Design of a Pipeline Bridge in Northern BC

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Appendices

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Appendix A: Preliminary Hydrotechnical and Geotechnical Assessment

PRELIMINARY HYDROTECHNICAL AND GEOTECHNICAL ASSESSMENT

EXECUTIVE SUMMARY

This report describes the hydrotechnical and geotechnical conditions of the proposed pipeline crossing at Bridge X Creek. The Information presented is based on desktop information, site visits and geophysical survey data. The Bridge X Creek crossing is located northeast of Hazelton, BC, along the proposed pipeline.

As part of the hydrotechnical assessment a flood frequency analysis was carried out to establish peak instantaneous flows at the Bridge X creek for different return periods including the 200-yr event. The bank erosion hazard was assessed by comparing historical aerial photographs, which enabled the identification of two hazard zones within the floodplain: the *zone of active influence* and the *zone of river influence*. The proposed location of the aerial crossing was assessed against flooding, scour and bank erosion. Recommendations are also provided for the aerial crossing freeboard considering local scour, ice jam scour and to accommodate the passing of large woody debris (LWD).

1.0 INTRODUCTION

1.1. Scope of Work

The objectives of this report are to provide the following for the proposed Bridge X Creek aerial crossing:

1. An overview of the expected surface and subsurface conditions at the site.
2. An overview of potential geohazards relevant to the proposed infrastructure.
3. A seismic hazard assessment of the site.
4. Updated hydrotechnical hazard assessment for the crossing.
5. Preliminary recommendations for foundations based on inferred ground conditions.
6. Recommendations for further work.

3.0 SITE SETTING

3.1. Site Morphology

The proposed aerial crossing is located in a constrained floodplain of the Bridge X Creek, approximately 475 m downstream of an existing forestry bridge. In the area of the proposed aerial crossing, the extent of the zone of active influence is within approximately 45 m of the valley bottom. The current bankfull width is about 15 m at the crossing location.

3.2. Surficial Geology

Surficial materials expected at the proposed crossing location include fluvial, colluvial, glaciofluvial and glacial till deposits.

The valley walls have moderately steep to steep gradients and are expected to be covered with a veneer of till sediments or colluvium, often with rock near their base. Given the site geology and topography at the crossing area, these slopes are prone to slumping, debris avalanches and rock falls.

Thick, hummocky, irregular glaciofluvial deposits are present in several locations throughout the interior plateau, and abandoned glaciofluvial fans and terraces are present in several valleys along the pipeline route. Glaciofluvial material is typically composed of coarse sands and gravels, deposited during the immediate post-glacial and earlier interglacial periods. It is commonly located in terraces above present river levels.

The floodplain of the creek is characterized by fluvial deposits comprising granular materials ranging in size from sands to boulders. Areas mapped as active floodplains have little vegetation and are regularly flooded. Low terraces are not typically flooded, but may be subject to bank erosion, or channel avulsion.

3.3. Bedrock

The geological map for Hazelton, BC (Evenchick et. al. 2008) indicates that the bedrock strata in the area of the proposed crossing likely comprise rocks from the Bulkley Canyon Formation. Typical lithologies associated with this geological unit include feldspathic sandstone, siltstone, mudstone, coal and minor volcaniclastic conglomerate.

Depositional cycles are commonly noted on bedrock exposures of the Bulkley Canyon Formation by an upward reduction on the lithology grain size, ranging from coarse-grained sandstone to carbonaceous siltstone, mudstone or coal.

4.0 FIELD INVESTIGATION

4.1. Site Visit and Surface Mapping

Five hand-dug test pits were excavated to depths of approximately 0.3 m for the purposes of characterizing the near-surface soils, and four bedrock outcrops were mapped. Locations of the hand-dug test pits and bedrock outcrops are presented in Drawing 01.

5.0 PRELIMINARY GEOTECHNICAL SITE CHARACTERIZATION

5.1. Surficial Geology

Three hand-dug test pits (Hand-dug Test Pit 3 to Hand-dug Test Pit 5) excavated in the floodplain (Drawing 01) indicated that the upper 0.3 m of overburden comprised dark brown fluvial sand and gravel, with traces of clay. The gravel particles were noted to be sub-rounded and comprised sedimentary clasts. The surficial sands and gravels were generally moist and based on the effort required to dig the hand dug pits, the compactness condition was assessed to be compact.

Till was encountered in two hand-dug test pits (Hand-dug Test Pit 1 and Hand-dug Test Pit 2) excavated in the south valley wall (Drawing 01) in the vicinity of the proposed crossing. These pits were excavated to a depth of approximately 0.3 m. The near-surface till comprised moist to wet, dark-brown to dark-gray, gravelly silt, and was assessed to be of soft consistency. Gravel sized particles within the till were observed to be sub-rounded to rounded and comprised sedimentary clasts.

Some colluvial material was exposed on a small slump (Slump 1) observed on a forestry road cut at the North bank of the valley (Drawing 01), and also on two other slumps (Slumps 2 and 3) identified further upstream of the crossing area (Drawing 01). These soils generally comprised light brown, dry, gravelly silt with trace sand. The gravel component was described as sub-angular to sub-rounded.

5.2. Bedrock

Bedrock outcrops were not observed along, or near the creek. Small bedrock exposures were identified on the south wall of the valley and large bedrock outcrops were also observed approximately 400 m upstream of the crossing, on the north side of the valley.

Data collected on four bedrock outcrops (Drawing 01) indicated that the rock typically comprised dark-brown, fine-grained, tabular siltstone (refer to photos in Appendix B). The rock mass was observed to be fractured, and two to three well developed sets of joints were consistently noted on the rock exposures. The spacing of discontinuities ranged from very closely to closely spaced. The discontinuities were generally noted to be of low persistence, although these observations were limited by the small size of the outcrops. The joints were generally rough, closed and with no apparent infilling. Based on a field estimate, the rock strength was classified as Medium Strong, with unconfined compressive strength (UCS) likely ranging from 25 to 50 MPa (ISRM, 1981).

5.4. Geohazards

This section has been prepared based on information collected through LiDAR topography, aerial photo interpretation as well as, helicopter and ground based reconnaissance. The assessment of terrestrial slope geohazards for the pipeline is ongoing and has been reported separately. A summary of slope geohazards relevant to the Bridge X Creek aerial crossing is included herein.

On the south wall of the valley, approximately five shallow debris slides have been identified (Drawing 01). Two of these features are located within approximately 50 m of the proposed south foundation, and similar shallow slides may pose a hazard during construction. These shallow slides are interpreted to be linked to the construction of the existing road forestry service road. Other evidence of slope instability has been observed near the proposed alignment along the south Creek valley wall. This includes the presence of curved trees, uneven, wet ground, and open joints and displaced blocks in bedrock outcrops.

The north valley wall is affected by an old, large, deep seated landslide, evident in air photos and LiDAR topography. A large slump block, forming part of the overall landslide, is shown in Drawing 01. This feature has been interpreted from ground reconnaissance as being abandoned, and no longer active; however this interpretation is subject to additional planned study by drilling and installation of an inclinometer. The slump block is not currently expected to pose a credible hazard to the proposed bridge structure, but will require consideration for placement and construction of pipe supports beyond the north embankment.

A number of relatively small, shallow debris slides have been noted on the north valley wall from air photo study. The locations of these features are identified in Drawing 01. Shallow landslides in this area appear to occur with a typical return period of 5 to 10 years. These shallow failures are not expected to affect the proposed aerial crossing structure.

Other geohazards observed in the area of the proposed Bridge X Creek, which do not pose a direct risk to the aerial crossing, include small slumps in the forestry road cuts and on steep sections of the river bank (Drawing 01).

5.5. Seismic Assessment

Spectral accelerations for the proposed Bridge X Crossing site were obtained from the Earthquakes Canada (EC) website http:/www.earthquakescanada.nrcan.gc.ca. These values are available for Site Class C, which is defined as very dense soil or soft rock, with time-averaged shear wave velocities in the top 30 m (Vs_{30}) ranging from 360 to 760 m/s (Earthquakes Canada, 2010).

Based on the results of the MASW survey undertaken at the proposed south location (Drawing 03), a Vs_{30} of 985 m/s was estimated for the site. It is important to note that the penetration of the MASW survey was about 24 m, and therefore some extrapolation was needed to complete the shear wave velocity profile to a depth of 30 m.

The estimated Vs_{30} of 985 m/s suggests that the Aerial Crossing site should be classified as Site Class B. Therefore, the spectral accelerations presented in Table 5-1 have been corrected using the appropriate amplification factors according to Finn and Wightman (2003).

Table 5-1. Corrected Spectral Accelerations for Site Class B at Bridge X Crossing using amplifications factors according to Finn and Wightman (2003).

Spectral Accelerations	Return Period Earthquake
	2,475-yr
Sa (0.2)	0.076
Sa (0.5)	0.048
Sa (1.0)	0.028
Sa (2.0)	0.018
PGA	0.040

5.6. Hydrotechnical Hazard Assessment

This section summarizes the hydrotechnical hazard assessment conducted for the aerial pipeline crossing of Bridge X Creek. Surveyed channel geometry and site observations were used to assess the potential hydrotechnical hazards for the proposed aerial crossing, including: general scour, ice jam scour, bank erosion, avulsion and large woody debris (LWD). The hazard assessment methodology is summarized here with considerations important to the aerial crossing. Hydrotechnical processes that have the potential to compromise an aerial crossing bridge span were evaluated based on judgment, qualitative assessment, and analysis of historic air photographs.

Figure 5-1. Upstream view of Bridge X Creek at the proposed aerial crossing.

5.6.1. Hydrotechnical Hazard Analysis

Flood Frequency Analysis

Flood quantiles at the Bridge X Creek site were estimated using a Flood Frequency Analysis (FFA). An FFA can either be conducted regionally using several hydrometric stations, or for a specific station where the peak flow estimates are then pro-rated by drainage area to the location of interest. The latter approach is generally only employed for locations on the same river system.

Flood quantiles were estimated using a regional analysis with available peak flow data from four Water Survey of Canada (WSC) hydrometric stations located near the Bridge X Creek site. The drainage area of this site was estimated to be 152 km^2.

Table 5-3. Peak Instantaneous Flow Estimates (Q_{IMAX}) at the Bridge X Creek

Aerial Crossing	Basin Area (km^2)	Q_{IMAX} for Given Return Periods (m^3/s)						
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
Bridge X Creek	152	16	23	28	36	43	50	59

General Scour

A scour analysis was completed for the Bridge X Creek site to evaluate general scour conditions for the proposed pipeline crossing. General scour refers to channel adjustments during a single flood event based on sediment inflow, bed material gradation and the sediment transport potential in a stream reach.

The scour analysis was conducted using peak flows presented in Table 5-3 above, a channel gradient of 1.5%, and a channel cross-section provided by a ground survey. Various empirical hydraulic equations have been developed to estimate scour depth during a peak flow event. Each method was designed based on a specific range of boundary conditions and care must be taken to select appropriate methods to apply to a given study site. The selection and effective use of these equations requires considerable engineering judgment, resulting in semi-quantitative results.

For the hydraulic analysis, available topographic data included a LiDAR survey and the aforementioned ground survey, complete with bathymetric data. Flow hydraulics were estimated using Manning's equation and a channel cross- section cut from the survey; the 200-year peak instantaneous flow was determined to correspond to a water elevation of 519.2 m above sea level (Drawing 06). The D_{50} of the channel substrate is approximately 105 mm based on a Wolman sample.

For the 200-year flood, a scour depth of 0.8 m below the channel thalweg (El. 517.7 masl) is estimated, which results in a scour elevation of approximately 516.9 masl. This scour depth is based on the Blench regime equation, which is considered to be suitably conservative for this crossing location. However, this estimate is for general scour only in the case of an open trench or HDD pipeline crossing. Local scour, which can occur, is considered in a section below.

Bank Erosion

To assess the historical lateral extents of the Bridge X Creek at the proposed aerial crossing, an evaluation of channel migration was undertaken by comparing historical aerial photographs. The aerial photographs were georeferenced to make the comparison, and the historic channel planform (the outline of the 1949 active channel) was digitized and then overlain on the 2013 orthophoto. Drawing 04 demonstrates a quantitative assessment of the lateral extents of the watercourse over the past 65 years (1949 - 2013).

The proposed crossing occurs through the constrained floodplain of Bridge X Creek, approximately 475 m downstream of an existing forestry service road bridge. The current bankfull width is 15 m at this location, reduced from approximately 25 m in 1949. Furthermore, Drawing 04 illustrates that the channel reaches within 100 m upstream and downstream of the proposed crossing have become increasingly stable as well: the active channel has narrowed by as much as 60 m and the river banks have become increasingly vegetated.

The wider channel in 1949 is likely related to a period of increased landslide activity on the valley sidelopes upstream of the crossing. Drawing 01 shows the paths of a number of historic debris slides, a majority of which have revegetated. However, there are a number of active debris slides located adjacent to Bridge X Creek upstream of the confluence with Creek B. A more unstable, wider channel is a typical response to an above average influx of sediment to a watercourse. It is therefore not unreasonable to hypothesis that the wider channel in 1949 is a result of a period of increased landslide activity and although these slides have since stabilized for the most part, another period of increased landslide activity could occur in the future during the life of the pipeline crossing.

5.6.2. Design Considerations

Location of Abutments

Drawing 05 shows the approximate extent of the Bridge X Creek floodplain, which is approximately 175 m wide at the proposed crossing. The floodplain is confined by moderate to steep gradient slopes on either side of the pipeline crossing. Two hazard zones have been identified on Drawing 05 that are potentially subject to flooding, scour, and bank erosion: the *zone of river influence* and the *zone of active influence*.

The *zone of active influence* represents the potential distance over which the active channel is likely to migrate based on air photo and topographic contour interpretation, as well as a site observations. The *zone of river influence* is essentially the width of the floodplain and it is assumed that the river could occupy any position of the floodplain in the future given sufficient time and suitable hydrologic and sediment supply conditions. The *zone of river influence* should be considered as a continuum hazard zone, such that as one moves away from the existing banks of the main river channel, the likelihood of the river occupying that position of the floodplain and impacting the pipe decreases until the edges of the zone are reached. The further the abutments of the aerial crossing are positioned away from the active channel, the lower the likelihood that lateral migration of the river would impact on the pipeline. The lowest likelihood would be associated with the abutments located on the outer margins of the *zone of river influence*.

However, hazard mitigation through avoidance is not always practical. Some rivers can have broad floodplains that are characterized by very low bank erosion rates. In such cases, it is not necessary that the aerial crossing span the entire floodplain. Therefore, within the broad *zone of river influence*, a *zone of active influence* has been subjectively delineated on Drawing 05. This zone represents an area that is considered most likely to be occupied by the active channel in the pipeline design life (assumed to be about 50 years). This zone is based on the historic assessment of lateral instability, and should not be considered as a distinct line: that is, having the outer abutments lie within or out of the active zone does not represent safe and unsafe conditions, respectively.

Given the greater uncertainty regarding the future position of the channel on the south side, it is recommended that the south (left) bank be protected with riprap. Class 100 riprap (100 kg, D_{50} ~ 450 mm) with a thickness of 0.7 m and an appropriate filter should be used at a minimum slope of 2H:1V. The riprap should extend to the upstream boundary of the right-of-way and about 10m downstream of the south abutment. For the upstream configuration, the riprap should curve inland at a moderate angle to minimize the potential for upstream erosion to undermine the riprap. Excavation of the floodplain soils would be required for some of the riprap placement and the armour layer would then be buried using the excavated soils.

Aerial Crossing Freeboard

At aerial crossings, a minimum clearance height (i.e. freeboard) is required above the 200-year flood (Q_{200}) elevation to account for passing of large woody debris (LWD) and to minimize the potential for any floating debris to come into contact with the pipeline. This freeboard also accounts for potential long-term channel aggradation and uncertainty in peak flow estimates.

LWD is defined as wood > 0.1 m diameter and > 1 m length which has been deposited on river banks or within the channel itself (Featherstone, Naiman, & Bilby, 1995). The potential for LWD to collect and become entrained is often evaluated for aerial span and bridge crossings. Assessments are typically conducted using a weighting system based on channel characteristics that favour LWD accumulation, including: channel form, channel roughness (i.e.: bed material, presence of boulders, and pools/riffles), bankfull width and channel slope. These assessments rely heavily upon field notes, photographs and other information collected from the crossing site.

Based on the site visit, 2 m of freeboard is recommended between the 200-year flood elevation and the soffit (Drawing 06).

Local Scour

Local scour refers to erosion of the streambed that is immediately adjacent to and caused by some obstruction to flow, such as a bridge pier or LWD. For the proposed aerial crossing, any of the foundations will potentially be located within the active channel within the lifespan of the pipeline. Local scour will therefore be evaluated under the pretense that the active channel could migrate laterally within the design life of the pipeline, exposing any of the foundations to high return period floods. Scour depth was assessed using the *CSU Equation for Scour* (Richardson et al., 1993). The estimated scour depth of 1.0 m is additive to the general scour estimate of 0.8 m, yielding a net scour depth of 1.8 m below which the foundations should be founded: 2.0 m (Elevation = 515.7 masl) is recommended for design.

Ice Jam Scour

Ice jam scour can occur during ice build-up reaching depths equal to ice thickness below the water surface at the upstream end of a jam (Mercer & Cooper, 1977). The influence of an ice cover on a channel involves complex interactions between the ice cover, ice roughness, fluid flow, sediment, bed geometry, water depth, and channel geometry, which can have a dramatic effect on sediment transport process and channel development, especially in narrow rivers (Hains and Zabilansky, 2005).

The potential stability of ice jam formation is determined in most cases by the width of the river channel and the river flow velocity. A river with high gradient and high flow velocity will not tend to allow the formation of stable ice jams. Similarly, wide rivers are less likely to form stable ice jams. Rivers prone to ice jam formation would generally have lower flow velocity and a narrowed section where ice could be accumulated (Mercer and Cooper, 1977). Rivers with fine-grained substrates (sand or finer) are also more susceptible to potential ice scour (Yaremko and Cooper, 1983).

A desktop study, including review of site photographs from the nearby Forestry Service road bridge, has concluded that the Bridge X Creek crossing site carries minimal risk of ice jam scour. While there is evidence of some ice cover formation on the north bank of Bridge X Creek, the watercourse was observed to be free flowing during winter low flow periods. See Figure 5-3 and Figure 5-4 for further detail.

Figure 5-3. Bridge X Creek as observed from the Forestry Service Road Bridge, facing downstream.

Figure 5-4. Bridge X Creek as observed from the Forestry Service Road Bridge, facing upstream.

6.0 FOUNDATION DESIGN

Given the interpreted depth to bedrock at the crossing site and the need to protect the bridge foundations from the creek scour, pile foundations are recommended for supporting the bridge.

It is recommended that the contractor have the ability to clean out the piles and/or remove obstructions, such as cobbles and boulders, by drilling, if needed. This may be accomplished using a down-hole hammer or dual-rotary drill or other suitable method. Concrete plugs may be installed once the piles have been cleaned out in order to improve the end bearing resistance and overall geotechnical pile capacity. This would result in shorter pile lengths being required to resist compressive loads. However, if uplift resistance or lateral capacity is shown to govern the required length of pile, the addition of a concrete plug at the base of the pile may be of limited benefit.

The preliminary pile design has been carried out so that the maximum Ultimate Limited State (ULS) axial load expected on the pile does not exceed the geotechnical axial resistance of the pile.

With respect to lateral resistance, the piles were sized so that the maximum lateral deflection at the top of the pile did not exceed 25 mm under the expected Service Limit State (SLS) loads.

Based on experience with similar geophysical surveys undertaken in other areas of the pipeline route with comparable geology, it is believed that the bedrock surface may be considerably deeper along the creek than the interpretation of the geophysical surveys.

In order to account for the uncertainty in bedrock elevation, two independent scenarios were analyzed and the piles were sized so as to satisfy the design criteria in either case. Given the overall consistency with respect to the subsurface conditions at each foundation location as suggested by the geophysical surveys, the two cases described below were assessed to be representative of each pile, regardless of the location. The maximum axial and lateral loads applied in the analyses represent estimated loading conditions on a single pile within the foundations.

- Case A: Competent bedrock was assumed to be at a depth of approximately 8.4 m below the existing ground surface (El. 512.8 masl). The piles would be driven through overburden soil (sands and gravels) and socketed into competent bedrock.
- Case B: Bedrock was assumed to be a depth such that the piles would be embedded in the overburden only. The overburden was assumed to comprise compact sand and gravel.

The analyses presented in the following sections are based on 914 mm x 12.7 mm (diameter x thickness) piles.

6.1. Geotechnical Axial Resistance

6.1.1. General Design Methodology

The unfactored axial pile resistance in compression is a combination of the pile shaft resistance (R_s) and the pile toe resistance (R_t). The unfactored axial pile resistance in tension is equivalent to the exterior pile shaft resistance only.

Various empirical methods are available to estimate these resistances based on the soil parameters and pile geometry. The recommendation is to use the Beta (effective stress) method for estimation of the unfactored axial pile resistance, R_{uc} and R_{ut}.

R_{uc} = unfactored axial pile resistance, in compression = $R_s + R_t$

R_{ut} = unfactored axial pile resistance, in tension = R_s

Rs = shaft resistance = As x qs

R_t = toe resistance = $A_t \times q_t$

$\phi_c R_{uc} \geq$ factored axial resistance in compression (refer to Table 6-2 for geotechnical resistance factor, ϕ_c)

$\phi_t R_{ut} \geq$ factored axial resistance in tension/uplift (refer to Table 6-2 for geotechnical resistance factor, ϕ_t)

where A_s = pile shaft/soil surface area (based on pile perimeter and pile length),

$q_s = \beta \times \sigma'$ = unit shaft friction,

A_t = pile toe cross-sectional area,

$q_t = N_t \times \sigma'_t$ = unit toe resistance,

β = shaft resistance coefficient,

N_t = toe bearing capacity factor,

σ' = vertical effective soil stress along pile shaft,

σ'_t = vertical effective soil stress at the pile toe.

The simplified equations above are representative of the case where the pipe piles were cleaned out to the pile toe following installation, or if the frictional resistance of the soil inside the pile shaft is ignored. Pile plugging was not considered in the present analyses

In order to ensure that interaction effects do not limit the axial capacity of the piles, a minimum spacing of 2.5 diameters is recommended (CSA, 2006).

Nt and β values were selected based on recommendations provided in Tables 18.1 and 18.2 of the Canadian Foundation Engineering Manual (CGS, 2006).

The soil located above the 200-year scour elevation (515.7 masl) as well as the contribution to the effective stress at depth caused by this material was ignored in the preliminary design.

The recommended geotechnical resistance factors for both compression and tension are presented in Table 6-2. Values are in accordance with the Canadian Foundation Engineering Manual (CGS, 2006). A resistance factor of ϕc=0.4 was used for factoring the compressive resistance of the pile.

Table 6-2. Recommended Geotechnical Resistance Factor (ϕ) for Pile Design (CGS, 2006)

Compression – static analysis	ϕc=0.4
Compression – dynamic analysis (with field measurement and analysis, e.g. PDA tests)	ϕc=0.5
Tension – static analysis	ϕt=0.3
Tension – using results of dynamic analysis	ϕt=0.4

6.1.2. Preliminary Pile Design for Axial Compression

Case A

For piles socketed into bedrock, it may be assumed that the geotechnical axial resistance will be limited by the buckling capacity of the pile.

Case B

For this case, the overburden was assigned a beta coefficient (ß) of 0.6, and N_t factor of 50. The results suggest that a 914 mm x 12.7 mm driven pipe pile would require a minimum embedment length of 15 m below the 200-yr scour elevation (515.7 masl) to develop adequate factored geotechnical axial resistance against the expected ULS compressive load.

6.2. Lateral Pile Capacity

6.2.1. General Design Methodology

The lateral resistance of piles is a function of the connection details at the pile head, the structural rigidity of the pile section, the installed length of the pile, and the soil resistance along the pile shaft. For preliminary design purposes, a lateral load analysis using assumed, non-liner p-y curves for each stratigraphic layer was conducted using the computer software LPILE, (by Ensoft, Inc.). The input parameters used for these analyses are presented in Table 6-3 for Case A and Case B, respectively. These values were selected based on recommendations provided by Reese and Wang (1989) with pipe piles driven in ground conditions similar to those expected here. The geotechnical properties for the weak bedrock strata have been conservatively selected following the guidance by Reese (1997) and are consistent with bedrock observations carried out in the field.

The preliminary lateral pile analyses conducted in this study considered single piles only and did not account for interaction effects resulting from closely spaced piles. No reduction factors are required for loading in the longitudinal direction.

Table 6-3. Geotechnical parameters adopted for preliminary pile design

Material	Geotechnical Parameter	Value
Sand (Reese)	Effective Unit Weight, γ' (kN/m^3)	9
	Friction Angle, ° (Deg)	33
	Initial Soil Modulus, k_s (MPa)	16.3
Weak Rock (Reese)	Effective Unit Weight, γ' (kN/m^3)	14
	Uniaxial Compressive Strength q_{ur} (MPa)	6.5
	Initial Rock Modulus, k_{ir} (MPa)	200 – 800
	Rock Quality Designation – RQD (%)	0
	Strain factor - k_{rm}	0.00005

6.2.2. Preliminary Pile Design for Lateral Resistance

Case A

The ground surface elevation is assumed to correspond to the 200 yr scour elevation.

The results from the LPILE analysis indicate that a 914 mm x 12.7 mm driven pipe pile, socketed a minimum of 4.2 m into bedrock, would experience a maximum deflection at the pile head on the order of 15 mm under the assumed conditions.

Case B

The analysis suggests that an embedment depth of 18 m below the 200 yr scour elevation would be required to resist the expected lateral loads under these assumed conditions. In that case, the 914 mm diameter x 12.7 mm driven pile would be expected to deflect approximately 21 mm at the pile head.

6.3. Pile Uplift Due to Frost Action

6.3.1. General Design Methodology

Based on the air freezing indices, a preliminary frost penetration depth of 2 m was calculated using the modified Berggren method (Aldrych and Paynter 1966) for the Bridge X aerial crossing site. In the absence of site specific geotechnical investigations, conservative values were selected for the ground thermal properties and ground surface conditions during winter.

A design adfreeze bond of 120 kPa was assumed following the recommendations from the Canadian Engineering Foundation Manual (CGS, 2006), where typical adfreeze bond stresses range for fine grained soils is proposed from 65 kPa to 100 kPa and for saturated gravel frozen to steel piles can be estimated at 150 kPa.

Based on the design bond stress, the pile surface area, the estimated frost-depth penetration of 2 m, and assuming a load factor of 1.3, a factored jack uplift force of 896 kN per pile was estimated for preliminary design purposes.

6.3.2. Preliminary pile uplift resistance

Case A.

The side-wall shear resistance was estimated based on a correlation with the rock unconfined compressive strength (UCS) from Wylllie (1999). For the calculation of the factored uplift resistance of the pile, a resistance factor of 0.5 was assumed. The basis for adopting a higher resistance factor (ϕt) to the one shown in Table 6-2, is that the jacking force from adfreeze is transient.

Based on a conservative UCS value, and on a proposed socketed length of 4.2 m into competent bedrock (refer to Table 6-5), the factored uplift resistance is estimated to be 2,900 kN, which exceeds the estimated factored adfreeze uplift load of 896 kN.

Case B.

The Canadian Engineering Foundation Manual (CGS, 2006) indicates that for cohensionless soils, such as sand and gravel, the ultimate uplift shaft resistance is about 75% of its value in compression. For the calculation of the factored uplift resistance of the pile, a resistance factor of 0.5 was assumed as discussed on Case A.

Following the design basis above, the proposed minimum embedment length of the pile below the 200-yr scour elevation (refer to Table 6-5), will provide an estimated factored uplift resistance of 1,250 kN, which exceeds the estimated factored adfreeze uplift load of 896 kN.

6.4. Pile Design Summary and General Recommendations

A summary of the overall findings of the preliminary design effort including the size, minimum embedment depth and factored axial resistance of the piles is presented in Table 6-5.

Table 6-5. Pile recommendations for the Bridge X Creek Aerial crossing

Pile details	Case A	Case B
Pile Diameter (mm) x Thickness (mm)	914 x 12.7	
Minimum total pile length to satisfy estimated axial and lateral loading (m)	13.1	24.0
Unbraced length – Pile length above 200-yr scour elevation (m)	6.0	
Minimum pile embedment below the 200-yr scour elevation to satisfy both axial and lateral loading requirements (m)	7.1	18.0
Minimum pile length socketed into bedrock (m)	4.2	Not applicable
Factored Geotechnical Axial Resistance in compression (kN)	Assumed to be equal to structural buckling capacity of the pile	1,300

A drive shoe is recommended to protect the pile toe and enhance pile driving penetration through the coarse fluvial soils. Specifically, the use of an inside shoe will minimize disturbance of the soil around the outside of the pile and allow maximum exterior shaft resistance. If an outside drive shoe is used, additional pile length may be required as a result of lower shaft resistance.

The Canadian Engineering Foundation Manual (CGS, 2006) suggests a value of 6,000 kJ times the cross-sectional area of the pile as the maximum hammer energy for driven steel pipe piles. For a 914 mm diameter pipe pile with a wall thickness of 12.7 mm, a maximum hammer energy of 216 kJ is recommended.

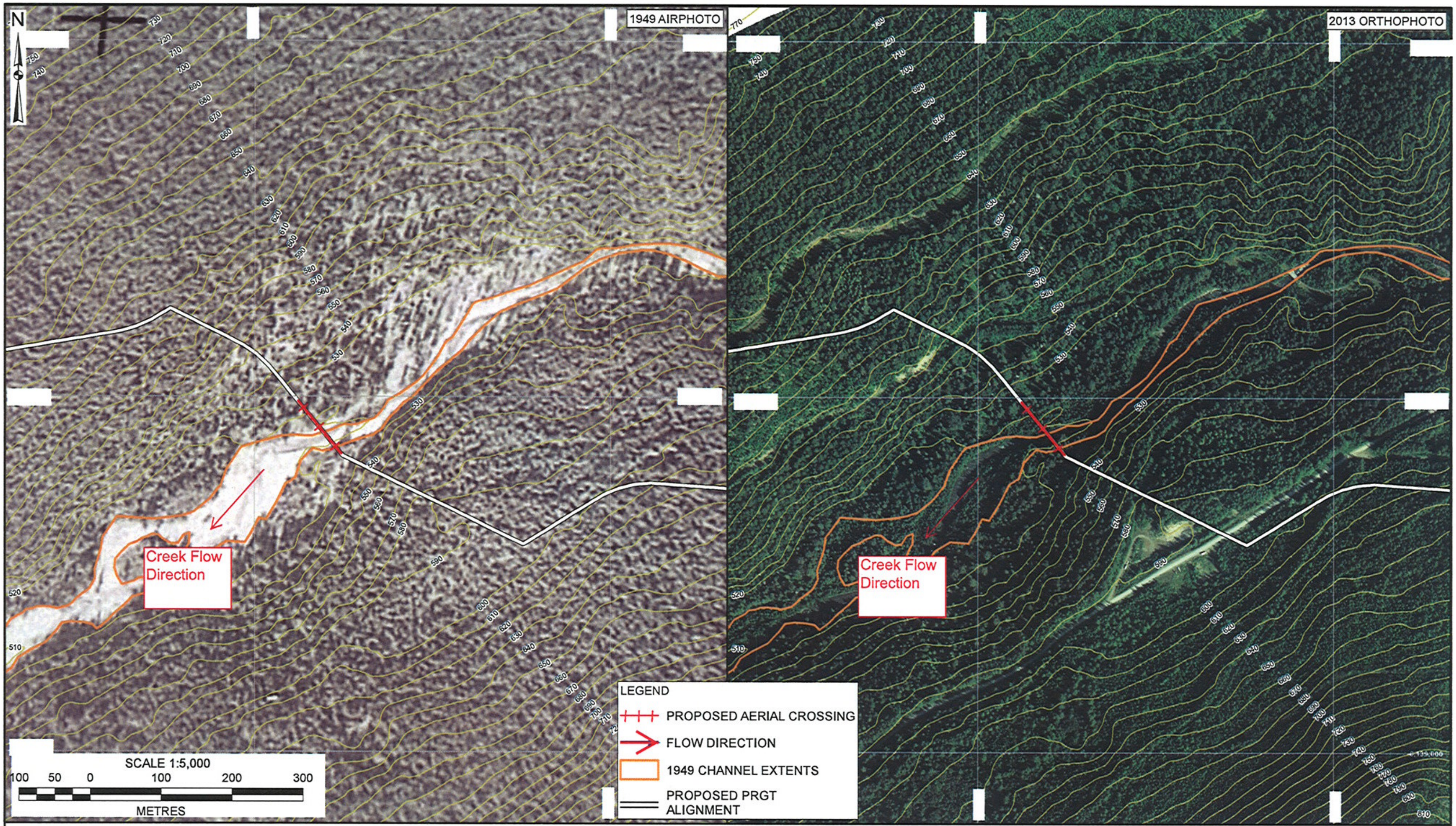
It is recommended that full-time inspection by an experienced geotechnical engineer be provided during pile installation. Comparison of pile tip elevations with the test hole records (once they become available) should be carried out on an ongoing basis while pile driving in addition to recording blow counts with penetration depth. Piles should be driven to the minimum tip elevation necessary to resist lateral design loads as well as compressive and tensile axial loads.

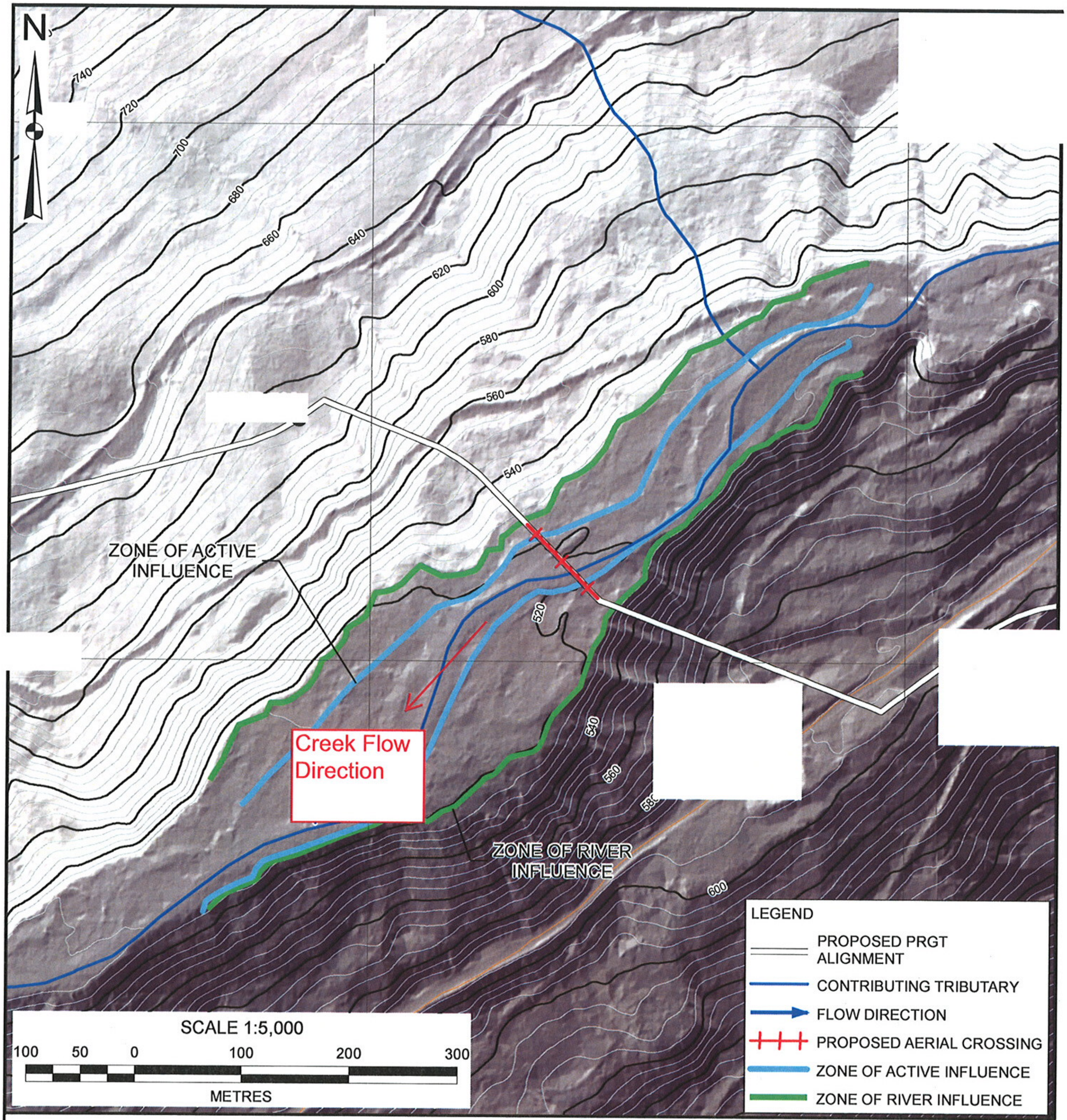
The contractor may wish to consider pre-boring to achieve adequate pile penetration if shallow refusal on cobbles/boulders become problematic, or if pile driving activities are conducted in winter conditions. In these cases, pre-boring should be limited to a diameter of 25 mm less than the proposed pile diameter to allow for good friction contact between the pile shaft and subsoil.

Due to the current uncertainty of the soil conditions at the proposed pile locations, Pile Dynamic Analysis (PDA) testing is recommended to confirm the axial design capacity of the piles. PDA testing should be conducted both during initial pile driving and restrike after a suitable waiting period has passed to allow pore pressures related to pile driving to dissipate.

It is recommended that all piles be analyzed with PDA, with re-strikes performed a minimum of 24 hours after initial driving. If relaxation occurs, the pile should be re-driven to the final driving criteria and the cycle repeated until the final driving criteria can be achieved during the pile re-strike. If PDA testing confirms that the pile has met the axial design capacity during re-strike, the pile may be considered to be acceptable. If PDA testing indicates that the axial design capacity has not been achieved and driving to further depth is not practical, the driving of additional pile(s) may be required. PDA tests must be carried out by an experienced geotechnical engineer.

The PDA measures the dynamic response of the pile-soil system during driving and uses these measurements to calculate an estimate of ultimate static pile-soil resistance. Following PDA field measurements the shaft and toe resistance should be further assessed by conducting CAPWAP analyses. CAPWAP is performed to determine the load-displacement relationship, and the distribution of the geotechnical resistance. CAPWAP allows analysis of soil/pile interactions and is a more refined calculation than the estimated capacities that are calculated directly during the field testing PDA procedure


1949 AIRPHOTO
2013 ORTHOPHOTO
N
Creek Flow Direction
Creek Flow Direction
LEGEND
PROPOSED AERIAL CROSSING
FLOW DIRECTION
1949 CHANNEL EXTENTS
PROPOSED PRGT ALIGNMENT
SCALE 1:5,000
100 50 0 100 200 300
METRES


N
740
720
700
680
660
640
620
600
580
560
540
520
540
560
580
600
ZONE OF ACTIVE INFLUENCE
Creek Flow Direction
ZONE OF RIVER INFLUENCE
LEGEND
PROPOSED PRGT ALIGNMENT
CONTRIBUTING TRIBUTARY
FLOW DIRECTION
PROPOSED AERIAL CROSSING
ZONE OF ACTIVE INFLUENCE
ZONE OF RIVER INFLUENCE
SCALE 1:5,000
100
50
0
100
200
300
METRES

Appendix B: Design Loads

PIPELINE

$\rho_{STEEL} = \sim 7850 \ kg/m^3$

$\gamma_{STEEL} = 77 \ kN/m^3$

$A_{STEEL} = \frac{\pi (1219)^2}{4} - \frac{\pi (1168)^2}{4} = 99\,280 \ mm^2$

$\boxed{W_{STEEL} = 7.64 \ kN/m}$

WATER

$\gamma_{WATER} = 9.81 \ kN/m^3$

$A_{WATER} = \frac{\pi}{4}(1166)^2 = 1\,068\,000 \ mm^2$

$\boxed{W_{WATER} = 9.81 \ kN/m^3 \ (1.068 \ m^2) = 10.5 \ kN/m}$

3.10 – WIND LOAD – REFERENCE WIND PRESSURE, q TABLE A3.1.1

3.10.1.2 - SMITHERS, 50 YR RETURN PERIOD 0.40 KPa

3.10.1.2 - NOT DEEP VALLEY ; NO ADDITIONAL EFFECT

3.10.1.3 – GUST EFFECT COEFF, $C_g = 2.5$

3.10.1.4 - WIND EXPOSURE COEFF, $C_E = 1.0$

3.10.2.3 - $C_V = 1.0$

$F_V = q \, C_e \, C_g \, C_v = (0.4 \ kPa)(1.0)(2.5)(1.0)$

$\boxed{F_V = 1.0 \ kPa}$ (VERTICAL WIND PRESSURE)

3.10.2.2 – $C_H = 2.0$

$\boxed{F_H = 2.0 \ kPa}$

ICE ACCRETION LOAD

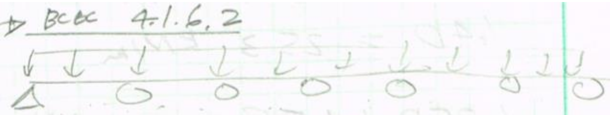
FIGURE A3.1.4 - 12 mm THICKNESS ON ONE SIDE

SNOW LOAD PER BCBC

$S = I_s [S_s (C_b C_w C_s C_a) + S_r]$

ASSUME $I_s = 1.0$ IMPORTANCE FACTOR

TABLE 4.1.6.2

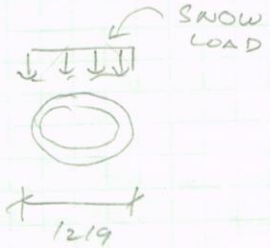


$S_s = 3.2$ kPa
$S_r = 0.2$ kPa

TABLE C-2 (DIV B, App. C)
(1 IN 50 YR EVENT)

$C_b = 0.8$ SENTENCE (2)

ASSUME $C_w = 1.0$
$C_s = 1.0$
$C_a = 1.0$
ASSUME RECTANGULAR DIST'N



$S = 1.0(3.2(0.8) + 0.2)$

$S = 2.76$ KPa MC

$W_{si} = (2.76 \text{ KPa})(1.219 \text{ m})$

$\boxed{W_{si} = 3.36 \text{ KN/m}}$ MC

ICE ACCRETION (CONT'D) FROM CHBDC 3.12.C

12 mm THICKNESS
(ONE SIDE OF PIPE)

$\gamma_{ICE} = 9.8 \text{ KN/m}^3$

ICE ACCUMULATED SURFACE

$L_I = \frac{\pi D}{2} = \frac{\pi (1219)}{2} = 1.915 \text{ m}$

$W_I = (9.8 \text{ kN/m}^3)(1.915 \text{ m})(0.012 \text{ m})$

$W_I = 0.225 \text{ KN/m}$

TOTAL SNOW LOAD

$W_s = W_{si} + W_I$ (ASSUME ICE ACCRETION IS SNOW LOAD)

$\boxed{W_s = 3.59 \text{ KN/m}}$
3.6

TOTAL DEAD LOAD (SEE NOTES, JAN. 17, 2015)

$W_{STEEL} + W_{WATER} = (10.5 + 7.64 \text{ kN/m})$

$\boxed{W_D = 18.1 \text{ KN/m}}$

WIND LOAD (SEE JAN. 17, 2013)

$W_{WIND} = (1.0 \text{ KPa})(1.22 \text{ m}) = \boxed{1.22 \text{ KN/m}}$

Assume construction during summer months

Load Cases:

1. Pressure Testing

   SW + Water Weight

2. Construction Loading

   SW + 20 workers + 2 kN tool cart

3. Dead Load (BCBC)

   $1.4D + 0.4S$ + Ice Accretion

   assume ice and snow occur @ the same time.

