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10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1895

# A SHORT, SELECTIVE, PERSONAL PERSPECTIVE ON THE ROLE OF STRUCTURAL CONSULTANTS AND DESIGN PRACTITIONERS IN THE DEVELOPMENT OF CANADIAN EARTHQUAKE CODES WITH A FEW EXAMPLES

Dr. Ron DeVall, Ph.D., P.Eng.[1]

## ABSTRACT

Practicing Structural Engineers provide an important and rich source of knowledge and experience that can be a valuable resource in the development of structural design codes. Their role can range from initiating changes and requirements to informing and shaping suggestions from others. Their comments can help decide the need for a change and whether it will address few or many structures, whether it will be practical or impractical in the field, whether it will be costly or not; and what the overall impact will be on the whole construction process, not just on the structural design. A few examples are given illustrating the involvement of structural designers and consultants in the development of earthquake code requirements and guidelines.

Keywords: Structural Engineers, Consultants, Earthquake, Codes

## INTRODUCTION

The intent is to show that structural consultants and design practitioners can be a rich source of experience, insight, and lessons learned (often from our occasional "less than perfect" job). We are always learning as the "usual" buildings almost always have something "unusual" about them and the "unusual" buildings often require special studies and additional special consultants. The perspective design practitioners bring can often help take "good science" – which is often not "good engineering" and develop it into "good engineering based on good science".

The following examples were chosen as they illustrate work that was initiated by consultants or had significant consultant involvement and that resulted in code changes, new code clauses or in "Design Guidelines". A list of some of the engineers involved in each example is given in the Appendix.

The presentation will cover the following topics and examples:

- A brief outline of the Canadian National Building Code (NBCC) System of Committees (the NBCC is a model code adopted by the Provinces).

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- A significant change to the 1984 Canadian Concrete Design Standard (CSA-A23.3).
- The idea that single storey buildings with flexible (wood or steel) roof diaphragms are a separate class of buildings and are not treated properly by our current codes.
- The introduction of 6-storey wood frame construction in the British Columbia Building Code in 2009 (and it will probably end up in NBCC 2015).

With a brief summary of:

- B.C. Schools Project – Peer Review Groups
- Design of Below Grade Structure Retaining Walls
- Canadian Seismic Strategic Network

**A Brief Outline of the Canadian Building Code System and Regulatory System**

- Building Codes are a provincial responsibility.
- The First National Research Council "Model" National Building Code was produced in 1941 and revised in 1953 then progressively after for use by the Provinces.
- It contained loading provisions and material design standards – which were subsequently spun off as design standards under the auspices of the Canadian Standards Association (CSA).
- The Provinces adopt the NBCC (often modified somewhat) – usually 1 to 3 years after publication.
- The NBCC and CSA Standards are written by volunteers, are broadly based, and are consensus documents. The NBCC is submitted for public review and every comment must be addressed.
- The NBCC and its Standing Committee's membership must satisfy a matrix of:
    - Regulatory members
    - Industry members, consultants and practitioners as well as "industry reps"
    - General interest – usually researchers
    - No one group can outvote the other two groups
    - CSA Standards Committees must also satisfy a broad membership matrix
    - Note that all members volunteer their time and only the NBCC group get travel expenses
- A simplified flow chart for the NBCC Commission, its relationship to the provincial code enacting bodies, its Standing Committees, and the Canadian Standards Association Design Standards is given in Figure 1.

