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Ontario Region

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BURLINGTON LIFT BRIDGE, TOWERS, AND PIERS - STRUCTURAL MODELLING, ANALYSIS and SURVEYS

PHASE II
RS8: FATIGUE REVIEW REPORT

MMM GROUP

MMM Group Limited
May 2014

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APPENDICES

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1. INTRODUCTION

MMM Group Ltd. (MMM) was retained by Public Works and Government Services Canada (PWGSC) to undertake a structural analysis including 3D model, evaluation of member capacities, and a fatigue review for the Burlington Lift Bridge.

This Report presents the results of the Fatigue Review of the bridge as outlined in RS8 in the Terms of Reference.

The Fatigue Review was restricted to main structural components and did not consider the open steel grating, lift cables, or any of the mechanical equipment (including the shafts of the sheaves).

A Key Plan showing the location of the structure and a General Arrangement Drawing have been provided in Appendix A.

2. EXISTING STRUCTURE

2.1 Structure Description

The Burlington Lift Bridge is a tower-drive steel truss vertical lift bridge designed in 1958 by C.C. Parker and Associates of Hamilton, Ontario and constructed between 1959 and 1960 by the Hamilton Bridge Division of the Bridge and Tank Company of Canada Limited. The bridge originally served both rail and highway traffic in a side-by-side configuration. A single rail track ran along the eastern half of the structure while two (2) traffic lanes were provided on the western half of the structure. In 1982 the bridge underwent a major rehabilitation to convert it to highway traffic only through the complete removal of the railway track and the addition of two new lanes of traffic.

The bridge is comprised of two 12.60m (41'- 4") approach spans, two 9.75m (32'- 0") tower spans, and one 112.78m (370'- 0") lift span. There is a 2.07m (6'- 9.5") wide sidewalk with an aluminum pedestrian hand railing cantilevered from the outside of the west truss. Two 3.375m wide northbound lanes and two 3.375m wide southbound lanes are provided on the bridge.

The substructure is comprised of two voided concrete substructures supporting the towers, and two concrete conventional closed abutments at each end of the approach spans.

2.1.1 Lift Span

The lift span is a steel through truss structure that is 15.54m (51'- 0") wide from centreline to centreline of the two trusses. Each truss is comprised of twelve 9.40m (30'- 10") panels which vary in depth from 13.87m (45'- 6") at the ends to 16.76m (55'- 0") at the midspan.

Truss members (i.e. verticals, diagonals, and top and bottom chords) are comprised of built-up steel sections. The built up members are typically comprised of rolled and plate components connected by rivets. Transverse floor beams and longitudinal stringers support an open steel grating deck. The sidewalk deck consists of a thin (50mm) concrete half-filled steel grating.

Portal and sway bracings are provided overhead at panel points.

2.1.2 Towers

There are two steel braced towers at either end of the lift span. Each tower is 15.90m (52'- 2") wide from centreline to centreline of the columns, and 9.75m (32'- 0") long from centreline to centreline of the columns. The towers are approximately 65m (213') high.

Tower members (i.e. columns, diagonal bracings, and horizontals) are comprised of built-up steel sections similar to the riveted sections of the lift span. The roadway passing through the towers is referred to as the "tower span" and is comprised of transverse floor beams and longitudinal stringers supporting a 190mm thick (7.5") concrete deck with a 65mm (2.5") asphalt wearing surface.

There is a 2.47m (8'- 1") wide sidewalk with an aluminum pedestrian barrier cantilevered from the west side of the tower.

Each tower is supported on a hollow concrete pier substructure resting on a pile foundation.

At the top of each tower is a machine room which houses the mechanical and electrical equipment including the sheaves and sheave girders.

2.1.3 Approach Spans

There are two approach spans at either end of the bridge. Each is 15.90m (52'- 2") wide and 12.60m (41'- 4") in length.

Transverse floor beams and longitudinal stringers support a 190mm thick (7.5") concrete deck with a 65mm (2.5") asphalt wearing surface. There is a 2.47m (8'- 1") wide sidewalk with an aluminum pedestrian barrier cantilevered from the west side of the approach spans.

Each approach span is simply supported by the tower piers at one end and a conventional closed reinforced concrete abutment at the other end of the span. Articulation is provided by fixed bearings at the concrete abutments, and expansion bearings at the tower piers.

3. FATIGUE REVIEW

3.1 Background

The Burlington Lift Bridge was designed circa 1958 using the American Railway Engineering Association (AREA) "Part 2" 1956 (for all movable components and structural components which support movable components), Canadian Standards Association (CSA) S1-1950 "Specifications for Steel Railway Bridges", and CSA S6-1952 "Specifications for Steel Highway Bridges". Cooper E-60 and H20-S16 design live loads were used for railway and highway components respectively. It is not clear how the live railway and highway loads were combined nor which code governed the design of the "mixed" use lift span.

It is interesting to note that the Cooper E60 design load consists of a series of axle loads approximately twice the length of the current Canadian Highway Bridge Design Code (CHBDC) S6-625-ONT Truck and with a total weight of all E60 axles of 3790 kN. This is six times the combined weight of the CL-625-ONT Truck. In addition, the Cooper E60 has a uniformly distributed load (UDL) of approximately 90 kN/m of track which is ten (10) times greater than the CHBDC lane load UDL.

Many of the truss and tower members are built up members. These members consist of a series of plate and rolled components connected with rivets. The rivets are predominantly 22 mm diameter (7/8 inch diameter) however 25 mm diameter (1 inch diameter) rivets were used in the east truss which was immediately adjacent to the railway track.

The steel in the structure consists of two (2) steel grades. One is a "carbon steel" conforming to CSA G40.4 (or ASTM A7) while the second steel type is an "alloy steel" conforming to ASTM 242-55. The impact or notch toughness properties of the steel are not known.

The Fatigue Review only examined in detail the stress ranges experienced in the lift truss under live loading in accordance with the requirements of the Canadian Highway Bridge Design Code (CHBDC). The tower members were examined for stress ranges due to the raising of the lift span. Ignoring the effects of wind loading on the raised span, no variation was found in the tower member during the lifting operation. Even if small variations in stress did occur during openings, the total number of opening/closing operations over the life of the bridge is less than 400,000 cycles which is below the fatigue endurance limit for even the "worst" Fatigue Category.

Wind loads do contribute to cyclic stresses in both the lift span and towers. However the CHBDC and other structural design codes do not address such cyclic loading when considering fatigue life. This is due to the unpredictability of the wind loading and the inability to assess the total number of anticipated cycles of such loading. It is generally believed that, for typical truss and girder bridges, the estimation of the wind load force applied during the design are conservative enough to address any fatigue effects due to the wind load which may occur over the life of the structure.

The Fatigue Review in this assignment was undertaken to the requirements of Fatigue Limit States Combination of Section 3 "Loads" from the CSA CHBDC S6-06 Supplement No. 3 (March 2013). For Fatigue Limit States, only one truck load was placed on the truss span (increased by the dynamic load allowance and placed in the centre of one travelled lane). This is consistent with Clause 3.8.4.1 c) of the CHBDC.

It should also be noted that the portion of the fatigue life "used" during the period 1960 to 1982, while the bridge functioned to carry railway traffic, has not been considered in this Fatigue Review. There is no known information on the type and frequency of rail traffic that crossed the bridge during that twenty-two (22) year period. This Fatigue Review has assumed the CHBDC S6-625-ONT Truck load was the fatigue design load since the time of construction.

Section properties used in the Fatigue Review were obtained from the 1959/1960 fabrication/erection drawings or the original design drawings where fabrication/erection drawings did not exist. The truss span has few available shop drawings (unlike the towers which have complete drawings set). As such, some approximations were required to calculate member net areas. These assumptions should be confirmed when additional detailed information becomes available.

4. FATIGUE OF RIVETED BRIDGES

4.1 Background

Much of the research and evaluation considerations for the fatigue of riveted bridges is based on a December 1987 report by Fisher, Yen and Wang entitled **"Fatigue and Fracture Evaluation for Rating Riveted Bridges"** published by the National Cooperative Highway Research Program (NCHRP) as Report 302(1). This Report (1) presents much of the background on the anticipated fatigue performance of riveted members currently used in the evaluation of riveted bridges.

Distortional and restraint cracking in rolled sections at connections is addressed in a number of reports and publications such as works by Fisher (2), Kulak, Fisher and Struik (3) and Fisher (4). In addition, fatigue evaluation is addressed by numerous Codes relating to the design and evaluation of bridges (5, 6, 7).

For riveted members, each of the American Railway Engineering and Maintenance of Way Association (AREMA), American Association for State Highway and Transportation Officials (AASHTO) and Canadian Standards Association (CSA) Codes use lower bounds from available test data on riveted sections to define the fatigue class and stress range in the detail classification system. The 1987 publication by Fisher (1) established a lower bound to Category D in the detail classification system. This is the same Category still used by most current Codes (5, 6, 7) for built-up riveted components.

Tests on riveted and built-up sections (1) demonstrated that fracture of a component of a built-up section did not immediately impair the load-carrying capacity of the member. At stress ranges less than 62 MPa, the riveted/built-up section experienced an "endurance limit" of a Category 'C' Detail. Following severing of a component in a built-up section, between 200,000 and 1,000,000 additional cycles of stress range were required before the load carrying capacity of the detail was completely destroyed. The fatigue strength used for riveted built up members is conservatively taken as Category D (48 MPa).

