INELASTIC EARTHQUAKE RESPONSE AND DESIGN OF MULTISTOREY TORSIONALLY UNBALANCED STRUCTURES

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To

My Father

Nikos

ABSTRACT

Structures exhibit coupled torsional and translational responses to earthquake ground motion input if their centres of floor mass and their centres of resistance do not coincide. However, torsional motions may occur even in nominally symmetric structures due to accidental eccentricity and torsional ground motions. The sources giving rise to accidental eccentricity include the difference between the assumed and actual distributions of mass and stiffness, asymmetric yielding strength, non-linear patterns of force-deformation relationships, and differences in coupling of the structural foundation with the supporting soil.

Symmetric and regular buildings that are properly designed have a much higher ability to survive a strong earthquake event than asymmetric buildings and their response to earthquake loading is far more straightforward to predict and design for. On the other hand, even though the response of asymmetric buildings is more unpredictable, designers still have to compromise structural regularity to accommodate functional and aesthetic needs. As a result, serious and widespread damage associated with structural asymmetry has been observed repeatedly in past major earthquakes.

In the first studies examining torsional effects in buildings, attention was focused on the elastic structural behaviour of single-storey buildings and the main purpose was to achieve a complete understanding of the effects of mass and stiffness eccentricities and to evaluate them by simple static models. However, as the response of real structures is mainly inelastic, these studies gave poor information on torsional behaviour and interest has moved towards non-linear response studies. In an effort to clarify some of the issues influencing the inelastic torsional response of multistorey asymmetric structures, this thesis presents a series of coherent parametric investigations.

These investigations include comparing the response of various reference models to the performance of code-designed torsionally unbalanced structures. An extensive parametric investigation of torsionally responding structures designed as stipulated by a selection of major earthquake building codes is presented and the adequacy of the static torsional provisions is assessed for a wide range of structural configurations and parameters. Detailed investigations of torsionally asymmetric structures incorporating frame elements oriented along both orthogonal axes of the structure are also conducted and the effect of including the second earthquake component to simultaneously excite the structural models is quantified. The relative merits and deficiencies of each code provision are discussed and a new proposed optimised method is tested. All fundamental conclusions from the investigations conducted are presented and various topics for further research are proposed, which are considered to be both necessary and pertinent for increasing and refining the knowledge and understanding the complex behaviour of multistorey torsionally asymmetric buildings.

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NOTATION

a .... building plan dimension parallel to the seismic excitation.

A .... peak ground acceleration.

A_c .... total cross-sectional area of a concrete section.

A_s .... total cross-sectional area of steel.

A_v .... ground acceleration normalised to a specific velocity value.

A_x .... UBC defined accidental eccentricity amplification factor.

A/V .... ratio of peak ground acceleration over peak ground velocity.

b .... building plan dimension perpendicular to the seismic excitation.

b/d .... width/depth beam dimensions.

β .... accidental eccentricity factor.

$\mathbf{C}$ .... global damping matrix.

C .... NZS lateral force coefficient.

$C_h(T,\mu)$ .... NZS basic seismic acceleration coefficient.

C_t .... EC8 factor for the period calculation of structures.

d .... lever arm between frame elements with respect to CR.

d_e .... linear displacement of a point of the structural system (EC8).

d_s .... displacement of a point of the structural system (EC8).

D .... maximum ground displacement.

δ .... EC8 amplification factor for torsional effects.

δ_{avg} .... average floor displacement.

δ_{max} .... maximum structural or element deformation.

δ_{yield} .... yield structural or element deformation.

Δ_i .... interstorey drift ratio.

Δt .... integration time-step.

Δx_i .... relative interstorey displacement between two adjacent floors.

e .... normalised static eccentricity.

e_1 .... EC8 additional eccentricity component.

e_a .... accidental eccentricity.

e_{d1}, e_{d2} .... first and second dynamic eccentricities.

e_{D1}, e_{D2} .... first and second design eccentricities.

e_g .... geometric eccentricity.

e_s .... stiffness or static eccentricity.

E_E .... seismic action effect.

EI .... stiffness gross-section.

f_{cd} .... design value of concrete cylinder compressive strength.

f_{ctm} .... mean axial tensile strength of concrete.

f_y .... yield strength of reinforcement.

f_{yd} .... design yield strength of reinforcement.

F_i .... lateral force of a storey level *i*.

F_t .... lateral force applied at the top of the structure.

ϕ_i .... vibration mode *i*.

g .... acceleration due to gravity.

γ_I .... EC8 importance factor of the structure.

h_i .... height of a storey level *i*.

H .... total building height.

I .... importance factor.

I_m .... mass moment of inertia.

k .... number of modes.

k_{iy}, k_{ix} .... stiffness of the i[th] element in the y- and x-directions, respectively.

$K_{\theta r}$ .... torsional stiffness about CR.

K_x .... total transverse structural stiffness.

K_y .... total lateral structural stiffness.

$\mathbf{K_T}$ .... current tangent stiffness matrix.

$\mathbf{K_0}$ .... the original elastic stiffness matrix.

l .... member length.

l_0 .... distance between inflection point (M=0) and end column section.

L_e .... distance between the two outermost lateral load-resisting elements.

L_s .... serviceability limit state factor.

L_u .... ultimate limit state factor.

λ .... floor plan aspect ratio.

m .... storey mass.

$\mathbf{M}$ .... global mass matrix.

M_b .... beam moment.

M_c .... column moment.

M_L .... local Richter magnitude.

M_y .... yield moment of a member.

μ .... ductility factor.

μ_δ .... displacement ductility.

μ_θ .... rotational ductility.

n .... number of storeys.

N, N_d .... axial load.

ν .... EC8 reduction factor to account for the lower return period of the seismic event.

ν_d .... normalised axial load.

O_s .... overstrength ratio.

q .... EC8 behaviour factor.

q_d .... EC8 displacement behaviour factor.

θ .... interstorey drift sensitivity coefficient.

θ_{max} .... maximum total rotation at the end of a member.

θ_p ....plastic hinge rotation at the end of a member.

θ_y ....yield rotation at the end of a member.

r_k .... stiffness radius of gyration about CR.

r_m .... mass radius of gyration about CM.

r_r .... mass radius of gyration about CR.

R .... NZS risk factor.

R_w .... UBC numerical coefficient factor.

ρ_1, ρ_2 .... tensile and compressive steel ratio of a beam section.

ρ_k .... normalised stiffness radius of gyration about CR.

ρ_m .... normalised mass radius of gyration about CM.

ρ_{min}, ρ_{max} .... minimum and maximum steel ratios of a structural element.

S .... UBC site coefficient factor.

S_a .... spectral acceleration.

$S_d(T)$ .... EC8 ordinate of the design spectrum at period T.

S_m .... NZS response spectrum scaling factor.

S_p .... NZS structural performance factor.

S_v .... spectral velocity.