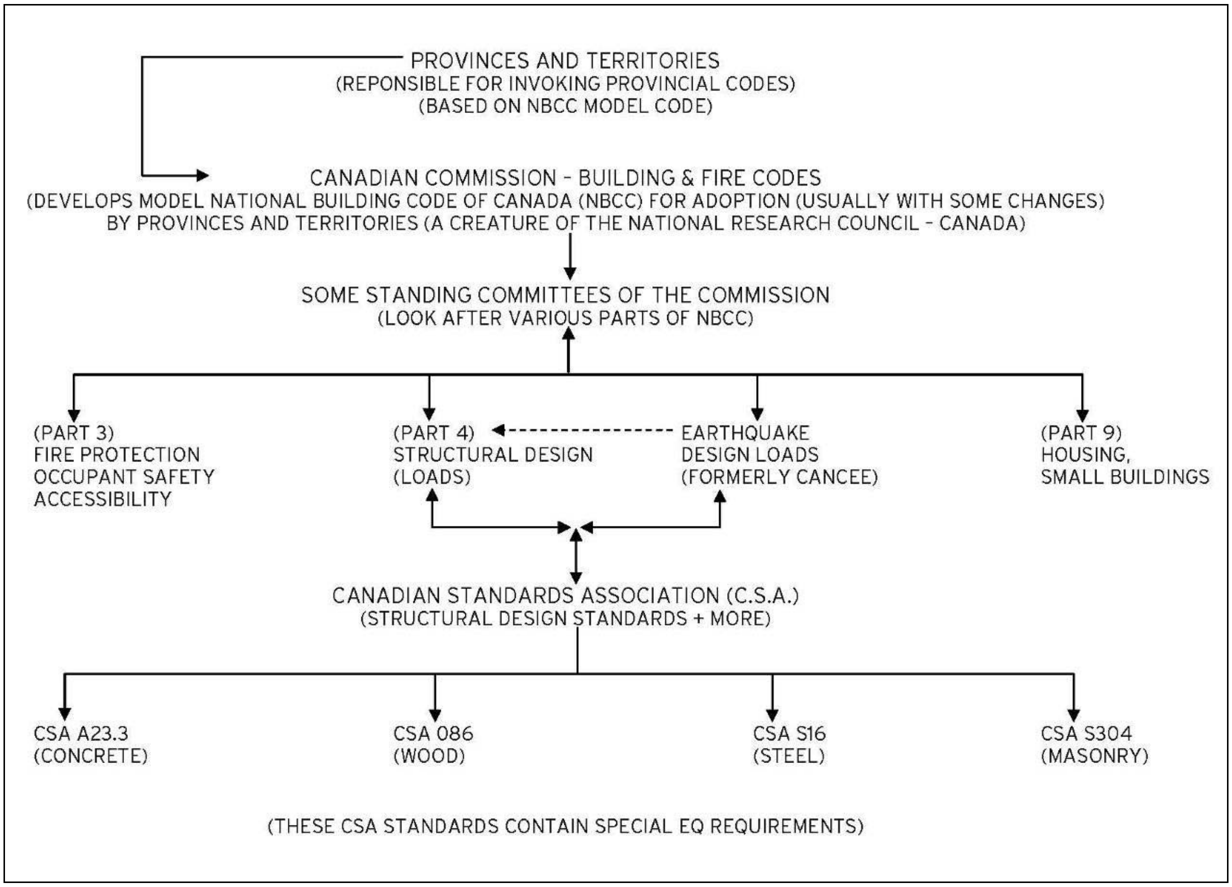


**Figure 1 – Relationship Between Provinces, National Building Code of Canada, and Canadian Design Standards**

### *Standing Committee on Earthquake Design (formerly Canadian National Committee for Earthquake Engineering – CANCEE)*

This committee was formed about 1965 and has a membership of:

- Members from the East and West "high" earthquake zones and from the central "very low" region
- Seismologists
- Geotechnical consultants and researchers
- Structural consultants, researchers, and specialists in analysis
- Representatives from the major CSA design standards
- About 24 people in total – some wearing several hats

### *Registration of Structural Engineers*

- Registered by self-regulating provincially authorized organizations.
- Typically – a bachelors degree and 4 years of practice is needed to register – no exams.
- Typically – cities do not check structural drawings (Toronto and Ottawa apparently do).

## DEVELOPMENTS IN A23.3 – CANADIAN CONCRETE DESIGN STANDARDS

### Development of Ductile Wall Requirements in CSA A23.3-1984 (Both Cantilever and Coupled Walls)

NBCC 1970 – Included Ductile Concrete Frames and Walls – but referenced the 1967 U.B.C. for design details. The NBCC Seismic Requirements were very similar to U.B.C. requirements.

CSA A23.3–1970 – No special requirements for ductile structures (basically NBCC said to go to the U.B.C.)

CSA A23.3–1973 – Added a whole section (Chapter 19) on special provisions for seismic design. Many of these were very similar to U.B.C. requirements, particularly for frames.

However, the wall section was different. It was based on the work of several practitioners and a researcher and required a ductility calculation, capacity design for shear, extra ties on the concentrated "tension zone" reinforcement and $v_c = 0.0$ in the hinge region. The foundation had to develop the walls capacity. The section on ductility referred to the Commentary which gave advice as to how to calculate the non-linear curvature demand in the hinge which was set at 3 (this turned out to be a problem).

During this time, Professor Tom Paulay at the University of Christchurch in New Zealand had been quite busy and had published extensively on earthquake design of concrete walls, in particular:

- 1975 – A book by Park and Paulay "Reinforced Concrete Structures" which had several sections on ductile walls – both cantilevered and coupled with diagonal reinforced headers.
- 1975 – An article in the Canadian Journal for Civil Engineering (CJCE) titled "Design Aspects of Shear Walls for Seismic Areas". This was very similar to the Park and Paulay 1975 book and brought his ideas to a broad Canadian audience.
- 1975 – An article published with Professor M. Uzemeri in the same CJCE journal titled "A Critical Review of the Seismic Design Provisions of the Canadian Code and Commentary". The main point was that a curvature ductility demand of 3 in the hinge was clearly very unconservative. See Figure 2, which are scans from this paper. Once again, this article was widely seen by a Canadian audience.
- This was a damning critique and the A23.3 Commentary section was withdrawn.
- 1977 – The 1973 A23.3 was "metrified" with some minor changes. The ductility calculation was still "based on generally accepted principles" but no Commentary reference was given nor were any other references.
- The next CSA A23.3 Concrete Design Standard was scheduled for 1984. By this time the 1982 New Zealand Concrete Design Standard was published incorporating Paulay's work and including a ductility equation and infused with capacity design principles. Tom Paulay also spent several months at the University of British Columbia in Vancouver in 1982 and gave a series of lectures in the evenings that was attended by a large number of Vancouver structural design engineers.

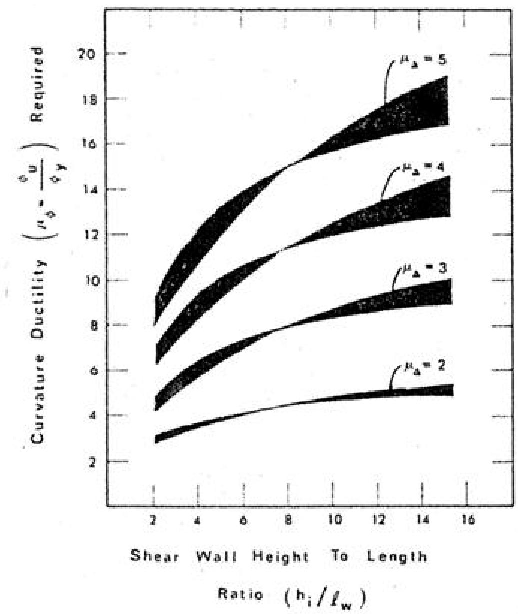


FIG. 5. Curvature ductilities ($\mu_\phi$) required to develop various system ductilities ($\mu_\Delta$) in cantilever shear walls with different height to length ratios.

