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SEISMIC DESIGN OF HIGH-RISE CONCRETE BUILDINGS: A DESIGNER'S PERSPECTIVE

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ABSTRACT

The paper describes the evolution of the design of high-rise buildings in Canada over forty years. Three buildings are used to illustrate the developing requirements. Examples of how changes to codes cause changes in practice are given. The new requirements of the 2004 edition of CSA A23.3 are discussed. The disconnect between code writers and designers as a limitation to improvements in building safety is examined.

Introduction

This paper is about the seismic design of high-rise concrete buildings and the evolution of the design approach over the last forty years. It is based on the author's 40 years of experience as a designer on the west coast of Canada and the U.S. Pacific North West, his 27 years of experience as a member of the Canadian Standards Association Technical Committee A23.3 and 18 years of experience as Chairman of the Clause 21 Sub-committee on special provisions for seismic design. This paper will attempt to describe where the design industry was at the start, where it got to in the 1980's, where it is today with the current code and where it needs to go in the future. This paper will also attempt to illustrate the author's belief that code writers need to try harder to frame their codes in a way that promotes an understanding of the reasons behind the requirements rather than just providing designers with a set of rules to follow.

Some Early Examples: 40 Years Ago

The first building where the author was responsible for the detailed design of the lateral force resisting system was a 25 story reinforced concrete office building called the Board of Trade Tower (Fig. 1). The lateral force resisting system was the central service core. There was an exterior frame but it was assumed to only carry gravity loading.

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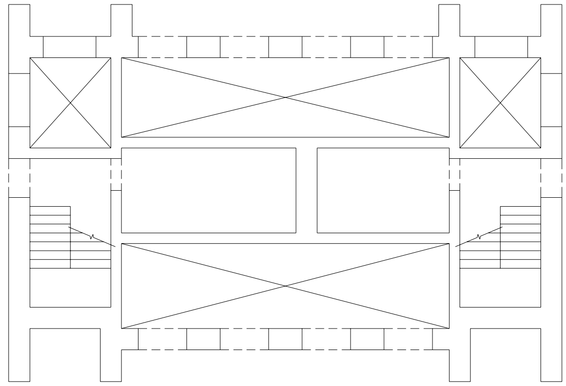
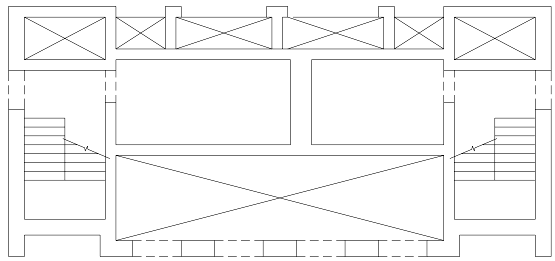


BASEMENT TO LEVEL 14



LEVEL 14 TO ROOF

Figure 1. Board of Trade Tower in Vancouver (left) and core plans of Board of Trade Tower (right). (Mike Sherman photo - RJC)

The analysis was very simple; a single element cantilever. Earthquake was treated by the Vancouver Building By-Law as just another load case along with wind load. There were no categories of lateral force resisting systems in the code, just one earthquake force for all systems. There was an allowable increase in design stresses of one third (133%) for both wind and earthquake loading but there were no special provisions for earthquakes contained in the material design provisions. The total code earthquake requirements took less than two full pages.

The design forces were determined using the following formula:

$$F = 4CW$$

$$\text{where} \quad C = \frac{0.15}{N + 4.5}$$

$$\text{and} \quad N = \text{Number of Stories above level under consideration}$$

Using this formula the base shear for a 25 story building is 0.043W. If this is increased by a load factor of 1.5 to make it equivalent to today's limits states design loads, the base shear would be 0.064W. This falls between a base shear of 0.073W for an R_d = 2.0 and 0.036W for an R_d = 3.5 building according to the 2005 NBCC. Thus, the building was designed for a base shear equivalent of an R_d = 2.35 building. As the building was designed without any special seismic detailing, it should be considered an R_d of 1.5 building and designed for a base shear of 0.105W. In that case, it was designed for only about 41% of the current NBCC 2005 base shear.

There was nothing in the code of the day to indicate that the design was for much less than the expected earthquake force and that energy dissipation in the lateral force resisting system was required during the design earthquake. This was even though researchers and code writers of the time knew this was the case and for whatever reason they chose to not convey this information to the designers.

The author's second major building was the Granville Square office building (Fig. 2). This building was to be the cornerstone of a major development on the shores of Vancouver Harbour over the Canadian Pacific Railway yards.

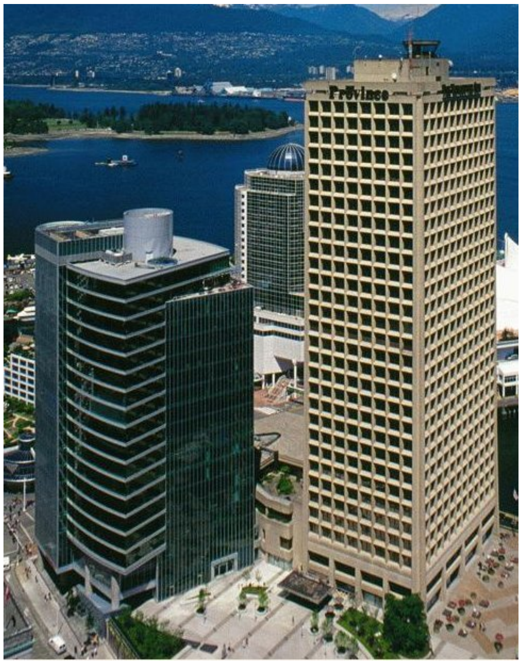


Figure 2. Granville Square office building in Vancouver on the right. (Mike Sherman photo - RJC)