SI .... spectrum intensity.

T_c .... EC8 parameter describing the elastic response spectrum.

T_x .... uncoupled transverse period.

T_y .... fundamental uncoupled lateral period.

u_g .... earthquake ground displacement.

u_i .... seismic deflection at a storey level i.

V .... maximum ground velocity.

V_i .... total shear force at a storey level i.

W_i .... total vertical force (weight) at a storey level i.

Ω .... fundamental torsional to lateral frequency ratio.

ω_i .... natural frequency of vibration i for the coupled system.

ω_θ .... fundamental uncoupled torsional frequency about CM.

$\omega_{\theta r}$ .... fundamental uncoupled torsional frequency about CR.

ω_y .... fundamental lateral frequency.

x_{CR}, y_{CR} .... the Cartesian co-ordinates of CR.

x_i, y_i .... normal distances from CR to the elements oriented in the y- and x-directions, respectively.

ξ .... structural damping ratio.

Z .... NZS and UBC zone factor.

ABBREVIATIONS

2D .... two – dimensional.

3D .... three – dimensional.

CM .... centre of mass.

CP .... centre of strength.

CQC .... complete quadratic combination.

CR .... centre of rigidity.

CS .... centre of stiffness.

Ea .... designed by including the accidental eccentricity provisions.

EC8 .... Eurocode 8 (1993).

GC .... geometric centre.

M-N .... moment-axial force interaction diagrams.

MDOF.......................... multi degree of freedom system.

Min .... designed by including the minimum steel ratios of the codes.

No ea .... designed by excluding the accidental eccentricity provisions.

No min .... designed by excluding the minimum steel ratios of the codes.

NZS .... New Zealand Standard (1992).

P-Δ .... P-delta effects.

RC .... reinforced concrete.

SDOF .... single degree of freedom system.

SRSS .... square root of the sum of squares.

SSP .... stiffness and strength proportional model.

SLS .... serviceability limit state.

SM .... symmetric reference model.

TB .... torsionally balanced reference model.

TU .... torsionally unbalanced model.

UBC .... Uniform Building Code (1994).

ULS .... ultimate limit state.

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CHAPTER 1

INTRODUCTION AND OBJECTIVES

1.1 INTRODUCTION

Earthquakes are one of nature's greatest hazards to life because throughout the years they have caused the destruction of cities and villages on nearly every continent. They are the least understood of the natural hazards and an important distinction of the earthquake problem is that the hazard to life is associated almost entirely with man-made structures. Except for earthquake-triggered landslides, the earthquake effects that cause extensive loss of life are collapses of bridges, buildings, dams, and other works of man. It is this fact that has led to the great emphasis placed on earthquake prediction in one of the world's great seismic regions, The People's Republic of China. Apparently such a prediction was effective in saving hundreds of lives during the Haicheng Earthquake, China, of February 1975. With a few hours of advance notice, people were evacuated from buildings into open fields where loss of life can be avoided.

However, a successful prediction cannot eliminate the earthquake hazard and, even if all the people are safely evacuated, the destruction of structures could be a disastrous loss to the regional economy. Only the design and construction of earthquake resistant structures can counter this aspect of the earthquake hazard. With effective application of engineering knowledge, the collapse of structures can be avoided and this achievement would greatly reduce the value of any earthquake prediction programme. Earthquake hazard poses a unique engineering design problem and the optimum engineering approach is to design the structure so as to avoid collapse in the most severe possible earthquake. This ensures no loss of life and accepts the possibility of damage when a major earthquake occurs and presents the structural engineer with a challenging problem, to achieve an economical design that is susceptible to some earthquake damage, but essentially proof against collapse in the greatest possible earthquake.

1.2 THE NATURE OF EARTHQUAKE GROUND MOTION

1.2.1 The Seismicity of the World

It is estimated that approximately 6,000 earthquake events are detected each year throughout the world. The number of earthquakes sensed by humans, but which induce no property damage or injury, is approximately 450, and another 35 seismic events cause only minor, non-structural damage. However, approximately 15 events per year can be extremely damaging, causing great loss of life. A list of some of the largest and most devastating earthquakes of the 20^{th} century is given in Table 1.2.1.

From earthquake ground motion readings at various seismographic observatories, the position of the centre of energy release in an earthquake can be calculated and a uniform picture of the earthquake distribution around the world has been obtained (Figure 1.2.1). Definite belts of seismic activity separate large oceanic and continental regions, themselves almost devoid of earthquakes. Other concentrations of earthquake sources can be seen in oceanic areas, for example, along the centre of the Atlantic and Indian Oceans. These are the sites of gigantic submarine mountain ranges called mid-oceanic ridges. The geological strains that prevail throughout this global ridge system are evidenced by great mountain peaks and deep rift valleys.

Dense concentrations of earthquakes coincide with island arcs, such as those of the Pacific and the eastern Caribbean (Figure 1.2.1). On the eastern side of the Pacific, the whole west coast of Central and South America suffers many earthquakes. In contrast, the eastern part of South America is almost entirely free from earthquakes. In Europe, the counties suffering from earthquakes are Turkey, Greece, Yugoslavia, Italy, Spain and Portugal. An earthquake in south-west Iberia on November 1, 1755 produced a great tsunami, which caused 70,000 deaths in Lisbon and Portugal. On December 27, 1939, in Erzincan, Turkey, 23,000 lives were lost from a major earthquake while similar earthquakes occurred in Turkey and Iran in recent years.

YEAR	LOCATION	FATALITIES	MAGNITUDE
1905	Kangra, India	157,000	8.3
1906	San Francisco, California	700	8.3
1908	Messina, Sicily	160,000	7.5
1920	Gansu, China	100,000	8.6
1923	Sagami Bay, Japan	100,000	8.3
1931	Hawke's Bay N. Zealand	255	7.9
1933	Long Beach, California	120	6.3
1939	Anatolia, Turkey	30,000	6.0
1950	Assam, India	30,000	8.4
1960	Agadir, Morocco	15,000	5.9
1960	Arauco, Chile	2,000	8.4
1962	Iran	12,000	7.3
1964	Anchorage, Alaska	114	8.5
1972	Managua, Nicaragua	10,000	6.2
1976	Guatemala City, Guatemala	23,000	7.5
1976	Tangshan, China	695,000	7.9
1978	Northeast Iran	25,000	7.7
1980	Algeria	20,000	7.5
1980	Southern Italy	3,000	6.8
1985	Mexico City, Mexico	7,000	8.1
1987	Ecuador	4,000	7.3
1988	Armenian USSR	45,000	6.9
1989	Northern California	67	7.1
1990	Northwest Iran	40,000	7.7
1992	Yuacca Valley, California	1	7.4
1992	Flores, Indonesia	2,500	7.5
1993	Northern Japan	185	7.8
1993	South-western India	13,000	6.4
1994	Northridge, California	55	6.7
1995	Kobe, Japan	5,500	7.2

Table 1.2.1 Some of the largest and most devastating earthquakes of the 20^{th} century.