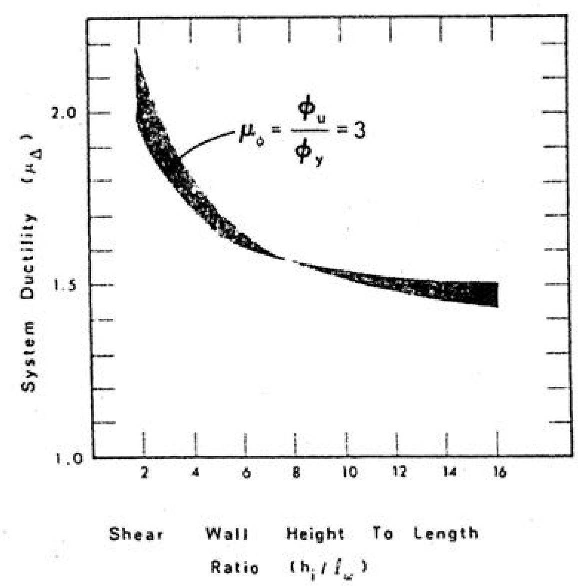


FIG. 6. System ductilities ($\mu_\Delta$) in cantilever shear walls with different height to length ratios corresponding to curvature ductility ($\mu_\phi$) of 3.

**Figure 2 – Figures from Paulay and Uzemeri Paper Critical of CSA A23.3 - 1973**

- In the meantime, the CSA A23.3 Committee was meeting and dealing with the start-up of "ductile wall" design given in the 1973 standard and having to withdraw the commentary on how to apply it. They decided that since most of A23.3 is closely based on ACI, the whole 1973 section on walls would be withdrawn and replaced by the ACI provisions. This news was brought back to the Vancouver consultants by consultants who were CSA A23.3 committee members.
- The Vancouver consultants reviewed the ACI wall provisions and felt that:
  - They did not reflect capacity design approaches.
  - They gave walls "too strong" in flexure.
  - They gave walls "too weak" in shear.
  - They gave undersized footings for the walls.
- The consultants revolted and said they would:
  - Write a new seismic section based on ACI for frames, and Paulay's work and the New Zealand code for walls, and submit it to the CSA A23.3 committee for their consideration.
  - It would be an extension, revision, and update of the wall provisions that appeared in A23.3-1973.
  - The Group consisted of several structural consultants with enthusiastic help from several of the professors of structural engineering at the University of BC (UBC).
  - The ductility equation was developed independently to ensure understanding.
- All this was done, and to provide some comfort to the CSA A23.3 committee the group asked Professor Tom Paulay and Professor Vitelmo Bertero to review the drafts and provide comments. Both very generously did so, and their comments were incorporated. Figures 3 and 4 show scans of excerpts from their correspondence.

- The package was submitted to A23.3, was accepted, and became part of the A23.3-1984 Design Standard.

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July 18, 1983

Professor Noel D. Nathan
Department of Civil Engineering
2324 Main Mall
The University of British Columbia
Vancouver, B.C., Canada
V6T 1W5

Ref: Comments on Proposed Code Provisions for Seismic Design of Shear Walls

Dear Noel:

I must apologize for not sending my comments before today. Unfortunately your letter and copy of code proposals arrived when I was in Japan.

I have gone over the proposed code provisions for the design of shear walls and attached are my specific comments on the different sections. There are many questions, probably because some of the terms used are defined more clearly in other chapters which I do not have.

In general I consider that philosophically the proposed changes are good and represent an improvement with respect to the 1982 proposed revisions to ACI 3-18-77. My major concerns are: first, that I believe that the profession will have a hard time to interpret and, therefore, to apply correctly the new proposals for the design of shear walls; and second, that the required design method for shear of walls, particularly in the case of the individual walls of a group of coupled walls, does not seem to be a sound method, according to the results of studies that I have been conducting for the past 10 years.

As you will notice from my comments, I consider that many of the definitions are vague. In many cases it will be necessary to illustrate with proper sketches the definitions of some terms. For example, the definition of $l_i$, $P_c$ ( which I believe should be $P_{ci}$) etc. Furthermore, in most of the cases it is not stated how the different terms should be computed (evaluated numerically). These are the cases of $C_c$, $M_{pi}$, $M_{up}$, $P_c$, $P_s$, $P_a$, $Y_w$, ($M_{st}$) etc. I am afraid that without some illustrations, numerical examples in the commentary, the designer will have a hard time figuring out how to compute these terms. Perhaps all these are defined in a separate chapter or other code which defines the seismic loads and the methods to estimate all the "demands" as well as the supplies. A comprehensive code should consider in an integrated way the two sides of the design equation, [ demands ≤ supplies ] i.e., should specify the seismic design-forces and how the different demands are estimated before regulating how the demanded stiffness, strength, stability, and ductility should be supplied.

**Figure 3 – Excerpt from Bertero Letter (5 pages of comments)**

Department of Civil Engineering
**University of Canterbury** Christchurch 1 New Zealand

13 June 1983

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MAIN
DIV.
NUMBER

Dr. R.H. DeVall
Read Jones Christoffersen Ltd
1161 Melville Street
Vancouver B.C.
CANADA V6E 2X7

Dear Ron,

I meant to write to you much earlier when I was hoping to study the first draft of your "Special Provisions for Seismic Design". However, other commitments prevented me from doing so. A few days ago I received the more up-to-date version of your draft and I spent the last two nights reading it.

It represents great improvement on the latest ACI Appendix A, with which we were very disappointed. Please find enclosed a number of comments. They represent my usual bluntness. However, the remarks are made with the conviction that they could possibly assist. I felt that your group made significant and very useful changes in the design of shear walls which are undoubtedly popular in B.C. Ductile frames on the other hand received no similar attention from your group. You will find numerous comments relevant to frames accordingly.

I enclose a xerox copy of Chapter 21 with my notes and the numbers which refer to the comments, typed separately. Also I enclose a paper, relevant to joint design.

I look forward to hearing from you in due course as to how things are developing.

In the hope that these notes will be of some help to your group I wish you success and remain,

Yours sincerely,

T. PAULAY
Professor of Civil Engineering.

Encls.

