Seismic damage potential to ductile and nominally ductile concrete frames

K.A. Hamdy[I], W.K. Tso[II], and A. Ghobarah[II]

ABSTRACT

Two reinforced concrete ductile moment resisting frames and two nominally ductile frames are designed according to the current Canadian concrete design code. For each type of frame design, one frame is 4 storeys high while the other is ten storeys high. The frames are analyzed dynamically using the computer program DRAIN-2D. The response parameters investigated are the storey drifts, the beam rotational ductility demands and the beam shear stresses. The above mentioned parameters are used to compare the equivalence of the seismic behaviour of nominally ductile frames and that of ductile frames. It was found that the behaviour of nominally ductile frames is less favourable than that of ductile frames mainly because the response shear stresses in the beams of nominally ductile frames are much larger than the design values.

INTRODUCTION

Moment resisting frames (MRF) are the most commonly used framing system for reinforced concrete structures. According to the current Canadian practice, designers are given two options for the seismic design of reinforced concrete frames (National Building Code of Canada, NBCC 1990). The first option is to design a ductile frame, which involves special design and detailing provisions to insure ductile behaviour. The second option is to design a nominally ductile frame. This option involves designing for twice the seismic lateral load as that for ductile frames, but without taking all the special provisions for good detailing in the design of the frame members. By allowing such a choice, the code implies that either type of frames will provide equivalent seismic performance under the design level earthquake disturbance.

The objective of this investigation is to compare the equivalence of the seismic behaviour of ductile and nominally ductile frames. To achieve this aim, two four storey and two ten storey frames are designed according to NBCC 1990 and CAN3-A23.3-M84. Each of the frames is subjected to earthquake records normalized to the design velocity. To avoid dependence on the characteristics of a single record, 15 records are used and the results are discussed based on a statistical analysis of the individual responses. The response parameters investigated are the total and interstorey drifts, the beam rotational ductility demands and the beam shear stresses.

[I] Research assistant, Civil Eng. Dept., McMaster University, Canada.

[II] Professor

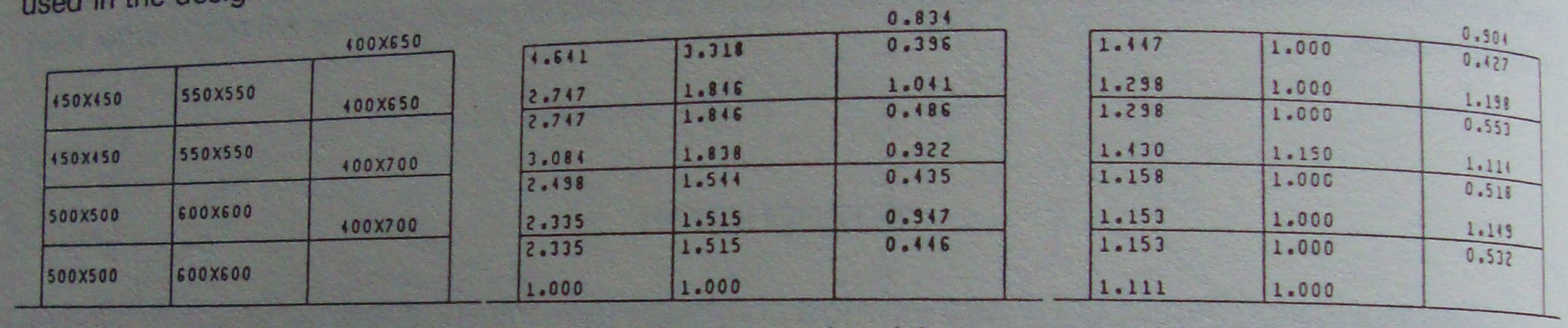
Each of the two buildings considered, consists of 3 bays in the E-W direction and 7 bays in the N-S direction. The bay widths are 8.0 m in each direction. One building is four storeys high and the other is ten storeys high. The storey height is 3.5 m. The concrete compressive strength is 30 MPa and the steel yield strength is 400 MPa. The seismic loading is assumed to be acting in the E-W direction. The typical interior E-W frame of each building is designed once as a ductile frame and once as a nominally ductile frame. This has resulted in four different frames.

DESIGN LOADINGS

The frames are designed for the critical combinations of gravity and seismic loads as per NBCC 1990. For seismic base shear calculations, the frames are assumed to be located in Quebec City. The force modification factor, R, is chosen according to the ductility level for which the frames are designed. For ductile frames R = 4.0 and for nominally ductile frames R = 2.0 .

DESIGN OF FRAME MEMBERS

For comparison purposes, the member dimensions are taken to be the same for all frames. The reinforcement ratio differs due to the variation of the seismic lateral load and the design approach. Figs. 1 and 2 show the member dimensions and the reinforcement ratios of the frames.The different approaches used in the design of ductile and nominally ductile frames are briefly described below.

