CTV Building Collapse Technical Investigation

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for
Canterbury Earthquakes Royal Commission

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Outline of Presentation

- Our Task
- Building features
- Investigation approach
- How did the building collapse?
- Why did the building collapse?

Our Task

- Investigation for DBH was focussed on:
  - Reasons for collapse of CTV Building
  - Implications for standards and practices

- The scope of the investigation was limited to identifying technical reasons for the collapse.

- Roles:
  - Dr Clark Hyland
    - Joint Author
    - Expert Panel Member
  - Mr Ashley Smith
    - Joint Author
    - Coordination of Non-Linear Seismic Analysis

Scientific Method

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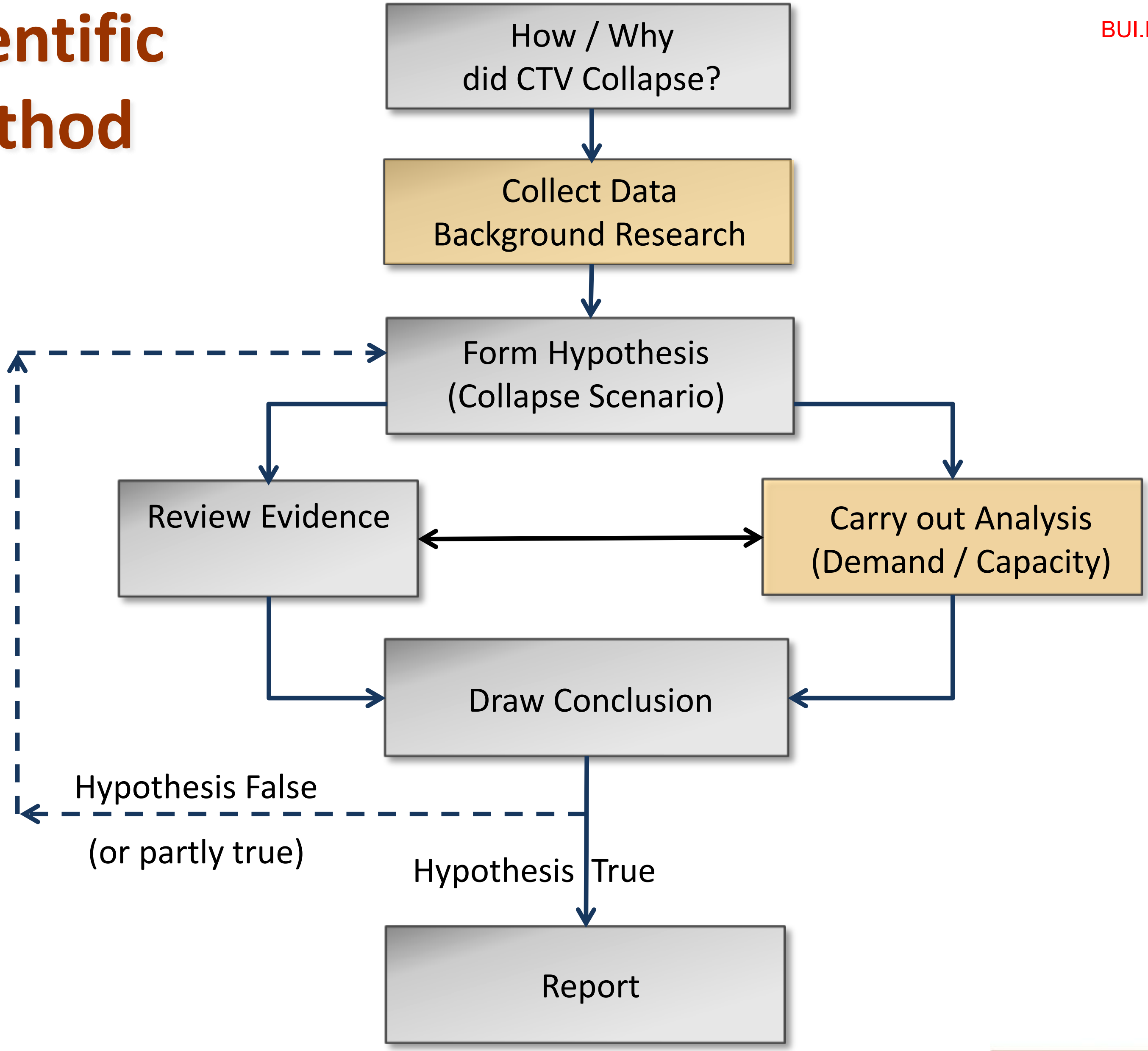

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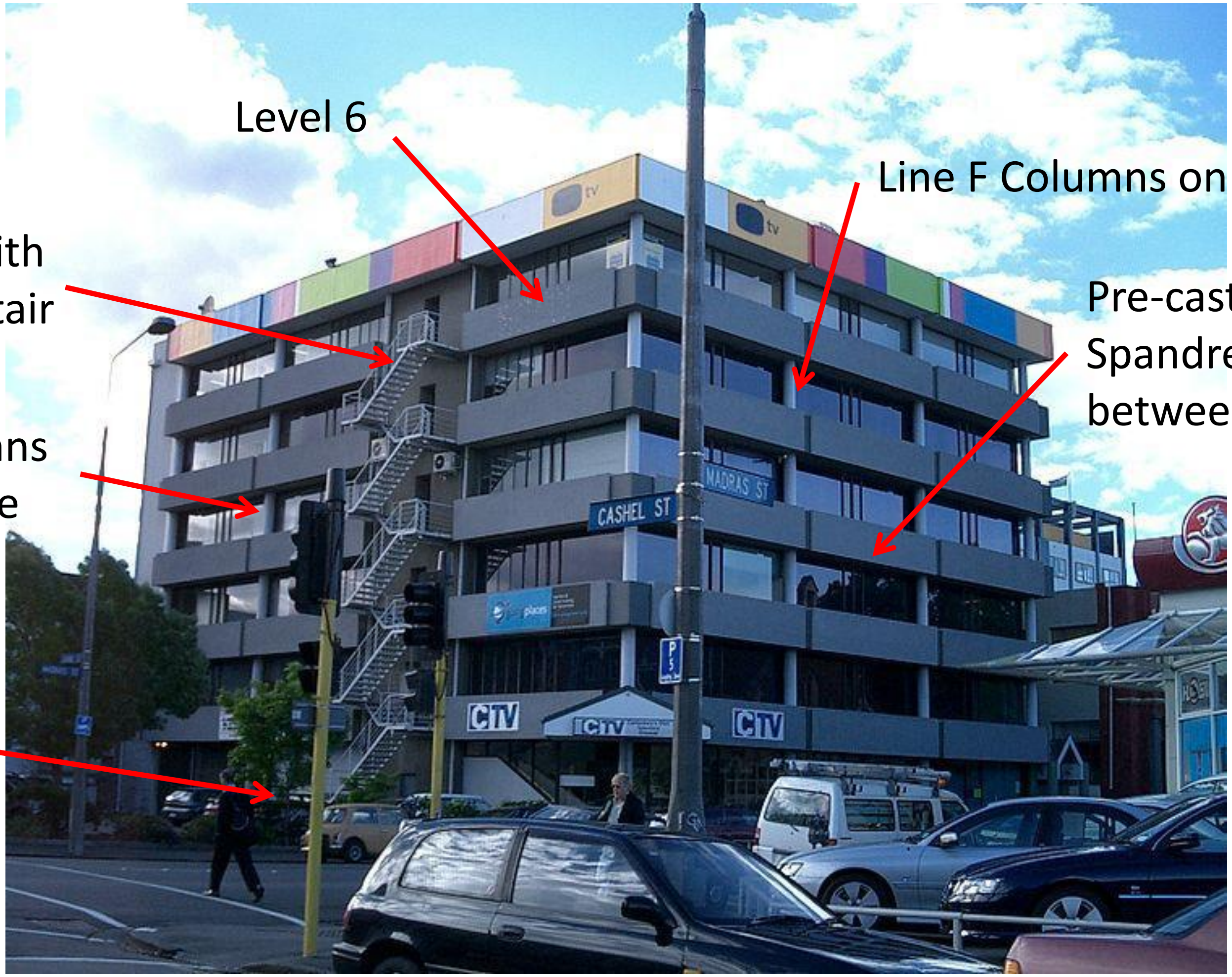
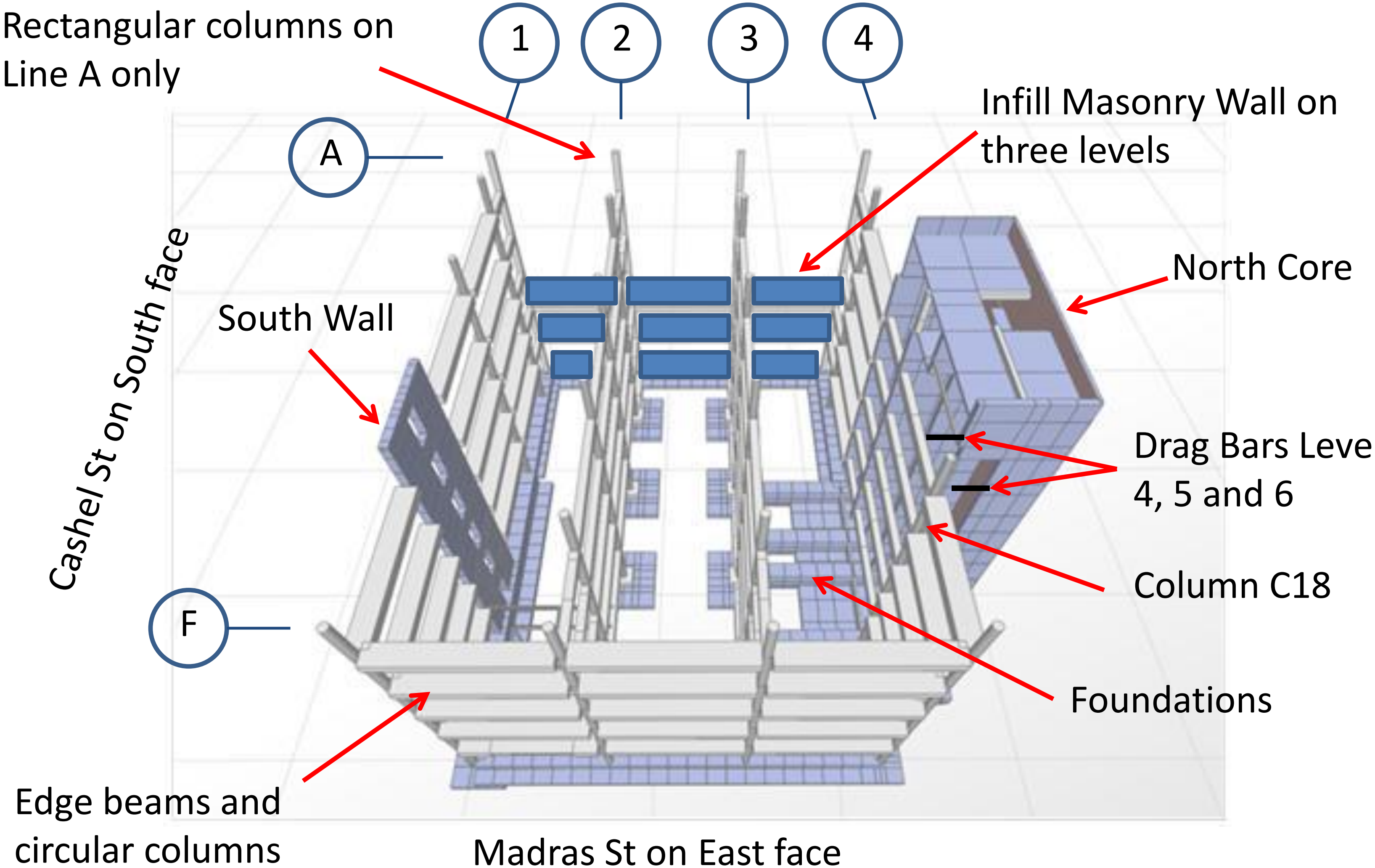


CTV Building in 2004 (viewed from southeast)

Building Features



Birds-eye view from east without slabs

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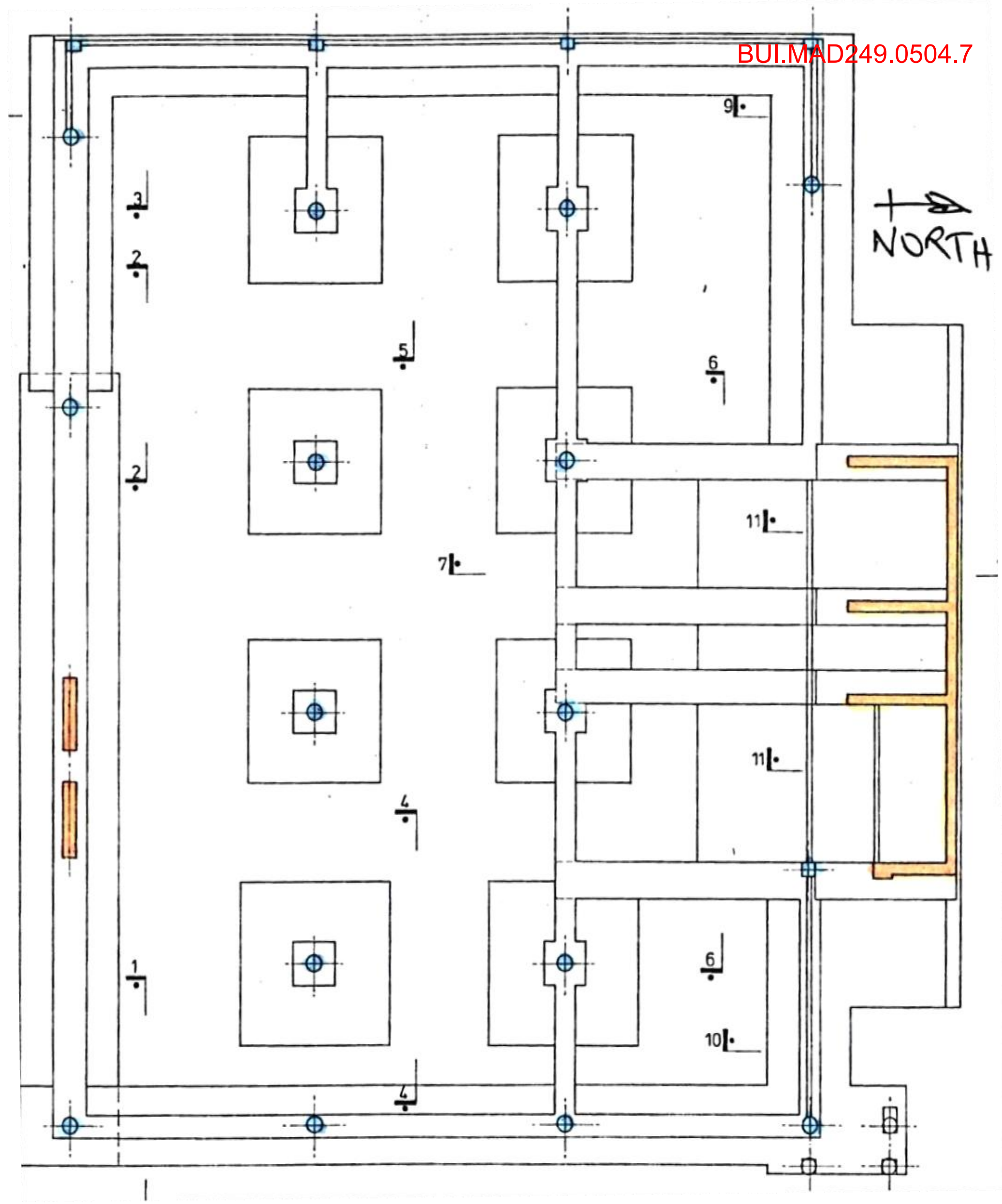
Foundation Plan



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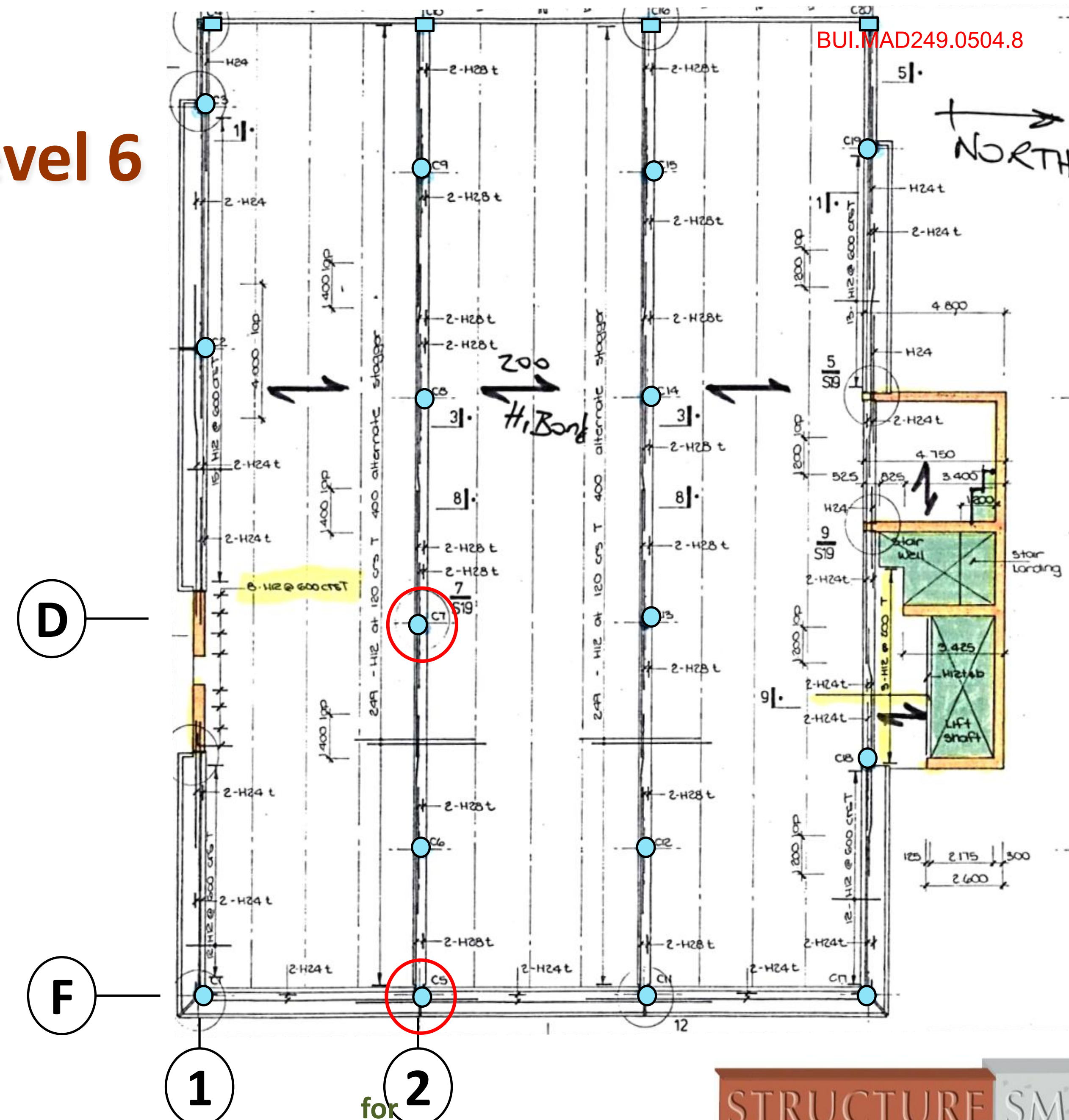
Floor Plan
Level 2 to Level 6



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North Core Wall with Column attached

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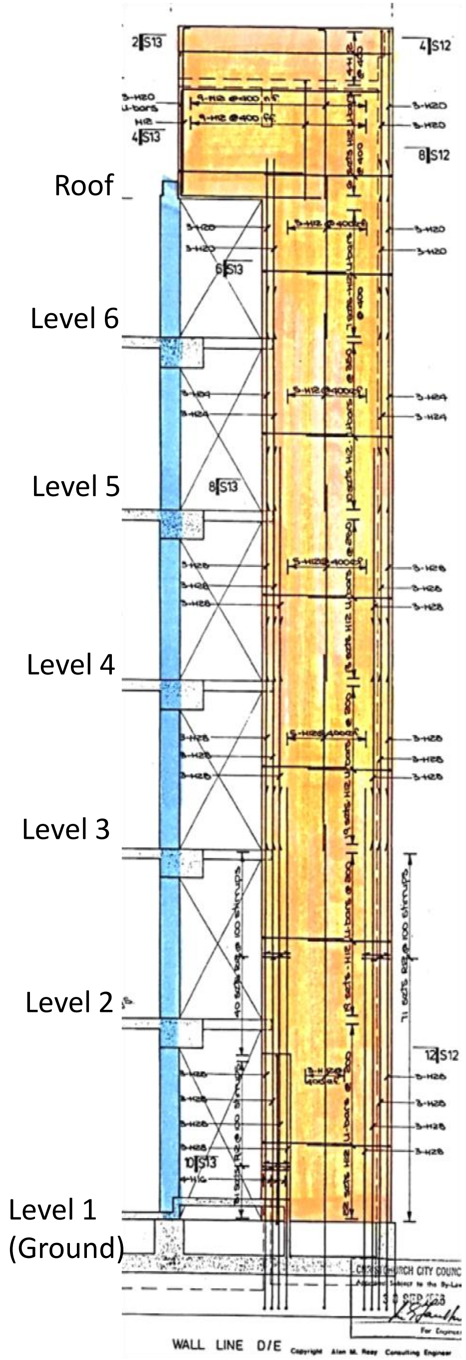

