SESOC

Seminar Series

2022

10 Tips for the Better Design of Low Rise Structures

aka low rise learning

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Low Rise Buildings

- These are common buildings
- 85-90% of what we are designing all day every day
- Repeated poor designs being seen

- How do we turn quite a few negatives into a positive…

Ten tips for designing better low rise structures

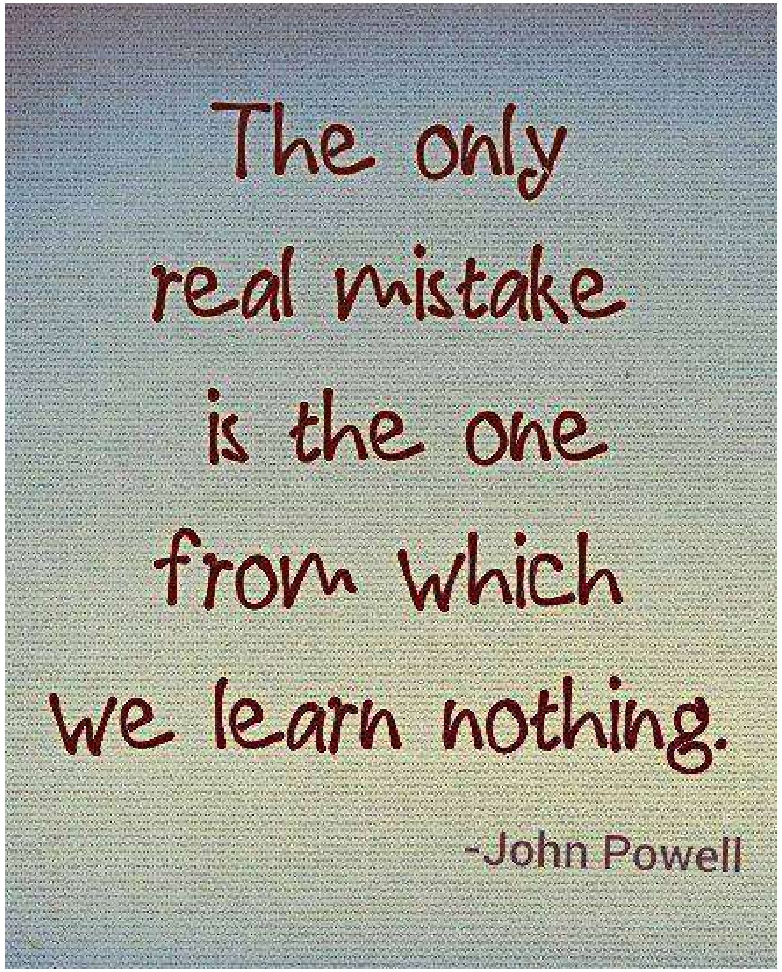
- We do not have a PhD
- We are not lecturers

- We are practicing structural engineers

- Be nice

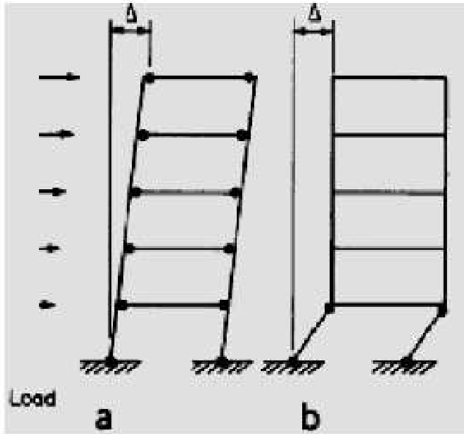
Please remember….

- Anonymized as much as possible – here to learn in a positive manner, we are not here to finger point
- Please respect that for all these buildings, none of this was intentional
- We are here today to try and spread the learnings, so these mistakes don't get repeated



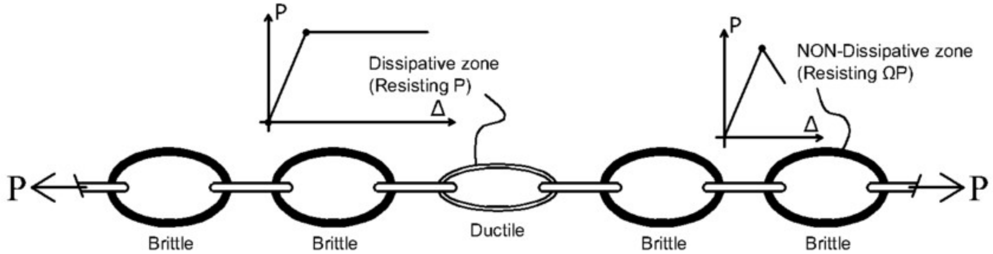
NZ Seismic Design Philosophy

- In NZ, we design structures with a ductile mechanism, and suppress undesirable failure modes - “Capacity Design”



Comparison of Energy Dissipating Mechanism with and without Strong Column - Weak Beam Concept

- For low rise, our design Standards typically do not require full capacity design



Designed for a level of EQ actions ⟵ ⟶ Requires Capacity design

	Brittle	Elastic Max μ=1	Nominally Ductile Max μ=1.25	Limited Ductile Max μ=3	Ductile Max μ=6
Timber	Glued connections, perp to grain connection failure. Use μ=1 or 1.25 if some load sharing			Some ductility, but not sufficient to be relied upon with certainty	Chosen mechanism to allow for large displacements
Concrete	Not allowed. Includes relying on concrete in tension & sections under min reo req'ts		Designed using μ=1.25 or less, NDPR	Designed using μ=3 or less, LDPR	Designed using μ=6 or less, DPR
Steel		Min displacement ductility demand under ULS EQ	Yield flanges	Form hinges	Strain harden hinges

Robustness - 'the ability to withstand or overcome adverse conditions'

- Robustness is achieved by capacity design
- But our Standards allow us to tap out of using capacity design
- Where is robustness then?

What is still needed for Low Rise...

- All Structures shall be configured with a clearly defined load path, or paths, to transfer the earthquake actions

  NZS1170.5 Clause 2.1.2

- All elements shall be capable of performing their required functions while sustaining the deformation of the structure

  NZS1170.5 Clause 2.1.2

- Structural elements and members shall be tied together to enable the structure to act as a whole in resisting seismic actions

  NZS1170.5 Clause 2.1.3

A quirk of the Standards?

1170.5 refers to 'Brittle Structures'

2.2.4 Brittle structures

A brittle structure is defined as a structure with structural components that are not capable of inelastic deformation without undergoing sudden and significant loss of strength. The structural ductility factor, μ, for brittle structures shall be taken as 1.0.

But what do the Material Standards say?

Brittle Structures are not within the scope…

2.6.1.2 *Classification of structures*

Structures subjected to earthquake forces shall be classified for design purposes as brittle structures, nominally ductile structures, structures of limited ductility or ductile structures, as specified below:

(a) Brittle concrete structures shall be those structures that contain primary seismic resisting members, which do not satisfy the requirements for minimum longitudinal and shear reinforcement specified in this Standard, or rely on the tensile strength of concrete for stability. Brittle structures are not considered in this Standard.

Concrete Standard NZS101

(4) *Elastic systems* (Category 4 systems)

These are expected to respond with minimal structural displacement ductility demand under the design level ultimate earthquake loads or effects and to resist collapse under a maximum considered earthquake as directed by the Loadings Standard. Elastic systems are not brittle systems: brittle systems are outside the scope of this Standard.

Steel Structures Standard NZS3404

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Connections too!

18.6.5 *Connections*

Connections between precast elements, and between precast and cast-in-place concrete elements, shall be designed to meet the following requirements:

(a) To control cracking due to restraint of volume change, temperature changes, and differential temperature gradients;

(b) To develop a failure mode by yielding of steel reinforcement or other non brittle mechanism.

Concrete Standard NZS101

Elastic (or nominally ductile) should not be brittle

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Designing for Uncertainty?

For low rise, it seems to be common to 'adopt a ductility', then design for a set load

The problem with this is that we are seeing structures with potentially brittle load paths

Robustness, Robustness, Robustness

- If we keep these 10 tips in mind, we should end up with robust structure

Tip #1

1		

Tip #1

Make sure your design matches your model

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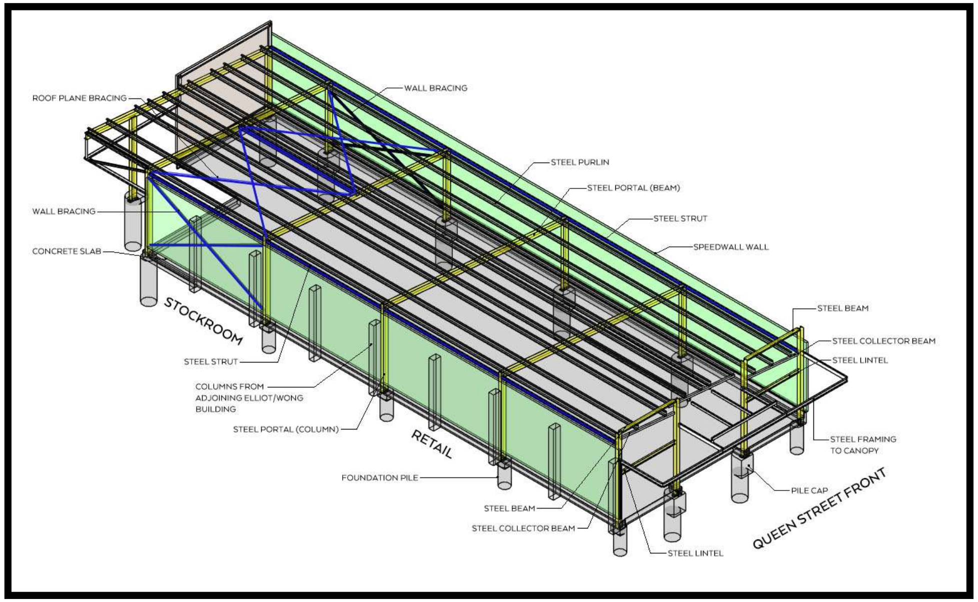
Building A

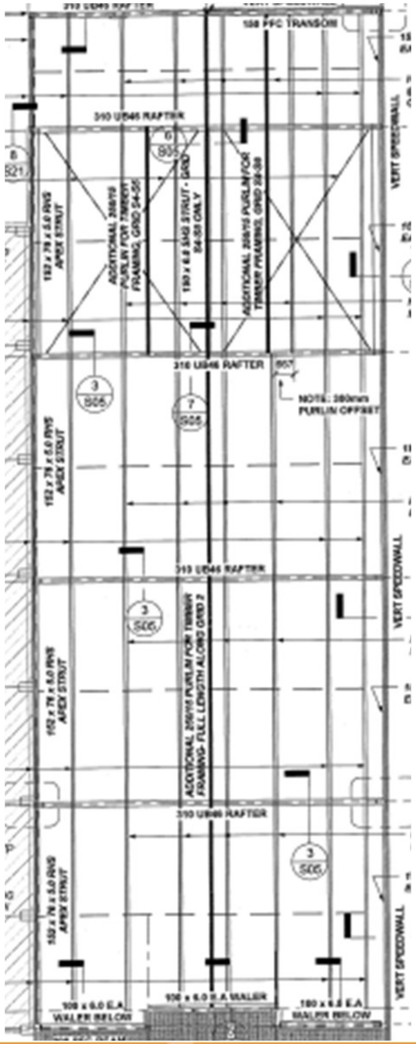


- Light weight low rise structure
- 330m^2 split into two tenancies
- Reinforced concrete foundations
- 310UB46 steel portal frames at 7.5m centres
- One side wall timber framed
- Other side wall Korok panels (aerated concrete – light weight)
- Tensions bracing in the plane of the roof and wall
- Typical steel framed facade


SAUNDERS
shoes synonymous with style
OUTLET STORE
trade aid
LJ Hooker
For Lease
Big save
FURNITURE
love local

Typical commercial building

Transverse Load Path



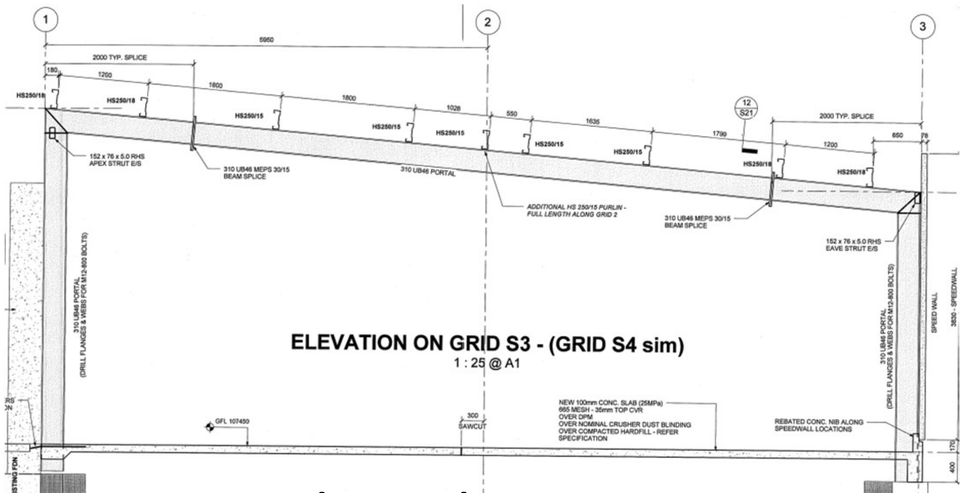
Steel Portal Frame

Steel Portal Frame

Steel Portal Frame

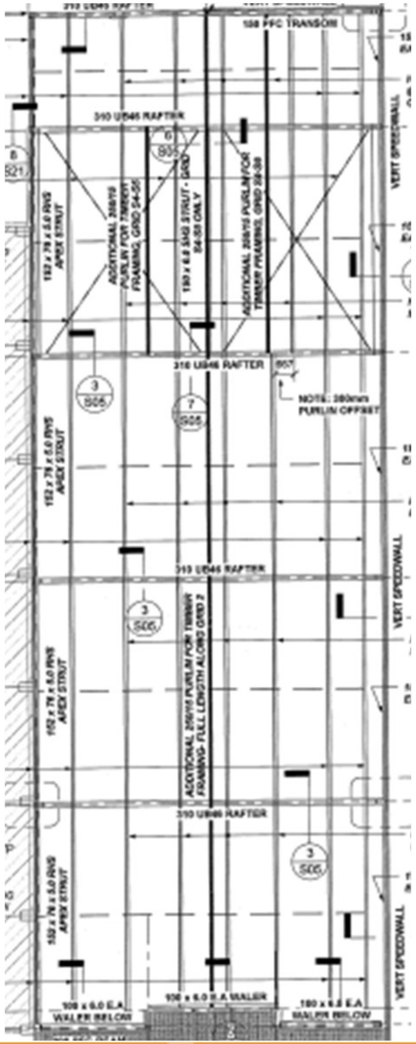
Steel Portal Frame

Façade steel frame



Steel Monoslope Frame

- 4 steel monosloped frames
- 1 façade frame



Steel Portal Frame

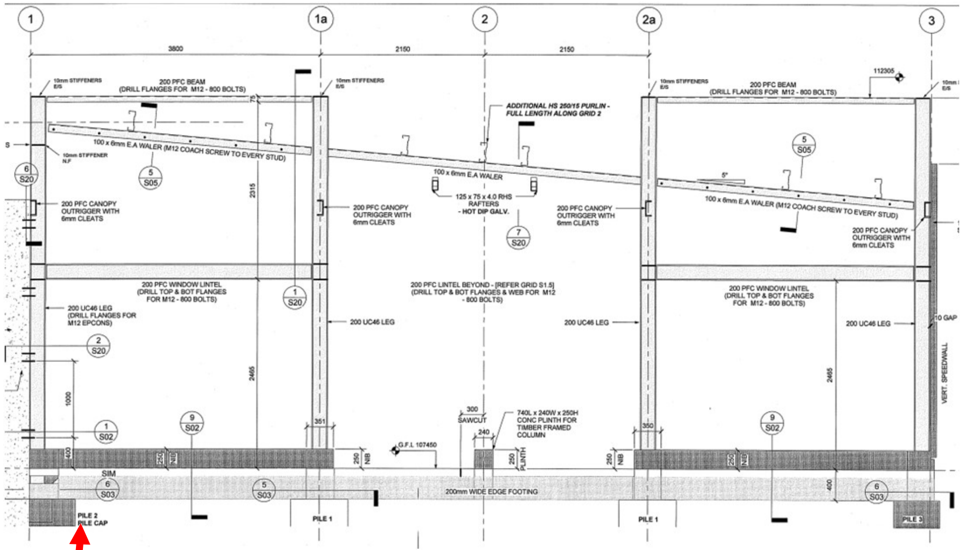
Steel Portal Frame

Steel Portal Frame

Steel Portal Frame

Façade steel frame

Transverse Load Path



Façade Steel Frame

- 4 steel monosloped frames
- 1 façade frame

Lets look at the shop front Facade

- Canopy cantilevers 2.9m out from building
- Parapet upstand for signage
- Steel frame facade

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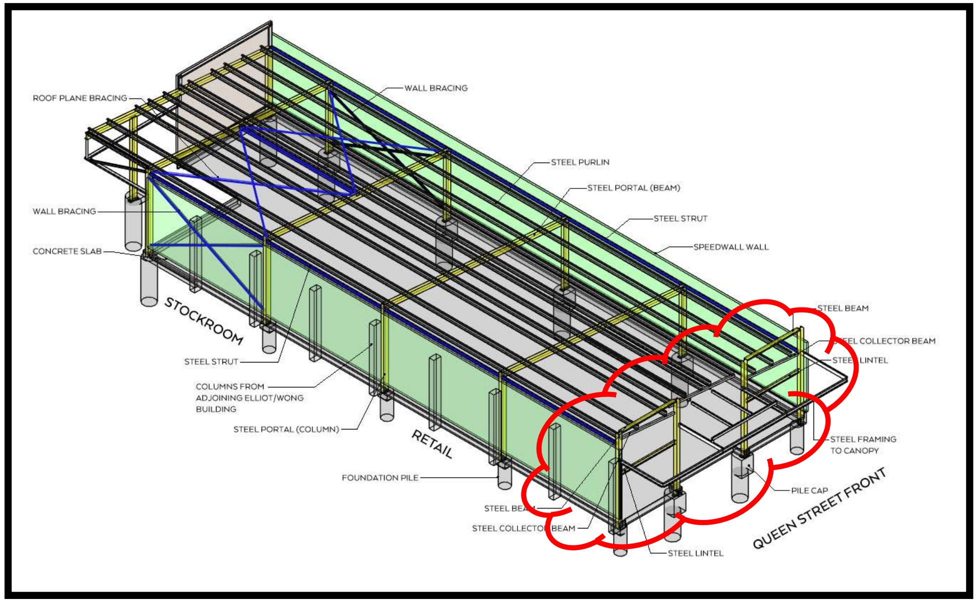
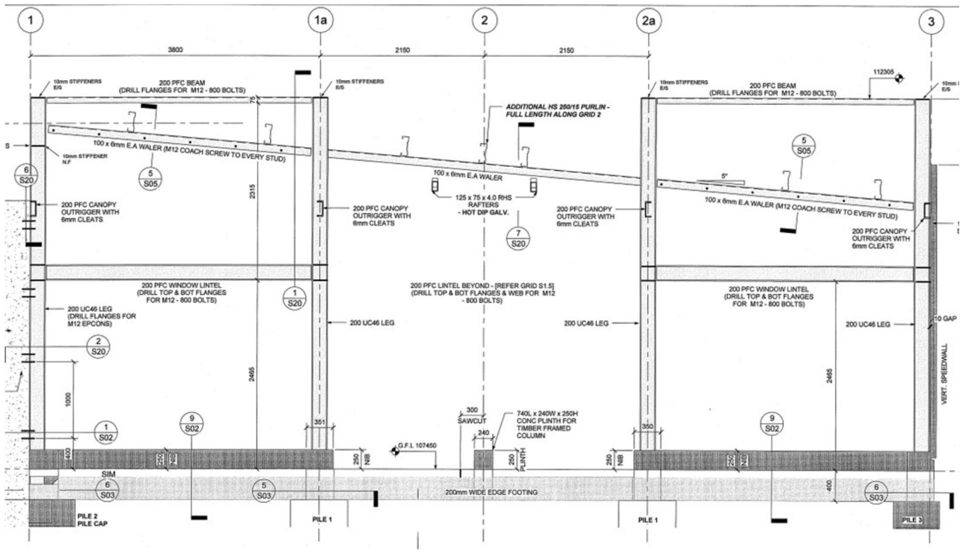
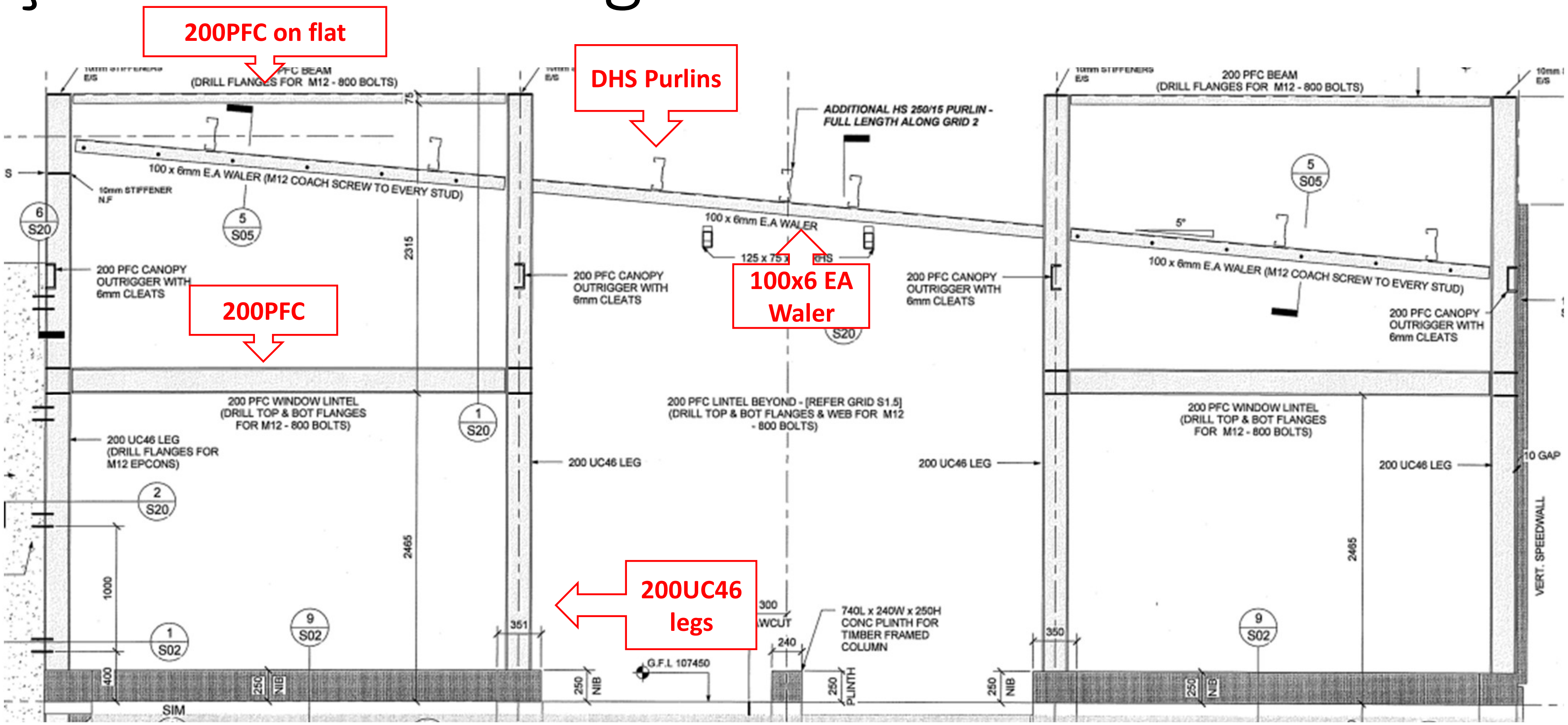
Going to the drawings



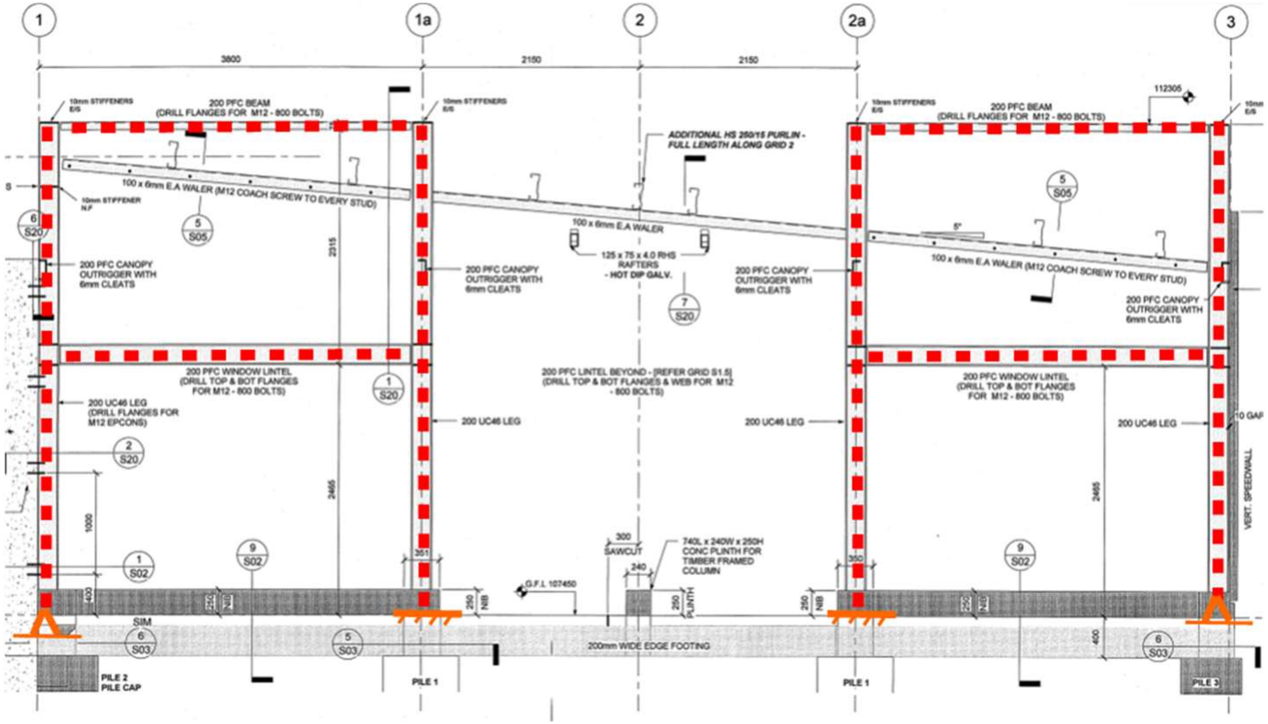


Façade steel frame elevation

Façade steel framing



Façade frame – in-plane actions

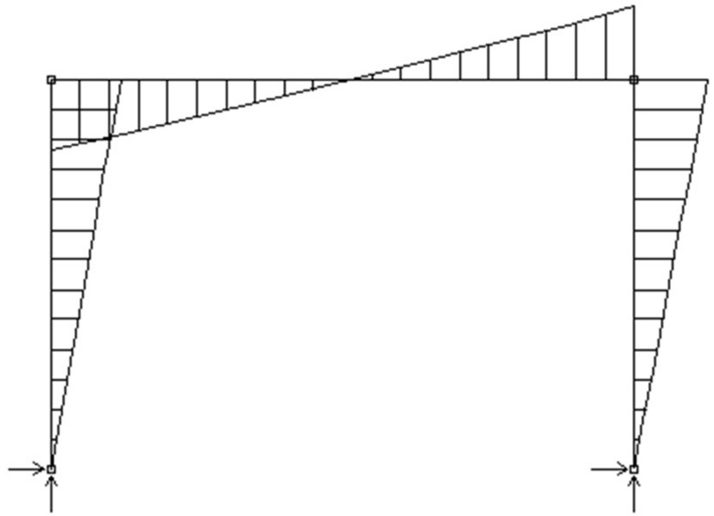


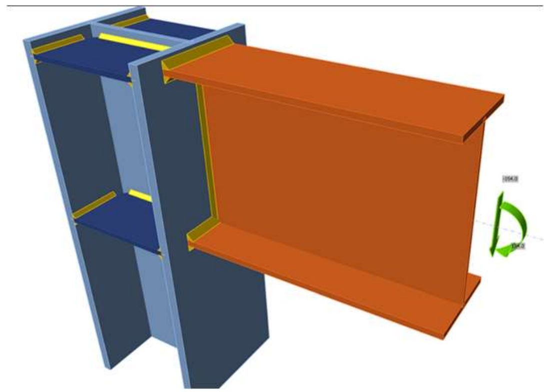
Model of Frames

- Original calculations modelled the façade as two 'frames'.
- Seismic actions resisted by each frame
- Model assumed moment joints at mid height and top rafter to both columns, and a fixed base to one column.

Reminder – what is a fixed (rigid) joint?

- A frame has rigid connections between each of its elements

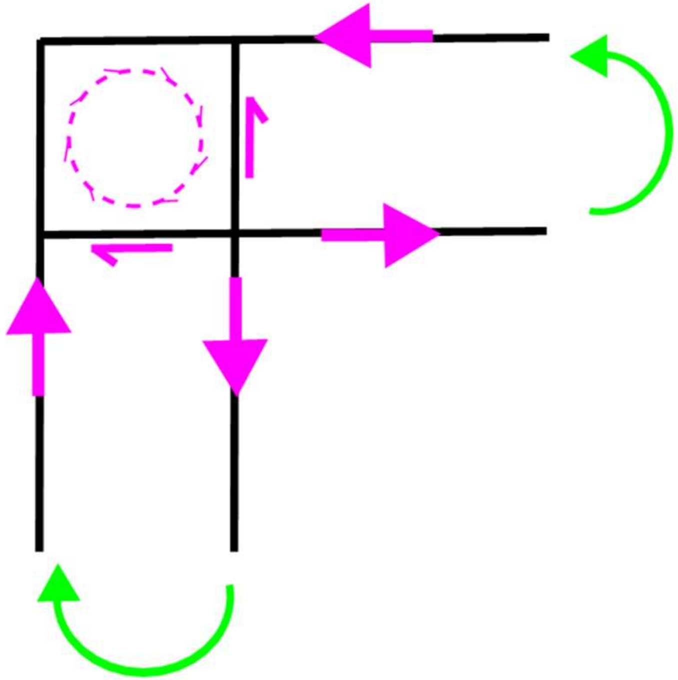




The rigid connection is critical

Reminder – what is a fixed (rigid) joint?

Moment, shear & axial forces need to be transferred from one bit of steel (ie rafter) to the next (ie column)



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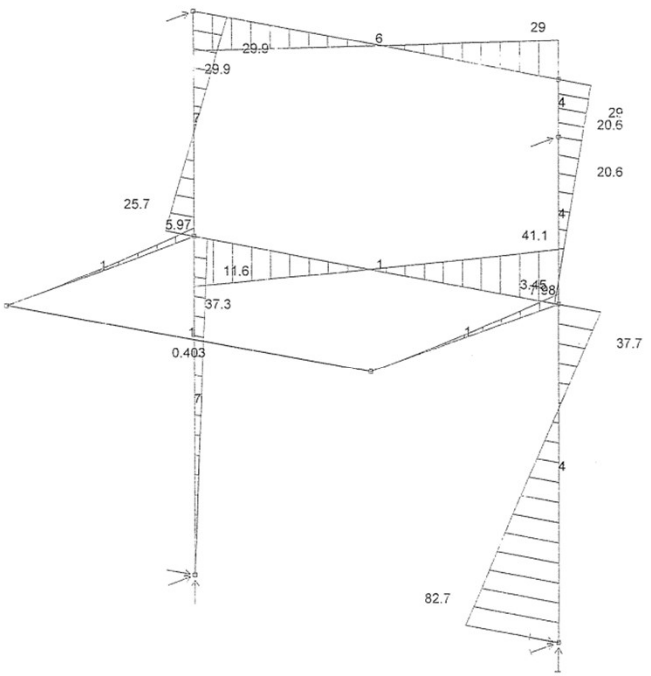
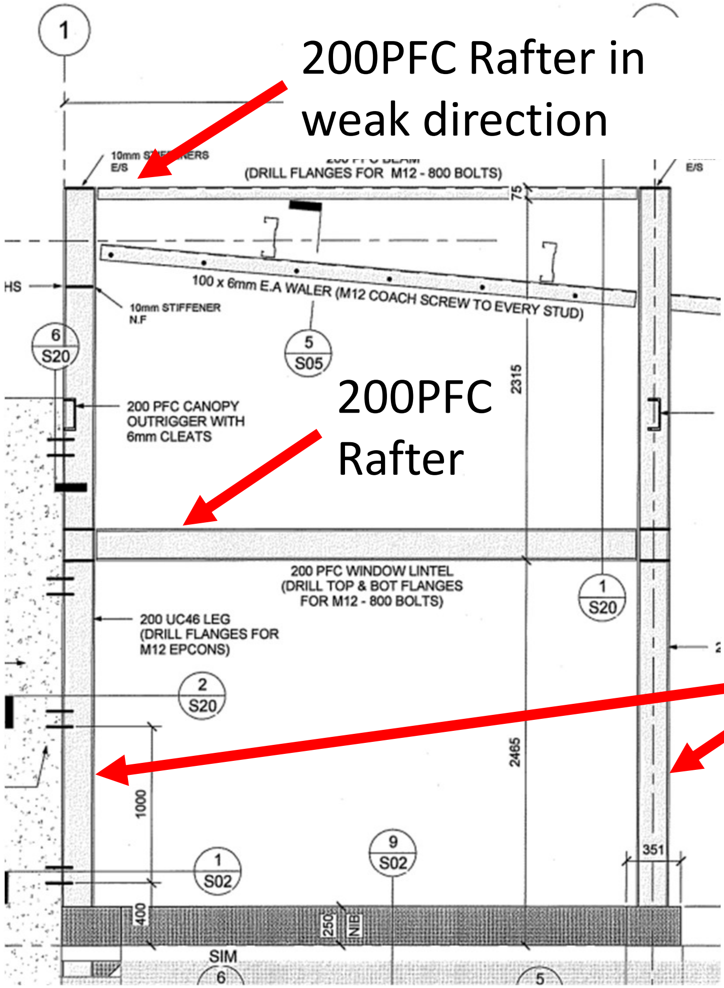
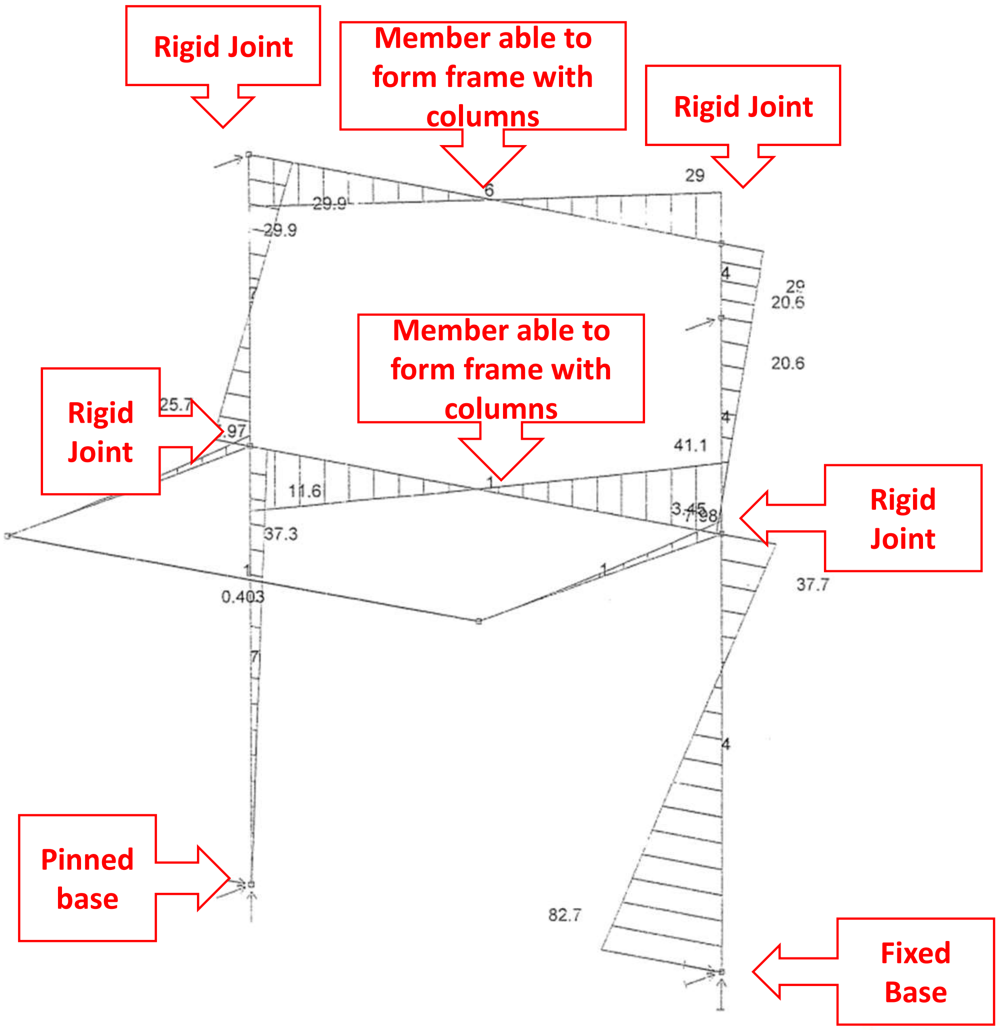
Lets look at how the model translated to design / detailing

Model vs Drawings





Model



Model assumes rigid joints between all steel members

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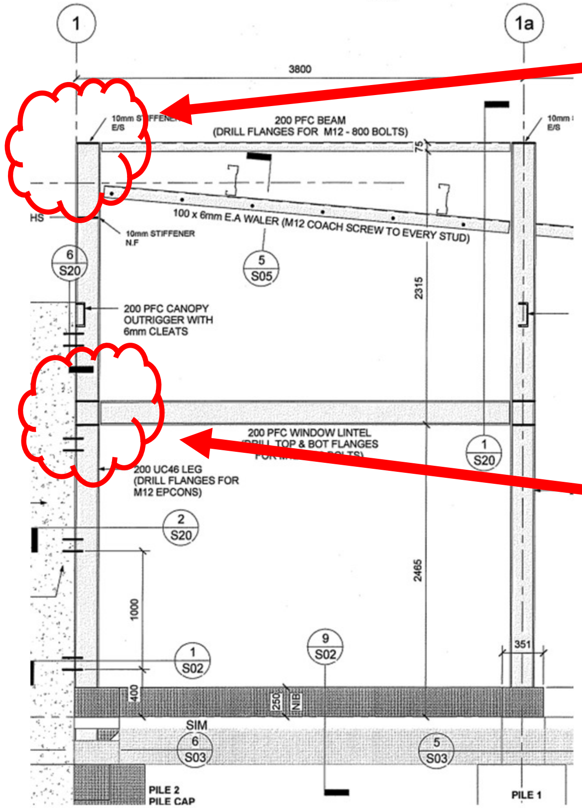
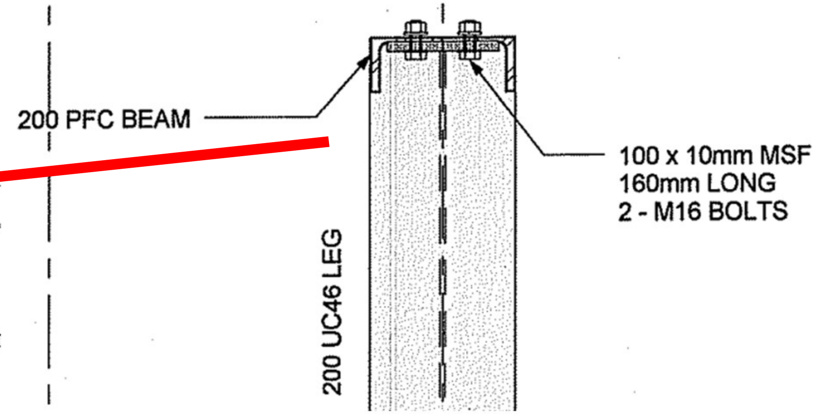
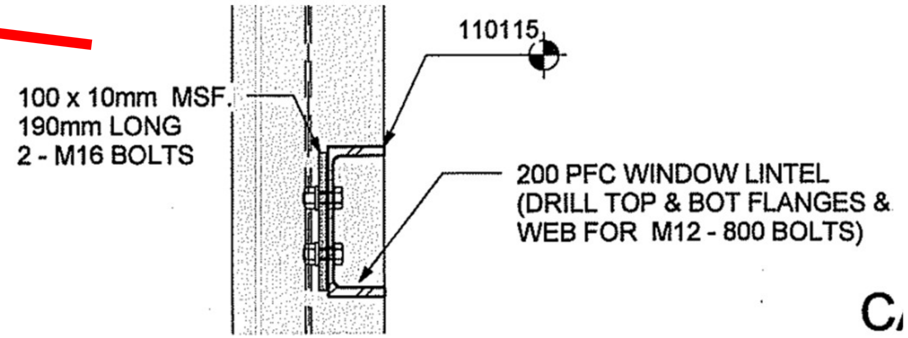
Drawings

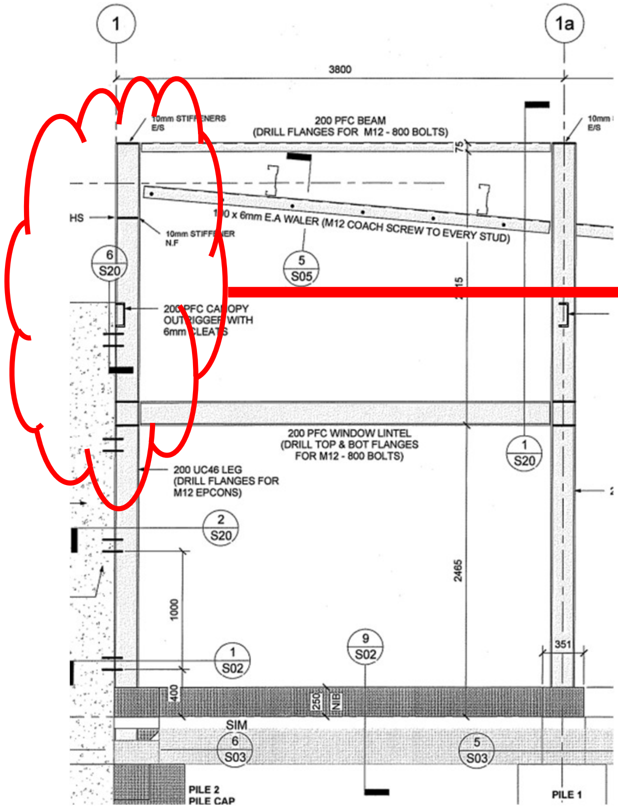




- UC has a bolted connection to a PFC in the weak direction



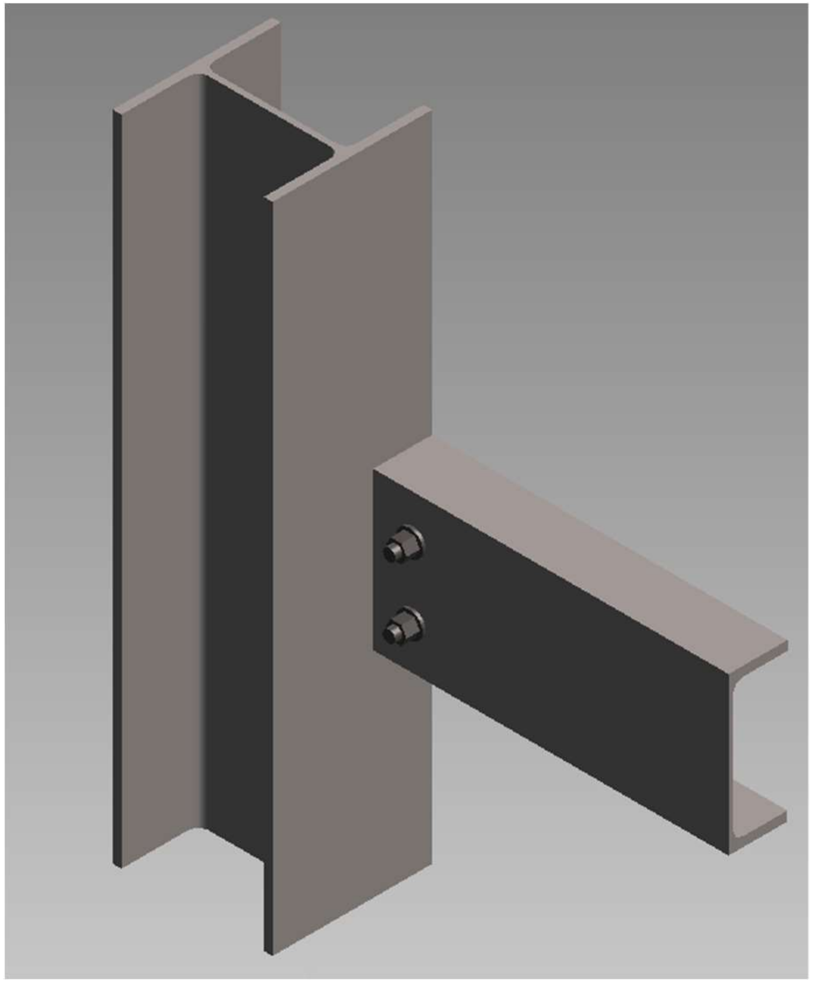
- UC has a bolted connection to a PFC in the strong direction


1
1a
3800
200 PFC BEAM
(DRILL FLANGES FOR M12 - 800 BOLTS)
75
HS
E.A WALER (M12 COACH SCREW TO EVERY STUD)
200 PFC WINDOW LINTEL
(DRILL TOP & BOT FLANGES
FOR M12 - 800 BOLTS)
200 UC46 LEG
(DRILL FLANGES FOR
M12 EPCONS)
1000
2465
351
400
SIM
PILE 2
PILE CAP
PILE 1

Rafter to Column Joint – Mid Height

How does this joint work?

- Column has a single welded cleat with 2 bolts
- Bolts then join to the PFC



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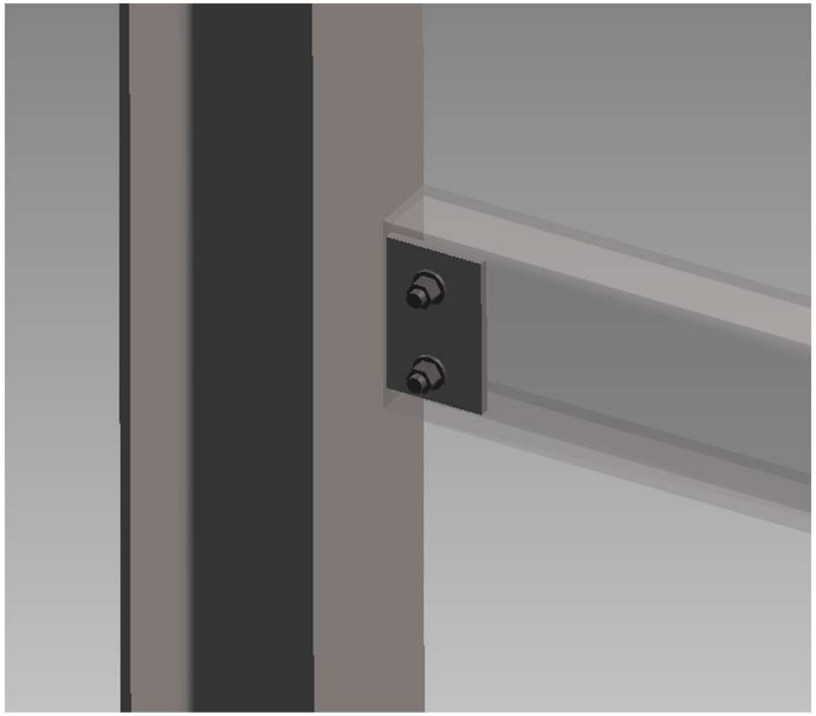


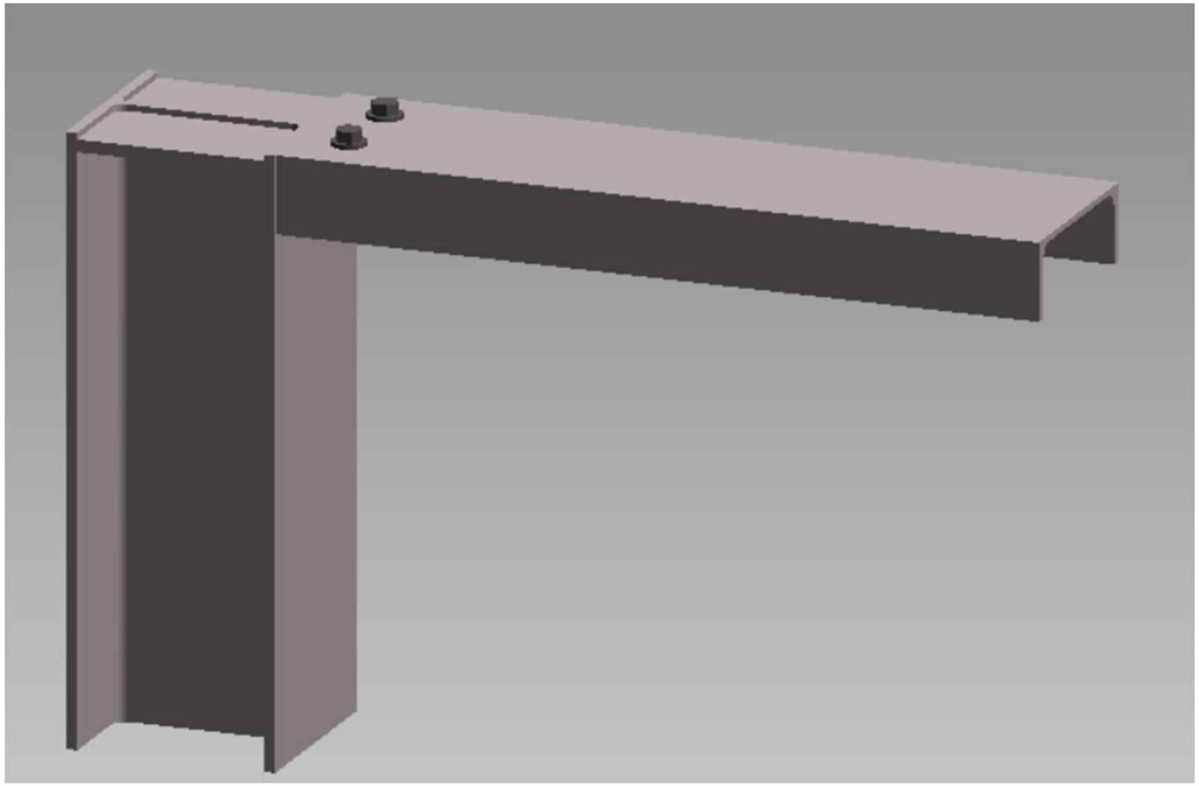
This is not a rigid joint!

- No way to transfer frame forces from UC to PFC – 2 bolt connection with a single cleat
- Eccentric tension/comp forces
- A PFC cannot form a frame with a UB/UC without poor and/or indirect loads paths

Rafter to Column Joint – At top Height

How does this joint work?

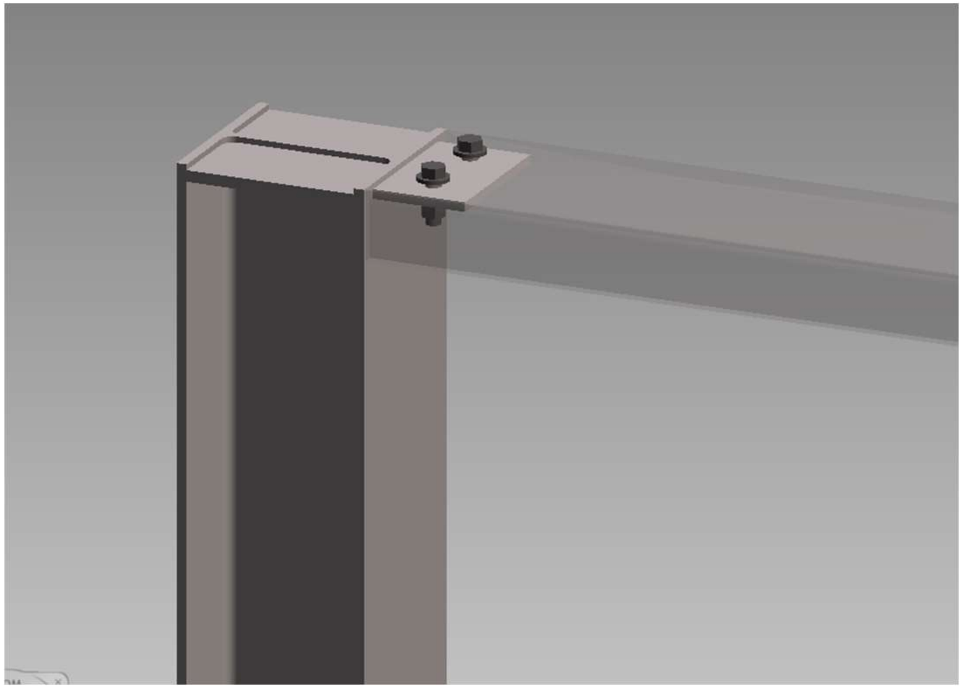
- Column has a single welded cleat with 2 bolts
- Bolts then join to the PFC which is in the weak direction



Rafter to Column Joint – At top Height

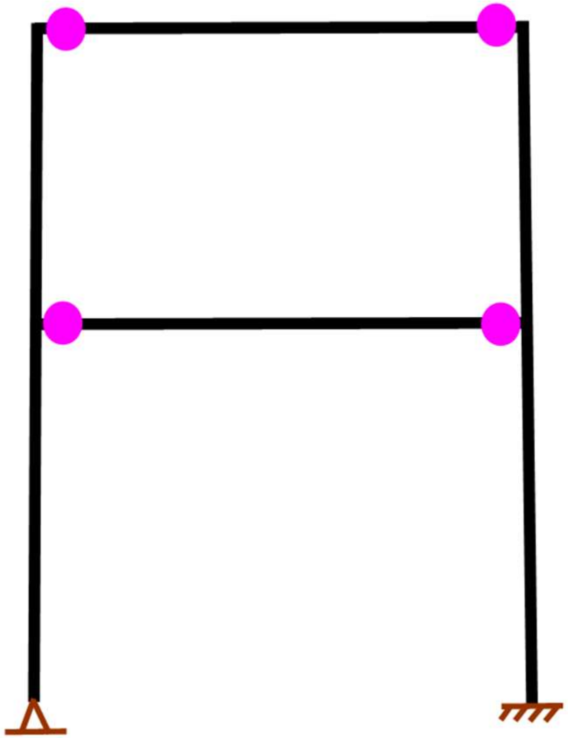
This is not a rigid joint!

- No load transfer mechanism for the frame actions from UC to the PFC – 2 bolt connection with a single cleat

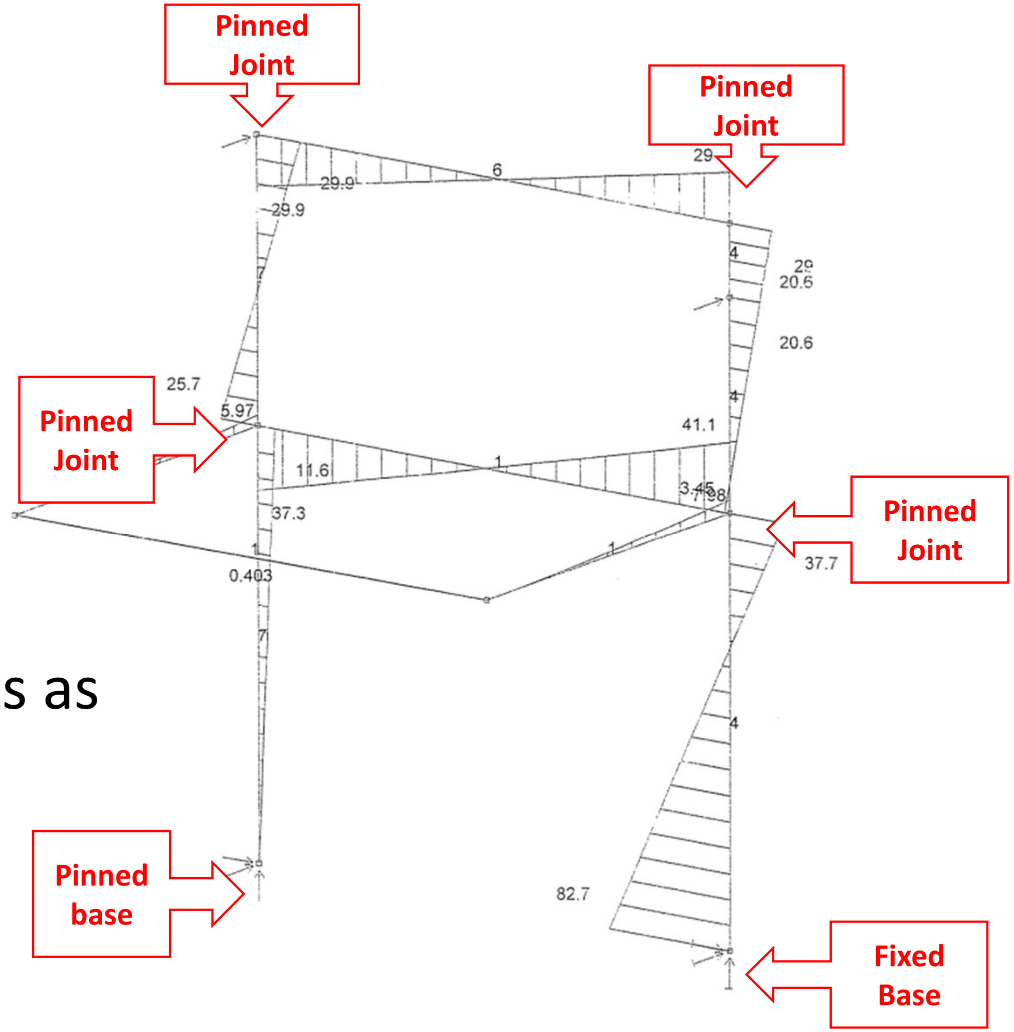


So what was the actual model?

So what was the Actual Model



- Pinned joints as appropriate

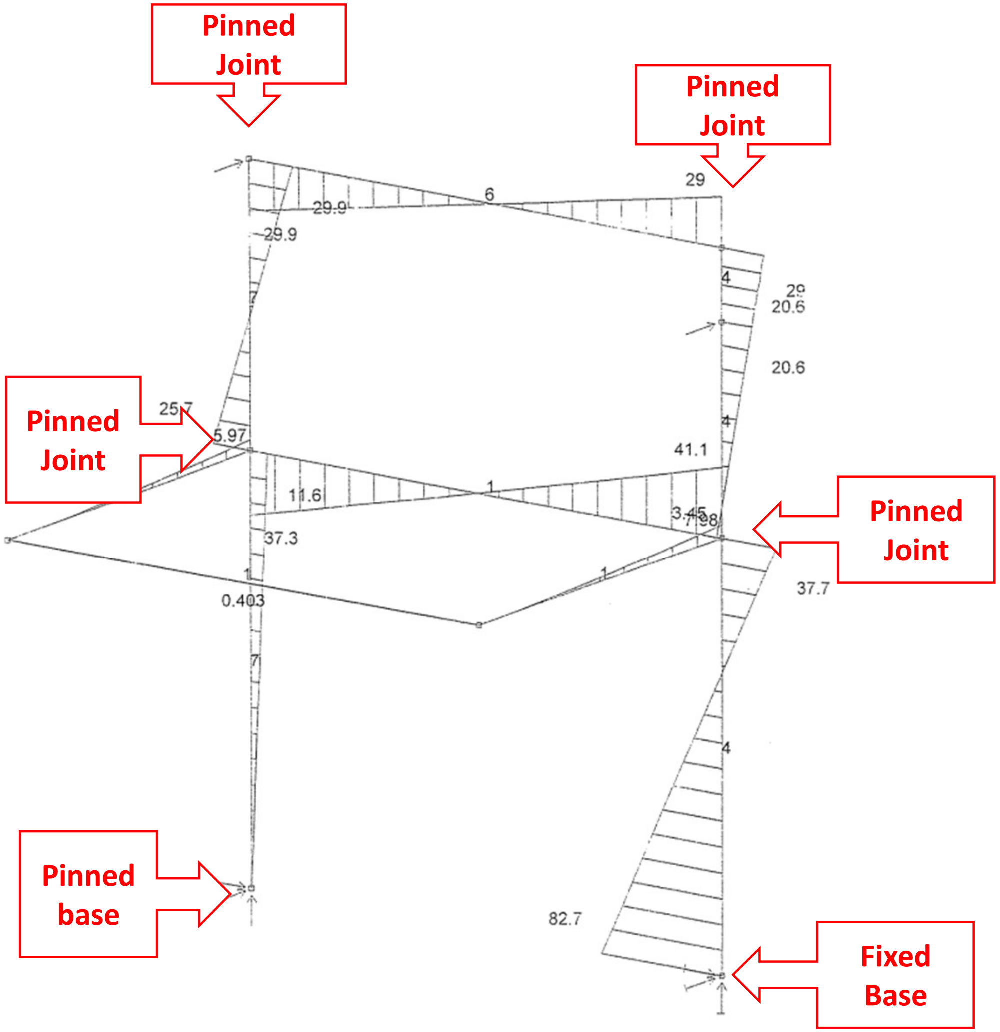


Actual Model

- A lot more flexible without the rigid joints
- Redistributes frame forces, and is stable only due to fixed base

Implications?

- Exceeds drift limits
- Members and base connections non-compliant as specified



What does good look like for a moment joint?

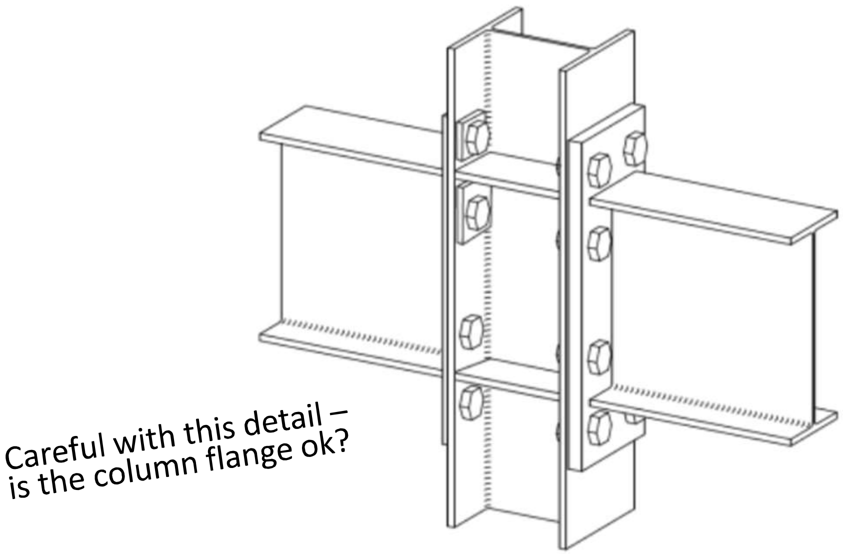


Figure 1: Moment End Plate Connection

SCNZ Steel Advisor CON1001 - Moment End Plate Column Side

Consideration of Load Path through the joint

- Proper and continuous load path for flange tension/compression forces
- Clear and direct load path – flanges aligned with stiffeners

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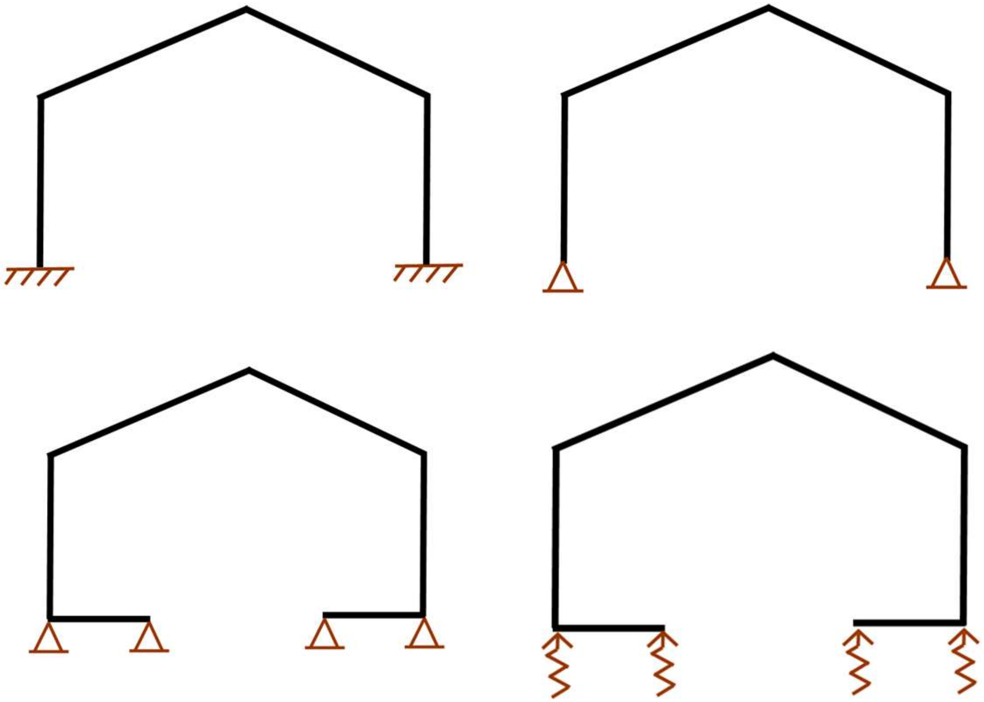
What does good look like for a moment joint?

SCNZ Steel Connections Guide

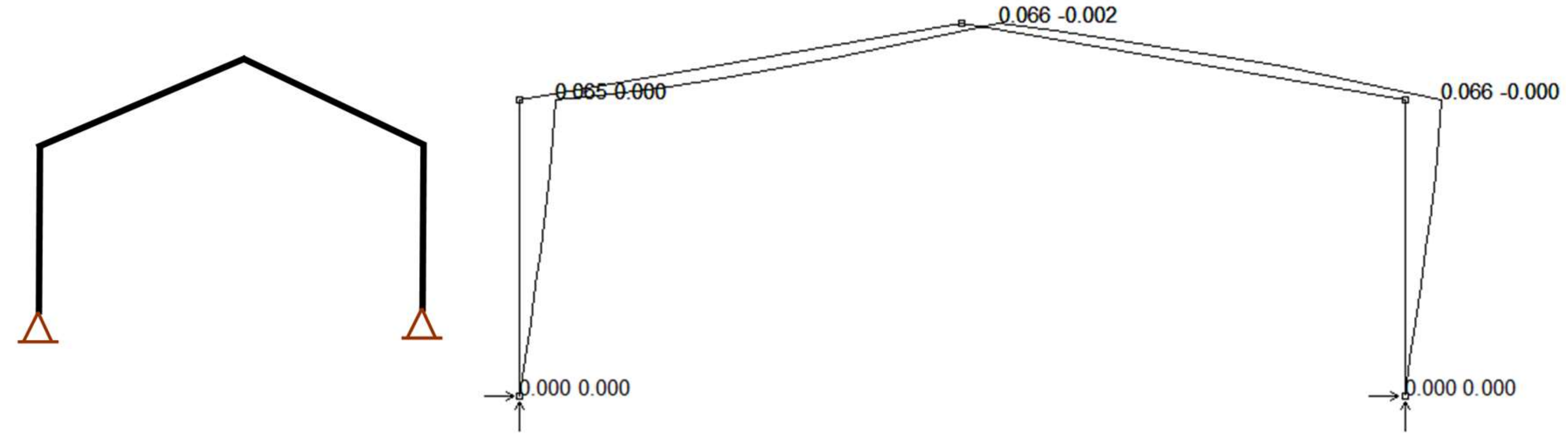
What tools are available to us?

- SCNZ Connections Guide provides some standard moment connections (note these are for steel to steel only – not suitable for steel to concrete!)
- Can be used with caution – make sure it fits all requirements (axial loads, geometry, fabrication tolerances)

Importance of understanding the uncertainty of modelling

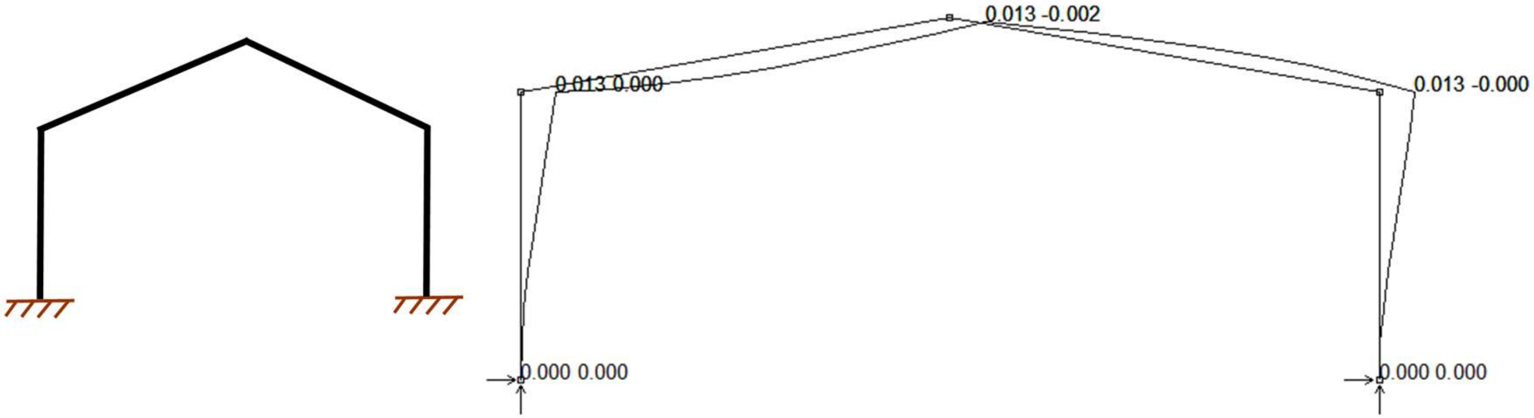


- We always make assumptions/ simplifications when we model
- Worth considering sensitivity analysis



- Traditional pinned base - deflection = height/60

With the click of a few buttons in the model…



- Fixed base deflection = height/307

How are we achieving fixity?
Ground beams?

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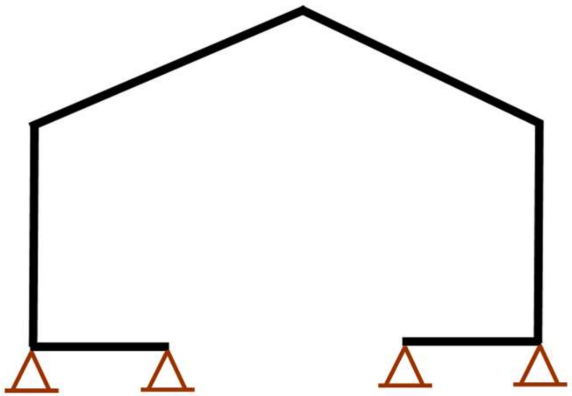
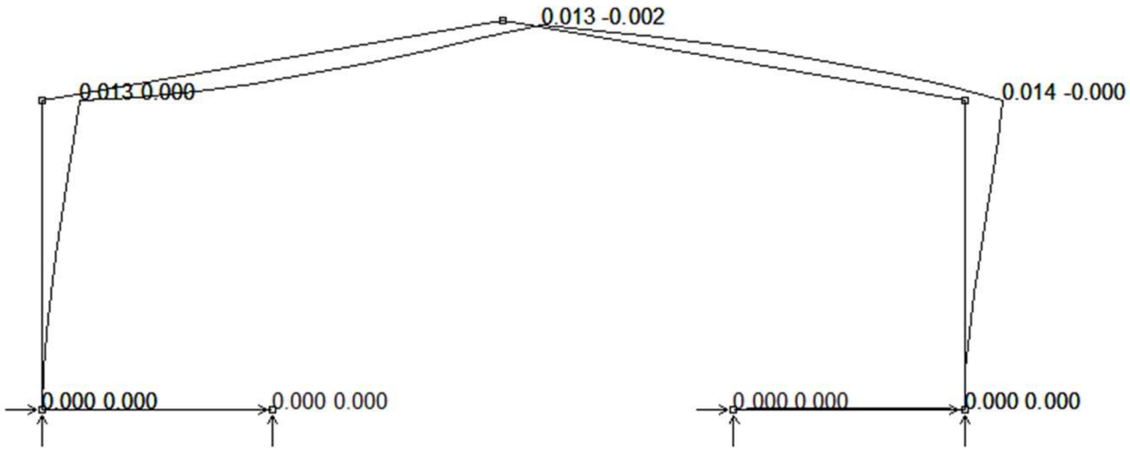
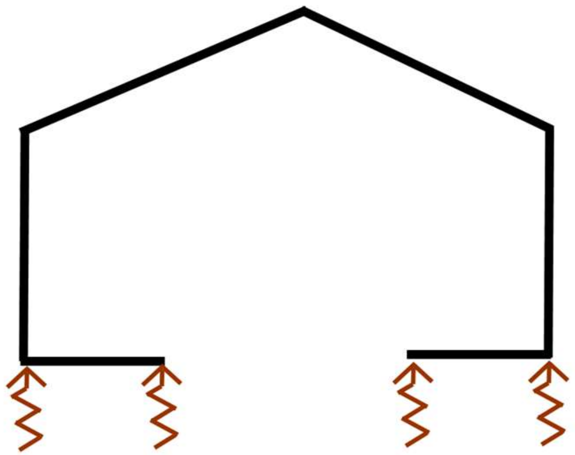
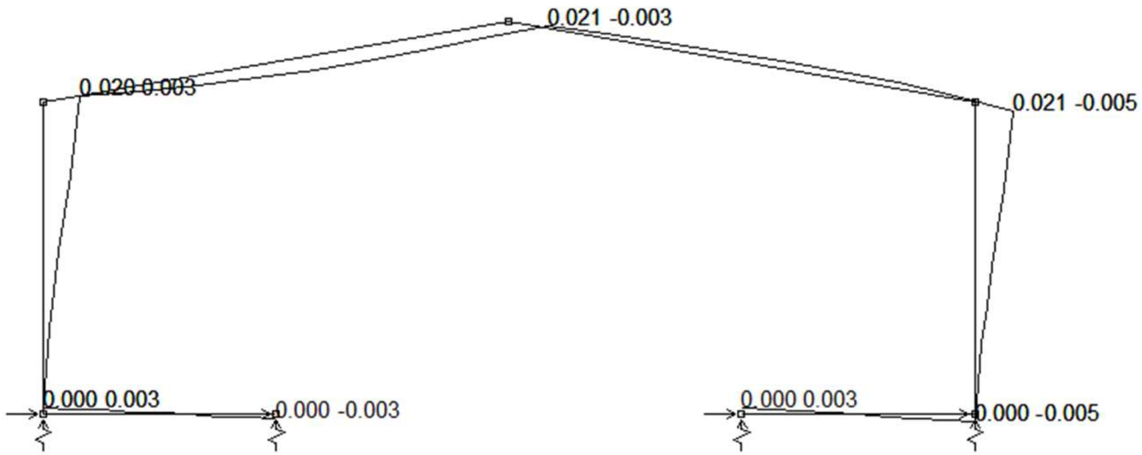




- Lets update our model and add the ground beams, with pin support at each end?
- Deflections = height/285



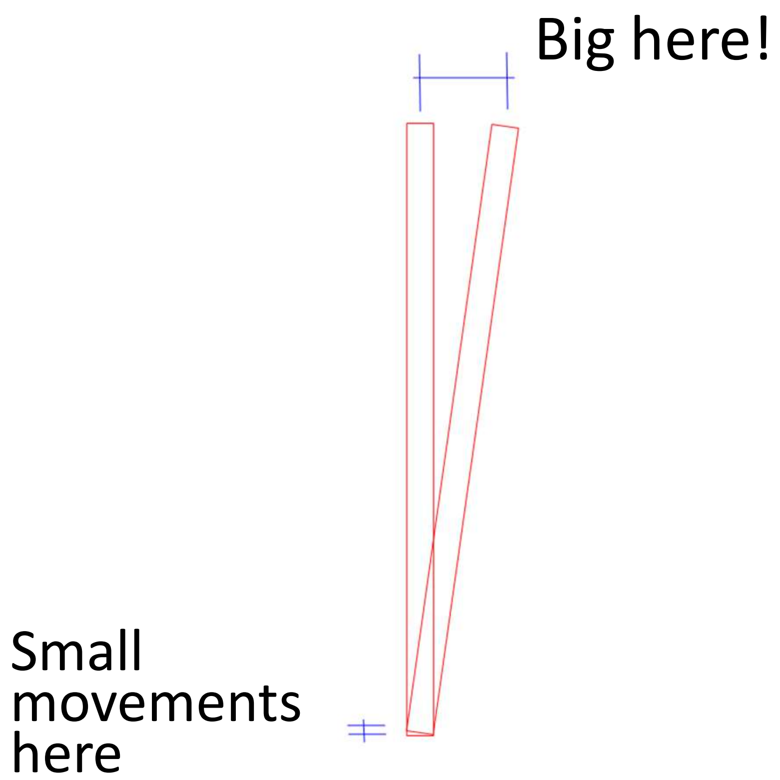


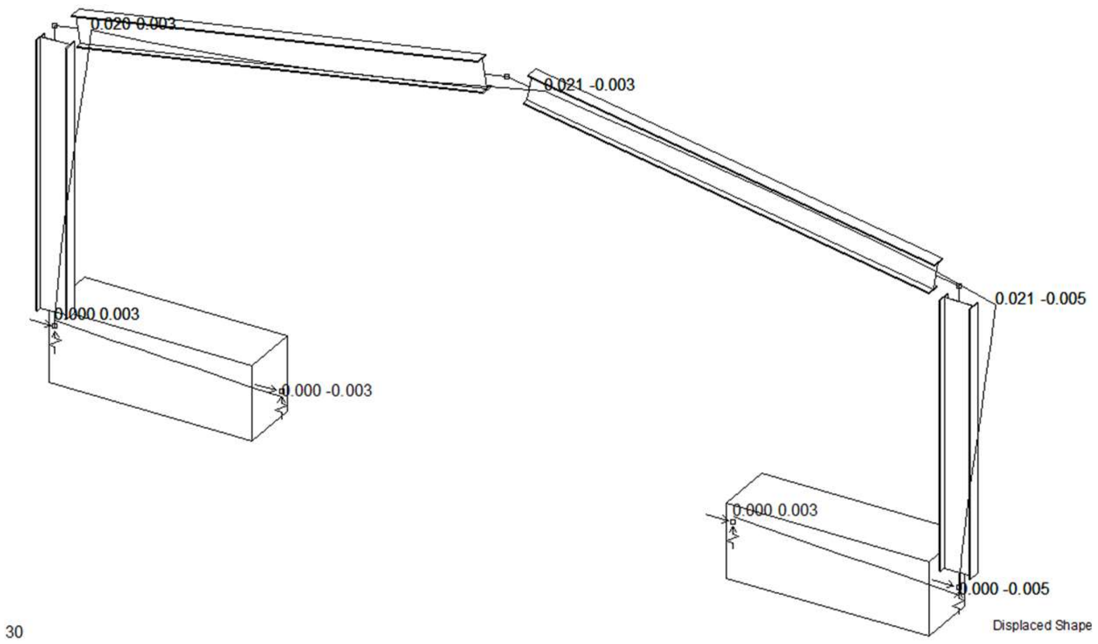
What about if we tweak our model – add a few spring supports, and allow a little foundation deflection…

- Deflection = height/190

What else can lead to more displacement?

- Watch your base plate details!
- Flexure of the base plate can lead to increased displacements
- Small rotation at the base plate can lead to large displacements at eaves level





All the details shown provide 'fixity' , but with different levels of modelling our displacements went from height/307 to height/190

The accuracy of our model, and all the assumptions that go into it really influence our structure!

Sometimes its best to complete upper bound/lower bound sensitivity checks

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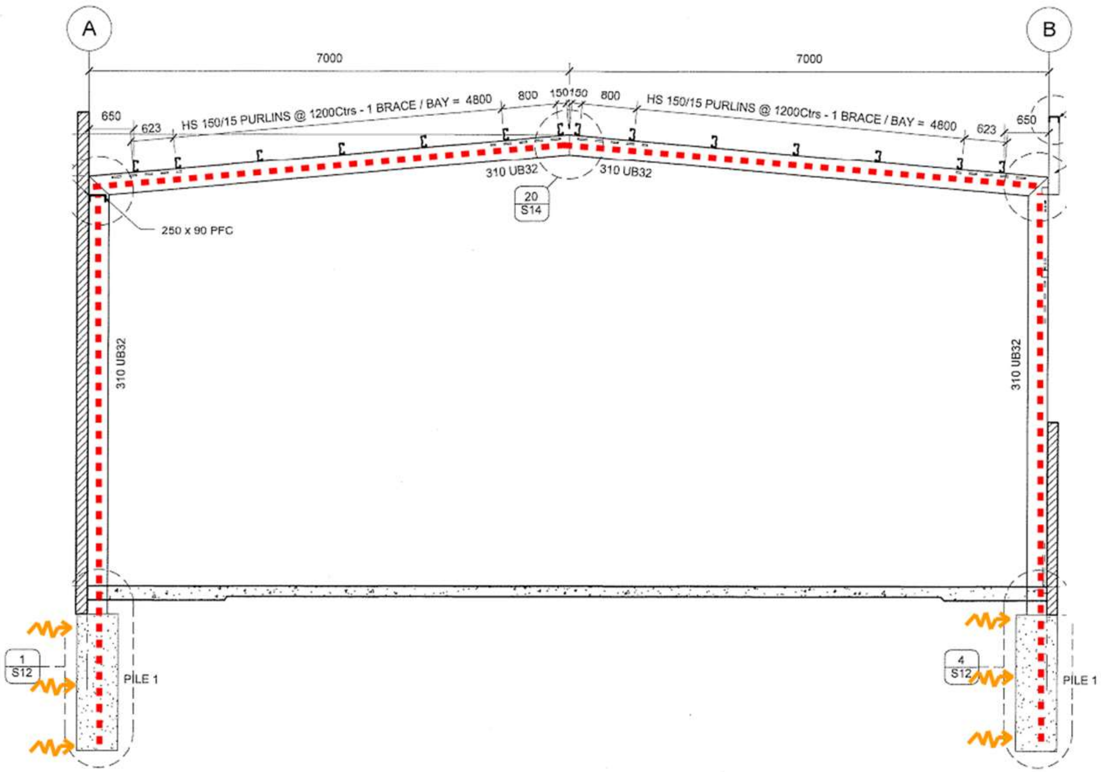
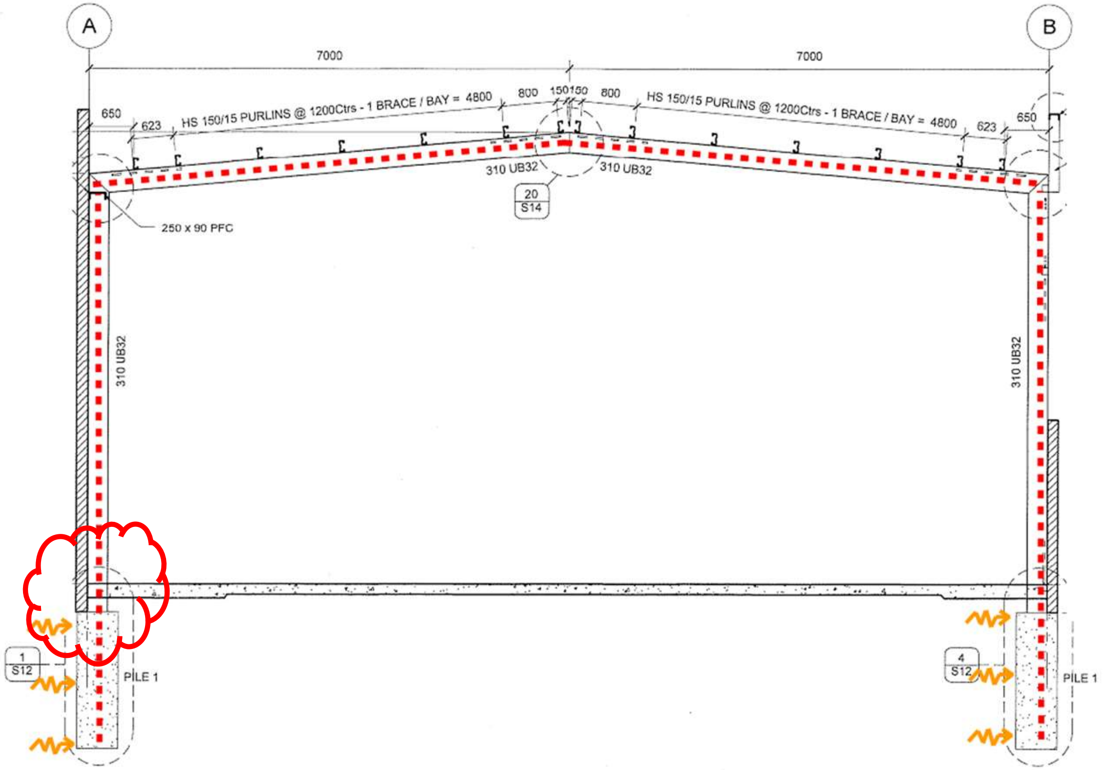
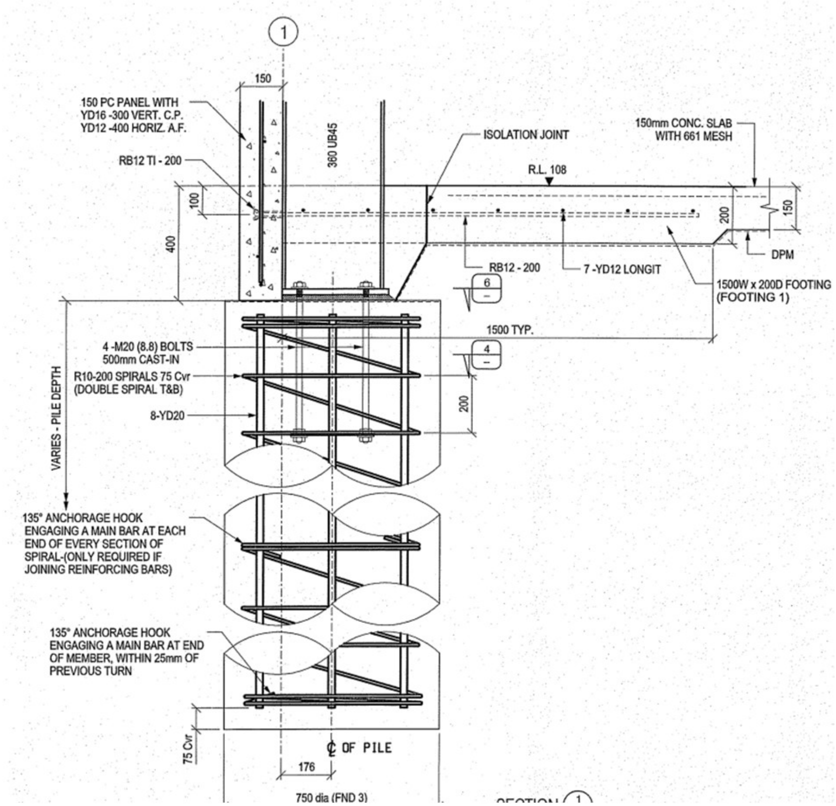


- What about if we try using piles?
- Increases uncertainty as we rely more on soil-structure interaction
- Depending on your soils, the pile could be likely to fail surrounding soils through bearing
- Likely to lead to increasing displacements of the system
- May need an L-Pile analysis to understand how the structure works?

Cannot just model as ‘fixed’base!

Connections…



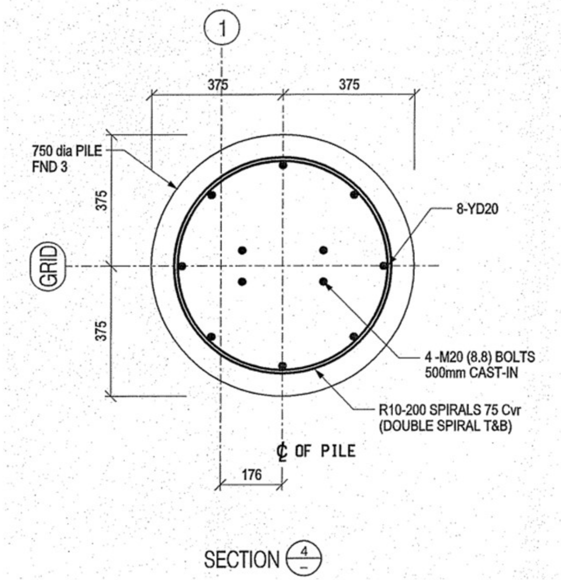


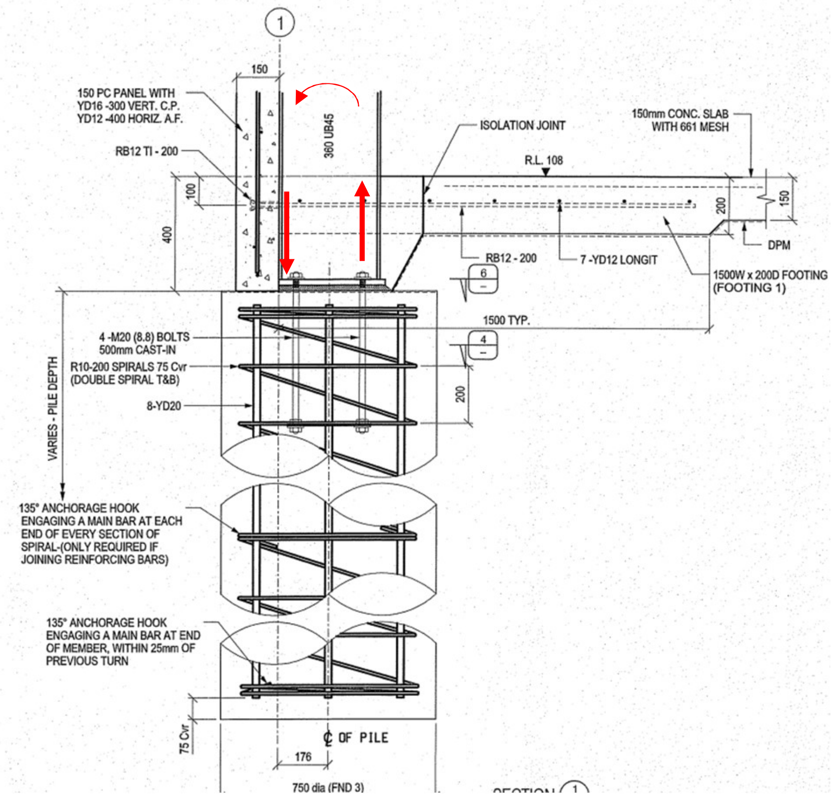
Frame to pile connection critical

Portal Connection to Pile

- No 90 degree hook at top of vertical reinforcing bars
- M20 bolts are cast in 500mm, so insufficient lap with reinforcing
- Base plate & welds also insufficient

This connection cannot transfer the loads required





Modelling vs Real Life

- Remember that a model is only a representation of a real structure!