Fatigue resistance/Category can be reduced to Category E (31 MP) as a result of "corrosion notched" sections. As the corrosion state of specific members is not currently known for the Burlington Lift Bridge, this situation was not considered in the Fatigue Review. (However, all of the stress ranges in the truss members examined in this Fatigue Review were less than even 31MPa by a significant amount).

5. FATIGUE REVIEW

5.1 Methodology

In this Fatigue Review the following methodology was followed:

1) The Bridge was evaluated to Fatigue Limit States as outlined in the CHBDC to establish the maximum range of forces in the members;

2) a) In axially loaded members (tension/compression) the range of stress was calculated based on the estimated net area (A_N);

   b) If the stress range (S_r) was less than 48 MPa (Category D) based on the net area, then fatigue is not considered to be consideration with that tension/compression member;

3) a) In a flexural member or combined flexure and tension/compression member, the range of stress will be calculated based on the net estimated section (A_N) at the point of maximum stress range;

   b) If the stress range (S_r) was less than the fatigue Category Threshold Stress Range for the detail of that component, then the flexural member will not be considered to be a limiting design constraint;

4) Any members from 2) or 3) where the stress range fails to meet that of Category D (riveted) or the appropriate Category for the details in flexural members, will be assessed for the remaining safe service life in accordance with NCHRP Report 299 (8); and

5) An overview of the existing structure drawings was made to assess any distortional stress or out-of-plane displacements that might contribute to cracks in the steel components.

5.2 Results of Stress Evaluation

The results of the above noted methodology are noted in Table 1 and Table 2 for a total of 13 members which were reviewed as part of this Fatigue Review. It should also be noted that the results are provided for the Table 1 and 2 for the lighter Highway truss; the stress ranges in the corresponding members of the Railway Truss are significantly lower than that shown in Table 1.

Truss Lift Span
Stress Range for Riveted Built-up Member
Table 1

No.	Member	Max. Force (KN)	Min. Force (KN)	Range (KN)	A_N (mm^2)	S_R (MPa)	CAT. D (MPa)	Acceptable	SFrame Member Number	Member Description
1	Lift-HwyT-U1L2	3149	2804	345	55781	6	48	Yes	22302	Diagonal
2	Lift-HwyT-L2U2	-13	-151	78	41025	2	48	Yes	22403	Vertical

3	Lift-HwyT-L6L5	6198	5796	402	71576	6	48	Yes	22204*	Bottom Chord
4	Lift-HwyT-L4U3	1773	1453	320	30309	11	48	Yes	22309	Diagonal
5	Lift-HwyT-U5L6	520	195	325	29107	11	48	Yes	22306	Diagonal
6	Lift-HwyT-L2L3	4972	4638	334	72051	5	48	Yes	22205*	Bottom Chord
7	Lift-Port-SWBC	10	-3	-13	8507	2	48	Yes	29305	Portal Sway Brace
8	Lift-Sway-SWBC	12	9	3	8469	1	48	Yes	29485	Int. Sway Brace
9	Lift-BLAT	67	37	30	10808	3	48	Yes	29213	Bottom Wind Brace
10	Lift-HWY 5-U3L3	613	364	249	47570	6	48	Yes	22404	Vertical

Truss Lift Span
Stress Range for Floorbeam/Stringer
Table 2

No.	Member/Description	Range Moment FLS (kN_m)	Section Property (mm^3)	Stress Range (MPa)	Category	Allowable S_R (MPa)	Acceptable
11	End Floor Beam	764	$46.9 x 10^6$	16	D	48	Yes
12	Interior Floor Beam	1529	$58.1 x 10^6$	26	D	48	Yes
13	Stringer	391	3.211×10^6	122	A	165	Yes

6. CONCLUSIONS

Based on this Fatigue Review all of the members are found to meet the applicable stress for Category D or A (as appropriate). None of the members investigated are anticipated to be critical for fatigue. We anticipate several factors contributed to this outcome.

We believe that the original design of the lift span for rail loading provided for a more severe cyclic live load than that currently being imposed on the lift span (or that used for the fatigue evaluation).

In addition, studies on riveted highway bridges designed and constructed in the period before 1970 have demonstrated that the actual maximum stress range will seldom ever exceed the fatigue limit applicable to riveted members (48 MPa). This is in part, related to the allowable stress design limits imposed on designs of that era.

It should also be noted that the above Review was based on approximations of the net area of the members reviewed. Given the stress range in members was found to be less than 16MPa, it is anticipated that should the net area be incorrect by 20% or more, none of the members reviewed would be unacceptable for fatigue. Category D.

It is also worth noting that the observed stress range in the built-up tension/compression members examined as part of this Fatigue Review was far less than the fatigue endurance limit of even Category E (31MPa). This is used as a lower bound estimate on the fatigue life of members which may contain "notches" due to corrosion.

As mentioned in Section 3.1, the first twenty-two (22) years of railway loading have not been considered in this evaluation. It is MMM Groups opinion that given the general long (greater than 80-100 years) life of railway bridges and few fatigue problems, we believe little of the fatigue endurance limit was utilized during that original period as a rail carrying structure. This, combined with the calculated low fatigue stress range in the members due to current CHBDC loading, would lead MMM to conclude that there is a very low probability of a fatigue issue developing in the lift span members over the expected life of the bridge.

MMM also completed a review of the available drawings of the lift span as well as reviewed the available photographs from previous inspections. This review focused primarily on the details of the floor beam to truss connections for the possibility of distortional or out-of-plane displacements leading to stress concentrations and cracks in the connection material.

Based on the available information, MMM has concluded that these connections are well "detailed" such that cracking of the connection material is not anticipated. This is supported by an absence of any such cracks observed during the recent cleaning and coating of the structural steel.

Overall, MMM group would conclude from this Fatigue Review, that failure of the built-up truss members, floor beam and stringers as a result of fatigue is not anticipated over the expected life span of the Burlington Lift Bridge.

7. REFERENCES

1. Fisher, J.W., Yen, B.T. Wang, D., "Fatigue and Fracture Evaluation For Rating Riveted Bridges", National Cooperative Highway Research Program Report 302, December 1987.
2. Fisher, J.W., "Fatigue and Fracture in Steel Bridges", John Wiley, 1984.Kulak, G.L., Fisher, J.W., Struik, J.H., "Guide to Design Criteria for Bolted and Riveted Joints", John Wiley, 1987.
3. Fisher, J.W., "Bridge Fatigue Guide - Design and Details", American Institute of Steel Construction, New York, 1977.
4. American Association of State Highway and Transportation Officials, "AASHTO LRFD Bridge Design Specifications", 6th Edition with 2013 Interim Revisions, 2013.
5. American Railroad Engineering and Maintenance-of-Way Association (AREMA), "Manual for Railroad Engineering", Chapter 15, 2013.
6. "Canadian Highway Bridge Design Code S6-06, Supplement No. 3, Canadian Standards Association, March 2013.
7. Moses, F., Schilling, C.G., and Raju, K.S., "Fatigue Evaluation Procedures for Steel Bridges", National Cooperative Highway Research Program Report 299, November 1987.

Report Prepared

By

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Senior Bridge Engineer
Partner