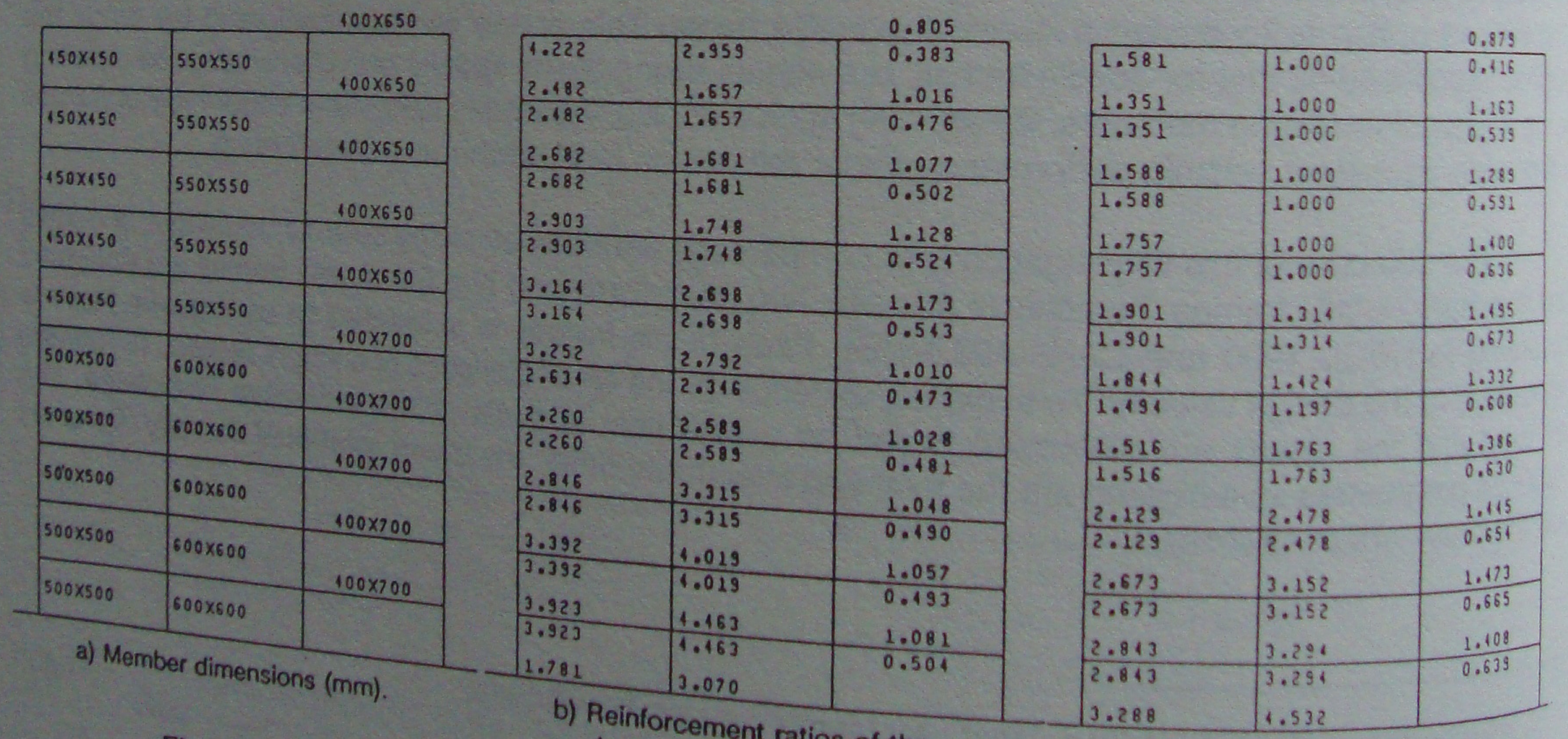


a) Member dimensions (mm).

b) Reinforcement ratios of the ductile frames (%).

c) Reinforcement ratios of the nominally ductile frames (%).

Figure 1. Member dimensions and reinforcement ratios of the four storey frames.



a) Member dimensions (mm).

b) Reinforcement ratios of the ductile frames (%).

c) Reinforcement ratios of the nominally ductile frames (%).

Figure 2. Member dimensions and reinforcement ratios of the ten storey frames.

Ductile frames

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

Nominally ductile frames

No special seismic design provisions are considered in the design of these frames. All the design actions are directly obtained from the results of the elastic static analysis. Detailing requirements are far less stringent than those of ductile frames.

DISCUSSION OF THE DESIGN RESULTS

The reinforcement ratios in the beams of nominally ductile frames are larger than those of the beams of ductile frames. Nevertheless, the columns of ductile frames usually contain more steel than those of nominally ductile frames This is the result of the strong column-weak beam requirement for ductile frames design. Thus, the overstrength possessed by the frames may vary significantly due to the variation in the design methodology. According to NBCC 1990, the overstrength factor (1/U) is assumed to be 1.67 for all structures regardless of type or design methodology. In order to determine their overstrengths, the designed frames were analyzed under a monotonically increasing static lateral loading. The lateral loads are distributed over the height according to NBCC 1990. The overall base shear-top displacement relationships of the frames are shown in figs. 3 and 4. For the ductile frames, the overstrength factor was 4.5 for the four storey frame and 3.0 for the ten storey frame. For the nominally ductile frames, the overstrength factor was 1.8 for the four storey frame and 1.7 for the ten storey frame. While the lateral overstrength factor of the ductile frames is significantly higher than that assumed in NBCC 1990, the corresponding factor of the nominally ductile frames is of the same order as that assumed in NBCC 1990. It should be noted that the design base shear for nominally ductile frames is twice that of ductile frames, therefore the actual lateral strengths of the frames designed based on either approach are comparable.

