Cowles Stadium, Wainoni, Christchurch

QUANTITATIVE ASSESSMENT OF STRUCTURE

- Report 3
- 20 December 2011

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Contents

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1. Executive Summary

- Analysis indicates that the chords of the trussed arches will achieve 70% New Building Standard (70%NBS) at which point yielding is likely and member buckling possible. If this occurs then the global stability of the arch trusses can not be assured. There does not seem to be a practical way of enhancing the chords to a higher level than this.
- Analysis further indicates that the existing web-members reach their buckling load before code (NZS1170.5) level loading and achieve 45%NBS. It is likely that this loading was exceeded during 22 February 2011 earthquake. Once buckled the webs become ineffective such that the building in its current state has very low tolerance to further earthquake loading (approximately 10%NBS). Practical ways of enhancing the web strength have been devised.
- The current buckling is explained as being a consequence of greater than loading code earthquake effects and also possible ground deformation, either momentary or permanent, creating stresses in the arches greater than those predicted by analysis.
- Replacement is advocated for all arch webs within the first 3.6m of the arch springing and also any members beyond this region showing buckling distress. Over the whole arch length web members should be laced together by welding a longitudinal tie rod along their mid-section to bring them up to 100%NBS standard (note that there is no lesser standard viable – it is a case of upgrade or don't upgrade). This proposal is shown in Appendix B.
- The longitudinal cross bracing members in each of the four corners of the hall fall short of meeting NBS demand, achieving only 55%NBS. To achieve 67%NBS the existing braces need to be upgraded with a stronger section and to achieve 100%NBS requires also an additional set of bracing each side.
- The roof bracing does not meet required strength capacity. To achieve 67%NBS requires upgrading the end two sets of braces in each corner of the building. To achieve 100%NBS will also require an additional bay of bracing, which must not be adjacent to any of the existing bays.
- The buttress frames appear to be adequate, including the special case where the frames each side of a modified frame (brace member removed) are being called on to carry additional load.
- Typical foundations are found to be satisfactory but those relating to the longitudinal cross bracing are deficient and require upgrading. This can be achieved either by adding further mass to the foundations or alternatively installing screw piles near each corner. The latter approach is preferred.

2. Introduction

2.1. Introduction

Cowles Stadium, located on Pages Road, Wainoni, Christchurch, is an indoor basketball stadium and has suffered significant buckling of some web members in the roof trusses, most probably as a result of the 22 February 2011 earthquake. There is also evidence of other non-critical earthquake damage to the building, which we understand is being attended to by others. Investigation indicated that the trusses seemed to have "narrowed" in some locations relating to the buckled members and also stadium staff had twice needed to adjust the basketball hoops with respect to their clearance above floor level. The continued stability of the roof structure could not be assured and hence the stadium was put out-of-use pending further investigation of the likely cause of the damage, the impact on stability, required repairs, the general rating of the building's earthquake strength in relation to current code requirements and any recommendations for improvement.

SKM was engaged by Christchurch City Council to carry out this investigation. SKM has produced two previous reports as the investigation into options developed, dated 25 July 2011 and 25 August 2011. Subsequent to the second report SKM was engaged to prepare documentation for the repair and upgrading of the superstructure. Late in this documentation phase the report of a parallel geotechnical investigation of the site came available the results of which had a direct bearing on the proposed remedial works which were therefore suspended pending investigation on the full impact.

This report presents the current situation and incorporates the two previous reports where still relevant.

3. Building and Damage Description

3.1. Building Description

Cowles Stadium (Photo 1) is primarily a single hall containing two basketball courts (Photo 2) plus tiered seating at one end equal in area to about a further half court. There is a lean-to area down each of the long sides of the hall which provide storage, changing/toilet facilities, kitchen and the like. The hall is orientated on a North-East axis with overall dimensions of the hall at 45.7m x 36.6m and the lean-tos at 6.1m wide. The South-East lean-to extends beyond the end of the hall by about 20m (Umpires Room) but this appears to be subsequent construction.

The hall roof is supported by seven shallow trussed arches spaced at 5.71m (Figure 1) plus the two end-walls. By "trussed arches" we mean that they are fabricated like trusses and look like trusses but their structural actions are, primarily, that of an arch hence working primarily in compression under gravity loads. However under seismic actions bending in the roof arches will occur and this is resisted partially in truss action. The trussed arches support simple timber purlins and sarking with aluminium cladding above.

The trussed arches take the form of two chords consisting of 4"x2" channels, both with toes down, interlaced with pairs of 10mm diameter steel rods at approximately 45 degrees (Photos 3 and 4).

The lean-tos form a braced buttress at each gridline to provide restraint to the arch thrusts and bracing against lateral loads. In the longitudinal direction lateral resistance is provided by simple cross bracing in each of the four end bays.

