ENVIRONMENTAL LOAD EVALUATION OF NIPIGON RIVER BRIDGE

by

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Abstract

If current climate trends continue, climate change will be inevitable and designing infrastructure which can withstand changing environmental loads will be a concern. Furthermore, current infrastructure will be affected and may require retrofitting or rehabilitation in order to meet safety and code requirements. The scope of this report is to determine the effect of increased environmental load factor coefficients on Nipigon River Bridge. An FEA model was created and the results from the model show that the bridge is sensitive to changes in environmental loads, particularly those of wind and temperature. An increase of 10% in wind and temperature load coefficients was enough to change the governing load combination and surpass the estimated moment capacities.

Acknowledgements

This report is completed for the fulfillment of my Master's Research Project. The topic of this report was agreed between me and Dr. Amleh, such being the impact of climate change on bridges and how increased snow, wind and temperature loads in the future can increase and damage existing infrastructure. Dr. Othman chose the scope of the report and decided Nipigon River Bridge as the focus. All technical information of the bridge which came in the form of the MTO bearing report was handed to me by Dr. Othman. Missing information was either researched or discussed with Dr. Othman. Use of software and capacity estimation was completed by myself and was submitted to Dr. Amleh and Dr. Othman for review.

I would like the thank Dr. Amleh and Dr. Othman for their support throughout the duration of this project. The completion of this project would not have been possible without their help and technical knowledge. It has been my pleasure to work with both of them.

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1.0 Introduction

Climate change is inevitable with current rates of emissions and engineers must be prepared to design new and existing structures to withstand the changing climate. Nipigon River Bridge is an example of how unaccounted for weather can cause failure in a bridge and upset the local population and economy that depends on it. Nipigon River Bridge is an important connection in the TransCanada highway that connects East and West Canada without any favorable detours. The 100 million dollar bridge failed in January of 2016 and again in May of 2018. The repair costs were about eight to twelve million dollars and took almost two years for all four lanes to be re-opened. Figure 1 illustrates the location of Nipigon River Bridge, located on the Northern side of Lake Superior in Ontario, Canada. Two possible detours are available (i) driving South around Lake Superior is an additional five hour detour (ii) driving around the under-developed roads of around Lake Nipigon takes about an additional ten hours. (Prokopchuk, 2018) (CBC News, 2018)

Figure 1. Nipigon River Bridge location

(Google Maps, 2019)

The bridge failure caused many vehicles to be stuck in traffic for many hours, which left families stranded in their cars during the winter. Transport trucks were unable to meet their destinations, putting pressure on the economy. This bridge is primarily used by trucks to travel across Canada using the TransCanada highway, however, the neighboring cities with a population of many thousands are also heavily dependent on the bridge. Therefore, it is critical that a bridge such as this one be designed to withstand climate loads, particularly because detours are not a reasonable option. (Prokopchuk, 2018)

Failure of Nipigon River Bridge is reported as the neglect to correctly tighten the bolts which anchor the cables to the bridge deck, and the combination of temperature shrinkage and wind excited oscillations of the cables. The high variance of temperatures which the Nipigon region experiences, causes a large degree of shrinkage and expansion demands on the cables. Additionally, the bridge is situated close to the open lake without any wind protections, which puts a large wind load on the cables, particularly during the winter months. A changing climate with high temperature variability is partly the cause for the failure. (CBC News, 2018)

Warming of the planet is particularly noticeable in Canada because of its proximity to the North Pole which has undergone extreme changes in the last few decades. The effect of climate change is a less predictable weather pattern that does not follow the historical data and extreme weather is being reported more frequently. The Jetstream that covers the globe runs across parts of Canada and America which separates the cold arctic air from the warm tropical air. The Jetstream has been weakened due to rising temperatures, slowing it down, and causing it to oscillate more North and South which can cause areas used to warmer weather to experience colder temperatures, and colder climates to receive bursts of warm tropical air at unexpected times in the year. The mixing of hot and cold air creates more extreme weather patterns which the Jetstream once separated. Furthermore, the overall increase in temperatures in Canada has led to soil which is typically frozen to thaw and soften. This softening which many buildings rest on can cause settling and other issues related to the foundations. The stiffness of frozen soil is typically four to eight times larger than that of thawed soil. Thus, design changes for future infrastructure and updating of current structures are necessary for a changing climate. (Mortillaro, 2018)

2.0 General Bridge Information and Assumptions

This report is based off the Ministry of Transportation Highway Standards Branch Office Report, Nipigon River Bridge West Abutment Bearing Technical Investigation (2016). All technical information regarding the bridge comes from this publication, and all remaining missing information was assumed such that it falls within reasonable values. The elevation view of the bridge is provided in figure 2. The general bridge specifications are listed in table 1.