APPENDIX A
KEY PLAN AND GENERAL ARRANGEMENT DRAWINGS

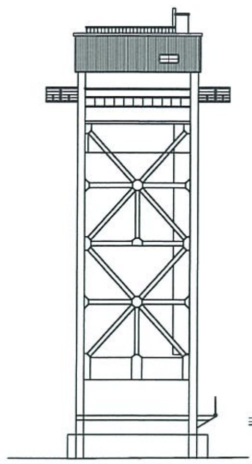
KEY PLAN

Burlington Lift Bridge, Burlington Ontario

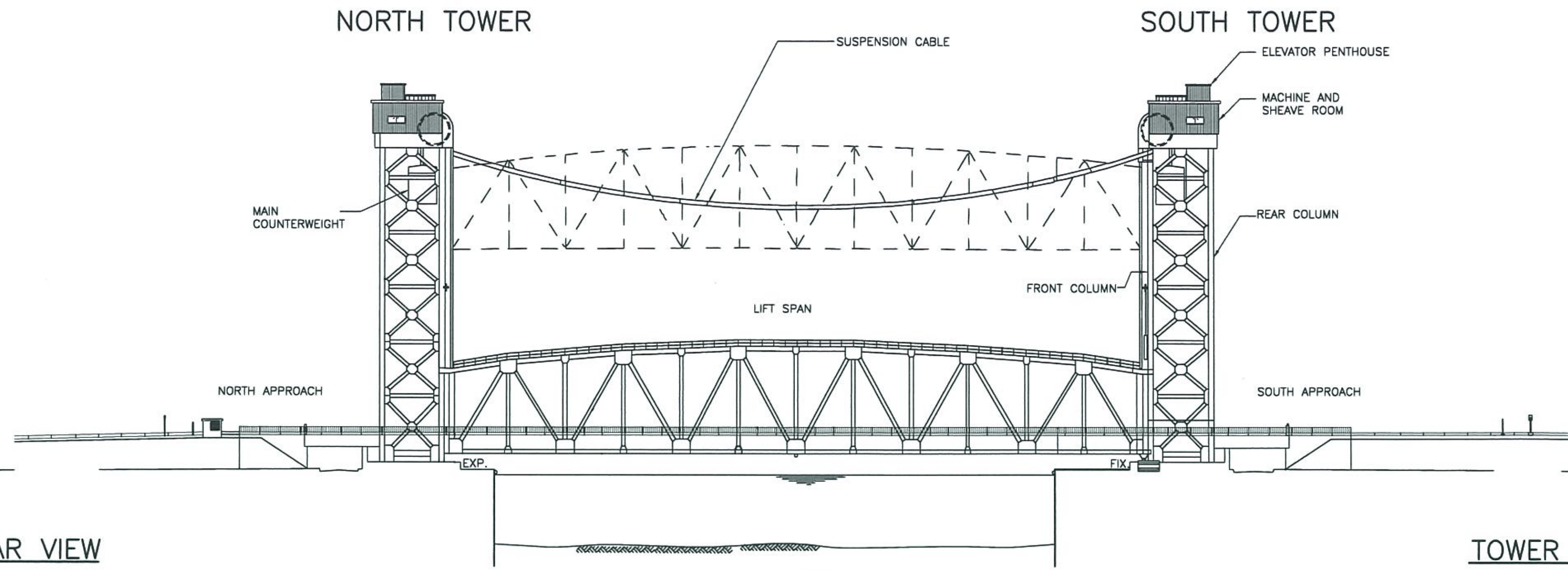
Scale: N.T.S

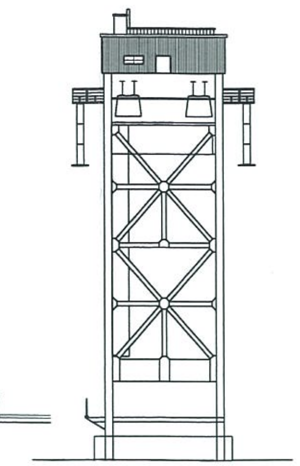
04 11

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres



TOWER – REAR VIEW

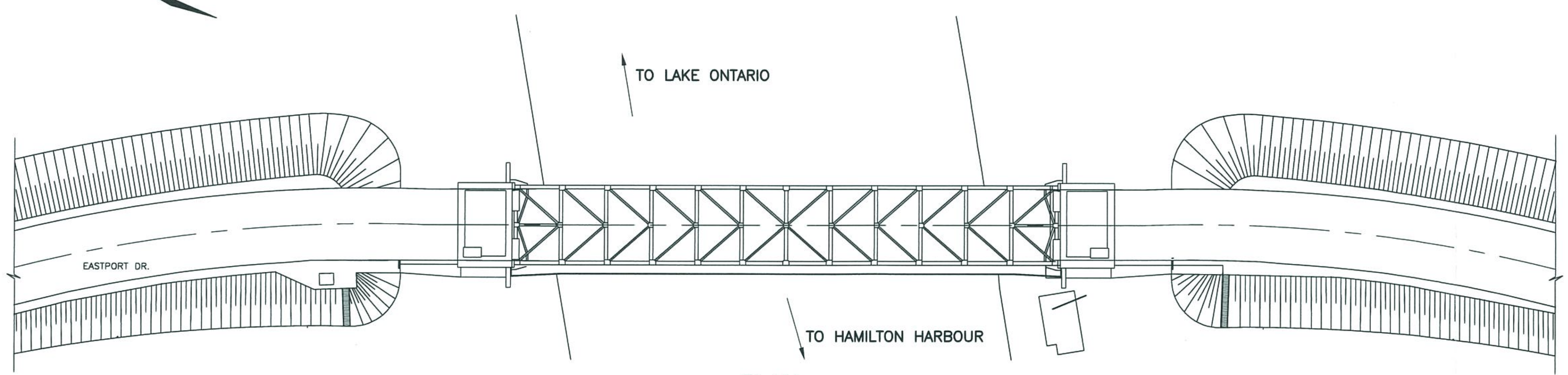




TOWER – FRONT VIEW

ELEVATION

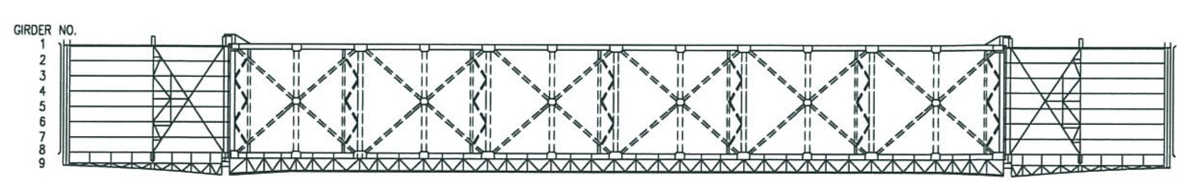
(WEST ELEVATION SHOWN, EAST ELEVATION SIMILAR)



PLAN

CITY OF BURLINGTON

CITY OF HAMILTON



PLAN VIEW – LIFT SPAN BOTTOM CHORD

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revision		date
04		
03		
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01		

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail No. / No. du détail
B drawing no. – where detail required / dessin no. – où détail exigé
C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet
Ontario

drawing title / titre du dessin
BURLINGTON CANAL
LIFT BRIDGE

GENERAL ARRANGEMENT

drawn by / dessine par

designed by / conc par

approved by / approuve par

bid / offre

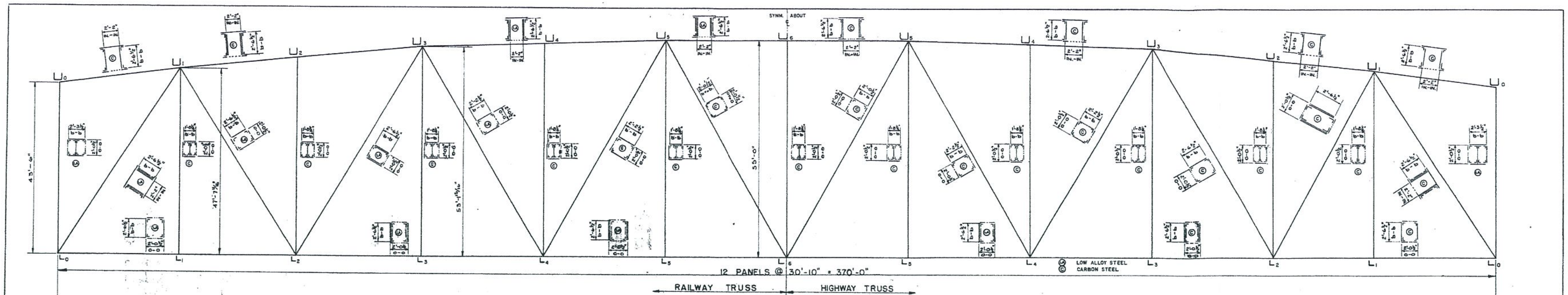
project manager / administrateur de projets

project date / date du projet: 2014-04-09

project no. / no. du projet: S3213009

drawing no. / dessine no.: S-1

GSC-A1 DATE PLOTTED: YYYY-MM-DD PLOT SCALE: 1:? CAD FILE:



RAILWAY TRUSS

	BOTTOM CHORDS L_0L_2	L_2L_4	L_4L_6	TOP CHORDS U_0U_1	U_1U_3	U_3U_5	U_5U_6
DEAD LOAD	-538	-1182	-1482		+925	+1380	+1825
LIVE LOAD RAILWAY	-596	-1275	-1535		+1004	+1468	+1572
HIGHWAY	-57	-124	-156		+97	+146	+160
IMPACT RAILWAY (D)	-150	-320	-386		+252	+366	+395
(R)	-7	-16	-19		+12	+18	+19
HIGHWAY	-6	-13	-16		+10	+15	+16
	-1354 @ NUS	-2930 @ NUS	-3594 @ NUS		+2300	+3395	+3687
V2 MINOR STRESS							
TOTAL							
BRAKING	-243	-208	-151				
30#/□' TRANS. WIND (DIRECT)	-96	-238	-306				
(OVERTURNING)	-64	-140	-175				
	-1757 @ 1.25 NUS = 1405 NUS	-3516 @ 1.25 NUS = 2810	-4228 @ 1.25 NUS = -3380				
DESIGN STRESS (KIPS)	-1406	-2930	-3594		+2300	+3395	+3687
BENDING MOM. (FT. KIPS)	24	48	56		54	63	70
L/R					29.0	30.3	31.6
L/B					9.8	9.8	9.8
ALLOWABLE UNIT STRESS (DIRECT)	27	27	27		16.26	15.58	15.54
KIPS/SQ. IN. (BENDING)	27	27	27		15.00	23.37	23.37
SECTION AREA REQ'D (GROSS)					145.0	177.2	194.6
SQ. IN. (NET)	53.5	112.0	137.3				
SECTION USED	2 WEB Rs. 30×7/16 (-8) 4 Ls. 8×6×1/2 (-8) 2 PERF. COV. Rs. (23-8)×3/8 (-4)	4 WEB Rs. 30×11/16 (-8) 2 DO. 14×1/2 (-4) 4 Ls. 8×6×1/2 (-8) 2 PERF. COV. Rs. (21-8)×3/8 (-4)	4 WEB Rs. 30×11/16 (-8) 2 DO. 30×1/2 (-8) 2 DO. 14×1/2 (-4) 4 Ls. 8×6×1/2 (-8) 2 PERF. COV. Rs. (20-8)×3/8 (-4)	2 WEB Rs. 30×1/2 4 Ls. 4×4×7/16 1 TOP COV. R. 38×1/2 1 PERF. R. (38×14)×1/2	4 WEB Rs. 30×1/2 2 TOP Ls. 4×4×7/16 2 BOTTOM Ls. 8×6×3/4 1 TOP COV. R. 38×1 1 PERF. BOTT. COV. R. (42-14)×7/8	4 WEB Rs. 30×3/4 2 TOP Ls. 4×4×7/16 2 BOTTOM Ls. 8×6×3/4 1 TOP COV. R. 38×1 1 PERF. BOTTOM COV. R. (42-14)×7/8	4 WEB Rs. 30×3/4 2 DO. 18×3/4 2 TOP Ls. 4×4×7/16 2 BOTTOM Ls. 8×6×3/4 1 TOP COV. R. 38×1 1 PERF. BOTT. COV. R. (42-14)×7/8
AREA OF SECTION (GROSS)					149.0	179.0	194.8
SQ. IN. (NET)	54.4	112.4	137.2				
SECTION MODULUS (CU. IN.)	491.0	706.0	815.0		1550	1680	1726
ACTUAL UNIT STRESS (DIRECT)	25.9	26.0	26.2		15.45	18.95	18.95
KIPS/SQ. IN. (BENDING)	0.58	0.8	0.83		0.419	0.46	0.49
$f_a/F_a + f_b/F_b$	0.982	.995	1.001		0.972	0.990	0.998
MATERIAL	LOW ALLOY	LOW ALLOY	LOW ALLOY	CARBON	CARBON	LOW ALLOY	LOW ALLOY