The developer was Marathon Realty, an arm of Canadian Pacific Railway and their approach was very unusual. They were building in an earthquake prone area so they hired one of the best known earthquake engineers of the time, Dr. Nathan Newmark, as a special consultant to the project. Dr. Newmark was Head of the Department of Civil Engineering at the University of Illinois at the time and had recently coauthored a book published in 1961 by the Portland Cement Association entitled *Design of Multistory Reinforced Concrete Buildings for Earthquake Motions*. This book was far ahead of any Canadian code of the time in describing earthquake effects and the structural ductility required to prevent building failure. After some discussion it was decided to use the 1965 edition of NBCC as it had introduced a new factor 'C' to recognize the benefit of a good system and good detailing. The equations were as follows:

$$V = K \times W$$

$$K = R \times C \times I \times F \times S$$

$$S = \frac{0.25}{9 + N} \quad \textit{Where } N = \textit{Total Number of Stories}$$

$$C = 0.75 \quad \textit{For Moment Frames Carrying 50\% of Design Shear and / or 'Special' Shear Walls}$$

Special shear walls had to be (to quote the code), "adequately reinforced to carry design shear forces in a ductile fashion" or the 'C' value had to be increased to 1.25. The base shear was 0.0368W for the non-ductile system and 0.0221W for the ductile system. These were less than the Vancouver Code base shear of 0.043W. Specifically, the 1965 NBCC base shear was about 85% of the Vancouver Code base shear for the non-ductile system and 50% of the Vancouver Code base shear for the ductile system. According to the 2005 NBCC for a ductile system with R_d = 4.0 and R_o = 1.7, the base shear is 0.030W, whereas according to the 1965 NBCC, the base shear was 1.5 x 0.0221W = 0.0332W. Note that these two values are within 10%. On the other hand, the difference is again very large for the non-ductile case, where the base shear according to the 2005 NBCC is 0.105W, while the base shear according to the 1965 NBCC is 1.5 x 0.0368W = 0.055W.

The analysis was more complicated for this building as there were both the shear walls and the frames. Therefore, the analysis had to be done by computer. The building was treated as two separate 2-D systems, one for each direction. Each direction consisted of a model of the core linked to a model of the exterior frame. This model complexity was about at the limit of what could easily be solved on the main frame computers of the day, which had the equivalent of 256K bytes (or .256M) of RAM.

The frames and shear wall were detailed in accordance with the suggestions contained in Newmark (1961). The frame columns and beams were reinforced in a similar fashion to the requirements in the current CSA A23.3 for Ductile Moment Frames, although not as stringently. The shear walls also followed the detailing suggestions in the book. The reinforcing steel contractor had never seen this type of detailing in a building with all the column ties and went bankrupt on the project.

The Past 20 Years

The bulk of the high-rise buildings in Vancouver were built in the years after Expo 86 through to today, and were designed to the 1985 and 1990 editions of the NBCC and the 1984 and 1994 editions of CSA A23.3. The first edition of CSA A23.3 with special provisions for seismic design was the 1973 edition. These provisions were updated slightly in the 1977 edition and then dramatically changed, particularly for walls and coupled walls, in the 1984 edition.

The accidental torsion requirement introduced in NBCC 1970 was doubled from $0.05D_n$ to $0.01D_n$ in NBCC 1985 and this, coupled with the improvements in computers, and the availability of 3D analysis programs capable of handling shear wall buildings, resulted in dramatic changes to the standard practice for high-rise buildings. Office buildings already had relatively large central cores, for functional purposes, which were easily able to resist the increased torsion as well as lateral forces. However, residential buildings typically had distributed individual shear walls, stair and elevator cores that shared the lateral loadings (Fig. 3). These arrangements were good for carrying lateral forces and were easy to analyze without computers but were not very stiff or efficient in carrying torsional forces.

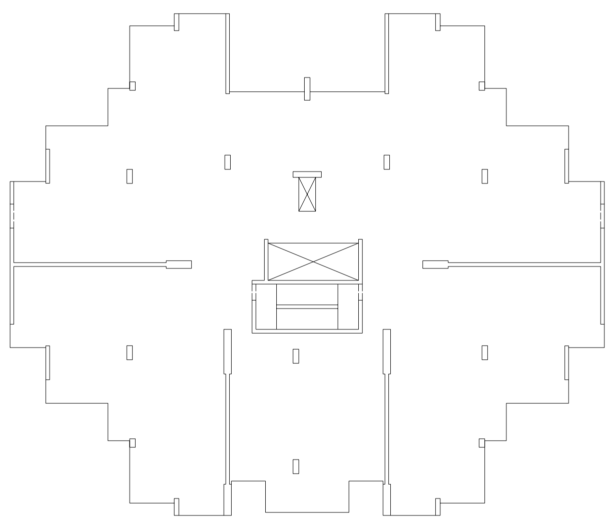
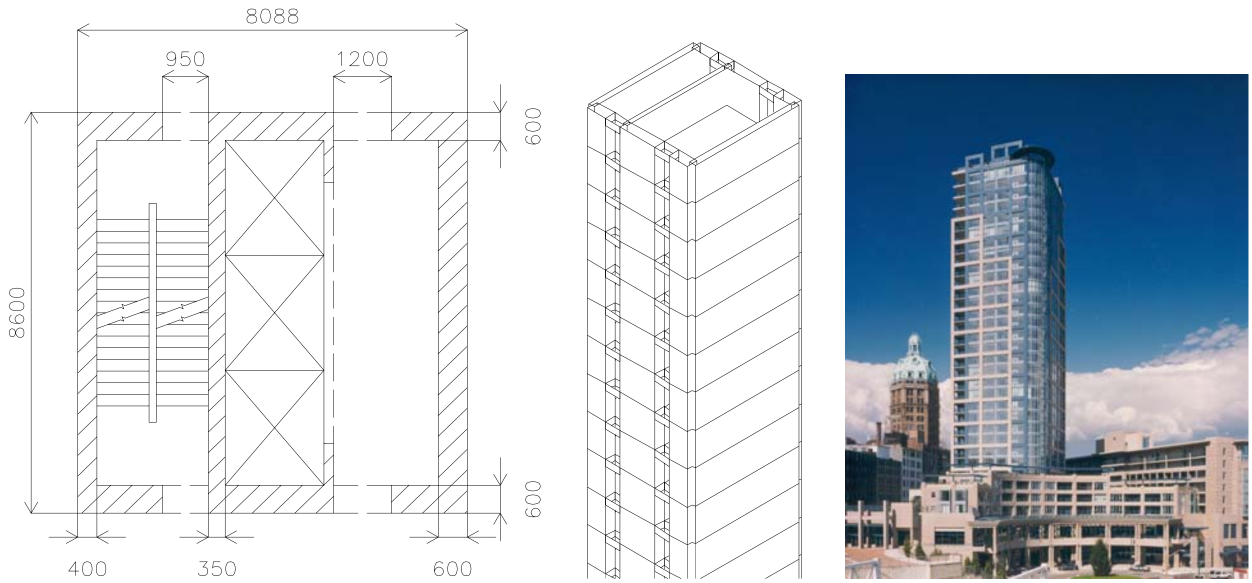


