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Overview of seismic provision changes in national building code of Canada, 1990

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ABSTRACT

This paper highlights the major and minor changes of the seismic provisions in NBCC-1990. The major changes include (1) Load factor for earthquake load, (2) Base shear formula, (3) Force modification factor R in place of structural coefficient K, (4) Story drift estimation, and (5) Anchorage force for parts of portions of building. The minor changes from NBCC 1985 include: (1) Importance factor (I), (2) Foundation factor (F) and (3) Top concentrated force on structure (F_t).

Finally, a comparison of the base shear calculated according to NBCC-85 and NBCC-90 is given to show that the base shears for most structural systems are similar, except unreinforced masonry (URM). The necessity to increase the base shear for URM appears to be justify in view of the experience of the Saguenay Earthquake in 1988.

INTRODUCTION

The seismic provisions in the National Building Code of Canada, 1990 (NBCC 90) represent the second instalment of a two stage changes of seismic provisions in Canada since 1980. The first stage changes include the use of peak ground velocity instead of peak ground acceleration as a site seismic intensity index; the recognition of different ground motion characteristic effect in the design of short period structures; and the adoption of a new set of seismic zoning maps with contours presented based on the risk probability level of 10% exceedance in 50 years. The reasons for these changes have been summarized (Heidebrecht and Tso, 1985); and these changes have been incorporated in the National Building Code of Canada, 1985 (NBCC 85).

In this paper, the second stage changes since 1985 that appear in NBCC 90 will be discussed. The changes are classified into major and minor changes and specific changes in each class will be presented below.

MAJOR CHANGES

There are major changes in three broad areas, namely (a) the specification of building strengths, (b) the estimation of story drifts, and (c) the specification of anchorage forces for parts or portions of buildings.

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(a) Building Strength Specification

Changes in building strength specification is more in form than in actual design values. These changes were made so that the information is presented in a more rational form to be implemented, and more reflective of the current seismic design philosophy. They will be discussed under three subheadings:

(a.1) Load Factor for Earthquake Loading

Unlike NBCC 85, the base shear value given the NBCC 90, V_{90}, is already calibrated to incorporate the load factor α_Q, in the context of Limit State Design load combination. Any seismic load effects obtained based on V_{90} are factored seismic load already and α_Q for earthquake load should be taken as unity. The change involved from NBCC 85 can be summarized in equations (1a) and (1b).

$$(V_f)_{85} = \alpha_Q V_{85} = 1.5 V_{85} \quad (1a)$$

$$(V_f)_{90} = \alpha_Q V_{90} = V_{90} \quad (1b)$$

where V_f denotes the factored seismic base shear.

Therefore, one should compare V_{90} with $1.5V_{85}$ to assess the net change in design base shear between the two editions of the Code.

The reason for this change is the desire to make an important distinction between the wind and earthquake design philosophies. Both loads appear as lateral loads in the design process and are often compared in order to choose the "critical" lateral loading source. However, most building designed according to the codified factored seismic base shears can be expected to have exceeded some forms of ultimate limit state (inelastic deformation, severe cracking, etc.) when they are exposed to the "design" seismic event. Unlike wind design, seismic design deemphasizes the importance of strength in favour of good post-elastic performance, commonly referred to as ductile behaviour. It is more appropriate therefore to specify the codified seismic base shear directly in values compatible with the ultimate limit state rather than with the serviceability state as is done in wind loading.

(a.2) Base Shear Formula

The base shear formula of NBCC 90 can be written in the form

$$V_{90} \left(\frac{1}{U}\right) = \frac{V_e}{R} \quad (2)$$

where V_e is the elastic base shear and R is the force modification factor dependent on the type of structural system used. U is called a factor representing level of protection based on experience and is taken to be equal to 0.6 for all structural systems. Alternatively, the factor (1/U) can be considered as an overstrength factor. It has been observed that buildings designed using a base shear value of V actually have a lateral strength that is substantially higher than V (Osteraas and Krawinkler 1989; Miranda and Bertero 1989; Fishinger and Fajfar 1990). Osteraas and Krawinkler (1989) have studied the behaviour of steel frame structures in Mexico city during the 1985 Michanocan earthquake and found an overstrength which increases from 2 for long period structures to about 13 for very short period structures. Similar investigation for low-rise reinforced concrete frame structures in Mexico City has been carried out by Miranda and Bertero (1989) and they found an overstrength factor greater than 2.5 for four-story building and greater than 5 for two-story buildings. Fishinger and Fajfar (1990) have summarized the results from various investigations of a specially designed and experimentally tested seven-story reinforced concrete frame-wall structure and showed overstrength factor of 3-4.

