SEISMIC RESPONSE OF REINFORCED CONCRETE STRUCTURES DESIGNED USING NORTH AMERICAN BUILDING CODES

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CHAPTER 1

INTRODUCTION

1.1 - General :

Over the years, earthquakes have been the cause of great disasters in the form of destruction of property, and loss of human lives. It is therefore essential to give special considerations to the design of structures in seismically active regions.

Research in the field of earthquake engineering is of recent origin, and the development of basic information is not yet complete. As the research progresses, more knowledge of the complex phenomenon of earthquake occurrence and its effects on building structures will be acquired. In the current design practice for aseismic design of structures, the following three methods of analysis are used :

i) Equivalent lateral load analysis,

ii) Modal analysis, and

iii) Time history analysis.

The first two methods are the most commonly used methods of analysis. The third method requires a suitable software and expertise and can be very costly. It is generally adequate to use the equivalent lateral load procedure for ordinary, symmetric buildings. This procedure requires only a static load analysis and is commonly used in North America.

In the present study, it is of interest to compare the seismic analysis (Equivalent Lateral Load Method) and design provisions of two building codes used in Canada and the United States. Many aspects of the Canadian Code requirements are similar to those of the U.S. Codes. However, the nature of code provisions for the design of buildings are such that their effects on the final design can vary immensely. For example, even though the development of the design base shear formula by the Canadian and the U.S. codes are similar, the two codes give rise to different design base shears. Thus the design seismic base shear can be significantly lower for a building on the Canadian side of the border than for a comparable building subjected to a presumably comparable seismic risk on the U.S. side of the border. Other differences also exist between the two codes, in terms of design and detailing requirements.

1.2 - Objective :

The primary objective of this study is to investigate the differences in the final designs of structural members designed by i) the Canadian Code and ii) the U.S. Code. The objective also includes dynamic inelastic response analysis of the structures designed using building codes to assess their performance during an actual earthquake.

1.3 - Scope :

The following gives the scope of this investigation :

- Selection of a frame and a shear wall structure for analysis and design.
- Determination of the equivalent static loads using the Canadian and the U.S. building codes.
- Static analysis of the selected structures using the equivalent static loads and the gravity loads.
- Proportioning and detailing of the selected structural members using the seismic provisions of the Canadian and the U.S. building codes.

- Modelling the structures for dynamic analysis and preparation of the necessary input data.
- Dynamic inelastic analysis of structures.
- Comparison of final designs resulting from the two codes of practice.
- Assessment of the performance of structures under dynamic earthquake forces.
- Presentation and discussion of results.

CHAPTER 2

BUILDING CODES CONSIDERED

2.1 - General :

As neighbors separated by a common border the technology exchange between Canada and the United States has for the greater part been quite good. Despite this close association, there exist significant differences in the building codes of the two countries.

The Canadian codes under consideration are the National Building Code of Canada 1985 (NBCC-1985) and the code for Design of Concrete Structures, (CAN3-A23.3-M84).

The U.S. codes of interest are the Uniform Building Code, 1982 edition (UBC 1982) and the Building Code Requirements for Reinforced Concrete (ACI-318.83).

2.2 - Background on Building Code Development:

i) NBCC : The building code which has been the forerunner of most seismic codes now in use in the world is that developed by the Structural Engineers Association of California, commonly known as the SEAOC code. The evolution of the current seismic provisions of the National Building Code (NBCC) and the various forms of Canadian seismic codes in use for nearly four decades have been very well summarized by Uzumeri [1].

In the first 1941 edition of NBC of Canada, the seismic provisions appeared in an appendix and were based on concepts presented in the 1937 UBC. The lateral earthquake force was assumed to act at the centre of gravity of the structure and to have magnitude given by the product of the building weight (which was taken as the dead load plus the live load) and a seismic base shear coefficient.

For a building located in a region where destructive earthquakes were probable, the value of the base shear coefficient varied from 0.02 to 0.05, depending on the bearing capacity of the soil. The first edition of the UBC in 1927 suggested a lateral base shear coefficient of 0.10 for buildings in high seismic risk areas.

The 1953 NBCC seismic requirements included two significant new features: A Canadian seismic zoning map and recognition of the influence of building flexibility in reducing the seismic force. A method was provided to calculate the horizontal force at any level which decreased with the number of stories above the storey under consideration.

The 1965 NBCC seismic requirements were influenced by the 1959 SEAOC code which represented the state of the art in earthquake engineering at that time in the U.S. This edition of NBCC provided a base shear formula:

$$V = RCIFSW \tag{2-1}$$

Where R =seismic regionalization factor; C = type of construction factor; I = importance factor; F = foundation factor; S = structural ductility factor and W = total weight. With the introduction of the above C factor, the NBCC recognized the influence of structural ductility on seismic response. The importance factor I and the foundation factor F were not in the existing UBC (1964) nor in the SEAOC code.

The 1965 NBC of Canada required the total lateral seismic force V to be distributed over the height of the building so that the force at any storey was proportional to the height of that storey and the weight of the floor. The 1965 NBCC also required seismic torsional moments in the horizontal plane of the building to be computed. The U.S. codes of that time did not include such explicit torsion requirements.

The 1970 NBCC included the revised Canadian zoning map. It was based on the peak horizontal acceleration amplitudes that had a probability of exceedence of 0.01 in one year (A_{100}). The zoning in the U.S. codes was based on the subjective Modified Mercali Intensity Scale of 1931. The minimum lateral seismic force was specified by the 1970 NBCC as:

$$V = 1/4\ RKCIFW \tag{2-2}$$

Where R, I, F and W remain same as in the 1965 NBCC, K = type of construction factor, C = structural flexibility factor which was made a direct function of the period of the structure. Approximate and empirical expressions were provided to evaluate the period of a structure.

In the 1975 NBCC, the minimum design lateral seismic force V is specified as:

$$V = ASKIFW \tag{2-3}$$

Where I, F and W remain essentially unchanged from the 1965 NBCC; whereas A = assigned horizontal design ground acceleration; S = seismic response factor which was made a direct function of the period of structure; and K = numerical coefficient reflecting the influence of the type of construction on damping, ductility and/or energy-absorption capacity of the structure. It is important to note that the assigned horizontal base shear did not change at all. In fact, the factor S was derived so that the 1975 NBCC base shear would be 80 percent of the 1970 NBCC base shear. (The 20 percent reduction was partly to counteract the effect of the increase in the overturning moment reduction factor). Thus the seismic design force is related to the traditional levels of seismic design force rather than that produced by the 100-year earthquake.

The 1977 NBCC had essentially the same seismic loading provisions as the 1975 edition. One major change was made: that was to restrict the dynamically determined seismic base shear to not less than 90 percent of the base shear determined by the equivalent static load procedure.

The 1980 NBCC edition, contains no major new developments, but there are a few specific changes. The most significant change related to the calcutation of design base shear is that the seismic response factor S is being modified to contain $\sqrt{T}$ in the denominator rather than $T^{1/3}$. This change is being made to provide clear agreement with other methods of predicting the seismic response of a structure.

In the current edition, namely NBCC 1985, a new factor (v) called zonal velocity ratio is introduced to take into account the fact that the behaviour of the structure may be affected in the intermediate period range.

ii) UBC : The first edition of the Uniform Building Code was published in 1927 and contained in the appendix a chapter on earthquake provisions, planned for optional use.

In 1928, the California State Chamber of Commerce recognized the need for a building code which would afford protection against earthquake damage through the inclusion of a section on seismic design.

The first mandatory seismic code used to any extent in the United States was published in 1933 following the March 10, 1933, Long Beach (Southern California) earthquake.

The second law, the Riley Act which became effective May 26, 1933, made provision for design and construction to resist seismic or wind forces.

In Los Angeles in 1943 it was recognized that more flexible buildings would be subjected to smaller earthquake loads. The formula used was:

$$F = CW \tag{2-4}$$

Where F represented the design force at any level, W was the structure dead weight plus 25 percent of snow load, and C the horizontal force factor, which varied depending on both seismic risk zone and the number of stories above the section of the structure under consideration.

In 1957, the seismology Committee of the Structural Engineers Association of California, undertook to develop a uniform seismic code which would resolve the important differences in the several codes used in seismic areas of the United States and particularly in California.

The UBC provisions were intended to be logically based with explicit consideration given to factors that are generally implicit in present code design provisions. A number of new concepts which are significant departures from existing seismic codes are included. These are:

- More realistic seismic ground motion intensities.
- Consideration of the effects of distant earthquakes on long period buildings.
- Response modification coefficients (reduction factors) which are based on consideration of the inherent capacity for energy absorption, damping associated with inelastic response, and observed past performance of various types of framing systems.

- Complexity of analysis and design dependent on importance or use factor, assigned building seismic performance category and seismic motion intensity.
- Simplified seismic response coefficient formulas related to fundamental period of the building but with certain restrictions.
- Material design stresses approaching yield.

Basically, the UBC code development was similar to the NBCC.

2.3 - Seismic Design Requirements of NBCC-85 :

The N.B.C.C.[4] refers the designer to the code for the design of concrete structures for buildings CAN3-A23.3-M84 [2]. Design conforming to the requirements of this code will conform to the reinforced concretre requirements of N.B.C.C. For the special provisions of seismic design the N.B.C.C. refers to chapter 21 of CAN3-A23.3-M84.

2.4 - Seismic Design Requirements of UBC-82 :

The design of reinforced concrete requirements of U.B.C.[5], are met by ACI-318.83 [3] in a manner similar to N.B.C.C and CAN3-A23.3-M84. The special seismic provisions of ACI-318.83 are included in appendix A.

2.5 - Reinforced Concrete Design Based on CAN3- A23.3-M84 :

Design of members for reinforced concrete buildings in Canada is performed using the code for the Design of Concrete structures for Buildings, Canadian Standars Association CAN3-A23.3-M84 [2]. Clause 21 of this code provides special provisions for seismic design. The following sections provide a brief discussion of the relevant portions of CAN3-A23.3-M84.

2.5.1 - Flexural Members of Ductile Frames :

In flexural members, the longitudinal reinforcement is provided such that the maximum reinforcement ratio, ρ, does not exceed 0.025. The lower limit on the reinforcement ratio is $1.4/f_y$, where f_y is the specified yield strength of steel.

The minimum positive moment capacity of flexural members at column connections is set at 50 percent of the negative moment capacity. The negative moment reinforcement calculated for the design negative moment along the girder and the positive moment reinforcement at the column connection are required to be continuous throughout the girder.

The web reinforcement is provided to develop the shears resulting from design gravity loads on the member and from the moment capacities of plastic hinges at the ends of the member produced by lateral displacement. The minimum size stirrup is number 10 (11.3mm diameter). The required area of shear reinforcement is given by the following formula (clause 11.3.6.1 of ref.2):

$$A_v = \frac{V_s S}{\phi_s f_y d} \qquad (2\text{-}5)$$

with a minimum area in square millimeters of:

$$A_v = 0.35\frac{b_w S}{f_y} \qquad (2\text{-}6)$$

when V_u is greater than $0.5V_c$

Where:

V_s= Factored shear resistance provided by shear reinforcement , newtons.

b_w= Web width, mm.

S= Spacing of stirrups, mm, not exceeding d/2.

f_y= specified yield strength of reinforcement, MPa.

For the design of flexural members, if the simplified method of clause 11.3 is used, the shear resistance provided by concrete (V_c) shall be assumed as zero for members conforming to clause 21.3 described as Ductile Frame Members subjected to flexure.

In regions of potential inelastic action, No.10 hoops or larger are required to be provided over lengths equal to twice the member depth. The spacing of hoops shall not exceed d/4 or 300mm (clause 21.9.2.1.2). Elsewhere in the beams, stirrups are required to be spaced at not more than d/2.

2.5.2- Ductile Frame Columns Subjected to Axial Loads and Bending :

The major seismic provisions for columns are found in chapter 21 of the code. These provisions deal mainly with the extra confinement reinforcement required for seismic resisting columns.

The design of columns, required to resist seismic loadings, is divided into two sections in CAN3-A23.3-M84. The governing criteria for design involves the value of the axial load: Columns shall be designed and detailed as flexural members when:

$$P_e \leq 0.1 A_g f_c' \qquad (2\text{–}7)$$

Where:

P_e= Maximum design axial load on a column during an earthquake.

A_g= Gross area of section.

f_c'= Compressive strength of concrete.

The concrete core of a column shall be confined by special transverse reinforcement when:

$$P_e > 0.1 A_g f_c' \qquad (2\text{-}8)$$

The length along the column that this special reinforcement is required is dependent on the strength of the columns and beams. The code states that at any beam-column connection the sum of the moment strengths of the columns at the design axial load shall be greater than the sum of the moment strength of the beams along each principle plane.

If this requirement is met this special reinforcement shall be placed according to the maximum of:

i) h, the overall thickness of column;

ii) 1/6 the clear span of the column;

iii) 450mm .

Should the moment strengths of the beams be larger than the column strengths, then the special transverse reinforcement shall be provided over the full length of the column.

The area of the required special lateral reinforcement hoop is calculated by:

$$A_{sh} = 0.3 \frac{S h_c f_c'}{f_{yh}} \left(\frac{A_g}{A_{ch}} - 1 \right) \qquad (2\text{-}9)$$

but not less than

$$A_{sh} = 0.12 \left(\frac{S h_c f_c'}{f_{yh}} \right) \qquad (2\text{-}10)$$

Where:

A_g = gross area of section, mm^2;

A_{ch} = cross sectional area of the core of a column, mm^2;

f'_c = compressive strength of concrete, MPa;

f_{yh} = yield strength of transverse reinforcement, MPa;

h_c = cross sectional dimension of column core, mm;

S = spacing of transverse reinforcement, mm.

The maximum spacing of hoops forming the lateral confinement shall not be greater than 100 mm.

A second requirement of the transverse reinforcement of columns involves the shear capacity of the column. There are actually two column shears to check. The second involves beam-column connections and will be discussed in the next section. The first shear requirement, requires that transverse reinforcement in the column shall be provided to ensure that the shear capacity of the column is at least equal to the applied shears at the formation of the plastic hinges in the frame due to the combination of lateral displacement and design gravity loads. The necessary areas of shear reinforcement can be calculated in similar manner to those of flexural members. However, the allowable shear stress taken by concrete (V_c) can be calculated using:

$$V_c = 0.2\lambda\phi_c\sqrt{f'_c}\left(1 - \frac{3N_f}{A_g f'_c}\right)b_w d \qquad (2\text{-}11)$$

Where:

A_g= gross area of concrete cross section, mm^2;

b_w= web width, mm;

d= distance from the extreme compression fibre to the centroid of longitudinal reinforcement, mm;

f'_c= compressive strength of concrete, MPa;

N_f= axial load normal to the cross section, N

ϕ_c= resistance factor for concrete;

λ= factor to account for low density concrete.

2.5.3 - Beam Column Connections in Ductile Frames :

The section of a column, in a ductile moment resisting frame, confined by the beams requires additional shear reinforcement. The design shear shall be equal to the maximum shear in the connection computed by taking into account the column shear and the shears developed from the yield forces of the beam reinforcement. The shear resistance of the joint is computed by:

$$V_j = 1.8\lambda\phi_c\sqrt{f_c'}A_j \tag{2-12}$$

The above equation is for joints not externally confined. A joint is considered to be externally confined, if members frame into all vertical faces of the joint and if at least 3/4 of each face of the joint is covered by the framing member.

2.5.4 - Shear Walls :

The shear walls, like the moment resisting frame components are part of a ductile system. The requirements for ductile shear walls are outlined in chapter 21 of CAN3-A23.3-M84.

The reinforcement requirements for shear walls include both distributed web reinforcement and concentrated end reinforcement to form boundary elements. The minimum area of horizontal and vertical steel is required to be 0.0025 times the cross sectional area of the wall. The vertical reinforcement shall be concentrated at each end of the wall with a minimum reinforcement of $0.001b_wl_w$. For regions of plastic hinge, the minimum area of concentrated reinforcement shall be $0.002b_wl_w$ at each end of the wall. In any region the concentrated reinforcement shall not exceed 0.06.

The concentrated reinforcement shall be tied as a column in accordance with Clause 7.6, and the ties shall be detailed as hoops.

The shear design of flexural walls is perhaps the best example of the ductility requirement. To estimate the design base shear, it is assumed that a plastic hinge will form at the base of the wall. The shear at the base when the wall develops a plastic hinge will be:

$$V = \frac{M_{pw} V_f}{M_f} \qquad (2\text{-}13)$$

Where:

M_{pw}= probable moment resistance of the wall;

M_f= factored moment at base;

V_f= factored shear at base.

The code provisions are set forth in such a manner, in order to prevent failure of overdesigned (less ductile) walls. An overdesigned wall will attract larger moments and shears, therefore the code requires that the less ductile wall be able to carry a larger shear.

2.6 - Reinforced Concrete Design Based on ACI- 318.83 :

Design of structural members for buildings situated in the U.S.A is based on the ACI Building Code Requirements for Reinforced Concrete, ACI-318.83 [3]. Provisions of Appendix A of ACI-318.83 should be followed to ensure ductile lateral load resisting systems. It is observed that the clauses relevant to the present design cases are different compared to those provided in the Canadian Code of practice [2].

2.6.1 - Flexural Members :

Flexural members are limited to axial compressive force of $0.10A_g f'_c$. The clear span to effective depth ratio should be 4 or more. Furthermore the width-to-width ratio is required to be less than 0.3. The width of a flexural member is further restricted to a minimum of 10 inches (250 mm). The maximum width is restricted to the width of the suporting member or three-quarters member depth, whichever is smaller.

Longitudinal reinforcement is provided such that the reinforcement ratio, ρ, is not less than $200/f_y$ nor is greater than 0.025 at any section. The minimum positive-moment strength at the face of the joint is set at one-half of the negative-moment strength provided at that face of the joint. It is also prescribed that the negative and positive moment strengths at any section along the length of the component shall not be less than one-fourth of the maximum moment strength provided at the face of either joint.

Design shear force is determined assuming that ultimate resisting moments of opposite sign act at the joint faces and that the member is loaded with the tributary gravity load along its span. The ultimate resisting moments are calculated using the properties of the member at the joint faces without strength factors and assuming that the stress in the tensile reinforcement is equal to $1.25f_y$. Special transverse reinforcement is provided within a distance equal to twice the effective depth, d, from the end of the member on both sides of a section where flexural yielding may occur. Hoops shall be used as web reinforcement in these regions and the spacing shall not exceed d/4.

2.6.2 - Members Subjected to Bending and Axial Load :

Some geometric constraints are specified in the building code for members subjected to bending and axial load. Accordingly the shortest cross sectional dimension, shall not be less than 12 inches (300 mm). The ratio of the shortest cross sectional dimension to the orthogonal dimension shall not be less than 0.4.

Calculation of longitudinal reinforcement is the same as that specified in CAN3-A23.3-M84 [2]. It is also required that at any joint and in the plane of the frame considered, the moment about the center of joint corresponding to the flexural strengths of columns (or column) shall exceed that corresponding to the flexural strengths of the beams framing into the joint. The columns framing into the affected joint where the above condition is not satisfied, shall be provided with a special lateral reinforcement. This reinforcement will be provided throughout the entire storey height.

The special lateral reinforcement consists of hoop steel. The minimum total cross sectional area of rectangular hoop reinforcement shall not be less than that given by the following equations:

$$A_{sh} = 0.3\frac{Sh_c f'_c}{f_{yh}}\left(\frac{A_g}{A_{ch}} - 1\right) \tag{2-14}$$

$$A_{sh} = 0.12\left(\frac{Sh_c f'_c}{f_{yh}}\right) \tag{2-15}$$

Where:

A_{sh} = total cross sectional area of hoop reinforcement, including supplementary cross-ties, having a spacing of S of maximum of 4 inches (10.16 cm) and crossing a section with a core dimension of h_c ,sq in.

A_g = gross area of section, sq in.

A_{ch} = cross sectional area of column measured out-to-out of transverse reinforcement, sq in.

f'_c = compressive strength of concrete, psi.

f_{yh} = specified yield strength of transverse reinforcement, psi.

h_c = cross sectional dimension of column core measured center-to-center of confining reinforcement, in.

S = spacing of transverse reinforcement, in.

When crossties are used, each end of a crosstie is required to engage a peripheral longitudinal reinforcing bar.

Lateral reinforcement is provided so that the shear strength of the member will be adequate to resist a design shear determined from the following two conditions:

(1) The static forces are applied to the member with the ultimate resisting moments acting at the faces of the joints calculated without capacity reduction factors.

(2) The maximum axial compressive design force is applied to the column.

2.6.3 - Joints :

The lateral reinforcement required for the column, framing into the joint, is required to be continuous throughout the joint. The shear stress in the joint, is limited to $20\sqrt{f'_c}$ psi and $15\sqrt{f'_c}$ psi for confined and unconfined joints respectively.

A laterally confined joint is a joint which, in the direction under consideration, has the opposite faces confined by members which are monolithic with the joint and cover 75 percent of the width and depth of the joint.

The shear force in the joint is determined from a consideration of the static forces in combination with the ultimate resisting moments of the flexural members at the faces of the joint. The ultimate moments of the flexural members are calculated without the strength reduction factor and with the assumption that the stress in the tensile reinforcement is equal to $1.25 f_y$. The shear stress is determined on the basis of the effective section bd, for rectangular sections.

2.6.4 - Shear Walls :

The criteria for shear wall design in ACI-318.83 is somewhat analogous to that of column design in both ACI-318.83 and CAN3-A23.3-M84. The initial design involves calculating the distributed steel. Both vertical and horizontal steel shall not be less than 0.0025 times the gross section of the wall. The concentrated reinforcement requirements depend on the axial load capacity of the shear wall.

The ACI code states that boundary elements shall be provided at boundaries and edges of structural walls for which the maximum extreme-fiber stress, corresponding to factored forces including earthquake effects, exceeds $0.2f'_c$. The resulting concentrated reinforcement shall be tied with transverse reinforcement specified for columns.

The shear requirements for shear walls in ACI involves the calculation of nominal shear strength, V_n, which shall be assumed not to exceed the shear force calculated by using Eq.(2.16).

$$V_n = A_{cv}(2\sqrt{f'_c} + \rho_n f_y) \tag{2-16}$$

Where:

A_{cv} = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, sq in.

ρ_n = ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv}.

f'_c and f_y as defined earlier.

2.7 - Comparison of the Building Codes :

The provisions of the building codes discussed in this chapter are compared in a tabular form in Table 2.1. The comparison is made between the analysis procedures outlined in the NBCC-85 and UBC-82, as well as the design procedures outlined in CAN3-A23.3-M84 and ACI-318.83. Provisions relevant to the building types and structural components, considered in this investigation are included in the comparison.

The comparison of the final designs of structural members are presented in chapter 6.

TABLE 2.1 GENERAL CODE COMPARISON

NBCC-85 and CAN3-A23.3-M84	UBC-82 and ACI-318.83
Analysis : **Design Seismic Base Shear :** V=v.S.K.I.F.W v-zonal velocity ratio $S=0.22/\sqrt{T}$-Seismic Response factor I-importance factor K-ductility factor F-foundation factor T=0.1N-fundamental period W-weight of the structure	 V=Z.I.K.C.S.W Z-zonal coefficient I-importance factor K-horizontal force factor $C=1/15\sqrt{T} \leqslant 0.12$ $S \geqslant 1.0$ $\quad$ $CS \leqslant 0.14$ T=0.1N-fundamental period W-weight of the structure
Vertical Distribution of Seismic Forces : Linear	 Linear
Design : **Girder Design :** **Geometric Constraints :** i) Length/Depth $\geqslant 4.0$ ii) Width/Depth $\geqslant 0.3$ iii) Width $\geqslant 250$ $<$ col. width +3/4 beam depth	 i) Length/Depth $\geqslant 4.0$ ii) Width/Depth $\geqslant 0.3$ iii) Width $\geqslant$ 10in. (254mm) $<$ col. width +3/4 beam depth
Maximum Factored Axial Comp. Force : $P_n \leqslant 0.1 A_g f'_c$ **Long. Reinforcement Ratio :** $\rho \geqslant 1.4/f_y$ (f_y in MPa) $\rho < 0.025$	 $P_n \leqslant 0.1 A_g f'_c$ $\rho \geqslant 200/f_y$ (f_y in Psi) $\rho < 0.025$

TABLE 2.1 GENERAL CODE COMPARISON (cont'd.)

NBCC-85 andCAN3-A23.3-M84	UBC-82 and ACI-318.83
Design for Shear:	
i) Capacity Reduction Factor $\phi = 0.85$	$\phi = 0.85$
ii) Design Shear Calculation based on : ultimate moments without capacity reduction factor & a steel stress of $1.25f_y$	same as CSA-M84
iii) Portion of Shear Carried by Concrete $V_c=0$; for ductile frame members conforming to clause 21.3. $V_c=0.2\lambda\phi_c\sqrt{f'_c}\,b_w d$ -for shear and flexure. $V_c=0.2\lambda\phi_c\sqrt{f'_c}\left(1-\frac{N_f}{N_r}\right)bw\,d$-for axial tension. if axial compressive force $>$ $0.10A_g f'_c$	$V_c=0$; if: $V_{seismic} \geq 1/2\ V_{design\ shear}$ and compressive design force $< 0.05f'_c A_g$
Column Design:	
Geometric Constraints:	
i) Axial Compressive Force $>0.10A_g f'_c$ ii) Shortest dimension>250mm iii) Min.Thickness/Max. Thick. >0.4.	i) Axial Compressive Force $>0.10A_g f'_c$ ii) Shortest dimension >12in.(305mm) Min. Thick./Max. Thick. >0.4.
ϕ for Compressive Strength:	
$\phi=0.70$	$\phi=0.50$ if Design Compressive Force $0.10A_g f'_c$ and special trans. reinforcement is not provided otherwise $\phi=0.70$

TABLE 2.1 GENERAL CODE COMPARISON (cont'd.)

NBCC-85 and CAN3-A23.3-M84	UBC-82 and ACI-318.83
Relative Flexural Strength of Columns :	
i) Σ Col.Strength>ΣBeam Strength at joint	i) Same as CAN3-A23.3-M84
ii) Otherwise, the member shall satisfy clause 21.8.1, and provided with transverse reinforcement over its full height.	ii) Otherwise, the member shall be provided with transverse reinforcement over their full height.
Design of Shear Reinforcement :	
Calculation of Design Shear based on: Moments due to formation of plastic hinges without capacity reduction factor.	Same as CAN3-A23.3-M84
Portion of Shear carried by Concrete :	
V_c =0, if member is detailed using Sec.21.3. $V_c \neq 0$ (See Clause 21.7.3.1)	V_c =0, if $V_{seismic} \geq 1/2\ V_{design}$ shear and compressive design force$<0.05f'_c\ A_g$
Capacity Reduction Fctor :	
ϕ=0.85	ϕ=0.85, if strength governed by flexure; ϕ=0.60, if strength governed by shear.
JOint Design :	
i) Design Shear based on column shear and shear due to yielding in beam reinforcement without capacity reduction factor and based on a steel stress of $1.25f_y$	i) Same as CAN3-A23.3-M84
ii) ϕ=0.85	ii) ϕ=0.85

TABLE 2.1 GENERAL CODE COMPARISON (cnot'd.)

NBCC-85 and CAN3-A23.3-M84	UBC-82 and ACI-318.83
iii) special lateral reinforcement of col. is continued up through the joint.	iii) Same as CAN3-A23.3-M84 iv) Allowable Shear Stress Lateral confined: normal weight aggregate concrete $20\sqrt{f'_c}A_j$; light weight agg. concrete $3/4(20\sqrt{f'_c}A_j)$ Lat. uncon.: $15\sqrt{f'_c}A_j$; $3/4(15\sqrt{f'_c}A_j)$
Shear Wall Design :	
Distributed Steel : Vertical : 0.0025 A_g Horizontal : 0.0025 A_g	Vertical : 0.0025 A_g Horizontal: 0.0025 A_g
Maximum Reinforcement Spacing: Plastic hinge regions: ≤300mm Other regions: ≤450mm	spacing ≤ 18in. (457mm)
Concentrated Reinforcement: provide boundary elements at each end of wall: $A_s min = 0.001 b_w l_w$ $A_s min = 0.002 b_w l_w$ (plastic hinge region)	provide boundary elements if: extreme fiber stress > $0.2f'_c$; Design boundary elements as columns.
Concentrated Reinforcement Confinement: Spacing: i) plastic hinge regions: smallest of: i) 6 longitudinal bar diameter ii) 24 tie diameter iii) 1/2 wall thickness iv) as required by Clause 21.5.7, if applicable. ii) Other regions as a column.	confinement as a column.

CHAPTER 3

PROPERTIES OF STRUCTURES CONSIDERED

3.1 - General :

Two reinforced concrete buildings were considered for analysis and design. The first building consisted of ductile moment resisting frames in transverse and longitudinal directions and represented a flexible structure. The second building consisted of frames and shear walls and represented a relatively stiff structure.

In selecting the structures, consideration was given to the effect of structural configuration and the flexibility of the lateral load resisting system.

The selection of these two types of structures permitted the comparison of code provisions in terms of different structural components. Columns, beams and beam-column joints selected from the moment resisting frame, and shear walls and coupling beams selected from the frame-shear wall structure, were used in the comparison.

3.2 - Frame building :

The frame structure considered for the study is illustrated in Fig.3.1. It is a 20-storey office building with 6 bays in the east-west direction at 8.0m, and 3 bays in the north-south direction at 8.0m. Column dimensions are tapered throughout the height, whereas beam dimensions are held constant. The floor to floor height is 4.0m except for the first storey where the height is increased to 6.0m due to architectural considerations. The structure is assumed to be fully fixed at the ground level. The locations chosen for design purposes were: Vancouver, British Columbia and Seattle, Washington. The properties of the members and materials are listed in Table 3.1.

3.3 - Frame-Shear Wall building :

The second structure selected for this study consists of a 20-storey reinforced concrete office building. Fig.3.2 shows a typical floor plan and the bay dimensions. It can be seen from the figure that the majority of the lateral loads are resisted by the end and core shear walls. The frames between the shear walls carry the majority of the gravity loads and help resist a small percentage of the lateral load. Member and material properties used are given in Table 3.1.

3.4 - Design Loads :

The basic dead and live loads prescribed by the NBCC 85 are similar to those prescribed by the UBC 82. In this study the same dead and live loads as prescribed by the two codes are used for the analysis of the structures. The interior transverse frame (column line 4 in Fig.3.1) and the exterior transverse shear wall (column line 1 in Fig.3.2) were used for the study. The dead and live loads used are shown in Table 3.2. In order for the effects of the lateral load to be pronounced the structure was designed for a high seismic risk zone. Vancouver, British Columbia was chosen as the design location for the NBCC . Vancouver is situated in Canada's highest seismic risk zone by the NBC standards. The comparable location for the UBC was chosen as Seattle, Washington. Although Seattle is not in the highest seismic zone according to the American code, its location just across the border from Vancouver does place it in a category comparable to the NBCC risk zone of Vancouver. The resulting zone classifications are; NBCC : zone 4, UBC : zone 3 .

The computation of seismic design loads are given in Appendix A in detail .

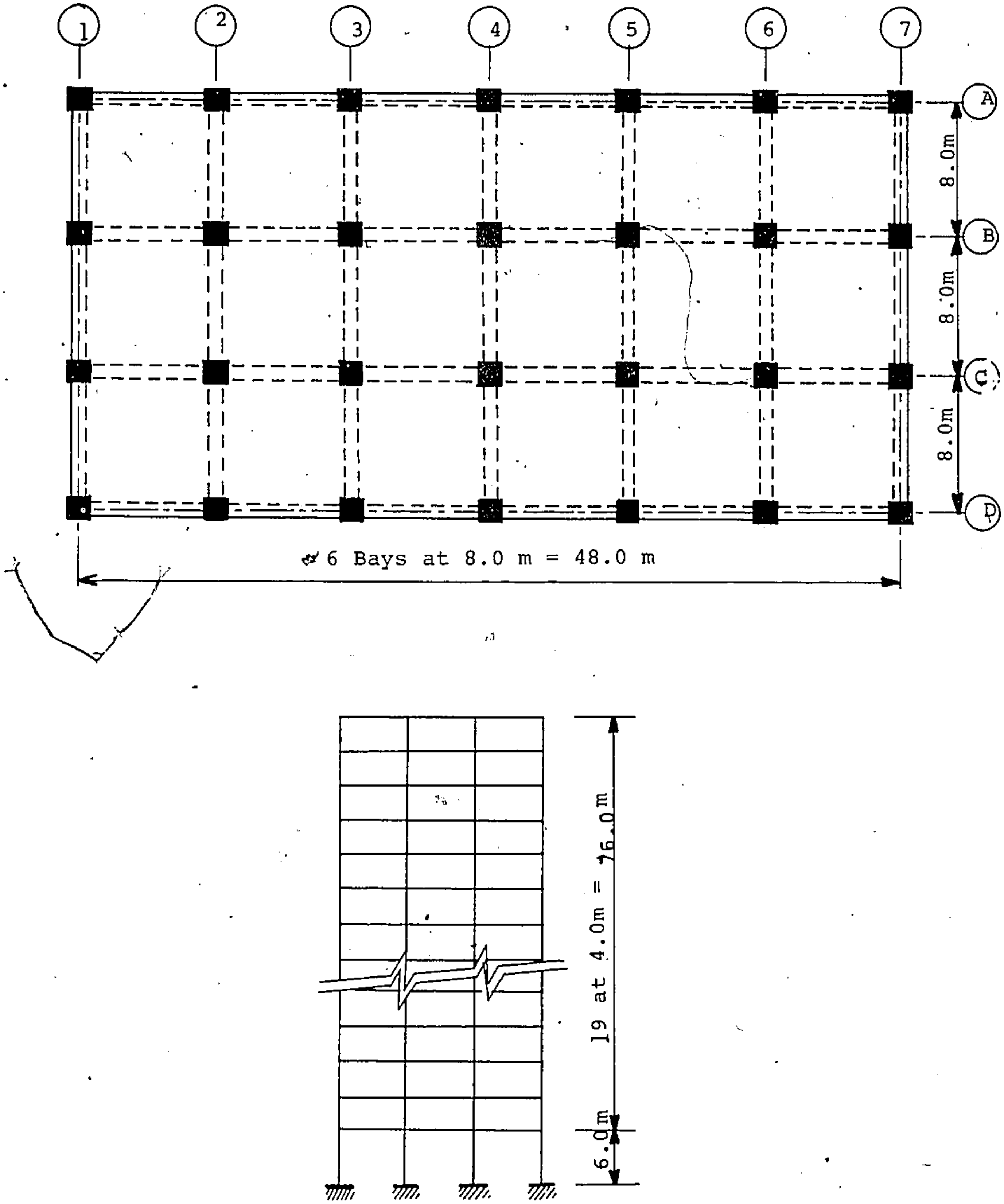


Figure 3.1 Plan and Elevation of Frame Building

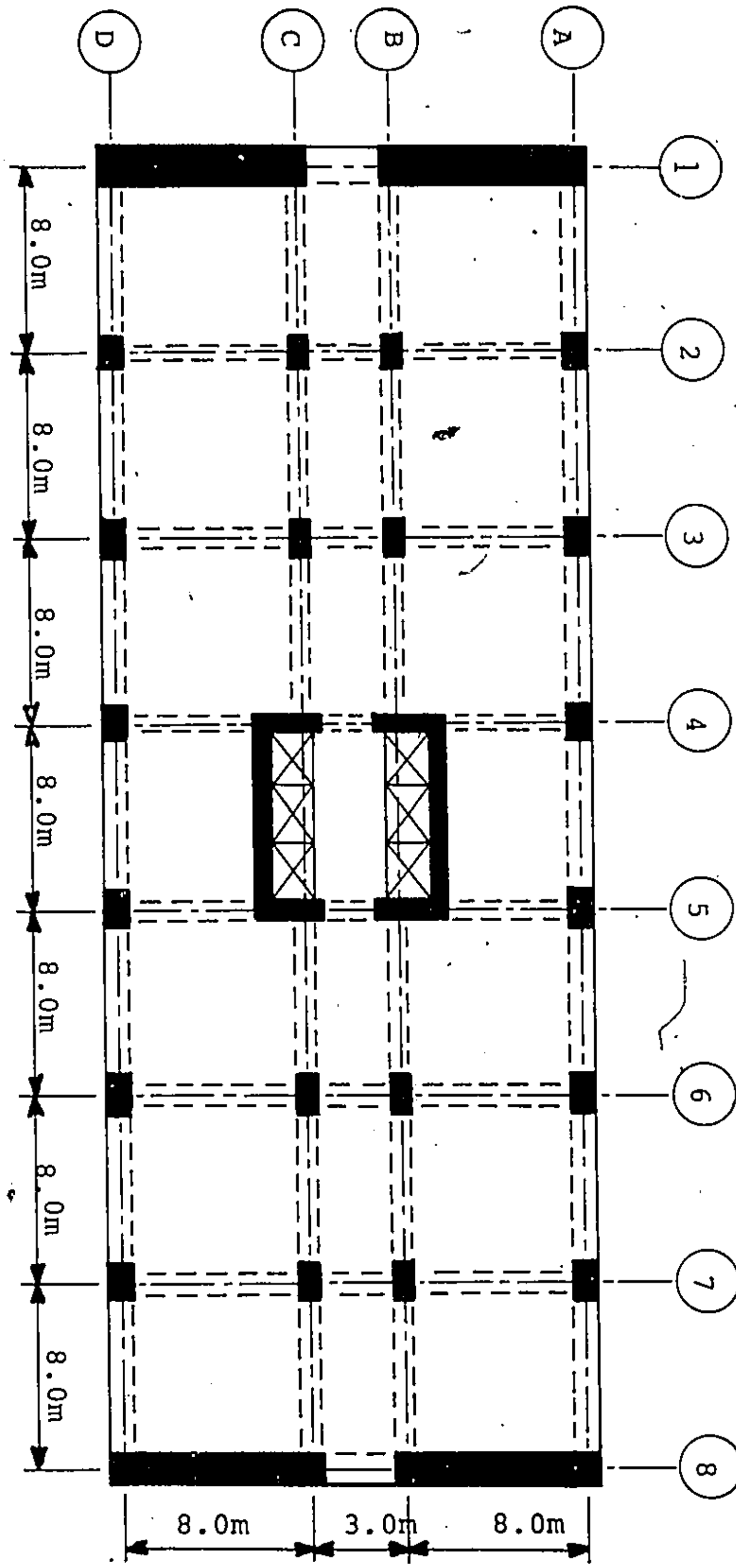


Figure 3.2 Plan of Shear Wall Building .