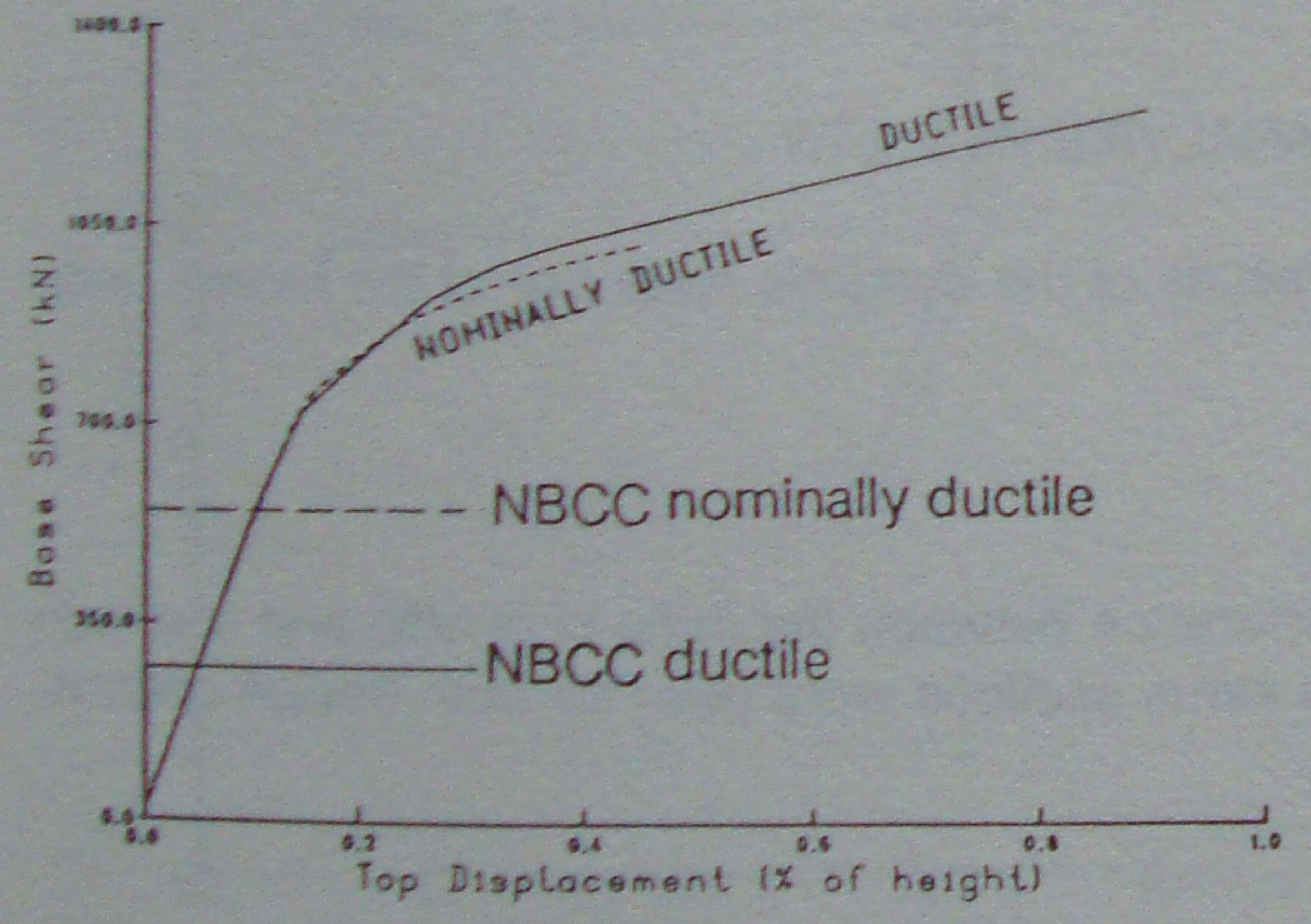


Figure 3. Base shear-top displacement curves of the four storey frames.

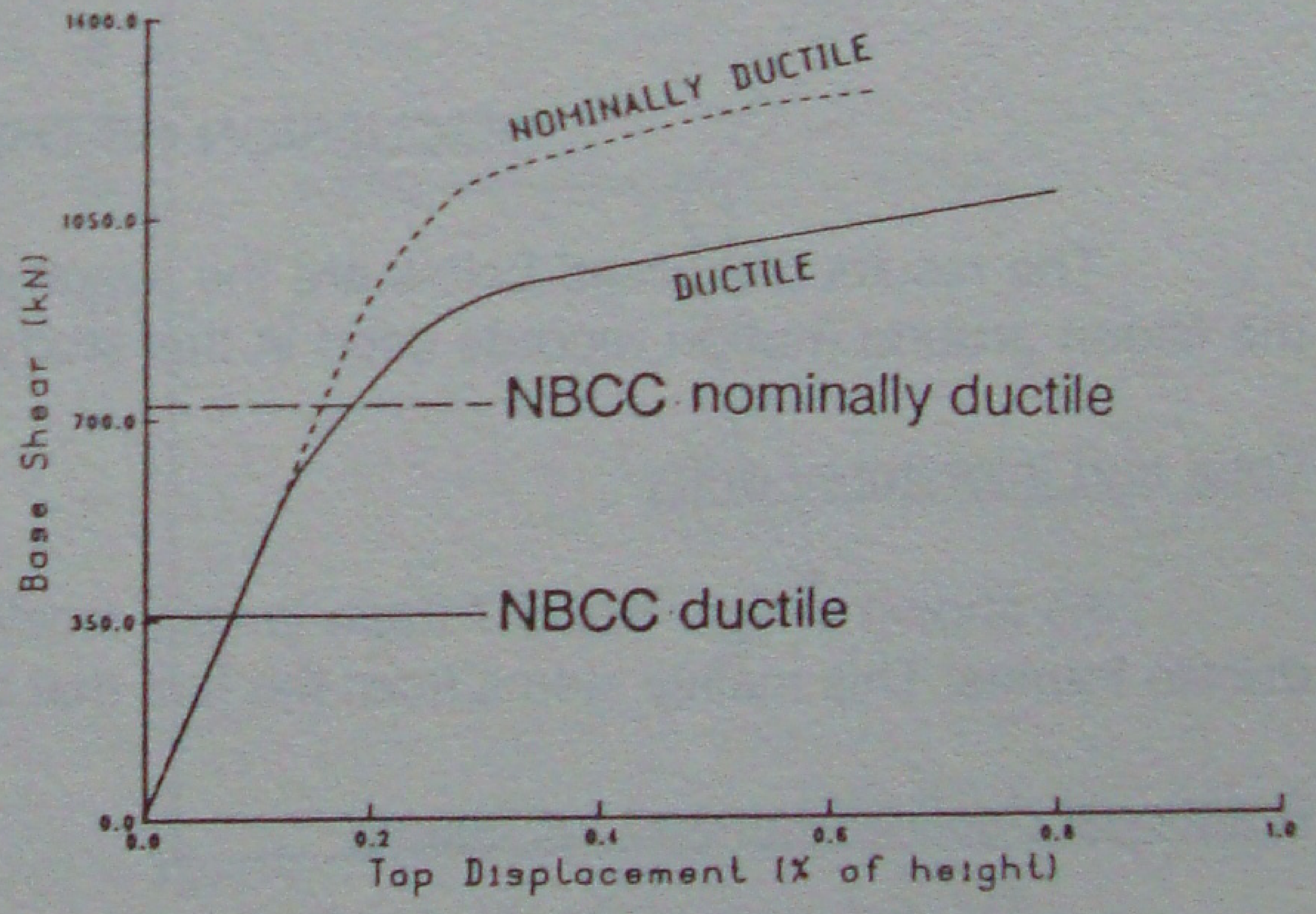


Figure 4. Base shear-top displacement curves of the ten storey frames.

DYNAMIC ANALYSIS PROCEDURE

The DRAIN-2D computer program (Kanaan and Powell 1973) in the dynamic analysis of the frames. The members of the ductile frames are modelled by the dual-component bilinear model developed by Clough, Benushad and Wilson (1965). The use of stable loops may be justified by the good detailing of the members of ductile frames. The members of the nominally ductile frames are modelled by elastic elements with nonlinear rotational springs at their ends. Strength deterioration and pinching are introduced in the hysteresis behaviour of the springs. The used hysteresis model was developed by Chung, Meyer and Shinozuka (1987). A typical hysteresis loop is shown in fig. 5. Fifteen records in the high acceleration-to-velocity (a/v) range (Naumoski et al. 1988) were selected as the ground motion input for this study. In a seismic region where $Z_a > Z_v$ according to the Canadian seismic zoning, the ground motions are expected to have high frequency contents and the records with high a/v ratio are considered representative of the expected ground motions.