**Figure 4 – Excerpt from Paulay Letter (11 pages of comments)**

***Developments since 1984:***

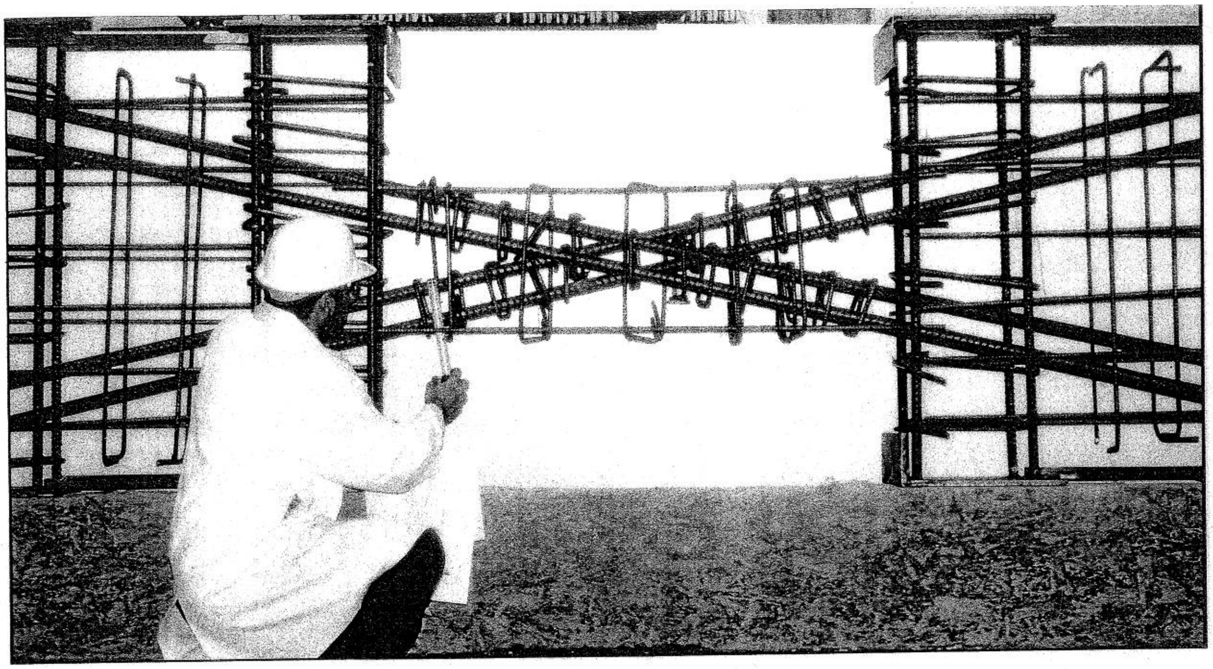
The group of consultants and UBC researchers continued to meet, with various topics discussed. Two tests performed by Professor Perry Adebar at UBC addressed topics of interest:

- A 60’ x 6’ wall was tested through non-linear cycles to obtain information on ductility, drift, and stiffness.
- Shallow diagonal headers are commonly used in flat plate residential buildings in Vancouver. Questions had been raised by several people about their effectiveness. The group felt these concerns unwarranted but felt a test should be done. A “typical” shallow header was tested at UBC by Professor Perry Adebar and the results are shown in Figure 5.
- The above, along with non-linear dynamic analysis of wall systems done at UBC by Professor Perry Adebar provided the basis for changes to the 2004 CSA A23.3 wall clauses which moved towards a displacement based approach.

There are Several Papers in the Conference on Possible Future CSA A23.3 Earthquake Design Requirements.

## Recent Experimental Results

(Perry Adebar, Riyadh Hindi and Emilio Gonzalez , "Full-scale Test of a Coupling Beam from a High-rise Ductile Core Wall Building" Department of Civil Engineering, University of British Columbia, *Report,* August 2000.)