In one location one of the diagonals of the south-east lean-to has been removed and its function replaced with a truss in the plane of the lean-to roof so as to shed the loads that the removed brace would have taken to the frame in the bay either side (one of which is the end wall). Thus the neighbouring buttress frame takes 50% more load than it would as a typical frame.

The buttress frame columns are supported on simple shallow pad foundations but there is a steel tie-rod passing over or through the soil between the pairs of footings under the main columns and the corresponding lean-to outer columns. These rods are corrosion protected.

The floor construction is simple timber floor supported on isolated pads.

3.2. Building Damage

The following description is not a comprehensive damage report or the consequence of a detailed damage survey, but a description of the general nature of damage observed during this investigation.

The damage of principle concern, that triggered the need for this investigation, is that a significant number of trussed-arch diagonals, primarily at the south-east ends of the arches have buckled substantially (Photos 3 and 4). Displacement of a previous build-up of dust on these diagonals suggests that this is recent, hence earthquake related, damage rather than as a result of some earlier demand. Generally this damage occurs only on the S-E side of the hall but it is understood that the diagonals on the N-W ends have been upgraded to heavier sections at some time in the past. The reason for this is not known but it seems likely that the building may have suffered similar distress in the past to have triggered this upgrading.

Measurements taken between the top and bottom chords of selected arches indicate that the chords have “narrowed” towards each other to some extent. This is consistent with the buckling of the webs.

The block walls to the perimeter show numerous cases of diagonal herring-bone cracking of minor nature (Photo 5).

The S-E lean-to extension has separated from its adjacent structure by about 30 – 40 mm and similar (but slightly less) movement is exhibited by the S-W wall of the hall relative to the hall floor (Photo 6).

Some cracks in external pavement were observed indicating some limited ground stretching and this would be consistent with movement of the umpires room and SW wall of the hall.

3.3. Information Provided

In addition to a site inspection SKM was provided with:

- The original structural drawings.
- Newly drafted floor plans, elevations and cross section.

4. Analyses and Interpretation

4.1. Analyses

SKM carried out several computer analysis studies utilising Microstran software and associated hand calculations of member capacities. The Microstran analyses consisted of:

- 2D typical cross section for the following load cases:
  1) Self-weight and Deadload
  2) Code seismic load based on available ductilities of 1.25 and 1.0[1] and soil type D[2]
  3) Snow Load
  4) Combination of 1 and 2
  5) Combination of 1 and 3
  6) Specified vertical displacement of brace foundation connection to simulate ground distortion during the earthquake. The value used was intuitive (see explanation below).
  7) Special case cross-section analysis for frame adjacent to modified frame thus picking up additional shear.
  8) Wind load and combinations with dead and self weight
- 3D model of whole building structure.

The specified displacement case was intended to study the momentary situation of the seismic wave rolling past the building or possibly permanent ground distortion. There is no code guidance or requirement to consider this phenomenon but it seemed relevant in trying to account for the building's distress. The case allowed for 100mm vertical displacement of both primary column supports which translated into a general compression of the roof arch. This value was probably unrealistically large but was selected to be easily scalable (pro-rata) for other displacements and to deliberately exaggerate the effects for clarification of interpretation. For this reason it was not combined with other load cases.

The output of these analyses were used to identify peak member actions and these were compared to member capacities for yielding and buckling assessed using both loading codes and steel code requirements.

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[1] Ductility is a measure of the structure's ability to yield when subject to overload rather than suffer a brittle failure. The numeric value is the ratio of yielded displacement to elastic displacement. Ductility of 1.0 implies an elastic response with no ability to yield. Ductility of 1.25 implies almost elastic but with a very nominal yielding ability. A modern ductile structure could have a ductility as high as 6.0.

[2] Soil Type D is a soft deep soil typical of the Christchurch area.

4.2. Load Derivation

To establish the New Building Standard (NBS) input seismic loads the following parameters (with respect to NZS1170.5) were used:

- Christchurch location $Z = 0.3$
- Assumed period $T = 0.7$ seconds (subsequently checked by Rayleigh method)
- Importance Level 3 (greater than 300 people) giving $R = 1.3$
- Life   50 years
- Distance from known fault $N(T,D) = 1.0$
- Ductility elastic $\mu = 1.0$ although the influence of $\mu = 1.25$ was also examined.
- Performance factor $Sp = 1.0$

These combine to give a seismic coefficient $C = 0.987$

With respect to NZS3404 *Steel Structures Standard* Cl 12.12.6.3.2 the structure was classified as Category 4 (elastic) with $Cs = 1.0$ for $\mu = 1.0$ or Category 3 with $Cs = 1.1$for $\mu = 1.25$.

