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Seismic assessment of existing buildings – core sections of the new Part C

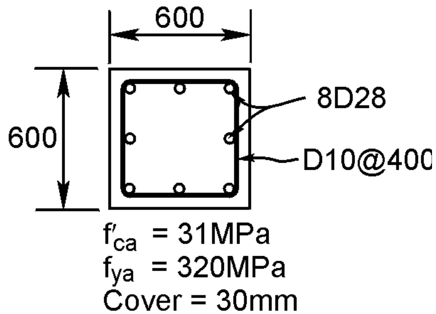
February 2017

Concrete Buildings - Section C5

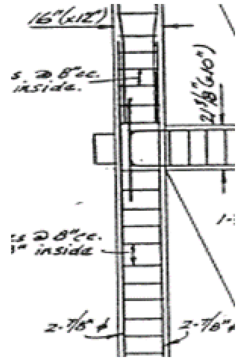
Presented by: Kam Weng Yuen

Key Features

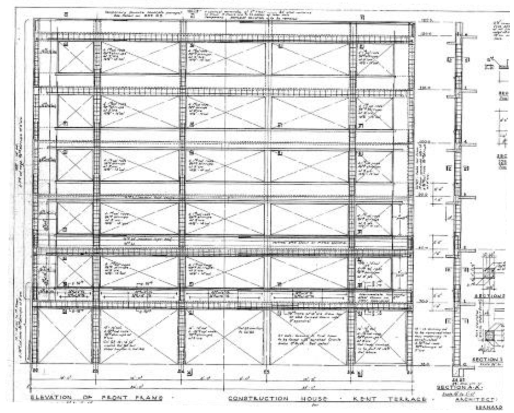
- **Improvement of General Assessment Framework – Emphasis on Analytical Procedure**
  - from Local to Global



Member



Subassembly



System

- **Introduction of Hierarchy of Strength Evaluation**
- **Schematic of Typical Deficiencies/Vulnerabilities**
- **Narrative/Tables on NZ Practice/Code Developments**

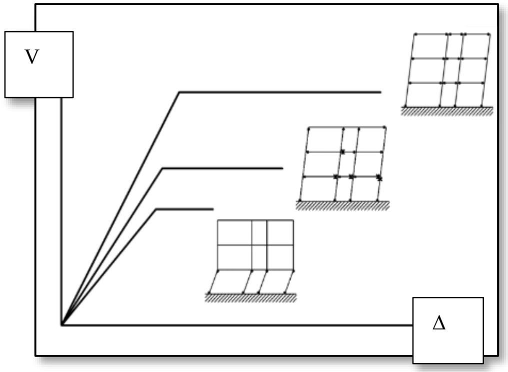
  - Material Properties (Steel and Concrete)

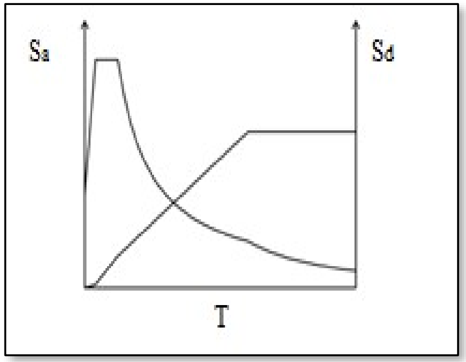
  - Structural Details (beams, columns, joints, walls)

Flow-Chart of Assessment Procedure

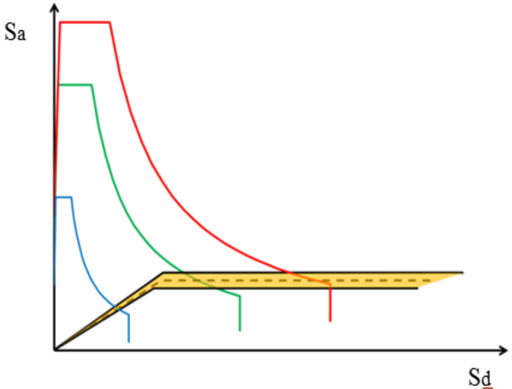
Step 1- CAPACITY

Step 2- DEMAND





Step 3 – CAPACITY vs. DEMAND (Behaviour)

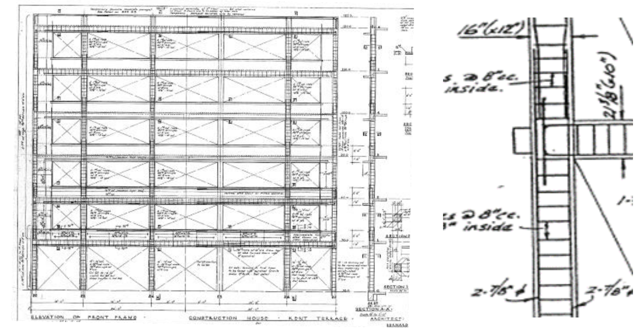


Chapter 5
Concrete Buildings

Leader: Stefano Pampanin

Step 1- CAPACITY

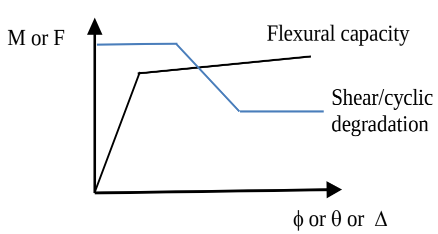
Building data:
- Geometry
- Material properties
- Structural details



1a- Component Level (beam, column, joint)

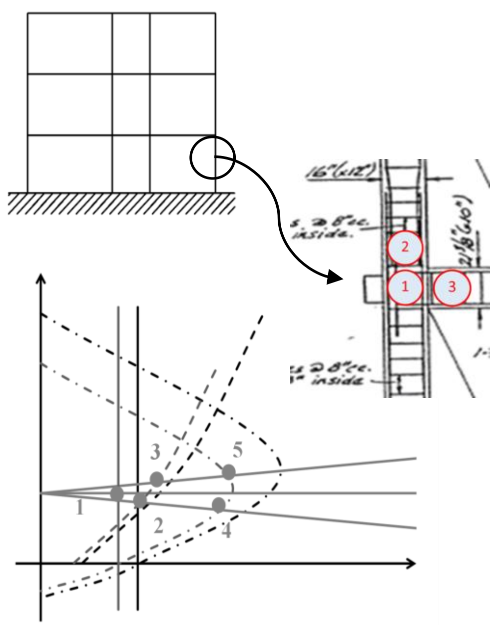
Evaluate strength and deformation capacity:
-Flexure, Shear, Flexure-shear interaction
- Cyclic degradation; Lap splices failure; Bi-directional effects

Outcomes (capacity curves):
Moment-curvature/rotation and/or Force-Displacement



1b- Subassembly Level

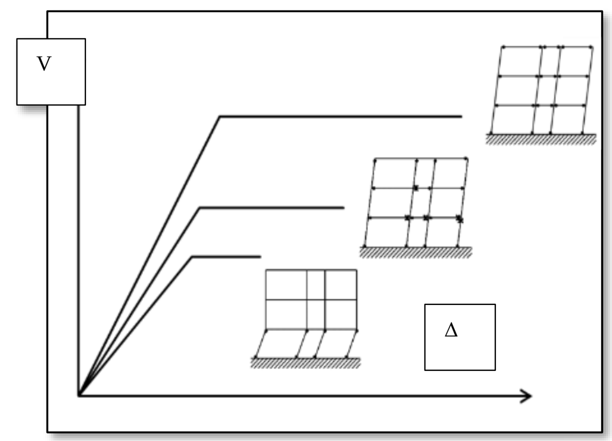
Evaluate the Hierarchy of Strength and sequence of events at a subassembly level



1c - Structural System Level

Identify the global mechanism

Evaluate the Global Capacity Curve (Force-Displacement)



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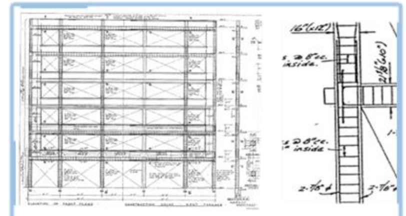

Building data:
- Geometry
- Material properties
- Structural details

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Knowing and understanding our Special Patients

