An Innovative Approach to Wind Strengthening of a Cantilevered Stadium Roof, Yarrows Stadium, Taranaki

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ABSTRACT: This case study outlines the assessment and innovative retrofit of a large cantilevered stadium roof at Yarrows Stadium, New Plymouth, Taranaki that sustained significant wind related damage in a high wind event in 2012. The stadium roof sustained damage to a significant area of roofing due to a roof fixing failure. However the investigation highlighted a significant change in the wind loading standard associated with cantilevered roofs. Through agreement with the then owner, the New Plymouth District Council (NPDC), a roof structure review by Beca was initiated.

The review identified that the stadium stand structure, constructed in 2001 had been designed and built prior to the current versions of AS/NZS1170.2 where cantilevered roofs are specifically covered. The upward wind loading of the truss imposed the most significant load in this case where the existing (steel circular hollow section) top chord was subjected to a significant compressive axial load that exceeded the capacity of the member. This risked structural failure at what is considered a wind event with a serviceability probability of exceedance. This meant that the stadium roof structure was subject to a relative risk of failure of 10 to 20 times that of a new roof structure of the same form. This can be considered similar to the relative risk associated with an earthquake prone building.

While there is no provision in the building act for retrofitting structures for wind loading, the recent changes to the wind loading standards are specific and significant. Given the level of relative risk and the number of potential affected parties, the NPDC commissioned Beca to design a retrofit to mitigate this risk.

A logical retrofit for this roof involved providing a restraint at the middle of the top chord of the truss, with suitable load paths to transfer restraint loads into existing load paths. This would stabilize the strut and protect it from an axial buckling failure.

The issues with such a solution that governed the design were: safety, access, environmental issues and work quality. The roof was 14m above ground level making access difficult and risky. The existing roofing system was underslung with fixings that failed under pedestrian/service loading (therefore access was limited to the purlin structure only, at 2.5m centers). The areas of works were exposed to all elements being exterior in nature. The expense and associated risks (safety, works length, quality and cost) of the works suggested an alternative solution that allowed for prefabrication and simple site assembly.

The retrofit included:

- A bespoke multifunctional innovative steel clamp that accommodated multiple members and alignments;
- It could be fabricated offsite and site fitted directly off a bespoke lifting plate assembly;
- Access was via Mobile crane lifted work platforms;
- A single member type and connection detail to be used for the entire strengthening works.
- Tactical use of friction grip bolts to facilitate rapid construction with site tolerance.

The retrofit allowed:

- Minimizing exposure of contractors to site risks such as height, weather and access.
- Improving the quality of the product by offsite fabrication.
- Improving the efficiency of all parties onsite including the ultimate cost of the retrofit.

The retrofit design was initially conceived in house and then collaboratively developed further with both the client and the contractor, Fitzroy Engineering Group limited (FEGL). Ultimately, the successful design and collaboration of all parties allowed the installation of 4 tons of structural steel on top of a relatively inaccessible stadium in three and a half working days.

1. BACK GROUND:

Beca Infrastructure Ltd (Beca) was commissioned by New Plymouth District Council (NPDC) following the failure of roof sheeting on the TSB Stand Roof at Yarrow Stadium, which occurred during high winds in April 2012. The scope of our initial engagement was to investigate the cause of failure and assist NPDC to make appropriate repairs that would allow the stadium to be used for events.
As this commission evolved it became evident that assessment was required of the following:

- Its structural condition.
- Capacity of the existing roof structure to resist wind loads relative to the current wind loadings standard.
- Consideration of serviceability issues relating to the TSB stand.

Yarrow Stadium is located near the CBD of New Plymouth and is a sports venue catering for top level sporting events, predominantly rugby. It seats approximately 26,000 people with the majority of spectators in the two main stands, the TSB Stand and the Yarrows Stand. These are located on the East and West sides of the main sports field respectively, which is aligned approximately North to South. Refer to Figure 1 below for a site aerial photo.