4.3. Findings

<u>Trussed Arches</u>

- Under load condition 4 (as section 4.1), based on the factors in section 4.2 (with ductility μ = 1.0), the chord members are unable to reach 100%NBS under combined axial load and bending. By back analysis we have determined that yield in the chord members occurs at 70%NBS with fairly even bending stresses along the full chord member length. Consequently in the event of loading greater than 70%NBS there is a risk of local chord buckling which could lead to structural failure of the arch.
- The above bullet point applies on the assumption that the rod web members remain functioning without buckling. If the web members fail in buckling (as has happened) then the chord would quickly lose its compression capacity at a lower %NBS value.
- The calculations indicated that the existing web member capacity is of the order of 45%NBS with buckling as the likely failure in the event of over-load. Overload clearly can and has happened and a buckling failure is a non-ductile and potentially unstable failure. Webs that have already buckled can be deemed to have close to zero strength capacity remaining leaving the arch-trusses vulnerable to collapse if repair and strengthening is not carried out.
- The specified displacement case indicated severe compression loads in chords and webs and also quite significant bending. Thus ground distortion can have a devastating effect on a structure of this type in addition to the effects of earthquake acceleration.

- We believe that what has occurred with the webs is that they were placed under significant compression due to seismic accelerations probably higher than code values but that the arch as a whole may also have simultaneously suffered significant end rotation as a consequence of ground displacement such that the webs were then subject to compression and high bending actions in excess of their capacities.
- Study of the photos (more than those reproduced with this report) indicated that the webs generally (though not entirely) buckled in the plane of the truss. This would be consistent with bending of the truss being a primary influence on the buckling behaviour whereas buckling due to compression alone would be random in direction.

Buttresses

These were found to be generally adequate for code loadings with significant reserve.

The special case frame that carries part of its neighbour's load was also adequate.

Roof Bracing

The roof bracing is overloaded and achieves only 39%NBS. However for loads under 67%NBS overstress is limited to the end two sets of braces in each corner.

Longitudinal Cross Bracing

The capacity of the bracing angles was found to meet only 55%NBS.

The wall linings were opened up to review the bracing joint details (which were not clear on the original drawings). The braces were welded to the face of the box-columns with weld capacity in excess of the member capacity.

Foundations

The foundation pads do not have sufficient mass to withstand the seismic uplift loads generated in all locations. Two scenarios were considered. These two scenarios were:

1. Analysis under NZS1170.5 seismic loads with ductility of 1.0 (elastic) and with Cs = 1.0 (NZS3404 12.12.6.3.2(d))
2. Analysis under NZS1170.5 seismic loads, allowing for a ductility of 1.25 (nominally ductile) and Cs = 1.1 (NZS3404 12.12.6.3.2(c))

In all cases scenario 1 controlled.

The uplift forces on foundations are generated from a quite complex interaction from three sources:

1. Roof truss arch action under gravity load tends to generate tension in the main column foundations with compression in the lean-to foundations.
2. The wall cross bracing in both main wall and lean-to walls tends to cause tension in the pad that is away from the direction of the earthquake – that is corresponding to the brace that is in tension at any one instant.
3. The buttress braces and wall bracing combine to provide a partial end-fixity to the roof bracing which can generate tension in both the main wall and lean-to foundations.

Generally the foundations associated with the longitudinal bracing do not have sufficient mass to resist these combined forces and consequently avoidance of foundation uplift tends to be the controlling strength limitation. The lift load is variable as indicated in the table below.

4.4. Summary of Code Compliance

The table below summarises findings for the various structural components from this analysis (capacities in excess of 100% are stated just as "Complies").

Case	Component	Ability to meet the load implied by NZS1170.5.
1	Arch Truss webs Ach Truss Webs - damaged Arch Truss chords	45%NBS <10%NBS 70%NBS
2	External brace buttressing the roof arches	Complies
3	Internal brace buttressing the roof arches	Complies
4	Roof bracing	39%NBS
5	Longitudinal bracing members in each corner, main walls	55%NBS
6	Longitudinal bracing members in each corner, lean-to walls	Complies
7	Joints to longitudinal bracing members and baseplate details	56%NBS
8	Foundations to typical cross section without bracing	Complies
9	Internal Foundation associated with longitudinal cross bracing#	Complies
10	End wall Foundation associated with main longitudinal cross bracing#	71%NBS
11	External Foundation associated with lean-to longitudinal cross	38%NBS

	bracing#	
12	Corner Foundation associated with lean-to longitudinal foundation#	32%NBS

The quoted values are based on the typical corner arrangement as exists for three corners of the building. The corner where the buttress has been modified will be more severe for cases 9 and 10 but less severe for cases 11 and 12.

