FORTH ESTUARY TRANSPORT AUTHORITY

FORTH ROAD BRIDGE

STRENGTHENING OF TRUSS END CONNECTIONS

FEASIBILITY STUDY OF PREFERRED OPTION

December 2009

Forth Estuary Transport Authority
Administration Block
South Queensferry
EH30 9SF

W. A. Fairhurst & Partners
225 Bath Street
Glasgow
G2 4GZ

CONTROL SHEET

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1. INTRODUCTION

W. A. Fairhurst & Partners were requested by the Forth Estuary Transport Authority (FETA) to develop an outline design for the strengthening / replacement of the existing connections between the stiffening truss and the main towers of the Forth Road Bridge.

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 connections between the stiffening truss and the main towers have overstress indices greater than 1.0.

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 design review workshop was held on 16 December 2008 at which several strengthening strategies were presented and discussed. The out come of the design review workshop was that a strengthening option that did not require the existing end post to be strengthened and that provided a new load path between the stiffening truss and the main towers should be developed further. A copy of the completed design review workshop handbook is contained in Appendix A.

In addition to the development of the preferred option the workshop identified further loading scenarios, which were considered as having a reasonable probability of occurring. These additional scenarios were to be analysed and the effects on the existing connection summarised. An updated summary of the assessment findings is detailed in this report. One particular aspect highlighted by FETA following the design review workshop was the effect that footway loading has on the forces carried by truss end connections. Fairhurst were requested to review the impact of footway loading on these connections.

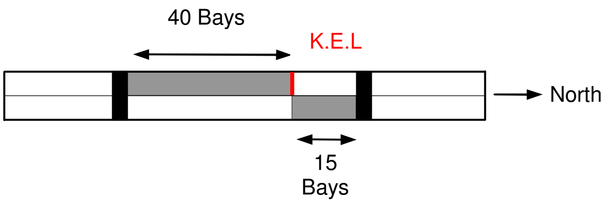
This report details the development of the preferred option including the effects that the preferred arrangement will have on the existing structure. Particular construction constraints and hazards identified during the study are also highlighted and details which will require particular attention at detailed design stage.

Consideration has also been given to the possibility of the new arrangement being utilised to support future works on and around the main towers of the bridge.

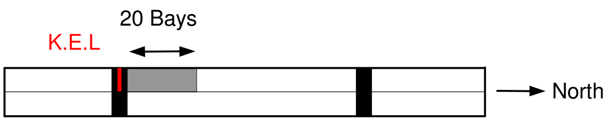
2. ADDITIONAL LOAD CASES

Additional loaded lengths identified during the design review workshop, which have a reasonable probability of occurring on the bridge and had not previously been analysed were undertaken. Generally it was found that torque cases resulted in the greatest end link forces being obtained. However although compliant with the code several of these load cases were considered to have a very small probability of occurring. To form the necessary loaded lengths in the position identified in the table in Appendix B would require incidents to occur on the structure concurrently. The effect of the different loaded lengths and position at which they start and stop can be seen in the table contained in Appendix B.

The previously identified critical load combination was a torque load case where 40 bays of the main span were loaded on the north bound carriageway and the remainder of the main span was loaded on the south bound carriageway. Footway loading is considered as occurring at the same location as the carriageway loading. The end link force under this arrangement of load is 4.82MN at Ultimate Limit State.



However following the additional analysis undertaken a load case was identified that gave higher end link forces than the load case above. The arrangement of load giving the highest end link forces is when 20 bays (362m) of the main span are loaded adjacent to one of the main towers, with no load on the opposite carriageway. Footway loading is restricted to a single footway adjacent to the loaded carriageway. The arrangement of loading is shown below.



This load case results in an end link force of 6.51MN at Ultimate Limit State.

During the assessment of the further load cases consideration was given to the footway loading considered and how much of an impact this loading has on the end link force, critical for the design of the replacement bracket. A summary of the end link forces for the critical load case and Bridge Specific Live Loading (BSALL) derivative load cases are shown in Table 1.

Carriageway Loading	HA	BSALL (2005)	BSALL (2005)	BSALL (2005)	BSALL 3.5T Restriction
Footway Loading Type	BD37/01 7.34kN/m (1.58kN/m^2)	BD37/01 approach 5.34 kN/m (1.15kN/m^2)	BSFLL (0.57kN/m) (0.12kN/m^2)	No Footway (0 kN/m)	BD37/01 approach (1kN/m) (0.22kN/m^2)
Total Footway Loading applied to the Bridge	2.662MN	1.937MN	0.207MN	0MN	0.363MN
End Link Force	**6.51MN**	**5.095MN**	**3.954MN**	**3.817MN**	**1.87 MN**

Table 1 End Link Forces for critical load case and its derivatives.

BD37/01 allows for footway loading to be reduced if two footways are considered where a structural member supports two or more notional traffic lanes. However this reduction is not applicable if only one footway is considered to be loaded in a particular load case. Therefore under the critical load case for the link members the 50% reduction has not been applied. The values given in the table in Appendix B are for footway loading complying with BD37/01 (Cl 6.5.1.2), reduction in footway loading have been applied if both carriageways are considered to contain traffic loading in a load case.

A sensitivity analysis to demonstrate the effect the footway loading has on the end link forces was undertaken.

When considering HA loading with a single footway loaded the force in the end link member drops from 6.51MN to 5.65MN when the footway loading is reduced by 50%. Likewise when considering BSALL loading the force in the end link drops from 5.095Mn to 4.45MN when the footway loading component is reduced by 50%.

Footway loading therefore contributes a significant proportion to the end link member forces, this is mainly due to the torque applied to the stiffening truss due to the load being positioned on the footways.

3. CRITICAL CONNECTION LOADING

Under the critical load case of Dead + Live Load (HA) at ultimate limit state a load of 6.51 Meganewtons is transferred from the end post through the link members to the pair of tower brackets. The critical load transferred through to the side span brackets is slightly less.

The critical load case for the connection is HA or BSALL applied to a single carriageway over 20 bays from one tower with no loading applied to the opposite carriageway. Only one footway, adjacent to the loaded carriageway, is considered to be loaded.

The split of load is as follows:

Dead (ULS)		0.87MN	(13.4% of total load)
Live Load	HA + Footway (ULS)	5.64MN	(86.6% of total load)
Total Load (ULS)		6.51MN	

When considering BSALL loading in place of HA the load carried by the connection reduces to 5.09MN, the split of the load is as follows.

Dead (ULS)		0.87MN	(17.1% of total load)
Live Load	BSALL + Footway (ULS)	4.22MN	(82.9% of total load)
Total Load (ULS)		5.09MN	

When considering BSALL loading with the BD37/01 approach footway live loading replaced with the BSFLL the load carried through the connection reduces to 3.95 MN. The split of load is as follows:

Dead (ULS)		0.87MN	(22.1% of total load)
Live Load	BSALL + BSFLL (ULS)	3.08MN	(77.9% of total load)
Total Load (ULS)		3.95MN	

4. SUMMARY OF ASSESSMENT RESULTS

The assessment results for the elements that were found to have overstress indices greater than 1.0 have been updated to reflect the increased load effects from the critical load case described in Section 3.

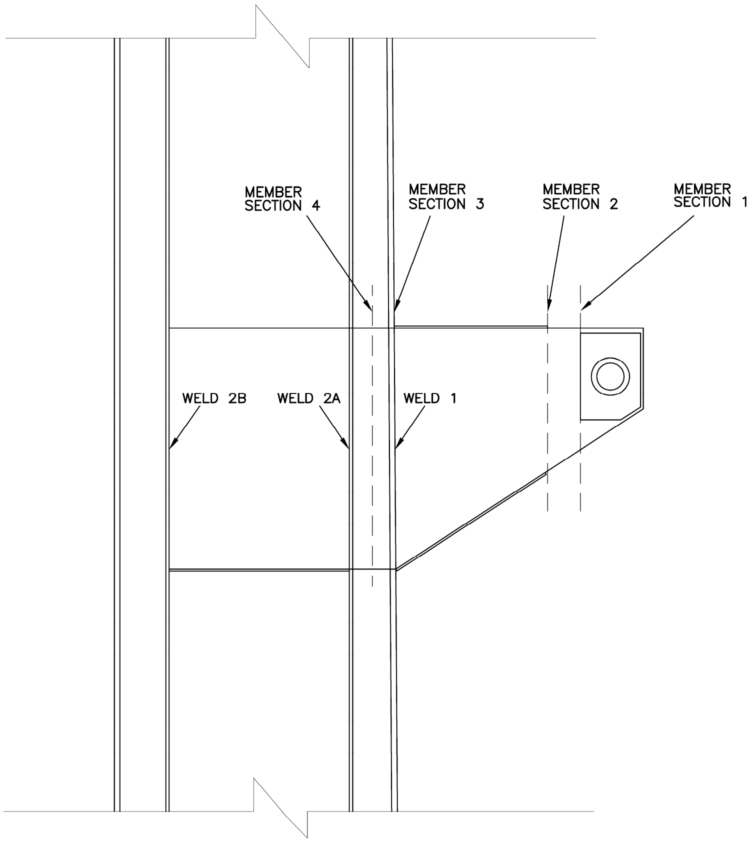


Figure 1

Updated Overstress Indices for Bracket

End Link Brackets

Loadcase	OI (Section 1)	OI (Section 2)	OI (Section 3)	OI (Section 4)	OI Max
Dead+Live + Footway ULS	**1.62**	**1.92**	**1.69**	**1.88**	**1.92**
Dead+BSALL + Footway ULS	**1.27**	**1.50**	**1.32**	**1.47**	**1.50**
Dead+BSALL + BSFLL ULS	**0.98**	**1.17**	**1.03**	**1.14**	**1.17**

Table 2 – Updated Summary of Overstress Indices (Main Span Brackets – members)

Welds

Loadcase	OI (Weld 1)	OI (Weld 2A)	OI (Weld 2B)	OI Max
Dead+Live + Footway ULS	**2.73**	**2.63**	**2.63**	**2.73**
Dead+BSALL + Footway ULS	**2.14**	**2.06**	**2.06**	**2.14**
Dead+BSALL + BSFLL ULS	**1.65**	**1.59**	**1.59**	**1.65**

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

Truss End Posts

Vertical Element	Loadcase	Limit State	Overstress Indices	Panel Point	*Load Effect
Main Members	Dead+Live + Footway	ULS	**2.23**	46	C
	Dead+BSALL + Footway	ULS	**1.75**	46	C
	Dead+BSALL + BSFLL	ULS	**1.35**	46	C

Table 4 – Updated Summary of Overstress Indices (Stiffening Truss End Post)
*** C – Compression T - Tension**

Truss End Post Welds

Dead + HA + Footway	(ULS)	**1.66**
Dead + BSALL + Footway	(ULS)	**1.30**
Dead + BSALL + BSFLL	(ULS)	**1.01**

Note these results supersede the values given in Section 4 of the Design Review Workbook in Appendix A, however the failure mechanisms detailed in section 4.2 and 4.4 are still applicable.