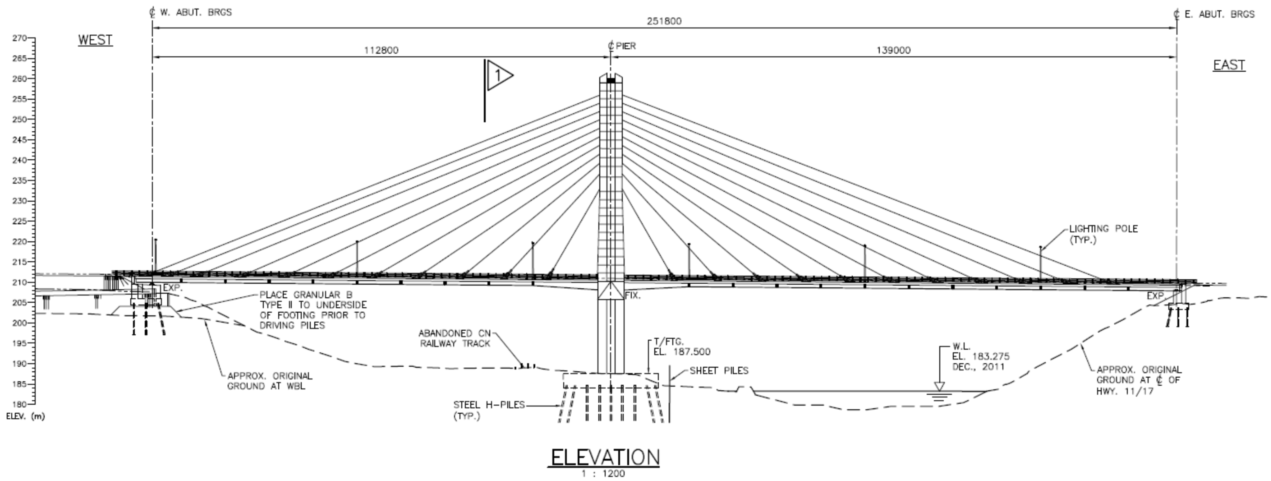

Figure 2. Elevation View of Nipigon River Bridge

(MTO, 2016)

General Information

1.0 General Specs

Number of Spans	2	
West Span Length	112.8	m
East Span Length	139	m
Deck Type	Precast Concrete on Steel Girder	
Number of Towers	3	
Cable Arrangement	Fan	

2.0 Deck

Width	16.03	m
Concrete Thickness	0.225	m
Lane Width	3.75	m

2.1 Girder

Number of Girders	3	
Flange Width	0.63	m
Flange Thickness	0.03	m
Web Length (inner)	1.2	m
Web Thickness	0.015	m

2.2 Diaphragm

Type	Steel Plate

	Length	1.2	m	*
	Width	0.03	m	*
	Spacing	3.6	m	**
3.0	*Pylon*			
	Type	Reinforced Concrete		
	Bottom Tower With	1.01	m	
	Bottom Tower Length	6.6	m	
	Top Tower With	1.01	m	
	Top Tower Length	5.5	m	
	Tower Height Above Deck	49	m	
	Tower Height Below Deck	22.5	m	
4.0	*Cable*			
	Min Tensile Yield Stress	2000	MPa	
	Elastic Modulus	200,000	MPa	
	Cable Diameter	0.2	m	*
	Largest Cable Tension	8172	kN	*
	Spacing	10.8	m	**
5.0	*Foundation*			
	Type	Steel Pile		