4.5. Detailed Engineering Evaluation Results

Our detailed engineering analysis indicates that in its damaged state Cowles Stadium achieves only approximately 10%NBS, limited by the arch-truss chords within the damaged web zones. The next webs beyond the damaged zone are also highly stressed and achieve only approximately 10%NBS. Thus there is a danger of an "unzipping" effect if the building is subjected to high loading with further webs buckling and the chords becoming unstable. If this happens then global structural buckling is likely with catastrophic collapse. Thus in its present state the building is Earthquake Prone, classifying as Grade E in the NZSEE system. This grade is summarised in the table below.

If the damaged webs were simply replaced with matching ones then the building remains marginally Earthquake Prone (<33%NBS) limited by inadequate hold-down mass in the foundations.

The Council policy states that since the %NBS for the current building is less than 33% the building is considered earthquake prone and so requires strengthening. Please note that structural strengthening is not required for buildings that have higher than 33%NBS but strengthening may be desirable by the building owner to reduce the risk of building damage or failure and decrease the risk to occupants.

Building	**Date of Drawings**	**%NBS Score**	**Risk**	**Grade**	**Structural performance**
Cowles Stadium, Pages Road, Aranui, Christchurch	1960	<20%	High	E	Earthquake Prone, strengthening legally required.

5. Recommended Remedial Work

In the following sections two levels of strengthening are considered being either 100%NBS or 67%NBS.

5.1. Trussed Arches

The arch truss webs require repair and strengthening before the stadium can be re-occupied. The following works are recommended: This work is independent of which level of strengthening is being considered.

- Any arch being worked on should be propped at ¼ and mid points with the support being provided via the top chord rather than the bottom. This is very high propping and will need careful planning. Notwithstanding that the prop loads will be quite light the props should be positioned and designed such that the loads are directly above the floor support pads and not via bending in the flooring or floor joists.
- Remove the following web members: all webs over the first three purlin spacings (approx 3.6m) whether buckled or not; all buckled members beyond this region, including any straight members that are between buckled members.
- Jacks (or similar) should be placed between the chords over these regions and the spacing between the chords returned to the original spacing (380mm top to top).
- The removed webs should be replaced using 16mm diameter grade 300 steel rod.
- Web members should be laced by welding a longitudinal 12mm rod to each web at mid-height of the truss.
- The above action does not need to be carried out in any regions where the 10mm rod has already been replaced with 16mm rod unless distress is exhibited.

This strengthening is explained in sketches contained in Appendix B

Although the chords were found to be overstressed at code level earthquake they were shown to be satisfactory at 70%NBS. We don't believe that there is any practical means to upgrade the capacity of the chords. Upgrading the webs as above should ensure that any future distress of the chords is local and not catastrophic.

5.2. Roof Bracing

The roof bracing requires strengthening of the existing bracing and supplementing it with additional bracing.

To meet **67%NBS** requires upgrading the end two sets of cross braces per end of each bay of roof bracing. This equates to 8 sets with 4 bracing rods in each set. The upgrade would consist of changing the current 16 diameter grade 250 rods to 20 diameter grade 300 rods.

To meet **100%NBS** requires the same upgrading as for 67%NBS plus the addition of a further complete bay of bracing across the roof, more or less in the centre of the building. The additional set of bracing must not share a common chord with the existing bracing (i.e. shall not be in the adjacent bay).

5.3. Longitudinal Wall Bracing

The cross bracing requires to be upgraded and the following work will be required:

Upgrading to 100%NBS will require removal of associated wall linings for access and replacement of the existing braces with heavier sections. Calculations indicate that 90x90x6 grade 300 equal angles would be suitable. In addition a further one set of supplementary bracing will be required each side of the building.

Upgrading to 67%NBS only can be achieved by replacing the existing bracing only, as above, without the need to supplement with additional bracing.

The bracing in the lean-to walls do not require upgrading.

5.4. Foundations

There is no point in upgrading the longitudinal bracing unless the foundations are also upgraded to be able to receive the load without going into uplift. Two possible means of upgrading have been explored, being increasing the weight of the foundations pads with mass concrete or alternatively utilising a deep hold-down device such as screw piles.

For increase in weight by means of mass concrete the following volumes have been assessed (the following values are based on a model with an extra bay of bracing for 100%NBS but no extra bay for 67%NBS):

Footing	100%NBS	67%NBS
Internal main column footing 1 in from each end wall	Nil	Nil
External main column footing at end wall	Nil	Nil
External lean-to footing 1 in from end wall	$4.9m^3$	$3.1m^3$

Corner footing	$4.2m^3$	$2.2m^3$

If screw piles are to be used for hold-down then they need to have a capacity of 225 kN. A total of 6 are required by calculation (one near each corner plus 1 each side for the additional braced bay for the 100% solution) however the presence of a transformer in one corner necessitates one pile either side of it, hence 7 total. For the typical corner these would be installed between the last two lean-to footings at each corner and slightly outside the line of the external wall. They would then be connected to the foundation pads either side by means of a new 6m long concrete beam per corner. This solution is shown in sketch format in Appendix B.