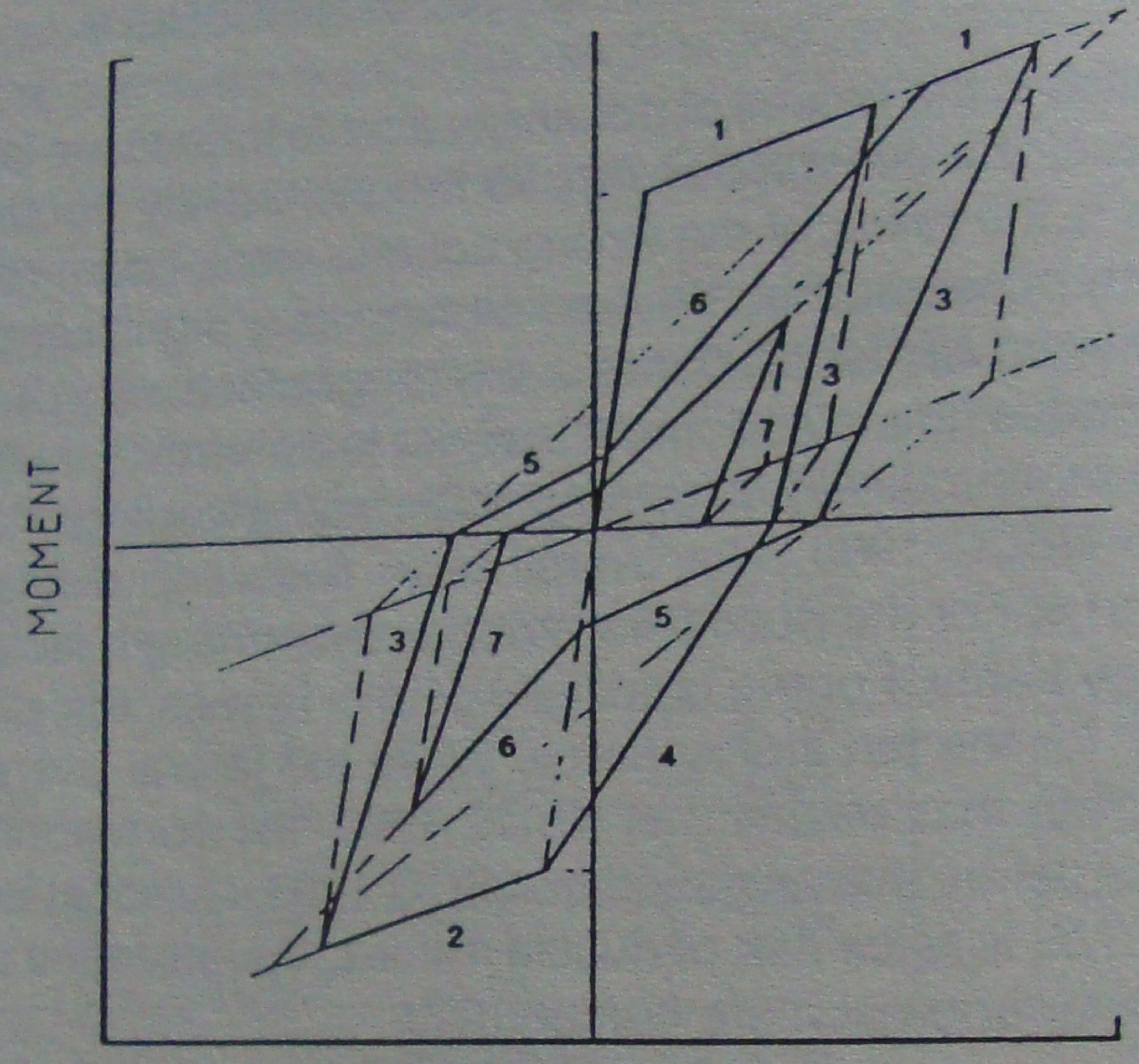


Figure 5. Typical moment-rotation relationship for the members of nominally ductile frames. (Chung, Meyer and Shinozuka, 1987)

DISCUSSION OF THE ANALYSIS RESULTS

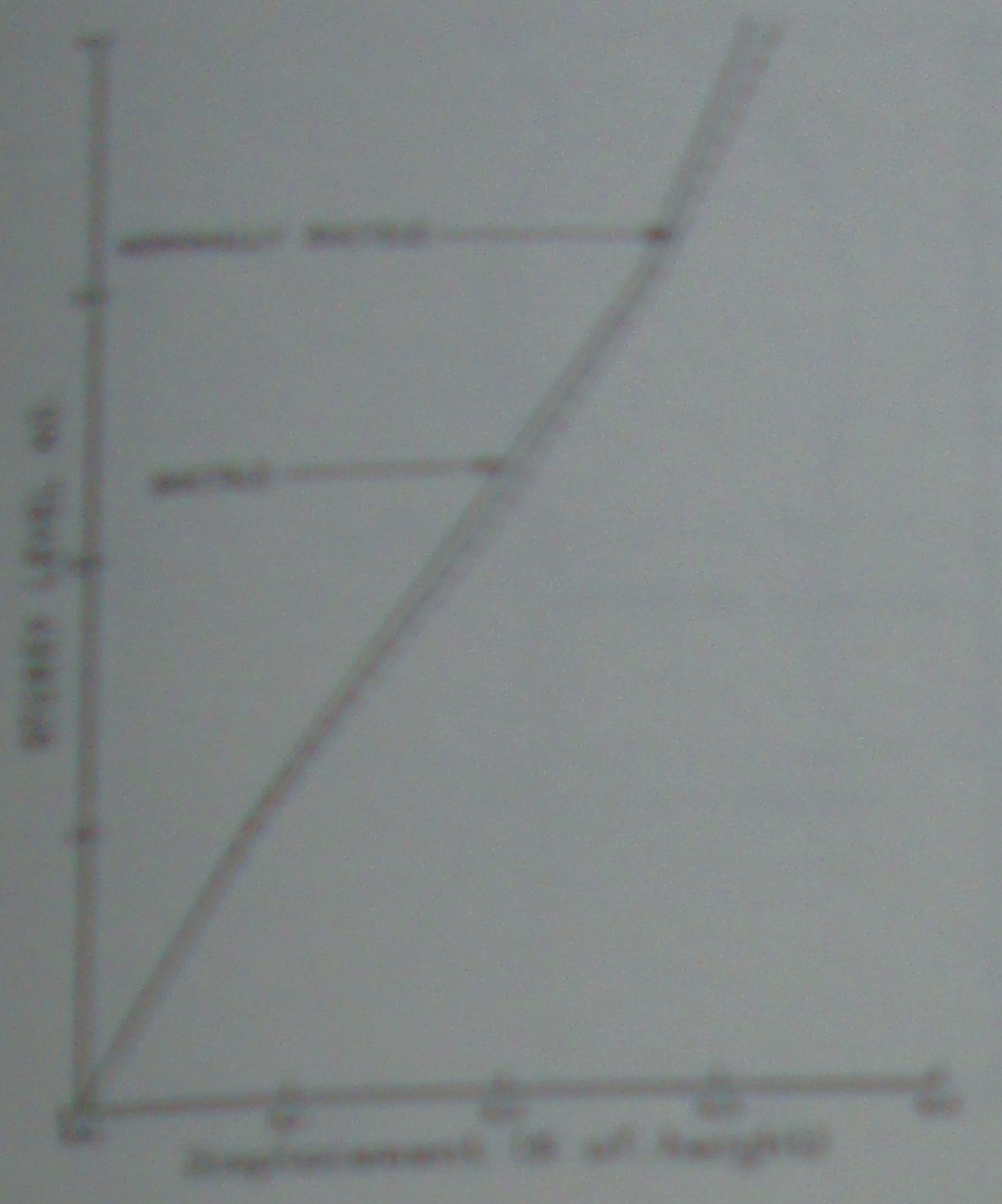
The results discussed below are the average results of the dynamic response of the frames under the fifteen ground motion records used in this study.

