Maple Engineering Services

Design of a retrofit structural system allowing the removal of the central support column of the pedestrian walkway spanning Cogswell Street

as part of

the Cogswell Redevelopment Program in Halifax, Nova Scotia

Design Report

Volume I of II

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Acknowledgements

The firm, Maple Engineering Services, would like to extend their sincerest gratitude towards our personal industry advisor, Raphael Lévesque, CPI., Prof. Denis Mitchell, Prof. Colin Rogers, Prof. William E. Taylor for taking time to provide technical advice, recommendations and their personal expertise throughout the completion of this project.

Further, the firm would like to extend this gratitude toward all other industry advisors and McGill Civil Engineering faculty involved with the grading, evaluation and feedback of our design project.

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1.0 Introduction

Maple Engineering Services has been asked by the City of Halifax to complete a segment of an urban redevelopment project named the Cogswell Redevelopment Project located around the Cogswell Interchange in Halifax, Nova Scotia. The aim of the redevelopment project is to increase green space and increase the capacity for pedestrians and cyclists in downtown Halifax.

The segment of the project that has been given to Maple Engineering Services involves the deconstruction of an on-ramp and the re-structuring of a pedestrian bridge linking a commercial building on the north side and a parking garage on the south side. The pedestrian bridge is made of concrete and is supported by a central concrete column. The re-structuring of the bridge involves deconstructing of the central concrete column. The bridge has glass windows that must be preserved. The bridge spans 35.6m and is 10.3m above street level. A functioning water pipe is locating on top of the bridge that must remain operational at all time. Disruption to pedestrian and street traffic must be minimised.

The design of the new bridge support system has proved to be a challenge for the engineering team at Maple Engineering Services. The team utilised key knowledge and techniques from previous design courses at McGill University, as well as expert advice and guidance from our industry advisor and McGill professors.

2.0 Project Objectives

Maple Engineering Services has been given requirements and constraints regarding the execution of the project. These factors shaped the design process of the bridge leading up to the final design. It also affected the construction plan. The following were the key considerations:

- Ensure the safety of the structure throughout deconstruction and construction while adhering to the NBCC ULS and SLS design
- Ensure the availability of construction materials used in the project
- Minimize deflection of the glass exterior & water pipes located on the bridge roof
- Minimize interruption of pedestrian and road traffic on & under the bridge
- Maximize the public use space under the bridge and along Cogswell Street, including the deconstruction of the Cogswell Street on-ramp

- Minimise the use of construction materials for economic and sustainable benefits

The bridge design had various physical constraints. The ends of the bridge were fixed to adjacent buildings. Property line constraints made it tricky to choose a location for the support columns while still maximizing green space below the bridge. One significant physical consideration was the mechanism of deconstructing the central column after having placed a temporary support system for the bridge. Certain construction methods must be used to carefully deconstruct the central column given the small space left around it.

Deflection was a critical limitation due to glass exterior and water pipe which are very sensitive to minute deflections. Our objective in the design was to limit the maximum deflection to 10mm to protect the glass exterior and water pipe from any damage.

3.0 Codes, Regulations, References, Software

3.1 Applicable Design Codes, Regulations and References

As the Cogswell Redevelopment Program involves both steel and concrete design, many resources were consulted throughout the duration of the project. In Canada, designers must satisfy the requirements of the *National Building Code of Canada* (NBCC 2015). This national code together with the commentaries provide general requirements for structural design and refers to the appropriate Canadian Standard Association (CSA) materials standards (i.e., concrete, steel, masonry, wood). The *CSA S16 Design Standard (2014)* was used for the design of steel structures throughout the entire project. *Handbook of Steel Construction 11th edition* was used as the fundamental reference for the steel design check and the material selection. Further, *CSA A23.3-14 Design Standard* and *Concrete Design Handbook 3rd edition* was used in the design of the concrete supporting columns and check feasibility of the concrete design.

Additionally, there were many external references that were contacted throughout the duration of the project. McGill Faculty of Civil Engineering and WPS. They were really helpful that provided professional suggestions and guidelines for the final design based on real-life examples and scenarios and contributed significantly to the constructability and the feasibility of the final design.

3.2 Design Software

Through the whole duration of the design report, much of the structural response analysis and the truss selection was completed with the assistance of software. The advancement of software not only enables us to quickly solve the difficulties during the duration of building design but also enables visualization of the structure by constructing its 3D model. Moreover, with the assistance of software, people can effectively locate problems and solve them. MAPLE will use the following software for the structural analysis, models, and design calculation:

- SAP 2000 for Structural Model and Structural Response Analysis
- SketchUp 2019 for 3D modelling of the structure and detail connections
- AutoCAD for bridge footing, column, bearing cap design
- Vray 4.0 for 3D model rendering and visualization
- Microsoft Excel for load calculations, connection design calculations

3.2.1 Structural Analysis and Design

SAP2000 provides a powerful function called Auto-selection for us to choose the proper size of trusses under different load combinations. After defining all loads and load combinations, SAP2000 automatically calculated the worst-case scenario that the preliminary design relied on. As all design provisions were defined in the setting, the Auto-selection function was able to output the minimum required truss sizes for each component of the bridge through tremendous calculations based on the worst-case scenario. The preliminary choice of trusses from software analysis provided a cornerstone for the ultimate design. Further, SAP2000 can be utilized to verify manual calculations, as well as complete analyses that are not feasible with manual calculations.

3.2.2 Structural 3D Modeling

Building 3D models had effectively facilitated the design process as most design details can be visualized, so that it provided us a clearer imagination that what would the final design look like. Especially when designing truss connections as our design project involved 11 different types of connections. Creating multiple 3D connection models facilitated the process of identifying design specifications, such as whether to use bolt connections or welded connections. SketchUp was used to construct both the truss cage design and connection design. Vray was used to render the final appearance of the structure as the construction was completed.

3.2.3 Design Calculations

All calculations were completed via Microsoft Excel. Excel enables us to create formulas that can be utilized in complex systematic problems. All related calculations will be displayed and described in different design sections of this report.

4.0 Work Breakdown Structure & Project Timeline

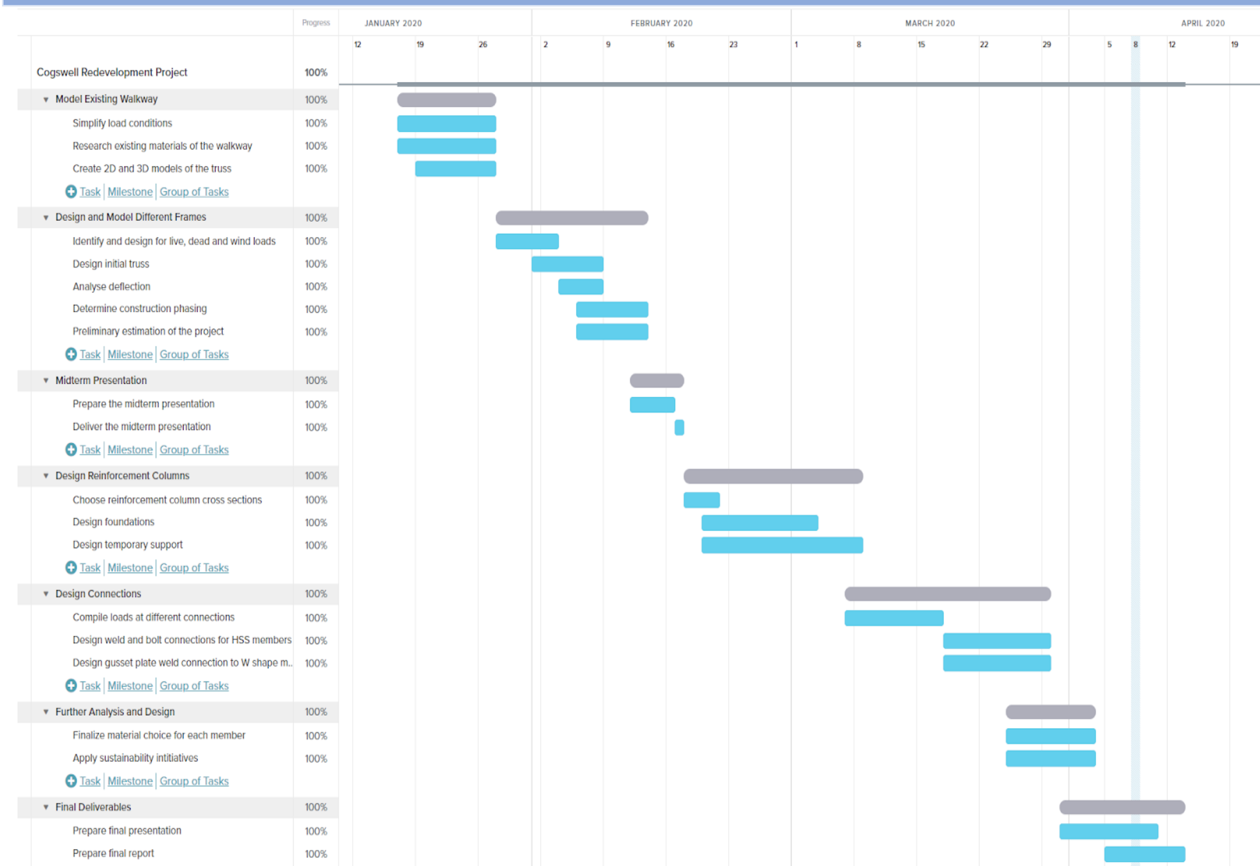

Figure 4.1: Project Timeline

As seen in Figure 1, the project had several main components. These main components were split into subtasks and divided among group members. Initially, most of the work had to be split sequentially and not everyone was able to work on their parts all at once. However, after the midterm presentation, all group members were able to work on different tasks at the same time.

The tasks were divided by roles. As project manager, Philippe ensured all members were focused on their tasks at hand and all the tasks were able to work in sync. He was also responsible for setting up group meetings with our advisor. Philippe oversaw all load calculations and directly designed all support connections. As Geotechnical Lead, Moied designed the shallow foundations and concrete column and cap. He also performed initial seismic load calculations that were verified in SAP2000. Adamo was responsible for the structural component of the project. He oversaw all calculations and the SAP2000 model and analysis, as well as designed the construction sequencing and procedure. As drafting technologist, Loren designed the entire model in SAP2000. Adamo and Loren collaborated to ensure the truss dimensions and member sizes were sufficient, as well as worked on connection design. Elia took the lead with regards to the connection design, estimation and sustainability concerns.

5.0 Modelling Existing Structure

5.1 Existing Physical Context & Given Plans

The given plans are found in the annex section of this report.

5.2 Existing Loads Acting on Central Support Column

It is assumed that the abutments on either end of the structure do not carry any loads. They simply provide lateral stability, and all loads are assumed to be acting on the central column. This

5.2.1 Dead Loads

The existing structure is made up of 8 double-tee spans. Spans are simply gravity-supported, with one-end being supported by an 8” bearing (building-side) and the other end being supported by the same bearing width on a central structural support.

The 8 double-tee spans are 5DT18 spans cross--section is shown in Figure 5.1. As noted in the figure, double tee units at roof share a similar cross-section, except no 2” composite concrete surface topping. Furthermore, double-tee sections are placed side-by-side, spanning a width of 10’. If we consider two spans, side-by-side, as one span, there are 4 spans; 2 top spans and 2 bottom spans.

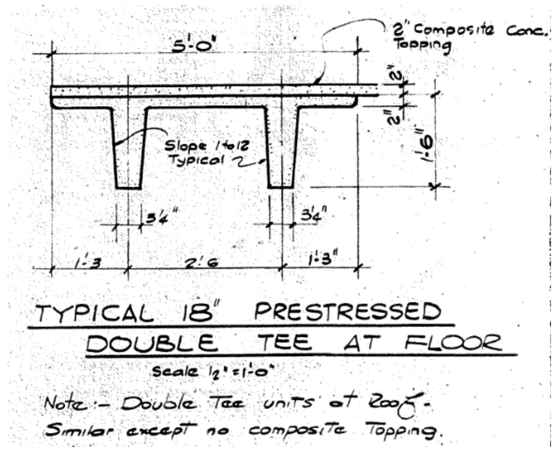

Figure 5.1: Double Tee Cross Section

Looking East, towards the structure, the top left spans have a length of 18 237 mm, the top right spans have a length of 17 539 mm, the bottom left spans have a length of 18 237 mm, and the bottom right spans have a length of 17 412 mm.

In calculating the dead load imposed by the structure, several assumptions are made:

- Top span cross slope is insignificant to load results
- Glass cladding does not influence load distribution on outside web
- Cladding, pipes & accessory self-weight is included in plan-prescribed "superimposed dead load"
- Steel reinforcement and prestressing tendons not considered in weight calculations

The unfactored dead loads, carried by the central support columns, are shown in Figure 5.2. Sample calculations are found in the annex section of this report.

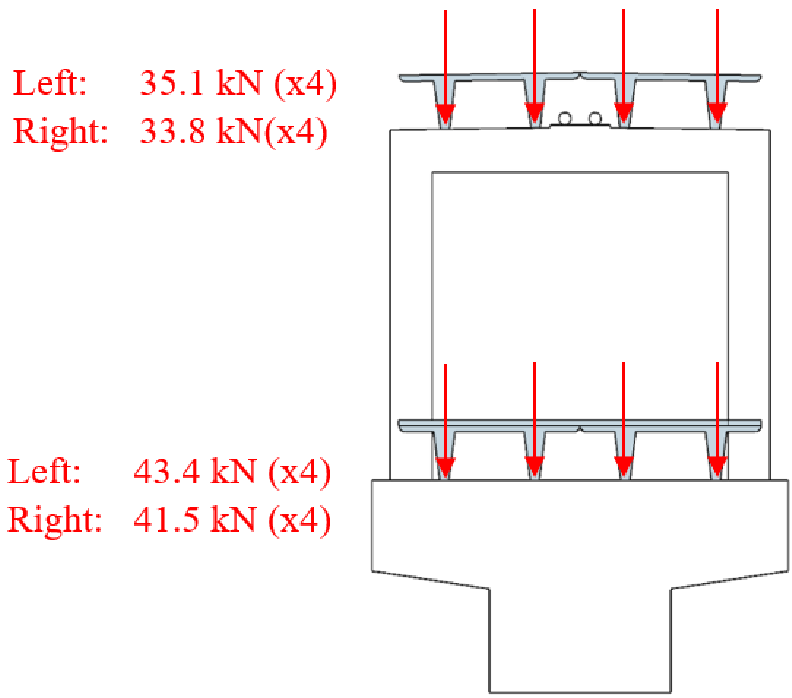

Figure 5.2: Dead Load Distribution on Central Support Column

5.2.2 Live Loads

The only applied live load on the bridge is a pedestrian load. This is done in accordance with Clause 3.8.9 of the CSA S6-14. The pedestrian load is assumed to be a uniform load across the entire span and results in a load of 339 kN. Calculation details can be found in the annex.

5.2.3 Wind Loads

Vertical Wind loads are calculated similarly in the CSA S6-14 as the NBCC 2015. The reference pressure (q) is taken from Appendix C Table C-2 of the NBCC 2015, which is equal to 0.59 kPa. The total vertical pressure applied either upwards or downwards on the structure, with calculation details found in the annex.

Multiplying this value by the same tributary area as for ice loads, the resultant point load is 180 kN.

5.2.4 Ice Accretion Loads

Ice accretion loads are to be calculated in accordance with Clause 3.12.6 of the CSA S6-14. During winter months, ice accumulates on the top flat roof of the existing structure. Assuming the unit weight of ice (γ) is 9.81 kN/m^3, an ice thickness (t) of 66 mm from Figure A3.1.4, the resulting ice point load (i) at the center is 71.7 kN. Calculation details can be found in the annex. Multiplying this value by the same tributary area as for ice loads, the resultant point load is 180 kN.

Lateral wind loads are calculated identically to vertical wind loads, but the vertical coefficient (C_v) is replaced with a horizontal coefficient (C_h) that is equal to 2.0 and the tributary area value above, with the resultant point load equal to 395 kN. Calculation details can be found in the annex.

5.2.5 Snow Loads

Snow loads are not considered in the CSA S6-14 because it is assumed that bridges and overpasses are cleared shortly after snowfall. However, this is not possible in this case. Therefore, snow loads are calculated according to the NBCC 2015. The importance factor is assumed to be 1.0, with the rest of the parameters obtained from Clause 4.1.6 and Appendix C Table C-2. The resultant point load is 235 kN. Snow load calculation details can be found in the annex.

5.2.6 Seismic Loads

The procedure in CSA s6.1-14 Canadian highway design chapter 4 was followed. Project located in Halifax-Nova Scotia with a site class C and PGA of 0.1. The Periodic Time Ta is then calculated (Ta=3.8s) which produces a uniform seismic load of 2.53Kn/m and a seismic point load of 95.3Kn. reduction factors in 4.17 table are used to accurately calculate the seismic load for each component of the structure.

Furthermore, The CSA highway code states that the final calculated seismic load is the combined total horizontal and transverse loads. Lastly a high importance/risk factor was chosen (1.5) to increase safety of the design

Refer to annex for detailed calculation.

5.3 Load Combinations

Load Combinations are done in accordance with CSA S6-14 Clause 3.5 As seen in Figure 1, all load combinations are depicted. There are several different forces that can be neglected, including: secondary prestress effects, loads due to earth pressure and hydrostatic pressure, wind load on traffic, differential settlement and/or movement of the foundation, stream pressure and ice forces or debris torrents and collision load arising from highway vehicles or vessels. These load combinations are used in the SAP2000 model after the loads have been calculated. ULS combinations 6 and 8 can be completely neglected due to neglecting the loads previously mentioned.

