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SEMINAR SERIES 2017

Fundamentals of Reinforced Concrete Seismic Design

Presented by

The New Zealand Concrete Society

Seminar Notes

(TR64)

Presented by

The New Zealand Concrete Society

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NZ Society for Earthquake Engineering Incorporated

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Sesoc

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Presenters:

Barry Davidson, Compusoft Engineering Ltd

Nicholas Brooke, Compusoft Engineering Ltd

Fundamentals of Reinforced Concrete Seismic Design

Session 1:
Concepts of Seismic Design In New Zealand

Barry Davidson
Courtesy of Compusoft Engineering Limited

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New Zealand Seismic Design Philosophy

- to protect lives
- minimize the cost and inconvenience of repair
- minimize the upfront cost of construction

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New Zealand Seismic Design Philosophy

- ISO2394[1.2] that requires buildings to be designed and constructed so that the annual earthquake fatality risk is of the order of 10^{-6}.
- Implied collapse
- Collapse Limit State
- Ultimate Limit State 10% probability in 50 year life

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Limit States

- The Serviceability Limit State for “Normal” structures, SL1.
  – This requires ensuring for “Normal” structures (Table 2 AS/NZS1170.0) that seismic induced displacements resulting from an earthquake that have annual probability of exceedance of 1/25 do not cause sufficient damage to structural and non-structural elements as to require them to be repaired.

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Limit States

- The Ultimate Limit State for “Normal” structures, ULS.
  – This requires that the design for “Normal” structures (Table 2 AS/NZS1170.0) from seismic actions resulting from an earthquake that has annual probability of exceedance of 1/500 must:
  – Avoid collapse of the structural system.
  – Avoid collapse or loss of support of parts of the structure representing a hazard to human life inside and outside the structure.

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Limit States

- The Ultimate Limit State for “Normal” structures, ULS.
  – Avoid damage to structural and nonstructural systems necessary for the building evacuation procedures that renders them inoperative

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Limit States

- The Collapse Limit State
  - Not explicitly designed for
  - "Hope" that the structure's additional strength from Conservative design procedures
    - strength redistribution,
    - soil structure interaction
    - non-structural damping
    - Conservative values for allowable material strains

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Limit States

The Collapse Limit State

Consequently, the magnitude of this additional strength would be expected to vary widely, and depend upon the type of structure, the design approach and the seismic region.

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New Zealand Seismic Design Philosophy

Christchurch experience

- Well designed buildings were NOT designed to survive without damage
- Well designed and constructed buildings should NOT have failed giving rise to injuries or death

IF we accept the sufficiency of the Standards

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Comparison with Design for Wind

Size of Extreme:Design Event

Wind 1.14:1.00 Earthquake 1.8:1.00

Design requirements

Wind – Size and shape of structure
Earthquake – Dynamics of structure

Predictability

Wind –Predictable (run and hide)
Earthquake - Unpredictable

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What is an Earthquake

- abrupt shaking of the ground

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What is an Earthquake

- abrupt shaking of the ground
  - Random motion

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What is an Earthquake

- abrupt shaking of the ground
  - Random motion
  - 10 – 60 seconds

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What is an Earthquake

- abrupt shaking of the ground
  - Random motion
  - 10 – 60 seconds
- Three orthogonal directions

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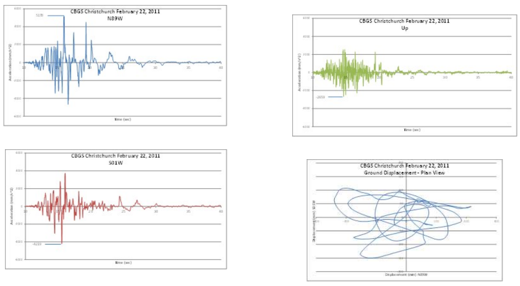
What is an Earthquake



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Variability and Magnitude of Earthquakes

Magnitude M	Average number above M
8	2
7	20
6	100
5	3000
4	15,000
3	more than 100,000

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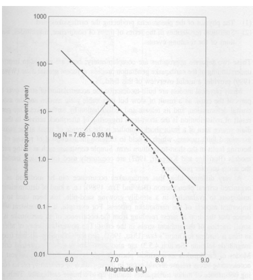
Variability and Magnitude of Earthquakes



$$\log N = a - bM$$

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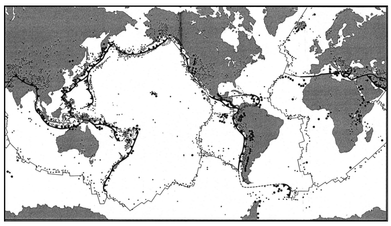
Variability and Magnitude of Earthquakes



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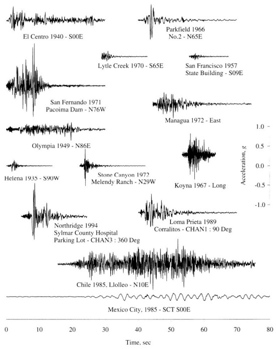
Variability and Magnitude of Earthquakes



At a site
Variability in:
- Amplitude
- Duration
- Frequency

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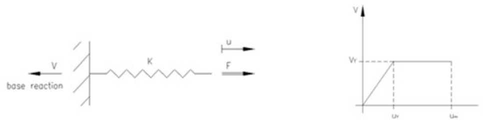
Fundamentals of Reinforced Concrete

Capacity Design

Solution to variability AND
Expense of Elastic Response



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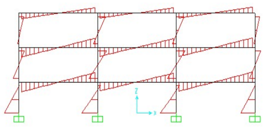
Capacity Design



Flexural Yielding

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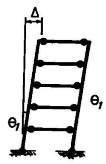
Capacity Design



Design Approach
- Envisage a "mechanism"
- That can support Gravity
- The hinges are fuses

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Capacity Design

Requirements

- The hinges need to be constructed to yield at a **well defined strength**.

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Capacity Design

Requirements

- The hinges need to be constructed to yield at a **well defined strength**.
- The hinges need to be detailed so that they may undergo a number of displacement reversals without losing appreciable strength. (**Not brittle**)

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Capacity Design

Requirements

- The hinges need to be constructed to yield at a **well defined strength**.
- The hinges need to be detailed so that they may undergo a number of displacement reversals without losing appreciable strength. (**Not brittle**)
- Other parts of the structure must be designed and constructed to resist in an **"elastic"** manner the actions that result from the nonlinear response of the structure with the fuses acting.

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Capacity Design

Strength Hierarchy

1. Design "Fuses"

- *S^* is Required Strength (from Analysis)*
- *$\emptyset S_i$ is the Dependable Strength (from Conservative material properties and Mechanics)*
- *$\emptyset$ is a Strength Reduction factor (less than 1)*
-

2. Design Structure Stronger than "Fuses"

- *Overstrength, "So"*
- λ_o is the Over Strength factor. The value of this factor will depend upon material characteristics.

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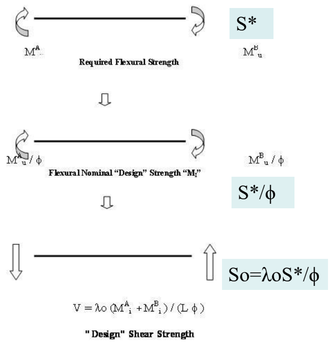
Capacity Design

Strength Hierarchy



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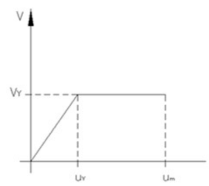
Ductility

$$^{u_m}/_{u_y} = \mu$$



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Ductility

- In Seismic Design

"Ductility" is interpreted as ability of a structure to undergo inelastic reversing deformations during an earthquake without a significant loss of strength.

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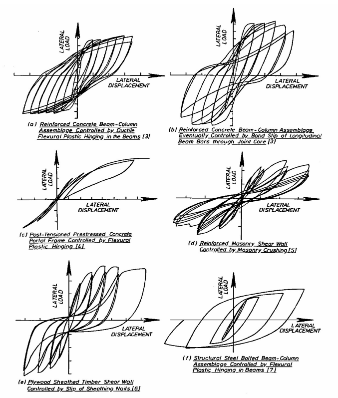

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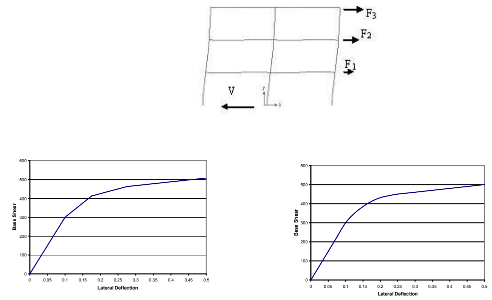
Ductility

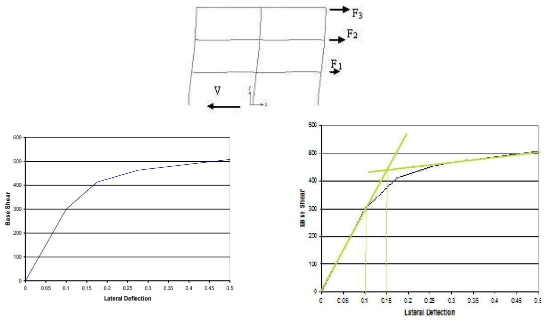
- In Seismic Design

- The differences between:
  - Structural displacement ductility
  - Member ductility
  - Curvature ductility

Ductility



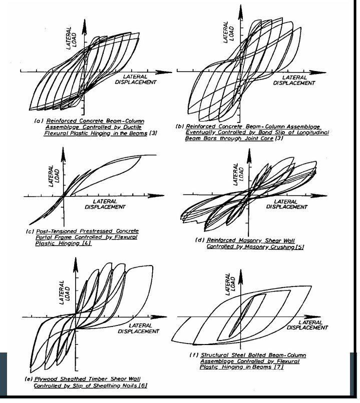
Ductility



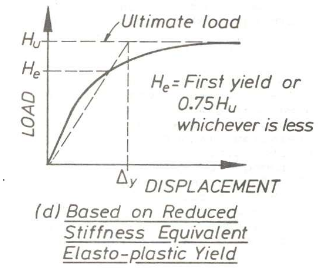
Ductility

Typical Test Results

What is ductility ?

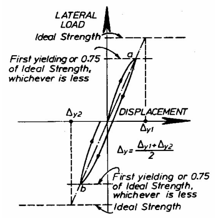


Ductility

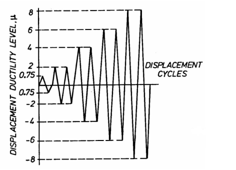


Ductility

Definition of yield



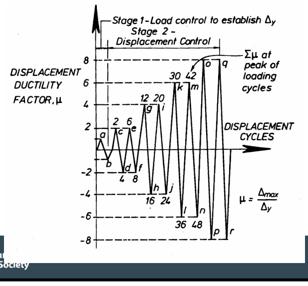
Definition of Load Sequence



Ductility

Definition of “failure” - 80% of maximum load

Definition of “effective” ductility $\mu = \sum\mu / 8$



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Summary

- Large variability in earthquakes
  - Size
  - Arrival
  - Duration
- Overcome through design
  - Life safety
  - Capacity Design

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Fundamentals of Reinforced Concrete Seismic Design

Session 2:
Framework for Seismic Design of Reinforced Concrete in New Zealand

Nic Brooke

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Overview

- The goal
- Legislative requirements
  - Means of compliance
  - Function of Standards
- Reliability of structures
  - Background
  - Achievement of reliability
  - Assessment vs design
- Ductility of reinforced concrete buildings
  - Impact on design actions
  - Member vs structure ductility
- Desirable characteristics for buildings

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Legislative requirements

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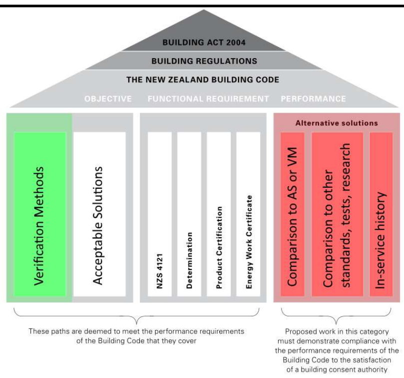

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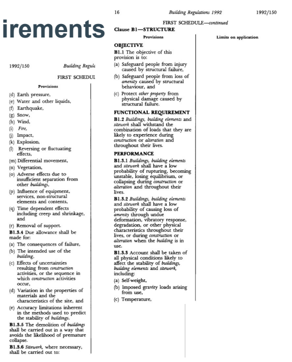
Building Code Requirements

- B1 – Structure
  - Objective:
    protect against injury and loss of amenity
  - Functional req.:
    withstand likely loads
  - Performance req:
    low probability of rupturing, becoming unstable…



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What is a 'low probability'?

- Specific Building Code requirements can only be inferred from methods deemed to show compliance
  - Acceptable Solutions
  - Verification Methods
- Reinforced concrete structural design – B1/VM1
  - NZS 1170
  - NZS 3101
  - Specific amendments by MBIE

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Any deviation is an Alternative Solution

- e.g. proprietary products
- Adequacy of Alternative Solutions must be demonstrated:
  - Comparison to Verification Method
  - Research
  - International Standards and Codes
- Reliability should be equivalent to a ‘B1/VM1’ structure

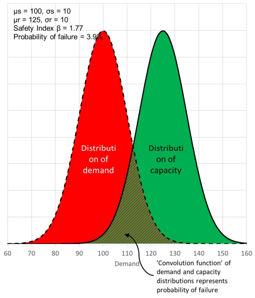
Reliability of concrete structures

Holistic aim of Standards

- Standards aim to achieve a certain level of reliability, e.g.
  - NZS 1170.5 - *target annual earthquake fatality risk in the order of 1×10^{-6} (ISO 2394:1998)*
  - NZS 4203 – *expressed in the form of a safety index, β…earthquake lateral loading – average value 1.75, range from 1.5 to 2.0*
- Detail of reliability theory is (hugely) complex – but simple in the big picture

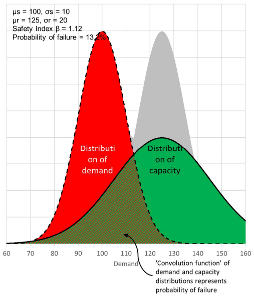
Basics of reliability

- Failure occurs if demand exceeds capacity
- Demand and capacity are uncertain
- Failure probability determined by ‘overlap’ of distributions



Basics of reliability

- Reliability reduced by:
  - Increased average demand
  - Decreased average capacity
  - Increased variability of demand or capacity



NZS 3101 measures to achieve reliability

- Clause 1.1.1.1
  - *This Standard sets out minimum requirements for the design of reinforced and prestressed concrete structures.*
- Forthcoming amendment:
  - *In addition to these requirements every load or force acting on a structure shall have one or more dependable load paths that can transfer the force to the foundation soils. Each load path shall satisfy the fundamental structural design requirements of equilibrium and displacement compatibility.*

Holistic measures

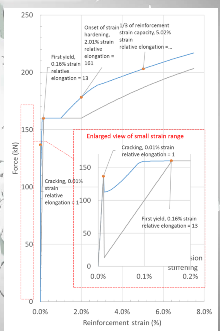
- Use of characteristic strengths
- Detailing
- Strength reduction (Φ) factors
- Prohibition of reliance on concrete in tension
- Aiming for section behaviour to be limited by reinforcement

Strength reduction factors

- Account for uncertainties
  - Minimum vs lower characteristic strength
    - Steel – min/5th%ile ≈ 10% - so "best possible" Φ = 1-0.1 = 0.9
    - Concrete in compression – min/5th%ile ≈ 25%
    - Concrete in tension – minimum effectively zero
  - Imperfection of design procedures
    - Simplifications
    - Limitations of calibration – i.e. extrapolation vs interpolation
  - Imperfection of element dimensions
- Calibrated in part to promote 'desirable behaviour – flexure vs shear etc

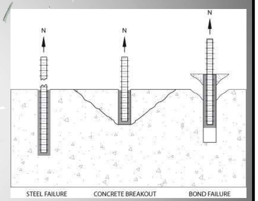
Concrete in tension

- Concrete is more brittle than glass
- Low strength
- Reduced further by shrinkage



Concrete in tension

- New clause:
  reinforcement with sufficient nominal strength to resist the total tension force required to maintain equilibrium of the structure must be provided
- Limited exceptions permitted – implicitly relied on in some instances
- Care required to avoid unwitting breach



Reliability in design vs assessment

- Assessment philosophies provide lower reliability
  - 'Non compliant' load paths and detailing
  - Less dependable material properties
  - Less stringent Φ factors

What is the %NBS of a new building?