4. Snow Load (BCBC)

   $1.25D + 1.5S$ + Ice Accretion

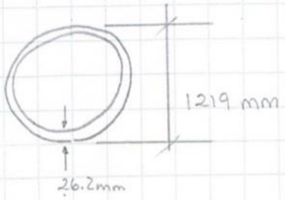
Case 1:

SW assume 152 × 152 × 9.5 $C_r = 796$ kN ($L = 5200$)

$M_L = 40.9$ kg/m $W_b = 401$ kN/m

$\gamma_s = 77$ kN/m³ $\gamma_w = 9.8$ kN/m³

$A_s = \frac{\pi}{4}(1219^2 - 1166^2) = 99280$ mm²



$W_s = 77$ kN/m³ $(99280$ mm²$) \times 10^{-6} = 7.64$ kN/m

$A_w = 1068000$ mm²

$W_w = 9.8$ kN/m³ $(1068600$ mm²$) \times 10^{-6} = 10.5$ kN/m

use S6 load combos 1.1D (treat water as dead since only applied once)
(1.1D for steel members)

pipe load:

$1.1(7.64 + 10.5) = 20$ kN/m

assume 0.25 kPa grating weight

0.25 kPa × 4.3m = 1.1 kN/m

1.1(1.1 kN/m) = 1.2 kN/m (factored)

factored beam self weight

1.1(401) = 440 N/m = 0.44 kN/m (per linear meter of beam)

Case 1 Summary:

pipe load + grating = 20 kN/m + 1.2 kN/m = 21.2 kN/m (factored)

beam self weight (per meter of beam) = .44 kN/m (factored)

Case 2

beam weight = 0.44 kN/m (factored; per linear meter of beam)

grating weight = 1.2 kN/m (factored)

pipe weight = 1.1(7.64) = 8.4 kN/m

assume tool cart already factored = $\frac{2\text{kN}}{0.5\text{m} \times 1\text{m}}$

20 maintanence workers:

assume worst case of all 20 workers standing on one panel near the center

Case 3:

unfactored loads

pipe = 7.64 kN/m beams = 0.4 kN/m (per linear meter of beam)

grating = 1.1 kN/m natural gas = 0.88 kg/m³ (9.81)(1.068 m²)
= 9N/m (insignificant)

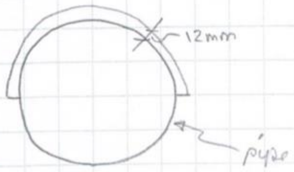
$$\text{snow} = I_s \left[S_s (C_b C_w C_s C_a) + S_r \right] \quad (\text{BCBC } 4.1.6.2)$$

$I_s = 1.0$ $C_b = 0.8$ (sentence 2) $C_a = 1.0$
$S_s = 3.2\,\text{kPa}$ $C_w = 1.0$
$S_r = 0.2\,\text{kPa}$ $C_s = 1.0$

snow = 2.76 kPa (assume snow can only accumulate over the area once
= 2.76 kPa (1.5m + 1.5m + 1.3m)
= 11.9 kN/m

ice accretion (S6 3.12.c)

12mm thickness $\gamma = 9.8\,\text{kN/m}^3$



$$\text{pipe} = \frac{\pi D}{2} \gamma t$$
$$= \frac{\pi}{2} (1.219)(9.8)(.012)$$
$$= 0.225\ \text{kN/m}$$

ice accretion on bridge members

$0.012\text{m}(9.8 \frac{\text{kN}}{\text{m}^3})(.152\text{m}) = 0.018\ \text{kN/m}$ (per linear meter of beam.)

ice accretion on grating

assume grating covers 25% of the area

$0.012\text{m}(9.8\,\text{kN/m}^3)(3\text{m})(0.25) = 0.088\ \text{kN/m}$

Grating weight = 0.25 kPa (4.3 m) = 1.1 kN/m

Pipe weight = 7.64 kN/m

Beam weight = 0.4 kN/m (per linear meter of beam)

Factored:

1.4 (1.1 + 7.64) = 11.8 kN/m

1.4 (0.4) = 0.56 kN/m (per linear meter of beam)

Snow load = 11.9 kN/m

Factored = 0.4 (11.9) = 4.76 kN/m

treat ice accretion load as snow load

0.4 (0.225 kN/m + 0.088 kN/m) = 0.13 kN/m

0.4 (0.02 kN/m) = 0.01 kN/m (per linear meter of beam.)

Combined Loads

11.8 + 4.76 + 0.13 = 16.69 kN/m

0.56 + 0.01 = 0.57 kN/m (per linear meter of beam)

Case 4: 1.25D + 1.5S + 1.3A (A = ice accretion S6-6)

combination of BCBC load case and S6 load case for ice accretion. BCBC load case taken since the case is more conservative than S6 case.

Snow = 11.9 kN/m

Dead:

1.1 kN/m + 7.64 kN/m = 8.74 kN/m

0.4 kN/m (per linear meter of beam)

Ice:

0.225 kN/m + 0.088 kN/m = 0.31 kN/m

0.018 kN/m (per linear meter of beam)

Factored:

1.25(8.74 kN/m) + 1.5(11.9 kN/m) + 1.3(0.31 kN/m) = 29.2 kN/m

1.25(0.4 kN/m) + 1.3(0.018 kN/m) = 0.52 kN/m (per linear meter of beam)

$\boxed{\text{Governing Load Case}}$

SLS Load Combo: $1.0D + 0.9S$ snow taken instead of live load since greater
(CSA S6-06 Table 3.1)

D:

SW of beams = dependent on option

SW of grating = 0.25 kPa (1.5m) = 0.375 kN/m

SW of pipe = = 7.64 kN/m

8.02 kN/m + Beam SW

S:

snow load unfactored = 2.76kPa

tributary widths: pipe = 1.219m/2 = 0.61m

grating = 1.5m

factored loads: pipe = 0.9(2.76kPa)(0.61m) = 1.5 kN/m

grating = 0.9(2.76kPa)(1.5m) = 3.7 kN/m

5.24 kN/m

Pipe Load, P:

$$\frac{7.64\ \text{kN/m}}{2} + 0.9(2.76\text{kPa}\ 0.61\text{m}) = 5.34\ \text{kN/m}$$

Service Load, Q:

Beam SW + 0.375kN/m + 0.9(2.76kPa × 1.5m) = 4.1 kN/m + Beam SW

Appendix C: Preliminary Superstructure Design Calculation

Project: Capstone Design Project
Done by: Myles C
Date: 08/02/2015
Subject: Preliminary Pratt Truss Design
Checked by: Amos K
Page: 1 of 5

Option 1 - 16 panels

Top Chord Members

$C_f = 2281\ kN \quad L = 4.375m \quad L_d = 0.9(4.375m) = 3.938m$

HSS 254×254×9.5 $\quad C_r = 2520\ kN \quad L = 4000\ mm$

Bottom Chord Members

$T_f = 2231\ kN \quad L = 4.375m \quad L_d = 3.938m$

$T_r = \phi A_g F_y = 0.9(9090mm^2)(300MPa) = 2454\ kN$

HSS 254×254×9.5

Vertical Members

$C_f = 595\ kN \quad L = 5m \quad L_d = 0.7(5) = 3.5m$

HSS 152×152×4.8 $\quad C_r = 627kN \quad L = 3600mm$

Diagonal Members

$T_f = 790kN \quad L = 6.64m \quad L_d = 4.648m$

Check HSS 152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760mm^2)\ 300MPa = 745\ kN$

$L/r \leq 300 \quad 4648/59.9 = 78$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members $= 16(5m + 6.64m) = 186.24m/truss$

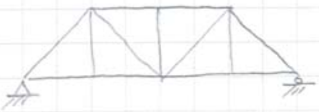
Length of Chord members $= 2 \times 70m - 4.375m = 135.6m/truss$

$Wt. = 186.24m(0.213 kN/m) + 135.6m(0.7 kN/m) = 134.6\ kN/truss$

total truss Wt = 269.2kN

# of connections:

2 per vertical + 2 per diagonal (assume # connections for chords constant between options)



panels = 4

vertical = 3

diagonals = 4

connections = $2(n+n-1)$
$= 2(2n-1)$

where n = # panels

connections = $2(2(4)-1) = 14$

16 panels = 62 connections

Option 2 - 14 panels

Top Chord Members

$C_f = 2383$ kN $L = 5$m $L_d = 0.9(5000) = 4500$ mm

HSS 254×254×9.5 $C_r = 2340$ kN $L = 4800$ mm

Bottom Chord Members

$T_f = 2316$ kN $L = 5$m $L_d = 4.5$m

Check HSS 254×254×9.5

$T_r = \phi A_g F_y = 0.9(9090)300 = 2454$ kN

$L/r < 300$ $4500/99.1 = 45$ ✓

HSS 254×254×9.5

Vertical Members

$C_f = 579$ kN $L = 5$m $L_d = 0.7(5) = 3.5$m

HSS152×152×4.8 $C_r = 627$ kN $L = 3600$mm

Diagnal Members

$T_F = 819$ kN $L = 7.07$m $L_d = 0.7(7.07) = 4.95$m

Check HSS152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760)(300) = 745$ kN

$L/r < 300$ $4950/59.9 = 82.6$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members $= 16(5 + 7.07) = 193.12$ / truss

Wt. $= 193.12^{m}(0.213 \text{ kN/m}) + 135\text{m}(0.7 \text{ kN/m}) = 135.6$ kN/truss

total truss Wt. = 271.2kN

\# connections = 54

Option 3 - 12 panels

Top Chord

$C_f = 2523$ kN $L = 5.833$m $L_d = 0.9(5833) = 5250$

HSS 305 × 305 × 9.5 $C_r = 2970$ $L = 5200$

Bottom Chord

$T_f = 2353$ kN $L = 5.833$m $L_d = 5.25$m

Check above

$T_r = \phi A_g F_y = 0.9(11000)(300) = 2970$ kN

$L/r = \frac{5250}{120} = 44 < 300$ ✓

HSS305 × 305 × 9.5

Vertical

$C_f = 561$ kN $L = 5$m $L_d = 0.7(5) = 3.5$m

HSS 152 × 152 × 4.8 $C_r = 627$ kN $L = 3600$mm

Diagonals

$T_f = 863$ kN $L = 7.68$m $L_d = 0.7(7.68) = 5.376$m

check above

$T_r = \phi A_g F_y = (0.9)(\quad)(300) = 869$ kN

$L/r = 3500/59.9 = 58 < 300$ ✓

Wt. of Steel

Length of Web members = 16 (5m + 5.833m) = 173.3 m / truss

Wt. = 173.3m (0.213 kN/m) + 134.2m (0.849 kN/m) = 150.85 kN/truss

total truss Wt. = 301.7 kN

connections = 46

Option 4 - 10 panels

Top Chord

$C_f = 2626 kN$ $L = 7m$ $L_d = 0.9(7) = 6.3m$

HSS 305 × 305 × 9.5 $C_r = 2700 kN$ $L = 6400mm$

Bottom Chord

$T_f = 2395 kN$ $L = 7m$ $L_d = 6.3m$

$T_r = \phi A_g F_y = 0.9(11000)300 = 3470$

$L/r = 6300/120 = 53 < 300$ ✓

Vertical

$C_f = 540 kN$ $L = 5m$ $L_d = 0.7(5) = 3.5m$

HSS 152 × 152 × 4.8 $C_r = 627 kN$ $L = 3600 mm$

Diagonal

$T_f = 929 kN$ $L = 8.6m$ $L_d = 0.7(8.6) = 6.02m$

$T_r = \phi A_g F_y = 0.9(2760)300 = 869 kN$

$L/r = \frac{6020}{59.9} = 101 < 300$ ✓

Wt. of Steel

Length of Web members = 16(5 + 8.6)m = 217.6 m/truss

Wt. = 217.6m(0.213kN/m) + 133 m (0.849 kN/m) = 159.3 kN/truss

total truss Wt. = 318.6kN

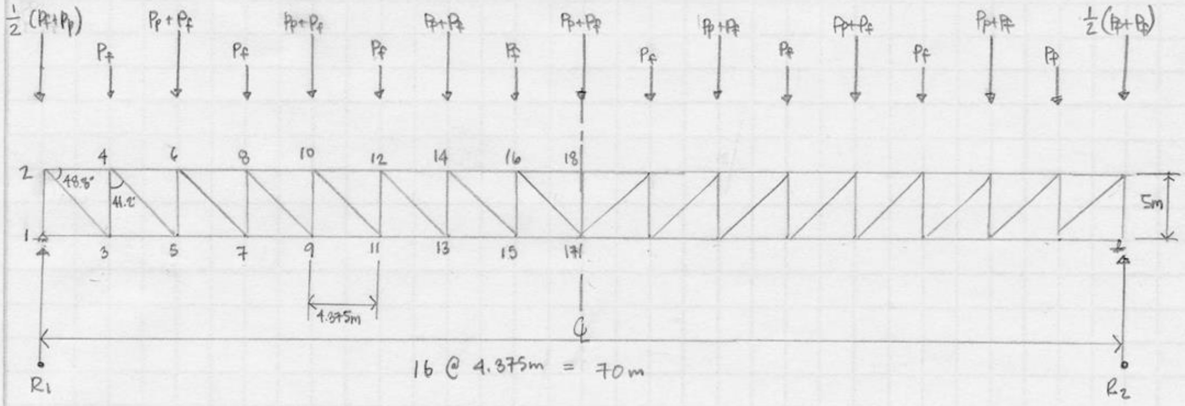
connections = 38

PRATT TRUSS ANALYSIS

IDEAL DEPTH TO SPAN RATIO ~ 10 - 15 (CIDECT 1984)

FOR THE ANAYSIS BELOW, WILL USE AN INITIAL DEPTH OF 5m $\left(\frac{70m}{5m} = 14 \right)$

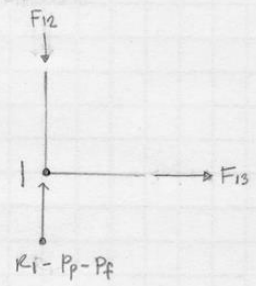
DESIGN REQUIRES
- EQUAL NUMBER OF PANELS
- TRY 16 PANELS → $\frac{70m}{16} = 4.375m$ (PANEL WIDTH)



PIPE LOAD IS SUPPORTED AT LEAST 10m INTERVALS

METHOD OF JOINTS ANALYSIS

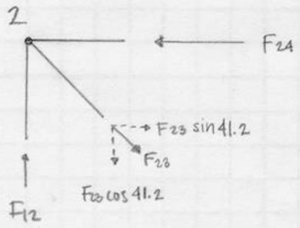
JOINT 1



$$F_{12} = R_1 - \frac{1}{2}(P_f + P_p) \quad (C)$$

$$F_{13} = 0 \quad \text{(ZERO FORCE MEMBER)}$$

JOINT 2



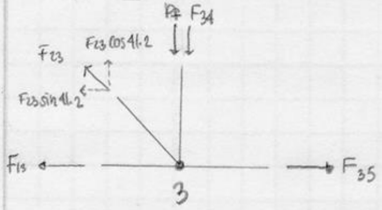
$$F_{23} = \frac{F_{12}}{\cos 41.2} \quad (T)$$

$$F_{24} = F_{23} \sin 41.2 \quad (C)$$

MOJA Inženjer

Project: Capstone Design Project
Subject: Preliminary Pratt Truss Forces
Done by: Amos K
Checked by: Myles C
Date: 10/02/2015
Page: 2 of 7

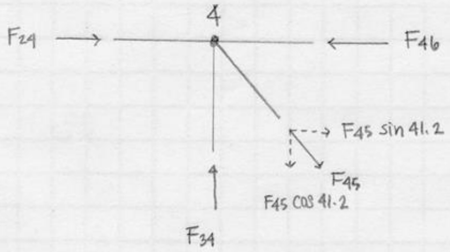
JOINT 3



$F_{34} = F_{28} \cos 41.2 - P_f \quad (C)$

$F_{35} = F_{23} \sin 41.2 + \cancelto{0}{F_{13}} \quad (T)$

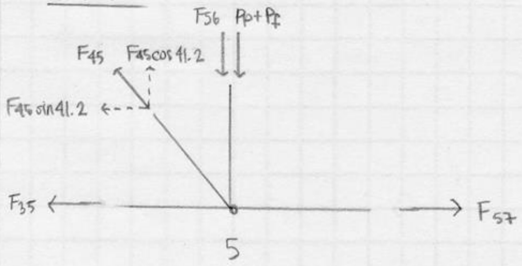
JOINT 4



$F_{45} = \frac{F_{34}}{\cos 41.2} \quad (T)$

$F_{46} = F_{24} + F_{45} \sin 41.2 \quad (C)$

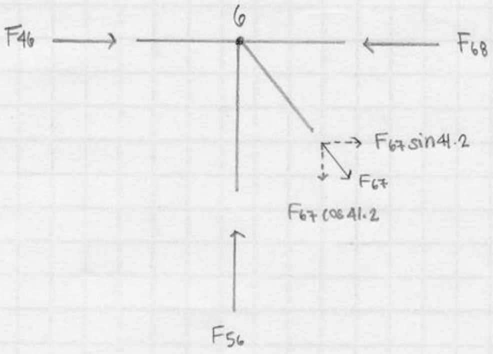
JOINT 5



$F_{56} = F_{45} \cos 41.2 - P_p - P_f \quad (C)$

$F_{57} = F_{35} + F_{45} \sin 41.2 \quad (T)$

JOINT 6



$F_{67} = \frac{F_{56}}{\cos 41.2} \quad (C)$

$F_{68} = F_{46} + F_{67} \sin 41.2 \quad (T)$

PRATT TRUSS ANALYSIS SPREADSHEET

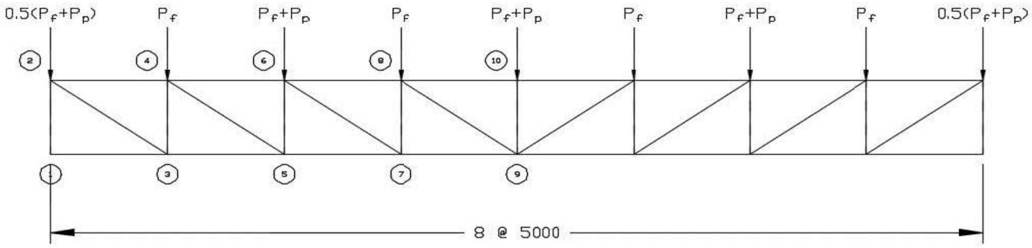
LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

OPTION 2 DRAWING



OPTION 1

TRUSS PARAMETERS			APPLIED LOADS		
Number of panels	10		Pipe load, P_P	60	kN
Panel length	4.000	m	Factored load, P_F	36	kN
Member length (V)	3.500	m	Reaction force, R	329	kN
Member length (D)	5.32	m			
Pipe support length	8.00	m			
Preliminary size	203x203x6.4				
Size width	0.203	m			
Dead load, DL	0.377	kN/m			
Total member length (4.375m section)	16.8	m			

MEMBER	TYPE	FORCE (kN)	
2-4	TC	322	(C)
4-6	TC	602	(C)
6-8	TC	774	(C)
8-10	TC	904	(C)
10-12	TC	924	(C)
1-3	BC	0	(T)
3-5	BC	322	(T)
5-7	BC	602	(T)
7-9	BC	774	(T)
9-11	BC	904	(T)
1-2	V	282	(C)
3-4	V	246	(C)
5-6	V	150	(C)
7-8	V	114	(C)
9-10	V	18	(C)
10-11	V	-18	(C)
2-3	D	428	(T)
4-5	D	373	(T)
6-7	D	227	(T)
8-9	D	173	(T)
9-10	D	27	(T)

Chord member size	178x178x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m
Truss weight	39.1	kN

OPTION 2

TRUSS PARAMETERS

Number of panels	8	
Panel length	5.000	m
Member length (V)	3.500	m
Member length (D)	6.10	m
Pipe support length	10.00	m
Preliminary size	203x203x6.4	
Size width	0.203	m
Dead load, DL	0.377	kN/m
Total member length (5.000m section)	19.6	m

APPLIED LOADS

Pipe load, P_P	75	kN
Factored load, P_F	44	kN
Reaction force, R	327	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	382	(C)
4-6	TC	700	(C)
6-8	TC	848	(C)
8-10	TC	933	(C)
1-3	BC	0	(T)
3-5	BC	382	(T)
5-7	BC	700	(T)
7-9	BC	848	(T)
1-2	V	267	(C)
3-4	V	223	(C)
5-6	V	104	(C)
7-8	V	59	(C)
9-10	V	-59	(C)
2-3	D	466	(T)
4-5	D	389	(T)
6-7	D	181	(T)
8-9	D	104	(T)

Chord member size	203x203x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m

Truss weight	37.4	kN

OPTION 3

TRUSS PARAMETERS

Number of panels	6	
Panel length	6.667	m
Member length (V)	3.500	m
Member length (D)	7.53	m
Pipe support length	6.67	m
Preliminary size	203x203x8.0	
Size width	0.203	m
Dead load, DL	0.466	kN/m
Total member length (5.000m section)	24.4	m

APPLIED LOADS

Pipe load, P_P	50	kN
Factored load, P_F	61	kN
Reaction force, R	332	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	527	(C)
4-6	TC	843	(C)
6-8	TC	949	(C)
1-3	BC	0	(T)
3-5	BC	527	(T)
5-7	BC	843	(T)
1-2	V	277	(C)
3-4	V	166	(C)
5-6	V	55	(C)
7-8	V	-55	(C)
2-3	D	595	(T)
4-5	D	357	(T)
6-7	D	119	(T)

Chord member size	203x203x8.0	
Web member size	89x89x6.4	
Web member weight	0.153	kN/m

Truss weight	44.3	kN

OPTION 4

TRUSS PARAMETERS

Number of panels	4	
Panel length	10.000	m
Member length (V)	3.500	m
Member length (D)	10.59	m
Pipe support length	10.00	m
Preliminary size	254x254x8.0	
Size width	0.254	m
Dead load, DL	0.59	kN/m
Total member length (5.000m section)	34.1	m

APPLIED LOADS

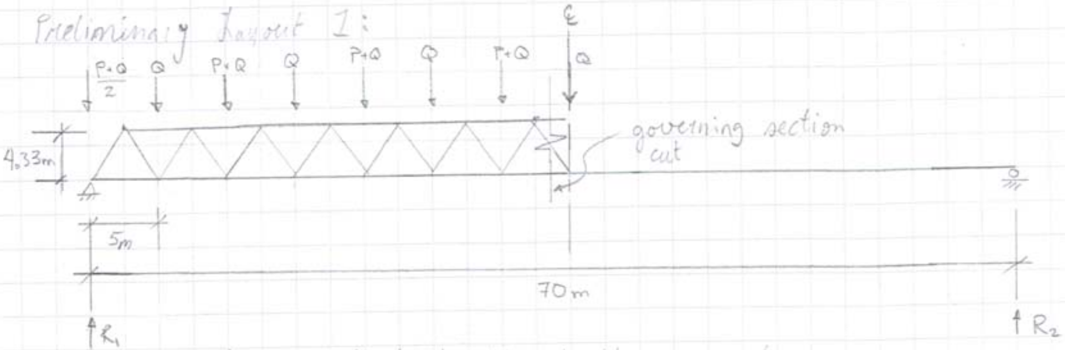
Pipe load, P_P	75	kN
Factored load, P_F	95	kN
Reaction force, R	340	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	729	(C)
4-6	TC	972	(C)
1-3	BC	0	(T)
3-5	BC	729	(T)
1-2	V	255	(C)
3-4	V	85	(C)
5-6	V	-85	(C)
2-3	D	772	(T)
4-5	D	257	(T)

Chord member size	254x254x8.0	
Web member size	102x102x8.0	
Web member weight	0.217	kN/m

Truss weight	53.5	kN

Preliminary Layout 1:



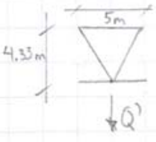
all loads applied to the bottom cord
assume all loads act @ the nodes

P is the loads which are from the pipeline
Q is the remaining other loads

$$P = \underset{(snow)}{2.76\,kPa}\ \underset{(pipe\ w)}{(1.3\,m)}\ \underset{(length)}{(10\,m)}\ \underset{(factor)}{(1.5)} + \underset{(ice)}{.225\,kN/m}\ \underset{(length)}{(10\,m)}\ \underset{(factor)}{(1.3)} + \underset{(SW)}{7.64\,kN/m}\ \underset{(length)}{(10\,m)}\ \underset{(factor)}{(1.25)}$$

$$= 53.82 + 2.925 + 95.5$$

$$P = 152\,kN$$



$$Q' = (2)20m(0.52\,kN/m) = 20.8\,kN$$

Q' is the loads that are per linear meter of members

$$Q = Q' + \underset{(factor)}{1.25}\underset{(grating\ SW)}{(1.1\,kN/m)}\underset{(length)}{(5m)} + \underset{(factors)}{1.3}\underset{(ice)}{(0.088\,kN/m)}\underset{(length)}{(5m)} + \underset{(factor)}{1.5}\underset{(snow)}{(2.76\,kPa)}\underset{(width)}{(3m)}\underset{(lengt}{(5m}$$

$$= 20.8\,kN + 6.9\,kN + 0.57\,kN + 62.1\,kN$$

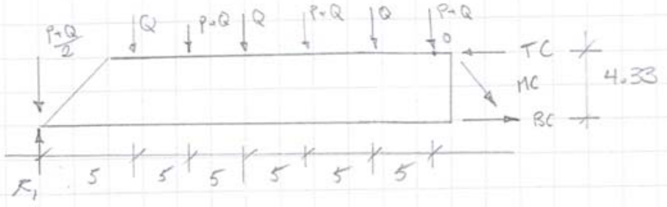
$$Q = 90.4\,kN$$

FBD

$$R_1 = R_2$$

$$R_1 = \frac{P+Q}{2} + 3Q + 3\overset{242.4kN}{(P+Q)} + \tfrac{1}{2}Q = \frac{242.4}{2} + 3(90.4) + 3(242.4) + 90.4/2$$

$$R_1 = 1165\ kN$$



$\Sigma F_v = 0$

$R_1 - \frac{P+Q}{2} - 3Q - 3(P+Q) = MC_v$

$1165\ kN - 242.4\ kN/2 - 3(90.4\ kN) - 3(242.4\ kN) = 45.4\ kN = MC_v$

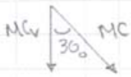
$\Sigma M_0 = 0$ ⟳

$BC\ (4.33m) + (P+Q)(10+20+30/2)m + Q(5+15+25)m - R_1(30m) = 0$

$BC = \frac{R_1(30m) - (P+Q)(45m) - Q(45m)}{4.33m}$

$= \frac{1165kN(30m) - (242.4kN)45m - 90.4kN(45m)}{4.33m}$ } 19974 kNm

$BC = 4613\ kN$



TAN = O/A

$MC_H = MC_v\ TAN(30)$
$= 45.4\ TAN(30)$
$= 26.2\ kN$

$\Sigma F_H = 0$

$BC + MC_H = TC$

$4613kN + 26.2kN = 4639\ kN = TC$

TC = 4639 kN (C)
MC = 45.2 kN (T)
BC = 4613 kN (T)

resisted by two trusses

$C_f = 2320\ kN$ $T_f = 2307\ kN$

Hollow Structural Section Connections and Trusses (HSS CT) from CISC pg 53

$\frac{kL}{r} = 0.9\frac{kL}{r}$ for chord members

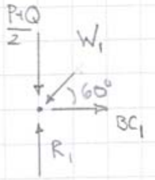
$= 0.9(5000) = 4500\ mm$

From Handbook of Steel Construction (HSC) design tables

HSS 254 × 254 × 9.5 $C_r = 3050$ (L = 4800)

Size the web members - Governing members near the supports

Method of Joints



$\Sigma F_V = 0$

$R_1 - \frac{P+Q}{2} + \sin 60\ W_1 = 0$

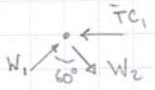
$1165 kN - \frac{242 kN}{2} - \sin(60)\ W_1 = 0$

$W_1 = 1206 kN$ (C)

$\Sigma F_H = 0$

$BC_1 - \cos 60\ W_1 = 0$

$BC_1 = 603 kN$ (T)



$\Sigma F_V = 0$

$\cos 30\ W_1 - \cos 30\ W_2 = 0$

$W_1 = W_2$

$W_2 = 1206 kN$ (T)

$\Sigma F_H = 0$

$2 \sin 30\ W_1 - TC_1 = 0$

$TC_1 = 1206 kN$ (C)

Governing Web Loads

$C_f = 603 kN$

$T_f = 603 kN$

from HSSCT pg 53 $\frac{kL}{r} = 0.7 \frac{kL}{r}$

$= 0.7 (5000)$

$= 3500 mm$

from HSC design table

HSS 127 × 127 × 8.0 $C_r = 681 kN$ $(L = 3600)$

check HSSCT guidelines pg 383

1. 127 > 0.35 (254) = 88.9 ✓
2. 127/254 = 0.5 > 0.01 (254/9.5 + .1) = 0.27 ✓
3. class 1 or 2 ✓
4. 127/8 = 16 < 35 ✓
5. 254/9.5 = 28 > 15 and < 35 ✓

6. minimum web member size
$= 127(0.63)$
$= 80$ mm

7. $0.5(1-\beta) < g/b_0$
$0.5(1 - 127/254) = 0.25 < g/254$

minimum gap = 63.5mm

8. absolute min gap = $4(8) = 32$ mm

9. max eccentricity $= -0.55(4.33) = 2382$ mm
$= 0.25(4.33) = 1083$ mm

recheck guidelines if 89 × 89 × 8.0 used

1. $\frac{127 + 89}{2} = 108 > 89$ ✓

2. $89/254 = 0.35 > 0.27$ ✓

3. class 1 or 2 ✓

4. $89/8 = 11 < 35$ ✓

assume # connections for chords constant between options

connections = 2 per diagonal

panels = 2
diagonals = 4
connections = 8

connections = $2(2)n = 4n$
where n = # panels
$4(2) = 8$ ✓

16 panels = 64 connections
14 panels = 56 connections
12 panels = 48 connections
10 panels = 40 connections

Weights:

16 panels: member length = 4.375m
$W_c = 0.849$ kN/m chord weight
$W_w = 0.213$ kN/m web weight

$W = 2W_w(L \cdot n) + W_c(140 - L)$ where L = member length, n = # panels

$= 2(0.213\text{ kN/m})(4.375\text{m} \cdot 16) + 0.849\text{ kN/m}(140\text{m} - 4.375\text{m})$

$= 29.82\text{kN} + 115.15\text{ kN}$

$= 145$ kN / truss

$= 290$ kN

14 panels: member length = 5m $W_c = 0.700$ kN/m $W_w = 0.279$ kN/m

$W = 2(0.279\text{ kN/m})(5\text{m} \cdot 14) + 0.700\text{ kN/m}(140\text{m} - 5\text{m})$

$= 39.06\text{kN} + 94.5\text{kN}$

$= 133.5$ kN/ truss

$= 267$ kN

12 panels: $L = 5.833\,m$ $W_c = 0.714\,kN/m$ $W_w = 0.278\,kN/m$

$$W = 2(0.278\,kN/m)(5.833\,m \times 12) + 0.714\,kN/m(140\,m - 5.833\,m)$$
$$= 38.9\,kN + 95.8\,kN$$
$$= 134.7\,kN/truss$$

$$= 269.4\,kN$$

10 panels: $L = 7\,m$ $W_c = 0.590\,kN/m$ $W_w = 0.401\,kN/m$

$$W = 2(0.401\,kN/m)(7\,m \times 10) + 0.590\,kN/m(140\,m - 7\,m)$$
$$= 56.14\,kN + 78.47\,kN$$
$$= 134.61\,kN/truss$$

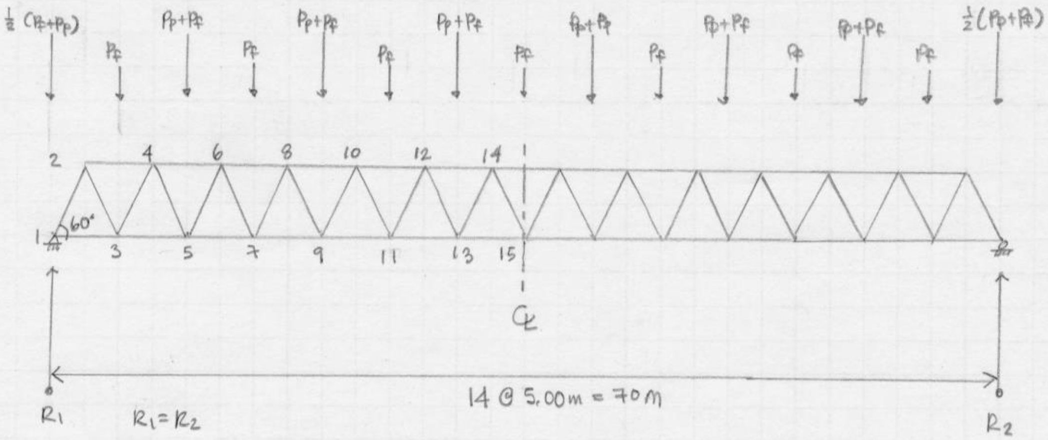
$$= 269.22\,kN$$

WARREN TRUSS CALCULATION

EQUILATERAL TRIANGLES IN DESIGN, $\theta = 60°$

- TRY 14 PANELS → $\frac{70m}{14} = 5.00\,m$ (PANEL WIDTH)

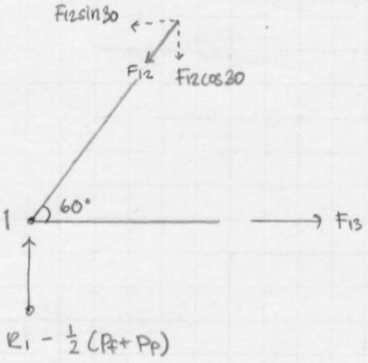
P_p = PIPE LOAD
P_f = FACTORED LOAD



PIPE IS SUPPORTED AT LEAST 10m INTERVALS DUE TO DEFLECTION REQUIREMENTS

METHOD OF JOINTS ANALYSIS

JOINT 1

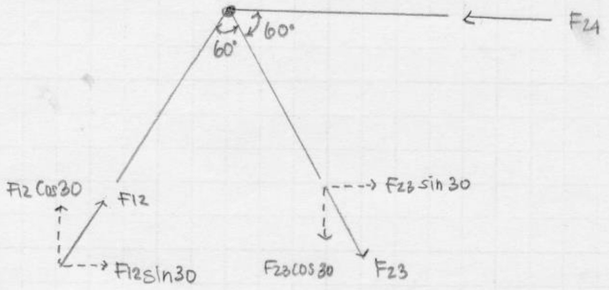


$$R_1 = \frac{1}{2}(P_p + P_f) + F_{12}\cos 30$$

$$F_{12} = \frac{R_1 - \frac{1}{2}(P_p + P_f)}{\cos 30}$$

$$F_{13} = F_{12}\sin 30$$

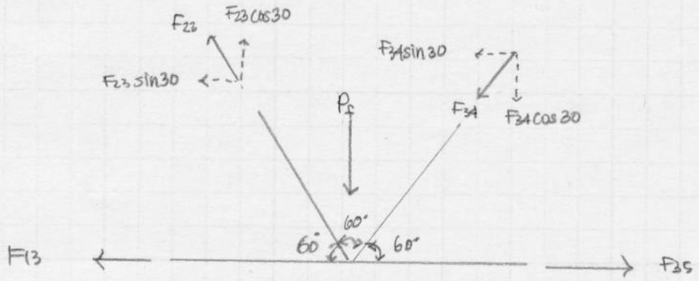
JOINT 2



$$F_{23} = \frac{F_{12}}{\sin 30}$$

$$F_{24} = F_{12}\sin 30 + F_{23}\sin 30$$

JOINT 3



$$F_{34} = \frac{F_{23}\cos 30 - P_f}{\cos 30}$$

$$F_{35} = F_{13} + (F_{23} + F_{34})\sin 30.$$

WARREN TRUSS ANALYSIS SPREADSHEET

LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

FINAL OPTION (40m - A TRUSS)

TRUSS PARAMETERS

Number of panels	8	
Member length	5.000	m
Depth	4.33	m
Pipe support length	5.00	m
Preliminary size	203x152x6.4	
Size width	0.152	m
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	37	kN
Factored load, P_F	43	kN
Reaction force, R	322	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-325	(C)
4-6	TC	-557	(C)
6-8	TC	-697	(C)
8-10	TC	-743	(C)
1-3	BC	163	(T)
3-5	BC	441	(T)
5-7	BC	627	(T)
7-9	BC	720	(T)
1-2	D	-325	(C)
2-3	D	325	(T)
3-4	D	-232	(C)
4-5	D	232	(T)
5-6	D	-139	(C)
6-7	D	139	(T)
7-8	D	-46	(C)
8-9	D	46	(T)

Chord member size	203x152x6.4	
Web member size	114x114x4.8	
Web member weight	0.157	kN/m
Truss Weight	37.1	kN

FINAL OPTION (30m - B TRUSS)

TRUSS PARAMETERS

Number of panels	6	
Member length	5.000	m
Depth	4.33	m
Pipe support length	5.00	m
Preliminary size	203x152x6.4	
Size width	0.152	m
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	37	kN
Factored load, P_F	43	kN
Reaction force, R	241	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-232	(C)
4-6	TC	-372	(C)
6-8	TC	-418	(C)
1-3	BC	116	(T)
3-5	BC	302	(T)
5-7	BC	395	(T)
1-2	D	-232	(C)
2-3	D	232	(T)
3-4	D	-139	(C)
4-5	D	139	(T)
5-6	D	-46	(C)
6-7	D	46	(T)

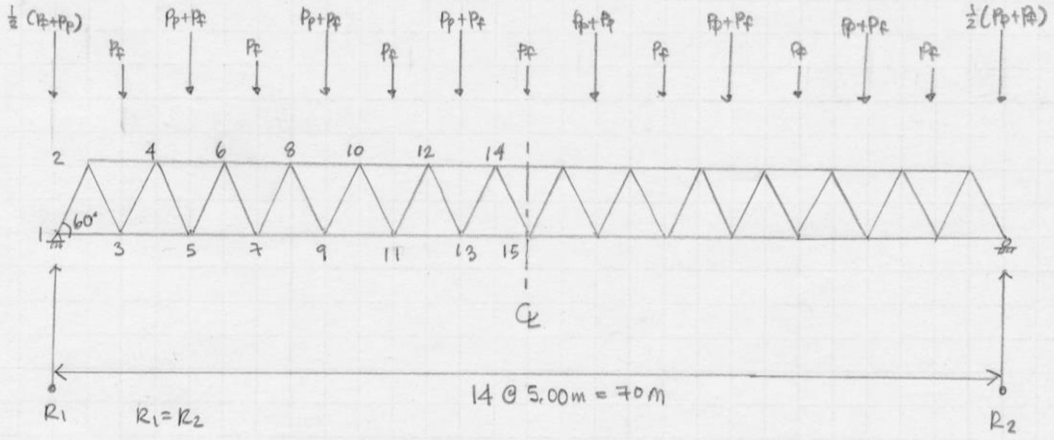
Chord member size	203x152x6.4	
Web member size	114x114x4.8	
Web member weight	0.157	kN/m
Truss Weight	33.9	kN

WARREN TRUSS CALCULATION

EQUILATERAL TRIANGLES IN DESIGN, $\theta = 60°$

- TRY 14 PANELS → $\frac{70\,m}{14} = 5.00\,m$ (PANEL WIDTH)

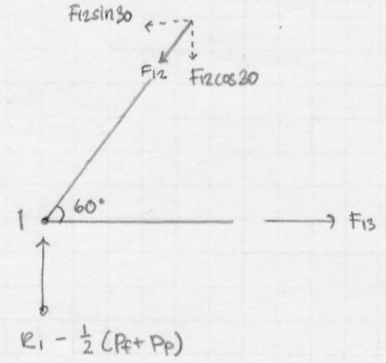
P_P = PIPE LOAD
P_f = FACTORED LOAD



PIPE IS SUPPORTED AT LEAST 10m INTERVALS DUE TO DEFLECTION REQUIREMENTS

METHOD OF JOINTS ANALYSIS

JOINT 1



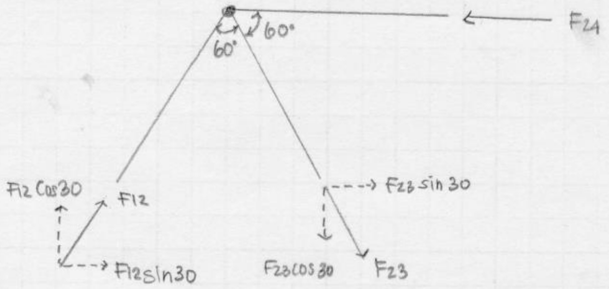
$R_1 = \frac{1}{2}(P_p + P_f) + F_{12}\cos 30$

