Forth Estuary Transport Authority

FAIRHURST

FORTH ESTUARY TRANSPORT AUTHORITY

68952B

STRENGTHENING OF TRUSS END CONNECTIONS

Handbook for

DESIGN REVIEW WORKSHOP

16 December 2008

Revision 01

CONTENTS

1.0 INTRODUCTION

Assessments of the suspended structure and truss end connections undertaken by W. A. Fairhurst & Partners have identified that several of the key elements forming these connection between the stiffening truss and the main towers have overstress indices greater than 1.0. W. A. Fairhurst & Partners have investigated the options available to strengthening these elements.

The elements that have overstress indices greater than 1.0 are:

Main plates of the tower bracket, externally and internally
Welds connection the bracket to the tower
Stiffening Truss end post member
Welds connecting the end post to the top chord gusset plate.

This workbook summaries the results of the assessment of the stiffening truss end connections previously undertaken, for the purpose of discussion at the workshop only.

2.0 OBJECTIVE OF THE WORKSHOP

The object of this workshop is to review the proposals for the remedial works to strengthen the connection between the main towers and stiffening truss and establish the following:

- Define the scope of works
- Establish the level of strengthening required
- Determine if further analysis is required
- Select an option to be developed under detail design.

ATTENDEES

The following individuals have been invited to attend the meeting.

Barry Colford	Forth Estuary Transport Authority
	Forth Estuary Transport Authority
	Forth Estuary Transport Authority
Colin Clark	W. A. Fairhurst & Partners
	W. A. Fairhurst & Partners
	W. A. Fairhurst & Partners
	W. A. Fairhurst & Partners

AGENDA

• Review of existing layout and defects	09:00pm
• Options review	09:15pm
• Bracket Solutions	09:30am
• End Post Solutions	10:30pm
• Summary Discussion	11:30pm
• Way Forward	11:45pm
• Lunch	12:00pm

3.0 DESCRIPTION OF EXISTING CONFIGURATION

3.1 Original Construction Details

The links connecting the main towers to the ends of the stiffening trusses are formed from mild steel H sections. The link embers are connected to the bottom chord of the truss and to a pair of cantilevered support brackets from the main towers. These connections are formed with pins made from high tensile steel. Details of the links are shown in Figure 1,2 & 3

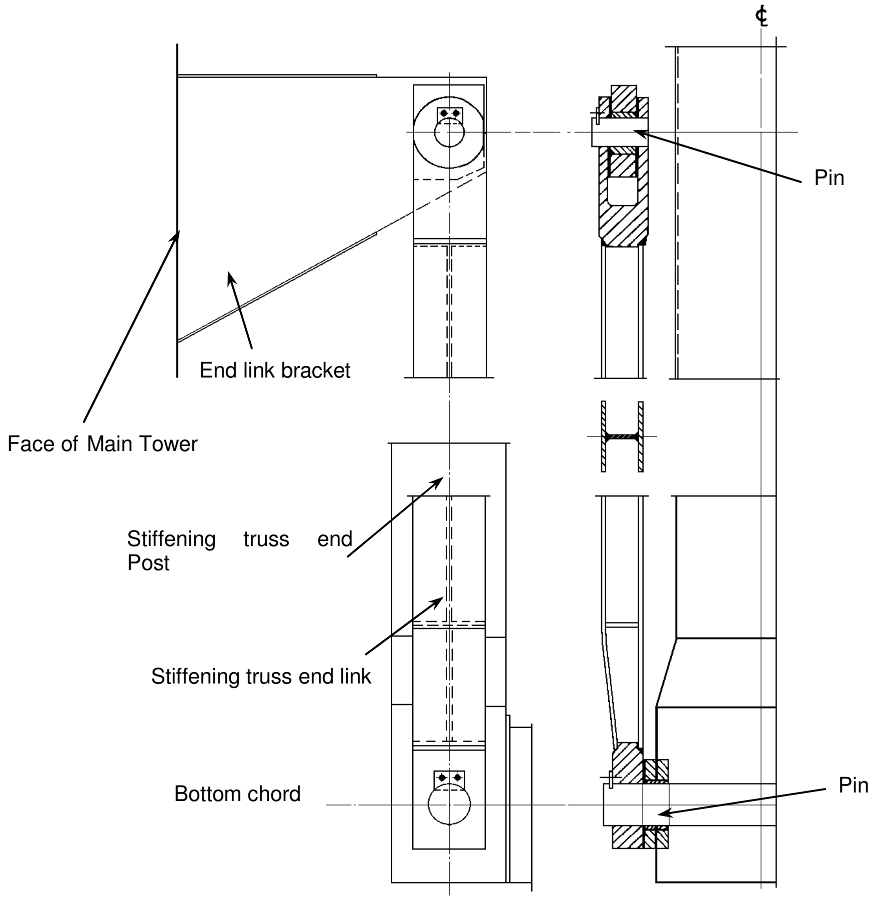


Figure 1 – Stiffening Truss End Link connection to Tower

Figure 2 – Stiffening Truss End Post and Link

Figure 3 – Main Tower End Link Brackets

A general arrangement drawing showing the area local to this connection is contained in Appendix B.

The link members are not designed to resist lateral forces. These forces are transferred to the main towers by the lateral thrust members. Sliding bearings which form part of the arrangement restrain the lateral loads but allow the trusses to move longitudinally in the axis of the bridge.