The minimum size screw pile will achieve 100%NBS hold-down and hence there is no 67%NBS case to consider.

6. Cost Assessments

A rough order of costing for the various options has been carried out by Rawlinsons. Their report is included as an appendix to this report but the following is a précis of the cost comparisons. Note that the values include for contractor P&G and Margin but exclude CCC charges, consultant fees and GST:

Repair and upgrading of all roof trusses (limited to 70%NBS) $90,000

Including temporary propping and access scaffold.

This work is common to, and must be added to, all of the bracing upgrading options presented below.

Upgrading fire protection (as per Fire Report) $176,000

This work is common to, and must be added to, all of the bracing upgrading options presented below

Upgrading bracing to 100%NBS

Upgrading existing vertical bracing	$60,000
Add additional vertical bracing	$157,000
Upgrade roof bracing	$43,000
Upgrading foundations by means of screw piling	$173,000
Total Bracing Option A1	**$433,000**
Extra/over to use mass concrete in lieu of screw piling	$26,000
Total Bracing Option A2	**$459,000**

Upgrading bracing to 67%NBS

Upgrading existing vertical bracing	$57,000
Upgrade existing roof bracing	$8,000
Upgrading foundations by means of screw piling	$173,000
Total Bracing Option B1	**$238,000**
Extra/over to use mass concrete in lieu of screw piling	$-38,000
Total Bracing Option B2	**$200,000**

The above pricing is further summarised in combination with the required ground improvements in a subsequent report.

7. Conclusion

We have undertaken a detailed engineering evaluation of the Cowles Stadium Structure to consider the seismic capacity of the building compared with New Building Standard (NBS). The outcome of this analysis indicates that in its damaged state the building has a capacity less than 20%NBS limited by the trussed arch chords. This capacity shows that the building is classified as earthquake prone and strengthening to a minimum of 67% will be required when the consentable repairs are undertaken.

We have also provided two possible strengthening options and associated cost estimates to inform the client and enable a decision over the future of the building. Option one being repair and strengthening to 67% of code and option two being repair and strengthening to 100% of code. The trussed arch chords cannot be reasonably strengthened to 100% of new building standard and hence the trussed arch chords only reach 70% of NBS, the remainder of the strengthening for this option has been designed to 100% of NBS.

We make the following additional recommendations if the building is to be repaired:

- A detailed strengthening design should be undertaken to confirm that the concept strengthening and the associated estimate is appropriate.
- A full strengthening and repair specification should be prepared accounting for the damage contained in the damage assessment report and strengthening as confirmed by the detailed design.

This report shall be read in conjunction with the SKM geotechnical report dated 24 November 2011 and a subsequent report which further develops and combines the possible structural and geotechnical solutions.

8. APPENDIX 1 - FIGURES & PHOTOGRAPHS

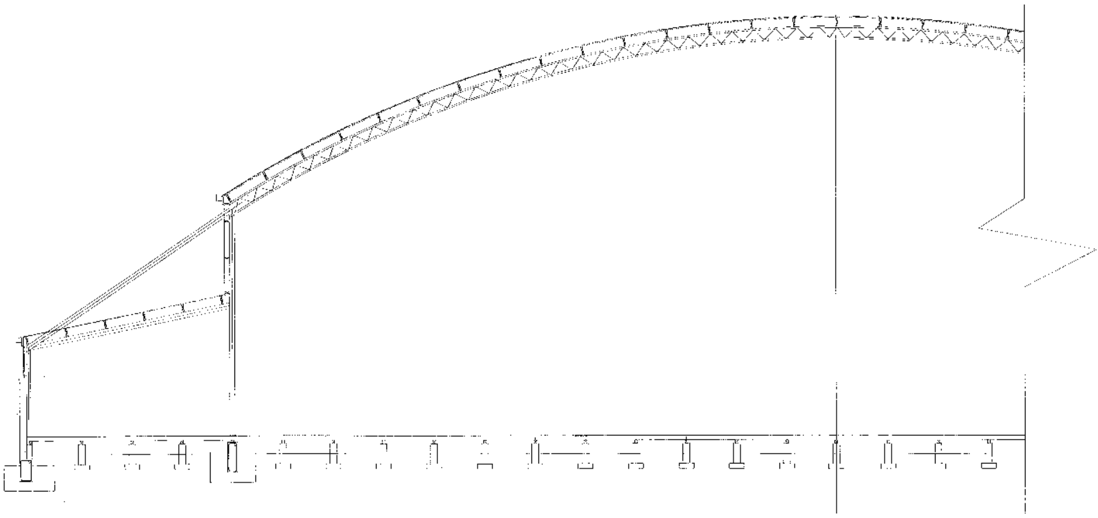


Figure 1 Typical cross section

Photo 1 – External view.

