Burlington Canal Lift Bridge
Analysis of Load-Carrying Capacity

Prepared for:

Public Works and Government Services Canada

Prepared by:

Delcan

October 2007

625 Cochrane Drive, Suite 500, Markham, ON Canada L3R 9R9
Tel: 905.943.0500 • Fax: 905.943.0400
www.delcan.com

October 18, 2007 OUR REF:BT3252

Mr. Abbas Khan, P.Eng.
Project Manager
PWGSC, Real Property Services - Ontario Region
4900 Yonge Steet
Toronto, ON M2N 6A6

Dear Sir

**Re: Burlington Canal Lift Bridge
Report on Analysis of Load-Carrying Capacity / MTO Overweight Vehicle Permitting Process**

Introduction

The Burlington Canal Lift Bridge was designed and constructed between 1958 to 1962 as a structure serving both road and rail traffic over the canal. The bridge has since been modified to accommodate four lanes of road traffic. The design was completed in accordance with road and rail design codes applicable at that time.

From time to time a number of oversized and overweight vehicles have been noticed crossing the bridge by PWGSC. These overweight vehicles are not currently authorized by the Ontario Ministry of Transportation (MTO) to use the crossing and the Ministry has flagged the structure to allow Highway Traffic Act (HTA) loading only as part of their permitting process. The HTA loading is also referred to as the legal load limit and is comparable to the current design truck loading. However, the Ministry does not have any means to ensure that the vehicles adhere to the route specified in the permit, and since it is possible to access the QEW from either side of the bridge, these vehicles are occasionally crossing the structure instead of using the QEW Burlington Skyway structure.

As a result of the observed exceptional vehicles using the bridge on an occasional basis, PWGSC requested Delcan to conduct an analysis of the bridge in order to determine the load-carrying capacity of the structure as well as to investigate MTO's current permitting practices with respect to overweight vehicles.

Analysis of Load-Carrying Capacity of the Bridge

A three-dimensional finite element analysis (3D FEA) model of the structure was created and analyzed. The model comprises of the main lift span, tower spans and approach spans. The geometric properties and material properties of the structural steel members and concrete deck slabs have been defined according to the original contract drawings dated in

1958, and the modification drawings dated in 1982. The steel grating was replaced in 2000 and conforms to the prevailing design code at that time. The results from the analysis have been compared with the structural section capacities calculated based on the original contract drawings and the material properties specified in the Canadian Highway Bridge Design Code (CHBDC) based on the year of construction.

Three main live load cases were considered in the analysis. The first two cases consisted of a design truck / lane load positioned in either the outside or inside lanes with the remaining lanes having a reduced loading based on the probability of consecutively loaded lanes in accordance with the CHBDC. The CHBDC design truck GVW is equivalent to the maximum legal GVW of 63,500 kg. The third case considered a single vehicle traveling down the centerline of the bridge with a GVW of 120,000kg and the same axle spacing as the design truck. The ultimate live load carrying capacity of the structure was derived from these load cases by determining the reserve capacity above the applied loading.

In order to ensure representative modeling of the bridge, the 3D FEA model was calibrated and tested using the self weight of the structure. The force effects of the truss members, obtained from the analysis, were compared with the stresses which were included on the original contract drawings, and these values have been found generally to be in agreement. The outputs from the three load cases described above have been compared with the calculated member capacities and the load carrying capacities of the various elements were derived.

It has been found that the main truss members (chords, diagonals, verticals) and floor beams of the truss and tower spans have significant reserve capacity as a result of the original design accounting for railway loading, while the concrete deck slab in the tower spans and approach spans have adequate capacities and were found to be the governing elements of the bridge with minimal reserve capacity. Hence, based on the analysis performed, it was found that the structure is adequate to resist the current design truck loading, with the stringers and deck elements of the tower and approach spans being the most sensitive to increased axle loads and spacing.

PWGSC has noted that the welds connecting the steel grating to the stringers are routinely breaking under current vehicle loads. This issue has been noted to occur with routine traffic and may or may not be an issue with overweight vehicles depending on the actual distribution of the loads. The cracking of the welds to the stringers does not affect the structural performance of the grating, and since it is occurring under current highway loading, this issue cannot be directly linked to overweight permitted vehicles and should continue to be considered as a maintenance issue rather than a load capacity issue. However, welds within the steel grating are also noted to be breaking which can lead to reduced load carrying capacity of the grating if not addressed with regular maintenance repairs. These weld failures have been noted in the annual inspection report as a result of metal fatigue from truck loads, and hence the presence of occasional overweight vehicles is not considered to be a contributing concern, provided ongoing maintenance is addressed.

MTO Oversize / Overweight Permitting Practices

For MTO permitting purposes, vehicles exceeding the maximum legal GVW of 63,500 kg but less than 120,000 kg, are accommodated by evaluating the load effect of the actual vehicle configuration identified in the permit application by the applicant (Carrier). Based on detailed information of the axle spacing and loading, the permit vehicle is compared to the legal load limit using the modified Ontario bridge formula which compares the load effect of the actual vehicle configuration against the legal weight. The modified Ontario bridge formula was developed by MTO to assess the impact of different vehicle configurations and has been used as the basis to develop the legal load configurations provided in the current HTA. Higher GVW loads do not necessarily produce larger load effects in structures compared to the CHBDC design truck since the load is typically distributed along a longer vehicle length to keep axle and tire loads within acceptable limits.

For a vehicle that has higher weight than the maximum allowable GVW for its configuration using the modified formula, additional allowance factors are also considered to utilize the safety factors built into the bridge design. These factors take into consideration controlling the operation of the permitted vehicle, limiting traffic on the structure at the same time, adjusting the magnitude of the impact load (a function of vehicle speed), and the location of the vehicle on multi-lane structures.

Using the modified Ontario bridge formula in conjunction with certain allowance factors defined by MTO as part of the permit review process, MTO issues permits for vehicles up to a maximum GVW of 120,000 kg on their bridge structures which meet current highway traffic act loading with the appropriate traffic control and travel restrictions. Based on the above system, it should be noted that police escort is typically required for GVW in excess of 100,000 kg where strict speed and traffic control are required to be enforced.

MTO classifies GVW in excess of 120,000 kg to be "superloads" and require the Carrier to satisfactorily justify the moves, and to produce independent detailed engineering analysis and evaluation of all structures along the route. MTO also escorts these "superloads" to ensure strict compliance with the permit and route restrictions.

Conclusions and Recommendations

The Burlington Canal Lift Bridge has been found to be structurally adequate under the design truck load, lane load and maximum wheel load, as specified by the CHBDC. The main load carrying elements of the bridge can sustain larger loads than the design vehicle; however, the deck and stringers in all spans are sensitive to localized increases in vehicle loadings.

The MTO issues permits for Carriers with GVW ranging from the legal weight of 63,500 kg to a maximum weight of 120,000 kg, which is almost double the legal weight. These are justified by the MTO on a rational analysis basis and can be shown to produce a load effect on the structure well within the design limitations for these structures.

It is understood that PWGSC is concerned with the reliability of the overweight loadings which are given in applications to obtain a permit, on account of the fact that the MTO does not have any proactive measures currently in place to ensure the permitted loadings are adhered to strictly. Accordingly, it is considered prudent that restrictions on permitted overweight vehicles crossing the Burlington Canal Lift Bridge continue to apply.

In order to better achieve the restriction of overweight vehicles and the awareness of this restriction by Carriers, we recommend that PWGSC consider one or both of the following two options:

- Request of MTO that the current restriction on MTO's database, which limits the allowable truck loadings over the Burlington Canal Lift Bridge to the legal GVW limit identified by the Highway Traffic Act, be maintained.
- Post the bridge with an appropriate sign to increase awareness and reinforce this restriction.

If PWGSC chooses to place signage on the structure, the following wording would be suggested: "MTO PERMITTED OVERWEIGHT VEHICLES NOT ALLOWED ON STRUCTURE". This conveys the message to the specific Carriers with permits without raising concern to the general public using the bridge, which may perceive that the structure may be inadequate or unsafe if a maximum load posting was used.

It should be noted that signage of the structure will not prevent overweight vehicles from crossing the Burlington Canal Lift Bridge, but is considered to be a reasonable effort to increase awareness and educate Carriers that they are not adhering to the restrictions of their MTO permit.

We trust the above report is satisfactory for your immediate needs. If you have questions concerning the report contained herein, please do not hesitate to call.

Yours very truly,

Brent Archibald, P. Eng.
Technical Director

Karen Liu, P.Eng.
Structural Engineer

c: Joanne Crabb

:BIA
J:\TOR\BT3252\Docs\BTJ Load Carrying Anaylsis\01bia001_ak_draft.doc

APPENDICES

Appendix A – General Arrangement

Appendix B – Finite Element Model

Appendix C – FEA Output

Appendix D – Capacity Calculation Notes

APPENDIX A

GENERAL ARRANGEMENT

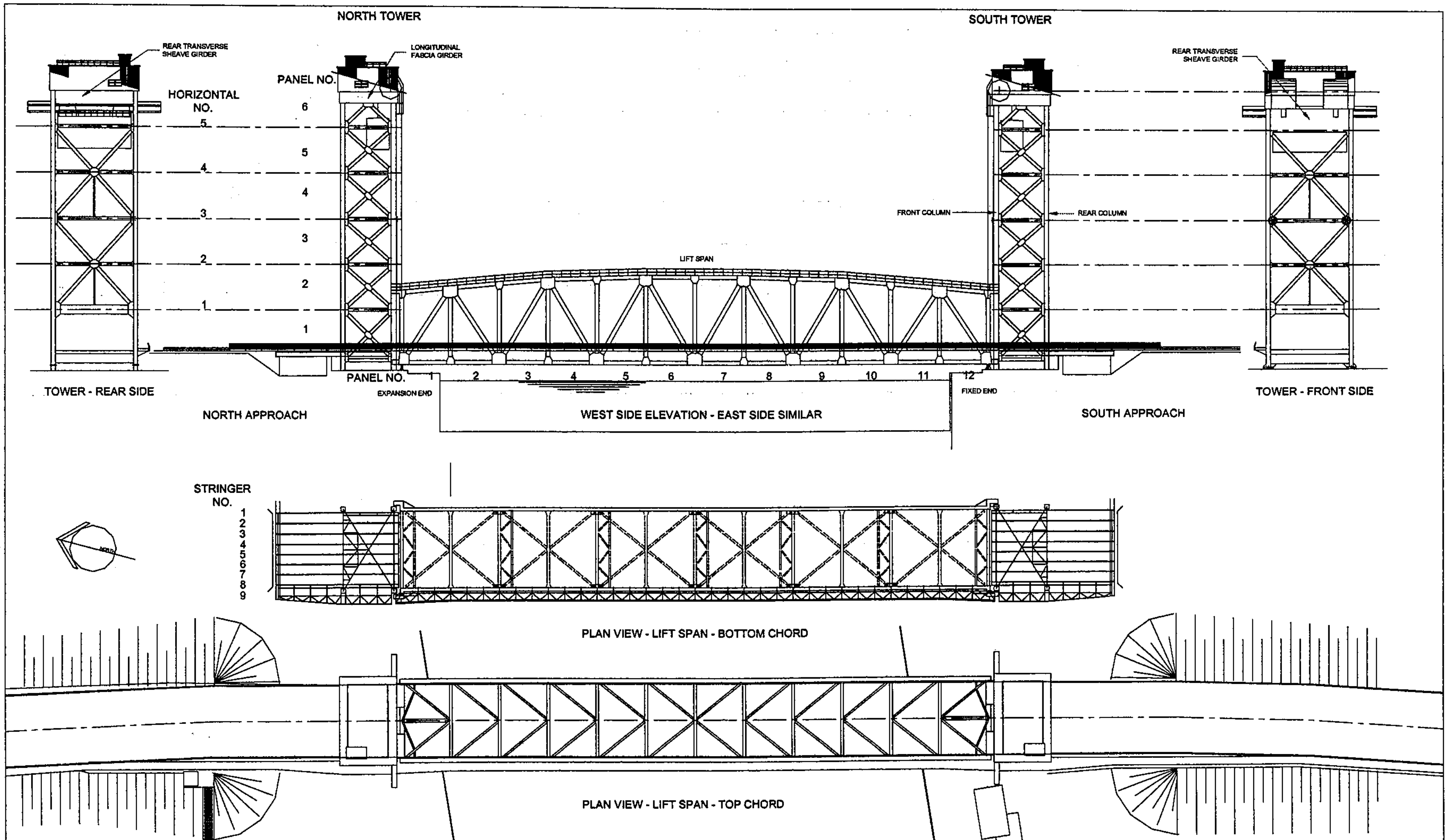


Fig. 1 - GENERAL ARRANGEMENT

APPENDIX B

FINITE ELEMENT MODEL

Title: BCLB Analysis of Load-Carrying Capacity

Model Units: N,m,kg,s,C
Report Units: N,m,kg,s,C

Model Title: Burlington Bridge
Model File: BCLB

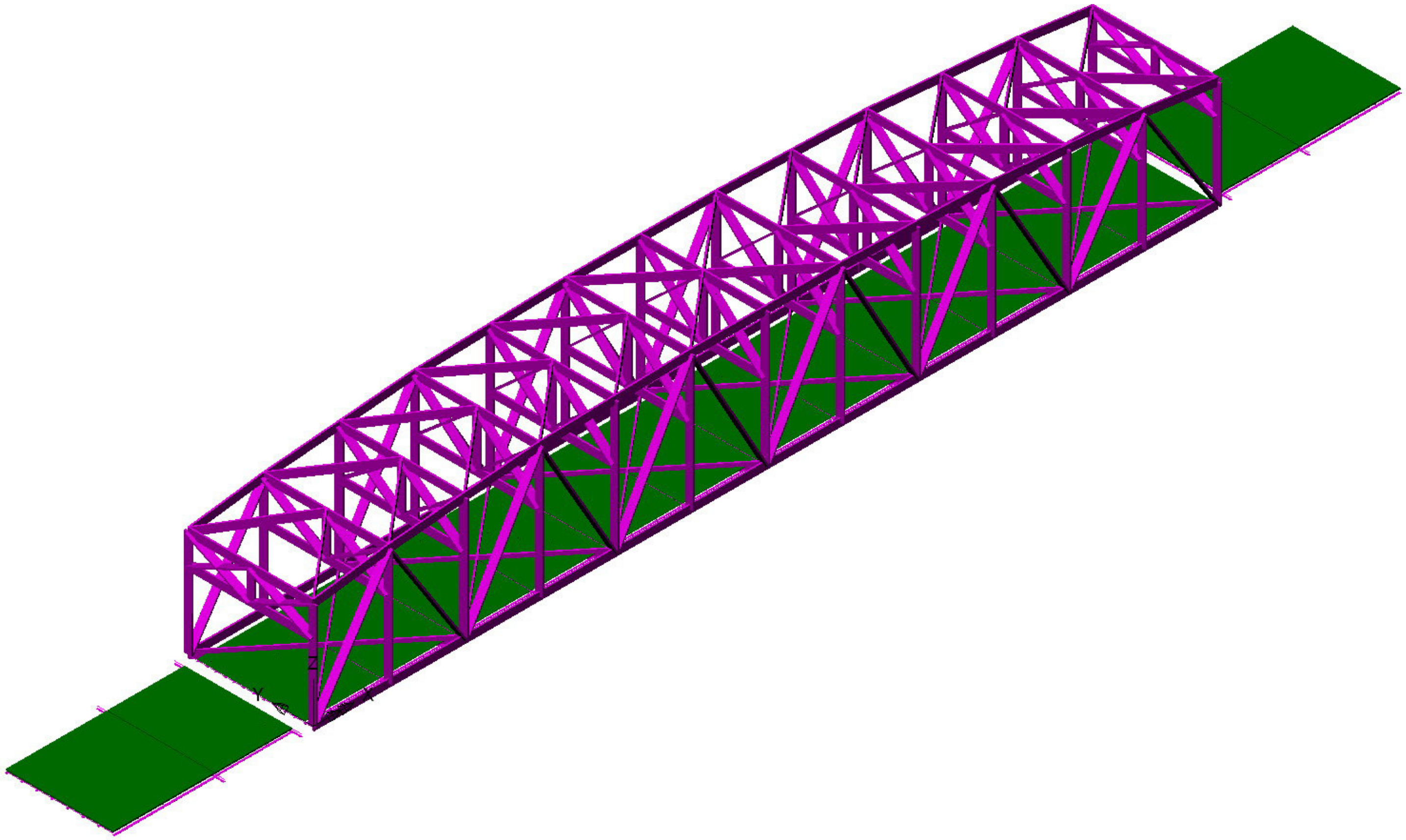


Figure B01: 3D FEA Model Overview of Lift Span, Deck Structure for Tower and Approach Spans

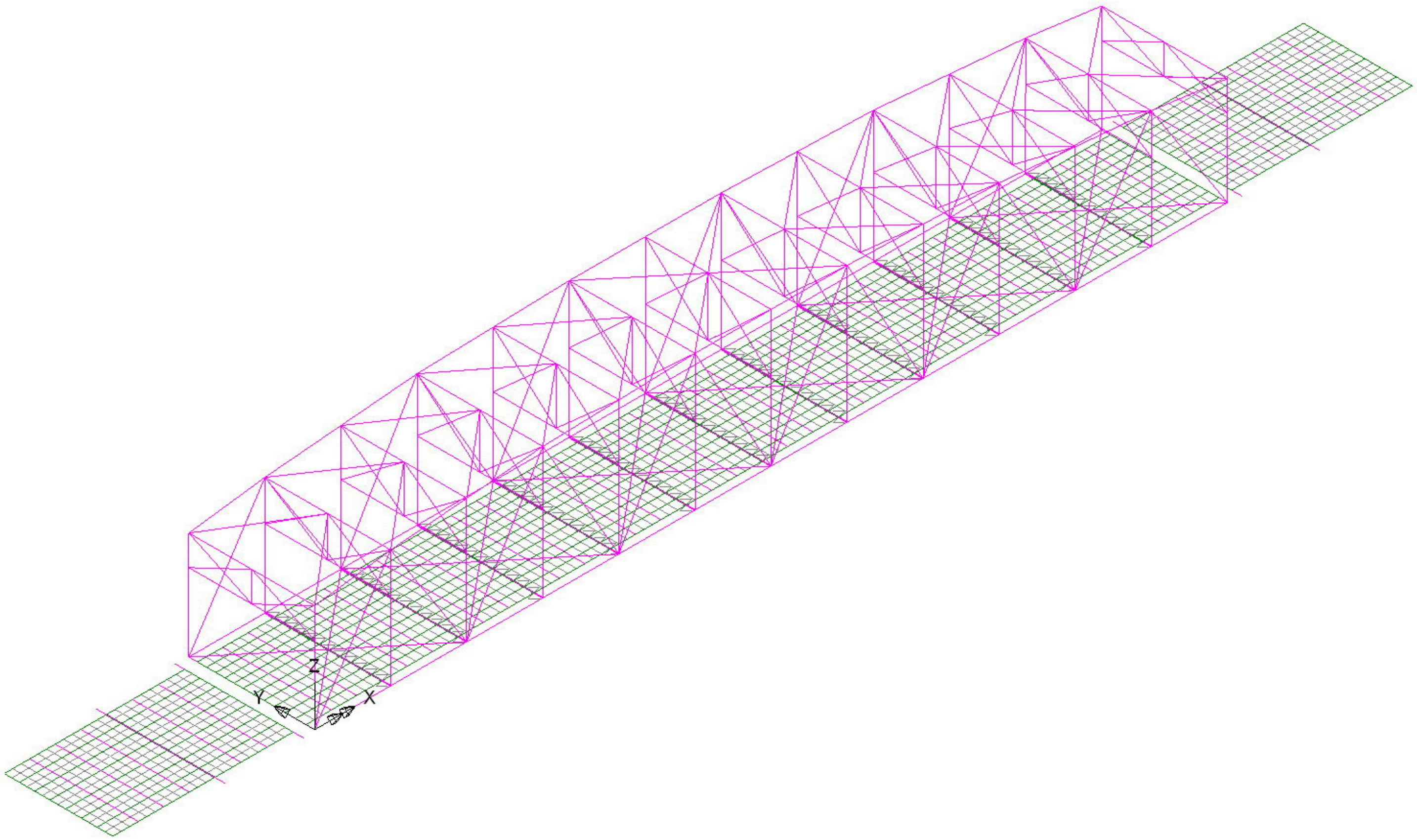


Figure B02: 3D FEA Model Overview Showing Element Mesh

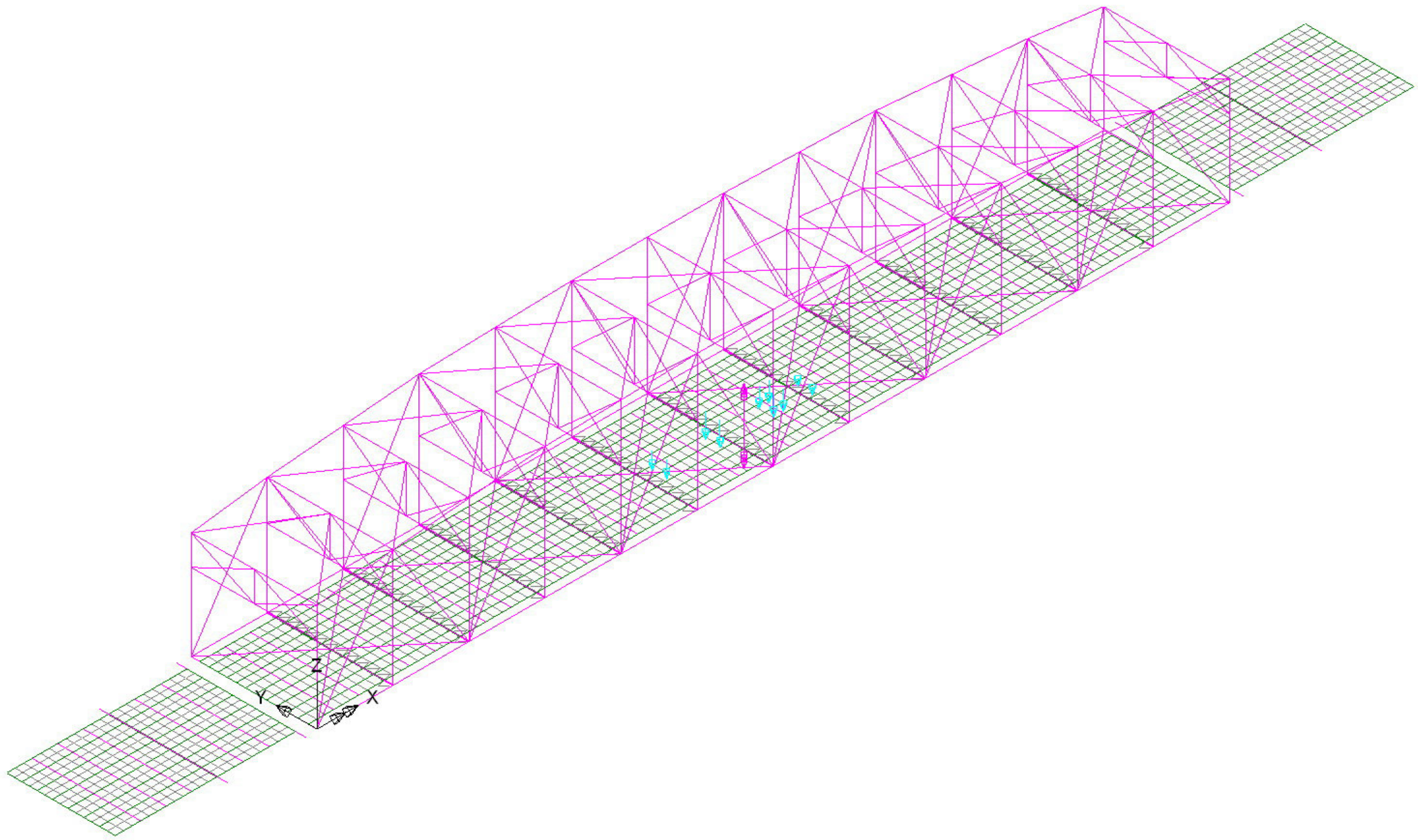


Figure B03: 3D FEA Model Showing Single Truck Loading Condition Applied Directly onto Deck Surface

