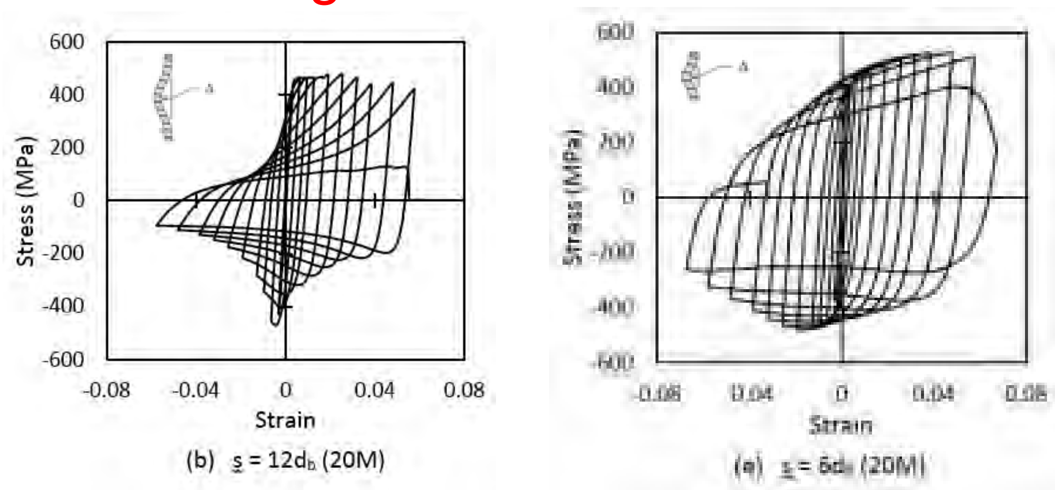
Bar Buckling – Function of s/db



Adomat, N., 2017, M.Eng. Thesis, McGill University
Howard, B., 2017, M.Eng. Thesis, McGill University

Performance Criteria - Damage Indicators

Life Safety: The structure shall not collapse and it shall be possible to evacuate the bridge safely.

- **Probable Replacement**: Bridge shall remain in place, but may be unusable. It may require extensive repair or replacement. Members shall be capable of supporting dead + 30% of live load, excluding impact, but including P-delta effects.
- Concrete Structures: Damage does not cause crushing of the confined concrete core. Reinforcing steel tensile strains shall not exceed 0.075, except that for steel reinforcing of 35M or larger the strains shall not exceed 0.060.

Strain Limits in 2019 CHBDC (S6) (BC MoTI Supplement)

Damage Level	Concrete Strain	Reinforcing Steel Strain
Minimal	$\varepsilon_c < 0.006$	$\varepsilon_s < 0.010$
Repairable	*NS*	$\varepsilon_s < 0.025$
Extensive	$\varepsilon_{cc} < 0.8\ \varepsilon_{cu}$	$\varepsilon_s < 0.05$
Probable Replacement	$\varepsilon_{cc} < \varepsilon_{cu}$	$\varepsilon_s < 0.075$ (30M or smaller) $\varepsilon_s < 0.060$ (35M or larger)

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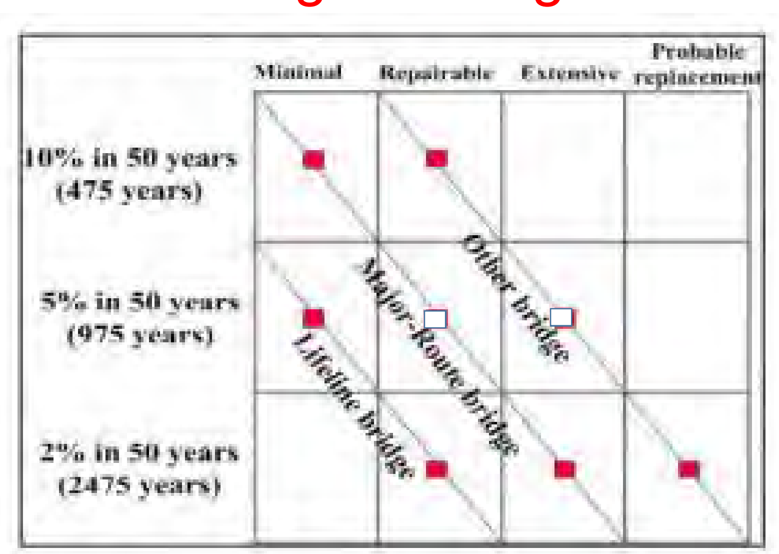
S6-19 Bridge Damage Levels



Required

Not Required

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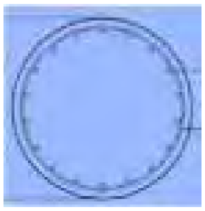
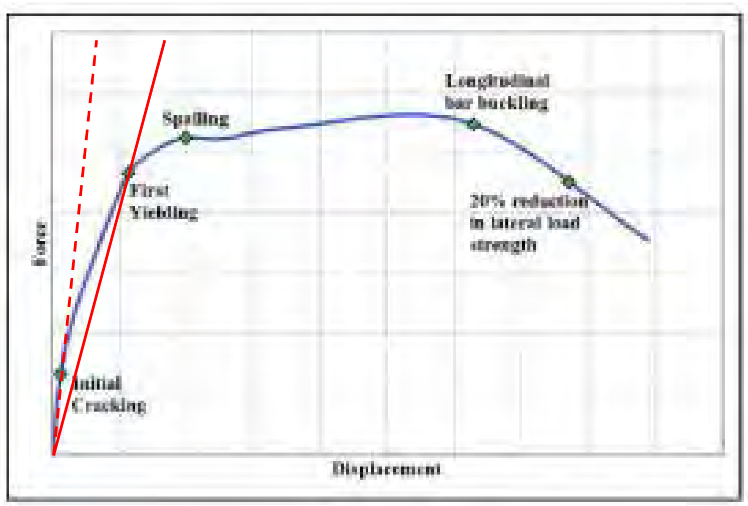
Cracked Stiffness and ISPA

Prediction using Response-2000



unspalled

Spalled Confined concrete



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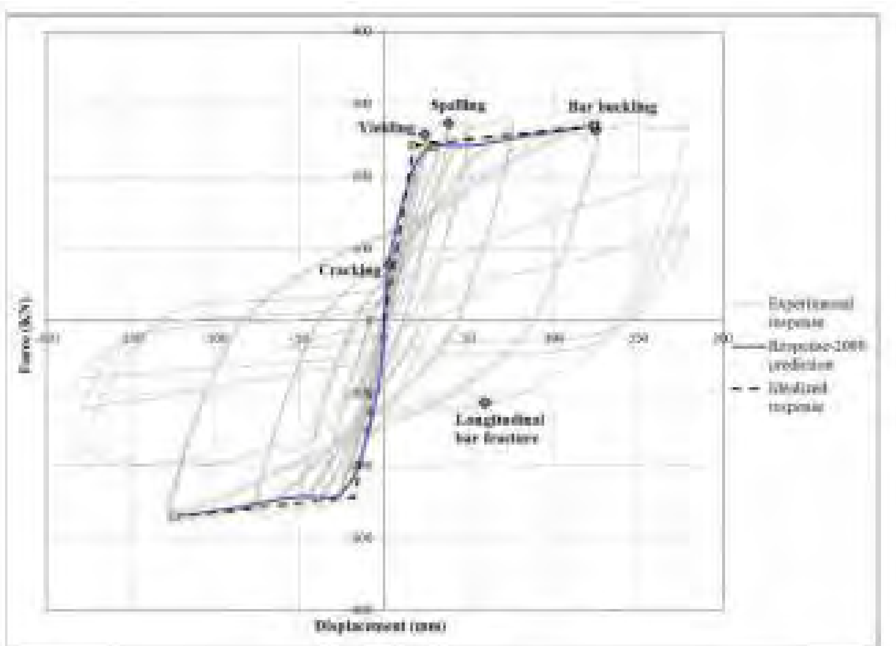
ISPA

Column 415



Lehman, Moehle, 2000, “Seismic Performance of Well-Confined Concrete Bridge Columns”, PEER, Berkeley, CA.

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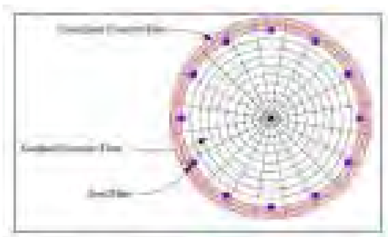
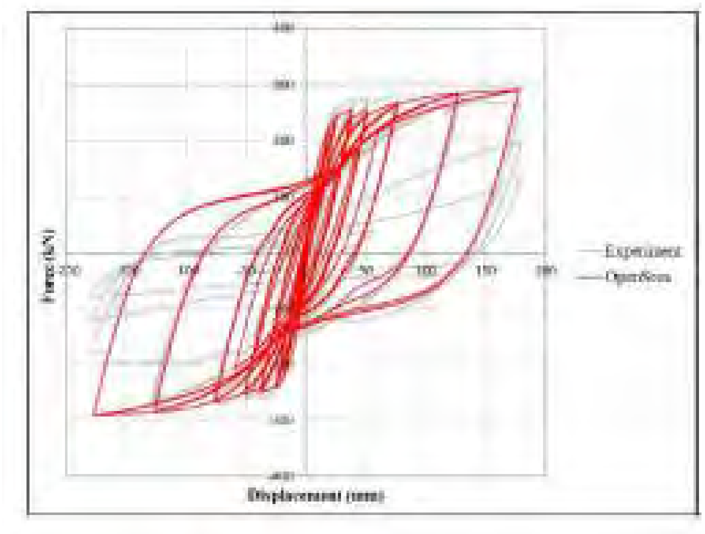
OpenSees

- Fibre Model
- Distributed plasticity





Column 415 tested by Lehman and Moehle, PEER, 2000.

Anghaie, H. "Predicting Seismic Response of Circular Bridge Columns", M.Eng. Thesis, McGill University, 2014.

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Expected Nominal Resistance for Design

- Expected nominal resistance:
- Use ϕ_c and ϕ_s = 1.0 (nominal)
- Expected yield strength:
- f_{ye} =1.2f_y and 1.1f_y (for R< 3)
- Expected concrete compressive strength:

$$f'_{ce} = 1.25 f'_c$$

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Shear – Capacity Design

Design shear, V_f
= shear to hinge column at probable flexural strength

$$V_r = V_c + V_s$$

$$V_c = \phi_c \beta \sqrt{f'_c} b_v d_v$$

$$V_s = \frac{\phi_s f_y A_v d_v \cot \theta}{s}$$

Probable resistance =
Nominal expected resistance x 1.3

Shear Failure Chile 2010

Short Column of Bio-Bio River Bridge

Mitchell, Huffman, Tremblay, Saatcioglu, Palermo, Tinawi, and Lau, "Damage to Bridges due to the February 27, 2010 Chile Earthquake", CSCE J., 40(8), July 2013.

Maximum Moment – Extended Pile Bents

4.7.5.2.4
Plastic hinge region shall be considered to extend from a low point of three times the maximum cross-section dimension below the calculated point of maximum moment, to an upper point at a distance of not less than the maximum cross-section dimension but not less than 500 mm, above the ground line.

Mitchell, Tinawi and Sexsmith. (1991). "Performance of Bridges in the 1989 Loma Prieta Earthquake - Canadian Design Concerns", CJCE, V18, N4.

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Flared Columns

4.7.5.2.4
For flared columns and columns attached to partial - height walls, the top and bottom flares and the height of the walls shall be considered in determining the effective column height.

Note: Flare reduces the effective height, L, of column and hence increases the shear, V for end moment, M

V = M/L

Mitchell, Bruneau, Williams, Anderson, Sexsmith,. (1995). "Performance of Bridges in the 1994 Northridge Earthquake", CJCE, V22, N2.

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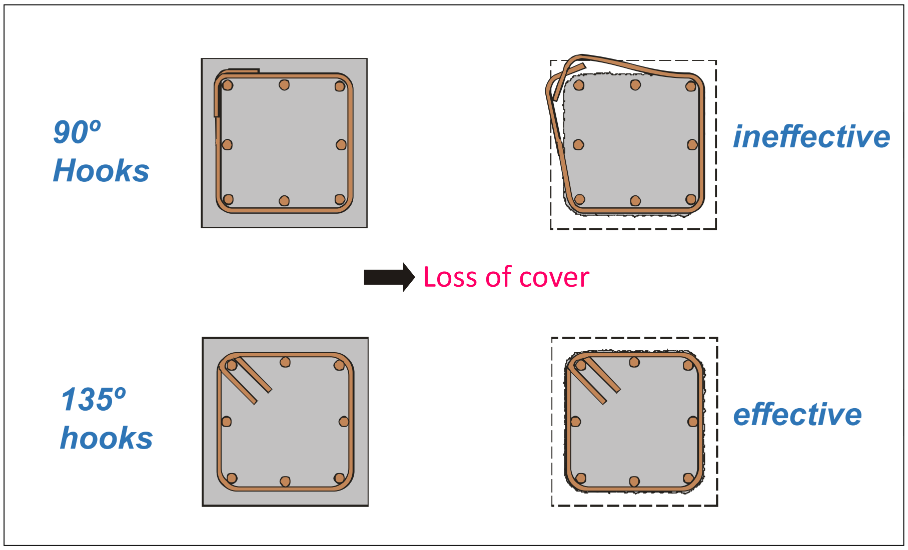
Advantages of Performance-Based Design

- Design based on functional objectives of service and damage states with explicit demonstration of meeting performance criteria
- Allows designers the flexibility of choosing materials and design options
- Provides consistent expectation of structural performance for different levels of seismic events
- Accommodates new technology and innovation

Challenges of Evaluation and Retrofit of Existing Bridges

- Major role played by Regulatory Authority (advisors)
- Remaining service life? (probabilistic approach)
- Performance indicators? (judgement required)
- Damage limits? (depends on specific details)
- Full or partial retrofit? – staged retrofit?

Anchorage Details



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Shear Failure Loma Prieta, CA 1989

Mitchell, Tinawi and Sexsmith. (1991). "Performance of Bridges in the 1989 Loma Prieta Earthquake - Canadian Design Concerns", CJCE, V18, N4.

