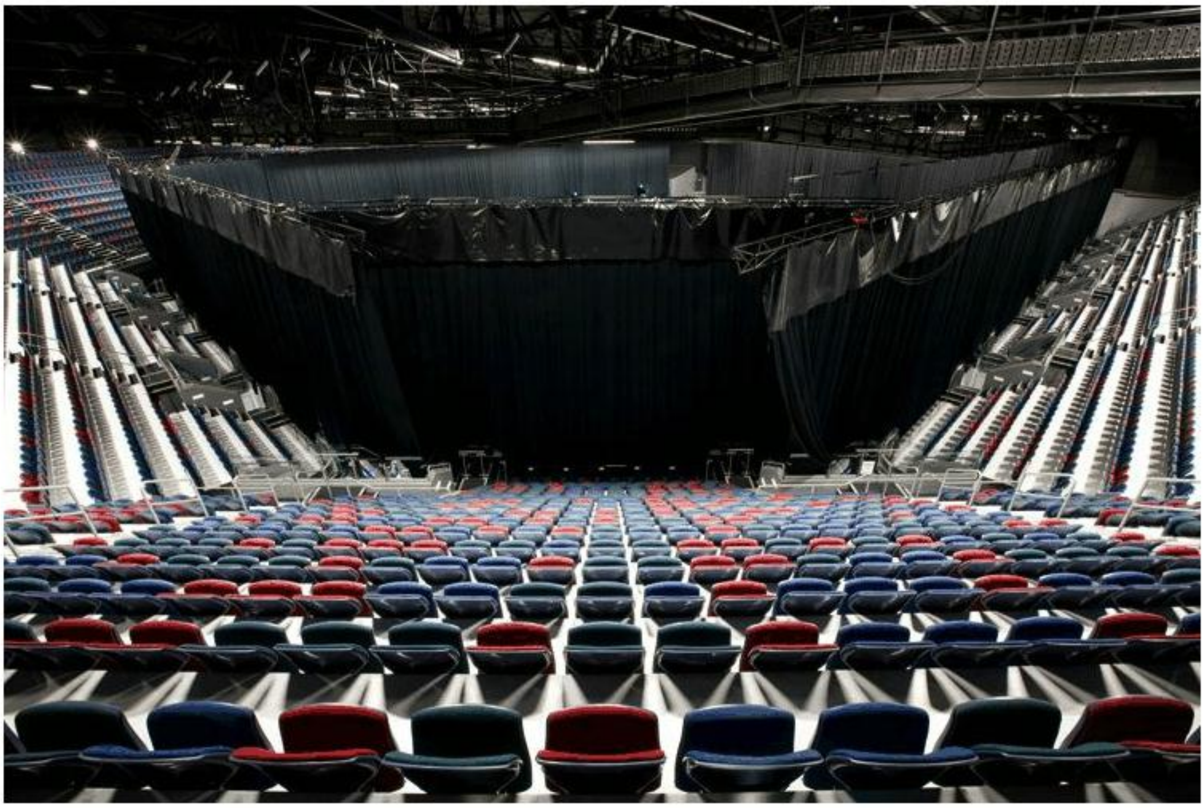
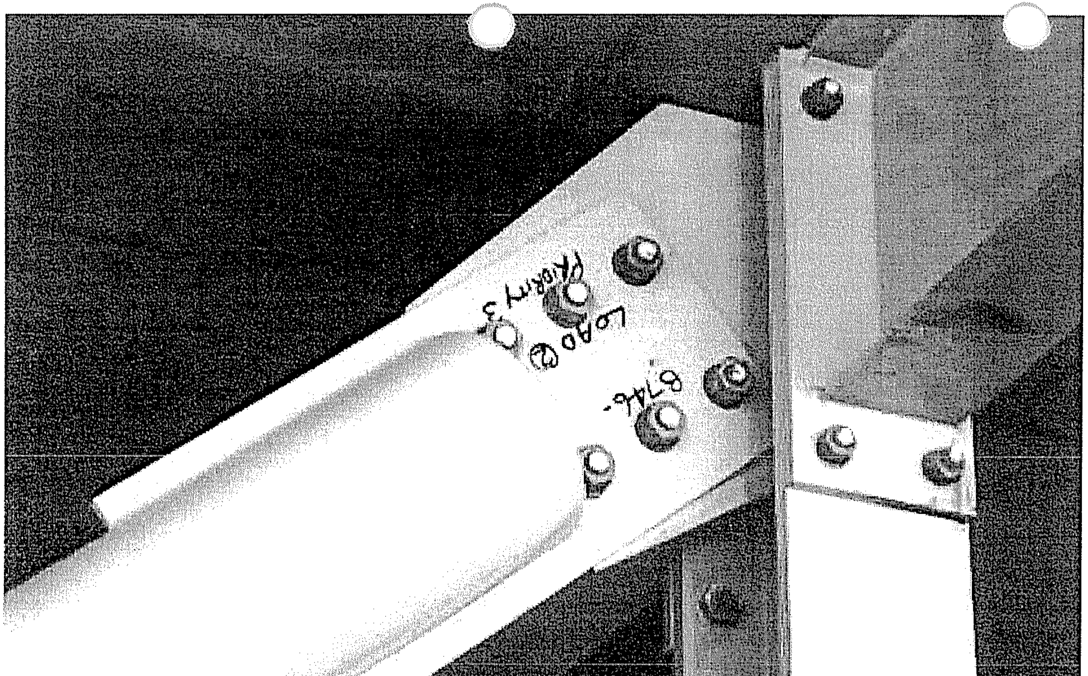
Waitakere Trusts Stadium