Photo 2 – Internal view

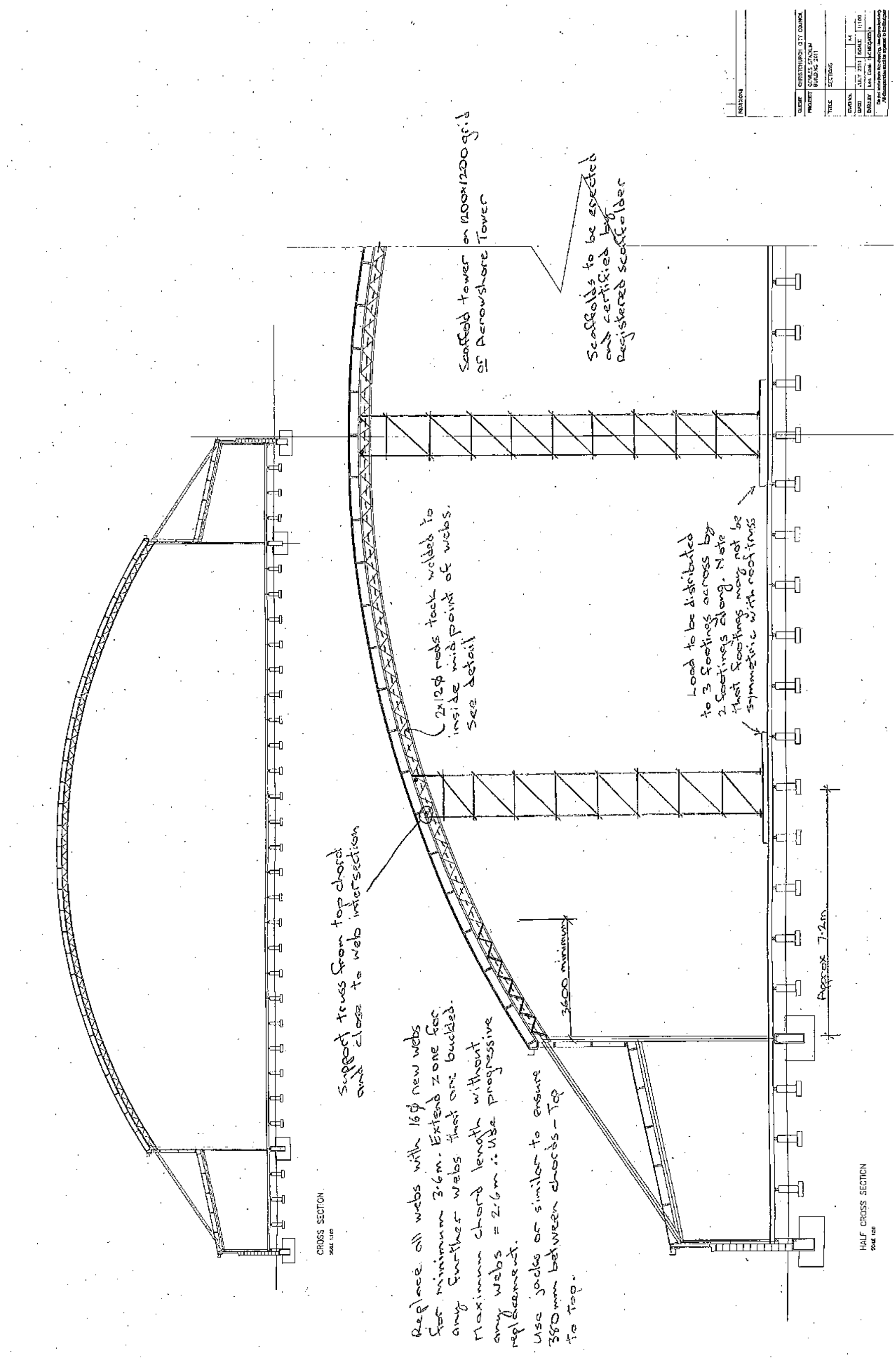
Photo 3 – Buckled web members and bent lower chord