TABLE3.1 a) MEMBER PROPERTIES

FRAME STRUCTURE Members *	b (mm)	h (mm)
Beams	500	750
Columns: -ground to 6th floor -7th to 13th floor -14th to 20th floor	 1000 750 500	 1000 750 500
SHEAR WALL STRUCTURE Members*	b (mm)	h (mm)
Girders	350	500
Spandrels	300	500
Coupling Beams: -1st to 6th floor -7th to14th = -15th to 20 =	 575 400 300	 600 600 600
Columns: -ground to 10th floor -10th to 20th =	 350 250	 500 400
Walls: -1st to 6th floor -7th to 14th = -15th to 20th =	 575 400 300	

b) MATERIAL PROPERTIES

Material	Strength (Mpa)	Density (Kg/m³)	E (Mpa)
Steel	400**	+	200000
Concrete	30***	2400	27386

*Note Slabs are 200mm thick
** yield strength of steel
*** 28 day compressive strength
+ Assumed to be negligible

TABLE 3.2 SPECIFIED DESIGN LOADS

LIVE LOADS :	
-Typical floor	2.4 KN/m^2
-Roof	1.0 KN/m^2
-Snow Load	1.9 KN/m^2
SUPERIMPOSED DEAD LOADS* :	
-Partitions	0.65 KN/m^2
-Ceilings	0.35 KN/m^2
-Roofing	0.35 KN/m^2
EARTHQUAKE AND WIND LOADS** :	
As per NBCC-85 [4] and UBC-82 [5] for Vancouver B. C., and Seattle, Washington respectively.	

* Dead loads due to structure weight is computed using normal density concrete.

** For further details see Appendix A.

CHAPTER 4

ANALYSIS AND DESIGN

4.1 - General :

Analysis and design of structures were performed using the appropriate clauses of the building codes. Static analyses of structures under lateral and gravity loads were conducted first. The analyses results were used to find design forces. Selected structural members and subassemblies were proportioned and detailed following the seismic provisions of the selected building codes. Details of these procedures are presented in this chapter.

4.2 - Static Load Analysis :

The static analyses of the structures were accomplished using computer program FR4C [6]. This program analyzes plane frames consisting of prismatic one-dimensional members. The stiffness method of frame analysis is used. The full set of joint equilibrium equations are processed by a modified Gaussian elimination and back substitution procedures. The necessary input data for this program are the modulus of elasticity E, area A, and moment of inertia I for each member, as well as structure geometry and loading.

The frames in the north-south direction of the buildings were considered for analysis. The static analysis was performed separately under lateral loads and gravity loads.

4.2.1 - Equivalent Lateral Load Analysis :

Three dimensional structures were idealized into two dimensional frames for the purpose of analysis. This was accomplished by lumping all identical frames into one frame. In the case of the frame structure, all five of the interior frames were combined to form one frame, and the exterior frames were combined to form another frame. The member properties of the lumped structure were obtained by multiplying individual member properties by the number of individual frames lumped.

For the purpose of lateral load analysis, the slabs were assumed to act as rigid diaphragms, distributing shear to the individual frames in proportion to their stiffnesses. The two lumped frames were connected together by means of rigid links to ensure equal horizontal displacement. The rigid links were connected to the frames by hinges to be able to transmit the lateral loads and not the moments. Figures 4.1 and 4.2 illustrate the lumped models used for lateral load analyses of the frame and shear wall structures respectively .

Each frame member was modelled as a line element. However, finite widths of members were considered. Member properties within the clear span length were assigned to the line elements. Member ends, integral with the adjoining members, were assigned infinite stiffness. This is shown in figure 4.3.

The details of the relevant building code provisions are discussed in sections 2.3 and 2.4.

The details of the lateral load calculations are given in Appendix A. A summary of the computed earthquake loads is provided in Table 4.1. The resulting shear force envelopes, along the height of each structure, is shown in Figs. 4.4 and 4.5. Figures 4.7 through 4.13 illustrate lateral load analysis results. The analysis result, for the members considered for design, are tabulated in Table 4.2 and 4.3 for the frame and shear wall structures respectively.

4.2.2 - Gravity Load Analysis :

The model used for the gravity load analyses is shown in figure 4.6. The model simulates a typical floor frame. This was used for both the exterior and interior frames, with appropriate loads and member properties.

The frames were analyzed using pattern loading. Unfactored dead and live loads were used in the analyses. Table 4.4 present the governing values of the analysis results for an intermediate floor.

4.3 - Design Forces :

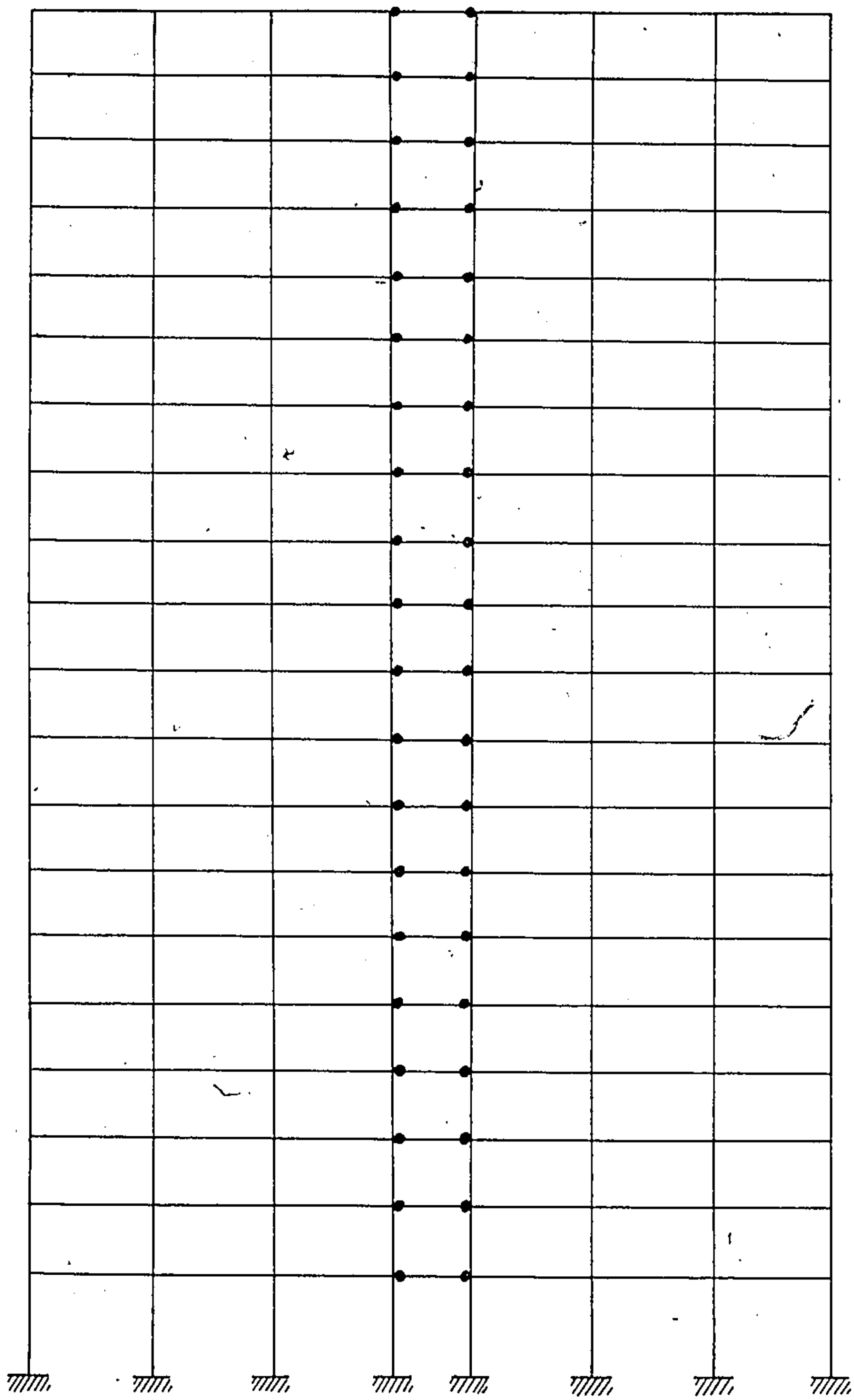
The buildings described in chapter 3 were analyzed to arrive at member end forces. Load factors and load combination factors employed by the two code procedures were different. The load combinations investigated for the two code procedures are given in Table 4.5. The coefficients associated with different load combinations and the load factors are prescribed by the respective codes. The unfactored loads for dead load (D), live load (L), and earthquake load (E) are obtained from Tables 4.2 through 4.4. Factored design quantities are shown in Tables 4.6 and 4.7 for the frame and shear wall structures respectively.

4.4 - Proportioning and Detailing Members :

Typical members of the two structures considered in this investigation were selected for proportioning and detailing. These members were selected from the critical regions of the structures. First storey exterior and interior beams, columns and beam-column joints of an interior frame of the frame structure were designed first. First storey shear walls and coupling beams of the shear wall structure were designed next.

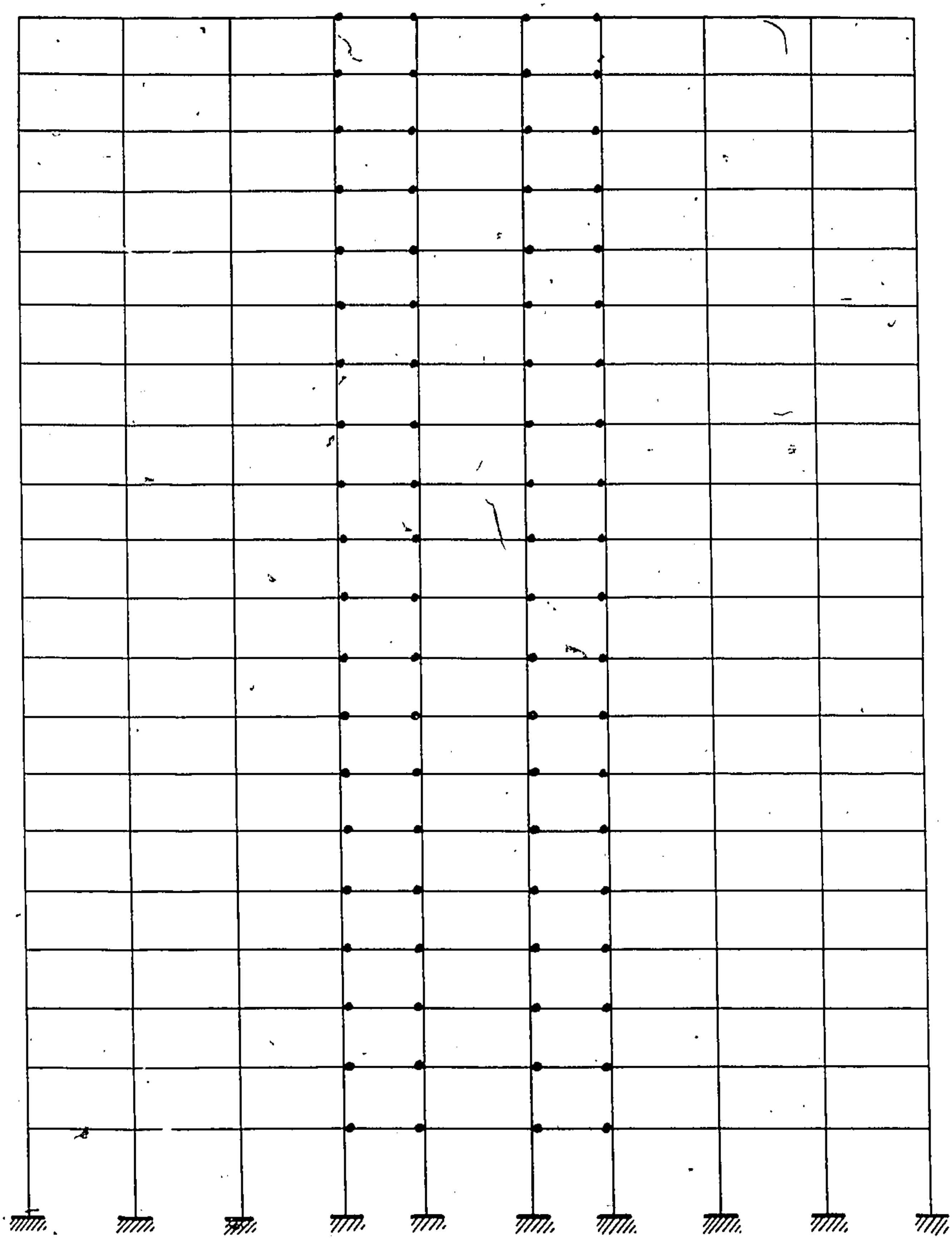
Preliminary member sizes used in the analyses were used in design. The amount and detailing of longitudinal and transverse reinforcements were determined using the seismic provisions of two design codes. The building codes considered in design were CAN3-A23.3-M84 and ACI-318.83. Each structural component was designed twice using the two building codes . Figures 4.14 through 4.26 illustrate the final designs and reinforcement detailings for each structural component considered. The comparison of the final designs between those designed following the Canadian and American practice is presented in Chapter 6.

Figure 4.1 FRAME BUILDING MODEL



MODEL FOR COMPUTER ANALYSIS

Figure 4.2 SHEAR WALL BUILDING MODEL



MODEL FOR COMPUTER ANALYSIS

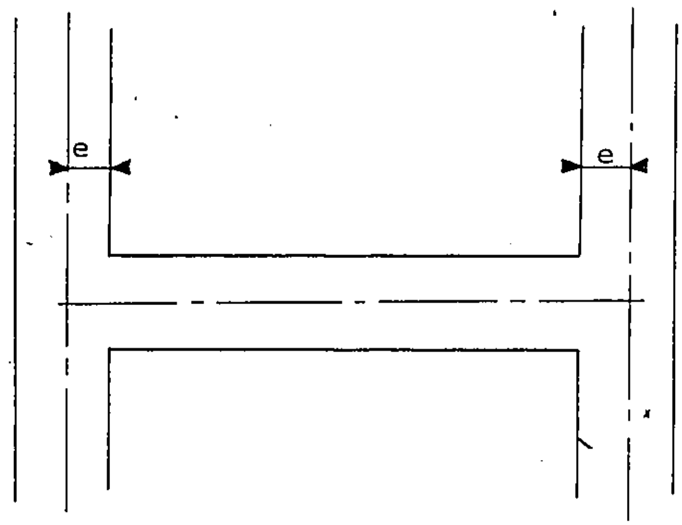
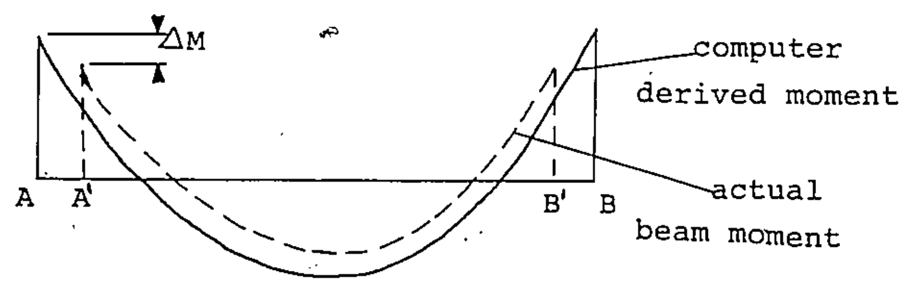


Figure4.3 EFFECT OF END ECCENTRICITY ON BENDING MOMENT



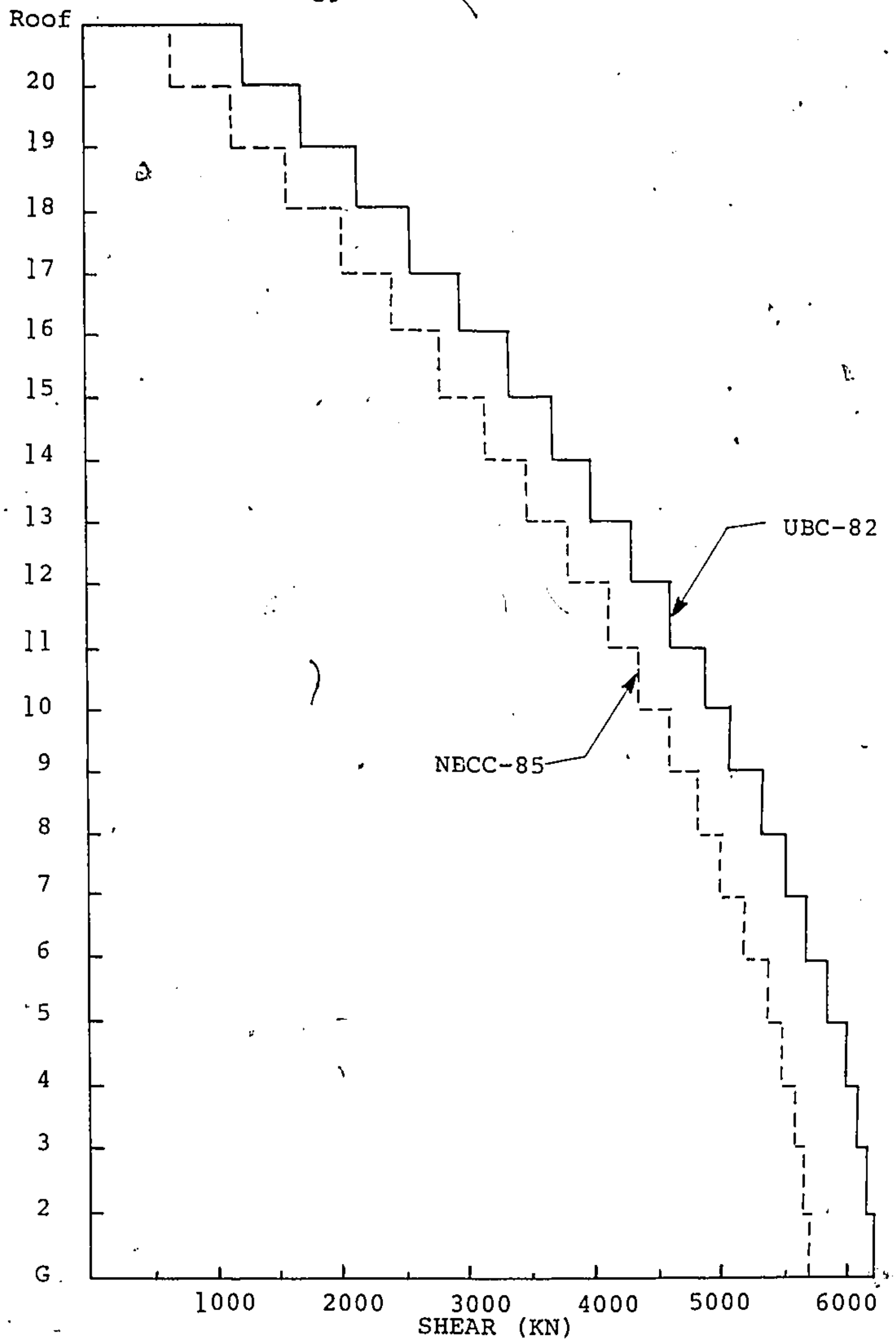


Figure 4.4 SHEAR FORCE ENVELOPES (FRAME STRUCTURE)

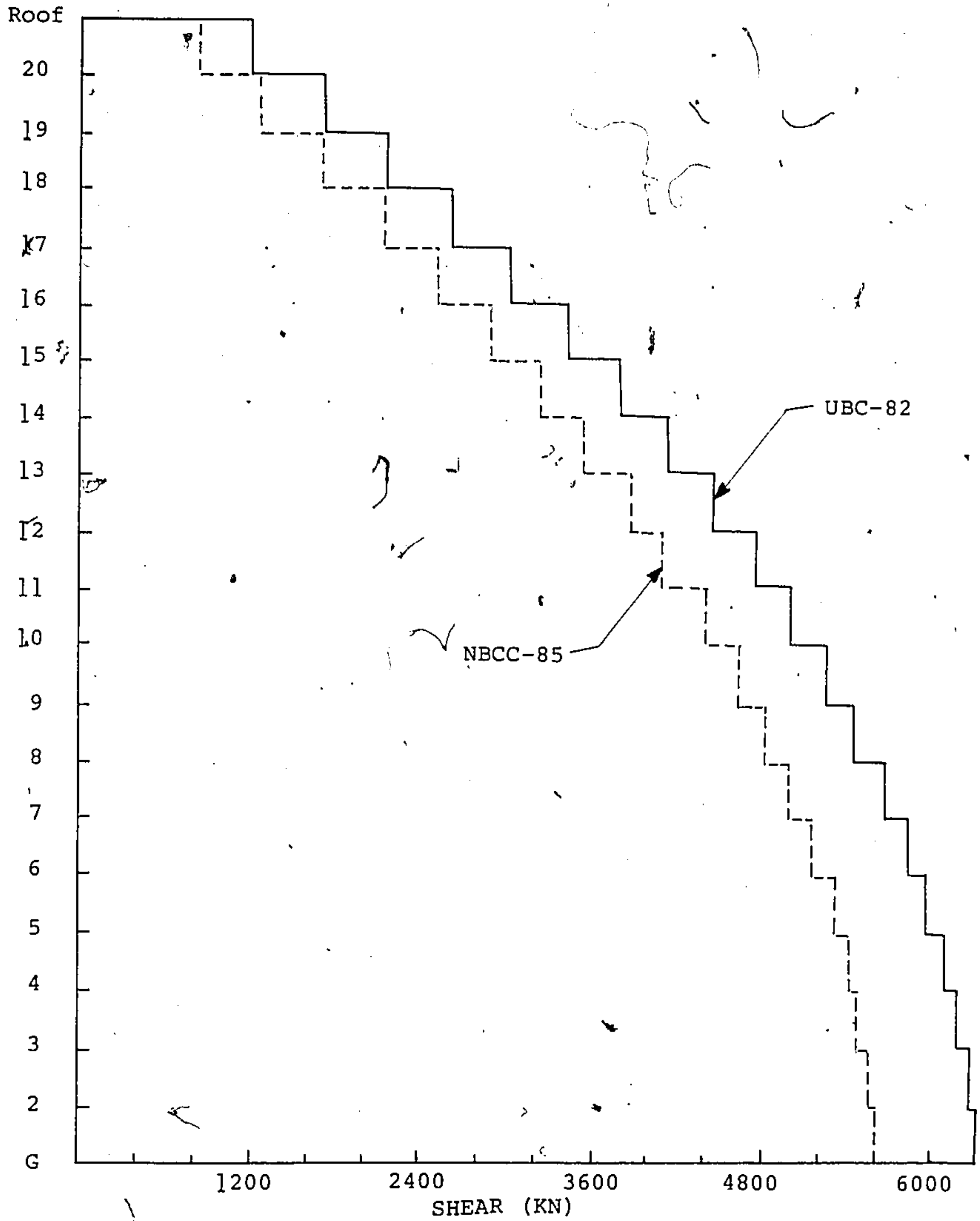


Figure 4.5 SHEAR FORCE ENVELOPES (SHEAR WALL STRUCTURE)