2003-2004

The Vector Arena

2005-2006

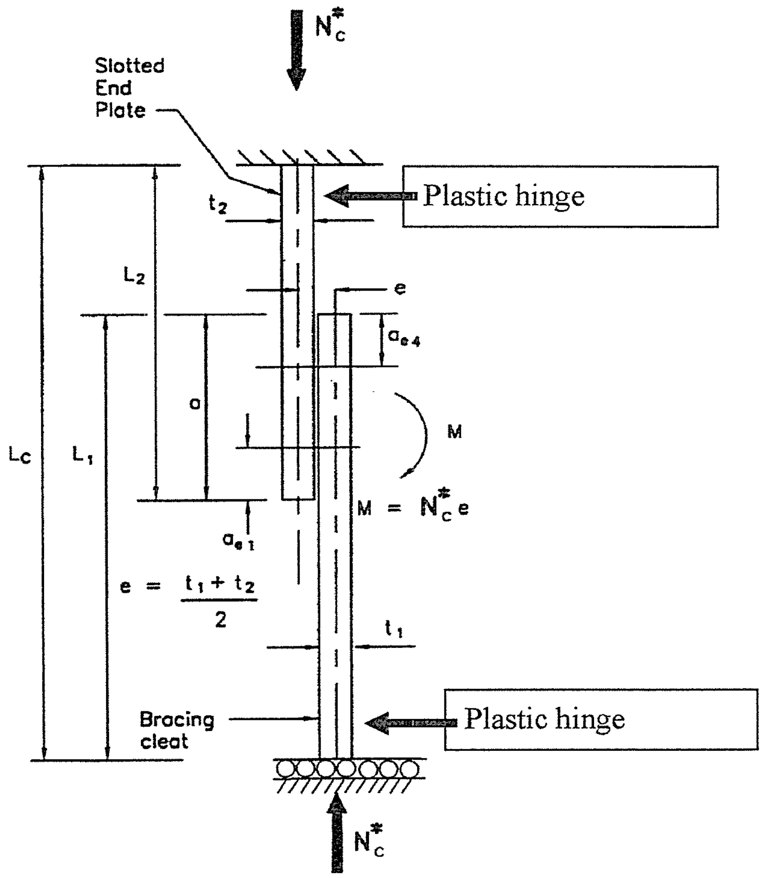


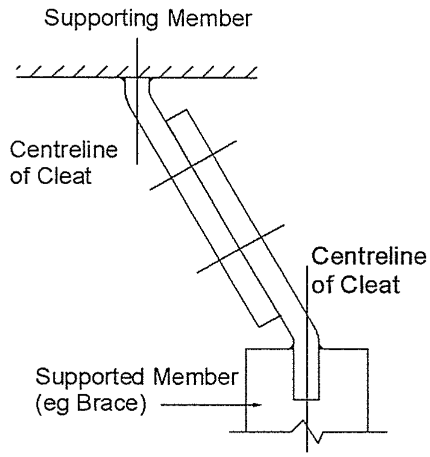

PRIORITY 3
LOAD 2
8746

Design Procedure for Eccentrically Loaded Cleats in Compression

Date of procedure: 6th July 2006

written by Dr G Charles Clifton, HERA Senior Structural Engineer, with input from a number of consulting engineers.





Sway mode failure of this type of connection

Notes in regard to the above diagram:
1 The failure mode of this type of connection is a sway mode, where the bottom support in the above diagram is not restrained against lateral movement. In practice this is the supported member end. The inset shows a diagrammatic representation of this mode of failure. This sway mode of failure will govern for any practical connection of this type and requires the formation of only two hinges, one in each cleat as shown.

2 If the supporting members at the end of either cleat cannot resist the applied moment, or are elastically flexible, then the elastic rotational stiffness of the connection will be reduced, as will the moment capacity because of the larger P-Delta effect. This connection is particularly susceptible to loss of strength due to loss of rotational support and for this reason detailed recommendations for the support conditions for both cleats are given.

First principles design procedure to NZS 3404 for the eccentrically loaded cleat in compression

Refer to the picture above in conjunction with the following:

1. Calculate the first order moment associated with the eccentricity, e, between the cleat centrelines. To this should be added an out of tolerance allowance as the capacity of this type of connection is strongly influenced by any out of tolerance between the ends which is in the same direction as the eccentricity, e. NZS 3404 allows the ends to be up to 3mm offset (see Clause 15.3.5(d)) so the eccentricity used should accommodate this. The first order moment is therefore

   $M^*_m = N^*_c(e+3\text{mm})$. equation 1

2. Determine the second-order sway multiplier on this moment using NZS 3404 Clause 4.4.3.3. This is δ_s and is determined from Clause 4.4.3.3.2 (a) (iii) with N_{oms} calculated from Clause 4.8.2. In this calculation, use an effective length of $1.2L_c$ when the cleats are effectively fixed at both ends, where L_c is the clear length of the connection. The each end fixed case is Case No 4 from NZS 3404 Fig 4.8.3.2. The variable I_{cl}, which is required for N_{oms}, is the minor axis moment of inertia of the cleats; use an average of the I for each cleat if the cleats are different sizes (either or both the width and the thickness may be different).