The stiffening truss end posts are formed from a steel box section, the dimensions of the main plates forming the end post are:

Webs (parallel to length of truss)	457mm * 11.1mm
Flanges	708mm * 9.5mm

The end post widens out at its base where the pinned connection to the link members is made.

It is understood that this detail has not been modified since the time of the original construction.

3.2 Current Loading

Under the critical load case of Dead + Live Load (HA) at ultimate limit state a load of 4.82 Meganewtons is transferred from the end post through the link members to the pair of tower brackets. The load transferred through to the side span brackets is slightly less.

The critical load case for the connection is HA or BSALL applied to the single carriageway over 40 bays from one tower and the opposite carriageway loaded for the remainder of the loaded length from the opposite tower.

The split of load is as follows:

Dead (ULS)		0.87MN	(18.0% of total load)
Live Load	HA Load (ULS)	3.95MN	(82.0% of total load)
Total Load (ULS)		4.82MN	

When considering BSALL loading in place of HA loads the total load carried through the connection reduces to 3.80 Meganewtons. The split of load is as follows:

Dead (ULS)		0.87MN	(22.9% of total load)
Live Load	BSALL (ULS)	2.93MN	(77.1% of total load)
Total Load (UL:S)		3.80MN	

4.0 SUMMARY OF ASSESSMENT FINDINGS

4.1 Bracket Assessment Results

The results of the assessment of the brackets are given in the following reports prepared by W. A. Fairhurst & Partners: Assessment of Connections Between Stiffening Truss and Main and Side Towers Dated March 2008 and the addendum Report titled: Assessment of End Link Brackets At Main Towers dated September 2008. A summary of the finding of these reports is given below for the brackets on the main span.

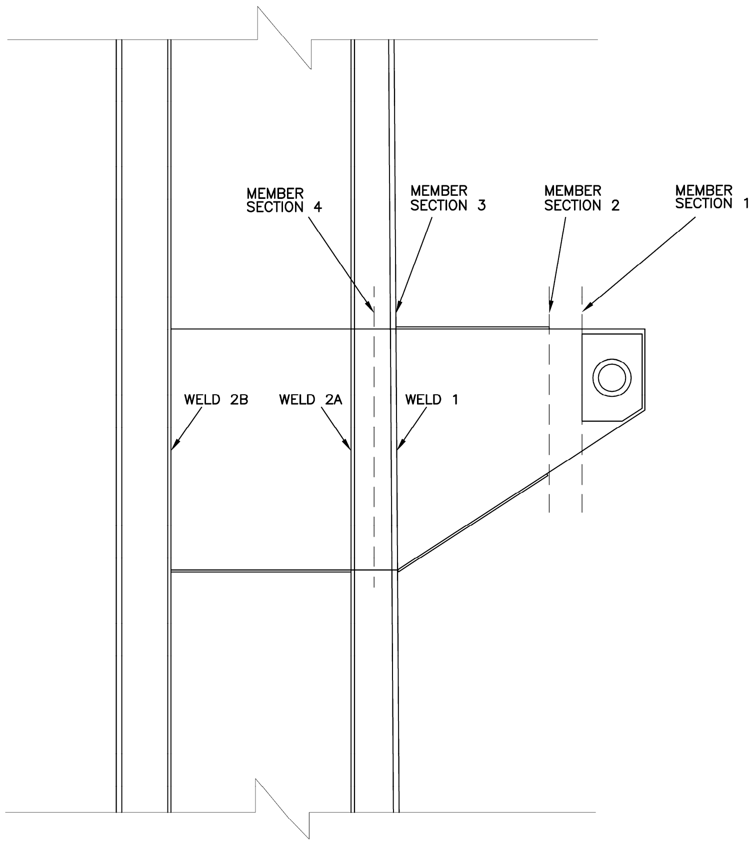


Figure 4 – Main Tower End Link Brackets

The End Link Brackets

Loadcase	OI (Section 1)	OI (Section 2)	OI (Section 3)	OI (Section 4)	OI Max
Dead ULS	0.22	0.26	0.23	0.25	0.26
Dead+Wind (transverse) ULS	0.30	0.36	0.32	0.35	0.36
Dead+Wind (longitudinal) ULS	0.34	0.40	0.35	0.39	0.40
Dead+Wind (transverse)+Live ULS	1.08	1.28	1.13	1.26	1.28
Dead+Wind longitudinal)+Live ULS	1.09	1.29	1.14	1.27	1.29
Dead+Live ULS	1.20	1.42	1.25	1.39	1.42
Dead+BSALL ULS	0.95	1.12	0.99	1.10	1.12

Table 1 – Summary of Overstress Indices (Main Span Brackets – members)

Welds

Loadcase	OI (Weld 1)	OI (Weld 2A)	OI (Weld 2B)	OI Max
Dead ULS	0.36	0.35	0.35	0.36
Dead+Wind (transverse) ULS	0.51	0.49	0.49	0.51
Dead+Wind (longitudinal) ULS	0.57	0.55	0.55	0.57
Dead+Wind (transverse)+Live ULS	1.82	1.76	1.76	1.82
Dead+Wind longitudinal)+Live ULS	1.84	1.78	1.78	1.84
Dead+Live ULS	2.02	1.95	1.95	2.02
Dead+BSALL ULS	1.60	1.54	1.54	1.60

Table 2 – Summary of Overstress Indices (Main Span Brackets - welds)

The side span brackets are shorter than those on the main span and therefore the load effects are reduced. Under Dead + Live (HA at ULS the critical Overstress index is 1.02 for the brackets. However the overstress indices for the welds between the tower and the bracket remain high. A summary of the overstress indices for the welds is given below.