$F_{12} = \frac{R_1 - \frac{1}{2}(P_p + P_f)}{\cos 30}$

$F_{13} = F_{12}\sin 30$

Project: Capstone Design Project
Subject: Preliminary Warren Truss Forces
Done by: Amos K
Checked by: Myles C
Date: 24/01/2015
Page: 2 of 5

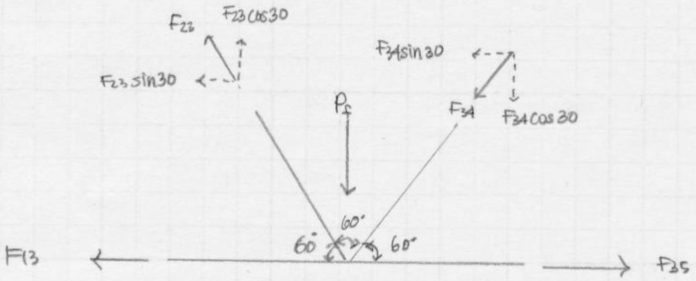
JOINT 2



$$F_{23} = \frac{F_{12}}{\sin 30}$$

$$F_{24} = F_{12}\sin 30 + F_{23}\sin 30$$

JOINT 3



$$F_{34} = \frac{F_{23}\cos 30 - P_f}{\cos 30}$$

$$F_{35} = F_{13} + (F_{23} + F_{34})\sin 30.$$

WARREN TRUSS ANALYSIS SPREADSHEET

LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

FINAL OPTION (40m - A TRUSS)

TRUSS PARAMETERS

Number of panels	8	
Member length	5.000	m
Depth	4.33	m
Pipe support length	5.00	m
Preliminary size	203x152x6.4	
Size width	0.152	m
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	37	kN
Factored load, P_F	43	kN
Reaction force, R	322	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-325	(C)
4-6	TC	-557	(C)
6-8	TC	-697	(C)
8-10	TC	-743	(C)
1-3	BC	163	(T)
3-5	BC	441	(T)
5-7	BC	627	(T)
7-9	BC	720	(T)
1-2	D	-325	(C)
2-3	D	325	(T)
3-4	D	-232	(C)
4-5	D	232	(T)
5-6	D	-139	(C)
6-7	D	139	(T)
7-8	D	-46	(C)
8-9	D	46	(T)

Chord member size	203x152x6.4	
Web member size	114x114x4.8	
Web member weight	0.157	kN/m
Truss Weight	37.1	kN

FINAL OPTION (30m - B TRUSS)

TRUSS PARAMETERS

Number of panels	6
Member length	5.000 m
Depth	4.33 m
Pipe support length	5.00 m
Preliminary size	203x152x6.4
Size width	0.152 m
Dead load, DL	0.327 kN/m
Total member length (5.833m section)	20.0 m

APPLIED LOADS

Pipe load, P_P	37 kN
Factored load, P_F	43 kN
Reaction force, R	241 kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-232	(C)
4-6	TC	-372	(C)
6-8	TC	-418	(C)
1-3	BC	116	(T)
3-5	BC	302	(T)
5-7	BC	395	(T)
1-2	D	-232	(C)
2-3	D	232	(T)
3-4	D	-139	(C)
4-5	D	139	(T)
5-6	D	-46	(C)
6-7	D	46	(T)

Chord member size	203x152x6.4
Web member size	114x114x4.8
Web member weight	0.157 kN/m
Truss Weight	33.9 kN

Project: Capstone Design Project Done by: Myles C Date: 08/02/2015
Subject: Preliminary Pratt Truss Design Checked by: Amos K Page: 1 of 5

Option 1 - 16 panels

Top Chord Members

$C_f = 2281\ kN$ $L = 4.375m$ $L_d = 0.9(4.375m) = 3.938m$

HSS 254×254×9.5 $C_r = 2520\ kN$ $L = 4000\ mm$

Bottom Chord Members

$T_f = 2231\ kN$ $L = 4.375m$ $L_d = 3.938m$

$T_r = \phi A_g F_y = 0.9(9090mm^2)(300MPa) = 2454\ kN$

HSS 254×254×9.5

Vertical Members

$C_f = 595\ kN$ $L = 5\ m$ $L_d = 0.7(5) = 3.5m$

HSS 152×152×4.8 $C_r = 627kN$ $L = 3600\ mm$

Diagonal Members

$T_f = 790kN$ $L = 6.64m$ $L_d = 4.648m$

Check HSS 152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760mm^2)\ 300MPa = 745\ kN$

$L/r \le 300$ $4648/59.9 = 78$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members = 16(5m + 6.64m) = 186.24m/truss

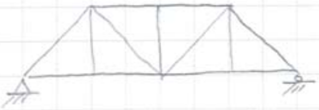
Length of Chord members = 2×70m − 4.375m = 135.6m/truss

Wt. = 186.24m(0.213 kN/m) + 135.6m (0.7 kN/m) = 134.6 kN/truss

total truss Wt = 269.2kN

# of connections:

2 per vertical + 2 per diagonal (assume # connections for chords constant between options)



\# panels = 4

\# vertical = 3

\# diagonals = 4

$$\# \text{ connections} = 2(n+n-1)$$
$$= 2(2n-1)$$

where n = # panels

$$\# \text{ connections} = 2(2(4)-1) = 14$$

16 panels = 62 connections

Option 2 - 14 panels

Top Chord Members

$C_f = 2383$ kN $L = 5$m $L_d = 0.9(5000) = 4500$ mm

HSS 254×254×9.5 $C_r = 2340$ kN $L = 4800$ mm

Bottom Chord Members

$T_f = 2316$ kN $L = 5$m $L_d = 4.5$m

Check HSS 254×254×9.5

$T_r = \phi A_g F_y = 0.9(9090)300 = 2454$ kN

$L/r < 300$ $4500/99.1 = 45$ ✓

HSS 254×254×9.5

Vertical Members

$C_f = 579$ kN $L = 5$m $L_d = 0.7(5) = 3.5$m

HSS152×152×4.8 $C_r = 627$ kN $L = 3600$mm

Diagnal Members

$T_F = 819$ kN $L = 7.07$m $L_d = 0.7(7.07) = 4.95$m

Check HSS152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760)(300) = 745$ kN

$L/r < 300$ $4950/59.9 = 82.6$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members = 16 (5+7.07) = 193.12 / truss

Wt. = $193.12^m(0.213\text{kN/m}) + 135\text{m}(0.7\text{kN/m}) = 135.6$ kN/truss

total truss Wt. = 271.2kN

connections = 54

Option 3 - 12 panels

Top Chord

$C_f = 2523$ kN L = 5.833m $L_d = 0.9(5833) = 5250$

HSS 305×305×9.5 $C_r = 2970$ L = 5200

Bottom Chord

$T_f = 2353$ kN L = 5.833m $L_d = 5.25$m

Check above

$T_r = \phi A_g F_y = 0.9(11000)(300) = 2970$ kN

$L/r = \frac{5250}{120} = 44 < 300$ ✓

HSS305×305×9.5

Vertical

$C_f = 561$ kN L = 5m $L_d = 0.7(5) = 3.5$m

HSS 152×152×4.8 $C_r = 627$ kN L = 3600mm

Diagonals

$T_f = 863$ kN L = 7.68m $L_d = 0.7(7.68) = 5.376$m

check above

$T_r = \phi A_g F_y = (0.9)(\quad)(300) = 869$ kN

$L/r = 3500/59.9 = 58 < 300$ ✓

Wt. of Steel

Length of Web members = 16(5m + 5.833m) = 173.3 m / truss

Wt. = 173.3m(0.213 kN/m) + 134.2m(0.849 kN/m) = 150.85 kN/truss

total truss Wt. = 301.7 kN

connections = 46

Option 4 - 10 panels

Top Chord

$C_f = 2626\text{kN}$ $L = 7\text{m}$ $L_d = 0.9(7) = 6.3\text{m}$

HSS 305 × 305 × 9.5 $C_r = 2700\text{kN}$ $L = 6400\text{mm}$

Bottom Chord

$T_f = 2395\text{kN}$ $L = 7\text{m}$ $L_d = 6.3\text{m}$

$T_r = \phi A_g F_y = 0.9(11000)300 = 3470$

$L/r = 6300/120 = 53 < 300$ ✓

Vertical

$C_f = 540\text{kN}$ $L = 5\text{m}$ $L_d = 0.7(5) = 3.5\text{m}$

HSS 152 × 152 × 4.8 $C_r = 627\text{kN}$ $L = 3600\text{mm}$

Diagonal

$T_f = 929\text{kN}$ $L = 8.6\text{m}$ $L_d = 0.7(8.6) = 6.02\text{m}$

$T_r = \phi A_g F_y = 0.9(2760)300 = 869\text{kN}$

$L/r = \frac{6020}{59.9} = 101 < 300$ ✓

Wt. of Steel

Length of Web members = $16(5 + 8.6)\text{m} = 217.6\text{m/truss}$

$Wt. = 217.6\text{m}(0.213\text{kN/m}) + 133^{m}(0.849\text{kN/m}) = 159.3\text{kN/truss}$

total truss Wt. = 318.6kN

connections = 38

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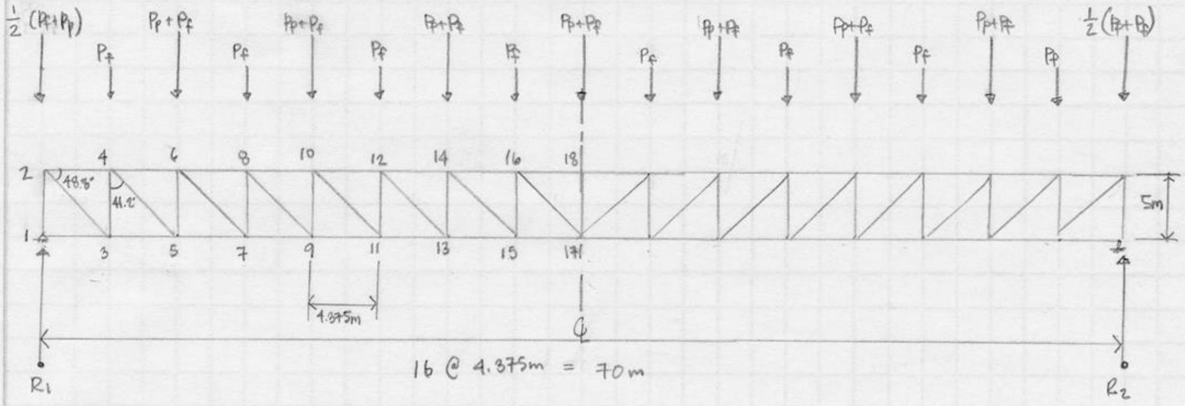
PRATT TRUSS ANALYSIS

IDEAL DEPTH TO SPAN RATIO ~ 10 - 15 (CIDECT 1984)

FOR THE ANAYSIS BELOW, WILL USE AN INITIAL DEPTH OF 5m $\left(\frac{70m}{5m} = 14 \right)$

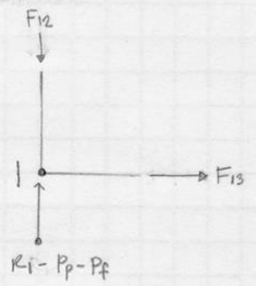
DESIGN REQUIRES
- EQUAL NUMBER OF PANELS
- TRY 16 PANELS → $\frac{70m}{16} = 4.375m$ (PANEL WIDTH)



PIPE LOAD IS SUPPORTED AT LEAST 10m INTERVALS

METHOD OF JOINTS ANALYSIS

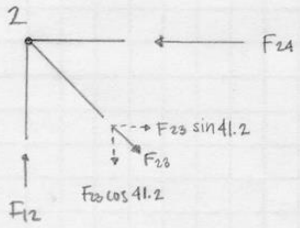
JOINT 1



$F_{12} = R_1 - \frac{1}{2}(P_f+P_p)$ (C)

$F_{13} = 0$ (ZERO FORCE MEMBER)

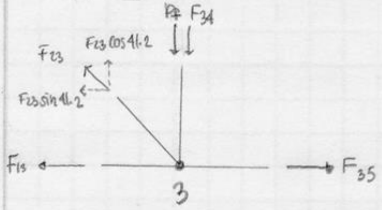
JOINT 2



$F_{23} = \frac{F_{12}}{\cos 41.2}$ (T)

$F_{24} = F_{23} \sin 41.2$ (C)

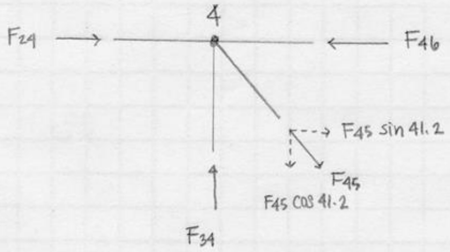
JOINT 3



$$F_{34} = F_{23} \cos 41.2 - P_f \quad (C)$$

$$F_{35} = F_{23} \sin 41.2 + \cancelto{0}{F_{13}} \quad (T)$$

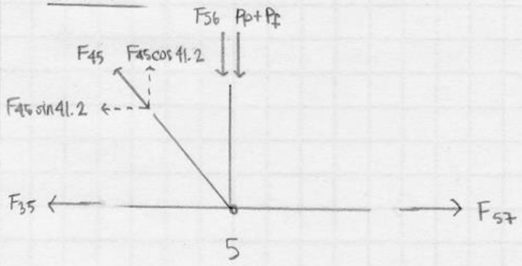
JOINT 4



$$F_{45} = \frac{F_{34}}{\cos 41.2} \quad (T)$$

$$F_{46} = F_{24} + F_{45} \sin 41.2 \quad (C)$$

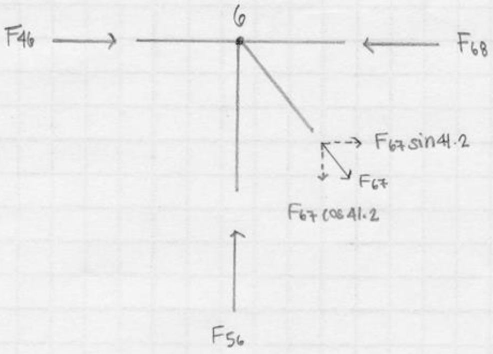
JOINT 5



$$F_{56} = F_{45} \cos 41.2 - P_p - P_f \quad (C)$$

$$F_{57} = F_{35} + F_{45} \sin 41.2 \quad (T)$$

JOINT 6



$$F_{67} = \frac{F_{56}}{\cos 41.2} \quad (C)$$

$$F_{68} = F_{46} + F_{67} \sin 41.2 \quad (T)$$

PRATT TRUSS ANALYSIS SPREADSHEET

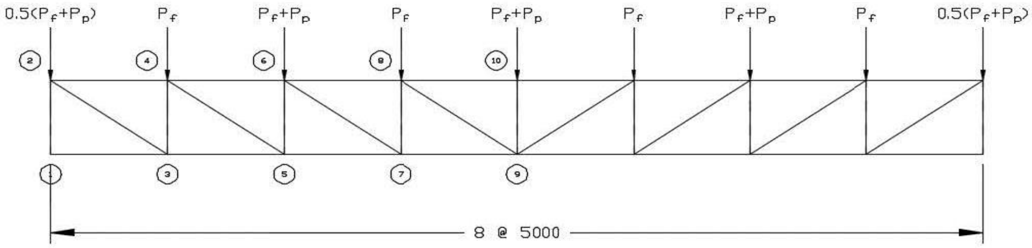
LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

OPTION 2 DRAWING



OPTION 1

TRUSS PARAMETERS

Number of panels	10	
Panel length	4.000	m
Member length (V)	3.500	m
Member length (D)	5.32	m
Pipe support length	8.00	m
Preliminary size	203x203x6.4	
Size width	0.203	m
Dead load, DL	0.377	kN/m
Total member length (4.375m section)	16.8	m

APPLIED LOADS

Pipe load, P_P	60	kN
Factored load, P_F	36	kN
Reaction force, R	329	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	322	(C)
4-6	TC	602	(C)
6-8	TC	774	(C)
8-10	TC	904	(C)
10-12	TC	924	(C)
1-3	BC	0	(T)
3-5	BC	322	(T)
5-7	BC	602	(T)
7-9	BC	774	(T)
9-11	BC	904	(T)
1-2	V	282	(C)
3-4	V	246	(C)
5-6	V	150	(C)
7-8	V	114	(C)
9-10	V	18	(C)
10-11	V	-18	(C)
2-3	D	428	(T)
4-5	D	373	(T)
6-7	D	227	(T)
8-9	D	173	(T)
9-10	D	27	(T)

Chord member size	178x178x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m

Truss weight	39.1	kN

OPTION 2

TRUSS PARAMETERS

Number of panels	8	
Panel length	5.000	m
Member length (V)	3.500	m
Member length (D)	6.10	m
Pipe support length	10.00	m
Preliminary size	203x203x6.4	
Size width	0.203	m
Dead load, DL	0.377	kN/m
Total member length (5.000m section)	19.6	m

APPLIED LOADS

Pipe load, P_P	75	kN
Factored load, P_F	44	kN
Reaction force, R	327	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	382	(C)
4-6	TC	700	(C)
6-8	TC	848	(C)
8-10	TC	933	(C)
1-3	BC	0	(T)
3-5	BC	382	(T)
5-7	BC	700	(T)
7-9	BC	848	(T)
1-2	V	267	(C)
3-4	V	223	(C)
5-6	V	104	(C)
7-8	V	59	(C)
9-10	V	-59	(C)
2-3	D	466	(T)
4-5	D	389	(T)
6-7	D	181	(T)
8-9	D	104	(T)

Chord member size	203x203x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m
Truss weight	37.4	kN

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OPTION 3

TRUSS PARAMETERS			**APPLIED LOADS**		
Number of panels	6		Pipe load, P_P	50	kN
Panel length	6.667	m	Factored load, P_F	61	kN
Member length (V)	3.500	m	Reaction force, R	332	kN
Member length (D)	7.53	m			
Pipe support length	6.67	m			
Preliminary size	203x203x8.0				
Size width	0.203	m			
Dead load, DL	0.466	kN/m			
Total member length (5.000m section)	24.4	m			

MEMBER	TYPE	FORCE (kN)	
2-4	TC	527	(C)
4-6	TC	843	(C)
6-8	TC	949	(C)
1-3	BC	0	(T)
3-5	BC	527	(T)
5-7	BC	843	(T)
1-2	V	277	(C)
3-4	V	166	(C)
5-6	V	55	(C)
7-8	V	-55	(C)
2-3	D	595	(T)
4-5	D	357	(T)
6-7	D	119	(T)

Chord member size	203x203x8.0	
Web member size	89x89x6.4	
Web member weight	0.153	kN/m
Truss weight	44.3	kN

Project:	Capstone Design Project	Done by:	Amos K	Date:	10/02/2015
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OPTION 4

TRUSS PARAMETERS

Number of panels	4	
Panel length	10.000	m
Member length (V)	3.500	m
Member length (D)	10.59	m
Pipe support length	10.00	m
Preliminary size	254x254x8.0	
Size width	0.254	m
Dead load, DL	0.59	kN/m
Total member length (5.000m section)	34.1	m

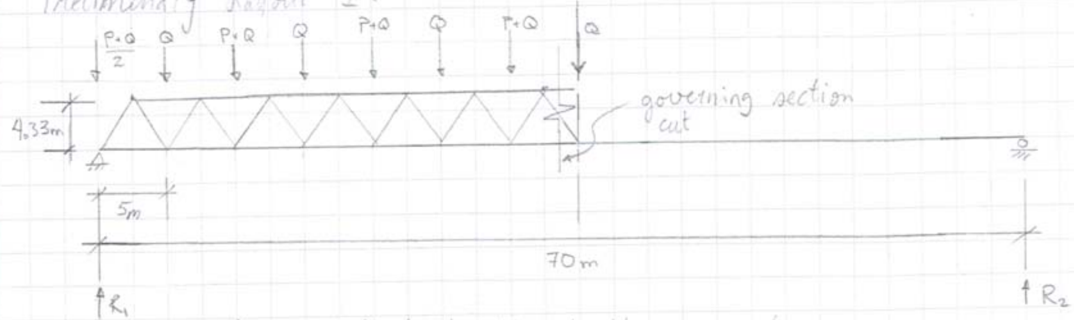
APPLIED LOADS

Pipe load, P_P	75	kN
Factored load, P_F	95	kN
Reaction force, R	340	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	729	(C)
4-6	TC	972	(C)
1-3	BC	0	(T)
3-5	BC	729	(T)
1-2	V	255	(C)
3-4	V	85	(C)
5-6	V	-85	(C)
2-3	D	772	(T)
4-5	D	257	(T)

Chord member size	254x254x8.0	
Web member size	102x102x8.0	
Web member weight	0.217	kN/m
Truss weight	53.5	kN

Preliminary Layout 1:

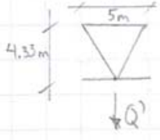


all loads applied to the bottom cord
assume all loads act @ the nodes

P is the loads which are from the pipeline
Q is the remaining other loads

$P = 2.76\,\text{kPa}(1.3\,\text{m})(10\,\text{m})(1.5) + .225\,\text{kN/m}(10\,\text{m})(1.3) + 7.64\,\text{kN/m}(10\,\text{m})(1.25)$
(snow) (pipe w) (length)(factor) (ice) (length)(factor) (SW)(length) (factor)
$= 53.82 + 2.925 + 95.5$

$P = 152\,\text{kN}$



$Q' = (2)20\,\text{m}(0.52\,\text{kN/m}) = 20.8\,\text{kN}$

Q' is the loads that are per linear meter of members

(factor)(grating SW)(length) (factors)(ice) (length) (factor)(snow)(width)(lengt
$Q = Q' + 1.25(1.1\,\text{kN/m})(5\,\text{m}) + 1.3(0.088\,\text{kN/m})(5\,\text{m}) + 1.5(2.76\,\text{kPa})(3\,\text{m})(5\text{m}$
$= 20.8\,\text{kN} + 6.9\,\text{kN} + 0.57\,\text{kN} + 62.1\,\text{kN}$

$Q = 90.4\,\text{kN}$

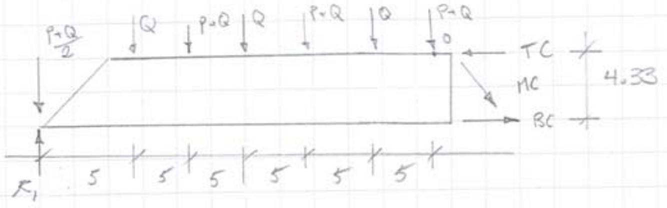
FBD

$R_1 = R_2$

242.4kN

$R_1 = \frac{P+Q}{2} + 3Q + 3(P+Q) + \frac{1}{2}Q = \frac{242.4}{2} + 3(90.4) + 3(242.4) + 90.4/2$

$R_1 = 1165\,\text{kN}$



$\Sigma F_v = 0$

$$R_1 - \frac{P+Q}{2} - 3Q - 3(P+Q) = MC_v$$

$$1165\text{ kN} - 242.4\text{kN}/2 - 3(90.4\text{ kN}) - 3(242.4\text{kN}) = 45.4\text{ kN} = MC_v$$

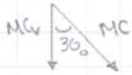
$\Sigma M_o = 0$ ↻

$$BC\,(4.33\text{m}) + (P+Q)(10+20+30/2)\text{m} + Q(5+15+25)\text{m} - R_1(30\text{m}) = 0$$

$$BC = \frac{R_1(30\text{m}) - (P+Q)(45\text{m}) - Q(45\text{m})}{4.33\text{m}}$$

$$= \frac{\left.1165\text{kN}(30\text{m}) - (242.4\text{kN})45\text{m} - 90.4\text{kN}(45\text{m})\right\} 19974\text{ kNm}}{4.33\text{m}}$$

$$BC = 4613\text{ kN}$$



$\text{TAN} = O/A$

$$MC_H = MC_v \,\text{TAN}(30)$$
$$= 45.4\,\text{TAN}(30)$$
$$= 26.2\text{ kN}$$

$\Sigma F_H = 0$

$$BC + MC_H = TC$$

$$4613\text{kN} + 26.2\text{kN} = 4639\text{ kN} = TC$$

$$\boxed{\begin{array}{lll} TC = & 4639\text{ kN} & (C) \\ MC = & 45.2\text{ kN} & (T) \\ BC = & 4613\text{ kN} & (T) \end{array}}$$

resisted by two trusses

$$C_f = 2320\text{ kN} \qquad T_f = 2307\text{ kN}$$

Hollow Structural Section Connections and Trusses (HSS CT)
from CISC pg 53

$$\frac{KL}{r} = 0.9\frac{L}{r} \quad \text{for chord members}$$

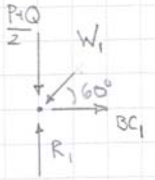
$$= 0.9(5000) = 4500\text{ mm}$$

From Handbook of Steel Construction (HSC) design tables

$$\text{HSS } 254 \times 254 \times 9.5 \qquad C_r = 3050 \quad (L = 4800)$$

Size the web members - Governing members near the supports

Method of Joints



$\Sigma F_v = 0$

$R_1 - \frac{P+Q}{2} + \sin 60\, W_1 = 0$

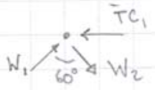
$1165\text{kN} - \frac{242\text{kN}}{2} - \sin(60)\, W_1 = 0$

$W_1 = 1206\text{kN}$ (C)

$\Sigma F_H = 0$

$BC_1 - \cos 60\, W_1 = 0$

$BC_1 = 603\text{kN}$ (T)



$\Sigma F_v = 0$

$\cos 30\, W_1 - \cos 30\, W_2 = 0$

$W_1 = W_2$

$W_2 = 1206\text{kN}$ (T)

$\Sigma F_H = 0$

$2\sin 30\, W_1 - TC_1 = 0$

$TC_1 = 1206\text{kN}$ (C)

Governing Web Loads

$C_f = 603\text{kN}$

$T_f = 603\text{kN}$

from HSSCT pg 53 $\frac{kL}{r} = 0.7\frac{kL}{r}$

$= 0.7(5000)$

$= 3500\text{mm}$

from HSC design table

HSS127×127×8.0 $C_r = 681\text{kN}$ $(L = 3600)$

check HSSCT guidelines pg 383

1. $127 > 0.35(254) = 88.9$ ✓
2. $127/254 = 0.5 > 0.01(254/9.5+.1) = 0.27$ ✓
3. class 1 or 2 ✓
4. $127/8 = 16 < 35$ ✓
5. $254/9.5 = 28 > 15$ and < 35 ✓

6. minimum web member size
$= 127(0.63)$
$= 80$ mm

7. $0.5(1-\beta) < g/b_0$
$0.5(1 - 127/254) = 0.25 < g/254$

minimum gap = 63.5mm

8. absolute min gap = $4(8) = 32$mm

9. max eccentricity $= -0.55(4.33) = 2382$ mm
$= 0.25(4.33) = 1083$ mm

recheck guidelines if 89 × 89 × 8.0 used

1. $\frac{127+89}{2} = 108 > 89$ ✓

2. $89/254 = 0.35 > 0.27$ ✓

3. class 1 or 2 ✓

4. $89/8 = 11 < 35$ ✓

assume # connections for chords constant between options

connections = 2 per diagonal

panels = 2

diagonals = 4

connections = 8

connections = 2(2)n = 4n

where n = # panels

4(2) = 8 ✓

16 panels = 64 connections

14 panels = 56 connections

12 panels = 48 connections

10 panels = 40 connections

Weights:

16 panels: member length = 4.375m

W_c = 0.849 kN/m chord weight
W_w = 0.213 kN/m web weight

$W = 2W_w(L \cdot n) + W_c(140 - L)$ where L = member length
n = # panels

$= 2(0.213\ kN/m)(4.375m \cdot 16)$
$+ 0.849\ kN/m\ (140m - 4.375m)$
$= 29.82kN + 115.15\ kN$

$= 145\ kN$ / truss

$= 290\ kN$

14 panels: member length = 5m W_c = 0.700 kN/m W_w = 0.279 kN/m

$W = 2(0.279\ kN/m)(5m \cdot 14) + 0.700\ kN/m(140m - 5m)$
$= 39.06kN + 94.5kN$

$= 133.5\ kN$/ truss

$= 267\ kN$

12 panels: $L = 5.833\text{m}$ $W_c = 0.714\text{kN/m}$ $W_w = 0.278\text{kN/m}$

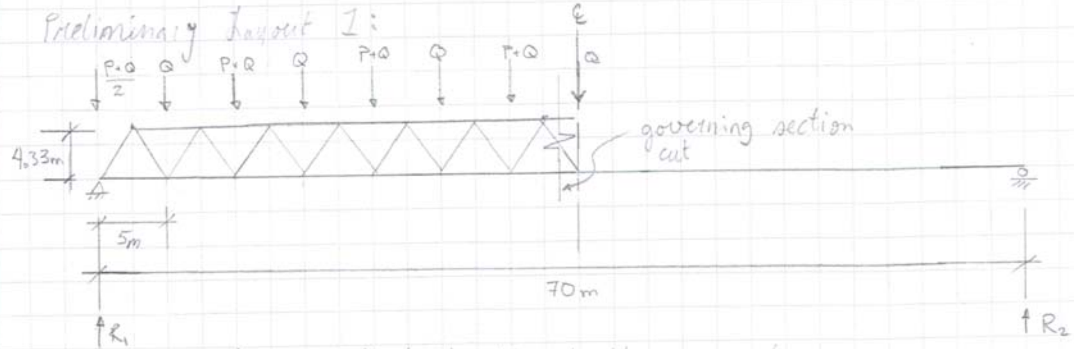
$$\begin{aligned} W &= 2(0.278\text{kN/m})(5.833\text{m} \times 12) + 0.714\text{ kN/m}(140\text{m} - 5.833\text{m}) \\ &= 38.9\text{kN} + 95.8\text{kN} \\ &= 134.7\text{ kN/truss} \\ &= 269.4\text{ kN} \end{aligned}$$

10 panels: $L = 7\text{m}$ $W_c = 0.590\text{ kN/m}$ $W_w = 0.401\text{ kN/m}$

$$\begin{aligned} W &= 2(0.401\text{ kN/m})(7\text{m} \times 10) + 0.590\text{ kN/m}(140\text{m} - 7\text{m}) \\ &= 56.14\text{kN} + 78.47\text{ kN} \\ &= 134.61\text{ kN/truss} \\ &= 269.22\text{ kN} \end{aligned}$$

Project: Capstone Design Project Done by: Myles C Date: 26/01/2015
Subject: Preliminary Warren Truss Design Checked by: Amos K Page: 1 of 6

Preliminary Layout 1:

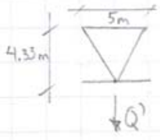


all loads applied to the bottom cord
assume all loads act @ the nodes

P is the loads which are from the pipeline
Q is the remaining other loads

$$P = \underset{(snow)}{2.76\,kPa}\ \underset{(pipe\ w)}{(1.3\,m)}\ \underset{(length)}{(10\,m)}\ \underset{(factor)}{(1.5)} + \underset{(ice)}{.225\,kN/m}\ \underset{(length)}{(10\,m)}\ \underset{(factor)}{(1.3)} + \underset{(SW)}{7.64\,kN/m}\ \underset{(length)}{(10\,m)}\ \underset{(factor)}{(1.25)}$$
$$= 53.82 + 2.925 + 95.5$$

$P = 152\,kN$



$Q' = (2)20m(0.52\,kN/m) = 20.8\,kN$

Q' is the loads that are per linear meter of members

$$Q = Q' + \underset{(factor)}{1.25}\ \underset{(grating\ SW)}{(1.1\,kN/m)}\ \underset{(length)}{(5m)} + \underset{(factors)}{1.3}\ \underset{(ice)}{(0.088\,kN/m)}\ \underset{(length)}{(5m)} + \underset{(factor)}{1.5}\ \underset{(snow)}{(2.76\,kPa)}\ \underset{(width)}{(3m)}\ (5m$$
$$= 20.8\,kN + 6.9\,kN + 0.57\,kN + 62.1\,kN$$

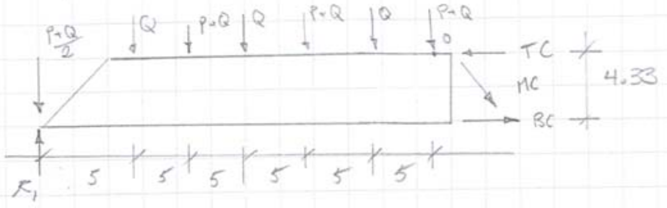
$Q = 90.4\,kN$

FBD

$R_1 = R_2$

$$R_1 = \frac{P+Q}{2} + 3Q + 3\overset{242.4kN}{(P+Q)} + \tfrac{1}{2}Q = \frac{242.4}{2} + 3(90.4) + 3(242.4) + 90.4/2$$

$R_1 = 1165\,kN$



$\Sigma F_v = 0$

$R_1 - \frac{P+Q}{2} - 3Q - 3(P+Q) = MC_v$

$1165\ kN - 242.4\ kN/2 - 3(90.4\ kN) - 3(242.4\ kN) = 45.4\ kN = MC_v$

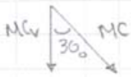
$\Sigma M_o = 0$ ⟳

$BC(4.33m) + (P+Q)(10+20+30/2)m + Q(5+15+25)m - R_1(30m) = 0$

$$BC = \frac{R_1(30m) - (P+Q)(45m) - Q(45m)}{4.33m}$$

$$= \frac{1165kN(30m) - (242.4kN)45m - 90.4kN(45m)}{4.33m} \} 19974\ kNm$$

$BC = 4613\ kN$



TAN = O/A

$MC_H = MC_v\ TAN(30)$
$= 45.4\ TAN(30)$
$= 26.2\ kN$

$\Sigma F_H = 0$

$BC + MC_H = TC$

$4613kN + 26.2kN = 4639\ kN = TC$

TC = 4639 kN (C)
MC = 45.2 kN (T)
BC = 4613 kN (T)

resisted by two trusses

$C_f = 2320\ kN$ $T_f = 2307\ kN$

Hollow Structural Section Connections and Trusses (HSS CT) from CISC pg 53

$\frac{kL}{r} = 0.9\frac{kL}{r}$ for chord members

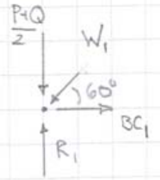
$= 0.9(5000) = 4500\ mm$

From Handbook of Steel Construction (HSC) design tables

HSS 254 × 254 × 9.5 C_r = 3050 (L=4800)

Size the web members - Governing members near the supports

Method of Joints



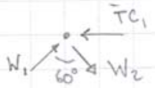
$\Sigma F_V = 0$

$R_1 - \frac{P+Q}{2} + \sin 60\, W_1 = 0$

$1165\text{kN} - \frac{242\text{kN}}{2} - \sin(60)\, W_1 = 0$ $W_1 = 1206\text{kN}$ (C)

$\Sigma F_H = 0$

$BC_1 - \cos 60\, W_1 = 0$ $BC_1 = 603\text{ kN}$ (T)



$\Sigma F_V = 0$

$\cos 30\, W_1 - \cos 30\, W_2 = 0$

$W_1 = W_2$ $W_2 = 1206\text{kN}$ (T)

$\Sigma F_H = 0$

$2\sin 30\, W_1 - TC_1 = 0$ $TC_1 = 1206\text{ kN}$ (C)

Governing Web Loads

$C_f = 603\text{kN}$ from HSSCT pg 53 $\frac{kL}{r} = 0.7\frac{kL}{r}$

$T_f = 603\text{kN}$

$= 0.7(5000)$

$= 3500\text{ mm}$

from HSC design table

HSS 127×127×8.0 $C_r = 681\text{ kN}$ $(L = 3600)$

check HSSCT guidelines pg 383

1. $127 > 0.35(254) = 88.9$ ✓
2. $127/254 = 0.5 > 0.01(254/9.5 + .1) = 0.27$ ✓
3. class 1 or 2 ✓
4. $127/8 = 16 < 35$ ✓
5. $254/9.5 = 28 > 15$ and < 35 ✓

6. minimum web member size
$= 127 (0.63)$
$= 80 \text{ mm}$

7. $0.5(1 - \beta) < g/b_0$
$0.5(1 - 127/254) = 0.25 < g/254$

minimum gap = 63.5mm

8. absolute min gap = 4(8) = 32mm

9. max eccentricity $= -0.55(4.33) = 2382 \text{ mm}$
$= 0.25(4.33) = 1083 \text{ mm}$

recheck guidelines if 89 × 89 × 8.0 used

1. $\frac{127 + 89}{2} = 108 > 89$ ✓

2. $89/254 = 0.35 > 0.27$ ✓

3. class 1 or 2 ✓

4. $89/8 = 11 < 35$ ✓

assume # connections for chords constant between options

connections = 2 per diagonal

panels = 2

diagonals = 4

connections = 8

connections = $2(2)n = 4n$

where n = # panels

$4(2) = 8$ ✓

16 panels = 64 connections

14 panels = 56 connections

12 panels = 48 connections

10 panels = 40 connections

Weights:

16 panels: member length = 4.375m

$W_c = 0.849$ kN/m chord weight

$W_w = 0.213$ kN/m web weight

$W = 2W_w(L \cdot n) + W_c(140 - L)$ where L = member length, n = # panels

$= 2(0.213\,\text{kN/m})(4.375\text{m} \cdot 16) + 0.849\,\text{kN/m}(140\text{m} - 4.375\text{m})$

$= 29.82\,\text{kN} + 115.15\,\text{kN}$

$= 145\,\text{kN / truss}$

$= 290\,\text{kN}$

14 panels: member length = 5m $W_c = 0.700$ kN/m $W_w = 0.279$ kN/m

$W = 2(0.279\,\text{kN/m})(5\text{m} \cdot 14) + 0.700\,\text{kN/m}(140\text{m} - 5\text{m})$

$= 39.06\,\text{kN} + 94.5\,\text{kN}$

$= 133.5\,\text{kN/ truss}$

$= 267\,\text{kN}$

12 panels: $L = 5.833_m$ $W_c = 0.714 kN/m$ $W_w = 0.278 kN/m$

$$W = 2(0.278 kN/m)(5.833_m \times 12) + 0.714 kN/m (140_m - 5.833_m)$$
$$= 38.9 kN + 95.8 kN$$
$$= 134.7 kN / truss$$

$$= 269.4 kN$$

10 panels: $L = 7m$ $W_c = 0.590 kN/m$ $W_w = 0.401 kN/m$

$$W = 2(0.401 kN/m)(7m \times 10) + 0.590 kN/m (140_m - 7m)$$
$$= 56.14 kN + 78.47 kN$$
$$= 134.61 kN / truss$$

$$= 269.22 kN$$

Project: Capstone Design Project
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Done by: Amos K
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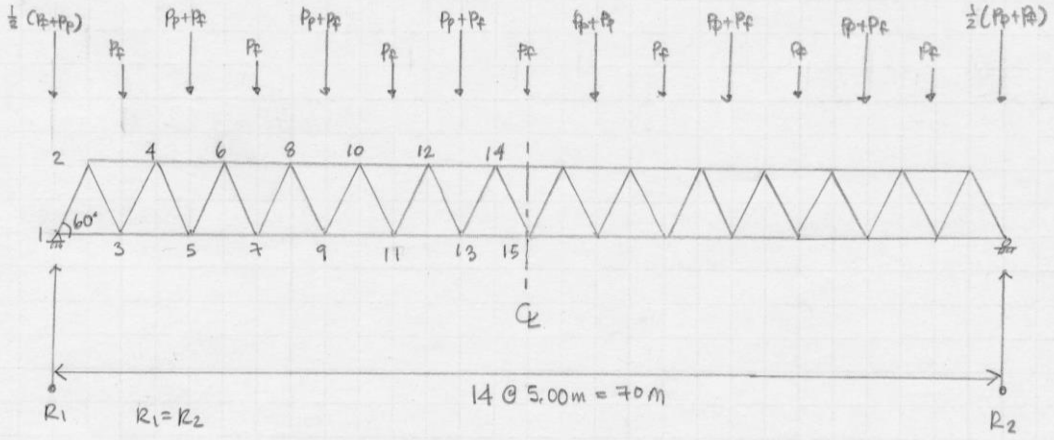
WARREN TRUSS CALCULATION

EQUILATERAL TRIANGLES IN DESIGN, θ = 60°

- TRY 14 PANELS → $\frac{70m}{14}$ = 5.00 m (PANEL WIDTH)