   NOTES FOR STEP 2: When calculating I_{cl}, the design width and thickness of the cleat must first be determined. The design thickness is the cleat thickness, t. The effective width is as given below. This effective width times the thickness forms the gross cross section for the above checks.

   When calculating the effective section for compression, the potential loss of area for bolt holes does not need to be considered – ie the gross area, being $b_{e,N}$ x t, is used. This is for two reasons; first, because the critical location for combined actions involving compression and bending is always in the clear length of cleat outside of the area where bolt holes are present and, secondly, because the sway failure mode of the connection limits the peak axial load that can be achieved such that net area effects within the overlapping region never adversely affect the result.

   When calculating the effective area for bending, the effect of bolt holes does not need to be considered – ie the gross area, being $b_{e,M}$ x t, is used, for the same reasons as given above for compression.

   If the cleat into the supporting member is wider than required to support just the brace under consideration, then the effective width of cleat for design must be determined.. See section 10.7.2.3.4 of the HERA Design Guide Volume 1, HERA Report R4-80, for guidance on applying the "Whitmore section" concept to determine this equivalent width of cleat. Experimental tests and finite element modelling have shown that the Whitmore section width is appropriate for axial load determination but is conservative for moment capacity determination in cleats where the available width of the cleat in which the plastic hinge will form is greater than the Whitmore width. From this body of work a width of at least 1.25 times the Whitmore width can be mobilised for the moment resistance. This leads to the following design recommendations for the effective width of the cleat to be used:

   For axial load capacity determination, $b_{e,N}$ = min (Whitmore width; actual width of cleat)
   For moment capacity determination, $b_{e,M}$ = min (1.25*Whitmore width; actual width of cleat)

3. At least one support must be effectively fixed. If the support at the other end is not effectively fixed, then use an appropriate effective length factor determined from NZS 3404 Clauses 4.8.3.3. and 4.8.3.4 in conjunction with Figure 4.8.3.3. (b). This effective length factor will be greater than

the value of 1.2 used in step 2 above and guidance on how to determine this is given in the NOTES below.

GENERAL NOTE FOR STEP 3: Effective fixity of the cleat to the supporting member is achieved when the supporting member can resist the design moment and the ratio of cleat to support stiffness complies with NZS 3404 Clause 4.8.3.4.1(b). The effective length factor of 1.2 for the sway case is based on based on the rotational stiffness of both supports complying with this limit when applying NZS 3404 Figure 4.8.3.3(b). The modifying factors of NZS 3404 Table 4.8.3.4 are incorporated into these requirements; the connection is a sway member when applying that table.

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The criteria below are based on supported or supporting member cleat supports which will deliver this requirement.

NOTE FOR STEP 3 WITH REGARD TO THE CLEAT CONNECTED TO THE SUPPORTED MEMBER:
Effective fixity of the cleat to the supporting member is achieved when equation 2 is met:

$$\frac{\left[\frac{I_{cl}}{L_c}\right]_{cleat}}{\left[\frac{0.5I}{L}\right]_{\sup portingmember}} \leq 0.6 \qquad \text{equation } 2$$

where:
I_{cl} and L_c are from step 2
I for the supporting member is calculated about the same axis as I_{cl}
L is the length of the supporting member

COMMENTARY TO EQUATION 2: this is based directly on meeting NZS 3404 Clause 4.8.3.4.1(b). The modifying factor on the supporting member stiffness, β_e, is taken as 0.5, which is the minimum value from NZS 3404 Table 4.8.3.4 and is realistic for a supported member with this type of connection at each end.

NOTE FOR STEP 3 WITH REGARD TO THE CLEAT CONNECTED TO THE SUPPORTING MEMBER:
Effective fixity of the cleat to the supported member can be assumed to be provided when any one of the following four criteria are met:

i. All member(s) framing into the supporting cleat are in compression and the cleat is framing into a structural hollow section member that has full twist restraint at each end. Either the supporting cleat is welded into the wall of the structural hollow section member, with the wall thickness at least 0.75x the cleat thickness, or else the supporting cleat is passed through the structural hollow section member and is welded into both faces.
ii. All member(s) framing into the supporting cleat are in compression and the cleat is fixed into the flange of an I section that has full twist restraint to NZS 3404 at the connection and which is directly applied to the flange into which the cleat is connected. The I section must be a hot rolled member or if welded have double sided welds between the flange and the web over the connection region.
iii. The supporting cleat supports one or more tension members in addition to the compression member under design. The tension member(s) are in close proximity to the compression member (as measured at the face of the cleat, the clear distance along the face of the cleat between the compression member and any adjacent tension member must be no greater than $22xt_{cl}$, and should be closer if practicable) and the tension load is similar or greater magnitude to the compression load.