5. REVIEW OF PREFERRED OPTION

The preferred option to strengthen the existing truss end link connection is to provide a new load path between the stiffening truss and the main towers. This solution is intended to remove the requirement to strengthen any of the components of the original connection. The design of the new connection arrangement would be based on HA loading in accordance with BD37/01.

Drawing 79866/001 contained in Appendix C provides an outline of the option. This solution takes into account the construction constraints identified in the design review workshop handbook.

The proposal is to install two brackets to the face of the tower above deck level, link members will connect the tower brackets to a new bracket fixed to the top chord of the stiffening truss. The connection between the link members and the tower / top chord brackets will be formed with pins. The existing link members would be removed however we consider that the existing tower brackets, although under strength, are retained. These brackets could be designed to support the structure in the temporary condition should the need arise to undertake maintenance of the new arrangement.

We consider that it is feasible for new brackets to be fixed to the main towers above deck level. The brackets would be positioned at a level above the existing tension splices at level 5A to avoid clashing. We consider that positioning the brackets at Level 5B is the most suitable location. Confined space working will be required within the tower during installation, however we consider that sufficient access is available to carry out the works. Depending on the final detail for connecting the new brackets to the towers modifications to the main tower plates and stiffeners may be required. Local strengthening around the proposed bracket location may also be required during installation. However the requirement for strengthening is dependent on the final detail developed.

The new tower brackets will be sized and detailed in order to support the link members, making due allowance for the range of movement of the link member. The brackets can also be designed to accommodate temporary works, supporting loads arising during the load transfer between the existing link arrangement and the new connection arrangement. We consider that the most suitable method of fixing the new brackets to the towers is by welding the bracket to the towers in a similar manner to how the original bracket was connected to the towers. Consideration has also been given for the bracket to support future loading from a dropped object structure, for details refer to Section 7.0 of this report. We consider that the brackets and connections can be designed to accommodate these additional forces.

The most suitable method of connecting the new brackets to the top chord is by welding the bracket to the web plates of the top chord member. The width of weld is restricted by the thickness of the existing web plate thickness, which governs the design of the bracket. We consider that a bolted option is not feasible given the configuration of the stiffening truss connections to the cross girder and lateral members.

The bracket will be welded to the top chord directly above the end post, the pin hole in the bracket will be slightly eccentric from the centreline of the end post member, this results in a bending moment in the top chord of the stiffening truss, however the effects of this bending moment have been reviewed and do not result in the top chord becoming overstressed. Loading is carried, from the diagonal member in the stiffening truss through the gusset plates which form the web of the top chord. A smaller proportion of load is transferred to the end post from the cross girder. The resulting forces on the end post are now greatly reduced and the member is no longer acting in compression. Strengthening of the end post member, which was considered to be very problematic, due to the constraints identified in the design review work shop handbook, is no longer required.

The proposed link members would be rolled hollow steel sections with machined forks welded to either end. We consider that members with a low torsional stiffness are required to minimise torque, due to the rotation in plan of the deck, being transferred to the main tower brackets.

The top chord connection would be designed to accommodate temporary loads arising during the load transfer. The permanent top chord brackets can be detailed to accommodate temporary works, similar to the arrangement of the existing top chord / hanger connection. It has been established that it is unlikely that a new bracket will be capable of carrying full traffic loading during the load transfer operation and that restriction on traffic using the bridge will be required. The magnitude of temporary loading that the bracket can accommodate will be dependent on the final detail.

The key advantages and disadvantages of the preferred option are described below.

Advantages

- The proposed option would address all overstressing issues, removing the need to strengthen the end post and accommodate the critical load of 6.51MN.

- All elements of the proposed configuration could be accessed to allow for construction, erection and any future maintenance required.

- The proposed option would not require any of the loads to be redistributed to other elements on the bridge.

- Temporary works could be easily reinstated in the future to aid maintenance if required.

Disadvantages

- The proposed arrangement will not be capable of sustaining impact loads detailed in BD60/04.

- Existing electrical distribution boxes and services would need to be relocated and the proposed option would noticeably alter the appearance of the bridge. Consultation with Historic Scotland will be required before finalising the concept for detailed design.

6. EFFECTS ON EXISTING STRUCTURE

The proposed arrangement of the replacement connection, although similar to the original configuration, will introduce different effects to parts of the existing structure that require further analysis. The following elements have been considered.

- The proposed location of the bracket is 13.2m above that of the original bracket. Therefore the global effects of transferring the load from the link to the new bracket at the proposed location were considered.
- The proposed Link members are 9.7m in length compared to the original members which were approximately 5.4m, consideration was given to the effect on the articulation of the bridge.
- The connection is made to the top chord of the stiffening truss rather than the bottom chord. The effect of changing the point at which the truss is supported from was considered with respect to the articulation of the stiffening truss and forces acting on the members.

The effects of the proposed arrangement on the structural elements of the Forth Road Bridge have been modelled using the 3D computer model used in the assessment of the stiffening truss. The principles of the analysis remain unchanged from those used in the assessment of the stiffening truss.

Summary of findings

Articulation of the stiffening truss is only marginally effected with the introduction of the new connection arrangement. The maximum difference in longitudinal movement of the truss being calculated as 10mm. The rotation of the truss in the vertical plane was also only slightly effected with differences in rotation of the end of the truss being assessed to be in the order of 0.01 degrees.

Global stresses in the towers were calculated based on the revised arrangement. The stresses calculated showed that under the critical load case for the tower connection there was a small increase in compressive stress in the main plates of the tower at the location of the new bracket. This increase was approximately 7N/mm^2 however the stresses in the main plates remained below the design capacity of the plates.

Under the critical load case for the main towers the changes to the stresses acting on the main plates of the tower increased by 2.2N/mm^2 at the location of the new connection however the stresses at the critical section of the tower the stresses remained unchanged.

Local stresses around the new bracket and connection will be dependent on the final detail and method and sequence of installation.

Overall the proposed arrangement of the replacement connection between the stiffening truss and main towers has very little effect on how the structure operates at present.

7. ACCOMMODATION OF FUTURE WORKS

Dropped Object Canopy

Consideration has been given to the possibility of utilising the new connection arrangement to support the centre section of the Dropped Object Canopy (DOC). The dropped object canopy is a structure that has been designed to prevent objects dropped during painting of the main towers reaching the road deck below. The centre section has been designed to also support the weight of the painting gantry and supplies used during the painting works.

We consider that the new tower brackets could be used to support a modified DOC structure. The position of the new tower brackets is slightly higher than the top of existing DOC support columns. Therefore the minimum clearance between the DOC structure and road check will not be compromised.

The existing DOC plate girder arrangement is arranged such that two cantilever sections of plate girder support a simply supported infill section of plate girder from halving joints. The plate girders are supported from a pair of columns connected to the splice plate of the main towers. Out of balance loading on the DOC, when only the centre span or outer spans are loaded, is supported by the columns acting as a couple. The current spacing of the columns is 1.75m.

The spacing of the tower brackets will be determined by the final detail for the proposed connection. The primary function of the tower brackets is to support the stiffening truss and therefore the brackets would require to be detailed for that purpose. We consider that the spacing of the new tower brackets will be dictated by the top chord connection layout, giving brackets in the region of 1000mm apart, which is significantly less than the current spacing of DOC support columns.

If the current arrangement of the DOC was to be supported from the new brackets, the vertical force applied to the inner bracket would be in the region of 2.4MN at ULS in addition to the load carried by the link member (3.25MN). If the DOC plate girders were to be made continuous and only supported from one bracket the vertical load applied to that bracket would be in the region of 0.55MN at ULS. This would significantly reduce the size of the connection to the towers. We consider that the detailed design of the new connection arrangement is based on supporting a modified DOC centre section as described. The current and proposed arrangement of the DOC structure can be see below in Figures 2 & 3 respectively. The main modifications required being the existing halving joints would be replaced by full strength splices and stiffeners relocated.