However, destructive earthquakes do occur from time to time in Romania, Germany, Austria, Switzerland and even in the North Sea region and Scandinavia. For example, on October 8, 1927, an earthquake occurred in Austria and caused damage in an area south-east of Vienna. This earthquake was felt in Hungary, in Germany and Czechoslovakia at distances of 250 km from the centre of the disturbance. The seismicity of the North Sea is sufficiently significant to require attention to earthquake resistant design of oil platforms there. Even, in Great Britain, damaging earthquakes occurred in historical times. One recent moderate earthquake there was in north Wales on July 19, 1984.

1.2.2 Causes of Earthquakes

Earthquake is the ground motion caused by a sudden release of energy in the earth's lithosphere, which arises from stresses built up during tectonic processes. The most coherent explanation of the majority of earthquakes is in terms of what is called **plate tectonics.** The basic idea is that the earth's outermost part, called the **lithosphere,** consists of several large and fairly stable slabs called plates, which extend to a depth of 80 km. Moving plates of the earth's surface provide an explanation for a great deal of the seismic activity of the world. The boundaries of the lithospheric plates coincide with the geographical zones, which experience frequent earthquakes. However, while the simple plate-tectonic theory is an important one for a general understanding of earthquakes, it does not explain all seismicity in detail, for within continental regions, away from boundaries, devastating earthquakes sometimes occur.

Earthquakes may also be produced by the underground detonation of nuclear and chemical devices, which release enormous quantities of nuclear energy. Underground **nuclear explosions** produced substantial earthquakes, up to magnitude 7.0, at a number of test sites around the world and resultant seismic waves have travelled throughout the Earth's interior to be recorded at distant seismographic stations. Earthquakes may also be caused by **volcanic eruptions** and, consequently, volcanoes and earthquakes occur together along the margins of plates around the world. Despite tectonic connections between volcanoes and earthquakes, there is no evidence that moderate to shallow earthquakes are not essentially all of tectonic, **elastic-rebound type** (Reid, 1911). Those earthquakes that can be reasonably associated with volcanoes are relative rare.

1.3 CONCEPTS OF SEISMIC DESIGN

1.3.1 Seismic Design Philosophy

The importance of a rational design philosophy becomes paramount when seismic considerations dominate the design because higher risks of damage are accepted under seismic design forces than under other comparable extreme loads, such as live load and wind forces. Design forces are generally too high to be resisted in the elastic range of the material response, and it is common to design for strengths that are a fraction, perhaps as low as 8 to 25 %, of that corresponding to elastic response. The structures are expected to survive an earthquake by large inelastic deformations and energy dissipation corresponding to the material distress. The full strength of the building can be developed while resisting forces resulting from much smaller earthquakes, which occur much more frequently than the design-level earthquake.

In the 1920's and 1930's, the incorporation of seismic design procedures in building design was first adopted. In the absence of reliable measurements of ground motions and lack of detailed knowledge of the dynamic response of structures, seismic inertia forces could not be estimated with any reliability and design for lateral forces corresponding to 10% of the building weight was adopted. By the 1960's, accelerograms giving information on ground accelerations were becoming more generally available. The development of sophisticated computer-based analytical procedures facilitated a closer examination of the seismic response of multi-storey structures and proved that the seismic design was insufficient to ensure an adequate strength to the structures. However, this lack of strength did not always result in failure, or even in severe damage. Provided that the structural strength could be maintained without excessive degradation as inelastic deformations developed, the structures could survive a severe earthquake and be repaired economically. When inelastic deformations reduced the strength of the structure, as often occurs with shear failure, severe damage or collapse was common. It was then concluded that some inelastic modes of deformation lead to failure while others provide ductility that is the essential attribute of maintaining the strength of a structure subjected to reversals of inelastic deformations under seismic excitation.

It has now become accepted that seismic design should encourage structures to possess ductility by carefully choosing the locations where inelastic deformations may occur, termed as plastic hinges. In conjunction with the careful selection of a structural configuration, required strengths for undesirable inelastic deformation modes are deliberately amplified in comparison with those for desired inelastic modes. Thus, for concrete and masonry structures, the required shear strength must exceed the required flexural strength to ensure that inelastic shear deformations, associated with large deterioration of stiffness and strength leading to failure, will not generally occur.

1.3.2 Seismic Design Limit States

It is customary to consider various levels of protection based upon different degrees of efforts to minimise damage that may be caused by a seismic event. While regions of seismicity are now reasonably well defined, the prediction of a seismic event within the projected life span of a building is extremely crude. Nevertheless, estimates must be made for potential seismic hazards in affected regions in an attempt to optimise between the degree of protection and its cost. It must be appreciated that the boundaries between different intensities of ground shaking, requiring each of the following levels of protection to be provided, cannot be defined precisely.

Serviceability Limit State For relatively frequent earthquakes, which induce minor intensity of ground shaking, no damages needing repair should occur to the structure. The design effort should limit displacements and ensure adequate strengths in all components while the structure remains essentially elastic. Reinforced concrete (RC) buildings may develop considerable cracking, but no significant yielding of the reinforcement or crushing of concrete should result. The frequency with which the occurrence of an earthquake corresponding to this limit state should be anticipated depends on the importance of preserving functionality of the building, but the return period is typically in the order of 10-50 years.

Damage Control Limit State For ground shaking of intensity greater than that corresponding to the serviceability limit state, some damage may occur. Yielding of reinforcement may result in wide cracks that require repair and also crushing of concrete may occur. A second limit state may be defined marking the boundary between economically repairable damage and damage that is irreparable or which cannot be repaired economically. Ground shaking corresponding to the damage control limit state should have a low probability of occurrence during the expected life of the building, with a return period of several hundred years.

Survival Limit State In modern seismic design, very strong emphasis is placed on the criterion that any loss of life should be prevented, even during the strongest ground motion feasible for the site. For most buildings, extensive damage due to severe events has to be accepted, but collapse must not occur. Therefore, the designer needs to concentrate on structural qualities to ensure that relatively large displacements can be accommodated without significant loss in lateral force resistance and that the integrity of the structure to support gravity loads is maintained.

1.4 TORSIONAL EFFECTS IN BUILDINGS

1.4.1 Causes of Torsional Building Response

Buildings subjected to earthquake ground motions undergo varying oscillatory motions, which may cause failure or major structural damage, if they have not been appropriately allowed for in the design. Therefore, structural engineers have to be familiar with building properties and earthquake characteristics that invoke torsional, coupled with translational, response when the structures are subjected to seismic activity. The magnitude and influence of the torsional component of the ground motion can only be estimated and incorporated into the analysis and design of the building's load resisting capacity. However, the most critical factor leading to torsional motions in buildings is structural asymmetry caused by mass, stiffness or strength distribution.