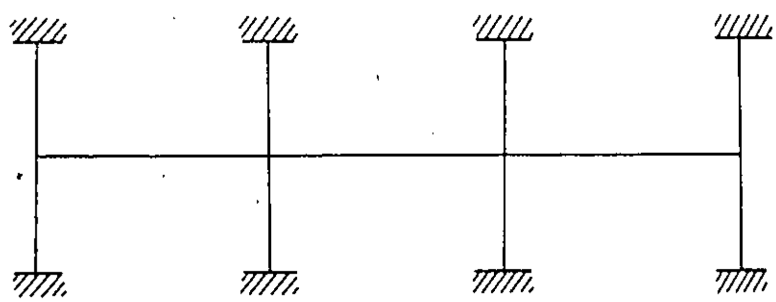


Figure 4.6 TYPICAL FLOOR FRAME used in gravity load analysis

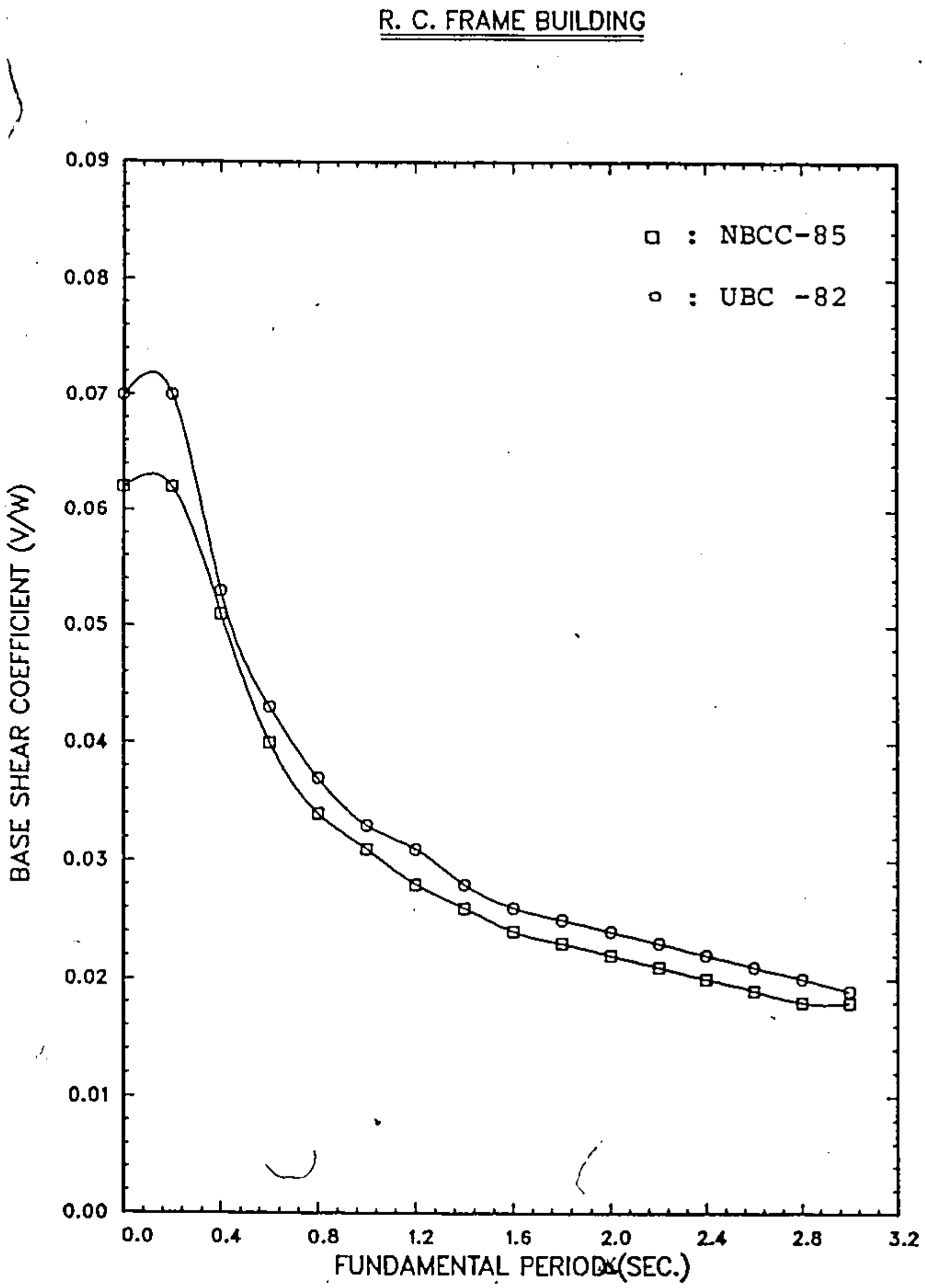


Fig.4.7 VARIATION OF BASE SHEAR COEFFICIENT WITH THE FUNDAMENTAL PERIOD OF THE BUILDING

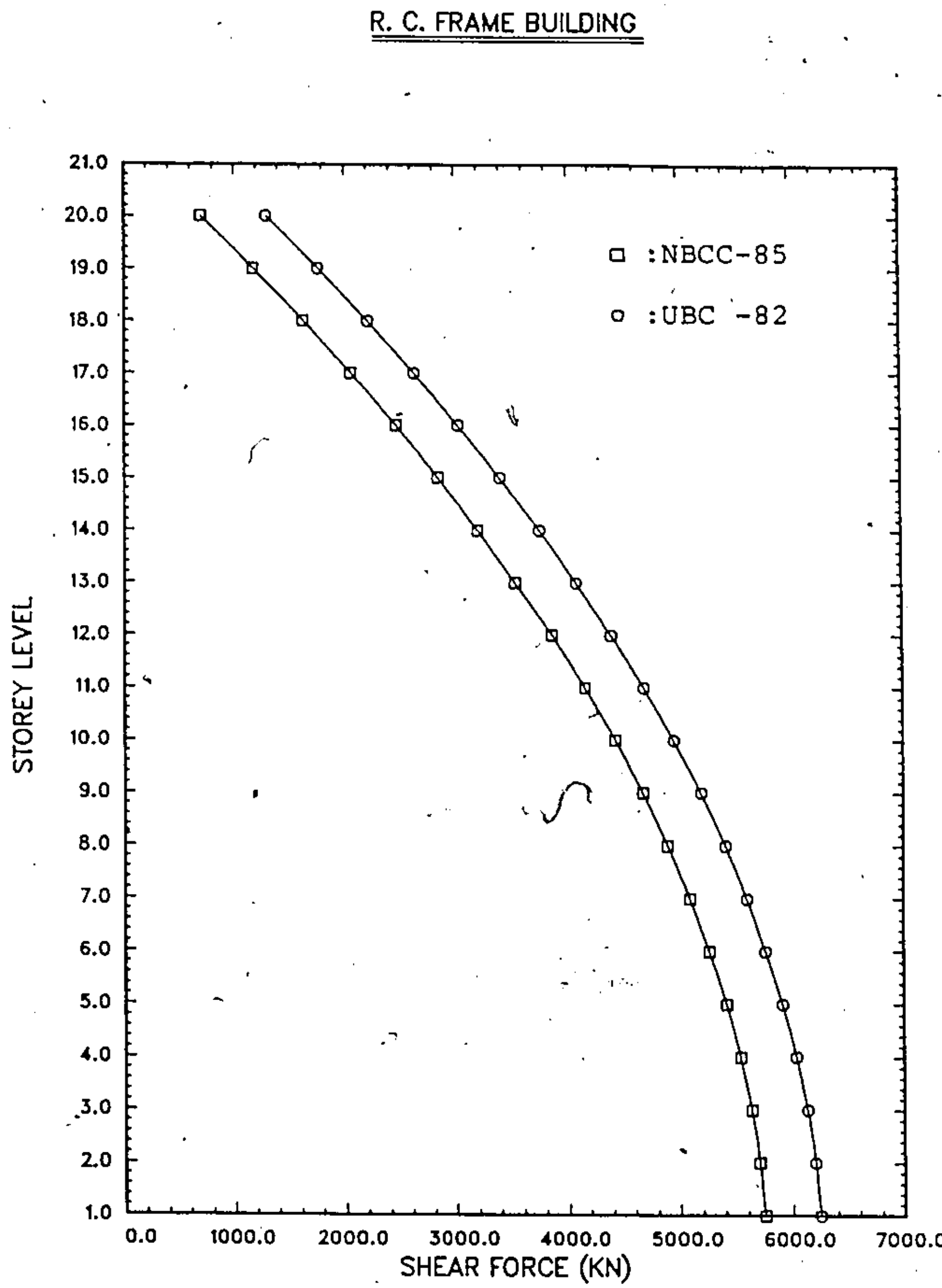


Fig. 4. 8 STOREY SHEARS FOR THE FRAME BUILDING

R. C. FRAME BUILDING (TAPERED COLUMNS)

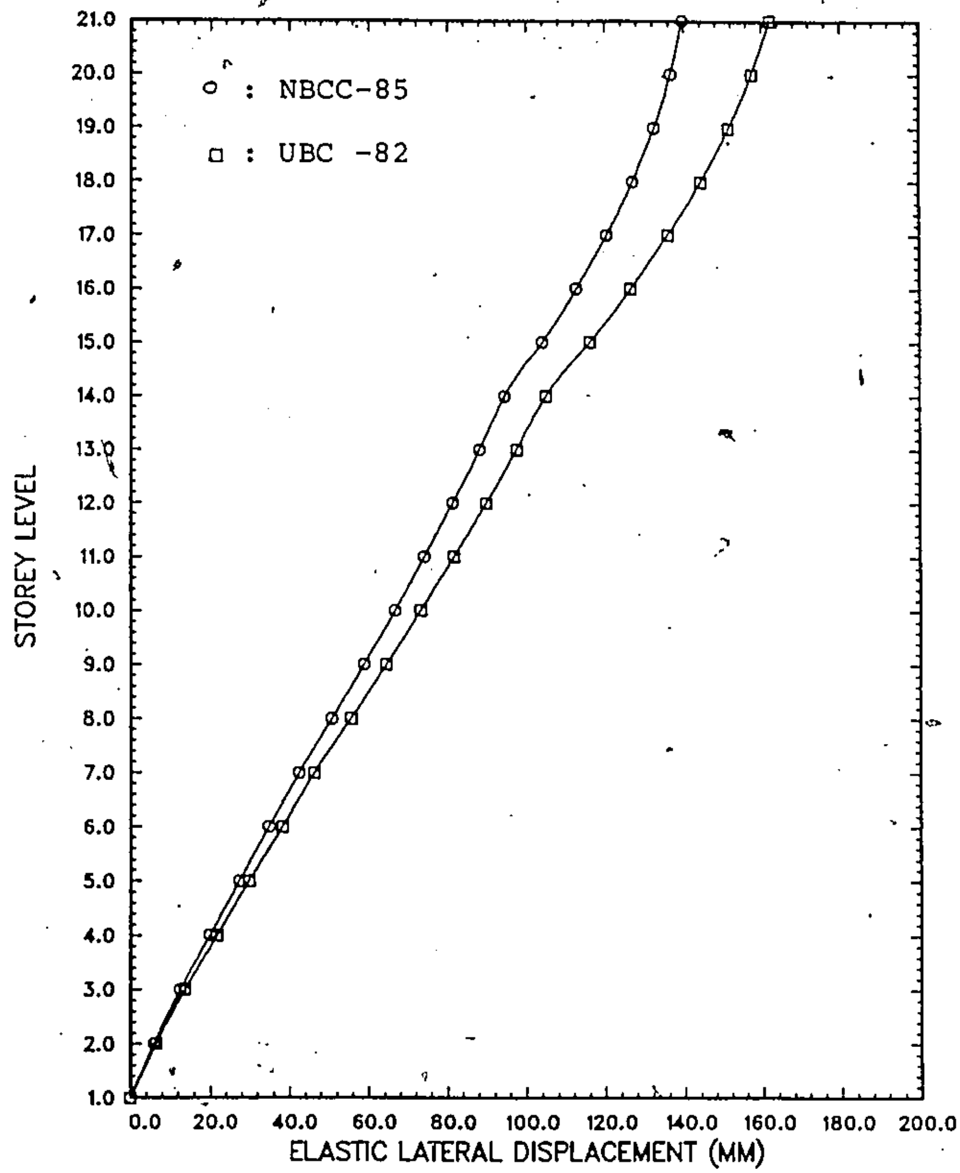


Fig.4.9 ELASTIC LATERAL DISPLACEMENTS DUE TO EARTHQUAKE LOADING

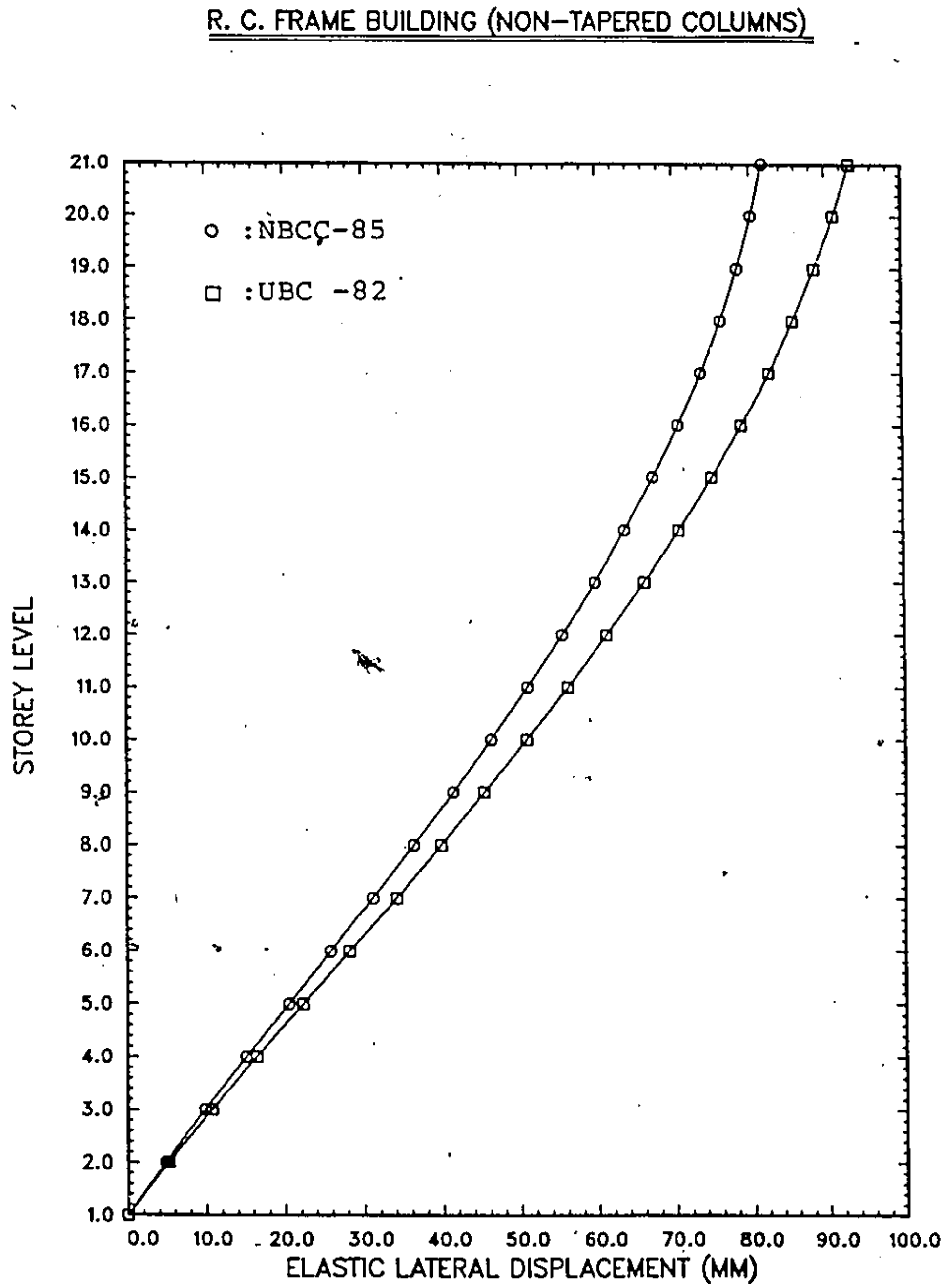


Fig.4.10 ELASTIC LATERAL DISPALCEMENTS DUE TO EARTHQUAKE LOADING

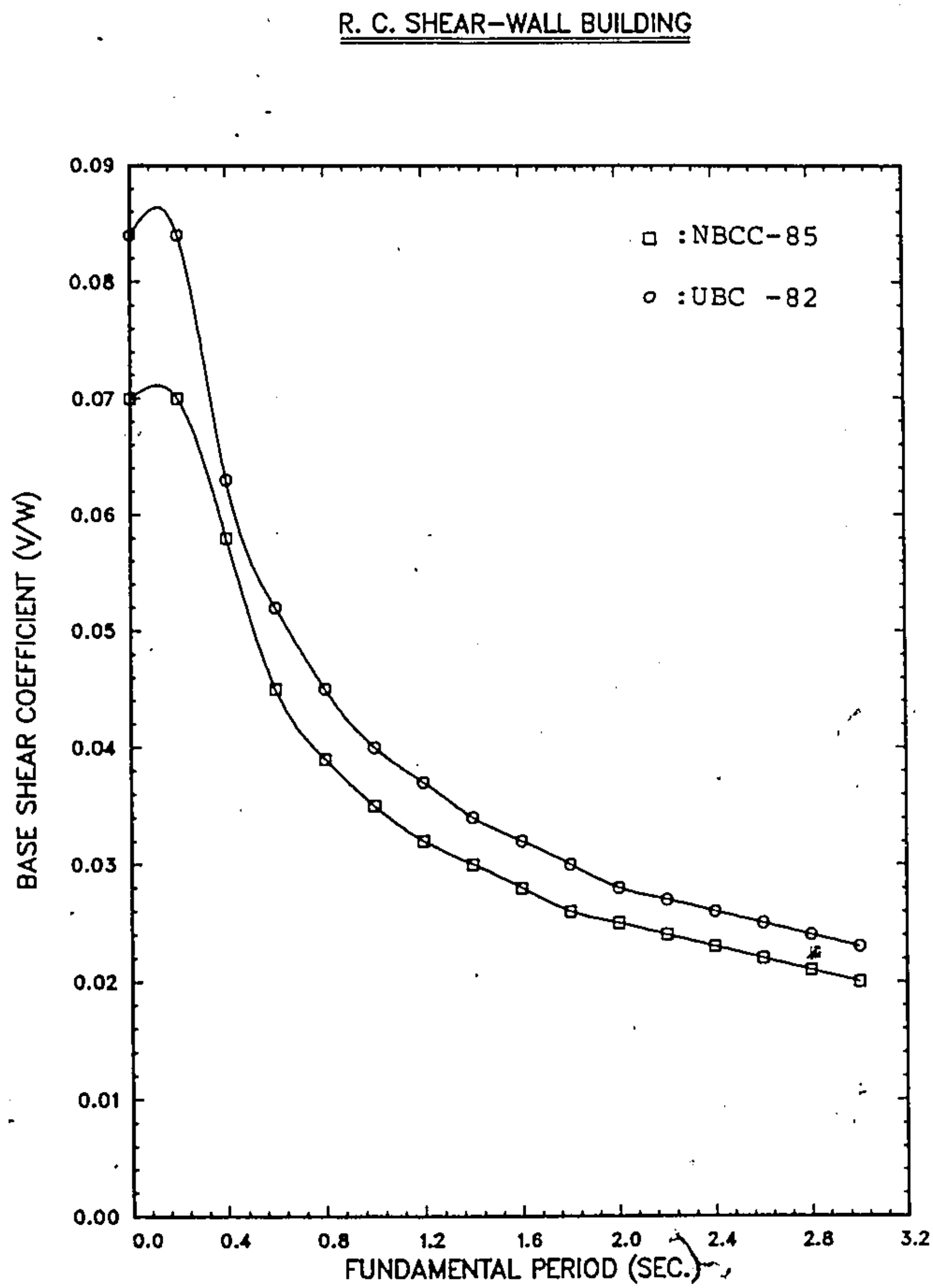


Fig.4.11 VARIATION OF BASE SHEAR COEFFICIENT WITH THE FUNDAMENTAL PERIOD OF THE BUILDING

R. C. SHEAR-WALL BUILDING

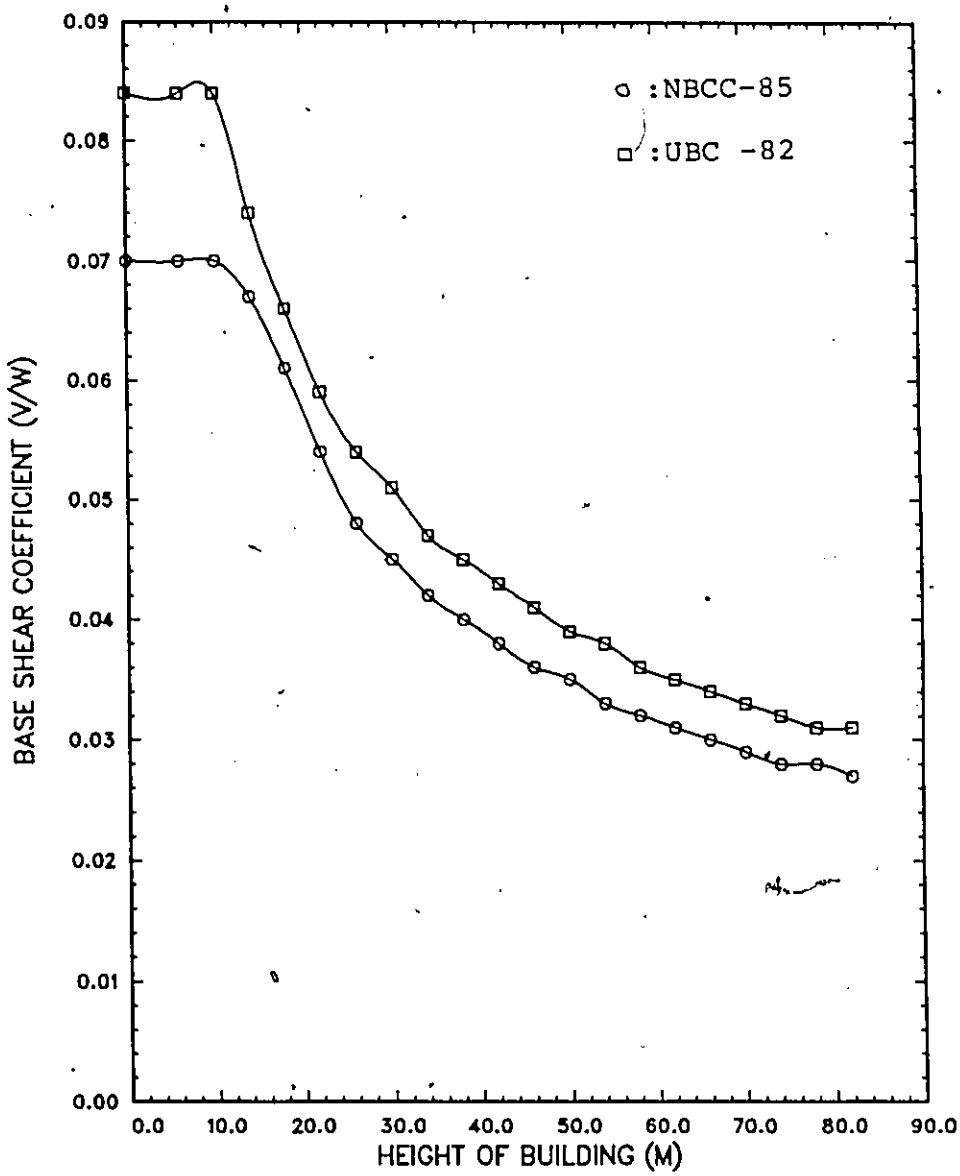


Fig.4.12 VARIATION OF BASE SHEAR COEFFICIENT WITH THE HEIGHT OF THE BUILDING

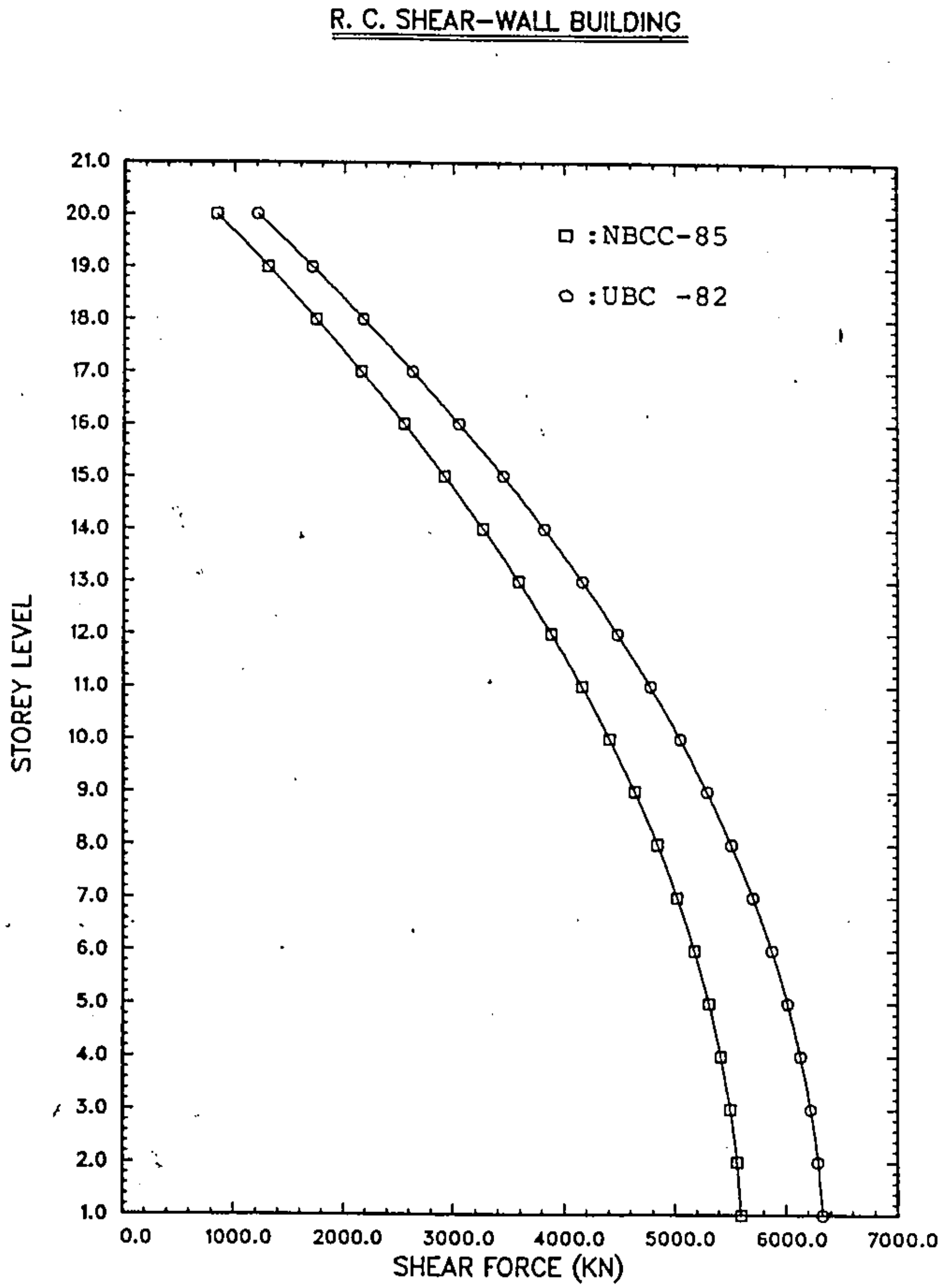


Fig. 4.13 STOREY SHEARS FOR THE SHEAR-WALL BUILDING

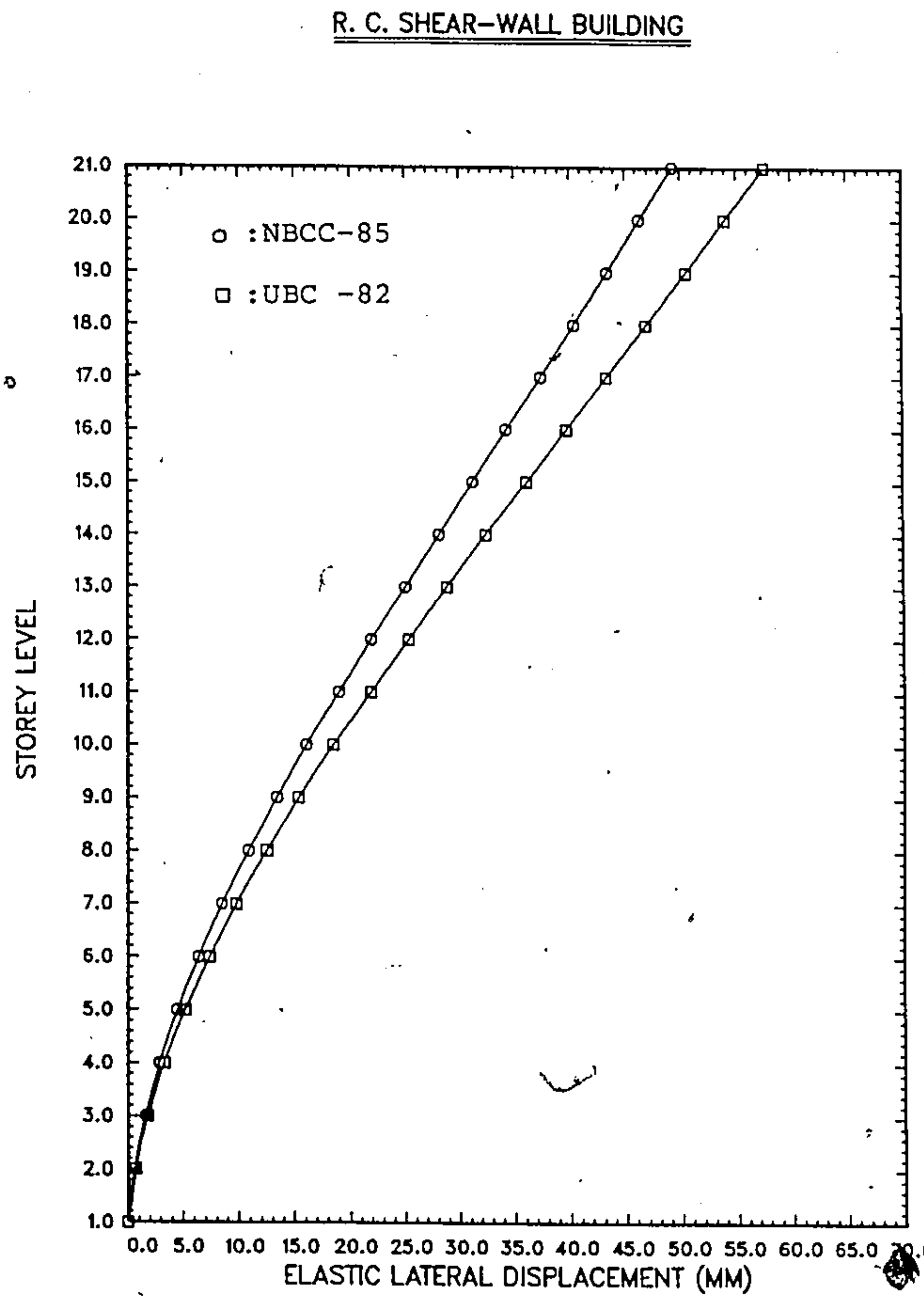


Fig.4.14 ELASTIC LATERAL DISPLACEMENTS DUE TO EARTH-QUAKE LOADING

Figure 4.1 5 FLEXURAL MEMBER REINFORCEMENT (CAN3-A23.3-M84)

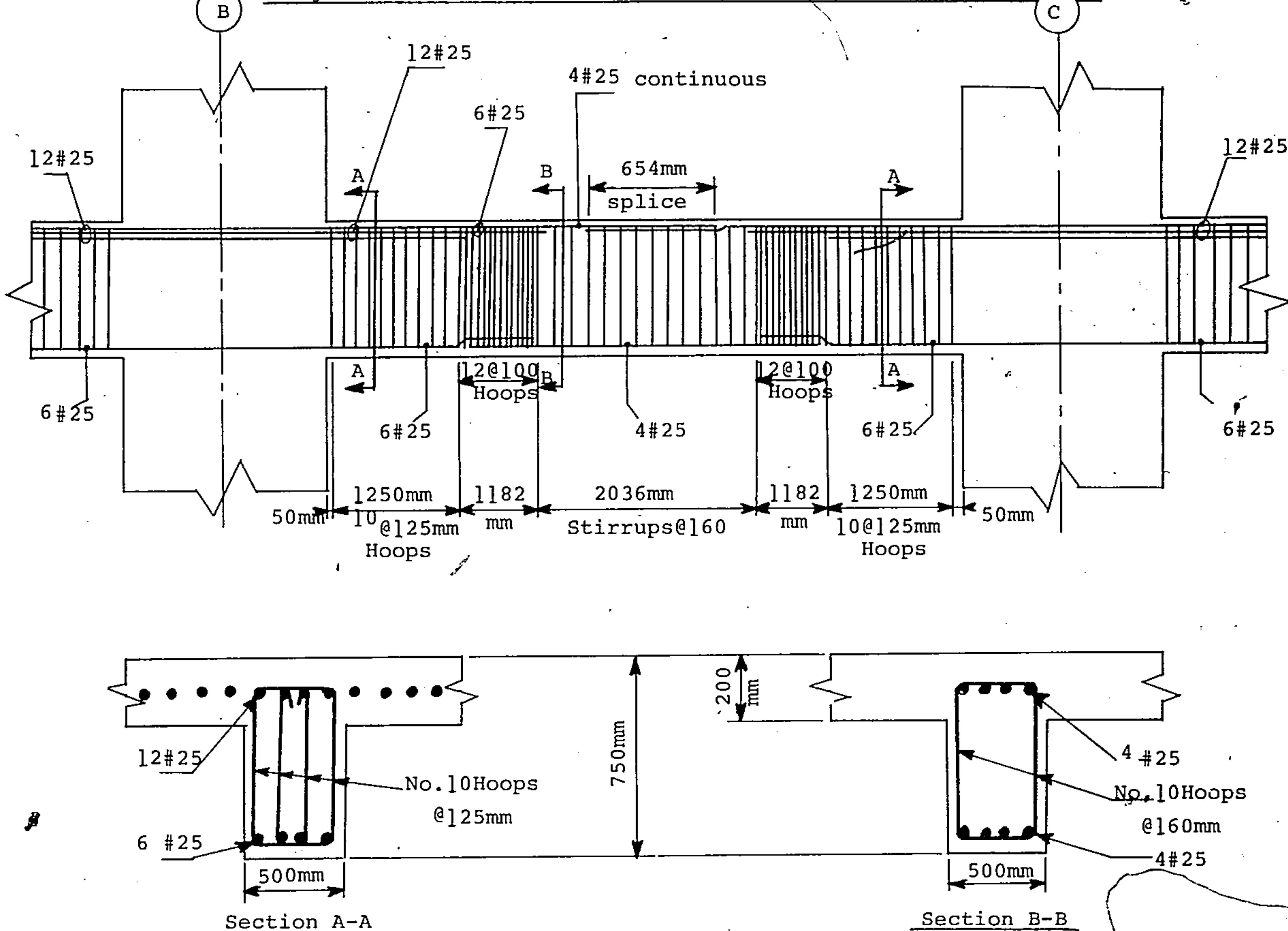
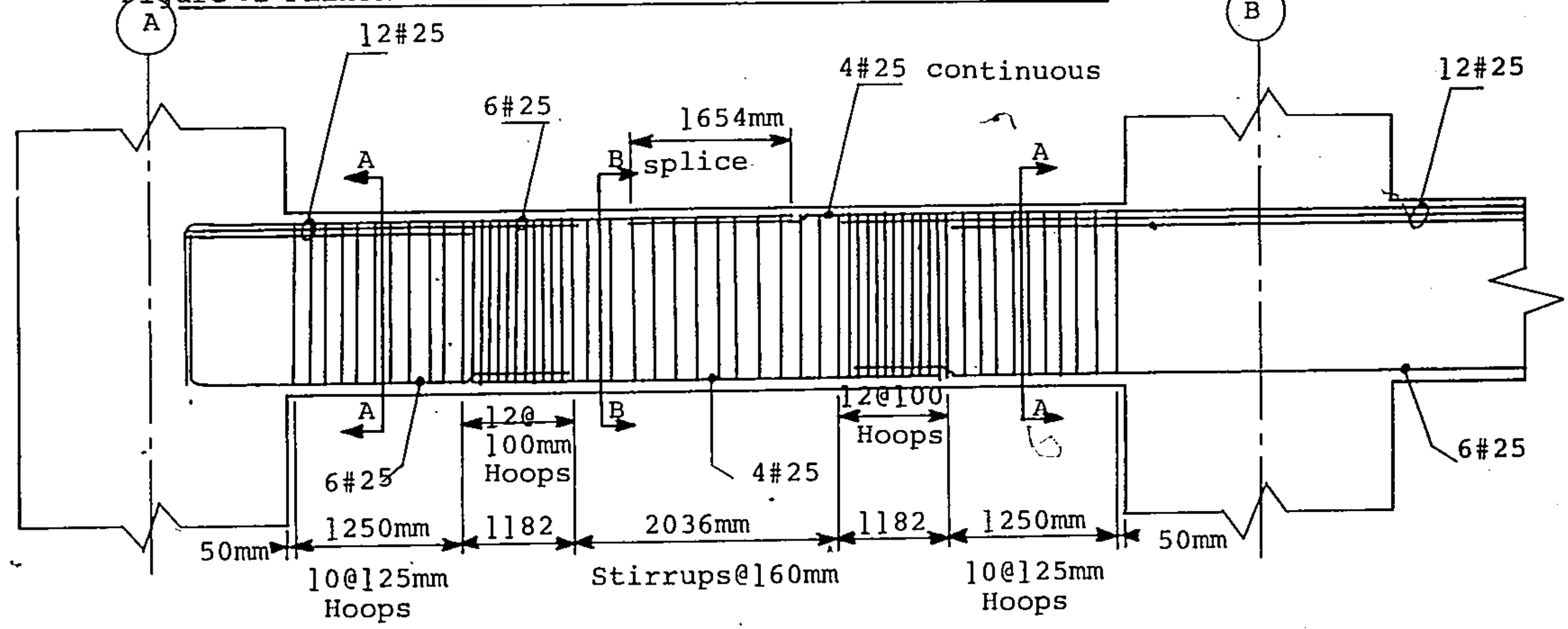


Figure 4.16 FLEXURAL MEMBER REINFORCEMENT (CAN 3-A23.3-M84)



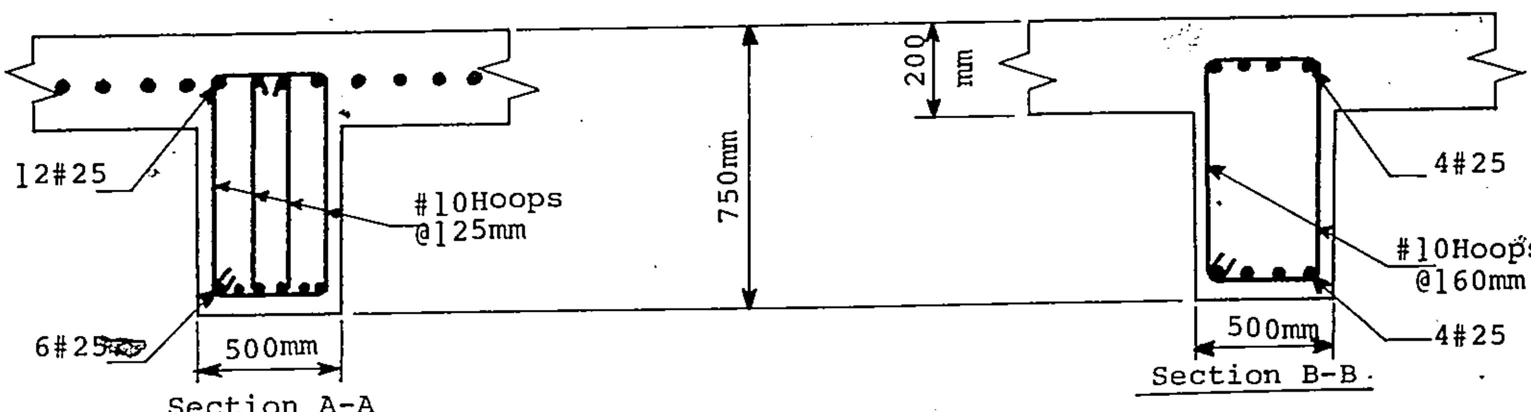


Figure 4.17 FLEXURAL MEMBER REINFORCEMENT (ACI318.83)

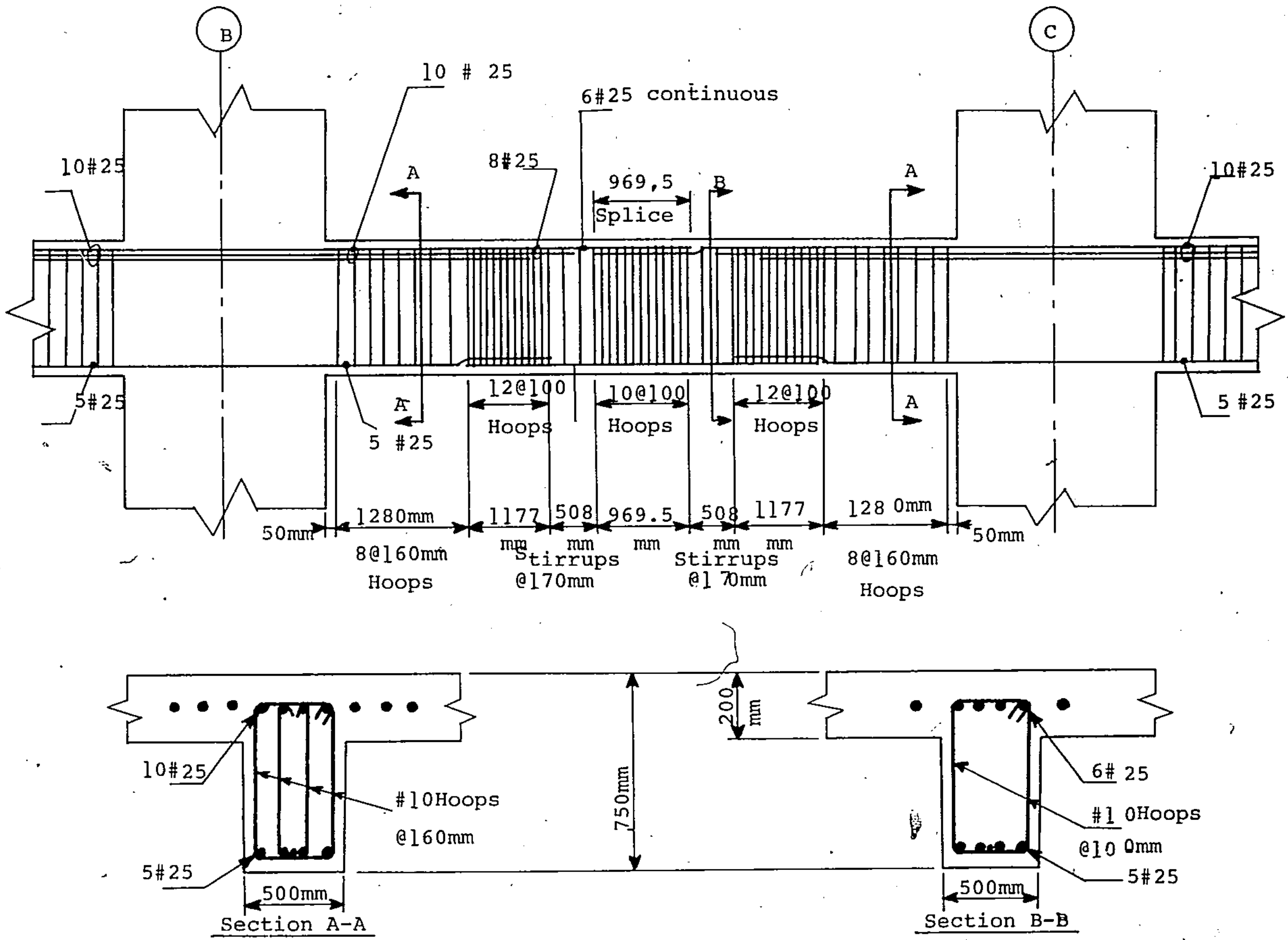


Figure 4.18 FLEXURAL MEMBER REINFORCEMENT (ACI 318-83)