	DIAGONALS L_0U_1	U_1L_2	L_2U_3	U_3L_4	L_4U_5	U_5L_6	VERTICALS U_0L_0	U_1L_1	U_2L_2	U_3L_3	U_4L_4	U_5L_5	U_6L_6
DEAD LOAD	+985	-707	+520	-392	+212 +212	-86 -57	-991	-80	+35	-88	+40	-91	+42
LIVE LOAD RAILWAY	+1090	-810	+650	-553	+428 -188	-346 +248		-211		-211		-211	
HIGHWAY	+110	-83	+68	-58	+46 -20	-36 +28		-41		-41		-41	
IMPACT RAILWAY (D)	+274	-203	+163	-139	+108 -47	-87 +62	-195 (D.L.)	-123		-123		-123	
(R)	+14	-10	+8	-7	+5 -2	-4 +3		-3		-3		-3	
HIGHWAY	+11	-8	+7	-6	+5 -6	-4 +3		-12		-12		-12	
	+2484	-1821	+1416	-1155	+804 -51	-565 +287	-1186	-470		-478		-481	
V2 MINOR STRESS	(+56) (2½ ν.)				+26 -26	-143 +143							
TOTAL					+830 -77	-708 +430							
DESIGN STRESS (KIPS)	+2540	-1821	+1416	-1155	+830 -77	-708 +430	-1186	-470	+35	-478	+40	-481	+42
BENDING MOM. (FT. KIPS)	(1120) Y-Y (69) X-X	78	78	60	67 67	51 51	755	478	478	478	478	478	478
L/R	59.0		69.2		79.3	80.2			56.1				
L/B	(12) Y-Y (18) X-X		30.1		30.8	30.9			27.2				
ALLOWABLE UNIT STRESS (DIRECT)	16.41	27	19.32	27	14.74 20	20 14.69	24	20	13.32	20	13.32	20	13.32
KIPS/SQ. IN. (BENDING)	(22.49) Y-Y (21.86) X-X	27	20.21	27	14.79 20	20 14.78	24	20	18.45	20	18.45	20	18.46
SECTION AREA REQ'D (GROSS)	207.0		79.9		64.5	35.6			39.9		40.3		40.4
SQ. IN. (NET)		72.7		46.9	10.1	40	101.6	57.4		57.9		58.0	
SECTION USED	4 WEB Rs. 30×3/4 2 DO. 18×3/4 2 TOP Ls. 4×4×7/16 2 BOTTOM Ls. 8×6×3/4 1 TOP COV. R. 38×1 1 PERF. BOTT. COV. R. (42-14)×7/8	2 WEB Rs. 30×5/8 (-8) 2 WEB Rs. 30×1/2 (-8) 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (22-8)×3/8 (-4)	4 WEB Rs. 30×1/2 4 Ls. 4×4×3/8 2 PERF. COV. R. (22-8)×3/8	2 WEB Rs. 26×3/4 (-8) 4 Ls. 4×4×3/8 (-8) 2 PERF. COV. Rs. (22-8)×3/8 (-4)	2 WEB Rs. 26×7/8 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (22-8)×3/8	2 WEB Rs. 24×9/16 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (23-8)×3/8	2 Rs. OUTSIDE WEBS 23×3/4 1 CENTRE WEB 23×7/8 4 Ls. 5×5×3/4 4 Ls. 4×4×3/4 2 COVER Rs. 27×1/2 2 CENTRE WEB 13×3/4	1 CENTRE WEB 23×1/2 2 SIDE PERF. Rs. (23-10)×3/8 4 Ls. 5×5×3/8 4 Ls. 4×4×1/2 2 FLG. Rs. 19½×1/2	1 CENTRE WEB 23×1/2 2 SIDE PERF. Rs. (23-10)×3/8 4 Ls. 6×5×1/2 4 Ls. 4×4×3/8 2 FLG. Rs. 19½×3/8	AS U_1L_1	AS U_2L_2	AS U_1L_1	AS U_2L_2
AREA OF SECTION (GROSS)	206.00		81.5		66.9	48.7			66.3		66.3		66.3
SQ. IN. (NET)		74.1		48.8	58.3	39.8	104.24	60.6		60.6		60.6	
SECTION MODULUS (CU. IN.)	(1761) X-X (1886) Y-Y	502	589	329	445 GROSS 348 NET	269 NET 341 GROSS	744 NET	512 NET	555 GROSS	512 NET	555 GROSS	512 NET	555 GROSS
ACTUAL UNIT STRESS (DIRECT)	12.32	-24.6	+17.40	-23.70	+12.40 -1.48	-17.80 +8.92	11.21	7.76	0.53	7.90	0.61	7.91	0.64
KIPS/SQ. IN. (BENDING)	(7.15) Y-Y (0.47) X-X	-1.9	+1.56	-2.17	+1.81 -2.35	-2.26 +1.79	12.18	11.20	10.36	11.20	10.36	11.20	10.36
$f_a/F_a + f_b/F_b$	1.005	0.982	0.979	0.960	0.863 0.192	1.003 0.730	0.975	0.968	0.602	0.955	0.608	0.956	0.610
MATERIAL	LOW ALLOY	LOW ALLOY	LOW ALLOY	CARBON	CARBON	CARBON	LOW ALLOY	CARBON	CARBON	CARBON	CARBON	CARBON	CARBON

HIGHWAY TRUSS

	BOTTOM CHORDS L_0L_2	L_2L_4	L_4L_6	TOP CHORDS U_0U_1	U_1U_3	U_3U_5	U_5U_6
DEAD LOAD	-538	-1182	-1482		+925	+1380	+1825
LIVE LOAD RAILWAY	-154	-331	-400		+261	+381	+408
HIGHWAY	-115	-254	-318		+188	+287	+527
SIDEWALK	-27	-60	-75		+47	+70	+77
IMPACT RAILWAY (D)	-39	-83	-100		+66	+96	+102
(R)	-7	-16	-19		+12	+18	+19
HIGHWAY	-12	-26	-32		+20	+50	+53
	-892	-1952	-2426		+1529	+2272	+2492
V2 MINOR STRESS					-	-	-
TOTAL							
BRAKING	±77	±66	±49				
30#/□' TRANS. WIND (DIRECT)	±96	±238	±308				
(OVERTURNING)	±64	±140	±175				
	-1129 @ 1.25 NUS = 904	-2396 @ 1.25 NUS = 1917	-2958 @ 1.25 NUS = 2366				
DESIGN STRESS (KIPS)	-904	-1952	-2426		+1529	+2272	+2492
BENDING MOM. (FT. KIPS)	-24	-42	-42		58	58	57
L/R					29.4	28.9	29.2
L/B					9.8	9.8	9.7
ALLOWABLE UNIT STRESS (DIRECT)	20	20	27		16.26	16.27	16.26
KIPS/SQ. IN. (BENDING)	20	20	27		19.00	19.00	19.00
SECTION AREA REQ'D (GROSS)					96.5	148.0	156.8
SQ. IN. (NET)	46.8	102.6	92.6				
SECTION USED	2 WEB Rs. 30×7/16 4 Ls. 8×6×1/2 2 PERF. COV. Rs. (23-8)×3/8	2 WEB Rs. 30×11/16 2 WEB Rs. 30×7/16 2 WEB Rs. 14×1/2 4 Ls. 8×6×1/2 2 PERF. COV. Rs. (21-8)×3/8	2 WEB Rs. 30×7/16 2 WEB Rs. 30×9/16 2 WEB Rs. 14×1/2 4 Ls. 8×6×1/2 2 PERF. COV. Rs. (20-8)×3/8	2 WEB Rs. 30×1/2 4 Ls. 4×4×7/16 1 TOP COV. R. 38×1/2 1 PERF. R. (38×14)×1/2	2 WEB Rs. 30×11/16 2 Ls. 4×4×7/16 TOP 2 Ls. 8×6×9/16 BOTT. 1 TOP COV. R. 38×11/16 1 PERF. BOT. COV. R. (42-14)×3/4	2 WEB Rs. 30×1 2 Ls. 4×4×7/16 TOP 2 Ls. 8×6×3/4 BOTT. 1 TOP COV. R. 38×1 1 BOTT. PERF. R. (42-14)×7/8	2 Ls. 4×4×7/16 2 WEB Rs. 30×1/2 2 WEB Rs. 30×5/8 2 Ls. 8×6×3/4 1 TOP COV. R. 38×1 1 PERF. B. COV. R. (42-14)×7/8
AREA OF SECTION (GROSS)					100.5	149.0	156.5
SQ. IN. (NET)	55.5	101.8	101.00				
SECTION MODULUS (CU. IN.)	501 NET	585 NET	649 NET		1025 GROSS	1547 GROSS	1581 GROSS
ACTUAL UNIT STRESS (DIRECT)	16.30	19.20	24.00		15.20	15.25	15.90
KIPS/SQ. IN. (BENDING)	0.56	0.93	0.76		0.46	0.42	0.43
$f_a/F_a + f_b/F_b$	0.843	1.007	0.917		0.959	0.959	1.00
MATERIAL	CARBON	CARBON	LOW ALLOY	CARBON	CARBON	CARBON	CARBON