Loadcase	OI (Weld 1)	OI (Weld 2A)	OI (Weld 2B)	OI Max
Dead ULS	0.29	0.24	0.24	0.29
Dead+Wind (transverse) ULS	0.36	0.30	0.30	0.36
Dead+Wind (longitudinal) ULS	0.42	0.35	0.35	0.42
Dead+Wind (transverse)+Live ULS	1.54	1.28	1.28	1.54
Dead+Wind longitudinal)+Live ULS	1.54	1.28	1.28	1.54
Dead+Live ULS	1.71	1.42	1.42	1.71
Dead+BSALL ULS	1.19	0.99	0.99	1.19

Table 3 – Summary of Overstress Indices (Side Span Brackets - welds)

4.2 Bracket Failure Reasons

The cantilever part of the bracket primarily fails as a result of buckling as there is no lateral restrain to the member. This is true for all the sections with the exception of section2 which is the point where the top and bottom flanges are curtailed.

The effect of buckling reduces the capacity of the member by between 20% - 36%. However at the critical section the reduction is approximately 20%. Therefore there is an underlying problem of insufficient member strength at this section 2; the point at which the flanges are curtailed.

At section 4 the bracket passes through a slot in the main tower plates and stiffener, there are no flanges at this section as a result the load carried by the flanges is transferred to the rectilinear section of plate causing stresses greater than permitted under yield checks. This section of the bracket is restrained from bucking by the main tower plates and stiffeners.

The welds fail as a result of insufficient weld material to carry the applied loads. The bracket pivots about the main plate of the tower where its welded to the plates on one side only (weld 1) as the back face is concealed by the tower stiffeners. The weld between the inner cell plate and the bracket (Weld 2) also fails as a result of insufficient weld material to carry the load. This is an intermittent weld and as a result has approximately half the capacity of weld 1.

4.3 Truss End Post Assessment Results

The results of the assessment of the end post are detailed in W A Fairhurst & Partners report titled Stiffening Truss Assessment Report dated May 2008. An extract of the overstress indices for the end post is given below. It should be noted that the overstress indices for the end post at panel point 44 are only slightly less than the values below.

Vertical Element	Loadcase	Limit State	Overstress Indices	Panel Point	Load Effect
Main Members	Dead + Wind	ULS SLS	0.41 0.28	46 46	C C
	Dead + HA	ULS SLS	1.65 1.16	46 46	C C
	Dead + BSALL	ULS SLS	1.34 0.95	46 46	C C

Table 4 – Summary of Overstress Indices Main Span End Posts

The overstress index for the main member for the end posts on the side span truss, under Dead + HA at ULS is 1.57. Under BSALL this drops to 1.28.

The welds that connect the end post to the gusset plate on the top chord are overstressed. The overstress index under the critical load case of Dead + Live at ULS is 1.23. When BSALL loading is considered this value drops to 0.93.

4.4 End Post Failure Reasons

The plates that form the end post are relatively thin in comparison to the width of plate. Under current standards the effective area of the plate to be used in determining its capacity to carry load is significantly lower than the gross area of the plate. Only 43% of the flange plates are considered effective whilst 67% of the web plates are effective. As a result the effective area used in the assessment is only 53.3% of the gross area of the member

The reduction in strength of the member as a result of overall buckling due to member geometry is approximately 7%.

Therefore it can be seen that overall buckling of the member is not the main reason that the overstress indices are high. The main reason for the high overstress indices is the plates forming the member are considered to be slender and such the member is only 53% effective. Either additional cross sectional area should be provided or the existing plates stiffened to enable a greater proportion of the gross are to be considered effective.

The welds between the end post and the gusset fail as a result of insufficient weld material to carry the load. The length of the weld can not be extended therefore the weld would have to be increased in size to reduce stresses.

4.0 CONSTRUCTION CONSTRAINS

There are several construction constraints that limit options to strengthen the brackets and end posts these include:

- External access is limited to the bracket due to the position and movement of the stiffening truss.
- Limited space to attach strengthening works
- Limited space and access within the towers to strengthen the welds
- Part of the bracket is concealed by the tower main plate stiffeners
- Limited access to the faces of the end post.
- No alternative load path.
- Elements of the tower are already highly stressed
- Bridge to remain open during works
- Limited access as a result of ongoing / planned woks. *(Dropped Object Canopy/ DEMAG joint replacement).*

The main constraint to undertaking strengthening of the brackets and end post is the limited amount of access to the connection.