The factors that contribute to the overstrength include the higher material strength realised than the nominal values specified in design, the many nominal or minimum design requirements in material codes irrespective of strength demand, the contributions to the lateral strength of elements such as stair and floor slabs, and the force redistribution effect due to the redundancy of most structural systems. The product of the base shear V_{90} and the overstrength factor (1/U) leads to an estimate of the actual lateral yield strength of the building. In NBCC 1990, the overstrength factor is taken to be equal to 1.67 and is applicable to all structural systems with all periods. Equation (2) simply states that the actual lateral strength of the building should be equal to the elastic strength demand V_e, reduced by a factor R which is a function of the ductility capacity of the structural system concerned.

(a.3) Force Modification Factor R

In all previous editions of NBCC, the influence of structural system choice is characterized by the "K" factor in the base shear expressions. The replacement of the K factor by the force modification factor is more than a mathematical exercise. For the mathematical oriented, it can be recognised that

$$KR \approx 3 \tag{3}$$

A comparison of the K and R values for different structural systems can be found in Table (1). There are two significant benefits using the R factor in NBCC 90. First, by relating it explicitly with the elastic force demand V_e, it draws attention to the designers the implication of choosing any R value in the seismic design load. For example using an R value of 4 implies that the design load is only a quarter of the design load needed if the building is to remain undamaged when exposed to the design seismic event. The building can therefore be excited well into the inelastic range and the survival (non-collapse) of the building relies heavily on its ductility. This line of reasoning will encourage designers to combine loading and detailing requirements when choosing the structural systems.

The second significant benefit is the direct linking of the R values assigned to different structural systems in NBCC 90 on one hand, to the design and detail requirements of these structural systems, as specified by the different material codes in Canada on the other. This direct linking is consistent with the limit state design approach. The material code of design specifications serve as the guidelines to provide appropriate capacities to satisfy the seismic demands set out by the seismic provisions of NBCC 90. Since careful design and detail requirements for ductile behaviour in the post elastic range have been stated in many Canadian material codes (Chapter 21 in CSA.A2.3.3-M-84 and Clause 27 and Appendix D of CAN/CSA-S16.1-M89), the designers are guided by NBCC 90 and these material codes to end up with a proper balance between strength and ductility in their design.

(a.4) Comparison of Factored Base Shears Between NBCC 85 and NBCC 90

In concluding this discussion of change in strength specification, it is illustrative to compare the factored base shear specified according to NBCC 85 and NBCC 90 for different structural systems. For this comparison, it is assumed that both the foundation factor F and importance factor I are equal to unity and the buildings are located either in Vancouver, B.C., ($v = 0.2$, $Z_a = Z_v$) or Montreal, Quebec ($v = 0.1$, $Z_a > Z_v$). As shown in Fig. (1), the difference in factored base shear for ductile concrete or steel moment resisting frame is minimal. The same observation applies to ductile steel braced frame ($K = 1.0$, $R = 3.0$); and concrete moment resisting frame or steel braced frames with nominal ductility ($K = 1.3$, $R = 2.0$); as shown in Figs. (2) and (3) respectively. For the more ductile systems having $R = 3.0$ or 4.0, the NBCC 90 factored base shears are slightly less than those specified in NBCC 85, while the reverse is true for the systems with nominal ductility having $R = 2.0$. This bias in favour of more ductile structural systems is more dramatically shown in the comparison of base shear for ductile flexural walls on one hand, and the unreinforced masonry system on the other. Through research in New Zealand and United States, it has been shown that concrete walls properly designed and detailed, can behave in a ductile manner. These design rules have been incorporated in the Canadian concrete code for seismic design. In recognition of such advances, the design base shear for ductile flexural wall specified by NBCC 90 is substantially lower than those specified by NBCC 85, as shown in Fig. (4).