Total and interstorey drifts

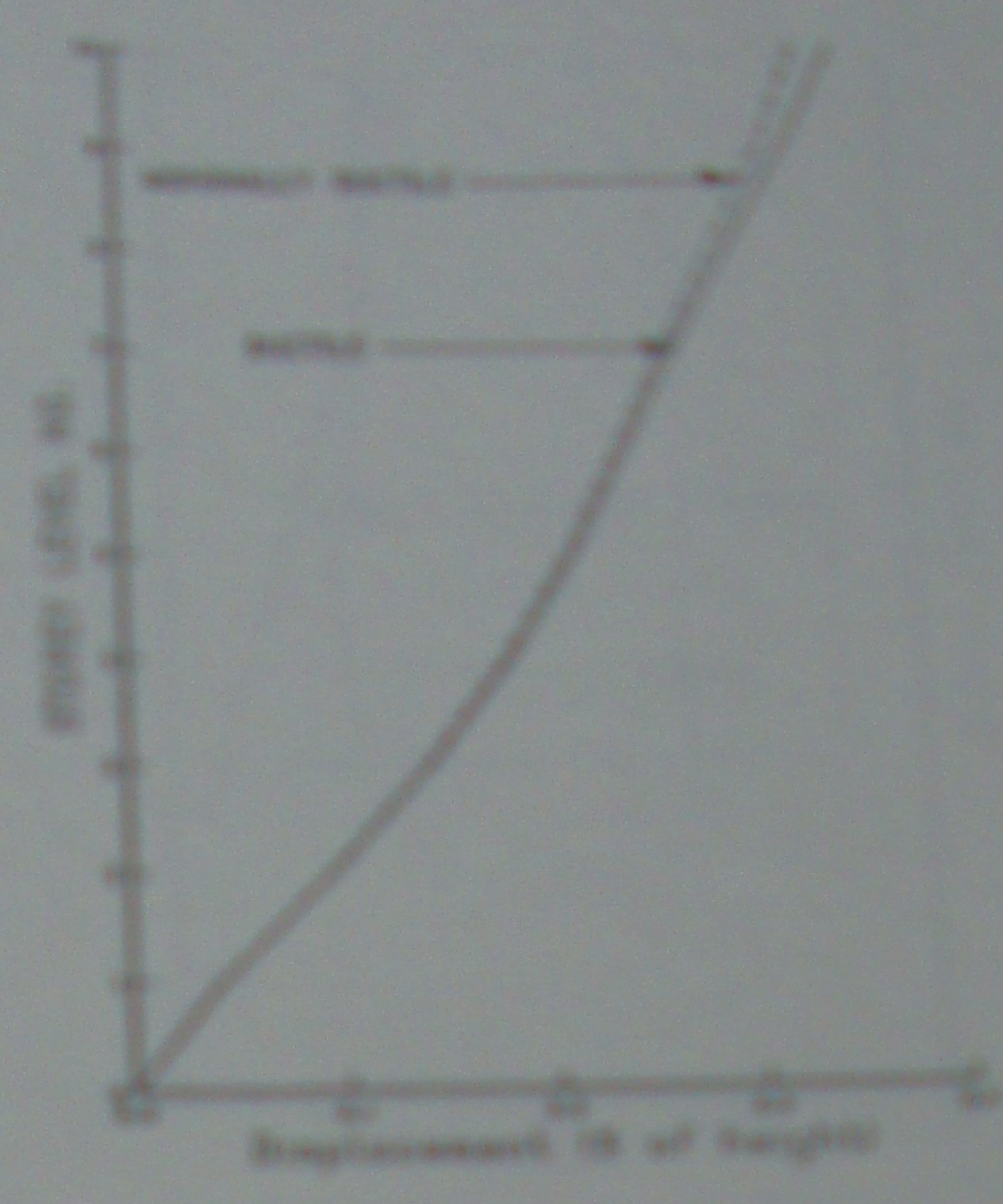
As shown in figs. 6 and 7 there is very small difference between the drifts of ductile and nominally ductile frames. This mainly stems from the fact that the same sections were used for both design options.

Rotational ductility demand for beams

The maximum rotational ductility demands at the beam ends of are shown in fig. 8. It can be seen that the ductility demand for ductile frames is larger than that for nominally ductile frames. Nevertheless, the difference is smaller that what would be implied by the difference in the design base shear of the two frames (base shear of a ductile frame is half that of a nominally ductile frame).