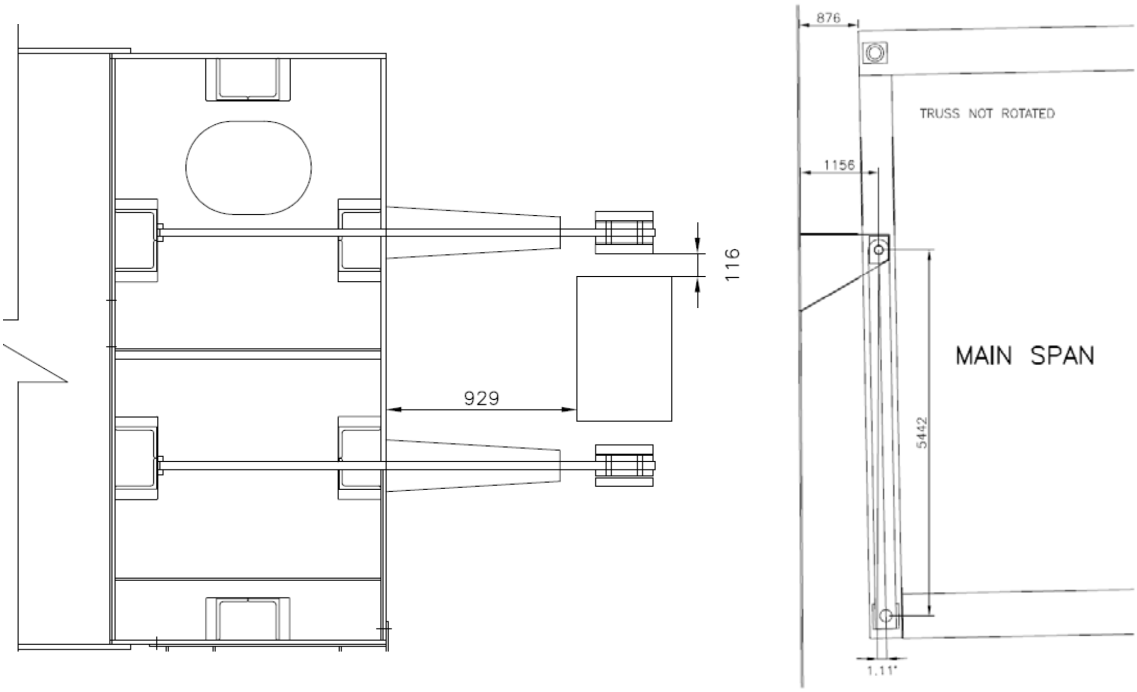


Figure 5 – Starting Position of Stiffening Truss Relative to Towers

The arrangement of the connection allows the stiffening truss to move longitudinally through either thermal expansion / contraction or under the action of applied wind / traffic loading.

Under 50mph wind loading the calculated longitudinal movement of the stiffening truss is 279mm. This range of movement increases to 607mm when wind gusting at 78mph wind is considered.

The maximum longitudinal movement of the stiffening truss under live loading is 553mm in either direction. The load case causing this movement is when one side span is fully loaded with live load which pulls the main cable tight causing the main span stiffening truss to move toward the tower.

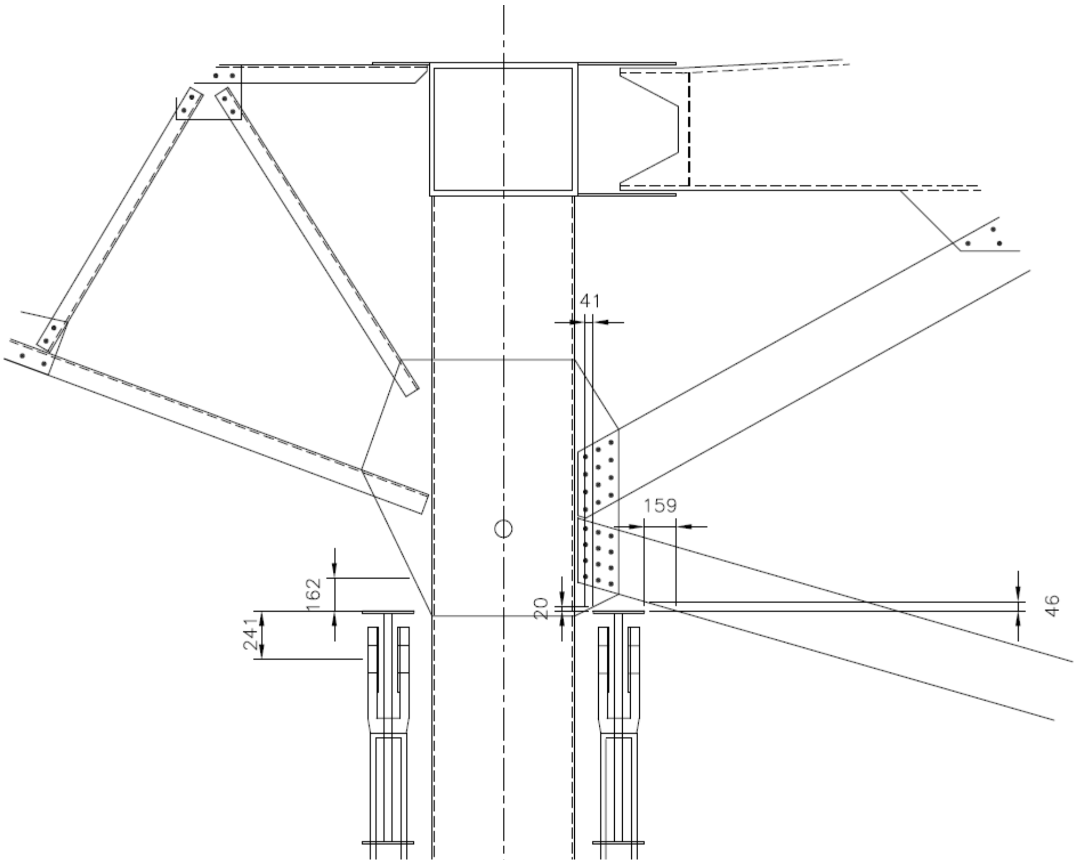


Figure 6 –Relationship between top of bracket and side span cross girder. Main span similar

Due to the movement of the stiffening truss it can be seen that the available space between the face of the main tower and the stiffening truss is severely limited. We consider that it would be unsafe for an operative to undertake remedial work in this area between the brackets or above the top of the brackets.

