AECOM

Forth Road Bridge Stiffening Truss Assessment Category 3 Check
Assessment Check Report

Forth Estuary Transport Authority
November 2010

Prepared by: Checked by:

Forth Road Bridge Stiffening Truss Assessment Category 3 Check

Rev No	Comments	Checked by	Approved by	Date
1	First Draft	CPEC	CPEC	05/06/09
2	Second Draft	CPEC	CPEC	Aug 09
3	First Issue	CPEC	CPEC	Nov 10

Enterprise House, 160 Croydon Road, Beckenham, Kent, BR3 4DE
Telephone: 0870 905 0906 Fax: 020 8663 6723 Website: http://www.aecom.com

Job No 60047345 Reference 60047345/R001

This document has been prepared by AECOM Limited ("AECOM") for the sole use of our client (the "Client") and in accordance with generally accepted consultancy principles, the budget for fees and the terms of reference agreed between AECOM and the Client. Any information provided by third parties and referred to herein has not been checked or verified by AECOM, unless otherwise expressly stated in the document.

No third party may rely upon this document without the prior and express written agreement of AECOM.

c:\documents and settings\jamess1\my documents\assessment report aecom 3 dec 2010.doc

Table of Contents

Assessment Report

1 Introduction

Aecom, formerly Faber Maunsell (FM) has been commissioned by the Forth Estuary Transport Authority (FETA) to carry out an independent assessment of the deck including the stiffening truss, cross girders, orthotropic deck and side span RC deck of the Forth Road Bridge.

The Consulting Engineering Company, W.A. Fairhurst & Partners (WAF), have carried out an assessment of the above elements and a report entitled “Stiffening Truss Assessment Report” issue 5 dated March 2008 was issued by WAF. It is Aecom’s understanding that the brief is to carry out an independent assessment of these elements and compare Fairhurst’s results with those obtained from this independent review.

In this report, Aecom will concentrate primarily on the assessment of the bridge in accordance with established UK practices using wherever appropriate assessment live loads. We believe that the prime aim of this report is to first establish those areas of the deck truss which fail the assessment. Where apparent failures occur we will aim to explain the reasons for this apparent failure and wherever possible provide justification. We will then report on the extent and approximate magnitude of the overstress at the areas which fail their assessment, when the bridge is subjected to the full design loads.

2 Sources of Information

We have used the following sources of information in the assessment check.

- Relevant As-Built drawings and drawings of works carried after completion of the bridge have been obtained from the Forth Estuary Transport Authority Administration building.
- Institution of Civil Engineers Proceedings of the Forth Road Bridge (1965) – A complete record of the design and construction of the Forth Road Bridge by engineers responsible for the project.
- Resident Engineer's Report (February 1965) – Report on Completion of steelwork, concreting of side span roadway decks, surfacing of roadway and footway/cycle tracks, and electrical installations (Mott, Hay & Anderson & Freeman Fox and Partners).
- Resident Engineer's Report (September 1962) - Report on Cable Spinning (Mott, Hay & Anderson & Freeman Fox and Partners).
- Various W.A. Fairhurst reports of survey work carried out on the bridge reviewed of the Forth Estuary Transport Authority General Managers Office.
- BSALL Report dated November 2006, produced by W.A. Fairhurst.
- Wind Tunnel Testing of Deck Structure – University of Glasgow (April 2006)
- Bridge-Specific Footway Live Loading Report dated June 2006, produced by W.A. Fairhurst

3 Assessment Methodology

3.1 Introduction and General Approach to Analysis

In this report, Aecom will concentrate primarily on the assessment of the bridge in accordance with established UK practices using wherever appropriate assessment live loads.

Aecom acknowledges that the assessment procedure set out in BD 50/92 is that the bridge should be first assessed using the nominal HA and KEL as specified in BD37 reduced by the appropriate reduction factor. The second stage is to then re-assess using a Bridge Specific Assessment Live Loading (BSALL) if the bridge does not pass the initial assessment. The AIP prepared by WAF specifies that the assessment should be checked for both HA (and HB) and BSALL but does not provide any further guidance on the priority or the order in which the checks should be made.

We believe that the prime aim of this report is to first establish those areas of the deck truss which fail the BSALL assessment. Where apparent failures occur we will aim to explain the reasons for this apparent failure and wherever possible provide justification. We will then report on the extent and approximate magnitude of the overstress at the areas which fail their assessment, when the bridge is subjected to the full HA design loads.

We have therefore carried out the assessment in two stages as follows:

Stage One

- Dead + BSALL
- Dead + BSALL + 50mph wind
- Dead + Full Wind

Stage Two

- Dead + HA
- Dead + HA + Full Wind
- Dead + HB

The assessment will generally be carried out in accordance with the AIP prepared by WAF. However, as mentioned above, the emphasis will be placed on the assessment in accordance with standard assessment procedures.

The assessment has been undertaken in accordance with BD56/96: Assessment of Steel Highway Bridges and Structures and BD 21/01: Assessment of Highway Bridges and Structures. In addition and in accordance with BD 50/92 a Bridge Specific Assessment Live Loading (BSALL) has been developed by WAF and the most recent version of June 2006 has been used in the assessment. The derivation of the BSALL has not been checked by Aecom.

3.2 Dead Load

Nominal dead load was calculated for each component of the suspended bridge structure. A breakdown of the loadings is summarised in Table 3.1 below for both as-built loads and loads applied to the structure after construction. The majority of information was obtained from drawings believed to be that of As-Built record.

The majority of drawings obtained are fabrication drawings. However, it should be noted that any certainty of having the latest revisions of each drawing or the access to information of any late changes to any such details cannot be fully guaranteed, and information received is that obtained from the Forth Estuary Transport Authority records which is believed to be correct.

An independent review of all nominal dead loading on the bridge was undertaken by AECOM with the exception of surfacing thicknesses and densities. A recording of thicknesses and densities was carried out by W.A. Fairhurst in 2004 and is believed by AECOM to be a more accurate assessment of the actual loading on the bridge. Table 3.1 shows the loadings calculated by AECOM.

The loadings given in the ICE Proceedings are slightly greater than the AECOM loads, the difference being 3.3% and 1.3% for the side span and main span respectively. However, this initial comparison omits superimposed dead load from services and other miscellaneous items that were added to the bridge subsequent to its opening. When these additional loads are added to the initial dead load and superimposed dead load (for both the AECOM and WAF values) then the loading matches that of the ICE Proceedings more closely.

In the document, Resident Engineer's Report 1965, Chapter 6 records information referring to the deflections of the suspended structure. In this it explains "an apparent lightness in suspended structure weight of 8.5% in the south side span, 7.5% in the main span and 10% in the north side span". This could explain the lower loadings calculated by AECOM and therefore indicate that these loads are more likely a correct assessment of actual loadings on the bridge.

It is thought that during spinning, the free cable geometry was set to achieve the correct theoretical profile of the deck using calculated weights of the suspended structure. The theoretical weights tabulated in the ICE proceedings have therefore been used to calculate pre-tension forces in the main cables for the structure model used in this assessment. The self-weight dead load and superimposed dead load applied are those calculated by AECOM, including the additional superimposed dead loads introduced after the bridge was opened.

Table 3.1 Comparison of Design Loads and As-built Loads

Forth Road Bridge Suspended Weight Check (Full width of Deck)	ICE Proceedings Paper Main span kN/m	ICE Proceedings Paper Side span kN/m	AECOM Estimate Main span kN/m	AECOM Estimate Side span kN/m
Cables, hangers & details	38.803	39.882	127.386	101.432
Stiff trusses, laterals, cgs	48.186	44.982		
Deck – Steel	31.121	10.984		
Ftwy steel	11.409	11.670		
Deck – Concrete	0.000	76.855	0	76.622
Deck – Asphalt	14.580	15.626	23.865	24.413
Ftwy rubber/bit	0.981	0.981		
Parapets etc	4.773	4.838		
Drainage	0.360	0.327		
Electric & telephone cables	0.981	0.981		
Zinc spray & paint	2.059	2.059		
Total (Original Construction)	153.252	209.186	151.251	202.467

Additional Loading since first Construction

Street lighting Water & air pipes	0.523	0.507
Under deck walkways Runway beams	5.818	5.933
Halving joint billets	0.216	0.000
Total Additional Loading	6.557	6.440

Grand Total	**153.252**	**209.186**	**157.808**	**208.907**

3.3 Vehicle Loading

3.3.1 BSALL

For the stiffening truss live loading intensities for the critical loaded lengths were obtained from the WAF BSALL dated June 2006.

The following Table 3.2 is an extract from the above report and summarises the range of load intensities used in the assessment

Table 3.2 Summary of BSALL Loading

Loaded Length (m)	Lane 1(kN/m)	Lane 2 Factor	Lane 3 Factor	Lane 4 Factor
100	20.45	0.67	0.33	0.33
200	16.70	0.67	0.33	0.33
300	15.08	0.67	0.33	0.33
408	14.20	0.67	0.33	0.33
1006	12.48	0.67	0.33	0.33
1414	12.16	0.67	0.33	0.33
1823	11.96	0.67	0.33	0.33

Influence lines were created to establish regions where vehicle and footway loading would adversely affect each element. Due to the non-linear behaviour of the structure it was necessary to establish the most adverse loaded lengths by trial and error, since the cusped influenced line method set out in BD37/01 demonstrably did not tend to produce the most adverse result. Typically the loaded length varied between 175m and 250m.

3.3.2 HA Loading

As reported above, HA loading was applied in order to establish the severity of the overstress primarily for those members which did not pass the assessment to BSALL in combination with wind load where this was the m ore critical.

The full HA loading was applied in accordance with BD 37/01 and typically the lane factors were the same as those for the BSALL.

Table 3.3 Summary of Lane Factors in accordance with BD 37/01

Lane 1 Factor	Lane 2 Factor	Lane 3 Factor	Lane 4 Factor
1.0	0.67	0.33	0.33

3.4 HB Loading

In the AIP Clause 4.1.2 it is specified that 45 units of HB loading should be considered and that it should not be considered acting in combination with bridge specific live loading. It has been assumed that the bridge specific live loading refers to the loading stated in clause 5.25 to 5.27 of BD21/01 and it has therefore been assumed that the HB shall be considered acting on its own. It is assumed that vehicles equivalent to HB would cross the bridge with a police escort and the opposite carriageway closed.

HB loading was applied to the bridge but it was found that it was typically less critical than HA for global effects.

3.5 Pedestrian Live Loading

Pedestrian loading was initially taken from the AIP. The footway loading applied in conjunction with the BSALL in the assessment of the stiffening truss was calculated using BD 37/01 Clause 6.5.1(b) with the nominal HA UDL described in the clause replaced with the nominal BSALL. If the above loading contributes to an apparent overstress, the pedestrian loading has been replaced with the Bridge Specific Footway Live Loading (BSFLL) Report produced by WAF in June 2006. FM have not checked the derivation of the BSFLL. See section 6.1.4 for more detail.

3.6 Wind Loading

In the member checks carried out, wind loading was applied in 2 forms:

- Full wind load in accordance with BD 37
- 50 mph wind in combination with BSALL

Full Wind Load

In accordance with the AIP, nominal wind loads applied were calculated from the parameters determined by the University of Glasgow wind tunnel tests. These were used in lieu of the wind loading given by Clause 5.3 of BD37/01.

By inspection, the critical wind direction for bending and shear effects in the truss is 0 degrees yaw, that is, in the transverse direction with respect to the bridge. A direction factor of 1.00 was assumed to apply to this direction.

It was noted in the review of the Glasgow wind tunnel test report that the drag coefficient in the transverse direction for the existing bridge was estimated at 0.2856. The report then compares this figure with the drag coefficient used in the original design of the bridge of 0.2318. The current drag coefficient is therefore approximately 23% higher than the original as the test now takes into consideration the effects of all the parapets, additional walkways etc. The effect of this increased coefficient on the assessment is discussed where it is critical in Section 5 and 6 below.

A check was made to compare the bending moments due to wind loading with those results stated in the ICE Proceedings. The results from the Aecom LUSAS model for maximum transverse bending moments in the deck (calculated from chord forces) are compared here to those given in the ICE proceedings for the original design in the table below. The Proceedings do not fully explain how the bending moment was estimated. Aecom has estimated the bending moment based on the average chord force in both chords for the transverse bending effects only, omitting vertical bending and torsion. Therefore the comparison of results below are indicative but it serves to illustrate that there is a reasonable comparison.

Table 3.4 Comparison of Wind Moments

	Transverse Moment due to Wind	
	Aecom LUSAS Model	**ICE Proceedings (Fig. 2.44)**
Side Span	249,120	240,000 *(340×10^6 lb.ft)*
Main Span)	500,070	460,000 *(175×10^6 lb.ft)*

The above indicates that there is a good agreement on the side span and a reasonable agreement (within 9%) on the main span.

Differences could be explained by the following:

- The transverse force drag coefficient of 0.2856 used by Faber Maunsell was obtained from the University of Glasgow wind tunnel testing report. The University of Glasgow report states that wind tunnel tests carried out for the original design found an equivalent drag coefficient of 0.2316 which is a difference of 23%.
- Wind speeds calculated under BD37/88 are 52.5m/s (117.4mph) for the side spans and 49.8m/s (111.4mph) for the main span. This results in wind pressures 14% and 2.6% respectively higher than would be calculated using the 110mph wind speed used in the ICE Proceedings section 2.126.

50 mph wind load

The AIP states that a maximum wind speed of 50 mph shall be taken in combination with BSALL. The application of using 50 mph has been reviewed by FM against FETA's operating procedure:

The following is a summary of the Strong Wind Procedures set up by FETA:

Table 3.5 Summary of FETA Strong Wind Procedure

Wind Speed	Restrictions
Winds with gusts **exceeding 35 mph** with a rising wind pattern	40mph speed limit on bridge
Winds with gusts **exceeding 45 mph** with a rising wind pattern	Bridge closed to double-decked buses
Winds with gusts **exceeding 50 mph** with a rising wind pattern	Bridge closed to high-sided and wind susceptible vehicles including HGVs, vans, bikes
Winds with gusts **exceeding 65 mph** with a rising wind pattern	Bridge closed to all vehicles except cars 30 mph speed limit on bridge
Winds with gusts **exceeding 80 mph** with a rising wind pattern	Bridge closed to all traffic

Table 3.5 indicates that a pro-active approach to traffic management is in place and that at wind speeds exceeding 50 mph, the bridge is closed to HGVs and vans. Aecom considers that it is reasonable to consider a load combination of BSALL + 50 mph wind. If the wind exceeds 50 mph, the loading reduces to that of cars only. The resulting vehicular load would be so substantially less than BSALL that it is not significant.

In the course of carrying out the assessment, it became apparent that overstresses due to dead plus wind were significant over a large extent of the structure. We therefore reviewed the loading and the design codes used in deriving the loads. The AIP states that the application of the wind loading will be based on BD37/88 rather than BD 37/01 as the former allows for loaded lengths greater than 400m. We have compared the results between BD 37/88 and BD 37/01 for the areas of apparent overstress.

In BD 37/88, the maximum wind gust speed V_c is calculated from

$V_c = v\ K_1 S_1 S_2$

in which v is the mean hourly wind speed and this is modified by the other factors including S_2 which is a gust factor based on height above ground and horizontal wind loaded length.

Without live load, on the side span based on a loaded length of 408 m, V_c is 52.5 m/s

Without live load on the main span, based on a loaded length of 1000 m V_c is 49.8 m/s

Transverse and vertical forces arising from wind loads are calculated using the formulae $X = \frac{1}{2}\rho V^2 BWC_x$ and $Z = \frac{1}{2}\rho V^2 BWC_z$ respectively. Torque on the truss is calculated by $M_y = \frac{1}{2}\rho V^2 B^2 WC_{My}$. These are taken from the University of Glasgow wind tunnel tests with V_c substituted as V.

In BD 37/01, the maximum design wind gust speed is calculated from

$V_d = S_g\ V_s$

In which V_s is the site hourly mean wind speed and S_g is the gust factor.

BD 37/01 was developed in a similar form to the wind load code for buildings, BS 6399 Part 2 which was first published in 1995. In the above equation Vs is modified by an altitude and direction factor. For the direction factor the most conservative value of 1.0 for wind blowing from a south westerly direction. The gust factor is modified by a bridge factor (for loaded lengths up to 400m) together with terrain, fetch correction and topography factors, and is hence a more rigorous approach. Throughout we have used conservative values for these factors.

For the maximum loaded length of 400m dealt with in BD 37/01, V_d is estimated to be 46.3 m/s. This value has been substituted into the equations produced by the University of Glasgow.

We have compared the results obtained from BD 37/88 and BD 37/01 and have concluded that typically the wind pressure applied in BD37/88 is higher than BD 37/01. For the side span, wind pressures would be approximately 28% higher and for the main span wind pressures would be approximately 15% higher. The effect of reducing the wind pressure to BD 37/01 will be discussed in the relevant sections below for those members which fail the assessment at Stage One under Dead + Wind or Dead + BSALL + 50mph Wind.

Several discussions have been held with WAF regarding the use of BD37/01 for derivation of wind forces. We recognise that BD37/01 only covers loaded lengths up to 400m and it may be argued that it can not be applied to the Forth Road Bridge. BD37/01 states that for bridges outside the 400m loaded length limit consideration should be given to the effects of dynamic response due to turbulence taking due account of lateral, vertical and torsional effects. As explained above, the wind pressures are calculated using the formulas from BD37 but using the lateral, vertical and torsional coefficients from the wind tunnel tests. Therefore, whilst the use of BD37/01 cannot be fully justified, there is a reasonable argument for its use to explain why there is no visible distress in the elements critically affected by wind.

Therefore, we have provided further advice for those members which are critically affected by wind. Furthermore, we have applied a conservative approach by using the coefficients for loaded length of 400m for loaded lengths greater than 400m.

