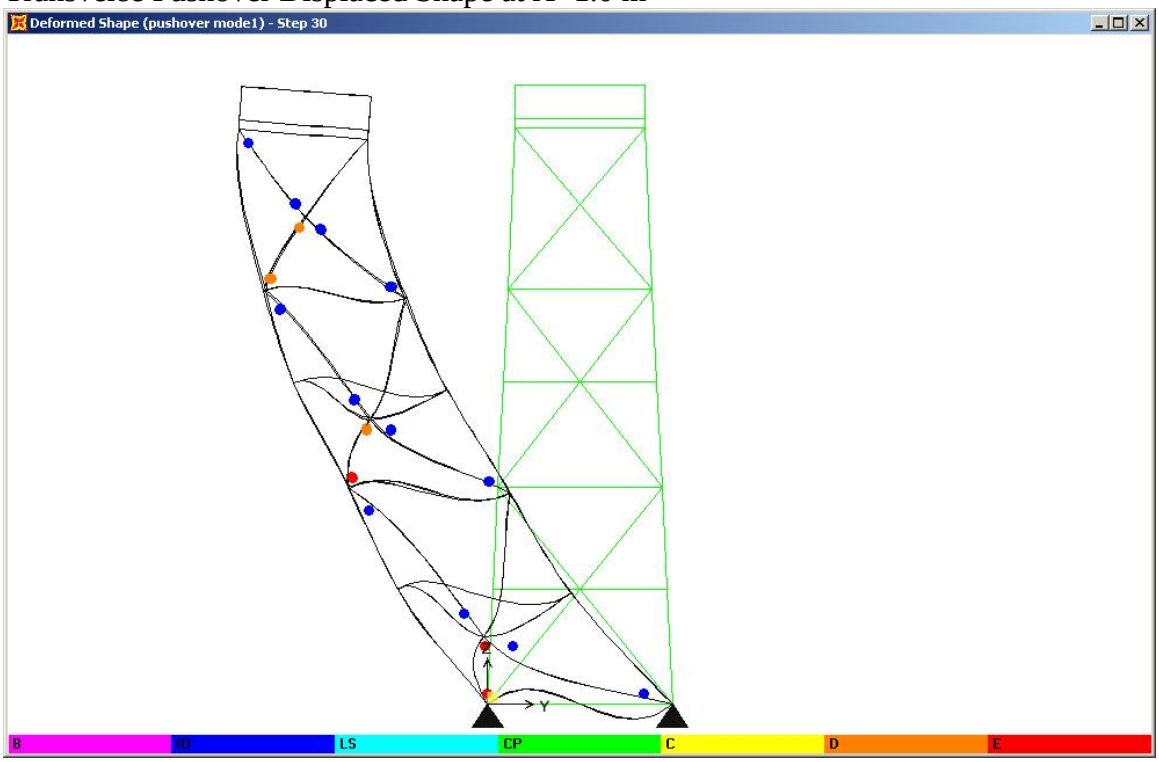


Longitudinal Pushover Curve to 1.0 m

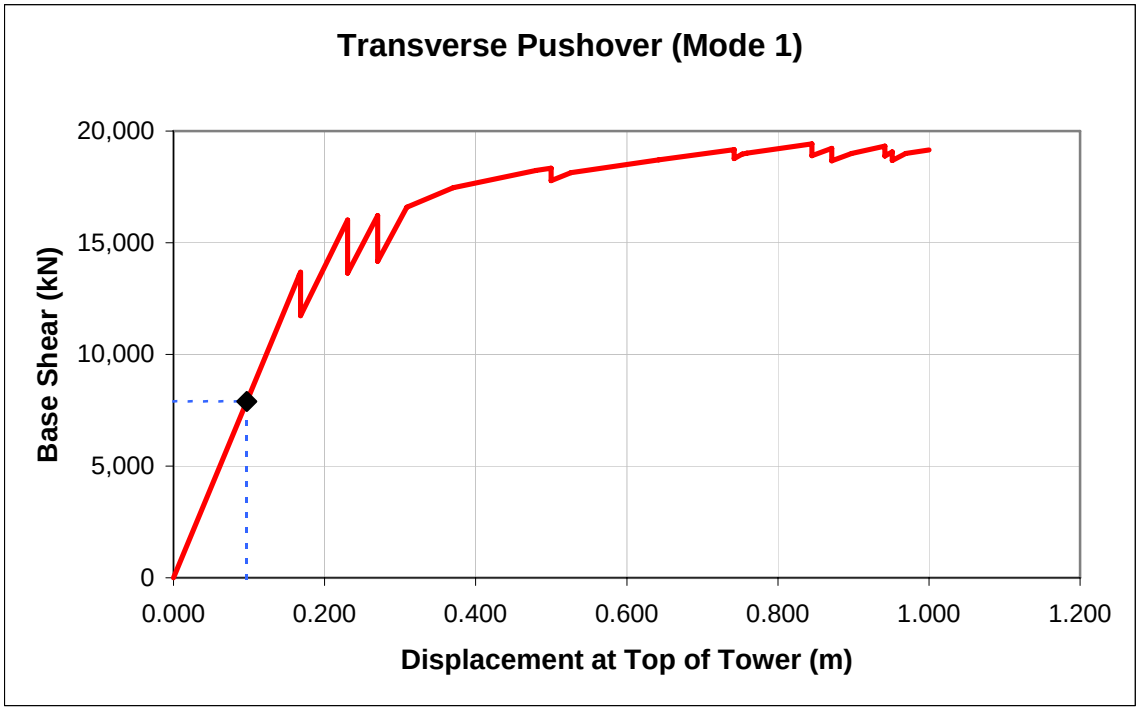


Shear reductions are due to compression member buckling (tension members take over)

Transverse Pushover Displaced Shape at X=1.0 m



Transverse Pushover Curve to 1.0 m



Shear reductions are due to compression member buckling (tension members take over)

FIGURES

Fig 2-1 – Tower C shown (Towers A&B sim)

Fig 2-2 – Temporary Tower ABC Prelim. Config.

Fig 2-3 – Conceptual Bolted Connection Details

Fig 2-4 – Temporary Tower G – Prelim. Config. (2 part tower)

Fig 2-5 – Conceptual Deck Support Cradle

Fig 2-6 – Fender Framing Options

Fig 3-1 – Temporary Structures No. 2

Fig 3-2 – Construction Load No. 1

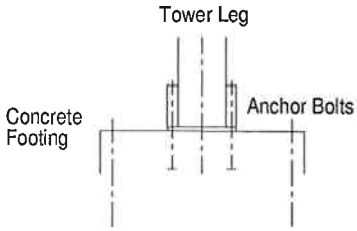
Fig 3-3 – Partial Typical Section No. 2

Fig 3-4 – Vessel Impact Diagram

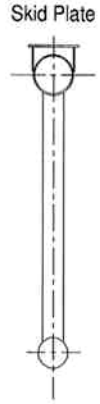
Fig 3-5 – Construction Sequence No. 2

Fig 3-6 – Wind Load Diagram

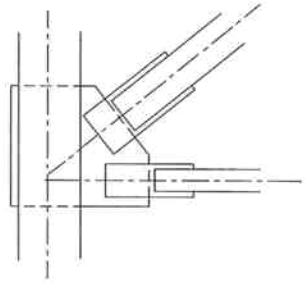
Fig 4-1 – Tower G – Jacket/Pile modeling



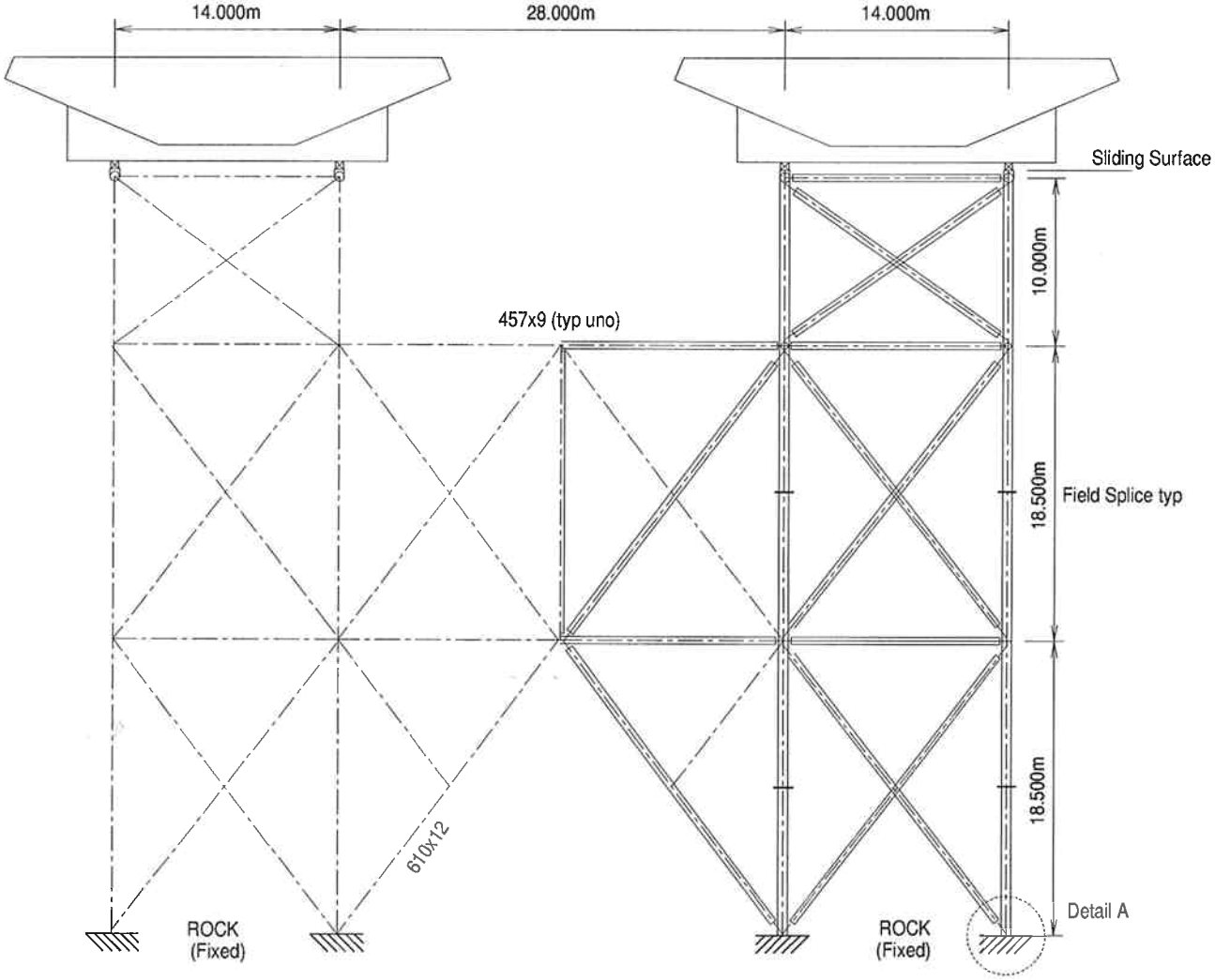
Tower Leg Base Connection A



Skid Truss Section B

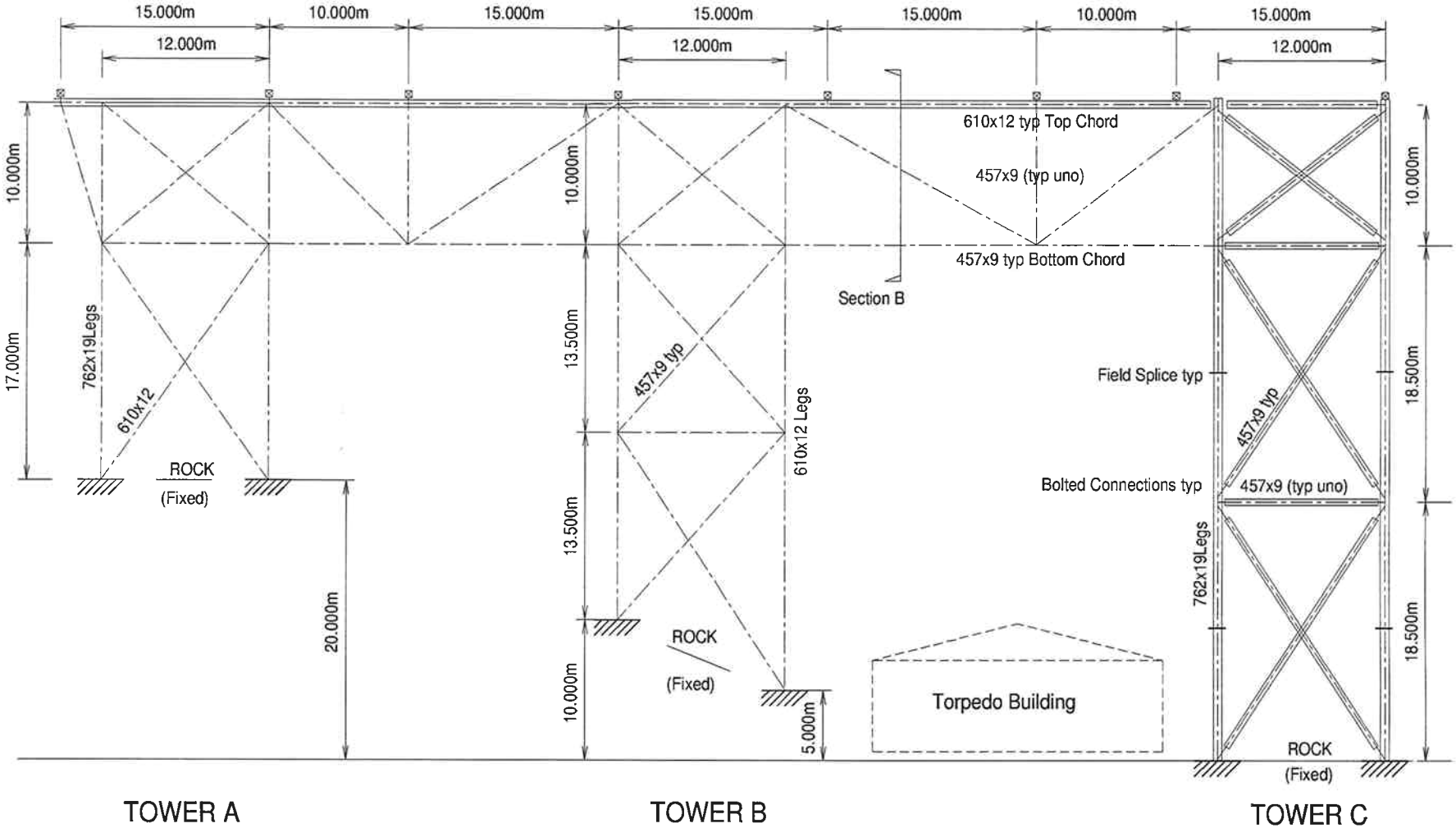


Typical Bolted Connection



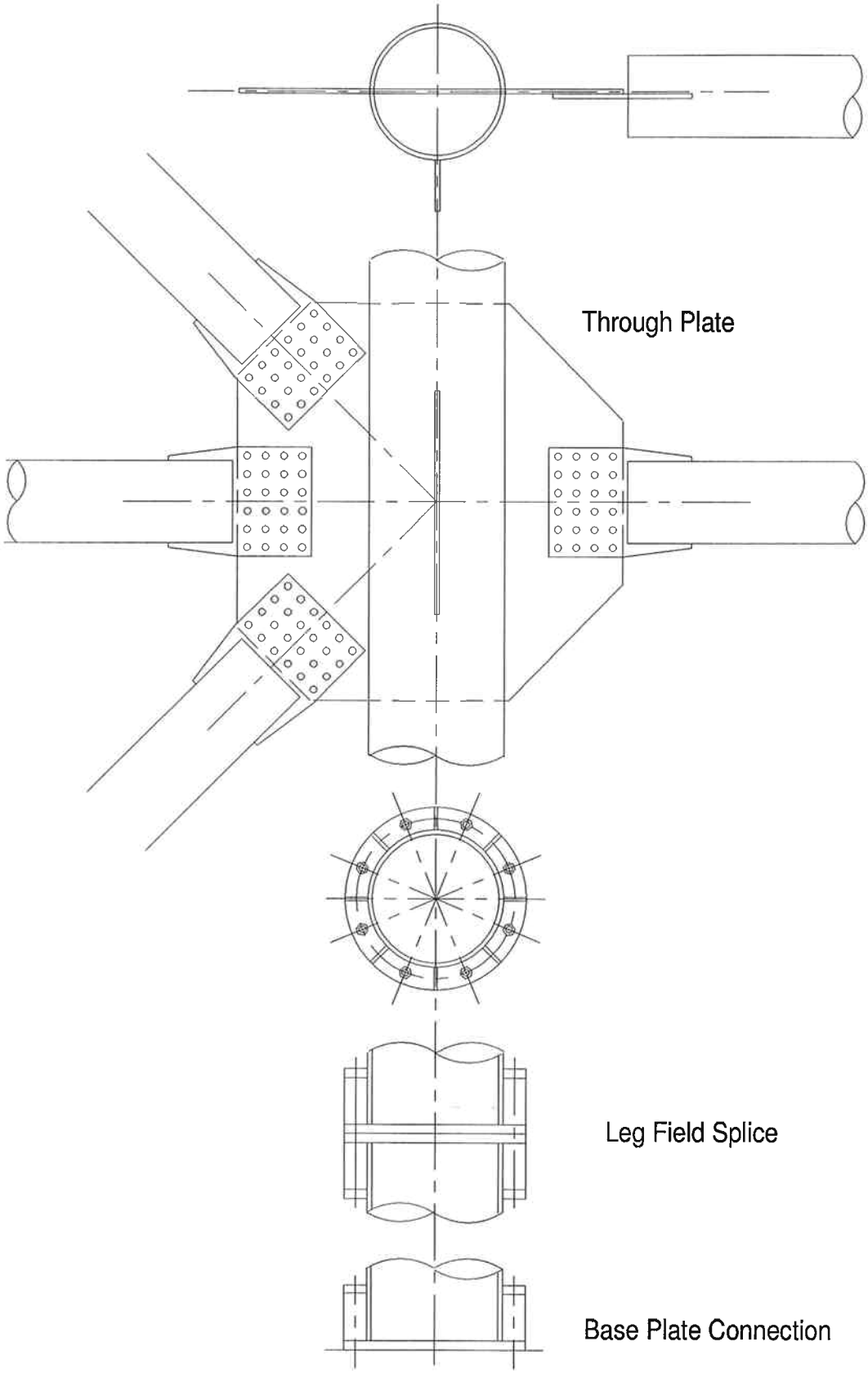
Tower C shown (Towers A&B sim)

Fig 2-1



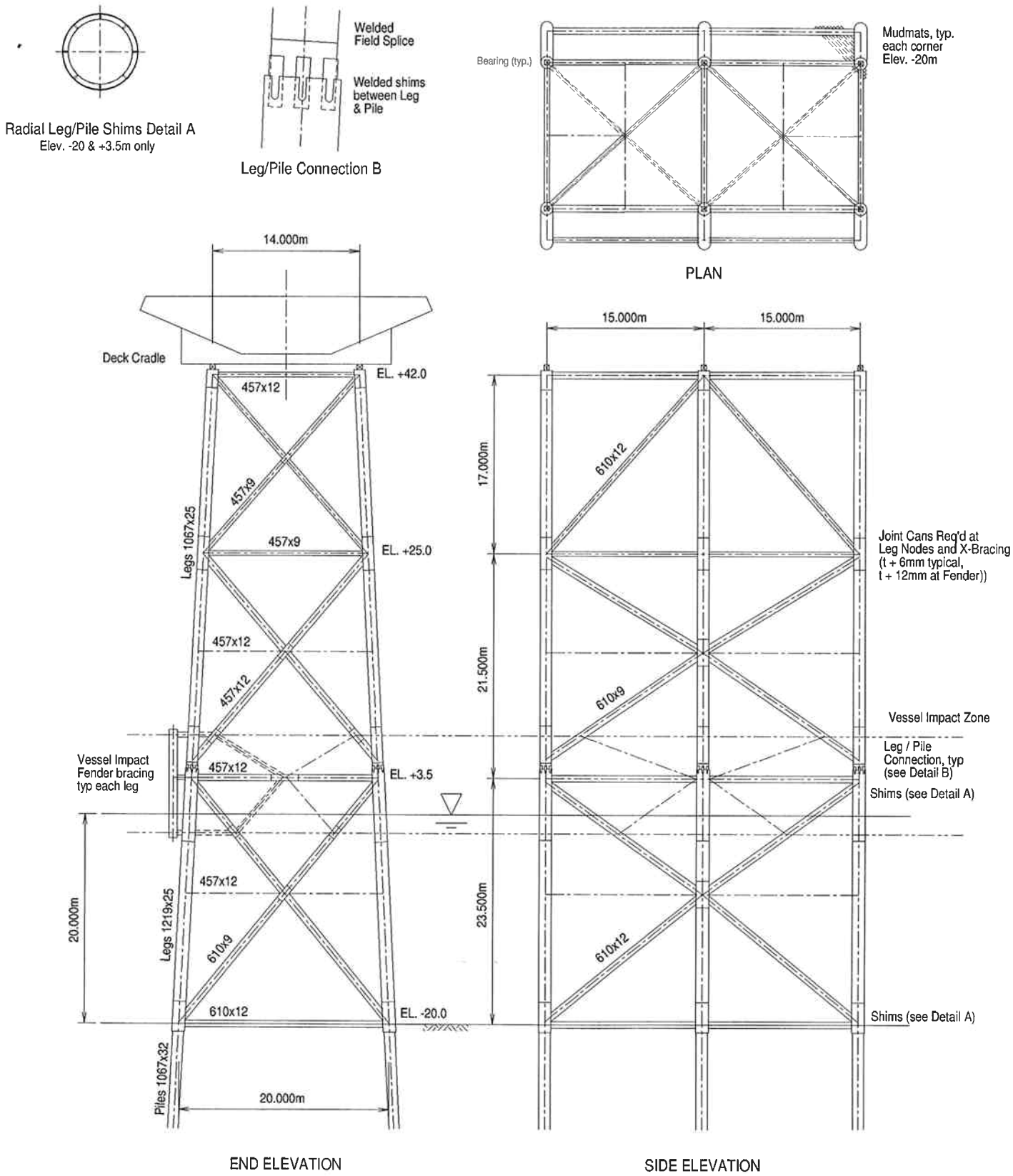
Temporary Tower ABC Prelim. Config.

Fig 2-2



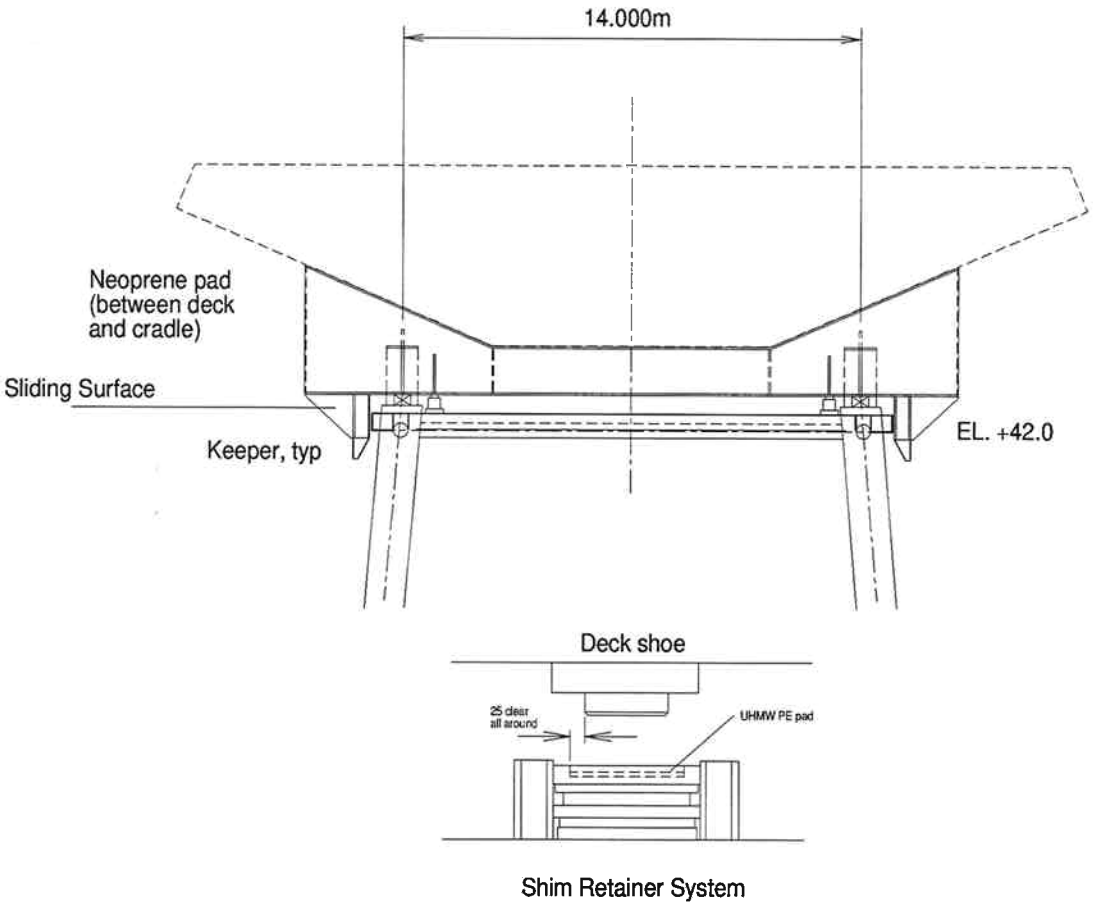
Conceptual Bolted Connection Details

Fig 2-3



Temporary Tower G - Prelim. Config.
(2 part tower)

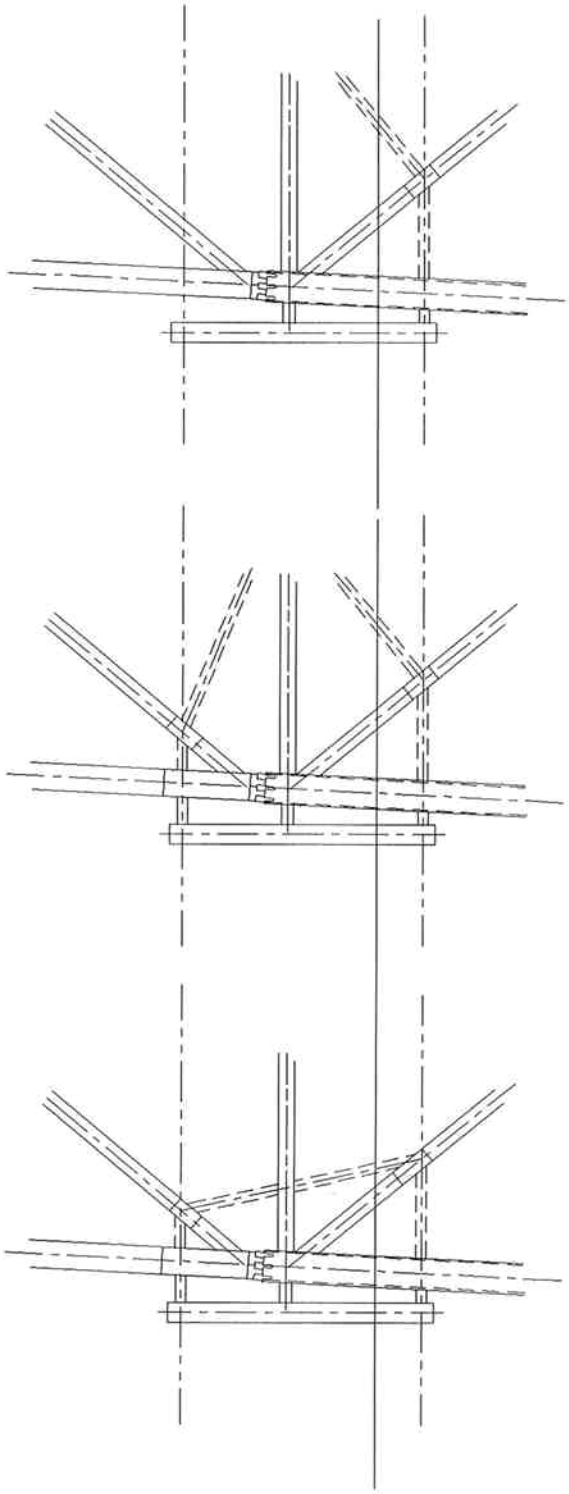
Fig 2-4



Conceptual Deck Support Cradle

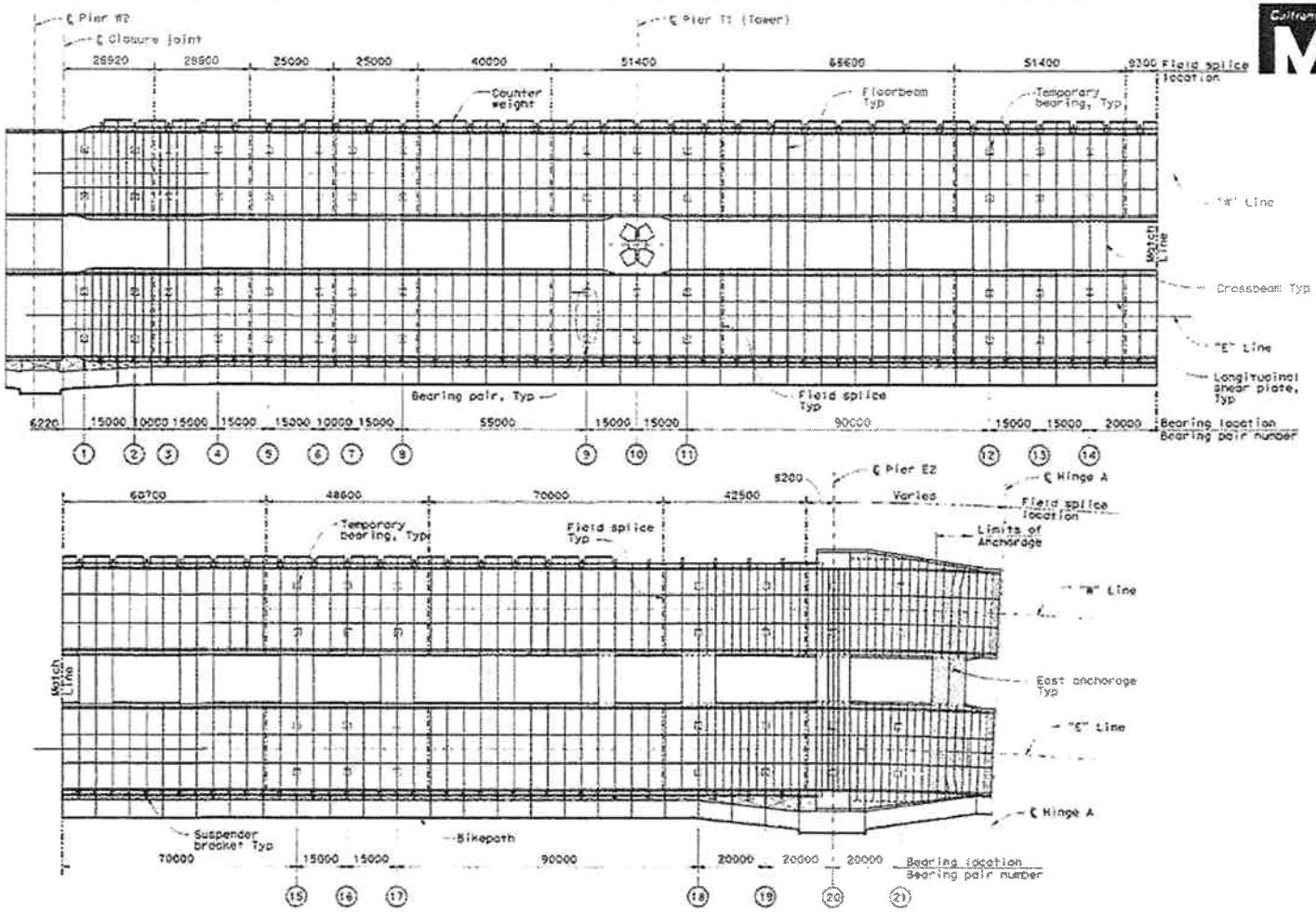
Fig 2-5

50



Fender Framing Options

Fig 2-6



FIELD SPLICE AND TEMPORARY TOWER BEARING LOCATION

NTS

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	976	1204

REGISTERED ENGINEER - CIVIL

12-6-00

PLANS APPROVAL DATE

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DETAILS	C. Wibell	CHECKED	J. Lopez-Jara
QUANTITIES	C. Wibell	CHECKED	J. Lopez-Jara

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

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KILOMETER POST 13.2/13.9

CU 04
EA 012071

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

TEMPORARY STRUCTURES NO.2

559

Fig 3-1
52

Table 1. Envelope of Vertical Forces of Temporary Towers and Temporary Bearings During Construction

Temporary Tower	Supported load per temporary tower, per box KN (See Note 1)	Temporary bearing pair number	Compression load per bearing pair, per box girder KN (See Note 1)	Tension load per bearing pair, per box girder KN (See Notes 1 & 2)
Temporary Tower A	17100	1	14900	-
		2	600	-
		3	4600	-
Temporary Tower B	11850	4	7700	-
		5	4700	-
		6	2550	-
Temporary Tower C	12650	7	1800	-
		8	12300	-
Temporary Tower D	24500	9	8200	-
		10	2700	-
		11	15850	-
Temporary Tower F	28400	12	13250	-
		13	3050	-
		14	13500	-
Temporary Tower G	28550	15	13500	-
		16	2850	-
		17	13150	-
Temporary Tower H	16800	18	16800	-
Temporary Tower E2	32850	19	9800	-
		20	26250	8300
		21	31050	-

Note: (a) No load factors are applied to these loads

(b) The above loads do not include wind or seismic effects

Table 2. Lift Weights

LIFT NO.	Estimated Weight, metric tons
1	830
2	546
3	459
4	459
5	912
6	2117
7	943
8	943
9	892
HL 1	734
HL 2	1259
HL 3	1285
HL 4	1285

(see Note 4)

HL=Heavy Lift

NOTES:

1. The loads shown do not account for construction loads and equipment, impact, eccentricities or accidental torsion. These values are for information only. The Contractor shall verify these numbers and submit for Engineer Approval.
2. The tension values shown were established assuming that all temporary bearings carry no tension (except for bearing pair No. 20). The Contractor shall carry out the proper analysis which account for proper boundary conditions based on this erection plan.
3. The temporary towers shall be designed to resist the following lateral loads:
   - Seismic Loads: Based on site-specific temporary tower response spectra (5% damping)
   - Wind Loads: ANSI/ASCE 7-95 Standard.
4. The Contactor may reduce the weight of this lift by having an additional splice. However an additional temporary tower may be required.

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	977	1204

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PROJECT ENGINEER

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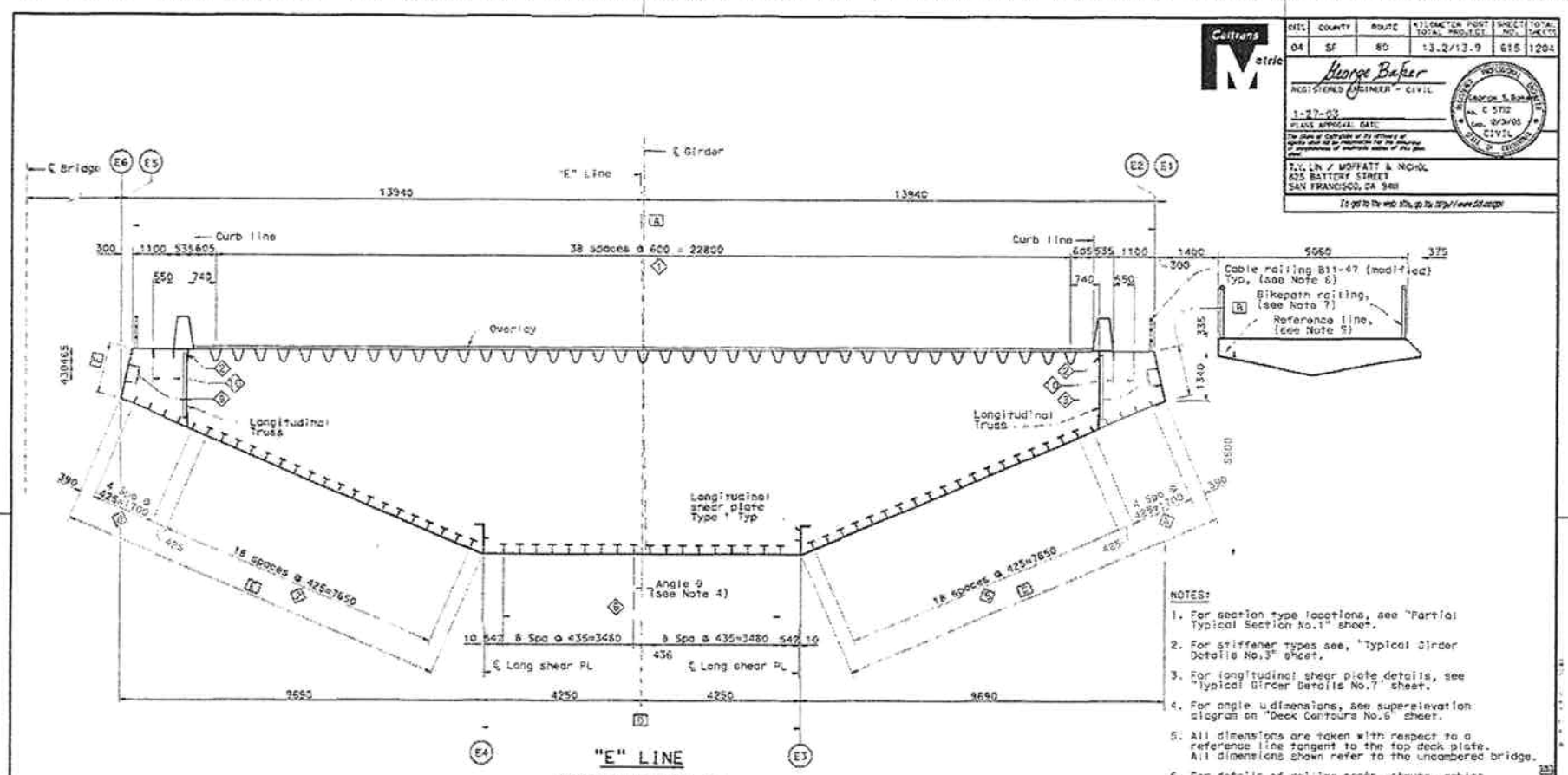
EA 01206F1

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION LOAD NO.1

Fig 3-2
53



"E" LINE

SECTION TYPE 1A

1:50

NOTES:

1. For section type locations, see "Partial Typical Section No.1" sheet.
2. For stiffener types see, "Typical Girder Details No.3" sheet.
3. For longitudinal shear plate details, see "Typical Girder Details No.7" sheet.
4. For angle ω dimensions, see superelevation diagram on "Deck Contours No.6" sheet.
5. All dimensions are taken with respect to a reference line tangent to the top deck plate. All dimensions shown refer to the uncambered bridge.
6. For details of railing posts, struts, cables and cable hardware, see "Caltrans Standard Plans B11-47" sheet. For details of post and strut connections, see "Caltrans Standard Plans B11-7" sheet. For base details, see "Typical Girder Details No.12" sheet.
7. For details of bikepath railing, see "Bikepath Details" sheets.
8. For longitudinal truss, see "Typical Girder Details No.12" sheet.

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	615	1204

George Baker
REGISTERED ENGINEER - CIVIL
1-27-03
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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PARTIAL TYPICAL SECTION NO.2

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

DESIGN C. Baker · DRAWN T. McKeone
DETAILS V. Quiyas · CHECKED T. McKeone
QUANTITIES D. Baker · CHECKED M. Roberts

R. Gonzalez
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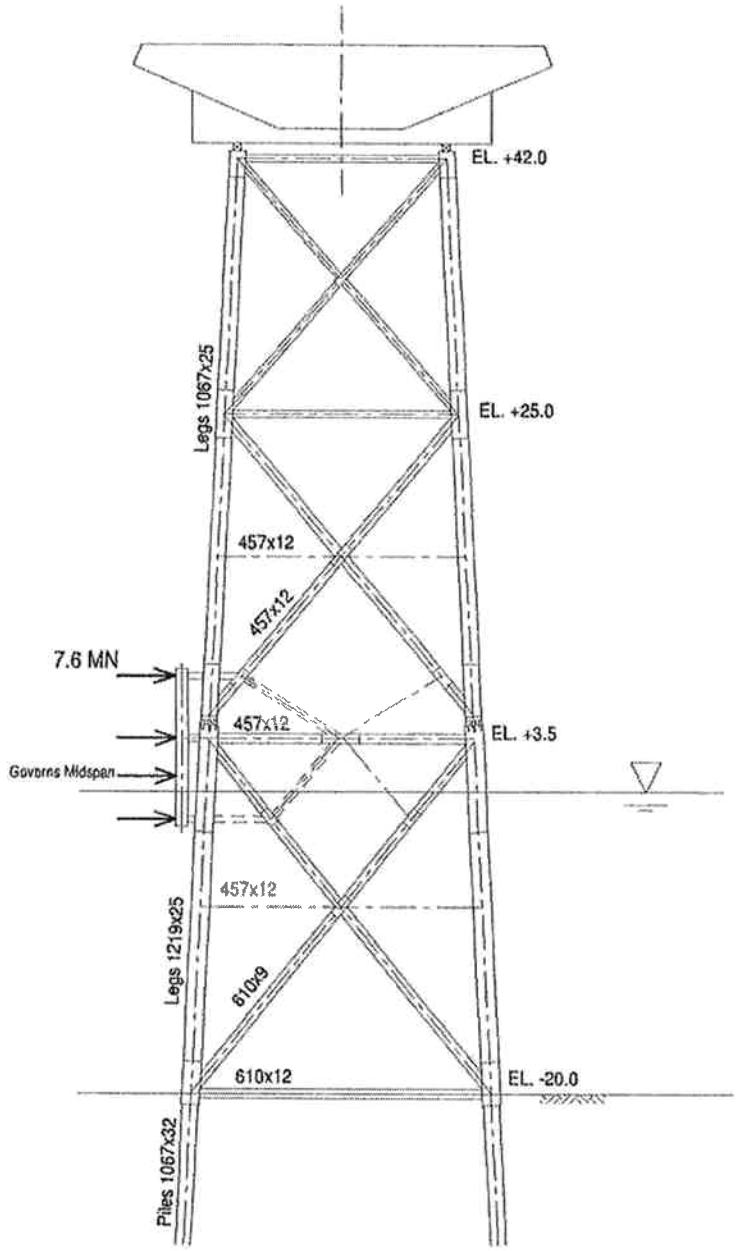
EA 012081

34-0006L/R
13.2/13.9

198

Fig 3-3
54



END ELEVATION

Vessel Impact Diagram

Fig 3-4

9. Temporary Towers:

The following minimum horizontal design response spectrum chart shall be used in the design of the temporary towers.

Period (Second)	PSA (g)
0.01	0.7
0.25	0.7
1.00	0.2
2.00	0.1
3.00	0.06
4.00	0.05
5.00	0.05
6.00	0.05

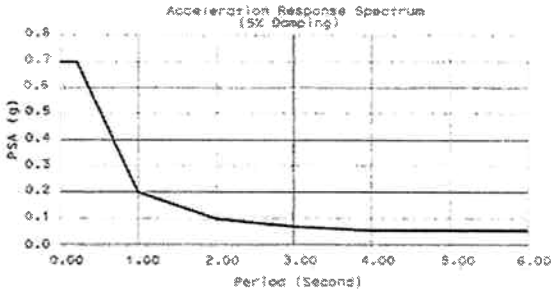


Fig 3-5

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
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825 BATTERY STREET
SAN FRANCISCO, CA 94111

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PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Vallooran/V. Tom/Y.L./M.L./J.C.
DESIGN ENGINEER

DESIGN: M. Nader — CHECKED: C. Sale
DETAILS: C. Mibelli — CHECKED: J. Lopez-Jara
QUANTITIES: C. Mibelli — CHECKED: J. Lopez-Jara

R. Montgomery
PROJECT ENGINEER

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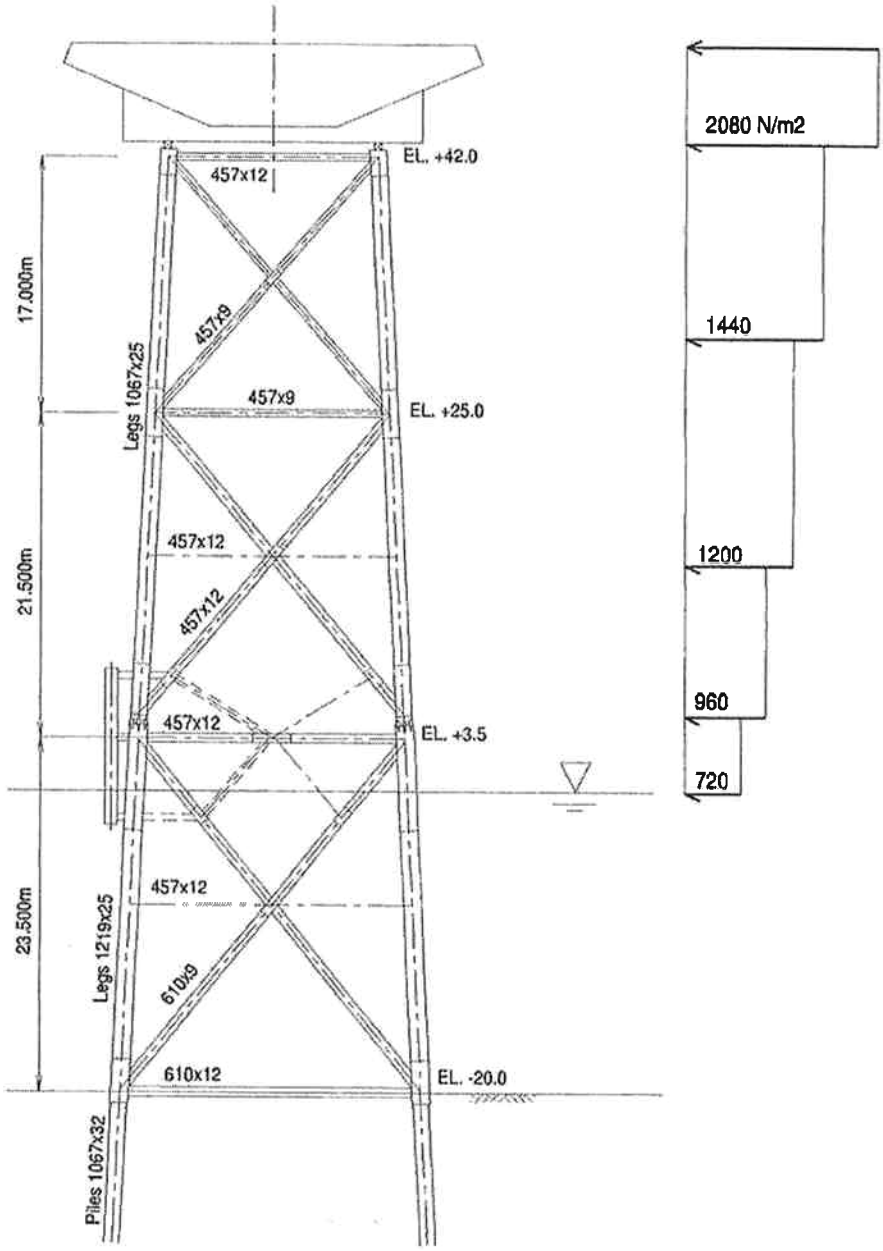
KILOMETER POST 13.2/13.9

CU 04
EA 0120F1

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)
CONSTRUCTION SEQUENCE NO.2

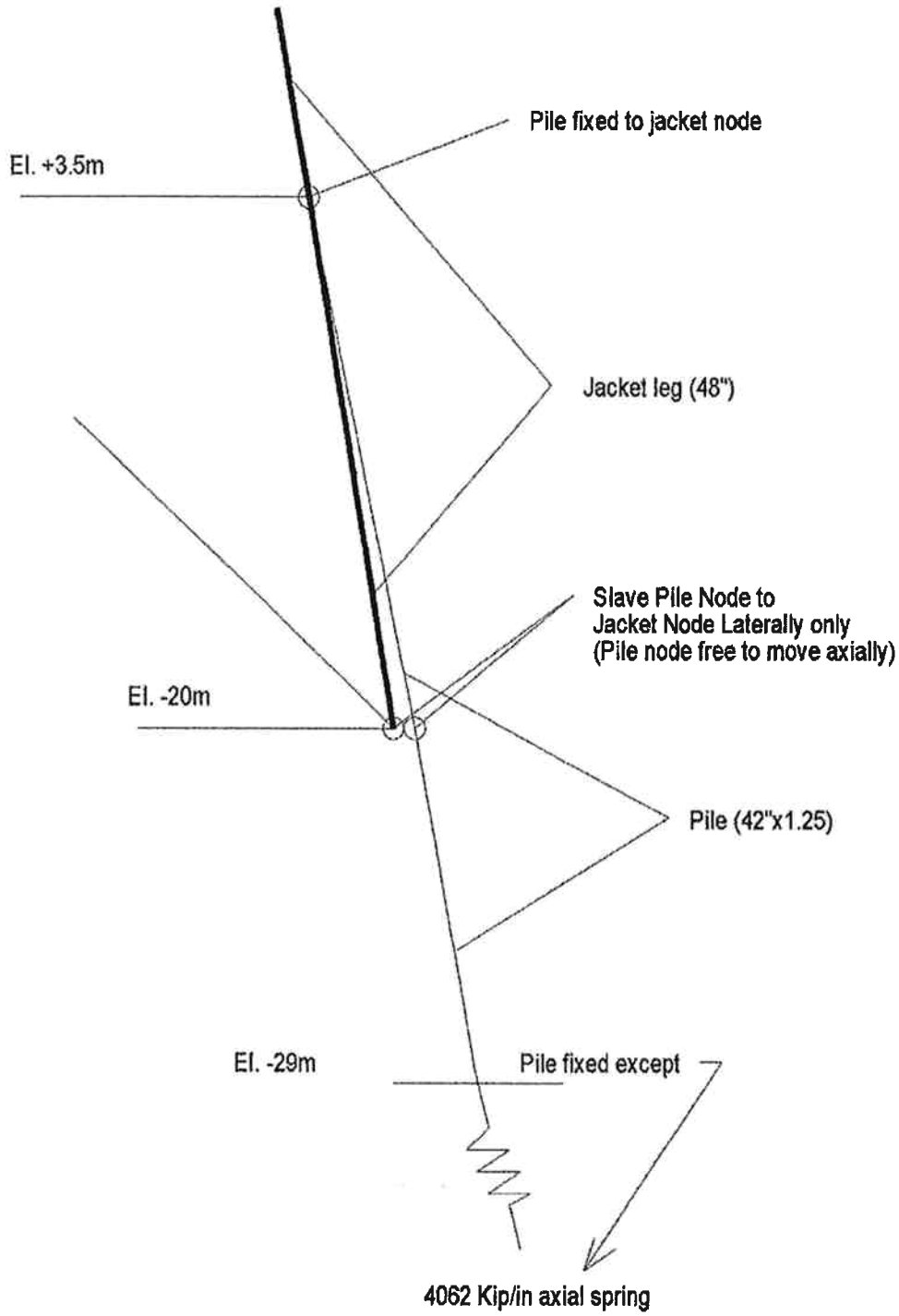
551



END ELEVATION

Wind Load Diagram

Fig 3-6



Tower G - Jacket /Pile modeling

Fig 4-1

APPENDICES

A. Design Drawings

B. Special Provisions

C. Sample calculations

Example API Member Check
Example API Joint Check including Member Capacity Check
Example Bolted Joint Check including Member Capacity Check

D. Misc. Hand/Spreadsheet Calculations

Preliminary Design and Loads Development
Various Pile Checks
Impact Fender & Bracing Check

NOT FOR CONSTUCTION

Appendix A

Design Drawings

Temporary Tower drawings
(Sheets 967 thru 978 of 1204 - Contract No. 04-0120F4)

NOT FOR CONSTUCTION

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	967	1204

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SAN FRANCISCO, CA 94111

To get to the web site, go to http://www.dot.ca.gov

General:

1. The construction sequence shown in "Construction Sequence 2 through 7", "Tower Construction Sequence", "Pier W2 Construction Sequence", "Hinge A Construction Sequence", "Hinge K Construction Sequence", "Tower Fender Construction Sequence No.1 and No.2", "Temporary Structures" and "Construction Loads" sheets, are schematic and are for information only. They are not intended to depict ways and means to be used by the Contractor. The Contractor shall verify all values shown on these sheets and submit them to the Engineer for review and approval.