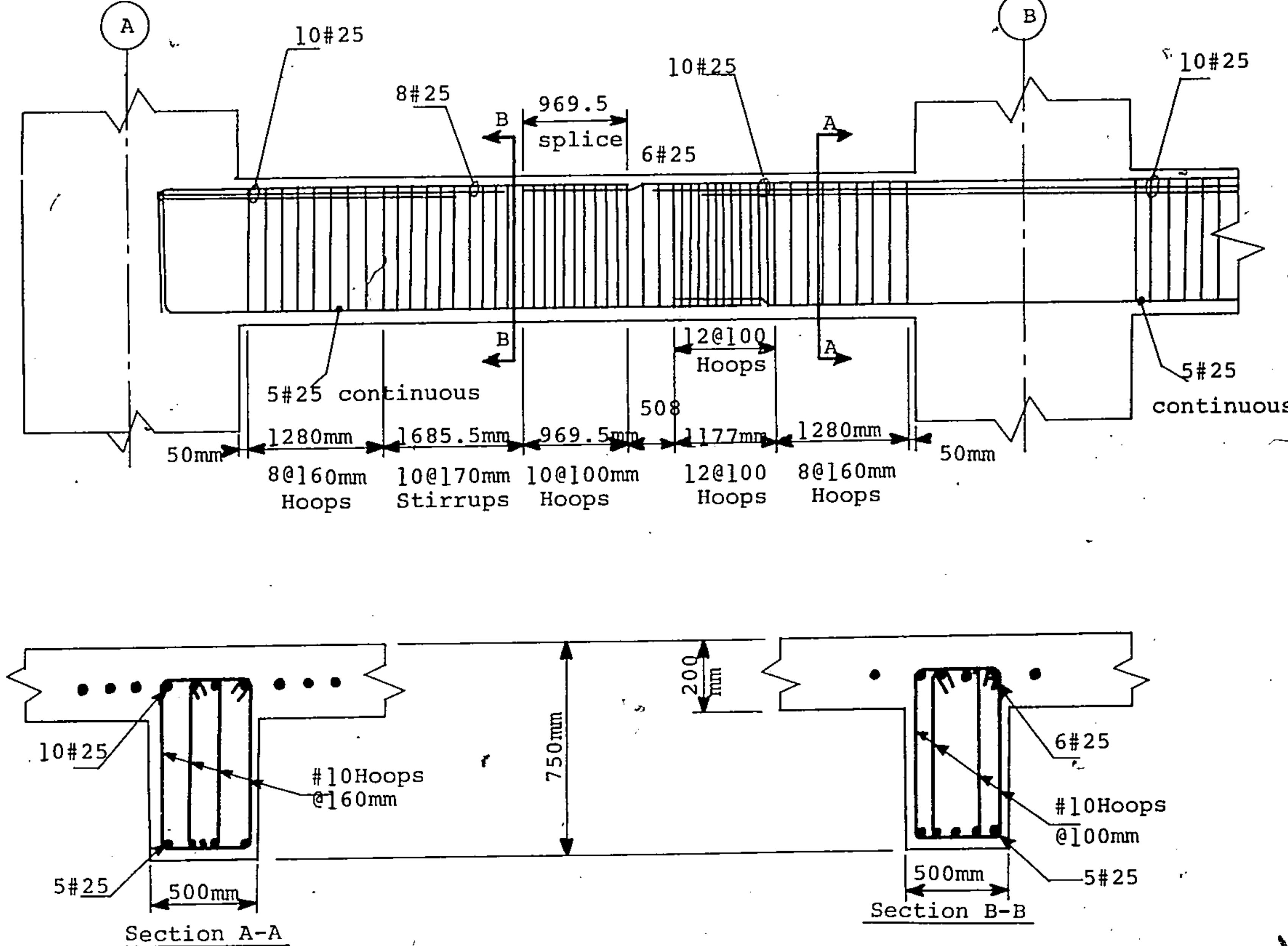
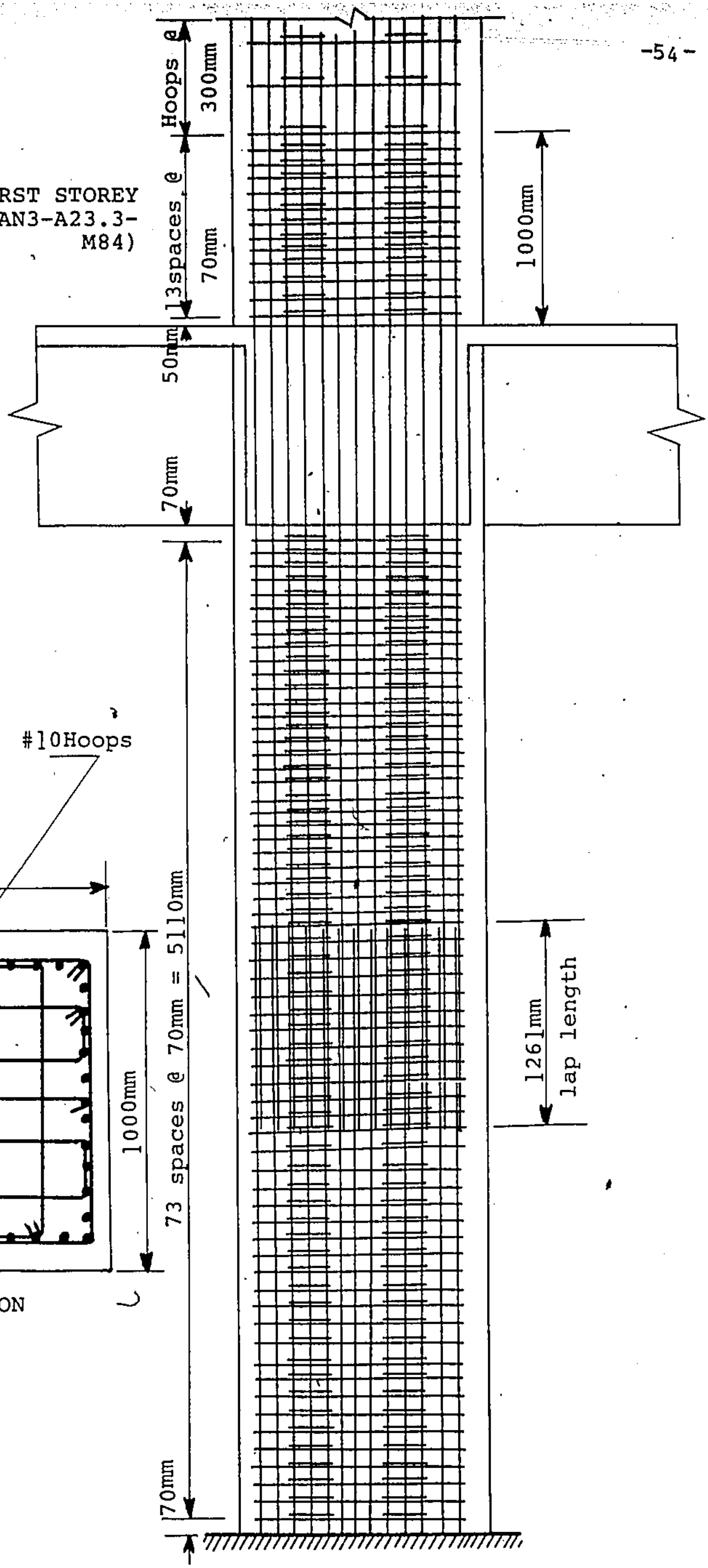
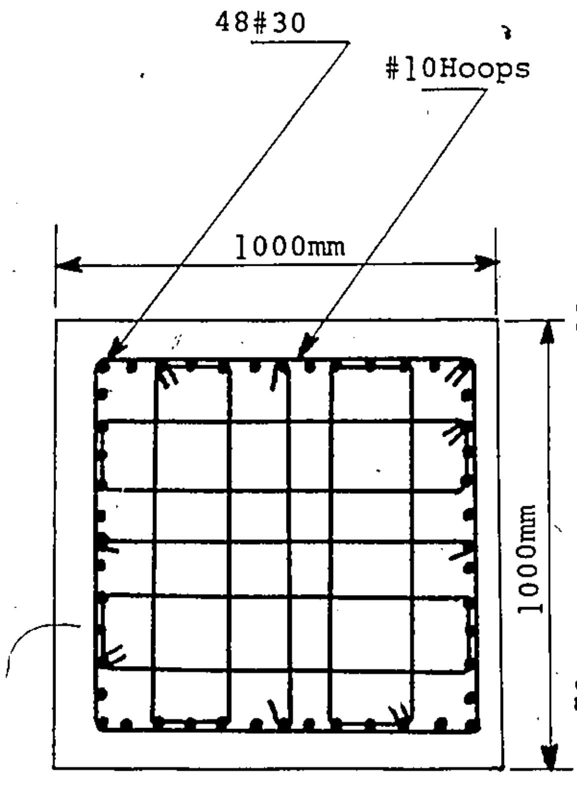


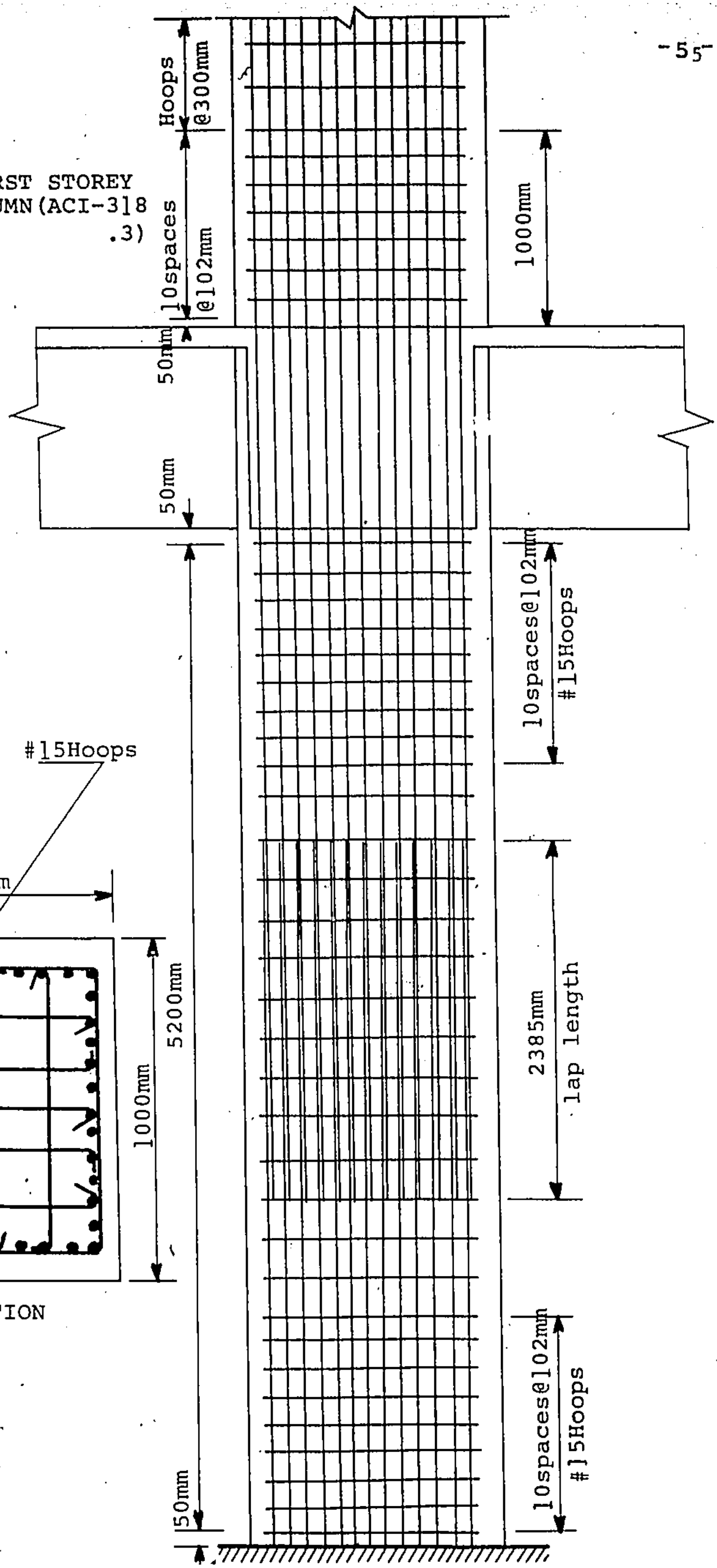
Figure 4.19 FIRST STOREY INTERIOR COLUMN (CAN3-A23.3-M84)

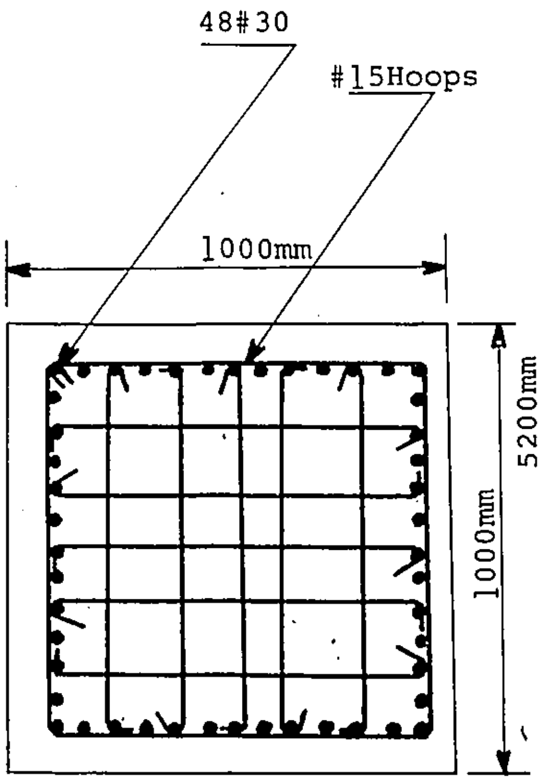




CROSS-SECTION

Figure4.20FIRST STOREY INTERIOR COLUMN(ACI-318.3)





CROSS-SECTION

Figure 4.21 FIRST STOREY EXTERIOR COLUMN (CAN3-A23.3-M84)

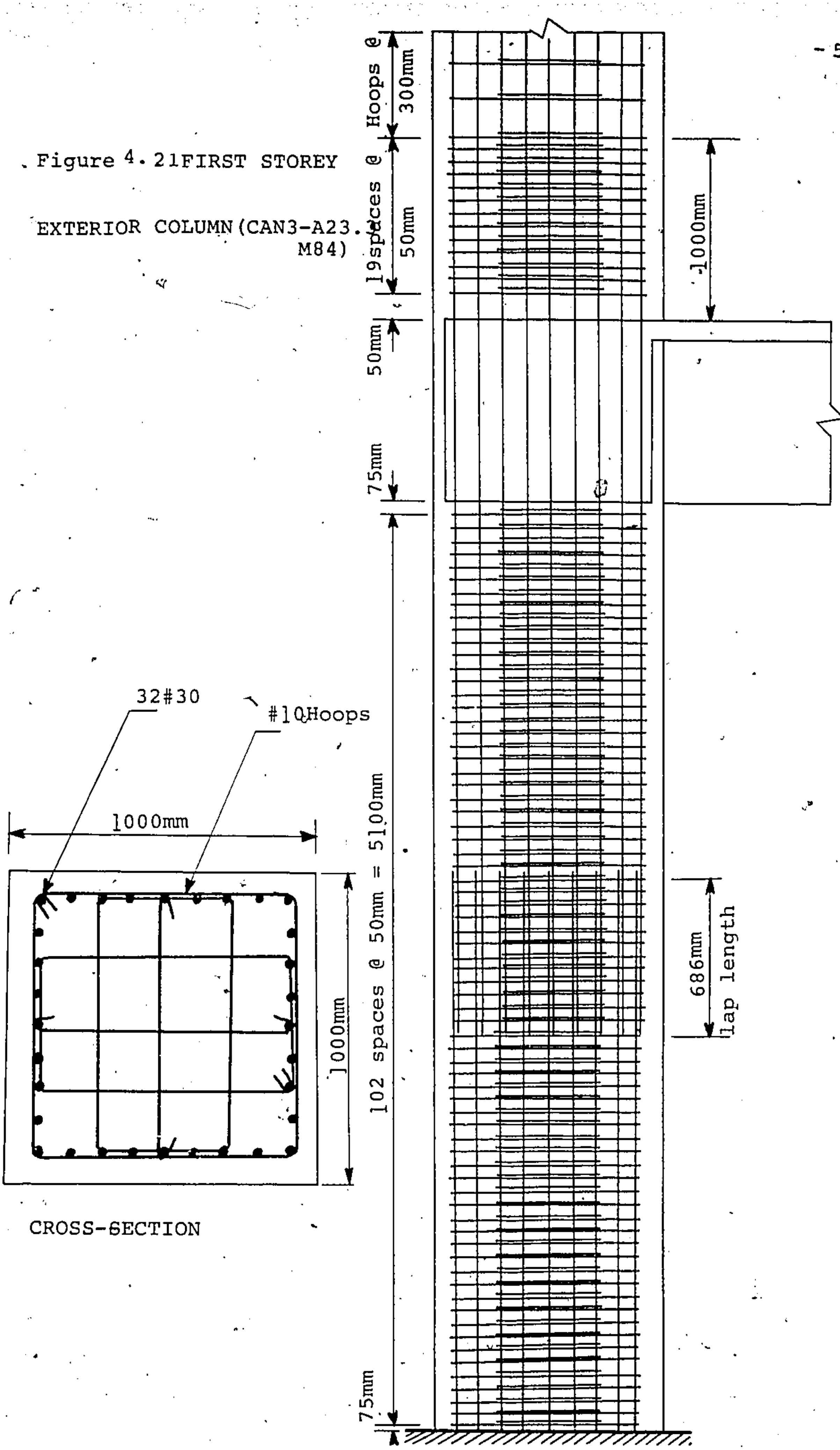
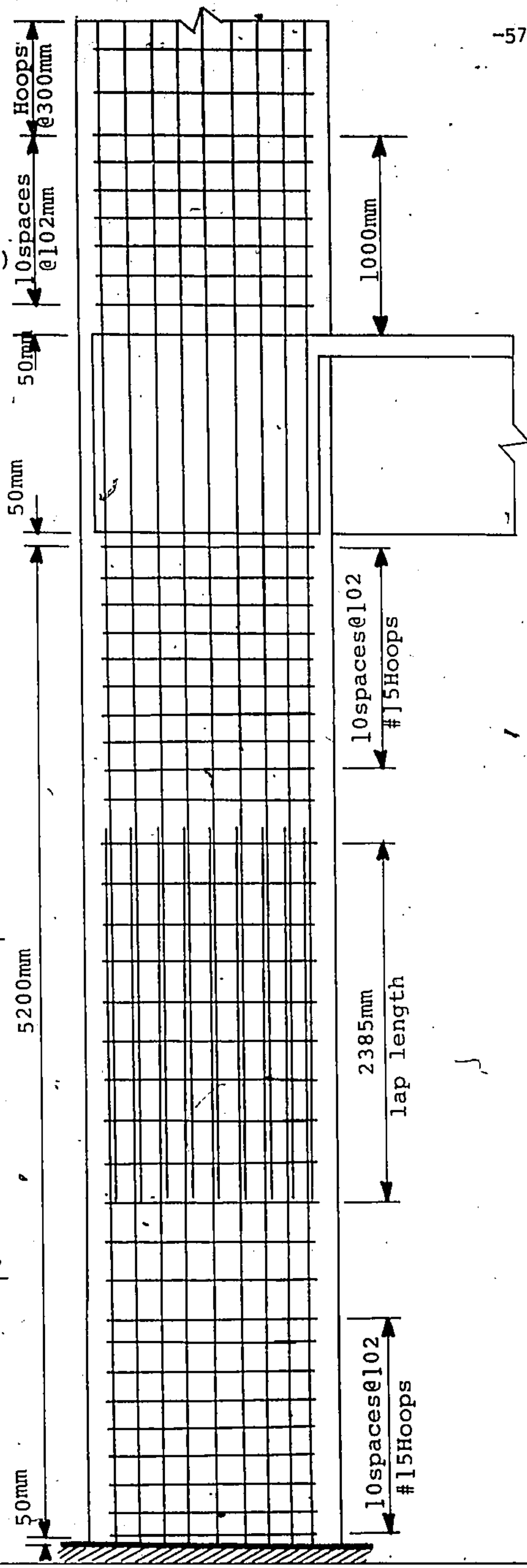
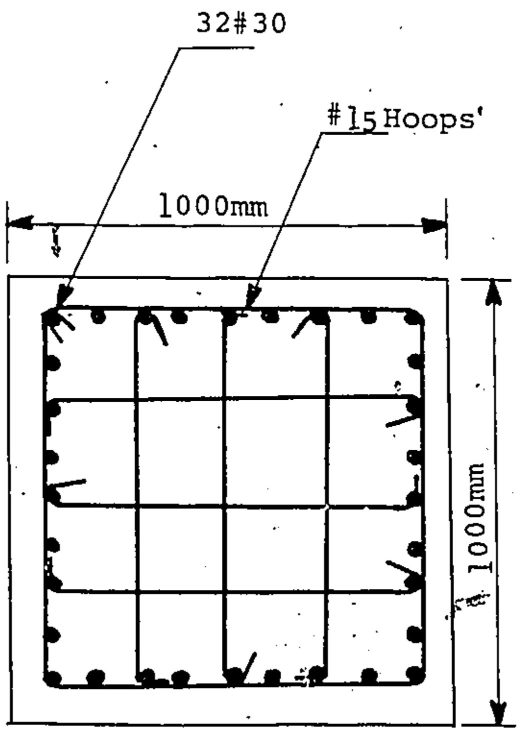


Figure 4.22 FIRST STOREY EXTERIOR COLUMN (ACI-318.3)





CROSS-SECTION

Figure 4.23 INTERIOR BEAM-COLUMN CONNECTIONS

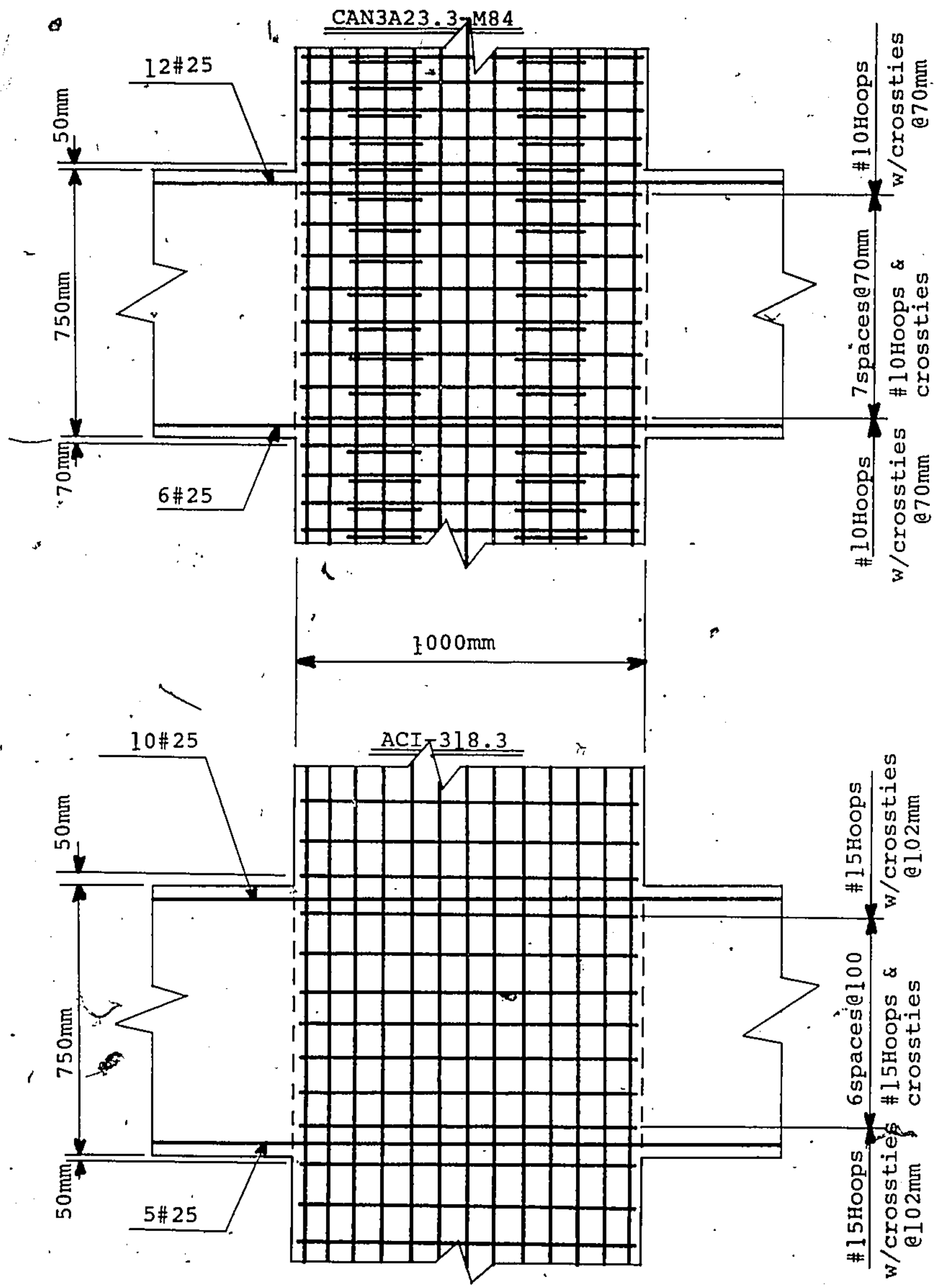
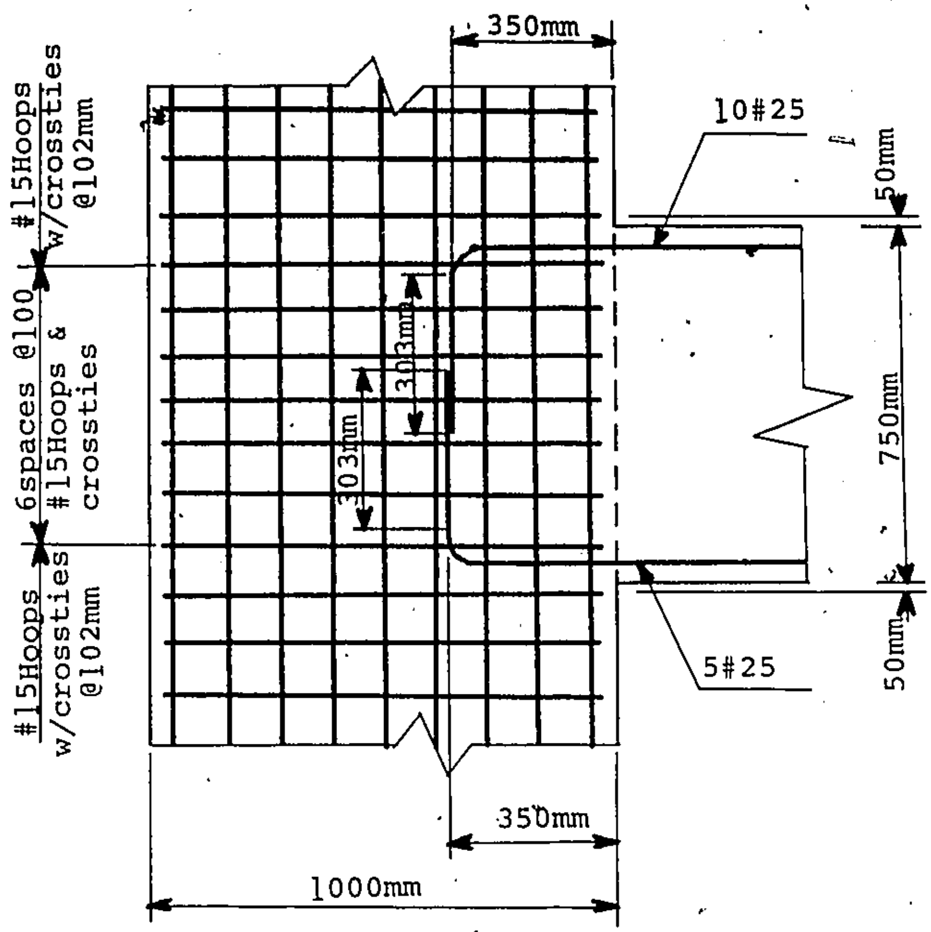
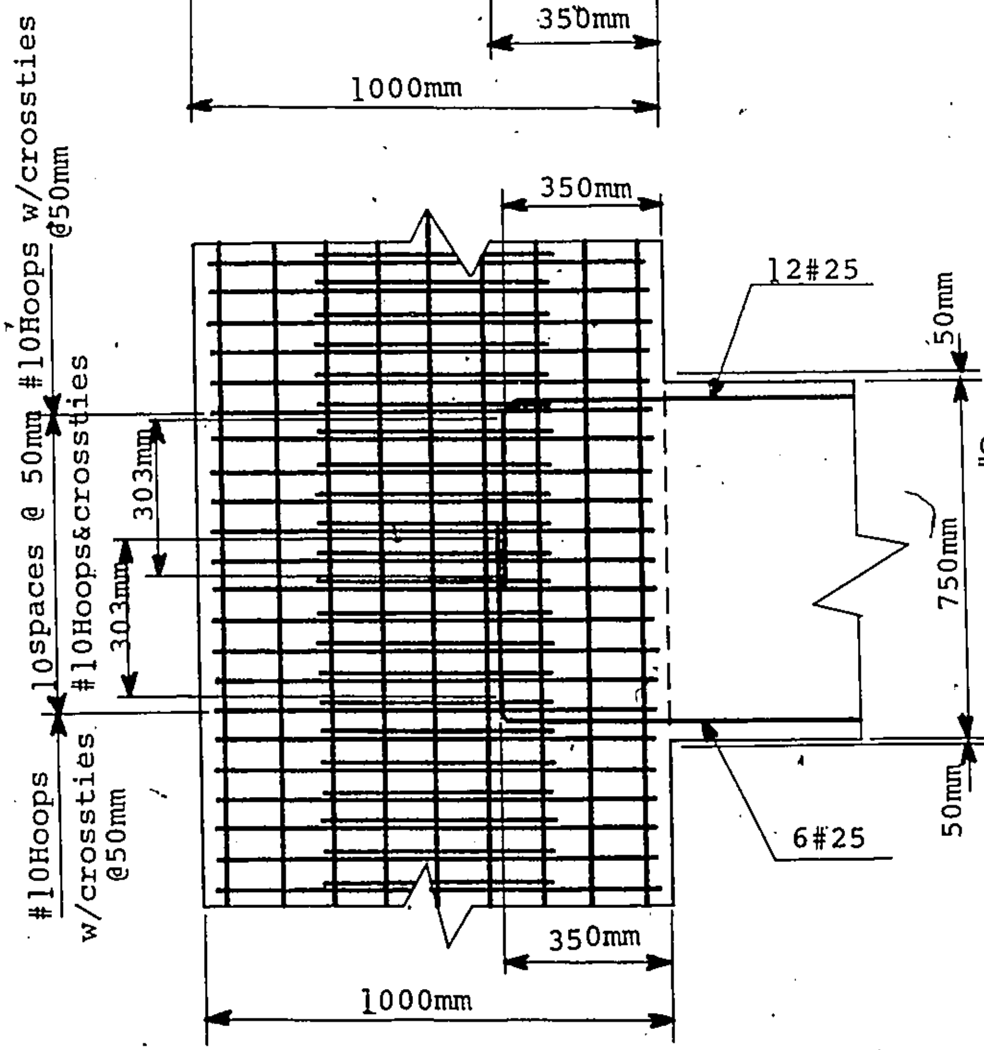


Figure 4.2 4 EXTERIOR BEAM- COLUMN CONNECTIONS



ACI318.3



CAN3-A23.3-M84

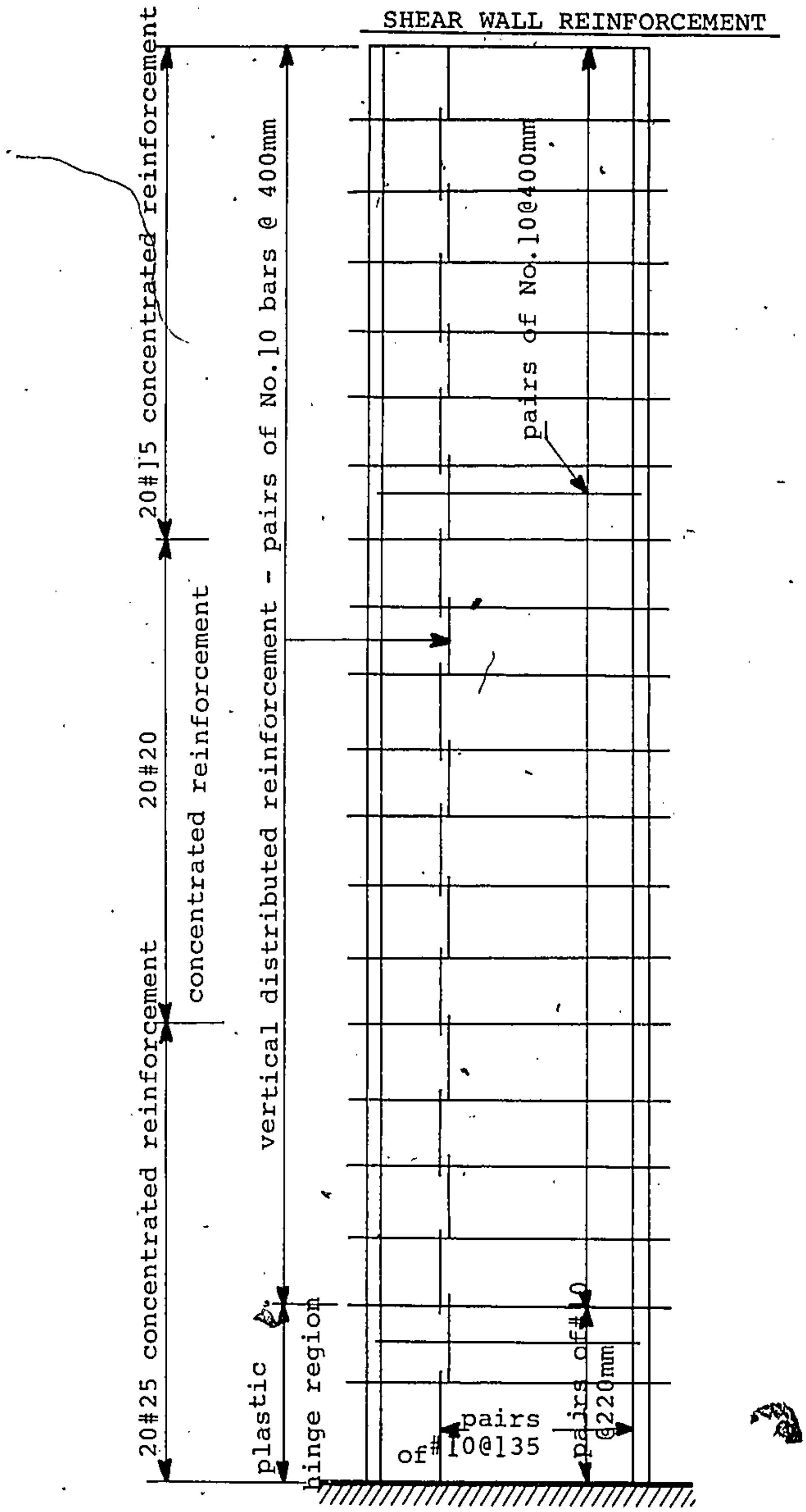


Figure 4.25   Reinforcement Details (CAN3-A23.3-M84)

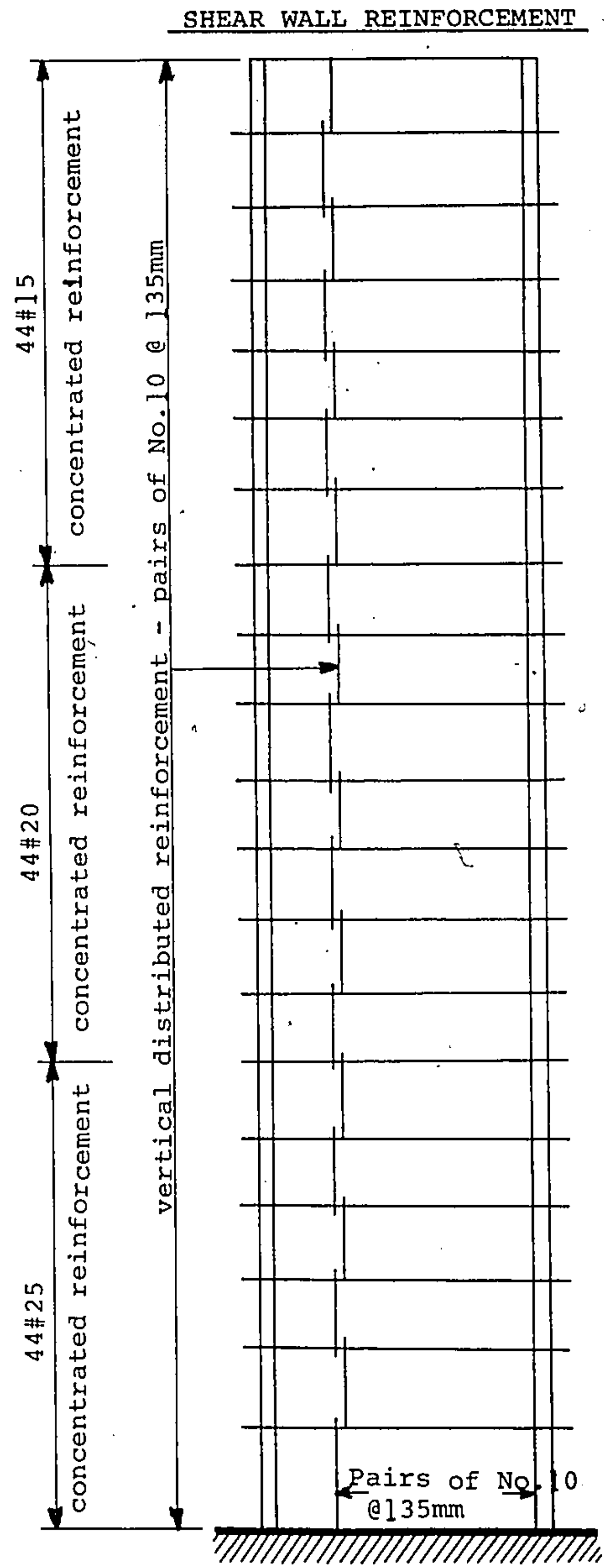


Figure 4.26 Reinforcement details (ACI.318-83)