Table 5.1: Load Combination Summary

	Permanent loads			Transitory loads					Exceptional loads			
Loads	*D*	*E*	*P*	*L**	*K*	*W*	*V*	*S*	*EQ*	*F*	*A*	*H*
Fatigue limit state												
FLS Combination 1	1.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	Table 3.2	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	Table 3.2	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	Table 3.2	1.00	0.45§	0.45	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.40§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.75§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

6.0 Modelling and Analysis

6.1 Truss Geometry

For the Cogswell Redevelopment Program, the main objective of the truss configuration design is to hold the entire concrete bridge without exceeding the maximum allowable deflection. The reason to choose steel truss as a part of our retrofitting project is that the steel truss cage is one of the most effective and economical retrofitting strategies selected by developers and local infrastructure managers for the bridge span of more than 100 feet. Due to the consideration of the constructability that the project consists of multiple construction stages, we developed two designs corresponding to a temporary configuration and a permanent configuration respectively.

For two steel truss configurations, the software SketchUp was used to construct the 3D model and SAP2000 was used to design the specific size of each steel member as well as check the shear forces along each truss, to check the moments as well as check the deflection under different load combinations. The section 9.1 describes the first step of design process which mainly defined the dimension of truss system without specifying the exact size for each member. The section 9.2 further explores the design details based on the outputs from Auto-Selection function in SAP2000. Finally, the section 9.3 shows the structural response and results of force analysis for the updated structure.

6.1.1 Temporary Configuration

Figure 6.1 shows the general views of the temporary configuration. The span length for each section is 2.946m. A total of 10 spans, having a total length of 29.463m, were designed as the cage structure to hold the original bridge. The height of the cage structure is equivalent to the length of the span, therefore, creating a square section with a bracing angled at 45 degrees, which is relatively easier to analyze in the software. The width of the cage structure is 3.81m. For the two spans that are adjacent to the center of the cage, the single-bracing trusses are replaced by the cross-bracing trusses to improve the structure resistance in both lateral and vertical directions. The design of underlying diagonal trusses also provides significant resistance to lateral deflection. Moreover, the role of two temporary bearing beams that are 1.2192m apart located at the center of the bridge is to be used as a supporting point to jack the existing bridge off the central support that is planned to be removed. The original bridge consists of two long concrete slab that spanned at 18.237m and 17.412m, the removal of central RC column will make these two concrete slabs act as two cantilevers. To ensure that the formation of two cantilevers does not make the deflection exceed the allowable limit, two sets of temporary bearing beams must be installed to bear the weight.

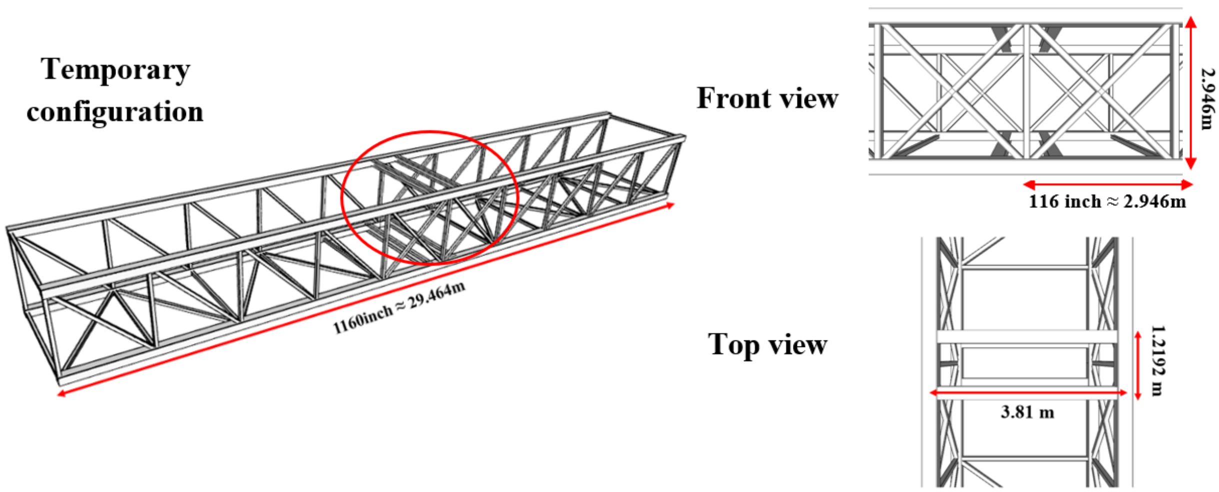

Figure 6.1: Dimension of the temporary configuration of the truss cage

6.1.2 Permanent Configuration

Figure 6.2 shows the general views of the permanent configuration. The permanent structure generally has the same configuration as the temporary structure excepting frames in the central area. Instead of two temporary bearing beams, one central bearing beam with a larger size was installed on the cage structure.

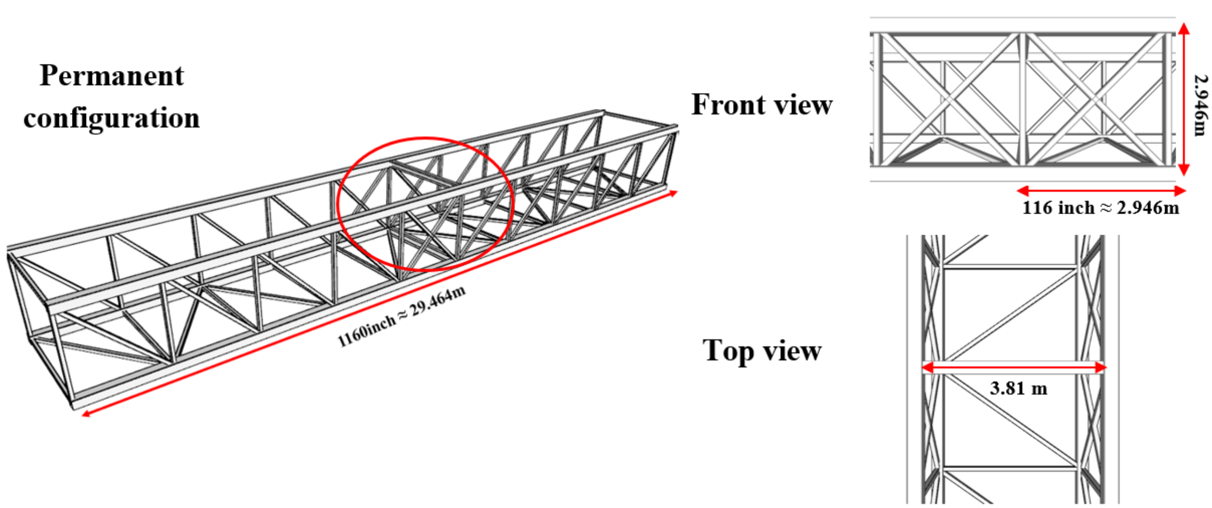

Figure 6.2: Dimension of the permanent configuration of the truss cage

6.2 Member Size Specifications

After defining all load cases and load combinations, the structural analysis indicated that the worst-case scenario is under the load combination of 1.1D+1.5L+0.45W. As mentioned in section 7.1. The cage structure mainly consists of three sections, which are W-sections, WT sections, and HSS sections. Although SAP2000 can design the minimum required size for each individual truss, the real-life construction doesn't allow us to have plenty of distinct steel trusses for this project as the manufacturing difficulties and production costs would be unbelievably great if we insist doing so. Therefore, the cage structure was decomposed into seven groups. Central bearing trusses and horizontal trusses were made of W shape steel, and cross-bracing trusses were made of WT shape steel. The rest of the four groups were made of HSS shapes. In SAP2000, the setting for the Auto-Selection list ranges from W100x19 to W360x990 for the W section, L25x25x3 to L200x200x30 for the WT section, and HSS 32x32x3 to HSS 305x305x13 for the HSS section. The software optimizes the structural behavior and prevents potential failures by selecting suitable truss sizes that can resist the worst-case scenario. The default design code is based on CSA S16-14. At the end of the design, trusses within the same group are supposed to have a uniform size. Table 6.1 shows the output of the auto-selection function in SAP2000.

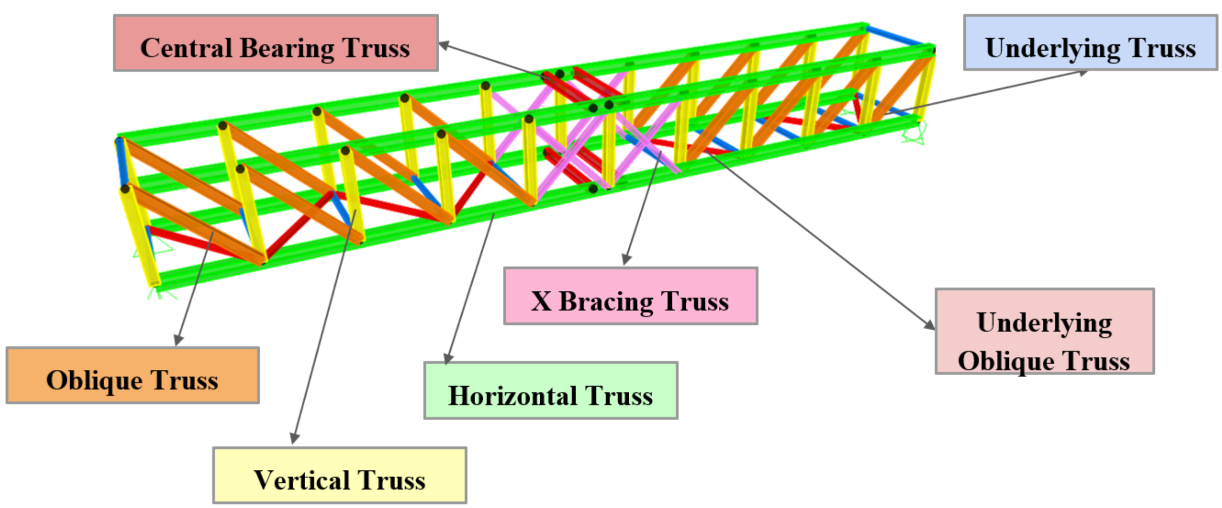

Figure 6.3: Grouping methodology for permanent& temporary configurations of the truss cage

Table 6.1: Specified truss sized for each design group

Group name	Truss size
Central Bearing Truss	W360x51
Horizontal Truss	W360x216
X Bracing Truss	WT180
Underlying Truss	HS152
Oblique Truss	HS305D
Vertical Truss	HS305V
Underlying Oblique Truss	HS152D

6.3 Material Specifications

6.3.1 Material selection for steel

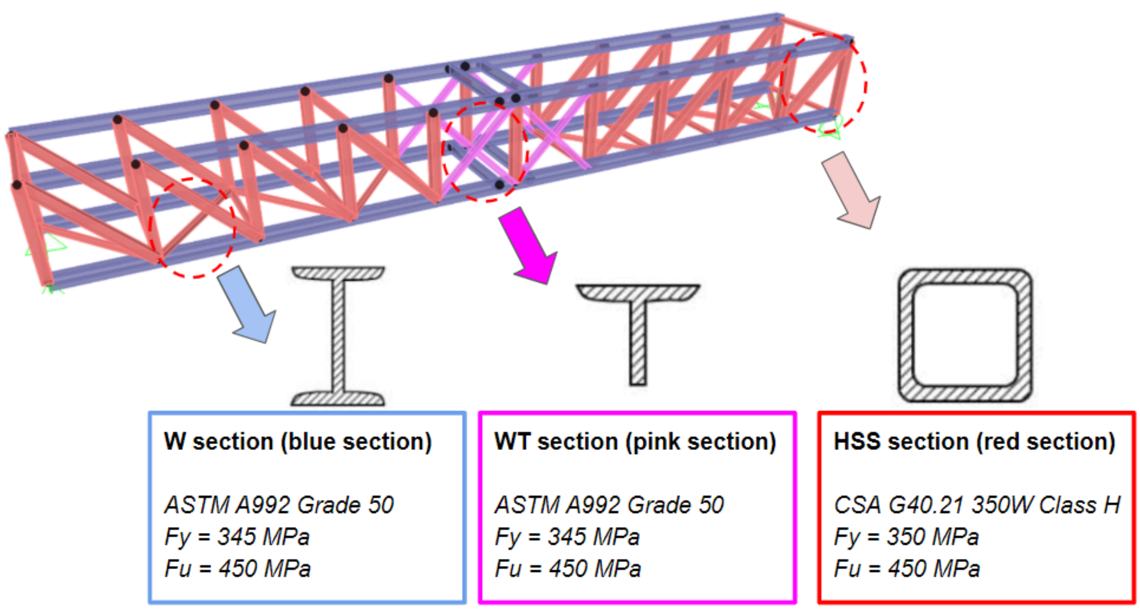

Figure 6.4: Steel materials used in each part of the truss cage

The main structural components of the truss cage are to be constructed using steels. The steel used throughout the cage will be of normal density that is 7850 kg/m^3. According to Figure 6.4, the cage structure mainly consists of three sections, which are W-sections, WT sections, and HSS sections. Due to different manufacturing methods, there is a distinction in materials that can be used to produce such sections. For the Cogswell Redevelopment Program, ASTM A992

Grade 50 is utilized for all W sections and WT sections, because, compared to the traditional ASTM A572 Grade 50 steel, A992 has a better material definition. It has an upper limit on the yield strength of 345Mpa, a minimum tensile strength of 450Mpa a specified maximum yield-to-tensile ratio of 0.85 and a specified maximum carbon equivalent of 0.47%. These qualities enable structural engineers to quickly have a systematic analysis of the structure composed of ASTM A992 steels. Further, as with A572 Grade 50, A992 presents some attractive cost benefits over A572 as there are often no grades extras for Grade 50 steel and no overall cost penalties, hence, most buildings will have a greater economy by using A992. From the constructability concept, the composition of A992 lends itself to improved weldability, adding to its utility for all types of construction projects. This steel is used to fabricate structural steel components with high corrosion resistance, essential for the bridge projects that are exposed to harsh weather conditions. The detail material information is shown in Table 6.2.

For all HSS sections in the project, G40.21 350W Class H Steel (Table 6.4) is utilized because of two reasons. First, 350W Class H Steel has a higher axial capacity in accordance with CSA S16-14. Clause 13.3.1 of CSA S16-14 allows for n=2.24 when specifying CSA Class H material. This will lead to higher axial capacities than those calculated for CSA Class C or ASTM A500. Take 8x8x1/4 and 12x8x1/4 as two examples (Table 6.3), The CISC Handbook of Steel Construction (10 edition) indicates that the axial strength of Class H is 36% more than that of A500 and 23% more than that of Class C for 8x8x1/4, and it is 41% more than the axial strength of A500 and 20% more than that of Class C for 12x8x1/4. This higher axial capacity could lead to using smaller sections and subsequent mass savings over Class C and A500, as much as 20%. Even though there is a cost premium for Class H over Class C and A500, the potential for mass savings will translate to project cost savings.

Furthermore, there is a certain restriction for the available dimensions for the HSS G40.21 350 W Class H Steel in the manufacturing process. For Square tubes, the dimensions should fall into a range that from 0.5"x 0.5" to 22"x22". For Rectangular tubes, the dimensions should fall into a range that from 1.5"x 0.75" to 24"x12". The available thickness of the tube ranges from 0.10" to 0.75" depending on the size required. Thus, we carefully filtered out infeasible truss sizes when creating the Auto-Selection list in SAP2000 to meet the manufacturing requirement.

Table 6.2: Material Specification for ASTM 992A Steel

Specification	ASTM 992A
Yield Strength	345-450 MPa
Tensile Strength	450 MPa
Elongation %in 2 in	21%
Carbon	0.23% Maximum
Nickel	0.45% Maximum
Manganese	0.50%-1.50%
Phosphorous	0.035% Maximum
Sulphur	0.045% Maximum
Silicon	0.40% Maximum
Copper	0.6% Maximum
Grain Refining Elements	0.10% Maximum

Table 6.3: Comparison of 350W Class H Steel to Class C and A500 Steel.

8 X 8 X ¼	A500, GRADE C	CSA CLASS C	CSA CLASS H
KL= 6m (19'-8')	829 Kn (186 kips)	920 Kn (207 kips)	1130 Kn (254 kips)

* Source: CISC Handbook of Steel Constructions, Tenth Edition (2014).

Class H:
36% more capacity than A500
23% more capacity than Class C

12 X 8 X ¼	A500, GRADE C	CSA CLASS C	CSA CLASS H
KL= 8m (26'-3")	675 Kn (152 kips)	780 Kn (178 kips)	951 Kn (214 kips)

Class H:
41% more capacity than A500
20% more capacity than Class C

Table 6.4: Material Specification for G40.21 350W Class H Steel

Specification	G40.21
Strength Levels	50W/ 350W
Yield Strength	350 MPa Minimum
Tensile Strength	450-620 MPa
Elongation %in 2 in	22 Minimum
Chemistry Levels	50W/ 350W
Carbon	0.23% Maximum
Manganese	0.50%-1.50%
Phosphorous	0.040% Maximum

Sulphur	0.050% Maximum
Silicon	0.40% Maximum
Grain Refining Elements	0.10% Maximum

6.3.2 Material Selection for concrete

Two supporting columns for the truss cage is composed of reinforced concrete. The concrete used throughout the building will be of normal density and will a design unit weight of 24 kN/m^3. Reinforced concrete elements will follow the requirements for obtaining a 2-hour fire-resistance rating as specified in the NBCC 2015. Since the supporting columns are continuously exposed to the outside and the concrete is in an unsaturated condition exposed to freezing and thawing, but not to chlorides. The class of exposure should be regarded as Type F-2. According to the CSA A23.1 Requirements for C, F, N, R, S and A classes of exposure, the maximum water-to-cementing materials ratio for F-2 is 0.55, the minimum specified compressive strength is 25 MPa and age at test is 28 days. Additionally, CSA A23.1 indicates that the range of air content for the Type F-2 concrete should be 5%- 8%, 4%- 7% and 3%- 6% for 10 mm, 14-20 mm and 28-40 mm aggregate sizes, respectively. In our design project, 20 mm aggregate size was utilized, hence, the air content should be control within a range of 4%-7%.