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South Wall Elevation

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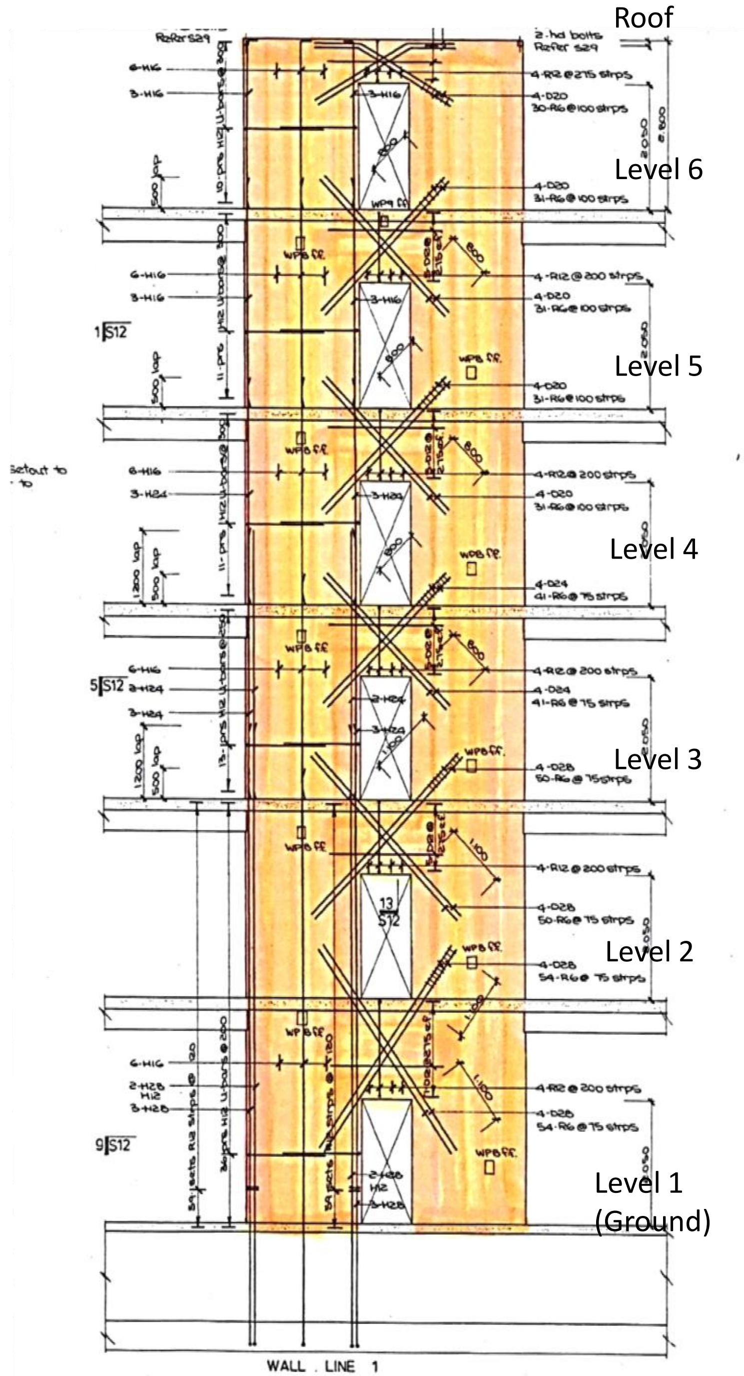
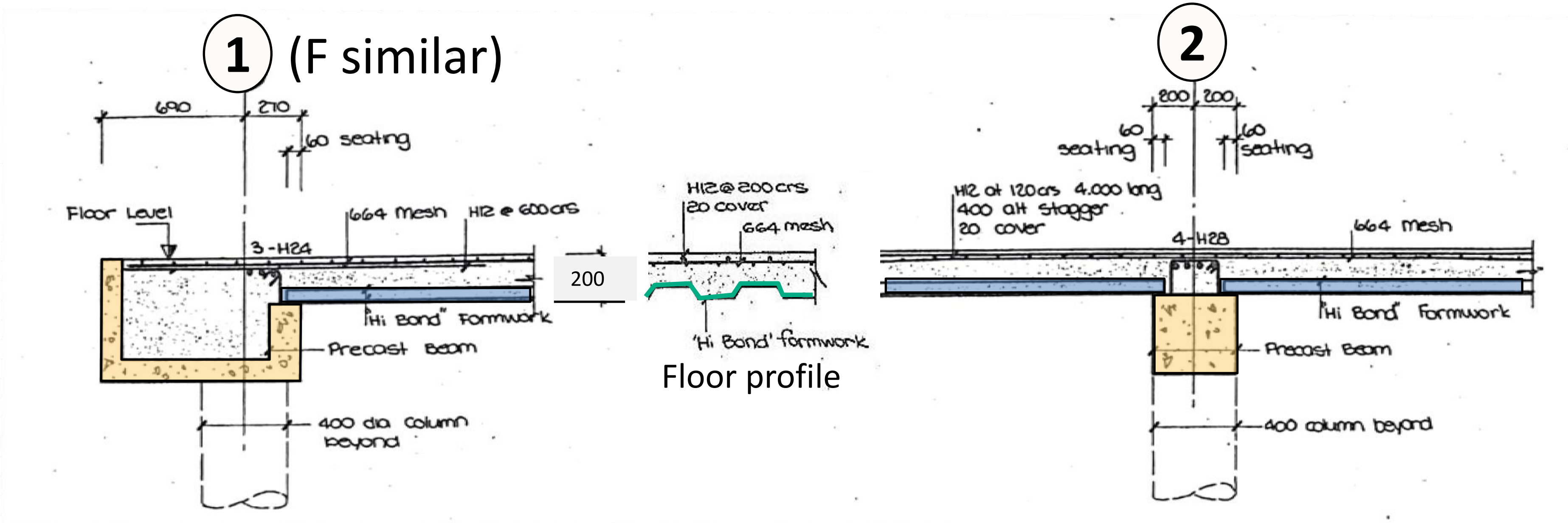
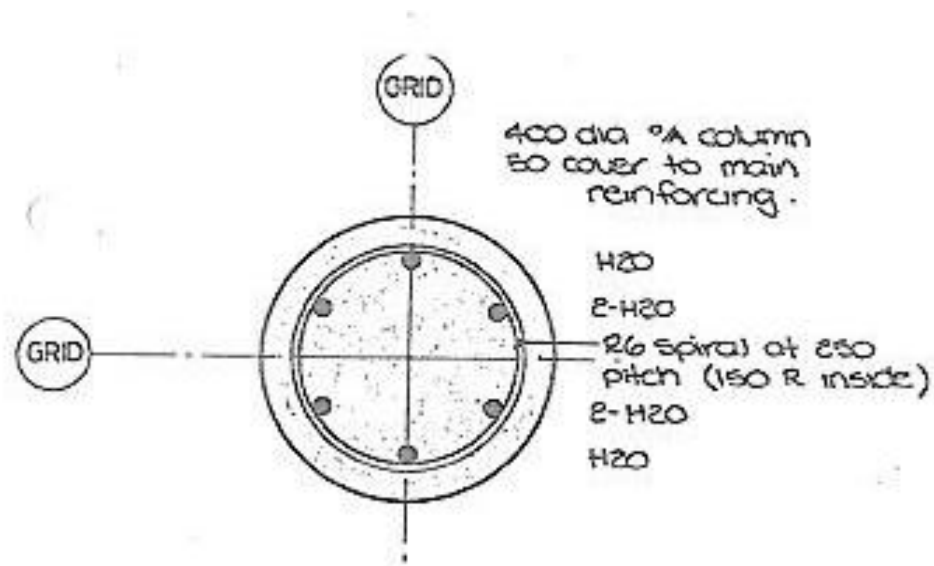
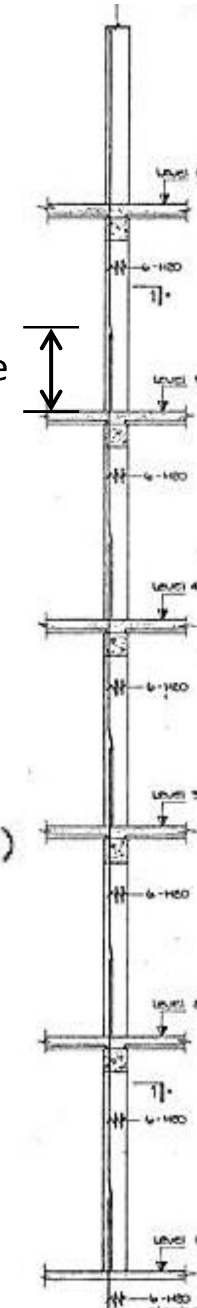


Beam and Floor Sections

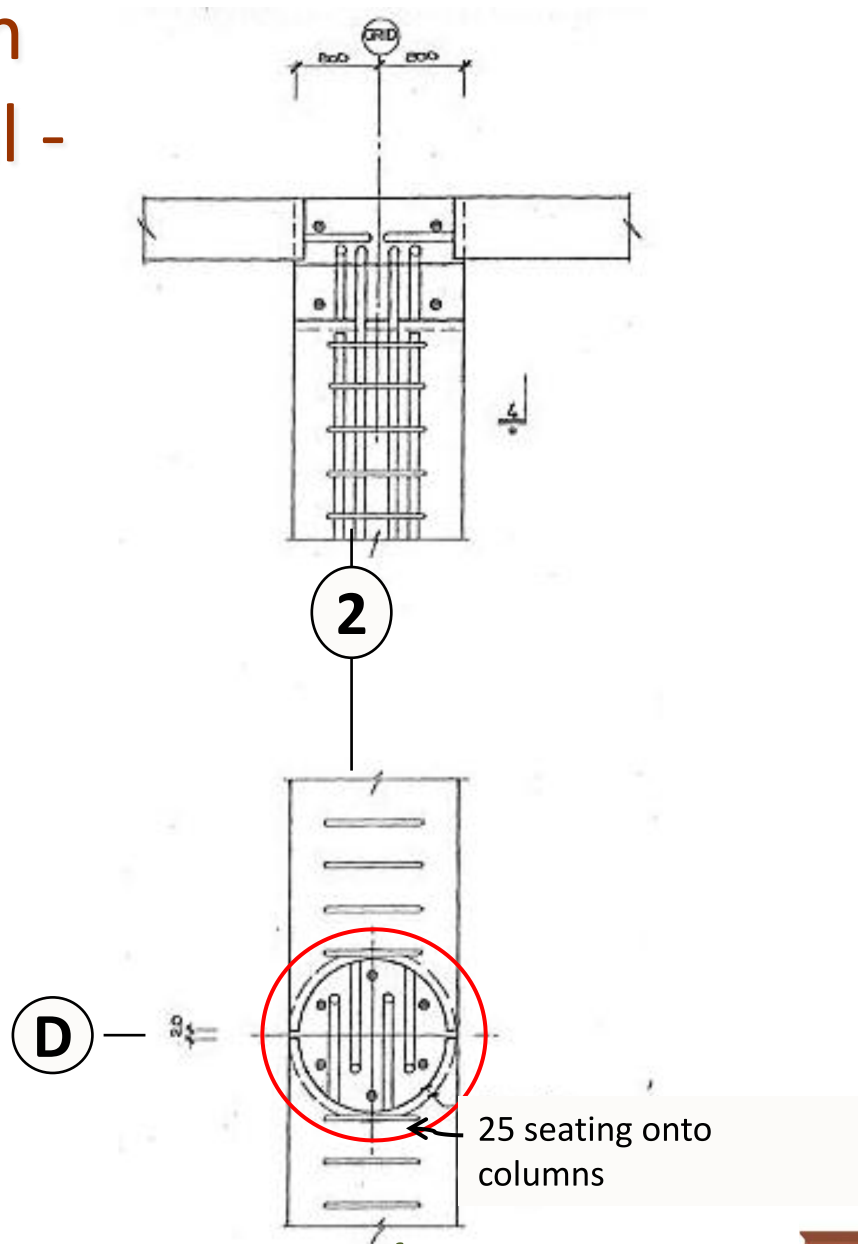


Column
Elevation & Section

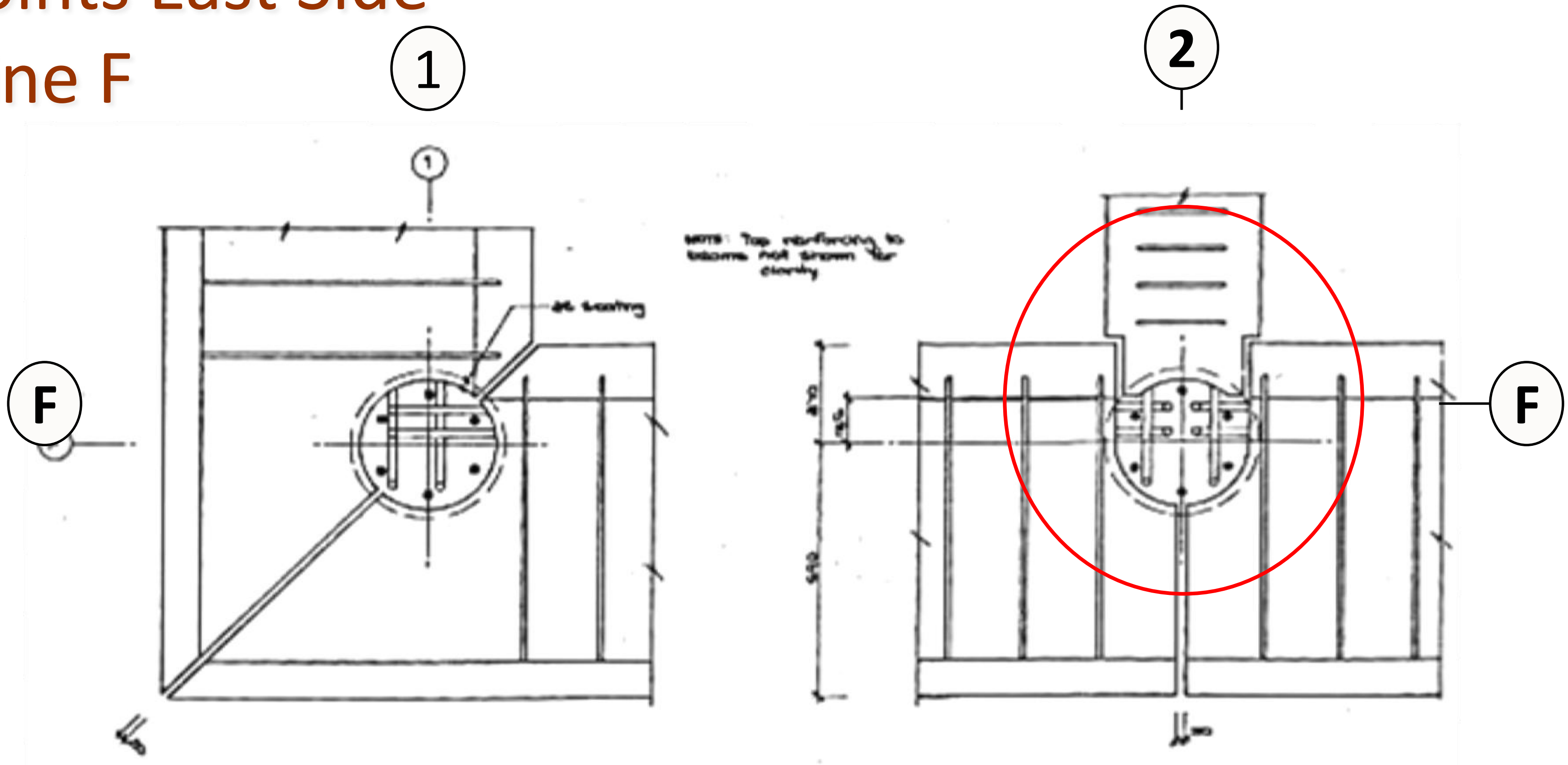




Beam-Column Joints Internal - Line 2



Beam-Column Joints East Side – Line F



Masonry Infill West Side

Three masonry panels in each bay between columns

Elevation (1 of 3 bays)

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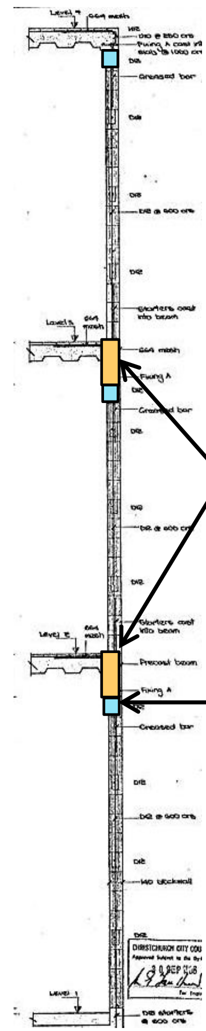


Level 4

Level 3

Beams at levels 2 & 3 only

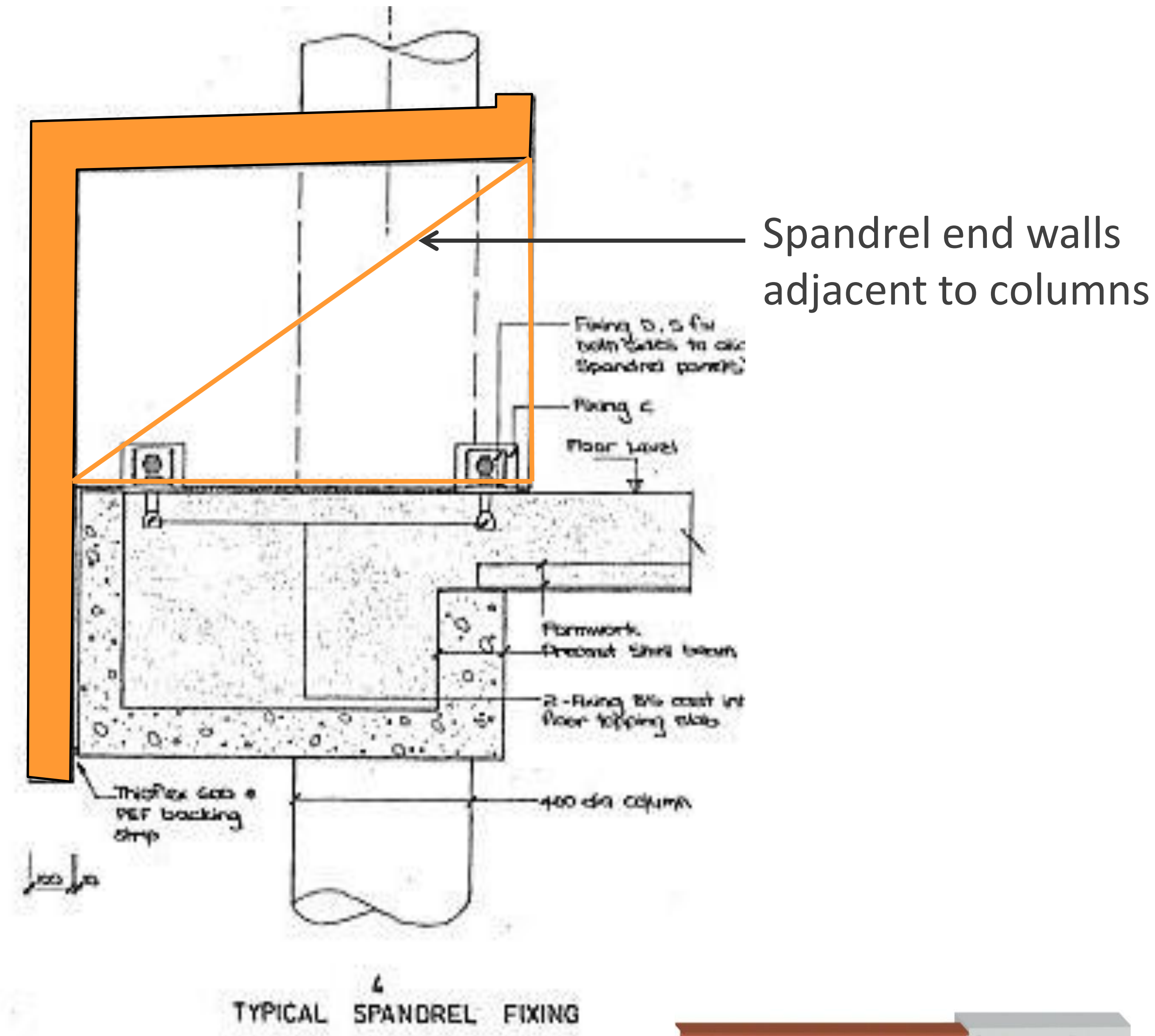
Level 2

Top block possibly un-grouted at each level (vertical dowel Connection)

Level 1 (Ground)

Section

Precast Spandrels



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Investigation Approach

- Collapsed Condition
- Witness Interviews
- Site Examination
- Materials Testing
- Structural Analysis
- Compliance Checks
- Collapse Scenario Evaluation

CTV Building after the Collapse

- View from west
  - Fire just started near north end
  - Level 4 to 6 cladding pushed north
    - Little debris otherwise on this side
  - Diagonal cracking to masonry infill
  - No liquefaction

North Core

Level 4, 5 and 6 Cladding

Level 2 masonry with diagonal cracking

No liquefaction

View from Les Mills from west Immediately after the collapse

CTV Building after the Collapse

- View from South
  - Prior to fire starting
  - Level 5 slab hanging from North Core
  - Level 6 slab supported by drag bars
  - No debris south of the building
    - Cars parked in front of South Wall undamaged