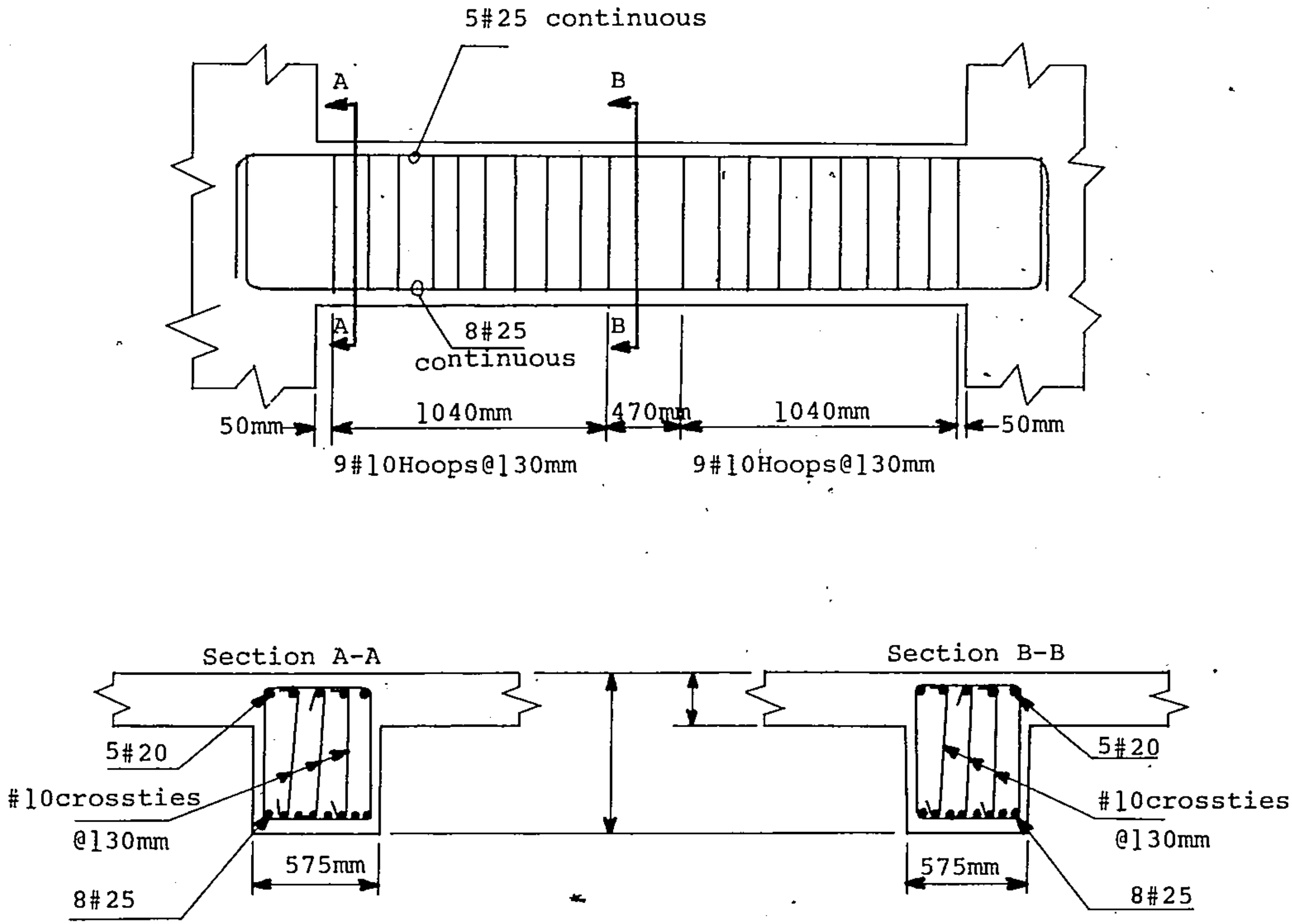


Figure 4.2 7 Shear Wall Coupling Beam (for both codes)

TABLE 4.1 SUMMARY OF EQUIVALENT STATIC EARTHQUAKE LOADS

Floor Level	Frame structure		Shear wall structure	
	NBCC -85	UBC -82	NBCC -85	UBC -82
Roof	699.36	1296.86	841.6	1209.8
20	478.18	468.68	463.4	498.5
19	453.66	444.65	439.6	472.9
18	429.14	420.61	415.8	447.4
17	404.61	396.58	392.1	421.9
16	380.09	372.54	368.3	396.2
15	355.57	348.51	344.6	370.7
14	331.05	324.47	320.8	345.2
13	324.14	317.70	297.0	319.6
12	298.21	292.28	273.3	294.0
11	272.28	266.87	249.5	268.5
10	246.35	241.45	228.2	245.5
9	220.41	216.04	204.2	219.6
8	194.48	190.62	180.1	193.8
7	168.55	165.20	156.1	168.0
6	153.92	150.86	132.1	142.1
5	125.94	123.43	108.1	116.2
4	97.95	96.00	84.1	90.5
3	69.96	68.56	60.1	64.6
2	45.59	44.68	38.3	41.2
G	0.00	0.00	0.0	0.0

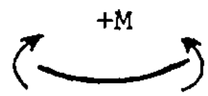


TABLE 4.2 DESIGN LATERAL LOADS (FRAME STRUCTURE)

	NBCC-85 & CAN3-A23.3-M84		UBC-82 & ACI-318.83	
	Beam AB	Beam BC	Beam AB	Beam BC
moment at jointA & B(KN.m)	±442	±434	±481	±473
moment near, mid-span (KN.m)	±25	0	±26	0
moment at jointB & C (KN.m)	±432	±434	±470	±473
	Col. AE	Col. BF	Col. AE	Col. BF
moment at A&B (KN.m)	19	167	19	167
moment at E&F (KN.m)	1126	1203	1126	1203
axial load (KN)	1766	117	1766	117
shear (KN)	191	229	191	229

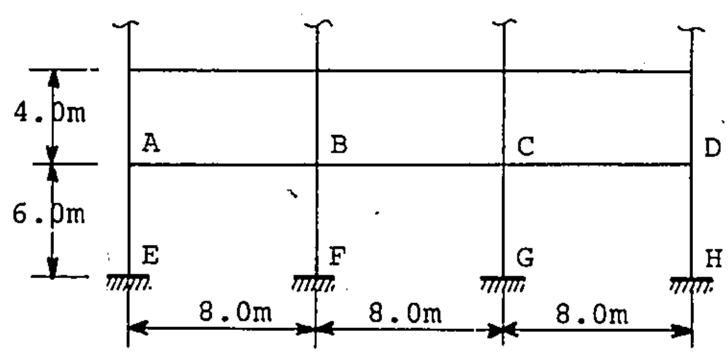
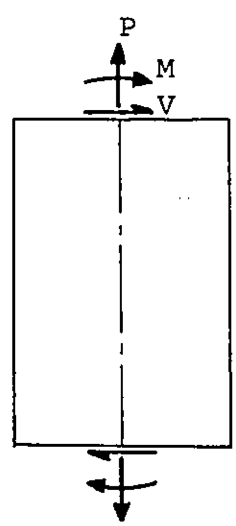


TABLE 4.3 DESIGN LATERAL LOADS (SHEAR-WALL STRUCTURE)

	NBCC-85 & CAN3-A23.3-M84	UBC-82 & ACI-318.83
moment (KN.m)	-18537 +24445	-22929 +31547
shear (KN)	1231	1436
axial load (KN)	6561	8008



positive end actions

TABLE 4.4 DESIGN GRAVITY LOADS (FRAME STRUCTURE)

	NBCC-85 & CAN3-A23.3-M84		UBC-82 & ACI-318.83	
	Beam AB	Beam BC	Beam AB	Beam BC
dead load moment at A&B (KN.m)	-186	-224	-186	-224
at mid-span	+116	+104	+116	+104
at B&C	-240	-224	-240	-224
live load moment at A&B (KN.M)	-71	-85	-71	-85
at mid-span	+43	+41	+43	+41
at B&C	-86	-79	-86	-79
	Col. AE	Col. BF	Col. AE	Col. BF
dead load moment at A&B (KN.m)	119	6	119	6
at E&F	63	2	63	2
live load moment at A&B	44	40	44	40
at E&F	23	20	23	20
dead load (KN)	6235	10367	6235	10367
live load (KN)	627	1253	627	1253

DESIGN GRAVITY LOADS (SHEAR-WALL STRUCTURE)

	NBCC-85 & CAN3-A23.3-M84	UBC-82 & ACI-318.83
dead load (KN), P_D	1123.7	1123.7
live load (KN), P_L	760.7	760.7

TABLE 4.5 LOAD COMBINATIONS

NBCC-1985	UBC-1982
1.25D+1.5L	1.4D+1.7L
1.25D+1.5E	
1.25D+0.7(1.5L+1.5E)	0.75(1.4D+1.7L+1.87E)
0.85D+1.5E	0.9D+1.43E

D:Dead Load

L:Live Load

E:Earthquake Load

TABLE 4.6 FACTORED DESIGN LOADS (SHEAR-WALL STRUCTURE)

	Axial load (KN)	Moment (KN.m)	shear (KN)	Axial load on boundary element (KN)
CAN3-A23.3-M84 &NBCC-85				
1.25D+1.5L	2546	nominal	nominal	1273
1.25D+0.7(1.5L+1.5E)	9093	25667	1293	7755
0.85D+1.5E	10797	36668	1847	9982
1.25D+1.5E	11247	36668	1847	10207
ACI-318.83 & UBC-82				
0.75(1.4D+1.7L+1.87E)	13381	44248	2014	12221
1.4D+1.7L	2866	nominal	nominal	1433
0.9D+1.43E	12462	45115	2054	11870

TABLE 4.7 FACTORED DESIGN LOADS(FRAME STRUCTURE)

DESIGN MOMENTS (KN.m)						
NBCC-85 & CAN3-A23.3-M84						
	BeamAB			BeamBC		
	jointA	jointB	midspan	jointB	jointC	midspan
1.25D+1.5L	338.5	428	210	408	399	74
0.85D+1.5E	505	852	136	841	841	9
1.25D+1.5E	895	948	182	931	931	13
1.25D+0.7(1.5L+1.5E)	770.5	843	216	825	819	56
ACI-318.83 & UBC-82						
	Beam AB			Beam BC		
	jointA	jointB	midspan	jointB	jointC	midspan
1.4D+1.7L	381	236	482	458	215	448
0.75(1.4D+1.7L+1.87E)	960	214	1021	1007	161	1000
0.9D+1.43E	855	142	888	878	94	878

TABLE 4.7 FACTORED DESIGN LOADS (FRAME STRUCTURE)

DESIGN MOMENTS (KN.m)				
NBCC-85 & CAN3-A23.3-M84				
	column AE		column BF	
	axial load (KN)	moment (KN.m)	axial load (KN)	moment (KN.m)
1.25D+1.5L	8734	113	14838	33
0.85D+1.5E	7949	1742	8988	1806
1.25D+1.5E	10443	1767	13134	1807
1.25D+0.7(1.5L+1.5E)	10307	1285	14397	1287
	ACI-318.83 & UBC-82			
	column AE		column BF	
	axial load (KN)	moment (KN.m)	axial load (KN)	moment (KN.m)
1.4D+1.7L	9795	242	16644	76
0.75(1.4D+1.7L+1.87E)	9823	2233	12647	1715
0.9D+1.43E	8137	1666	9498	1722

CHAPTER 5

DYNAMIC INELASTIC RESPONSE ANALYSIS

5.1 - General :

The two structures selected for this investigation were analyzed under dynamic loading. This provided an understanding of dynamic inelastic response of structures, designed on the basis of current North American building codes.

A computer program was used to conduct nonlinear dynamic analyses. The structures were modeled for computer analysis. It was concluded that the 1940 El Centro, E-W record would produce critical or near critical response in structures with a fundamental period of 2.0 seconds or longer. The structures considered in this investigation are predicted to fall under this category upon yielding of the critical members. Furthermore, the intensity of the earthquake record is increased by 50 percent, to obtain an intensity comparable to the high seismic risk zones of the building codes.

The details of the analysis procedure, structural modelling, and the results of nonlinear dynamic analyses are presented and discussed in this chapter.

5.2 - Analysis Procedure :

The dynamic inelastic analysis was carried out using program DRAIN-2D developed at the university of California at Berkely by A.E.Kanaan and G.H.Powell in 1972. It was then modified by the Portland Cement Association, Illinois in 1979. The program was later implemented to the university of Ottawa AMDAHL computer. The program has the capabilities to analyze plane inelastic structures under seismic excitation. The structural stiffness matrix is formulated by the direct stiffness method, with nodal displacements as unknowns. Dynamic response is determined using step-by-step integration by assuming a constant response acceleration during each time step. In the original program the moment-rotation rules used were proposed by Takeda and Sozen. This basic hysteresis loop for the decreasing stiffness beam element is shown in Fig.5.1 and was developed for members under constant level of axial force. The analytical model for hysteretic moment-rotation relationship was later modified by Saatcioglu [11] to include moment-axial force interaction. This model, shown in Fig.5.2, was used in the analyses.

Input for the program consists of Cectional capacities, nodal inertia masses, element stiffnesses, strain-hardening coefficients and ground accelerograms. Structure geometry and the properties of the model representing the prototype structure, also form part of the input data.

5.3 - Modelling for Computer Analysis :

Proper modelling of a structure for computer analysis cannot be overemphasized if reliable results are to be obtained. The structural members used in this investigation are modelled by means of line elements. Each column and beam member between joints is represented by a line element. It is extremely important to specify properties of these line elements properly so that both elastic and inelastic behavior of individual members can be simulated accurately. While the load deformation relationship for the elastic region is straightforward, representation of hinging regions of walls, columns and beams requires special attention.

DRAIN-2D accounts for inelastic action by allowing the formation of plastic hinges at the ends of line elements. Thus each element consists of an "elastic beam" and two potential "point hinges" at each end as shown in Fig.5.3. Stiffness of elastic beam and point hinges should be specified, such that total chord rotation of a line element in the model is equal to chord rotation of the real member.

The program utilizes a plane frame idealization similar to that in the static analysis procedure. To reduce the computer time necessary to analyze the structure, only one such frame was used for each analysis. The model for the frame structure consisted of one interior frame that was fixed at the base. For the shear wall structure, an exterior coupled wall was modelled.

Figures 5.4 and 5.5 illustrate the models used in the computer analysis.

5.4 - Dynamic Analysis of Frame structure :

An interior frame, in the short direction, was selected for dynamic analysis. The critical members of the frame, at the second floor level, had been designed twice using UBC-82 and NBCC-85 building codes. Both designs produced similar capacities with the same cross-sectional dimensions. Dead load tributary to each interior frame was assigned to each node as structure mass. Table 5.1 provides a summary of structural properties used in the analysis.

The frame was first analyzed under 1.5 times the 1940 El Centro Earthquake, E-W record. Axial load couple created during response produced excessive compressive force. High axial compression, combined with bending, created excessive compression in the extreme fibres of exterior columns, producing concrete crushing. Once the failure surface, defined by an interaction diagram, was exceeded in the analytical model for the hystretic force deformation relationship, a message was printed by the program indicating crushing of column concrete.

Next, the column capacity was increased to ensure elastic column response. The analysis was carried out twice under the 1940 El Centro Earthquake, E-W record, first with the intensity as recorded and second with 1.5 times this record.

The analysis results are presented in the form of maximum response quantities along the height of the structure. Figures 5.6 through 5.18 illustrate maximum moments, shears, axial forces and rotational ductilities for various frame members.

5.5- Dynamic Analysis of shear Wall Structure:

An exterior coupled shear wall was selected for dynamic analysis. The wall and the coupling beams had been previously designed following the requirements of UBC-82 and NBCC-85 building codes. Therefore two sets of coupled walls were analyzed using two different member capacities. Total mass at each floor level was proportioned on the basis of previous static analysis and the relative stiffnesses of frames. Hence each exterior coupled wall was assigned 45% of the floor mass. structural properties used in the analysis are shown in Table 5.2. It should be noted that a strength and stiffness taper was introduced to both the walls and the beams to reflect the actual construction practice.

The structure that was designed following the NBCC-85 code, was analyzed under 1.5 times the El Centro, E-W record. The results indicated failure of shear walls under net axial tension caused by strong coupling of the beams and relatively weak capacity of the walls. The same structure was analyzed under the same ground motion with intensity reduced to the actual intensity of the record. Although the response was somewhat improved, failure of walls under net axial tension was indicated. No further analysis was conducted on this structure.

The structure that was designed to conform the UBC-82 code performed well under the 1940 El Centro, E-W record. However, when the intensity of the input motion was increased by 50%, excessive coupling and wall failure under net axial tension was observed.

The same structure was further analyzed with increased wall strength . In this analysis, uniform wall strength was used throughout the height of the structure and the wall capacity was increased by 50%. The structure was subjected to 1.5 times the El Centro E-W record, and showed reasonably well response with high ductility beam demands. The results of the analyses are shown in figu res 5.19 through 5.27.

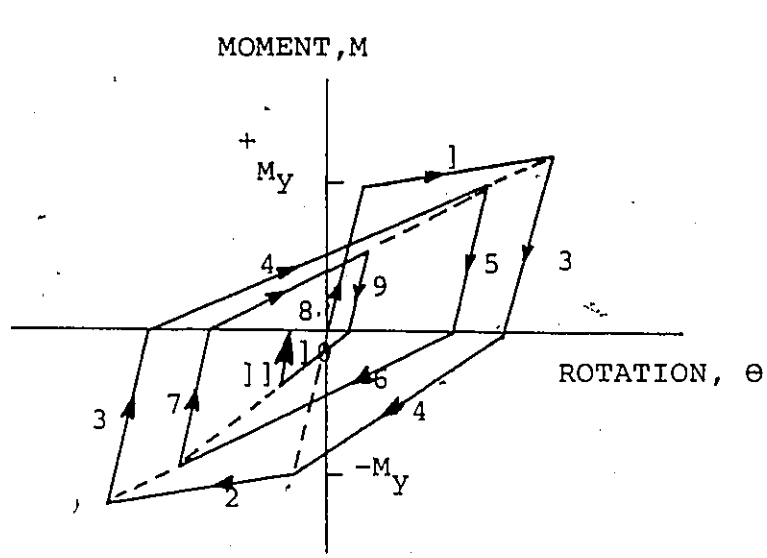


Figure 5.1 Takeda's Hysteretic Loop

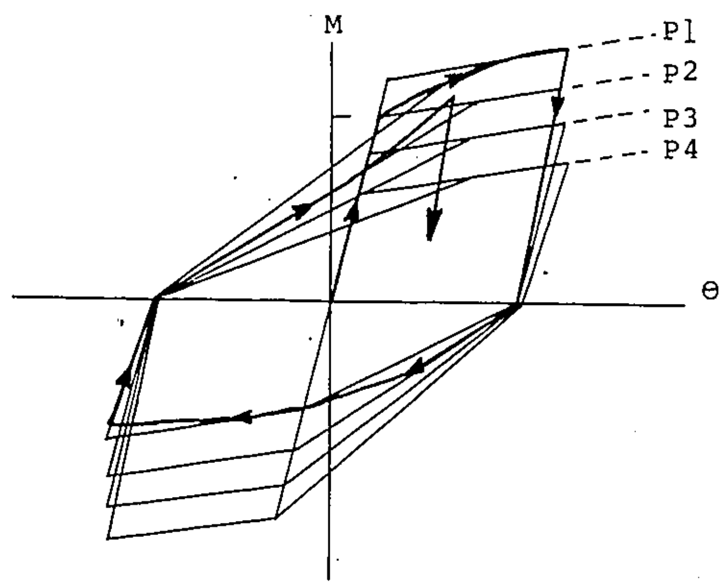


Figure 5.2 Hysteretic Loop Under Changing Axial Forces

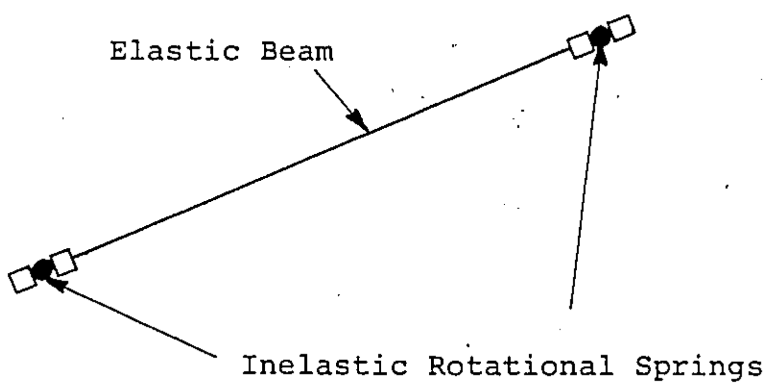
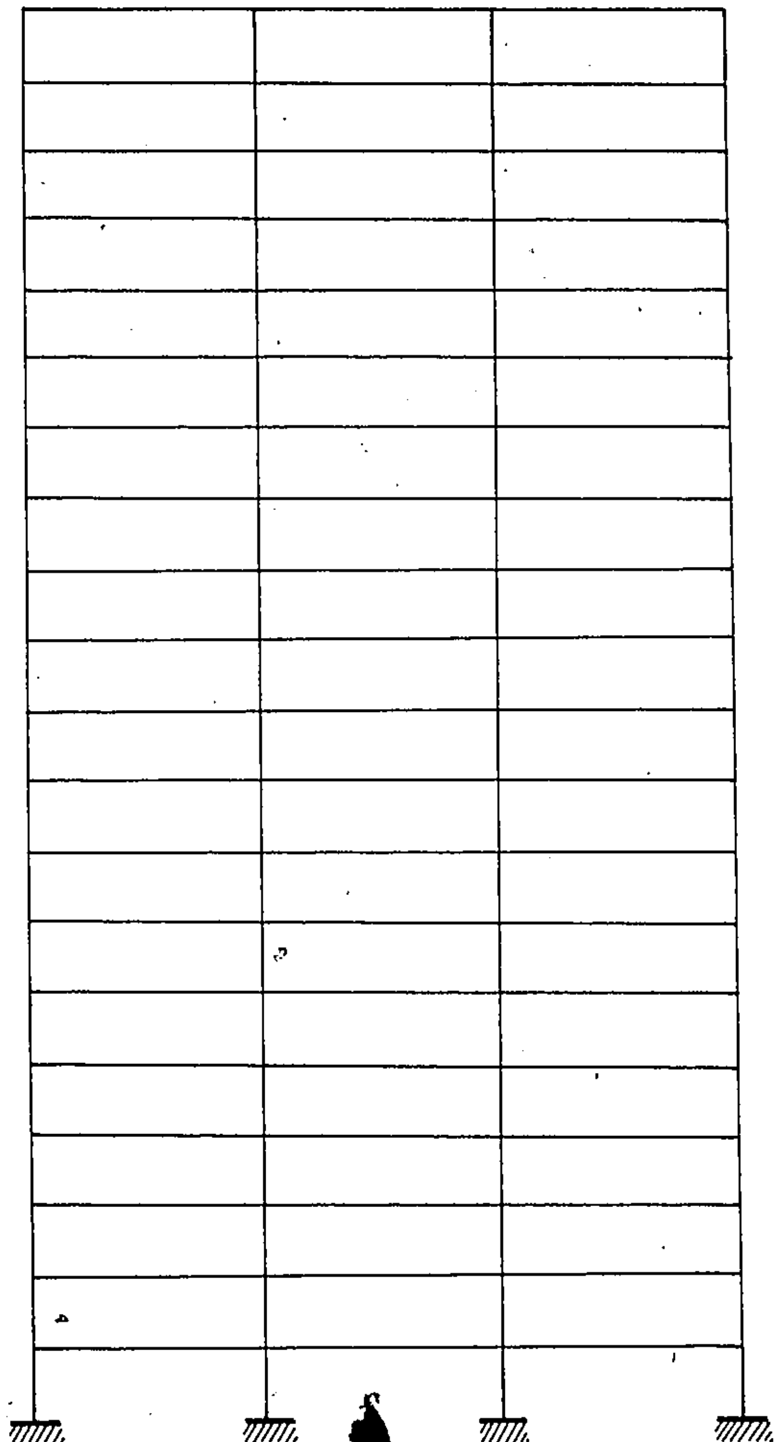


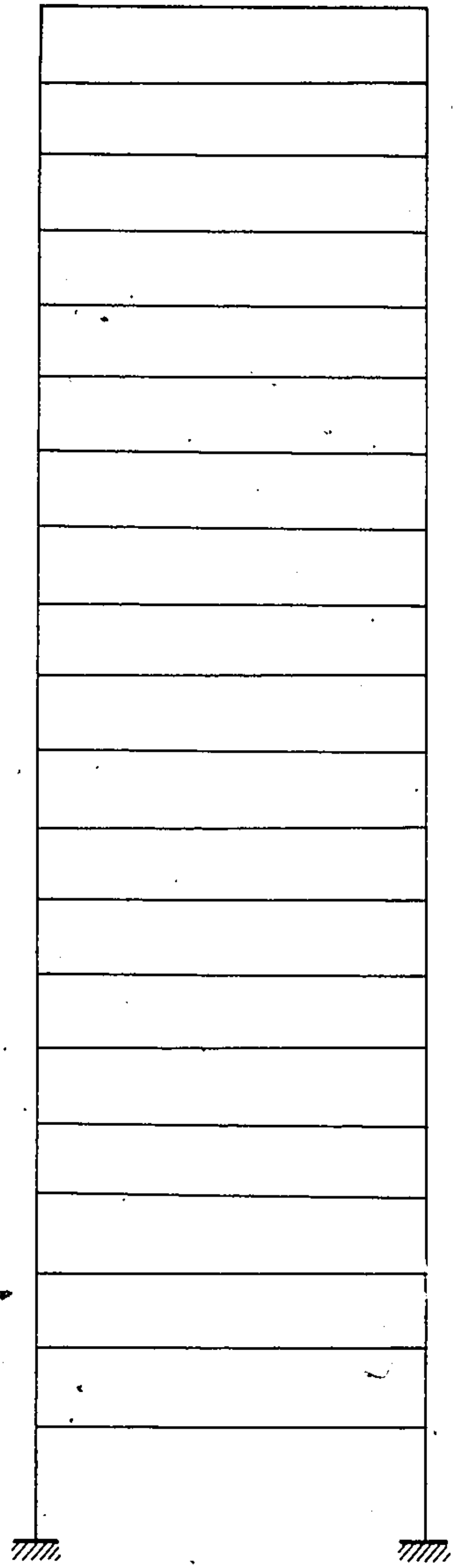
Figure 5.3 ELEMENT IDEALIZATION

Figure 5.4 FRAME BUILDING MODEL



MODEL FOR COMPUTER ANALYSIS (DRAIN-2D)

Figure 5.5 SHEAR WALL BUILDING MODEL



MODEL FOR COMPUTER ANALYSIS (DRAIN-2D)