For the same reason detailed above there is limited amount of space available to strengthen the tower bracket. Fixing additional steelwork to the top of the bracket is considered not feasible as it would clash with the stiffening truss.

Fixing additional steel to the bottom of the bracket can only be taken up to a limited point after which there would be a clash with the link members.

Figure 7 – View on bracket from within the tower, inner cell of the tower is topmost in the photograph

Access within the tower to undertake strengthening work is consider feasible although space and access is limited. Working in this location will be considered to be confined space working.

It can be seen in Figure 7 that the brackets pass though the tower plate stiffeners and are therefore currently inaccessible. Strengthening of the stiffeners would be required to enable access slots / holes to be cut in order to strengthen the concealed section of bracket.

5.0 STRENGTHENING STRATEGIES

There are 5 strengthening strategies that have been considered, these are:

- Undertake strengthening of accessible elements
- Retain existing load path and strengthen members
- Retain part of the existing load path, strengthen members and provide new details
- Provide new completely load path and details
- Reduce loading on the connection

5.1 Undertake strengthening of accessible elements

To undertake strengthening to the accessible elements only is not considered to be a suitable option. The construction constraints that affect the connection would result in a very limited amount of strengthening being undertaken which would not address the problems identified in the assessments.

We consider that the weld between the tower and the bracket could be strengthened from with in the tower. This would require the main stiffeners to be strengthened such that an access hole could be cut through the stiffener at the bracket enabling a run of weld to be made to between the inner face of the tower and the bracket.

The welds between the inner cells of the tower and the bracket can be strengthened relatively easily by infilling the “misses” of the intermittent weld thereby creating a continuous run of weld.

It may also be possible to strengthen part of the bracket from within the tower. The section of bracket that is currently concealed by the stiffener could be strengthened by welding on flange. This would again require the stiffener to be strengthened and an access hole cut. Careful detailing would be required to allow welding on to the existing high strength steel plates.

5.2 Retain existing load path and strengthen elements

By retaining the existing load path all the elements identified as being overstressed will require to be strengthened or replaced.

In order to strengthen the bracket and end post it is considered necessary for the link arms to be removed in order to provide sufficient access of the elements. Likewise complete replacement of the exiting bracket will require the link arms to be removed.

Removal of the link arms will therefore require temporary works to provide a new load path. Details of the proposed temporary bracket are shown in figure 8.

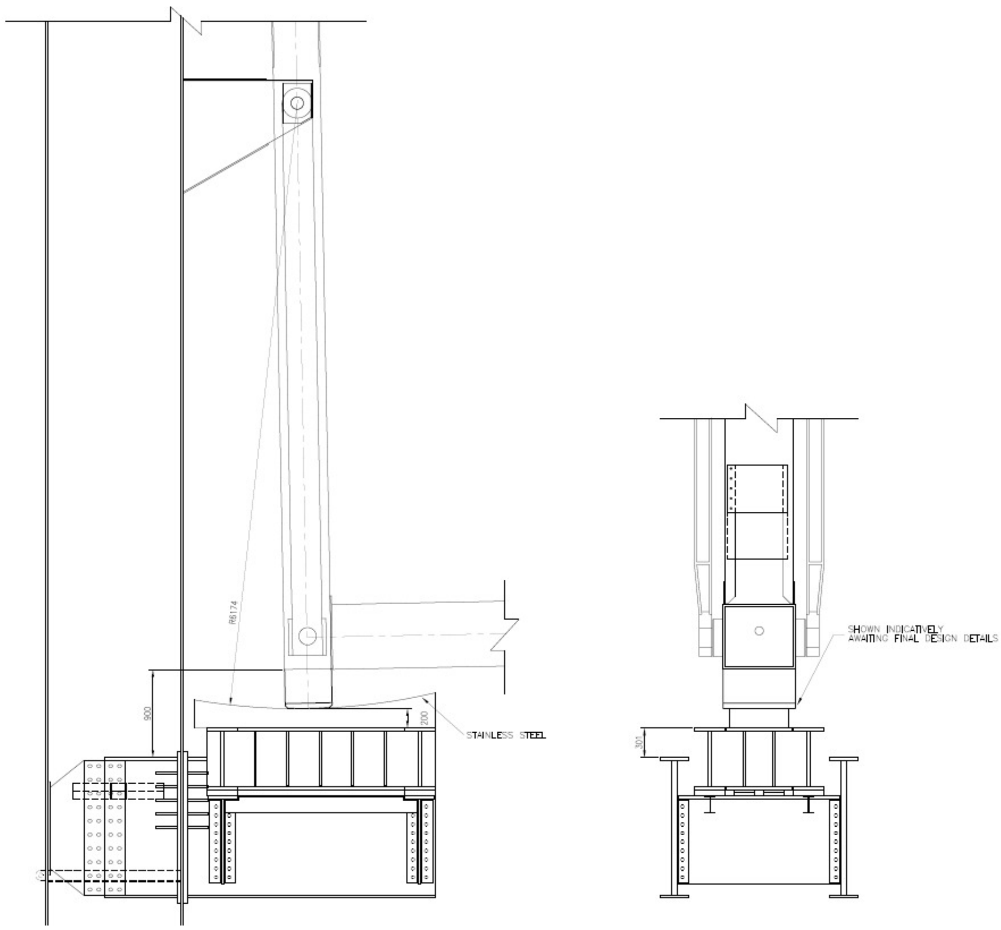


Figure 8 – Proposed Temporary Bracket

Details of the options considered for strengthening the individual components are contained in Appendix A