6.4 Failure Check

The load combination used for the analysis is 1.1D+1.5L+0.45W, the self-weight of the structure as well as a superimposed dead load acting on the cage structure is included in the dead load.

6.4.1 Axial force resistance

From the axial force diagram for both the temporary and permanent configuration, there is a negligible difference between these two diagrams. For the temporary configuration (Figure 6.5), the maximum tensile force of 1448.973 kN occurred at the bottom truss that is parallel to the bridge, the maximum compressive force of 1392.311 kN occurred at the top horizontal truss.

For the permanent configuration (Figure 6.6), the maximum tensile force of 1446.532 kN occurred at the top truss, and the maximum compressive force of 1390.911 kN occurred at the top horizontal truss. Although SAP2000 has already defined the size of the truss without violating any design book criteria from CSA S16-14, manual calculations have been made to confirm the software setting is accurate. According to the gross cross-section yielding formula

$$T_r = \emptyset A_g F_y$$

The calculated maximum allowable tensile force Tr of the gross cross-section is 8538 kN for both configurations as they both have the same horizontal trusses of W360x216. There are still other formulas from the design provision to check the design feasibility, more details will be discussed in the connection design section.

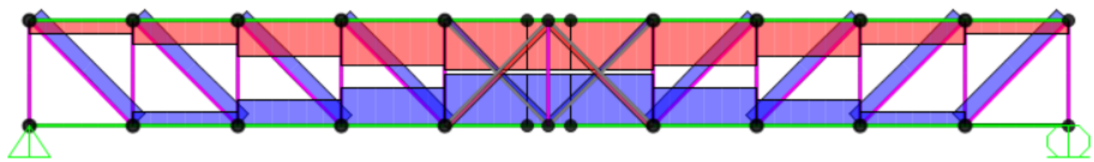

Figure 6.5: Axial force diagram for the temporary configuration under 1.1D+1.5L+0.45W

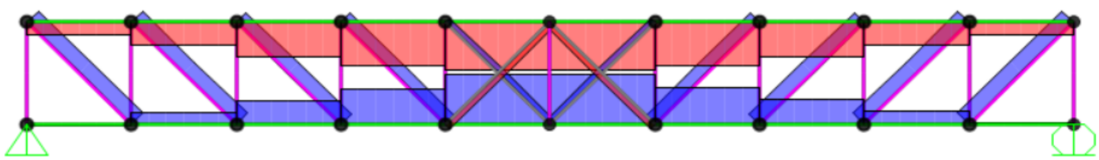

Figure 6.6: Axial force diagram for the permanent configuration under 1.1D+1.5L+0.45W

6.4.2 Shear force resistance

From the shear force analysis in SAP 2000 (Figure 6.7), there was a shear force of 135.379kN acting on the bottom horizontal truss W360x216 and a shear force of 158.412kN acting on the central bearing truss W360x51. According to the formula,

$$V_r = \emptyset A_w F_s$$

where $F_s = 0.66F_y$, and A_w =d*w for rolled shapes. The maximum allowable shear forces for W360x216 and W360x51 are 1333.028kN and 526.711kN respectively, which satisfied the design provision for shear forces. Additionally, the design also satisfies:

$$\frac{h}{w} \leq 439 * sqrt(\frac{k_v}{F_y})$$

$$W360X216:\ 21.734 \leq 54.617\ \ (ok)$$

$$W360X50:\ \ 49.17 \leq 54.617\ \ (ok)$$

Where k_v =5.34 for shear buckling coefficient for an unstiffened web. The shear forces for other members can be neglected as the value of their shear forces ranges from 0.916kN to 6.2kN

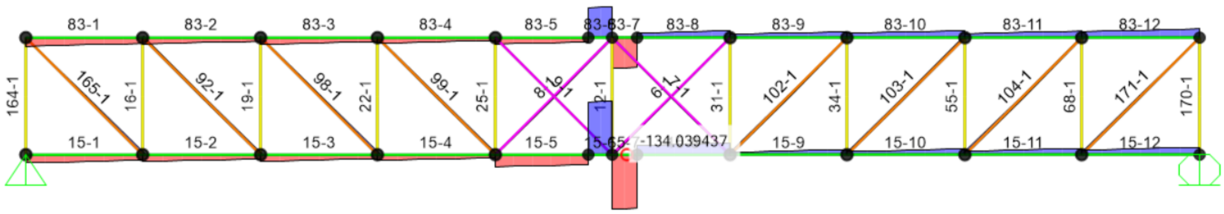

Figure 6.7: Axial force diagram for the permanent configuration under 1.1D+1.5L+0.45W

6.4.3 Moment resistance

The analysis of moments has also been done with SAP2000 (Figure 6.8). Excepting the central bearing truss W360x51, the rest of trusses has a negligible bending moment that ranges from 1.1789 kNm to 12.3326 kN-m. Therefore, the manual calculation for the moment check has been applied to the central bearing truss. To calculate the allowable moments of the truss W360x51, the first step is to define the section classification. Refer to the CSA S16-14 Clauses 11.2 and Clauses 27.7.2.7 (Table 6.5), W360x51 falls into the Class 1 category as following:

$$\frac{b_{el}}{t} \leq \frac{145}{sqrt(F_y)})\ and\ \ \frac{h}{w} \leq \frac{1700}{sqrt(F_y)}$$

$$7.3707 \leq 7.8065\ \ and\ \ 45.97\ \ \leq 91.52\ (ok)$$

Where b_{el} =85.5mm, t = 11.6mm, h =332.8mm and w =7.24mm

The next step is to calculate M_r based on the formula

$$M_r\ =\ \emptyset Z_x F_y$$

The maximum allowable moments of the truss W360x51 is 277.898 kNm, which is greater than the actual M_f of 172.945 kNm. Therefore, the central bearing beam is capable of bearing the existing loads and moments.

Table 6.5: Maximum width-to-thickness ratios: Elements in flexural compression

Description of Element	Maximum Width-to-Thickness Ratios		
	Class 1	Class 2	Class 3
Flanges of I-sections or T-sections in bending about the major axis; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact with an axis of symmetry in the plane of loading	$\frac{b_{el}}{t} \leq \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}$

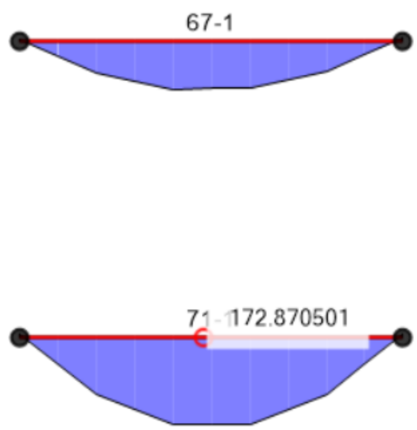

Figure 6.8: The maximum bending moment along central bearing truss W360x51

6.4.4 Deflection

To determine the load limit based on deflection, it is necessary to check the deflection limit for both steel cage structure and the concrete bridge under 1.1D+1.5L+0.45W. For the steel structure check, the maximum deflection occurred at the midspan of the truss W360x216 which is 19.9mm. According to the CSA S16-14 provision, it belongs to the category that simple span members of floors supporting construction susceptible to cracking. Therefore. The maximum allowable deflection is 1/360 of the entire span length of the W360x216. The length of the span is 29.46m. In the case of the steel truss, the maximum deflection is:

$$\frac{\ell_n}{360} = \frac{29460}{360} = 81.83m$$

Hence, the steel member W360x216 satisfies the deflection check. Then, the bridge is composed of two concrete slabs that have a span length of 18.237m and 17.412m respectively. The initial assumption for the cage structure and the concrete bridge is that they both have the same deflection as they are deflected coincidently. A long-term deflection was check was carried out on the bridge slab. From the Table 9.3 (maximum permissible computed deflection) of the Concrete Design Handbook, the deflection of slabs supporting non-structural elements likely to be damaged by large deflection should not exceed $\ell_n/480$. ℓ_n was taken as the most critical span length of the slab, which is 18.237m.

In the case of the typical slab the maximum deflection is:

$$\frac{\ell_n}{480} = \frac{18.237}{480} = 37.99\ mm$$

According to the software output, the maximum long-term deflection was 19.9mm. The deflection requirements are satisfied.

,

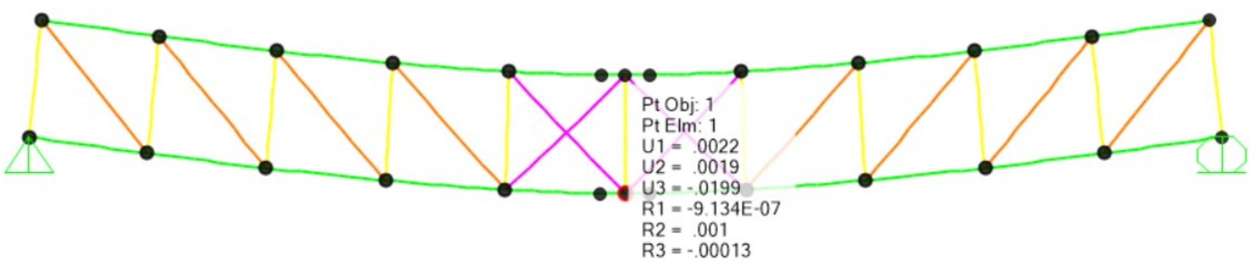

Figure 6.9: The maximum deflection for the cage structure

7.0 Detailed Design

7.1 Truss Connection Design

A detailed connection design follows the SAP 2000 frame analysis. The SAP 2000 frame analysis is used to determine the appropriate member sizes based on the assignment of members into different groups, henceforth referred to as “member families.”

Table 7.1: Member families

Member Group	Member Size Designation
Long bending members	W 360x216
Oblique HSS	HSS 305x305x13
Vertical HSS	HSS 305x305x13
Horizontal HSS Bracing	HSS 152x152x8
Oblique HSS Bracing	HSS 152x152x8
Central Cross Bracing	WT 180x
Central Bearing Beam	W 360x51

The colours in Table 7.1 refer to the colours assigned to the member groups in the SAP 2000 models.

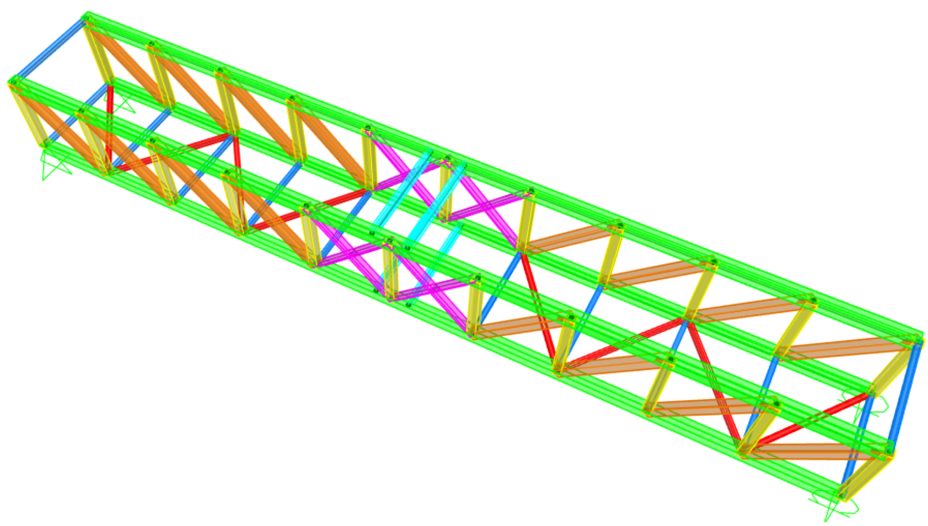

Figure 7.1: SAP2000 model, temporary configuration

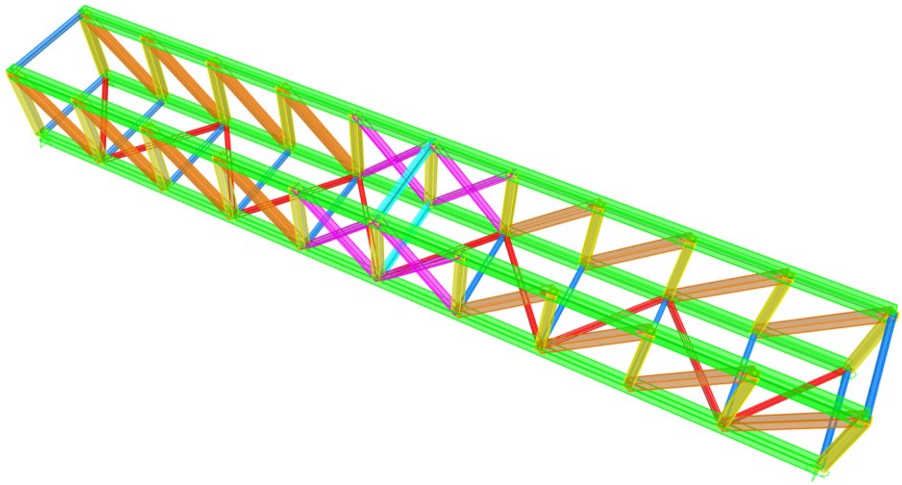

Figure 7.2: SAP2000 model, permanent configuration

An assumption is made with regards to the permanent configuration of the frame to facilitate the connection design. It is assumed that the temporary configuration of the frame brings about the worst loading in all members of the structure. Therefore, the analysis results of the temporary configuration from SAP 2000 is used to design the connections.

A further assumption is made prior to the detailed connection design. It is assumed that all members are pin-connected, and therefore shear, torsion and moment effects are ignored for the design of the connections. This assumption is made to simplify the connection design and is reinforced by a visual analysis of the exported results which show relatively low shear, torsion and moment forces for all members except for W-shape members. The central bearing W-shape members are left out of this assumption, however only shear is evaluated for the connection of the central bearing W-shape.

7.1.1 Methodology Used

The frame connections are designed to resist the worst loads in all members, which is assumed to come from the temporary configuration of the frame.

First, the results of the analysis data are exported to a Microsoft Excel file. SAP 2000 exports analysis results in a tabular format. Each row in the table corresponds to a point on an individual frame element and the loads acting at that point. The table is exported with the following headers:

Table 7.2: SAP2000 analysis result table headers

Frame	Station	Output Case	P	V2	V3	T	M2	M3	Frame Element
number	m		KN	KN	KN	KN-m	KN-m	KN-m	

Frame: the number assigned to the individual frame element in the model

Station (m): the point along the frame element at which the loads are acting

Output Case: the load case evaluated (following the analysis assumption, the case of 1.1D+1.5L+0.45W is used)

P (kN): The axial force acting on the individual frame element at the station along the element

V2 (kN): The shear force in the X-Y plane acting on the individual frame element at the station along the element

V3 (kN): The shear force in the X-Z plane acting on the individual frame element at the station along the element

T (kN-m): The torsional force acting on the individual frame element at the station along the element

M2 (kN-m): The moment in the X-Y plane acting on the individual frame element at the station along the element

M3 (kN-m): The moment in the X-Z plane acting on the individual frame element at the station along the element

A summary of the analysis results is found in the annex section of this report.

7.1.2 Advice Received

Following the export of the analysis data, two design methods are evaluated prior to carrying forward with the design process:

Dennis D'Aronco, ing. advises Maple Engineering Services to use the CISC publication entitled *Hollow Structural Section – Connections and Trusses* by Jeffrey A. Packer and J.E. (Ted) Henderson (1997) as a guideline for the connection of our HSS members. The publication is based on CSA Standard S16.1-94 and is a compendium of design information from 1997 on the topic of HSS connections and trusses. Although the publication is in-depth, it focuses on the **connection between trusses composed of HSS chord and web members**. The recommendation was valid at the time, as it followed the mid-term presentation where it was still assumed that our group would be using HSS sections throughout. However, the final design involves the use of W sections for the chords of the truss. Therefore, the publication by Packer & Henderson is not used for the design of the connections.

Raphael Levesque, ing. advises Maple Engineering Services to work to our strengths & simplify the connection design to **basic welded & bolted gusset plate connections**. Levesque proposes that endplates are welded to all truss members, and gusset plates are welded to the flange & web of the W section truss chords. A bolted connection connects both welded plates to resist the axial force in the truss members.

Maple Engineering Services selects Raphael Levesque's proposed design as it offers a much simpler approach to the design of the connections and uses our knowledge and materials (course packs, design guides, design codes) our group members have accrued in our courses at McGill, notably the content from:

CIVE 317: Structural Engineering 1 (Prof. Sherif Kamel)
CIVE 318: Structural Engineering 2 (Prof. Colin Rogers)
CIVE 462: Design of Steel Structures (Prof. Colin Rogers)
CISC/ICCA "Handbook of Steel Construction, Eleventh Edition." (2017)

7.1.3 Conceptual Connection Modeling & Design

Following the decision to design our connections as basic welded & bolted gusset plate connections, conceptual drawings are drafted using Google SketchUp to aid visually in the design steps and provide rough sketches & details of the future connections. An example of such is shown in Figure 7.3. The other conceptual connection models done in SketchUp are found in the annex section of this report.