* assumed values

** dimension varies across length / width of bridge

Table 1. General Nipigon River Bridge Information

The cable dimensions were assumed based off typical cable bridge dimensions. Dongting River Bridge is a cable stayed bridge with a cable diameter with 0.199 m, an initial cable tension of 3150 kN and supports main two spans of 310 m. Therefore, the information shown in table 1 in regards to a similar bridge such as this one is not unreasonable. Additionally, the material properties of the cables are within the industry norm. The spacing of the cables along the tower were inferred from the scale shown in the elevation view in figure 1. Spacing of the diaphragms was explicitly stated, but the locations of the cable-deck connections was not, instead, a close estimate of 10.8 m was inferred from the drawings. It is typical for cable bridges to choose a cable spacing of about 10 m along the length of the deck. (Ni, et al, 2007)

3.0 Procedure

A model of the bridge was created using CSiBridge and a non-linear static and modal analysis was completed using a spine model. The load combinations used are shown in table 2. Omissions in the model include the slope of the deck both laterally and transversely. The load distribution method as described in the MTO report uses the outermost cable on the West side of the bridge as an anchor by pulling the cable into the abutment. When the truck is on the East span, the load is carried through by the nearest cables and the load is transferred to the outermost cable where it is attached to the abutment as shown in figure 3. Trucks located on the West span transfer the load to the nearest back stays which compress (but remain in tension) while the remainder of the load is transmitted to the girders and tower. Tension in the back stays is transferred to the girder using a fin plate. This bridge behaviour was not directly implemented in the CSiBridge model, instead the West abutment was placed directly beneath the outermost cable without the cable-abutment attachment. The cable-abutment connection is a method for creating a fixed connection, therefore this is not necessary in the model in the since the abutment in the software is modelled as a perfectly fixed connection.

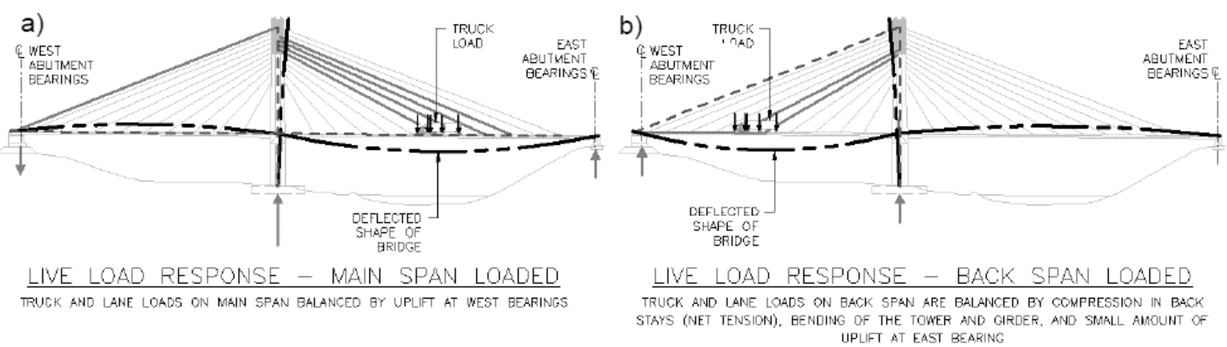

Figure 3. Bridge behaviour of truck passing on (a) East span (b) West span

(MTO, 2016)

Two analyses were completed to determine the effect of snow loading on the bridge. First a baseline analysis was completed to determine the current governing load combination, and a second analysis with elevated environmental loads. The baseline analysis is necessary because of missing information during the modelling of the bridge. Rebar spacing and sizes were not provided so the positive and negative moment capacities were designed based off the factored moments in the model. Calculations for moment and shear were completed to ensure that the results of model were reasonable and comply with the Canadian Highway Bridge Design Code (CHBDC). Moment and shear design calculations are provided in Appendix A, and material and bridge properties are located in Appendix B.

Elevated environmental load analysis was completed and compared with the baseline results. Environmental loads in this report is referred to as the ice, wind, and temperature loads. The comparison between elevated environmental load factors and baseline results serves to determine the degree of importance that environmental loads play on the bridge.