- Design or assessment – ΦS ≥ S*
  - Design: Φ = 0.85, nominal strength – S_n
  - Assessment: Φ = 1.0, expected strength – $S_E \approx 1.1S_n$
- Impact is significant

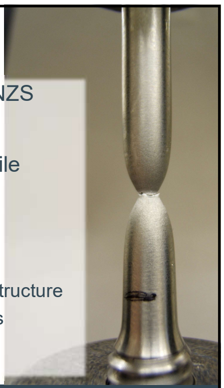
$$\Phi S_n \geq 100\%NBS$$
$$0.85\frac{S_E}{1.1} \geq 100\%NBS$$
$$S_E \geq \frac{100}{0.85} \times 1.1 = 129\%NBS$$

- With some allowance for rationalisation ≥ 150%NBS

Structure and member ductility

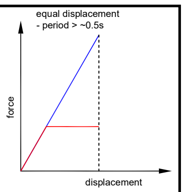
Types of ductility

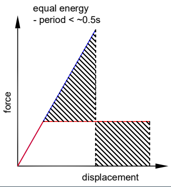
- All structures complying with NZS 3101 should be ductile
- Ductile structures require ductile materials
- Two types of ductility require consideration
  - Displacement ductility – whole structure
  - Member ductility – plastic hinges



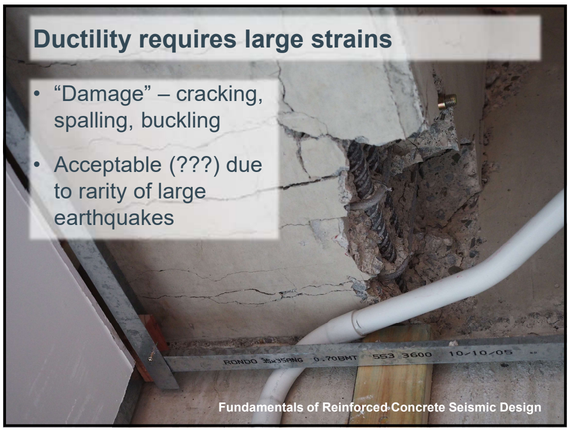
Structure ductility

- Earthquakes impose displacements rather than forces
  - required displacement similar irrespective of ductility
  - reduced design actions





Ductility requires large strains



- "Damage" – cracking, spalling, buckling
- Acceptable (???) due to rarity of large earthquakes

Structure ductility

- Limited depending on structural type and detailing
- Limited/fully ductile similar
- Often not possible to use full ductility
  - drift limits
  - serviceability

Type of structure	Reinforced concrete
1. Nominally ductile structures In-plane action of singly reinforced walls	1.25
2. Structures of limited ductility	
(a) Moment resisting frame	3
(b) Walls	3
(c) Cantilever face loaded walls (single storey only)	2
3. Ductile structures	
(a) Moment resisting frame	6
(b) Walls	
(i) Two or more cantilevered	5
(ii) Two or more coupled	6
(iii) Single cantilever	4

Nominal vs limited/full ductility

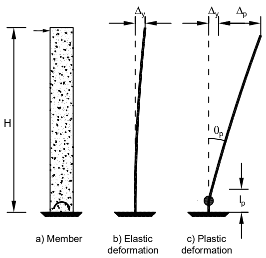
	Nominally ductile	Limited/full ductility
Capacity design	No…but	Yes
Sp	0.9	0.7
Design	x.3 clauses…but	x.4 clauses
Hinge detailing	Check rotations, detail accordingly	Check rotations, detail accordingly

Clause 2.6.6.1

- *When the structural system is such that under seismic actions larger than anticipated, mechanisms could only develop in the same form as those permitted … for ductile structures… the selected structure is exempt from the additional seismic requirements of all sections of this Standard.*
- *When a mechanism could develop in a form which is not permitted for ductile or limited ductile structures, the relevant mechanism or mechanisms shall be identified. Potential plastic hinge regions shall be identified, and detailed for ductile or limited ductile plastic regions such that the material strain limits… are not exceeded, in accordance with the additional seismic design requirements of this Standard.*

Member ductility

- Not the same as structural ductility
  - Elastic deformation = result of curvature along whole member
  - Plastic deformation = curvature in hinge only
- Influenced by
  - Geometry
  - Structural ductility



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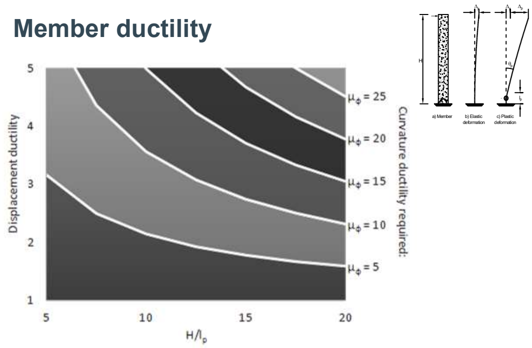
Member ductility



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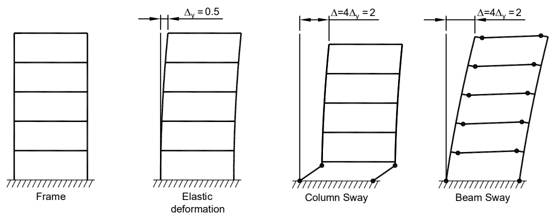
Impact of sway profile

- In 'complex' structure, location and number of hinges has an effect



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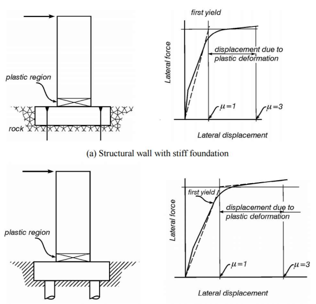
Impact of foundations



Fenwick & Dhakal 2007

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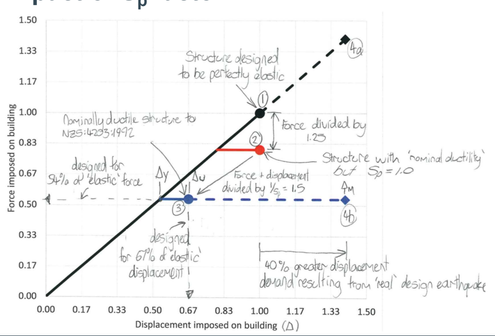
Impact of S_p factor



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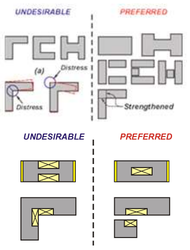
Desirable characteristics for buildings

What is undesirable performance?

- Unpredictable behaviour - not readily explainable by analysis
- Excessive torsional response
- Concentration of damage, especially at specific stories
- Damage disproportionate to earthquake intensity

How is chance of indesirable behaviour minimised?

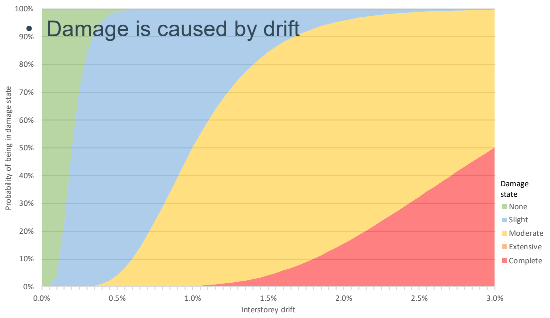
- Regularity
- Robustness of connection between different parts of a building
- Avoidance of unwanted load paths
- “Elastic” strength and stiffness of the building
- Predictable occurrence of plastic deformations in desirable, locations
- Ability to sustain plastic deformations.



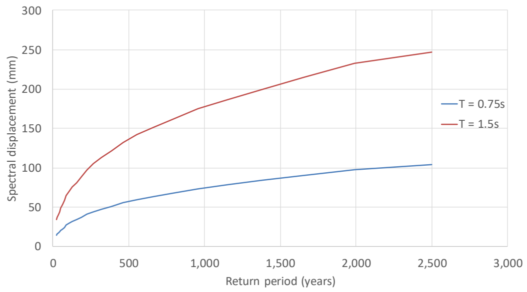
Paulay & Priestley 1992

Impact of stiffness on damage

- Damage is caused by drift

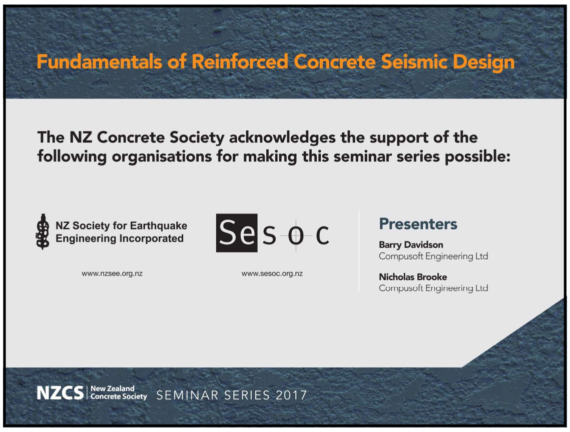


Large drifts occur less frequently in stiff buildings



What is desirable?

- Strong, stiff building
- Regular and predictable
- Detailed to sustain large ductilities – just in case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Presenters
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Fundamentals of Reinforced Concrete Seismic Design

Session 3:
Earthquake Design Actions

Barry Davidson
Courtesy of Compusoft Engineering Limited

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Earthquake Design Loads -Overview

- Design Loads
- Design Actions (Member Actions) Deformations
- Controlling Actions (fuses)

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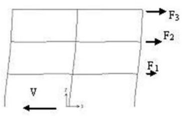
Equivalent Static Method – A Quick Review

- $F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}$



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Equivalent Static Method – A Quick Review

- $F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}$
- $V = C_d(T_1)W_t$

- $F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}$
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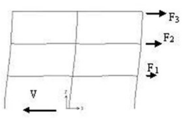
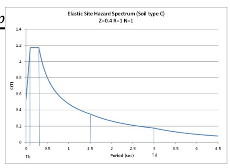
Equivalent Static Method – A Quick Review

- $F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}$
- $V = C_d(T_1)W_t$
- $C_d = \frac{C(T)S_p}{k_\mu}$





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Seismic Response of a Structure

- NZS1170.5 Cl. 2.1.3 - “Localised Actions”
- “Structural elements and members shall be tied together to enable the structure to act as a whole in resisting seismic actions.”

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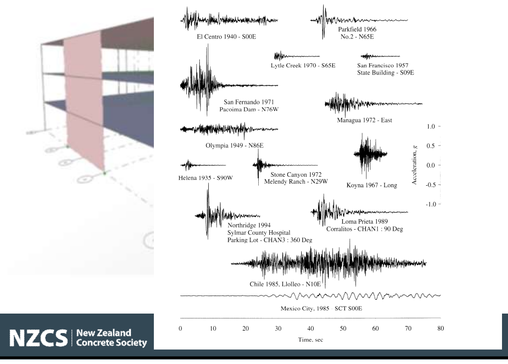
Seismic Response of a Regular "Elastic" Structure



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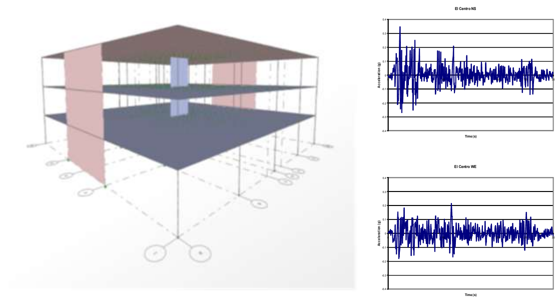
Seismic Response of a Regular "Elastic" Structure



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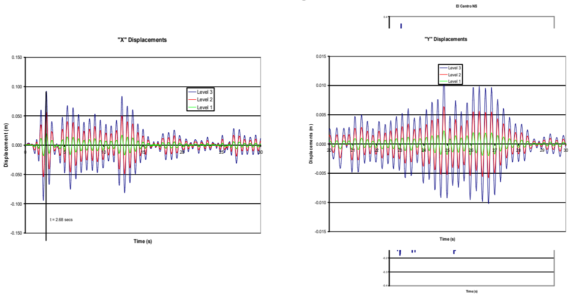
Seismic Response of a Regular "Elastic" Structure



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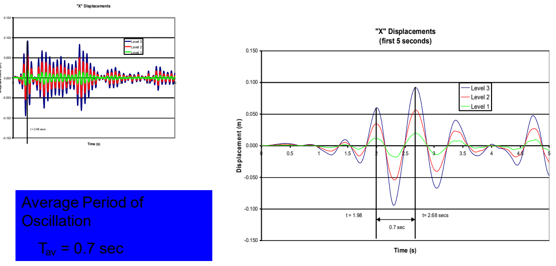
Seismic Response of a Regular "Elastic" Structure



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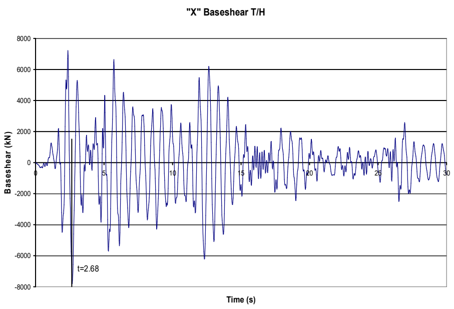
Seismic Response of a Regular "Elastic" Structure



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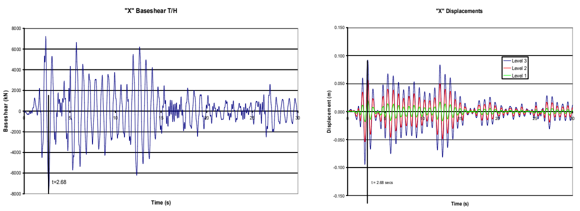
Seismic Response of a Regular "Elastic" Structure



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Seismic Response of a Regular "Elastic" Structure

Conclusions

Structures respond to a somewhat random seismic ground vibration

- by "oscillating" back and forward in a harmonic manner
- where all parts of the structure move together
- displace in an approximately consistent shape
- the amplitude of the oscillations varies
- the period of oscillation is relatively constant

- harmonic nature of the base shear
- time of occurrence of its maximum same as maximum displacements.