Christchurch, built 1950s

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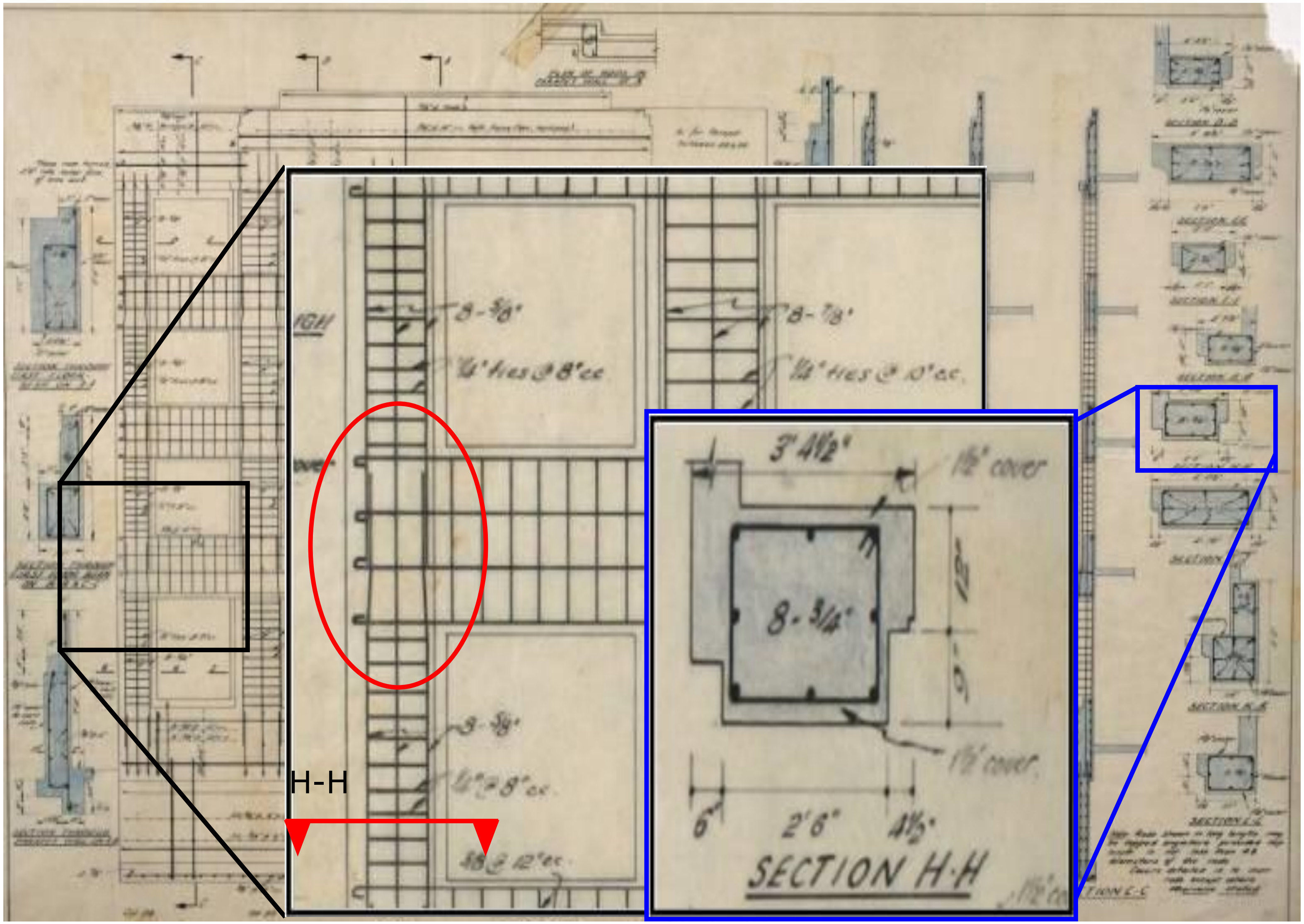
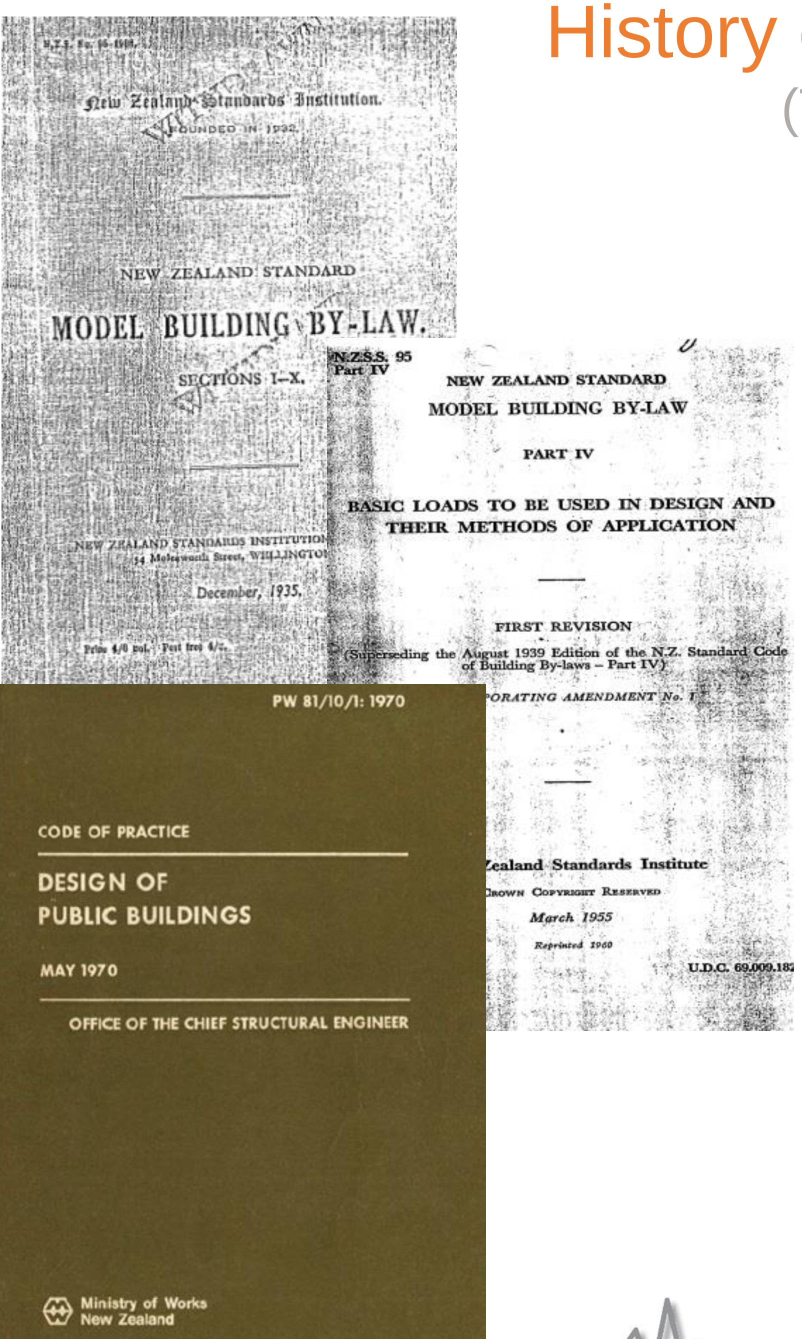


Christchurch, built 1950s

History of NZ Design Standards

(Text and Appendix C5A)



Appendix C5A –

Evolution of NZ Concrete Design Standard

Period	Loading Standard	Concrete Standard	Major changes
Pre-1957	1935 Model Bylaws	No seismic provisions	While there were no specific seismic requirements, 135 degree hooks were already shown for stirrups in RC construction (clause 409). Maximum spacing of stirrups was 2/3 of the internal lever arm (clause 616). Development of plain round longitudinal bars was often by 180 degree hooks.
1957-1964	NZSS 95 - Pt IV Basic Loads to be used and methods of application (1955)	UK concrete Code of Practice, CP114:1957 (No seismic provisions) and NZSS 95, Pt V (1939)	Section properties of members were permitted to be based on gross sections, transformed un-cracked sections, or transformed cracked sections (Fenwick and MacRae, 2009).
1964-1968/71	NZSS 1900 Basic Design Loads Chapter 8 (1964)	Design and Construction, Concrete, Chapter 9.3, 1964 (No seismic provisions)	Essentially, no seismic details were specified. It is likely that reinforcement was inadequately anchored for seismic actions, particularly in columns. Plain round bars were used extensively during this period.
1968/71-1982	Ministry of Works Code of Practice: 1968	Ministry of Works Code of Practice: 1968	Ultimate Limit State (ULS)/Limit State Design (LSD) recommended. Detailing requirements introduced for (i) beam-column joints; (ii) column confinement. **Capacity design** introduced between beams and columns (though no allowance for beam over-strength due to slab reinforcement contribution).
	NZ4203:1976	ACI 318:1971 or provisional NZ Concrete Standard, NZS 3101:1970	Ultimate Strength Design used Strength Reduction Factors of 0.9 for beams, 0.75 for confined columns and 0.7 for unconfined columns. Member stiffness for seismic analysis recommended as 75% gross section stiffness.

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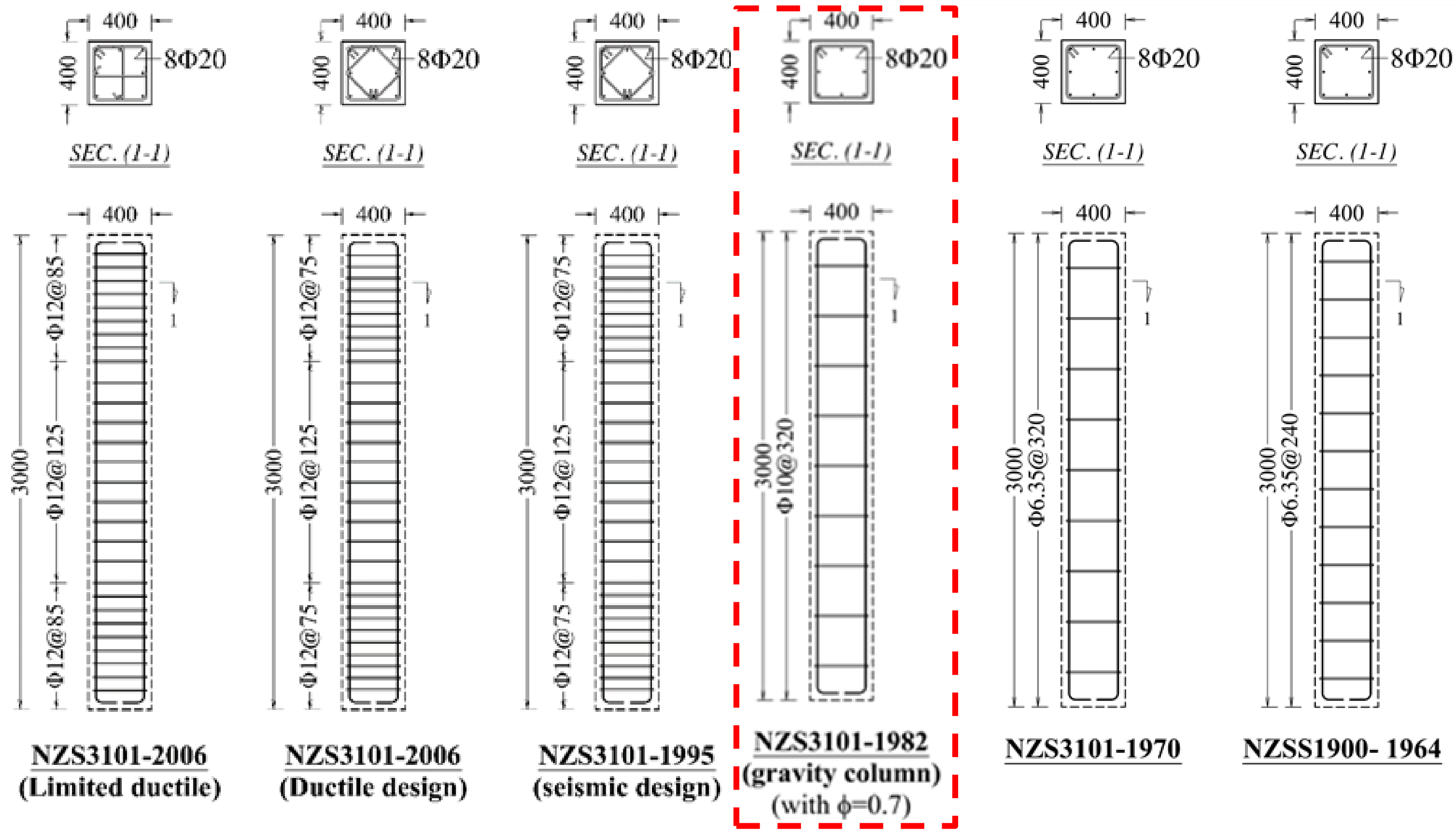
Example of typical column layouts with seismic design according to different New Zealand concrete standards from the mid-1960s onwards - Figure C5.16 (from Niroomandi et al., 2015)



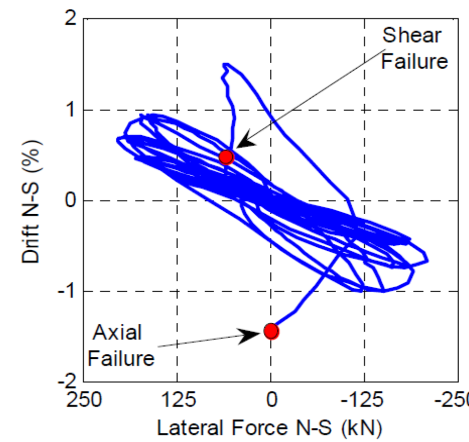
CAREFUL with pre-1995 and post-1982 non-Ductile Columns!