APPENDIX C

FEA OUTPUT

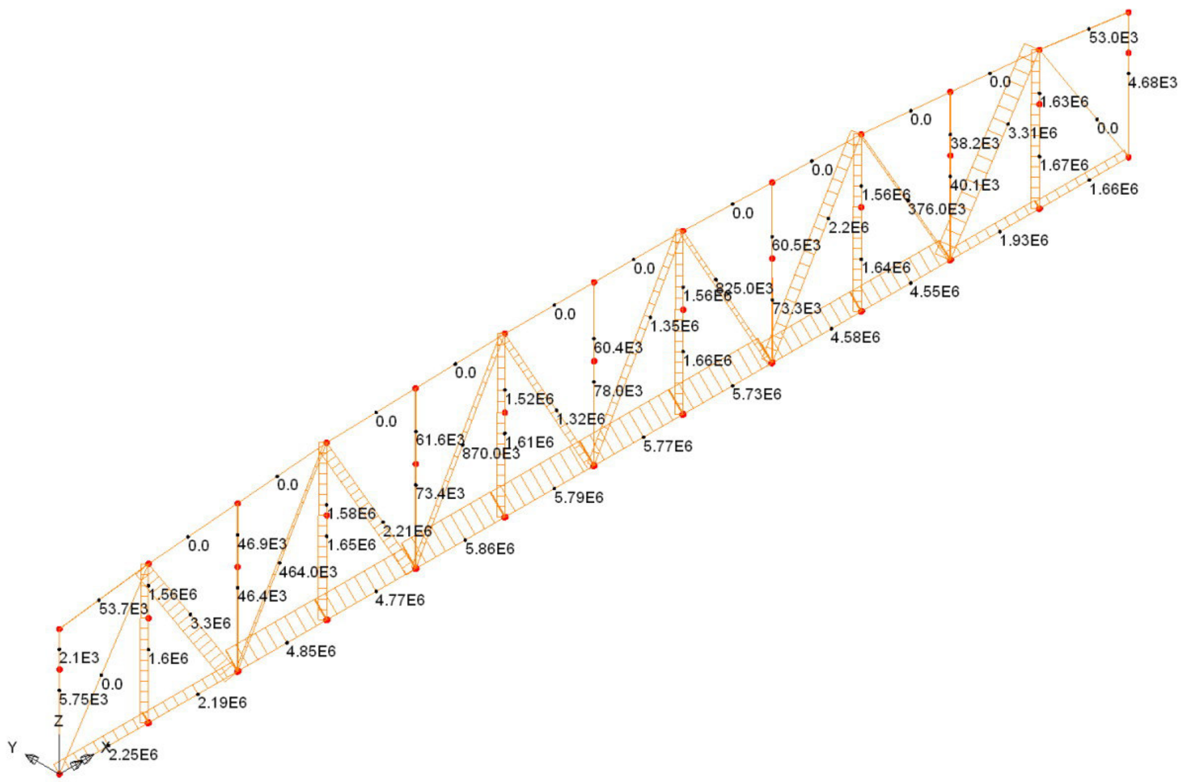


Figure C01: Lift Span Highway Truss Members Axial Forces-Combination 4-Maximum Tension

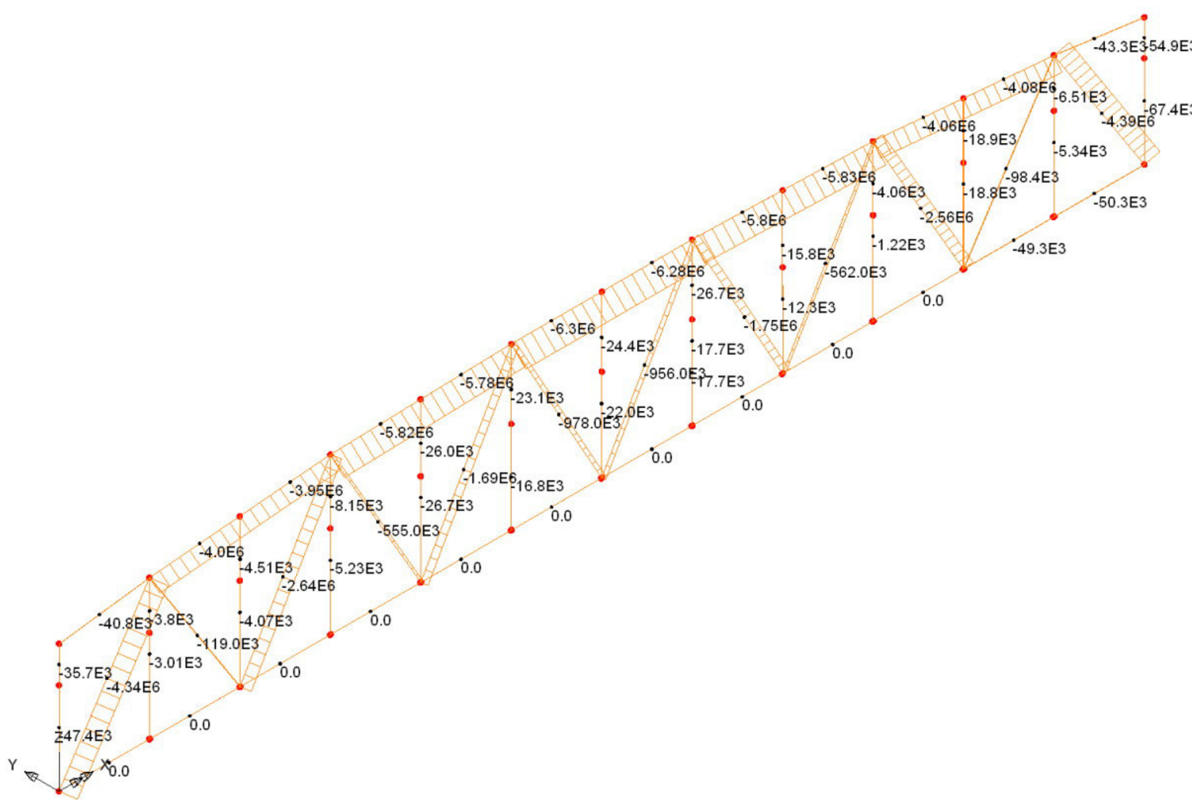


Figure C02: Lift Span Highway Truss Members Axial Forces-Combination 4-Maximum Compression