View from Cashel Street from south Immediately after the collapse

CTV Building after the Collapse

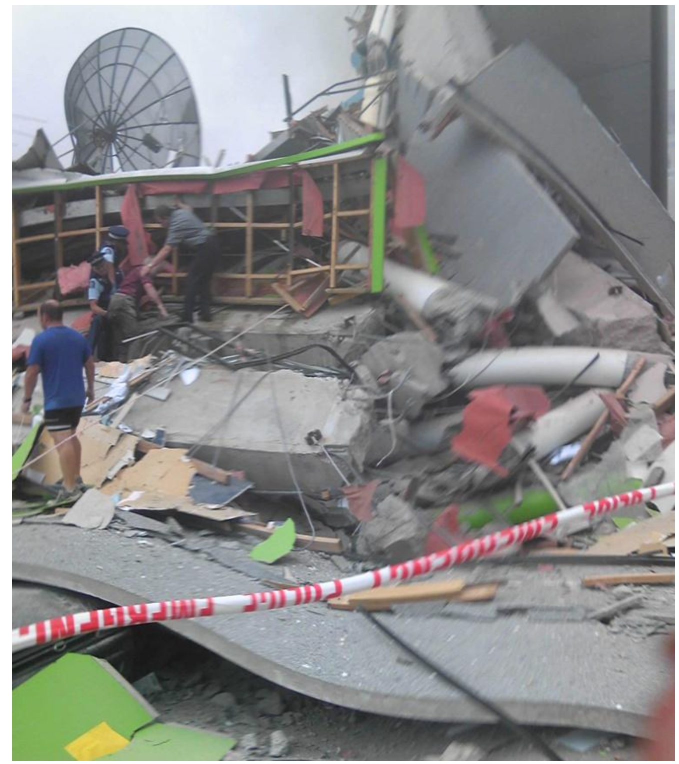
- View from southeast
  - Smoke from fire in background
  - No liquefaction
  - Slight eastward throw
    - Cars in Madras Street crushed by edges beams and Spandrel panels
  - Columns fractured

View from southeast Immediately after the collapse

CTV Building after the Collapse

- View from East Lines 2 to 4
  - Smoke from fire in background
  - No liquefaction
  - Cars in Madras street crushed
  - Columns fractured

View from Blackwells from east Line 2 to 4 shortly after the collapse at 1:21 pm



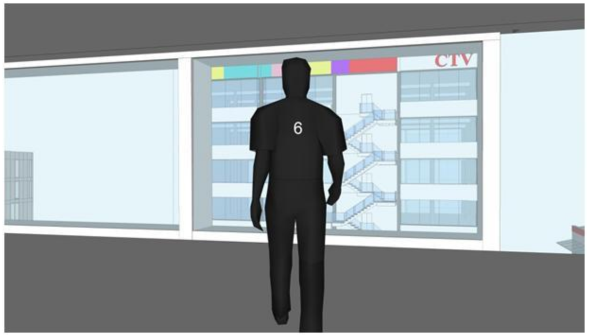
View from east at Line 4 North Core Immediately after the collapse at 1:00 pm

Witness Interviews

- 18 Eye-witnesses Interviewed
  - 6 were in the building during the collapse
    - Levels 1, 4 and 6
  - Views of the collapse from East, South and West
    - 3D perspectives
    - Fly-through video

Witness Observations

- Witness 14: Gutteridge
  - Twisting, bursting, columns breaking
- Witness 8 and 15: Hawker and Spencer
  - Upper portion came down as a unit
- Witness 6: May
  - Top leaned to east then collapsed straight down
- Witness at Level 4 Lifts: Godkin and Horsley
  - Floor started collapsing and undulating near South Wall before sharp west-east lurch



Site Examination

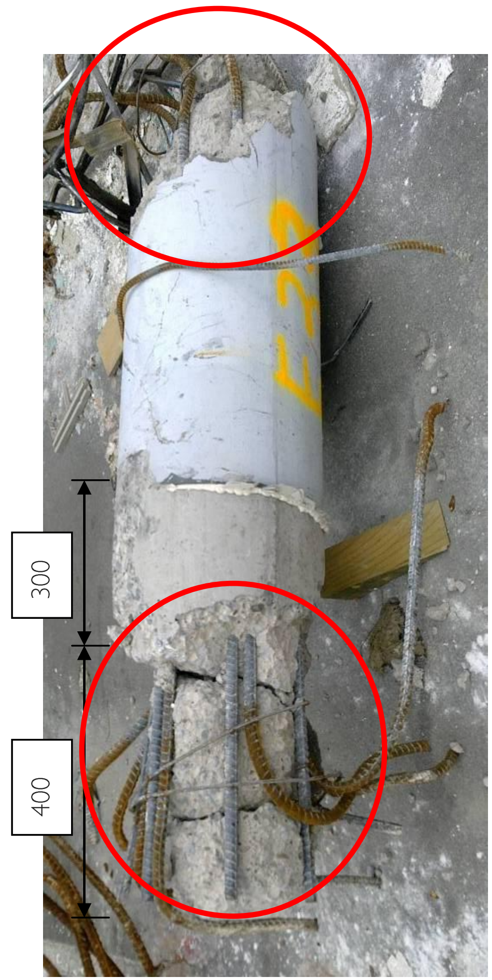
- Salvaged Structural components
- North Core examination
- Levels Survey
- Foundations excavation and examination
- Column extraction and testing at Burwood

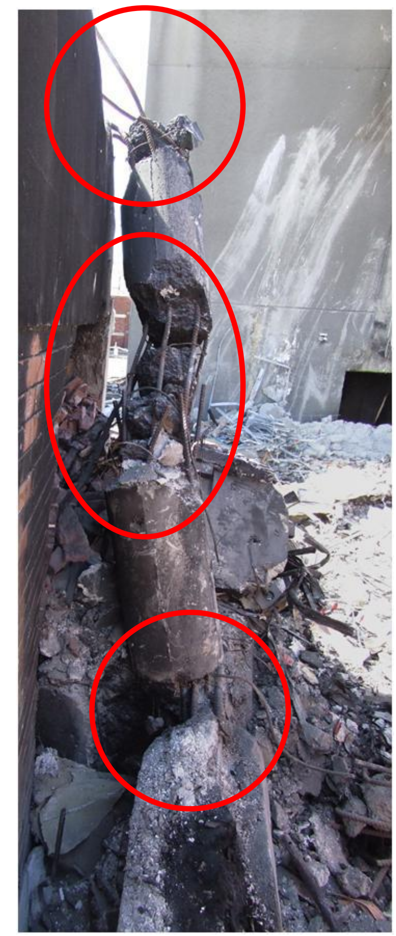
Salvaged Structural Components

- South Wall Level 5 to 6
  - Soft concrete at door head
    - Non-compliant
  - Smooth construction joints
    - Non-compliant

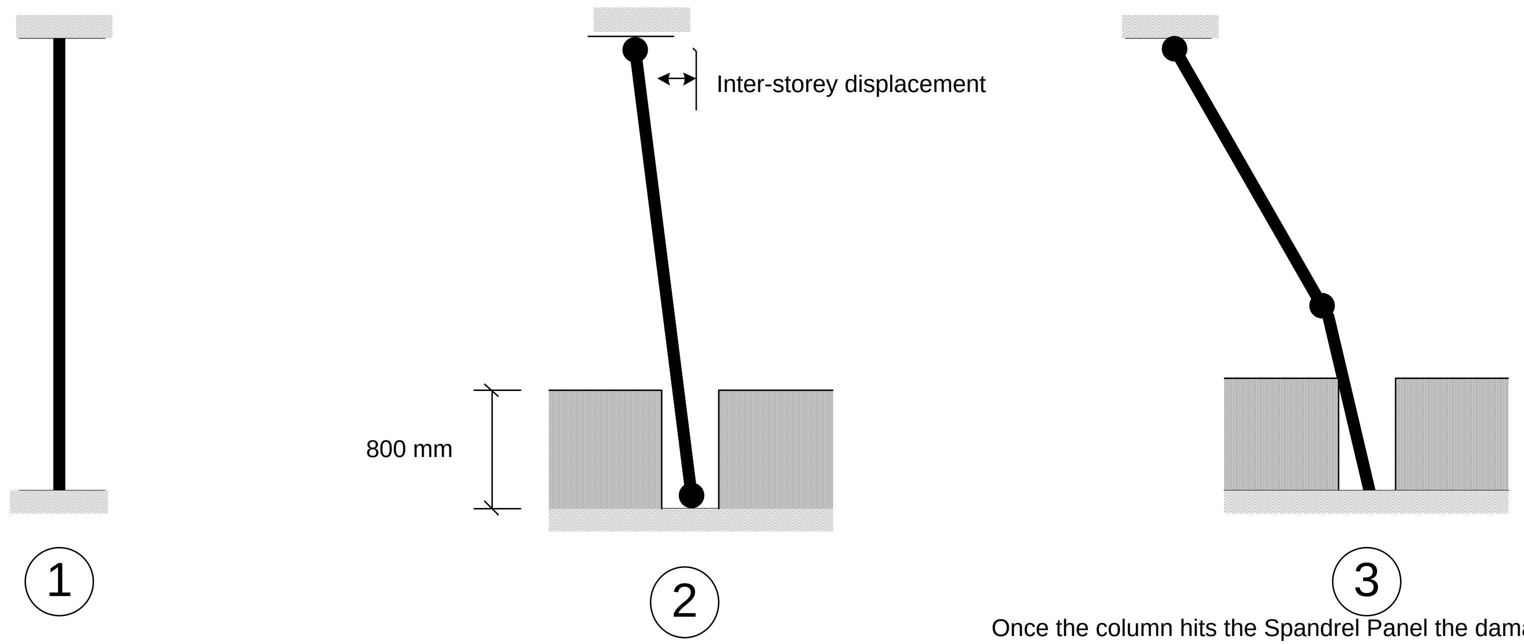
Salvaged Structural Components

- Column Hinging
  - Base
  - Mid-height
    - Vertical reinforcing steel termination zone
    - Spandrel Panel contact?
  - Head





Salvaged Structural Components



Damage developing at the head and base of the column as the inter-storey drifts increase.

Once the column hits the Spandrel Panel the damage at the column head increases markedly to loss of load carrying capacity. Damage may also possibly develop near contact with the Spandrel Panel. Distress may develop at compression splice ends 1200 mm above base due to lack of confining steel

- Possible East and North face column damage sequence

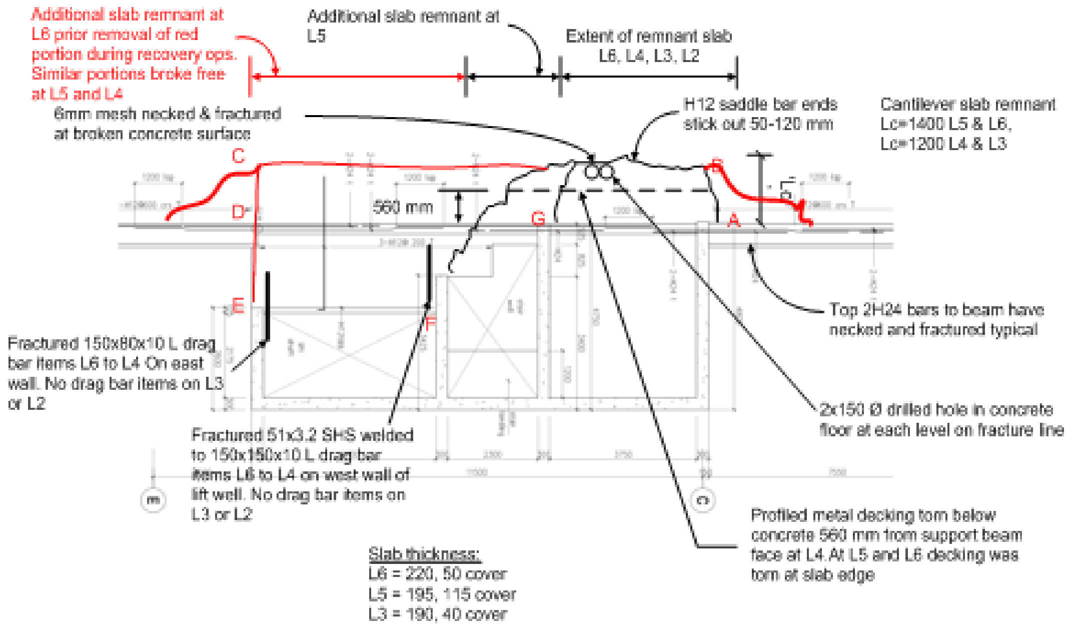
South Wall Condition

- South Wall
  - Heavy compression spalling at east end
  - Fan like flexural cracking
    - Cantilever behaviour
    - Masonry infill of door

North Core Condition

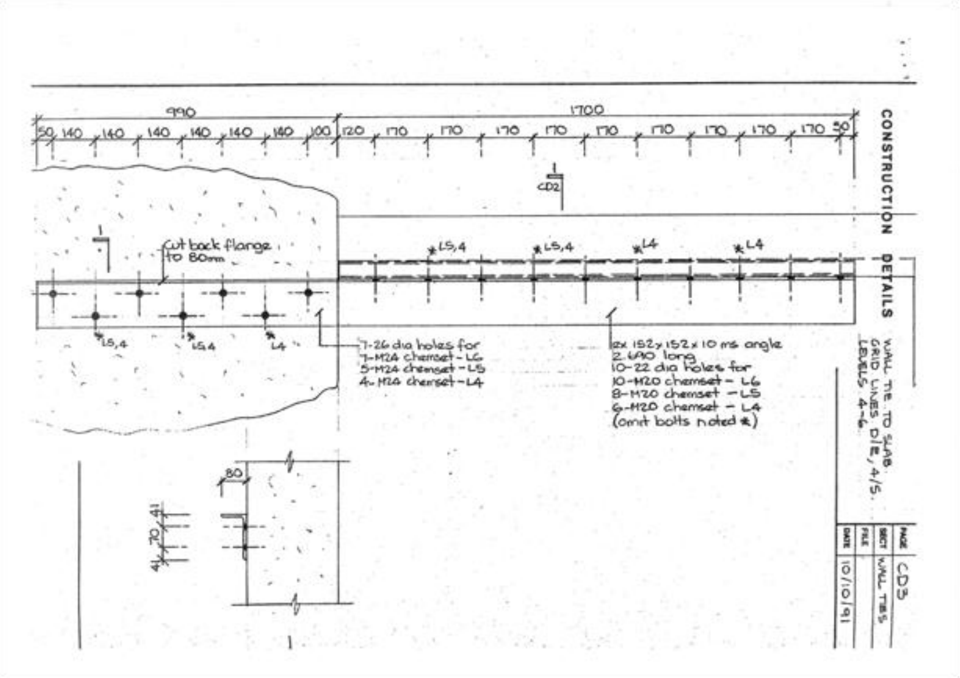
- Hairline cracking to North Core
- Fire effects on inner faces

North Core Examination



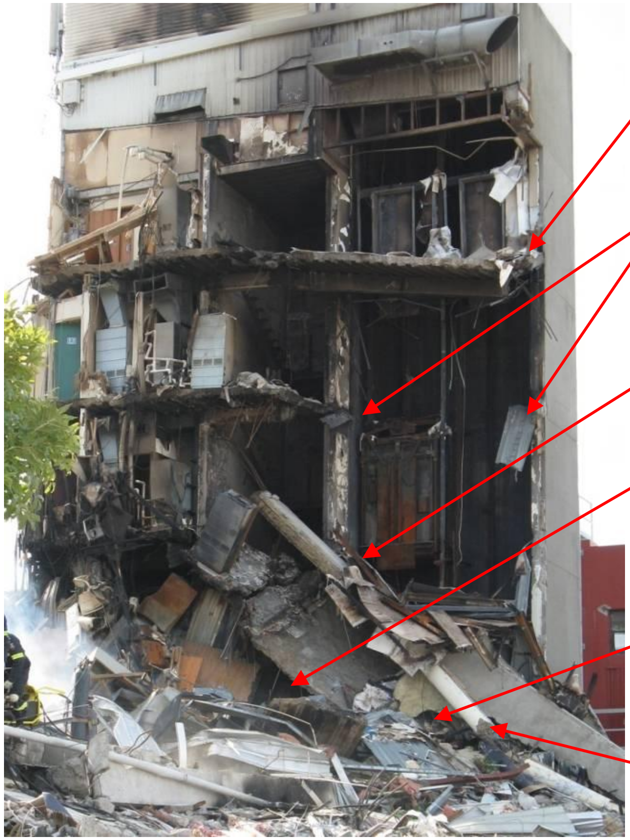
- Drag Bars at Levels 4 to 6
- Slab broken away at end of saddle bars
  - 1200 mm south of beam
- No or little slab reinforcing connection to some walls

North Core Examination



- Drag Bars connection to some walls

North Core Examination



L6 Drag Bar still attached to and holding up slab

L5 Drag Bars

L5 Line 4 precast concrete beam after L5 slab has rotated off

L4 slab failure along ends of H12 saddle bars 1200mm off Line 4 similar to L5 and L6

Failure surface runs diagonally from inside face of edge beam to the ends of the slab saddle bars

Column Line 4 D/E L4-5 and L5-6 with beam-column joint pullout at L5

North Core Examination

L5 Drag Bar

L5 slab from in front of lifts between Walls D/E and D

L5 slab failure surface 1200 mm out from Line 4 at end of H12 saddle bars

North Core Examination

- Slabs leaning against North Core
- Indicates loss of support along Line 3 prior to breaking off North Core

North Core Examination

- Drag Bars slab anchors at Levels 4, 5 and 6 upright behind wall tips
  - Slab appeared to have rotated off after collapse further south in building
- Some Drag Bars had been cut off during recovery operations

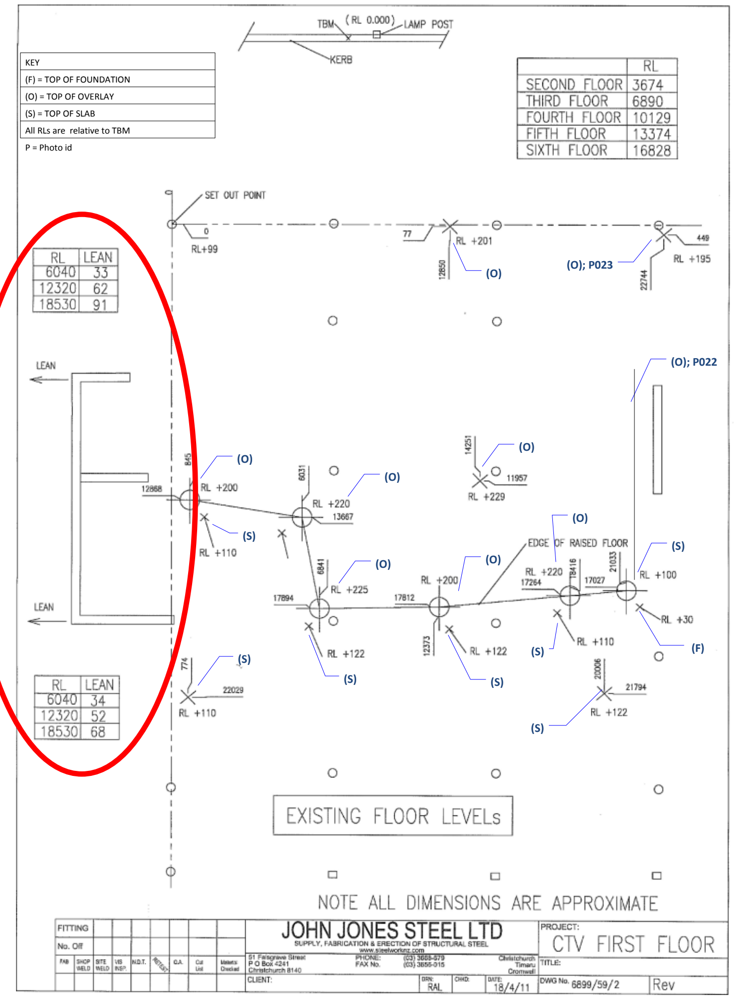
Level 4 Drag Bars

North Core Examination

- Beam bottom bars not cast into North Core L3 to L6
  - Non-compliant

Levels Survey

- No sign of settlement of the ground floor
- North Core had a northwards displacement
  - More at the eastern end



Foundations Examination

- No liquefaction was found adjacent to the foundations.
- No signs of uplift
- No signs of damage to the foundation beams

Materials Testing

- Concrete cores from walls, slabs and columns
- Reinforcing steel and decking
  - Compliant
- Drag Bar threaded anchors
  - Compliant
  - Results used to assess Drag Bar capacity

Concrete Quality

- Wall Concrete
  - Satisfactory
  - Localised door head issue
- Slab Concrete
  - Satisfactory
- Beam Concrete
  - Satisfactory
- Column Concrete
  - Non-compliances