‘Non-Ductile’ Gravity Columns

Damage observed after the
4 Sept 2010 Canterbury earthquake

Figure C5.2: Experimental tests on ‘1982’ detailing (Boys et al., 2008)

- Limited Rotation and Drift Capacity (1-1.5%)
- Significant Effects of Bi-directional Loading



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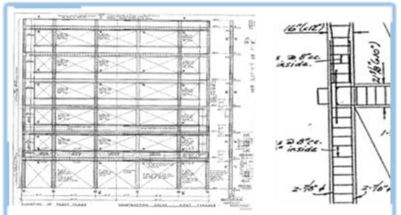

Building data:
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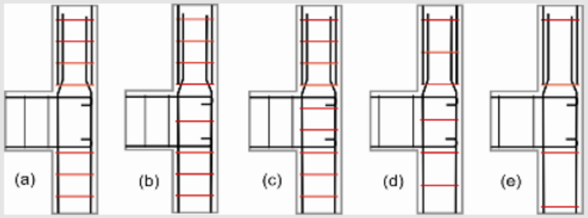
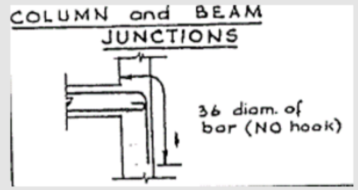
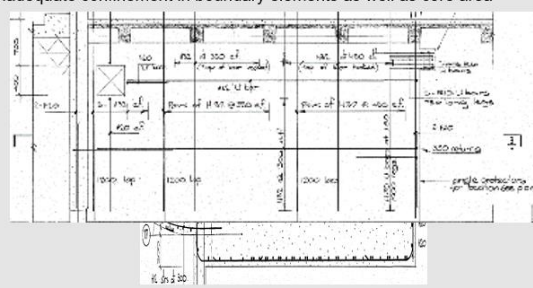
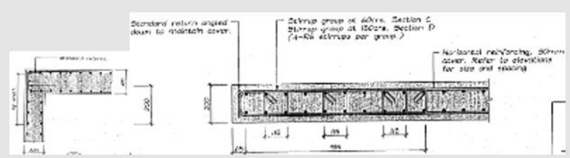
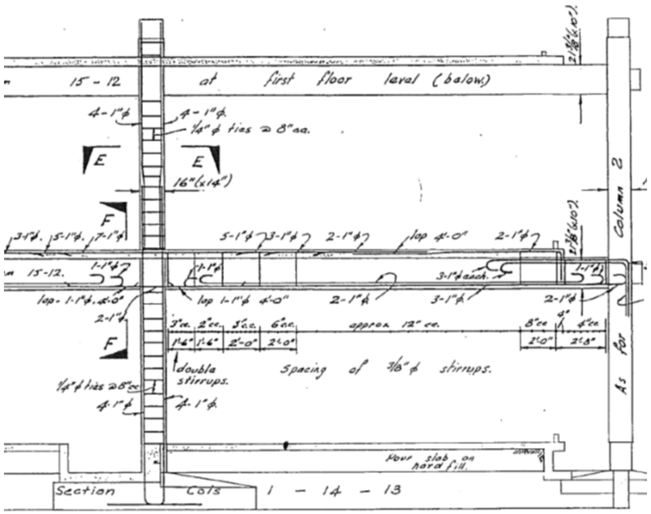
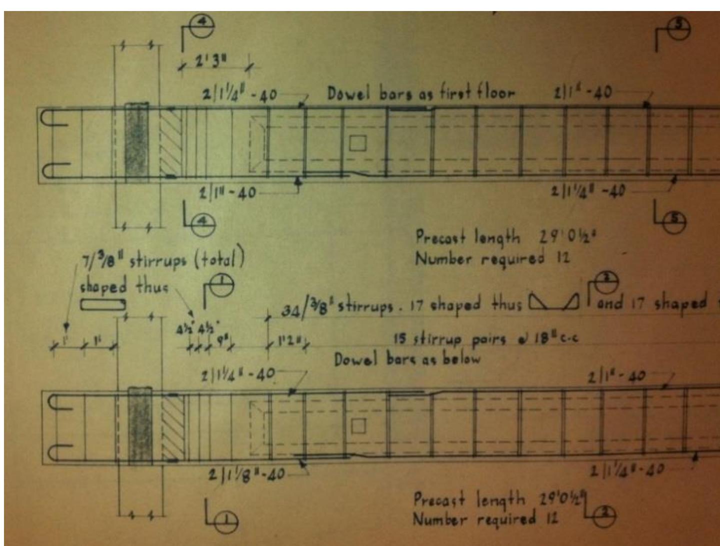
Tables C5.1-C5.2: Typical/expected structural deficiencies and observed damage/failure mechanism in pre mid-1970s Canterbury RC buildings

Component or global structure	Typical deficiency	Observed damage
Beam-column joints	Lack or total absence of horizontal and/or vertical transverse reinforcement in the joint panel zone.  Figure: Schematic illustrations of joint traverse reinforcement in pre-1970s buildings related to column stirrups and design assumptions: (a)-(b) Joint neglected in design or considered as a construction joint (c)-(d)-(e) Joints treated as part of column, therefore quantity of joint stirrups depended on column stirrup spacing and beam depth	Shear damage/failure in joint area with potential loss of gravity load bearing capacity in column  
Walls	Inadequate confinement in boundary elements as well as core area  Figure: Structural drawings of wall reinforcement and confinement details  Figure: Structural drawings of confinement details at wall corner and boundary element	Crushing, spalling of concrete; bar buckling; out-of-plane failure  Photos: Spalling of concrete at wall end, and buckling failure  Photos: Shear failure at ground floor wall

Typical Structural Deficiencies of RC Buildings





- inadequate transverse reinforcement and detailing in plastic hinge regions, beam-column joints and wall systems
- insufficient lap splices in columns, beams and walls
- insufficient longitudinal reinforcement ratio in walls leading to single crack opening, when compared to a spread plastic hinge, and failure in tension of the rebars
- lower quality of materials (concrete and steel) when compared to current practice, in particular:
  - use of low grade plain round (smooth) bars for both longitudinal (until mid-1960) and transverse reinforcement
  - low-strength concrete (up to or below 20-25 MPa)
- lack of or inadequate capacity design considerations
- lack of displacement compatibility between the lateral load resisting systems, the floor-diaphragms and the gravity load bearing systems (e.g. non-ductile columns with limited confinement details and drift capacity)
- plan and vertical irregularity
- potential brittle failure mechanisms due to interaction with secondary elements, i.e. spandrel beams, masonry infills, facades causing

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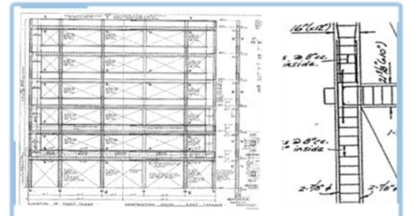

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Material **Properties** and Testing

- In the Guidelines **reference default values** for the mechanical properties of the reinforcing steel based on standard of the time are provided

- **Appendix C5C** - Overview of the evolution of concrete and steel reinforcing **material properties specifications and design requirements** in New Zealand Standards

<table>
<tr><th>1949</th><th>1962</th><th>1964</th><th>1968</th><th>1972</th><th>1973</th><th>1989</th><th>2001</th></tr>
<tr><td colspan="4">NZS 197:1949 (BS 785:1938)
Rolled steel bars and drawn steel wire for concrete reinforcement (Yield stress varied with diameter, minimum value was 227 MPa, Refer to Appendix C5D.1)</td><td>NZS 3423P:1972
Hot rolled plain round steel bars of structural grade for reinforced concrete
"Grade" 40,000 psi (275 MPa)</td><td rowspan="3">NZS 340 2P: 1973
Hot rolled steel bars for the reinforcement of concrete
Grade 275 MPa
Grade 380 MPa</td><td rowspan="3">NZS 3402: 1989
Steel bars for the reinforce-ment of concrete
Grade 300 MPa
Grade 430 MPa</td><td rowspan="3">AS/NZS 4671:2001
Steel reinforcing material
Grade 300 MPa
Grade 500 MPa</td></tr>
<tr><td></td><td colspan="2">NZSS 1693:1962
Deformed steel bars of structural grade for reinforced concrete
"Grade" 33000 psi (227 MPa)</td><td colspan="2">NZSS 1693:1962 (Amendment 1:1968)
Deformed steel bars of structural grade for reinforced concrete
"Grade" 40000 psi (275 MPa)</td></tr>
<tr><td></td><td></td><td colspan="3">NZS 1879:1964
Hot rolled deformed bars of HY 60 (High Yield 60,000 psi) for reinforced concrete
Grade" 60,000 psi (415 MPa)</td></tr>
</table>

Material Properties and **Testing**

- Material Testing should be targeted to elements within the most critical/uncertain mechanism,
  e.g. as part of hierarchy of strength evaluation.