iv. The supporting cleat frames into any supporting member where equation 3 is satisfied:

$$\frac{\left[\frac{I_{cl}}{L_c}\right]_{cleat}}{\sum\left[\frac{\beta_e I}{L}\right]_{\sup portingmember}} \leq 0.6 \qquad \text{equation } 3$$

where:

β_e is given by NZS 3404 Table 4.8.3.4

when calculating $(\beta_e I/L)$ for any supporting member, the effective I/L must be used where the stiffness is generated through a number of components connected in series. See HERA Design Guides Volume 1 section 4.6.3.2 for guidance on implementation of this requirement; in step 2 of that example an effective stiffness is being calculated but the principle is the same

If the supporting member cleat does not meet one of these four criteria, the rotational stiffness must be determined and the effective length for second order effects determined from NZS 3404 Clause 4.8.3.3.

COMMENTARY TO EQUATION 3: each of these criteria is based on meeting the ratio of equation 3. In the first case, where the cleat is welded to the SHS wall, the ratio of I supporting member/I cleat will be at least 42% and given that there are two supporting members to each cleat and these members are shorter and more confined than the cleat (β_e = 1.0) the limit on thickness will always ensure that equation 3 is met. In the second case, the cleat is connected into a supporting member flange which has direct twist restraint, which is more severe than the rotational stiffness requirement for this connection. In the third case, performance in practice shows that the out-of-plane stiffness of the cleat under combined reactions onto it combining compression and tension will exceed the twist restraint required. In this case the value of 22 is taken from case no 2 of NZS 3404 Table 5.2 for a LW or HW member. That value is associated with being able to reach the yield capacity in a plate with the one supported edge in tension and the unsupported edge in compression. In this case the supported edge is at the outgoing tension member and the unsupported edge is at the outgoing compression member, so the case is applicable to the face of the gusset plate supporting these two members. In the fourth case, the equation to be met is directly given.

4. If δ_s determined from step 2, and step 3 when appropriate, is greater than 1.3, the connection is overly sway sensitive and must be stiffened.

5. Multiply the first order moment from step 1 by this multiplier δ_s to give the design moment M*.

6. Determine, for each cleat, the nominal section moment capacity reduced to take account of axial force, M_{rey}. The relevant equation for this check on a flat plate in weak axis bending is given in NZS 3404 Clause 8.4.2.2.1, however use the elastic section modulus for the cleat rather than the plastic section modulus as follows:

$$M_{rey} = M_{sey}\left(1-\frac{N^*}{\phi N_c}\right) \qquad \text{equation 4}$$

where:

M_{sey} is calculated using the elastic section modulus, $Z = b_e t^2/6$, not the effective section modulus from NZS 3404. The reason for this is in the commentary to step 6 below

N_c is the member compression capacity, calculated from NZS 3404 Clause 6.3 using α_b = 0.5 and using the clear length of the cleat as the effective length, L_e.

The clear length is (L_1-a) for cleat t_1 in the above figure and (L_2-a) for cleat t_2. For inclined connections into gusset plates the design effective length is determined from Design Guides Vol 1 Fig 10.38 with an effective length factor of 1.0.

COMMENTARY TO STEP 6: the elastic section modulus is used instead of the plastic section modulus for this connection. This is considered necessary because of the sensitivity of this connection to initial fit-up and to the presence of residual stresses, plus the rapid drop in compression capacity if the connection is loaded beyond its peak load carrying capacity. What happens is that this connection is very sensitive to the magnitude of the moment at the plastic hinge locations. Once the combination of moment and axial load cause the outer fibres of the plate to yield, the cleat's rotational stiffness against sidesway decreases and the rate of moment increase due to P-Delta effects increases over that calculated. This causes the peak load to be reached soon after initial yielding of the face of the cleats occurs and before the plastic section is developed. It is conservative to base the moment on first yielding however there is insufficient understanding of the connection behaviour to go beyond this at present. However, this is mitigated slightly by calculating the resistance of the connection as the combined resistance of both cleats from eqn 5.

7. The combined moment capacity of both cleats (reduced for axial force) must then be greater than the applied design moment M^* i.e.

$$M^* \leq \phi\left(M_{ry1} + M_{ry2}\right) \qquad \text{equation 5}$$

8. The axial load and moment developed in each cleat must be resisted by the member supporting the cleat and by all the connections thereto, and this needs to be checked for each particular case. The supporting member needs to be able to resist N^* and $1.3M_{rey}$ N^* is the design axial compression load. $1.3M_{rey}$ is the nominal reduced moment capacity of the cleat with an overstrength factor which is intended to ensure that the supporting member does not start to yield prior to the formation of the collapse condition in the cleats. This is to prevent loss of rotational support to the cleats which would increase the P-Delta effects on the connection and lower the connection capacity. The 1.3 factor is applied because M_{rey} is based on the elastic modulus.

9. The connections between the cleats and the supporting members should be either complete penetration butt welds or balanced double sided fillet welds. In the latter case, these must be designed to transfer the design actions from step 7 into the supporting member. The complete penetration butt weld will not fail when the cleat develops a plastic hinge adjacent to the weld and the above design requirements for the fillet weld will ensure that the same performance is achieved.