- Reality will always be different

Tip #1

Make sure your design matches your model

Or

Make sure that your structure is accurately modelled

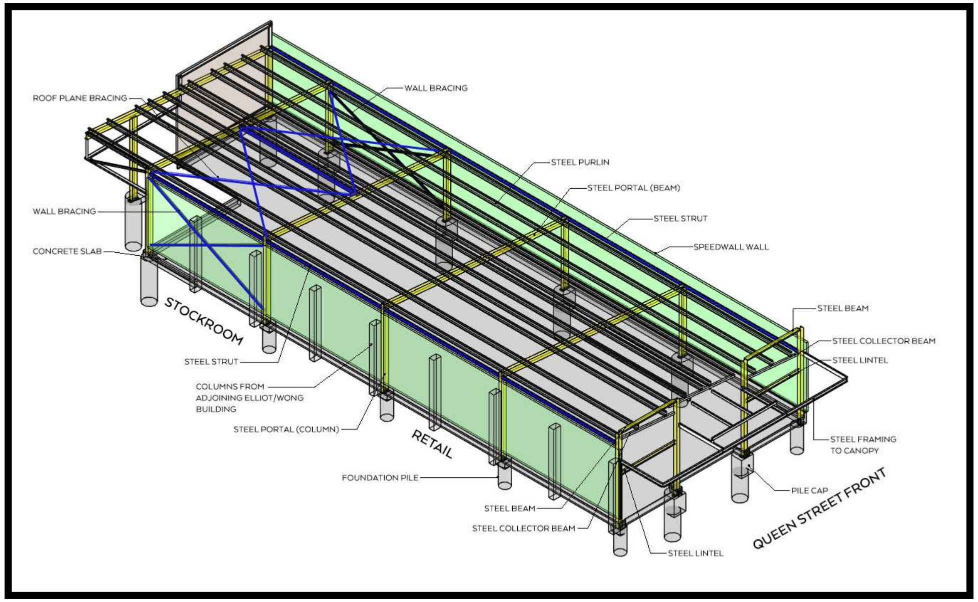
Tip #2

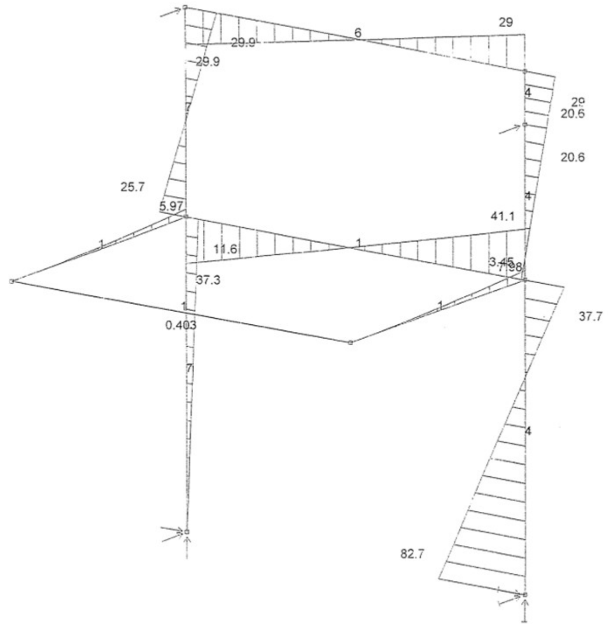
1	Make sure your design matches your model	☑
2		

Tip #2

Make sure your have a load path

Building A - Facade

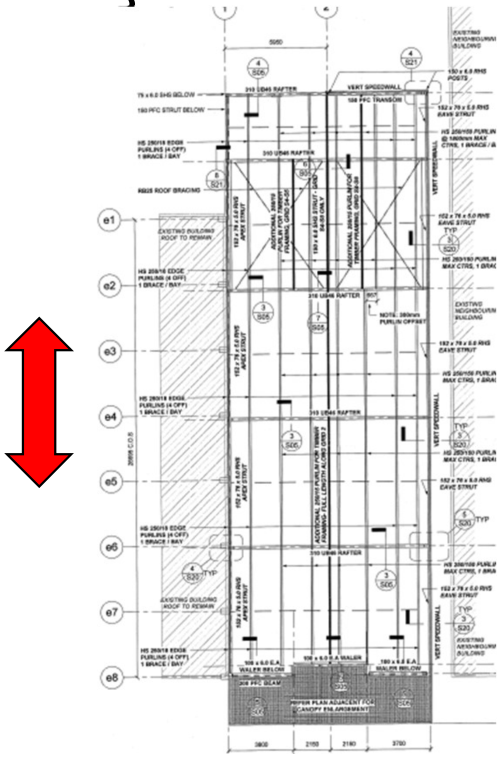




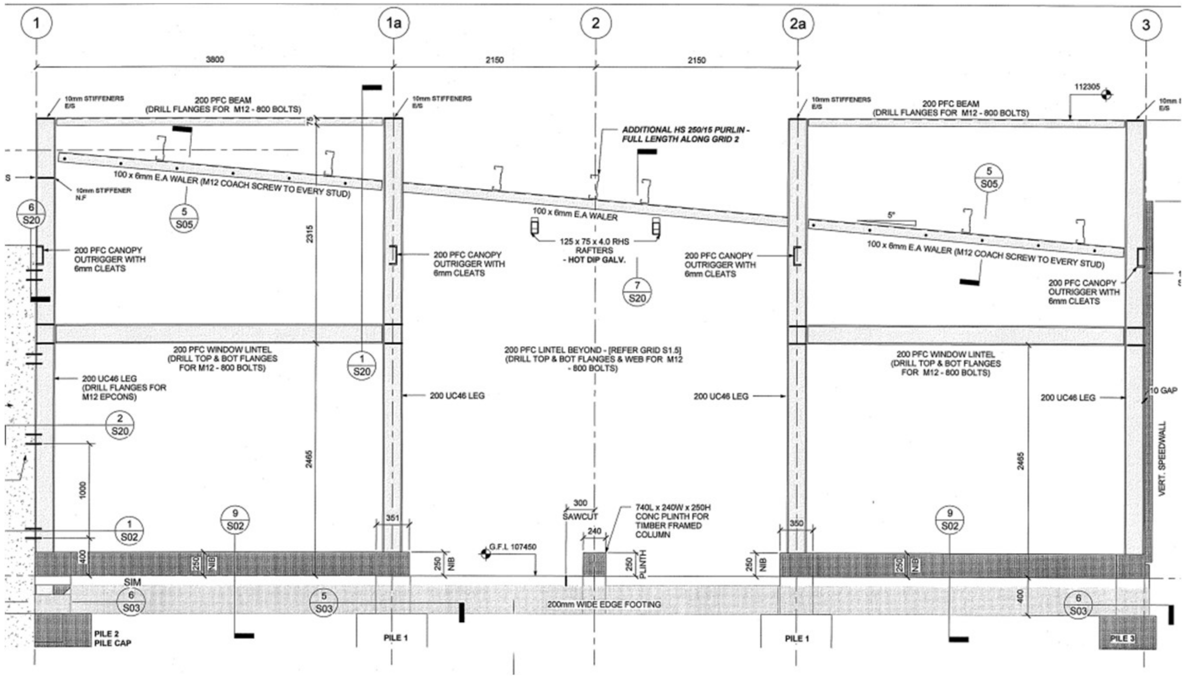
Same building as per previous example

What about out-of-plane?

Façade frame – Out-of-Plane actions



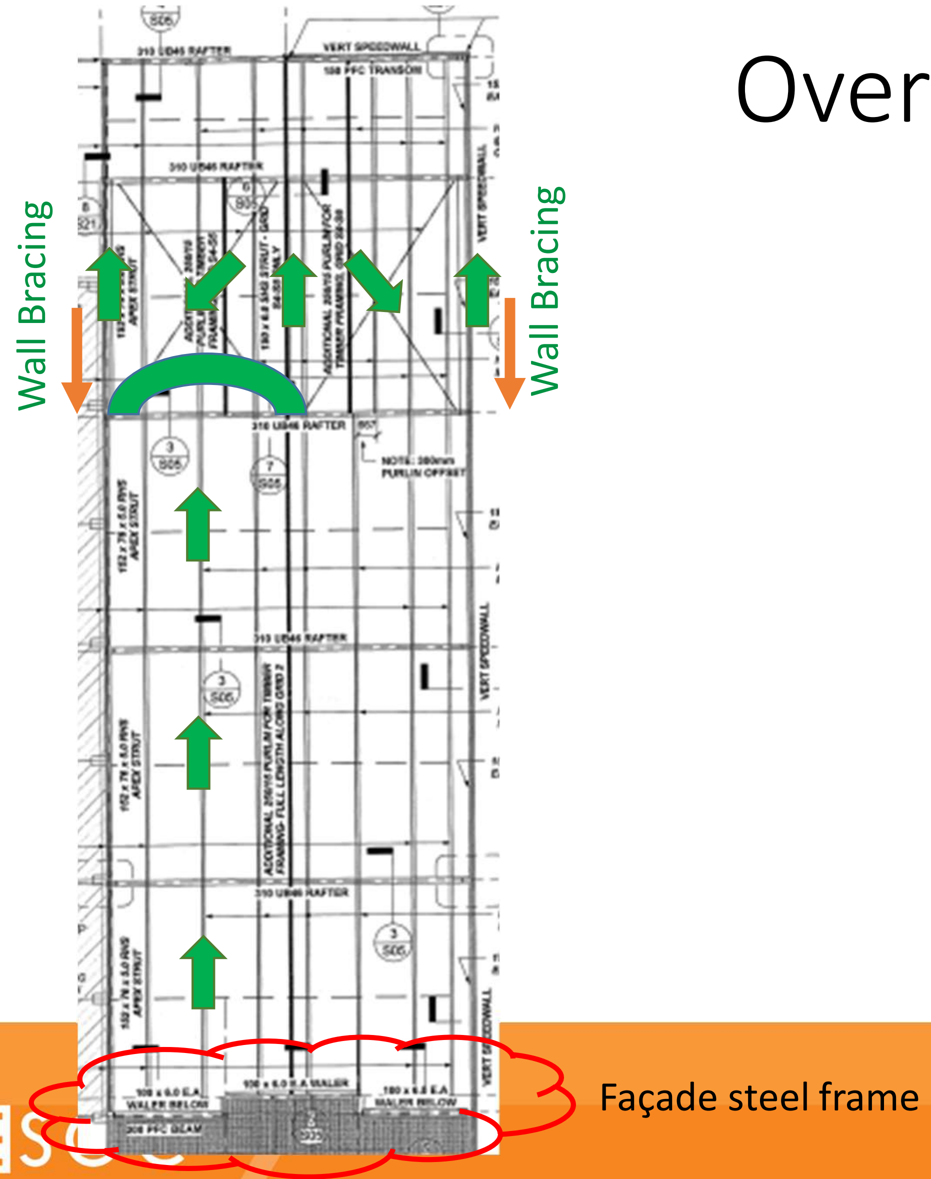
Roof Framing Plan



Façade Elevation

What about out-of-plane?

Overall Longitudinal Load Path



Out of Plane (façade and end wall)
+ roof loads

to DHS purlins as struts

to Rafter bends in weak direction

to Central strut

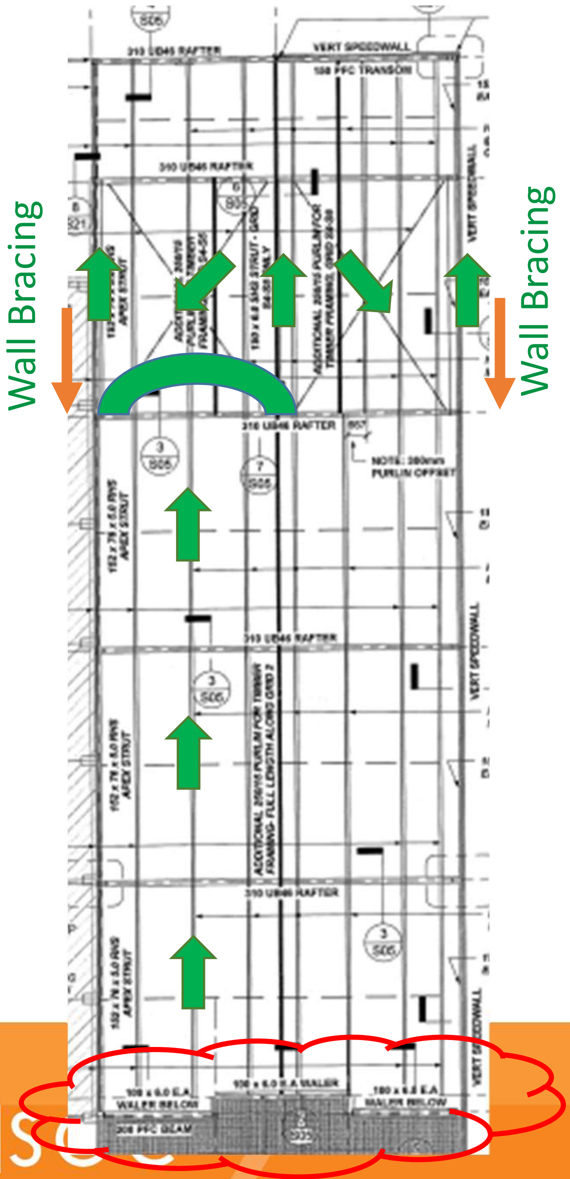
to Tension roof plane bracing

to Strut at eaves

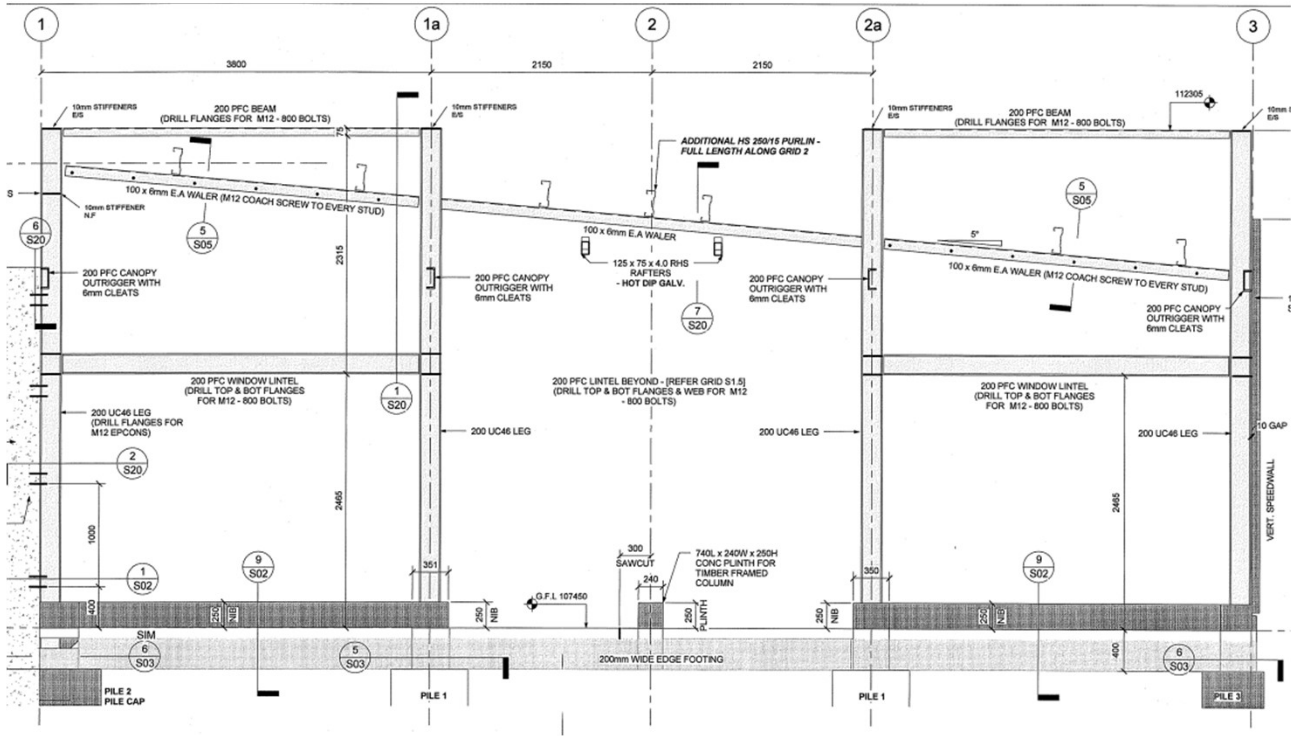
to Wall plane tension bracing

to Foundations

Longitudinal Load Path



Façade steel frame

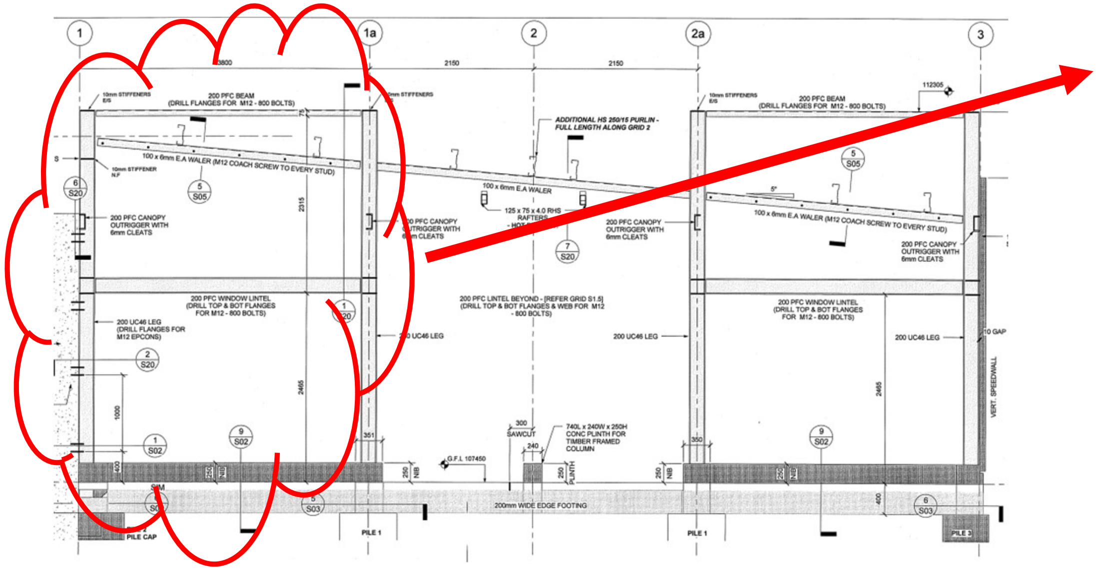


Façade Steel Frame

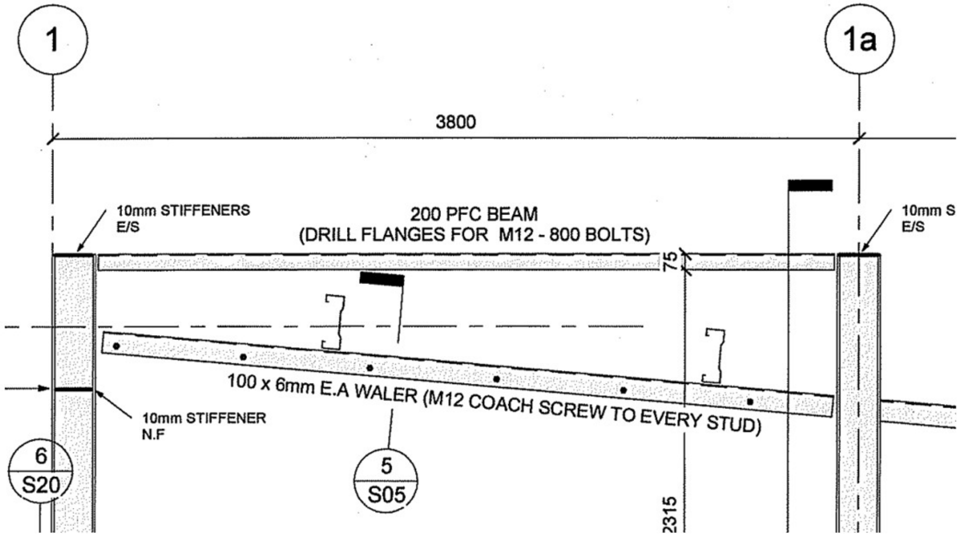
Façade out-of-plane

- The weight of the façade and the canopy needs to be supported out-of-plane and tied back to the roof plane, to be transferred via roof plane bracing to the side wall braces.
- Some connection at the eaves as there is an eaves strut
- On first look, it appears the DHS purlins could provide support out-of-plane

Façade Out-of-Plane



- Purlins supported on an EA waler bolted to timber framing



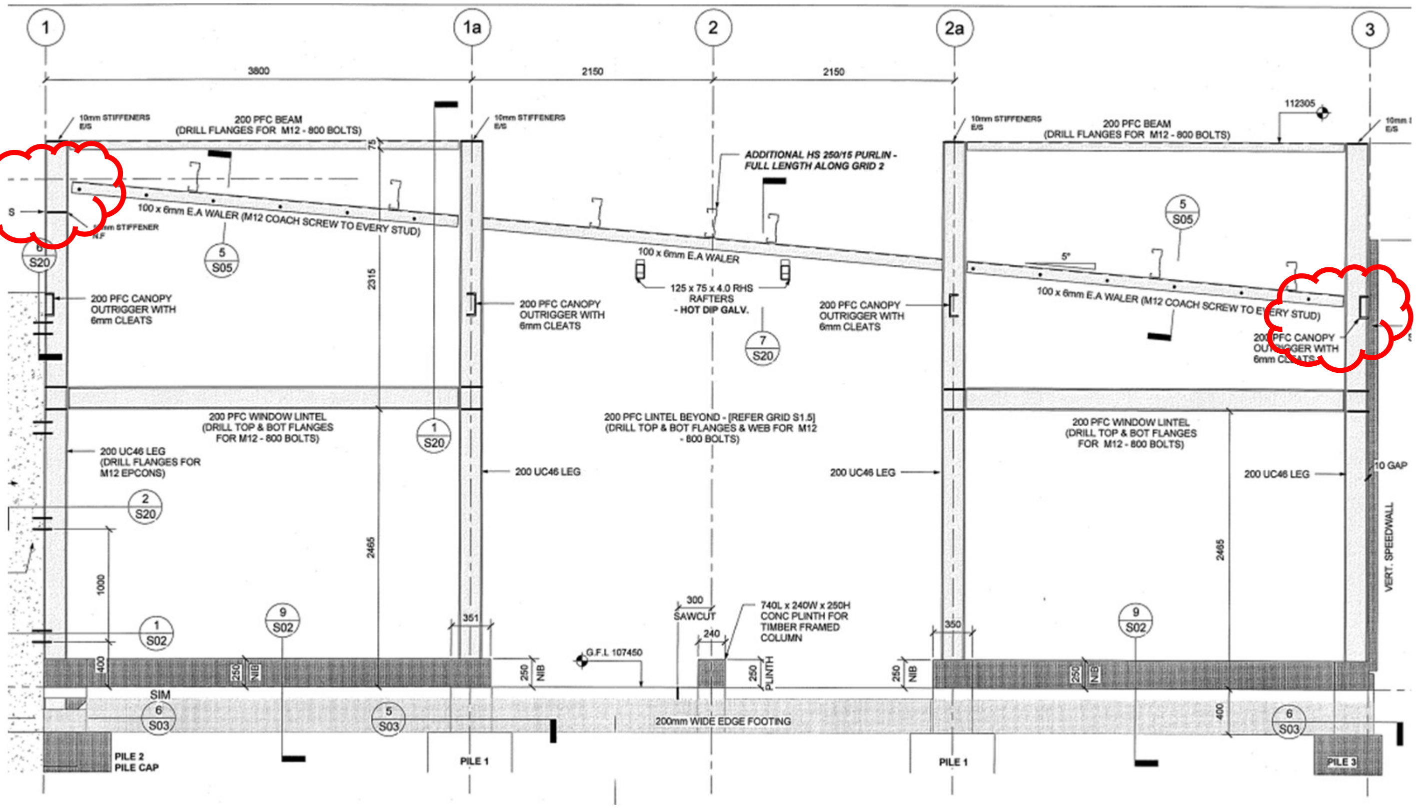
But the EA Waler is not connected to the UC columns!

Hard to spot!

No connection

Façade Out-of-Plane

A connection is present at the eaves via an eaves strut



There is no load path from façade frame to DHS purlins

& therefore

There is no out-of-plane load path for the facade

Façade Out-of-Plane

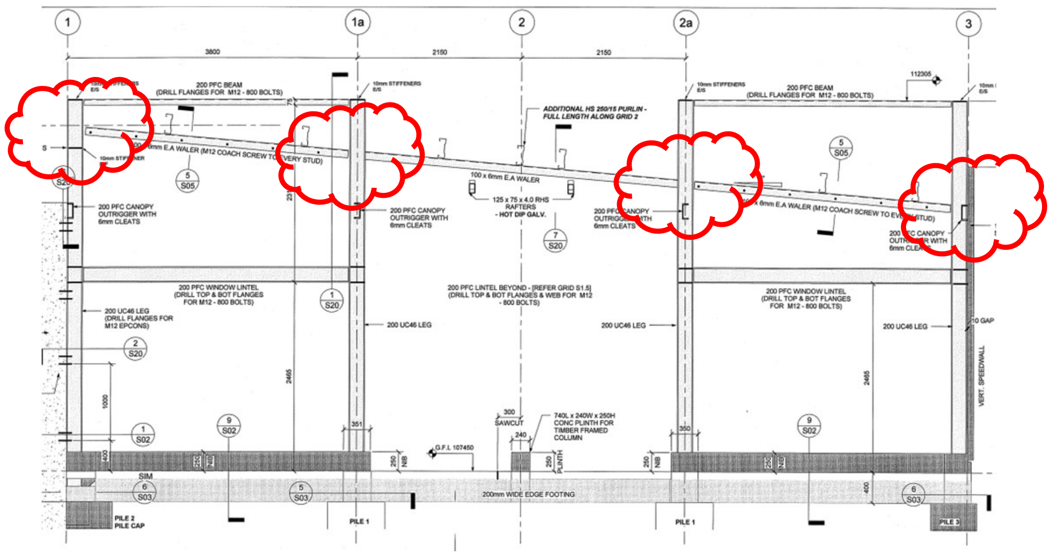
Implications?

- We have no clear & direct load path
- Reliance is on secondary load paths
- Non-compliant structure
- Not robust

&

- Potentially unsafe structure

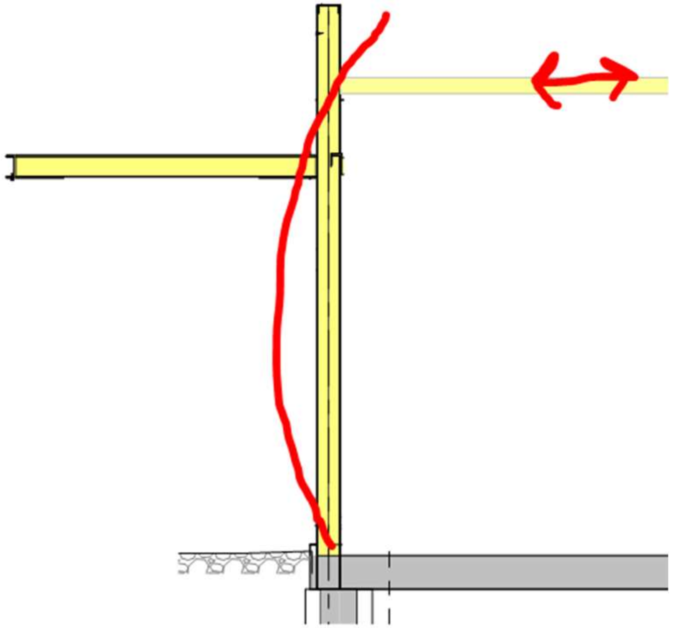
What does good look like for a façade OOP?



Consideration of Load Path of the facade

- Provide a direct load path – connect all façade columns to struts
- Struts transfer loads to side with tension roof plane bracing
- Either add wall braces in this bay; or
- Make sure you can transfer loads via the eaves strut to the wall bracing bay

What would a load path look like?



- Direct load path – connect façade to struts
- Struts transfer loads to side with tension roof plane bracing
- Either add wall braces in this bay; or
- Make sure you can transfer loads via the eaves strut to the wall bracing bay

Tip #2

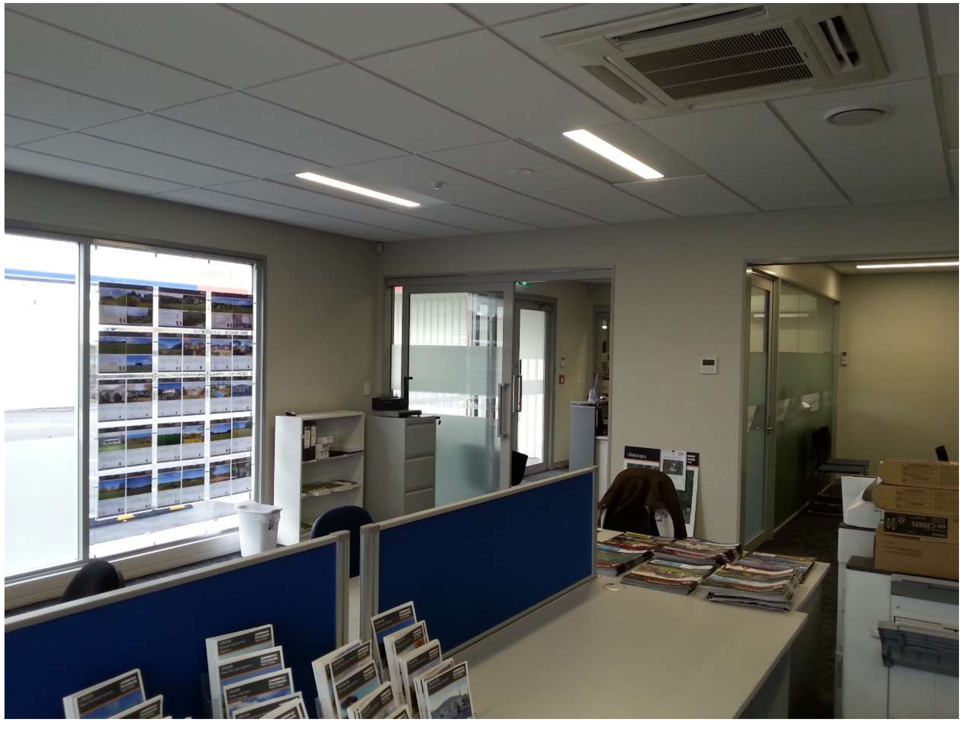
Make sure your have a load path

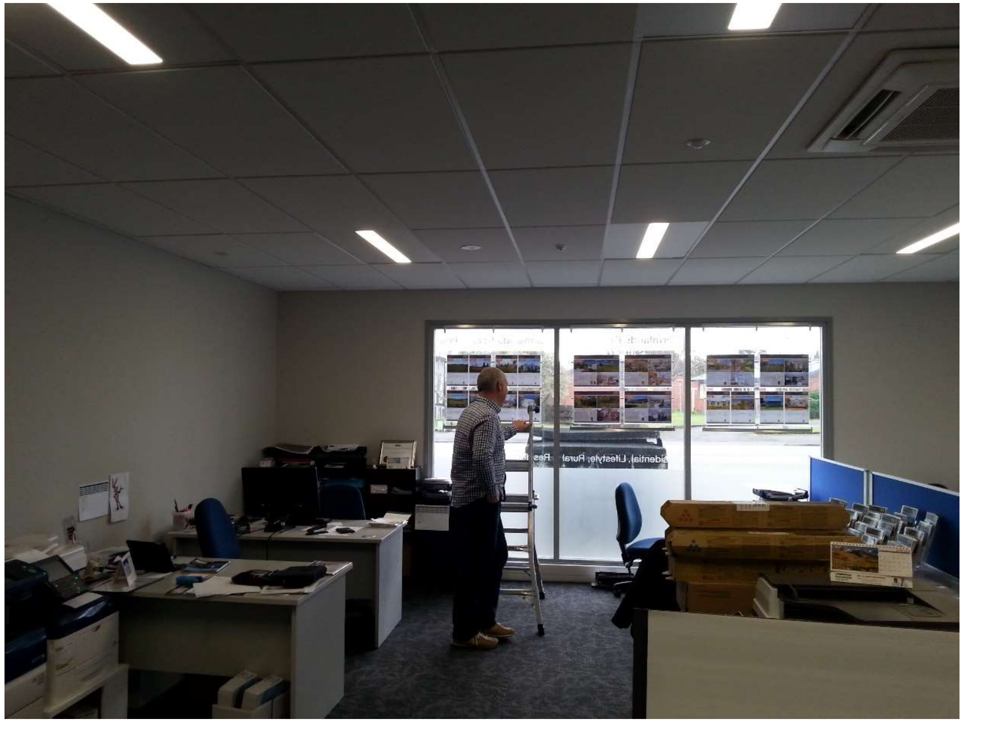
Building B

- Single storey extension to an existing building
- 11.4m x 13.4m
- Timber framed walls with plasterboard linings
- Masonry block boundary wall
- Posi-strut roof framing
- Suspended ceiling

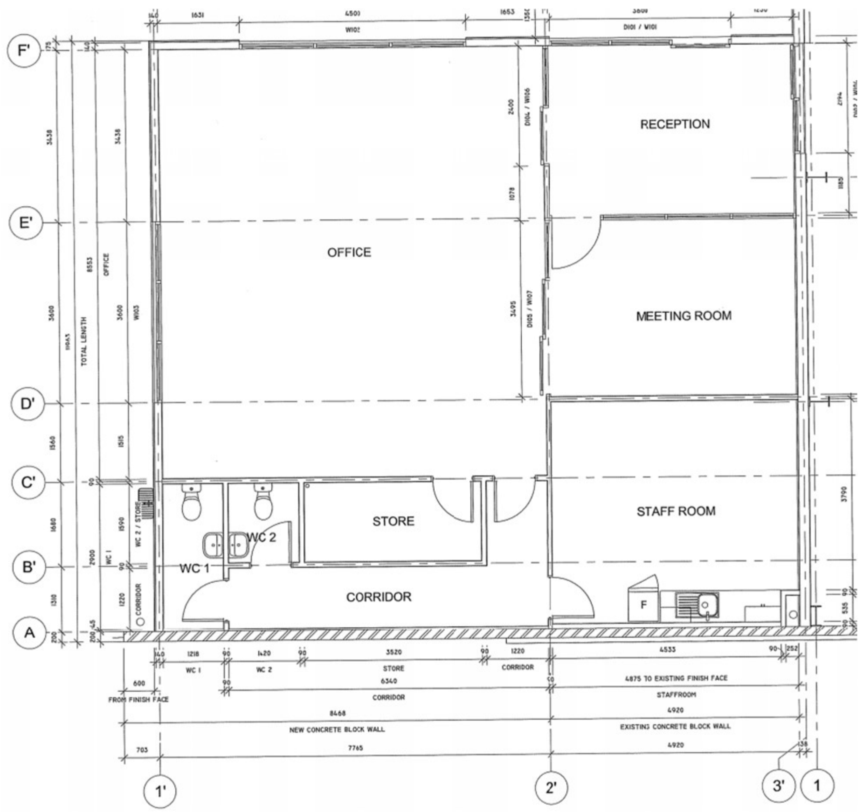

Real Estate


Farmlands
co-operative
Rural Supplies
Nutrition
Horticulture
Grain & Seed
Fuel & Lubricants
Finance
Real Estate
Livestock



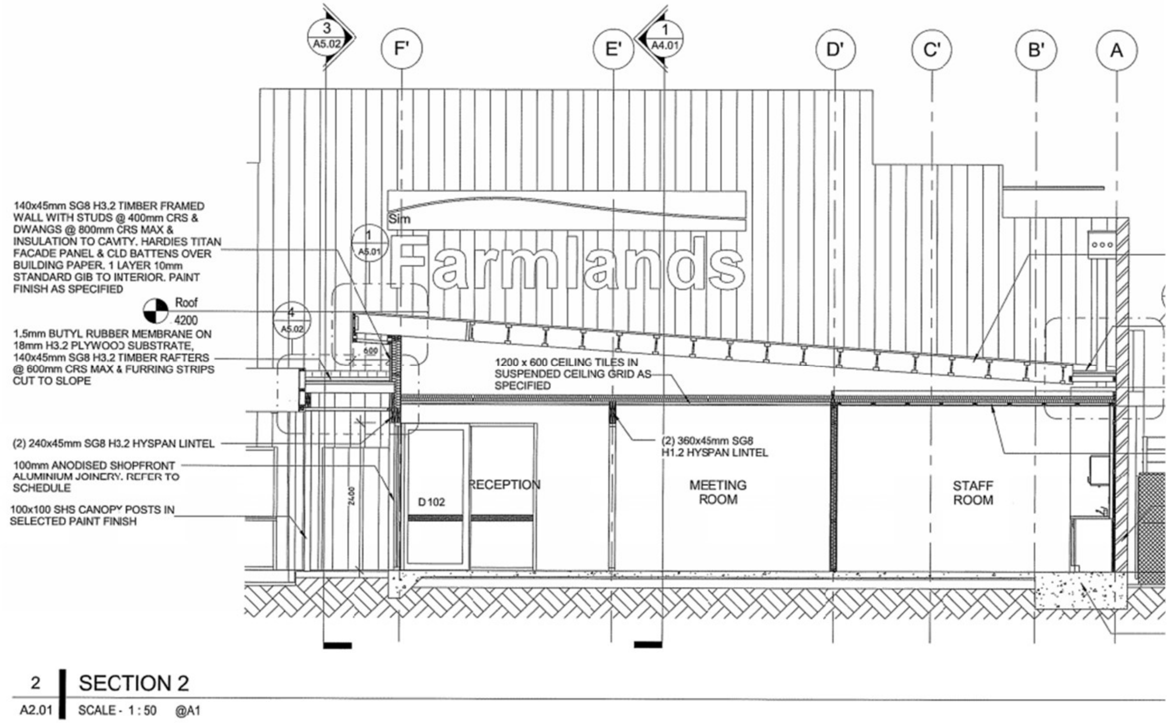


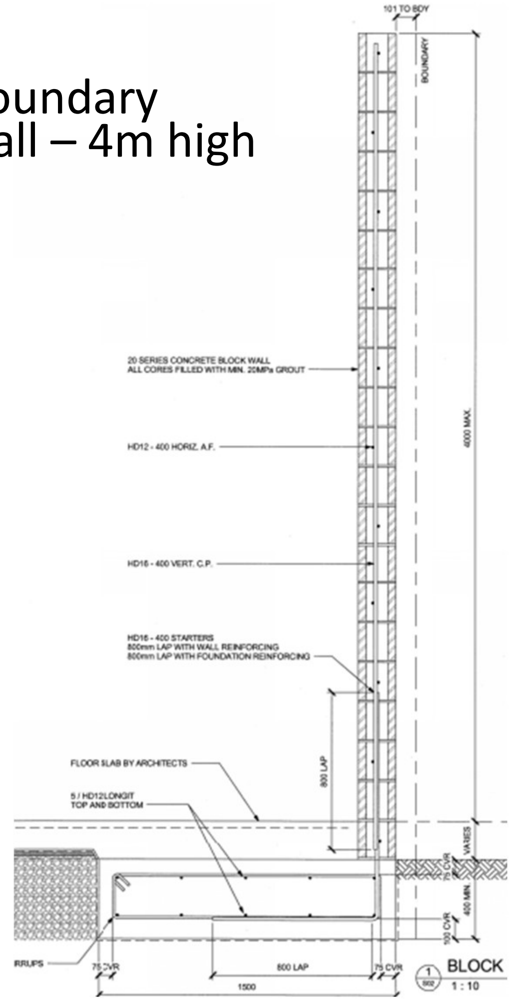
Building B

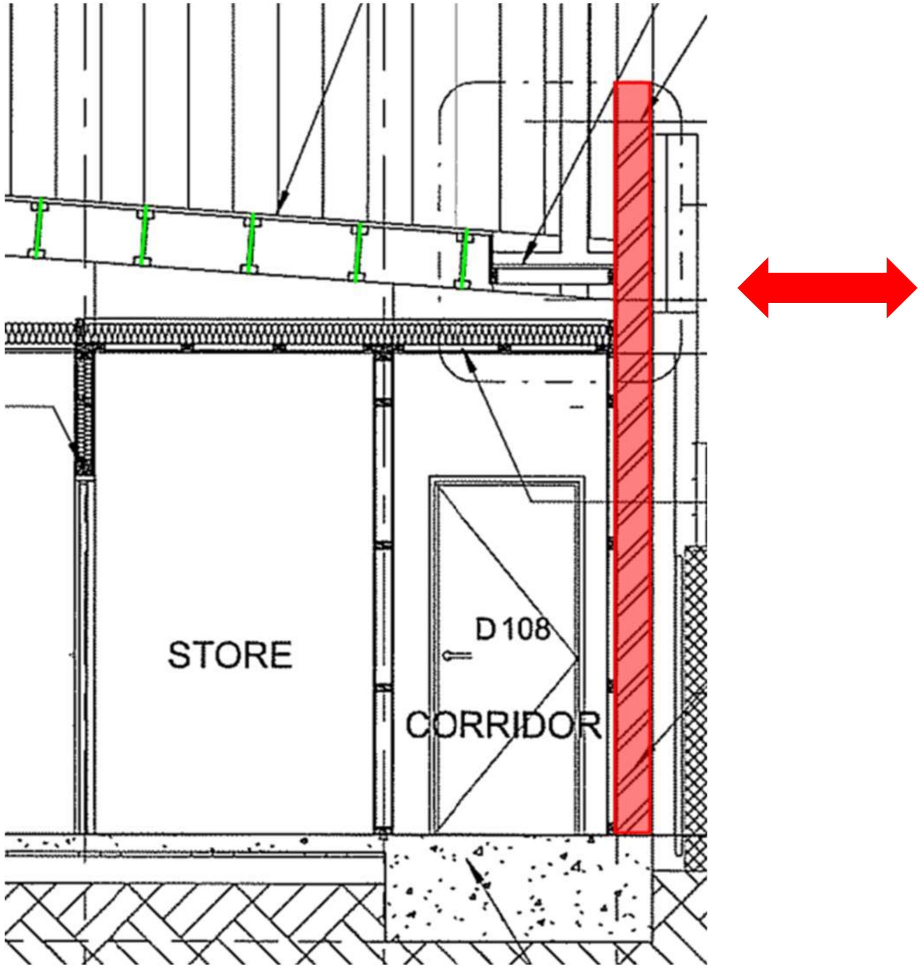

F'
E'
D'
C'
B'
A
RECEPTION
OFFICE
MEETING ROOM
STORE
STAFF ROOM
WC 2
WC 1
CORRIDOR
1'
2'
3'
1


F'
E'
D'
C'
B'
A
F'/2 - 1.4 - GS1-N
F'/3 - 1.2 - GS1-N
3438
M/2 - 3.4 - GS1-N
Q/3 - 1.1 - GS1-N
3600
Q/2 - 3.5 - GS'-N
1560
C'/1 - 5.3 - GS1-N
D'/1 - 4.8 - GS1-N
1680
M/1 - 2.9 - GS1-N
N/1 - 1.6 - GS1-N
O/1 - 1.6 - GS1-N
P/1 - 3.5 - GS1-N
Q/1 - 3.8 - GS1-N
1310
7765
5023
182
1'
2'
3'

Masonry block boundary masonry block wall – 4m high







How is the wall braced out-of-plane?

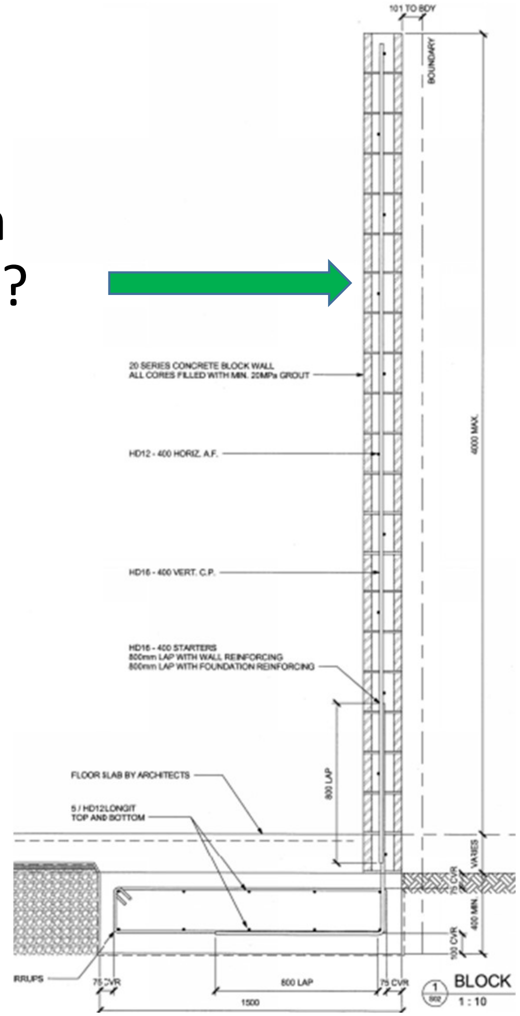
- Designed to cantilever for the post-fire case of 0.5kPa face load
- However no load path for ULS seismic actions
- Posi-strut rafters are parallel to the wall

There is no load path for the wall out-of-plane

SESOC

In theory the wall could have been propped by some structure at ceiling level

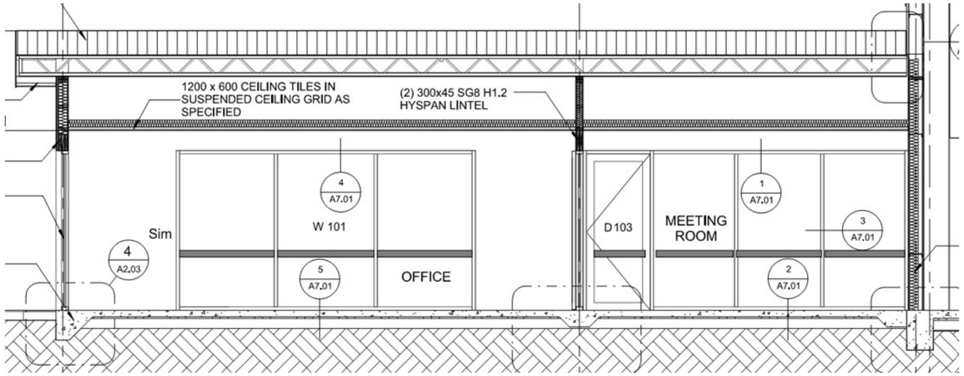
Ply Diaphragm at ceiling level?



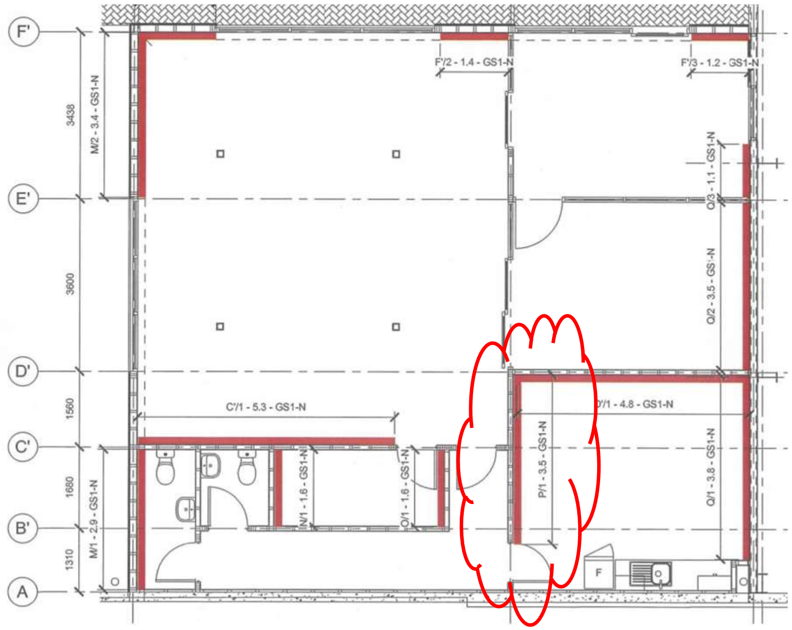
- Ceiling diaphragm could have transferred loads to the in-plane walls.
- The in-plane walls would have needed to be plywood shear walls
- Ceiling diaphragm, walls and connections all needed to be specific engineer designed

Tip 2 – Make sure you have a load path

Same building, but lets look at the rest of the bracing system



- A suspended ceiling has been specified by the architect
- The engineer has provided a bracing plan
- But there are gaps…



- This is meant to be a bracing wall
- Linings stop just above the suspended ceiling
- No bracing load path for roof to wall, and wall bracing element not constructed properly

Tip 2 – Make sure you have a load path

What should we be seeing?

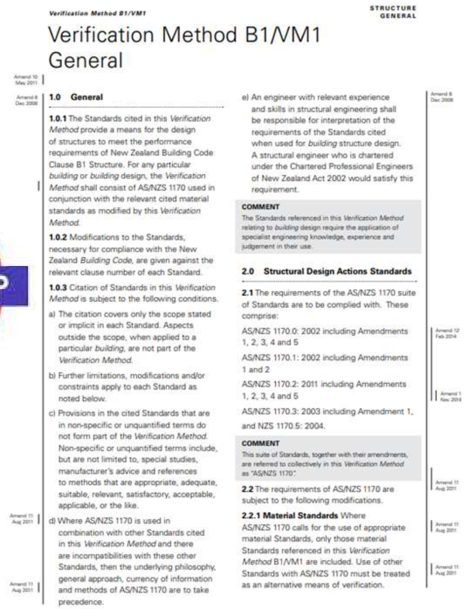
A load path!

- Bracing wall linings should have gone all the way to the top plate

- Blocking between posi-struts to stop roll-over and transfer roof loads to top plate

Watch the gaps between B1/AS1 and B1/VM1
(or the gaps between where 3604 finishes and SED starts)

Acceptable Solution



Verification Method B1/VM1

STRUCTURE GENERAL

Verification Method B1/VM1
General

1.0 General

1.0.1 The Standards cited in this *Verification Method* provide a means for the design of structures to meet the performance requirements of New Zealand Building Code Clause B1 Structure. For any particular *building* or *building* design, the *Verification Method* shall consist of AS/NZS 1170 used in conjunction with the relevant cited material standards as modified by this *Verification Method*.

1.0.2 Modifications to the Standards, necessary for compliance with the New Zealand *Building Code*, are given against the relevant clause number of each Standard.

1.0.3 Citation of Standards in this *Verification Method* is subject to the following conditions.

a) The citation covers only the scope stated or implicit in each Standard. Aspects outside the scope, when applied to a particular *building*, are not part of the *Verification Method*.

b) Further limitations, modifications and/or constraints apply to each Standard as noted below.

c) Provisions in the cited Standards that are in non-specific or unquantified terms do not form part of the *Verification Method*. Non-specific or unquantified terms include, but are not limited to, special studies, manufacturer's advice and references to methods that are appropriate, adequate, suitable, relevant, satisfactory, acceptable, applicable, or the like.

d) Where AS/NZS 1170 is used in combination with other Standards cited in this *Verification Method* and there are incompatibilities with these other Standards, then the underlying philosophy, general approach, currency of information and methods of AS/NZS 1170 are to take precedence.

e) An engineer with relevant experience and skills in structural engineering shall be responsible for interpretation of the requirements of the Standards cited when used for *building* structure design. A structural engineer who is chartered under the Chartered Professional Engineers of New Zealand Act 2002 would satisfy this requirement.

COMMENT
The Standards referenced in this *Verification Method* relating to *building* design require the application of specialist engineering knowledge, experience and judgement in their use.

2.0 Structural Design Actions Standards

2.1 The requirements of the AS/NZS 1170 suite of Standards are to be complied with. These comprise:

AS/NZS 1170.0: 2002 including Amendments 1, 2, 3, 4 and 5

AS/NZS 1170.1: 2002 including Amendments 1 and 2

AS/NZS 1170.2: 2011 including Amendments 1, 2, 3, 4 and 5

AS/NZS 1170.3: 2003 including Amendment 1, and NZS 1170.5: 2004.

COMMENT
This suite of Standards, together with their amendments, are referred to collectively in this *Verification Method* as "AS/NZS 1170".

2.2 The requirements of AS/NZS 1170 are subject to the following modifications.

2.2.1 Material Standards Where AS/NZS 1170 calls for the use of appropriate material Standards, only those material Standards referenced in this *Verification Method* B1/VM1 are included. Use of other Standards with AS/NZS 1170 must be treated as an alternative means of verification.

Verification Method

- How are the out-of-plane walls supported and load transferred to the in-plane walls?
- NZS3604 assumes you have a ceiling to help out here.
- What about the walls parallel to the posi-strut rafters? How are they supported out-of-plane?

There is no load path to transfer out-of-plane wall loads to in-plane walls

Specific Engineer Designed elements

- Other items which needed specific design
  - Load path to bracing walls (roof plane bracing?)
  - Parapet framing and support
  - Support of top plate
- Vague load path leads to uncertainty
- If in doubt
  - Design it!
  - Tell the architect it is SED scope
  - Make sure there is a load path!

What’s the compliance pathway? B1/AS1 or B1/VM1?

- All building work must comply to the building code
- If it is not clearly covered by an Acceptable Solution, then it needs to be designed using the Verification Method
- A structural engineer needs to use the Verification Method and provide a PS1

Verification Method B1/VM1

STRUCTURE
GENERAL

Verification Method B1/VM1
General

1.0 General

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SESOC

Importance of the 'Part Only' statement with your PS1

- Clearly define what the specific engineer designed scope was with your 'part only' attachment to your PS1
- If its not covered in B1/AS1, then its part of the SED scope!

SCHEDULE to PS1

Please include an itemised list of all referenced documents, drawings, or other supporting materials in relation to this producer statement below:

- Design of boundary masonry block wall
- Design of roof plane bracing
- Bracing design
- Design of parapet

Tip #2

Make sure you have a load path

Tip #3

1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3		

Tip #3

Node your connections

Node: (noun) a point in a network or diagram at which lines or pathways intersect or branch

Building C

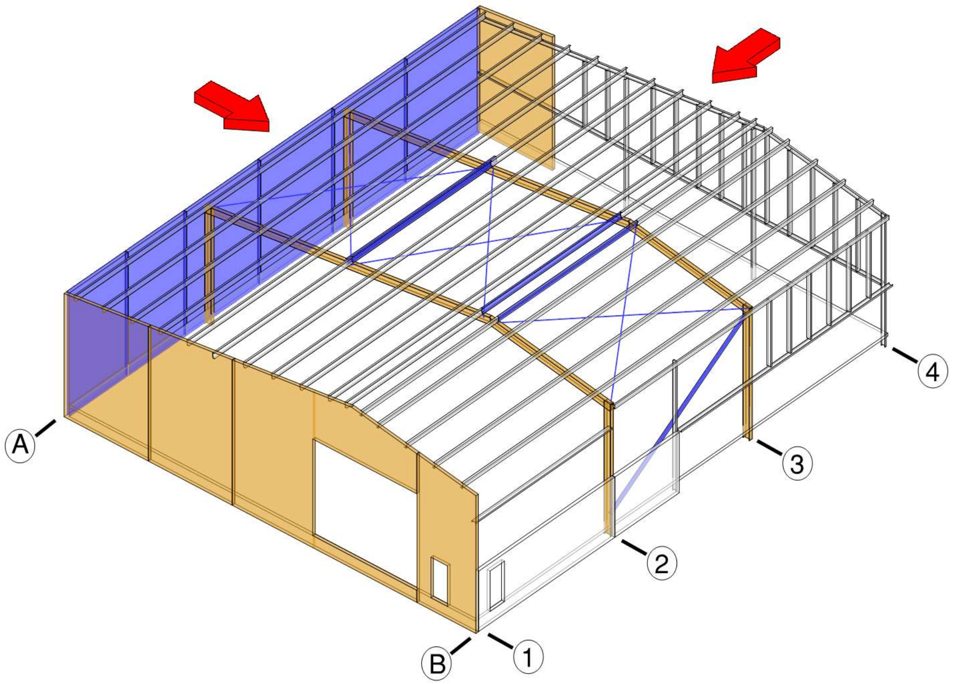
- Built 2006
- Typical modern building
- Single storey 25mx21m


Customer Entry


Customer Entry


Pressure Test Gauge

Building C

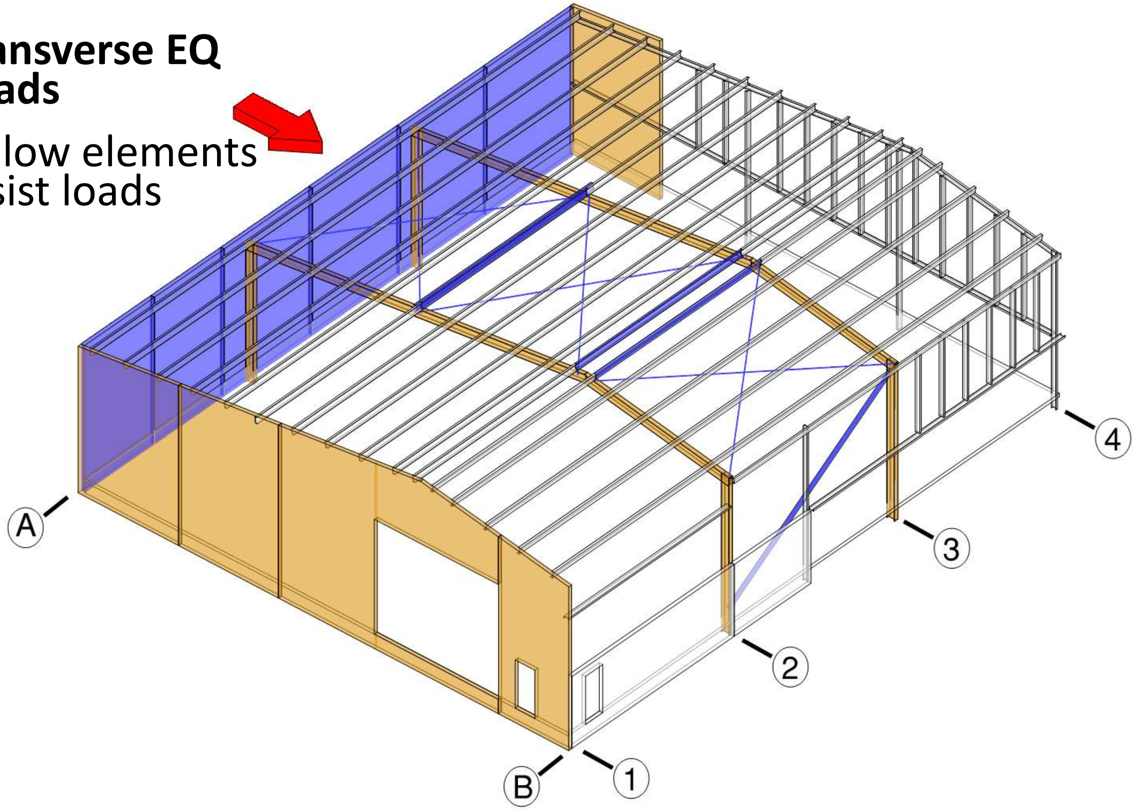


- 544m^2 single storey structure
- Reinforced concrete foundations
- 360UB45 steel portal frames at 8.36m centres
- 150mm precast concrete wall panels
- Tension bracing in the plane of the roof
- Tension/compression strut one side wall
- Mezzanine in part of the building

Transverse direction

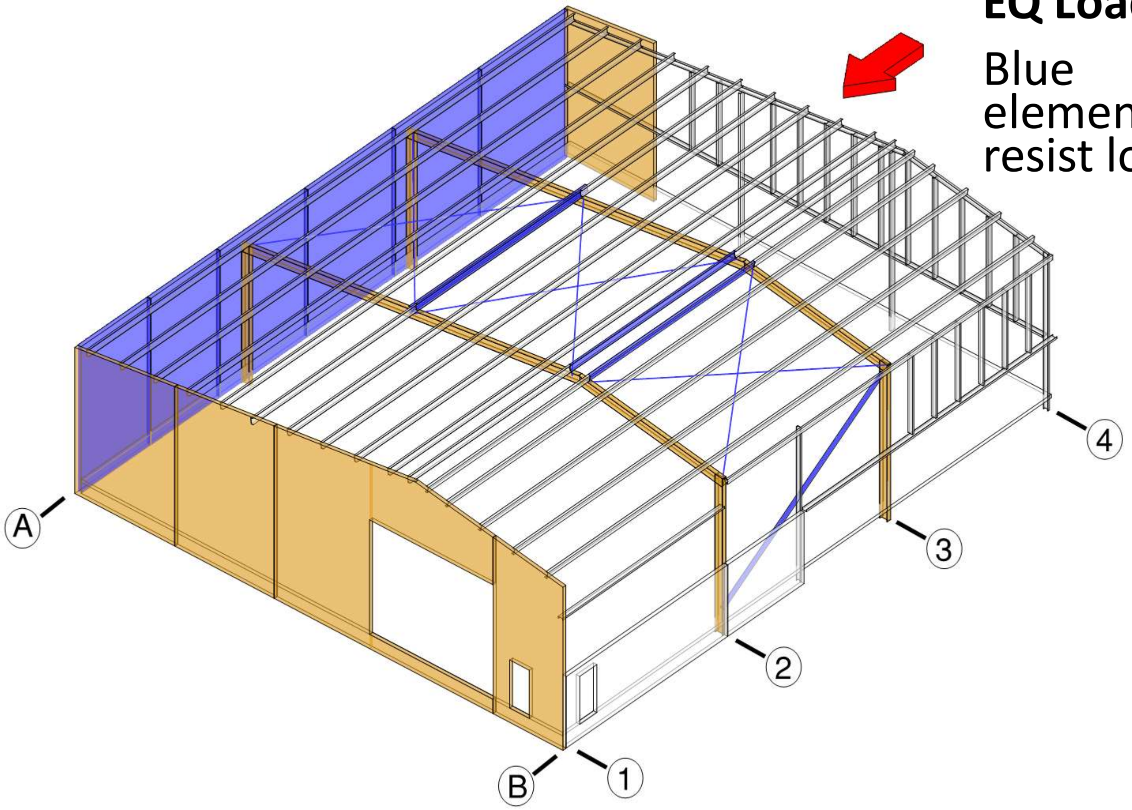
Transverse EQ Loads

Yellow elements resist loads



- Seismic loads resisted by the steel portal frames and the in-plane precast concrete panel end walls (shown yellow)

Longitudinal direction

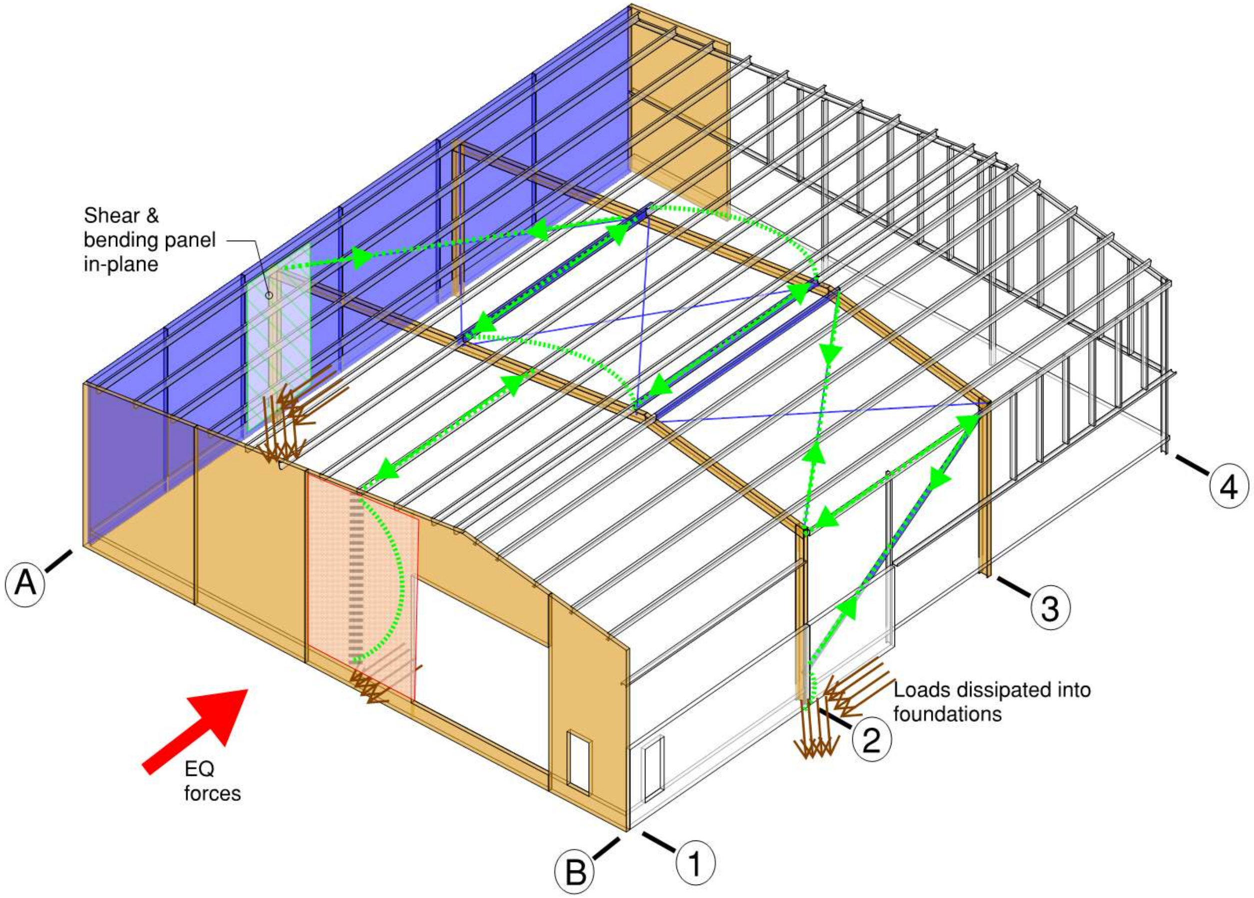


Longitudinal EQ Loads

Blue elements resist loads

- Loads resisted by in-plane walls Grid A, and tension/compression brace on Grid B (shown blue)
- Out-of-plane end walls supported at top by a collector, in turn supported by roof purlins acting as struts transferring loads back to roof plane cross bracing.
- Roof plane cross bracing transfers loads to the primary elements on Grid A and B.

Lets Follow a Load Path - Grid 1 panel out-of-plane



<u>Load Path</u>

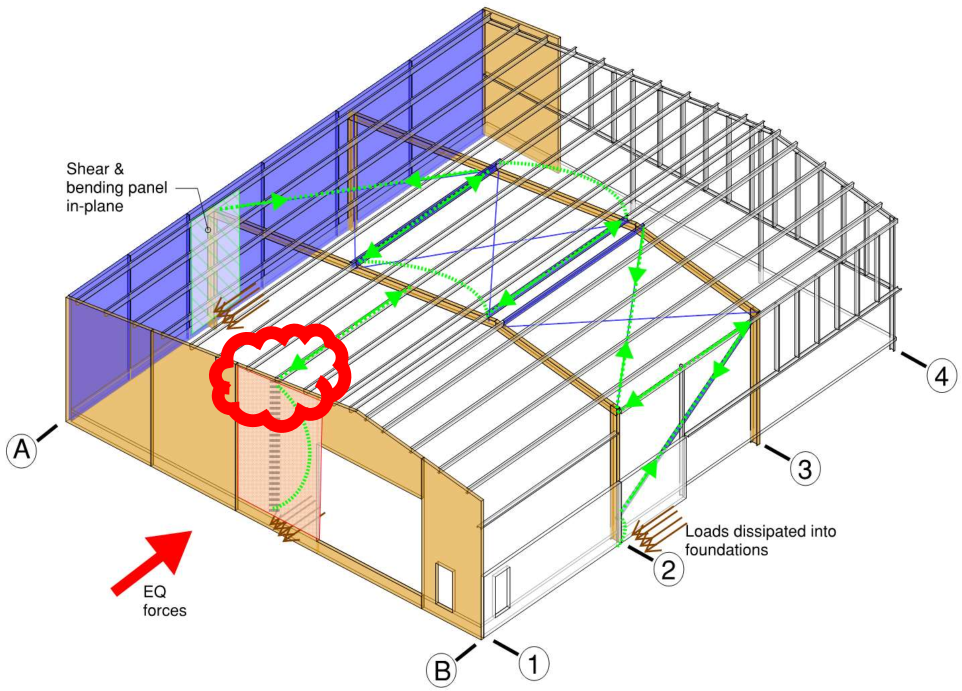
- Panel spans out-of-plane
- Propped at top by purlins
- Rafter in weak direction bending
- Roof plane bracing transfers loads to side elements
- Panels in plane, strut in tension

SESOC

A Reminder…

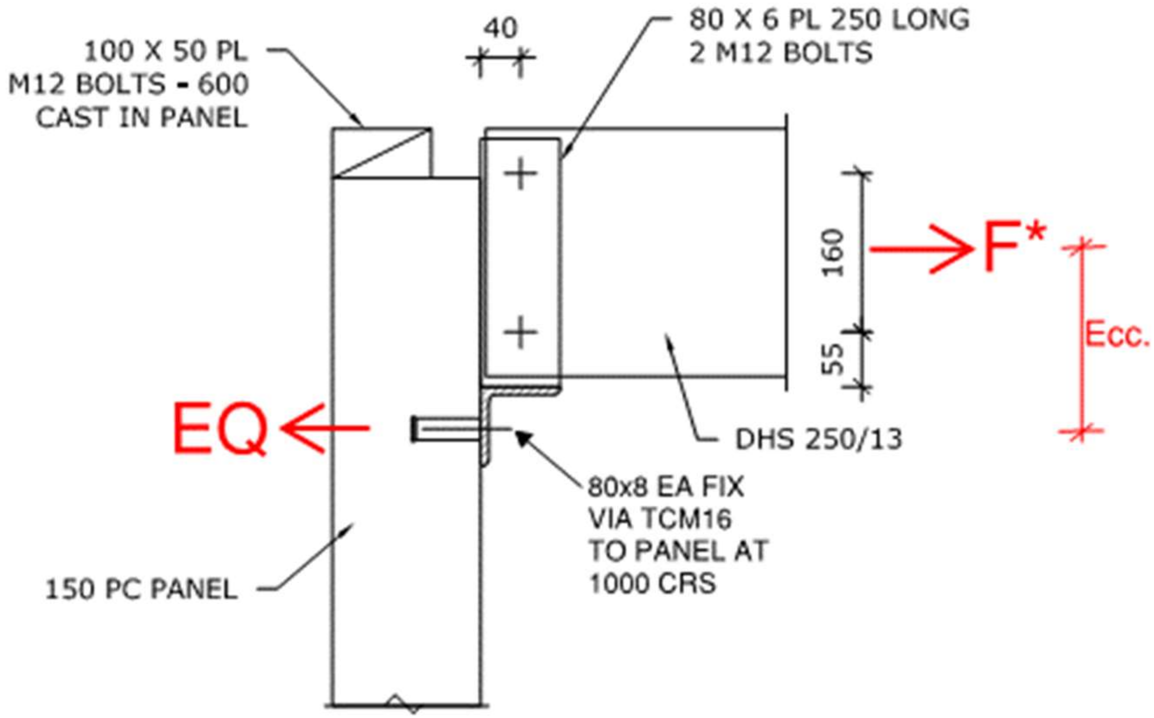
A chain is only as strong as its weakest link

Lets look at some connections…



Joint between panels and purlins

Panel out-of-plane to DHS



Load path just to get from panel into DHS Strut

Insert to EA

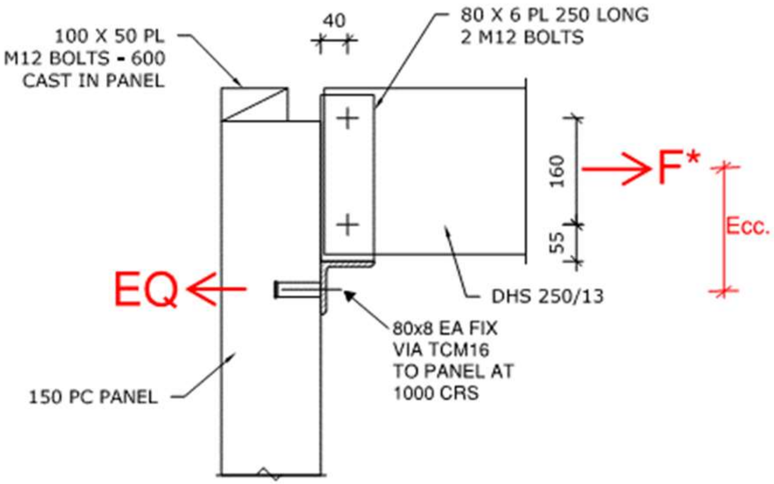
EA bending to location of purlin

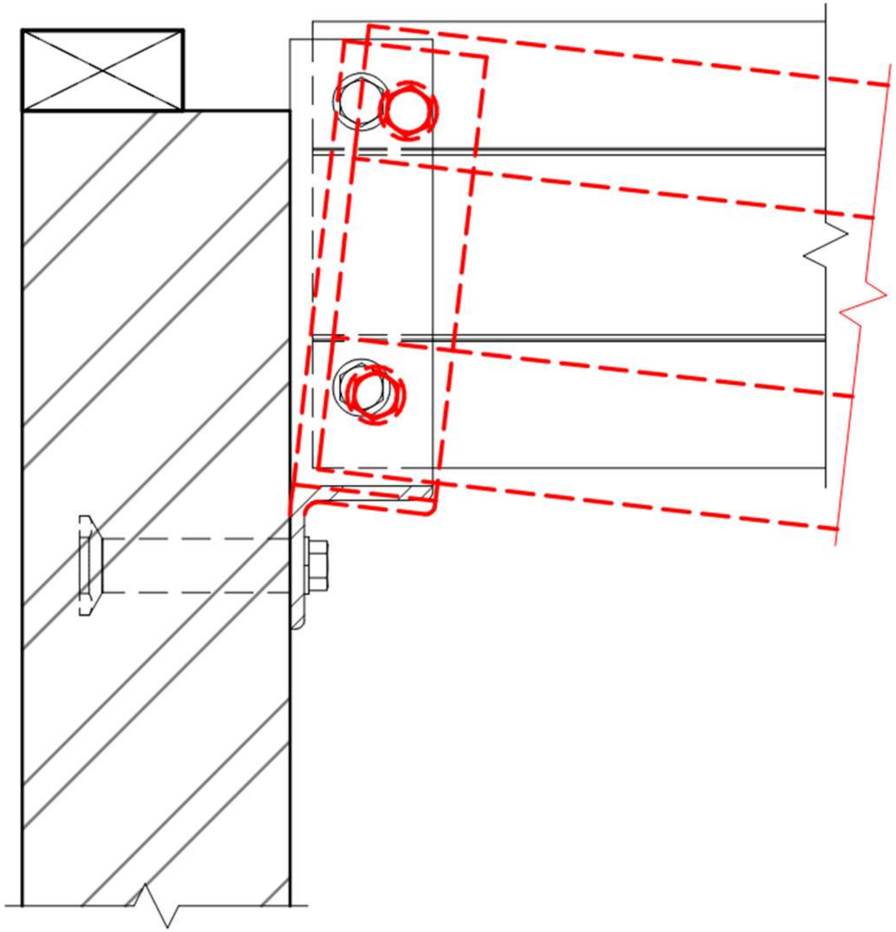
EA weld to cleat

Cleat bending to bolts

Bolts in shear to DHS purlin

Purlin to act as strut





Note the prying action on the bolts

What is the size of action F*?

- Assume purlin spacing 1.4m, panel 6.6 High
- Tributary weight: 1.4x6.6/2x0.15x24=16.6kN
- Z=0.42, Deep Soils, Ductility 1.25
- Sp=0.9, T=0.4, kmu=1.14

Global Cd(T) =3x.42x.9/1.14=0.994

F* =0.9945x16.6=16.4kN

- (Note - Parts Fph=0.42x1.12x2x2(Wp=16.6) x.85 =26.5kN)

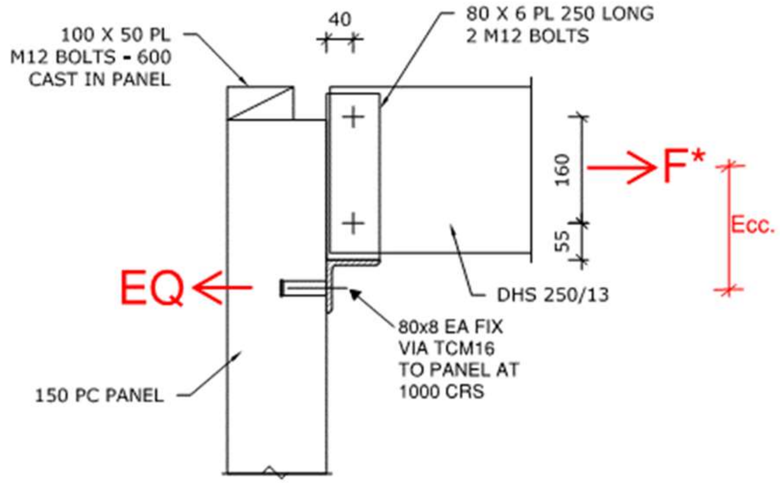
Joint moment due to eccentricity:

- 16.4x0.5(0.125+0.04+0.035/2)=1.4kNm

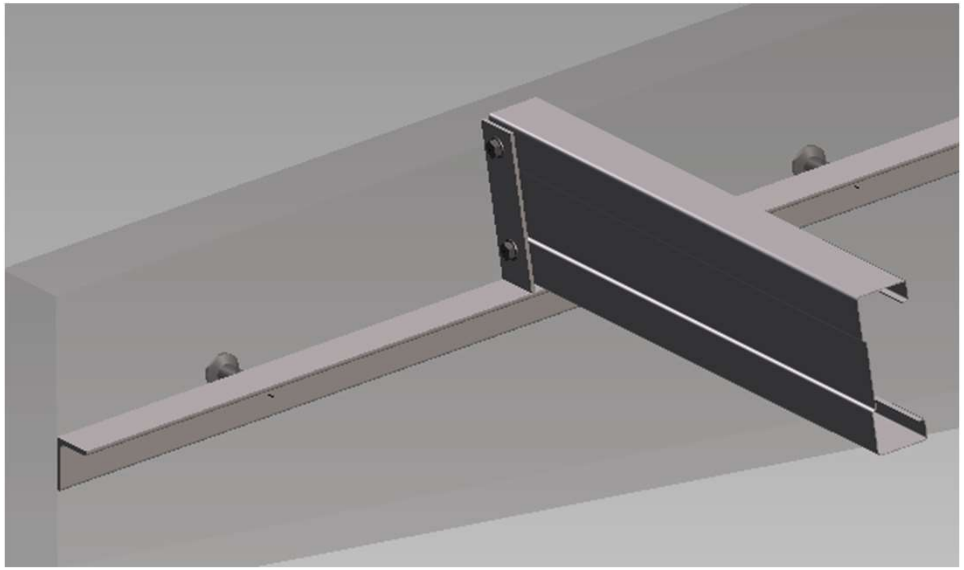
Purlin Bolt Load

- 16.4/2+ (Moment/0.160=8.8)=17kN
- In excess of capacity (From Dimond handbook for 12mm bolts and 1.25mm wall) = 14kN – note that this is worse if you consider Parts actions

Also consider the prying force on the bolts!



Panel out-of-plane to DHS

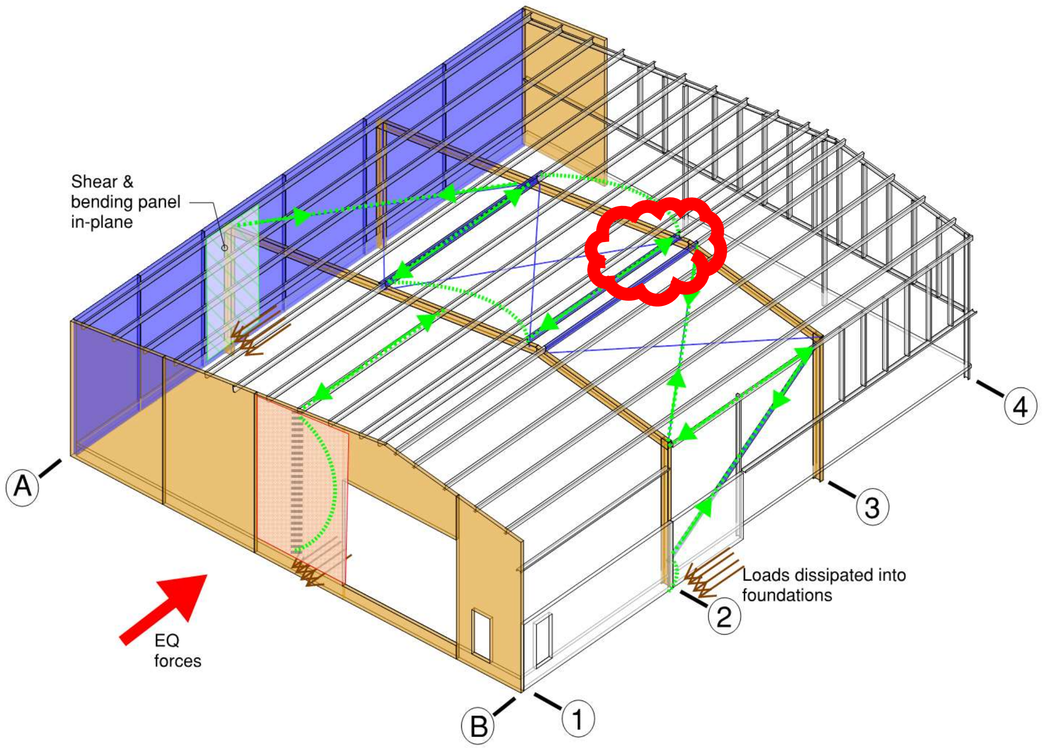


Why is this a poor connection?

- Highly eccentric
- Prying at inserts
- M12 bolts in thin walled DHS (usually with oversized holes)
- EA in torsion
- Shallow insert – cracked zone?

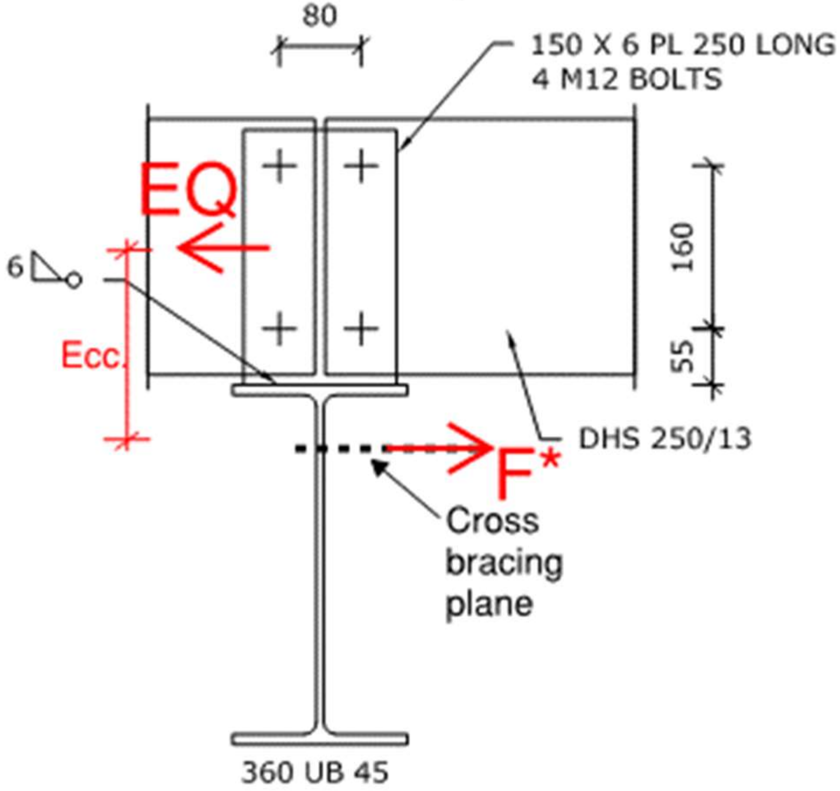
This connection cannot transfer the loads required

What about the next connection?



Joint between the purlins and the tension bracing

DHS to roof plane bracing



Load path just to get from DHS Strut to tension bracing

DHS purlin axial loads/strut

Bolts in shear to cleat

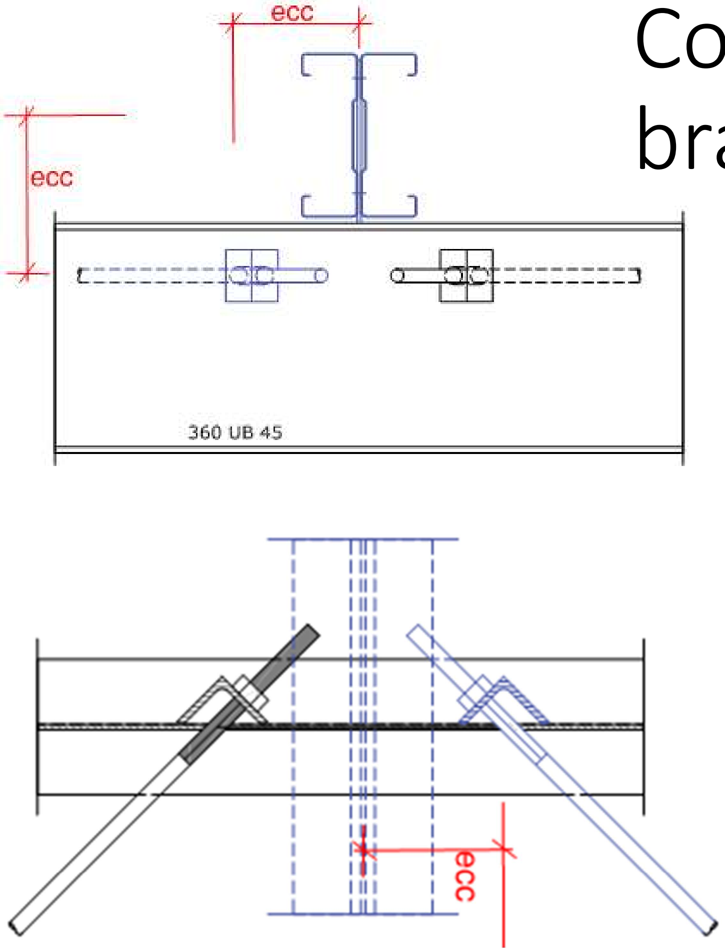
Cleat bending to weld

Weld to UB rafter

Rafter weak direction bending to brace cleats

Bracing cleats to tension braces

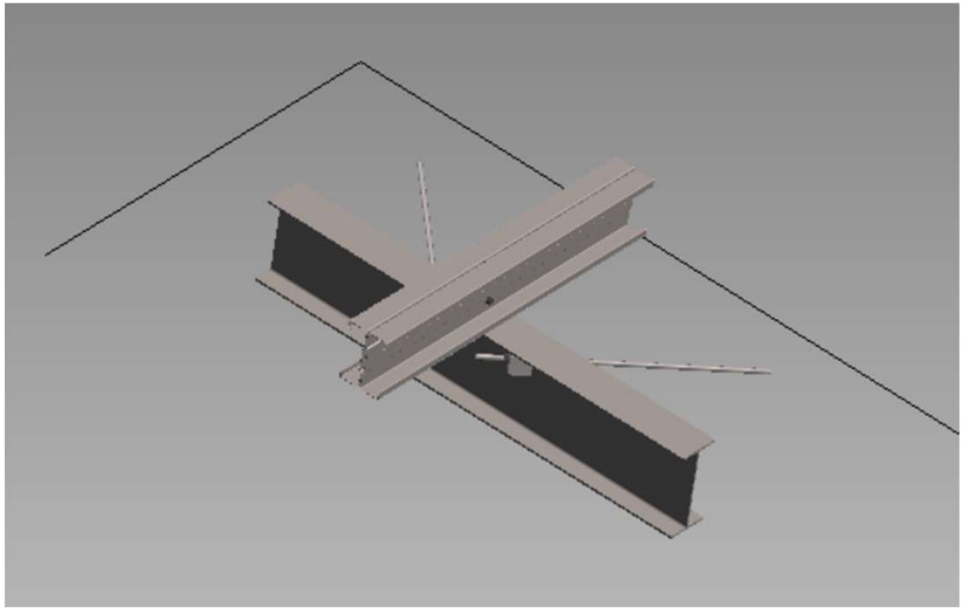
Connection detail – Roof Plane bracing to strut



Why is this a poor connection?