For design of unreinforced masonry structures, NBCC 90 prescribes a higher base shear than NBCC 85, as shown in Fig. (5). The justification of this increase in base shear is due to the poor seismic performance of such structures in many earthquakes. An evaluation of the base shear specification on this class of structure in view of their relatively poor performance in the 1988 Saguenay earthquake ($M = 5.7$) in Quebec province has been made (Tso and Zhu 1991). Damage survey in the epicentral area showed that a number of short period unreinforced masonry buildings have suffered damage (Mitchell et al. 1990).

The design base shear for short period unreinforced masonry buildings in the Saguenay region has been increased from NBCC 80 to NBCC 85, recognising the vulnerability of this type of structure to the high frequency content of ground motions likely to occur associated with earthquakes in this region. The level of design base shear is raised again in NBCC 90. The seismic resistance coefficient (design base shear to seismic weight) for short period unreinforced masonry buildings in the Saguenay region are shown in Fig. (6). Plotted in the same figure is the spectral accelerations computed on the two horizontal components of ground motions measured at Chicoutimi, a town 36 km from the epicentre of the Saguenay earthquake. The spectral acceleration plot can be interpreted as the seismic strength demand while the three codified seismic resistance coefficient curves can be treated as the strength supply curves. Comparisons of the supply and demand curves show that even the increase of base shear of NBCC 85 from NBCC 80 is not sufficient to ensure short period unreinforced masonry structures to withstand the ground shaking at Chicoutimi. The further increase of base shear for this type of structural system in NBCC 90 appears to be a step in the right direction.

(b) Estimate of Story Drifts

Reliable estimates of the maximum story drift during the earthquake is essential to limit the damage of nonstructural elements. Many of the nonstructural elements need to be protected from damage as they are essential for the functioning of the buildings. This is particularly crucial in the cases of designing post-disaster buildings such as hospitals and fire stations. Another reason for building drift estimation is to allow adequate separation between adjacent buildings to prevent them from pounding each other. Observation of the many building damages caused by pounding in Mexico City in 1985 illustrates the importance of such consideration in design.

Unlike NBCC 85 where the story drift is taken as three times the elastic drift based on code load, NBCC 90 specifies that the story drift is estimated as the product of force modification R and the elastic drift based on the code load. This modification recognized that the inelastic deflection and the elastic deflection for buildings are approximately the same. R times the code load is essentially the elastic load. Drift provisions in NBCC 90 implies that the inelastic drift can be estimated by calculating the elastic deformation based on the elastic load demand on the structure.

To emphasize the importance of drift on post-disaster buildings, NBCC 90 limits the drift to 1% of story height for these type of buildings but consider drift up to 2% of story height to be acceptable for other buildings.

(c) Anchorage Forces for Part and Portion of Buildings

Unlike NBCC 85, NBCC 90 has different clauses to specify design anchorage force for architectural components, and for mechanical and electrical components. The anchorage force for architectural components follows the same format as NBCC 85, namely, $(V_p)_{90} = v(S_p)(W_p)$. The S_p values are tabulated and the values presented in NBCC 90 is approximately 1.5 times the corresponding values in NBCC 85 to reflect that $(V_p)_{90}$ is the factored load already and no further load factor is needed.

Mechanical equipment is generally mounted on isolation dampers to reduce the vibrational effect under normal operation. As a result, it forms a dynamic system when attached to the building and further dynamic amplification can result depending on the condition of mounting.

Also a building filters and amplifies the ground motion and the resulting floor motion depends to a degree on the height of the building. Since the anchorage force depends on the severity of the floor motion, it must also be location dependent. Based on these considerations, NBCC 90 defines the S_p factor for mechanical/electrical equipment by

$$S_p = C_p A_r A_x \tag{4}$$

where C_p values are tabulated for different types of equipment, A_r is the connection factor and A_x is the location factor. A_r ranges from unity for rigid items rigidly mounted to 4.5 for flexible items flexibly mounted. A_x ranges from 1.0 at the ground floor to 2.0 at the top of the building. As a result of such subclassifications, the S_p values for equipment can be as large as 9 times those for the equipment that were located and mounted in the most favourable manner. It is felt that this large factor in design anchorage force represents a small premium to ensure that equipment will not lose its function because it dislodges from its normal position during the seismic shaking.