2. The Contractor shall develop an Erection Plan consistent with his operations. This Erection Plan shall consist of a step-by-step construction sequence along with analyses, calculations, drawings, camber and all other relevant material to substantiate the proposed construction methods. This Erection Plan shall be sealed by a Professional Engineer licensed in California and shall be submitted to the Engineer for review and approval

3. The Contractor shall perform the necessary analyses to prove that the structure is stable and the stresses are within the requirements of the design criteria for each construction stage. Global and local analyses shall be performed to establish these stresses.

4. All temporary structures attachments and components used to reinforce the structure shall be removed after dead load transfer. The structure segments shall be returned to their original intended design proportions and weight. All work of cutting and grinding associated with temporary structure removal shall be documented in the Erection Plan and shall be approved by the Engineer before implementation. The shop drawings shall clearly indicate all temporary structure attachments. Temporary reinforcement and its removal shall be at the Contractor's cost.

Construction Requirements:

1. The Contractor shall erect a bridge structure such that the suspension system introduces dead load moments in the suspended box girders within the allowable bounds shown in Chart A. Such moments are controlled by appropriate dimensioning and adjustments of the suspenders which influence the loading on the cable. The box girders may be cambered to the extent that the profile grade is achieved, to the specified tolerances, under the Contractors proposed dead load moments. The Contractor's attention is directed to the requirement to have small dead load moment at the W2 Closure joint. See note 7.

2. Box Girder Erection Requirements:

   a. The Contractor shall provide temporary towers to support the box girders. The Contractor has the option to support the box girders between these temporary towers with temporary trusses.

   b. Navigational clearance required by the Coast Guard shall be maintained. The temporary towers shall provide for minimum horizontal and vertical clearances as shown in View A-A.

   c. The box girders shall be erected in their entirety from Hinge A to Hinge K, including all cross beams, before the cable is erected. The Contractor may choose to leave part of the box girder "open" at the east anchorage to have better access for erecting the cable and to accommodate the existing pipe beams at Hinge A.

   d. Adjacent segments of box girders or cross beams shall be aligned and restrained from relative movement before field splicing. This alignment may be achieved by applying moments at the interfaces, applying forces at the temporary tower bearings or by any other means.

3. Suspender Erection Requirements:

   a. The suspender loads shall be verified based on a construction sequence analysis that uses the fabricated and installed weights, and that confirms the achievement of the specified box girder dead load moment requirements. The installed suspender lengths, measured between the cable and the working points, shall be calculated based on the prescribed suspender loads and shall be consistent with the cable and girder geometry of the completed structure, as determined by the Contractor's construction sequence analysis.

   b. The Contractor is alerted to the fact that the "free hanging" cable does not lie in the same plane as the "loaded cable". Dead load transfer will cause translation and rotation of the cable to meet its position under the design dead load condition. The Contractor shall make the necessary provisions to compensate for such rotations in order to orient the cable bands correctly and to minimize the friction over the rope saddles.

   c. Cable band bolts shall be tightened to specified values before and after the load is transferred.

   d. The jacking of the west jacking saddle shall be coordinated with the dead load transfer through the suspenders in order to provide equal cable force at either side of the deviation saddle and ensure that there will be no wire slippage at the west deviation saddle.

   e. The Contractor shall take the necessary measures to prevent contact between the cable and the box girder. The Contractor shall also ensure that cable angle break does not exceed 6 degrees anywhere along the cable during construction.

4. Geometry Control:

   The alignment, grade profile and dimensions of the bridge at the end of construction shall be consistent with the Plans. The Contractor shall survey the geometry of the cable and deck profile and check against calculated geometry throughout the construction stages.

5. Weight Control:

   The weight of the suspended structure (box girder, cross beam, bikepath, cable system, pipe beam, deck overlay, utilities, access platforms, barrier, counter weight and all other components that contribute to the superstructure weight shall be accurately monitored using an approved weight control procedure.

6. **Movements During Construction:**

   The Contractor shall take the necessary measures to a low for cable system movements during the construction particularly at the top of the Tower (Tower Saddle), Pier E2 Bearing and Shear Key, East Saddle and Cable in main span and back span. The attention of the Contractor is directed to the fact that measures have to be taken to avoid potential interference between the cable and the box girder in the vicinity of the east saddle and between the cable and the tower saddle.

7. If the Contractor elects to use the Alternative Camber Method, the box girder dead load moments shall be as specified on "Camber Schedule-B" sheet.

8. The Contractor may erect temporary work platforms west of Pier W2 to facilitate the Cable erection and suspender load transfer. The Contractor shall include the platform in his Erection Plan. The platforms shall be located within 17 m west of CL Pier W2 (along Bridge alignment) and shall not interfere with the Existing Bridge. These platforms shall be removed after suspender load transfer is completed.

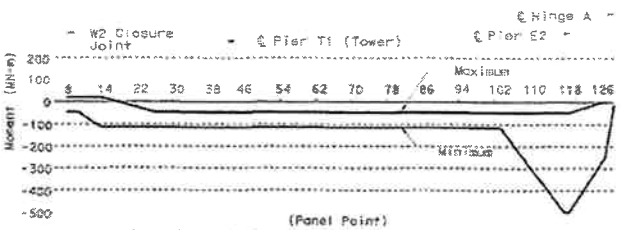
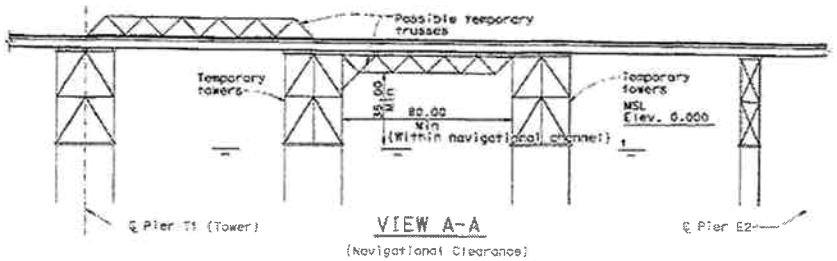


Chart A: Allowable range for Box Girder Dead Load Moment

Note: This range accounts for the reactions from the Skyway Structures and Yerba Buena Transition Structures at Hinges A & K respectively. See "Suspension Cable Layout" sheets.



VIEW A-A

(Navigational Clearance)

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Y. Lee

DESIGN M. Nader
DETAILS C. Mibelli
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CHECKED J. Lopez-Jara

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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

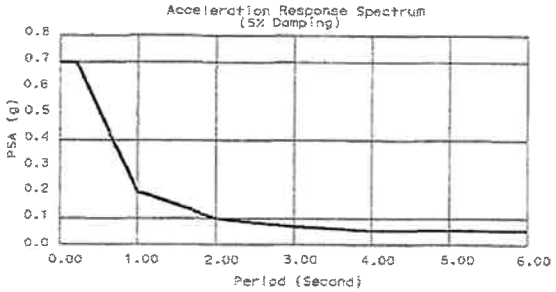
CONSTRUCTION SEQUENCE NO.1

550

9. Temporary Towers:

The following minimum horizontal design response spectrum chart shall be used in the design of the temporary towers.

Period (Second)	PSA (g)
0.01	0.7
0.25	0.7
1.00	0.2
2.00	0.1
3.00	0.06
4.00	0.05
5.00	0.05
6.00	0.05



DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	568	1204

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100 % PS&E

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

DESIGN: M. Nader; DETAILS: C. Mibelli; QUANTITIES: C. Mibelli

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R. Monzonarez, PROJECT ENGINEER

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EA 0120F1

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

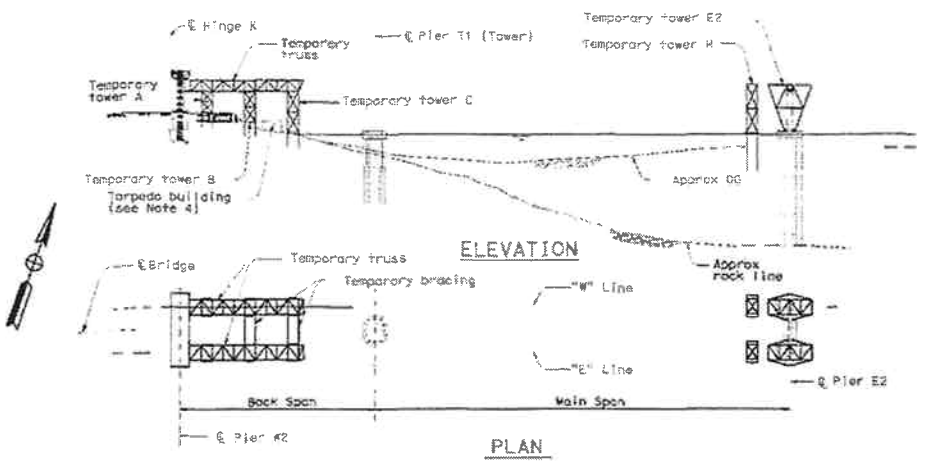
CONSTRUCTION SEQUENCE NO.2

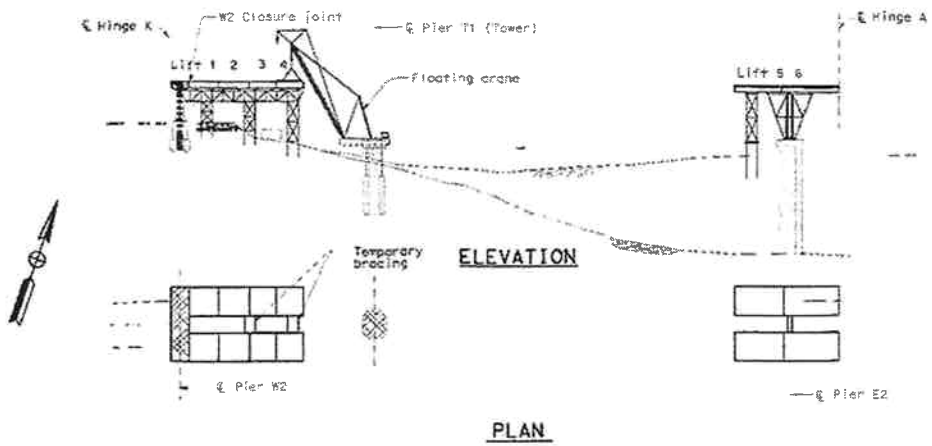
551

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	969	1204

REGISTERED ENGINEER - CIVIL
1-27-03
PLANS APPROVAL DATE

T.Y. LIN / MOFFATT & NICHOL
825 BATTERY STREET
SAN FRANCISCO, CA 94111





Step 1

a. Construct Foundations for Pier W2, Pier E2 and Tower, (By Others).

b. Construct Pier W2 and Pier E2, (By Others).

c. Construct Pier W2 Cap beam and Pier E2 Cap beam. For Pier W2 construction, see "Pier W2 Construction Sequence" sheet.

d. Drive/drill temporary tower piles.

e. Erect temporary towers A, B, C, H, E2 and temporary truss for back span (including all foundations).

Step 2

a. With floating crane. Lift four (4) box girder sections for each roadway on temporary truss (Lifts 1, 2, 3 and 4). Roll each box girder section west into position, align and make up box girder section splices. Connection between W2 concrete cap beam and steel box girder (W2 closure joint) is to be made in Step 7. Temporary vertical and transverse supports between the box girder and Pier W2 cap beam shall be made instead.

b. Lift two (2) box girder sections for each roadway (Lifts 5 and 6), onto temporary tower E2, align and make up box girder section splices. Lift 6 needs to be tied down at Pier E2 until it has been spliced with lift 5. The tie down shall be removed after splicing.

LEGEND:

Denotes previously erected sections.

NOTES:

1. Erection procedure continued on "Construction Sequence No.3" sheet.
2. Contractor is responsible for developing a detailed construction guide manual for Pier W2 and Pier E2 cap beams that includes, but is not limited to, construction scheme, stressing sequence, and relevant construction engineering calculations. The construction guide manual shall be submitted by the Contractor to the Engineer for review and approval, prior to start of construction.
3. Skyway temporary towers AE & AW not shown on "Construction Sequence No.2 through No.7" sheets for clarity.
4. Torpedo building to remain in place.
5. Fender at pier E2 (by others) not shown.

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PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Wanzenriedt/V. Tan/T.L./W.L./F.C.
DESIGN OVERSIGHT
Y. Lin

DESIGN M. Nader — CHECKED C. Seim
DETAILS C. Mitelli — DRAWN J. Lopez-Jara
QUANTITIES C. Mitelli — DRAWN J. Lopez-Jara

R. Wanzenriedt
PROJECT ENGINEER

CU 04
EA 012071

BRIDGE NO. 34-0006L/R
PREFIX POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION SEQUENCE NO.3

552

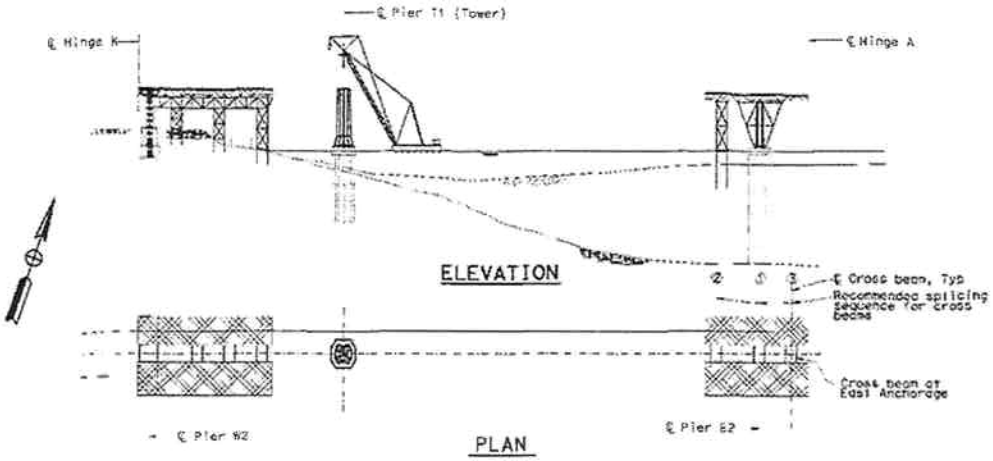
100 % PS&E

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	970	1204

REGISTERED ENGINEER - CIVIL

12-6-04
PLANS APPROVAL DATE

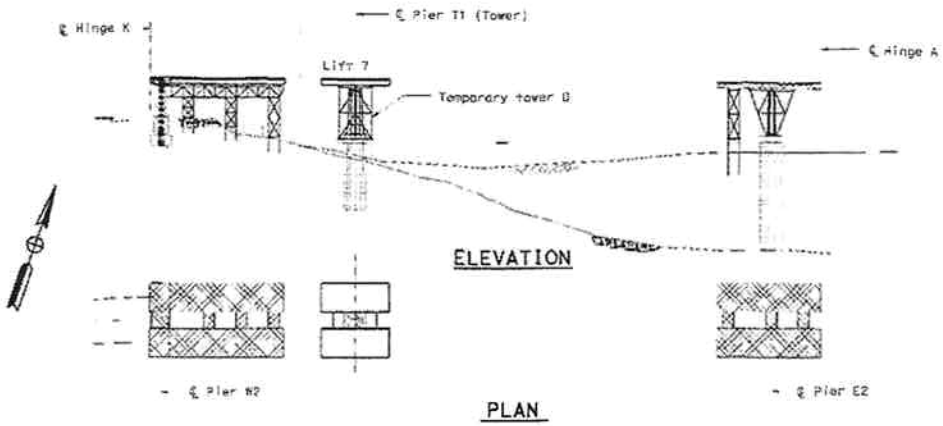
T.Y. LIN / MOFFATT & NICHOL
825 BATTERY STREET
SAN FRANCISCO, CA 94111



Step 3

a. Erect tower up to tower splice no.1 elevation and make up all tower splices. For tower erection procedure, see "Tower Construction Sequence" sheet.

b. Erect cross beams, align and make up splices to erected box girder sections. Box girders at East Anchorage shall be jacked transversly in order to align and splice cross beam (see Note 3).



Step 4

a. Drive piles for temporary tower D.

b. Erect temporary tower D, lift one (1) box girder section for each roadway, align and make up splices (Lift 7).

c. Erect cross beams, align and make up splices to erected box girder sections.

NOTES:

1. Erection procedure continued on "Construction Sequence No.4" sheet.
2. Falsework bracing not shown for clarity.
3. For cross beam at East Anchorage, see "Cross Beam at East Anchorage" sheets.
4. Tower could be erected to splice no.4 in step 3a instead of splice no.1 to benefit from the accessibility before the box girder (Lift 7) is erected.
5. Fender at Pier E2 (by others) not shown.

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION SEQUENCE NO.4

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

DESIGN: M. Noori; DETAILS: C. Mibelli; QUANTITIES: C. Mibelli; CHECKED: C. Sein; J. Lopez-Jara; J. Lopez-Jara

R. Wolfzoder/V. Toor/T.L./M.L./F.C.
DESIGN OVERSIGHT: Y. Lin

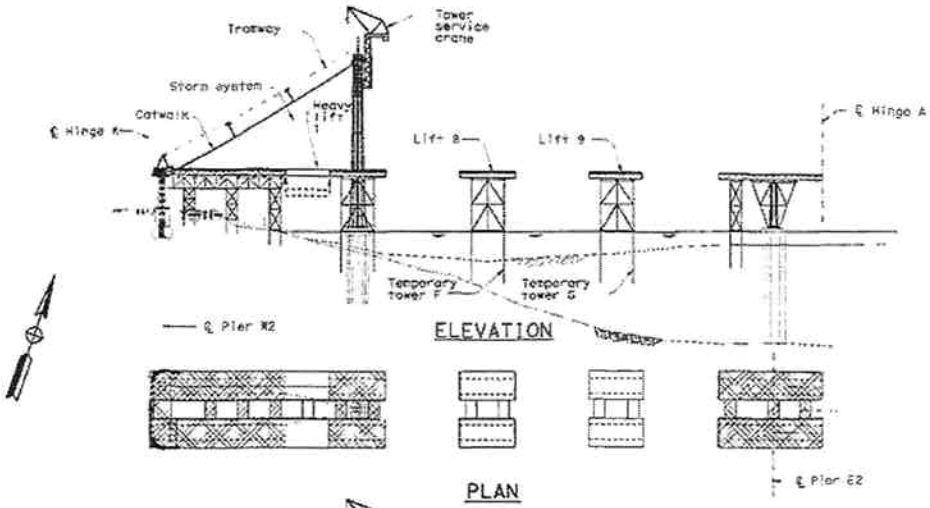
R. Montanarez, PROJECT ENGINEER

MODEL NO. 34-9006L/R
KILOMETER POST 13.2/13.9

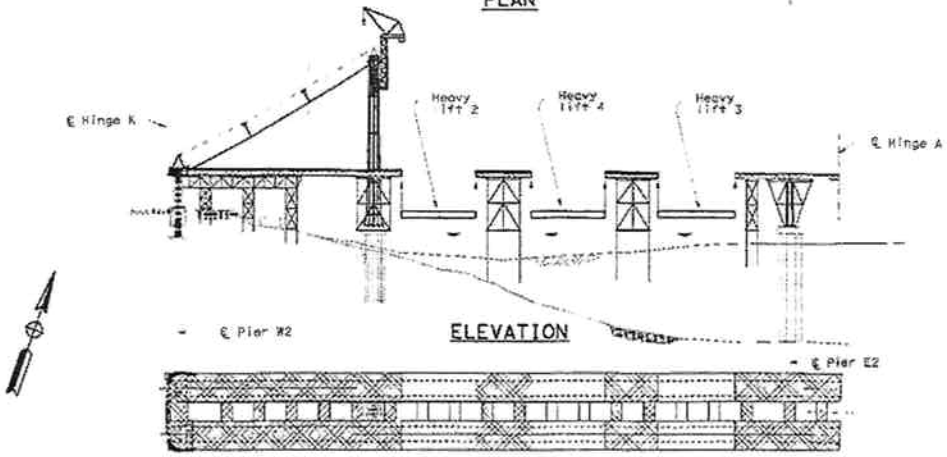
CU 04
EA 0170F1

553

100% PS&E



PLAN



PLAN

Step 5

a. Drive piles for Temporary towers F and G, and erect temporary towers F and G.

b. Erect one box girder section, for each roadway over Temporary towers F and G (lifts 8 and 9).

c. Perform heavy lift 1 for each roadway.

d. Erect cross beams, align and make up splices to erected box girder sections.

e. Jack as required, align and make up splices.

f. Complete tower construction, see "Tower Construction Sequence" sheet.

g. Erect catwalk, storm system and tramway in the back span.

Step 6

a. Perform heavy lift 2 for each roadway align box girder sections and make up splices.

b. For heavy lift 3 and 4, use same procedure.

c. Erect cross beams, align and make up splices to erected deck sections.

NOTES:

1. Erection procedure continued on "Construction Sequence No.5" sheet.
2. Heavy lifts may be performed by a floating crane or any other means.
3. Navigational clearances required by Coast Guard shall be maintained.
4. Tower should be surveyed after step 5f.
5. Fender at Pier E2 (By Others) not shown.

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	871	1204

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12-6-04
PLANS APPROVAL DATE
T.Y. LIN / MOFFATT & NICHOL
625 BATTERY STREET
SAN FRANCISCO, CA 94111

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

R. Vallzadeh/V. Toon/Y.L./N.L./F.C.
DESIGN ENGINEERS
Ryan Vallzadeh / Vincent Toon / Y. Lin
STAMP DATE 04/08/04

DESIGN W. Nomer — CHECKED C. Selin
DETAILS C. Wibelitz — CHECKED J. Lopez-Jara
QUANTITIES C. Wibelitz — CHECKED J. Lopez-Jara

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzocharles
PROJECT ENGINEER

CU 04
EA 012001

BRIDGE NO. 34-0006L/R
ALIGNMENT POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION SEQUENCE NO.5

554

100 % PAGE

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	972	1204

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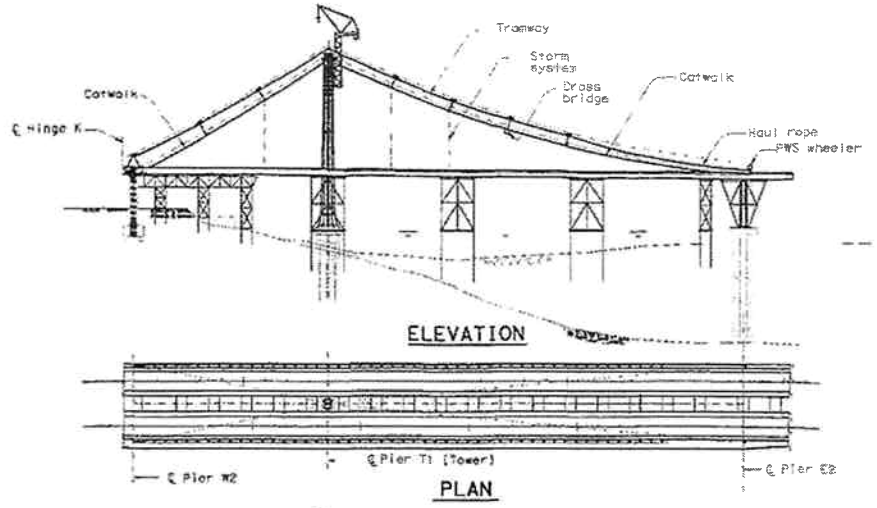
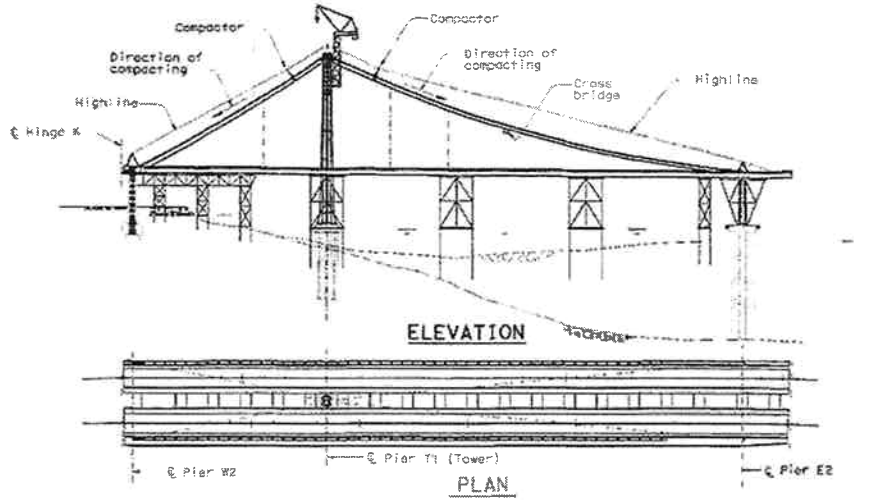
12-6-04

PLANS APPROVAL DATE

T.Y. LIN / MOFFATT & NICHOL
825 BATTERY STREET
SAN FRANCISCO, CA 94111





Step 7

a. Erect catwalk, storm system, tramway PWS rollers and cross bridge on the main span and side span.

b. Install, splice and tension haul rope and mount hauling wheels.

c. Tie down box girder at Pier E2.

d. Jack box girder sections vertically, at temporary bearings, to achieve desired rotation at W2 closure joint. (Estimated jacking forces are in the order of 200 KN per bearing pair).

e. Make W2 closure joint.

f. Remove temporary connection between W2 cap beam and deck sections.

g. Erect bikepath, counter weight and barriers. Final tightening of barrier connection to box girder is made in Step 11c.

Step 8

a. Remove tie down between box girder and Pier E2.

b. Release falsework bearings in longitudinal direction. (See "Temporary Structures" sheets).

c. Tower saddle should either be allowed to slide in the longitudinal direction or the tower has to be tied back before step 5g. See "Construction Load" sheets for expected movements during construction.

d. Start cable erection operations.

e. After cable erection, with one compactor on each cable, in each span, compact cable downhill from tower top .

NOTES:

1. Erection procedure continued on "Construction Sequence No.6" sheet.
2. Deck geometry shall be surveyed and checked after step 7b.
3. Fender at Pier E2 (by others) not shown.

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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION SEQUENCE NO.6

DESIGN: M. Nader — CHECKED: C. Sele
DETAILS: C. Mibelli — CHECKED: J. Lopez-Jara
QUANTITIES: C. Mibelli — CHECKED: J. Lopez-Jara

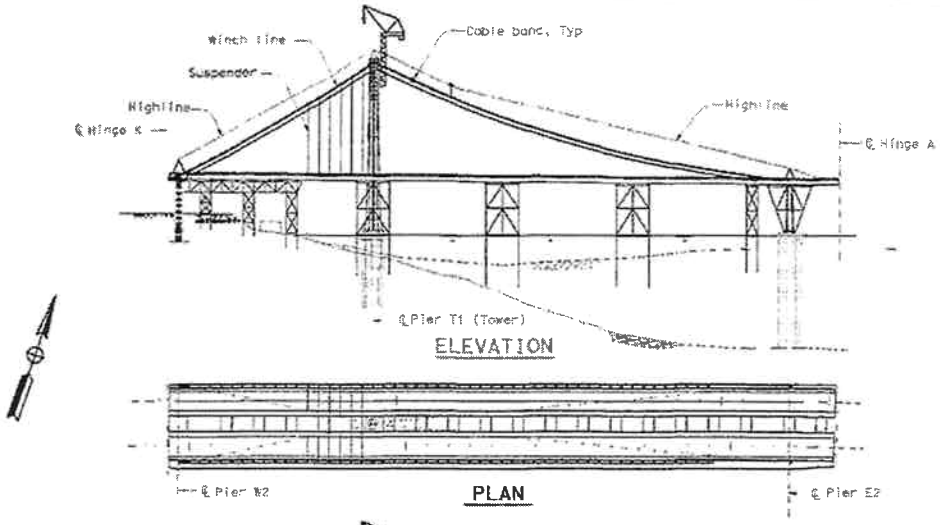
R. Montemayor
PROJECT ENGINEER

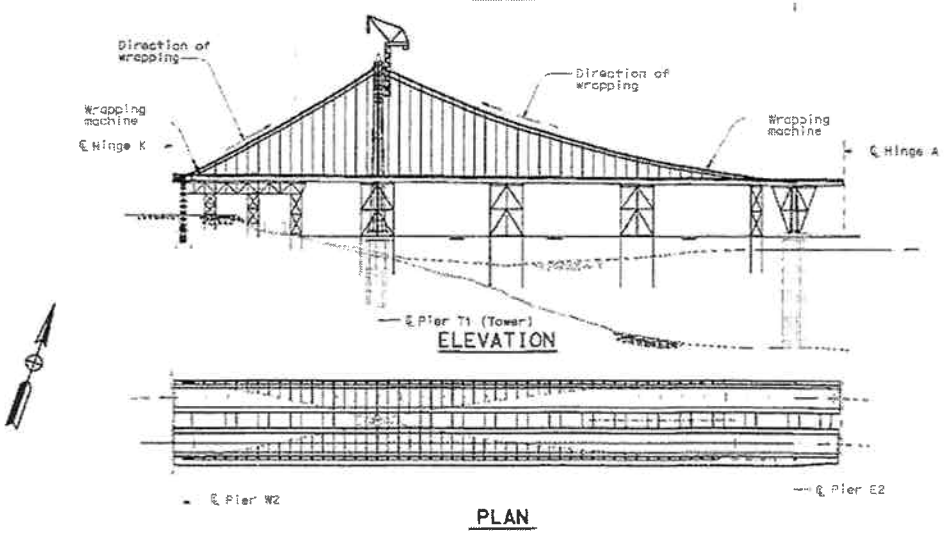
34-0006L/R
13.2/13.9

CU 04
EA 012071

555

100% PGAR





DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	973	1204

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12-6-04
PLANS APPROVAL DATE

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Step 9

a. Secure catwalk to cable.

b. Remove storm system and cross bracing

c. Erect cable bands and install suspenders at each cable and in each span.

d. Erect post and hand rail on top of cable.

Step 10

a. Transfer load to suspenders. This should be done by jacking the suspenders (6-10 at a time and simultaneously) in cycles towards the suspender brackets at the deck.

b. The jacking of the jacking saddle should be coordinated with the suspender load transfer to ensure that there is no slippage of the cable at the deviation saddle.

c. Place wrapping machine on each cable and in each span.

d. Wrap uphill.

e. Remove catwalk and tower service crane.

f. Remove all falsework towers except for temporary bearings and temporary tower E2.

g. Construct fender at tower. For tower fender construction, see "Tower Fender Construction Sequence" sheets.

NOTES:

1. Erection procedure continued on "Construction Sequence No.7" sheet.

2. Deck and cable geometry shall be surveyed and checked during load transfer to cable.

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PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Montezemolo
PROJECT ENGINEER

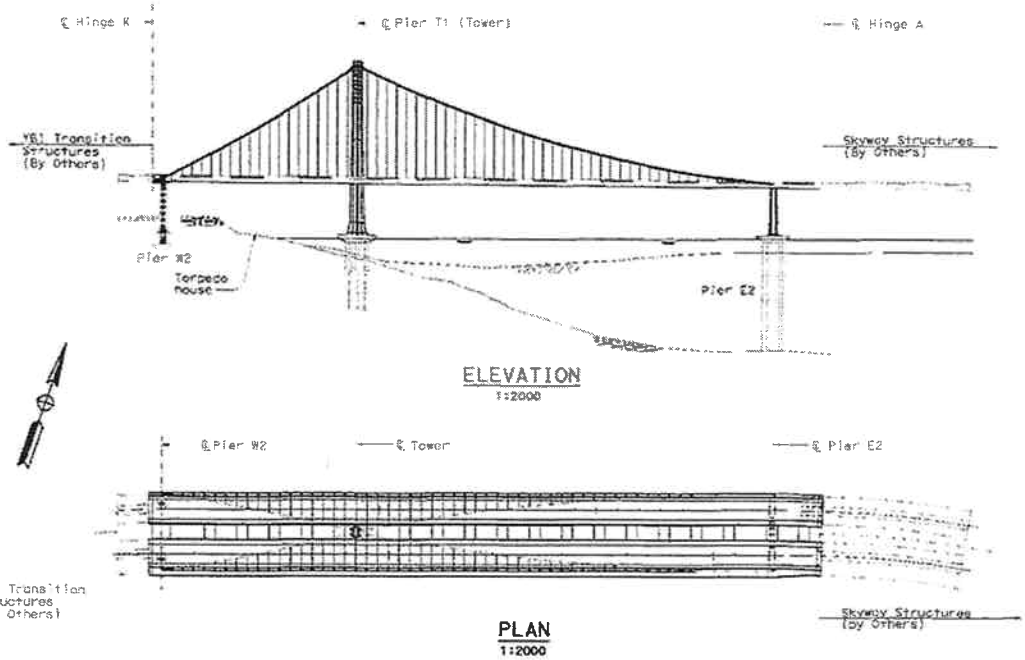
34-0006L/R
13.2/13.9

CU 04
EA 0120F1

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION SEQUENCE NO.7

556



DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	974	1204

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Step 11

a. Erect Hinge A. For Hinge A erection sequence, including the removal of skyway temporary towers AE & AW, see "Hinge A Construction sequence" sheet. Erection of Hinge K Bearing is by Others.

b. Place wearing surface and utilites.

c. Tighten barrier connection to the box girder. Install and tension suspenders at tower.

d. Make permanent connection of bearings and shear keys at Pier E2.

e. Remove temporary bearings and temporary tower E2.

f. Install Hinge A expansion joint. Installation of Hinge K expansion joint by Others.

Notes:

1. Fender at Pier E2 (by others) not shown.

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R. Hernandez
PROJECT ENGINEER

34-0006L/R
13.2/13.9

CU 04
EA 0120F1

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION SEQUENCE NO.8

557

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	975	1204

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12-6-06
PLANS APPROVAL DATE

Norman N. Nichol
No. C 064426
12/31/05
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STATE OF CALIFORNIA

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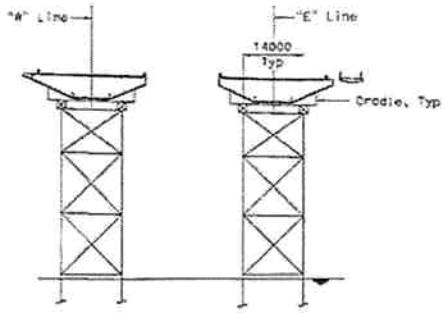
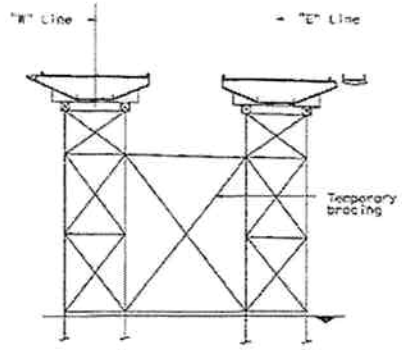
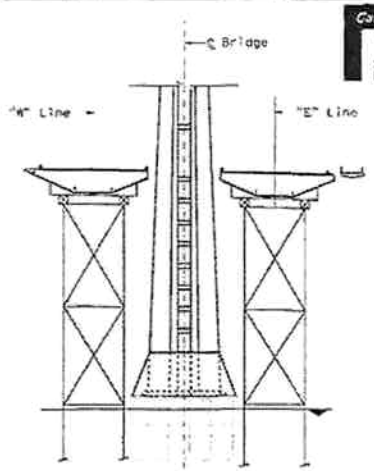
825 BATTERY STREET
SAN FRANCISCO, CA 94111



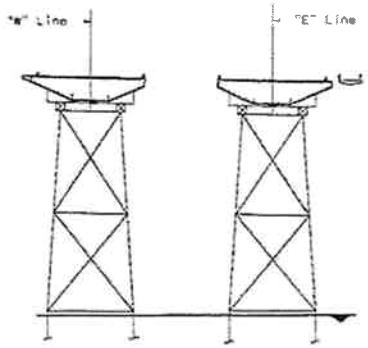
ELEVATION AT TEMPORARY TOWER A
NTS



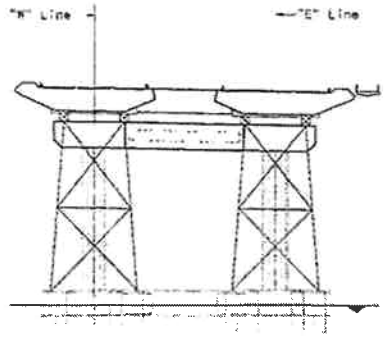
ELEVATION AT TEMPORARY TOWERS B AND C
NTS



ELEVATION AT TEMPORARY TOWER D
NTS



ELEVATION AT
TEMPORARY TOWERS F, G AND H
NTS



ELEVATION AT TEMPORARY TOWER E2
NTS
(See Note 4)

NOTES:

1. The box girder should be supported on a cradle during construction in order to evenly distribute the bearing reactions.
2. The clearance between the bottom of the cradle and top of falsework should be sufficient to account for camber and deflection during construction.
3. The box girders shall be secured to the falsework in the longitudinal, transverse and vertical directions. The box girders shall be released in the longitudinal direction before the cable erection starts.
4. The cradle at PP119 (℄ Pier E2) shall be designed to permit the installation of the permanent Pier E2 bearing and shear key.
5. Fender at Pier E2 (by others) not shown.

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N. Valtzoden/V. Toor/Y.L./N.L./F.C.
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Y. Lee
STRUCTURE

DESIGN M. Noor — CHECKED C. Seih
DETAILS C. Mitchell — CHECKED J. Lopez-Jong
QUANTITIES C. Mitchell — CHECKED J. Lopez-Jong

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

R. Montemayor
PROJECT ENGINEER

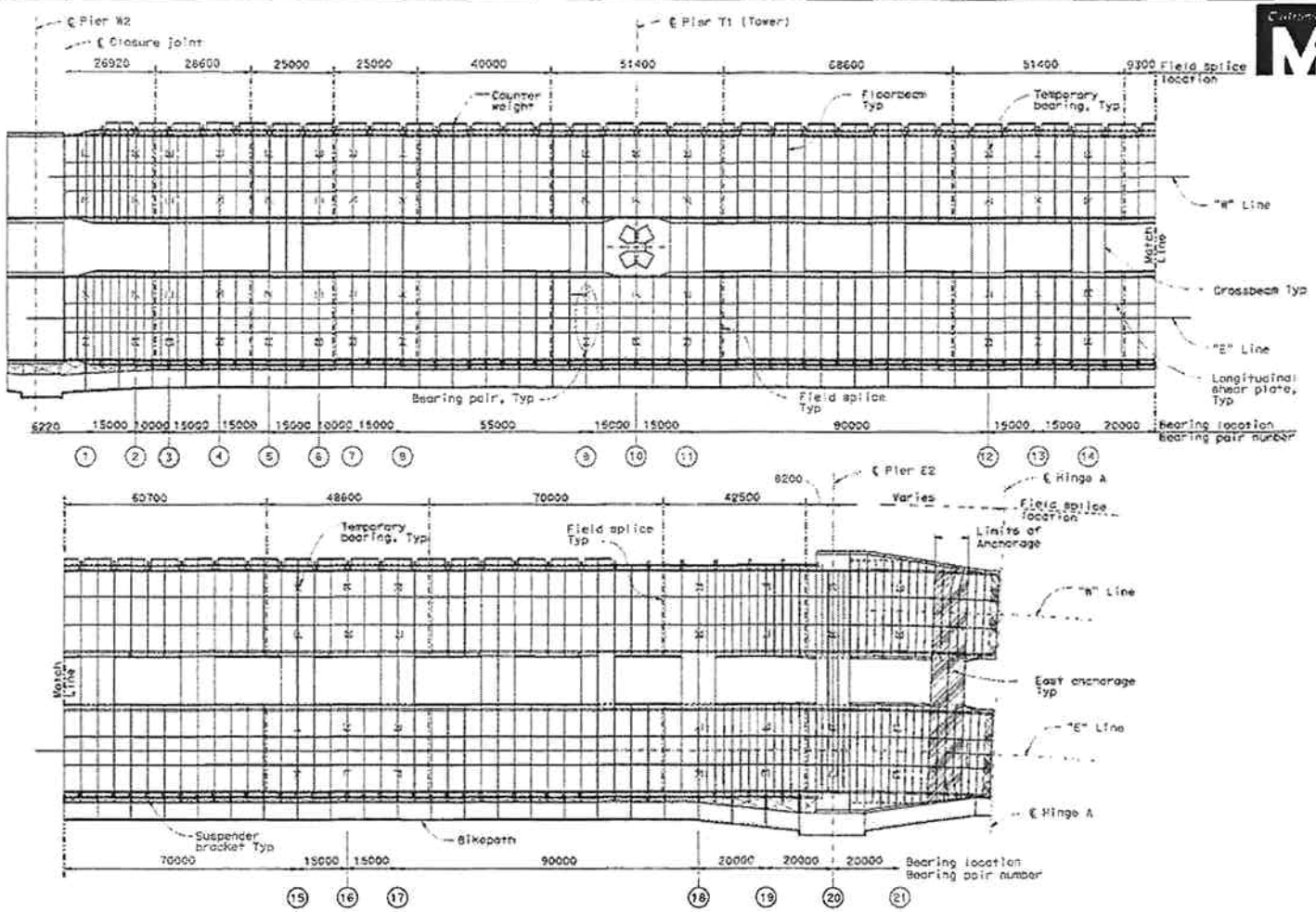
CU 04
EA 0120F1

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

TEMPORARY STRUCTURES NO.1

SHEET 558



FIELD SPLICE AND TEMPORARY TOWER BEARING LOCATION

NTS

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DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	976	1204

REGISTERED ENGINEER - CIVIL

12-6-04

PLANS APPROVAL DATE

T.Y. LIN / MOFFATT & NICHOL
625 BATTERY STREET
SAN FRANCISCO, CA 94111

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DETAILS: C. Mibelli; J. Lopez-Jara
QUANTITIES: C. Mibelli; J. Lopez-Jara

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

CU 04
EA 012041

BRIDGE NO. 34-0006L/R
ALIGNMENT POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

TEMPORARY STRUCTURES NO.2

559

Table 1. Envelope of Vertical Forces at Temporary Towers and Temporary Bearings During Construction

Temporary Tower	Supported load per tesporary tower, per box kN (See Note 1)	Tesporary bearing pair number	Compression load per bearing pair, per box girder kN (See Note 1)	Tension load per bearing pair, per box girder kN (See Notes 1 & 2)
Temporary Tower A	17100	1	14900	-
		2	600	-
		3	4800	-
Temporary Tower B	11950	4	7700	-
		5	4700	-
		6	2550	-
Temporary Tower C	12650	7	1600	-
		8	12300	-
Temporary Tower D	24500	9	8200	-
		10	2700	-
		11	15850	-
Temporary Tower F	28400	12	13250	-
		13	3050	-
		14	13500	-
Temporary Tower G	28550	15	13500	-
		16	2850	-
		17	13150	-
Temporary Tower H	16800	18	16800	-
Temporary Tower E2	32850	19	9800	-
		20	26250	8300
		21	31050	-

Note: (a) No load factors are applied to these loads

(b) The above loads do not include wind or seismic effects

Table 2. Lift Weights

LIFT NO.	Estimated Weight, metric tons
1	830
2	546
3	459
4	459
5	912
6	2117
7	943
8	943
9	892
HL 1	734
HL 2	1259
HL 3	1285
HL 4	1285

(see Note 4)

HL=Heavy Lift

NOTES:

1. The loads shown do not account for construction loads and equipment, impact, eccentricities or accidental torsion. These values are for information only. The Contractor shall verify these numbers and submit for Engineer Approval.
2. The tension values shown were established assuming that all temporary bearings carry no tension (except for bearing pair No. 20). The Contractor shall carry out the proper analysis which account for proper boundary conditions based of this erection plan.
3. The temporary towers shall be designed to resist the following lateral loads:
   - Seismic Loads: Based on site-specific temporary tower response spectra (5% damping)
   - Wind Loads: ANSI/ASCE 7-95 Standard.
4. The Contactor may reduce the weight of this lift by having an additional splice. However an additional temporary tower may be required.