3.1 Baseline Analysis

The baseline analysis was completed with the four load combinations shown in table 2 which come from the CHBDC. All load combinations which include ice and temperature loads factors were chosen, along with load combination 1. Load combination 1 was also included to see if the ice and wind load factors are of significance to the governing load combination. In the bridge model, the wind loads used to describe the winds experienced by Nipigon River Bridge came from Thunder Bay data which is the nearest city with wind recordings to the bridge. It is however likely that Nipigon River Bridge experiences higher winds due to the shape of the surrounding area and the river beneath it which creates a very exposed landscape from where strong winds over the lake can interact with the bridge. The 100 year wind pressure that the bridge experiences according to Thunder Bay data is 430 Pa. (CHBDC, 2014)

	Factor				
Combination	**α_D**	**α_L**	**α_W**	**α_K**	**α_A**
1	1.20	1.50	0.00	0.00	0.00
2	1.20	1.25	0.45	1.00	0.00
3	1.20	0.00	1.40	1.25	0.00
4	1.20	0.00	0.75	0.00	1.30

Table 2. Load combinations

(CHBDC, 2014)

3.2 Elevated Environmental Load Analysis

The ice, wind, and temperature loads factor coefficients were increased by 50%, 20%, and 10% for load combinations 2 to 4. Each load was increased by the same percentage. A more detailed analysis is necessary to determine which of the load factors is the most sensitive to the bridge.

4.0 Baseline Results

Results from the baseline analysis show that the governing load combination is number 2. It was initially believed that load combination 1 would be the governing combination. This indicates that the bridge is sensitive to changes in temperature and wind which is typical for cable stayed bridges. The maximum moment, shear, and deflections are provided in table 3.

	Positive Moment (kNm)	**Negative Moment (kNm)**	**Shear (kN)**
Maximum	31116	33653	13865
Capacity	31263	34097	17010

Table 3. Baseline analysis results with load combination 2

The capacities in the table are close to the maximum possible capacities that can be achieved with the provided design according to the hand calculations in Appendix A. This is a good indicator that the bridge model is reliable. It is however possible that the real capacities of the bridge are different due to missing information in the bearing report. These capacities serve only check to the baseline analysis results.

Currently, the CHBDC does not specify the maximum allowable deflection of cable-stayed bridges. Thus, the AASHTO design code was used for to provide the maximum allowable deflection which is stated in equation 1, where L is the span length. The service dead and live load cannot exceed the prescribed limit.

$$\boldsymbol{\delta}_{\boldsymbol{max}} = \frac{\boldsymbol{L}}{\mathbf{400}} \tag{1}$$

$$0.348m = \frac{139}{400}, 0.282 = \frac{112.8}{400}$$

$$\boldsymbol{\delta}_{\boldsymbol{analysis\ East\ span}} = 0.248m < 0.348m$$

$$\boldsymbol{\delta}_{\boldsymbol{analysis\ West\ span}} = 0.206m < 0.282m$$

The analysis deflection is 71% and 73% of the maximum allowable deflection for the East and West spans, respectively. Service and live load deflection of the bridge is shown in figure 4.

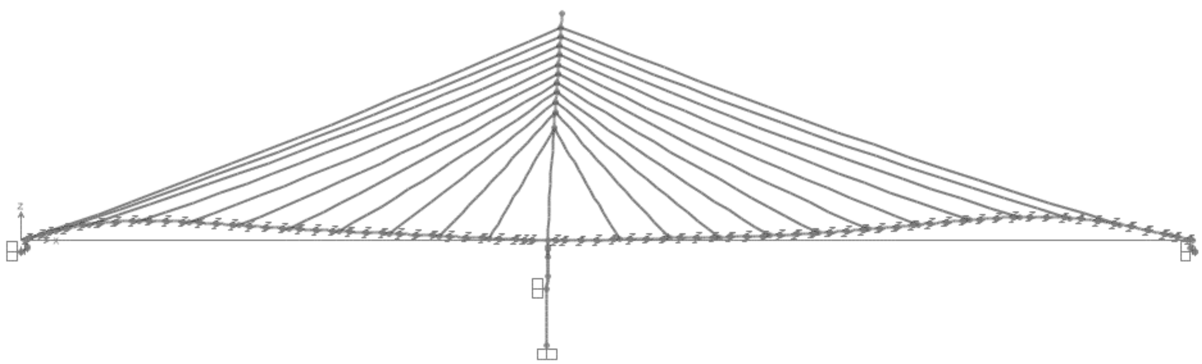

Figure 4. Span deflection under service dead and live load

(MTO, 2016)