Figure 3. Typical floor plan of a residential building core with distributed individual shear walls and stair and elevator cores to share the lateral loadings.

When 3D analysis programs were first used, it was quickly discovered that, in most cases, the first mode of vibration for these buildings was torsional. Residential high-rise practice quickly changed to central cores with diagonally reinforced coupling beams connecting the wall segments together (Fig. 4a) to create stiff “torque boxes” in which the first two modes of vibration were lateral and the third mode was torsional (Fig. 4a and b). These buildings had much better structural properties than the old systems, architects liked the layout flexibility they provided and contractors quickly learned how to construct them economically.



(a) (b) (c)

Figure 4. (a) Residential floor plan having central cores with diagonally reinforced coupling beams connecting wall segments; (b) three-dimensional view of residential high-rise “torque boxes”; (c) example residential high-rise with “podium”.

The other factor that encouraged the use of “torque box” cores was the new requirements of CSA A23.3-M84. The significant requirement was the limitation on compression zone depth introduced as a way of ensuring wall ductility. This lead to the use of walls with compression flanges, often flanges that were capable of taking the total specified gravity load on the wall. Some designers used to say they proportioned their cores so that they could “stand the core on the flanges”.

The complexity of urban buildings started to increase around this time as podiums were added to buildings. The podiums were added as a result of city planners requiring developments to include features that would enhance the street level liveability of neighbourhoods such as retail, townhouses and other functions. The ability of the average consultant to purchase powerful programs, such as ETABS, capable of performing modal dynamic analysis of these complex buildings, exposed a series of problems with the NBCC seismic design requirements. The rules the code created for the use of dynamic analysis, scaling the results to the force level determined by the static method, worked very well for relatively uniform buildings, but could give very unconservative results for the very types of buildings suggested for dynamic analysis by the code.

The other problem was the application of code requirements for torsion that required amplification of torsional eccentricity, that is easy to do for uniform buildings where the shear center can be readily determined, but for irregular buildings where the shear center is a function of the loading pattern, no uniform approach was ever developed. It was not until the publication of NBCC 2005 that these problems were addressed and so in the interim period, there was a wide disparity in the approach adopted by engineers in BC, where there is no enforcement of structural design standards by any authority having jurisdiction.

There is also far too much blind faith within the engineering community in the results produced by linear analysis programs and a mistaken belief that dynamic modal analysis gives superior seismic design. Dynamic analysis certainly does give a better idea of the behaviour of buildings in the elastic range of deformation than any static method, but once yielding commences, only non-linear methods can provide any insight into the actual building behaviour.

Recent US Experience

The author was involved in the design of a building in Bellevue Washington for a Canadian client and architect. They both wanted to build a 400 ft. high Vancouver style building without a moment frame around the exterior in Bellevue where the Uniform Building Code requires at least a 25% frame for buildings over 240 ft. high. The US codes have long been written around the use of frames for seismic resistance starting after the 1906 San Francisco earthquake, where it was observed that buildings with complete frames had performed the best. The wall buildings that did not perform well were typically made of masonry.

The UBC code contained a list of acceptable systems with limitations such as maximum height but the list also included an “Undefined System” category. The code provided a list of requirements to be met for the “Undefined” category including non-linear dynamic analysis for a suite of earthquakes and independent Peer Review. The City of Bellevue was approached to see if they would consider looking at the possibility of using a coupled wall system as an “Undefined System” and, after some thought, they agreed to consider the possibility. This project was not the first project in the Greater Seattle area to attempt the “Undefined System” approach but it was proposed to be much higher than any of the others. They selected a Peer Reviewer, a well known engineering group from San Francisco. A team was assembled for the project that included Dr. Perry Adebar from UBC.

The typical central core systems used in Vancouver are a coupled wall system in one direction and an uncoupled (cantilever) wall system in the other. For the Bellevue project a system with coupled walls in both directions was developed, even though openings in the walls were not required in one of the directions (Fig. 5). The reason for this is that coupled walls are similar to frames in that they have numerous plastic hinges distributed throughout the structure.