	DIAGONALS L_0U_1	U_1L_2	L_2U_3	U_3L_4	L_4U_5	U_5L_6	VERTICALS U_0L_0	U_1L_1	U_2L_2	U_3L_3	U_4L_4	U_5L_5	U_6L_6
DEAD LOAD	+988	-707	+520	-392	+212 +212	-86 -57	-991	-80	+35	-88	+40	-91	+42
LIVE LOAD RAILWAY	+283	-211	+166	-142	+110 -49	-90 +64		-55		-55		-55	
HIGHWAY	+224	-170	+137	-118	+94 -41	-78 +56		-80		-80		-80	
SIDEWALK	+50	-37	+30	-25	+19 -7	-15 +11	-195 (D.L.)	-27		-27		-27	
IMPACT RAILWAY (D)	+71	-53	+42	-36	+28 -12	-23 +16		-32		-32		-32	
(R)	+14	-10	+8	-7	+5 -2	-4 +3		-3		-3		-3	
HIGHWAY	+23	-17	+14	-12	+10 -12	-6 +6		-24		-24		-24	
	+1650	-1205	+919	-732	+478 +89	-504 +101	-1186	-301		-309		-312	
V2 MINOR STRESS	(+94) (W+2½ν)					-51 +51							
TOTAL	+1744 @ 1.25 NUS = 1395					-355 +152							
DESIGN STRESS (KIPS)	+1395	-1205	+919	-732	+478	-355 +152	-1186	-301	+35	-309	+40	-312	+42
BENDING MOM. (FT. KIPS)	1590	72	65	51	68	51 51	755	392	392	392	392	392	392
L/R	59.0		67.0		76.8	80.2							
L/B	14.0		30.1		30.8	30.9							
ALLOWABLE UNIT STRESS (DIRECT)	15.63	20	15.26	20	14.85	20 14.70	24	20	13.32	20	13.32	20	13.32
KIPS/SQ. IN. (BENDING)	(18.92) Y-Y (18.28) X-X	20	15.00	20	14.79	20 14.78	24	20	18.45	20	18.45	20	18.45
SECTION AREA REQ'D (GROSS)	199.0		66.9		32.9	16.3			35.1		35.5		33.6
SQ. IN. (NET)		66.5		40.8		22.3	101.6	43.0		43.5		43.5	
SECTION USED	2 Ls. 4×4×7/16 4 WEB Rs. 30×3/4 2 WEB Rs. 18×3/4 2 Ls. 8×6×3/4 BOTT. 1 TOP COV. R. 38×1 1 P.B. COV. R. (42-14)×7/8	2 WEB Rs. 30×1 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (22-8)×3/8	2 WEB Rs. 30×3/4 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (22-8)×3/8	2 WEB Rs. 26×9/16 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (22½-8)×3/8	2 WEB Rs. 26×11/16 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (22-8)×3/8	2 WEB Rs. 24×9/16 4 Ls. 4×4×3/8 2 PERF. COV. Rs. (23-8)×3/8	2 Rs. OUTSIDE WEBS 23×3/4 1 CENTRE WEB 23×7/8 4 Ls. 5×5×3/4 4 Ls. 4×4×3/4 2 COVER Rs. 27×1/2 2 CENTRE WEB 13×3/4	1 CENTRE WEB 23×1/2 2 SIDE PERF. Rs. (23-10)×3/8 4 Ls. 5×5×3/8 4 Ls. 4×4×1/2 2 FL. Rs. 19½×1/2	1 CENTRE WEB 23×1/2 2 SIDE PERF. Rs. (23-10)×3/8 4 Ls. 5×5×1/2 4 Ls. 4×4×3/8 2 FL. R. 19½×3/8	AS U_1L_1	AS U_2L_2	AS U_1L_1	AS U_2L_2
AREA OF SECTION (GROSS)	206.0		66.6		57.1	48.7			66.3		66.3		66.3
SQ. IN. (NET)		69.3		41.8		40.8	104.24	60.6		60.6		60.6	
SECTION MODULUS (CU. IN.)	(886) Y-Y (1761) X-X	485.2 NET	517 GROSS	307 NET	405 GROSS	277 NET 341 GROSS	744 NET	512 NET	555 GROSS	512 NET	555 GROSS	512 NET	555 GROSS
ACTUAL UNIT STRESS (DIRECT)	6.77	-17.40	+13.80	17.50	8.40	-8.70 +3.13	11.21	4.97	0.53	5.10	0.61	5.18	0.64
KIPS/SQ. IN. (BENDING)	(9.64) Y-Y (0.47) X-X	-1.78	+1.50	1.99	1.72	-2.19 +1.78	12.18	9.20	8.46	9.20	8.48	9.20	8.48
$f_a/F_a + f_b/F_b$	0.966	0.960	1.004	0.973	0.577	0.545 0.354	0.975	0.709	0.499	0.715	0.505	0.718	0.507
MATERIAL	CARBON	CARBON	CARBON	CARBON	CARBON	CARBON	LOW ALLOY	CARBON	CARBON	CARBON	CARBON	CARBON	CARBON

LIFTING GIRDER

VERTICAL SHEAR (KIPS)	ROPE LOAD	1201
	GIRDER WEIGHT	-31
	TOTAL	1170
VERTICAL MOMENT (FT.-KIPS)	ROPE LOAD	11002
	GIRDER WEIGHT	-296
	TOTAL	10706
HORIZONTAL SHEAR TEMPERATURE (KIPS)		7.75
HORIZONTAL MOMENT TEMPERATURE (FT. KIPS)		57.2
WEB REQ'D. SQ. IN.		117
SECTION	WEB	169 × 7/8
	ANGLES	4 Ls. 8 × 8 × 3/4
	SIDE PLATES	4 Rs. 16 × 1/2
	TOP COVER PLATES	1 R. 36 × 1/2
	BOTTOM COVER PLATES	1 R. 26 × 3/4
	TOP FLANGE ANGLES	2 Ls. 4 × 5 × 1/2
	BOTTOM FLANGE ANGLES	2 Ls. 4 × 5 × 1/2
NET SECTION MODULUS VERT. BENDING (CU. IN.)		9015
NET SECTION MODULUS HORIZ. BENDING (CU. IN.)		177 (TOP FLG.)
MAXIMUM KIPS/SQ. IN. BENDING STRESS	VERT. MOMENT	14.3
	HORIZ. MOMENT	2.9
	TOTAL	17.2 (TOP)
ALLOWABLE UNIT STRESS (KIPS/SQ. IN.)		15.2 (BOTT.) 18.0 (TOP)
WEB AREA PROVIDED SQ. IN.		148.0
MATERIAL		CARBON

GENERAL NOTES:

1. GENERAL NOTES ON SHEET NO. 2 APPLY HERE.
2. CAMBER: THE LIFT SPAN TRUSSES SHALL BE SO CAMBERED THAT THE STRUCTURE WILL RETAIN ITS GEOMETRIC SHAPE UNDER FULL DEAD LOAD PLUS HALF LIVE LOAD.
3. MATERIAL: MEMBERS FOR WHICH STRUCTURAL LOW-ALLOY STEEL SHALL BE USED ARE DESIGNATED WITH (LA). ALL OTHER MEMBERS ARE STRUCTURAL CARBON STEEL. CONNECTING MATERIALS BETWEEN LOW-ALLOY MEMBERS AND BETWEEN LOW-ALLOY AND CARBON STEEL MEMBERS SHALL BE LOW-ALLOY STEEL, AND ALL OTHER CONNECTING PARTS, LACING AND DIAPHRAGMS SHALL BE CARBON STEEL. ALL RIVETS TO BE CARBON STEEL.
4. RIVETS: 1" Ø RIVETS TO BE USED FOR RAILWAY SIDE MAIN TRUSS. 7/8" Ø RIVETS FOR HIGHWAY SIDE MAIN TRUSS AND ALL FLOOR AND BRACING SYSTEMS, EXCEPT AS NOTED.
5. ALLOWABLE ERECTION STRESSES:
   D.L. + ERECTION LOAD - 1.25 NORMAL UNIT STRESSES.
   D.L. + ERECT. LOAD + 30 P.S.F. WIND - 1.33 NORMAL UNIT STRESSES.
6. SIGN CONVENTIONS:
   + FOR COMPRESSION.
   - FOR TENSION.
7. MOMENT:
   MX-X FOR MOMENT ABOUT HORIZONTAL AXIS.
   MY-Y FOR MOMENT ABOUT VERTICAL AXIS.
8. MINIMUM THICKNESS OF MATERIAL: 3/8" EXCEPT FOR FILLS AND WHERE NOTED OTHERWISE.
9. MINIMUM THICKNESS OF GUSSET PLATES: 1/2" UNLESS NOTED OTHERWISE.