R. C. FRAME BUILDING (NON-TAPERED COLUMNS)

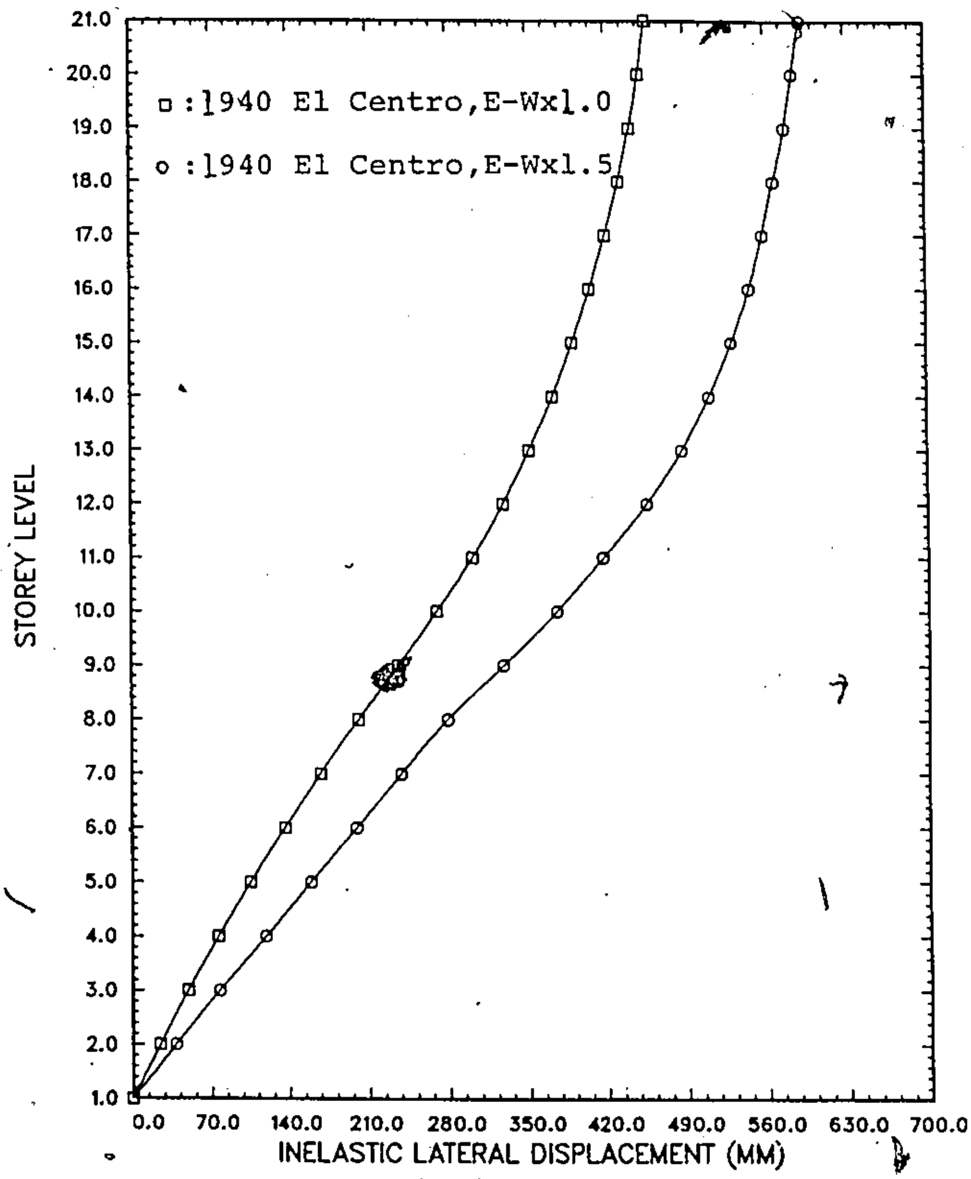


Fig.5. 6 INELASTIC LATERAL DISPLACEMENTS DUE TO EARTH-QUAKE LOADING

R. C. FRAME BUILDING

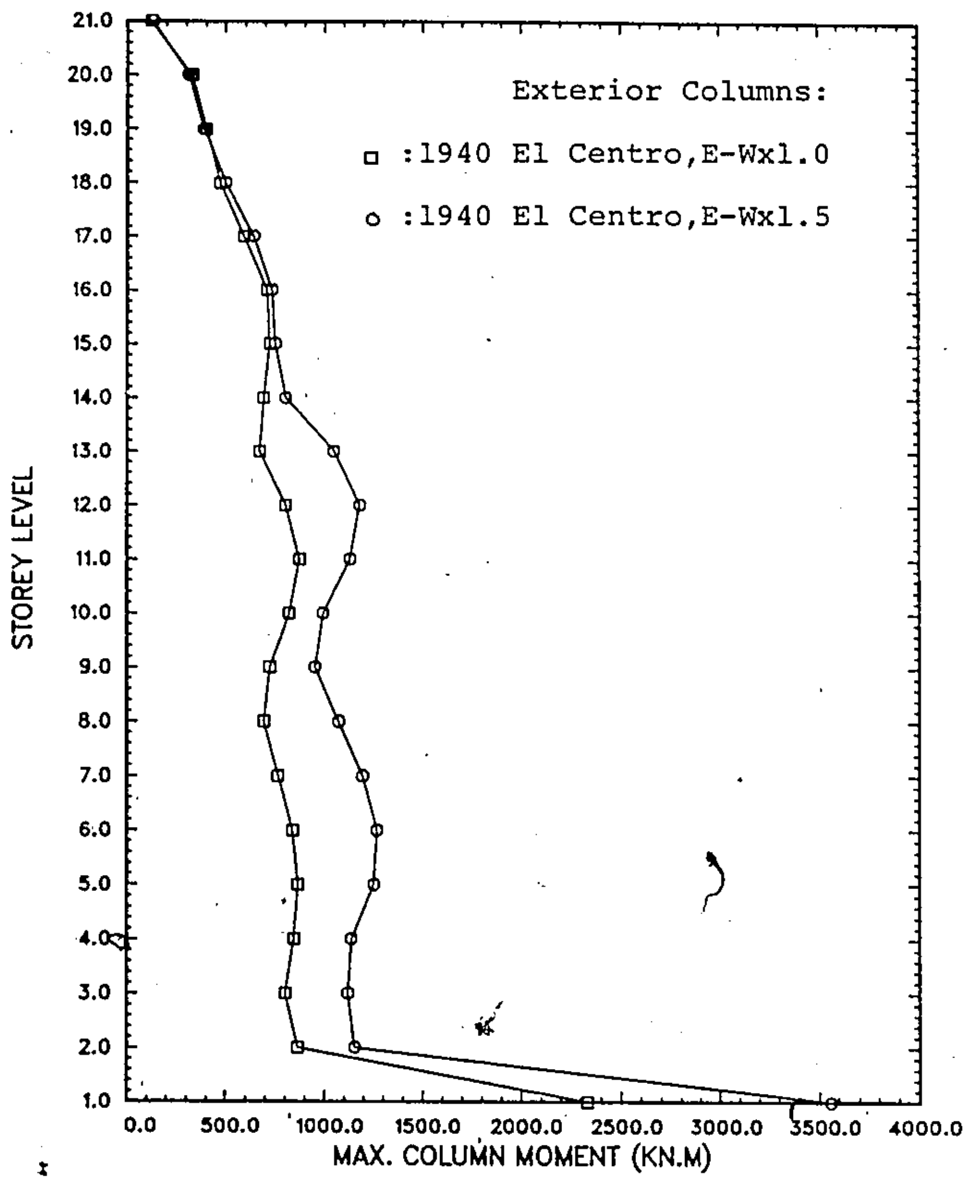


Fig.5.7 MAXIMUM COLUMN MOMENTS FOR THE FRAME BUILDING

R. C. FRAME BUILDING

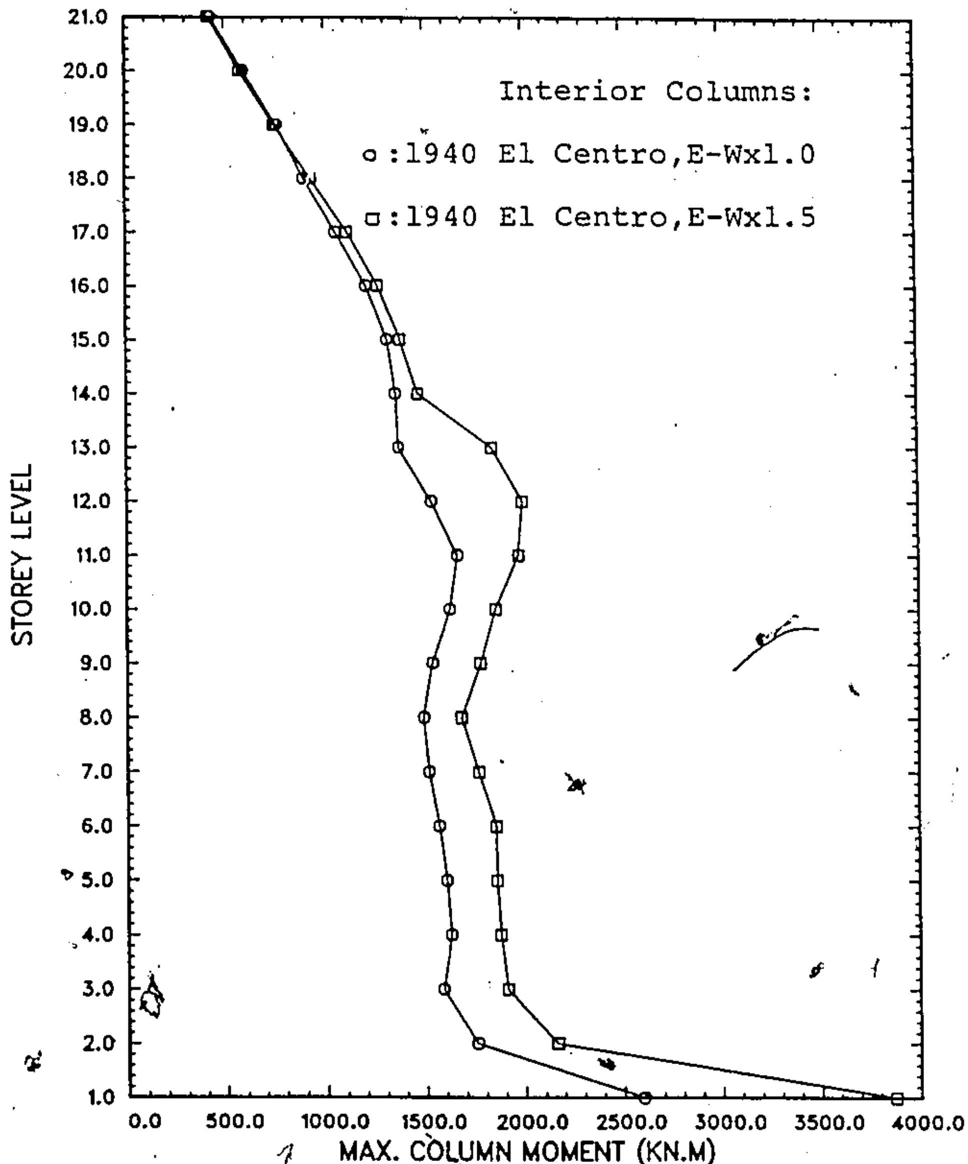


Fig. 5. 8 MAXIMUM COLUMN MOMENTS FOR THE FRAME BUILDING

R. C. FRAME BUILDING

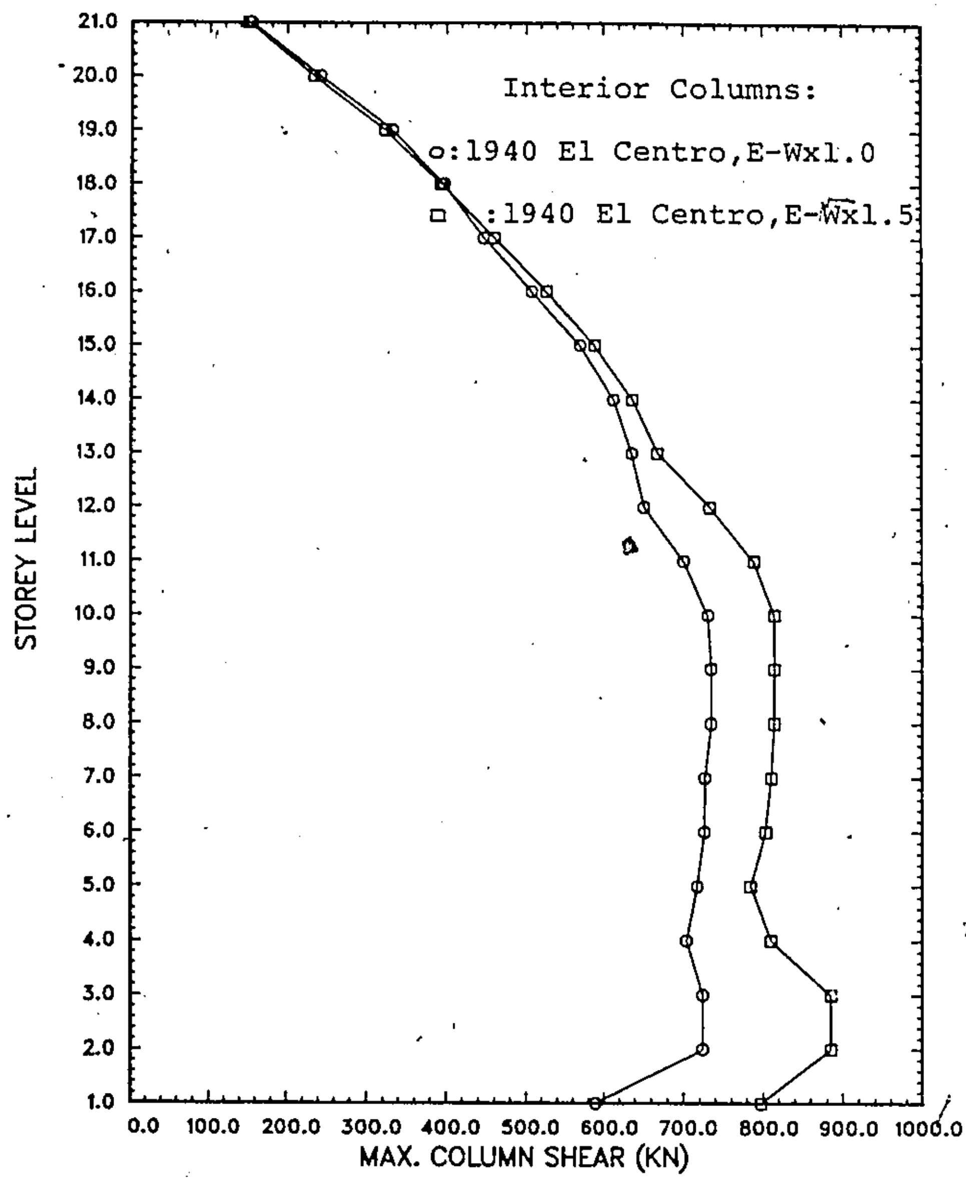


Fig.5.9 MAXIMUM COLUMN SHEARS FOR THE FRAME BUILDING

R. C. FRAME BUILDING

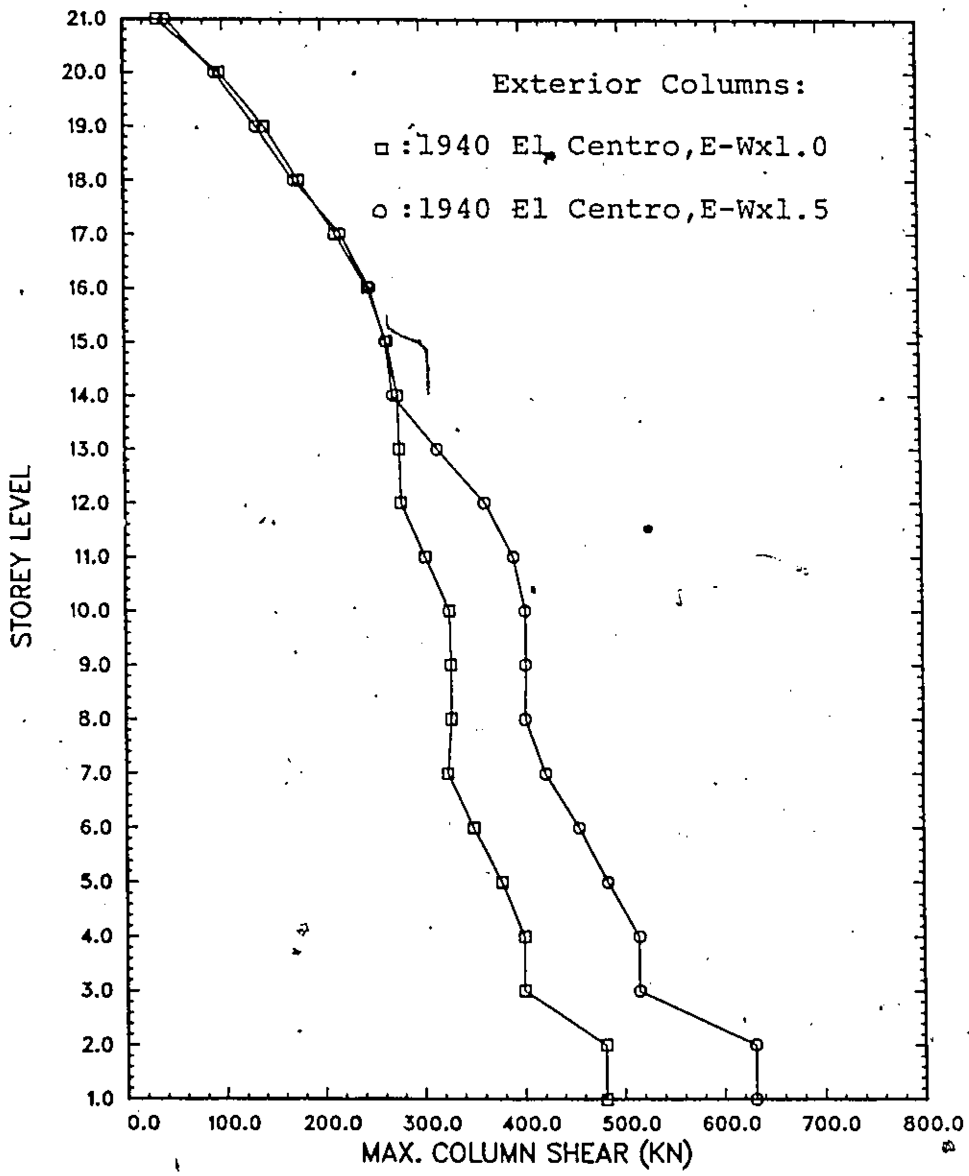


Fig.5.10 MAXIMUM COLUMN SHEARS FOR THE FRAME BUILDING

R. C. FRAME BUILDING

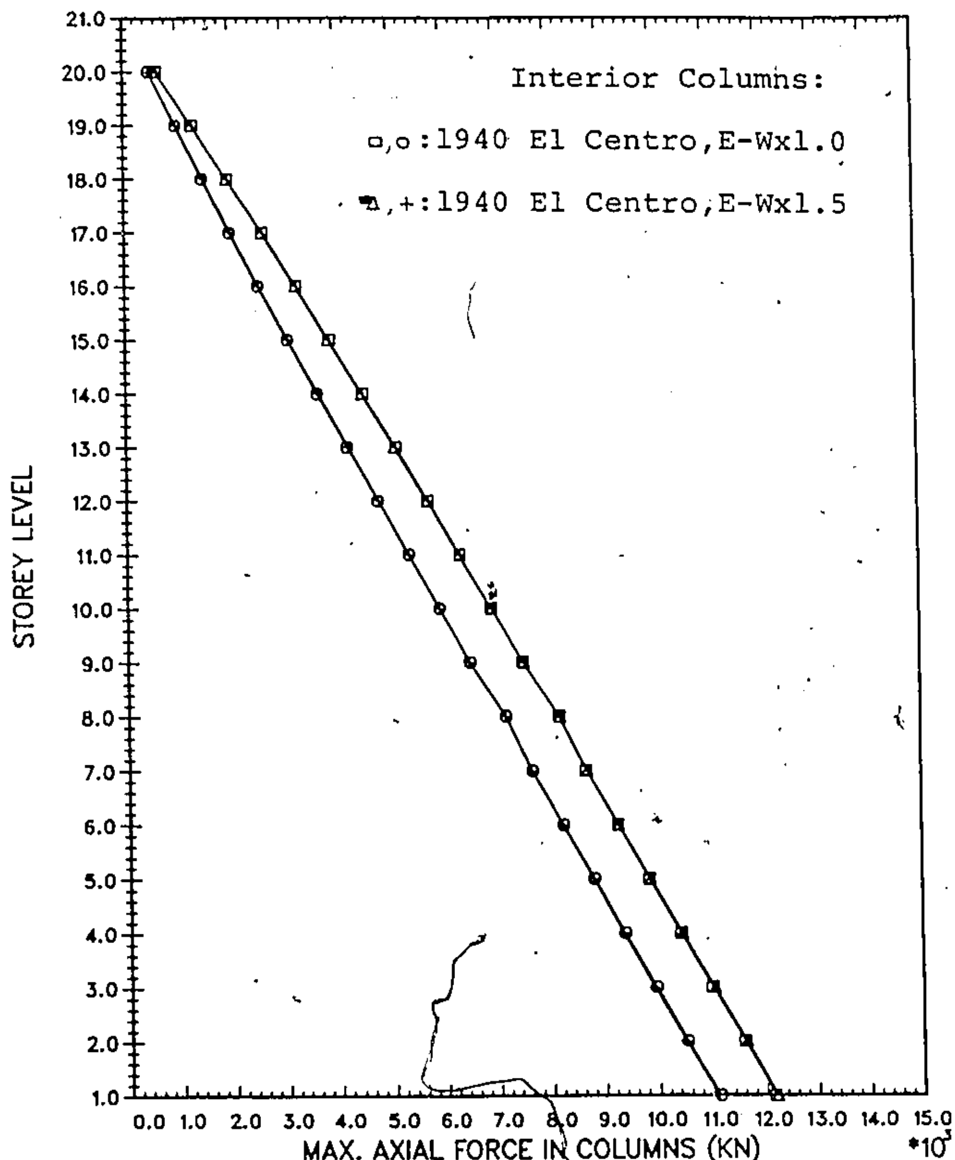


Fig.5.11 MAXIMUM AXIAL FORCE IN COLUMNS (FRAME Bldg.)

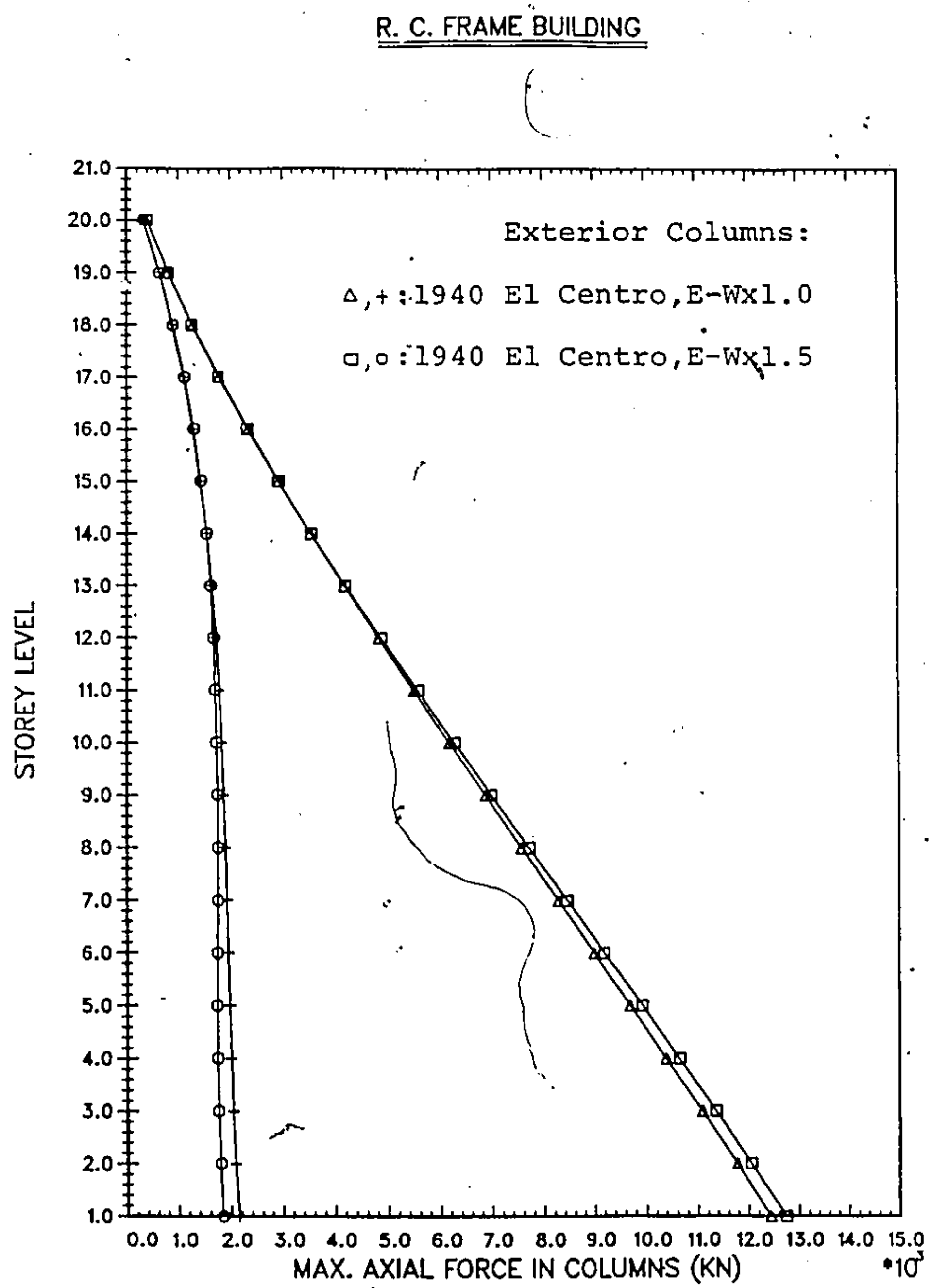


Fig.5.12 MAXIMUM AXIAL FORCE IN COLUMNS (FRAME Bldg.)

R. C. FRAME BUILDING

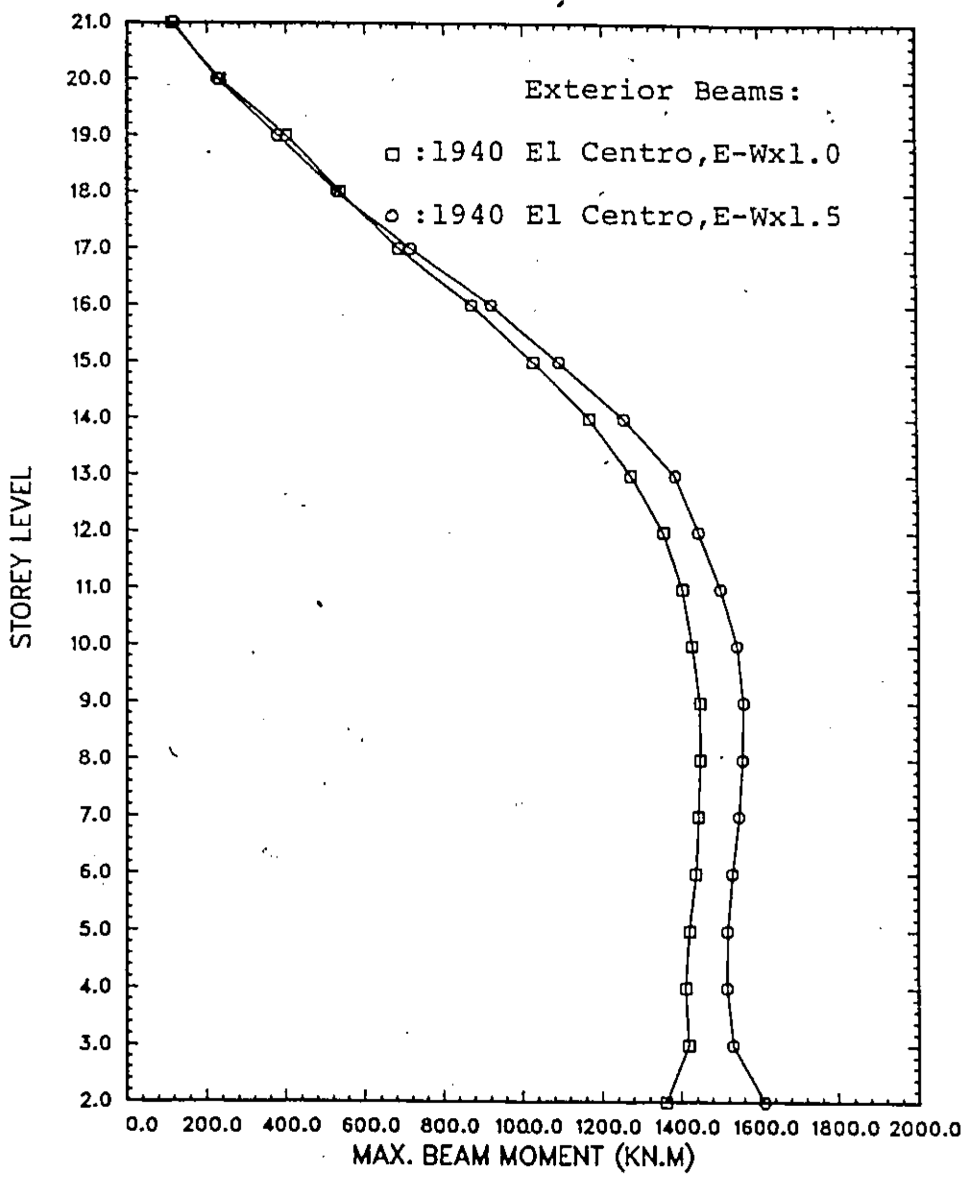


Fig.5.13 MAXIMUM BEAM MOMENTS (FRAME Bldg.)

R. C. FRAME BUILDING

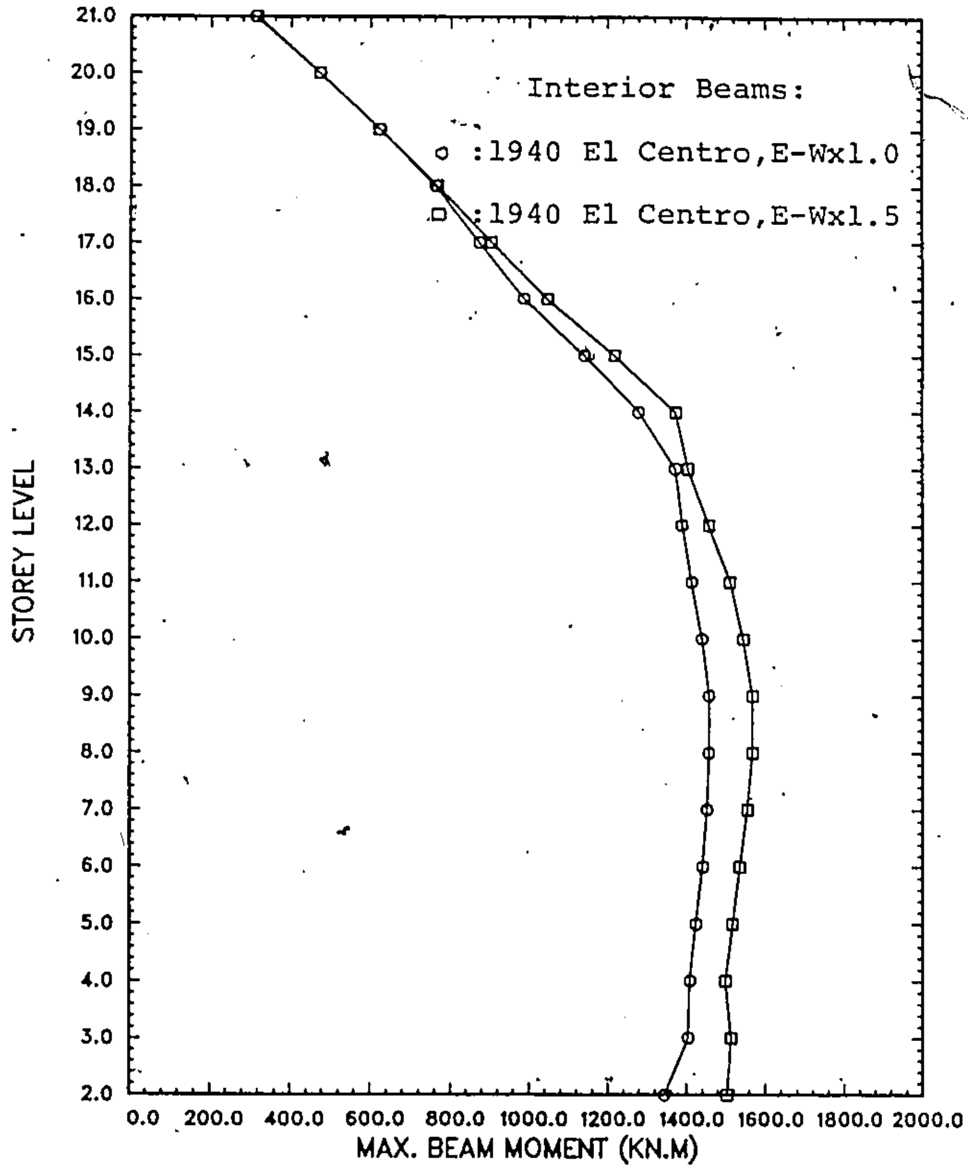


Fig.5.14 MAXIMUM BEAM MOMENTS (FRAME Bldg.)

R. C. FRAME BUILDING

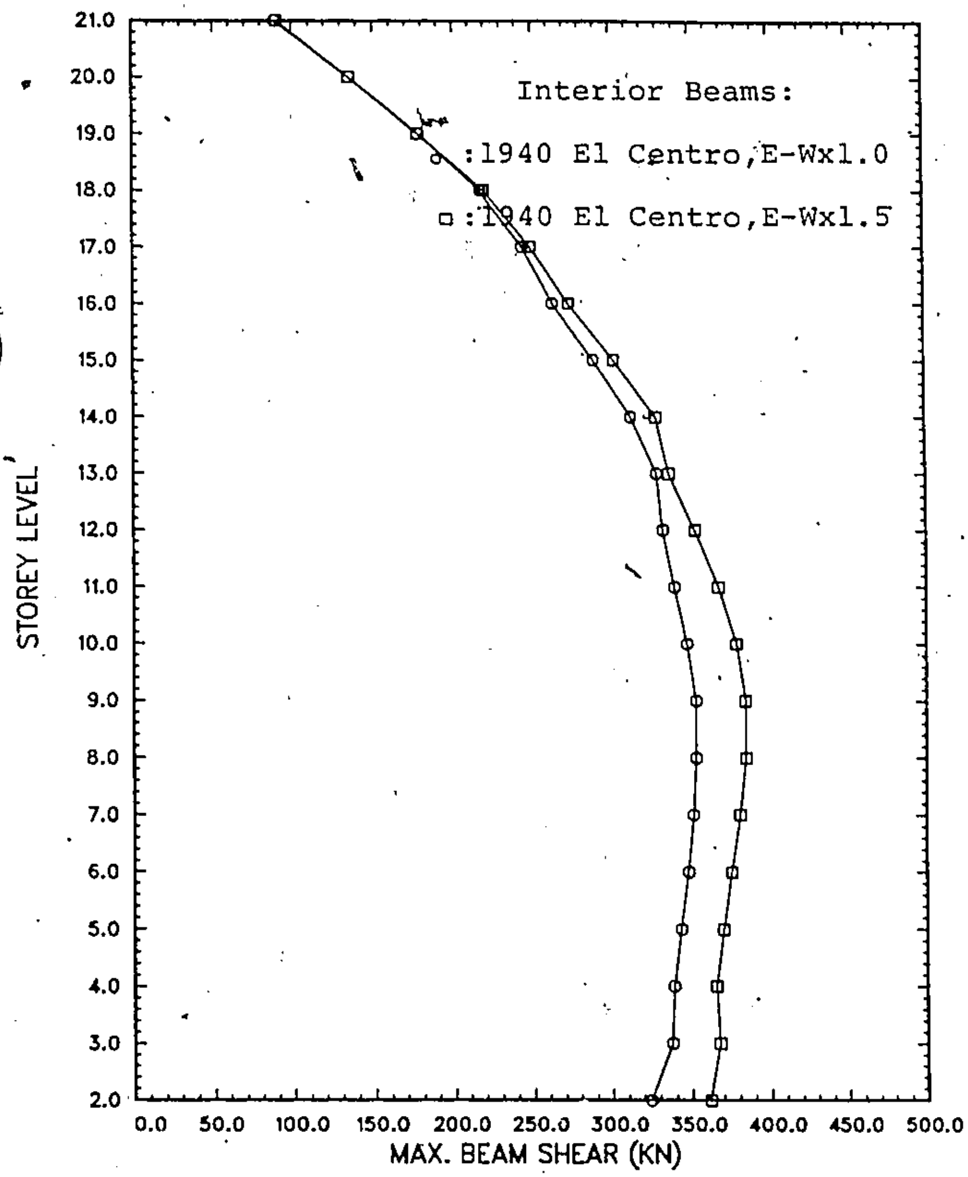


Fig.5.15 MAXIMUM BEAM SHEARS (FRAME Bldg.)

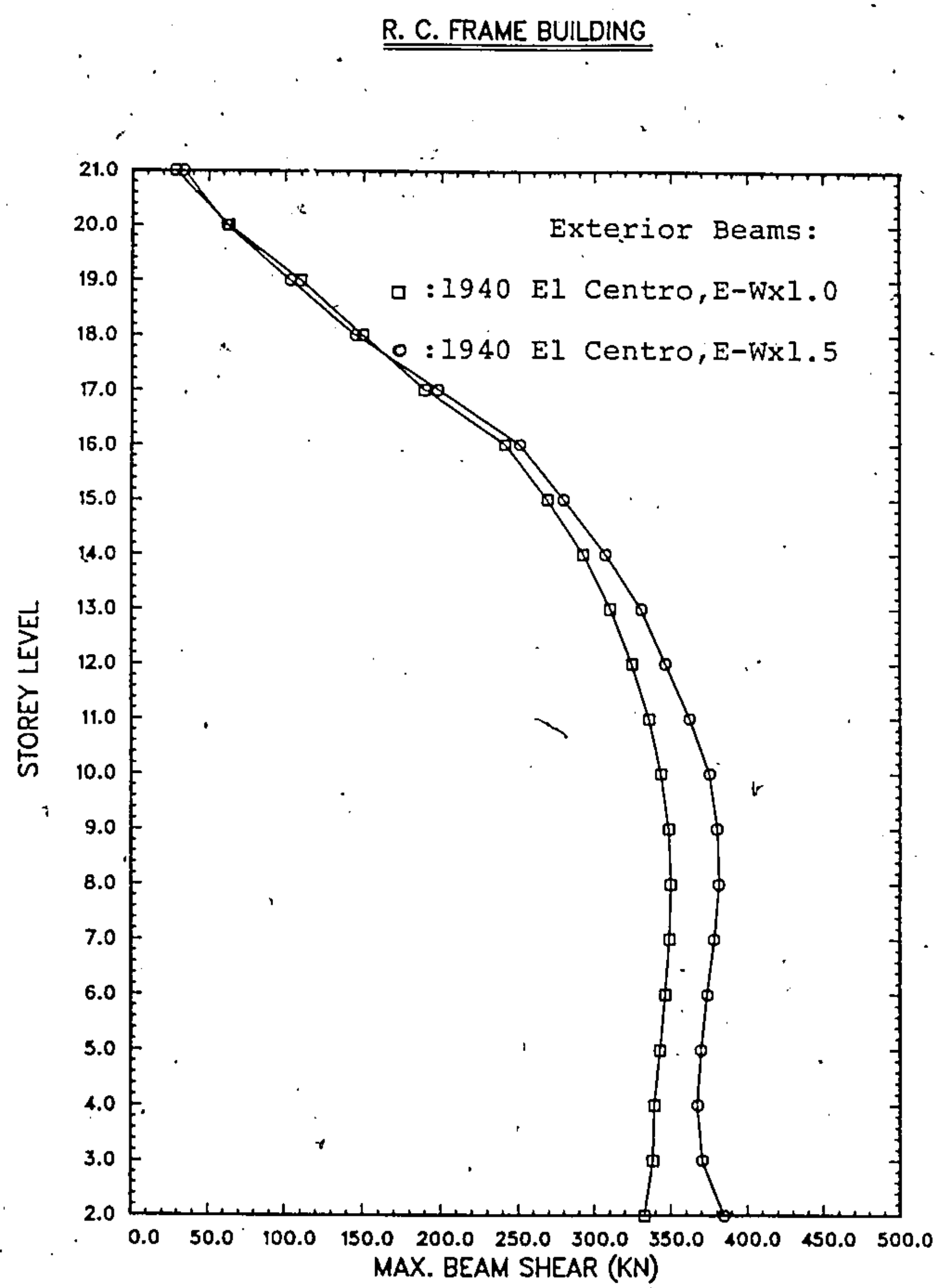


Fig.5.16 MAXIMUM BEAM SHEARS (FRAME Bldg.)

R. C. FRAME BUILDING

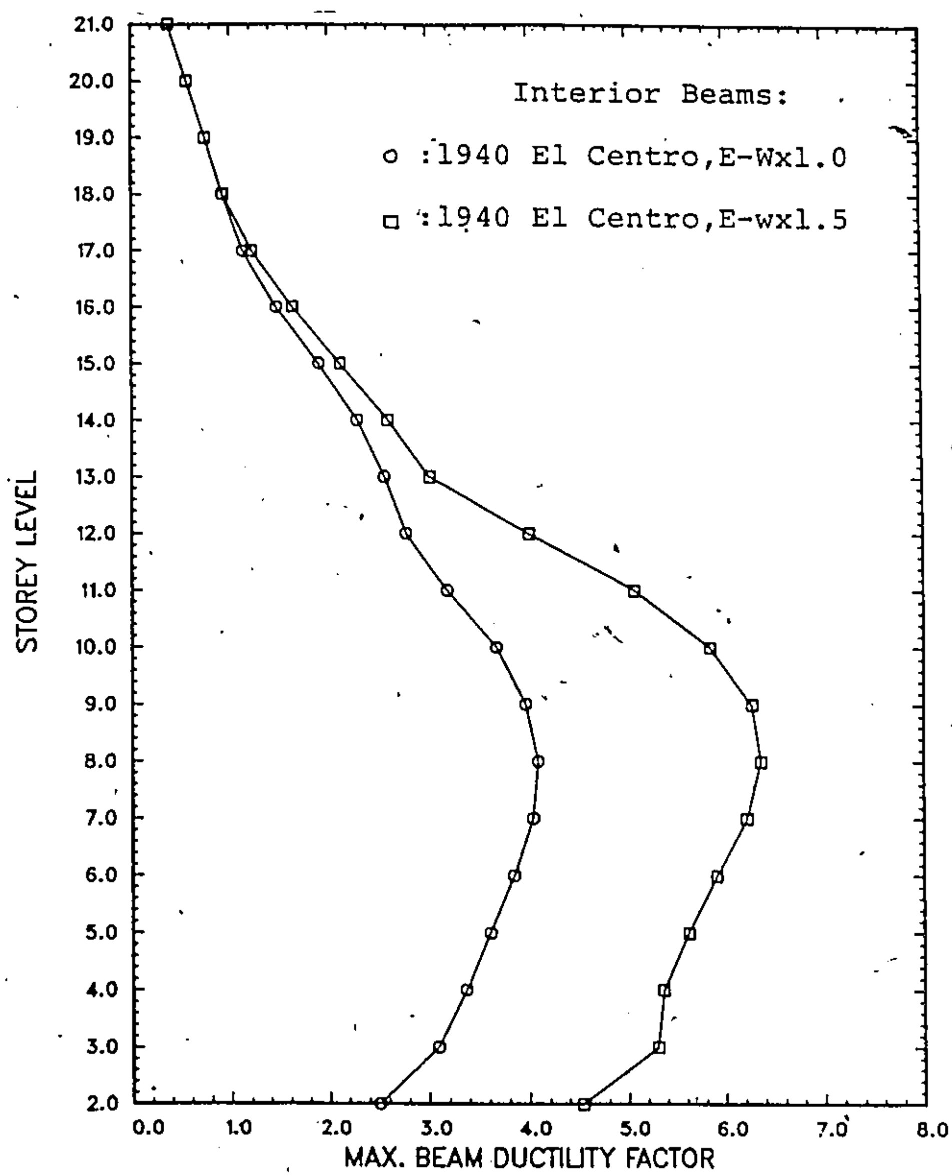


Fig.5.17 MAXIMUM BEAM DUCTILITY FACTOR (FRAME Bldg.)

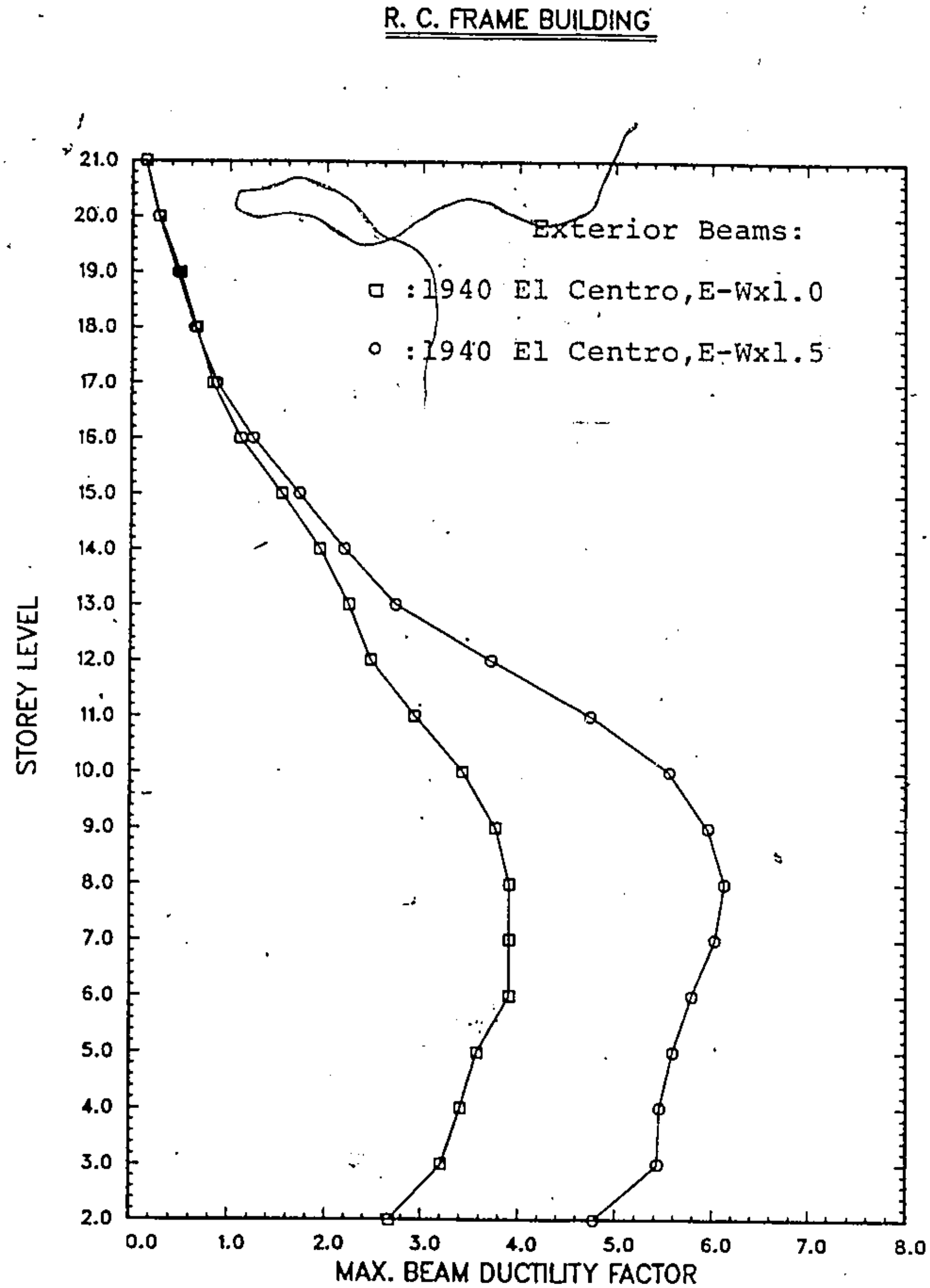


Fig.5.18 MAXIMUM BEAM DUCTILITY FACTOR (FRAME Bldg.)