5.0 Elevated Environmental Load Factor Results

Results from the elevated environmental loads are summarized in table 4, where the moments and shear results are shown for each percent increase. Therefore, the new governing load combination, is combination 3 for all percent increases. This combination has the factor coefficients for both wind and temperature loads, and does not take live load into consideration. These findings indicate that the bridge is very sensitive to increases in wind and temperature loading as quantified in table 5. Table 5 shows the percentage increase of moment and shear with respect to be baseline results for each percentage increase in environmental loading. Thus, as shown in table 5, there is a positive non-linear relationship between environmental load and positive moment. A smaller percent increase is experienced by negative moment and shear for every environmental load percent increase.

Percent	Positive Moment (kNm)			Negative Moment (kNm)			Shear (kN)		
Increase	Comb 2	Comb 3	Comb 4	Comb 2	Comb 3	Comb 4	Comb 2	Comb 3	Comb 4
50	43710	53457	37614	36498	38644	35123	15606	16920	14769
20	36070	39364	26949	34749	35497	32649	14549	15004	13280
10	33646	34856	23878	34197	34472	31849	14214	14381	12799

Table 4. Results of environmental load factor increase

Percent Increase	Positive Moment (kNm)	Negative Moment (kNm)	Shear (kN)
50	71.8	14.8	22.0
20	26.5	5.5	8.2
10	12.0	2.4	3.7

Table 5. Percent increase in moment and shear per percent increase in environmental load coefficient

6.0 Discussion

If the findings in table 4 are compared with the capacities in table 3, the shear capacity of the bridge is large enough to carry even a 50% increase in environmental loading. However, even a 10% increase in environmental loading surpasses the estimated positive and negative moment capacities. The analysis of the bridge began with a 50% increase in environmental loading to see if environmental loading is significant. In the case that the results remained the same, it could be concluded that environmental loading is insignificant because it is unreasonable to believe that the environmental loading would increase by 50%. Thereafter, the 20% and 10% environmental load increase was completed. Since the governing load combination changed, it can be concluded that even a reasonable 10% increase in environmental loading can have a significant impact the bridge. Even though the negative moment capacity was surpassed by a 10% load increase, it is possible that the actual capacity of the bridge is large enough to carry the moment. However, the positive moment increased by a larger percentage and surpassed the estimated capacity by 3,593 kNm. This is a substantial increase in moment it is possible that the actual positive moment capacity of the bridge is insufficient to carry the load.

The findings of this report match closely with the bridge failures reported in 2016. Large positive moments are located on the East span, and large negative moments are located on the West span as shown in figure 5a. The deflection diagram is shown in figure 5b to better illustrate the bridge behaviour. All absolute maximum moments occur at the same bridge locations for all load combinations at all percent environmental load coefficient increases. The effect of the bridge failure caused the West span deck to lift due to improper tightening of the cable anchors. It is possible that when truck loading on the East span caused the cable tension to increase over the West span, thereby increasing the negative moment. However, an inconsistency with the findings of this report with the bridge failure is that the bridge is more susceptible to increases in positive moment rather than negative moment.

