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Seismic design of R/C frames – A Canadian code perspective

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ABSTRACT: Three ductile and three nominally ductile frames were designed according to the current Canadian practice. For each type of frame design, one frame was four storeys high while the others were ten and eighteen storeys high. The frames were analyzed under a monotonically increasing static loading to determine the effect of the design methodology on their overstrengths. Then, the frames are analyzed dynamically under earthquake excitation using a modified version of DRAIN-2D. The response parameters investigated are the total and interstorey drift, the beam rotational ductility demand and the demand shear forces in the beams. The results of this investigation show some deficiencies in the seismic performance of nominally ductile frames. In particular, the demand shears attained in the beams during the dynamic response are much larger than the beam shear capacity. Corrective measures to the current concrete design provisions of nominally ductile frames are proposed.

1 INTRODUCTION

According to the current Canadian practice the designer has two options for the design of reinforced concrete frames in seismic regions, a ductile frame or a nominally ductile frame. For the design of a ductile frame, the National Building Code of Canada (NBCC 1990) specifies a relatively low level of the seismic lateral strength. The survival of such frames when exposed to severe ground shaking depends on their inherent ductility. Therefore, the design and detailing must conform to the requirements given by the Canadian concrete design code (CAN3-A23.3-M84) for ductile frames to ensure a ductile behaviour of the frame. To prevent the occurrence of a storey side-sway mechanism, the columns are designed to be stronger than the beams. To eliminate the possibility of shear failure, the design shear forces are calculated by assuming that the plastic hinges at the beam ends develop their probable strengths in opposite directions to produce maximum shear in the members in addition to the shear forces due to the factored gravity loading.

For the design of a nominally ductile frame, NBCC 1990 specifies twice the value of the seismic lateral loading as that of a ductile frame. No special provisions are specified for the design of the frame members and the detailing requirements are less stringent than those of ductile frames. Thus, in the beams of nominally ductile frames, the design shear forces are obtained directly from the combinations of factored gravity and seismic loading without any modifications. By giving the designer the choice between ductile and nominally ductile frames, the code implies that either type of frame design will provide acceptable seismic performance under the design level earthquake. Paultre and Mitchell (1991) designed a six storey building once as a ductile and once as a nominally ductile frame. The two frames were analyzed under different ground motions. They concluded that the current Canadian design and detailing requirements for nominally ductile frames are adequate. This conclusion may not be generalized to all nominally ductile frames since they did not consider frames of any other heights. The objective of this study is to determine the equivalence or the difference of the seismic performance of ductile and nominally ductile reinforced concrete frames designed exactly according to the Canadian codes, both for low-rise and high-rise frames.

To achieve this objective, three buildings were designed once as ductile and once as nominally ductile frames according to NBCC 1990 and CAN3-A23.3-M84. One of the buildings was four storeys, the second was ten storeys while the third was eighteen storeys. The frames were first analyzed under a monotonically increasing static loading to study their overstrength. Then the frames were analyzed dynamically under earthquake excitation using the computer program DRAIN-2D. A new element was added to the program to model the beams of the nominally ductile frame. The new element exhibits stiffness and strength deterioration and pinching in the hysteresis loops. The response parameters investigated in this study are the total and interstorey drifts, the beam rotational ductility demands and the beam demand shears.

2 STRUCTURAL CONFIGURATION

Each of the buildings considered consists of three bays in the E-W direction and seven bays in the N-S

direction. The bay widths are 8.0 metres in both directions. The first building is four storeys high, the second building is ten storeys high and the third is eighteen storeys high. The storey height is constant at 3.5 metres for all buildings. The concrete compressive strength is 30 MPa and the steel yield strength is 400 MPa. The seismic loading is considered to be acting only in the E-W direction. The typical interior E-W frame of each building was designed once as a ductile frame and once as a nominally ductile frame.

3 DESIGN LOADING

The frames are designed for the critical combinations of gravity and seismic loading as given by the National Building Code of Canada, NBCC 1990. The dead load includes the self weight of the structural components, the partitions and the mechanical services loading. The live load is taken as that of an ordinary office building.

3.1 *Seismic loading*

The seismic base shear is given by the following equations according to NBCC 1990;

$$V = (V_e/R)U \quad (1)$$

and

$$V_e = vSIFW \quad (2)$$

where v is the zonal velocity ratio, S is the seismic response factor, I is the importance factor, F is the foundation factor, W is the total weight of the building, R is the force modification factor, U is equal to 0.6.