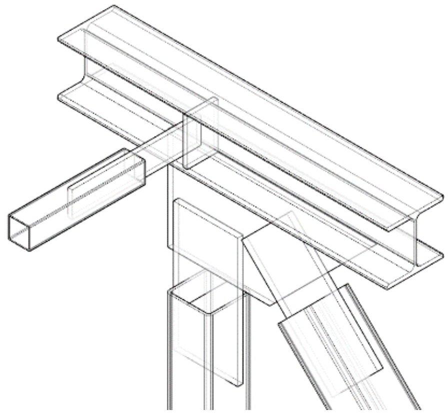

Figure 7.3: Example of 3D Google SketchUp connection model

HSS sections providing lateral bracing (in the X-Z plane) are connected by a member-welded endplate. The endplate is connected by a bolted connection to a stiffener-type plate, welded to the inside of the truss chord web. The HSS sections which make up the vertical truss (in the X-Y plane) are also connected by a member-welded endplate. The endplate is connected by a bolted connection to a gusset plate which is welded to the outside of the flange of the W shape. This is the way all truss members are connected throughout the frame.

Using Microsoft Excel's VLOOKUP, MATCHIF, IF, ABS functions, the frame element load data was treated to output the maximum (positive and negative) loads for each member family. The summary of the data treatment is found in Table 7.3.

Table 7.3: Summary of maximum forces in all truss members

	Member Family	***P*** *(kN)*	***V2*** *(kN)*	***V3*** *(kN)*	***T*** *(kN-m)*	***M2*** *(kN-m)*	***M3*** *(kN-m)*
Max +ve	W 360x216	*1449.0*	*135.4*	*8.9*	*6.6*	*15.7*	*72.1*
Max -ve		*-1390.9*					
Max +ve	Oblique HSS 305x305x13	*466.0*	*6.2*	*0.7*	*0.6*	*2.4*	*10.3*
Max -ve				*-0.8*		*-3.6*	
Max +ve	Vertical HSS 305x305x13	*75.0*	*0.9*	*17.8*	*0.5*	*27.4*	*12.3*
Max -ve		*-355.7*	*-5.6*	*-18.5*	*-3.3*		
Max +ve	Horizontal HSS 152x152x7.9	*11.0*	*2.9*		*0.1*	*1.8*	*3.8*
Max -ve		*-7.3*		*-0.9*	*0.0*		
Max +ve	Oblique HS 152x152x7.9	*42.9*	*1.0*		*0.1*	*0.5*	*1.9*
Max -ve		*-30.5*	*-1.8*	*-0.2*			
Max +ve	W 360x51*	*20.6*	*316.8*			*1.8*	*346.0*
Max -ve				*-1.0*	*0.0*		
Max +ve	WT 180x81	*151.4*	*1.5*	*1.5*	*0.0*	*3.4*	*1.2*
Max -ve		*-165.6*		*-1.1*		*-3.6*	

Note: if a cell is blank, the force does not have an opposite-direction component

* the loads for the W 360x51 member are loads arising from the **permanent** configuration

Recall the assumption made which allows us to neglect the shear, torsion and moment forces in all members except W section members, where shear is also considered.

The connection design which follows is made based on the assumption that the forces outlined in this table are the maximum forces that arise in all member families and arise from the load case 1.1D + 1.5L + 0.45W.

The connections are split into three groups:

- Connections in the X-Y plane
- Connections in the X-Z plane
- Connection in the X-Z plane of the W 360x51 beam

7.1.4 Connections in the X-Y Plane

The conceptual schematic shown in Figure 7.4 is used as the outline in the X-Y plane connection design. As all the parts of the connection are in some way inter-connected with a certain degree of universality, the design process chosen is not one with discrete steps but cross-checks that happen throughout to ensure the conformity to all pre-determined restraints, both geometric and ones which facilitate the universality of all connections and therefore eventually simplify the construction (both in reducing time & cost).

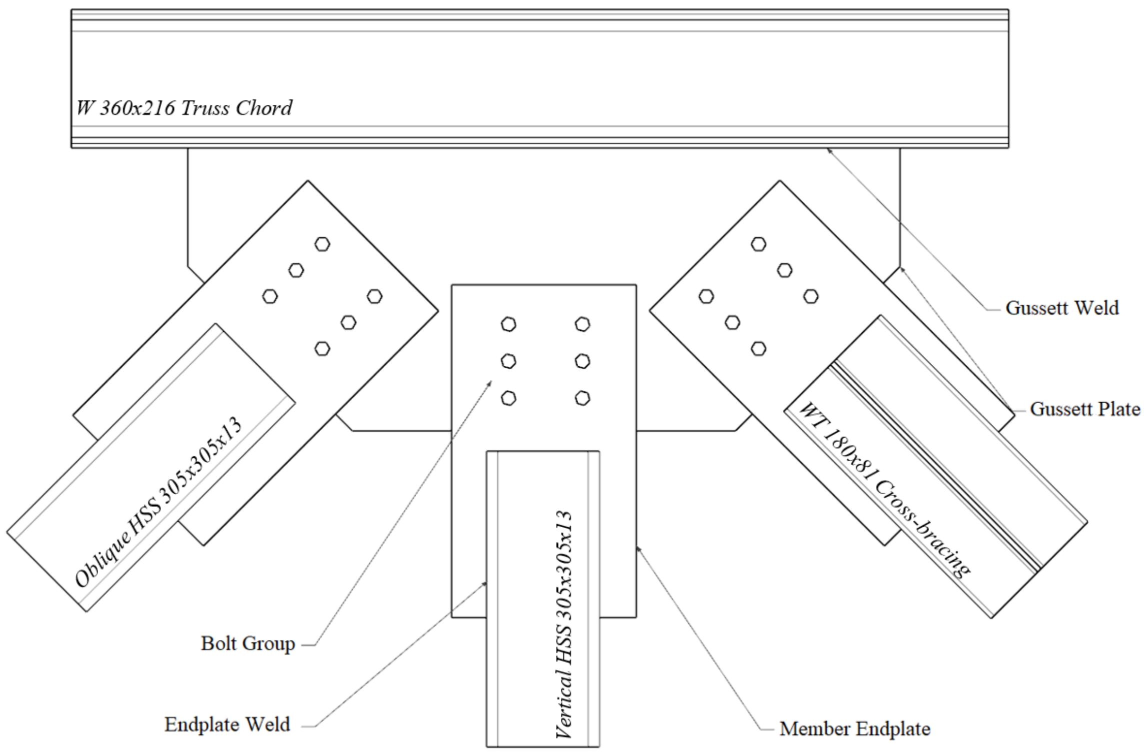

Figure 7.4: Conceptual reference for all X-Y plane connections

Although the connection shown in Figure 7.4 serves as a visual reference for all connections in the X-Y plane, it also serves as an illustration of the assumed worst-case loading scenario that can act on a gusset plate in the truss. Therefore, the X-Y plane connection design begins with the design of the gusset plate minimum weld.

7.1.4.1 Gusset plate

The connection involving an oblique HSS 305x305x13, a vertical HSS 305x305x13 and a diagonal WT 180x81 cross-bracing member is assumed to be the connection which brings about the strongest forces (both in shear and in tension) acting on a designed gusset plate weld.

If all members can exhibit either tension or compression, with 3 connecting members, there are 8 loading combination at this specific point. However, from the analysis results in Table 7.3, and, from a logical structural analysis, it is known that the oblique HSS 305x305x13 members only exhibit tension. Therefore, the loading combination is narrowed down to 4 scenarios. Furthermore, cases in which the vertical HSS 305x305x13 members are in compression are not considered for the design of the weld, as members pushing axially perpendicular to the weld do not influence the weld resistance. Therefore, two loading combinations are left to be evaluated:

Case 1: Oblique HSS 305x305x13 in tension, vertical HSS 305x305x13 in tension, and WT 180x81 in tension

From TABLE, the maximum axial forces in tension for all these members are 466 kN, 75 kN, and 151.4 kN, respectively.

Case 2: Oblique HSS 305x305x13 in tension, vertical HSS 305x305x13 in tension, and WT 180x81 in compression

From TABLE, the maximum axial forces in tension for the oblique and vertical HSS 305x305x13 members, respectively, are 466 kN and 75 kN. The maximum axial force in the WT 180x81 member in compression is 165.6 kN.

Table 7.4 summarizes the two load cases and their components. The worst loads acting parallel (shear) and perpendicular (tension) to the weld are highlighted in green: 331.8 kN and 893.2 kN, respectively.

Table 7.4: Load case components for gusset weld design

HS305D	HS305V	WT180	HS305Dx	HS305Dy	HS305V	WT180x	WT180y	Shear	Tension
			x (kN)	y (kN)	y (kN)	x (kN)	y (kN)	Rx (kN)	Ry (kN)
T	T	T	-244.8	396.6	355.7	87.0	140.9	-157.9	893.2
T	T	C	-244.8	396.6	355.7	-87.0	-140.9	-331.8	611.4

The weld which resists this worst-case loading scenario is a 574.4 mm, 10mm fillet weld with an E49 electrode. An interaction check between the shear and tension forces acting on the weld is shown in the annex section of this report as well as calculations for the weld check.

A detail of the weld and gusset plate dimensions for a three-member plate connection are shown in Figure 7.5. As not all connections in the X-Y plane involve three separate members, Maple Engineering Services prescribes two other different geometric possibilities for gusset plate connections, shown in Figure 7.6 and Figure 7.7. The gusset plates for both one-member and two-member connections must have a minimum of 574.4mm weld, on either side of the plate.

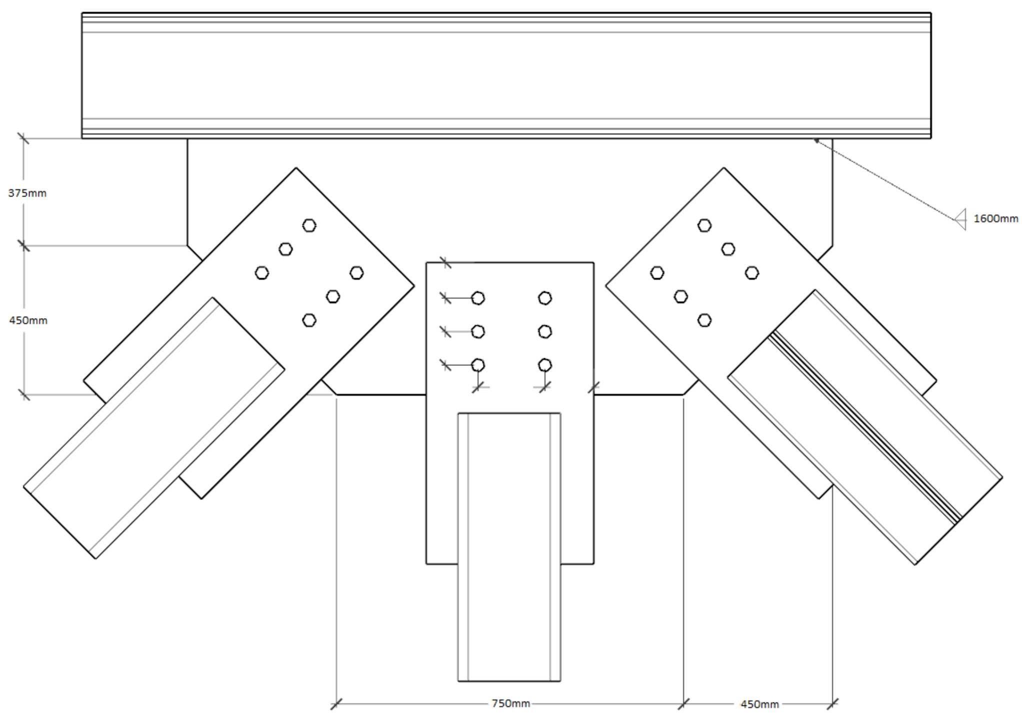

Figure 7.5: X-Y Plane, Three-member gusset plate connection detail

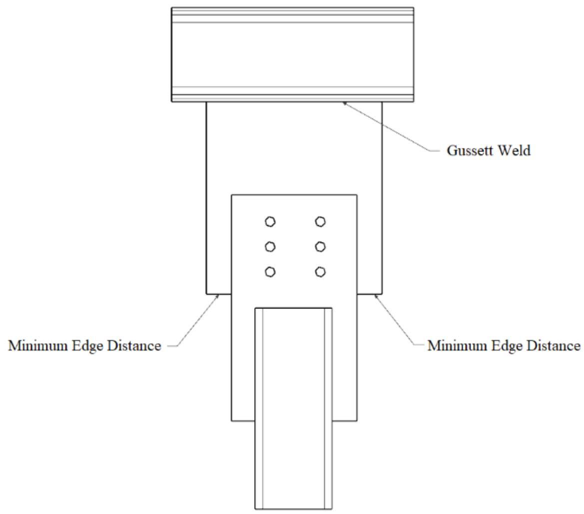

Figure 7.6: X-Y Plane, One-member gusset plate connection detail

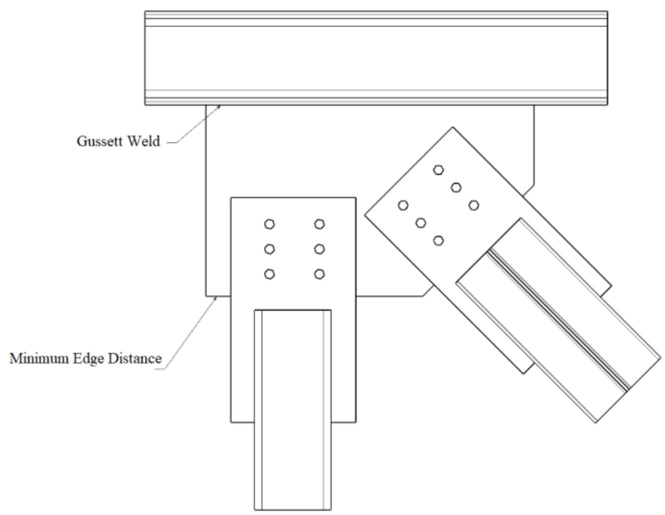

Figure 7.7: X-Y Plane, Two-member gusset plate connection detail

Both the one-member configuration, shown in Figure 7.6 and the two-member configuration, shown in Figure 7.7, must use the same weld dimensions as those found to resist the worst load case scenario of the three-legged configuration, shown in Figure 7.5. As a measure to protect the structure from oxidization, which is accelerated by water having access to small gaps & cracks, Maple Engineering Services further prescribes that all welds connecting gusset plates to W-members must be welded along the whole length of the gusset plate.

7.1.4.2 Member endplate

The members in the X-Y plane are the vertical and oblique HSS 305x305x13 members as well as the WT 180x81 cross-bracing members. To reduce cost & time, both in design and construction, Maple Engineering Services has decided to design for a universal member end-plate and a universal weld which can apply to *both* the WT 180x81 and the HSS 305x305x13 members.

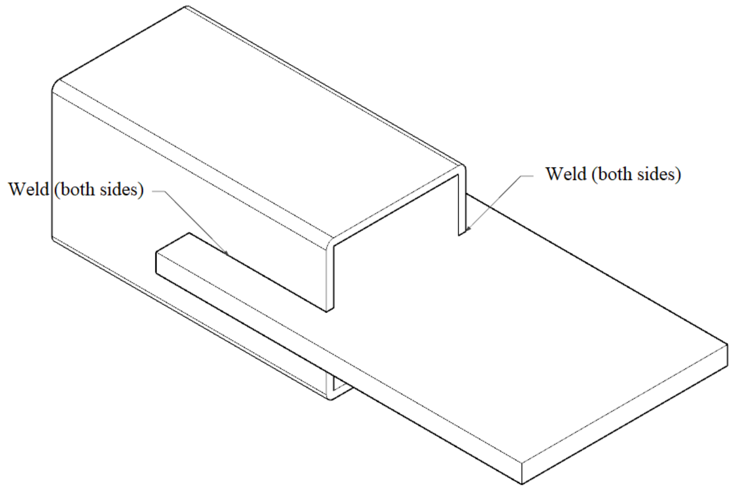

Figure 7.8: X-Y Plane, HSS endplate conceptual isometric detail

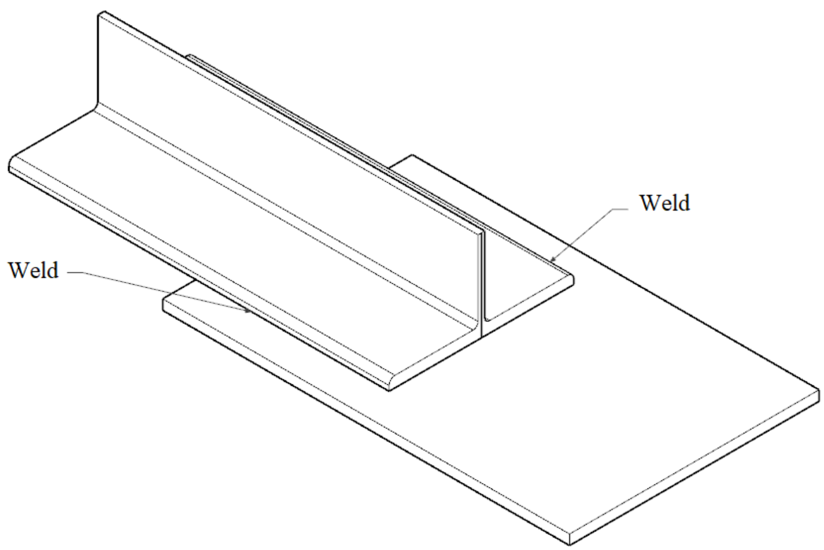

Figure 7.9: X-Y Plane, WT endplate conceptual isometric detail

As both members share the same endplate and weld dimensions, the weld must resist the worst load from either member. From Table 7.3, the worst axial load among all HSS 305x305x13 & WT 180x81 members is 466 kN in tension. Furthermore, the weld group must comprise only *two* welds, as the WT 180x81 endplate is not a sandwich connection like the HSS 305x305x13 endplate connection is. The minimum weld which resists this load is a 75 mm, 10 mm fillet weld. For safety, we assume an 80 mm long, 10 mm fillet weld. The thickness of the endplate is 30 mm, the same thickness as the gusset plate. The endplate width is 500 mm and the length of the endplate is 800mm. Details of the connections for both the HSS305x305x13 endplates and WT 180x81 endplates are shown in Figures 7.10 and 7.11, respectively. The sample calculations for the design of this connection is found in the annex section of this report.