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Seismic Response of a Regular "Elastic" Structure

Summary of Observations

1. While the Ground Motion was quite random
   - Motion of structure was "harmonic"

2. All floors moved essentially in unison

Single degree of Freedom

3. "Period" of response independent of earthquake

$T = 2\pi\sqrt{(M_{eff} / K_{eff})}$

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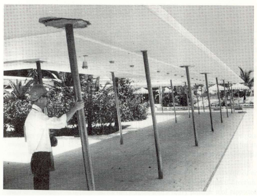
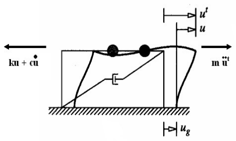
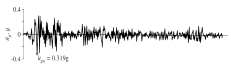
Single Degree of Freedom Oscillator (SDF)







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Single Degree of Freedom Oscillator (SDF)

$m(\ddot{u} + \ddot{u}_g) + c\,\dot{u} + ku = o$

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Single Degree of Freedom Oscillator (SDF)

$m(\ddot{u} + \ddot{u}_g) + c\,\dot{u} + ku = o$

$(\ddot{u} + \ddot{u}_g) + c/m\,\dot{u} + k/m\,u = o$

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Single Degree of Freedom Oscillator (SDF)

$m(\ddot{u} + \ddot{u}_g) + c\,\dot{u} + ku = o$

$\omega = 2\pi / T$ (rads/s)

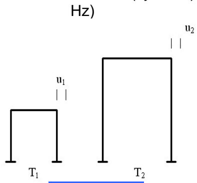
$f = 1 / T$ (cycles/s) (Hz)

$(\ddot{u} + \ddot{u}_g) + c/m\,\dot{u} + k/m\,u = o$

$k/m = \omega^2$

$c/m = 2\,\xi\,\omega$

$(\ddot{u} + \ddot{u}_g) + 2\,\xi\,\omega\,\dot{u} + \omega^2\,u = o$



If $\xi_2 = \xi_1$ and

$T_2 = T_1$, then

$u_2 = u_1$

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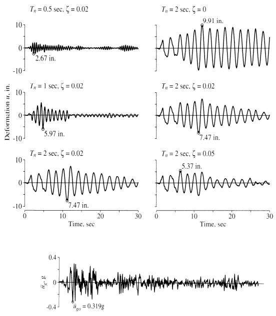
Single Degree of Freedom Oscillator (SDF)



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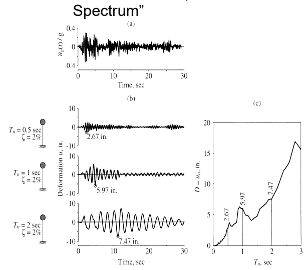
Single Degree of Freedom Oscillator (SDF)

Definition of "Displacement Spectrum"



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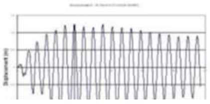
Relationship between displacement, velocity, acceleration

T = 3.0 sec

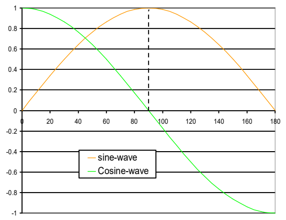
ζ = 0.001



d = D sin (2π t/T)

v = (2π/T) D * cos (2π t/T)

a = - (2π/T)^2 D sin (2π t/T)



At time of max displacement

sin (2π t/T) = 1

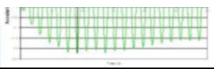
d = D = - 0.4983 (m)

v = 0

A = (2π/3)^2 0.4983 = 2.186 m/s^2

= 0.223 g

v_{max} ~ 2π/3 * 49.94 = 104.59



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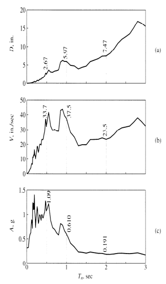
Pseudo Spectra

Pseudo velocity = displacement * ω, Pseudo acceleration = displacement * ω^2.



($\omega = 2\pi$ /T)

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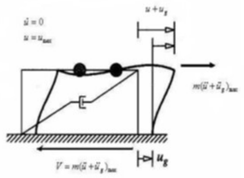
Static Interpretation of Response Spectra

m (ü + $ü_g$) + c ú + ku = o

u = u_{max}

ú = 0



Ext F = m (ü + $ü_g$)$_{max}$

Result u_{max}

Int F = k u_{max}

Equilibrium

V = Ext = m (ü + $ü_g$)$_{max}$

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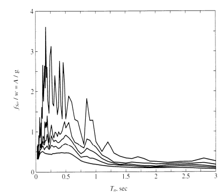
Elastic Spectra



Linear Response Spectra (different Damping)

- Observe Response Spectrum is plotted

A / g vs Period

V = F = m (ü + $ü_g$)$_{max}$

V = $\frac{w}{g}$ (ü + $ü_g$)$_{max}$)

V / w = (A / g)

Or, the response spectrum plotted as a function of "g", or "A/g"

is a representation of Base shear (V) in terms of seismic weight (W)

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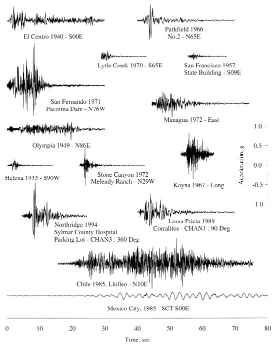
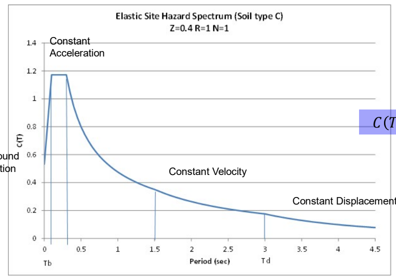


These all have different spectra

However, the shape of the spectra illustrate trends

Design Spectra



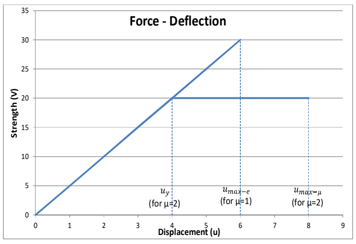
Constant Hazard Spectrum

$C(T) = C_h(T)ZR \quad (T, D)$

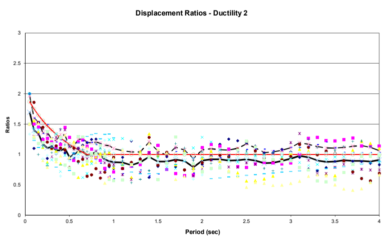
Inelastic Design Spectra

- In similar approach as "elastic" hazard spectra
  - Developed from "trends" in shape
    - "Equal Displacement" hypothesis

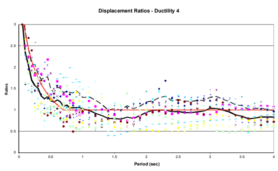
Inelastic Design Spectra

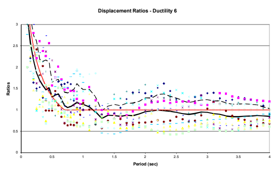


Inelastic Design Spectra



Inelastic Design Spectra





Inelastic Design Spectra

- Observations

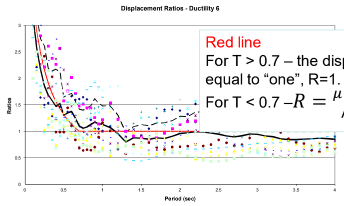
- There is large scatter in the results.
- Over the period range 0.7 – 4.0 seconds, the mean ratio is approximately equal to one.

$$u_{max} \quad = u_{max-e}$$

- This endorses the equal ductility displacement hypothesis.

- The mean plus one standard deviation exceeds "one" over this period range.
- The larger the ductility ratio, the larger the scatter. The variability observed is a consequence of the nature of earthquake loading.

Inelastic Design Spectra



Red line

For T > 0.7 – the displacement ratio is equal to "one", R=1.

For T < 0.7 – $R = \mu / [\frac{(\mu-1)T}{0.7}+1]$

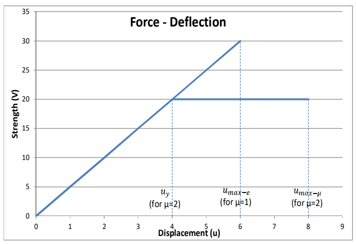
$$k_\mu = \mu$$
$$k_\mu = \frac{(\mu-1)T}{0.7}+1$$

Inelastic Design Spectra

- Summary

- If we divide the elastic strength by k_μ we assume that the system will yield and have a displacement associated with μ

Inelastic Design Spectra



As we are performing an elastic analysis, the displacement calculated is u_y

To obtain the maximum displacement, we need to scale by μ

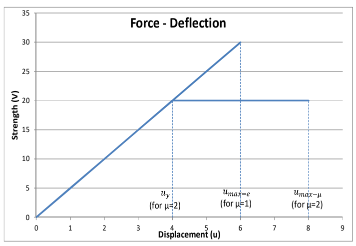
Inelastic Design Spectra

- Summary (incl. Sp)

- If we divide the elastic strength by k_μ and multiply by "Sp" we assume that the system will yield and have a displacement associated with μ/Sp

Inelastic Design Spectra

- Summary(incl. Sp)

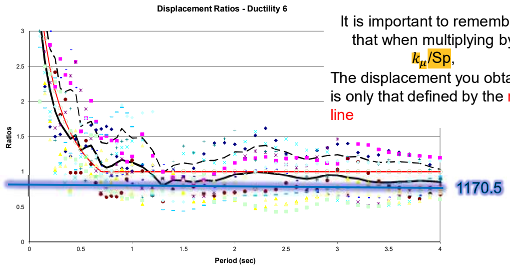


As we are performing an elastic analysis, the displacement calculated is u_y

To obtain the maximum displacement, we need to scale by

μ/Sp

Inelastic Design Spectra



It is important to remember, that when multiplying by k_μ/Sp,
The displacement you obtain is only that defined by the red line

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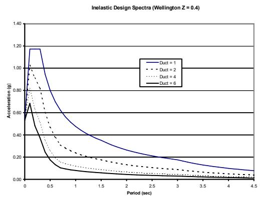
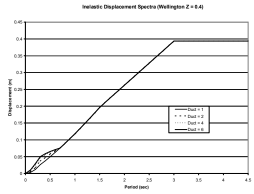
Inelastic Design Spectra





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Fundamentals of Reinforced Concrete Seismic Design

Session 4:
Deformation, Ductility, and Stiffness of Concrete Members

Nic Brooke

NZCS New Zealand Concrete Society SEMINAR SERIES 2017

Overview

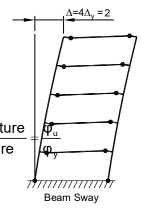
- NZS 3101 approach to detailing of plastic hinges
- Deformation demands in plastic hinges
  - Impact of k_{dm}
  - Relationship to detailing requirements
- Design requirements to achieve ductility
  - Reinforcement properties
  - Detailing
  - Anchorage
- Effective stiffness of concrete members

Why is detailing required to achieve ductility?

- Ductile building behaviour results from concentrated plastic deformations
  - Most commonly flexural plastic hinges
  - Other scenarios exist – e.g. coupling beams

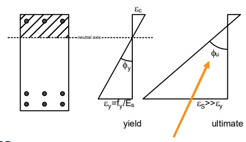
$$\mu_\varphi = \frac{\text{ultimate curvature}}{\text{yield curvature}} = \frac{\varphi_u}{\varphi_y}$$

- Plastic deformation creates large tension and compression strains
- Detailing ensures strains can be sustained



Quantification of plastic deformation

- Fundamentally – larger strains require more stringent detailing
- For flexure – conveniently quantifiable based on curvature $\mu_\varphi = \frac{\text{ultimate curvature}}{\text{yield curvature}} = \frac{\varphi_u}{\varphi_y}$
- Integration of curvature gives a rotation
  - Thus easily linked to displacement of building
  - Integration of plastic curvature gives plastic rotation

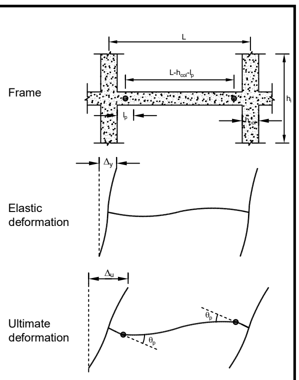


Plastic rotation

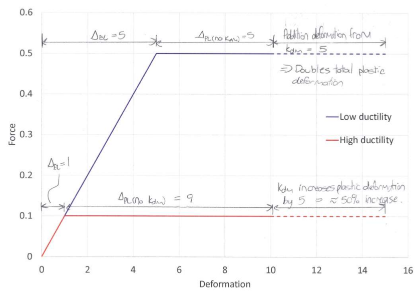
- Calculated from deformed shape and geometry
  - e.g. for a frame

$$\theta_p = \frac{(\Delta_u - \Delta_y)}{h_i}\frac{L}{(L - h_c - l_p)}$$

- Based on drift – so k_{dm} increases demand

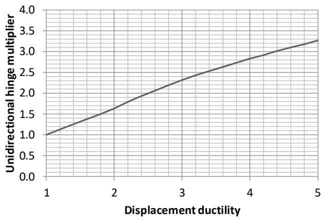


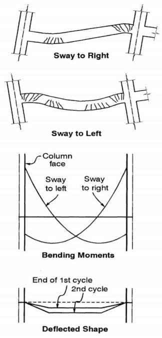
Significance of k_{dm} for low ductility buildings



Plastic rotation

- Uni-directional hinges "crank"
  - Note error in NZS 3101





Fenwick & Megget 1993

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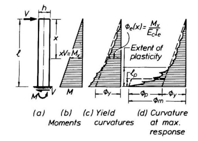
Calculation of plastic curvature

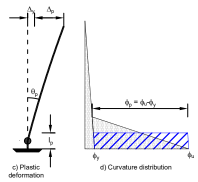
Paulay & Priestley 1992

- Curvature (and strain) vary over length
- For convenience – 'effective' plastic hinge length defined:

$$\theta_p = l_p \phi_p$$

- NOT the length of yielding (ductile detailing length)





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NZS 3101 approach to detailing of plastic hinges

- NZS 3101 requires local plastic deformation demands be assessed irrespective of the ductility class of a building
  - Hinges at the top of a high-ductility building might not experience substantial yielding
  - Some hinges in a poorly configured nominally ductile building might be subject to large plastic deformations
- Level of detailing required for hinge depends on the local demand

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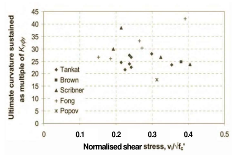
NZS 3101 procedure

- NZS 3101 uses a 'material strain index' – K_d
- Developed from experiments:
  - Collect experimental results
  - Categorise each test to its detailing
  - Determine plastic deformation, θ_p, 'failure'
  - Convert θ_p to non-dimensional index (K_d) by reference to notional yield curvature and effective plastic hinge length.
- Despite appearances - K_d is not a real curvature ductility value – it is a convenient index based on similar, but simplified/idealised, concepts.