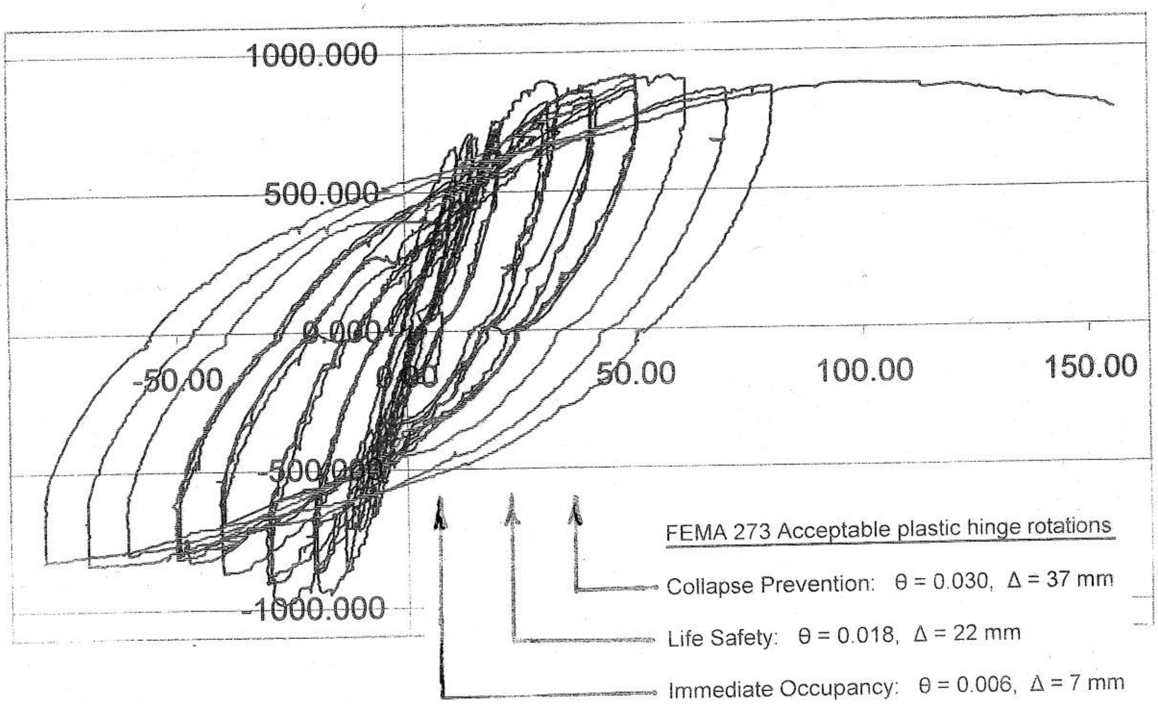



**Figure 5 – Test at UBC of Shallow Diagonal Header**

## ONE STOREY STEEL BUILDINGS WITH FLEXIBLE ROOF DIAPHRAGMS – CANADIAN STANDARD S16 – LIMIT STATES DESIGN OF STEEL STRUCTURES.

- 1989 – The first specific seismic design requirements were introduced in a new separate clause in CSA-S16. (Coincidently, a fabricator with plants on each side of the border had a partially constructed steel moment frame building damaged in the Loma Prieta earthquake. Cracks in the bottom flange beam to column welds were attributed to the state of construction, the cracked welds had not been tested, and had all been done by a welder who had recently been let go. They were fixed up and construction continued). A requirement for connections to be designed for AgFy of braces was introduced and these forces were to apply to other connections "participating in the lateral system". This was felt to be somewhat ambiguous by many designers. Designers familiar with concrete design felt this should apply to diaphragms, drag struts, and foundations – but it became a topic of discussion amongst designers as to what precisely to do – particularly when it came to the steel deck as this was particularly difficult to deal with. I suspect there is many a building designed in the early days where "capacity design" stops at the underside of the deck.
- 2001 – The CSA S16 standard now explicitly states that a capacity design approach is to be used – from diaphragm to drag strut through the lateral system and into the footings/foundations. The steel deck is the most problematic design challenge as the AgFy requirement can generate very large forces. Designers of one storey tilt-ups and block wall buildings are also having difficulties in getting the steel deck to work.

2003 – CSA16 Issues a revision for "conventional construction" which allows design force reductions for the deck for cases where some ductility in the deck can be found.

**During this period a few things are going on:**

- Professor Robert Tremblay is starting to test steel deck diaphragms at Ecole Polytechnique in Montreal using various connection details.
- A consultant, Gerry Weiler, P.Eng., who designs tilt-up buildings with steel deck, gathers a group of other consultants in Vancouver to discuss this problem. Some are aware of Professor Tremblay's work.
- The group contacts Professor Tremblay to offer to work with him. This offer is taken up, and discussions begin.
- Professors in the structural group at UBC (Professor Carlos Ventura) have undertaken a project measuring the lateral periods of steel roof diaphragms in the field. They become part of the group.

Gerry Weiler, P.Eng., has a very good relationship with his clients, has managed to get them to use a variety of different connection techniques on their roofs, and has obtained data from the field, based on actual installations, of the cost and practicality of Button Punches vs. Screwed Side Laps as well as Puddle Welds vs. Welded Washers vs. Hilti Pins as deck attachments.

Tests performed by Professor Tremblay indicate that Puddle Welded Deck with Button Punch Side Laps perform very poorly, that Welded Washers are strong (but field installation shows them to be costly) and that the Hilti Pins show some ductility due to deck deformation. The direction for better, cost efficient performance seems to point to screwed side laps and pins instead of welds.

However, the consultant group are getting reports from the field identifying some issues with using pins.

- In a few cases, during the night after installation, some pins at deck laps had popped off.
- When thin, double angle top chords are used, the pins often did not set properly.
- "Hat sections" top chords often are not quite flat across the top and caused some problems in seating the pins.

The Hilti company has been working closely with the group since the beginning and responding with testing of their own to address issues as they arise.

***Where things stand now:***

- CSA S16 for 2009/10 will have guidelines on how to achieve a steel deck diaphragm with some ductility.
- Linear and non-linear analysis of one storey flexible diaphragm buildings indicate they behave completely differently from rigid diaphragm buildings and are dominated by the behaviour and period of the diaphragm. In some cases a long period elastic diaphragm increases demand on the lateral system, while the usual thinking would be that a long period should reduce demand. See Figure 6.
- A group of researchers and consultants led by Professor Tremblay will be working on developing a separate and distinct set of requirements for R values, overstrengths, periods, etc. for the class of one storey flexible diaphragm buildings for inclusion in the 2015 NBCC and CSA-S16.

If you are interested in this topic, there are several papers being presented in the conference by Professor Robert Tremblay and others.