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	977	1204

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12-6-04
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525 BATTERY STREET
SAN FRANCISCO, CA 94111

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION LOAD NO.1

CU 04
EA 0120F1

560

12

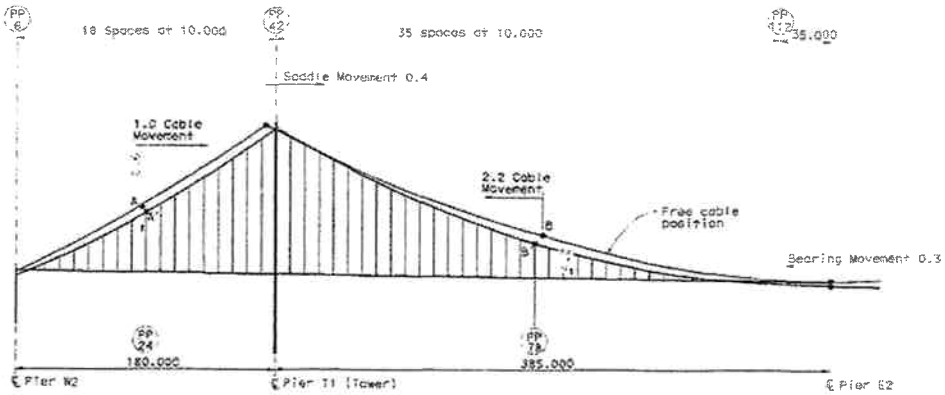
DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	978	1204

REGISTERED ENGINEER - CIVIL

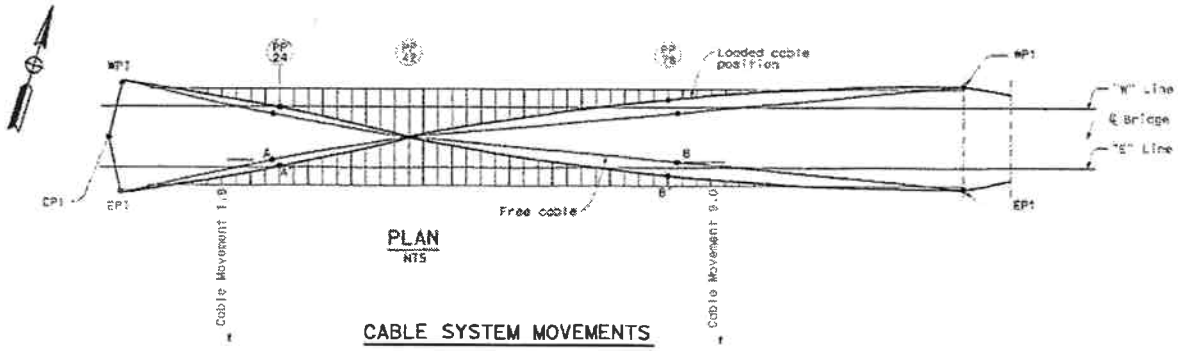
12-6-06

PLANS APPROVAL DATE

T.Y. LIN / MOFFATT & NICHOL
525 BATTERY STREET
SAN FRANCISCO, CA 940



ELEVATION FOR EB (WB similar)
NTS



PLAN
NTS

CABLE SYSTEM MOVEMENTS

NOTES:

1. These movements are for information only, and shall be recalculated by the Contractor and approved by the engineer.
2. "A" and "B" denote free-cable location and "A'" and "B'" denote the corresponding loaded location.
3. The cable movements shown do not consider the interference between the cable and the box girder in the vicinity of the East Saddles, or the interference between the cable and the Tower Saddle troughs.
4. The contractor shall take the necessary measures to prevent contact between the cable and the box girder. The contractor shall also insure that the cable angle brake does not exceed 6 degrees, during the operation of tensioning the suspenders to transfer load to the cable.

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

CU 04
EA 0170F1

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CONSTRUCTION LOAD NO.2

561

Appendix B

Special Provisions Section 10-1.41 Temporary Towers

NOT FOR CONSTUCTION

10-1.41 TEMPORARY TOWERS

Temporary towers used for the construction of the bridge superstructure shall conform to the information shown on the plans and these special provisions.

The temporary towers shown on the approved working drawings shall be designed and constructed by the Contractor in conformance with the requirements in Section 49, "Piling," and Section 51-1.06, "Falsework," of the Standard Specifications and these special provisions.

Temporary towers shall include jacking assemblies and appurtenant items necessary to jack and support the superstructure, and compensate and maintain the proper roadway alignment and profile.

GENERAL

Attention is directed to the following sections of these special provisions regarding permit restrictions and regulations that may impact temporary tower design and construction:

1. Relations with U. S. Coast Guard
2. Relations with Regional Water Quality Control Board
3. Relations with United States Fish and Wildlife Service
4. Relations with California Department of Fish and Game
5. Relations with National Marine Fisheries Service
6. Maintaining Traffic
7. Relations with Bay Conservation Development Commission
8. Relations with Army Corps of Engineers

Temporary tower foundation information and piling design procedures (including example calculations) are included in the "Information Handout" available to the Contractor as provided for in "Project Information" of these special provisions and Section 2-1.03, "Examination of Plans, Specifications, Contract, and Site of Work," of the Standard Specifications.

Attention is directed to "Marine Pile Driving Energy Attenuator," of these special provisions."

SCHEDULE OF VALUES

Attention is directed to "Accelerated Working Drawings Submittal" in these special provisions.

The Contractor shall submit, for approval by the Engineer, a schedule of values detailing the cost breakdown of the contract lump sum item for furnish and remove temporary towers. The schedule of values shall reflect the items, work, quantities and costs required to furnish, erect, and remove temporary towers, except for costs required to prepare working drawings as included in the contract lump sum price paid for "Accelerated Working Drawings Submittal". The Contractor shall be responsible for the accuracy of the quantities and cost used in the schedule of values.

The sum of the costs for the items of work listed in the schedule of values shall equal the contract lump sum price for furnish and remove temporary towers.

No adjustment in compensation will be made in the contract lump sum price paid for furnish and remove temporary towers due to differences between the quantities shown in the schedule of values furnished by the Contractor and the quantities required to complete the work as shown on the plans and specified in these special provisions.

The schedule of values shall be submitted to the Engineer within the time required for submittal of the Interim Baseline Schedule, as specified in "Progress Schedule (Critical Path)," of these special provisions. When approved in writing by the Engineer, the schedule of values will be used to determine progress payments for furnish and remove temporary towers during the progress of the work. No progress payment for furnish and remove temporary towers will be made until the schedule of values is approved in writing by the Engineer.

The schedule of values shall categorize the work by tower location and shall quantify the values into the following progress payment milestones:

A. Furnish temporary tower material
B. Erect temporary tower
C. Remove temporary tower

The Contractor may propose additional milestones for progress payment subject to approval by the Engineer.

WORKING DRAWINGS

The Contractor shall submit to the Engineer working drawings and design calculations for temporary towers, in conformance with the requirements in "Working Drawings," of these special provisions. The Contractor shall allow the following review times, after notification is received from the Engineer, for the review of temporary tower working drawings and design calculations:

Location	Review Time
AE and AW	50 Days
BE and BW	50 Days
CE and CW	50 Days
DE and DW	50 Days
EE and EW	50 Days
FE and FW	50 Days
GE and GW	50 Days
HE and HW	50 Days

When several temporary tower drawings or calculations are submitted simultaneously, or additional drawings or calculations are submitted for review before the review of previously submitted drawing or calculation has been completed, the Contractor shall designate the preferred sequence in which the plans and calculations are to be reviewed. The time to be provided for the review of any drawings and calculations in the sequence shall be not less than the review time specified above for that plan, plus 10 days for each plan of higher priority which is still under review.

Working drawings for temporary towers shall include, the following:

A. Locations of the temporary towers needed to support the deck
B. Complete details showing the proposed construction sequence, including the deck and foundations.
C. Concrete placing diagram.
D. Method and plans for release of tower to superstructure connections.
E. Erection methods and removal plans.
F. Descriptions of equipment and associated loads, including the loads shown on the plans.
G. Complete details for providing grade adjustment to compensate for anticipated settlement and proper fit up.
H. Complete details and descriptions of the displacement monitoring system described in these special provisions.
I. Details and descriptions of equipment and methods proposed for monitoring the erection, position, settlement and deflection of temporary towers.
J. Schedule of taking displacement measurements.
K. Pile Data Table including the design loads, bearing value, and required pile tip elevations.
L. Details for pile installation techniques, spread footings, tie-downs, driven piles and CIDH piles, as appropriate.
M. Details for alternative installation techniques for installing driven piles, in case pile refusal is encountered more than 3 meters above the design penetration. Techniques shall include the effects on bearing value and any design modifications.

Design calculations for temporary towers shall include, at a minimum, the following:

A. Summary of computed stresses in the temporary towers for all design loading conditions. Computed stresses shall include effects from any grade adjustment assumed by the Contractor to ensure proper grade and fit up.
B. Assessment of lateral stiffness and significant periods of vibration of the temporary towers system.
C. Calculations for providing grade adjustment to compensate for anticipated settlement and ensure proper fit up.
D. Calculation of pile setup period(s).
E. Calculation of pile alignment and location tolerances.
F. Driven pile refusal criteria.
G. Driving system submittal.
H. Dynamic monitoring equipment and setup.
I. CIDH drilling system submittal.
J. Pile handling submittal.
K. Design pile length.
L. Schedule.

SLOPE RESTORATION

Attention is directed to the existing steep slope at Yerba Buena Island between Bent W2 and Building No. 262 (Torpedo Building). The slope, and any other areas affected by the construction and removal of temporary towers, shall be restored to the original (existing) grade by the Contractor and approved by the Engineer.

The Contractor shall submit to the Engineer slope restoration working drawings that include existing contour grading, restoration procedures, erosion control treatment, materials, and design calculations for any earth reinforcement and support structure for the restoration of the slope. Areas excavated shall be returned to pre-excavation conditions.

Any work that will change the existing contour grade of the slope will not be allowed before slope restoration working drawings are approved.

TEMPORARY TOWER FOUNDATIONS

The types of foundations used for the temporary towers will vary along the length of the bridge. Spread footings, footing tie-downs, driven piling and cast-in-drilled-hole (CIDH) piling may be used, depending on the surficial materials and the depth to bedrock. The recommendations presented in the Information Handout for the allowable bearing pressures, tie-down bond strength, CIDH bearing capacity and driven piling capacity shall be adopted in the design of these types of foundations. Blasting, including the use of explosives for excavation of foundations, will not be permitted.

SPREAD FOOTINGS AND TIE-DOWNS

Spread footings and tie-downs shall be designed and constructed in accordance with the design recommendations in the Information Handout, the Standard Specifications, and these special provisions.

Spread footings shall be constructed in a dry or dewatered condition.

Equipment or methods for drilling tie-down sockets in rock shall not result in a smooth hole. The Contractor shall provide to the Engineer for approval information on the roughness that is expected for the sockets.

DRIVEN PILING

Driven piling used to support temporary towers located in the bay shall be steel pipe piling. The pile sections shall be compact, capable of plastic deformations without local buckling.

The requirements in Section 49-1.03, "Determination of Length," of the Standard Specifications shall not apply.

Driven piling shall be of such length as required to develop the minimum bearing value, as defined in the Information Handout and to obtain the design penetration as shown on the approved working drawings. Attention is directed to the use of Pile Dynamic Monitoring of piles which are designed to be tipped into rock to obtain end bearing capacity.

Driving Equipment

Diesel and steam hammers shall not be used to install driven piling.

Jetting and drilling in conformance with Section 49-1.05, "Driving Equipment," of the Standard Specifications shall not be used.

Pile Dynamic Monitoring

Where required to obtain verification of rock contact for end bearing, as shown on the approved working drawings, or as otherwise required by the Engineer, the Contractor shall conduct dynamic monitoring of pile driving and conduct penetration and bearing analyses based on a wave equation analysis. The analysis shall be signed by an engineer who is registered as a Civil Engineer in the State of California and submitted to the Engineer prior to completion of temporary tower erection.

The Contractor's monitoring equipment shall be capable of generating a continuous computer printout of monitoring results.

Each pile to be dynamically monitored shall be fitted with two sets of attachments. These attachments shall be located on opposite sides of the pile.

Bearing Criteria

The first and second paragraphs in Section 49-1.08 "Bearing Value and Penetration," of the Standard Specifications shall not apply. Pile tip elevation and bearing capacity shall be determined as shown in the Information Handout using capacity curves where appropriate. Where the locations of the temporary towers are outside the limits shown in the Information Handout, the Contractor shall determine the bearing capacity of the piles at that location, using the same method specified in the Information Handout and this capacity shall be submitted to the Engineer for approval.

Piles shall be driven to the design tip elevation and a minimum bearing value of not less than the design loading shown on the approved working drawings, unless otherwise specified in these special provisions or permitted in writing by the Engineer. Pile bearing values shall be determined using skin friction only, except where the pile is tipped in rock.

For piles that encounter driving refusal above the approved design tip elevation, the Contractor shall notify the Engineer in writing. The Contractor shall evaluate these piles and propose alternative pile installation techniques. No additional compensation will be allowed for developing and using alternative pile installation methods.

The Contractor shall provide a pile driving log at the completion of driving each pile and with the notification of driving refusal.

CIDH PILING

Drilling Equipment

The fifth paragraph in Section 49-4.03 "Drilled Holes" of the Standard Specifications shall not apply.

Drilling equipment shall be selected and operated to ensure that the surfaces of sockets drilled in rock are not smooth.

The Contractor shall provide to the Engineer for approval information on the roughness that is expected for the sockets.

Bearing Criteria

Pile tip elevation and bearing capacity shall be determined as shown in the Information Handout. End bearing shall not be used in the determination of bearing capacity. The bearing capacity calculations shall be submitted to the Engineer for approval.

TEMPORARY TOWER DESIGN

Temporary towers shall consist of ductile steel braced frames with welded or bolted connections used for field erection splices. Welded connections performed in the field or in the Contractor's fabrication facilities shall be designed in accordance with AISC or API RP2A for hot rolled sections and steel tubular sections, respectively. The Contractor shall provide 2 copies of the stated codes to the Engineer. Timber walkways and decks will be permitted.

Cable bracing and tie-rod bracing will not be permitted.

Timber connections shall be designed in conformance with the procedures, stresses and loads permitted in the Falsework Manual as published by the California Department of Transportation, Division of Structures, Division of Structure Construction.

The construction equipment loads shall be the actual weight of the construction equipment, material and personnel, but in no case shall be less than 960 N/m2 of deck surface area and 1100 N/m along the deck edges.

The design of temporary towers shall conform to both the design service load and ultimate limit state criteria set in these special provisions. The service load criteria will be met by the load combinations stated in "Design Load Combinations for Load Factor Design" and the ultimate limit state criteria will be met by the pushover analyses in "Seismic Design Loads".

The following codes shall be used to detail and to establish temporary tower capacities:

Subject	Design Code or Reference	Year
Structural Steel – All Members and Connections except Tubular Members**	AISC-LRFD*	1999
Structural Steel – Tubular Members and Connections**	API RP2A-LRFD	July 1993
Concrete	AASHTO-LRFD Bridge Construction Specifications	2nd Edition
Falsework	State of California Dept. of Transportation – Falsework Manual	Revision 32, November 2001

* With reference to AISC Seismic Provisions for Structural Steel Buildings, 1997.

** All connections shall be designed to be stronger than the connected members. For connections between rolled sections and tubular members the most stringent code of AISC-LRFD and API RP2A-LRFD shall govern.

A single code, specification or recommended practice shall be applied consistently for a specific structure type or load.

Design of temporary towers shall account for any change in the loads imposed on the tower by the bridge superstructure due to the construction sequence of the bridge.

Temporary towers shall be designed to adequately support the superstructure of the bridge without exceeding a demand-to-capacity ratio in the box girder of 1.0 for all design load combinations.

The temporary tower design calculations shall demonstrate that the total design settlement of temporary towers does not exceed 25 mm at the mudline for the governing design load combination.

The twentieth paragraph of Section 51-1.06A "Falsework Design and Drawings," of the Standard Specifications shall not apply.

The design of temporary towers shall be based on assumed loads that are equal to or greater than those described in this section.

The Contractor shall be responsible for the proper evaluation of the falsework materials and design of the falsework to safely carry the actual loads imposed.

The fifth and sixth paragraphs of Section 51-1.06A(1), "Design Loads," of the Standard Specifications shall not apply.

Vertical Loads

Temporary towers and their foundations shall be designed to carry the anticipated total effective tower load. The total effective tower load shall be determined including, at a minimum, the effects of the following:

A. Tower and_foundation, including fenders;
B. Load from supported bridge; and
C. Contractor's equipment and live load.

The vertical loads from the supported bridge shall be calculated by the Contractor for the selected construction method. Vertical loads shown on the plans are for information only.

Seismic Design Loads

Temporary towers shall be designed to be ductile and to resist horizontal seismic loads as defined below in combination with the appropriate vertical loads covering controlling construction stages. The seismic analyses shall consider the interactions of the temporary towers with the bridge superstructure at all appropriate stages. Seismic design loads need not be considered during lifting operations.

The temporary towers shall remain serviceable and capable of carrying the design loads.

Analysis and design calculations shall correctly incorporate all contributing mass, stiffness, loading and energy dissipation characteristics of the temporary towers including geotechnical and structural components and hydrodynamic added mass. P-delta effects shall be considered in the analyses. Assumed scour depth shall be 0.5 m plus one pile diameter.

Modal spectral analysis with sufficient number of modes to capture at least 90% of the mass of the structure shall be used to establish the peak seismic displacements. To establish the ultimate limit state, a longitudinal and transverse static push-over analyses shall be used to verify the stability of the temporary tower and its ductility. Tower vertical load carrying members and connections shall remain elastic. Structural steel of the bracing members may yield. Extreme fibers of the structural steel piles may yield to a maximum of 2% strain. The_temporary tower displacement capacity shall exceed the demands corresponding to 1.50 times the peak seismic displacements. Pushover analyses of the temporary towers shall be carried out based upon displaced shapes defined, at a minimum, as the primary transverse and longitudinal mode shapes. The mode shapes shall be determined from modal analyses of the temporary towers and the supported bridge superstructure.

Seismic demands shall be determined for two independent horizontal loading conditions in perpendicular directions. The directions are defined to be in the longitudinal axis of the bridge and the transverse axis of the bridge. In order to account for directional uncertainty of earthquake motions, the demands resulting from analyses of the two perpendicular seismic loading directions shall be combined into two load cases as follows:

EQ load case 1 1.0 longitudinal and 0.3 transverse
EQ load case 2 0.3 longitudinal and 1.0 transverse

Acceleration levels applied to the temporary towers shall be generated using the Acceleration Response Spectrum shown on plan sheet "Construction Sequence 2." The response spectrum can be used for all locations along the length of the self-anchored suspension bridge as input motion. These lateral loadings supersede the 0.02 g lateral load requirement in the California Department of Transportation Falsework Manual.

These Seismic Design Loads are the minimum required during construction. The Contractor may elect to design and construct the temporary towers for a greater level of loading. No additional compensation will be allowed nor extension of time will be granted due to the Contractor's use of loading that exceeds the required minimum.

Within 90 days of contract award, the Contractor shall hold a meeting between the Engineer, the Contractor and the Contractor's designer of the temporary towers. The Contractor shall present to the Engineer preliminary temporary tower design and details and methods of analyses he proposes to use.

Vessel Impact Design Loads

Temporary towers shall be designed for accidental vessel impact and coincident wind, and current loads calculated in accordance with API RP2A. Tide and current information are included in Section 3.4 of the Ship Collision Report contained in the Information Handout.

Attention is directed to the AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges (1991) regarding design for accidental impact.

Temporary towers shall be designed to resist loads from impact of the Contractor's equipment. At a minimum, temporary towers shall be designed to resist an accidental impact load of 7.6 MN from any direction caused by a barge, acting between elevation +7.6 m and –1.8 m NGVD.

The rake of the bow of a 76 m x 14 m hopper barge shall be used to determine whether the impact force will occur on the fenders or whether the vessel will impact the tower substructure or foundation.

Temporary towers shall not be used for mooring the Contractor's vessels, unless they are specifically designed for mooring loads, including the effects of the eccentric application of these loads.

In lieu of designing the temporary towers for a static accidental impact load of 7.6 MN, the Contractor may design and supply an impact energy absorption system in order to prevent damage to the temporary tower structures at no additional cost to the State. The minimum impact force associated with an empty hopper barge, having a displacement of 800 Tonnes, drifting into the temporary tower from any direction at a design current of 2.0 knots (1.029m/sec), shall be used for design. The barge mass as well as the hydrodynamic added mass shall be considered acting between elevation +7.6 m and –1.8 m NGVD. The rake of the bow of a 76 m x 14 m hopper barge shall be used to determine whether the impact force will occur on the fenders or whether the vessel will impact the tower substructure or foundation.

Wind Loads

Temporary towers shall be designed to resist horizontal wind loads as determined from the wind criteria set forth in Section 10-1.59, "Steel Structures," subsection "ASSEMBLY," of these special provisions and the ANSI/ASCE 7-95 code.

The temporary towers shall have an Importance Factor of 1.15 per ANSI/ASCE 7-95, for wind load purposes only.

Design Load Combinations for Load Factor Design

Temporary towers shall be designed using the following load combinations:

1.4 DL
1.1 DL + 1.3 LL
1.0 DL + 1.0 LL + 0.5 Wind + 1.0 Current + 1.0 Vessel Impact
1.0 DL + 1.0 LL + 1.0 EQ
1.0 DL + 1.0 LL + 1.3 Wind +1.3 (Wave & Current)

Temporary Tower Foundations - Driving System Submittal

The Contractor's temporary tower working drawings shall include a driven pile installation schedule that is in conformance with "Progress Schedule (Critical Path Method)," of these special provisions.

Prior to installing driven piling at a given temporary tower location, the Contractor shall provide a driving system submittal for that tower location, including a driveability analysis, in conformance with the provisions in "Working Drawings," of these special provisions. Technical data for all proposed driving systems (i.e., each hammer that may be brought onto the site) shall be included in the submittal.

The driving system submittal shall be based on the soil profiles shown in the Information Handout and shall contain an analysis showing that the proposed driving systems will install piling to the Contractor's design tip elevation, as shown on the approved working drawings without overstressing the piles. Submittals shall include the following:

A. Complete description of soil parameters used, including soil quake and damping coefficients, skin friction distribution, percentage shaft friction, and total soil resistance to driving.
B. List of all hammer operation parameters assumed in the analysis, including manufacturer's rated energy, fuel settings, stroke limitations, and hammer efficiency.
C. Driveability studies that are based on a wave equation analysis using a computer program that has been approved by the Engineer. Driveability studies shall model the Contractor's proposed driving systems, including the hammers, capblocks, pile cushions, followers and driving shoes, as well as determine driving resistance and pile stresses for assumed site conditions. For open-ended steel pipe piles, soil resistance to driving shall be computed for both plugged and unplugged cases. The range of soil resistance to driving and the percentage shaft resistance shall be determined for site conditions ranging from 5 meters above to 5 meters below the Contractor's design tip elevation shown on the approved working drawings. Separate analyses shall be completed at elevations above the design tip elevations where difficult driving or pile splices are anticipated. Driveability analysis results shall include plots of the following:

    1. Maximum pile head and pile toe compressive stress versus blows per 250 mm.
    2. Soil resistance to driving versus blows per 250 mm.

D. Copies of all test results from any previous pile load tests, dynamic monitoring, and all driving records used in the analyses.
E. Completed "Pile and Driving Data Form," which is shown in these special provisions.
F. Estimate of pile penetration due to self-weight and the weight of the hammer.

The Contractor shall allow the Engineer 15 days to review a driving system submittal.

The Contractor shall use the driving system and installation methods described in the approved driving system submittal for each temporary tower location. Any change in hammers from those submitted and approved by the Engineer shall also meet the requirements for driving system submittals. Revised and new driving system submittals shall be approved by the Engineer prior to using corresponding driving systems on temporary tower piling. The Contractor shall allow the Engineer 15 days to review each revised and each new driving system submittal after a complete set has been received, as determined by the Engineer.

Approval of pile driving equipment shall not relieve the Contractor of his responsibility to drive piling free of damage to the design penetration.

Temporary Tower Foundations – CIDH Drilling System Submittal

The Contractor's temporary tower working drawings shall include a CIDH pile installation schedule that is in conformance with "Progress Schedule (Critical Path Method)," of these special provisions.

Prior to installing CIDH piling at a given temporary tower location, the Contractor shall provide a drilling system submittal for that tower location, including drilling rig details, drilling fluids, drilling casing, fluid handling system and cuttings disposal system. Technical data for the proposed drilling systems shall be included in the submittal.

MANUFACTURED ASSEMBLIES

Manufactured assemblies shall conform to the provisions in Section 51-1.06A(2) "Design Stresses, Loadings, and Deflections," of the Standard Specifications and these special provisions.

All jacks shall be equipped with a load cell for determining the jacking force. Pressure gages shall have an accurately reading dial at least 150 mm in diameter. Each jack shall be calibrated by a private laboratory approved by the Transportation Laboratory within 6 months prior to use and after each repair, unless otherwise directed. Each jack and its gage shall be calibrated as a unit with the cylinder extension in the approximate position that it will be at final jacking force and shall be accompanied by a certified calibration chart. Load cells shall be calibrated and provided with an indicator by which the jacking force is determined.

Jacks for temporary towers shall be load-rated for at least 125% of the design service loads.

CONSTRUCTION

The construction of temporary towers at each location shall not begin until the Engineer has reviewed and approved the drawings for that location.

The Contractor shall be responsible for monitoring the erection, position, settlement and deflection of temporary towers in accordance with the requirements of this section and submitting logs of these deflections and settlements to the Engineer. Deflection logs shall be provided to the Engineer within 12 hours of recording the measurements.

The third paragraph of Section 51-1.06B, "Falsework Construction," of the Standard Specifications shall not apply.

Welding, welder qualification, and inspection of welding for all steel members shall conform to the requirements of AWS D1.1, except that all CJP welds that sustain tension shall be tested by UT or RT.

Prior to erecting bridge members on the temporary towers, an engineer for the Contractor who is registered as a Civil Engineer in the State of California shall inspect the temporary towers, including grade adjustment and displacement monitoring systems, for conformity with the working drawings. The Contractor's registered engineer shall certify in writing that the temporary towers, including grade adjustment and displacement-monitoring systems, substantially conform to the working drawings, and that the material and workmanship are satisfactory for the purpose intended. A copy of this certification shall be submitted to the Engineer and shall be available at the site of the work at all times.

Grade adjustment operations shall be carefully controlled and monitored to prevent imparting distortion and excessive stresses that would damage the structure.

Adequate means shall be employed to prevent unplanned lateral and longitudinal movement of the temporary tower during jacking. The temporary towers, jacks, and the superstructure shall be stable during all phases of the operation. The jacking system shall be designed such that pressure loss of any hydraulic system cannot cause movement after jacking operations. This may be accomplished by means of a mechanical lock off of the jacks, replacement of the jacks by supports that can be cast in, or alternative methods. The Contractor's jacking methods shall include provisions for blocking up the superstructure from the temporary towers such that the gap between the temporary tower and the superstructure during all phases of the jacking operation does not exceed 25 mm.

Temporary towers shall be protected from damage during construction.

Fenders and navigation lighting for temporary towers shall conform to the requirements in "Relations with the United States Coast Guard" of these special provisions.

REMOVING TEMPORARY TOWERS

When no longer required, temporary towers shall be completely removed, unless otherwise directed by the Engineer. Temporary tower piling shall be removed at least 1.0 meter below the original mudline in-bay, and at least 1.0 meter below the original ground in-land.

Working drawings for the existing Skyway temporary towers (AE) and (AW) are included in the Information Handout, available to the Contractor as provided for in "Project Information" of these special provisions and Section 2-1.03, "Examination of Plans, Specifications, Contract, and Site of Work," of the Standard Specifications.

Section 51-1.06C, "Removing Falsework," of the Standard Specifications shall not apply.

MEASUREMENT AND PAYMENT

The contract lump sum price paid for furnish and remove temporary towers, shall include full compensation for furnishing all labor, materials, tools, equipment and incidentals, and for doing all the work involved in furnishing and removing temporary towers, including designing, constructing and maintaining temporary towers, furnishing and installing temporary tower foundations, temporary tower foundation installation submittals, monitoring and redriving piles, necessary grade adjustment and displacement monitoring, as shown on the plans, and all work involved with slope restoration, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer. Section 49-6, "Measurement and Payment," of the Standard Specifications shall not apply.

Costs for preparing working drawings in excess of amounts allocated in "Accelerated Working Drawings Submittal" item shall be considered as included in contract prices paid for the various items of work and no additional compensation will be allowed therefor.

The contract lump sum price paid for remove Skyway temporary tower, of the types listed in the Engineer's Estimate, shall include full compensation for furnishing all labor, materials, tools, equipment and incidentals, and for doing all the work involved in removing Skyway temporary towers, as shown on the plans, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer.

MARINE PILE DRIVING ENERGY ATTENUATOR

This work shall consist of designing, furnishing, installing, operating, monitoring, maintaining, and removing an air bubble curtain system to attenuate underwater energy generated by driving piles that are 2.5 meters or greater in diameter for the temporary towers. For purposes of this specification, pile installation refers to all the activities involved with driving a single pile; pile driving refers to the time when the hammer is physically driving the pile.

Attention is directed to "Relations with United States Coast Guard," of these special provisions regarding navigation requirements.

Attention is directed to the following sections of these special provisions regarding permit restrictions and regulations that may impact attenuator system design, operation, and removal:

A. Relations with California Department of Fish and Game
B. Relations with Regional Water Quality Control Board
C. Relations with U.S. Army Corps of Engineers
D. Relations with Bay Conservation Development Commission
E. Relations with United States Coast Guard
F. Relations with United Fish and Wildlife Service
G. Relations with National Marine Fisheries Service
H. Maintaining Traffic

The approved attenuator system shall be operating prior to beginning pile driving at any given pile location. If the attenuator fails, as determined by the Engineer, pile driving shall immediately stop. Piling driving at any given location shall not resume until the attenuator system at that location is again operating in conformance with the requirements of this section, as determined by the Engineer.

Failure of the attenuator system shall include, but not be limited to, the following methods of failure as determined by the Engineer:

A. The pressure or flow rate in any meter falls below 90% of its determined operating value during the pile driving operation. Operating values based on calculated data will be determined during the performance test.

B. During inspection of the perforated pipe the Engineer determines that erosion of the holes or debris has clogged the holes that will degrade the performance of the system.

The Contractor shall make provisions for the Engineer to inspect the bubble curtain system for proper operation before each deployment and as necessary during deployment. Proper operation during deployment will be determined by

observation of the digital meters (air flow) and analog gauges (pressure) in the monitoring station and by other methods developed by the Engineer.

The Contractor shall provide adequate means to prevent light from pile driving operations from shining directly into the water. At least 15 minutes prior to and during pile driving operations, the Contractor shall not shine light directly into the water in areas adjacent to piles being driven.

General

An air bubble curtain system is generally composed of an air compressor(s), supply lines to deliver the air, distribution manifolds or headers, perforated aeration pipes, and a frame. The frame facilitates transport and placement of the system, keeps the aeration pipes stable, and provides ballast to counteract the buoyancy of the aeration pipes in operation.

Air bubble curtain system shall conform to the following:

A. Air bubble system shall consist of multiple and concentric layers of perforated aeration pipes stacked vertically in accordance with the following:

Water Depth (m)	No. of Layers
0 to less than 5	2
5 to less than 10	4
10 to less than 15	7
15 to less than 20	10
20 to less than 25	13

B. Pipes in any layer shall be arranged in a geometric pattern, which shall allow for the pile driving operation to be completely enclosed by bubbles for the full depth of the water column and for a radial dimension of no more than 0.5 meters as measured from the outside surface of the pile.
C. The lowest layer of perforated aeration pipes shall be designed to ensure contact with the mudline without sinking into the bay mud.
D. The system shall provide a bubble flux of 2.0 cubic meters per minute per linear meter of pipe in each layer. Air holes shall be 1.6 mm in diameter and shall be spaced approximately 20 mm apart. Air holes shall be placed in four adjacent rows along the pipe to provide uniform bubble flux.
E. Meters shall be provided in accordance with the following:

1. Pressure meters shall be installed at all inlets to aeration pipelines (manifolds) and at points of lowest pressure in each branch of the aeration pipeline pipelines (manifolds).
2. The Flow meters shall be installed in the main line at each compressor and at each branch of the aeration pipelines (manifold). In applications where the feedline from the compressor is continuous from the compressor to the aeration pipe inlet, the flow meter at the compressor can be eliminated.
3. Flow meters shall be installed according to the manufacturer's recommendation based on either laminar flow or non-laminar flow, which ever applies.

Gauges shall be installed above the water line and shall be easily accessible to the Engineer. The Contractor shall keep a continuous electronic log of all meters and gauges when the system is operating. Readings shall be logged every 1 minute and at other times, as determined by the Engineer, when variation in the readings exceed 10%. The Contractor shall maintain a graphical plot showing the variation of the meter readings with time.

Air pressure and air flow meters and gauges shall be calibrated by a private laboratory approved by the Engineer prior to use in the attenuator system. Meters shall be accurate to within 2 percent.

The contractor shall conduct a performance test of the bubble curtain, prior to any pile driving operations, in order to confirm the calculated pressures and flow rates at each manifold ring.

The Contractor shall submit an inspection/performance report in conformance with "Working Drawings," of these special provisions within 72 hours following the performance test.

The Contractor shall monitor the condition of the attenuator system and prepare inspection reports daily during pile installation operations and no less than every other day during periods of no activity.

The Contractor's design, installation, maintenance, monitoring, operation and removal of the attenuator system shall take into account the site conditions and the requirements of pile installation. Factors to be taken into account include anchoring, moving, and dismantling the system; configuration of bay bottom; water velocity; water-surface conditions; air and water temperatures; and positioning of pile and pile-driving equipment relative to the bubble curtain system.

Water velocity at the site is expected to vary from zero to 3 knots and vary in direction due to changes in tidal flow. The

design of the system shall ensure that the system extends from bay bottom to the water surface during maximum water-current conditions and accommodates tidal changes.

The pile-driving barge shall be isolated from the noise-producing operations. This isolation shall be such that noise from the pile driving operation is not transmitted through the barge to the water column. The barge deploying or containing the pile-driving equipment is not required to be contained within the system.

The Contractor shall completely remove the attenuator system at the completion of the project and the system will remain the property of the Contractor.

Working Drawings

The Contractor shall submit working drawings with supplement for the attenuator system to the Engineer for approval in conformance with the provisions in "Working Drawings," of these special provisions, except as otherwise noted.

Working drawings with supplement shall be signed by a Mechanical Engineer who is registered in the State of California. Working drawings shall include the following:

A. Complete details of the system including mechanical and structural details.
B. Details of anchorage components, air compressors, supply lines, distribution manifolds, aeration pipes and frame.
C. Details of proposed means of isolating noise-producing systems on the pile-driving barge.
D. Details of meters, gauges, and recording devices.
E. Description of measures taken to avoid shining light into the water during pile driving operations.
F. Details of the manufacturer's recommendations for installation of the flow meters in conditions of laminar flow and non-laminar flow.

The supplement to the working drawing shall include the following:

A. Independently checked design calculations.
B. Materials list including the name of the manufacturer and the source, model number, description, and standard of manufacture.
C. Manufacturer's descriptive data and catalog cuts for all products proposed for the system including air compressors.
D. Calculations showing pressure loss in the piping system and estimated flows from the most removed orifice of the aeration piping.

Within 40 days after the approval of the contract, the Contractor shall submit working drawings, with supplements, to the Engineer. The Contractor shall allow the Engineer 20 days to review the working drawings. If revisions are required, as determined by the Engineer, the Contractor shall revise and resubmit the working drawings within 15 days of receipt of the Engineer's comments. The Contractor shall allow the Engineer 10 days to review the revised working drawings.

The Contractor shall submit inspection reports in conformance with "Working Drawings," of these special provisions within 48 hours following inspection.

Full compensation for furnishing all labor, materials, tools, equipment, and incidentals and for doing all the work involved in furnishing marine pile driving energy attenuators, complete in place, including designing, installing, inspecting, operating, maintaining, monitoring, recording, and removing the attenuator system, as specified in these special provisions, and as directed by the Engineer shall be considered as included in the contract lump sum price paid for temporary towers and no additional compensation will be allowed therefor.

Full compensation for isolation of pile-driving barge from pile installation noise shall be considered as included in the contract lump sum price paid for temporary towers and no additional compensation will be allowed therefor.

Appendix C

Sample calculations

Example API Member Check
Example API Joint Check including Member Capacity Check
Example Bolted Joint Check including Member Capacity Check

NOT FOR CONSTUCTION

TOWER G: MEMBER STRENGTH CHECK – API RP-2A LRFD 1993

The automated code check procedure in the program SAP was used to determine utilization ratios for all frame members in Tower G under the following load combinations:

Load Comb	DL	LL	Wind	Wave + Current	Seismic	Impact
LCMB4	1.4	-	-	-	-	-
LCMB7	1.1	1.3	-	-	-	-
LCMB8	1.0	1.0	0.5	1.0	-	1.0 (in Y)
LCMB9	1.0	1.0	0.5	1.0	-	1.0 (in X)
LCMB10	1.0	1.0	0.5	1.0	-	1.0 (angular)
LCMB13	1.0	1.0	-	-	1.0Ex+0.3Ey	-
LCMB14	1.0	1.0	-	-	-1.0Ex+0.3Ey	-
LCMB15	1.0	1.0	-	-	0.3Ex+1.0Ey	-
LCMB16	1.0	1.0	-	-	0.3Ex-1.0Ey	-
LCMB17	1.0	1.0	1.3	1.3	-	-

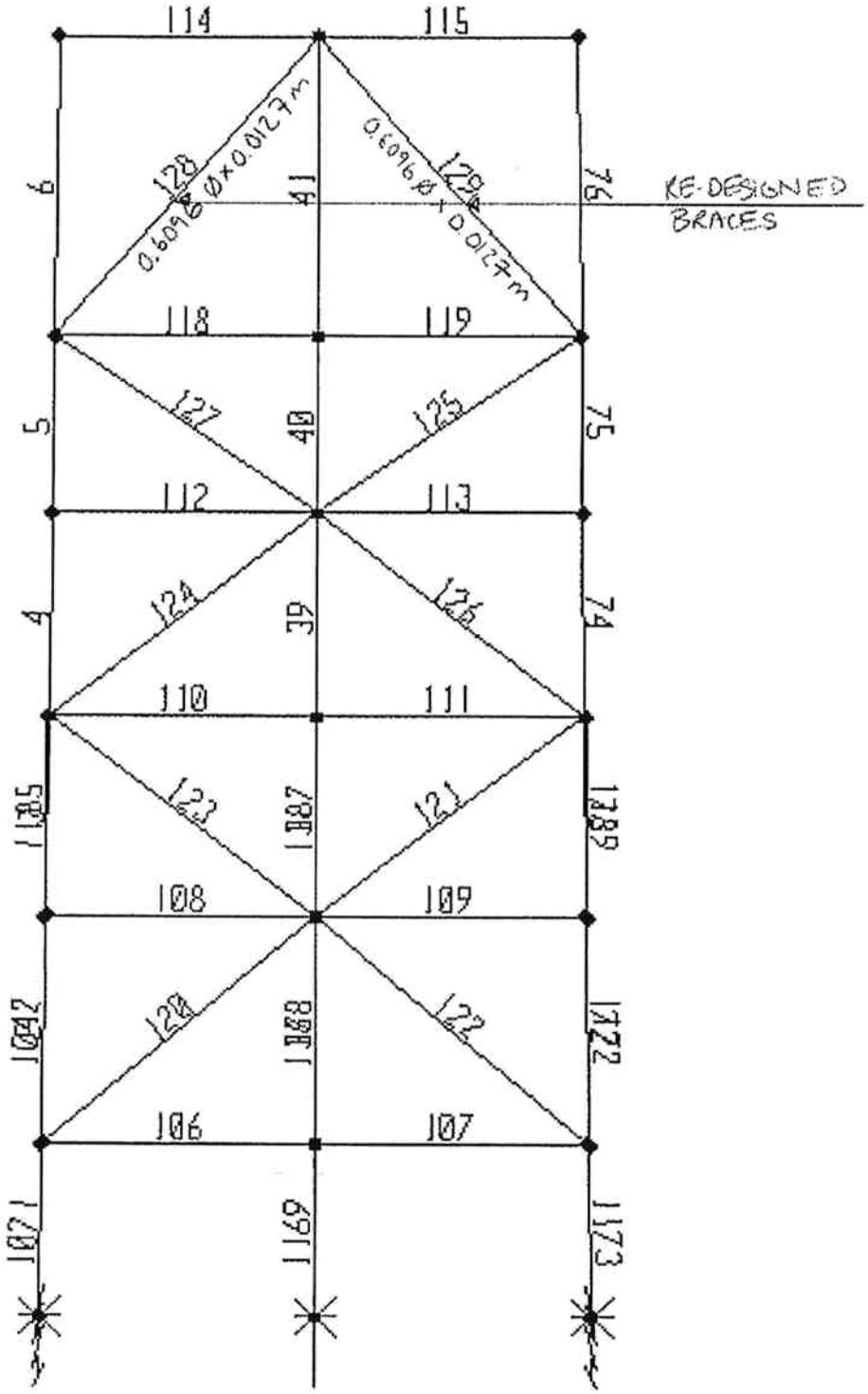
The program-computed effective length factors for buckling (K values) were overwritten as follows: A blanket K value of 1.0 was conservatively assumed for all leg and brace members. In addition, the program-computed unbraced length factors were overwritten with a blanket 1.0 factor. The unbraced length factor adjusts the node-to-node member length to take account of the true length of the member, in particular at the cross joints. A factor of 1.0 is justified, since one of the diagonal members will be in tension and thus restrain the opposite diagonal member in compression from buckling out-of-plane.

The API code check in SAP appears to use a combination of AISC and API parameters, so it is not a true API member strength check. Accordingly, we used the automated code check in SAP (with the above over-writes) to carry out a first-pass screening of the members, followed by a hand calculation according to API RP 2A LRFD (1993) to compute member strength utilization ratios of the most highly stressed member.

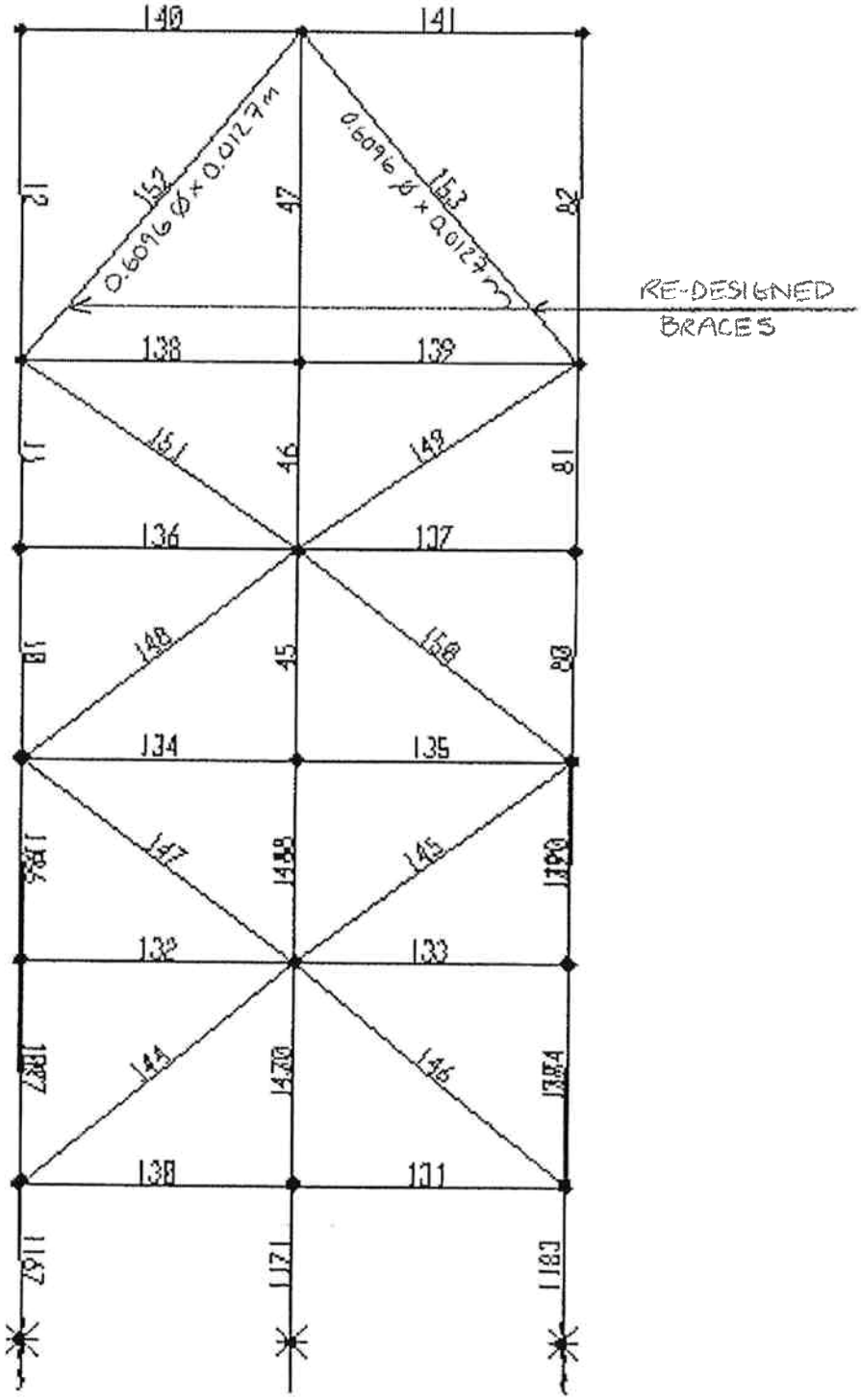
An initial code check in SAP indicated that the 0.4572m diameter by 0.0127m diagonal braces located on the two longitudinal frames in the upper bay are overstressed. These four braces were re-designed to be 0.6096m diameter by 0.0127m wall thickness and the automated code check was repeated. The listing of utilization ratios (as output by SAP), in descending order is provided on the following pages. Each of the six dummy pile members below mudline (members 1071, 1167, 1169, 1171, 1173, 1183) has a utilization ratio in excess of 1.0. The automated procedure for these six members is not appropriate, since the unbraced length of the piles below mudline is much less than the 9m length of the dummy pile members.

All other members have utilization ratios less than 1.0. The most highly stressed member is the diagonal in the top bay of Frame A (member 128). A detailed hand calculation according to the requirements of API RP 2A LRFD (1993) is presented for this particular

member and demonstrates that the utilization ratio is below 1.0. Combined axial compression and bending controls, with a utilization ratio of 0.855.