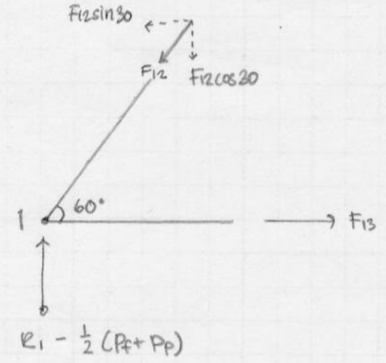
P_P = PIPE LOAD
P_F = FACTORED LOAD



PIPE IS SUPPORTED AT LEAST 10m INTERVALS DUE TO DEFLECTION REQUIREMENTS

METHOD OF JOINTS ANALYSIS

JOINT 1



$R_1 = \frac{1}{2}(P_p+P_f) + F_{12}\cos 30$

$F_{12} = \frac{R_1 - \frac{1}{2}(P_p+P_f)}{\cos 30}$

$F_{13} = F_{12}\sin 30$

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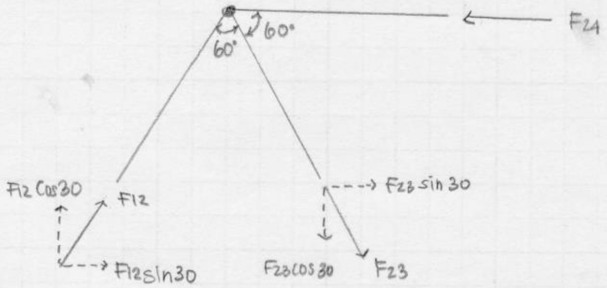
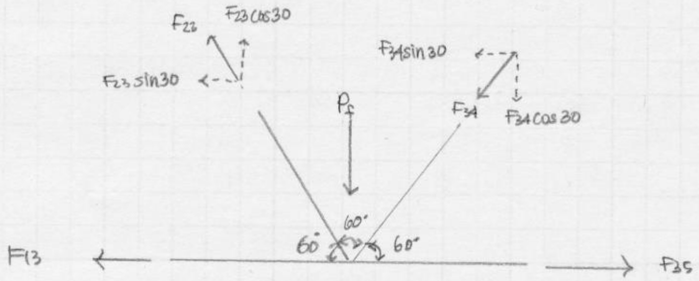
JOINT 2



$$F_{23} = \frac{F_{12}}{\sin 30}$$

$$F_{24} = F_{12}\sin 30 + F_{23}\sin 30$$

JOINT 3



$$F_{34} = \frac{F_{23}\cos 30 - P_f}{\cos 30}$$

$$F_{35} = F_{13} + (F_{23} + F_{34})\sin 30.$$

WARREN TRUSS ANALYSIS SPREADSHEET

LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

FINAL OPTION (40m - A TRUSS)

TRUSS PARAMETERS

Number of panels	8	
Member length	5.000	m
Depth	4.33	m
Pipe support length	5.00	m
Preliminary size	203x152x6.4	
Size width	0.152	m
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	37	kN
Factored load, P_F	43	kN
Reaction force, R	322	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-325	(C)
4-6	TC	-557	(C)
6-8	TC	-697	(C)
8-10	TC	-743	(C)
1-3	BC	163	(T)
3-5	BC	441	(T)
5-7	BC	627	(T)
7-9	BC	720	(T)
1-2	D	-325	(C)
2-3	D	325	(T)
3-4	D	-232	(C)
4-5	D	232	(T)
5-6	D	-139	(C)
6-7	D	139	(T)
7-8	D	-46	(C)
8-9	D	46	(T)

Chord member size	203x152x6.4	
Web member size	114x114x4.8	
Web member weight	0.157	kN/m
Truss Weight	37.1	kN

FINAL OPTION (30m - B TRUSS)

TRUSS PARAMETERS

Number of panels	6	
Member length	5.000	m
Depth	4.33	m
Pipe support length	5.00	m
Preliminary size	203x152x6.4	
Size width	0.152	m
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	37	kN
Factored load, P_F	43	kN
Reaction force, R	241	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-232	(C)
4-6	TC	-372	(C)
6-8	TC	-418	(C)
1-3	BC	116	(T)
3-5	BC	302	(T)
5-7	BC	395	(T)
1-2	D	-232	(C)
2-3	D	232	(T)
3-4	D	-139	(C)
4-5	D	139	(T)
5-6	D	-46	(C)
6-7	D	46	(T)

Chord member size	203x152x6.4	
Web member size	114x114x4.8	
Web member weight	0.157	kN/m
Truss Weight	33.9	kN

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Option 1 - 16 panels

Top Chord Members

$C_f = 2281\ kN$ $L = 4.375m$ $L_d = 0.9(4.375m) = 3.938m$

HSS 254×254×9.5 $C_r = 2520\ kN$ $L = 4000\ mm$

Bottom Chord Members

$T_f = 2231\ kN$ $L = 4.375m$ $L_d = 3.938m$

$T_r = \phi A_g F_y = 0.9(9090mm^2)(300MPa) = 2454\ kN$

HSS 254×254×9.5

Vertical Members

$C_f = 595\ kN$ $L = 5\ m$ $L_d = 0.7(5) = 3.5m$

HSS 152×152×4.8 $C_r = 627kN$ $L = 3600\ mm$

Diagonal Members

$T_f = 790kN$ $L = 6.64m$ $L_d = 4.648m$

Check HSS 152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760mm^2)\ 300MPa = 745\ kN$

$L/r \le 300$ $4648/59.9 = 78$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members = 16(5m + 6.64m) = 186.24m/truss

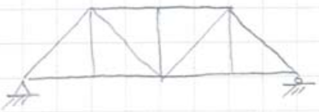
Length of Chord members = 2×70m − 4.375m = 135.6m/truss

Wt. = 186.24m(0.213 kN/m) + 135.6m (0.7 kN/m) = 134.6 kN/truss

total truss Wt = 269.2 kN

\# of connections:

2 per vertical + 2 per diagonal (assume # connections for chords constant between options)



\# panels = 4

\# vertical = 3

\# diagonals = 4

$\# \text{ connections} = 2(n+n-1)$
$= 2(2n-1)$

where n = # panels

$\# \text{ connections} = 2(2(4)-1) = 14$

16 panels = 62 connections

Option 2 - 14 panels

Top Chord Members

$C_f = 2383$ kN $L = 5$m $L_d = 0.9(5000) = 4500$ mm

HSS 254×254×9.5 $C_r = 2340$ kN $L = 4800$ mm

Bottom Chord Members

$T_f = 2316$ kN $L = 5$m $L_d = 4.5$m

Check HSS 254×254×9.5

$T_r = \phi A_g F_y = 0.9(9090)300 = 2454$ kN

$L/r < 300$ $4500/99.1 = 45$ ✓

HSS 254×254×9.5

Vertical Members

$C_f = 579$ kN $L = 5$m $L_d = 0.7(5) = 3.5$m

HSS152×152×4.8 $C_r = 627$ kN $L = 3600$mm

Diagnal Members

$T_F = 819$ kN $L = 7.07$m $L_d = 0.7(7.07) = 4.95$m

Check HSS152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760)(300) = 745$ kN

$L/r < 300$ $4950/59.9 = 82.6$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members $= 16(5 + 7.07) = 193.12$/truss

Wt. $= 193.12^{m}(0.213$ kN/m$) + 135$m$(0.7$kN/m$) = 135.6$ kN/truss

total truss Wt. = 271.2kN

connections = 54

Option 3 - 12 panels

Top Chord

$C_f = 2523$ kN   L = 5.833m $L_d = 0.9(5833) = 5250$

HSS 305×305×9.5 $C_r = 2970$ L = 5200

Bottom Chord

$T_f = 2353$ kN   L = 5.833m $L_d = 5.25$m

Check above

$T_r = \phi A_g F_y = 0.9(11\,000)(300) = 2970$ kN

$L/r = \frac{5250}{120} = 44 < 300$ ✓

HSS305×305×9.5

Vertical

$C_f = 561$ kN   L = 5m $L_d = 0.7(5) = 3.5$m

HSS 152×152×4.8 $C_r = 627$ kN   L = 3600mm

Diagonals

$T_f = 863$ kN   L = 7.68m $L_d = 0.7(7.68) = 5.376$m

check above

$T_r = \phi A_g F_y = (0.9)(\quad)(300) = 869$ kN

$L/r = 3500/59.9 = 58 < 300$ ✓

Wt. of Steel

Length of Web members = 16 (5m + 5.833m) = 173.3 m / truss

Wt. = 173.3m (0.213 kN/m) + 134.2m (0.849 kN/m) = 150.85 kN/truss

total truss Wt. = 301.7 kN

connections = 46

Option 4 - 10 panels

Top Chord

$C_f = 2626 kN \quad L = 7m \quad L_d = 0.9(7) = 6.3m$

HSS 305 × 305 × 9.5 $\quad C_r = 2700 kN \quad L = 6400 mm$

Bottom Chord

$T_f = 2395 kN \quad L = 7m \quad L_d = 6.3m$

$T_r = \phi A_g f_y = 0.9(11000)300 = 3470$

$L/r = 6300/120 = 53 < 300$ ✓

Vertical

$C_f = 540 kN \quad L = 5m \quad L_d = 0.7(5) = 3.5m$

HSS 152 × 152 × 4.8 $\quad C_r = 627 kN \quad L = 3600 mm$

Diagonal

$T_f = 929 kN \quad L = 8.6m \quad L_d = 0.7(8.6) = 6.02m$

$T_r = \phi A_g F_y = 0.9(2760)300 = 869 kN$

$L/r = \frac{6020}{59.9} = 101 < 300$ ✓

Wt. of Steel

Length of Web members = $16(5 + 8.6)m = 217.6 m/truss$

$Wt. = 217.6m(0.213 kN/m) + 133^{\overset{m}{\vee}}(0.849 kN/m) = 159.3 kN/truss$

total truss Wt. = 318.6kN

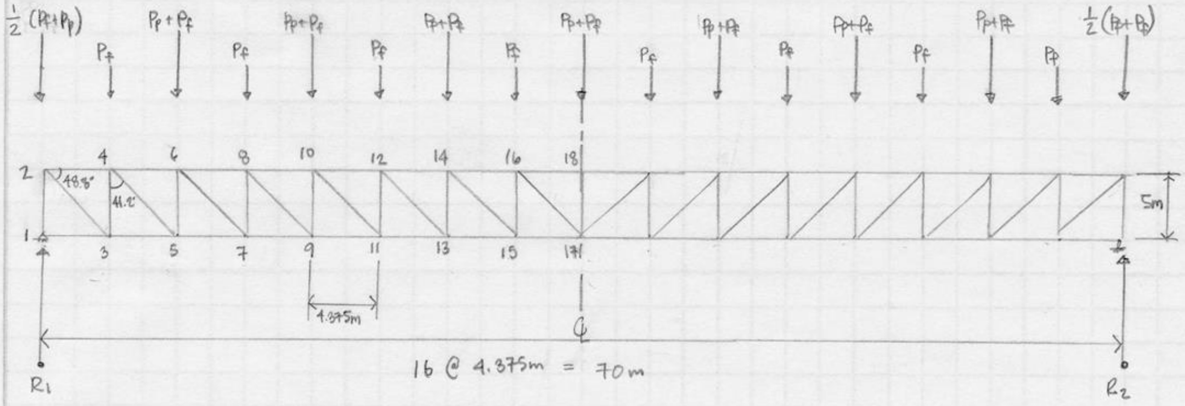
connections = 38

PRATT TRUSS ANALYSIS

IDEAL DEPTH TO SPAN RATIO ~ 10 - 15 (CIDECT 1984)

FOR THE ANAYSIS BELOW, WILL USE AN INITIAL DEPTH OF 5m $\left(\frac{70m}{5m} = 14\right)$

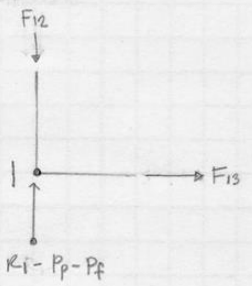
DESIGN REQUIRES
- EQUAL NUMBER OF PANELS
- TRY 16 PANELS → $\frac{70m}{16} = 4.375m$ (PANEL WIDTH)



PIPE LOAD IS SUPPORTED AT LEAST 10m INTERVALS

METHOD OF JOINTS ANALYSIS

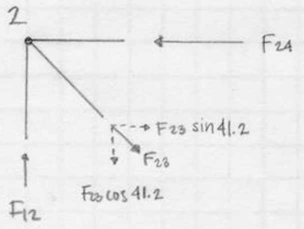
JOINT 1



$F_{12} = R_1 - \frac{1}{2}(P_f + P_p)$ (C)

$F_{13} = 0$ (ZERO FORCE MEMBER)

JOINT 2



$F_{23} = \frac{F_{12}}{\cos 41.2}$ (T)

$F_{24} = F_{23} \sin 41.2$ (C)

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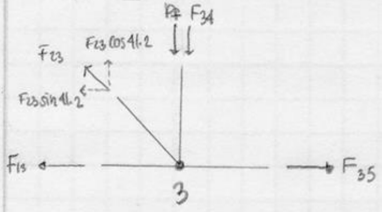
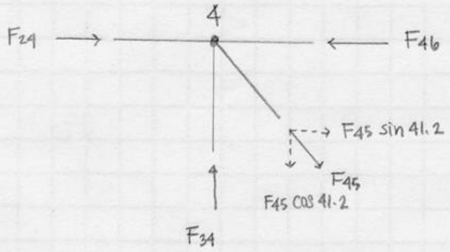
JOINT 3



$$F_{34} = F_{28} \cos 41.2 - P_f \quad (C)$$

$$F_{35} = F_{23} \sin 41.2 + \cancel{F_{13}}^{0} \quad (T)$$

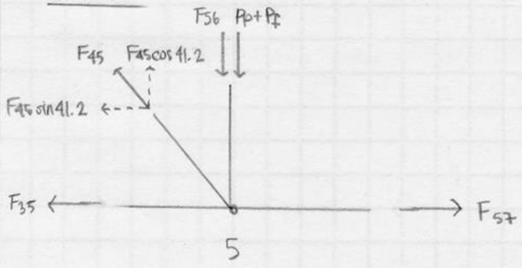
JOINT 4



$$F_{45} = \frac{F_{34}}{\cos 41.2} \quad (T)$$

$$F_{46} = F_{24} + F_{45} \sin 41.2 \quad (C)$$

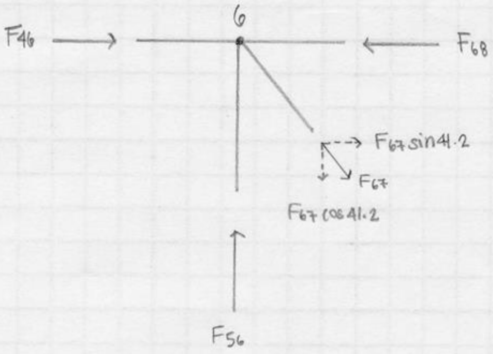
JOINT 5



$$F_{56} = F_{45} \cos 41.2 - P_p - P_f \quad (C)$$

$$F_{57} = F_{35} + F_{45} \sin 41.2 \quad (T)$$

JOINT 6



$$F_{67} = \frac{F_{56}}{\cos 41.2} \quad (C)$$

$$F_{68} = F_{46} + F_{67} \sin 41.2 \quad (T)$$

PRATT TRUSS ANALYSIS SPREADSHEET

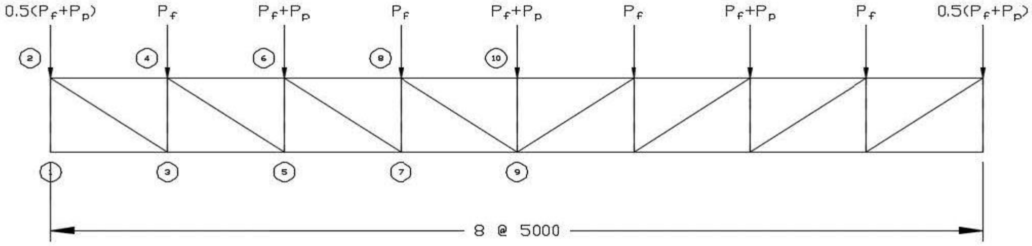
LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

OPTION 2 DRAWING



OPTION 1

TRUSS PARAMETERS

Number of panels	10	
Panel length	4.000	m
Member length (V)	3.500	m
Member length (D)	5.32	m
Pipe support length	8.00	m
Preliminary size	203x203x6.4	
Size width	0.203	m
Dead load, DL	0.377	kN/m
Total member length (4.375m section)	16.8	m

APPLIED LOADS

Pipe load, P_P	60	kN
Factored load, P_F	36	kN
Reaction force, R	329	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	322	(C)
4-6	TC	602	(C)
6-8	TC	774	(C)
8-10	TC	904	(C)
10-12	TC	924	(C)
1-3	BC	0	(T)
3-5	BC	322	(T)
5-7	BC	602	(T)
7-9	BC	774	(T)
9-11	BC	904	(T)
1-2	V	282	(C)
3-4	V	246	(C)
5-6	V	150	(C)
7-8	V	114	(C)
9-10	V	18	(C)
10-11	V	-18	(C)
2-3	D	428	(T)
4-5	D	373	(T)
6-7	D	227	(T)
8-9	D	173	(T)
9-10	D	27	(T)

Chord member size	178x178x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m
Truss weight	39.1	kN

OPTION 2

TRUSS PARAMETERS

Number of panels	8	
Panel length	5.000	m
Member length (V)	3.500	m
Member length (D)	6.10	m
Pipe support length	10.00	m
Preliminary size	203x203x6.4	
Size width	0.203	m
Dead load, DL	0.377	kN/m
Total member length (5.000m section)	19.6	m

APPLIED LOADS

Pipe load, P_P	75	kN
Factored load, P_F	44	kN
Reaction force, R	327	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	382	(C)
4-6	TC	700	(C)
6-8	TC	848	(C)
8-10	TC	933	(C)
1-3	BC	0	(T)
3-5	BC	382	(T)
5-7	BC	700	(T)
7-9	BC	848	(T)
1-2	V	267	(C)
3-4	V	223	(C)
5-6	V	104	(C)
7-8	V	59	(C)
9-10	V	-59	(C)
2-3	D	466	(T)
4-5	D	389	(T)
6-7	D	181	(T)
8-9	D	104	(T)

Chord member size	203x203x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m

Truss weight	37.4	kN

OPTION 3

TRUSS PARAMETERS

Number of panels	6	
Panel length	6.667	m
Member length (V)	3.500	m
Member length (D)	7.53	m
Pipe support length	6.67	m
Preliminary size	203x203x8.0	
Size width	0.203	m
Dead load, DL	0.466	kN/m
Total member length (5.000m section)	24.4	m

APPLIED LOADS

Pipe load, P_P	50	kN
Factored load, P_F	61	kN
Reaction force, R	332	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	527	(C)
4-6	TC	843	(C)
6-8	TC	949	(C)
1-3	BC	0	(T)
3-5	BC	527	(T)
5-7	BC	843	(T)
1-2	V	277	(C)
3-4	V	166	(C)
5-6	V	55	(C)
7-8	V	-55	(C)
2-3	D	595	(T)
4-5	D	357	(T)
6-7	D	119	(T)

Chord member size	203x203x8.0	
Web member size	89x89x6.4	
Web member weight	0.153	kN/m
Truss weight	44.3	kN

OPTION 4

TRUSS PARAMETERS

Number of panels	4	
Panel length	10.000	m
Member length (V)	3.500	m
Member length (D)	10.59	m
Pipe support length	10.00	m
Preliminary size	254x254x8.0	
Size width	0.254	m
Dead load, DL	0.59	kN/m
Total member length (5.000m section)	34.1	m

APPLIED LOADS

Pipe load, P_P	75	kN
Factored load, P_F	95	kN
Reaction force, R	340	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	729	(C)
4-6	TC	972	(C)
1-3	BC	0	(T)
3-5	BC	729	(T)
1-2	V	255	(C)
3-4	V	85	(C)
5-6	V	-85	(C)
2-3	D	772	(T)
4-5	D	257	(T)

Chord member size	254x254x8.0	
Web member size	102x102x8.0	
Web member weight	0.217	kN/m
Truss weight	53.5	kN

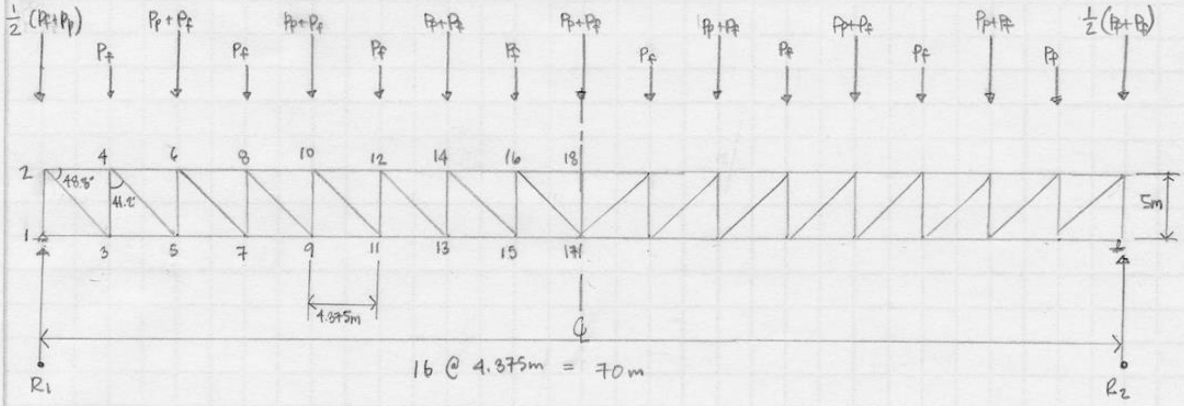
PRATT TRUSS ANALYSIS

IDEAL DEPTH TO SPAN RATIO ~ 10 - 15 (CIDECT 1984)

FOR THE ANAYSIS BELOW, WILL USE AN INITIAL DEPTH OF 5m $\left(\frac{70m}{5m} = 14 \right)$

DESIGN REQUIRES

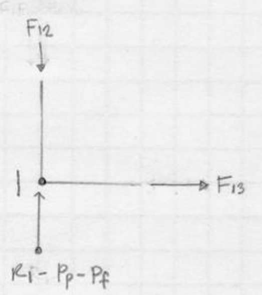
- EQUAL NUMBER OF PANELS
- TRY 16 PANELS → $\frac{70m}{16} = 4.375m$ (PANEL WIDTH)



PIPE LOAD IS SUPPORTED AT LEAST 10m INTERVALS

METHOD OF JOINTS ANALYSIS

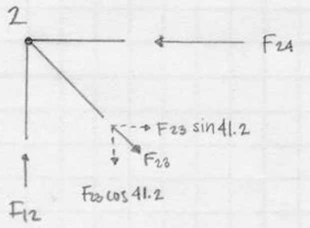
JOINT 1



$F_{12} = R_1 - \frac{1}{2}(P_f + P_p)$ (C)

$F_{13} = 0$ (ZERO FORCE MEMBER)

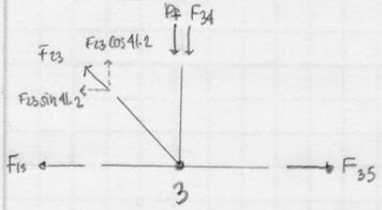
JOINT 2



$F_{23} = \frac{F_{12}}{\cos 41.2}$ (T)

$F_{24} = F_{23}\sin 41.2$ (C)

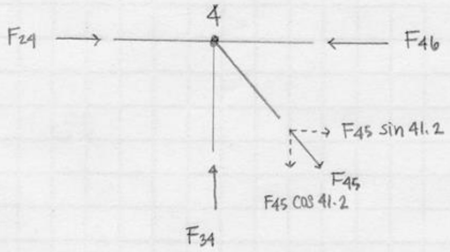
JOINT 3



$$F_{34} = F_{23}\cos 41.2 - P_f \quad (C)$$

$$F_{35} = F_{23}\sin 41.2 + \cancelto{0}{F_{13}} \quad (T)$$

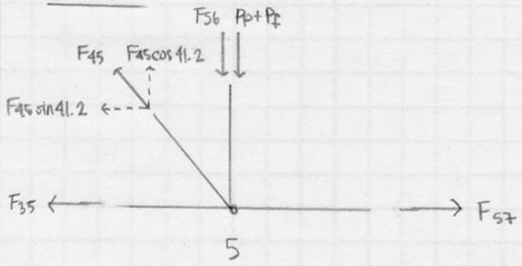
JOINT 4



$$F_{45} = \frac{F_{34}}{\cos 41.2} \quad (T)$$

$$F_{46} = F_{24} + F_{45}\sin 41.2 \quad (C)$$

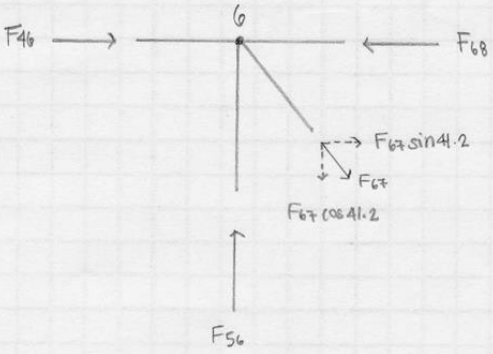
JOINT 5



$$F_{56} = F_{45}\cos 41.2 - P_p - P_f \quad (C)$$

$$F_{57} = F_{35} + F_{45}\sin 41.2 \quad (T)$$

JOINT 6



$$F_{67} = \frac{F_{56}}{\cos 41.2} \quad (C)$$

$$F_{68} = F_{46} + F_{67}\sin 41.2 \quad (T)$$

PRATT TRUSS ANALYSIS SPREADSHEET

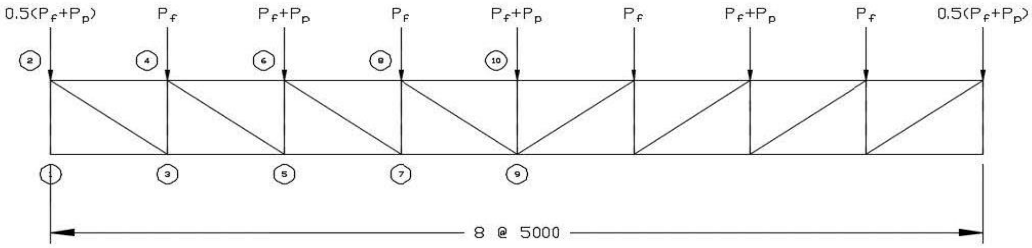
LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

OPTION 2 DRAWING



OPTION 1

TRUSS PARAMETERS

Number of panels	10	
Panel length	4.000	m
Member length (V)	3.500	m
Member length (D)	5.32	m
Pipe support length	8.00	m
Preliminary size	203x203x6.4	
Size width	0.203	m
Dead load, DL	0.377	kN/m
Total member length (4.375m section)	16.8	m

APPLIED LOADS

Pipe load, P_P	60	kN
Factored load, P_F	36	kN
Reaction force, R	329	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	322	(C)
4-6	TC	602	(C)
6-8	TC	774	(C)
8-10	TC	904	(C)
10-12	TC	924	(C)
1-3	BC	0	(T)
3-5	BC	322	(T)
5-7	BC	602	(T)
7-9	BC	774	(T)
9-11	BC	904	(T)
1-2	V	282	(C)
3-4	V	246	(C)
5-6	V	150	(C)
7-8	V	114	(C)
9-10	V	18	(C)
10-11	V	-18	(C)
2-3	D	428	(T)
4-5	D	373	(T)
6-7	D	227	(T)
8-9	D	173	(T)
9-10	D	27	(T)

Chord member size	178x178x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m
Truss weight	39.1	kN

OPTION 2

TRUSS PARAMETERS			**APPLIED LOADS**		
Number of panels	8		Pipe load, P_P	75	kN
Panel length	5.000	m	Factored load, P_F	44	kN
Member length (V)	3.500	m	Reaction force, R	327	kN
Member length (D)	6.10	m			
Pipe support length	10.00	m			
Preliminary size	203x203x6.4				
Size width	0.203	m			
Dead load, DL	0.377	kN/m			
Total member length (5.000m section)	19.6	m			

MEMBER	TYPE	FORCE (kN)	
2-4	TC	382	(C)
4-6	TC	700	(C)
6-8	TC	848	(C)
8-10	TC	933	(C)
1-3	BC	0	(T)
3-5	BC	382	(T)
5-7	BC	700	(T)
7-9	BC	848	(T)
1-2	V	267	(C)
3-4	V	223	(C)
5-6	V	104	(C)
7-8	V	59	(C)
9-10	V	-59	(C)
2-3	D	466	(T)
4-5	D	389	(T)
6-7	D	181	(T)
8-9	D	104	(T)

Chord member size	203x203x6.4	
Web member size	89x89x4.8	
Web member weight	0.119	kN/m
Truss weight	37.4	kN

OPTION 3

TRUSS PARAMETERS		
Number of panels	6	
Panel length	6.667	m
Member length (V)	3.500	m
Member length (D)	7.53	m
Pipe support length	6.67	m
Preliminary size	203x203x8.0	
Size width	0.203	m
Dead load, DL	0.466	kN/m
Total member length (5.000m section)	24.4	m

APPLIED LOADS		
Pipe load, P_P	50	kN
Factored load, P_F	61	kN
Reaction force, R	332	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	527	(C)
4-6	TC	843	(C)
6-8	TC	949	(C)
1-3	BC	0	(T)
3-5	BC	527	(T)
5-7	BC	843	(T)
1-2	V	277	(C)
3-4	V	166	(C)
5-6	V	55	(C)
7-8	V	-55	(C)
2-3	D	595	(T)
4-5	D	357	(T)
6-7	D	119	(T)

Chord member size	203x203x8.0	
Web member size	89x89x6.4	
Web member weight	0.153	kN/m
Truss weight	44.3	kN

OPTION 4

TRUSS PARAMETERS

Number of panels	4	
Panel length	10.000	m
Member length (V)	3.500	m
Member length (D)	10.59	m
Pipe support length	10.00	m
Preliminary size	254x254x8.0	
Size width	0.254	m
Dead load, DL	0.59	kN/m
Total member length (5.000m section)	34.1	m

APPLIED LOADS

Pipe load, P_P	75	kN
Factored load, P_F	95	kN
Reaction force, R	340	kN

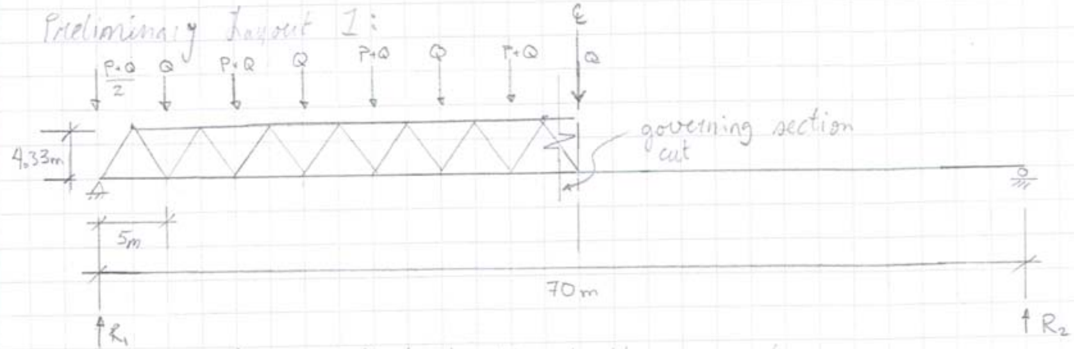
MEMBER	TYPE	FORCE (kN)	
2-4	TC	729	(C)
4-6	TC	972	(C)
1-3	BC	0	(T)
3-5	BC	729	(T)
1-2	V	255	(C)
3-4	V	85	(C)
5-6	V	-85	(C)
2-3	D	772	(T)
4-5	D	257	(T)

Chord member size	254x254x8.0	
Web member size	102x102x8.0	
Web member weight	0.217	kN/m

Truss weight	53.5	kN

Project:	Capstone Design Project	Done by:	Myles C	Date:	26/01/2015
Subject:	Preliminary Warren Truss Design	Checked by:	Amos K	Page:	1 of 6

Preliminary Layout 1:

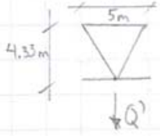


all loads applied to the bottom cord
assume all loads act @ the nodes

P is the loads which are from the pipeline
Q is the remaining other loads

$P = 2.76\,kPa\,(1.3m)(10m)(1.5) + .225\,kN/m\,(10m)(1.3) + 7.64\,kN/m\,(10m)(1.25)$
(snow) (pipe w) (length)(factor) (ice) (length)(factor) (SW)(length)(factor)
$= 53.82 + 2.925 + 95.5$

$P = 152\,kN$



$Q' = (2)20m(0.52\,kN/m) = 20.8\,kN$

Q' is the loads that are per linear meter of members

(factor)(grating SW)(length) (factors)(ice) (length) (factor)(snow)(width)(lengt
$Q = Q' + 1.25(1.1\,kN/m)(5m) + 1.3(0.088\,kN/m)(5m) + 1.5(2.76\,kPa)(3m)(5m$
$= 20.8\,kN + 6.9\,kN + 0.57\,kN + 62.1\,kN$

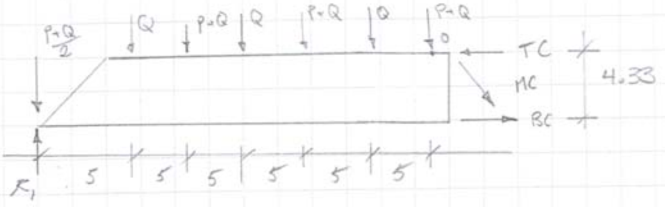
$Q = 90.4\,kN$

FBD

$R_1 = R_2$

$R_1 = \frac{P+Q}{2} + 3Q + 3\underset{242.4kN}{(P+Q)} + \frac{1}{2}Q = \frac{242.4}{2} + 3(90.4) + 3(242.4) + 90.4/2$

$R_1 = 1165\,kN$



$\sum F_v = 0$

$R_1 - \frac{P+Q}{2} - 3Q - 3(P+Q) = MC_v$

$1165\,kN - 242.4\,kN/2 - 3(90.4\,kN) - 3(242.4\,kN) = 45.4\,kN = MC_v$

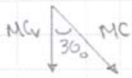
$\sum M_0 = 0 \quad \circlearrowright +$

$BC\,(4.33m) + (P+Q)(10+20+30/2)m + Q(5+15+25)m - R_1(30m) = 0$

$BC = \frac{R_1(30m) - (P+Q)(45m) - Q(45m)}{4.33m}$

$= \frac{1165kN(30m) - (242.4kN)45m - 90.4kN(45m)}{4.33m} \Big\}\ 19974\ kNm$

$BC = 4613\ kN$



TAN = O/A $\quad MC_H = MC_v\,TAN(30)$

$= 45.4\,TAN(30)$

$= 26.2\,kN$

$\sum F_H = 0$

$BC + MC_H = TC$

$4613kN + 26.2kN = 4639\,kN = TC$

$$\boxed{\begin{array}{lll} TC = & 4639\,kN & (C) \\ MC = & 45.2\,kN & (T) \\ BC = & 4613\,kN & (T) \end{array}}$$

resisted by two trusses

$C_f = 2320\ kN \qquad T_f = 2307\ kN$

Hollow Structural Section Connections and Trusses (HSS CT)
from CISC pg 53

$\frac{kL}{r} = 0.9\frac{kL}{r}$ for chord members

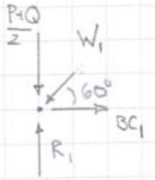
$= 0.9(5000) = 4500\,mm$

From Handbook of Steel Construction (HSC) design tables

HSS 254 × 254 × 9.5 $\quad C_r = 3050 \quad (L = 4800)$

Size the web members - Governing members near the supports

Method of Joints



$\Sigma F_V = 0$

$R_1 - \frac{P+Q}{2} + \sin 60\, W_1 = 0$

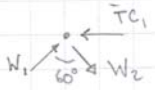
$1165kN - \frac{242kN}{2} - \sin(60)\, W_1 = 0$

$W_1 = 1206kN$ (C)

$\Sigma F_H = 0$

$BC_1 - \cos 60\, W_1 = 0$

$BC_1 = 603kN$ (T)



$\Sigma F_V = 0$

$\cos 30\, W_1 - \cos 30\, W_2 = 0$

$W_1 = W_2$

$W_2 = 1206kN$ (T)

$\Sigma F_H = 0$

$2\sin 30\, W_1 - TC_1 = 0$

$TC_1 = 1206kN$ (C)

Governing Web Loads

$C_f = 603kN$
$T_f = 603kN$

from HSSCT pg 53 $\frac{kL}{r} = 0.7\frac{kL}{r}$
$= 0.7(5000)$
$= 3500mm$

from HSC design table

HSS127×127×8.0 $C_r = 681kN$ ($L = 3600$)

check HSSCT guidelines pg 383

1. $127 > 0.35(254) = 88.9$ ✓
2. $127/254 \sim 0.5 > 0.01(254/9.5+.1) = 0.27$ ✓
3. class 1 or 2 ✓
4. $127/8 = 16 < 35$ ✓
5. $254/9.5 = 28 > 15$ and < 35 ✓

6. minimum web member size
$= 127(0.63)$
$= 80 \text{ mm}$

7. $0.5(1-\beta) < g/b_0$
$0.5(1 - 127/254) = 0.25 < g/254$

minimum gap = 63.5mm

8. absolute min gap = 4(8) = 32mm

9. max eccentricity = $-0.55(4.33) = 2382$ mm
$= 0.25(4.33) = 1083$ mm

recheck guidelines if 89 × 89 × 8.0 used

1. $\frac{127 - 89}{2} = 108 > 89$ ✓

2. $89/254 = 0.35 > 0.27$ ✓

3. class 1 or 2 ✓

4. $89/8 = 11 < 35$ ✓

assume # connections for chords constant between options

connections = 2 per diagonal

panels = 2
diagonals = 4
connections = 8

connections = $2(2)n = 4n$
where n = # panels
$4(2) = 8$ ✓

16 panels = 64 connections
14 panels = 56 connections
12 panels = 48 connections
10 panels = 40 connections

Weights:

16 panels: member length = 4.375m
W_c = 0.849 kN/m chord weight
W_w = 0.213 kN/m web weight

$W = 2W_w(L \cdot n) + W_c(140 - L)$ where L = member length, n = # panels

$= 2(0.213\text{ kN/m})(4.375\text{m} \cdot 16) + 0.849\text{ kN/m}(140\text{m} - 4.375\text{m})$
$= 29.82\text{kN} + 115.15\text{ kN}$

$= 145\text{ kN / truss}$

$= 290\text{ kN}$

14 panels: member length = 5m W_c = 0.700 kN/m W_w = 0.279 kN/m

$W = 2(0.279\text{ kN/m})(5\text{m} \cdot 14) + 0.700\text{kN/m}(140\text{m} - 5\text{m})$
$= 39.06\text{kN} + 94.5\text{kN}$

$= 133.5\text{ kN/ truss}$

$= 267\text{ kN}$

12 panels: $L = 5.833\text{m}$ $W_c = 0.714\text{ kN/m}$ $W_w = 0.278\text{ kN/m}$

$$W = 2(0.278\text{ kN/m})(5.833\text{m} \times 12) + 0.714\text{ kN/m}(140\text{m} - 5.833\text{m})$$
$$= 38.9\text{ kN} + 95.8\text{ kN}$$
$$= 134.7\text{ kN/truss}$$

$$= 269.4\text{ kN}$$

10 panels: $L = 7\text{m}$ $W_c = 0.590\text{ kN/m}$ $W_w = 0.401\text{ kN/m}$

$$W = 2(0.401\text{ kN/m})(7\text{m} \times 10) + 0.590\text{ kN/m}(140\text{m} - 7\text{m})$$
$$= 56.14\text{ kN} + 78.47\text{ kN}$$
$$= 134.61\text{ kN/truss}$$

$$= 269.22\text{ kN}$$

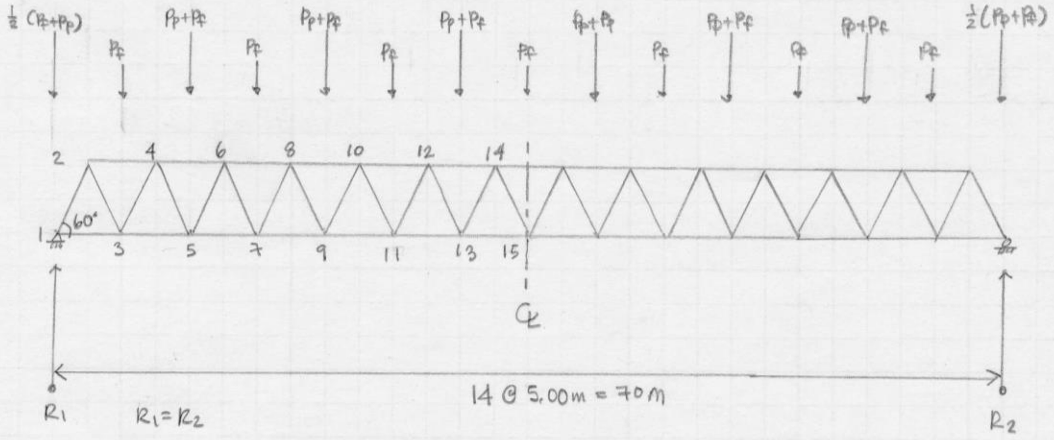
Project:	Capstone Design Project	Done by:	Amos K	Date:	24/01/2015
Subject:	Preliminary Warren Truss Forces	Checked by:	Myles C	Page:	1 of 5

WARREN TRUSS CALCULATION

EQUILATERAL TRIANGLES IN DESIGN, $\theta = 60°$

- TRY 14 PANELS → $\frac{70m}{14} = 5.00$ m (PANEL WIDTH)