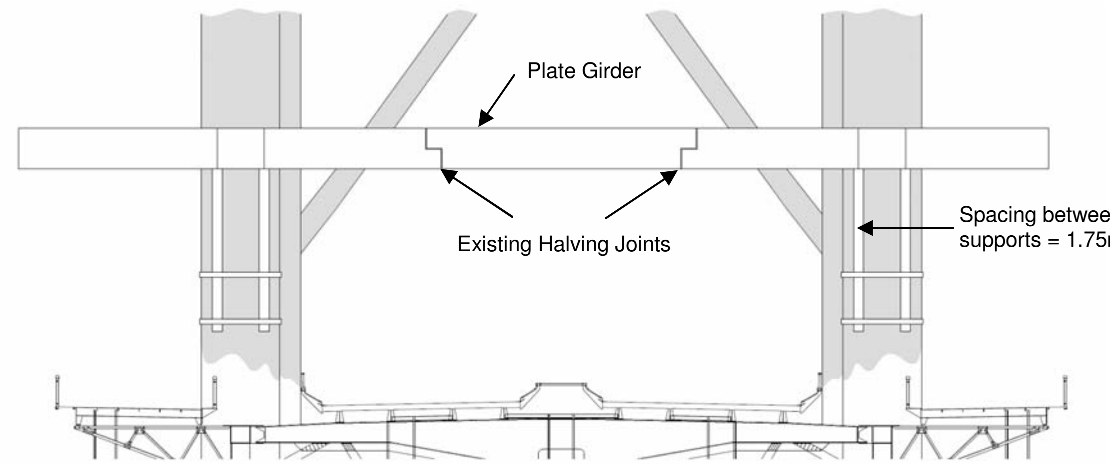


Figure 2 – Existing D. O. C configuration

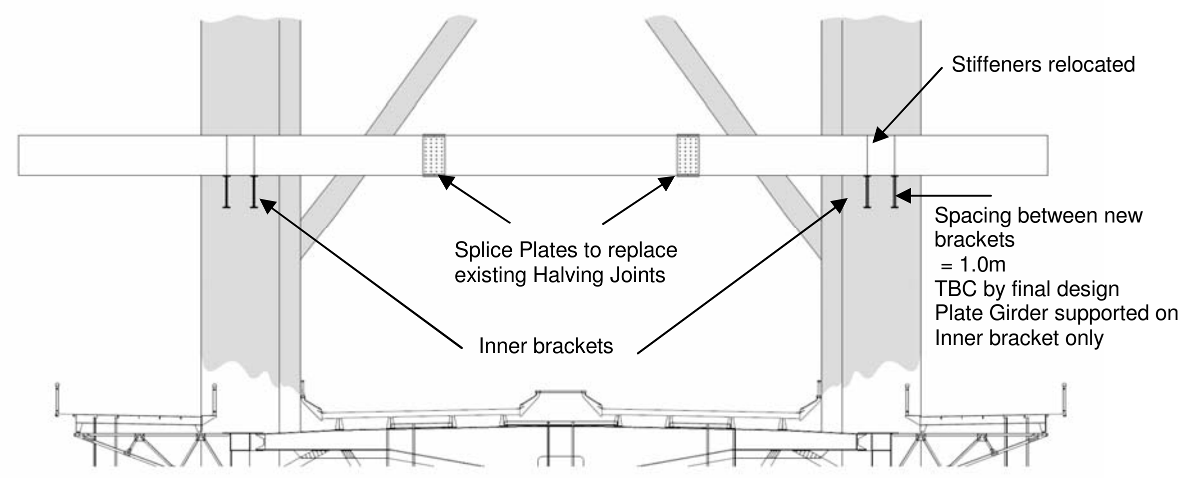


Figure 3 – Modified D. O. C configuration

The DOC structure currently resists wind loading from either direction and transfers the wind loading back to the main tower either through the support columns or through a wind connection fixed to the main tower cell cove plates. We consider that it is feasible to design the new tower brackets such that they can be used to support these transverse and longitudinal loads. The nominal loads that each supporting bracket will be required to resist are given below:

Longitudinal	130kN
Transverse	95kN

In the temporary erection condition both brackets would be utilised to support the DOC plate girder sections until full continuity of the plate girder and plan bracing system is achieved.

The outer frames of the DOC are supported directly from the top chord of the stiffening truss and therefore the loads from this part of the DOC structure will be carried through the new link connection. The load from the outer frame is approximately 320kN nominal.

Main Tower Wind Shielding

We understand that FETA are considering installing wind shielding around the main towers to reduce the effects of wind on bridge users local to the towers. However no details of the proposed wind shielding are available at present.

Tower Impact Barriers

We understand that FETA are considering installing higher containment barriers (Corus type H4a) local to the main towers. We do not have final details of how the higher containment barriers will be connected to the road deck / towers however we consider that there will sufficient space available to erect the proposed brackets and link members. Final details of the higher containment barrier and fixings will be required before undertaking the detailed design of the replacement link arrangement.

8. DETAIL DESIGN ISSUES

Through the study we have identified several key elements that will require careful consideration at detailed design stage. These elements include the following:

- Relocation of services local to the end of the stiffening truss at the main towers.
- Fabrication and construction tolerances for load transfer sequence. Specification of dimensions to be surveyed by any contractor prior to fabrication and erection.
- Detailing to accommodate temporary works for load transfer operations.
- Detailing to avoid fatigue susceptible details.
- Detailing of corrosion protection, site welding is considered the most suitable means of connecting the elements to the existing structure.
- Details for maintenance and lubrication of the bearings. Consideration of automated system of lubrication although maintenance of this system will be required.
- Detailing of any connection and method of installation of the new tower brackets. Consideration of local effects around the towers and the requirement to strengthen these areas if required.
- Details of the proposed tower wind shielding and impact barriers will be required and their effect on the proposed arrangement taken into account.

We note that the appearance of the proposed connection arrangement will alter the profile of the existing structure. Consultation with Historic Scotland and other statutory bodies may be required.

9. SIGNIFICANT HAZARDS IDENTIFIED

As part of the study we have identified a series of hazards which will require to be considered during detailed design. The hazards listed below may or may not be able to be removed during the detailed design stage; however cognisance should be taken of them:

Construction Hazards

Hazard	Details
Working at height	Working above water and carriageway.
Hot working	On-site welding required inside the tower legs. On-site welding required externally at height above carriageway.
Confined space	The Main Towers are classified as a confined space.
Moving structure	The stiffening truss moves longitudinally from the effects of wind and / or traffic loading.
Services	Attention should be given to the location of any remaining existing services which can not be relocated.
Site access/delivery	Restricted/difficult access to inside the towers. Delivery of material and equipment to work areas.
Working above carriageway	Traffic may be live on the carriageways during some operations to install / repaint part of the tower brackets.
Working adjacent to a live carriageway	It is anticipated that top chord connection works will be undertaken when carriageways are open to traffic.
Manual handling	Positioning steel plates inside the outer cells shall be more difficult due to restricted/difficult access routes inside towers.
Interface with public	Footways may be open to public/cyclist during works.
Dust / debris	Removal of existing protective coatings on structure.

Maintenance Hazards

Working at height over live carriageway	Lubrication of moving parts, inspection of moving parts.

10. CONCLUSION

W. A. Fairhurst & Partners have developed the preferred option for the replacement of the connections between the stiffening truss and main towers of the Forth Road Bridge. The detail has been based on providing a new load path between the stiffening truss and main towers, which relieves load from the truss end posts. The preferred option developed can be designed to carry HA loading in accordance with BD37/01.

Additional load cases run identified a more critical load case than previously reported. A review of the effect of footway loading showed that the footway loading contributed a significant proportion of the load in the end link member at ULS.

The new arrangement has little effect on how the bridge functions, the articulation is effectively unchanged and the governing global stresses in the towers do not exceed the permissible design stresses.

We have identified that there are several key areas that will require particular attention during the detailed design of the connection in order to produce a robust detail.

Throughout this study process we have also identified several hazards which have not been removed by design that remain for both the construction and maintenance phase of the proposed detail. Further consideration of minimising the risk from these hazards should be given during the detailed design stage.

APPENDIX A

DESIGN REVIEW WORKSHOP HANDBOOK

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

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Appendix A Component Strengthening Options
Appendix B General Arrangement drawing

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

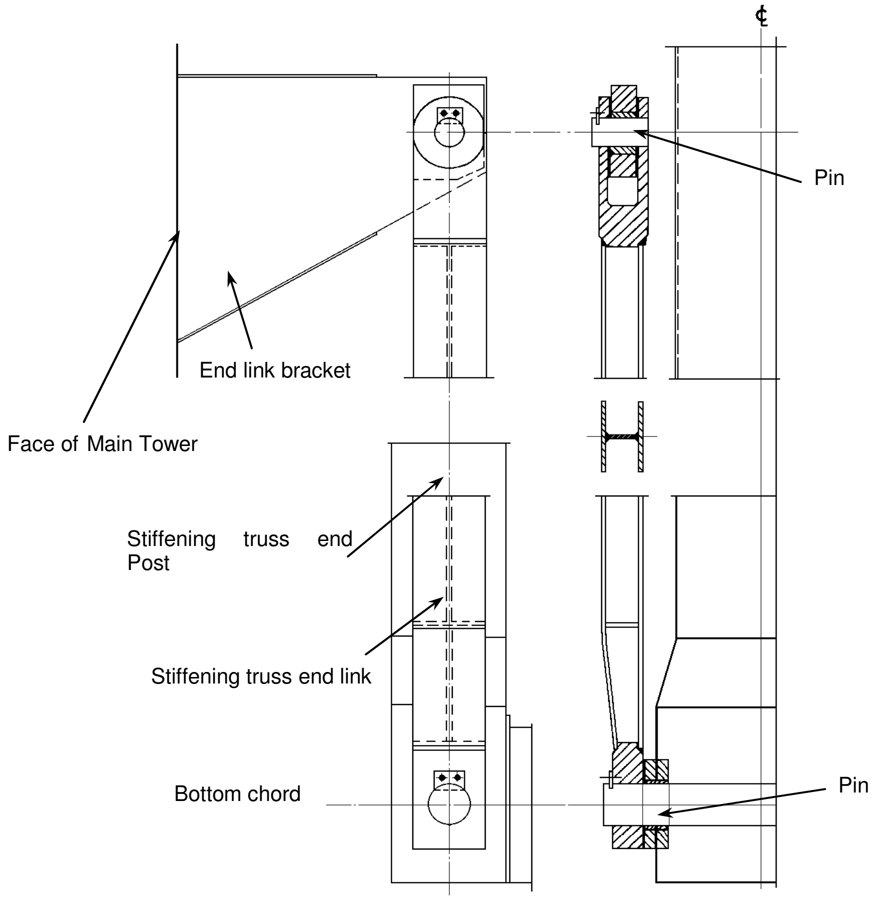


Figure 1 – Stiffening Truss End Link connection to Tower

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	

VALUES SUPERSEDED

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.

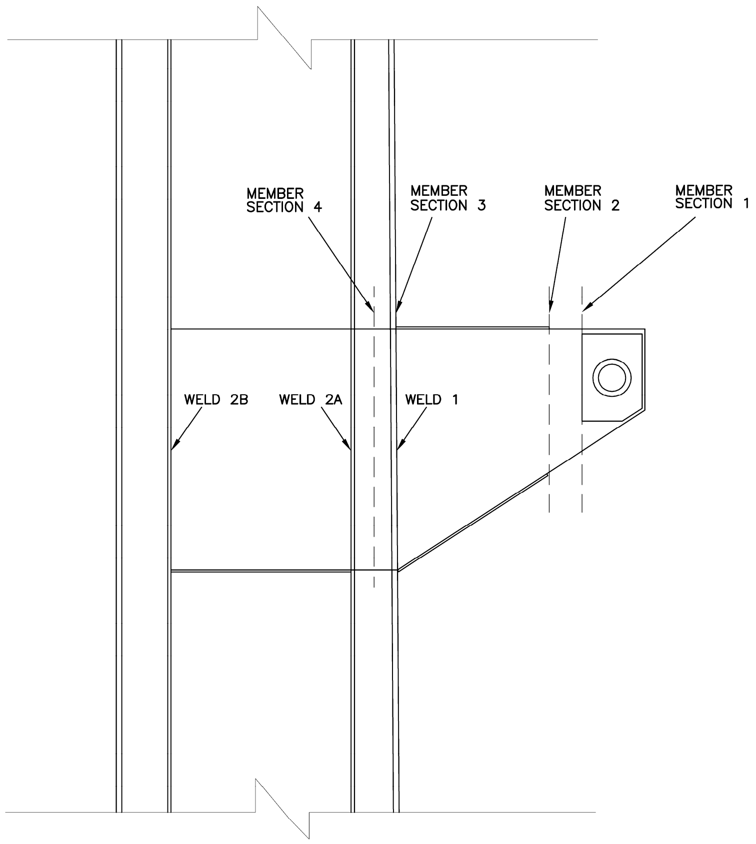


Figure 4 – Main Tower End Link Brackets

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)

TABLE SUPERSEDED

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)

TABLE SUPERSEDED

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)

TABLE SUPERSEDED

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

TABLE SUPERSEDED

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.