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NZS 3101 Procedure

- Calculate notional yield curvature: $\phi_y = \dfrac{2\varepsilon_y}{h}$
- Calculate plastic hinge length $0.25h \le l_p = 0.25\dfrac{M_e}{V_e} \le 0.5h$
- Calculate plastic rotation demand, θ_p
  - Based on drift, so increased by k_{dm}
- Calculate plastic curvature: $\phi_p = \dfrac{\theta_p}{l_p}$
- Calculate material strain index $K_d = \dfrac{\phi_p + \phi_y}{\phi_y}$
- Detail accordingly

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Detailing requirements

- Determined based on magnitude of K_d

Member type	Specifics	Nominally ductile	Limited ductile	(Fully) ductile
Beams and columns		3	11	19
Walls	Singly reinforced	0.8	n/a	n/a
	Doubly reinforced	4	6	14
	Doubly reinforced with boundary elements	n/a	9	16

- Check K_d early – can be critical for structure configuration

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Example

- 750 mm deep beam, Grade 300 reinforcement, 5.8 m span and 800 mm columns

$$\phi_y = \frac{2\varepsilon_y}{h} = \frac{2\times 0.0015}{750} = 4\times 10^{-6}\text{rad/mm}$$

- Assuming $V_e = 2M_e/l_{clear}$

$$0.25h \le l_p = 0.25\frac{M_e}{V_e} \le 0.5h$$

$$187.5 \le l_p = 0.25\frac{5000}{2} = 625 \le 375$$

$$l_p = 375\text{mm}$$

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Example

- Assume frame is nominally ductile
  - Storey height 3200 mm, k_{dm} = 1.5
  - Elastic interstorey displacement 12 mm (from Etabs say)
  - Design interstorey displacement ≈ 1.35 × 12 = 16 mm (greater than μ = 1.25 × at critical storey due to sway)
- Design drift = 1.5 × 16/3200 = 0.0075
- Elastic drift = 12/3200 = 0.00375

$$\theta_p = (\delta_{des} - \delta_y)\frac{L}{(L - h_c - l_p)}$$

$$= (0.0075 - 0.00375)\frac{5800}{5800 - 800 - 375} = 0.0047\text{rad}$$

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Example

- Calculate plastic curvature:

$$\phi_p = \frac{\theta_p}{l_p} = \frac{0.0047}{375} = 12.5\times 10^{-6}\text{rad/mm}$$

- Calculate strain index:

$$K_d = \frac{\phi_p + \phi_y}{\phi_y} = \frac{12.5+4}{4} = 4.1$$

Member type	Nominally ductile	Limited ductile
Beams and columns	3	11

- So hinge is limited ductility – even though structure is nominally ductile

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How do you achieve this

- Appropriate reinforcement properties
- Suitable detailing

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Reinforcement properties

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Seismic reinforcing steel

- Use of low ductility rebar can be catastrophic

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Seismic reinforcing steel

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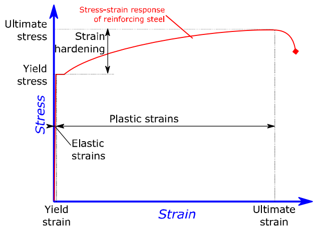
New Zealand reinforcing steel

- Two key requirements
  - Ultimate strain capacity
  - Significant strain hardening potential



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New Zealand reinforcing steel

- Only Grade ‘E’ reinforcement is acceptable

Property	Grade 300E	Grade 500E
Lower characteristic yield strength	≥ 300 MPa	≥ 500MPa
Upper characteristic yield strength	≤ 380 MPa	≤ 600 MPa
Strain hardening ratio	1.15 – 1.5	1.15 – 1.4
Ultimate strain	≥ 15%	≥ 10%
Notes	Mild steel, so neither MA nor Q&T	Can be produced by micro-alloying (MA) or quenching and tempering (Q&T).

- AS/NZS 4671 lists three types of Grade 500 reinforcement – L, N, E
  - yield strength of N/L grades not tightly controlled
  - low ultimate strain
  - low ratio of ultimate to yield strength

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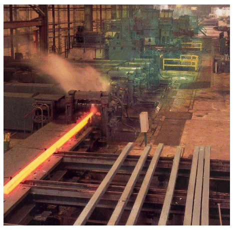
Grade 500E Reinforcement

- Two manufacturing methods
  - micro alloy (MA)
  - quenched & tempered (Q+T)



- QT less robust on site
  - must NEVER be welded
  - must NEVER be threaded
  - must NEVER be heated

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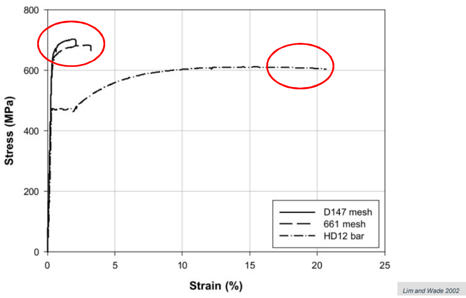
Cold drawn mesh



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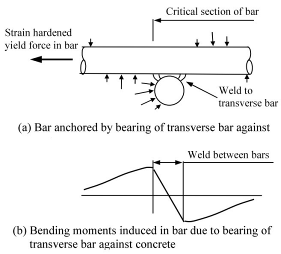
Anchorage

- Actually worse than stated



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Detailing for ductility

Detailing for ductility

- Four key requirements
  - Reinforcement content
  - Transverse reinforcement
  - ‘Compression’ reinforcement
  - Bar anchorage
- Level of detailing required depends on hinge rotation demand

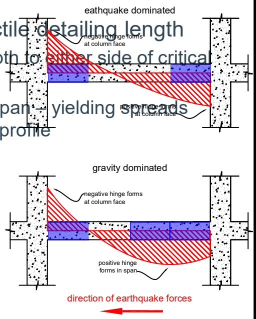
Detailing for ductility

Table C10.2 – Design of reinforced columns and piers (Continued)

	Design issue	Nominally ductile seismic design philosophy	Ductile seismic design philosophy
Transverse reinforcement within potential plastic hinge region	Minimum diameter for transverse reinforcement	Same as outside plastic hinge region	Same as for nominally ductile
	Maximum vertical spacing of ties		In DPRs smallest of $h/4$, $b/4$, diameter/4 or 6 d_b (10.4.7.4.5(a), 10.4.7.5.5 (a)) In LDPRs smallest of $h/4$, $b/4$, diameter/4 or 10 d_b (10.4.7.4.5(b), 10.4.7.5.5 (b))
	Anti-buckling reinforcement		For rectangular hoops and ties In DPRs & LDPRs $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s_h}{d_b}$ (10.4.7.5.1) Regions protected from hinging $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}} \frac{s_h}{d_b}$ (10.4.7.5.3) Spirals or circular hoops In DPRs & LDPRs $p_s = \frac{A_{st}}{110 d''} \frac{f_y}{f_{yt}} \frac{1}{d_b}$ or (10.4.7.4.1) In regions protected from hinging $A_{te} = \frac{A_{st} f_y}{155 d'' f_y d_b}$ (10.4.7.4.4)
	Confinement reinforcement		For rectangular hoops and ties In DPRs $A_{sh} = \frac{(1.3 - p_t m) s_h h''}{3.3} \frac{A_g}{A_c} \frac{f'_c}{f_{yt}} \frac{N^*_o}{\phi f'_c A_g} - 0.006 s_h h''$ (10.4.7.5.1) In LDPRs and regions protected from hinging use 70 % of this area (10.4.7.5.2 and 10.4.7.5.3) Spirals or circular hoops In DPR $p_s = \frac{(1.3 - p_t m)}{2.4} \frac{A_g}{A_c} \frac{f'_c}{f_{yt}} \frac{N^*_o}{\phi f'_c A_g} - 0.0084$ (10.4.7.4.1)

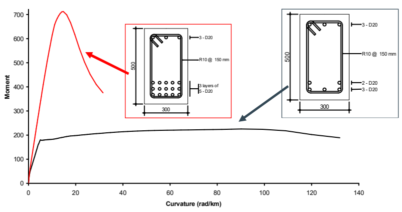
Detailing for ductility

- Detailing extends over ductile detailing length
  - generally twice member depth to either side of critical section
  - longer length for hinges in span – yielding spreads more due to flatter moment profile



Reinforcement content

- Must be within a specific band to achieve ductility
  - Cracking strength exceeds flexural strength if too low
  - Compression governed if too high

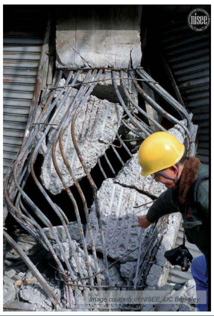


Compression reinforcement

- Increases beam curvature du
  - C_c reduced so smaller NA dept
  - helps hold core concrete in pla
- Provide transverse reinforcer compression reinforcement b

Transverse reinforcement

- Three roles
  - Shear strength
  - Confinement
  - Anti-buckling



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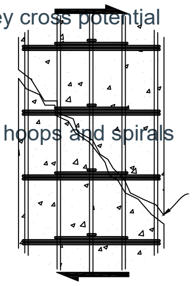
Shear strength

- Stirrup sets only effective if they cross potential (45 degree) crack
- Note differing contributions for hoops and spirals



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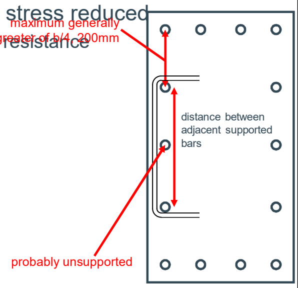
Bar buckling

- Average compression stress reduced
  - Reduces compression resistance
- Promotes spalling
  - further load reduction
- High strains at buckle
  - Promotes bar rupture



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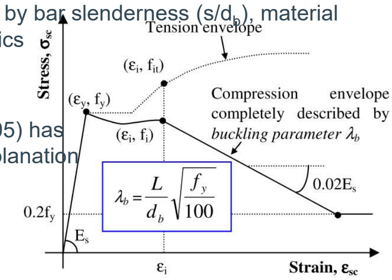
Bar buckling

- Determined by bar slenderness (s/d_b), material characteristics
- Dhakal (2005) has detailed explanation



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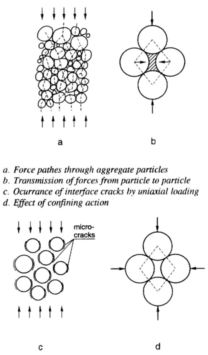
What is confinement?

- Axial strain causes transverse
  - Splitting failure at low strain
- Confinement
  - restricts dilation
  - greater strength and ductility



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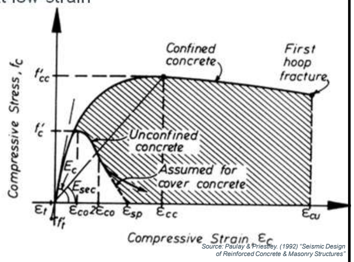
What is confinement?

- Axial strain causes transverse tension
  - Splitting failure at low strain
- Confinement
  - restricts dilation
  - greater strength and ductility



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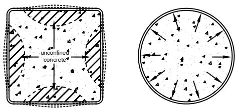
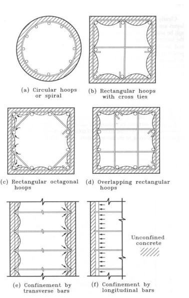
Confinement effectiveness

- Affected by reinforcement layou
  - Circular more effective
  - Longitudinal spacing important fo





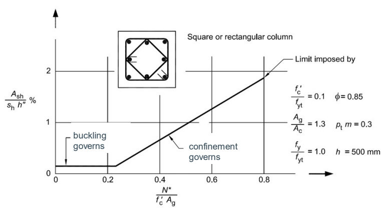
Priestley et al. (2006)

Confined core

- MUST be able to (
- Gross area < 1.5 x

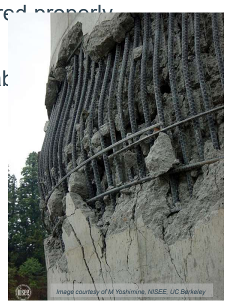
Confinement and buckling

- Interaction between effects
  - Difficult to quantify
  - Buckling governs for low axial loads (and beams)



Transverse reinforcement – anchorage

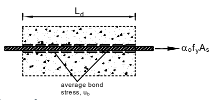
- VITAL that spirals are anchored properly
- Lap splices are NOT acceptab
- Three permitted methods
  - 135° hooks engaging longitudinal bar
  - welded splices
  - mechanical couplers



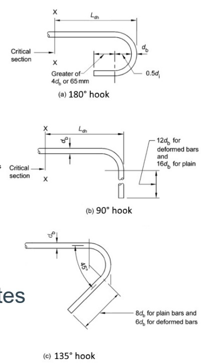
Anchorage

- Straight development
  - bar anchored by bond

$$L_d \geq \frac{d_b}{4u_b}\alpha_o f_y$$

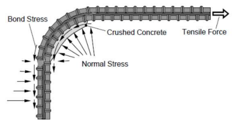


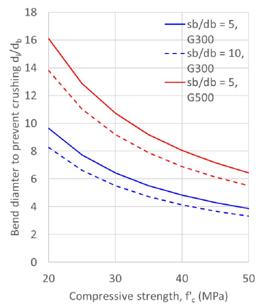
- Standard hooks
- Mechanical anchorage
  e.g. welded or threaded footplates



Hooked anchors

- Remember to check bend diameters!





Lap splices

- Severely detrimental to hinges
  - Progressive unzipping
  - Stress raiser at bar terminations
  - Concentration of yielding = less rotation capacity

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Stiffness of concrete members

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Stiffness of concrete members

- Most commonly means ‘flexural’ stiffness
  - shear stiffness typically high, important in some cases
  - axial stiffness typically less important
- Flexural stiffness is determined by *EI*, not just *I*

$$M = \underline{EI}\phi$$

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Concrete material stiffness

- Young’s modulus related to compressive strength

$$E_c = 3320\sqrt{f_c'} + 6900$$

- Typically take $f_c' + 10MPa$ in above
  - estimate of average stiffness
- Considerable scatter!

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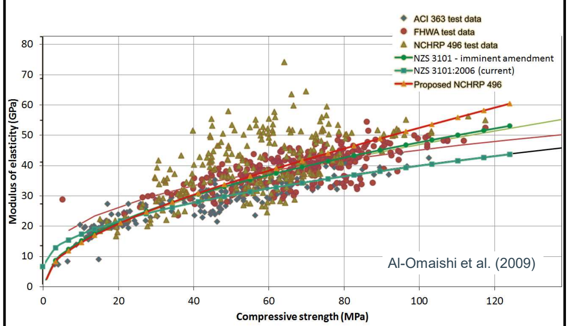
Concrete material stiffness



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NZS 3101 effective stiffness

Type of member	Ultimate limit state		Serviceability limit state		
	f_y = 300 MPa	f_y = 500 MPa	μ = 1.25	μ = 3	μ = 6
1 Beams					
(a) Rectangular [¶] (valid range)	0.43 I_g 0.5% ≤ p ≤ 1.75%	0.32 I_g 0.3% ≤ p ≤ 1.4%	I_g	0.7 I_g	0.50 I_g
(b) T and L beams [¶] (valid range)	0.40 I_g 0.75% ≤ p ≤ 1.75%	0.30 I_g 0.45% ≤ p ≤ 1.75%	I_g	0.7 I_g	0.50 I_g
2 Columns					
(a) $N^*/A_g f_c' > 0.5$	0.80 I_g (1.0 I_g) [‡]	0.80 I_g (1.0 I_g) [‡]	I_g	1.0 I_g	As for the ultimate limit state values in brackets
(b) $N^*/A_g f_c' = 0.2$	0.55 I_g (0.66 I_g) [‡]	0.50 I_g (0.66 I_g) [‡]	I_g	0.8 I_g	
(c) $N^*/A_g f_c' = 0.0$	0.40 I_g (0.45 I_g) [‡]	0.30 I_g (0.35 I_g) [‡]	I_g	0.7 I_g	
3 Walls [¶]					
(a) $N^*/A_g f_c' = 0.2$	0.48 I_g	0.42 I_g	I_g	0.7 I_g	As for the ultimate limit state values
(b) $N^*/A_g f_c' = 0.1$	0.40 I_g	0.33 I_g	I_g	0.6 I_g	
(c) $N^*/A_g f_c' = 0.0$	0.32 I_g	0.25 I_g	I_g	0.5 I_g	
4 Coupling beams					
Diagonal and conventional reinforcement [*]	$0.40 I_g \alpha_c$	$0.33 I_g \alpha_c$	I_g	0.5 I_g	As for the ultimate limit state values

NOTES –

(‡) The values in brackets apply to columns which have a high level of protection against plastic hinge formation in the ultimate limit state.