- **Appendix C5C:**
  Overview of alternative techniques:
  destructive
  semi-destructive
  non-destructive

Table C5C.1: Overview of destructive, semi-destructive and non-destructive tests for investigating concrete material properties (from De Pra, Bianchi and Pampanin, 2015; Malek et al., 2015)

Method	Capability/Use	Advantages	Disadvantages
DESTRUCTIVE TESTS			
Compressive test	Strength of concrete	Direct evaluation of concrete strength from compressive tests on cylindrical specimens	Disturbance of the sample, so excessive damage to obtain a representative core of concrete Previous test with pacometer necessary to individuate the regions without bars
SEMI-DESTRUCTIVE TESTS			
Pull-out	In-place estimation of the compressive and tensile strengths	In-place strength of concrete can be quickly measured	Pull-out device must be inserted in a hole drilled in the hardened concrete Only a limited depth of material can be tested
Pull-off/tear-off	Direct tension test	In situ tensile strength of concrete Determining bond strength between existing concrete and repair material	Sensitivity to rate of loading
Penetration probe (Windsor probe)	Estimation of compressive strength, uniformity and quality of concrete Measuring the relative rate of strength development of concrete at early ages	The equipment is easy to use (not requiring surface preparation) The results are not subject to surface conditions and moisture content	Minimum edge distance and member thickness are requested Not precise prediction of strength for concrete older than 5 years and where surface is affected by carbonation or cracking
NON-DESTRUCTIVE TESTS			
Visual tests	The first step in investigating concrete material	Quick evaluation of damage	No detailed information
Rebound hammer	Measuring surface hardness of concrete to estimate compressive strength	The assessment of the surface layer strength	Results can only suggest the hardness of surface layer
Durability test: Concrete electrical resistivity	Measuring the ability of the concrete to conduct the corrosion current	Inexpensive, simple and many measurements can be made rapidly	Not reliable at high moisture contents
Durability test: Permeability	To evaluate the transfer properties of concrete (porosity)	Useful method to evaluate the risk of leaching, corrosion and freezing	Thickness limitation Age, temperature dependent Sufficient lateral sealing

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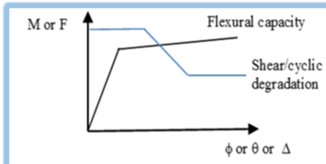

Evaluate strength and deformation capacity:
-Flexure, Shear, Flexure-shear interaction
- Cyclic degradation; Lap splices failure; Bi-directional effects

<u>Outcomes (capacity curves):</u>
Moment-curvature/rotation and/or Force-Displacement

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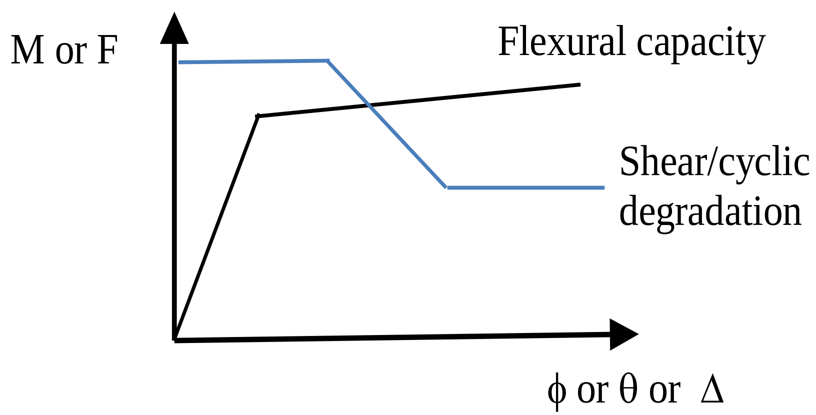
Step 1a – Element Capacities

Evaluate the Capacity Curves for each element:

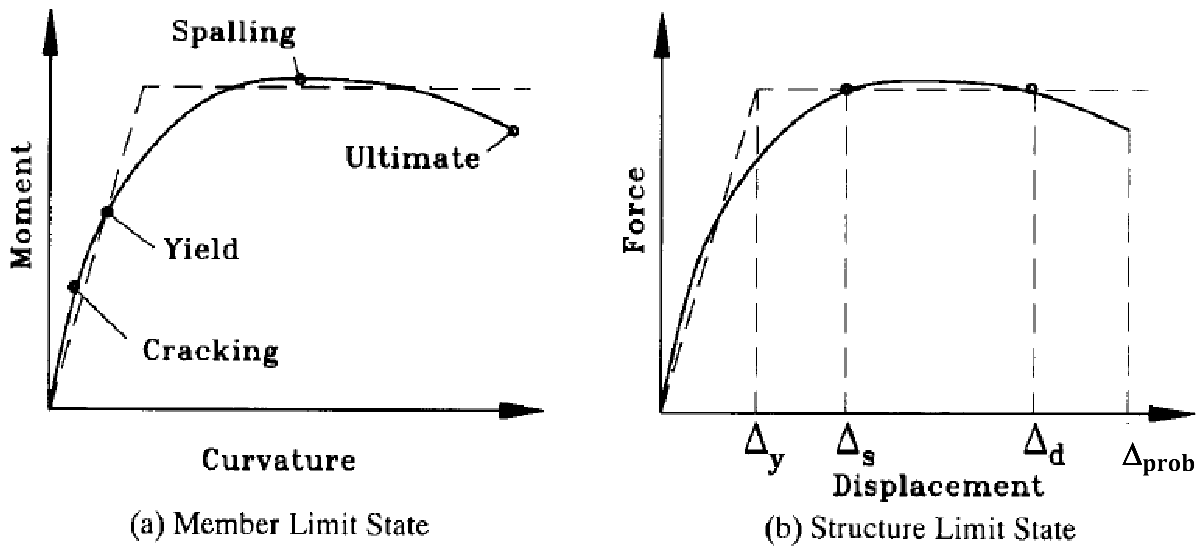
- moment-curvature/rotations and/or
- force-displacement (mostly for columns and walls)



Strength and deformation capacities at key limit states (i.e. yielding and ultimate capacity) for:

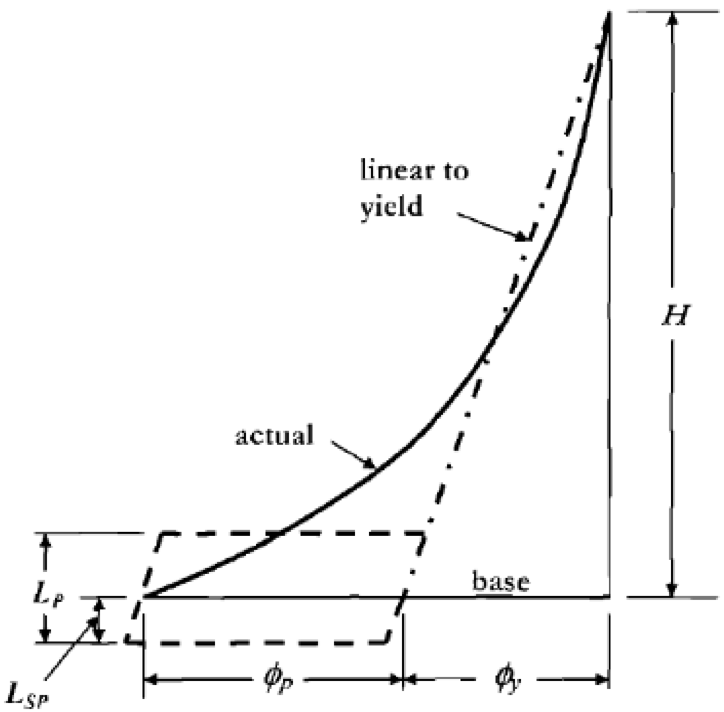
- Flexure and shear (including bar buckling, cyclic degradation, lap splices, bi-directional effects)

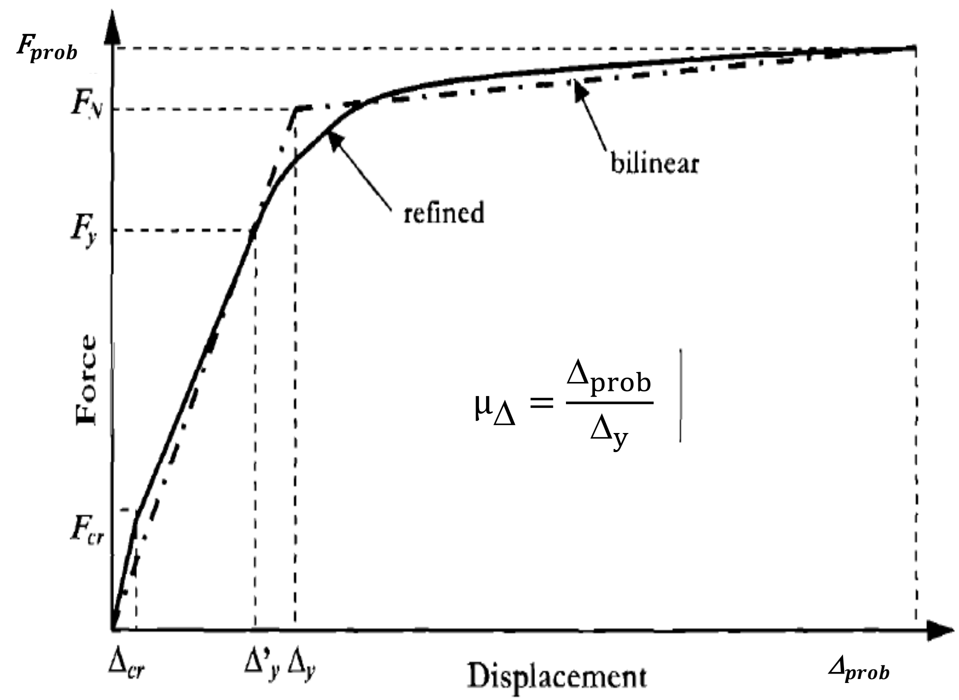
Strain Limits at Probable Capacity



Concrete	$\varepsilon_{cu} = 0.004 + \dfrac{1.4\rho_v f_{yh}\,\varepsilon_{su}}{f_{cc}} (\leq 0.015)$ (confined core)
Steel	$\varepsilon_s = 0.06\ (\leq 0.6\varepsilon_{su})$

From Moment-Curvature to Force- Displacement





$$\Delta_{\mathrm{prob}} = \Delta_{\mathrm{y}} + \Delta_{\mathrm{p}}$$

$$\Delta_{\mathrm{y}} = \phi_{\mathrm{y}} \frac{H^2}{3}$$

$$\Delta_{\mathrm{p}} = \phi_{\mathrm{p}} \boldsymbol{L_{\mathrm{p}}} H = (\phi_{\mathrm{prob}} - \phi_{\mathrm{p}}) \boldsymbol{L_{\mathrm{p}}} H$$

What plastic hinge length?

$$L_{\mathrm{p}} = kL_{\mathrm{c}} + L_{\mathrm{sp}} \geq 2L_{\mathrm{sp}}$$

(spread plastic hinge)

Shear Capacity - Beams

$$\boxed{V_p = 0.85(V_c + V_s)}$$

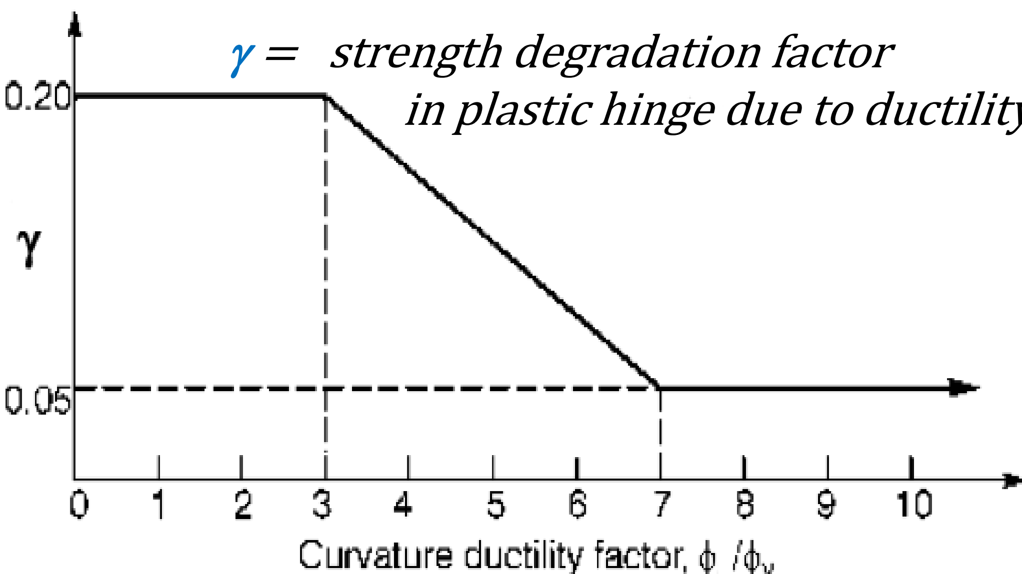
$$V_c = \alpha\beta\gamma\sqrt{f'_c}(0.8A_g)$$