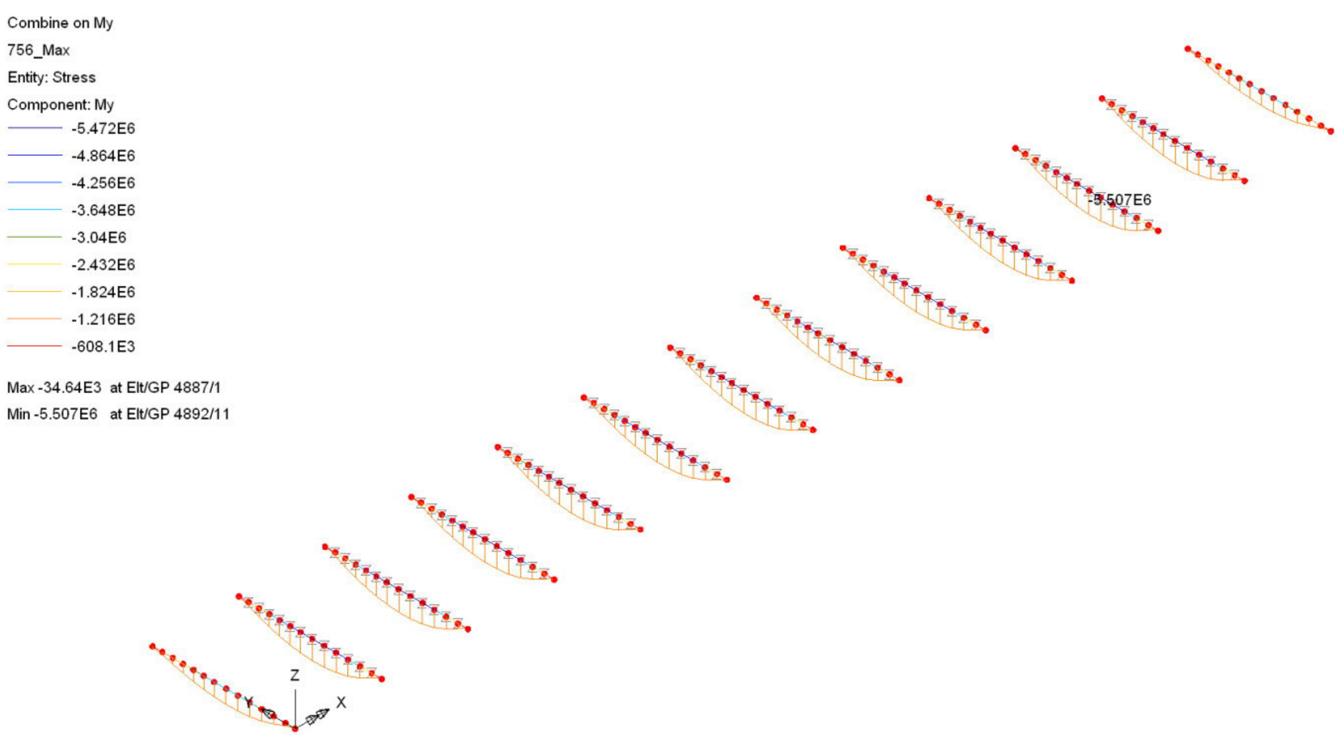


Figure C03: Lift Span Floor Beams - Maximum Moment

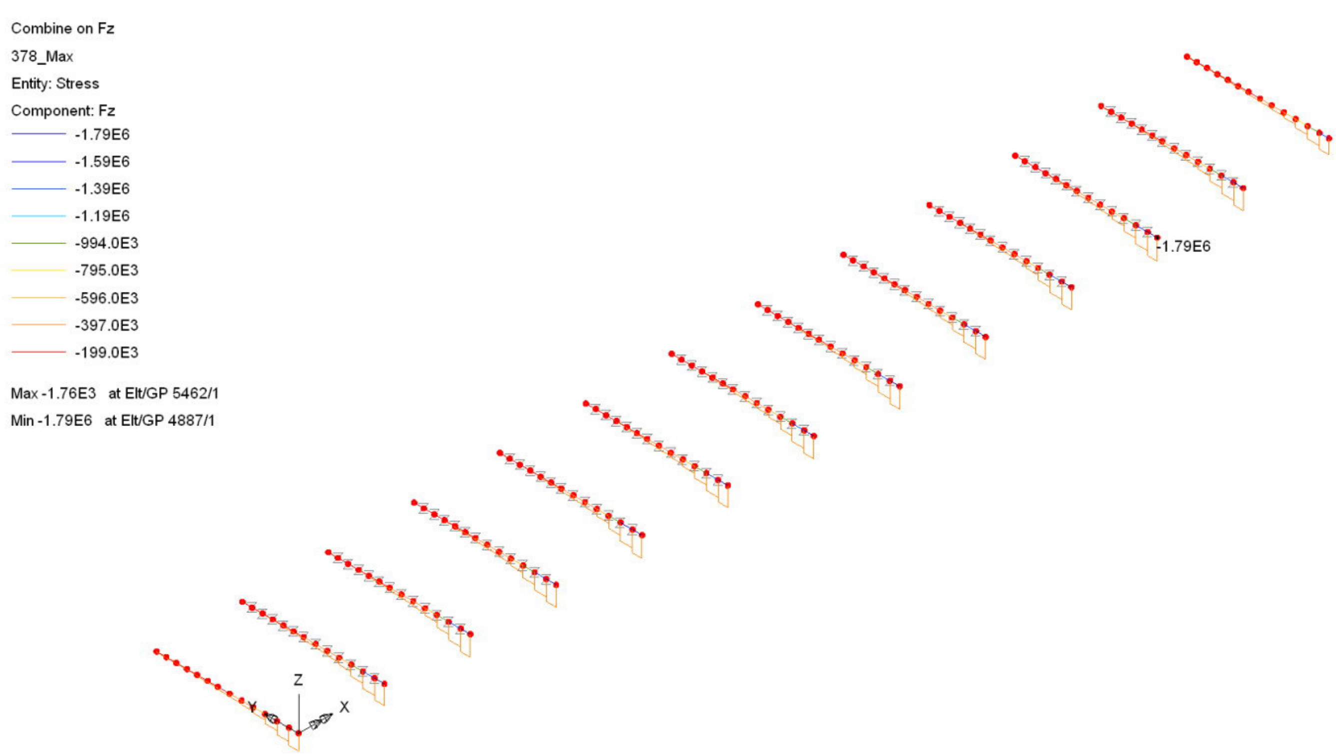


Figure C04: Lift Span Floor Beams - Maximum Shear Force

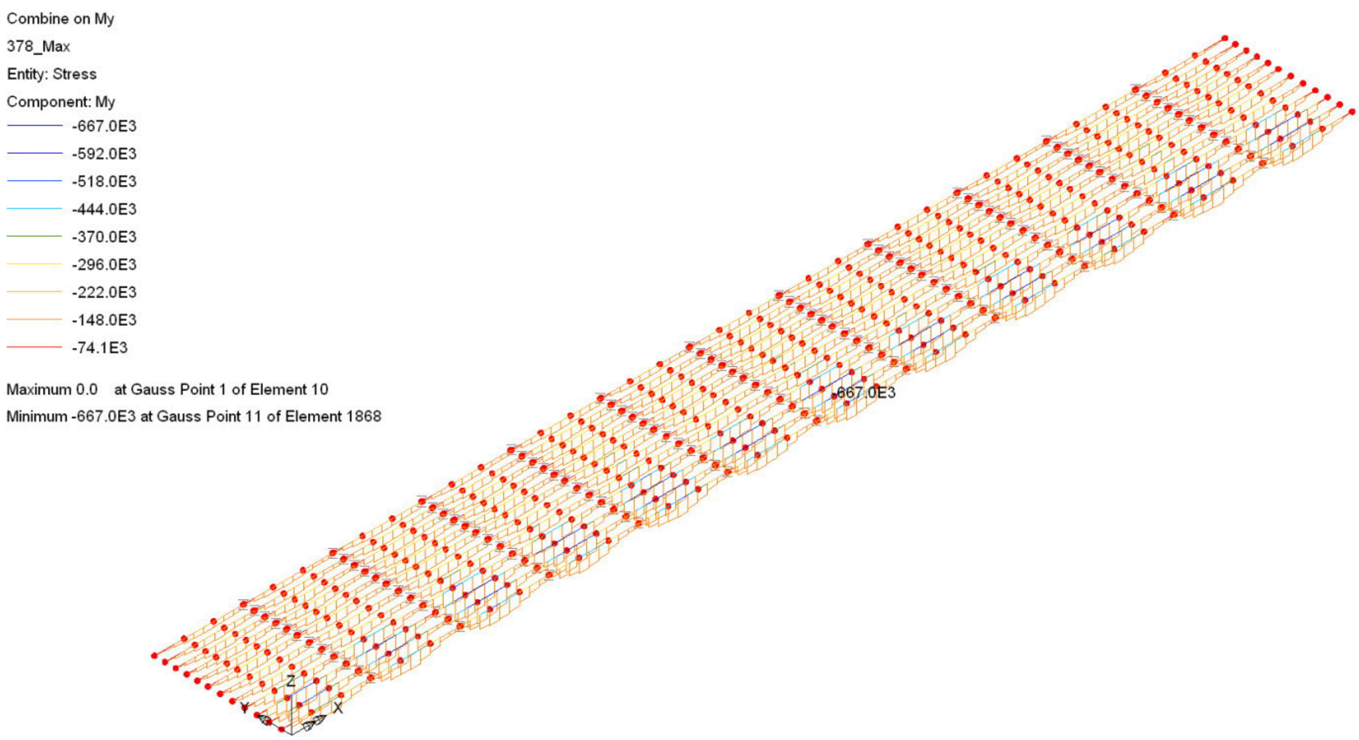


Figure C05: Lift Span Stringers - Maximum Moment

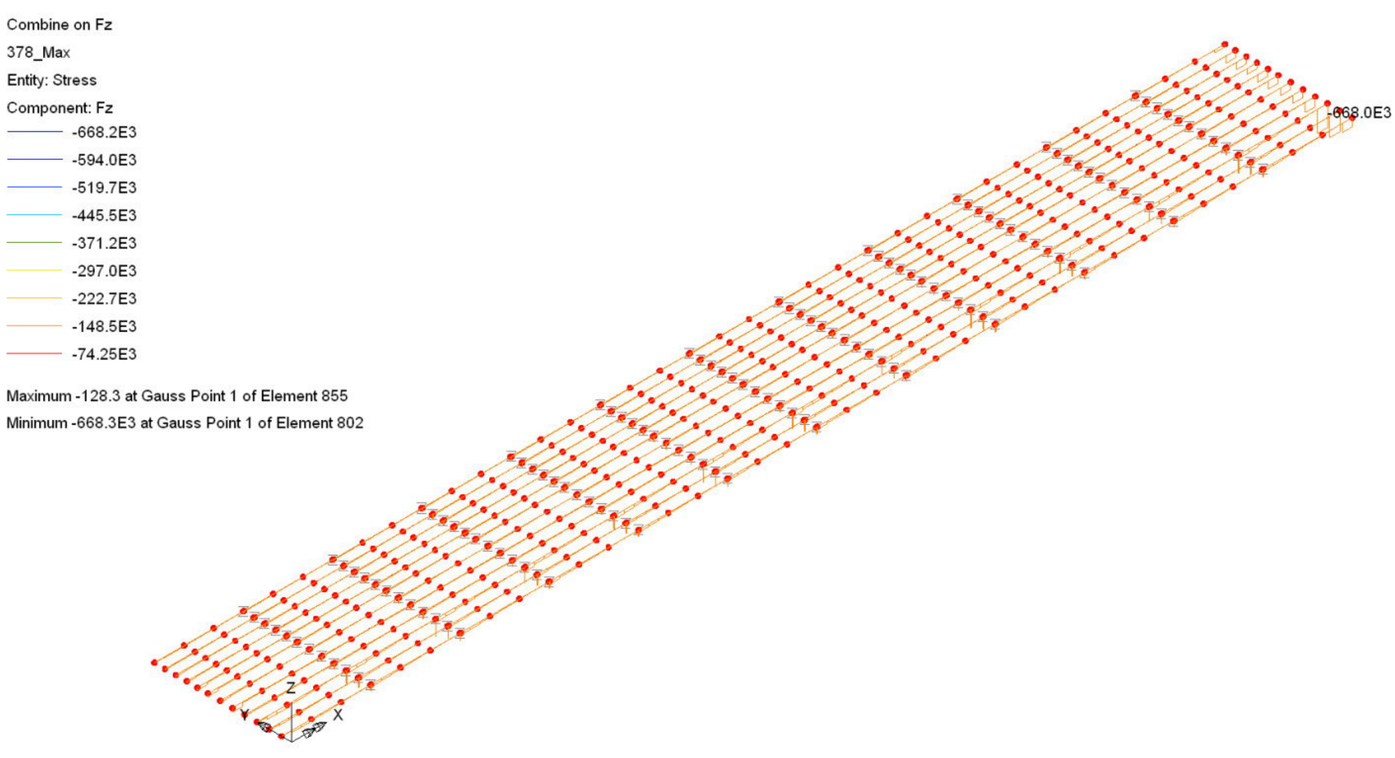


Figure C06: Lift Span Stringers - Maximum Shear Force

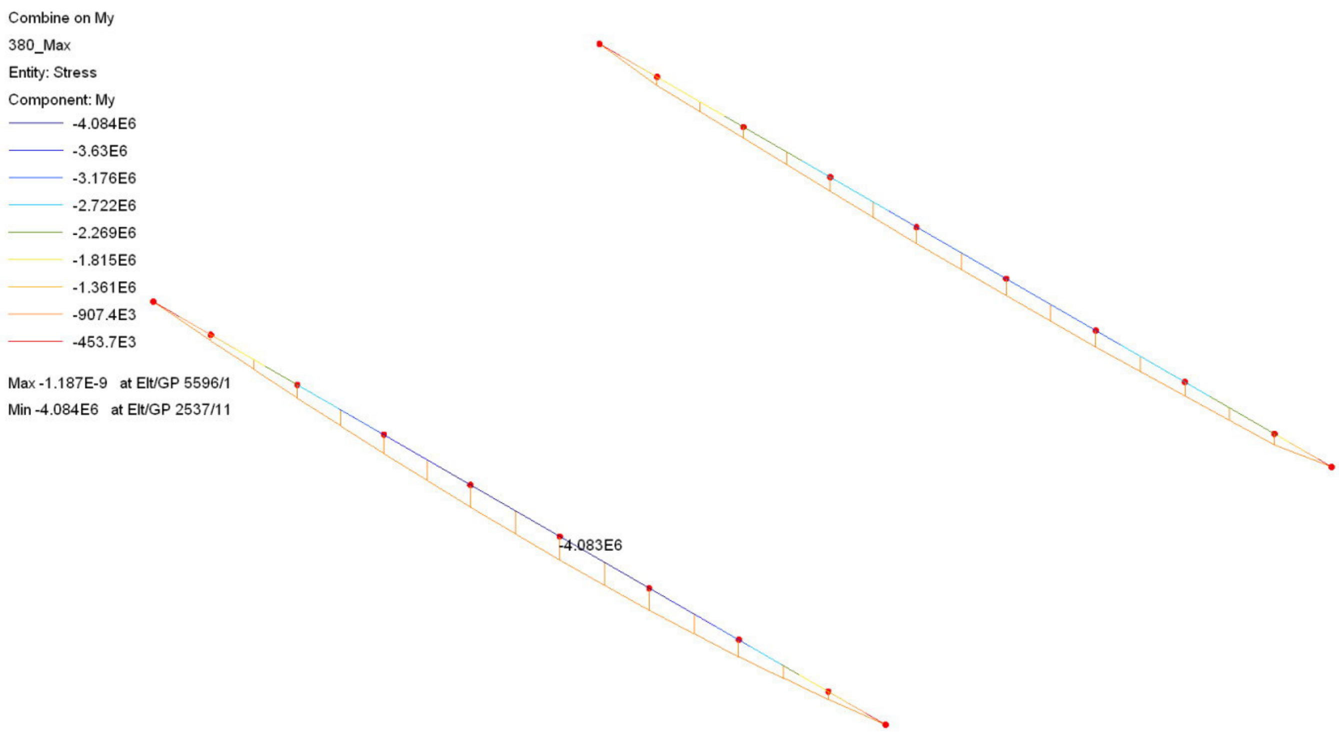


Figure C07: Tower Span Floor Beams - Maximum Moment

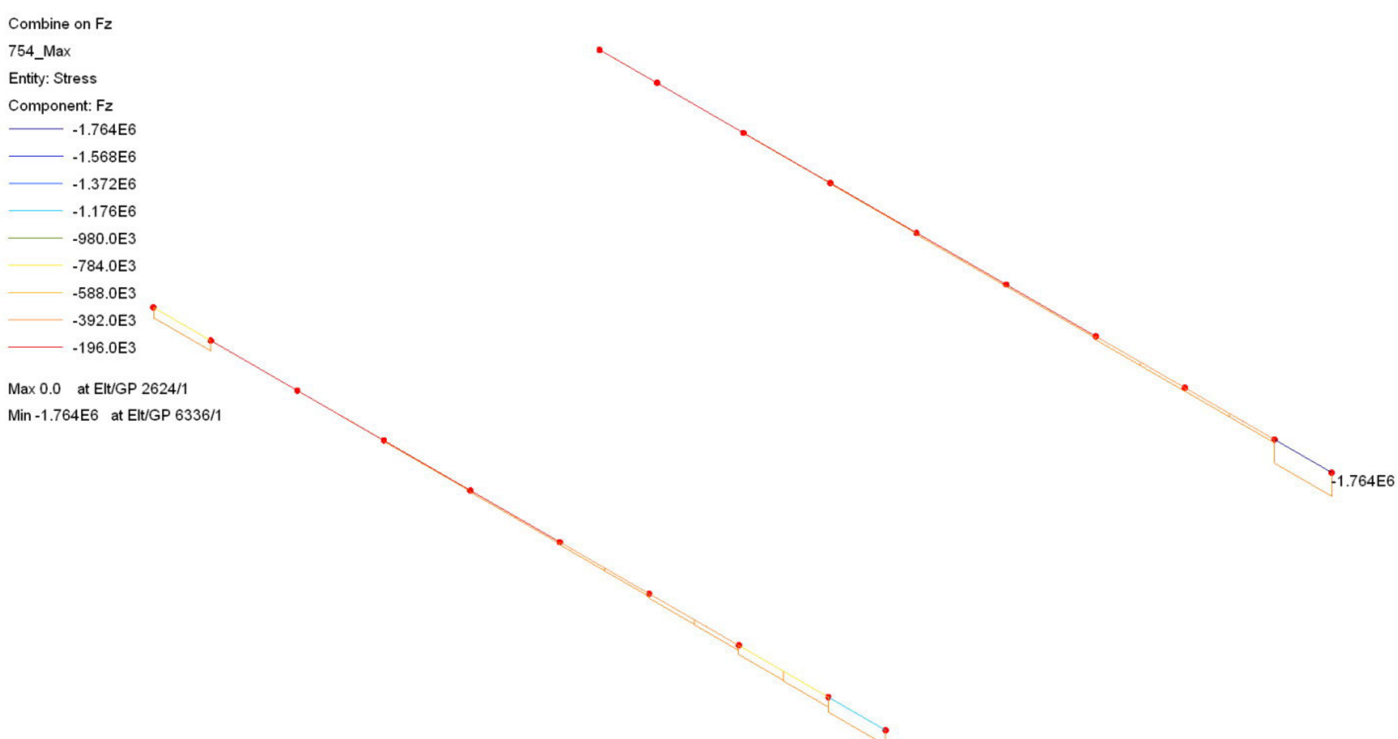


Figure C08: Tower Span Floor Beams - Maximum Shear Force

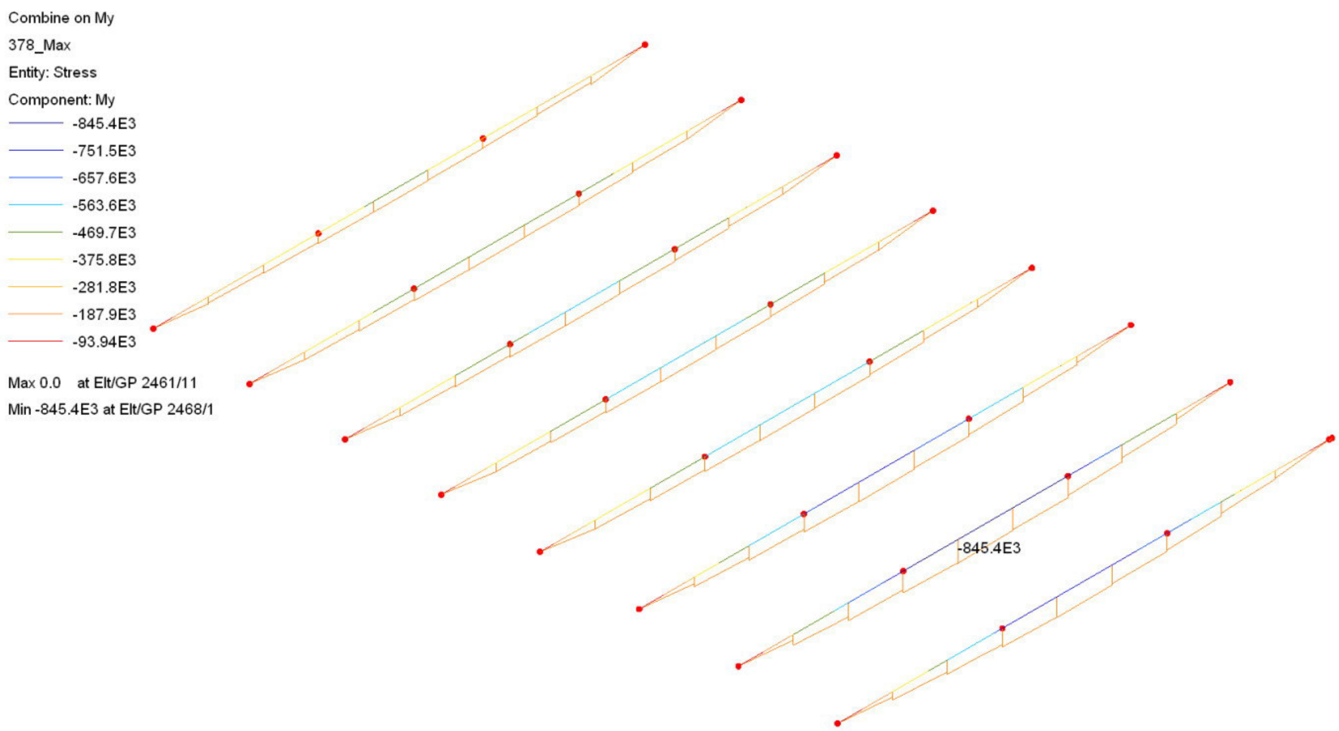


Figure C09: Tower Span Stringers - Maximum Moment

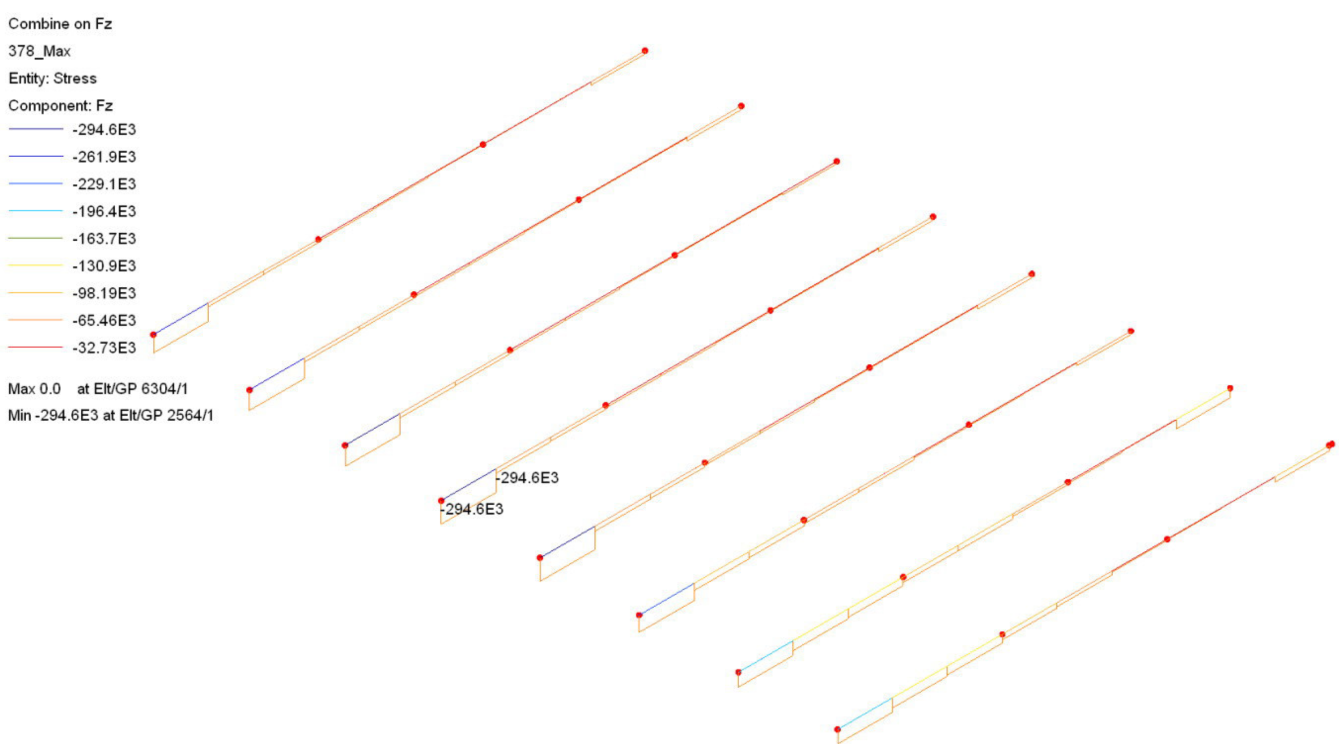


Figure C10: Tower Span Stringers - Maximum Shear Force

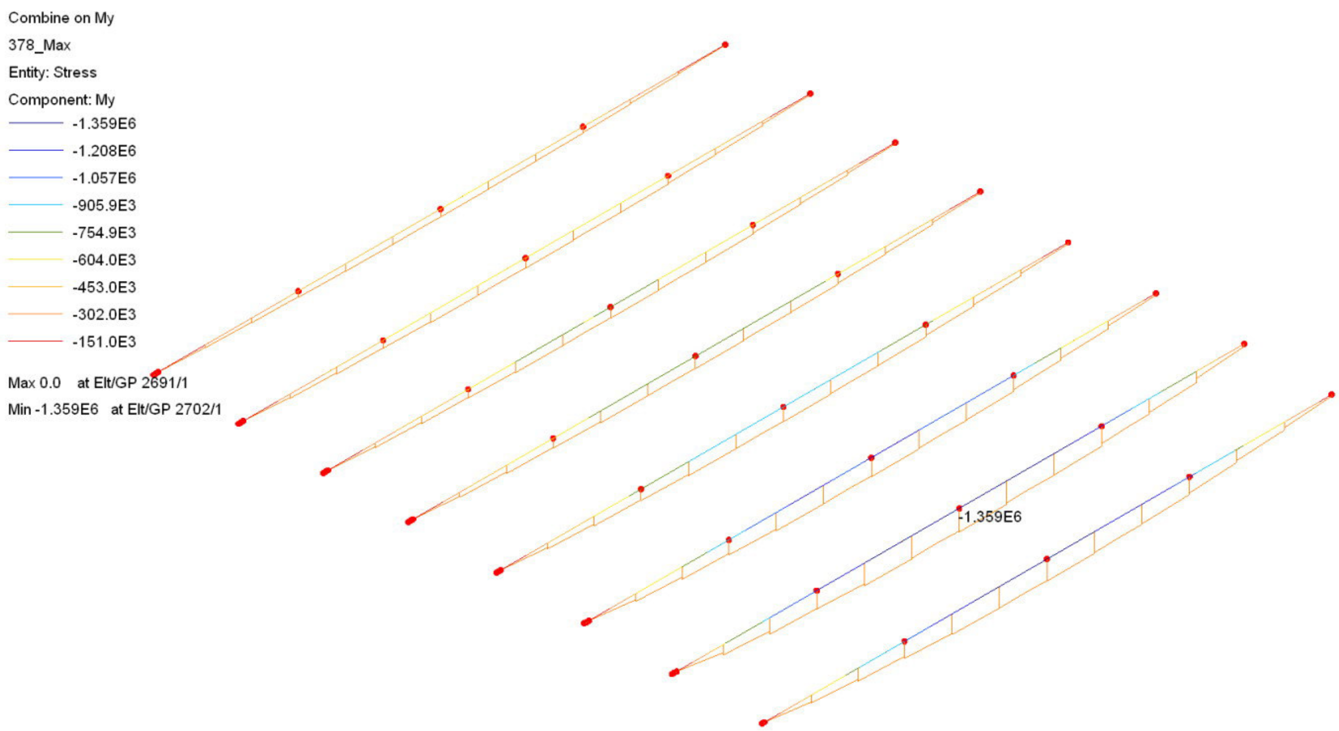


Figure C11: Approach Span Stringers - Maximum Moment

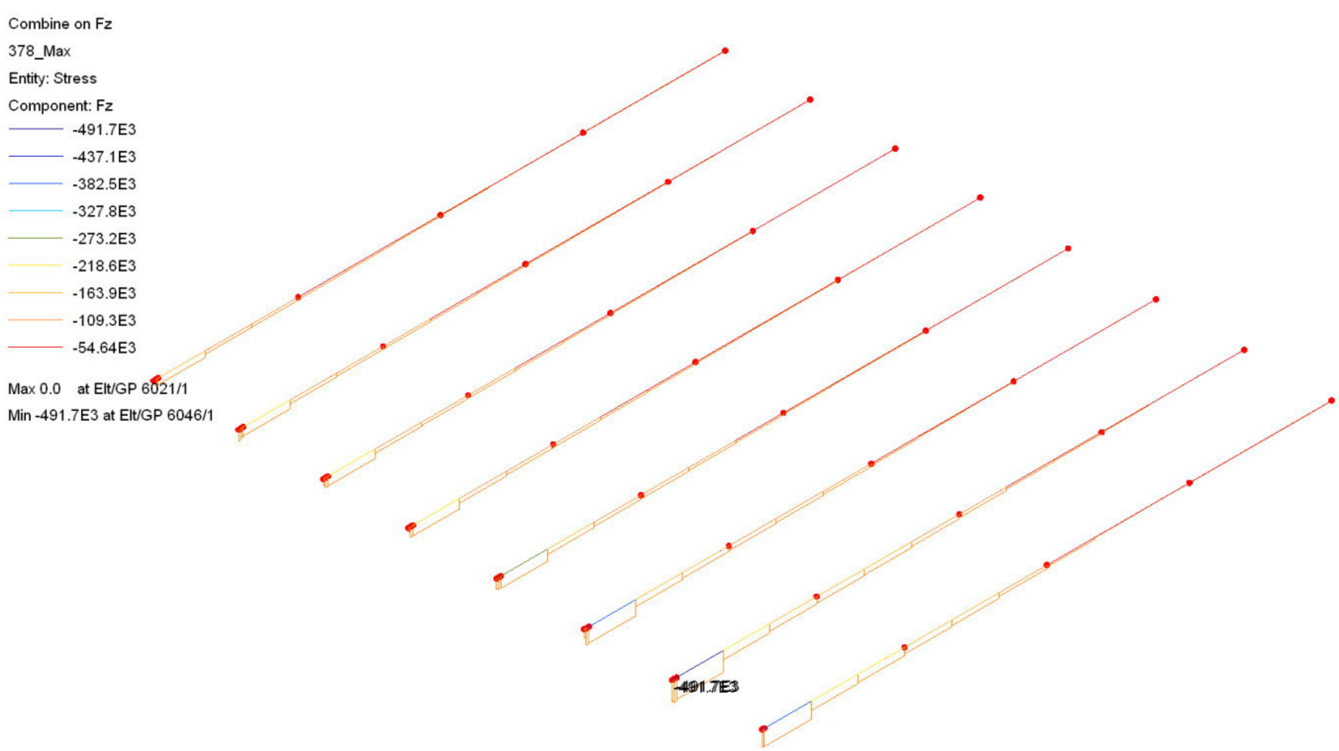


Figure C12: Approach Span Stringers - Maximum Shear Force

APPENDIX D

CAPACITY CALCULATION NOTES

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity	JOB NO. BT3252BTJ00
	Lift Span Highway Member Properties - with Perforation	PAGE of
MADE BY: KL	DATE: Jun. 4, 07	CHECKED BY: DATE:

		Bottom Chords			Top Chords				Diagonals						Verticals						
		$L_0 L_2$	$L_2 L_4$	$L_4 L_6$	$U_0 U_1$	$U_1 U_3$	$U_3 U_5$	$U_5 U_6$	$L_0 U_1$	$U_1 L_2$	$L_2 U_3$	$U_3 L_4$	$L_4 U_5$	$U_5 L_6$	$U_0 L_0$	$U_1 L_1$	$U_2 L_2$	$U_3 L_3$	$U_4 L_4$	$U_5 L_5$	$U_6 L_6$
Total Depth d (in)		31.25	31.25	31.25	31.25	31.25	31.25	31.25	32.375	31.25	31.25	27.25	27.25	25.25	24.5	24.5	24.5	24.5	24.5	24.5	24.5
Web-1	#	2	2	2	2	2	2	2	4	2	2	2	2	2	2	1	1	1	1	1	1
	h	30	30	30	30	30	30	30	30	30	30	26	26	24	23	23	23	23	23	23	23
	w	7/16	11/16	7/16	1/2	11/16	1	1/2	3/4	1	3/4	9/16	11/16	9/16	1/2	1/2	1/2	1/2	1/2	1/2	1/2
	A (in^2)	26.25	41.25	26.25	30	41.25	60	30	90	60	45	29.25	35.75	27	23	11.5	11.5	11.5	11.5	11.5	11.5
Web-2	#		2	2				2	2						1	2	2	2	2	2	2
	h		30	30				30	18						23	13	13	13	13	13	13
	w		7/16	11/16				5/8	3/4						7/8	3/8	3/8	3/8	3/8	3/8	3/8
	A (in^2)		26.25	41.25				37.5	27						20.125	9.75	9.75	9.75	9.75	9.75	9.75
Web-3	#		2	2											2						
	h		14	14											13						
	w		1/2	1/2											3/4						
	A (in^2)		14	14											19.5						
Top Angles	#	2	2	2	2	2	2	2	2	2	2	2	2	2	4	4	4	4	4	4	4
	Section	8x6x1/2	8x6x1/2	8x6x1/2	4x4x7/16	4x4x7/16	4x4x7/16	4x4x7/16	4x4x7/16	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	5x5x3/4	5x5x5/8	5x5x1/2	5x5x5/8	5x5x1/2	5x5x5/8	5x5x1/2
	A (mm^2)	4350	4350	4350	2140	2140	2140	2140	2140	1850	1850	1850	1850	1850	4490	3790	3060	3790	3060	3790	3060
	A (in^2)	13.485	13.485	13.485	6.634	6.634	6.634	6.634	6.634	5.735	5.735	5.735	5.735	5.735	27.838	23.498	18.972	23.498	18.972	23.498	18.972
Bot. Angles	#	2	2	2	2	2	2	2	2	2	2	2	2	2	4	4	4	4	4	4	4
	Section	8x6x1/2	8x6x1/2	8x6x1/2	4x4x7/16	8x6x9/16	8x6x3/4	8x6x3/4	8x6x3/4	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/4	4x4x1/2	4x4x3/8	4x4x1/2	4x4x3/8	4x4x1/2	4x4x3/8
	A (mm^2)	4350	4350	4350	2140	3250	6420	6420	6420	1850	1850	1850	1850	1850	3530	2430	1850	2430	1850	2430	1850
	A (in^2)	13.485	13.485	13.485	6.634	10.075	19.902	19.902	19.902	5.735	5.735	5.735	5.735	5.735	21.886	15.066	11.470	15.066	11.470	15.066	11.470
Top Cover/ Perforated Plate	#	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
	b	15	13	12	38	38	38	38	38	14	14	14.5	14	15	27	19.5	19.5	19.5	19.5	19.5	19.5
	t	3/8	3/8	3/8	1/2	11/16	1	1	1	3/8	3/8	3/8	3/8	3/8	1/2	1/2	3/8	1/2	3/8	1/2	3/8
	A (in^2)	5.6	4.88	4.5	19.0	26.1	38	38	38.0	5.3	5.3	5.4	5.3	5.6	13.5	9.8	7.3	9.8	7.3	9.8	7.3
Bottom Cover/ Perforated Plate	#	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
	b	15	13	12	24	28	28	28	28	14	14	14.5	14	15	27	19.5	19.5	19.5	19.5	19.5	19.5
	t	3/8	3/8	3/8	1/2	5/8	7/8	7/8	7/8	3/8	3/8	3/8	3/8	3/8	1/2	1/2	3/8	1/2	3/8	1/2	3/8
	A (in^2)	5.63	4.88	4.50	12.00	17.50	24.50	24.50	24.50	5.25	5.25	5.44	5.25	5.63	13.5	9.8	7.3	9.8	7.3	9.8	7.3
Calc. A_{gross} (in^2)		64.470	118.22	117.47	74.27	101.58	149.04	156.54	206.04	81.97	66.97	51.60	57.72	49.72	139.35	79.31	66.32	79.31	66.32	79.31	66.32
Calc. A_{gross} (m^2)		0.0416	0.0763	0.0758	0.0479	0.0655	0.0962	0.1010	0.1329	0.0529	0.0432	0.0333	0.0372	0.0321	0.0899	0.0512	0.0428	0.0512	0.0428	0.0512	0.0428
A_{gross} on dwg. (in^2)						100.5	148.0	156.5	206.0		66.6		57.1	48.7			66.3		66.3		66.3
A_{net} on dwg. (in^2)		55.5	101.8	101.0						69.3		41.8		40.8	104.2	60.6		60.6		60.6	
A_{net} on dwg. (m^2)		0.0358	0.0657	0.0652	0.0479	0.0648	0.0955	0.1010	0.1329	0.0447	0.0430	0.0270	0.0368	0.0263	0.0673	0.0391	0.0428	0.0391	0.0428	0.0391	0.0428
$S_{y\text{-}y}$ on dwg. (in^3)		501	535	649		1025	1547	1581	1886	485.2	517.0	307.0	405.0	277.0	744.0	512.0	555	512	555	512	555
$S_{x\text{-}x}$ on dwg. (in^3)									1761					341							
$I_{y\text{-}y}$ (in^4)		7828.125	8359.375	10140.63	--	16015.63	24171.88	24703.13	30529.63	7581.25	8078.125	4182.875	5518.125	3497.125	9114	6272	6798.75	6272	6798.75	6272	6798.75
$I_{x\text{-}x}$ (in^4)									28506.19												
$I_{y\text{-}y}$ (m^4)		0.003258	0.003479	0.004221	--	0.006666	0.010061	0.010282	0.012707	0.003156	0.003362	0.001741	0.002297	0.001456	0.003794	0.002611	0.00283	0.002611	0.00283	0.002611	0.00283
$I_{x\text{-}x}$ (m^4)									0.011865												
Tension/Compression		T	T	T		C	C	C	C	T	C	T	C	T	T	T	C	T	C	T	C
Length (m)		18.796	18.796	18.796	9.42	9.436	9.402	9.402	17.297	17.297	18.732	18.732	19.219	19.219	13.259	14.521	15.363	16.204	16.484	16.764	16.764
F_y (MPa)		230	230	290	230	230	230	230	230	230	230	230	230	230	290	230	230	230	230	230	230
F_u (MPa)		420	420	434	420	420	420	420	420	420	420	420	420	420	434	420	420	420	420	420	420
T_r/C_r (kN)		7824	14350	17952		12976	19135	20205	23170	9769	6850	5892	5348	5751	18528	8543	7245	8543	6991	8543	6926
Material		Carbon	Carbon	LA	Carbon	Carbon	Carbon	Carbon	Carbon	Carbon	Carbon	Carbon	Carbon	Carbon	LA	Carbon	Carbon	Carbon	Carbon	Carbon	Carbon
		$L_0 L_2$	$L_2 L_4$	$L_4 L_6$	$U_0 U_1$	$U_1 U_3$	$U_3 U_5$	$U_5 U_6$	$L_0 U_1$	$U_1 L_2$	$L_2 U_3$	$U_3 L_4$	$L_4 U_5$	$U_5 L_6$	$U_0 L_0$	$U_1 L_1$	$U_2 L_2$	$U_3 L_3$	$U_4 L_4$	$U_5 L_5$	$U_6 L_6$
		Bottom Chords			Top Chords				Diagonals						Verticals						

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity	JOB NO. BT3252BTJ00	
	Lift Span Railway Member Properties - with Perforation	PAGE	of
MADE BY: KL	DATE: Jun. 4, 07	CHECKED BY:	DATE:

		Bottom Chords			Top Chords				Diagonals						Verticals						
		$L_0 L_2$	$L_2 L_4$	$L_4 L_6$	$U_0 U_1$	$U_1 U_3$	$U_3 U_5$	$U_5 U_6$	$L_0 U_1$	$U_1 L_2$	$L_2 U_3$	$U_3 L_4$	$L_4 U_5$	$U_5 L_6$	$U_0 L_0$	$U_1 L_1$	$U_2 L_2$	$U_3 L_3$	$U_4 L_4$	$U_5 L_5$	$U_6 L_6$
Total Depth d (in)		31.25	31.25	31.25	31.25	31.25	31.25	31.25	32.375	31.25	31.25	27.25	27.25	25.25	24.5	24.5	24.5	24.5	24.5	24.5	24.5
Web-1	#	2	2	4	2	4	4	4	4	2	4	2	2	2	2	1	1	1	1	1	1
	h	30	30	30	30	30	30	30	30	30	30	26	26	24	23	23	23	23	23	23	23
	w	7/16	11/16	11/16	1/2	1/2	3/4	3/4	3/4	5/8	1/2	3/4	7/8	9/16	1/2	1/2	1/2	1/2	1/2	1/2	1/2
	A (in^2)	26.25	41.25	82.5	30	60	90	90	90	37.5	60	39	45.5	27	23	11.5	11.5	11.5	11.5	11.5	11.5
Web-2	#		2	2				2	2	2					1	2	2	2	2	2	2
	h		30	30				18	18	30					23	13	13	13	13	13	13
	w		11/16	1/2				7/16	3/4	1/2					7/8	3/8	3/8	3/8	3/8	3/8	3/8
	A (in^2)		41.25	30				15.75	27	30					20.125	9.75	9.75	9.75	9.75	9.75	9.75
Web-3	#		2	2											2						
	h		14	14											13						
	w		1/2	1/2											3/4						
	A (in^2)		14	14											19.5						
Top Angles	#	2	2	2	2	2	2	2	2	2	2	2	2	2	4	4	4	4	4	4	4
	Section	8x6x1/2	8x6x1/2	8x6x1/2	4x4x7/16	4x4x7/16	4x4x7/16	4x4x7/16	4x4x7/16	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	5x5x3/4	5x5x5/8	5x5x1/2	5x5x5/8	5x5x1/2	5x5x5/8	5x5x1/2
	A (mm^2)	4350	4350	4350	2140	2140	2140	2140	2140	1850	1850	1850	1850	1850	4490	3790	3060	3790	3060	3790	3060
	A (in^2)	13.485	13.485	13.485	6.634	6.634	6.634	6.634	6.634	5.735	5.735	5.735	5.735	5.735	27.838	23.498	18.972	23.498	18.972	23.498	18.972
Bot. Angles	#	2	2	2	2	2	2	2	2	2	2	2	2	2	4	4	4	4	4	4	4
	Section	8x6x1/2	8x6x1/2	8x6x1/2	4x4x7/16	8x6x3/4	8x6x3/4	8x6x3/4	8x6x3/4	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/8	4x4x3/4	4x4x1/2	4x4x3/8	4x4x1/2	4x4x3/8	4x4x1/2	4x4x3/8
	A (mm^2)	4350	4350	4350	2140	6420	6420	6420	6420	1850	1850	1850	1850	1850	3530	2430	1850	2430	1850	2430	1850
	A (in^2)	13.485	13.485	13.485	6.634	19.902	19.902	19.902	19.902	5.735	5.735	5.735	5.735	5.735	21.886	15.066	11.470	15.066	11.470	15.066	11.470
Top Cover/ Perforated Plate	#	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
	b	15	13	12	38	38	38	38	38	14	14	14.5	14	15	27	19.5	19.5	19.5	19.5	19.5	19.5
	t	3/8	3/8	3/8	1/2	1	1	1	1	3/8	3/8	3/8	3/8	3/8	1/2	1/2	3/8	1/2	3/8	1/2	3/8
	A (in^2)	5.6	4.9	4.5	19.0	38.0	38	38	38.0	5.3	5.3	5.4	5.3	5.6	13.5	9.8	7.3	9.8	7.3	9.8	7.3
Bottom Cover/ Perforated Plate	#	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
	b	15	13	12	24	28	28	28	28	14	14	14.5	14	15	27	19.5	19.5	19.5	19.5	19.5	19.5
	t	3/8	3/8	3/8	1/2	7/8	7/8	7/8	7/8	3/8	3/8	3/8	3/8	3/8	1/2	1/2	3/8	1/2	3/8	1/2	3/8
	A (in^2)	5.63	4.88	4.50	12.00	24.50	24.50	24.50	24.50	5.25	5.25	5.44	5.25	5.63	13.5	9.8	7.3	9.8	7.3	9.8	7.3
Calc. A_{gross} (in^2)		64.470	133.22	162.47	74.27	149.04	179.04	194.79	206.04	89.47	81.97	61.35	67.47	49.72	139.35	79.31	66.32	79.31	66.32	79.31	66.32
Calc. A_{gross} (m^2)		0.0416	0.0859	0.1048	0.0479	0.0962	0.1155	0.1257	0.1329	0.0577	0.0529	0.0396	0.0435	0.0321	0.0899	0.0512	0.0428	0.0512	0.0428	0.0512	0.0428
A_{gross} on dwg. (in^2)						149.0	179.0	194.8	206.0		81.5		66.9	48.7			66.3		66.3		66.3
A_{net} on dwg. (in^2)		54.5	112.4	137.2						74.1		48.8	52.3	39.8	104.24	60.6		60.6		60.6	
A on dwg. (m^2)		0.0352	0.0725	0.0885	0.0479	0.0961	0.1155	0.1257	0.1329	0.0478	0.0526	0.0315	0.0432	0.0314	0.0673	0.0391	0.0428	0.0391	0.0428	0.0391	0.0428
S_{y-y} on dwg. (in^3)		491	706	815		1550	1680	1726	1886	502.0	589.0	329.0	445.0	269.0	744.0	512.0	555	512	555	512	555
S_{x-x} on dwg. (in^3)									1761				342	341							
I_{y-y} (in^4)		7671.875	11031.25	12734.38	--	24218.75	26250	26968.75	30529.63	7843.75	9203.125	4482.625	6063.125	3396.125	9114	6272	6798.75	6272	6798.75	6272	6798.75
I_{x-x} (in^4)									28506.19												
I_{y-y} (m^4)		0.003193	0.004592	0.0053	--	0.010081	0.010926	0.011225	0.012707	0.003265	0.003831	0.001866	0.002524	0.001414	0.003794	0.002611	0.00283	0.002611	0.00283	0.002611	0.00283
I_{x-x} (m^4)									0.011865												
Tension/Compression		T	T	T		C	C	C	C	T	C	T	C	T	T	T	C	T	C	T	C
Length (m)		18.796	18.796	18.796	9.42	9.436	9.402	9.402	17.297	17.297	18.732	18.732	19.219	19.219	13.259	14.521	15.363	16.204	16.484	16.764	16.764
F_y (MPa)		290	290	290	230	230	290	290	290	290	290	290	230	230	290	230	230	230	230	230	230
F_u (MPa)		434	434	434	420	420	434	434	434	434	434	434	420	420	434	420	420	420	420	420	420
T_r/C_r (kN)		9687	19978	24386		19255	28660	31068	27682	13171	9534	8674	6112	6865	18528	8543	7245	8543	6991	8543	6926
Material		LA	LA	LA	Carbon	Carbon	LA	LA	LA	LA	LA	LA	Carbon	Carbon	LA	Carbon	Carbon	Carbon	Carbon	Carbon	Carbon
		$L_0 L_2$	$L_2 L_4$	$L_4 L_6$	$U_0 U_1$	$U_1 U_3$	$U_3 U_5$	$U_5 U_6$	$L_0 U_1$	$U_1 L_2$	$L_2 U_3$	$U_3 L_4$	$L_4 U_5$	$U_5 L_6$	$U_0 L_0$	$U_1 L_1$	$U_2 L_2$	$U_3 L_3$	$U_4 L_4$	$U_5 L_5$	$U_6 L_6$
		Bottom Chords			Top Chords				Diagonals						Verticals						

Section Properties of Floor Beams, Stringers on Lift, Tower and Approach Spans

	Lift Span			***Tower Span***			***Approach Span***
	Floor Beams		Stringers	Floor Beam		Stringers	Stringers
	Intermediate	End		Front	Back		
Web	1-78x9/16	1-71x1/2	24WF84	1-71x1/2	1-71x5/8	27WF102	33WF130
Angles	4-8x8x7/8	4-8x8x3/4		4-8x8x1/2	4-8x8x1/2		
Cover Plates	2-20x3/4	2-20x1/2		2-20x1/2	2-20x1/2		
A (m^2)	0.0817	0.0654	0.01590	0.055766	0.061492	0.01940	0.0247
I_x (m^4)	0.05899	0.0459	0.000985	0.031493	0.033045	0.00151	0.00278
I_y (m^4)	0.000697	0.00052	0.000039	0.000435	0.000439	0.000058	0.000090

DELCAN	SUBJECT:	Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity	JOB NO.	BT3252BTJ00
		Lift Span Truss Member Force due to Dead Load and Live Loa	PAGE	1 of 1
	MADE BY: KL	DATE: Jun. 12, 07	CHECKED BY:	DATE:

Axial Force F_x on Highway Truss Members under Self Weight and Factored Live Load

		Bottom Chords			Top Chords				Diagonals						Verticals						
		$L_0 L_2$	$L_2 L_4$	$L_4 L_6$	$U_0 U_1$	$U_1 U_3$	$U_3 U_5$	$U_5 U_6$	$L_0 U_1$	$U_1 L_2$	$L_2 U_3$	$U_3 L_4$	$L_4 U_5$	$U_5 L_6$	$U_0 L_0$	$U_1 L_1$	$U_2 L_2$	$U_3 L_3$	$U_4 L_4$	$U_5 L_5$	$U_6 L_6$
	Tension/Compression	T	T	T		C	C	C	C	T	C	T	C	T	T	T	C	T	C	T	C
Member Force due to Dead Load	Unfactored F_x (kN) (LUSAS)	2151	4741	5916		-3850	-5670	-6170	-4140	2940	-2130	1600	-882	366	-197	266	-194	317	-208	284	-227
	Factored F_x (kN) (LUSAS)	2366	5215	6508		-4235	-6237	-6787	-4554	3234	-2343	1760	-970.2	403	-217	292	-213	348	-229	312	-250
Member Force due to Live Load	Factored F_x (kN): LUSAS Comb1-Max	1100	2400	2930		0	0	0	0	1660	379	1270	721	983	9.34	1110	46.6	1130	49.7	1110	52.7
	LUSAS Comb1-Min	0	0	0		-1940	-2860	-3080	-2090	-102	-1430	-455	-1100	-815	-10.7	-4.13	-8.43	-2.31	-27.8	-11.4	-24.8
	LUSAS Comb2-Max	1060	2340	2870		0	0	0	0	1600	359	1230	688	949	9.34	1060	45.4	1090	46.5	1070	49
	LUSAS Comb2-Min	0	0	0		-1880	-2780	-3000	-2010	-96	-1380	-431	-1060	-780	-9.37	-4.13	-7.69	-1.69	-27.5	-9.57	-24.8
	LUSAS Comb3-Max	517	1150	1420		0	0	0	0	785	174	604	335	466	0	521	23.7	531	22.1	524	22.4
	LUSAS Comb3-Min	0	0	0		-921	-1370	-1480	-984	-45.2	-679	-209	-523	-380	-2.67	0	-2.94	0	-13	-1.19	-11.1
	LUSAS Comb4-Max	2250	4850	5860		0	0	0	0	3310	464	2210	870	1350	5.75	1630	46.9	1580	61.6	1520	60.4
	LUSAS Comb4-Min	0	0	0		-4080	-5830	-6303	-4390	-119	-2640	-562	-1750	-978	-67.4	-5.34	-18.8	-5.23	-26.7	-17.4	-22
	LUSAS Comb5-Max	2070	4490	5470		0	0	0	0	3020	400	2010	756	1200	5.74	1480	42.5	1470	63.1	1480	66.3
	LUSAS Comb5-Min	0	0	0		-3740	-5370	-5830	-4000	-101	-2410	-490	-1570	-853	-59.4	-6.52	-16.7	-6.2	-24.7	-21.1	-22.2
	LUSAS Comb6-Max	874	1920	2370		0	0	0	0	1260	139	820	267	456	0	551	17.4	561	21.4	567	20.5
	LUSAS Comb6-Min	0	0	0		-1570	-2290	-2490	-1670	-33.2	-998	-174	-622	-304	-21.8	0	-5.79	-0.0876	-8.16	-1.73	-7.14
Total Member Force	Factored $F_{x, TOT}$ (kN) (LUSAS)	4616	10065	12368		-8315	-12067	-13090	-8944	6544	-4983	3970	-2720	1753	-207	1922	-232	1928	-257	1832	-275
	Factored $F_{x,TOT}$ (kN) (S-Frame)	4747	10330	12927		-8127		-13285	-8836	6287		3672	-2233	1215	5046					1385	-281
Member Capacity	Member Capacity (kN)	7824	14350	17952		-12976	-19135	-20205	-23170	9769	-6850	5892	-5348	5751	18528	8543	-7245	8543	-6991	8543	-6926
	Remaining Capacity (kN)	3208	4285	5584		-4661	-7068	-7115	-14226	3225	-1867	1922	-2628	3999	18735	6620	-7013	6614	-6734	6710	-6652
	Member Capacity Reserve Ratio	1.69	1.43	1.45		1.56	1.59	1.54	2.59	1.49	1.37	1.48	1.97	3.28							
	Live Load Capacity (kN)	5458	9135	11444		-8741	-12898	-13418	-18616	6535	-4507	4132	-4378	5349	18744	8250	-7031	8194	-6762	8230	-6677
	Live Load Capacity Reserve Ratio	2.43	1.88	1.95		2.14	2.21	2.13	4.24	1.97	1.71	1.87	2.50	3.96							

DELCAN	SUBJECT:	Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity	JOB NO.	BT3252BTJ00
		Other Member Forces due to Dead Load and Live Load	PAGE	1 of 1
	MADE BY:	KL DATE: Jun. 19, 07 CHECKED BY:	DATE:	

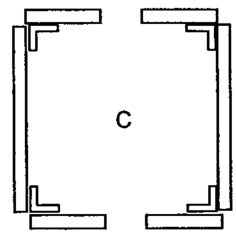
Moment and Shear on Floor Beams, Stringers and Deck Slabs under Self Weight and Factored Live Load