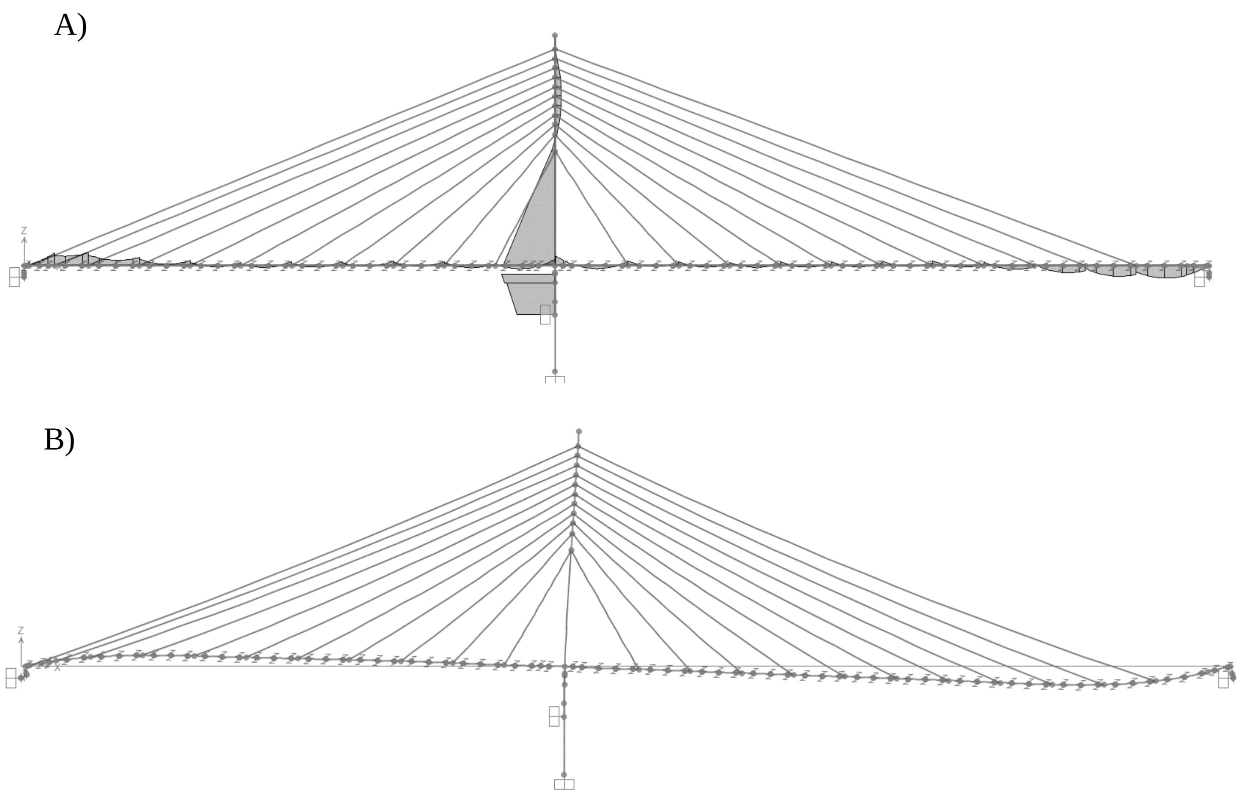

Figure 5. Load combination 2 (a) moment diagram (b) deflection diagram

Cable tensions were not commented on in this report since the bridge failure was not due to material failure. Additionally, the findings in the results indicate that the cable tensile stress does not exceed the maximum allowable yield stress for all load combinations and increases in environmental load coefficients. It was assumed that the towers, and abutments have enough capacity to withstand all loads and that bridge failure will first occur due to moment and shear. A more sensitive analysis is required for all connections and welds in order to accurately determine the governing failure method.

7.0 Conclusion

This report aims to determine the effect of environmental load increase on Nipigon River Bridge and report if the findings are of concern due to climate change. It can be seen from the analysis results that even a small increase in environmental load factors can cause a relatively large increase in moment demand. The results of this report can be summarized as follows.

- Environmental load factor coefficients were increased by 10%, 20%, and 50%, and for all load combinations, and the load governing load combination changed from combination 2 to combination 3.
- Estimated shear capacity is not exceeded for all percent environmental load coefficient increases.
- Percent positive moment increases more per percent increase of environmental load factor coefficient, but percent negative moment increases less per percent increase of environmental load factor coefficient.
- Estimated positive and negative moment capacities are exceeded for all percent load factor coefficient increases for governing load combination 3.
- The results of the model show that the bridge is susceptible to a reasonable 10% increase in environmental load factor.

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Appendix A – Estimated Moment and Shear Capacity Calculations

Concrete

f'_c		25	MPa
α_1	$0.85-0.0015f'_c$	0.81	
φ_c		0.65	

Steel

F_y		350	MPa
φ_s		0.9	
A_s		55800	mm^2
b		630	mm
t		30	mm
w		15	mm

Positive Moment Capacity

T - Beam

Case 1	Assume NA is in Concrete a<t			
b	$\min(0.25L, (S_1+S_2)/2)$	16030	mm	
a	$\varphi_s F_y A_s/(\alpha_1 \varphi_c f'_c b)$	83	mm	**OK**
T_r	$\varphi_s F_y A_s$	17577	kN	
M_{rc1}	$T_r(t_{sl}-a/2+d/2)$	10871	kNm	