- Highly eccentric
- Relies on indirect web bending to transfer loads
- M12 bolts in thin walled DHS
- Bending of cleat

DHS to roof plane bracing

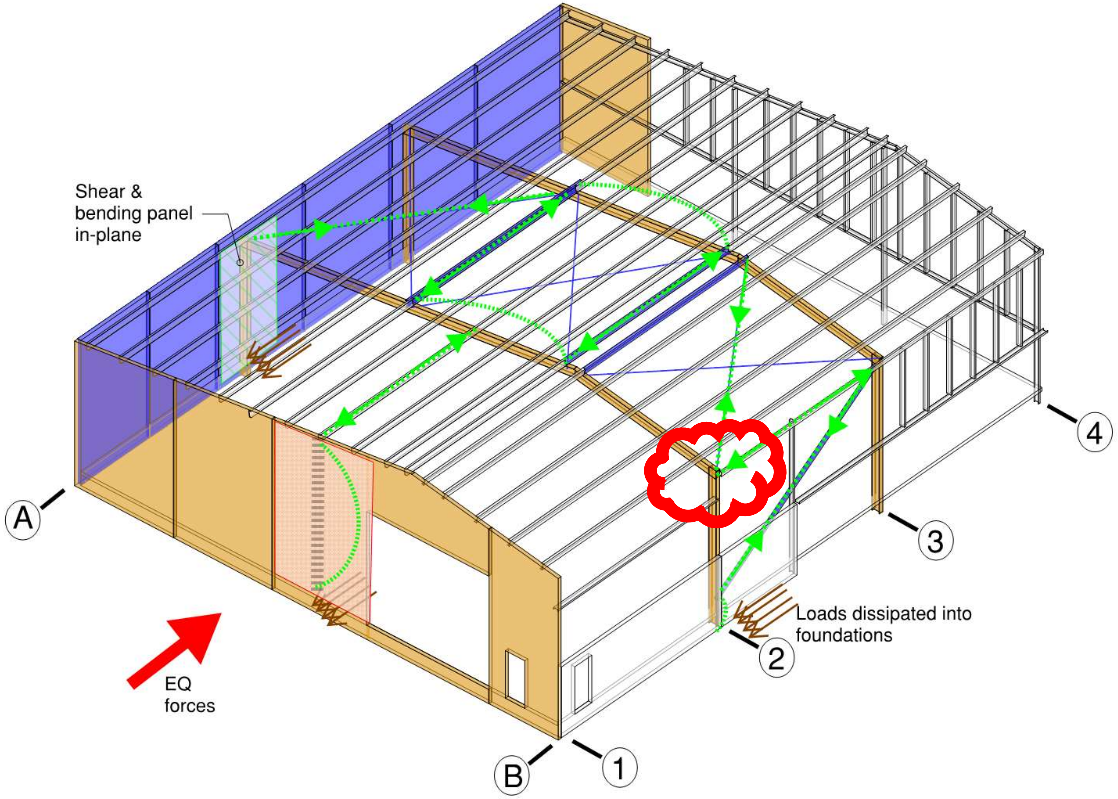


Why is this a poor connection?

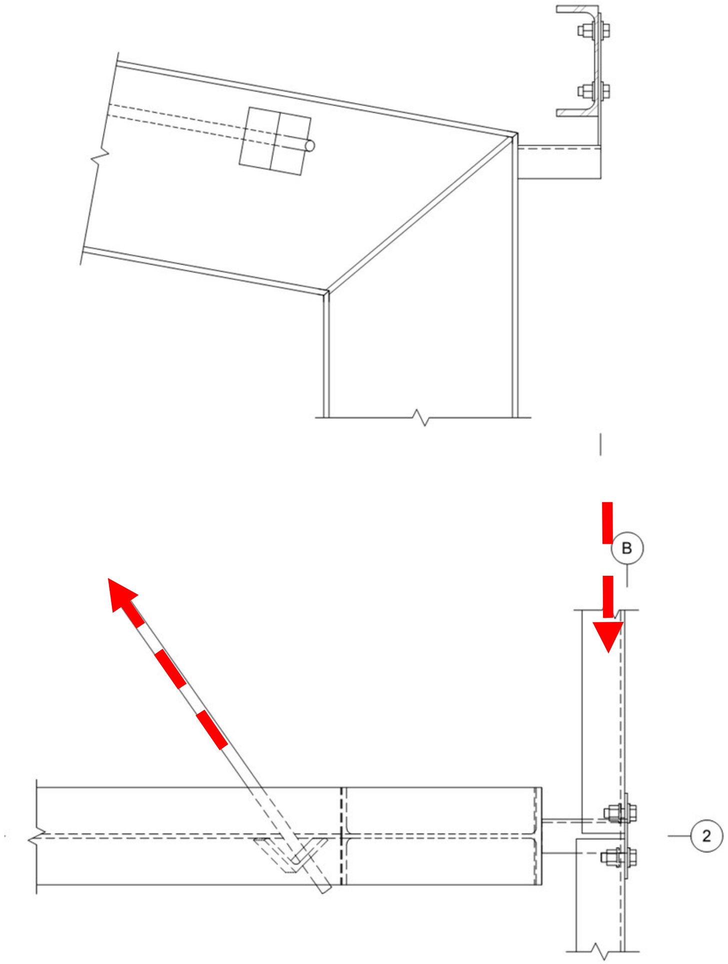
- Highly eccentric
- M12 bolts in thin walled DHS (usually with oversized holes)
- Relies on roof purlins to act as a strut
- Relies on indirect load path – weak direction bending of UB

This connection cannot transfer the loads required

Joint of Tension brace to strut



Tension brace to strut



Load path just to get from tension brace into PFC strut

Tension bracing bear against cleat

Indirect web bending to stiffener

Stiffener welds to column

Column bending in weak direction

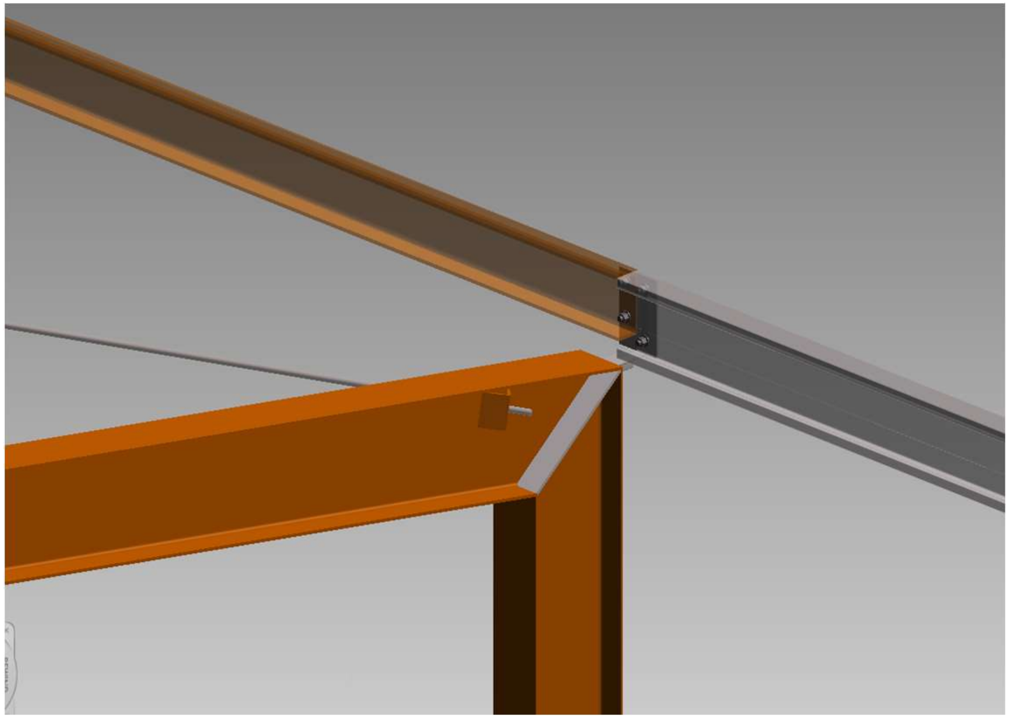
Weld to EA stub

EA stub in torsion to cleat

Cleat in bending to bolts

Bolts in shear to PFC

Tension brace to strut

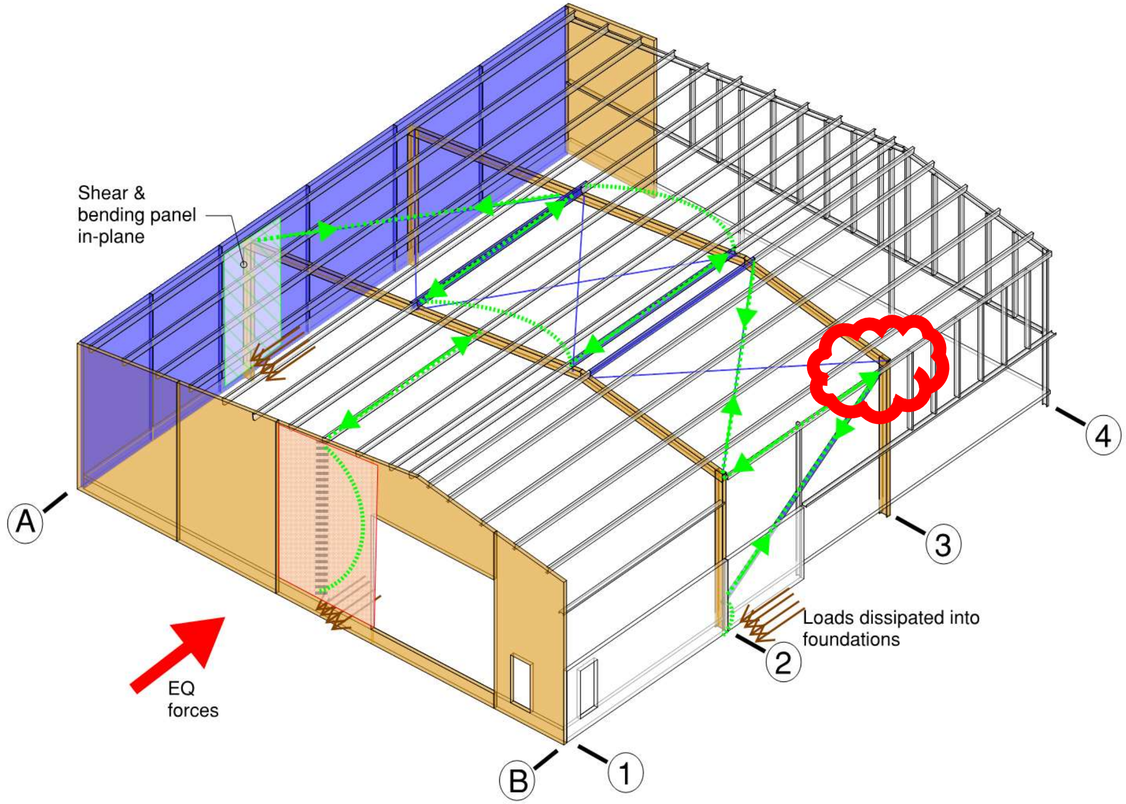


Why is this a poor connection?

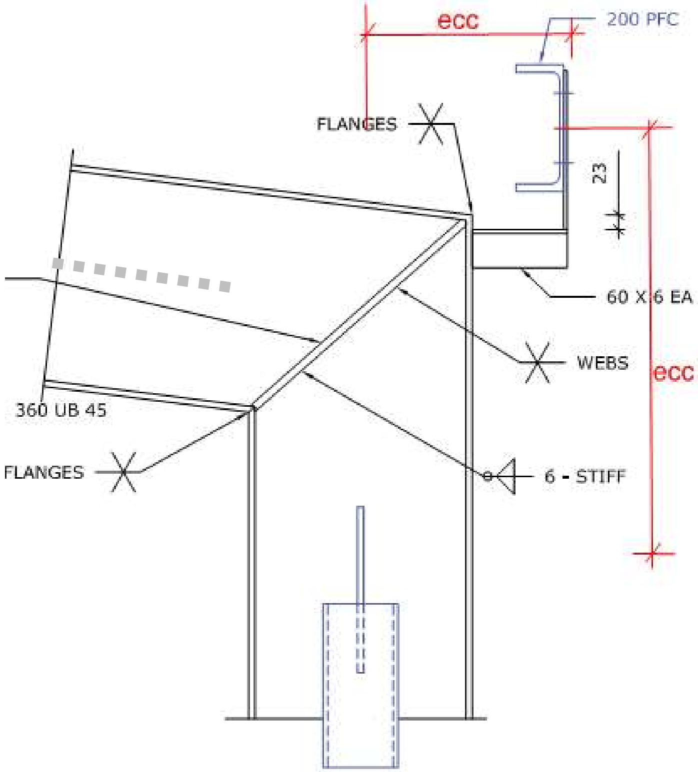
- Highly eccentric
- 2 bolts only to cleat
- EA in torsion
- Relies on indirect load path – weak direction bending of UB
- Not designed

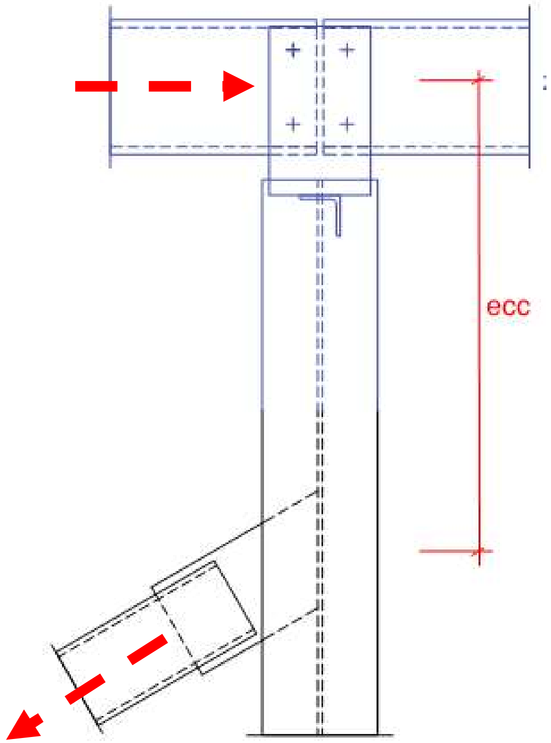
This connection was not designed and cannot transfer the loads required

Joint of Strut to Angled wall brace



Strut to wall brace





Load path just to get from Strut into angled wall strut

PFC bolted to vertical cleat

Cleat bending to EA stub

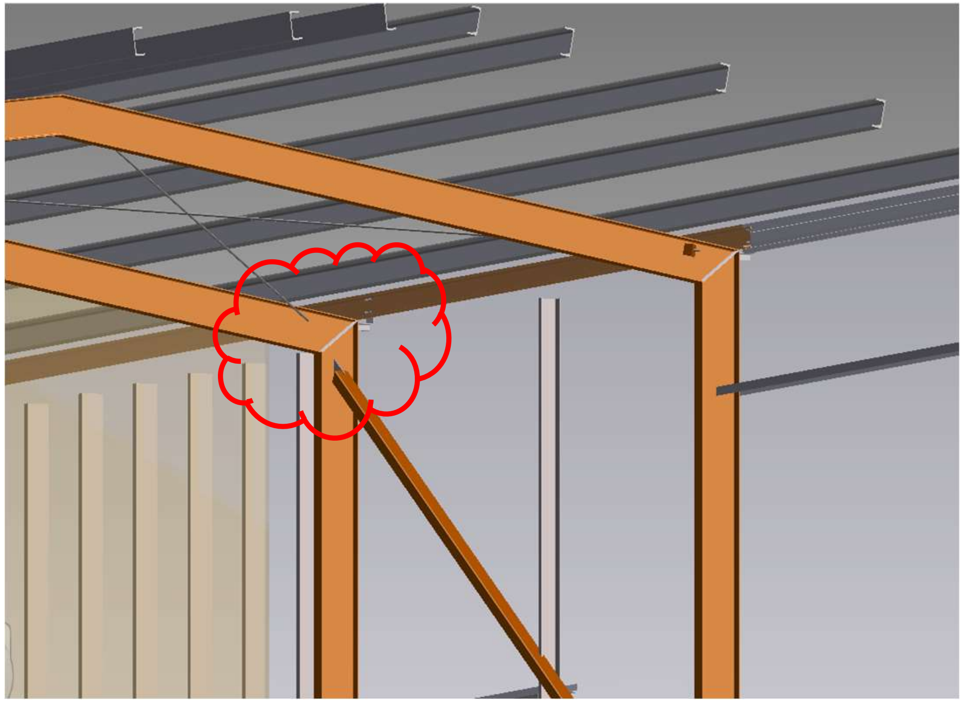
EA stub in torsion

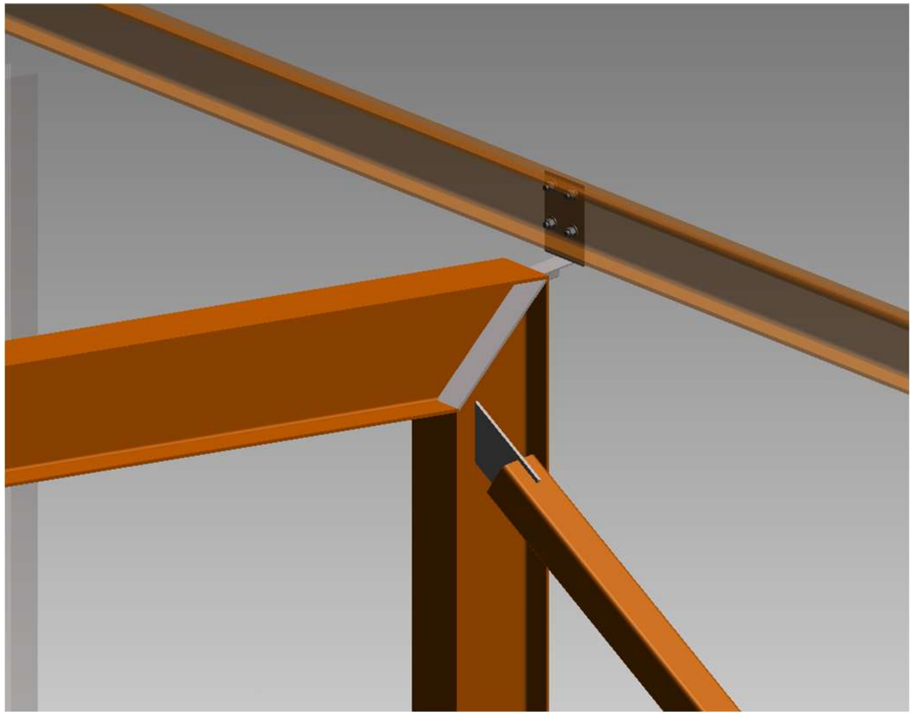
Weld to Rafter

Rafter in weak direction bending

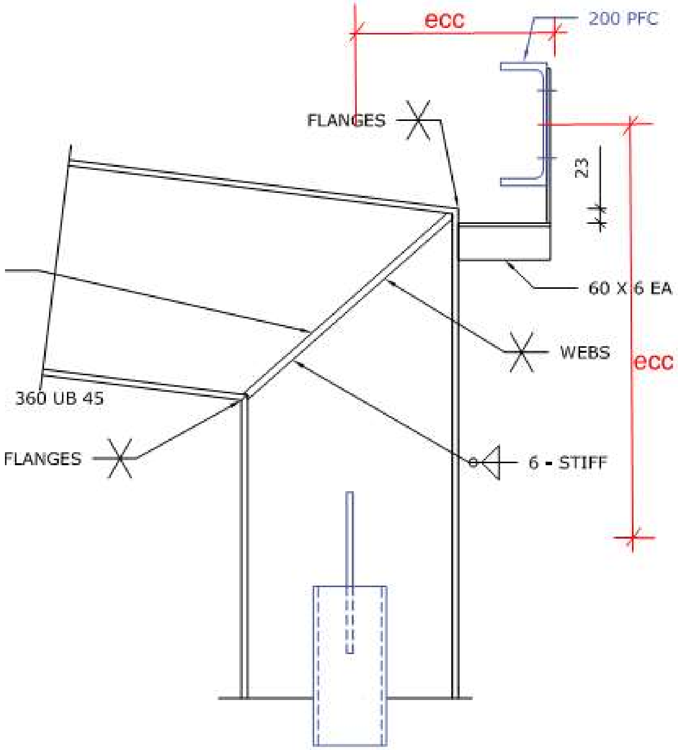
Cleat to angled wall strut

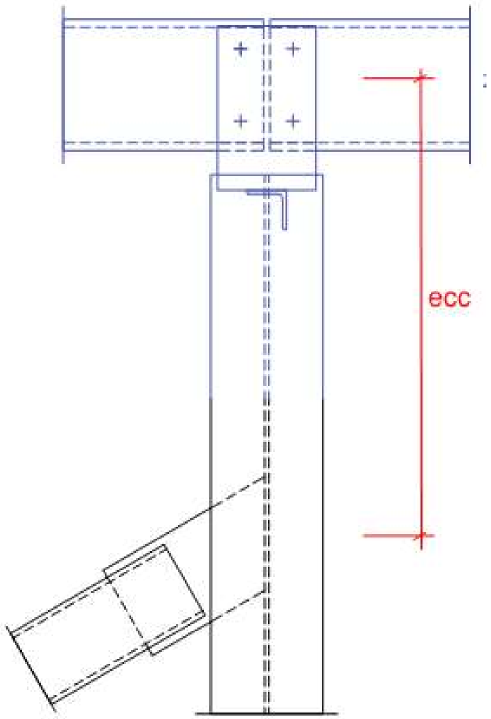
Strut to wall brace





Strut to wall brace



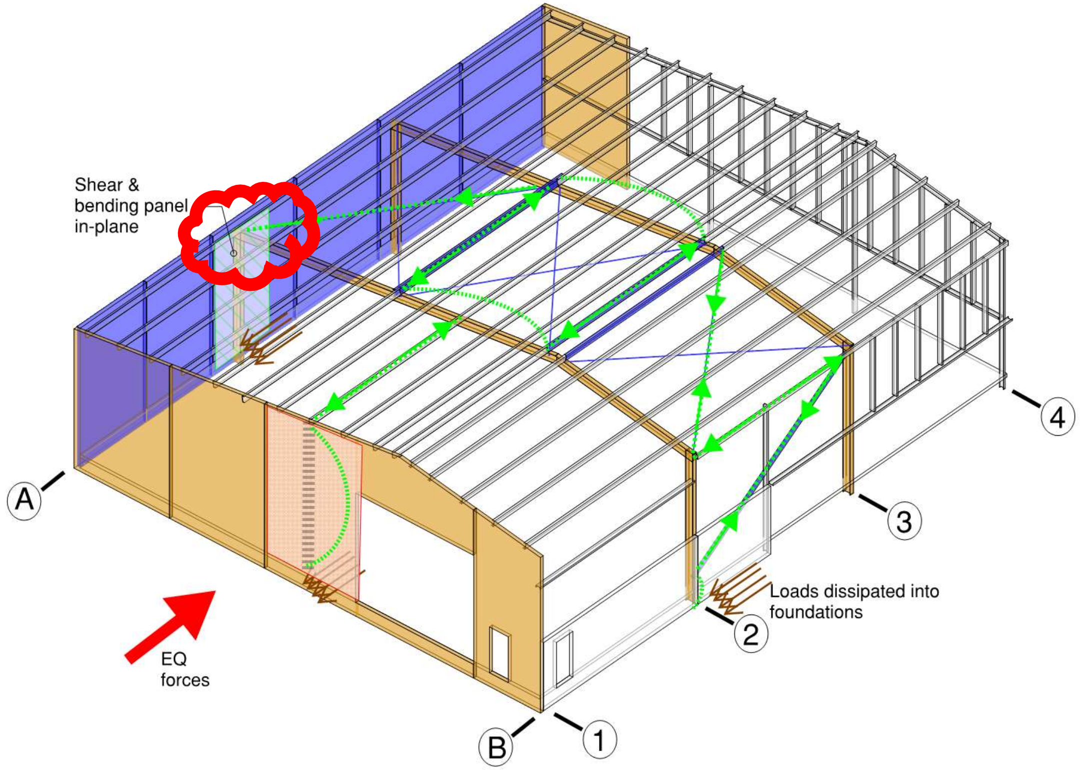


Why is this a poor connection?

- Highly eccentric
- 2 bolts only to cleat
- EA in torsion
- Relies on indirect load path – weak direction bending of UB
- Not designed

This connection was not designed and cannot transfer the loads required

Join of Tension brace to in-plane wall



Load path to get from tension brace into in-plane wall

Tension brace bears on cleat

Indirect web bending to angled stiffener weld

Angled stiffener weld to column

Column bending in weak direction

Column bear against single bolt in shear

- No details on the plans
- Generic detail for tension brace

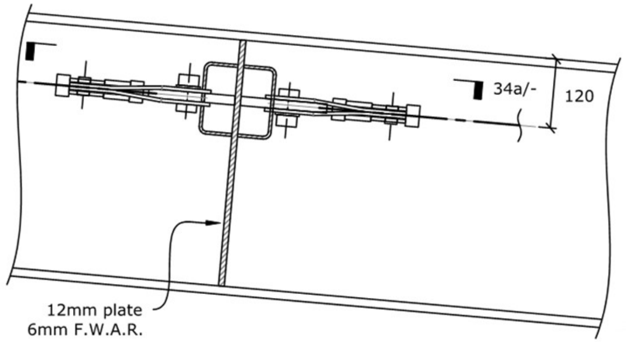
Weak links in the chain

- We have all these structural members joined with weak links

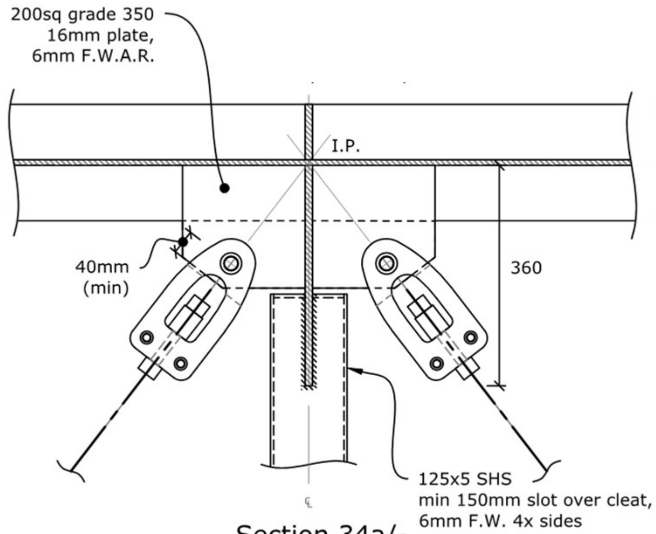
- Implications are that the connections will fail, potentially in a sudden brittle manner

What does good look like for Connections?

- Robustness, robustness, robustness
- Keep it concentric
- ‘Node’ all your joints, consider and nominate Intersection Points of all members – finish the triangle!
- Consider what happens if the EQ is a little bigger – perhaps design cleats for overstrength of your braces

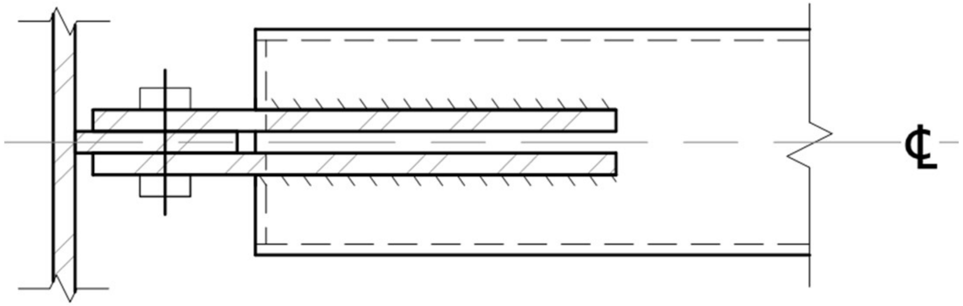


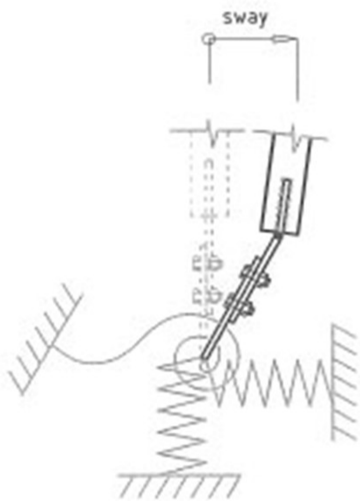
Section 34/S30
1:10



Section 34a/-
1:10

Keep your strut connection concentric

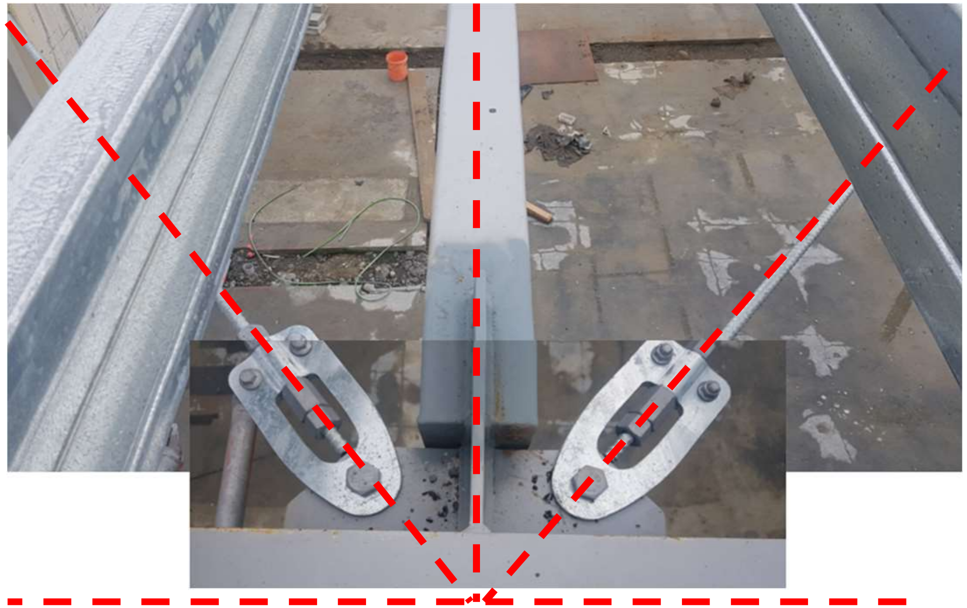




It is important for strut connection to have no eccentricities

Practice Advisory 12: Unstiffened eccentric cleat connections in compression

What does good look like for Connections?



Don't make life hard for yourself – avoid having to design your way out of eccentric connections

Keep it simple…

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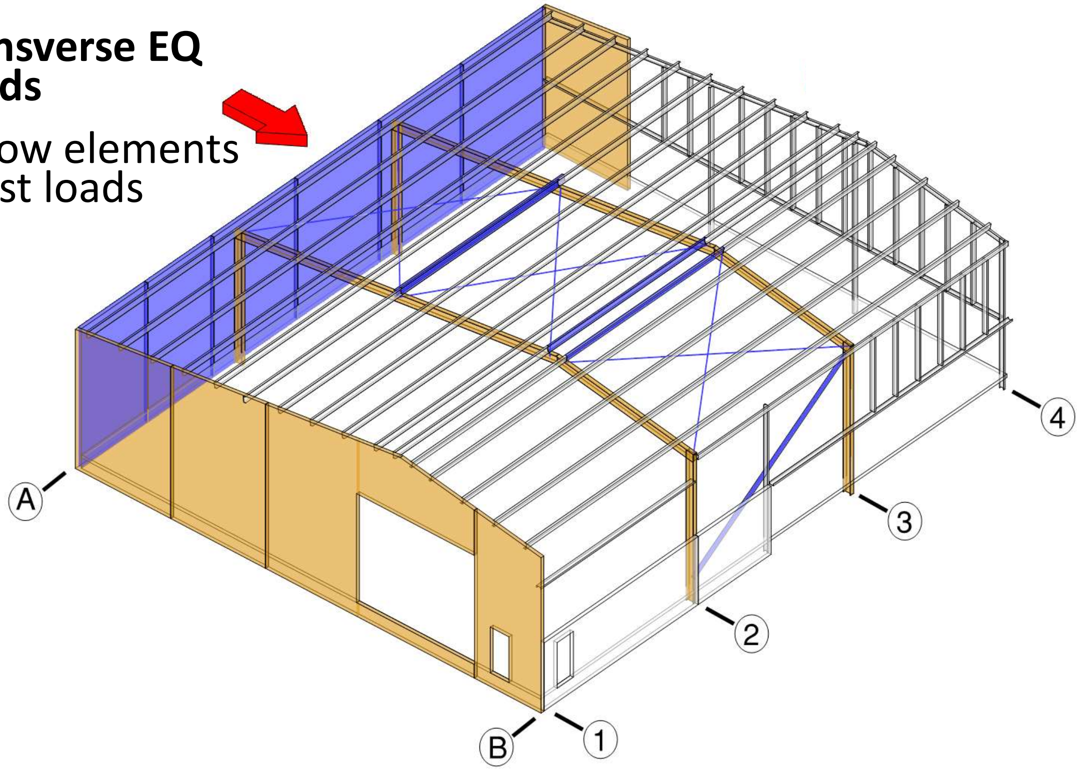
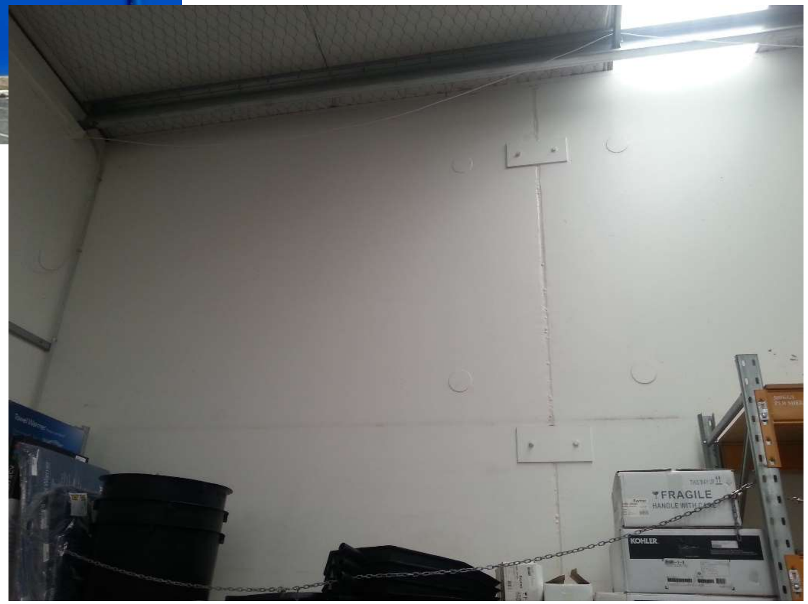
One more load path..

Transverse EQ Loads

Yellow elements resist loads



- Grid A panels out-of-plane
- No collector at eaves level
- Panels do not span horizontally
- Rely on cantilever base connection only



- Tip 2– Make sure you have a load path

Tip #3

Node your connections

Tip #4

1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4		

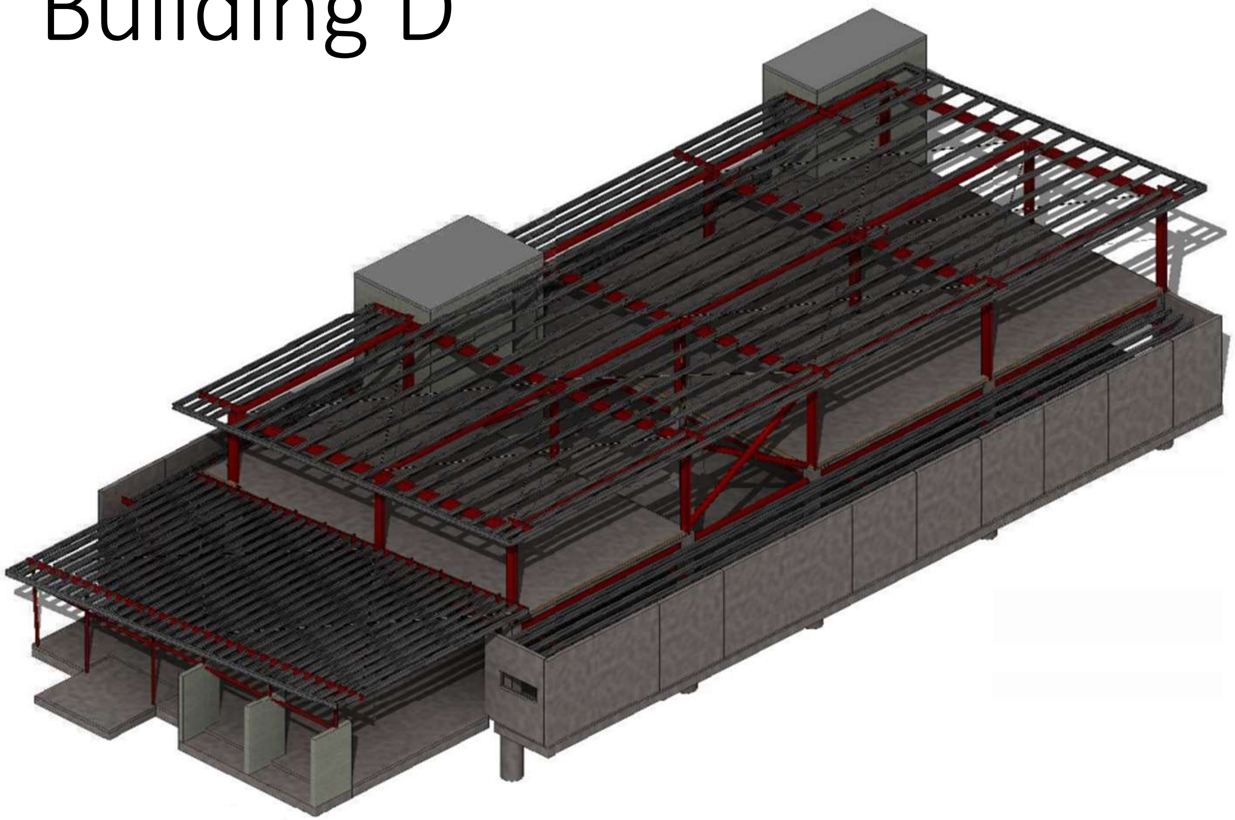
Tip #4

Connections are critical

Building D

- Built in 2012
- Floor area 1527m^2
- Single storey section at front
- Two storey at rear

Building D



- Designed as four separate structural systems
  1. Main 2 storey section
  2. Masonry block stair towers
  3. Front single storey section
  4. Ground floor cantilever precast concrete panels

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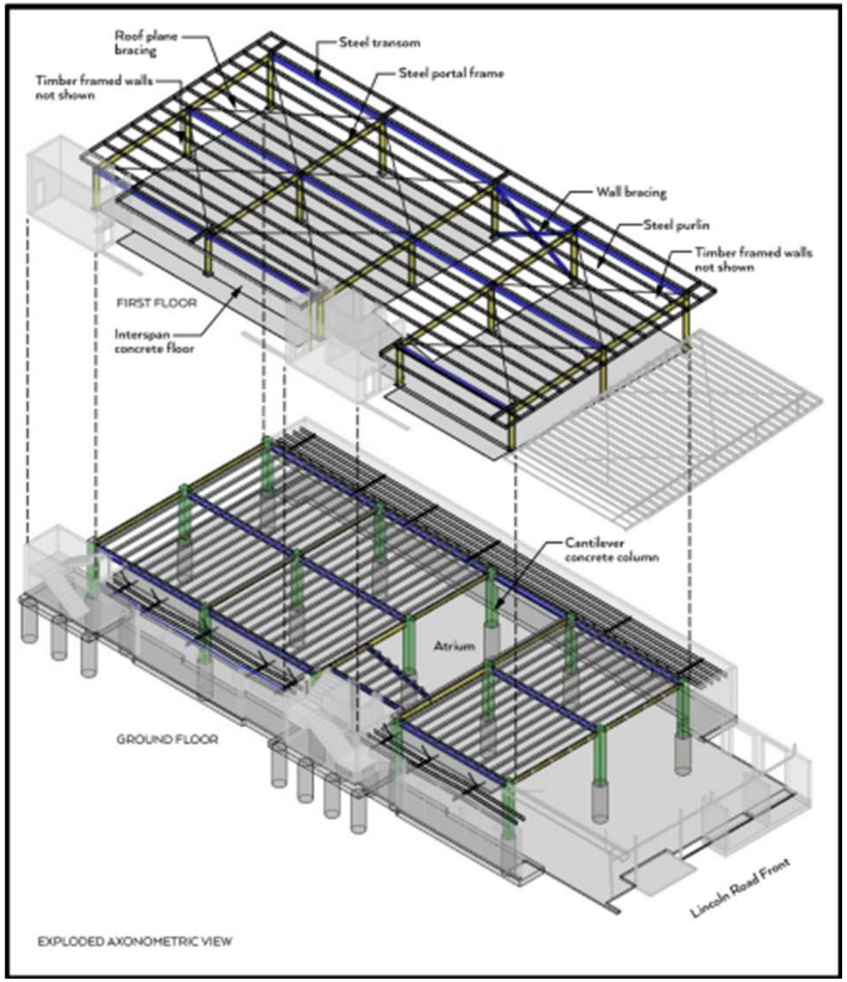


Main two storey section

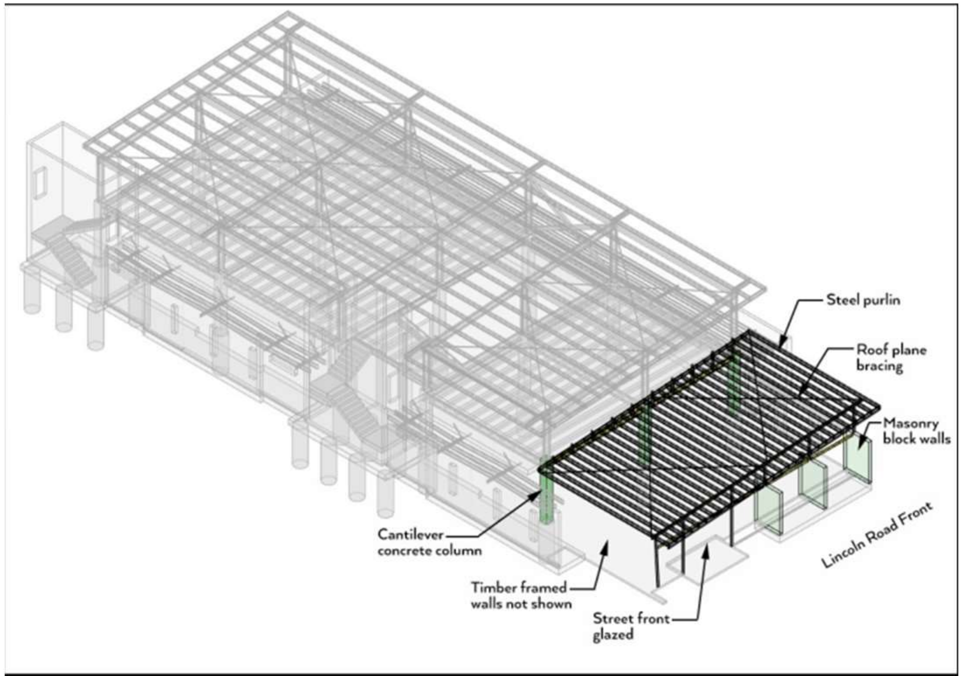
- Foundations and floor slab reinforced concrete
- Ground floor formed with cantilever concrete columns
- First floor steel portal frames with tension roof plane bracing and X brace in wall
- Suspended reinforced concrete Interspan first floor supported on steel beams spanning between cantilever concrete columns
- Timber framed infills forming first floor walls


GCC517
TG6680
DBA759
CGU88
DTR193


0800 4 SCAFIT
0800 4 SCAFIT
0800 4 SCAFIT
0800 4 SCAFIT
DANGER
CONSTRUCTION SITE

Main two storey section

Front Single Storey Section



- Formed with DHS purlins supported on a combination of cantilever masonry block walls, structural steel and timber framing

Front Single Storey Section

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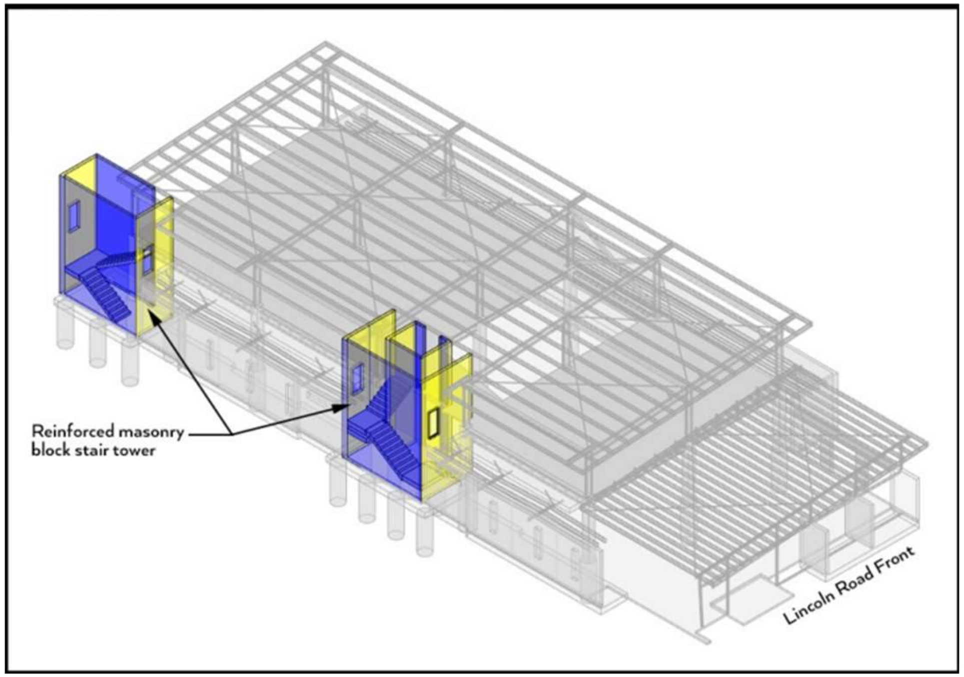
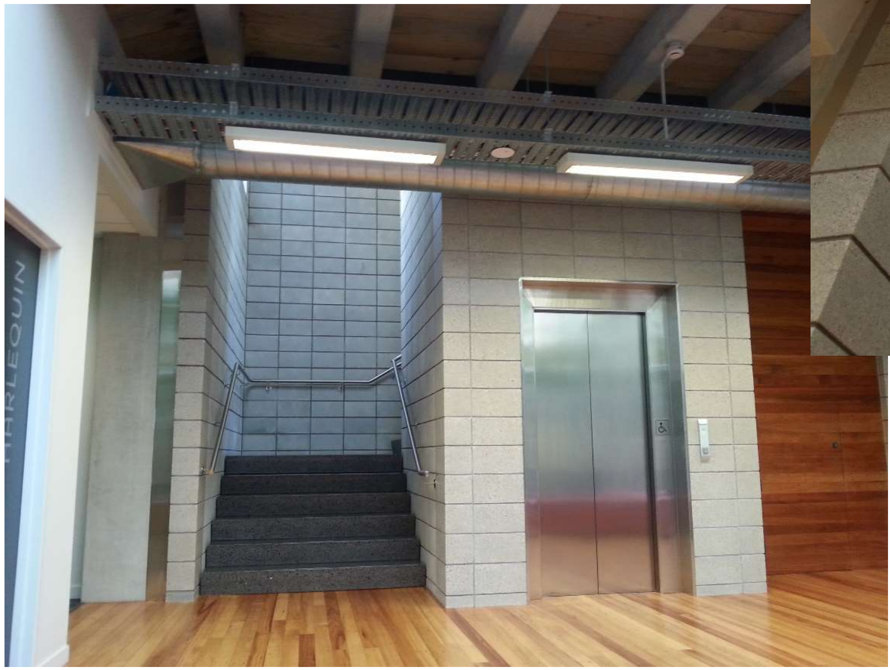
Front Single Storey Section

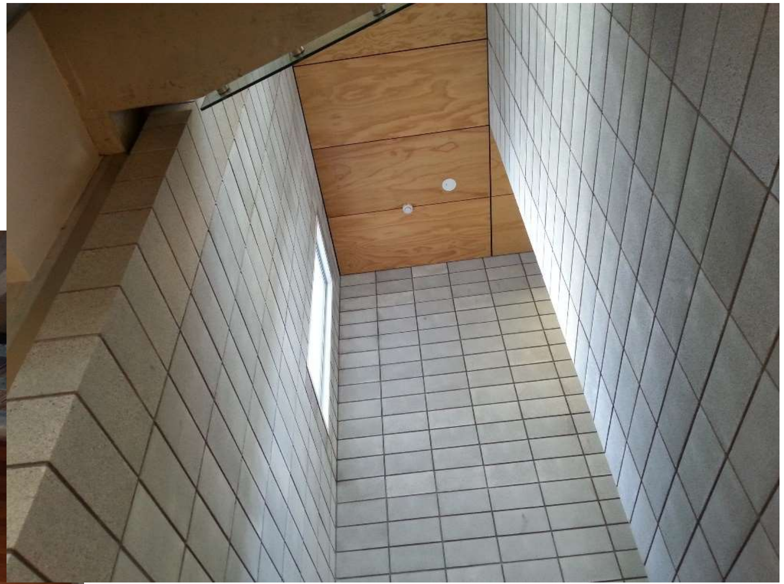
Stair Towers

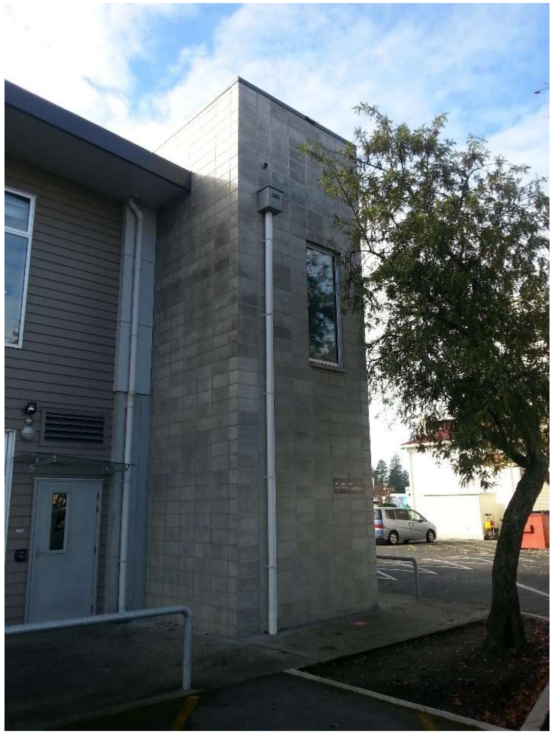


- 200 series reinforced masonry block stair towers

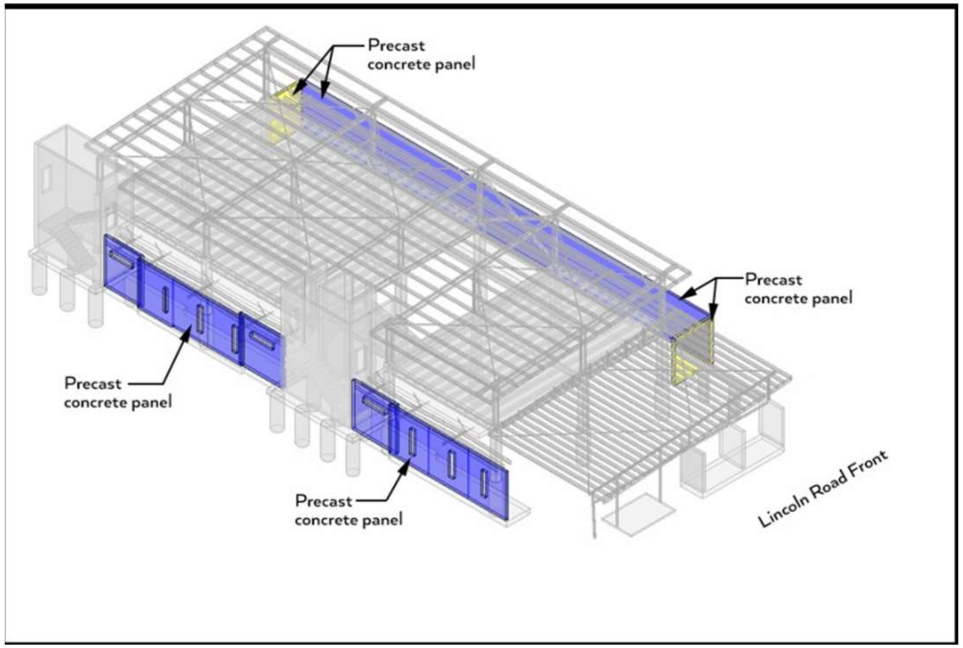
Stair Towers







Cantilever Precast Concrete Panels



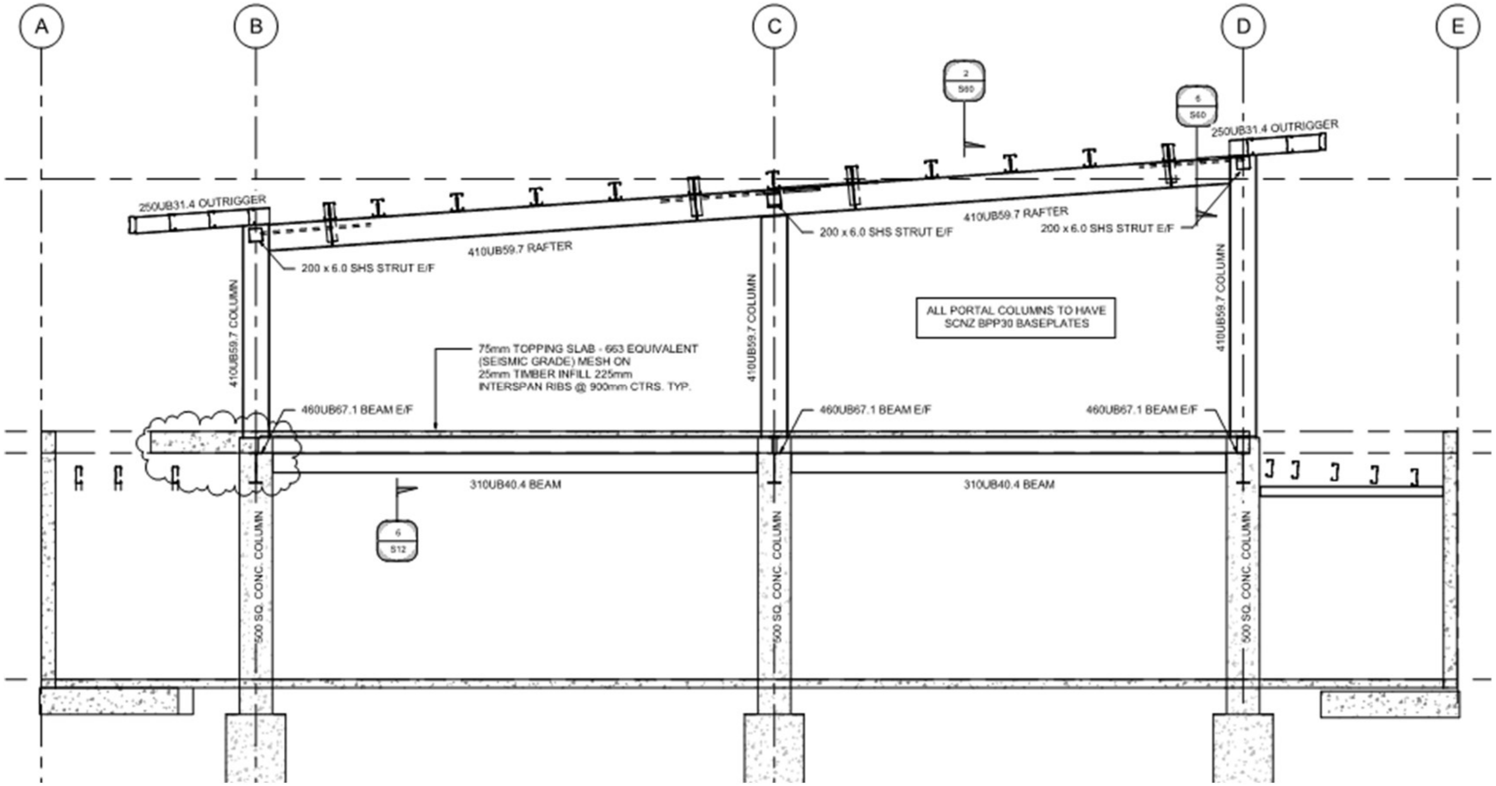
- Cantilever reinforced concrete precast panels
- Drossbach connection to footing

Cantilever Precast Concrete Panels

Transverse Connections

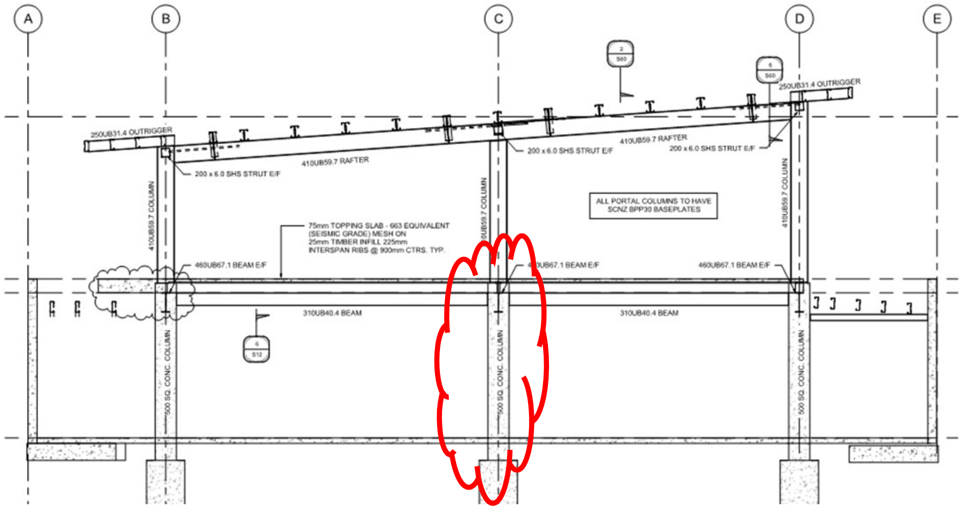
- Lets look at some of the more critical connections for the load path in the transverse direction

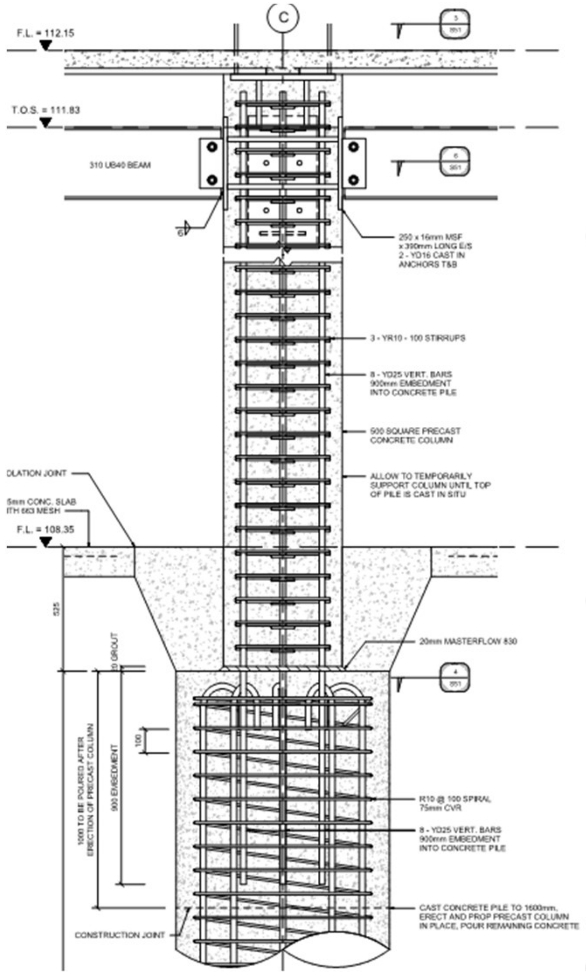
First floor – steel portal frames



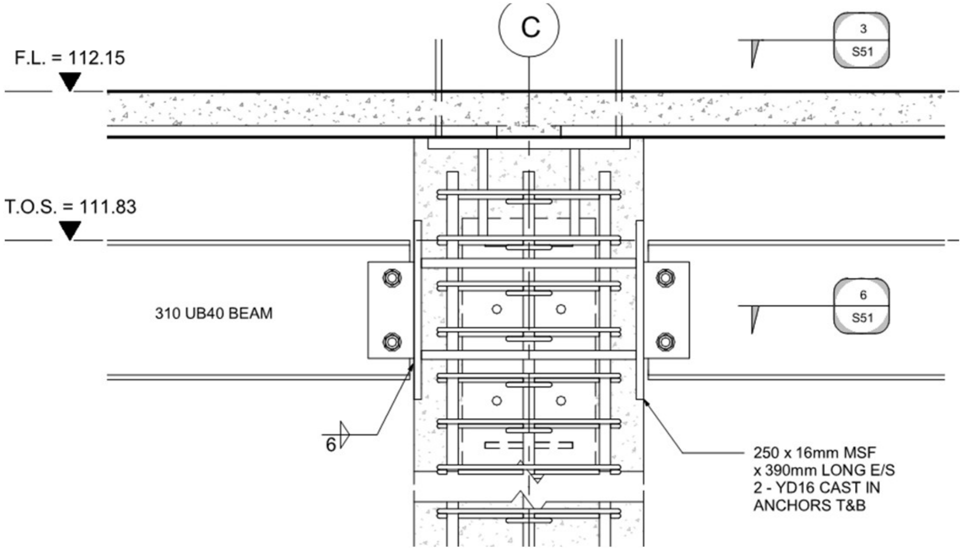
- Steel portal frames, with columns supported by cantilever concrete columns

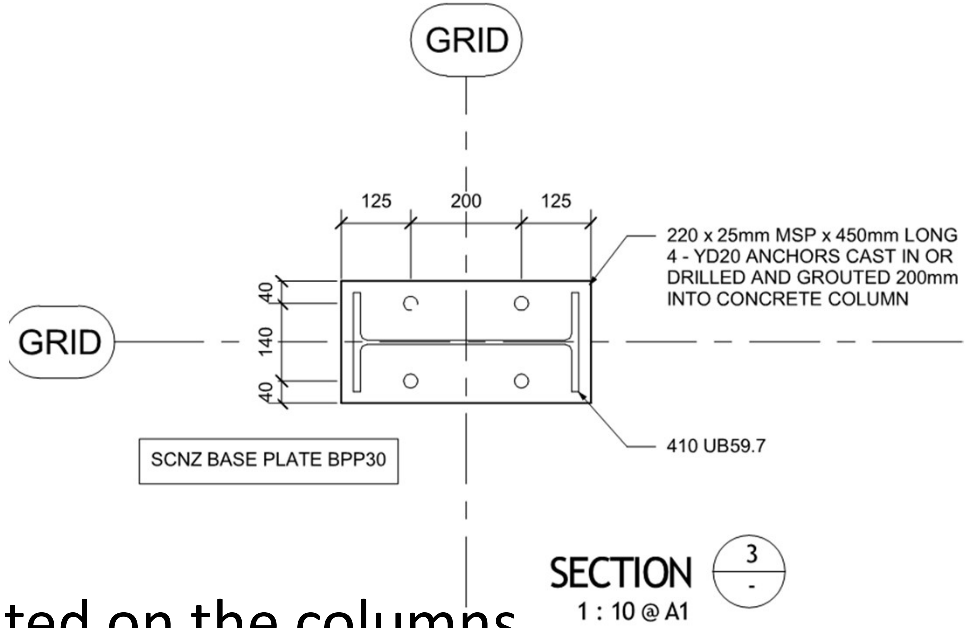
Concrete column to portal frame





Portal frame connection to RC Columns





- Steel portal frame has a base plate seated on the columns
- 4-YD20 anchors drilled & grouted 200mm into top of precast column

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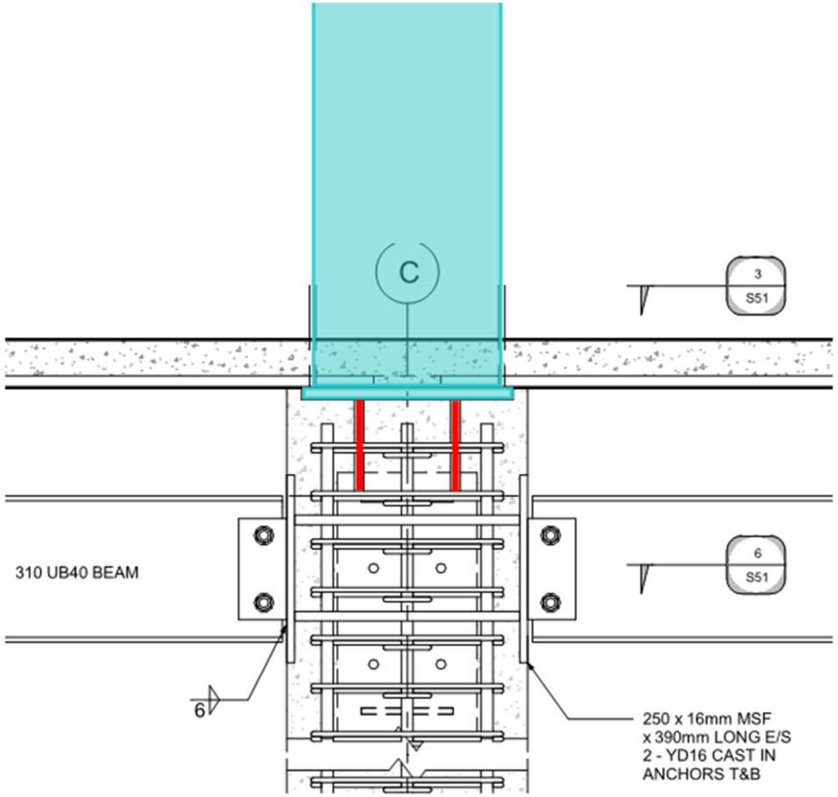
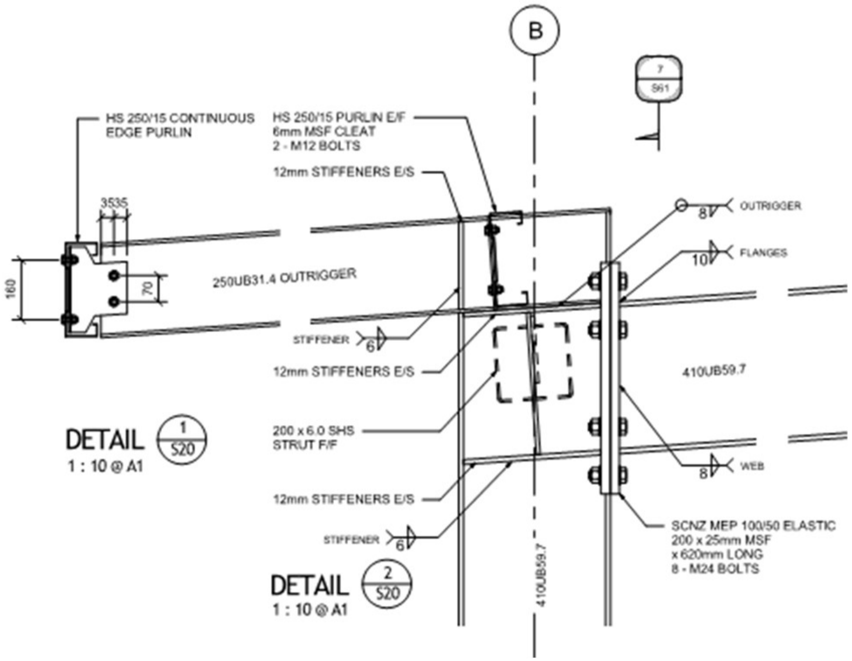
Portal frame connection to PC columns



- Shallow embedment (only 200mm!
- Insufficient development to column reinforcing
- 'Grouted' into PC column

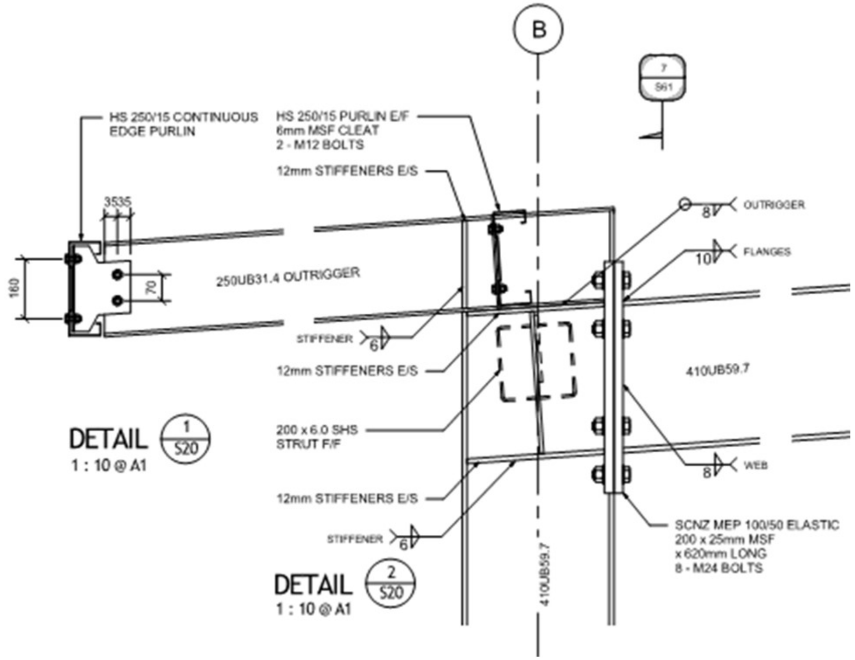
This connection was not designed and cannot transfer the loads required

What about the Portal Frame Knee Joint?



- This is a critical joint for steel portal frames
- Constantly seeing this poorly detailed
- SCNZ has guidance and connections guide which we can refer to

Portal Frame Knee Joint?



This connection cannot transfer the loads required

- Flange weld is 10mm fillet weld (not enough for tension force from a 410UB60 flange)
- Bolts are 8-M24 (not specified as TB)
- 12mm Grade 250MPa stiffeners specified (not enough for tension force from a 12.8mm Grade 320MPa flange)

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TABLE 3.1-3(B)

UNIVERSAL BEAMS
GRADE 300
PROPERTIES FOR ASSESSING SECTION CAPACITY TO AS 4100

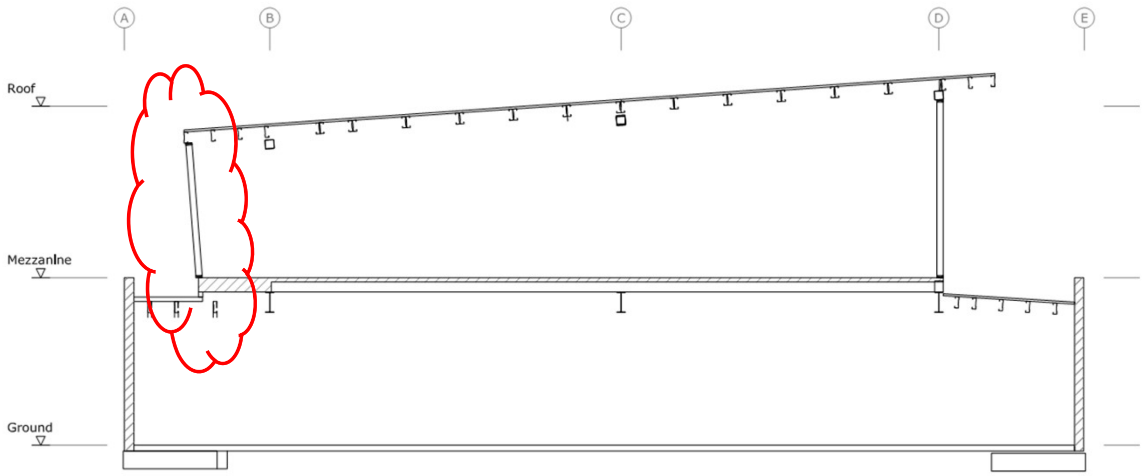
Designation	Yield Stress Flange f_{yf}	Yield Stress Web f_{yw}	Form Factor k_f	About x-axis Compactness (C, N, S)	About x-axis Z_{ex}	About y-axis Compactness (C, N, S)	About y-axis Z_{ey}
	MPa	MPa			10^3mm^3		10^3mm^3
610UB125	280	300	0.950	C	3680	C	515
113	280	300	0.926	C	3290	C	451
101	300	320	0.888	C	2900	C	386
530UB 92.4	300	320	0.928	C	2370	C	342
82.0	300	320	0.902	C	2070	C	289
460UB 82.1	300	320	0.979	C	1840	C	292
74.6	300	320	0.948	C	1660	C	262
67.1	300	320	0.922	C	1480	C	230
410UB 59.7	300	320	0.938	C	1200	C	203
53.7	320	320	0.913	C	1060	C	173
360UB 56.7	300	320	0.996	C	1010	C	193
50.7	300	320	0.963	C	897	C	168
44.7	320	320	0.930	N	770	N	140
310UB 46.2	300	320	0.991	C	729	C	163
40.4	320	320	0.952	C	633	C	139
32.0	320	320	0.915	N	467	N	86.9
250UB 37.3	320	320	1.00	C	486	C	116
31.4	320	320	1.00	N	395	N	91.4
25.7	320	320	0.949	C	319	C	61.7
200UB 29.8	320	320	1.00	C	316	C	86.3
25.4	320	320	1.00	N	259	N	68.8
22.3	320	320	1.00	N	227	N	60.3
18.2	320	320	0.990	C	180	C	34.4
180UB 22.2	320	320	1.00	C	195	C	40.7
18.1	320	320	1.00	C	157	C	32.5
16.1	320	320	1.00	C	138	C	28.4
150UB 18.0	320	320	1.00	C	135	C	26.9
14.0	320	320	1.00	C	102	C	19.8

Notes: (1) For Grade 300 sections the tensile strength (f_u) is 440 MPa.
(2) C= Compact Section; N= Non-compact Section; S= Slender Section.
(3) All references to Grade 300 refer to the OneSteel specification of 300PLUS™ Steel or AS/NZS 3679.1 Grade 300.

UB properties

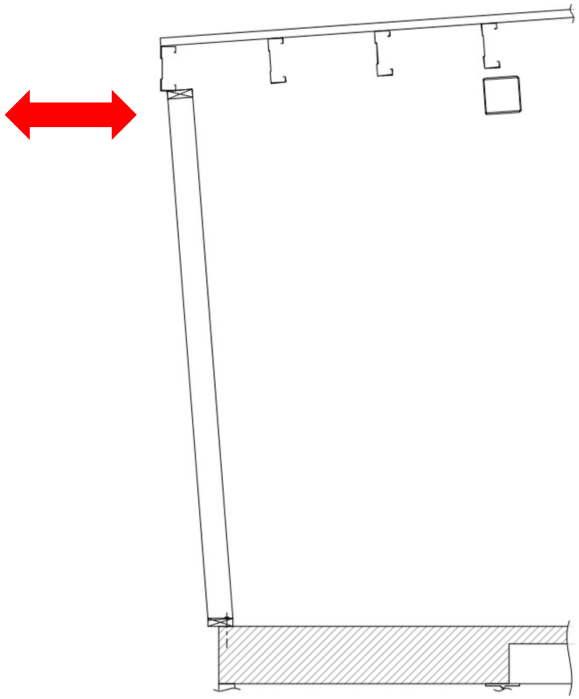
- UB flange yield stresses are provided – can vary from 280MPa to 320MPa
- Keep these in mind when designing joints!

What about the side walls?



- First floor timber framed side walls
- Along one side, the wall rakes, and is offset from the steel framing
- There is suspended ceiling

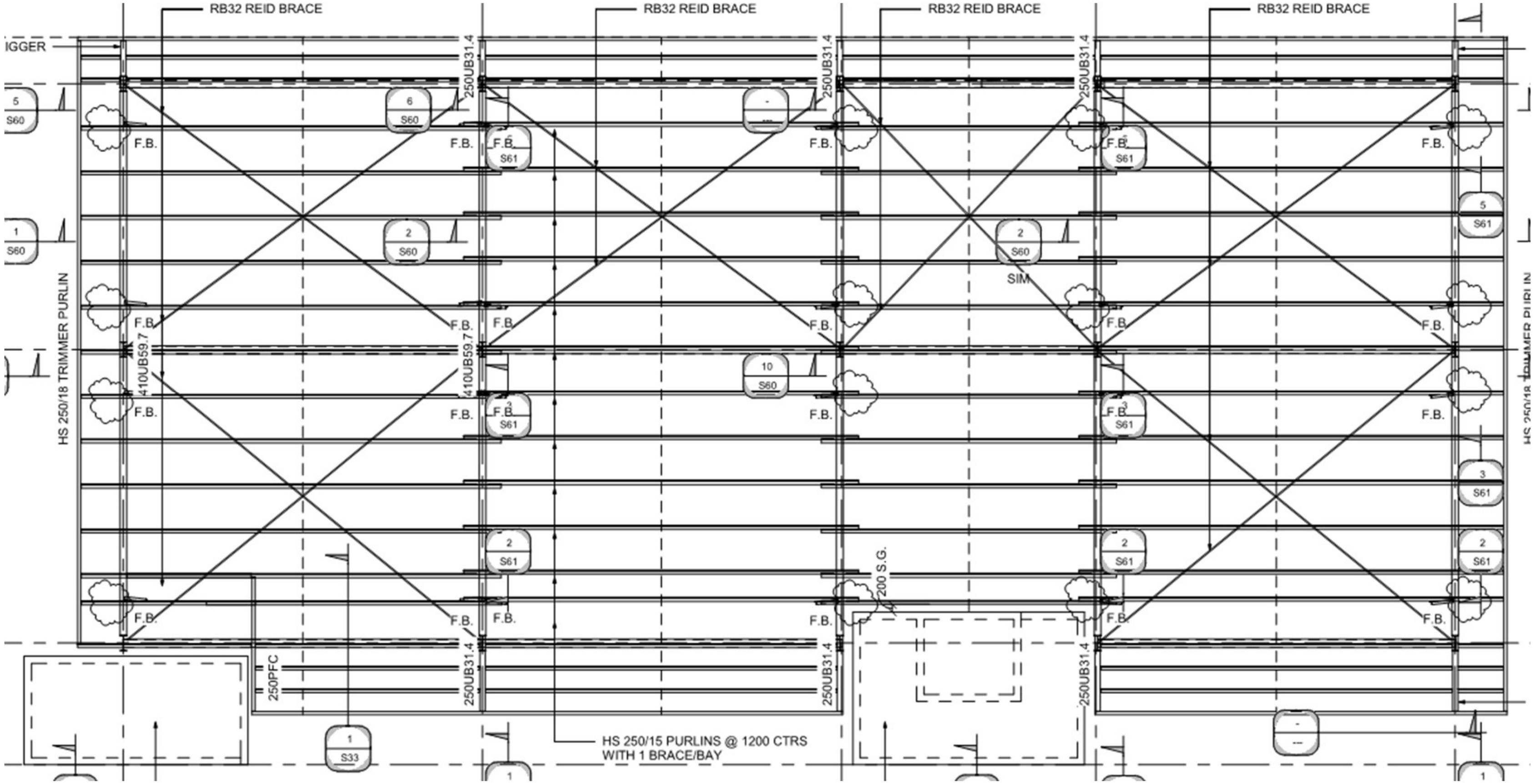
Side walls



- There is no out-of-plane support for this wall framing
- Some indirect load path through the box gutter on this line & weak direction bending of the DHS roof purlin – this is not a direct engineered load path

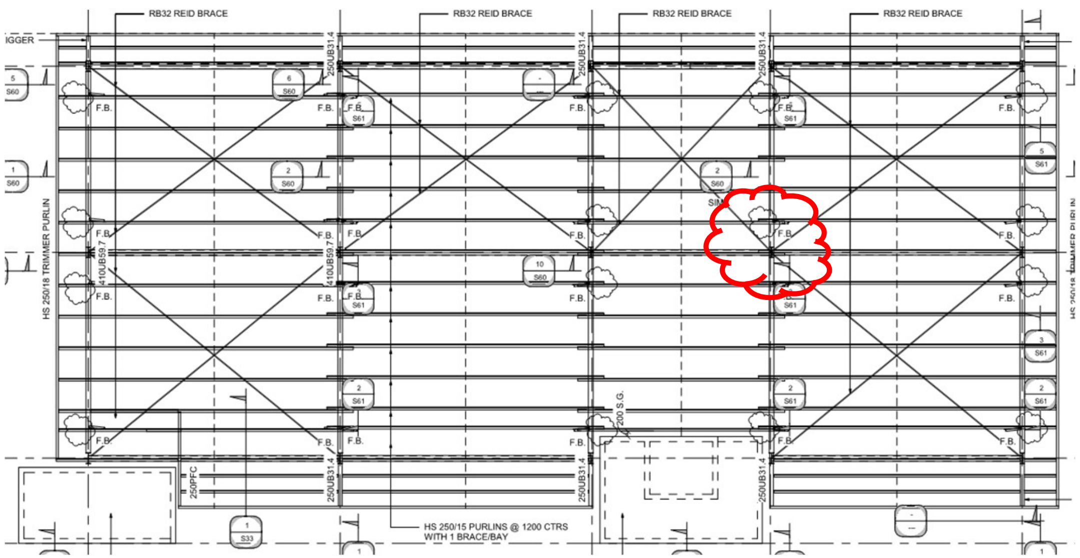
Longitudinal Connections

- Lets look at some of the more critical connections for the load path in the longitudinal direction

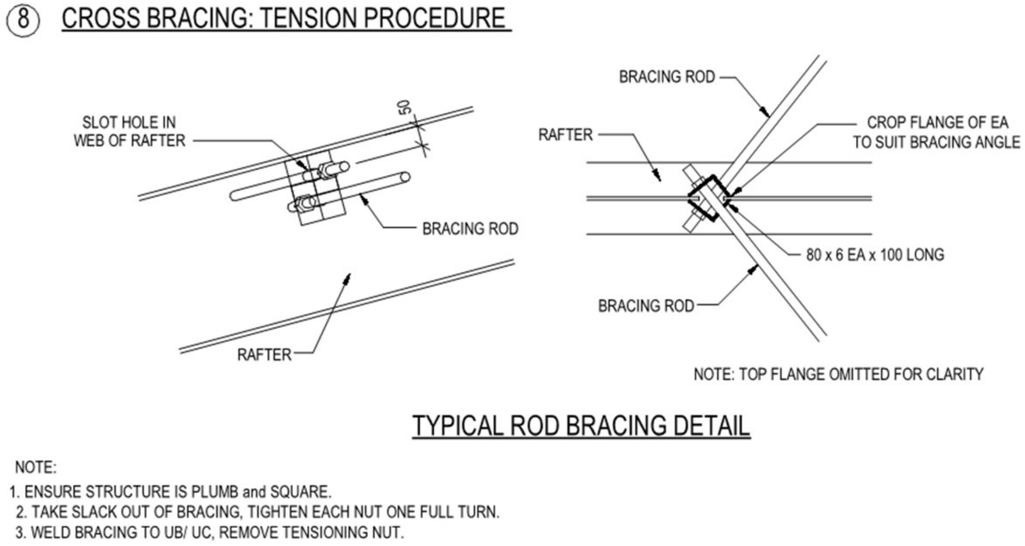


- Roof plane tension only bracing

Roof plane bracing



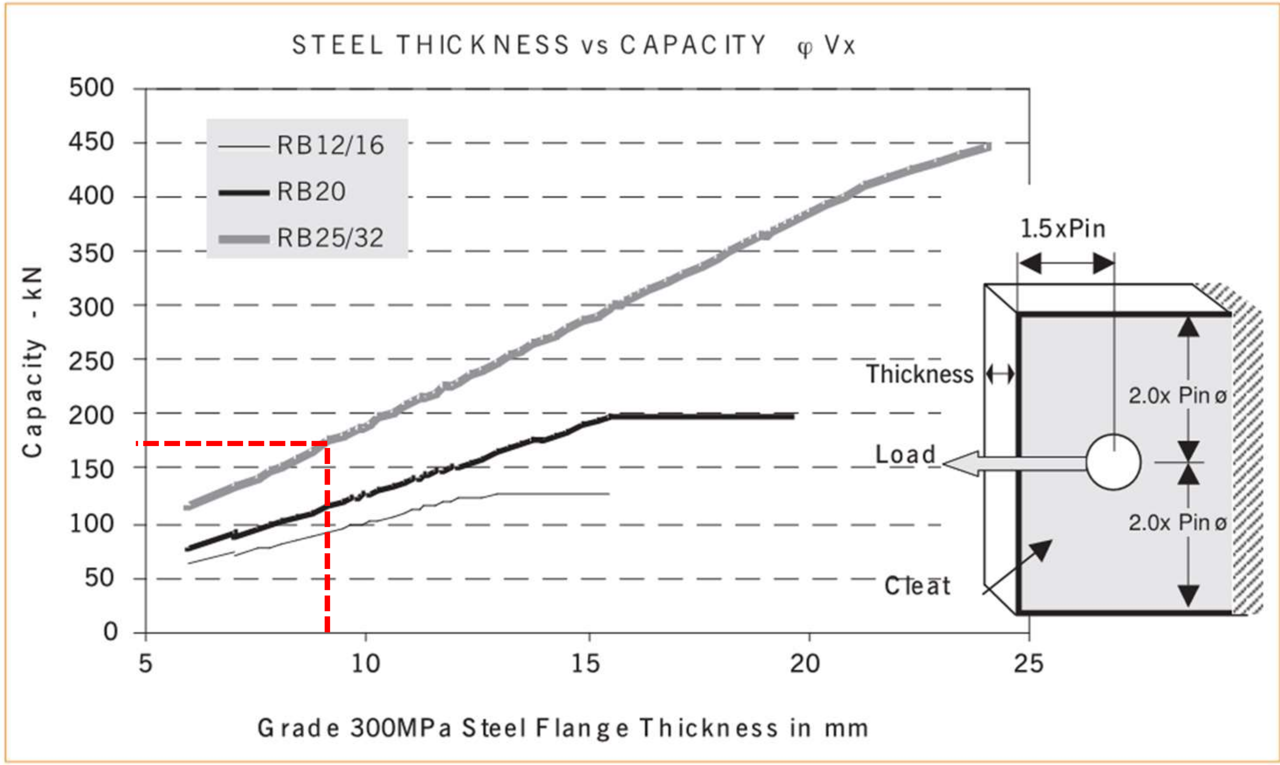
- Roof plane tension only bracing are RB32 Reid braces
- RB32 capacity is 462kN
- What's the connection detail?



- Generic detail on drawings, which could not be built due to portal cleat arrangement
- On site as-built was an 8mm cleat welded to the web of the portal

Reidbrace connection

Graph 3. – Pin Flange Connection Capacity φVx

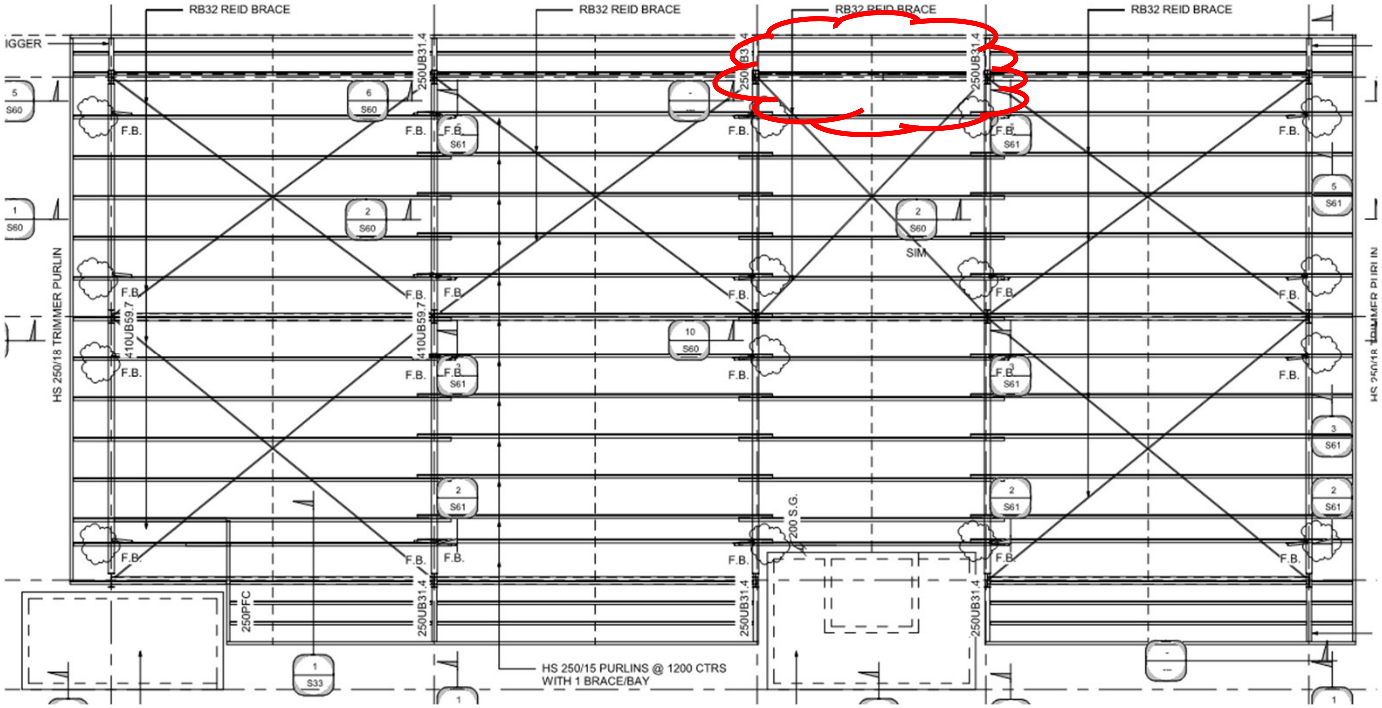


Graph 1.