R. C. SHEAR-WALL BUILDING

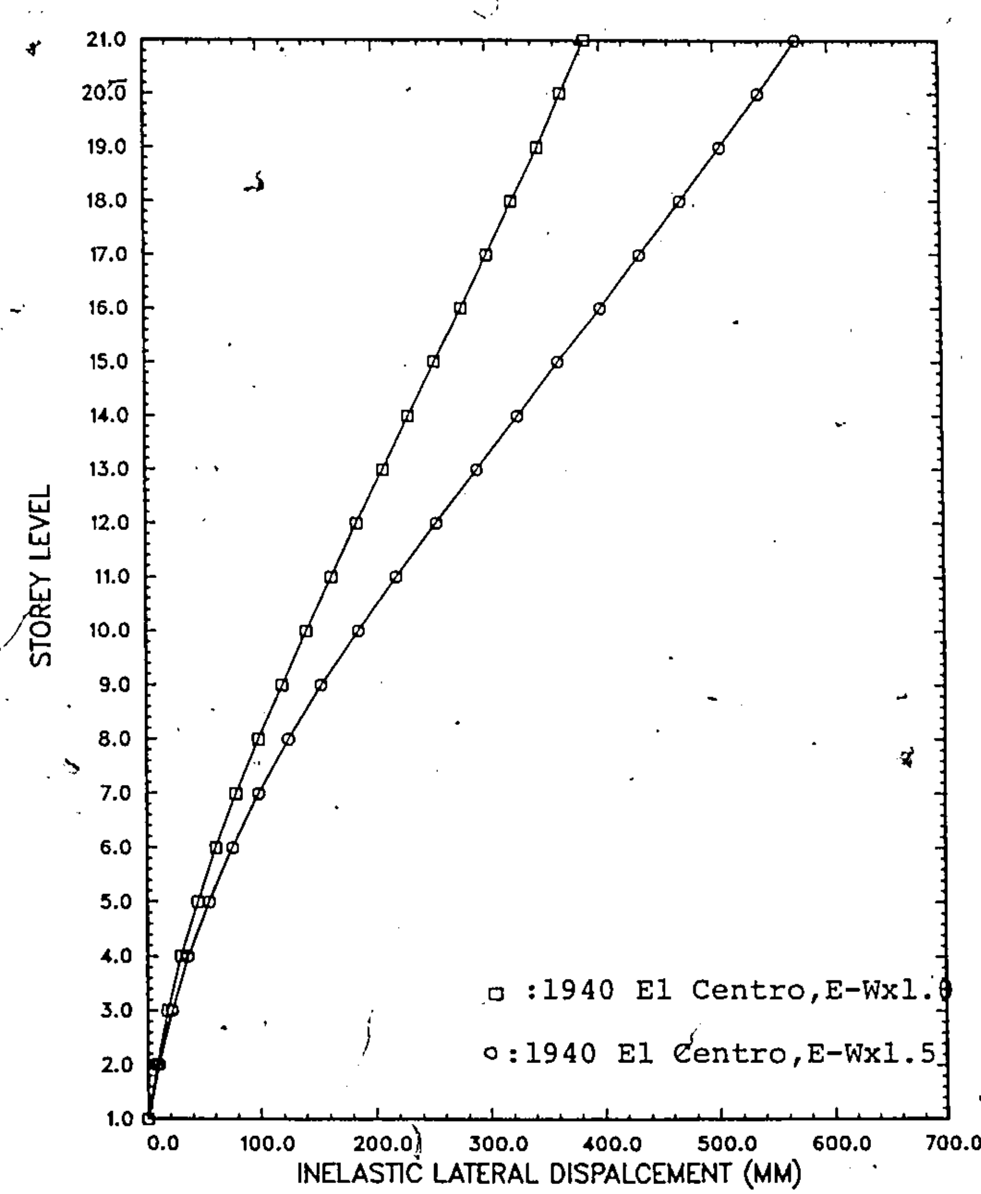


Fig.5.19 INELASTIC LATERAL DISPLACEMENTS DUE TO EARTHQUAKE LOADING

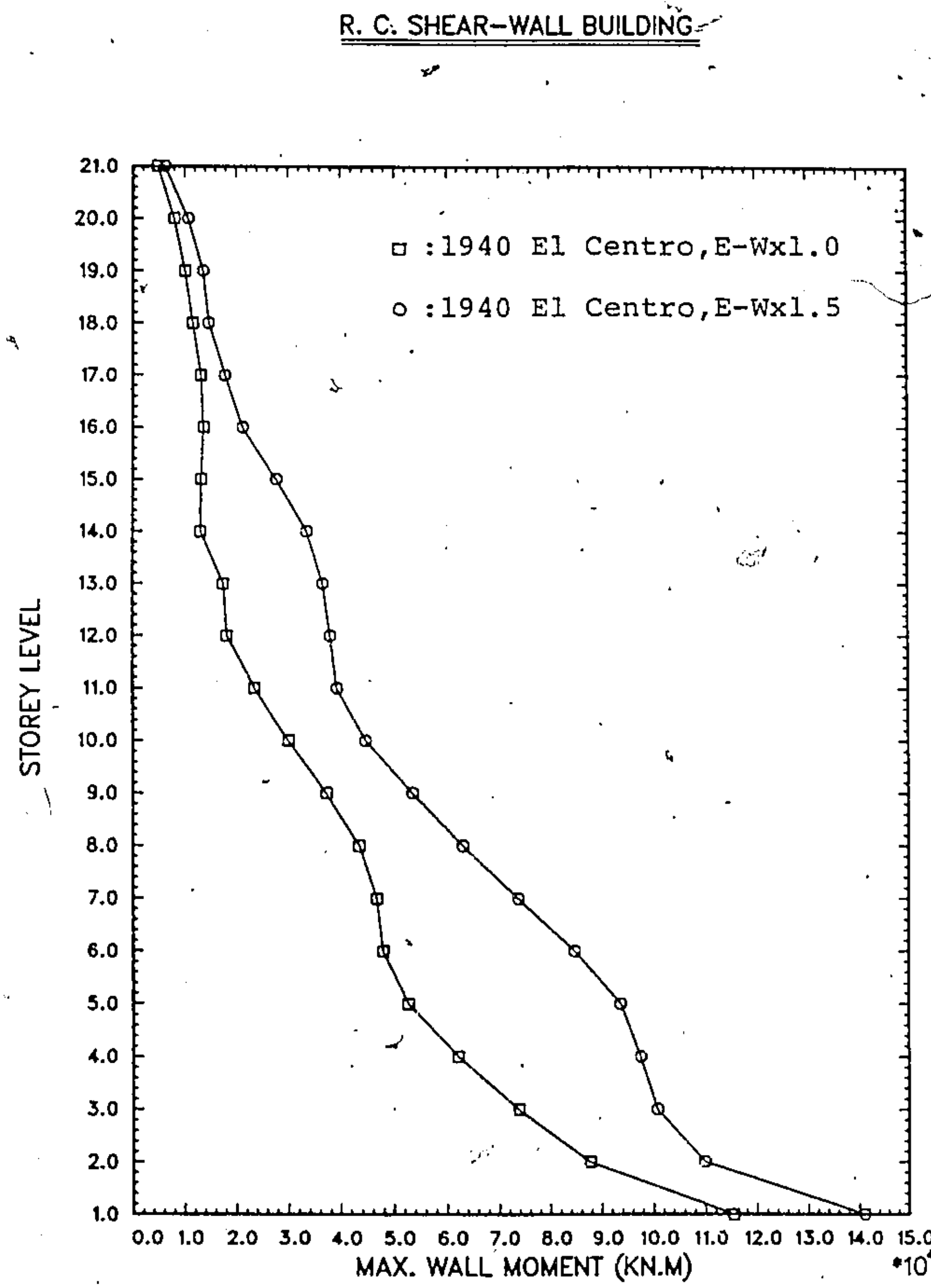


Fig.5.20 MAXIMUM WALL MOMENTS FOR THE SHEAR-WALL Bldg.

R. C. SHEAR-WALL BUILDING

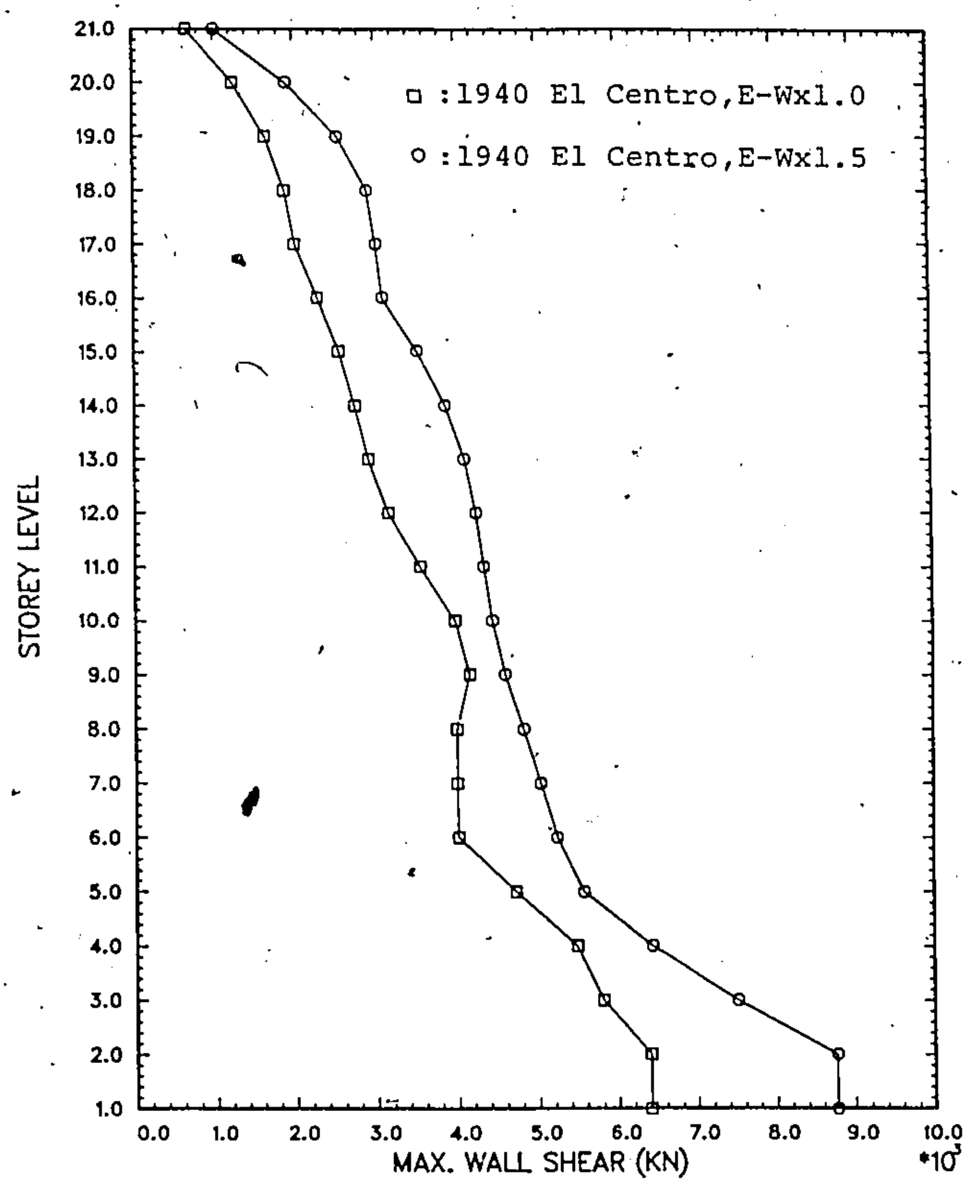


Fig.5.21 MAXIMUM WALL SHEARS FOR THE SHEAR-WALL Bldg.

R. C. SHEAR-WALL BUILDING

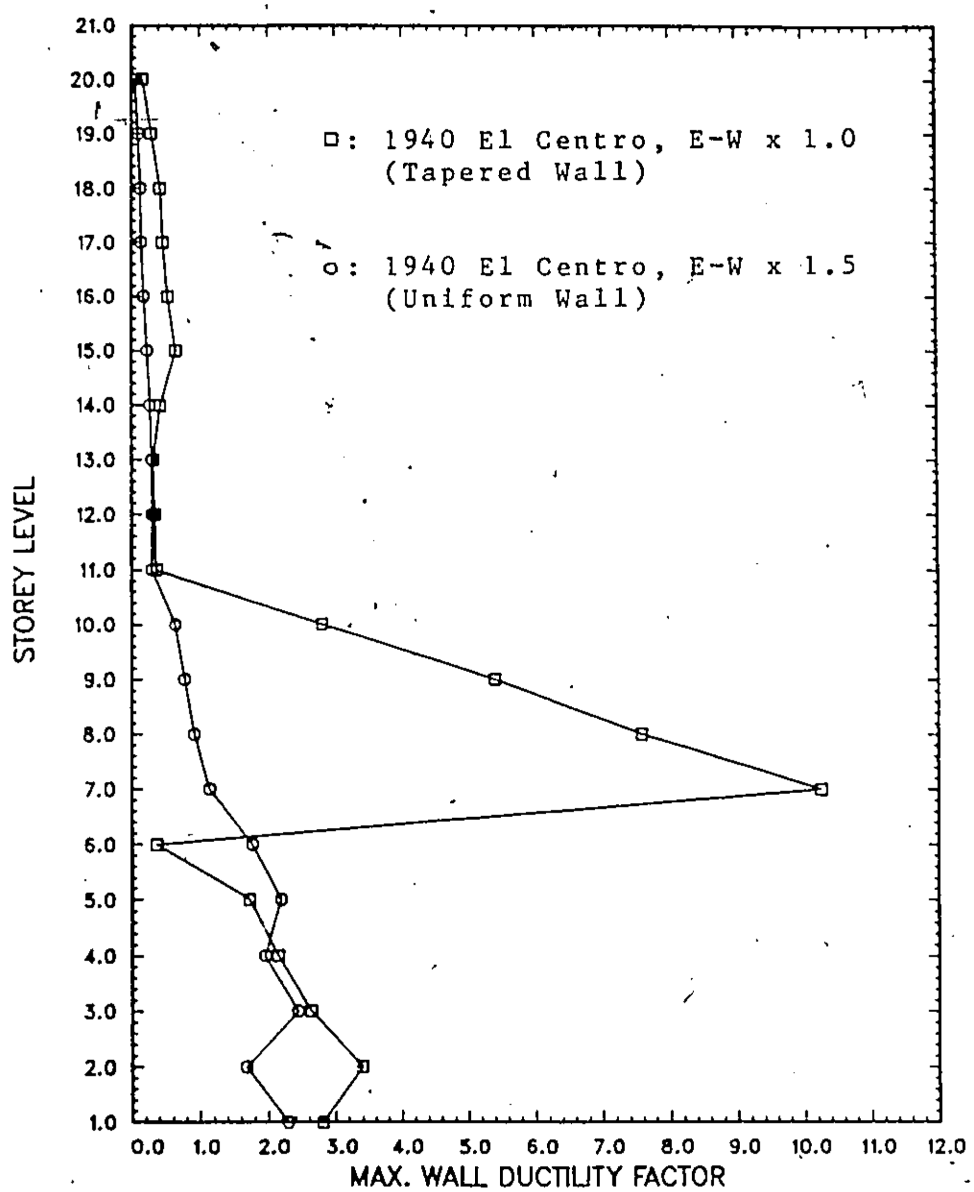


Fig.5.22 MAXIMUMWALL DUCTILITY FACTORS (SHEAR-WALL Bldg.)

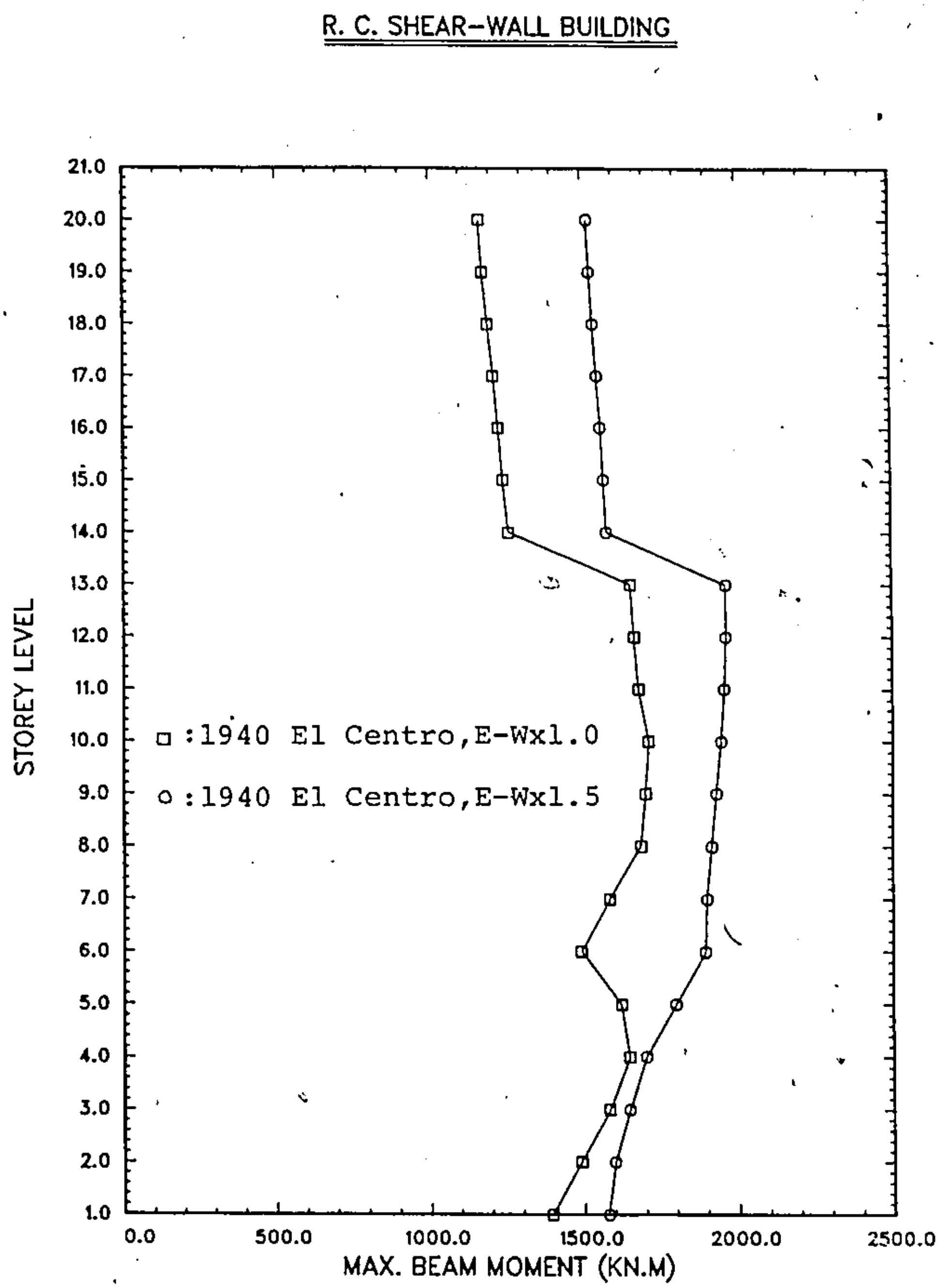


Fig.5.23 MAXIMUM COUPLNG BEAM MOMENTS (SHEAR-WALL Bldg.)

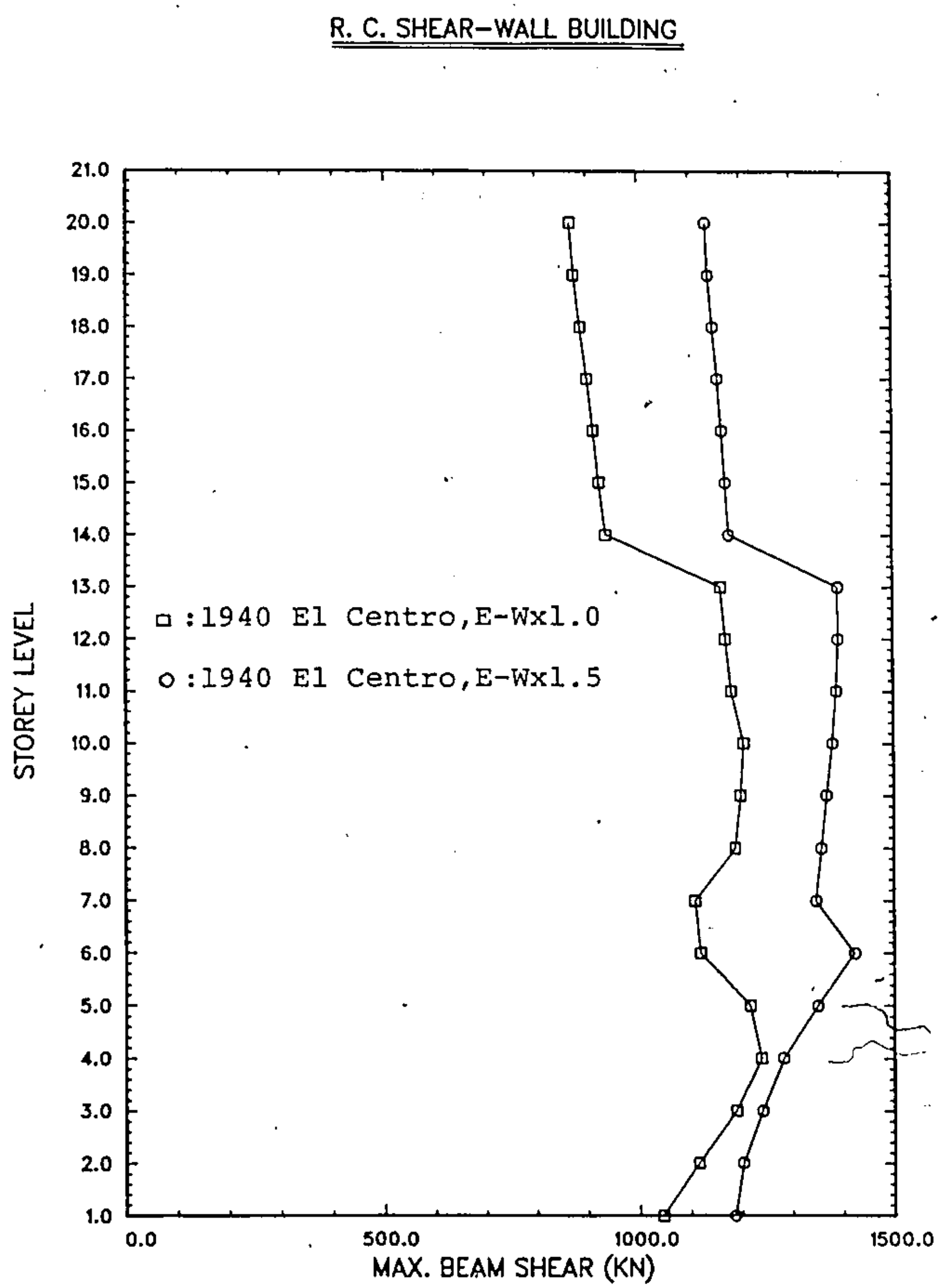


Fig.5.24 MAXIMUM COUPLING BEAM SHEARS (SHEAR-WALL Bldg.)

R. C. SHEAR-WALL BUILDING

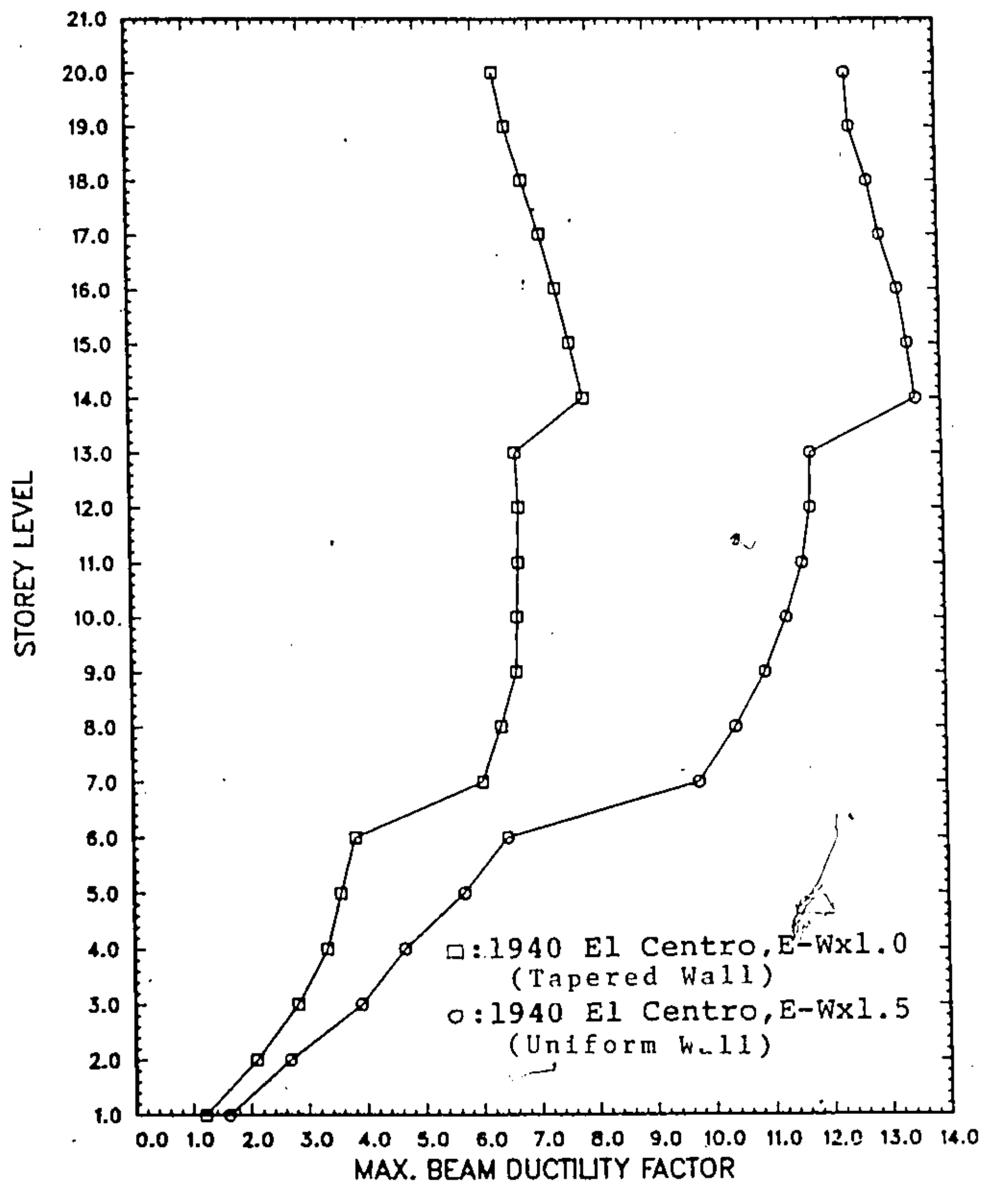


Fig.5. 25 MAXIMUM COUPLING BEAM DUCTILITY FACTORS

(SHEAR-WALL Bldg.)

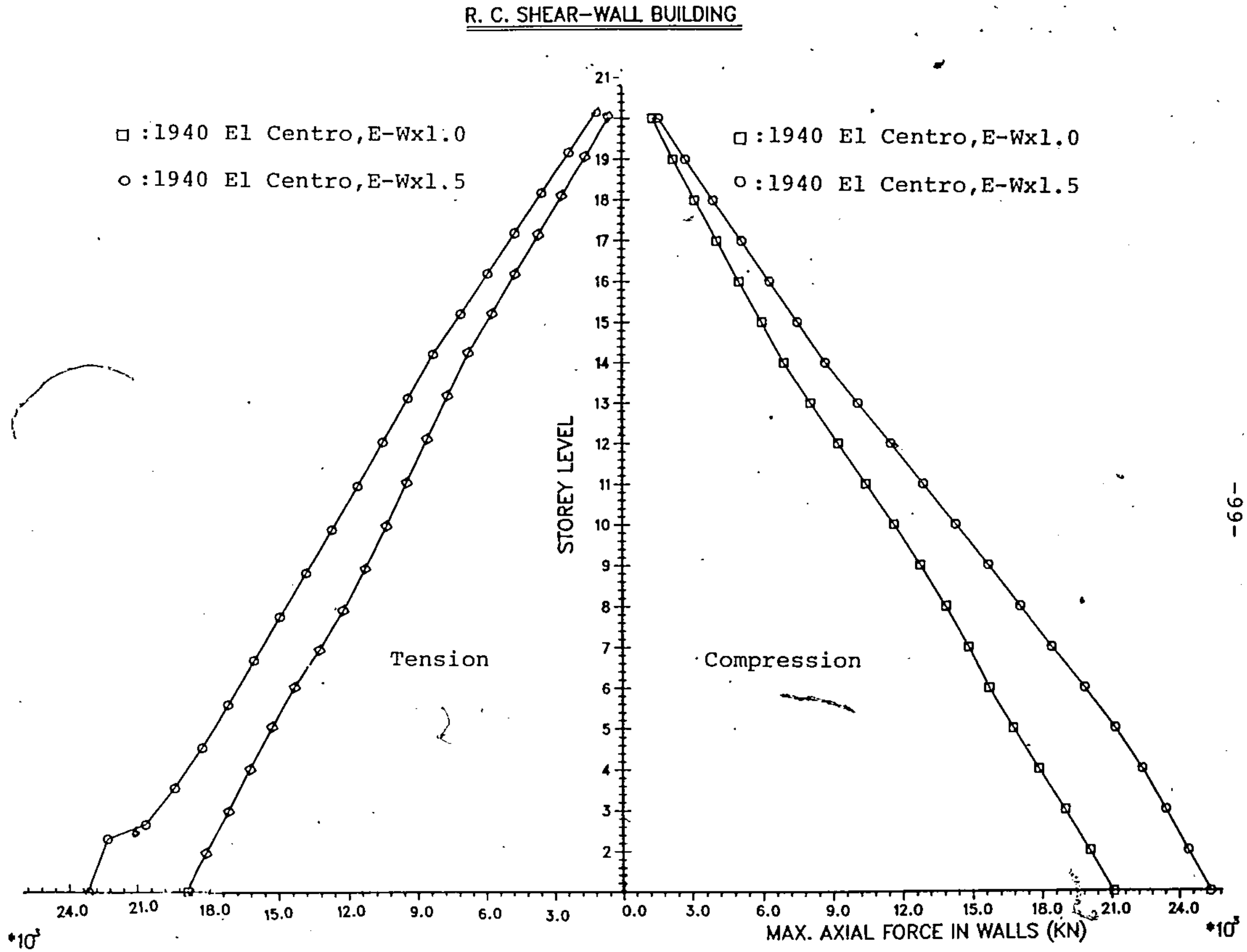


Fig.5.26 MAXIMUM AXIAL FORCE IN WALLS (SHEAR-WALL BUILDING)

TABLE 5.1 MEMBER PROPERTIES FOR FRAME STRUCTURE

	NBCC-85	UBC-82
Column Stiffness Parameters:		
EI(10^3 KN.m^2)	1141	1141
GA(10^3 KN)	2331	2331
EA(10^3 KN)	13693	13693
Beam Stiffness Parameters:		
EI(10^3 KN.m^2)	427.2	427.2
GA(10^3 KN)	833	833
EA(10^3 KN)	9243	9243
Column Yield Moment (KN.m):		
at balanced point:		
-at base	6003	6003
-at 7th floor	4981	4981
-at 15th floor	4276	4276
at zero axial load:		
-at base	3721	3721
-at 7th floor	2238	2238
-at 15th floor	1522	1522
Column Axial Load Capacity: (KN)		
at balanced point:		
-at base	12009	12009
-at 7th floor	11619	11619
-at 15th floor	11501	11501
at zero moment:		
-at base	12320	12320
-at 7th floor	7840	7840
-at 15th floor	4480	4480
Beam Yield Moment : (KN.m)	4603	4603
Mass per Floor: (Kg) at roof	37.61	37.61
at 7th floor	46.79	46.79
at base	49.23	49.23

TABLE 5.2 MEMBER PROPERTIES FOR SHEAR WALL STRUCTURE

	NBCC-85	UBC-82
Wall Stiffness Parameters:		
EI(10^3 KN.m^2)-at base	381970	381970
GA(10^3 KN) -at base	11193	11193
EA(10^3 KN) -at base	65743	65743
Stiffness Taper[1] (Step Variation)	1.00 EI at base 0.70 at 7th floor 0.52 at 15th floor	
Beam Stiffness Parameters:		
EI(10^3 KN.m^2)-at base	200.70	200.70
-at 7th floor	151.80	151.80
-at 15th fl.	122.13	122.13
GA(10^3 KN)-at base	748.456	748.456
-at 7th floor	520.665	520.665
-at 15th fl.	390.499	390.499
EA(10^3 KN)-at base	6558.98	6558.98
-at 7th floor	5121.21	5121.21
-at 15th fl.	4299.62	4299.62
Wall Yield Moments: (KN.m)		
-at base	74932	74932
-at 7th floor	46006	46006
-at 15th floor	31285	31285
Beam Yield Moments: (KN.m)		
-at base	1290	1290
-at 7th floor	1010	1010
-at 15th floor	825	825
Mass per floor: (Kg)		
-at base	251.68	251.68
-at 7th floor	237.21	237.21
-at 15th floor	234.66	234.66
-at roof	204.59	204.59

Note:

1 The same taper also applies to "GA" and "EA"

CHAPTER 6

DISCUSSION AND COMPARISON OF RESULTS

6.1 - General :

The results presented in chapter 4 show considerable differences in both the design and analysis aspects of the two codes. The differences and similarities between the final designs are discussed in this chapter. Emphasis is placed on the reasons leading to the differences in final designs.

6.2 - Comparison of Analysis Procedures :

Each of the two building codes considered provides a formula for base shear to be used in the Equivalent Lateral Load Analysis Method. The differences in the final base shear value are made up of contributions from different components of the base shear formula. The distribution of seismic base shear along the height of the structure, load combinations, consideration of simultaneous occurrence of seismic forces in orthogonal directions, second order effects and many other aspects of analysis, when considered in the text of the two codes, bear some differences as discussed below.

The first factor of the two base shear formulae relates to the severity of ground motion. In general this factor represents peak ground acceleration ratios and amplitudes. This basic definition loosely fits the two codes. In NBCC 1985 factor v, and in UBC 1982 factor Z is used for this purpose.

The size of the base shear and resultant lateral loads is also a function of the building use. Both NBCC-85 and UBC-82 include an importance factor in their base shear equation. These two codes recognize the need for some structures to remain operative after the earthquake, such as those for public services and hospitals. The role of importance factor (I) in NBCC-85 or UBC-82 is implicitly being performed by detailing requirements for different 'Seismic Performance Categories'. Depending upon the severity of seismic zone and the importance of the building, its seismic performance category is fixed. The importance factor assigned to the structures considered in this investigation was 1.0 as the structures were not deemed to be post-disaster buildings.

The ductility and damping ability of a structure is of utmost importance in determining structural response to earthquake motion. With this in mind, it stands to reason that a structure's ductility should play a part in determinig the base shear and lateral loads for which it is to be designed. This fact is recognized in the building codes. Factor "K" in the base shear equations of NBCC-85 and UBC-82 accouts for the ductility and damping ability of a structure. The "K" factor also reflects the redundancy of the load path in a structure. Redundancy in a structure is a desirable characteristic. The resulting "K" factor for the frame structure considered were 0.7 and 0.67 based on NBCC-85 and UBC-82 respectively. The "K" factor for the shear-wall structure was 0.8 for both NBCC-85 and UBC-82.