4 Global Modelling

4.1 General

To obtain the member forces due to various transient vehicle load combinations, a 3D LUSAS finite-element model was constructed to suit the handling the geometric nonlinearity of the structure. All transient combination loadings are analysed concurrent with the permanent loads calculated in the existing structure under nominal dead loading.

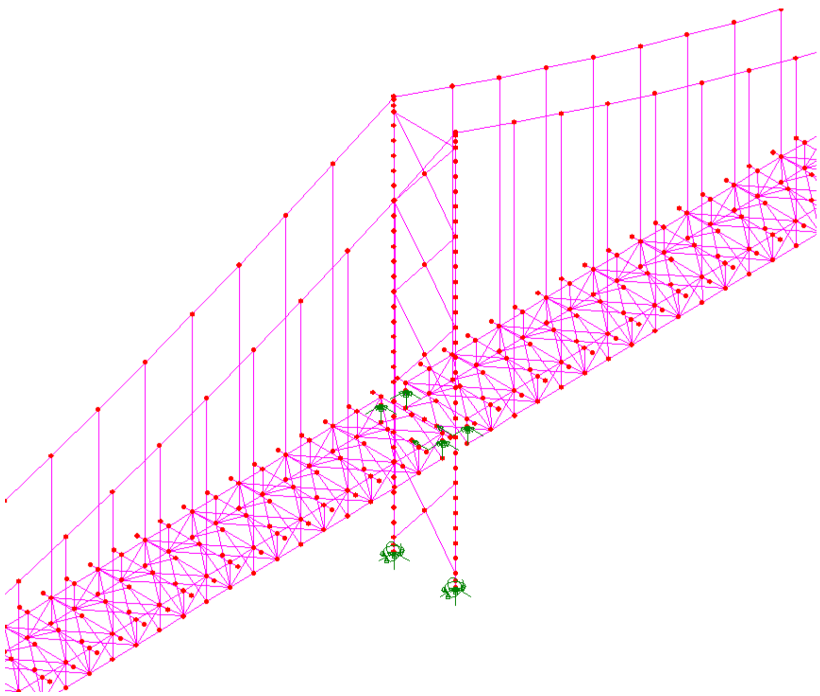


Figure 4.1 - Graphical View of 3D Global Model of Forth Road Bridge.

The actual dead load of the bridge is critical to the profile of the main cable. Not only does the suspended permanent load have a direct effect on the geometry of the cable, it also determines the actual pre-tension in the cables.

The bridge would have originally been designed to BS153 using unfactored loads. The behaviour of a suspension bridge is non-linear. Using current design codes bridges are designed to ultimate as well as serviceability limit states using loads multiplied by factors and compared generally against yield stresses divided by factors. This presents a complication when applying factored dead loads to suspension bridges as there will be a resulting dead load deflection and hence bending moments in the stiffening truss. The bridge would have been designed in accordance with best practice such that there would be zero bending moments in the truss. The WAF AIP clause 4.3.2 suggests that a load factor, γ_{fl}, of 1.08 for the dead load of the concrete and 1.20 for the superimposed dead load surfacing at ULS. Aecom considers that this is conservative as it introduces a dead load deflection in the truss. However, the effect is relatively small and, in the interest of comparing results, these dead load factors have been adopted. However, where overstresses occur this will be a contributory factor.

4.2 Deck Truss Joint Fixity

The initial global analysis was carried out assuming that all joints were fully fixed. However, as the assessment proceeded it became apparent that there were several theoretical overstresses in the main members in the longitudinal stiffening truss. Research was therefore carried out to determine if the deck truss could be re-assessed assuming pin jointed connections.

In BD 56 Clause 12.2.1 it states that “In trusses with stiff joints stresses due to axial deformation of members may be ignored” at ULS. In Clause 12.3 it states that trusses can be analysed either assuming that the joints are pin-jointed or if they are rigid. The SCI commentary on BS5400-3:2000 reinforces these points and clarifies that the stresses referred to in Clause 12.2.1 refer to bending and shear stresses. It further adds that: “At the ultimate limit state, redistribution of flexural stresses in the region of stiff joints will occur and hence such stresses will not affect the ultimate capacity of the member and may be ignored.

The issue of secondary effects is discussed in the ICE Proceedings section 2.133. It states that “The secondary stresses due to fixity at the joints were calculated and it was found that they nowhere exceeded 20% of the allowable axial stress.” We consider that this is a reference to BS153 Part 3B Clause 21e which states the following:

> “Where, for any combination of loading, all secondary stresses due to eccentricity of connections and off-joint loading generally, and to the elastic deformation of the structure combined with the rigidity of the joints, are computed and combined with the co-existent axial stresses in accordance with Clause 23, the appropriate allowable working stresses shall be increased by 20 per cent”.

In section 2.97 of the ICE Proceedings, it states the following: “When calculated secondary stresses were combined with primary stress the allowable stress was increased by 20%”.

We therefore conclude that the secondary stresses were ignored or dealt with as described in section 2.97 of the ICE Proceedings in the original design. This is further backed up by table 2.15 and figure 2.46 of the ICE Proceedings which refer to comparisons of actual and allowable axial forces in the top chord.

On this basis, further analysis was carried out assuming pin joints and it was determined that there was an insignificant change in the axial loads in the top and bottom chords. There was an increase in the axial force in the diagonals but, as will be seen later, under Stage One load combinations with BSALL loading or Dead + Wind, the diagonals are well within their capacity and this increase was ignored.

The assessment was therefore carried out in 2 stages. Initially the analysis and assessment was carried out assuming that the joints were fixed. For the elements which had theoretical overstresses, the process was repeated assuming that the joints are pinned.

5 Stiffening Truss Assessment

5.1 Introduction

The stiffening truss assessment was carried out using the live loading described in Section 3 of this report. The elements were assessed in 2 stages to the following load combinations. The footway live load effects were added to each relevant combination.

Stage One

Combination 1: Dead + BSALL

Combination 2: Dead + BSALL + 50mph wind

Combination 3: DL + Full Wind

The bridge was then assessed for the effects of full HA and HB loadings determined in accordance with the AIP using the following load combinations

Stage Two

Combination 4: DL + HA

Combination 5: DL + HA + Full Wind

Combination 6: DL + HB (less critical than HA)

5.2 Top and Bottom Chords

In order to establish the maximum chord forces in the chords it was first necessary to establish the critical loaded lengths for the top and bottom chords. The ICE Proceedings described the process followed in the original design in which loaded lengths varying between 410ft (125m) and 820ft(250m) were applied at the 1/8th point on the main span. It was then established that a loaded length of 615ft (187m) was the most critical length and this length was applied at sufficient locations along the length of the bridge.

We carried out a similar exercise to establish the critical loaded lengths. The behaviour of the structure is highly non-linear and it was observed that the most adverse effects did not occur when the model was loaded based on influence lines derived from a linear solution. This meant that application of the most adverse live loading was best established by trial and error.

For bending in the main span, knife edge loads produce the greatest moments in the truss under their point of application, and uniformly distributed loads produce maximum moments at the centre of the loaded length. This concurs with the analysis by the bridges' designer.

For this investigation, axial compression and tension in the chords was taken as a proxy for bending in the truss. Lengths of 100m, 150m, 175m, 200m and 250m HA loads were calculated and combined with knife-edge load at the point under investigation as well as matching lengths of footway loading. Ultimate load factors were applied to live loads. The loaded lengths were applied symmetrically with respect to the longitudinal centreline of the bridge with the greatest lane factors applied in the lanes adjacent to the side of the truss under investigation. A one-eighth point, a quarter point and the mid point on the main span are checked for each of the above load cases in order to empirically determine a reasonably accurate lower bound solution.

We established that the critical loaded lengths were as follows:

- BSALL Side Span – 175m
- BSALL Main Span – 175m
- HA Side Span – 250m
- HA Main Span – 175

Using the above loaded lengths the loads were placed to investigate the chord forces at each panel point.

As part of the calibration of the model, analysis of the model with loading to BS153 has been carried out for calibration of the model and comparison to the bridge designers' assumptions.

The model was loaded with the live loading intensities and lane factors taken from BS 153 and it was checked against the data provided on page 63 of the ICE Proceedings.

The result of this analysis is that at panel point 60, axial force in top chords = 7398kN (C) and 7213kN (C), Axial force in bottom chords = 7597kN (T) and 7511kN (T).

Truss bending moment (approx) = (7398+7213+7597+7511)/2 x 8.382m = 62844kN.m = 91,833,719 lb.ft in 2 trusses.

This is almost identical to the value of 90.79×10^6 lb.ft tabulated by the designer.

5.2.1 Top Chord Main Members

The summary of the results of the Top Chord is presented in Figures 5.1 and 5.2 The utilisations of the members have been expressed as Overstress Indices similar to the approach taken by WAF to simplify comparison of results.

It can be seen that under DL + BSALL the maximum overstress indices are all less than 1.0.

Under DL + Wind, the maximum overstress indices are also less than 1.0. .

Under DL + BSALL + Wind it can be seen that the maximum overstress indices are less than 1.0.

Although there are no overstresses in the assessment of the top chord, we carried out analysis of the HA load cases. It can be seen that, under DL + HA the maximum overstress indices are 1.3 in the side span and 1.22 in the main span. Under DL + HA + Wind, the maximum overstress indices are approximately 1.48 in the side span and 1.83 in the main span.

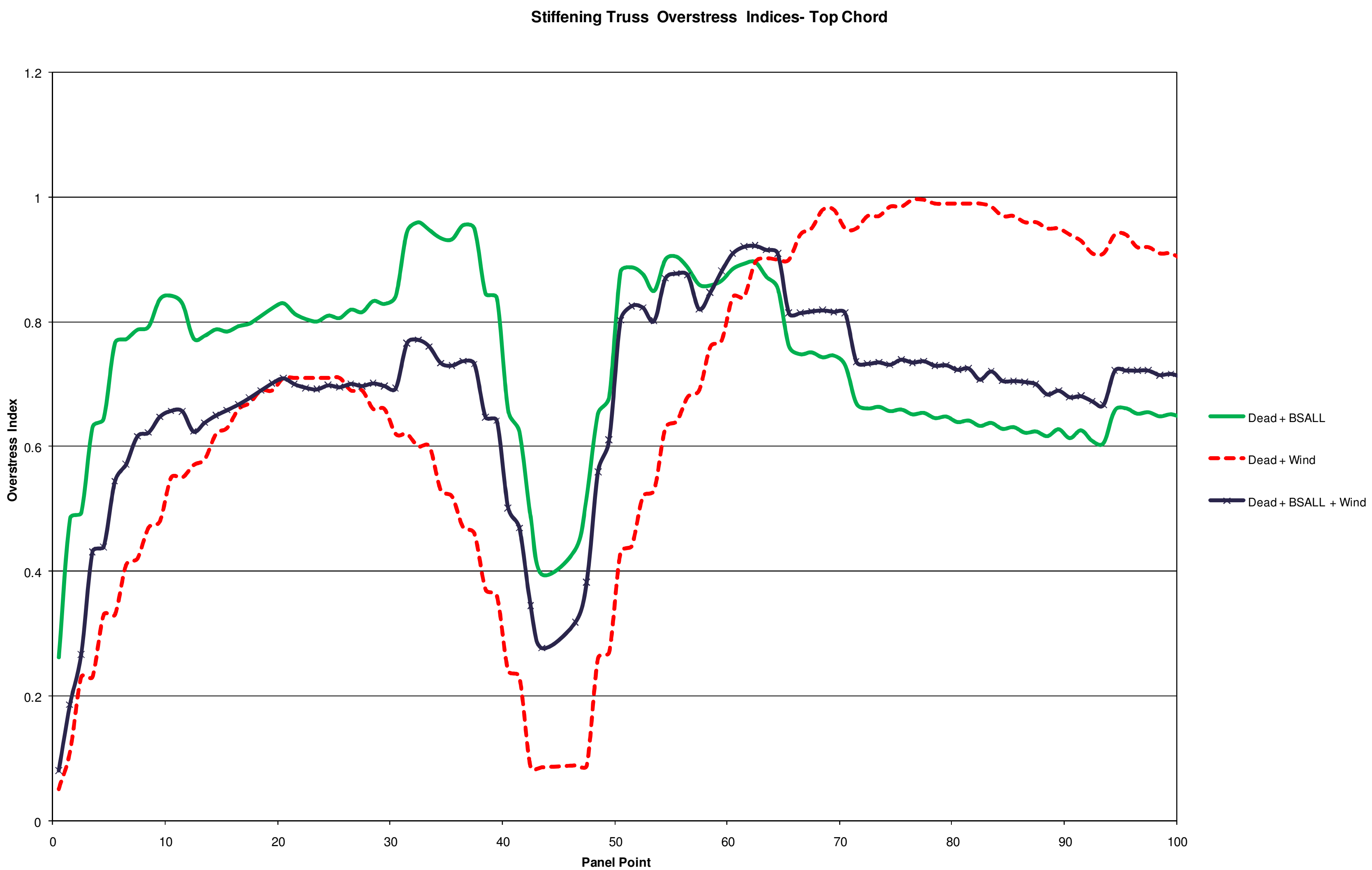


Figure 5.1 **Summary of Top Chord Overstress Indices (BSALL and Dead + Wind Load Combinations)**

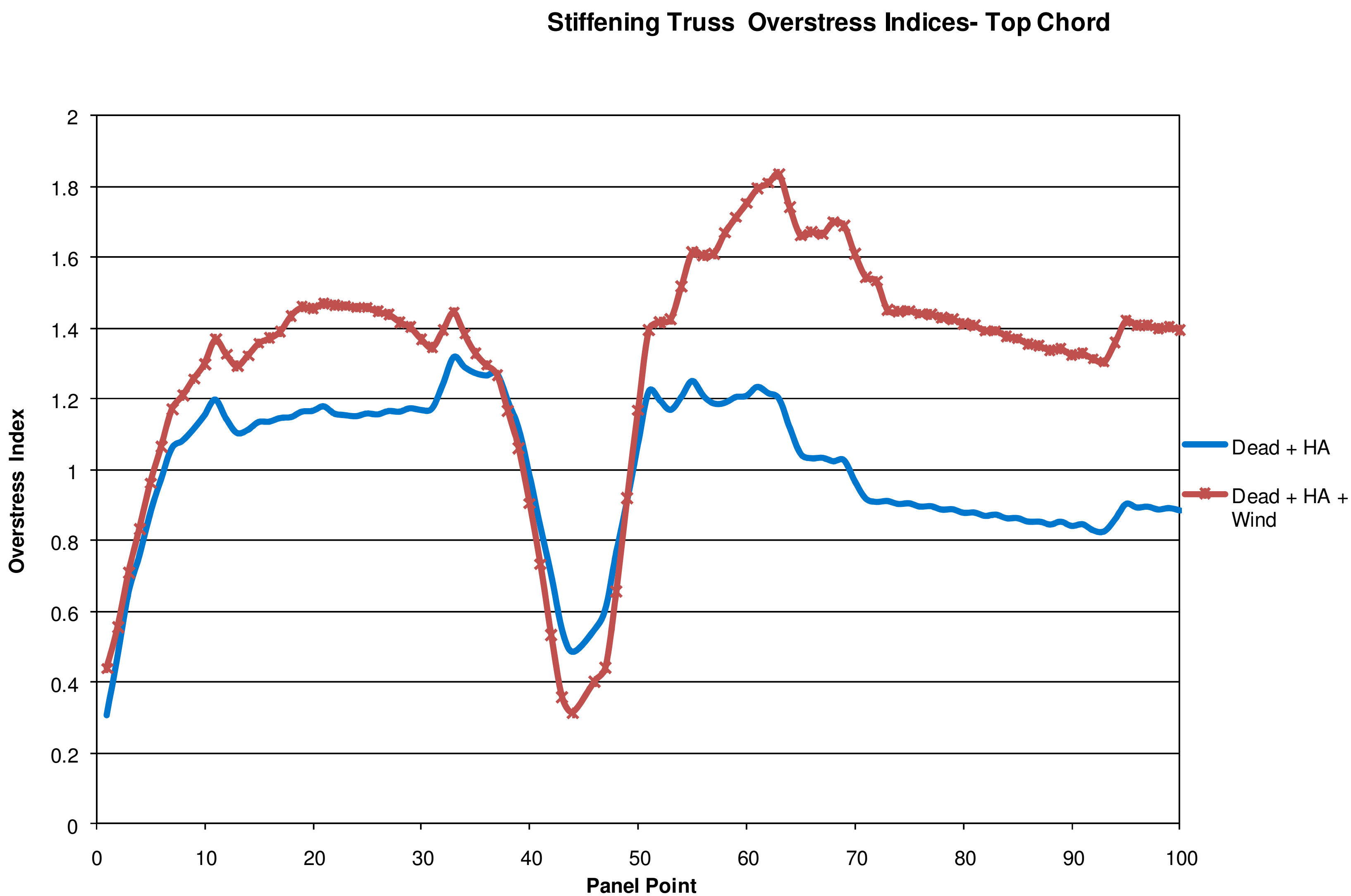


Figure 5.2 Summary of Top Chord Overstress Indices (HA Load Combinations)

5.2.2 *Top Chord Connections*

Typically, the bolts used in the structure are Roberts bolts which are friction grip bolts with a waisted shank. The bolts have a diameter of 7/8 inch measured at the threads in 15/16 inch holes. The waisted shank diameter has been taken as ¾ inch.