Column Extraction at Burwood

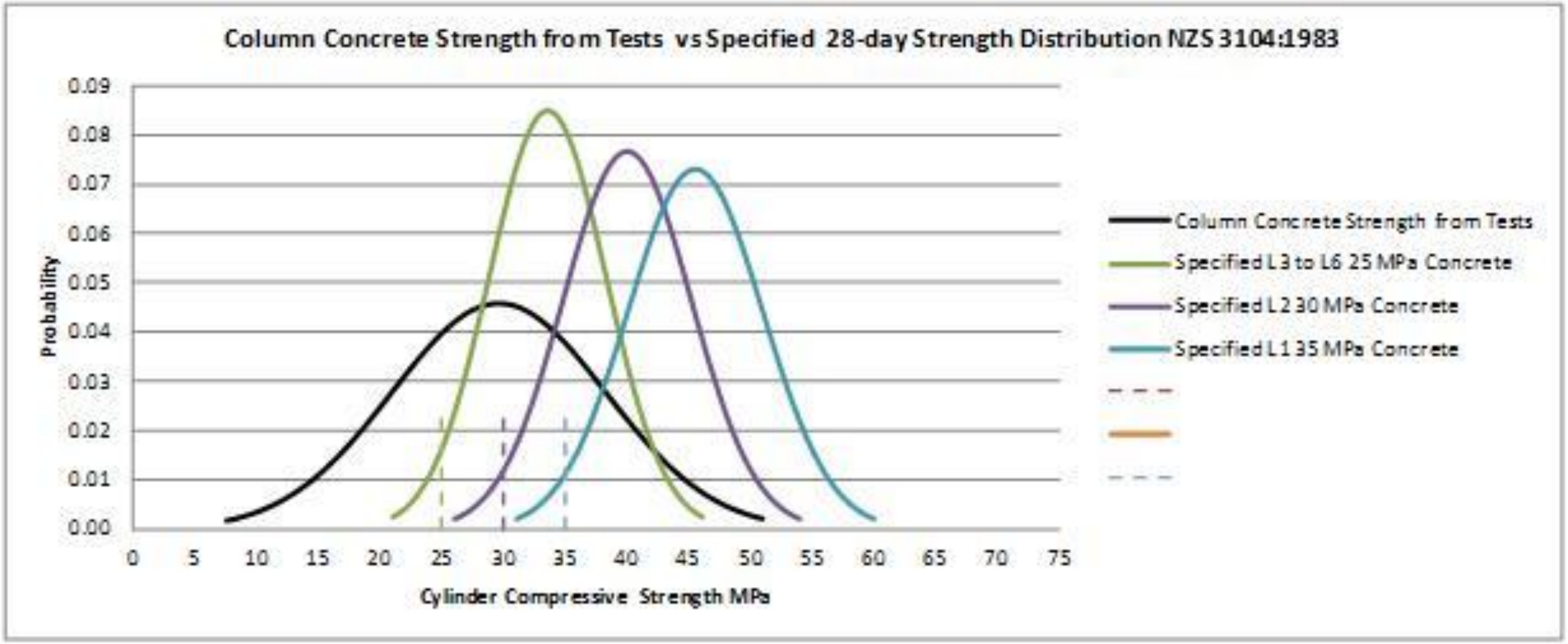
The CTV Building debris field at the Burwood Eco Landfill from which columns were extracted

CTV Building columns extracted for examination and testing

Column Concrete

- Cores extracted and tested
  - NZS3112:1986
- Rebound Hammer testing
  - Calibrated to core tests ASTM C805
- Comparison to concrete production statistical limits
  - NZS 3104:1983
- Comparison to 25 % aged
  - Concrete is known to increase in strength with time
- Density
  - Low in some cases

Column Concrete



- This indicates that the concrete in a significant proportion of the columns may have had strengths less than the minimum specified

Reinforcing Steel

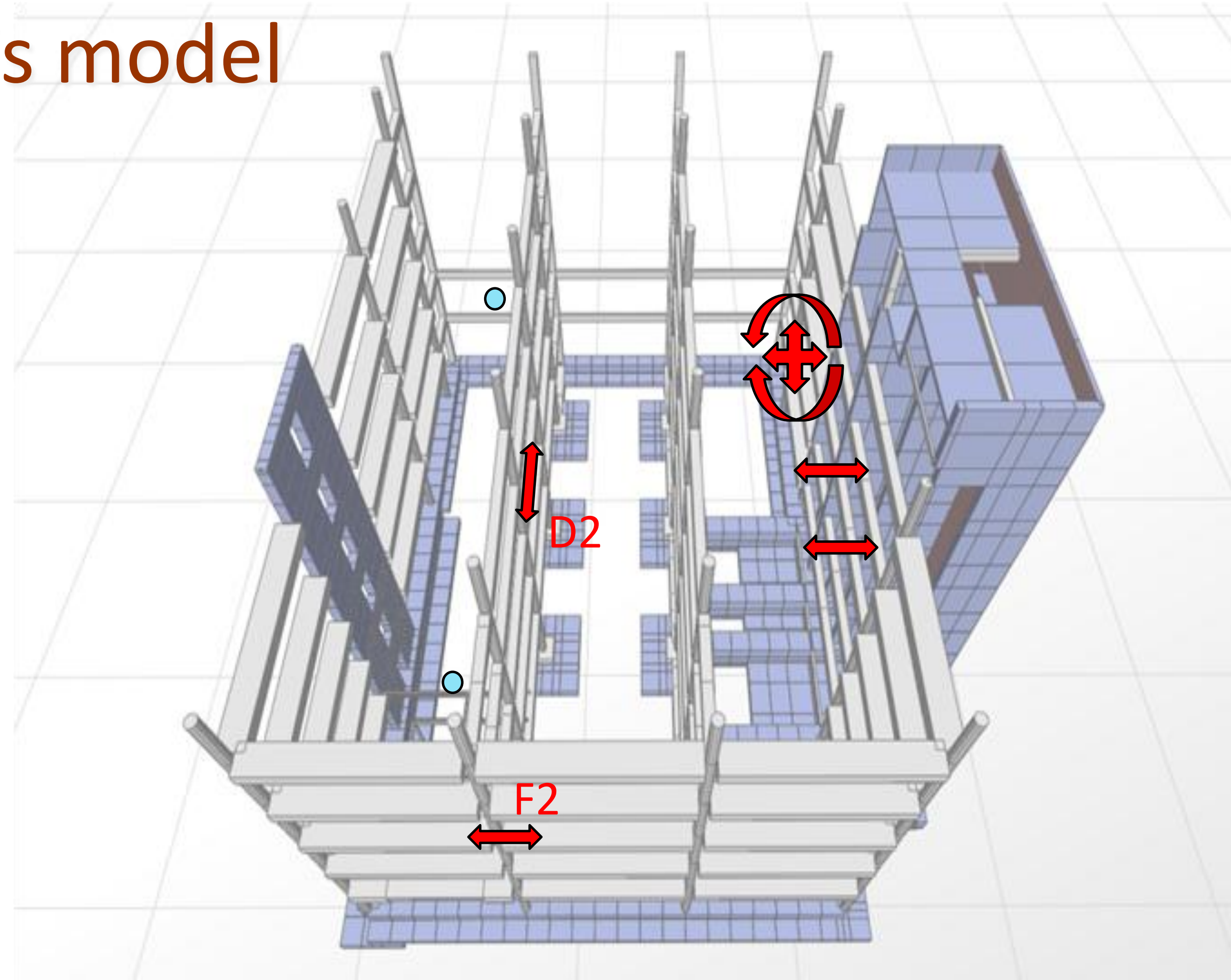
- South Wall
  - Compliant
  - Some yielding at base only

Structural Analysis

- ERSA
  - Elastic Response Spectrum Analysis
- NTHA (StructureSmith)
  - Nonlinear Time History Analysis
- NPA (StructureSmith)
  - Nonlinear Push-over Analysis
- Column drift capacity
  - CUMBIA / MATLAB
- Drift Compatibility
  - ERSA drifts compared to CUMBIA and NPA drift capacities
  - Could the columns cope with the drifts of the South Wall and North Core?

Non-Linear

Analysis model



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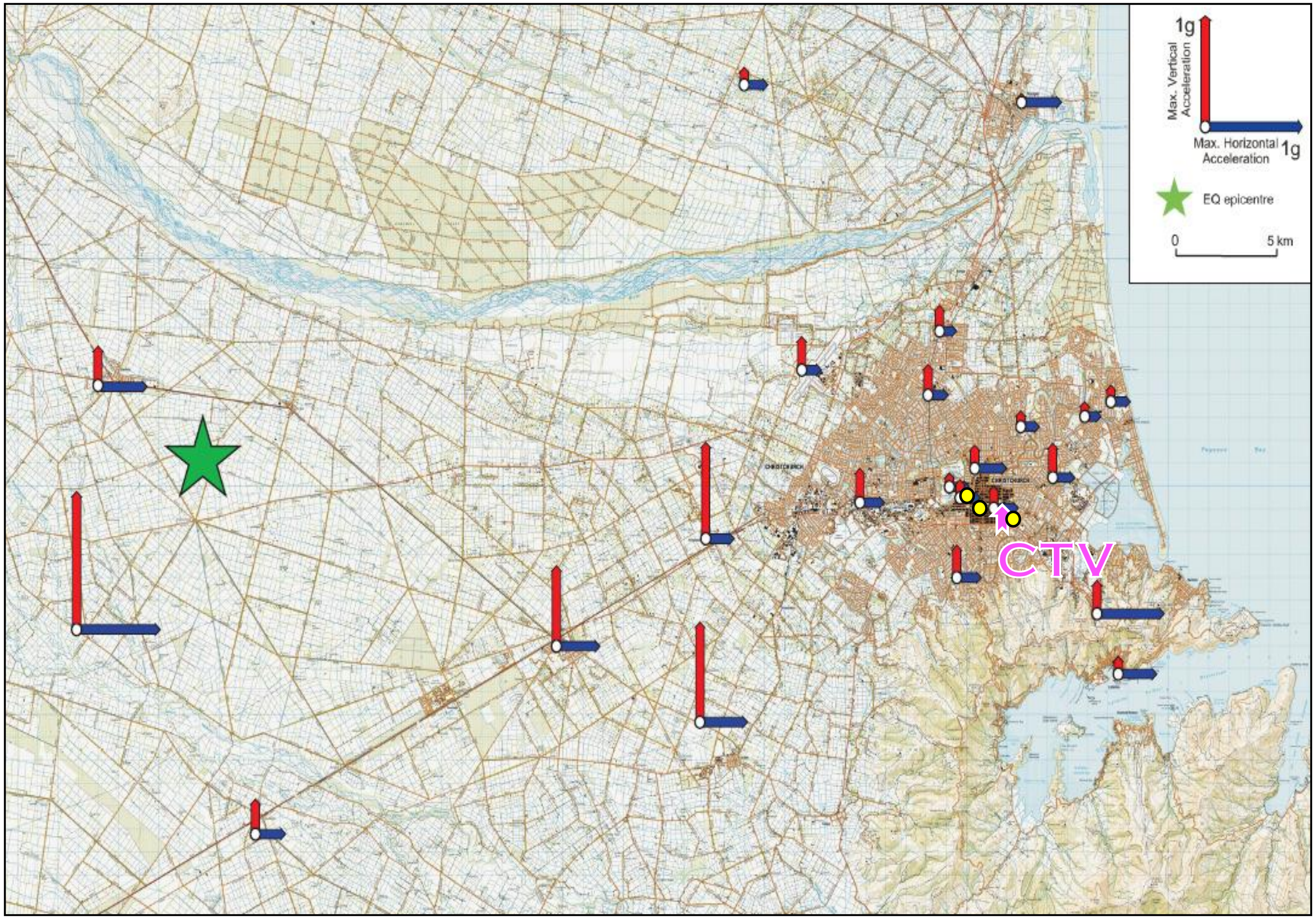
Recorded Peak Ground Accelerations – 4 September 2010



(Source: EQC-GNS Geonet)

Effects of 4 September 2010

- Estimated floor movements approx. 50% of 22 February
- Limited structural damage reported
- No evidence to indicate significant effects on earthquake performance

Column cracking Level 6

Spalling plaster at masonry wall

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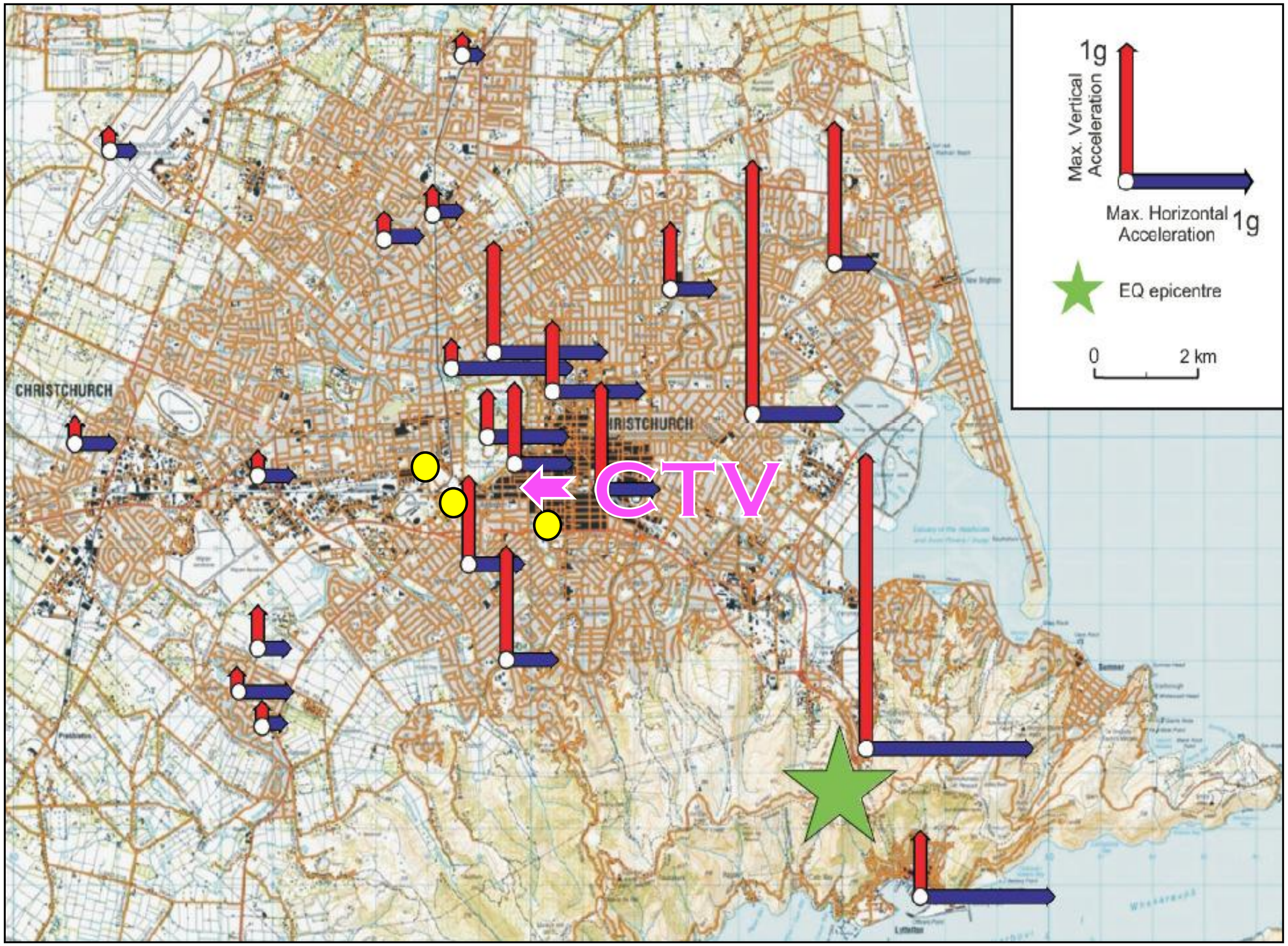
Recorded Peak Ground Accelerations – 22 February 2011



(Source: EQC-GNS Geonet)

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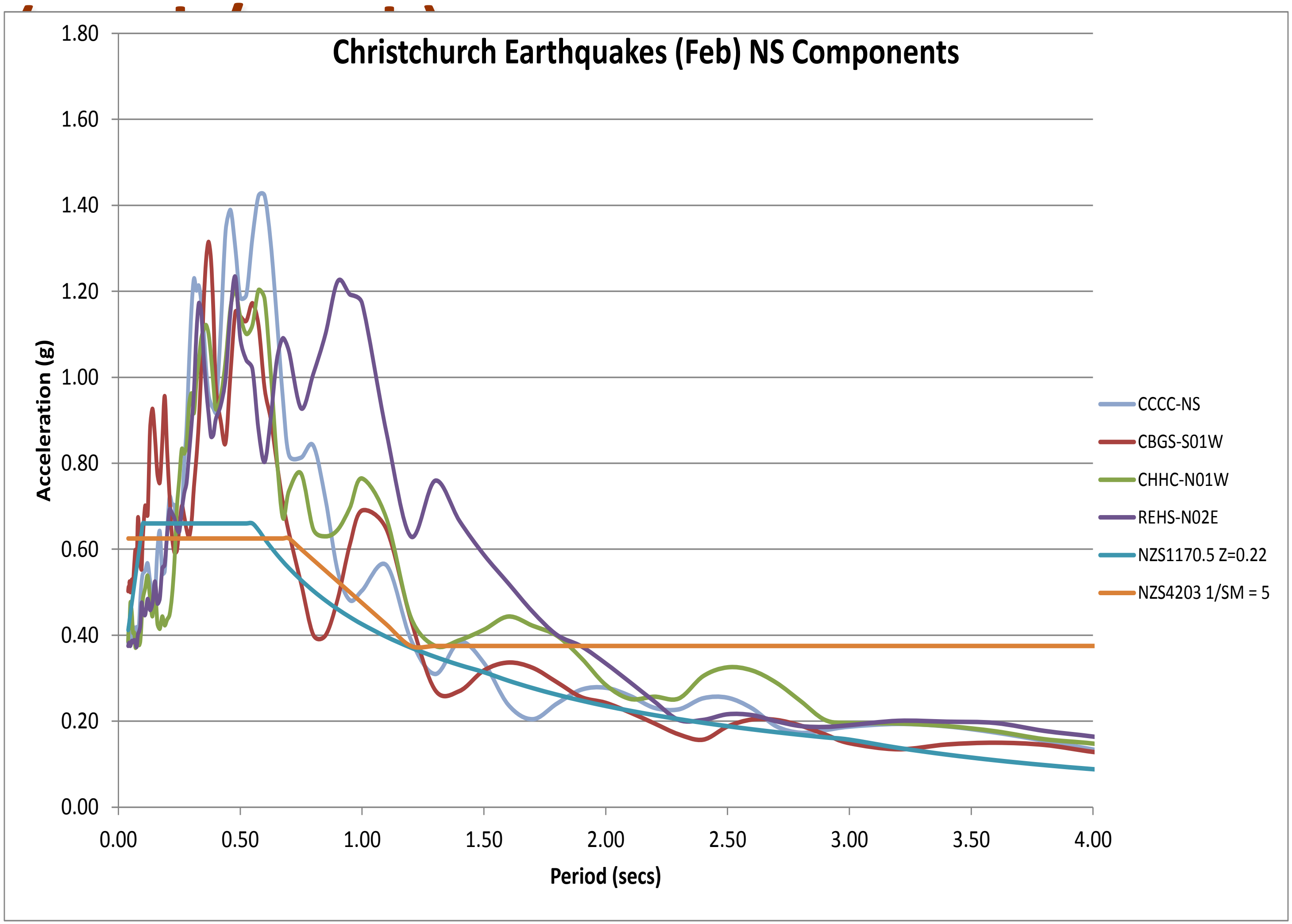
Acceleration response spectra



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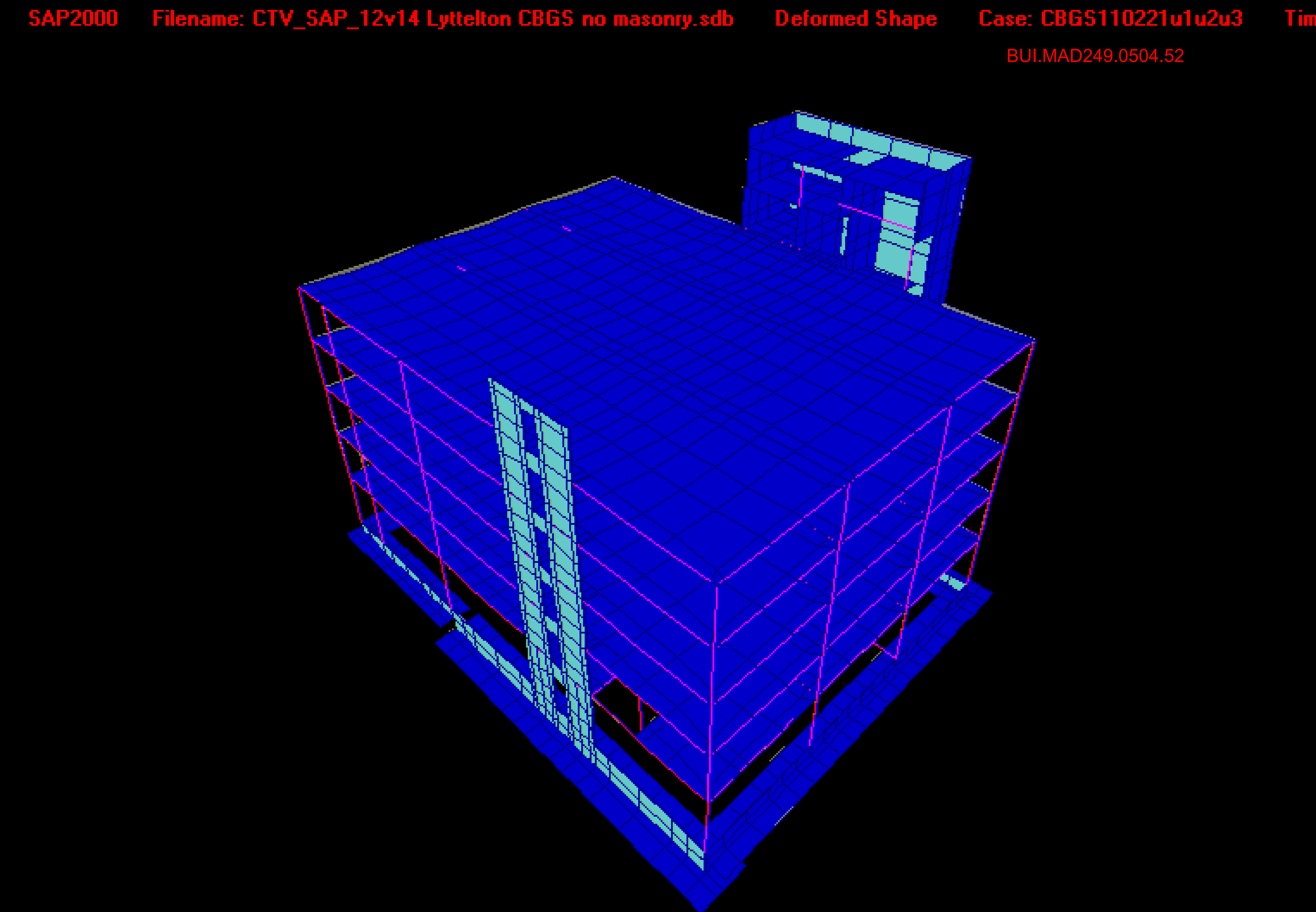
3-D Animation: 22 February 2011