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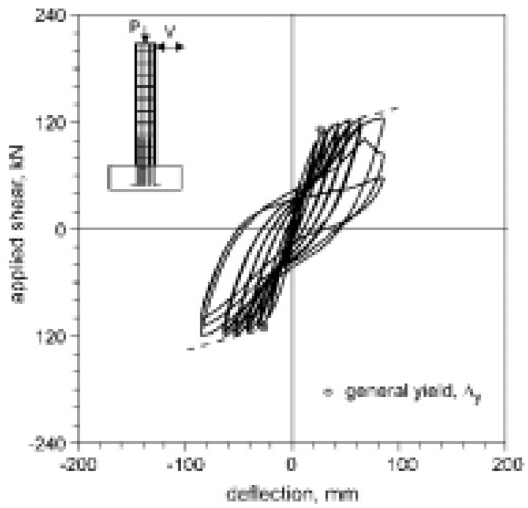
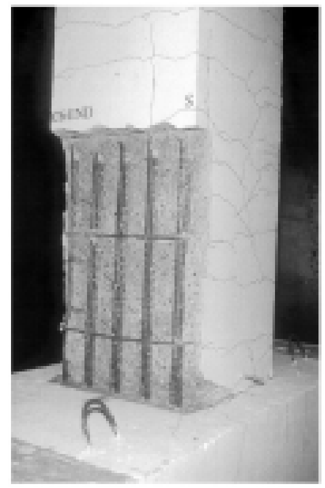
Lap Splices





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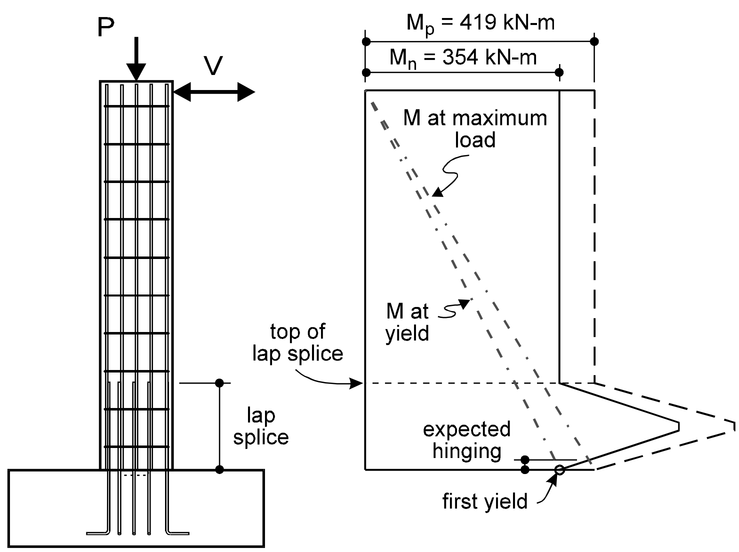
Lap Splices



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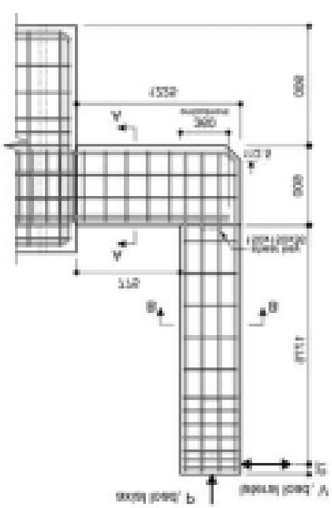
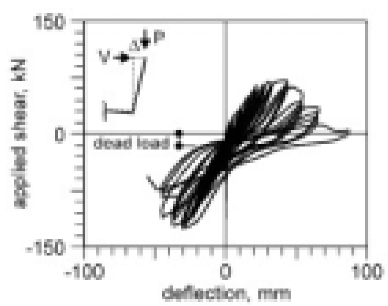
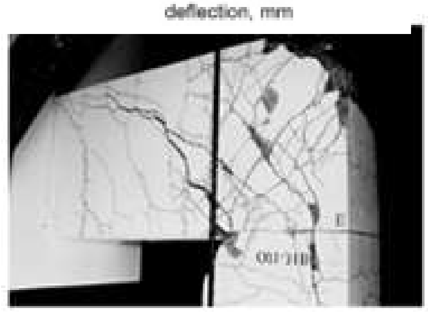
Outrigger Beam-Column Connections

Mitchell, D., "Aspects of Seismic Evaluation and Retrofit of Canadian Bridges", ACI SP-197, "Behavior and Design of Concrete Structures for Seismic Performance", 2002.

Outrigger







Griezic, A., "Seismic Evaluation and Retrofit of Bridge Columns and Joints", Ph.D. thesis, McGill University,1996.

Advantages of PBD – Evaluation and Retrofit

- Recognizes the difficulties and limitations of seismic retrofit
- Choice of Performance Levels and Performance Criteria controlled by Regulatory Authority/Owner
- Enables assessment of risk and cost-benefit of different retrofit strategies.
- Permits vital retrofits at reduced levels and permits staged upgrades

Towards PBD of Buildings

- Proposed additions to NBCC 2020 (subject to public review!)
- Targeting "more important" buildings in moderate to high seismic categories
- Aim is to improve resiliency
- Limit "damage" at lower level (more frequently occurring)earthquakes

Performance Levels

- Initially for "more important" buildings
- Post-Disaster buildings (e.g., hospitals)
- High Importance (e.g., schools)
- Importance triggers performance requirements for lower hazard level
- Depends on seismic hazard (Seismic Performance Category)
- Performance levels
  - Limit drift to prevent structural and non-structural damage
  - Components and connections required to behave "elastically"

Acknowledgements

SEISMIC DESIGN OF BRIDGES:

- Significant effort in developing and refining the PBD approach by the S6 Subcommittee on Seismic Design
- Key input from the development of the BC MoTI Supplement to S6

SEISMIC DESIGN OF BUILDINGS:

- Members of the Standing Committee on Earthquake Design (SCED)

Denis Mitchell, McGill University

Session 2

Performance Based Seismic Design of Buildings

TOWARDS THE PERFORMANCE BASED SEISMIC DESIGN OF UNUSUAL IRREGULAR & TALL BUILDINGS IN BC

By Dr. P. Adebar, University of British Columbia

Abstract

Consistent with a worldwide "epidemic," the City of Vancouver has recently declared that "all higher buildings must establish a significant and recognizable new benchmark for architectural creativity…" The result is some awe-inspiring structures, but a growing concern is that these highly irregular buildings will not be habitable after a small earthquake. At the same time, there is an increased awareness that current building code requirements for collapse prevention do not provide for sufficiently resilient cities. In this presentation, the author will discuss some of the proposed changes to the NBC 2020, such as (i) a requirement for Normal importance buildings, more than 30 m tall and in high seismic regions, to have a gravity-load frame that remains elastic for a 10% in 50-year earthquake, and; (ii) new requirements for sloped-column irregularity in buildings. The author will also discuss how a completely different approach, from a national model building code on a 5-year development cycle, is needed if structural engineers are to have the "tools" they need to adequately design the buildings that architects are currently "conjuring up."

Keywords: building codes, concrete shear wall buildings, gravity-load frame, high-rise buildings, irregularity.

Biography

Dr. Perry Adebar is a Professor of Structural Engineering at the University of British Columbia. He is a member of Technical Committee CSA A23.3, and the Chair of the Sub-committee on Seismic Design, a member of the Canadian Standing Committee on Earthquake Design, a Director of the Structural Engineers Association of BC, and he contributes his structural engineering expertise to the Vancouver's HUSAR Team – Canada Task Force 1.

NRC-MOST/NCREE Taiwan
Workshop Earthquake Engineering
NRC Ottawa Oct. 7-8 2019

Towards the **Performance Based Seismic Design** of **Unusual Irregular & Tall Buildings** in BC

Perry Adebar
Professor of Structural Engineering
University of British Columbia

Topics Discussed:

- Irregularities in concrete buildings
- Designing irregularities away
- Overhang wall irregularity – concentration of strains
- Concrete walls not part of the SFRS
- "Unusual irregular"
- Design approach for *unusual irregular* buildings
- 2020 NBC: Sloped-column irregularity
- 2020 NBC: SLE check for tall buildings & high seismicity

There is an increasing awareness that designing buildings for *life safety/collapse prevention* performance for a 2% in 50-year seismic demand (current objective of building code) is not sufficient.

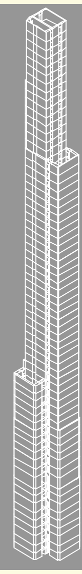
Providing opportunity for 'building owners' to choose a higher performance level is not a solution in Canada.

We need to provide designers with the "tools" they need to design better performing buildings.

Opinion:
For concrete buildings, irregularities strongly influence performance

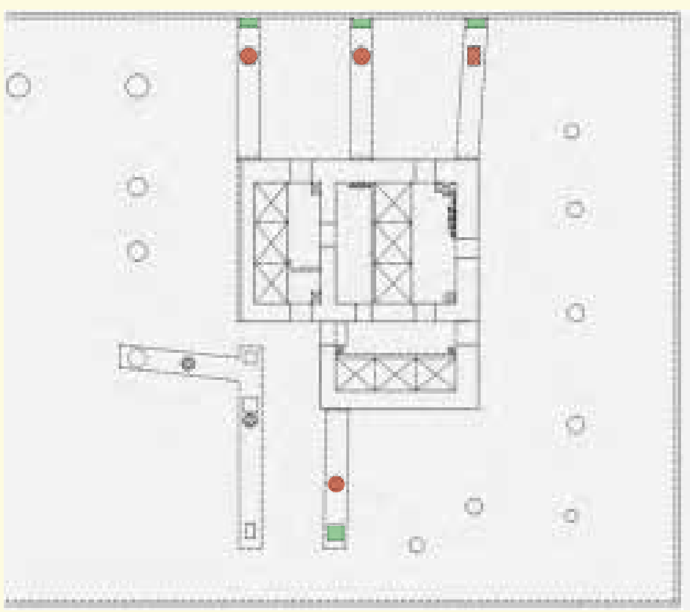
Most buildings have some irregularities.

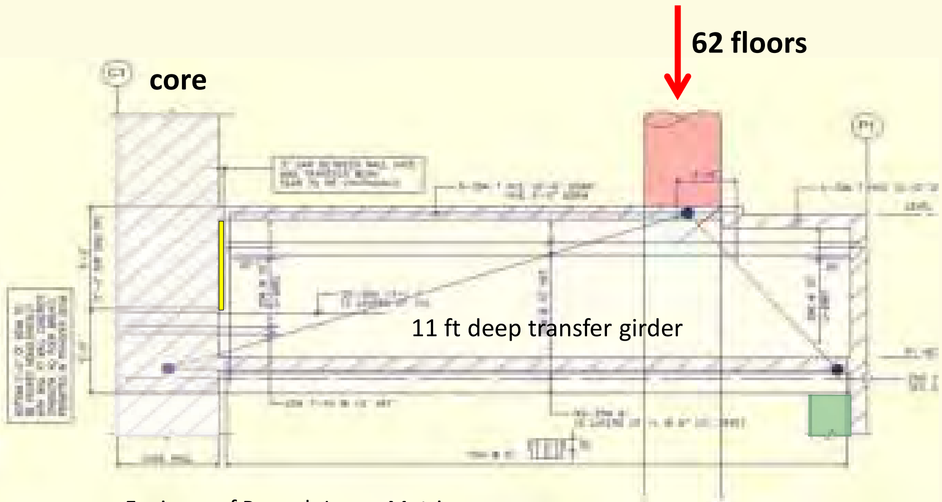
Good engineering can help reduce effect.
Example…



Living Shangri-La Vancouver

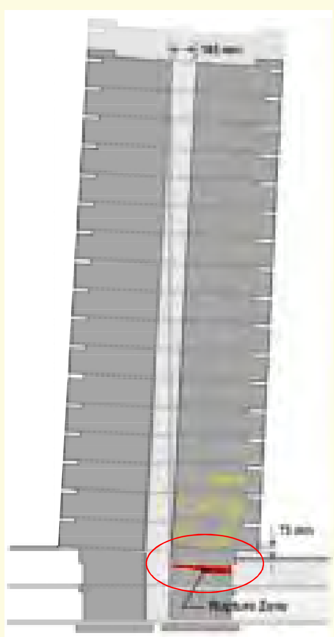
62 stories
201 m
(659 ft)
Built 2008





Engineer of Record: James Mutrie

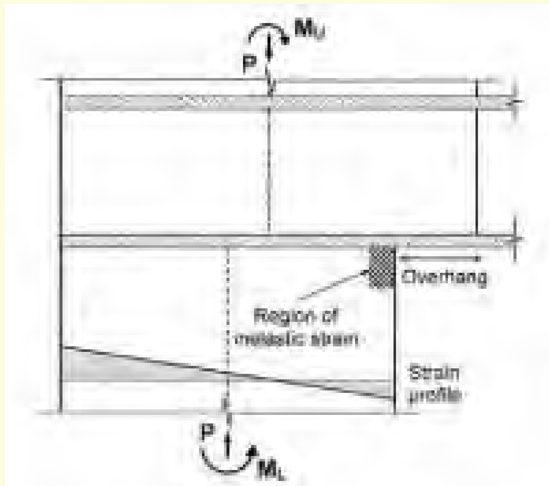
Sherstobitoff, Cajiao, Adebar, "Repair of 18-Story Shear Wall Building Damaged in 2010 Chile Earthquake," *Earthquake Spectra*, June 2012.



Irregularities: Lessons learned from 2010 Chile (Maule) Earthquake

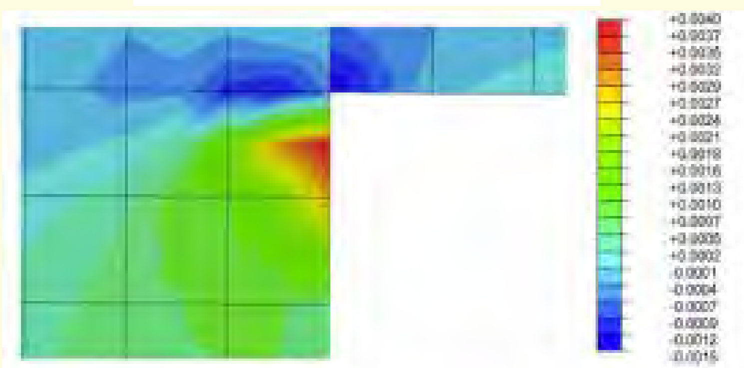
10

Overhanging wall irregularity



11

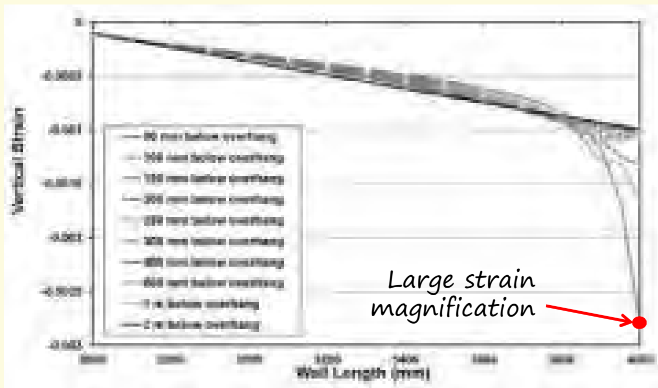
Nonlinear Finite Element Analysis



Contours of horizontal strains for a 1.0 m overhang when the maximum compression strain in the uniform strain region is 0.002.

Adebar, and Mahmoodi, "Compression failure of thin concrete shear walls with overhanging wall above," 10NCEE, 2014.

Nonlinear Finite Element Analysis



Adebar, and Mahmoodi, "Compression failure of thin concrete shear walls with overhanging wall above," 10NCEE, 2014.

Conclusion:
Small irregularities in concrete walls
can cause large increases in maximum strains
that may result in significant damage to the building
when subjected to a relatively small earthquake.

These strain increases cannot be determined
using current (nonlinear dynamic) analysis models.

Modern tall buildings in BC should not have overhanging walls as part of the SFRS

NBC Clause 4.1.8.10.(3)
(paraphrased)

For buildings with $T_a \geq 1.0$ s, and
where $I_E F_v S_a(1.0)$ is greater than 0.25,
concrete shear walls part of the SFRS
shall be continuous from top to foundation and shall not
have *In-plane Discontinuity* or *Out-of-Plane Offset*.

Modern building:

Concrete walls
not part of SFRS

Photo courtesy of Andy Metten

2018 BC Building Code (2014 CSA A23.3):

Significant new requirements to ensure gravity-load frames can tolerate movements of the SFRS.