The detailed original fabrication drawings and the ICE Proceedings (Reference 2.2) indicate that the steelwork was originally grit blasted after fabrication and sprayed with metallic zinc. Neither the drawings nor the ICE Proceedings specify the preparation at the connection surfaces. However, the Resident Engineer's Report (Reference 2.3) records the following:

> "All external contact surfaces were zinc sprayed, whilst the internal contact surfaces were wire-brushed only. It was found that on the majority of chord splice plates and cross girder gussets, which were painted on site, there were a number of narrow paint runs on the contact faces due to paint running through the bolt holes. This paint was removed by hand scraping prior to erection.
>
> In a few isolated cases where contact faces had been painted in error, the paint was removed by sand blasting which satisfactorily stripped the paint but left the zinc coating."

Using the information above, in accordance with BD56, the slip factor, μ, for the external surfaces sprayed with zinc is 0.40. This figure has been used for all bolted connections.

According to the RE's report on construction, the bolts were intended to be stressed to 26 tons. The shank diameter is ¾ inch, with an area of 0.442 sq in; hence installation stress is 58.9 tons/sq in. The steel used is En16V, with a minimum tensile strength of 65 tons/sq in, i.e. the tightening tension is 58.9/65 which is equivalent to 91% x UTS

The assessment has been carried out using. proof loads of 26 tons and a friction factor of 0.4. However, the friction factor has been reduced by 10% in accordance with BD56 14.5.4.4 This reduction factor is applied to higher grade bolts with high proof load

Under Dead + BSALL and Dead + BSALL + Wind all overstress indices for both SLS and ULS are less than 1.0.

Under Dead + Wind, the maximum overstress index is 1.33 at SLS at panel point 78 in the main span. As the axial force in the top chord is fairly constant at this location in the truss, the apparent overstress applies to a substantial length of the main span. In the side span, the maximum overstress index is 0.91. As explained in Section 3.4 above, the wind load pressure in the main span when calculated in accordance with BD 37/88 is approximately 13% greater than in BD 37/01. If the 10% reduction for higher grade bolts is relaxed, the maximum overstress in the main span reduces to 1.07.

Under Dead + HA, the maximum overstress occurs in the main span is 1.08.

Under Dead + HA + Wind, the maximum overstress occurs in the main span at panel point 71 and is 1.99. The maximum overstress in the side span at panel point 22 is 1.49.

Table 5.1 Summary of Maximum Top Chord Overstress Indices

Top Chord Element	Load case	Limit State	Overstress Indices	Panel Point
Main Plates	Dead + BSALL	ULS	0.96	33
	Dead + Wind	ULS	0.99	77
	Dead + BSALL +50mph Wind	ULS	0.92	61
	Dead + HA	ULS	1.3	33
	Dead + HA + Wind	ULS	1.83	63
Bolts	Dead + BSALL	SLS	<1.0	
	Dead + Wind (BD37/88)	SLS	0.91	22 (Side Span)
	Dead + Wind (BD37/88)	SLS	1.33	78 (Main Span)
	Dead + BSALL +50mph Wind	SLS	<1.0	
	Dead + HA	SLS	1.08	61
	Dead + HA + Wind (Main Span)	SLS	1.99	71
	Dead + HA + Wind (Side Span)	SLS	1.49	22

Table 5.2 Summary of extent of Overstressed Bolts in Top Chord

	Load Case	Extents of Overstressed Panels (all SLS)
Top Chord Bolts	Dead + Wind	Panel 62-101
	Dead + BSALL	-
	Dead + BSALL + Wind	-
	Dead + HA	Panel 54-79
	Dead + HA + Wind	Panel 10-36, 50-101

5.2.3 *Top Chord Splice Plates*

The Top Chord splice plate tensile capacities are reduced by the presence of bolt holes. If the requirements of BD5 6 are applied strictly there are significant sections which are theoretically significantly overstressed. As no significant distress has been reported we have looked at each of the assumptions to determine if they can be relaxed.

Typically the top and bottom chords are spliced using inner and outer splice plates for the flanges and webs. In some areas the inner web splice is omitted. According to BD56 clause 14.4.3.2 the effective area of outer plies shall be reduced by 0.8 for splices made by HSFG bolts acting in friction. It could be argued that at ULS the bolts are acting in bearing and shear and this clause does not apply.

The requirements of BD56 clause 14.4.1 are such that where practicable the cover material should be provided to communicate the proportional load in each part of the section. On this basis, the inner splice plates indicate the most severe overstresses. However, it could be argued that in the ultimate condition, the bolts will slip at different rates and the loads will be redistributed such that an average stress is applied to each of the splice plates. By following this method the severity and extent of the overstress is reduced.

Table 5.3 Summary of Maximum Top Chord Splice Plate Overstress Indices (Assuming 0.8 factor on area and average stress on all splice plates)

Top Chord Splice Plate Location	Load case	Limit State	Overstress Indices	Panel Point	Extents of Overstressed Panels (all ULS)
Side Span	Dead + Wind	ULS	1.50	22	10-36
Main Span	Dead + Wind	ULS	1.55	78	50-100

Table 5.4 Summary of Maximum Top Chord Splice Plate Overstress Indices (Assuming 1.0 factor on area and average stress on all splice plates)

Top Chord Splice Plate Location	Load case	Limit State	Overstress Indices	Panel Point	Extents of Overstressed Panels (all ULS)
Side Span	Dead + Wind	ULS	1.20	22	14-32
Main Span	Dead + Wind	ULS	1.24	78	64-100

These theoretical overstresses will reduce even further if wind to BD37/01 is applied. Applying these reductions to Table 5.4, the side span OI will reduce by 28% from 1.20 to 0.94. The main span OI will reduce by 15% from 1.24 to 1.08.

In summary, if the most optimistic set of assumptions is used, there is still a theoretical overstress.

5.2.4 Bottom Chord Main Members

The summary of the results for the Bottom Chord is presented in Figures 5.3 and 5.4.

Under Dead + BSALL it can be seen that the maximum overstress indices are all less than 1.0..

Under Dead + BSALL + 50 mph Wind the maximum overstress indices are all less than 1.0..

Under Dead and Wind loads it can be seen that there is an apparent failure of the bottom chords with maximum overstress indices of approximately 1.1 in the side span at panel point 24. The main span shows a theoretical marginal overstress of approximately 1.15. This is based on the drag coefficients obtained from the Glasgow University wind tunnel test. The bridge would have been designed in accordance with the best practice at that time and the bridge would have been designed safely. There appear to be no signs of distress to the members. A possible reason for this apparent failure is due to the increased drag coefficient. The critical members under Dead + Wind have been re-analysed to BD 37/01 and the maximum overstress indices fall below 1.0..

Further checks were carried at SLS assuming that no slip had occurred and the moments arising from the fixity of the joints were included. It was concluded that these critical members when analysed in this more rigorous method were adequate.

Under Dead + HA the maximum overstress indices are approximately 1.15 in the side span at panel point 20 and in the main span at panel point 56.

Under Dead + HA + Wind the maximum overstress indices are approximately 1.28 at panel point 22 in the side span and 1.55 at panel point 64 in the main span.

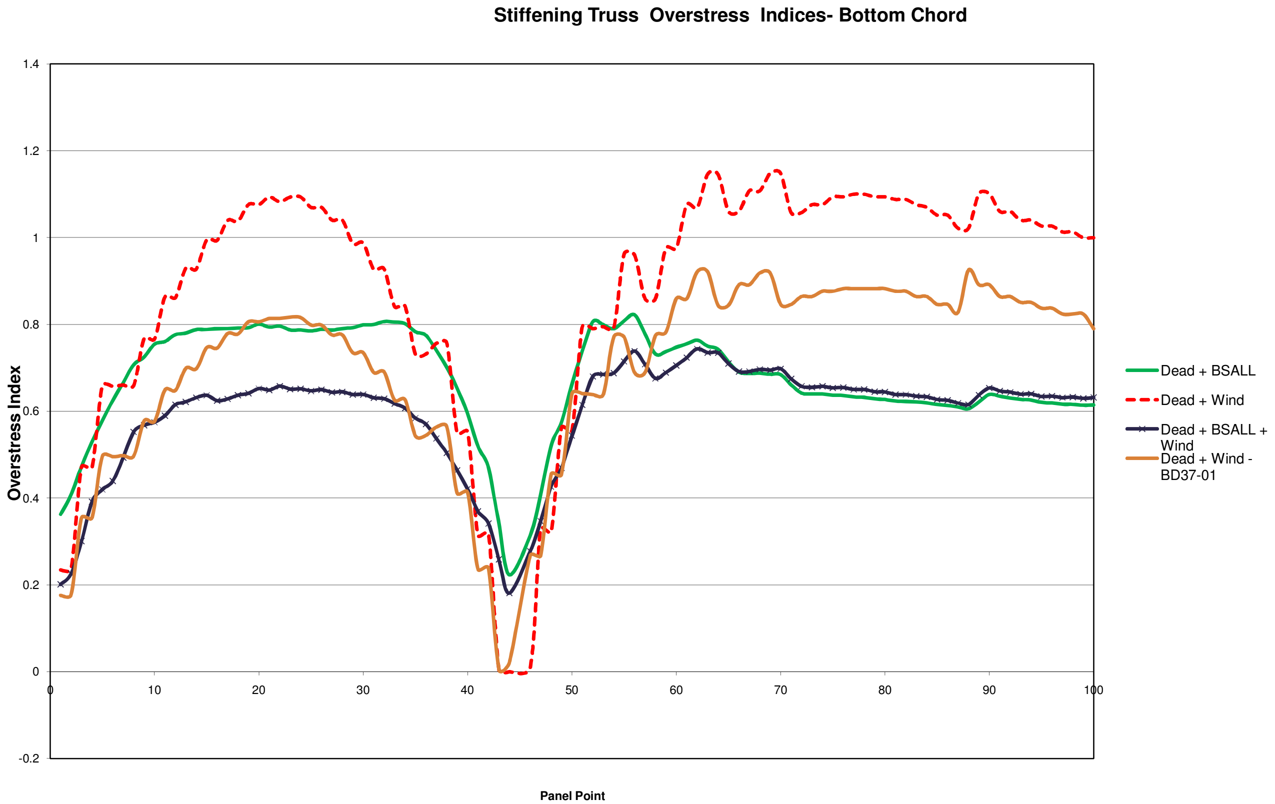


Figure 5.3 Summary of Bottom Chord Overstress Indices (BSALL and Dead + Wind Load Combinations)

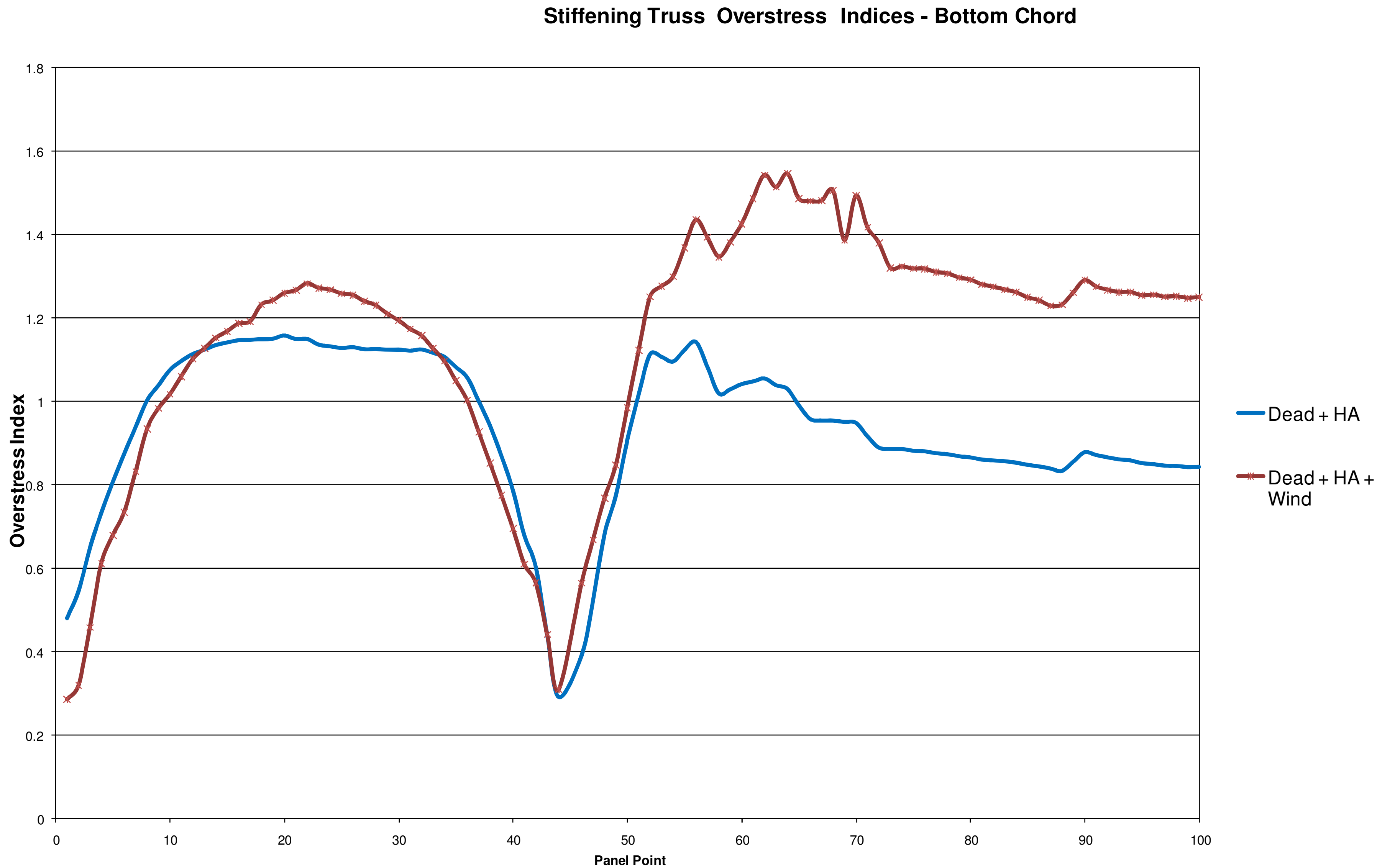


Figure 5.4 Summary of Bottom Chord Overstress Indices (HA Load Combinations)

5.2.5 *Bottom Chord Connections*

Under Dead + BSALL all overstress indices for both SLS and ULS are less than 1.0.

Under Dead + Wind, the maximum overstress index are less than 1.0.

Under Dead + HA + Wind, the maximum overstress occurs in the main span at panel point 68 and is 1.99 at SLS. The maximum overstress in the side span at panel point 22 is 1.28.

Table 5.5 Summary of Maximum Bottom Chord Overstress Indices

Bottom Chord Element	Load case	Limit State	Overstress Indices	Panel Point
Main Plates	Dead + BSALL	ULS	0.82	56
	Dead + Wind (BD37/88) Dead + Wind (BD37/01)	ULS	1.15 (0.92)	64
	Dead + BSALL +50mph Wind	ULS	0.74	62
	Dead + HA	ULS	1.15	20
	Dead + HA + Wind	ULS	1.55	62
Bolts	Dead + BSALL	SLS	Not Critical	
	Dead + Wind (BD37/88)	SLS	0.95	69
	Dead + BSALL +50mph Wind	SLS	Not Critical	
	Dead + HA	SLS	Not Critical	
	Dead + HA + Wind	SLS	1.99	68

Table 5.6 Summary of extent of Overstressed Bolts in Bottom Chord

	Load Case	Extents of Overstressed Panels (all SLS)
Bottom Chord Bolts	Dead + Wind	-
	Dead + BSALL	-
	Dead + BSALL + Wind	-
	Dead + HA	Panel 56-85
	Dead + HA + Wind	Panel 9-36, 51-101

5.2.6 Bottom Chord Splice Plates

The Bottom Chord splice plates were initially also found to have overstresses but not as severe as the top chord. For the same reasons stated in section 5.2.3 of this report, the overstresses have been re-examined and the approaches are summarised below:

Table 5.7 Summary of Maximum Bottom Chord Splice Plate Overstress Indices (Assuming 0.8 factor on area and average stress on all splice plates)

Bottom Chord Splice Plate Location	Load case	Limit State	Overstress Indices	Panel Point	Extents of Overstressed Panels (all ULS)
Bottom Chord Splice Plates	Dead + Wind	ULS	0.61	23	
	Dead + Wind	ULS	1.14	77	66-101

Table 5.8 Summary of extent of Overstressed Splice Plates in Bottom Chord (Assuming 1.0 factor on area and average stress on all splice plates)

Bottom Chord Splice Plate Location	Load case	Limit State	Overstress Indices	Panel Point	Extents of Overstressed Panels (all ULS)
Bottom Chord Splice Plates	Dead + Wind	ULS	0.49	23	
	Dead + Wind	ULS	0.91	77	

5.3 Diagonal and Vertical Members

The assessment of the vertical members is reported in Section 6 – Cross Girders.

The critical loaded lengths for the diagonal members were obtained in a similar manner to the main chords by trial and error.

It was found that knife-edge loads, represented as point loads in the model, cause the greatest axial force in the member when they are applied two panels from a given diagonal.

Uniformly distributed BSALL loading reduces in intensity as its loaded length increases. To determine the critical loaded length for BSALL, lengths of 100m, 150m, 200m, 250m and 300m uniformly distributed load were calculated and combined with knife-edge load along with matching lengths of footway loading. Ultimate load factors were applied to live loads. The loaded lengths were applied symmetrically with respect to the longitudinal centreline of the bridge with the greatest lane factors applied in the lanes adjacent to the side of the truss under investigation.

Diagonals in the truss adjacent to the main tower, at a quarter point and at the mid point on the main span were checked for each of the above load cases in order to determine the critical loaded length. The following loaded lengths were established:

- BSALL Side Span – 150m

- BSALL Main Span – 150m
- HA Side Span – 250m
- HA Main Span – 250m

The summary of the results for the diagonals is presented in Figure 5.5

Under Dead + BSALL the diagonals have overstress indices less than 1.0.