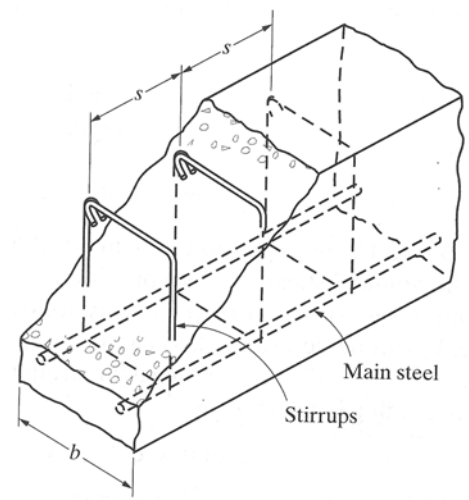
$$V_s = \frac{A_v f_{yt} d}{s}$$

$$1 \leq \alpha = 3 - \frac{M}{VD} \leq 1.5$$

$$\beta = 0.5 + 20\rho_l \leq 1$$

$\gamma =$ *strength degradation factor in plastic hinge due to ductility*



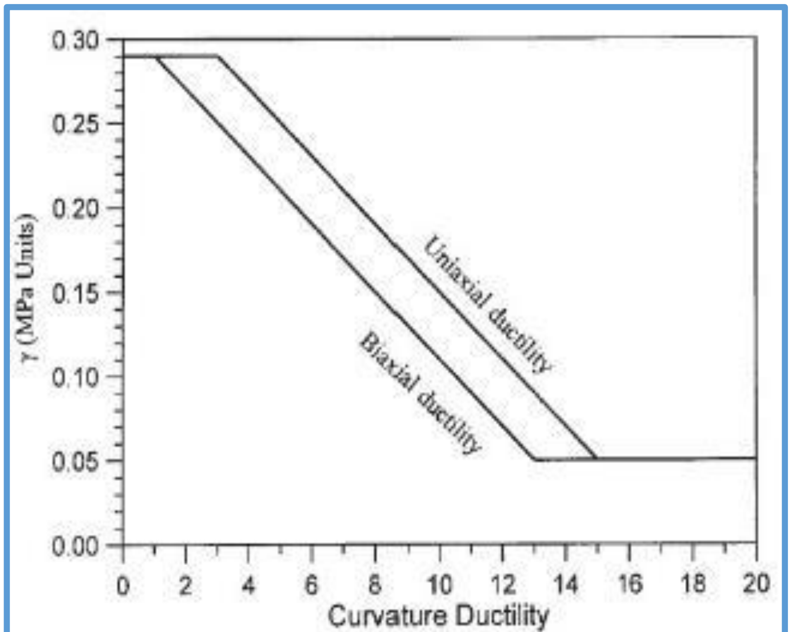


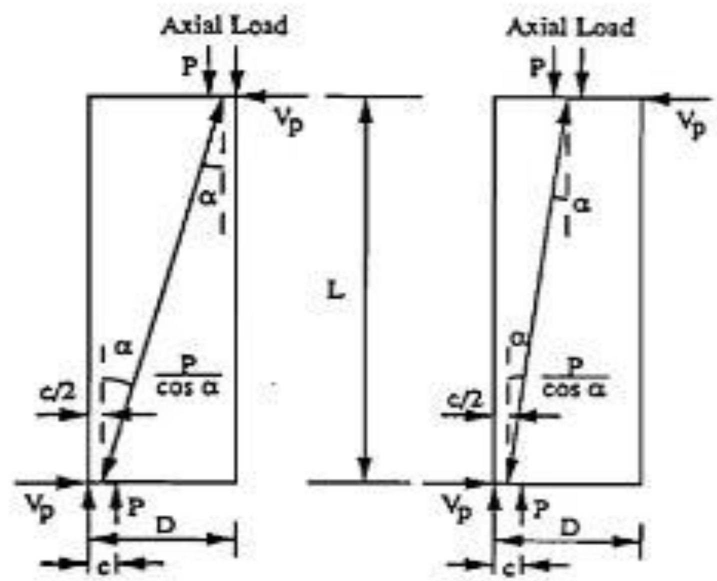
Shear Capacity – Columns

$$\boxed{V_p = 0.85\,(V_c + V_s + V_n)}$$

$\gamma =$ *shear strength degradation factor in the plastic hinge region due to ductility*

$$V_n = N^*\tan\alpha$$

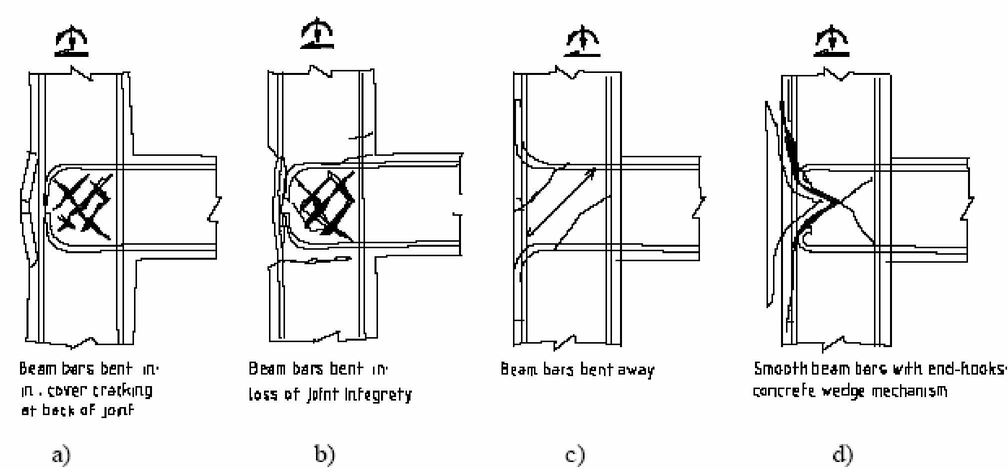




(a) Reversed Bending (b) Single Bending

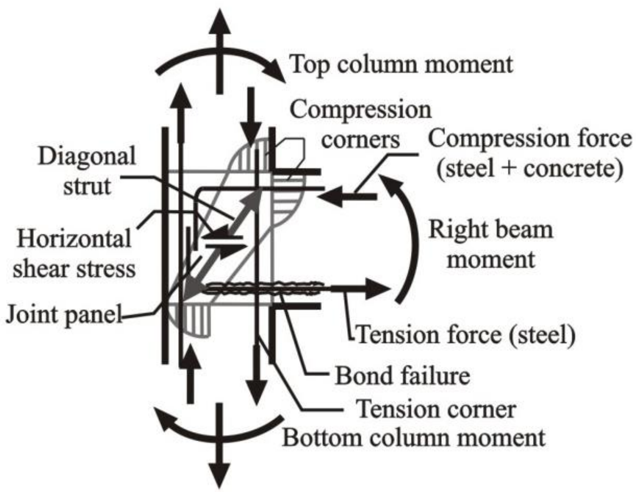
Beam-Column Joints Behaviour

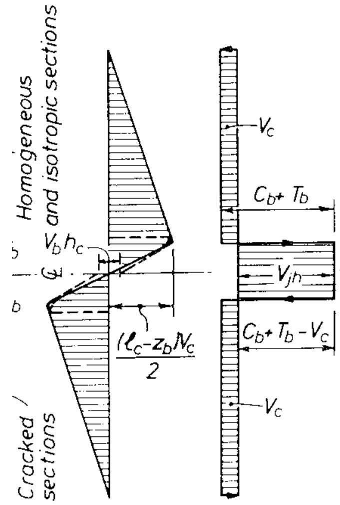
- Depends on the type of joints and detailing
- Affected by Variation of Axial load
- Affected by Bi-directional Loading



Shear Capacity – Joints (1/2)

$$\boxed{V_{\mathrm{pjh}} = 0.85\, v_{\mathrm{ch}} b_j h}$$





Nominal Shear Stress

$$V_{jh} = Ts - V_c$$

$$v_{jh} = \frac{V_{jh}}{b_{je} h_c} = v_{jv} = \frac{V_{jv}}{b_{je} h_b} = v_j$$

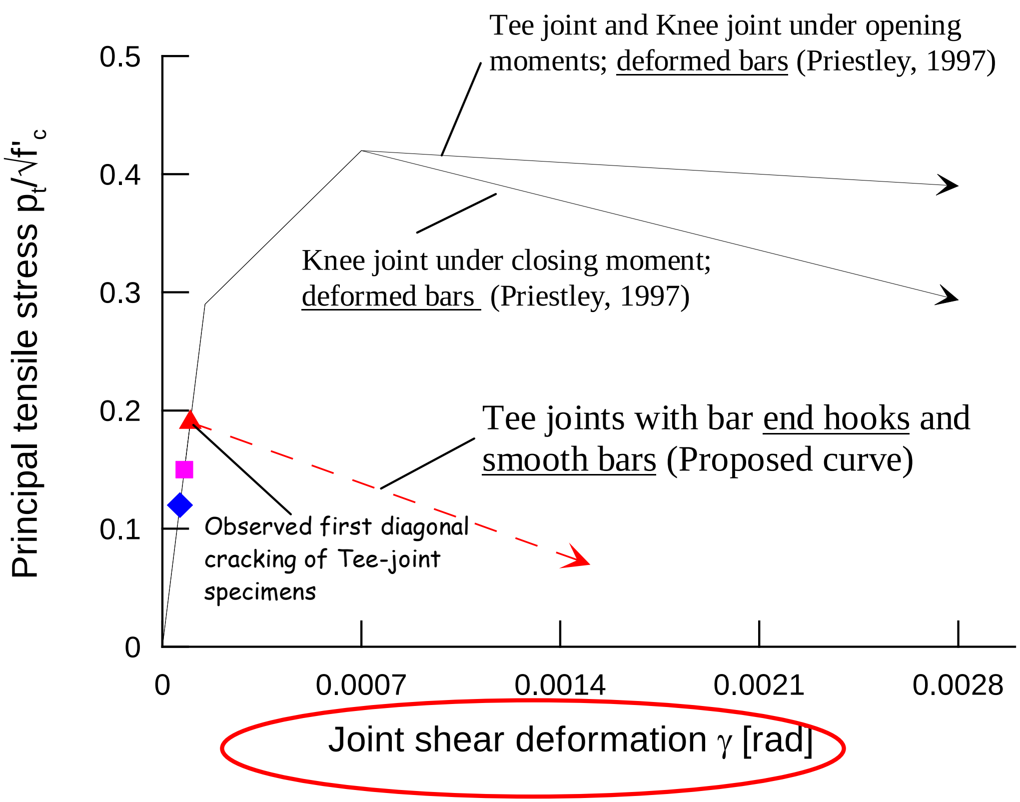
Principal Stresses

$$p_{t,c} = \frac{f_v + f_h}{2} \pm \sqrt{\left(\frac{f_v - f_h}{2}\right)^2 + v_j^{\,2}}$$

$$\boxed{v_{\mathrm{ch}} = v\mathrm{j} = p_t \sqrt{\left(1 + \frac{N^*}{A_g k \sqrt{f'_c}}\right)}}$$