- Ground shakes, causing building to move
- Notice:
  - Lateral (sideways) movement of floors
  - Twisting of floors
  - Strain on columns



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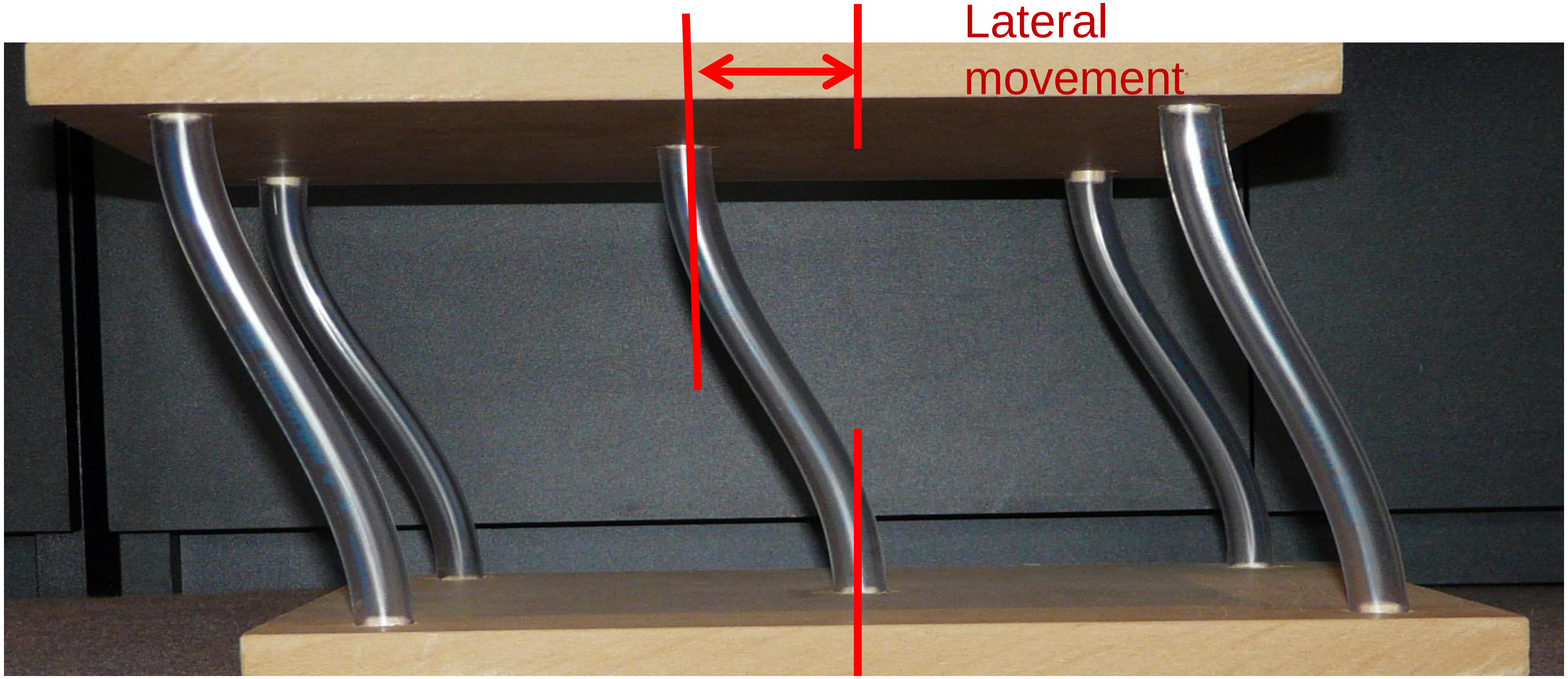
Columns under strain

Lateral movement of floors causes columns to distort

Distortion causes bending and shear in columns



Column drift demand and capacity

Δ_{total} = **lateral movement between floors**

= inter-storey drift demand

Δu = total drift capacity of column

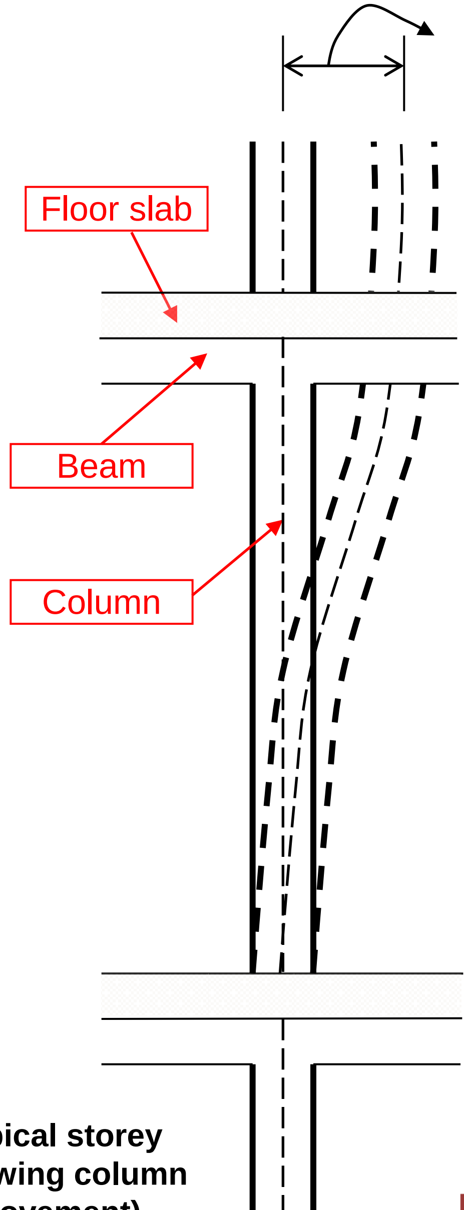
$= \Delta y + \Delta p$

Δ_y = drift when steel first yields

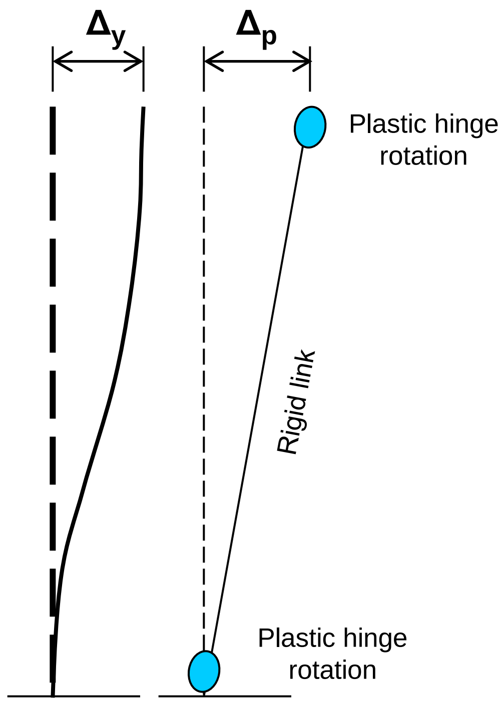
= ***elastic*** displacement

Δ_p = drift after steel yield up to failure

= ***plastic*** displacement

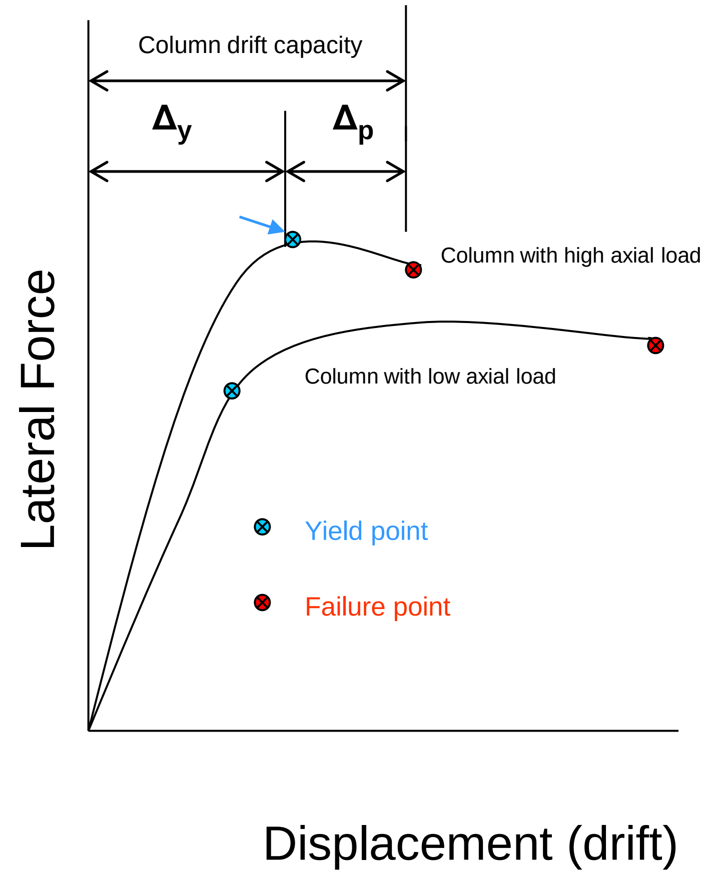


Typical storey (showing column movement)



(a) Elastic (b) Plastic

Components of 'drift'



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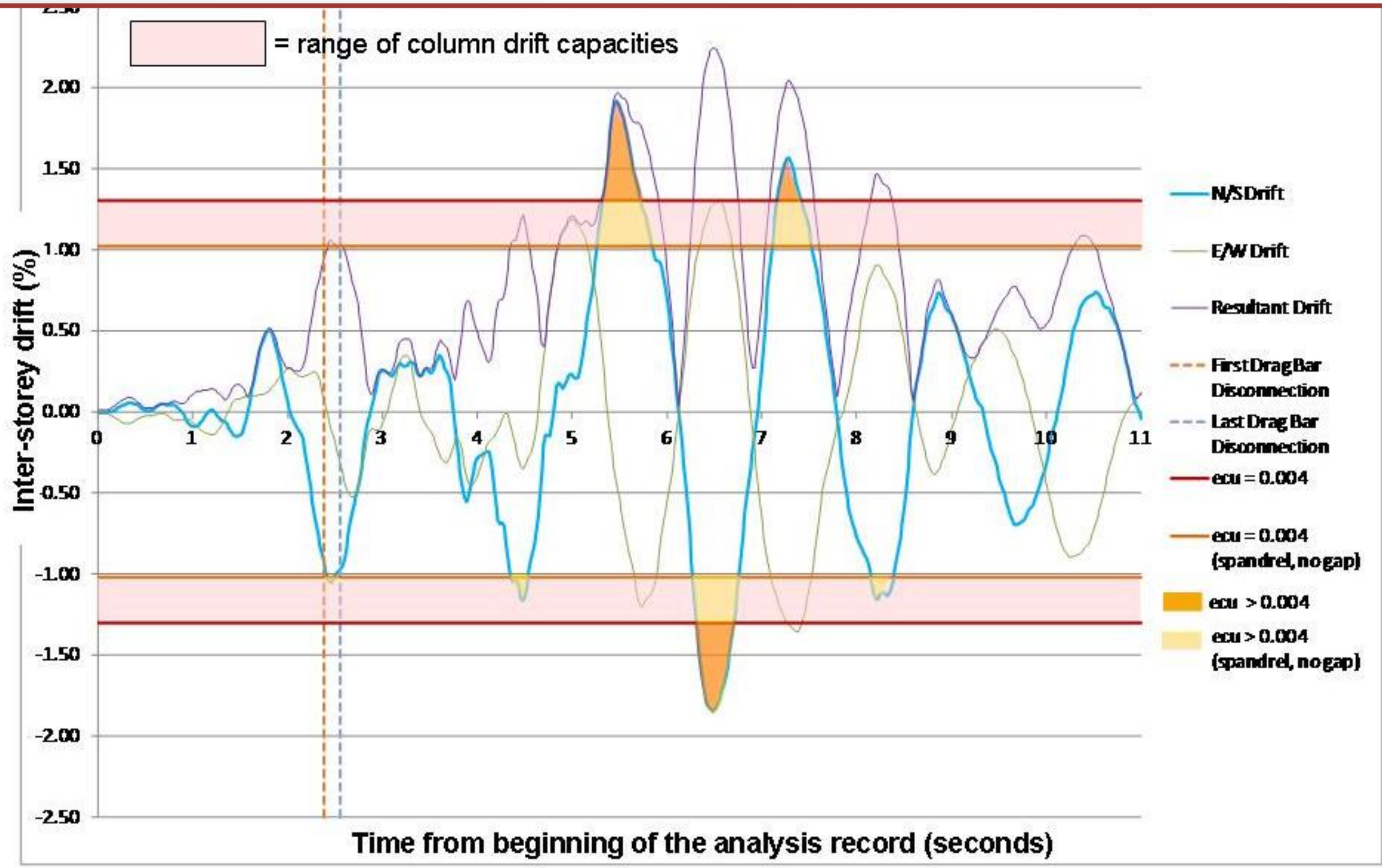
Column F2 Level 3 – 22 Feb 2011 – CBGS Record



Comparison of drift demand and capacity

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Drift demand vs. capacity for column at grid D2

Level	Axial Load (kN)	Drift Capacity (% of storey height)			E/W Drift Demand 22 Feb CHHC (%)	Ratio Demand / Capacity (Ɛcu=0.004)
		First Yield	Nominal Strength	Ɛcu=0.004		
5	324	0.71	0.85	1.31	1.65	1.26
4	681	0.85	0.90	1.20	1.85	1.54
3	1038	no yield	0.89	1.10	1.86	max. 1.69
2	1328	no yield	0.95	1.08	1.76	1.63
1	1682	no yield	0.90	0.96	1.46	1.52

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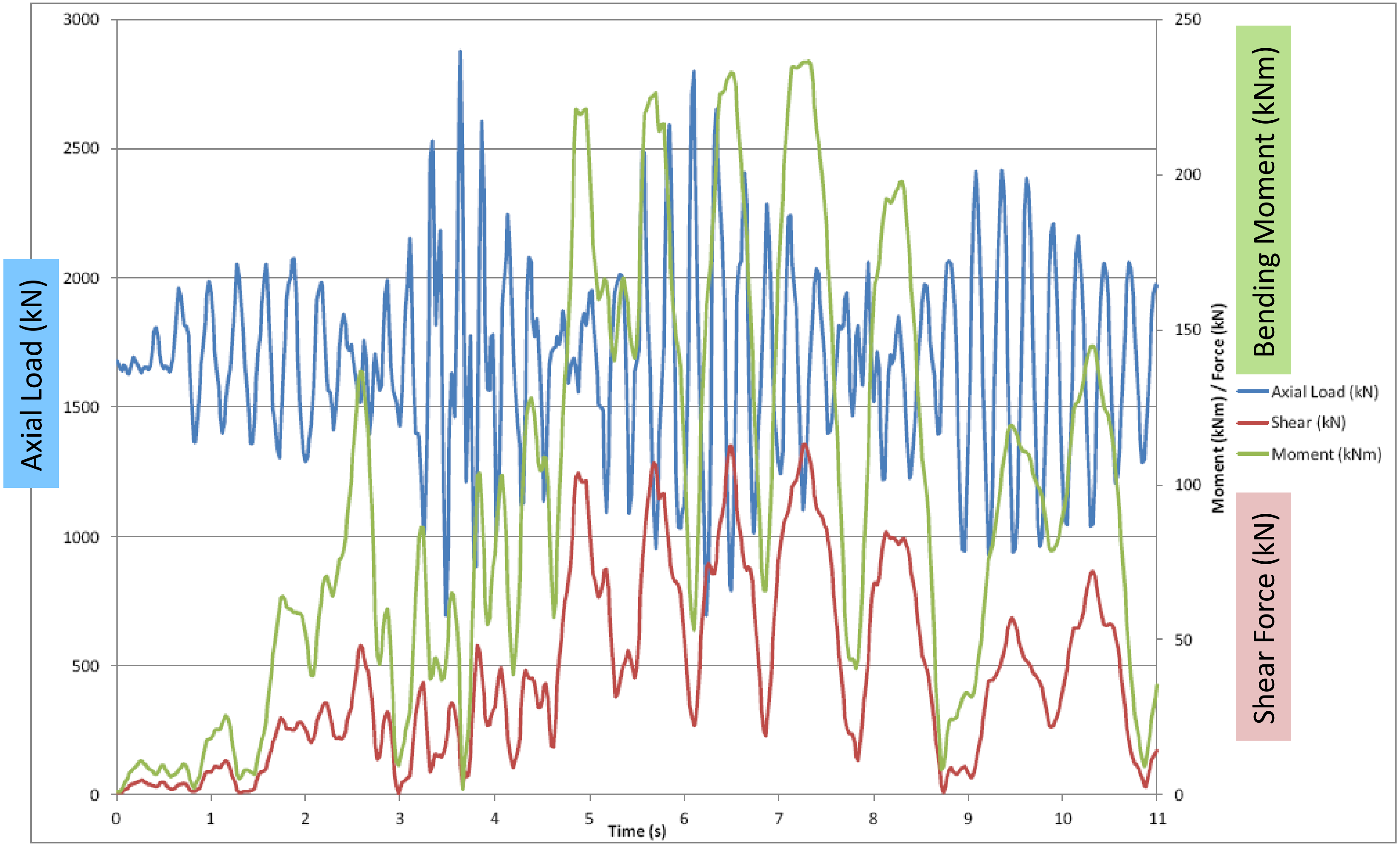
Column Actions Grid D2 Level 1 - 22 Feb 2011



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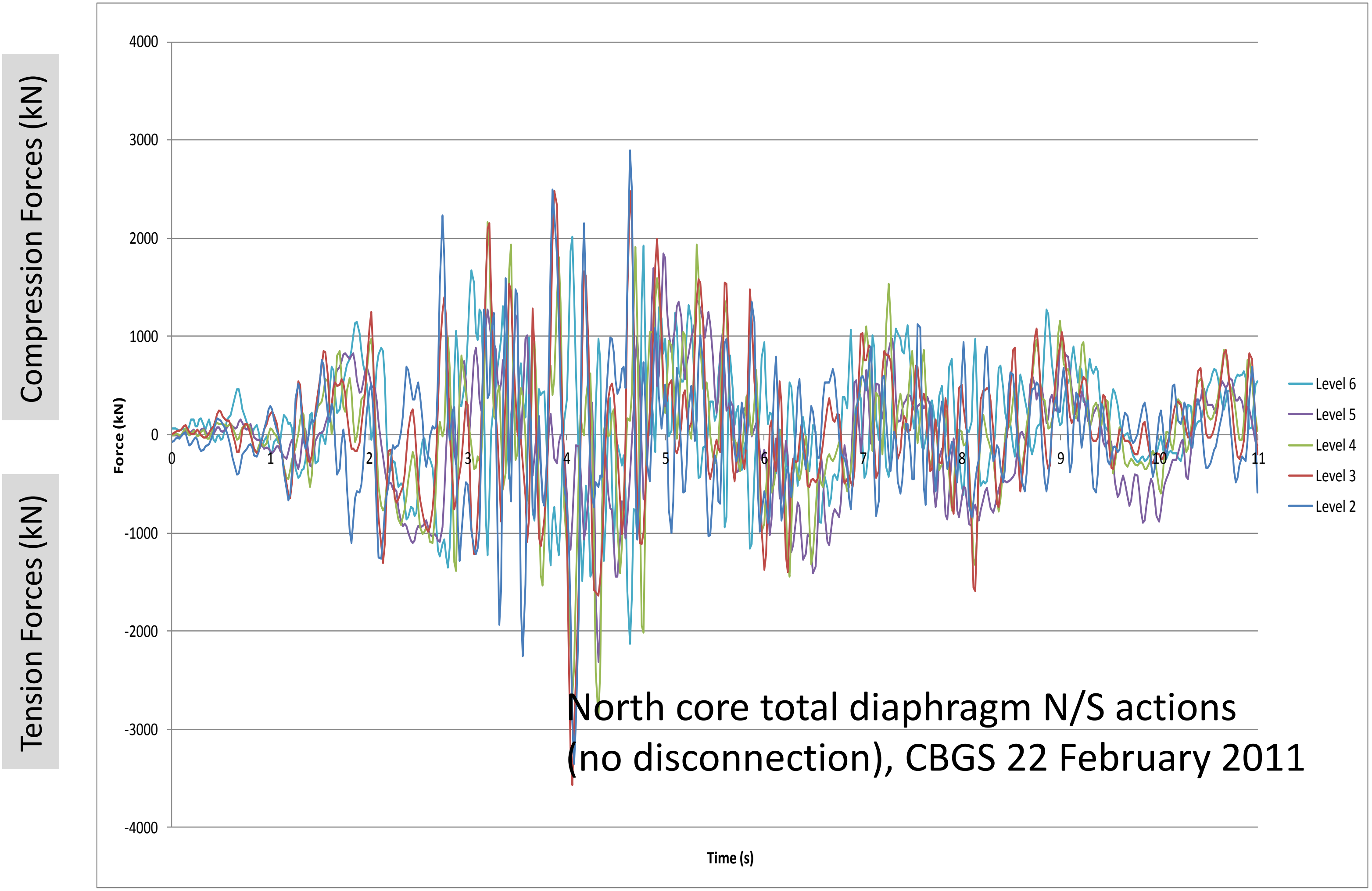
Floor Connection to North Core