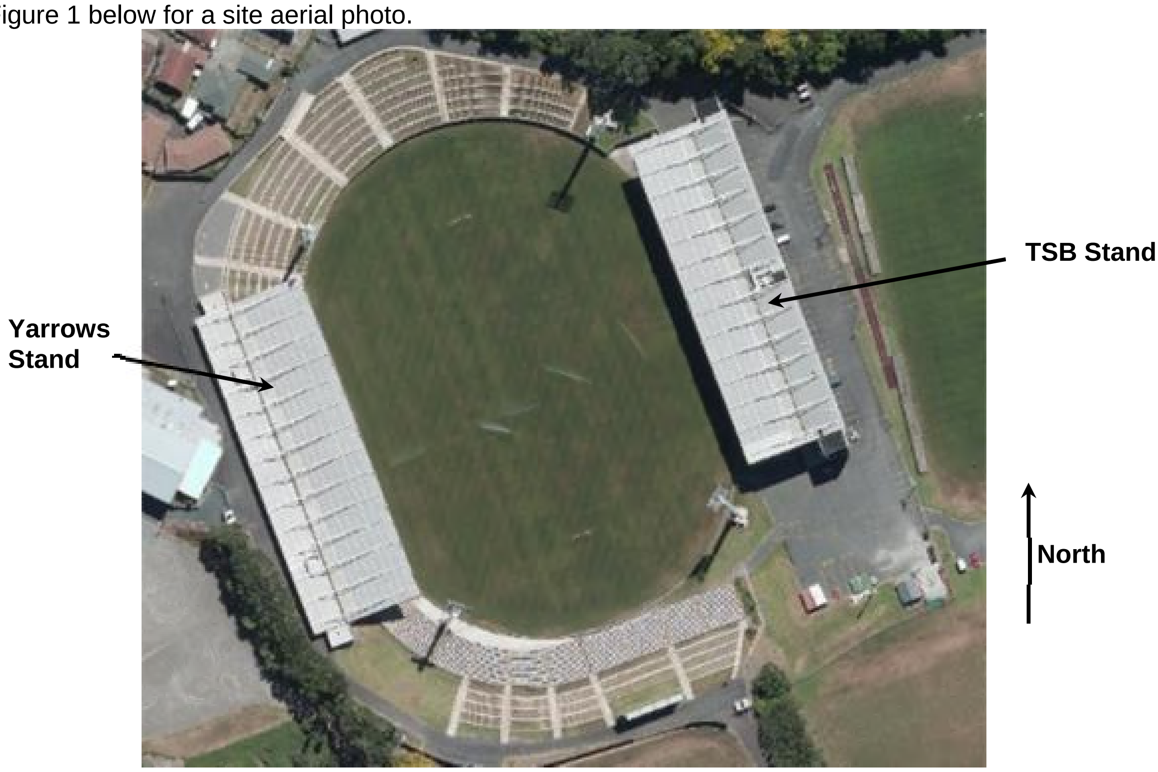

Figure 1 – Plan of Yarrows Stadium

The TSB Stand (and Yarrows stand), the subject of strengthening was constructed in 2001 in its current form. Since July 2013, the ownership of the stadium has resided with the Taranaki Regional Council (TRC) and NPDC, with Management and Operations undertaken by the NPDC. A Joint committee was established (comprising TRC and NPDC) to provide overall governance and a joint vision.

2. EXISTING STADIUM STAND DESCRIPTION:

The steel roof structure above the top floor level on both stands consists of heavy steel columns that form a portal frame with a steel truss, which cantilevers 21m over the bleachers towards the main field. The steel truss (7.3m centres) is constructed from circular hollow sections (CHS) with a steel universal beam section (UB) as the bottom chord located immediately above the underslung roof, consisting of profiled aluminum roofing supported below steel rectangular hollow sections (RHS). The total roofed area of each stand is in the order of 3,000m^2.

For the purposes of discussion in later sections, the critical sections of the roof steel structure are identified in Figure 2. The same terms apply when discussing the East or West Stand roof structures.
The entire TSB Stand structure is supported on piles as it is sited near to the middle of the original fill zone of the valley. This three storey structure has a double height middle storey. The ground floor contains the sports facilities, the first floor contains the public concourse and the top floor contains the members area, known as the Legends Lounge. The lateral resisting structure consists of a braced steel structure, with both eccentrically (K-braces) and concentrically braced (rod bracing), in the north-south direction and steel moment resisting frames in the east-west direction. Concrete bleachers are supported on raking steel beams and the suspended floors are generally constructed from hollowcore precast concrete units with an in-situ concrete topping slabs. A typical section of the TSB Stand is provided in Figure 2.