Under Dead + Wind, the overstress indices are all less than 1.0

Under Dead + HA the overstress indices are generally less than 1.0 but there are overstresses adjacent to the side and main towers with a maximum index of 1.17.

5.3.1 Diagonal Member Connections

Under Dead + BSALL, all overstress indices are less than 1.0

Under Dead + Wind all overstress indices are less than 1.0

Under Dead + HA, the maximum overstress index is between panel point 0 and 1 is 1.02 at SLS.

Table 5.9 Summary of Maximum Diagonal Member Overstress Indices

Diagonal Element	Load case	Limit State	Overstress Indices	Panel Point
Main Plates	Dead + BSALL	ULS	0.95	11-12
	Dead + Wind (BD37/88)	ULS	0.36	50-51
	Dead + BSALL +50mph Wind	ULS	Not critical	
	Dead + HA	ULS	1.17	46-47
	Dead + HA + Wind	ULS	Not critical	
Bolts	Dead + BSALL	SLS	Less than 1.0	
	Dead + Wind (BD37/88) Dead + Wind (BD37/01)	SLS	Less than 1.0	
	Dead + BSALL +50mph Wind	SLS		
	Dead + HA	SLS	1.02 (0.89)	0-1
	Dead + HA + Wind	SLS		

Table 5.10 Summary of extent of Overstressed Bolts in Diagonals

	Load Case	Extents of Overstressed Panels (all SLS)
Diagonal Member Bolts	Dead + Wind	-
	Dead + BSALL	-
	Dead + BSALL + Wind	-
	Dead + HA	Panel 0-1, Panel 46-47
	Dead + HA + Wind	-

5.3.2 Diagonal Member Gusset Plates

Table 5.11 Summary of Maximum Diagonal Member Gusset Plate Overstress Indices

Gusset Plate Location	Load case	Limit State	Overstress Indices	Panel Point
Side Span	Dead + BSALL	ULS	0.79	2-3
	Dead + HA	ULS	0.98	2-3
Main Span	Dead + Wind	ULS	0.25	47-48

Table 5.12 Summary of extent of Overstressed Gusset Plates in Diagonals

	Load Case	Extents of Overstressed Panels (all ULS)
Diagonal Member Gusset Plates	Dead + Wind	0
	Dead + BSALL	0
	Dead + BSALL + Wind	0
	Dead + HA	0
	Dead + HA + Wind	0

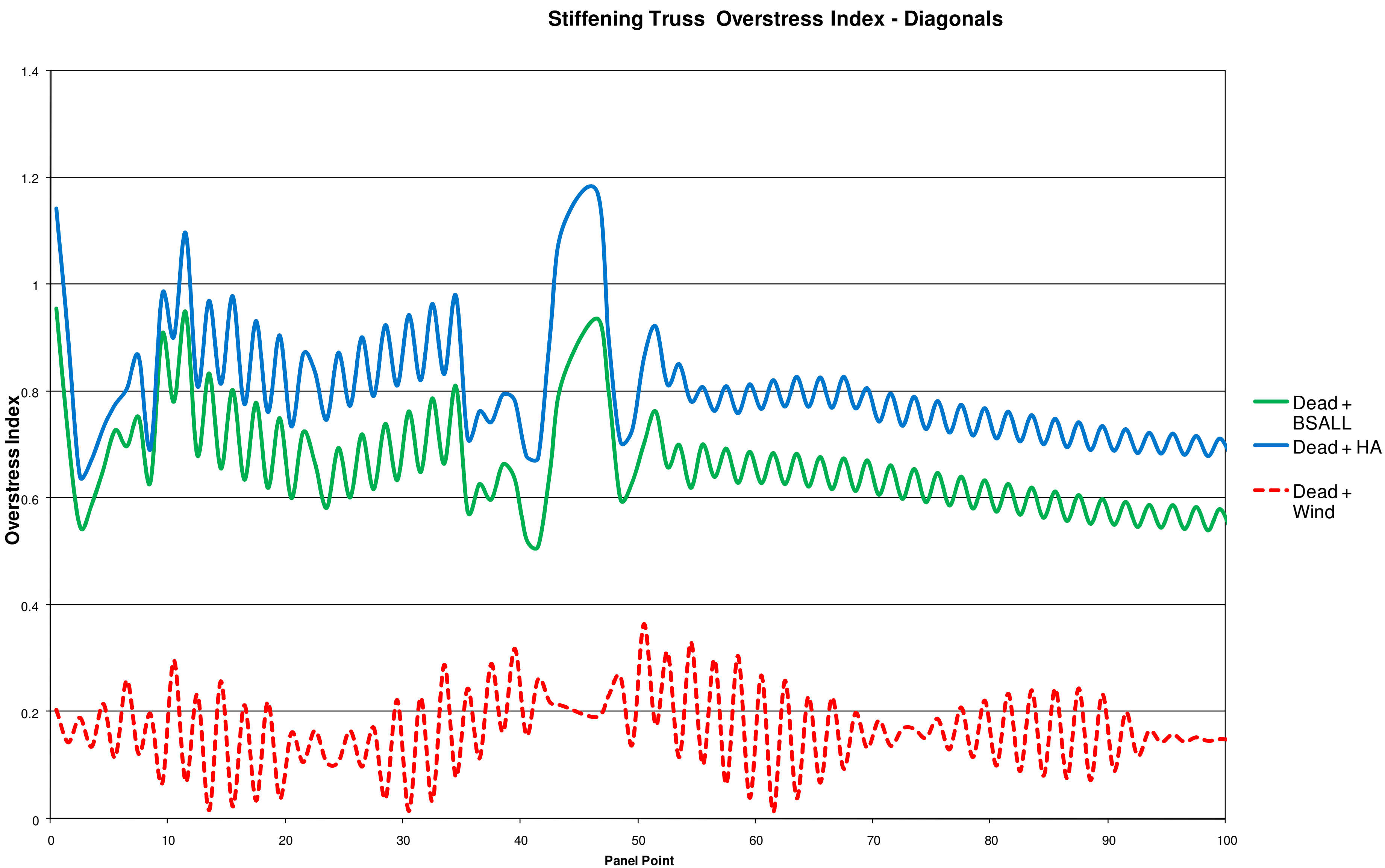


Figure 5.5 Summary of Diagonals Overstress Indices

5.4 Top and Bottom Lateral Bracing

The critical loaded length was derived in a similar method to the main chords and diagonals. The critical loaded length for BSALL and HA for the main span and side span was determined to be 175m.

5.4.1 Top Lateral Bracing Members

An effective length of 0.7 has been taken in the assessment of the top lateral members. The summary of the overstress indices for the top laterals is shown in Figures 5.6 and 5.7.

Under Dead + BSALL, the overstress indices are less than 1.0.

Under Dead + Wind, the maximum overstress occurs adjacent to the main towers and is 1.55 at panel point 46-47. If the gamma m for compression members is reduced in accordance with BD56, the OI reduces by approximately 10%. If the wind pressures to BD37/01 are applied this is insufficient to reduce the overstress factors below 1.0 , The members closest to the side tower are also overstressed at 1.20.

Under Dead + BSALL + 50mph Wind the overstress indices are less than 1.0.

Under Dead + HA the maximum overstress is 1.54 at panel point 4-5.

Under Dead + HA + Wind, the maximum overstress is 2.45 at panel point 48-49.

5.4.2 Top Lateral Bracing Connections

Under dead + BSALL + wind, the maximum overstress index is 1.01 at SLS at panel point 2-3. For the reasons noted above, this can be reduced to less than 1.0.

Under dead + HA + wind the maximum overstress index is 1.85 at SLS at panel pint 2-3

Under dead + wind the maximum overstress is 1.32 at panel point 47-48. Under the reduction factors noted above, this can be reduced to 1.01.

Table 5.13 Summary of Maximum Top Lateral Overstress Indices

Top Lateral Element	Load case	Limit State	Overstress Indices	Panel Point
Main Plates	Dead + BSALL	ULS	<1.0	
	Dead + Wind (BD37/88)	ULS	1.55	46-47
	Dead + BSALL +50mph Wind	ULS	<1.0	
	Dead + HA	ULS	1.54	4-5
	Dead + HA + Wind	ULS	2.45	48-49
Bolts	Dead + BSALL	SLS		
	Dead + Wind (BD37/88) Dead + Wind (BD37/01)	SLS	1.32 (1.01)	47-48
	Dead + BSALL +50mph Wind	SLS	1.01	2-3
	Dead + HA	SLS		
	Dead + HA + Wind Dead + Wind (BD37/01)	SLS	1.85 (1.53)	2-3

Table 5.14 Summary of extent of Overstressed Bolts in Top Laterals

	Load Case	Extents of Overstressed Panels (all SLS)
Top Lateral Member Bolts	Dead + Wind	Panel 46-52
	Dead + BSALL	-
	Dead + BSALL + Wind	Panel 2-3, Panel 47-51
	Dead + HA	Panel 2-5, Panel 41-43, Panel 47-63
	Dead + HA + Wind	Panel 0-15, Panel 34-43, Panel 47-67

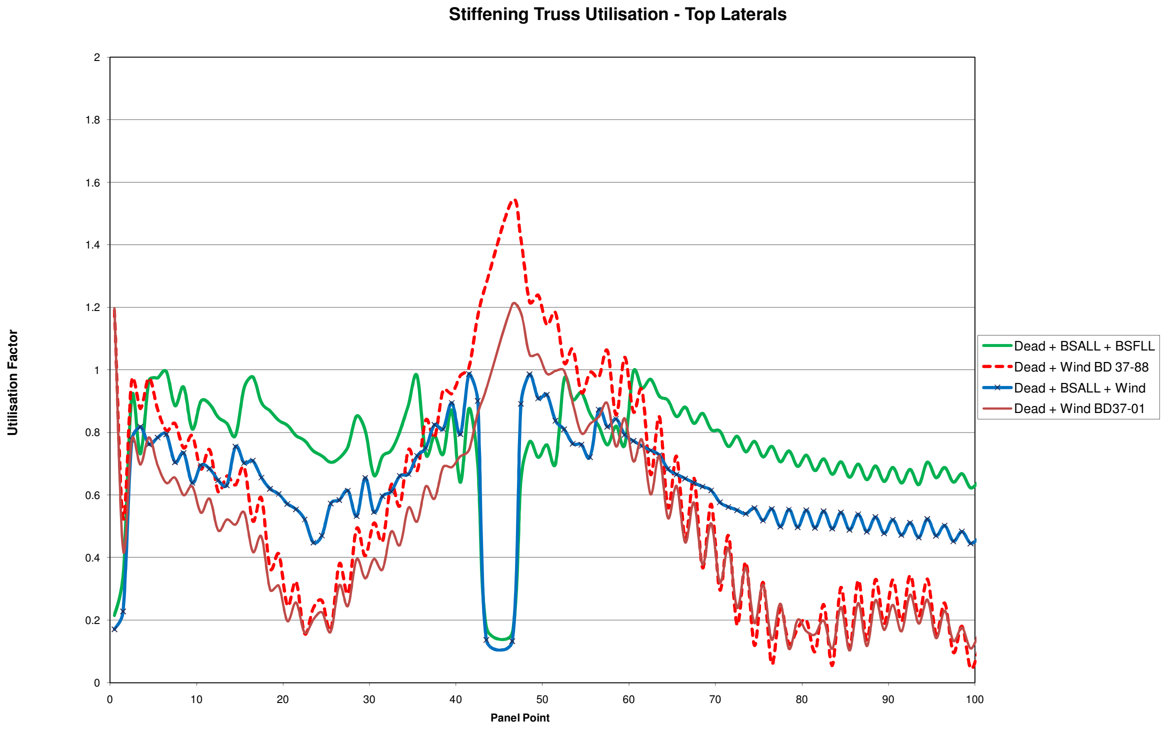


Figure 5.6 Summary of Top Laterals Overstress Indices (BSALL and Dead + Wind Load Combinations)

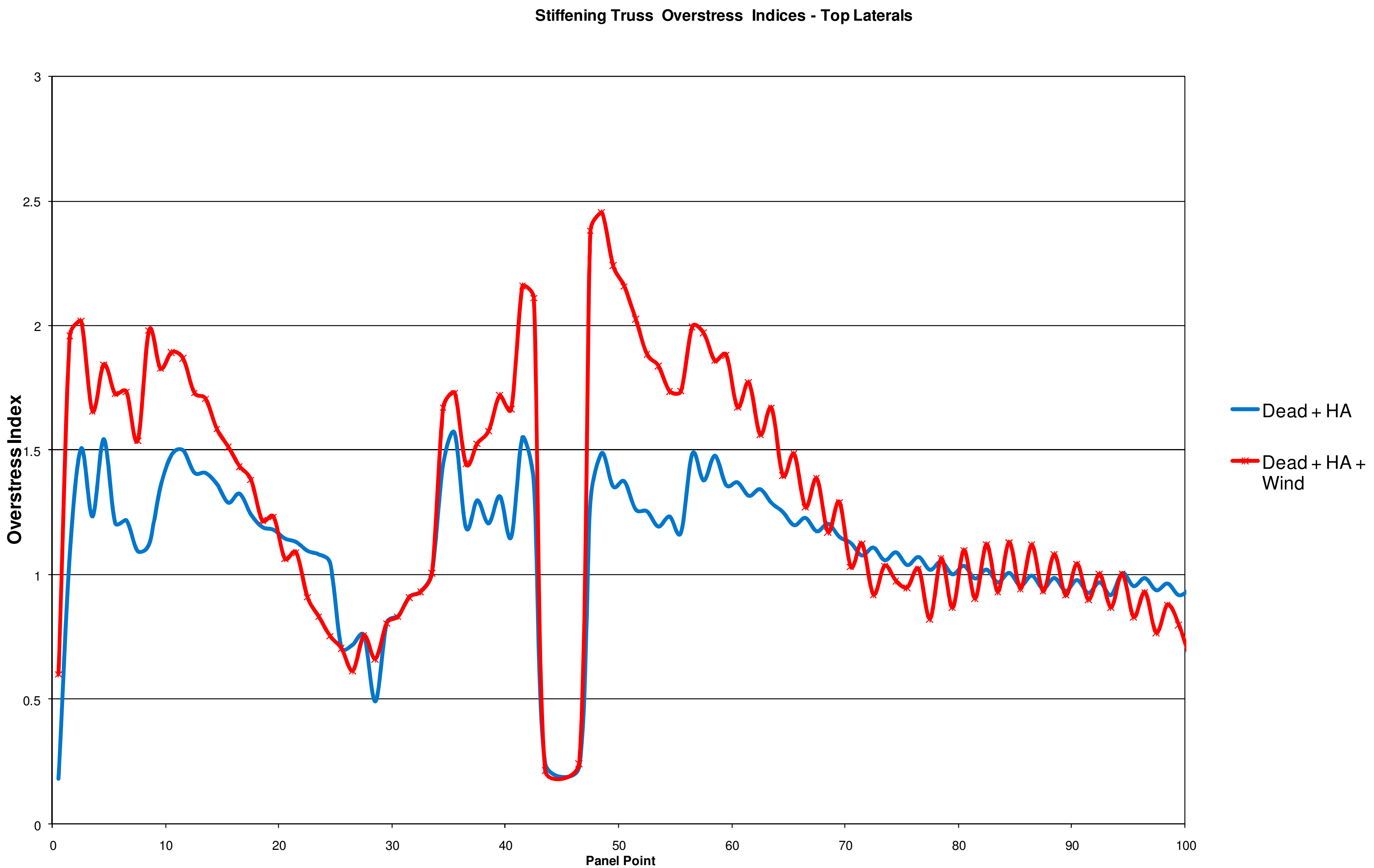


Figure 5.7 **Summary of Top Laterals Overstress Indices (HA Load Combinations)**

5.4.3 Top Lateral Bracing Gusset Plates

Table 5.15 Summary of Maximum Top Lateral Bracing Gusset Plate Overstress Indices

Top Lateral Gusset Plate Location	Load case	Limit State	Overstress Indices	Panel Point
Side Span	Dead + BSALL + 50mph wind	ULS	0.82	2-3
	Dead + HA + Wind	ULS	1.26	2-3
Main Span	Dead + Wind	ULS	1.05	46-47

Table 5.16 Summary of extent of Overstressed Gusset Plates in Top Laterals

	Load Case	Extents of Overstressed Panels (all ULS)
Top Lateral Member Gusset Plates	Dead + Wind	Panel 42-47
	Dead + BSALL	-
	Dead + BSALL + Wind	-
	Dead + HA	-
	Dead + HA + Wind	Panel 2-7, Panel 41-43, Panel 47-55

5.4.4 Bottom Lateral Bracing Members

The summary of results for the Bottom Laterals is shown in Figures 5.8 and 5.9

Under Dead + BSALL the maximum overstress indices are less than 1.0.

Under Dead + Wind, the maximum overstress indices are less than 1.0.

Under Dead + BSALL + 50mph Wind the maximum overstress indices are less than 1.0

Under Dead + HA the maximum overstress is1.32

Under Dead + HA + Wind, the maximum overstress is 1.2

5.4.5 Bottom Lateral Bracing Connections

Under dead + BSALL the maximum overstress is 0.88 at SLS.

Under Dead + HA the maximum overstress is 1.08 at SLS at panel point 2-3.

Under dead + wind the maximum overstress indices are less than 1.0.