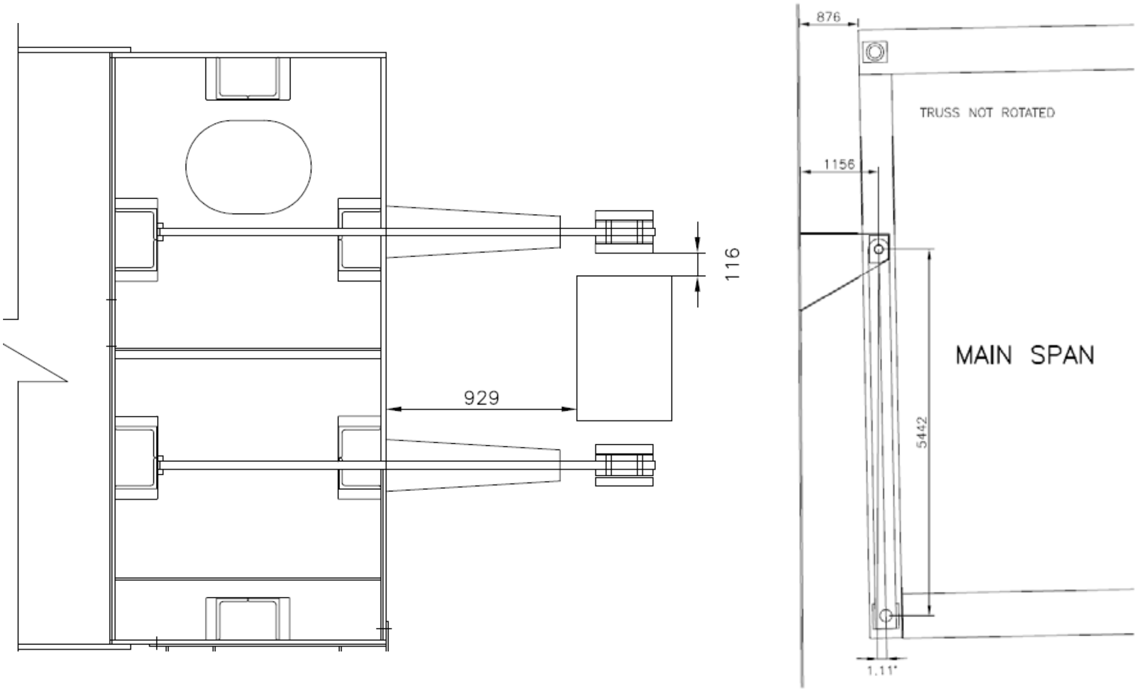


Figure 5 – Starting Position of Stiffening Truss Relative to Towers

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.

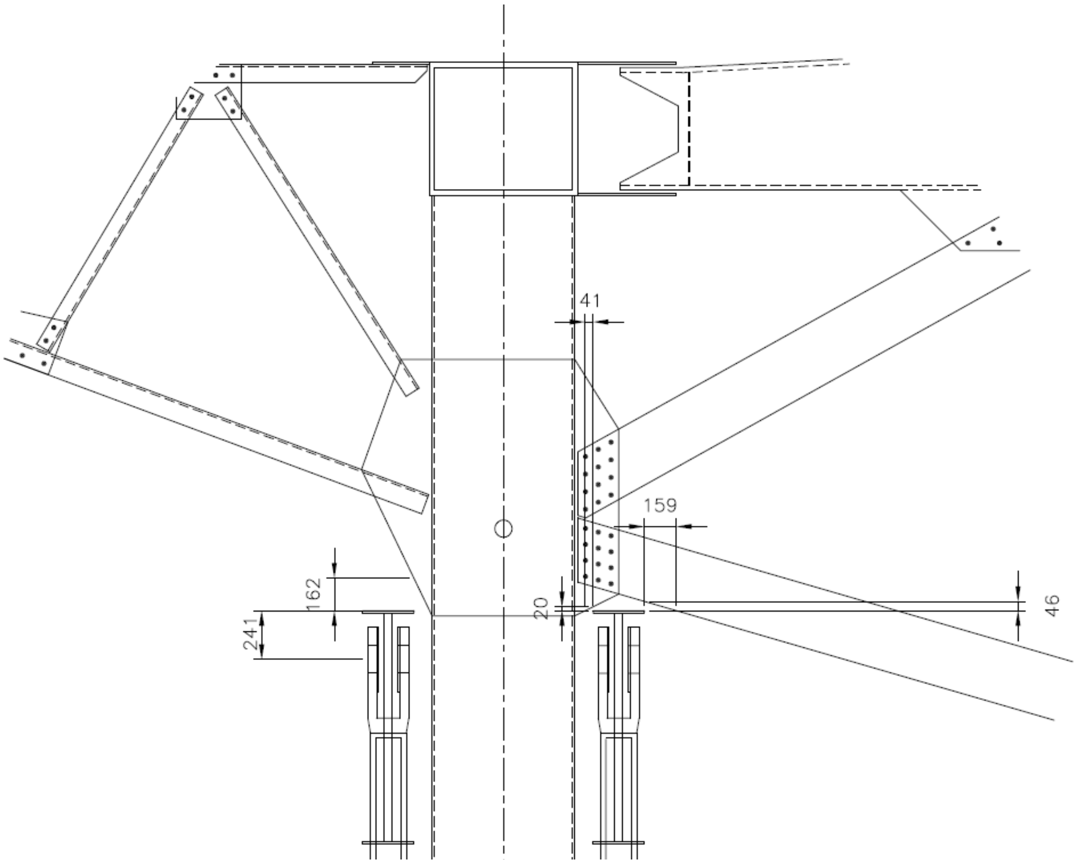


Figure 6 –Relationship between top of bracket and side span cross girder. Main span similar

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.

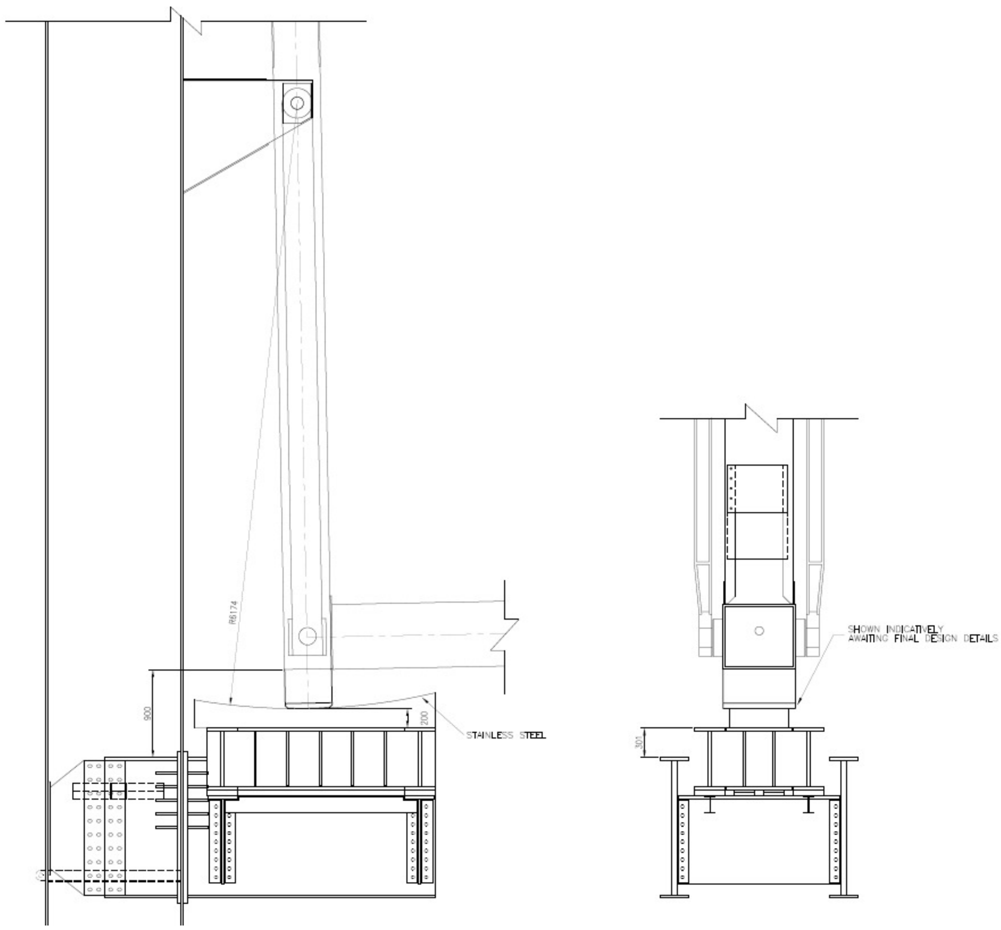


Figure 8 – Proposed Temporary Bracket

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.

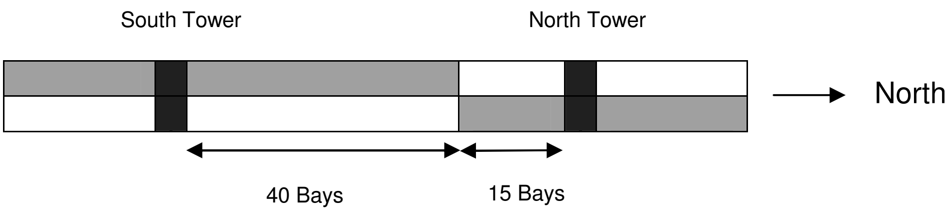
6.1 Additional Load Cases to be Considered

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.