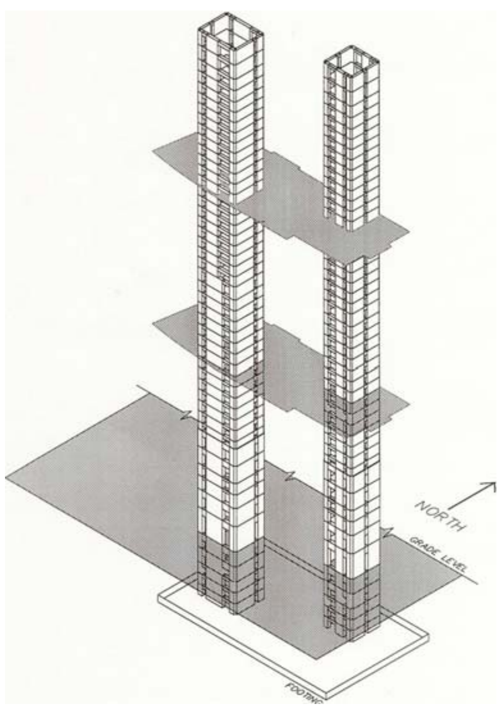


Figure 5. Central core system used in the Bellevue project in Vancouver.

The systems were proportioned for a force level equal to that required for a shear wall plus a 25% frame system. A push-over analysis of the chosen systems adopting the displacement based approach was then started to judge the potential system performance. The results of the push-over analysis are shown in Fig. 6.

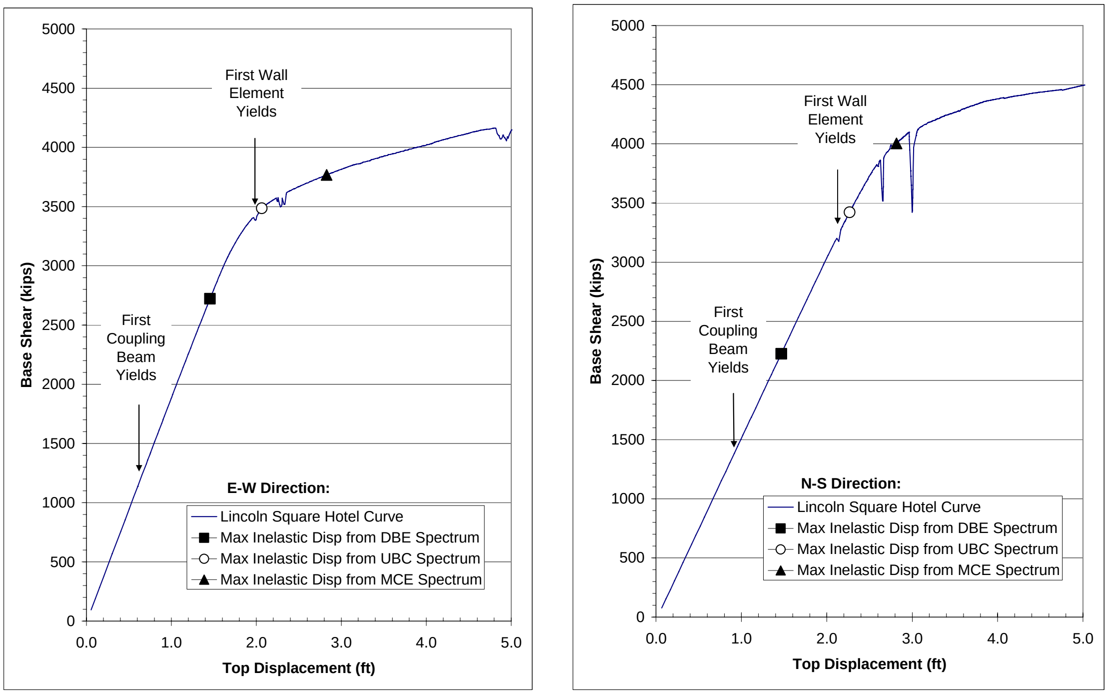


Figure 6. Results of the push-over analysis.

When a comfort level from the push over results was developed, a non-linear time history analysis of the systems was started, using site specific time history records developed under the direction of a well known seismologist. It was soon found, particularly for the hotel/residential building, that the rotational demands on the coupling beams were quite large. Conservative rotations limits using limits suggested in the FEMA documents were previously developed, as acceptance criteria, and rotations in excess of the criteria were reached. Considerable time was spent “tuning” the coupling beams, both the length and the depth, at locations over the height of the building, to keep all rotations within the criteria. The records used were spectrum matched to a site specific spectrum, but the results were not consistent from record to record as can be seen from the charts that follow (Figs. 7, 8, 9 and 10). As would be expected, the records that produced the largest interstory drifts also produced the largest chord rotations. Dynamic shear magnification in the walls was also found, which was not unexpected, as it was mentioned in CSA A23.3-M84 and the explanatory notes to CSA A23.3-94.