Table 5.17 Summary of Maximum Bottom Lateral Overstress Indices

Bottom Lateral Element	Load case	Limit State	Overstress Indices	Panel Point
Main Plates	Dead + BSALL	ULS	<1.0	
	Dead + Wind (BD37/88)	ULS	<1.0	
	Dead + BSALL +50mph Wind	ULS	<1.0	
	Dead + HA	ULS	1.32	47-48
	Dead + HA + Wind	ULS	1.20	47-48

Bottom Lateral Element	Load case	Limit State	Overstress Indices	Panel Point
Bolts	Dead + BSALL	SLS	0.88	2-3
	Dead + Wind (BD37/88)	SLS	0.46	47-48
	Dead + BSALL +50mph Wind	SLS	Not critical	
	Dead + HA Dead + HA (91% UTS)	SLS	1.08 (0.94)	2-3
	Dead + HA + Wind	SLS		

Table 5.18 Summary of extent of Overstressed Bolts in Bottom Laterals

	Load Case	Extents of Overstressed Panels (all SLS)
Bottom Lateral Member Bolts	Dead + Wind	-
	Dead + BSALL	-
	Dead + BSALL + Wind	-
	Dead + HA	Panel 1-3, Panel 41-43, Panel 47-53
	Dead + HA + Wind	-

5.4.6 Bottom Lateral Gusset Plates

Table 5.19 Summary of Maximum Bottom Lateral Gusset Plate Overstress Indices

Bottom Lateral Gusset Plate Location	Load case	Limit State	Overstress Indices	Panel Point
Side Span	Dead + BSALL + 50mph wind	ULS	0.82	2-3
	Dead + HA + Wind	ULS	1.00	2-3
Main Span	Dead + Wind	ULS	0.43	47-48

Table 5.20 Summary of extent of Overstressed Gusset Plates in Bottom Laterals

	Load Case	Extents of Overstressed Panels (all ULS)
Bottom Lateral Member Gusset Plates	Dead + Wind	0
	Dead + BSALL	0
	Dead + BSALL + Wind	0
	Dead + HA	0
	Dead + HA + Wind	0

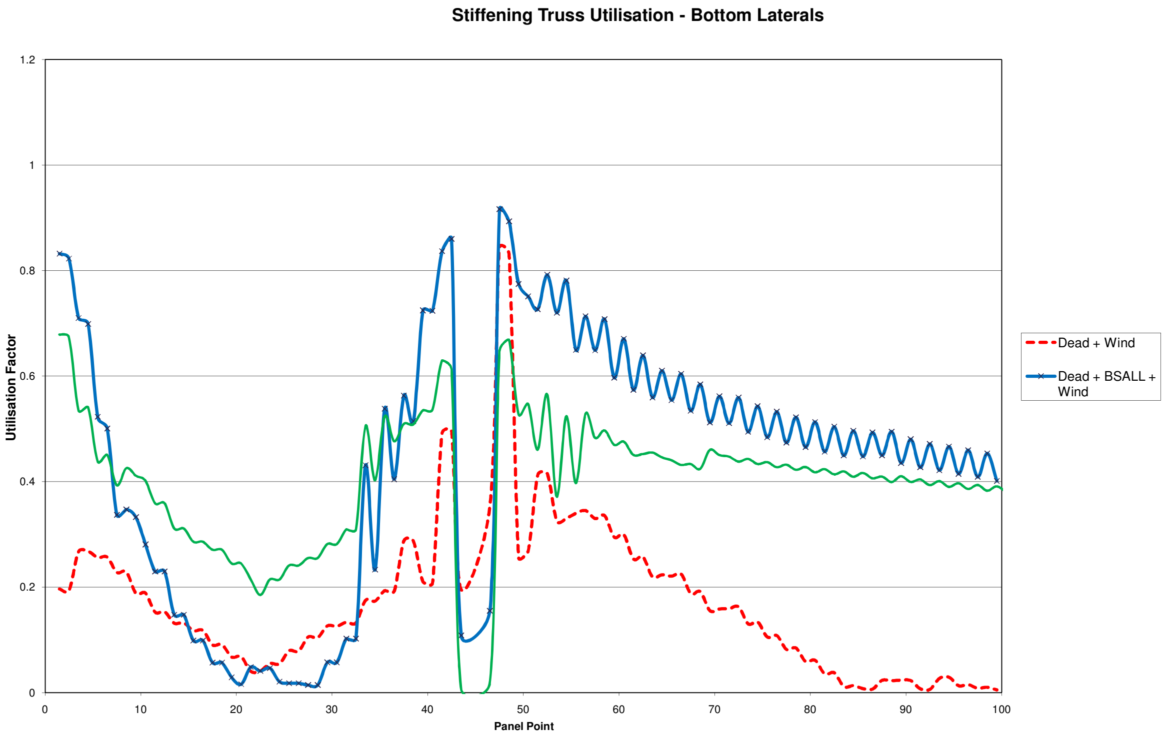


Figure 5.8 Summary of Bottom Laterals Overstress Indices (BSALL and Dead + Wind Load Combinations)

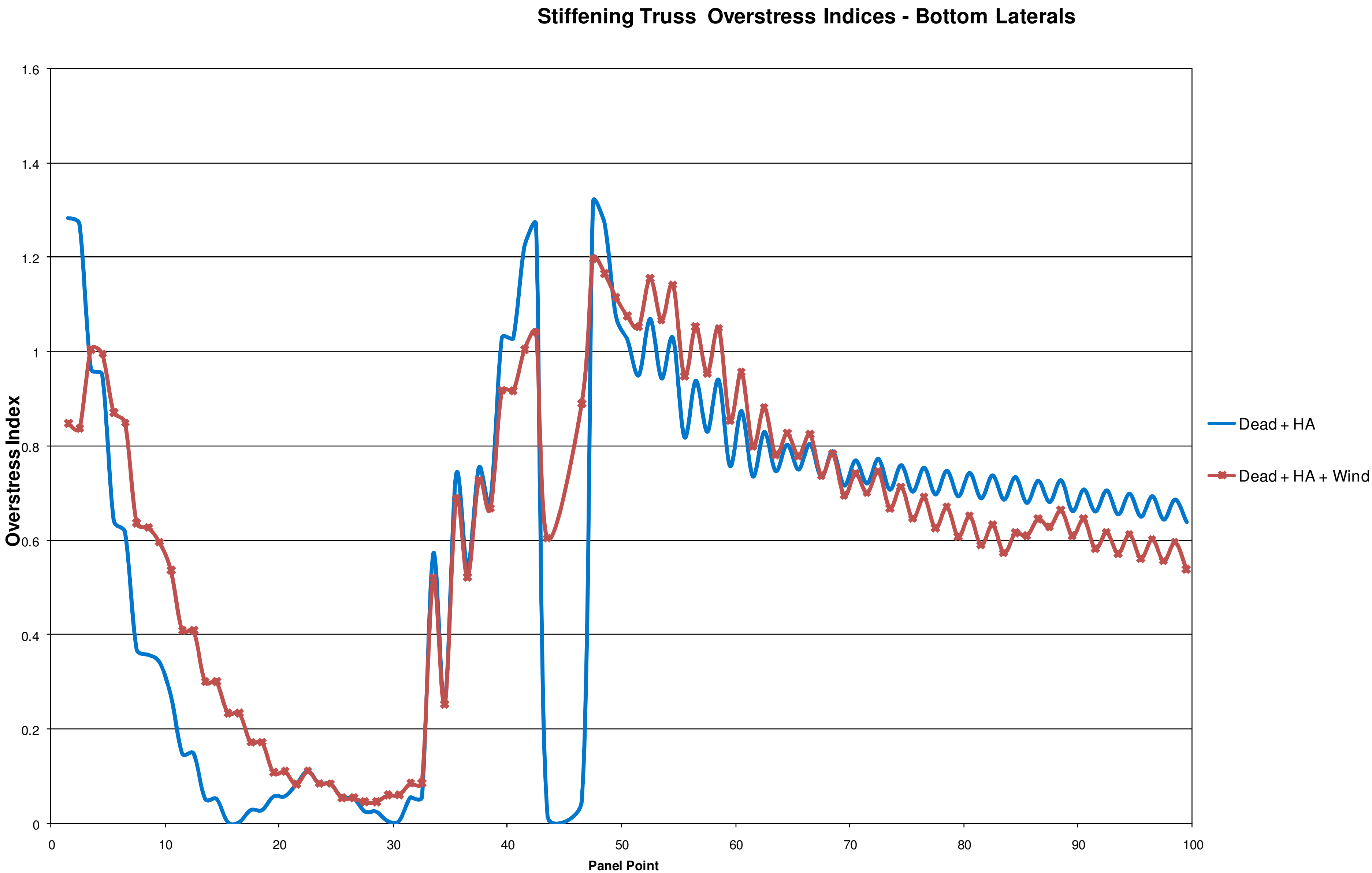


Figure 5.9 Summary of Bottom Laterals Overstress Indices (HA Load Combinations)

Table 5.21 Summary of Main Members Maximum Overstress Indices

	Dead + BSALL	Dead + BSALL + 50 mph Wind	Dead + Wind	Dead + HA	Dead + HA + Wind
Top Chord	0.96	0.92	0.99	1.30	1.83
Bottom Chord	0.82	0.74	1.15 (0.92)	1.15	1.55
Diagonals	0.95		0.36	1.17	
Verticals	Refer to Section 6				
Top Laterals	0.99	0.99	1.55	1.54	2.45
Bottom Laterals	<1.0	<1.0	<1.0	1.32	1.20

5.5 Lateral Thrust Members at Main Towers

Lateral thrust members transfer the lateral wind load reaction from the stiffening truss to the main tower. The thrust members are connected to the cross box between the tower legs just below the deck. The ends of the stiffening truss fit either side of it to form a sliding joint which permits the deck to move longitudinally. The lateral loads are transferred via a box girder which is propped by 2 diagonal I sections.

Since the original construction, leaf bearings have been introduced between the main box member and the cross beam to eliminate bending moments between the box beam and cross beam.

By inspection, the critical load case for the lateral thrust members is Dead + Wind, since wind is the dominant transverse force on the bridge. The maximum transverse wind force occurs on the main span side and all the overstress indices stated below correspond to the main span side.

Table 5.22 Summary of Lateral Thrust Member Overstress Indices:

Structural Element	Loadcase	Overstress Index
Main Longitudinal Box Member	Dead + Wind at 30 degrees	0.85
Connection Main Box Member to Leaf Bearing (assuming M20 HSFG bolts)	Dead + Wind at 30 degrees	0.78
Diagonal I section bracing members	Dead + Wind at 10 degrees	0.87
Bracing Member connection to Tower	Dead + Wind at 10 degrees	0.88
Leaf Bearing by Glacier (see further comments below)	Dead + Wind at 30 degrees	0.99

The leaf bearings were introduced in 1988 and are shown on Glacier Drawings A2/GPF 3471 sheets 1 and 2. The drawings indicate that the maximum transverse load capacity is 2100 kN. The maximum SLS applied load is 2086 kN. There are several key dimensions missing from the above drawings but based on our interpretation of the above drawings an assessment has been made of the primary structural elements making up the bearings. Based on this interpretation involving scaling some of the dimensions, It is possible to verify the capacity at 2100 kN at SLS working loads.

The transverse forces are transferred to the lateral thrust member from the ends of the truss via bearings attached to the end cross girders at PP 44 and 46. The maximum calculated force is 5895 kN and the bearings are comprised of 3inch diameter pins which are 2 feet long and are high tensile steel to BS 548. The pins bear onto castings out of Meehanite Grade A. BS 1452:1948 refers to Meehanite with UTS varying from 10 to 26 ton/sq in (154 to 355 N/mm2). Referring to the Historical Structural Steelwork Handbook by BCSA, this give guidance to allowable compressive stresses for cast iron of 8.0 ton/sq in (123.5 N/mms) since 1900. All these values are greater than the calculated bearing pressure between the pin and the Meehanite casting which is calculated to be 91 N/mm2 at nominal load factors.

It is concluded that the bearings are adequate.

5.6 Stiffening Truss End Links

The end links transfer the vertical reaction from the end of the stiffening truss in tension back to the end link brackets at the main towers. Each bottom chord is supported by two end links which are formed from welded sections. Each bottom chord is connected to the two end links by a single 7-inch diameter steel pin. The tops of the end links are welded to fork couplings and each fork coupling is connected to the end link bracket by a 5-inch diameter steel pin.

The critical load case for the end link members is Dead + Live (no wind) since wind loads tend to cause uplift in the bridge deck.

The main welded members were found to be adequate at Dead + BSALL with an overstress index of 0.91. Under Dead + HA the overstress index increases to 1.14.

It has been assumed that the pins are prevented from rotating within the fork couplings by keep plates which are screwed to the adjacent connecting part and slotted into a groove in the pin. Under BD56/96, it is a requirement that the end distance of the connecting part should be a minimum length equal to the width of the connecting part measured at the pin. The width of the fork coupling is 1 foot whereas the end distance is only 6 inches. Therefore it does not strictly comply with the assessment code. A more detailed FE analysis has been carried out of the connection and it has been conclude that the non-compliant end distance does not reduce the calculated capacities.

5.7 End Link Brackets at Main towers

The end link brackets support the stiffening truss end links via the 5 inch diameter pins. The end link brackets are welded I sections which are welded to the main towers. The flanges are curtailed 1 foot back from the pin location and the remaining web section is strengthened with 2 No 1¼ inch cheek plates. The bracket web projects through and is welded to the nearside tower external vertical plate and the vertical angle stiffener. The end of the web is welded to the next angle stiffener. Although there is a horizontal diaphragm at the same level as the bottom flange of the bracket, there is no diaphragm at the top flange level. It has been assumed that the moment in the end link bracket will be transferred through the web. On this basis checks have been made at the root of the cantilever and at the critical location where the flanges are curtailed.

The moment fixity of this cantilever bracket is provided by two sets of vertical welds, one (2no. 5/16”/7/16” welds running the full height of the web and 2no. intermittent 5/16”/7/16” running the height of the web intermittent with 4” weld/4” gaps) at the face of the tower and the other (2no. 5/16”/7/16” welds running the full height of the web intermittent with 4” weld/4” gaps; net length 2’ per weld) at the termination of the portion of the web that is inside the tower connecting to the vertical angle stiffeners. These sets of welds allow development of a couple composed of a pair of vertical forces that resists the bending developed in the cantilever. The most critical weld was found to be the weld at the end of the web within the tower.

In order to assess the brackets, a finite element buckling analysis was carried out as the geometry of the cantilever is non-uniform. Hand calculations were also carried out to verify the results.

Table 5.23 Summary of End Link Bracket Overstress Indices

Element	Loadcase	Overstress Index
Main Members (Main Span)	Dead + BSALL + BSFLL	1.20 (See note below)
	Dead + HA	2.47
Welds (Main Span front face of Main Tower at root of cantilever)	Dead + BSALL + BSFLL	1.42
	Dead + HA	2.49
Welds (Main Span at end of bracket within Main Tower)	Dead + BSALL + BSFLL	1.95
	Dead + HA	3.40
Main Members (Side Span)	Dead + BSALL + BSFLL	0.98
	Dead + HA	1.93
Welds (Side Span front face of Main Tower at root of cantilever)	Dead + BSALL + BSFLL	1.40
	Dead + HA	2.37
Welds (Side Span at end of bracket within Main Tower)	Dead + BSALL + BSFLL	1.50
	Dead + HA	2.54

Note (Main Member Main Span): The OI is based on BD 56 which uses the yield stress of material at 16mm thickness and a gamma m factor of 1.20. If the approach from BS 5400 Part3: 2000 is used, in which the yield stress is reduced to 232 N/mm2 and gamma m reduced to 1.05, the OI reduces slightly to 1.15.

5.8 Rockers at Side Towers

The vertical reaction from the stiffening truss at the side towers is transferred via rockers which are welded I sections with steel slab ends with spherical bearing surfaces at the top and bottom. The top and bottom are connected to the stiffening truss and the side tower respectively by 2 No screwed rods tensioned to 55 tons. The contact stresses were calculated using BS5400 Part 9.

Table 5.24 Summary of Overstress Indices at Side Tower Rockers

Link Element	Loadcase	Overstress Index
Main Members	Dead + BSALL	0.72
	Dead + HA	1.27
Welds	Dead + BSALL	0.32
	Dead + HA	0.57
Contact Stress	Dead + BSALL	0.67
	Dead + HA	1.19

5.9 Lateral Bearings at Side Towers

The lateral thrust bearing at the side towers consists of a steel block which is welded to the top laterals. This bears against a convex-shaped surface of a welded bracket which is cast into the concrete side tower. The steel block is anchored to the side tower by 3 screwed rods 64.4mm in diameter to EN 15Q to BS 970. These bars pass through the tower, post-tensioned to 80 tons and clamped against the concrete face. The rods pass through mild steel tubes and it is assume that the rods have not been grouted in.

The bearings are subject to longitudinal load and shear loads. The most critical longitudinal load is tension as a result of longitudinal wind from wind blowing at 50 degrees as stated in the University of Glasgow Wind Tunnel tests. It is noted that the maximum longitudinal wind load coefficient is 0.1327. The equivalent coefficient used for the original design was 0.1028 and it is therefore 29% higher.

The maximum ULS loads applied to the Side Towers are;

- Maximum longitudinal load (wind at 50 degrees) 4570 kN
- Associated transverse load 2170 kN

- Maximum transverse load (wind at 0 degrees) 4152 kN
- Associated longitudinal load 600 kN

Using the maximum tension obtained from the wind tunnel tests obtained from BD 37/88, the prestress in the bars is exceeded. There is also an apparent overstress of 1.35 against the yield stress of the bars. If the wind load is estimated using BD 37/01, the overstress reduces.. By inspection, it can be seen that with the further reduction of 29% used in the original design, the bars would have been theoretically adequate.

The shear resistance of the system is less obvious to estimate. The clamping force, provided by the 3 bars, in theory provides shear resistance through friction between the steel bracket and the concrete face of the side tower. However, most of the critical load cases create high tensile forces which significantly reduce the prestress force in the bars. The shear resistance of the 3 post-tensioned bars has been ignored as it is not known if the bars are grouted within the steel tubes.