Buildings exhibit coupled torsional and translational response to lateral ground motion if their centres of mass CM do not coincide with their centres of rigidity CR calculated at floor levels. The building's centres of rigidity are defined as the set of points at floor levels through which the set of applied lateral forces does not produce rotation of the floor slabs. This can be clearly demonstrated by observing the response of the model shown in Figure 1.4.1. The earthquake ground motion $\ddot{u}_g(t)$ excites the structural model in the lateral y-direction, acting through CM and loads the structure with an inertial force of magnitude $m\,\ddot{u}_g(t)$, where m is the mass of the floor diaphragm. This lateral inertial force is resisted by the structural elements, which induce an equal, but opposite, resisting force acting through CR, which is equivalent to the centre of stiffness CS and the shear centre SC for single-storey systems.

The structure is assumed to possess a uniform mass distribution when CM is located at the geometric centre GC whereas an asymmetric stiffness distribution exists when CR is located away from GC. In this case, the earthquake induced inertia force and the structural resisting force are separated by the stiffness eccentricity or static eccentricity e_s. Thus, such a configuration induces a torsional moment that in turn invokes torsional rotation of the floor diaphragm. This onset of an earthquake-induced torque may also eventuate from mass asymmetry or strength asymmetry.

The maximum response of the structural frame elements may be amplified or deamplified by the presence of torsional response mode coupled with translational response. These effects are particularly critical for elements located near the periphery of the floor diaphragm and their response is either amplified or deamplified depending directly upon the contribution from torsional and translational response modes. If these modes produce displacements that are in phase, the element response is amplified considerably whereas, if out of phase, the response may be decreased.

Furthermore, torsional response may even occur in nominally symmetric structures (where CR and CM coincide with each other) due to accidental eccentricity or torsional ground motion. The accidental eccentricity may be caused by differences between assumed and actual distributions of mass and stiffness, asymmetric yielding strength of the elements, differences in actual and modelled post-yield deformations, non-linear material properties, non-linear force-deformation relationships, contributions of non-structural components, and differences in coupling of the structural foundation with the soil. Moreover, the shifting of CR due to changes in the stiffness distribution of the structural elements can also cause force-deformation relationships that may induce torsion.

A consequence of such non-quantifiable sources of torsion is that buildings, which may appear nominally symmetric with apparently equal distributions of stiffness, strength and mass, may still be prone to some degree of torsional rotations. It is therefore important that current seismic building standards and engineers take account of both the accidental and the non-quantifiable torsional sources, in addition to the dynamic or quantifiable sources, and ensure adequate strength and ductility for structural components that may be prone to severe damage or failure.

1.4.2 Consequences of Torsion in Buildings

The collapse and severe damage of structures subjected to ground motion due to past earthquakes demonstrated mistakes made in structural design and construction, and proved the inadequacy of some building codes regulations. The presence of torsional rotation in structures may induce additional structural response such that the severity of total damage is significantly increased. Code provisions take into account torsional response in order to allocate additional strength to critical elements. However, torsional provisions may in certain cases be inadequate and further research is required.

Mexico City (1985) The Mexico City Earthquake of 19 September 1985 is considered to be a typical example of recent earthquake having great engineering significance. In Mexico, earthquake resistant design provisions were first introduced as early as 1942 and ever since up-dated codes were introduced, including the Emergency Regulations issued after the 28 July 1957 Mexico City earthquake and the 1976 edition of the Mexican Building Code. Despite this, the 1985 Mexico City earthquake of magnitude 8.1 caused about 10,000 human fatalities, 200,000 people lost their homes, 200 buildings collapsed and 1,000 buildings were damaged severely. Among the collapsed buildings, about 70% were built after 1957, when the Emergency Regulations were introduced (Esteva, 1987). Such statistics for a city having been built in accordance with reasonably modern seismic provisions were evidence of the need for further investigations into the effectiveness and adequacy of current building standards.

All post-earthquake field investigation reports identified torsional effects as one of the major causes of failure in buildings. Rosenblueth and Meli (1986) revealed that 15% of the cases of failure were caused by pronounced stiffness or structural asymmetry and that 42% of the buildings, which collapsed or suffered severe damage were corner buildings with asymmetric layout of masonry in-fill walls. A large number of buildings experienced either severe damage or total collapse due to torsional effects caused by the fact that the centres of mass and resistance did not coincide (Popov, 1987). Further inspection of collapsed, or heavily damaged, structures revealed buildings with irregular floor-plan configurations and eccentrically located infilled masonry walls, columns and diagonally braced frames. Permanent torsional deformations were evident in many structures having buckled corner columns (Mitchell et al, 1986).

As a result of this earthquake, emergency code changes were implemented with respect to the 1985 Mexican Building Standard. The consequence of having numerous structures that failed due to torsion imposed a code amendment limiting the torsional eccentricity to only 20% of the plan dimension of the structure, but further analysis have to be conducted on inelastically torsionally responding structures. The lessons learnt from the Mexico City disaster must be succeeded by a detailed re-examination of earthquake resistant design principles of the building codes (Chandler, 1986).

Northridge, California (1994) A more recent example of earthquake-induced torsional effects in buildings was observed in the 1994 Northridge Californian Earthquake of magnitude 6.7, followed by four aftershocks of magnitude 5. These intense earthquake ground motions produced significant torsional effects in reinforced concrete (RC) multi-storey structures, where it was clearly evident that double diagonal shear failures were a direct consequence of torsional rotations (Goltz, 1994).

Kobe, Japan (1995) Since the 1926 code, Japan's seismic codes have been as advanced as any in the world. Japanese engineers upgraded their standards after the 1968 Tokacki-oki Earthquake and the 1971 San Fernando Earthquake. During the period 1971 and 1980, lessons learned in previous earthquakes were included in the design of major buildings and, in the last several years, U.S. and Japanese professionals have been working together to understand the seismic behaviour of the structures and to upgrade seismic design codes. In general, buildings constructed using the current code performed well in the Kobe earthquake. However, a number of newer buildings were severely damaged and structures that also did poorly included older houses and mid-rise concrete structures constructed prior to the early 1980s using the same non-ductile details employed in high-seismic U.S. regions up until the early 1970s. Among the causes for collapse was the irregular distribution of shear walls or concrete frames that resulted in substantial torsion causing the structures to twist as well as to sway due to earthquake loading. Irregular RC multi-storey buildings with random arrangement of shear walls, weak storeys, shears walls stiffer or weaker than the other vertical structural elements, ground soft storeys, isolated columns on flexible sides, large openings in the lowest storeys resulted in severe damage and collapse (EEFIT, 1997). All post-earthquake field investigations recognised torsion as one of the major causes of failure.