- RB32 – Min UTS = 462kN
- 8mm cleat – cleat pin capacity ≈175kN

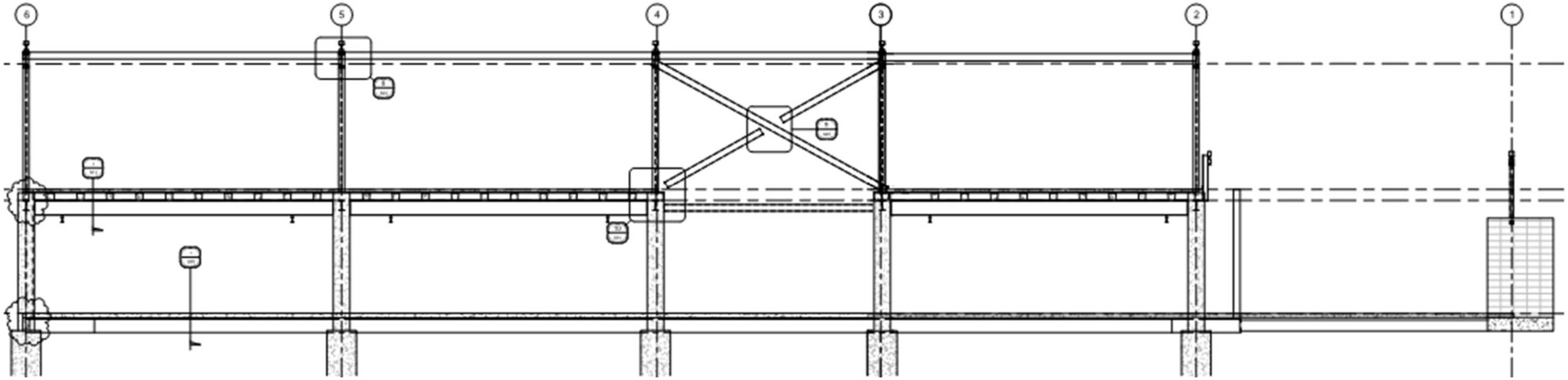
This connection cannot transfer the loads required & would have a connection failure

Wall plane bracing



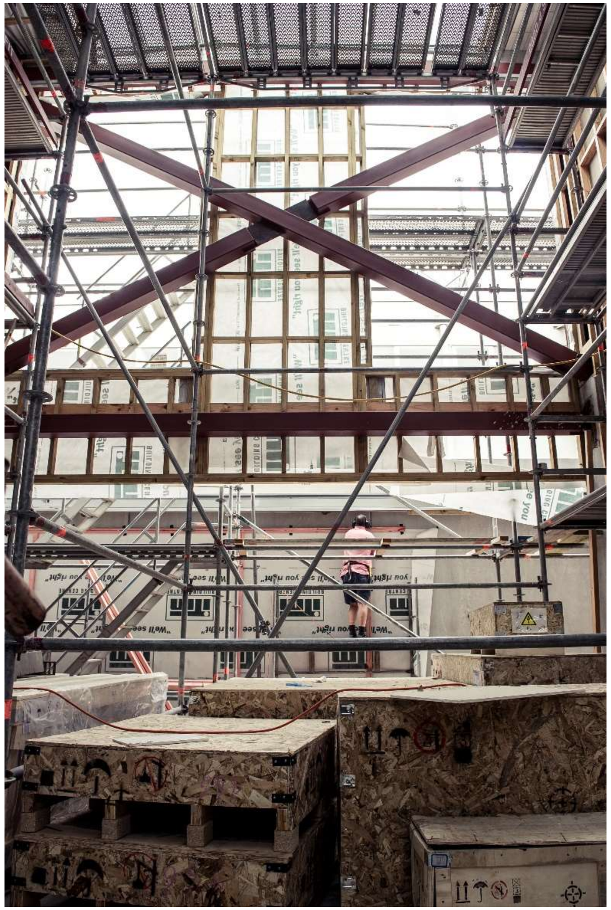
- Wall plane bracing in one bay on one side only

Wall plane bracing

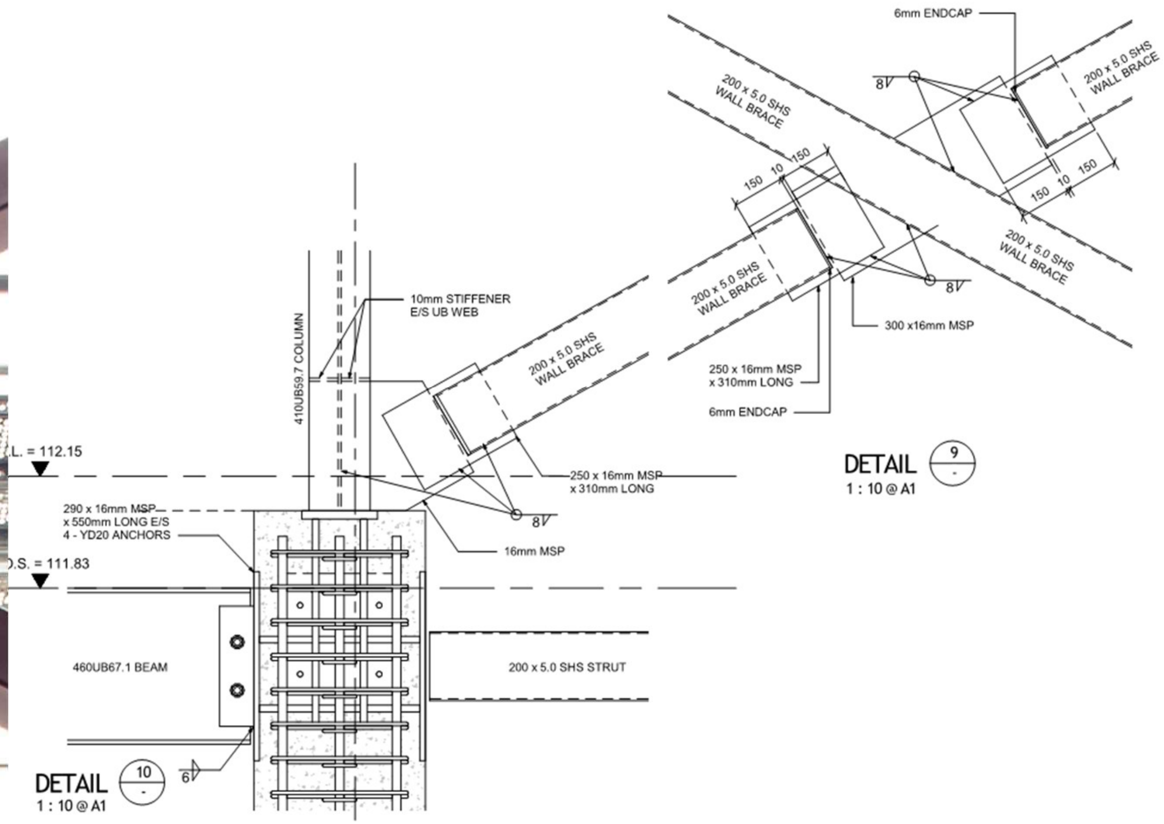


- Tension/compression brace in one bay to brace entire first floor roof and end walls out-of-plane

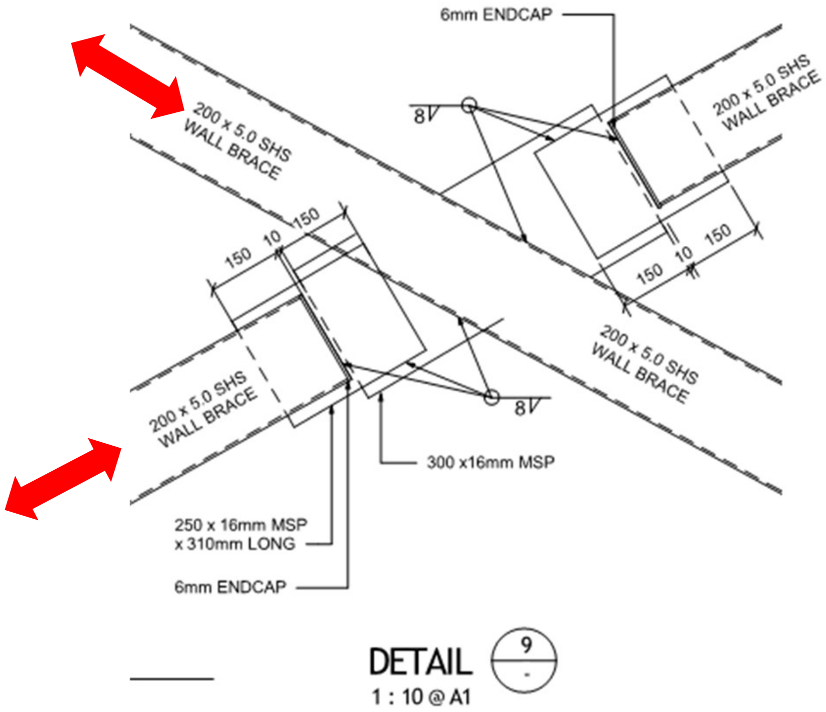
Wall plane X bracing



X brace connections

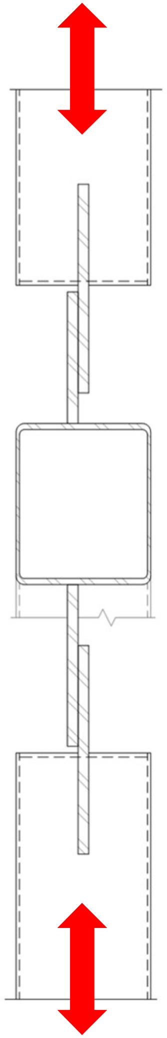


X brace connections



- 200x5 SHS in compression
- 16mm cleat slotted centrally into SHS and welded
- 16mm cleat piece welded to SHS cleat and wall of crossing SHS
- SHS plate element bending? To transfer loads to adjacent cleat?

This connection cannot transfer the loads required & the member would not satisfy slenderness limits



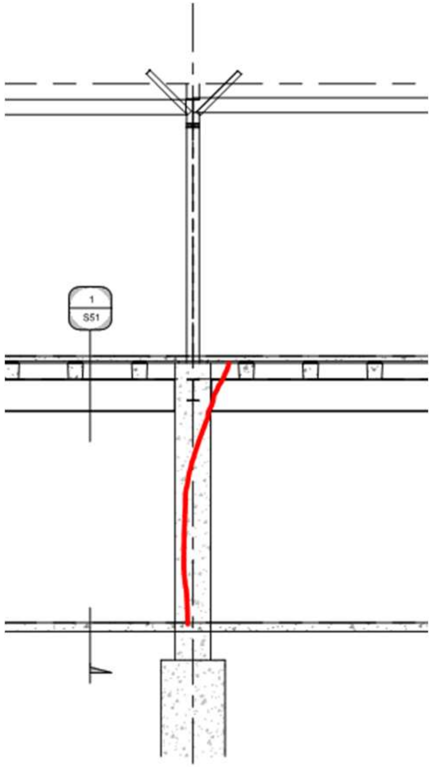
This connection cannot transfer the loads required
No redundancy in bracing – this is the only structural element to transfer first floor/roof forces to ground floor

Seismic Systems Overall

- Initial concept of four structural systems all separated very difficult to implement in practice
- Seismic jointing needed between all elements
- Adds complexity to the structural design, architectural detailing and building processes

The multiple connections/details require careful attention to detail if you go down this route

Beam-Column Joint



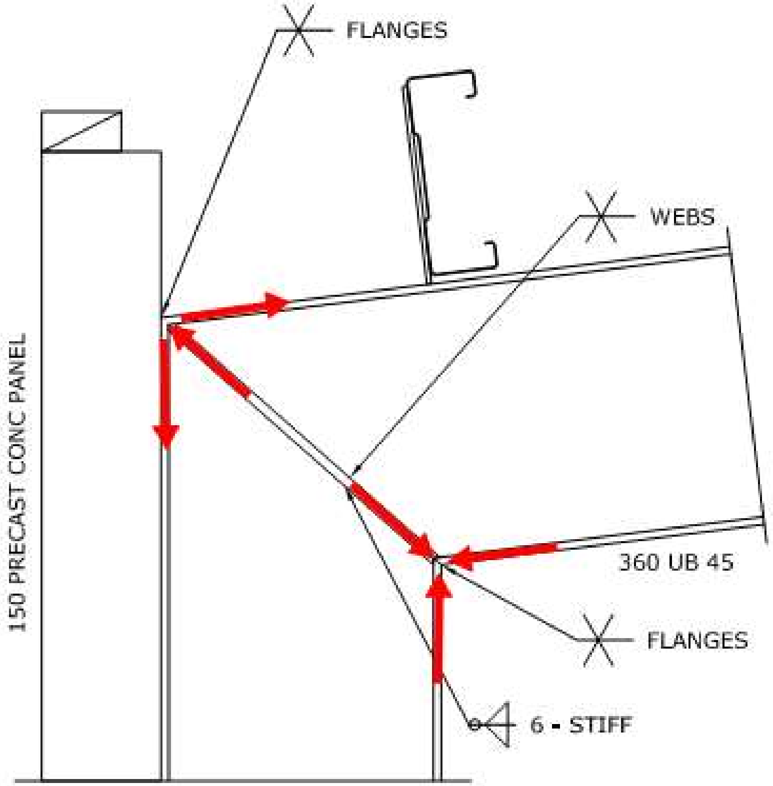
- Rotation will occur as the flexible cantilever columns resist lateral loads
- Connection of the steel beams has no allowance for rotation

Tip #4

Connections are critical

Some drawings of knee joint details?

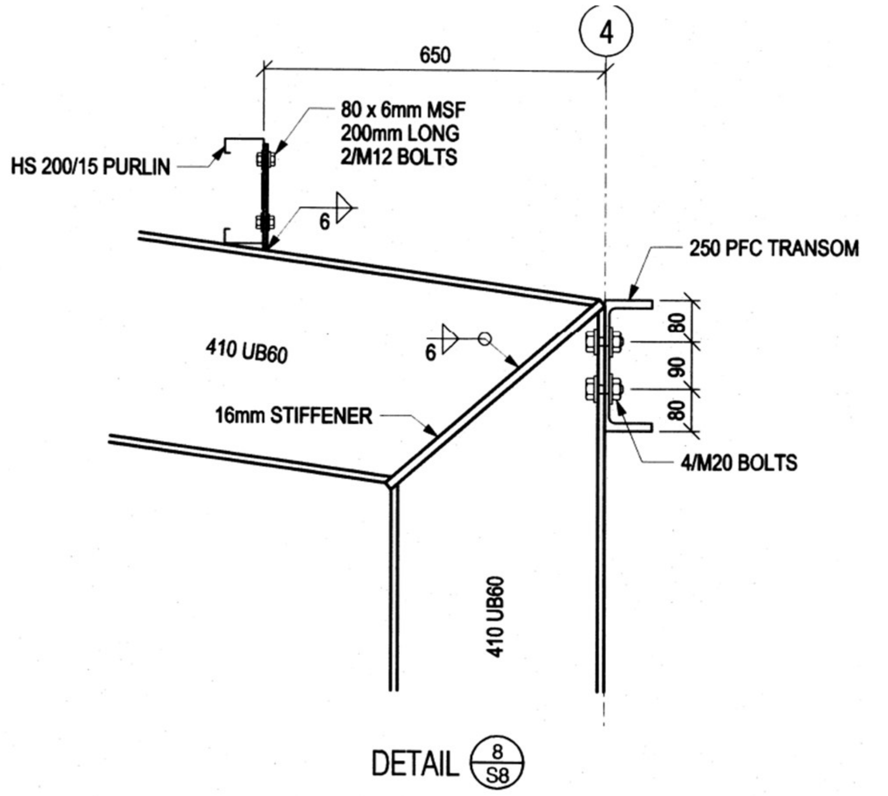
Diagonal stiffener plate?



- Grade 250MPa 10mm diagonal plate – to resist incoming forces from two 9.8mm Grade 320MPa flanges
- No lateral restraint to inside flange
- Acute angle full penetration butt weld

This connection cannot transfer the loads required & is not a recommended detail

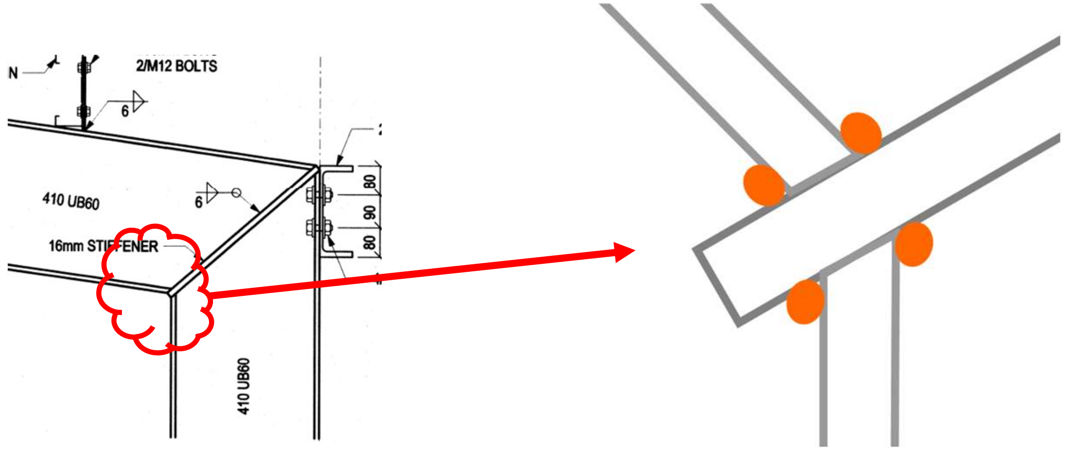
Steel Portal Frame Knee Joint



- Critical knee joint
- 16mm Grade 250 plate stiffener to resist component of loads from two 12.8mm Grade 320 thick flanges
- But the steel pieces are joined with a 6mm fillet weld all around!

This connection cannot transfer the loads required & is not a recommended detail

Steel Portal Frame Knee Joint



- Flange – 170mmx12.8mmx320MPa = 696kN
- Weld - 2x 170mm x 0.835 = 283kN

This connection cannot transfer the loads required

Moment forces transfer through flanges

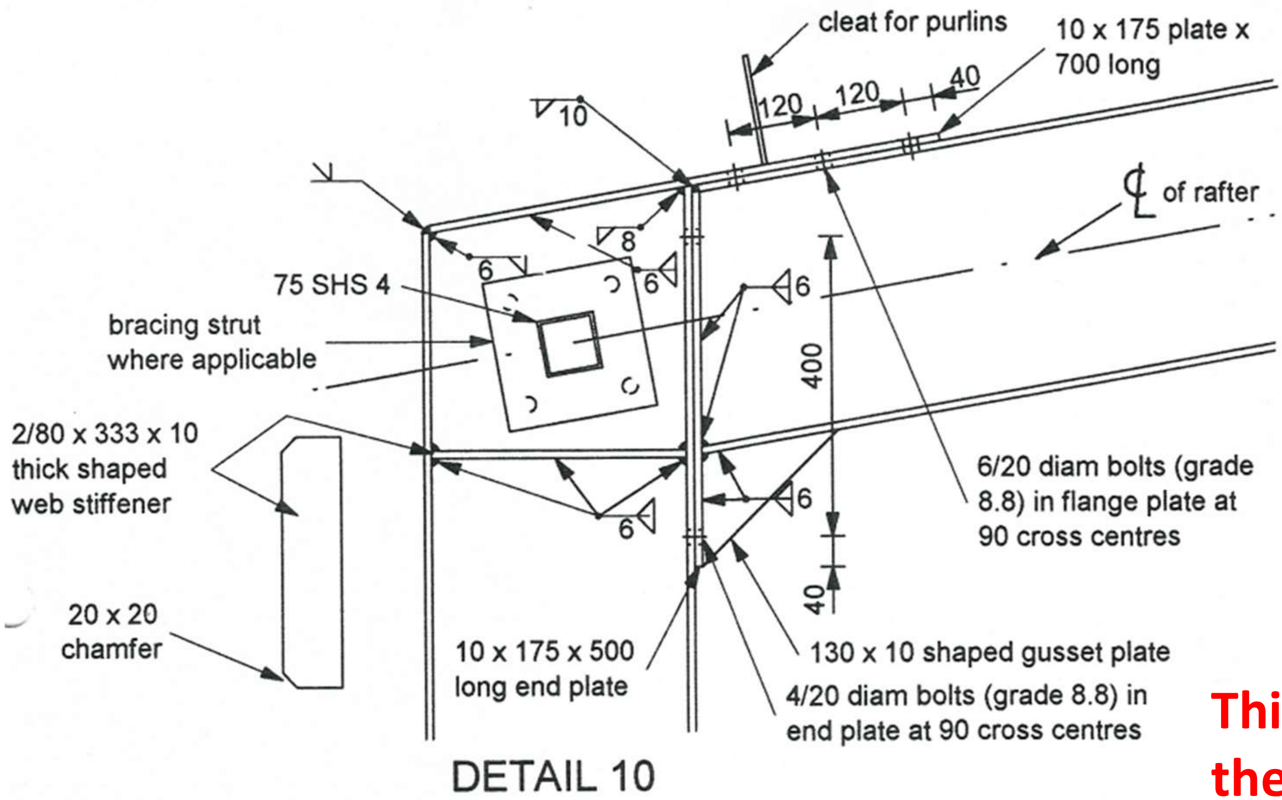
Tension component in column flange

Tension flange welded to stiffener

Stiffener resists couple from flanges

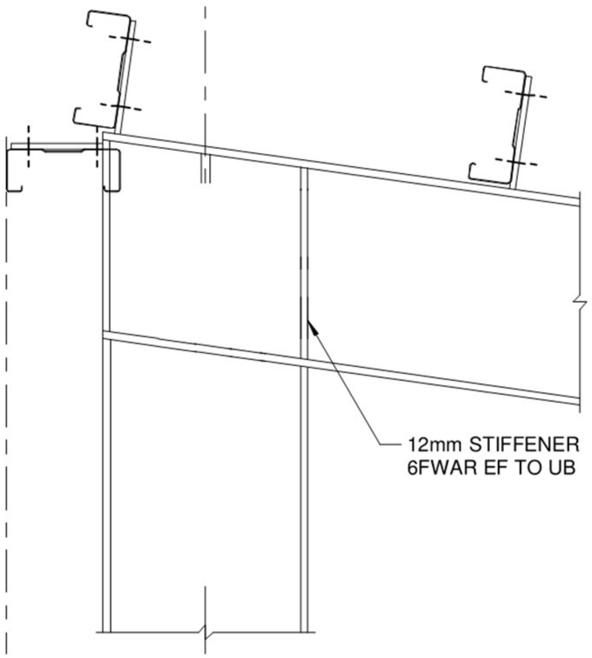
Weld to rafter flange

Tension component in rafter flange



- Multiple issues with this joint
- Insufficient weld sizes
- Insufficient plate sizes
- Eccentricities

This connection cannot transfer the loads required & is not a recommended detail



- 360UB57
- Flange is a 172mm x 13mm Grade 320MPa
- 12mm stiffener with 6mm FW would not be enough for the flange forces

This connection cannot transfer the loads required & is not a recommended detail

Moment Joint tips

- Use available guides – SCNZ has some helpful material
- Sweat the details!
- Be careful of stiffeners sizes relative to flanges
- Watch out for weld sizes
- This is a critical detail – be careful of load path

Robustness, robustness, robustness

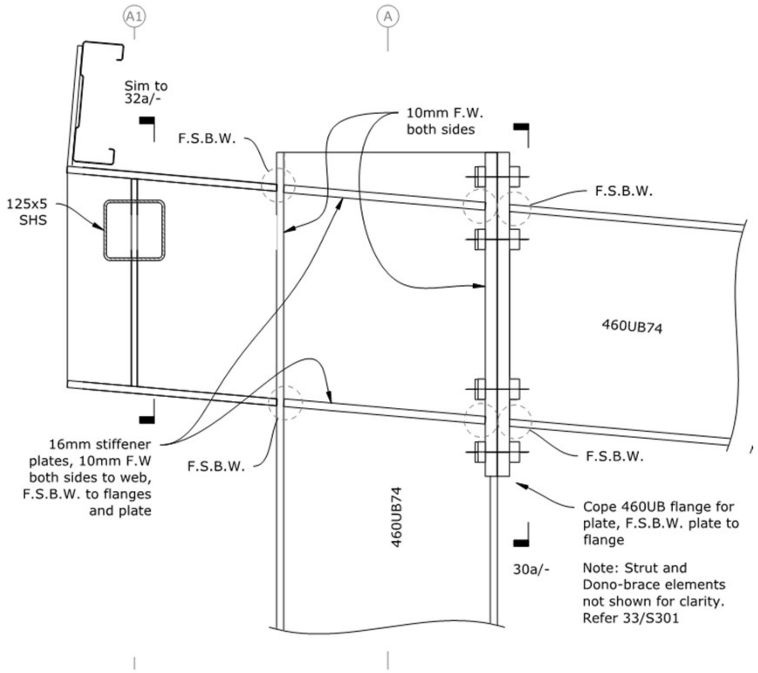
(source https://www.youtube.com/watch?v=TdY2AodUfks&t=505s)

- If the knee joint fails, you lose gravity support for the roof
- We want the hinge to form in the rafter (away from the joint!)

- Do not want
  - Sudden bolt failure
  - Weld failure
  - Tear out
- Can live with
  - Plate yielding
  - Member hinges
  - Web panel yielding

SESOC

Moment Joint tips



Section 30/S30 & S50
1:10

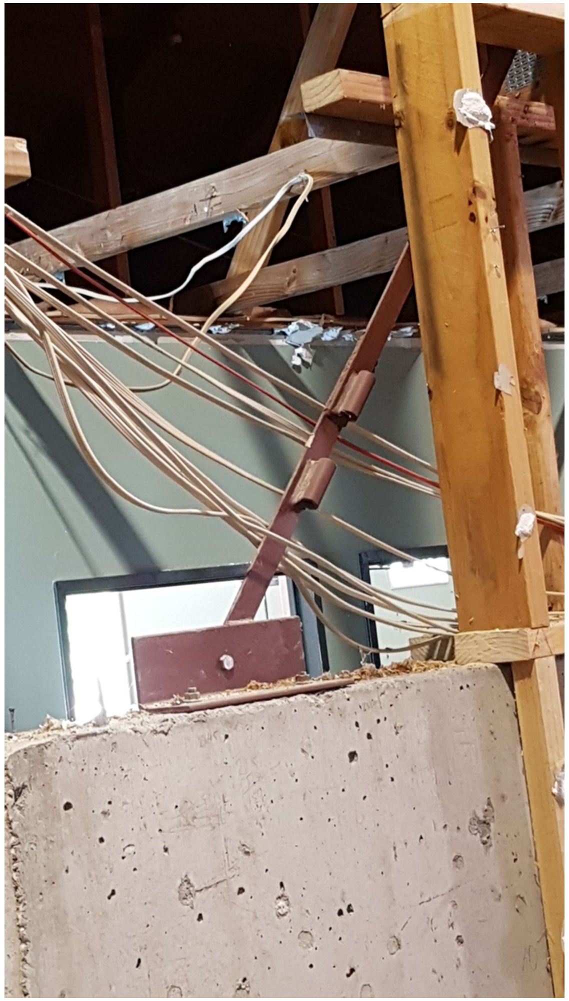
SCNZ Steel Advisor - Moment End Plate – Column Side (CON1001)

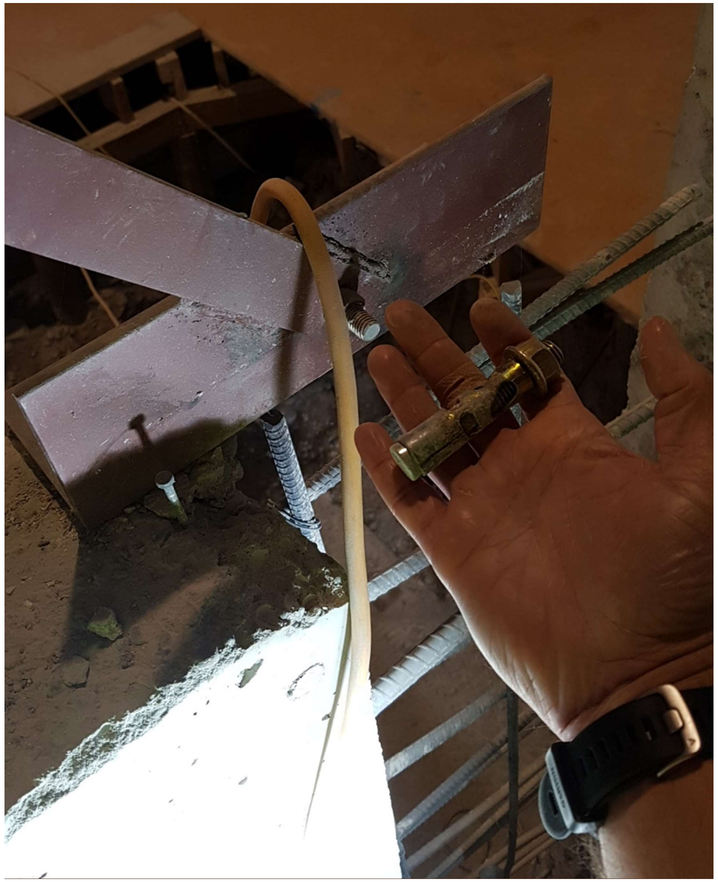
- Step 1 – Design continuity stiffeners
- Step 2 - Determine if Column Flange Backing Plates are required
- Step 3 - Determine column flange tension equivalent tee stub length
- Step 4 - Determine column flange tension capacity for each bolt row
- Step 5 - Determine column web tension capacity
- Step 6 - Calculate moment capacity
- Step 7 - Detail backing plates
- Step 8 - Determine column flange bolt bearing capacity
- Step 9 - Determine column panel zone shear action
- Step 10 - Calculate panel zone shear capacity and detail doubler plate

& this is just for the Column Side!

Some drawings of tension brace connections?

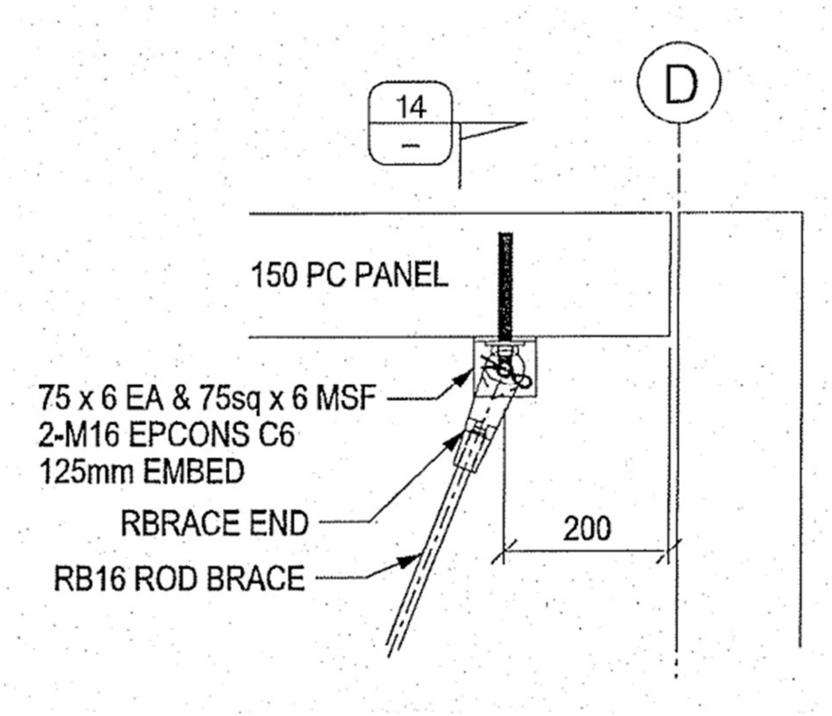
Tension Bracing details – example 1

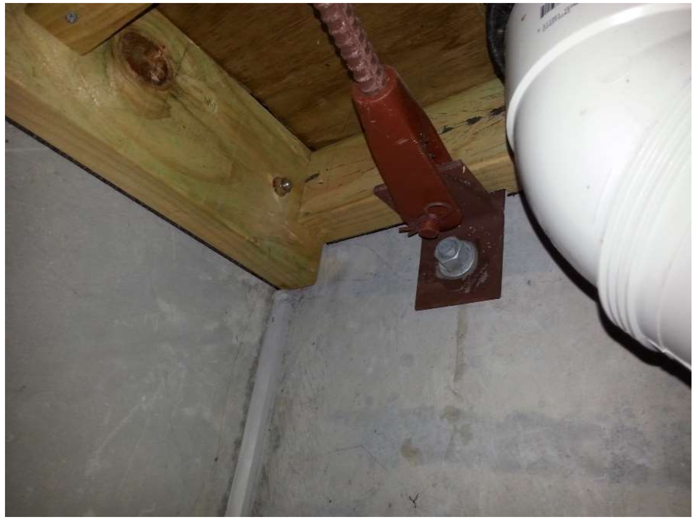




This connection cannot transfer the loads required & is not a recommended detail

Some Tension Bracing details



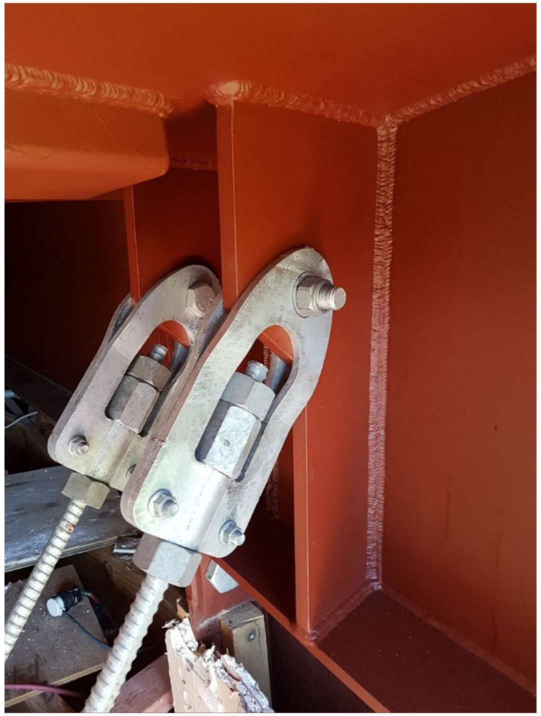


This connection cannot transfer the loads required & is not a recommended detail

Tension Bracing connection tips

- A connection failure needs to be avoided
- This would be a sudden brittle failure mechanism
- Avoid
  - Plate tear-out
  - Weld failures
  - Bolt failures
  - Indirect load paths

Tension Bracing connection tips



- Node your connections
- line up your intersection points for tension bracing with struts – finish the triangle
- Consider designing cleats for the overstrength of the brace
- Pay careful attention to
  - Grade of plate
  - Edge distances
  - Size of welds

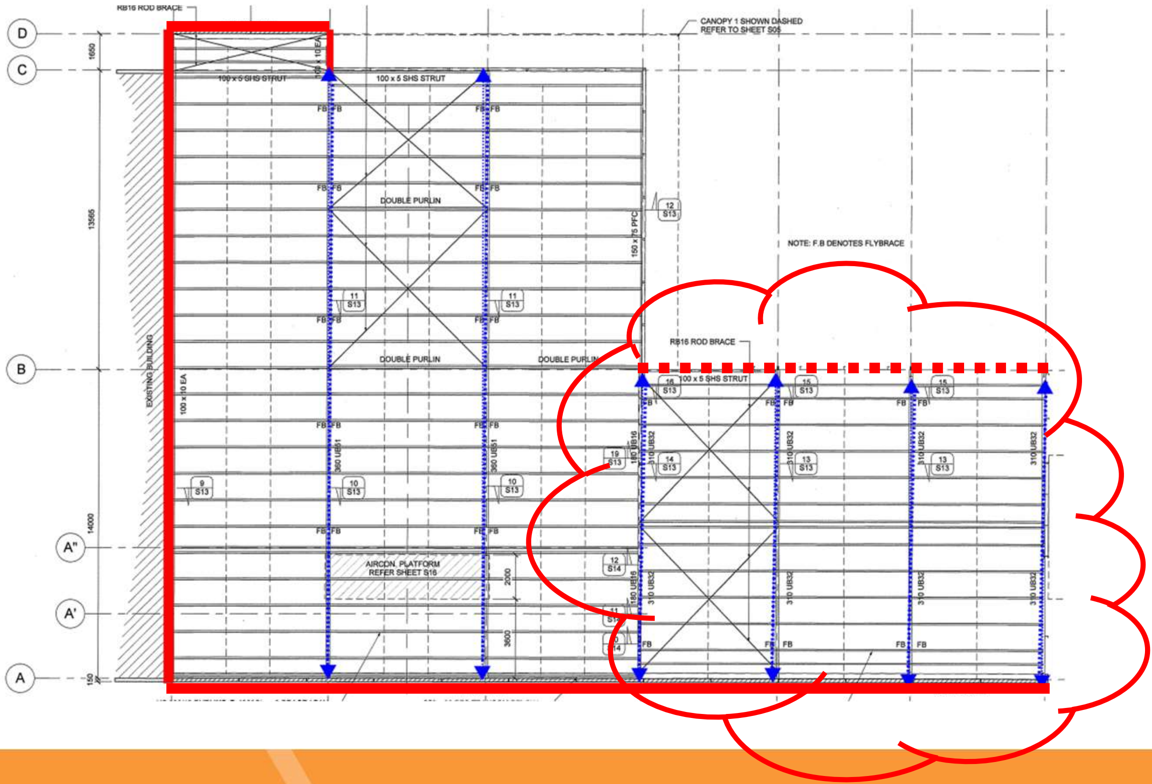
Tip #5

1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5		

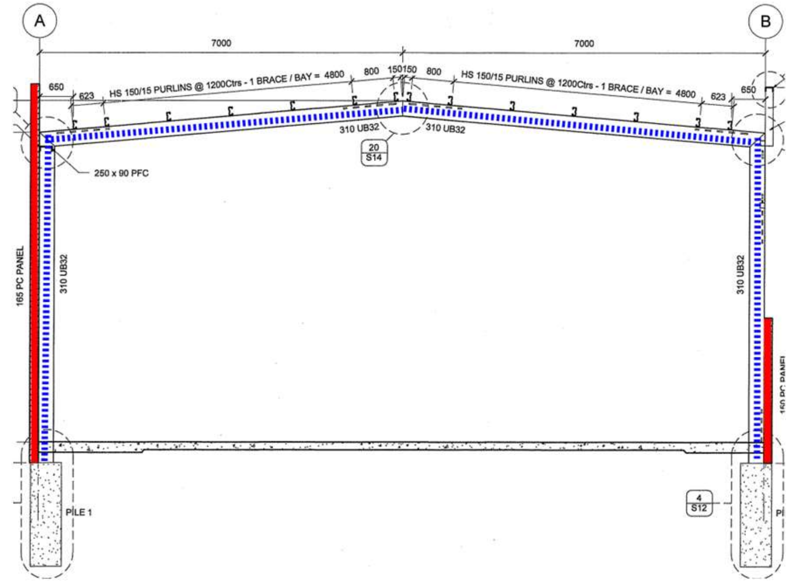
Tip #5

If you 'adopt a ductility', make sure you can actually get ductility

Building E – Storage Area Frames

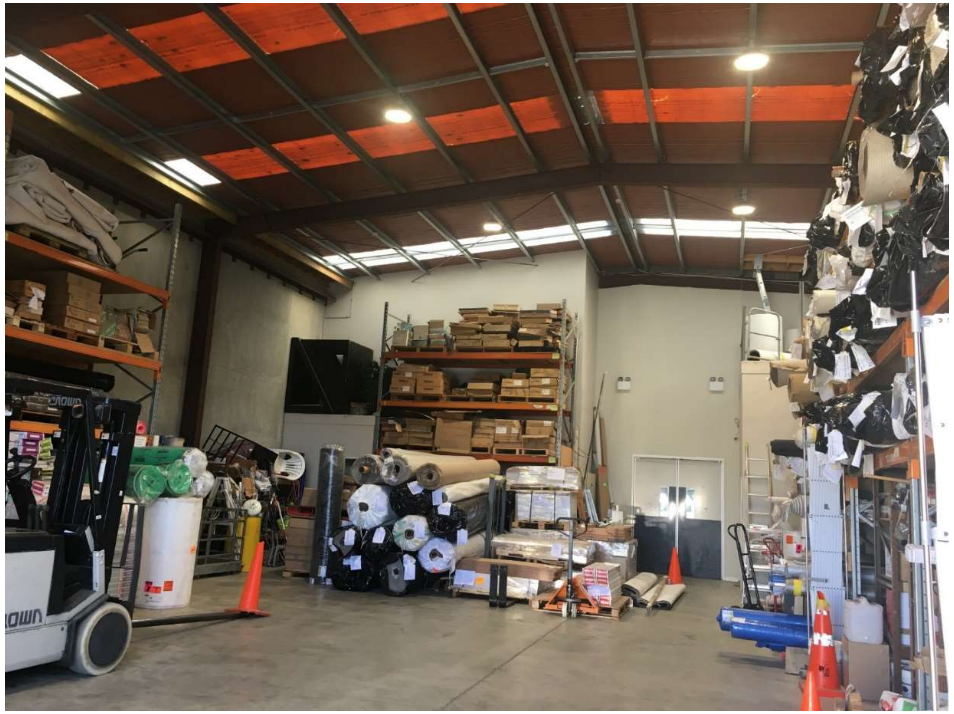


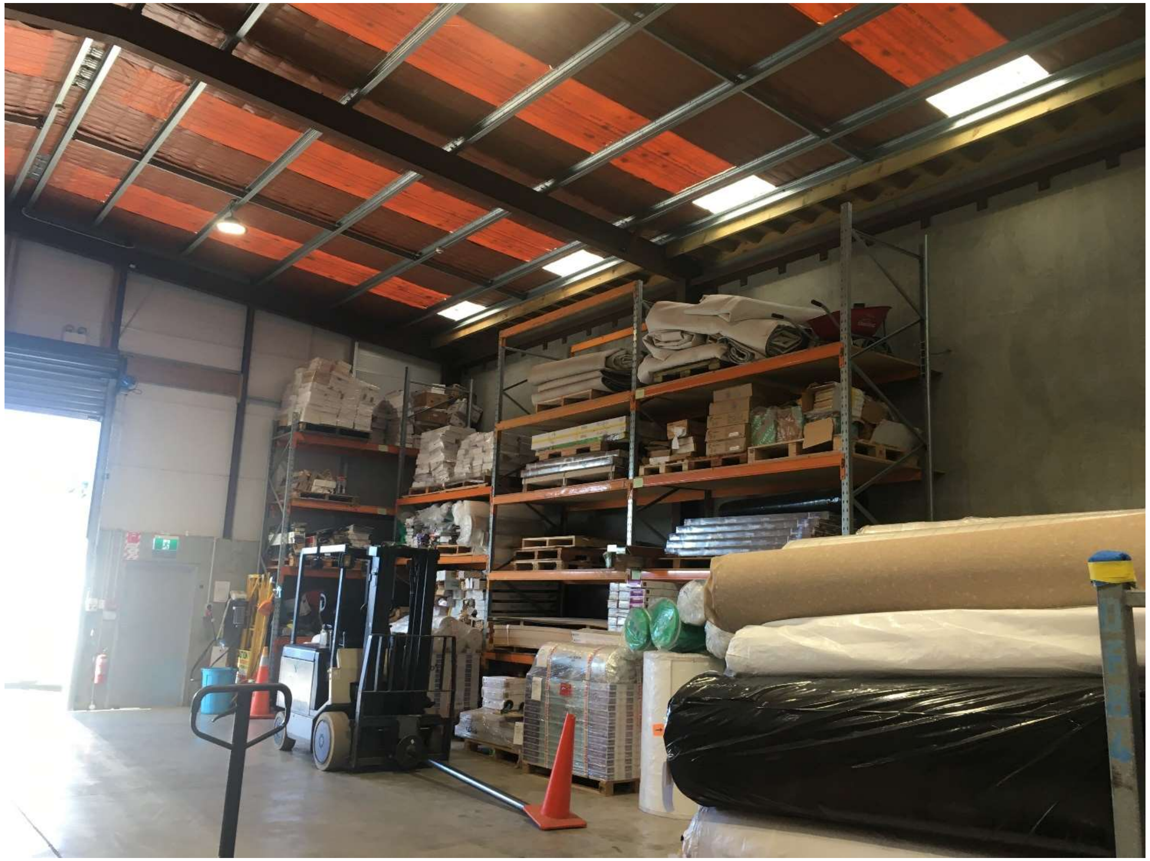
- 310UB32 frames at 6m centres
- 6m knee height
- Precast panels (in red)

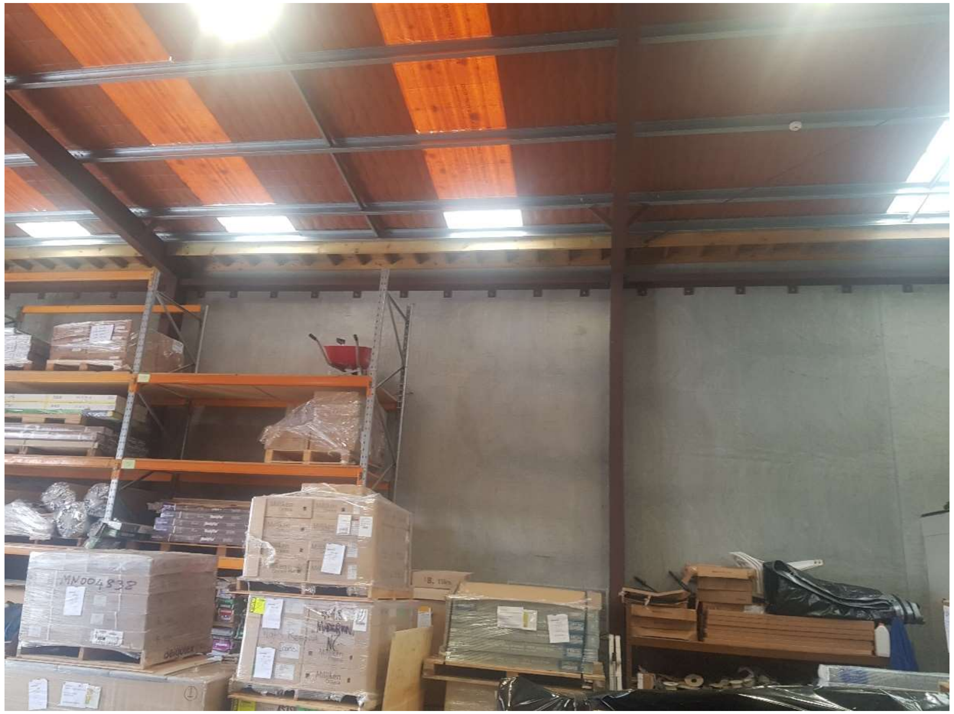



Court
NO PARKING

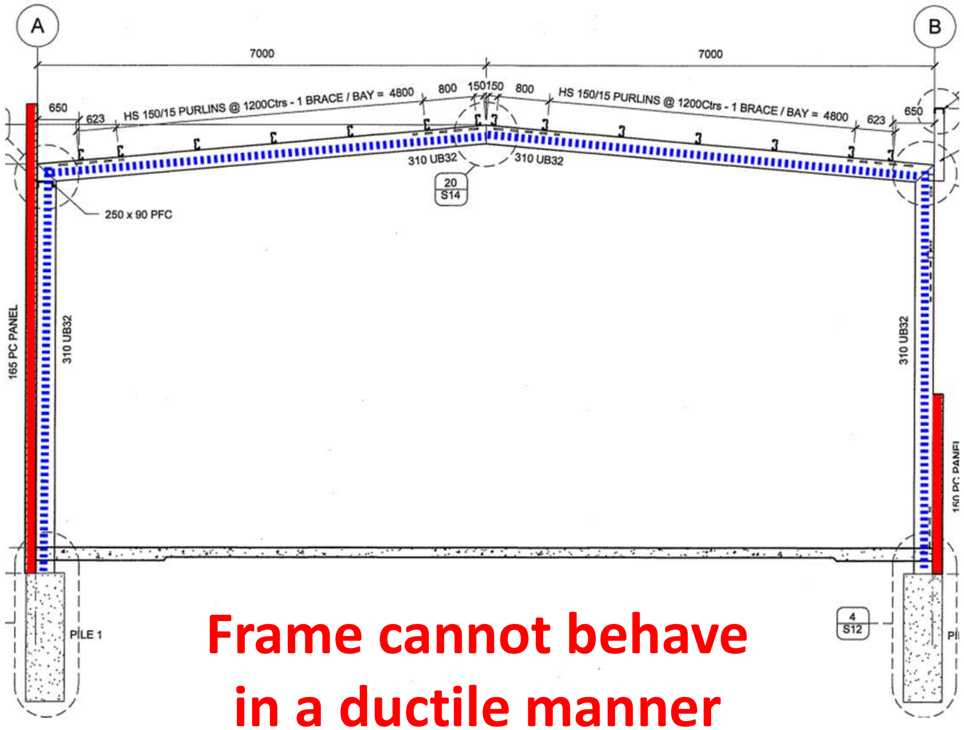

McKenzies
Curtain Court
carpetcourt







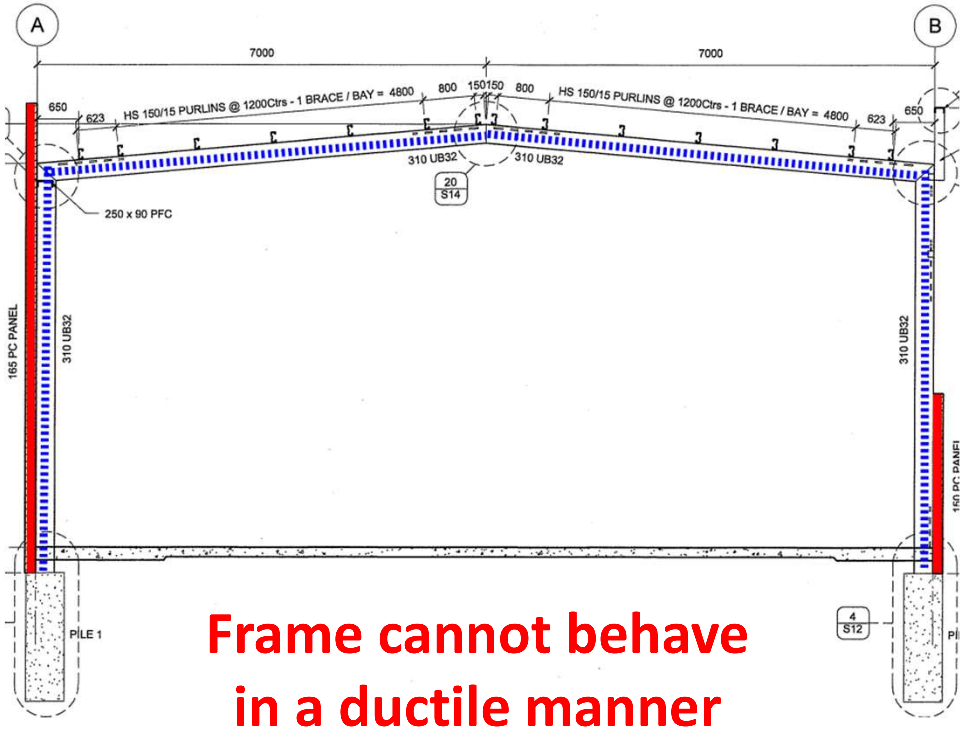
Storage Area Frames



Frame cannot behave in a ductile manner

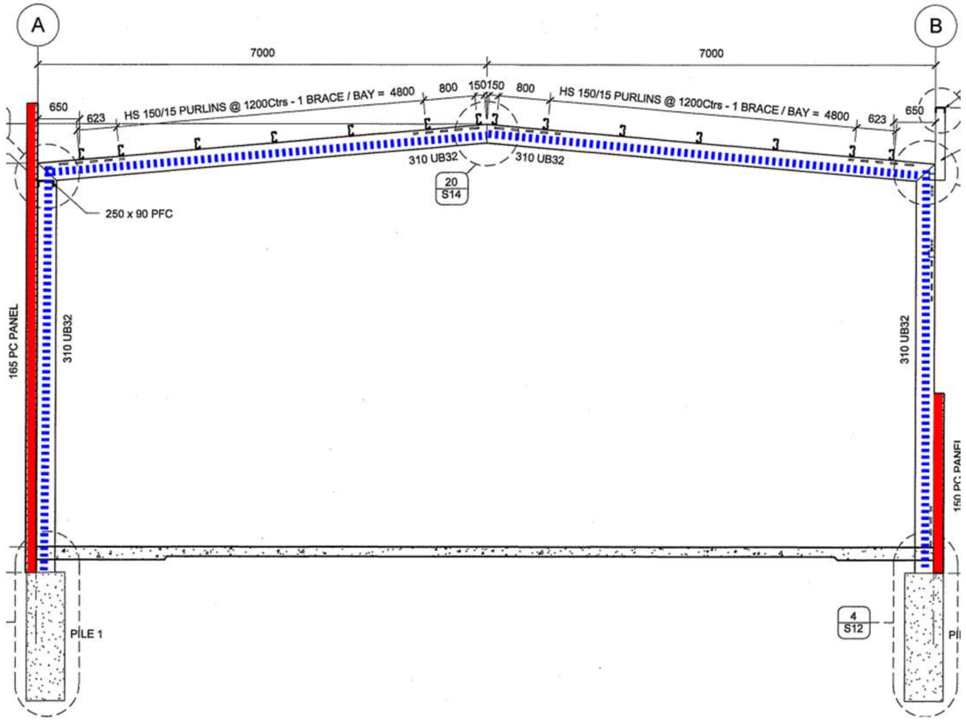
- No fly braces – no lateral restraint to rafter bottom flange, ie segment will buckle before yield can be achieved
- No lateral restraint to column, apart from at knee where collector provides restraint
- 310UB32 is a category 4 member , ie the UB elements will locally buckle before the member can reach yield

Storage Area Frames



- Foundations not designed to ensure yielding, ie not designed or checked for the overstrength capacity of the frame

Storage Area Frames



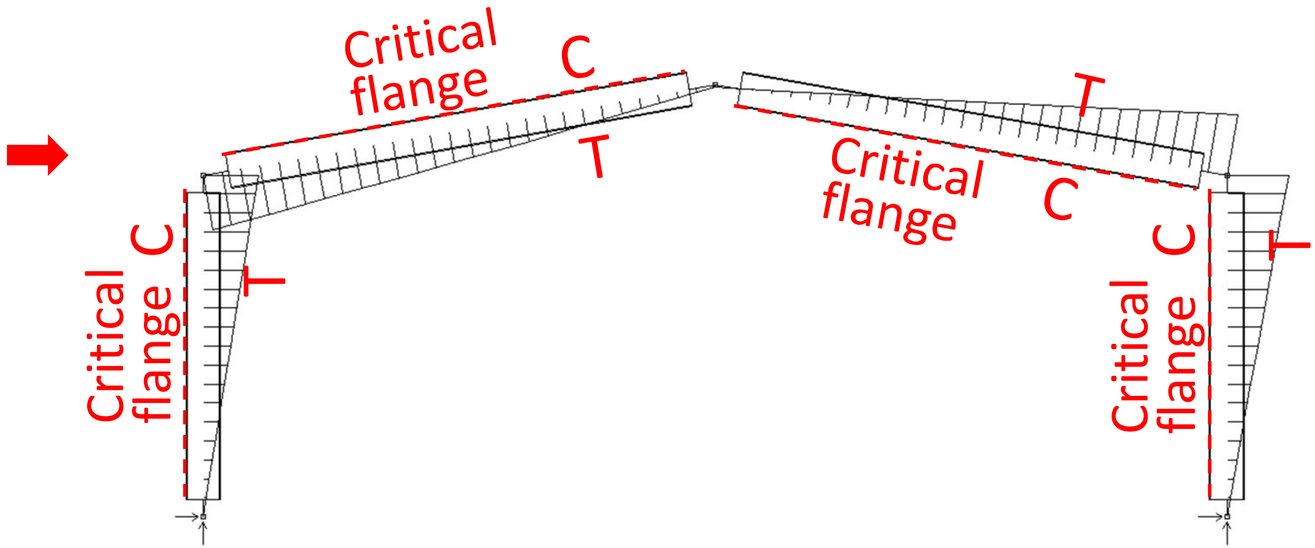
- Even if we assume a rigid fixed base, drifts are in the order of 4.5%
- The foundations and piles are not likely to be rigidly fixed, so will add to this deflection

Frame is undersized and too flexible

Tip #5

If you ‘adopt a ductility’, make sure you can actually get ductility

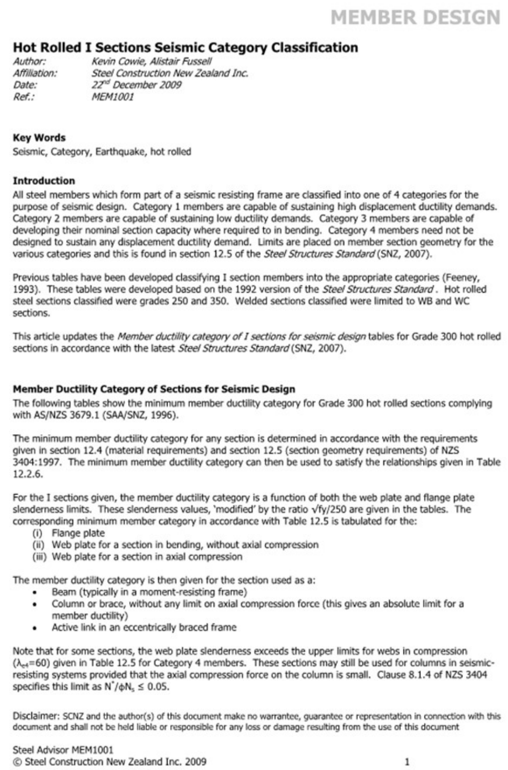
What does good look like – adopt a mu



- Why not use category 1 or 2 steel members?
- Laterally restrain the members
- Be careful with connections – keep it robust
- When running your analysis be mindful of where your critical flange is (compression flange)

Steel Member Category

Category 1	cross-sections are those which can form a plastic hinge
Category 2	capable of sustaining low ductility demands
Category 3	capable of developing their nominal section capacity where required to in bending
Category 4	local buckling will occur before the attainment of yield stress in one or more parts of the cross-section



MEMBER DESIGN

Hot Rolled I Sections Seismic Category Classification

Author: *Kevin Cowie, Alistair Fussell*
Affiliation: *Steel Construction New Zealand Inc.*
Date: *22nd December 2009*
Ref.: *MEM1001*

Key Words

Seismic, Category, Earthquake, hot rolled

Introduction

All steel members which form part of a seismic resisting frame are classified into one of 4 categories for the purpose of seismic design. Category 1 members are capable of sustaining high displacement ductility demands. Category 2 members are capable of sustaining low ductility demands. Category 3 members are capable of developing their nominal section capacity where required to in bending. Category 4 members need not be designed to sustain any displacement ductility demand. Limits are placed on member section geometry for the various categories and this is found in section 12.5 of the *Steel Structures Standard* (SNZ, 2007).

Previous tables have been developed classifying I section members into the appropriate categories (Feeney, 1993). These tables were developed based on the 1992 version of the *Steel Structures Standard*. Hot rolled steel sections classified were grades 250 and 350. Welded sections classified were limited to WB and WC sections.

This article updates the *Member ductility category of I sections for seismic design* tables for Grade 300 hot rolled sections in accordance with the latest *Steel Structures Standard* (SNZ, 2007).

Member Ductility Category of Sections for Seismic Design

The following tables show the minimum member ductility category for Grade 300 hot rolled sections complying with AS/NZS 3679.1 (SAA/SNZ, 1996).

The minimum member ductility category for any section is determined in accordance with the requirements given in section 12.4 (material requirements) and section 12.5 (section geometry requirements) of NZS 3404:1997. The minimum member ductility category can then be used to satisfy the relationships given in Table 12.2.6.

For the I sections given, the member ductility category is a function of both the web plate and flange plate slenderness limits. These slenderness values, 'modified' by the ratio $\sqrt{fy/250}$ are given in the tables. The corresponding minimum member category in accordance with Table 12.5 is tabulated for the:

(i) Flange plate
(ii) Web plate for a section in bending, without axial compression
(iii) Web plate for a section in axial compression

The member ductility category is then given for the section used as a:

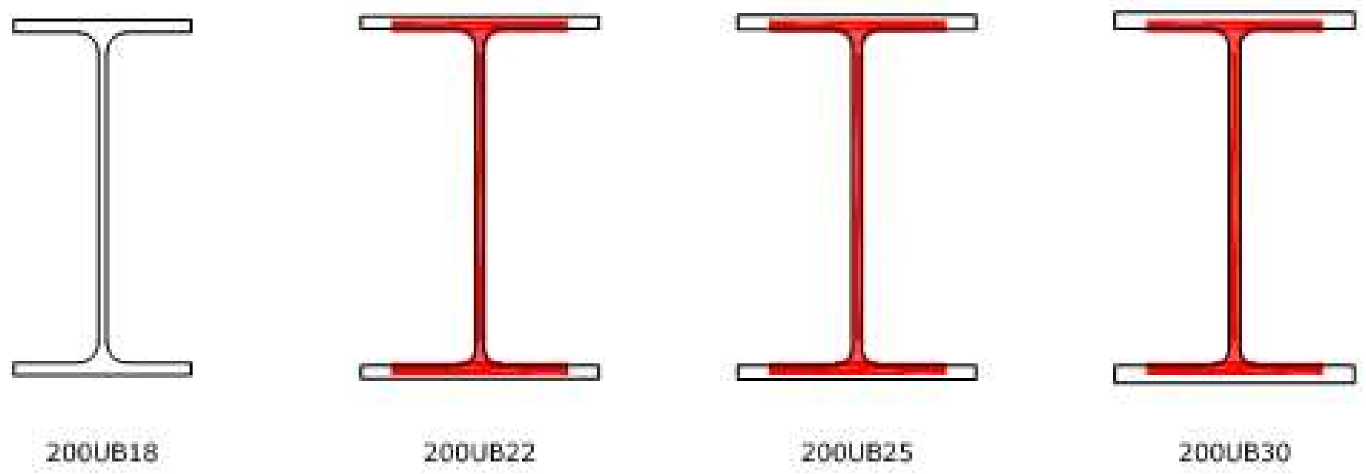
- Beam (typically in a moment-resisting frame)
- Column or brace, without any limit on axial compression force (this gives an absolute limit for a member ductility)
- Active link in an eccentrically braced frame

Note that for some sections, the web plate slenderness exceeds the upper limits for webs in compression (λ_{e4}=60) given in Table 12.5 for Category 4 members. These sections may still be used for columns in seismic-resisting systems provided that the axial compression force on the column is small. Clause 8.1.4 of NZS 3404 specifies this limit as $N^*/\phi N_s \leq 0.05$.

Steel Advisor MEM1001
 1

SCNZ – Hot Rolled Steel Section Member Classification

Choose Category



- Category 1
- OK for ductility
- (can be μ>3)

- Category 4
- Elastic only
- (max μ=1)

- Category 3
- Nominally ductile
- (max μ=3)

- Category 1
- OK for ductility
- (can be μ>3)

Category 1 can yield….

Category 4 will locally buckle!

Why not just use a category 1 or 2 steel section?

Table 3: Seismic Section Classification for Beam Columns – Grade 300 Universal Beams to AS/NZS 3679.1:1996

Designation		Web (Constant Axial Load Limit) Category 1, 2 $N^*_g/\phi N_s \le$	Category 3 $N^*_g/\phi N_s \le$	Flange Category
610 UB	125	0.216	0.709	1
610 UB	113	0.192	0.634	1
610 UB	101	0.154	0.519	1
530 UB	92.4	0.194	0.640	1
530 UB	82	0.168	0.562	1
460 UB	82.1	0.243	0.795	1
460 UB	74.6	0.211	0.695	1
460 UB	67.1	0.183	0.608	1
410 UB	59.7	0.197	0.650	1
410 UB	53.7	0.185	0.615	1
360 UB	56.7	0.256	0.835	1
360 UB	50.7	0.223	0.731	1
~~360 UB~~	~~44.7~~	~~0.201~~	~~0.665~~	~~3~~
310 UB	46.2	0.250	0.815	1
310 UB	40.4	0.216	0.708	1
~~310 UB~~	~~32~~	~~0.177~~	~~0.588~~	~~4~~
250 UB	37.3	0.400	0.964	1
~~250 UB~~	~~31.4~~	~~0.326~~	~~0.918~~	~~3~~
250 UB	25.7	0.217	0.712	1
200 UB	29.8	0.678	1.000	1
~~200 UB~~	~~25.4~~	~~0.572~~	~~1.000~~	~~3~~
~~200 UB~~	~~22.3~~	~~0.357~~	~~0.938~~	~~4~~
200 UB	18.2	0.262	0.853	1
180 UB	22.2	0.814	1.000	1
180 UB	18.1	0.597	1.000	1
180 UB	16.1	0.451	1.000	1
150 UB	18	1.000	-	1
150 UB	14	0.787	1.000	1

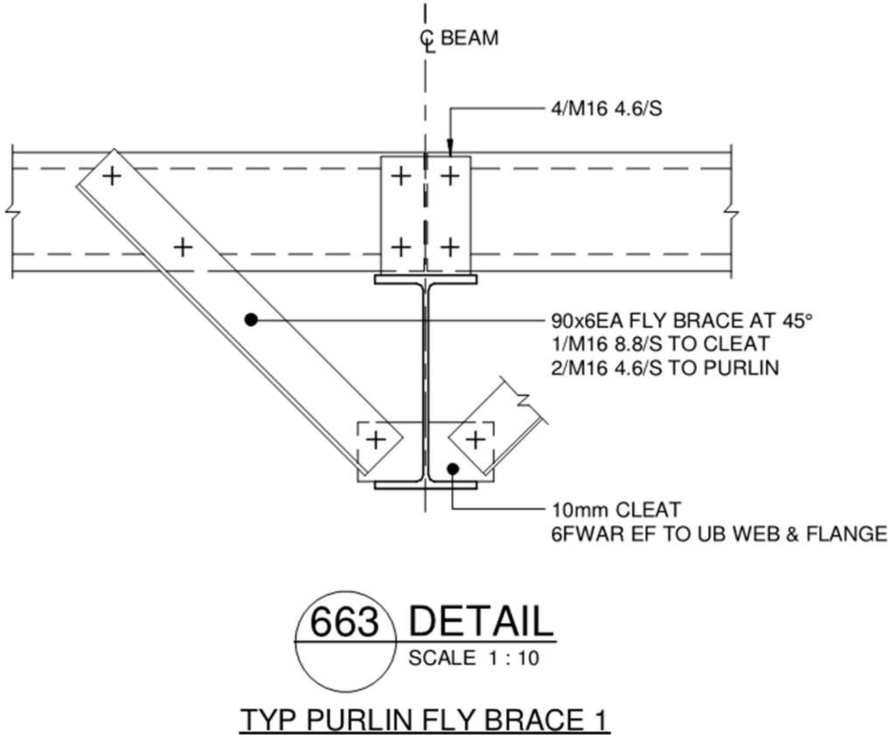
- indicates no axial load limit applies, web slenderness values comply with the requirements for category 1 or 2 elements. Refer to table 1

Looking at only category 1 or 2 steel members doesn't restrict us too much!

Hot Rolled I Sections Seismic Category Classification

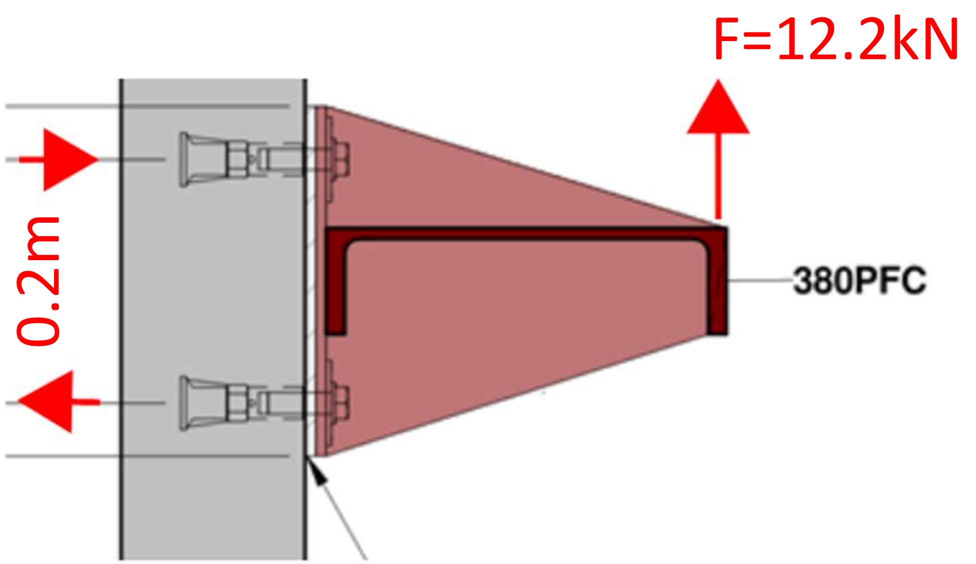
Author: *Kevin Cowie, Alistair Fussell*
Affiliation: *Steel Construction New Zealand Inc.*
Date: *22nd December 2009*
Ref.: *MEM1001*

Lateral Restraint



- Fly braces – rather cheap structural component!
- Make sure you consider segment lengths (points of lateral restraint to the critical flange!) when you are designing.

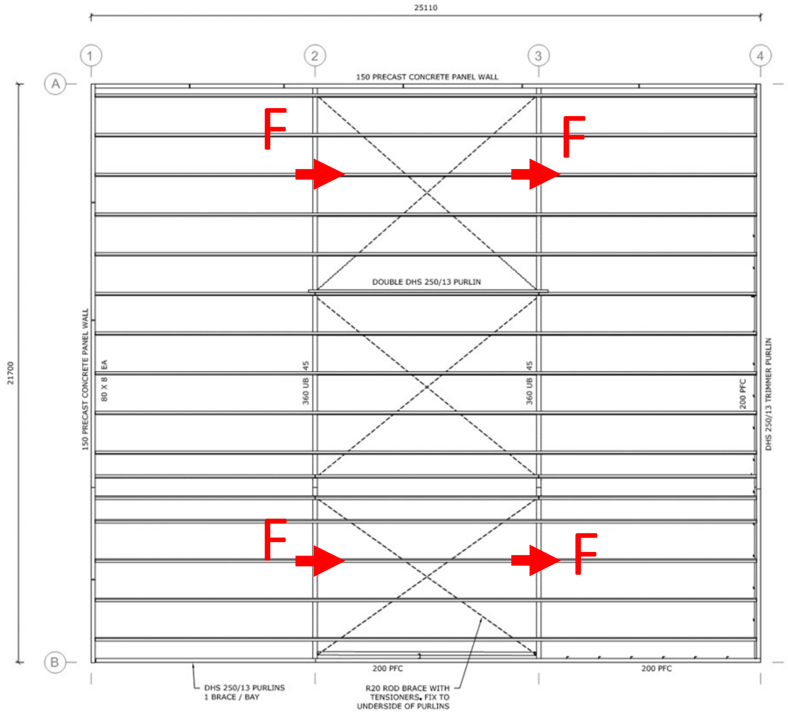
Consider the load path of your lateral restraint



These loads need to be resolved too!

F =0.025x100x17.5x280MPa
=12.25kN

Consider the load path of your lateral restraint



- Load path of lateral restraining force also requires consideration – a load that needs to be resolved

Frame – choosing a ductility

SESOC Journal

"ADOPT A DUCTILITY" FOR STEEL PORTAL FRAME STRUCTURES

First published in the Proceedings of the NZSEE Auckland Conference, 13 - 15 April 2018

M.Grant[1] and S.Lanser[2]

SUMMARY

Steel portal frames are a simple and commonly used structural form. It is also common to design portal frames to support heavy precast concrete cladding panels. This paper aims to outline the Steel Structures Standard NZS 3404 requirements for the seismic design of portal frames and show that the premise of 'adopt a ductility' can be irrelevant in highly seismic zones.

1 WHAT DOES 'ADOPT A DUCTILITY' MEAN?

1.1 What does ductility mean

What does it mean when we 'adopt a ductility'? For steel portal frames, the premise is that we as designers can nominate the level of ductility to reduce seismic design loads, and then use the relevant Standards to ensure that the required ductility is achieved.

For steel structures, the adoption of ductility is a design method which exploits the well-known ability of steel to deform plastically under load once its yield strength is reached (Henderson, 2015).

For a steel section to form a stable plastic hinge capable of cyclic deformation past yield, and for a stable overall ductile mechanism to develop, all premature localised and member failure mechanisms need to be suppressed. This is achieved through appropriate section selection, capacity design and restraint to prevent connection failure, and local and lateral buckling.

The main purpose of this paper is to highlight the fact that should a structural engineer choose to design a steel portal frame structure for seismic loads less than fully elastic by adopting a ductility factor greater than 1, then this must be accompanied by a fully implemented capacity design criteria. For portal frames in high seismic areas providing lateral support to precast concrete wall panels, SLS and ULS deflection limits may govern the size of the steel members required, thereby restricting the maximum ductility that can be used.

1.2 Inelastic Mechanism

To clarify the NZS 1170.5 and NZS 3404 ductile design intentions, it is helpful to look at seismic design the same way we assess existing buildings. If we assess a portal frame structure, using the recommended Simple Lateral Mechanism Analysis (SLaMA) methodology, how would we approach this? SLaMA methodology dictates that we must first identify the capacity of each sub-assemblage to identify our failure mechanism. As we apply lateral load to our portal frame, what happens first?

- For a portal frame formed from open I sections with no fly bracing, it is likely that lateral torsional buckling of a segment will happen well before a plastic hinge can develop.
- For a frame with a poorly detailed knee joint, it is likely that joint failure will occur before a plastic hinge can develop
- For a frame with poor connection to the foundation, it is possible that connection failure will occur before a plastic hinge can develop.

All sub-assemblages of a frame should be checked to determine what will fail first. If we adopt the above thinking in design, it assists with understanding what the relevant Standards are requiring of us as designers.

1.3 Tell the structure what to do

As a designer we must be mindful of what we are requiring of our structure when we adopt a level of ductility.

To achieve ductility, we as designers are aiming for our steel section to form stable plastic hinges at pre-determined locations that remain stable under cyclic loading. This means that we cannot have premature failure of any other items. In addition, for plastic hinges to occur it is a fundamental requirement that stability of the members is maintained, both locally and globally (Henderson 2015). This means in practice the application of capacity design. This process requires that;

- A dependable overall mechanism can develop which precludes a collapse mechanism;
- That the slenderness of the plate elements of a (rafter or

PAPER CLASS & TYPE: GENERAL REFEREED

[1] BE, CMEngNZ, CPEng, Principal LGE Consulting
[2] BE, CMEngNZ, CPEng, Principal LGE Consulting

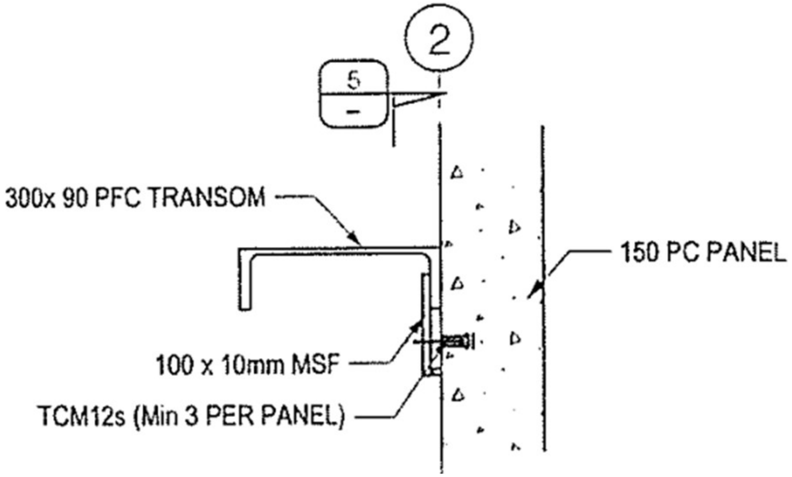
- Make sure you check drift!
- Low rise frames typically deflection controlled
- Even though capacity design may not be mandated, keep robustness in mind!
- Refer SESOC Journal article

- Adopt A Ductility For Steel Portal Frame Structure SESOC Journal Vol31 No1 APR 2018 Pdf

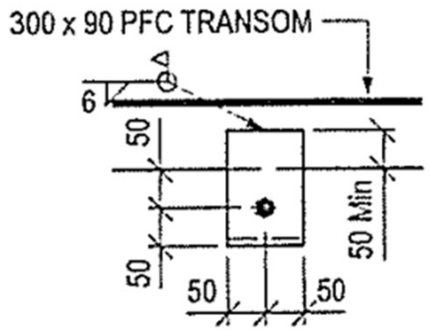
SESOC

What about collectors?

Precast Panel top connection - PFC Collector



- Spans 8.6m
- Supports 150 PC Panels 7m high
- Has no lateral restraint to inner flange
- Load is applied at the outer flange

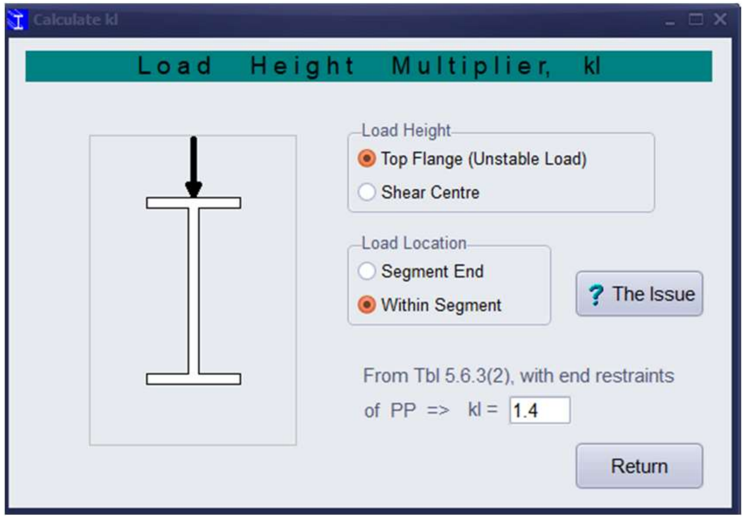


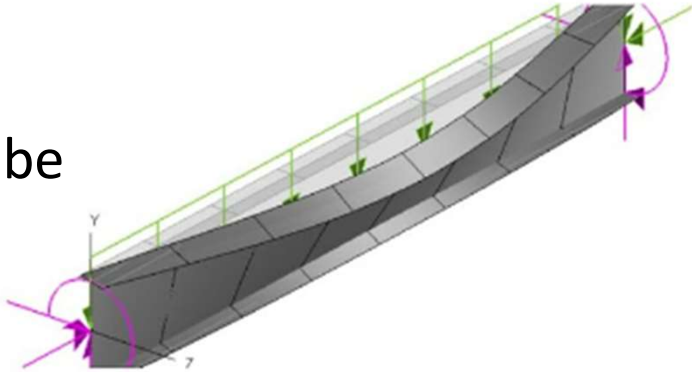
Connection to PFC highly eccentric!

Designer adopted μ=2 & went with a 300PFC

PFC Collector

- Singly symmetrical steel sections cannot be used in members requiring ductility (ie, don't use PFCs if you are adopting a ductility above 1.25!)
- Segment checks on the PFC were not completed (Use memdes)
- Load height factor was not considered (be careful when using memdes!)





PFC Collector – supports PC Panels OOP

μ=2, is limited ductile which needs a category 2 member

Note: You would not typically let a collector be a yielding component!

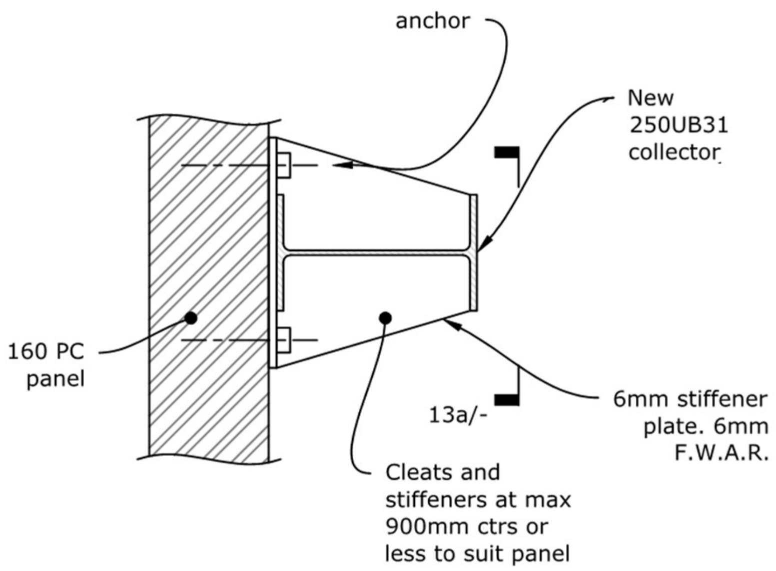
NZS3404 Clause 12.5.2.1

"The yielding regions of category 1 or 2 members shall be double symmetrical sections"

Implications

- PFC was undersized and inadequate to support the panels out-of-plane

What does good look like?



- Collector should have been designed as a part. Refer MBIE determination 2013/057

- Not structurally logical to assign this type of collector as a yielding element

Tip #5

If you ‘adopt a ductility’, make sure you can actually get ductility

Tip #6

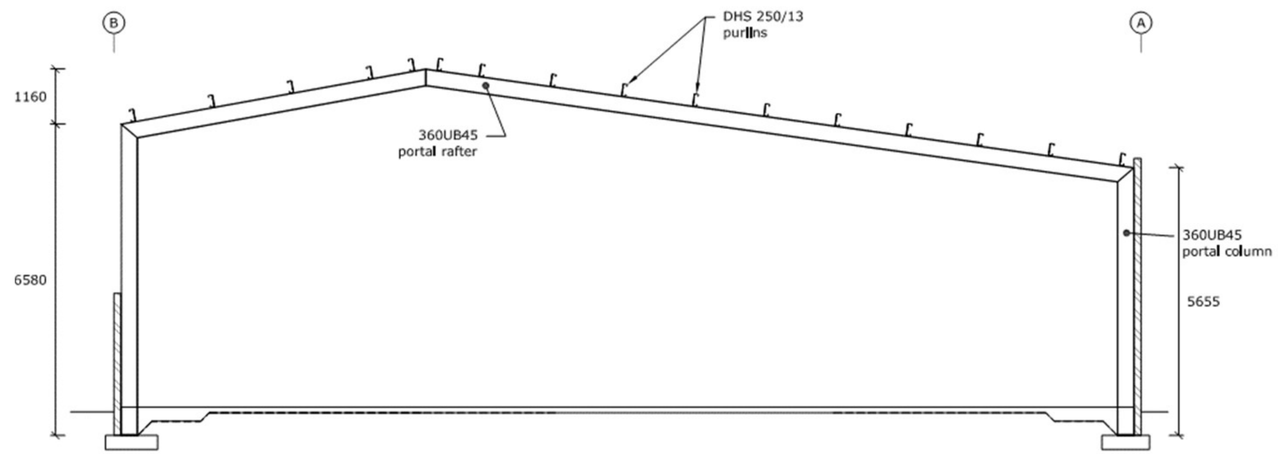
1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5	If you ‘adopt a ductility’ make sure you can actually get ductility	☑
6		

Tip #6

Do checks

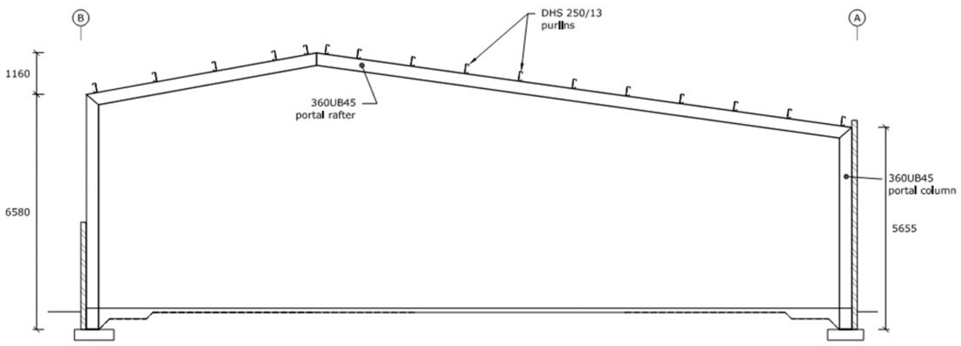
…check your maths, check your loads, check your base shears

Portal frame – weights & base shears



- Simple portal frame
- Weights to be resisted are
  - Roof purlins & roofing
  - Portal frame self weight
  - Roof struts/bracing
  - Tributary area of side walls

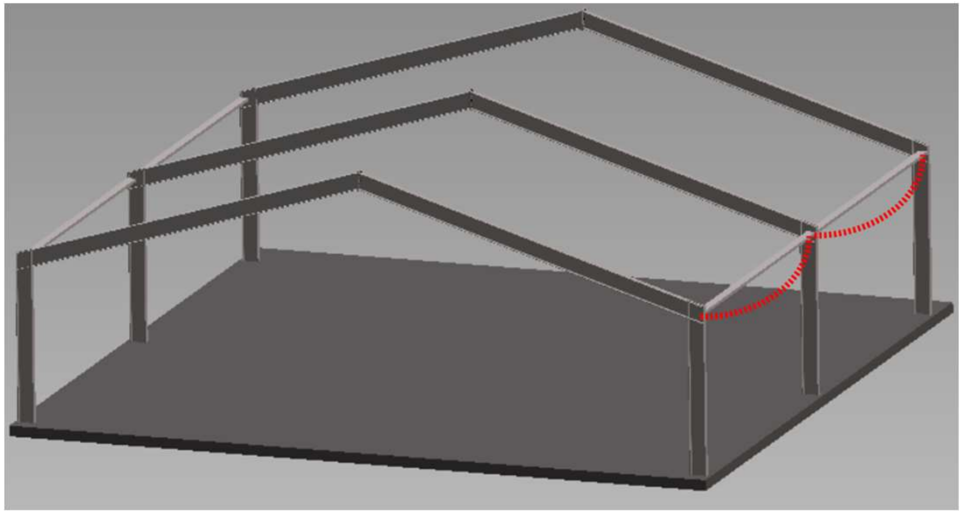
Portal frame sizing

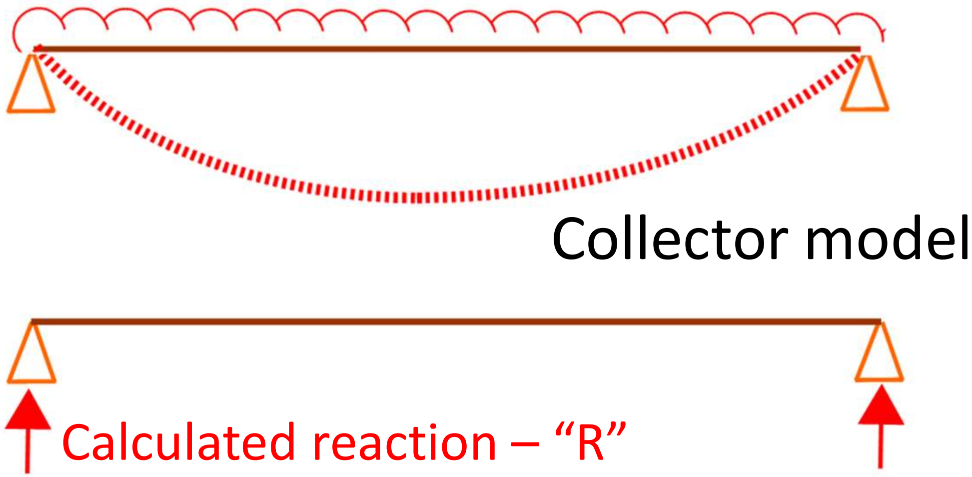


- For one building the frames were well undersized
- Typical portal frame, with precast panels supported at the top via a PFC Collector

Where did things go wrong?

Portal frame – where can things go wrong?