The most feasible options for strengthening the brackets is with a combination of supplementary web plates and lateral restraints. The supplementary web plates would be attached to the outer faces of the brackets only.

Alternatively the existing bracket could be completely replaced.

The end post would require to be strengthened, options considered for strengthening the end post include the provision of stiffeners to maximise the area of the existing member that can be considered effective.

5.3 Retain part of the existing load path and strengthen

This option is similar to the above option however rather than strengthen or replace the existing brackets the temporary brackets could be utilised as the permanent solution. The end post would still require to be strengthened.

The existing brackets and links would be retained to resist uplift forces.

The bracket requires to be sized to accommodate the longitudinal and associated vertical movement of the truss. This will require a curved bearing plate to be machined.

The major limitation of this solution is the issue of maintenance associated with the bearing.

5.4 Provide new load path and details

This option would require the load from the stiffening truss to be transferred to the tower through a series of new details. The load would be transferred from the connection at the top chord of the stiffening truss. This could be achieved by providing a new hanger or link at panel points 44 and 46 to carry the load.

New hangers / links would have to be supported from brackets fixed to the tower at a higher level. In the case of a new hanger there is insufficient space to adopt the cable band type connection utilised else where as a result of the main cable sleeve at the tower saddle. The position of the new bracket would have to be located above the tower splice plates between portions four and five of the towers.

The top chord of the truss at this location would require to be strengthened in order to connect the hangers / links.

A significant amount of work would be required to relocate the electrical distribution boxes which are located in the vicinity of the proposed connection point.

Figure 9 – Existing Electrical Distribution Box at Main Towers

This option would noticeably alter the appearance of the bridge.

Transfer of load from the existing arrangement to the new hangers / links would be complicated due to the differences in stiffness of the two load paths.

5.5 Reduce loading on the connection

This option would require load to be redistribute to the adjacent hangers by adjusting the length of the hangers. However the amount of load required to be redistributed is consider too high for this to be feasible.

6.0 OUTCOME OF WORKSHOP

The section of the workshop handbook details the discussions and the decisions that were made during the workshop in order to progress the project.

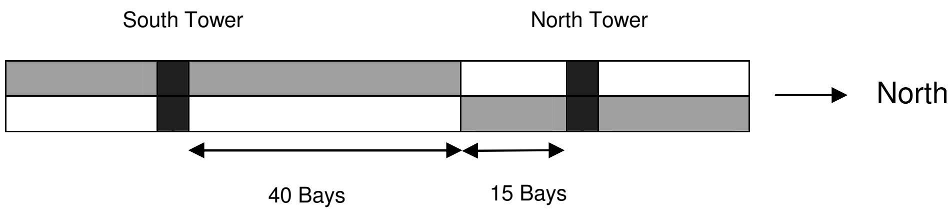
6.1 Additional Load Cases to be Considered

FETA requested that further load cases were analysed to establish the effects on the truss end link connections. The critical load case previously identified in section 3.2 is considered very unlikely top occur although it is required to be included in the assessment of the connections.

The following load cases are to be analysed that are considered to represent a more realistic configuration of traffic. The additional load cases are also considered to represent a scenario where a new Forth crossing is constructed and loading on the existing bridge is restricted.

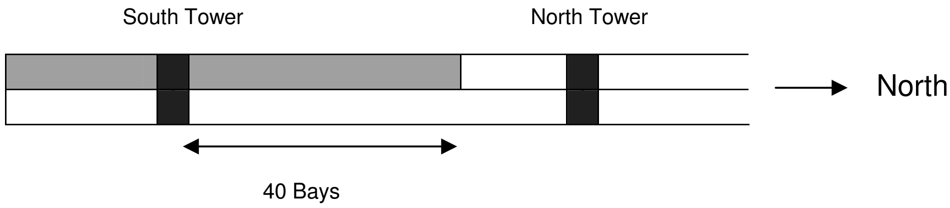
1. & 2. Dead + Live: - Full BSALL and also BSALL with 3.5 tonne restriction

North bound carriageway: South side span + 40 bays loaded from South tower on the main span.
South bound carriageway: 15 bays of the main span loaded from the North tower and north side span loaded.



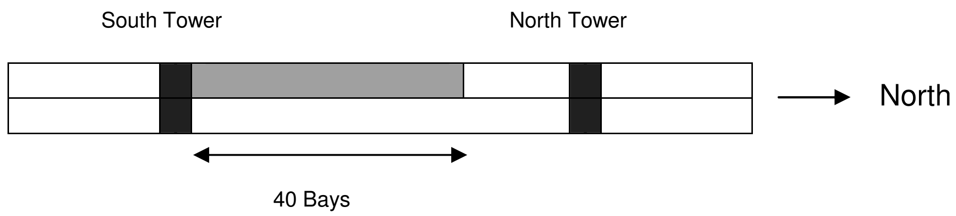
3. & 4. Dead + Live: - Full BSALL and also BSALL with 3.5 tonne restriction

North bound carriageway, south side span + 40 bays loaded from South tower on the main span.
South bound carriageway closed to traffic.