REGISTERED PROFESSIONAL ENGINEER — CIVIL — I. M. WALLACE — PROVINCE OF ONTARIO

NO.	REVISIONS	BY	DATE

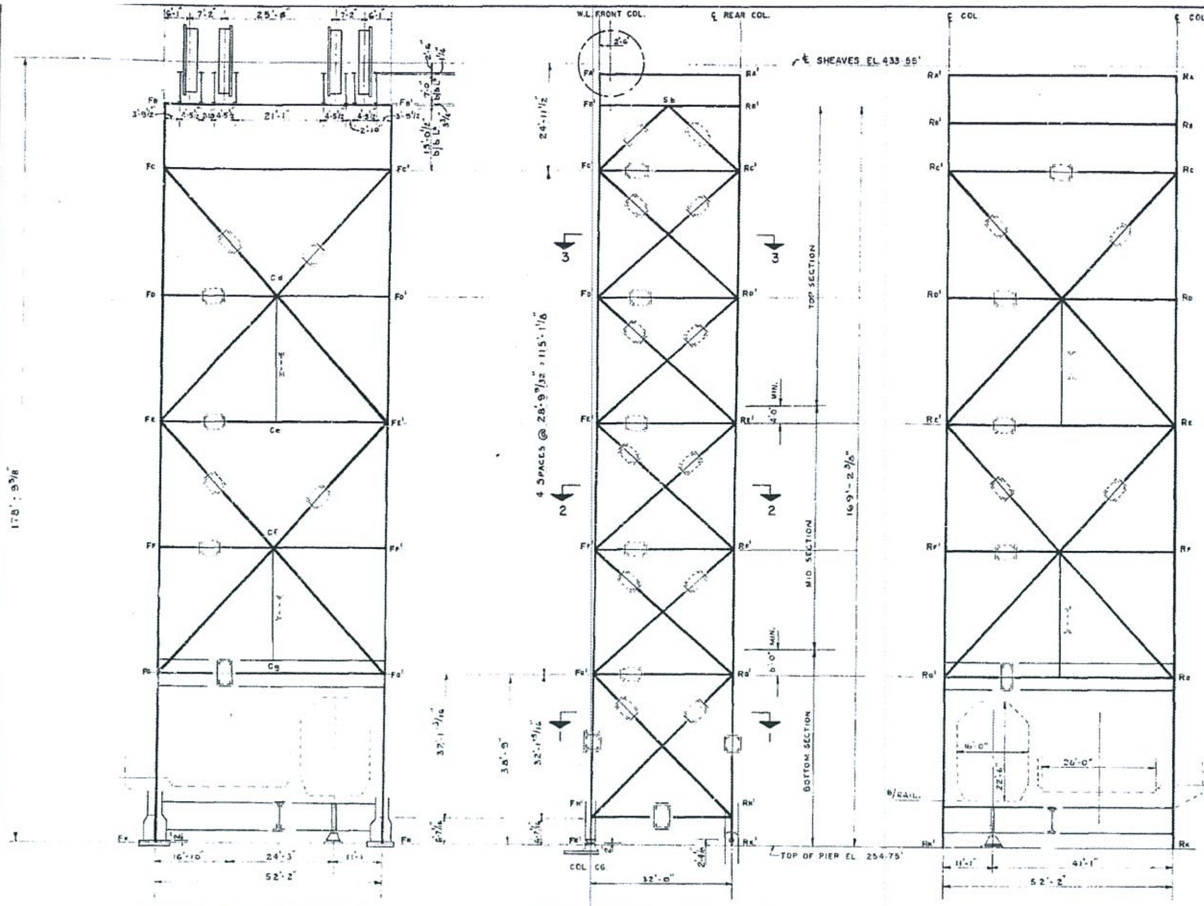
DEPARTMENT OF PUBLIC WORKS
CANADA
DEVELOPMENT ENGINEERING BRANCH
STRUCTURES DIVISION

C.C. PARKER & ASSOCIATES LTD.
CONSULTING ENGINEERS
HAMILTON ONTARIO

BURLINGTON CANAL LIFT BRIDGE

LIFT SPAN TRUSSES - STRESS SHEET

RECOMMENDED DATE 1-12-56	DESIGN D.J.D.	CHKD. R.W.A.	APPROVED DATE 13/1/58 (CHIEF STRUCTURES DIVISION)	DEPARTMENT PROJECT NO. SD6-4-77
C.C. Parker C.C. PARKER & ASSOC. LTD.	DRAWN B.L.	CHKD. R.K.C.C.	APPROVED DATE 14-1-58 (CHIEF ENGINEER)	CONTRACT NO. 2
	TRACED W.E. & D.H.	CHKD. A.T.		SHEET 3 OF 62
	JOB NO. H-538			



LOAD TO LIFT 4000,000 LBS.

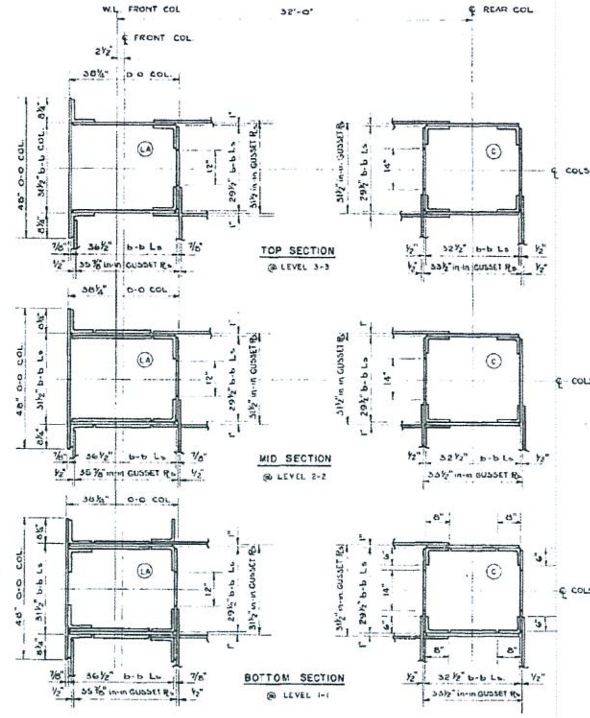
SHEAVE LOAD

SPAN 4000 x 1/8 =	500
COUNTERWEIGHT	500
ROPES & ATTACHMENTS	35
SHEAVE SHAFT & BEARINGS	65
OPERATING FORCE & IMPACT	40
TOTAL PER SHEAVE	1140

BEARING LOADS

554 PER BEARING AT G1 & G4

586 PER BEARING AT G2 & G3



TOWER COLUMNS

SCALE ½"=1'-0"

SHEAVE GIRDERS LONGITUDINAL TRANSVERSE

GIRDER		G1 & G1A MOMENT	G1 & G1A SHEAR	G2, G2A, G3, G3A MOMENT	G2, G2A, G3, G3A SHEAR	G4 G4A MOMENT	G4 G4A SHEAR	G6 G6A MOMENT	G6 G6A SHEAR	G7 MOMENT	G7 SHEAR	G8 MOMENT	G8 SHEAR
DEAD LOAD		37'K	25K	36'K	31K	900'K	79K				294K		226K
SHEAVE LOAD		1282	513	1355	542	850	512				2109		172
IMPACT. SHEAVE LOAD		258	103	270	108	170	103				422		34
DL.+ SL.+ IMPACT		1577	641	1661	581	1920	694	312'K	31K	26187'K	2825	4768'K	432
COUNTERWEIGHT ON GIRDERS				3500	350								
DL.+ CTWT ON GIRDERS				3802	429								
NET SECTION MODULUS REQ'D IN³	@ 18.0 K/□"	1052				1280		208					
	@ 22.5 K/□"			2025								3180	
	@ 24.0 K/□"									14100			
GROSS AREA OF WEB REQ'D IN²	@ 11.0 K/□"		58.5		62		63.2		3				
	@ 14.0 K/□"										202		39.3
SECTION	WEB FLANGE	84 x 3/4		84 x 3/4		84 x 3/4		84 x 1/2		1 ℞ 180 x ½ @ MID SPAN 2 ℞s 180 x 9/16 @ END SPAN		1 ℞ 132 x ½"	
	Ls AND	4 Ls 8 x 6 x 3/4		4 Ls-8 x 8 x 7/8		4 Ls-8 x 6 x 3/4		4 Ls-6 x 6 x 1/2		4 Ls- 8 x 8 x 1		4 Ls-8 x 6 x 5/8	
	FLANGE ℞s												
	COVER ℞s									4 ℞s-26 x 7/8		2 ℞s-20 x 1/2	
GROSS AREA OF WEB PROV'D IN²		63.0		63.0		63.0		42.0		247		66	
NET SECTION MODULUS PROV'D IN³		1800		2260		1800				14300		3640	
MATERIAL		CARBON		CARBON		CARBON		CARBON					
STIFFENERS @ SUPPORT		8 Ls-6 x 4 x 7/8		8 Ls-7 x 4 x 7/8		8 Ls-7 x 4 x 7/8				8 Ls-8 x 4 x 3/4			
STIFFENERS UNDER CONC. L.		8 Ls-6 x 4 x 7/8		8 Ls-7 x 4 x 7/8		8 Ls-7 x 4 x 7/8				8 Ls-8 x 4 x 3/4			
INTERMEDIATE STIFFENERS		2 Ls-5 x 3½ x 3/8		2 Ls-6 x 4 x 3/8		2 Ls-5 x 3½ x 3/8		2 Ls-5 x 3½ x 3/8		2 Ls-8 x 4 x 3/4		2 Ls-7 x 4 x 5/8	
MATERIAL		CARBON STEEL								LOW ALLOY		CARBON STEEL	