1.5 THE NECESSITY FOR THIS STUDY

Symmetric and regular buildings that are properly designed have a much higher ability to survive a strong earthquake than asymmetric buildings and their response to earthquake loading is far more straightforward to predict and design for. On the other hand, even though the response of asymmetric buildings is more unpredictable, designers still have to compromise structural regularity to accommodate functional and aesthetic needs. As a result, serious and widespread damage associated with structural asymmetry has been observed repeatedly in past major earthquakes (see Section 1.4.2).

In the first studies examining torsional effects in buildings, attention was focused on the elastic structural behaviour. The main purpose was to achieve a complete understanding of the effects of mass and stiffness eccentricities and to evaluate them by simple static models. However, as the response of real structures is mainly inelastic, these studies gave poor information on torsional behaviour and interest has moved towards non-linear response studies. The literature review presented in Chapter 2 revealed that specific inconsistencies and discrepancies exist in the current understanding of the inelastic dynamic response of torsionally responding structures. Consequently, it is considered fundamentally important to further investigate and re-evaluate the effectiveness of code defined static torsional provisions. The inelastic response of asymmetric structures is highly dependent on numerous structural parameters and on the simplifying assumptions of the theoretical and analytical modelling. Therefore, it is highly significant to identify the influence of key structural issues and system parameters by employing realistic and, at the same time, simplified structural models.

Regarding the inelastic response of multi-storey torsionally asymmetric buildings, there has been little research carried out and consequently, the seismic code provisions and recommendations are based on conclusions from single-storey research studies (refer also to Section 2.2). It is therefore of crucial importance to analyse multi-storey buildings responding in the inelastic range, to compare their response with the response of single-storey structures and, finally, to recommend possible improvements and alterations to the existing earthquake resistant design codes.

1.6 OBJECTIVES OF THE STUDY

The principal objectives of this study can be summarised as follows:

1. Define consistent, realistic and simple structural models including appropriate structural and response parameters for the investigation of the inelastic torsional response of asymmetric multi-storey structures.
2. Define various types of reference models, examine the factors influencing their inelastic seismic response and choose the most appropriate ones to compare their response to the response of corresponding asymmetric buildings.
3. Investigate the effect of torsion on the inelastic dynamic response of a wide range of multi-storey regularly asymmetric structures with different structural characteristics.
4. Study the factors that influence the inelastic response of the asymmetric models, i.e. mass, stiffness and strength distributions, the number of floor levels, the amount of the static eccentricity, the incorporation of the accidental eccentricity provisions, and the influence of the static torsional provisions of different seismic codes.
5. Present analytically the section design factors, which influence the response of torsionally asymmetric models, i.e. the minimum reinforcement requirements imposed by the seismic codes for the design of the structural elements and the capacity design.
6. Examine first the inelastic response of torsionally asymmetric multi-storey structures having structural elements oriented only perpendicular to the direction of the earthquake ground motion and loaded by the lateral earthquake component (uni-directional loading).

7. Then investigate the response of asymmetric models including transverse structural elements and subjected simultaneously to both earthquake components (bi-directional loading) and compare their response to the response of asymmetric models without transverse elements.
8. Compare the static torsional provisions of different seismic codes and evaluate their effectiveness to appropriately design torsionally asymmetric structures.
9. Assess the efficiency of a new proposed optimised method and investigate whether this method can be successfully applied to multi-storey asymmetric structures.
10. Propose any possible recommendations and improvements for the existing torsional provisions of the earthquake resistant codes and indicate areas for future research.

1.7 ORGANISATION OF THE THESIS

The following outline discusses the general content of the thesis and provides a logical sequence by which each of the objectives stated in the previous section are fulfilled, investigated and presented.

Chapter 1: Introduction and Objectives provides some general information on the nature of earthquake ground motions, presents the seismicity of the world, and indicates the causes of the earthquake events. Moreover, the basic concepts of current seismic design philosophies and the reasons of torsional building response are also presented, followed by the necessity, the objectives and the organisation of the thesis.

Chapter 2: Literature Review presents an historical review and assessment of the research carried out on the elastic and inelastic response of single-storey and multi-storey asymmetric buildings. A brief discussion is given on the conclusions drawn from single-storey elastic analyses, followed by a more detailed examination of investigations into the inelastic torsional behaviour of single-storey structures. More emphasis is placed to the recent elastic and inelastic studies of multi-storey structures. The final part of this chapter is organised in the form of major issues and includes a detailed presentation of the fundamental structural definitions widely used in torsional studies.

Chapter 3: Structural Parametric Modelling is concerned with the structural configuration and geometry of the models incorporated in this study. The definitions of the system parameters adopted are specified and critical structural parameters are also analysed.

Chapter 4: Design Analysis Procedure concerns the dynamic parametric modelling decisions and assumptions of the thesis. It presents the elastic analysis methodology and the torsional provisions of the codes incorporated in the design. Details about the reinforcement design of the structural elements are included, and information about the non-linear dynamic analysis is presented concerning the selection of the seismic input, the computer program specifications and the response parameters adopted.

Chapter 5: Inelastic Seismic Response of the Reference Models examines the inelastic dynamic response characteristics of various reference models. Reference models with different mass, stiffness and strength distributions are analysed, compared and assessed for their ability to represent an acceptable level of inelastic response. The effect of including, or excluding, the section design requirements and the accidental eccentricity provisions, as stipulated by the seismic codes, are among the parameters examined.

Chapter 6: Inelastic Seismic Response of Multi-storey Regularly Asymmetric Models presents the results of inelastic dynamic analyses performed to identify the influence of various structural and design parameters. Asymmetric systems having different structural configurations and designed according to various seismic codes enable the performance the code provisions to be evaluated and assessed for their effectiveness in controlling the additional responses arising in asymmetric structures. The design parameters considered include the accidental eccentricity provisions, the influence of the minimum reinforcement requirements for the design of structural elements, the static eccentricity, the number of floor levels and the stiffness distribution of the models.

Chapter 7: Comparison of the Inelastic Seismic Response of Multi-storey Regularly Asymmetric Models to the Response of their Reference Models compares the inelastic response of the asymmetric multi-storey models (examined in Chapter 6) to the response of their corresponding reference models. The influence of the factors investigated in Chapter 6 is re-evaluated based on the comparison between the behaviour of various asymmetric models and the behaviour of similar models responding purely in translation.

Chapter 8: Inelastic Seismic Response of Multi-storey Regularly Asymmetric Building Models with Transverse Frame Elements extends the investigations of torsionally asymmetric structures by examining structural models incorporating frame elements oriented along both orthogonal axes. The effect of including additional elements perpendicular to the ground motion direction is assessed for various code-defined structural systems. Furthermore, the effect of including the second earthquake component to simultaneously excite the structural models is quantified with respect to some key structural configurations and system parameters.

Chapter 9: Evaluation of a Recently Proposed Equivalent Static Force Procedure presents the relative merits and deficiencies of the existing seismic code provisions based on the results of the inelastic analyses carried out in Chapters 6 – 8 and examines the applicability of a new method proposed by Duan and Chandler (1997).