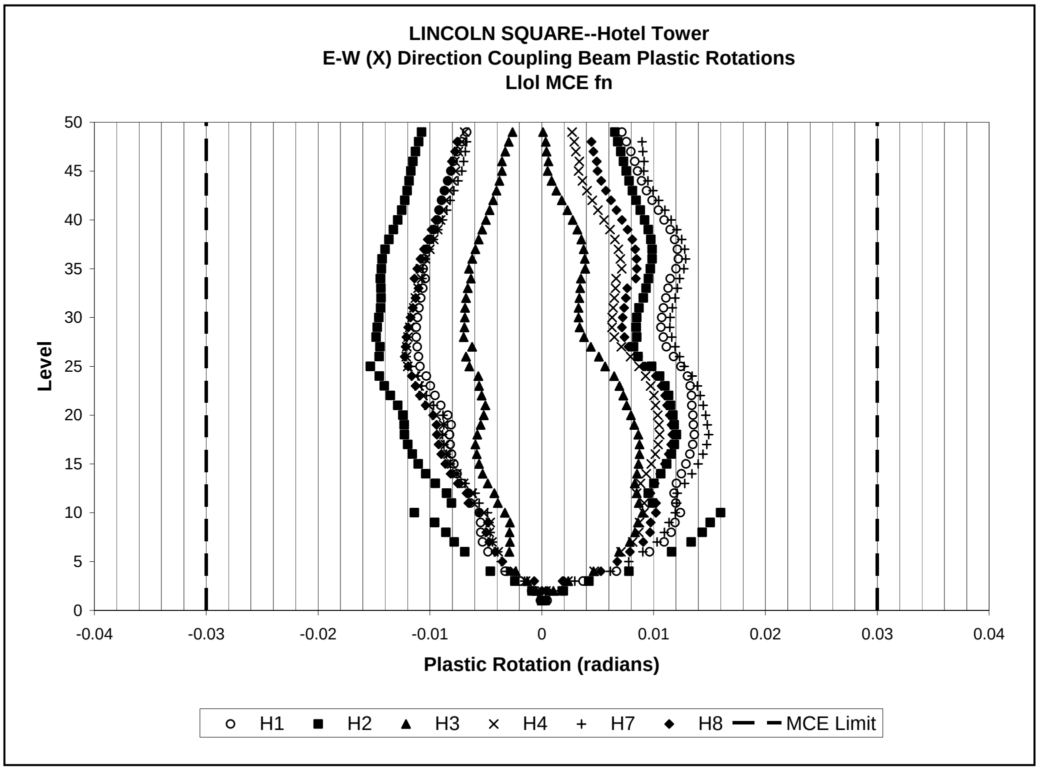


Figure 7. Rotation analysis results for Llol MCE fn.

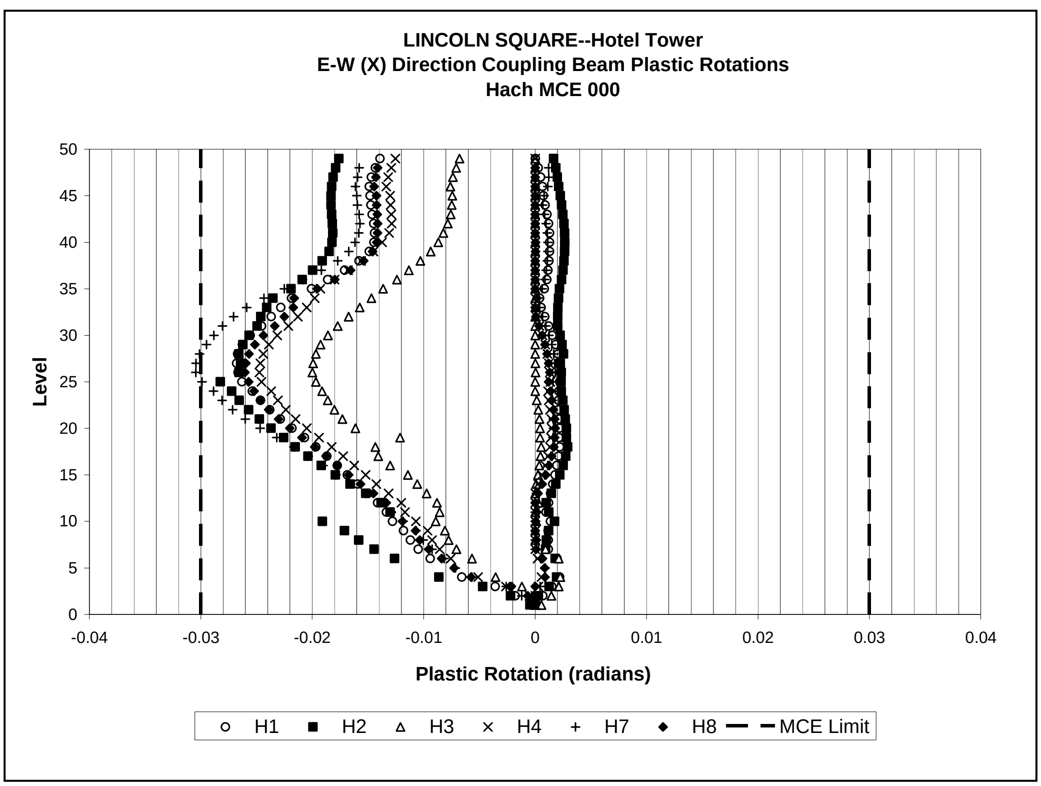


Figure 8. Rotation analysis results for Hach MC E 000.