L-Beam

Case 1	Assume NA is in Concrete a<t			
b	$\min(0.1L, 0.5S_{clear}+b_f)$	8330	mm	
a	$\varphi_s F_y A_s/(\alpha_1 \varphi_c f'_c b)$	160	mm	**OK**
T_r	$\varphi_s F_y A_s$	17577	kN	
M_{rc2}	$T_r(t_{sl}-a/2+d/2)$	10196	kNm	

Total M_{rc}	$M_{rc1} + 2M_{rc2}$	**31263**	kNm

Shear Capacity

V_r	$\varphi_s A_w F_s$	5670	kN
Total V_r	$3V_r$	**17010**	kN

Concrete

f'_c		25	MPa
α_1	$0.85-0.0015f'_c$	0.81	
φ_c		0.65	
t_{sl}		225	mm

Reinforcement

Bar size		35	M
Bar area		1000	mm^2
φ_r		0.9	

Steel Girder

F_y		350	MPa
φ_s		0.9	
A_s		55800	mm^2
d		1200	mm
b		630	mm
t		30	mm
w		15	mm

Negative Moment Capacity

T - Beam

b	$\min(0.25L, (S_1+S_2)/2)$	16030	mm	
Bar spacing		400	mm	
A_r	$A_r b/\text{spacing}$	40075	mm^2	
Case 1	Plastic NA in flange of steel section			
T'_r	$\varphi_r F_{yr} A_r$	12623625	N	
T_r	$(\varphi_r F_{yr} A_r - T'_r)/2$	2476688	N	
A_{cr}	$C_r/(\varphi_s F_y)$	7863	mm^2	
d_{cr}	$(1/w)[C_r/(\varphi_s F_y-(b-w)t]$	12	mm	**OK**
y_{cr}	$(b-w)t^2+w(d_{cr})/(2A_r)$	6	mm	
y_{tr}	$(A_s d/2-A_{cr} y_{cr})/(A_s-A_{cr})$	697	mm	
e	$y_{tr}-y_{cr}$	691	mm	
e'	$y_{tr}-t_{sl}/2$	810	mm	
M_{rc1}	$T_r e+T'_r e'$	11935	kNm	

L-Beam

b	min(0.25L, $(S_1+S_2)/2$)	8330	mm	
Bar spacing		155	mm	
A_r	A_rb/spacing	53742	mm^2	
Case 1	Plastic NA in flange of steel section			
T'_r	$\varphi_r F_{yr} A_r$	16928709	N	
T_r	$(\varphi_r F_{yr} A_r - T'_r)/2$	324145	N	
A_{cr}	$C_r/(\varphi_s F_y)$	1029	mm^2	
d_{cr}	$(1/w)[C_r/(\varphi_s F_y-(b-w)t]$	2	mm	**OK**
y_{cr}	$(b-w)t^2+w(d_{cr})/(2A_r)$	1	mm	
y_{tr}	$(A_s d/2-A_{cr} y_{cr})/(A_s-A_{cr})$	611	mm	
e	$y_{tr}-y_{cr}$	610	mm	
e'	$y_{tr}-t_{sl}/2$	689	mm	
M_{rc2}	$T_r e+T'_r e'$	11858	kNm	
Total M_{rc}	$M_{rc1} + 2M_{rc2}$	**34097**	kNm	

Appendix B – Material Properties

Steel Properties

Material	Fy	Fu	EffFy	EffFu
Text	KN/m2	KN/m2	KN/m2	KN/m2
A709Gr50	344738	448159	379212	492975
A992Fy50	344738	448159	379212	492975

Concrete Properties

Material	Fc	eFc	LtWtConc
Text	KN/m2	KN/m2	Yes/No
4000Psi	27579	27579	No

Cable Properties

CableSect	Material	Diameter	Area	TorsConst	I	AS	TotalWt	TotalMass
Text	Text	m	m2	m4	m4	m2	KN	KN-s2/m
Cable	A992Fy50	0.2	0.031416	0.000157	0.000079	0.02827	12353	1260

Tower Dimensions

Cross-Section	Length	Width
Text	m	m
Tower Bottom	6.60	5.5
Tower Top	1.01	1.01