		Lift Span						Tower Span								Approach Span			
		Intermediate Beams		End Beams		Stringers		Front Beams		Back Beams		Stringers		Slab		Stringers		Slab	
		M_y	F_z	M_y	F_z	M_y	F_z	M_y	F_z	M_y	F_z	M_y	F_z		M	M_y	F_z		M
Member Force due to Dead Load	Unfactored (LUSAS)	-989	-220	-567	-148	-37	-23	-921		-1094		-102.0	-40	$UF._{conc}$	-1.3	-199	-84	$UF._{conc}$	-1.3
	Factored (LUSAS)	-1088	-242	-624	-163	-41	-25	-1013	-212	-1203	-489	-112.2	-44	$F._{conc.}$	-1.56	-219	-92	$F._{conc.}$	-1.56
Member Force due to Live Load	LUSAS Comb1-Max	713	1192	632	882	693			605		1174			$UF._{asphalt}$	-0.704			$UF._{asphalt}$	-0.704
	LUSAS Comb1-Min	-5294	-1790	-3744	-1332	-667	-668	-4073	-1220	-4060	-1763	-845	-295	$F._{asphalt}$	-1.056	-1359	-492	$F._{asphalt}$	-1.056
	LUSAS Comb2-Max	650	1196	594	761				609		1180								
	LUSAS Comb2-Min	-5344	-1760	-3780	-1164	-532	-487	-4084	-1065	-4080	-1542.3	-686	-258	$F._{wheel\ load}$	-42.37	-1080	-379	$F._{wheel\ load}$	-42.37
	LUSAS Comb3-Max	306	560	292	381				348		556								
	LUSAS Comb3-Min	-3580	-558	-2510	-416	-432	-438	-2593	-383	-2590	-556	-464	-130			-765	-299		
	LUSAS Comb4-Max	764	1226	751	715				575		1252								
	LUSAS Comb4-Min	-5359	-1742	-4080	-1242	-588	-594	-3826	-1110	-3820	-1764	-763	-257			-1245	-466		
	LUSAS Comb5-Max	700	1239	697	849				583		1270								
	LUSAS Comb5-Min	-5507	-1550	-3640	-1100	-475	-436	-3873	-984	-3870	-1574	-628	-233			-1006	-363		
	LUSAS Comb6-Max	327	593	315	405				340		611.7								
	LUSAS Comb6-Min	-3794	-592	-2460	-414	-410	-448	-2510	-371	-2510	-612	-444	-131			-755.7	-319		
Total Member Force	Factored (LUSAS)	-6595	-2032	-4704	-1495	-707	-693	-5097	-1432	-5283	-2253	-958	-339		-45.0	-1578	-584		-45.0
	Factored (Simplifed Method)					485													
	Factored (kN) (S-Frame)	5311	1333			538													
Member Capacity	Member Capacity	12497	3542	9540	2790	802	859	6477	2237	6875	2797	1093	1067	Old Slab	46.3	1665	1557	Old Slab	46.3
														New Slab	47.5			New Slab	47.5
	Sufficiency (Yes/No)	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes		Yes	Yes	Yes		Yes
	Live Load Capacity	11408	3300	8916	2627	761	834	5464	2025	5671	2308	980	1022		43.7	1446	1465		43.7
	Live Load Capacity Reserve Ratio *	2.07	1.84	2.19	1.97	1.14	1.25	1.34	1.66	1.39	1.31	1.16	3.47		1.03	1.06	2.98		1.03
	Live Load Capacity Reserve Ratio **					1.57													

Note: * Live Load Capacity/Max M_y from LUSAS analysis output

** Live Load Capacity/M_y calculated using the simplified method specified in CHBDC

LIFT SPAN



Bottom Chord L_0-L_2 Tension member

Built-up section

Angle (4)	Web (2)	Top plate (1)	Bot plate (1)
8x6x1/2	30x7/16	(23-8)x3/8	(23-8)x3/8
203x152x13			

Design Criteria

Member Length = 9.4206 m *dwg. 3/62*

Total width =	20	in	tal Depth =	31.25	in	*(2-6 1/2"+2*3/8")*
	508	mm		793.75	mm	

	Angle			Web			Top plate		Bot plate
A =	4350	mm^2	w =	7/16	in	t =	3/8	in	3/8
y =	62.8	mm		11.1125	mm		9.525	mm	9.525
x =	37.3	mm	h =	30	in	b =	15	in	15
b =	203			762	mm		381	mm	381
d =	152								
t =	12.7								

$F_{y,\ carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u,\ carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.95			
S_{net} =	501	in^2		*dwg. 3/62*
	323	10^3 mm^3		

Calculate Area

A_{angle}=4A= 17400 mm^2 A_{web}=2*hw= 16935.45 mm^2 A_{cp}=2*ht= 7258.05 mm^2

A_{gross} =	41594	mm^2	0.0416	m^2

A_{net} = 55.5 in^2 *dwg 3/62*

A_{net} =	35806	mm^2	0.03581	m^2

Class Check

b' = b/2 = 381 mm
h' = h-2d = 458 mm

Flange (treated as rectangular hollow section)

b/t = 40.0
670/sqrt(F_y) = 44.2 — *Tab 10.9.2.1, p451*
Flange is Class 3

Flange (treated as perforated cover plates)

840/sqrt(F_y) = 55.4
Flange is Class 3

Web (in axial compression)

h/w = 41.2
670/sqrt(F_y) = 44.2
Web is Class 3 — *Fig 2-7, Handbook p2-21*

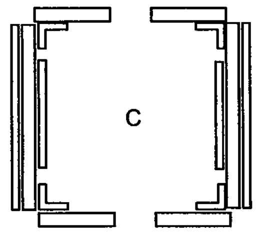
Both Flange and web are Class 3

Check Tensile Resistance

$T_{r.1} = \phi_s A_{net} F_y =$ 7823.7 kN — *Cl 10.8.2 (a), p 447*
$T_{r.2} = 0.85\phi_s A_{ne} F_u =$ 12143.7 kN — *Cl 10.8.2 (b), p 448*
$T_r = \min(T_{r.1}, T_{r.2}) =$ 7823.7 kN

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity		JOB NO. BT3252BTJ00	
	Member Capacity - Lift Span - Bottom Chord L_2-L_4		PAGE 1 of 1	
	MADE BY: KL	DATE: May. 23, 07	CHECKED BY:	DATE:

LIFT SPAN



Bottom Chord L_2-L_4 Tension member

Built-up section

Angle (4)	Web (2)	Web (2)	Web (2)	Top plate (1)	Bot plate (1)
8x6x1/2	30x11/16	30x7/16	14x1/2	(21-8)x3/8	(21-8)x3/8
203x152x13					

Design Criteria

Member Length = 9.4206 m *dwg. 3/62*

Total width =	20	in	tal Depth =	31.25	in	*(2-6 1/2"+2*3/8")*
	508	mm		793.75	mm	

	Angle			Web				Top plate	Bot plate	
A =	4350	mm^2	w =	11/16	7/16	1/2	t =	3/8	3/8	in
y =	62.8	mm		17.4625	11.1125	12.7		9.525	9.525	mm
x =	37.3	mm	h =	30	30	14	b =	13	13	in
b =	203			762	762	355.6		330.2	330.2	mm
d =	152									
t =	12.7									

$F_{y, carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u, carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.95			
S_{net} =	535	in^2		*dwg. 3/62*
	345	10^3 mm^3		

Calculate Area

A_{angle}=4A= 17400 mm^2 A_{web}=2*hw= 52580.54 mm^2 A_{cp}=2*ht= 6290.31 mm^2

A_{gross} =	76271	mm^2	0.0763	m^2

A_{net} = 101.8 in^2 *dwg 3/62*

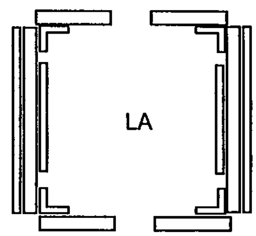
A_{net} =	65677	mm^2	0.06568	m^2

Check Tensile Resistance

$T_{r.1} = \phi_s A_{net} F_y$ =	14350.5	kN	*Cl 10.8.2 (a), p 447*
$T_{r.2} = 0.85\phi_s A_{ne} F_u$ =	22274.5	kN	*Cl 10.8.2 (b), p 448*
$T_r = \min(T_{r.1}, T_{r.2})$ =	14350.5	kN	

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity	JOB NO. BT3252BTJ00
	Member Capacity - Lift Span - Bottom Chord L_4-L_6	PAGE 1 of 1
	MADE BY: KL · DATE: May. 23, 07 · CHECKED BY:	DATE:

LIFT SPAN



Bottom Chord L_4-L_6 Tension member

Built-up section

Angle (4)	Web (2)	Web (2)	Web (2)	Top plate (1)	Bot plate (1)
8x6x1/2	30x11/16	30x7/16	14x1/2	(20-8)x3/8	(20-8)x3/8
203x152x13					

Design Criteria

Member Length = 9.4206 m *dwg. 3/62*

Total width = 20 in / 508 mm   tal Depth = 31.25 in / 793.75 mm   *(2-6 1/2"+2*3/8")*

	Angle			Web					Top plate	Bot plate	
A =	4350	mm^2	w =	11/16	7/16	1/2		t =	3/8	3/8	in
y =	62.8	mm		17.4625	11.1125	12.7			9.525	9.525	mm
x =	37.3	mm	h =	30	30	14		b =	12	12	in
b =	203			762	762	355.6			304.8	304.8	mm
d =	152										
t =	12.7										

$F_{y, carbon}$ = 290 MPa *Low Alloy* *Table 2-2, "Design of Steel Structures", p54*

$F_{u, carbon}$ = 434 MPa *from Zuzana's notes* *Edwin H. Gaylord*

E = 200000 MPa

ϕ = 0.95

S_{net} = 649 in^2 *dwg. 3/62*

419 $10^3\ mm^3$

Calculate Area

A_{angle}=4A= 17400 mm^2 A_{web}=2*hw= 52580.54 mm^2 A_{cp}=2*ht= 5806.44 mm^2

A_{gross} = 75787 mm^2	0.0758 m^2

A_{net} = 101.0 in^2 *dwg 3/62*

A_{net} = 65161 mm^2	0.06516 m^2

Check Tensile Resistance

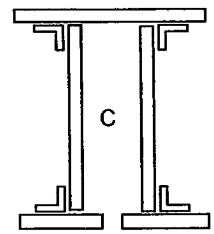
$T_{r.1} = \phi_s A_{net} F_y$ = 17951.9 kN *Cl 10.8.2 (a), p 447*

$T_{r.2} = 0.85\phi_s A_{ne} F_u$ = 22836.1 kN *Cl 10.8.2 (b), p 448*

$T_r = \min(T_{r.1}, T_{r.2})$ = 17951.9 kN

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity	JOB NO. BT3252BTJ00	
	Member Capacity - Lift Span - Top Chord U_1-U_3	PAGE 1 of 1	
	MADE BY: KL · DATE: May. 23, 07	CHECKED BY:	DATE:

LIFT SPAN



Top Chord U_1-U_3 Compression Member

Built-up section

Angle (2)	Angle (2)	Web (2)	Top plate (1)	Bot plate (1)
4x4x7/16	8x6x9/16	30x11/16	38x11/16	(42-14)x5/8
102x102x11	203x152x14			

Design Criteria

							*(2-6 1/2"+2*3/8")*	
Member Length =	9436	mm	ɔtal width =	20	in	tal Depth =	31.25 in	*dwg. 3/62*
				508	mm		793.75 mm	

A_{gross} = 100.5 in^2 S = 1025 in^3 I = 6666.2 10^6 mm^4 *dwg 3/62*

A_{gross} = 64838.6 mm^2 16796.74 $*10^3$ mm^3

$F_{y,\ carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u,\ carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.9			

Check Compressive Resistance (Cl 10.9.3)

$C_{r,1} = \phi_s A_g F_y(1+\lambda^{2n})^{-1/n}$ *Cl 10.9.3.1, p452*

$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$ *Cl 10.9.3.1, p452*

n = 1.34 *fabricated shapes*

K = 1.0

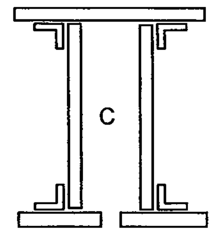
$r = (I/A)^{0.5}$ = 320.6 mm

λ = 0.32

$C_{r,1}$ = 12976 kN

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity			JOB NO. BT3252BTJ00
	Member Capacity - Lift Span - Top Chord U_3-U_5			PAGE 1 of 1
	MADE BY: KL	DATE: May. 23, 07	CHECKED BY:	DATE:

LIFT SPAN



Top Chord U_3-U_5 Compression Member

Built-up section

Angle (2)	Angle (2)	Web (2)	Top plate (1)	Bot plate (1)
4x4x7/16	8x6x3/4	30x1	38x1	(42-14)x7/8
102x102x11	203x152x19			

Design Criteria

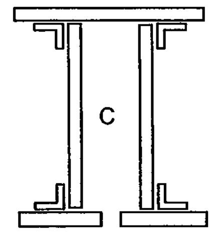
							*(2-6 1/2"+2*3/8")*		
Member Length =	9402	mm	otal width =	20	in	tal Depth =	31.25	in	*dwg. 3/62*
				508	mm		793.75	mm	
A_{gross} =	148.0	in^2	S =	1547	in^3	I =	10061.1	10^6 mm^4	*dwg 3/62*
A_{gross} =	95483.7	mm^2		25350.79	$*10^3$ mm^3				

$F_{y, carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u, carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.9			

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y (1+\lambda^{2n})^{-1/n}$			*Cl 10.9.3.1, p452*
$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$			*Cl 10.9.3.1, p452*
n =	1.34		*fabricated shapes*
K =	1.0		
$r = (I/A)^{0.5}$ =	324.6	mm	
λ =	0.31		
$C_{r.1}$ =	19135	kN	

LIFT SPAN



Top Chord U_5-U_6 Compression Member

Built-up section

Angle (2)	Angle (2)	Web (2)	Top plate (1)	Bot plate (1)
4x4x7/16	8x6x3/4	30x1/2	38x1	(42-14)x7/8
102x102x11	203x152x19			

Design Criteria

Member Length = 9398 mm   ɔtal width = 20 in / 508 mm   tal Depth = *(2-6 1/2"+2*3/8")* 31.25 in / 793.75 mm *dwg. 3/62*

A_{gross} = 156.5 in^2 S = 1581 in^3 I = 10282.2 10^6 mm^4 *dwg 3/62*

A_{gross} = 100967.5 mm^2 25907.95 $*10^3$ mm^3

$F_{y, carbon}$ = 230 MPa *A36 Carbon steel, 36 ksi* *Table 2-2, "Design of Steel Structures", p54*

$F_{u, carbon}$ = 420 MPa *A36 Carbon steel, 58-80 ksi* *Edwin H. Gaylord*

from Zuzana's notes

E = 200000 MPa

ϕ = 0.9

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y (1+\lambda^{2n})^{-1/n}$ *Cl 10.9.3.1, p452*

$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$ *Cl 10.9.3.1, p452*

n = 1.34 *fabricated shapes*

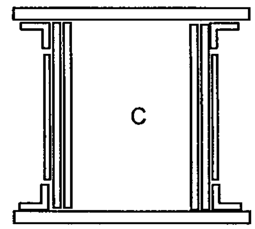
K = 1.0

$r = (I/A)^{0.5}$ = 319.1 mm

λ = 0.32

$C_{r.1}$ = 20205 kN

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity			JOB NO. BT3252BTJ00	
	Member Capacity - Lift Span - Diagonal L_0-U_1			PAGE 1 of 1	
	MADE BY: KL	DATE: May. 24, 07	CHECKED BY:	DATE:	



LIFT SPAN

Diagonal Member L_0-U_1 Compression Member

Built-up section

Angle (2)	Angle (2)	Web (4)	Web (2)	Top plate (1)	Bot plate (1)
4x4x7/16	8x6x3/4	30x3/4	18x3/4	38x1	(42-14)x7/8
102x102x11	203x152x19				

Design Criteria

Member Length =	17297	mm	otal width =	20	in	tal Depth =	32.375	in	*dwg. 3/62*
				508	mm		822.325	mm	
A_{gross} =	206.0	in^2	S =	1886	in^3	I =	12707.4	10^6 mm^4	*dwg 3/62*
A_{gross} =	132903.0	mm^2		30906	$*10^3$ mm^3				

$F_{y, carbon}$ = 230 MPa — *A36 Carbon steel, 36 ksi* — *Table 2-2, "Design of Steel Structures", p54*

$F_{u, carbon}$ = 420 MPa — *A36 Carbon steel, 58-80 ksi* — *Edwin H. Gaylord*

from Zuzana's notes

E = 200000 MPa

ϕ = 0.9

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y (1+\lambda^{2n})^{-1/n}$ — *Cl 10.9.3.1, p452*

$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$ — *Cl 10.9.3.1, p452*

n = 1.34 — *fabricated shapes*

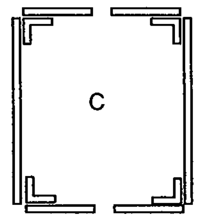
K = 1.0

$r = (I/A)^{0.5}$ = 309.2 mm

λ = 0.60

$C_{r.1}$ = 23170 kN

LIFT SPAN



Diagonal Member U_1-L_2 Tension Member

Built-up section

Angle (4)	Web (2)	Bot plate (1)
4x4x3/8	30x1	(22-8)x3/8
102x102x9.5		

Design Criteria

Member Length = 17297 mm *dwg. 3/62*

Total width = 20 in tal Depth = 31.25 in
508 mm 793.75 mm

$F_{y,\,carbon}$ = 230 MPa *A36 Carbon steel, 36 ksi* *Table 2-2, "Design of Steel Structures", p54*
$F_{u,\,carbon}$ = 420 MPa *A36 Carbon steel, 58-80 ksi* *Edwin H. Gaylord*
from Zuzana's notes

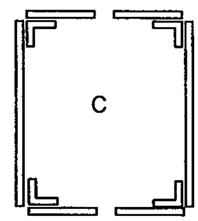
E = 200000 MPa
ϕ = 0.95

A_{gross} = 82.0 in^2 A_{net} = 69.3 in^2 *dwg 3/62*
A_{gross} = 52884 mm^2 A_{net} = 44710 mm^2

Check Tensile Resistance

$T_{r.1} = \phi_s A_{net} F_y$ = 9769.0 kN *Cl 10.8.2 (a), p 447*
$T_{r.2} = 0.85\phi_s A_{ne} F_u$ = 15163.3 kN *Cl 10.8.2 (b), p 448*
$T_r = \min(T_{r.1}, T_{r.2})$ = 9769.0 kN

LIFT SPAN



Diagonal Member L_2-U_3 Compression Member

Built-up section

Angle (4)	Web (2)	Plate (2)
4x4x3/8	30x3/4	(22-8)x3/8
102x102x9.5		

Design Criteria

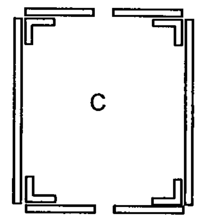
							*(2-6 1/2"+2*3/8")*		
Member Length =	18732	mm	ɔtal width =	20	in	tal Depth =	31.25	in	*dwg. 3/62*
				508	mm		793.75	mm	
A_{gross} =	66.6	in^2	S =	517	in^3	I =	3362.4	10^6 mm^4	*dwg 3/62*
A_{gross} =	42967.7	mm^2		8472.112	$*10^3$ mm^3				

$F_{y, carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u, carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.9			

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y(1+\lambda^{2n})^{-1/n}$			*Cl 10.9.3.1, p452*
$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$			*Cl 10.9.3.1, p452*
n =	1.34		*fabricated shapes*
K =	1.0		
$r = (I/A)^{0.5}$ =	279.7	mm	
λ =	0.72		
$C_{r.1}$ =	6850	kN	

LIFT SPAN



Diagonal Member U_3-L_4 Tension Member

Built-up section

Angle (4)	Web (2)	Plate (2)
4x4x3/8	26x9/16	(22.5-8)x3/8
102x102x9.5		

Design Criteria

Member Length = 18732 mm *dwg. 3/62*

Total width = 20 in / 508 mm   tal Depth = 27.25 in / 692.15 mm

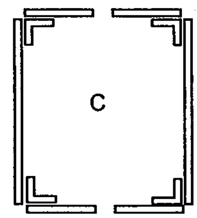
$F_{y, carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u, carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.95			

A_{gross} =	51.6	in^2	A_{net} =	41.8	in^2	*dwg 3/62*
A_{gross} =	33290	mm^2	A_{net} =	26968	mm^2	

Check Tensile Resistance

$T_{r.1} = \phi_s A_{net} F_y$ =	5892.4	kN	*Cl 10.8.2 (a), p 447*
$T_{r.2} = 0.85\phi_s A_{ne} F_u$ =	9146.1	kN	*Cl 10.8.2 (b), p 448*
$T_r = \min(T_{r.1}, T_{r.2})$ =	5892.4	kN	

LIFT SPAN



Diagonal Member L_4-U_5 Compression Member

Built-up section

Angle (4)	Web (2)	Plate (2)
4x4x3/8	26x11/16	(22-8)x3/8
102x102x9.5		

Design Criteria

							(2-6 1/2"+2*3/8")		
Member Length =	19219	mm	ɔtal width =	20	in	tal Depth =	27.25	in	*dwg. 3/62*
				508	mm		692.15	mm	
A_{gross} =	57.1	in^2	S =	405	in^3	I =	2296.8	10^6 mm^4	*dwg 3/62*
A_{gross} =	36838.6	mm^2		6636.761	$*10^3$ mm^3				

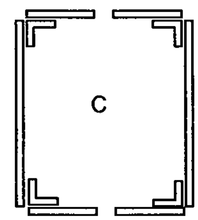
$F_{y,\ carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u,\ carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.9			

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y (1+\lambda^{2n})^{-1/n}$			*Cl 10.9.3.1, p452*
$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$			*Cl 10.9.3.1, p452*
n =	1.34		*fabricated shapes*
K =	1.0		
$r = (I/A)^{0.5}$ =	249.7	mm	
λ =	0.83		
$C_{r.1}$ =	5348	kN	