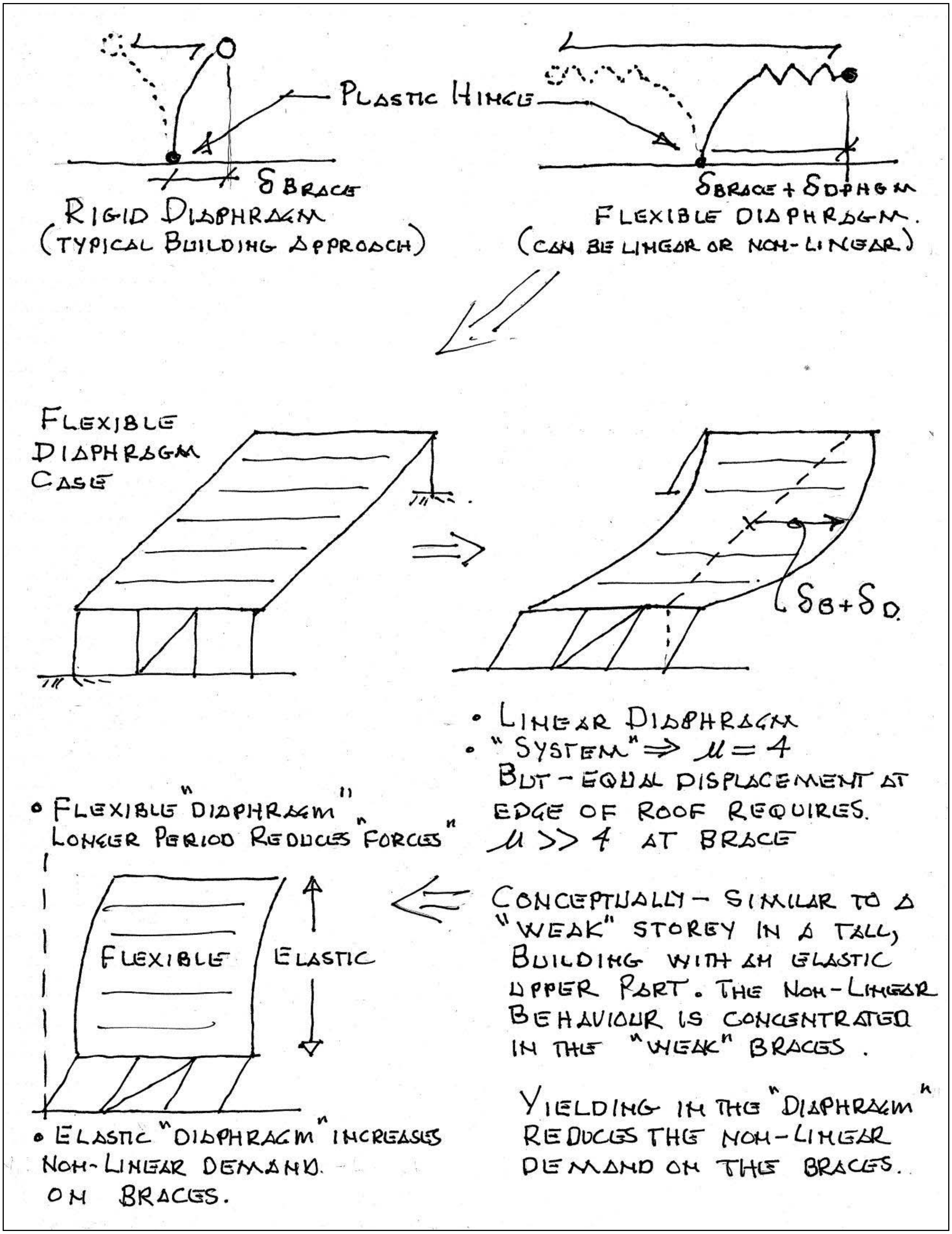



**Figure 6 – Behaviour of One Storey Buildings with Flexible Diaphragms**

**The Introduction of 6 Storey Wood Frame Construction in the British Columbia Building Code.**

Generally, requests for code changes are submitted with supporting documentation to the appropriate Standing Committee for consideration. This is all done on a 5 year code cycle that includes a public review session. Occasionally changes are made quickly and these changes are usually emergency changes driven by a pressing public safety concern. However, a change can also be made quickly when political decisions drive them and that is the case here.

The background to the change is the perilous state of the forestry economy with the big wood producing provinces of British Columbia and Quebec hit hardest. Small interior towns dependent upon logging, sawmills, and pulp and paper, have had there economic base severely reduced. This has come about through trade and tariff disputes with U.S. producers, the high Canadian dollar compared to the U.S. dollar, the crash of the U.S. housing market, and the Pine Beetle which is surviving warm winters and cheerfully eating its way through the BC pine forests and leaving them dead.

Codes are a Provincial responsibility and the premier of British Columbia suddenly decreed that the current 3 storey limit on wood frame building construction would be raised to 6 storeys. The act would be introduced within a few months and come into force about 6 months later.

This took the design community by surprise and several issues were raised such as fire protection, building envelope, and structural concerns.

The main structural concerns were shrinkage and – by far the most problematic – lateral design in a high seismic zone. The typical 4 storey building (made 3 stories through the clever use of the definition of where grade is) was already difficult to design.

The Province engaged consultants to review all of these issues. Lateral analysis of some designs was also undertaken by Forintec, a wood research facility at the University of British Columbia.

Several structural consultants expressed concerns about sway mechanisms forming at the lower levels as wood frame plywood shear wall platform construction is particularly susceptible to this. It is difficult to use the "strong continuous column" strategy used in concrete moment frames and steel moment and braced frames to mitigate against this type of failure.

These structural consultants formed a small group and offered to help (an offer accepted) by developing a few representative shear wall designs and performing some linear and non-linear analysis. This group included a Professor of Structural Engineering (Analysis Specialist) from UBC.

***They found that:***

- The usual approach to design of these buildings was to use assume them to be stiff, short period buildings and use a static analysis. This is a reasonable approach as NBCC has a high frequency, short period spectral cut-off that runs flat to about a 0.5S period.

- When this approach was used on a "typical" 6 storey wood building with 40mm (1.5") of concrete topping the deflections exceeded the limits and the hold downs became almost impossible to deal with.
- When this became apparent, the engineers being very clever engineers, decided to do a dynamic analysis. They could use a "rational" period up to 2 times the "static design rule" period and could take the base shear as 80% of the base shear calculated using this longer period. Using this approach, the deflections, overturning, and base shear were much reduced – and all was well (the period was about 1 second).
- However, when a suite of 10 earthquake records scaled to Vancouver were applied and analyzed using 2 non-linear approaches, about 5 of the 10 records resulted in large sway deformations and "collapse". This was deemed unacceptable.

***The solution:***

- In order to produce a design to avoid this, the model was strengthened which also results in a stiffer structure since for wood panel shear walls the strength and stiffness are pretty much a function of the number of nails.