(¶) For additional flexibility, within joint zones and for conventionally reinforced coupling beams refer to the text.

(*) The value of α_c is given in Table C6.6

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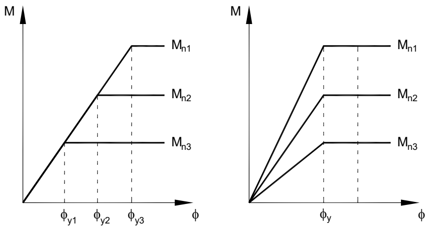
Is constant stiffness realistic?



(a) tradition - assumption of constant member stiffness

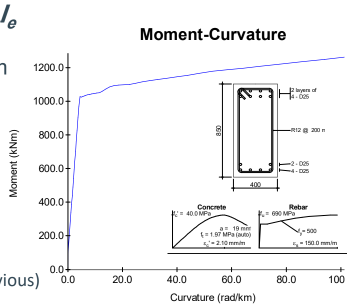
(b) realistic - assumption of constant yield curvature

Determining E_cI_e

- Analysis of beam
  - $M_n \approx 1050$ kNm
  - $\phi_y \approx 4.5\times10^{-6}$

$$\frac{E_cI_e}{E_cI_g} = \frac{M_n}{\phi_y E_c I_g}$$

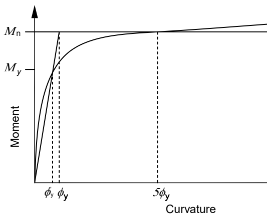
  - Therefore ≈ 0.41 (cf 0.40 from previous)



'Complex' sections

- For columns, walls etc, response is complicated by progressive yielding of reinforcement
- Define 'corner' point based on:
  - first reinforcement yield
  - nominal strength

$$\phi_y = \frac{M_n}{M_y}\phi'_y$$

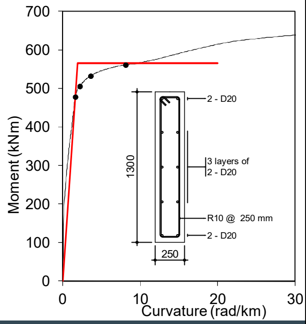


'Complex' sections

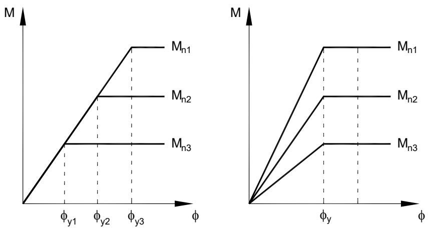
- Example
  - My ≈ 478 kNm
  - $\phi'_y = 1.62\ rad/km$

$$\phi_y = \frac{M_n}{M_y}\phi'_y = \frac{574}{478}\times 1.62$$
$$= 1.94\ rad/km$$

- Thus $\frac{E_cI_e}{E_cI_g} = 0.23$



What about this?

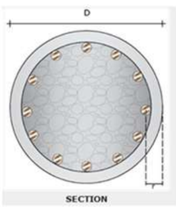


(a) tradition - assumption of constant member stiffness

(b) realistic - assumption of constant yield curvature

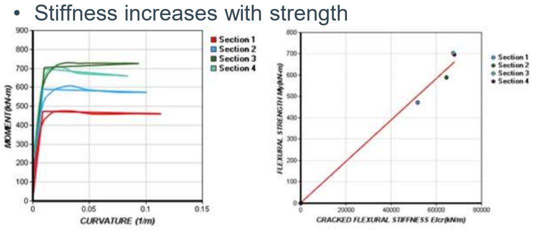
Parametric analysis

SECTION NUMBER:	1	2	3	4
			SECTION PROPERTIES	
SECTION DIAMETER D(mm)	600	600	600	600
COVER UP MAIN TO REINFORCEMENT REBAR CENTRE r (mm)	50	50	50	50
DIAMETER OF MAIN REBARS (mm)	20	20	20	20
# BARS OF MAIN REINFORCEMENT	8	12	16	12
DIAMETER OF TRANSVERSE REINFORCEMENT(mm)	10	10	10	10
SPACING OF TRANSVERSE REINFORCEMENT (mm)	150	150	150	150
TYPE OF TRANSVERSE REINFORCEMENT	Spirals	Spirals	Spirals	Spirals
AXIAL LOAD (kN)	900	900	900	1800
SHEAR SPAN (mm)	2500	2500	2500	2500
			MATERIAL PROPERTIES	
CONCRETE COMPRESSIVE STRENGTH (MPa)	35	35	35	35
LONG STEEL YIELDING STRESS (MPa)	500	500	500	500
TRANSVERSE STEEL YIELDING STRESS (MPa)	500	500	500	500
STEEL HARDENING RATIO	1.25	1.25	1.25	1.25



Parametric analysis results

- Yield curvature approximately constant
- Stiffness increases with strength



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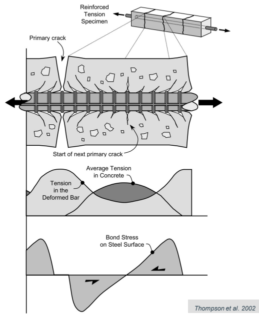
Member vs section stiffness

- Stiffness from MC analysis is <u>cracked section</u>
- Members are stiffer than this due to tension stiffening



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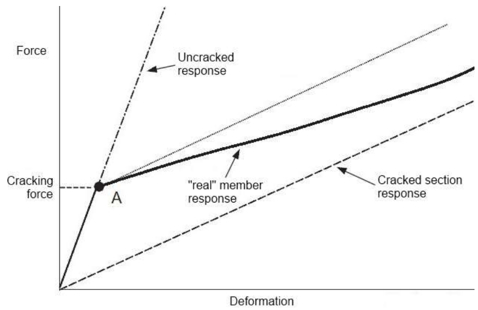
Member vs section stiffness



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Fundamentals of Reinforced Concrete Seismic Design

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www.nzsee.org.nz

Sesoc
www.sesoc.org.nz

Presenters

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Compusoft Engineering Ltd

Nicholas Brooke
Compusoft Engineering Ltd

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Fundamentals of Reinforced Concrete Seismic Design

Session 5:
Seismic Analysis

Barry Davidson
Courtesy of Compusoft Engineering

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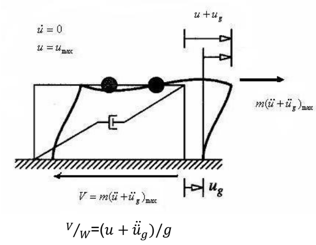
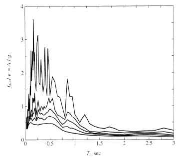
Inertia Forces as Equivalent Static Forces





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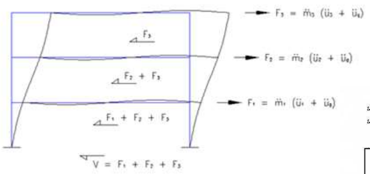
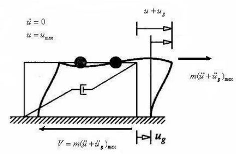
Inertia Forces as Equivalent Static Forces



Difficulty



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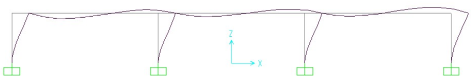
Degrees of Freedom

One Level Structure



Assuming –
Rigid Floor diaphragm

One Degree of Freedom-
The shape is fixed
Amplitude of all/any action is defined by the value of floor displacement

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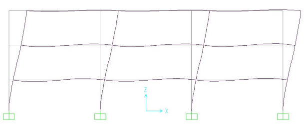
Degrees of Freedom



- Three level with rigid floor assumption

Three Degrees of Freedom-
The shape is defined by amplitude of Three floors
All/any action is defined by knowledge Of three floor displacements

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Modeshapes and Frequencies

- When structures vibrate, whether from seismic or other loads, the deflected shapes they take can be shown to be made up from proportions of “shapes” called their modeshapes.

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Modeshapes and Frequencies

- When structures vibrate, whether from seismic or other loads, the deflected shapes they take can be shown to be made up from proportions of “shapes” called their modeshapes.
- The modeshapes depend **only** on the stiffness and mass of the structure and are independent of the source of vibration.

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Modeshapes and Frequencies

- When structures vibrate, whether from seismic or other loads, the deflected shapes they take can be shown to be made up from proportions of “shapes” called their modeshapes.
- The modeshapes depend **only** on the stiffness and mass of the structure and are independent of the source of vibration.
- The magnitude of the vibration and the relative contribution of each modeshape to the defected shape of the structure at any time is dependent of the vibration source.

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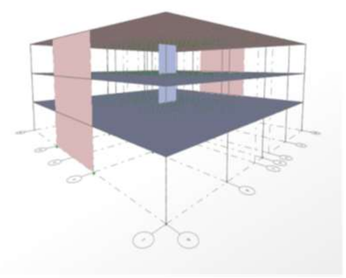
Modeshapes and Frequencies (Example)



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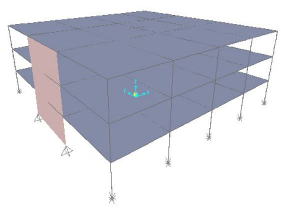
Mode 1



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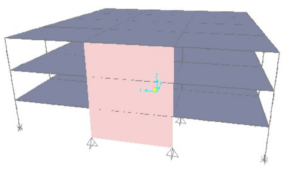
Mode 2 (Mode 1 Y)



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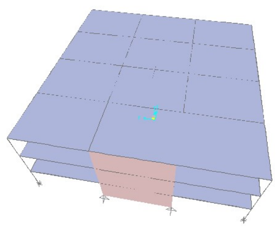
Mode 3 (Mode 1 Rotation)



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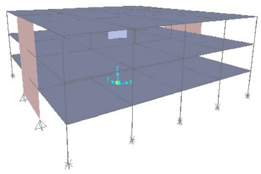
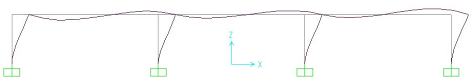
Mode 4 (Mode 2 X)



Modeshapes

One Level Structure

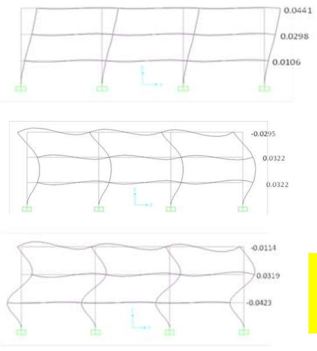


Assuming –
Rigid Floor diaphragm
All mass lumped at floor level

One Modeshape-
The shape is fixed

Amplitude of all/any action is defined by the value of floor displacement

Modeshapes



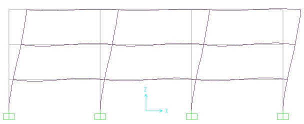
Three Level Structure

Assuming –
Rigid Floor diaphragm
All mass lumped at floor levels

Three Modeshapes-
The shape of each is fixed.

For each mode,
The amplitude of all/any action is defined by the value of a floor displacement

Analysis



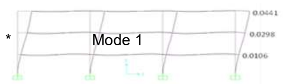
- Three level with rigid floor assumption

- Requires the displacements of three floors to complete the description and allow actions to be defined

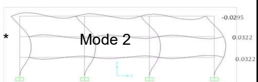
OR

Analysis

α *

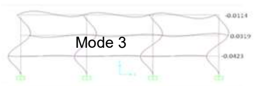


β *

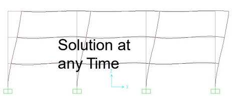


+

+ γ *

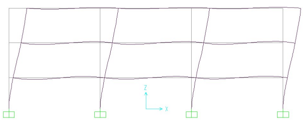


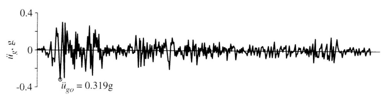
=



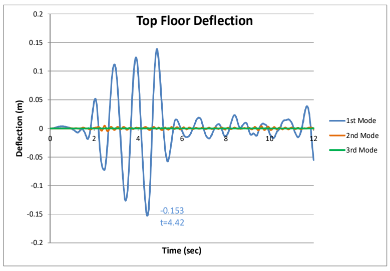
- Requires the components of three shapes to complete the description and allow actions to be defined

Modal Time History Analysis of Three Storey Frame to El Centro



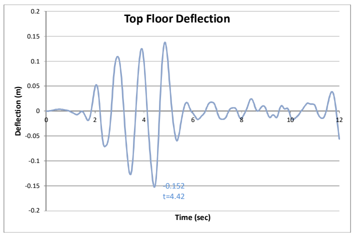


Modal Time History Analysis of Three Storey Frame to El Centro



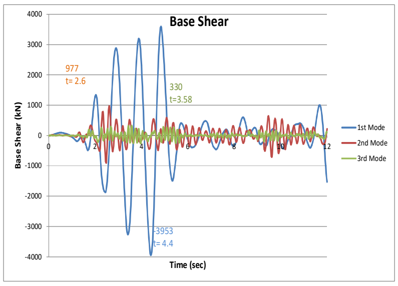
Modal components of top floor displacement

Modal Time History Analysis of Three Storey Frame to El Centro



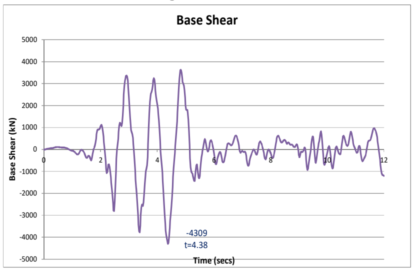
Combined solution (at each time step)
- Each modal component is added at each time step

Modal Time History Analysis of Three Storey Frame to El Centro



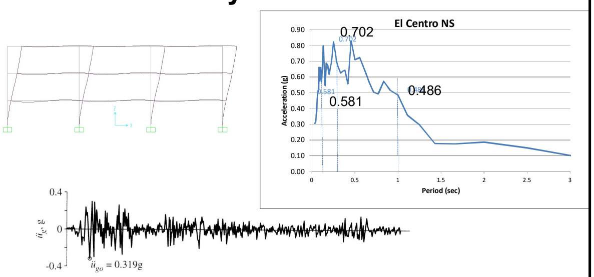
Modal components of base shear

Modal Time History Analysis of Three Storey Frame to El Centro

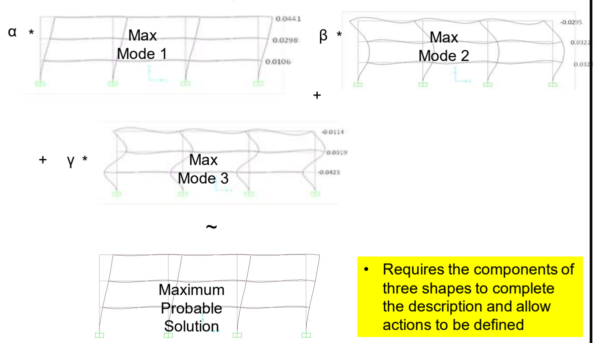


Combined solution (at each time step)
- Each modal component is added at each time step

Modal Response Spectrum Analysis of Three Storey Frame to El Centro



Modal Response Spectrum Analysis of Three Storey Frame to El Centro

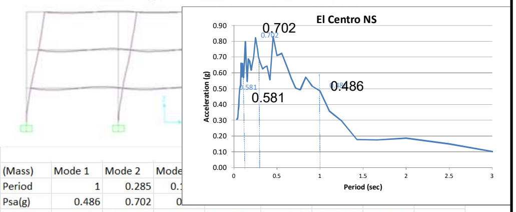


- Requires the components of three shapes to complete the description and allow actions to be defined

Modal Response Spectrum Analysis of Three Storey Frame to El Centro

- Combination Procedures
  - These spectral responses are combined to obtain **maximum probable** responses.
  - The NZS1170.5 recommends two methods
    - The Square Root of Sum of Squares (SRSS)
    - The Complete Quadratic Combination (CQC). Thus using the SRSS method the **maximum probable** responses for:
- .