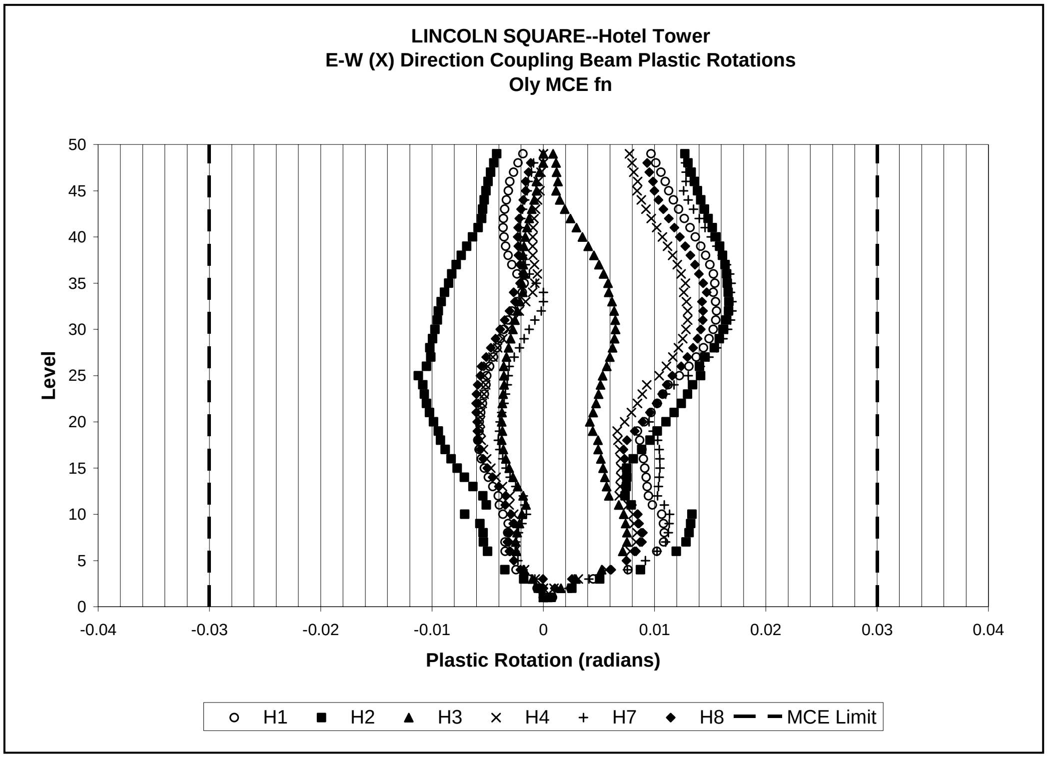


Figure 9. Rotation analysis results for Oly MCE fn.

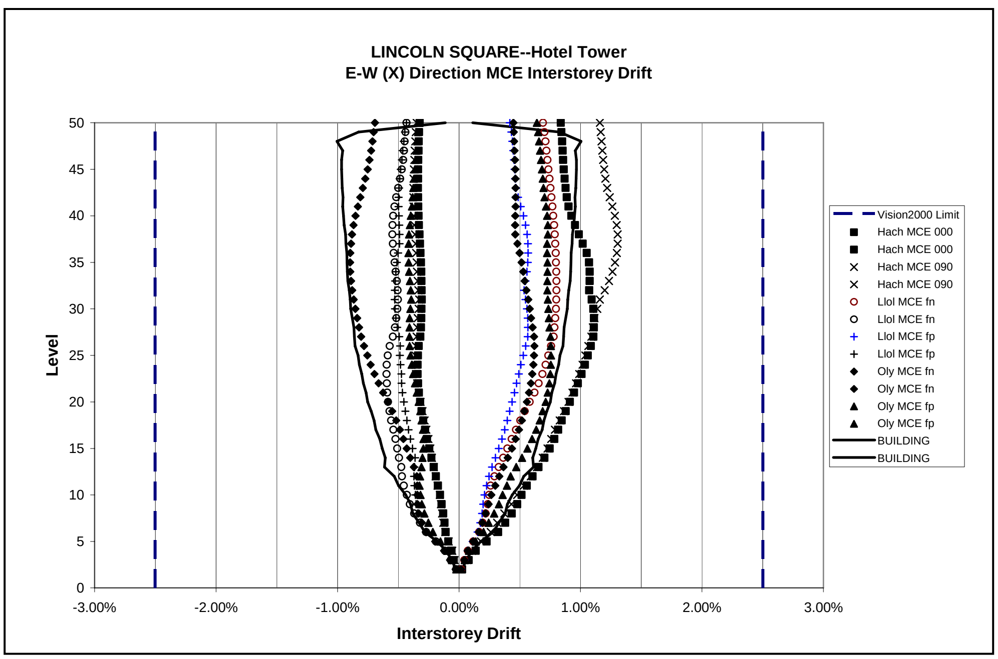


Figure 10. Direction of Interstorey Drift.

Since the completion of the Bellevue project, there have been many more buildings in the US that have used core walls and have been designed using a similar approach. The Los Angles Tall Building Structural Design Council published a document formalizing the procedure in 2005. Both the peer reviewer and the seismologist involved in the Bellevue project contributed to the document.

2004 Canadian Concrete Code Provisions

CSA A23.3 Clause 21 was extensively rewritten in the 2004 edition. The base document was expanded to include clauses on conventional construction, precast concrete, diaphragms and foundations. There are significant changes throughout the standard but the most important ones are in General covering systems and effective stiffness's, in Ductile Walls covering rotational demand and rotational capacity of shear walls and coupling beams, in Moderately Ductile systems covering approaches to tilt-up walls and the new class of squat shear walls and in the section on non-SFRS members covering slab-column connections.

The general section of Clause 21 has been rewritten to make it clear that the clauses are written to cover the "systems" (SFRS) recognized in NBCC and then only if the systems are regular. These thoughts are expanded in the explanatory notes published by the Cement Association of Canada. Designers in the past were known to treat Clause 21 as a "kit of parts" to be assembled in any way necessary to easily suit the functional/architectural layout desired for the building. This approach can easily result in situations where the plastic hinge rotational demands are far in excess of those provided by the detailing rules written for regular systems. If constraints of the project are such that the standard SFRS's cannot be used, then any system employed must be subject to a non-linear analysis and treated as an equivalent under NBCC. This can be a very onerous and expensive procedure and should only be undertaken after all other approaches are exhausted.

The approach to walls was changed from the ductility approach introduced in 1984 to a displacement based approach. The authors attempted to make it clear what they were trying to accomplish by introducing a seismic "limits states" approach.

$$\theta_{ic} > \theta_{id}$$

Where ic = inelastic rotation capacity
id = inelastic rotation demand

The approach of many codes could have been adopted, by combining all the relationships into one equation, but that obscures the rational for the requirement. It is hoped that the code writers try to keep in mind that a designer who understands what the code is trying to accomplish is much more likely to deliver a design that meets both the intent as well as the letter of the code. Such designs will undoubtedly perform better in an earthquake.

The approach taken allows the calculation of demand and the capacity required. The building can be modified to change either the demand (by making the building less flexible) or the capacity (by changing the compression strain depth) or both to find an optimum solution. The 1984/94 CSA had a one size fits all approach.