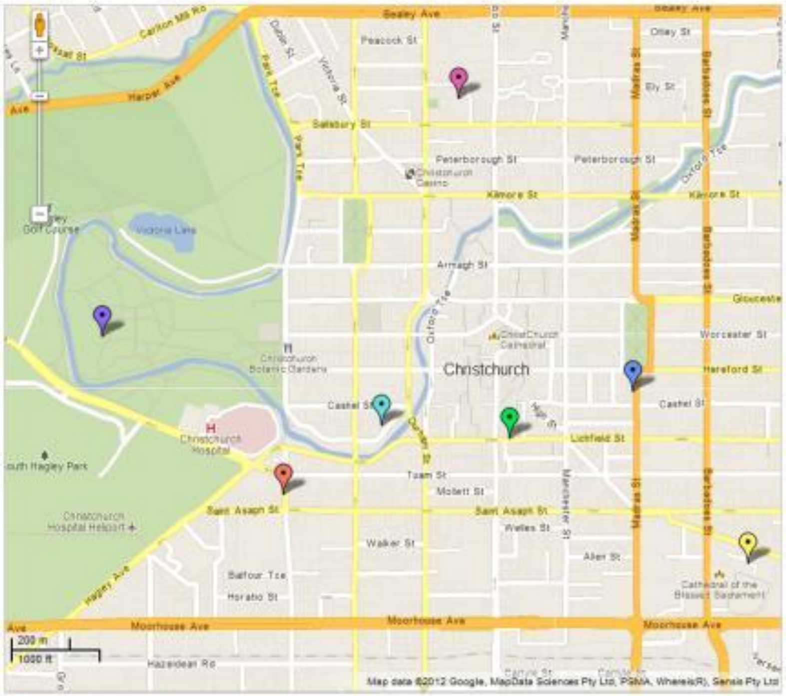
Comments on NTHA

- Inelastic behaviour using earthquake records
- Earthquake records used without scaling
- Calibration to observed damage debatable
- Appears to over predict damage
  - Drag Bar failure predicted in September Earthquake
  - Drag Bar failure predicted very early in February Aftershock
    - Site evidence and L4 witness testimony indicates Drag Bars did not fail before collapse started elsewhere

ERSA

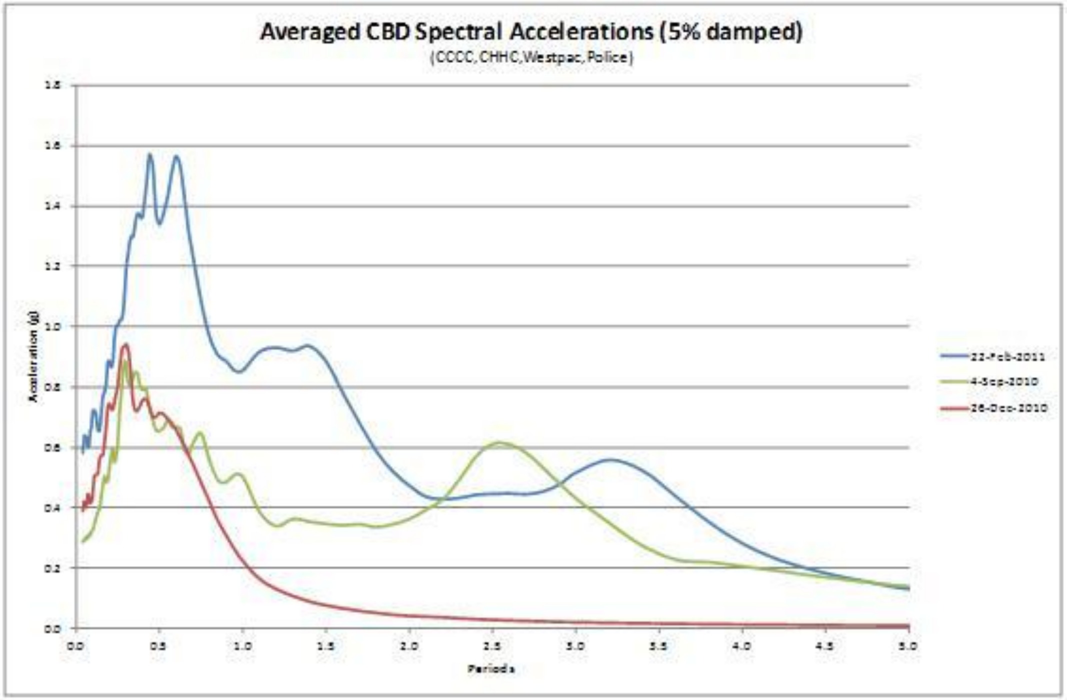
- 3D elastic structural behaviour
  - Standard design spectra for compliance checks
    - Effect of masonry infill contact considered
  - CBD earthquake records spectra
    - September Earthquake response 2.0 x December Aftershock
    - February Aftershock 2.2 x September Earthquake
      - Tables 15 to 17

ERSA Earthquake Records



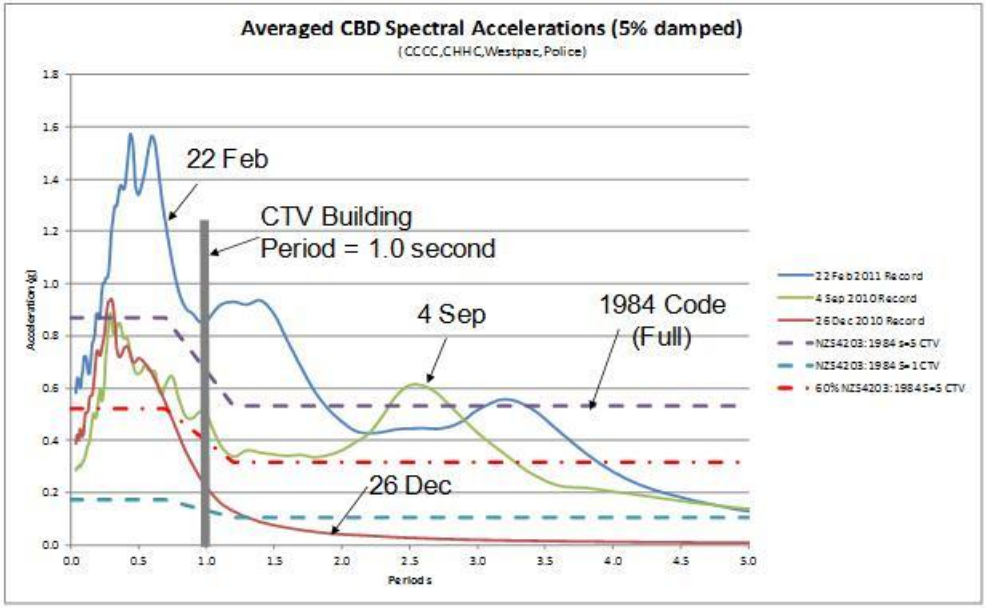
- GNS recording stations used

ERSA Earthquake Records



- Development of Response Spectra for comparative ERSA

Earthquake Records vs Design Spectra

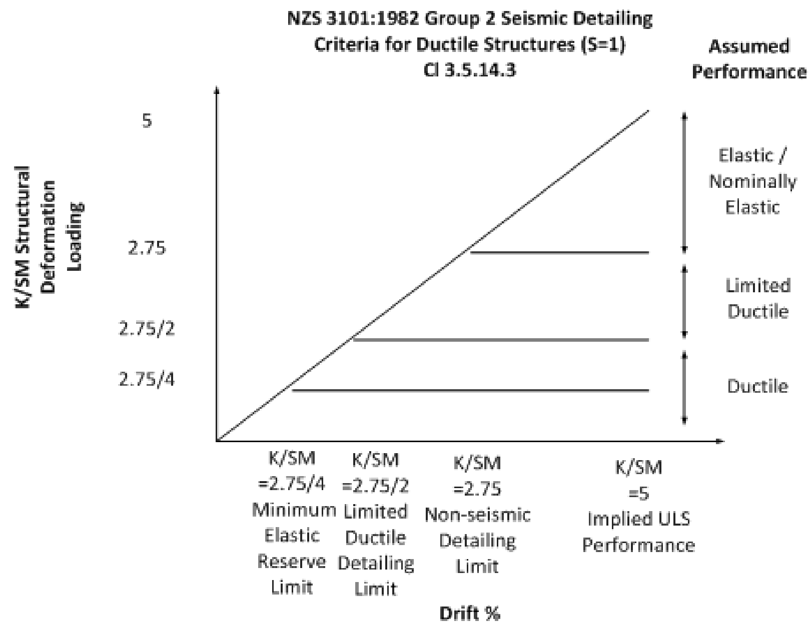


- Earthquake Loadings Standard (NZS 4203:1984)
  - Ductile response
- Design spectra vs Earthquake spectra
  - Calibration issues

Compliance Checks to Standards

- Compliance checks to Standards using ERSA
  - Walls "complied" with inter-storey drift limits
    - Column C/1 K/SM=2.75 drift 0.80% at Level 5 < 0.83% (Table 13)
      - Design practitioners would likely have used more conservative development of "dependable capacity" as measure of elastic limit.
    - Should have been adequate to protect columns
      - Implied safe ultimate ULS drift performance of Standard at S=5 loads
        - » 0.83% x 5/2.75 = 1.51%
    - However lack of symmetry means check would under predict drifts
  - Columns non-compliant for seismic spiral reinforcing limits
    - Elastic deformation limits less than K/SM=2.75 drifts (Table 13 and 14)
  - Columns non-compliant for spiral reinforcing for shear under imposed drifts
    - Application of the drifts causes high shears in columns heads
    - Minimum shear requirements not satisfied to NZS 3101:1982 (p.110)

Column Drift Limits



- At K/SM = 2.75 loads (2.75 x S=1 loads Fig 162)
  - Elastic behaviour required if no additional seismic reinforcing provided
    - 55% ULS drifts
    - Note NZS 3101:1982 Appendix B and ACI 318-71
    - Working Stress vs Strength Design
  - Additional seismic reinforcing required if elastic behaviour exceeded at or less than that demand
    - Stiff columns would require more reinforcing than more flexible columns to give safe performance
- 1.51% safe drift performance appeared to be expected by the Standards
  - Drift performance of 1.51% at S=5 loads
  - Principle of equivalent ductile displacement is basis of NZ seismic deign standards

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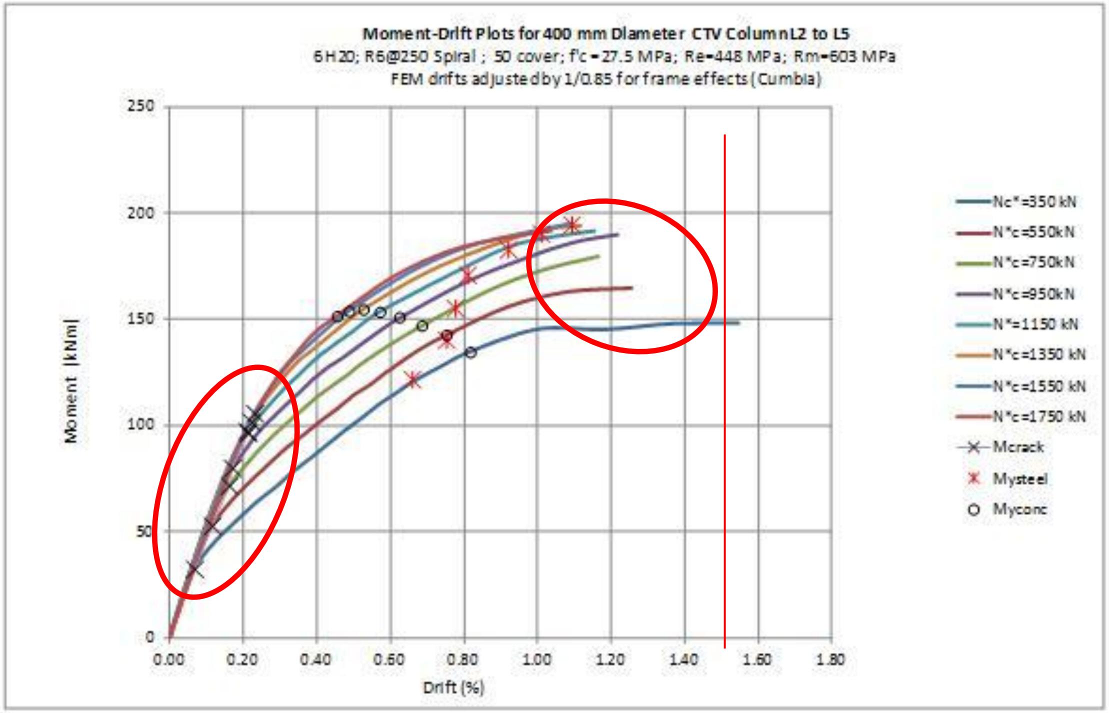
Column Drift Capacity



- At average tested concrete strength of 27.5 MPa (Fig 159)
  - Cracking at 0.10 to 0.35% drifts
- Level 2 to 4 columns North and East faces 1.15 to 1.45% drift capacity (Table 13 and 14)
  - Less than 1.51% safe drift performance expected by Standard

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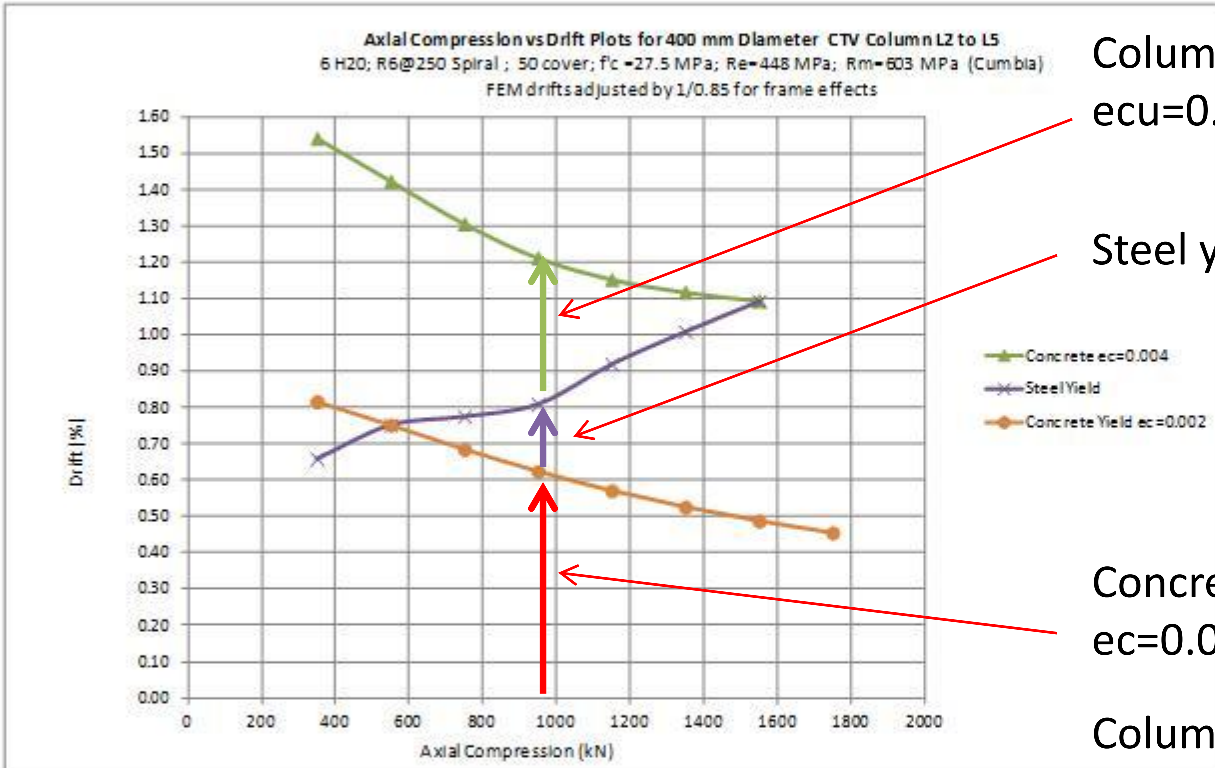
Column Drift Capacity



Column failure limit
ecu=0.004 at 1.20% drift

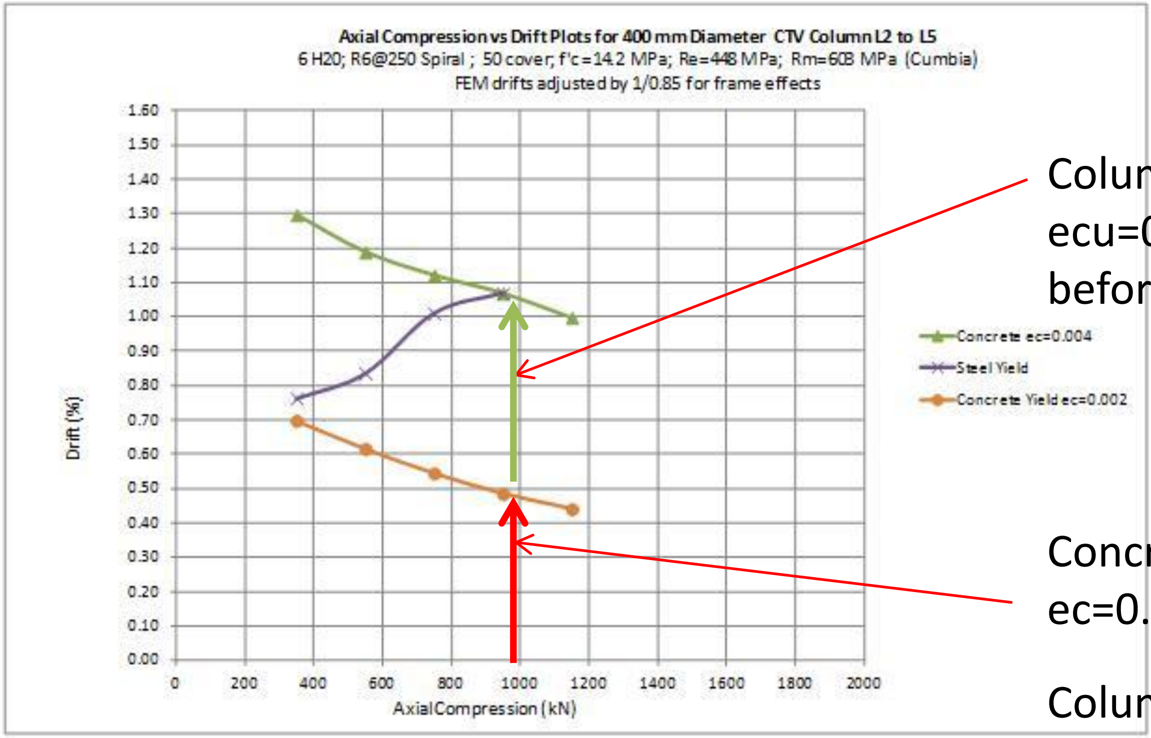
Steel yield at 0.80% drift

Concrete yield limit
ec=0.002 at 0.60% drift

Column F/2 Level 2
with 995 kN axial load

- At average tested concrete strength of 27.5 MPa
- Drift capacity reduces with increased axial load from vertical acceleration
  - Table13 for C/1 and Table 14 for F/2 axial loads and drift limits

Column Drift Capacity



Column failure limit ecu=0.004 at 1.05% drift before steel yields

Concrete yield limit ec=0.002 at 0.48% drift

Column F/2 Level 2 with 995 kN axial load

- At lower 5%ile tested concrete strength of 14.2 MPa
- Drift capacity reduces with reduced concrete strength