10. Because this connection is sensitive to the erection tolerances, an allowance for the NZS 3404 maximum permitted out of tolerance of 3mm has been included in the procedure (see step 1). It is important that the misalignment of the installed connection is not greater than this. In practice this extent of misalignment will be just visible. It means that during the Construction Review, any of these connections which show visible misalignment should have the extent of this measured and be straightened to be within the NZS 3404 tolerances if required.

11. This connection reaches its peak load at small axial deformation and then undergoes an appreciable drop in load carrying capacity if the axial deformation demand on the connection increases beyond this point. For this reason, its use must be restricted in seismic-resisting systems to applications where this behaviour will not compromise the performance of the structure. These restrictions on seismic-resisting system use are that:
    a. The connection cannot be used to connect category 1, 2 or 3 primary members in a seismic-resisting system

b. The connection can be used to connect category 4 members in a seismic-resisting system, provided that equation 6 is satisfied for the connections to those members:

$$0.7\phi N_c \geq N^*_{G+Qu} \qquad \text{equation 6}$$

where:

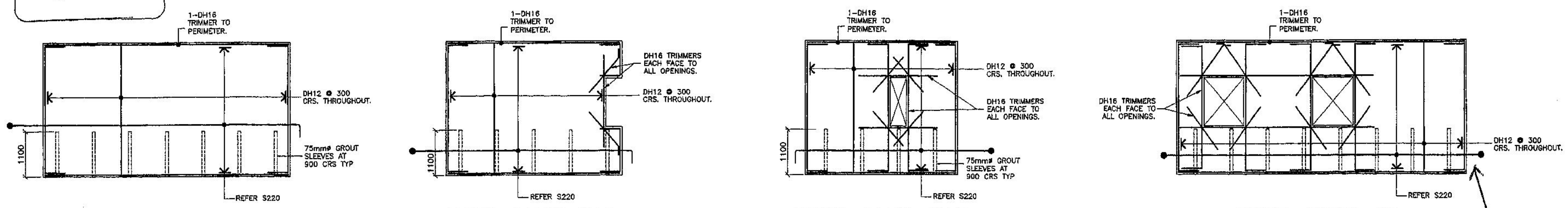
ϕN_c is the design compression capacity of the connection from satisfying equations 4 and 5 above

N^*_{G+Qu} is the design compression load on the connection from the dead and combination live loading associated with earthquake actions

*COMMENTARY TO STEP 11: The restriction of (a) is to ensure that the connection is not used to connect members that are designed to respond inelastically in a severe earthquake. However, even when the members are not designed for inelastic response, as is the case with category 4 members, they may be subject to overloading during earthquake sufficient to push the connection past the peak load and into the permanently deformed state. If this happens, the load carrying capacity is reduced. The aim of equation 6 is to ensure that, if the connection is displaced 5 times that associated with reaching the peak load, the reduced load carrying capacity will still be greater than the compression loading on the connection due to vertical loading associated with earthquake actions. This will ensure that the connection can reach this state and still be able to carry the imposed gravity actions following the earthquake. Tests and analyses have shown that when the connection is deformed to 5x the deformation associated with the peak compression load, the compression load that can be carried is around 0.55 to 0.6 times the peak load. This design procedure will give a reserve factor of at least 1.3 between peak load and design capacity, hence a ratio of at least 0.55*1.3 = 0.72 between deformed compression capacity and design capacity. This is rounded to 0.7 in equation 6.*