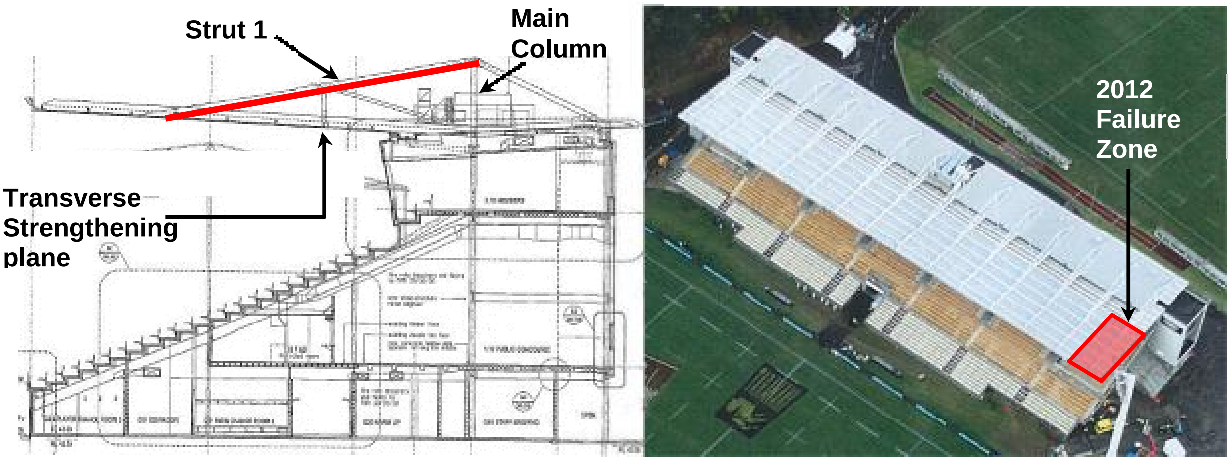

Figure 2 & 3 – TSB Stand Typical Section and Perspective View

3. HIGH WIND DAMAGE:

In March 2012, the TSB Stand suffered high wind related damage from a gust event(s) from the south west which loaded the underslung roof such that it caused the roofing to puncture through the fixings that held the roofing to the underside of the RHS purlins.
The damage occurred on the SW tip of the TSB Stand and covered an area of 90 to 120m^2. Refer to Figure 3 above for an annotation of the failure area.
During the subsequent investigation of the roof cladding failure and inspection of the balance of the roof for damage, it was raised to the stadium owner that the understanding of high wind loads relative to large cantilever roofs had changed since the design of the original structure. Therefore the engineering scope for the stadium was extended to allow assessment of the primary structure as well. The intent of this was to establish whether the structure was exposed to higher wind related risk than was previously understood and to then be able to manage that risk in an informed manner. This wind structural assessment, the findings and associated implementation is the subject of this paper.

4. ROOF ASSESSMENT:

Wind Loading Standards
The structure was designed under NZS4203:1992, which does not specifically cover large cantilevered roofs. However an assumed basis for design was likely derived from one of the existing similar cases within the standard such as for an exposed roof.
The Australian wind loading standard, AS1170.2 (1989) initially introduced provisions to cover large cantilevered roofs in 2002, after research into wind behaviour around stadium, stand like large cantilever roofs by Melbourne and Cheung in 1985. This research, amongst others, suggested that the net pressure coefficients around these types of structures were notably higher than had been previously assumed.
In 2004, New Zealand and Australia combined their loadings standards into AS/NZS1170, Appendix D5

of 1170.2 (wind loadings standard for Australia and New Zealand) formalized the findings from earlier works. In this standard, the suggested net pressure distribution on a large cantilever roof was represented by a triangular profile that tapered to the support region of the cantilever. The pressure could be either positive or negative, but the magnitude of the coefficient changed with the sign and with the relativity of the area concerned relative to the structures edge.

In 2011 AS/NZS1170.2 Appendix D5 was revised, due to further work in this area. The significant change was to increase the net pressure distribution from a triangular form to that of a trapezoid. This was notified by IPENZ in 2011 as part of the draft update of AS/NZS1170.2, and was the basis for the advice to assess the roof for wind loading. The current (and previous) pressure distribution coefficients for wind aligned toward the tip of the cantilever is as follows in Figure 5:

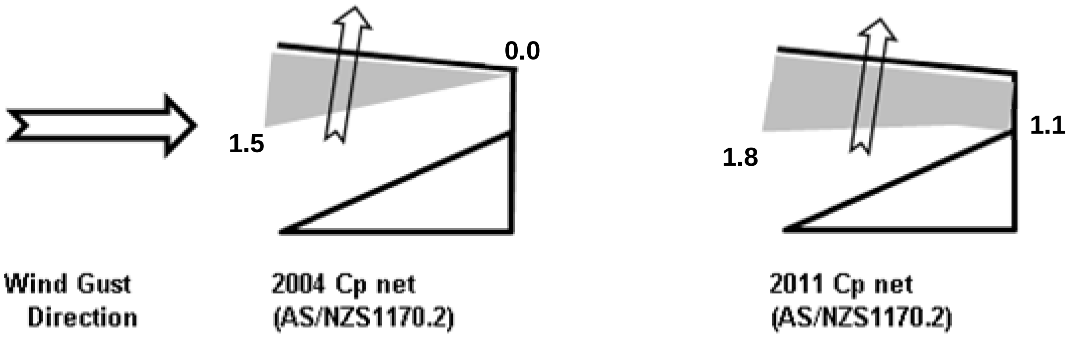

Figure 5 – Cantilever Roof Wind Pressure Coefficients in 1170.2

The above illustrates the increasing magnitude of the Cpn for these types of roofs that occurred over time and the significance of the uplift case which was critical in this instance. The following was also notable within Appendix D5 relevant to these types of roofs:

- Downward loads from wind are also significant.
- Roof loading varies relative to the offset from the sides of the stand.
- These significant load cases only occur when the wind direction is within a 90 degree plan arc of the wind direction indicated in Figure 5 above, in other cases the wind load on the roof is as per other typical structures. Therefore the TSB Stand roof is exposed to governing wind loads from the NW, W and SW directions.
- The method in AS/NZS1170.2 Appendix D also includes dimensional and dynamic amplification limitations and allowances respectively and non-conformance can still require wind tunnel testing.
- AS/NZS1170.2 Appendix D does not reference localized pressure factors (as per other roofing shapes) for vortices or other mechanisms that are often critical for secondary elements (ie purlins and roof fixings). Therefore testing and judgment of the engineer is required in these instances.

Assessment of the Roof Structure

The steel structure of the roof was computer modelled in spacegass as a 3D frame and assessed under various wind loading cases from all directions. The model extent was such that the vertical frames were extended to the foundation level with a sensitivity analysis applied to the level of rotational restraint in plane of the truss/frame portal. Out of plane the main trusses of the stadium were half modelled to assess structural capacity in the truss out of plane direction.

It was found however that the AS/NZS1170.2 Appendix D conditions were the only case that caused actions on any structural elements such that they did not meet their current code capacity under NZS3404.

Detailed Assessment of the Critical Strut

In the uplift load combination depicted in Figure 5, Strut 1 assumes a compression load. When under compression this element is 15m long approximately (L) from the bottom chord intersection to the main column at the highest point of the stadium. Therefore buckling sensitivity is the most significant issue to derive capacity. In doing so we considered:

- The potential for rotational restraint at the bottom chord and the midspan vertical strut. However these mechanisms were either weak or governed by poor connections (purlin to chord).
- The out of plane restraint and stiffness of the main column where it cantilevered out of plane above the purlin plane to restrain Strut 1.

- The ability of the main column to also offer torsional restraint (in plan) to Strut 1, thereby offering an out of plane moment end connection to strut 1.

Given the above issues, the effective length of strut 1 relative to compression buckling was **Ke X L = Le**, where Ke was in the range of 0.85 to 1.0. This was to reflect the potential of the boundary conditions to offer some rotational restraint available by the strut end conditions. Induced moments were also included in the analysis, but given the bending stability of the section and the small moments involved, compression buckling governed.

The results of the assessment were as follows in Table 1, under the load combination of 0.9G & Wuplift:

Table 1: Strut 1 – Wind Assessment Results

Ke	Phi.Ncy (kNm)	Phi.Mcy (kNm)	N*/Phi.Ncy + M*/Phi.Mcy <= 1.0 (vs 100% of 1170.2)	V,des (m/s)	Strut 1 - V,allowable (m/s)	AEP
1.00	327	139	1.81	41.0	32.0	17 years
0.85	455	139	1.30	41.0	36.0	76 years
0.50	1,097	139	0.63	41.0	56.0	>>5,000 years

Comments on the above table are as follows:

- V,des derived from an IL3 wind event (1000 year AEP), and an Mz,cat = (15m,C3) = 0.89.
- V,actual derives from a gust speed that generates a unity check in the capacity calculation.
- The AEP derives from a reverse calculation of the equation, AS/NZS1170.2, table 3.1 region A7 to determine the event risk that would not cause failure. See a part extract from AS/NZS1170 below to depict this (note this is for V,r rather than V,des).

V_{5000}	50
V_{10000}	51
V_R ($R \geq 5$ years)	$67–41R^{-0.1}$

- Therefore the strut had 10 to 20 times the relative risk of a new structure designed to current standards. (This could be considered as equivalent to an earthquake prone building under seismic loading).

Legislation

As referenced above, if the actions were derived from seismic loading and the level of risk was such that strengthening would be required of the structure under the Building Act. But, no such legislation exists for wind loading.