For this study, the buildings are assumed to be located in Quebec City for which the zonal velocity ratio is equal to 0.15 and $Z_a > Z_v$. The importance and foundation factors are both taken as unity. The seismic response factor is calculated based upon the fundamental natural period of the building. The period T is taken as 0.1 N where N is the number of storeys. The value of the force modification factor, R, is chosen according to the ductility level for which the frames are designed. For ductile frames R = 4.0 and for nominally ductile frame R = 2.0. The seismic lateral loading is distributed to the frames according to their relative stiffness.

4 DESIGN OF THE FRAME MEMBERS

The bending moments, axial forces and shearing forces required for the design are obtained using the SUPERETABS computer program (Wilson et al. 1975). For comparison purposes, the member dimensions are taken to be the same for ductile and nominally ductile frames for each building height. Full details of the member dimensions and the reinforcement ratios of the frames are given by Hamdy (1992).

4.1 *Ductile frames*

The main aim of designing ductile frames is to avoid brittle failure and storey side-sway mechanisms. The seismic design provisions of chapter 21 of CAN3-A23.3-M84 (1984) are to be followed. The main features of the design are i) strong column-weak beam, ii) design shear forces based on the probable strength of plastic hinges and iii) good detailing.

4.2 *Nominally ductile frames*

All the design actions including beam design shears are directly obtained from the results of the elastic static analysis. The code allows the use of the concrete shear resistance in the design of beams. Minimum stirrups (No. 10 @ d/4) are specified in the plastic hinge zone to provide confinement for concrete and steel. In addition to the concrete shear resistance allowed by the code, the minimum stirrups will be sufficient to resist the factored design shears. Therefore, the minimum stirrups will be provided in the beams of all three nominally ductile frames.

5 LATERAL STRENGTH OF THE FRAMES

To evaluate the lateral strength of the frames, they were analyzed under a monotonically increasing static loading. A modified version of DRAIN-2D was used in the analysis (Zhu, 1989). The static loads were distributed over the height according to NBCC 1990. The first plastic hinges appeared in the beams in ductile frames, while in nominally ductile frames the hinges appeared first in the columns. The ultimate strength of the frames was taken as the lateral strength at a displacement equal to R/0.6 times the displacement at code level loading. Table (1) gives the values of the overstrength factors (Ultimate strength/Code base shear) of the frames considered here. The overstrength factor decreases with the increase of the frame height. This can be attributed to the fact that the strength of the members of low-rise frames is usually governed by gravity loading and minimum code requirements rather than by the seismic lateral loading demand.

The U factor used in NBCC 1990 can be interpreted as some measure of overstrength of buildings when they are designed according to NBCC 1990 and the relevant material codes. It has been suggested (Tso, 1991) that 1/U can be considered as the overstrength factor. Thus, the code assumes an overstrength factor of 1.67 for all types of structures. The overstrength factors of ductile frames are significantly higher than that assumed in NBCC 1990, while the corresponding factors for nominally ductile frames are of the same order as the value assumed in the Code.

Table (1) Overstrength factors of the frames.

Number of storeys	Four	Ten	Eighteen
Ductile	4.47	3.00	2.40
Nominally Ductile	1.82	1.74	1.50

6 DYNAMIC ANALYSIS PROCEDURE

6.1 *Modelling*

The DRAIN-2D computer program (Kanaan and Powell 1973) was used in the dynamic analysis of the frames. The members of the ductile frames are modelled by the dual-component bilinear model developed by Clough, Benuska and Wilson (1965). The use of stable loops may be justified by the good detailing expected in members of ductile frames. With relatively poor detailing, the beams of nominally ductile frames are expected to show some degree of strength deterioration and/or pinching under cyclic loading as shown by the experiments performed by the authors (Hamdy et al. 1992). The bilinear or the stiffness degrading model available in the original version of DRAIN-2D would not be adequate in this case. Therefore, a new element based on the work of Chung et al. (1987) was added to the program in order to model the members of nominally ductile frames. The new element can exhibit stiffness degradation, strength deterioration and pinching in the hysteresis loops. The element consists of a linear elastic element with non-linear rotational springs at its ends. All plastic deformation effects are introduced by means of the moment-rotation relationships of the end springs, whose general behaviour is shown in figure 1.