ELEVATION - FRAME A



ELEVATION - FRAME B

Table: Steel Design 1 - Summary Data - API RP2A-LRFD 97

SAP 2000

Frame Text	DesignSecl Text	DesignType Text	Status Text	Ratio Unitless	RatioType Text	Combo Text	Location m	ErrMsg Text	WarnMsg Text
1183	1PILE	Column	Overstressed	1.363988	PMM	LCMB8	9.01891	No Messages	No Messages
1173	1PILE	Column	Overstressed	1.173066	PMM	LCMB8	9.0757	No Messages	No Messages
1071	1PILE	Column	Overstressed	1.097808	PMM	LCMB9	9.0757	No Messages	No Messages
1171	1PILE	Column	Overstressed	1.077678	PMM	LCMB8	9.01891	No Messages	No Messages
1167	1PILE	Column	Overstressed	1.047181	PMM	LCMB10	9.01891	No Messages	No Messages
1169	1PILE	Column	Overstressed	1.026603	PMM	LCMB9	9.0757	No Messages	No Messages
128	4DIAG	Brace	No Messages	0.837575	PMM	LCMB13	0	No Messages	No Messages
152	4DIAG	Brace	No Messages	0.823979	PMM	LCMB13	0	No Messages	No Messages
129	4DIAG	Brace	No Messages	0.778753	PMM	LCMB9	0	No Messages	No Messages
99	5DIAG	Brace	No Messages	0.727094	PMM	LCMB8	0	No Messages	No Messages
1174	1PILE	Column	No Messages	0.711229	PMM	LCMB8	12.4653	No Messages	No Messages
126	4DIAG	Brace	No Messages	0.688433	PMM	LCMB9	0	No Messages	No Messages
1077	1PILE	Column	No Messages	0.639215	PMM	LCMB13	12.45591	No Messages	No Messages
1042	1PILE	Column	No Messages	0.637906	PMM	LCMB13	12.39903	No Messages	No Messages
100	5DIAG	Brace	No Messages	0.637428	PMM	LCMB8	10.82717	No Messages	No Messages
1172	1PILE	Column	No Messages	0.620774	PMM	LCMB13	12.40842	No Messages	No Messages
95	4DIAG	Brace	No Messages	0.617318	PMM	LCMB8	14.45339	No Messages	No Messages
1170	1PILE	Column	No Messages	0.60869	PMM	LCMB8	12.4653	No Messages	No Messages
153	4DIAG	Brace	No Messages	0.58779	PMM	LCMB13	0	No Messages	No Messages
96	4DIAG	Brace	No Messages	0.583847	PMM	LCMB8	0	No Messages	No Messages
1186	1PILE	Column	No Messages	0.57721	PMM	LCMB13	11.0627	No Messages	No Messages
1185	1PILE	Column	No Messages	0.576196	PMM	LCMB13	11.0627	No Messages	No Messages
1190	1PILE	Column	No Messages	0.572615	PMM	LCMB8	11.05331	No Messages	No Messages
1168	1PILE	Column	No Messages	0.561532	PMM	LCMB15	12.40842	No Messages	No Messages
1189	1PILE	Column	No Messages	0.556051	PMM	LCMB13	11.05331	No Messages	No Messages
86	6HORZ	Beam	No Messages	0.532065	PMM	LCMB8	17.742	No Messages	No Messages
1187	1PILE	Column	No Messages	0.524396	PMM	LCMB15	11.05331	No Messages	No Messages
1188	1PILE	Column	No Messages	0.522234	PMM	LCMB15	11.05331	No Messages	No Messages
124	4DIAG	Brace	No Messages	0.482814	PMM	LCMB13	0	No Messages	No Messages
148	4DIAG	Brace	No Messages	0.478186	PMM	LCMB13	0	No Messages	No Messages
74	2LEG	Column	No Messages	0.470269	PMM	LCMB8	0	No Messages	No Messages
82	2LEG	Column	No Messages	0.44387	PMM	LCMB4	0	No Messages	No Messages
41	2LEG	Column	No Messages	0.434406	PMM	LCMB4	0	No Messages	No Messages
101	5DIAG	Brace	No Messages	0.433849	PMM	LCMB9	0	No Messages	No Messages
81	2LEG	Column	No Messages	0.433604	PMM	LCMB4	10.10141	No Messages	No Messages
169	5DIAG	Beam	No Messages	0.43267	PMM	LCMB10	0	No Messages	No Messages
66	5DIAG	Brace	No Messages	0.43083	PMM	LCMB15	0	No Messages	No Messages
76	2LEG	Column	No Messages	0.428511	PMM	LCMB4	17.52015	No Messages	No Messages
125	4DIAG	Brace	No Messages	0.42433	PMM	LCMB13	0	No Messages	No Messages
75	2LEG	Column	No Messages	0.423121	PMM	LCMB4	10.1016	No Messages	No Messages
80	2LEG	Column	No Messages	0.419066	PMM	LCMB4	11.42325	No Messages	No Messages
170	5DIAG	Beam	No Messages	0.419036	PMM	LCMB10	11.61657	No Messages	No Messages
47	2LEG	Column	No Messages	0.416246	PMM	LCMB4	17.52015	No Messages	No Messages
40	2LEG	Column	No Messages	0.412632	PMM	LCMB4	10.1016	No Messages	No Messages
31	5DIAG	Brace	No Messages	0.408374	PMM	LCMB8	0	No Messages	No Messages
121	3DIAG	Brace	No Messages	0.407629	PMM	LCMB9	0	No Messages	No Messages
149	4DIAG	Brace	No Messages	0.406417	PMM	LCMB13	0	No Messages	No Messages
29	5DIAG	Brace	No Messages	0.404434	PMM	LCMB15	0	No Messages	No Messages
11	2LEG	Column	No Messages	0.400685	PMM	LCMB4	10.10141	No Messages	No Messages
127	4DIAG	Brace	No Messages	0.397247	PMM	LCMB9	0	No Messages	No Messages
46	2LEG	Column	No Messages	0.396821	PMM	LCMB4	10.10141	No Messages	No Messages
5	2LEG	Column	No Messages	0.390676	PMM	LCMB4	10.1016	No Messages	No Messages
10	2LEG	Column	No Messages	0.3885	PMM	LCMB4	11.42325	No Messages	No Messages

Frame Text	DesignSecl Text	DesignType Text	Status Text	Ratio Unitless	RatioType Text	Combo Text	Location m	ErrMsg Text	WarnMsg Text
98	4DIAG	Brace	No Messages	0.387707	PMM	LCMB9	0	No Messages	No Messages
39	2LEG	Column	No Messages	0.387386	PMM	LCMB4	11.42315	No Messages	No Messages
45	2LEG	Column	No Messages	0.38506	PMM	LCMB4	11.42325	No Messages	No Messages
67	5DIAG	Brace	No Messages	0.385043	PMM	LCMB15	0	No Messages	No Messages
4	2LEG	Column	No Messages	0.383597	PMM	LCMB4	11.42315	No Messages	No Messages
176	5DIAG	Beam	No Messages	0.382747	PMM	LCMB8	0	No Messages	No Messages
94	3DIAG	Brace	No Messages	0.374659	PMM	LCMB8	14.17092	No Messages	No Messages
102	5DIAG	Brace	No Messages	0.372166	PMM	LCMB9	0	No Messages	No Messages
12	2LEG	Column	No Messages	0.372122	PMM	LCMB4	0	No Messages	No Messages
28	4DIAG	Brace	No Messages	0.367006	PMM	LCMB8	0	No Messages	No Messages
97	4DIAG	Brace	No Messages	0.362487	PMM	LCMB9	0	No Messages	No Messages
27	4DIAG	Brace	No Messages	0.360678	PMM	LCMB8	0	No Messages	No Messages
6	2LEG	Column	No Messages	0.359964	PMM	LCMB13	17.52015	No Messages	No Messages
30	5DIAG	Brace	No Messages	0.358778	PMM	LCMB15	0	No Messages	No Messages
64	5DIAG	Brace	No Messages	0.355674	PMM	LCMB15	0	No Messages	No Messages
32	5DIAG	Brace	No Messages	0.355118	PMM	LCMB8	0	No Messages	No Messages
61	4DIAG	Brace	No Messages	0.354377	PMM	LCMB15	0	No Messages	No Messages
150	4DIAG	Brace	No Messages	0.352905	PMM	LCMB13	0	No Messages	No Messages
60	4DIAG	Brace	No Messages	0.351144	PMM	LCMB15	0	No Messages	No Messages
111	6HORZ	Beam	No Messages	0.347322	PMM	LCMB9	0	No Messages	No Messages
93	3DIAG	Brace	No Messages	0.335014	PMM	LCMB8	0	No Messages	No Messages
25	4DIAG	Brace	No Messages	0.325101	PMM	LCMB15	0	No Messages	No Messages
62	4DIAG	Brace	No Messages	0.31525	PMM	LCMB15	0	No Messages	No Messages
65	5DIAG	Brace	No Messages	0.309452	PMM	LCMB15	0	No Messages	No Messages
26	4DIAG	Brace	No Messages	0.30658	PMM	LCMB15	0	No Messages	No Messages
120	3DIAG	Brace	No Messages	0.295001	PMM	LCMB9	0	No Messages	No Messages
63	4DIAG	Brace	No Messages	0.289414	PMM	LCMB15	0	No Messages	No Messages
151	4DIAG	Brace	No Messages	0.279213	PMM	LCMB13	0	No Messages	No Messages
581	5DIAG	Beam	No Messages	0.211376	PMM	LCMB8	0	No Messages	No Messages
181	5DIAG	Beam	No Messages	0.205153	PMM	LCMB9	22.40586	No Messages	No Messages
110	6HORZ	Beam	No Messages	0.203887	PMM	LCMB9	15	No Messages	No Messages
145	3DIAG	Brace	No Messages	0.201445	PMM	LCMB13	0	No Messages	No Messages
144	3DIAG	Brace	No Messages	0.198722	PMM	LCMB13	0	No Messages	No Messages
567	5DIAG	Beam	No Messages	0.198575	PMM	LCMB9	0	No Messages	No Messages
568	5DIAG	Beam	No Messages	0.196557	PMM	LCMB9	11.61657	No Messages	No Messages
89	7HORZ	Beam	No Messages	0.191785	PMM	LCMB4	0	No Messages	No Messages
92	3DIAG	Brace	No Messages	0.181371	PMM	LCMB8	14.16966	No Messages	No Messages
582	5DIAG	Beam	No Messages	0.171758	PMM	LCMB8	12.02617	No Messages	No Messages
19	7HORZ	Beam	No Messages	0.171452	PMM	LCMB4	0	No Messages	No Messages
54	7HORZ	Beam	No Messages	0.169737	PMM	LCMB4	0	No Messages	No Messages
58	3DIAG	Brace	No Messages	0.162253	PMM	LCMB8	15.9688	No Messages	No Messages
122	3DIAG	Brace	No Messages	0.154454	PMM	LCMB9	19.50282	No Messages	No Messages
91	3DIAG	Brace	No Messages	0.153686	PMM	LCMB8	15.9688	No Messages	No Messages
575	5DIAG	Beam	No Messages	0.152722	PMM	LCMB10	0	No Messages	No Messages
107	6HORZ	Beam	No Messages	0.150388	PMM	LCMB9	0	No Messages	No Messages
73	2LEG	Column	No Messages	0.14845	PMM	LCMB8	11.0627	No Messages	No Messages
131	6HORZ	Beam	No Messages	0.14553	PMM	LCMB13	0	No Messages	No Messages
569	5DIAG	Beam	No Messages	0.141272	PMM	LCMB10	0	No Messages	No Messages
570	5DIAG	Beam	No Messages	0.140918	PMM	LCMB10	0	No Messages	No Messages
146	3DIAG	Brace	No Messages	0.133894	PMM	LCMB13	19.50288	No Messages	No Messages
147	3DIAG	Brace	No Messages	0.131763	PMM	LCMB13	0	No Messages	No Messages
175	5DIAG	Beam	No Messages	0.131046	PMM	LCMB13	0	No Messages	No Messages
139	7HORZ	Beam	No Messages	0.127356	PMM	LCMB4	0	No Messages	No Messages
576	5DIAG	Beam	No Messages	0.127019	PMM	LCMB10	12.02617	No Messages	No Messages
85	6HORZ	Beam	No Messages	0.126767	PMM	LCMB8	9.4	No Messages	No Messages

Frame Text	DesignSecl Text	DesignType Text	Status Text	Ratio Unitless	RatioType Text	Combo Text	Location m	ErrMsg Text	WarnMsg Text
138	7HORZ	Beam	No Messages	0.126454	PMM	LCMB4	0	No Messages	No Messages
123	3DIAG	Brace	No Messages	0.124941	PMM	LCMB13	0	No Messages	No Messages
56	3DIAG	Brace	No Messages	0.124711	PMM	LCMB8	15.9688	No Messages	No Messages
579	5DIAG	Beam	No Messages	0.12007	PMM	LCMB4	0	No Messages	No Messages
577	5DIAG	Beam	No Messages	0.116953	PMM	LCMB4	0	No Messages	No Messages
580	5DIAG	Beam	No Messages	0.116146	PMM	LCMB4	12.02617	No Messages	No Messages
578	5DIAG	Beam	No Messages	0.114355	PMM	LCMB4	12.02617	No Messages	No Messages
113	7HORZ	Beam	No Messages	0.113535	PMM	LCMB9	15	No Messages	No Messages
137	7HORZ	Beam	No Messages	0.112901	PMM	LCMB10	15	No Messages	No Messages
134	6HORZ	Beam	No Messages	0.110945	PMM	LCMB10	15	No Messages	No Messages
130	6HORZ	Beam	No Messages	0.110626	PMM	LCMB13	15	No Messages	No Messages
23	3DIAG	Brace	No Messages	0.109756	PMM	LCMB8	15.9688	No Messages	No Messages
106	6HORZ	Beam	No Messages	0.108493	PMM	LCMB13	15	No Messages	No Messages
13	6HORZ	Beam	No Messages	0.108084	PMM	LCMB9	20	No Messages	No Messages
184	5DIAG	Beam	No Messages	0.106522	PMM	LCMB4	0	No Messages	No Messages
87	7HORZ	Beam	No Messages	0.105302	PMM	LCMB8	0	No Messages	No Messages
156	5DIAG	Beam	No Messages	0.105269	PMM	LCMB4	0	No Messages	No Messages
182	5DIAG	Beam	No Messages	0.105097	PMM	LCMB4	0	No Messages	No Messages
168	5DIAG	Beam	No Messages	0.104731	PMM	LCMB4	11.61657	No Messages	No Messages
59	3DIAG	Brace	No Messages	0.104562	PMM	LCMB8	0	No Messages	No Messages
36	5DIAG	Beam	No Messages	0.103931	PMM	LCMB4	0	No Messages	No Messages
119	7HORZ	Beam	No Messages	0.10266	PMM	LCMB4	0	No Messages	No Messages
118	7HORZ	Beam	No Messages	0.099866	PMM	LCMB4	0	No Messages	No Messages
57	3DIAG	Brace	No Messages	0.098268	PMM	LCMB8	14.16966	No Messages	No Messages
88	7HORZ	Beam	No Messages	0.095788	PMM	LCMB8	0	No Messages	No Messages
21	3DIAG	Brace	No Messages	0.092554	PMM	LCMB10	15.9688	No Messages	No Messages
72	2LEG	Column	No Messages	0.090166	PMM	LCMB10	12.46435	No Messages	No Messages
24	3DIAG	Brace	No Messages	0.085094	PMM	LCMB8	0	No Messages	No Messages
52	7HORZ	Beam	No Messages	0.084422	PMM	LCMB8	0	No Messages	No Messages
133	6HORZ	Beam	No Messages	0.083995	PMM	LCMB10	0	No Messages	No Messages
1	5DIAG	Beam	No Messages	0.083884	PMM	LCMB4	20.51828	No Messages	No Messages
7	5DIAG	Beam	No Messages	0.083884	PMM	LCMB4	20.51828	No Messages	No Messages
108	6HORZ	Beam	No Messages	0.082822	PMM	LCMB9	0	No Messages	No Messages
48	6HORZ	Beam	No Messages	0.082359	PMM	LCMB10	20	No Messages	No Messages
49	6HORZ	Beam	No Messages	0.081838	PMM	LCMB8	9.402	No Messages	No Messages
38	2LEG	Column	No Messages	0.078277	PMM	LCMB8	0	No Messages	No Messages
22	3DIAG	Brace	No Messages	0.078193	PMM	LCMB10	14.16966	No Messages	No Messages
79	2LEG	Column	No Messages	0.076156	PMM	LCMB8	11.0627	No Messages	No Messages
135	6HORZ	Beam	No Messages	0.075822	PMM	LCMB10	0	No Messages	No Messages
83	6HORZ	Beam	No Messages	0.075425	PMM	LCMB13	20	No Messages	No Messages
109	6HORZ	Beam	No Messages	0.074612	PMM	LCMB9	0	No Messages	No Messages
78	2LEG	Column	No Messages	0.071756	PMM	LCMB8	12.46445	No Messages	No Messages
136	7HORZ	Beam	No Messages	0.068678	PMM	LCMB13	0	No Messages	No Messages
37	2LEG	Column	No Messages	0.063123	PMM	LCMB8	12.46435	No Messages	No Messages
44	2LEG	Column	No Messages	0.062801	PMM	LCMB8	11.0627	No Messages	No Messages
112	7HORZ	Beam	No Messages	0.060179	PMM	LCMB13	0	No Messages	No Messages
51	6HORZ	Beam	No Messages	0.057462	PMM	LCMB10	0	No Messages	No Messages
9	2LEG	Column	No Messages	0.057126	PMM	LCMB10	11.0627	No Messages	No Messages
132	6HORZ	Beam	No Messages	0.057035	PMM	LCMB13	0	No Messages	No Messages
8	2LEG	Column	No Messages	0.053009	PMM	LCMB10	12.46445	No Messages	No Messages
14	6HORZ	Beam	No Messages	0.052277	PMM	LCMB10	9.402	No Messages	No Messages
84	6HORZ	Beam	No Messages	0.050994	PMM	LCMB8	0	No Messages	No Messages
2	2LEG	Column	No Messages	0.050156	PMM	LCMB13	0	No Messages	No Messages
16	6HORZ	Beam	No Messages	0.049684	PMM	LCMB4	17.742	No Messages	No Messages
3	2LEG	Column	No Messages	0.049349	PMM	LCMB8	11.0627	No Messages	No Messages

Frame Text	DesignSecl Text	DesignTyp Text	Status Text	Ratio Unitless	RatioType Text	Combo Text	Location m	ErrMsg Text	WarnMsg Text
18	7HORZ	Beam	No Messages	0.046869	PMM	LCMB8	8.32	No Messages	No Messages
114	7HORZ	Beam	No Messages	0.044831	PMM	LCMB4	0	No Messages	No Messages
115	7HORZ	Beam	No Messages	0.044831	PMM	LCMB4	0	No Messages	No Messages
140	7HORZ	Beam	No Messages	0.044831	PMM	LCMB4	0	No Messages	No Messages
141	7HORZ	Beam	No Messages	0.044831	PMM	LCMB4	0	No Messages	No Messages
43	2LEG	Column	No Messages	0.044055	PMM	LCMB8	12.46445	No Messages	No Messages
20	7HORZ	Beam	No Messages	0.039053	PMM	LCMB4	0	No Messages	No Messages
55	7HORZ	Beam	No Messages	0.039053	PMM	LCMB4	0	No Messages	No Messages
90	7HORZ	Beam	No Messages	0.039053	PMM	LCMB4	0	No Messages	No Messages
50	6HORZ	Beam	No Messages	0.038426	PMM	LCMB8	9.4	No Messages	No Messages
15	6HORZ	Beam	No Messages	0.033965	PMM	LCMB15	9.4	No Messages	No Messages
157	DECKDUM	Beam	No Messages	0.033612	PMM	LCMB7	0	No Messages	No Messages
158	DECKDUM	Beam	No Messages	0.033612	PMM	LCMB7	0	No Messages	No Messages
159	DECKDUM	Beam	No Messages	0.033612	PMM	LCMB7	0	No Messages	No Messages
160	DECKDUM	Beam	No Messages	0.033612	PMM	LCMB7	0	No Messages	No Messages
17	7HORZ	Beam	No Messages	0.028997	PMM	LCMB8	8.324	No Messages	No Messages
53	7HORZ	Beam	No Messages	0.026225	PMM	LCMB15	0	No Messages	No Messages
42	DECKDUM	Beam	No Messages	0.00732	PMM	LCMB7	0	No Messages	No Messages
71	DECKDUM	Beam	No Messages	0.00732	PMM	LCMB7	7	No Messages	No Messages
155	DECKDUM	Beam	No Messages	0.00732	PMM	LCMB7	0	No Messages	No Messages
167	DECKDUM	Beam	No Messages	0.00732	PMM	LCMB7	0	No Messages	No Messages
33	2LEG	Column	No Messages	0.000183	PMM	LCMB4	0	No Messages	No Messages
34	2LEG	Column	No Messages	0.000183	PMM	LCMB4	0	No Messages	No Messages
68	2LEG	Column	No Messages	0.000183	PMM	LCMB4	0	No Messages	No Messages
69	2LEG	Column	No Messages	0.000183	PMM	LCMB4	0	No Messages	No Messages
103	2LEG	Column	No Messages	0.000183	PMM	LCMB4	0	No Messages	No Messages
104	2LEG	Column	No Messages	0.000183	PMM	LCMB4	0	No Messages	No Messages
35	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
70	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
77	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
105	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
116	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
117	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
142	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
143	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
154	DECKDUM	Beam	No Messages	0			0	No Messages	No Messages
161	DECKDUM	Column	No Messages	0			0	No Messages	No Messages
162	DECKDUM	Column	No Messages	0			0	No Messages	No Messages
163	DECKDUM	Column	No Messages	0			0	No Messages	No Messages
164	DECKDUM	Column	No Messages	0			0	No Messages	No Messages
165	DECKDUM	Column	No Messages	0			0	No Messages	No Messages
166	DECKDUM	Column	No Messages	0			0	No Messages	No Messages

FRAME A , BRACE 128

ORIGINAL 0.4572m Ø × 0.0127m BRACE DOES NOT MEET STRENGTH REQUIREMENTS.

THUS, TRY 0.6096m Ø × 0.0127

LOCATION O "LMB13"

$P_u = -2366.57$ KN
$(M_u)_{major} = M_3 = -68.8961$ kN·m
$(M_u)_{minor} = M_2 = -4.3574$ kN m
$T_u = -0.8089$ kN m
$(V_u)_{major} = V_2 = -15.215$ kN
$(V_u)_{minor} = V_3 = -0.308$ kN

PROPERTIES

0.6096m Ø × 0.0127m
$A = 0.0238 m^2$
$A_s = 0.0123 m^2$
$I = 0.001061 m^4$
$J = 0.002122 m^4$
$S = 0.003481 m^3$
$Z = 0.004526 m^3$
$r = 0.2111$

AXIAL COMPRESSION:

$$\lambda = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} \qquad K=1.0 \qquad L=23.0642 m$$

WHERE $F_y = \min\{F_{xe}, F_{xc}\}$

$F_{xe} = 2C_x E (t/D) = (2)(0.3)(200,000)(0.0127/0.6096) = 2500$ MPa

$F_{xc} = F_y = 345$ MPa for $\frac{D}{t} = 48 \leq 60$

$\Rightarrow F_y = 345$ MPa

$$\lambda = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} = \frac{(1.0)(23.0642)}{(\pi)(0.2111)}\sqrt{\frac{345}{200{,}000}} = 1.444 > \sqrt{2}$$

$$F_{cn} = \frac{1}{\lambda^2}F_y = \frac{1}{(1.444)^2}F_y = 0.479\,F_y = \underline{165.36\ MPa}$$

$$f_c = \frac{-2366.57/1000}{0.0238} = 99.44\ MPa$$

EQN D.2.2-1 $$\frac{f_c}{\phi_c F_{cn}} = \frac{99.44}{(0.85)(165.36)} = \underline{\underline{0.707}}$$

BENDING:

$$\frac{D}{t} = \frac{0.6096}{0.0127} = 48 \qquad \frac{10340}{F_y} = 29.97 < \frac{D}{t} < \frac{20680}{F_y} = 59.94$$

$\Rightarrow$ D.2.3-2b: $$F_{bn} = \left[1.13 - 2.58\frac{F_y D}{E t}\right]\left(\frac{Z}{S}\right)F_y$$

$$= \left[1.13 - 2.58\frac{(345)(48)}{(200{,}000)}\right]\cdot\left[\frac{0.004526}{0.001061}\right]\cdot F_y$$

$$= 1.19\,F_y = \underline{411.06\ MPa}$$

$$M_p = f_y \cdot Z = 345 \times 1000 \times 0.004526 = 1561.47\ kN\cdot m$$

$$M_y = f_y \cdot S = 345 \times 1000 \times 0.003481 = 1200.95\ kN\cdot m$$

$$(M_u)_{major},\ (M_u)_{minor} < M_y \Rightarrow f_b = \frac{M}{S}$$

$$(f_b)_{major} = \frac{(M_u)_{major}}{S} = \frac{68.8961/1000}{0.003481} = 19.792\ MPa$$

$$(f_b)_{minor} = \frac{(M_u)_{minor}}{S} = \frac{4.3574/1000}{0.003481} = 1.252\ MPa$$

EQN D 2.3-1 $\dfrac{(f_b)_{major}}{\phi_b F_{bn}} = \dfrac{19.792}{0.95 \times 411.06} = \underline{\underline{0.0507}}$

$\dfrac{(f_b)_{minor}}{\phi_b F_{bn}} = \dfrac{1.252}{0.95 \times 411.06} = \underline{\underline{0.0032}}$

SHEAR:

$F_{vn} = \dfrac{F_y}{\sqrt{3}} = 0.577 F_y = \underline{199.19 \text{ MPa}}$

$(f_v)_{major} = \dfrac{2V}{A} = \dfrac{(2)(15.215/1000)}{0.0238} = 1.279 \text{ MPa}$

EQN D.2.4-1 $\dfrac{(f_v)_{major}}{\phi_v F_{vn}} = \dfrac{1.279}{0.95 \times 199.19} = \underline{\underline{0.0068}}$

TORSIONAL SHEAR:

$F_{vtn} = \dfrac{F_y}{\sqrt{3}} = 0.577 F_y = 199.19 \text{ MPa}$

$f_{vt} = \dfrac{M_t D}{2 J_p} = \dfrac{(0.8089/1000)(.6096)}{(2)(0.002122)} = 0.116 \text{ MPa}$

EQN D.2.4-3 $\dfrac{f_{vt}}{\phi_v F_{vtn}} = \dfrac{0.116}{(0.95)(199.19)} = \underline{\underline{0.0006}}$

COMBINED AXIAL COMPRESSION AND BENDING:

EULER BUCKLING $\lambda = \lambda_y = \lambda_z = 1.444$

$$F_{ey} = \frac{F_y}{\lambda_y^2} = F_{ez} = \frac{F_y}{\lambda_z^2} = \frac{F_y}{\lambda^2} = \frac{F_y}{1.444^2} = 0.479 F_y = 165.36 \text{ MPa}$$

EQN D.3.2-1:

$$\frac{f_c}{\phi_c F_{cn}} + \frac{1}{\phi_b F_{bn}} \sqrt{\left[\frac{C_{my} f_{by}}{\left(1 - \frac{f_c}{\phi_c F_{ey}}\right)}\right]^2 + \left[\frac{C_{mz} f_{bz}}{\left(1 - \frac{f_c}{\phi_c F_{ez}}\right)}\right]^2}$$

$$= \frac{99.44}{(0.85)(165.36)} + \frac{1}{(0.95)(411.06)} \sqrt{\left(\frac{(0.85)(19.792)}{(1 - 99.44/0.85/165.36)}\right)^2 + \left(\frac{(0.85)(1.252)}{(1 - 99.44/0.85/165.36)}\right)^2}$$

$$= 0.707 + \frac{57.625}{(0.95)(411.06)} = 0.707 + 0.1476$$

$$= \underline{\underline{0.855}}$$

Note. C_m was conservatively assumed as 0.85
$C_m = 1.0 - 0.4 \frac{f_c}{\phi_c F_c} = 1.0 - (0.4)\frac{(99.4)}{(0.85)(165.4)} = 0.72$
could be used instead.

EQN D.3.2-2:

$$1 - \cos\left[\frac{\pi}{2} \cdot \frac{f_c}{\phi_c F_{xc}}\right] + \frac{\sqrt{f_{by}^2 + f_{bz}^2}}{\phi_b F_{bn}}$$

$$= 1 - \cos\left(\frac{\pi}{2} \cdot \frac{99.44}{(0.85)(345)}\right) + \frac{\sqrt{19.792^2 + 1.252^2}}{0.95 \times 411.06}$$

$$= 1 - 0.861 + 0.0508 = 0.139 + 0.0508 = \underline{\underline{0.189}}$$

EQN D.3.2-3: $\frac{f_c}{\phi_c F_{xc}} = \frac{99.44}{0.85 \times 345} = \underline{\underline{0.339}}$

MOFFATT & NICHOL	CALTRANS	JOB NO 4141-05	
	SFOBB TEMP TOWERS	SHEET 5 OF 5	
		DESIGNED JRM	DATE 18 OCT '05
		CHECKED JAO	DATE 28 OCT 05

SUMMARY:

	API EQN.	UTILIZATION
AXIAL COMPRESSION	D.2.2-1	0.707
BENDING	D.2.3-1	0.051
SHEAR	D.2.4-1	0.007
TORSIONAL SHEAR	D.2.4-3	0.0006
COMBINED STRESSES	D.3.2-1	0.855
"	D.3.2-2	0.189
"	D3.2-3	0.339

Tower C Alone API Code Member Check SAP 2000

Frame	DesignSect	DesignType	Status	Ratio	RatioType	Combo	Location m
46	30LEG	Column	No Messages	0.97	PMM	LCMB4	0
123	30LEG	Column	No Messages	0.96	PMM	LCMB4	0
126	30LEG	Column	No Messages	0.96	PMM	LCMB6	1
49	30LEG	Column	No Messages	0.96	PMM	LCMB6	1
124	30LEG	Column	No Messages	0.79	PMM	LCMB4	0
47	30LEG	Column	No Messages	0.77	PMM	LCMB4	0
125	30LEG	Column	No Messages	0.68	PMM	LCMB2	0
119	30LEG	Column	No Messages	0.68	PMM	LCMB4	18.5
48	30LEG	Column	No Messages	0.68	PMM	LCMB2	0
42	30LEG	Column	No Messages	0.68	PMM	LCMB4	18.5
43	30LEG	Column	No Messages	0.49	PMM	LCMB4	0
122	30LEG	Column	No Messages	0.49	PMM	LCMB1	0
45	30LEG	Column	No Messages	0.47	PMM	LCMB1	0
120	30LEG	Column	No Messages	0.47	PMM	LCMB4	0
521	30LEG	Column	No Messages	0.19	PMM	LCMB4	10.0
520	30LEG	Column	No Messages	0.19	PMM	LCMB6	10.0
524	24STUB	Beam	No Messages	0.44	PMM	LCMB6	0
522	24STUB	Beam	No Messages	0.41	PMM	LCMB6	0
525	24CHRD	Beam	No Messages	0.18	PMM	LCMB6	0
523	24CHRD	Beam	No Messages	0.17	PMM	LCMB6	0
571	24CHRD	Beam	No Messages	0.12	PMM	LCMB6	14
179	24CHRD	Beam	No Messages	0.08	PMM	LCMB6	14
181	18HORZ	Beam	No Messages	0.41	PMM	LCMB4	0
127	18HORZ	Beam	No Messages	0.35	PMM	LCMB6	0
50	18HORZ	Beam	No Messages	0.35	PMM	LCMB6	0
180	18HORZ	Beam	No Messages	0.26	PMM	LCMB2	0
182	18HORZ	Beam	No Messages	0.25	PMM	LCMB4	0
51	18HORZ	Beam	No Messages	0.13	PMM	LCMB6	0
128	18HORZ	Beam	No Messages	0.11	PMM	LCMB6	0
178	18HORZ	Beam	No Messages	0.09	PMM	LCMB4	0
629	18DIAG	Brace	No Messages	0.90	PMM	LCMB6	0
628	18DIAG	Brace	No Messages	0.90	PMM	LCMB6	0
627	18DIAG	Brace	No Messages	0.84	PMM	LCMB6	0
626	18DIAG	Brace	No Messages	0.84	PMM	LCMB6	0
1	18DIAG	Brace	No Messages	0.83	PMM	LCMB1	5.2
2	18DIAG	Brace	No Messages	0.80	PMM	LCMB1	5.2
587	18DIAG	Brace	No Messages	0.78	PMM	LCMB6	0
622	18DIAG	Brace	No Messages	0.78	PMM	LCMB4	0
588	18DIAG	Brace	No Messages	0.77	PMM	LCMB6	0
585	18DIAG	Brace	No Messages	0.76	PMM	LCMB6	0
586	18DIAG	Brace	No Messages	0.75	PMM	LCMB6	0
621	18DIAG	Brace	No Messages	0.75	PMM	LCMB4	0

612	18DIAG	Brace	No Messages	0.72	PMM	LCMB4	0
613	18DIAG	Brace	No Messages	0.72	PMM	LCMB4	0
611	18DIAG	Brace	No Messages	0.70	PMM	LCMB4	0
614	18DIAG	Brace	No Messages	0.69	PMM	LCMB4	0
570	18DIAG	Brace	No Messages	0.68	PMM	LCMB4	0
435	18DIAG	Brace	No Messages	0.67	PMM	LCMB4	0
583	18DIAG	Brace	No Messages	0.66	PMM	LCMB4	0
623	18DIAG	Brace	No Messages	0.66	PMM	LCMB4	0
582	18DIAG	Brace	No Messages	0.66	PMM	LCMB4	0
580	18DIAG	Brace	No Messages	0.64	PMM	LCMB4	0
624	18DIAG	Brace	No Messages	0.64	PMM	LCMB4	0
433	18DIAG	Brace	No Messages	0.63	PMM	LCMB4	0
581	18DIAG	Brace	No Messages	0.62	PMM	LCMB4	0
434	18DIAG	Brace	No Messages	0.60	PMM	LCMB4	0
616	18DIAG	Brace	No Messages	0.53	PMM	LCMB6	0
617	18DIAG	Brace	No Messages	0.53	PMM	LCMB6	0
228	18DIAG	Brace	No Messages	0.51	PMM	LCMB6	0
121	18DIAG	Brace	No Messages	0.51	PMM	LCMB4	0
112	18DIAG	Brace	No Messages	0.50	PMM	LCMB4	0
222	18DIAG	Brace	No Messages	0.49	PMM	LCMB4	0
217	18DIAG	Brace	No Messages	0.48	PMM	LCMB4	0
618	18DIAG	Brace	No Messages	0.47	PMM	LCMB6	0
251	18DIAG	Brace	No Messages	0.47	PMM	LCMB6	0
619	18DIAG	Brace	No Messages	0.47	PMM	LCMB6	0
255	18DIAG	Brace	No Messages	0.47	PMM	LCMB6	0
575	18DIAG	Brace	No Messages	0.44	PMM	LCMB6	0
576	18DIAG	Brace	No Messages	0.43	PMM	LCMB6	0
577	18DIAG	Brace	No Messages	0.42	PMM	LCMB6	0
256	18DIAG	Brace	No Messages	0.42	PMM	LCMB6	0
578	18DIAG	Brace	No Messages	0.41	PMM	LCMB6	0
226	18DIAG	Brace	No Messages	0.37	PMM	LCMB4	7.8
573	18DIAG	Beam	No Messages	0.36	PMM	LCMB6	0
130	18DIAG	Brace	No Messages	0.34	PMM	LCMB4	7.8
225	18DIAG	Brace	No Messages	0.33	PMM	LCMB4	0
129	18DIAG	Brace	No Messages	0.31	PMM	LCMB4	0
207	18DIAG	Brace	No Messages	0.23	PMM	LCMB6	8.6
177	18DIAG	Brace	No Messages	0.20	PMM	LCMB6	0
173	18DIAG	Brace	No Messages	0.20	PMM	LCMB6	8.6
137	18DIAG	Brace	No Messages	0.19	PMM	LCMB6	0
229	18DIAG	Beam	No Messages	0.11	PMM	LCMB6	0
477	18DIAG	Beam	No Messages	0.10	PMM	LCMB2	18.4

	D	t	D/t
30LEG	0.762	0.025	30.0
18DIAG	0.762	0.013	60.0
18HORZ	0.457	0.010	38.4
24CHRD	0.610	0.016	38.4
24STUB	0.610	0.025	24.0

TOWER G: JOINT PUNCHING SHEAR CHECK – API RP-2A LRFD 1997

The automated code check procedure in the program SAP was used to determine utilization ratios for all tubular joints in Tower G under the following load combinations:

Load Comb	DL	LL	Wind	Wave + Current	Seismic	Impact
LCMB4	1.4	-	-	-	-	-
LCMB7	1.1	1.3	-	-	-	-
LCMB8	1.0	1.0	0.5	1.0	-	1.0 (in Y)
LCMB9	1.0	1.0	0.5	1.0	-	1.0 (in X)
LCMB10	1.0	1.0	0.5	1.0	-	1.0 (angular)
LCMB13	1.0	1.0	-	-	1.0Ex+0.3Ey	-
LCMB14	1.0	1.0	-	-	-1.0Ex+0.3Ey	-
LCMB15	1.0	1.0	-	-	0.3Ex+1.0Ey	-
LCMB16	1.0	1.0	-	-	0.3Ex-1.0Ey	-
LCMB17	1.0	1.0	1.3	1.3	-	-

Joint classification should be based on load pattern as well as joint geometry. The automated joint check procedure in SAP does neither. Thus, overwriting of default parameters is required when using SAP. Other structural codes may be available that perform the API code checks and that base joint classification either on joint geometry only or joint geometry and load pattern.

SAP uses a default joint classification of T&Y. This is conservative in cases where the joint is classified as 100% or partial K, but non-conservative in the case of X joints. For this reason, the following scheme was utilized. The default T&Y joint classification was accepted for all joints, with the exception of the X joints. For these joints, the joint classification was re-defined to X. This is a conservative approach. If joints fail under this procedure, it is recommended that the load pattern in the joint be determined for each load condition to determine the correct joint classification. An example of this procedure is shown on the following sheets, which show a hand calculation of the API LRFD joint punching shear check for Joint 11, including determination of joint classification. For all design load combinations, with the exception of LCMB9 and LCMB15 max, the joint is classified as a K Joint. In the case of LCMB9 and LCMB15 max, the upper and lower brace, respectively, is defined as a partial K and T&Y. The remainder of the hand calculation is carried out for the controlling design load combination, LCMB4. The results are identical to the SAP results when the joint classification is overwritten and defined as K in SAP. They are also identical to a spreadsheet that was developed, which was used for those joints that failed the automated SAP code check and required increased can thickness. Note that for the automated code check of the full model, this particular joint (Joint 11), although a K joint, was checked as a T&Y joint per the screening procedure described above.

The listing of utilization ratios (as output by SAP), in descending order is provided on the pages following the design example. A total of 8 joints have utilization ratios that exceed 1.0. Seven out of the eight joints are X joints. The overstress due to punching shear can be remedied in 7 of the 8 joints with a 1/8" increase in wall thickness of the chord member. One joint is marginal with the 1/8" increase in wall thickness, but is well below the 1.0 limit when the wall thickness is increased by ¼". A summary of the utilization ratios for the originally over-stressed joints and their revised utilization ratios with increased can thickness are provided in tabular format and details of the calculations (original and revised utilization ratios) are provided on the pages that follow.

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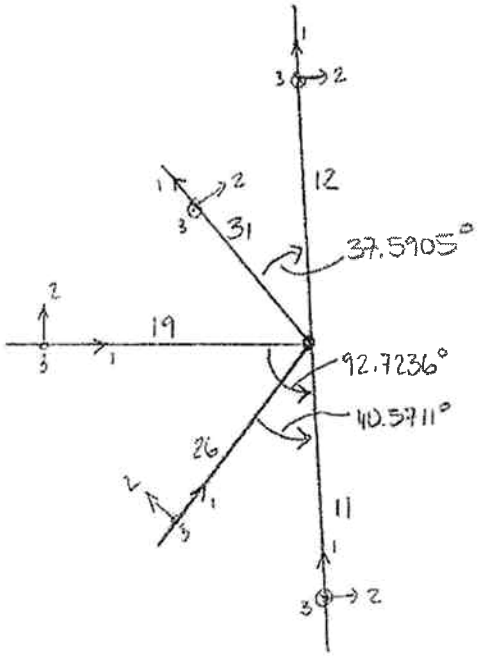
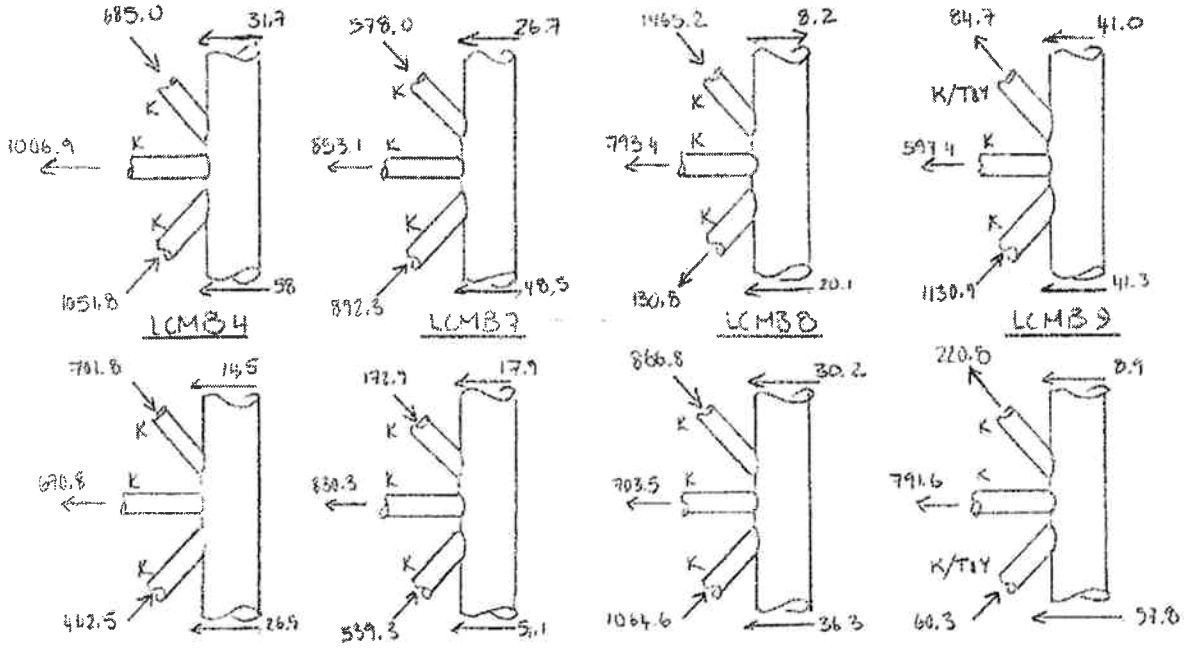
		JAD	28 OCT 05

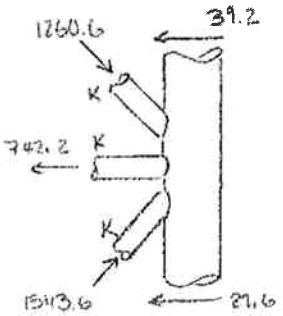
JOINT 11



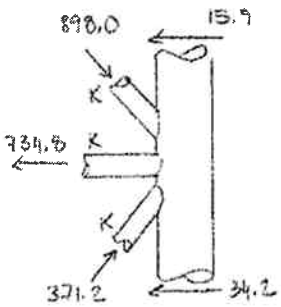


5.1

MOFFATT & NICHOL			
FIRM	CALTRANS	JOB NO.	4141
PROJECT	SFOBB TEMP TOWERS	SHEET	2 OF 4
DESIGNED BY	TOWER 6	CHKD BY	JRM DATE 11 OCT '05
		REVIEWED	JAO DATE 28 OCT 05



LCMB 15 MIN



LCMB 17

11, 12	"2LEG"	1.2192 Ø × 0.0254 m (48" Ø × 1")	A = 0.0953 m²	S = 0.0279 m³
19	"7HORZ"	0.4572 Ø × 0.0127 m (18" Ø × 1/2")	A = 0.0177 m²	S = 0.001918 m³
26	"4DIAG"	0.6096 Ø × 0.0127 m (24" Ø × 1/2")	A = 0.0238 m²	S = 0.003481 m³
31	"5DIAG"	0.4572 Ø × 0.0127 m (18" Ø × 1/2")	A = 0.0177 m²	S = 0.001918 m³

S-1

MOFFATT & NICHOL	CALTRANS	4141	
	SFOBB TEMP TOWERS	3	4
	TOWER G	JRM	12 OCT '05
		JAD	28 OCT 05

$\theta = 92.7236°$
$g = 0$
$t = 0.0127$
$T = 0.0254$
$d = 0.4572$
$D = 1.2192$

$$\tau = \frac{t}{T} = \frac{0.0127}{0.0254} = 0.5$$

$$\beta = \frac{d}{D} = \frac{0.4572}{1.2192} = 0.375$$

$$\gamma = \frac{D}{2T} = \frac{1.2192}{2 \times 0.0254} = 24$$

Q_u FOR K JOINT

Axial Tension / Compression

$Q_g = 1.8 - 4g/D = 1.8$ for $\gamma > 20$

$Q_u = (3.4 + 19\beta)\, Q_g$

$= (3.4 + 19 \times 0.375) \times 1.8 = \underline{18.945}$

IPB

$Q_u = 3.4 + 19\beta$

$= 3.4 + 19 \times 0.375 = \underline{10.525}$

OPB

$Q_\beta = 1.0$ for $\beta < 0.6$

$Q_u = (3.4 + 7\beta)\, Q_\beta$

$= (3.4 + 7 \times 0.375) \times 1.0$

$= \underline{6.025}$

LIMB 4 chord $P = -9777.37$ kN $M_{ipb} = 253.29$ kN·m $M_{opb} = -257.369$ kN·m

$$f_{ax} = \frac{9777.37/1000}{0.09526} = \underline{102.6 \text{ MPa}} \qquad f_{ipb} = \frac{253.92/1000}{0.02785} = \underline{9.1 \text{ MPa}}$$

$$f_{opb} = \frac{257.37/1000}{0.02785} = \underline{9.2 \text{ MPa}}$$

$$A = \sqrt{\frac{f_{ax}^2 + f_{ipb}^2 + f_{opb}^2}{\phi_q F_y}} = \frac{\sqrt{102.6^2 + 9.1^2 + 9.2^2}}{0.95 \times 345} = \underline{0.316}$$

$(Q_f)_{ax} = 1.0 - 0.030 \times 24 \times 0.316^2 = 0.9283$

$(Q_f)_{ipb} = 1.0 - 0.045 \times 24 \times 0.316^2 = 0.8924$

$(Q_f)_{opb} = 1.0 - 0.021 \times 24 \times 0.316^2 = 0.9498$

BRACE: $P_D = -1006.8$ kN $\quad M_{ipb} = 40.27$ kN·m $\quad M_{opb} = 3.28$ kN·m

$f_{ax} = \dfrac{1006.9/1000}{0.017735} = \underline{56.8 \text{ MPa}}$ $\qquad f_{ipb} = \dfrac{40.27/1000}{0.001918} = \underline{21.0 \text{ MPa}}$

$f_{opb} = \dfrac{3.28/1000}{0.001918} = \underline{1.7 \text{ MPa}}$

$$P_{uj} = \frac{F_y T^2}{\sin\theta} Q_u Q_f = \frac{345000 \times 0.0254^2}{\sin(92.7236°)} \cdot 18.945 \times 0.9283$$

$$= \underline{3918.9 \text{ kN}} \qquad \text{(Tension or Compression)}$$

$$(M_{uj})_{ipb} = \frac{F_y T^2}{\sin\theta}(0.8d) Q_u Q_f = \frac{345000 \times 0.0254^2}{\sin(92.7236°)}(0.8)(0.4572) \cdot Q_u Q_f$$

$$= 81.503 \cdot Q_u Q_f = (81.503)(10.525)(0.8924) = \underline{765.5 \text{ kN·m}}$$

$$(M_{uj})_{opb} = (81.503)(6.025)(0.9498) = \underline{466.4 \text{ kN·m}}$$

EQN 3.2: $\dfrac{P_D}{\phi_j P_{uj}} = \dfrac{1006.8}{0.95 \times 3918.9} = \underline{\underline{0.270}}$

EQN 3.3: $\left(\dfrac{M_D}{\phi_j M_{uj}}\right)_{ipb} = \dfrac{40.27}{0.95 \times 765.5} = \underline{\underline{0.055}}$

$\left(\dfrac{M_D}{\phi_j M_{uj}}\right)_{opb} = \dfrac{3.28}{0.95 \times 466.4} = 0.007$

EQN 3-4: $1 - \cos\left\{\dfrac{\pi}{2}\left(\dfrac{P_D}{\phi_j P_{uj}}\right)\right\} + \sqrt{\left(\dfrac{M_D}{\phi_j M_{uj}}\right)_{ipb}^2 + \left(\dfrac{M_D}{\phi_j M_{uj}}\right)_{opb}^2}$

$= 1 - \cos\left(\dfrac{\pi}{2} \cdot 0.270\right) + \sqrt{0.055^2 + 0.007^2}$

$= 0.0886 + 0.055 = \underline{\underline{0.144}}$

Joint 11 Chord 11 Brace 19 LCMB4

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
19	11	92.7236	0.0127	0.0254	0.4572	1.2192	0.5	0.375	24	0.017735	0.001918	0
										0.095261	0.027851	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
100%	0%	0%	0%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi
Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord LCMB9	-9777.37	253.2917	-257.369	102.6	9.1	-9.2	0.316	0.9283	0.8924	0.9498	3918.7	3918.7	765.5	466.4	0.270	0.055	0.145
Brace LCMB9	-1006.82	40.271	3.2762	56.8	21.0	1.7											

Q,beta	1
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	18.945	18.945	10.525	6.025
K,gap	18.945	18.945	10.525	6.025
T&Y	10.525	10.525	10.525	6.025
X w/o diaphragm	10.525	8.275	10.525	6.025
X w/ diaphragm	10.525	10.525	10.525	6.025
Qu, eff	18.945	18.945	10.525	6.025

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.95	0.95	0.95	0.95

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SAP OUTPUT (1 of 2)

SAP2000 v9.1.8 10/14/05 16:21:45

Table: API Punch Check 1 =- Summary Data - API RP 2A-LRFD 97

Joint Text	Chord Text	Brace Text	Combo Text	Ratio Unitless	Equation Text	ErrMsg Text	WarnMsg Text
47	95	98	LCMB9	1.493885	E.3-4	Section is (	No Messages
51	99	102	LCMB8	1.474344	E.3-4	Section is (	No Messages
24	37	121	LCMB9	1.436689	E.3-4	Section is (	No Messages
9	26	28	LCMB8	1.393915	E.3-4	Section is (	No Messages
29	40	126	LCMB9	1.215053	E.3-4	Section is (	No Messages
42	91	94	LCMB8	1.118269	E.3-4	Section is (	No Messages
28	61	62	LCMB15	1.077876	E.3-4	Section is (	No Messages
45	74	121	LCMB9	1.064954	E.3-4	Section is (	No Messages
32	64	67	LCMB15	0.971299	E.3-4	No Messag	No Messages
13	29	31	LCMB8	0.92005	E.3-2	No Messag	No Messages
44	80	86	LCMB8	0.877395	E.3-2	No Messag	No Messages
52	82	100	LCMB8	0.834474	E.3-2	No Messag	No Messages
49	82	96	LCMB8	0.787558	E.3-2	No Messag	No Messages
50	75	99	LCMB8	0.766797	E.3-2	No Messag	No Messages
19	169	569	LCMB10	0.744906	E.3-2	No Messag	No Messages
27	46	148	LCMB13	0.74315	E.3-2	No Messag	No Messages
12	6	128	LCMB13	0.656936	E.3-2	No Messag	No Messages
26	39	111	LCMB9	0.652985	E.3-2	No Messag	No Messages
11	12	152	LCMB13	0.647966	E.3-2	No Messag	No Messages
34	41	128	LCMB13	0.647842	E.3-2	No Messag	No Messages
33	47	152	LCMB13	0.638865	E.3-2	No Messag	No Messages
39	78	93	LCMB8	0.633665	E.3-2	No Messag	No Messages
25	45	169	LCMB10	0.632067	E.3-2	No Messag	No Messages
2	2	120	LCMB9	0.612367	E.3-2	No Messag	No Messages
7	4	124	LCMB13	0.580451	E.3-2	No Messag	No Messages
6	10	148	LCMB13	0.575423	E.3-2	No Messag	No Messages
40	72	122	LCMB9	0.559077	E.3-2	No Messag	No Messages
30	47	54	LCMB4	0.515079	E.3-2	No Messag	No Messages
31	41	54	LCMB4	0.514048	E.3-2	No Messag	No Messages
22	43	145	LCMB13	0.513634	E.3-2	No Messag	No Messages
53	76	102	LCMB9	0.482533	E.3-2	No Messag	No Messages
15	6	32	LCMB8	0.458918	E.3-2	No Messag	No Messages
14	12	30	LCMB15	0.440887	E.3-2	No Messag	No Messages
23	56	58	LCMB8	0.414361	E.3-2	No Messag	No Messages
18	156	567	LCMB9	0.411305	E.3-2	No Messag	No Messages
4	21	23	LCMB15	0.3923	E.3-2	No Messag	No Messages
1	8	144	LCMB13	0.386522	E.3-2	No Messag	No Messages
205	577	581	LCMB8	0.369572	E.3-2	No Messag	No Messages
21	37	56	LCMB8	0.34137	E.3-2	No Messag	No Messages
20	43	58	LCMB8	0.278427	E.3-2	No Messag	No Messages
46	80	182	LCMB10	0.212392	E.3-2	No Messag	No Messages
43	73	582	LCMB8	0.200141	E.3-2	No Messag	No Messages
41	78	578	LCMB8	0.189576	E.3-2	No Messag	No Messages
64	575	579	LCMB10	0.177534	E.3-2	No Messag	No Messages
8	11	181	LCMB4	0.119309	E.3-4	No Messag	No Messages