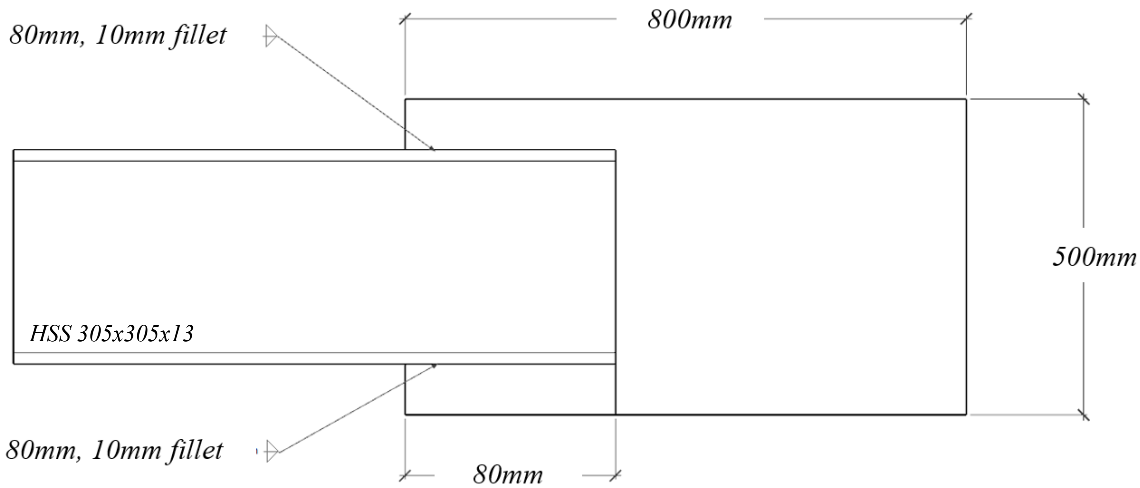

Figure 7.10: X-Y Plane, HSS endplate connection detail

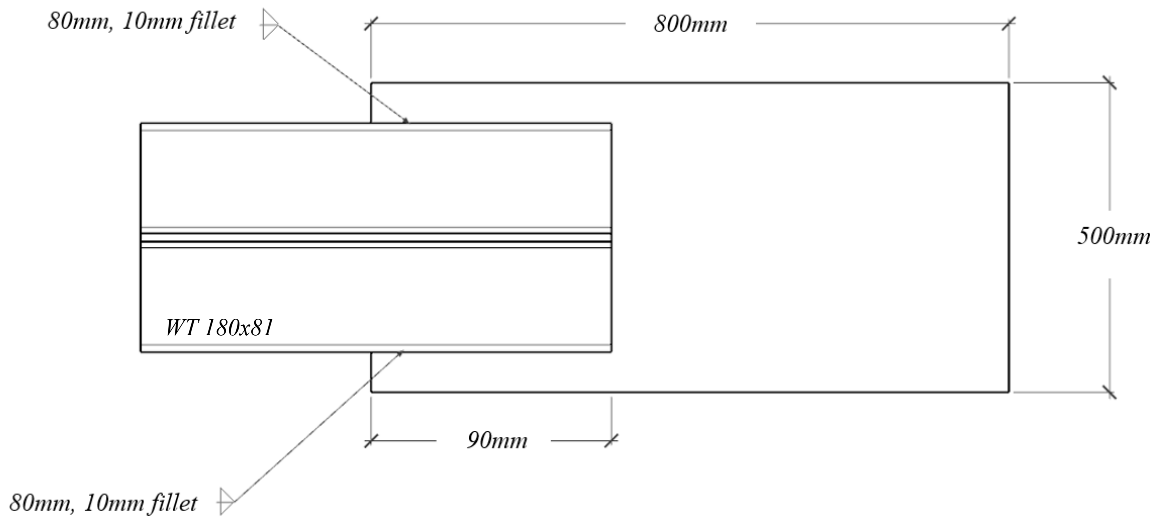

Figure 7.11: X-Y Plane, WT endplate connection detail

7.1.4.3 Bolt group

Like the weld group, the bolt group is designed to be universal among HSS 305x305x13 and WT 180x81 members in the X-Y plane. The bolt group is therefore designed to resist the worst load from either member. The worst axial load among all HSS 305x305x13 & WT 180x81 members is 466 kN in tension. The dimensions of the bolt group which resist this force in tension are found following an analysis of the bolted connection. The procedure is comprised of multiple checks found in Prof. Colin Rogers' *Design of Steel Structures Course Pack* (2019). The sample calculations of this procedure can be found in annex of this report. The detail of the bolt group is shown in Figure 7.12.

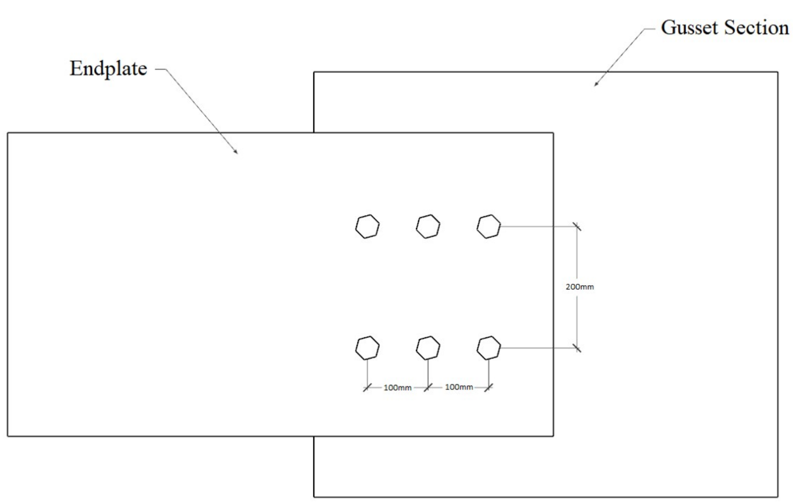

Figure 7.12: X-Y Plane, Bolt group

7.1.5 Connections in the X-Z Plane

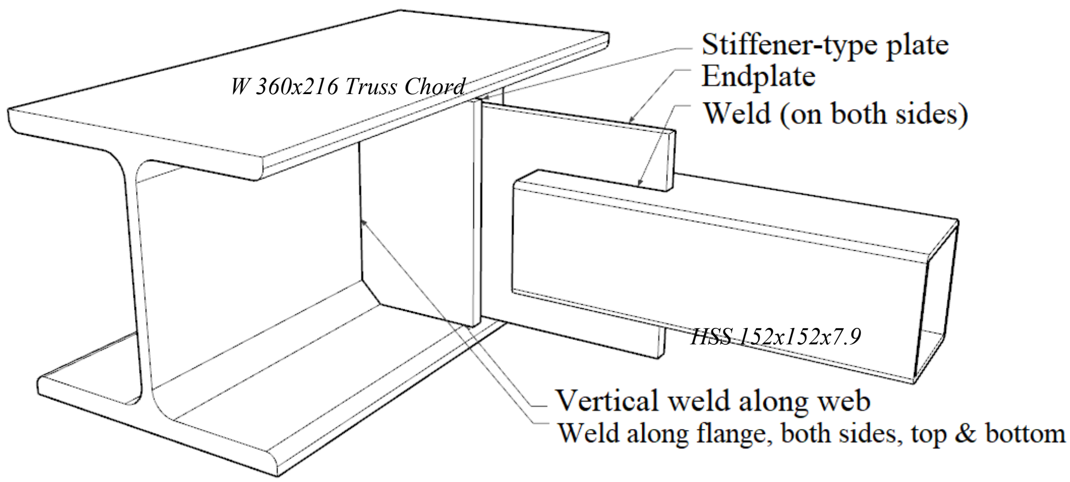

Figure 7.13: Conceptual reference for all X-Z plane connections

Figure 7.13 serves as a visual reference for all connections in the X-Z plane. Whether connections are one, two, or three-membered (two diagonal and one horizontal HSS member), the design is considered for only one HSS 152x152x7.9 member. This approach is a necessary simplification as the interaction between three endplates is beyond our knowledge and scope of work.

7.1.5.1 Stiffener-type plate

To connect the HSS member to the truss' W-shaped chord, a stiffener-type plate is used. The word "stiffener" does not infer that the plate is evaluated and is designed to perform as a shear or bearing stiffener. The plate is designed to resist the worst axial load, in tension, from an HSS 152x152x7.9 member acting in the X-Z plane. The worst axial load, in tension, from an HSS 152x152x7.9 member, is 42.9 kN. The weld group comprises four welds parallel to the axial load, two on the top and two on bottom of the plate (on either side), as well as two welds perpendicular, along the web of the W section (on both sides of the plate). A detail for the stiffener-type plate and its' welded connection to the truss chord is shown in Figure 7.14. The sample calculations for the design of this connection are found in the annex section of this report.

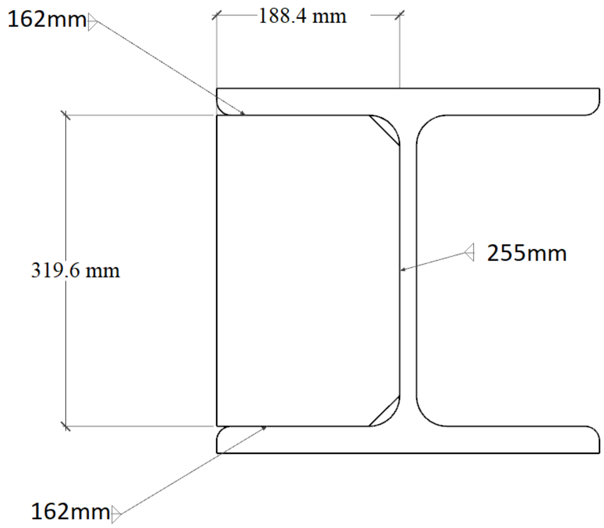

Figure 7.14: Stiffener-type plate for X-Z plane connections

7.1.5.2 Member endplate

Like the member endplates in the X-Y plane, in the X-Z plane, the welds connecting the members to their endplates are designed to resist the worst axial load of 42.9 kN in all HSS 152x152x7.9 members. FIGURE shows a conceptual isometric view of the sandwiched endplate design. Four welds, on either side and above and below the plate, must be designed to resist the worst axial load of 42.9 kN. A detail of the designed connection is shown in Figure 7.16. The sample calculations for the design of this connection are like those for the X-Y plane member endplate, which are found in ANNEX.

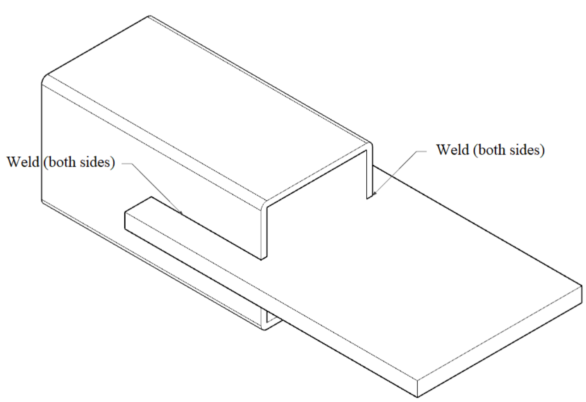

Figure 7.15: Conceptual isometric view of X-Z plane member endplate

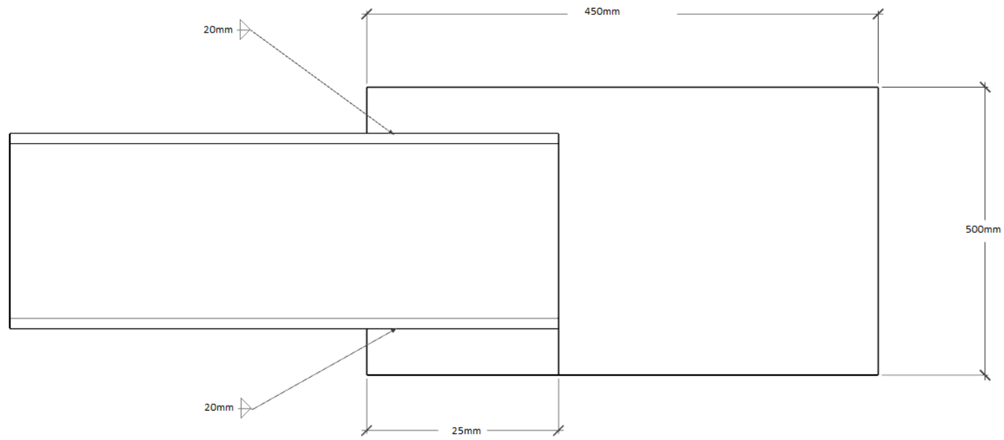

Figure 7.16: X-Z plane member endplate weld & plate detail

7.1.5.3 Bolt group

The bolt group is designed to be universal among all HSS 152x152x7.9 members and their plates. The bolt group is therefore designed to resist the worst axial load, 42.9 kN in tension, in all HSS 152x152x7.9 members. The dimensions of the bolt group which resist this force in tension are found following an analysis of the bolted connection. The procedure is comprised of multiple checks found in Prof. Colin Rogers' *Design of Steel Structures Course Pack* (2019). The sample calculations of this procedure are like those for the endplate bolt group calculations for members in the X-Y plane which can be found in the annex section of this report. The detail of the bolt group designed to conform with all the checks is shown in Figure 7.17.

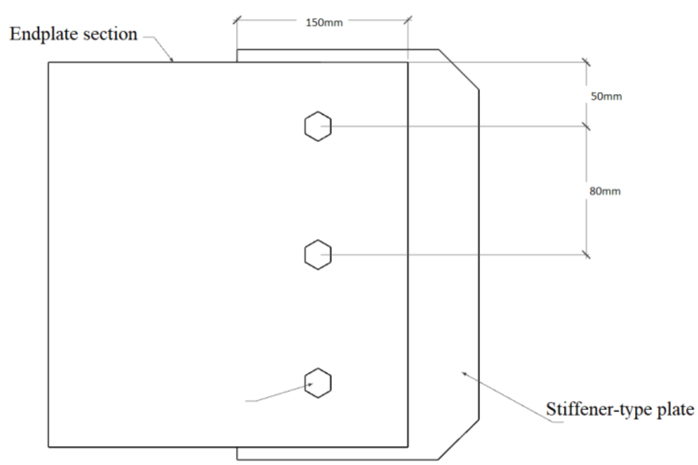

Figure 7.17: X-Z plane member bolt group

7.1.6 Bearing Beam Connection

As described in the structural analysis portion of this report, the temporary and permanent configurations of the frame differ in that the W 360x51 members in the temporary configuration share the loads from the existing structural spans amongst *four* members; two on top and two on the bottom of the truss. In the permanent configuration, the W 360x51 member family is comprised of a singular member on top & a singular member on the bottom, supporting twice the loads from the existing structure when compared to the temporary configuration. Therefore, all loads in Table 7.3 are loads arising from the temporary configuration, following the assumption that the temporary configuration produces the worst load scenario among all members *except* the W 360x51 member, where, Table 7.3 shows the loads acting on that member from the permanent configuration.

Furthermore, while previously assumed that shear, torsion and moment effects are neglected due to all members being pin-connected and all secondary forces being relatively insignificant compared to their axial counterparts, the assumption for the W 360x51 bearing member is that it is still pin connected, so the connection must only resist axial and shear effects. Moment and torsion effects are neglected due to this assumption.

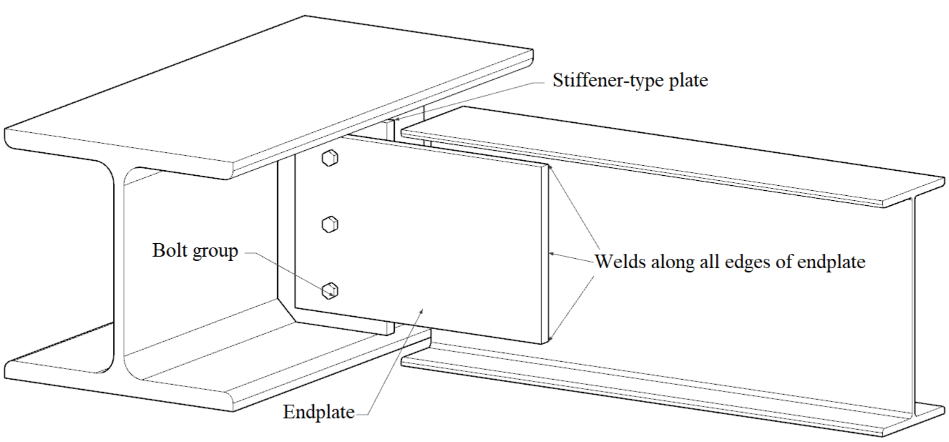

Figure 7.18: Conceptual reference for bearing beam connection

7.1.6.1 Stiffener-type plate

The stiffener-type plate used is the same geometry as the stiffener-type plate for connections in the X-Z plane. The weld is evaluated to resist the shear load of 316.8 kN. The sample calculations for the design of this connection is found in the annex section of this report.