Apartment Building - Auckland 2003

A1



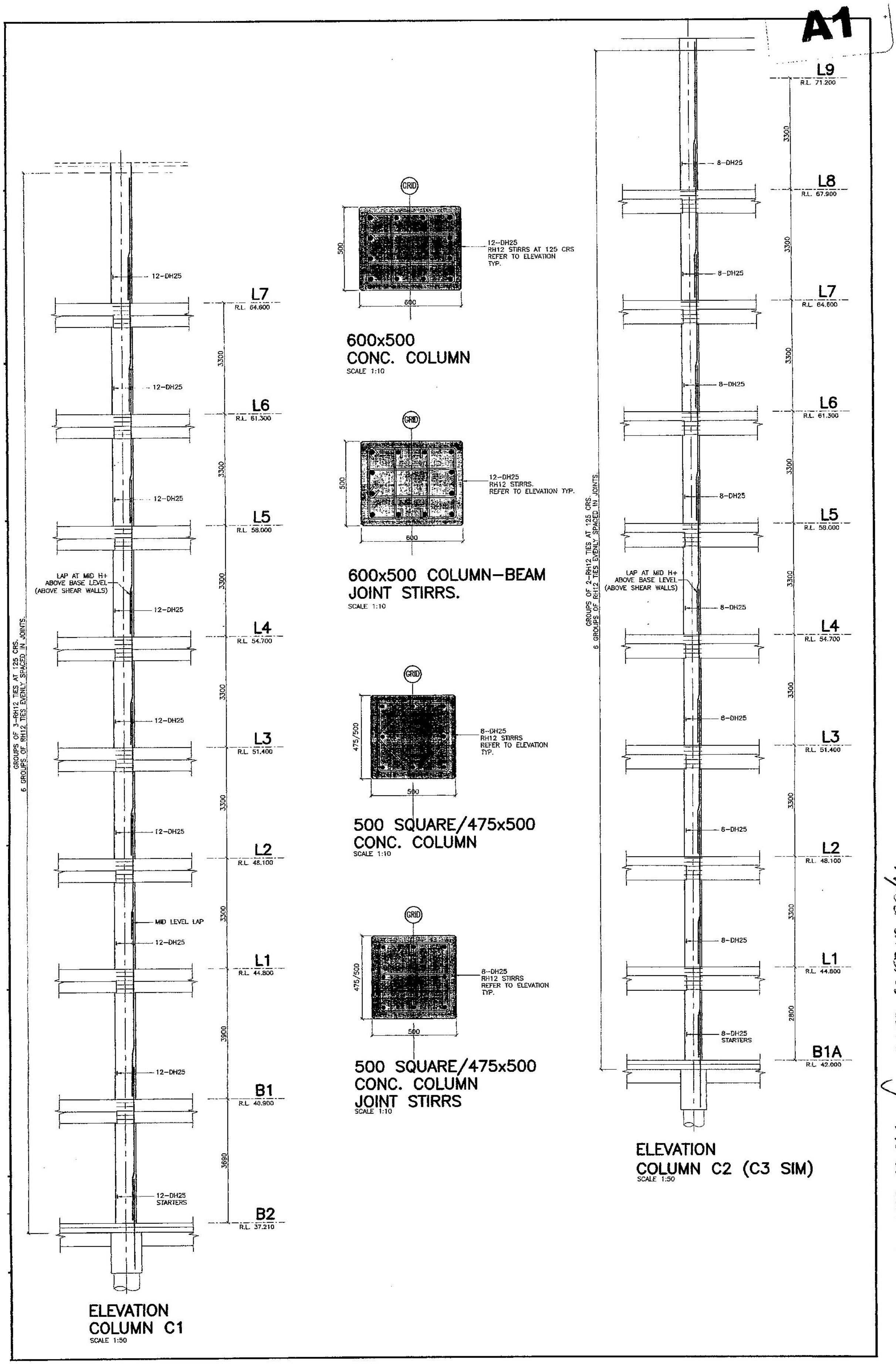
REINF. PANEL A1
(PANELS A2-A6, B1-B11,B15,B16,C1 SIMILAR)
SCALE: 1:50

REINF. PANEL B12
SCALE: 1:50

REINF. PANEL B13
(PANELS B14 SIMILAR)
SCALE: 1:50

REINF. PANEL C2
(PANELS C3-C5 SIMILAR)
SCALE: 1:50

Starters must project into columns (typ.)