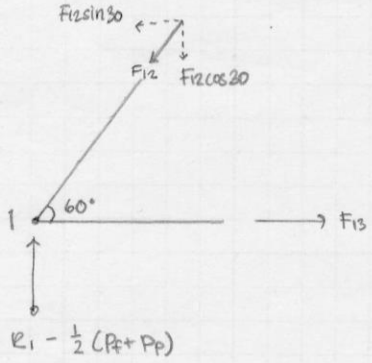
P_p = PIPE LOAD
P_f = FACTORED LOAD



PIPE IS SUPPORTED AT LEAST 10m INTERVALS DUE TO DEFLECTION REQUIREMENTS

METHOD OF JOINTS ANALYSIS

JOINT 1

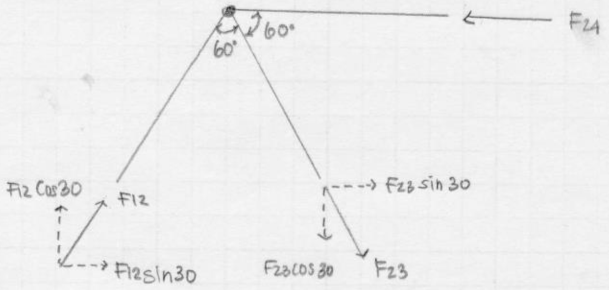


$$R_1 = \frac{1}{2}(P_p+P_f) + F_{12}\cos 30$$

$$F_{12} = \frac{R_1 - \frac{1}{2}(P_p+P_f)}{\cos 30}$$

$$F_{13} = F_{12}\sin 30$$

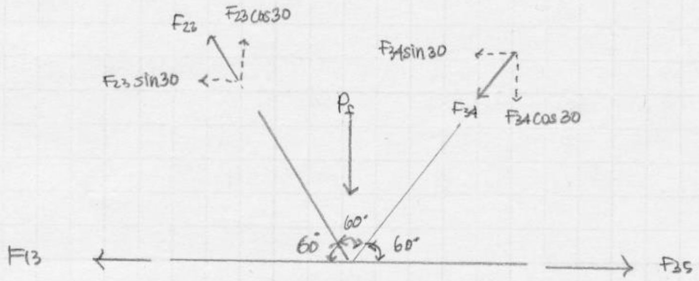
JOINT 2



$$F_{23} = \frac{F_{12}}{\sin 30}$$

$$F_{24} = F_{12}\sin 30 + F_{23}\sin 30$$

JOINT 3



$$F_{34} = \frac{F_{23}\cos 30 - P_f}{\cos 30}$$

$$F_{35} = F_{13} + (F_{23}+F_{34})\sin 30.$$

WARREN TRUSS ANALYSIS SPREADSHEET

LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	1.10	kN/m
Snow load, S	2.76	kPa
Pipe ice accretion load, A_{Pipe}	0.26	kN/m
Member ice accretion, A_{Member}	0.12	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

FINAL OPTION (40m - A TRUSS)

TRUSS PARAMETERS

Number of panels	8	
Member length	5.000	m
Depth	4.33	m
Pipe support length	5.00	m
Preliminary size	203x152x6.4	
Size width	0.152	m
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	37	kN
Factored load, P_F	43	kN
Reaction force, R	322	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-325	(C)
4-6	TC	-557	(C)
6-8	TC	-697	(C)
8-10	TC	-743	(C)
1-3	BC	163	(T)
3-5	BC	441	(T)
5-7	BC	627	(T)
7-9	BC	720	(T)
1-2	D	-325	(C)
2-3	D	325	(T)
3-4	D	-232	(C)
4-5	D	232	(T)
5-6	D	-139	(C)
6-7	D	139	(T)
7-8	D	-46	(C)
8-9	D	46	(T)

Chord member size	203x152x6.4	
Web member size	114x114x4.8	
Web member weight	0.157	kN/m
Truss Weight	37.1	kN

FINAL OPTION (30m - B TRUSS)

TRUSS PARAMETERS

Number of panels	6	
Member length	5.000	m
Depth	4.33	m
Pipe support length	5.00	m
Preliminary size	203x152x6.4	
Size width	0.152	m
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	37	kN
Factored load, P_F	43	kN
Reaction force, R	241	kN

MEMBER	TYPE	FORCE (kN)	
2-4	TC	-232	(C)
4-6	TC	-372	(C)
6-8	TC	-418	(C)
1-3	BC	116	(T)
3-5	BC	302	(T)
5-7	BC	395	(T)
1-2	D	-232	(C)
2-3	D	232	(T)
3-4	D	-139	(C)
4-5	D	139	(T)
5-6	D	-46	(C)
6-7	D	46	(T)

Chord member size	203x152x6.4	
Web member size	114x114x4.8	
Web member weight	0.157	kN/m
Truss Weight	33.9	kN

Option 1 - 16 panels

Top Chord Members

$C_f = 2281$ kN $L = 4.375$m $L_d = 0.9(4.375\text{m}) = 3.938$m

HSS 254×254×9.5 $C_r = 2520$ kN $L = 4000$ mm

Bottom Chord Members

$T_f = 2231$ kN $L = 4.375$m $L_d = 3.938$m

$T_r = \phi A_g F_y = 0.9(9090\text{mm}^2)(300\text{MPa}) = 2454$ kN

HSS 254×254×9.5

Vertical Members

$C_f = 595$ kN $L = 5$m $L_d = 0.7(5) = 3.5$m

HSS 152×152×4.8 $C_r = 627$kN $L = 3600$mm

Diagnal Members

$T_f = 790$kN $L = 6.64$m $L_d = 4.648$m

Check HSS 152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760\text{mm}^2)\,300\text{MPa} = 745$ kN

$L/r \leq 300$ $4648/59.9 = 78$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members = 16(5m + 6.64m) = 186.24m/truss

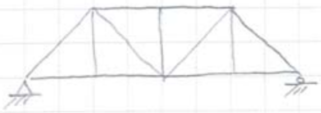
Length of Chord members = 2× 70m - 4.375m = 135.6m/truss

Wt. = 186.24m(0.213 kN/m) + 135.6m (0.7 kN/m) = 134.6 kN/truss

total truss Wt = 269.2 kN

# of connections:

2 per vertical + 2 per diagonal (assume # connections for chords constant between options)



\# panels = 4

\# vertical = 3

\# diagonals = 4

\# connections = 2(n + n - 1)
= 2(2n - 1)

where n = # panels

\# connections = 2(2(4) - 1) = 14

16 panels = 62 connections

Option 2 - 14 panels

Top Chord Members

$C_f = 2383$ kN $L = 5m$ $L_d = 0.9(5000) = 4500$ mm

HSS 254×254×9.5 $C_r = 2340$ kN $L = 4800$ mm

Bottom Chord Members

$T_f = 2316$ kN $L = 5m$ $L_d = 4.5m$

Check HSS 254×254×9.5

$T_r = \phi A_g F_y = 0.9(9090)300 = 2454$ kN

$L/r < 300$ $4500/99.1 = 45$ ✓

HSS 254×254×9.5

Vertical Members

$C_f = 579$ kN $L = 5m$ $L_d = 0.7(5) = 3.5m$

HSS152×152×4.8 $C_r = 627$ kN $L = 3600$mm

Diagnal Members

$T_F = 819$ kN $L = 7.07m$ $L_d = 0.7(7.07) = 4.95m$

Check HSS152×152×4.8

$T_r = \phi A_g F_y = 0.9(2760)(300) = 745$ kN

$L/r < 300$ $4950/59.9 = 82.6$ ✓

HSS 152×152×4.8

Wt. of Steel

Length of Web members = 16 (5+7.07) = 193.12 / truss

Wt. = $193.12^{m}(0.213\text{ kN/m}) + 135\text{m}(0.7\text{ kN/m}) = 135.6$ kN/truss

total truss Wt. = 271.2kN

connections = 54

Option 3 - 12 panels

Top Chord

$C_f = 2523$ kN $L = 5.833$m $L_d = 0.9(5833) = 5250$

HSS 305×305×9.5 $C_r = 2970$ $L = 5200$

Bottom Chord

$T_f = 2353$kN $L = 5.833$m $L_d = 5.25$m

Check above

$T_r = \phi A_g F_y = 0.9(11000)(300) = 2970$ kN

$L/r = \frac{5250}{120} = 44 < 300$ ✓

HSS305×305×9.5

Vertical

$C_f = 561$ kN $L = 5$m $L_d = 0.7(5) = 3.5$m

HSS 152×152×4.8 $C_r = 627$kN $L = 3600$mm

Diagonals

$T_f = 863$ kN $L = 7.68$m $L_d = 0.7(7.68) = 5.376$m

check above

$T_r = \phi A_g F_y = (0.9)(\quad)(360) = 869$ kN

$L/r = 3500/59.9 = 58 < 300$ ✓

Wt. of Steel

Length of Web members = 16 (5m + 5.833m) = 173.3 m / truss

Wt. = 173.3m (0.213 kN/m) + 134.2m (0.849 kN/m) = 150.85 kN/truss

total truss Wt. = 301.7 kN

connections = 46

Option 4 - 10 panels

Top Chord

$C_f = 2626 kN$ $L = 7m$ $L_d = 0.9(7) = 6.3m$

HSS 305 × 305 × 9.5 $C_r = 2700 kN$ $L = 6400mm$

Bottom Chord

$T_f = 2395 kN$ $L = 7m$ $L_d = 6.3m$

$T_r = \phi A_g F_y = 0.9(11000) 300 = 3470$

$L/r = 6300/120 = 53 < 300$ ✓

Vertical

$C_f = 540 kN$ $L = 5m$ $L_d = 0.7(5) = 3.5m$

HSS 152 × 152 × 4.8 $C_r = 627 kN$ $L = 3600 mm$

Diagonal

$T_f = 929 kN$ $L = 8.6m$ $L_d = 0.7(8.6) = 6.02m$

$T_r = \phi A_g F_y = 0.9(2760) 300 = 869 kN$

$L/r = \frac{6020}{59.9} = 101 < 300$ ✓

Wt. of Steel

Length of Web members = $16(5 + 8.6)m = 217.6 m/truss$

$Wt. = 217.6m(0.213 kN/m) + \overset{m}{133}(0.849 kN/m) = 159.3 kN/truss$

total truss Wt. = 318.6kN

connections = 38

Appendix D: Superstructure Deflection Calculations

Project:	Capstone Design Project	Done by:	Amos K	Date:	10/02/2015
Subject:	Preliminary Warren Truss Deflection	Checked by:	Myles C	Page:	1 of 6

WARREN TRUSS DEFLECTION SPREADSHEET

LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	0.38	kN/m
Snow load, S	2.76	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

OPTION 1

TRUSS PARAMETERS

Number of panels	12	
Member length	3.333	m
Depth	2.887	m
Pipe support length	10.00	m
Preliminary size	203x203x6.4	
Moment of inertia	3.13E+07	mm^4
Area	4900	mm^2
Dead load, DL	0.5	kN/m
Total member length (4.375m section)	13.3	m
Truss moment of inertia	2.05E+10	mm^4

APPLIED LOADS

Pipe load, P_P	53	kN
Service load, P_S	20	kN
Reaction, R	229	

POINT	DEFLECTION (mm)
1	3
2	6
3	33
4	11
5	13
6	24
SUM	91

OPTION 2

TRUSS PARAMETERS

Number of panels	10	
Member length	4.000	m
Depth	3.46	m
Pipe support length	8.00	m
Preliminary size	203x203x6.4	
Moment of Inertia	3.13E+07	mm^4
Area	4900	mm^2
Dead load, DL	0.377	kN/m
Total member length (5.000m section)	16.0	m
Truss moment of inertia	2.95E+10	mm^4

APPLIED LOADS

Pipe load, P_P	43	kN
Service load, P_S	22	kN
Reaction, R	219	kN

POINT	DEFLECTION (mm)
1	3
2	17
3	8
4	28
5	5
SUM	61

OPTION 3

TRUSS PARAMETERS

Number of panels	8	
Member length	5.000	m
Depth	4.33	m
Pipe support length	10.00	m
Preliminary size	178x178x6.4	
Moment of Inertia	2.50E+07	mm^4
Area	4250	mm^2
Dead load, DL	0.327	kN/m
Total member length (5.833m section)	20.0	m

APPLIED LOADS

Pipe load, P_P	53	kN
Service load, P_S	27	kN
Reaction, R	215	kN

Truss moment of inertia	3.99E+10	mm^4

POINT	DEFLECTION (mm)
1	3
2	18
3	8
4	13
SUM	43

Project:	Capstone Design Project	Done by:	Amos K	Date:	10/02/2015
Subject:	Preliminary Warren Truss Deflection	Checked by:	Myles C	Page:	5 of 6

OPTION 4

TRUSS PARAMETERS

Number of panels	6	
Member length	6.667	m
Depth	5.77	m
Pipe support length	6.67	m
Preliminary size	203x203x6.4	
Moment of Inertia	3.13E+07	mm^4
Area	4900	mm^2
Dead load, DL	0.377	kN/m
Total member length (7.000m section)	26.667	m

APPLIED LOADS

Pipe load, P_P	36	kN
Service load, P_S	37	kN
Reaction, R	166	kN

Truss moment of inertia	8.17E+10	mm^4

POINT	DEFLECTION (mm)
1	6
2	10
3	6
SUM	22

OPTION 5

TRUSS PARAMETERS

Number of panels	4	
Member length	10.000	m
Depth	8.66	m
Pipe support length	10.00	m
Preliminary size	203x203x6.4	
Moment of Inertia	3.13E+07	mm^4
Area	4900	mm^2
Dead load, DL	0.377	kN/m
Total member length (7.000m section)	40.000	m
Truss moment of inertia	1.84E+11	mm^4

APPLIED LOADS

Pipe load, P_P	53	kN
Service load, P_S	56	kN
Reaction, R	166	kN

POINT	DEFLECTIONS (mm)
1	5
2	4
SUM	9

WARREN TRUSS DEFLECTION SPREADSHEET

LOADS

Pipe self-weight, SW_{Pipe}	7.64	kN/m
Grating self-weight, $SW_{Grating}$	0.38	kN/m
Snow load, S	2.76	kPa

DIMENSIONS

Pipe diameter	1219	mm
Walkway width (one side)	1.50	m
Span	40	m

OPTION 1

TRUSS PARAMETERS

Number of panels	10	
Member length	4.000	m
Depth	3.500	m
Diagonal length	5.315	m
Pipe support length	8.00	m
Preliminary size	203x203x6.4	
Moment of inertia	3.13E+07	mm^4
Area	4900	mm^2
Dead load, DL	0.377	kN/m
Total member length (4.375m section)	16.8	m
Truss moment of inertia	3.01E+10	mm^4

APPLIED LOADS

Pipe load, P_P	43	kN
Service load, P_S	23	kN
Reaction, R	220	kN

POINT	DEFLECTIONS (mm)
1	3
2	16
3	8
4	27
5	5
SUM	60

OPTION 2

TRUSS PARAMETERS			**APPLIED LOADS**		
Number of panels	8		Pipe load, P_P	53	kN
Member length	5.000	m	Service load, P_S	28	kN
Depth	3.50	m	Reaction, R	218	kN
Diagonal length	6.103	m			
Pipe support length	10.00	m			
Preliminary size	203x203x6.4				
Moment of Inertia	3.13E+07	mm^4			
Area	4900	mm^2			
Dead load, DL	0.377	kN/m			
Total member length (5.000m section)	19.6	m			

Truss moment of inertia	3.01E+10	mm^4

POINT	DEFLECTION (mm)
1	5
2	25
3	11
4	18
SUM	59

OPTION 3

TRUSS PARAMETERS

Number of panels	6	
Member length	6.667	m
Depth	3.50	m
Diagonal length	7.530	m
Pipe support length	6.67	m
Preliminary size	203x203x8.0	
Moment of Inertia	3.79E+07	mm^4
Area	6050	mm^2
Dead load, DL	0.466	kN/m
Total member length (5.833m section)	24.4	m

APPLIED LOADS

Pipe load, P_P	36	kN
Service load, P_S	39	kN
Reaction, R	223	kN

Truss moment of inertia	3.71E+10	mm^4

POINT	DEFLECTION (mm)
1	13
2	23
3	13
SUM	49

OPTION 4

TRUSS PARAMETERS

Number of panels	4	
Member length	10.000	m
Depth	3.50	m
Diagonal length	10.595	m
Pipe support length	10.00	m
Preliminary size	254x254x8.0	
Moment of Inertia	3.13E+07	mm^4
Area	4900	mm^2
Dead load, DL	0.377	kN/m
Total member length (7.000m section)	34.1	m

Truss moment of inertia	3.01E+10	mm^4

APPLIED LOADS

Pipe load, P_P	53	kN
Service load, P_S	54	kN
Reaction, R	214	kN

POINT	DEFLECTION (mm)
1	33
2	24
SUM	56

APPENDIX E: LATERAL MEMBER DESIGN CALCULATIONS

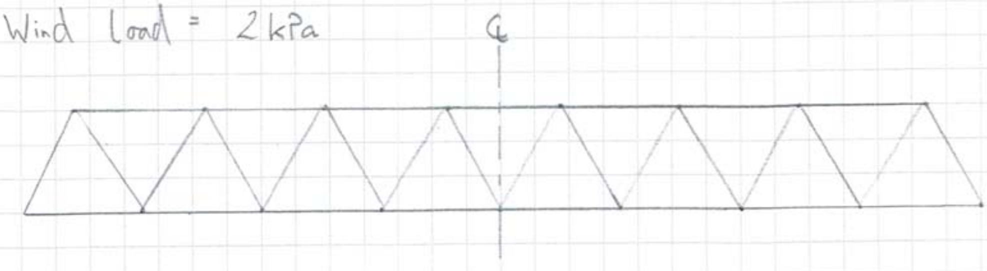
Project:	Capstone Design Project	Done by:	Myles C	Date:	23/02/2015
Subject:	Cross/Sway Bracing	Checked by:	Amos K	Page:	1 of 4

Cross Bracing

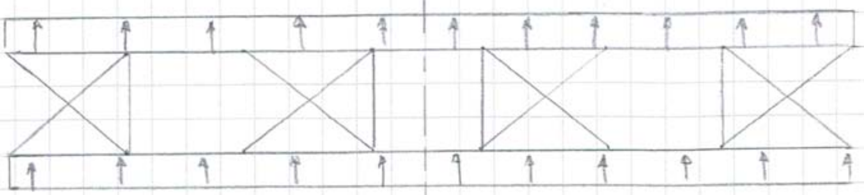
designed to resist wind load and 1% of compression chord force
(CSA S6-06 10.10.9.1)

Chord load = 744 kN   1% = 7.4 kN

Wind Load = 2 kPa



PROFILE ℄



PLAN @ TOP CHORD

PLAN @ BOTTOM CHORD

Wind load of 2 kPa on both sides of the truss and pipeline

- assume pipe wind load goes to bottom bracing
- assume half of truss wind load goes to bottom bracing.

Pipe wind load = 2 kPa (1.219m) = 2.4 kN/m

Truss wind load:

member surface area per panel = 2(5m)(0.114m) + 2(5m)(0.178m)

= 1.14 m² + 1.78 m² = 2.92 m²

Truss wind load = $2\text{ kPa}(2.92\text{ m}^2)(8) = 46.7\text{ kN}$

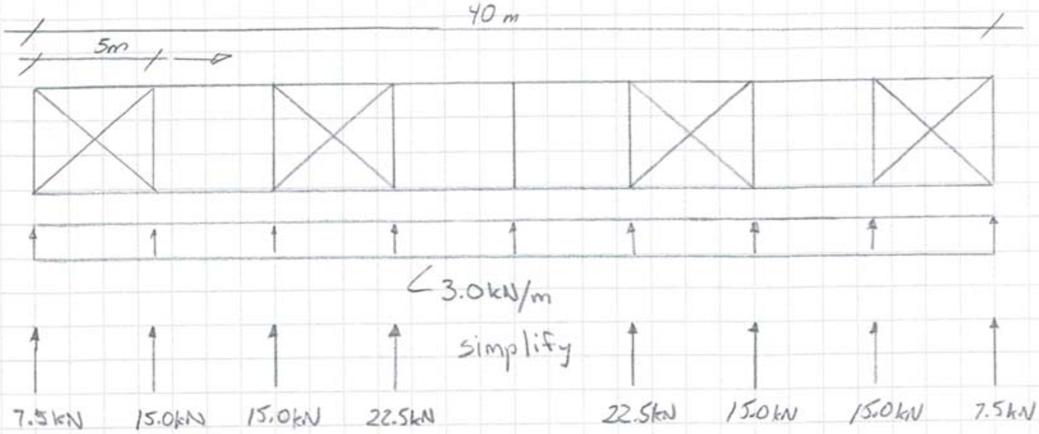
$46.7\text{ kN}/40\text{ m} = 1.2\text{ kN/m}$

Top chord wind load = $\frac{1.2\text{ kN/m}}{2} = 0.6\text{ kN/m}$

Bottom chord wind load = $\frac{1.2\text{ kN/m}}{2} + 2.4\text{ kN/m} = 3.0\text{ kN/m}$

Bottom bracing governs.

Check web member size (HSS114×114×4.8)



design x bracing as tension only

governing load is 22.5 kN

HSS114×114×4.8 $T_r = 438\text{ kN}$ (CISC HSC pg 4-54; based on gross area)

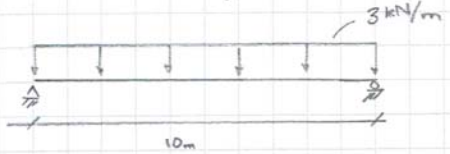
if member takes compression load

$C_r = 244\text{ kN} >> 22.5\text{ kN}$

Bracing should be sufficient however check with SAP.

X bracing size = 114×114×4.8
lateral bracing size = 114×114×4.8
pipe support size = 178×127×13

Check chords against bending in central span.
- treat as simply supported which is conservative



$M_{MAX} = \frac{w l^2}{8} = \frac{3 kN/m (10m)^2}{8}$

$= 37.5 \text{ kNm}$

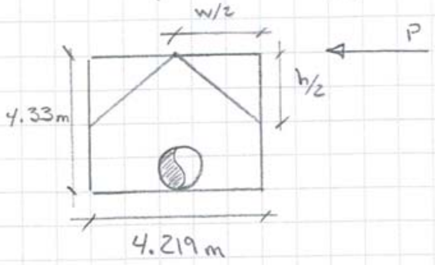
HSS 178 × 178 × 6.4 $Z = 271 \times 10^3 \text{ mm}^3$

$M_r = \phi Z F_y = 0.9 (271 \times 10^3 \text{ mm}^3)(350 \text{ MPa})$

$= 85 \text{ kNm}$

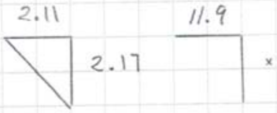
$M_r = 85 > M_f = 37.5$

Truss requires support to prevent the truss from tipping over



simplify wind load to a point load @ the top

max P = 0.6kN/m(7.5m) + 7.4kN
= 11.9kN



$x = 11.9\left(\frac{2.17}{2.11}\right) = 12.2$

required force in members. = $\sqrt{11.9^2 + 12.2^2}$ = 17.0 kN

design for one member taking the full load

compare with web member size HSS 114×114×4.8

CISC HSC pg 4-54 C_r = 496 kN (L ~ 2400mm)

check design with SAP.

Case 1: Pressure Testing

Case 2: Snow

Constant Load = 7.64 kN/m

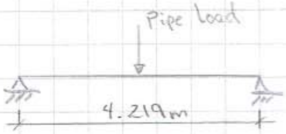
Water = (10.5 kN/m) 1.1 = 11.5 kN/m ← governs

Snow = 2.76 kPa (1.219 m) = 3.36 kN/m

Governing Load: 1.1(7.64 + 10.5) kN/m = 19.95 kN/m

Treat pipe as continuous beam under UDL
(CISC HSC pg 5-160)

Force = 1.143 (19.95 kN/m) (10m) = 228 kN



$M_{MAX} = Pl/4 = 228\,kN(4.219m)/4 = 240\,kNm$

CSA S6-06 13.6

Compare a HSS and a W shape member
↳ look for member < 178mm depth

W shape W150x37 $I_x = 22.2 \times 10^6 mm^4$, $I_y = 7.07 \times 10^6 mm^4$, $Z_x = 310 \times 10^3 mm^3$
$Z_y = 140 \times 10^3 mm^3$, $J = 193 \times 10^3 mm^3$, $C_w = 40 \times 10^9 mm^6$

$M_p = ZF_y = 310 \times 10^3 mm^3 (350 MPa) = 109 kNm$

$$M_u = \frac{\omega_2 \pi}{L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

G = 70 000 MPa
E = 200 000 MPa

$= 119 kNm$

$M_r = \phi M_u = 107 kNm$

HSS shape check HSS 178×127×13

$M_r = \phi Z F_y = 0.9(378 \times 10^3 mm^3)(350 N/mm^2) = 120 kNm$

check HSS 178×178×13

$M_r = \phi Z F_y = 0.9(484 \times 10^3 mm^3)(350 N/mm^2) = 152 kNm$

Need to reduce support length on pipe to lessen loads

New pipe support length = 5m.

$Force = 1.143 (19.95 kN/m)(5m) = 114 kN$

$M_{MAX} = Pl/4 = 114 kN (4.219m)/4 = 120 kNm$

use HSS 178×127×13

$M_r = 120 kNm \geq M_f = 120 kNm$

check HSS 178×178×9.5 (lighter than 178×127×13)

$M_r = \phi Z F_y = 0.9 (385 \times 10^3 mm^3)(350MPa)$
$= 121 kNm$

use 178×178×9.5 since lighter and higher M_r.

APPENDIX F: DETAILED DESIGN CALCULATIONS

PRELIMINARY MEMBER SELECTIONS

The values for compressive resistances, Cr, are interpolated from the CISC Handbook of Steel Construction Tenth Edition (2011). Using G40.21 350W Class H. See pages 4-74 to 4-81.

For HSS chord members

Effective length factor

$K_c := 0.9$

Distance between chord panel points

$L_c := 5.00$m

For HSS web members

Effective length factor

$K_w := 0.7$

Distance between web panel points

$L_w := 4.33$ m

Central top chord

Central top chord is the governing member and is in compression.

Load in member 8a-10a is -744 kN.

$F_{8a10a} := -744$ kN

$K_c \cdot L_c = 4.5$ m

Try HSS 203x152x6.4

$A_{c1} := 4250$ mm^2

$Cr_{c1} := -1030$ kN (From CISC Handbook)

$Cr_{c1} > F_{8a10a}$

Although a smaller size can be chosen, it will be later shown that HSS 178x178x6.4 was chosen for deflection limits.

Checking section classification limits

$\text{b\_el\_c1} := \frac{152}{2} = 76$ mm

$\text{t\_c1} := 6.4$ mm

$\frac{\text{b\_el\_c1}}{\text{t\_c1}} = 11.9$

$\text{Fy} := 350$ MPa

$\frac{420}{\sqrt{\text{Fy}}} = 22.4$

$\frac{420}{\sqrt{\text{Fy}}} > \frac{\text{b\_el\_c1}}{\text{t\_c1}}$ therefore, 178x178x6.4 is a Class 1 section.

Utilization is $\frac{-744}{-1030} = 0.72$

Central bottom chord

Central bottom chord is the governing member and is in tension.

Load in member 7a-9a is 710 kN

$\text{F\_7a9a} := 710$ kN

$\phi := 0.9$

$A_{min} = T_f/\phi F_y$

$\text{Amin\_c2} := \frac{\text{F\_7a9a} \cdot 1000}{\phi \cdot \text{Fy}} = 2254\,\text{mm}^2$

Try HSS 203x152x6.4 (Same as top chord)

Using uniform sizes for chord members for practical reasons (e.g. constructability, purchasing, etc.)

$\text{A\_c2} := 4250\ \text{mm}^2$

$\text{A\_c2} > \text{Amin\_c2}$

$\text{Tf\_c2} := \frac{\phi \cdot \text{A\_c2} \cdot \text{Fy}}{1000} = 1339$ kN

Utilization is $\frac{710}{1213} = 0.59$

Compression diagonal at the end of truss

Compression diagonal at the end of truss is the governing member and is in compression.

Load in member 1a-2a is 325 kN

$F\_1a2a := -325$ kN

$K\_w \cdot L\_w = 3.0$

Try HSS 114x114x4.8

$A\_w1 := 2040$ mm^2

$Cr\_w1 := -490$ kN (From CISC Handbook)

$Cr\_w1 > F\_1a2a$

Checking section classification limits

$b\_el\_w1 := \frac{114}{2} = 57$ mm

$t\_w1 := 4.8$ mm

$\frac{b\_el\_w1}{t\_w1} = 11.9$

$\frac{420}{\sqrt{Fy}} > \frac{b\_el\_w1}{t\_w1}$ therefore, 114x114x4.8 is a Class 1 section.

Utilization is $\frac{-325}{-490} = 0.66$

Tension diagonal at the end of truss

Tension diagonal at the end of the truss is the governing member and is in tension.

Load in member 2a-3a is 325 kN

$F\_2a3a := 325$ kN

$Amin\_w2 := \frac{F\_2a3a \cdot 1000}{\phi \cdot Fy} = 1032$ mm^2

$A\_w1 := 2040$ mm^2

$A\_w1 > Amin\_w2$

$Tf\_w2 := \frac{\phi \cdot A\_w1 \cdot Fy}{1000} = 643$ kN

Utilization is $\frac{325}{583} = 0.56$

Design summary with trial members

Chord members: HSS 203x152x6.4

Web members: HSS 114x114x4.8

RESISTANCE OF GAP K CONNECTIONS

Looking at Panel Point 3 and 4 (Looking at critical panel points)

Checking the validity limits for a gap connection with sqaure chords and webs from CISC Hollow Structural Section: Connections and Trusses (CISC HSS).

Confirm that a gap connection is feasible. Looking at Maximum β (average width of web members relative to the chord width) based on allowable eccentricity limits, chord aspect ratio and inclination of web members. See CISC HSS Figure 3.11.

Chord member height, h_0

Chord member width, b_0

$h\_0 := 203$ mm

$b\_0 := 152$ mm

$$\frac{h\_0}{b\_0} = 1.336$$

$\theta\_1 := 60$ degrees

$\theta\_2 := 60$ degrees

$\beta := 1.0$ (From Figure 3.11)

Maximum allowable β is 1.0.

Web member height, h_1

Web member width, b_1

$h\_1 := 114$ mm

$b\_1 := 114$ mm

$$\beta\_actual := \frac{(2 \cdot b\_1)}{(2 \cdot b\_0)} = 0.75$$

Therefore, OK. use β = 0.75 $\beta := 0.75$

For gap connections to be valid, the average width of the two web members (both web members for all panel points have the same width) must be at at least 0.35 times the chord width.

$\beta \geq 0.35$ therefore, OK.

Width of the web members must be at least a hundredth of the chord width-to-thickness ratio, plus 0.1.

$\frac{b\_1}{b\_0} = 0.75$

$t\_0 := 6.4$ mm

$t\_1 := 4.8$ mm

$0.01 \cdot \frac{b\_0}{t\_0} + 0.1 = 0.338$

$0.75 \geq 0.378$ therefore, OK.

Both chord and web members confirmed as Class 1 sections.

Maximum width to thickness ratio of tension web members is 35.

$\frac{b\_1}{t\_1} = 24 \quad 24 \leq 35$ therefore, OK.

Width to thickness ratio of the chord must be between 15 and 35.

$\frac{b\_0}{t\_0} = 24$ therefore, OK.

Width of the smaller web member must be at least 0.63 times the width of the web member. As both web members are the same size, OK.

With the above checks, the use of gap connections are valid.

Connection resistance of tension diagonal 2-3

Looking at Panel Point 3 (Looking at critical panel points).

Refer to Fig. 3.23 Web member effiency for square HSS K and N gap connections from CISC HSS.

$$\frac{b\_0}{t\_0} = 24$$

$C\_KGap1 := 0.34$

Web members connecting to a tension chord, $f\_n1 := 1$

The connection resistance is given by

$$N\_1 := C\_KGap1 \cdot \frac{t\_0}{t\_1} \cdot \frac{1}{\sin\left(\theta\_1 \cdot \frac{\pi}{180}\right)} \cdot f\_n1 \cdot A\_w1 \cdot \frac{Fy}{1000} = 374 \text{ kN}$$

Tension force in diagonal 2-3 is 330 kN.

$374 \geq 325$ therefore, OK.

Connection resistace of compression diagonal 3-4

The connection resistance is the same as tension diagonal 2-3

$N\_2 := N\_1 = 374$ kN

Compression force in diagonal 3-4 is 232 kN.

$374 \geq 232$ therefore, OK.

Panel Point 3 resistance of gap K connections are OK.

Connection resistance of tension diagonal 4-5

$P\_f := 43$ kN

$P\_p := 37$ kN

$$M\_f1 := \frac{3}{32}\cdot P\_f\cdot L\_c + \frac{3}{32}\cdot (P\_f + P\_p)\cdot L\_c = 57.7 \text{ kNm}$$

$F\_4a6a := -558$ kN

$$S\_c1 := 225\cdot 10^{3} = 225 \times 10^{3}$$

$$n := \frac{F\_4a6a}{A\_c1\cdot \frac{Fy}{1000}} = -0.375$$

$$f\_n2 := 1.3 + \frac{0.4}{\beta}\cdot n = 1.1 \qquad f\_n2 := 1$$

$$N\_3 := C\_KGap1\cdot \frac{t\_0}{t\_1}\cdot \frac{1}{\sin\left(\theta\_1\cdot \frac{\pi}{180}\right)}\cdot f\_n2\cdot A\_w1\cdot \frac{Fy}{1000} = 374 \text{ kN}$$

Tension force in diagonal 4-5 is 232 kN.

$374 \geq 232$ therefore, OK.

Panel Point 4 resistance of gap connections are OK.

Gap dimensions

$0.5\cdot(1 - \beta) = 0.125$

g is the gap dimension.

$\frac{g}{b\_0} \geq 0.125$ and $g \geq 2\cdot t\_1$

$g := 20$

A gap of 20mm satisfies both of these requirements.

BOLTED FLANGE-PLATE SPLICE CONNECTION

Designing splices in bottom chord member 7a-9a and top chord member 8a-10a.

$F\_7a9a = 710$

$F\_8a10a = -744$

Splice connection for bottom chord 7-9

Try 6 bolts.

Applied load per bolt, $P\_fb := \frac{710}{6} = 118$ kN

Assume M22 bolts, 24mm hole diameter. $T\_rb := 189$ kN From CISC (Handbook)

$d := 22$ mm

$d\_prime := 24$ mm

Bolt pitch should generally be about 4 to 5 bolt diameters.

Initial pitch, $p := 100$ mm

Initial distance from blot line to edge of face $b := 45$ mm

$$\delta := 1 - \frac{d\_prime}{p} = 0.76$$

$$b\_prime := b - \frac{d}{2} + t\_0 = 40.4 \quad \text{mm}$$

Parameter, $K := \frac{4 \cdot b\_prime \cdot 1000}{0.9 \cdot p \cdot Fy} = 5.13$

Plate thickness

$$t\_min := \left[\frac{K \cdot P\_fb}{(1 + \delta \cdot 1)}\right]^{0.5} = 18.6\text{mm}$$

$$t\_max := \left[\frac{K \cdot P\_fb}{(1 + \delta \cdot 0)}\right]^{0.5} = 24.6\text{mm}$$

Try 3/4 inch plate (19 mm) $tp1 := 19$ mm

Maximum effective a is $a := 1.25 \cdot b = 56.25$ mm

Ratio of the sagging plate moment to the hogging plate moment.

$$\alpha := \left(\frac{K \cdot T\_rb}{tp1^2} - 1\right) \cdot \left[\frac{a + \left(\frac{d}{2}\right)}{\delta \cdot (a + b + t\_0)}\right] = 1.39$$

α is greater than 1. This would cause major flexure in the plate and prying forces. Try a thicker plate closer to the maximum thickness.

Try 7/8 inch plate (22.2 mm) $tp2 := 22.2 \quad mm$

$$\alpha := \left(\frac{K \cdot T\_rb}{tp2^2} - 1\right) \cdot \left[\frac{a + \left(\frac{d}{2}\right)}{\delta \cdot (a + b + t\_0)}\right] = 0.795$$

Since α is between 0 and 1, choose α = 0.795 and 3/4" plate thickness.

$$N\_c1 := \left(\frac{tp2^2}{K}\right) \cdot (1 + \delta \cdot \alpha) \cdot 6 = 925 \text{kN}$$

$925 \geq 710$ therefore, OK.

Actual bolt load (including prying)

$$a\_prime := a + \frac{d}{2} = 67.25 \text{ mm}$$

$$\alpha\_p := \left(K \cdot \frac{P\_fb}{tp2^2} - 1\right) \cdot \frac{1}{\delta} = 0.305$$

$$T\_fb := P\_fb \cdot \left[1 + \frac{b\_prime}{a\_prime} \cdot \left(\frac{\delta \cdot \alpha\_p}{1 + \delta \cdot \alpha\_p}\right)\right] = 132 \quad kN$$

Prying tension is $T\_fb - P\_fb = 13.4$ kN per bolt

DESIGN OF WELDED JOINTS

Effective weld lengths

For K and N gap connections and θ greater or equal to 60 degress, effective length = a+b+c = 2h/sinθ+b.

$$l\_weld := 2\cdot\frac{h\_1}{\sin\left(\theta\_1\cdot\frac{\pi}{180}\right)} + b\_1 = 377$$ mm

Weld length can also be calculated by

$$Ka := \left[\frac{\left(\frac{h\_1}{\sin\left(\theta\_1\cdot\frac{\pi}{180}\right)} + b\_1\right)}{(h\_1 + b\_1)}\right] = 1.077$$

$$l\_weld2 := Ka\cdot[4\cdot\pi\cdot t\_1 + 2\cdot(b\_1 - 4\cdot t\_1) + 2\cdot(h\_1 - 4\cdot t\_1)] = 474$$ mm

Values from this method are conservative for small wall thicknesses as this equation uses perimeters for the thickest wall HSS for each size. Therefore, use length of weld of 377 mm.

Default method of sizing welds

Looking at Table 3-46, the 90 degree fillet size to develop wall strength for a wall thickness of 4.8 mm is 8 mm.

$$Tr\_w := 0.9\cdot A\_w1\cdot\frac{Fy}{1000} = 643$$ kN

$$\phi\_w := 0.67$$

$$Fu := 450$$ MPa

$$Vr\_w := 0.67\cdot\phi\_w\cdot 8\cdot\frac{Fu}{1000}\cdot l\_weld = 610$$ kN

Appendix G: Bearing Calculations

CONTACT PRESSURE - S6 11.6.2.4

CYLINDRICAL SURFACES

LOAD DUE TO SERVICEABILITY

$$P_s \leq 8 \left[\frac{L D_1}{1 - \frac{D_1}{D_2}} \right] \frac{F_y^2}{E_s}$$

$F_y = 460 MPa$ (ASSUMED)

$E_s = 200$ GPa

FOR MIDDLE PIER:

$P_s = 201 kN + 201 kN \left(\frac{3}{4}\right)$ (NOTES FEB. 17, 2015)

$\underline{P_s = 352 kN}$

$$352000 \leq 8 \left[\frac{(200) D_1}{1 - \frac{D_1}{D_2}} \right] \left(\frac{400^2}{200\ 000} \right)$$

ASSUME L = 200 mm

$$275 \leq \frac{D_1}{1 - \frac{D_1}{D_2}}$$

TRY $\underline{D_1 = 260}$, $\underline{D_2 = 300}$ ✓

SPHERICAL SURFACES

$$P_s \leq 40 \left[\frac{D_1}{1 - \frac{D_1}{D_2}} \right]^2 \frac{F_y^3}{E_s^2}$$

MIDDLE PIER: $\underline{P_s = 352 kN}$

$$\sqrt{\frac{(352000)(200\,000)^2}{40\ (400)^3}} \leq \frac{D_1}{1 - \frac{D_1}{D_2}}$$

$$2345 \leq \frac{D_1}{1 - \frac{D_1}{D_2}}$$

TRY $D_1 = 200$
$D_2 = 300$ ✗

$D_1 = 500$
$D_2 = 600$ ✓

→ TRY SPHERICAL S6 11.6.4

→ LATERAL:

$$\beta = \tan^{-1} \left[\frac{H_u}{P_{ub}} \right]$$

HORIZ. LOAD ON BEARING (ULS)

MIN. DEAD LOAD (ULS)

TABLE 3.2

$H_u = 277 kN$

$P_{ub} = (0.95)(201 kN)$

$\underline{P_{ub} = 191 kN}$

$$\beta = \tan^{-1} \left[\frac{277}{191} \right]$$

$\beta = 55.41$

$$R \leq \frac{D}{2 \sin(\beta + \theta_u)}$$

$$\frac{500}{2 \sin(55.41}$$

ELASTOMERIC BEARING

h_e = TOTAL EFFECTIVE ELASTOMER THICKNESS

L = SMALLER DIMENSION OF BEARING

R = RADIUS OF BEARING (ASSUME FLAT)

PLAIN BEARINGS : S6 11.6.6.3

$L \geq 5h_e$

$R \geq 3h_e$

$10mm < h_e < 30mm$

LAMINATED BEARINGS: S6 11.6.6.3

$L \geq 3h_e$

$R \geq 2h_e$

→ USE PLAIN BEARINGS.