MINOR CHANGES

It should be noted that the minor changes described herein do not imply that they are insignificant changes. They are grouped under this category because the changes can be implemented in a relatively straight forward manner, assuming one is familiar with the corresponding NBCC 85 provision clauses.

(a) Importance Factor I

While NBCC 85 assigned the same importance factor $I = 1.3$ to both post disaster buildings and schools, a distinction is made in NBCC 90 between these two types of buildings with a higher importance factor ($I = 1.5$) applicable to post disaster buildings. The poor performance in many of the hospitals in Mexico City during the 1985 earthquake re-emphasized the necessity of the availability of post-disaster buildings after an earthquake. This leads to a more stringent strength requirement as well as drift limitation for such buildings. The changes of the importance factor I can be summarized in Table (2).

(b) Foundation Factor F

The amplification of ground motion by thick layers of soft soil was observed in Mexico City during the 1985 Michanocan earthquake and again in the San Francisco bay area during the 1989 Loma Prieta earthquake. In Mexico City, the amplification of peak ground surface acceleration can reach as high at 4 (Romo and Seed 1987), depending on the depth of soil. Typical amplification is in the range of 2 to 3. To accommodate the large amplification effect of deep soil deposit, NBCC 90 increases the soil factor F for such soil conditions from 1.5 to 2.0.

(c) Top Force F_t

To allow for the higher modal contribution on the story shear distribution along the height of a building, NBCC adopted the procedure of adding a top force F_t in its formula for lateral seismic force distribution. Since the influence of higher modal contribution is much more related to the fundamental period T of the building, as opposed to its height to width ratio, NBCC 90 decides to relate F_t directly to the fundamental period T instead of the height to width aspect ratio of the building.

Many of the damages in buildings in Mexico City during the 1985 earthquake occurred around the mid-height level. This could be caused by inadequate allowance of the higher modal contribution. Based on the Mexican experience, NBCC 90 increases the upper limit of F_t from 15% to 25% of base shear. This upper limit of F_t would be reached when $T = 3.6$ sec. For buildings with a period beyond 3.6 second, it is likely that the actual earthquake force distribution will be determined by a more refined analysis than the formula given by the code.

CONCLUSIONS

Substantial upgrading and improvements have been made to the seismic provisions in the National Building Code of Canada since 1980. The present Canadian seismic loading code (NBCC 90) has incorporated a number of lessons learned from many major earthquakes around the world in the 1980's. It contains a number of innovative features among other seismic codes. First, it explicitly coupled the seismic design strength, through the force modification factor R, to specific material base design codes. This coupling provides direct guidance to Canadian engineers to design and detail structures such that these structures will have the expected seismic behaviour in the event of an earthquake. Second, the current Canadian code is one of the few seismic codes that explicitly recognizes the overstrength of buildings from their design values. This recognition is expressed in the form of (1/U) = 1.67 in NBCC 90, independent of structural systems used. As more research results become available, this factor may be refined to reflect its dependence on both structural period and structural systems used in future editions of the Code. Third, the Canadian Code recognises different seismic regions in Canada are likely to experience ground motions having substantially different frequency content, and makes allowance for the effect on these different types of ground motions on short period structures. The 1988 Saguenay earthquake occurred in a seismic region where the ground motions are expected to contain energy in the high frequency range and all measured records in the epicentral area confirmed this expectation. This demonstrated the necessity to differentiate the different types of ground motions as is currently done.

In summary, the current Canadian seismic code (NBCC 90) is at the forefront of seismic code for buildings and provides guidelines for the safe designs of buildings in seismic regions in Canada.