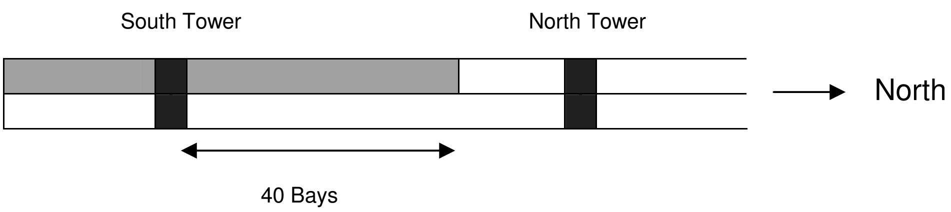
1. & 2. Dead + Live: - Full BSALL and also BSALL with 3.5 tonne restriction

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.



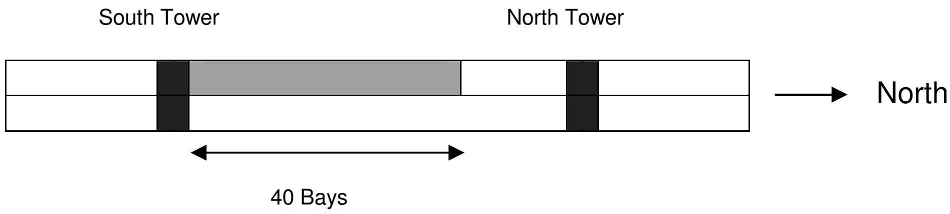
3. & 4. Dead + Live: - Full BSALL and also BSALL with 3.5 tonne restriction

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

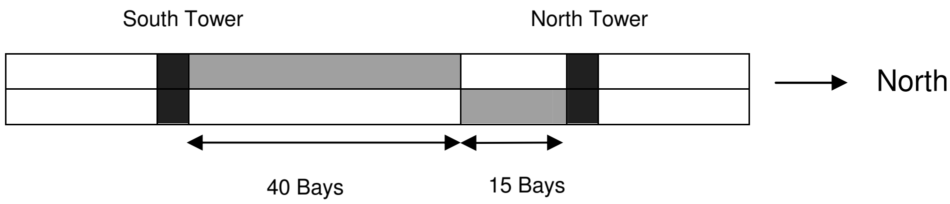


5. & 6. Dead + Live:- Full BSALL and also BSALL with 3.5 tonne restriction

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 take cognisance of the requirement to strengthen the end posts of the side span stiffening trusses.

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.

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.	

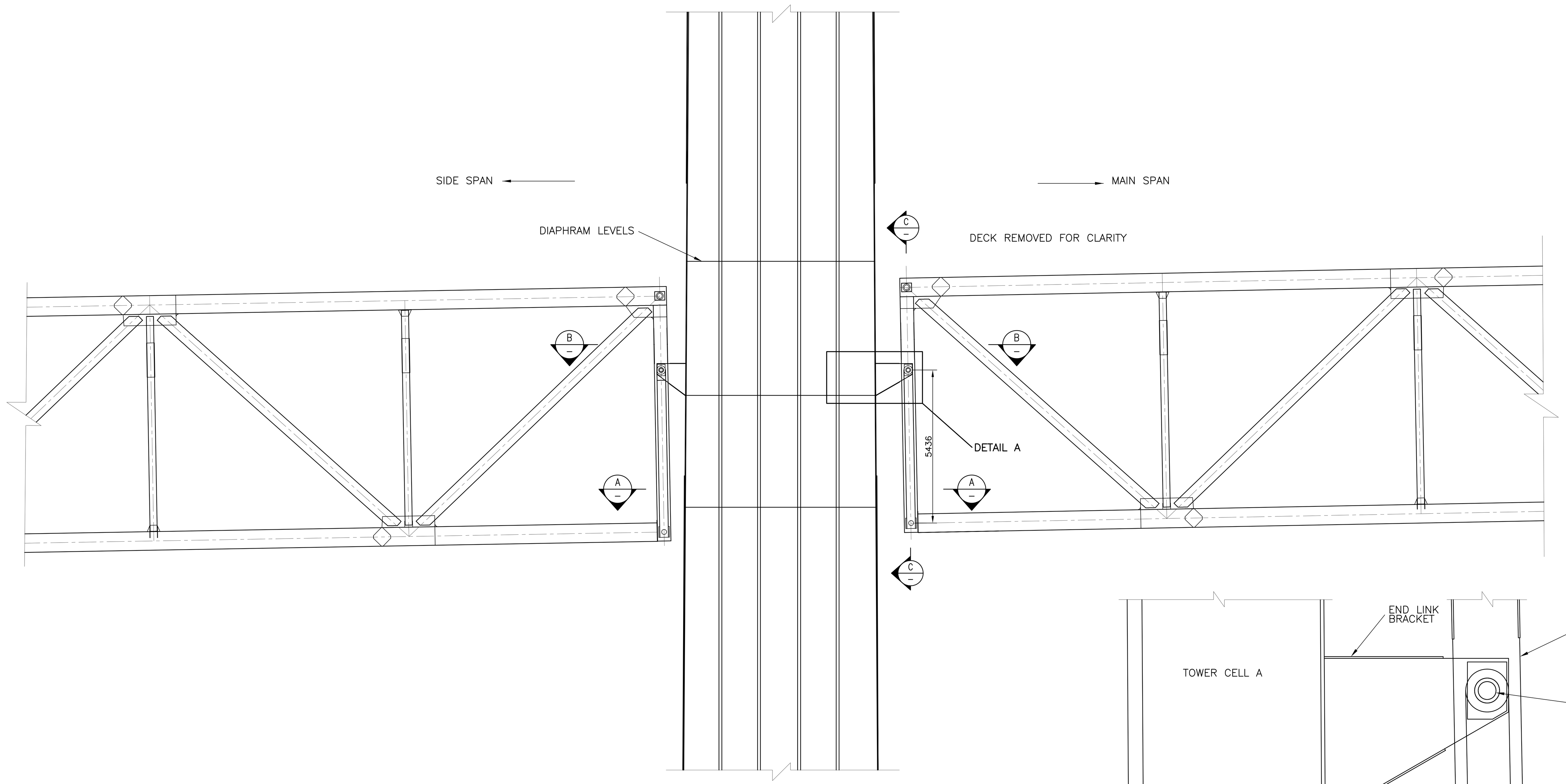
Element	**Remedial measure**	**Limitations / drawbacks**	**Feasible**
End Post Welds	Do Nothing	• Overstress index at BSALL ULS is below 1.0	-
	Reduce loading	• Alternative load path required	Feasible
	Increase weld size	• Access to weld on back face of end post is severely restricted.	Feasible

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

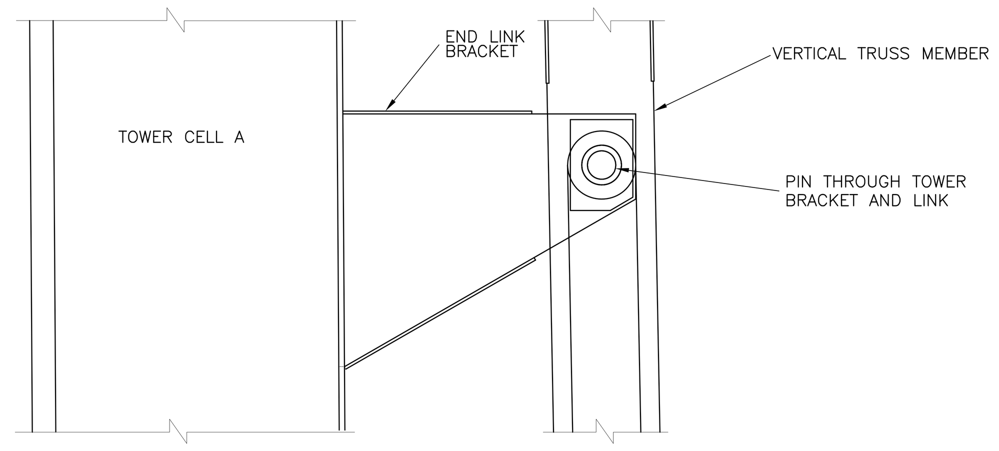
Both options require a similar bracket to be installed on the face of the tower which will require confined space working.