SAP OUTPUT (2 of 2)

5	2	579	LCMB4	0.116108	E.3-4	No Messag No Messages
3	9	575	LCMB4	0.115585	E.3-4	No Messag No Messages
48	74	87	LCMB8	0.095506	E.3-2	No Messag No Messages
58	71	159	LCMB7	0.074228	E.3-3 (OPB)	No Messag No Messages
59	42	157	LCMB7	0.074228	E.3-3 (OPB)	No Messag No Messages
63	155	158	LCMB7	0.074228	E.3-3 (OPB)	No Messag No Messages
60	154	160	LCMB7	0.074225	E.3-3 (OPB)	No Messag No Messages
61	77	158	LCMB7	0.074225	E.3-3 (OPB)	No Messag No Messages
10	4	112	LCMB13	0.061267	E.3-4	No Messag No Messages
62	160	167	LCMB7	0.016176	E.3-3 (OPB)	No Messag No Messages
16	161	35	LCMB13	1.05E-16	E.3-3 (OPB)	No Messag No Messages
35	166	70	LCMB13	1.05E-16	E.3-3 (OPB)	No Messag No Messages
54	165	105	LCMB13	1.05E-16	E.3-3 (OPB)	No Messag No Messages
17	162	35	LCMB13	9.03E-17	E.3-3 (OPB)	No Messag No Messages
36	163	70	LCMB13	9.03E-17	E.3-3 (OPB)	No Messag No Messages
55	164	105	LCMB13	9.03E-17	E.3-3 (OPB)	No Messag No Messages

Joint ID	Chord ID	Brace ID	Load Comb	Chord T (m)	Utilization Ratio	Controlling Equation	Chord T (m)	Utilization Ratio	Controlling Equation	Chord T (m)	Utilization Ratio	Controlling Equation
47	95	98	LCMB9	0.0127	1.494	E.3-4	0.015875	0.809	E.3-2			
51	99	102	LCMB8	0.0127	1.474	E.3-4	0.015875	0.768	E.3-2			
24	37	121	LCMB9	0.0254	1.437	E.3-4	0.028575	1.024	E.3-4	0.03175	0.806	E.3-2
9	26	28	LCMB8	0.0127	1.394	E.3-4	0.015875	0.770	E.3-2			
29	40	126	LCMB9	0.0254	1.215	E.3-4	0.028575	0.876	E.3-2			
42	91	94	LCMB8	0.01905	1.118	E.3-4	0.022225	0.769	E.3-2			
28	61	62	LCMB15	0.0127	1.078	E.3-4	0.015875	0.651	E.3-2			
45	74	121	LCMB9	0.0254	1.065	E.3-4	0.028575	0.784	E.3-2			

Joint 47 Chord 95 Brace 98 LCMB9

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
98	95	75.715	0.0127	0.015875	0.6096	0.6096	0.8	1	19.2	0.023815	0.003481	0
										0.029611	0.004284	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	0%	100%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi
Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj: Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord LCMB9	-410.114	9.2398	24.9141	13.9	2.2	5.8	0.046	0.9988	0.9981	0.9991	2007.2	2640.0	978.3	816.7	0.809	0.053	0.761
Brace LCMB9	-2029.82	49.5404	-13.9244	85.2	14.2	-4.0											

Q,beta	1.796407
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	40.32	40.32	22.4	18.68263
K,gap	40.32	40.32	22.4	18.68263
T&Y	22.4	22.4	22.4	18.68263
X w/o diaphragm	22.4	29.46108	22.4	18.68263
X w/ diaphragm	22.4	22.4	22.4	18.68263
Qu, eff	22.4	29.46108	22.4	18.68263

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

Joint 51 Chord 99 Brace 102 LCMB8

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
102	99	80.594	0.0127	0.015875	0.4572	0.4572	0.8	1	14.4	0.017735	0.001918	0
										0.02201	0.002347	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	0%	100%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi
Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB8	-2598.78	12.8127	-9.7637	118.1	5.5	-4.2	0.361	0.9437	0.9156	0.9606	1863.1	2450.4	661.1	578.5	**0.768**	**0.035**	**0.681**
Brace																	
LCMB8	1287.642	21.8517	7.0048	72.6	11.4	3.7											

Q,beta	1.796407
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	40.32	40.32	22.4	18.68263
K,gap	40.32	40.32	22.4	18.68263
T&Y	22.4	22.4	22.4	18.68263
X w/o diaphragm	22.4	29.46108	22.4	18.68263
X w/ diaphragm	22.4	22.4	22.4	18.68263
Qu, eff	22.4	29.46108	22.4	18.68263

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

Joint 24 Chord 37 Brace 121 LCMB9

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
121	37	53.591	0.01905	0.03175	0.762	1.2192	0.6	0.625	19.2	0.044464	0.008057	0
										0.118443	0.03427	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	0%	100%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord LCMB9	-263.773	-195.272	12.5572	2.2	-5.7	0.4	0.019	0.9998	0.9997	0.9999	6599.5	4985.8	4022.7	2050.5	**0.806**	**0.024**	**0.726**
Brace LCMB9	-3818.52	93.0967	14.0547	85.9	11.6	1.7											

Q,beta	1.001304
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	27.495	27.495	15.275	7.785137
K,gap	27.495	27.495	15.275	7.785137
T&Y	15.275	15.275	15.275	7.785137
X w/o diaphragm	15.275	11.54003	15.275	7.785137
X w/ diaphragm	15.275	15.275	15.275	7.785137
Qu, eff	15.275	11.54003	15.275	7.785137

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

Joint 9 Chord 26 Brace 28 LCMB8

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
28	26	75.715	0.0127	0.015875	0.6096	0.6096	0.8	1	19.2	0.023815	0.003481	0
										0.029611	0.004264	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	0%	100%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	flpb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB8	112.033	-45.9032	1.5245	3.8	-10.7	0.4	0.035	1.0000	1.0000	1.0000	2009.7	2643.2	980.1	817.4	**0.770**	**0.049**	**0.696**
Brace																	
LCMB8	-1933.95	45.4534	6.3011	81.2	13.1	1.8											

Q,beta	1.796407
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	40.32	40.32	22.4	18.68263
K,gap	40.32	40.32	22.4	18.68263
T&Y	22.4	22.4	22.4	18.68263
X w/o diaphragm	22.4	29.46108	22.4	18.68263
X w/ diaphragm	22.4	22.4	22.4	18.68263
Qu, eff	22.4	29.46108	22.4	18.68263

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

Joint 29 Chord 40 Brace 126 LCMB9

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
126	40	52.709	0.0127	0.028575	0.6096	1.2192	0.444444	0.5	21.33333	0.023815	0.003481	0
										0.106884	0.031087	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	0%	100%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB9	-7638.56	3.9101	207.7542	71.5	0.1	6.7	0.219	0.9693	0.9540	0.9785	4427.6	3397.9	2125.0	1165.9	0.876	0.016	0.824
Brace																	
LCMB9	-2828.09	31.4256	-7.8711	118.8	9.0	-2.3											

Q,beta	1
Q,g	1.6

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	23.22	23.22	12.9	6.9
K,gap	23.22	23.22	12.9	6.9
T&Y	12.9	12.9	12.9	6.9
X w/o diaphragm	12.9	9.9	12.9	6.9
X w/ diaphragm	12.9	12.9	12.9	6.9
Qu, eff	12.9	9.9	12.9	6.9

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

Joint 42 Chord 91 Brace 94 LCMB8

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
94	91	77.533	0.01905	0.022225	0.762	0.762	0.857143	1	17.14286	0.044464	0.008057	0
										0.051652	0.009283	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	0%	100%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB8	2271.426	4.4186	17.5118	44.0	0.5	1.9	0.134	1.0000	1.0000	1.0000	3909.4	5141.8	2383.2	1987.7	0.769	-0.007	0.678
Brace																	
LCMB8	-3758.57	-71.3368	-13.7367	84.5	-8.9	-1.7											

Q,beta	1.796407
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	40.32	40.32	22.4	18.68263
K,gap	40.32	40.32	22.4	18.68263
T&Y	22.4	22.4	22.4	18.68263
X w/o diaphragm	22.4	29.46108	22.4	18.68263
X w/ diaphragm	22.4	22.4	22.4	18.68263
Qu, eff	22.4	29.46108	22.4	18.68263

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

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Joint 28 Chord 61 Brace 62 LCMB15

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
62	61	75.728	0.0127	0.015875	0.6096	0.6096	0.8	1	19.2	0.023815	0.003481	0
										0.029611	0.004284	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	0%	100%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj: Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord LCMB15	-1843.51	-51.5817	3.497	62.3	-12.0	0.8	0.193	0.9784	0.9677	0.9849	1966.3	2586.1	948.3	805.1	0.651	0.009	0.490
Brace LCMB15	-1600.3	7.9091	-4.0772	67.2	2.3	-1.2											

Q,beta	1.796407
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	40.32	40.32	22.4	18.68263
K,gap	40.32	40.32	22.4	18.68263
T&Y	22.4	22.4	22.4	18.68263
X w/o diaphragm	22.4	29.46108	22.4	18.68263
X w/ diaphragm	22.4	22.4	22.4	18.68263
Qu, eff	22.4	29.46108	22.4	18.68263

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

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Joint 45 Chord 74 Brace 121 LCMB9

BRACE	CHORD	θ	t	T	d	D	τ	β	γ	A	S	gap
121	74	53.591	0.01905	0.028575	0.762	1.2192	0.666667	0.625	21.33333	0.044464	0.008057	0
										0.106884	0.031087	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	0%	100%	0%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj: Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord LCMB9	-9046.57	-1075.93	272.8914	84.6	-34.6	8.8	0.280	0.9497	0.9246	0.9648	5077.9	5077.9	3013.5	1602.7	0.784	0.033	0.699
Brace LCMB9	-3780.77	93.363	-0.6088	85.0	11.6	-0.1											

Q,beta	1.001304
Q,g	1.8

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	27.495	27.495	15.275	7.785137
K,gap	27.495	27.495	15.275	7.785137
T&Y	15.275	15.275	15.275	7.785137
X w/o diaphragm	15.275	11.54003	15.275	7.785137
X w/ diaphragm	15.275	15.275	15.275	7.785137
Qu, eff	15.275	15.275	15.275	7.785137

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.9	0.95	0.95	0.95

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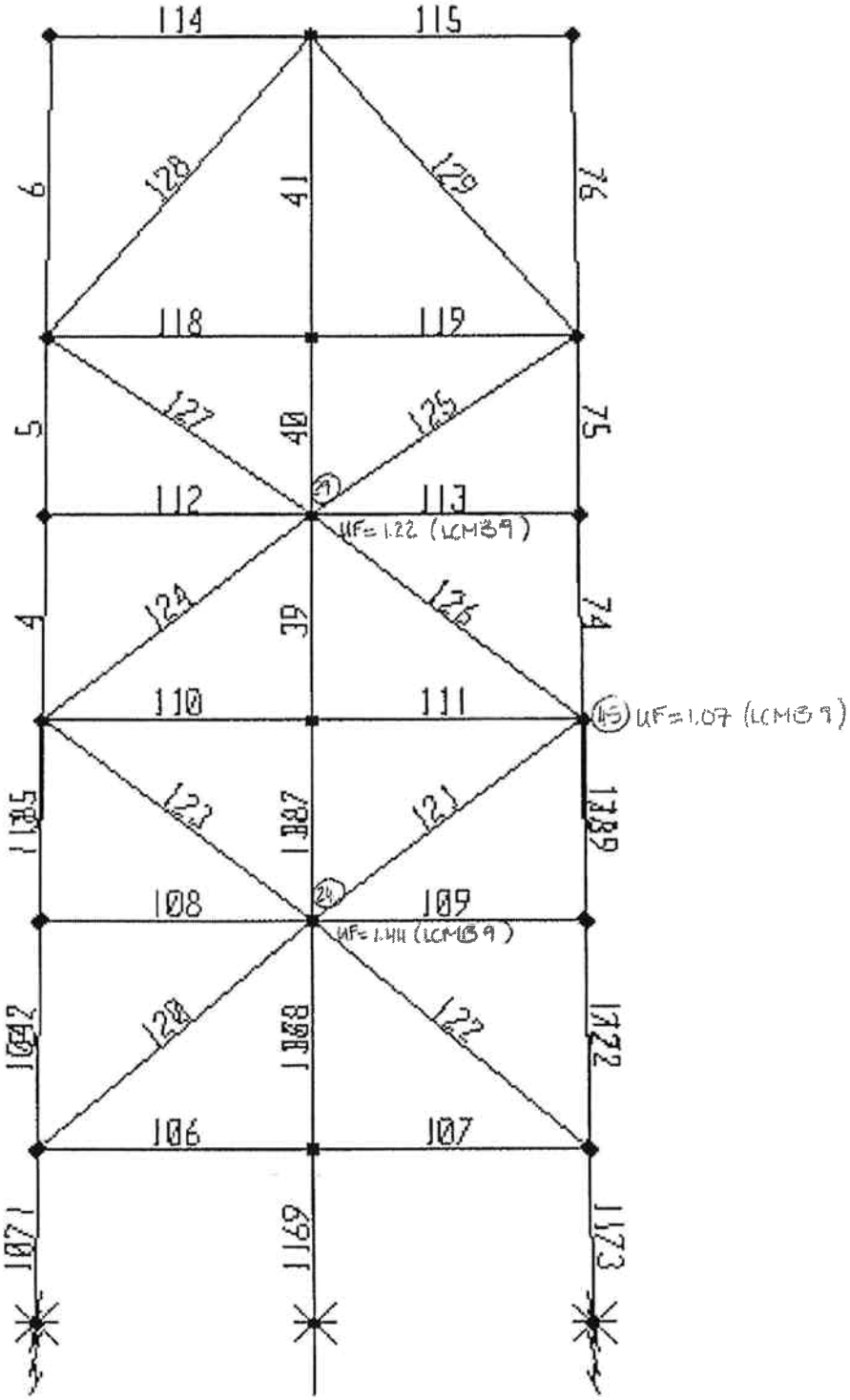
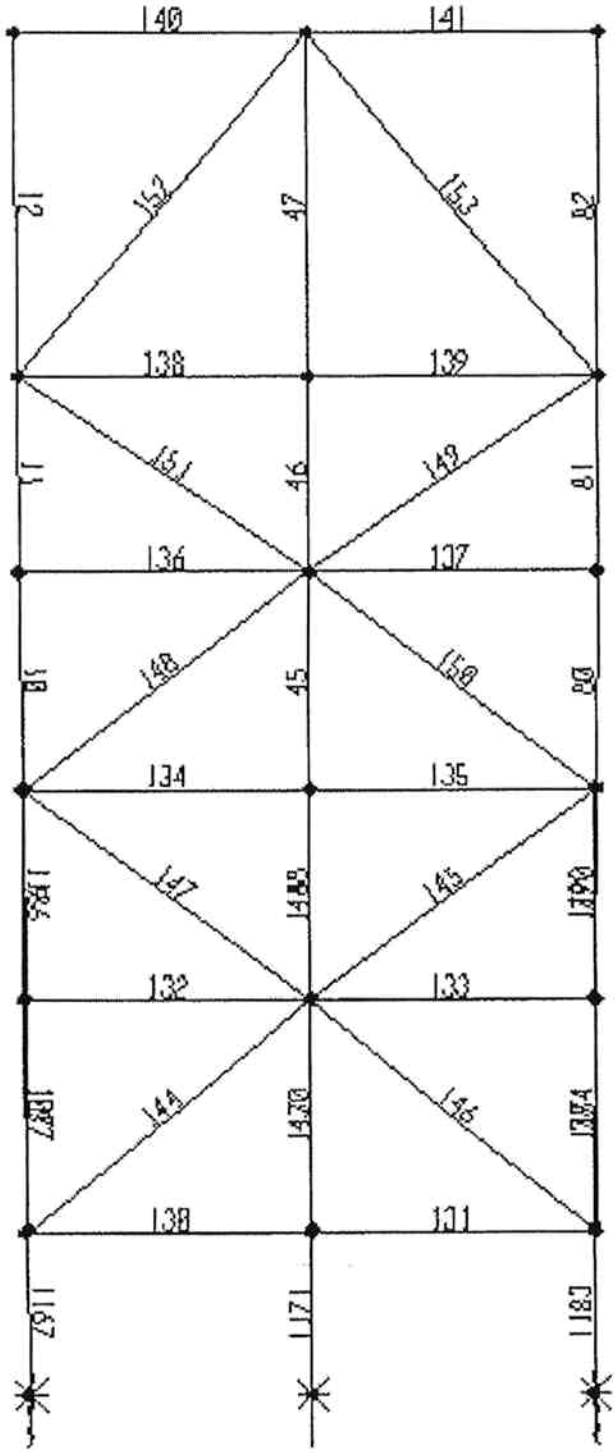
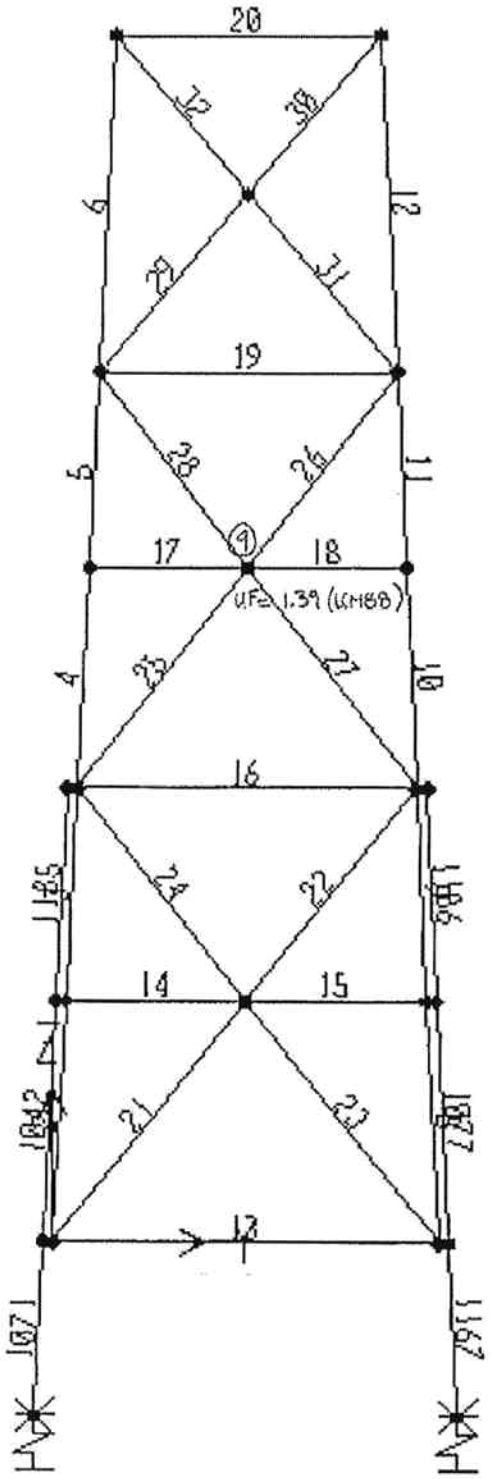


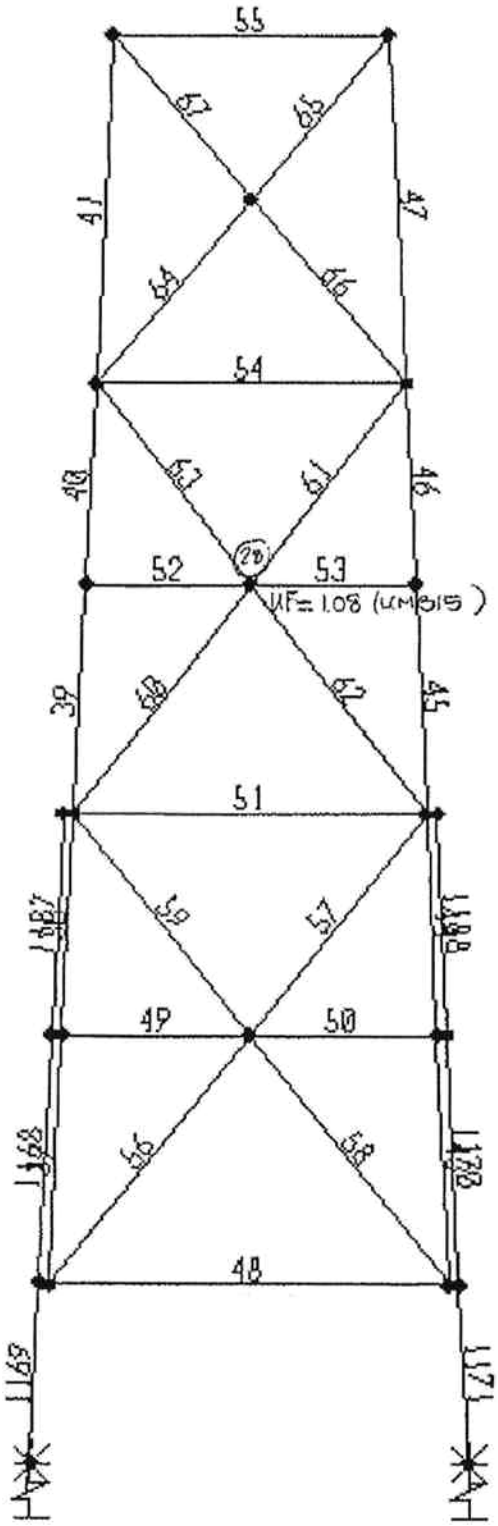
ELEVATION - FRAME A



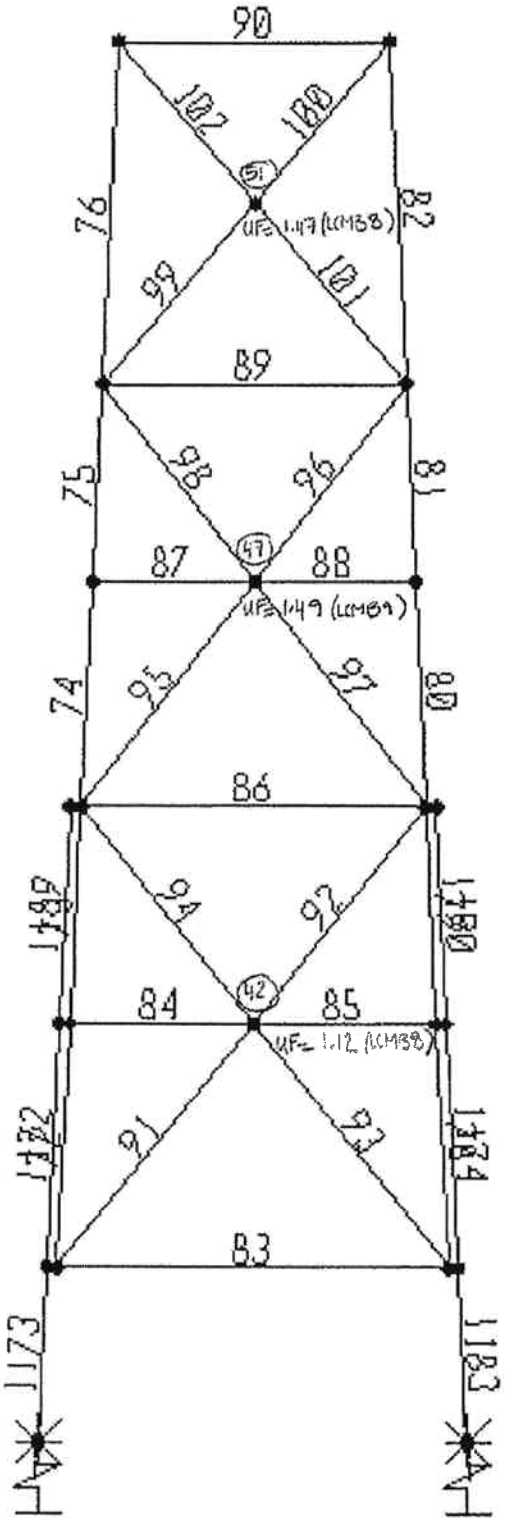
ELEVATION - FRAME B



ELEVATION - FRAME 1



ELEVATION - FRAME 2



ELEVATION - FRAME 3

Joint Capacity Check

CHORD D	T	BRACE d	t	θ	τ	β	γ	A	S	gap
1.219	0.050	0.610	0.012	50.0	0.24	0.500	12.19	0.0225	0.0033	2
								0.1836	0.0516	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	100%	0%	0%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Note: This uses an equivalent section due to presence of pile in leg.

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj: Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB9	(8,000)	800	400	43.6	15.5	7.8	0.143	0.9925	0.9888	0.9948	14417.3	14417.3	7004.5	3769.2	**0.567**	**0.047**	**0.44**
Brace																	
LCMB9	7,773	310.90	155	345.0	94.2	47.1											

Q,beta	1
Q,g	1

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	12.90156	12.90156	12.90156	6.90
K,gap	12.90156	12.90156	12.90156	6.90
T&Y	12.90156	12.90156	12.90156	6.90
X w/o diaphragm	12.90156	9.90	12.90156	6.90
X w/ diaphragm	12.90156	12.90156	12.90156	6.90
Qu, eff	12.90156	12.90	12.90156	6.90

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.95	0.95	0.95	0.95

MEMBER CAPACITY JOINT CHECKS

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Joint Capacity Check

CHORD D	T	BRACE d	t	θ	τ	β	γ	A	S	gap
1.067	0.032	0.610	0.012	30.0	0.375	0.571	16.67188	0.0225	0.0033	2
								0.1040	0.0261	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	100%	0%	0%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB9	(8,000)	800	400	76.9	30.6	15.3	0.257	0.9670	0.9505	0.9769	9740.0	9740.0	4669.0	2490.7	**0.840**	**0.070**	**0.85**
Brace																	
LCMB9	7,773	310.90	155	345.0	94.2	47.1											

Q,beta	1
Q,g	1

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	14.25511	14.25511	14.25511	7.40
K,gap	14.25511	14.25511	14.25511	7.40
T&Y	14.25511	14.25511	14.25511	7.40
X w/o diaphragm	14.25511	10.83	14.25511	7.40
X w/ diaphragm	14.25511	14.25511	14.25511	7.40
Qu, eff	14.25511	14.26	14.25511	7.40

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.95	0.95	0.95	0.95

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Joint Capacity Check

CHORD D	T	BRACE d	t	θ	τ	β	γ	A	S	gap
1.067	0.032	0.610	0.009	40.0	0.28125	0.571	16.67188	0.0170	0.0025	2
								0.1040	0.0261	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	100%	0%	0%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi

Fy	345	MPa	Phi,q	0.95

Capacities, Puj, Muj

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB9	(8,000)	800	400	76.9	30.6	15.3	0.257	0.9670	0.9505	0.9769	7576.3	7576.3	3631.8	1937.5	0.814	0.068	0.81
Brace																	
LCMB9	5,859	234.35	117	345.0	93.3	46.6											

Q,beta	1
Q,g	1

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	14.25511	14.25511	14.25511	7.40
K,gap	14.25511	14.25511	14.25511	7.40
T&Y	14.25511	14.25511	14.25511	7.40
X w/o diaphragm	14.25511	10.83	14.25511	7.40
X w/ diaphragm	14.25511	14.25511	14.25511	7.40
Qu, eff	14.25511	14.26	14.25511	7.40

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.95	0.95	0.95	0.95

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Joint Capacity Check

CHORD D	T	BRACE d	t	θ	τ	β	γ	A	S	gap
1.067	0.035	0.457	0.012	40.0	0.342857	0.428	15.24286	0.0168	0.0018	2
								0.1135	0.0283	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	100%	0%	0%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi
Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj: Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB9	(8,000)	800	400	70.5	28.2	14.1	0.236	0.9746	0.9619	0.9822	7393.3	7393.3	2667.8	1510.6	0.824	0.091	0.85
Brace																	
LCMB9	5,788	231.51	116	345.0	127.3	63.6											

Q,beta	1
Q,g	1

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	11.53777	11.53777	11.53777	6.40
K,gap	11.53777	11.53777	11.53777	6.40
T&Y	11.53777	11.53777	11.53777	6.40
X w/o diaphragm	11.53777	8.97	11.53777	6.40
X w/ diaphragm	11.53777	11.53777	11.53777	6.40
Qu, eff	11.53777	11.54	11.53777	6.40

Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.95	0.95	0.95	0.95

Joint Capacity Check

CHORD D	T	BRACE d	t	θ	τ	β	γ	A	S	gap
1.067	0.029	0.457	0.009	40.0	0.310345	0.428	18.39655	0.0127	0.0014	2
								0.0946	0.0239	

K, lap	K,gap	T&Y	X w/o	X w/ diaphragm
0%	100%	0%	0%	0%

Fy	345.0	MPa	50.0	ksi
Fu	517.5	MPa	75.1	ksi
Fy	345	MPa	Phi,q	0.95

	Pax (kN)	Mipb=M3 (kN-m)	Mopb=M2 (kN-m)	fax (MPa)	fipb (MPa)	fopb (MPa)	A	Qf(ax)	Qf(ipb)	Qf(opb)	Capacities, Puj, Muj: Puj, tens (kN)	Puj,compr (kN)	Muj,ipb (kN-m)	Muj,opb (kN-m)	EQN 3-2	EQN 3-3	EQN 3-4
Chord																	
LCMB9	(8,000)	800	400	84.6	33.5	16.7	0.282	0.9560	0.9340	0.9692	4979.0	4979.0	1778.5	1023.4	0.924	0.103	1.02
Brace																	
LCMB9	4,370	174.80	87	345.0	125.6	62.8											

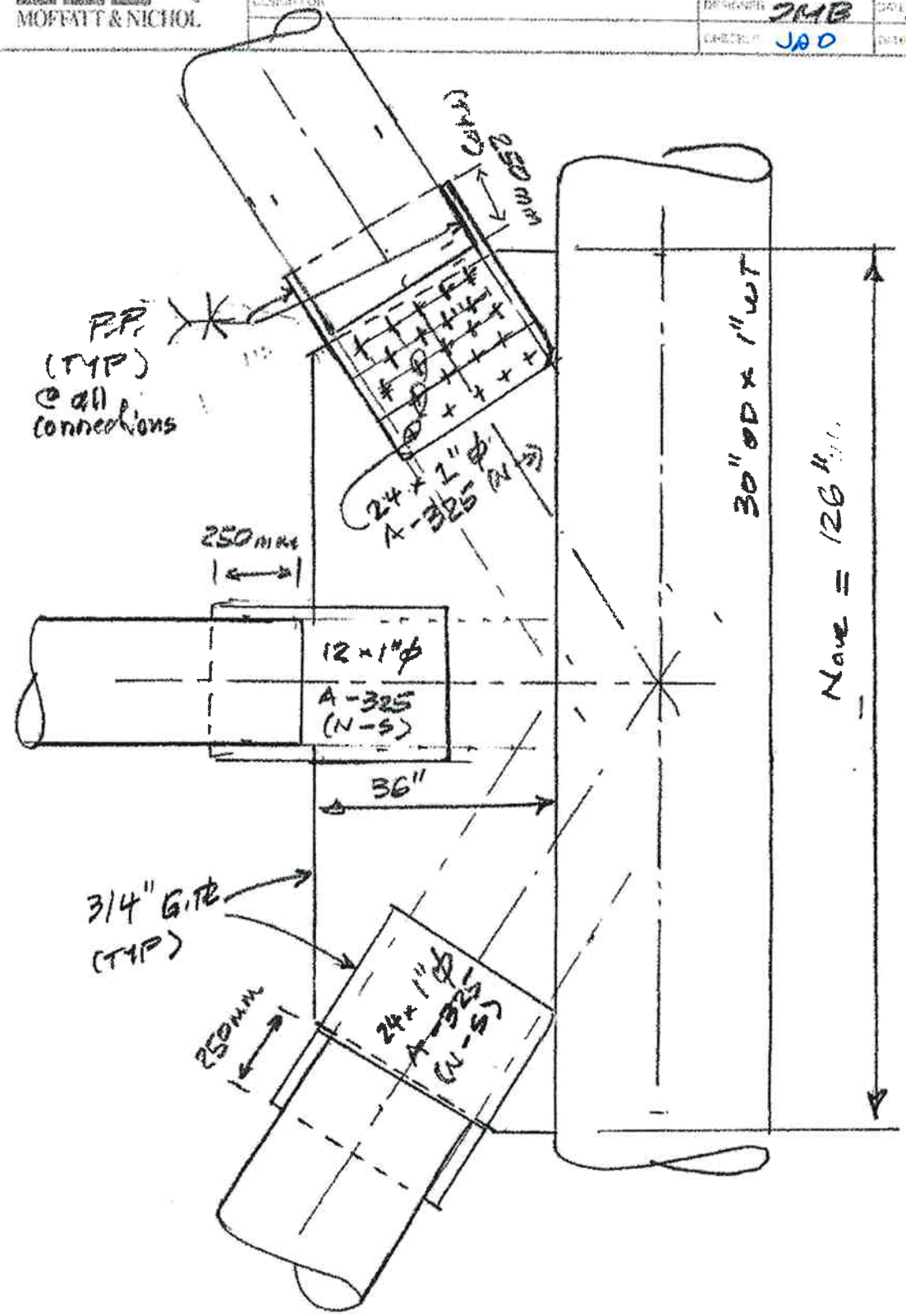
Q,beta	1
Q,g	1

Qu	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	11.53777	11.53777	11.53777	6.40
K,gap	11.53777	11.53777	11.53777	6.40
T&Y	11.53777	11.53777	11.53777	6.40
X w/o diaphragm	11.53777	8.97	11.53777	6.40
X w/ diaphragm	11.53777	11.53777	11.53777	6.40
Qu, eff	11.53777	11.54	11.53777	6.40

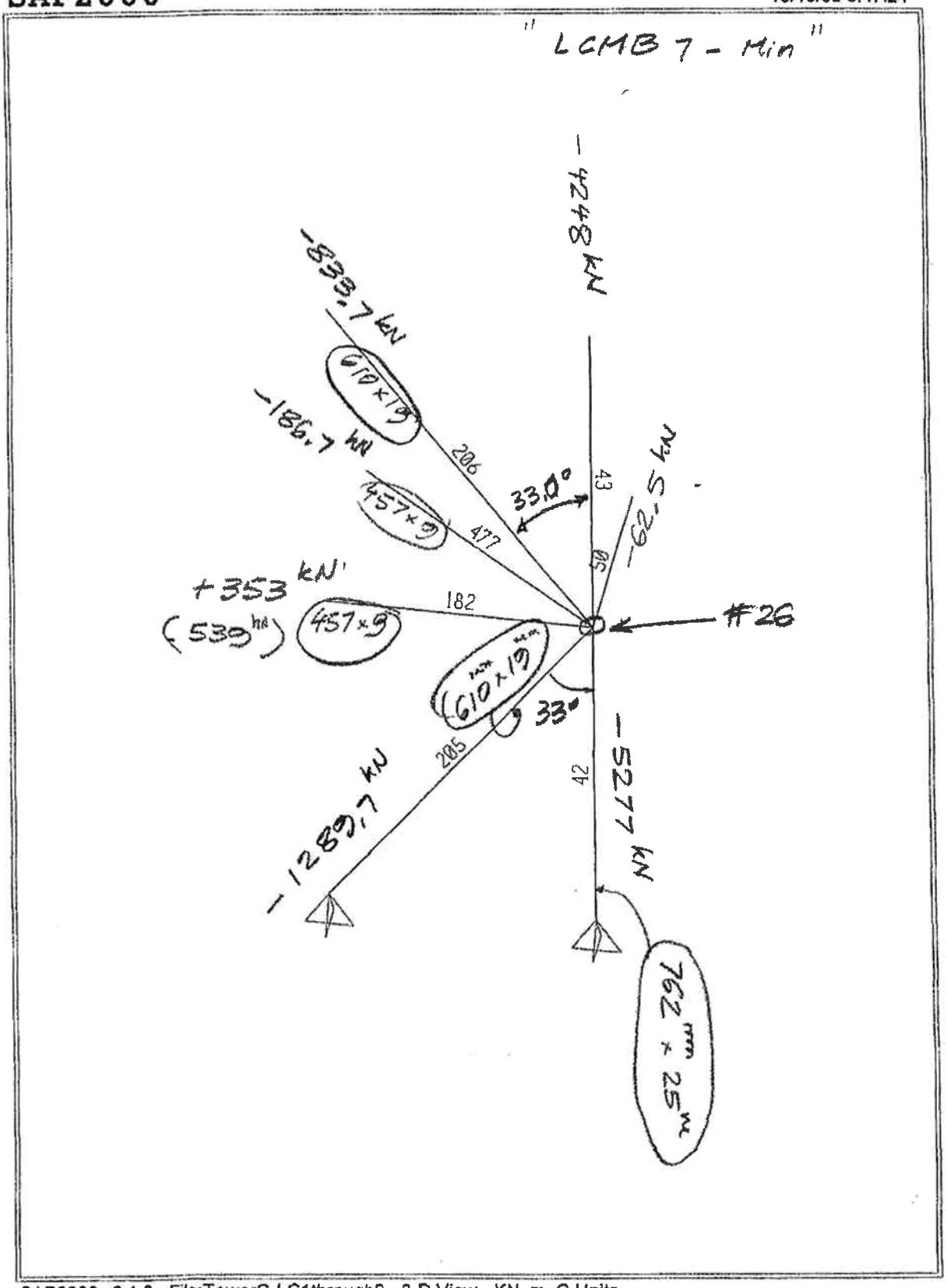
Phi,j	Axial Tension	Axial Compr	Bending ipb	Bending opb
K, lap	0.95	0.95	0.95	0.95
K,gap	0.95	0.95	0.95	0.95
T&Y	0.90	0.95	0.95	0.95
X w/o diaphragm	0.90	0.95	0.95	0.95
X w/ diaphragm	0.90	0.95	0.95	0.95
Phi,j, eff	0.95	0.95	0.95	0.95

MOFFATT & NICHOL

CLIENT	TEMP. TOWER	JOB NO.	4141-05
PROJECT	JOINT #26	SHEET	9
LOCATION		DESIGNED	JMB
		CHECKED	JAO
		DATE	10-19-05
		DATE	28 OCT 05

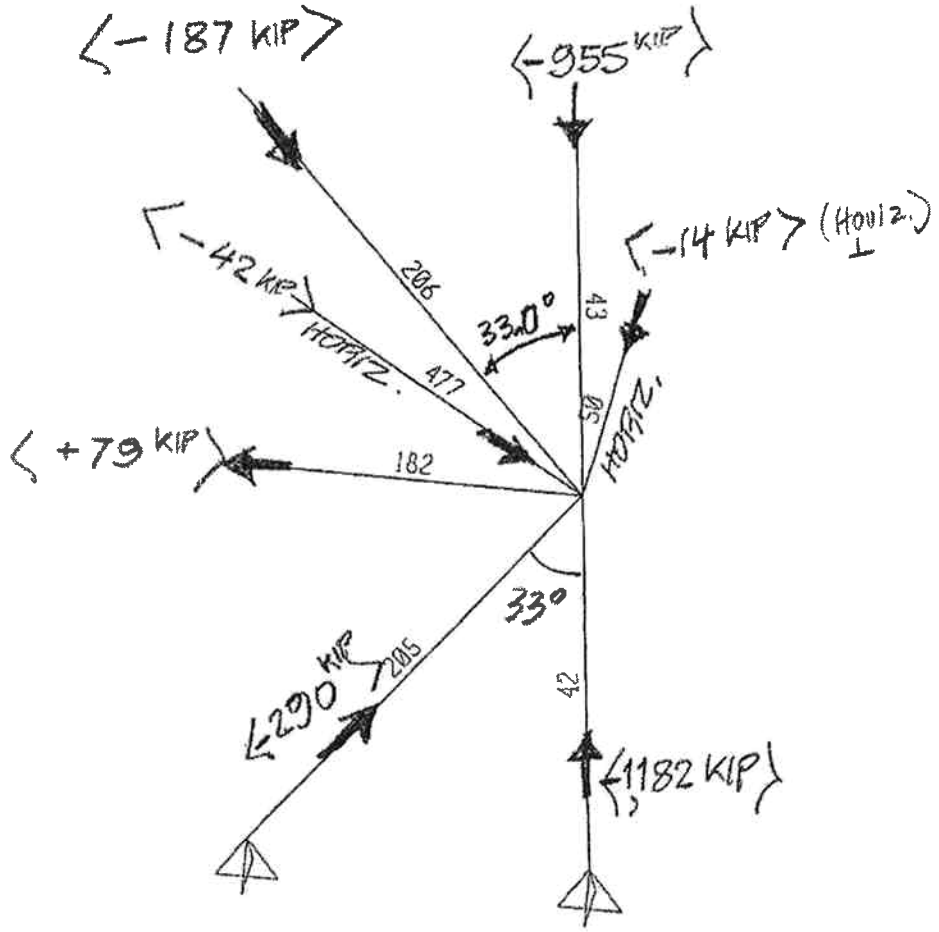


TYPICAL BOLTED JOINT DESIGN



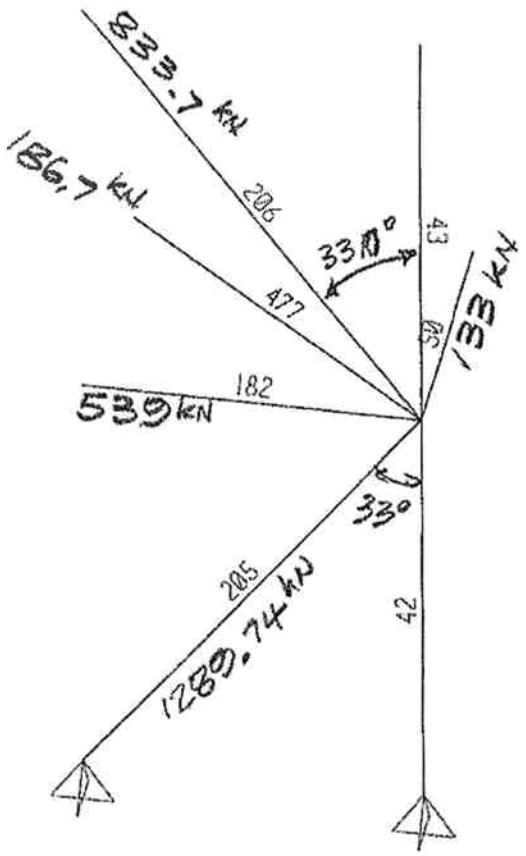
SAP2000 v9.1.8 - File:TowerC-LC1through9 - 3-D View - KN, m, C Units

"LCMB 7 - Min"
Ult. Member Demands in KIPS



SAP2000 v9.1.8 - File:TowerC-LC1through9 - 3-D View - KN, m, C Units

MAXIMUM MEMBER AXIAL ULT. DEMAND



SAP2000 v9.1.8 - File:TowerC-LC1through9 - 3-D View - KN, m, C Units

MOFFATT & NICHOL	TEMP. TOWER C	4141-05
	JOINT # 26	/
		DMB 10-20-05
		JAD 28OCT05

TEMPORARY TOWER "C"

JOINT #26 : A-572 Steel

$F_y = 50$ KSI
$F_u \geq 65$ KSI

"K" JOINT IN VERTICAL PLANE

Provide typical gusset plate $t_{gus.} = 20$mm

1. BOLTS

Provide A-325 1"ϕ HS BOLTS

1"ϕ A-325 BOLTS N-S

ϕR_{nv} (N-S) = 28.3 KIP/BOLT
= 125.9 kN/BOLT

Member	COMBT-No. Bolt (KIP)	n required 1"ϕ A-325 N-S	n provided for DEMAND
205	-290 <-1289.7 kN>	11	12
206	-187 <-833.7 kN>	7	8
182	+79 (+121) MAX <539 kN>	3 (4)	4
50	-14 (-30) MAX <133 kN>	1 (2)	4
477	-42 <-186.7 kN>	2	4

2 GUSSET PLATES: USE 3/4" TYP. → 19mm

BOLT BEARING: $\phi R_{nb} = \phi 2.4 F_u d t_{GPL}$

F_u (Gusset PL) = 65 KSI for A-572

$\phi R_n = (0.75)(2.4)(65)(1.0)(.75)$
$= 87.75$ KIP/BOLT (does not govern)

MORATT & NICOL	TEMP TOWER C	9141-05
	JOINT #26	2
		JMB 10-20-05
		JAD 28 OCT 05

API "CONNECTOR" REQUIREMENT:

Provide 50% of member capacity

Member capacities are computed according to API LRFD

Tension: $P_a^t = \phi_t F_y A_o$ (D.2.1-1)

Compression: $P_a^c = \phi_c F_{cr} A_o$ (D.2.2-1)

See EXCEL spreadsheet for results

NOTE

Engineering Judgement is used to choose between compression capacity vs. tension capacity with respect to bolts and welds.

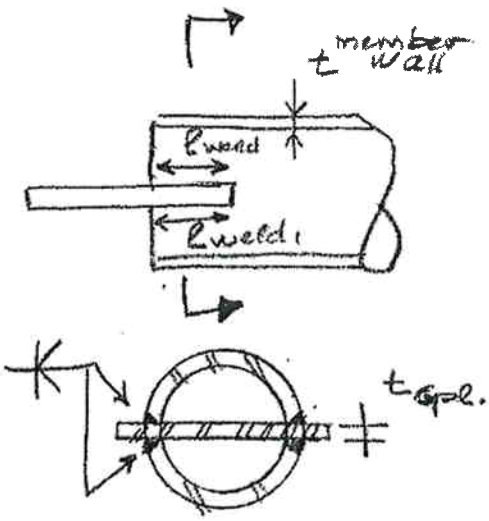
Given the very low demand in tension the 50% compression capacity limit is applied to establish the "minimum" required number of 1" Φ A-325 (N-S) HS Bolts per member connection as well as the weld length.

WELD

The weld of the member ~~gusset~~ plate connection is designed as partial penetration "pre-approved" groove weld with an effective weld "throat" / "thickness" larger than 70% of the member pipe wall thickness.

MERRITT & NICHOL	TEMP TOWER C	414-06	
	POINT #26	3	
		JMB	6-20-05
		JAD	28 OCT 05

$$\phi R_{nw} = \phi\, t_{weld}^{eff} (0.6 F_{Exx})\, \ell_{weld}$$



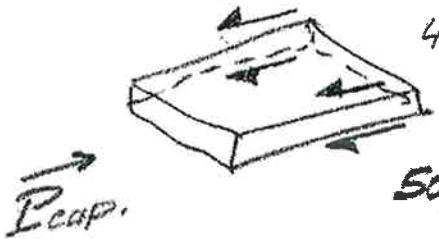
$\phi = 0.80$

$t_{weld}^{eff} = 0.70\, t_{member\ wall}$

$F_{Exx} = 70\ ksi \rightarrow 483\ MPa$

ℓ_{weld} = length of "partial" penetration groove weld on each side.

The Excel spreadsheet computes ℓ_{weld} on the basis of the 50% member capacity retained (comp.).