Modal Response Spectrum Analysis of Three Storey Frame to El Centro



Level	(Mass)	Mode 1	Mode 2	Mode					
	Period	1	0.285	0.1					
	Psa(g)	0.486	0.702	0					
3	339.7893	0.0441	0.0295	0.0114	0.1529152	-0.01739	0.001847	0.153912	0.152
2	339.7893	0.0298	-0.0322	-0.0319	0.1033304	0.018984	-0.00517	0.105187	
1	339.7893	0.0106	-0.0322	0.0423	0.0367551	0.018984	0.006853	0.041932	
P		28.7122	-11.8586	7.407407					

Modal Response Spectrum Analysis of Three Storey Frame to El Centro

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    - The Square Root of Sum of Squares (SRSS)
    - The Complete Quadratic Combination (CQC). Thus using the SRSS method the **maximum probable** responses for:
- Top floor deflections would be $\sqrt{(0.153)^2} = 0.153m$, cf. -0.152m.

Modal Response Spectrum Analysis of Three Storey Frame to El Centro

- $F_j^i = m_j \emptyset_j^i P_i S_{Ai}$
- $P_i = {\sum_j \emptyset_j^i m_j} \Big/ {\sum_j {\emptyset_j^i}^2 m_j}$ modal participation factor

						Inertia Force (kN)				
Level	(Mass)	**Mode 1**	**Mode 2**	**Mode 3**		**Mode 1**	**Mode 2**	**Mode 3**	**Max Prob**	**T/H**
	Period	1	0.285	0.1515						
	Psa(g)	0.486	0.702	0.581						
3	339.8	0.0441	0.0295	0.0114		2051	-819	164	2215	
2	339.8	0.0298	-0.0322	-0.0319		1386	894	-458	1711	
1	339.8	0.0106	-0.0322	0.0423		493	894	607	1187	
P		28.7122	-11.8586	7.407407	Base Shear	3930	968	313	4060	4309

Modal Response Spectrum Analysis of Three Storey Frame to El Centro

- Combination Procedures
  - These spectral responses are combined to obtain **maximum probable** responses.
  - The NZS1170.5 recommends two methods
    - The Square Root of Sum of Squares (SRSS)
    - The Complete Quadratic Combination (CQC). Thus using the SRSS method the **maximum probable** responses for:
- Base shear $\sqrt{3930^2 + 968^2 + 313^2} = 4060\ kN$ cf. -4309 kN.

Modal Response Spectrum Analysis General Comments

- Comments

– Signs
  - Response Spectrum Influence
  - Combination procedure Influence

– Derived Actions and Responses
  - As we calculate **Maximum Probable Responses**, then other actions derived from these **may**, or **may not** be maximum probable values

Modal Response Spectrum Analysis of

Inertia Forces					Interstorey Shears			
Mode 1	Mode 2	Mode 3	Max Prob	From MP IF	Mode 1	Mode 2	Mode 3	Max Prob
2051	-819	164	2215					
1386	894	-458	1711	2215	2051	-819	164	2215
493	894	607	1187	3926	3437	75	-294	3450
				5114	3930	968	313	4060

						Inertia Force (kN)				
Level	(Mass)	Mode 1	Mode 2	Mode 3		Mode 1	Mode 2	Mode 3	Max Prob	T/H
	Period	1	0.285	0.1515						
	Psa(g)	0.486	0.702	0.581						
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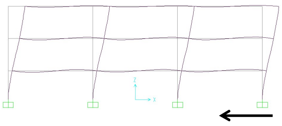
Modal Response Spectrum Analysis General Comments

2215=(2015^2+(-819)^2+164^2)^0.5 →



1711 →

1187 →

← 5114=2215+1711+1187

Inertia Forces					Interstorey Shears			
Mode 1	Mode 2	Mode 3	Max Prob	From MP IF	Mode 1	Mode 2	Mode 3	Max Prob
2051	-819	164	2215					
1386	894	-458	1711	2215	2051	-819	164	2215
493	894	607	1187	3926	3437	75	-294	3450
				5114	3930	968	313	4060

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Comments on- The NZS1170.5 Modal Response Spectrum Method

Percentage Mass (90% required !)

$V_i = P_i^2 S_{Ai}$ if $(\sim S_{Ai} = g)$, then

With reference to F=ma, P_i^2 is mass in ith mode

Level	(Mass)	Mode 1	Mode 2	Mode 3
	Period	1	0.285	0.1515
	Psa(g)	0.486	0.702	0.581
3	339.8	0.0441	0.0295	0.0114
2	339.8	0.0298	-0.0322	-0.0319
1	339.8	0.0106	-0.0322	0.0423
P		28.7122	-11.8586	7.407407
P^2		824.3903	140.6275	54.86968
% Mass		80.87	13.80	5.38

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Comments on- The NZS1170.5 Modal Response Spectrum Method

- **Base Shear Scaling**

– The problem arises in that the building being analysed most likely does not have a large amount of mass participating in its first mode

– It would be more rational to scale to equivalent static base shear calculated at the period of the mode that introduced say > 60% of the mass of the building.

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The Equivalent Static Method

- Observed from previous example that the response of the three level regular frame was dominated by 1st Mode

Storey Displacement (m)					Interstorey Shear (kN)				
Mode 1	Mode 2	Mode 3	Max Prob	M1 Error(%)	Mode 1	Mode 2	Mode 3	Max Prob	M1 Error(%)
0.153	-0.017	0.002	0.154	0.648	2051	-819	164	2215	7
0.103	0.019	-0.005	0.105	1.765	3437	75	-294	3450	0
0.037	0.019	0.007	0.042	12.346	3930	968	313	4060	3

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The Equivalent Static Method

- Assume for Regular low rise buildings

– First Mode Dominates response

– First mode takes a pre guessed shape (NZ a straight line)

- $\emptyset_j^1 = \alpha h_j$
- Then substituting into previous equations for Modal Response Spectrum analysis

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The Equivalent Static Method

- $F_j = {w_j h_j V}/{\sum_j w_j h_j}$
  - $V_1 = P_1^2 S_{A1}$

- $F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}$
  - $V = C_d(T_1)W_t$

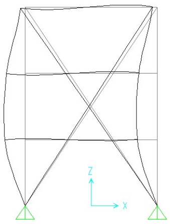
(Usually P^2 ~ 0.8*Wt)

The Equivalent Static Method

- **What if you use the wrong modeshape?**
  - ESM (assumes linear mode) is restricted for structures
- Have a height less 10m, or
- Have a period less 0.4 seconds, or
- Are regular and have a period less than 2 seconds

The Equivalent Static Method

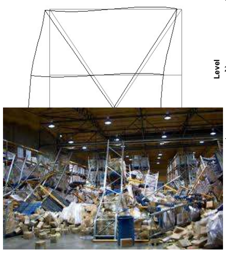
- Steel Storage Rack - braced

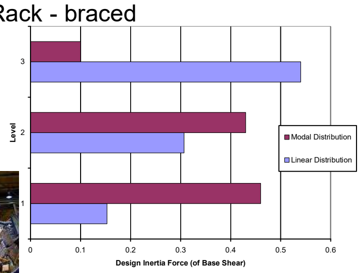


Expected First Modeshape

The Equivalent Static Method

- Steel Storage Rack - braced





Fundamentals of Reinforced Concrete Seismic Design

Session 6:
Walls, Elongation, and
Displacement-Sensitive Elements

Nic Brooke

NZCS New Zealand Concrete Society SEMINAR SERIES 2017

Summary

- Walls
  - Detailing
  - Non-planar walls
  - Strut-and-tie design
  - Interpretation of analysis
- Plastic hinge elongation
- Design of displacement-sensitive items
  - Stairs
  - Cladding

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Walls

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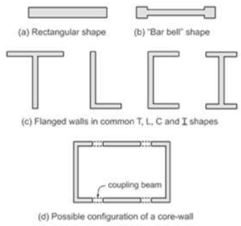
Walls are little different

- Compared to beams and columns, walls:
  - Are typically 'long'
    - small axial loads create significant flexural strength
    - often low reinforcement requirement
  - Need to have bars along length
  - More flexibility in section shape



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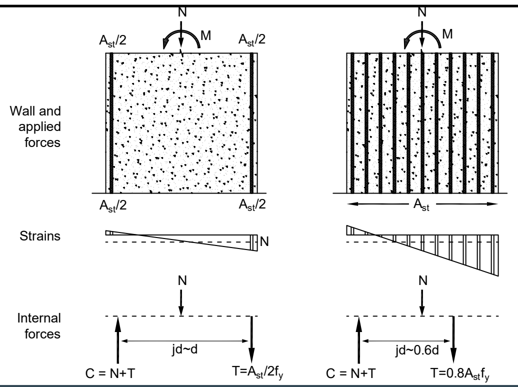

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Concentrating reinforcement

- Promotes distributed cracking
  - locally higher reinforcement ratio acts as initiator
- Results in stiffer section
  - more EA at a bigger lever arm
- Other construction benefits
  - transfer to foundations
  - effective confinement simpler

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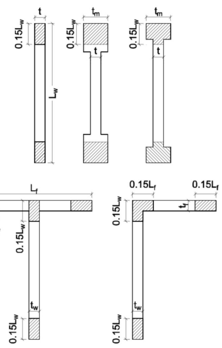
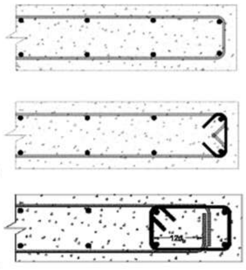
NZS 3101 requirements

- Increased minimum reinforcement requirement for end regions
  - End regions $\rho_{min} = \dfrac{\sqrt{f'_c}}{2f_y}$
  - Elsewhere $\rho_{min} = \dfrac{\sqrt{f'_c}}{4f_y}$



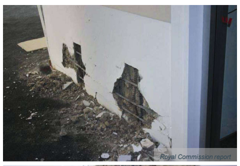
Transverse reinforcement

- Provides shear strength, confinement, prevents buckling – as in other members
- New requirements for anchorage
  - Continuous U
  - Hooks and crosslink
  - Hooks within 'column' cage

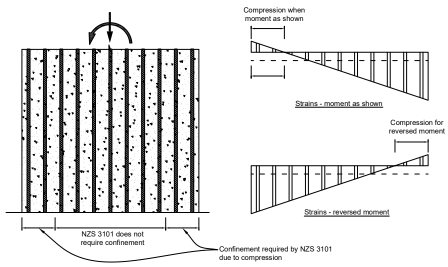


Wall confinement

- Very important!
- Recent realisation: it's not only the end regions that need confining



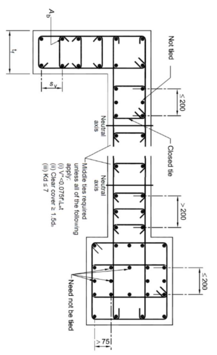
Wall confinement



- What about when the wall is 'in between'?

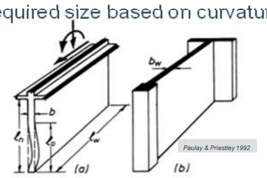
'Constrainment' reinforcement

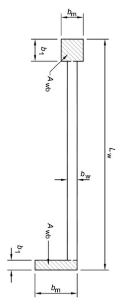
- Prevents buckling of central bars
- Required where spalling anticipated:
  - Fully ductile plastic hinge
  - Shear demand is high
  - Close spaced vertical reinforcement
  - Cover concrete is thin



Boundary regions

- Stabilise entire wall
- Allow well confined end regions
- Flanges can fill role
- Required size based on curvature



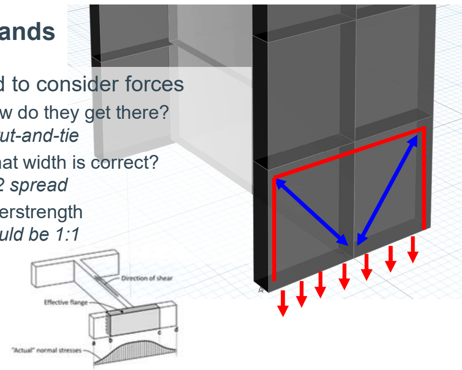


Singly reinforced walls

- Cannot be effectively confined
- Stability under bi-directional loading uncertain
- Consequently limited to nominally ductile design
- Φ = 0.7 in upcoming amendment
- Intent of combined changes is for little plasticity to occur during MCE

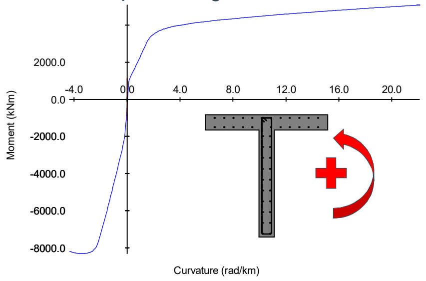
Outstands

- Need to consider forces
  - how do they get there? *strut-and-tie*
  - what width is correct? *1:2 spread*
  - overstrength *could be 1:1*

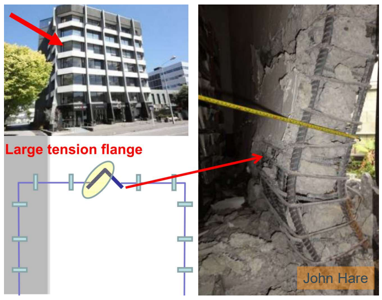


Asymmetric sections

- Watch out for unequal strength

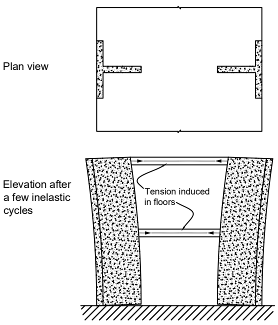


Asymmetric sections



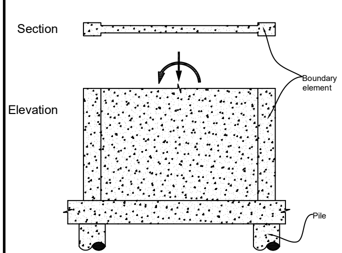
Asymetric sections

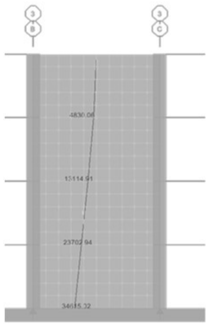
- Solutions can be hard or easy
  - design for the forces
  - reconsider the reinforcement arrangement



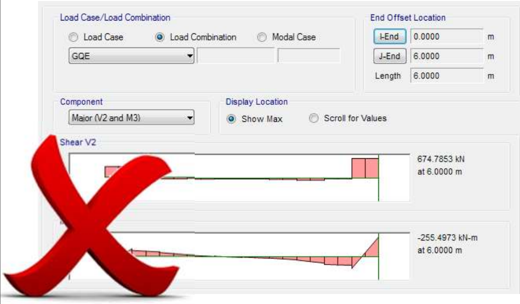
Wall analysis output

- Tread carefully when modelling ground beams etc.