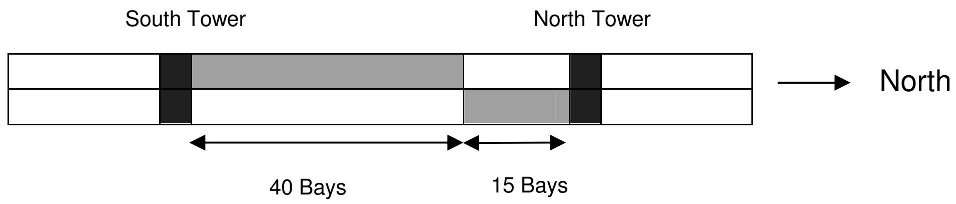


5. & 6. Dead + Live:- Full BSALL and also BSALL with 3.5 tonne restriction

North bound carriageway: 40 bays loaded from South tower on the main span.
Southbound carriageway closed to traffic



7. Critical load case Dead + Live:- BSALL with 3.5 tonne restriction.



W. A. Fairhurst & Partners are to supplement the previous assessment report with the out come of the additional load cases analysed.

W. A. Fairhurst & Partners to consider a load case comprising buses only, frequency of buses is around 20 buses per hour, and if greater than BSALL loading with 3.5 tonne restriction apply to the above load cases and include in the supplement to the previous assessment report.

6.2 Comparison of BSALL with Original Design Loading

During the workshop a comparison was made between the original design loading for the Forth Road Bridge and the Bridge Specific Assessment Live Loading (BSALL) calculated based on 2005 data. A comparison of these loads have been made in W. A. Fairhurst & Partners' report titled Bridge Specific Live Loading Report Dated June 2066.

The total BSALL loading, including lane factors, for the critical case for the main span, loaded length of 1006 metres, is 29.08 kN/m.

The total live loading for the critical case for the main span, based on the original design loading criteria, BS 153: 1954 is 15.54 kN/m.

The BSALL is 87% greater than the original design loading for the critical case for the truss end connection for the main span. Live loading amounts to approximately 77% of the total load applied to the stiffening truss end link connection.

A similar comparison was made for the critical load case for the truss end connection of the side span, where the critical loaded length is 408 metres. This comparison shows that the BSALL is approximately 32% greater than the original design loading.

6.3 Strengthening of End Link Connections

All strengthening design to be undertaken to full HA loading.

Initial discussions proposed the creation of a new load path for the main span connection only and locally strengthen the welds of the side span brackets. However this approach does not to cognisance of the requirement to strengthen the end posts of the side span stiffening trusses.

W. A. Fairhurst & Partners should proceed with the option to strengthen the truss end links of both main span and side span by creating a new load path between the truss and main towers as detailed in Section 5.4. The new arrangement will be located above deck level and will comprise of a bracket connected to the towers that supports a link member connected to the top chord of the stiffening truss.

Local strengthening of the tower and stiffening truss will be required and the existing services that are currently located around the towers will require to be relocated.

W. A. Fairhurst & Partners are to prepare a cost estimate and programme for undertaking this work.

W A Fairhurst & Partners
Glasgow

GAM/68952B 28 January 2009

APPENDIX A

COMPONENT STRENGTHENING OPTIONS

Sections highlighted represent the decisions made during the workshop regarding the method of strengthening the connections.

COMPONENT STRENGTHENING

Element	Remedial measure	Limitations / Drawbacks	Feasibility
External Section of Bracket	Reduce loading	• Alternative load path required / restrictions on bridge.	Feasible
	Increase section modulus by deepening section at top flange	• No scope to increase depth of section at top flange due to movement of truss, clash with cross girder • Severely limited access to make attachments to bracket.	No feasible
	Increase section modulus by deepening section at bottom	• Limited access to make attachments to bracket, only one side of each bracket is consider to have any safe access • No scope to increase depth at section 2-2 as links would clash in permanent condition. Can not make a weld / connection at section 2-2 • Difficulties in welding single sided butt weld in thick plate.	Not feasible to strengthen entire member
	Increase section modulus by web plates	• Access to install web plates limited by range of movement of truss. • Web plates can only be provided on one side of bracket. • Web plate can only extend to a certain distance unless link arm is remove which will require temporary supports. • Reattaching link arms may be difficult. • Require thick weld plate ~ 25mm maximum plate thickness 31.75mm • Eccentric loading on plate require to be stabilised require to cut a slot in the main tower plates and stiffeners. •	Feasible
	Provide Lateral restraint and increase section modulus	• Lateral restrain on its own is not sufficient for the entire member. • Require to enhance section modulus of section 2-2 refer to above • Access to install lateral restraint is limited by movement of the truss.	Feasible

Element	Remedial measure	Limitations / Drawbacks	Feasibility
External Section of Bracket Continued	Provide a new bracket	• Limited access to install new bracket, only one side of each bracket is consider to have any safe access, severely limited space • Difficultly in providing a sealing weld between the outer face of the tower and the bracket. • Requires link arms to be removed. • Depth of section limited by position of cross girder gusset and diaphragm in tower. • May require to modify link arms to suit configuration of bracket.	Feasible