21.11 Members not considered part of the SFRS

21.11.1.1 Application

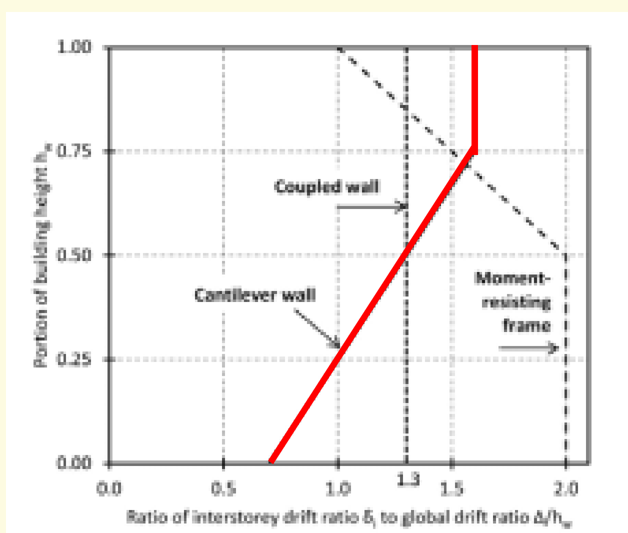
Independent of R_d used to design the SFRS, shall apply to all members not part of SFRS unless building is located where $I_E F_a S_a(0.2) \le 0.35$; or maximum interstorey drift ratio < 0.005.

21.11.2.2 Simplified analysis of buildings

The demands induced in members of a gravity-load frame shall be determined at each level by subjecting the frame to the interstory drift given in Fig. 21-1 for that level.

Cracking of concrete: an upper-bound estimate of effective stiffness shall be used in order to determine a safe estimate of induced forces

Prescribed inter-story drift demands over full building height
Gravity-load frame must be designed to tolerate drifts



Foundation rotation must be added as a an additional uniform interstorey drift demand over the full height of building

Conclusion:
The (fairly) new requirements in
Clause 21.11 of CSA A23.3 will help to ensure
that the gravity-load frames in concrete buildings
will not collapse for 2% in 50 year hazard,
and will perform better at lower hazard levels.

"Unusual irregular"

The City of Vancouver has recently declared that:
"all higher buildings must establish a significant and recognizable new benchmark for architectural creativity… "

Developers, Architects, City Officials, …

and some Structural Engineers…

believe unusual irregular buildings that

“meet the code” will perform like all other buildings

The Globe and Mail, Aug. 11, 2016:

… “the less uniform and more irregular the structure of a high-rise building is, the more likely it will be damaged.”

Statement by practice adviser with the Architectural Institute of BC:

“All buildings are required to reach a certain level and there are certain things like irregularities that create issues for seismic [standards], but the code requires that you design to address it. It basically equalizes everything out and it will comply with the code,” Ms. Gatensby said. “Everything ends up getting to the same level.”

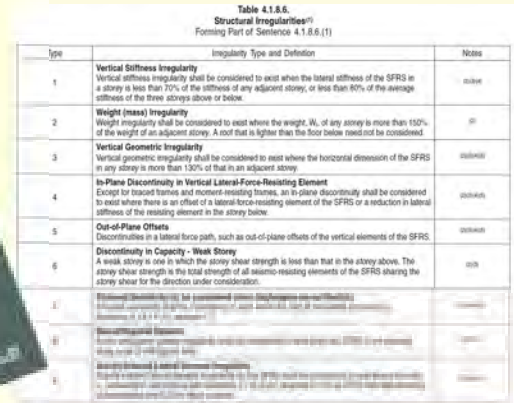


Table 4.1.8.6.
Structural Irregularities[1]
Forming Part of Sentence 4.1.8.6.(1)

Type	Irregularity Type and Definition	Notes
1	**Vertical Stiffness Irregularity** Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the SFRS in a storey is less than 70% of the stiffness of any adjacent storey, or less than 80% of the average stiffness of the three storeys above or below.	[illegible]
2	**Weight (mass) Irregularity** Weight irregularity shall be considered to exist where the weight, W_i, of any storey is more than 150% of the weight of an adjacent storey. A roof that is lighter than the floor below need not be considered.	[illegible]
3	**Vertical Geometric Irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the SFRS in any storey is more than 130% of that in an adjacent storey.	[illegible]
4	**In-Plane Discontinuity in Vertical Lateral-Force-Resisting Element** Except for braced frames and moment-resisting frames, an in-plane discontinuity shall be considered to exist where there is an offset of a lateral-force-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the storey below.	[illegible]
5	**Out-of-Plane Offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS.	[illegible]
6	**Discontinuity in Capacity - Weak Storey** A weak storey is one in which the storey shear strength is less than that in the storey above. The storey shear strength is the total strength of all seismic-resisting elements of the SFRS sharing the storey shear for the direction under consideration.	[illegible]
7	[illegible]	[illegible]
8	[illegible]	[illegible]
9	[illegible]	[illegible]

Question:
What do you call a document where you summarize simple procedures for how to design a building of the ***type that has been built before****?*

Answer:
Building code.

Question:
What do you use to design a building with a ***new type of irregularity****?*

Answer:
Something more than ***the building code****.*

How to design a unusual irregular building:

(for collapse prevention/life safety and other performance levels)

'Building Code Plus' + State-of-the art analysis "tools" + Rigorous Peer Review

update *(frequently)*

Update Building Code *(every 5 years)*

Finally,

two proposed changes for 2020 NBC

to **ensure better performance of**

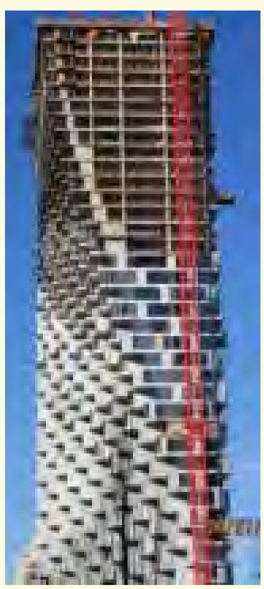
unusual irregular and tall buildings

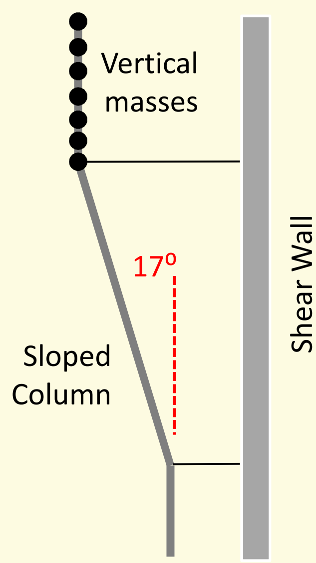
- Sloped-column irregularity
- SLE check for tall buildings & high seismicity

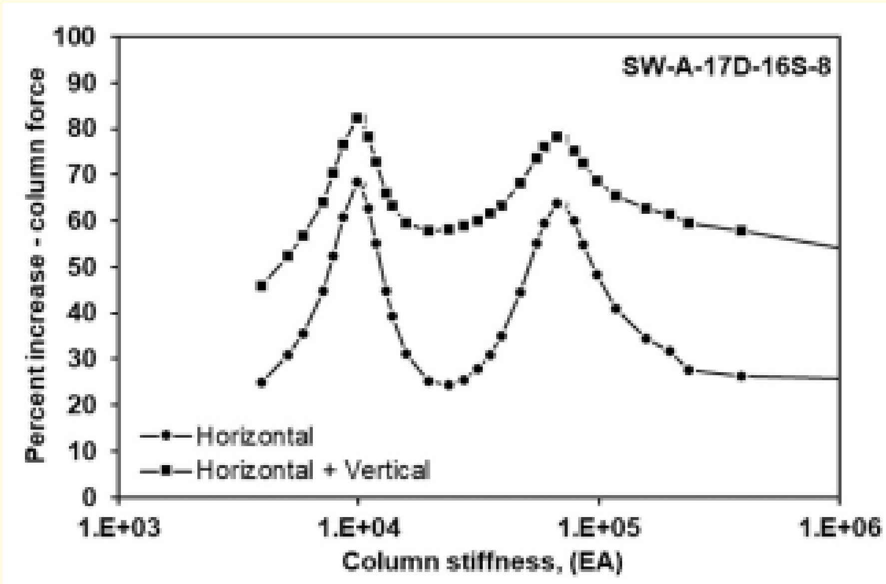
Draft 2020 NBC: Sloped-Column Irregularity

Table 4.1.8.6.
Structural Irregularities
Forming Part of Sentence 4.1.8.6.(1)

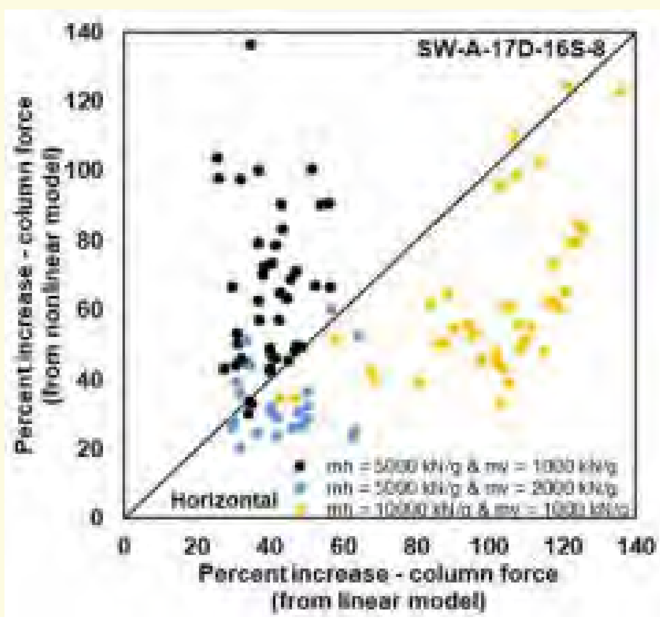
Type	Irregularity Type and Definition	Notes
1	Vertical Stiffness Irregularity Vertical stiffness irregularity shall be considered to exist when the drift ratio in a storey under lateral seismic force is greater than 125% of the drift ratio of the storey above	[illegible]
8	Non-orthogonal Systems A non-orthogonal system irregularity shall be considered to exist when the SFRS is not oriented along a set of orthogonal axes.	[illegible]
9	Gravity-Induced Lateral Demand Irregularity Gravity-induced lateral demand irregularity on the SFRS shall be considered to exist where the ratio, α, calculated in accordance with Sentence 4.1.8.10.(5), exceeds 0.1 for an SFRS with self-centering characteristics and 0.03 for other systems.	[illegible]
10	Sloped-Column Irregularity Sloped-column irregularity shall be considered to exist when vertical member(s) supporting more than 5% of the total building weight are inclined more than 2 degrees from the vertical.	[illegible]







Adebar P and Mahmoodi M, “Sloped-column Irregularity in High-rise Shear Wall Buildings,” 17WCEE.



Adebar P and Mahmoodi M, “Sloped-column Irregularity in High-rise Shear Wall Buildings,” 17WCEE.

Draft 2020 NBC

Normal importance buildings
in seismic category SC4
and height more than 30 m
all structural framing elements not considered part of the SFRS
to be **elastic** for the demand from earthquake with 10% probability of exceedance in 50 years.

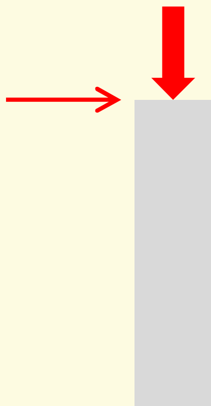
Intent is to reduce damage to gravity-load frames in ***irregular buildings***.

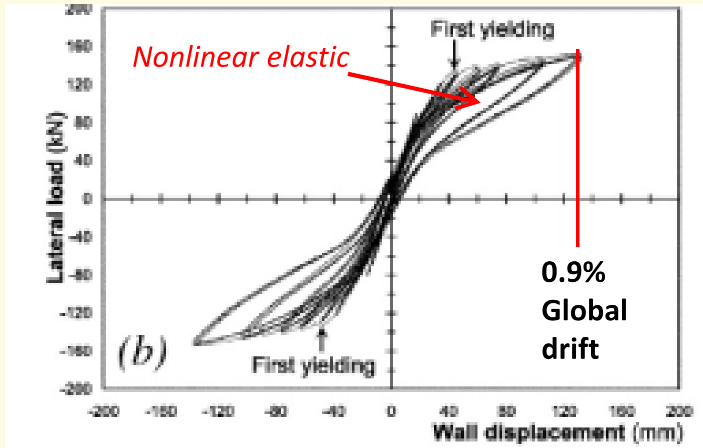
Rather than specify for which irregularities an additional service-load check must be done, we require that for all buildings, the gravity-load frames must remain elastic; but only the design of irregular buildings is expected to be affected.

Cost impact: minimal or negative – the main impact is to constrain architect's creativity – less irregularity means lower construction costs.

Definition of "elastic"

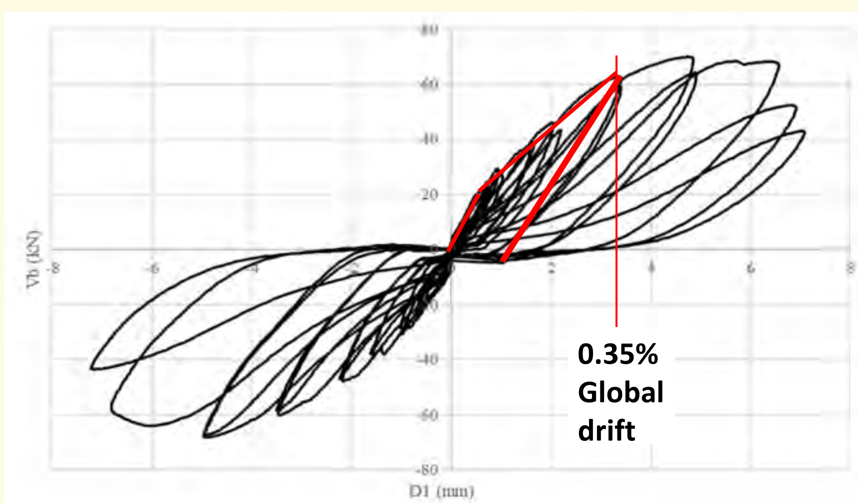
"**Yielding**" of a flexurally-dominated member subjected to significant axial compression





Perry Adebar – University of British Columbia Slide 35

Shear-dominated member



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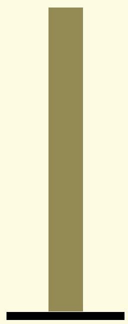
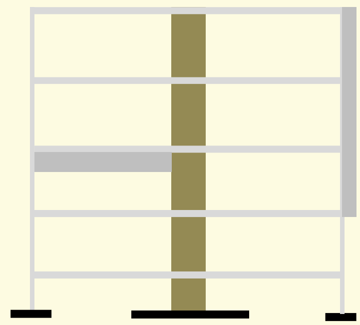
For collapse evaluation:



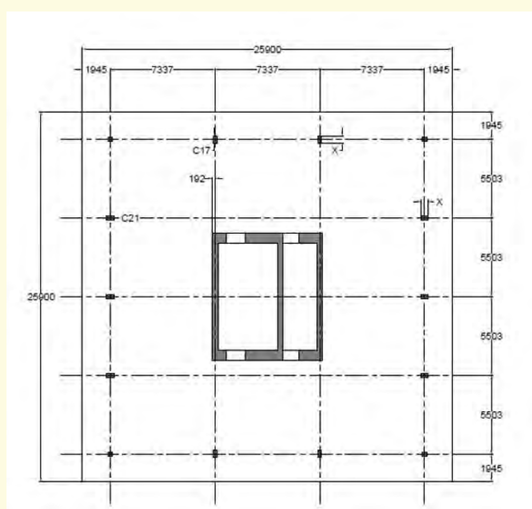
SFRS Only

For service-level evaluation:



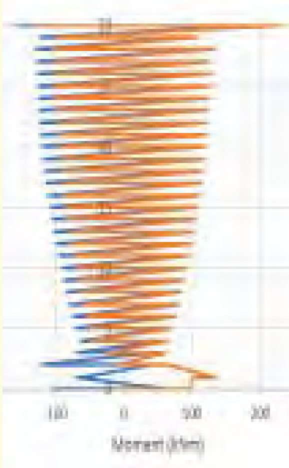
SFRS and Structural Members Not Considered Part of SFRS

Case Study Typical 30-story Vancouver Residential Tower

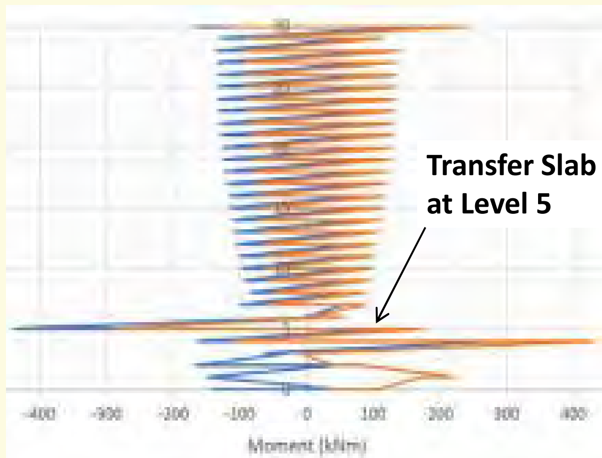


Column bending moments:

Regular building

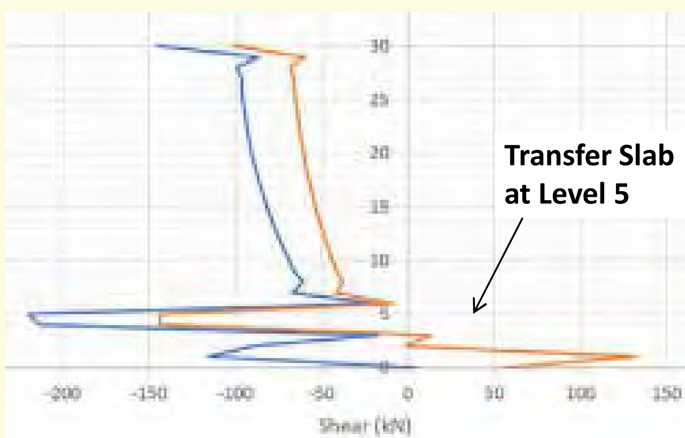


Irregular building



Perry Adebar – University of British Columbia   Slide 39

Column shear forces:



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Trial analyses by RJC – Three buildings

No design changes required to meet new NBC requirement

From: Bob Neville, RJC

DESIGN BASE SHEAR FORCES FOR RC BUILDINGS CONSIDERING SEISMIC RELIABILITY AND LIFE-CYCLE COSTS

By Dr. C.K. Chiu, National Taiwan University of Science and Technology

Abstract

The object of this work is to propose a novel estimation procedure for optimal design base shear forces for RC buildings while considering the seismic reliability and life-cycle costs (LCCs) incurred by life-cycle earthquake events. By simulating life-cycle earthquake events within a specified period and using nonlinear dynamic analysis, including earthquakes occurrences and their peak ground accelerations, this work also derives the damage states of an RC building considering the effect of the cumulative damage. Additionally, besides life-cycle earthquake events, a simplified model is developed to modify the structural properties of a structure without seismic repair after earthquakes. Given the uncertainty of the occurrence time and PGAs of earthquake events, the seismic reliability and expected current values of LCCs are calculated using Monte Carlo simulation. Therefore, optimal design base shear forces for RC buildings calculated via the same procedure can be derived and utilized when making decisions on the seismic level of a building based on safety and economic considerations.

Keywords: life-cycle cost (LCC), seismic reliability, reinforced concrete, building, cumulative damage.

Biography

Dr. Chien-Kuo Chiu received his Ph.D. (2008) from Architecture at University of Tokyo. Since 2009, he is a Professor at National Taiwan University of Science and Technology (NTUST). Currently, he is the Vice Dean of Faculty of Engineering in NTUST. His present research interests include life-cycle assessment, maintenance, and deterioration assessment.

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- INTRODUCTION/Destination
- *STRUCTURAL CAPACITY DEGRADATION DUE TO CORROSION*
- STRUCTURAL DAMAGE DUE TO EARTHQUAKES
- ESTIMATION MODEL OF LIFE-CYCLE COSTS
- ESTIMATION PROCEDURE FOR OPTIMAL DESIGN BASE SHEAR FORCES OF RC BUILDINGS BASED ON SEISMIC RELIABILITY and LCCs
- NUMBERICAL CASE STUDY: SECOND DIVISION ZONE IN TAIPEI BASIN, TAIWAN

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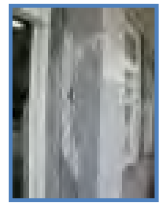
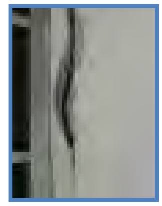
INTRODUCTION





Demand has increased for a system that can identify the optimal seismic design and life-cycle maintenance strategies for RC structures based on minimal LCCs, including losses incurred by the effect of cumulative damage.

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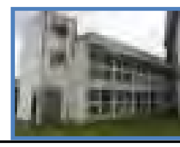

Decision-Making Supporting System of Life-Cycle Maintenance Strategies for Deteriorating Reinforcing Concrete Buildings

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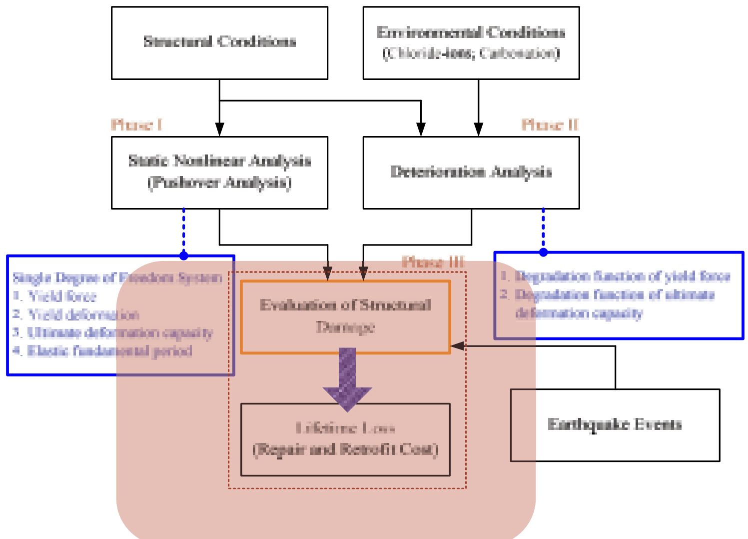
<u>Chiu, C. K.</u>, and Lin, Y. F. 2014. Multi-objective Decision-making Supporting System of Maintenance Strategies for Deteriorating Reinforced Concrete Buildings, Automation in Construction, 39(1), 15-31. (SCI)

Estimation Procedure



__Chiu, C. K.__, Tu, F. J. and Hsiao, F. P. 2015. Lifetime Seismic Performance Assessment for Chloride-Corroded Reinforced Concrete Buildings, Structure and Infrastructure Engineering, 11(3), 345-362. (SCI)

INTRODUCTION

Destination

The object of this work is to propose a novel estimation procedure for optimal design base shear forces for RC buildings while considering the ***seismic reliability*** and ***life-cycle costs (LCCs)*** incurred by life-cycle earthquake events.

Life-cycle performance-based Design

Optimal design base shear forces for RC buildings calculated via the same procedure can be derived and utilized when making decisions on the seismic level of a building based on safety and economic considerations. The proposed method can help both owners and investors to identify LCCs of RC buildings due to seismic structural damage within a specified service life.

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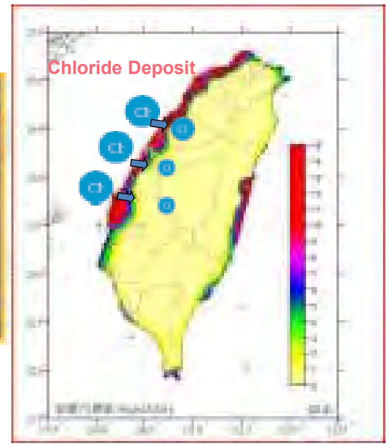
STRUCTURAL CAPACITY DEGRADATION DUE TO CORROSION

Image base from Google Earth

Chiu, C. K., Tu, F. J. and Fan, C. 2014. Risk Assessment of Environmental Corrosion for Reinforcing Steel Bars Embedded in Concrete in Taiwan, Natural Hazards, 75(1), 581-611. (SCI)



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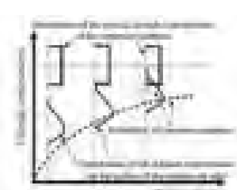
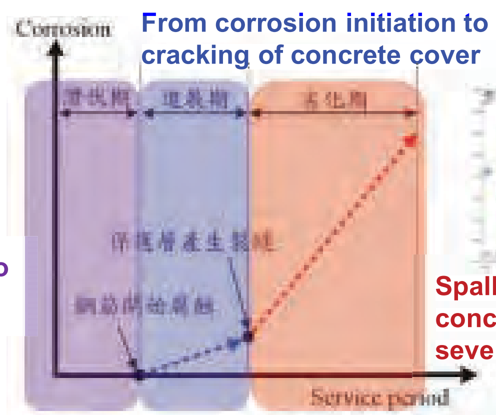
DETERIORATION-CHLORIDE INGRESS



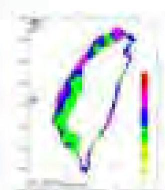
From construction to corrosion initiation

$$\frac{\partial Cl}{\partial t} = D(x,t)\frac{\partial^2 Cl}{\partial x^2}$$

Fick's Equation



From corrosion initiation to cracking of concrete cover

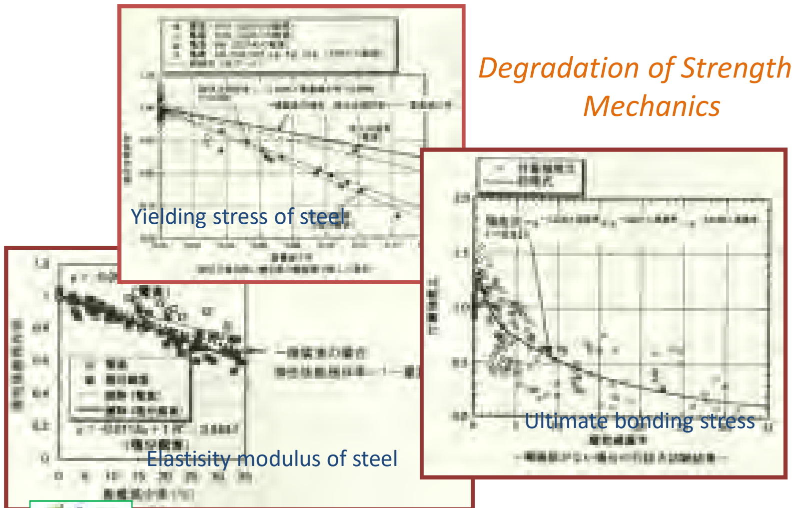


Spalling of concrete cover, severe corrosion

$$Cl(t) = (C_0 - C_{init})\left[1 - erf\left(\frac{x}{2\sqrt{D_p \times t}}\right)\right] + C_{init}$$

$$\phi(t) = \phi_0 - \frac{1.0508(1-w/c)^{-1.64}}{d}(t - T_{corr})^{0.71}$$

MODIFICATION OF MATERIAL PROPERTIES DUE TO CORROSION



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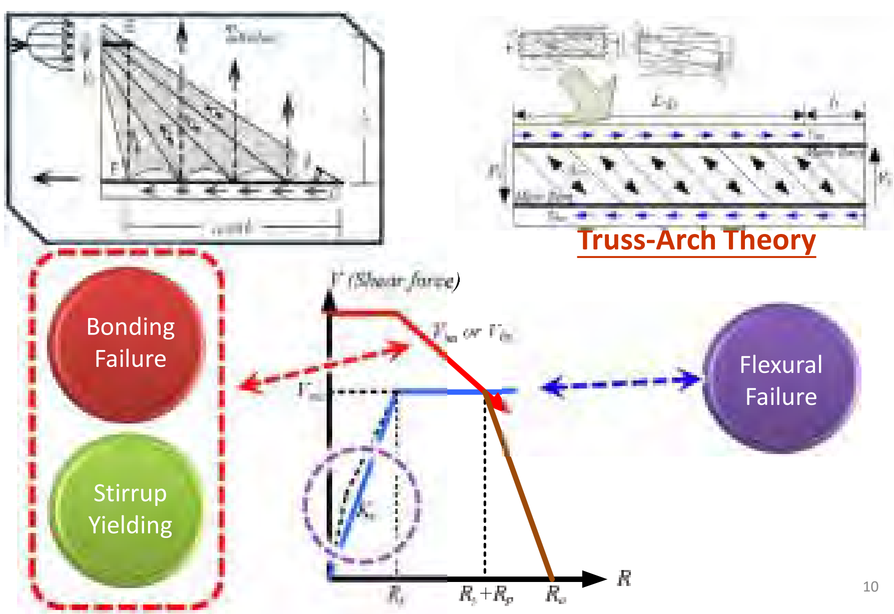
BEHAVIOR OF RC MEMBERS



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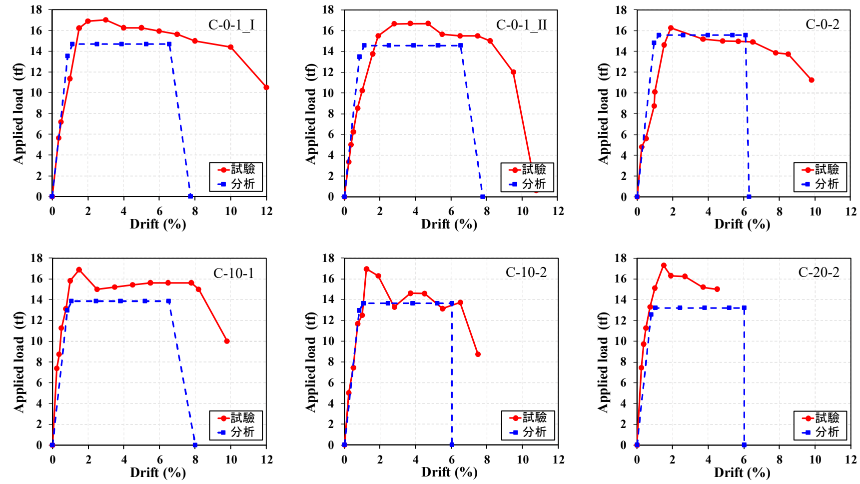
BEHAVIOR OF RC MEMBERS (Experiment and Analysis)