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Table (1): K and R Factors for Structural Systems

Structural Systems	K	R
Ductile concrete or steel moment resistant frame	0.7	4.0
Ductile flexural wall	1.0	3.5
Ductile steel braced frame	1.0	3.0
Concrete moment resisting frame with nominal ductility	1.3	2.0
Steel braced frame with nominal ductility	1.3	2.0
Reinforced masonry	2	1.0

Table (2): Importance Factor I

Occupancy	NBC-85	NBC-90
Post disaster buildings	1.3	1.5
Schools	1.3	1.3
Other occupancy	1.0	1.0

Table (3): Foundation Factor F

Category	Type and Depth		F	
			NBC-85	NBC-90
1	Rock, stiff soil	< 15m deep	1.0	1.0
2	Stiff soil Loose coarse grained soil Soft fine grained soil	> 15m deep < 15m deep	1.3	1.3
3	Loose coarse-grain soil	> 15m deep	1.5	1.5
4	Soft fine-grained soil	> 15m deep	1.5	2.0

DUCTILE CONCRETE & STEEL MRF
(K = 0.7, R = 4.0)

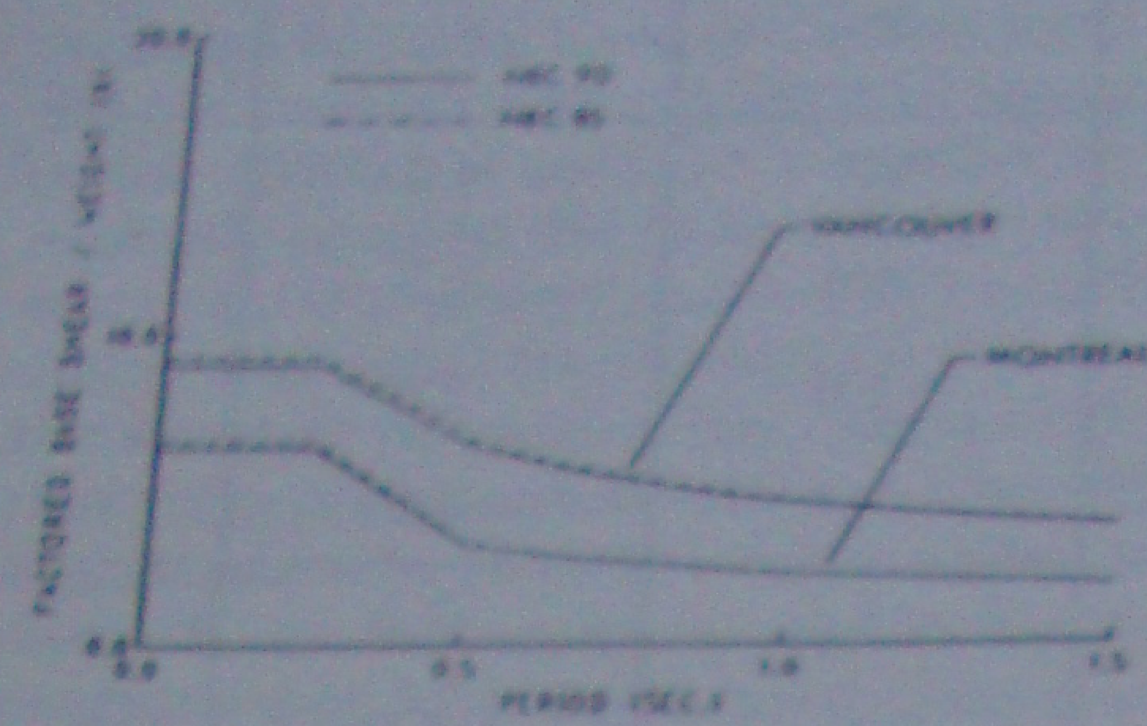

Fig.(1) Factored Base Shear for Ductile MRF

DUCTILE STEEL BRACED FRAME
(K = 1.0, R = 3.0)