Element	Remedial measure	Limitations / drawbacks	Feasibility
Internal section of bracket	Reduce loading	• Alternative load path required / restrictions on bridge.	Feasible
	Increase section modulus by deepening section at op flange	• Need to be compatible with strengthening option for external section of bracket.	Not feasible
	Increase section modulus by web plates	• Need to be compatible with strengthening option for external section of bracket. • Require to cut a slot in tower stiffeners and main plates, which will require strengthening prior to works. • Locked in dead load stresses. Web plates to be sized to accommodate this. • Confined space working	feasible
	Increase section modulus by providing flanges	• Require to cut access slot in existing stiffeners. • Require to weld to main tower plates for continuity of load path. • Locked in dead load stresses, flanges will require to be larger than those on the external part of the bracket. • Confined space working.	Feasible
	Provide new bracket	• Require to strengthen and cut existing tower plate stiffeners to gain access. • Confined space working. • Significant temporary works required.	Feasible

Element	Remedial measure	Limitations / drawbacks	Feasiblity
Bracket Welds	Reduce loading	• Alternative load path required / restrictions on bridge.	Feasible
Weld 1	Provide additional weld between outer face of tower and bracket	• Limited access to undertake this option. • No access to weld on inner faces of the pair of brackets, limited access to outer face welds. • Welding to high strength steel.	Not Feasible
	Provide additional weld between inner face of tower and bracket.	• Area is currently concealed by stiffener which will require to be strengthened and cut open. • Welding to high strength steel. • Confined space working.	Feasible
	Enhance connection by bolted arrangement	• Access to install bolts limited. • Insufficient space to fix new steelwork to top flange; Section would required to be attached to the bottom of bracket. • Bolted connection would be eccentric as no access to inner face of brackets. • Confined space working.	Feasible
Weld 2 a & B	Strengthen existing intermittent weld into a continuous line of weld	• Limited access within tower. • Welding to high strength steel. • Confined space working.	Feasible

Element	Remedial measure	Limitations / drawbacks	Feasible
End Posts	Reduce loading	• Alternative load path required / restrictions on bridge.	Feasible
	Reduce effective length of member	• Not sufficient increase in member strength 7% max.	-
	Reduce effective width of plates by providing external stiffeners and ring stiffeners.	• 2 web are inaccessible due to link members, link members would require to be removed to enable stiffeners to be fixed. • Back face of end post inaccessible due to movement of the truss. • Difficult to access area of end post between brackets. • Welding to high strength steel. • Limited depth available for stiffeners on webs.	Feasible
	Reduce effective width of plates by providing Internal stiffeners	• Require to cut large access holes on the open flange to enable lengths of stiffeners and diaphragms to be installed. • Welding to high strength steel. • Load distribution.	Feasible
	Reduce effective width of section by providing ring stiffeners	• Limited access to back face of end post. • Stiffeners required to be provided at close spacing. • Diffiult to access area of end post between brackets.	Not Feasible
	Increase thickness of plate on end post by adding supplementary plates	• Difficult to access area of end post between brackets. • Load distribution.	Feasible
	Fill end post with material to limit buckling of plates	• Require material to restrain buckling in inward and outward directions. • External ties / ring stiffeners may still be required.	

Element	Remedial measure	Limitations / drawbacks	Feasible
End Post Welds	Do Nothing	• Overstress index at BSALL ULS is below 1.0	-
	Reduce loading	• Alternative load path required	Feasible
	Increase weld size	• Access to weld on back face of end post is severely restricted.	Feasible

Element	Remedial measure	Limitations / drawbacks	Feasible
Temporary Support Bracket	Bracket and bearing supporting bottom of the end post	• Maintenance detail, although short term • Modifications required to end post at bottom chord	Feasible
	Hanger / Link	• A new hanger would require to be connected to the top chord. • Strengthening of top chord connection required • Working above carriageway, closures required • Space in outer cells reduces with the height of the tower.	Feasible

Both options require a similar bracket to be installed on the face of the tower which will require confined space working.

APPENDIX B

GENERAL ARRANGEMENT DRAWING

Rev.	Date	Description	Drawn	Checked	Approved

NOTES:

1. DIMENSIONS IN MILLIMETERS U.N.O.
2. DO NOT SCALE USE ONLY NOTED DIMENSIONS.
3. IT IS THE CONTRACTOR'S RESPONSIBILTY TO VERIFY ALL DETAILS SHOWN ON OR REFERRED TO BY THIS DRAWING WHERE DIFFERENCES ARE FOUND FROM THE INFORMATION PROVIDED THE CONTRACTOR WILL NOTIFY THE ENGINEER.

Client: FORTH ESTUARY TRANSPORT AUTHORITY

Project Title: FORTH ROAD BRIDGE STRENGTHENING OF STIFFENING TRUSS CONNECTIONS TO MAIN/SIDE TOWERS

Drawing Title: GENERAL ARRANGEMENT OF EXISTING STRUCTURE LOCAL TO MAIN TOWERS

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Scale at A0: AS SHOWN	Status:	CAD File Name: ps0001-0005
Drawn: N.Mc	Checked:	Approved:
Date: 16/04/08	Date:	Date:
Drawing No.: 68952/0002		Revision: –