Photo 4 Buckled web members

Photo 5 – Typical external wall cracking

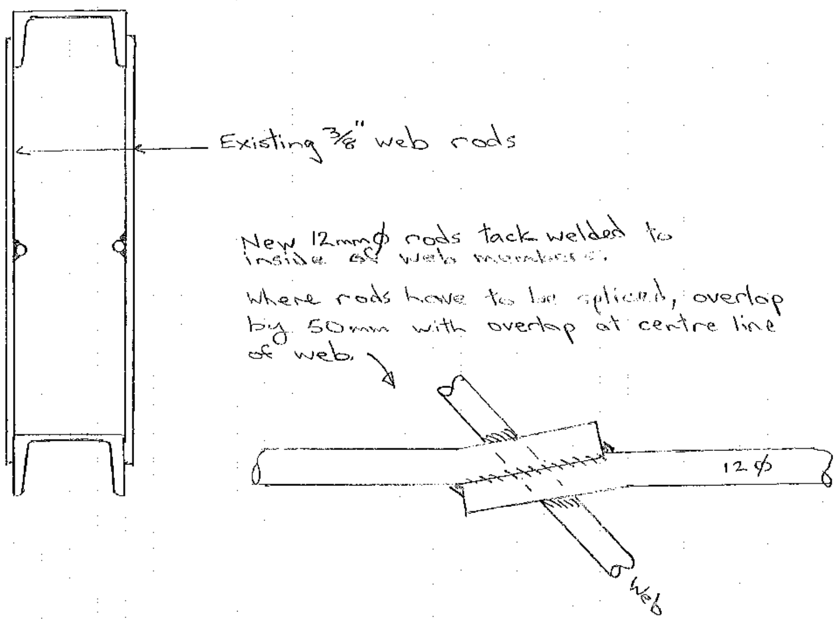
Photo 6 – Movement of South-West wall

9. APPENDIX 2 - STRENGTHENING SKETCHES


Scaffold tower on 1200x1200 grid or Acrowshore Tower
Scaffolds to be erected and certified by Registered scaffolder
2x12φ rods tack welded to inside mid point of webs. See detail
Load to be distributed to 3 footings across by 2 footings along. Note that footings may not be symmetric with roof truss
Support truss from top chord and close to web intersection
3600 minimum
Approx 7.2m
Replace all webs with 16φ new webs for minimum 3.6m. Extend zone for any further webs that are buckled.
Maximum chord length without any webs = 2.6m ∴ Use progressive replacement.
Use jacks or similar to ensure 360mm between chords - Top to top.
CROSS SECTION
HALF CROSS SECTION

SKM

	Job No. ZP01020
	Calc. Series
Client CCC	Page SK1
Job Name Cowles Stadium Repairs.	By TWR
Calcs Title Stiffening of Unbuckled Webs.	Date 4/8/11



SKM

Job No. ZP01020

Calc. Series

Client CCC

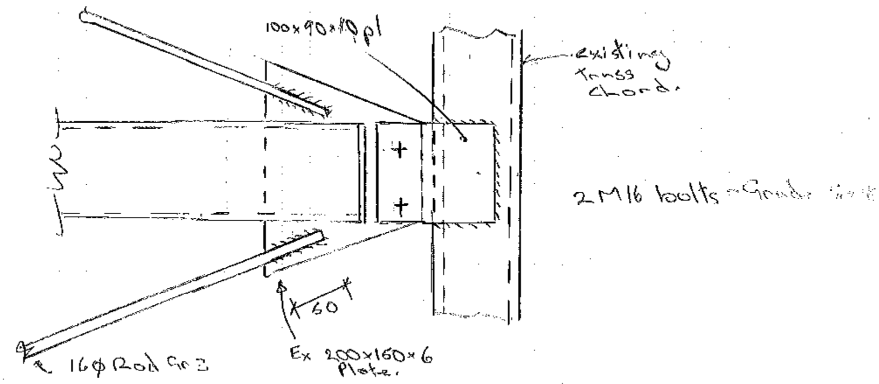
Page

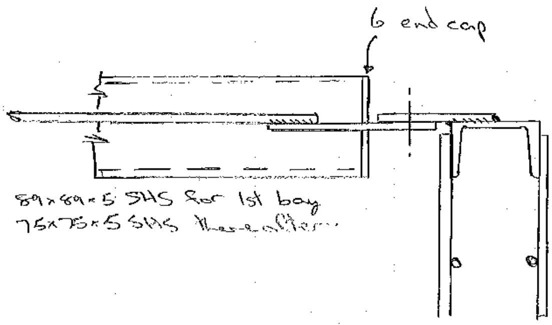
Job Name Cowles Stadium.