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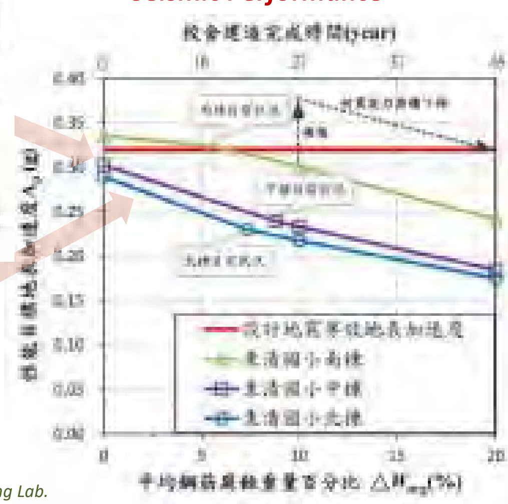
Degradation of Seismic Performance

Deterioration Degree of an RC Column

Seismic Performance



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ULTIMATE STORY SHEAR FORCE

Ultimate Shear Force of a Corroded RC member

$$Q_u = \min(Q_{Mu}, Q_{Su})$$

$$Q_{Su} = \min\left(\frac{2M_{bu}}{L}, V_{bu}, V_u\right)$$

$$Q_{Mu} = \frac{(M_{yu})_T + (M_{yu})_B}{L} = \frac{2M_{yu}}{L}$$

Ultimate Story Shear Force

Ductility-Controlled

$$E_o^i = \sqrt{G_{i1}^2 + G_{i2}^2 + G_{i3}^2} = \sqrt{(C_{i1} \times F_{i1})^2 + (C_{i2} \times F_{i2})^2 + (C_{i3} \times F_{i3})^2}$$

$$E_o^i = (C_{i1} + (C_{i2} + C_{i3}) \times \alpha_2) \times F_{i1}$$ **Strength-Controlled**

Failure Mode

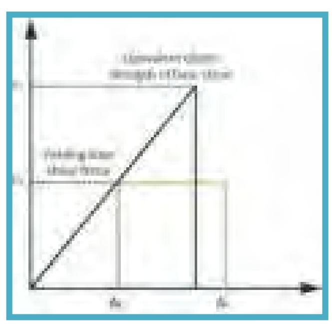
Group	Failure mode	Ductility (F)
G1 (Corrosion-induced)	Shear failure	1.0
G2 (Corrosion-induced)	Flexural failure	$\frac{1}{0.75(1+0.05\mu)}\sqrt{2\mu-1}$
G3 (Without any corrosion)	Flexural failure	$\frac{1}{0.75(1+0.05\mu)}\sqrt{2\mu-1}$

SEISMIC PERFORMANCE OF AN DETERIORATING RC BUILDING

$$E_e = \min\left(\frac{1}{A_j} E_e^j\right), j = 1,2,3,\ldots,n$$ **Equivalent elastic strength of base shear**

$$E_y = \min\left(\frac{1}{A_j} E_y^j\right), j = 1,2,3,\ldots,n$$ **Yielding base shear force**

$$D_u = \begin{cases} \left(\left(\frac{E_e}{E_y}\right)^2 + 1\right)/2, & \text{for a structure with shortperiod} \\ \frac{E_e}{E_y}, & \text{for a structure with long period} \end{cases}$$



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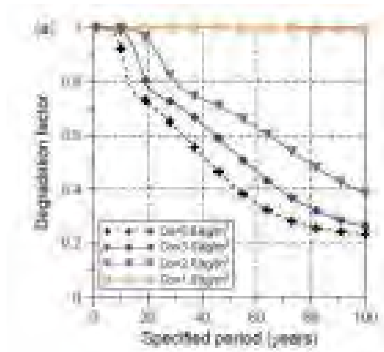
DEGRADATION FUNCTION FOR THE STRUCTURAL CAPACITY

For a corroded RC building, we assume yielding force and ductility of the SDOF system decrease over time after corrosion starts, as shown below.

$$F_y = F_{yo} \times g_1(t)$$

$$\mu_u = \mu_{uo} \times g_2(t)$$

$$\delta_y = \frac{F_y}{k_y} = \frac{F_{yo} \times g_1(t)}{k_{yo}} = \delta_{yo} \times g_1(t)$$



This work adopted the equivalent elastic strength of base shear and yielding base shear force to define time-dependent degradation functions, g1(t) and g2(t) for the RC building in the case study.

$$g_1(t) = \frac{E_y(t)}{E_{yo}}$$

$$g_2(t) = \begin{cases} \dfrac{((E_e(t)/E_y(t))^2 + 1)}{((E_{eo}/E_{yo})^2 + 1)}, & \text{for a structure with short period} \\ \dfrac{(E_e(t)/E_y(t))}{(E_{eo}/E_{yo})}, & \text{for a structure with long period} \end{cases}$$

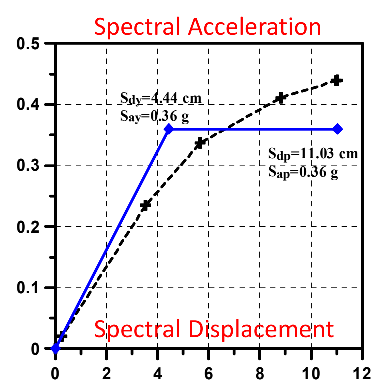
STRUCTURAL DAMAGE DUE TO EARTHQUAKES

Δ Seismic Damage Index

The model developed by Park and Ang (1985), the most widely used model in literature (Cosenza et al., 2009), is a linear combination of maximum deformation response and hysteretic energy. The damage index of this model is expressed as

$$D_{P\&A} = \frac{\delta_M}{\delta_u} + \frac{\beta}{F_y \delta_u} \int dE$$

Absorbed hysteretic energy



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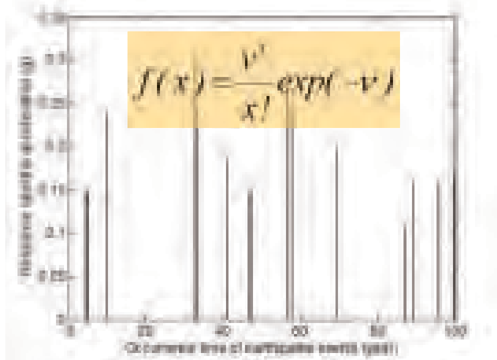
STRUCTURAL DAMAGE DUE TO EARTHQUAKES

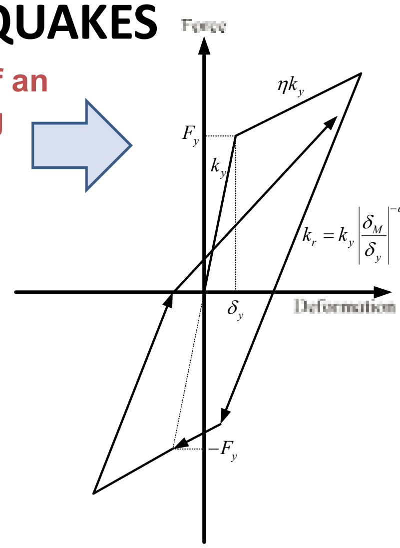
Δ Nonlinear Dynamic Analysis of an SDOF System for an RC Building

Δ Life-cycle Earthquake Events

Poisson's Distribution

$$f(x)=\frac{v^x}{x!}exp(-v)$$





Hysteretic behavior descripted in the Takeda model.

Equivalent Linearization Method for Maximum Deformation Response

Generally, the iterative calculation needs to be applied in the equivalent linearization method for deformation response of the structural SDOF system.

$$D_{P\&A}=\frac{\delta_M}{\delta_u}+\frac{\beta}{F_y\delta_u}\int dE$$

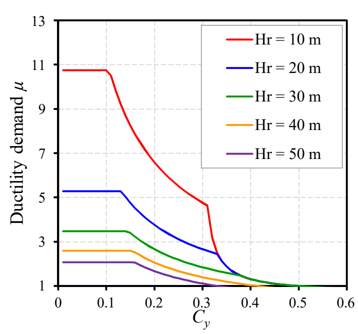
Response spectral acceleration

$$\delta_M=[\frac{1}{b}-\frac{c\times S_A}{S_{ay}}]^{-2}\times\delta_y$$

$$\delta_M=[a\frac{S_V}{\omega_y\delta_y}+b]^2\times\delta_y$$

$$\delta_M=\frac{1}{4}[b+\sqrt{b^2+\frac{4a\times S_D\times\omega_y^2}{S_{ay}}}]^2\times\delta_y$$

(Okano et al. 2002)



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Modified equivalent linearization method (MELM)

$$F_h = \frac{1}{B_s} = \frac{\left(\frac{1.5}{40\xi_{eq}+1}+0.5\right)}{1.1}$$

$$F_h = \frac{1}{B_1} = \left(\frac{1.5}{40\xi_{eq}+1}+0.5\right)$$

$$T_{eq} = \sqrt{\mu} \times T_y$$

$$\xi_{eq} = 0.212\frac{(\mu-1)}{\mu}+0.05$$

$$C_y = \frac{F_y}{mg} = \frac{S_{ay}}{g}$$

Spectral acceleration $S_A F_h = S_{ay}$ ➡ $$\Delta_M = \frac{1}{1.354-\left(\frac{0.177}{1.1C_y g/S_A - 0.5}\right)} \times \Delta_y$$

Spectral velocity $\frac{2\pi}{T_{eq}} S_V \cdot F_h = S_{ay}$ ➡ $$\frac{1}{\sqrt{\mu}}\left(\frac{1.5\mu}{11.48\mu-8.48}+0.5\right) = \frac{\sqrt{C_y H_r R_y g}}{S_V}$$

Spectral displacement $\left(\frac{2\pi}{T_{eq}}\right)^2 S_D \cdot F_h = S_{ay}$ ➡ $$\Delta_M = \frac{-B+\sqrt{B^2-194.7C}}{22.96C} \times \Delta_y$$

$$B = -8.48C - 7.24 \text{、} C = {H_r R_y}/{S_D}$$

Chiu, C. K., Nugroho, L., Gautama, S. and Hsiao, F. P. (2019), "Reliability-based Constant-damage Ductility Demand Spectra of Mid-rise RC Building Structures Using the Equivalent Linearization Method", Structure and Infrastructure Engineering, Online published. (SCI)

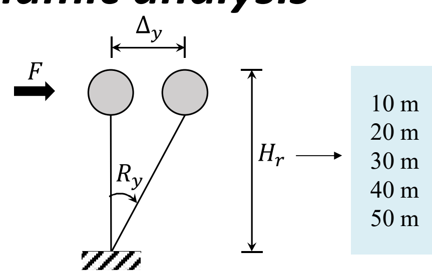
Nonlinear dynamic analysis

SDOF system Input:
1) Hysteretic model
2) Damping ratio
3) Elastic period
4) C_y

Ground motion input

Nonlinear Dynamic Analysis (NDA)
SDF.exe

Response time histories



SDOF System

$$\Delta_y = H_r \times R_y \qquad T_y = 2\pi\sqrt{\frac{H_r \times R_y}{C_y \times g}}$$

Δ_y = yield displacement
H_r = equivalent height of SDOF system
R_y = yield rotation (1/150)
T_y = elastic period (s)
C_y = yield base shear coefficient
g = gravitational acceleration (9.81 m/s^2)

Collected ground motion records

Earthquake	Date	Magnitude (M_L)	Source depth (km)	Number of records
Chi-Chi	1999/09/21	7.3	8.0	133
Hualien	2002/03/31	6.8	9.6	100
Jiaxian	2010/03/04	6.3	5.0	12
Meinong	2016/02/06	6.6	14.6	95
Chiayi	1999/10/22	6.4	12.1	8

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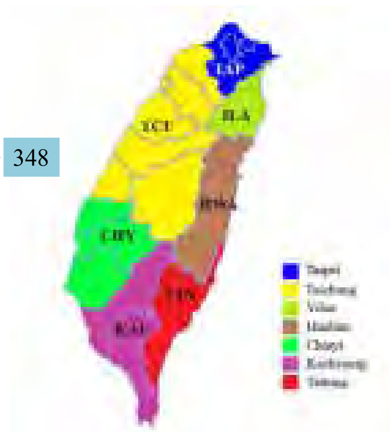
Site Condition	Characteristics	Number of records
Taipei Basin	Located in Taipei	100
Soil type 1	$V_{S30} > 270$ m/s	100
Soil type 2	$180 \le V_{S30} \le 270$ m/s	100
Soil type 3	$V_{S30} < 180$ m/s	48

Near-fault effect is not considered!



Station Map

Modification factor α_T

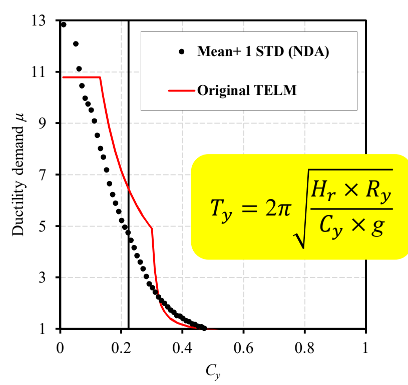
$$S_A = \frac{2\pi}{T_{eq}} S_V(\alpha_T T_{eq})$$

$$S_D = \frac{T_{eq}}{2\pi} S_V(\alpha_T T_{eq})$$

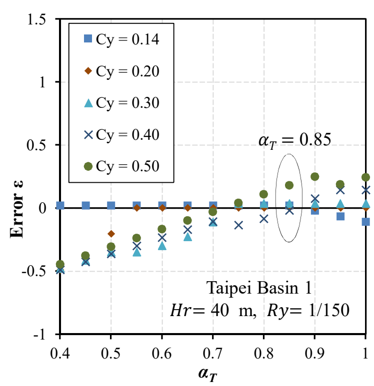
Selection of α_T:

1) Error as minimum as possible
2) Conservative result

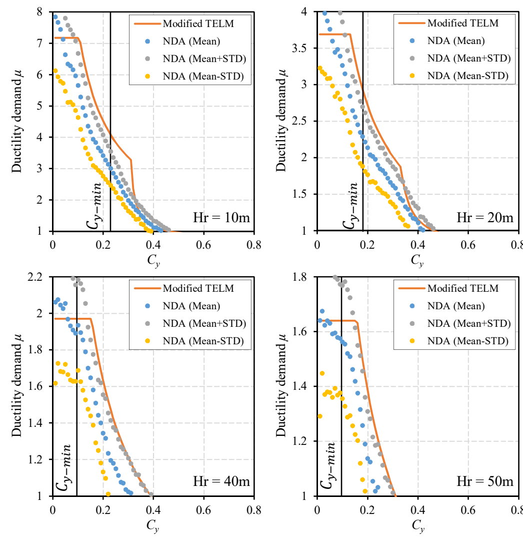
$\alpha_T = 0.85$



$$T_{eq} = \sqrt{\mu} \times T_y$$



Comparison Results

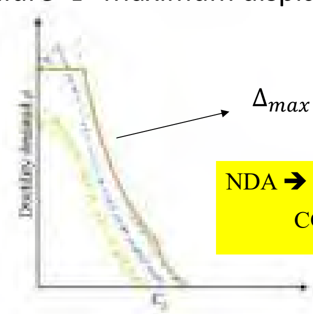


Micro zone:
Taipei Basin 2

Ductility demand spectrum using reliability analysis approach

Failure ➔ Maximum displacement > Δ_2

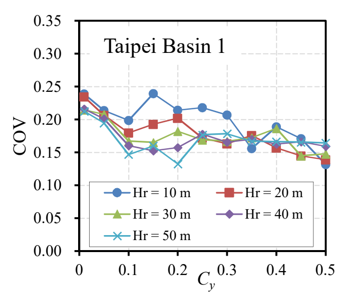
$$\Delta_2 = \left[\frac{2(\mu - 1) \times R_y}{3} + R_y\right] \times H_r$$



$$\Delta_{max} = \mu_\Delta + \sigma_\Delta$$

NDA ➔ COV = **0.165**

$$\text{COV} = \frac{\sigma_\Delta}{\mu_\Delta}$$



$$P_f(\Delta \geq \Delta_2) = 1 - \Phi\left[\frac{\ln \Delta_2 - \lambda}{\xi}\right]$$