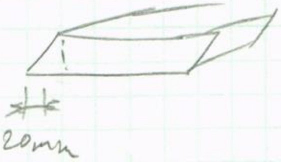
$$h_e \leq \frac{L}{5}$$

$$h_e \leq \frac{200}{5} = \underline{\underline{40mm}}$$

COMPRESSIVE DEFORMATION LIMIT: $0.07\, h_e = \underline{\underline{2.8mm}}$ (SLS)
S6. 11.6.6.4

ROTATION LIMIT : $0.14 h_e$
$= 5.6mm$

SHEAR DEFLECTION LIMIT: $0.5h_e = 20mm$



BEARING PRESSURE - 11.6.6.7

$$\frac{201\,000\ N}{200^2} = 5.0\ MPa = NEED\ 4.5$$

TRY 250^2

$$\frac{201\,000}{250^2} = \underline{3.2\ MPa}\ ✓$$

⇒ $\underline{250 \times 250}$ ⇐ USE FOR BOTH PIERS

BEARING SPREADSHEET

WEST PIER

INPUT	
Factored rxn force (single truss)	322 kN

BEARING		
Required Bearing Area, s^2	0.0168 m^2	
Required Bearing length, s	130 mm	
Actual Bearing Length Increment	50 mm	
Actual Bearing Length	150 mm	
Actual Bearing Area	0.0225 m^2	
Allowable Bearing Load	430 kN	**OK**

Note: Assumed square bearing surface

MIDDLE PIER

INPUT	
Factored rxn force (single truss)	564 kN

BEARING		
Required Bearing Area, s^2	0.0295 m^2	
Required Bearing length, s	172 mm	
Actual Bearing Length Increment	50 mm	
Actual Bearing Length	200 mm	
Actual Bearing Area	0.04 m^2	
Allowable Bearing Load	765 kN	**OK**

Note: Assumed square bearing surface

EAST PIER

INPUT	
Factored rxn force (single truss)	242 kN

BEARING		
Required Bearing Area, s^2	0.0127 m^2	
Required Bearing length, s	112 mm	
Actual Bearing Length Increment	50 mm	
Actual Bearing Length	150 mm	
Actual Bearing Area	0.0225 m^2	
Allowable Bearing Load	430 kN	**OK**

Note: Assumed square bearing surface

APPENDIX H: PIER DESIGN CALCULATIONS

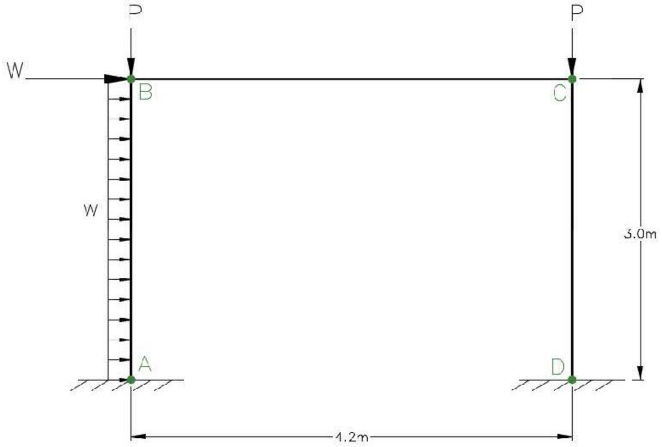
Project:	Capstone Design Project	Done by:	Omnirey L	Date:	10/02/2015	
Subject:	Bridge Piers	Checked by:	Myles C	Page:	1	of 1 1

SAP ANALYSIS SPREADSHEET

Middle Pier

W	277 kN
w	1.5 kN/m
P	367 kN
P_c	2.10552E+15

REACTION	RESULTS
	SAP (kN, kNm)
A_x	145
A_y	275
M_a	375
B_x	140
B_y	275
M_b	193
C_x	140
C_y	460
M_c	191
D_x	138
D_y	460
M_d	362
Beam Shear	92



Beam Governing

Moment	193 kNm
Shear	92 kN

Column Governing

Moment	375 kNm
Shear	145 kN
Axial	460 kN

Amplified by Slenderness Effects

δ 1.00

Project:	Capstone Design Project	Done by:	Omnirey L	Date:	10/02/2015
Subject:	Bridge Piers	Checked by:	Myles C	Page:	2 of 11

MIDDLE BEAM SPREADSHEET

INPUT						
Length	3.5	m	Beam Width	0.8	m	<- equal to column base
Depth	0.6	m	Self Weight	11.3	kN/m	
Mf	210	kNm				

REINFORCEMENT						
Top Reinf						
reinf size	25M		Ab	500	mm^2	
No. of Bars	5		db	25	mm	
cover	70	mm	As	2500	mm^2	
Spacing	150	mm	Center of steel	82.5	mm	(measured from top)
Bottom Reinf						
reinf size	25M		Ab	500	mm^2	
No. of Bars	5		db	25	mm	
cover	70	mm	As	2500	mm^2	
Spacing	150	mm	Center of steel	82.5	mm	(measured from bottom)

Note: Compression steel = Tension Steel

CHECKS				
clear distance b/w bars		125	>1.5db	
Reinforcement Limits (Compression)				
As / Ag (Max)	0.0104	0.08	OK	CSA 8.8.5.6
(As*fy)/(Ag*f'c) (Min	0.139	0.135	OK	
Tr=Cr'=	900	kN		
Mr=	392	kNm	OK	
As =	5000	mm^2		
Asmin =	1315	mm^2	OK	CSA A23.3 10.5.1.2
Crack Control				CSA A23.3 10.6.1
z=	14180	N/mm	OK	
Upper limit:	25000	N/mm		
				root fc 5.4772256
Shear check				
Vf=	112	kN		
dv	466	mm		CSA S6 8.9.1.5
bw	800	mm		CSA S6 8.9.1.6
β	0.157			CSA A23.3 11.3.6.3 / CSA S6 8.9.3.6
Vrmax	1876	kN		
Vc	240	kN		CSA S6 8.9.3.4
Vr =	240	kN	OK	

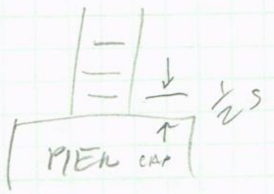
CHECKS CONT.							
Transverse Reinf Requirement Check		122	kN	**SHEAR REINF NOT REQD**	CSA S6 8.9.1.2	Stirrups for compression reinf provided (CSA S6 8.14.3)	
Av/s	Min transverse reinf	0.657	mm^2/mm				
Total Area reqd		2300.435	mm^2				
Thickness of Stirrup		11	mm				
Perp area of one stirrup		10120	mm^2				
spacing		300	mm		<- code minimum; not required for shear		
Min Reinforcement Check							
CSA S6 8.8.4.3							
fcr		2.2	Mpa				
y		530	mm				
I		9.925E+09	mm^4				
Mcr		41.03	kNm		CSA S6 8.8.4.4		
factored Mcr		49.23	kNm	**OK**			
Max Reinforcement Check							
CSA S6 8.8.4.4							
c=	300		mm				
d	518		mm				
c/d	0.6		< 0.5	**NOT OK**	*See section 4.2.1 for assumption used		
Development Length in Tension					Development Length in Compression		
CSA S6 8.15.2.3					CSA S6 8.15.3.1		
k1	1.3				ld min	440	mm
k2		1			ld'	456	mm
k3	1						
Mod factor	-		CSA S6 8.15.2.5		Mod factor	0.75	CSA S6 8.15.3.2
ld		1068	mm		ld	440	mm
Hook length	CSA S6 8.15.5.3						
ld'		456	mm				
400MPa		1					
		0.7					
		0.8					
ld	256		mm				

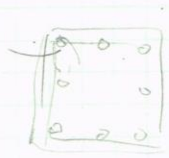
*Denotes automated calculation

SC 8.14.3 – STIRRUPS REQ'D FOR COMPRESSION REINFORCEMENT IN FLEX. COMPONENTS

→ STIRRUPS 10M @ 300mm (CLOSED) ✓ 12.5 + 15

8.14.3 TIES FOR COLUMNS
→ USE 300mm





DEVELOPMENT LENGTH : S6 8.15.2.3

$0.18\, k_1 k_2 k_3 \frac{f_y}{f_u} d_b$

$k_1 = 1.3$
$k_2 = 1.0$
$k_3 = 1.0$
} 8.15.2.4

MODIFICATION FACTOR : 8.15.2.5 $\frac{A_s \text{ REQ'D}}{A_s \text{ PROVIDED}} = \frac{M_f}{M_r} = \frac{322}{392}$ = ~~0.82~~

DON'T APPLY (8.15.2.5)

IN COMPRESSION : (8.15.3.1)

$l_b = \frac{0.1 (f_y) d_b}{f_{cr}}$

$= \frac{0.1 (400)(25mm)}{2.2}$

$= 454mm$

CHECK MIN:

$= 0.\overset{200}{044} (400MPa)(25$

440 mm

⇒ 450

MOD FACTOR: ~~0.82~~ × 0.75 = ~~0.62~~

⇒ l_b = ~~280~~ 340 mm

⇒ USE 350mm

MIN. BEND DIAMETER 150 mm – 25M
(INSIDE DIA) 200mm – 30M

BASIC DEV. LENGTH:

$$\frac{40\ D_v}{f_{cr}} = \frac{40(25)}{2.2} = 450\ mm$$

×0.8
×0.7

254 mm

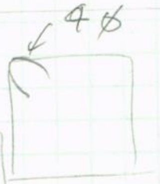
⇒ 300mm

COMBINATION DEV. LENGTH = S6 8.15.6.
(TOO ACCOUNT FOR COMPRESSION REQ'MENTS AS WELL)

LAP SPLICES → MIN. 0.2 (SPLICE LENGTH)
SPACING 150 mm

(S6 8.15.9.1)

FOR TIES



S6. 8.14.13

1.5 × AGG SIZE = 40

MAX AGG. SIZE = 25mm

S6 8.14.2.1

MIDDLE PIER COLUMN SPREADSHEET

INPUT

Unsupported length	5	m
Cover	70	mm
b	800	mm
h	800	mm
A_g	640,000	mm^2
Reinforcement Size	30M	
d_b	30	mm
A_b	700	mm^2
No. of Bars	8	
A_s	5600	mm^2
Assumed size of ties	10M	
d_{tie}	11	mm

AXIAL LOAD RESISTANCE W/ NO ECCENTRICITY

P_{r0}	13,507	kN
P_{rmax}	10,805	kN

DEPTH OF NEUTRAL AXIS BASED ON BALANCED CONDITION

d	704	mm
c	448	mm

POINTS IN THE INTERACTION DIAGRAM

REINF. LAYER	NO. OF BARS	A_{si}	d_i
1	3	2100	96
2	2	1400	400
3	3	2100	704

STRAINS IN REINFORCEMENT LAYERS

c (mm)	700	500	448	300	75
ε_{s1}	0.0030	0.0028	0.0028	0.0024	-0.0010
ε_{s2}	0.0015	0.0007	0.0004	-0.0012	-0.0152
εs3	0.0000	-0.0014	-0.0020	-0.0047	-0.0294

*Denotes yielding

STRESSES IN REINFORCEMENT LAYERS

c (mm)	700	500	448	300	75
f_{s1}	400	400	400	400	-196
f_{s2}	300	140	75	-233	-400
f_{s3}	-4	-286	-400	-400	-400

*Denotes yielding

FACTORED AXIAL RESISTANCE

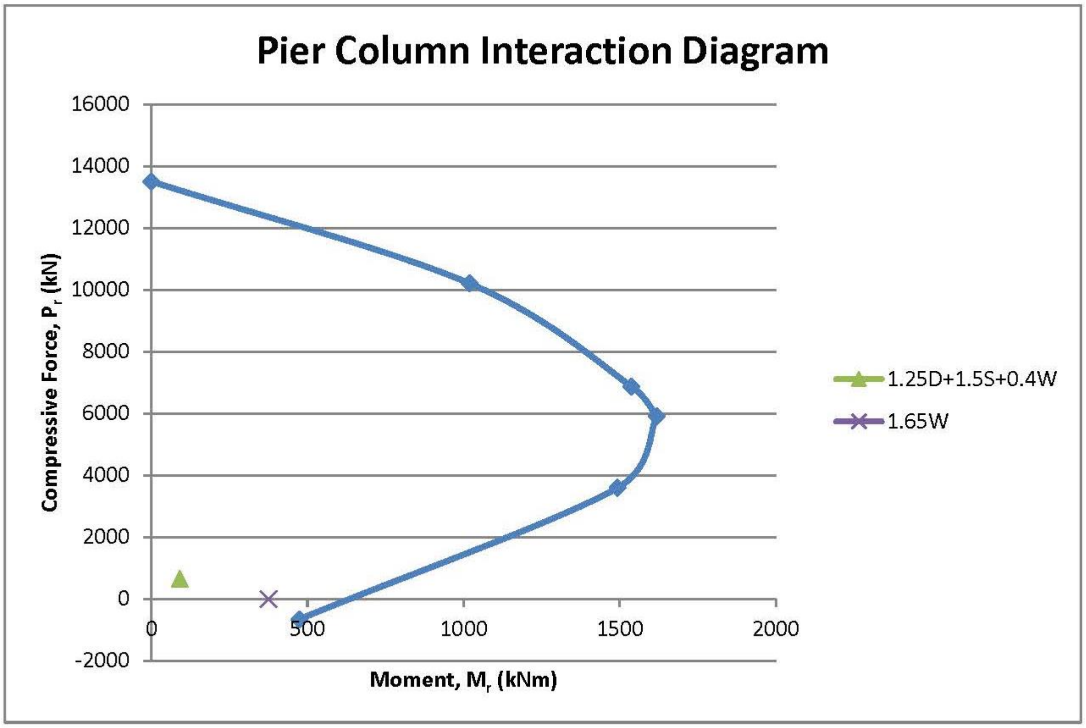
c (mm)	700	500	448	300	75
C_r (kN)	9078	6484	5810	3891	973
F_{rs1}	756	756	756	756	-370
F_{rs2}	378	176	95	-294	-504
F_{rs3}	-8	-540	-756	-756	-756
P_r (kN)	10204	6877	5904	3597	-658

FACTORED MOMENT RESISTANCE

c	700	500	448	300	75
M_{rc} (kNm)	788	1143	1159	1034	356
M_{rs1}	230	230	230	230	-113
M_{rs2}	0	0	0	0	0
M_{rs3}	2	164	230	230	230
M_r (kNm)	1020	1537	1619	1494	474

Interaction Diagram	
P_r (kN)	M_r (kNm)
13507	0
10204	1020
6877	1537
5904	1619
3597	1494
-658	474

LOAD CASE	P_r (kN)	M_r (kNm)
1.25D+1.5S+0.4W	644	91
1.65W (Lateral only)	-	-
1.65W (Lateral only)	0	375



ADDITIONAL CHECKS

BUCKLING CHECK			
P	322	kN	
e	0	mm	
I	3.413E+10	mm^4	
Le	10000	mm	
Pcr, Euler Buckling Resist	84,221	kN	OK

SHEAR CHECK				
Vf	145	kN		
dv	643.5	mm	CSA S6 8.9.1.5	
Max Shear Resist:	2895.75	kN	CSA S6 8.9.3.3	OK
β	0.18		CSA S6 8.9.3.6	
Vc	381	kN	CSA S6 8.9.3.4	OK

SLENDERNESS CHECK			
k	2		
r	230.94011		
kl/r	43.30127	If less than 100, can ignor slenderness effect	CSA A23.3 10.13.2 CSA S6 8.8.5.3

PILECAP SPREADSHEET

INPUT						
thickness	1.04	m	Beam Width	1.61	m	
depth	1.61	m	Pile Dev	0.05	m	CSA A23.3 15.2.3
P	555	kN	(Compression)			
Mf	221	kNm	(Bending)			

REINFORCEMENT						
Note: Compression steel = Tension Steel						
Top Reinf						
reinf size	25M		Ab	500	mm^2	
No. of Bars	6		db	25	mm	
cover	100	mm	As	3000	mm^2	
Spacing	250	mm	Center of steel	112.5	mm	(measured from top)
Bottom Reinf						
reinf size	25M		Ab	500	mm^2	
No. of Bars	6		db	25	mm	
cover	100	mm	As	3000	mm^2	
Spacing	250	mm	Center of steel	112.5	mm	(measured from bottom)

CHECKS						
clear distance b/w bars	225	>1.5db				
Tr=Cr'=	1080	kN				
Mr=	1496	kNm	OK			
As =	6000	mm^2				
Asmin =	3348.8	mm^2	OK	CSA A23.3 10.5.1.2 (Slab)		
Crack Control						
z=	16710	N/mm		CSA A23.3 10.6.1		
Upper limit:	25000	N/mm	OK			
Shear check						
Vf=	145	kN				
dv	1348	mm	CSA S6 8.9.1.5		root fc	5.4772256
bw	1610	mm	CSA S6 8.9.1.6			
β	0.098		CSA A23.3 11.3.6.3 / CSA S6 8.9.3.6			
Vrmax	10924	kN				
Vc	873	kN	CSA S6 8.9.3.4			
Vr =	873	kN	OK			

Project:	Capstone Design Project	Done by:	Omnirey L	Date:	10/02/2015
Subject:	Bridge Piers	Checked by:	Myles C	Page:	10 of 11

CHECKS CONT.

Transverse Reinf Requirement Check	713	kN	**SHEAR REINF NOT REQD**	CSA S6 8.9.1.2	Stirrups for compression reinf provided (CSA S6
Min transverse reinf	1.323	mm^2/mm			
Total Area reqd	1375.660	mm^2			
Thickness of Stirrup	11	mm			
Perp area of one stirrup	31020	mm^2			
spacing	300	mm		<- code minimum; not required for shear	
Min Reinforcement Check					
CSA S6 8.8.4.3					
fcr	2.2	Mpa			
y	1510	mm			
I	4.619E+11	mm4			
Mcr	670.22	kNm		CSA S6 8.8.4.4	
factored Mcr	804.27	kNm	**OK**		
Max Reinforcement Check					
CSA S6 8.8.4.4					
c=	805	mm			
d	1498	mm			
c/d	0.5	< 0.5	**NOT OK**	*See section 4.2.1 for assumption used	

*Denotes automated calculation

Project:	Capstone Design Project	Done by:	Omnirey L	Date:	10/02/2015
Subject:	Bridge Piers	Checked by:	Myles C	Page:	1 of 1 1 1

Material Properties

Steel		
Parameter	Value	Reference
Φ_s	0.9	CSA S6 8.4.6
f_y	400 Mpa	CSA S6 8.4.2.1.3
E_s	200,000 MPa	CSA S6 8.4.2.1.4
ε_s	0.002	-

Reinforcing Bars				
Nominal Bar	Diameter (mm)	Area (mm^2)	Perimeter (mm)	Mass per Length (kg/m)
10M	11	100	36	0.8
15M	16	200	50	1.6
20M	20	300	61	2.4
25M	25	500	79	4.0
30M	30	700	94	5.5
35M	36	1000	112	8.0
45M	44	1500	137	12.0
55M	56	2500	177	20.0

Concrete		
Parameter	Value	Reference
ρc	2400 kg/m3	
γc	23.5 kN/m3	
Φ_c	0.75	CSA S6 8.4.6
f'_c	30 MPa	CSA S6 8.4.1.2
α1	0.81	CSA A23.3 10.1.7
β1	0.90	CSA A23.3 10.1.7
E_c	25,000 MPa	(see notes, Jan 31)
ε_{cmax}	0.0035	CSA A23.10.1.3
λ	1	CSA A23.3 8.6.5
fcr	2.2 MPa	CSA S6 8.4.1.8

Appendix I: Pile Design Calculations

FROM GEOTECH REPORT

CFEM CLAUSES SHOWN

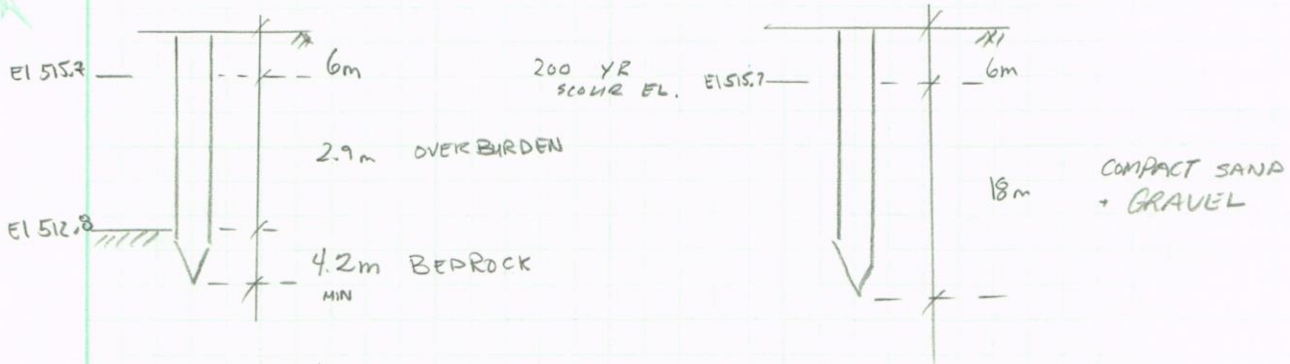
$\phi_c = 0.4$
$\phi_T = 0.3$
} CFEM CLAUSE 18.2.1 ✓ ok

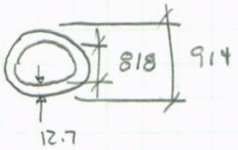
PRELIM DESIGN OF PILE CASE B

$\beta = 0.6$ $N_T = 50$ (TABLE 18.2 - DRIVEN PILE MED SAND)
(TABLE 18.1 - DRIVEN PILES MED SAND)

MIN SPACING = 2.5D

CASE A CASE B

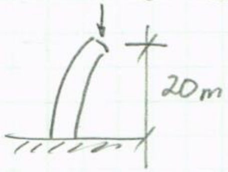




INDIVIDUAL PILE STRENGTH (18.2.1)

$$R = \sum_{i=0}^{L} \underbrace{C_{qs}\,\Delta z}_{\text{SHAFT FRICTION}} + \underset{\text{PILE TOE BEARING}}{A_q} - \underset{\text{WEIGHT OF PILE}}{W_P}$$

FOR CASE A PILE COMP RESIST IS ASSUMED TO BE BEARING CAPACITY



SHAFT FRICTION

ABOVE CRITICAL DEPTH:

$$q_{s1} = (\sigma_v')(K_s \tan \delta)$$

$$q_{s1} = \left(\frac{\gamma \cdot D_c}{2}\right)(K_s)(\tan \delta) \quad ;$$

$$q_{s1} = \left(\frac{9\,kN}{m^3} \cdot \frac{9.14m}{2}\right)(0.8)(0.4)$$

$$\underline{\underline{q_{s1} = 13.1\ kN/m^2}}$$

BELOW CRIT. DEPTH:

$$q_{s2} = (\sigma_v')(K_s \tan \delta)$$

$$q_{s2} = (\gamma \cdot (D_c))(K_s)(\tan \delta)$$

$$q_{s2} = \frac{9\,kN}{m^3} \cdot (19.14)(0.8)(0.4)$$

$$\underline{q_{s2} = 26.3\ kN/m^2}$$

PILE TOE BEARING CAP.

$$q_t = N_t \cdot \sigma_t' \quad ; \quad N_t = 50 \quad , \quad \sigma_t' = 9 \cdot 9.14 = 82.2\ kN/m^2$$

$$q_t = 50 \cdot (82.2\ kN/m^2)$$

$$\underline{\underline{q_t = 4113\ kN/m^2}}$$

TOTAL PILE BEARING CAP.

$$C = \pi D$$

$$= \pi (0.914)$$

$$\underline{C = 2.87m}$$

$$A = \frac{\pi(.914^2 - 0.888^2)}{4} \text{ (WITHOUT CONC. PLUG)}$$

$$\underline{A_1 = 0.0368\ m^2}$$

$$\underline{A_2 = 0.6561\ m^2} \text{ (WITH PLUG)}$$

-> WEIGHT OF STEEL PILE

$$\underline{\gamma_{STEEL} = 77\ kN/m^3}$$

$$W_p = \gamma \cdot A_1 \cdot L$$

$$= 77\ kN/m^3 \cdot 0.0368m^2 \cdot 24m$$

$$\underline{\underline{W_p = 68\ kN}}$$

FROM SPONSOR

$$\underline{1300\ kN}$$

FOR FULL PILE-LENGTH W/O CONSIDERING SCOUR LEVE
+ CONC. PLUG

$$R = \sum_{z=0}^{L} C q_s \Delta z + A q_t - W_p$$

$$= 2.87m\left(13.1\ \frac{kN}{m^2}(9.14m) + 26.3\ \frac{kN}{m^2}(14.8m)\right) + (0.0368)(4113) - 68$$

$$= 1461\ kN + 151\ kN - 68 \longrightarrow (\overset{0.6/0.8}{0.75})(1461) + 151\,kN - 68$$

$$\boxed{R = \underline{\underline{1544\ kN}}}$$

IF 0.6 = K_s $\underline{\underline{R = 1178\ kN}}$

EMPIRICAL METHOD (CFEM) → CASE B

$S = S_p + S_s$; $S_s = S_{ss} + S_{st}$

S_p ↳ ELASTIC DEFORMATION OF PILE SHAFT

S_{ss} → PILE SETTLEMENT CAUSED BY LOAD TRANSMITTED ALONG SHAFT

S_{st} → SETTLEMENT OF PILE TOE CAUSED BY LOAD TRANSMITTED @ TOE

$$S_p \approx 0.75 \frac{QL}{A_p E_p}$$

$QL = P_L = 154\ kN + 367\ kN$

154 kN ↳ PIER CONC. WEIGHT

367 kN (GRAVITY @ TOE)

$QL = 521\ kN$ ← GOVERNS

OR 460 kN (LATERAL FROM EXCEL)

$P_s = 24m\ (0.0368m^2)\ (77\ kN/m^3)$

0.0368m² ↳ AREA

$P_s = 68kN$

$\Rightarrow P = 521\ kN + 68\ kN = 590\ kN = Q_c$

$S_p = 0.75\ (590\ kN)\ (24m) / 0.0368m^2 (200\,000\ KPa)$

$= 1.4\ mm$

$$S_E = \frac{C_t\, Q_{BA}}{d\, q_t} = \frac{0.09\ Q_{bu}}{0.914\ (4113)}$$

0.09 ↳ DRIVEN

$$S_s = \frac{C_s\, Q_{SA}}{L\, q_t}$$ (ASSUME SHAFT CARRIES ALL LOAD SINCE PILE CAP. NOT REACHED)

$C_s = 0.93 + 0.16(L/D)^{0.5} = 0.93 + 0.16\,(24/0.914)^{0.5}$
$= 1.75$

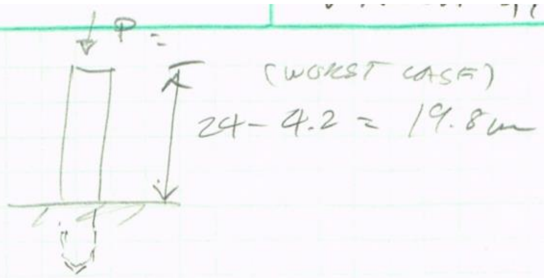
$$S_s = \frac{1.75\ (590\ kN)}{24m\ (4113\ kN/m^3)}$$

$= 10.5mm$ ⟹ TOTAL DEFLECTION
$= 1.4 + 10.5 = 12mm$

PILE SETTLEMENT

CASE A

$$\delta = \frac{PL}{EA}$$



$$\delta = \frac{590\,000\ N\ (19\,800\ mm)}{(200\,000\ MPa)(0.0368 \times 1000^2\ mm^2)}$$

Δ = 2mm ✓ MINIMAL

ACCORDING TO CFEM, GENERALLY INSIGNIFICANT DEFLECTION DUE TO ROCK SETTLEMENT

⇒ FROST HEAVE CANNOT BE CALCULATED

→ NO GRADATION OF INSITU THERMAL GRADIENT

→ RETAINED BY GEOTHERMAL SIMULATION

EVANS + DUNCAN'S CHARTS

$V_c = \lambda B^2 E R_1 \left(\frac{\sigma_p}{E R_1}\right)^m (\epsilon_{50})^n$ EQN. 16.8

$\lambda = 1.00$ (SAND)

$\sigma_p = 2 C_{p\phi} \gamma B \tan^2(45 + \phi'/2)$ EQN 16.14

$B = 914\text{mm}$ [DIAMETER OF PILE]

$\phi = 33°$ [INTERNAL FRICTION ANGLE]

$C_{p\phi} = \frac{\phi'}{10} = 3.3$

$\gamma = \frac{\gamma + \gamma_b}{2} = \frac{9 + 9.81\ \text{kN/m}^3}{2} = \underline{9.41\ \text{kN/m}^3}$

OR

$\gamma = \gamma_b = \underline{9\ \text{kN/m}^3}$ (UNSURE) ⇐ CONSERVATIVE

$\Rightarrow \sigma_p = 2(3.3)(9\ \text{kN/m}^3)(0.914\text{m})[\tan^2(45 + 33/2)]$

$\sigma_p = \underline{184\ \text{kPa}}$

$R_1 = \frac{I}{\pi B^4 / 64}$; $I = 3.734 \times 10^9\ \text{mm}^4$ (JAN. 21 NOTES)

$B = 914\text{mm}$

$R_1 = \frac{3.734 \times 10^9\ \text{mm}^4}{\pi (914\ \text{mm})^4 / 64}$

$\underline{R_1 = 0.109}$ [MOMENT OF INERTIA RATIO]

$E = 200\ \text{GPa}$ (STEEL)

$\epsilon_{50} = 0.002$ [AXIAL STRAIN AT WHICH 50% OF SOIL IS MOBILIZED] (TABLE 16.2)

$m = 0.57$
$n = -0.22$ } TABLE 16.3

$V_c = (1.0)(0.914\text{m})^2 (200\ 000\ 000\ \text{kPa})(0.109) \left[\frac{184\ \text{kPa}}{200 \times 10^6\ \text{kPa}\,(0.109)}\right]^{0.57} 0.002^{-}$

$\underline{V_c = 91\ 650\ \text{kN}}$ [CHARACTERISTIC SHEAR LOAD

16.5 EVANS AND DUNCAN'S CHARTS

Evans and Duncan (1982) developed a convenient method of expressing the lateral load-deflection behavior in chart form. They compiled these charts from a series of *p-y* method computer analyses using the computer program COM624.

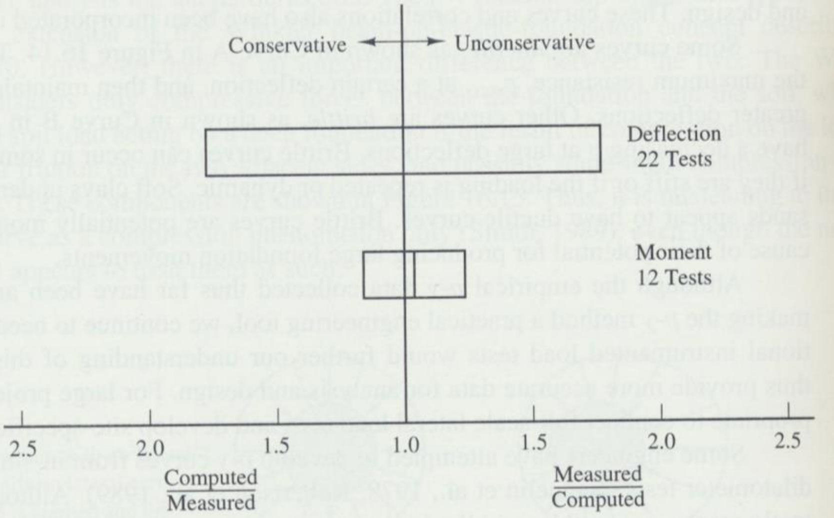


Figure 16.16 90 percent confidence interval for computed lateral deflection and bending moment predictions from *p-y* analysis (based on data from Reese and Wang, 1986). The line in the middle of each bar represents the average prediction, and the number to the right is the number of data points.

Characteristic Load and Moment

Evans and Duncan defined the *characteristic shear load*, V_c, and *characteristic moment load*, M_c, as follows:

$$\boxed{V_c = \lambda\, B^2\, ER_I \left(\frac{\sigma_p}{ER_I}\right)^m (\epsilon_{50})^n} \tag{16.8}$$

$$\boxed{M_c = \lambda\, B^3\, ER_I \left(\frac{\sigma_p}{ER_I}\right)^m (\epsilon_{50})^n} \tag{16.9}$$

$$R_I = \frac{I}{\pi B^4/64} \tag{16.10}$$

$= 1.00$ for solid circular cross sections

$= 1.70$ for solid square cross sections

For plastic clay and sand:

$$\lambda = 1.00 \tag{16.11}$$

For brittle clay[1]:

$$\lambda = (0.14)^n \tag{16.12}$$

For clay:

$$\sigma_p = 4.2\, s_u \tag{16.13}$$

For sand:

$$\sigma_p = 2\, C_{p\phi}\, \gamma\, B \tan^2 (45 + \phi'/2) \tag{16.14}$$

Where:

V_c = characteristic shear load
M_c = characteristic moment load
λ = a dimensionless parameter dependent on the soil's stress-strain behavior
B = diameter of foundation
E = modulus of elasticity of foundation
= 29,000,000 lb/in^2 (200,000 MPa) for steel
= 57,000 $\sqrt{f'_c}$ lb/in^2 (4700 $\sqrt{f'_c}$ MPa) for concrete
= 1,600,000 lb/in^2 (11,000 MPa) for Southern pine or Douglas fir
f_c' = 28-day compressive strength of concrete (lb/in^2, MPa)
R_I = moment of inertia ratio (dimensionless)
σ_p = representative passive pressure of soil
ϵ_{50} = axial strain at which 50 percent of the soil strength is mobilized (see Table 16.2)
m, n = exponents from Table 16.3
I = moment of inertia of foundation
= $\pi B^4/64$ for solid circular cross sections
= $B^4/12$ for square cross sections
Also see tabulated values in Chapter 12
s_u = undrained shear strength of soil from the ground surface to a depth of 8 pile diameters
ϕ' = effective friction angle of soil (deg) from ground surface to a depth of 8 pile diameters
$C_{p\phi}$ = passive pressure factor = $\phi'/10$
γ = unit weight of soil from ground surface to a depth of 8 pile diameters. If the groundwater table is in this zone, use a weighted average of γ and γ_b, where $\gamma_b = \gamma - \gamma_w$ (the buoyant unit weight in the zone below the groundwater table.

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[1] A brittle clay is one with a residual strength that is much less than the peak strength.

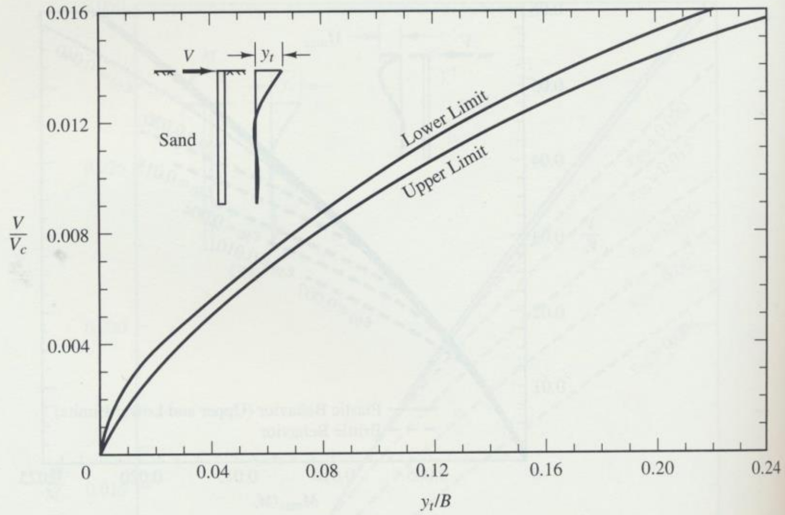


Figure 16.22 Shear load vs. lateral deflection curves for free-head condition in sand (Evans and Duncan, 1982).

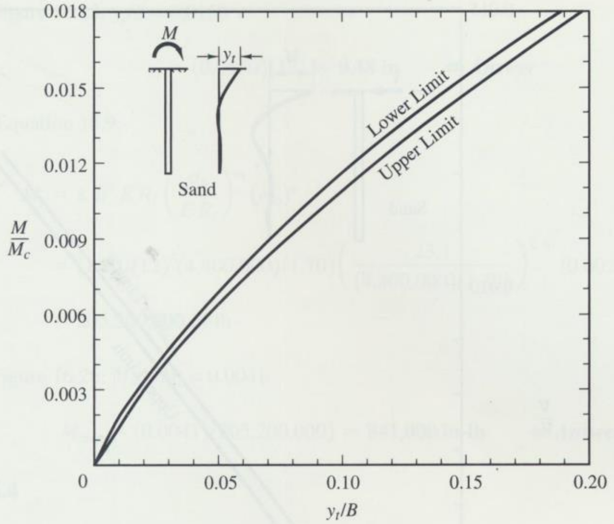


Figure 16.24 Moment load vs. lateral deflection curves for free-head condition in sand (Evans and Duncan, 1982).

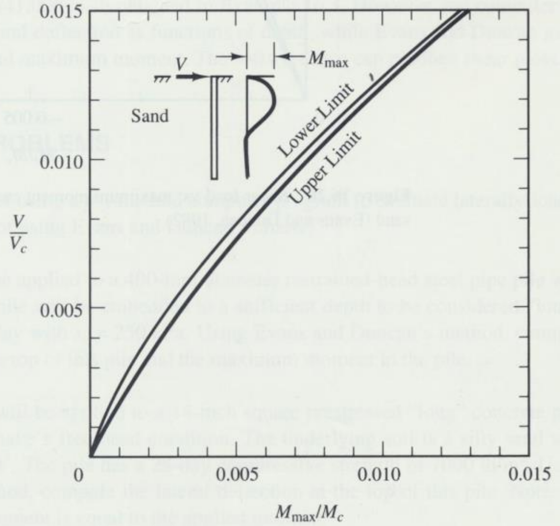


Figure 16.25 Shear load vs. maximum moment curves for free-head condition in sand (Evans and Duncan, 1982).

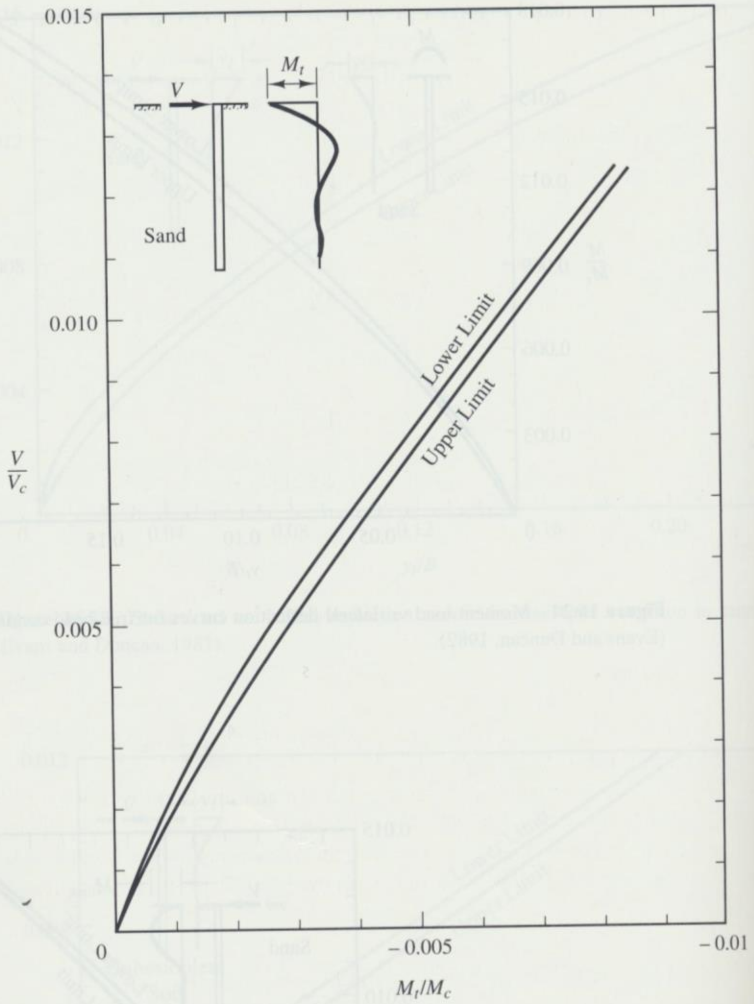


Figure 16.26 Shear load vs. maximum moment curves for restrained-head condition in sand (Evans and Duncan, 1982).

APPENDIX J: RIPRAP DESIGN CALCULATIONS

Project:	Capstone Design Project	Done by:	Omnirey L	Date:	26/03/2015
Subject:	Riprap Design Spreadsheet	Checked by:	Amos K	Page:	1 of 1

RIPRAP SPREADSHEET

Average channel velocity, v_{avg}	4.2	m/s
Local flow velocity, v_{ss}	3.36	m/s
Gravity, g	9.81	m/s^2

PARAMETER VALUES

Safety factor, S_f	1.2	
Stability coefficient, C_s	0.3	
Velocity distrib. coefficient, C_v	1	
Thickness coefficient, C_t	1	
Side-slope factor, K_1	0.9	* For 2H/1V
Rock relative density, s	2.65	

OUTPUT

Local flow depth, y (m)	D_{30} (mm)
0.5	311

REVETMENT

D_{30}	311	mm
Assumed D_{50}	389	mm
Rounded D_{50}	400	mm
Calculated thickness	700	mm

	Depth of scour below stream bed (m)	Volume of launching trench (m^3/m)
West Pier	3.5	8.2
Middle Pier	3.5	8.2
East Pier	3.5	8.2

APPENDIX K: CONSTRUCTION LIFTING CALCULATIONS

Maximum size crane for access 150 Ton.

Truss weights (using SAP): 8 panel = 4 × 31 kN = 124 kN
6 panel = 4 × 22 kN = 88 kN

from sponsors recommendation factor by 1.5

8 panel = 185.5 kN = 18 900 kg = 41 600 lb
6 panel = 141 kN = 14 400 kg = 31 600 lb

half of long span = 20m = 65.6'

LS-238H (Link Belt): 80' boom 70' radious
max lift = ~~36 500~~ lb (over toe)
28 700 lb (360°)

Need to reduce truss weight or lessen radious.

- truss weight cannot be reduced enough without removing too many members.
- upon reviewing the drawing radious can be reduced to 60'

Need to look @ 360° max lift since crane must reposition load.

70' boom 60' radious max 360° lift = 35 300 lb

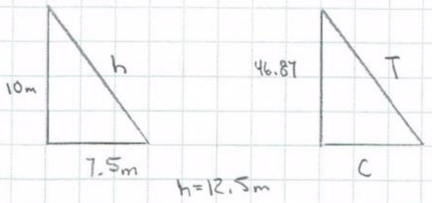
Review other cranes.

HC238H 70' @ 60' = 44 600 lb (crane truck)
CKS1350 21.3m @ 18m = 26 600 kg (crawler crane)
21.3m @ 20m = 21 700 kg

Truss can be lifted @ 60'.

Further calculations based off Kobelco CKS1350 crane.
(see attached)

Check cable tension and compression induced on truss.