7.1.6.2 Endplate

The endplate weld group is composed of two welds parallel to the member span, and one weld perpendicular to the member span. The welds parallel to the member span are to span the whole length of the overlap of the plate and the central bearing member. The weld group is designed to resist a shear force, perpendicular to the member length, of 316.8 kN, as shown in Table 7.3. The weld is not checked to resist the max axial force of 20.6 kN as this force is relatively small compared to the shear force, which the weld is checked to resist. Figure 7.18 shows the details of the connection between the member and the endplate, as well as the geometry of the endplate. The sample calculations for the design of this connection are found in the annex section of this report.

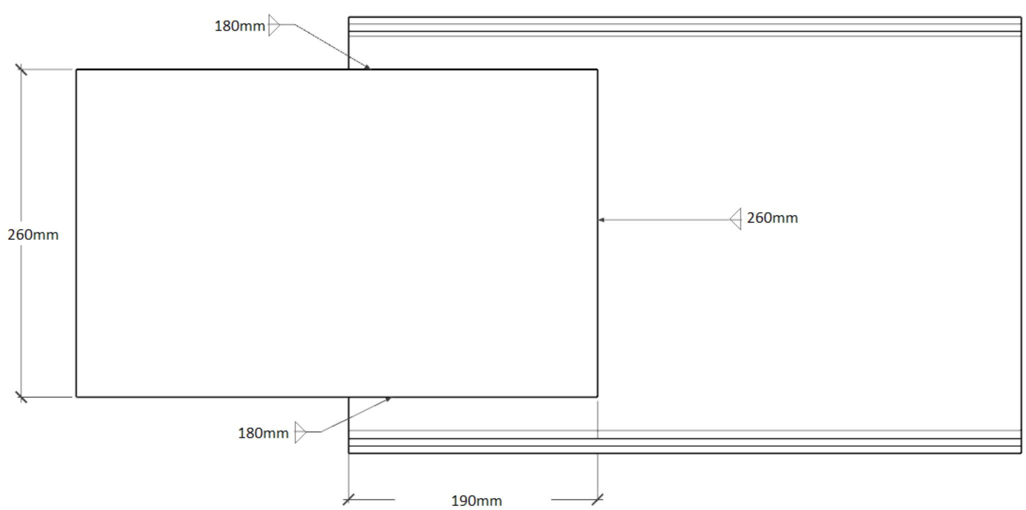

Figure 7.19: Bearing beam, endplate weld

7.1.6.3 Bolt group

Like the weld group, the bolt group is designed to resist the worst shear load of 316.8 kN. The dimensions of the bolt group which resist this force in shear are found following an analysis of the bolted connection outlined by the procedures in Prof. Colin Rogers' *Design of Steel Structures Course Pack* (2019). The sample calculations of this procedure can be found in the annex section of this report. The detail of the bolt group is shown in Figure 7.18.

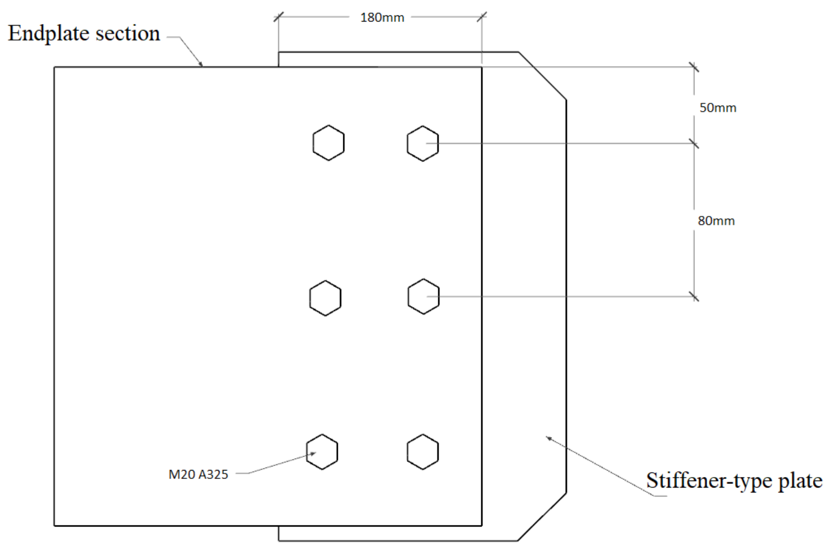

Figure 7.20: Bearing beam, bolt group

7.2 Column Support Connection Design

The support connection design encompasses the transfer of forces from the truss structure to the outer columns. The chosen type of support system is a roller and pin type because of the zero moment values at each support. A total of 4 supports are needed, 1 at each lower truss corner. On one end, there are 2 pin supports that restrict movement in each direction, while there are roller supports on the other end. The roller supports allow for thermal expansion and contraction in the longitudinal direction.

7.2.1 Loads

A thorough load analysis is conducted in SAP2000, where the different SLS and ULS load combinations are inputted. The software analysis provides the forces present at each support. The load combinations can be found in Table 1, where the highlighted values represent the maximum force in the respective directions at the support joints.

Table 7.5: Load Combination Forces at Support Joint

Load type	Vertical Load (kN)	Longitudinal Load (kN)	Lateral Load (kN)
FLS1	310.83	0.14	0.11
SLS1	306.59	0.14	0.11
SLS2	38.15	0.11	0.10
ULS 1	367.35	0.05	0.16
ULS 2	363.11	0.16	0.08
ULS 3	373.38	33.7	26.22
ULS 4	353.6	104.5	81.63
ULS 5	295.31	27.93	26.9
ULS 7	376.39	56.07	43.74
ULS 9	362.4	0.17	0.21

The support connection design will be based off the worst-case scenario. In this case, the maximum vertical, longitudinal and lateral loads a support must resist are 376.39 kN, 104.5 kN and 81.63 kN, respectively. The vertical load can be interpreted as a compressive downwards force, while the longitudinal and lateral loads as shear forces, in directions perpendicular to each other.

7.2.2 Types and Components

There are 3 primary components to the support connections that transfer the forces to the columns. Each component serves a different purpose and is critical to the overall functioning of the support. The different types of supports are also explained in more detail in this section, specifically referring to the functionality of the components and its load transfer mechanisms.

Elastomeric Bearings

Elastomeric bearings act as a cushion between the column and truss. They deform vertically as the loads change due to differences in live, snow, earthquake and wind loads. Specifically, laminated bearings will be used for these supports. Laminated bearings are made of high-quality natural rubber that make them ideal for bridge applications, specifically for seismic vibration and shock absorbing applications.

Base Plates

Base plates act as a compressive and shear load transfer material between the I beam and elastomeric bearing. The vertical load from the I beam is directed onto the base plate. It ensures the elastomeric bearing deforms evenly by having an even pressure distribution at the top of the elastomeric bearing. In addition, they are welded to the I-beam and have two holes for the anchor rods in order to transfer shear forces. They also serve as the mechanism that allows movement for thermal expansion.

Anchor Rods

Anchor rods serve as a tensile and shear transfer mechanism between the truss and column. However, due to the different load combinations and forces acting at the joint, no tensile stresses can develop in the anchor rod. The vertical compressive load is too great for any possibility of an upward tensile load; therefore, the anchor rods will only need to resist shear in the lateral and longitudinal directions. Fillet welds connecting the I beam to the base plate will resist any longitudinal and lateral shear and transfer the shear to the anchor rods. The anchor rods are placed in the column cap and through the base plate holes, and the shear forces will be transferred into the column.

Guided Supports

Guided supports restrict movement in the vertical and lateral directions. The induced loads are the same as those for fixed supports, except for longitudinal shear loads. The base plate is free to move along with the I beam in the longitudinal direction due to thermal expansion forces. This is possible by maintaining the same weld design as fixed supports yet welding 10M bars to the bottom of the base plate in order to reduce friction between the elastomeric bearing and base plate. This is because the coefficient of friction between the elastomeric bearing and base plate is very high due to the elastomeric bearing material. The anchor rods are still in place in the base plate holes, yet the holes are lengthened in order to allow the base plate to move and remain in place.

7.2.3 Elastomeric Bearing Design

Elastomeric bearings must be designed in accordance with the CSA S6-14 Clause 11.6.6. The most restricting clause, 11.6.6.7 places limits on the allowable pressure of the bearing. The pressure calculations are shown in the annex. The permanent loads represent the loads that are always present (i.e. dead loads), whereas the total and average loads represent the maximum load combination value, as seen in Table 7.5. With dimensions of 50 mm x 200 mm x 300 mm, all limits are satisfied, as seen in Table 7.6. This satisfies the geometric requirements for laminated bearings, where the length must be at least 3 times greater than the thickness. This also means that its width is shorter than the pile cap width, ensuring it constantly sits on a flat stable surface. The exact material and number of layers is to be chosen by the contractor, yet they must respect the shape factor and material limitations. This design is to be used at all 4 supports.

Table 7.6: Elastomeric Bearing Pressure Distribution Combinations

	SLS Permanent	**SLS Total**	**SLS average**	**ULS Permanent**	**ULS Total**
Actual (MPa)	0.635833333	5.1098333	5.1098333	6.04	6.273166667
Limits (MPa)	4.5	7	7	7	10

7.2.4 Base Plate Design

Base plates must be designed such that their resistance is greater than the compressive load. Their thickness must be of sufficient size to ensure the moment resistance of the plate is greater than the induced moment from the compressive load. In addition, the W beam ends must be checked for enough resistance for local and overall buckling.

There are some geometrical restrictions for the base plate. The base plate length must be significantly larger than 394 mm (width of W beam flange) in order to be longer than the I beam and have enough area for anchor rod installation on each end. In addition, the load is assumed to be applied at the center of the base plate, and center of the W beam, ensuring no eccentricities exist. The values found in Table 7.7 represent the calculated restrictions of the base plate, with the steps found in the annex.

Table 7.7: Base Plate Design Limitations

Bearing Stress (MPa)	**Min. Required Bearing Area (mm^2)**	**Factored Moment (Nmm/mm width)**	**Min. Thickness (mm)**	**Min. Thickness Deflection (mm) (Check)**
16.575	22,708	565,000	83.7	20.6

In order to satisfy these requirements, a base plate with size 85 mm x 275 mm x 600 mm for all 4 supports is chosen. Using these dimensions and those of the W beam, the web local plastic buckling and web overall buckling bearing capacities are 1752 and 1127 kN, respectively, using in the annex, ensuring no end stiffeners are needed for a compressive load of 376.39 kN.

7.2.5 Anchor Rod Design

The CSA S6-14 Clause 10.19 does not specify how additional shear planes affect the calculations; therefore, it is assumed that the lateral and longitudinal forces can be summated. The maximum factored shear is equal to 132.6 kN at an angle of 38 degrees from the longitudinal (x) axis. Considering the minimum anchor rod diameter is 30 mm, the factored shear resistance for 2 anchor rods is 391.9 kN, greater than the factored shear force. For the anchor rods to meet this strength, A325 Anchors will be installed. Calculation details can be found in the annex.

The bearing resistance of the concrete must also be checked in order to ensure the concrete can resist the shear forces transmitted by the anchor rods. In the annex, the bearing resistance of the concrete can be rearranged to find the minimum embedded length, which is 81 mm. The embedded depth must equal to or smaller than 20 times the interfacial diameter as well. These calculations can be found in the annex.

Typically, anchor rods must be headed, but since there is no tensile force, there is no need. They must also be threaded on each end for 150 mm to ensure sufficient friction between the anchor and the concrete and tightening of the nuts. This anchor rod design is to be used for all 4 supports. The anchor rods must have a minimum of 8 mm free on both ends, along with a clear 5 mm between the base plate and above nut so that vertical deformations can occur.

To ensure that there is sufficient space for the base plates to move with the W beam at the guided support, using the thermal expansion formula in the annex, it is estimated that there can be movement of at least 27 mm. This is assuming a thermal coefficient of expansion of 1.3 x 10-5/oC and using Clause 3.9.4.1 of the CSA S6-14 to determine the maximum and minimum effective temperatures. The maximum hole size allowed in the CSA is discussed in Clause 10.18.4.2 and limits the size to 75mm. This allows for enough movement of the base plate. In addition, it specifies 10 mm washers must be used, along with nuts and counter nuts.

7.2.6 Fillet Weld Design

7.2.6.1 Beam-Base Plate Weld

The fixed support fillet weld design must be stronger than the induced longitudinal and lateral shear in order to ensure the base plate and W beam remain firmly connected and the shear can be transmitted via anchor rods to the concrete column. Yet, the guided support fillet weld design must be stronger than the induced lateral shear and frictional force between the 10M bars and the base plate.

In order to prevent corrosion between the base plate and the edge of the W beam flange; an 8 mm deep weld will be done along both lengths (longitudinal axis) of the flange. The weld metal shear resistance of the fixed support is found to be 952.7 kN in the annex, with the matching E49 electrode. To calculate the shear resistance of the weld metal fracture, the line of action of the induced force is a factor. For the fixed support, as mentioned in section 12.4, the line of action with respect to the longitudinal axis is 38 degrees. However, the line of action at the guided support is 14 degrees. To find this angle, the magnitude of the frictional force must be found. The frictional force is equal to the frictional coefficient between the 10M bars and the base plate (assuming to be 0.7) multiplied by the normal force. The normal force is the weight of the base plate + the concentrated vertical load. Knowing the lateral shear force and the frictional force (325.4 kN), the resultant and its line of action can be found in the annex. The resultant shear force is equal to 335 kN. Calculations for the welding strength of 952.7 kN for the fixed support and 852.5 kN for the guided support are found in the annex. The fixed support weld strength is greater than the 132.6 kN induced shear force, and the guided support weld strength is greater than the 335 kN resultant shear and frictional forces. The base metal shear resistance is found to be 888.8 kN in the annex, meaning that this design is adequate.

7.2.6.2 Base Plate – 10M Bar Weld

The fillet weld design between the base plate and 10M bars must resist similar forces as the fillet weld design between the base plate and W beam of the guided support, however the friction coefficient of the frictional force is different. Considering the material of the elastomeric bearing is to be chosen by the contractor, the coefficient can range from 0.3 to 0.8. This means that a conservative value of 0.8 must be chosen. This results in a frictional force of 371.9 kN. Using the weld shear design in the annex, a 6mm deep weld across the length of a 300 mm long bar will provide 543 kN of shear resistance, and another 18.1 kN at the end of the bar. Even though this is greater than the factored shear force, several bars must be used to ensure the compressive pressure is distributed evenly onto the elastomeric bearing. The bars must be at least 6mm apart, meaning that (200 mm wide bearing divided by 16 mm) 12 bars can be placed along the length of the bearing.

7.2.7 Finalized Column Support Connection Design

The final fixed and guided support connection design can be found in Figure 1, with the fixed support on the left, and guided support on the right.

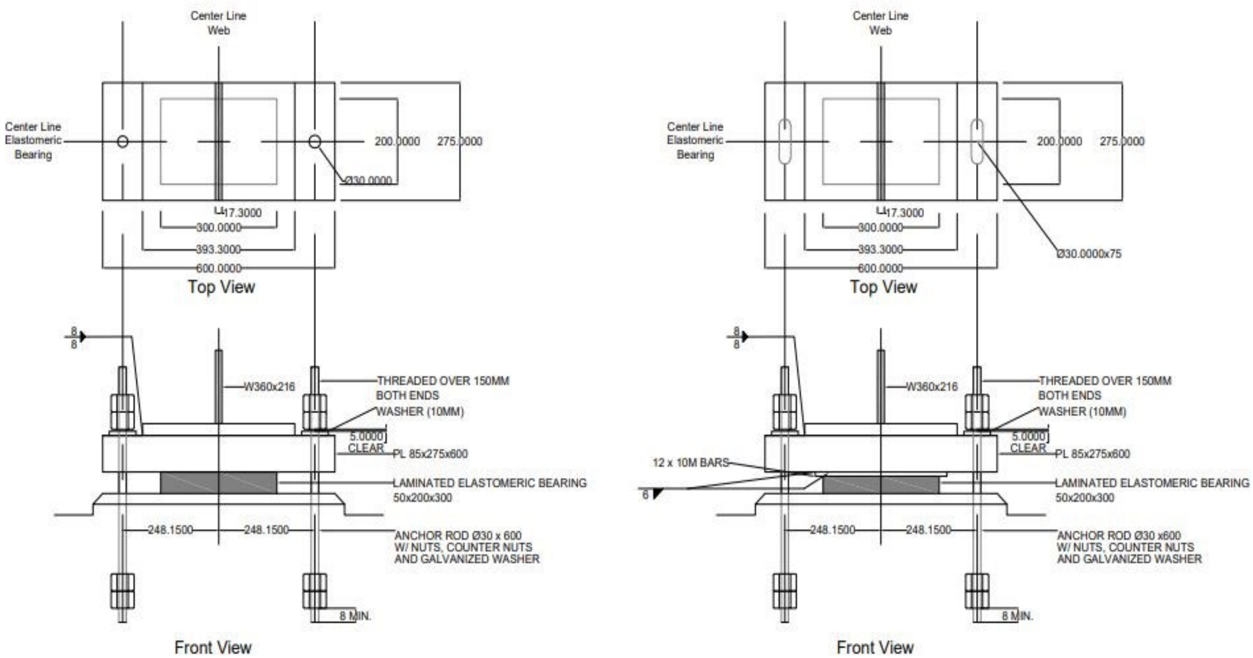

Figure 7.21: Finalized support connection details

8.0 Reinforced Concrete Structure Design

Two identical reinforcement structures serve as the main support system for the truss structure. Reinforcement structure are free on one and pinned on the other to prevent expansion stresses. The structures are not connected to the pathway but are only connected to the edges of the truss members. This mainly due to our objective of not changing the shear and bending moment of the pathway.