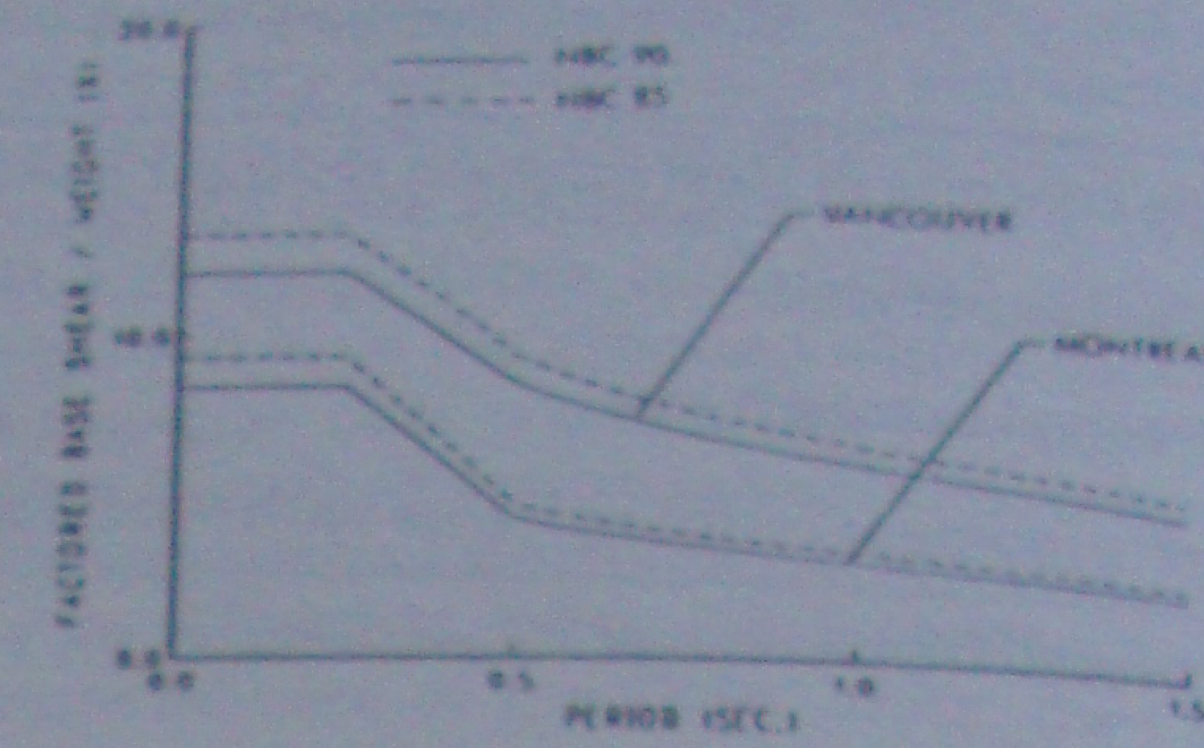

Fig. (2) Factored Base Shear for Ductile Braced Frame

CONCRETE MRF WITH NOMINAL DUCTILITY
STEEL BF WITH NOMINAL DUCTILITY
(K = 1.3, R = 2.0)