The seismic response factor "S", and the numerical coefficient "C" are the literals given by NBCC, and UBC respectively, to the next factor in the base shear equations. The basic function of this factor is to reflect the dependence of seismic acceleration on the fundamental period of the structure. Both factors are a function of the period of the structure, "T". NBCC and UBC show the factor to be a function of the square root of "T". In order to obtain the value of the seismic response factor, "T" must first be estimated. For preliminary sizing, it is advisable that the base shear and the value of T be conservative. Thus, the value T should be smaller than the actual period T of the structure. Given this, the equation for the estimation of T given in the codes result in conservative estimates of the building period. The value of T is expressed as a function of building height in the codes. It should be remembered however, that these expressions provide only an approximate T. It is recommended that, wherever possible, the fundamental period of the building, T, should be determined based on the properties of the seismic resisting system.

Foundation properties of a structure can have a sizeable effect on the characteristics and magnitude of earthquake motions. Soils such as soft to medium-stiff clays can actually amplify the motions in certain frequency ranges. This can result in more intense shaking due the development of quasi resonance state between the structure and soil [6]. In order to assess the possibility of poor soil conditions increasing earthquake effects, each of the two codes includes a foundation factor in their base shear equation. The structures considered were assumed to be resting on solid bedrock. Thus, the resulting foundation factor for the two codes was 1.0 sec.

The factors listed above lead to different base shears when calculated using different code procedures. For the structures chosen, the variation of base shear coefficient with respect to i) the period of the building, and ii) the height of the building are shown in figures 5.6, 5.23, and 5.24 for the two structures considered.

Since the early development of seismic codes in the 1920's the base shear has been a function of the buildings dead weight. Today other factors have been added, some have been changed, but the base shear equations still incorporates W (building weight). The value of W includes all the dead weight of the structure including partitions, ceiling and permanent equipment. This weight also includes a fraction of the roof snow load where applicable. The resulting base shear values found for NBCC are considerably smaller than the ones for UBC.

The final step in the equivalent lateral load procedure of the two codes involves the distribution of the base shear along the building height. NBCC and UBC adopt a linear distribution with a concentrated force (F_t) at the top of the building. This additional concentrated force, used for slender buildings, accounts for a redistribution of load to the top storeys due to increases in the top storey amplitudes. The distribution of shears along the height of each selected structure are compared in figures 5.7 and 5.25. The complete calculation, and use of the distribution equations for NBCC and UBC are presented in Appendix A.

6.3 - Design Differences :

The basic philosophy of seismic design is the same in both codes. Each code requires the building to have ductility proportional to the reduction in the design elastic seismic force acting on the building. The codes recommend that the energy dissipation during an earthquake should occur in the beams, leaving the gravity load supporting columns mostly unaffected. Both codes require the joints to remain rigid during an earthquake. In spite of, the degree of concern towards these design objectives, the final designs obtained by the use of the two codes may be different. Sometimes the provisions of a given code may result in a large base shear. But, the same code may have varying (lower) load factors and load combination factors, thus resulting in comparable design forces and final designs.

This section discusses the implications of the design provisions of the two concrete design codes on the final design of structures previously mentioned.

6.3.1 - Beams :

In the calculation of design shear, the two codes recommend a steel stress of $1.25f_y$, basically because of the likelihood of strain hardening at hinge locations as well as the possibility of having an actual yield stress in excess of the design yield stress. The governing load cases were found to be 1.25D+1.5E, and 0.75(1.4D+1.7L+1.87E) for NBCC and UBC respectively.

Calculation of the contribution of concrete to shear resistance is different in the two codes. NBCC simplified method of shear design requires the shear carried by concrete to be zero. In the UBC, the shear carried by concrete should be taken as zero only if the following two conditions are satisfied simultaneously:

i) seismic shear on the member is greater than half of the design shear on the member, and

ii) the axial compressive force on the member is less than $0.05f'_cA_g$.

The code similarities also extend to the requirement of positive moment resistance at column connections. This requirement proved to be the governing one in the design of positive reinforcement of the beams.

The limitations on the geometry of the cross section as set forth by CAN3-A23.3-M84 are comparable to those given in ACI-318.83. The lower limit of four on length-to-depth ratio is set to avoid the stubby components whose shear strength is known to deteriorate at a faster rate under load reversals. The width-to-depth is set to encourage compact cross-sections having a reasonably low risk of lateral instability in non-linear range of response. The maximum width limitation is related to the problem of efficient transfer of moment from girder to column.

The special confinement reinforcement is required over a distance of 2d from each support face in both codes.

Because of the similarities in the ACI-318.83 and the CAN3-A23.3-M84 Building Codes, in terms of flexural members, the final designs of beams do not show a significant difference. The final designs, as can be seen in Fig 4.11 through 4.17, call for approximately the same flexure and shear reinforcement. Total required steel, in kilograms is given in Table 6.1. The results indicate that the Canadian Code calls for approximately 4% more steel in beams as compared to those designed by using the American Code.

6.3.2 - Columns :

As for the other structural members, column desgin by ACI-318.83 basically differs from that by CAN3-A23.3-M84 in its design for shear. Like for the beams, the governing load cases for exterior columns were found to be 1.25D+1.5E, and 0.75(1.4D+1.7L+1.87E) for NBCC and UBC respectively. For interior columns the governing load cases were found to be 1.25D+1.5L and 1.4D+1.7L for NBCC and UBC respectively.

The design philosophy behind the column shear provisions of the two codes is similar. The requirement of strong column weak beam is found in both codes. The basis of the column design shears is determined from a consideration of the static forces of the joints calculated without capacity reduction factors and the maximum axial compressive design force on column. CAN3-A23.3-M84 has an additional provision, regarding column confinement at the ground floor. Accordingly the column at the base of the structure should be confined over its full height. As explained earlier in the design of girders, ACI-318.83 prescribes that the shear carried by concrete should be taken as zero under certain conditions.

All of the different provisions stated above, produce different shear design by different codes. In the present examples, columns designed by the two codes do not require any shear reinforcement. Only the minimum tie steel is provided. The increased steel area by CAN3-A23.3-M84 for column confinement at base of structures is justified by the fact that the performance of structures in actual earthquakes and results of non-linear dynamic analyses indicate that these columns tend to undergo considerable inelastic deformations.

The special detailing requirement of crosstie engaging the peripheral longitudinal reinforcing bar, is similar in the two codes. This special requirement enhances the strength and ductility of the confined core of the column. The main function of the lateral reinforcement is to provide confinement for the concrete and lateral support for the reinforcement. The amount of transverse reinforcement so required may also be used to resist shear.

Because of differences in the ACI-318.83 and the CAN3-A23.3-M84 Building Codes, the final designs of columns show certain differences. The final designs, as can be seen in Fig 4.18 through 4.21, call for approximately the same flexure reinforcement, however shear reinforcement is more than 50% for CAN3-A23.3-M84. Total required steel in kilograms is given in Table 6.1. The results indicate that the Canadian Code calls for approximately 6% more steel in columns as compared to those designed by using the American Code.

6.3.3 - Joints :

Under the action of an earthquake, the beam-column connections are one of the design engineers greatest concerns. All the two codes under consideration recognize the need for proper transverse reinforcement for the beam-column joint. In addition to the column confinement and shear steel, further transverse reinforcement is required by the two codes, in the area of the beam column connection. The design shear force is calculated in a manner similar to the regular column shear, by taking into account column shears and shear due to yield forces of the beam reinforcement. These design recommandations require that the transverse reinforcement provide the difference between the joint shear and the shear carried by concrete. As in the case of beams, the two codes require that all calculations require a steel yield stress of $1.25f_y$ and no capacity reduction factor. The two codes restrict the design joint shear stress to a maximum allowable joint shear stress given in Table 4.2. It is further required to continue the confinement reinforcement of columns into the framing joint. This requirement is based on the evaluatios of the existing data on strength of joints subjected to moment reversals. These tests indicated that the strength of the joint is relatively insensitive to the amount of transverse reinforcement, provided there is a minimum amount of steel and the shear stress on the joint is limited to the prescribed values.

It is true that without a beam on the exterior face there is less reinforcement to produce yielding induced shear. However, because the beam reinforcement is not continuous, but rather it ends at the column, all of it must pass through the column and firmly anchored in the column confined region. With the interior columns, code provisions stated that the beam reinforcement could be spread within specified limits in the flange of the beams. This allowed some of the beam reinforcement to pass around the column and reduce the design shear force.

The provisions of the two codes are similar in their determination of the allowable concrete shear stress V_c. The equations given by the codes are a function of the axial load.

The final designs are given in Fig 4.22 and 4.23. Total required steel in kilograms is given in Table 6.1. The results indicate that the Canadian Code calls for approximately 9% more steel in joints as compared to those designed by using the American Code.

6.3.4 - Shear Walls :

The comparison of shear wall designs based on the two codes indicates, large differences in reinforcement requirements. ACI-318.83 requires an average of about twice the reinforcement required by CAN3-A23.3-M84 wall design. This can be seen in Table 6.1. The reasons behind these differences can be understood by examining the code provisions. In the CAN3-A23.3-M84 requirements for coupled shear walls, walls are required to be proportioned so that a significant amount of overturning moment is resisted by axial loads resulting from vertical shear in the coupling members. The provisions of CAN3-A23.3-M84 recognizes the need for both distributed and concentrated reinforcement that varies substantially from ACI-318.83.

Concentrated reinforcement in CAN3-A23.3-M84 is based on providing adequate reinforcement to resist all factored load effects including earthquake effects.

According to the ACI code, boundary elements are provided at boundaries of structural walls for which the maximum extreme-fiber stress , corresponding to factored forces including earthquake effects, exceeds $0.2f'_c$. A calcualted compressive stress of $0.2f'_c$ in a member is assumed to indicate that integrity of the entire structure is dependent on the ability of that member to resist substantial compressive forces under severe cyclic loading.

The governing load case for the design of ACI shear wall was found to be $f_c > 0.2f'_c$. This resulted in the shear wall having boundary elements designed to resist the total load effects. Adequate lateral reinforcement are required to provide concrete confinement in the boundary elements.

The governing load case for CAN3-A23.3-M84 was 1.25D+1.5E while for ACI-318.83 it was 0.9D+1.43E. The final consideration in the design of the shear wall main reinforcement is the size of the loads. Tab.A.2 in Appendix A shows a considerable difference in the base shear values of the two codes. The high rigidity of the shear walls attract the majority of this shear. This factor in itself can change the reinforcement required by a given code.

The provisions for shear in CAN3-A23.3-M84 include possible excess shear due to overdesign. One of the major concerns of most seismic codes is the ductility of a structure. The more ductile a structure the greater the possibility of dissipating earthquake induced energy. A shear wall that is overdesigned for moment resistance can attract more shear force than that for which it was designed. In such cases, additional shear capacity is needed to prevent premature shear failure.

The horizontal distributed steel of both codes was calculated as $0.0025A_g$. This proved to be more than sufficient to resist the shear forces.

CAN3-A23.3-M84 code requires special confinement reinforcement for the concentrated steel for plastic hinge region. ACI-318.83 requirements called only for regularly spaced confinement steel similar to column design.

The shear wall coupling beams produced similar amount of steel reinforcement. The final wall and coupling beam designs are shown in Figs 4.18 through 4.20. The comparisons of the amount of reinforcement required by each code are given in Table 6.1.

6.4 - Structural Performance Under Dynamic Loading :

The structures designed on the basis of the building codes used in this investigation were analysed under a recorded motion. The details of the dynamic inelastic analyses are provided in chapter 5. Performance of these structures as indicated by the analysis results are discussed in this section.

6.4.1 - Frame Structure :

The initial analysis of the frame structure under the 1940 El Centro , E-W record indicated excessive compression in the first storey columns. The moment and the axial force generated in the first storey columns showed a capacity demand that was more than the capacity required by the building codes. Therefore the first analysis indicated crushing of columns under the given earthquake motion.

In the subsequent analyses, the column capacity was increased to prevent column failure. It was found that an increase of approximately 35% of the Canadian Code requirement and 10% of the American Code requirement produced elastic column response. The results also indicated beam rotational ductility requirements of up to 4.0.

However, the E-W component of the 1940 El Centro record is generaly considered a representative of a medium intensity eartquake. Therefore the same analysis was repeated under a 50% increased intensity. The accelerogram was multiplied by a factor of 1.5 for this purpose. In this case, the anlysis results indicated approximately 100% and 60% increase in the first storey column capacities relative to the Canadian and American codes respectively. The maximum beam ductility requirement was increased to 6.0, indicating a need for increased beam capacity.

The results indicate that the member strength required by the American code is closer to the requirements indicated by dynamic response history analyses.

6.4.2 - Shear Wall structure :

The shear wall structure designed on the basis of the Canadian code was first analysed using the 1940 El Centro E-W record. The results indicated excessive tension in the walls producing net axial tension sufficient to cause yielding of the section. Subsequent analyses with increased wall capacity did not significantly improve the results. Therefore it was concluded that the strength requirement of the Canadian code is substanqially less than the requirement indicated by dynamic response history analysis.

The shear wall structure designed on the basis of the American code showed a favorable behaviour under dynamic loading. However, the results indicated excessive yielding at the seventh floor level due to strength and stiffness taper. In the subsequent analyses, uniform wall thickness was used and this problem was prevented. Increased intensity of earthquake produced high ductility demands in both the walls and the coupling beams. A 50% increase in wall strength was required to obtain a wall ductility demand of 2.3.

The results indicate that the capacity requirements of the American code for shear wall structures are closer to those indicated by dynamic response history analysis than those of the Canadian code. The results further indicate that walls with uniform stiffness (or thickness) along the structure height perform better under dynamic load conditions than those with stiffness taper.

TABLE 6.1 AMOUNT OF STEEL REQUIRED

FRAME

BEAMS :

	Top bars (Kg)	Bot. bars (Kg)	shear steel (Kg)	joint steel (Kg)	Total (Kg)
CAN3-A23.3-M84	881.2	624.5	166.9	1767.6	3440.2
ACI-318.83	945.4	579.4	171.5	1616.4	3312.7

COLUMNS :

	Ext. col. (Kg)	Int. col. (Kg)	Trans. steel ext.col. (Kg)	Trans. steel int.col. (Kg)	Total (Kg)
CAN3-A23.3-M84	1276.8	1915.3	628.8	813.3	9268.2
ACI-318.83	1474.4	2211.6	290.0	406.0	8763.9

SHEAR-WALL

Wall: Concentrated reinforcement:	CAN3-A23.3-M84	ACI-318.83
Longitudinal steel:	942	2072
Lateral steel:	446	728
Distributed steel:		
Vertical steel	279	279
Horizontal steel	171	279
Coupling Beam:		
Longitudinal steel	118	71
Lateral steel	50	50
TOTAL STEEL (Kg):	2006	3480

CHAPTER 7

CONCLUSIONS

Differences in the seismic risk zones meeting at the American, Canadian border are the first sign of existing differences in the seismic codes of the two countries. It is unlikely that the intensity of earthquake ground motions changes substancially on either side of the border. The performance of a building at the border between Canada and the United States, depends upon whether it is designed by the Canadian code or the U.S. code. Even though the general philosophy and the development of the two codes are similar, there are certain differences at different stages in the design process.

The design of the structures studied, indicates that the basic design philosophy of each code to be similar. Basic provisions such as basing beam shear design on the formation of plastic hinges are found in both codes. However, the similarity in philosophy and basic provisions did not prevent design differences. The difference in designs ranged from insignificant to considerable. Some of the major observations in this study are as follows:

1. The base shears resulting from UBC-82 are larger than the NBCC-85 values.

For a 20-storey ductile moment resistant reinforced concrete frame, UBC-82 base shear is about 10 percent more than the NBCC-85 base shear. On the other hand for a 20-storey shear wall building, UBC-82 base shear is about 15 percent more than the NBCC-85 base shear.

2. In the UBC-82, calculation of the foundation factor in the formula for design seismic base shear could give different values for different site periods.
3. The base shear due to wind loads for the structure as calculated by the NBCC-85 wind pressure zoning maps was about 45 percent larger than that calculated using the specified design wind pressures for the particular site as provided by the UBC-82.
4. In general ACI-318.83 required larger amount of reinforcement for the shear wall structures.
5. The majority of CAN3-A23.3-M84 additional reinforcement was for resisting shear for the case of joints.
6. In general shear requirements of the two codes are similar.
7. The design provisions for shear walls for both codes are similar in their requirements of boundary elements, shear, and confinement reinforcement.
8. The shear wall design of ACI-318.83 resulted in more than twice the vertical concentrated reinforcement for CAN3-A23.3-M84 mainly due to the increased design loads.
9. With the exception of shear walls the final ACI-318.83 and CAN3-A23.3-M84 designs are similar.
10. Structures designed on the basis of the American code showed better performance under dynamic loading.

APPENDIX A

COMPUTATION OF LOADS

A.1- Material Properties :

Concrete strength (28 days) : 30 MPa

Steel strength : 400 MPa

A.2- Member sizes :

Frame structure:

Columns : 1000 mm x 1000 mm - Ground to 6th floor

750 mm x 750 mm - 7th to 13th floor

500 mm x 500 mm - 14th to 20th floor

Beams : 500 mm wide, and 750 mm deep

Slab : 200 mm thick

Storey height : 4m typical floor

6m 1st floor

Shear-Wall Structure:

Walls : 575 mm - 1st floor to 6th floor

400 mm - 7th floor to 14th floor

300 mm - 15th floor to 20th fl.

Columns : 350 mm x 500 mm - Base to 10th floor

250 mm x 400 mm - 10th to 20th floor

Girders : 350 mm x 500 mm

Spandrels: 300 mm x 500 mm

Coupling Beams: 575 mm x 600 mm - 1st to 6th floor

400 mm x 600 mm - 7th to 14th floor

300 mm x 600 mm - 15th to 20th floor

Slab : 200 mm thick

Storey height : 4m typical floor

6m 1st floor.

Live Loads :

- Typical floor : 2.4 KN/m^2
- Roof : 1.0 KN/m^2
- Snow Load : 1.9 KN/m^2 (Vancouver)

Dead Loads :

- Concrete : 2400 Kg/m^3
- Partitions : 0.65 KN/m^2
- Ceilings : 0.35 KN/m^2
- Roofing : 0.35 KN/m^2

A.3 - Wind Loads :

NBCC-85 :

$$P = qC_eC_gC_p$$

Where:

P= specified external pressure

q= reference velocity pressure = $0.55 KN/m^2$ (Vancouver)

C_e= exposure factor = 1.5

C_g= gust effect factor = 2.0

C_p= external pressure coefficient = 0.7-(-0.5) = 1.2

$$P = 0.55 \times 1.5 \times 2.0 \times 1.2 = 1.98 KN/m^2$$

$$V_{base} = 1.98 \times 48.0m \times 82.0m = 7793.3KN$$

UBC-82 :

$$P = C_eC_qq_sI$$

Where:

P= design wind pressure

C_e= exposure and gust factor coefficient for severe exposure= 2.1

C_q= pressure coefficient= 0.8

q_s= wind stagnation pressure= 17 psf

I= importance factor= 1.0

$$P = 2.1 \times 0.8 \times 17 \times 1.0 = 28.6\ psf$$

structure width = 48.0m x 3.281 = 157.5 ft

structure height = 82.0m x 3.281 = 269.0 ft

structure location = Seattle, Washington

$$V = 28.6 \times 269.0 \times 157.5 \times 1/1000 = 1212\ Kips$$

$$V = 1212/0.2248 = 5390\ KN$$

A.4 - Calculation of Seismic Loads :

Equivalent Lateral Load procedure is chosen as the method of lateral load analysis. The lateral loads are applied in the form of concentrated loads at each storey level.

A.4.1 - Base Shear by NBCC-85 :

According to section 4.1.9 of Ref. [4], the minimum lateral seismic force, V, assumed to act nonconcurrently in any direction on the building shall be given by:

$$V = v.S.K.I.F.W \qquad \text{(A-1)}$$

Where:

v= Zonal velocity ratio

S= Seismic response factor $=0.22/\sqrt{T}$

T= fundamental period of vibration of the building in seconds

T= 0.1N (for frame structures)

$T = 0.09h_n/\sqrt{D_s}$ (for shear wall structures)

N= Total number of storeys

h_n= Height above the base to level n

D_s= Dimension of the lateral force-resisting system in a direction parallel to the applied forces.

K= Numerical coefficient that reflects the material and type of construction, damping, ductility of the structure.

I= Importance factor.

F= Foundation factor.

W= Weight of the structure.

The lateral seismic force shall be distributed along the height of the building in accordance with the following formula:

$$F_x = (V - F_t)W_x h_x / (\Sigma_{i=1}^{n} W_i h_i) \qquad \text{(A-2)}$$

Where:

F_x= Portion of V to be concentrated at the top of the structure.

h_i, h_n, h_x= Height above the base (i=0) to level "i", "n", or "x", respectively.

W_i, W_x= Portion of W which is located at or assigned to level "i" or "x", respectively.

$$F_t = 0.004V(h_n/D_s)^2 \leq 0.15V \qquad \text{(A-3)}$$

a) For the frame structure situated in zone 4:

v = 0.20

S = 0.1556

T = 2.0 sec.

K = 0.7

I = 1.0

F = 1.0

W = 263930 KN

The base shear is given by:

$$V = 0.20 \times 0.1556 \times 0.7 \times 1.0 \times 1.0 \times 263930$$

$$V = 5749.45\ KN$$

Portion of load concentrated at top of structure is given by:

$$F_t = 0.004 \times 5749.45(82.0/24.0)^2$$

$$F_t = 268.47\ KN$$

$$0.15V = 0.15 \times 5749.45$$

$$0.15V = 862.42\ KN$$

$$F_t < 0.15V$$

$$268.47KN < 862.42KN \qquad O.K.$$

V is distributed along the height of the building according to formula (A-2) (see Table A-1).

<u>b) For the shear-wall structure situated in zone 4 :</u>

v = 0.20

S = 0.1692

T = 1.69 sec.

k = 0.8

I = 1.0

F = 1.0

W = 206731 KN

$$V = 0.20 \times 0.1692 \times 0.8 \times 1.0 \times 1.0 \times 206731$$

$$V = 5596.62\ KN$$

V is distributed along the height of the building according to formula (A-2) (see Table A-2).

A.4.2 - Base Shear by UBC-82 :

According to section 2312 of Ref. [5], the building shall be designed to resist minimum total lateral seismic force assumed to act non-concurrently in the direction under consideration of the building in accordance with the following formula:

$$V = Z.I.K.C.S.W \qquad \text{(A-4)}$$

Where:

Z= Numerical coefficient dependent upon the zone.

I= Occupancy importance factor.

K= Numerical coefficient.

C= Numerical coefficient $= 1/(15\sqrt{T})$

T= Fundamental elastic period of vibration of the building in seconds.

T= 0.1N (for frame structures)

T=$0.05h_n/\sqrt{D}$ (for shear wall structures)

N= Total number of stories.

S= Numerical coefficient for site-structure resonance.

W= Total dead load of the building.

The total lateral force shall be distributed over the height of the building in accordance with formula (A-5):

$$F_t = 0.07TV \qquad \text{(A-5)}$$

$$F_t \leq 0.25V$$

<u>a) For the frame structure situated in zone 3:</u>

Z = 3/4

I = 1.0

K = 0.67

C = 0.0471

T = 2.0 sec.

S = 1.0

W = 263930 KN

$$V = 3/4 \times 1.0 \times 0.67 \times 0.0471 \times 1.0 \times 263930$$

$$V = 6246.63\ KN$$

$$F_t = 0.07TV \leq 0.25V$$

$$F_t = 0.07 \times 2.0 \times 6246.63$$

$$F_t = 874.53\ KN$$

$$0.25V = 1561.66\ KN$$

$$874.53KN < 1561.66\ KN$$

V is distributed along the height of the building according to formula (A-2) (see Table A-1).

<u>b) For the shear-wall structure situated in zone 3 :</u>

Z = 3/4

I = 1.0

K = 0.80

C = 0.051

T = 1.70 sec.

S = 1.0

W = 206731 KN

$$V = 0.75 \times 1.0 \times 0.80 \times 0.051 \times 1.0 \times 206731$$

$$V = 6325.97\ KN$$

V is distributed along the height of the building according to formula (A-2) (see Table A-2).

For the building designed by NBCC, the lateral load is governed by the wind load, whereas for UBC is governed the seismic load.

However, for sake of comparison wind loads were assumed not to govern.

TABLE A.1 DISTRIBUTION OF BASE SHEAR OVER THE HEIGHT OF THE FRAME BUILDING

Level	Height h (m)	Weight W (KN)	Lateral Seismic Force Fx (KN)	
			NBCC-85	UBC-82
Roof	82	10740	699.36	1296.86
20	78	12530	478.18	468.68
19	74	12530	453.66	444.65
18	70	12530	429.14	420.61
17	66	12530	404.61	396.58
16	62	12530	380.09	372.54
15	58	12530	355.57	348.51
14	54	12530	331.05	324.47
13	50	13250	324.14	317.70
12	46	13250	298.21	292.28
11	42	13250	272.28	266.87
10	38	13250	246.35	241.45
9	34	13250	220.41	216.04
8	30	13250	194.48	190.62
7	26	13250	168.55	165.20
6	22	14300	153.92	150.86
5	18	14300	125.94	123.43
4	14	14300	97.95	96.00
3	10	14300	69.96	68.56
2	6	15530	45.59	44.68
1	0	0	0.00	0.00
Total		263930	5749.44	6246.59

TABLE A.2 DISTRIBUTION OF BASE SHEAR OVER THE HEIGHT OF THE SHEAR-WALL BUILDING

Level	Height h (m)	Weight W (KN)	Lateral Seismic Force Fx (KN) NBCC-85	UBC-82
Roof	82	8996	841.6	1209.8
20	78	10321	463.4	498.5
19	74	10321	439.6	472.9
18	70	10321	415.8	447.4
17	66	10321	392.1	421.9
16	62	10321	368.3	396.2
15	58	10321	344.6	370.7
14	54	10321	320.8	345.2
13	50	10321	297.0	319.6
12	46	10321	273.3	294.0
11	42	10321	249.5	268.5
10	38	10432	228.2	245.5
9	34	10432	204.2	219.6
8	30	10432	180.1	193.8
7	26	10432	156.1	168.0
6	22	10432	132.1	142.1
5	18	10432	108.1	116.2
4	14	10432	84.1	90.5
3	10	10432	60.1	64.6
2	6	11069	38.3	41.2
1	0	0	0.0	0.0
Total		206731	5597.3	6326.2

APPENDIX B

DESIGN CALCULATIONS

B.1- Design by CAN3-A23.3-M84 :

No.	Load Combinations:
1	1.25D+1.5L
2	1.25D+ 0.7(1.5L+1.5E)
3	0.85D+1.5E
4	1.25D+1.5E

B.1.1- Design of First Storey Girders of the Frame Structure:

DESIGN MOMENTS (KN.m)						
NBCC-85 & CAN3-A23.3-M84						
	Beam AB			Beam BC		
	Joint A	Joint B	mid span	Joint B	Joint C	mid span
1.25D+1.5L	338.5	428	210	408	399	191
1.25D+0.7(1.5L+1.5E)	770.5	843	216	825	819	56
0.85D+1.5E	505	444	61	841	841	9
1.25D+1.5E	895	948	182	931	931	130

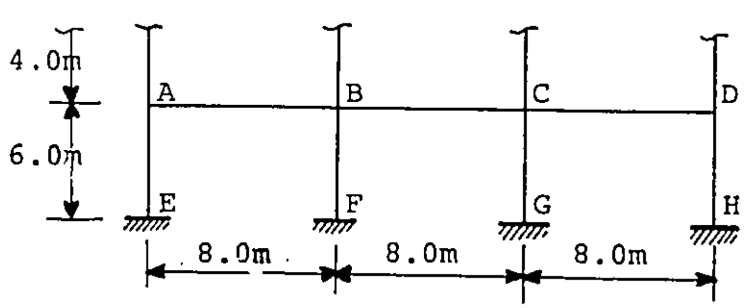
Design Parameters:

b_w = 500mm

d = 650mm

ρmax = 0.025 (longitudinal steel)

ρmin = 0.0035 (longitudinal steel)



Girder Design Steps	
1. Governing load combination	1.25D+1.5E
2. Design negative moment (KN.m)	-948
3. Area of steel required, A_s (mm^2)	5500
4. Design positive moment (KN.m)	+216
5. Area of steel required, A'_s (mm^2)	1500
6. Design nominal shear, V_n (KN)	685
7. Shear carried by concrete V_c (KN)	Nil
8. Shear steel required (KN)	685
9. Lateral steel provided total of 4 legs (mm^2)	$400mm^2$ @ 125mm

B.1.2- Design of First Storey Columns of the Frame Structure

Design Parameters:

ρ_{max} = 0.06 (longitudinal steel)

ρ_{min} = 0.01 (longitudinal steel)

P_b = 12008 KN (exterior column)

P_b = 11581 KN (interior column)

Cover= 90mm

Analysis Results	Interior column load combination at 1st fl.				Exterior column load combination at 1st fl.			
	1	2	3	4	1	2	3	4
Axial load (KN)	14838	14397	8987	13134	8734	10306	7949	10443
Moment (KN.m)	32.5	1287	1806	1807	113	1285	1742	1767

Column Design Steps	Interior Col.	Exterior Col.
1. Governing load combination for longitudinal steel	1.25D+1.5L	1.25D+1.5E
2. Percentage of longitudinal steel provided	3.4	2.2
3. Confinement reinforcement steel , total area (mm^2)	$700mm^2$ @ 70mm	$500mm^2$ @ 50mm
4. Design shear, V_f (KN)	393.5	369.9
5. Shear carried by concrete v_c (KN)	1380	1181
6. Shear reinforcement , (mm^2)	Nil	Nil
7. Total area of lateral ties	$200mm^2$ @ 70mm	$200mm^2$ @ 50mm

B.1.3- Design of First Storey Beam Column Joints of the Frame Structure:

Joint Design Steps	Interior Joint	Exterior Joint
1. Shear due to beam reinforcement (KN)	4500	3000
2. Shear from column above connection (KN)	906	604
3. Net shear in connection (KN)	3594	2396
4. Shear resistance of the joint (KN)	5915	5915
5. Total lateral reinforcement (mm^2)	$700mm^2$ @ 70mm	$500mm^2$ @ 50mm
6. Is the joint confined* ?	No	No

* A joint is considered confined if : Members frame into all four sides of the joint and where each member width is at least three-fourths of the column width.

B.1.4- Design of Shear-Walls of the Shear-Wall Structure:

Unfactored Loads at First Floor Level					
Storey	Seismic, E			Dead, D	Live, L
	M_E^* (KN.m)	V_E (KN)	P_E (KN)	P_D (KN)	P_L^{**} (KN)
1	24445	1231	6561	1124	761

* J factor and effect of accidental eccentricity included

** Includes live load reduction

Load combination	Axial load (KN)	Bending moment (KN.M)	Horizontal shear (KN)	Axial Load on boundary element (KN)
1	2546	nominal	nominal	1273
2	9093	25667	1293	7755
3	10797	36668	1847	9982
4	11247	36668	1847	10207

Shear-Wall Design Steps	
1. Governing load combination	1.25D+1.5E
2. Design moment (KN.m)	36668
3. Axial load on wall (KN)	11247
4. Axial load on boundary element (KN)	10207
5. Design shear at base (KN)	4495
6. Concentrated reinforcement (mm^2)	10000
7. Percentage of distributed reinfocement	0.25%
8. Percentage of concentrated reinforcement	2.17%
9. Shear reinforcement (mm^2)	400
10. Shear resistance at base (KN)	5731

B.1.5- Coupling Beam Design for the Coupled Walls of the Shear Wall Structure

Load combination No.	Design Moments (KN.m) 1	2	3	4
at joint	40	408	302	398
at midspan	20	15	18	10

Design Steps	
1. Governing load case	1.25D+1.5E
2. Design negative moment (KN.m)	-408
3. Area of steel required (mm^2)	2435
4. Desigm positive moment (KN,m)	+359
5. Area of steel required (mm^2)	2336
6. Design shear (KN)	540
7. Shear carried by concrete (KN)	Nil
8. Lateral reinforcement provided	$400mm^2$ @ 130mm
9. Maximum shear provided (KN)	810

B.2- Design by ACI-318.83 :

No.	Load Combinations:
1	1.4D+1.7L
2	0.75(1.4D+1.7L+1.87E)
3	0.9D+1.43E

B.2.1 -Design of First Storey Girders of the Frame Structure:

DESIGN MOMENTS (KN.m)						
UBC-82 & ACI-318.83						
	Beam AB			Beam BC		
	Joint A	Joint B	mid span	Joint B	Joint C	mid span
1.4D+1.7L	381	482	236	458	448	215
0.75(1.4D+1.7L+1.87E)	960	1021	214	1007	1000	161
0.9D+1.43E	855	888	142	878	878	94

Design Parameters:

b_w = 500mm

d = 650mm

ρ_{max} = 0.025

ρ_{min} = 0.0034

Girder design steps	
1. Governing load combination	0.75(1.4D+1.7L+1.87E)
2. Design negative moment (KN.m)	-1021
3. Area of steel required (mm^2)	4957
4. Design positive moment (KN.m)	+521
5. Area of steel required (mm^2)	2136
6. Nominal design shear (KN)	611
7. Shear carried by concrete (KN)	296
8. Shear steel required (KN)	423
9. Lateral steel provided	$400mm^2$ @ 160mm

B.2.2- Design of First Storey Columns of the Frame Structure:

Design Parameters:

ρ_{max} = 0.06 (longitudinal steel)

ρ_{min} = 0.01 (longitudinal steel)

P = 12008 KN (exterior column)

P = 11581 KN (interior column)

Cover= 90mm

Analysis Results	Interior column load combination at 1st fl.			Exterior column load combination at 1st fl.		
	1	2	3	1	2	3
Axial load (KN)	16644	12647	9498	9795	9823	8137
Moment (KN.m)	76	1715	1722	242	2223	1666

Column Design Steps	Interior col.	Exterior col.
1. Governing load combination for longitudinal steel	1.4D+1.7L	0.75(1.4D+1.7L+1.87E)
2. Percentage of longitudinal steel provided	3.4	2.2
3. Confinement reinforcement steel (mm^2)	$700mm^2$ @ 102mm	$500mm^2$ @ 102mm
4. Design shear, V_u (KN)	1714	1448
5. Shear carried by concrete V_c (KN)	932	797
6. Shear reinforcement, (mm^2)	1400	1000
7. Total area of lateral ties	$200mm^2$ @ 70mm	$200mm^2$ @ 50mm

B.2.3- Design of First Storey Beam Column Joints of the Frame Structure:

Joint Design Steps	Int. Joint	Ext, Joint
1. Shear due to beam reinforcement (KN)	3750	2500
2. Shear from column above connection (KN)	379	238
3. Net shear in connection (KN)	3371	2262
4. Shear resistance of the joint (KN)	5796	5796
5. Total lateral reinforcement (mm^2)	$700mm^2$ @100mm	$500mm^2$ @100mm
6. Is the joint confined*?	No	No

* A joint is considered confined if ; Members frame into all four sides of the joint and where each member width is at least three-fourths of the column width.

B.2.4- Design of Shear-Walls of the Shear-Wall Structure:

Storey	Unfactored Loads at First Floor Level				
	Seismic, E			Dead, D	Live, L
	M_E^* (KN.m)	V_E (KN)	P_E (KN)	P_D (KN)	P_L^{**} (KN)
1	35423	1391	7616	1124	761

* Includes effect of accidental eccentricity

** Includes live load reduction.

Load combination	Axial Load (KN)	Bending moment (KN.m)	Horizontal shear (KN)	Axial Load on boundary element (KN)
1	2866	nominal	nominal	1433
2	12831	49681	1950	12626
3	11902	50655	1989	12283

Shear-Wall Design Steps	
1. Governing load combinations	0.75(1.4D+1.7L+1.87E)
	0.9D+1.43E
2. Design moment (KN.m)	59655
3. Axial load on wall (KN)	12831
4. Axial load on boundary element (KN)	12626
5. Design shear at base (KN)	5616
6. Concentrated reinforcement (mm^2)	22000
7. Percentage of concentrated reinforcement	3.83%
8. Percentage of distributed reinforcement	0.25%
9. Shear reinforcement (mm^2)	400
10. Shear resistance at base (KN)	10270

B.2.5- Coupling Beam Design for the Coupled Walls of the Shear Wall Structure:

	Design Moments (KN.m)		
Load combination No.	1	2	3
at joint	45	435.5	431
at midspan	22.5	17	11

Design Steps	
1. Governing load case	0.75(1.4D+1.7L+1.87E)
2. Design negative moment (KN.m)	-435.5
3. Area of steel required (mm^2)	2404
4. Design positive moment (KN.m)	+389
5. Area of steel required (mm^2)	2064
6. Design shear (KN)	557
7. Shear carried by concrete (KN)	Nil
8. Lateral reinforcement provided	$400mm^2$ @ 130mm
9. Maximum shear provided (KN)	810

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