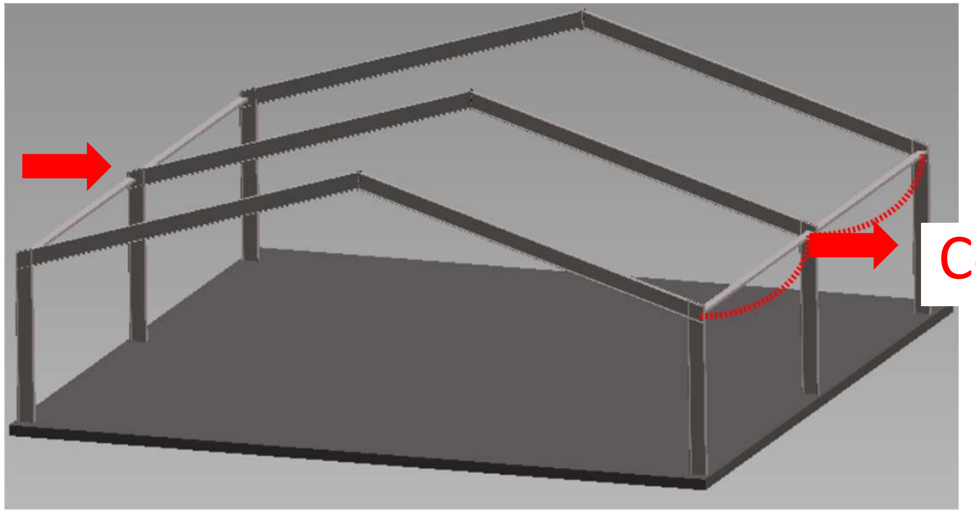




- Upon examination of the calculations…
- Collector checked with Cd(T)=0.412 (ie assuming μ=2)

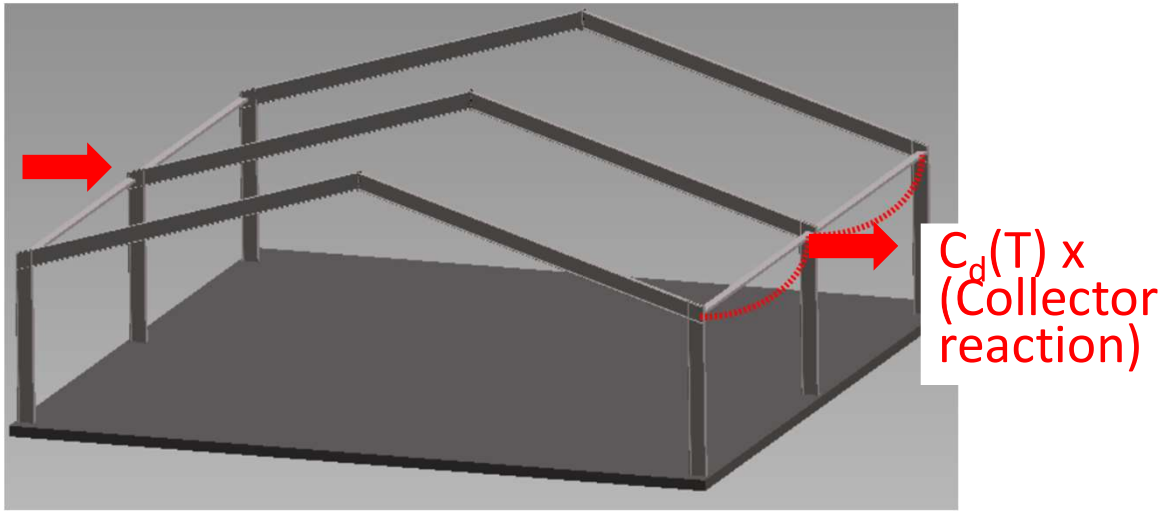
Note: this is incorrect – collector should have been designed as a part. Refer MBIE determination 2013/057.

Portal frame – where can things go wrong?



Reaction from the collector model was then applied to the portal frames

Portal frame – where can things go wrong?



- But these loads were then reduced again by 0.412
- So effective $C_d(T)=0.412 \times 0.412=0.24$

Implications

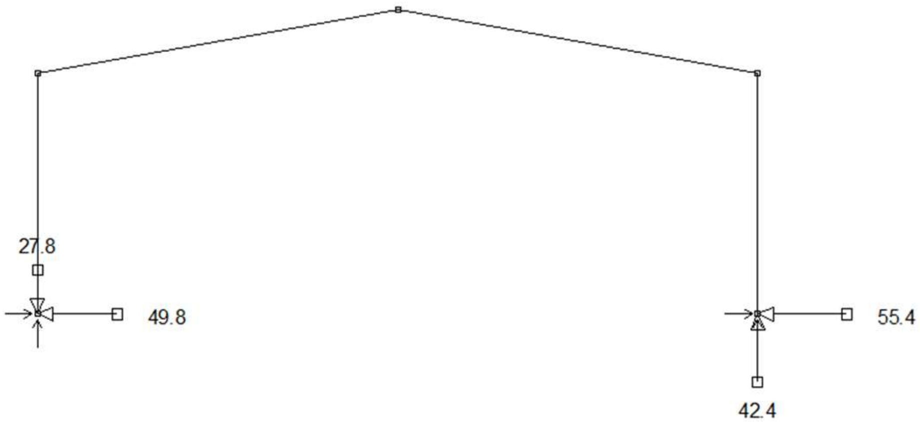
- The portal frame was grossly undersized for the loads it was required to resist
- Drifts exceeded allowable

Tip #6

Do checks

…check your maths, check your base shears

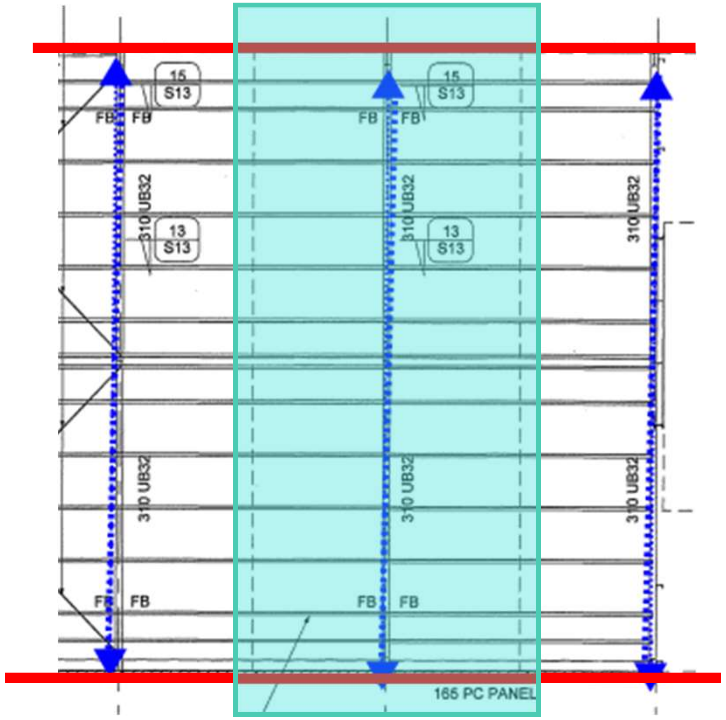
Back of the Envelope checks…

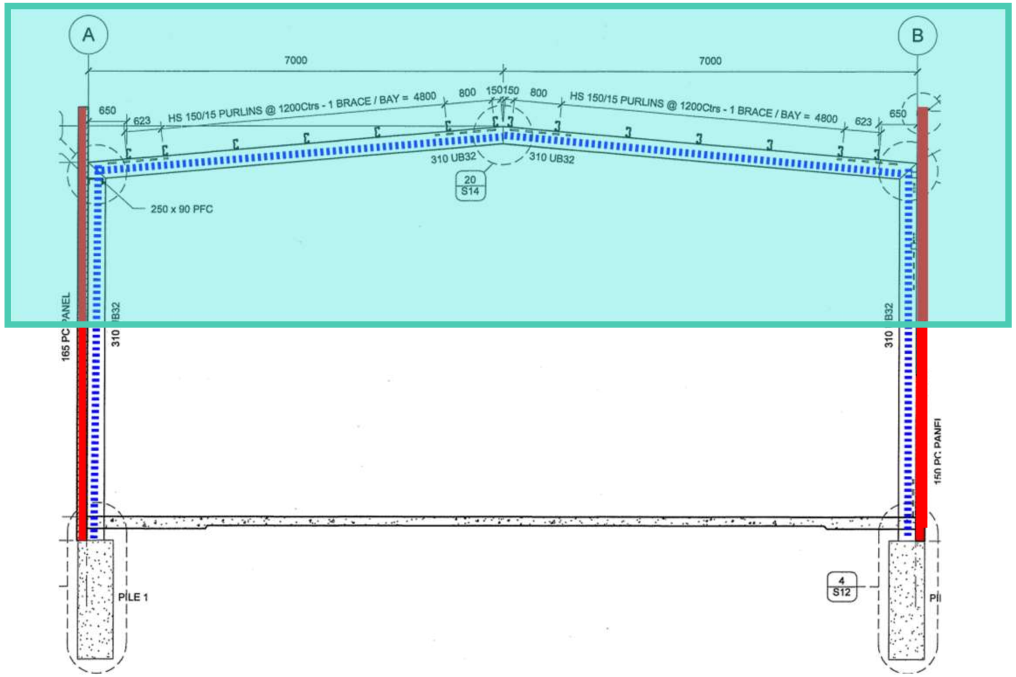


- Always a good idea to do some back of the envelope checks on loads
- Simple idea
  - Check your reactions – sum the base shears, then compare that with your weights
  - Do the numbers look about right?

SESOC

Trib Weight Check





- Quick calc to check weights/base shears

Tip #6

Do checks

…check your maths, check your base shears

Tip #7

1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5	If you 'adopt a ductility' make sure you can actually get ductility	☑
6	Do check ins – ie base shear total is right?	☑
7		

Tip #7

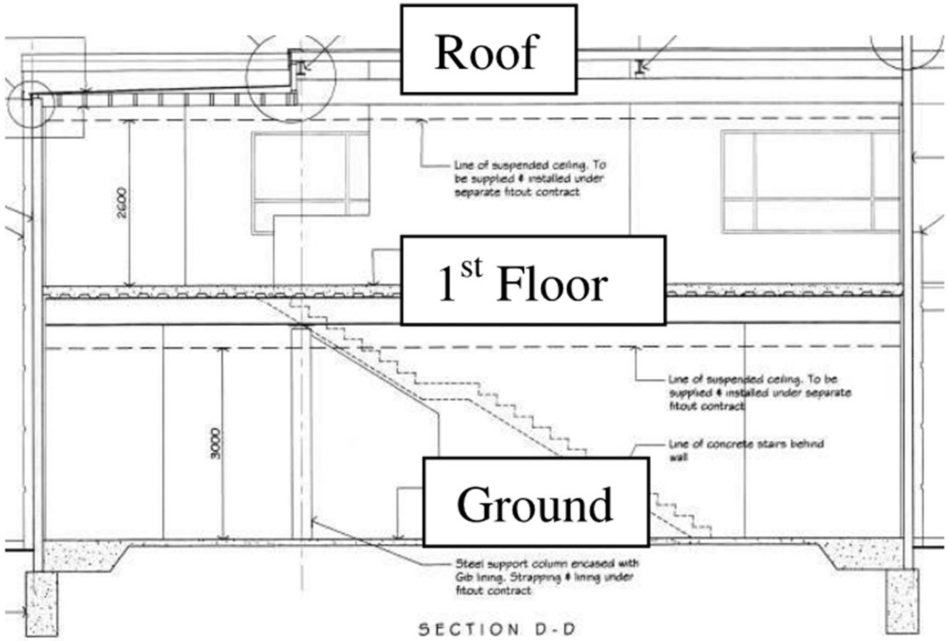
Sometimes you need to say no

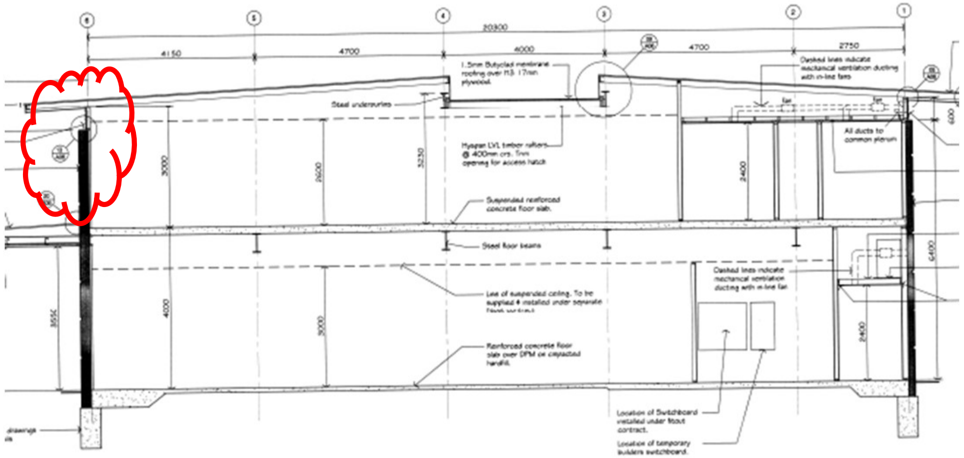
Or

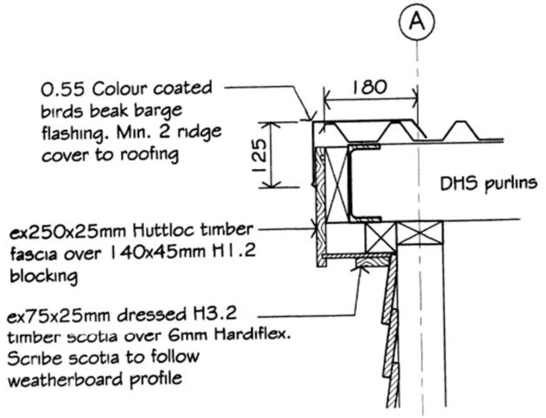
Early collaboration can save you some headaches!

Building F

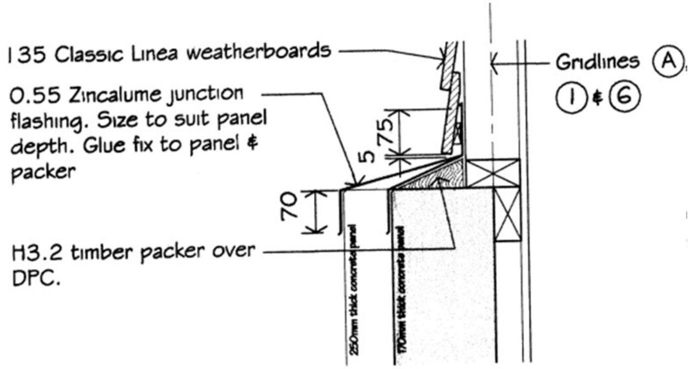

ASB
Masterton
ASB Bank


Roof
1st Floor
Ground
2600
3000
Line of suspended ceiling. To be supplied & installed under separate fitout contract
Line of suspended ceiling. To be supplied & installed under separate fitout contract
Line of concrete stairs behind wall
Steel support column encased with Gib lining. Strapping & lining under fitout contract
SECTION D-D





01 / A05 SOFFIT SCOTIA TO LINEA MAIN ROOF SOFFIT



13 / A04 TYPICAL HORIZONTAL JUNCTION TOP OF CONCRETE PANELS

- The architects details shows an external weatherboard for the top portion of the walls
- Precast panels were shown stopping short of the roof to accommodate this feature


ASB
Masterton
ASB
Bank

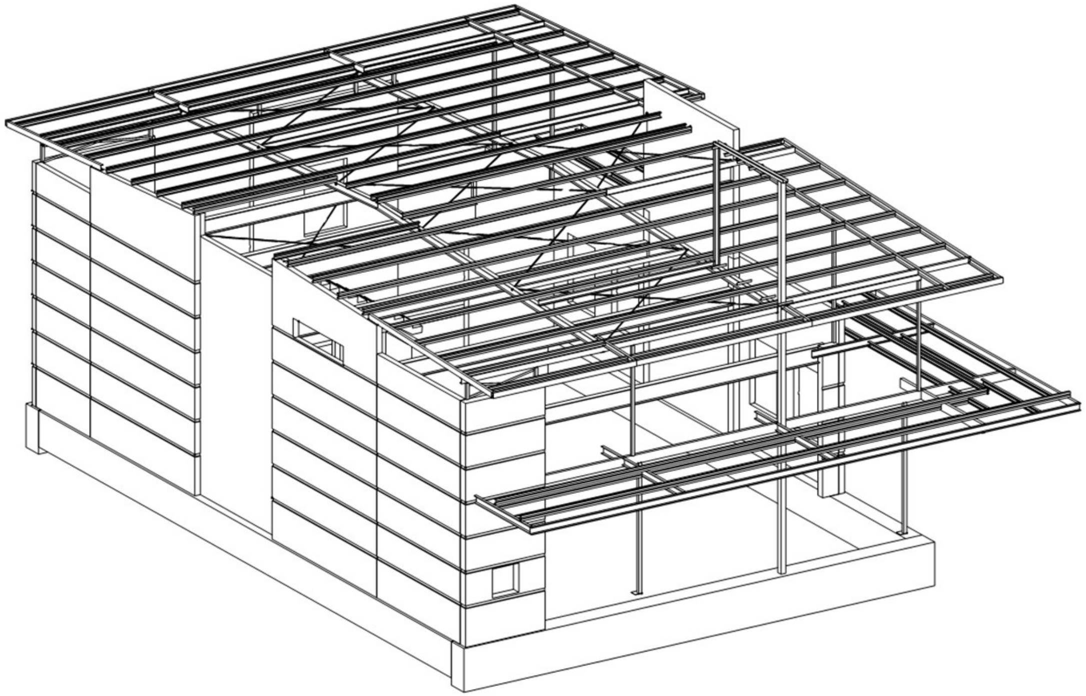

ASB

Fitting a round Structural peg into a square Architectural hole…

- What were the implications of running with this architectural details?

- Lets look at the structural system…

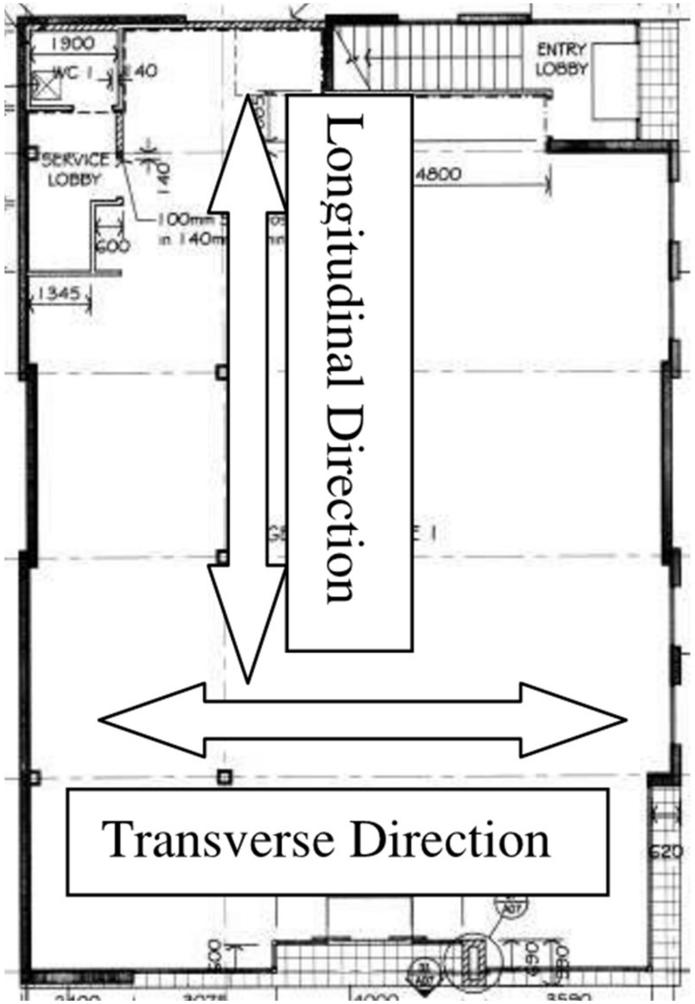
Building F

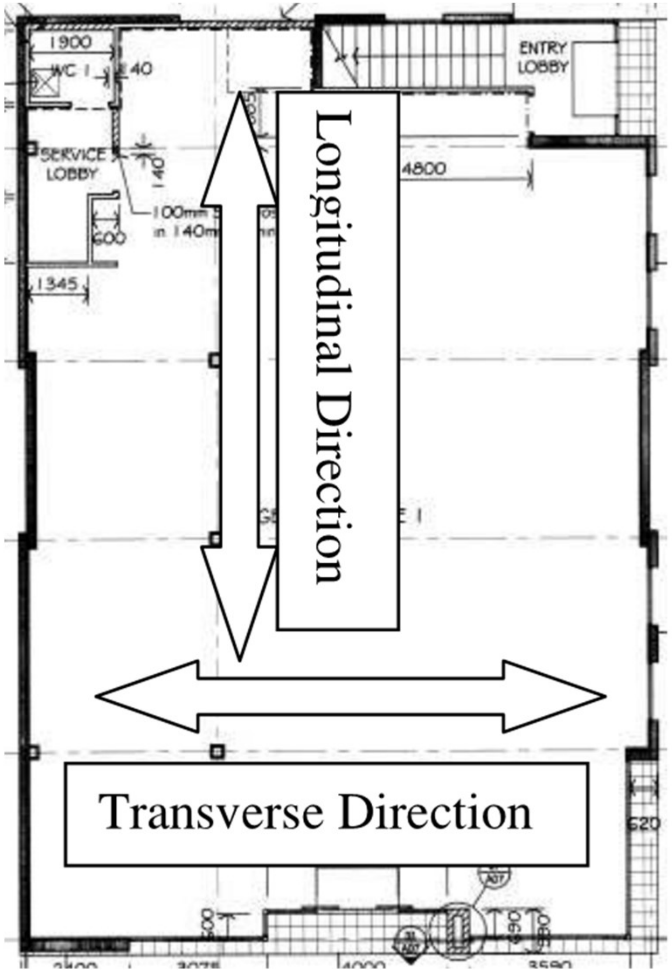


- Two storey rectangular structure
- 13.9m x 20.3m
- 170 thick precast panels
- Light weight roof
- Steel rafters to support roof purlins
- Roof plane tension bracing
- Suspended first floor slab

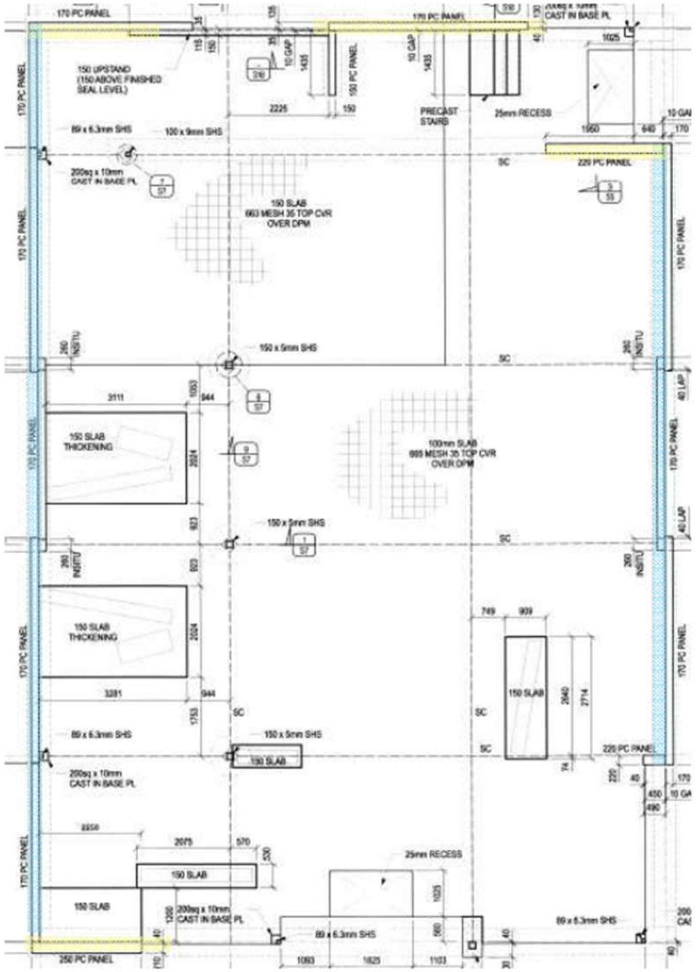
Seismic System

- Braced in both directions by precast panels acting in-plane

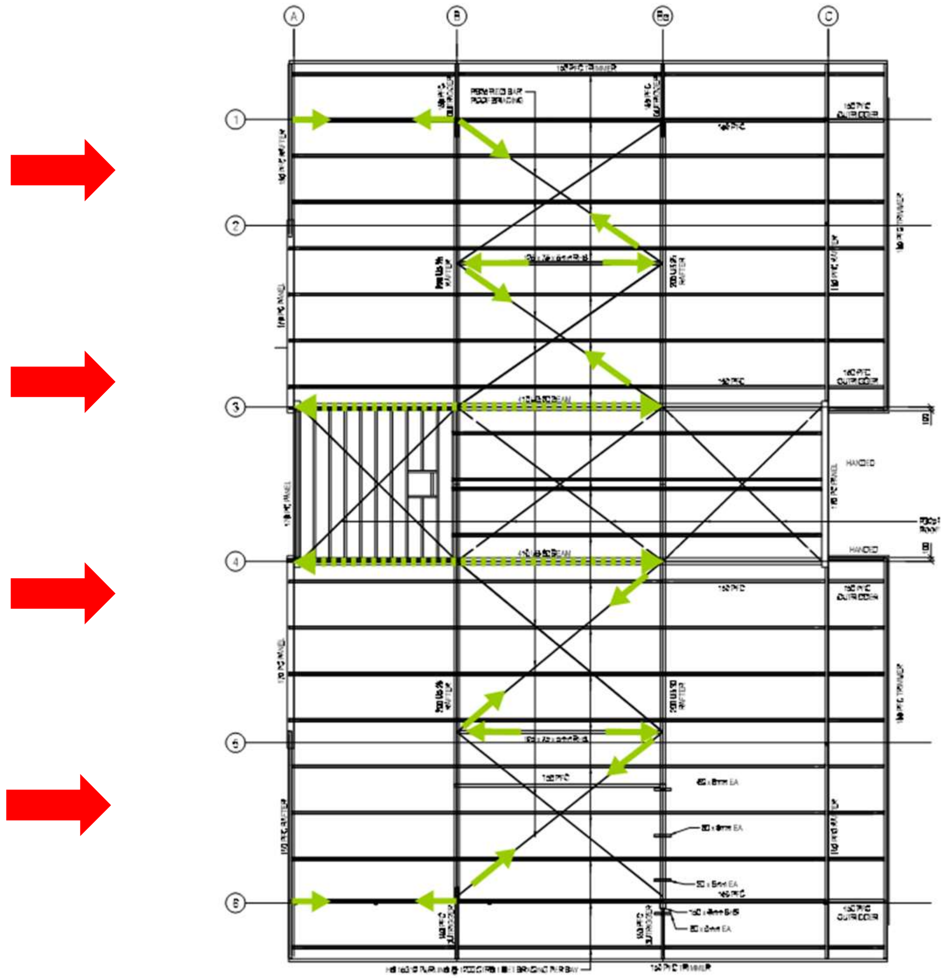




1900
WC 1
ENTRY LOBBY
SERVICE LOBBY
4800
1345
Longitudinal Direction
Transverse Direction



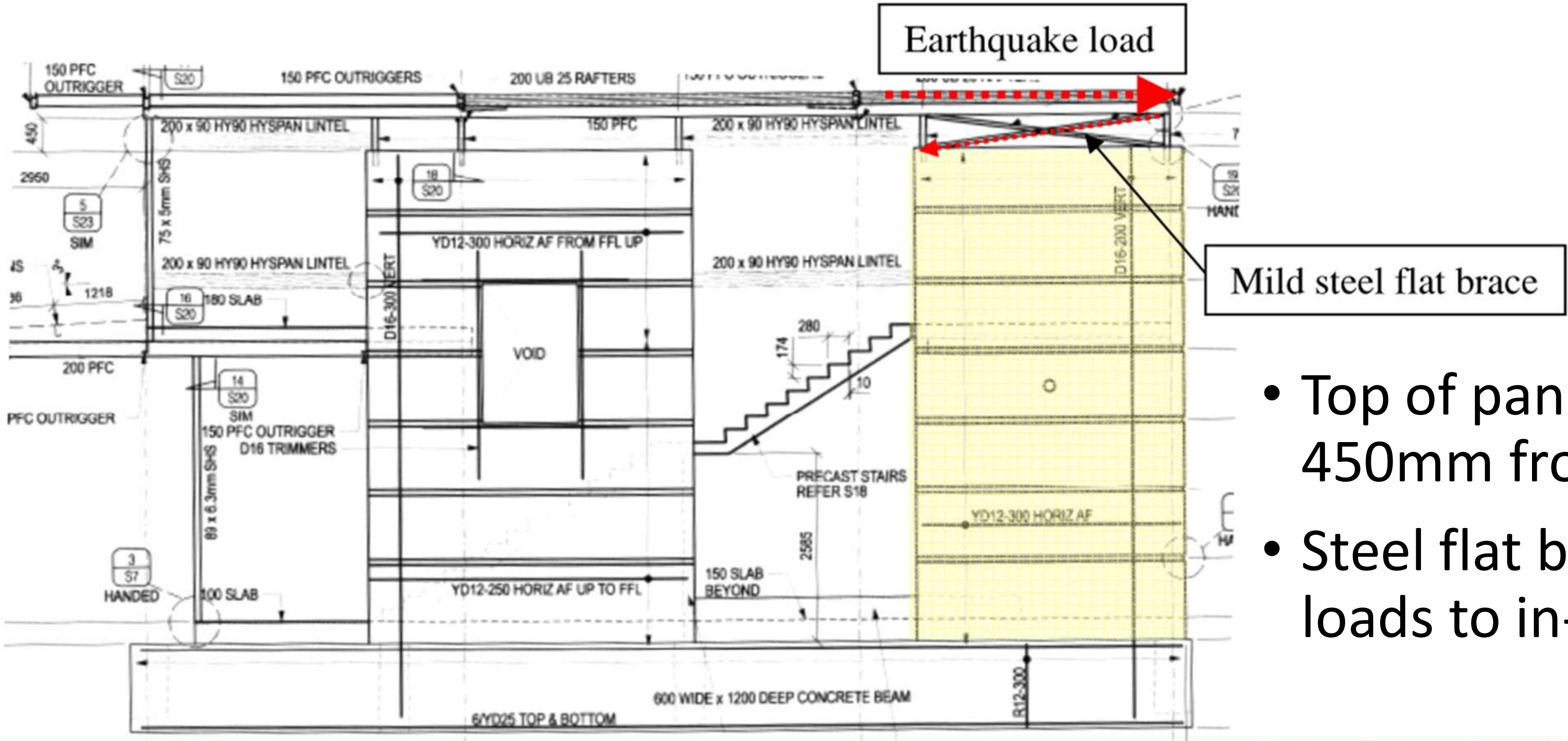
150 SLAB THICKENING
150 SLAB THICKENING
25mm RECESS



Transverse – Roof Plane

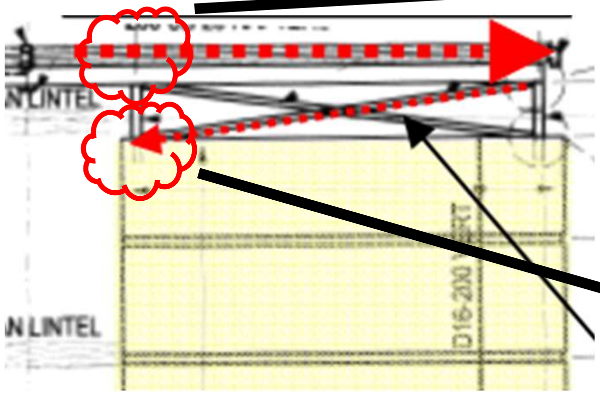
- Side wall panels propped via steel roof beams
- Load transferred to tension bracing
- Tension bracing transfer loads to strut on side wall
- Precast panel on side walls are 450mm down from roof level

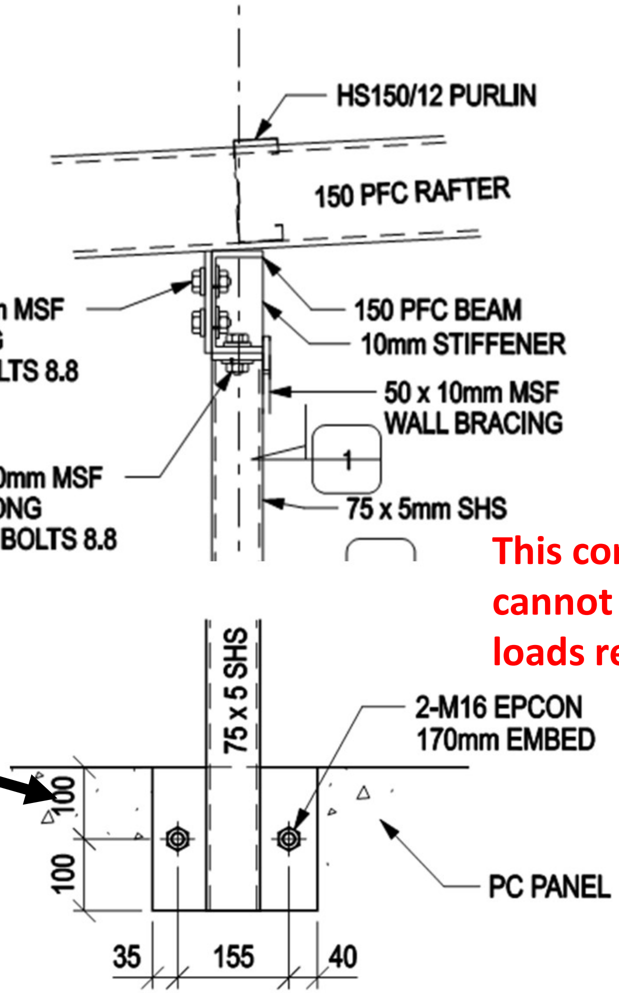
Transverse – roof plane to wall in-plane



- Top of panels down 450mm from roof plane
- Steel flat brace transfers loads to in-plane panel

Transverse – Connections





This connection cannot transfer the loads required

- No details for flat connection
- Flat is too small for load required to be transferred
- Connection is eccentric
- Two M16 epoxied bolts into the PC panel are not sufficient

Tip 4
Connections are critical

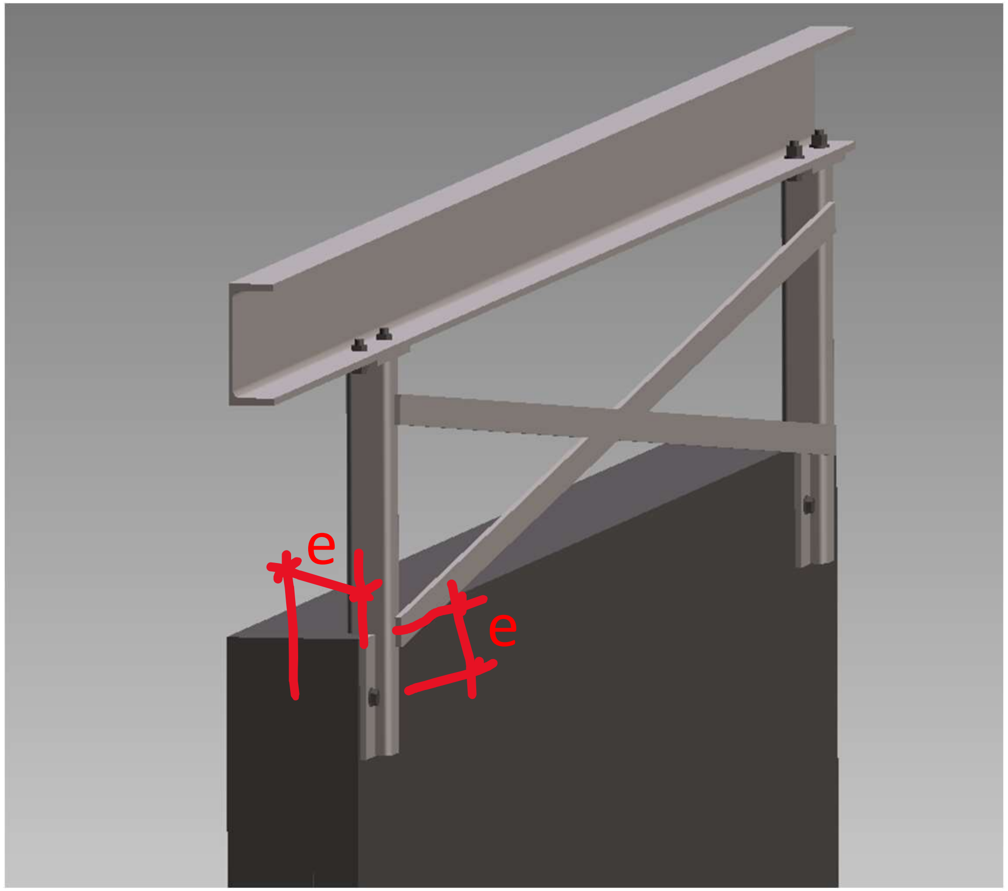
- Strip away the architectural

Eccentric connection

Focusing on one component of this load path – brace to PC Panel

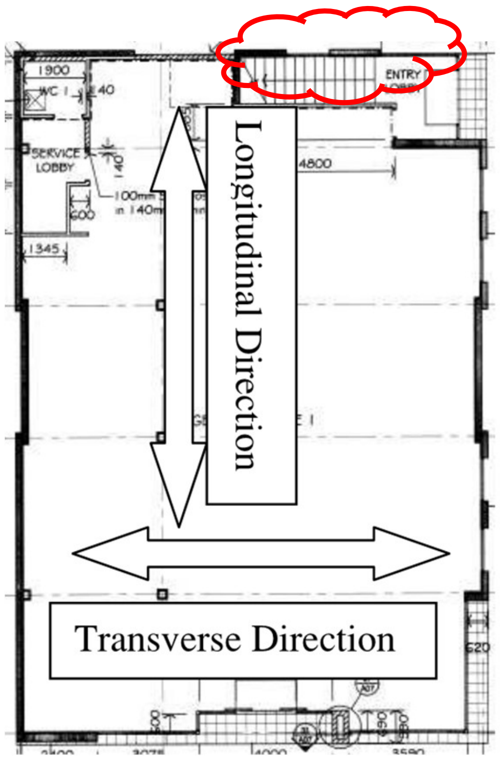
Vertical eccentricities – tension in steel flat is offset from bolt group

Horizontal eccentricates – Applied bolt shear is offset from panel centreline



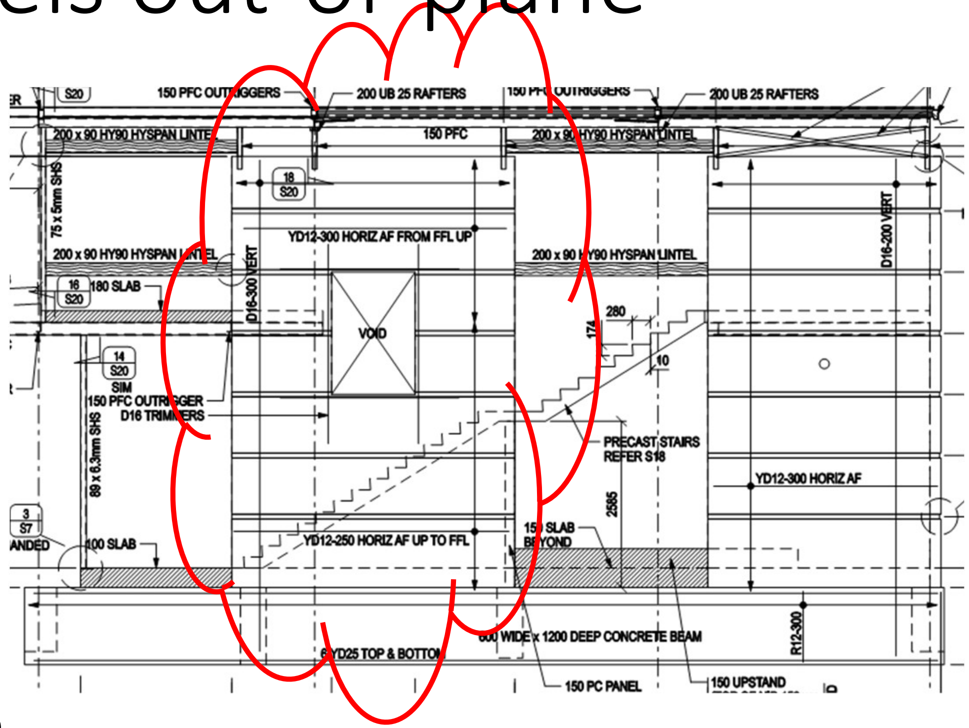
Already a poor load path, made worse…

Longitudinal –End Panels out-of-plane



Plan

- Out-of-plane support



Elevation of End Wall

Longitudinal – End Panels out-of-plane

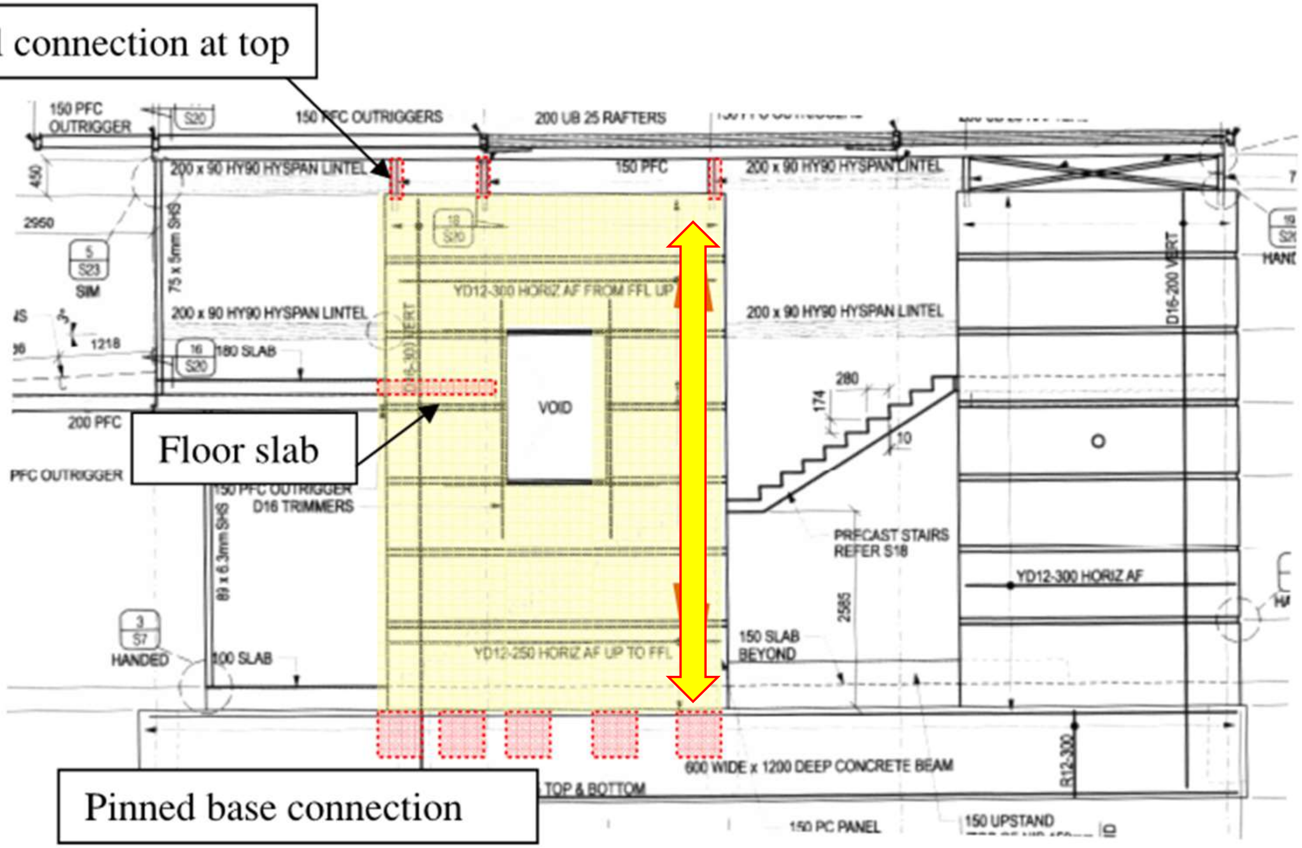


Figure 8 – Elevation of rear wall (Grid 6)

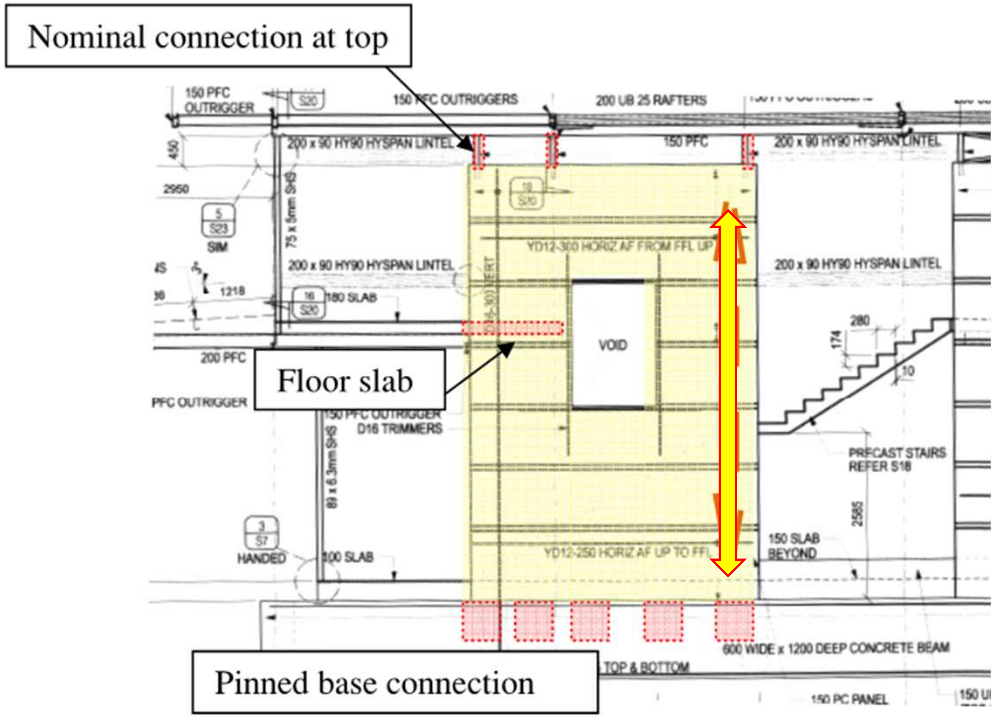


Figure 8 – Elevation of rear wall (Grid 6)

Out-of-plane

What is securing this panel out-of-plane?

- Pinned base connection to footings
- Partial connection to the first floor slab plate one side of the window
- Roof level – three SHS members?

Panel out-of-plane load path

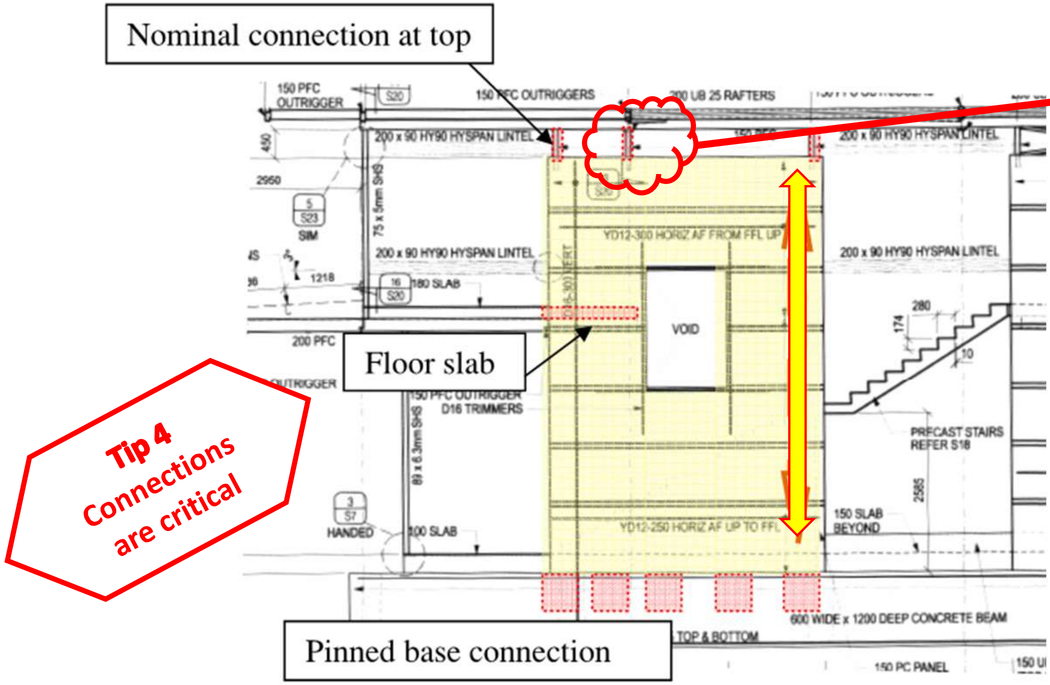
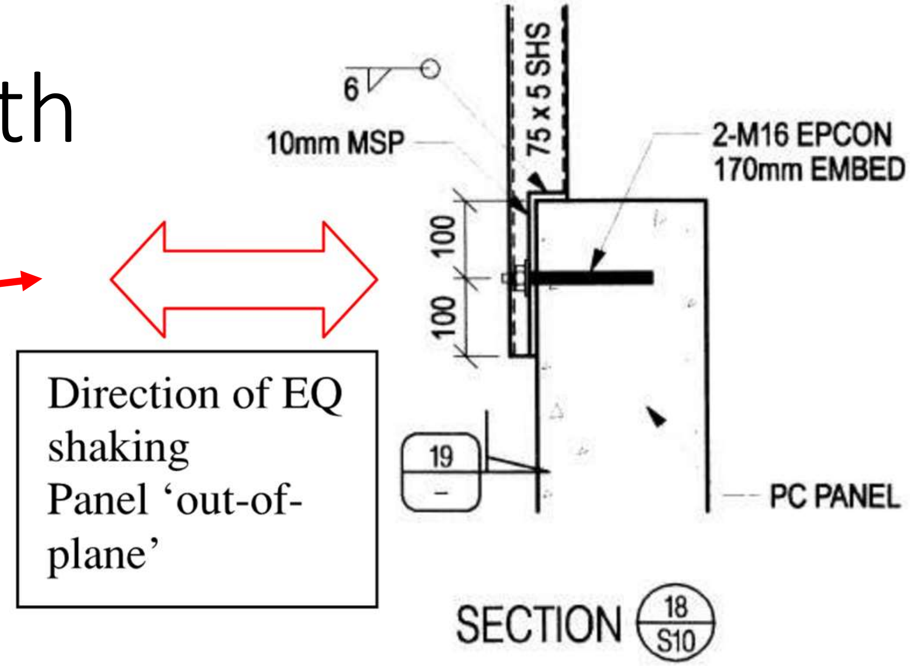


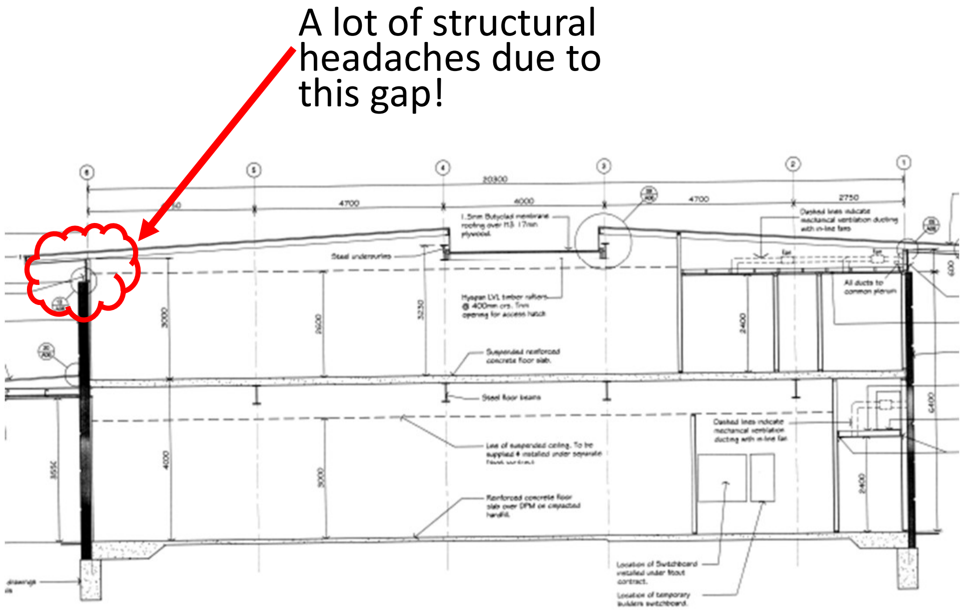
Figure 8 – Elevation of rear wall (Grid 6)



- No load path to roof plane bracing
- Connection at the top of panel has not been designed for out-of-plane forces – bolts would pry, SHS would bend

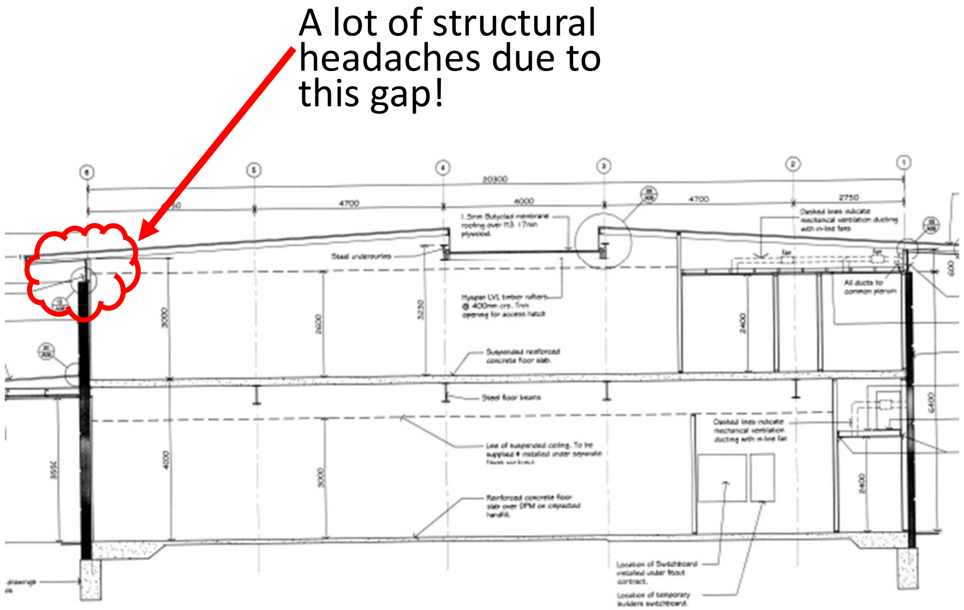
This connection cannot transfer the loads required

The implications of the architectural feature?



- Architect left a very small space both vertically and horizontally to fit in primary structure
- Made the load path more complex
- Adds several unnecessary potential weak links in the chain

How could this have been kept simple?



- Finish the load path!
- The panel could have been extended up to the roof plane – removing the eccentricities and providing a direct load path for the transfer of in-plane roof bracing loads, and out-of-plane support of the panels
- Early collaboration with the architect may have been able to make the engineers life a lot easier

Tip #7

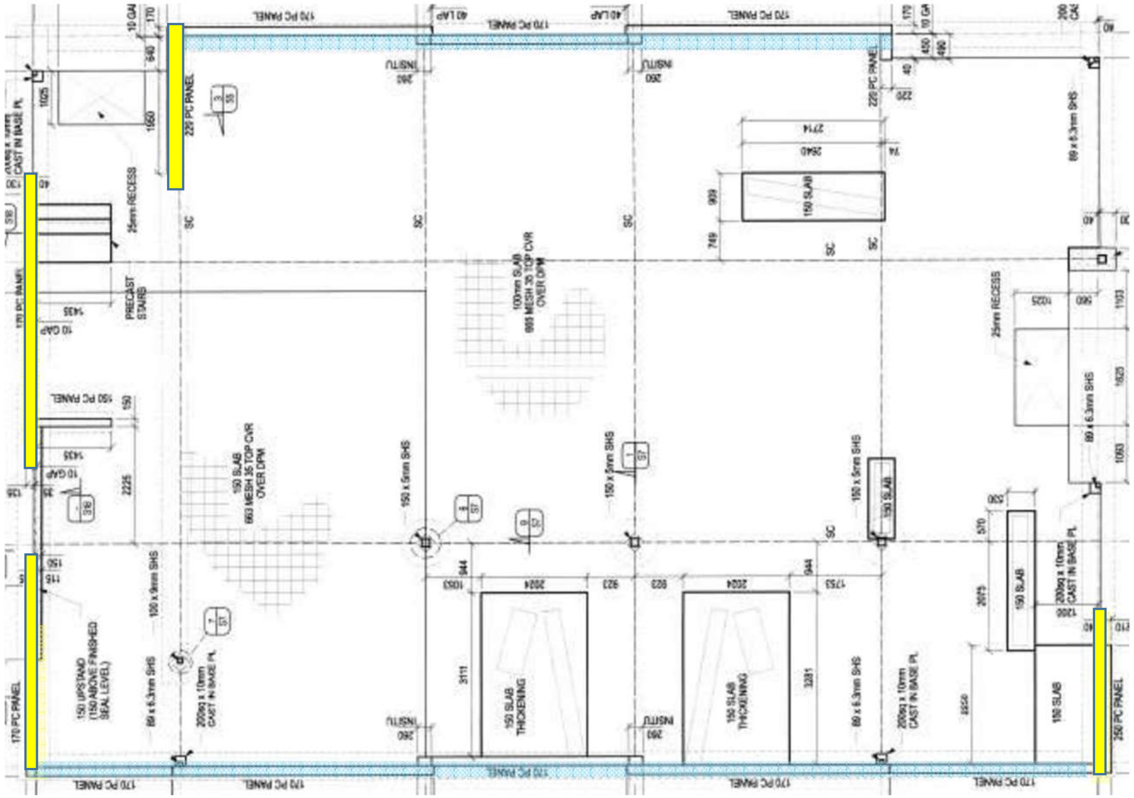
Sometimes you need to say no

Or

Early collaboration can save you some headaches!

Another example

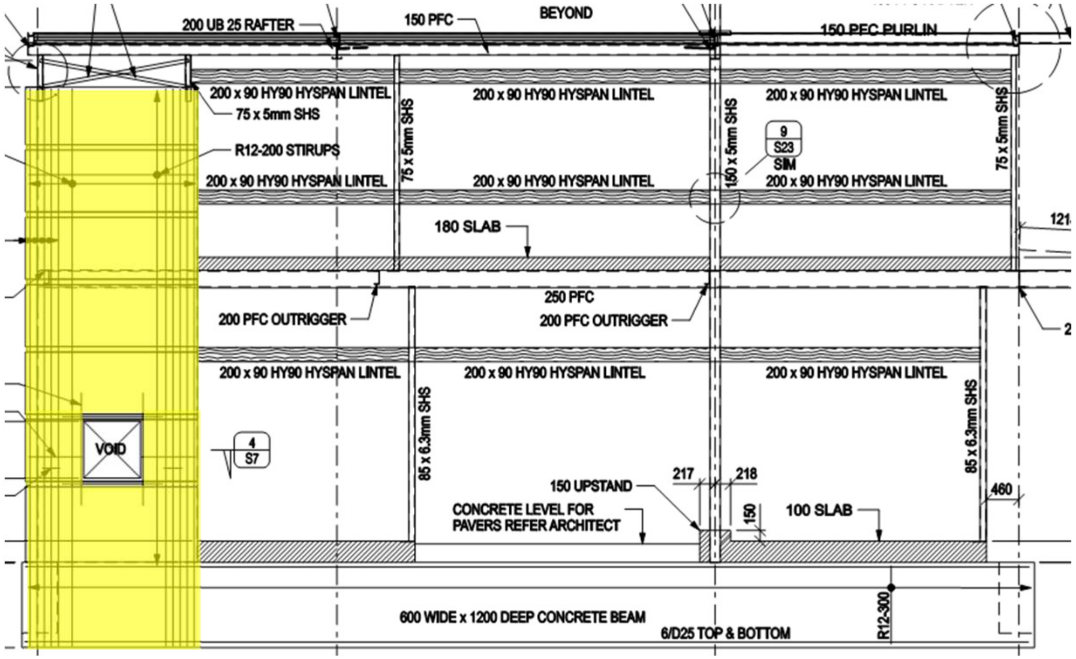
Sometimes a ‘No’ is needed



Transverse System

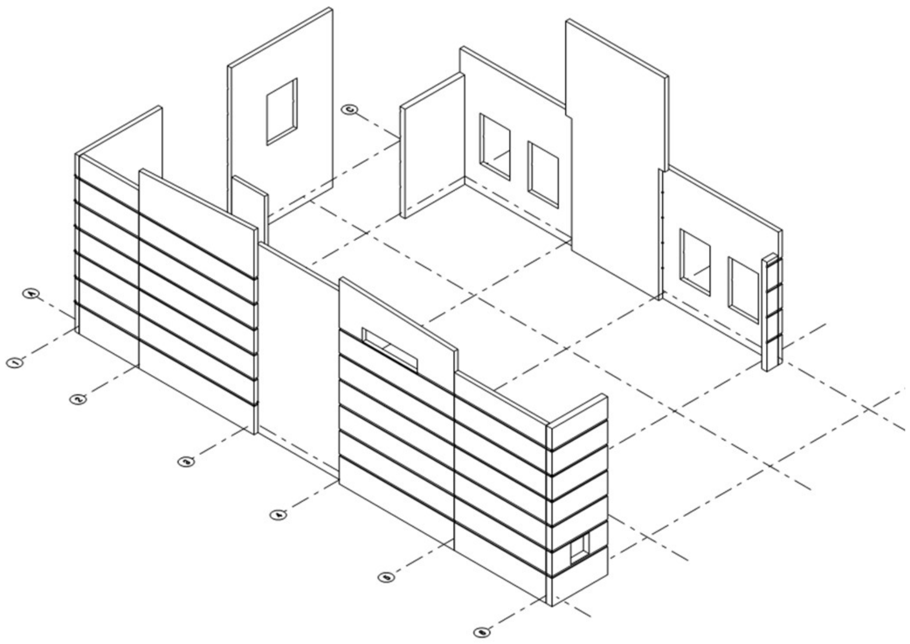
- Loads are resisted by precast panels acting in-plane
- Not many panels in total
- And of these panels, only one panel in plane towards the front half of the building

Single panel bracing front half of building



- Rest is glazed
- Void accommodated on request of architect
- During construction void size was realised to be inadequate

Single panel bracing front half of building



- Additional cutting of the concrete was approved
- Reinforcing & concrete cut
- Critical load path, with little redundancy already, allowing an additional reduction in strength….sometimes we have to say no on site!

Tip #7

Sometimes you need to say no

Or

Early collaboration can save you some headaches!

What does good look like?

- Solving problems is what being an engineer is about
- Sometimes we can be flexible, but sometimes it has to be a ‘no’
- Don’t make life hard for yourself – keep the structure as simple as it can be
- Advocate for structure to be where it needs to be
- Harder to design – typically will be harder to build, with less ability to work with minor construction tolerances (errors!) on site

Tip #7

Sometimes you need to say no

Or

Early collaboration can save you some headaches!

Tip #8

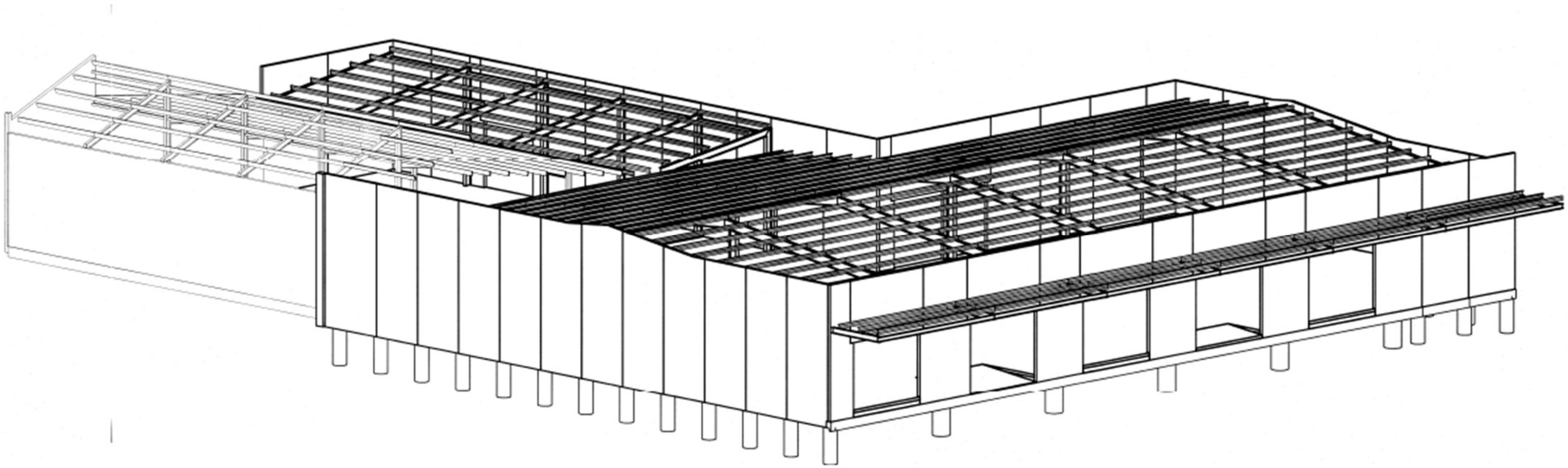
1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5	If you 'adopt a ductility' make sure you can actually get ductility	☑
6	Do check ins – ie base shear total is right?	☑
7	Sometimes you need to say no	☑
8		

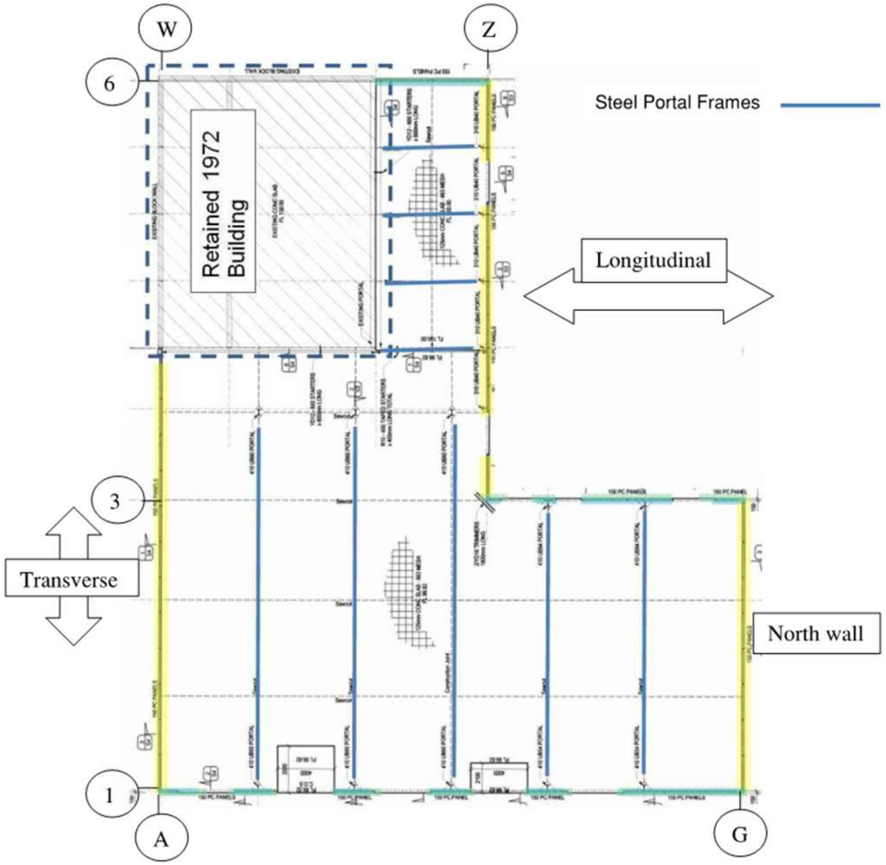
Tip #8

Make sure you co-ordinate structural with architectural

SESOC

Building G



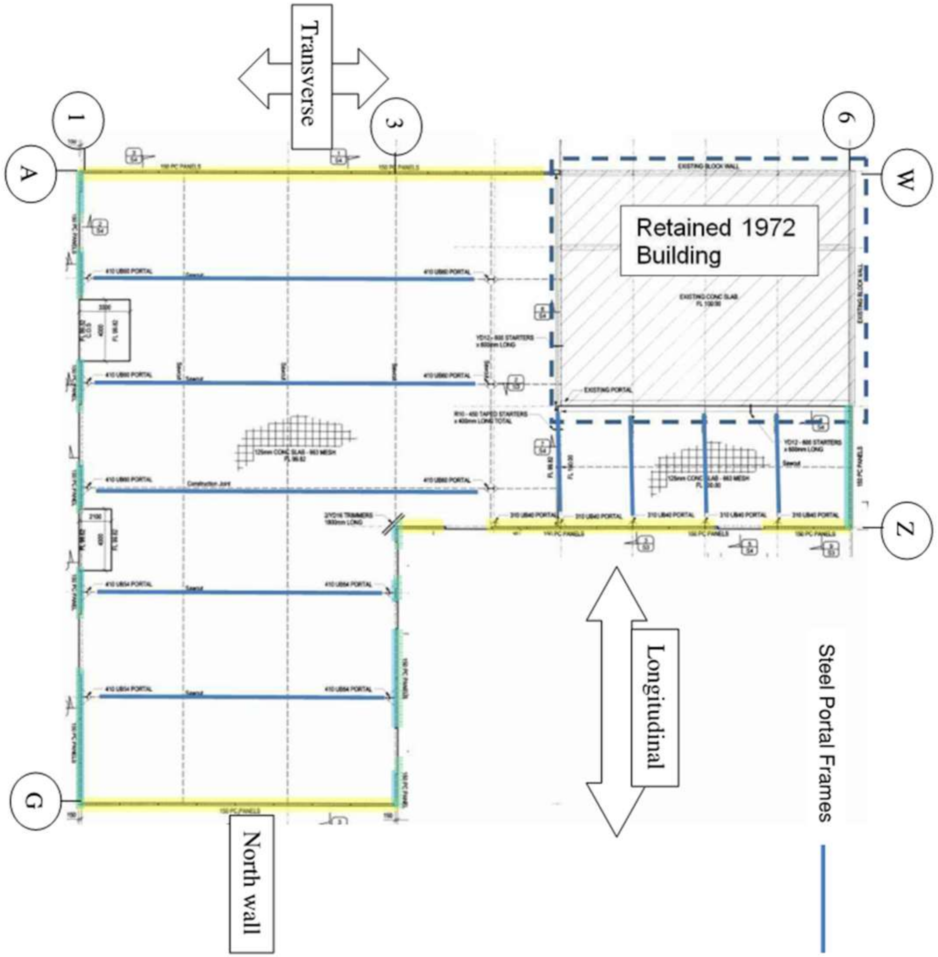


Building G

- Single storey L-shaped structure
- 410UB60 Steel portal frames at 6.8m centres spanning 26.9m
- 150 thick precast panels
- Light weight roof
- Roof plane tension bracing


HERITAGE BOOKS
THE BUNKER
ASB


KOMATSU
KOMATSU 45
LOADER



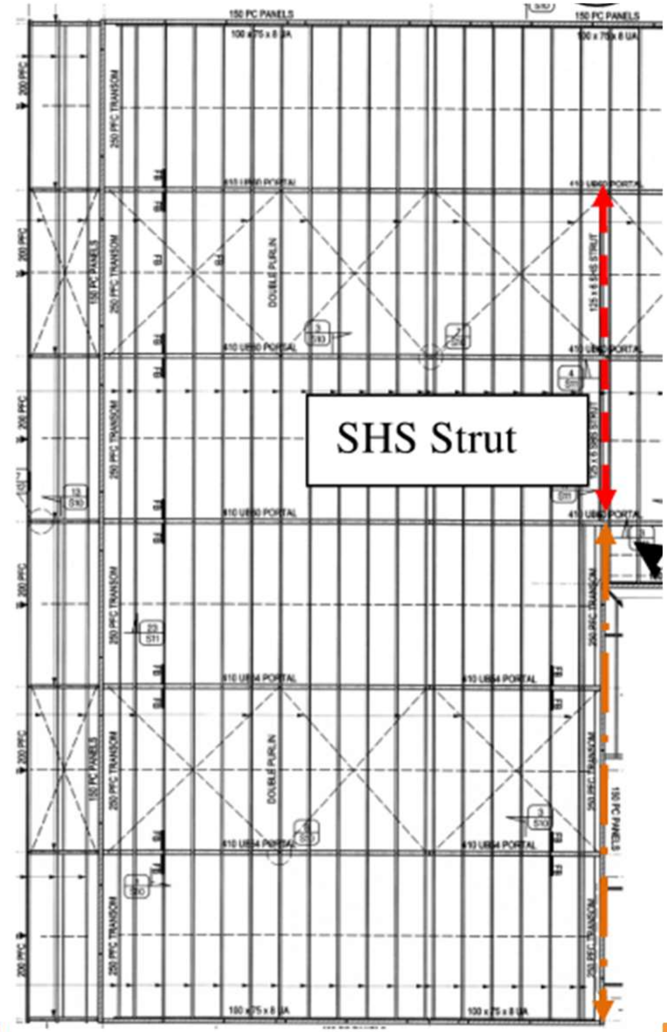
Seismic System

Transverse

- Steel portal frames
- In-plane precast panels at end walls

Longitudinal

- Roof plane bracing transfer
- In-plane precast panels Grid 1 & 2

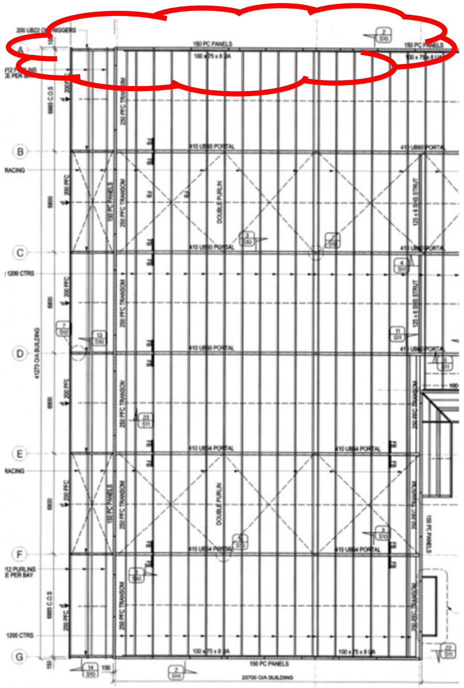


Longitudinal System

Load Path

- End panels propped by purlins
- Purlins transfer loads to rafters
- Rafters bend in weak direction
- Roof plane bracing transfers loads to the sides
- One side is an SHS strut, the to panels in plane
- Other side is to panels in-plane

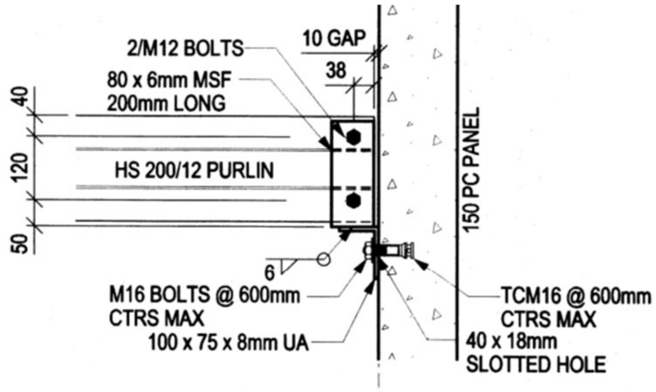
Longitudinal System



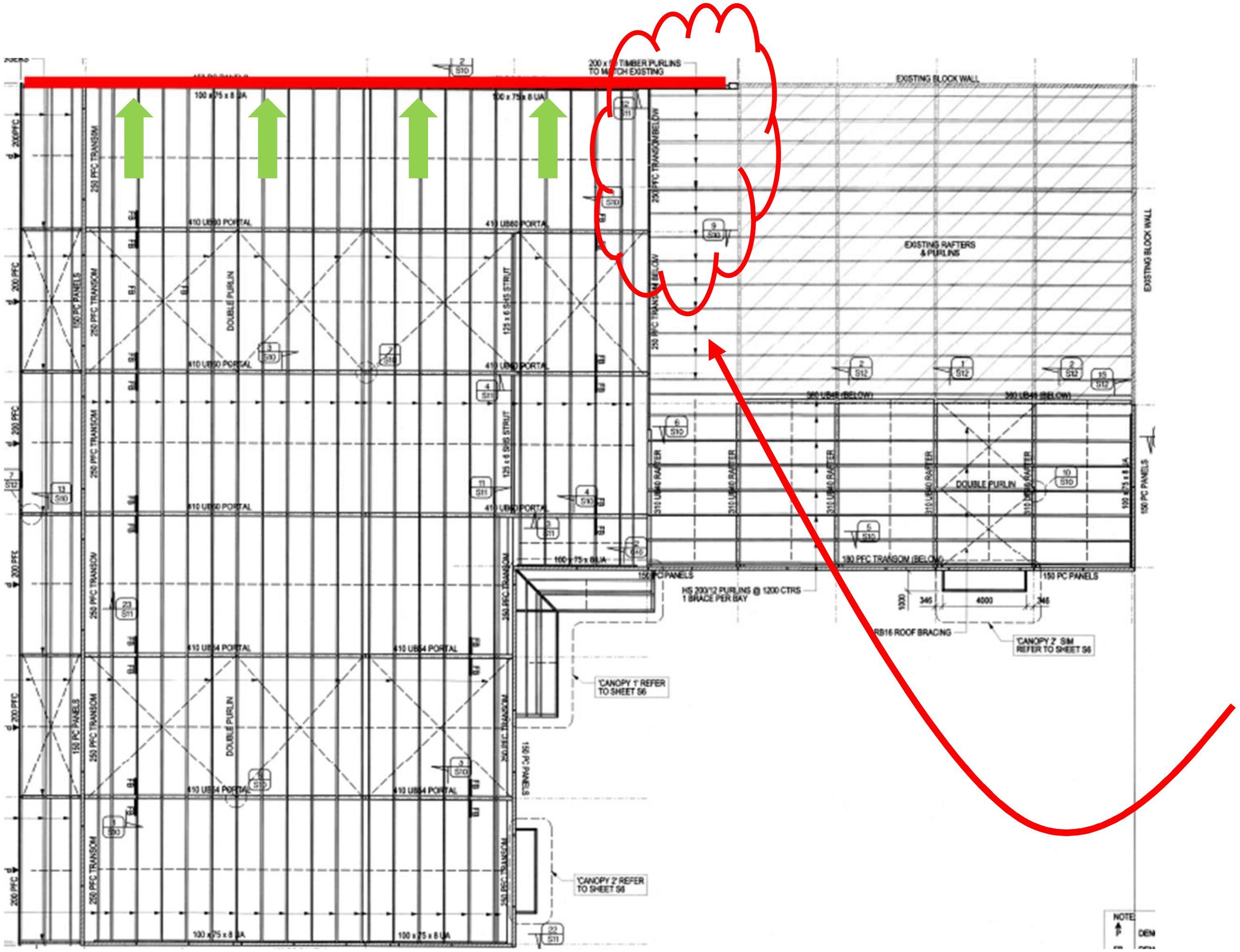
Gable end wall

- Precast panels propped by DHS purlins
- Same issues as previous buildings…

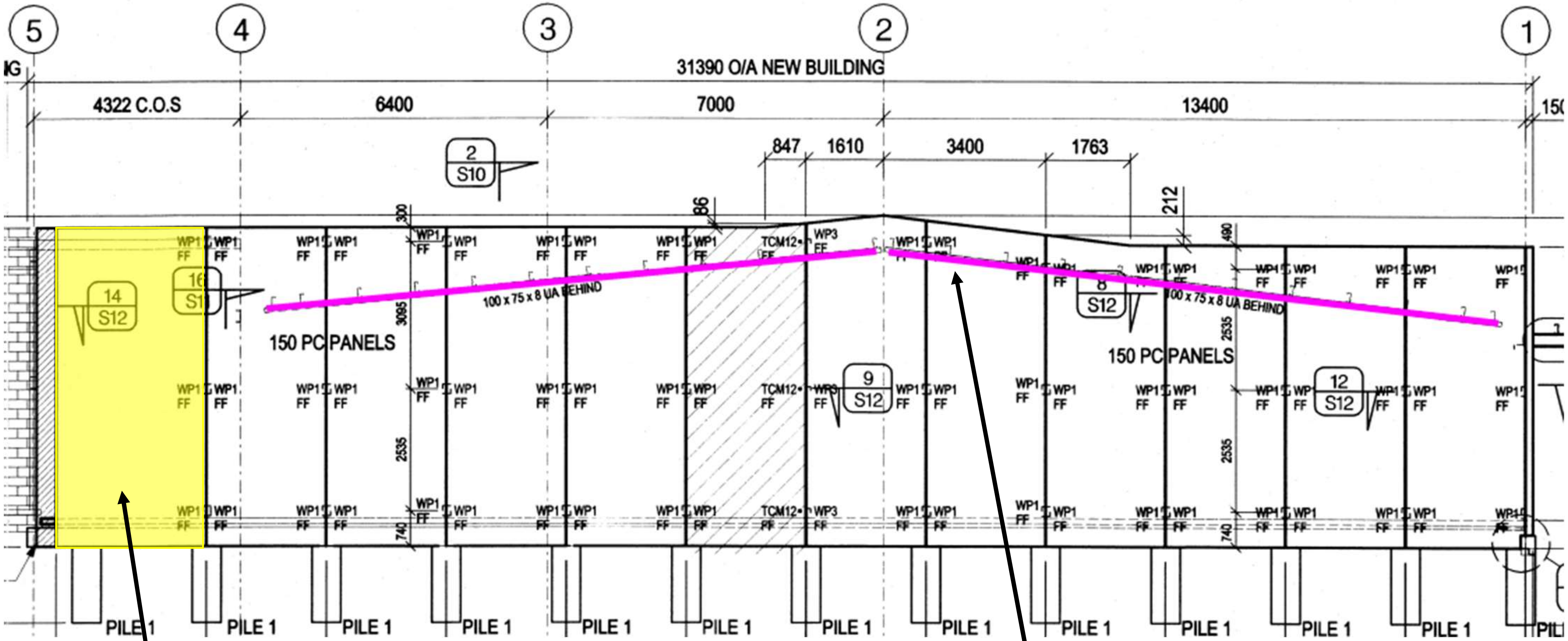
…Apart from one quirk



This connection cannot transfer the loads required



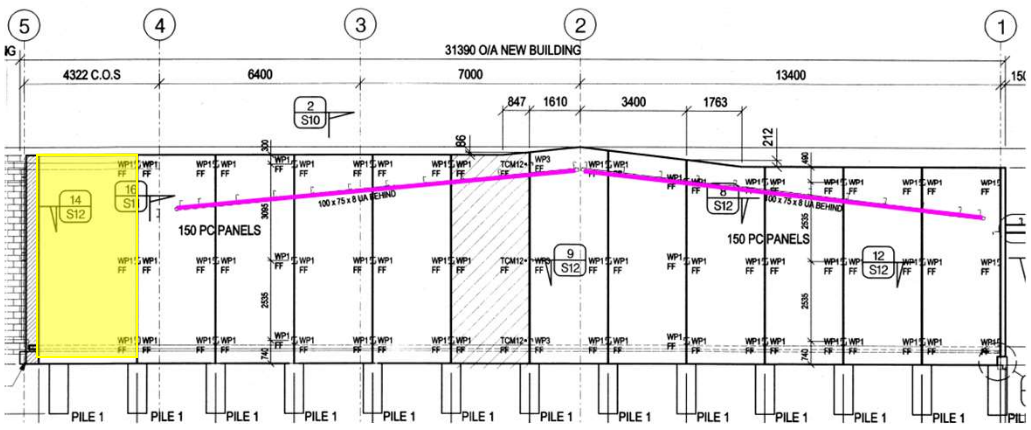
- Gable end wall precast concrete panels
- Propped by the DHS purlins
- But there is a length of panel which is at a transition where the purlins are parallel to the wall



But what is supporting this panel out-of-plane?

Line of EA connected to DHS purlins

Hold onto all your bits!

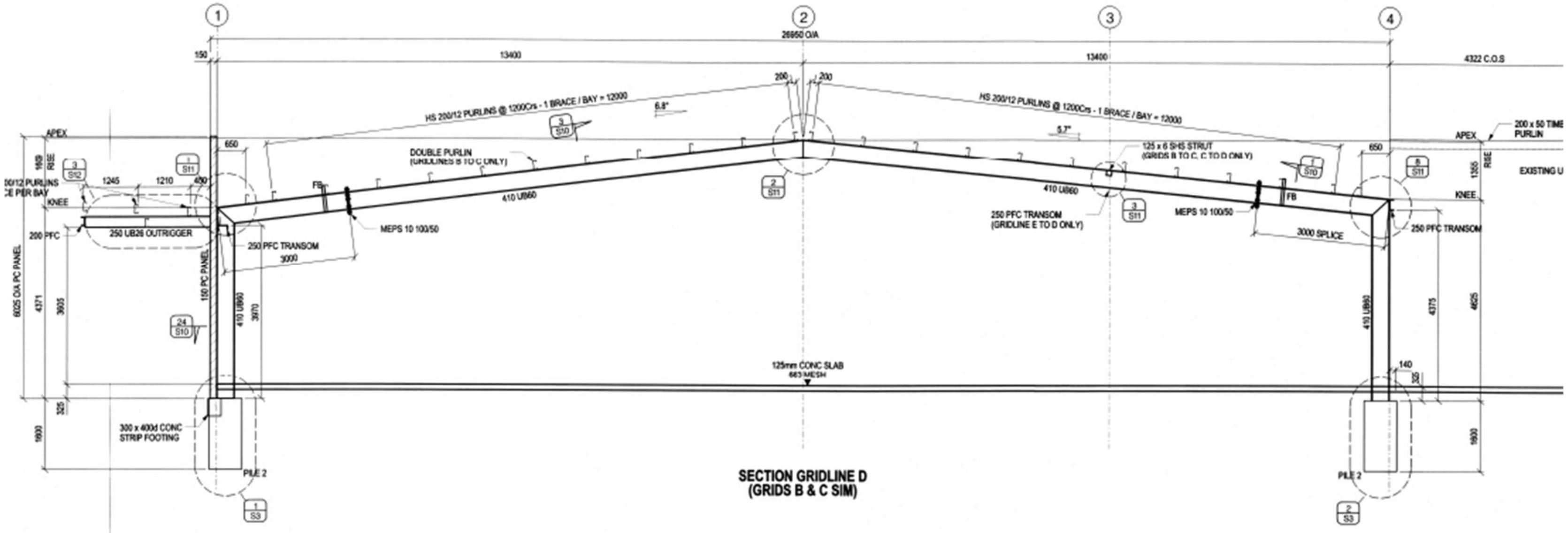


- 6.7m high precast panel with no support out-of-plane

Tip 2 – Make sure you have a load path

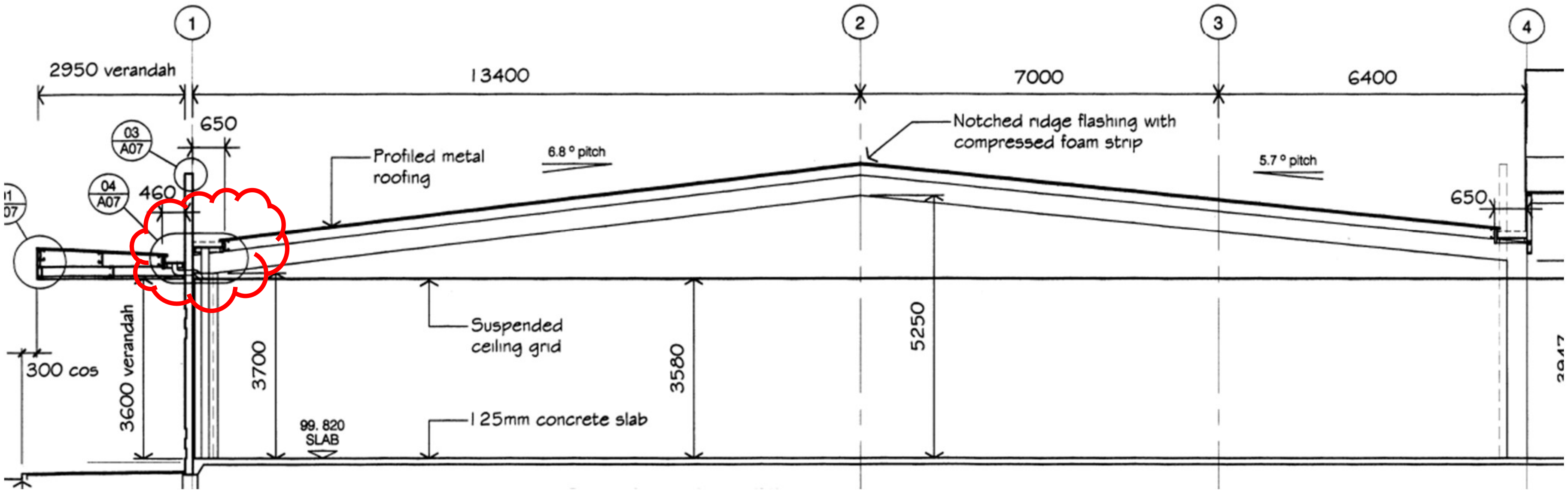
Lets look at some structural/architectural interfaces…

Transverse Steel Portal Frames

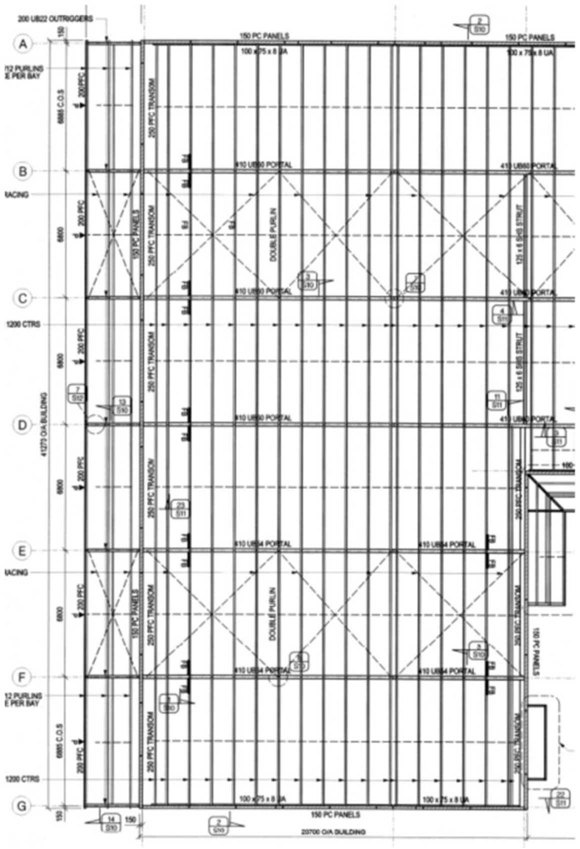


- 410UB60 portal frames

Architectural Requirements



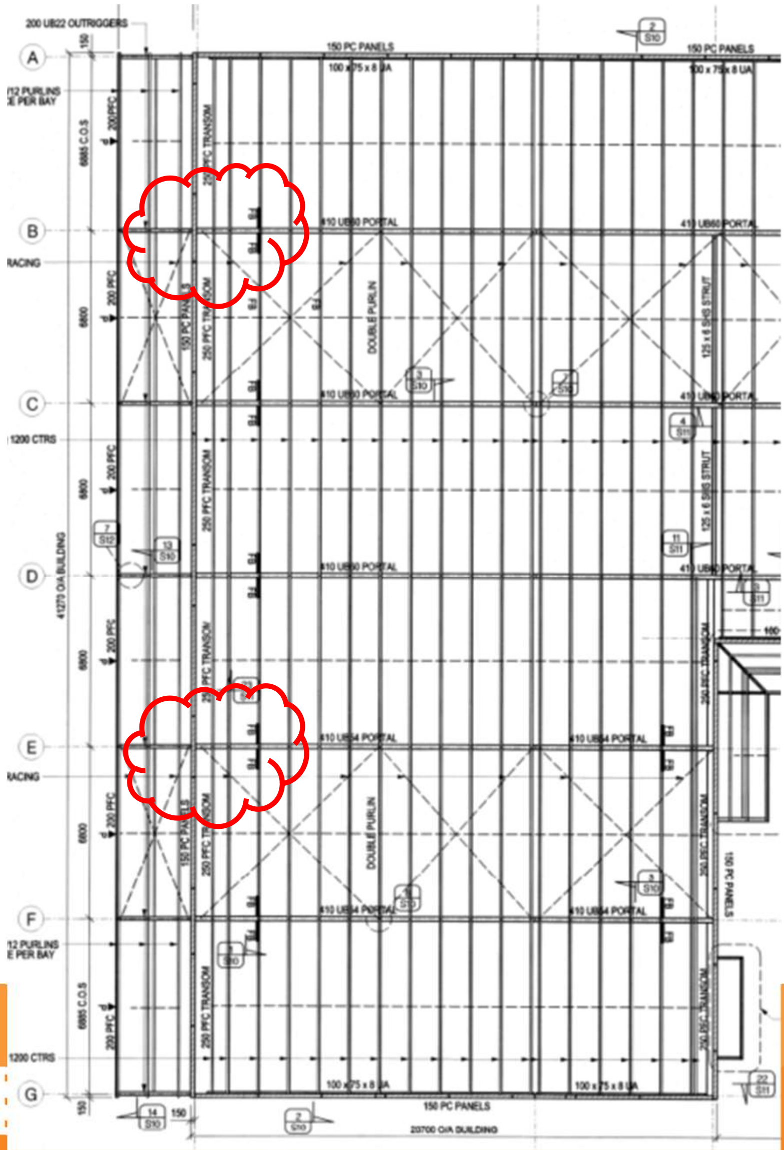
- Box gutter along this wall line
- Falls over the 6 metres between portal frames, with a downpipe at each frame



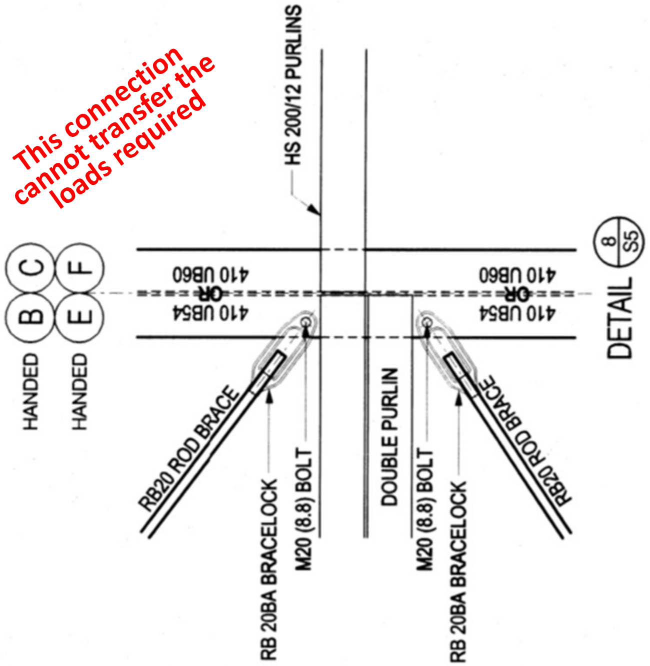
Lets look at how the structure fits with the architectural

- Roof plane bracing system transfers seismic actions to the external wall lines
- But this is also where the roof falls to
- How do these two things work?