Apartment Building - Auckland 2003

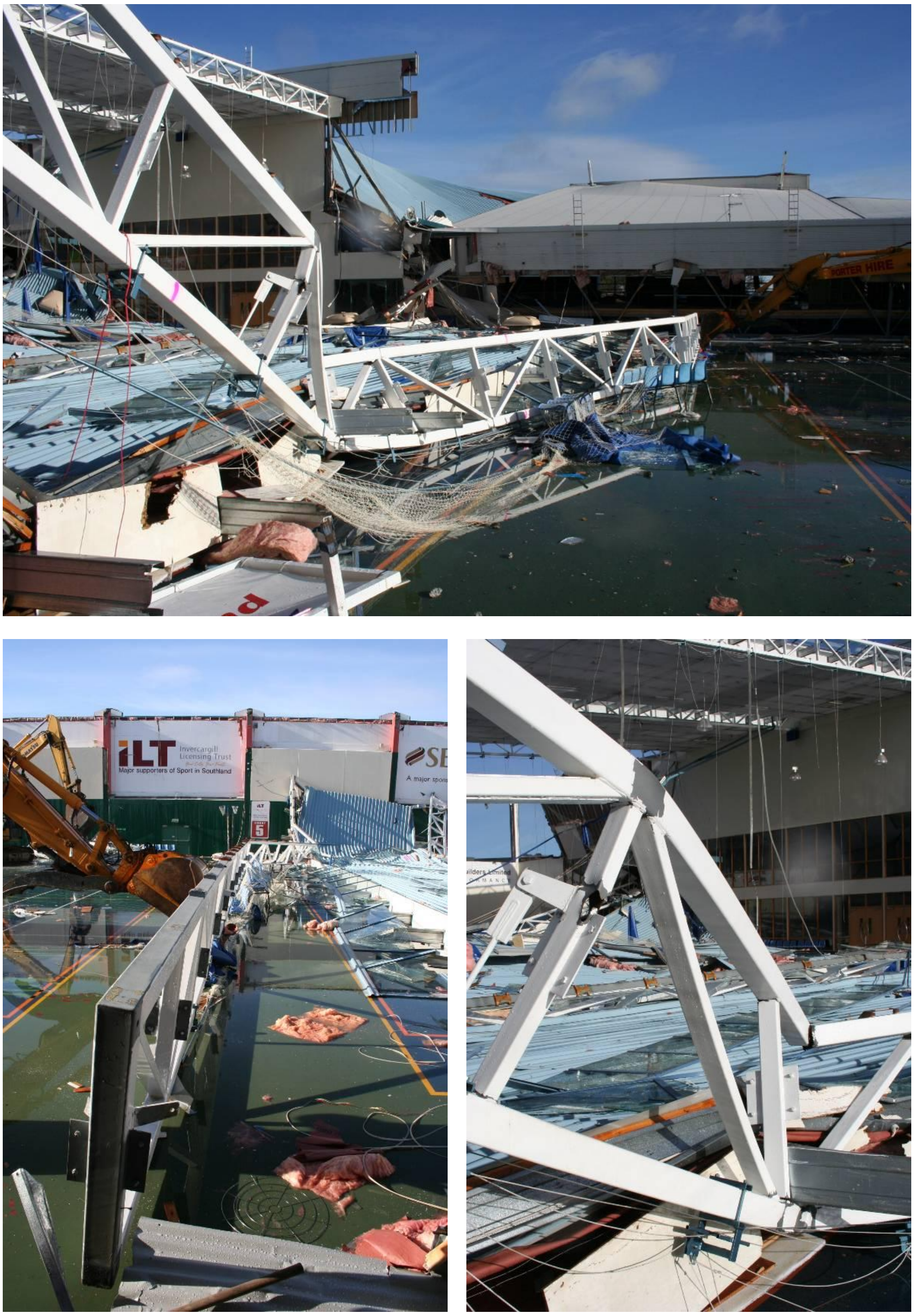


Figure 28 - Truss T4 prior to being moved on 25 September 2010 (clockwise from top); (a) View from south wall; (b) Top chord splice without packer plate failure and reduced wall thickness bottom chord failure zones at quarter point. Diagonal rod bracing cleat has pulled out of vertical strut on eastern side; (c) View from northern end of Truss 4 showing westward offset from mid-span

Figure 24 - Truss T2 (Clockwise from top left) (a) Sheared head of column C12; (b) Mid span failure of top chord; (c) End butting of side plates without the specified complete penetration butt weld; (d) Typical top chord fit up at packer plates with 3-sided bearing at quarter point splice; (e) Top chord packer plate at mid span showing no evidence of the specified complete penetration butt weld; (f) Truss T2 in salvage yard with TCM still attached to column C12 end plate.

Christchurch Construction Pre-Earthquakes

Where is the seismic separation?

Owner demands answers on why her building failed
The Press – Ben Heather 16/06/2011

DEJECTED: Michelle Crouch wants to know why her new building performed so poorly in the earthquake.

<u>The 'First Building of the Rebuild'</u>

Devastated by the 13 June 2011 aftershock

Precast Building Under Construction

Devastated on 23 December 2011

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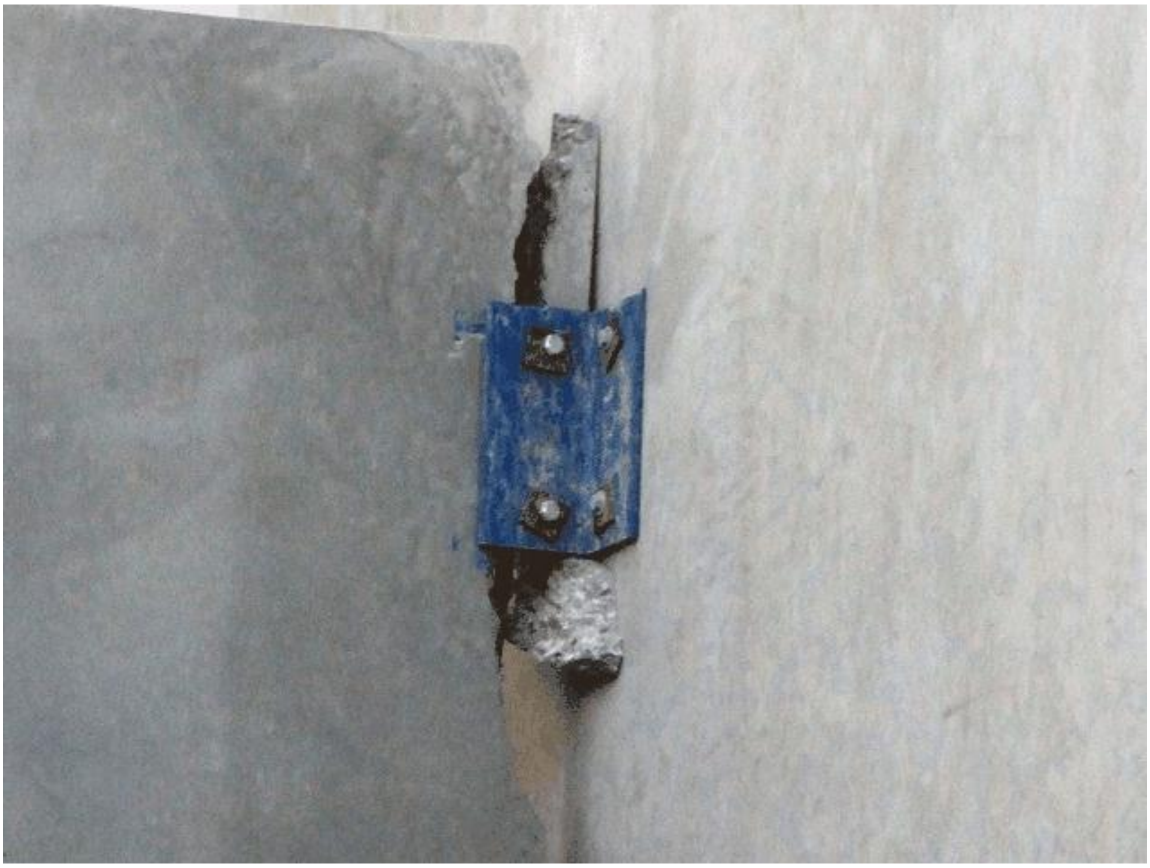