4 sets of equal "weld transmitting forces".

$$50\%\, P_{cap} \geq 4\phi R_n$$

$$\hookrightarrow \quad \ell_{weld} = \frac{(50\%\, P_{cap})}{4\,\phi\, t_w^{eff} (0.6 F_{Exx})}$$

MOFFATT & NICHOL

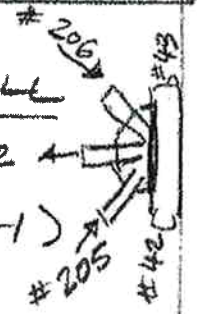
Subject:	TEMP TOWER C	Job No.	4141-05
Project:	JOINT #26	Sheet No.	4
Calculation No.		Computed by:	JMB 10-20-05
		Checked by:	JAO 28 OCT 05

connector Summary:

$t_{GUSSET} = 19$ mm
Bearing OK

Member		A-325 1"φ N Bolts PROVIDED	ℓ welds (mm) P.P. ($0.7t^m$) PROVIDED	ℓ welds (mm) min. required
diags	205	24	250	225.8
	206	24	250	"
horiz ty.	182	12	250	241.8
hor. long.	50	12	250	214.38
hor. diag	477	12	250	138.42
leg	42	Simplify: 2 sets of fasteners	simplify: 1 length only.	
	43			

GUSSET PL ACTION ON MAIN LEG ("chord")

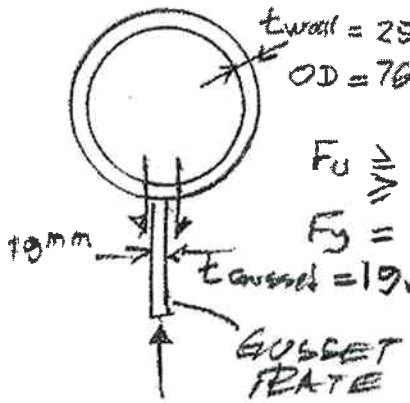


PUNCHING SHEAR THROUGH LEG WALL

(AISC for HSS - Nov 10 - 2000)

$$f\,t_{p\,GUSSET} \overset{?}{<} 2\,(0.6F_u\,t_{WALL}) \qquad (C9\text{-}B\text{-}1)$$

$f_{max\,gusset} \simeq F_y = 50$ ksi $= 345$ MPa



$F_u \geq 65$ ksi (≥ 450 MPa)
$F_y = 50$ ksi (345 MPa)

$$F_y\,t_p \overset{?}{<} 2\,(0.6F_u\,t_{wall})$$

$$t_p \overset{?}{<} 2\,(0.6\,\frac{F_u}{F_y}\,t_{wall})$$

$$t_p \overset{?}{<} (2)(0.6)\left(\frac{450}{345}\right)(25)$$

$$\underbrace{t_p}_{19^{mm}} \overset{?}{<} 39\text{ mm}$$

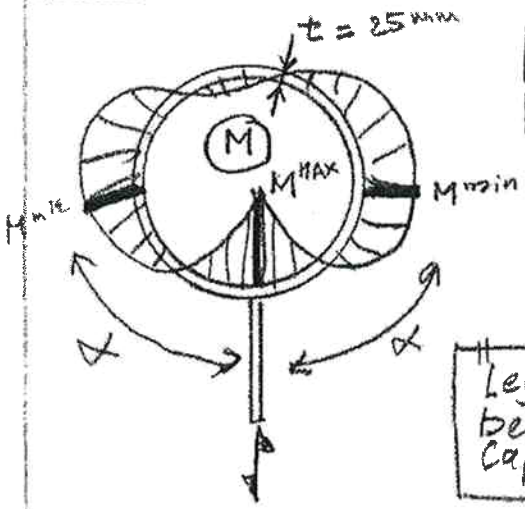
$$19^{mm} < 39^{mm} \quad \boxed{OK}$$

$$(C9.3\text{-}2) \rightarrow t_p < \frac{F_u}{F_{yp}}\,t = \frac{(450)}{(345)}(25) = 32.6^{mm} \quad \boxed{OK}$$

MOFFATT & NICHOL	TEMP TOWER C JOINT #26	4141-05 5
		JHB 10-20-05 JAO 28 OCT 05

treated as "T" "K joint"

JOINT #26 LEG WALL LOCAL BENDING @ K joint



$|M^{max}| \cong 0.24\,pR$ @ load application location

$|M^{min}| = -0.103\,pR$ @ $\alpha \cong 75°$

$\begin{cases} t = 25\text{ mm} \\ R = D/2 = \frac{1}{2}(762-25) \cong 369\text{ mm} \end{cases}$

$|M| \overset{?}{<} \phi_b M_n$ (wall)

$\phi_b = 0.95$

"Leg" wall bending capacity $\rightarrow M_n = F_y S = F_y \frac{t^2}{6}$ (MPa/m)

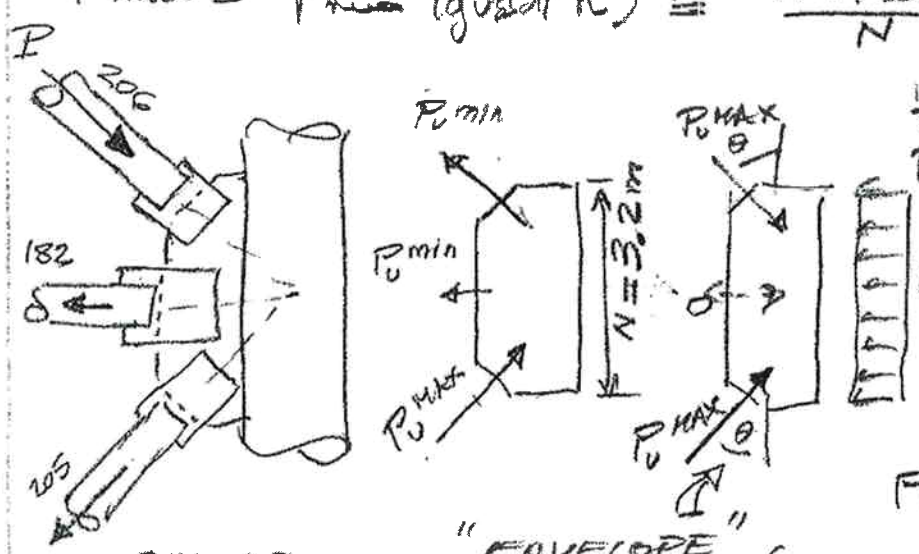
$= (345)\left(\frac{25^2}{6}\right)(10^{-6})$ (MPa)

$\cong 35.9 \times 10^{-3}$ kNm/m

$\boxed{\phi M_n = (.75)(35.9) = 34.1 \frac{kNm}{m}}$

$|M^{ult}|_{Demand} = 0.24\, P^{ult}_{max} R$

$P_{max} = P^{ult}_{max}$ (gusset PL) $\cong \frac{\Sigma P_i \text{ horiz. comp.}}{N}$



$\theta \cong 33°$

$P_U^{MAX}(206) = 0.834$ MN

$P_U^{max}(205) = 1.29$ MN

$\Sigma P_H = \Sigma P \sin\theta$

$\Sigma P_H = (0.834 + 1.29)\sin 33°$

$\Sigma P_H = 1.157$ MN

$P_U^{max} = \frac{\Sigma P_H}{N} = \frac{1.157}{3.2}$

EQUILIBRIUM CONFIGURATION

"ENVELOPE" LOADING (non-equilibrium situation with max P_{gusset})

$$P_u^{max} = \frac{1.157}{3.2} \cong 0.362 \text{ MN/m}$$

$$\boxed{\left|M^{max}_{\substack{in\\ \text{wall of}\\ \text{Leg}}}\right| \cong 0.24\, P_u^{max} R = (0.24)(0.362)(0.369) \cong 0.0321 \text{ MN m/m}}$$

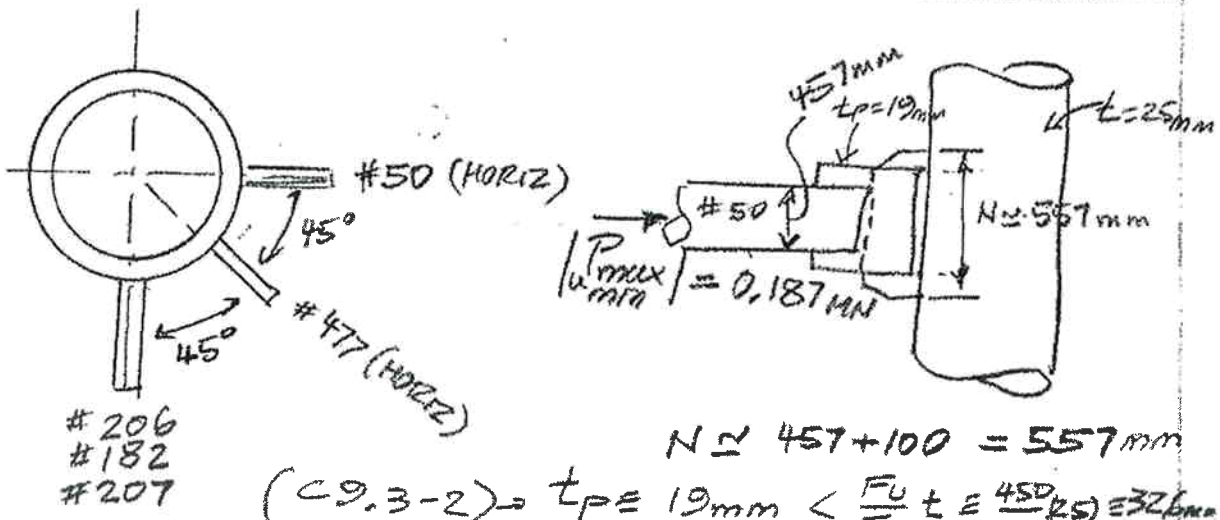
Leg wall ult. Bending capacity check:

$$\left|M^{max}\right| \cong 0.0321 \text{ MNm/m} < \phi_b M_n \cong 0.034 \text{ MNm/m}$$

OK

NOTE: The load configuration considered treats the wall bending as in a "T" joint which is much more conservative.

PUNCHING SHEAR THROUGH LEG WALL @ 45° (Point 26) - Member #477



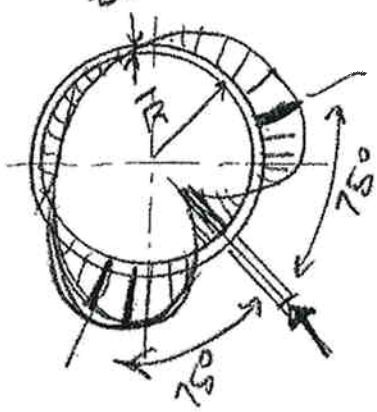
$N \cong 457 + 100 = 557$ mm

$(C9.3\text{-}2) \rightarrow t_p \cong 19\text{mm} < \frac{F_u}{F_{yp}} t \cong \frac{450}{345}(25) \cong 32\text{mm}$ OK

JOINT #26 LEG WALL LOCAL BENDING @ MEMBER #477 GUSSET PL CONNECTION

$\bar{R} = 0.369\,m$

$t = 25\,mm$



$|M_{wall}^{max}|$ @ Load application location

$|M_{wall}^{max}| = 0.24\,pR$

$P = P_u^{max}$

$P_u^{max} = \dfrac{P_{u\,\#477}^{max}}{N}$

$P_{u\,\#477}^{max} = 0.187\ MN$

$P_u^{max} = \dfrac{0.187}{0.557} = 0.337\ MN/m$

$|M_{u\,wall}^{max}| = (0.24)(0.337)(0.369) = 0.0298\ MNm/m$

$\phi M_{n\,wall}^{leg} = 0.0341\ MNm/m$

Check: $|M^{max}| = 0.0298\ MNm/m < \phi M_n \equiv 0.0341\ MNm/m$

$\boxed{OK}$

JOINT #26 @ CONNECTION WITH MEMBER #50
(SAME GUSSET PL GEOMETRY AS FOR #477)

Punching Shear OK (same gusset $t_p < 32.6\,mm$ 18 mm)

Local Leg Wall Bending

$|P_{u\,\#50}^{max}| \equiv 0.133\ MN \rightarrow |M_{wall}^{max}| = 0.24\,p_u R$

$P_u = \dfrac{P_{u\,\#50}^{max}}{N} = \dfrac{0.133}{0.557}$

MOFFATT & NICHOL

SUBJECT	TEMP. TOW.	JOB NO.	4141-05
PROJECT		SHEET	8
SUBJECT	JOINTS	PREPARED BY	JAB 10-20-05
		CHECKED BY	JAD 28 OCT 05

$p_u = 0.133/N = 0.133/0.557 = 0.239$ MN/m

$N = 557$ mm } Gusset PL for #50
$t_p = 19$ mm } same as for #477

$\rightarrow |M^{max}_{u\ weld}| = (0.24)\underbrace{(0.239)}_{p_u}\underbrace{(0.369)}_{R}$

$= 0.0212$ MNm/m

$|M^{max}_{u\ weld}| = 0.0212$ MNm/m $< \phi M_n^{weld} = 0.0341$ MNm/m

$\boxed{OK}$

TEMPORARY TOWERS API - LRFD AXIAL CAPACITY OF MEMBERS

OD (mm)	WT (mm)	ID (mm)	Ao (m2)	D/t	Fy (MPa)	*API LRFD Pa tensile (MN)*	*Member Tension Capacity Number of A325 Bolts 1.0 in. N-S*	*Member Tension Capacity Number of A325 Bolts 1.25 in. N-S*	*Member Tension Capacity Number of A325 Bolts 1.5 in. N-S*	*MBR. CAP PPen. Weld Length (mm)*
610	12	586	0.023	50.8	345.0	***7.39***	***58.7***	***37.5***	***26.1***	***948***
610	9	592	0.017	67.8	345.0	***5.57***	***44.2***	***28.3***	***19.7***	***953***
457	12	433	0.017	38.1	345.0	***5.50***	***43.7***	***27.9***	***19.4***	***705***
457	9	439	0.013	50.8	345.0	***4.15***	***33.0***	***21.1***	***14.7***	***710***

MEMBER CAPACITY REQUIREMENTS

BOLTED CONNECTIONS

SAP2000

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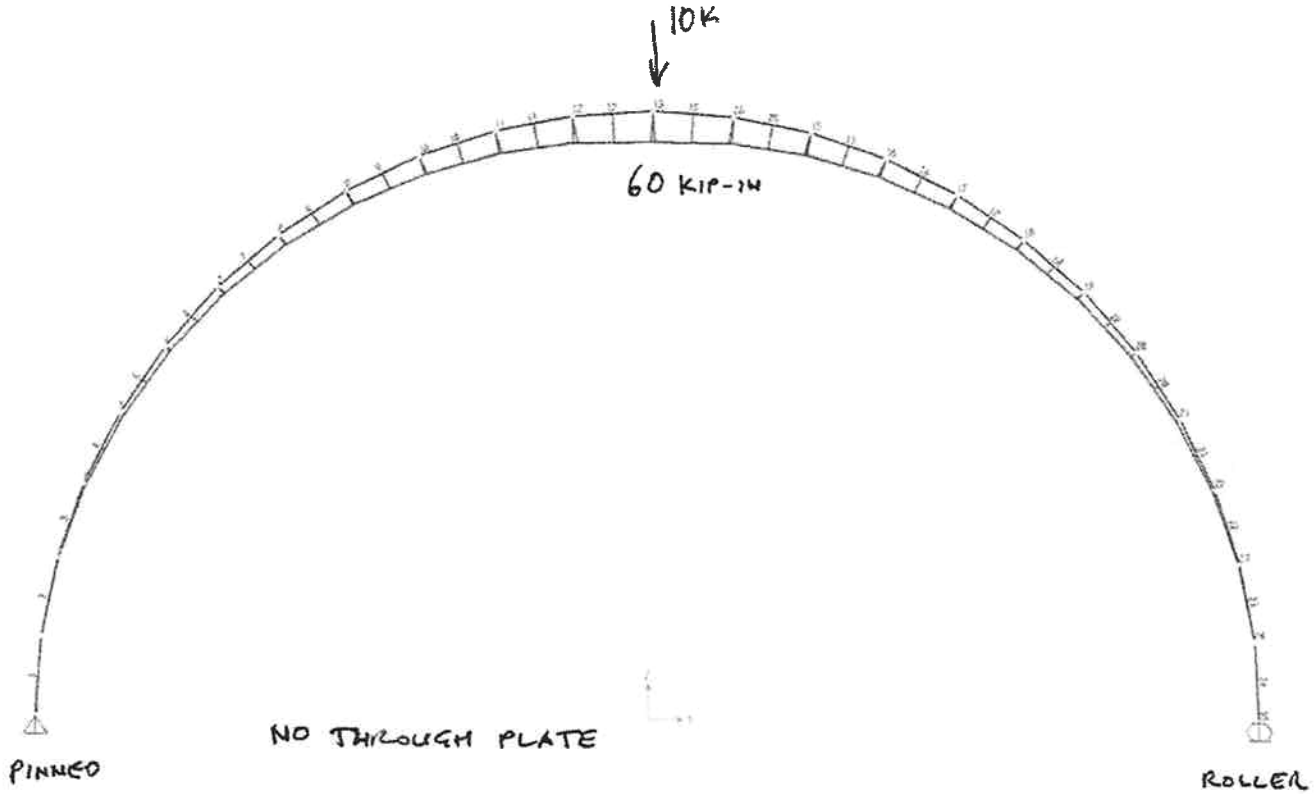

PARTIAL RING ANALYSIS

SAP2000 v9.1.1 - File:ring - Moment 3-3 Diagram (DEAD) - Kip, in, F Units

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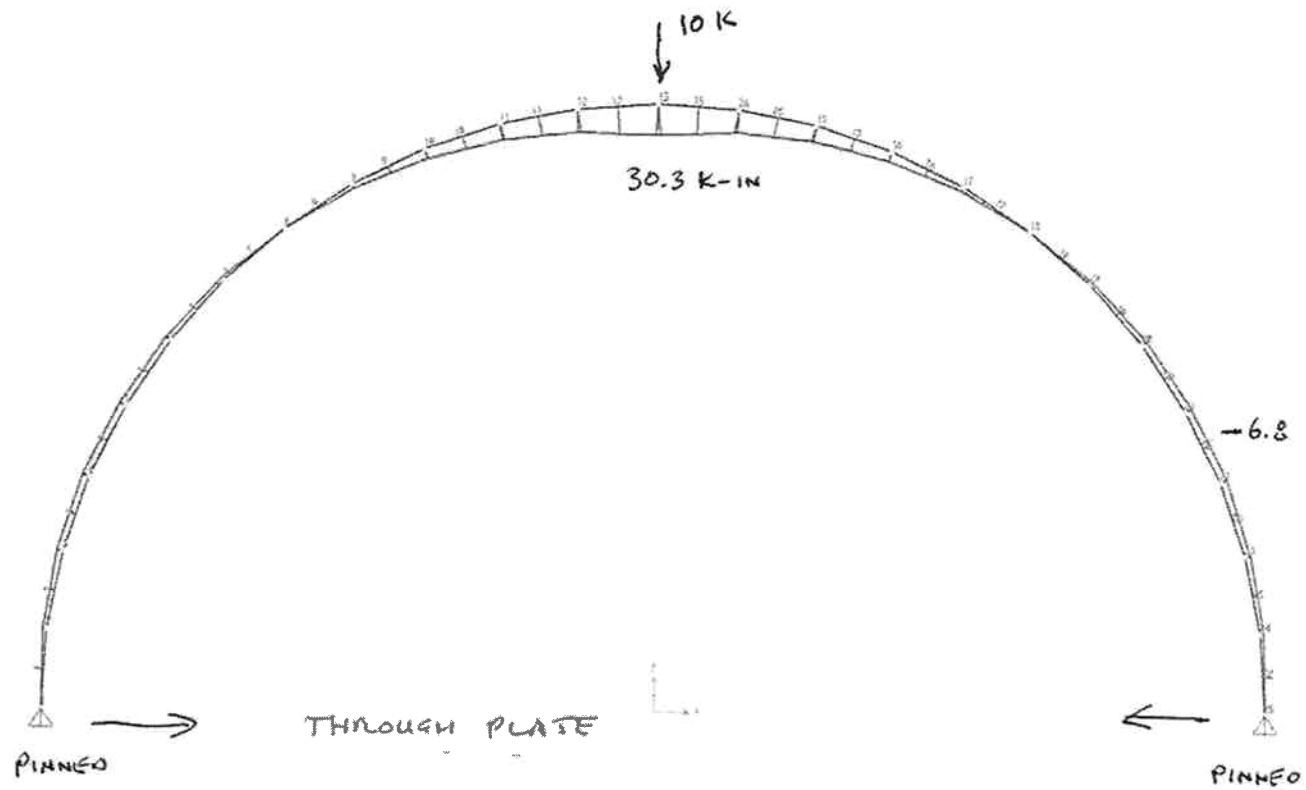


PARTIAL RING ANALYSIS

SAP2000 v9.1.1 - File:ring - Moment 3-3 Diagram (DEAD) - Kip, in, F Units

Appendix D

Misc. Hand/Spreadsheet Calculations

Preliminary Design and Loads Development
Various Pile Checks
Impact Fender & Bracing

NOT FOR CONSTUCTION

Preliminary Design Procedures

The following section contains the preliminary design calculations for Tower G, in spreadsheet form, adapted from classical design procedures.

Basically, it estimates the base shear and overturning moment of the tower by applying a series of simplifying assumptions, which in turn are used to estimate the pile axial and lateral load requirements. Once the pile size is known, the remainder of the jacket members are sized accordingly: leg diameters are about 150mm larger in order to provide the desired pile clearance. The bracing sizes were initially chosen based on L/r and D/t ratios but were checked against an estimate of demand obtained by resolving the lateral forces through the bracing down to the mudline for the governing load case.

Initially, the Dead and Live Load cases for Tower G were narrowed down to four conditions where the box girder sections were installed in three steps: Lift #9, Heavy Lift #3 (eccentric), Heavy Lift #4 and the Table 1 loads envelope.

Next, the seismic demand was estimated by determining the appropriate mass conditions for the above Dead Load cases along with an approximation of the tower stiffness based on the moment area of the legs with an adjustment factor for the cross-bracing stiffness. This produced an estimate of the tower period for the different load conditions and resulted in lateral accelerations obtained from the specified design spectrum. The total lateral force was thus estimated along with the associated overturning moment, which was resolved into lateral and axial pile forces.

The Wind Load force was also determined by spreadsheet along with the overturning moment, and was also resolved into pile forces. The wave force and moment determined for a 1m diameter tubular was applied to the gross projected area of the tower portion in the water, as was the current force and moment.

Comparing all of the above load results for the piles, the seismic forces governed and an appropriate pile wall thickness and penetration was selected for the confirmation analyses.

It should be noted that a series of preliminary pile analyses were performed for the soil profile provided in the Fugro/EMI Geotechnical report section for the temporary towers. These initially included 36, 42 and 48-inch diameter piles from which the final pile size was chosen. Based on the maximum lateral shear per pile as well as the maximum axial force, a 1.067m diameter by 32mm pile was selected with a design penetration of about 52m.

Once all of the computer analyses were completed, the piles were code checked by spreadsheet for the governing load conditions. Overall, Impact governed the pile wall thickness, while Seismic governed their penetration.

- It can be seen from the axial pile response curve that the maximum total vertical displacement is about 30mm under the most severe seismic load case; however, this is a transient load and the structure is expected to settle back to its original Dead Load displacement of about 15mm. In order to achieve the required settlement limit of 25mm, the pile penetration could be increases to about 55m or the pile thickness increased to 38mm.

Independent Load cases

Dead Load
All structural steel in tower
Pile steel above mudline
Estimated deck steel (50 tonnes)

Deck Sections in order of erection sequence taking into account inter-connectivity as it develops.

Tower added mass
Equivalent to gross buoyancy of all member below MSL (including legs)

Live Load
Construction loads specified as 960 N/m^2, and 1100 N/m at deck edges

Wind load
Wind on deck and tower above MSL (Broadside case only)

Earthquake Load
Results from a Response Spectrum Analysis (RSA)

Vessel Impact
7.6 MN applied between Elev. +7.6m and –1.8m
Applied as a point load at three elevations from the two orthogonal directions as well as a diagonal direction

Load Combinations (LRFD)

1.4 DL
1.4 Tower, deck and piles weight above mudline; buoyancy below MSL

1.1 DL + 1.3 LL
1.1 Tower, deck and piles above mudline, buoyancy below MSL, deck weight + 1.3 Live Load

1.0 (DL + LL + 0.5 Wind + Current + Vessel Impact)
1.0 (Tower, deck and piles above mudline, buoyancy below MSL, deck weight + Live Load + 0.5 Wind + Current + Vessel Impact)
Apply wind and current broadside and vary vessel impact angle only.

1.0 (DL + LL + EQ)
1.0 (Tower, deck and piles above mudline, buoyancy below MSL, deck weight + Live Load + EQ) (Including added mass)

1.0 (DL + LL + 1.3 Wind)
1.0 (Tower, deck and piles above mudline, buoyancy below MSL, deck weight + Live Load + 1.3 Wind (broadside only)

Analysis & Design of Temporary Tower C

Load Combinations

Load Comb. #	DL	LL	WIND	EQK
LCMB1	1.4 (L_1)	-	-	-
LCMB2	1.4 (L_T)	-	-	-
LCMB3	1.4 (L_T+L_{HI})			
LCMB4	1.1(L_T+L_{HI})	1.3	-	-
LCMB5	1.0(L_T+L_{HI})	1.0	-	1.0Ex + 0.3Ey
LCMB6	1.0(L_T+L_{HI})	1.0	-	-1.0Ex + 0.3Ey
LCMB7	1.0(L_T+L_{HI})	1.0	-	0.3Ex + 1.0Ey
LCMB8	1.0(L_T+L_{HI})	1.0	-	0.3Ex – 1.0Ey
LCMB9	1.0 (L_T+L_{HI})	1.0	1.3	-

Notes:

1. The load combinations in the specification correspond to the above list as shown below:
   - Spec. Combination 1 (1.4D) – LCMB1, 2 & 3
   - Spec. Combination 2 (1.1D+1.3L) – LCMB4
   - Spec. Combination 4 (1.0D+1.0L+1.0 Eqk) – LCMB5, 6, 7 & 8
   - Spec. Combination 5 (1.0D+1.0L+1.3W) – LCMB9.
2. Different dead loads are used in the combinations. These are as follows:
   - L_1 – 1st Lift (830T)
   - L_T – Total dead load on structure (Table 1 loads)
   - L_{HI} – Heavy lift 1 cantilevered at East end of support structure (Table 1)

Analysis & Design of Temporary Tower G

Load Combinations

Load Comb. #	DL	LL	WIND	WAVE &CURRENT	EQK	IMPACT
LCMB1	1.4 (M2)	-	-	-	-	-
LCMB2	1.4 (M3)	-	-	-	-	-
LCMB3	1.4 (M4)	-	-	-	-	-
LCMB4	1.4 (M5)	-	-	-	-	-
LCMB5	1.1 (M3)	1.3	-	-	-	-
LCMB6	1.1 (M4)	1.3	-	-	-	-
LCMB7	1.1 (M5)	1.3	-	-	-	-
LCMB8	1.0 (M5)	1.0	0.5	1.0	-	1.0 (Dir Y)
LCMB9	1.0 (M5)	1.0	0.5	1.0	-	1.0 (Dir X)
LCMB10	1.0 (M5)	1.0	0.5	1.0	-	1.0 (Angular)
LCMB11	1.0 (M2)	1.0	-	-	1.0Ex + 0.3Ey	-
LCMB12	1.0 (M2)	1.0	-	-	0.3Ex + 1.0Ey	-
LCMB13	1.0 (M5)	1.0	-	-	1.0Ex + 0.3Ey	-
LCMB14	1.0 (M5)	1.0	-	-	-1.0Ex + 0.3Ey	-
LCMB15	1.0 (M5)	1.0	-	-	0.3Ex + 1.0Ey	-
LCMB16	1.0 (M5)	1.0	-	-	0.3Ex – 1.0Ey	-
LCMB17	1.0 (M5)	1.0	1.3	1.3	-	-

Notes:

1. The load combinations in the specification correspond to the above list as shown below:
   - Spec. Combination 1 (1.4D) – LCMB1, 2, 3, & 4.
   - Spec. Combination 2 (1.1D+1.3L) – LCMB5, 6, 7.
   - Spec. Combination 3 (1.0D+1.0L+0.5W+1.0(Wave/Current/Impact) – LCMB8, 9, 10.
   - Spec. Combination 4 (1.0D+1.0L+1.0 Eqk) – LCMB11, 12, 13, 14, 15, 16.
   - Spec. Combination 5 (1.0D+1.0L+1.3W+1.3Wave/Current) – LCMB17.
2. Different dead loads are used in the combinations. These are as follows:
   - M2 – Model 2: Deck weight + 1st Lift
   - M3 – Model 3: Deck weight + 1st Lift + ½ Heavy lift (one side)
   - M4 – Model 4: Deck weight + 1st Lift + Full Heavy lift (both sides)
   - M5 – Model incorporating Towers G, F, H (pier E2), deck etc (full model) with Table 1 loads applied to Tower G.

Table 1. Envelope of Vertical Forces at Temporary Towers and Temporary Bearings During Construction

Temporary Tower	Supported load per temporary tower, per box kN (See Note 1)	Temporary bearing pair number	Compression load per bearing pair, per box girder kN (See Note 1)	Tension load per bearing pair, per box girder kN (See Notes 1 & 2)
Temporary Tower A	17100	1	14900	-
		2	600	-
		3	4600	-
Temporary Tower B	11850	4	7700	-
		5	4700	-
		6	2550	-
Temporary Tower C	12650	7	1800	-
		8	12300	-
Temporary Tower D	24500	9	8200	-
		10	2700	-
		11	15850	-
Temporary Tower F	28400	12	13250	-
		13	3050	-
		14	13500	-
Temporary Tower G	28550	15	13500	-
		16	2850	-
		17	13150	-
Temporary Tower H	16800	18	16800	-
Temporary Tower E2	32850	19	9800	-
		20	26250	8300
		21	31050	-

Note: (a) No load factors are applied to these loads

(b) The above loads do not include wind or seismic effects

Table 2. Lift Weights

LIFT NO.	Estimated Weight, metric tons
1	830
2	546
3	459
4	459
5	912
6	2117
7	943
8	943
9	892
HL 1	734
HL 2	1259
HL 3	1285
HL 4	1285

HL=Heavy Lift

Dead Load & Live Load TOWER G

Dead load	Deck Lift #9		Heavy Lift #4	Intermediate case		Heavy Lift #3	Table 1	
Jacket	614 Tonnes	(Assumed)	614	614		614	614	
Piles	463		463	463		463	463	
Deck #9	892 Tonnes		892	892		892		
Deck #3			1285	1969.7525	328	1285		
Deck #4				642.5	161	1285	2800	
				@ 24.3 m				
					260			
Total	16.28 MN		22.57 MN			28.87 MN	34.97 MN	
	1,661 Tonnes		2,303 Tonnes			2,946 Tonnes	3,569 Tonnes	
	3,654 kips		5,067 kips			6,481 kips	7,851 kips	
	2.71 MN		3.76 MN			4.81 MN	8.01 MN	
6 Legs/Piles	277 Tonnes		384 Tonnes	Max	7.34 MN	491 Tonnes	817 Tonnes	/pile
	609 kips		845 kips		749 Tonnes	1,080 kips	1,797 kips	
					1,648 kips			
					0.67 MN			
				Min	68 Tonnes			
				(Tension)	150 kips			

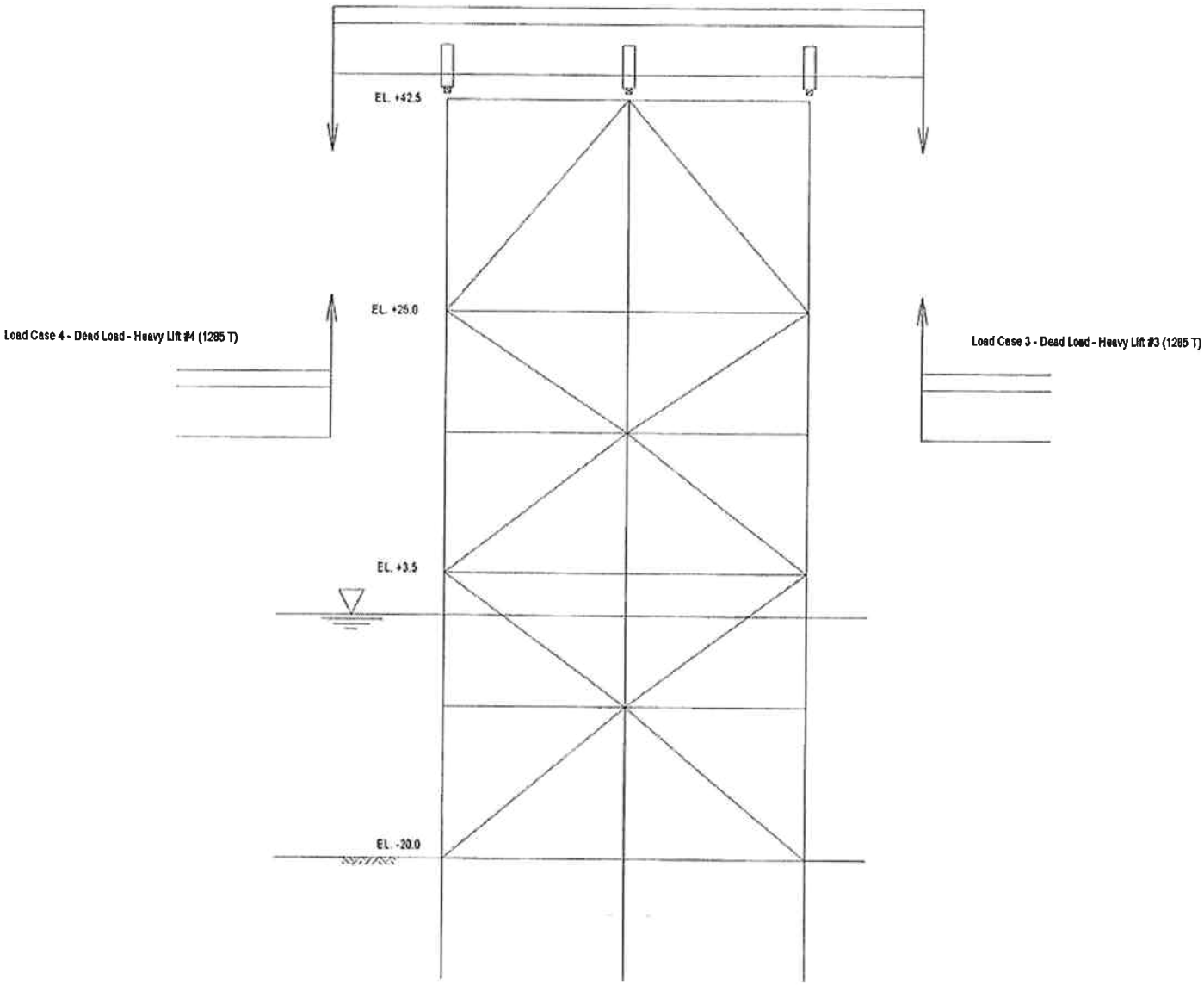
Live load	0.96 KN/m^3		1.1 KN/m					
Deck area	48.6	13.94	677.5 m^2			118.6	13.94	1653.3 m^2
			650 KN					1,587 KN
Deck edges	48.6	13.94	125.1 m			118.6	13.94	265.1 m
			138 kN					292 kN
	total		788 kN	average	1,333 kN			1,879 kN
			0.79 MN		1.33 MN			1.88 MN
			80.4 Tonnes		136.1 Tonnes			192 Tonnes total
			177 kips		299 kips			422 kips
	4.27 MN		5.84 MN		11.35 MN	7.69 MN		12.48 MN
Pile ult required =	435 Tonnes		596 Tonnes		1,158 Tonnes	784 Tonnes		1,273 Tonnes /pile
	958 kips		1,311 kips		2,547 kips	1,726 kips		2,801 kips

		A	I	S	
42	1	129	27081	1290	
Stress check		1,867 kips		3500 in-kips	
		14.50		2.71	17.21 ksi
					0.34

Load Case 1 - Dead Load - Tower Only

Load Case 2 - Dead Load - Tower plus Deck Lift #9 (892 T)

Load Case 5 - Dead Load - Tower plus Bearing loads per Table (see drawings)



TOWER G - Independent Load Cases

MOFFATT & NICHOL ENGINEERS	CLIENT CALTRANS	JOB NO.	
	PROJECT SFOBB TEMP TOWERS	SHEET OF	
	DESIGN FOR	DESIGNER JAO	DATE
	LIVE LOAD	CHECKER	DATE

LIVE LOAD

960 N/m² plus 1100 N @ DECK EDGES

TYPICAL DECK WIDTH = 2x 13.94 = 27.88m

PER METER OF DECK LENGTH

$$\frac{960 \times 27.88}{1000} = 26.8 \text{ KN}$$

FOR A 50m LENGTH

$$\frac{960 \times 27.88 \times 50}{1000} = 1338 \text{ KN}$$

DECK EDGE = 2x 27.88 + 2x 50 = 155.8m

$$\frac{155.8 \times 1100}{1000} = 171 \text{ KN}$$

1509 KN = 154 TONNES ✓

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Seismic TOWER G

Loads

	Lift #9	Heavy lifts #3 & #4	Table 1
Jacket	600 Tonnes	600 Tonnes	600 Tonnes
Piles	450	450	450
Deck lift #9	892	892	
Heavy lift #3		1285	
Heavy lift #4		1285	
Load per pile	277 Tonnes	491 Tonnes	817 Tonnes
	609 kips	1,080 kips	1,797 kips
	10.70 MN	23.29 MN	28.55 MN
Dynamic mass	1,092 Tonnes	2,377 Tonnes	2,913 Tonnes
	2,402 kips	5,229 kips	6,409 kips
E steel =	29,000 ksi		
	4,176,000 ksf		
Height	72.5 m		
	237.9 ft		

Approx. Period

Spectrum

Period	PSA
0.2	0.7
0.4	0.55
0.6	0.45
0.8	0.35
1	0.2
2	0.1
3	0.075
4	0.05

Jacket Stiffness

Try 42 in diam x 1.00 in piles D/t = 42

Area = 128.81 in^2 = 0.89 ft^2

I jacket =	8,665 ft^4		5,777 ft^4											
K =	3,227 kips/ft		2,151 kips/ft		3,227		2,151		3,227		2,151			
Mass =	74.61 kips sec^2/ft		74.61 kips sec^2/ft		162.40		162.40		199.04		199.04			
Period =	0.96 sec		1.17 sec		1.41		1.73		1.58		1.91			
	0.80	0.35	1.00	0.20	1.00	0.20	1.00	0.20	1.00	0.20	1.00	0.20		
	1.00	0.20	2.00	0.10	2.00	0.10	2.00	0.10	2.00	0.10	2.00	0.10		
PSA	0.23		0.18		0.16		0.13		0.14		0.11			

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Seismic (Cont'd)

Loads	Lift #9			Heavy lifts #3 & #4			Table 1		
Lateral	2.50 MN	1.96 MN		3.70 MN	2.97 MN		4.11 MN	3.11 MN	
	255 Tonnes	200 Tonnes		378 Tonnes	303 Tonnes		419 Tonnes	317 Tonnes	
	561 kips	440 kips		832 kips	668 kips		923 kips	698 kips	
Lateral Per Pile	0.42 MN	0.33 MN		0.62 MN	0.49 MN		0.68 MN	0.52 MN	
	42.5 Tonnes	33.3 Tonnes		63.0 Tonnes	50.5 Tonnes		69.9 Tonnes	52.9 Tonnes	
	93 kips	73 kips		139 kips	111 kips		154 kips	116 kips	
Height	72.5 m	72.5 m		72.5 m	72.5 m		72.5 m	72.5 m	
	238 ft	238 ft		238 ft	238 ft		238 ft	238 ft	
Moment	181.1 MN-m	142.0 MN-m		268.6 MN-m	215.1 MN-m		298.0 MN-m	225.4 MN-m	
	18,481 Tonne-m	14,487 Tonne-m		27,408 Tonne-m	21,948 Tonne-m		30,403 Tonne-m	22,996 Tonne-m	
	133,392 ft-kips	104,563 ft-kips		197,822 ft-kips	158,417 ft-kips		219,444 ft-kips	165,977 ft-kips	
Couple	6.04 MN	4.73 MN		8.95 MN	7.17 MN		9.93 MN	7.51 MN	
	616 Tonnes	483 Tonnes		914 Tonnes	732 Tonnes		1,013 Tonnes	767 Tonnes	
	1,355 kips	1,062 kips		2,010 kips	1,610 kips		2,230 kips	1,686 kips	
Axial Per Pile	3.0 MN	2.4 MN		4.5 MN	3.6 MN		5.0 MN	3.8 MN	
	308 Tonnes	241 Tonnes		457 Tonnes	366 Tonnes		507 Tonnes	383 Tonnes	
	678 kips	531 kips		1,005 kips	805 kips		1,115 kips	843 kips	
Total Axial Pile load =	1,287 kips	Comp	1,140 kips	2,085 kips	Comp	1,885 kips	2,912 kips	Comp	2,640 kips /pile
	5.74 MN		5.09 MN	9.31 MN		8.41 MN	13.00 MN		11.79 MN
	(69) kips	Tension	78 kips	75 kips	Tension	275 kips	682 kips	Tension	954 kips /pile
	(0.31) MN		0.35 MN	0.34 MN		1.23 MN	3.05 MN		4.26 MN

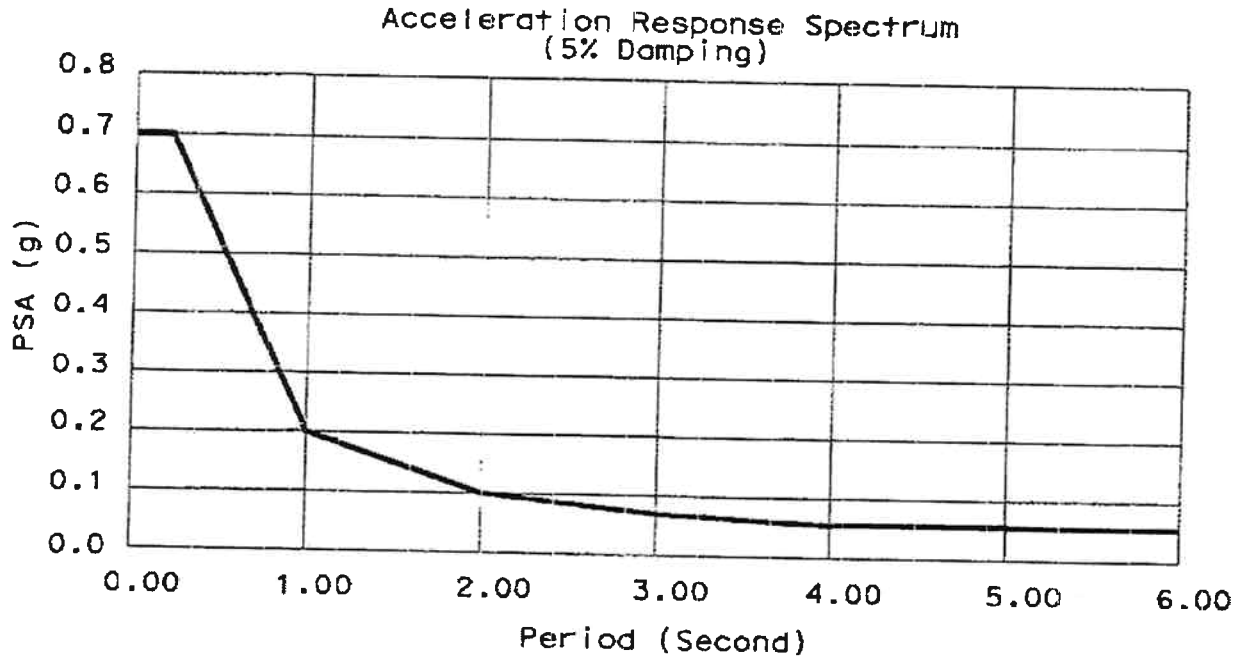
A		I	S	
42	1.00	129	27081	1290

1,287 kips	35000 in-kips			2,912 kips	35000 in-kips		
9.99	27.14	37.13 ksi		22.61	27.14	49.75 ksi	
		0.74				0.99	

9. Temporary Towers:

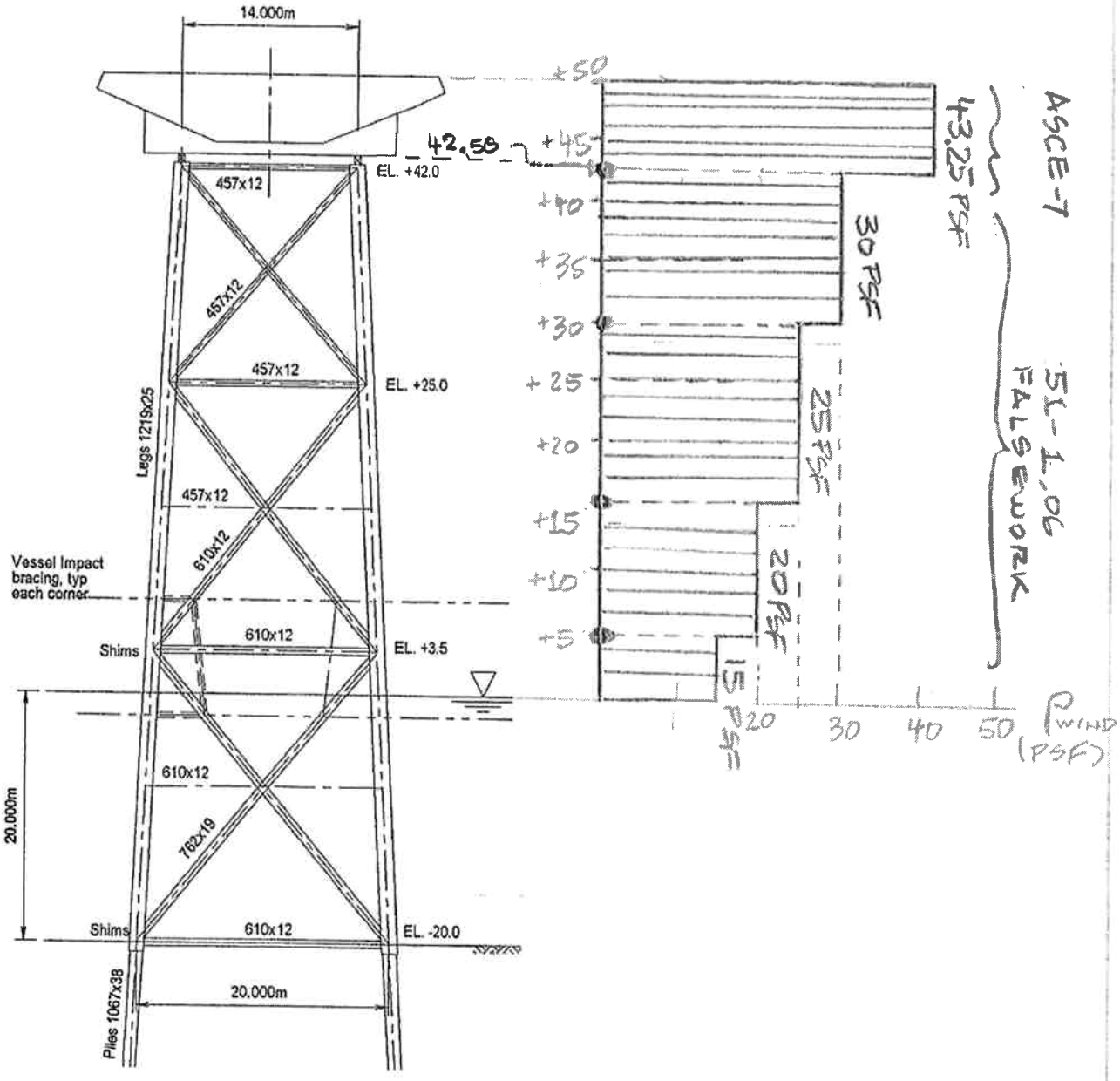
The following minimum horizontal design response spectrum chart shall be used in the design of the temporary towers.

Period (Second)	PSA (g)
0.01	0.7
0.25	0.7
1.00	0.2
2.00	0.1
3.00	0.06
4.00	0.05
5.00	0.05
6.00	0.05



CLIENT	SAS	JOB NO	4141-05
PROJECT	TEMPORARY TOWERS	SHEET	OF
DESIGN FOR		DESIGNER JMB	DATE 9-26-05
		CHECKER	DATE

DESIGN WIND: 100mph gust @ 50 meter.



END ELEVATION

Wind, Wave & Current

Wind

TEMPORARY TOWERS - WIND LOADING - ASCE 7 - 95								
ELEVATION (m)	ELEVATION (ft)	Kz	qz (N/m2)	qz (psf)	G	Cf	p(dyn) qzGCf (N/m2)	p(dyn) qzGCf (psf)
46.25	151.74	1.54	1623.36	33.92	0.85	1.50	2069.78	43.25
42.5	139.44	1.52	1602.28	33.48	0.85	0.80	1089.55	22.77
33.75	110.73	1.46	1539.03	32.16	0.85	0.80	1046.54	21.87
25	82.02	1.38	1454.70	30.40	0.85	0.80	989.19	20.67
14.25	46.75	1.26	1328.20	27.75	0.85	0.80	903.18	18.87
3.5	11.48	1.03	1085.75	22.69	0.85	0.80	738.31	15.43

V = 100.00 mph at 50 m Gust (Spec. Prov.)