REACTION IN KIPS PER COLUMN

			FRONT COLUMN SPAN UP RWY	SPAN UP HWY	SPAN DOWN RWY	SPAN DOWN HWY	CTWT JACKED RWY	CTWT JACKED HWY	REAR COLUMN SPAN UP RWY	SPAN UP HWY	SPAN DOWN RWY	SPAN DOWN HWY	CTWT JACKED RWY	CTWT JACKED HWY
VERTICAL	DEAD LOAD		881	1002	881	1002	735	822	410	588	410	588	460	657
	SHEAVE LOAD		1920	1920	1920	1920	819	819	160	160	160	160	261	231
	MESSENGER CABLE		170	170	170	170	170	170	-159	-159	-159	-159	-159	-159
	TOTAL STATIC LOAD		2971	3092	2971	3092	1724	1811	411	589	411	589	552	759
	2/3 DEAD LOAD		1981	2061	1981	2061	1149	1207	274	393	274	393	368	506
	ROOF LIVE LOAD		66	66	66	66	66	66	72	72	72	72	72	72
	SHEAVE LOAD IMPACT		384	384	384	384	—	—	32	32	32	32	—	—
	TOTAL D.L. + L.L. + IMPACT		3421	3542	3421	3542	1790	1273	515	693	515	693	624	831
	30 P.S.F.	TRANS. WIND	± 156	± 156	± 198	± 198			± 682	± 682	± 200	± 200		
		LONG WIND	± 973	± 973	± 652	± 649			± 973	± 973	± 652	± 649		
		45° WIND	± 798	± 798	± 642	± 635			± 1170	± 1170	± 661	± 611		
	MAX DL + L.L. + IMPACT + 30 P.S.F. WIND		+4394	+4515	+4073	+4191			+1685	+1863	+1167	+1342		
	50 P.S.F.	TRANS. WIND			± 326	± 326	± 326	± 326			± 326	± 326	± 326	± 326
		LONG. WIND			± 1080	± 1080	± 1070	± 1080			± 1080	± 1080	± 1080	± 1080
		45° WIND			± 994	± 994	± 994	± 994			± 994	± 994	± 994	± 994
	MAX. D.L. + 50 P.S.F. WIND				±4051	±4172	±2804	±2891			±1491	±1669	±1632	±1839
	2/3 D.L. -30 P.S.F. WIND		+1008	+1008	+1329	+1412			-896	-777	-378	-256		
	2/3 D.L. -50 P.S.F. WIND				+901	+981	+69	+127			-806	-687	-712	-574
HORIZONTAL	MESSENGER CABLE		-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15
	TRANS. WIND		± 164	± 12	± 131	± 11	± 131	± 11	± 113	± 5	± 131	± 6	± 131	± 6
	LONG WIND		± 138	± 138	± 137	± 137	± 137	± 137	± 138	± 138	± 137	± 137	± 137	± 137
	L.L. WIND													
	NOSING													

FRONT BRACING

		Fc Fc' / Fc' Fc	Fe Fg' / Fe' Fg	Fd Fd'	Fe Fe'	Ff Ff'	Fg Fg'	Cd Ce / Cf Cg
2½% COLUMNS KIPS		± 112	± 117	+75 -63	+77 -65	+78 -66	+80 -80	
TRANSVERSE WIND KIPS		± 184	± 280	+104 -26	+133 -15	+199 -42	+133 -15	
MOMENT FROM TRAN WIND - FT - KIPS				154	210	210	2795 210	
DESIGN STRESS KIPS		± 296	± 397	+179 -89	+210 -80	+276 -108	+215 -95	
L/T		61.0	61.3	39.7	41.0	41.2	39.4	
ALLOW. UNIT STRESS K/IN²	COMPRESSION	15.5	15.5	16.0* 16.4**	16.0* 16.4**	16.0* 16.4**	16.1* 19.0**	
	TENSION	20.0	20.0	20.0	20.0	20.0	20.0	
AREA REQ'D IN²	GROSS	21.9	28.6	20.1	25.5	30.2	115	
	NET	16.2	22.3	5.2	4.7	6.5	99	
SECTION		4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 1/2 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 1/2 2 ℞s 24 x 3/8	4 Ls 8 x 8 x 3/4 2 ℞s 37 x 5/8 2 ℞s 70 x 1/2	4 Ls 4 x 3 x 3/8
AREA OF SECTION - IN²	GROSS	25.7	30.3	25.7	25.7	30.3	144.5	
	NET	21.2	31.5	21.2	21.2	31.5	119	
SECTION MODULUS - IN³ (GROSS)				322	336	399	3200 1020	
MATERIAL		CARBON STEEL						

* UNIT AXIAL STRESS ** UNIT FLEXUAL STRESS

REAR BRACING

Rc' Rc / Rc Rz'	Re' Rg / Re Hg'	Rc' Rc	Rd' Rd	Re' Re	Rf' Rf	Rg' Rg	Md Me / Mf Mg
± 22	± 25	± 25	± 17	± 19	± 21	± 18.3	
± 95	± 98	± 87	± 11	—	± 31	—	
		544				1443	
± 117	± 123	± 112	± 28	± 19	± 52	± 19.1	
64.7	64.7	43.4	63.7	63.7	63.7	43	
15.3	15.3	16.0* 17.9**	15.4* 16.3**	15.4* 16.3**	15.4* 16.3**	16.0* 16.3**	
20.0	20.0	20.0	20.0	20.0	20.0	20.0	
10.5	10.9	38.7			7.5	28.7	
7.3	7.5	32.6			3.2	21.4	
4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 1/2 2 ℞s 24 x 3/8	4 Ls 6 x 6 x 3/4 2 ℞s 35 x 1/2 2 ℞s 32 x 1/2	4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 7 x 4 x 3/4 2 ℞s 70 x 1/2 2 ℞s 32 x 1/2	4 L 4 x 3 x 3/8
25.7	25.7	86.8	22.7	22.7	22.7	118.8	
21.2	21.2	69.8	18.2	18.2	18.2	93.8	
CARBON STEEL							

FRONT TOWER COL / REAR TOWER COL*

MEMBER		Fc Fc / Fc' Fc' (FRONT)	Fe Fe / Fe' Fe' (FRONT)	Fd Fe / Fd' Fe' (FRONT)	Rb Rc / Rb' Rc' (REAR)	Rc Rb / Rc' Rb' (REAR)	Rb Rc / Rb' Rc' (REAR)
DEAD LOAD KIPS		+ 571	+ 687	+ 781	+ 438	+ 509	+ 570
SHEAVE LOAD KIPS		+ 1920	+ 1920	+ 1920	+ 160	+ 160	+ 160
IMPACT - SHEAVE LOAD KIPS		+ 384	+ 384	+ 384	+ 32	+ 32	+ 32
AUX. CTWT + 20% IMPACT KIPS		—	+ 89	+ 89	—	- 2	- 2
MESSENGER CABLE KIPS		+ 66	+ 120	+ 148	- 55	- 109	- 137
TOTAL DIRECT LOAD WITHOUT WIND KIPS		+ 2941	+ 3200	+ 3322	+ 575	+ 590	+ 623
TRANSV. WIND KIPS		± 73	± 333	± 635	± 35	± 127	± 261
DIRECT LOAD + TRANSV. WIND KIPS		+ 3014	+ 3533	+ 3957	+ 610	+ 717	+ 884
TRANSV. WIND MOMENT FT. - KIPS		736	1324	4000	37	+ 604	1862
LONG. WIND KIPS		± 166	± 556	± 988	± 267	± 651	± 1082
DIRECT LOAD + LONG. WIND KIPS		+ 3107	+ 3756	+ 4310	+ 842	+ 1241 - 113	+ 1705 - 511
LONG. WIND MOMENT FT.-KIPS		373	373	433	—	—	—
45° WIND KIPS		± 170	± 628	± 1147	± 216	± 545	± 963
DIRECT LOAD + 45° WIND KIPS		+ 3111	+ 3828	+ 4469	+ 791	+ 1135 - 7	+ 1586 - 392
TRANSV. WIND MOM. (45° WIND) FT-KIPS		679	935	2823	27	427	1315
LONG. WIND MOM. (45° WIND) FT. KIPS		263	263	306	—	—	—
MOM. FROM SHEAVE GIRDER	TRANSV. FT.-KIPS	573	36	—	499	—	—
	LONGIT. FT.-KIPS	43	15	119	416	2	2
AREA REQ'D IN² (GR)	D.L. + SHEAVE	183	165	176.5	90	36	39
	D.L. + SHEAVE + TRANS. W.	182	206	345	76	60	121
	D.L. + SHEAVE + LONG. W.	167	173	200	85	86	85
	D.L. + SHEAVE + 45° W.	189	216	326	84	73	132
L/R		21	26.9	31.7	28.4	28.4	35.9
ALLOW. UNIT STRESS K/IN²		19.8 X 1.25 = 24.7	19.7 X 1.25 = 24.6	19.5 X 1.25 = 24.4	16.3 X 1.25 = 20.4	16.3 X 1.25 = 20.4	16.1 X 1.25 = 20.1
SECTION		1 ℞ 48 x 7/8 4 Ls 8 x 8 x 1 2 ℞s 36 x 1 1 ℞ 29 x 7/8	1 ℞ 48 x 7/8 4 Ls 8 x 8 x 1 2 ℞s 36 x 1 1 ℞ 29 x 7/8 2 ℞s 20 x 1 2 ℞s 28 x 1	1 ℞ 48 x 7/8 4 Ls 9 x 8 x 1 2 ℞s 36 x 1 1 ℞ 29 x 7/8 2 ℞s 20 x 1 2 ℞s 28 x 1 4 Ls 8 x 8 x 1	2 ℞s 32 x 1 4 Ls 8 x 6 x 1 2 ℞s 29 x 1/2	2 ℞s 32 x 1 4 Ls 8 x 6 x 1 2 ℞s 29 x 1/2	2 ℞s 32 x 1 4 Ls 8 x 6 x 1 2 ℞s 29 x 1/2 2 ℞s 16 x 1
AREA OF SECTION - IN² (GR)		188.87	284.87	345.75	131	131	163
SECTION MODULUS IN³ (GR)	TRANSV. ENTIRE SECT.	1720	2575	3250	1635	1635	2045
	LONG. ENTIRE SECT.	1920	2170	2990	1165	1185	1205
MATERIAL		LOW ALLOY			CARBON		