An *Equivalent 'Moment-Curvature' for* the Joint

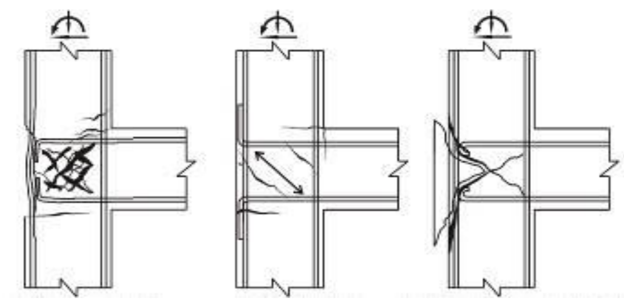
$$p_t = \boldsymbol{k}\sqrt{f'_c}$$



Interior Joints

- k = 0.8 for interior joints

Exterior Joints



- k = 0.4 deformed bars anchored into the joint core
- k = 0.3 deformed bars anchored away from the joint core
- k = 0.2 plain round bars with end hooks

Shear Capacity – Joints (2/2)

$$\boxed{V_{\mathrm{pjh}} = 0.85\, v_{\mathrm{ch}} b_j h}$$

$$p_{t,c} = -\frac{f_a}{2} \pm \sqrt{\left(\frac{f_a}{2}\right)^2 + v^2{}_j}$$

$$v_{ch} = k\sqrt{f'_c}\sqrt{\left(1 + \frac{N^*}{A_g k\sqrt{f'_c}}\right)}$$

$$\boxed{V_{\mathrm{pjh}} = 0.85k\sqrt{f'_{\mathrm{c}}}\sqrt{\left(1 + \frac{N^*}{A_{\mathrm{g}}k\sqrt{f'_{\mathrm{c}}}}\right)}\, b_{\mathrm{j}}h}$$

$$\leq 1.92\sqrt{f'_c}b_jh$$

Road Map

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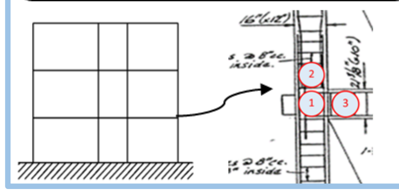
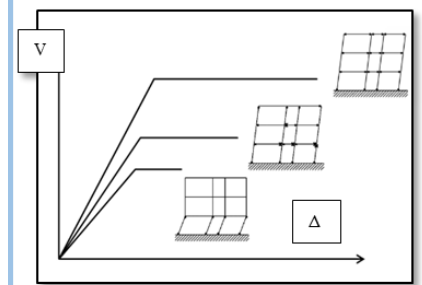
Evaluate the Hierarchy of Strength and sequence of events at a subassembly level



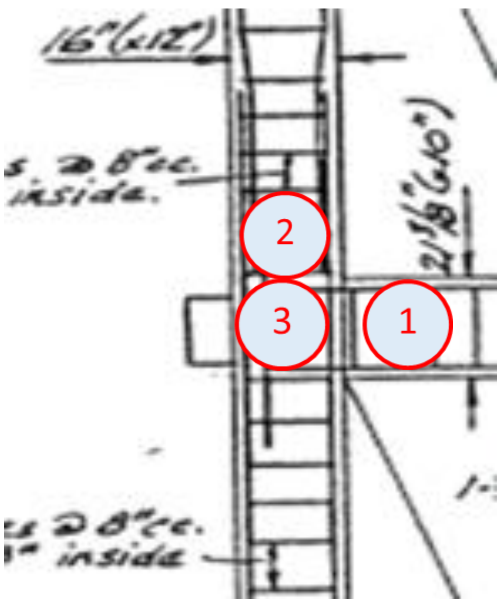
Identify the global mechanism Global Capacity Curve



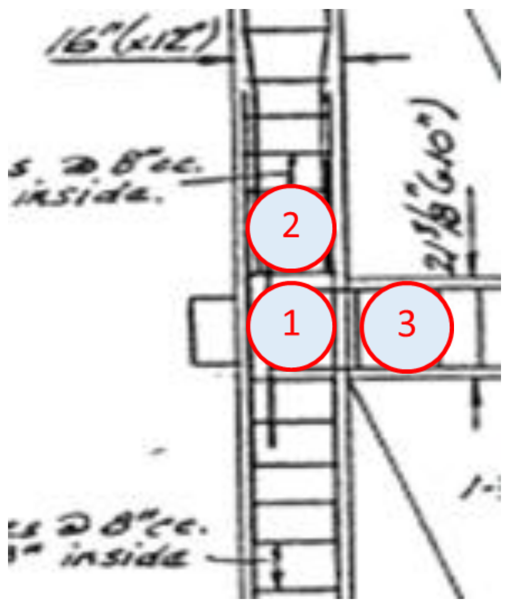
Step 1b - Hierarchy of Strength

- Evaluation of Hierarchy of Strength and Sequence of Events between connected elements
- Identification of Mechanism at a subassembly level.