APPENDIX B

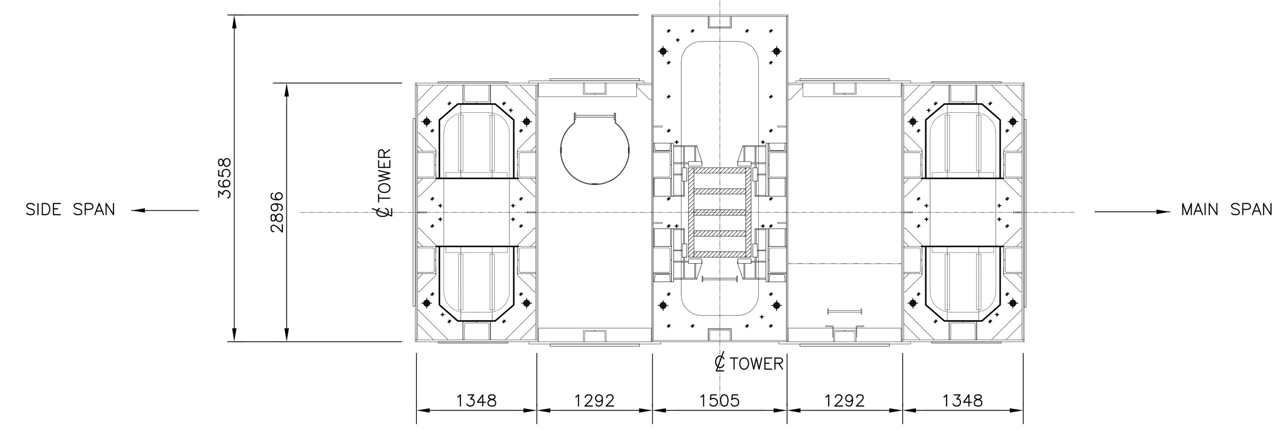
GENERAL ARRANGEMENT DRAWING



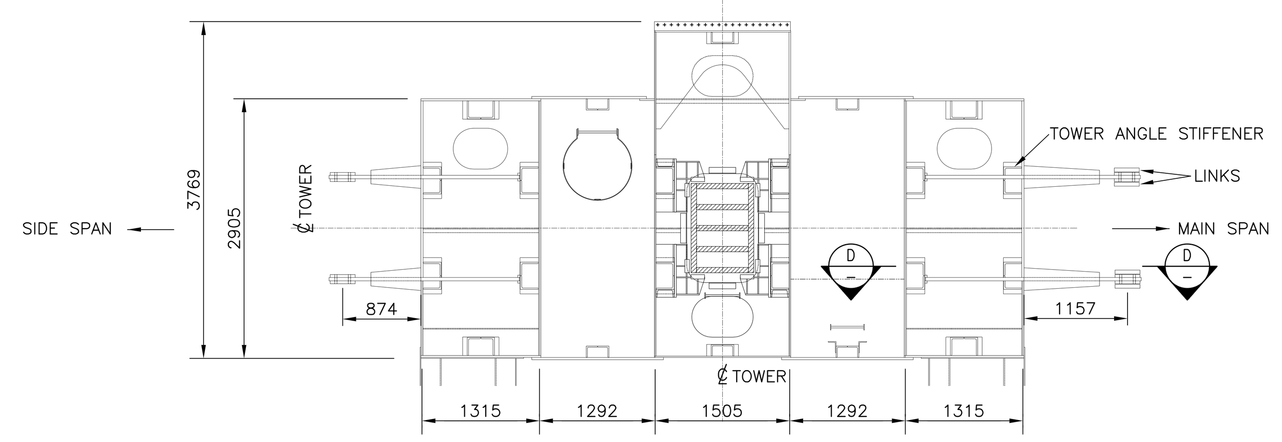
EXISTING NORTH WEST LEG & SOUTH EAST LEG ELEVATION
SCALE 1:100