However, both TRC and NPDC deemed it appropriate to investigate strengthening of the stands due to the following:

- The level of increased relative risk;
- The number of parties that may attend a large event;
- The nearby residential housing;
- The unknown extent of area affected by a structural failure of such a large roof.

5. STRENGTHENING SCHEME

From review of Table 1 above, it is clear that stabilizing Strut 1 laterally at its midspan is all that is required to allow the existing structure to operate as close as practical to a new structure.

To stabilize the strut:

- A transverse strut and brace system, and;
- A localized fly brace system to brace the vertical strut below Strut 1 at midspan to the RHS purlins to either side of the truss.

The first option of the two was selected for the following rational:

1. A strut and brace system did not clash with another reroof project to be atop the existing RHS's.
2. The fly brace system utilized existing elements and connections that were either weak and/or highly loaded.
3. The fly brace option would have required hot work very close to the existing roofing which was already known to have leakage and durability issues (reference the roofing failure).

See an elevated half section of the transverse strut and brace system adopted below in Figure 6.

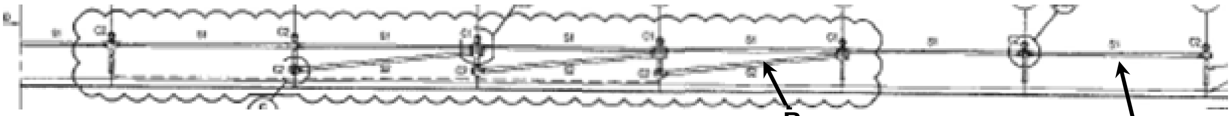

Figure 6 –Transverse Strut and Brace Strengthening Concept

4. CONCEPT DISCUSSION

Due to the location of works, a number of issues became more important to consider. These were generally as follows:

Construction Safety

The works were required to be fitted on top of the stadium roof, 14m in the air and across roofing that had historically been subject to damage by servicemen. The underslung roofing was already the subject of a water tightness review of existing leakage. There was an area of age/UV embrittled opaque roofing that separated the area of works from the more accessible top fixed roofing over the members lounge areas. This area of roofing had a 5m fall beneath it without mesh. It had also been installed in such a manner that repair/replacement of this roof would be expensive.

Construction Access

Due to the roof height and accessibility of the works areas, access was required to be considered early to assist in mitigating construction safety, maintaining quality and planning for a reasonable construction form. The more awkward the access, the longer contractors would be exposed to the risks of working on the roof.

Construction Quality

This was a concern for both existing stand features and for any new elements. The risk of construction activity damaging the existing roof was high, while any new works up on the roof would be of lesser quality due to the extreme access issues.

Safety in Design

Due to the preceding issues and both the TRC and NPDC appreciating them, we were able to bring in the contractor early to derive the benefits of the client, operators and the contractor (Fitzroy Engineering Group Limited - FEGL) being on board with a safety in design process as well as specifically addressing the issues above. This allowed a collaborative approach throughout the project life.

5. DETAILED DESIGN:

From early conception the intent of the design was to have the following features:

- Transverse collector horizontal struts that pick up stabilizing loads (assuming they are all additive, and of the incremental order of 2.5% of the primary axial load – reference NZS3404 1997).
- The collector struts are to then be diagonally braced to nominally above the bottom chord of the truss.
- The existing vertical strut within the truss then transfers the stabilizing actions to the existing roof plane struts and braces, thereby distributing actions into an existing structural system that has redundancy for this type of load combination.

The structural braces were selected as CHS's for various rational, such as:

- Ease of coating.
- The ability to accommodate significant compression loads as well as tension.
- A CHS could be consistently and efficiently used for all member types while remaining consistent with the existing structure.

The Strut/Brace Design

The CHS strut design was a conventional approach (127CHS), however the connection methodology was job specific, developed with the issues noted in Section 4 in mind. The strut/brace connection design was generally as follows:

- The connection cleat was a knife plate shop welded into the CHS.
- Connection of the cleat to the connection on the primary structure was via friction grip (TF) bolting and slotted holes.
- The slotted holes allowed for additional site tolerance beyond a site measure, while the TF bolting allowed for a non-slip primary load path connection to accommodate the connection play deriving from the slotted holes and the use of bolts in shear to take the axial strut/brace loads.
- To mobilize the accumulated stabilizing action loads, the TF bolt interfaces utilized a specific localized coating system. The steel to steel interfaces were coated in an inorganic zinc coating to both provide some corrosion protection while still providing a high friction surface interface to maximize the connection capacity.
- In the event of actions exceeding the

connections capacity (which is smaller than the member), the joint would slip (while still maintaining the joint TF capacity) into a new position 10mm offset whereby the bolting can then behave also in single shear as well. Therefore the connection can be said to have considerable redundancy in it. The 10mm deformation in the strut connections would also have associated deformations in the restrained Strut 1. However the order of these additional moment actions is insignificant relative to the length of Strut 1 and can be easily accommodated.