V = 86.50 mph at 10 m. (factored down to El. +10m)
V = 38.67 m/s

I = 1.15

TEMPORARY TOWERS - WIND LOADING - ASCE 7 - 95 / 51-1.06 FALSEWORK - 100 mph gust @ 50m.												
ELEVATION (m)	ELEVATION (ft)	Kz	qz (N/m2)	qz (psf)	G	Cf	p(dyn) qzGCf (N/m2)	p(dyn) qzGCf (psf)	p FALSEW. (N /m2)	p (FALSEW.) (psf)	p DESIGN (N/m2)	p DESIGN (psf)
46.25	151.74	1.54	1623.36	33.92	0.85	1.50	2069.78	43.25	1440.00	30.00	2079.49	43.25
42.50	139.44	1.52	1602.28	33.48	0.85	0.80	1089.55	22.77	1440.00	30.00	1440.00	30.00
40.00	131.23	1.50	1581.19	33.04	0.85	0.80	1075.21	22.47	1440.00	30.00	1440.00	30.00
35.00	114.83	1.47	1549.57	32.38	0.85	0.80	1053.71	22.02	1440.00	30.00	1440.00	30.00
33.75	110.73	1.45	1528.49	31.94	0.85	0.80	1039.37	21.72	1440.00	30.00	1440.00	30.00
30.00	98.43	1.43	1507.40	31.50	0.85	0.80	1025.04	21.42	1200.00	25.00	1200.00	25.00
25.00	82.02	1.38	1454.70	30.40	0.85	0.80	989.19	20.67	1200.00	25.00	1200.00	25.00
20.00	65.62	1.33	1401.99	29.30	0.85	0.80	953.35	19.92	1200.00	25.00	1200.00	25.00
15.00	49.21	1.27	1338.74	27.98	0.85	0.80	910.35	19.02	960 / 1200	20 / 25	960 / 1200	20 / 25
14.25	46.75	1.26	1328.20	27.75	0.85	0.80	903.18	18.87	960.00	20.00	960.00	20.00
10.00	32.81	1.17	1233.33	25.77	0.85	0.80	838.67	17.53	960.00	20.00	960.00	20.00
5.00	16.40	1.03	1085.75	22.69	0.85	0.80	738.31	15.43	720 / 960	15 / 20	720 / 960	15 / 20
0.00	0.00	1.03	1085.75	22.69	0.85	0.80	738.31	15.43	720.00	15.00	720.00	15.00

V = 100.00 mph at 50 m
V = 44.70 m/s

TEMPORARY TOWERS - WIND LOADING - ASCE 7 - 95 / 51-1.06 FALSEWORK - 100 mph gust @ 50m.												
ELEVATION (m)	ELEVATION (ft)	p DESIGN (N/m2)	p DESIGN (psf)	h or diam. (m)	L (m)	Atributary (m2)	Wind Load F (kN)	Global Shear (kN)	M global (kNm)	Pile spacing (m)	N Leg/Pile (kN)	N Diag (kN)
46.25	151.74	2079.49	43.25	6.50	118.60	770.90	1603	1603	6012			
42.50	139.44	1440.00	30.00	1.00	32.00	32.00	46	1649	10134	14	241	416
40.00	131.23	1440.00	30.00	15.40	5.00	77.00	111	1760	18935	15	421	428
35.00	114.83	1440.00	30.00	15.40	5.00	77.00	111	1871	28289	16	589	457
30.00	98.43	1200.00	25.00	15.40	5.00	77.00	92	1963	38106	16	794	485
25.00	82.02	1200.00	25.00	15.40	5.00	77.00	92	2056	48384	17	949	510
20.00	65.62	1200.00	25.00	15.40	5.00	77.00	92	2148	59125	17	1159	534
15.00	49.21	1080.00	20 / 25	15.40	5.00	77.00	83	2231	70281	18	1302	558
10.00	32.81	960.00	20.00	15.40	5.00	77.00	74	2305	81807	18	1515	579
5.00	16.40	840.00	15 / 20	15.40	5.00	77.00	65	2370	93657	19	1643	598
0.00	0.00	720.00	15.00	15.40	2.50	38.50	28	2398	99651	20	1661	615
-2.50	-8.20					1457.40	2398	2398				
-20.00	-65.62							2398	147603	22	2236	

Shear	400	kN/pile	
Axial	1,661	kN/pile	Broadside
	2,001	kN/pile	Diagonal
	1,760		45 deg
	1,618		X diag
Wind center	41.6	m	

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MOFFATT & NICHOL ENGINEERS	CLIENT CALTRANS	JOB NO.	
	PROJECT SFOBB TEMP TOWERS	SHEET OF	
	DESIGN FOR	DESIGNER JAD	DATE
	WIND LOAD	CHECKER	DATE

WIND LOAD

HEIGHT OF DECK CENTROID = 46.25 m

100 MPH (3 SEC GUST) @ EL +50 m

ADJUST WIND VELOCITY DOWN TO +30 m (REF EL)
USING TABLE 6.3 FACTORS FOR EXP. D (ASCE 7-95)

$$\sqrt{\frac{100^2 \times 1.16}{1.55}} = 86.5 \text{ MPH @ EL +30 m}$$

$K_z = 1.54$ 86.5 MPH = 38.7 m/s $I = 1.15$ SPEC. PROV.

$q_z = 0.613\, K_z K_{zt} V^2 I$ $K_{zt} = 1.0$

$= 0.613 \times 1.54 \times 1.0 \times 38.7^2 \times 1.15$

$= 1623 \text{ N/m}^2$

GUST FACTOR $G = 0.85$ FOR EXP D SECT 6.6 ASCE

FORCE COEFFICIENT $C_f = 1.5$ TABLE 6.8 ASCE

$p = q_z G C_f = 1623 \times 0.85 \times 1.5 = 2069 \text{ N/m}^2$

S-1

Wave TOWER 6

Water depth =	20.0 m	65.6 ft	Cd =	0.65		(API RP2A)
			Cm =	1.5		
Height =	2.0 m	6.56 ft	(Fetch length limited)			(Generated per COE Shore Protection Manual)
Period =	6.0 sec					
Wave force on a 1m diameter pipe:		12.4 kN	2.79 kips			(Linear wave theory)
Wave moment:		164.0 kN-m	121 ft-kips			
Height of Wave force resultant =		13.2 m	43.4 ft			

Broadside

Exposed area		#	diam. D	length L	angle	sine	Area			
	Legs	6	1.219	21.6	90	1.000	157.98			
in-plane	Diag Brace	4	0.762	30.4	90	1.000	92.66			
out-of-plane	Diag Brace	6	0.762	25.0	51	0.777	69.03			(sine^2 for horiz component of normal force)
in-plane	Horiz Br	2	0.610	28.2	90	1.000	34.40			
out-of-plane	Horiz Br	3	0.610	17.6	0	-	-			
	Note: Mudline horiz ignored.						354.1	m^2	3,811	ft^2

Total wave force =	220 kN	49.4 kips					
Wave moment =	2,904 kN-m	2,142 ft-kips	Maximum pile force (wave only) =	48.4 kN	10.88 kips		

End-on

Exposed area		#	diam. D	length L	angle	sine	Area			
	Legs	6	1.219	21.6	90	1.000	157.98			
in-plane	Diag Brace	6	0.762	25.0	90	1.000	114.30			
out-of-plane	Diag Brace	4	0.762	30.4	40	0.643	38.28			(sine^2 for horiz component of normal force)
in-plane	Horiz Br	3	0.610	17.6	90	1.000	32.21			
out-of-plane	Horiz Br	2	0.610	28.2	0	-	-			
	Note: Mudline horiz ignored.						342.8	m^2	3,690	ft^2

Total wave force =	213 kN	47.8 kips					
Wave moment =	2,812 kN-m	2,074 ft-kips	Maximum pile force (wave only) =	46.9 kN	10.53 kips		

Member Buoyancy / Added Mass

						Volume				
Legs	(not buoyant)	1.219	6	20	1.17	140.05	m^3	Buoyancy =	188	Tonnes
Diags		0.762	6	25	0.46	68.41				
		0.762	4	30.4	0.46	55.45		Added Mass =	328	Tonnes
Horiz		0.61	3	17.6	0.29	15.43				
		0.61	2	27.6	0.29	16.13				
Mudline Horiz		0.61	3	18.8	0.29	16.48				
		0.61	2	27.6	0.29	16.13				
						328.09	Tonnes			

WAVE FORCE DISTRIBUTION ON CYLINDRICAL PILE

SFOBB SASS

WAVE HEIGHT =	6.56	FT	DEPTH =	65.6 FT
PERIOD =	6	SEC	SLOPE =	.00001
DEEPWATER LENGTH, Lo =	184.3	FT	LENGTH, L =	180.5 FT
BREAKING WAVE HEIGHT, Hb =	25.5	FT	DIAMETER =	3.3 FT

PHASE	0	15.0	30.0	45.0	60.0	75.0	90.0	135.0	180.0
SURF DSP	3.5	3.5	3.3	2.9	2.0	0.6	-0.8	-2.4	-3.0
HEIGHT			LB FORCE ON 1		FT SEGMENTS				
71.0	5	5							
70.0	34	65	72	53					
69.0	32	62	79	94	62				
68.0	30	58	76	90	96	21			
67.0	28	56	73	87	93	97			
66.0	26	53	69	84	89	94	78		
65.0	24	50	67	80	86	91	92	9	
64.0	23	48	64	77	83	88	89	52	-13
63.0	21	46	61	74	80	85	86	50	-21
62.0	20	43	58	71	77	82	84	49	-20
61.0	19	41	56	69	75	79	81	48	-19
60.0	17	39	54	66	72	77	78	46	-17
59.0	16	38	52	64	69	74	75	45	-16
58.0	15	36	49	61	67	72	73	44	-15
57.0	14	34	47	59	65	69	71	43	-14
56.0	13	33	46	57	62	67	68	42	-13
55.0	13	31	44	55	60	65	66	40	-13
54.0	12	30	42	53	58	63	64	39	-12
53.0	11	29	40	51	56	60	62	38	-11
52.0	10	27	39	49	54	58	60	37	-10
51.0	10	26	37	47	53	57	58	36	-10
50.0	9	25	36	46	51	55	56	35	-9
49.0	8	24	35	44	49	53	54	34	-8
48.0	8	23	33	43	47	51	53	33	-8
47.0	7	22	32	41	46	50	51	32	-7
46.0	7	21	31	40	44	48	49	31	-7
45.0	7	20	30	38	43	47	48	31	-7
44.0	6	19	29	37	42	45	46	30	-6
43.0	6	19	28	36	40	44	45	29	-6
42.0	5	18	27	35	39	42	43	28	-5
41.0	5	17	26	34	38	41	42	27	-5
40.0	5	17	25	32	37	40	41	27	-5
39.0	5	16	24	31	35	39	40	26	-5
38.0	4	15	23	30	34	37	39	25	-4
37.0	4	15	22	29	33	36	37	24	-4
36.0	4	14	22	29	32	35	36	24	-4
35.0	4	14	21	28	31	34	35	23	-4
34.0	3	13	20	27	31	33	34	23	-3
33.0	3	13	20	26	30	32	33	22	-3
32.0	3	12	19	25	29	32	32	21	-3
31.0	3	12	19	25	28	31	32	21	-3
30.0	3	12	18	24	27	30	31	20	-3
29.0	3	11	17	23	27	29	30	20	-3
28.0	2	11	17	23	26	28	29	19	-2
27.0	2	11	17	22	25	28	28	19	-2
26.0	2	10	16	22	25	27	28	19	-2

25.0	2	10	16	21	24	26	27	18	-2
24.0	2	10	15	20	23	26	27	18	-2
23.0	2	9	15	20	23	25	26	17	-2
22.0	2	9	15	20	22	25	25	17	-2
21.0	2	9	14	19	22	24	25	17	-2
20.0	2	9	14	19	22	24	24	16	-2
19.0	2	9	14	18	21	23	24	16	-2
18.0	2	8	13	18	21	23	23	16	-2
17.0	2	8	13	18	20	22	23	16	-2
16.0	1	8	13	17	20	22	23	15	-1
15.0	1	8	13	17	20	22	22	15	-1
14.0	1	8	12	17	19	21	22	15	-1
13.0	1	8	12	17	19	21	22	15	-1
12.0	1	8	12	16	19	21	21	14	-1
11.0	1	7	12	16	19	21	21	14	-1
10.0	1	7	12	16	18	20	21	14	-1
9.0	1	7	12	16	18	20	21	14	-1
8.0	1	7	12	16	18	20	21	14	-1
7.0	1	7	11	16	18	20	20	14	-1
6.0	1	7	11	16	18	20	20	14	-1
5.0	1	7	11	15	18	20	20	14	-1
4.0	1	7	11	15	18	20	20	14	-1
3.0	1	7	11	15	18	19	20	14	-1
2.0	1	7	11	15	18	19	20	14	-1

TOTAL FORCE (LBS)

548	1445	2035	2545	2724	2789	2718	1626	-358

MOMENT (FT-KIPS)

29	69	95	116	121	119	113	64	-17

Current

TOWER G

MSL = 20.0 m 65.6 ft

Maximum Current at MSL = 3.0 knots (Bay area Tide and Current tables)
1.54 m/sec 5.06 ft/sec

Current velocity profile (using 1/7th power distribution)

	Depth	x/depth	1/7th	Vc		Fc		Force		Moment	
	m			m/sec	ft/sec	N/m2	lbs/ft^2	N	lbs	N-m	lb-ft
MSL	0	1.00	100%	1.54	5.06	797.9	16.7				
	5	0.75	96%	1.48	4.86	734.9	15.3	3,831.9	861.5	67,058	49,462
	10	0.50	91%	1.40	4.59	654.5	13.7	3,473.5	780.9	43,419	32,026
	15	0.25	82%	1.27	4.15	536.9	11.2	2,978.6	669.6	22,339	16,477
Mudline	20	0.00	34%	0.52	1.71	90.9	1.9	1,569.7	352.9	3,932	2,900
								11,854	2,665	136,749	100,865

Height of Current force = 11.5 m 37.8 ft

Broadside Area = 354.1 m^2 3,811 ft^2 (see wave area above)

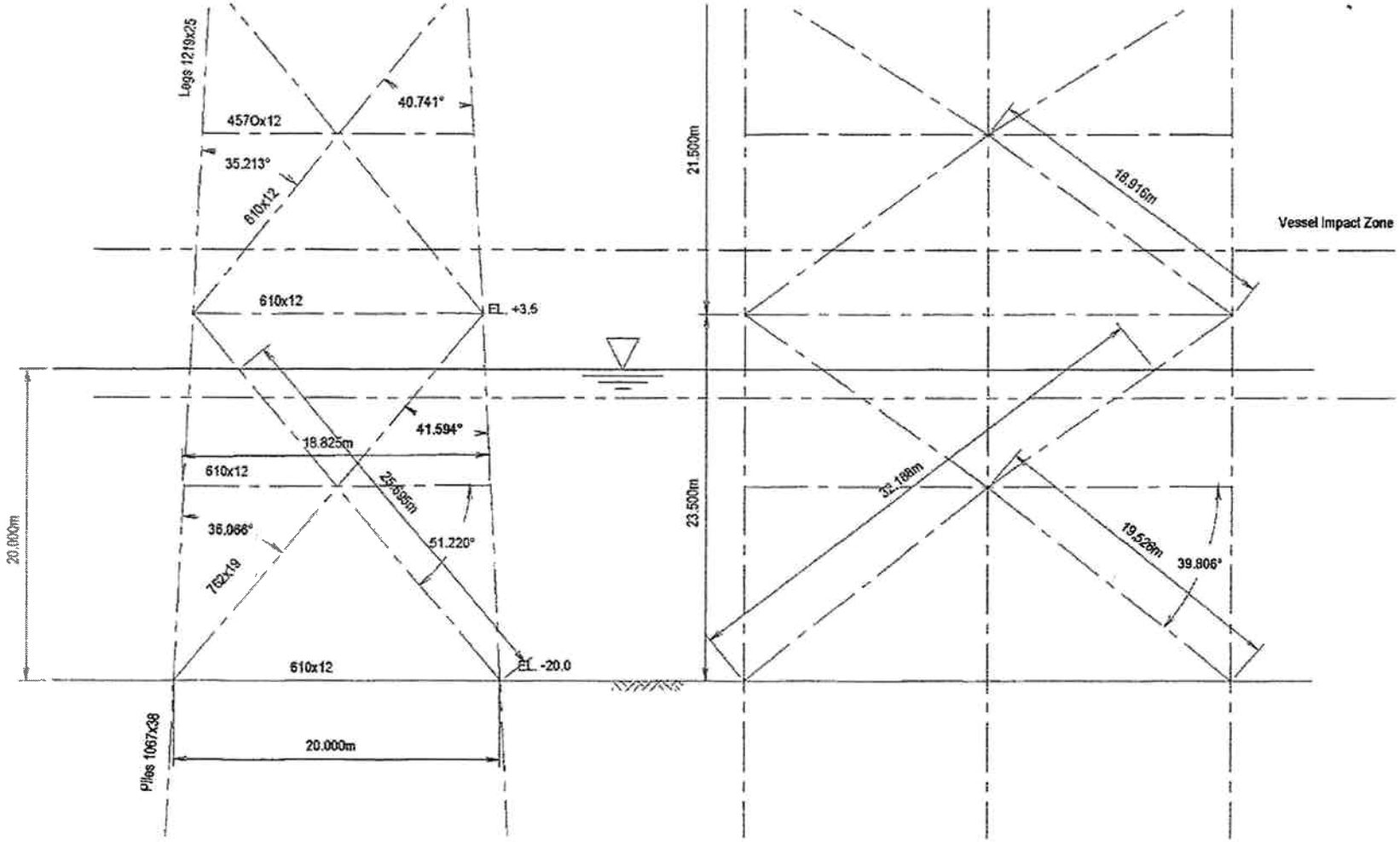
Total current force = 209.9 kN 47.2 kips
Current moment = 2,421 kN-m 1,786 ft-kips Maximum pile force (current only) = 40.3 kN 9.07 kips /pile

End-on Area = 342.8 m^2 3,690 ft^2 (see wave area above)

Total current force = 203.2 kN 45.7 kips
Current moment = 2,344 kN-m 1,729 ft-kips Maximum pile force (current only) = 39.1 kN 8.78 kips /pile

Wind, Wave & Current Summary

DL	8.01 MN	(Table 1 loads)
LL	0.31	Full deck
Wind	1.66	Full deck (Broadside)
Wave	0.05	Broadside
Current	0.04	Broadside
	10.07 MN	



Temporary Tower G - Prelim. Config.

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Pile Calculations

PILE STRESS CHECKS

Seismic
Pile check

		Area	I	S	r	Z	
42	1.25	160.0	33,248	1,583	14.41	2,058	23.5 m
							77.1 ft

14,000	kN	6,246		l/r	64.19
1,429	Tonnes				
3,149	kips	4,506		Cc =	107
	19.68 ksi	34.16	ksi	Fy =	50 ksi
		53.83	ksi total		
				alpha =	0.85
API D.3.2-2					
1	- 0.75	+	0.55	Fcn =	41.0 ksi
		=	0.81	Fey =	69.5 ksi
			OK		
				Cm =	0.72
				Fxe =	518 ksi
				Fxc =	50 ksi
					0.98
				Fbn =	63.7 ksi

Impact
Pile check

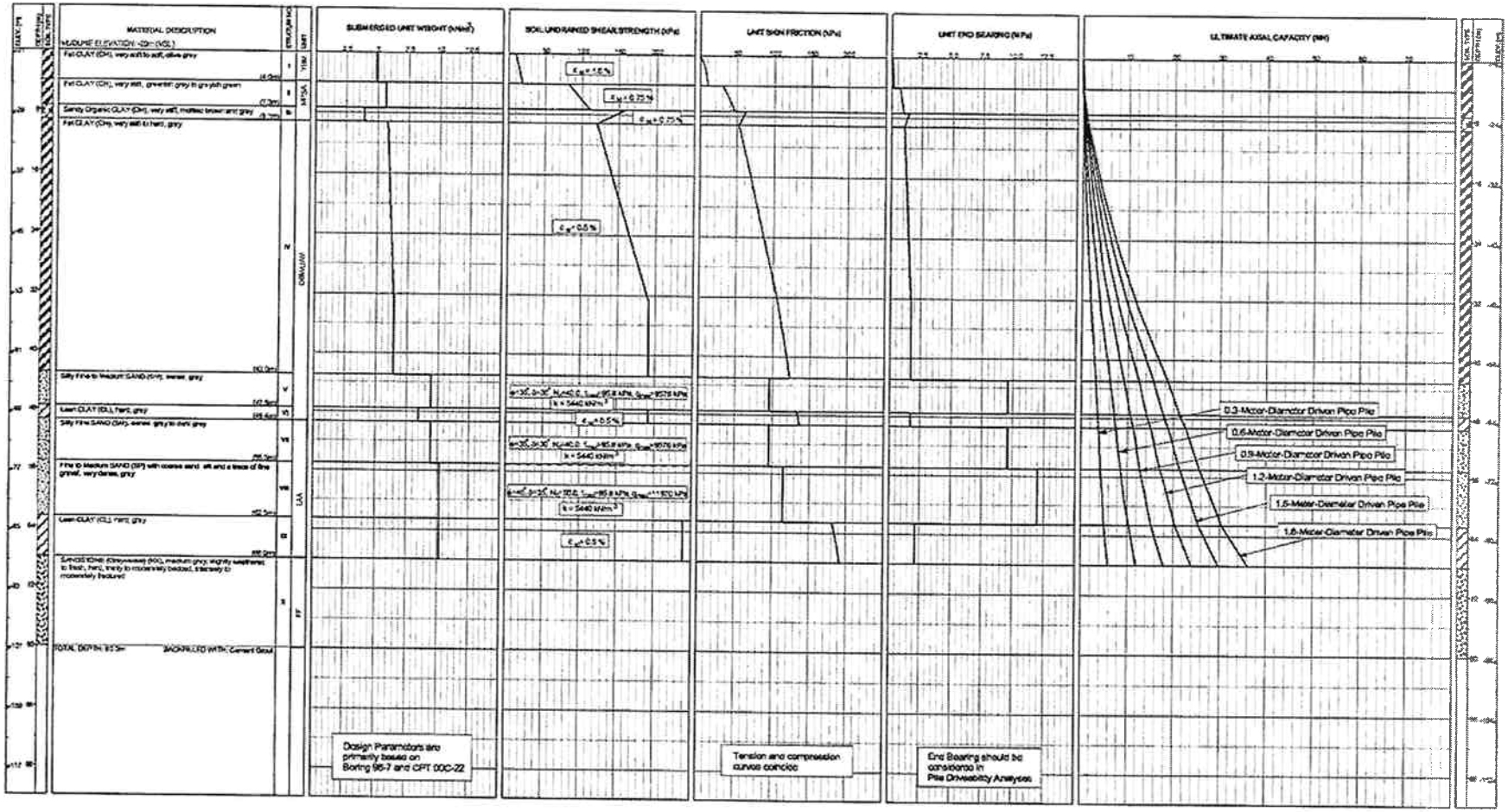
LC8 Y direction Broadside

		Area	I	S	r	Z	
42	1.25	160.0	33,248	1,583	14.41	2,058	23.5 m
							77.1 ft

11,236	kN	10,836	kN-m	l/r	64.19
1,146.53	Tonnes				
2,527	kips/ leg	7,818	kip-ft	Cc =	107
	15.79 ksi	59.26	ksi	Fy =	50 ksi
		75.05	ksi total		
				alpha =	0.85
API D.3.2-2					
1	- 0.83	+	0.96	Fcn =	41.0 ksi
		=	1.13	Fey =	69.5 ksi
			NG		
				Cm =	0.72
Check without Capacity Reduction Factors					
	15.79 ksi	59.26		Fxe =	518 ksi
		75.05			
				Fxc =	50 ksi
1	- 0.88	+	0.91		
					0.98
		=	1.03		
Pile is "essentially elastic"			say OK	Fbn =	63.7 ksi

Impact		LC9	X direction	End on						
Pile check				Area	I	S	r	Z		
	42		1.25	160.0	33,248	1,583	14.41	2,058	23.5	m
									77.1	ft
	9,000	kN			8,776	kN-m		l/r	64.19	
	918.37	Tonnes								
	2,024	kips/ leg			6,332	kip-ft		Cc =	107	
			12.65	ksi	47.99	ksi		Fy =	50	ksi
					60.64	ksi total				
								alpha =	0.85	
	API D.3.2-2									
	1		-	0.89	+	0.78		Fcn =	41.0	ksi
					=	0.88		Fey =	69.5	ksi
						OK				
								Cm =	0.72	
								Fxe =	518	ksi
								Fxc =	50	ksi
									0.98	
								Fbn =	63.7	ksi

Impact		LC10	45 degrees							
Pile check				Area	I	S	r	Z		
	42		1.25	160.0	33,248	1,583	14.41	2,058	23.5	m
									77.1	ft
	10,914	kN			8,556	kN-m		l/r	64.19	
	1,113.67	Tonnes								
	2,455	kips/ leg			6,173	kip-ft		Cc =	107	
			15.34	ksi	46.79	ksi		Fy =	50	ksi
					62.13	ksi total				
								alpha =	0.85	
	API D.3.2-2									
	1		-	0.84	+	0.76		Fcn =	41.0	ksi
					=	0.91		Fey =	69.5	ksi
						OK				
								Cm =	0.72	
								Fxe =	518	ksi
								Fxc =	50	ksi
									0.98	
								Fbn =	63.7	ksi



AXIAL PILE DESIGN PARAMETERS AND RESULTS
Main Span Temporary Tower G (Westbound)
0.3-, 0.6-, 0.9-, 1.2-, 1.5- and 1.8-Meter-Diameter Pipe Piles

PLATE 7.12a

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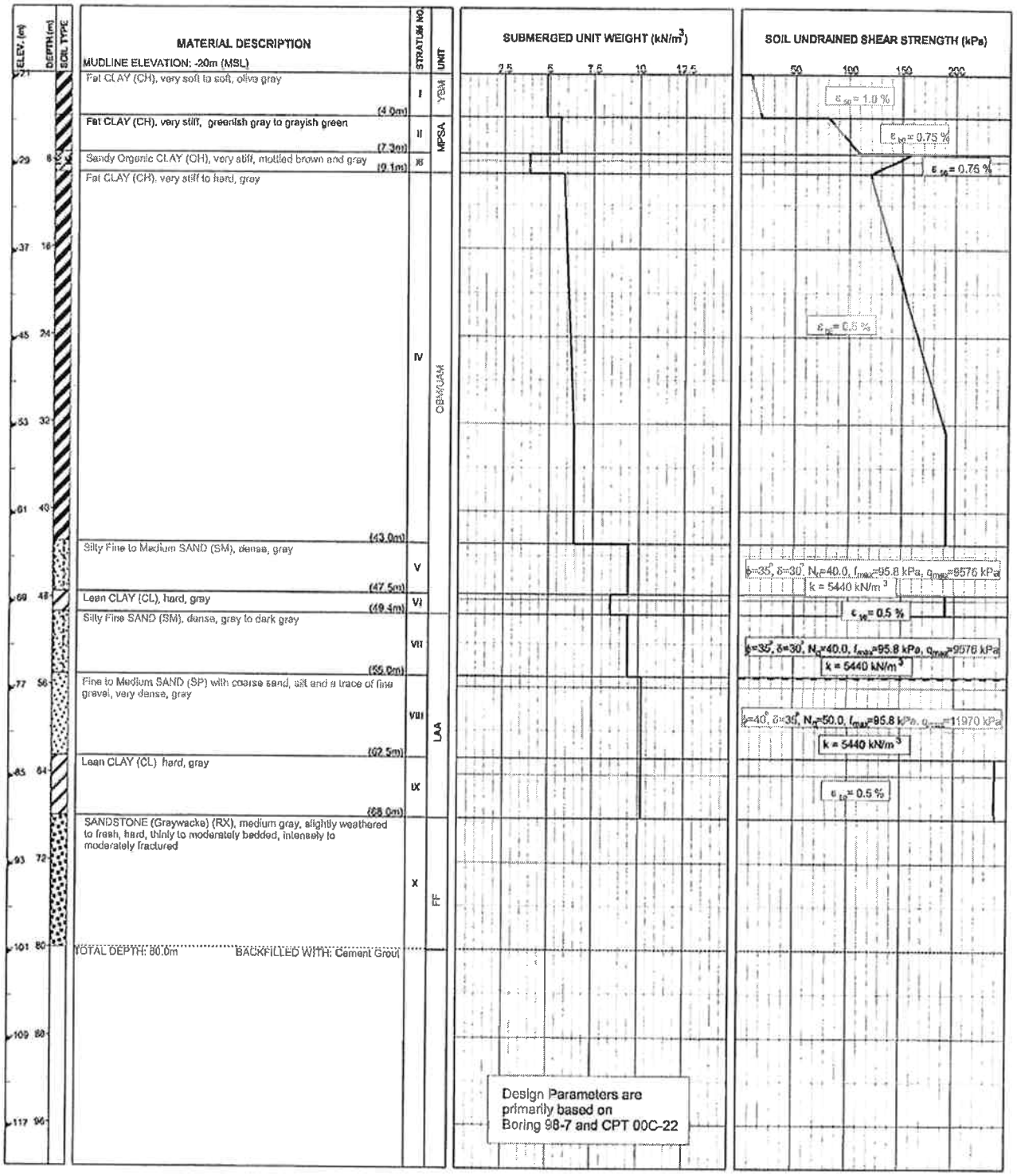
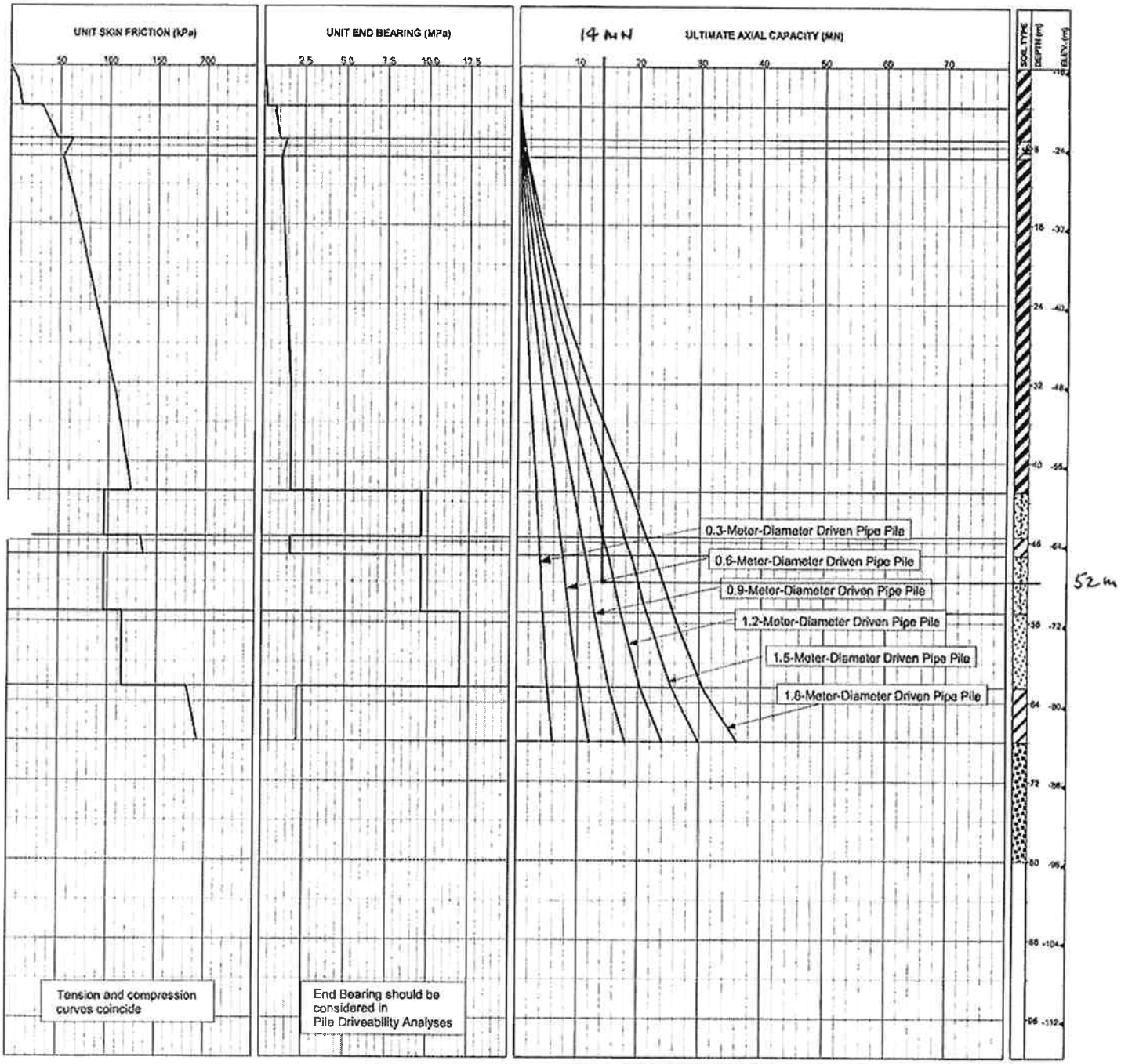
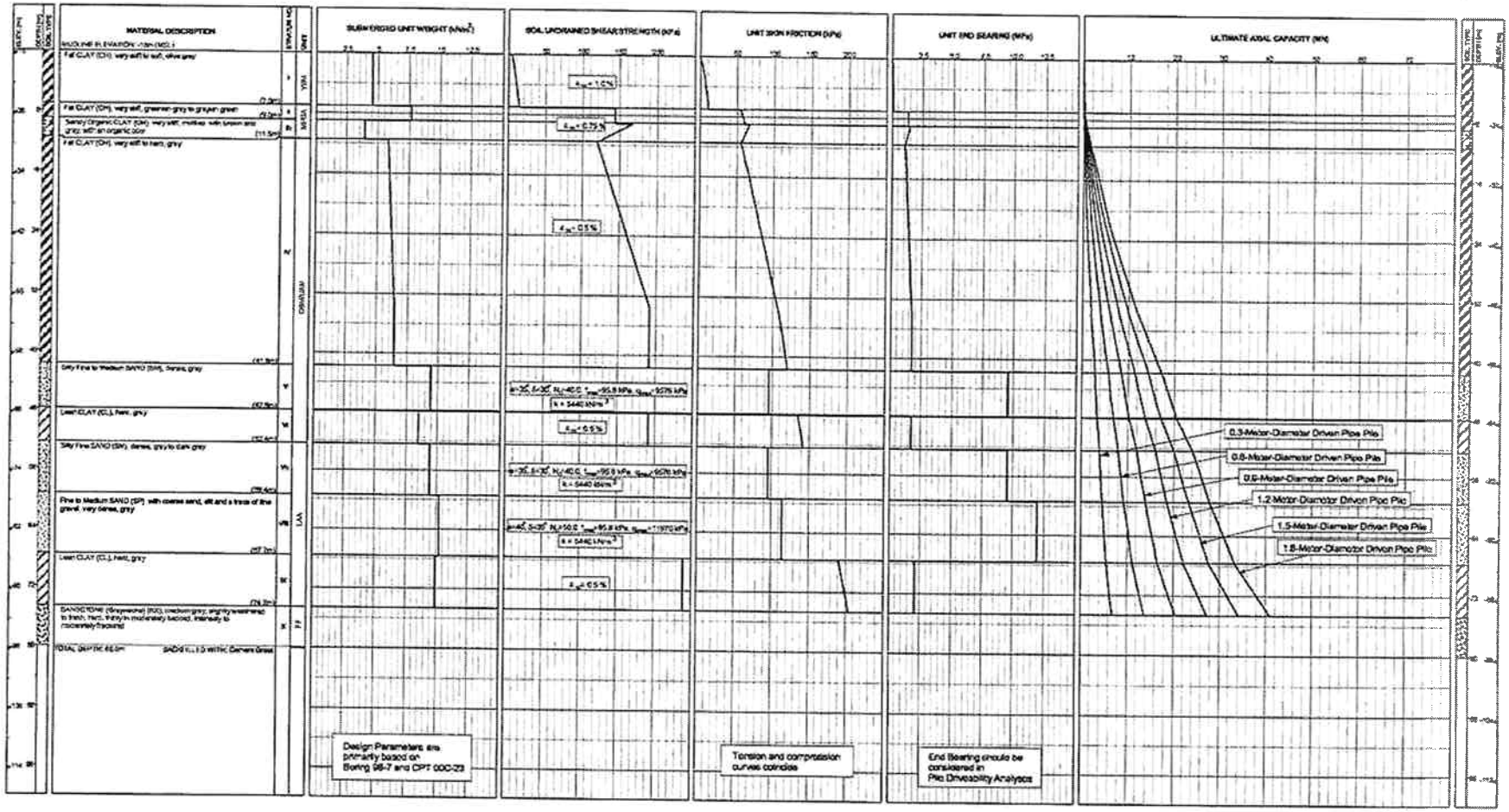

ELEV. (m)
DEPTH (m)
SOIL TYPE
MATERIAL DESCRIPTION
MUDLINE ELEVATION: -20m (MSL)
STRATUM NO.
UNIT
Fat CLAY (CH), very soft to soft, olive gray
(4.0m)
Fat CLAY (CH), very stiff, greenish gray to grayish green
(7.3m)
Sandy Organic CLAY (CH), very stiff, mottled brown and gray
(9.1m)
Fat CLAY (CH), very stiff to hard, gray
(43.0m)
Silty Fine to Medium SAND (SM), dense, gray
(47.5m)
Lean CLAY (CL), hard, gray
(49.4m)
Silty Fine SAND (SM), dense, gray to dark gray
(55.0m)
Fine to Medium SAND (SP) with coarse sand, silt and a trace of fine gravel, very dense, gray
(62.5m)
Lean CLAY (CL) hard, gray
(68.0m)
SANDSTONE (Graywacke) (RX), medium gray, slightly weathered to fresh, hard, thinly to moderately bedded, intensely to moderately fractured
TOTAL DEPTH: 80.0m
BACKFILLED WITH: Cement Grout
I
II
III
IV
V
VI
VII
VIII
IX
X
YBM
MPSA
OBM/UAM
LAA
FF
SUBMERGED UNIT WEIGHT (kN/m³)
2.5
5
7.5
10
12.5
SOIL UNDRAINED SHEAR STRENGTH (kPa)
50
100
150
200
ε50 = 1.0 %
ε50 = 0.75 %
ε50 = 0.75 %
ε50 = 0.5 %
φ=35°, δ=30°, Nq=40.0, fmax=95.8 kPa, qmax=9576 kPa
k = 5440 kN/m³
ε50 = 0.5 %
φ=35°, δ=30°, Nq=40.0, fmax=95.8 kPa, qmax=9576 kPa
k = 5440 kN/m³
φ=40°, δ=35°, Nq=50.0, fmax=95.8 kPa, qmax=11970 kPa
k = 5440 kN/m³
ε50 = 0.5 %
Design Parameters are primarily based on Boring 98-7 and CPT 00C-22

PILE PENETRATION REQ'D



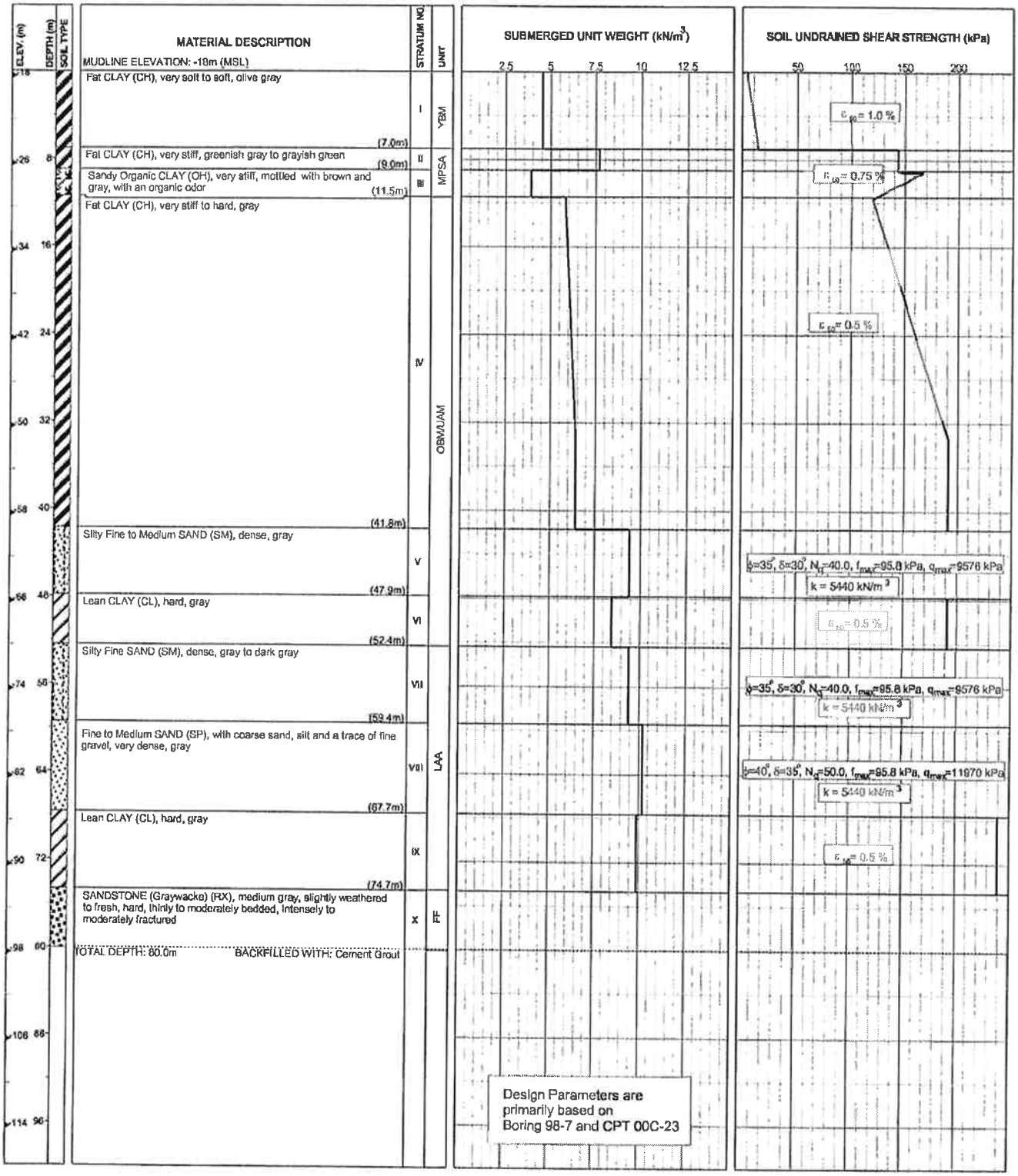
AXIAL PILE DESIGN PARAMETERS AND RESULTS
Main Span Temporary Tower G (Westbound)
0.3-, 0.6-, 0.9-, 1.2-, 1.5- and 1.8-Meter-Diameter Pipe Piles

PLATE 7.12a

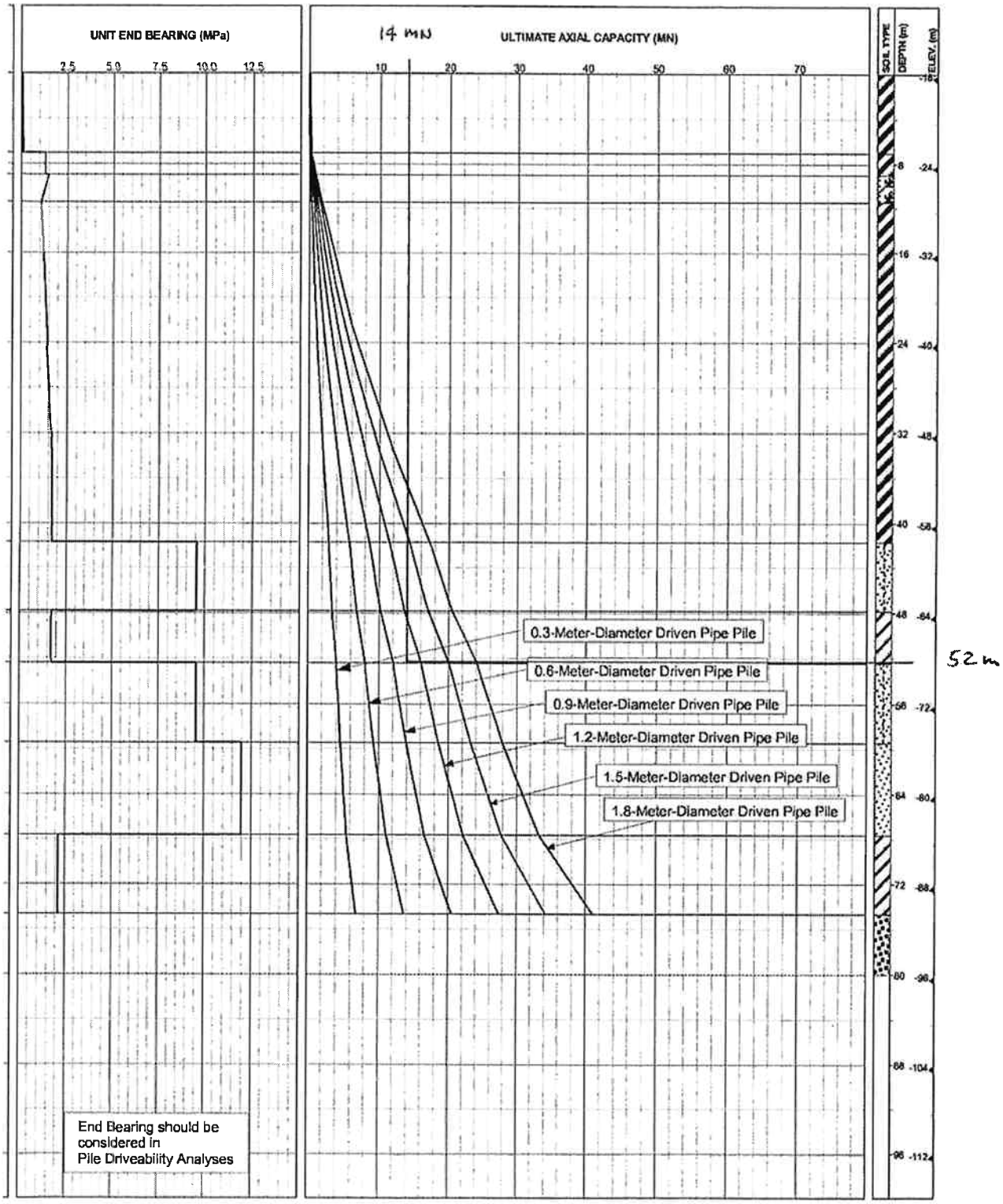


AXIAL PILE DESIGN PARAMETERS AND RESULTS
Main Span Temporary Tower G (Eastbound)
0.3-, 0.6-, 0.9-, 1.2-, 1.5- and 1.8-Meter-Diameter Pipe Piles

PLATE 7.12b


ELEV. (m)
DEPTH (m)
SOIL TYPE
MATERIAL DESCRIPTION
MUDLINE ELEVATION: -18m (MSL)
STRATUM NO.
UNIT
Fat CLAY (CH), very soft to soft, olive gray
(7.0m)
Fat CLAY (CH), very stiff, greenish gray to grayish green
(9.0m)
Sandy Organic CLAY (OH), very stiff, mottled with brown and gray, with an organic odor
(11.5m)
Fat CLAY (CH), very stiff to hard, gray
(41.8m)
Silty Fine to Medium SAND (SM), dense, gray
(47.9m)
Lean CLAY (CL), hard, gray
(52.4m)
Silty Fine SAND (SM), dense, gray to dark gray
(59.4m)
Fine to Medium SAND (SP), with coarse sand, silt and a trace of fine gravel, very dense, gray
(67.7m)
Lean CLAY (CL), hard, gray
(74.7m)
SANDSTONE (Graywacke) (RX), medium gray, slightly weathered to fresh, hard, thinly to moderately bedded, intensely to moderately fractured
TOTAL DEPTH: 80.0m
BACKFILLED WITH: Cement Grout
I
II
III
IV
V
VI
VII
VIII
IX
X
YBM
MPSA
OBM/UAM
LAA
FF
SUBMERGED UNIT WEIGHT (kN/m³)
2.5
5
7.5
10
12.5
SOIL UNDRAINED SHEAR STRENGTH (kPa)
50
100
150
200
ε50 = 1.0 %
ε50 = 0.75 %
ε50 = 0.5 %
φ=35°, δ=30°, Nq=40.0, fmax=95.8 kPa, qmax=9576 kPa
k = 5440 kN/m³
ε50 = 0.5 %
φ=35°, δ=30°, Nq=40.0, fmax=95.8 kPa, qmax=9576 kPa
k = 5440 kN/m³
φ=40°, δ=35°, Nq=50.0, fmax=95.8 kPa, qmax=11970 kPa
k = 5440 kN/m³
ε50 = 0.5 %
Design Parameters are primarily based on Boring 98-7 and CPT 00C-23