By TNR

Calcs Title Roof Bracing Upgrade

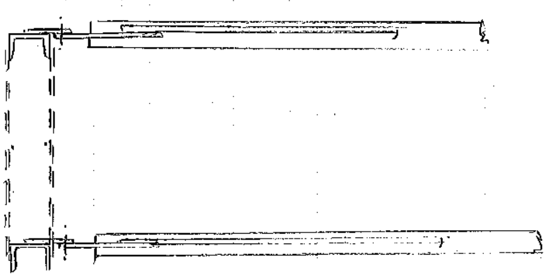
Date 8/8/11





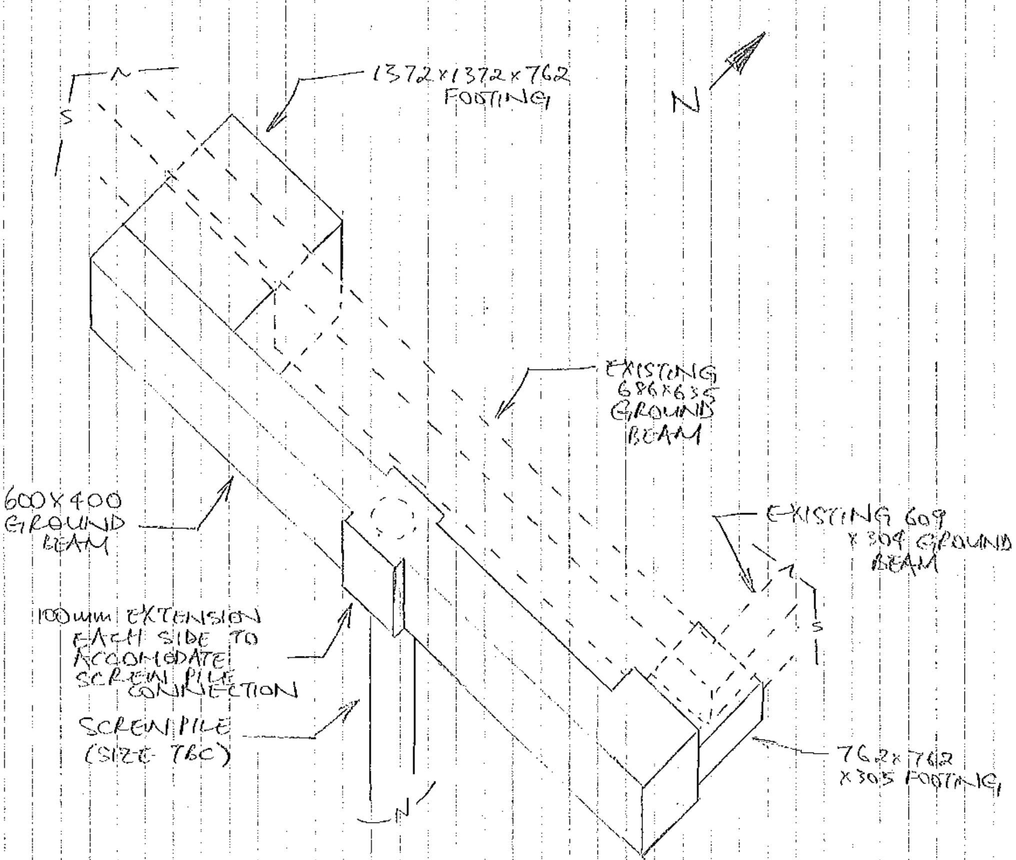
The option drawn achieves 75% of Code Loading.

To achieve 100% of Code substitute rod bracing with Reidbrace RB16 including end fittings. Change plates to 12mm.



3D VIEW OF EXTERIOR CORNER OF FOUNDATIONS:

ROUGH SCALE 1:500



NB: EXISTING GROUND BEAMS SHOWN FOLLOW LINE OF EXTERIOR WALL OF STADIUM

STEEL REINFORCING WEIGHT APPROXIMATELY 280 kg

10. APPENDIX 3 - QS ESTIMATES

RAWLINSONS

QUANTITY SURVEYORS

CONSTRUCTION CONSULTANTS

COST ENGINEERS

29 November 2011

SKM
Level 2, 12-16 Nicholls Lane
Parnell
AUCKLAND

Attention: Trevor Robertson

Dear Trevor,

COWLES STADIUM PROPOSED REINSTATEMENT

Please find attached our preliminary estimates of the structural reinstatement options for the Cowles Stadium as per your Seismic Assessment reports dated 25 July & 25 August 2011 and the Holmes Fire design report dated 18 August 2011 and subsequently updated by email 28 November 2011.

Main Bracing - 100% of Code	60,000.00
Main Bracing – 67% of Code	57,000.00
Trussed Arches Reinstatement	90,000.00
Upgrade Roof Bracing – 100% of Code	43,000.00
Upgrade Roof Bracing – 67% of Code	8,000.00
Additional Bracing V's Mass Concrete Options:	
Additional Bracing Bay – 100% of Code	157,000.00
Concrete Option 1 – Mass Concrete 100% of Code	199,000.00
Concrete Option 1a – Mass Concrete 67% of Code	135,000.00
Concrete Option 2 – Screw Pile	173,000.00
Fire Report Assessment	
Fire Upgrade as HF Report	176,000.00

The above estimates include for contractor P&G and margin, contingency and escalation but exclude professional fees and CCC fees.

Please do not hesitate to contact me should you require any further information.

Yours faithfully
Rawlinsons Limited

Andrew Millard
Director

O:\CSKM001\CSKM001 Cowles Stadium EQ Reinstatement\Reinstatement & Fire Upgrade Cover Letter & Summary 29 Nov 2011.docx

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