The following diagrams (Figs. 11 and 12) illustrate the derivation of the code rotational demand and rotational capacity formula:

$$\theta_{id} = \frac{\left(\Delta_f R_o R_d - \Delta_f \gamma_w\right)}{\left(h_w - \frac{\ell_w}{2}\right)} \geq 0.004$$

$$Design\,Displacement = \Delta_f R_o R_d$$

$$Elastic\,Displacement = \Delta_f \gamma_w$$

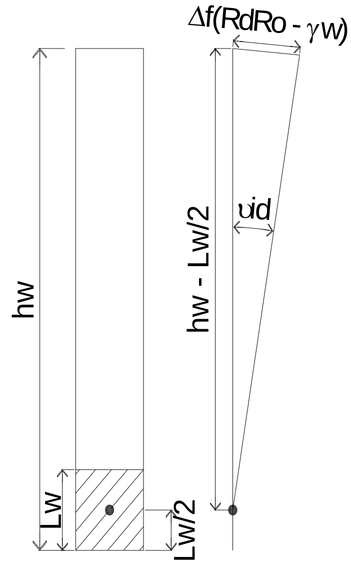


Figure 11. Rotational demand formula.

$$\theta_{ic} = \left(\frac{\varepsilon_{cu}\ell_w}{2c} - 0.002\right) \leq 0.025$$

$$Plastic\,Hinge\,Length = \frac{\ell_w}{2}$$

$$\theta_{ic} = \frac{\ell_w}{2}\left(\frac{\varepsilon_{cu}}{c} - \frac{\varepsilon_{sy} + \varepsilon_{cy}}{\ell_w}\right)$$

$$\theta_{ic\,\max} = \frac{\ell_w}{2}\left(\frac{\varepsilon_{s\max}}{\ell_w}\right)$$

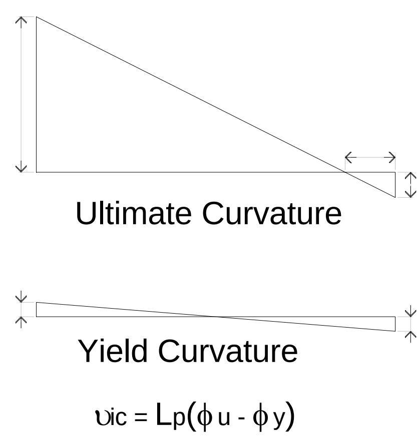


Figure 12. Rotational capacity formula.

The rotational demand relationships for coupled walls was derived from work done by White and Adebar (2004) and is an approximation of the more complex deformation of coupled walls (Fig. 13).

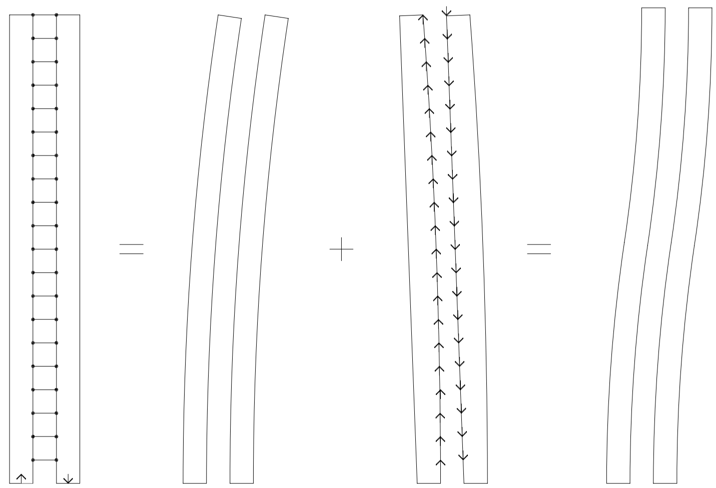


Figure 13. Rotational demand relationship for coupled walls.

The rotational demand at the base of a coupled wall is not related to the top displacement in the same way as a solid wall and is in fact more due to the fact that the coupling beams tend to pull the top of the walls back. A reasonable approximation of this effect was found to be determining the rotational demand by taking the top deflection as the design displacement and not subtracting the elastic portion of the displacement as is done for un-coupled walls. The relation derived was

$$\theta_{id} = \frac{\Delta_f R_d R_o}{h_w} - \frac{\Delta_f \gamma_w^2}{R_d R_o}$$

but since the last term is small the code equation reduces to:

$$\theta_{id} = \frac{\Delta_f R_d R_o}{h_w}$$

The relation for coupling beam rotation is derived from the wall rotation and the floor rotation (Fig. 14). For the code equation it was found that a relation based on the top design displacement produced good correlation with the results of non-linear analysis

$$\theta_{id} = \left(\frac{\Delta_f R_0 R_d}{h_w}\right)\frac{\ell_{cg}}{\ell_u}$$