It was initially assumed that the 4 No 38mm diameter holding down bolts would contribute to the transverse shear resistance but it was found that these were severely overstressed.

Having noted the high overstress in the bolts, alternative and additional load paths were examined. Therefore calculations were carried out to determine the bond between the cast-in channels and the concrete. It was estimated that the bond stress required to resist the applied transverse load was 1.9 N/mm2. This has been compared against published values for bond between steel and concrete and is comparable to the bond stress of 1.9 N/mm2 for plain bars in compression in grade 30 concrete in accordance with BS5400 Part 4.

It has been noted that there is a large range of published data relating to allowable bond strength between steel and concrete and the figure of 1.9 N/mm2 as at the high end. It is understood that there are no signs of distress at this location on the bridge. It is recommended that this area is inspected regularly for any signs of distress.

Table 5.25 Summary of Overstress Indices at Side Tower Lateral Bearings

Link Element	Loadcase	OI
Lateral Bearing	Dead + Wind (ULS)	1.0
Bars	Dead + Wind (ULS)	1.35

6 Cross Girder Assessment

6.1 Introduction and Method of Analysis

In order to simplify the computing it was decided not to fully model all the cross girders on the bridge. The majority of the cross girders were therefore simplified in the model by using K-bracing on its side. A preliminary assessment of the more critical cross girders was carried out and these critical girders which were fully modelled. Full cross girders were therefore added immediately adjacent to the side and main towers as well as the neighbouring girders. Rigorous checks were made to ensure that the simple cross girders did not affect the analysis.

Full cross girders were also added at in-span sections within the main and side spans. In addition independent 3-D models were created up to 8 bays in length were created to provide a further check of the member forces.

6.2 Loading

The loading for the cross girders is not specifically addressed in the AIP and FM have therefore carried out a detailed examination. In the first instance, the critical loaded length for the critical cross girders was determined. The critical loaded length was estimated to be typically 10-12 m for in-span cross girders. For the end cross girders the critical load case were torsion load cases. The approximate loaded length for the side span end cross girders at PP1 and PP 44 was found to be approximately 300m long.

As described above, our first consideration was to check for assessment loading to BD 21/01. The BSALL loading derived by WAF is for loaded lengths in excess of 100m and therefore, the assessment load had to be derived from BD21/01. In accordance with BD 21/01 we therefore carried out the following checks:

- Assessment HA live loading in accordance with BD 21/01 Clause 5.1
- Critical Vehicles in accordance with BD 21/01 Annex D.
- HB single vehicle only
- Wind Loading

6.2.1 Assessment HA Loading

Carriageway Loaded Length 2-50m

For loaded lengths between 2 and 50m, the assessment live loading should be taken as HA UDL and KEL modified by the reduction factor K. It has been assumed that the surface condition is poor, based on the relatively thin mastic asphalt surfacing and the joints in the road panels at 60ft (18.288m) centres. On this basis the reduction factor, K, has been taken as 0.91.

In addition and an adjustment factor, AF, is applied to the HA UDL and KEL. This factor is a lateral bunching factor which takes into account the possibility that, in slow moving situations more lanes of traffic than the marked or notional lanes could use the bridge. However, the impact factors incorporated in the HA for higher speeds should not be considered together with the maximum lateral bunching. On this basis, BD21/01 allows an AF which is divided into the HA loading. Typically for loaded lengths less than 20m, the AF = al/2.5 where al = 3.65. In other words, the AF equals 1.46 and the HA UDL and KEL are reduced by 1/1.46.

In accordance with BD21/01, Clause 5.24, the lane factors are as follows:

Lane 1 Factor	Lane 2 Factor	Lane 3 Factor	Lane 4 Factor
1.0	1.0	0.5	0.4

Carriageway Loaded Length 50-100m

For loaded lengths between 50 and 100m, the assessment live loading should be taken as HA UDL and KEL modified by the reduction factor K of 0.91 as above. However, under BD 21/01, the adjustment factor, AF, = 1.0 for loaded lengths greater than 40m. The lane factors are as above. It was found that these loaded lengths were generally not critical.

Carriageway Loaded Lengths in excess of 100m

For these loaded lengths the assessment live loading was taken from the 2006 BSALL.

6.2.2 Critical Vehicles to Annex D

The load effects from the critical vehicles in BD 21/01 Annex D were derived using the lane factors 1.0 for lanes 1 and 2; 0.5 for lane 3 and 0.4 for lane 4.

6.2.3 HB Loading

In the AIP Clause 4.1.2 it is specified that 45 units of HB loading should be considered and that it should not be considered acting in combination with bridge specific live loading. It has been assumed that the bridge specific live loading refers to the loading stated in clause 5.25 to 5.27 of BD21/01 and it has therefore been assumed that the HB shall be considered acting on its own. It is assumed that vehicles equivalent to HB would cross the bridge with a police escort and the opposite carriageway closed.

6.2.4 Pedestrian Live Loading

The footway loading was initially applied In accordance with the original AIP which used footway loading based on the principles of BD 37/01 Clause 6.5.1 but the formula used the BSALL loading intensity and not the HA loading intensity .For those members which were theoretically overstressed the loading was reduced to the Bridge Specific Footway Live Loading derived by WAF in June 2006.

Footway loaded length 2-36m

For loaded lengths less than 100m, it is unclear from the AIP what the load intensity should be. We have therefore looked at two scenarios.

Conservatively, the loaded length could be taken from BD37/01. For the typical cross girders with a loaded length of 10-12m, the load intensity would be 5 kN/sqm. We have also looked at the load effects taken from the Bridge Specific Footway Live Loading (BSFLL) Report produced by WAF in June 2006.

Footway Loaded Length in excess of 100m

We have also looked at two scenarios for loaded lengths in excess of 100m. For the critical cross girders, the footway loading is based on the principles of BD 37/01 Clause 6.5.1 but the formula uses the BSALL loading and not the HA loading. We have also used the BSFLL loading.

6.2.5 Main Members Results

In the following section, the overstress indices will be reported for the various cross girder types. The cross girder types referred to as CG1, CG2, CG3 etc refer to the original design designation. The cross girder types are summarised below:

- CG1: Panel Point 1 end side span adjacent to side tower
- CG2: Panel points 2, 4, etc to 42 side span typical cross girders at hanger locations
- CG3: Panel points 3 and 43 side span
- CG5: Panel points 5,7 etc to 39 side span typical cross girders between hangers

- CG6: Panel point 44 end side span adjacent to main tower
- CG7: Panel point 46 end main span adjacent to main tower
- CG8: Panel points 47, 49 etc to 53,59 main span typical cross girders between hangers
- CG10: Panel points 48,50 etc to 58 main span typical cross girders at hanger location
- CG11: Panel points 57 and 59 on main span
- CG20: Panel points 60, 62 etc to 100 main span typical cross girders at hanger locations
- CG21: Panel points 61,63 to 101 main span typical cross girders between hangers

6.2.6 Assessment HA Loading/ Critical Vehicles to Annex D

The evaluation of the effective length of the individual members is critical to the assessment of the members. The most critical typical member was found to be the lower diagonal member particularly in the side span. BD 56 provides the following guidance depending on whether the web members of the cross girder or truss are treated as a single triangulated system or a multiple intersection system:

Member	**Effective Length**	
	Buckling in Plane of Truss	Buckling normal to truss
Web Single Triangulated System	0.70 times distance between intersection with chords	0.85 times distance between intersection with chords
Web Multiple Intersection System	0.85 times the greatest intersection between any two intersections	0.70 times distance between intersection with chords

Initially it was considered that the lower diagonal member should be treated as a web member in a multiple intersection system and it was found that the limiting slenderness was obtained from buckling in the plane of the truss. On this basis, the lower diagonal members were found to be adequate for Dead + BSALL + BSFLL for the cross girders at PP1 and just adequate for the typical main span cross girders. However, the typical side span cross girders in the side span were overstressed.

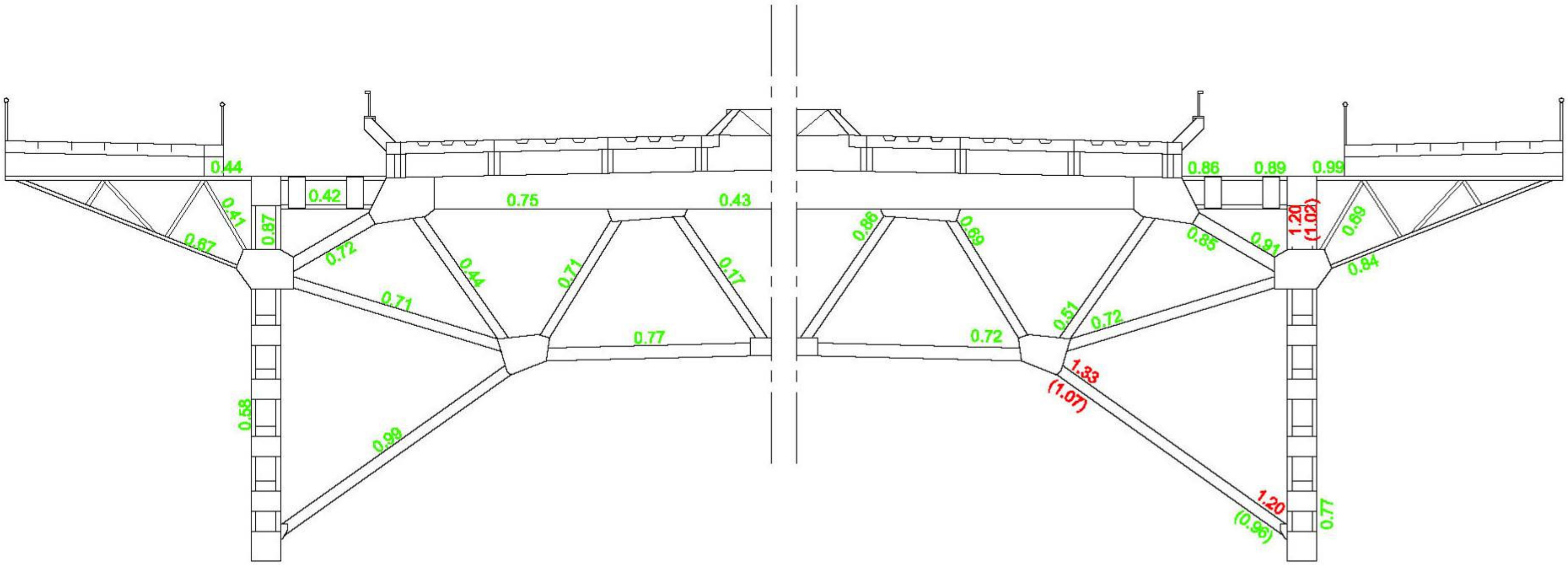
The intentions of the original design are difficult to discern and it was therefore decided to carry out a critical buckling analysis of the system using a local model consisting of 8 bays. This indicated that the system behaved more closely to that of a single triangulated system and the side span cross girders were re-checked for the effective length 0.7 times the length of the member. As the bottom chord of the cross girder is in tension this member tends to provide restraint to the lower diagonal member.

The results for the Assessment Loading/Critical Vehicles are presented in Figure 6.1 to 6.4. Assessment HA loading is typically less critical than the Critical Vehicle loading.

For the typical cross girders (CG2, CG3, CG5, CG8, CG10, CG11, CG20, CG21) in both the main and side spans the maximum overstress indices are less than 1.0. The maximum index is 0.99 in the lower diagonal member at the side span intermediate cross girders (CG3/5).

For cross girder at panel point 1 adjacent to the side tower, the maximum overstress is 1.01 in the lower diagonal member1 when subject to a torsion load case with BSALL and footway loading (in accordance with AIP) for 300m in the side span. However, if the AIP footway loading is reduced to the BSFLL loading the overstress index by inspection will be less than 1.0

For cross girders adjacent to the main towers (CG6 at panel point 44 and CG7 at panel point 46), the maximum overstress index is 1.61 in the vertical members. This overstress is based on the footway loading as stated in the AIP. If the BSFLL footway loading is used, the overstress index reduces to 1.2. A more rigorous analysis involving non-linear buckling analysis of this member would reduce this overstress further. At this stage, the non-linear buckling analysis has not been carried out.



Member Overstress Indices

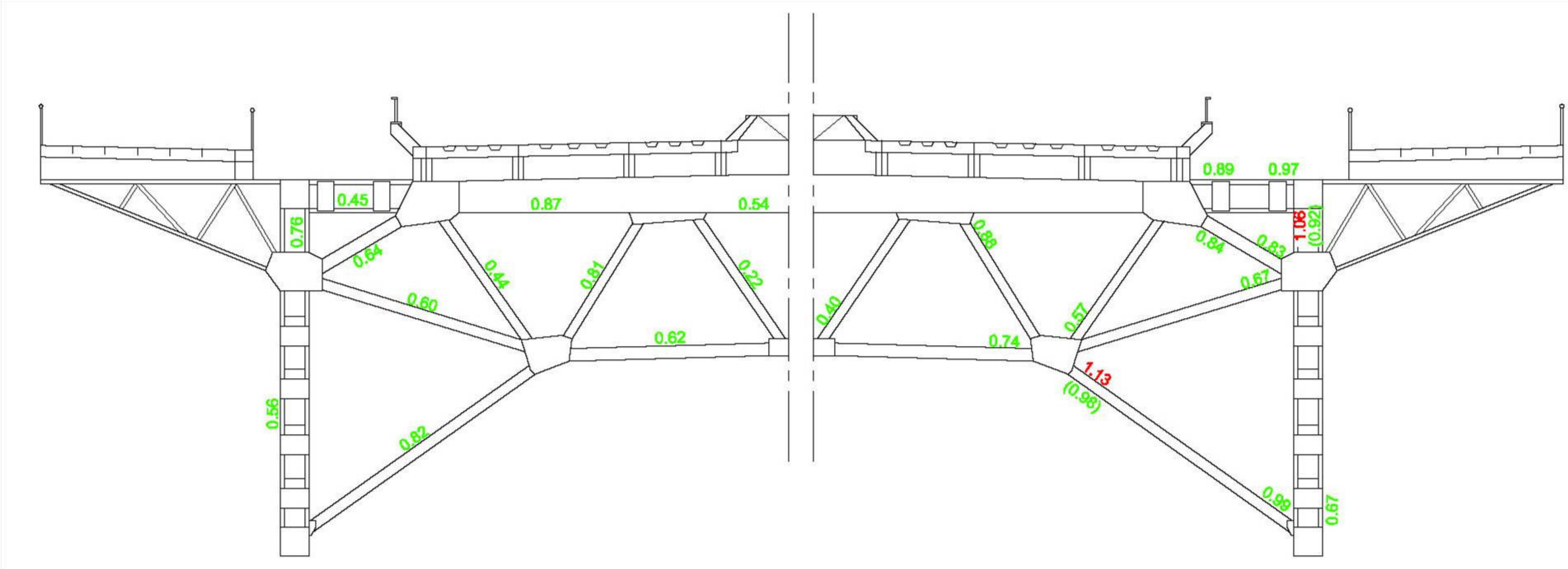
PP 2-43

Bolts Overstress Indices

PP 2-43

(Figures in brackets indicate indices for relaxation of 10% reduction and BSFLL footway loading)

Figure 6.1 Side Span Typical Cross Girders (CG2, CG3, CG5)
HA Assessment Load/ Critical Vehicle Load



Member Overstress Indices

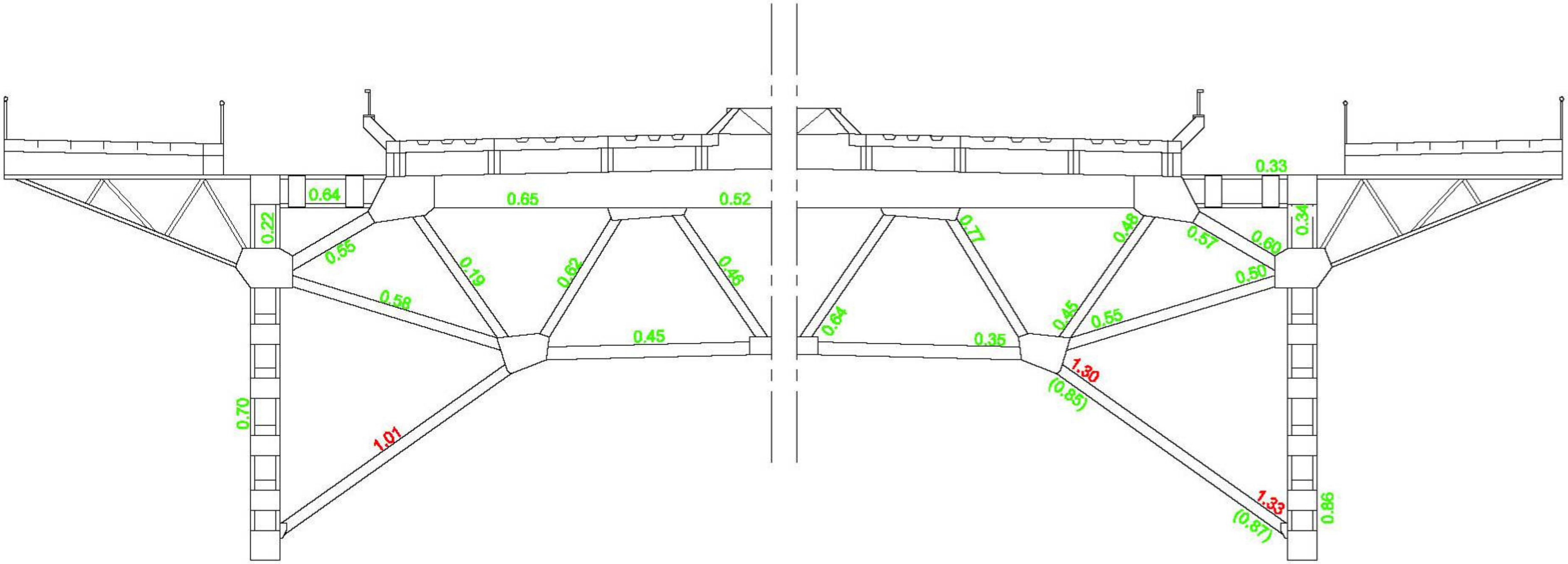
PP 47-101

Bolt Overstress Indices

PP 47-101

(Figures in brackets indicate indices for relaxation of 10% reduction and BSFLL footway loading)