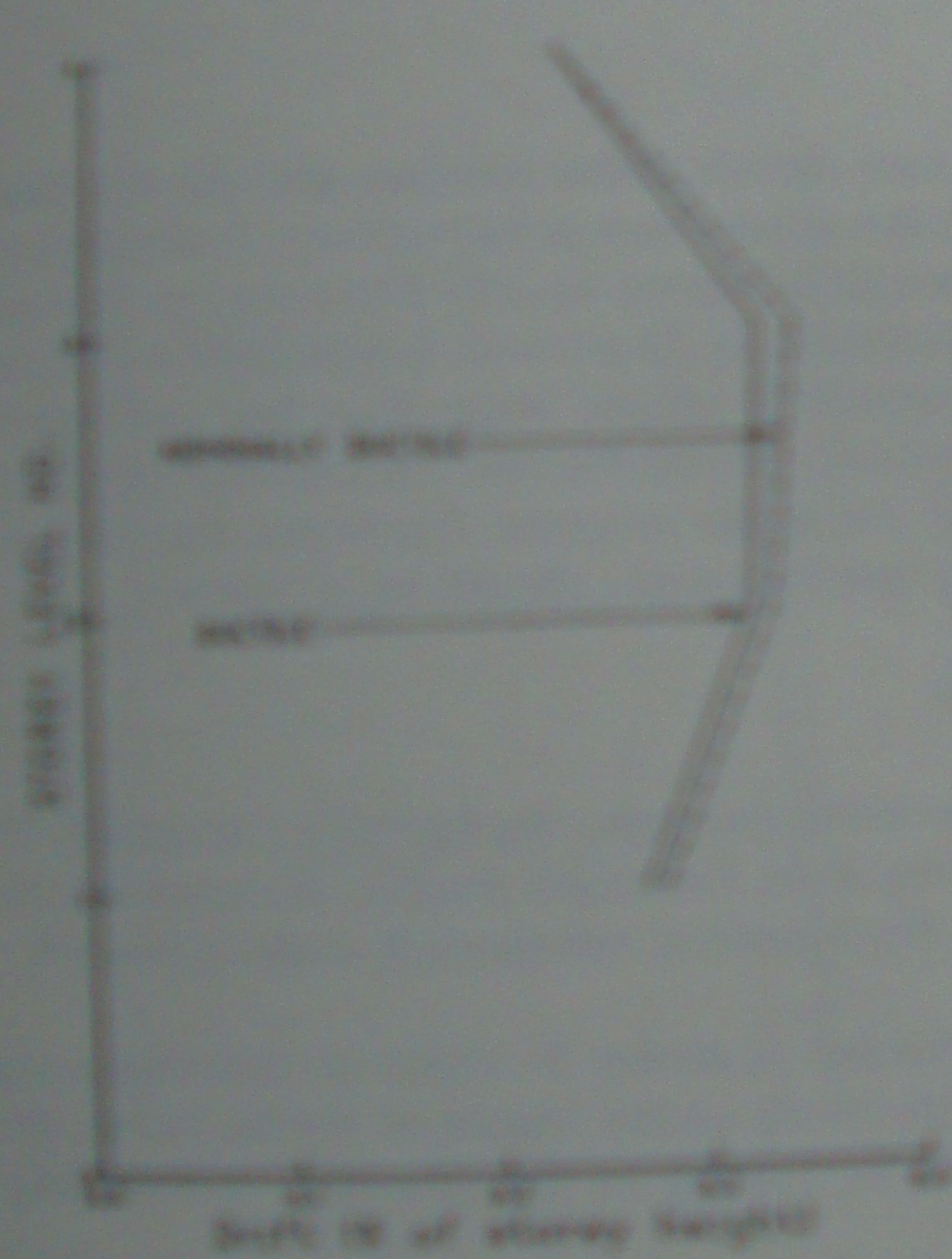


a) Four storey frames.

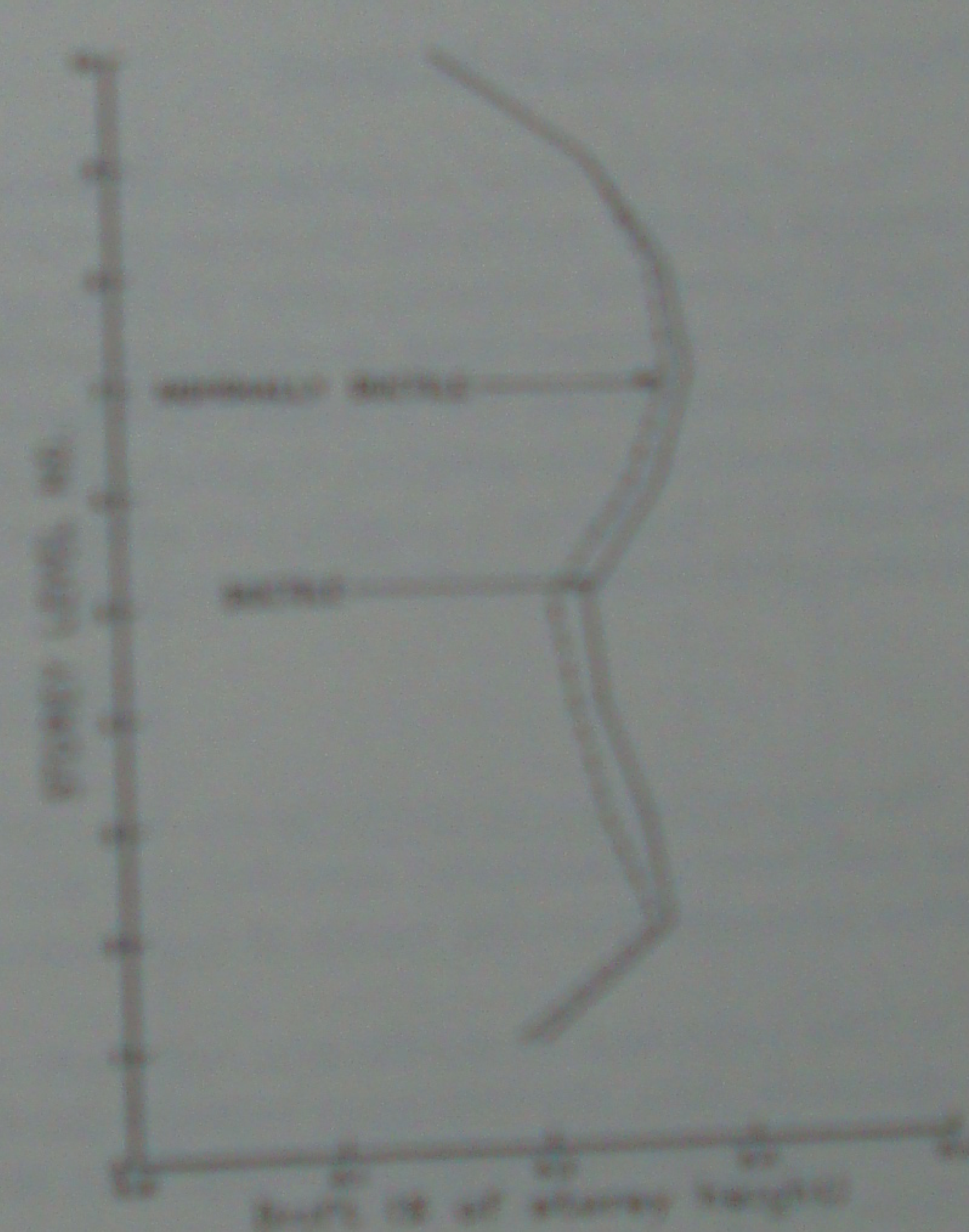


b) Ten storey frames.

Figure 6. Lateral displacements of the frames (% of height).