(Lognormal distribution)

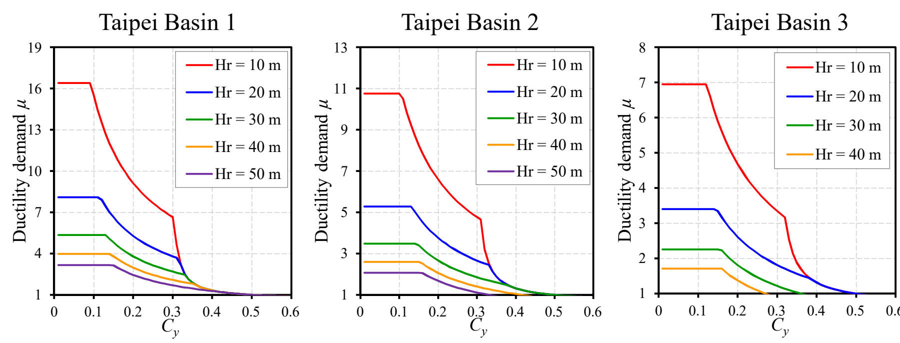
$$\xi = \sqrt{\ln\left(1 + \frac{{\sigma_\Delta}^2}{{\mu_\Delta}^2}\right)} \quad \lambda = \ln \mu_\Delta - \frac{1}{2}\xi^2$$

$$\mu = \frac{1}{2}\left\{\frac{3e\left[\Phi^{-1}\left(1 - P_f\right) \times \xi + \lambda\right]}{H_r R_y} - 1\right\}$$

Ductility demand spectrum using reliability analysis approach

- Failure probability = 10%

$\mu \downarrow$ as $C_y \uparrow$
$\mu \downarrow$ as $H_r \uparrow$



$$\mu = \frac{1}{2}\left\{\frac{3e\left[\Phi^{-1}\left(1-P_f\right)\times\xi+\lambda\right]}{H_r R_y} - 1\right\} \longrightarrow$$

Maximum deformation calculated using MELM

*****Chiu, C. K.**, Nugroho, L., Gautama, S. and Hsiao, F. P. (2019), "Reliability-based Constant-damage Ductility Demand Spectra of Mid-rise RC Building Structures Using the Equivalent Linearization Method", Structure and Infrastructure Engineering, Online published. (SCI)*

Simplified Evaluation Method for Hysteretic Energy

A measurement of the distribution of cycle amplitudes is n_{eq} representing the number of cycles at the maximum response deformation δ_M of the SDOF system that a structure could develop in order to dissipate the total amount of hysteretic energy E_h.

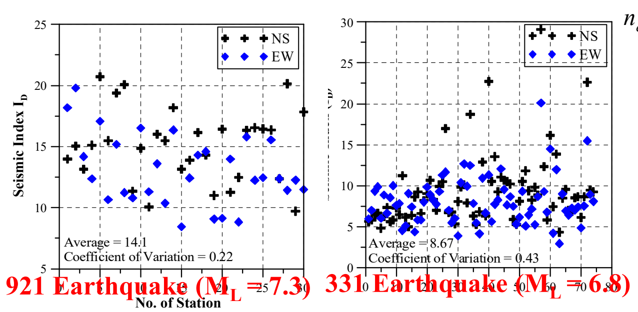
$$D_{P\&A} = \frac{\delta_M}{\delta_u} + \frac{\beta}{F_y \delta_u}\int dE$$

$$n_{eq} = \frac{E_h}{F_y(\delta_M - \delta_y)} = \left(\frac{E_h}{M}\right)\frac{1}{S_{ay}(\delta_M - \delta_y)}$$

$$\frac{E_h}{M} = (\mu_M - 1)n_{eq}\left(\frac{S_A}{\omega_y}\right)^2 (F_h)^2$$

$$n_{eq} = 1 + 0.18\left(\frac{1}{F_h} - 1\right)^{3/5} I_D \tau^{-1/2}$$

(Manfredi 2001)



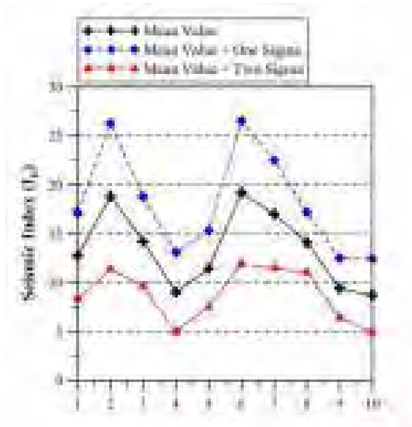
Seismic Index

$$I_D = \frac{I_E}{PGA \times PGV}$$

$$I_E = \int_0^{t_E} a(t)^2\,dt$$

Simplified Evaluation Method for Hysteretic Energy

The ten major earthquakes that occurred in Taipei over the past decade were chosen to analyze the seismic index, I_D (e.g., the 921 Chi-Chi Earthquake (Richter magnitude (M_L = 7.3, 1999), the 0614 Earthquake (M_L = 6.3, 2001), and the 331 Earthquake (M_L = 6.8, 2002); the number preceding an earthquake is its occurring date). *Simulation results indicate that mean values ± 1σ (σ is standard deviation) of seismic indices of selected earthquakes were 5–25.*



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Simplified Evaluation Method of Hysteretic energy

Correlation between the ground motion indices and n_{eq}:

Index	Definition	Correlation of coefficient
I_D	$I_D = \frac{I_E}{\text{PGA PGV}}$ $I_E = \int_0^{t_E} a(t)^2 dt$	0.43
τ	$\tau = \frac{t_d V_{max}}{2\pi SD_{max}}$	0.52
$\frac{\text{CAV}}{\text{PGV}}$	$\text{CAV} = \int_0^{t_E} \lvert a(t) \rvert dt$	0.54

Several SDOF systems from various site conditions are selected to perform the optimization process

2288 data points

$$n_{eq} = 1 + 0.048\left({}^{1}/_{F_h} - 1\right)^{0.6}\left(\frac{\text{CAV}}{\text{PGV}}\right)$$

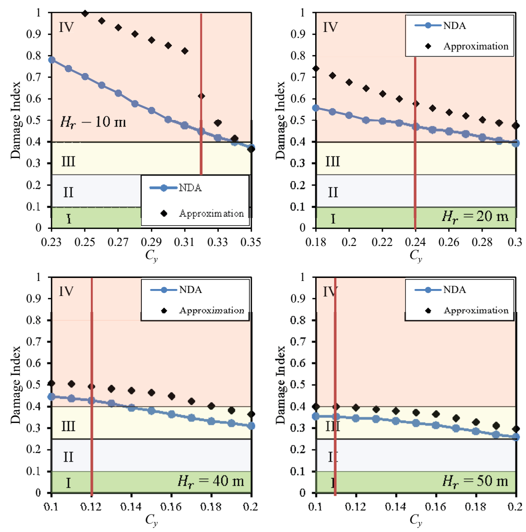
AE = 0.88
SE = 0.63

$$\frac{E_h}{M} = 2.83(\mu_c - 1)^{0.84} n_{eq}\left(\frac{S_a}{\omega_y}\right)^2 (F_h)^2$$

AE = 0.55
SE = 0.37

28

Damage Index for Taipei Basin 2 ($R = 4.8$)



Note:

- Large overestimation is observed in $H_r = 10$ m
- For other equivalent height, approximation method could predict the damage index relatively well

29

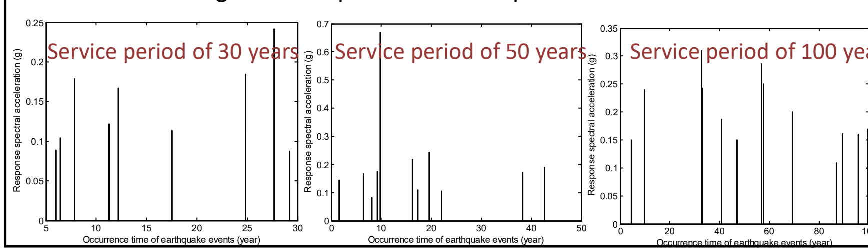
Life-cycle Earthquake Events

For convenience, the occurrence of earthquakes can be assumed to be a Poisson process with a mean rate appropriate for a region. In a Poisson process, the time intervals between two events follow an exponential distribution. Therefore, **the time of occurrence** of the (M+1)th earthquake is derived as follows:

$$t_{M+1} = t_M + \Delta t$$

$$f(\Delta t) = \left(\frac{\nu}{T_H}\right)\exp\left(-\frac{\nu \Delta t}{T_H}\right)$$

The Hazard curve of response spectral acceleration for a selected site, was utilized to determine the response spectral acceleration of each earthquake for a building within a specified service period.



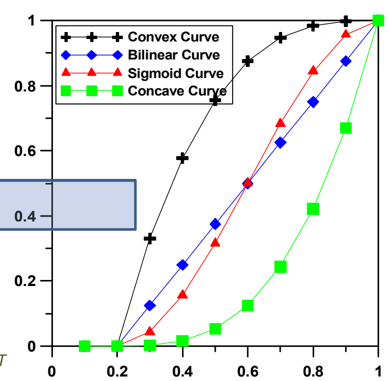
ESTIMATION MODEL OF LIFE-CYCLE COSTS (LCCs)

Δ Modeling of Seismic Repair Costs

According to Takahashi *et al.* (2006), repair costs after seismic structural damage are strongly correlated with the ratio of the maximum deformation response to the ultimate deformation, which is defined in this work as the damage repair index, D_R.

$$D_R = \frac{\delta_M}{\delta_u}$$

$$C_{Rep} = -\left(\frac{1-D_R}{1-\gamma}\right)^3 + 1$$



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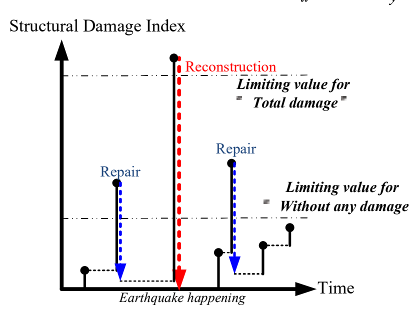
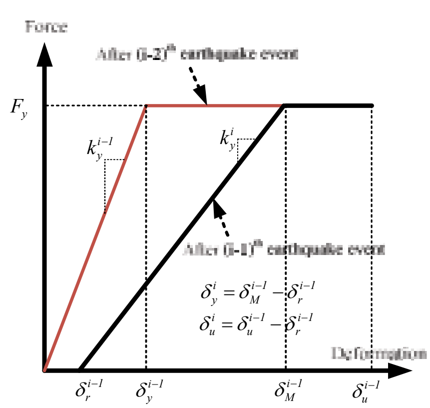
ESTIMATION MODEL OF LIFE-CYCLE COSTS (LCCs)

Δ Modeling of Cumulative Structural Damage Incurred by Earthquakes

$$\left(D_{P\&A}\right)_i = \left(D_{P\&A}\right)_{i-1} + \frac{\delta_M^i}{\delta_u^i} + \beta \frac{\int dE}{F_y^i \delta_u^i}$$





Modification of the structural properties of an RC building based on residual and maximum deformations

ESTIMATION MODEL OF LIFE-CYCLE COSTS (LCCs)

Δ Modeling of Life-cycle Costs

An objective function related to the expected current value of LCCs, L_I, is defined as

$$C_{Life} = C_I + \sum_{j=1}^{N}((C_{Rep,j} + C_{f,j}) \times \frac{1}{(1+k)^{T_j}})$$

$$L_I = E\left[-C_{Life}\right]$$

C_{Life} is the current value of LCCs for one calculation in the MCS

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SEISMIC RELIABILITY

Δ Estimation Method for the Seismic Reliability of RC Buildings

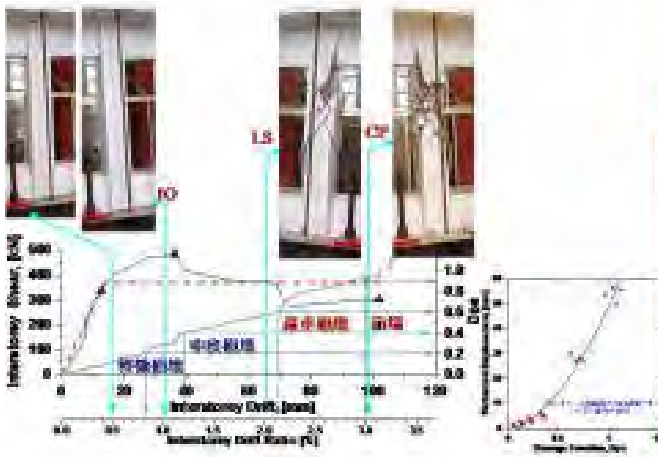
By incorporating the uncertainty in the occurrence and intensity of earthquake events in a specified period, the probability of failure with a specified damage state can be estimated using MCS.

$$P_f(D_{alw}) = P\left[(D_{P\&A} - D_{alw}) > 0\right] = P\left[\left(D_{P\&A}\left(\delta_u, \delta_y, F_y, \delta_M, \int dE\right) - D_{alw}\right) > 0\right]$$

Damage State	$D_{P\&A}$	*Description of Damage State*
Without any damage	<0.2	Slight cracks in non-structural components.
Slight damage	0.2-0.4	Slight cracks in structural components (e.g., beams and columns).
Medium damage	0.4-0.6	Flexure shear cracks in the top or bottom ends of columns. Spalling of the concrete cover. Shear cracks in the middle part of columns connected to windowsills. Obvious damage in non-structural components. Loosening of stirrups in the top or bottom ends of columns.
Serious damage	0.6-0.9	Crushed concrete in column cores. Extensive loosening of stirrups. Buckling of main bars
Total damage	>0.9	Extensive damage to columns. Extensive crushing of core concrete in columns and columns without loading capacity. Partial or total building collapse, or close to collapsing.

Seismic Structural Damage

Damage States Defined by Full Scale Experiments



Typical Low-rise RC School Buildings in Taiwan

OPTIMAL DESIGN BASE SHEAR FORCES OF RC BUILDINGS

Δ Combined Objective Function based on Seismic Reliability and LCCs

$$F_{ws} = w_1\left(\frac{R_I - R_{I,min}}{R_{I,max} - R_{I,min}}\right) + w_2\left(\frac{L_I - L_{I,min}}{L_{I,max} - L_{I,min}}\right)$$

Each objective function can be assigned a weighting value based on the decision-maker or user; a combined objective function value is then achieved through means of the linear or nonlinear combination of all weighted objective functions.

*__Chiu, C. K.__, Jean, W. Y. and Chuang, Y. T. 2013. Optimal Design Base Shear Forces for Reinforced Concrete Buildings Considering Seismic Reliability and Life-cycle Costs, *Journal of the Chinese Institute of Engineers*, 36(4), 458-470.

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NUMBERICAL CASE STUDY
SECOND DIVISION ZONE IN TAIPEI BASIN

$$V_d = \frac{IW}{1.4\alpha_y}\left(\frac{S_{aD}}{F_u}\right)_m$$

$$V_y = V_d \times 1.4\alpha_y = IW\left(\frac{S_{aD}}{F_u}\right)_m$$

Code-compatible yielding base shear forces and design base shear forces.