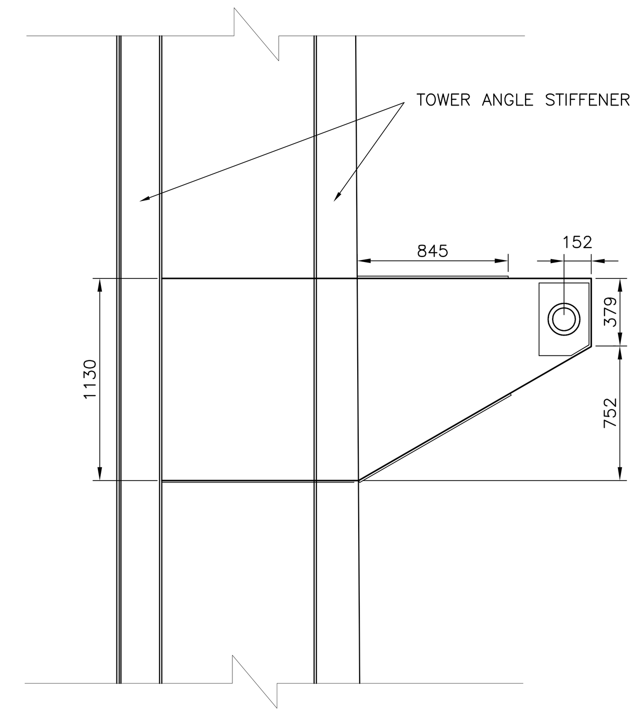
DETAIL A
SCALE 1:20



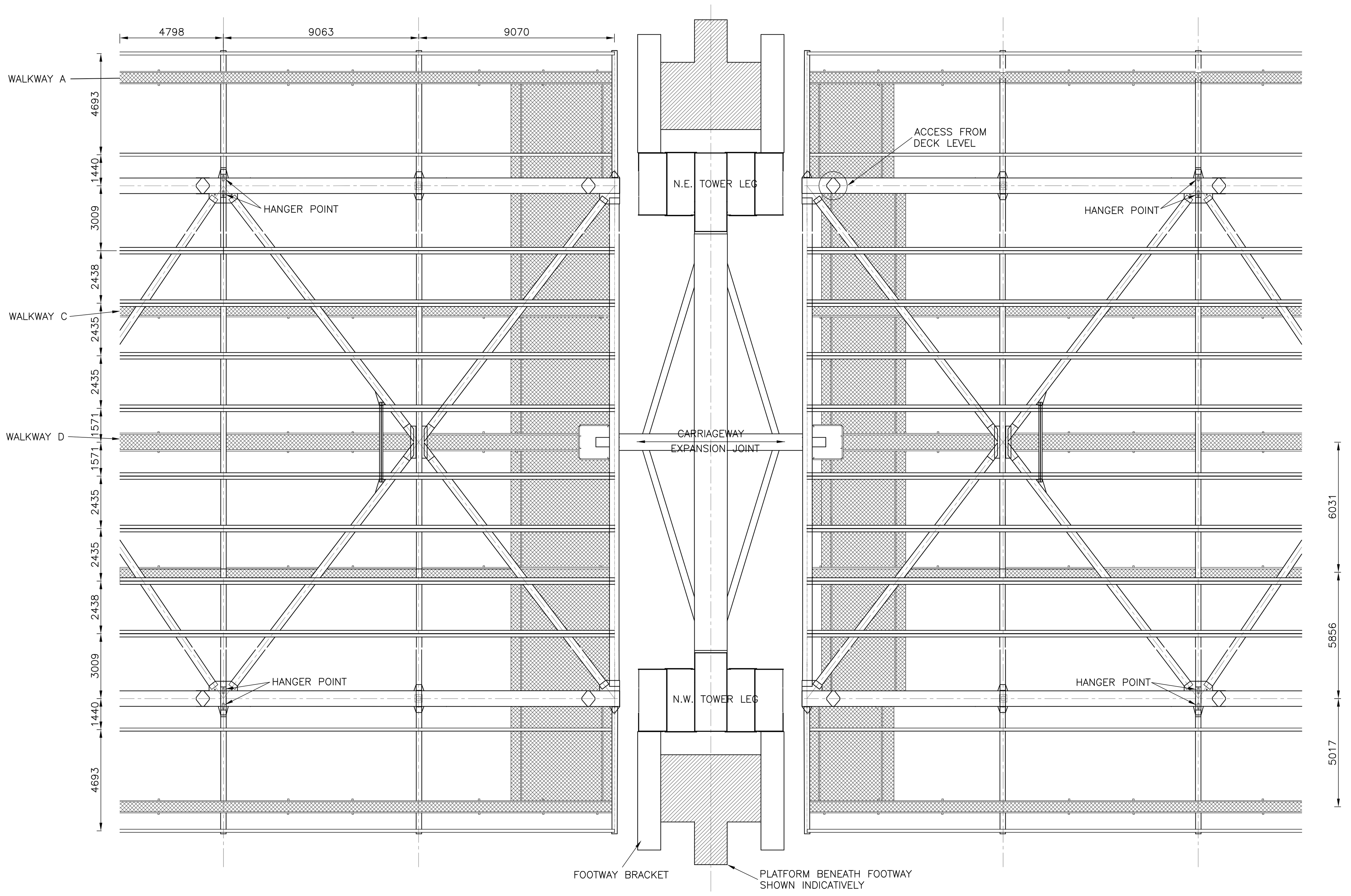
SECTION A–A
SCALE 1:50



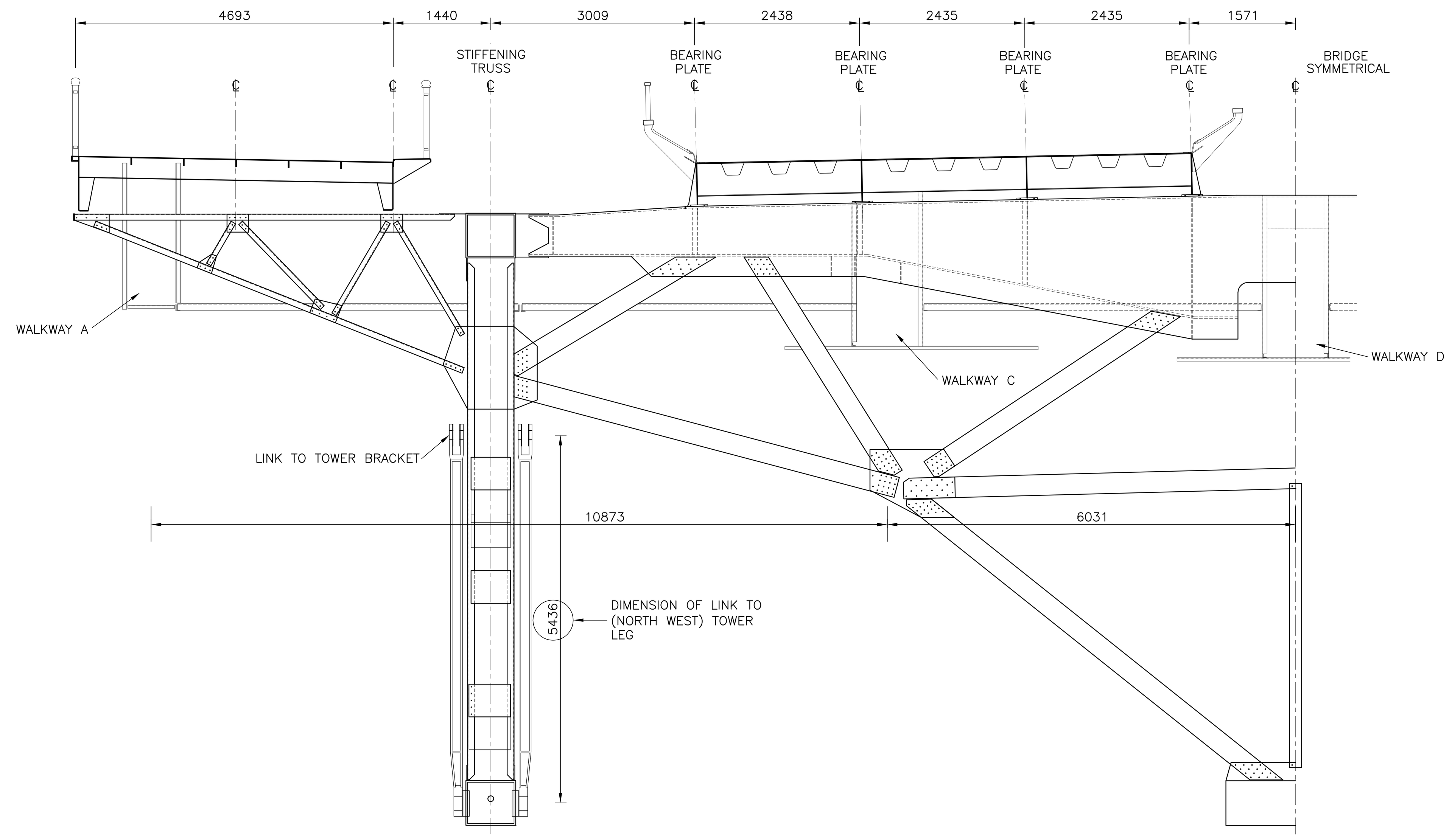
SECTION B–B
SCALE 1:50



LINK BRACKET
SCALE 1:25

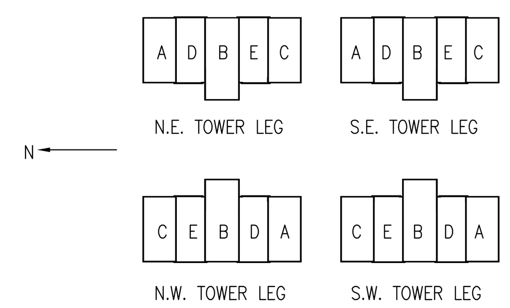


PLAN ON NORTH TOWER BENEATH DECK LEVEL
SOUTH TOWER SIMILAR BUT ROTATED 180°
SCALE 1:125



SECTION C–C
CROSS GIRDER OF MAIN SPAN AT MAIN TOWER
SCALE 1:50

IN PROGRESS



KEY PLAN

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

FAIRHURST
CONSULTING STRUCTURAL AND CIVIL ENGINEERS
11 Woodside Terrace Glasgow G3 7XQ
Tel.: 0141 332 8754 Fax.: 0141 332 5083

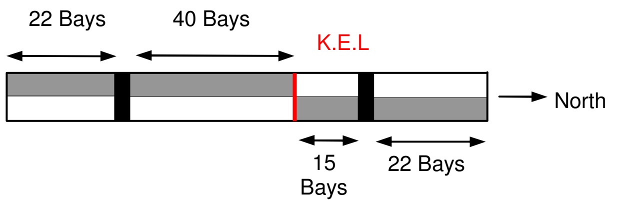
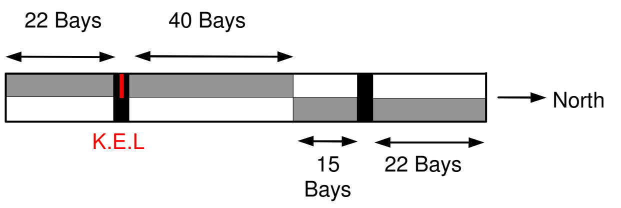
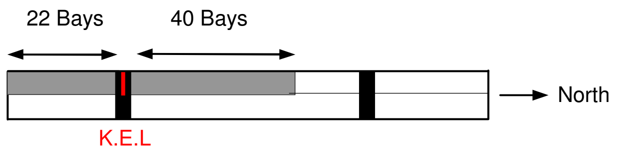
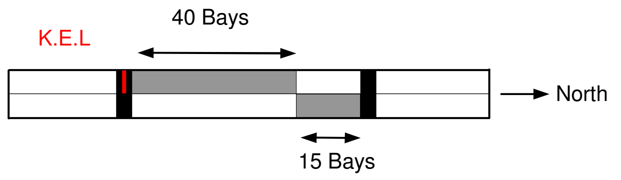
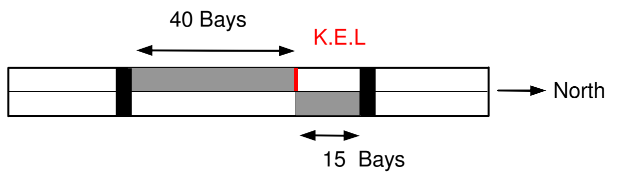
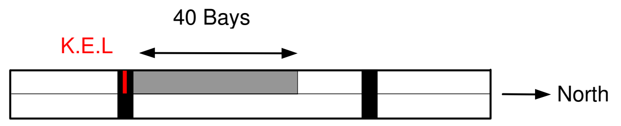
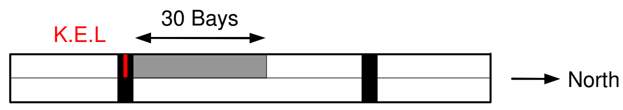
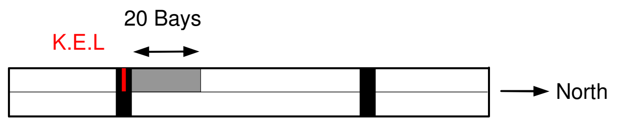
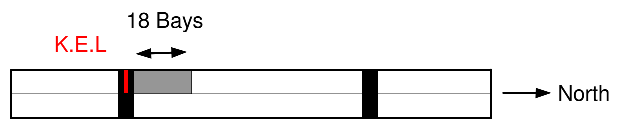
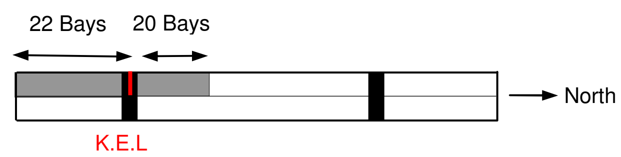
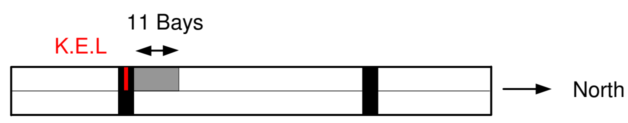
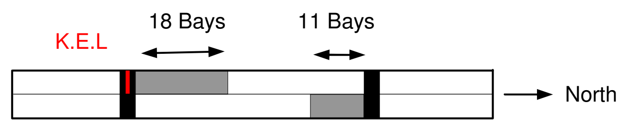
Scale at A0	Status	CAD File Name
AS SHOWN		ps0001-0005

Drawn	Checked	Approved
N.Mc		
Date: 16/04/08	Date:	Date:

Drawing No.	Revision
68952/0002	–

APPENDIX B

END LINK FORCES FOR ADDITIONAL LOAD CASES

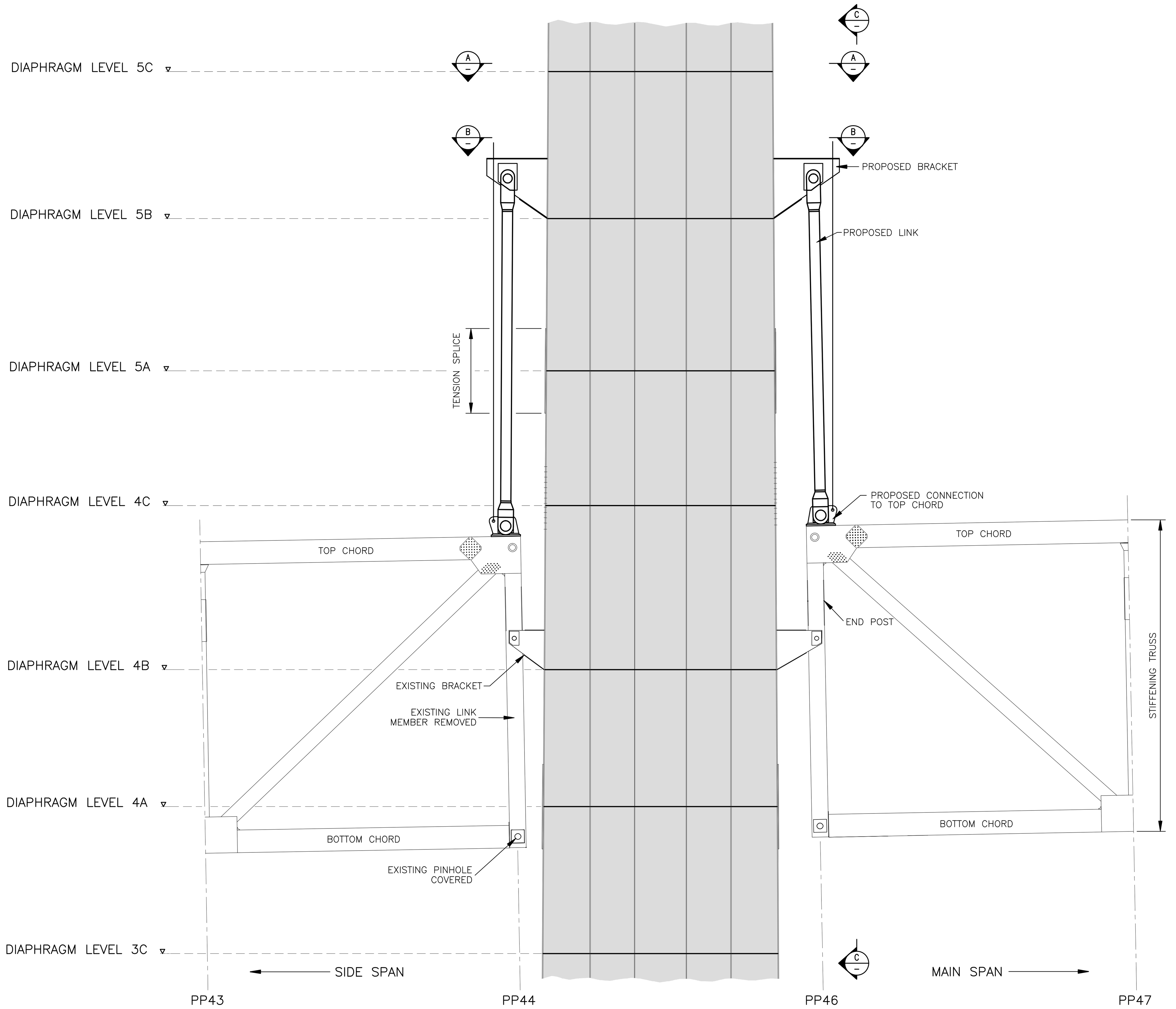
SUMMARY OF END LINK FORCES		MAXIMUM END LINK LOAD FOR DEAD + LIVE at ULS (SLS values shown in brackets where applicable)				
LOAD CASE	CARRIAGEWAY LOADING	HA	BSALL	BSALL	BSALL	BSALL 3.5T
	FOOTWAY LOADING	(BD 37/01)	(BD 37/01)*2	No Footway	BSFLL	(BD 37/01) *2
1.		-	3.28	-	-	1.35
2.		-	3.53	-	-	1.61
3.		-	3.96	-	-	*1 1.65
4.		-	3.94	-	-	1.69
5.		4.82	3.68	-	-	1.44
6.		5.996	4.65	-	-	1.84
7.		6.35	4.83	-	-	-
8.		6.48	-	-	-	-
9.	 CRITICAL LOADCASE	6.51 (5.175)	5.095 (4.064)	3.817 (3.192)	3.954 (3.285)	1.869
10.		6.49				
11.		5.06	3.89	3.27	3.35	1.64
12.		5.45	5.106	-	-	-
13.		4.97	3.85	-	-	FRB Proceedings Max Torque Case

*1 50% load reduction permitted by code has been included however this should NOT have been applied as only 1No. footway is loaded. Load case is not fully compliant.
*2 BSALL footway loading is derived using BSALL carriageway loading adopting BD 37/01 approach.