The reinforcement structure has a total length of 13.1m and above ground level height of 10.3m.

It spans 4.774M and is free to move on one end to eliminate expansive stresses. The structure consists of 3 main component substructures:

1. The column pier Cap beam
2. Rectangular reinforcement column
3. Reinforced square footing

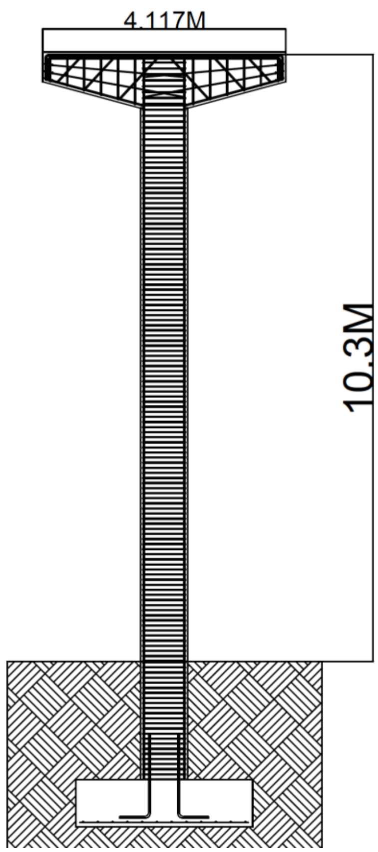

Figure 8.1: Reinforced Concrete Truss Support Structure

Each component was designed separately in accordance with CSA design code. The following section will summarize the design for each component while references the appropriate sample calculations in the annexes.

8.1 Column Pier cap Beam

The column cape is a vital component of the reinforcement structure. It serves the main purposes of funneling vertical axial load from the pathway bridge and the support to truss downwards towards the reinforcement and the footing. The column cape experiences both shear and moment and thus, it was crucial to take extra care when designing the reinforcement for the cap beam. The cap beam was designed in accordance with the procedure in the CSA along with embedded formulas for the effective depth taken from "Response of variable- depth Reinforced Concrete Pier Cap Beams" scientific paper

The designed Pier cap has the following cross section as shown in the figure.

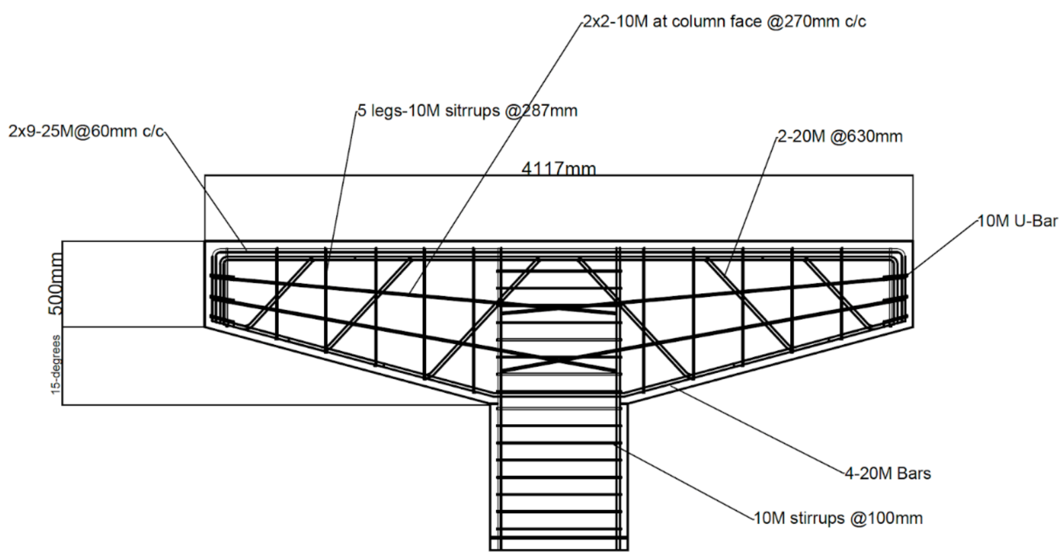

Figure 8.2: Reinforced column pier cap

- Span= 4117mm
- Thickness = 800mm
- Depth 1= 500mm
- Depth 2= 950
- 2x9-25M @60mm c/c horizontal bars spanning the cape
- 10M U-Bars
- 5 legs-10M stirrups @287mm
- 2x2-10M at column face
- 2-20M inclined bars @558mm
- 4-20M bars

- 10M stirrups @100mm

Extra care was taken to the design of the reinforcement after calculating the required area of reinforcement (12454.9). the reinforcements specifications are shown on the cross-section figure. it was designed in reference to the scientific paper highlighting some designing choices, a 2x9-25M @60mm c/c horizontal bars spanning the cape and anchoring at 90 degrees on the edges, 2-20M inclined bars @558mm and 5 legs-10M stirrups @287mm play a key role in the confinement of the concrete.

Moment CSA procedure

Factored moment at column face is 1907.98 kNm

Analysis was done on the two critical depths DV1 and DV2 (500mm and 811mm) for both shear and moment. The minimum flexural reinforcement at both depths is 901.25mm^2 and 1459.88 mm^2 respectively and calculated moment resistance Mr is 3624.1kNm.

Mr> Mf

Detailed sample calculations in the annex

Shear CSA

The preliminary analysis for shear involved both Simple and general method procedures. The simple method provided a conservative estimate for the shear resistance and thus the simple method is used and presented in this report. Factored shear Vf is 481.72kN at the critical depth dv1 500.11

Factored shear resistance is = 512.13kN at critical depth/section

Vr>Vf

8.2 Rectangular reinforced columns

Reinforcement columns were designed to transfer the gravity loads of the steel truss along with the pathway bridge and the cape to the footings. Moreover, the columns needed to resist uniform lateral loads but only one moment at the bridge end that is pinned. No architectural or structural drawings were provided for the design of the columns, thus the design initiated by creating the cross section of the columns which reinforcement bars, stirrups and cover/spacing. First, for a long duration resistance, the concrete stress must be reduced by a factor, α_1. This factor can be determined by reducing $0.0015f'_c$ to 0.85; thus, α_1 was equal to 0.805.

Reinforcing rebar were needed to proud axial stiffness and carry/resist lateral loads such as wind and seismic. Bars were set up along with stirrups and ties to provide lateral confinement of the concrete as well as to meet lateral (seismic and wind) design requirements.

The reinforced column cross section is 800 x 500 mm. a cover of 65 was chosen along with 6-30M bars and 10M stirrups spaced at 100 mm along the vertical height of the column. This resulted in a steel percentage of 1.75% which satisfied the requirement of being between 1% and 4%.

The critical factored axial load, P_f was equal to 1033.2kN. No tensional load was assumed to be applied to these columns as the tensional loads were insignificant compared to the axial load and the moment. The maximum factored axial load resistance, $P_{r,max}$, was found to be 5129 kN. With further investigations into the balance loads, the balanced axial load resistance 1871Kn. Mbr was found to be 1094.9kN.

The column is significantly stronger than the required factored forces. This is mainly due to the goal of increasing stability and structural safety in our project. Section was designed in accordance with CSA code.

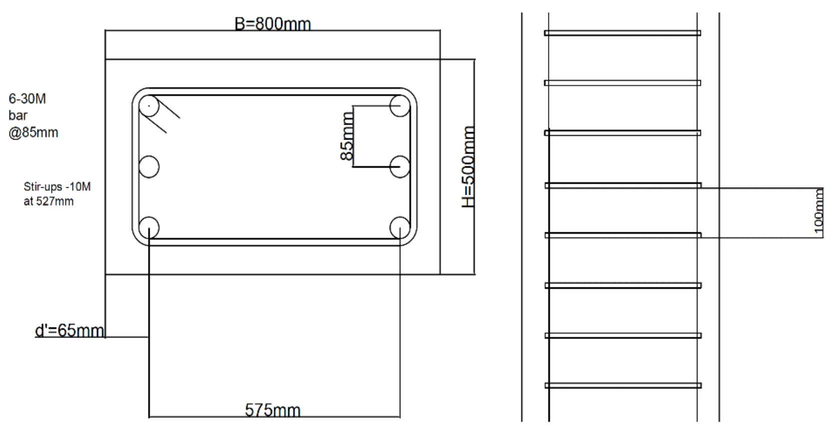

Figure 8.3: Reinforced rectangular column section

8.3 Reinforced square footing design

A 3x3m square footing is designed for each reinforced column. Firstly, the concrete used for the design of these short columns had a specified compressive strength, *f'-c*, = 25 MPa and the steel yield strength of 400 MPa. The most critical load was found to be 1128 kN . Moreover, a thickness of 800mm was assumed for the square footing. Provided the allowable soil pressure of 220 kN/m^3 by the geotechnical survey, **we conclude that the dimensions of the footing is acceptable.**

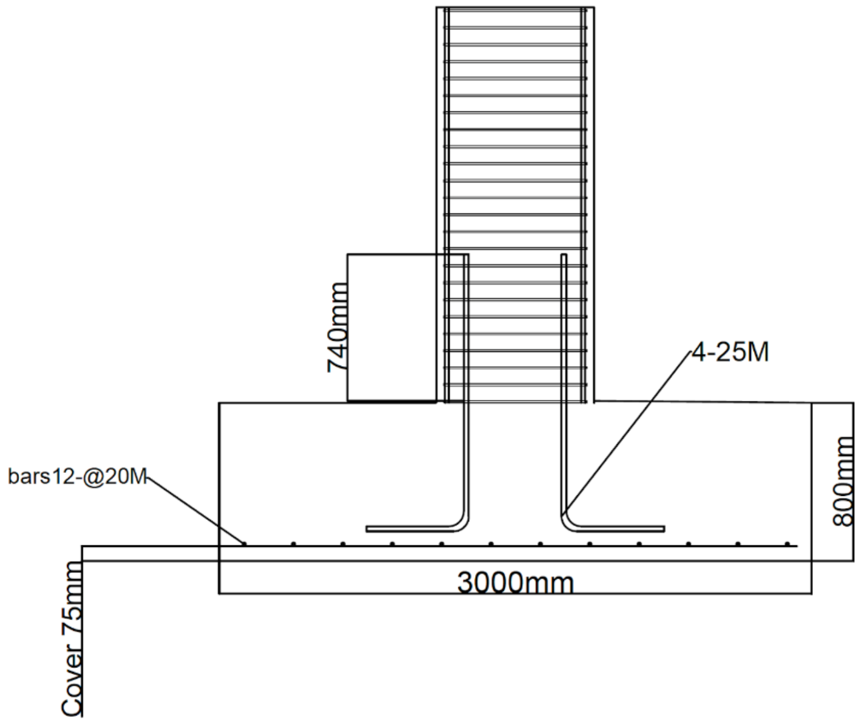

Figure 8.4: Reinforced square footing

Shear capacity was also considered in the design of the footing. With a cover of 75 mm and 20M bars, the effective length, d, was found to be 700 mm and the critical section was located at a distance, d_v, of 630 mm from the face of the wall. Since the footing had a height greater than 350 mm, the simplified shear design method could not be applied. The value of shear carried by the concrete was equal to 972.4 kN.

As for the moment, mf is 227.5KnM. The required flexural reinforcement As is 3635.0mm^2 , depth $d_{avg\ (flexure)}$ is 705mm and Mr was calculated to be 844KnM. Bearing capacity is 17680KN.

Detailed step by step procedure is in the annex.

9.0 Constructability & Estimation

9.1 Construction Phasing & Recommendations

As part of our mandate to design a structural system which would enable the removal of the existing central support column of the Cogswell Street Pedway, the client has also asked for Maple Engineering Services' expertise in construction to provide recommendations to a potential general contractor in the construction of the proposed design.

Our firm has divided the construction phases into discrete sections and outlined specific recommendations for each phase relating to design criteria and construction feasibility. Our firm's proposed design is feasible **only** if all recommendations in the section are followed.

9.1.1 Deconstruction of Cogswell Street On-Ramp

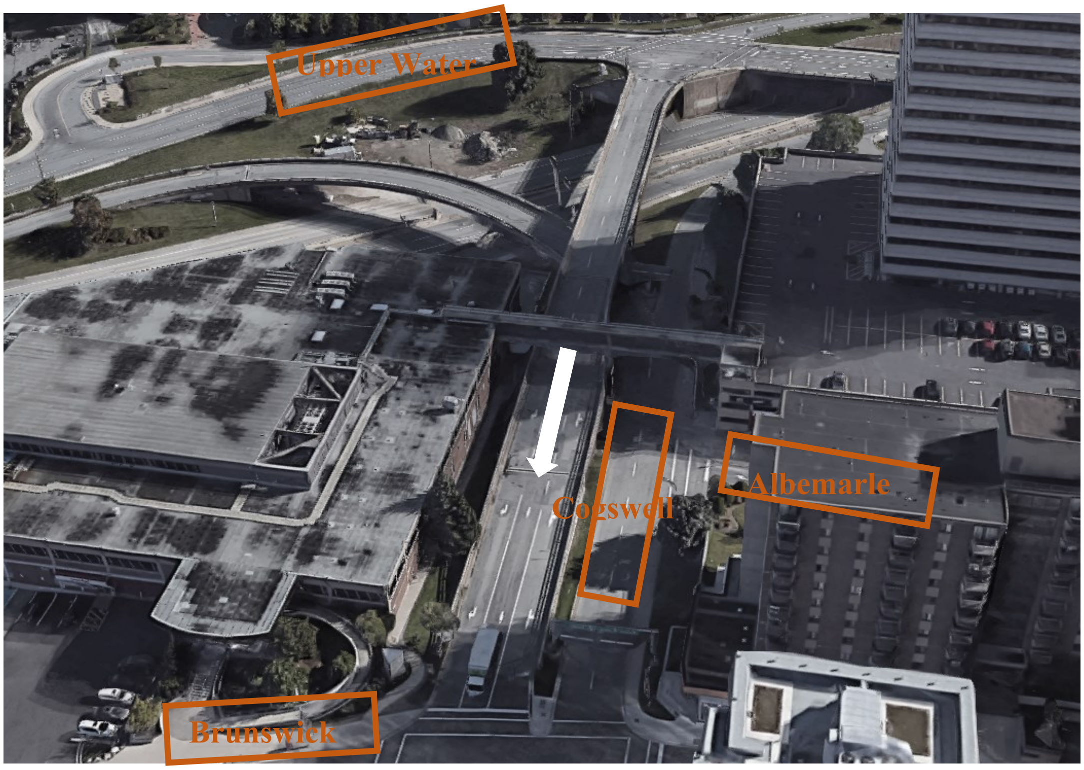

Figure 9.1: Cogswell Street Road Network

The scope of our mandate not only encompasses all work relating to the existing pedestrian walkway spanning Cogswell St., but the client has asked for recommendations regarding the deconstruction of the existing on-ramp. The on-ramp in question is a one-way on-ramp, connecting the eastern Upper Water St./Barrington St. intersection to the western Brunswick St./Cogswell St. intersection. As shown in Figure 9.1, the on-ramp spans below the pedestrian bridge, and our firm provides recommendations regarding the deconstruction of this on-ramp.

The full existing structural drawings are found in the annex of this report. A cut section, looking East, from the plan provided 'A8 Bridge from North Pad to Trade Mart.pdf' across the centerline of the pedestrian walkway is shown in Figure 9.2. The clearances provided in the elevation view show that less than 1m clearance between the central support column and the rightmost face of the on-ramp. Furthermore, the foundations for the vertical supports of the on-ramp are relatively close to the singular foundation for the central column of the pedestrian walkway. For these reasons, our firm recommends that no deconstruction work of the on-ramp (both above and below-ground) within a radius of 10m from the pedestrian walkway's central support column is to be done with a hydraulic chipping hammer. All deconstruction work within this radius must be done by saw-cutting or other minimal vibration deconstruction methods.

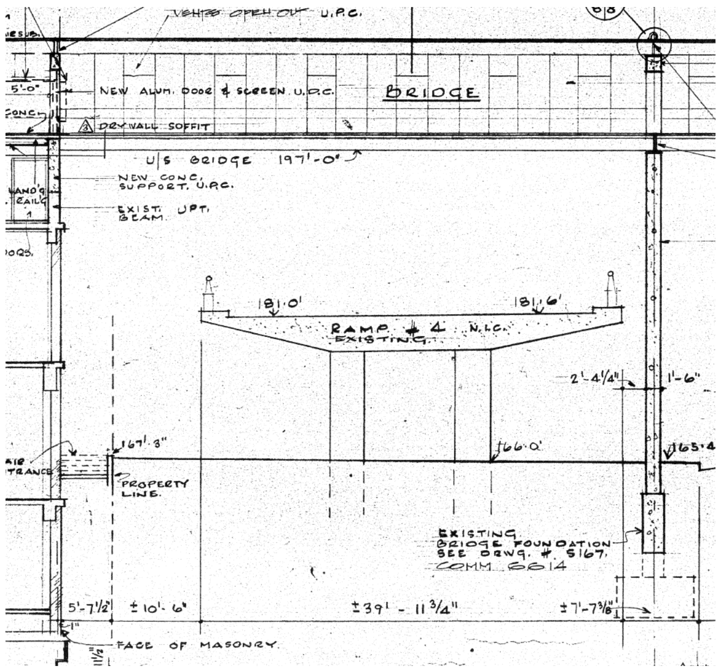

Figure 9.2: Elevation view of on-ramp and pedestrian bridge

9.1.2 Installation of Concrete Support Columns

Our firm does not provide any specific construction recommendations for the concrete support column construction. Construction must be done within the physical constraints of the existing structures, clearances and property lines.