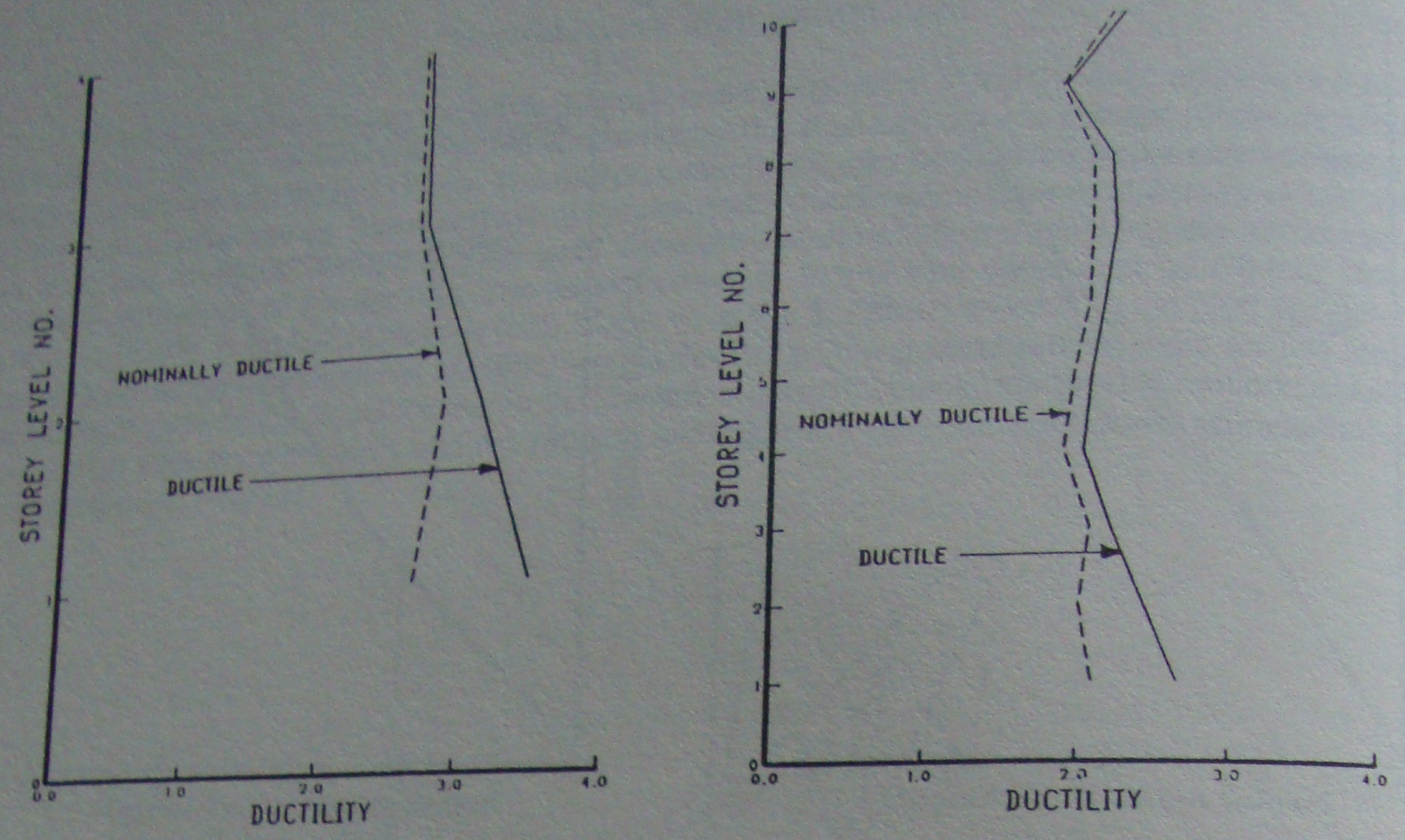


a) Four storey frames.



b) Ten storey frames.

Figure 7. Interstorey drifts of the frames (% of storey height).



a) Four storey frames. b) Ten storey frames.

Figure 8. Beam rotational ductility demands.

Maximum shear stresses in the beams

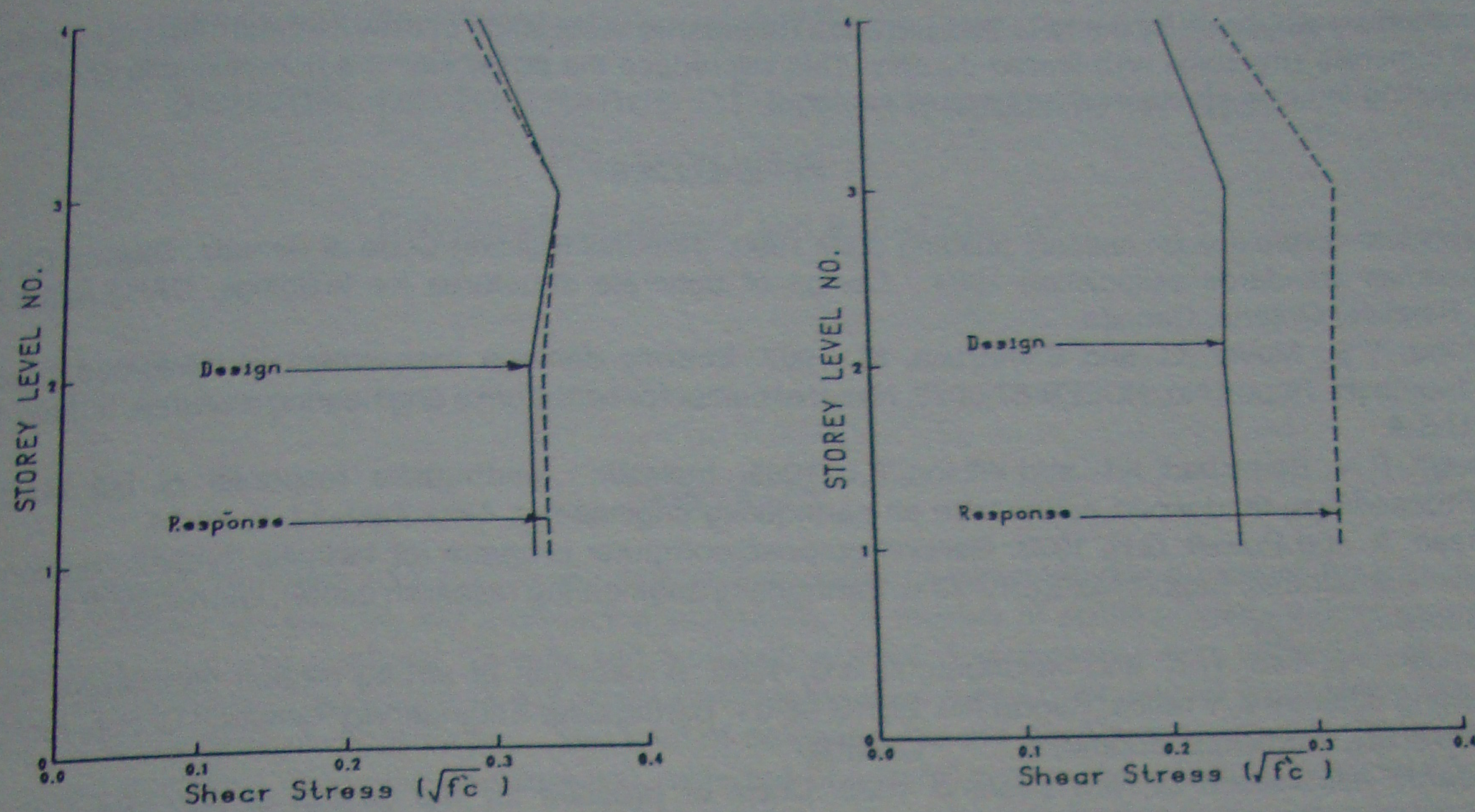
Figs. 9 and 10 show a comparison between the beam design shear stresses and the maximum shear stresses attained during the response for the four and ten storey frames respectively. It can be seen that in ductile frames the response shear stresses exceed the design values by only 6 percent. In nominally ductile frames the response shear stresses exceed the design values by 30 percent. It should be mentioned here that in ductile frames, the provided transverse reinforcement is usually larger than that required for shear, because of the other detailing requirements. But in nominally ductile frames, the provided transverse reinforcement in the beams is usually just sufficient to resist the shear stresses. Therefore, the response shear stress being larger than the design value for nominally ductile frames should be of concern.