Chapter 10: Conclusions and Recommendations for Future Research summarises all the fundamental conclusions reached from the investigations conducted throughout the thesis. Various topics for future research are proposed, which are considered to be both necessary and pertinent for increasing and refining the understanding of the complex behaviour of multi-storey torsionally asymmetric buildings.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Buildings are seldom symmetric and almost invariably have non-symmetrical distribution of mass and/or stiffness in plan. Even in structures with symmetric geometry, possible variations in quality, or method of construction, and uncertainties in live and dead load distributions may introduce asymmetry (refer also to Section 1.4.1). Asymmetry results in significant coupling between the translational and torsional response of the structures and, as a result, the induced lateral and torsional forces can exceed the design values and cause widespread damage or failure. Chandler (1986) observed that during the Mexico City Earthquake, 1985, 15% of severe damage, or collapse, was caused by pronounced stiffness asymmetry (refer to Section 1.4.2).

Ayre (1938) published the first journal papers regarding the translational and torsional response of structures and concluded that torsional rotations in structures should be eliminated, or controlled, by appropriate design measures. These papers were the beginning of a long series of investigations, which endeavoured to comprehend the effects of translational and torsional coupling in elastically responding single-storey systems. In the 1970's, the number of studies on torsionally responding structures to earthquake loading was drastically increased due to the general and widespread availability of computer power necessary for detailed dynamic parametric analyses. Extensive research and numerous investigations were conducted into torsional building response and structural models were subjected to real seismic ground motions.

In early studies, the response of asymmetric structural systems was assumed elastic and only later the inelastic response of torsionally responding structures was investigated. During the last few years, further research was conducted on the inelastic response of multi-storey structures, and it was observed that the response of torsionally asymmetric multi-storey buildings couldn't be accurately predicted by simplified single-storey model analysis. In Section 2.2, the fundamental conclusions drawn from single-storey elastic and inelastic studies are outlined. In an effort not to be repetitive, reference is made to published literature reviews for such studies whereas a more detailed overview of recent literature (Section 2.3) is focused on the complex problem of torsion in multi-storey elastic and inelastic systems.

2.2 REVIEW OF SINGLE-STOREY STUDIES

2.2.1 Single-storey Elastic Studies

The elastic response of single-storey structures has been analysed extensively, since Kan and Chopra (1977c) indicated that the coupled elastic response of a multi-storey regularly asymmetric building with N-storeys may be approximated by the dynamic properties of a symmetric (uncoupled) N-storey model and an equivalent asymmetric coupled single-storey model. Therefore, elastic parametric studies have been focused on the understanding of the response of coupled single-storey systems, since their behaviour may be generally applicable to particular cases of multi-storey structures. Numerous researchers conducted investigations into the elastic response of asymmetric structures by employing various structural models and parameters. The key parameters identified as significantly affecting the elastic response of asymmetric structures are the static or stiffness eccentricity e_s, the uncoupled torsional to lateral frequency ratio Ω, the uncoupled lateral period of the system T_y, and the structural damping ratio ξ.

Various researchers confirmed the effect of the static eccentricity and of the torsional to lateral frequency ratio on the elastic response of the structures. The effect of the uncoupled lateral period could not be identified due to the idealised response spectrum analysis used in many studies (Kan and Chopra, 1977a, 1977b, 1977c;

Dempsey and Irvine, 1979; Tso and Dempsey, 1980; Tsicnias and Hutchinson, 1981, 1982). The significant influence of the fundamental period could be observed in studies that adopted time-history analysis, using real earthquake records (Dempsey and Tso, 1982; Chandler and Hutchinson, 1986, 1987a, 1987b, 1988a, 1988b; Rutenberg and Pekau, 1987). Other researchers who also studied the torsional response of linear asymmetric buildings include Penzien (1969), Gibson et al. (1972), Douglas and Trabert (1973), Keintzel (1973), Rutenberg et al. (1978), Hejal and Chopra (1989a, 1989b). The achievements of elastic single-storey studies were summarised and reviewed by Chandler (1988) and led to important fundamental conclusions regarding the torsional elastic structural response, which could be summarised as follows:

1. Lateral-torsional coupling induces building responses under earthquake loading that, in certain cases, are larger than accounted for in current code provisions. A dynamic torque is produced about the centre of resistance, which may significantly exceed the product of the horizontal storey shear and of the static eccentricity. This is also supported by damage evidence from post-earthquake surveys.
2. Corresponding amplification of the dynamic torque results in reductions of the horizontal storey shears relative to the equivalent symmetric, or uncoupled, structure.
3. When the building's natural torsional and lateral frequencies are close, strong coupling occurs between the lateral and the torsional motions in essentially symmetric systems and the effects of the two previous conclusions are amplified.
4. Proper design procedures for torsional effects should include consideration of a number of factors: building frequencies, locations of a building's centres of rigidity, adequate design of vertical members, accidental eccentricity effects due to a variety of causes, and non-linear structural response.

2.2.2 Single-storey Inelastic Studies of 1980's

During the 1980's, researchers started to investigate the inelastic torsional response of single-storey asymmetric buildings subjected to intense earthquake ground motions and loaded beyond their yield capacity. These early investigations (Irvine and Kountouris, 1980; Kan and Chopra, 1981a, 1981b; Syamal and Pekau, 1985; Tso and

Sadek, 1984, 1985; Bozorgnia and Tso, 1986; Tso and Bozorgnia, 1986; Bruneau and Mahin, 1987; Sadek and Tso, 1988, 1989; Chandler and Duan, 1990, 1991a) drew contradictory conclusions regarding the influence of various structural parameters on the inelastic torsional response of asymmetric structures. The response of torsionally asymmetric systems loaded into the inelastic range was found to be more complicated than their elastic response and highly dependent on the structural model configuration. In addition to the parameters influencing the elastic structural response, the inelastic response is also dependent on the location, number and force-deformation characteristics of the load-resisting elements. Since the structure is loaded beyond its yield capacity, its response is also dependent on the yield distribution and strength of the individual elements. Chandler and Duan (1990, 1991a) identified the contradictory conclusions from the early inelastic studies of single-storey structures and pointed out the sources of such discrepancies. A brief outline of the most important conclusions drawn from early inelastic studies is presented in the following sections.

(a) Uncoupled Torsional to Lateral Frequency Ratio

Although the uncoupled torsional to lateral frequency ratio Ω was found to be a very important structural parameter for the response of elastic systems, for structures responding in the inelastic range contradictory results were presented mainly due to different definitions employed (refer also to Section 2.4.2). One way to define Ω (Equation (2.4.6)) is to vary the mass radius of gyration r_m of the floor diaphragm (Equation (2.4.8)) by varying the plan aspect ratio λ of the structure. A second way to define Ω is to change the torsional stiffness K_θ of the system by changing the position or the stiffness distribution of the structural elements.