Figure 6.2 Main Span Typical Cross Girders (CG20, CG21)
HA Assessment Load/ Critical Vehicle Load



Member Overstress Indices

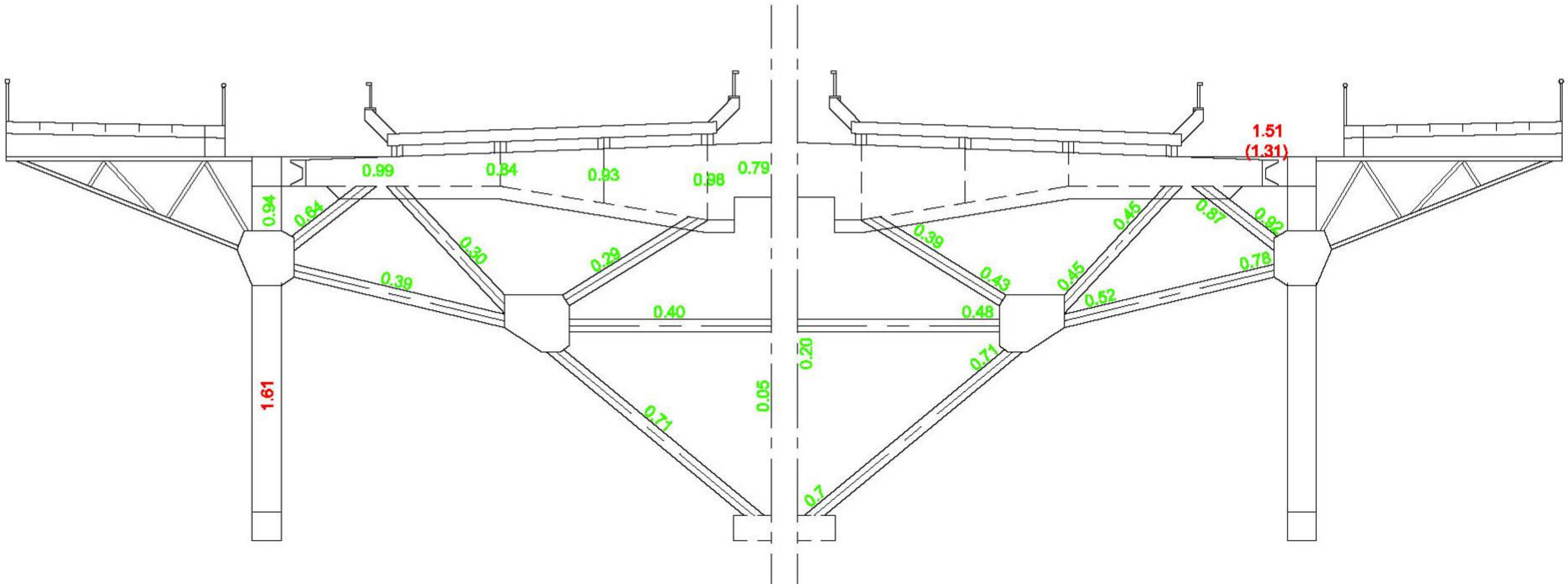
PP 1

Bolt Overstress Indices

PP 1

(Figures in brackets indicate indices for relaxation of 10% reduction and BSFLL footway loading)

Figure 6.3 Cross Girder at PP1 (CG1)
HA Assessment Load/ Critical Vehicle Load



Member Overstress Indices

PP 44/46

Bolt Overstress Indices

PP 44/46

(Figures in brackets indicate indices for relaxation of 10% reduction and BSFLL footway loading)

Figure 6.4 Cross Girders at PP 44, 46 (CG6,7)
HA Assessment Load/ Critical Vehicle Load

6.2.7 *Full HA Loading*

The critical loaded length was established by trial and error and was found to be approximately 10-12m for the typical cross girders.

The results for the HA Loading on cross girders are shown on Figures 6.5 to 6.8. For the typical side span cross girders, the maximum overstress index is 1.36 in the lower diagonal member. The upper vertical members have an overstress index of 1.30

For the typical main span cross girders, there is an overstress index of 1.23 in the vertical and 1.20 in the top chord. The lower diagonal member has an overstress index of 1.16 and one of the internal diagonal members has an overstress index of 1.06.

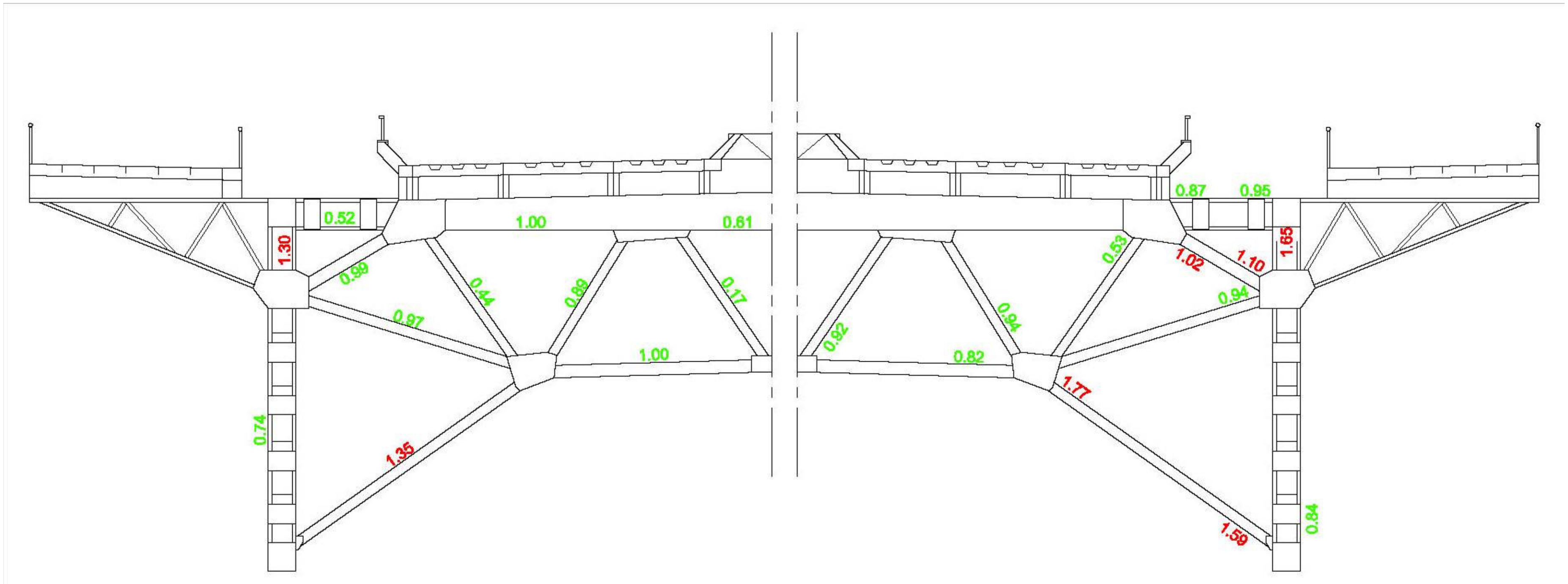
For cross girder CG1, the maximum overstress index is 1.29 in the lower diagonal member.

For the cross girder CG6 and CG7, the maximum overstress index is 2.13 in the vertical member. The top chord also has a small overstress of 1.02.

6.2.8 *HB Loading*

Under a single HB vehicle the top chord has a maximum overstress index of 1.16.

All other overstress indices are less than 1.0.



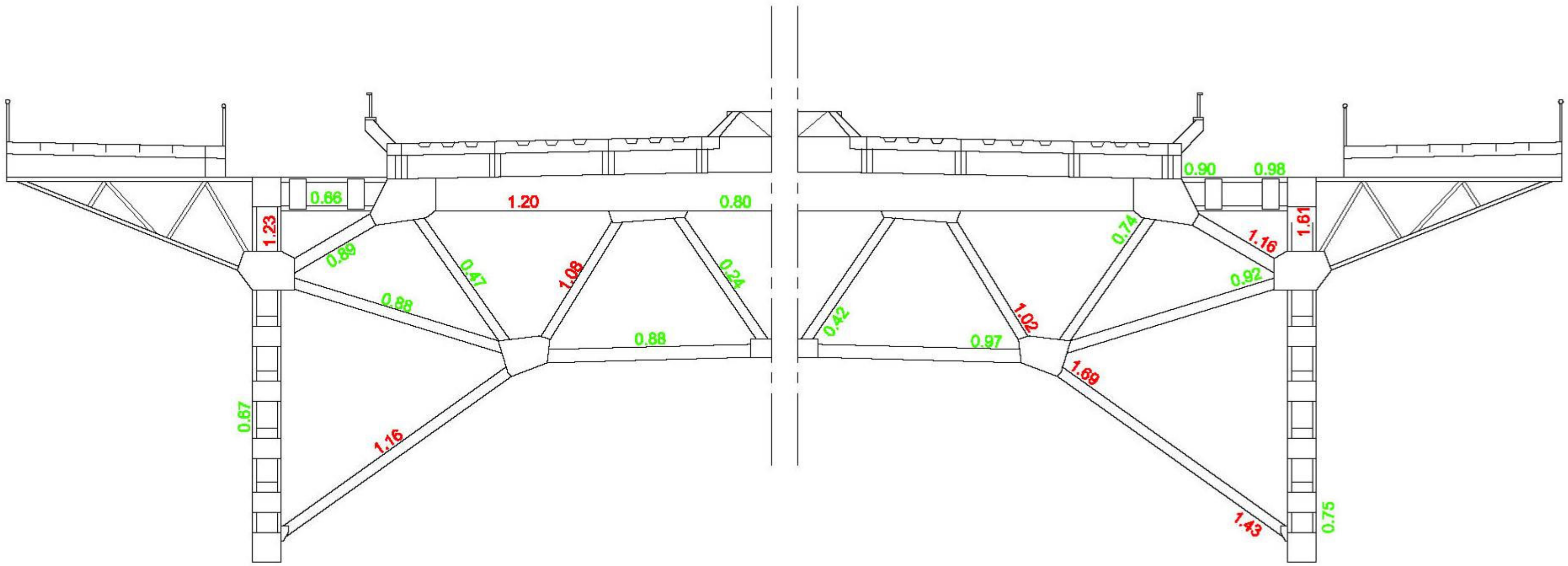
Member Overstress Indices

PP 2-43

Bolt Overstress Indices

PP 2-43

Figure 6.5 Side Span Typical Cross Girders (CG2, CG3, CG5)
Full HA Load



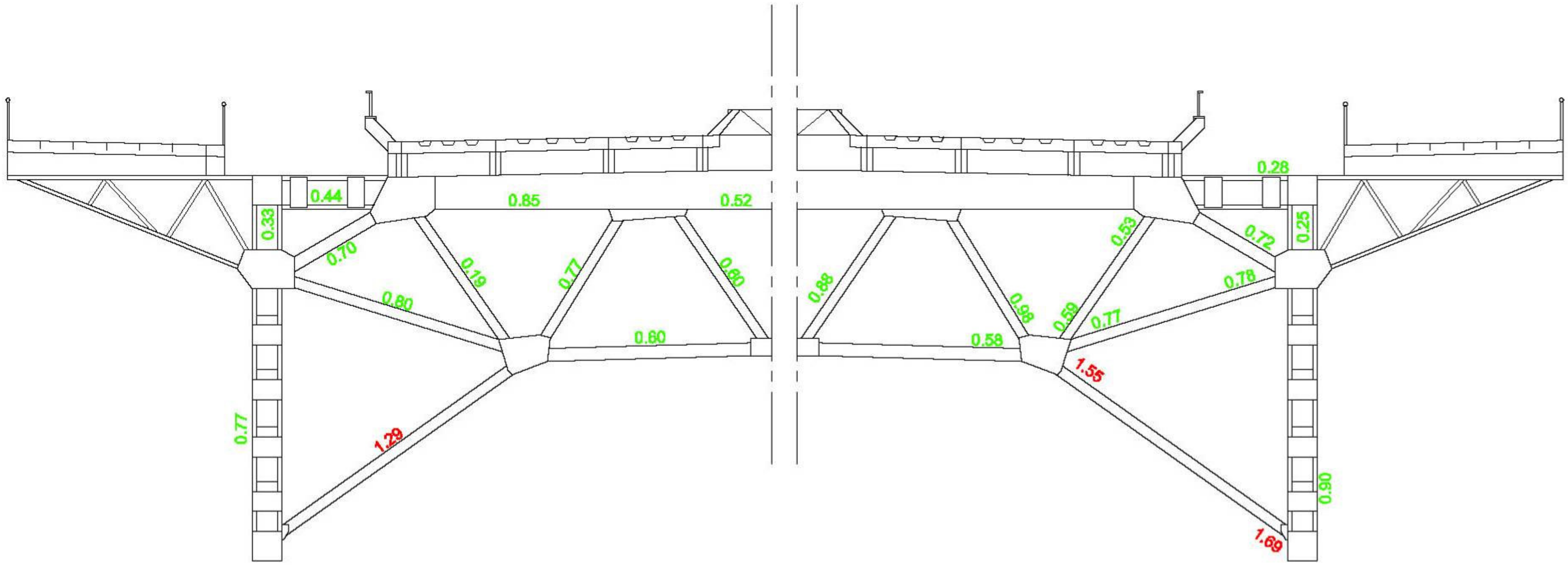
Member Overstress Indices

PP 47-101

Bolt Overstress Indices

PP 47-101

Figure 6.6 Main Span Typical Cross Girders (CG8, CG10, CG11, CG20, CG21)
Full HA Load



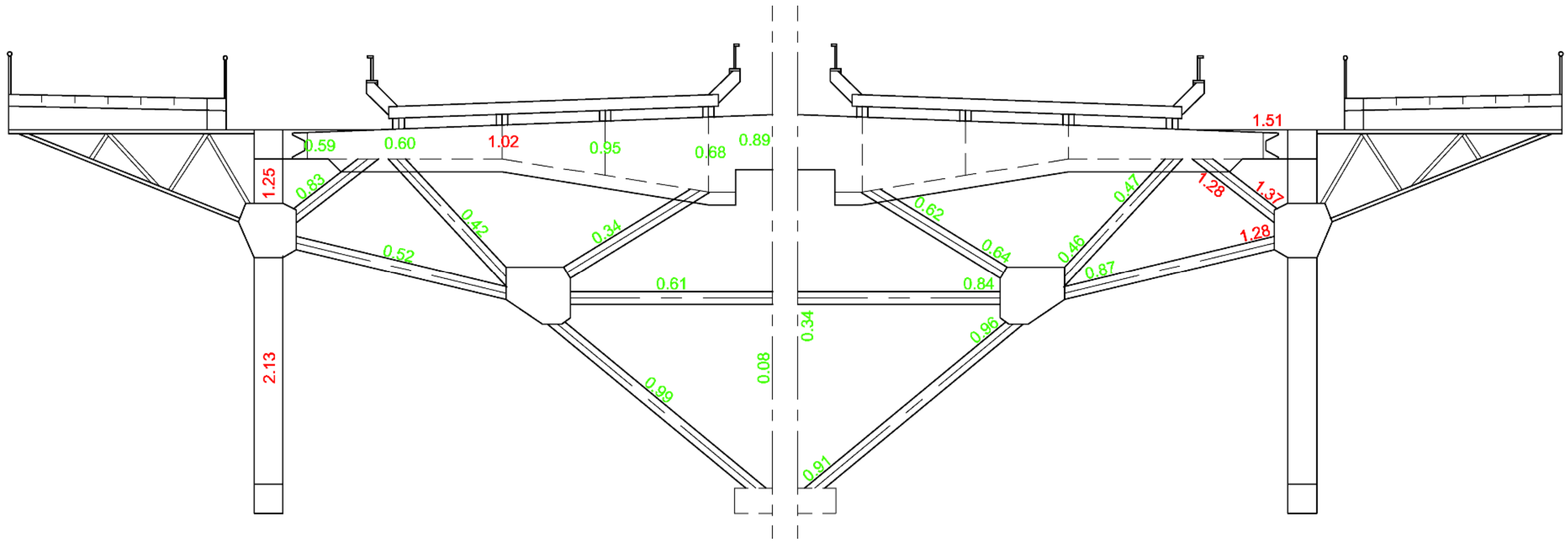
Member Overstress Indices

PP 1

Bolt Overstress Indices

PP 1

Figure 6.7 Cross Girder at PP1 (CG1)
Full HA Load



Member Overstress Indices

PP 44/46

Bolt Overstress Indices

PP 44/46

Figure 6.8 Cross Girders at PP 44, 46 (CG6,7)
Full HA Load

6.3 Bolted Connections and Gusset Plates

6.3.1 Assessment HA and Critical Vehicle Loading

The results for the bolted connections are presented in Table 6.1 below.