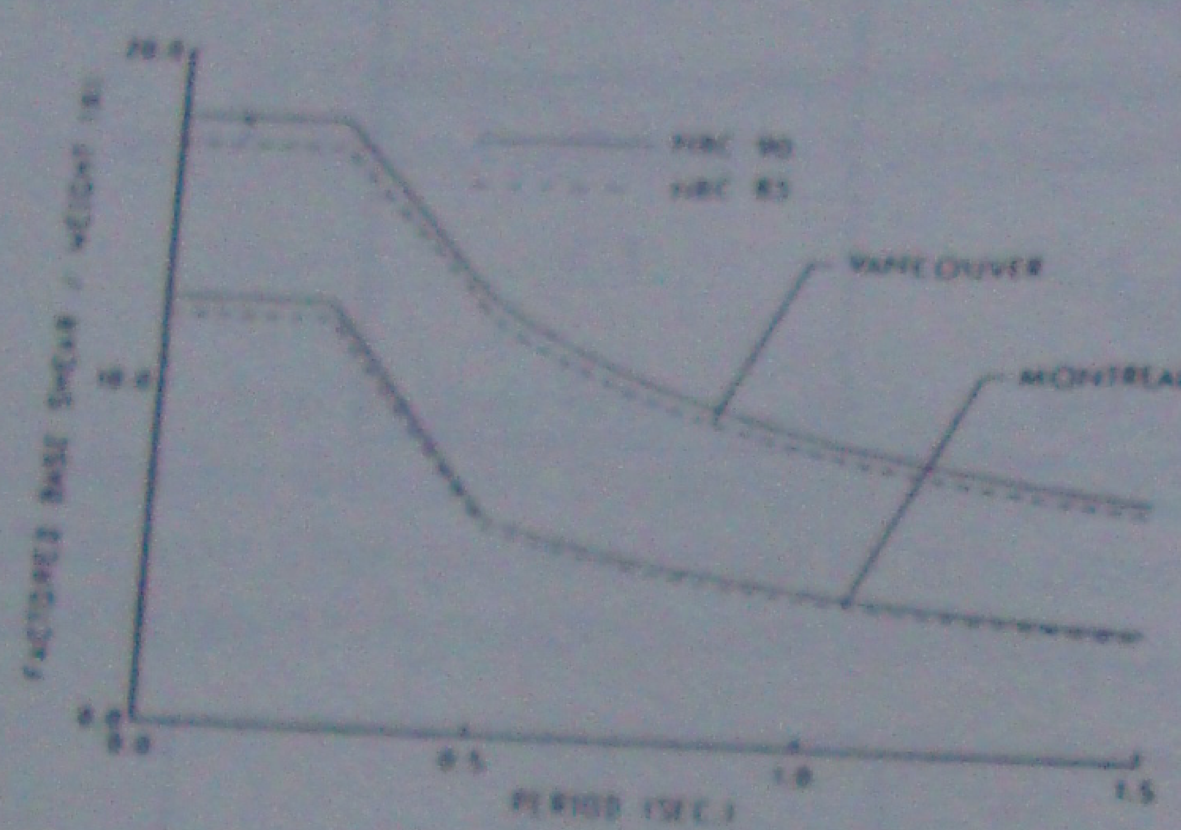

Fig. (3) Factored Base Shear of Frames of Nominal Ductility

DUCTILE FLEXURAL WALL
(K = 1.0, R = 3.5)