Bruneau and Mahin (1987) employed the first approach and observed that the ductility demand of the flexible-edge element increases with increasing Ω while other researchers (Tso and Sadek, 1984, 1985; Bozorgnia and Tso, 1986; Syamal and Pekau, 1985) employed the second approach and found the opposite. Although the effect of Ω is highly complex, the approach to define it by varying the torsional stiffness of the system is more appropriate because the positions of the structural elements do not change and the models employed correspond to physically real structures.

(b) Number of Load Resisting Elements

The inelastic response of asymmetric models is highly affected by the structural configuration and the number of structural elements. Kan and Chopra (1981a, 1981b) employed a single-element model, which was inadequate to provide information about the ductility demand of critical elements. Furthermore, the two-element model employed by Irvine and Kountouris (1980) and Bruneau and Mahin (1987) was statically determinate and underestimated the peak ductility demand of the flexible-edge element. Later studies (Tso and Sadek, 1984, 1985; Tso and Bozorgnia, 1986) proved that a three-element model (refer also to Section 3.2.1) could adequately predict the ductility demand of the structural elements.

(c) Stiffness Eccentricity

The influence of the stiffness eccentricity (Section 2.4.3) depends on the number of the load-resisting elements of the models employed. Studies employing two-element models (Irvine and Kountouris, 1980) indicated that the stiffness eccentricity does not influence the ductility demand of the structural elements. This contradicts conclusions of studies with three-element models (Tso and Sadek, 1984, 1985; Bozorgnia and Tso, 1986), which are statically indeterminate and hence dependent upon the stiffness distribution. Based on the investigation of three-element models, the ductility demand of the flexible-edge element increases significantly for moderate to large stiffness eccentricity and for Ω less than unity.

(d) Strength Eccentricity

Contradictory remarks were also reached about the influence of the strength eccentricity due to studies employing models with different numbers of structural elements. Increasing the strength eccentricity increased rapidly the element ductility demand, particularly for systems with low uncoupled torsional to lateral frequency ratios, while, for models with similar stiffness and strength eccentricities (having small to moderate eccentricities), the element ductility demand was sensitive to both of them.

2.2.3 Single-storey Inelastic Studies of 1990's

This section contains a critical review of the latest inelastic studies of single-storey asymmetric structures while the findings of the research conducted after 1990 are presented based on knowledge from earlier inelastic studies. Various researchers (Goel and Chopra, 1990a, 1990b, 1991a, 1991b; Chopra and Goel, 1991; Tso and Ying, 1990, 1992; Tso and Zhu, 1992a, 1992b; Zhu and Tso, 1992; Rutenberg et al., 1992a, 1992b; Duan, 1991; Duan and Chandler, 1993; Chandler and Hutchinson, 1992; Correnza et al., 1992a, 1992b; Correnza, 1994; Chandler et al., 1994, 1995; De Stefano et al., 1993a, 1993b) investigated the influence of key structural parameters and evaluated the effectiveness of current code torsional provisions. The conclusions drawn are still ambiguous due to inconsistencies in model design, structural configuration, definition of parameters, and application of different code provisions. The fundamental conclusions from the 1990's inelastic single-storey studies may be summarised as follows:

1. Inelastically responding asymmetric systems need additional structural parameters to be defined: strength distribution, overstrength ratio, strength reduction factor, and characteristics of the load-resisting elements.
2. Increases in the uncoupled torsional to lateral frequency ratio represent increase in the structure's torsional stiffness and, therefore, result in decreasing torsional effects.
3. Stiffness-eccentric structures give larger inelastic torsional response compared to mass-eccentric structures for similar stiffness and strength distributions whereas the opposite is true for a strength distribution nearly symmetric.
4. In code-designed systems, there is little strength eccentricity because the strength is distributed symmetrically about the centre of mass. As a result, there is only minimal additional ductility demand on the flexible side whereas large additional element deformations may still occur.
5. Additional displacement demand at the flexible-side elements is dependent on the static eccentricity, the torsional stiffness of the structure, and the distance of the element from the centre of rigidity while it is largely independent of the design code employed and whether the system is mass-eccentric or stiffness-eccentric.

6. For the stiff-side elements, only moderate strength reductions should be allowed while, for the flexible-side elements, a strength increase should be employed in order to control additional torsional response.

The following parameters have been identified as areas of concern, where the use of differing definitions, or the making of divergent assumptions, has resulted in a basic lack of agreement between the results and conclusions of various research studies:

(a) **Accidental Eccentricity**

The accidental eccentricity may be caused by a torsional ground motion component, by variations in material characteristics and by differences between actual and assumed distributions of mass, stiffness and strength. The inclusion, or not, of the accidental eccentricity component, as stipulated in the static torsional provisions of the existing seismic codes, has been a source of inconsistency in approaches employed by various researchers. In many studies, the accidental component was excluded from the determination of strength distribution in asymmetric systems (Duan, 1991; De Stefano et al., 1993a, 1993b). This was justified on the premise that if the accidental eccentricity is not modelled in the dynamic analysis, the performance of the structure may not realistically represent torsional structural behaviour.

The opposite approach was employed by other researchers (Tso and Ying, 1990, 1992; Rutenberg et al., 1992a, 1992b; Tso and Zhu, 1992a, 1992b; Zhu and Tso, 1992) who included the accidental eccentricity in the design eccentricity expressions and, therefore, increased the strength of the structural elements and reduced their response. Wong and Tso (1994) noticed that using the response spectrum analysis without including accidental torsion could lead to excessive additional ductility demand on the stiff-edge element. Moreover, Chandler and Duan (1993) concluded that including the accidental eccentricity in the design eccentricity expressions and ignoring the uncertainties and the torsional ground motion in the inelastic dynamic analysis is misleading. Therefore, the influence of the accidental eccentricity is difficult to quantify from the results of previous studies due to the incompatible model configurations, normalisation procedures, and response parameters (refer to Section 2.4.4).

(b) Reference Structural Models

In order to access the influence that torsional oscillations induce on a structure, a reference model is commonly adopted, and its inelastic dynamic response is compared to the response of the torsionally unbalanced (TU) model. The reference model has the same fundamental properties as the TU model, such as structural period, total lateral stiffness and total torsional stiffness. Two different reference systems have commonly been employed in the past: the symmetric reference model (SM) and the torsionally balanced reference model (TB) (refer also to Section 3.3).