For the typical side span intermediate cross girders (CG5), the bolted connections at the top and bottom ends of the lower diagonal member have an overstress of 1.33 and 1.20 respectively. If the connections are re-assessed with a relaxation of the 10% reduction (see section 5.2.2 of this report) and the footway loading reduced to the BSFLL, the overstress indices reduces to 1.07 and 0.96 respectively. At the top of the verticals for the typical side span cross girders at the hangers (CG2), the overstress index is 1.20. This reduces to 1.02 if the connections are re-assessed as section 5.2.2 of this report and the footway loading reduced to the BSFLL

For the typical main span cross girders at the hangers (CG20), the maximum overstress index at the top of the verticals is 1.06. If the connection is re-analysed assuming BSFLL loading on the footways and section 5.2.2 of this report, the overstress index reduces to 0.92. All other connections in the main span typical hanger cross girders (CG20) and intermediate cross girders (CG21) are less than 1.0 with a maximum of 0.99 at the bottom of the lower diagonal member.

At the cross girder at panel point 1 (CG1) adjacent to the side tower, the maximum overstress index for the connections at the top and bottom of the lower diagonal member are 1.30 and 1.33 respectively. If the connections are re-assessed with section 5.2.2 of this report and the footway loading reduced to the BSFLL, the overstress indices reduces to 0.85 and 0.87 respectively.

At the cross girder at panel point 44/46, the maximum overstress index is 1.51 at the connection between the cross girder top chord and the main top chord under dead plus wind load. This reduces to 1.31 if the connections are re-assessed with the proof load equal to 91% of the UTS and the footway loading reduced to the BSFLL.

All other bolted connections for the cross girders are satisfactory subject to assessment loading to BD21/01.

The gusset plates for all cross girders subject to load combinations with BSALL have overstress indices less than 1.0. There is a theoretical overstress index of 1.063 for the gusset plate between the top chord and the vertical in panel points 44 and 46. However, if the wind loads are reanalysed to BD 37/01, the overstress index is less than 1.0.

6.3.2 Full HA Loading

Many of the bolted connections are overstressed based on SLS friction value. The maximum overstress index is 1.77 at the upper end of the lower diagonal member on the typical side span cross girders.

Under HA, the maximum overstress index for the gusset plates is 1.31 in the central bay of the bottom chord for the cross girders located at hanger locations in both the main and side spans.. There are minor overstress indices of 1.04 in the end bay of the bottom chord in the typival side span cross girders at the hanger locations. There is a minor overstress of 1.04 in the lower diagonal members of the cross girders located between hangers.

At panel points 44 and 46, the maximum overstress is 1.2 in the lower central diagonal and 1.18 in the top chord.

6.3.3 Single HB Loading

Many of the bolted connections are overstressed based on SLS friction value. The maximum overstress index is 1.23 at the upper end of the lower diagonal member on the typical side span cross girders.

Table 6.1 Summary of Maximum Overstress Indices for Cross Girders

Cross Girder Element	Load Case	Limit State	Stress Indices	Cross Girder/ Member
Main Members	Dead + BSALL+ Fway	ULS	1.61	CG6/ CG7 Vertical members
	Dead + HA + Fway	ULS	1.35	CG3, CG5 (Lower diagonal)
	Dead + HA + Fway	ULS	1.30	CG2, (Vertical)
	Dead + HA + Fway	ULS	1.20	CG8, CG10, CG11, CG20 (Top Chord)
	Dead + HA + Fway	ULS	1.23	CG10, CG20 (Verticals)
	Dead + HA + Fway	ULS	1.16	CG8, CG11, CG21 (Lower diagonal)
	Dead + HA + Fway	ULS	1.29	CG 1 (Lower diagonal)
	Dead + HA + Fway	ULS	2.13 1.02	CG6 (Vertical) CG6 (Top Chord)
	Dead + HA + Fway	ULS	1.59 1.16	CG7 (Vertical) CG7 (Top Chord)
Bolts	Dead + Critical Vehicle + FWay (Dead + Critical Vehicle + BSFLL)	SLS	1.33 (1.07)	CG3, CG5 (Lower diagonal)
	Dead + Critical Vehicle + FWay (Dead + Critical Vehicle + BSFLL)	SLS	1.20 (1.02)	CG2 (Verticals)
	Dead + Critical Vehicle + FWay (Dead + Critical Vehicle + BSFLL)	SLS	1.06 (0.92)	CG10, CG20 (verticals)
	Dead + Critical Vehicle + FWay (Dead + Critical Vehicle + BSFLL)	SLS	1.13 (0.98)	CG8, CG11, CG21 (Lower diagonal)
	Dead + BSALL+ FWay (Dead + BSALL + BSFLL)	SLS	1.33 (0.87)	CG1 (Lower diagonal)
	Dead + Wind (BD 37/88)	SLS	1.51	CG6, CG7 (Top Chord)
	Dead + HA + Fway	SLS	1.77	CG3, CG5 (Lower Diagonal)
	Dead + HA + Fway	SLS	1.65	CG2 (Verticals)
	Dead + HA + Fway	SLS	1.10	CG2, CG3, CG5 (End diagonal)
	Dead + HA + Fway	SLS	1.69	CG8, CG11, CG21 (Lower diagonal)
	Dead + HA + Fway	SLS	1.61	CG10, CG20 (Verticals)
	Dead + HA + Fway	SLS	1.16	CG8, CG10, CG11, CG20, CG21 (End diagonal)
	Dead + HA + Fway	SLS	1.02	CG8, CG10, CG11,

				CG20, CG21 (Internal diagonal)
	Dead + HA + FWay	SLS	1.69	CG1 (Lower diagonal)
	Dead + HA + Fway	SLS	1.28	CG6, CG7 (Bottom chord)
	Dead + HA + Fway	SLS	1.37	CG6, CG7 (End diagonal)

7 Main Span Deck and Concrete Deck Assessment

7.1 Introduction and Method of Analysis

The orthotropic deck was analysed as a grillage, representing one 60 foot length road panel, to obtain longitudinal bending stresses. Finite element models were also created to assess the local transverse bending under the wheel loads.

The Stringers check does not include impact from crash barrier

7.2 Loading

In accordance with the AIP, the orthotropic deck including the longitudinal stringers was assessed for 45 units of HB loading. No other loads were applied at the same time.

7.3 Orthotropic Deck

The orthotropic deck on the main span consists of a flange plate ½ in thick and troughs 8 in deep and ¼ in thick. All plates are mild steel to BS15.

In the ICE Proceedings, Section 2.139 onwards describes load tests carried out by the Building Research Station on experimental panels made to an earlier design. The tests were carried out using a 11¼ ton HB wheel load which produced a local transverse stress of 14.3 ton/sq.in (220 N/mm2) based on 9/16 in deck plating and an unsupported width of plate of 15in. The final design used a ½ in. plate and an unsupported width of 13.5in (343mm) between troughs. The reduction in the unsupported width partly compensates for the reduction in the thickness of the plate and the stress level in the plates would be expected to exceed the test result of 220 N/mm2. The unsupported length of the deck plate increases to 360m adjacent to the stringers.

A finite element model was created of a complete 60 foot long deck panel and subjected to the nominal (unfactored) load from a 45 unit HB vehicle. The stringers tend to provide stiffer support to the deck plate and peak hogging moments occur over the stringers. The maximum unfactored peak stress obtained was 265N/mm2 (factored stress at SLS of 291 N/mm2) as a hogging moment over the stringers. However, a common feature of finite element models is that they do produce high local peaks and it is possible to average results over an appropriate length. In order to reduce the peak stress to the yield stress of 247 N/mm2, the stresses need to be averaged over a length of approximately 300 mm measured along the length of the bridge.

Under the Assessment Code, BD 56, it is stated in clause 9.10.3.2 that, provided a serviceability check is satisfactory, no account need be taken of local bending stresses when checking a stiffened flange at the ultimate limit state. It is likely that this clause was written for a box girder, for example, where the top flange will be stiffened to support the wheel loads. We have considered that the orthotropic deck is a stiffened flange of the road deck approximately 60 ft long and spanning between 3 cross girders.

For comparison, we have considered the orthotropic deck plate used on the Severn Suspension Bridge. On that bridge, the troughs are at 12 inch centres, 0.45 mm thick but are made of high yield steel. Subject to the same load, Severn Bridge would have a lower overstress index.

We have also carried out a check at ULS and at ULS, the maximum factored peak stress obtained in the LUSAS analysis is 344 N/mm2.

No account has been taken of composite action between the deck plate and the surfacing.

In summary, the deck plate is just adequate when analysed at Serviceability if the stresses are averaged over a length of approximately 300mm. At ULS, the deck plate appears to be overstressed when analysed using classic bending theory. However, it has been shown that, in practice, for large loads, membrane stresses are set up and the ultimate load capacity is in the order of 15-20 times the ultimate load computed in accordance with ordinary bending plate theory.

For the trough stiffeners, both SLS and ULS checks are required. The maximum overstress index at SLS is 0.96 and, at ULS is 0.88

The maximum overstress index for the stringers is 0.99.

The maximum overstress for the cross beams is 0.86.

7.4 Stringers

In the side span deck construction, each of the carriageways is comprised of four welded steel stringers (565 mm deep) composite with a 218mm thick concrete slab. The main span consists of four welded steel stringers (643mm deep) with the orthotropic deck spanning approximately 3m between steel cross beams. The Deck panels are supported on mild steel bearing plate on top of cross girders, which are approximately 9.65m apart. The deck panels are continuous over two spans with a halving joint at about 1.5m from cross girder at hanger locations.

The steel stringers were designed very economically as would be expected for a repetitious item and the maximum overstress index is 0.99.

7.5 Side Span Concrete Deck Assessment

The side span deck is comprised of a reinforced concrete slab, cast on a steel plate that is stiffened by inverted T-sections running longitudinally. It is reasonable to consider the concrete as a (one-way) slab that spans transversely between the “beams” that are effectively created by these stiffeners. For analysis and assessment of the concrete deck, we have idealised a section of the deck as a grillage model. By inspection, the critical load case is Combination 1 HA + HB; the relatively small transverse spans mean that HB wheel loads will give the most adverse bending and shear actions in the slab. This confirms the AIP produced by WAF that the deck should be checked using HB.

Table 7.1 Summary of Overstress Indices for Side Span Concrete Deck

Load Case	SLS or ULS	Overstress Index	Comment
Longitudinal Reinforcement	SLS	1.08	Edge Stringer
Longitudinal Reinforcement	SLS	0.77	Internal Stringer
Transverse Reinforcement	ULS	0.85	
Shear	ULS	0.59	
Punching Shear	ULS	0.31	

7.5.1 Summary of Loads Applied to Deck

7.5.1.1 Concrete Deck

According to reference 2.0.3, the density of placed and compacted concrete is 143.0lb/ft^3 and slab thickness is 8 7/16”. These correspond to 22.47kN/m^3 and 214.3mm in respectively in S.I. Units. The nominal dead load due to self-weight of concrete is therefore 4.82 kN/m^2. It is likely that that the concrete is not required to support its entire self-weight since the steel part of the deck effectively acts as permanent falsework. However, due to uncertainty of the effects of creep it is thought prudent to assume that the concrete is required to support its entire self weight.

7.5.1.2 Superimposed Dead Load from Surfacing:

1 ½" of asphalt was placed during the original construction. However, the surfacing has been replaced several times during the lifetime of the bridge. To account for the possibility of local variations in surfacing thickness the full γ_{fl} of 1.75 from Table 1 of BD37/01 has been used, whereas a reduced value of 1.20 have been used elsewhere in this assessment.

7.5.1.3 Live Loads

The deck was checked for HA + HB loads. Influence surface generation and live load optimisation was used to ascertain the most adverse configuration of live load.in a grillage model. These were enveloped to obtain the most onerous load effects. A finite element model was used to compute additional local sagging moments.

7.5.1.4 ULS Capacity of Slab: Bending

The following data and assumptions were used in the analysis of the slab:

- Minimum concrete cube strength f_{cu} (28 days) is 4,500psi. This equates to 31.0N/mm^2.
- Reinforcing bars are mild steel to BS785. f_y = 36,000psi which equates to 248 N/mm^2.
- Transverse bars are ¾" at 6" spacing, which equates to a reinforcement area (A_s) of 1870mm^2 /m top and bottom (refer as-constructed drawing ACD 4124A and Resident Engineers' report).

8 Assessment Summary

The deck structure for the bridge has been assessed in the first stage to load combinations including BSALL in association with appropriate wind loads. Wherever, apparent overstresses have been obtained we have provided an explanation wherever possible to justify accepting the overstress. In the second stage we have assessed the bridge to load combinations including HA in association with appropriate wind loads. The load combinations were therefore grouped as follows:

Stage One

- Dead + BSALL
- Dead + BSALL + 50mph wind
- Dead + Wind

Stage Two

- Dead + HA
- Dead + HA + Wind
- Dead + HB
-

The extent and magnitude of the overstresses are provided for the main steel members in Sections 5, 6 and 7 of this report. The extent of the overstresses is summarised in Figure 8.1 below:

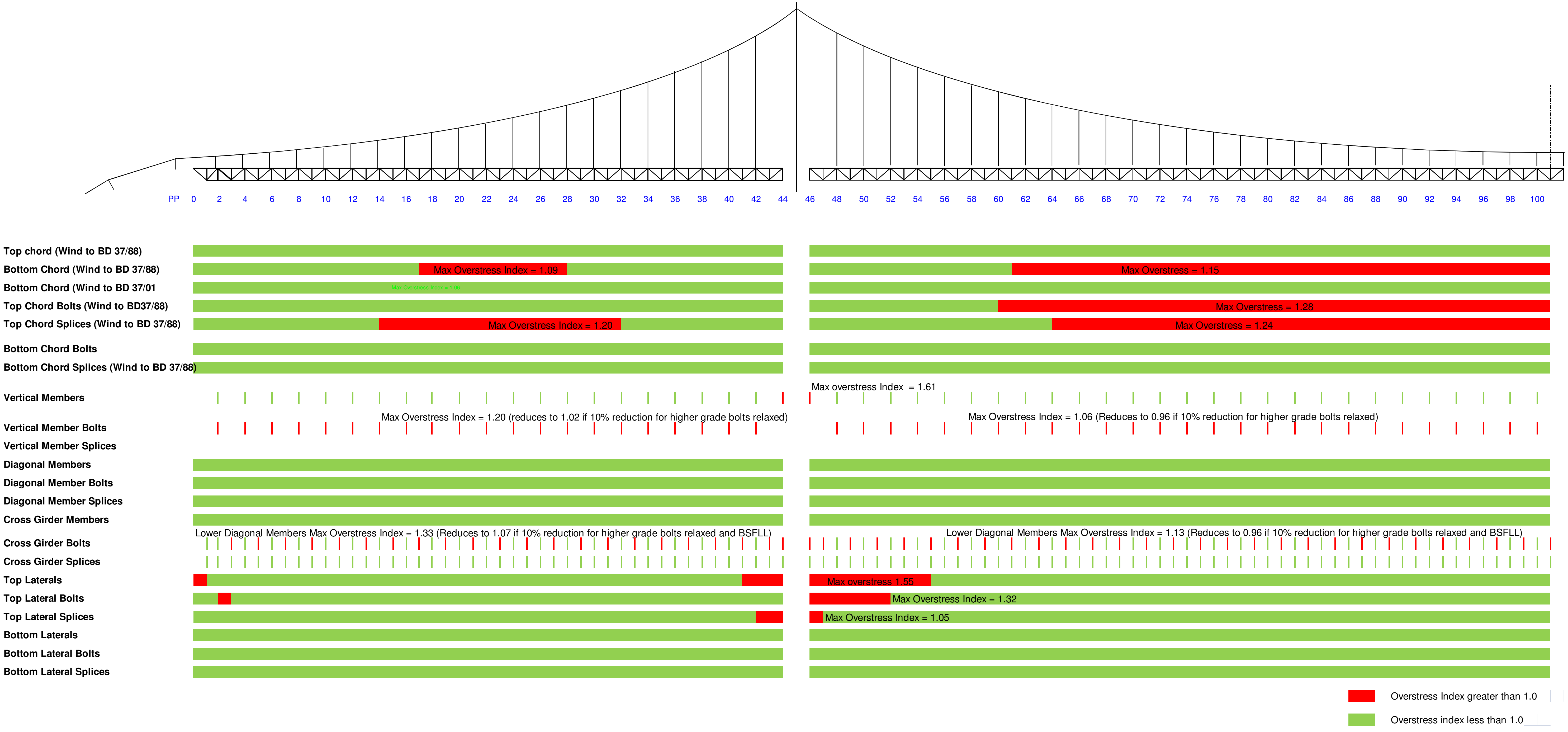


Figure 8.1 Summary Extent of Overstresses
(Stage One Load Cases)

Table 8.1 Summary of Percentage of Overstressed Elements

Stage One

Element	Main Members	Splice/ Gusset Plates	Bolts
Top Chords	0%	54%	39%
Bottom Chords	55% (0% Wind to BD 37/01	0%	0%
Diagonals	0%	0%	0%
Verticals	2%	0%	50% (overstress reduces to 1.02 if BSFLL used and relaxation of 10% reduction to section 5.2.2 of this report)
Top Lateral Bracing	10%	3%	7%
Bottom Lateral Bracing	0%	0%	0%
Cross Girders	0%	0%	5%

Stage Two

Element	Main Members	Splice/ Gusset Plates	Bolts
Top Chords	85%	90%	81%
Bottom Chords	80%	86%	79%
Diagonals	6%	0%	2%
Verticals	50%	0%	50%
Top Lateral Bracing	80%	18%	47%
Bottom Lateral Bracing	20%	0%	13%
Cross Girders	10%	7%	18%

From the above tables it can be seen that at Stage One, the top chords and diagonal main members are all adequate. If the Bottom Chord is re-analysed using BD 37/01, the bottom chord main members are adequate. A small percentage of the verticals, top lateral bracing and cross girder main members are overstressed. The bottom lateral members are adequate. A significant number of bottom chord and top chord splice plates are overstressed.

At Stage Two, significant numbers of all main members, gusset plates and bolts are overstressed.