Ground beam actions?



Ground beam actions

- Wall requires 2-HD20 @ 250 c/c
- At nominal strength, all bars in web at yield
  → UDL on the beam

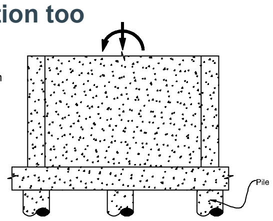
$$F_{dist} = \frac{n_b A_b f_y}{s_b} = 2\times\frac{20^2\pi}{4}\times500\times\frac{1000}{250} = 1257kN/m$$

- V* ≈ 3000-4000 kN   500%-700% of analysis

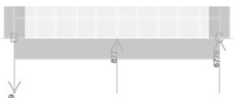
(or lump reinforcement!)

Piles need consideration too

- Third pile added:

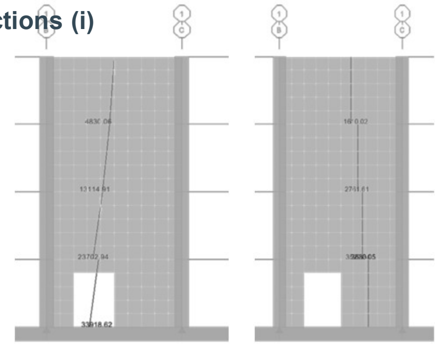


- From analysis:

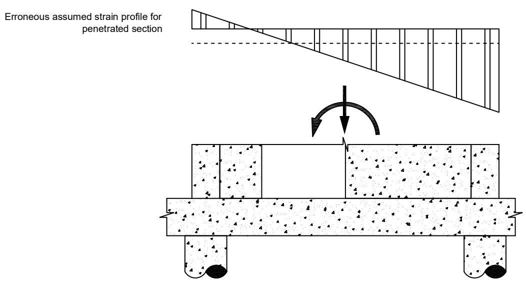


- Based on UDL from bars yielding

Actions (i)

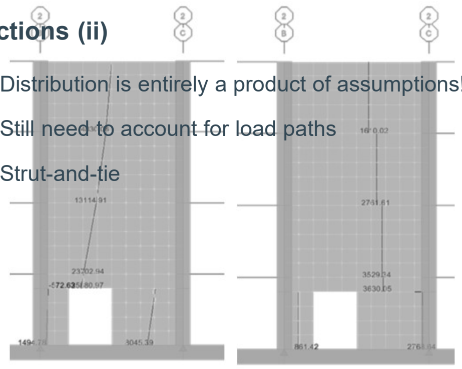


Wrong approach



Actions (ii)

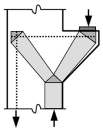
- Distribution is entirely a product of assumptions!
- Still need to account for load paths
- Strut-and-tie



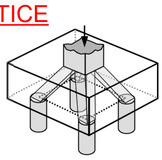
Strut-and-tie

- Every engineer should be using strut-and-tie!
  - walls
  - diaphragms
  - corbels (and seating)
  - foundations
  - unusual connections etc.

} BEST PRACTICE



iii. Corbel



iv. Deep Pile Cap

16.4.4 *Method of design*

Brackets and corbels shall be designed by either method (a), or method (b) where appropriate:

(a) Brackets or corbels with a span to effective depth ratio (a/d) of 1.8 or less may be designed by the strut and tie method.

11.4.8 *Walls with openings*

Openings in structural walls shall be so arranged that unintentional failure planes across adjacent openings, do not reduce the shear or flexural strength of the structure. For ductile cantilever walls with irregular openings appropriate analyses such as based on strut-and-tie models shall establish rational paths for the internal forces. Capacity design procedures shall be used to ensure that the horizontal shear reinforcement will not yield before the flexural strength of the wall is developed.

14.3.2 *Design of pile caps*

Pile caps shall be designed using either flexural theory or a strut-and-tie approach.

13.3.9 *Strength of diaphragms in shear*

The strength design of diaphragms for shear shall be based upon strut and tie models in accordance with Appendix A.

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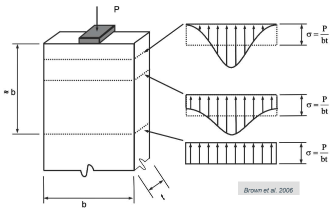
Why S+T?

- Design approaches assume plane sections before deformation remain plane after deformation (Bernoulli)
- Doesn't hold in 'D-regions'



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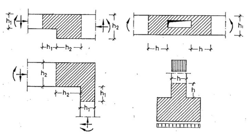
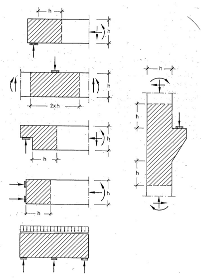
Where S+T

- Anywhere there is a substantial discontinuity



…unless there is a well validated alternative



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Lets go back to session 2

- Clause 1.1.1.1
  - *This Standard sets out minimum requirements for the design of reinforced and prestressed concrete structures.*
- Forthcoming amendment:
  - *In addition to these requirements every load or force acting on a structure shall have one or more dependable load paths that can transfer the force to the foundation soils. Each load path shall satisfy the fundamental structural design requirements of equilibrium and displacement compatibility.*

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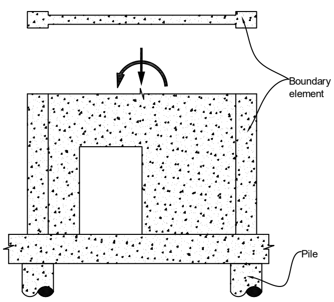
In practice

- Identify D-region(s)
- Determine boundary conditions
- Truss model to transfer forces
- (Reinforcement, detailing nodes etc.)



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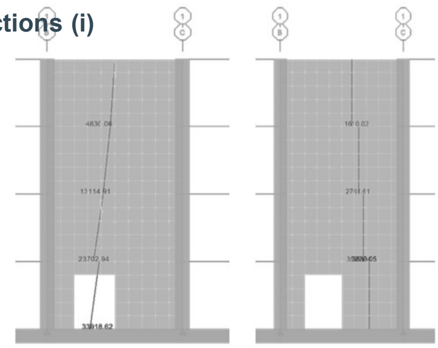
Actions (i)



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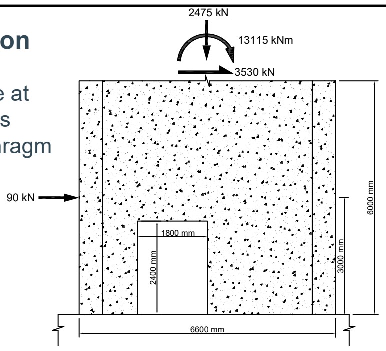
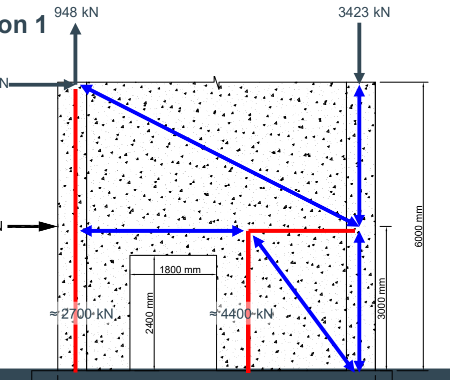
D-region

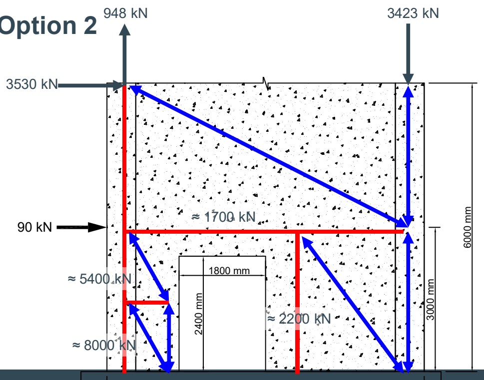
- Force at side is diaphragm force



Option 1

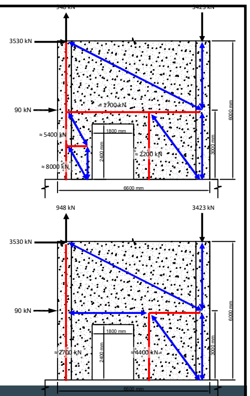


Option 2



Choices

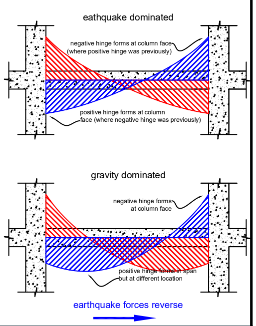
- Influence of ground beam?
- Consider both directions of loading
- Different choices affect the level of damage



Hinge elongation

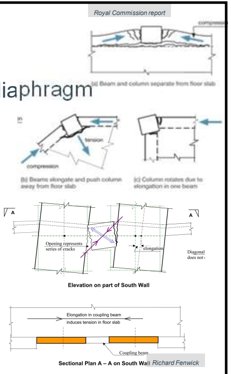
Elongation of plastic hinges

- Plastic hinges elongate during reverse cyclic loading
  - Reinforcement yields in tension but not in compression
  - Cracks wedged by concrete
  - Worse for uni-directional hinges in span – little moment reversal



Effects of Elongation

- Separation of columns from diaphragm
- Seating/connection
  - stairs
  - precast floors
  - cladding
- Change of strength heirarchy
  - frames
  - coupled walls



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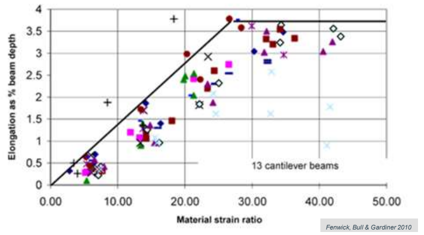
How much?

- Dependent on drift
  - some will occur even in nominally elastic frames



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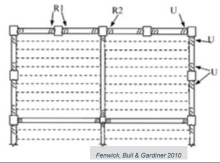
How much?

- Amendment 3
  - Design level/ULS: $\sigma_{el} = 2.6\frac{\theta_p}{2}(d - d') \leq 0.036h_b$
  - Peak MCE: $\sigma_{el,MCE} = \sigma_{el}\frac{1.5}{S_p} \leq 0.036h_b$
- Can be reduced by restraint: e.g. 'R1' – max $0.02h_b$



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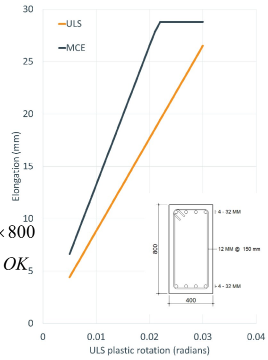
Example

- 800 mm deep beam, 60 mm to reinforcement centroids
- Say $\theta_p = 0.012$

$$\sigma_{el} = 2.6\frac{0.012}{2}(740 - 60') \leq 0.036 \times 800$$
$$= 11mm \qquad \leq 29mm\ OK_S$$



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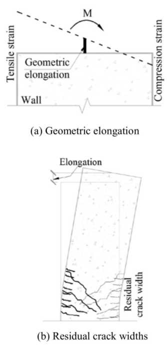
Elongation occurs in walls too

- Causes wall to 'grow' vertically
- Geometric component significant
- Growth can cause wall to 'pick up' more weight
  - Substantial increase in axial force
  - Confinement/buckling?
  - Cannot be predicted yet
  - NZS 3101 – (hopefully) conservative limit on axial force



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Displacement-sensitive items

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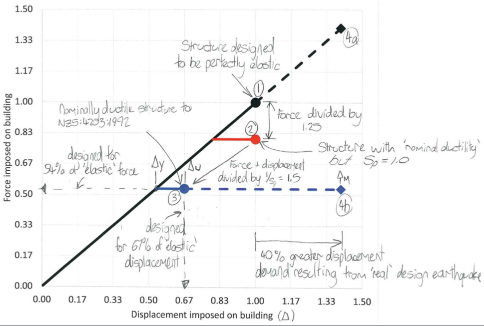
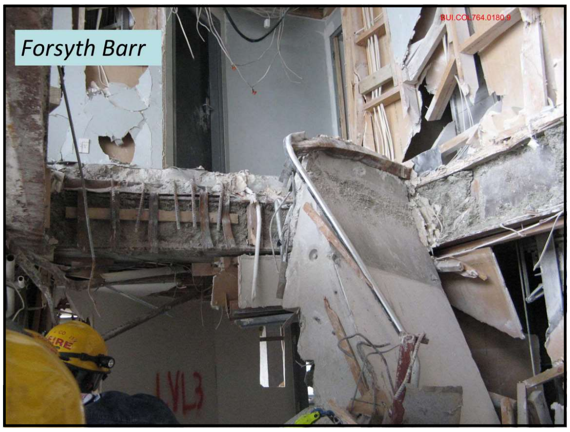
Back to session 2 again



Peak displacement demands

- NZS 1170:
  - *A structure should have a small margin against collapse in the most severe earthquake shaking to which it is likely to be subjected. The maximum considered…*
- For some items it is untenable to assume the peak displacement "doesn't matter"
  - Items that will fall off a ledge at a certain displacement
  - Items that will "jam" – change of load path

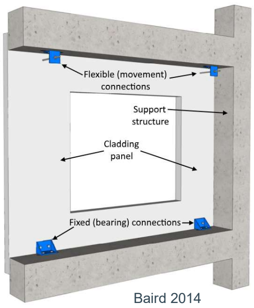


Peak displacement demands

- Amendments to Standards require the 'MCE peak displacement' to be considered
  - Stairs and ramps to be functional
  - No collapse of precast floor units and cladding panels
- Peak MCE defined as: $\Delta_{peakMCE} = \Delta_{ULS} \dfrac{1.5}{S_p}$
  - NZS 1170 says $2/S_p$!!!
- Plus allowance for deformation arising from elongation and/or rotation of the supports

Cladding

- Aim – avoid the cladding becoming a load path
  - Fixed supports at base
  - Movement connections at top
- Connections must be tied into structure – LOADPATHS!
- Connection should be ductile 'weak link'



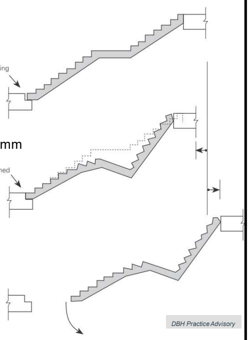
Baird 2014

Stairs

- Substantial displacements to accommodate:
  - 3m storey, 1.5% ULS drift

$$\Delta_{peak,MCE} = \Delta_{ULS}\frac{2}{Sp} = (0.015 \times 3000)\frac{2}{0.7} = 129mm$$

  - Plus 2x hinges elongating – say $0.036h_b$ = 29 mm
  - Total > 200 mm after tolerances
- Opening, closing, and transverse