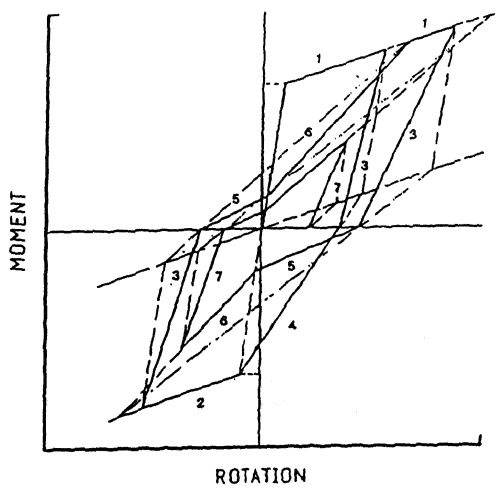

Figure 1. Typical moment-rotation relationship of the new element for DRAIN-2D (Chung et al. 1987).

6.2 *Ground motion input*

To avoid dependence on the characteristics of a single record, a total of fifteen earthquake records were used as the ground motion input for this study. All the records were in the high a/v (peak ground acceleration-to-velocity ratio) range (> 1.2g/cm/s) to reflect the seismic zoning ($Z_a > Z_v$) at Quebec City. A complete description of the characteristics of these records is given by Naumoski et al. (1988). All the ground motion records were normalized to a common maximum velocity equal to the design velocity of Quebec City. The response parameters discussed below are based on a statistical analysis of the individual responses to each ground motion record.

7 DYNAMIC ANALYSIS RESULTS

7.1 *Total and interstorey drifts*

As shown in figures 2 and 3 there is very small difference between the drifts of ductile and nominally ductile frames. This mainly stems from the fact that the same sections were used for both design options.

7.2 *Rotational Ductility Demand for Beams*

The maximum rotational ductility demands at the beam ends are shown in figure 4. It can be seen that the ductility demand for ductile frames is larger than that for nominally ductile frames. Nevertheless, the difference is minor considering that the base shear of a ductile frame is half that of a nominally ductile frame.

7.3 *Beam shear demand vs beam shear capacity*

7.3.1 *Ductile frames*

Figure 5 shows that there is excellent agreement between the maximum response shear forces and the design values in the beams of ductile frames. Moreover, the capacity of the provided stirrups exceeds the demand shears.

7.3.2 *Nominally ductile frames*

An experimental investigation performed by the authors (Hamdy et al. 1992) showed that the concrete contribution to the shear resistance of the beams of nominally ductile frames is unreliable when the beam is subjected to ductility demands similar to those observed here. Therefore, the beam shear capacity depicted in the following figures will consist only of the stirrups resistance. Figure 6 shows that in the beams of the nominally ductile frames the response shear forces exceeded the factored design shears and the capacity of the minimum stirrups by approximately 30 percent.

In the six-storey nominally ductile frame analyzed by Paultre and Mitchell (1991), the response shears exceeded the design values from NBCC by 30 to 40 percent. For that particular frame, the capacity of the minimum stirrups as provided by the code (No. 10 @ d/4 spacing) was adequate to resist the maximum induced shear forces. This will not always be true for all nominally ductile frames. A frame sustaining larger loads (e.g. frames analyzed here)

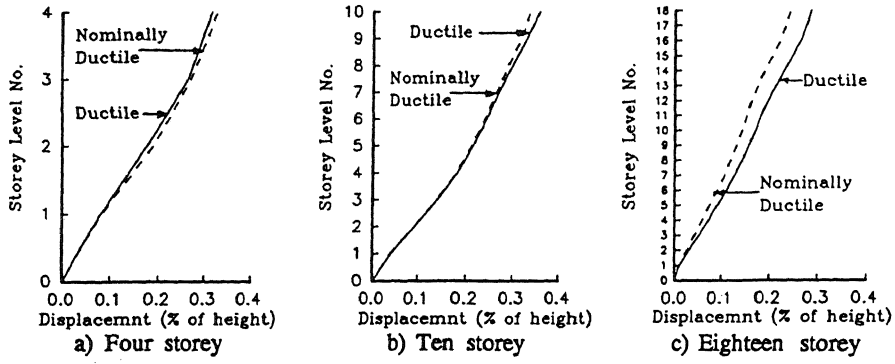

Figure 2. Envelopes of lateral displacements of the frames (% of height).