CONCLUSIONS

The seismic behaviour of ductile and nominally ductile reinforced concrete frames is compared. The response parameters investigated were the total and interstorey drifts, the beam ductility demands and the beam shear stresses. From the results of this investigations, the following conclusions could be drawn;

1) The behaviour of ductile and nominally ductile frames is similar in terms of deflections and ductility demands. Also, the difference between the ductility demands on the beams of ductile frames and those of nominally ductile frames is smaller than what would be implied by the difference in the design seismic base shear.

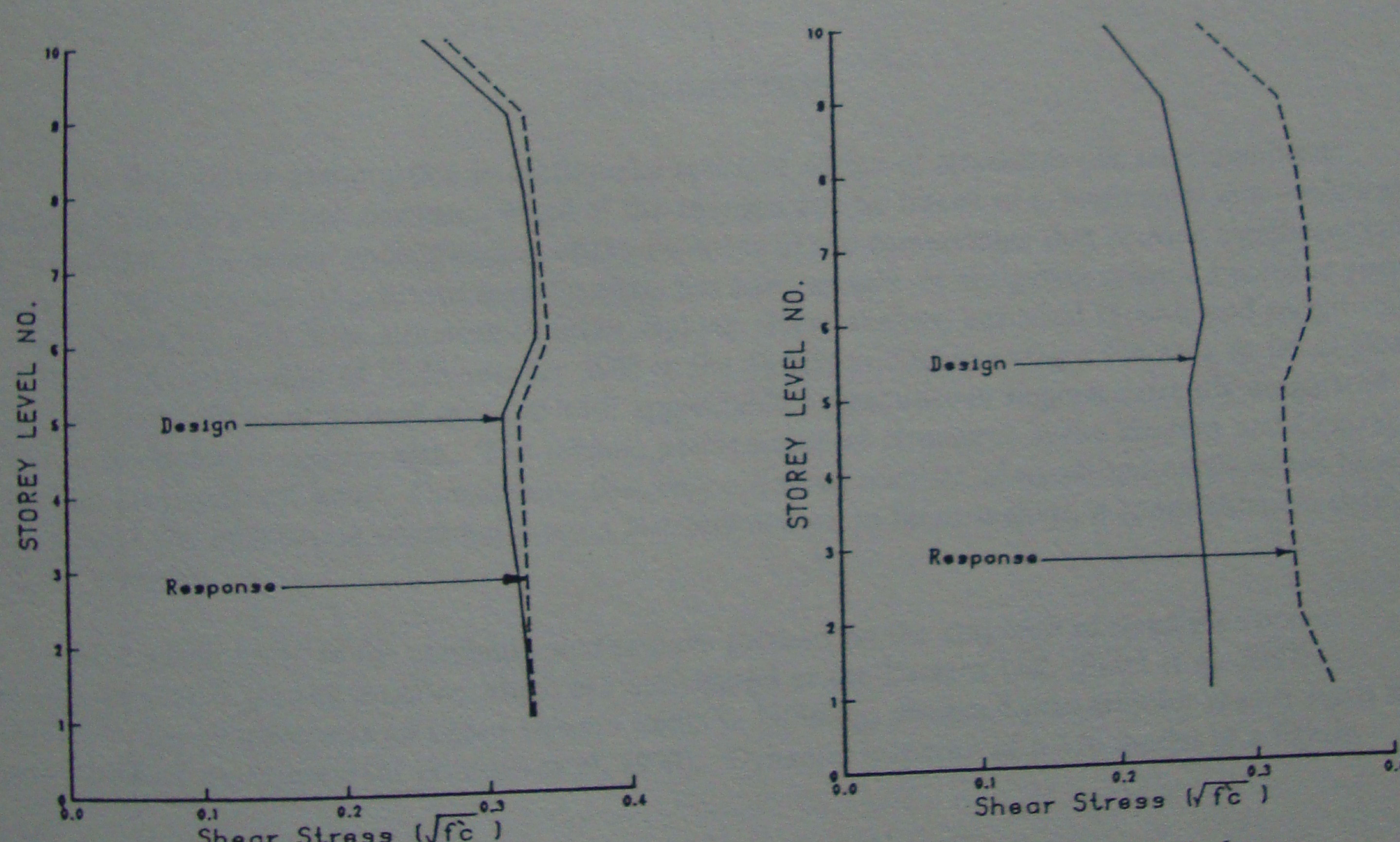
2) The beams of nominally ductile frames undergo shear stresses which are much larger than their design values. Therefore, the design shear forces used for nominally ductile frames should be increased from those obtained directly from the elastic static analysis. This approach would be consistent with the



a) Ductile frame.

b) Nominally ductile frame.

Figure 9. Comparison between the design and response shear stresses in the beams of the four storey frames.



a) Ductile frame.

b) Nominally ductile frame.

Figure 10. Comparison between the design and response shear stresses in the beams of the ten storey frames.

procedure employed by the New Zealand code (Standards association of New Zealand 1982) for the design of concrete structures with limited ductility. This will reduce the possibility of a non-desirable shear failure resulting in large cracks and irreparable damage.

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