$C = 46.87\left(\frac{7.5}{10}\right) = 35\text{ kN}$

$T = 46.87\left(\frac{12.5}{10}\right) = 59\text{ kN}$

C = compression applied to chord
T = tension in cable

assume lifting cable splits 10 m above truss
↳ taken from review of online lifts and to be conservative

boom is 24 m above the ground (see attached)
↳ allows for 14 m of lift

T = 59 kN = 6014 kg

Crosby 7/8" Screw Pin Anchor Shackle (see attached)

Working load 6.5 ton = 6500 kg

Shackle pin diameter = 1"

Lifting Lug Design:

1 1/8" = 29 mm

Check gross steel area: $\sigma = \frac{T}{A} = \frac{59 \times 10^3\text{ N}}{29 \times 25} = 81\text{ MPa} < f_y = 300$

CSA S16-09 13.2

$T_r = 0.75\,\phi\,A_{NE}F_y$ (pin connection)

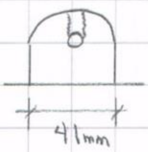
$A_{NE} \geq T_r / 0.75\phi F_y = 59\text{ kN} / 0.75(0.9)(300)\text{ MPa}$

$A_{NE} \geq 291\text{ mm}^2 \approx 300\text{ mm}^2$

$$\text{min width} = \frac{A_{n\geq}}{t} = \frac{300}{25} = 12mm$$

$$w \geq 10mm$$

check block shear $T_r = \phi_u \left[U_t A_N F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2} \right]$



$$A_{gv} = 2\left(\frac{12mm}{2}\right) 25mm = 300mm^2$$
$$A_N = 0$$

$$T_r = 0.75 \left[0 + 0.6(300mm^2) \frac{(300+450)MPa}{2} \right]$$

$$T_r = 67.5kN > T_f = 59\ kN$$

Weld Requirements

design as 8mm fillet weld
begin with 2" PL

$$V_r = 0.67 \phi_w A_m F_u$$
$$V_r = 0.67 \phi_w A_w X_u \left(1 + 0.5 \sin^{1.5}\theta\right) M$$
$$T_r = \phi_w \sqrt{(A_n F_u)^2 + (A_w X_u)^2}$$

load is @ 53° angle

$$V_r = 0.67(0.67)(25mm \times 50mm)(450\ MPa) = 250\ kN/side = 500kN$$

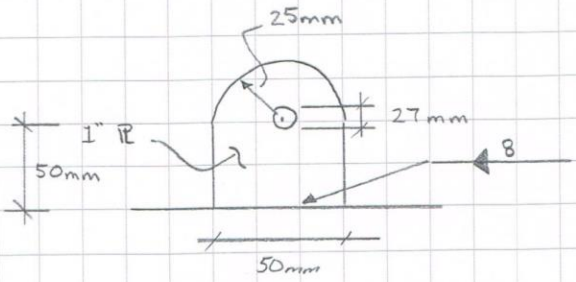
CISC HSC pg 3-41 8mm weld 45° (conservative)
$V_r = 1.61\ kN/mm$ E49XX electrode

$$V_r = 1.61(100) = 160\ kN$$

$$T_r = 0.67 \sqrt{\underbrace{(0.5 \times 2 \times 25mm \times 8mm \times 450MPa)^2}_{90kN} + \underbrace{(2 \times 25mm \times 8mm \times 490MPa)^2}_{196kN}}$$

$$= 144\ kN > T_f = 59kN$$

Lifting Lug Summary



adjust shackle hole to
27mm S16-09

check PL against bending

⊥ load = 35kN

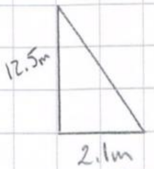
$M = 35\text{kN}(50\text{mm}) = 1.75\text{ kNm}$

$\sigma = \frac{My}{I}$ $I = \frac{bd^3}{12} = \frac{25(50)^3}{12} = 260\times10^3\text{ mm}^4$

$= \frac{1.75\times10^6\text{Nmm}(25\text{mm})}{260\times10^3\text{ mm}^4}$

$\sigma = 168\text{ MPa} < F_y = 300\text{ MPa}$

check lifting lug including 3rd dimension.



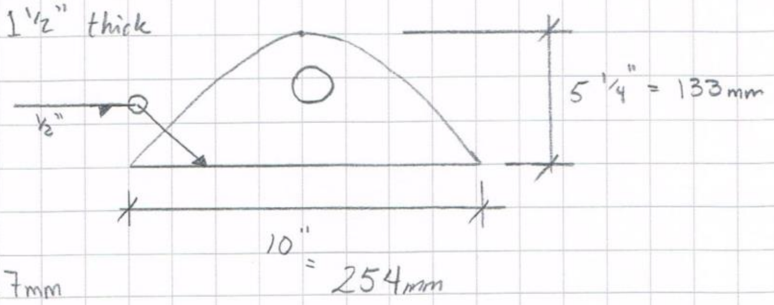
$$T = 59\,\text{kN}\left(\frac{12.7\,\text{m}}{12.5\,\text{m}}\right) = 60\,\text{kN}$$

$$h = \sqrt{2.1^2 + 12.5^2}$$

$$= 12.7\text{m}$$

1 kN change in tension is a 1.7% change which is insignificant to the lifting lug.

Winsafe Corp. 10T lifting lug.



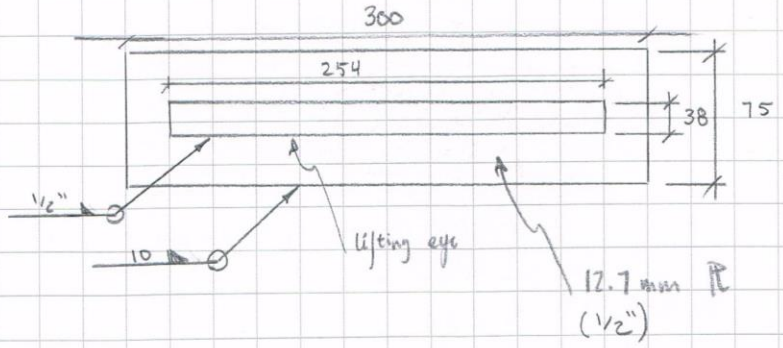
½" = 12.7mm

manufacturer requires 12.7mm minimum thickness for welding to to achieve full strength.

Welded to HSS 178 × 178 × 6.4

12.7 > 6.4 ∴ ℞ needed to distribute load

weld ½" ℞ to HSS to distribute load.



ground composed of fluvial sand and gravel
(geotechnical report test pits)

assume that material excavated for rip-rap and pier installation
composed of same material

from Reinforced Concrete Design Table 14.1

Sandy gravel or gravel	150 kPa
Sand, silty sand, clayey sand, silty gravel, clayey gravel	100 kPa

Empty crane ground pressure = 106 kPa

lifting weight = 186 kN (factored, 1.5)

track area = $910mm \times 7895mm = 7184450\ mm^2$ / track

$= 14.37 \times 10^6 mm^2$

$= 14.37\ m^2$

increase ground capacity when loaded

$186 \times 10^3 N / 14.37 \times 10^6 mm^2 = 13\ kPa$

factor by 1.5 to account for uneven pressure = 20 kPa

ensure soil can resist 136 kPa

Material is fluvial sand and gravel
· small grain sand and gravel

soil likely somewhere between two values shown in Table 14.1

→ material is a sand and gravel
→ fluvial is small grain could stray towards silty sand

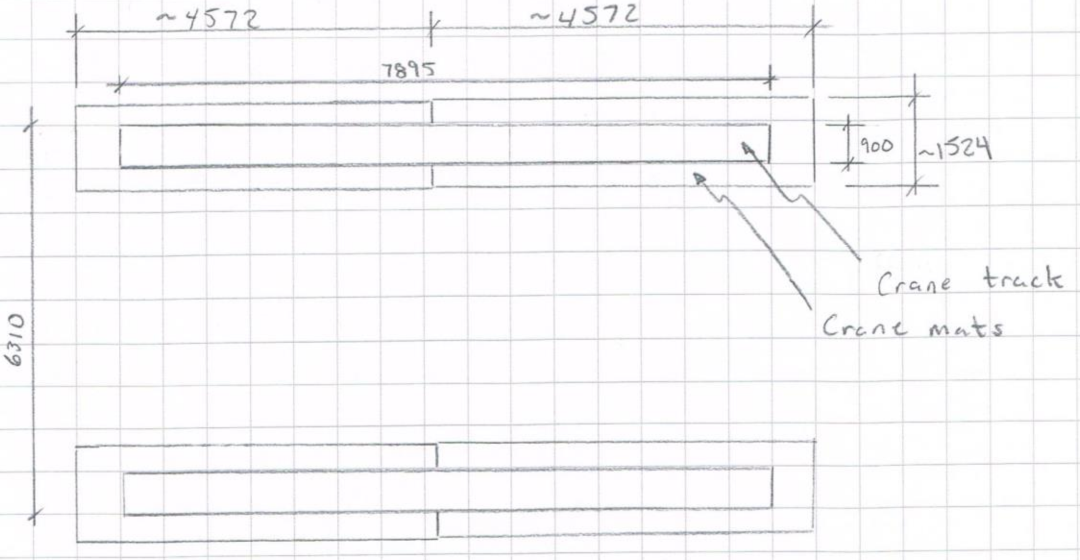
design for max pressure of 100 kPa (conservative)

$B_f = 136 \text{kPa} > B_r = 100 \text{kPa}$

need to design crane mats to reduce stress

crane mat required to double track area and half pressure

typical crane mat 15' × 5' × 8"
= 4572 × 1524 × 203



Bearing pressure from configuration shown

$$136 \text{ kPa} \times 14.37 \text{ m}^2 = 1954 \text{ kN}$$

$$B_f = \frac{1954 \text{ kN}}{2(2 \times 4572)(1524) \text{ mm}^2} = 70 \text{ kPa} < B_r = 100 \text{ kPa} \checkmark$$

$$\text{area per track} = (2 \times 4572 \text{ mm})(1524 \text{ mm}) = 13.9 \text{ m}^2$$

Crane mats shall provide 14m² of area per track.

LIFTING CAPACITIES

Crane Boom Lifting Capacities

Counterweight: 55.0 t
Carbody Weight: 10.8 t

Unit: metric ton

Boom length (m) / Working radius (m)	15.2	18.3	21.3	24.4	27.4	30.5	33.5	36.6	39.6	42.7	45.7	48.8	51.8	Boom length (m) / Working radius (m)
4.5	4.5m/135.0													**4.5**
5.0	131.1	5.1m/128.4	5.6m/117.2											**5.0**
6.0	110.4	110.1	109.6	6.1m/107.8	6.7m/95.1									**6.0**
7.0	95.1	94.8	93.3	91.1	89.3	7.2m/84.2	7.7m/75.3							**7.0**
8.0	79.5	79.9	79.1	77.4	75.9	74.6	72.4	8.2m/67.8	8.8m/61.7					**8.0**
9.0	67.7	68.8	68.5	67.2	66.0	64.9	62.5	61.5	60.0	9.3m/56.3	9.8m/51.8			**9.0**
10.0	58.4	59.0	59.0	58.8	58.3	57.4	56.5	55.0	53.6	52.2	50.9	10.4m/47.8	10.9m/44.2	**10.0**
12.0	44.3	45.7	45.6	45.4	45.2	45.2	45.1	44.9	44.1	43.0	42.0	41.0	40.0	**12.0**
14.0	33.5	37.1	37.0	36.8	36.6	36.5	36.5	36.3	36.2	36.1	35.6	34.7	33.9	**14.0**
16.0	14.8m/29.3	30.0	31.0	30.8	30.6	30.5	30.4	30.2	30.1	30.0	29.9	29.8	29.3	**16.0**
18.0		17.5m/24.8	26.6	26.4	26.2	26.1	26.0	25.8	25.7	25.6	25.4	25.3	25.2	**18.0**
20.0			21.7	23.0	22.8	22.7	22.6	22.4	22.3	22.2	22.0	21.9	21.7	**20.0**
22.0			20.1m/21.3	19.9	20.1	20.0	19.9	19.7	19.6	19.5	19.3	19.2	19.0	**22.0**
24.0				22.8m/18.5	18.0	17.9	17.7	17.5	17.4	17.3	17.1	17.0	16.8	**24.0**
26.0					25.4m/16.0	16.1	16.0	15.7	15.6	15.5	15.3	15.2	15.0	**26.0**
28.0						14.2	14.5	14.2	14.1	13.9	13.8	13.6	13.5	**28.0**
30.0						28.1m/14.1	13.2	12.9	12.8	12.7	12.5	12.3	12.2	**30.0**
32.0							30.7m/12.5	11.8	11.7	11.5	11.4	11.2	11.1	**32.0**
34.0								33.3m/10.9	10.8	10.6	10.4	10.3	10.1	**34.0**
36.0									9.7	9.8	9.6	9.4	9.2	**36.0**
38.0										8.9	8.8	8.7	8.5	**38.0**
40.0										38.6m/8.6	8.1	8.0	7.8	**40.0**
42.0											41.2m/7.5	7.4	7.2	**42.0**
44.0												43.9m/6.5	6.7	**44.0**
46.0													5.9	**46.0**
48.0													46.5m/5.7	**48.0**
Reeves	10	10	9	8	8	7	6	6	5	5	4	4	4	**Reeves**

Boom length (m) / Working radius (m)	54.9	57.9	61.0	64.0	67.1	70.1	73.2	76.2	Boom length (m) / Working radius (m)
10.0	11.4m/40.1	11.9m/38.4							**10.0**
12.0	39.1	38.2	12.5m/35.8	13.0m/33.4	13.5m/26.7				**12.0**
14.0	33.2	32.5	31.7	30.9	26.7	14.1m/26.7	14.6m/24.4	15.1m/20.4	**14.0**
16.0	28.7	28.1	27.4	26.7	26.3	25.7	22.7	19.4	**16.0**
18.0	25.1	24.6	24.0	23.4	23.0	22.5	20.6	17.5	**18.0**
20.0	21.6	21.5	21.2	20.7	20.4	19.9	18.8	15.8	**20.0**
22.0	18.9	18.8	18.6	18.4	18.1	17.7	17.1	14.3	**22.0**
24.0	16.7	16.6	16.4	16.2	16.2	15.8	15.4	13.0	**24.0**
26.0	14.9	14.7	14.6	14.4	14.4	14.2	13.8	11.8	**26.0**
28.0	13.4	13.2	13.1	12.9	12.8	12.7	12.4	10.7	**28.0**
30.0	12.1	11.9	11.7	11.6	11.5	11.4	11.2	9.7	**30.0**
32.0	10.9	10.8	10.6	10.4	10.4	10.2	10.0	8.8	**32.0**
34.0	10.0	9.8	9.6	9.4	9.4	9.2	9.1	8.0	**34.0**
36.0	9.1	8.9	8.8	8.6	8.5	8.4	8.2	7.2	**36.0**
38.0	8.4	8.2	8.0	7.8	7.8	7.6	7.4	6.5	**38.0**
40.0	7.7	7.5	7.3	7.1	7.1	6.9	6.7	5.8	**40.0**
42.0	7.1	6.9	6.7	6.5	6.5	6.3	6.1	5.2	**42.0**
44.0	6.5	6.4	6.2	6.0	5.9	5.7	5.5	4.6	**44.0**
46.0	6.0	5.9	5.7	5.4	5.3	5.2	4.9	4.0	**46.0**
48.0	5.3	5.4	5.2	4.9	4.9	4.7	4.4	3.5	**48.0**
50.0	49.2m/4.8	4.7	4.7	4.5	4.4	4.2	4.0	2.9	**50.0**
52.0		51.8m/4.1	4.2	4.1	4.0	3.8	3.6	2.4	**52.0**
54.0			3.6	3.6	3.5	3.4	3.2		**54.0**
56.0			54.4m/3.5	3.0	3.1	3.0	2.8		**56.0**
58.0				57.1m/2.8	2.6	2.5	2.4		**58.0**
60.0					59.7m/2.2	2.1			**60.0**
Reeves	3	3	3	3	2	2	2	2	**Reeves**

Note: Ratings according to EN13000.

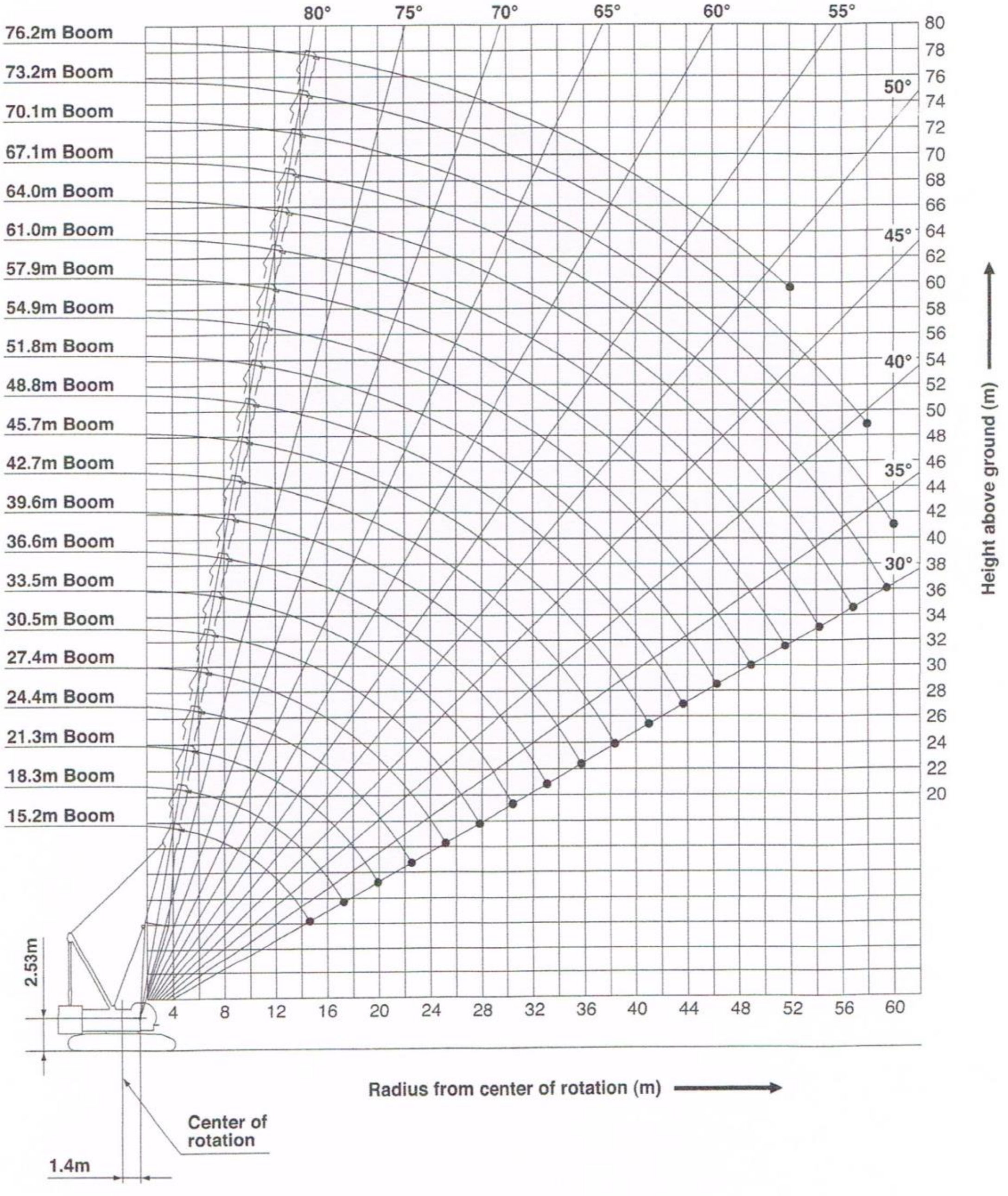
Ratings shown in ▭ are determined by the strength of the boom or other structural components.

Lifting capacities may vary depending on hook used or with/without auxiliary sheave.

Please refer rated chart in operator's cabin.

WORKING RANGES

Crane Boom



CROSBY FORGED SHACKLES

Screw Pin Anchor Shackles

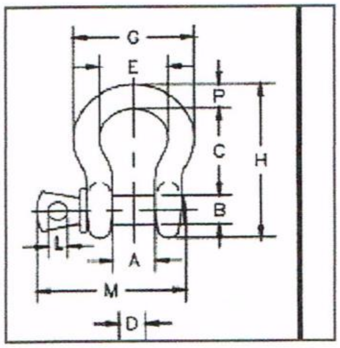
G-209 **S-209**

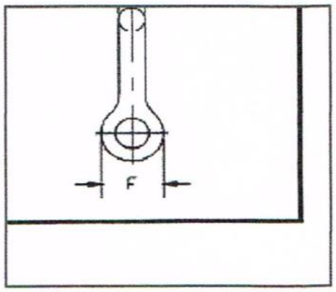
Screw pin anchor shackles meet the performance requirements of Federal Specification RR-C-271D Type IVA, Grade A, Class 2, except for those provisions required of the contractor.

- Shackles are Quenched and Tempered and can meet DNV impact requirements of 42 joules at -20C.
- Working Load Limit permanently shown on every shackle.
- Forged - Quenched and Tempered, with alloy pin.
- Capacities 1/3 thru 55 metric tons.
- Look for the Red Pin¨. . .the mark of genuine Crosby quality.
- Shackles can be furnished proof tested with certificates to designated standards, such as ABS, DNV, Lloyds, or other certification. Charged for proof testing and certification available when requested at the time of order.
- Hot Dip galvanized or Self-Colored.
- Fatigue Rated.

Nominal Size (in.)	Working Load Limit (t)*	Stock No.		Weight Each (lbs.)	Dimensions (in.)										
		G-209	S-209		A	B	C	D	E	F	G	H	L	M	P
3/16	1/3	1018357	—	.06	.38	.25	.88	.19	.60	.56	.98	1.47	.16	1.12	.19
1/4	1/2	1018375	1018384	.10	.47	.31	1.13	.25	.78	.61	1.28	1.84	.19	1.38	.25
5/16	3/4	1018393	1018400	.19	.53	.38	1.22	.31	.84	.75	1.47	2.09	.22	1.66	.31
3/8	1	1018419	1018428	.31	.66	.44	1.44	.38	1.03	.91	1.78	2.49	.25	2.03	.38
7/16	1-1/2	1018437	1018446	.38	.75	.50	1.69	.44	1.16	1.06	2.03	2.91	.31	2.38	.44
1/2	2	1018455	1018464	.72	.81	.63	1.88	.50	1.31	1.19	2.31	3.28	.38	2.69	.50
5/8	3-1/4	1018473	1018482	1.37	1.06	.75	2.38	.63	1.69	1.50	2.94	4.19	.44	3.34	.69
3/4	4-3/4	1018491	1018507	2.35	1.25	.88	2.81	.75	2.00	1.81	3.50	4.97	.50	3.97	.81
7/8	6-1/2	1018516	1018525	3.62	1.44	1.00	3.31	.88	2.28	2.09	4.03	5.83	.50	4.50	.97
1	8-1/2	1018534	1018543	5.03	1.69	1.13	3.75	1.00	2.69	2.38	4.69	6.56	.56	5.07	1.06
1-1/8	9-1/2	1018552	1018561	7.41	1.81	1.25	4.25	1.16	2.91	2.69	5.16	7.47	.63	5.59	1.25
1-1/4	12	1018570	1018589	9.50	2.03	1.38	4.69	1.29	3.25	3.00	5.75	8.25	.69	6.16	1.38
1-3/8	13-1/2	1018598	1018605	13.53	2.25	1.50	5.25	1.42	3.63	3.31	6.38	9.16	.75	6.84	1.50
1-1/2	17	1018614	1018623	17.20	2.38	1.63	5.75	1.54	3.88	3.63	6.88	10.00	.81	7.35	1.62
1-3/4	25	1018632	1018641	27.78	2.88	2.00	7.00	1.84	5.00	4.19	8.86	12.34	1.00	9.08	2.25
2	35	1018650	1018669	45.00	3.25	2.25	7.75	2.08	5.75	4.81	9.97	13.68	1.22	10.34	2.40
2-1/2	55	1018678	1018687	85.75	4.13	2.75	10.50	2.71	7.25	5.69	12.87	17.84	1.38	13.00	3.13

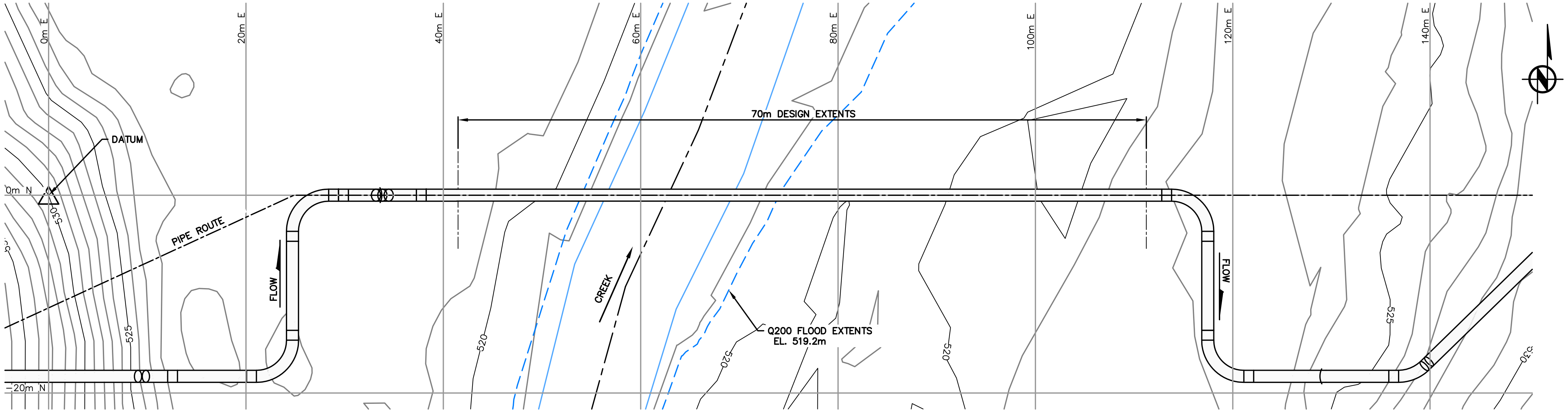
NOTE: Maximum Proof Load is 2.0 times the Working Load Limit. Minimum Ultimate Strength is 6 times the Working Load Limit. For Working Load Limit reduction due to side loading applications, see page 75.



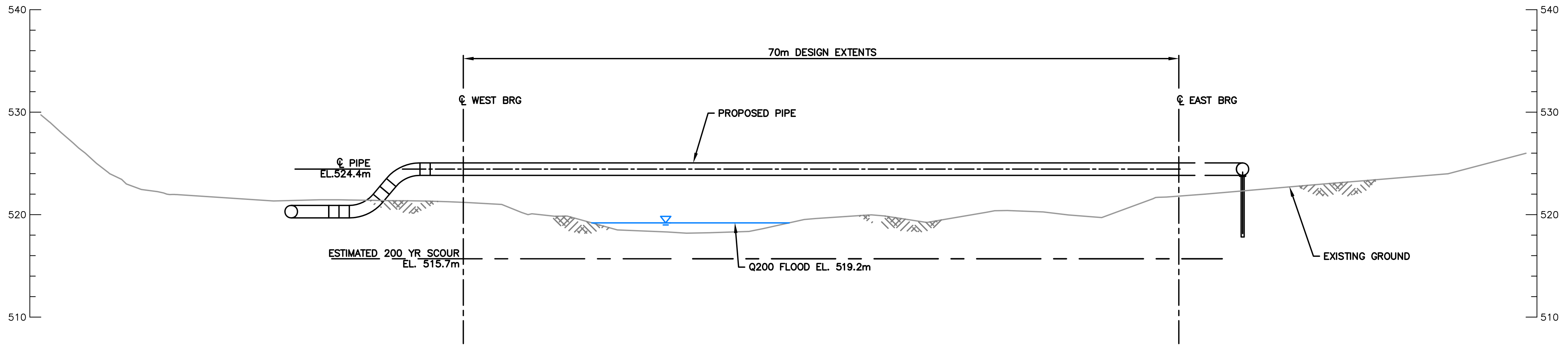


APPENDIX L: DESIGN DRAWING PACKAGE

MOJA
Inženjer



EXISTING SITE PLAN



EXISTING SITE PROFILE

NOT FOR CONSTRUCTION

PAPER SIZE: 11" x 17"

REV. NO.	REVISIONS	DATE	DRAWN	APPR'D
1	ISSUED FOR CLIENT REVIEW	MAR 16, 2015	OL	MC

MOJA
Inženjer

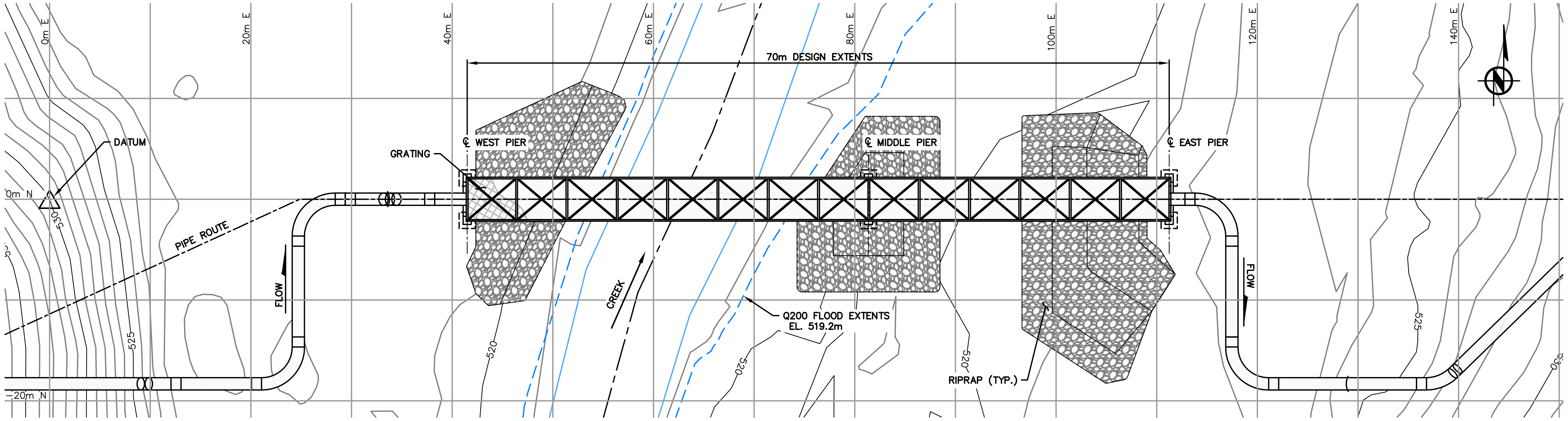
PROJECT
CIVL 7090 - CAPSTONE DESIGN PROJECT

DATE MAR. 16, 2015	NAME OL
SCALE 1:500	CHECK MC

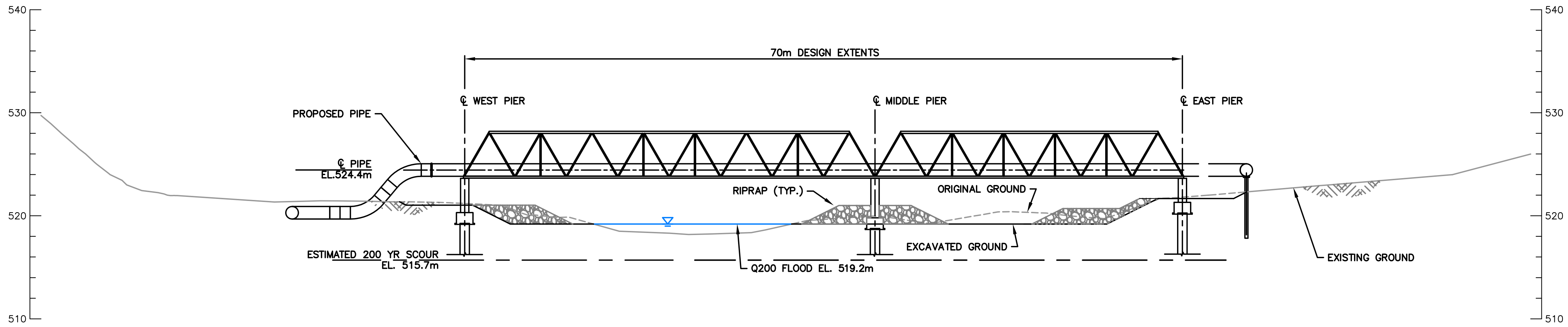
DWG. TITLE
EXISTING SITE PLAN AND PROFILE

DWG. No.
D01

REV.
1



PLAN



PROFILE

NOTES:

1. BOTTOM OF TRUSS STRUCTURE AND FULL GRATING EXTENT NOT SHOWN FOR CLARITY.

NOT FOR CONSTRUCTION

PAPER SIZE: 11" x 17"

REV. NO.	REVISIONS	DATE	DRAWN	APPR'D
1	ISSUED FOR CLIENT REVIEW	MAR 16, 2015	OL	MC

PROJECT
CIVL 7090 - CAPSTONE DESIGN PROJECT

DATE
MAR. 16, 2015

NAME
OL

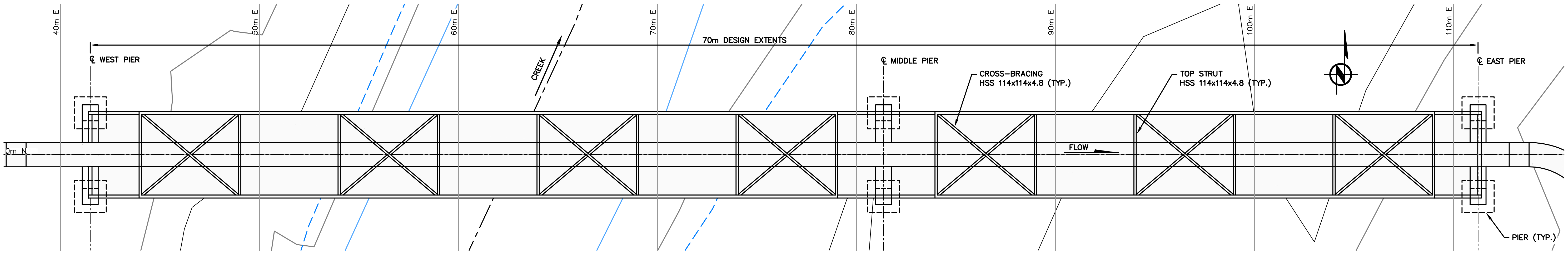
SCALE
1:500

CHECK
MC

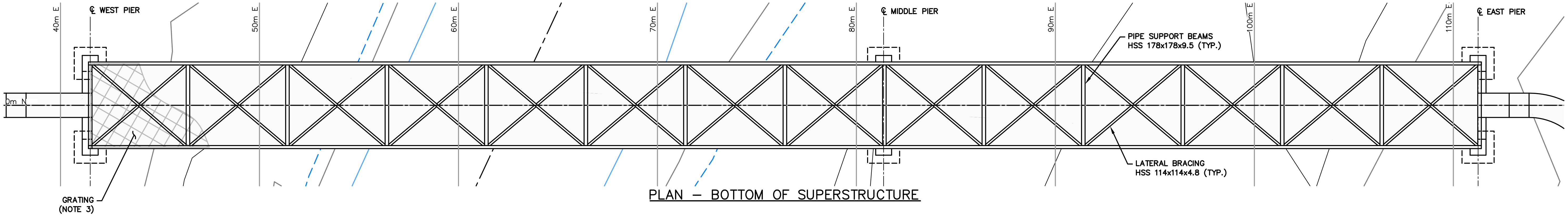
DWG. TITLE
SITE GENERAL ARRANGEMENT

DWG. No.
D02

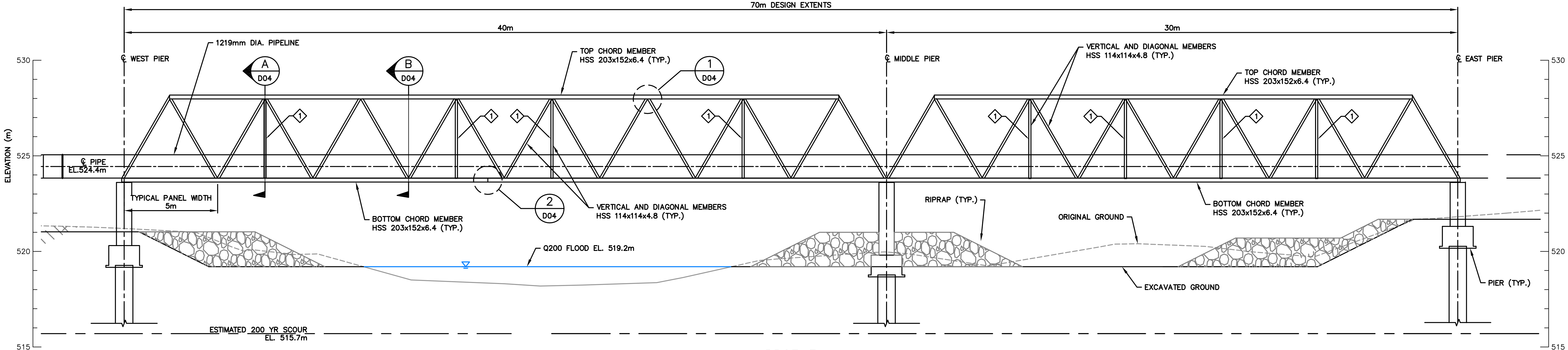
REV.
1



PLAN – TOP OF SUPERSTRUCTURE



PLAN – BOTTOM OF SUPERSTRUCTURE



PROFILE

LEGEND:

◇1 SWAY–BRACING LOCATION

NOTES:

1. TOPOGRAPHY BELOW BRIDGE SUPERSTRUCTURE NOT SHOWN FOR CLARITY.
2. HSS MEMBERS SHALL BE CSA G40.21.
3. GRATING SHALL BE INSTALLED TO COVER 70m DESIGN EXTENTS.