9.1.3 Installation of Truss Cage Structure

Our firm's design solution revolves around the client's need to keep the pedestrian walkway serviceable for the maximum amount of time possible throughout the physical implementation of the design. Therefore, clearances have been provided in the geometry of the temporary truss configuration to allow the truss to be installed *around*, and not interfere with, the existing structure. Furthermore, enough clearance has been provided to allow the careful deconstruction of the central support structures by non-intrusive and minimal vibration deconstruction methods. Throughout the installation of the truss cage structure, the general contractor's operations need not physically interfere with the existing structure.

Our firm provides important recommendations regarding the installation of the vertical trusses (in the X-Y plane). The vertical truss panels must be pre-assembled prior to installation: all oblique HSS 305, vertical HSS 305 and WT 180 cross-bracing must be connected to the truss chords, and all shear-type plates (described in the "Connections" section of this report) must be welded in their appropriate locations. Prior to lifting the two vertical truss panels into place, an engineer must submit a rigging plan to the client, signed & stamped by a professional engineer registered to Engineers Nova Scotia. The rigging plan must state that the rigging method chosen by the general contractor ensures the safe installation, accounting for lateral toppling effects, of the vertical truss panels. Finally, all lateral members (members in the X-Z plane) must arrive to the site with endplates attached and be ready to be bolted to the shear-type plates pre-welded to the W chord members.

9.1.4 Bridge Jacking Process

W 360x51 beams have been selected for the temporary *and* permanent central bearing (contact between existing and truss structures). Our firm recommends a jacking method to essentially lift the existing structure off its' central support structure and allow for the careful deconstruction of the central support column. This process will allow for the cancellation of an unacceptable deflection of 20mm found in our structural analysis. The client, due to serviceability restraints, prescribed an allowable deflection of 10mm. Conceptually, the process our firm recommends is as follows:

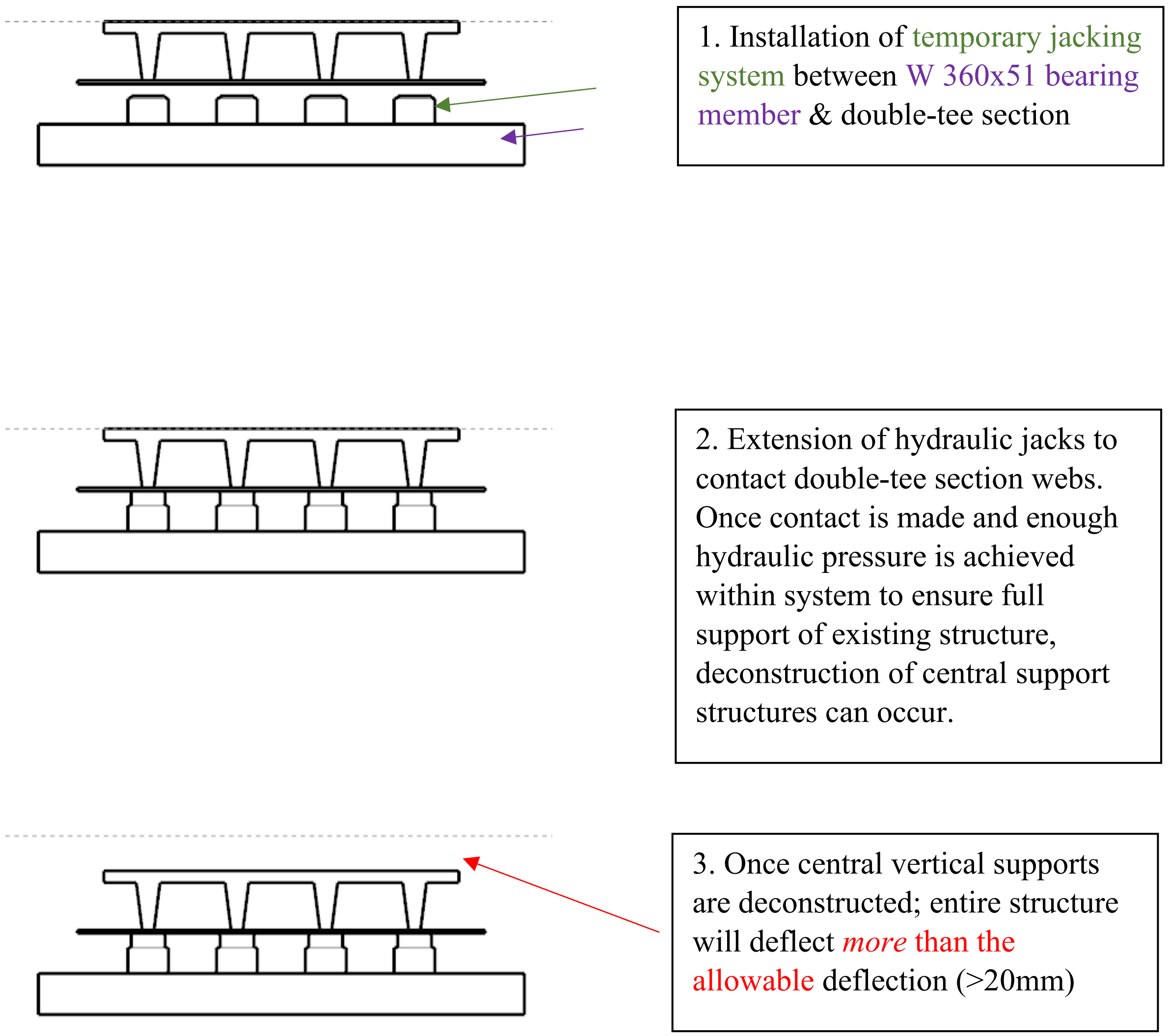

Figure 9.3: Jacking process (1/2)

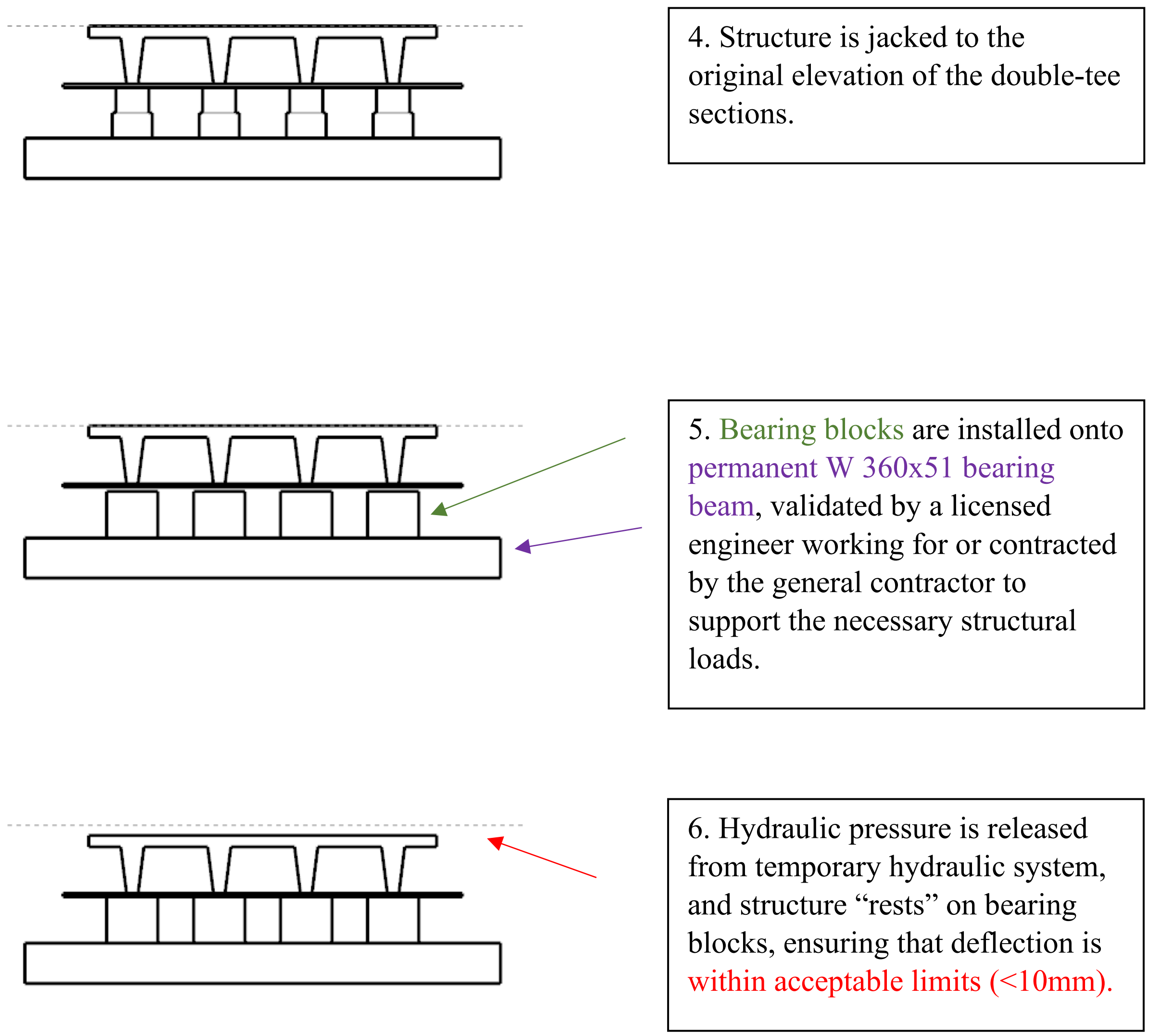

Figure 9.4: Jacking process (2/2)

9.1.5 Landscape Renewal

At the request of the client, our firm has recommended elements of a conceptual landscaping renewal of the space below the pedestrian walkway once the on-ramp and central vertical column are removed. The recommendations will allow for the renewal of a central part of Halifax, encourage pedestrian traffic and drive pedestrian traffic towards nearby commercial units. The recommendations are as follows:

- Only allowing vehicular traffic on Cogswell St. after 9:00PM.
- Including a pedestrian walkway at the Albemarle St. & Cogswell St. intersection protected by a three-way traffic signal

- Constructing a paved park area between the *Apple Self Storage* building and the one-lane Cogswell St., including fountains, benches, games court and ample lighting

Our firm recommends the landscaping works be contracted to a landscaping contractor specializing in clean construction, LEED certified and located in the Greater Halifax Area to promote more sustainable initiatives in the Cogswell Redevelopment Program.

A conceptual 3D model of the landscape renewal is shown in Figure 9.6.

Figure 9.5: Proposed landscape renewal, conceptual

9.2 Cost Estimation & Feasibility

Using the Accelerated Bridge Construction (ABC) Decision Making and Economic Modeling Tool (Doolen & Emami, 2011), developed in 2011 for the use of Federal Highway Administration (FHA), the cost estimates for this project were developed. They are shown in the table below.

Materials	0.83$M
Labour	1.09$M
Equipment	0.92$M
Overhead Costs	0.54$M
Administration	0.51$M
Subtotal	**3.89$M**
Markup (12%)	0.46$M
Total	**4.36$M**

10.0 Conclusions

Throughout the duration of the design project, from the project initiation to the completion, we have learned several precious things. The completion of every project requires professional knowledge from various fields, including the project management methodology, geotechnical information, structure analyzing techniques, and mathematical principles, to effectively come up with a unique engineering solution to the problem. The final design reflects a combination of knowledge from undergraduate courses, researches, and discussions with structural experts and related specialists. At the beginning of the project, it was challenging to come up with a proper initial design to handle the problem as the bridge design had various physical constraints. The ends of the bridge were fixed to adjacent buildings. Property line constraints made it tricky to choose a location for the support columns while still maximizing green space below the bridge. Brainstorming with team members and consulting professors about the feasibility of the design effective help us to create our first preliminary design that would be both easy to work with and reflective of the engineering knowledge. Several design codes were utilized as the base-stone of our design, including the *National Building Code of Canada* (NBCC, 2015), the *Handbook of Steel Construction 11th edition* and *Concrete Design Handbook 3rd edition.* It was learned throughout the project that, instead of arbitrarily creating the design, any completion of successful projects strictly requires the combination of design regulation with industry knowledge in order to make it feasible for construction. Moreover, throughout tremendous adjustments and fixations of the preliminary design, it was learned how design is a very iterative process. For example, our first preliminary design in SAP2000 consisted of both concrete bridge and the steel cage structure. After modeling every component and assigning all necessary loads, we were suggested to simplify the SAP2000 model as the structure was too complex to be accurate. A lot of work was required to take the many iterations of designs and present them in a concise and accurate way for an audience. During the duration of the design, strong and effective communications between team members had greatly facilitated the process of the project. Finally, this design project has been one of the most precious experiences in our lives, the objective stated at the beginning of this semester has been successfully achieved. We are now equipped with skills and experiences that we can bring into many future applications in our careers.

10.1 Design Summary

The primary objective of this project was to design a structural system that can support the existing pedestrian walkway because of the rehabilitation of Cogswell Street that ultimately required the demolition of the existing support column. This was achieved by designing a truss structure surrounding the existing structure and transferring the loads at the existing load transfer point (center diaphragm). This truss structure transferred these loads to 2 outer columns with shallow foundations. Secondary objectives included re-designing the road environment, reducing cost and implementing sustainability initiatives that can be found in this report.

For this design to come to realization, it was determined that firstly, the loads on the existing structure must be found. Then, an initial truss structure was designed that can resist the existing forces. Once the truss design was chosen, the truss connections, support connections and columns were designed. The truss connections were designed by identifying the different types of connections and analyzing the worst-case scenario for each connection. Using these worst-case scenarios, the connections were designed such that they could resist the worst-case applied forces. The support connections were similarly designed, using the worst-case loading scenarios at each support joint. A load transferring mechanism was designed that transferred the compressive and shear loads to the columns. Finally, the columns were designed such that the pile cap, column and shallow foundations could resist induced shear, compressive and moment forces.

These designs were chosen for their efficiency and ability to resist loads. They were also chosen for cost optimization. For example, a typical connection type is used in order to simplify constructability and material purchasing, as well as to decrease the amount of time needed to design each connection.

The project timeline was well respected, and all deadlines had been met. While there were some unexpected design challenges, our group asked for the proper advice from external advisors and professors.

10.2 Closing Remarks & Lessons Learned

"I have always been fascinated by large scale mega civil engineering. Pursuing my degree at McGill university has given me the opportunity to meet exceptional individuals and professors. Working on this project has been such a privilege and I will always remember my academic journey at McGill"

Moied Haddadin

"Learning about different design codes, regulations, methods & procedures in completing this project has opened my eyes about the real meaning of being a design engineer. There is ample work involved, even in completing a student project, but it's rewarding, and I look forward to participating in other projects down the line in my professional career. I've learned that structural projects need to be looked at holistically and am eager to eventually learn to use BIM and live rendering to avoid the natural pitfalls of losing sight of the whole project's scope while designing a specific section.

It has been a pleasure working on the project with my teammates & sharing the design studio space (while it lasted) with other students & members of the faculty, continually learning and adding to my knowledge that I've gained throughout my time at McGill."

Adamo Bernola

"Working as a group on such a complex project for the first time is not an easy task to accomplish. Luckily, my teammates and I all worked very well together, and the work was efficiently distributed. The team was organized, and they communicated their issues and objectives clearly. While it was certainly interesting to apply the engineering and scientific principles learned over the years, it was even more interesting to learn how this knowledge is to be applied in an industrial and professional capacity.

Working and helping on nearly all aspects of the design has certainly opened my eyes to the challenge of understanding and applying structural engineering principles. However, the true challenge of ensuring all design components are well-integrated into the overall structure design is a testament to the overall functionality of a group. Our group applied these engineering principles to our greatest extent and collaborated as efficiently as possible. I look forward to applying and developing my engineering skills in the future, as well as working with and organizing a team that is both competent and eager to learn."

Philippe Allard

"The best way to acquire knowledge is to practice it. Throughout my four-year undergraduate life, I have learned ample theorical knowledge regarding structural engineering. The design project provides a great chance for me to utilize my knowledge of scientific and mathematical principles

to solve technical problems. This undergraduate experience has cultivated strong 'problem-solving skills' as I have always been self-motivated to find solutions to unique problems and to collaborate with my team members. This quality provides me a great opportunity to confront difficulties and enables me to quickly handle the new skills and techniques in my career path. Watching the design of the bridge from zero to the completion provides me a sense of fulfillments. It consolidates my determination to pursue structural engineering in the future."

Loren Yan

"This project has truly been an eye-opener for me. Being able to apply the knowledge and design thinking that we've been practising into a comprehensive project based on a real existing project. I enjoyed working with everybody, planning the design process, applying it and seeing the bridge come to life with the renders. Seeing firsthand the complexity involved with such a project gave me more respect towards the engineering profession and how crucial it is to design with safety in mind as our intricate work will impact tens of thousands of people. I hope this project is one of many exciting projects that I will get to work on in my career. "

Elia Abu-Manneh

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