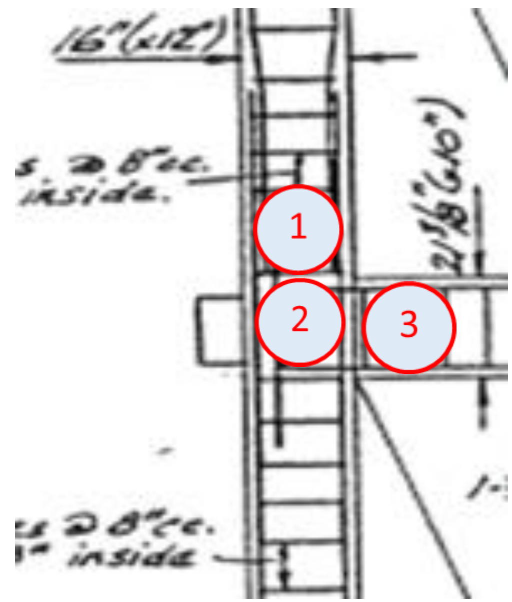
Beam hinging



Joint Shear Failure

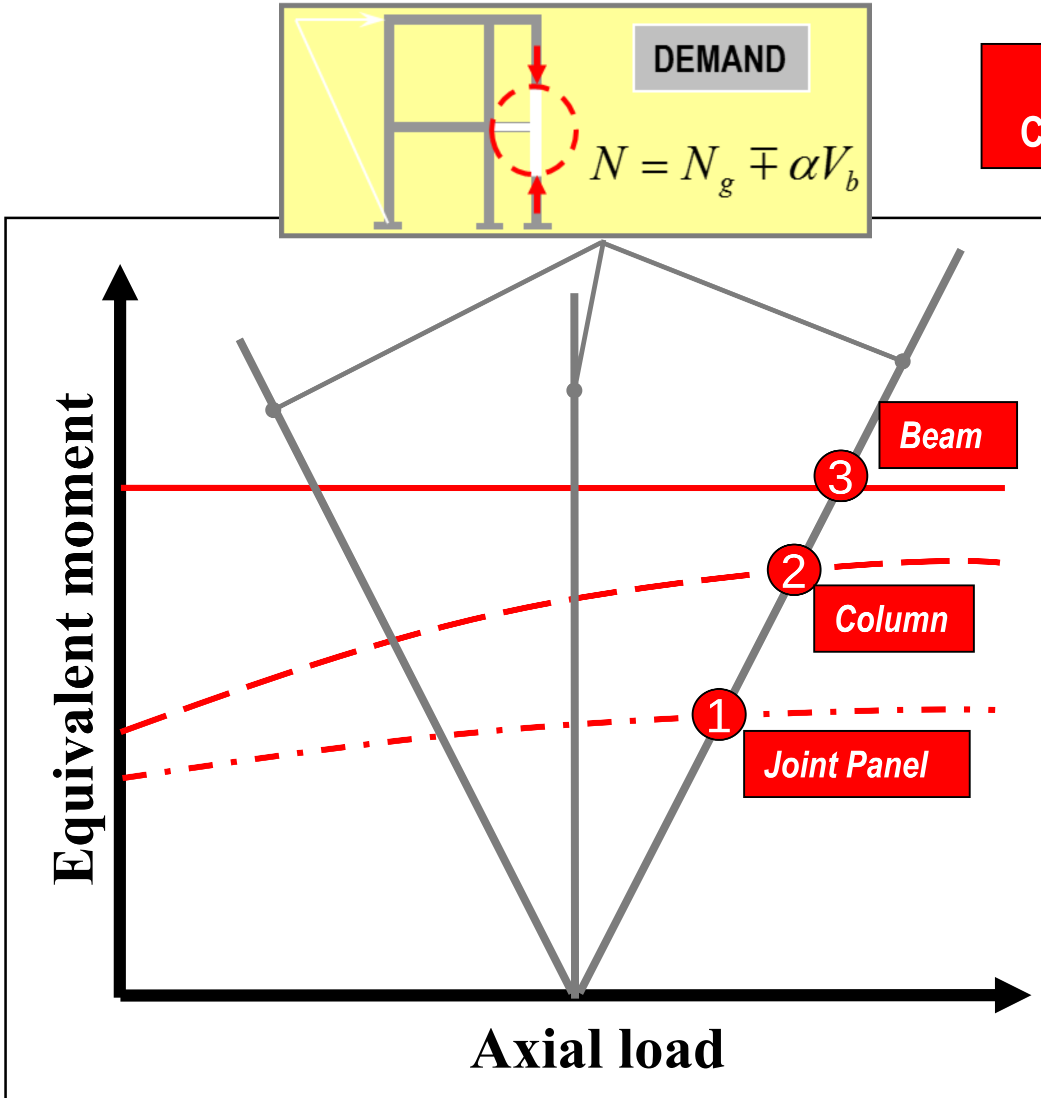


Column hinging

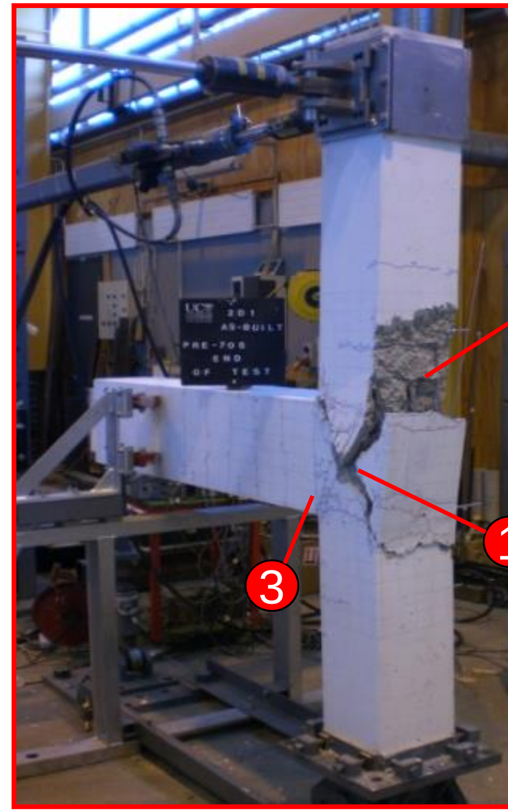


Evaluation of Hierarchy of Strength

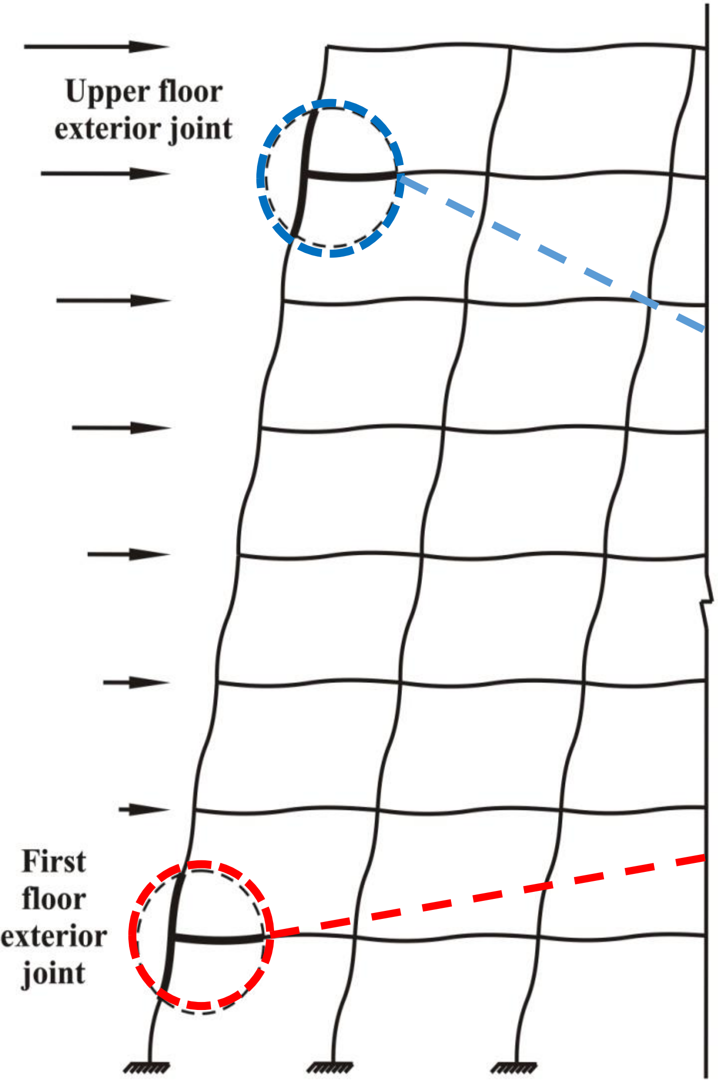
M-N Interaction Diagram



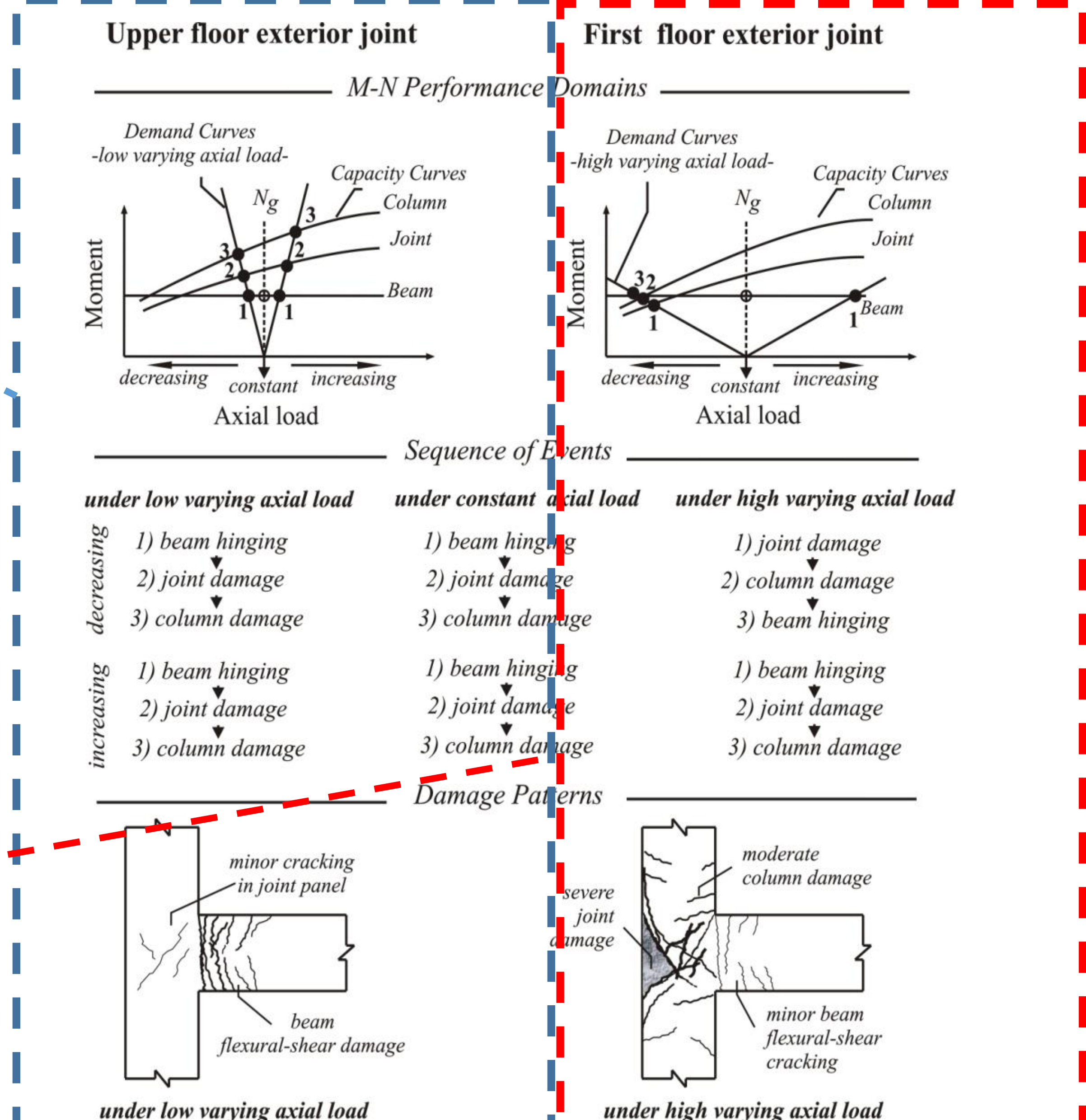
As-Built Joint Capacity Curves



From Agkuzel, 2011



Laterally loaded frame

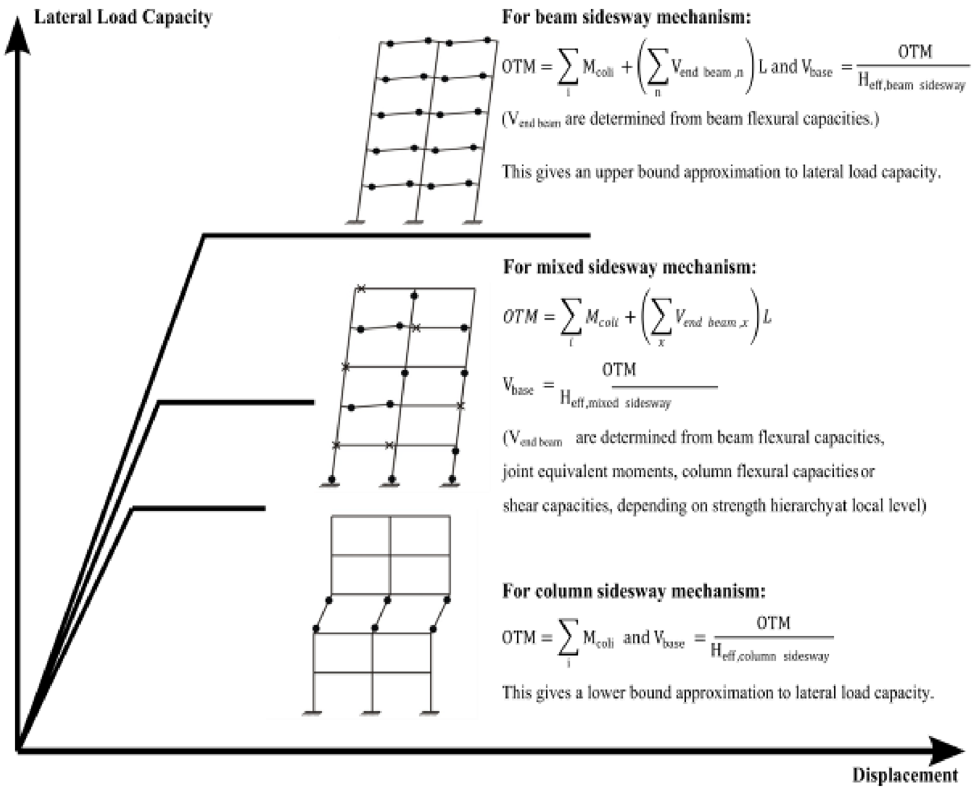


Hierarchy of strength and sequence of events for two types of exterior joints

Step 1c – Global Mechanism

The likely global or system level mechanisms are determined.