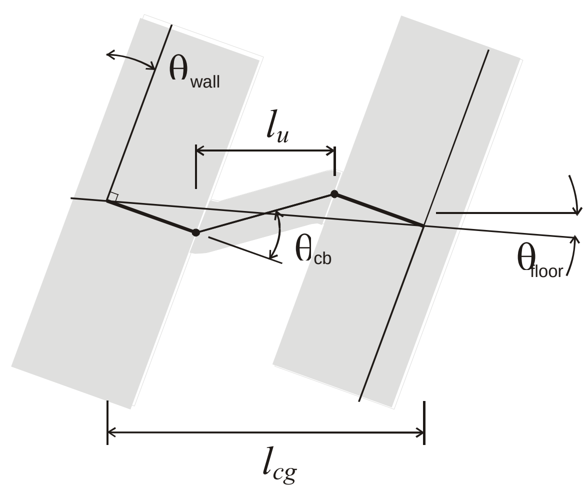
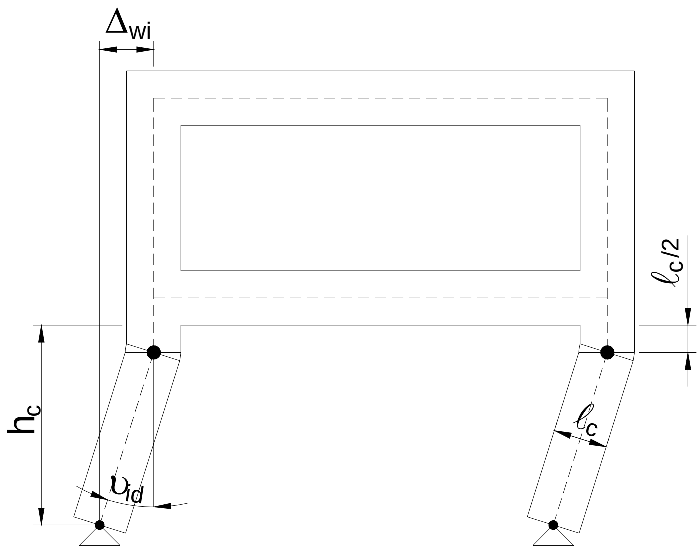


Figure 14. Relation for coupling beam rotation from wall and floor rotation.

Another example of a case where it is possible to simply calculate the rotational demand is the framed tilt-up buildings, which are seen more commonly on the West Coast. Fig. 15 is a diagram taken from the CAC explanatory notes.



$$\theta_{id} = \frac{\Delta_{wi}}{h_c - \frac{\ell_c}{2}}$$

Figure 15. Rotational demand for frame tilt-up buildings.

Future Directions

The 2004 CSA A23.3 introduced displacement based design, but the surface has only just been scratched. There is a need for greater understanding of the non-linear displacement of the building as a whole and the local rotations at the hinges. Most real buildings are not at all like the simple uniform structures used to develop the concepts contained in codes. What is needed are relatively simple methods that allow designers to estimate the non-linear behaviour of complex structures as well as the simple ones currently covered. For frames and even relatively simple coupled wall systems with dimensional variations over the building height, the determination of the rotational demands is not simple. Currently, the only method available is the use of non-linear computer programs to either do a push-over analysis or a complete time history analysis, which may not be a realistic approach for routine design.

Currently, for structures with recognized systems (mainly frames), where it is not possible to easily determine the rotational demands, designers take the “cook book” approach, where it is taken on faith that no matter what the building configuration is, the detailing rules provided in the code will give a satisfactory result. This one size fits all approach does not give designers any understanding of whether they are providing the required rotational capacity, more than they need or perhaps, in more cases than they care to admit, less than they need. Without knowing rotational demands, it is not possible to take a rational “limits states” approach to seismic design of all buildings and it should be possible to do just that.

The R_d values in the code, and not just for concrete but for all materials, have more to do with the intuition of code writers than they do with rational analysis. More work needs to be done to either confirm the R_d values or make changes where necessary. Powerful non-linear programs are now available for this task. Designers need to have good relations that tell them the rotational capacity of member hinges as a function of axial compression and detailing. These capacities should then be compared with the demands seen from non-linear analyses. This type of study is needed to give confidence to the R_d values.

There is also a long way to go in the consideration of what is needed to protect the gravity load carrying elements from the deleterious effects of non-linear building drift. A section in the CAC Explanatory notes was added after the code was complete to try and cover the case of the deformation demand on gravity load carrying columns over the height of a shear wall plastic hinge. This is a subject that requires a lot more study and there will hopefully be a lot more in the next code edition.

For shear walls, there is an issue that is well known but has yet to be properly resolved and that is what is often termed “dynamic shear magnification”. This is just a fancy term to cover the fact that a moment hinge in a shear wall is not a shear hinge and therefore does not limit shears in the wall above the hinge location. It has been mentioned in the 1984, 1994 and now the 2004 editions of A23.3 but there is still no agreed method of dealing with the problem. What this refers to is the load effects and one would normally think this is an NBCC Part 4 issue and therefore designers should be looking to CANCEE for a resolution. CANCEE takes the position that it is a concrete issue since concrete shear walls are the only moment yielding SFRS. All other SFRS’s have only shear yielding systems. The New Zealand code has a magnifier in their code and a modified form of this relation was suggested in the Explanatory Notes to the 1994 edition of CSA A23.3, but there is nothing in the current Explanatory Notes to CSA A23.3. It is urgent that this issue be addressed as soon as possible, for without some sort of shear magnifier the designed shear walls are potentially unsafe.

One very important message that must be conveyed to designers, and probably to architects and owners as well, is that this is definitely not simple stuff. As long as buildings continue to be designed for far less than the expected seismic forces, seismic design will not be simple. The skills needed to design a building so that it does not fail under the influence of gravity and wind loads are not sufficient to design a building so that it fails “nicely” in an earthquake. Seismic design requires an additional knowledge and skill set above and beyond that required for standard design. The behaviour of all but the simplest of buildings in the non-linear range is very complex indeed, involving the interaction of many components; beams, columns, diaphragms, braces, walls and foundations. This need for greater understanding is highlighted by the extreme reaction of designers in seismic zones to the introduction of code clauses like NBCC 2005 Clause 4.8.15.1. This is a very simple clause that says, in essence, one must design diaphragms to be stronger than the SFRS so that the SFRS is the energy absorber, not the diaphragm. This reaction indicates that, to date, educators and code writers have been doing a very poor job of telling designers how to think about the behaviour of their buildings in an earthquake. Designers have come a long way since 1966 when they were kept completely in the dark, but there is still a long way to go.

Acknowledgements

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