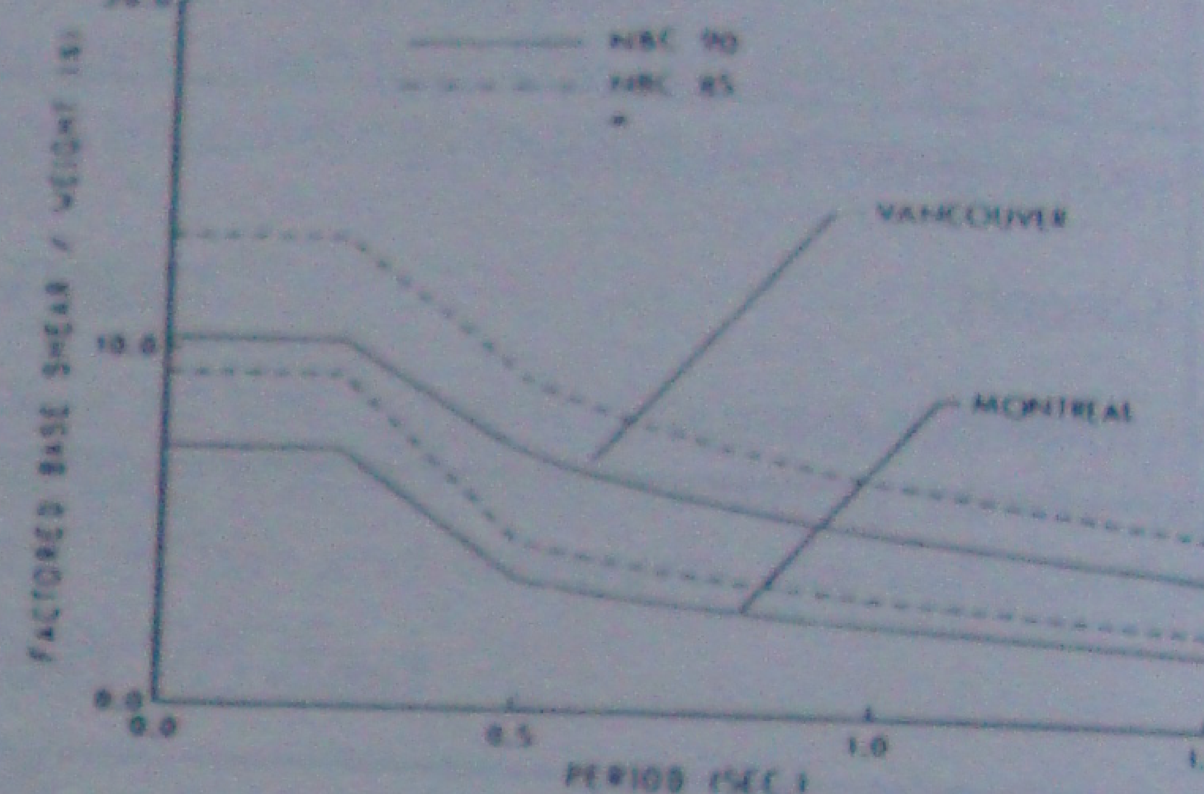

Fig.(4) Factored Base Shear of Ductile Flexural Walls

UNREINFORCED MASONRY
(K = 2, R = 1.0)

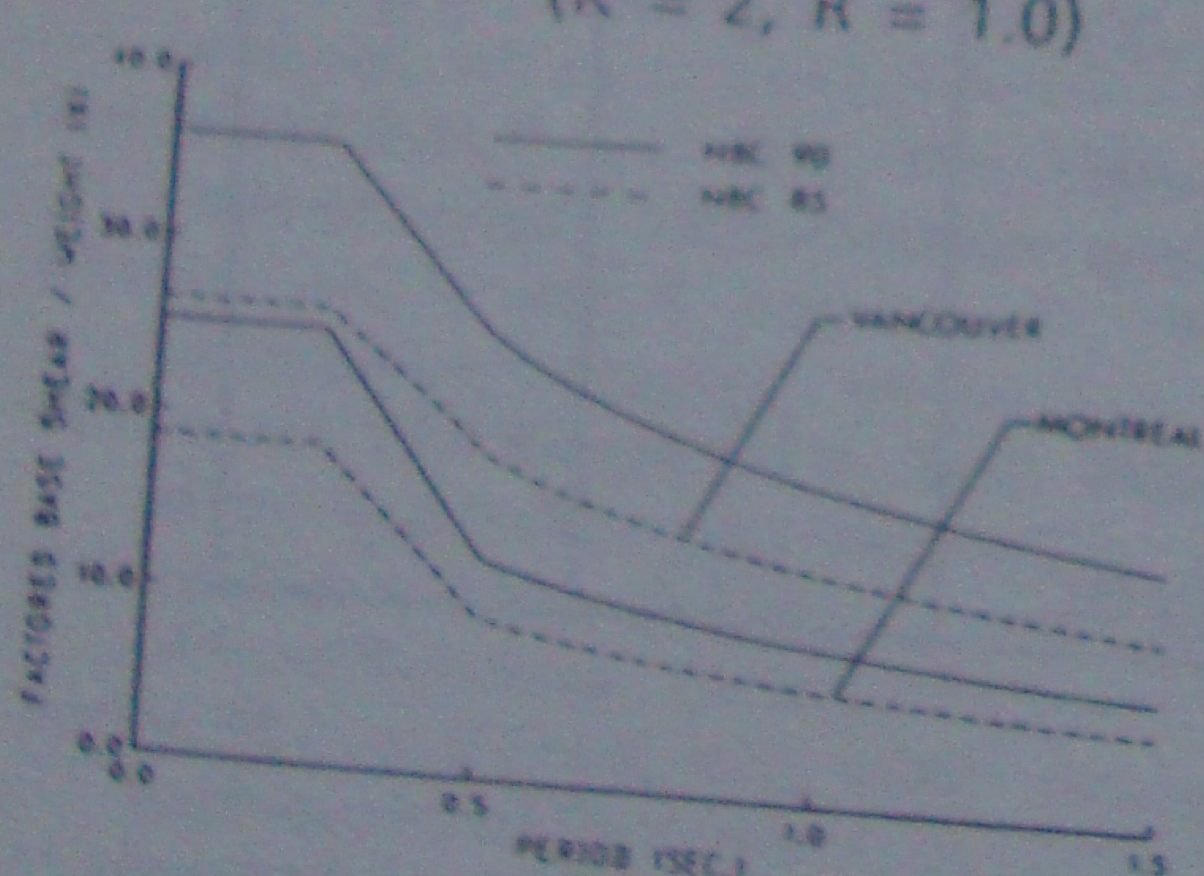

Fig.(5) Factored Base Shear of Unreinforced Masonry