Global capacity curve (Lower, Upper bounds, Actual) are evaluated



Evaluation of %NBS

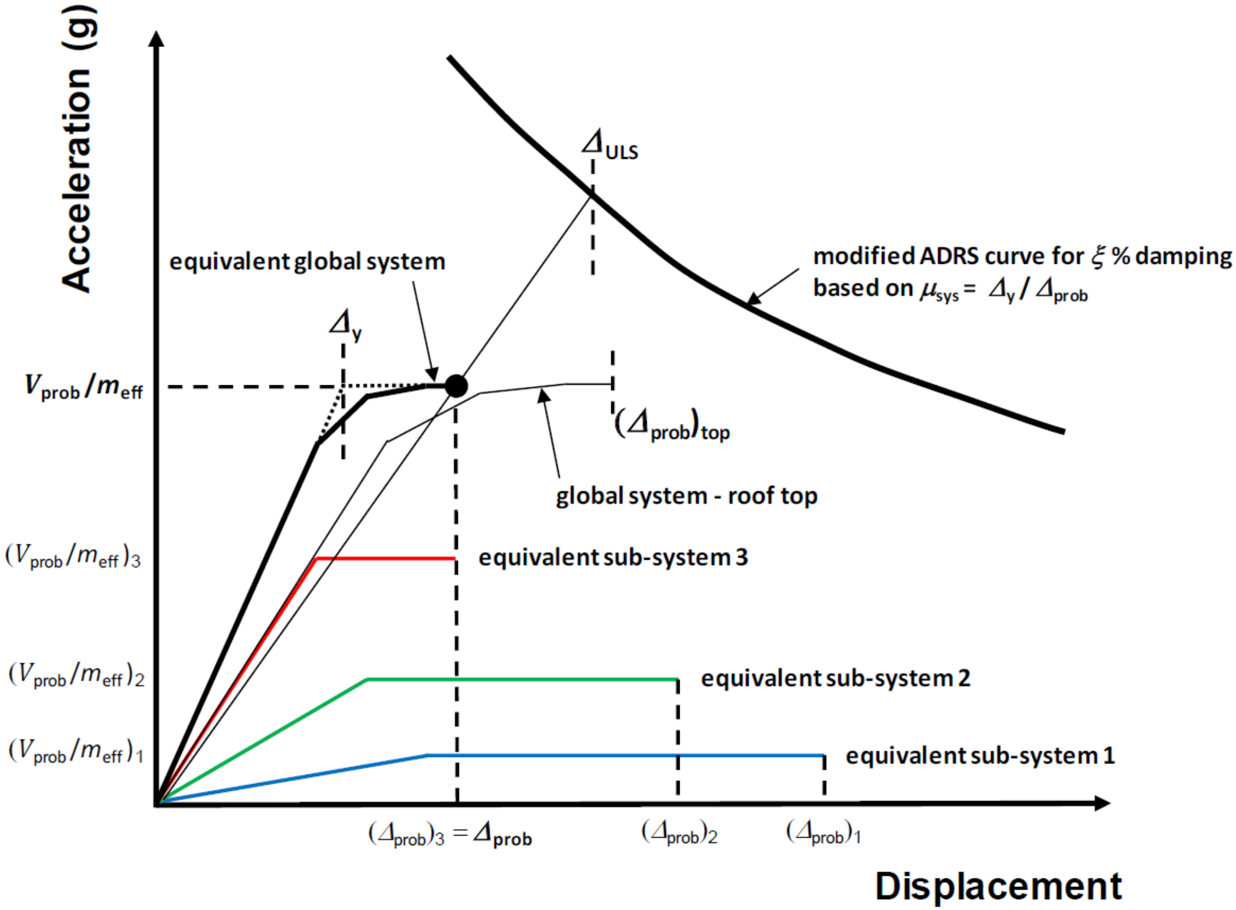
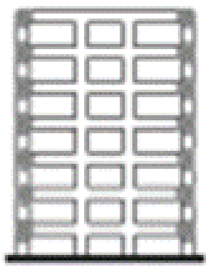
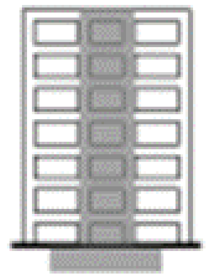


Figure C2.6: Derivation of seismic score using SLaMA

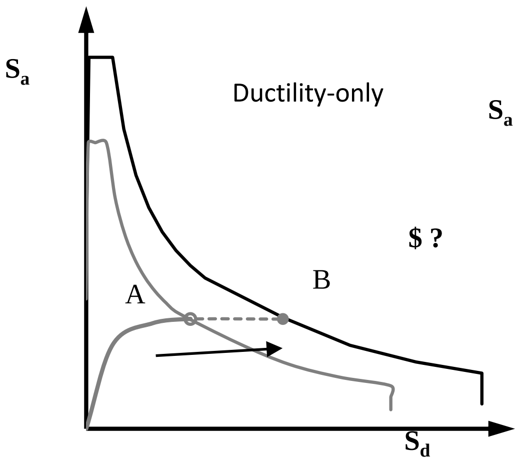
Comparing Alternative Retrofit Options

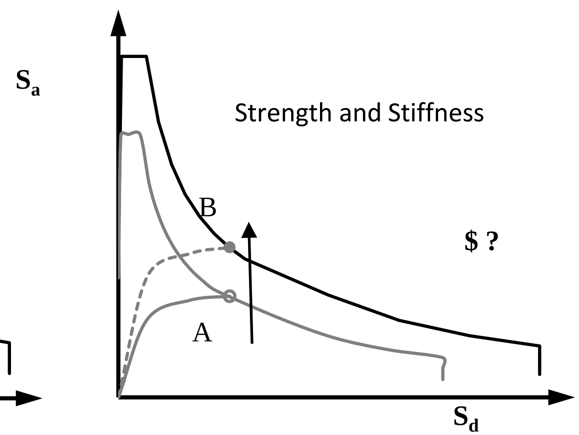


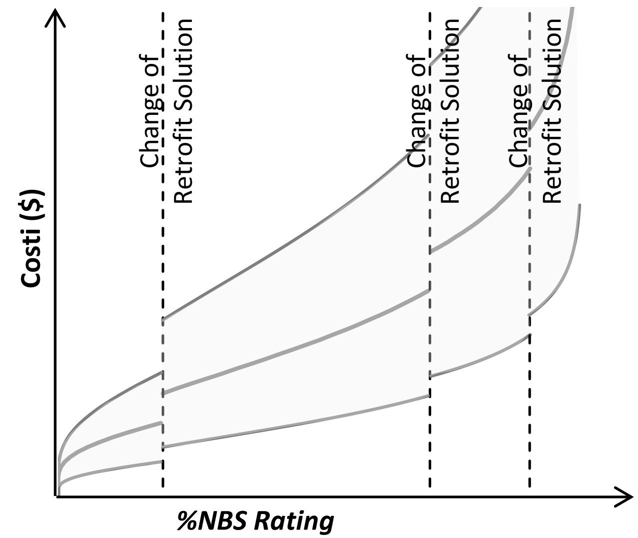
Option 2
(FRP)



Option 3
(Shear Wall)



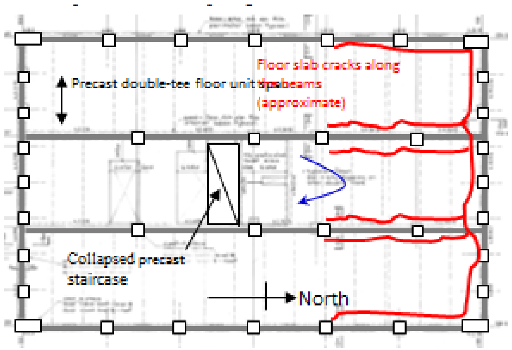




Pampanin et al.

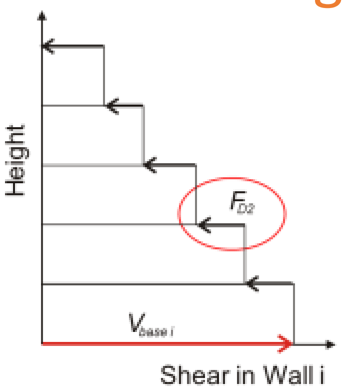
(Let us not forget about)

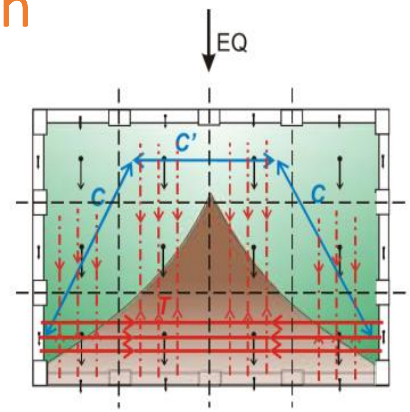
Diaphragms



Diaphragm Assessment

Strength





Demand Floor Forces and Internal Actions

(Appendix C5E.9- C5E.11)

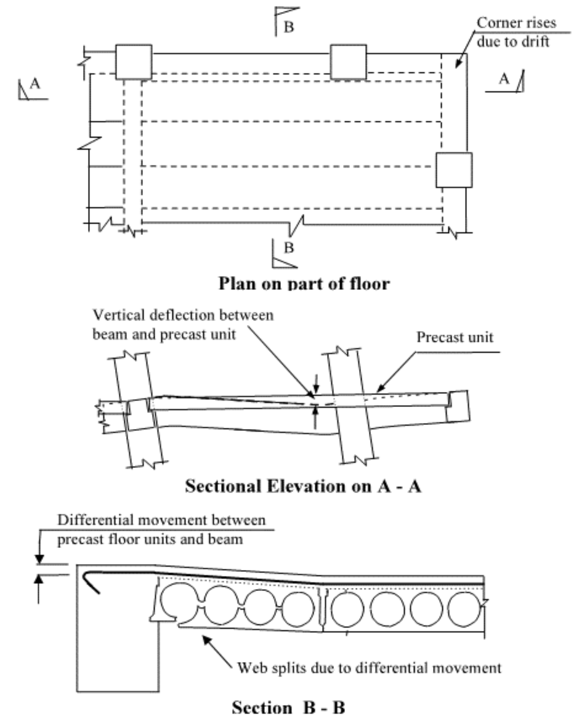
Capacity elements
(NZS3101:2006)

$$\%NBS = 100\,\frac{0.9R_{\mathrm{prob}}}{K_{\mathrm{d}}R_{\mathrm{E},\mu=1.25}}$$

demand

(Strength-based %NBS)

Deformation/Drift



$$\%NBS = 100\,\frac{\theta_{\mathrm{SC}}}{K_{\mathrm{d}}\theta_{\mathrm{SD}}}$$

(Drift-based %NBS)

QUESTIONS?