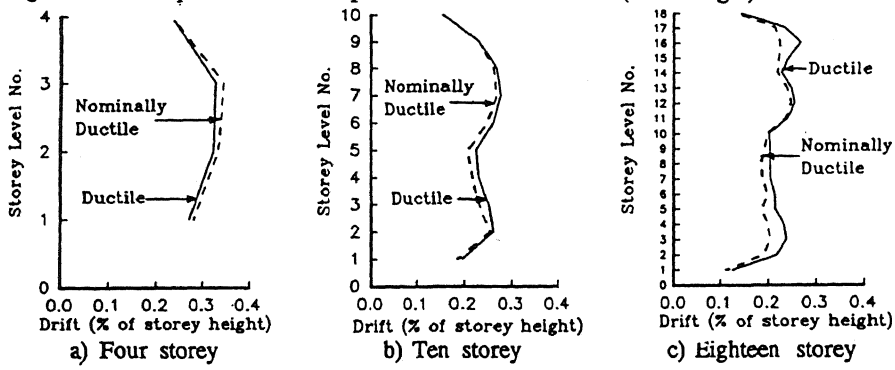

Figure 3. Envelopes of drift indices of the frames (% of storey height).

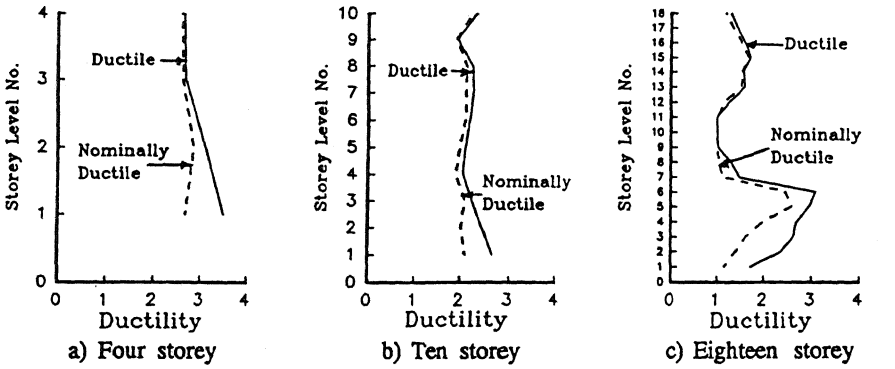

Figure 4. Beam rotational ductility demands.

will undergo larger shear forces while the minimum stirrups will remain the same, resulting that the shear demand exceeds the capacity. This may result in a brittle shear failure which must be avoided. The factored design shears should be increased to be more reflective of the demand shears.

8 MODIFIED PROCEDURE FOR DESIGN SHEAR CALCULATIONS IN NOMINALLY DUCTILE FRAMES

Since the factored design shears are smaller than the response values, an increase in the design values is required. Such an approach is currently used by the New Zealand concrete design code (SANZ, 1982) for *structures of limited ductility* (which are equivalent to nominally ductile frames in the Canadian context). A constant magnification factor for all storey levels is applied to the shear forces due to seismic lateral loading before using the conventional loading combinations. The New Zealand code does not permit the use of limited ductility structures in buildings higher than four storeys. The same approach is used by the ACI-318 concrete design code (ACI, 1989) for the design of *structures in moderate seismic zones*, which can be used for all building heights. Application of the New Zealand

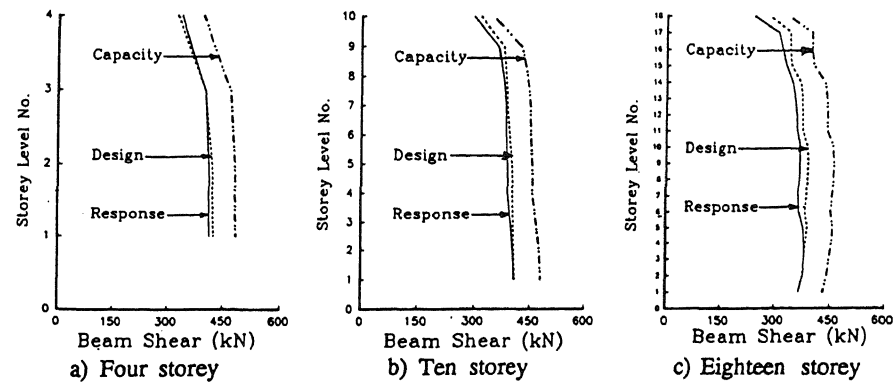

Figure 5. Comparison between design and response shears in ductile frames.