The solution was basically:

- Use the static approach – or –
- Use the static approach using a "rational" period calculation and multiply the forces by 1.2 – or –
- Use a response spectrum dynamic approach with the forces multiplied by 1.2 after scaling the dynamic base shear up (no scaling down) to 100% of the "usual static" approach.

These were accepted, included in the Provincial Government order, and also appear as part of the Association of Professional Engineers and Geologists of BC guidelines for these buildings that are posted on their website. Selected pages are reproduced in Figures 7 and 8.

**PROVINCE OF BRITISH COLUMBIA**

**REGULATION OF THE MINISTER OF HOUSING AND SOCIAL DEVELOPMENT**

*Local Government Act*

Ministerial Order No. M121

DEPOSITED
APR 3 2009
B.C. REG. 146/2009

I, Rich Coleman, Minister of Housing and Social Development, order that the Schedule of B.C. Reg. 1/2009 is amended

(a) *in Sentence 3.2.2.45 (3), as enacted by section 5, by striking out* "Clause 3.2.2.45.3.(v) or (vi)" *and substituting* "Subclause 3.2.2.45.(1)(d)(v) or (vi)" *and by striking out* "noncombustible" *and substituting* "noncombustible", *and*

(b) *by adding the following sections:*

13 *Add the following new Sentence 4.1.8.11.(11):*

11) Where the fundamental lateral period, $T_a$, is determined by Clause 4.1.8.11.(3)(d) for *buildings* constructed with 5 or 6 *storeys* of continuous *combustible* construction as permitted by Article 3.2.2.45. and having an SFRS of nailed shear walls with wood-based panels, the lateral earthquake force, V, as determined in Sentence (2) shall be multiplied by 1.2.

14 *Replace Sentence 4.1.8.12.(6) with the following:*

6) Except as required by Sentence (7) or (10), if the base shear, $V_d$, obtained in Sentence (5) is less than 80% of the lateral earthquake design force, V, of Article 4.1.8.11., $V_d$ shall be taken as 0.8V.

15 *Add the following new Sentence 4.1.8.12.(10):*

10) The base shear, $V_d$, shall be taken as 100% of the lateral earthquake design force, V, as determined by Article 4.1.8.11. for *buildings*

a) constructed with 5 or 6 *storeys* of continuous *combustible* construction as permitted by Article 3.2.2.45.,
b) having an SFRS of nailed shear walls with wood-based panels, and
c) having a fundamental lateral period, $T_a$, as determined by 4.1.8.11.(3).(d).

April 3/09
Date

Minister of Housing and Social Development

*(This part is for administrative purposes only and is not part of the Order.)*

**Authority under which Order is made:**

Act and section:- *Local Government Act*, R.S.B.C. 1996, c. 323, s. 692

Other (specify):-

April 2, 2009 R 355/2009/27

APEGBC • April 2009/Revised June 10, 2009

*APEGBC Technical & Practice Bulletin*
*5 and 6 Storey Wood Frame Residential Building Projects (Mid-Rise)* 40

**Figure 7 – Except from BC Building Regulation Enabling 6 Storey Wood Frame Buildings**

3.4.3 It is recommended that there be a start up meeting with the contractor to clarify issues related to the implementation of the design drawings which would address such matters as drilled holes and notching which is allowable in structural members as well as shrinkage issues. This meeting should include the mechanical and plumbing trades.

3.4.4 Shop drawing design and submission requirements for specialty structural elements such as trusses, guardrails, canopies, windows etc. should be stated on the structural drawings.

3.4.5 *Field Review* requirements for the specialty structural elements should be stated on the structural drawings.

3.4.6 Assurance letter requirements for the engineer designing specialty structural elements should be specified on the structural drawings. It is recommended that Schedule S, as contained in APEGBC's *Bulletin K: Letters of Assurance and Due Diligence*, should be used as an assurance letter for all specialty structural engineering services provided.

### 3.5 DESIGN AND DETAILING OF WOOD SHEARWALLS AND DIAPHRAGMS

(Some of these requirements apply only where seismic forces govern lateral design)

3.5.1 **SHEARWALL DESIGN FORCE LEVELS (Please refer to Appendix A – Ministerial Order No. M121 regarding seismic design requirements for mid-rise buildings)**

Design for a force level determined by one of the following three procedures:

a) Design forces determined in accordance with Clause 4.1.8.11 of the *BCBC* using $T_a$ determined using 4.8.1.11.(3).(c);

b) Design forces determined in accordance with Clause 4.1.8.11 of the *BCBC* using $T_a$ determined using methods of engineering mechanics with $T_a$ not greater than permitted by Clause 4.8.1.11.(3).(d).(iii) of the *BCBC* with the forces multiplied by 1.2;

c) Design forces determined by Linear Dynamic Analysis in accordance with Clause 4.1.8.12 of the *BCBC*, with the forces multiplied by 1.2 and $V_d$ determined using 100% V.

3.5.2 **DESIGN**

a) The design of shearwalls and diaphragms shall be to the requirements of CSA O86-09 Clause 9 – Lateral-Load-Resisting System.

b) For the purposes of Clause 4.1.8.9.(1) of the *BCBC* height limits (m), the SFRS height shall be taken as the vertical distance from the ground floor to the center of mass of the roof. For sloping ground floors, the average elevation should be taken. *Note: This definition is based on the assumption that any structure below the ground floor is a concrete box with stiff walls on all 4 sides.*

c) No type 4 and 5 seismic irregularities as defined in Clause 4.1.8.6 of the *BCBC* are allowed in the wood framed portion of the building where $I_eF_aS_a(.2)$ >=.35. Where type 4 and 5 irregularities are allowed, capacity design principals must be used to transfer shear forces down to the base. Buildings with these irregularities will likely be more susceptible to soft storeys.

d) Buildings with L, T, E and other similar plan layouts, where the wings have a length greater than the base width should be separated into rectangular building sections that avoid re-entrant diaphragm corners (see the sketches provided below).