Base Shear Force	*R=3.0*	*R=4.0*	*R=5.0*
V_y	0.34	0.30	0.27
V_d	0.16	0.14	0.13

*Unit is W.

Structural properties and conditions for estimating LCCs.

Parameter of the unloading degrading stiffness, α	0.5
Ratio of the stiffness after yielding to initial stiffness, η	0.0
Ratio of yielding base shear forces to the total gravity load of a building, V_y/W	0.20 - 0.65
Elastic fundamental period, T_y	0.6 sec
Specified service life	40 years
Discount rate, k	0.03
Mean times of earthquake occurrence in one year, ν/T_H	0.2

NUMBERICAL CASE STUDY
SECOND DIVISION ZONE IN TAIPEI BASIN

Time History Acceleration Data of Earthquakes for the Second Division Zone in Taipei Basin

This work choses three time history acceleration data that were recorded in the second division zone in Taipei City during two earthquakes that caused major damage in Taipei City

Name	*Earthquake*	*Original PGA (gal)*
TAP003EW	19990921 (921 Earthquake)	127.4
TAP013EW	2002331 (331 Earthquake)	83.3
TAP014EW	19990921 (921 Earthquake)	106.9

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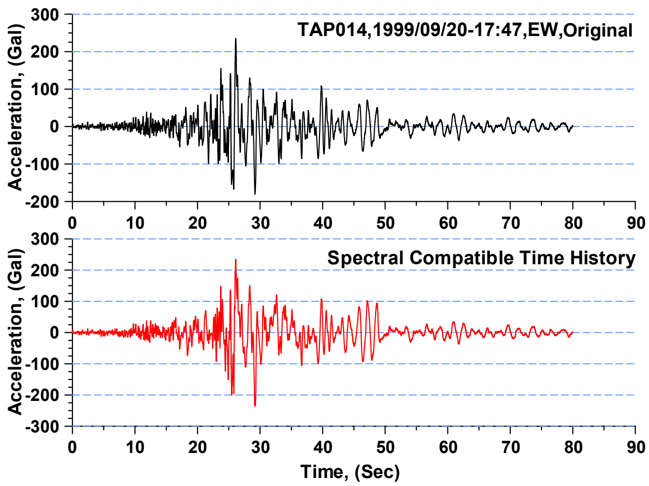
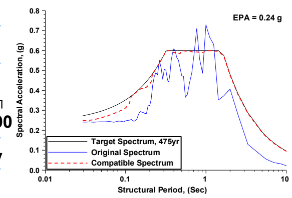
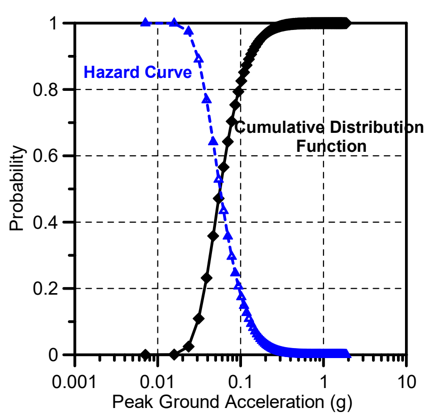
Time history data of accelerations formulated based on TAP014EW (19990921, $M_L = 7.3$)



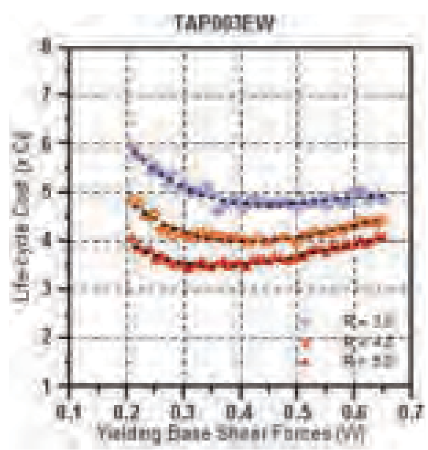


NUMBERICAL CASE STUDY SECOND DIVISION ZONE IN TAIPEI BASIN

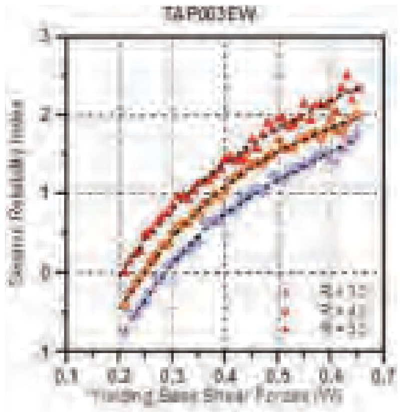
Hazard curve and CDF of peak ground accelerations within a specified service life of 40 years for the building site



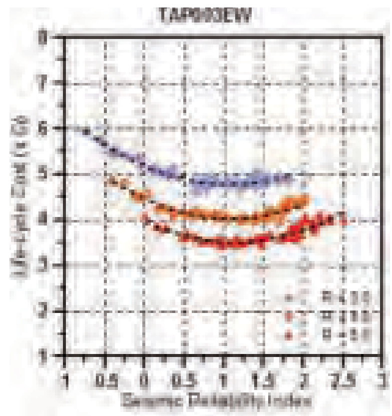
LCCs corresponding to yielding base shear forces within a specified service life of 40 years



NUMBERICAL CASE STUDY
SECOND DIVISION ZONE IN TAIPEI BASIN

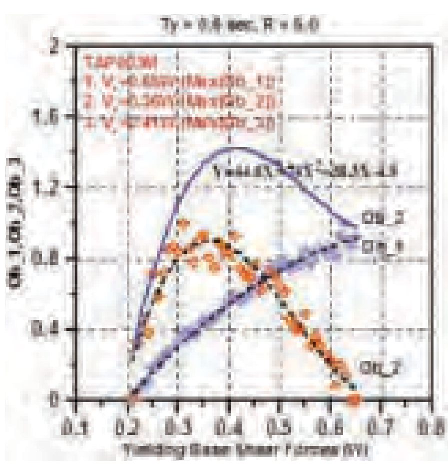


Seismic reliability indices corresponding to yielding base shear forces within a specified service life of 40 years



LCCs corresponding to seismic reliability indices within a specified service life of 40 years

NUMBERICAL CASE STUDY
SECOND DIVISION ZONE IN TAIPEI BASIN



Optimal yielding base shear forces for the ductility capacity of 5.0.
(Time history acceleration data of earthquakes is formulated by TAP003EW)

NUMBERICAL CASE STUDY SECOND DIVISION ZONE IN TAIPEI BASIN

Minimal LCCs

TAP003EW	*R=3.0*	*R=4.0*	*R=5.0*
Max (Ob_1)	0.310 (1.94)	0.310 (2.21)	0.310 (2.38)
Max (Ob_2)	*0.210 (1.31)*	*0.195 (1.39)*	*0.171 (1.32)*
Max (Ob_3)	0.310 (1.94)	0.229 (1.64)	0.195 (1.50)

TAP014EW	*R=3.0*	*R=4.0*	*R=5.0*
Max (Ob_1)	0.310 (1.94)	0.310 (2.21)	0.310 (2.38)
Max (Ob_2)	*0.205 (1.28)*	*0.195 (1.39)*	*0.195 (1.50)*
Max (Ob_3)	0.252 (1.58)	0.243 (1.74)	0.229 (1.76)

TAP013EW	*R=3.0*	*R=4.0*	*R=5.0*
Max (Ob_1)	0.310 (1.94)	0.310 (2.2)	0.310 (2.38)
Max (Ob_2)	*0.205 (1.28)*	*0.195 (1.4)*	*0.176 (1.35)*
Max (Ob_3)	0.310 (1.94)	0.238 (1.7)	0.205 (1.58)

Optimal design base shear forces for a specified service life of 40 years

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CONCLUSION

The proposed method can help both owners and investors to identify LCCs of RC buildings due to seismic structural damage within a specified service life. Their assets can then be managed using a novel procedure. Future works attempting to devise maintenance approaches should integrate this method with the deteriorating scenario of a selected RC building, seismic hazards and damage incurred by carbonation or chloride ions.

Thank you for listening

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HYBRID SIMULATION FOR EARTHQUAKE AND MULTI-HAZARD PERFORMANCE BASED DESIGN OF STRUCTURES

By Dr. D. Lau, Carleton University

Abstract

This presentation will introduce some new experimental and simulation research facilities currently being implemented at Carleton University. These facilities are part of the new multi-hazard test facilities under a partnership between Carleton University and University of Ottawa and NRC. The new facilities at Carleton include high performance hybrid simulation equipment, multi-unit mobile shake table earthquake motion simulators and modular reaction walls. The presentation will give brief summary of some recently completed research projects on the seismic performance of concrete, heavy timber-steel hybrid buildings and bridges and fire following earthquake multi-hazard risk and performance assessment by hybrid simulation.

Keywords: buildings, bridges, seismic performance, seismic risk assessment, hybrid simulation, experiments.

Biography

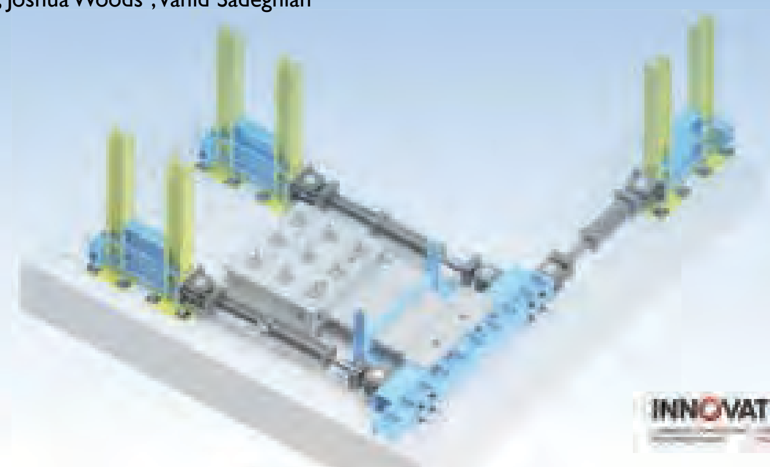
Dr. David Lau is a Professor in Civil Engineering at Carleton University. His research interests include earthquake engineering of bridges and buildings and other critical structures. He has participated in collaborations between Canada and Taiwan for many years.

NRC-MOST Taiwan Workshop on Earthquake Engineering Technologies
Ottawa, 7-9 October 2019

Hybrid Simulation for Earthquake and Multi-Hazard Performance Evaluation of Structures

David Lau[1], Jeffrey Erochko[1], Joshua Woods[2], Vahid Sadeghian[1]
[1] *Carleton University*
[2] *Polytechnique Montreal*

S. Miller, X. Cheng, I. Shaheen
A. Hassan, Z. Yu



Carleton UNIVERSITY

INNOVATION.CA

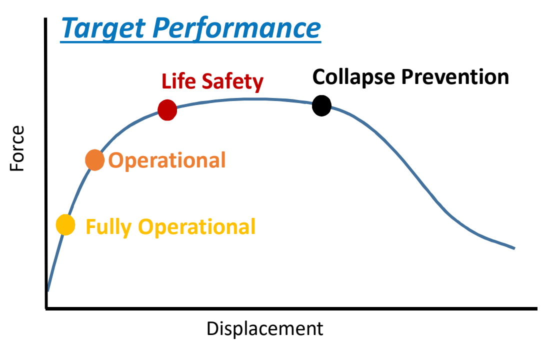
Structural Design Approach

Carleton UNIVERSITY

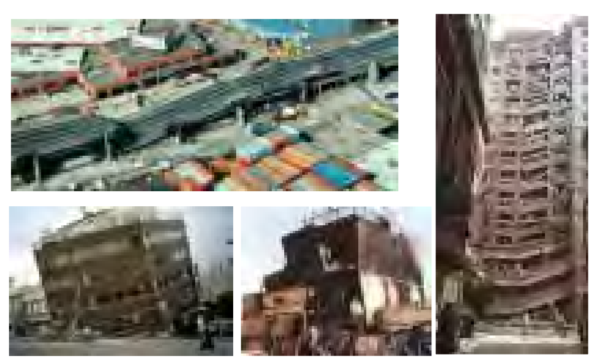
- **Prescriptive code-based design → Performance-Based Design**
- **Component vs system performance**
- **Silo Approach → Multi-Hazards**

e.g. fire following earthquake wind and earthquake

Target Performance



Performance curve for structure under seismic hazard



3

Performance-Based Seismic Design (PBSD)

- Eliminate restrictions and constraints of code-based prescriptive design approach
- Promote creativity and innovations in design
- Need to demonstrate the design can achieve the target performance

Verification by Computer Modelling

- Modelling assumptions
- Uncertainties and accuracy
- Verification of computer models

Verification by Physical Testing

- Scale model test
- Scaling and size effect

Challenges in Implementation of PBSD

- Target performance of PBSD is inherently based on ***entire structural system behaviour***, not individual structural components
- To account for the interaction effects between different structural components, need to test large scale or full size prototype structural system
- Prototype or full-scale test is prohibitively expensive and even impractical

New Experimental Method

- Hybrid Simulation/Testing Methodology

Hybrid Simulation Methodology

Hybrid Simulation

- Combine the efficiency of computer modelling and the accuracy of physical testing
- Testing of entire structural system behaviour
- Full or large scale structural system test
- Build on the foundation of substructuring techniques

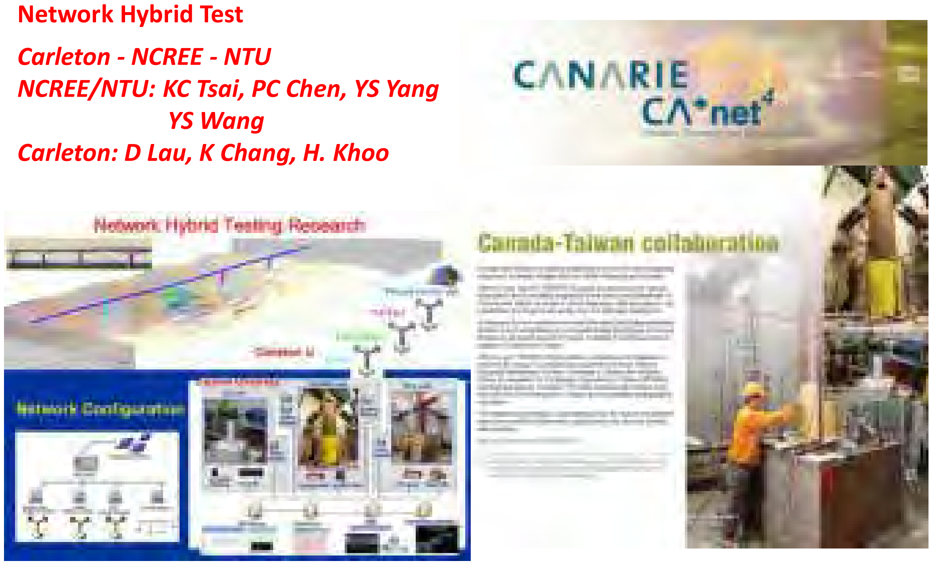
Structuring Engineering + Advanced Network + High Performance Computing + Control Theory

- Pseudynamic hybrid simulation (quasi-static)
- Real-time hybrid simulation (RTHS)
- Distributed hybrid simulation
  promote collaboration and share resources

International Canada-Taiwan Collaboration

World First Geographically Distributed Network Hybrid Test

Carleton - NCREE - NTU
NCREE/NTU: KC Tsai, PC Chen, YS Yang YS Wang
Carleton: D Lau, K Chang, H. Khoo