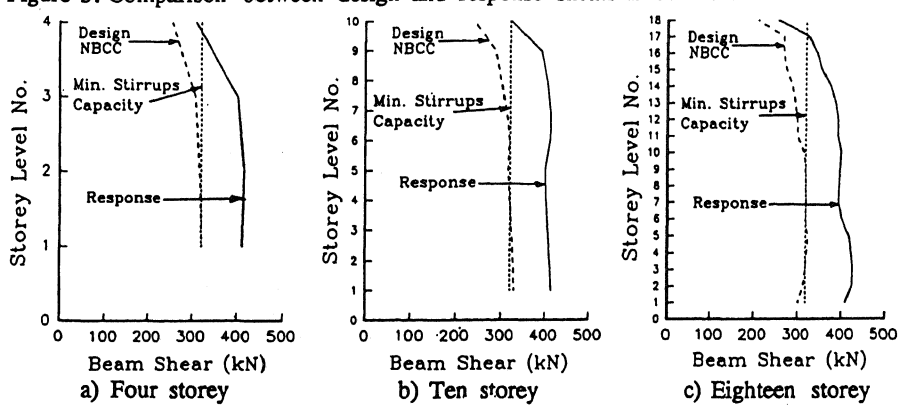

Figure 6. Comparison between design and response shears in nominally ductile frames.

and ACI approaches to the buildings analyzed in this study has shown good agreement between the magnified design shear forces, V'_f, and the response values for the four storey frame only. For the ten and eighteen storey frames, the design shears were overestimated in the lower storeys and underestimated in the upper storeys.

The procedure suggested here involves applying a magnification factor to the shear forces due to the seismic lateral loading, V_Q, with the shear forces due to gravity loads remaining unchanged. Since the Canadian codes allow the use of nominally ductile frames for all building heights, it is desirable to consider the number of storeys and the storey level number in calculating the magnification factor. The suggested magnification factor is a function of the total number of storeys in the frame and the storey level under consideration as given by equation 3.

$$(V'_Q)_i = \left(2+\frac{2.5}{N-i}\right)(V_Q)_i \qquad (N-i\geq 1) \qquad (3)$$

where i is the storey level under consideration, N is the total number of storeys, V_Q is the shear force in the beam due to seismic lateral loading, V'_Q is the magnified shear force due to seismic lateral loading.

The magnified design shear force V'_f will be the greatest value resulting from the following combinations;

$$V'_f = \begin{array}{l} 1.25\,V_D + 1.5\,V_L \\ 1.25\,V_D + 1.0\,V'_Q \\ 1.25\,V_D + 0.7(1.5\,V_L + 1.0\,V'_Q) \end{array} \qquad (4)$$

where V_D is the Shear force due to dead load and V_L is the Shear force due to live load.

It is suggested here that the contribution of concrete to the beam shear resistance should be ignored in the design. The reason is the relatively large ductility demands attained during earthquake excitation which would diminish the concrete shear resistance as shown by experiments (Hamdy et al. 1992). Therefore, the stirrups should be designed to resist the total magnified design shear forces, V'_f. Figure 7 shows a comparison between the shear demands, the modified design shears and the capacity of the provided stirrups for the three nominally ductile frames considered in this study. The modified design shears are in close agreement with the demand shears. Moreover, the capacity of the provided stirrups always exceeded the shear demands, thus preventing any shear problems in the beams.

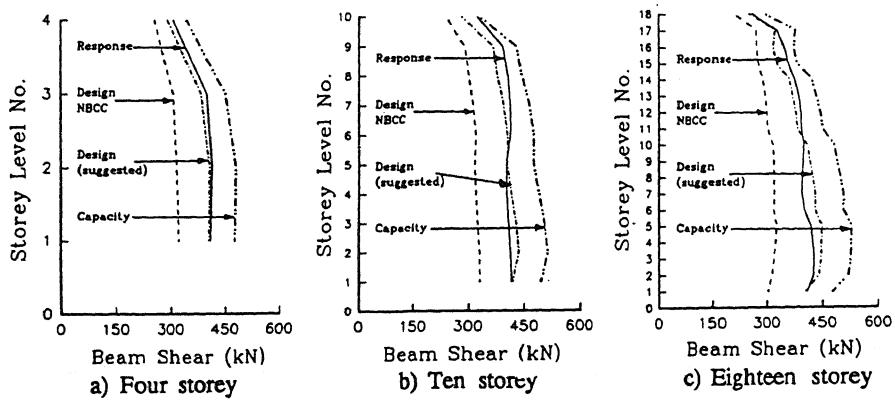

Figure 7. Comparison between the response and the suggested design shears in nominally ductile frames.