- Refer to Figure 6 for an elevation of the strut/brace members, and refer to Figure 8 for the connection details of the strut/braces.

The Strut/Brace Design

The Structural Steel clamp adopted in the detailed design is the element that allowed the works to progress. The clamp design provided for the following considerations:

- It forms the connection of the struts/braces to the primary structure.
- Provides vertical cleats for the new strut/braces, while also allowing for clamping the connection in two halves around the existing primary structure.
- With this scheme the clamps, struts/braces are all easily removable and able to be fully fabricated and coated offsite.

The success of the joint relies on the clamping action around the existing vertical strut in the roof truss. To address this, the following was conducted to make provision for construction conditions:

- Early site measuring of the 163CHS existing struts was conducted by FEGL.
- An allowance for a UV resistant rubber sleeve was made between the existing CHS outer diameter and the inner diameter of the clamp.
- TF bolting was again adopted to mobilize a clamping action on the 163CHS.
- All TF bolts also used load indicating washers to assist in site QA.
- Refer to Figure 8 for and extract of the clamp and strut/brace connection details used.

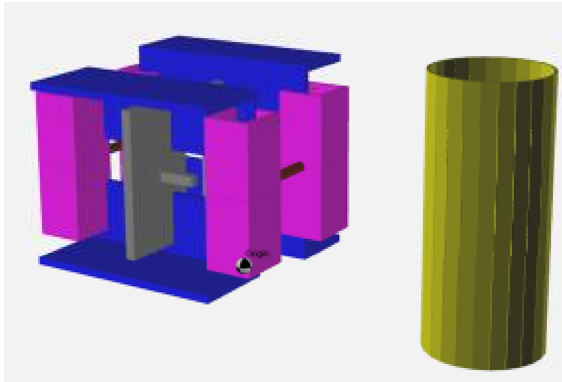

Figure 7 – Space Gass Clamp Model

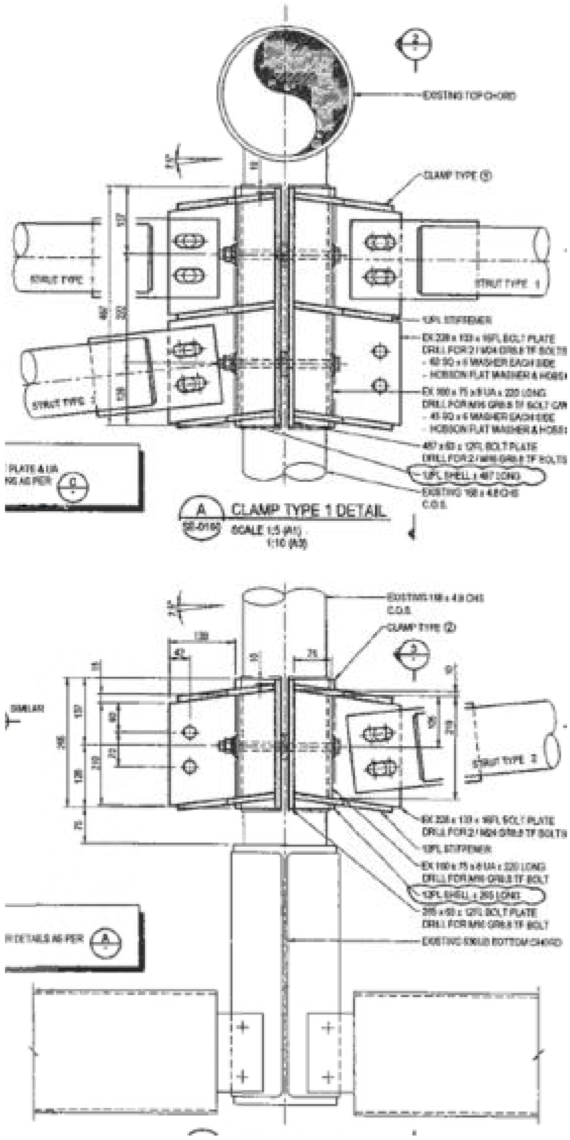

Figure 8 – Clamp detail drawing extract

To assess the influence of the stabilizing actions and clamping actions on the clamp parts as well as the existing structure, calculations were conducted by hand then verified by the use of a Space Gass model of the clamp (refer to Figure 7). The findings of this exercise raised the following:

- That the clamping load as well as action on the existing 163CHS vertical strut was acceptable even in consideration of the existing loads in the CHS from uplift actions.
- That the loading distribution the clamp was very sensitive to its support conditions (ie the rubber sleeve and clamp stiffness).
- That the clamp bolting to the existing strut had to be minimized closely to the design actions as when TF bolting is torqued to minimum tension and the clamp has room to deform, then the clamps take on significant load.
- To address even minor loading the clamps needed to be significantly stiffened in the detailed design, thereby taking the single clamp half weight to 28kg, and the double clamp half weight to 50kg. Installation methodology became topical due to difficulty manhandling of these weights.
- The required clamping load to keep the clamp in position and to transfer the vertical actions into the existing structure was relatively minor, again this mechanism had redundancy such that in the event of significant loading the clamps could deform while still maintaining their clamping capability.
- All TF bolts also used load indicating washers to assist in site QA.

The detailed design of the stadium wind strengthening included only two components, the clamp and the strut. These only varied nominally by:

- Length, for the struts versus the braces and site measurement variations.
- The single versus the double clamp that took the braces.

A last note on the clamps, due to the enclosed spaces on the clamp bolting stiffening angles, the clamps were hot dip galvanized initially. Top coating was via a 2 pot epoxy paint system. Therefore the clamps have a very long time before probable first maintenance is likely to be required. However as an added measure all stiffeners that were likely to trap water were pitched at 7.5 degrees to the horizontal to actively shed water rather than retain it and the probable associated chloride laden deposits. Therefore the coating system on the clamp is more durable that the surrounding systems, they can also be locally removed to be recoated if required.

We considered using the bolting arrangement to fit the clamp and connect the strut, but found that the need for high friction in the strut connection and tolerance in the clamp fitting were incompatible or overly stressed the clamp. Significant interaction was conducted between Beca and FEGL during the entire detailed design, the workshop drawings, testing and fabrication stages ultimately deriving benefits during the construction stage.

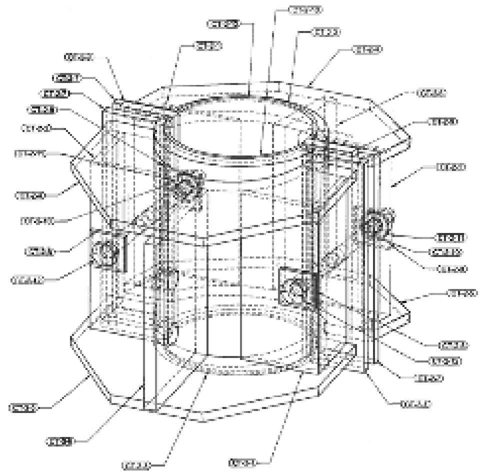

Figure 9 – FEGL Workshop ISO of Clamp

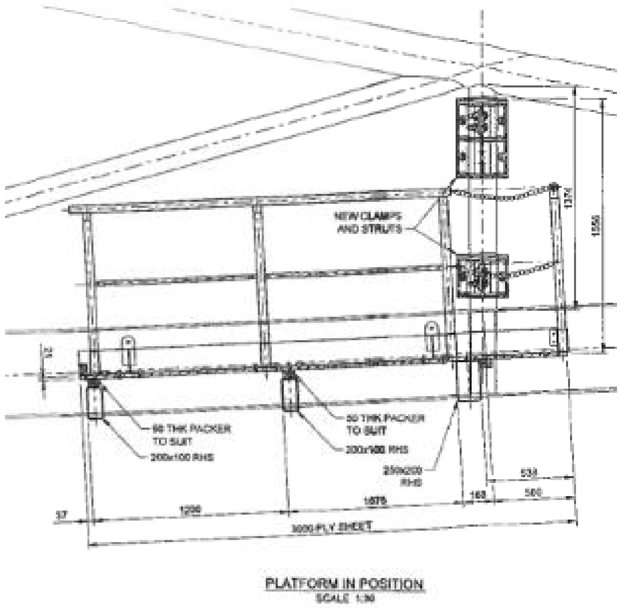

Figure 10 – FEGL Shop Drawing of Craned Access Platforms

6. CONSTRUCTION:

The workshop drawing, fabrication and coatings and installation methodologies derived mainly from FEGL and their aim to provide the best product in the best way they could. Their experience and depth in combination with their active interaction in the Safety In Design (SID) workshop and interactive discussion of detail and works methodology led to many benefits for the project. A likely non-complete list of these are as follows:

Workshop Drawings:

All shop drawings for every plate and element were produced in Steel Pro. They included complete material take offs, weights and site measured variations. Refer to Figure 9 for a sample isometric shop drawing of the clamp.