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity Member Capacity - Lift Span - Diagonal U_5-L_6		JOB NO. BT3252BTJ00 PAGE 1 of 1	
	MADE BY: KL	DATE: Jun. 6, 07	CHECKED BY:	DATE:

LIFT SPAN



Diagonal Member U_5-L_6 Tension Member

Built-up section

Angle (4)	Web (2)	Bot plate (1)
4x4x3/8	24x9/16	(23-8)x3/8
102x102x9.5		

Design Criteria

Member Length =	19219	mm	*dwg. 3/62*			
Total width =	20	in	tal Depth =	25.25	in	
	508	mm		641.35	mm	
$F_{y,\ carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*			*Table 2-2, "Design of Steel Structures", p54*
$F_{u,\ carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*			*Edwin H. Gaylord*
			from Zuzana's notes			
E =	200000	MPa				
ϕ =	0.95					

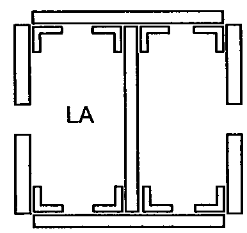
A_{gross} =	48.7	in^2	A_{net} =	40.8	in^2	*dwg 3/62*
A_{gross} =	31419	mm^2	A_{net} =	26323	mm^2	

Check Tensile Resistance

$T_{r.1} = \phi_s A_{net} F_y$ =	5751.5	kN	*Cl 10.8.2 (a), p 447*
$T_{r.2} = 0.85\phi_s A_{ne} F_u$ =	8927.3	kN	*Cl 10.8.2 (b), p 448*
$T_r = \min(T_{r.1}, T_{r.2})$ =	5751.5	kN	

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity		JOB NO. BT3252BTJ00	
	Member Capacity - Lift Span - Vertical Member U_0-L_0		PAGE 1 of 1	
	MADE BY: KL	DATE: May. 23, 07	CHECKED BY:	DATE:

LIFT SPAN



Vertical Member U_0-L_0 Tension Member

Built-up section

Angle (4)	Angle (4)	Web (2)	Web (1)	Web (2)	Top plate (1)	Bot plate (1)
5x5x3/4	4x4x3/4	23x1/2	23x7/8	13x3/4	(23-8)x3/8	(23-8)x3/8
203x152x19	102x102x19					

Design Criteria

Member Length = 13258.8 m *dwg. 3/62*

Total width = in — 0 mm; tal Depth = 24.5 in — 622.3 mm

$F_{y, carbon}$ =	290	MPa	*Low Alloy*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u, carbon}$ =	434	MPa	*from Zuzana's notes*	*Edwin H. Gaylord*

E = 200000 MPa

ϕ = 0.95

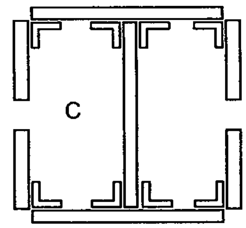
A_{gross} =	139.35	in^2	A_{net} =	104.2	in^2	*dwg 3/62*
A_{gross} =	89903	mm^2	A_{net} =	67251	mm^2	

Check Tensile Resistance

$T_{r.1} = \phi_s A_{net} F_y$ =	18527.8	kN	*Cl 10.8.2 (a), p 447*
$T_{r.2} = 0.85\phi_s A_{ne} F_u$ =	23568.6	kN	*Cl 10.8.2 (b), p 448*
$T_r = \min(T_{r.1}, T_{r.2})$ =	18527.8	kN	

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity		JOB NO. BT3252BTJ00	
	Member Capacity - Lift Span - Vertical Member $U_{1,3,5}$-$L_{1,3,5}$		PAGE 1 of 1	
	MADE BY: KL	DATE: May. 23, 07	CHECKED BY:	DATE:

LIFT SPAN



Vertical Member U_1-L_1, U_3L_3, U_5L_5 Tension Member

Built-up section

Angle (4)	Angle (4)	Web (1)	Web (2)	Top plate (1)	Bot plate (1)
5x5x5/8	4x4x1/2	23x1/2	(23-10)x3/8	19 1/2x1/2	20 1/2x1/2
127x127x16	102x102x13				

Design Criteria

Total width =		in	tal Depth =	24.5	in
	0	mm		622.3	mm

$F_{y,\,carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u,\,carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.95			

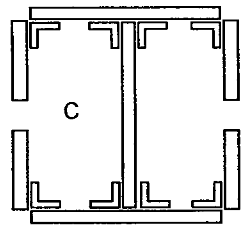
A_{gross} =	79.31	in^2	A_{net} =	60.6	in^2	*dwg 3/62*
A_{gross} =	51168	mm^2	A_{net} =	39097	mm^2	

Check Tensile Resistance

$T_{r.1} = \phi_s A_{net} F_y$ =	8542.6	kN	*Cl 10.8.2 (a), p 447*
$T_{r.2} = 0.85\phi_s A_{ne} F_u$ =	13259.6	kN	*Cl 10.8.2 (b), p 448*
$T_r = \min(T_{r.1}, T_{r.2})$ =	8542.6	kN	

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity		JOB NO. BT3252BTJ00
	Member Capacity - Lift Span - Vertical Member U_2-L_2		PAGE 1 of 1
	MADE BY: KL	DATE: May. 23, 07	CHECKED BY:	DATE:

LIFT SPAN



Vertical Member U_2-L_2 Compression member

Built-up section

Angle (2)	Angle (2)	Web (2)	Top plate (1)	Bot plate (1)
4x4x7/16	8x6x3/4	30x1/2	38x1	(42-14)x7/8
102x102x11	203x152x19			

Design Criteria

Member Length =	15363	mm	otal width =	20	in	tal Depth =	24.5	in	*dwg. 3/62*
				508	mm		622.3	mm	
A_{gross} =	66.3	in^2	S =	555	in^3	I =	2829.9	10^6 mm^4	*dwg 3/62*
A_{gross} =	42774.1	mm^2		9094.821	$*10^3$ mm^3				

$F_{y, carbon}$ = 230 MPa *A36 Carbon steel, 36 ksi* *Table 2-2, "Design of Steel Structures", p54*

$F_{u, carbon}$ = 420 MPa *A36 Carbon steel, 58-80 ksi* *Edwin H. Gaylord*

from Zuzana's notes

E = 200000 MPa

ϕ = 0.9

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y(1+\lambda^{2n})^{-1/n}$ *Cl 10.9.3.1, p452*

$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$ *Cl 10.9.3.1, p452*

n = 1.34 *fabricated shapes*

K = 1.0

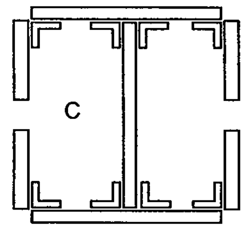
$r = (I/A)^{0.5}$ = 257.2 mm

λ = 0.64

$C_{r.1}$ = 7245 kN

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity		JOB NO. BT3252BTJ00	
	Member Capacity - Lift Span - Vertical Member U_4-L_4		PAGE 1 of 1	
	MADE BY: KL	DATE: May. 23, 07	CHECKED BY:	DATE:

LIFT SPAN



Vertical Member U_4-L_4 Compression member

Built-up section

Angle (2)	Angle (2)	Web (2)	Top plate (1)	Bot plate (1)
4x4x7/16	8x6x3/4	30x1/2	38x1	(42-14)x7/8
102x102x11	203x152x19			

Design Criteria

Member Length =	16484	mm	ɔtal width =	20	in	tal Depth =	24.5	in	*dwg. 3/62*
				508	mm		622.3	mm	

A_{gross} = 66.3 in^2 S = 555 in^3 I = 2829.9 10^6 mm^4 *dwg 3/62*

A_{gross} = 42774.1 mm^2 9094.821 $*10^3$ mm^3

$F_{y,\ carbon}$ = 230 MPa *A36 Carbon steel, 36 ksi* *Table 2-2, "Design of Steel Structures", p54*

$F_{u,\ carbon}$ = 420 MPa *A36 Carbon steel, 58-80 ksi* *Edwin H. Gaylord*

from Zuzana's notes

E = 200000 MPa

ϕ = 0.9

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y(1+\lambda^{2n})^{-1/n}$ *Cl 10.9.3.1, p452*

$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$ *Cl 10.9.3.1, p452*

n = 1.34 *fabricated shapes*

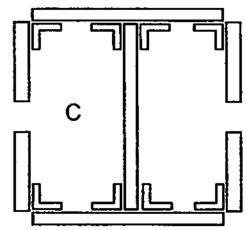
K = 1.0

$r = (I/A)^{0.5}$ = 257.2 mm

λ = 0.69

$C_{r.1}$ = 6991 kN

LIFT SPAN



Vertical Member U_6-L_6 Compression member

Built-up section

Angle (2)	Angle (2)	Web (2)	Top plate (1)	Bot plate (1)
4x4x7/16	8x6x3/4	30x1/2	38x1	(42-14)x7/8
102x102x11	203x152x19			

Design Criteria

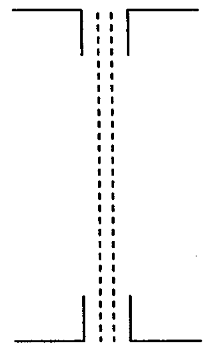
Member Length =	16764	mm	otal width =	20	in	tal Depth =	24.5	in	*dwg. 3/62*
				508	mm		622.3	mm	
A_{gross} =	66.3	in^2	S =	555	in^3	I =	2829.9	10^6 mm^4	*dwg 3/62*
A_{gross} =	42774.1	mm^2		9094.821	$*10^3$ mm^3				

$F_{y,\ carbon}$ =	230	MPa	*A36 Carbon steel, 36 ksi*	*Table 2-2, "Design of Steel Structures", p54*
$F_{u,\ carbon}$ =	420	MPa	*A36 Carbon steel, 58-80 ksi*	*Edwin H. Gaylord*
			from Zuzana's notes	
E =	200000	MPa		
ϕ =	0.9			

Check Compressive Resistance (Cl 10.9.3)

$C_{r.1} = \phi_s A_g F_y(1+\lambda^{2n})^{-1/n}$			*Cl 10.9.3.1, p452*
$\lambda = (KL/r)[F_y/(\pi^2 E_s)]^{0.5}$			*Cl 10.9.3.1, p452*
n =	1.34		*fabricated shapes*
K =	1.0		
$r = (I/A)^{0.5}$ =	257.2	mm	
λ =	0.70		
$C_{r.1}$ =	6926	kN	

LIFT SPAN



Top Laterals (Carbon)

Built-up section

	Angle (4)	D. Lace
	7x4x7/16	2 3/4x1/2
	178x102x11	

Design Criteria

Total width =		in	Total Depth =	30.5	in	*(2-6 1/2")*
	229.4	mm		774.7	mm	
	Angle			Lace		
A =	2980	mm^2	t =	1/2	in	
y =	60.8	mm		12.7	mm	
x =	22.8	mm	w =	2 3/4	in	
b =	102			69.85	mm	
d =	178					
t =	11.1					
I_x =	9.88	10^6 mm^2				
I_y =	2.45	10^6 mm^2				

Calculate Area

A_{angle}=4A= 11920 mm^2

$A_{double\ lacing}$= 0.0254 m^2/m

A_{gross} = 0.03732 m^2

Calculate Moment of Inertia

$y_{x-x} = h/2 =$ 387.35 mm

$d_{x-x,\ angle} = y_{x-x}-y =$ 364.55 mm

$I_{x-x,\ tot} =$ 0.001594 m^4

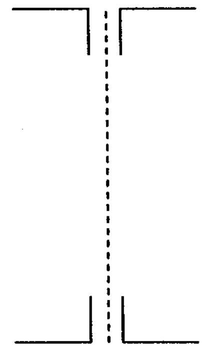
$y_{y-y} = b/2 =$ 114.7 mm

$d_{y-y,\ angle} = x+y_{y-y} =$ 73.05 mm

$I_{y-y,\ tot} =$ 0.000103 m^4

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity	JOB NO. BT3252BTJ00
	Member Properties - Bottom Laterals	PAGE 1 of 1
	MADE BY: KL / DATE: Jun. 4, 07 / CHECKED BY:	DATE:

LIFT SPAN



Bottom Laterals (Carbon)

Built-up section

	Angle (4)	D. Lace
	7x4x7/16	2 3/4x1/2
	178x102x11	

Design Criteria

Total width =		in	Total Depth =	31.25	in	*(2-7 1/4")*
	365.5	mm		793.75	mm	

	Angle			Lace	
A =	2980	mm^2	t =	1/2	in
y =	60.8	mm		12.7	mm
x =	22.8	mm	w =	2 3/4	in
b =	102			69.85	mm
d =	178				
t =	11.1				
I_x =	9.88	10^6 mm^2	9.656		
I_y =	2.45	10^6 mm^2			

Calculate Area

A_{angle}=4A= 11920 mm^2

$A_{double\ lacing}$= 0.0254 m^2/m

A_{gross} = 0.03732 m^2

Calculate Moment of Inertia

y_{x-x} = h/2 = 396.875 mm

$d_{x-x,\ angle}$ = y_{x-x}-y = 374.075 mm

$I_{x-x,\ tot}$ = 0.001678 m^4

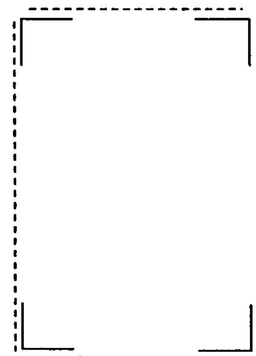
y_{y-y} = b/2 = 182.75 mm

$d_{y-y,\ angle}$ = x+y_{y-y} = 73.05 mm

$I_{y-y,\ tot}$ = 0.000103 m^4

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity		JOB NO. BT3252BTJ00
	Member Properties - Top Strut		PAGE 1 of 1
	MADE BY: KL	DATE: Jun. 4, 07	CHECKED BY:	DATE:

LIFT SPAN



Top Strut (Carbon)

Built-up section

Angle (4)	D.L.-sides	D.L.-top & bot.
5x5x3/8	2 3/4x1/2	3 3/4x1/2
127x127x9.5		

Design Criteria

Total width =	22	in	tal Depth =	31.5	in	*(2'-6 1/2"+2*1/2)*
	558.8	mm		800.1	mm	
	Angle			Lace		
A =	2330	mm^2	t =	1/2	in	
y =	35.3	mm		12.7	mm	
x =	35.3	mm	w =	2 3/4	in	
b =	127			69.85	mm	
d =	127					
t =	9.53					
I_x =	3.64	10^6 mm^2				
I_y =	3.64	10^6 mm^2				

Calculate Area

A_{angle}=4A= 9320 mm^2

$A_{double\ lacing}$= 0.0254 m^2/m $A_{single\ lacing}$= 0.0127 m^2/m

A_{gross} =	0.08552	m^2

Calculate Moment of Inertia

y_{x-x} = h/2 = 400.05 mm

$d_{x-x,\ angle}$ = y_{x-x}-y = 364.75 mm

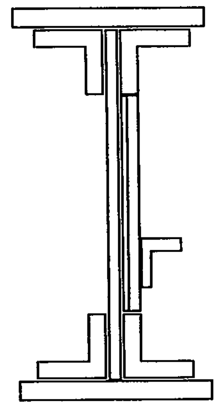
$I_{x-x,\ tot}$ =	0.001255	m^4

y_{y-y} = b/2 = 279.4 mm

$d_{y-y,\ angle}$ = x+y_{y-y} = 244.1 mm

$I_{y-y,\ tot}$ =	0.00057	m^4

LIFT SPAN



Intermediate Floor Beam dwg. 5/62

Built-up section

	Angle	Web	Cover plate	Long. Traction Bracing	Stiffener
	4	1	2		
	8x8x7/8	78x9/16	20x3/4	6x4x1/2	7x4x5/8
	203x203x22			152x102x13	172x102x16

Design Criteria

Total width, b = 20 in (508 mm) Depth, d = 80 in (2032 mm) *(6-6 1/2+2*3/4)*

Angle			Web			Cover plate		
A =	8520	mm^2	w =	9/16	in	t =	3/4	in
y =	58.9	mm		14.2875	mm		19.05	mm
x =	58.9	mm	h =	78	in	b =	20	in
t =	22.2	mm		1981.2	mm		508	mm
b =	203	mm						
d =	203	mm						
$A_{one\ leg}$ =	4506.6	mm^2						
$I_{angle,x}$ =	33	$10^6\ mm^4$	$I_{web,x}$ =	9258.91	$10^6\ mm^4$	$I_{cover,x}$ =	0.2927	$10^6\ mm^4$
$I_{angle,y}$ =	33	$10^6\ mm^4$	$I_{web,y}$ =	0.481522	$10^6\ mm^4$	$I_{cover,y}$ =	208.1157	$10^6\ mm^4$

Cqlculate Section Properties

Calculate Area

A_{angle}=4A= 34080 mm^2 A_{web}=hw= 28306.4 mm^2 A_{cp}=ht= 9677.4 mm^2 2A_{cp}= 19354.8 mm^2

A_{total} = 81741 mm^2 0.0817 mm^2

Calculate Centroid taken about the bottom of the section

$y_{top\ angles}$ = 1954.1 mm y_{web} = 1016.0 mm $y_{top\ cover}$ = 2022.5 mm
$y_{bot\ angles}$ = 78.0 mm $y_{bot\ cover}$ = 9.5 mm

$y_{centroid}$ = 1016 mm

Calculate Moment of Inertia

$I_{x-x} = 1/12*[bd^3-(b-w)(d-2t)^3]$ $S_{x-x} = 1/(6d)*[bd^3-(b-w)(d-2t)^3]$
$I_{y-y} = 1/12*[2tb^3-(d-2t)w^3]$ $S_{x-x} = 1/(6b)*[2tb^3-(d-2t)w^3]$

I_{x-x} = 29045.0 $10^6\ mm^4$ S_{x-x} = 28.588 $10^6\ mm^3$
I_{y-y} = 415.75 $10^6\ mm^4$ S_{y-y} = 1.637 $10^6\ mm^3$

G_{steel} = 77000 MPa ϕ = 0.95
E = 200000 MPa
F_y = 230 MPa

$I_{angles, t\text{-}x}$ = 30120 10^6 mm^4 $I_{web, t\text{-}x}$ = 9258.91 10^6 mm^4 $I_{cover, t\text{-}x}$ = 19606.84 10^6 mm^4
$I_{angles, t\text{-}y}$ = 281 10^6 mm^4 $I_{web, t\text{-}y}$ = 0.4815 10^6 mm^4 $I_{cover, t\text{-}y}$ = 416.231 10^6 mm^4

$I_{total, x}$ = 58986.0 10^6 mm^4 0.0590 m^4
$I_{total, y}$ = 697.36 10^6 mm^4 0.00070 m^4

$S_{total, x} = I_x/y$ = 58057.1 10^3 mm^3 0.000058 m^4
$S_{total, y} = I_y/y$ = 686.38 10^3 mm^3

<u>*Check Class*</u>

b' = b/2 = 254 mm
$t' = t_{cp}+t_{angle}$ = 41.25 mm
$h'_{web} = h_{web}-2^*d_{angle}$ = 1575.2 mm

Flange (Top and Bottom the same)

b'/t' = 6.2
200/sqrt(F_y) = 13.2 *Tab 10.9.2.1, p451*
Flange is Class 3

Web

h'_{web}/w_{web} = 110.3
1900/sqrt(F_y) = 125.3
Web is Class 3 *Tab 10.9.2.1, p451*

<u>*Check Moment Capacity*</u>

Note: *Flange Class is 3 and web is Class 3 , Stiffened plate girder, laterally unsupported*

<u>*Check Width-to-Thickness Ratios Webs*</u>

3150/sqrt(F_y) = 207.7 *Cl. 10.10.4.2(a)*
h/w ratio is OK *treated as longitudinal unstiffened*

<u>*Moment Required*</u>

M_f = 12000.0 kNm *from LUSAS Model*

<u>*Moment Resistance (Cl. 10.10.4.3)*</u>

a. Treat as girder without longitudinal stiffeners
b. Check if $2d_c/w > 1900/F_y^{0.5}$ *Cl . 10.10.4.3, p 458*

$2d_c/w$ = 142.2
$1900/Fy^{0.5}$ = 125.3
2dc/w>1900/sqrt(Fy), need to reduce the moment *Cl . 10.10.4.3, p 458*

c. Calculate reduction factor

Reduction Factor = $1.0-1/[300+(1200A_{cf})/A_w]^*[2d_c/w-1900/(M_f/\phi_s S)^{0.5}]$ *Cl . 10.10.4.3, p 458*

$A_{cf} = A_{cp}+2^*A_{angle}$ = 18690.6 mm^2

$A_w = A_{web}+2*A_{angle} =$ 37319.6 mm^2
$d_c = h_{tot}/2 =$ 1016.0 mm
Reduction Factor = 0.985
RD = 0.985

d. Calculate $M_r = \phi_s S_x F_y$ (10.10.3.2) *Cl. 10.10.3.2, p458*

$M_r = \phi_s S_x F_y =$ 12685.5 kNm *treat as laterally supported*

e. Calculate Reduced Moment Resistance

$M_{r,\,final} = RF*M_r =$ 12496.7 kNm

Shear Resistance cl 10.10.5.1

a/h ratio

a = 1295.4 mm
h = 1981.2 mm
a/h = 0.654
a/h<1 *cl. 10.10.5.1, p459*

use kv = 4+5.34/(a/h)^2

$k_v =$ 16.51

h/w = 138.7

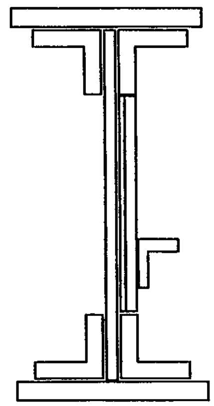
$502(k_v/F_y)^{0.5} =$ 134.51
$621(k_v/F_y)^{0.5} =$ 166.40
502(kv/Fy)^0.5<h/w<621(kv/Fy)^0.5