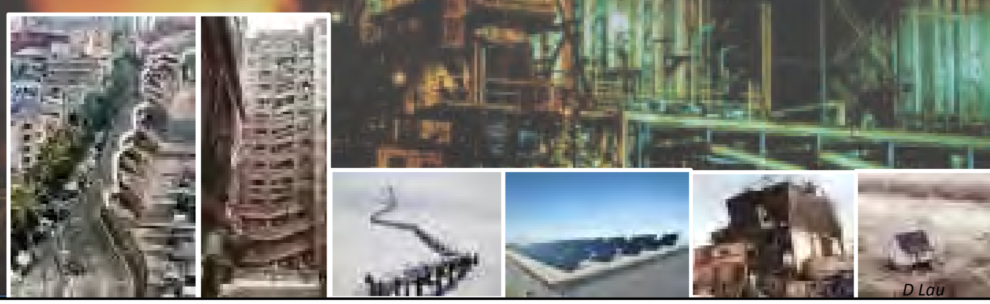
D Lau

Earthquake and Multi-Hazard Test Facility for Built Infrastructure Protection and Resilience

CFI joint Carleton-UOttawa facilities

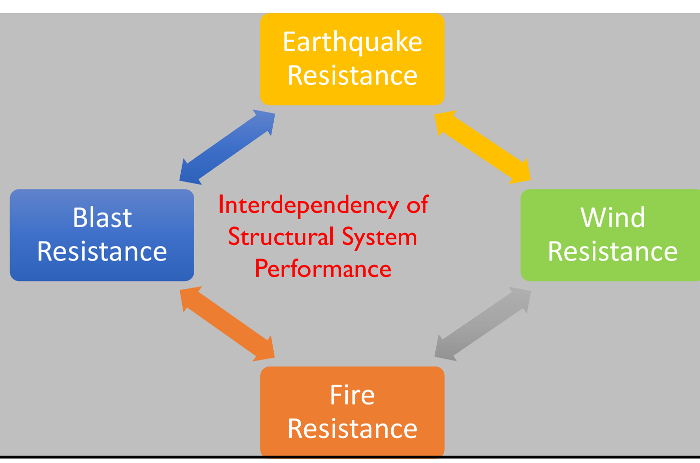
Carleton:
D Lau, J Erochko, G Hadjisophocleous, S Kenny, S Gruber, A. Braimah

U Ottawa:
M Saatcioglu, H Aoude, G Doudak, E Dragomirescu



Design for Hazards

- *Design for one hazard effects the performance of the others*
- *e.g. earthquake and wind*
- *e.g. earthquake and blast resistance*



Distributed Multi-Hazard Experimental Research Facilities

- Shared use resources
- Distributed multi-site hybrid simulation links
- Multi-hazard combinations

Carleton U Structure Lab
- EQ simulators
- Mobile shake table
- Realtime hybrid simulation
- Climate change chamber

Distributed hybrid simulation links

U Ottawa Structure Lab
- High axial load machine
- Shock tube
- High axial load PDHS
- Wind chamber

NRC Fire Lab
- PDHS
- Column, wall, floor slab furnaces

NRC Structure Lab (under plan)

Carleton CFI Facilities

- Earthquake Simulation System
  4 mobile, reconfigurable high performance shake tables
  Each shake table 6 DOF's

- Reconfigurable concrete block reaction wall

- Real-time and pseudo-dynamic hybrid simulation controller
- Real-time high speed actuators

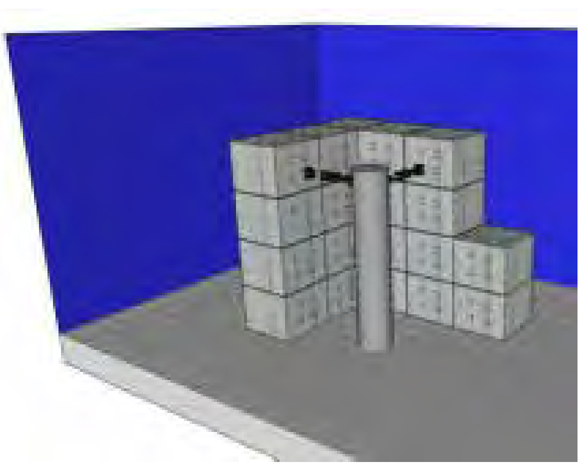
- Hybrid fire simulation facilities
  NRC-Carleton Collaboration

- Climate change hybrid simulation facilities

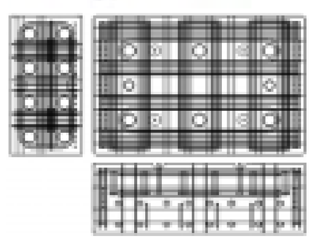
University of Ottawa CFI Facilities

- 2.5MN high axial load universal testing machine
- Pseudo-dynamic hybrid simulation controller
- Blast load simulation facility – Shock tube
- 3D wind pressure simulation chamber

Reconfigurable Block Reaction Wall (CU)



- 1.8m x 1.2m x 0.6m (High)
- 3 TON
- 6 - 47mm Post Tensioned Rods
- 2 Shear Pins on top and bottom



Provides maneuverability and flexibility in structural testing for application of lateral loads on test specimens

Mobile 6-d.o.f. Motion Table System

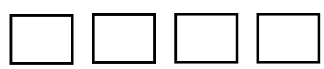
New type of shake table system
Electro-mechanical system
6 Deg-of-Freedom
3-Tonne Capacity each
Long stroke length

4 easily reconfigurable
motion simulation system

Mobile reconfigurable shake table system

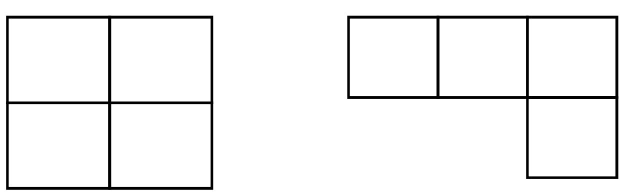
Flexible and Versatile

Use separately as multi-unit system

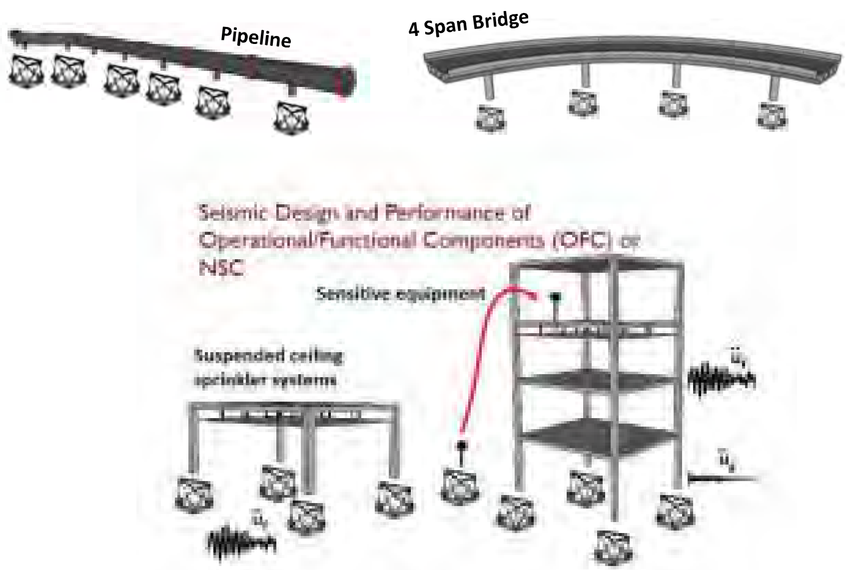


Combine to form a larger single-platform system
with higher payload capacity

4x capacity



Mobile reconfigurable shake table system



Experimental and Analytical Simulations of Suspended Non-Structural Systems in Super Tall Building under Long Period and Duration Earthquakes

ILEE (International Joint Research Laboratory of Earthquake Engineering)

Part 1: Large Scale Shake Table Tests at Tongji University

- K. Kasai (TIT, Japan)
  H. Jiang (Tongji, China)
- Large size ceiling
- Bracing and dampers
- Bi-directional horizontal motions

- **Analytical Investigation led by S. Motoyui (TIT, Japan)**

Part 2: Multi-Unit Shake Table Tests at Carleton University

- D. Lau, J. Erochko
- Vertical ground motion component
- Vertical and/or rotational motion components of tall building floor response motion inputs
- Different layout configurations

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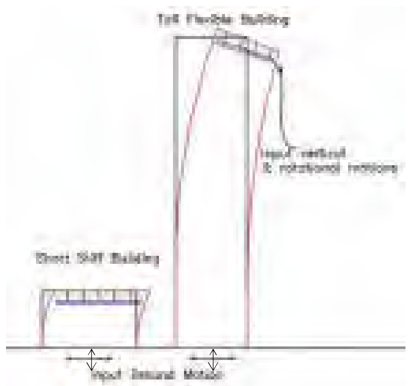
Tall Building Floor Response Motions to Suspended Ceilings



Multiple DOF Input Motions to Ceiling

17

Test Setup at Tongji Shake Table

Figure 18. Japanese Ceiling Test Setup at Tongji University

18

Japanese vs Canadian Ceilings

- Hanging Wires vs Rods
- Compression Post with 4 Diagonal Wires vs Diagonal Angle Braces

Figure 19. Canadian/Chinese Suspended Ceiling with Seismic Bracing [12]

Figure 20. Japanese Suspended Ceiling with Seismic Bracing

Testing of Japanese Suspended Ceiling

Figure 21. Japanese Suspended Ceiling (Left = Free-Free Boundary, Right = Fix-Free Boundary)

Results

Figure 23. Fixed-Free Ceiling Damage

Figure 24. Result of Japanese Tests

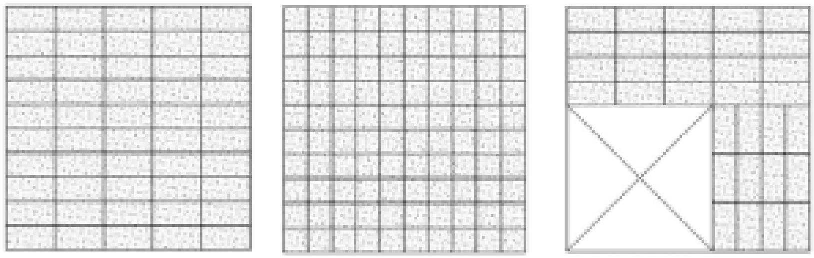
Part 2: Carleton Test Frame Design

- I Shapes, HSS Bracing, OWSJ Ceiling support
- 6m x 6m x 1.0m
- Pinned Connections – Easy assembly/disassembly

Figure 24. Side View of Test Frame

Figure 25. SAP2000 Model of Test Frame

Ceiling Configurations



(a) (b) (c)

Figure 27. Suspended Ceiling Configurations

a) 1200mm x 600mm tiles full frame b) 600mm x 600mm tiles full frame

c) 1200mm x 600mm tiles L-Shape

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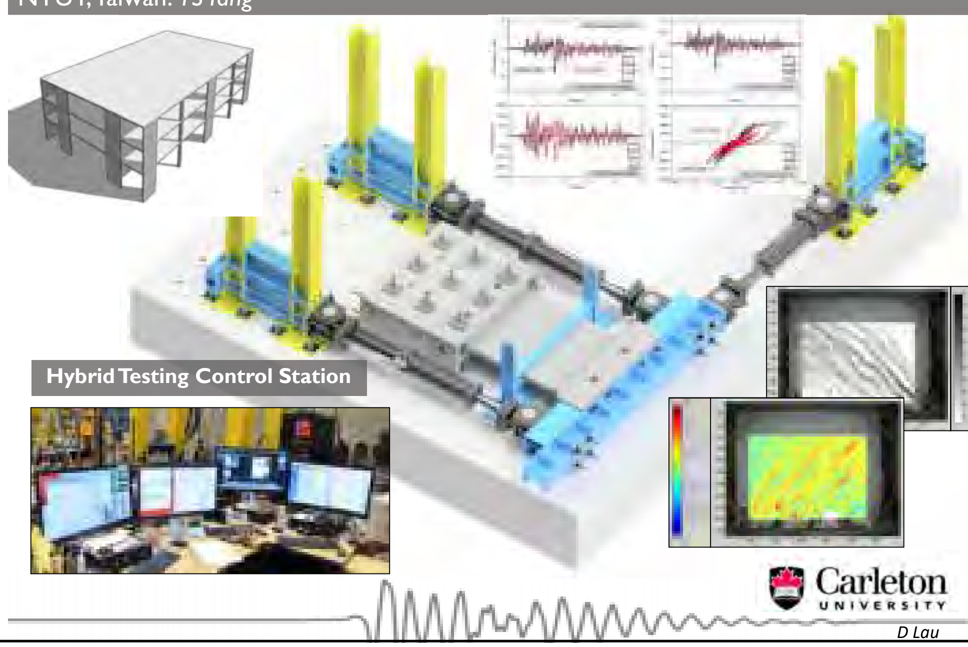
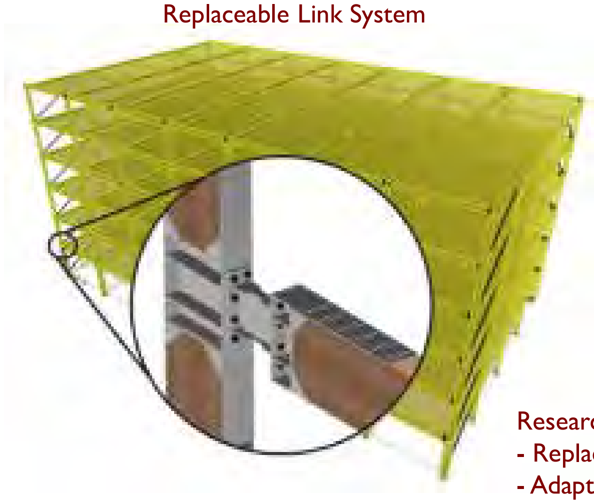
Building Scale Performance Evaluation of RC Shear Wall Building by Hybrid Simulation
Carleton U: *J Woods, D Lau, J Erochko*
NTUT, Taiwan: *YS Yang*



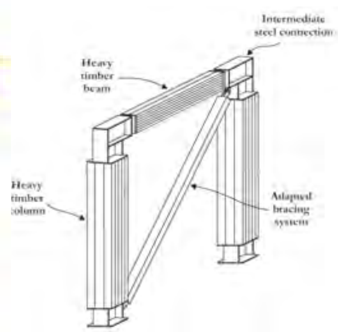
Carleton Hybrid Simulation 7-Storey Hybrid Timber Systems

Ryan Gohlich, Colin Gilbert and Dr. Jeffrey Erochko

Replaceable Link System



Advanced Bracing Systems



Research Interests:
- Replaceable Ductile Link Systems
- Adaptation of advanced bracing systems for heavy timber structures

D Lau

Seismic Assessment of a New Type Timber-Steel Structure using Hybrid Simulation

S. Miller, J. Woods, J. Erochko, and D. Lau

- Recently, a Multi-Hazard Research Facility has been established at Carleton University with the focus of hybrid simulation in multi-hazard applications;
- Hybrid simulation is used to investigate the feasibility of a new type of combined heavy timber-steel brace SFRS in mid-rise structures located in earthquake prone regions
- Incremental dynamic analysis/simulation is conducted to obtain fragility relationships of the performance of the new structural system
- **Green construction technology**