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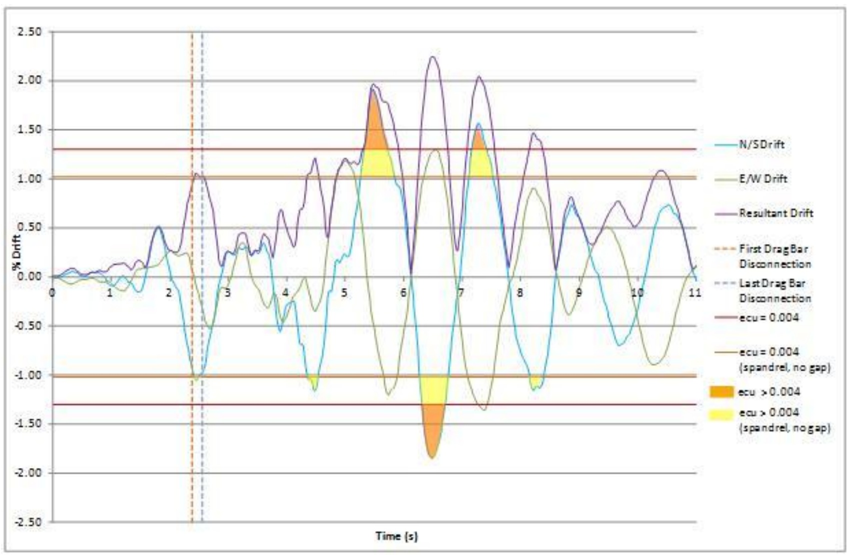
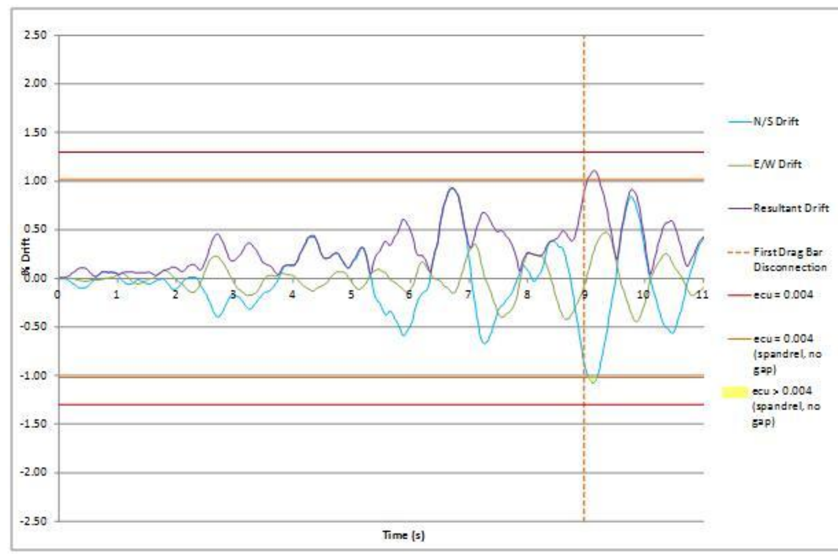
NTHA Drag Bar Failure Estimates

Column F2 Level 3 Drifts - CBGS, 22 February Lyttelton Aftershock, no masonry



Column F2 Level 3 Drifts - CBGS, 4 September Darfield Earthquake, no masonry

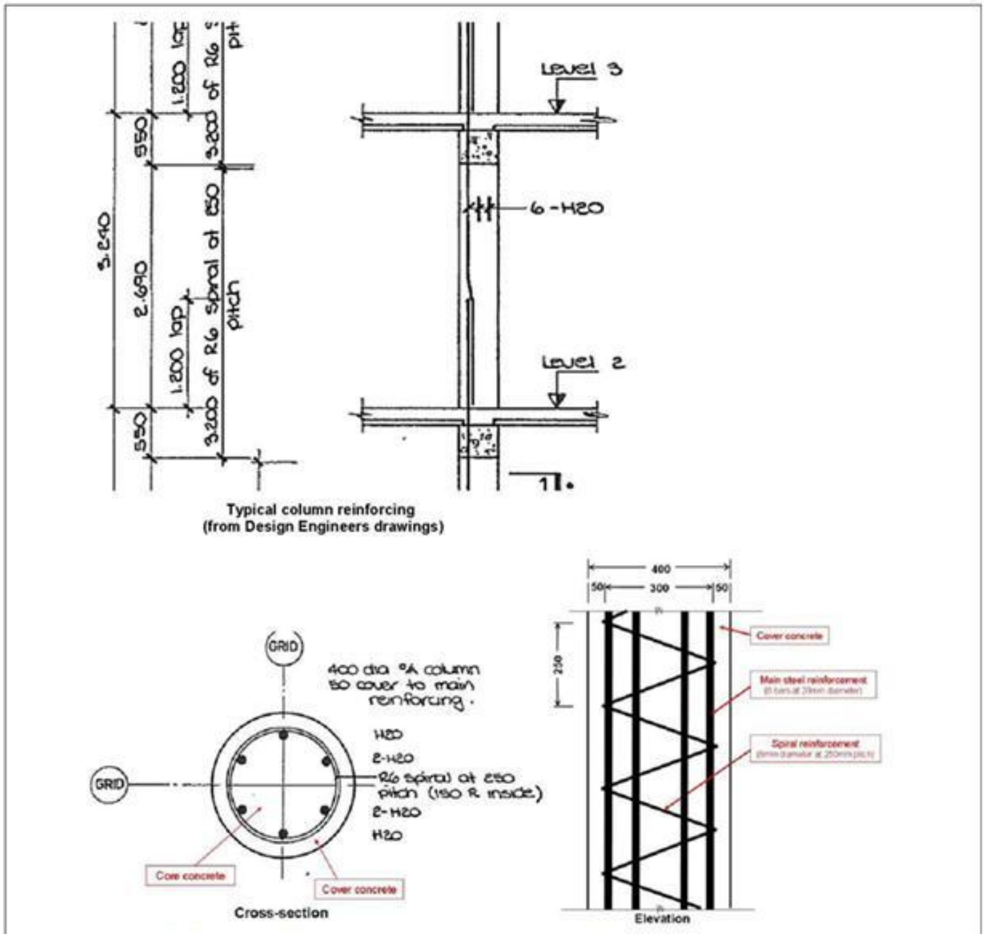


Drag Bar failure estimated to occur when 1% drift along Line F

Compliance Checks and Collapse Drift

- Compliance Checks with Standards
  - Design As-drawn :
    - Non-compliant column spiral reinforcing
    - Non-compliant slab diaphragm connections to North Core prior to remedial work
    - Non-compliant lack of seismic separation of Spandrel Panels from columns
  - As-built fitness for purpose check with defects:
    - Non-compliances as above
    - Non-compliant Level 3 to 6 beam connections to North Core
    - Non-compliant separation of masonry infill from Line A west face
    - Non-compliant masonry infill at Level 1 South Wall exit
    - Drag Bars unable to sustain ULS structural response
- Assessment of Inter-storey Drift on CTV Building at Collapse
  - Used tested properties and strengths
  - Estimated less than 1.0% drift along East face at collapse
    - Based on Drag Bar failure estimated at 1.0% prior to Drag Bar failure
    - Initiating at Level 3, 4 or 5 columns
    - Design standard expected 1.51% safe drift performance

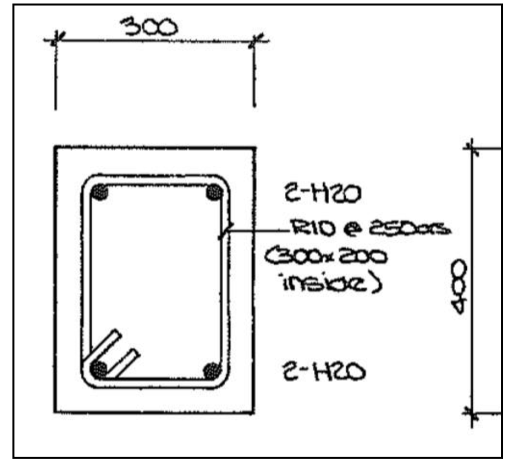
Concrete Columns



Typical column reinforcing (from Design Engineers drawings)

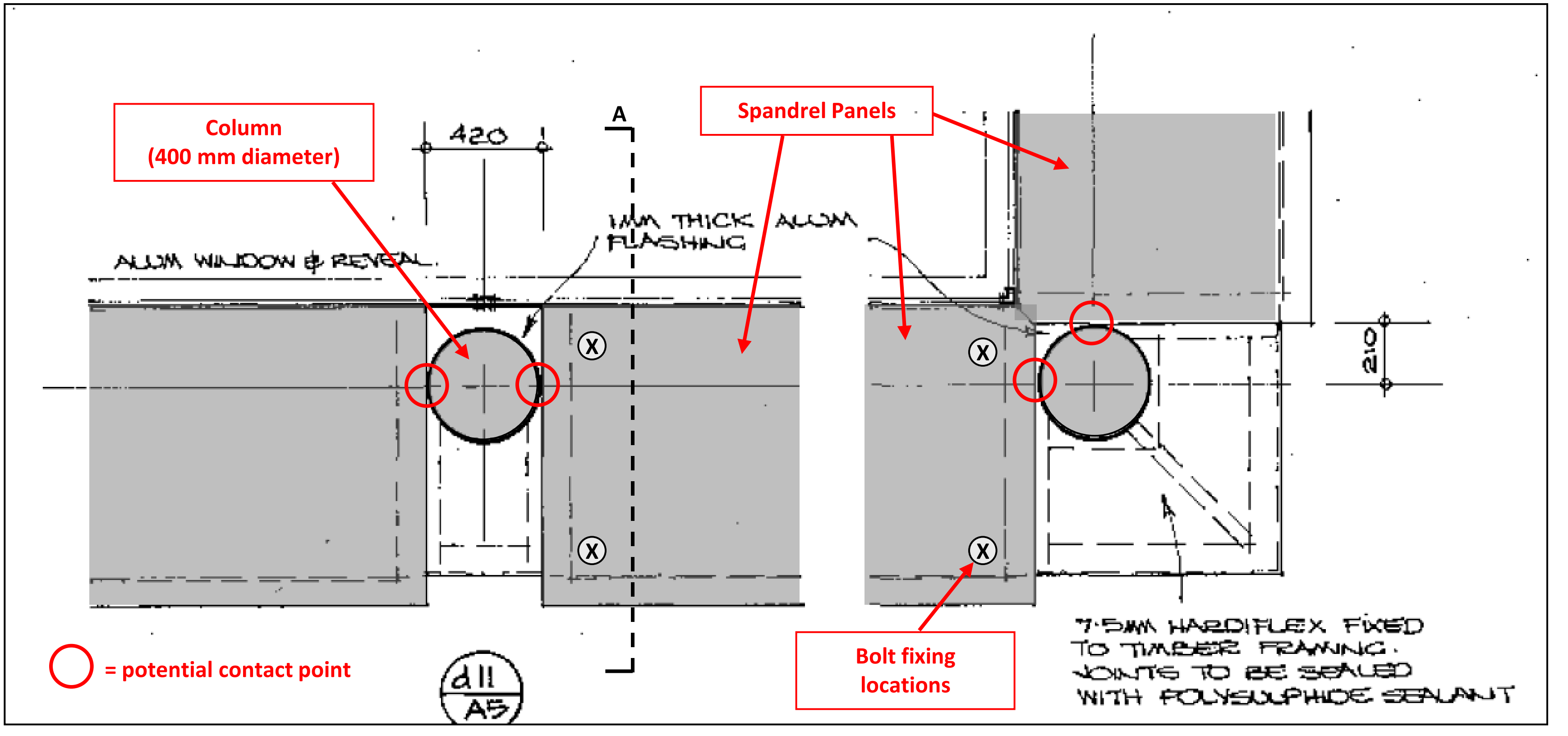
Cross-section

Elevation



- Same reinforcing in all main building columns
- Light spiral binding R6 @ 250 centres
  - Non-compliant
- Rectangular columns on Line A only

Spandrel Panels



Non-compliant : no seismic gaps specified between columns and Spandrel Panels

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Spandrel Panels and Columns

Line F columns on east face

Column C18

North Core on north face

Concrete edge beams

Pre-cast concrete Spandrel panels between columns on edge beams

CTV Building in May and October 1987 during construction (viewed from northeast)

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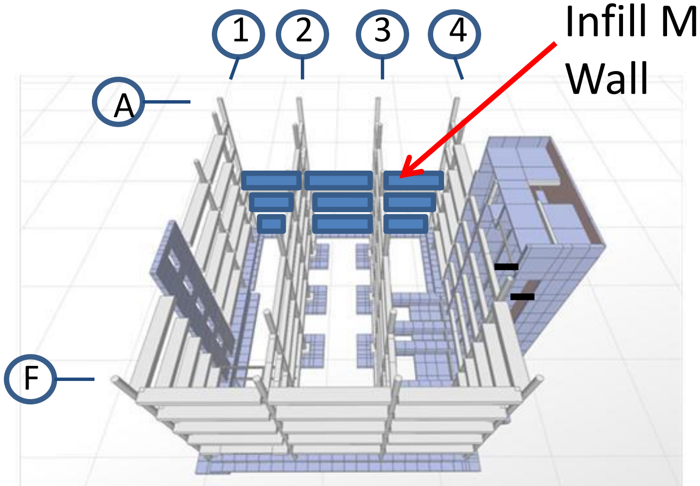
Masonry Infill Wall on West Face



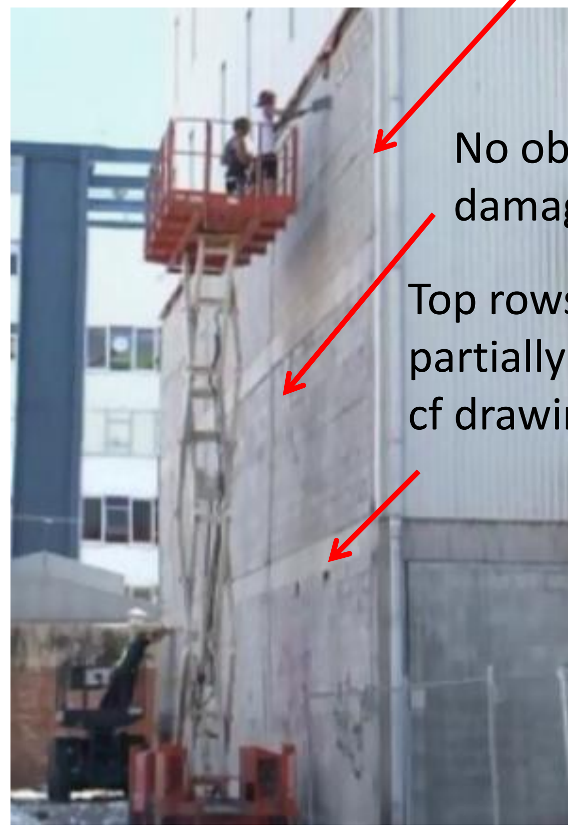
Infill Masonry Wall

No side gaps at columns

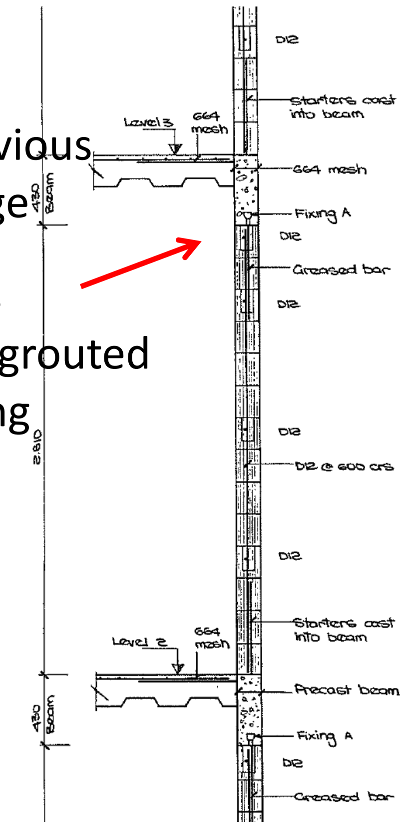
No obvious damage

Top rows partially grouted cf drawing

- Drawings showed
  - Grouted top row of masonry at each floor
  - 25 mm gaps on sides at columns
- Workers outside just before collapse found
  - Top rows partially grout filled
  - No gaps on sides at columns on outer face
  - No obvious damage from September eq.
- Staff inside found
  - Sealant and gaps on sides at columns

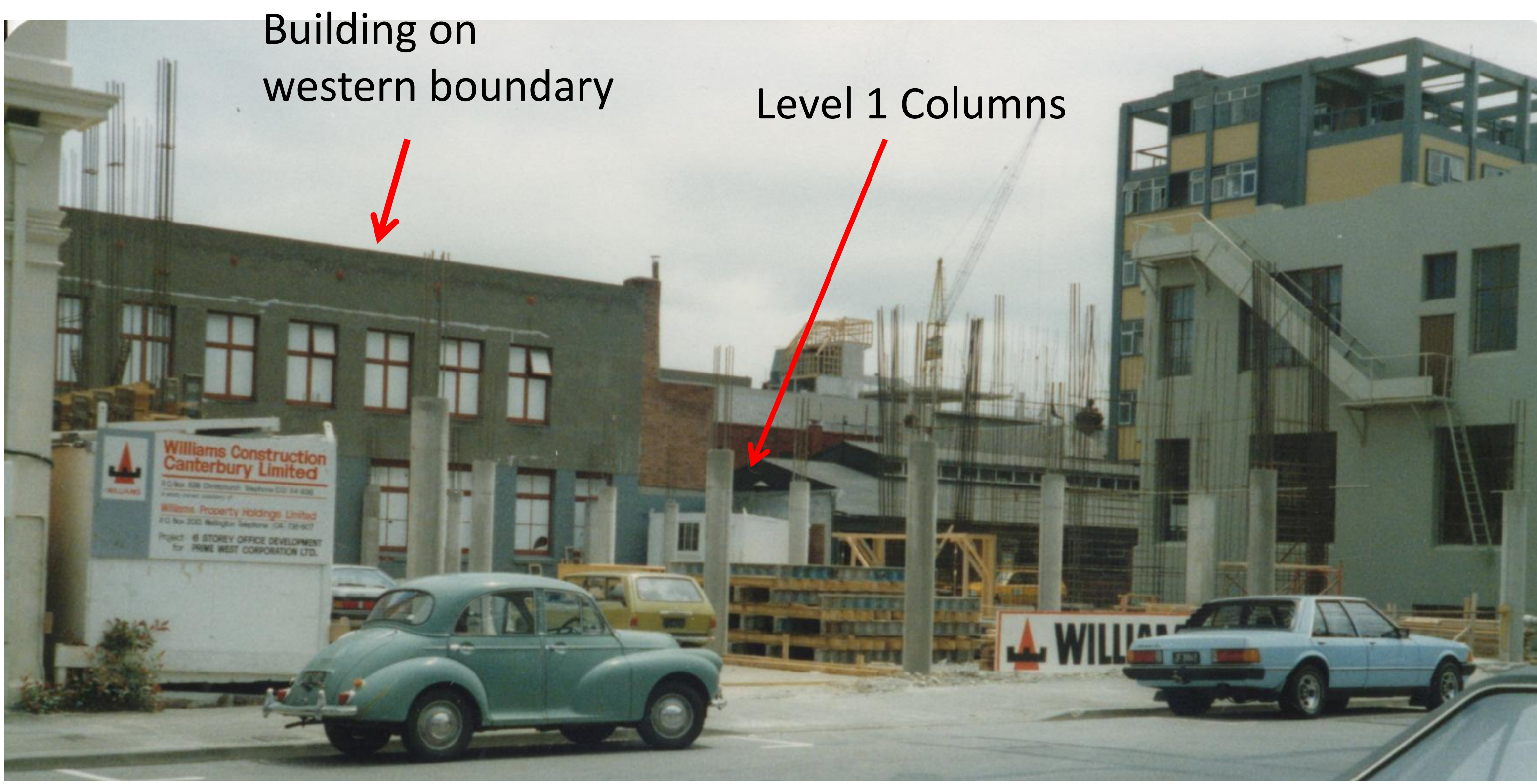


Workers preparing wall for cladding just prior to Eq (CTV News)



Section through wall from Drawings

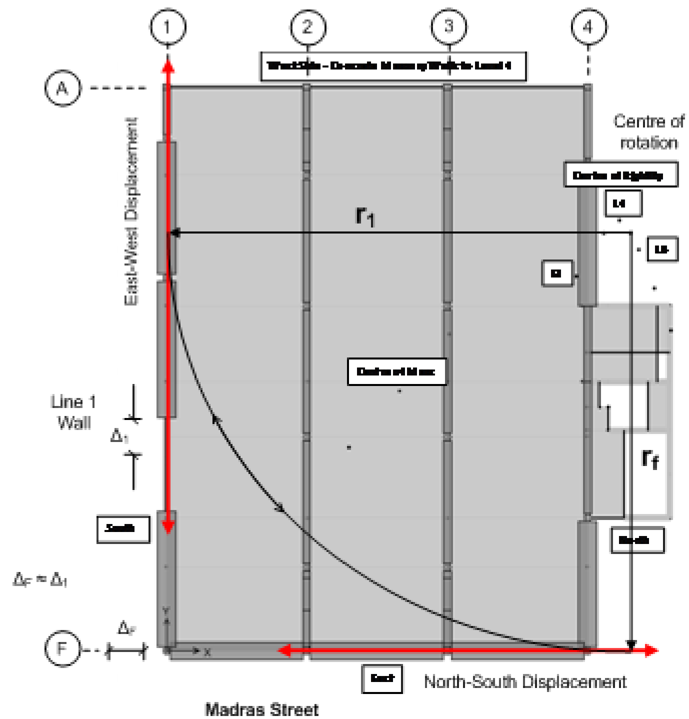
Adjacent Building



CTV Building in January 1987 during construction (viewed from southeast)

Effect of Masonry Infill on West Wall

- Increased torsional eccentricity
- Drifts on East face similar to drifts on South face
- May have reduced demand on South Wall