Illegal Construction – Auckland – 2004

Similar illegal construction seen in Christchurch – mid 2012

Jumping collapses venue floor

SAM SACHDEVA
Last updated 12:32 14/07/2012
Share

TOMMY ILL

Hip hop concert opening act Tommy Ill said the floor collapsed under the weight of the crowd.

Structural engineers will investigate why the floor of a new $2.5 million events centre collapsed during a Christchurch hip-hop concert.

A Canterbury University concert was abandoned last night after part of the floor at its new multimillion-dollar venue collapsed during a performance.

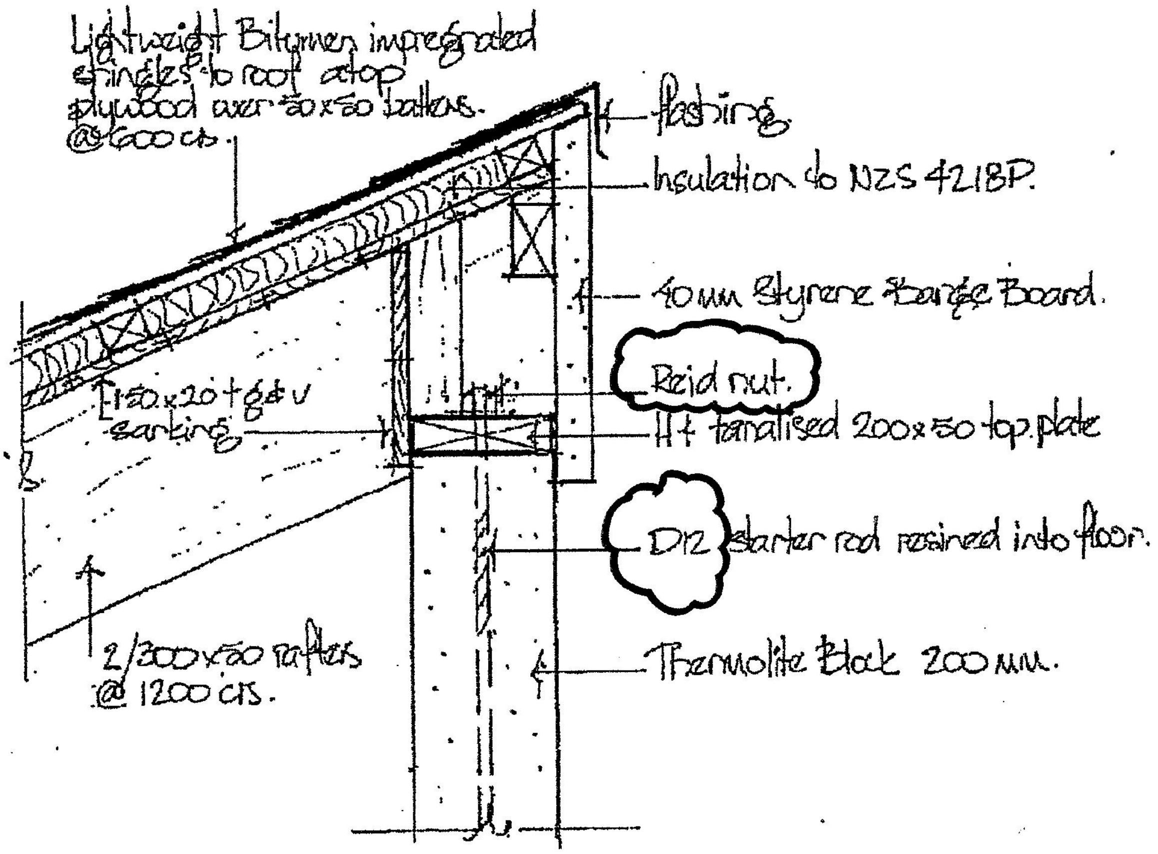
The concert, The Perfect Storm, was held at the university's new $2.5 million temporary events centre.

Designed by Christchurch architecture firm Warren and Mahoney, the centre opened in April after the university's original student bar was damaged in the February 2011 earthquake.

University spokesman John MacDonald said the university would consult structural engineers to determine the cause of the collapse.

"We are very thankful that no one was injured. The safety of our students is paramount, and we will need some time to determine what has happened and what we are going to do about it," MacDonald said.


External Thread
(Male)
Internal Thread
(Female)



ROOF BARGE DETAIL.

scale 1:20