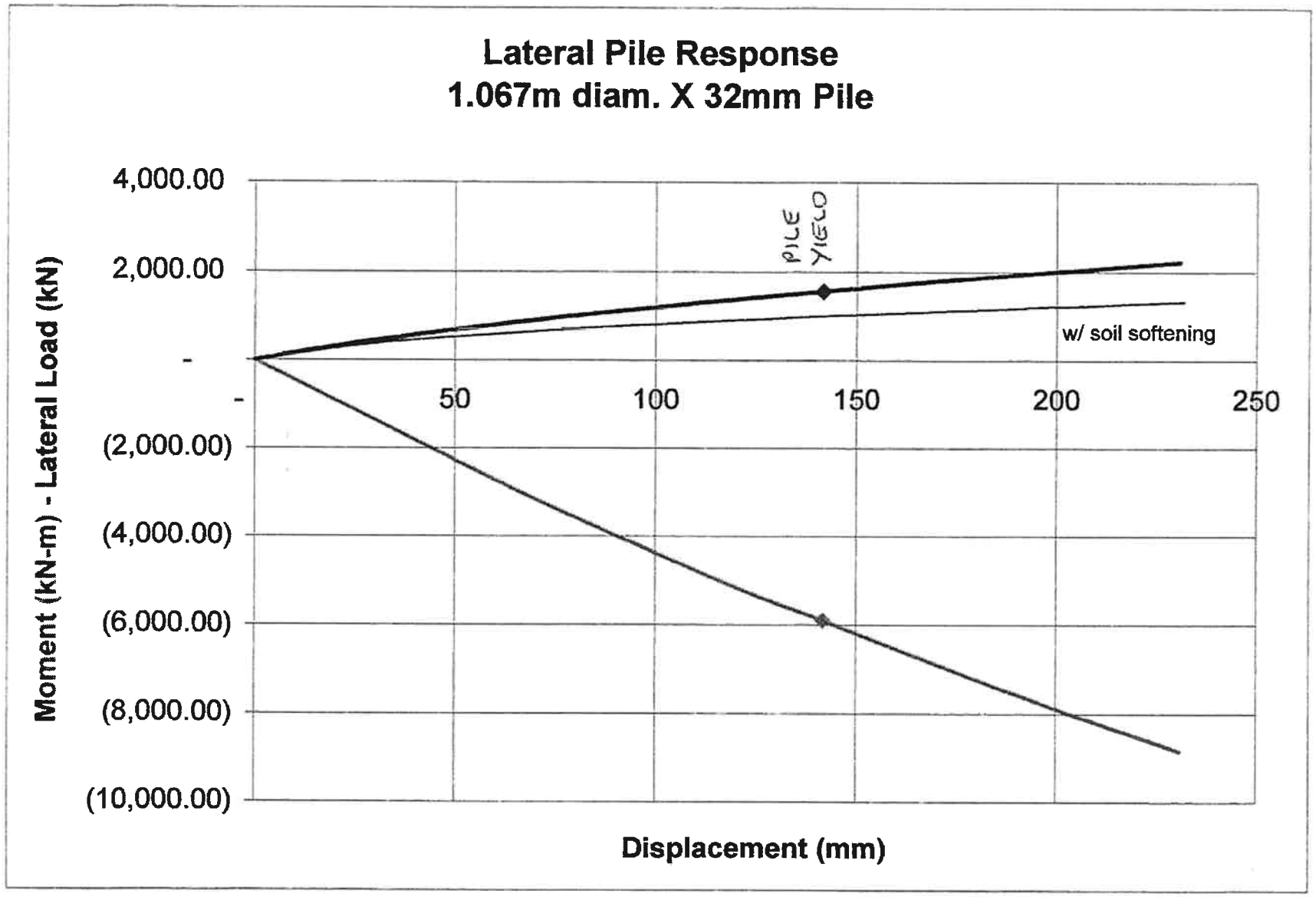
PILE PENETRATION REQ'D



AXIAL PILE DESIGN PARAMETERS AND RESULTS
Main Span Temporary Tower G (Eastbound)
0.3-, 0.6-, 0.9-, 1.2-, 1.5- and 1.8-Meter-Diameter Pipe Piles

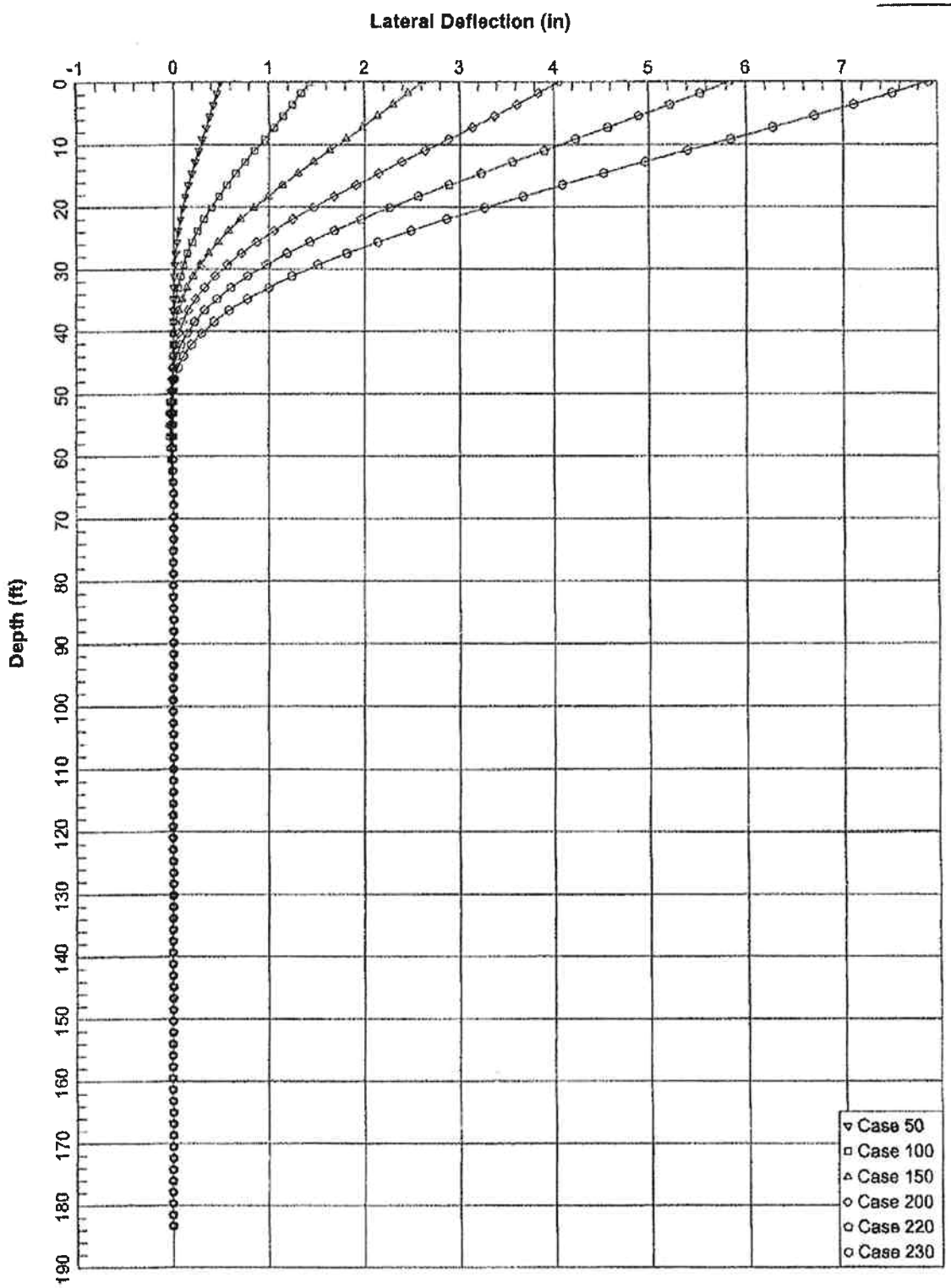
PLATE 7.12b

Axial Pile Response
1.067m diam. x 32mm Pile
Axial Load (MN)
20
18
16
14
12
10
8
6
4
2
-
0
10
20
30
40
50
60
70
80
90
100
Axial Displacement (mm)
API basic soil critreria
w/out API limits

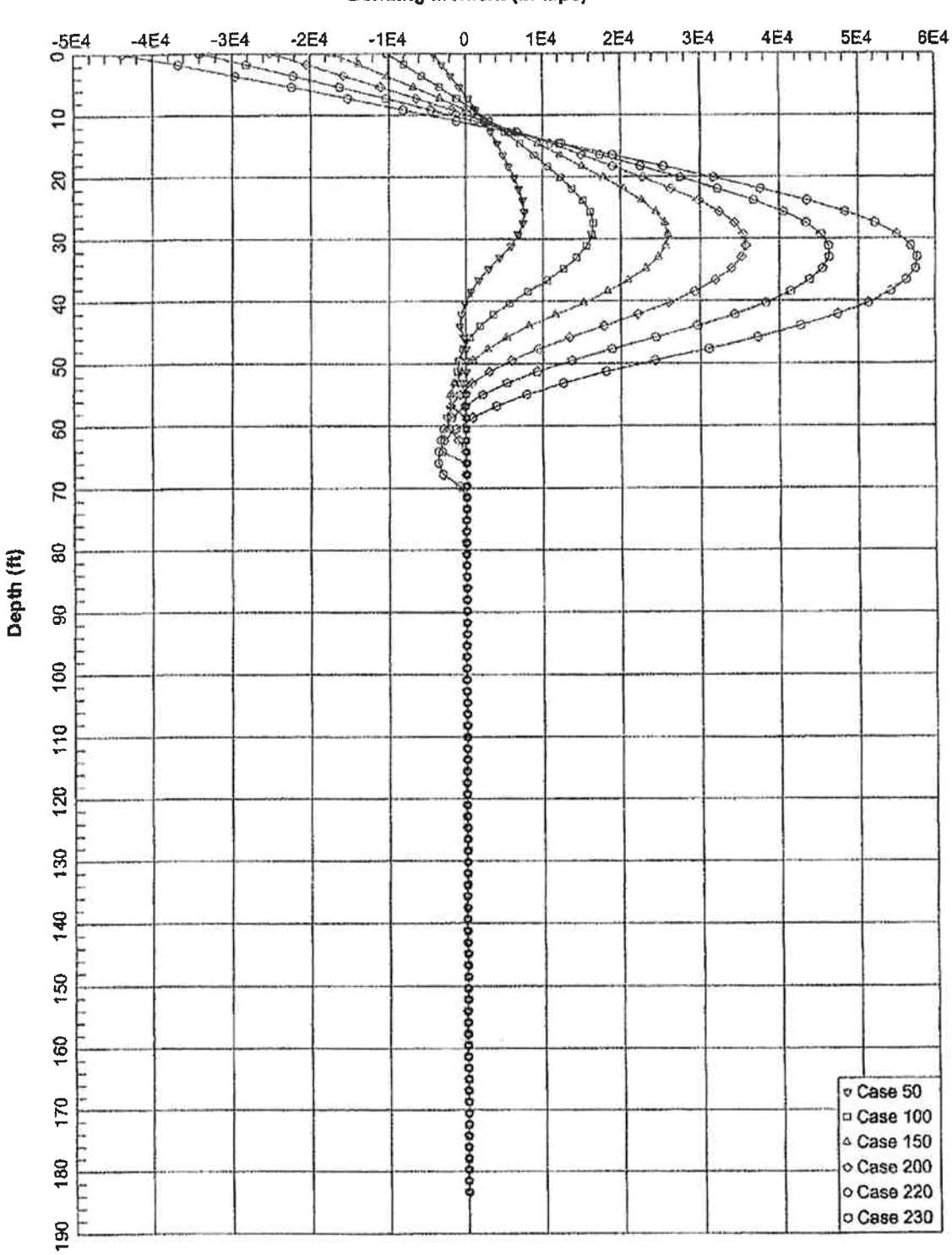

Lateral Pile Response
1.067m diam. X 32mm Pile
4,000.00
2,000.00
-
(2,000.00)
(4,000.00)
(6,000.00)
(8,000.00)
(10,000.00)
Moment (kN-m) - Lateral Load (kN)
50
100
150
200
250
Displacement (mm)
PILE YIELD
w/ soil softening

L PILE RESULTS

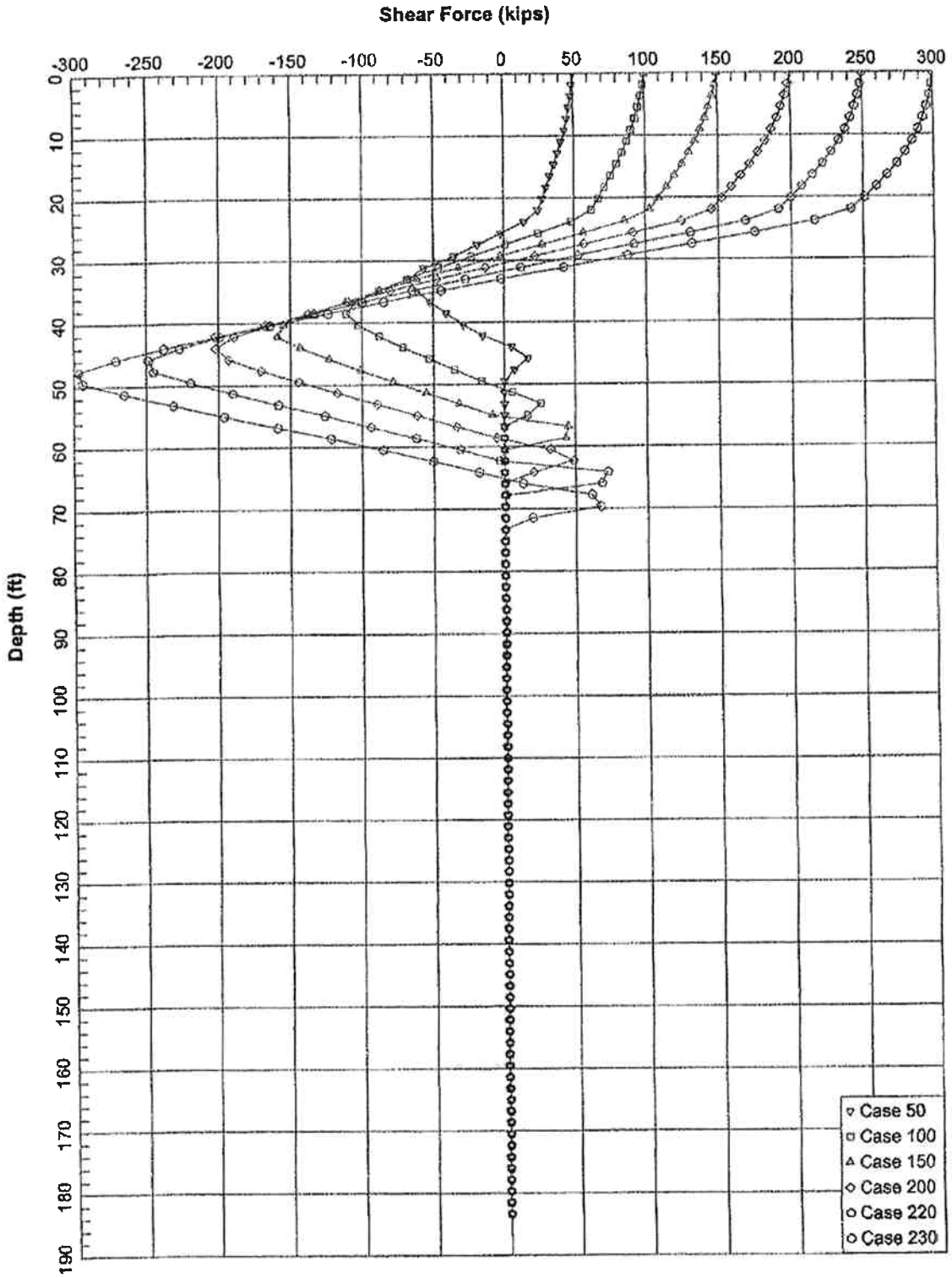
Temporary Tower CLAY W/O FREE WATER 42-in. x 1.25-in. H= 50, 100, 150, 200, 250, 300 kip N=1500 kip



Temporary Tower CLAY W/O FREE WATER 42-in. x 1.25-in. H= 50, 100, 150, 200, 250, 300 kip N=1500 kip

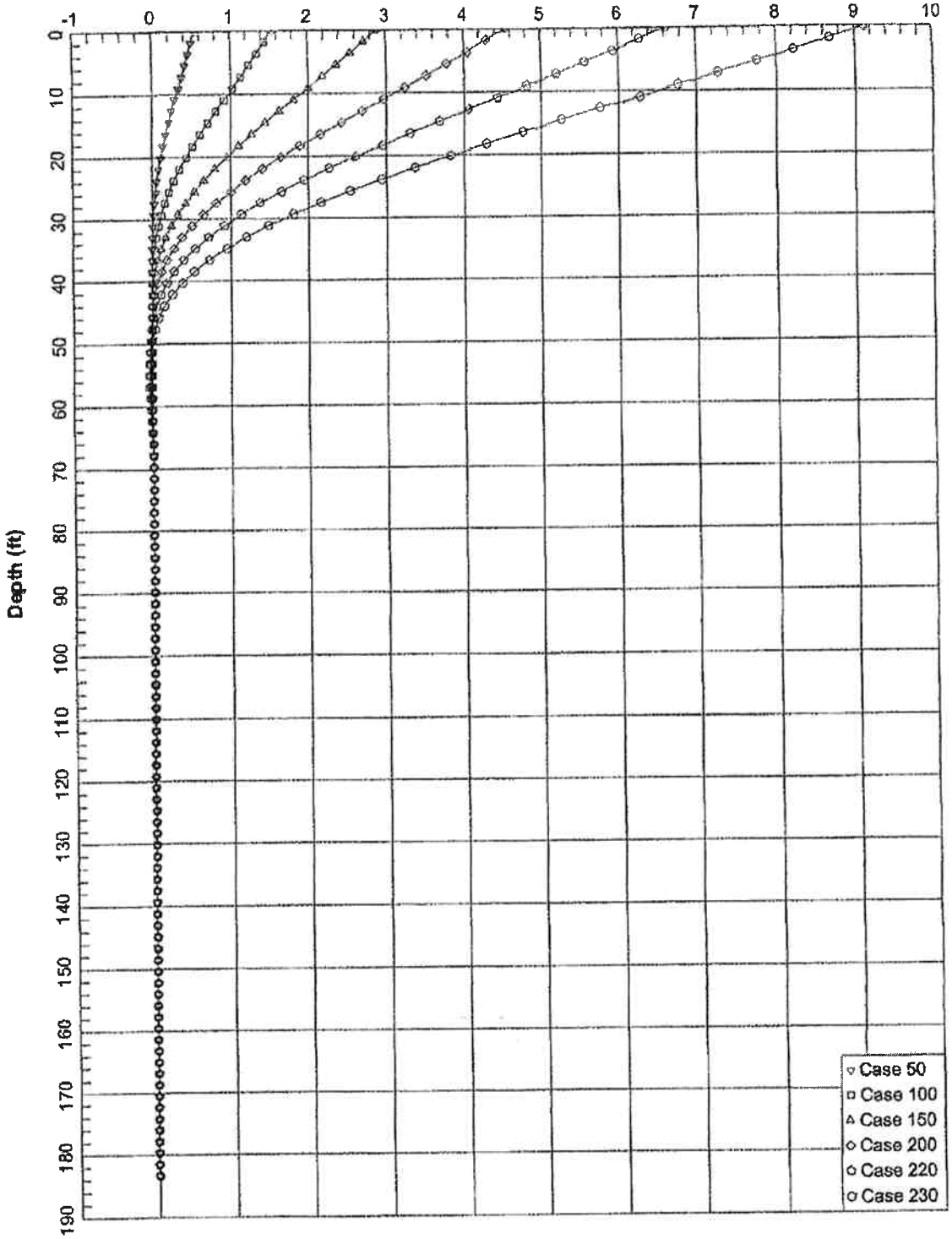


Temporary Tower CLAY W/O FREE WATER 42-in. x 1.25-in. H= 50, 100, 150, 200, 250, 300 kip N=1500 kip

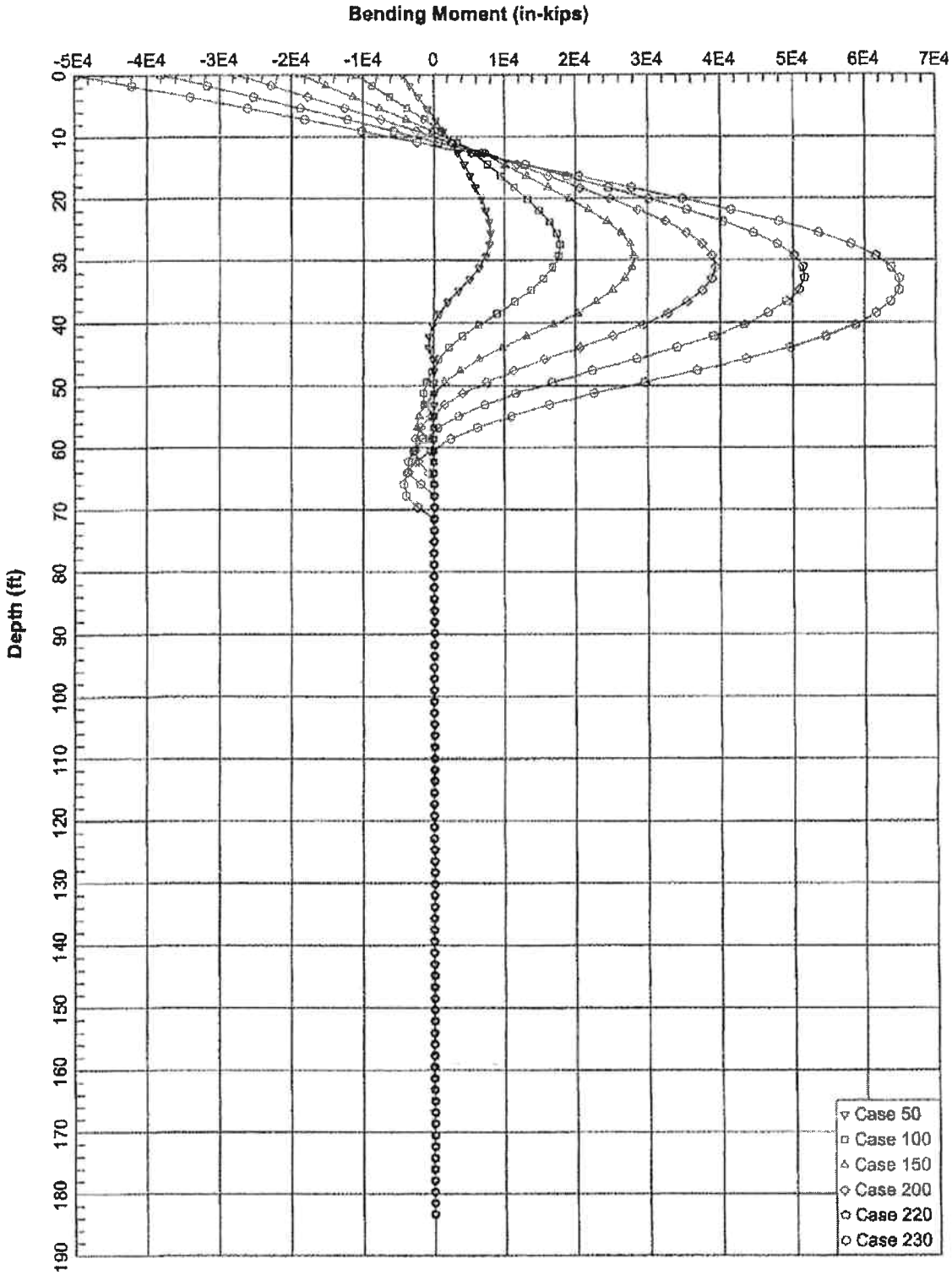


LPILE

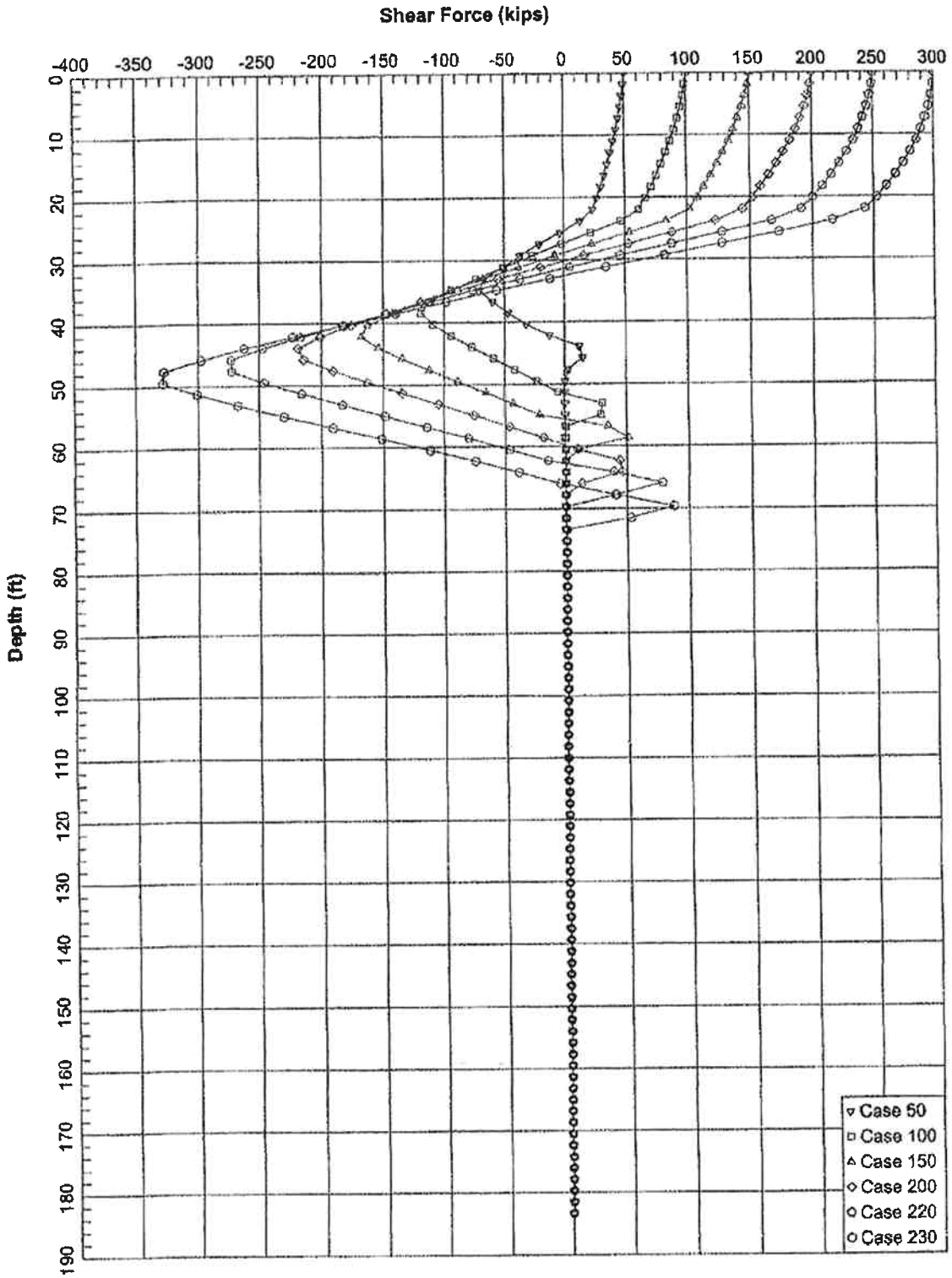
Temporary Tower CLAY W/O FREE WATER 42-in. x 1.25-in. H= 50, 100, 150, 200, 250, 300 kip N=3000 kip



Temporary Tower CLAY W/O FREE WATER 42-in. x 1.25-in. H= 50, 100, 150, 200, 250, 300 kip N=3000 kip



Temporary Tower CLAY W/O FREE WATER 42-in. x 1.25-in. H= 50, 100, 150, 200, 250, 300 kip N=3000 kip



ULTIMATE PILE CAPACITY - Cohesive Soil

(Per API RP 2A)

1 Enter pile diameter 42.00 in — 10.996 ft^2 Surface area/ft — 1.325 ft^2 End area — 8.296 ft^2 End area (plugged)

thickness 1.50 in

2. Enter the initial shear strength at the mudline

Co = 0.000 ksf

NOTE:
All units in Kips and ft unless noted.

3. Enter the increasing shear strength with depth

c = 0.000 ksf / ft

4. Depth may be modified to suit

	Depth	C ksf	GAMMA pcf	P'o ksf	EP	alpha	ALPHA	friction ksf	p kips	P-Ult Kips
0.0	0.0	0.10	28.7	0.00	0.00	0.00	0.00	0.00	0	0
23.0	23.0	0.31	28.7	0.66	0.48	0.72	0.72	0.23	29	29
0.0	23.0	2.96	49.0	0.66	4.50	0.34	0.34	1.02	0	29
6.6	29.5	2.96	49.0	0.98	3.02	0.38	0.38	1.12	77	106
0.0	29.5	3.51	25.5	0.98	3.58	0.36	0.36	1.27	0	106
8.2	37.7	2.50	25.5	1.19	2.11	0.42	0.42	1.04	104	210
0.0	37.7	2.50	36.9	1.19	2.11	0.42	0.42	1.04	0	210
67.3	105.0	3.97	41.4	3.97	1.00	0.50	0.50	1.98	1118	1328
32.2	137.1	3.97	41.4	5.30	0.75	0.58	0.58	2.29	756	2084
0.0	137.1	3.97	60.5	5.30	0.75	0.58	0.58	2.29	0	2084
20.0	157.2	3.97	60.5	6.51	0.61	0.64	0.64	2.54	532	2616
0.0	157.2	3.97	54.1	6.51	0.61	0.64	0.64	2.54	0	2616
14.8	171.9	3.97	54.1	7.31	0.54	0.68	0.68	2.69	425	3041
0.0	171.9	3.97	60.5	7.31	0.54	0.68	0.68	2.69	0	3041
23.0	194.9	3.97	60.5	8.70	0.46	0.74	0.74	2.94	711	3752

FUGRO/EMI Raw Data

Depth m	Depth ft	Gamma kN/m^3	Gamma pcf	C kPa	C ksf
0.0	0.0	4.50	28.7	5	0.10
7.0	23.0	4.50	28.7	15	0.31
7.0	23.0	7.70	49.0	142	2.96
9.0	29.5	7.70	49.0	142	2.96
9.0	29.5	4.00	25.5	168	3.51
11.5	37.7	4.00	25.5	120	2.50
11.5	37.7	5.80	36.9	120	2.50
32.0	105.0	6.50	41.4	190	3.97
41.8	137.1	6.50	41.4	190	3.97
41.8	137.1	9.50	60.5	35 deg sand	
47.9	157.2	9.50	60.5	35 deg sand	
47.9	157.2	8.50	54.1	190	3.97
52.4	171.9	8.50	54.1	190	3.97
52.4	171.9	9.50	60.5	35 deg sand	
59.4	194.9	9.50	60.5	35 deg sand	

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API SOIL CALCS

ULTIMATE PILE CAPACITY - Cohesive Soil

Depth	α ksf	Q-Ult Kips Unplugged	Q-Ult Kips Plugged	Tension	Unplugged	Plugged	Depth	
0.0	0.9	1	8	0	1	8	0.0	Unplugged
23.0	2.8	4	23	29	59	52	23.0	Plugged
23.0	26.7	35	221	29	91	250	23.0	Unplugged
29.5	26.7	35	221	106	240	327	29.5	Unplugged
29.5	31.6	42	262	106	246	368	29.5	Unplugged
37.7	22.5	30	187	210	435	397	37.7	Plugged
37.7	22.5	30	187	210	435	397	37.7	Plugged
105.0	35.7	47	296	1,328	2,609	1,624	105.0	Plugged
137.1	35.7	47	296	2,084	4,067	2,381	137.1	Plugged
137.1	35.7	47	296	2,084	4,067	2,381	137.1	Plugged
157.2	35.7	47	296	2,616	5,093	2,912	157.2	Plugged
157.2	35.7	47	296	2,616	5,093	2,912	157.2	Plugged
171.9	35.7	47	296	3,041	5,912	3,337	171.9	Plugged
171.9	35.7	47	296	3,041	5,912	3,337	171.9	Plugged
194.9	35.7	47	296	3,752	7,283	4,048	194.9	Plugged

T-Z Curves

lbs/ft (of depth) @ Z displ. (in inches)

DEPTH	T1	T2	T3	T4	T5	
feet	0.067	0.130	0.239	0.336	0.420	
0.0	0	0	0	0	0	
23.0	253	842	1,684	2,246	2,496	
23.0	1,132	3,775	7,550	10,066	11,185	
29.5	1,251	4,170	8,340	11,119	12,355	
29.5	1,419	4,730	9,460	12,614	14,015	
37.7	1,157	3,857	7,714	10,286	11,429	
37.7	1,157	3,857	7,714	10,286	11,429	
105.0	2,209	7,365	14,729	19,639	21,821	
137.1	2,553	8,509	17,019	22,692	25,213	
137.1	2,553	8,509	17,019	22,692	25,213	
157.2	2,829	9,431	18,862	25,149	27,943	
157.2	2,829	9,431	18,862	25,149	27,943	
171.9	2,998	9,992	19,985	26,647	29,607	
171.9	2,998	9,992	19,985	26,647	29,607	
194.9	3,270	10,900	21,801	29,068	32,297	3,147

P-Y Curves

lbs/in (of depth) @ Y displ. (in inches)

DEPTH	Pus	Pud	P1	P2	P3
feet			0.53	1.58	4.20
0.0	91	274	46	66	91
23.0	765	822	383	551	765
23.0	5,757	7,779	2,879	4,145	5,757
29.5	6,661	7,779	3,331	4,796	6,661
29.5	7,601	9,203	3,800	5,473	7,601
37.7	6,409	6,574	3,204	4,614	6,409
37.7	6,535	6,574	3,267	4,705	6,535
105.0	22,083	10,409	5,204	7,494	10,409
137.1	27,783	10,409	5,204	7,494	10,409
137.1	28,547	10,409	5,204	7,494	10,409
157.2	32,207	10,409	5,204	7,494	10,409
157.2	31,915	10,409	5,204	7,494	10,409
171.9	34,587	10,409	5,204	7,494	10,409
171.9	34,906	10,409	5,204	7,494	10,409
194.9	39,106	10,409	5,204	7,494	10,409

Q-Z Curves

lbs @ Z displ. (in inches)

DEPTH	T1	T2	T3	T4	T5
feet	0.084	0.546	1.76	3.07	4.20
0.0	0	0	0	0	0
23.0	1,972	7,888	15,776	21,035	23,372
23.0	18,669	74,675	149,350	199,133	221,259
29.5	18,669	74,675	149,350	199,133	221,259
29.5	22,087	88,348	176,696	235,594	261,771
37.7	15,776	63,106	126,211	168,281	186,979
37.7	15,776	63,106	126,211	168,281	186,979
105.0	24,979	99,917	199,834	266,446	296,051
137.1	24,979	99,917	199,834	266,446	296,051
137.1	24,979	99,917	199,834	266,446	296,051
157.2	24,979	99,917	199,834	266,446	296,051
157.2	24,979	99,917	199,834	266,446	296,051
171.9	24,979	99,917	199,834	266,446	296,051
171.9	24,979	99,917	199,834	266,446	296,051
194.9	24,979	99,917	199,834	266,446	296,051

T-Z Curves

0.42	Z1	Z2	Z3	Z4	Z5
z/D	0.16	0.31	0.57	0.80	1
z	0.07	0.13	0.24	0.34	0.42
t/T	0.30	0.50	0.75	0.90	1

Q-Z Curves

4.2	Z1	Z2	Z3	Z4	Z5
z/D	0.02	0.13	0.42	0.73	1
z	0.08	0.55	1.76	3.07	4.20
q/Q	0.25	0.5	0.75	0.9	1

P-Y Curves

	Y1	Y2	Y3
y/Y	1.00	3.00	8.00
y/Y	0.53	1.58	4.20
p/P	0.50	0.72	1.00

ULTIMATE PILE CAPACITY - Cohesionless Soil

(Per API RP 2A)

1. Enter pile diameter 42.00 in — 10.996 ft^2 — 1.325 ft^2 — 8.296 ft^2
   thickness 1.50 in — Surface area/ft — Tip area — Plug area

2. Enter soil description, into table below, from the following (by depth)

VERY LOOSE SAND	MEDIUM SAND
LOOSE SAND-SILT	DENSE SAND-SILT
MEDIUM SILT	DENSE SAND
LOOSE SAND	VERY DENSE SAND-SILT
MEDIUM SAND-SILT	DENSE GRAVEL
DENSE SILT	VERY DENSE SAND

NOTE:
All units in Kips and ft unless noted.

3. Depth may be modified to suit

								Compression Ko = 0.8		Tension Ko = 0.8		
									Friction		Friction	
	Depth	Description	Angle	Limit f	Nq	Limit q	Gamma	Po	fc	Fc	ft	Ft
0.0	0.0	DENSE SAND	30	2.0	40	200	28.7	0.00	0.00	0.00	0.00	0.00
23.0	23.0	DENSE SAND	30	2.0	40	200	28.7	0.66	0.30	0.30	0.30	0.30
0.0	23.0	DENSE SAND	30	2.0	40	200	49.0	0.66	0.30	0.30	0.30	0.30
6.6	29.5	DENSE SAND	30	2.0	40	200	49.0	0.98	0.45	0.45	0.45	0.45
0.0	29.5	DENSE SAND	30	2.0	40	200	25.5	0.98	0.45	0.45	0.45	0.45
8.2	37.7	DENSE SAND	30	2.0	40	200	25.5	1.19	0.55	0.55	0.55	0.55
0.0	37.7	DENSE SAND	30	2.0	40	200	36.9	1.19	0.55	0.55	0.55	0.55
67.3	105.0	DENSE SAND	30	2.0	40	200	41.4	3.97	1.84	1.84	1.84	1.84
32.2	137.1	DENSE SAND	30	2.0	40	200	41.4	5.30	2.45	2.00	2.45	2.00
0.0	137.1	DENSE SAND	30	2.0	40	200	60.5	5.30	2.45	2.00	2.45	2.00
20.0	157.2	DENSE SAND	30	2.0	40	200	60.5	6.51	3.01	2.00	3.01	2.00
0.0	157.2	DENSE SAND	30	2.0	40	200	54.1	6.51	3.01	2.00	3.01	2.00
14.8	171.9	DENSE SAND	30	2.0	40	200	54.1	7.31	3.38	2.00	3.38	2.00
0.0	171.9	DENSE SAND	30	2.0	40	200	60.5	7.31	3.38	2.00	3.38	2.00
23.0	194.9	DENSE SAND	30	2.0	40	200	60.5	8.70	4.02	2.00	4.02	2.00

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API SOIL CALCS

ULTIMATE PILE CAPACITY - Cohesionless Soil

Depth	Bearing q	Q	Comp Friction	Compressior	Tension Friction	Tension	Bearing Annulus	Bearing Plug	Unplugged	Plugged	Depth	
0.0	0	0	0	0	0	0	0	0	0	0	0.0	
23.0	26	26	38	38	38	38	35	218	109	292	23.0	Unplugged
23.0	26	26	0	38	0	38	35	218	109	292	23.0	Unplugged
29.5	39	39	27	66	27	66	52	325	179	443	29.5	Unplugged
29.5	39	39	0	66	0	66	52	325	179	443	29.5	Unplugged
37.7	48	48	45	111	45	111	63	395	277	568	37.7	Unplugged
37.7	48	48	0	111	0	111	63	395	277	568	37.7	Unplugged
105.0	159	159	882	992	882	992	211	1318	2125	2521	105.0	Unplugged
137.1	212	200	678	1,670	678	1670	265	1659	3486	3595	137.1	Unplugged
137.1	212	200	0	1,670	0	1670	265	1659	3486	3595	137.1	Unplugged
157.2	261	200	440	2,110	440	2110	265	1659	4335	4035	157.2	Plugged
157.2	261	200	0	2,110	0	2110	265	1659	4335	4035	157.2	Plugged
171.9	293	200	325	2,435	325	2435	265	1659	4961	4359	171.9	Plugged
171.9	293	200	0	2,435	0	2435	265	1659	4961	4359	171.9	Plugged
194.9	348	200	505	2,940	505	2940	265	1659	5935	4864	194.9	Plugged

T-Z Curves lbs/in (of depth) @ Z displ. (in inches)

DEPTH feet	T1 0.42
0.0	0
23.0	3,343
23.0	3,343
29.5	4,977
29.5	4,977
37.7	6,038
37.7	6,038
105.0	20,178
137.1	26,937
137.1	26,937
157.2	33,086
157.2	33,086
171.9	37,145
171.9	37,145
194.9	44,202

P-Y Curves lbs/in (of depth) @ Y displ. (in inches)

DEPTH feet	Pus	Pud	P1 0.525	P2 1.050	P3 2.100	P4 4.200
0.0	0	0	0	0	0	0
23.0	2,955	5,375	2,464	2,652	2,660	2,660
23.0	2,955	5,375	2,464	2,652	2,660	2,660
29.5	8,046	11,825	4,687	6,606	7,211	7,241
29.5	8,046	11,825	4,687	6,606	7,211	7,241
37.7	6,622	7,849	4,962	5,861	5,959	5,959
37.7	6,622	7,849	4,962	5,861	5,959	5,959
105.0	69,100	31,670	17,156	25,187	28,286	28,502
137.1	130,815	46,362	23,030	35,305	41,150	41,722
137.1	130,815	46,362	23,030	35,305	41,150	41,722
157.2	250,013	77,648	28,033	48,295	65,370	69,728
157.2	250,013	77,648	28,033	48,295	65,370	69,728
171.9	267,038	76,001	30,250	50,603	65,408	68,333
171.9	267,038	76,001	30,250	50,603	65,408	68,333
194.9	382,325	96,290	34,764	59,890	81,064	86,468

Q-Z Curves lbs @ Z displ. (in inches)

DEPTH feet	T1 0.06	T2 0.53	T3 1.76	T4 3.07	T5 4.20
0.0	0	0	0	0	0
23.0	126	8,370	66,962	159,432	218,400
23.0	126	8,370	66,962	159,432	218,400
29.5	187	12,462	99,698	237,377	325,174
29.5	187	12,462	99,698	237,377	325,174
37.7	227	15,119	120,956	287,990	394,507
37.7	227	15,119	120,956	287,990	394,507
105.0	758	50,527	404,214	962,414	1,318,375
137.1	954	63,587	508,696	1,211,182	1,659,154
137.1	954	63,587	508,696	1,211,182	1,659,154
157.2	954	63,587	508,696	1,211,182	1,659,154
157.2	954	63,587	508,696	1,211,182	1,659,154
171.9	954	63,587	508,696	1,211,182	1,659,154
171.9	954	63,587	508,696	1,211,182	1,659,154
194.9	954	63,587	508,696	1,211,182	1,659,154

T-Z Curves

0.42	Z1
z/Z	1
z	0.42
t/T	1

Q-Z Curves

4.20	Z1	Z2	Z3	Z4
z/D	0.015	0.125	0.42	0.73
z	0.06	0.53	1.76	3.07
q/Q	0.25	0.5	0.75	0.9

P-Y Curves

0	Z1	Z2	Z3
z/Z	0.19	0.36	0.58
z	0	0	0
t/T	0.5	0.7	0.85

Miscellaneous Calculations

, Tower C Alone Base Reactions

Joint	Load Case	F1 KN	F2 KN	F3 KN
25	TABLE1	322	146	2,577
30	TABLE1	(428)	579	6,314
71	TABLE1	428	(270)	2,934
74	TABLE1	(322)	(455)	5,958
		0	-	17,782
25	LIVE	4	5	72
30	LIVE	(6)	4	46
71	LIVE	6	(7)	76
74	LIVE	(4)	(2)	41
		0	-	235
25	TTWRX	572	386	4,952
30	TTWRX	571	388	4,964
71	TTWRX	573	389	4,963
74	TTWRX	571	387	4,950
		2,288	1,550	19,829
25	TTWRY	567	493	4,779
30	TTWRY	217	810	4,697
71	TTWRY	563	488	4,760
74	TTWRY	217	809	4,716
		1,564	2,599	18,953
25	WIND	(40)	(116)	(657)
30	WIND	64	(85)	(680)
71	WIND	40	(117)	657
74	WIND	(63)	(87)	680
		-	(405)	0

1.4DL		
450	204	3,608
(599)	810	8,840
599	(378)	4,107
(450)	(637)	8,341
1.1DL+1.3LL		
359	167	2,928
(478)	642	7,005
478	(306)	3,326
(359)	(503)	6,607
DL+LL+Eqx+0.3EQy		
1,068	685	9,035
203	1,213	**12,733**
1,176	259	9,401
310	172	12,364
DL+LL+Eqy+0.3EQx		
1,064	760	8,913
(45)	**1,509**	12,546
1,168	328	9,259
63	468	12,200
DL+LL+1.3Wind		
273	0	1,795
(351)	472	5,475
485	(429)	3,864
(408)	(570)	6,883
DL+LL-Eqx+0.3EQy		
(76)	(87)	**(870)**
(939)	438	2,805
29	(519)	(525)
(832)	(602)	2,463
DL+LL-Eqy+0.3EQx		
(69)	(225)	(645)
(479)	(111)	3,152
42	(648)	(262)
(372)	(1,150)	2,768

Maximum

12,733	kN	1,509	kN
2,864	kips	339	kips

use 8 41 in diam anchor bolts 82 in thick base plate

try SQ. 1.500 m footing q all = $\frac{4500}{B}$ = 3,000 kPa
4.92 ft 9,000 Ultimate

Footing Pressure 5,659 kPa OK

0.762 m diam leg Minimum footing thickness required 0.369 m

Vessel Impact Checks

Check for local Impact load 7,600 kN
1,700 kips

Fy = 345 MPa
50 ksi

E = 200,100 MPa
29,000 ksi

Fender

Max span 5.3 m 17.39 ft (Lower section)

Vertical	Diam.	thick	Area	I	S	Z
	1.067	0.025	0.083	0.011	0.021	0.028
	42.0	1.00	128.8	27,081	1,290	1,681

Moment = PL/4 = 9,998 kN-m fb = 473 MPa (S)
363 MPa (Z)
7,390 ft-kips 52.7 ksi (Z)
say OK Yield is good

Displ. = 0.010 m 0.41 in

Stubs	Diam.	thick	Area in^2
	0.762	0.010	0.023
	30.0	0.375	34.9

Axial load = 7600 kN fa = 338 MPa
1700 kips 48.7 ksi
OK Yield is good

Leg - Upper bay Leg & bracing share load according to relative stiffness

Impact height Elev. + 7.60 m 24.9 ft

Jacket Horizontal 3.50 m 11.5 ft

Bay height (to interm. horiz member) 10.75 m 35.3 ft

l/r 29.4

Impact location, a = 4.10 m b = 6.65 m
13.5 ft 21.8 ft

Fy = 345 MPa

alpha = 0.39

Leg	Diam.	thick	Area	I	S	Z	r
	1.067	0.032	0.103	0.014	0.026	0.034	0.366
	42.0	1.25	160.0	33,248	1,583	2,076	14.4

Fcn = 332 MPa

Fey = 2,291 MPa

Leg axial load = 9,000 kN fa = 87.2 MPa
2,024 kips 12.6 ksi

Cm = 0.93

K = P/displ = 685 kip/in 19% to leg (Compare w/ bracing check below)

Fxe = 518 MPa

Load P = 1.42 MN 317 kips

Fxc = 345 MPa

Moment = Pab/L = 3,565 kN-m fb = 137 MPa (S)
105 MPa (Z)
2,635 ft-kips 15.2 ksi (Z)

Fbn = 444 MPa

Displ. = 0.063 m 2.48 in 192 MPa (Z) OK
27.9 ksi (Z)

Check API D.3.2-1
0.04 + 0.03 = 0.07 OK

Leg/Pile - Lower bay Leg & bracing share load according to relative stiffness

Impact height	Elev.	(1.80) m		(5.9) ft		**Ignore Pile (conservative)**				
Bay height (to interm. horiz member)		11.75 m		38.5 ft					l/r	27.8
Impact location,	a =	5.30 m	b =	6.45 m						
		17.4 ft		21.2 ft						
									Fy =	50 ksi
Leg	Diam.	thick	Area	I	S	Z	r			
	1.219	0.025	0.095	0.017	0.028	0.036	0.422		alpha =	0.37
	48.0	1.00	147.7	40,790	1,700	2,209	16.6			
									Fcn =	48.3 ksi
Leg axial load =		9,000 kN			fa =	94.5 MPa				
		2,024 kips				13.71 ksi			Fey =	369 ksi
K = P/displ =		585 kip/in		18% to leg		(Compare w/ bracing check below)			Cm =	0.94
Load P =		1.35 MN		301 kips					Fxe =	363 ksi
Moment =	Pab/L =	3,885 kN-m			fb =	139 MPa (S)			Fxc =	50 ksi
						107 MPa (Z)				
		2,872 ft-kips				15.6 ksi (Z)			Fbn =	59.6 ksi
	Delta =	0.074 m		2.91 in		202 MPa (Z)				
						29.3 ksi (Z)		OK		
Check API D.3.2-1										
		0.33 +		0.22 =		0.55 OK				

Bracing check

(Upper bay)	Diam.	thick	Area	I	S	Z	r			
	0.457	0.019	0.026	0.001	0.003	0.004	0.155			
	18.0	0.75	40.6	1,515	168	219	6.10			
K = P/displ =		2,994 kip/in		81% to bracing		(Compare w/ leg checks above)				
length =	10 m		32.8 ft						l/r	64.5
Load P =		6.18 MN		1,383 kips					Fy =	345 MPa
	fa =	236 MPa		34.0 ksi					alpha =	0.85
		0.98 OK							Fcn =	282 MPa

Bracing check

(Lower bay)	Diam.	thick	Area	I	S	Z	r			
	0.610	0.013	0.024	0.001	0.003	0.005	0.211			
	24.0	0.50	36.9	2,549	212	276	8.31			
K = P/displ =		2,719 kip/in		82% to bracing		(Compare w/ leg checks above)				
length =	10 m		32.8 ft						l/r	47.4
Load P =		6.25 MN		1,399 kips					Fy =	345 MPa
	fa =	263 MPa		37.9 ksi					alpha =	0.63
Check API D.2.2-2a		0.99 OK							Fcn =	311 MPa