SESOC



- Roof plane bracing is meant to node here – transfer roof truss loads to the wall line in -plane



Generic detail in plans only

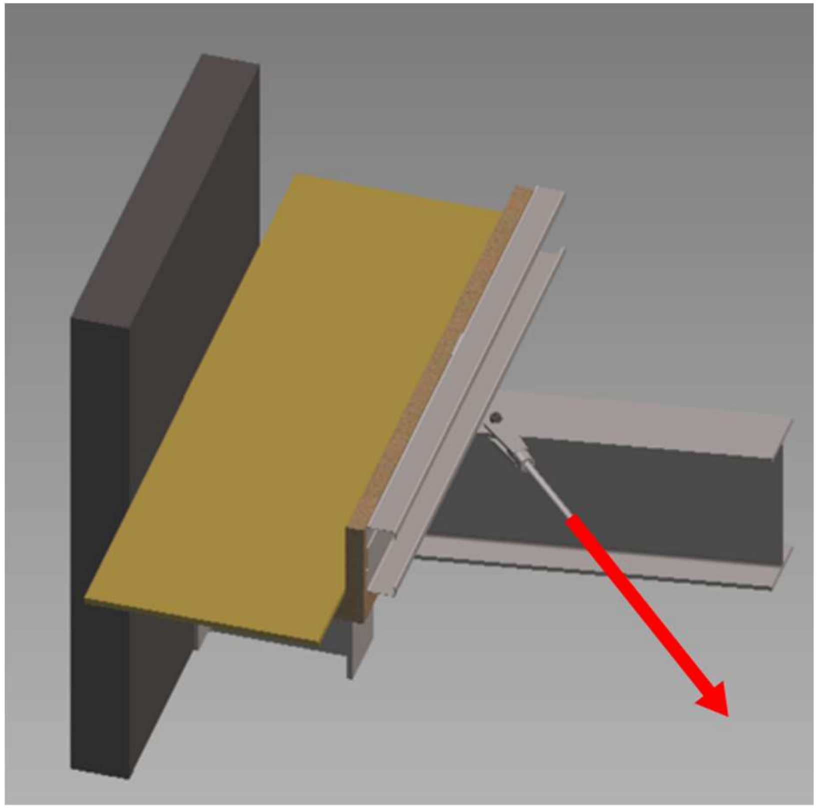
- RB20 bolted to UB flange
- Relies on purlins to form the truss

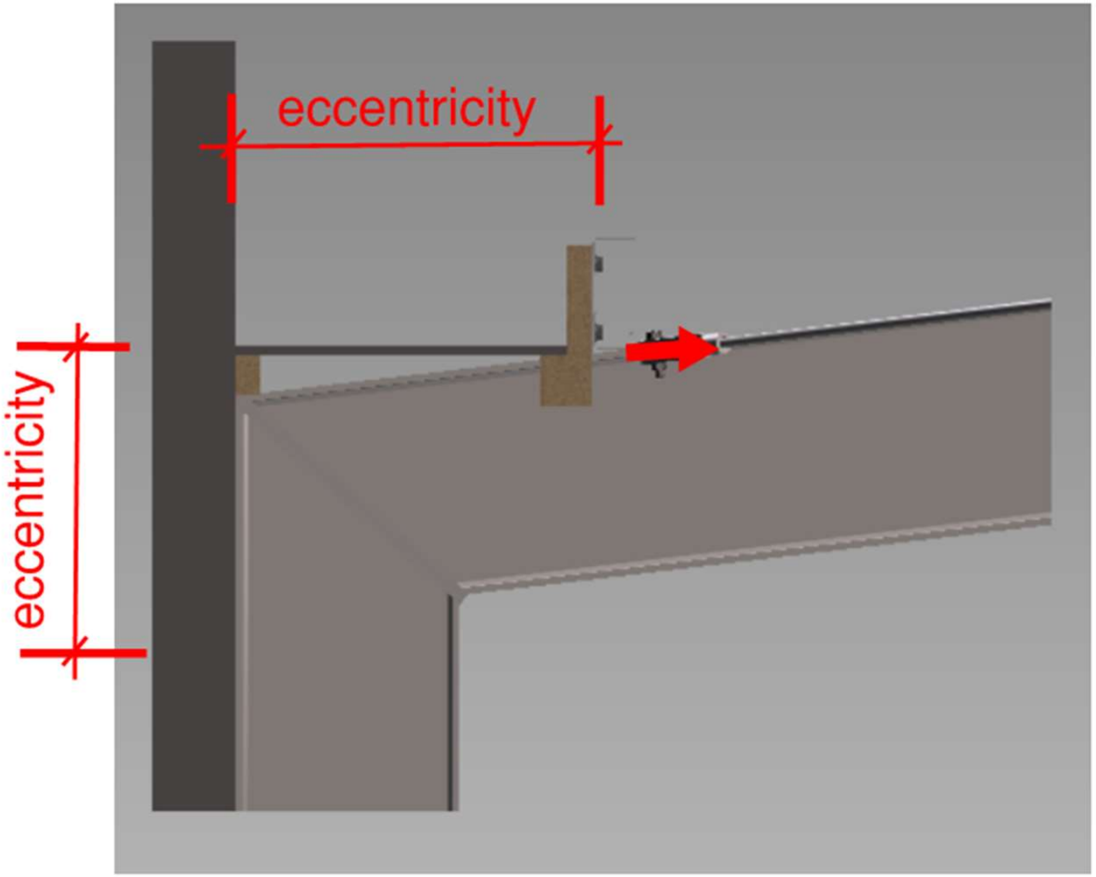
No detail at knee

- Box gutter along this wall line

- Roof plane bracing is meant to transfer forces to the precast panels in plane

- Bracing has therefore been installed well offset from wall




eccentricity
eccentricity

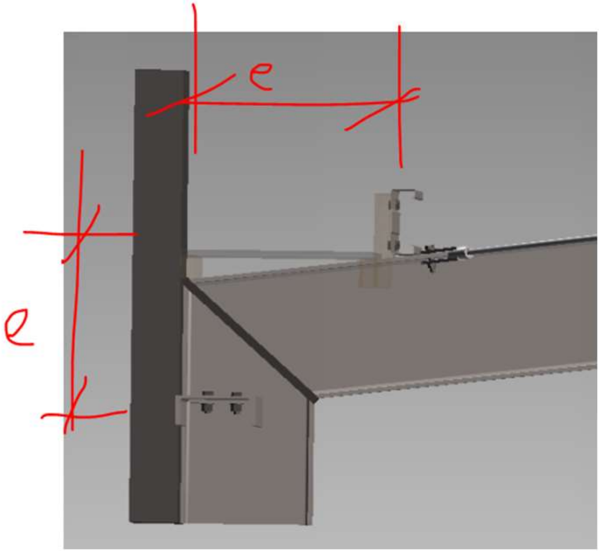
Follow the load path

Load path?

- Roof brace in tension (also limited by bolting to a 12.8mm flange with small edge distance!)
- Bend the portal frame rafter in the weak direction?
- Transfer to the column through the welded connection to the stiffener both sides?
- Column has a bolted connection to the PFC collector
- PFC collector to the panels

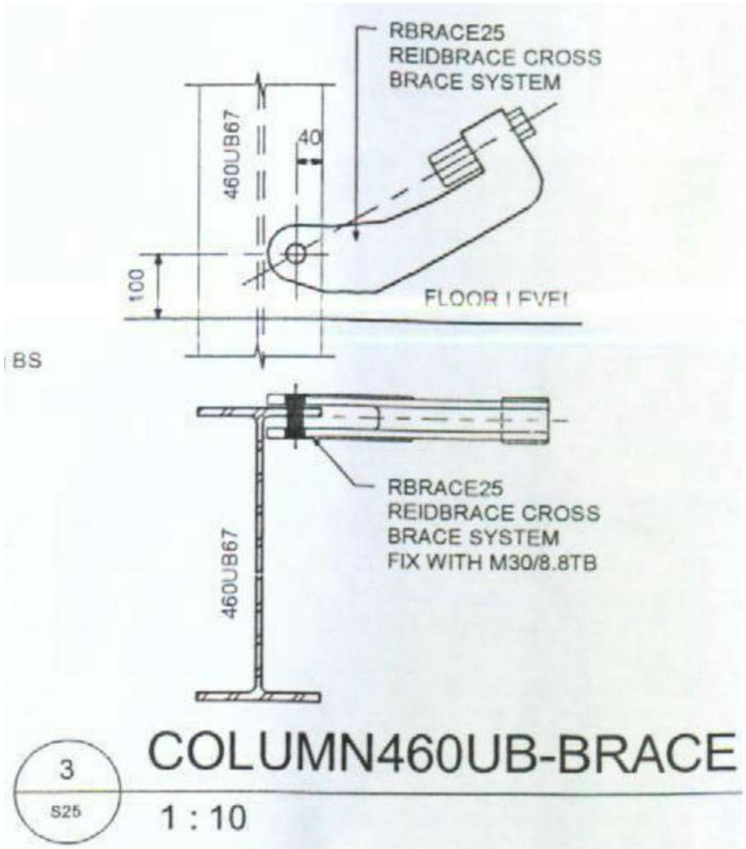
Implications

- Tension forces rely on indirect, and eccentric load paths to transfer to the in-plane walls

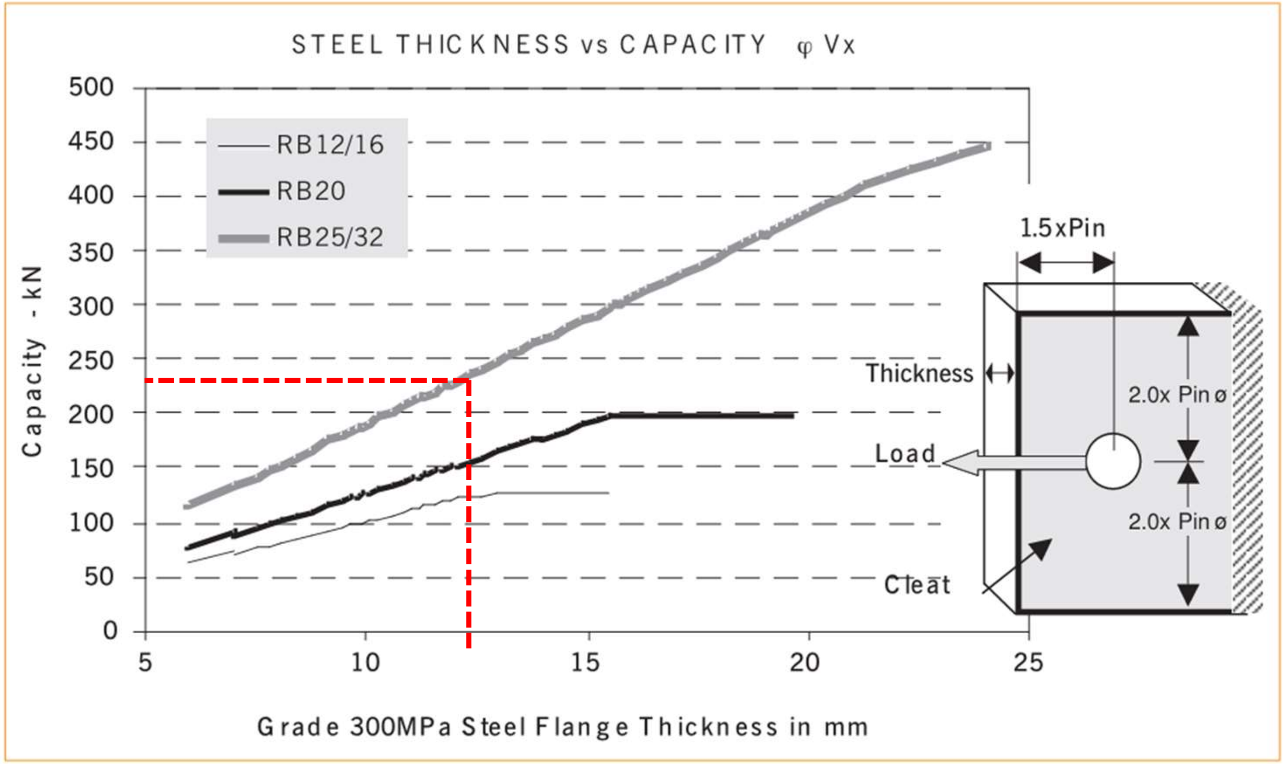


DHS/reidbrace connection

- RB25 – Min UTS = 282kN
- 410UB60 has a 12.8mm flange



Graph 3. – Pin Flange Connection Capacity φVx



Graph 1.

Connection controls!

What about the other end of the truss?

Strut to transfer roof plane loads to in-plane walls

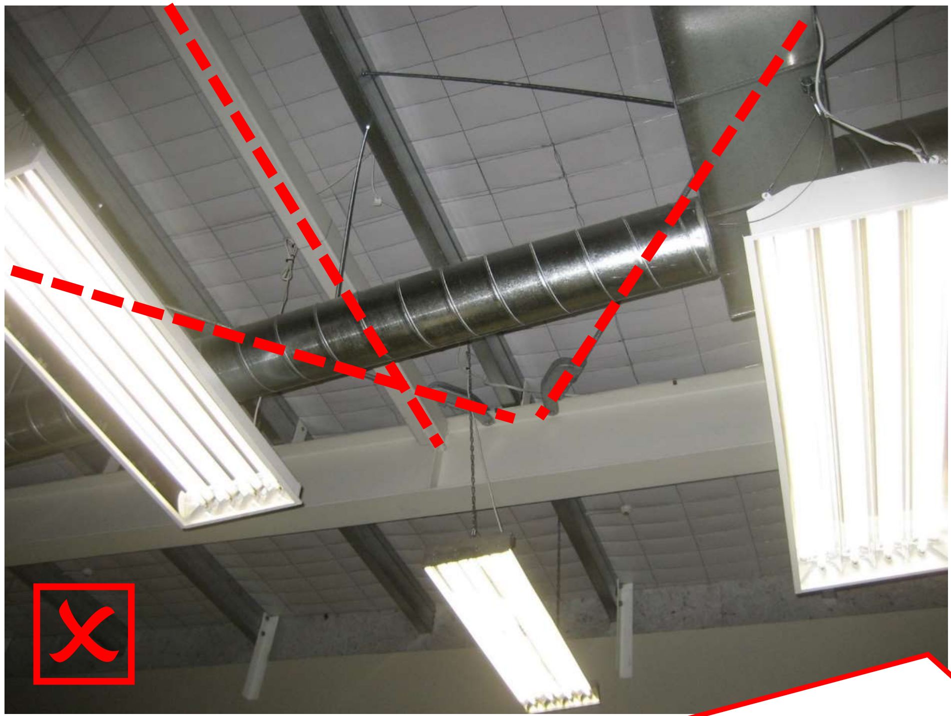
No detail on plans, so this was built

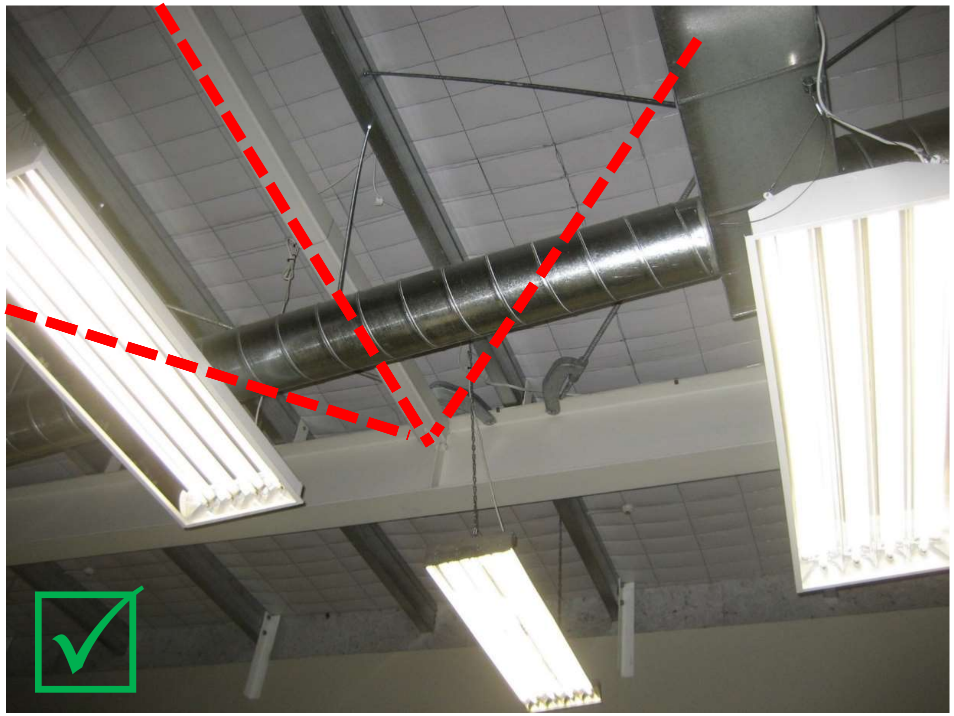
- Builder has 'noded' the bracing with the roof purlins, not with the strut that is meant to transfer the loads!

Implications

- Indirect, and eccentric load paths to transfer loads from the tension braces to the strut

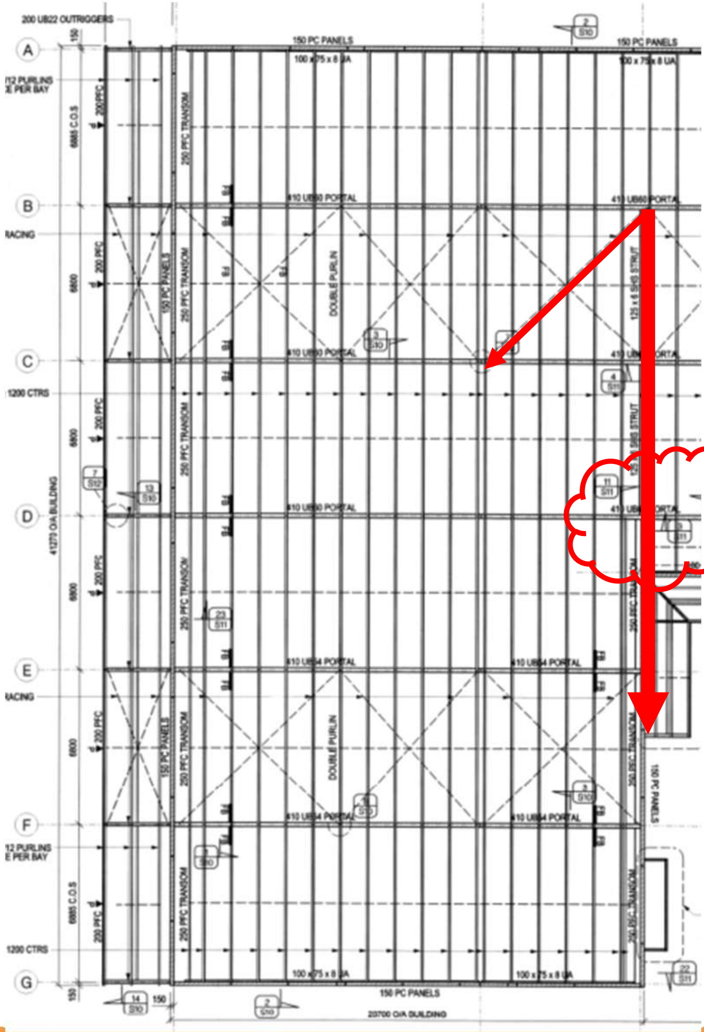
Tip 3– Node your connections





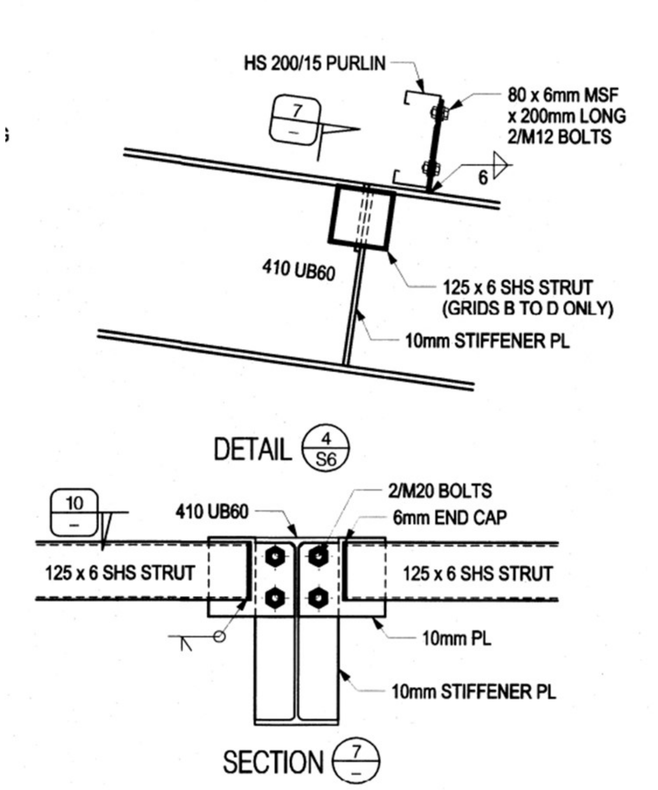
Tip 3– Node your connections

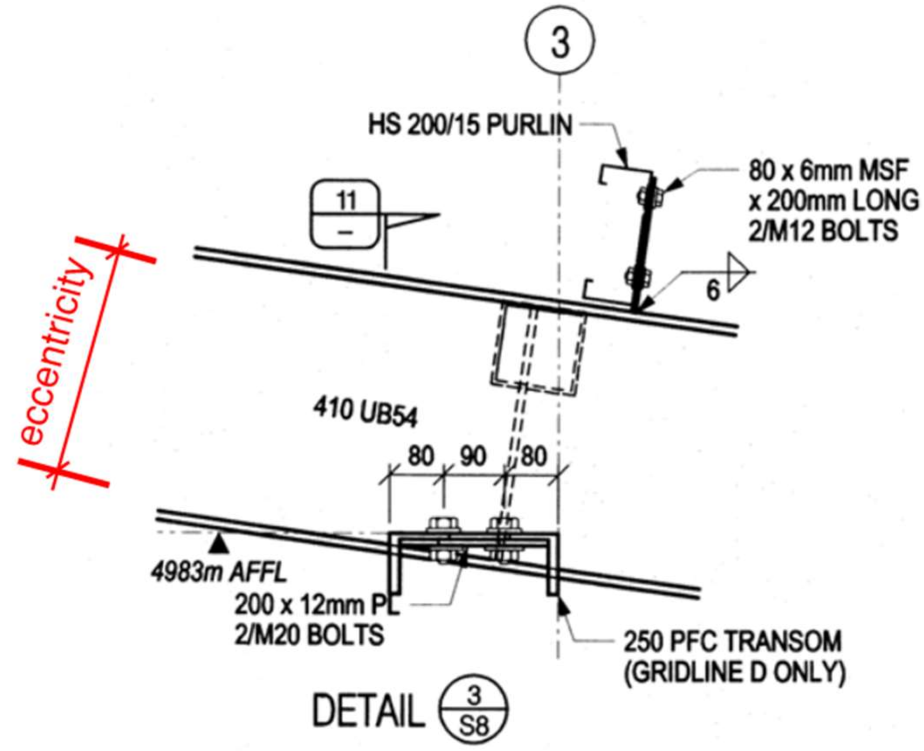
What about the next connection?



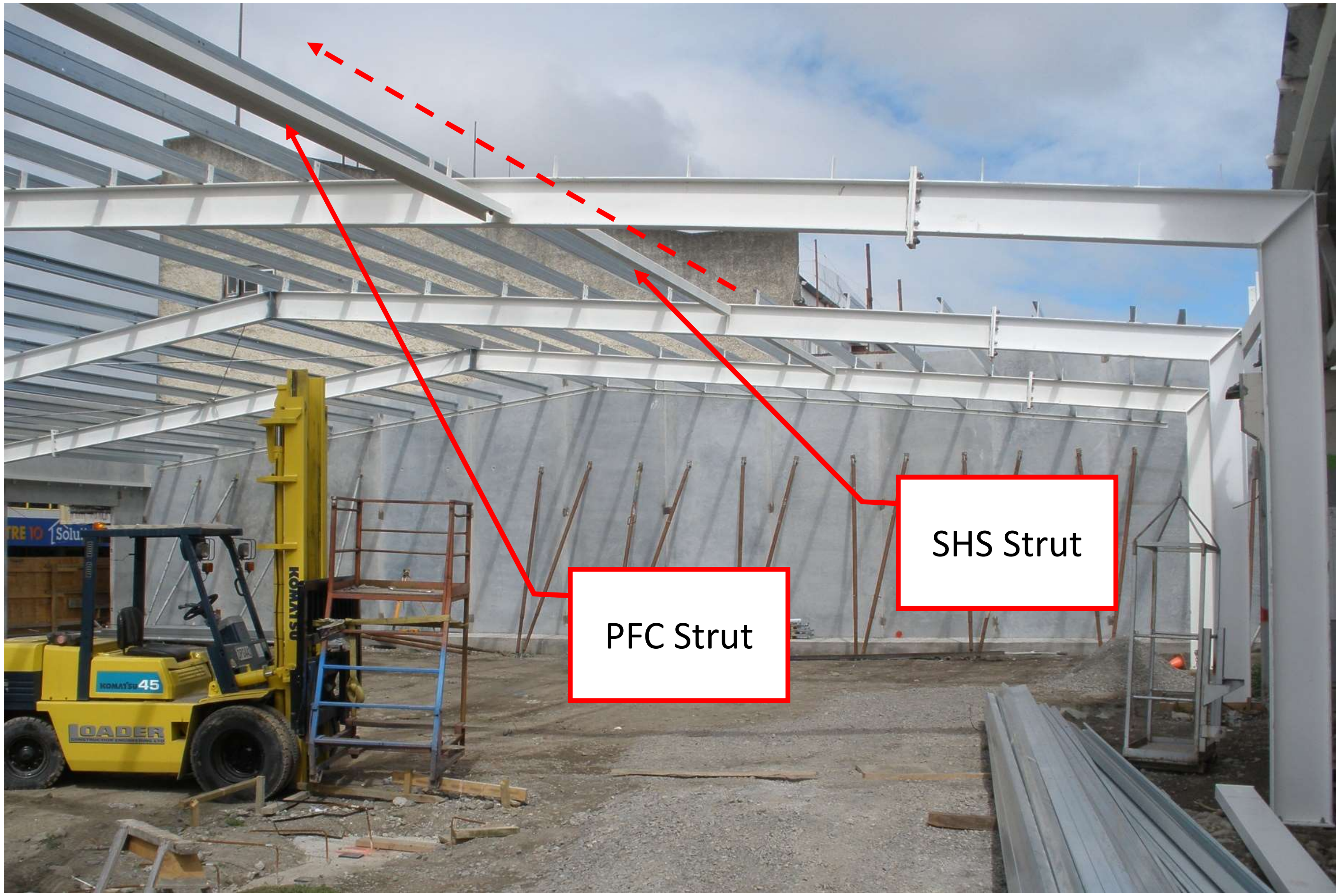
Strut continues each side of the portal frame rafter

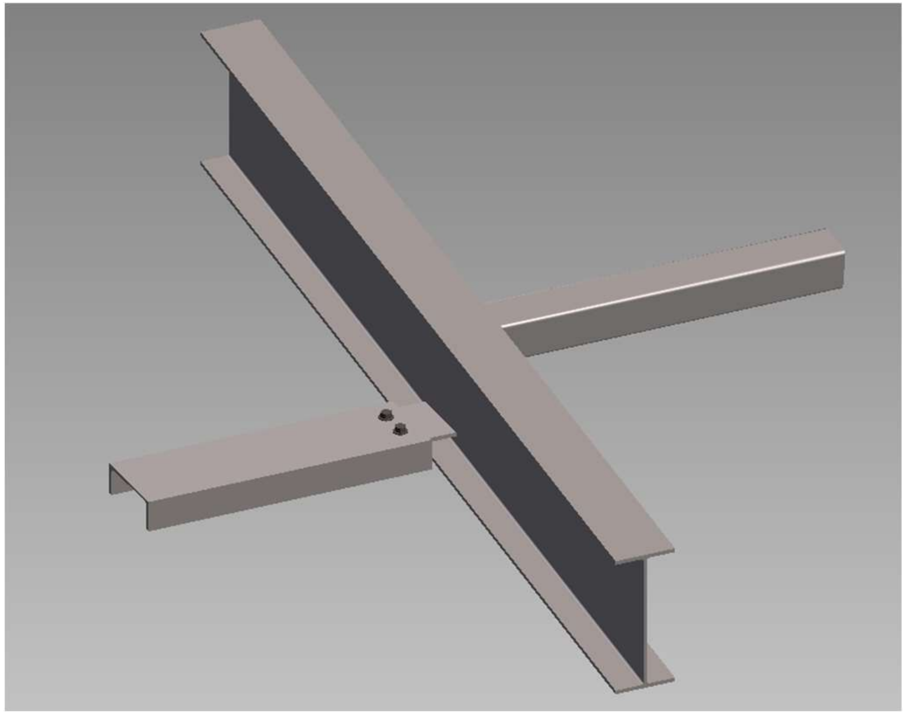
Strut transitions from an SHS to a PFC

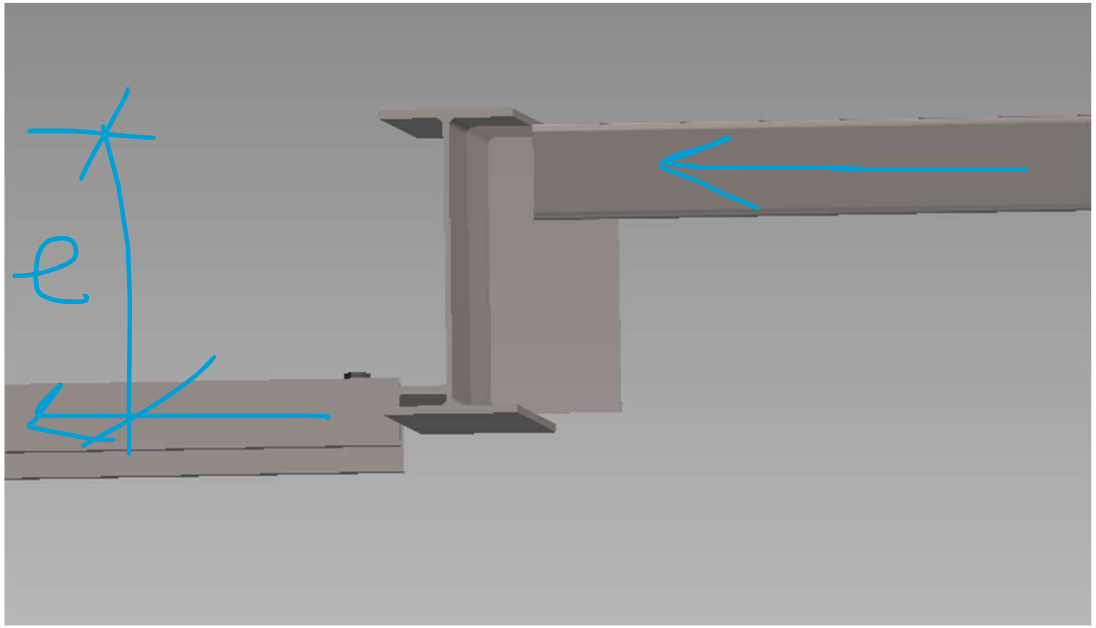




- Grid B to D
- Grid D to E


SHS Strut
PFC Strut





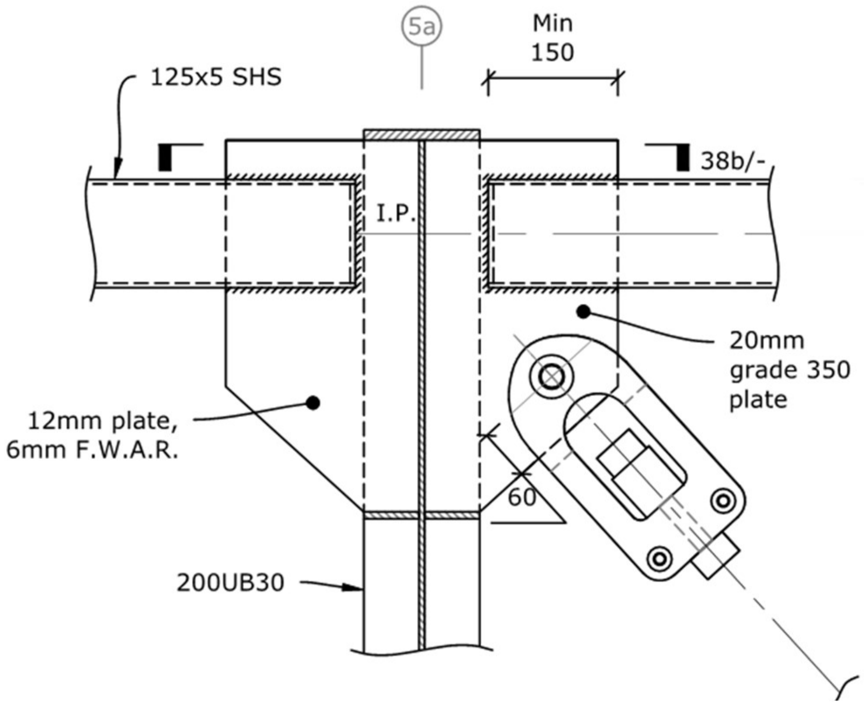
- Eccentric connection – will induce torsion on UB rafter

Implications

- Strut cannot transfer required seismic actions

Tip 3 – Node your connections

What would good look like?



- Node your connections –
- Make sure you have no eccentricities as forces transfer across joint into other members

Tip #8

Make sure you co-ordinate structural with architectural

Tip #9

1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5	If you 'adopt a ductility' make sure you can actually get ductility	☑
6	Do check ins – ie base shear total is right?	☑
7	Sometimes you need to say no	☑
8	Co-ordinate structural with architectural	☑
9		

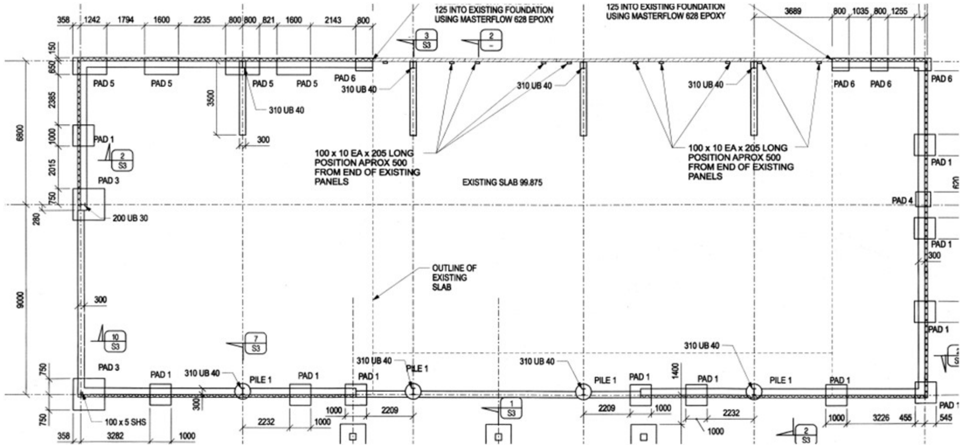
Tip #9

Keep durability in mind

Footing details

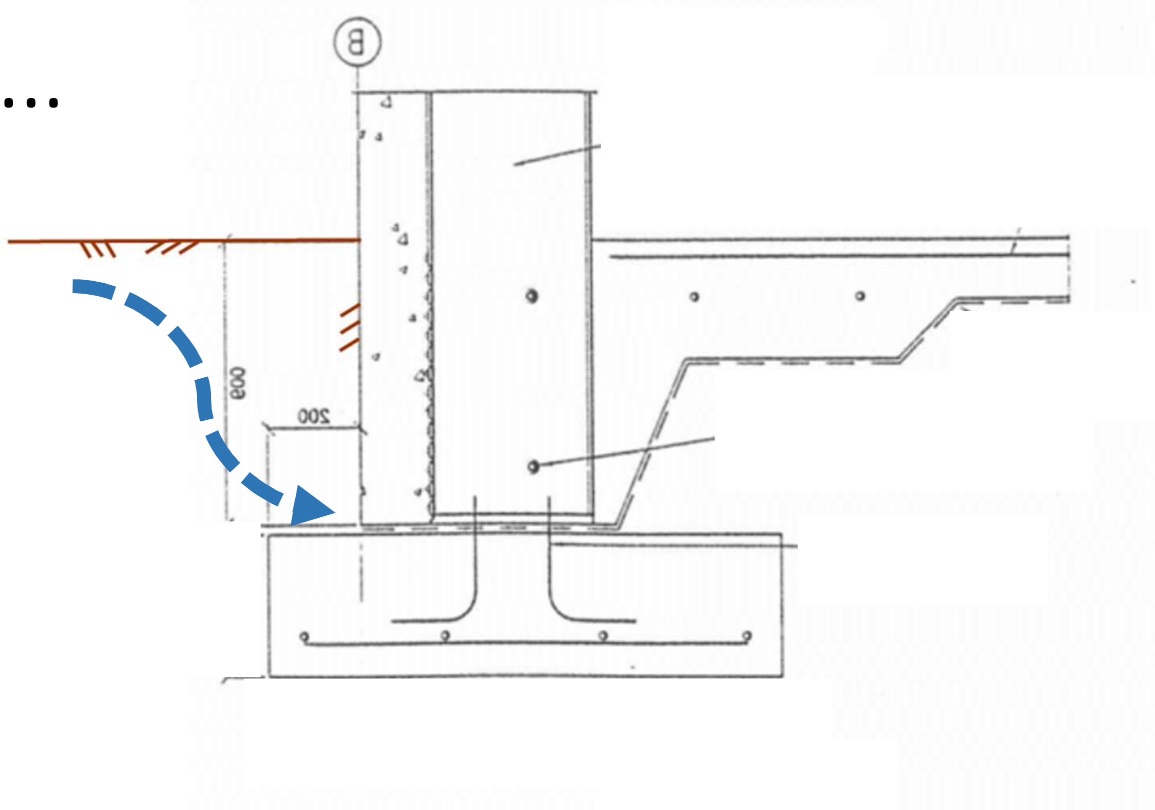
Typical warehouse

- Pad footings to support precast panels
- Panels positioned
- Slab poured against panels



Devil is in the details…

- Ground level is above the footing
- Steel portal extends to the footing
- No waterproofing on the outside edge at the gap between the footing and the panels
- Water ingress will occur



Implications – potentially unprotected structural steel susceptible to rusting

SESOC

Ground level

Un-sealed joint

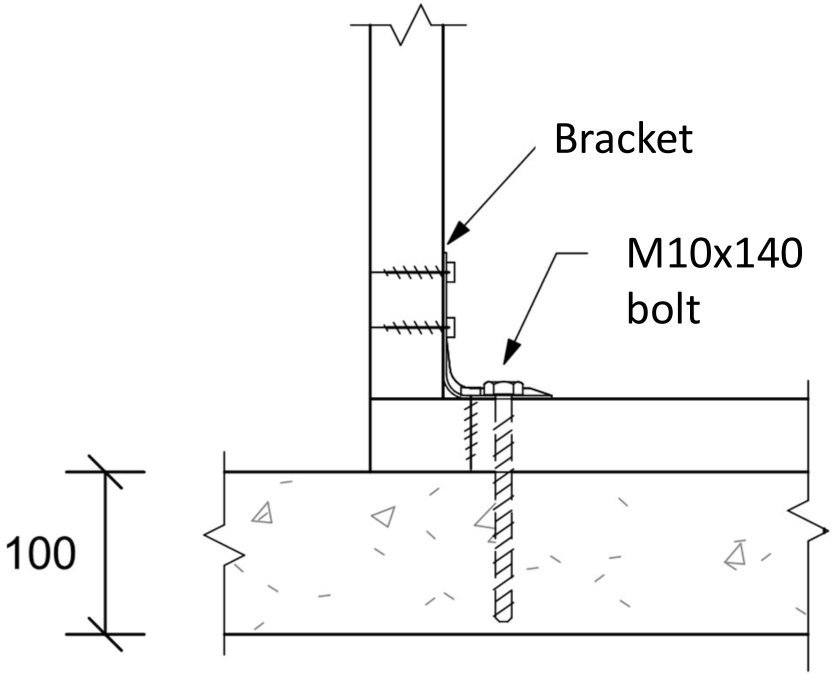
- Moisture seen entering at precast panel joint
- Start of rust to the column base
- Approx. 13 years old
- No chance for maintenance!

Another example

- Steel columns
- A typical structural detail shown only

Remember – steel is all about time to first maintenance

Hold downs

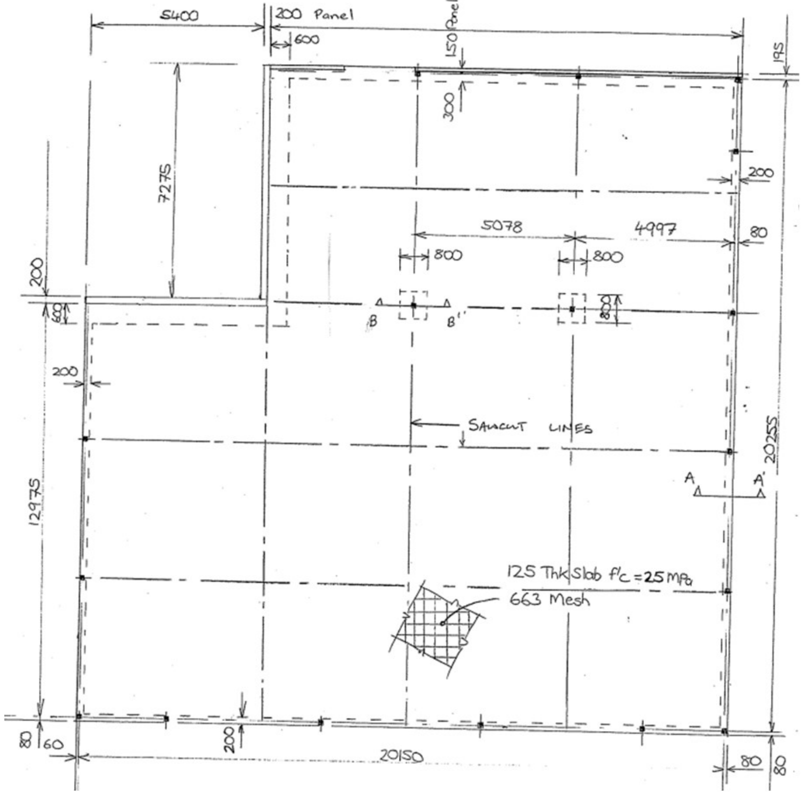


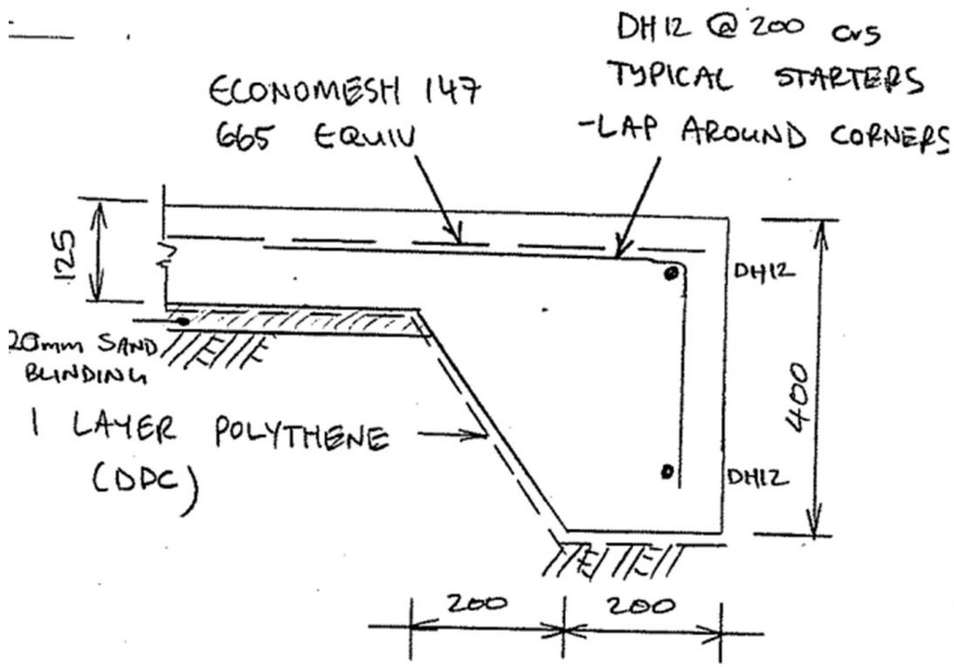
- Typical framing
- Supplied screw bolt is M10x140mm
- A minimum 120 thick slab is therefore needed to maintain cover (and to avoid punching the DPM when drilling!)

1) **Fixing to concrete floor or concrete masonry header block**
   1.1. Minimum concrete strength shall be 17.5 MPa
   1.2. Minimum edge distance to centre of screw bolt shall be 55mm
   1.3. Minimum embedment depth in concrete shall be 88mm
   1.4. Drill 10mm diameter hole x 95mm minimum depth
   1.5. In sea-spray zones, masonry header block shall be wa
   requirements of NZS4210:2001

Building H

- Built 2003
- Steel DHS roof beams providing gravity support to the roof
- Roof plane tension only bracing
- Tension/compression struts in the plane of the walls

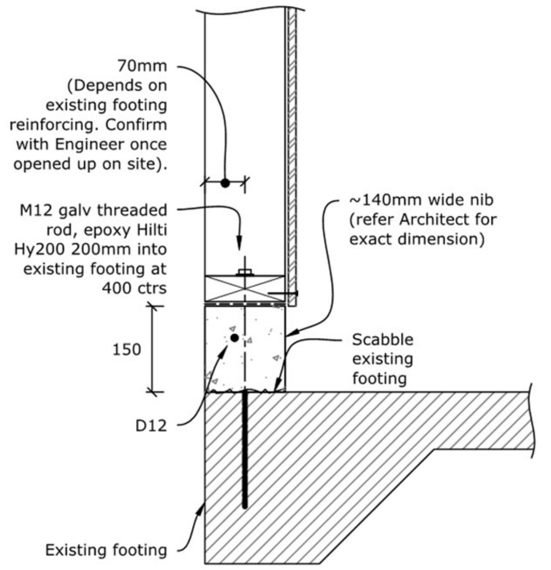


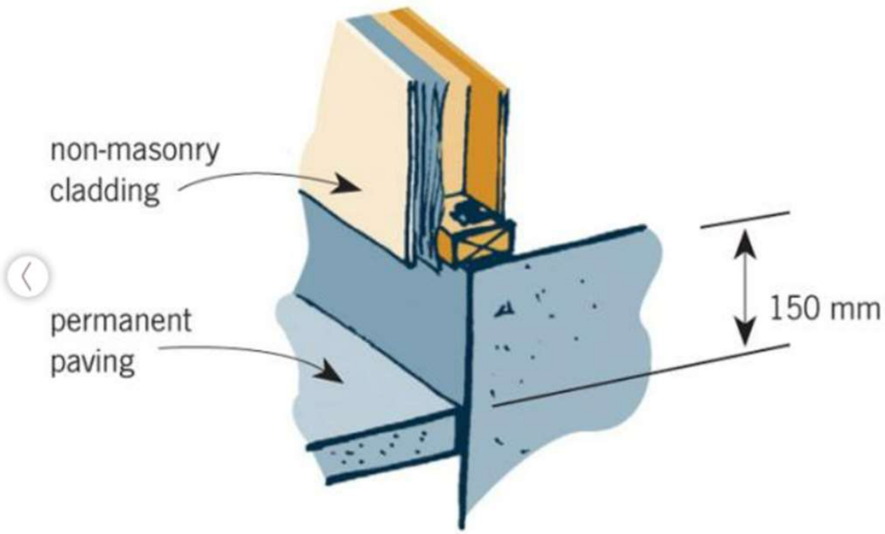


- Timber framed external wall
- Ground level at around slab level

- Moisture ingress mainly due to cladding system failure
- Bottom plate also damaged – insufficient clearance from the outside ground to the timber framing

- New nib added to get required clearances
- Tricky construction to retrofit – much easier to do when building new!





Non-masonry cladding with permanent paving.

E1 – Surface water

Not B1 but something we should be aware of

Tip #9

Keep durability in mind

Tip #10

1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5	If you 'adopt a ductility' make sure you can actually get ductility	☑
6	Do check ins – ie base shear total is right?	☑
7	Sometimes you need to say no	☑
8	Co-ordinate structural with architectural	☑
9	Keep durability in mind	☑
10		

Tip #10

Consider displacements & displacement compatibility

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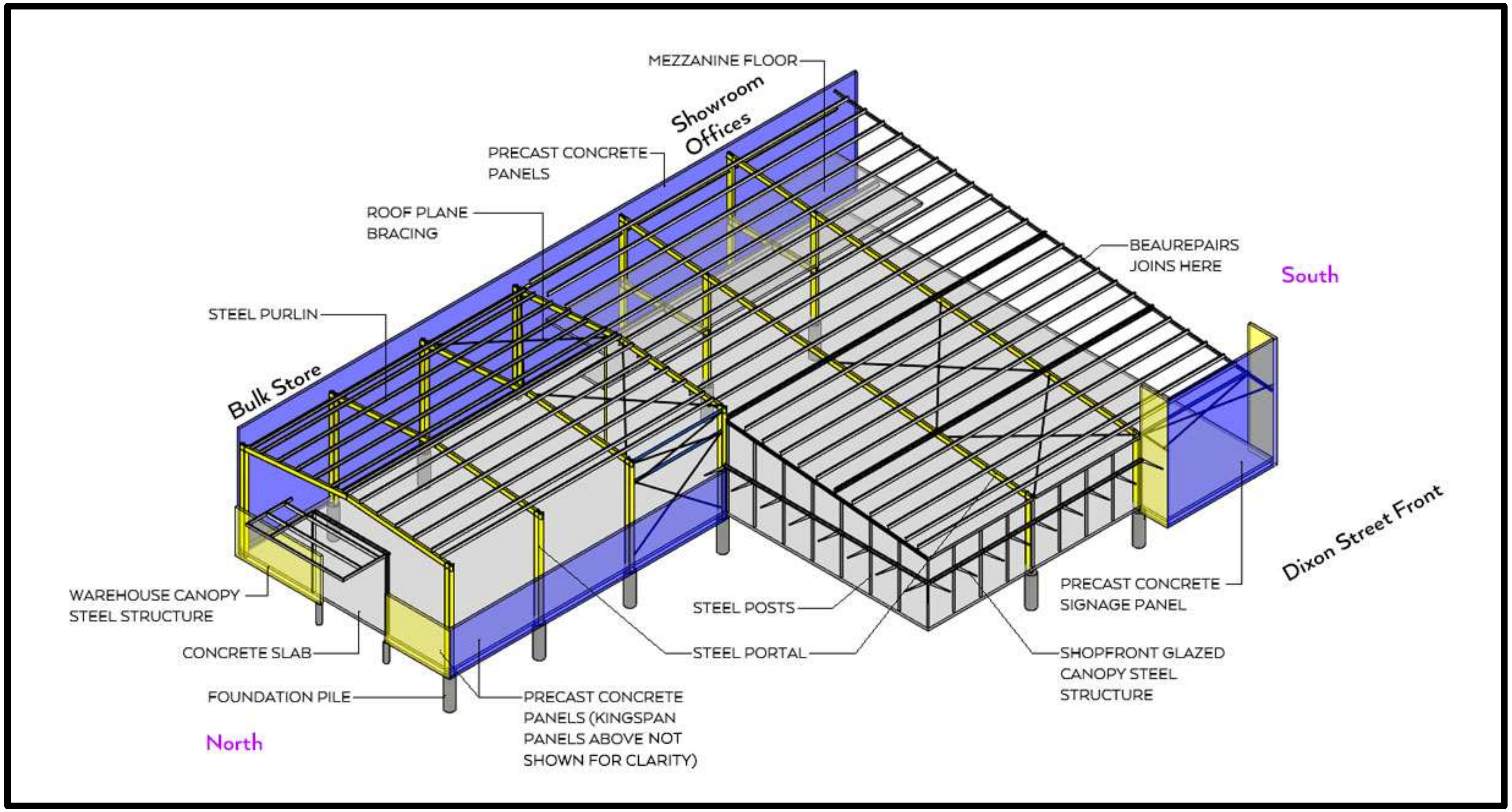
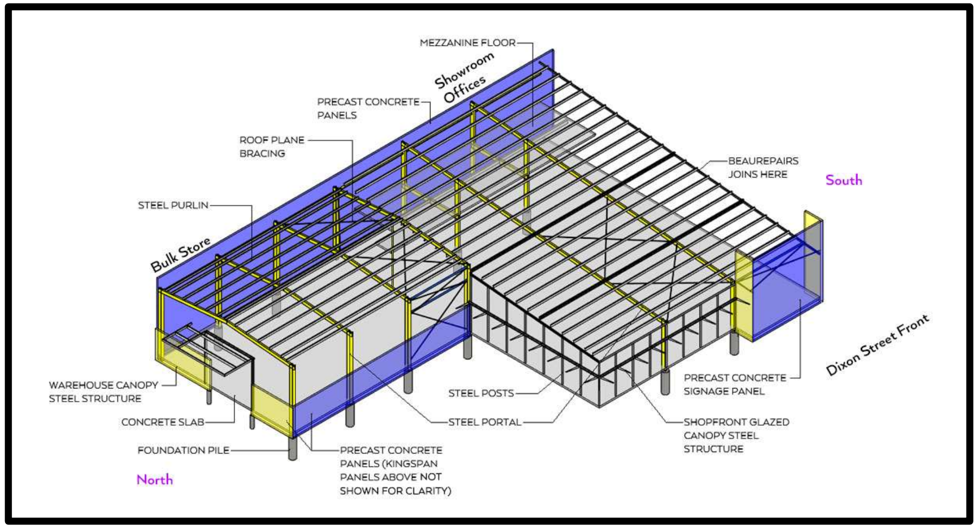
Building E – Retail Area


MEZZANINE FLOOR
Showroom
Offices
PRECAST CONCRETE PANELS
ROOF PLANE BRACING
BEAUREPAIRS JOINS HERE
South
STEEL PURLIN
Bulk Store
Dixon Street Front
WAREHOUSE CANOPY STEEL STRUCTURE
PRECAST CONCRETE SIGNAGE PANEL
STEEL POSTS
CONCRETE SLAB
STEEL PORTAL
SHOPFRONT GLAZED CANOPY STEEL STRUCTURE
FOUNDATION PILE
PRECAST CONCRETE PANELS (KINGSPAN PANELS ABOVE NOT SHOWN FOR CLARITY)
North

Building E

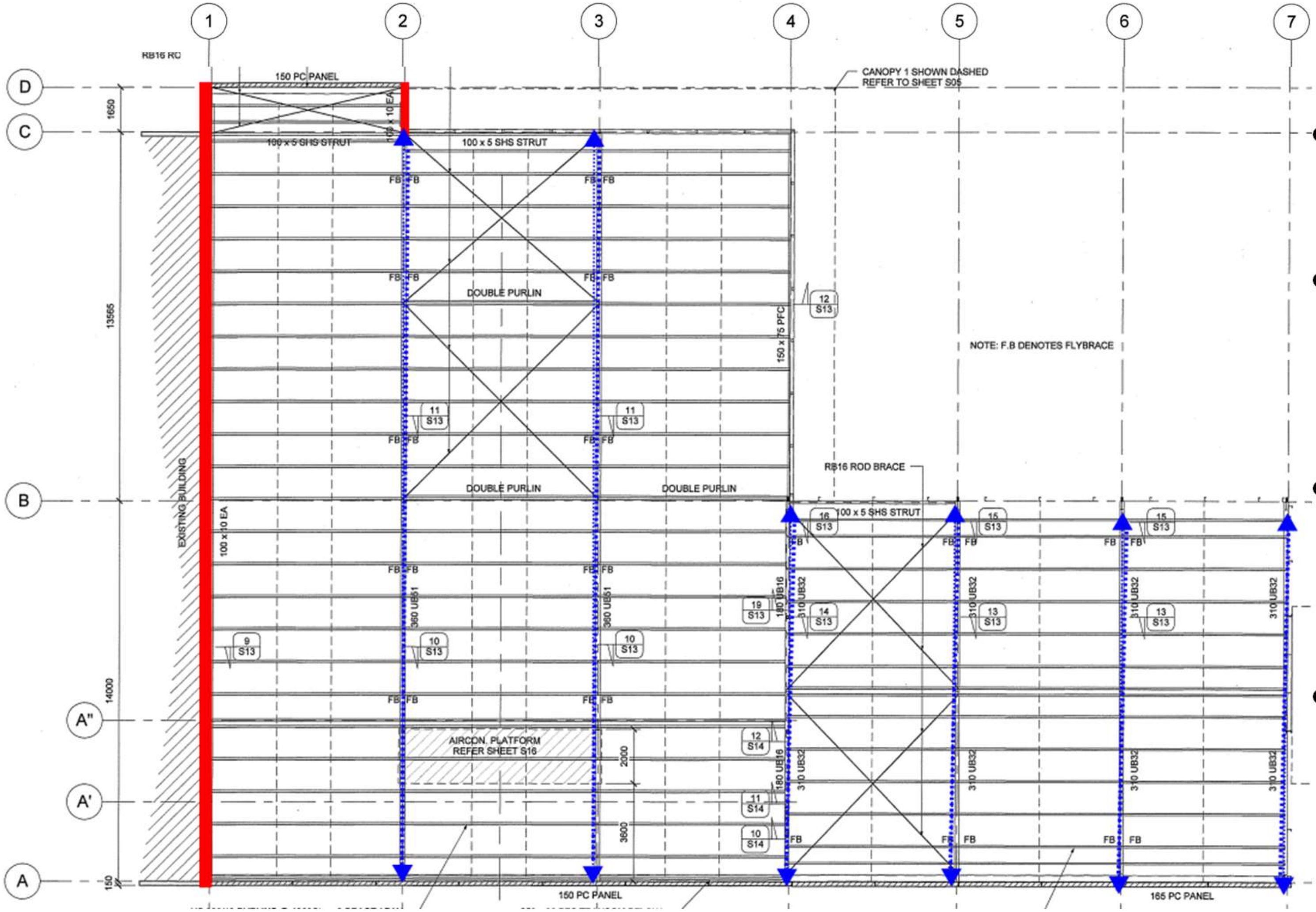


- Designed in 2010
- 1,010m^2
- Two areas – one retail and one storage
- Retail – raking monoslope steel frames
- Storage – steel portal frames
- 150thick precast concrete panels
- Glazing

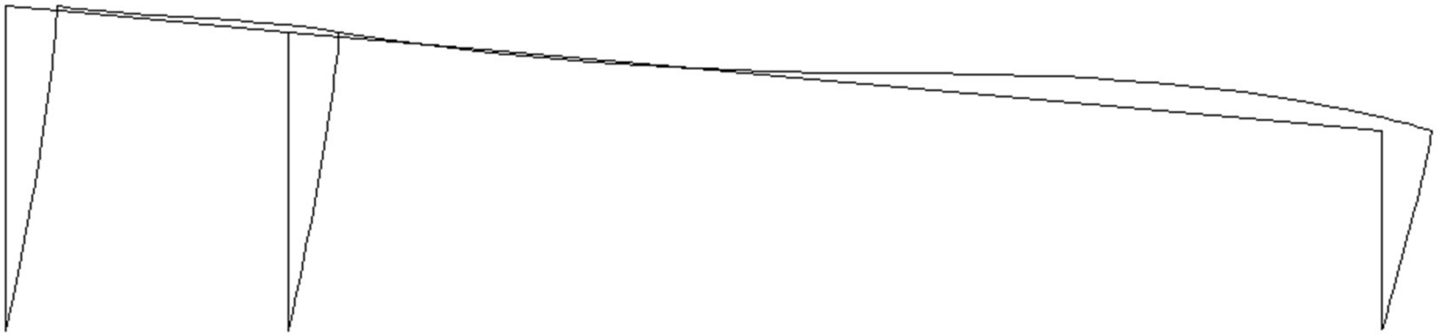

BUY
NOW
PAY
LATER*


MALMO
CurtainStudio
Bona

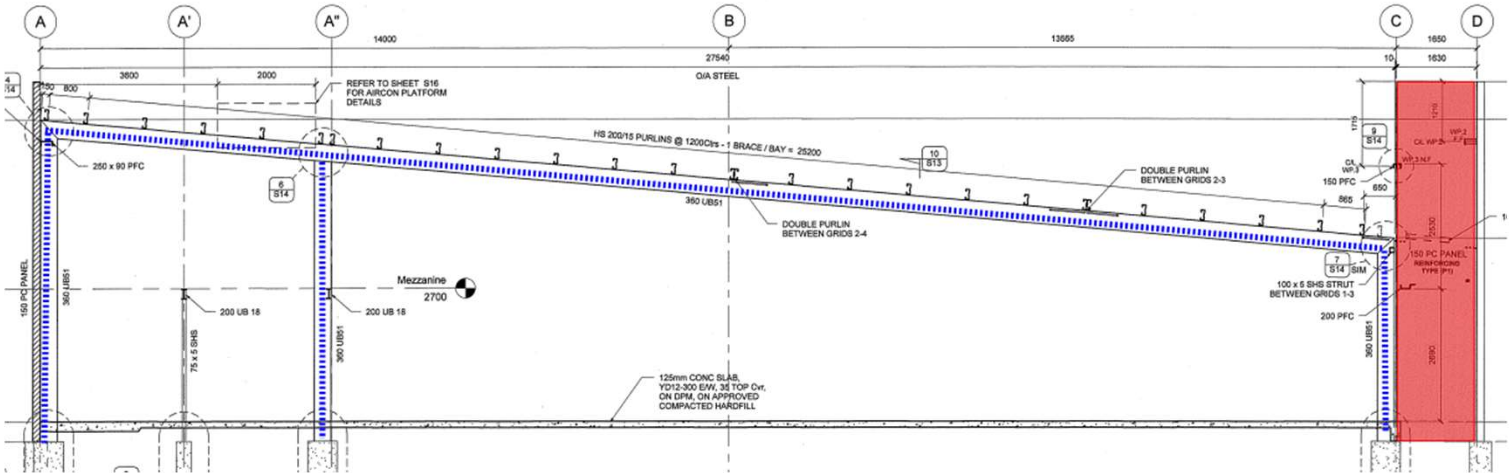
Not a good detail!



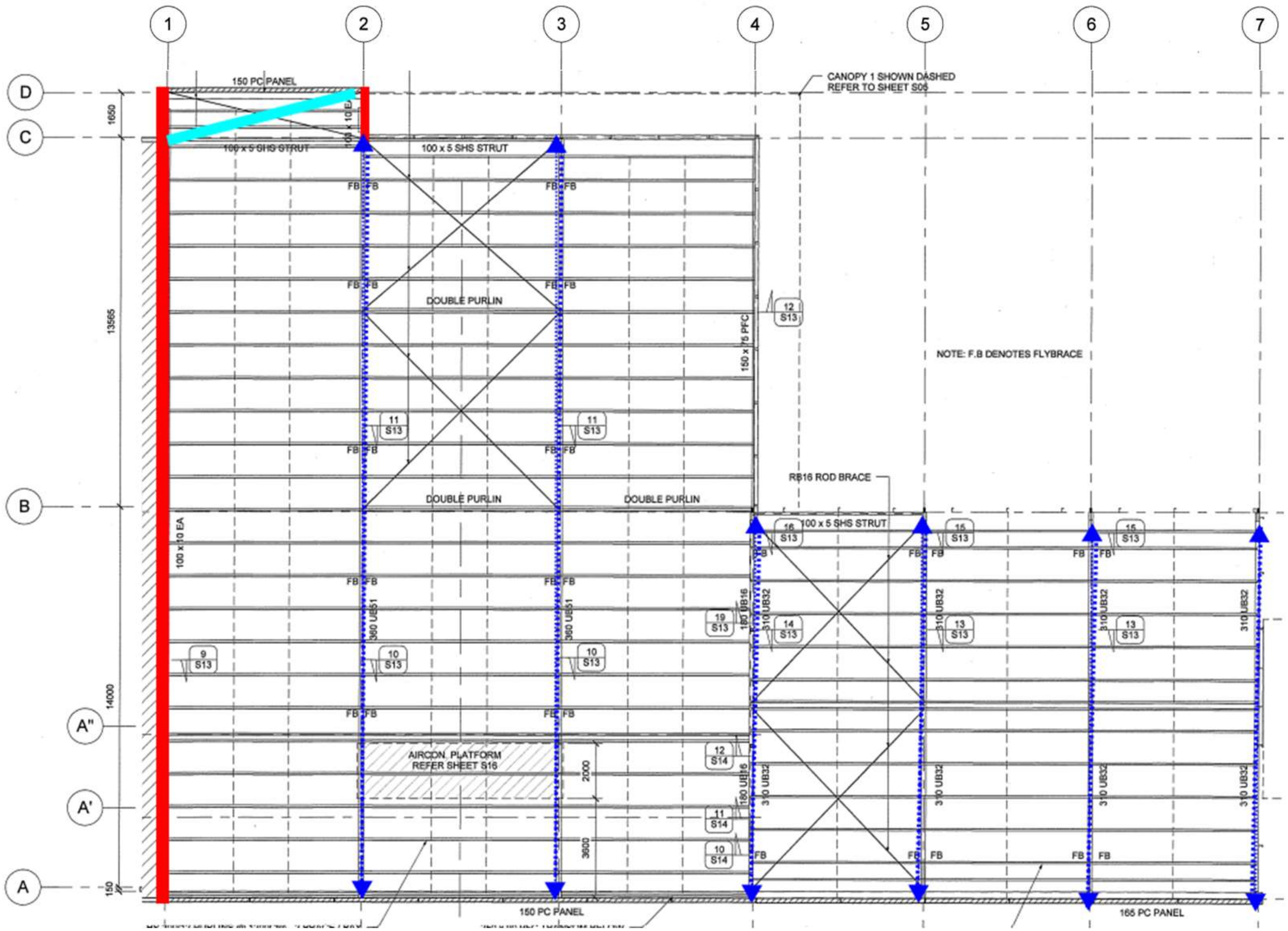
- Steel monoslope frame
- Sits hard against a precast concrete panel in-plane
- Portal frame will deflect – flexible frame
- Panels in-plane will have negligible deflection



Modelled deflected shape of frame



But the frame cannot deflect – it sits hard against a precast panel



- Even if there is some movement along Grid 2, the frame is ‘tethered’ to the long line of PC panels in plane on Grid 1
- The tension brace is meant to brace longitudinal loads
- But will also try and brace Grid 2, which is also tied to grid 3

Tip #10

Consider displacements & displacement compatibility

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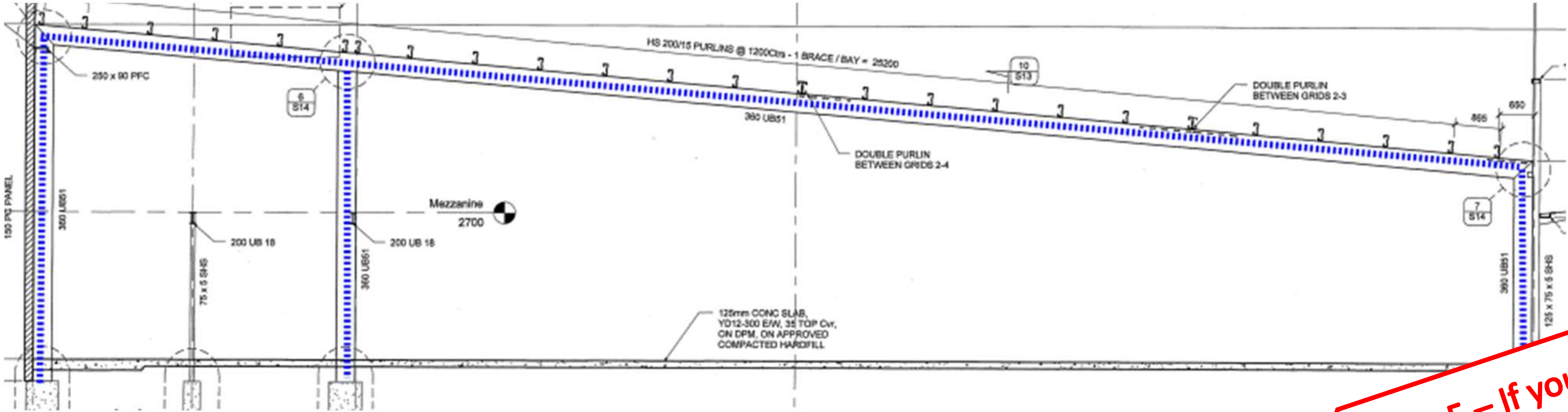
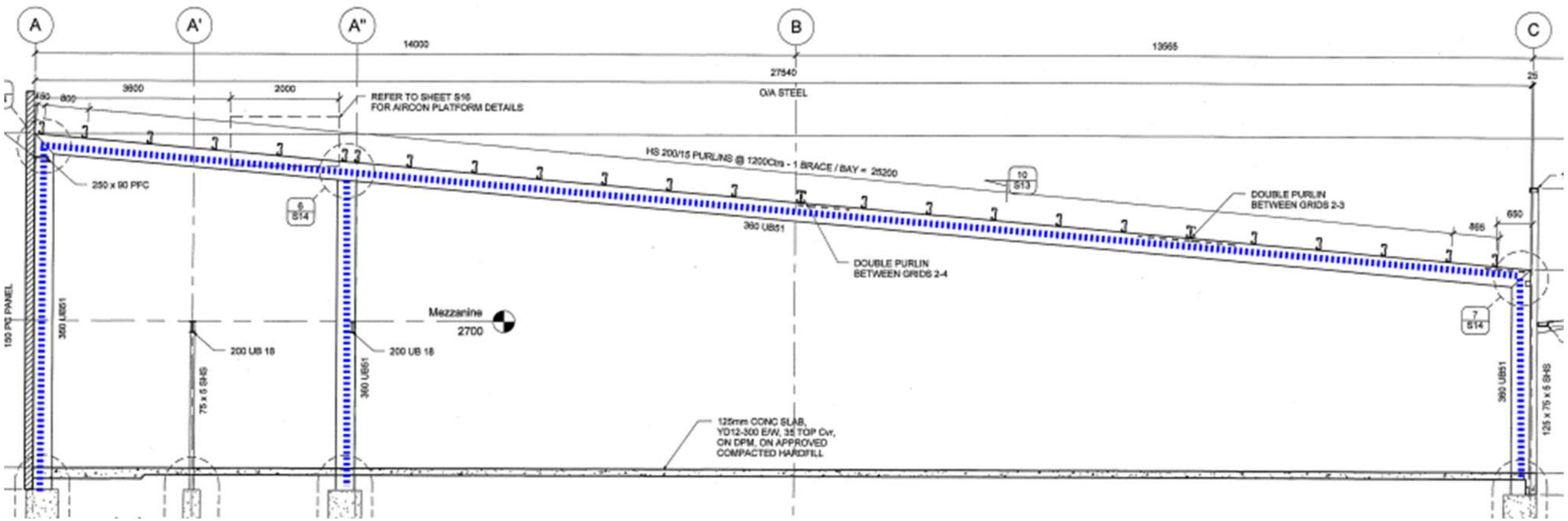
Lets also look at the frames



- 360UB45 frame, designer adopted μ=3
- A 360UB45 meets the sectional requirements
- However there is no fly bracing – no lateral restraint to the bottom flanges
- What about frame deflections?

Tip 5 – If you adopt a ductility, make sure you can actually get ductility

SESOC

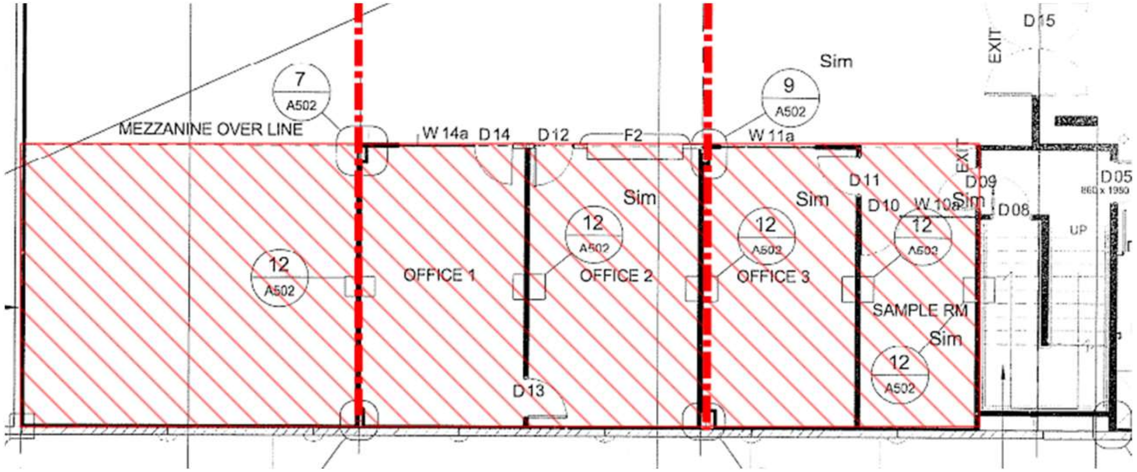


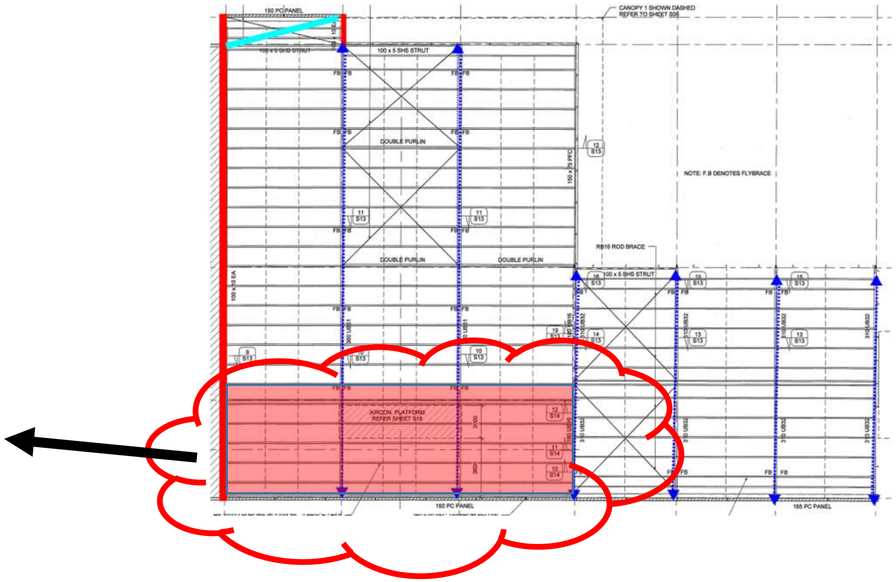
- Designer assumed a fixed base at the three column bases
- Even if we assume this could be achieved, what are the frame deflections?
- Drift = $\Delta \times k_{dm} \times \mu$
- Well exceeds drift limit of 2.5%

Frame is undersized and too flexible

Tip 10 – Consider Displacements

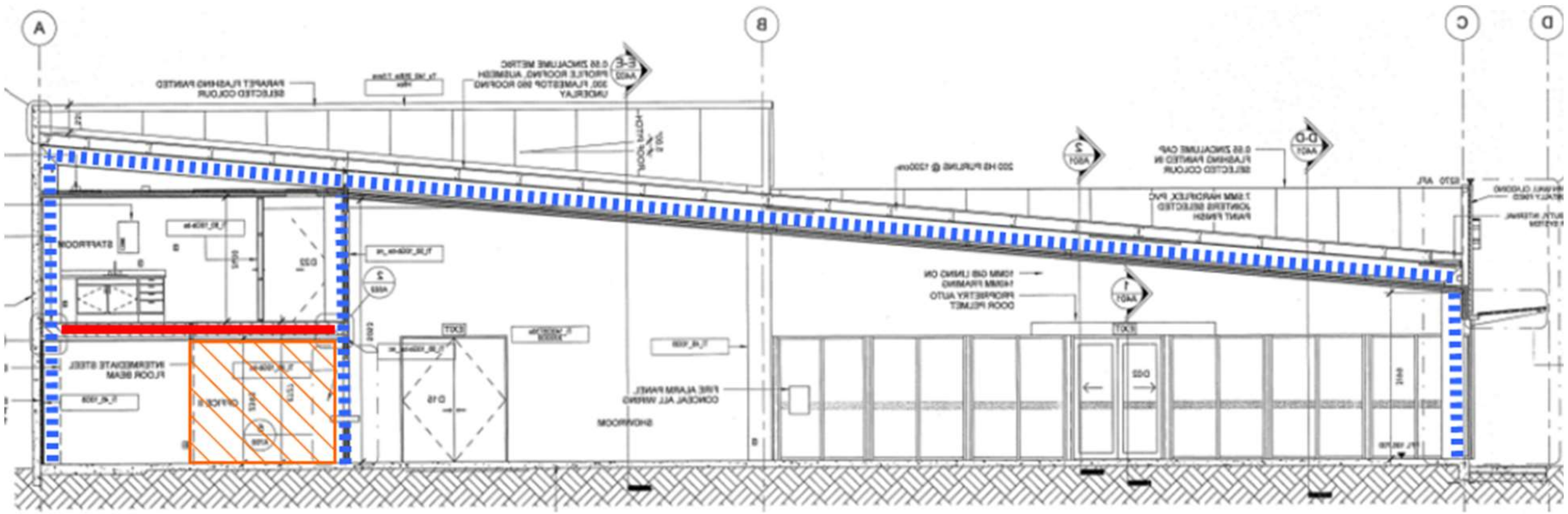
Back to Tip 10





What about the mezzanine?