$F_{cr} = 290(F_y k_v)^{0.5}/(h/w) =$ 128.9 MPa
$F_t = (0.5F_y - 0.866F_{cr})*\{1/[1+(a/h)^2]\} =$ 2.83 MPa
$F_s = F_{cr} + F_t =$ 131.72 MPa

$A_w = h*w =$ 28306.4 mm^2

$V_r = \phi_s A_w F_s =$ 3542 kN

LIFT SPAN



End Floor Beam dwg. 6/62

Built-up section

	Angle	Web	Cover plate	Long. Traction Bracing	Stiffener
	4	1	2		
	8x8x3/4	71x1/2	20x1/2	6x4x1/2	7x4x5/8
	203x203x19			152x102x13	172x102x16

Design Criteria

Total width, b = 20 in (508 mm) Depth, d = 80 in (2032 mm) *(6-6 1/2+2*3/4)*

Angle			Web			Cover plate		
A =	7390	mm^2	w =	1/2	in	t =	1/2	in
y =	57.8	mm		12.7	mm		12.7	mm
x =	57.8	mm	h =	71	in	b =	20	in
t =	19.1	mm		1803.4	mm		508	mm
b =	203	mm						
d =	203	mm						
$A_{one\ leg}$ =	3877.3	mm^2						
$I_{angle,x}$ =	29	$10^6\ mm^4$	$I_{web,x}$ =	6207.24	$10^6\ mm^4$	$I_{cover,x}$ =	0.0867	$10^6\ mm^4$
$I_{angle,y}$ =	29	$10^6\ mm^4$	$I_{web,y}$ =	0.307838	$10^6\ mm^4$	$I_{cover,y}$ =	138.7438	$10^6\ mm^4$

Cqlculate Section Properties

Calculate Area

A_{angle}=4A= 29560 mm^2 A_{web}=hw= 22903.18 mm^2 A_{cp}=ht= 6451.6 mm^2 $2A_{cp}$= 12903.2 mm^2

A_{total} =	65366	mm^2	0.0654	mm^2

Calculate Centroid taken about the bottom of the section

$y_{top\ angles}$ = 1961.5 mm y_{web} = 1016.0 mm $y_{top\ cover}$ = 2025.7 mm

$y_{bot\ angles}$ = 70.5 mm $y_{bot\ cover}$ = 6.4 mm

$y_{centroid}$ = 1016 mm

Calculate Moment of Inertia

$I_{x-x} = 1/12*[bd^3-(b-w)(d-2t)^3]$ $S_{x-x} = 1/(6d)*[bd^3-(b-w)(d-2t)^3]$

$I_{y-y} = 1/12*[2tb^3-(d-2t)w^3]$ $S_{x-x} = 1/(6b)*[2tb^3-(d-2t)w^3]$

I_{x-x} = 21704.4 $10^6\ mm^4$ S_{x-x} = 21.363 $10^6\ mm^3$

I_{y-y} = 277.15 $10^6\ mm^4$ S_{y-y} = 1.091 $10^6\ mm^3$

G_{steel} = 77000 MPa ϕ = 0.95
E = 200000 MPa
F_y = 230 MPa

$I_{angles, t\text{-}x}$ =	26542	10^6 mm^4	$I_{web, t\text{-}x}$ =	6207.24	10^6 mm^4	$I_{cover, t\text{-}x}$ =	13153.61	10^6 mm^4
$I_{angles, t\text{-}y}$ =	238	10^6 mm^4	$I_{web, t\text{-}y}$ =	0.3078	10^6 mm^4	$I_{cover, t\text{-}y}$ =	277.488	10^6 mm^4

$I_{total, x}$ =	45902.6	10^6 mm^4	0.0459	m^4
$I_{total, y}$ =	515.44	10^6 mm^4	0.00052	m^4

$S_{total, x} = I_x/y$ =	45179.7	10^3 mm^3	0.000045	m^4
$S_{total, y} = I_y/y$ =	507.32	10^3 mm^3		

Check Class

b' = b/2 = 254 mm
t' = $t_{cp}+t_{angle}$ = 31.8 mm
$h'_{web} = h_{web}-2*d_{angle}$ = 1397.4 mm

Flange (Top and Bottom the same)

b'/t' = 8.0
200/sqrt(F_y) = 13.2 *Tab 10.9.2.1, p451*
Flange is Class 3

Web

h'_{web}/w_{web} = 110.0
1900/sqrt(F_y) = 125.3
Web is Class 3 *Tab 10.9.2.1, p451*

Check Moment Capacity

Note: Flange Class is 3 and web is Class 3 , Stiffened plate girder, laterally unsupported

Check Width-to-Thickness Ratios Webs

3150/sqrt(F_y) = 207.7 *Cl. 10.10.4.2(a)*
h/w ratio is OK *treated as longitudinal unstiffened*

Moment Required

M_f = 9000.0 kNm *from LUSAS Model*

Moment Resistance (Cl. 10.10.4.3)

a. Treat as girder without longitudinal stiffeners
b. Check if $2d_c/w > 1900/F_y^{0.5}$ *Cl . 10.10.4.3, p 458*

$2d_c/w$ = 160.0
$1900/Fy^{0.5}$ = 125.3
2dc/w>1900/sqrt(Fy), need to reduce the moment *Cl . 10.10.4.3, p 458*

c. Calculate reduction factor

Reduction Factor = $1.0-1/[300+(1200A_{cf})/A_w]*[2d_c/w-1900/(M_f/\phi_s S)^{0.5}]$ *Cl . 10.10.4.3, p 458*

$A_{cf} = A_{cp}+2*A_{angle} =$ 14206.2 mm^2
$A_w = A_{web}+2*A_{angle} =$ 30657.8 mm^2
$d_c = h_{tot}/2 =$ 1016.0 mm
Reduction Factor = 0.966
RD = 0.966

d. Calculate $M_r = \phi_s S_x F_y$ (10.10.3.2) *Cl. 10.10.3.2, p458*

$M_r = \phi_s S_x F_y =$ 9871.8 kNm *treat as laterally supported*

e. Calculate Reduced Moment Resistance

$M_{r,\,final} = RF*M_r =$ 9539.8 kNm

Shear Resistance cl 10.10.5.1

a/h ratio

a = 1295.4 mm
h = 1803.4 mm
a/h = 0.718
a/h<1

use kv = 4+5.34/(a/h)^2

$k_v =$ 14.37

h/w = 142.0

$502(k_v/F_y)^{0.5} =$ 125.47
$621(k_v/F_y)^{0.5} =$ 155.22
502(kv/Fy)^0.5<h/w<621(kv/Fy)^0.5

$F_{cr} = 290(F_y k_v)^{0.5}/(h/w) =$ 117.4 MPa
$F_t = (0.5F_y - 0.866F_{cr})*\{1/[1+(a/h)^2]\} =$ 10.82 MPa
$F_s = F_{cr} + F_t =$ 128.23 MPa

$A_w = h*w =$ 22903.18 mm^2

$V_r = \phi_s A_w F_s =$ 2790 kN

LIFT SPAN

	Imperial	Metric
Stringer	***24WF84***	***W610x125***

Design Criteria

d =	612	mm	I_x =	985	10^6 mm^4	J =	1540	10^3 mm^4
b =	229	mm	I_y =	39.3	10^6 mm^4	C_w =	3450	10^9 mm^6
t =	19.6	mm	S_x =	3220	10^3 mm^3	G_{steel} =	77000	MPa
w =	11.9	mm	Z_x =	3670	10^3 mm^3			
F_y =	230	MPa	ϕ =	0.95		E =	200000	MPa
L =	3150	mm						

Class Check

b' = b/2 = 114.5 mm

h = d-2t = 572.8 mm

Flange (Top and Bottom the same)

b'/t = 5.8

145/sqrt(F_y) = 9.6 — *Tab 10.9.2.1, p451*

Flange is Class I

Web

h/w = 48.1

1100/sqrt(F_y) = 72.5

Web is Class I — *Tab 10.9.2.1, p451*

Check Moment Capacity — *Class 1, Laterally unsupported member*

Check if M_u>0.67M_p

Calculate M_p

$M_p = ZF_y$ = 844 kNm — *Class 1 section, cl 10.10.2.2, p457*

$0.67M_p$ = 566 kNm

Calculate M_u — for doubly symmetric sections

$M_u = (\omega_2\pi/L)[E_sI_yG_sJ+(\pi E/L)^2I_yC_w]^{0.5}$ — *cl 10.10.2.3*

ω_2 = 1.0 — *cl 10.10.2.3*

M_u = 2509 kNm

Case (a) Mu>0.67Mp — *case a applies*

Calculate M_r

$M_r = 1.15\phi M_p(1-0.28M_p/M_u) < \phi M_p$

M_r = 835.3 kNm

ϕM_p = 801.9 kNm

$M_{r,\,final} = min(M_r,\ \phi M_p)$

M_r = 802 kNm

Simplified Method — Cl 5.7.1.2.1, p174

$F_m = SN/(F(1+\mu C_f/100)$

$F =$	9.5		*Tab A 5.7.1.2.1, p215*
$C_f =$	0		
$S =$	1.2954	m	
$N =$	12		*# of stringers*

Calculate μ

$W_c =$	13.4112	m	
$n =$	4		*# of design lanes*
$W_e = W_c/n =$	3.3528	m	
$\mu = (W_e-3.3)/0.6 =$	0.088	<=1.0	

mu<=1.0, OK

Calculate F_m

$F_m = SN/(F(1+\mu C_f/100) =$ 1.64 >=1.05

Claculate Reduction Factor

$R_L =$	0.70		*Reduction factor for multilane loading, 3.8.4.2, p55* *4 lanes*
Reduction Factor = F_m*nR_L/N =	0.38		
$M_T =$	598	kNm	*LUSAS results "BCLB-stringer.mdl"*
$M_g = RF*M_T =$	228	kNm	
LL Factor =	1.7		
DLA =	0.25		
Factored $M_f =$	485	kNm	

Shear Resistance *cl 10.10.5.1, p459*

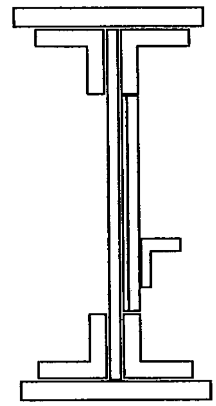
a/h ratio — unstiffened, a/h=infinity, $k_v = 5.34$ — *cl. 10.10.5.1, p459*

$k_v =$	5.34		no stiffeners, a/h is infinity
$h/w =$	48.1		
$502(k_v/F_y)^{0.5} =$	76.49		
$621(k_v/F_y)^{0.5} =$	94.62		

h/w<502(kv/Fy)^0.5

$F_{cr} = 0.577F_y =$	132.7	MPa
$F_t =$	0	MPa
$F_s = F_{cr} + F_t =$	132.71	MPa
$A_w = h*w =$	6816.32	mm^2
$V_r = \phi_s A_w F_s =$	859	kN

LIFT SPAN



End Floor Beam dwg. 14/62

Built-up section

	Angle	Web	Cover plate	Long. Traction Bracing	Stiffener
	4	1	2		
	8x8x1/2	71x1/2	20x1/2	6x4x1/2	7x4x5/8
	203x203x13			152x102x13	172x102x16

Design Criteria

Total width, b = 20 in, 508 mm   Depth, d = 72.375 in, 1838.325 mm   *(5-11 3/8+2*1/2)*

Angle			Web			Cover plate		
A =	4990	mm^2	w =	1/2	in	t =	1/2	in
y =	55.5	mm		12.7	mm		12.7	mm
x =	55.5	mm	h =	71	in	b =	20	in
t =	12.7	mm		1803.4	mm		508	mm
b =	203	mm						
d =	203	mm						
$A_{one\ leg}$ =	2578.1	mm^2						
$I_{angle,x}$ =	20.2	$10^6\ mm^4$	$I_{web,x}$ =	6207.24	$10^6\ mm^4$	$I_{cover,x}$ =	0.0867	$10^6\ mm^4$
$I_{angle,y}$ =	20.2	$10^6\ mm^4$	$I_{web,y}$ =	0.307838	$10^6\ mm^4$	$I_{cover,y}$ =	138.7438	$10^6\ mm^4$

Cqlculate Section Properties

Calculate Area

A_{angle}=4A= 19960 mm^2 A_{web}=hw= 22903.18 mm^2 A_{cp}=ht= 6451.6 mm^2 $2A_{cp}$= 12903.2 mm^2

A_{total} =	55766	mm^2	0.0558	mm^2

Calculate Centroid taken about the bottom of the section

$y_{top\ angles}$ = 1770.1 mm y_{web} = 919.2 mm $y_{top\ cover}$ = 1832.0 mm

$y_{bot\ angles}$ = 68.2 mm $y_{bot\ cover}$ = 6.4 mm

$y_{centroid}$ = 919 mm

Calculate Moment of Inertia

$I_{x-x} = 1/12*[bd^3-(b-w)(d-2t)^3]$ $S_{x-x} = 1/(6d)*[bd^3-(b-w)(d-2t)^3]$

$I_{y-y} = 1/12*[2tb^3-(d-2t)w^3]$ $S_{x-x} = 1/(6b)*[2tb^3-(d-2t)w^3]$

I_{x-x} = 17057.6 $10^6\ mm^4$ S_{x-x} = 18.558 $10^6\ mm^3$

I_{y-y} = 277.18 $10^6\ mm^4$ S_{y-y} = 1.091 $10^6\ mm^3$

G_{steel} = 77000 MPa ϕ = 0.95
E = 200000 MPa
F_y = 230 MPa

$I_{angles,\ t\text{-}x}$ =	14535	$10^6\ mm^4$	$I_{web,\ t\text{-}x}$ =	6207.24	$10^6\ mm^4$	$I_{cover,\ t\text{-}x}$ =	10751.46	$10^6\ mm^4$
$I_{angles,\ t\text{-}y}$ =	157	$10^6\ mm^4$	$I_{web,\ t\text{-}y}$ =	0.3078	$10^6\ mm^4$	$I_{cover,\ t\text{-}y}$ =	277.488	$10^6\ mm^4$

$I_{total,\ x}$ =	31493.3	$10^6\ mm^4$	0.0315	m^4
$I_{total,\ y}$ =	434.95	$10^6\ mm^4$	0.00043	m^4

$S_{total,\ x} = I_x/y$ =	34263.0	$10^3\ mm^3$	0.000034	m^4
$S_{total,\ y} = I_y/y$ =	473.20	$10^3\ mm^3$		

S_{net} = 1840 in^2 *from dwg. 13/62*
30152 $10^3\ mm^3$

Check Class

b' = b/2 = 254 mm
$t' = t_{cp} + t_{angle}$ = 25.4 mm
$h'_{web} = h_{web} - 2^*d_{angle}$ = 1397.4 mm

Flange (Top and Bottom the same)

b'/t' = 10.0
$200/\text{sqrt}(F_y)$ = 13.2 *Tab 10.9.2.1, p451*
Flange is Class 3

Web

h'_{web}/w_{web} = 110.0
$1900/\text{sqrt}(F_y)$ = 125.3
Web is Class 3 *Tab 10.9.2.1, p451*

Check Moment Capacity

Note: Flange Class is 3 and web is Class 3 , Stiffened plate girder, laterally unsupported

Check Width-to-Thickness Ratios Webs

$3150/\text{sqrt}(F_y)$ = 207.7 *Cl. 10.10.4.2(a)*
h/w ratio is OK *treated as longitudinal unstiffened*

Moment Required

M_f = 6000.0 kNm *from LUSAS Model*

Moment Resistance (Cl. 10.10.4.3)

a. Treat as girder without longitudinal stiffeners
b. Check if $2d_c/w > 1900/F_y^{0.5}$ *Cl. 10.10.4.3, p 458*

$2d_c/w$ = 144.8
$1900/Fy^{0.5}$ = 125.3
2dc/w>1900/sqrt(Fy), need to reduce the moment *Cl. 10.10.4.3, p 458*

c. Calculate reduction factor

Reduction Factor = $1.0-1/[300+(1200A_{cf})/A_w]*[2d_c/w-1900/(M_f/\phi_s S)^{0.5}]$ — *Cl. 10.10.4.3, p 458*

$A_{cf} = A_{cp}+2*A_{angle}$ = 11607.8 mm^2

$A_w = A_{web}+2*A_{angle}$ = 28059.4 mm^2

$d_c = h_{tot}/2$ = 919.2 mm

Reduction Factor = 0.983

RF = 0.983

d. Calculate $M_r = \phi_s S_x F_y$ (10.10.3.2) — *Cl. 10.10.3.2, p458*

$M_r = \phi_s S_x F_y$ = 6588.3 kNm — *treat as laterally supported*

e. Calculate Reduced Moment Resistance

$M_{r,\ final}$ = RF*M_r = 6476.8 kNm

Shear Resistance cl 10.10.5.1

a/h ratio

a = 971.55 mm

h = 1397.4 mm

a/h = 0.695

a/h<1 — *cl. 10.10.5.1, p459*

use kv = 4+5.34/(a/h)^2

k_v = 15.07

h/w = 110.0

$502(k_v/F_y)^{0.5}$ = 128.49

$621(k_v/F_y)^{0.5}$ = 158.95

h/w<502(kv/Fy)^0.5

$F_{cr} = 0.577F_y$ = 132.7 MPa

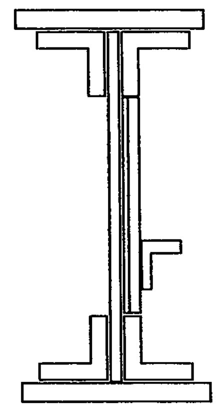
F_t = 0.00 MPa

$F_s = F_{cr} + F_t$ = 132.71 MPa

A_w = h*w = 17746.98 mm^2

$V_r = \phi_s A_w F_s$ = 2237 kN

LIFT SPAN



End Floor Beam dwg. 15/62

Built-up section

Angle	Web	Cover plate	Long. Traction Bracing	Stiffener
4	1	2		
8x8x1/2	71x5/8	20x1/2	6x4x1/2	7x4x5/8
203x203x13			152x102x13	172x102x16

Design Criteria

Total width, b =	20	in	Depth, d =	72.375	in	*(5-11 3/8+2*1/2)*
	508	mm		1838.325	mm	

Angle			Web			Cover plate		
$A =$	4990	mm^2	$w =$	5/8	in	$t =$	1/2	in
$y =$	55.5	mm		15.875	mm		12.7	mm
$x =$	55.5	mm	$h =$	71	in	$b =$	20	in
$t =$	12.7	mm		1803.4	mm		508	mm
$b =$	203	mm						
$d =$	203	mm						
$A_{one\ leg} =$	2578.1	mm^2						
$I_{angle,x} =$	20.2	$10^6\ mm^4$	$I_{web,x} =$	7759.05	$10^6\ mm^4$	$I_{cover,x} =$	0.0867	$10^6\ mm^4$
$I_{angle,y} =$	20.2	$10^6\ mm^4$	$I_{web,y} =$	0.601246	$10^6\ mm^4$	$I_{cover,y} =$	138.7438	$10^6\ mm^4$

Cqlculate Section Properties

Calculate Area

						A_{cp}=ht=	6451.6	mm^2
A_{angle}=4A=	19960	mm^2	A_{web}=hw=	28628.98	mm^2	$2A_{cp}$=	12903.2	mm^2

A_{total} =	61492	mm^2	0.0615	mm^2

Calculate Centroid taken about the bottom of the section

$y_{top\ angles} =$	1770.1	mm	$y_{web} =$	919.2	mm	$y_{top\ cover} =$	1832.0	mm
$y_{bot\ angles} =$	68.2	mm				$y_{bot\ cover} =$	6.4	mm

$y_{centroid} =$ 919 mm

Calculate Moment of Inertia

$I_{x\text{-}x} = 1/12*[bd^3-(b-w)(d-2t)^3]$ $S_{x\text{-}x} = 1/(6d)*[bd^3-(b-w)(d-2t)^3]$

$I_{y\text{-}y} = 1/12*[2tb^3-(d-2t)w^3]$ $S_{x\text{-}x} = 1/(6b)*[2tb^3-(d-2t)w^3]$

$I_{x\text{-}x} =$	18634.1	$10^6\ mm^4$	$S_{x\text{-}x} =$	20.273	$10^6\ mm^3$
$I_{y\text{-}y} =$	276.88	$10^6\ mm^4$	$S_{y\text{-}y} =$	1.090	$10^6\ mm^3$

G_{steel} = 77000 MPa ϕ = 0.95
E = 200000 MPa
F_y = 230 MPa

$I_{angles, t\text{-}x}$ =	14535	10^6 mm^4	$I_{web, t\text{-}x}$ =	7759.05	10^6 mm^4	$I_{cover, t\text{-}x}$ =	10751.46	10^6 mm^4
$I_{angles, t\text{-}y}$ =	161	10^6 mm^4	$I_{web, t\text{-}y}$ =	0.6012	10^6 mm^4	$I_{cover, t\text{-}y}$ =	277.488	10^6 mm^4

$I_{total, x}$ =	33045.1	10^6 mm^4	0.0330	m^4
$I_{total, y}$ =	439.21	10^6 mm^4	0.00044	m^4

$S_{total, x} = I_x/y$ =	35951.3	10^3 mm^3	0.000036	m^4
$S_{total, y} = I_y/y$ =	477.84	10^3 mm^3		