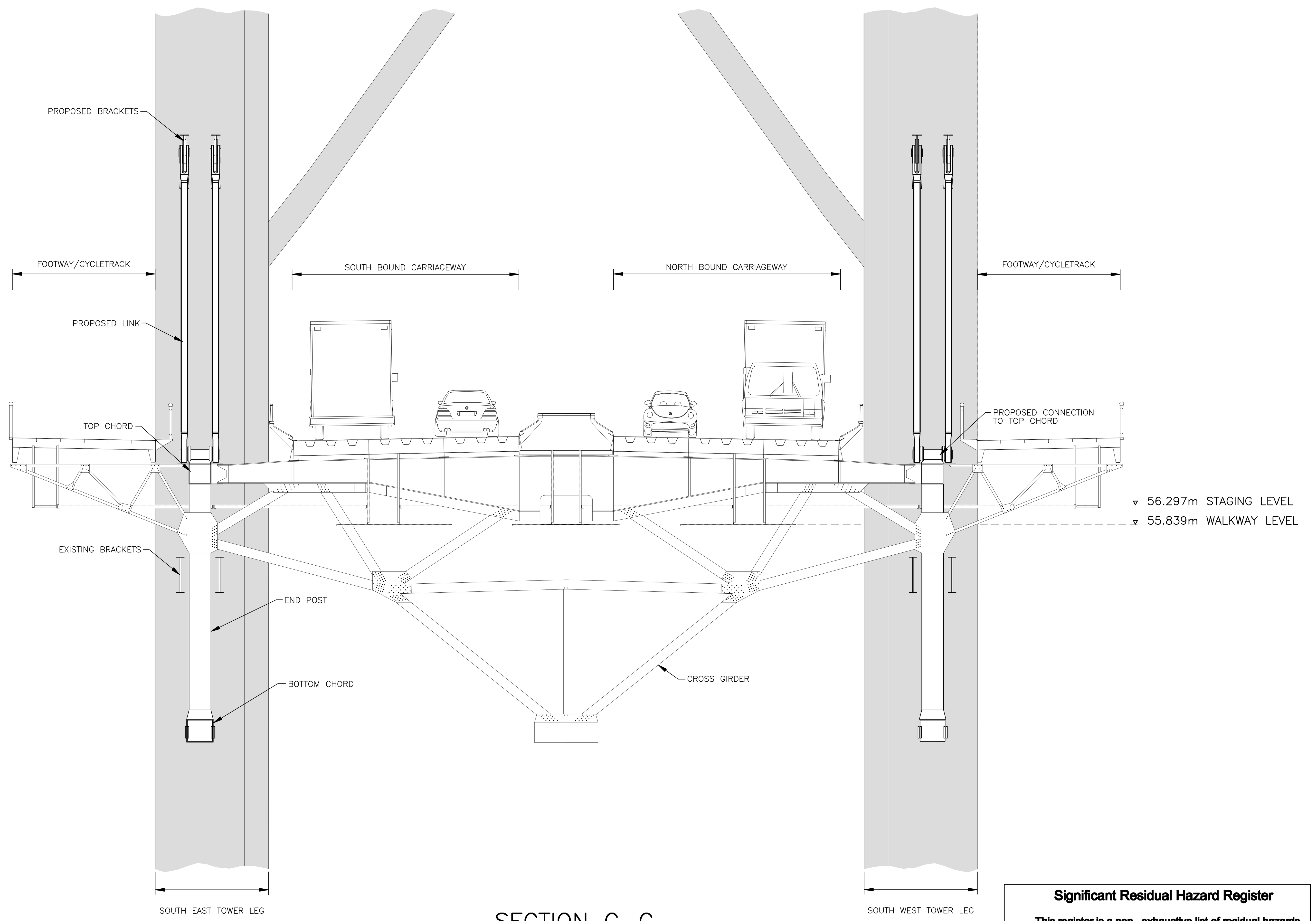
APPENDIX C

GENERAL ARRANGEMENT OF PREFERRED OPTION

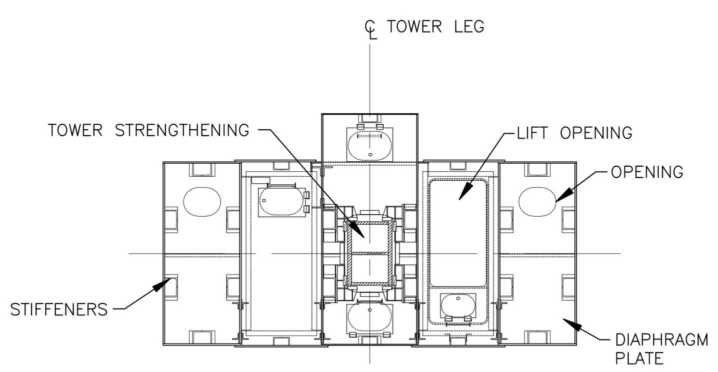
Do not scale from this drawing.



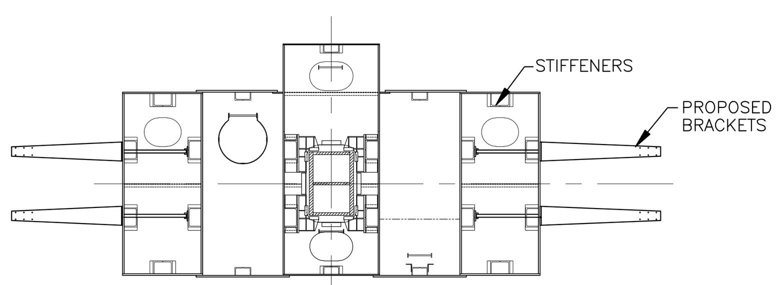
ELEVATION
SOUTH EAST LEG



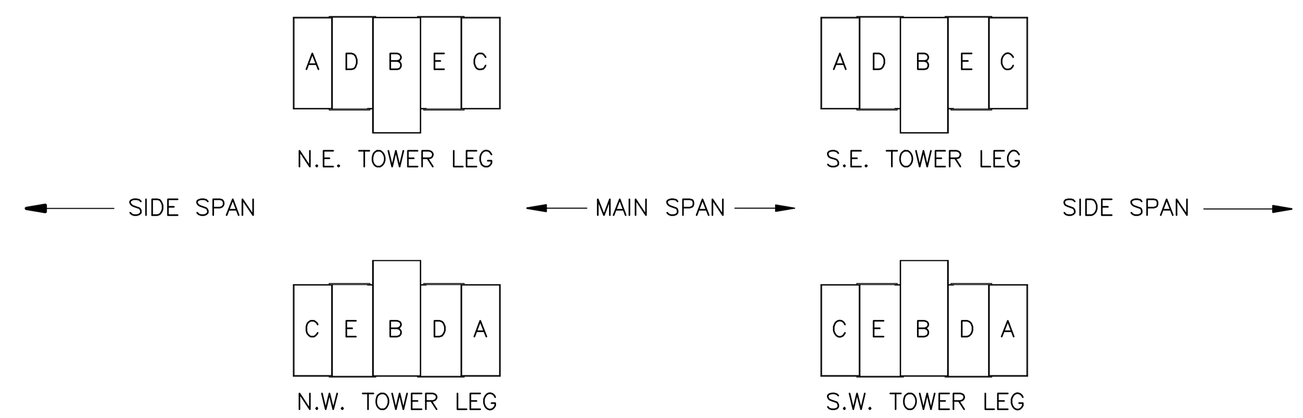
SECTION C–C
SOUTH EAST/SOUTH WEST LEG



SECTION A–A



SECTION B–B



KEY PLAN

Significant Residual Hazard Register

This register is a non - exhaustive list of residual hazards relating to the works shown on this drawing that have been identified during the design stage

Significant Residual Hazards	Reference
Working at height	Working above water and carriageway.
Hot working	On-site welding required inside the tower legs. On-site welding required externally at height above carriageway.
Confined space	Main towers are classified as a confined space.
Moving structure	Stiffening truss moves longitudinally in North-South direction.
Services	Attention should be given to the location of remaining existing services.
Site access/delivery	Restricted/difficult access inside main towers.
Working above carriageway	Traffic may be live on carriageways during works.
Manual Handling	Positioning steel plates inside outer cells of main tower shall be more difficult due to restricted access routes inside towers.
Interface with public	Footways may be open to public/ cyclist during works.
Existing structures	Existing structure shall be operational during works.

Rev.	Date	Description	Drawn	Checked	Approved
A	12/10/09	GENERAL REVISIONS	N.Mc	G.M	C.A.C

NOTES

1. DIMENSIONS IN MILLIMETRES U.N.O.
2. DO NOT SCALE USE NOTED DIMENSIONS ONLY.

REFERENCE DRAWINGS

W.A. FAIRHURST & PARTNERS
21511/15 – GENERAL ARRANGEMENT OF TOWERS (N.W. & S.E. LEGS) FIFTH & SIXTH PORTIONS

Client:
Forth Estuary Transport Authority

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

FAIRHURST
W.A. Fairhurst & Partners
225 Bath Street, GLASGOW, G2 4GZ
Tel: 0141 204 8800 Fax: 0141 204 8801

Scale at A1:	Status:	
1:100	PRELIMINARY	
Drawn: N.Mc	Checked: G.M	Approved: C.A.C
Date: 01/06/09	Date: 01/06/09	Date: 01/06/09

Drawing Title:
GENERAL ARRANGEMENT PREFERRED OPTION

Drawing No.: 79866/001
Revision: A