* REAR COLUMN STRESSES AND MOMENTS ARE CONTROLLED BY THE CONDITION WHEN SPAN IS DOWN WITH 50#/□' WIND

SIDE BRACING

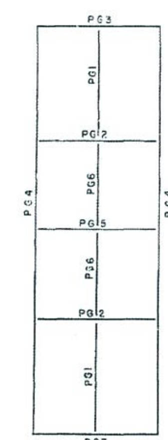
MEMBER		Fc' Rd' / Rc' Fd'	Fd' Re' / Rd' Fe'	Fe' Rf' / Re' Ff'	Ff' Rg' / Rf' Fg'	Fg' Rh' / Rg' Fh'	Fc' Rc'	Fd' Rd'	Fe' Re'	Ff' Rf'	Fg' Rg'	Fh' Rh'	Sb Fc' / Sb Rc'
2½% COLUMNS		± 84	± 103	± 106	± 108	± 109	± 74	± 76	± 77	± 79	± 80		± 102
MESSENGER CABLE		± 20	± 20	± 20	± 20	± 22							± 21
WIND	CASE GOVERNING			LONG WIND			L. W.	L. W.	L. W.	L. W.	L.W.		L. W.
	STRESS	± 108	± 136	± 146	± 161	± 191		± 52	- 104	- 56	- 104		± 33
DESIGN STRESS		± 212	± 259	± 272	± 289	± 322	± 74	± 127	+ 77 OR - 181	+ 79 OR - 135	+ 80 OR - 184		± 156
DESIGN MOMENT							76	104	208	194	208		
L/R		34.7	34.7	34.7	34.7	37.2	47.4	47.4	47.4	47.4	42.8		33.4
ALLOWABLE UNIT STRESS K/□"	COMPRESSION	16.2	16.2	16.2	16.2	16.1	16.1* 16.4**	16.0* 16.4**	15.9* 18.6**	15.9* 18.6**	16.0* 18.6**		16.2
	TENSION	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0		20.0
AREA REQ'D. IN² (GR)	GROSS	14.3	16.9	17.6	18.9	18.9	15.6	23.8	17.4	20.8	15.5		11.5
	NET	10.6	13.0	13.6	14.4	16.1	11.4	16.7	20.6	15.9	24.9		7.8
SECTION		4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 4 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 3/8 2 ℞s 24 x 3/8	4 Ls 6 x 4 x 1/2 2 ℞s 24 x 3/8	4 Ls 8 x 6 x 1 2 WEB ℞s 71 x 5/8 2 ℞s 30 x 3/4	4 Ls 4 x 4 x 1/2 2 ℞s 24 x 3/8
AREA OF SECTION - IN²	GROSS	22.7	22.7	22.7	22.7	22.7	25.7	25.7	25.7	25.7	30.2		26.3
	NET	18.2	18.2	18.2	18.2	18.2	21.2	21.2	21.2	21.2	26.7		20.8
NET SECTION MODULUS - IN³													
MATERIAL		CARBON STEEL											

* UNIT AXIAL STRESS ** UNIT FLEXUAL STRESS IN LATERAL DIRECTION

COUNTERWEIGHT GIRDERS

NORMAL CONDITIONS		P.G.1.	P.G.2	P.G.3.	P.G.4	P.G.5.	P.G.6
SHEAR KIPS	D.L.	520	257.2	257.3	253.0	—	—
	IMPACT	103	51.5	51.5	50.3	—	—
	TOTAL	623	308.7	308.8	303.3	—	—
WEB AREA REQUIRED AT 11 S.K.I.		56.6	28.1	28.1	27.6	—	—
MOMENT (FT.-KIPS)	D.L.	1499	1786	1788	1140	—	—
	20% IMPACT	299	353	354	230	—	—
	TOTAL	1798	2139	2142	1370	—	—
NET SECTION MOD REQ'D AT 18 K.S.I.		1200	1423	1430	913	—	—
EMERGENCY CONDITION *							
D.L. SHEAR (KIPS)		525	257.3	257.3	253.0		
WEB AREA REQ'D AT 13.75 K.S.I.		38.2	18.7	18.7	18.4		
D.L. MOMENT (FT.-KIPS)		3354	1786	1788	1140		
NET SECTION MOD REQ'D AT 22.5 KSI		1787	953	954	608		
SECTION USED	WEB	118 x 7/8	120 x 5/8	207 x 1/2	207 x 1/2	180 x 3/8	118 x 3/8
	FLANGE Ls TOP	2 Ls 8 x 6 x 5/8	2 Ls 6 x 6 x 1/2	1 Ls 8 x 4 x 1/2	1 Ls 6 x 4 x 1/2	2 Ls 4 x 4 x 3/8	2 Ls 4 x 4 x 3/8
	FLANGE Ls BOTT	2 Ls 8 x 6 x 5/8	2 Ls 6 x 6 x 1/2	1 Ls 5 x 5 x 1/2	1 Ls 4 x 4 x 1/2	2 Ls 4 x 4 x 3/8	2 Ls 4 x 4 x 3/8
	COVER PLATE	—	—	—	—	—	—
AREA OF WEB USED - SQ. IN.		103.3	75	103.5	103.5		
NET SECTION MOD. USED (TENSION)		3260	2580	4250	4140		
DEPTH		9'-10½"	10'-0½"	17'-3½"	17'-3½"	15'-0½"	9'-10½"
MATERIAL		CARBON					

* COUNTERWEIGHT HUNG FROM GIRDERS.



CWT GIRDER-LOCATION PLAN

NOTES:

1. FOR GENERAL NOTES SEE SHEET Nos. 2 & 3.
2. RIVETS: 1" Ø RIVETS TO BE USED FOR FRONT TOWER COLUMNS. 7/8" Ø RIVETS TO BE USED FOR REAR TOWER COLUMNS AND ALL BRACING MEMBERS.
3. CAMBER: TOWER COLUMNS SHALL BE CAMBERED FOR FULL TOWER DEAD LOAD AND SHEAVE LOAD.
4. LOADING CONDITIONS:
   CASE 1 — SPAN UP = DIRECT LOADS + 30 P.S.F. WIND.
   CASE 2 — SPAN DOWN = DIRECT LOADS + 50 P.S.F. WIND.
   CASE 3 — CWT. JACKED: DIRECT LOADS + 30 P.S.F. WIND.
5. FOR JACKING GIRDER: SEE SHEET No. 13.
6. SIDE BRACING DIAGONALS FgRh AND RgFh OF WEST FACE OF NORTH TOWER TO BE CHANGED TO PROVIDE FOR STAIR CASE TO ELEVATOR, SEE SHEET No. 31 FOR MATERIALS AND SECTIONS.

NO.	REVISIONS	BY	DATE

DEPARTMENT OF PUBLIC WORKS
CANADA
DEVELOPMENT ENGINEERING BRANCH
STRUCTURES DIVISION

C·C·PARKER & ASSOCIATES LTD
CONSULTING ENGINEERS
HAMILTON ONTARIO

BURLINGTON CANAL LIFT BRIDGE

TOWER-STRESS SHEET

REGISTERED PROFESSIONAL ENGINEER CIVIL I. M. WALLACE PROVINCE OF ONTARIO

RECOMMENDED	DATE 1-12-58.	DESIGN R.K.C.C.	CHKD. G.W.A.
C.C. Parker		DRAWN J.B.	CHKD. G.W.A.
C·C· PARKER & ASSOC· LTD·		TRACED M.C. & D.H.	CHKD. R.K.C.C.

APPROVED DATE 13/11/58 — [signature] CHIEF STRUCTURES DIVISION

APPROVED DATE 14.11.58 — [signature] CHIEF ENGINEER

DEPARTMENT PROJECT NO· SD6-4-77

CONTRACT NO· 2

SHEET 19 OF 62