DBH Practice Advisory

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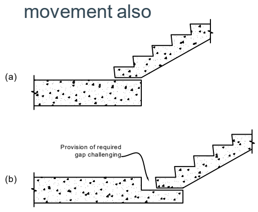
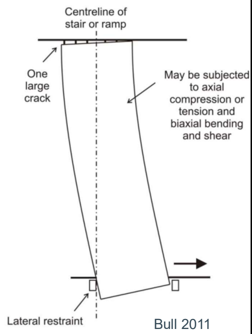
Stairs

- Fixed support at top
- Sliding at base
- Need to consider transverse movement also





Bull 2011

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Fundamentals of Reinforced Concrete Seismic Design

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NZ Society for Earthquake Engineering Incorporated
www.nzsee.org.nz

Sesoc
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Presenters

Barry Davidson
Compusoft Engineering Ltd

Nicholas Brooke
Compusoft Engineering Ltd

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Fundamentals of Reinforced Concrete Seismic Design

Session 8:
Floors and Diaphragms

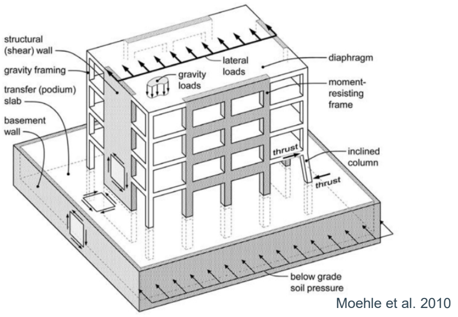
Nic Brooke

NZCS New Zealand Concrete Society SEMINAR SERIES 2017

Overview

- Roles of diaphragms and floors
- Diaphragm demands
- Diaphragm load paths
- Analysis to determine diaphragm forces
- Precast flooring
  - Factors requiring consideration
  - Seating design

Roles of floors and diaphragms

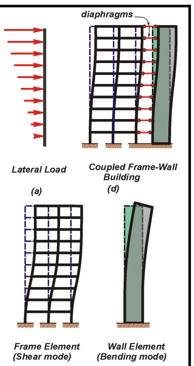


Moehle et al. 2010

Diaphragm demands

Diaphragms must resist forces from three sources

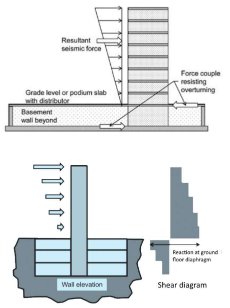
- Inertia
  - Acceleration of the mass of the diaphragm and objects it supports
- Transfer
  - Flow of forces from one vertical element to another – result from changes of stiffness of vertical elements
- Secondary effects
  - Beam elongation
  - Ratchetting of elements
  - etc.



Paulay & Priestley 1992

Kickbacks/backstay

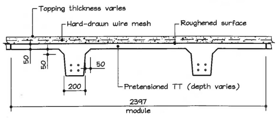
- Demands likely to exceed building base shear
- Consider a cantilever beam with backspan – reaction exceeds applied force

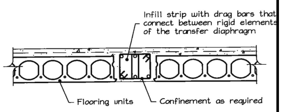


Moehle et al. 2010, Sabelli et al. 2010

Impact of stiffness on demands

- Transfer and kickback forces strongly influenced by stiffness
  - Diaphragm stiffness
  - Relative stiffness of elements
- Complex for precast floors
  - What contribution is mad
  - Different transverse/long





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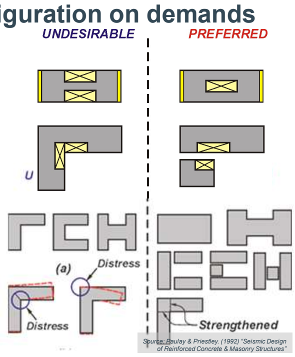
Impact of plan configuration on demands

- Complex shapes lead to concentrated of demands – stress raisers



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Diaphragm load paths

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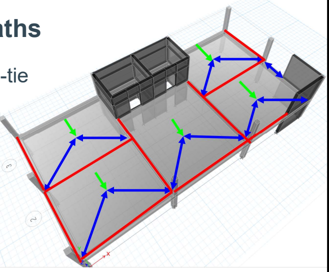
Design of load paths

- MUST use strut-and-tie
- Consider diaphragm as free body:
  - Forces for inertia
  - Reactions at vertical elements
- Nothing magic!
  - Ties – reinforcement on orthogonal axes
  - Struts – concrete
  - ≥ 25-30° degrees between struts and ties
  - Minimise cracking by minimising length of ties, matching elastic pattern of stresses



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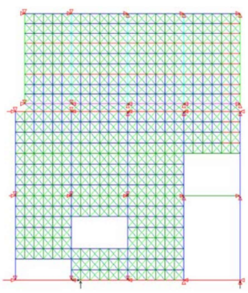
Finite element equivalents

- Various options in software:
  - Grillage
  - Rotating angle smeared crack elements – e.g. Darwin Pecknold



NZSEE 2016

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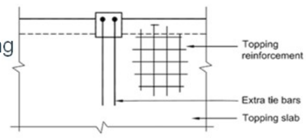
Minimum requirements

- Minimum quantity of reinforcement in orthogonal directions
- ≥75 mm topping on precast floors
- All vertical elements tied into the diaphragms
- ‘Drag’ elements to major vertical resisting elements



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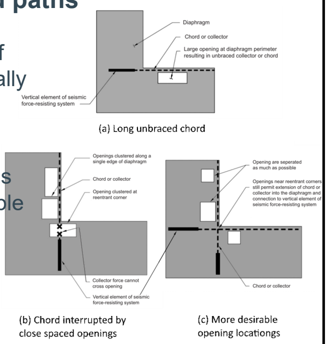
Disruption of load paths

- Size and position of openings can critically affect diaphragm function
- Some configurations are almost impossible to design



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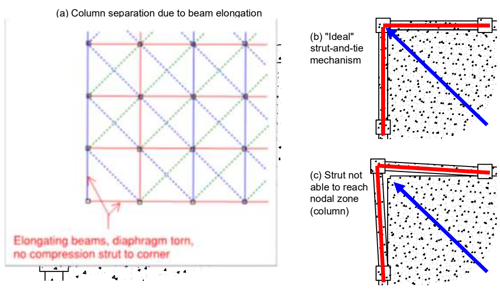
Specific issues



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Analysis to determine diaphragm demands

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'Regular' analysis methods not ideal

- Equivalent static: *non-conservative*
- Response spectrum: *difficult to obtain and interpret useful results*
- Parts and components: *inappropriate – diaphragm interacts with structure*
- (Non-linear) time history: *"perfect"... but*

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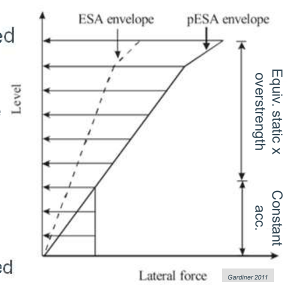
pESA – pseudo Equivalent Static Analysis

- Equivalent static analysis with a modified force envelope
  - Simple to implement
  - Calibrated against time history results
- Applied to whole building
  - Results used for diaphragm design only
  - Transfer forces obtained directly



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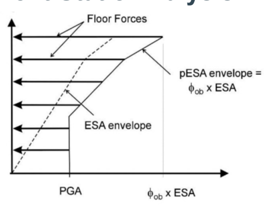
pESA – pseudo Equivalent Static Analysis

- Now covered in detail in NZS 1170.5
  - Limited to 9 storeys – ongoing research
  - 0.1b not to be included
- Building overstrength:
  - Capacity design – calculate explicitly, 2-4 typical
  - Otherwise – estimated: $\phi_{ob} = \frac{S_{n,prov}}{S_E^*}\frac{0.93}{S_p}\frac{f_{y,95}}{f_{y,nom}}$

$$\approx \frac{\left(\frac{1.2}{\phi}S^*\right)}{S^*}\frac{0.93}{S_p}1.2 = 1.75\ to\ 2.25$$



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Precast concrete flooring

Overview

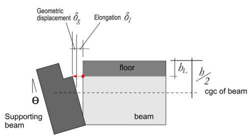
- Significant investigation recently of:
  - hollowcore flooring
  - double tee seating

Everything discussed in research has been seen in Christchurch or Wellington

Seating design

Seating must be sufficient to allow for

- At least 10 mm bearing length
- The length of cover anticipated to be lost,
- The peak MCE (i.e. 1.5 × ULS/Sp) elongation of the parallel beam;
- Peak MCE rotation of the support beam
- 20 mm spalling from end of precast unit;
- Shortening of the precast unit due to creep and shrinkage
- Hogging that may result from thermal effects
- Construction tolerances

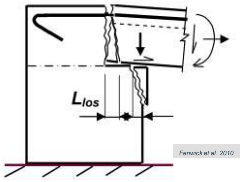


Example

- 275 mm deep floor (e.g. 200 hollowcore with 75 mm topping)
- 900 mm deep support beam
- ULS design plastic rotation is 0.02 radians

Example - spalling

- Spalling of the seat:
  - 25 mm (clear cover)
  - 16 mm (stirrup diameter)
  - 10 mm (half diameter of ledge longitudinal bar)
  - = 51 mm
- 15 mm allowance for spalling from unit



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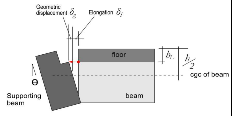
Example – elongation and rotation

- Elongation of parallel beams – length required 32 mm

$$\sigma_{el,peak.MCE} = 2.6\frac{\theta_p}{2}(d-d')\left(\frac{1.5}{S_p}\right) \le 0.036h_b$$
$$= 2.6\frac{0.02}{2}(850-50)\left(\frac{1.5}{0.7}\right) \le 0.036\times 900$$
$$= 46mm \le 32mm$$

- Rotation of support beams
  - Seat is 900/2 – 275 = 175 mm above centroid of beam
  - Peak rotation = 0.036 rad
  - Allowance = 0.036 × 175 = 6 mm



$$\theta_{p,peak.MCE} = \theta_{p,ULS}\frac{1.5}{S_p} \le 0.036$$
$$= 0.02\frac{1.5}{0.7} \le 0.036$$
$$= 0.043 \le 0.036$$

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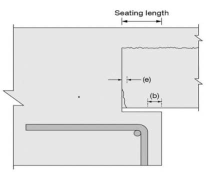
Example

Calculation of required seating length			
(a)	Bearing length	10 mm	
(b)	Spalling of ledge	51 mm	
(c)	Beam elongation	32.4 mm	
(d)	Geometric displacement	6.3 mm	
(e)	Spalling of unit	15 mm	
(f)	Shortening of unit	6 mm	0.6mm/metre
(g)	Tolerances	12.5 mm	NZS 3109
Req. length = sum of above =		134 mm	(round to 135 mm)



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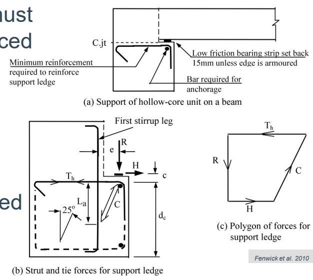
Seating

- Support ledges must always be reinforced
- Design as corbel
- Horizontal force must be considered



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Type specific issues

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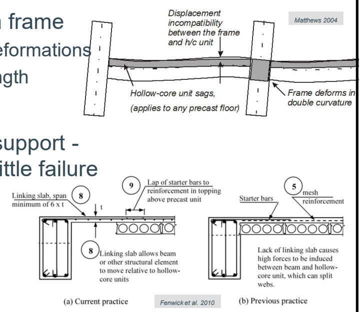
Hollowcore

- Separation from frame
  - Incompatible deformations
  - Unwanted strength enhancement
- Robustness of support - prevention of brittle failure



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Summary of practice

- Fenwick et al. 2010

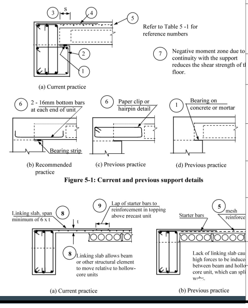


Table 5.1: Detailing practice near supports and adjacent to other structural elements

Reference In Fig.[1]	Reference Text[2]	Recommended practice	Previous practice	Consequences of previous practice
5.1–1	5.4.1	Use low friction bearing strip under hollow-core.	Mount hollow-core unit direct on concrete ledge or on a mortar pad.	Friction between hollow-core unit and support may increase the likelihood of a positive moment flexural crack forming in the units close to the support and of spalling from the support ledge.
5.1–2	5.4.1	Support ledge reinforced to tie it into support beam.	Some times the hollow-core unit was supported on cover concrete.	Spalling from support ledges may occur due to; loss of cover concrete in plastic hinge zones; bond cracking associated with longitudinal beam bars; prying action of precast units on support ledge.
5.1–3	5.4.1	Contact length between hollow-core and ledge to be length for construction tolerance plus the larger of; 75mm, span / 180 or 0.038h_b plus length required for bearing for1,500 < h_b <2,500.	NZS3101: 1995 required contact length greater than the larger of 50mm or span /180*. In earlier Standards no recommended contact lengths were given.	Contact length less than current recommended practice may cause premature loss of support due to spalling and relative movement of hollow-core units to supporting structure.
5.1–4	5.4.3	Starter bars must be extended to cover maximum negative moments induced near supports.	No guidance given on necessary extension of starter reinforcement.	Cutting of the starter bars short can result in a negative moment failure of the hollow-core floor.
5.1–5 5.2-5	5.4.3	Topping concrete to be reinforced with deformed reinforcing bars. Mesh should not be used (this includes ductile mesh).	Topping concrete generally reinforced with mesh.	The mesh has a low ductility, which can cause brittle failure in negative moments or in transmitting diaphragm forces.
5.1–6	5.4.2	Two cells broken out at each end, filled with concrete and reinforced with a plain round 16mm bar in the bottom of each cell.	Where a support length appears to be inadequate two or more cells were broken out and reinforced with paper clip or hairpin form of reinforcement.	The use of paper clip reinforcement increases the likelihood of a negative moment flexural failure at the end of the paper clip bars or where starter bars are terminated.
5.1–7	5.4.4.	Shear strength in negative moment zone checked as required by NZS 3101: 2006	Shear strength assumed to be adequate if it satisfied gravity loading conditions acting as a simply supported member.	Negative moments and axial tension induced in hollow-core floor due to elongation etc reduce the shear strength of the hollow-core units.
5.2–8	5.4.5	Linking slabs required to reduce forces induced in hollow-core units due to relative vertical movement of adjacent structural elements.	No linking slab provided.	Relatively small differential vertical movements between a hollow-core unit and an adjacent structural element causes webs to split, eventually allowing bottom half of unit to fail.
5.2–9	5.4.3	Starter bars lap reinforcing bars in topping above hollow-core units.	Starter bars lap mesh.	At the end of the starter bars there is a sharp drop in negative moment flexural strength.
5.3–10	5.4.6	Assess torsion due to differential displacement of supports.	No guidance given.	Differential displacement of supports can induce torsion in hollow-core units. As these contain no torsional reinforcement this may lead to torsional failure.

Double tees

- Issues around seating load path

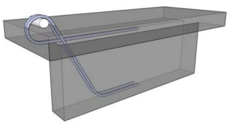
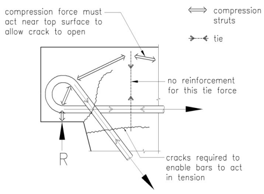


Figure 2 : Pigtail hanger not recommended



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Double tees

- Other options obviously more robust

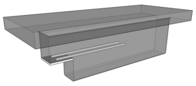

Figure 1 : Dapped end with armouring

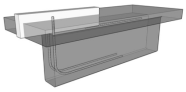

Figure 3 : Cazaly hanger recommended

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