S_{net} = 1920 in^2 *from dwg. 13/62*
31463 10^3 mm^3

<u>*Check Class*</u>

b' = b/2 = 254 mm
$t' = t_{cp}+t_{angle}$ = 25.4 mm
$h'_{web} = h_{web}\text{-}2^*d_{angle}$ = 1397.4 mm

Flange (Top and Bottom the same)

b'/t' = 10.0
200/sqrt(F_y) = 13.2   *Tab 10.9.2.1, p451*
Flange is Class 3

Web

h'_{web}/w_{web} = 88.0
1900/sqrt(F_y) = 125.3
Web is Class 3   *Tab 10.9.2.1, p451*

<u>*Check Moment Capacity*</u>

Note: *Flange Class is 3 and web is Class 3 , Stiffened plate girder, laterally unsupported*

<u>*Check Width-to-Thickness Ratios Webs*</u>

3150/sqrt(F_y) = 207.7   *Cl. 10.10.4.2(a)*
h/w ratio is OK   *treated as longitudinal unstiffened*

<u>*Moment Required*</u>

M_f = 6500.0 kNm   *from LUSAS Model*

<u>*Moment Resistance (Cl. 10.10.4.3)*</u>

a. Treat as girder without longitudinal stiffeners
b. Check if $2d_c/w > 1900/F_y^{0.5}$ *Cl . 10.10.4.3, p 458*

$2d_c/w$ = 115.8
$1900/Fy^{0.5}$ = 125.3
2dc/w<1900/sqrt(Fy), no need to reduce the moment   *Cl . 10.10.4.3, p 458*

c. Calculate reduction factor

Reduction Factor = $1.0-1/[300+(1200A_{cf})/A_w]*[2d_c/w-1900/(M_f/\phi_s S)^{0.5}]$ — *Cl. 10.10.4.3, p 458*

$A_{cf} = A_{cp}+2*A_{angle} =$ 11607.8 mm^2

$A_w = A_{web}+2*A_{angle} =$ 33785.2 mm^2

$d_c = h_{tot}/2 =$ 919.2 mm

Reduction Factor = 1.018

RF = 1.000

d. Calculate $M_r = \phi_s S_x F_y$ (10.10.3.2) — *Cl. 10.10.3.2, p458*

$M_r = \phi_s S_x F_y =$ 6874.7 kNm — *treat as laterally supported*

e. Calculate Reduced Moment Resistance

$M_{r,\,final} = RF*M_r =$ 6874.7 kNm

Shear Resistance cl 10.10.5.1

a/h ratio

a = 971.55 mm

h = 1397.4 mm

a/h = 0.695

a/h<1 — *cl. 10.10.5.1, p459*

use kv = 4+5.34/(a/h)^2

$k_v =$ 15.07

h/w = 88.0

$502(k_v/F_y)^{0.5} =$ 128.49

$621(k_v/F_y)^{0.5} =$ 158.95

h/w<502(kv/Fy)^0.5

$F_{cr} = 0.577F_y =$ 132.7 MPa

$F_t =$ 0.00 MPa

$F_s = F_{cr} + F_t =$ 132.71 MPa

$A_w = h*w =$ 22183.73 mm^2

$V_r = \phi_s A_w F_s =$ 2797 kN

TOWER SPAN

	Imperial	Metric
Stringer	***27WF102***	***W690x152***

Design Criteria

d =	688	mm	I_x =	1510	10^6 mm^4	J =	2200	10^3 mm^4
b =	254	mm	I_y =	57.8	10^6 mm^4	C_w =	6420	10^9 mm^6
t =	21.1	mm	S_x =	4380	10^3 mm^3	G_{steel} =	77000	MPa
w =	13.1	mm	Z_x =	5000	10^3 mm^3			
F_y =	230	MPa	ϕ =	0.95		E =	200000	MPa
L =	3230	mm						

Class Check

b' = b/2 = 127 mm
h = d-2t = 645.8 mm

Flange (Top and Bottom the same)

b'/t = 6.0
145/sqrt(F_y) = 9.6 — *Tab 10.9.2.1, p451*
Flange is Class I

Web

h/w = 49.3
1100/sqrt(F_y) = 72.5
Web is Class I — *Tab 10.9.2.1, p451*

Check Moment Capacity — *Class 1, Laterally unsupported member*

Check if M_u >0.67M_p

Calculate M_p

$M_p = ZF_y$ = 1150 kNm — *Class 1 section, cl 10.10.2.2, p457*
$0.67M_p$ = 771 kNm

Calculate M_u — for doubly symmetric sections

$M_u = (\omega_2\pi/L)[E_sI_yG_sJ+(\pi E/L)^2 I_yC_w]^{0.5}$ — *cl 10.10.2.3*
ω_2 = 1.0 — *cl 10.10.2.3*
M_u = 3891 kNm

Case (a) — Mu>0.67Mp — *Case a applies*

Calculate M_r

$M_r = 1.15\phi M_p(1-0.28M_p/M_u) < \phi M_p$
M_r = 1152.4 kNm
ϕM_p = 1092.5 kNm

$M_{r, final} = min(M_r, \phi M_p)$

M_r = 1093 kNm

Shear Resistance *cl 10.10.5.1, p459*

a/h ratio unstiffened, a/h=infinity, k_v = 5.34 *cl. 10.10.5.1, p459*

k_v = 5.34 no stiffeners, a/h is infinity

h/w = 49.3

$502(k_v/F_y)^{0.5}$ = 76.49

$621(k_v/F_y)^{0.5}$ = 94.62

h/w<502(kv/Fy)^0.5

$F_{cr} = 0.577F_y$ = 132.7 MPa

F_t = 0 MPa

$F_s = F_{cr} + F_t$ = 132.71 MPa

$A_w = h^*w$ = 8459.98 mm^2

$V_r = \phi_s A_w F_s$ = 1067 kN

APPROACH SPAN

	Imperial	Metric
Stringer	***33WF130***	***W840x193***

Design Criteria

d =	840	mm	I_x =	2780	10^6 mm^4	J =	3050	10^3 mm^4
b =	292	mm	I_y =	90.3	10^6 mm^4	C_w =	15100	10^9 mm^6
t =	21.7	mm	S_x =	6630	10^3 mm^3	G_{steel} =	77000	MPa
w =	14.7	mm	Z_x =	7620	10^3 mm^3			
F_y =	230	MPa	ϕ =	0.95		E =	200000	MPa
L =	3150	mm						

Class Check

b' = b/2 = 146 mm
h = d-2t = 796.6 mm

Flange (Top and Bottom the same)

b'/t = 6.7
145/sqrt(F_y) = 9.6 — *Tab 10.9.2.1, p451*
Flange is Class I

Web

h/w = 54.2
1100/sqrt(F_y) = 72.5
Web is Class I — *Tab 10.9.2.1, p451*

Check Moment Capacity — *Class 1, Laterally unsupported member*

Check if M_u>0.67M_p

Calculate M_p

$M_p = ZF_y$ = 1753 kNm — *Class 1 section, cl 10.10.2.2, p457*
$0.67M_p$ = 1174 kNm

Calculate M_u — for doubly symmetric sections

$M_u = (\omega_2\pi/L)[E_sI_yG_sJ+(\pi E/L)^2I_yC_w]^{0.5}$ — *cl 10.10.2.3*
ω_2 = 1.0 — *cl 10.10.2.3*
M_u = 7628 kNm

Case (a) — Mu>0.67Mp — *case a applies*

Calculate M_r

$M_r = 1.15\phi M_p(1-0.28M_p/M_u) < \phi M_p$
M_r = 1791.5 kNm
ϕM_p = 1665.0 kNm

$M_{r,\,final} = min(M_r,\ \phi M_p)$

M_r = 1665 kNm

Required Moment

M_f = 1223.4 kNm — from Zuzana's notes

V_f = 113.2 kN — *"SARESS63.MCD"*

Shear Resistance *cl 10.10.5.1, p459*

a/h ratio — unstiffened, k_v = 5.34 — *cl. 10.10.5.1, p459*

k_v = 5.34 — no stiffeners, a/h is infinity

h/w = 54.2

$502(k_v/F_y)^{0.5}$ = 76.49

$621(k_v/F_y)^{0.5}$ = 94.62

h/w<502(kv/Fy)^0.5

F_{cr} =0.577F_y = 132.7 MPa

F_t = 0 MPa

$F_s = F_{cr} + F_t$ = 132.71 MPa

A_w = h*w = 12348 mm^2

$V_r = \phi_s A_w F_s$ = 1557 kN

Tower Span-Transverse Reinforcement

Bottom Reinforcement

f'_c =	20	MPa	*14.6.3.2, p651*
α_1=0.85-0.0015f'_c =	0.82		
β_1=0.97-0.0025f'_c =	0.92		
ϕ_s =	0.9		
f_y =	350	MPa	*Tab 14.6.3.3, p651*
ϕ_c =	0.75		
b =	1000	mm	
15mm Rebar Diameter =	15	mm	
$A_{s\text{-}15M}$ =	176.7	mm^2	
Slab thickness, t_{slab} =	7.5	in	
	190.5	mm	
Clear Cover =	1.5	in, bot	
	38.1	mm	
d =	144.9	mm	
Spacing =	6	in, bot	
	152.4	mm	
# of 15M bars =	6.562	per metre	
$A_{trans.}$ =	1160	mm^2	
$T_s = \phi_s A_s f_y$ =	365.3	kN	
$a = T_s/(\phi_c \alpha_1 f'_c b)$ =	29.70	mm	
Lever arm =	130	mm	
$M_r = T_s$(Lever arm) =	47.5	kNm/m	

Top Reinforcement

f'_c =	20	MPa
α_1=0.85-0.0015f'_c =	0.82	
β_1=0.97-0.0025f'_c =	0.92	
ϕ_s =	0.9	
f_y =	350	MPa
ϕ_c =	0.75	
b =	1000	mm
15mm Rebar Diameter =	15	mm
$A_{s\text{-}15M}$ =	176.7	mm^2
Depth of Cap =	7.5	in
	190.5	mm
Clear Cover =	2	in, top
	50.8	mm
d =	132.2	mm
Spacing =	6	in, top
	152.4	mm
# of 15M bars =	6.562	per metre
$A_{trans.}$ =	1160	mm^2
$T_s = \phi_s A_s f_y$ =	365.3	kN
$a = T_s/(\phi_c \alpha_1 f'_c b)$ =	29.70	mm
Lever arm =	117	mm
$M_r = T_s$(Lever arm) =	42.9	kNm

Tower Span-Longitudinal Reinforcement

Bottom Reinforcement

f'_c =	20	MPa	*14.6.3.2, p651*
α_1=0.85-0.0015f'_c =	0.82		
β_1=0.97-0.0025f'_c =	0.92		
ϕ_s =	0.9		
f_y =	275	MPa	*Tab 14.6.3.3, p651*
ϕ_c =	0.75		
b =	1000	mm	
15mm Rebar Diameter =	15	mm	
$A_{s\text{-}15M}$ =	176.7	mm^2	
Slab thickness, t_{slab} =	7.5	in	
	190.5	mm	
Clear Cover =	1.5	in, bot	
	38.1	mm	
d =	144.9	mm	
Spacing =	9	in, bot	
	228.6	mm	
# of 15M bars =	4.374	per metre	
$A_{long.}$ =	773	mm^2	
$T_s = \phi_s A_s f_y$ =	191.3	kN	

f'_c =	20	MPa
α_1=0.85-0.0015f'_c =	0.82	
β_1=0.97-0.0025f'_c =	0.92	
ϕ_s =	0.9	
f_y =	400	MPa
ϕ_c =	0.75	
b =	1000	mm
15mm Rebar Diameter =	15	mm
$A_{s\text{-}15M}$ =	176.7	mm^2
Depth of Cap =	7.5	in
	190.5	mm
Clear Cover =	2	in, top
	50.8	mm
Spacing =	9	in, top
	228.6	mm
# of 15M bars =	4.374	per metre
$T_s = \phi_s A_s f_y$ =	278.3	kN

$a = T_s/(\phi_c \alpha_1 f'_c b) =$	15.55	mm
Lever arm =	137	mm
$M_r = T_s$(Lever arm) =	26.2	kNm/m

$a = T_s/(\phi_c \alpha_1 f'_c b) =$	22.63	mm
Lever arm =	121	mm
$M_r = T_s$(Lever arm) =	33.6	kNm

Empirical Design Method - cl 8.18.4

(b) Ratio of the spacing of the supporting beams to the thickness of the slab <=18

$S_{beams} =$ 1.8795 m

$S_{beams}/t_{slab} =$ 9.9

ratio<=18, OK

(c) the spacing of the supporting beams does not exceed 4.0 m

spacing < 4.0 m, OK

Cast-in-Place Deck Slabs

(a) Calculate ρ

$\rho_{transverse} = A_{s.transverse}/A =$ 0.008

rho, trans.>0.003, OK

$\rho_{long.} = A_{s.long.}/A =$ 0.005

rho, long.>0.003, OK

(b) OK

(f) spacing < 300 mm, OK

Concrete Deck Slabs 14.13.1.2, p665

t =	7.5	in
	190.5	mm
	t>175mm	
$d_l = d_t = d =$	144.9	mm
Transverse reinforcement		
spacing =	6	in
	152.4	mm
# of bars/m =	6.56	
$A_{sl} =$	1160	mm^2
Longitudinal reinforcement		
spacing =	9	in
	228.6	mm
# of bars/m =	4.37	
$A_{sl} =$	773	mm^2
$q = 50[A_{sl}/(bd_l)+A_{st}/(bd_t)] =$	0.667	

Read from Figure14.13.1.2.2(b) non-composite slab

$R_d =$	800	kN	*deck=190 mm, span=1.9 m, Figure 14.13.1.2.2(b), p666*
$F_q =$	1.08		*approximate, using q, read from figure 14.13.1.2.2(b), p666*
$F_c =$	0.87		*f'_c = 20 MPa*
$R_n = R_d F_q F_c =$	750	kN	

ϕ_{md} = 0.5
$R_f = \phi_{md} R_n$ = 375 kN

5.7.1.7 Transverse Bending Moments in Decks

Cl. 5.7.1.7.1, 186

Treat it as simple span deck using maximum wheel load of CL-625 Truck

M = (S+0.6)P/10

S =	1.9431	m	
P =	87.5	kN	
M = (S+0.6)P/10 =	22.3	kNm	
M_f = 80%M =	17.8	kNm	
Live Load Factor =	1.7		
DLA =	1.4		
F. M_f =	42.37	kNm	
UF. $M_{f.sw}$ =	1.32	kNm	*LUSAS model-"BCLB-Wheel load transverse on deck.mdl"*
Load factor =	1.2		
F. $M_{f.sw}$ =	1.584	kNm	
UF. $M_{f.asphalt}$ =	0.704	kNm	*LUSAS model-"BCLB-Wheel load transverse on deck.mdl"*
Load factor =	1.5		
F. $M_{f.asphalt}$ =	1.056	kNm	

Total F. M_f =	45.01	kNm

Conclusioin Mr>Mf, OK

DELCAN	SUBJECT: Burlington Canal Lift Bridge Analysis of Load-Carrying Capacity			JOB NO. BT3252BTJ00
	Tower Span and Approach Span Old Slab Capacity			PAGE / of 3
	MADE BY: KL	DATE: Jun. 18, 07	CHECKED BY:	DATE:

Tower Span-Transverse Reinforcement

Bottom Reinforcement				*Top Reinforcement*		
f'_c =	20	MPa	*14.6.3.2, p651*	f'_c =	20	MPa
α_1=0.85-0.0015f'_c =	0.820			α_1=0.85-0.0015f'_c =	0.820	
β_1=0.97-0.0025f'_c =	0.920			β_1=0.97-0.0025f'_c =	0.920	
ϕ_s =	0.9			ϕ_s =	0.9	
f_y =	275	MPa	*Tab 14.6.3.3, p651*	f_y =	275	MPa
ϕ_c =	0.75			ϕ_c =	0.75	
b =	1000	mm		b =	1000	mm
#5 Rebar Diameter =	15.875	mm		#5 Rebar Diameter =	15.875	mm
$A_{s\text{-}15M}$ =	197.9	mm^2		$A_{s\text{-}15M}$ =	197.9	mm^2
Slab thickness, t_{slab} =	7.5	in		Depth of Cap =	7.5	in
	190.5	mm			190.5	mm
Clear Cover =	1.0	in, bot	*dwg. 14*	Clear Cover =	1.0	in, top
	25.4	mm			25.4	mm
d =	157.16	mm		d =	157.16	mm
Spacing =	6	in, bot		Spacing =	6	in, top
	152.4	mm			152.4	mm
# of #5 bars =	6.562	per metre		# of #5 bars =	6.562	per metre
$A_{trans.}$ =	1299	mm^2		$A_{trans.}$ =	1299	mm^2
$T_s = \phi_s A_s f_y$ =	321.4	kN		$T_s = \phi_s A_s f_y$ =	321.4	kN
$a = T_s/(\phi_c \alpha_1 f'_c b)$ =	26.13	mm		$a = T_s/(\phi_c \alpha_1 f'_c b)$ =	26.13	mm
Lever arm =	144	mm		Lever arm =	144	mm
$M_r = T_s$(Lever arm) =	46.3	kNm/m		$M_r = T_s$(Lever arm) =	46.3	kNm

Tower Span-Longitudinal Reinforcement

Bottom Reinforcement						
f'_c =	20	MPa	*14.6.3.2, p651*	f'_c =	20	MPa
α_1=0.85-0.0015f'_c =	0.82			α_1=0.85-0.0015f'_c =	0.82	
β_1=0.97-0.0025f'_c =	0.92			β_1=0.97-0.0025f'_c =	0.92	
ϕ_s =	0.9			ϕ_s =	0.9	
f_y =	275	MPa	*Tab 14.6.3.3, p651*	f_y =	400	MPa
ϕ_c =	0.75			ϕ_c =	0.75	
b =	1000	mm		b =	1000	mm
#5 Rebar Diameter =	15.875	mm		#5 Rebar Diameter =	15.875	mm
$A_{s\text{-}15M}$ =	197.9	mm^2		$A_{s\text{-}15M}$ =	200	mm^2
Slab thickness, t_{slab} =	7.5	in		Depth of Cap =	7.5	in
	190.5	mm			190.5	mm
Clear Cover =	1.0	in, bot	*dwg. 14*	Clear Cover =	1.0	in, top
	25.4	mm			25.4	mm
d =	157.1625	mm				
Spacing =	9	in, bot		Spacing =	9	in, top
	228.6	mm			228.6	mm
# of 15M bars =	4.374	per metre		# of 15M bars =	4.374	per metre
$A_{long.}$ =	866	mm^2		$A_{long.}$ =	875	mm^2
$T_s = \phi_s A_s f_y$ =	214.3	kN		$T_s = \phi_s A_s f_y$ =	315.0	kN

$a = T_s/(\phi_c \alpha_1 f'_c b) =$	17.42	mm
Lever arm =	148.5	mm
$M_r = T_s$(Lever arm) =	31.8	kNm/m

$a = T_s/(\phi_c \alpha_1 f'_c b) =$	25.61	mm
Lever arm =	144	mm
$M_r = T_s$(Lever arm) =	45.5	kNm

Empirical Design Method - cl 8.18.4

(b) Ratio of the spacing of the supporting beams to the thickness of the slab <=18

$S_{beams} =$ 1.8795 m

$S_{beams}/t_{slab} =$ 9.9

ratio<=18, OK

(c) the spacing of the supporting beams does not exceed 4.0 m

spacing < 4.0 m, OK

Cast-in-Place Deck Slabs

(a) Calculate ρ

$\rho_{transverse} = A_{s.transverse}/A =$ 0.008

rho, trans.>0.003, OK

$\rho_{long.} = A_{s.long.}/A =$ 0.006

rho, long.>0.003, OK

(b) OK

(f) spacing < 300 mm, OK

Concrete Deck Slabs 14.13.1.2, p665

t =	7.5	in
	190.5	mm
	t>175mm	
$d_l = d_t = d =$	157.1625	mm
Transverse reinforcement		
spacing =	6	in
	152.4	mm
# of bars/m =	6.56	
$A_{sl} =$	1299	mm^2
Longitudinal reinforcement		
spacing =	9	in
	228.6	mm
# of bars/m =	4.37	
$A_{sl} =$	866	mm^2
$q = 50[A_{sl}/(bd_l)+A_{st}/(bd_t)] =$	0.689	

Read from Figure14.13.1.2.2(b) non-composite slab

$R_d =$	800	kN	*deck=190 mm, span=1.9 m, Figure 14.13.1.2.2(b), p666*
$F_q =$	1.08		*approximate, using q, read from figure 14.13.1.2.2(b), p666*
$F_c =$	0.87		*f'_c = 20 MPa*
$R_n = R_d F_q F_c =$	750	kN	

ϕ_{md} =	0.5	
$R_f = \phi_{md} R_n$ =	375	kN

5.7.1.7 Transverse Bending Moments in Decks *Cl. 5.7.1.7.1, 186*

Treat it as simple span deck using maximum wheel load of CL-625 Truck

M = (S+0.6)P/10

S =	1.9431	m	
P =	87.5	kN	
M = (S+0.6)P/10 =	22.3	kNm	
M_f = 80%M =	17.8	kNm	
Live Load Factor =	1.7		
DLA =	1.4		
F. M_f =	42.37	kNm	
UF. $M_{f.sw}$ =	1.32	kNm	*LUSAS model-"BCLB-Wheel load transverse on deck.mdl"*
Load factor =	1.2		
F. $M_{f.sw}$ =	1.584	kNm	
UF. $M_{f.asphalt}$ =	0.704	kNm	*LUSAS model-"BCLB-Wheel load transverse on deck.mdl"*
Load factor =	1.5		
F. $M_{f.asphalt}$ =	1.056	kNm	
Total F. M_f =	45.01	kNm	

Conclusioin Mr>Mf, OK