- Mezzanine floor loads were not applied to the frames
- Perhaps it was assumed that bracing walls under would provide bracing for the floor?
- Displacement compatibility? Frame should match the Gib in-plane SLS limits



- What about the mezzanine?
- Mezzanine floor loads were not applied to the frames
- Perhaps it was assumed that bracing walls under would provide bracing for the floor?
- Displacement compatibility? Frame should match the Gib in-plane SLS limits

Tip #10

Consider displacements & displacement compatibility

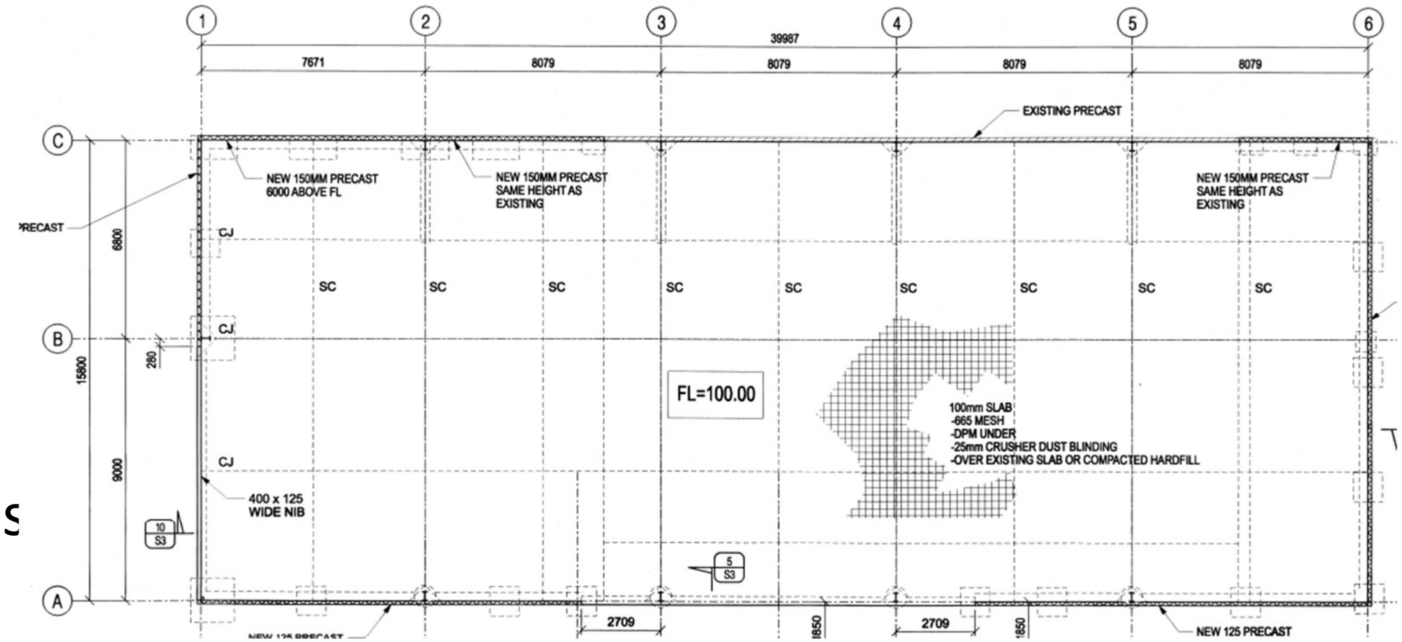
Ten tips for the better design of low rise structures

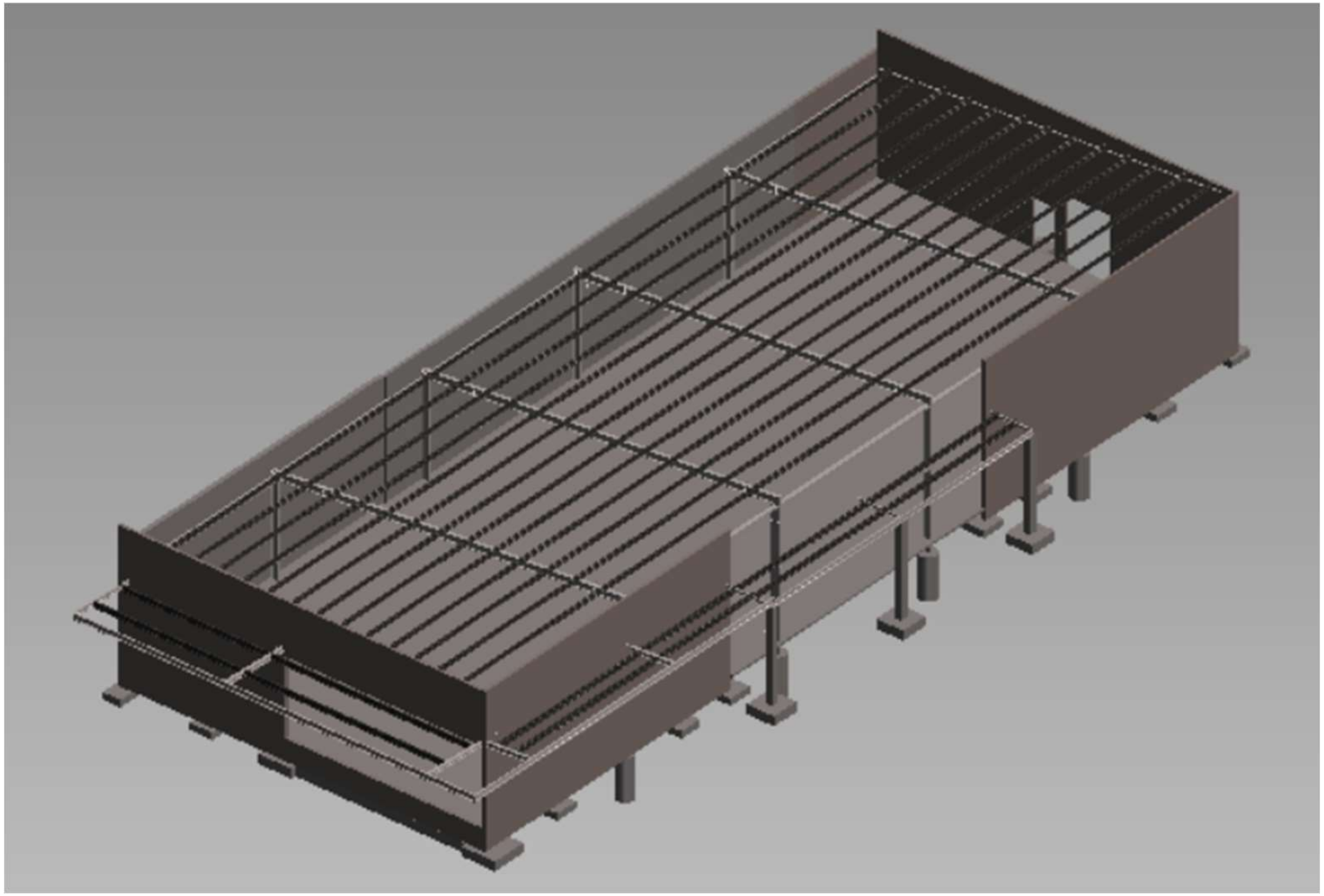
1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5	If you ‘adopt a ductility’ make sure you can actually get ductility	☑
6	Do check ins – ie base shear total is right?	☑
7	Sometimes you need to say no	☑
8	Co-ordinate structural with architectural	☑
9	Keep durability in mind	☑
10	Consider Displacements	☑

With these ten tips in mind, lets look at a few more buildings…

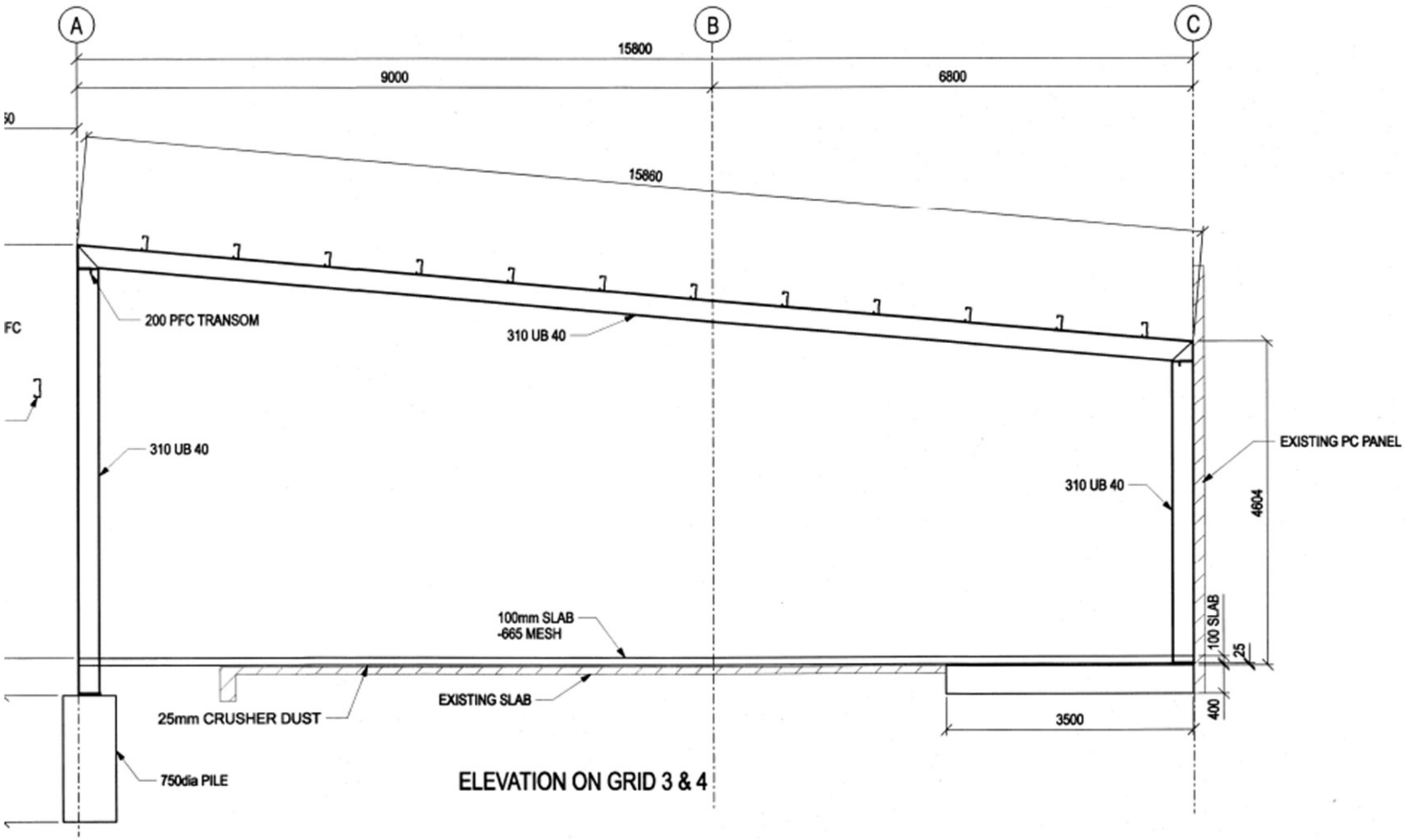
Building I

- 15.8m x 40m rectangle
- Steel portal frames
- 150 thick precast panels

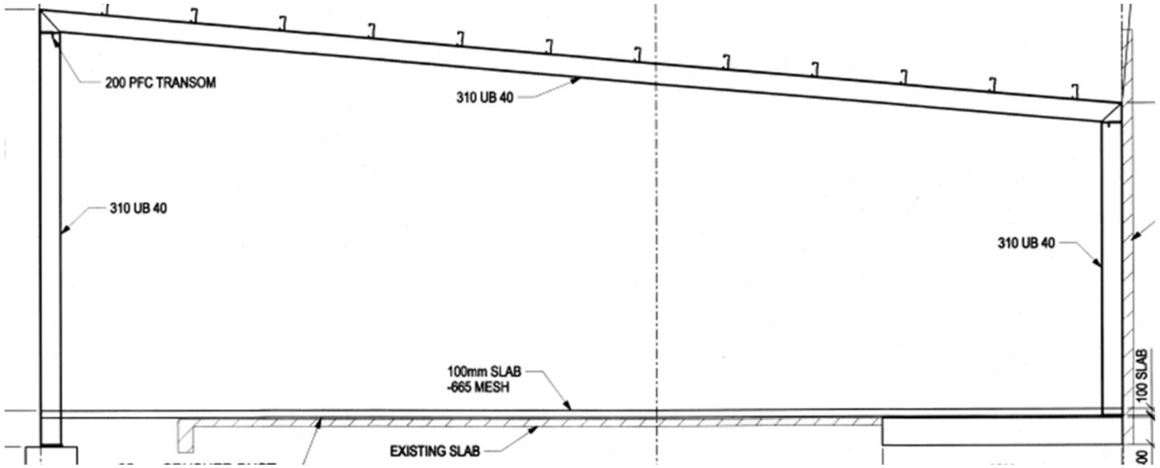





LJ Hooker
For
Lease



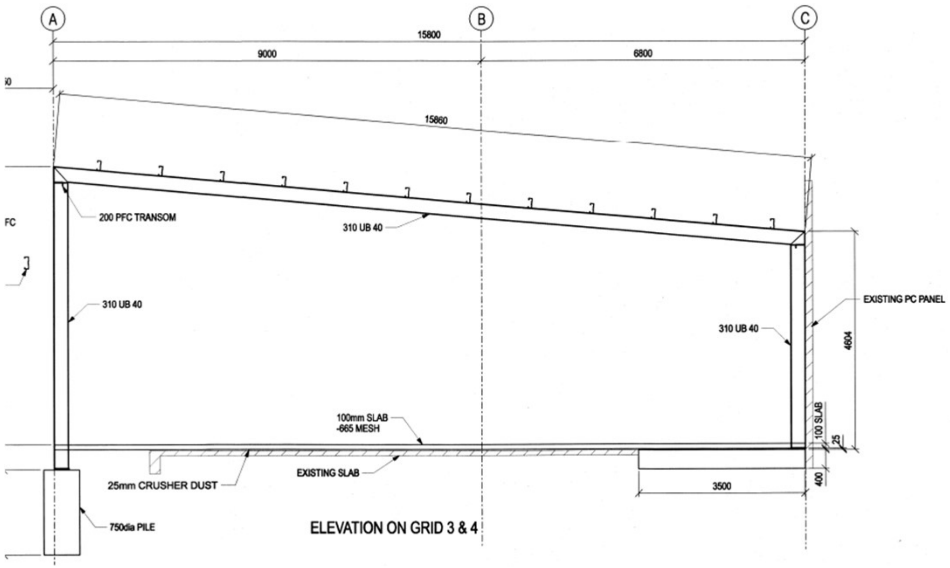
- 310UB40 frames at 8.1m centres
- Supporting 6.5m high panels



Members will buckle well before yield could occur

Tip 5 – If you adopt a ductility, make sure you can actually get ductility

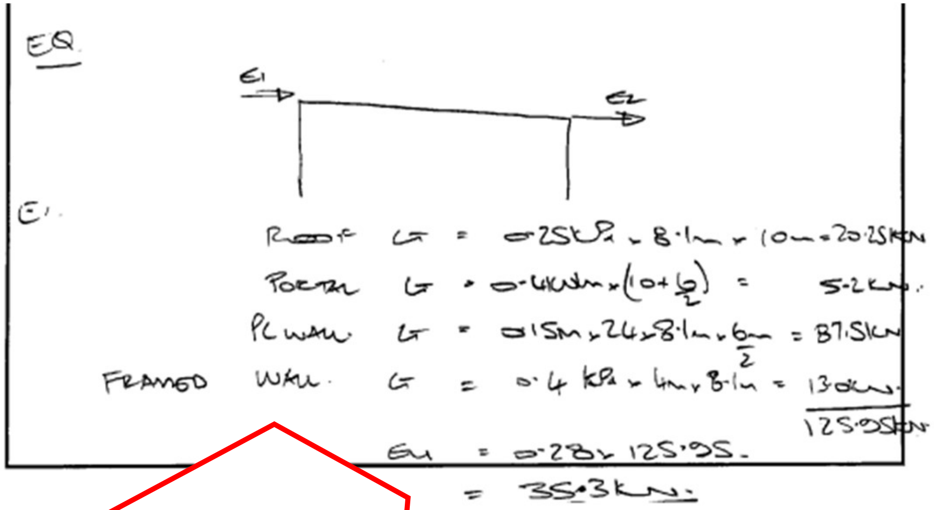
- Designer adopted a ductility μ=2
- Rafter length of 15.86m
- Purlins could provide lateral restraint to top flange of UB.
- No fly braces – no lateral restraint to bottom flange of UB.
- Segment length of rafter (depending on forces) could be 12-14m
- 310UB40 grossly undersized, even if the sections were properly restrained



- Drift was not checked
- Drift = deflection x k_{dm} x μ
- Drifts well in excess of 2.5%

Tip 10 – Consider Displacements

Frames tributary loads

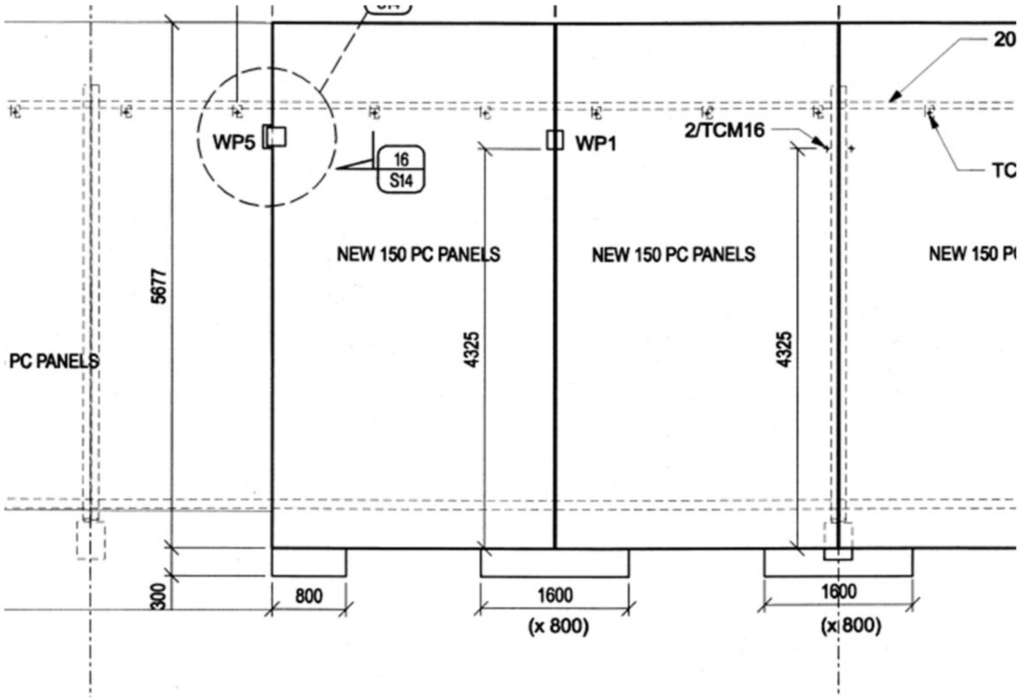


Tip 6 – check your maths!

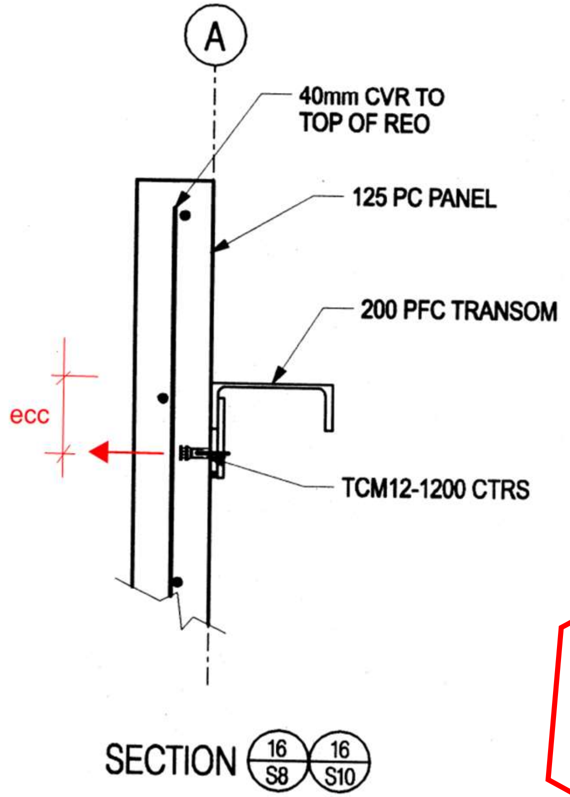
- In Calcs - weight per frame from walls = 100kN
- But portals at 8.1m centres, holding up PC panels around 6m high on both side walls
- Simple calc of wall weight

Wt=0.15m thick x24kN/m^3 x 8m length x 6/2m height x 2 sides = 173kN

Side wall Panels Out-of-Plane



- Panels supported by a 200PFC transom spanning 8m
- PFC Collector is grossly undersized for an 8m span
- No lateral restraint to the collector



Tip 4 – Connections are critical

- TCM12 inserts – shallow embedment
- Plate bending in weak direction to PFC
- Eccentric & induces torsion on PFC

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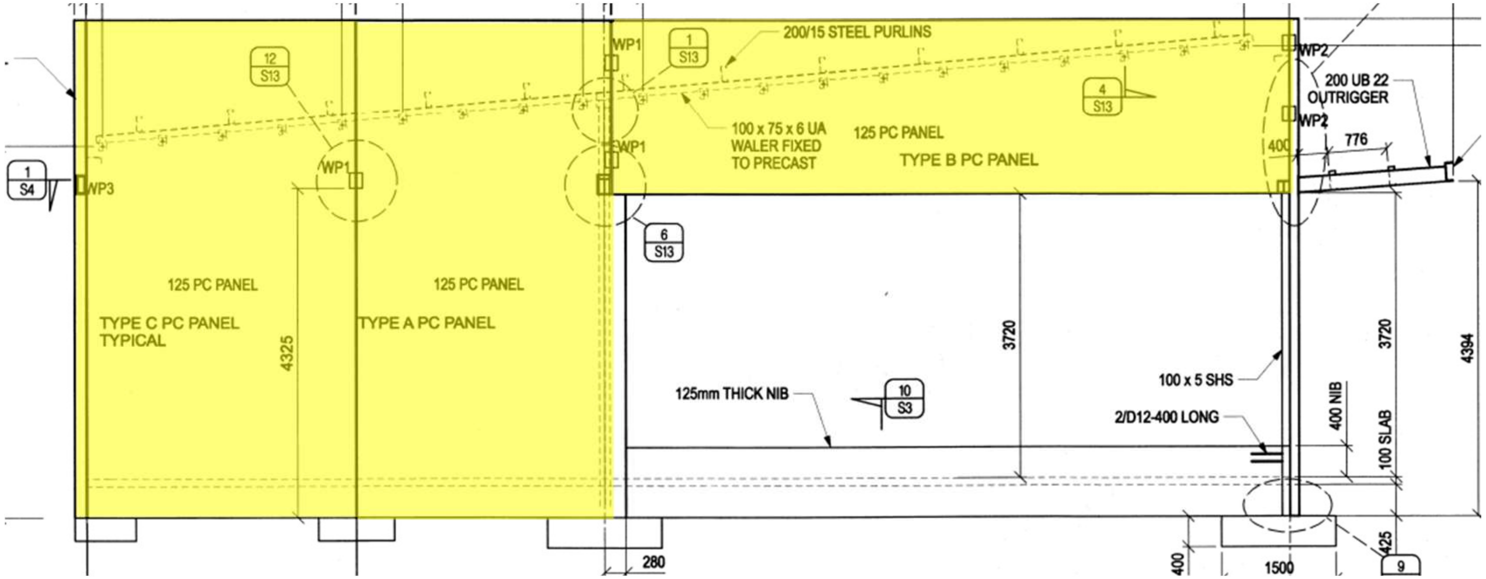
Panel connection out-of-plane

Eccentric connection to panels, no lateral restraint to PFC flanges
This connection and member cannot transfer the required forces

End Wall

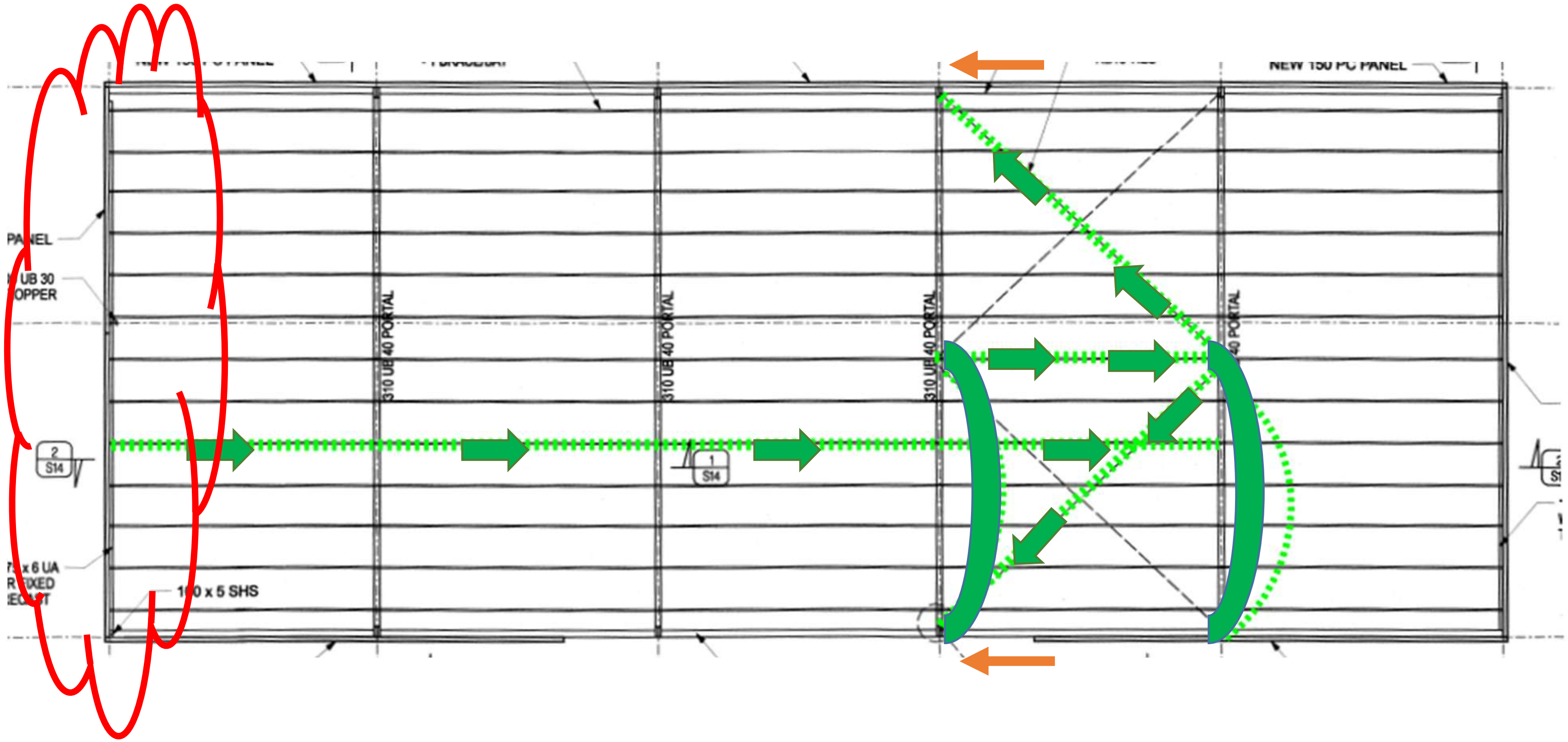


- 150 thick concrete panel sections
- Lintel panel over the window opening

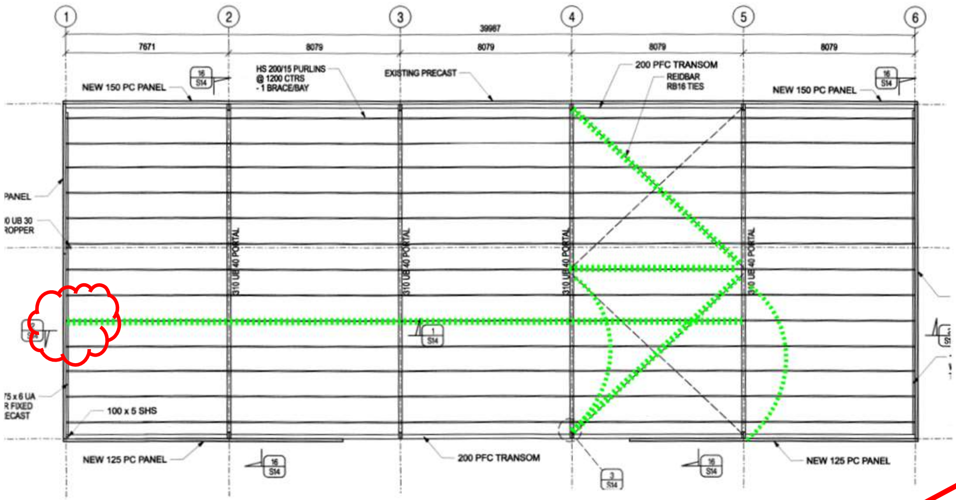
End wall out-of-plane

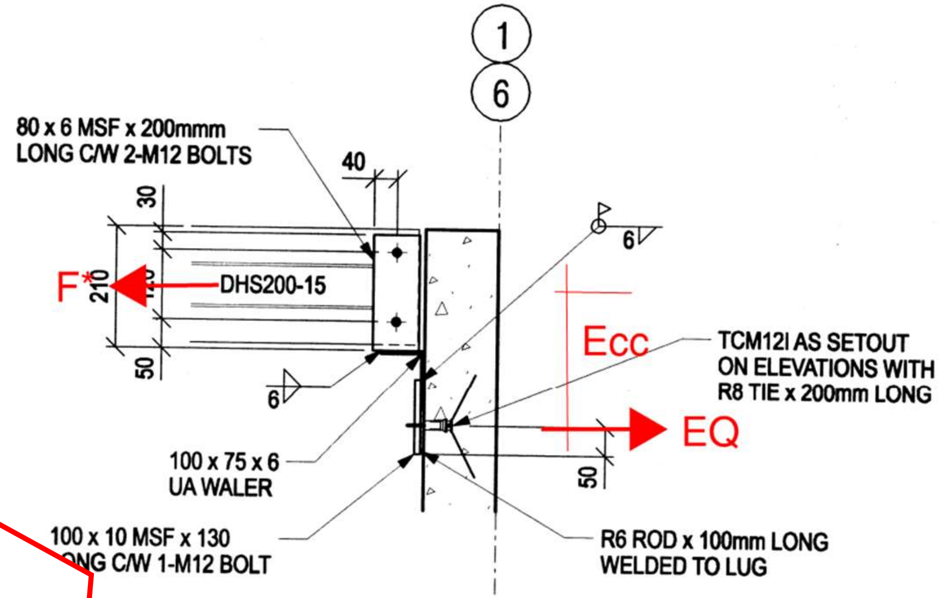
SESOC

What’s the load path?



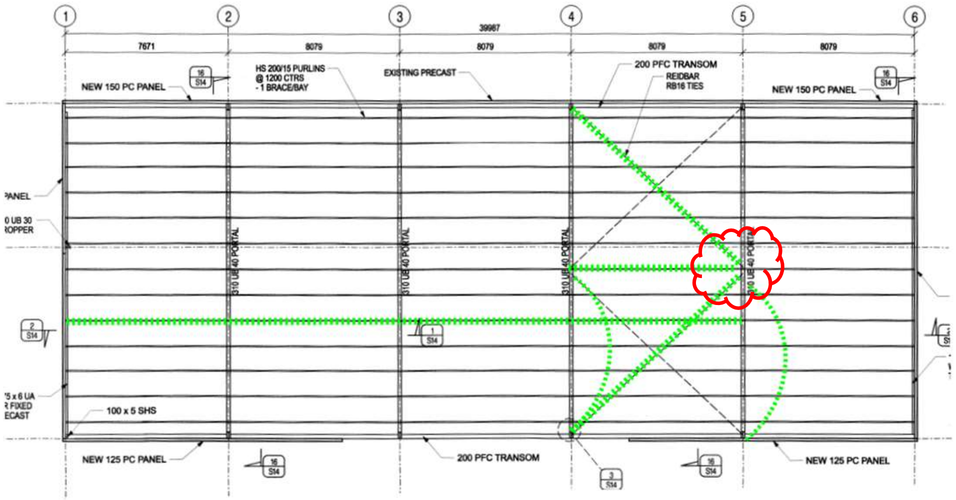
- End wall panel load path out-of-plane
- Panel is only secured at the top via the DHS purlins

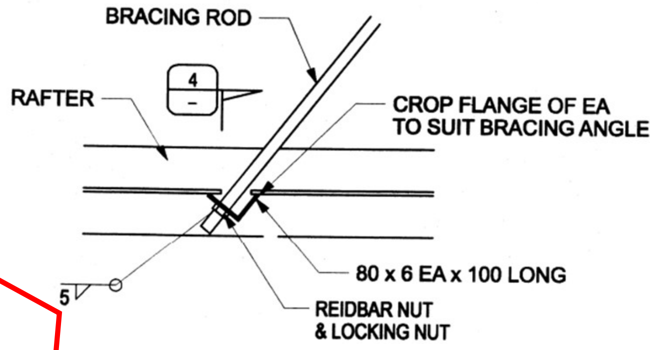


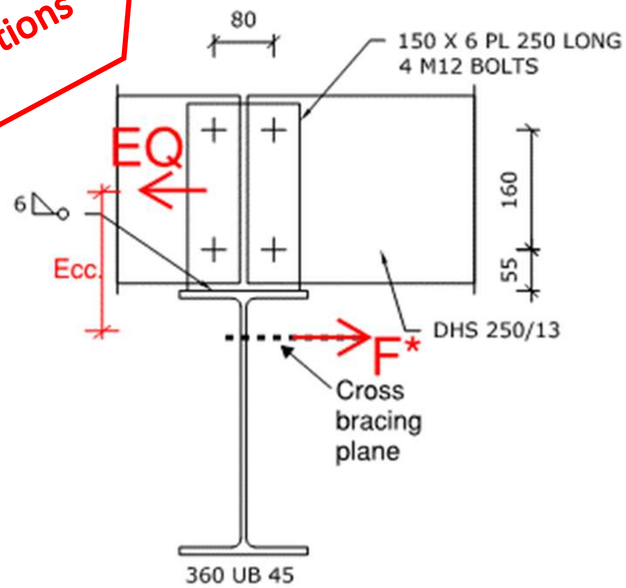


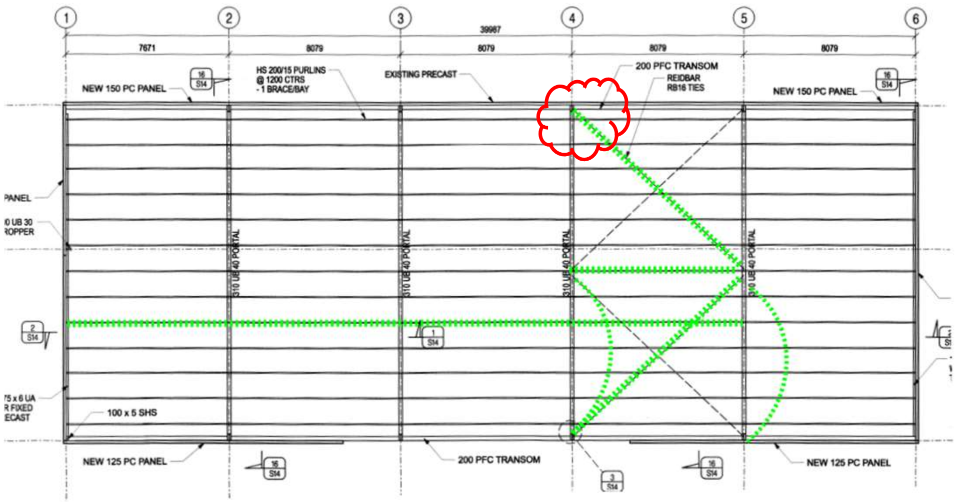
Tip 3 – Node all your connections

Tip 3 – Node all your connections









Connection load path

- Tension brace

??

- Precast panel in-plane

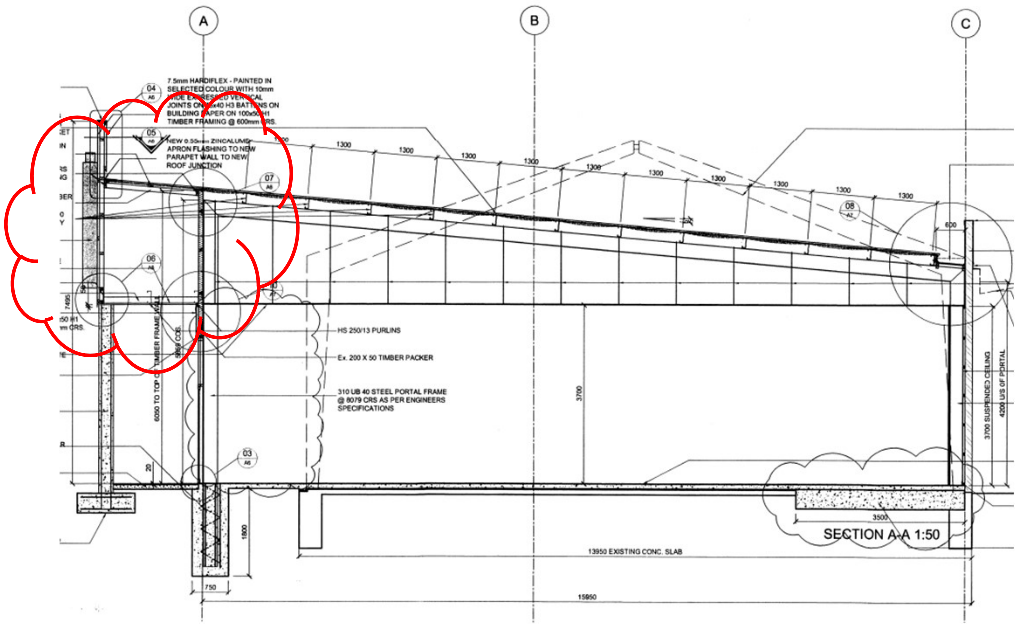
Tip 2- have a load path

No detail provided on the plans!

Eccentric connection from bracing, relying on indirect load paths to transfer from tension brace to the PFC collector

This connection and member cannot transfer the required forces

Timber framed Facade

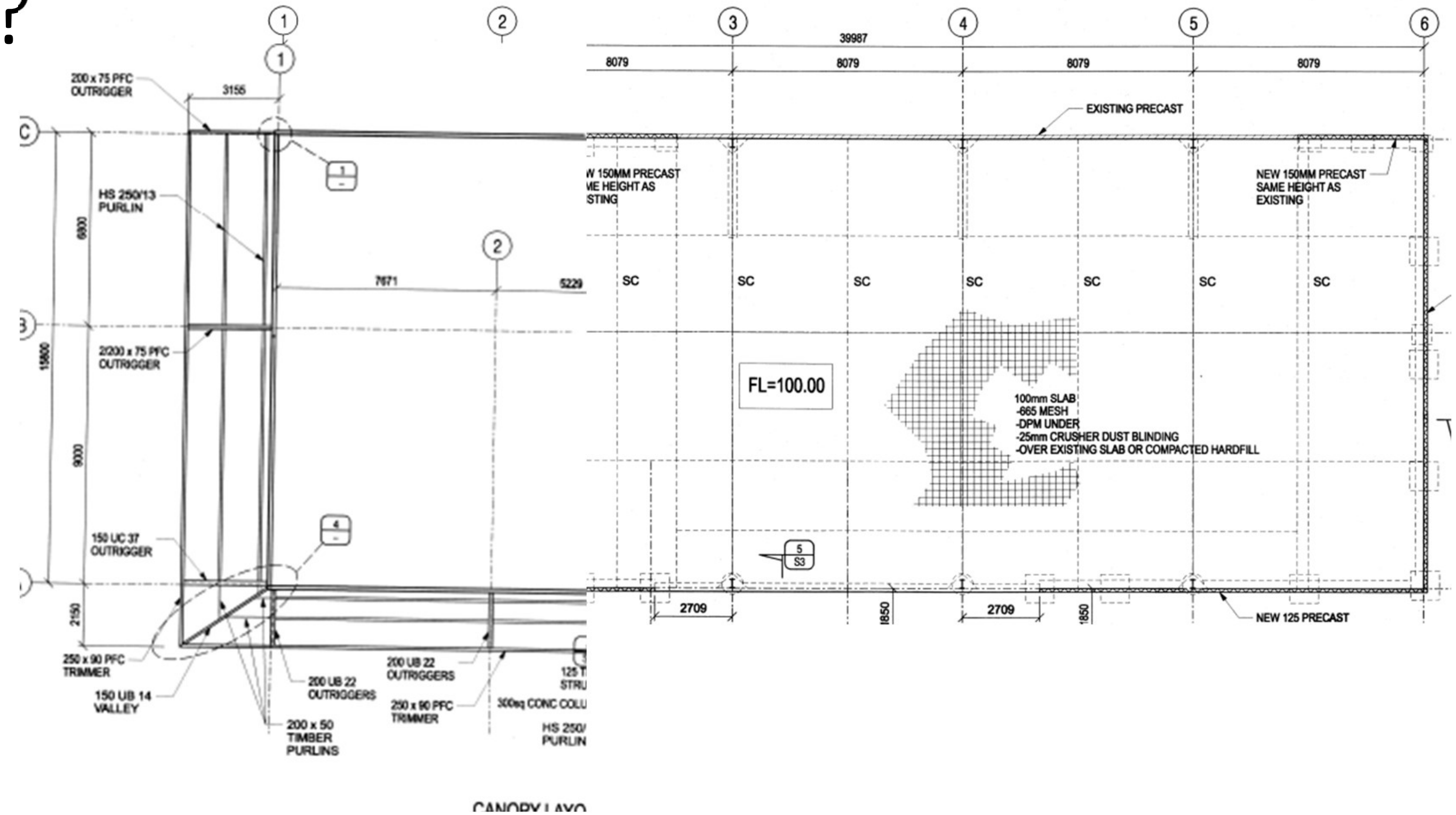


- Light weight, but still needs a load path
- Outside of the acceptable solution (NZS3604)
- Needs to be covered by SED
- No details on engineers' plans

Tip 2- have a load path

Save a dollar?

- Sections of older original buildings retained on boundary wall



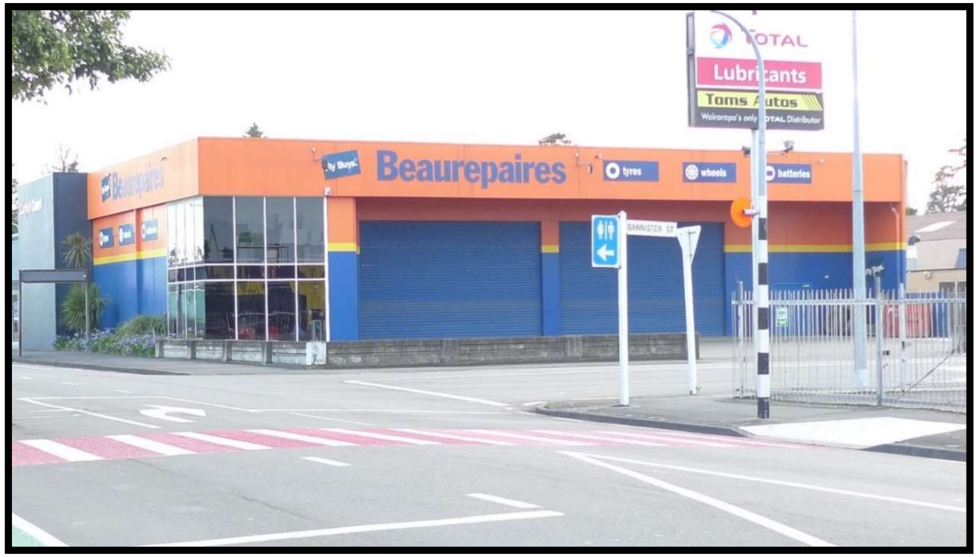
Tip 7 – sometimes you have to say no

- Walls are not flush – as you would expect with older walls
- Connection to the collector has been packed out

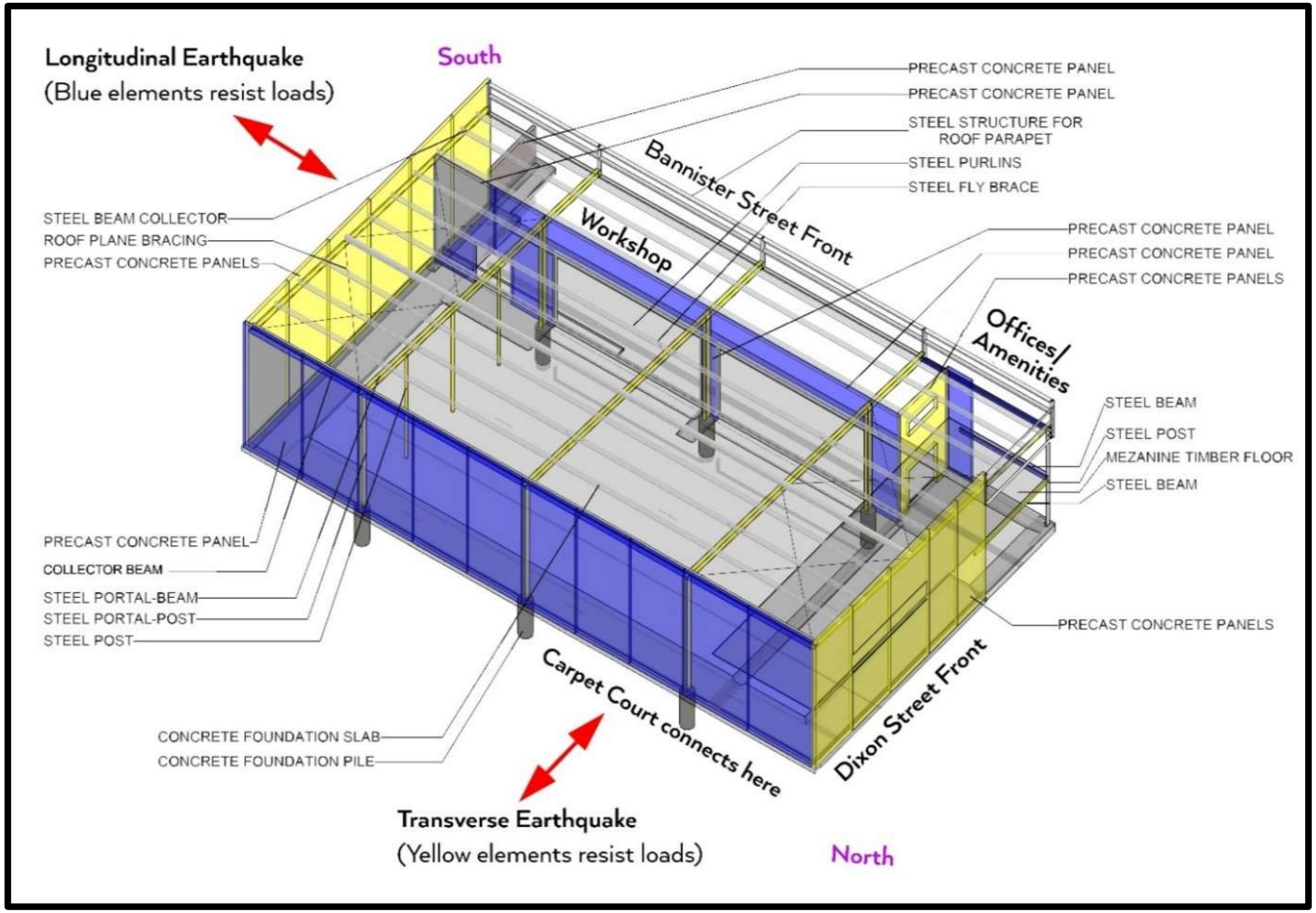
Tip 7 – sometimes you have to say no

What about plant?

Building J

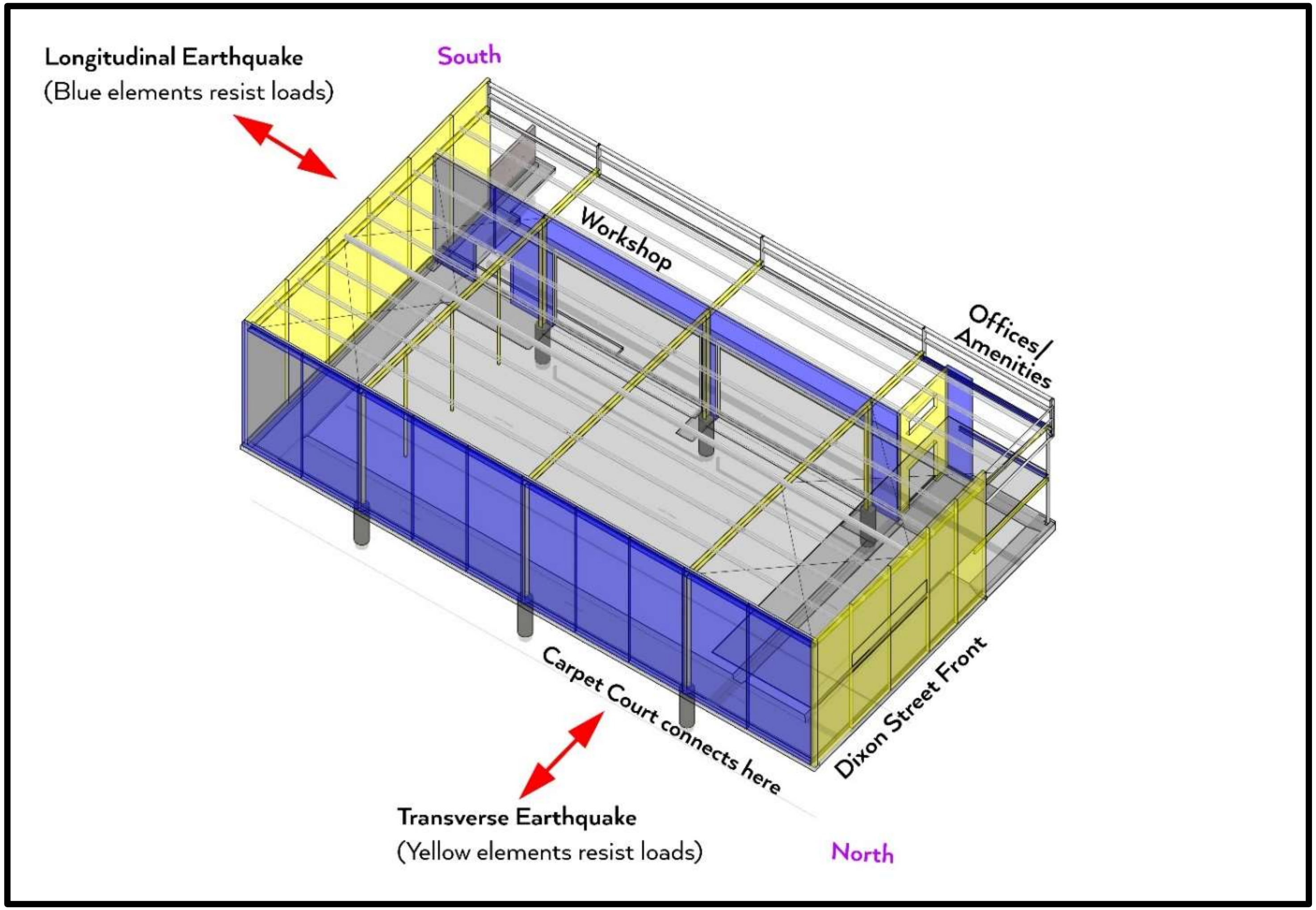


- Built in 2006-2007
- Floor area 345m^2
- Steel portal frames
- Precast panels to perimeter
- Mezzanine floor along on side of building


Longitudinal Earthquake
(Blue elements resist loads)
South
PRECAST CONCRETE PANEL
PRECAST CONCRETE PANEL
STEEL STRUCTURE FOR ROOF PARAPET
STEEL PURLINS
STEEL FLY BRACE
Bannister Street Front
Workshop
STEEL BEAM COLLECTOR
ROOF PLANE BRACING
PRECAST CONCRETE PANELS
PRECAST CONCRETE PANEL
PRECAST CONCRETE PANEL
PRECAST CONCRETE PANELS
Offices/ Amenities
STEEL BEAM
STEEL POST
MEZANINE TIMBER FLOOR
STEEL BEAM
PRECAST CONCRETE PANEL
COLLECTOR BEAM
STEEL PORTAL-BEAM
STEEL PORTAL-POST
STEEL POST
PRECAST CONCRETE PANELS
CONCRETE FOUNDATION SLAB
CONCRETE FOUNDATION PILE
Carpet Court connects here
Dixon Street Front
Transverse Earthquake
(Yellow elements resist loads)
North


Precast Concrete Panels
Beaurepaires
tyres
wheels
batteries
Mezzanine
Boxed in Steel Portal
Column Leg


Fly Brace
Steel Purlin
Collector beam
Steel Portal- Rafter
Steel Portal- Column
Precast Concrete Panels
Steel Post
Precast Concrete Panels
Collector beam



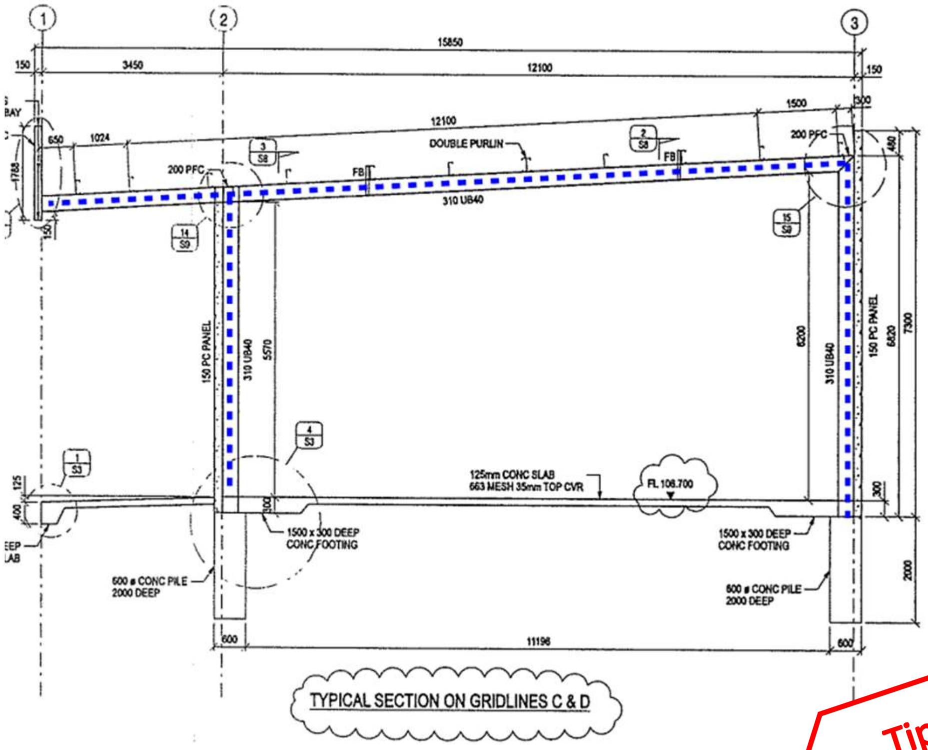
Transverse

- Steel portal frames
- End walls in-plane

Longitudinal

- Tension bracing in roof plane
- Panels in-plane at rear and front wall

Transverse

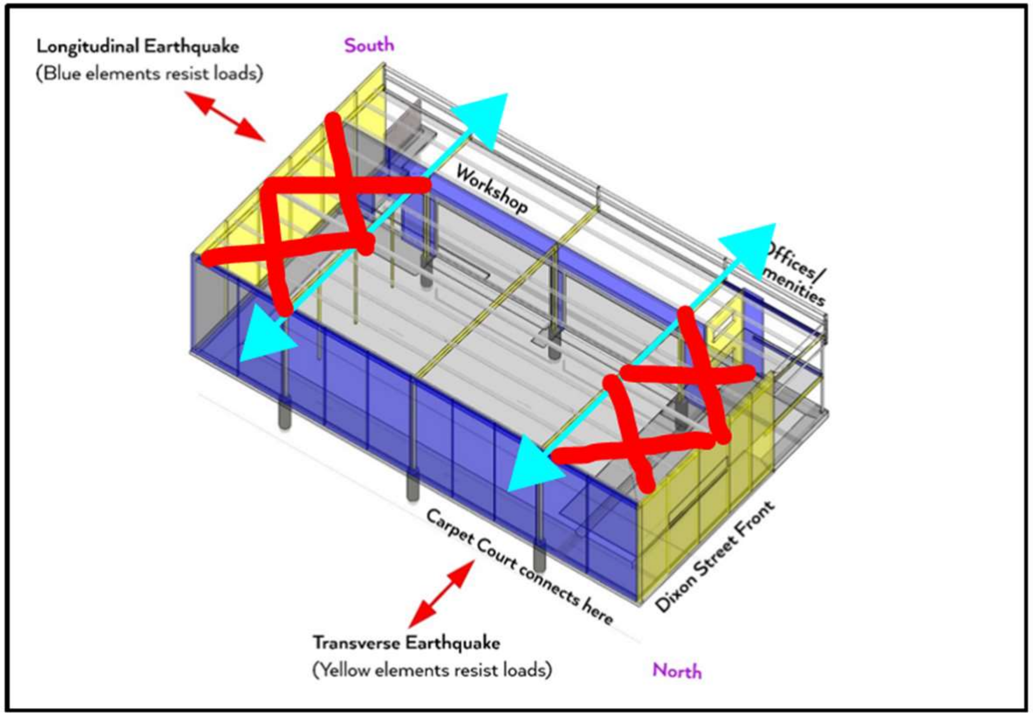


- 310UB40 portal frames at 7.9m centres
- 2 fly braces present – some lateral restraint to bottom flange
- Designer adopted μ=3
- No checks on drift completed
- Designer assumed a rigid fixed base

Frame is well undersized & too flexible

Tip 5 – If you adopt a ductility, make sure you can actually get ductility

Transverse



Displacement Compatibility

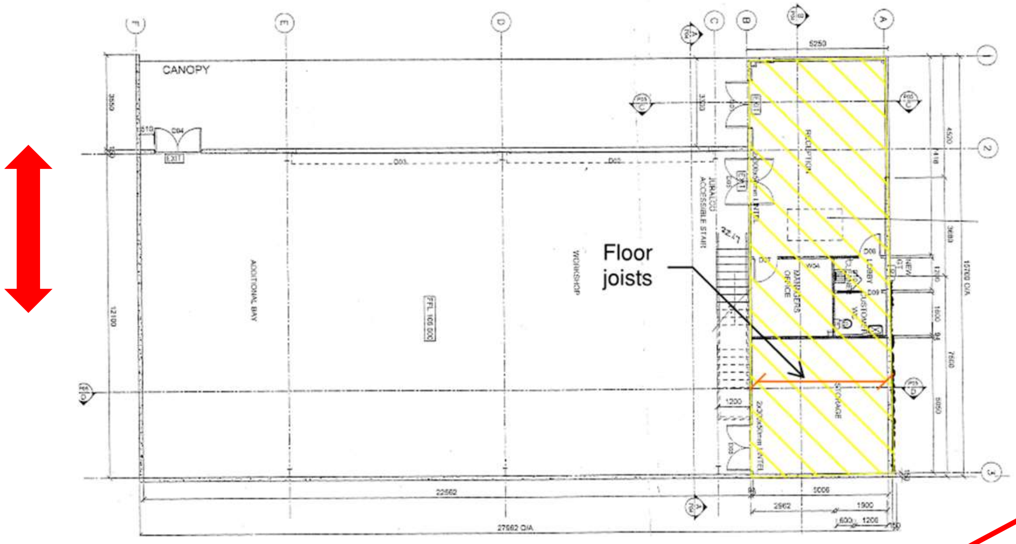
- Two frames are in effect ‘tethered’ to the side walls
- Side walls are precast panels in-plane and will be very stiff

Frames cannot act as intended

Tip 10 – Consider Displacements

SESOC

Transverse – what about the mezzanine?



- Bracing provided by in-plane precast panels on end wall
- No other walls nominated as bracing walls
- No engineered connection to the panels in-plane

Tip 2- have a load path

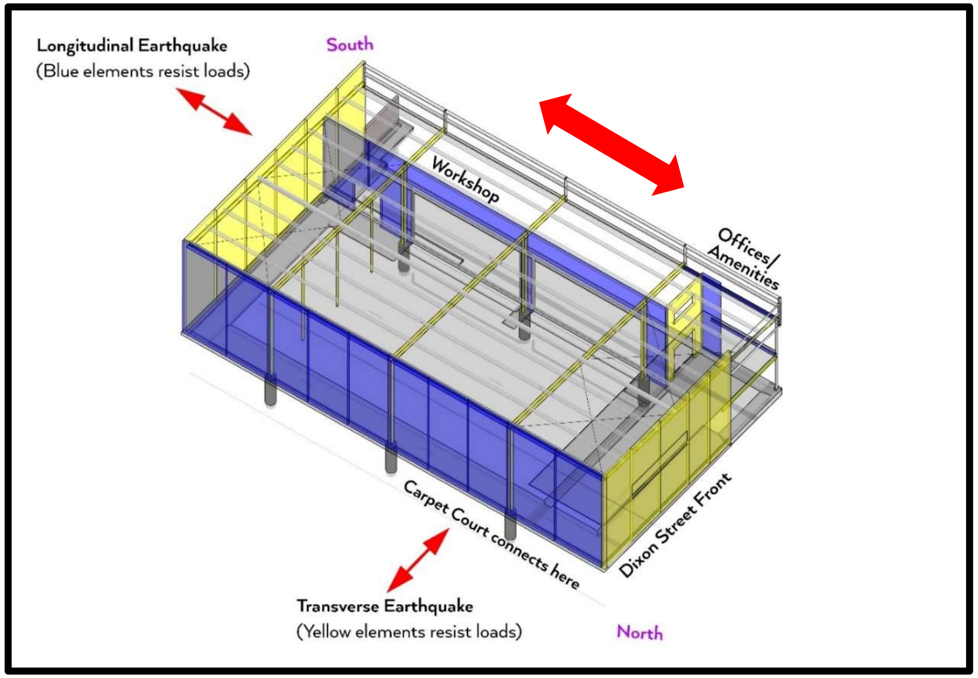
Floor bracing does not have a compliant load path

Transverse – what about the mezzanine?

- Front portion of floor is supported on steel beams and short lengths of panels – glazed shop front
- How are these floor loads dragged back to the in-plane panels?

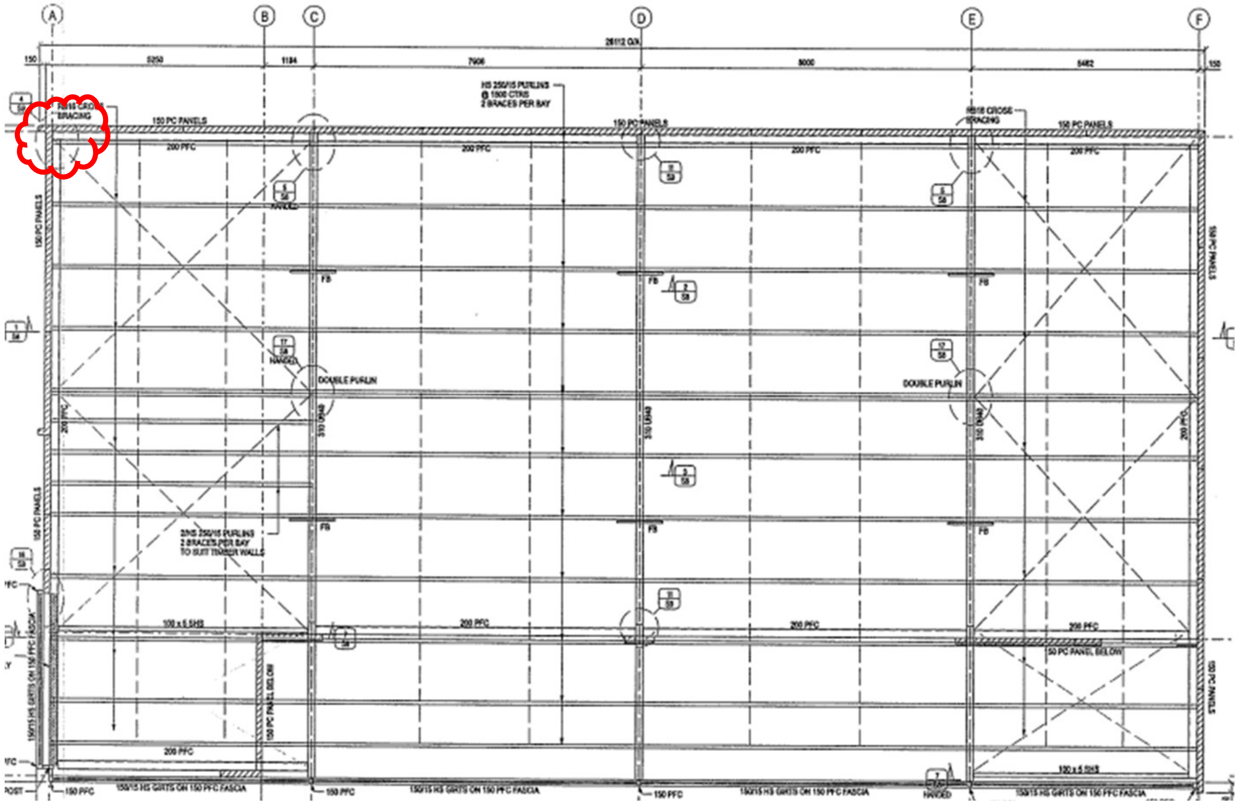
Floor bracing does not have a compliant load path

Longitudinal



- Concrete panel end walls out-of-plane
- Roof plane bracing to transfer loads to the walls in-plane

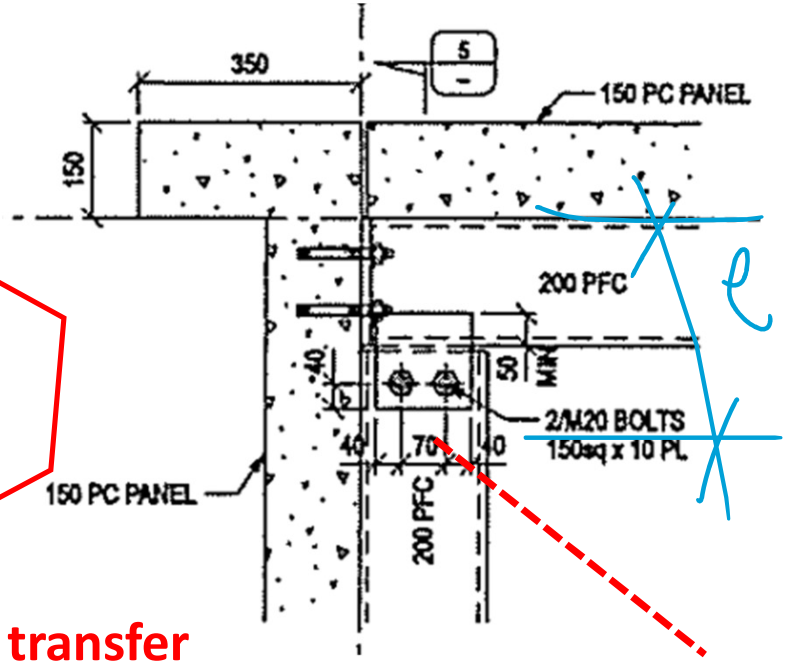
Longitudinal



Look at a connection

- Roof plane bracing to in-plane wall

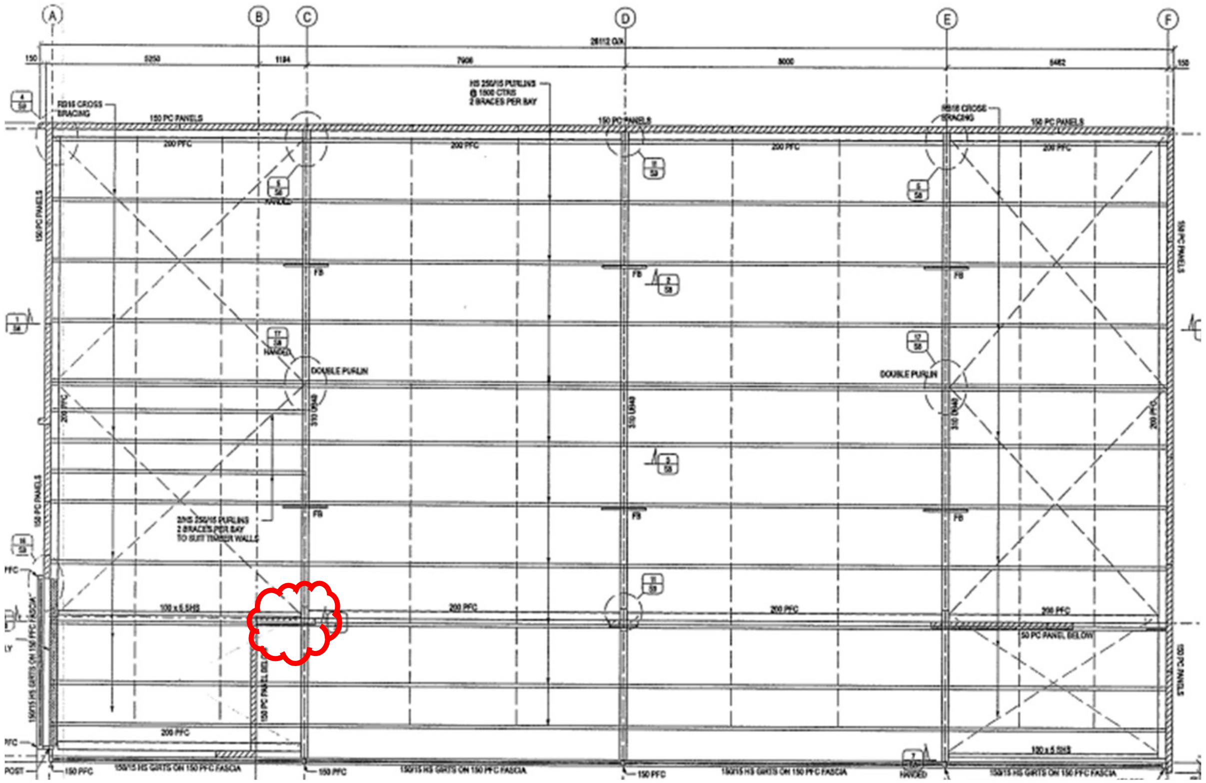
Roof plane bracing to in-plane wall



Tip 3 – Node all your connections

Eccentric connection, bolted join has to transfer collector out-of-plane plus tension brace loads to the in-plane PFC element

Longitudinal

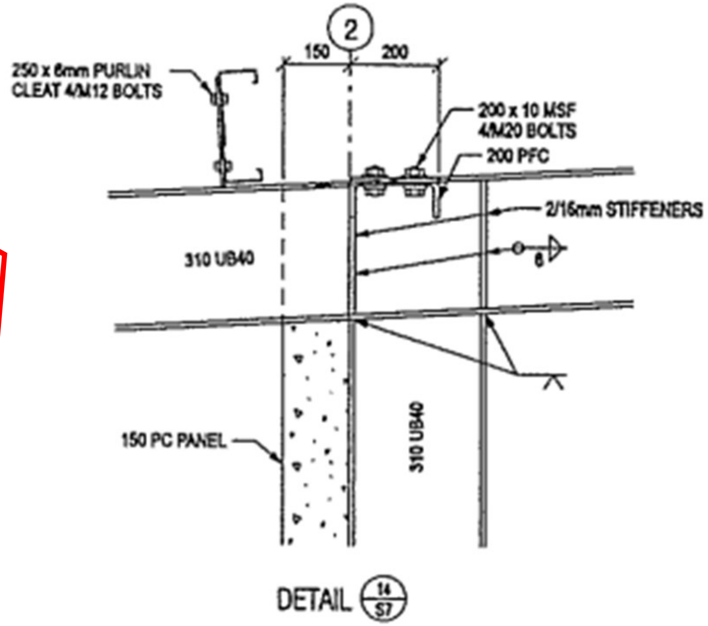


Look at a connections

- Roof plane bracing to wall strut

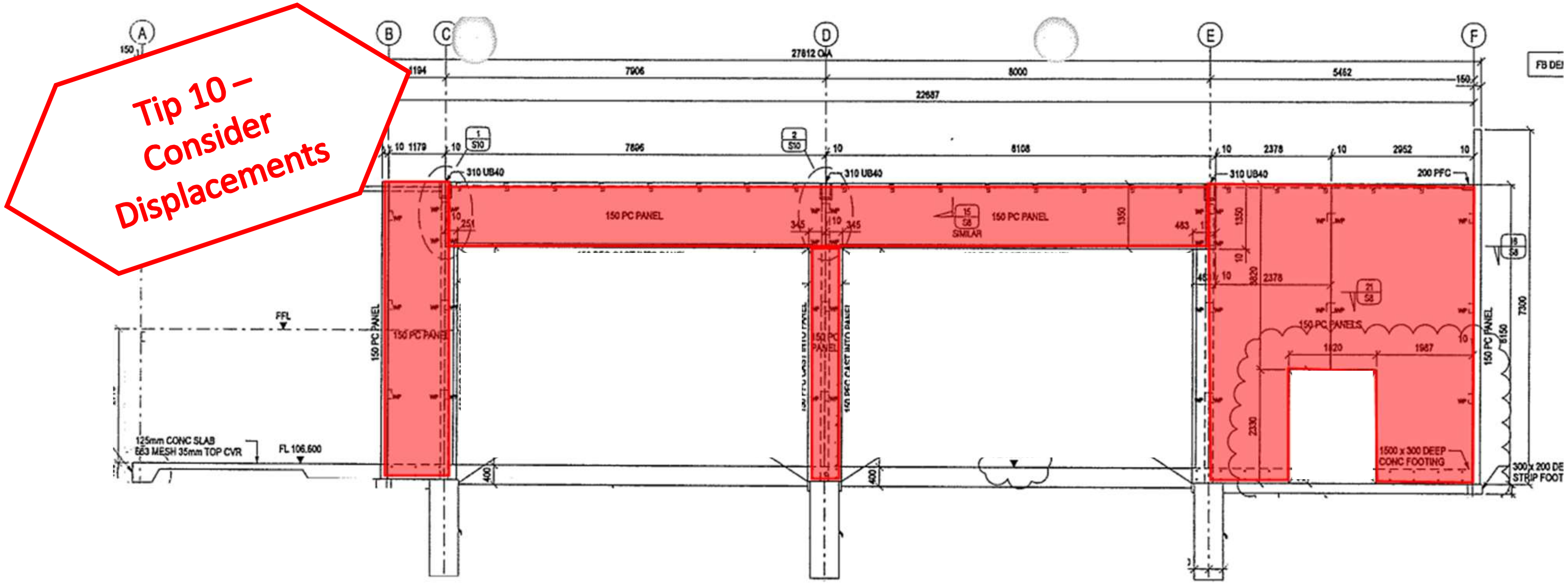
Roof plane bracing to wall strut

Tip 3 – Node all your connections



Eccentric connection and indirect load path from tension brace to the strut

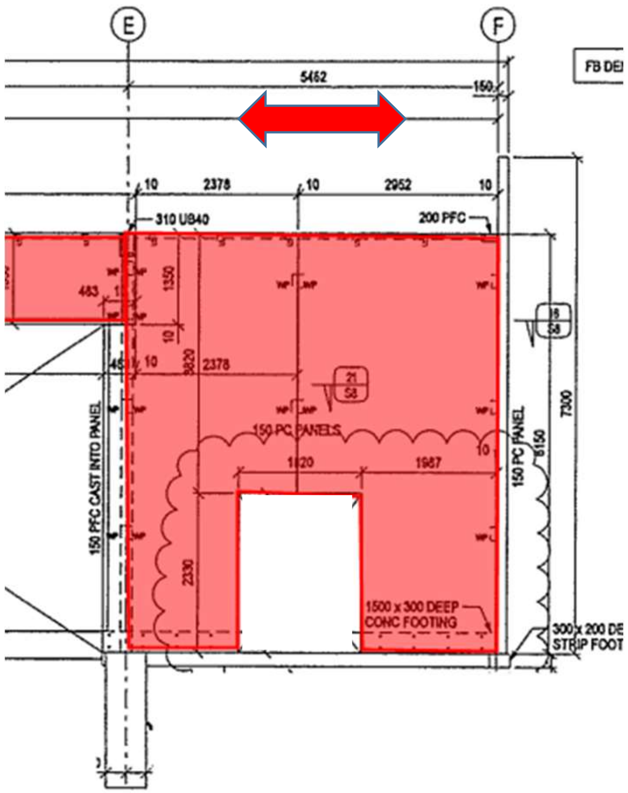
Longitudinal – Front wall in-plane



- Relative stiffness means the end panel will try and brace seismic loads

Longitudinal – Front wall in-plane

Tip 2- have a load path



- Singly reinforced panel in-plane
- Pile on one side, but no direct connection – pile support the portal frames
- Limited connection to return wall on other end of panel

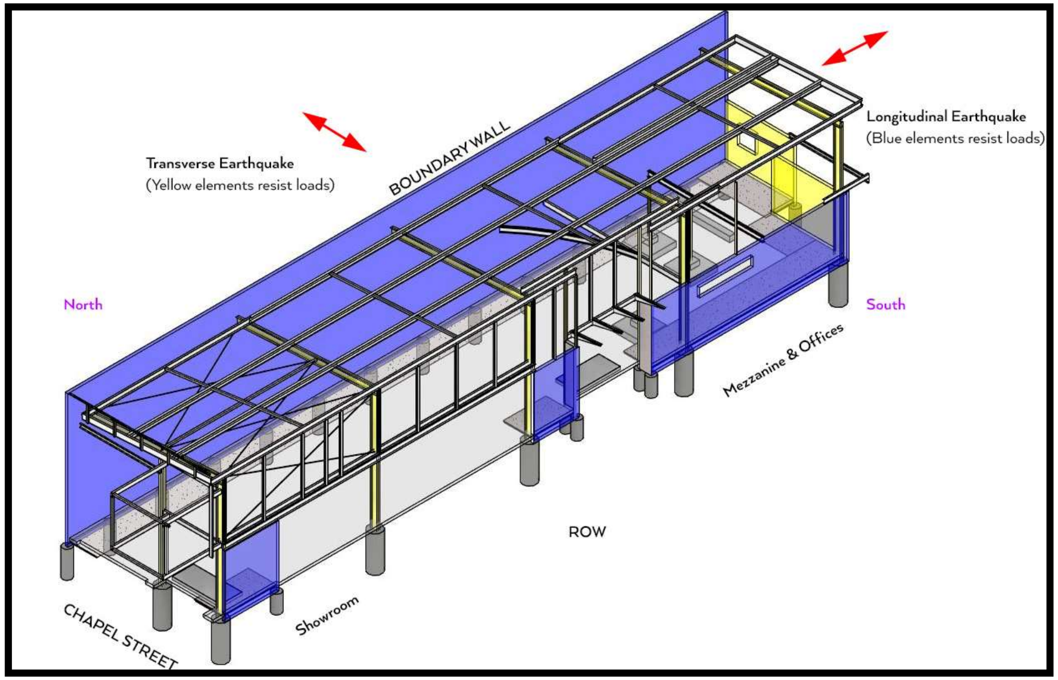
Inadequate load path for panel rocking

Building K

- Built 2011
- Typical modern building
- Single storey 36.7mx7.8m

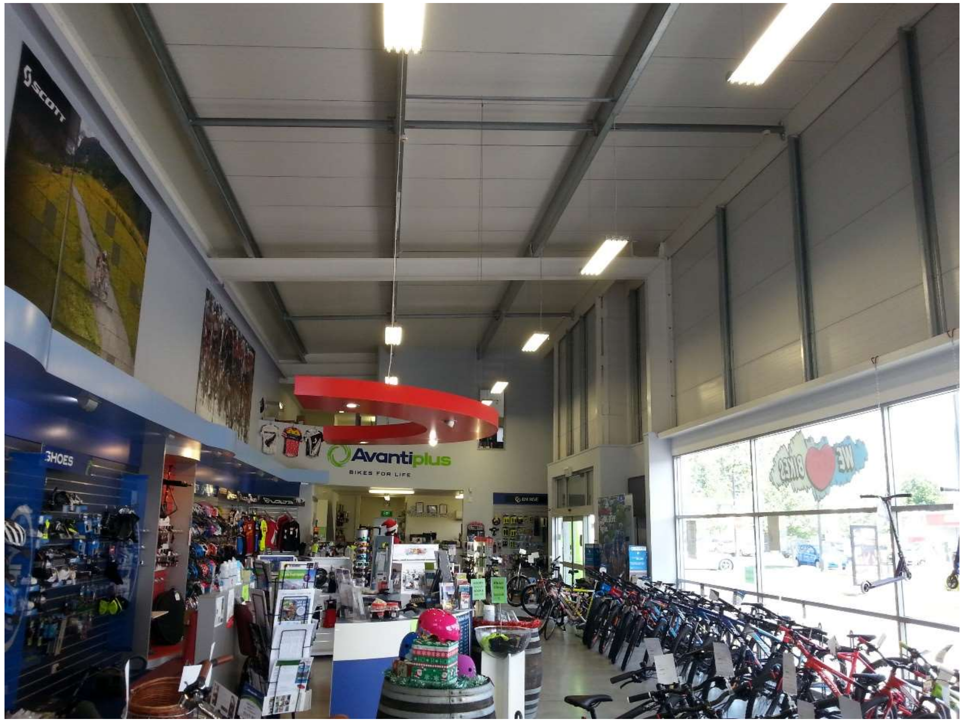
SESOC

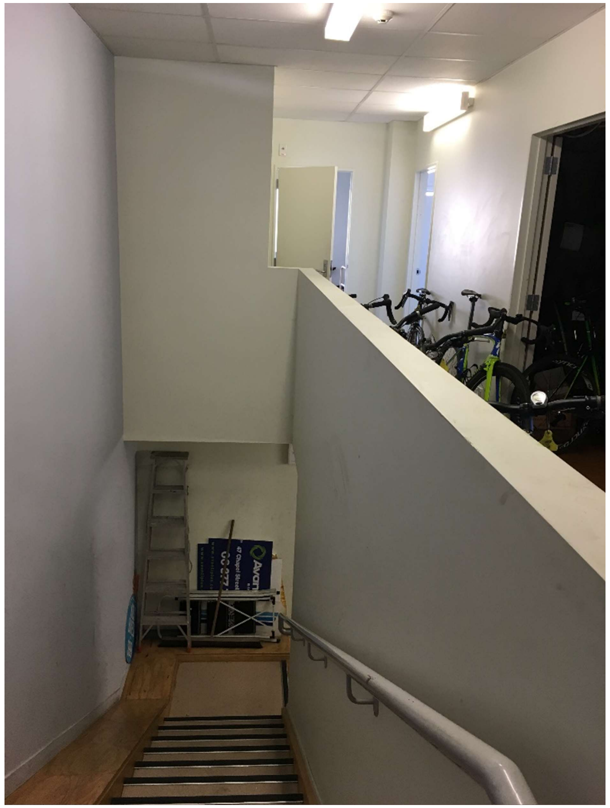
Building K



- 286m^2 low rise structure
- Reinforced concrete foundations
- 360UB45 steel portal frames at 8.6m centres
- 7.7m high 150 thick precast panels on the boundary
- Low level 150 thick precast panels around the other sides
- Mezzanine at the rear
- Tensions bracing in the plane of the roof and one side wall


Light. Stiff.
Agile.
CATEYE


SCOTT
Avantiplus
BIKES FOR LIFE



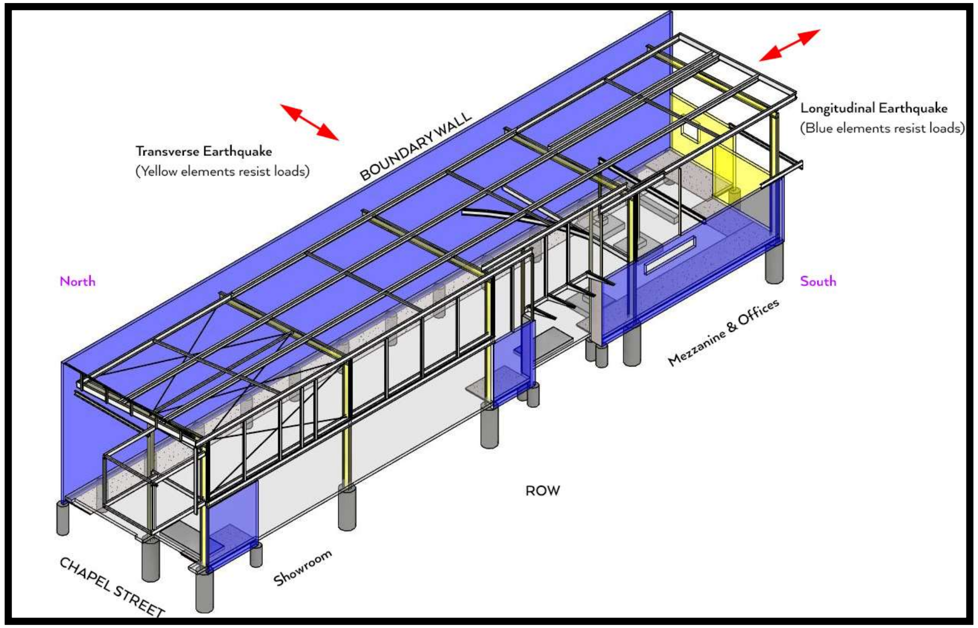
Lateral load resisting system

Transverse

- Earthquake loads are resisted by partial steel portal frames

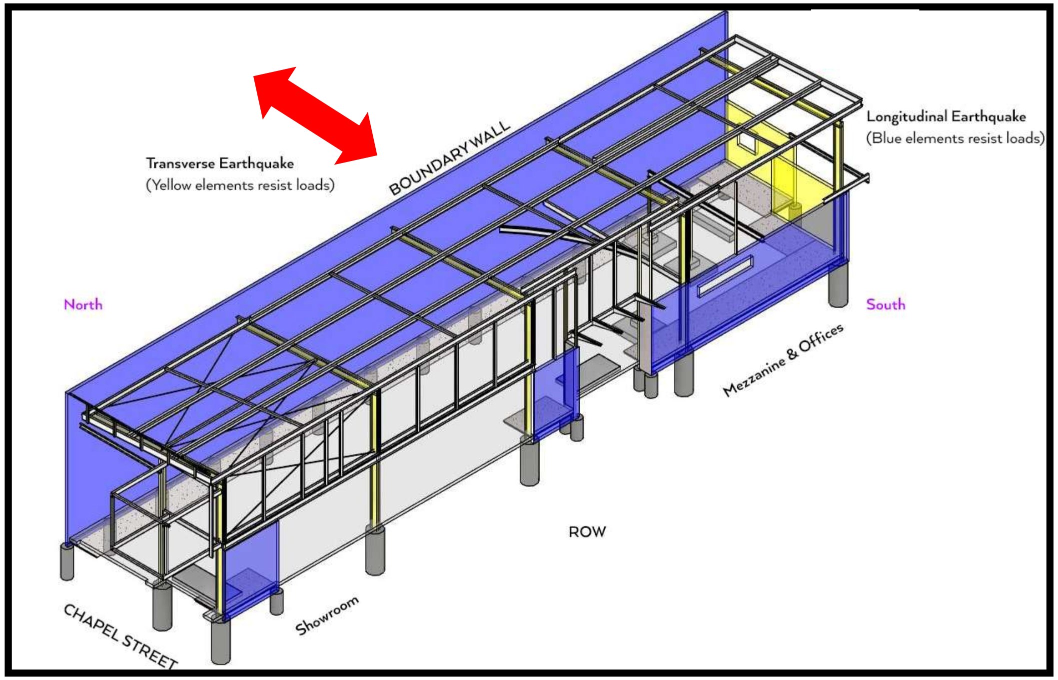
Longitudinal

- Roof plane tension bracing to side walls.
- One side walls in-plane panels
- Other side is tension bracing to mid height, then to in-plane low level panel



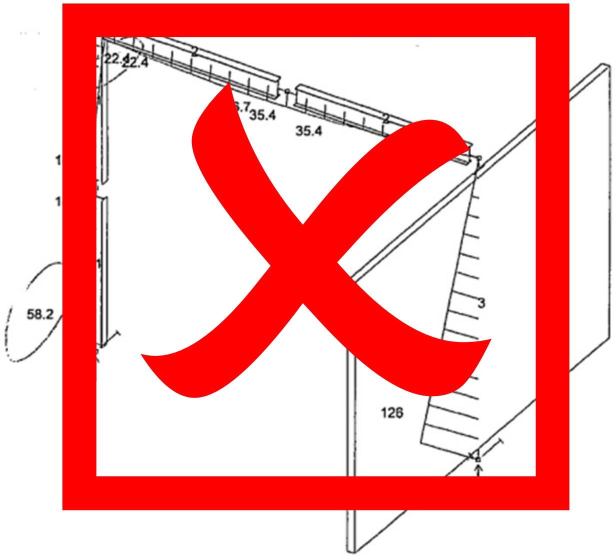
Follow a couple of load paths

Transverse Load Path



Lets look at the transverse partial frames

Incomplete Frames at 8m centres



- Rafter connection to the panel likely to have sudden brittle failure when structure deflects
- This 'system' is not robust, and relies on indirect and poor load paths

Incorrect Assumptions

- Precast panels cantilevering in the weak direction were used as part of the primary system – This is a brittle system
- Base fixity provided – perfectly rigid connection at top of pile - This is a not provided
- Drift not checked

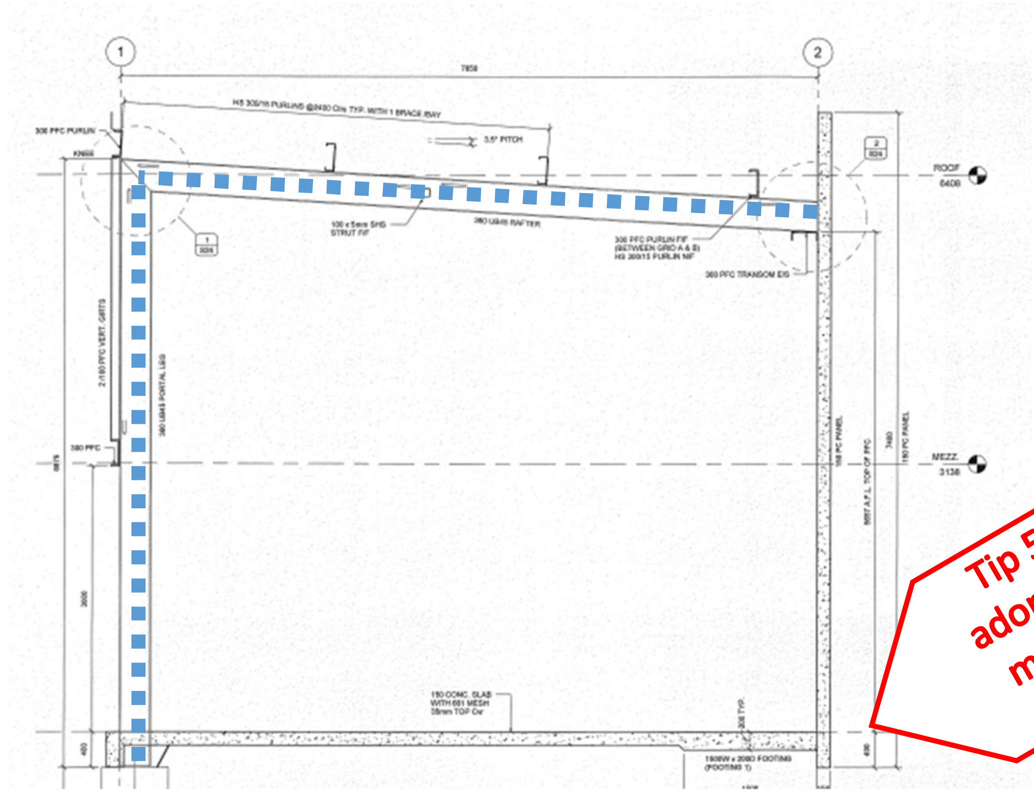
This system is not structurally logical or robust

Transverse

- Partial Portal Frames are 360UB45
- Designer also adopted μ=2, ie limited ductile which needs a category 2 member

360UB45 – does not meet Category 2 requirements

(A 360UB45 is category 3!)



Tip 5 – If you adopt a ductility make sure you can get it!

Implications

- ‘Frames’ can only be limited ductile – and are therefore undersized and non-compliant (even if frames were complete!)