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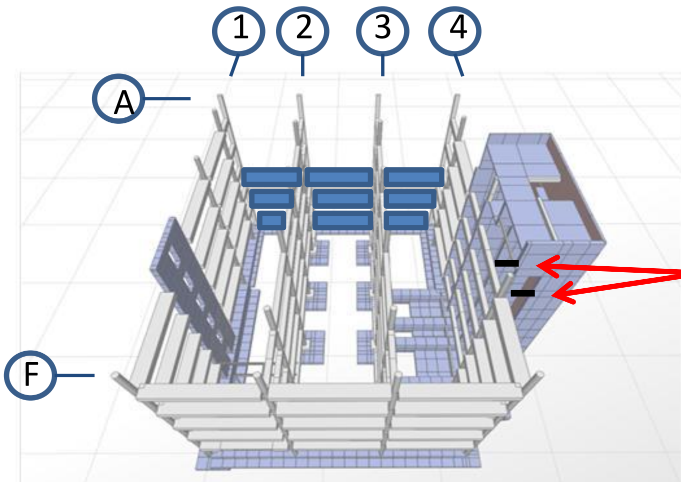
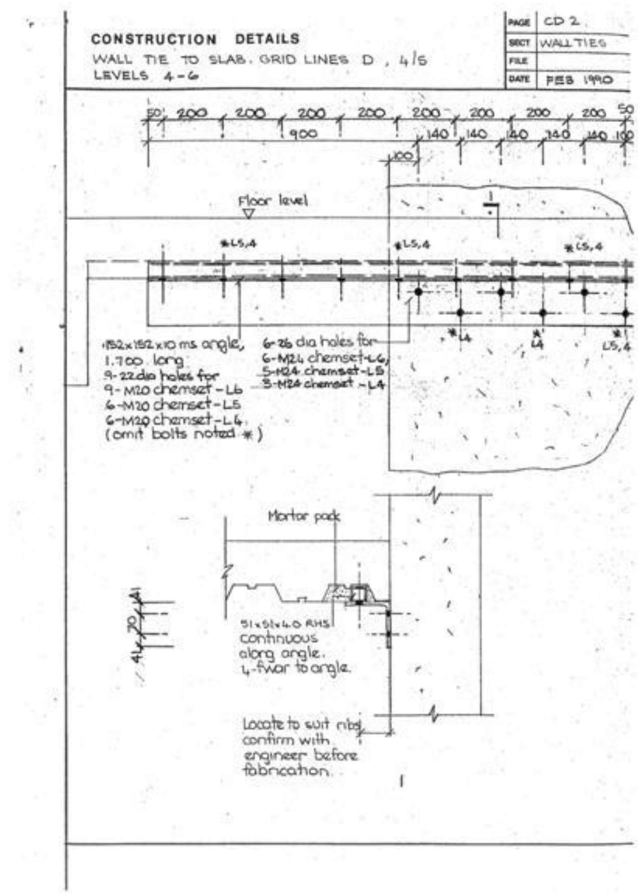
Drag Bars Added after Completion

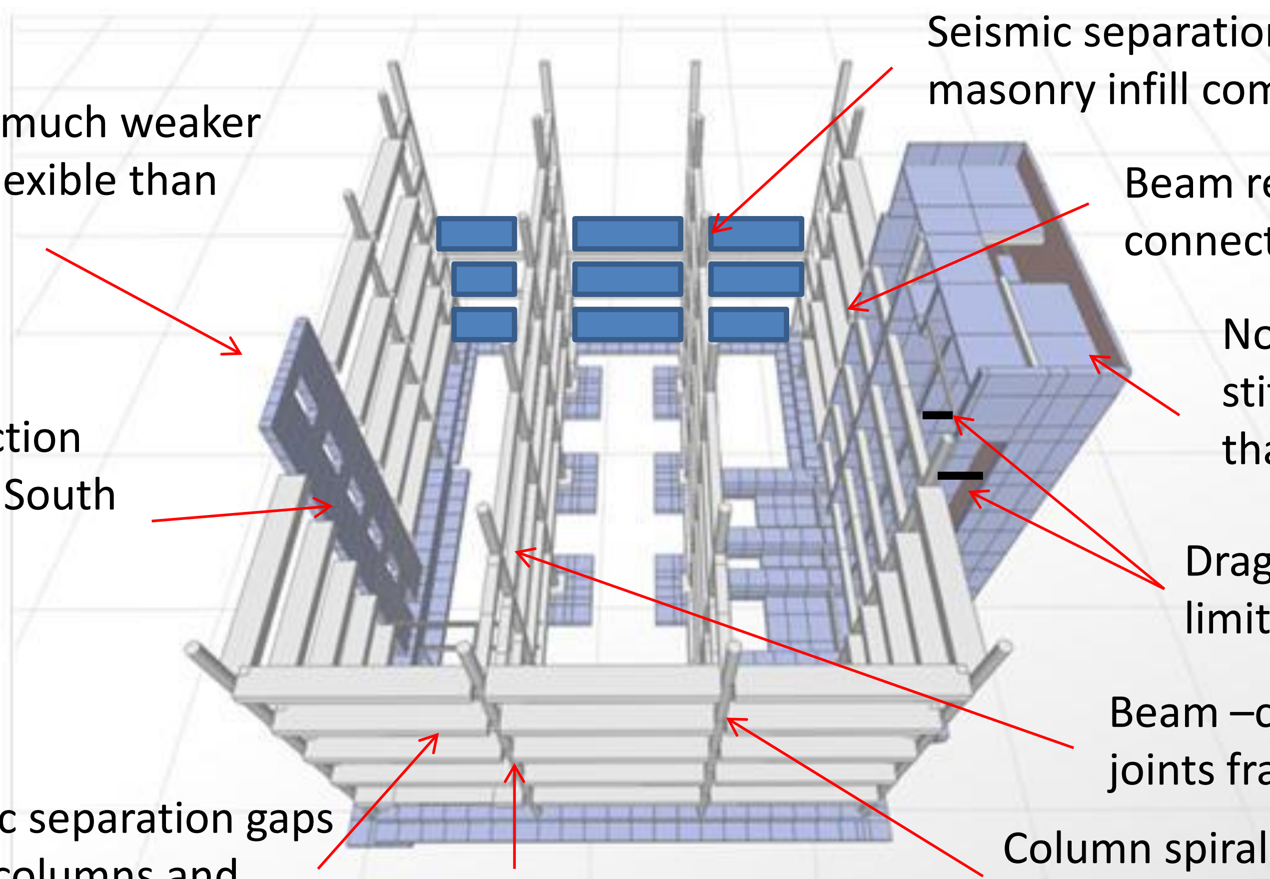


Drag Bars Level 4, 5 and 6



- Construction of building 1986-1987
- Design defect found in 1990
  - Building could separate from North Core in an Eq.
- Remedial work using Drag Bars designed in 1991
  - Steel angles epoxy bolted into walls and underside of slabs
  - No Drag Bars designed or installed at Level 2 or 3
  - Unable to sustain full design response of the structure
- No Building Consent application on Council files

Summary of Vulnerabilities



South Wall much weaker and more flexible than North Core

Smooth construction joints in South Wall

No seismic separation gaps between columns and Spandrel Panels

Some column concrete strength tests low

Seismic separation of masonry infill compromised

Beam reinforcing not connected into wall

North Core much stiffer and stronger than South Wall

Drag Bars had limited strength

Beam –column joints fragile

Column spiral reinforcing insufficient

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Collapse Scenarios

DEMAND

Components of demand on columns:

Gravity loads

Horizontal seismic loads and storey drifts

may have been influenced by:

- asymmetric shear walls
- masonry infill
- critical connections between floors and North Core

Vertical seismic loads

PROBABLE COLLAPSE SEQUENCE

East and/or south side column failure - at mid to high level

Possible floor diaphragm disconnection from North Core walls

Internal column failure - at mid to low level

Rapid progressive collapse

CAPACITY

Capacity of columns to support gravity loads and to withstand storey drifts may have been limited by:

Non ductile detailing of columns and beam-column joints

Insufficient gaps to Spandrel Panels

Low concrete strength

Vertical seismic loads

Capacity to resist progressive collapse may have been limited by:

Lack of redundant load paths and interconnection of structural elements for robustness

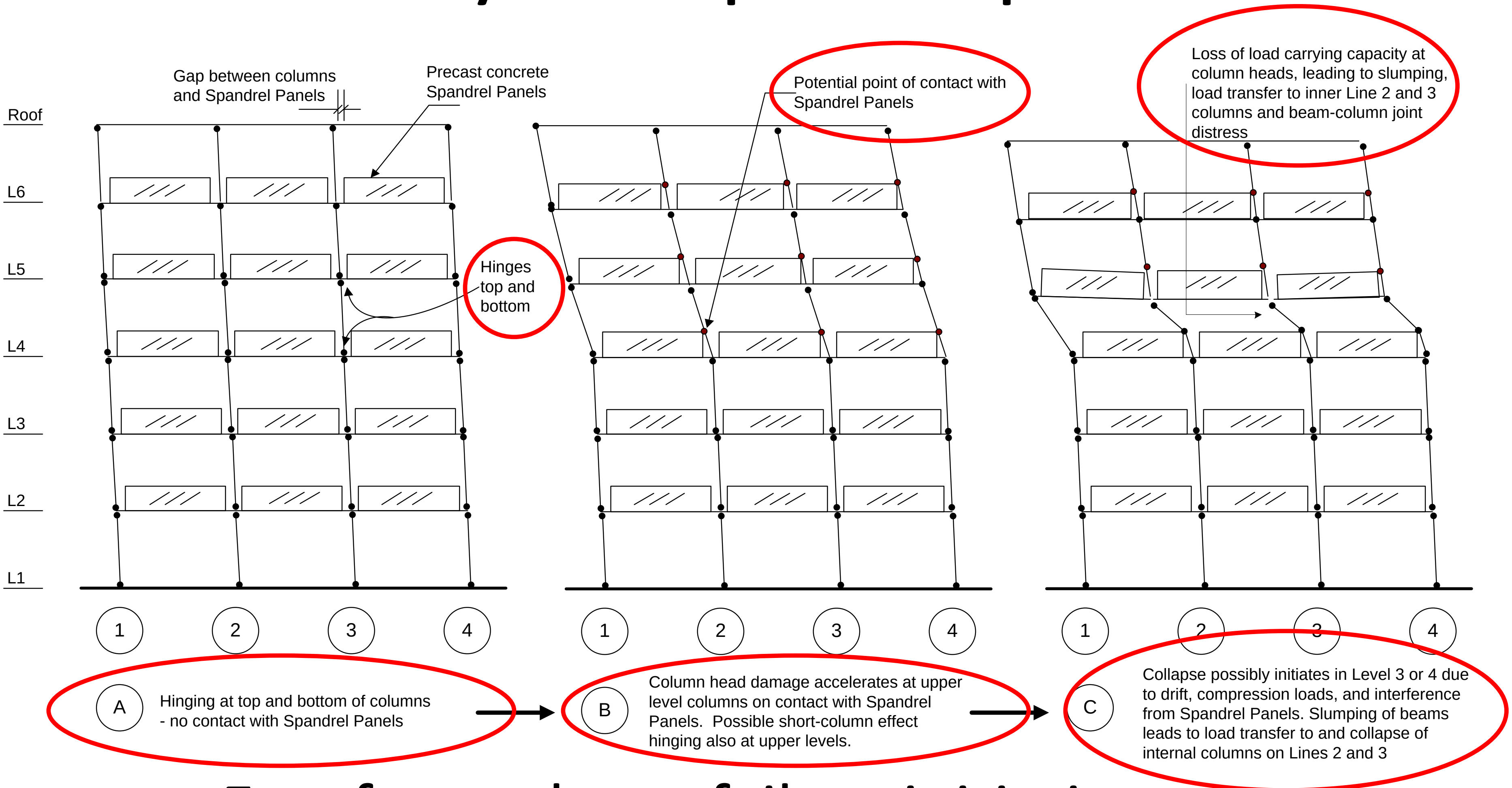
Demand = loads or displacements acting on structure

Capacity = ability of structure to resist loads or displacements

Collapse Initiation Scenarios

- Collapse Initiation Scenarios Examined
  1. East or South face column Failure on (Line F or 1)
  2. Internal column failure on Line 2 or 3
  3. Internal column failure following floor slab diaphragm disconnection at North Core
     - No Drag Bars at these levels
  4. Column failure following floor slab disconnection at North Core at Levels 4, 5 or 6
- Scenario 1 preferred
  – (Refer p.103 to107)

Likely Collapse Sequence



- East face column failure initiation

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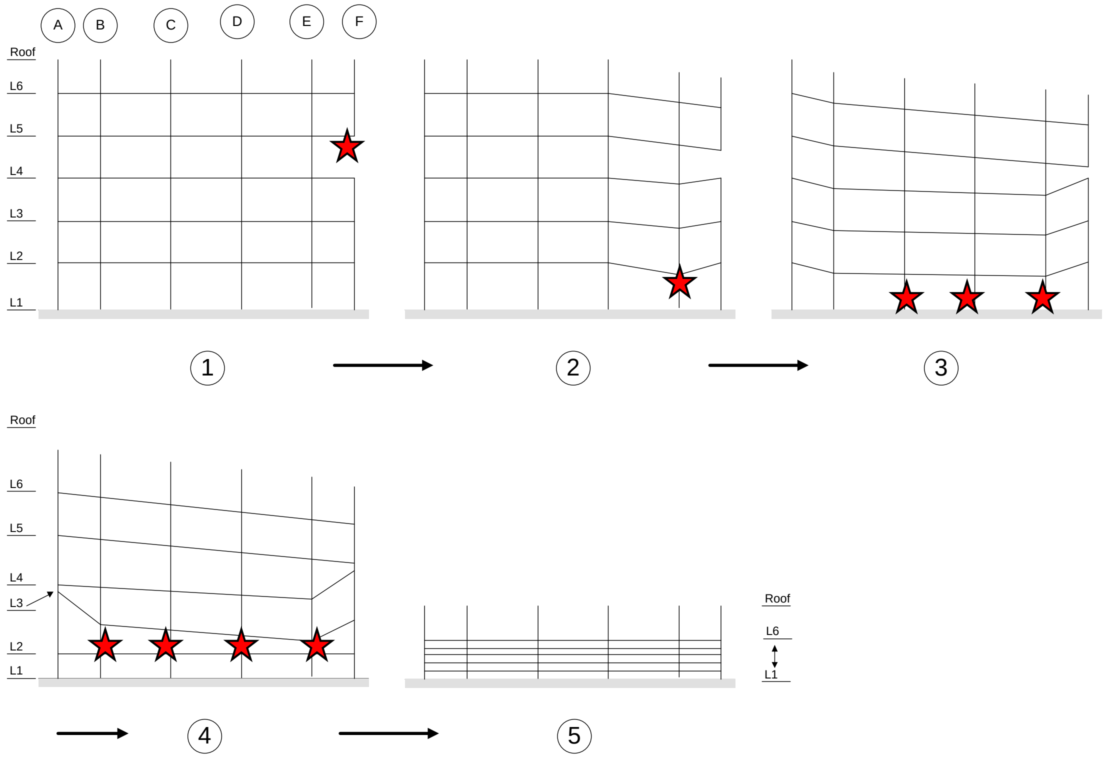
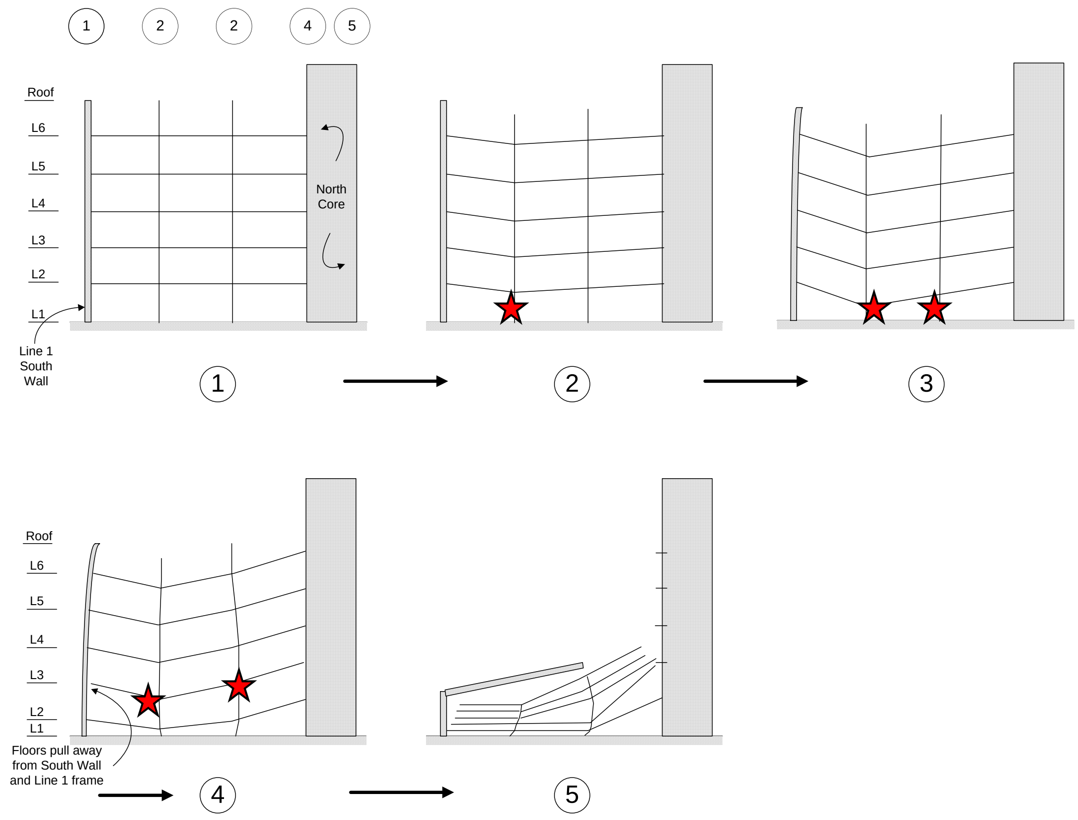
Likely Collapse Sequence



- Line 2 (east to west) column failure development

Likely Collapse Sequence



Line D Collapse Sequence

- Line C to D (south to north) column failure development

Why Did the CTV Building Collapse?

- Specific factors that contributed (or may have contributed) to the collapse include:
  - Severe earthquake aftershock
  - Column drift capacity substandard
  - Seismic gaps between columns and Spandrel Panels substandard
  - Some column concrete test strengths substandard
  - Unsymmetrical layout and large strength differential between South Wall and North Core
  - Seismic separation of masonry infill on west wall compromised
  - Substandard construction joints in South Wall

Likely or Possible Contributors to the Collapse of the CTV Building

(From p.31 of the report)

- The stronger than design-level ground shaking.
- The low displacement-drift capacity of the columns due to:
  - The low amounts of spiral reinforcing in the columns which resulted in sudden failure once concrete strain limits were reached.
  - The large proportion of cover concrete, which would have substantially reduced the capacity of columns after crushing and spalling.
  - Significantly lower than expected concrete strength in some of the critical columns.
  - The effects of vertical earthquake accelerations, probably increasing the axial load demand on the columns and reducing their capacity to sustain drift.
- The lack of sufficient separation between the perimeter columns and the Spandrel Panels which may have reduced the capacity of the columns to sustain the lateral building displacements.
- The plan irregularity of the earthquake-resisting elements which further increased the inter-storey drifts on the east and south faces.
- Increased displacement demands due to diaphragm (slab) separation from the North Core.
- The plan and vertical irregularity produced by the influence of the masonry walls on the west face up to Level 4 which further amplified the torsional response and displacement demand.
- The limited robustness (tying together of the building) and redundancy (alternative load path) which meant that the collapse was rapid and extensive.

Summary of Findings

BUI.MAD249.0504.86

- The earthquake aftershock was severe but the building appears to have collapsed at inter-storey drifts less than those expected by the Standards
- A number of collapse scenarios were considered. Collapse most likely initiated in substandard concrete columns along the east face of the building at Levels 3, 4 or 5.
- Columns designed in accordance with the standards would have been expected to be safe at drifts of 1.51%.
- The columns along the North and East faces of the CTV Building at Levels 2 to 4 were estimated to have drift capacities between 1.15 and 1.45%
- It appears that these East face columns may have failed at drifts of less than 1.0% prior to Drag Bar failure at the North Core

Summary of Findings

BUI.MAD249.0504.87

- Specific factors that contributed (or may have contributed) to the columns failures include:
  - Columns did not have the amount of spiral confining and shear reinforcing steel required by the design standard.
  - There was no specific seismic gaps between the Spandrel Panels and the Columns
  - The South Wall may have begun to yield and lose stiffness at drifts as low as of 0.40% due to structural asymmetry
  - Vertical accelerations may have reduced column drift capacity
  - Substandard construction joints in the South Wall may have slipped and increased inter-storey drifts..
  - The concrete in some of the columns had test strengths less than the minimum strength specified.
  - Seismic separation gaps between the Infill masonry on the west face and the structure appear to have been compromised and may have changed the response of the structure.

Summary of Findings

BUI.MAD249.0504.88

- Critical connections of the floors to some of the North Core walls were omitted in the original design and were only identified during a pre-purchase structural review 3 years after construction.
  - The Council did not have any record of the remedial works that were subsequently undertaken.
  - The Drag Bars installed could not sustain the ultimate design response of the structure.
- Most of the substandard design could be identified by a normal peer review
- Most of the substandard construction could be identified by normal inspection procedures.
- The building did not appear to have suffered significant structural damage in the 4 September 2011 Earthquake or 26 December 2010 Aftershock.
- The presentation is based on the findings of the CTV Building Collapse Investigation Report by Hyland Fatigue + Earthquake Engineering and StructureSmith Ltd and the Site Examination and Materials Testing Report by Hyland for the Department of Building and Housing
- The scope of the investigation was limited to identifying technical reasons for the collapse.