NOT FOR CONSTRUCTION

PAPER SIZE: 11" x 17"

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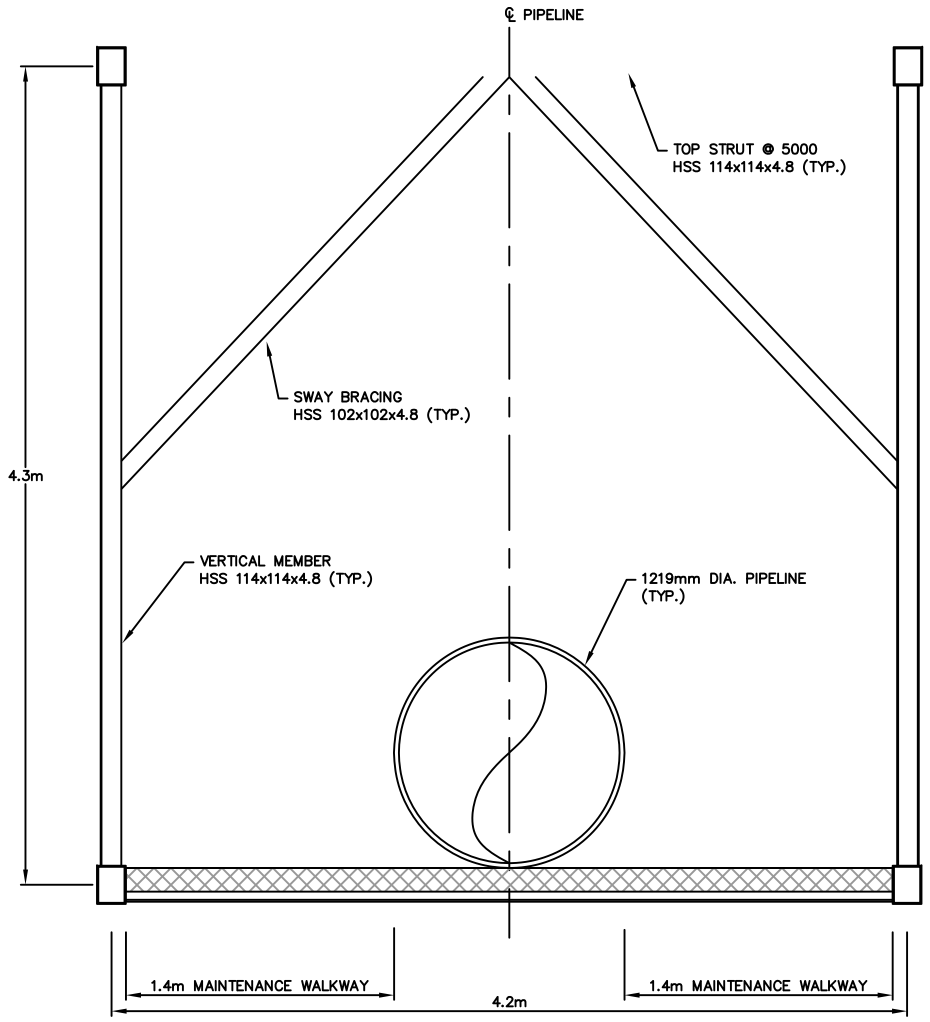
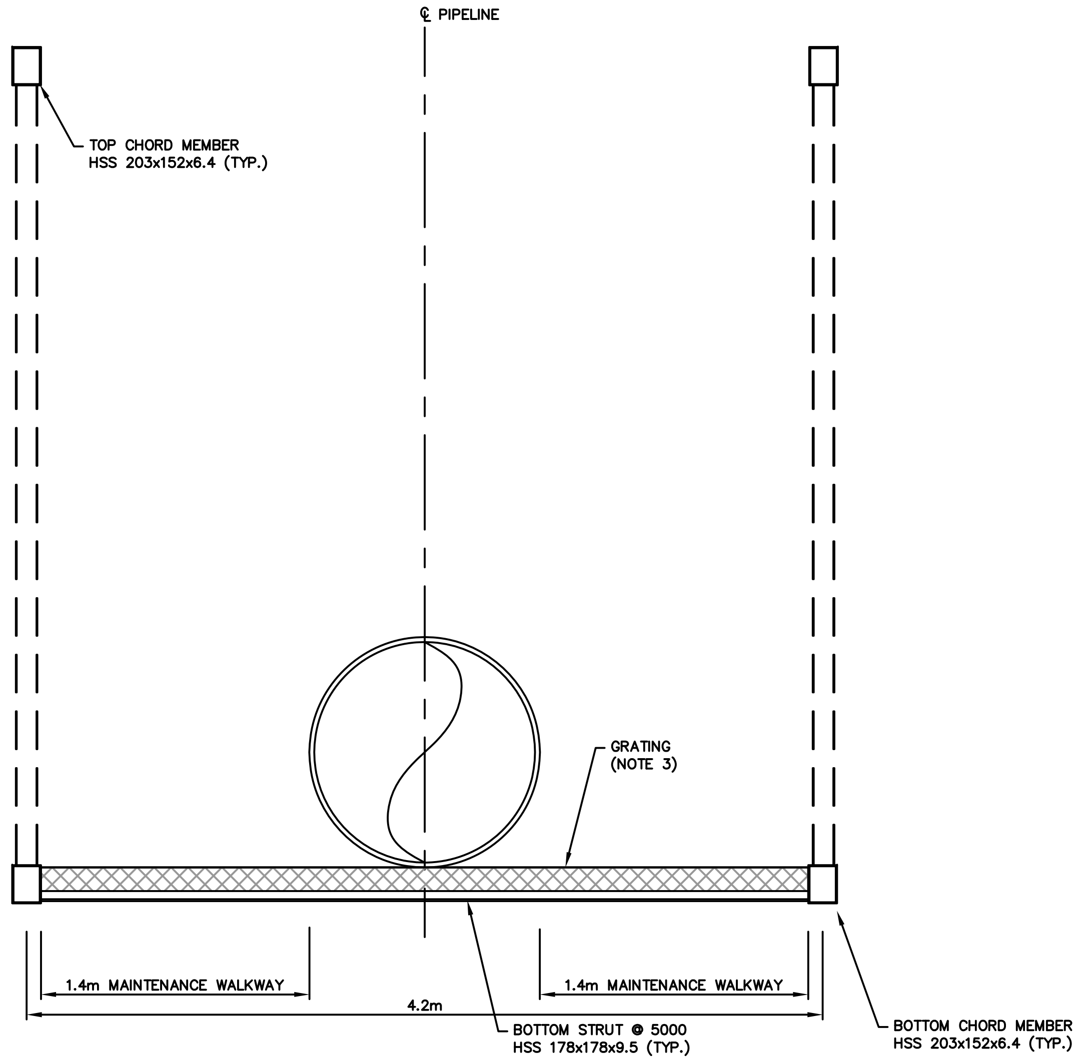
PROJECT
CIVL 7090 - CAPSTONE DESIGN PROJECT

DATE MAR. 16, 2015
NAME OL
SCALE 1:200
CHECK MC
DWG. TITLE
SUPERSTRUCTURE PLAN AND PROFILE
DWG. No. D03
REV. 1



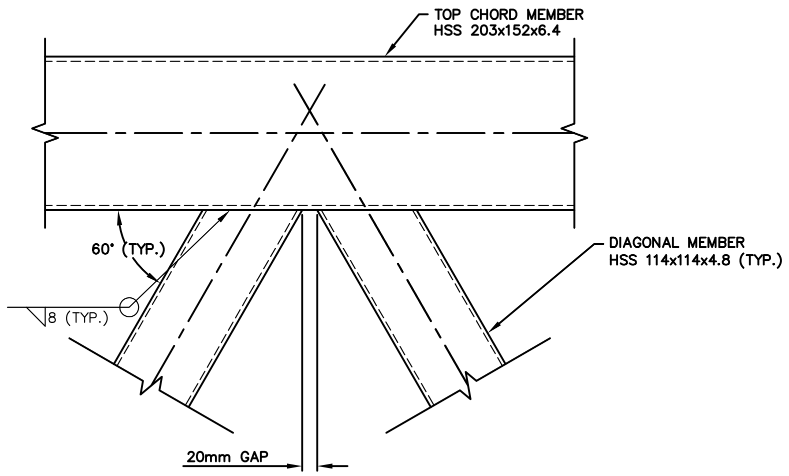
SECTION (A / D03) TYPICAL SUPERSTRUCTURE SECTION THROUGH SWAY BRACING

SCALE 1:40



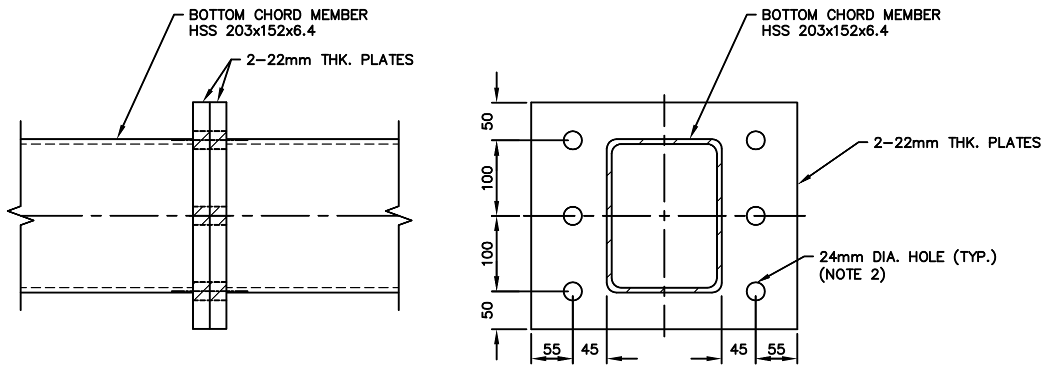
SECTION (B / D03) TYPICAL SUPERSTRUCTURE SECTION THROUGH BOTTOM STRUT

SCALE 1:40



DETAIL (1 / D03) TYPICAL TOP CHORD TO DIAGONAL MEMBER CONNECTION

1:10



SIDE VIEW

CROSS–SECTION

DETAIL (2 / D03) TYPICAL TENSION CHORD SPLICE CONNECTION

1:10

NOTES:

1. HSS MEMBERS SHALL BE CSA G40.21.
2. M22 BOLTS SHALL BE USED FOR THE TENSION CORD SPLICE CONNECTION.
3. GRATING AND GRATING CONNECTIONS TO BE DESIGNED BY OTHERS.

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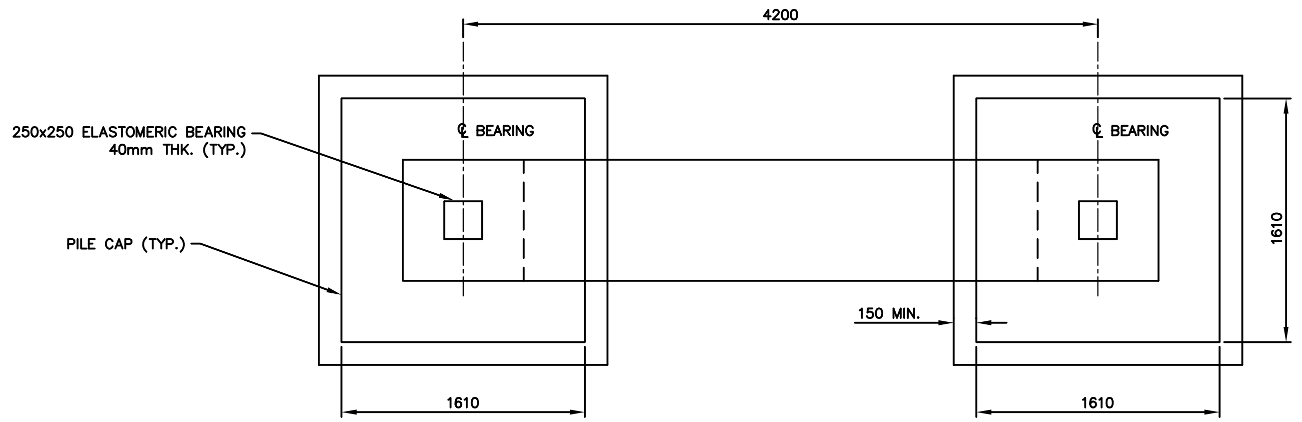
CIVL 7090 - CAPSTONE DESIGN PROJECT

DATE MAR. 16, 2015	NAME OL
SCALE AS SHOWN	CHECK MC
DWG. TITLE SUPERSTRUCTURE SECTIONS AND DETAILS	

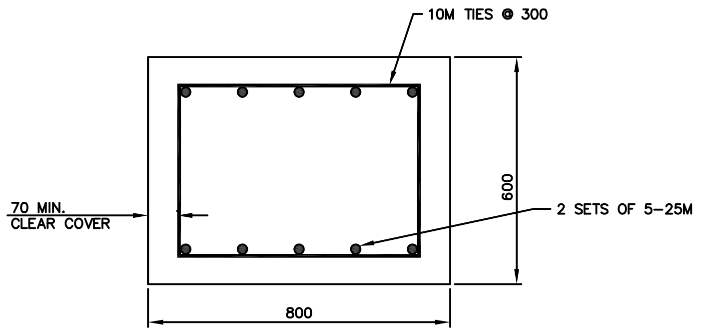
DWG. No.
D04

REV.
1

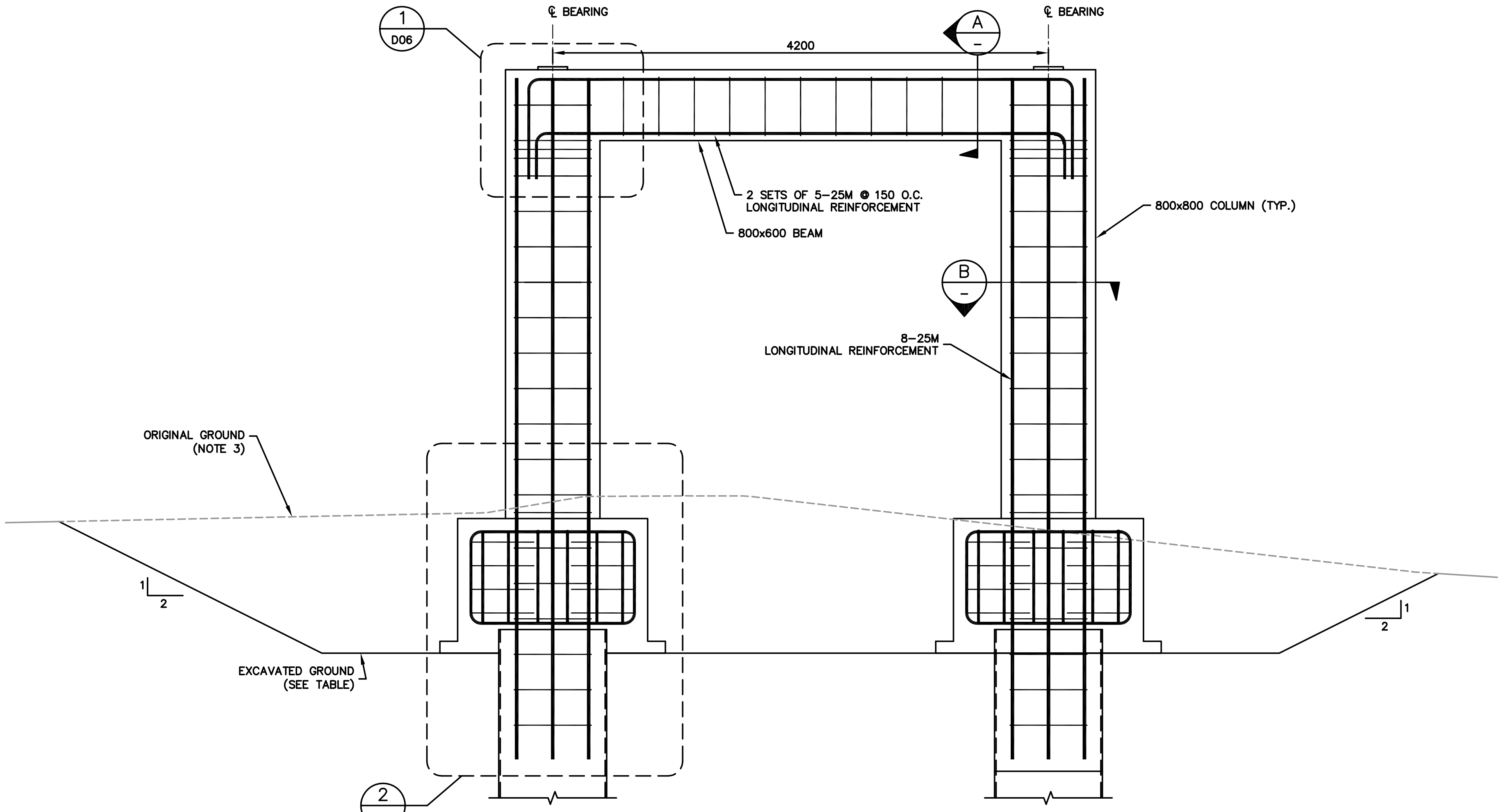


PLAN – PIER
SCALE 1:50

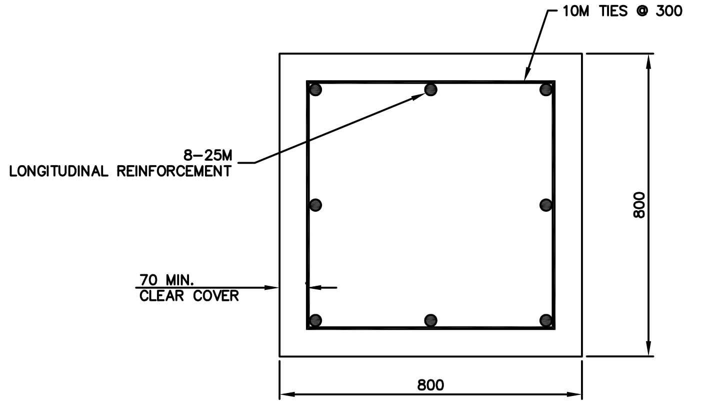
PIERS		
PIER	COLUMN HEIGHT (m)	EXCAVATION ELEVATION (m)
WEST PIER	3.3	519.2
MIDDLE PIER	3.8	518.7
EAST PIER	2.3	520.2



SECTION A TYPICAL BEAM CROSS–SECTION
SCALE 1:20



ELEVATION – PIER
SCALE 1:50



SECTION B TYPICAL COLUMN CROSS–SECTION
SCALE 1:20

NOTES:

1. CONCRETE SHALL HAVE A COMPRESSIVE STRENGTH AT 28 DAYS.
2. REINFORCEMENT SHALL HAVE A YIELD STRENGTH OF 400 MPa.
3. MIDDLE PIER GROUND PROFILE SHOWN.

NOT FOR CONSTRUCTION

PAPER SIZE: 11" x 17"

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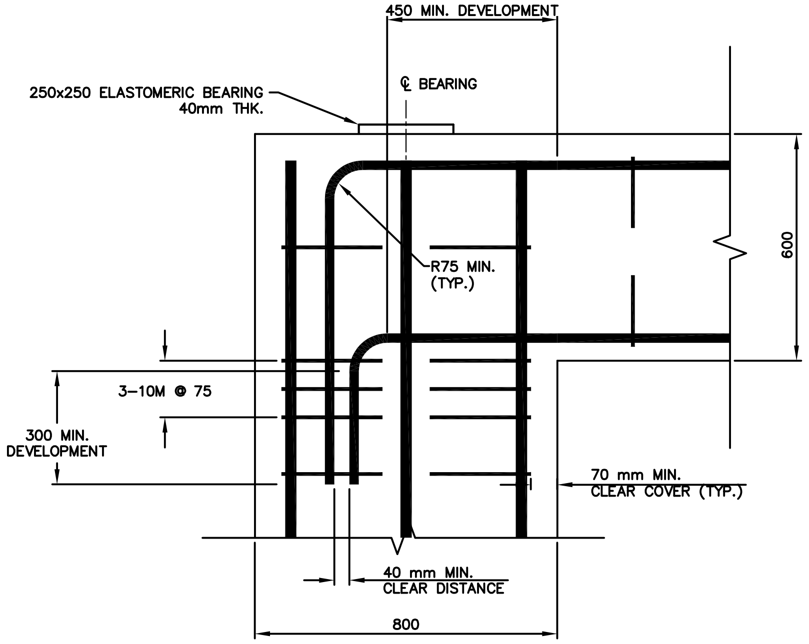
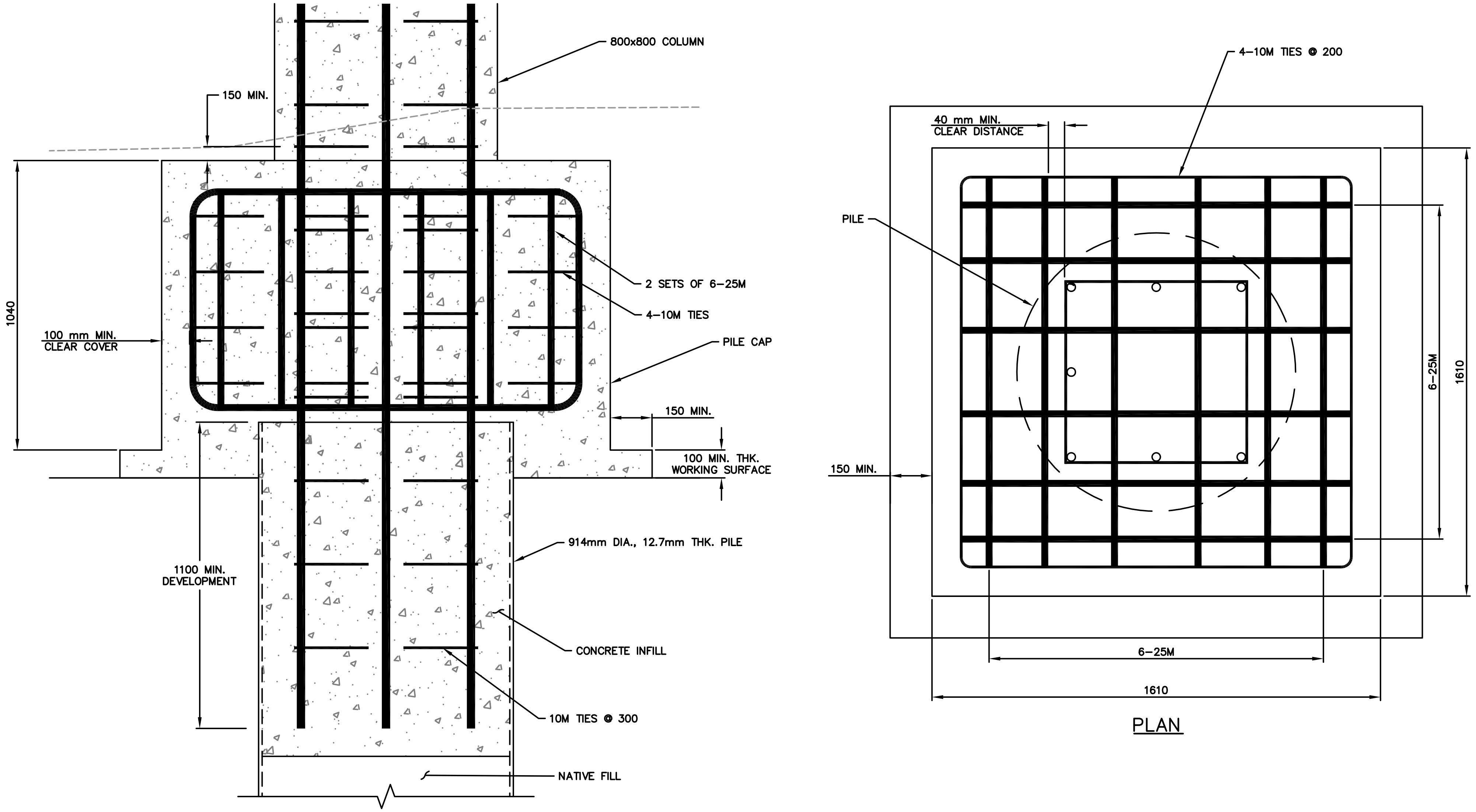
DATE MAR. 16, 2015
NAME OL
SCALE AS SHOWN
CHECK MC
DWG. TITLE
SUBSTRUCTURE ELEVATION AND DETAILS
SHEET 1

DWG. No.
D05
REV.
1



DETAIL 1 D05 TYPICAL BEAM TO COLUMN CONNECTION
1:20



ELEVATION

PLAN

DETAIL 2 D05 PILECAP
1:20

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PAPER SIZE: 11" x 17"

NOTES:

1. CONCRETE SHALL HAVE A COMPRESSIVE STRENGTH AT 28 DAYS.
2. REINFORCEMENT SHALL HAVE A YIELD STRENGTH OF 400 MPa.

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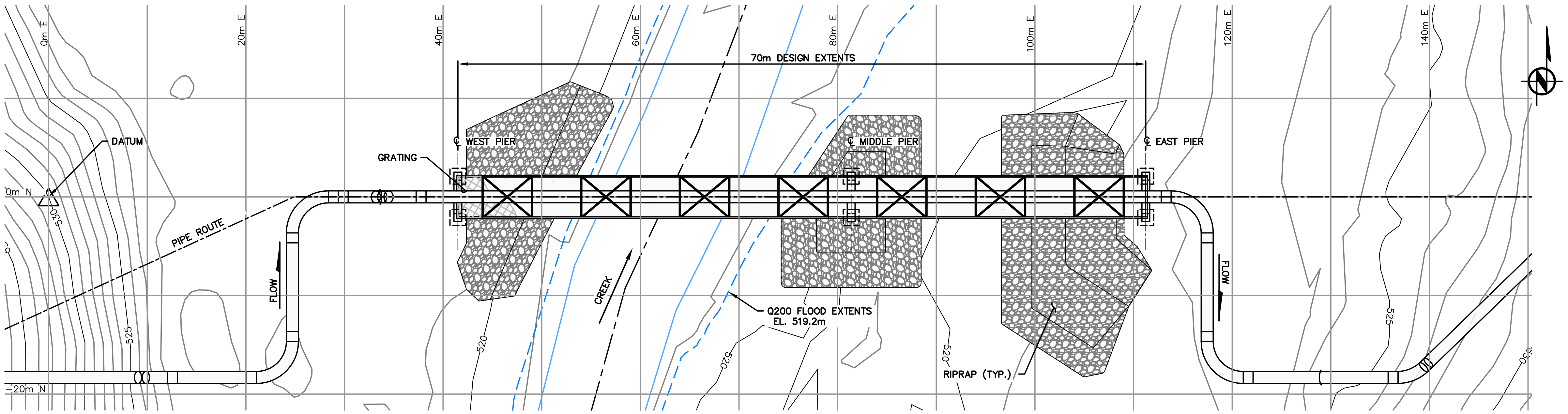
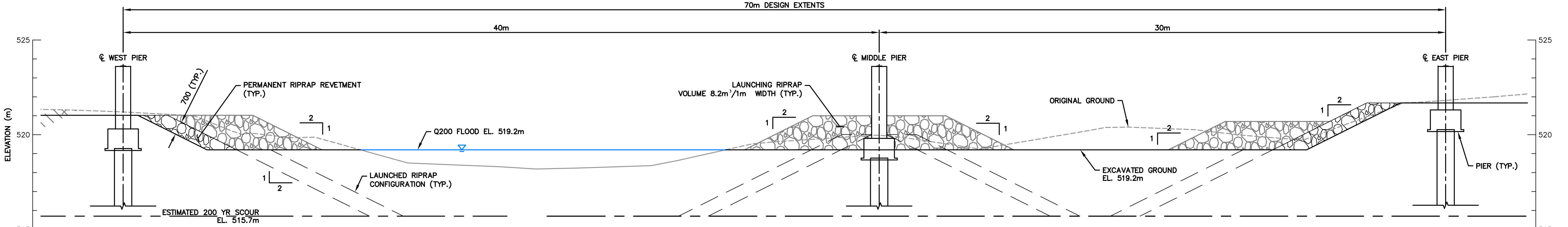
DATE MAR. 16, 2015
NAME OL
SCALE AS SHOWN
CHECK MC
DWG. TITLE
SUBSTRUCTURE ELEVATION AND DETAILS
SHEET 2
DWG. No. D06
REV. 1



PLAN
SCALE 1:500



PROFILE
SCALE 1:200

NOTES:

1. BRIDGE SUPERSTRUCTURE AND PIPELINE NOT SHOWN ON PROFILE FOR CLARITY.

NOT FOR CONSTRUCTION

PAPER SIZE: 11" x 17"

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PROJECT
CIVL 7090 - CAPSTONE DESIGN PROJECT

DATE: MAR. 16, 2015
NAME: OL
SCALE: AS SHOWN
CHECK: MC
DWG. TITLE: EROSION PROTECTION PLAN
DWG. No.: D07
REV.: 1

Appendix M: Construction Drawing Package

MOJA
Inženjer

EXCAVATION WORK POINTS

WORK POINTS	DESCRIPTION	NORTHING (m)	EASTING (m)	EXCAVATION ELEVATION (m)
WP1	BOTTOM OF EXCAVATION FOR FOUNDATION	4.1	39.5	519.2
WP2	BOTTOM OF EXCAVATION FOR FOUNDATION	4.1	43.4	519.2
WP3	BOTTOM OF EXCAVATION FOR FOUNDATION	−4.1	39.5	519.2
WP4	BOTTOM OF EXCAVATION FOR FOUNDATION	−4.1	43.4	519.2
WP5	BOTTOM OF EXCAVATION FOR RIPRAP	4.4	45.9	519.2
WP6	BOTTOM OF EXCAVATION FOR RIPRAP	9.1	56.8	519.2
WP7	BOTTOM OF EXCAVATION FOR RIPRAP	0.4	53.1	519.2
WP8	BOTTOM OF EXCAVATION FOR RIPRAP	−3.0	49.0	519.2
WP9	BOTTOM OF EXCAVATION FOR RIPRAP	−9.6	47.9	519.2
WP10	BOTTOM OF EXCAVATION FOR RIPRAP	−8.3	44.6	519.2
WP11	BOTTOM OF EXCAVATION FOR FOUNDATION	4.1	79.4	518.7
WP12	BOTTOM OF EXCAVATION FOR FOUNDATION	4.1	83.3	518.7
WP13	BOTTOM OF EXCAVATION FOR FOUNDATION	−4.1	79.4	518.7
WP14	BOTTOM OF EXCAVATION FOR FOUNDATION	−4.1	83.3	518.7
WP15	BOTTOM OF EXCAVATION FOR RIPRAP	6.6	80.5	519.2
WP16	BOTTOM OF EXCAVATION FOR CRANE PAD	−5.1	72.3	519.2
WP17	BOTTOM OF EXCAVATION FOR CRANE PAD	−13.2	71.8	519.2
WP18	BOTTOM OF EXCAVATION FOR CRANE PAD	−13.2	82.8	519.2
WP19	BOTTOM OF EXCAVATION FOR RIPRAP	−12.5	98.2	519.2
WP20	BOTTOM OF EXCAVATION FOR RIPRAP	−18.7	107.8	519.2
WP21	BOTTOM OF EXCAVATION FOR RIPRAP	−11.0	110.7	519.2
WP22	BOTTOM OF EXCAVATION FOR RIPRAP	−6.6	103.9	519.2
WP23	BOTTOM OF EXCAVATION FOR RIPRAP	7.7	103.9	519.2
WP24	BOTTOM OF EXCAVATION FOR RIPRAP	7.7	95.8	519.2
WP25	BOTTOM OF EXCAVATION FOR RIPRAP	2.9	89.5	519.2
WP26	BOTTOM OF EXCAVATION FOR FOUNDATION	4.1	109.3	520.2
WP27	BOTTOM OF EXCAVATION FOR FOUNDATION	4.1	113.2	520.2
WP28	BOTTOM OF EXCAVATION FOR FOUNDATION	−4.1	109.3	520.2
WP29	BOTTOM OF EXCAVATION FOR FOUNDATION	−4.1	113.2	520.2

STOCKPILE WORK POINTS

WORK POINTS	NORTHING (m)	EASTING (m)
WP30	32.7	8.5
WP31	9.7	8.5
WP32	9.7	42.4
WP33	32.7	42.4

SILT FENCE WORK POINTS

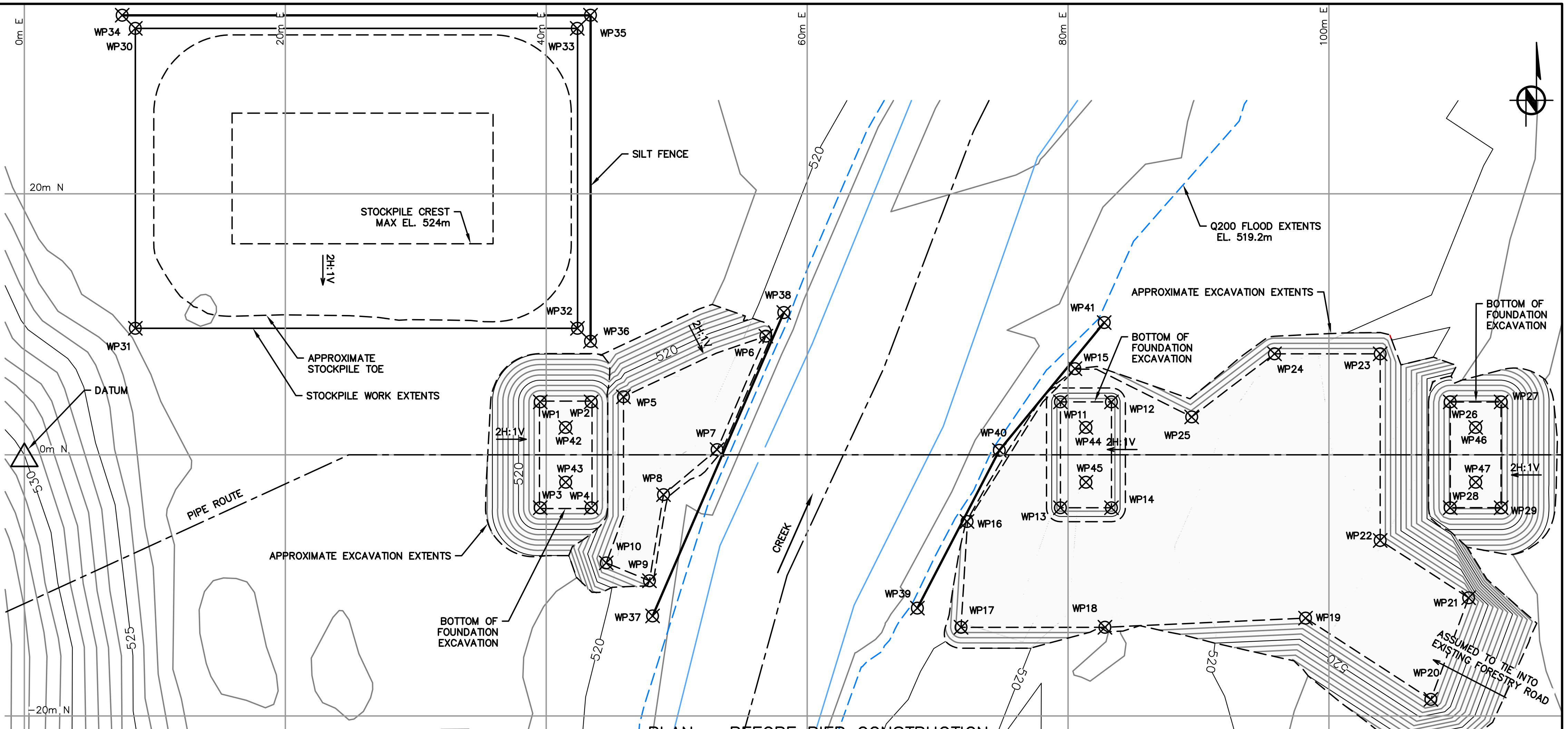
WORK POINTS	DESCRIPTION	NORTHING (m)	EASTING (m)
WP34	STOCKPILE SILT FENCE	33.7	7.5
WP35	STOCKPILE SILT FENCE	33.7	43.4
WP36	STOCKPILE SILT FENCE	8.7	43.4
WP37	WEST EXCAVATION SILT FENCE	−12.3	48.2
WP38	WEST EXCAVATION SILT FENCE	10.9	58.2
WP39	EAST EXCAVATION SILT FENCE	−11.7	68.5
WP40	EAST EXCAVATION SILT FENCE	0.3	74.7
WP41	EAST EXCAVATION SILT FENCE	10.1	82.8

PILES

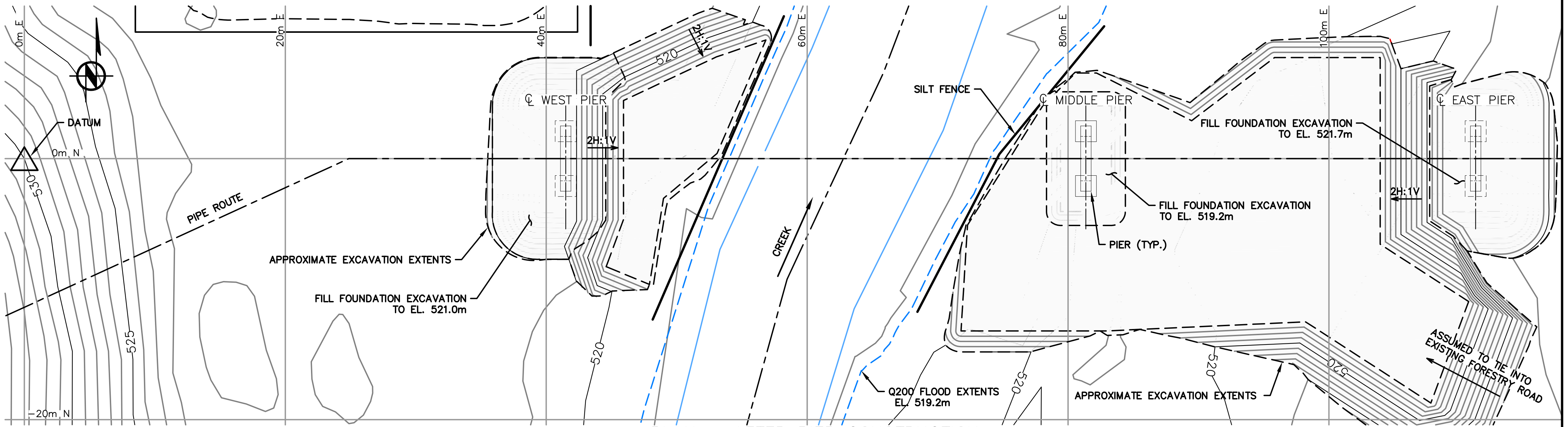
WORK POINTS	DESCRIPTION	NORTHING (m)	EASTING (m)
WP42	NORTH PILE OF WEST PIER	2.1	41.5
WP43	SOUTH PILE OF WEST PIER	−2.1	41.5
WP44	NORTH PILE OF MIDDLE PIER	2.1	81.4
WP45	SOUTH PILE OF MIDDLE PIER	−2.1	81.4
WP46	NORTH PILE OF EAST PIER	2.1	111.2
WP47	SOUTH PILE OF EAST PIER	−2.1	111.2

NOTES:

1. SEE DWG. C02 FOR NOTES.



PLAN – BEFORE PIER CONSTRUCTION



PLAN – AFTER PIER CONSTRUCTION

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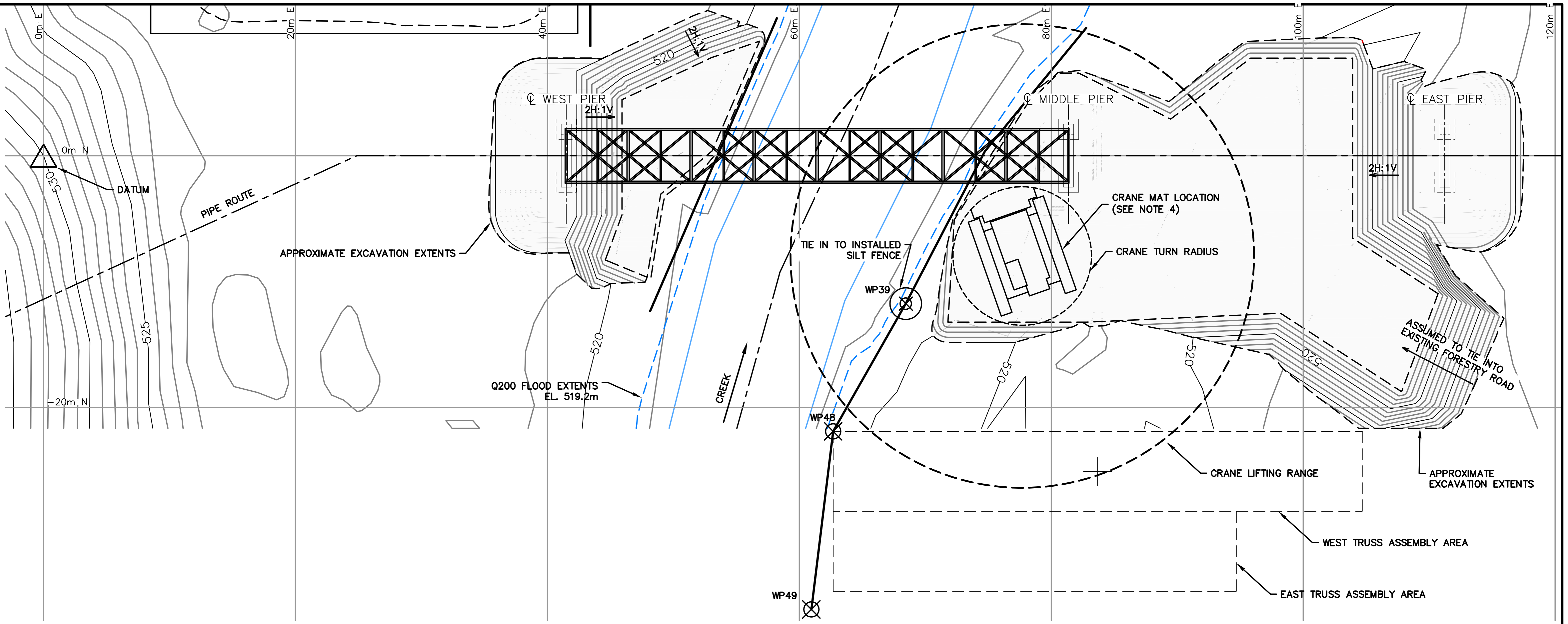
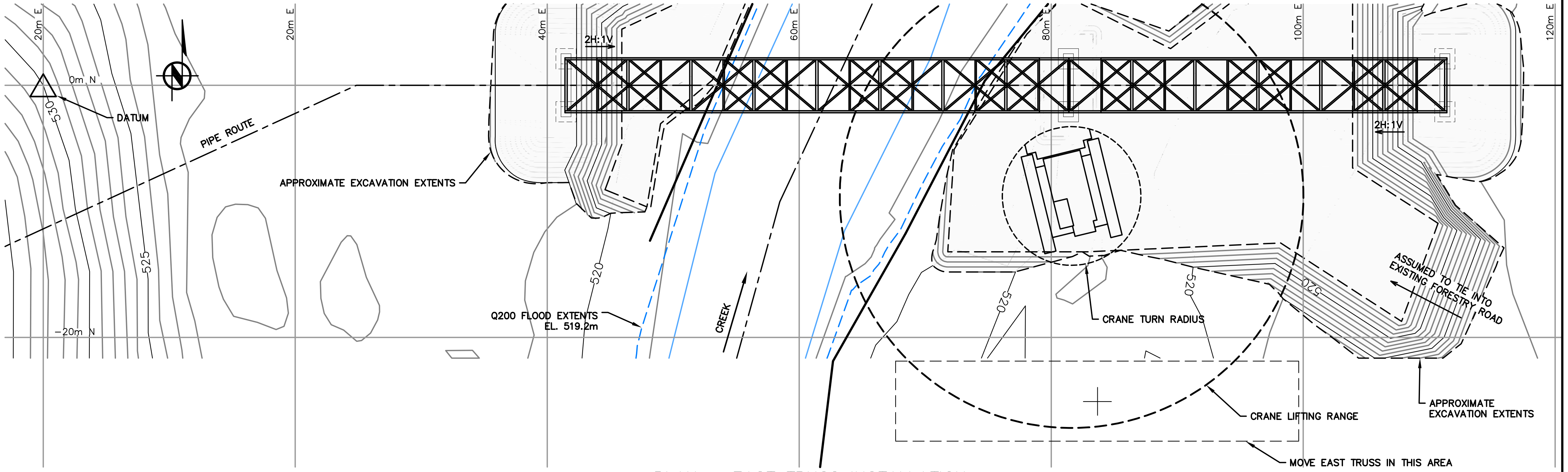
PROJECT
CIVL 7090 - CAPSTONE DESIGN PROJECT

DATE: MAR. 16, 2015
NAME: OL
SCALE: 1:400
CHECK: MC
DWG. TITLE: EXCAVATION AND SEDIMENTATION PLAN
DWG. No.: C01
REV.: 1

SILT FENCE WORK POINTS			
WORK POINTS	DESCRIPTION	NORTHING (m)	EASTING (m)
WP48	TRUSS ASSEMBLY SILT FENCE	−21.9	62.7
WP49	STOCKPILE SILT FENCE	−36.0	61.0



PLAN – WEST TRUSS INSTALLATION



PLAN – EAST TRUSS INSTALLATION

NOTES:

1. ALL SLOPE ARROWS ARE POINTING DOWNHILL.
2. ALL EXCAVATION AND STOCKPILE SLOPES ARE 2H:1V.
3. STOCKPILE AND TRUSS ASSEMBLY AREAS LIE OUTSIDE GIVEN TOPOGRAPHY.
4. PILES CAN BE INSTALLED WITH A 300mm DEVIATION FROM THE INDICATED COORDINATES.
5. PILES SHALL BE SOCKETED 4.2m INTO BEDROCK, OR WILL BE DRIVEN SUCH THAT THE PILE WILL HAVE A MINIMUM EMBEDMENT OF 18m BELOW ELEVATION 515.7m.
6. CRANE MATS SHALL BE INSTALLED ON CRANE LIFTING LOCATIONS WITH A MINIMUM SIZE OF 4572mm x 1524mm PER TRACK.
7. HAY BALES OR OTHER EQUIVALENT FLOW–CONTROL MEASURES SHALL BE USED IF CHANNELIZATION OCCORS ON CONSTRUCTION WORK AREA.
8. THE CONTRACTOR SHALL NOT ALLOW SEDIMENTATION TO ACCUMULATE MORE THAN 40% OF SILT FENCE HEIGHT.

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PROJECT
CIVL 7090 - CAPSTONE DESIGN PROJECT

DATE
MAR. 16, 2015

NAME
OL

SCALE
1:400

CHECK
MC

DWG. TITLE
BRIDGE SUPERSTRUCTURE LIFTING PLAN

DWG. No.
C02

REV.
1