**Figure 8 – Excerpt from APEGBC Technical Bulletin Forming Basis of BC Building Regulation – 6 Storey Wood Frame Buildings**

## VERY BRIEF SUMMARY OF ADDITIONAL TOPICS

### British Columbia Schools Upgrade Project

This is a $1.5 billion dollar program founded by the Province of British Columbia to assess and upgrade high risk schools in British Columbia.

A consultant, Graham Taylor, Ph.D., P.Eng., was responsible for the concepts and ideas for developing a new assessment tool that convinced the BC government to proceed with this program.

This consultant joined with the Association of Professional Engineers and Geologists of BC and the Civil Engineering Department of the University of BC (Professor Carlos Ventura and his students) to develop the tool. The assessment tool uses:

- A non-linear analysis approach.
- A suite of spectrum scaled records reflecting the geology and seismic history of BC.
- Several common prototype school structural systems, many of which are not recognized as appropriate systems in modern earthquake codes.
- A displacement backbone curve with a drift limit to define failure.
- A method to combine different systems.
- A different and unique approach to determine the total probability of "failure".

There are two Peer Review Groups – one external group from the U.S reviewing the "concepts" and the approaches used by the UBC group, and a BC group reviewing presentation, practicality, what designers want and need, and questioning all of the assumptions forming the basis of the document. All the Peer Reviewers are consultants and there is no question as to how helpful their comments have been in shaping the document.

For those interested, there are several presentations in the conference on this topic.

### Strategic Network Grant – "Reducing Urban Risk"

This is a very recent grant and the Network has a broad mandate. It involves several universities across the country that are involved in Earthquake Engineering Research and many researchers spread over many disciplines. It also includes several structural consultants and designers who will be contributing time, knowledge, and experience to the project.

The network is managed by Dr. Rene Tinawi and the Scientific Management Committee chair is Professor Dennis Mitchell of McGill University in Montreal.

This type of Broad Strategic Grant in the earthquake research area is quite new for Canada and there is a special section on it in the conference.

**Earthquake Loads and Design Approaches for Multilevel Below Grade Retaining Walls – A Small Task Group to Develop Design Guidelines**

This is a small volunteer task group initiated by a group of structural engineers in Vancouver, BC. The membership includes a preponderance of geotechnical consultants and geotechnical researchers.

The group is motivated by the fact that the 1/2475 “Mononobe-Okabe” lateral soil loads are over 3 times those of the 1/475 loads and are difficult to deal with, coupled with the apparent fact that there seems to be no history of damage to these types of walls.

The groups main “analysis person” is applying a suite of spectrum scaled ground motions to a “typical” 4 level wall below grade retaining wall design and performing a series of studies using a FLAC analysis. The intent is to develop a design methodology for local Vancouver designers and to submit it to the Standing Committee for Earthquake Design for consideration for inclusion in the NBCC.

## SUMMARY

The presentation is a brief personal perspective describing an overview of the Canadian Code System and how (using examples) Engineering Consultants and Practitioners have participated, informed, and helped shape (and improve!) the Earthquake Design Requirements for the NBCC and its referenced CSA Design Standards.

## Appendix

**A List (probably incomplete – my apologies to those missed) of Participants in Examples Given in Paper**

***CSA A23.3 – 1984 – Special Provisions for Seismic Design***

Vancouver Group

- Jim Mutrie, P.Eng. – Read Jones Christoffersen LTD.
- Carl Stewart, P.Eng. – Dominion Construction
- Don Nielle, PhD., P.Eng. – Jones Kwong Kishi
- G. Bevan-Pritchard, P.Eng. – Mackenzie, Snowball, Skalbahia
- Joe Harrison, P.Eng. – Tamm Tacy
- J. Eran, P.Eng., - Bush Bohlman
- Wim Jellma, P.Eng. – Sayers Engineering
- Ron DeVall, PhD., P.Eng. - Read Jones Christoffersen LTD.
- Nigel Brown, P.Eng. - Read Jones Christoffersen LTD.
- Professor Richard Spencer – University of BC
- Professor Noel Nathan – University of BC
- Professor Shel Cherry – University of BC

***Flexible Steel Diaphragms***

- Gerry Weiler, P.Eng. – Weiler, Smith, Bowers
- Kevin Lemieux, P.Eng. – Weiler, Smith, Bowers
- John Wallace, P.Eng. – Pomeroy
- Bob Neville, P.Eng. - Read Jones Christoffersen LTD.
- Rob Simpson, P.Eng. – Glotman, Simpson
- Professor Robert Tremblay – Ecole Polytechnique
- Professor Colin Rogers – McGill University
- Professor Carlos Ventura – University of BC

***6 Storey Wood Frame in BC***

- Jim Murtrie, P.Eng. – Jones Kwong Kishi
- Thomas Leung, P.Eng. – Thomas Leung Inc.
- Robert Malczyk, P.Eng. – Equilibrium Consulting
- Grant Newfield, P.Eng. - Read Jones Christoffersen LTD.
- Professor Emeritus Don Anderson – University of BC

***BC Schools Project***

- Graham Taylor, PhD., P.Eng. – T.G.B.
- Professor Carlos Ventura and students – University of BC

External Peer Reviewers (for UBC)

- Farzad Haeim, PhD., P.E., S.E – John Martin and Associates
- Michael Mehrain, PhD., P.E., S.E. – URS Corporation

Vancouver Peer Review Group (local + one US member)

- Andy Mill, P.Eng. – David Nairne and Associates
- Bob Hanson, PhD. – Consultant (ex University of Michigan)
- John Wallace, P.Eng. – Pomeroy
- Clint Low, P.Eng. – Bush Bohlman
- John Sherstobitoff, P.Eng. – Sandwell
- Ron DeVall, PhD., P.Eng. - Read Jones Christoffersen LTD.
- Tim White, PhD., P.Eng. – Bush Bohlman