100% prefabrication and coating:

All elements were able to be shop fabricated. Therefore weld and coating quality was high and able to be fully inspected. No rework was required on this project.

Workshop testing of fits and bolt tensioning:

A mockup of the braced assemble was developed in the workshop by FEGL, thereby allowing a test fit scenario prior to the elements getting to site. Refer to Figure 12 for photos of the shop assembly

Bespoke access platforms:

Due to the inaccessibility and safety concerns of accessing the roof, FEGL developed bespoke access/work platforms that spanned purlins and allowed discrete work areas at the clamps to be worked safely. These platforms, 3 in total were able to be lifted by crane from one position to the next very rapidly. Refer to Figure 10 for an extract of the shop drawing of these platforms

Clamp lifting jigs:

Clamps were efficiently fitted by use of a set of bespoke steel plate lifting jigs that allowed the clamps to be lowered by a spreader beam, separated above the top chord (Strut 1) and then lowered and fitted into position around the vertical 163CHS. The crane could then finely adjust the clamp heights prior to the fitting bolts being tightened to fit the clamp in place. Once fitted the jig/crane assemble would then bring up the next one for the next joint creating a reliable, repeatable, rapid process. Refer to Figure 11 for a sample of the shop drawing of this jig.

Load indicator washers:

Load indicator washers were adopted for use on all TF bolting, specifically “Squirter Washers”. They are so called due to the washers having “buttons” that deform flat at the appropriate bolt pretension and “squirt” out an indicator mark of paint. These allow a rapid review of tensioning completion to allow work to progress. However it was found in conjunction with FEGL that on the clamps with minor deformation (due to tensioning of the clamp bolts), bolt nuts do not activate the squirter buttons if the nut is not completely true. This can lead to over tightening of the TF bolts in an attempt to activate the washers. This was observed during the mockup testing stage (refer Figure 12) when FEGL were doing a parallel check using a torque wrench. Therefore, in this instance, they were useful to confirm a level of tensioning had been achieved, but not necessarily the final tension. Also, the clamp bolting to the existing strut did not critically need to be at full tension. Therefore torque limits were seen as conservative in this case to test on the lower side of the minimums, due to them acting in tension as well as being pretensioned.

Friction Grip bolting:

The use of TF bolting in both shear and in tension allowed the site assembly of a dimensionally stable structure with an abundance of construction tolerance, therefore avoiding lost time and roof top site works.

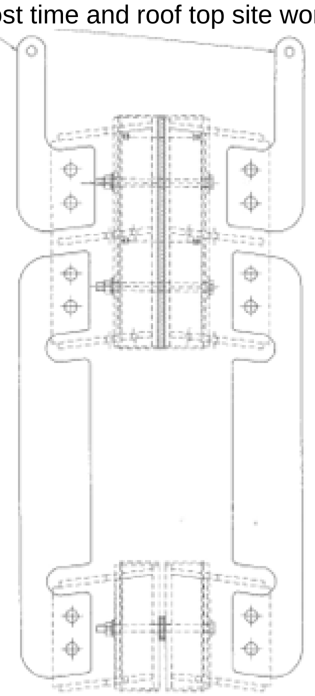

Figure 11 – FEGL Shop Drawing of Clamp Installation Jigs

Figure 12 – Photos from FEGL's workshop depicting the mock assembly of components

7. PROJECT RESULTS:

From review of the project at completion the following can be surmised:

- The design intent achieved a robust strengthening scheme through collaborative early contractor involvement, enabled by the client (TRC/NPDC) who was able to appreciate the benefits of this process from the outset.
- A safe construction process was established and followed through by the contractor including bespoke access and installation assemblies. No accidents occurred during this project.
- The combination of a considered design and construction methodology meant a quick, efficient construction.
- Offsite work also protected the existing structure and roof from site work related damage.
- The strengthening is fully removable and maintainable.
- High quality steel products were achieved by workshop fabrication and painting in combination with experienced contractors with formal quality assurance practices (FEGL).
- 40% cost reduction from estimates based on conventional connection methodology.
- Functional installation occurred at **3.5 working days** for a total of 4T of structural steel over a 100m transverse long truss. Therefore contractor risk exposure time was short.
- Subsequent installation of the same system on the opposing stand was fully installed between engineer's site visits, albeit with improved access to the work stations.