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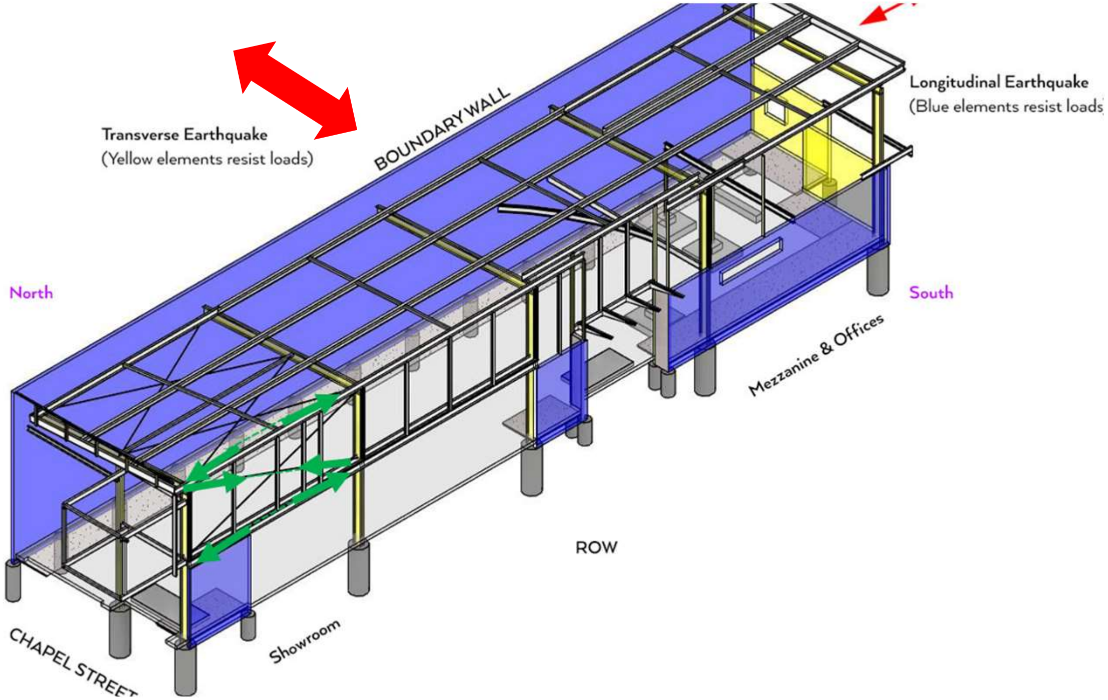
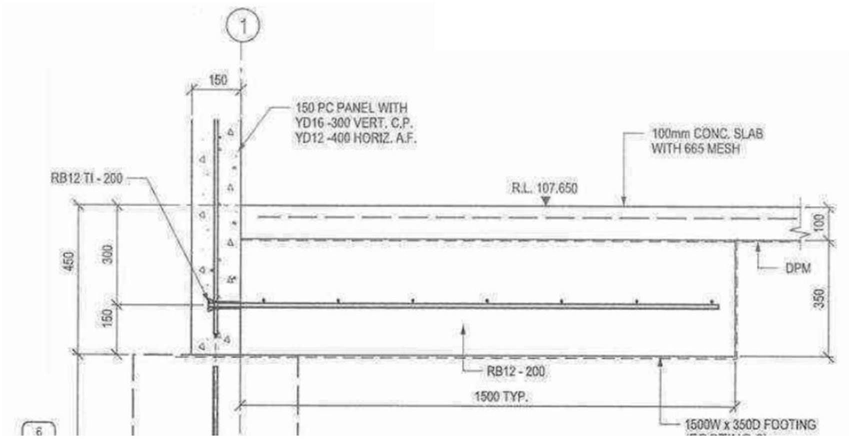
Side wall panels out-of-plane



Boundary Panels

- 150mm thick singly reinforced precast concrete panels 6.5m high
- 300PFC collector at top

Panel base connection



Tip 4 – Connections are critical

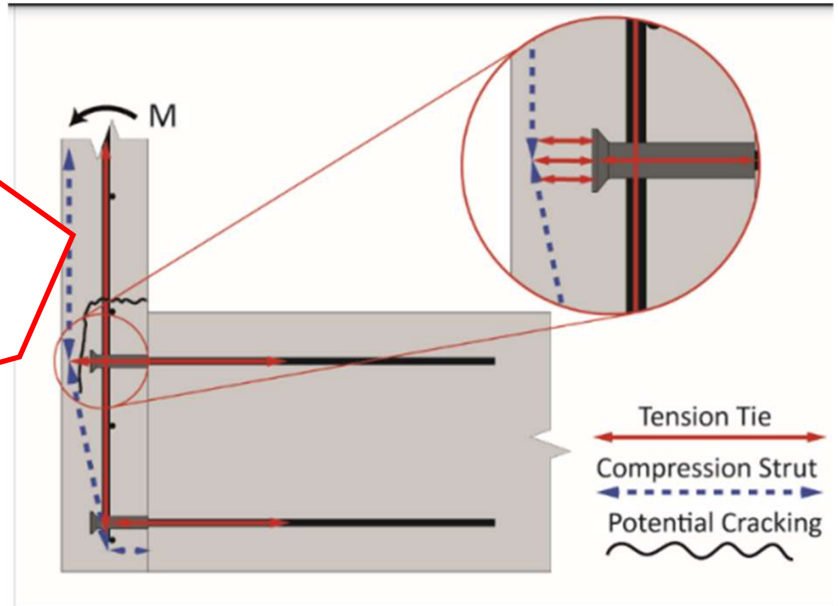


Figure 2: Strut and tie representation of load path for panel to foundation connection using threaded inserts with a shallow embedment depth

Hogan et al 2018

- Proprietary insert, reliance on concrete in tension in cracked zone

This connection cannot transfer the loads required

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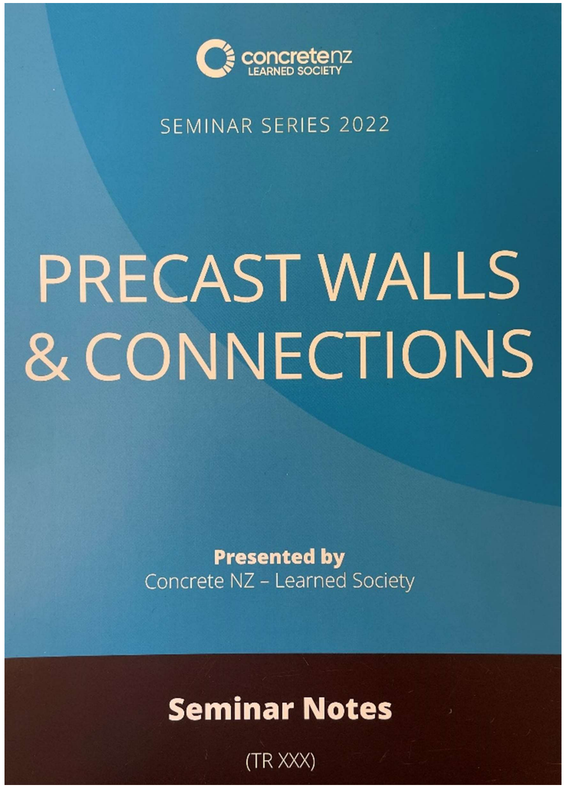
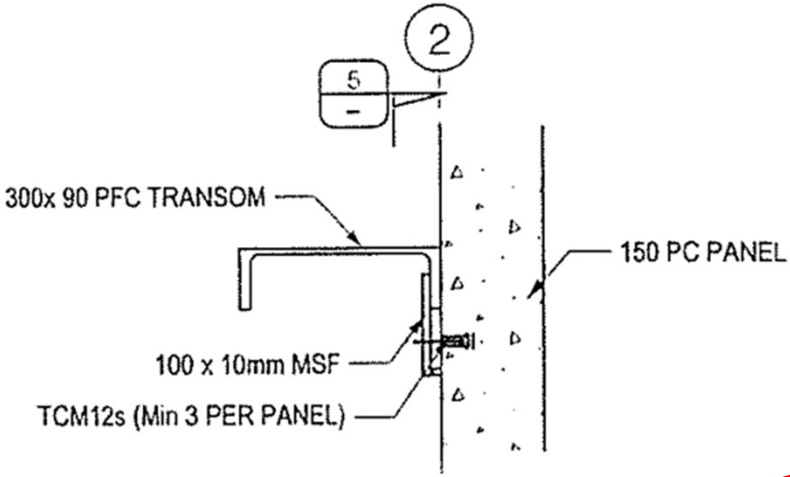
Precast panel base connections

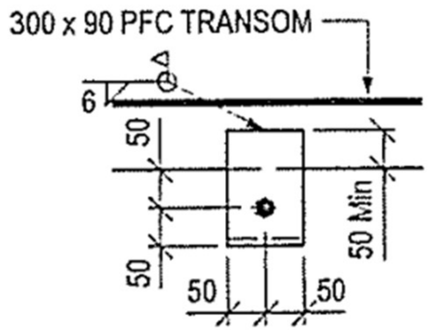


- Refer to latest information from Rick Henry & Lucas Hogan with recommended connection details

Precast Panel top connection - PFC Collector



- Eccentric connection
- No lateral restraint to inner flange
- Designer adopted μ=2 & went with a 300PFC



Connection to PFC highly eccentric!

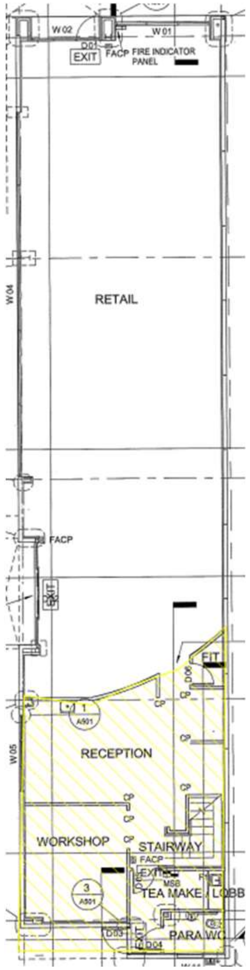
Tip 4 – Connections are critical

Tip 5 – If you adopt a ductility make sure you can get it

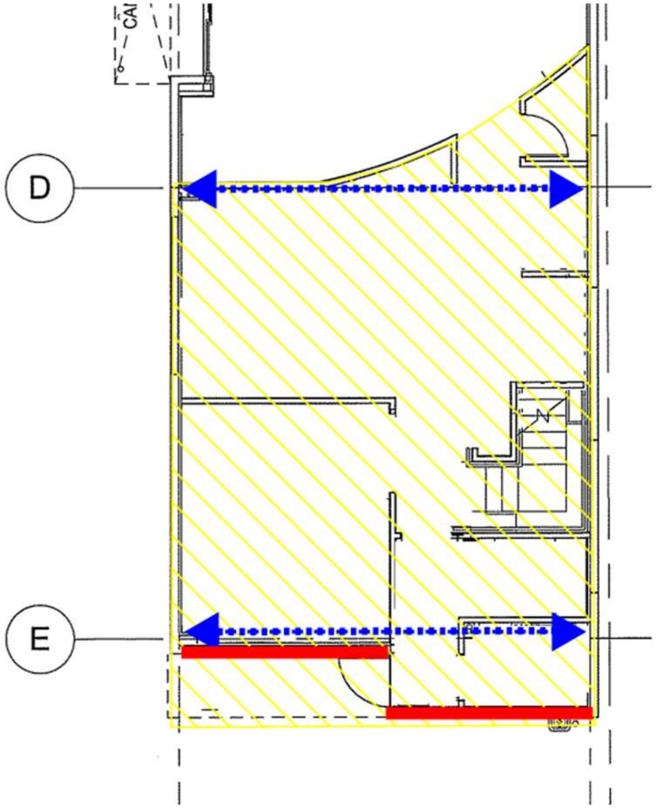
What about the mezzanine?

At the back of the building there is a mezzanine floor

- Transverse System is partial steel frames
- Flexible frames
- Mezzanine floor at the rear
- External walls under the mezzanine low level precast concrete panels

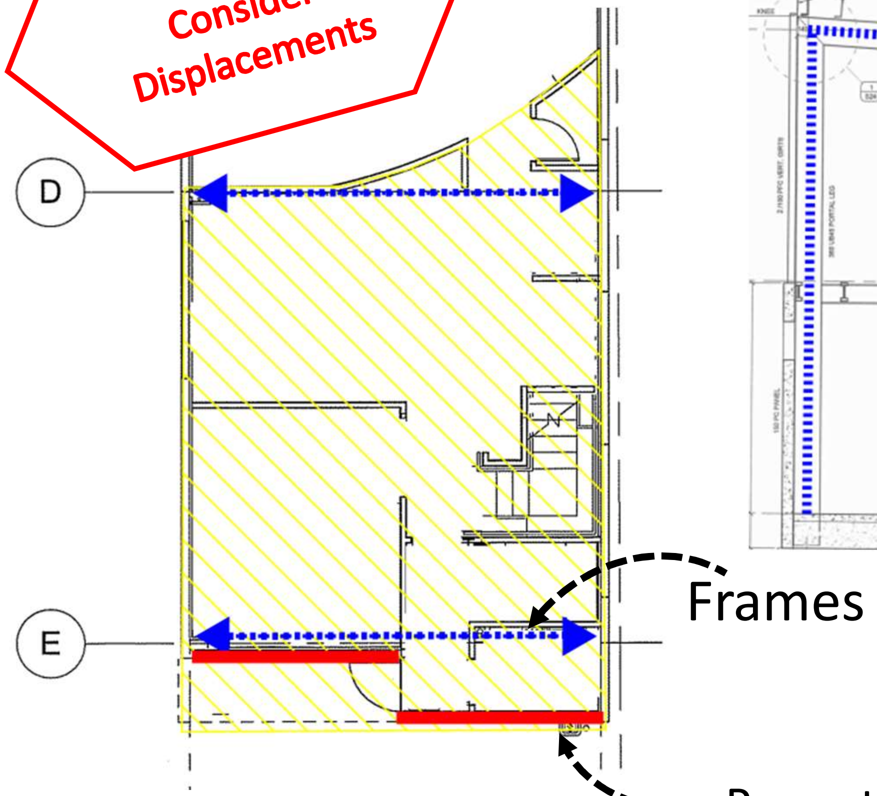


Displacement compatibility?

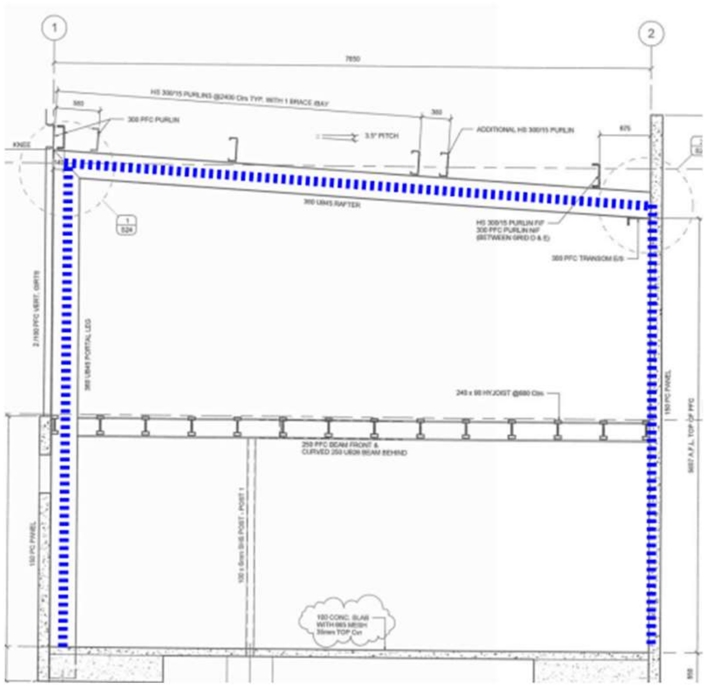


- Flexible steel frames on Grid D and E
- In-plane panels at rear
- Mezzanine floor will try and act as a diaphragm
- Gravity support to floor provided by rear panels as well as gravity beams connected to steel frames

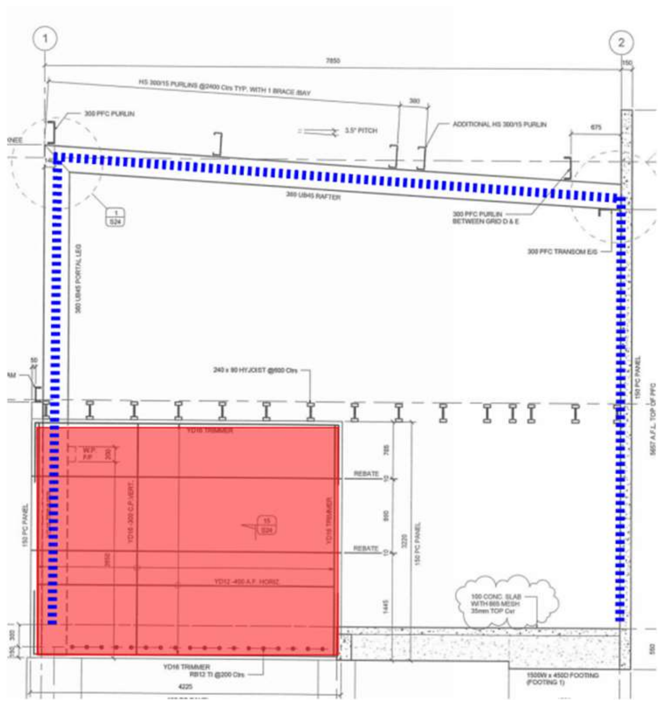
Tip 10 – Consider Displacements



Part Ground Floor Plan



Grid D



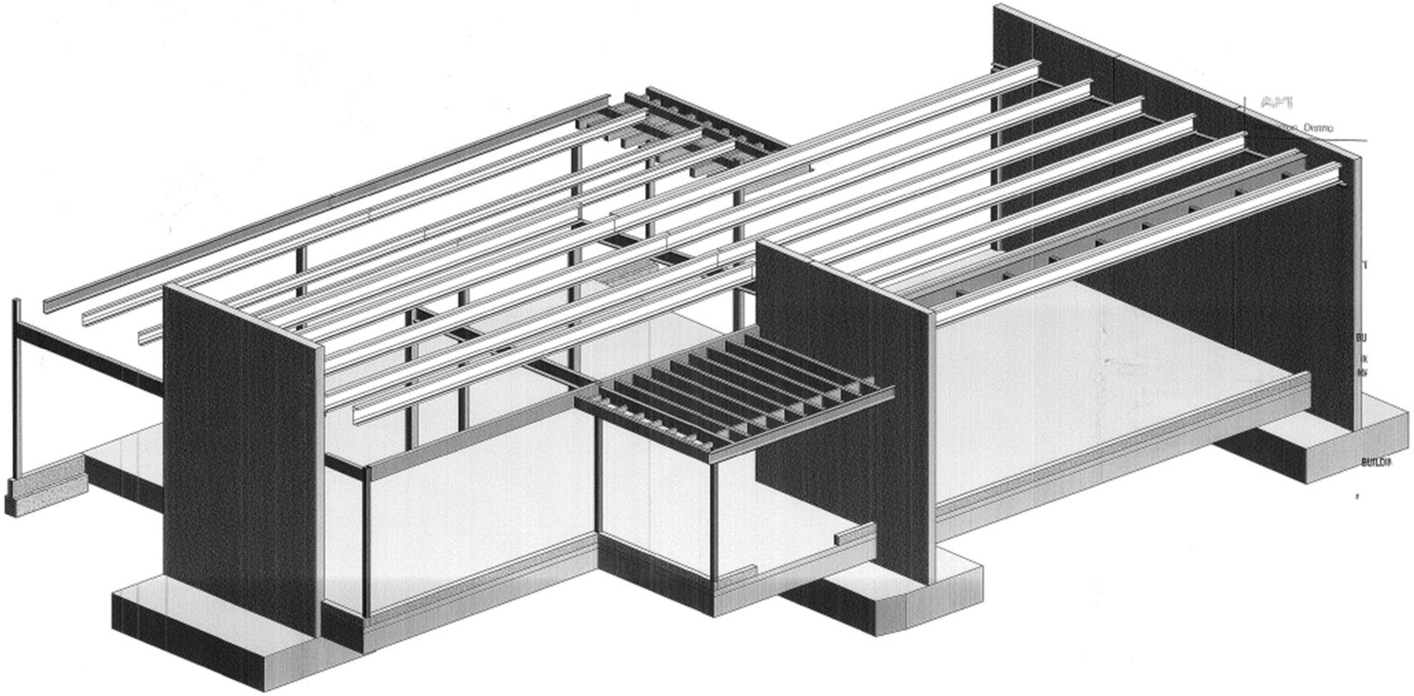
Grid E

Implications

- Frames cannot displace if secondary load paths tie the structure to the in-plane panels
- Load will go to the stiffest load path

Building L

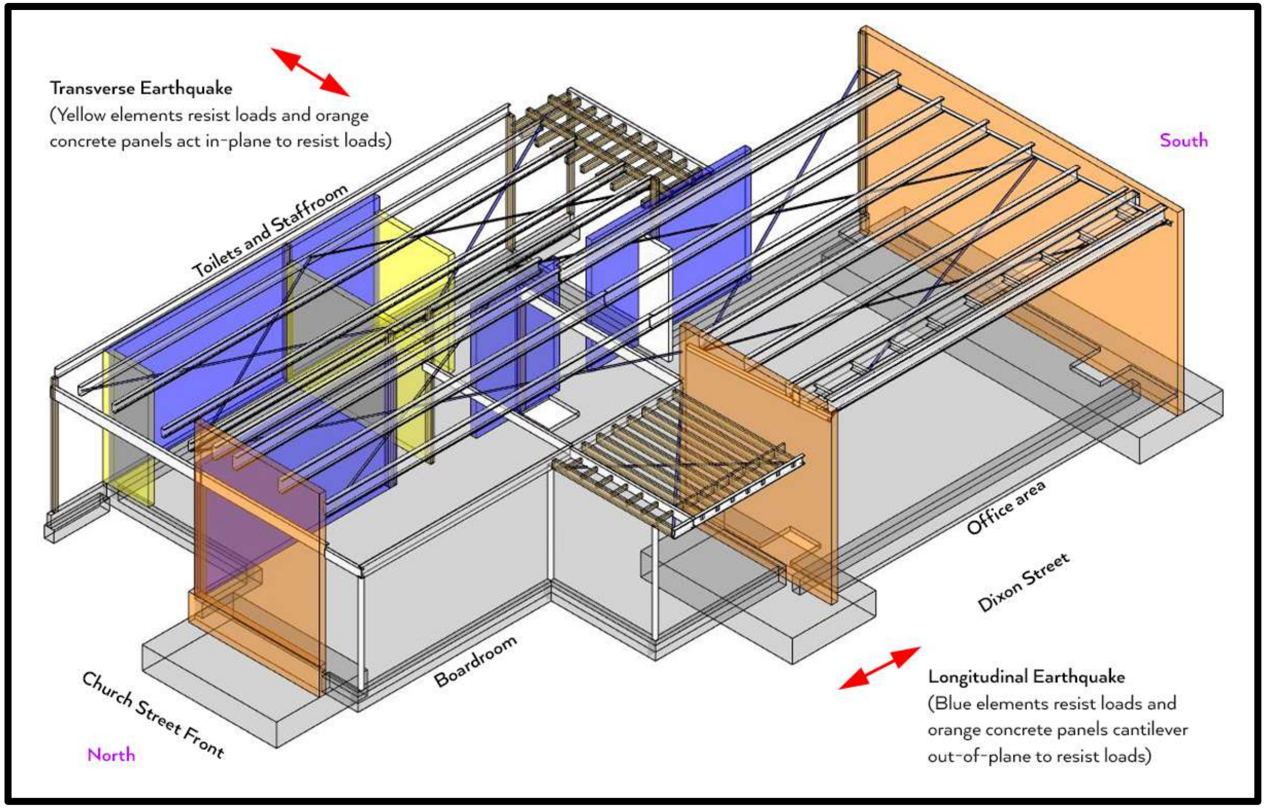
- Single storey
- 157m2



- Three cantilever precast concrete panels
- Steel gravity beams to support the DHS purlins forming the roof
- Timber framed walls at rear and internally


Precast Concrete
panel wall
Precast Concrete
panel wall
FMG


Precast Concrete
panel wall
FMG



Transverse

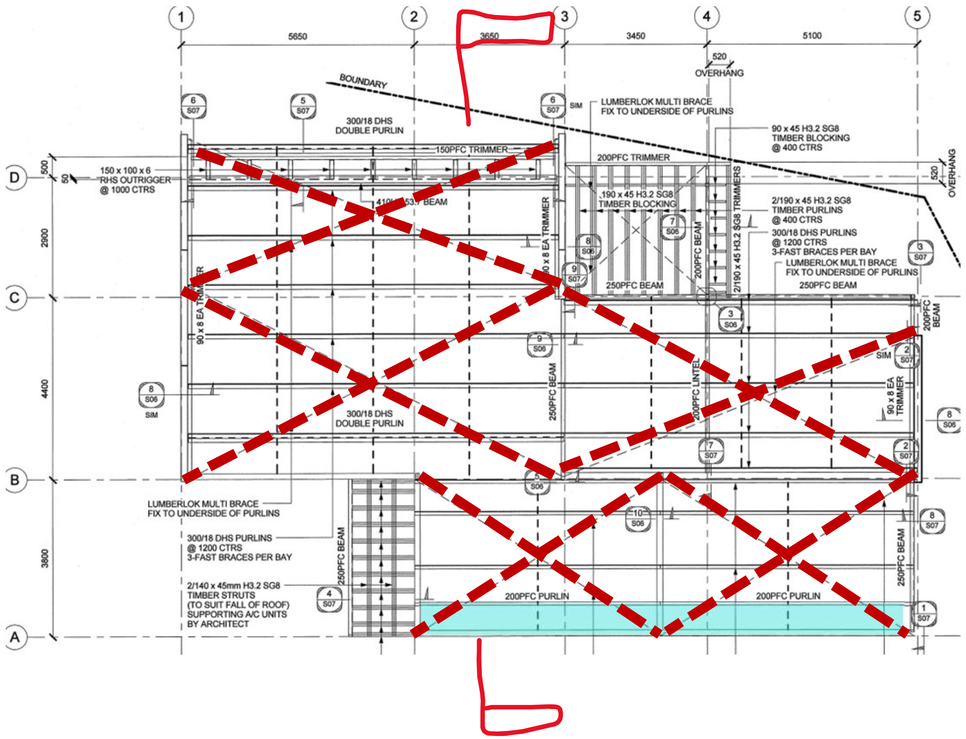
- In-plane plasterboard lined walls (yellow) and precast panels in-plane (orange)

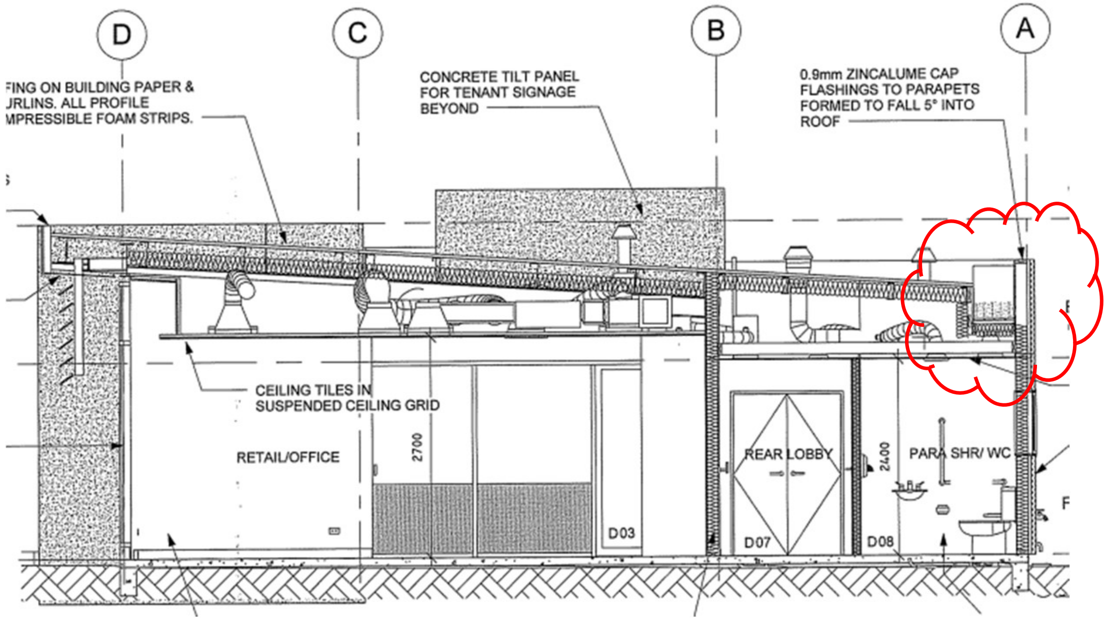
Longitudinal

- In-plane plasterboard lined walls (blue) and cantilevering precast panels out-of-plane (orange)

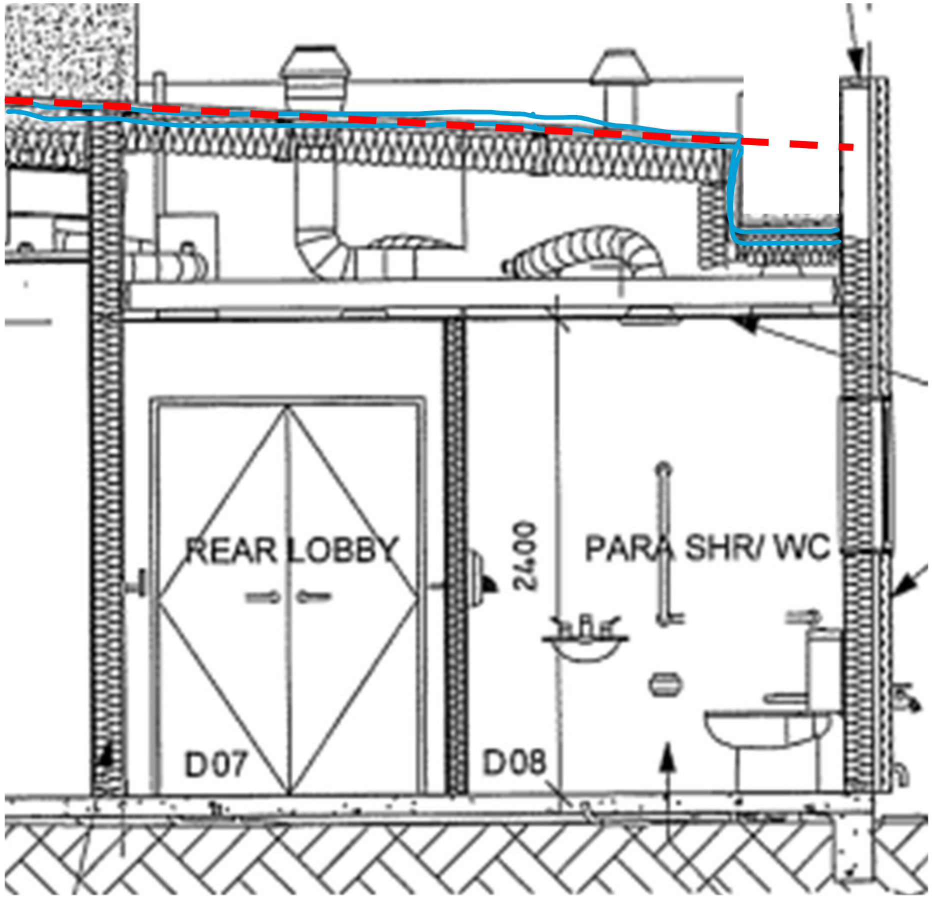
- Roof plane bracing present to transfer roof loads to in-plane elements

Roof plane bracing to transfer loads to in-plane walls





But there is a box gutter along one side of the building…

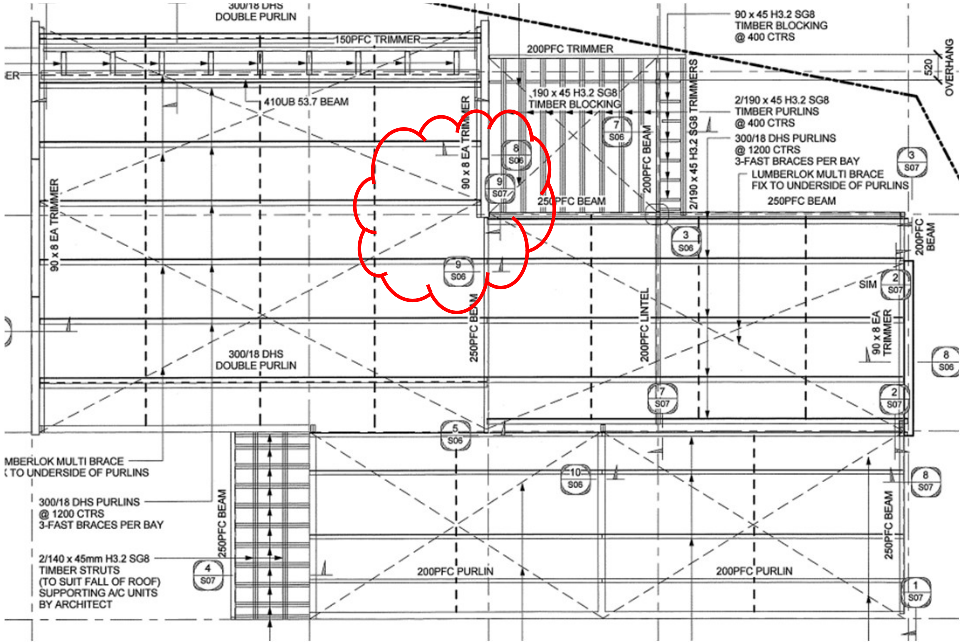


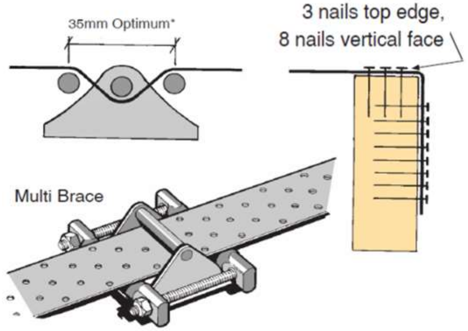
But roof plane is interrupted by the box-gutter – bracing cannot be installed

Tip 8 – Co-ordinate structural with architectural

What about the other roof plane bracing connections?

- Roof plane bracing – Lumberlok Multi Brace specified





MULTI BRACE

0.91mm G300 Z275 GALVANISED STEEL or
0.9mm STAINLESS STEEL 304-2B

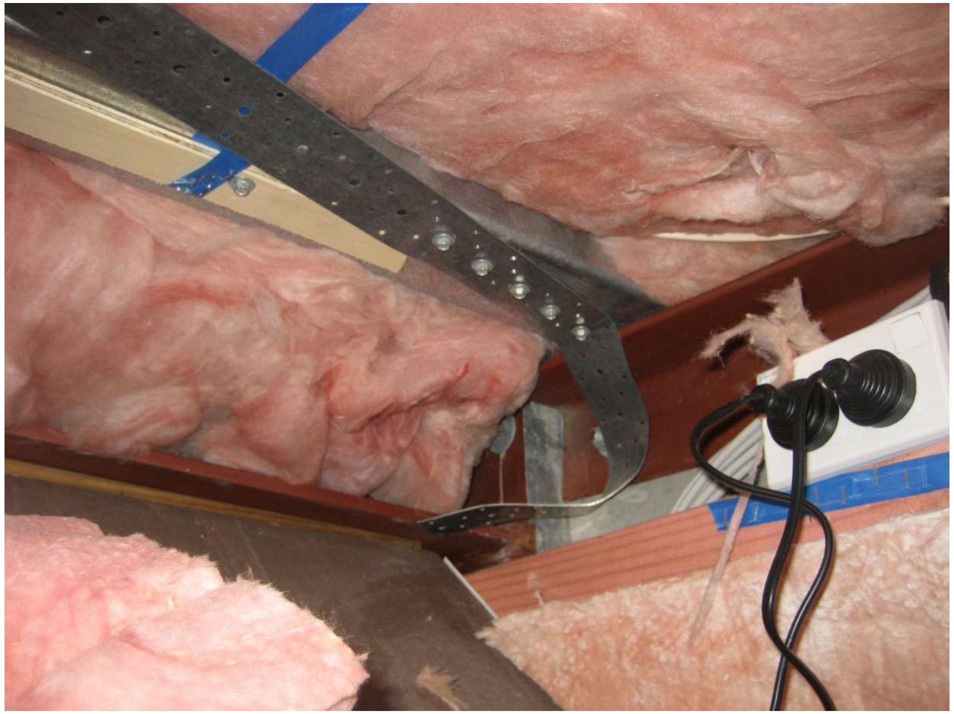
Characteristic Load	Multi-Brace / Multi-Brace with Tensioner#
Tension	14.8 kN
Elongation 0.2mm/m/kN includi… End nail fixing 30mm x 3… # Tensioner not availabl… tension during installa…	

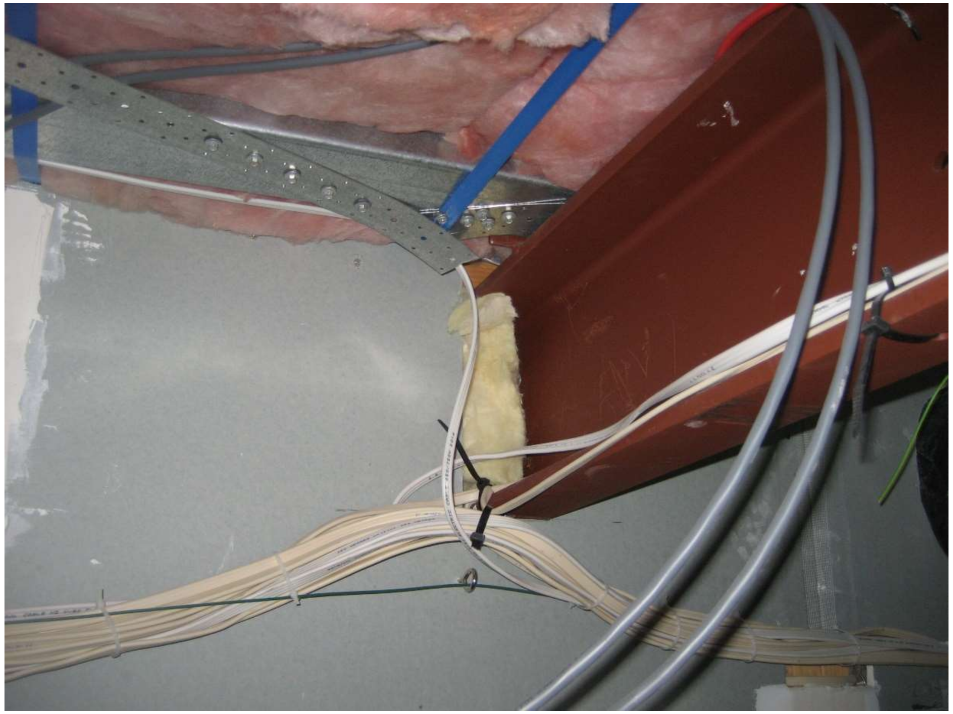
Tip 3 – Node your connections

- However we have a multibrace strap which is meant to 'node' to a precast panel and DHS roof purlin – the standard lumberlok detail is not suitable
- This connection would need to be specific engineer designed

Tip 4 – Connections are critical

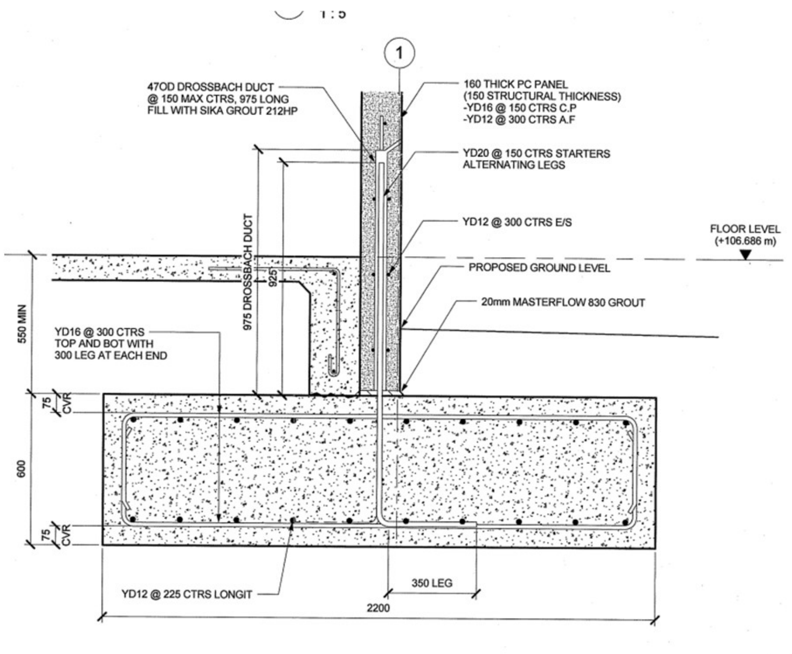
With no details the builders will make it up…





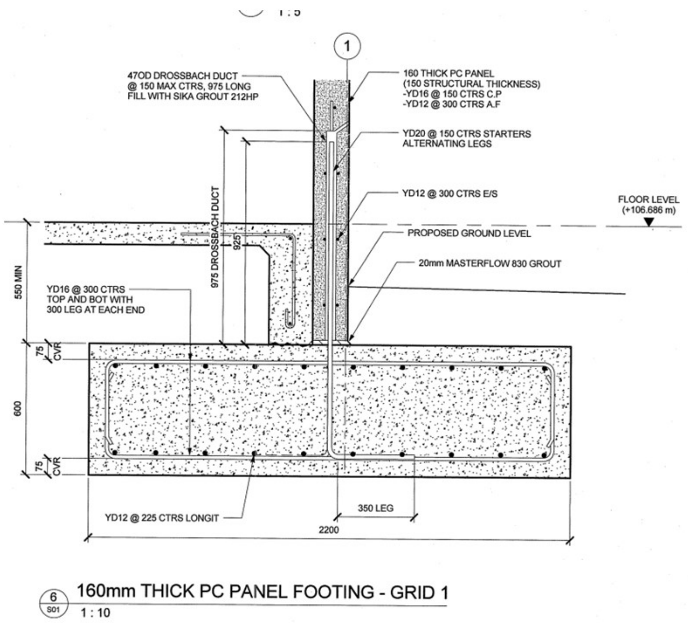
Lets look at the walls

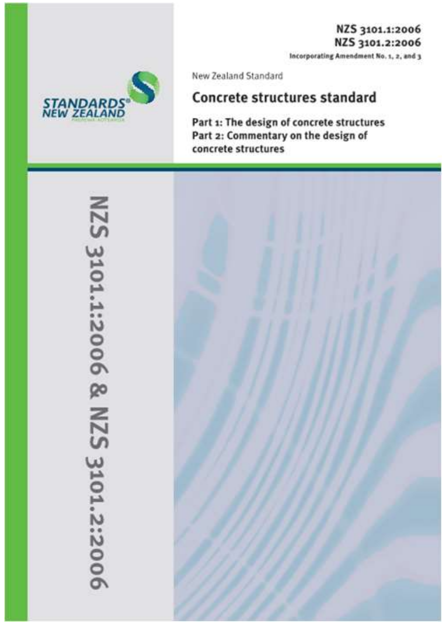
Cantilever panels



6 S01 160mm THICK PC PANEL FOOTING - GRID 1
1 : 10

- Singly reinforced face loaded cantilever panels is not a very robust system - an un-conventional structure
- What happens when the panel section reaches post-yield rotation limits? Brittle structure?
- Drossbachs – No confinement around drossbachs
- What alternate gravity support is there for roof loads?



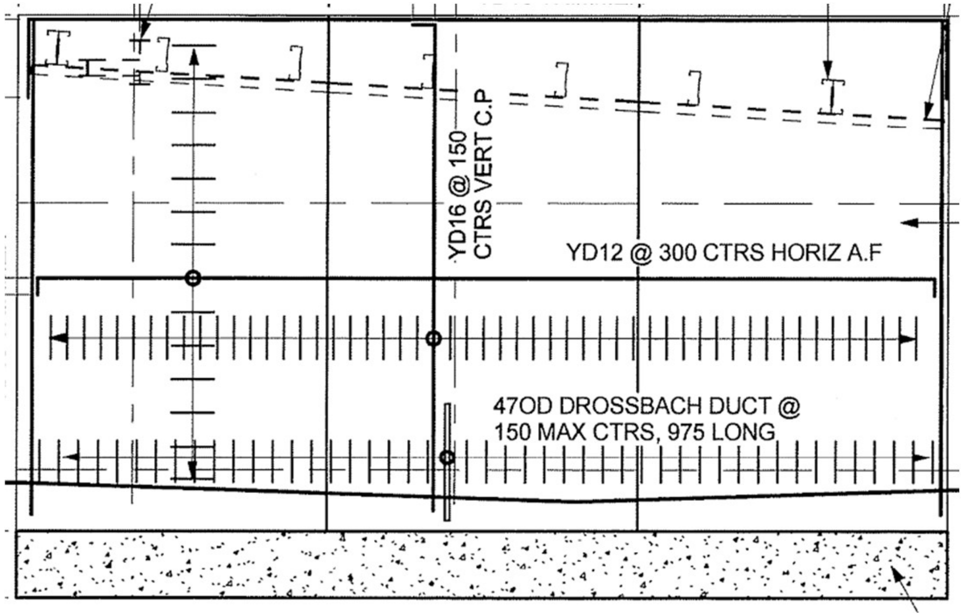


- Panel was highly reinforced to accommodate the out-of-plane actions in a high seismic zone

11.3.12.3 *Minimum and maximum area of reinforcement*

The ratio of vertical reinforcement in any section of a wall shall satisfy the limitations given in (a), (b) and (c) below:

(a) For actions causing bending about the minor axis of singly reinforced walls, the area of vertical reinforcement shall be such that at every section the distance from the extreme compression fibre to the neutral axis shall be equal to or less than $0.75c_b$;



- 150 thick panel with 20mm dia Grade 500 starters at 150 centres

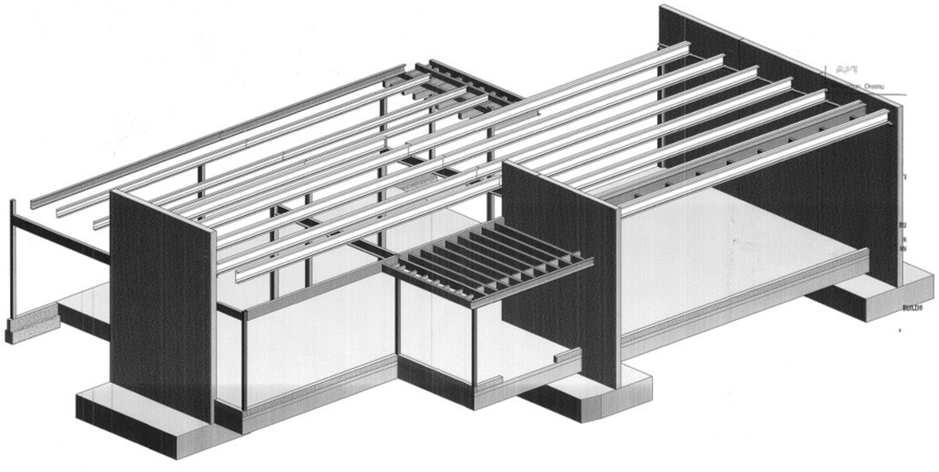
$$\rho_b = 0.85\frac{f_m'}{f_y}\beta_1\left(\frac{0.003E_s}{0.003E_s + f_y}\right)$$

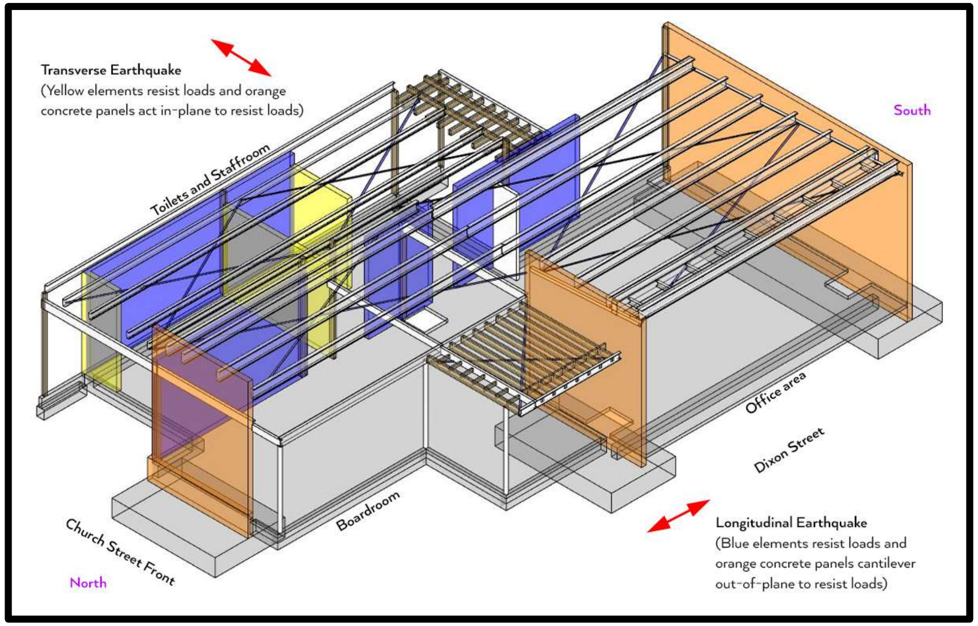
- Panel is over-reinforced – has a balanced failure

Tip ? –
Avoid brittle
failures

SESOC

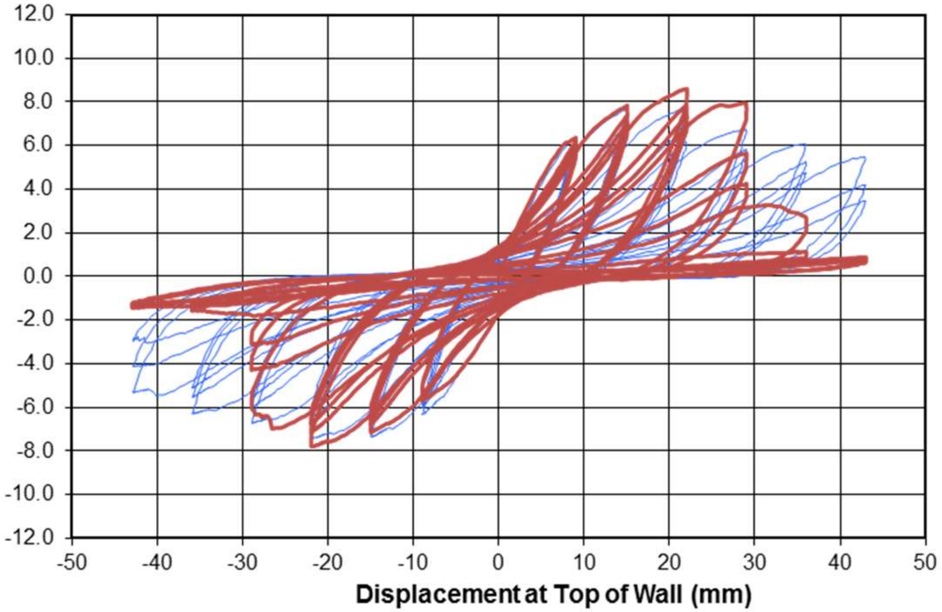
Mixed Systems





- Back portion of the building is light timber framed
- Bracing walls specified with standard Gib products

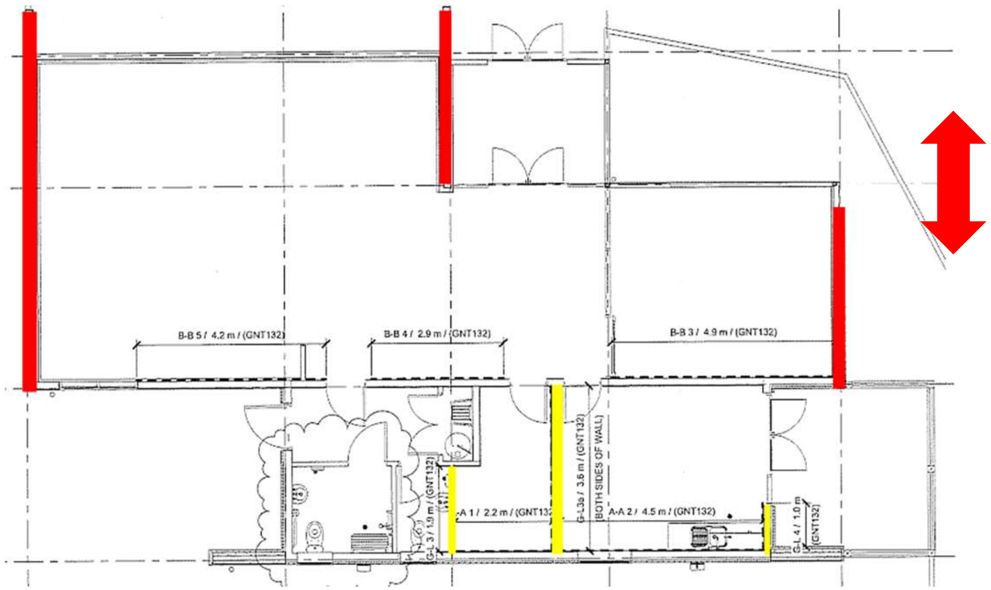
Deformation compatibility?



Typical P21 test for plasterboard lined wall

How much will a 150 thick precast panel deflect in-plane?

Transverse Direction



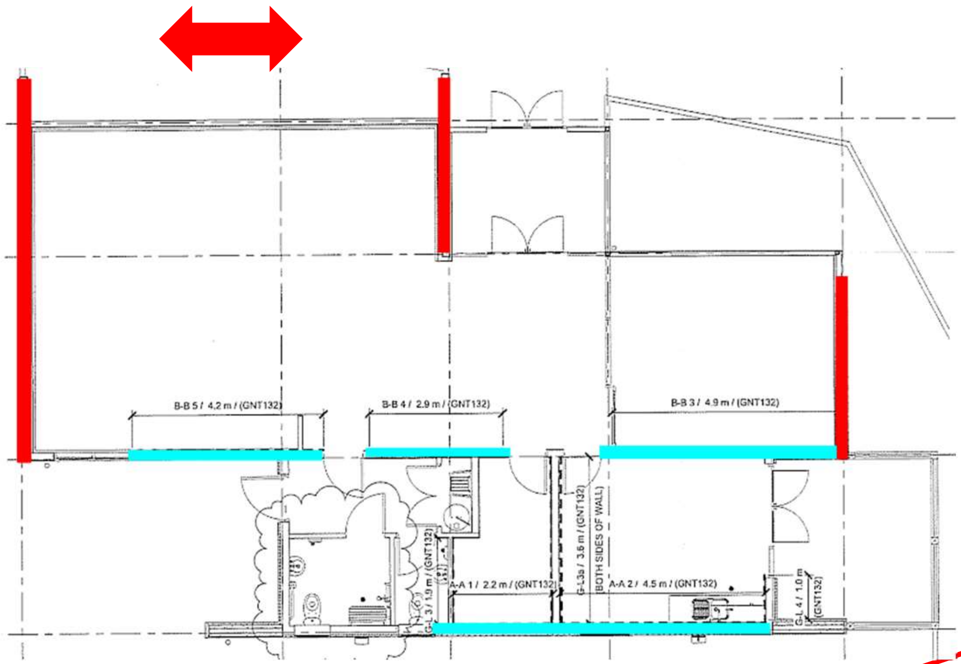
Tip 10 – Consider displacements

- We have in-plane precast concrete panels (red), and in-plane BL1-H bracing walls (yellow) acting in the same direction
- However the concrete walls will be very stiff & will have negligible deflection
- Roof plane bracing will try and tie the structure together

Differential Displacements means loads will try and be dragged to the in-plane concrete walls.

Likely lead to increased damage

Longitudinal Direction



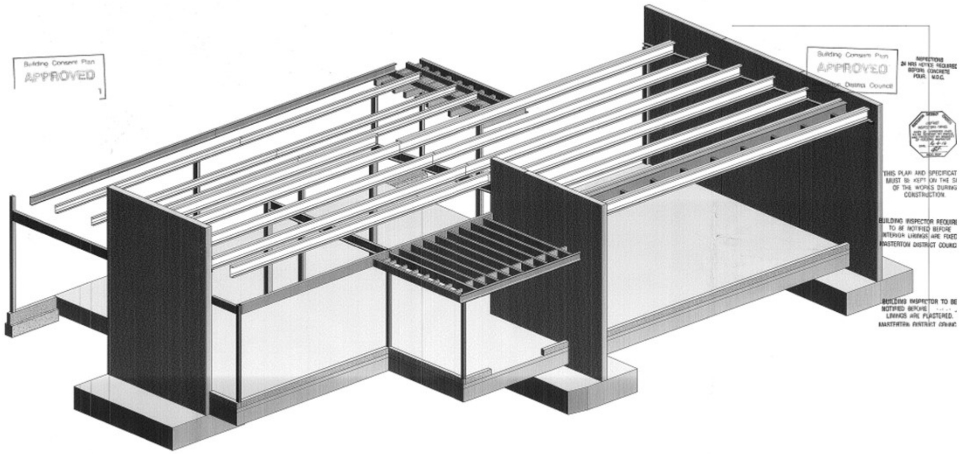
- We have out-of-plane precast concrete panels (red), and in-plane BL1-H bracing walls (blue) acting in the same direction

Again, the differential displacements means loads will try and be dragged to the in-plane concrete walls.

Likely lead to increased damage

Tip 10 – Consider displacements

What would have been some other options?

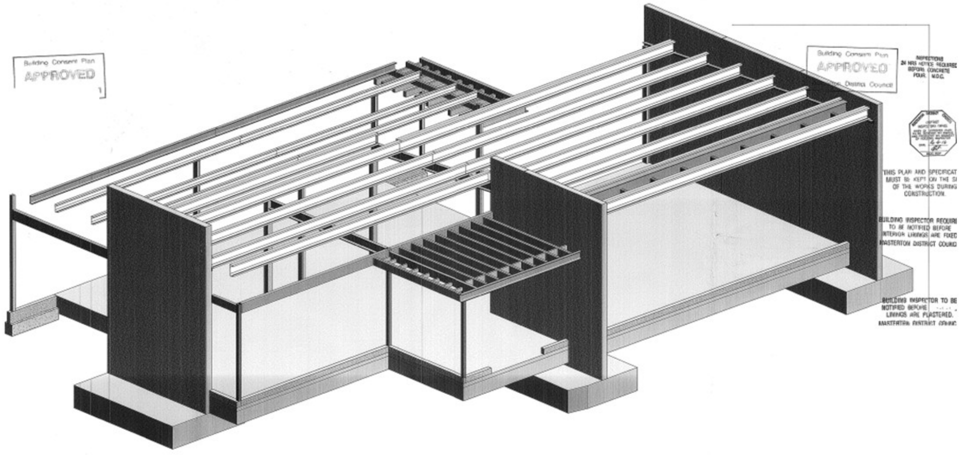


- Consider panels as cladding, primary system introduced (steel frames?) to support panels in out-of-plane direction

or

- Doubly reinforced as a minimum

Structural & Architectural

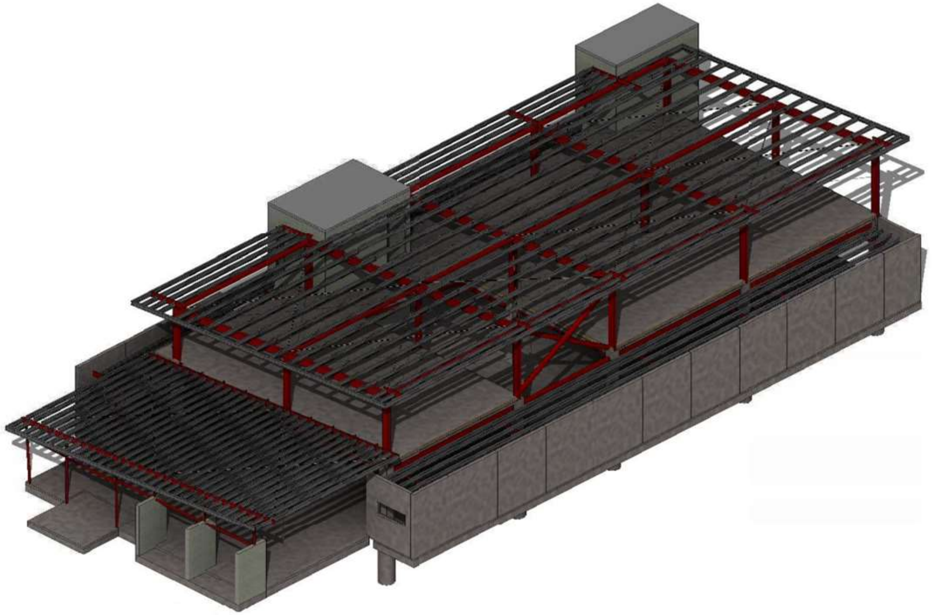


- The building is not on a boundary
- There is no functional reason for the precast panels (ie no fire rating needed!)
- Would the architect have been open to a light weight option?

- How could this have been kept simple?

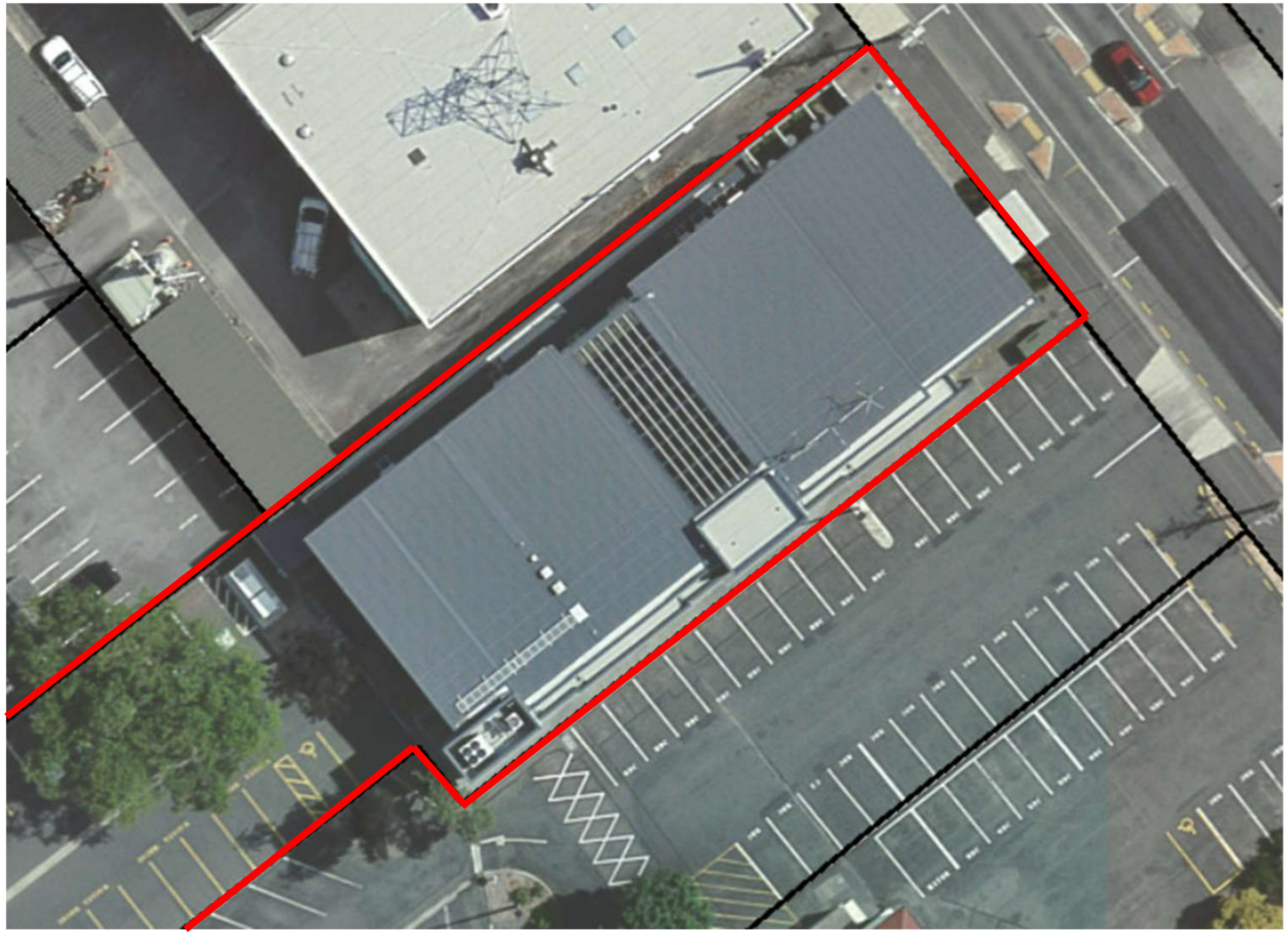
And last one….

Back to Building D



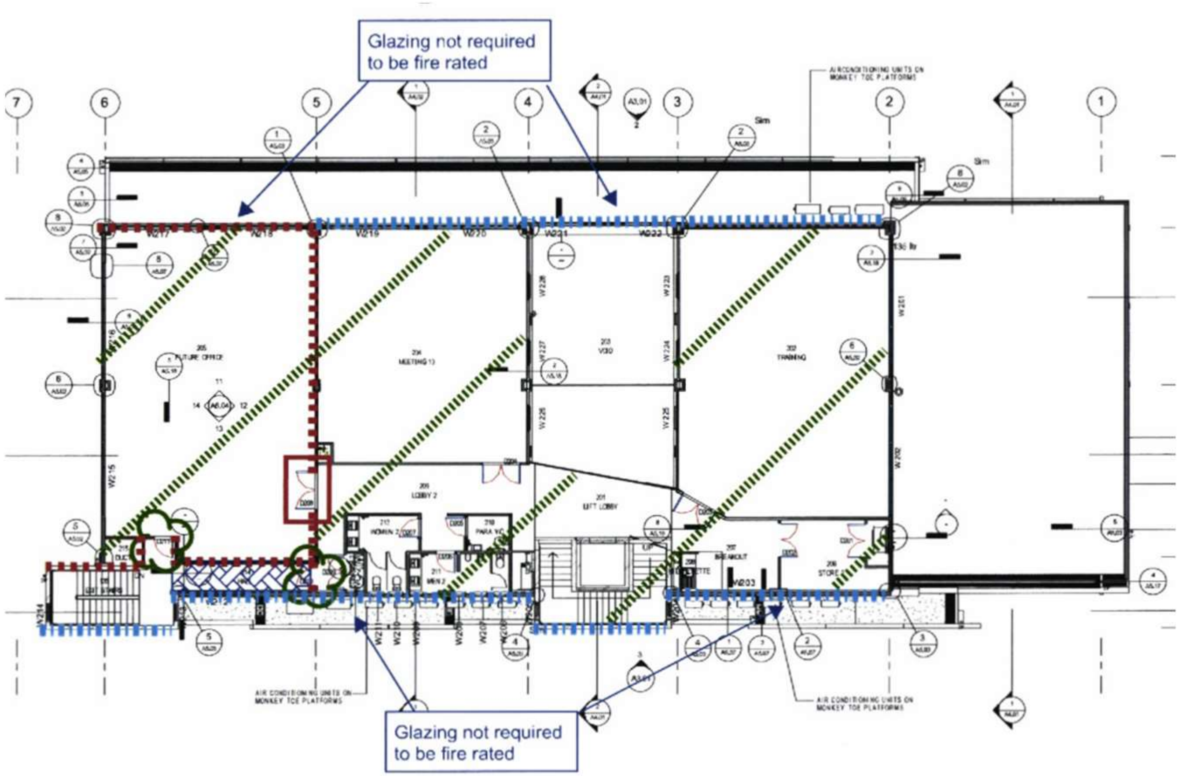
Consented in two stages

- Stage 1
  - Structural skeleton
- Stage 2
  - Architectural and fit-out



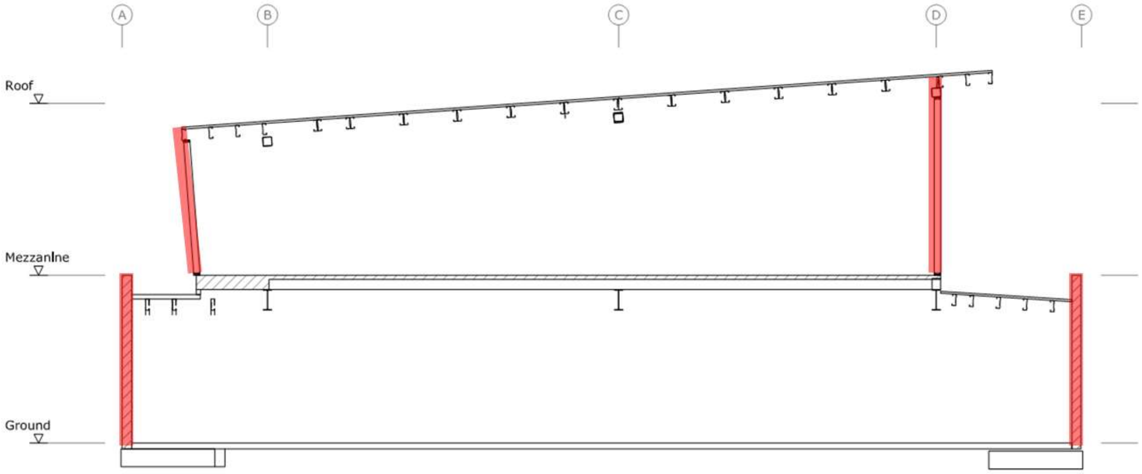
- Building is close to a boundary along both side walls
- Structure was designed, consent submitted before the fire report was provided to the engineer

Fire report



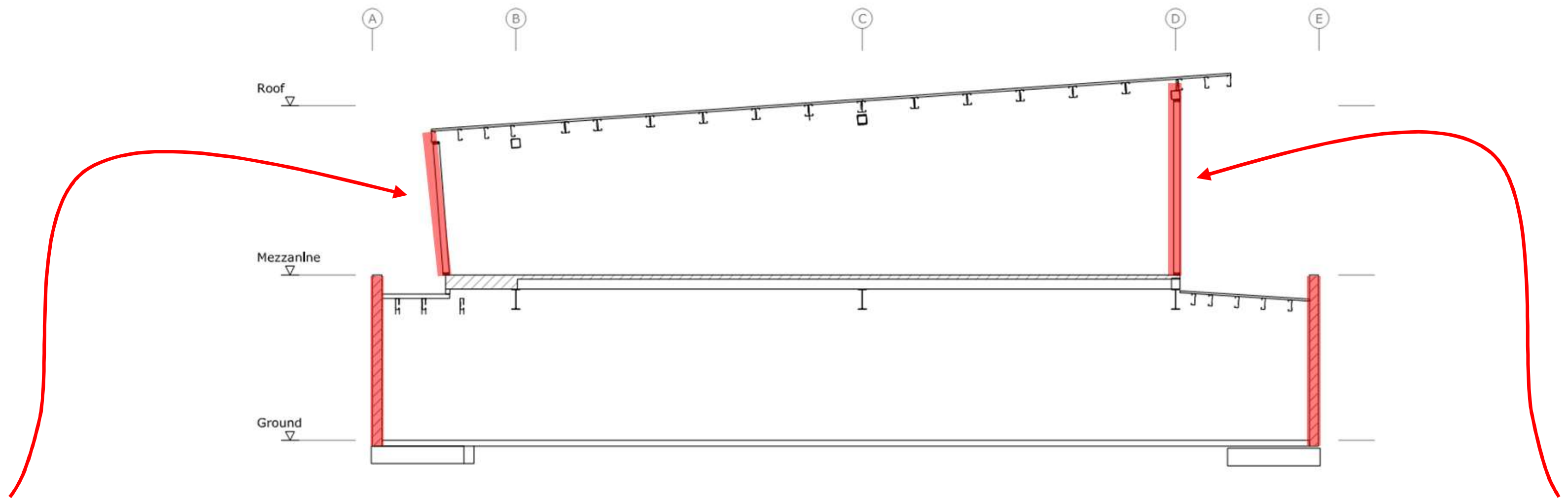
- Fire Engineer required that the side walls on both the ground floor and mezzanine required post-fire stability

What about the side walls?



- Ground floor has cantilever precast concrete panels – ok for required FRR
- What about the first floor?

What about the side walls?

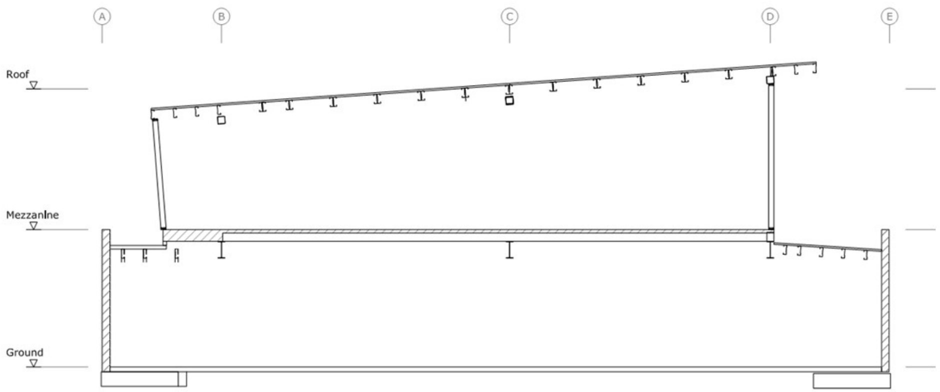


As talked about earlier – there was no engineered load path to support this wall out-of-plane

Elements that support the wall for post-fire stability require fire rating protection

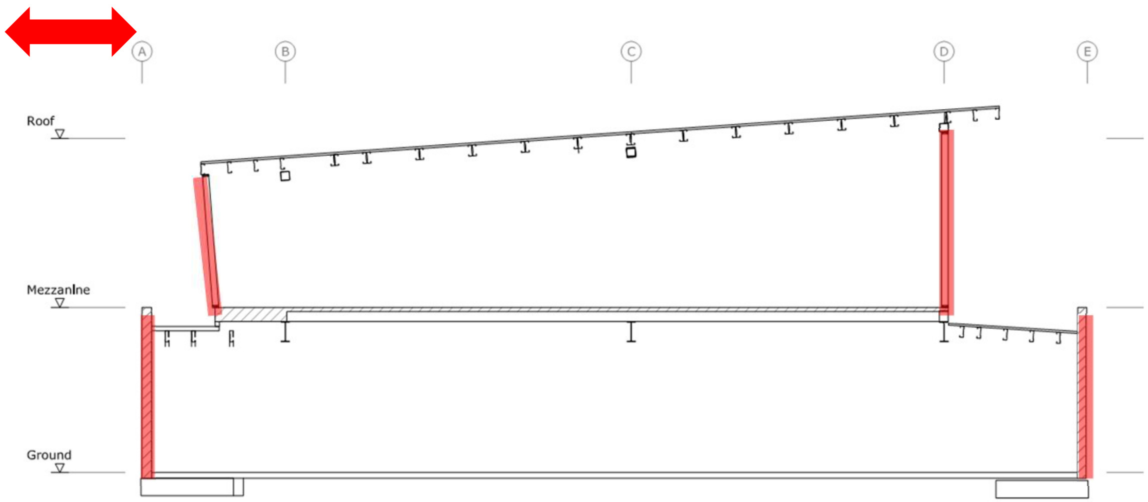
These walls span up to an SHS collector, which span between the portal frames

Post-fire protected load path?



- The portal frame columns could have been designed to cantilever & then been protected with a 120 minute system
- But the columns and base connections were not designed for this

Post-Fire load path?



Lesson?

- Make sure you have all the information you need to design a structure
- Location in NZ – eq, snow & wind loads
- Location on a site – fire requirements – get the fire report!

Designing for Uncertainty?

National Seismic Hazard Model - the revision programme

Te Tauira Matapae Pūmate Rū i Aotearoa
NSHM The New Zealand National Seismic Hazard Model
A GNS Science Led Research Programme

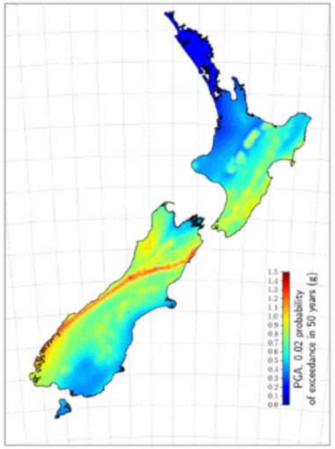
- What is the National Seismic Hazard Model?
- Who uses the NSHM?
- Revising the model
- Who is involved in revising the NSHM?
- Find out more

What is the National Seismic Hazard Model?
The New Zealand National Seismic Hazard Model (NSHM) is a model that calculates the likelihood and strength of earthquake shaking occurring in different parts of New Zealand.

In a nutshell – The NSHM is a collection of many different models that are combined together to estimate future earthquake shaking in New Zealand. These models represent the broad range (and uncertainty) of our knowledge about how earthquakes occur – and also about how earthquakes cause the surface of the earth to shake. We use a collection of different models because each model allows us to include a different scientific possibility. These results help understand the expected shaking in, for example, the next 10, 50 or 100 years.

The model incorporates scientific understanding of earthquakes acquired from diverse research fields ranging from paleoseismology, geodesy, and geophysics, through to engineering seismology.

The NSHM consists of two primary components:



This map shows the 2% probability of exceedance of peak ground acceleration (PGA) from earthquake shaking in any 50-year time window. These results were produced for NZ

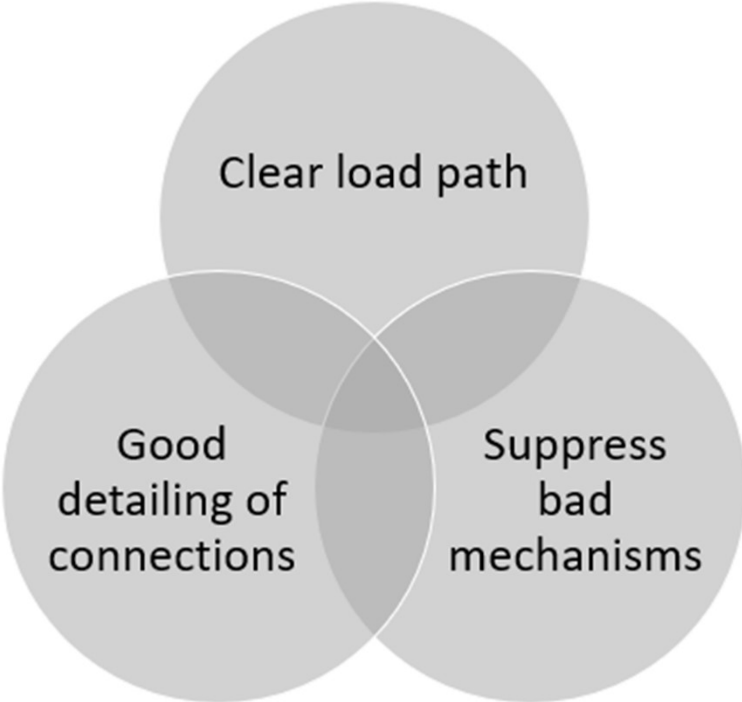
Designing for Uncertainty?

For low rise, most designs do not require capacity design

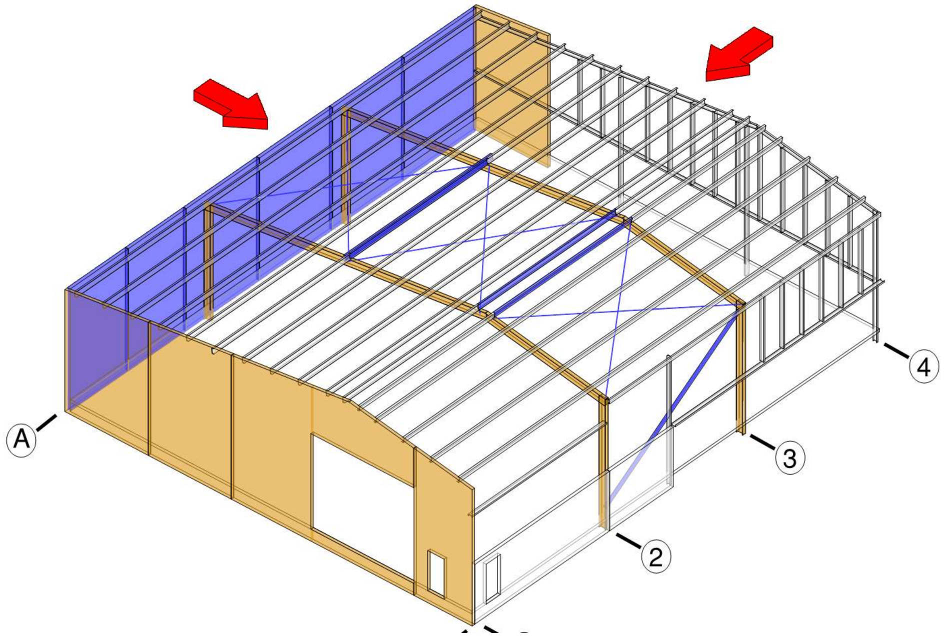
But without the safety net of capacity design, the principles of robustness need to be front and centre

Follow some simple principles

Shouldn't necessarily add cost to the build

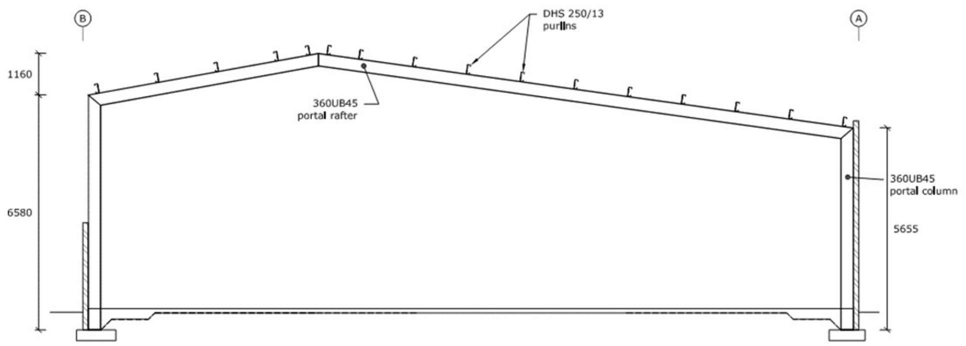


What is robustness for a common Warehouse Type Structures



- Steel frames in one direction
- Tension only bracing in the other

Portal Frame system



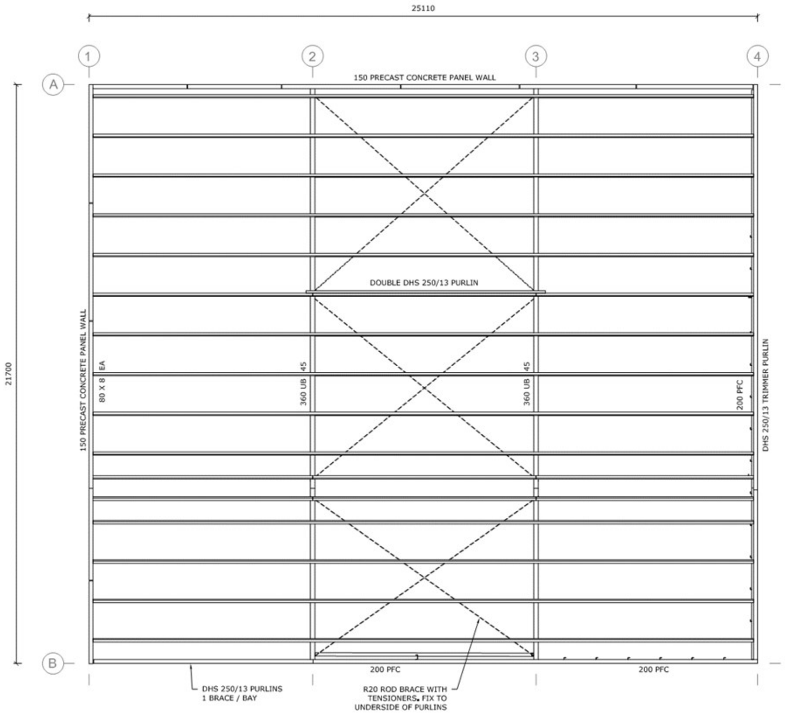
Max level of shaking design

- Lateral buckling of members possible
- Connections designed for upper limits only
- Category 4 member can be used

Designing for Robustness

- Well restrained members to prevent buckling
- Connections designed for overstrength capacity of members
- Category 1 or 2 member capable of yielding

Longitudinal Tension bracing System



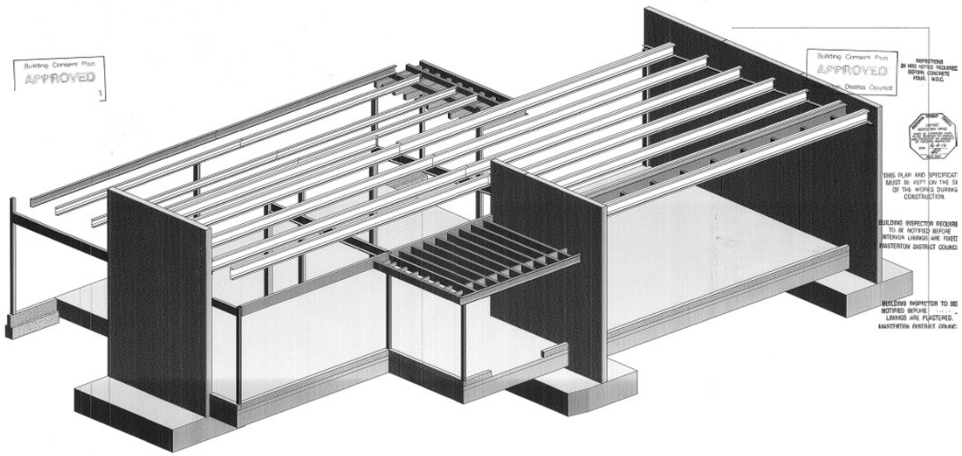
☒ **Max level of shaking design**

- DHS purlins as struts
- Connections designed for upper limits only
- Cast components on tension braces

☑ **Designing for Robustness**

- Structural steel struts, sized to prevent buckling
- Connections designed for overstrength capacity of members
- Seismic rated system, with cleats designed for overstrength

Low rise cantilever panel building



☒ **Max level of shaking design**

- Cantilever face loading designed for max shaking
- Some post yield capacity, but after this?

☑ **Designing for Robustness**

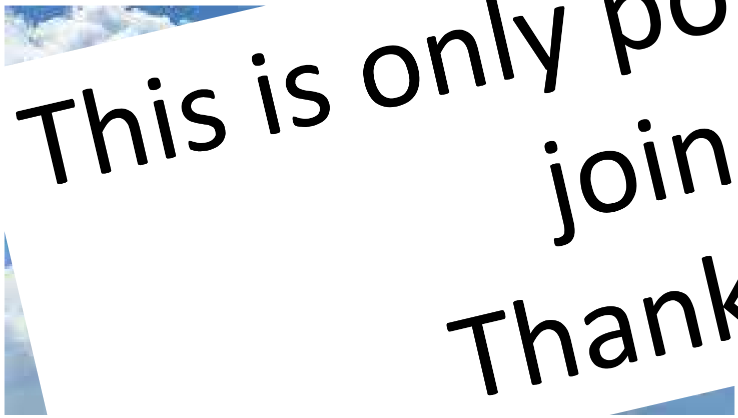
- Consider panels as cladding, primary system introduced to support panels in out-of-plane direction

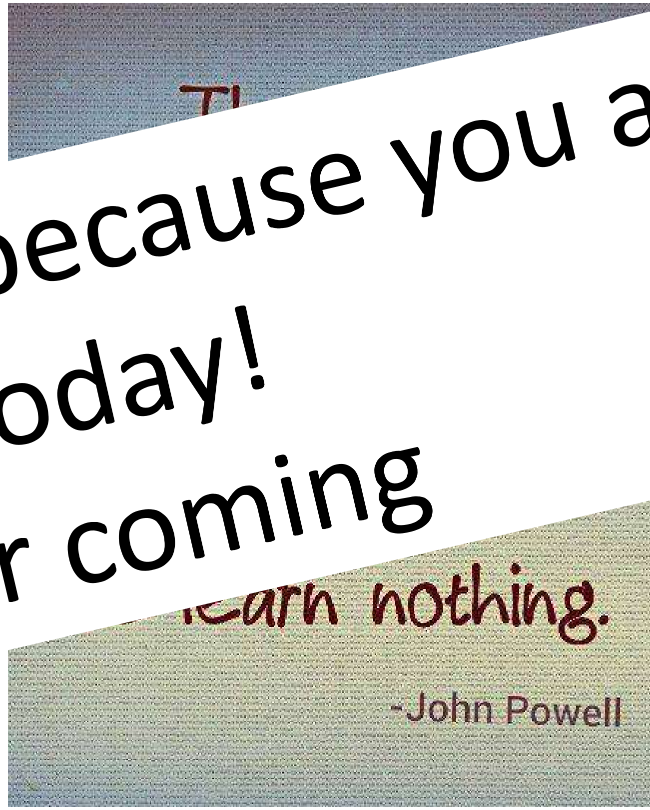
or

- Design for rocking of footings – but this forces us outside of B1/VM1
- Doubly reinforced as a minimum

A reminder … we are here to turn towards a positive

How do we turn quite a few negatives into a positive…

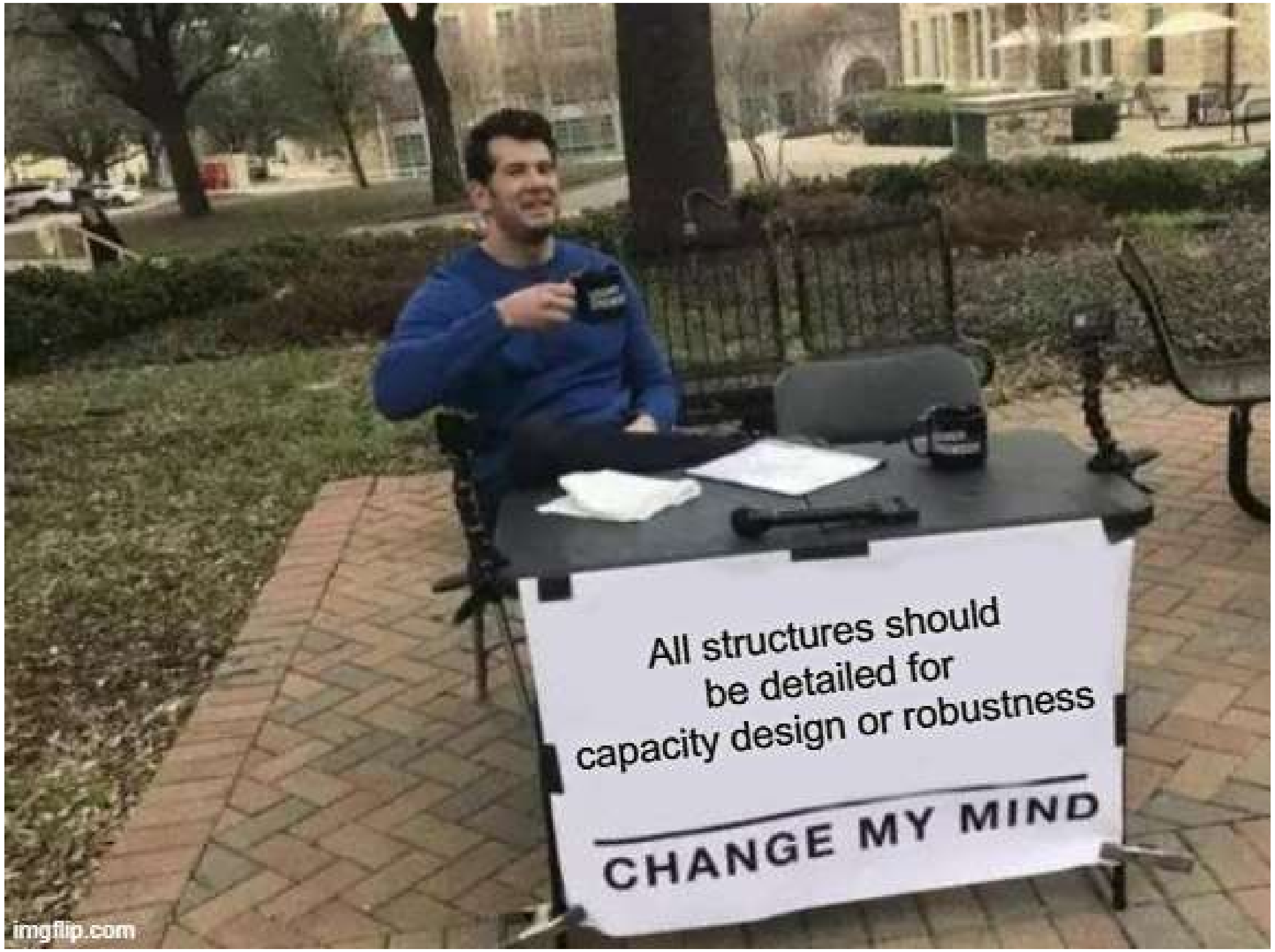




This is only possible because you all joined us today!
Thankyou for coming

Ten tips for the ~~better~~ design of robust low rise structures

1	Make sure your design matches your model	☑
2	Make sure you have a load path	☑
3	Node all of your connections	☑
4	Connections are critical	☑
5	If you 'adopt a ductility' make sure you can actually get ductility	☑
6	Do check ins – ie base shear total is right?	☑
7	Sometimes you need to say no	☑
8	Co-ordinate structural with architectural	☑
9	Keep durability in mind	☑
10	Consider Displacements	☑


All structures should
be detailed for
capacity design or robustness
CHANGE MY MIND
imgflip.com

Thankyou

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