9 SUMMARY AND CONCLUSIONS

A four storey, a ten storey and an eighteen storey building were designed as ductile and nominally ductile frames according to the current Canadian practice. The designed frames were analyzed statically to determine their overstrength factors and dynamically under earthquake excitation. The response parameters investigated were the total and interstorey drifts, the beam rotational ductility demands and the maximum shear forces in the beams. From the results of this investigation the following remarks could be drawn:

1. The overstrength factors of ductile frames were found to be significantly larger than the value assumed in NBCC 1990, while the corresponding factors of nominally ductile frames were in the same order as that assumed in the code.
2. Under earthquake excitation, the behaviour of ductile and nominally ductile frames is similar in terms of deflections and ductility demands. Also, the difference between the ductility demands on the beams of ductile frames and those of nominally ductile frames is smaller than what would be implied by the difference in the design seismic base shear.
3. The shear forces attained during the response in the beams of nominally ductile frames were found to be much larger than their capacity. Thus, an increase in the design shear forces in such frames is required.
4. A procedure for calculating the design shear forces in the beams of nominally ductile frames is suggested. The procedure is shown to give design shear stresses in good agreement with the response values for low-, medium and high-rise nominally ductile frames. The suggested procedure is also simple enough for use in the design office.

REFERENCES

American Concrete Institute (ACI) committee 318, 1989. *Building code requirements for reinforced concrete*, ACI 318-89., Detroit, Michigan, U.S.A.

Associate Committee on National Building Code, 1990. *National Building Code of Canada, NBCC 1990.*, Ottawa, Canada.

Canadian Standards Association (CSA), 1984. *Design of concrete structures for buildings, CAN3-A23.3-M84*, Rexdale, Ontario.

Chung, Y.S., Meyer, C. and Shinozuka, M. 1987. *Seismic damage assessment of reinforced concrete members*. Report No. NCEER-87-0022, Buffalo, N.Y., U.S.A.

Clough, R.A., Benuska, K.L. and Wilson, E.L. 1965. Inelastic earthquake response of tall buildings. *Proc. 3rd WCEE* II: 68-84, New Zealand.

Hamdy, K.A., 1992. *Seismic response of nominally ductile frames*. Ph.D. thesis (under preparation), McMaster University, Hamilton, Ontario, Canada.

Hamdy, K.A., Tso, W.K. and Ghobarah, A., 1992. Experimental study on the seismic response of beams in R/C frames of nominal ductility. *Proc. CSCE annual conference*, Quebec City, Canada.

Kanaan, A., and Powell, G. H., 1973. *General purpose computer program for inelastic dynamic response of plane structures.*, Report No. EERC 73-6, University of California, Berkeley, U.S.A.

Naumoski, N., Tso, W. K. and Heidebrecht, A. C. 1988. *A selection of strong motion earthquake records having different a/v ratios.*, Report No. EERG 88-01, McMaster University, Hamilton, Ontario.

Paultre, P. and Mitchell, D., 1991. Assessment of some Canadian seismic code requirements for concrete frame structures. *Canadian Journal of Civil Engineering* 18: 343-357.

Standard Association of New Zealand (SANZ), 1982. *Code of practice for the design of concrete structures, NZS 3101.*, Wellington, New Zealand.

Tso, W.K. 1991. Overview of seismic provision changes in national building code of Canada, 1990. *Proc. 6th CCEE*: 743-750, Toronto, Ontario.

Wilson, E. L., Hollings, J. P. and Dovey, H. H., 1975. *Three dimensional analysis of building systems (extended version).*, Report No. EERC 75-13, University of California, Berkeley, U.S.A.

Zhu, T.J., 1989. *Inelastic response of reinforced concrete frames to seismic ground motions having different characteristics.*, Ph.D. thesis, McMaster University, Hamilton, Canada.