The SM model has been employed by many researchers (Tso and Ying, 1990, 1992; Rutenberg et al., 1992a, 1992b; Duan, 1991; De Stefano et al., 1993a, 1993b) and its centres of mass, rigidity and strength coincide with the geometric centre of the floor diaphragm. The structural elements of the SM models have the same stiffness and strength distribution in order that the system responds purely in translation, in both the elastic and inelastic ranges. However, a TU stiffness-eccentric system has a different element strength distribution than the SM model and, therefore, no consistency is maintained between the structural configuration of the models. The TB reference system is more general than the SM system because it retains the same structural parameters as the TU model and maintains also the same stiffness distribution between the structural elements (Tso and Zhu, 1992a, 1992b; Zhu and Tso, 1992). The TB model, as concluded by several researchers (Correnza, 1994), can provide an appropriate base for assessing the significance of the inelastic torsional effects in asymmetric (TU) models.

The actual response of the reference model is critical in evaluating the performance of code-designed TU systems and, therefore, the reference system should also be designed with respect to the code provisions. As discussed by Correnza et al. (1992a, 1992b) and Chandler et al. (1994), such a reference model definition requires the accidental eccentricity to be incorporated in the design of the structural elements and increases their strengths.

(c) Natural Lateral Period

Although the influence of the natural lateral period is very important for the elastic response of the structures, its influence in the inelastic range of response is somewhat unclear since it is highly dependent on the structural configuration. Tso and

Ying (1990) observed that the inelastic response of the flexible side decreases with increasing period for highly inelastic systems, but for moderate inelastic systems no change was observed. In a later study, Tso and Ying (1992) observed that when the strength is distributed as a direct proportion of stiffness, the ductility demand at the stiff side increases with increasing structural period.

Rutenberg et al. (1992a, 1992b) found that the peak ductility demand is larger for short-period structures and decreases with increasing period for both stiffness-eccentric and mass-eccentric systems. The maximum displacement increased with the structural period and it was dependent on the frequency content of the earthquake ground motions. Duan (1991) noticed that the response of the structural elements decreased with increasing period and, the ductility demand was particularly large for very short-period systems. De Stefano et al. (1993a, 1993b) observed that structures generally sustain large degrees of damage in short-period range whereas, with increasing period, the damage sustained is reduced and remains constant with further increases in the period.

Goel and Chopra (1991a, 1991b) concluded that, for systems with high static eccentricities, the increase of torsional deformation is larger in short-period structures while, in systems with small static eccentricity, the largest increase in torsional deformation occurred in medium-period systems. Finally, Chandler and Hutchinson (1992) concluded that short-period systems exhibit greater coupling of lateral and torsional response and that the flexible side is more affected, especially for highly eccentric models. Hence, the period affects the inelastic response of asymmetric systems, but still needs further research to be carried out (refer also to Section 2.4.1).

(d) Transverse Elements

The contribution of the transverse structural elements (elements oriented perpendicularly to the earthquake direction) to the effective torsional stiffness and strength of inelastic asymmetric structures is highly significant. Goel and Chopra (1990a, 1990b) and Chopra and Goel (1991) concluded that, when laterally oriented elements (elements oriented parallel to the earthquake direction) are loaded into the inelastic range, the transverse structural elements remain elastic and they contribute to the total torsional stiffness of the system. Asymmetric systems in the short-period range are affected significantly by the inclusion of transverse elements due to the increased levels of yielding occurring in the short-period range.

Moreover, Duan (1991) observed that, when transverse structural elements are included, the structure should be analysed with a bi-directional earthquake loading. Furthermore, he noticed that the transverse structural elements are not particularly important in the short-period range. This discrepancy in results was explained by the fact that Goel and Chopra (1991) used a reduced design spectrum in the short-period range and, therefore, systems with short structural periods would undergo excessive yielding. Thus, the contribution of transverse structural elements to the inelastic seismic response of asymmetric structures is very important and further research should be carried out (see also Section 3.4.6).

2.3 REVIEW OF MULTI-STOREY STUDIES

Real multi-storey building structures have a much more complex behaviour than single-storey buildings and they are significantly affected by higher vibration modes. In RC structures, the strength design depends strongly on the minimum reinforcement requirements of the design codes, the standardisation of reinforcement arrangements in structural elements, and the capacity design criteria adopted by most modern seismic codes. Unfortunately, there is lack of investigations on multi-storey buildings that take into account all these aspects.

2.3.1 Multi-storey Elastic Studies

Douglas and Trabert (1973) examined the coupled bending and torsional vibrations of a nominally symmetric 22-storey RC building and noticed that its response was influenced by the choice of the accelerogram and its orientation with respect to the structure. The simultaneous application of two orthogonal components of ground motion significantly influenced the response of elements in systems with coupled bending and torsional modes of vibration. For structures fairly long and narrow in plan, the rotation of the floor diaphragm had a significant effect on the response of specific frames and the importance for providing ductility in the design of buildings was proved.

Boroschek and Mahin (1992) examined the dynamic torsional behaviour of an existing 13-storey frame building and its response was recorded during different earthquakes. Parametric studies were also performed on analytical models to examine the effect of material and geometric non-linearity, accidental eccentricity and bi-directional ground motion. The torsional behaviour in nearly regular space frames increased the ductility demand in elements located far away from the centre of rotation. These effects were more severe for elastic structures, rather than inelastic ones, and they were highly dependent on the ground motion characteristics and the static eccentricity.

Chandler et al. (1993) examined the validity of the extrapolation of single-storey asymmetric building results to a certain special class of multi-storey buildings. For these multi-storey buildings, the locations of the centres of mass and rigidity were found on two vertical straight lines (regularly asymmetric models). Although there were qualitative similarities between the seismic torsional response of the special class of multi-storey buildings and the corresponding single-storey models, the contribution of the higher translational and torsional modes in the response of the multi-storey models could lead to significant numerical differences. The distribution of torsional coupling effects was not uniform over the model height, and it was concluded that, for the purpose of parametric studies, it was reasonable to adopt a uniform vertical mass distribution and a gradually tapered vertical stiffness distribution. The single-storey approximation was found to be satisfactory for estimating the torsional coupling effects in the lower floor levels of multi-storey buildings, particularly for low-rise buildings and/or those with large static eccentricities. Multi-storey structures exhibited greater reduction in storey shear in comparison with equivalent single-storey models, especially in the upper floors of taller buildings having large static eccentricities. The response of the upper storeys was usually less well approximated by the single-storey model.

For buildings with irregular distribution of static eccentricity, the amplification of eccentricity for storeys without any static eccentricity was of comparable magnitude to storeys with non-zero eccentricities. The presence of large static eccentricities at the upper floor levels of multi-storey buildings produced greater values of eccentricity amplification at those levels than at the lower levels. However, for the inverse case with large eccentricities in the lower storeys, the vertical distribution of the eccentricity amplification was nearly uniform. The torsional response at a given level was not solely influenced by the magnitude of the static eccentricity at that storey, but it was equally, or even more, influenced by the eccentricities at other storeys.

2.3.2 Multi-storey Inelastic Studies

Sedarat and Bertero (1990a, 1990b) considered a 7-storey RC frame-wall structure with seven by three bays and a symmetrical distribution of stiffness was assumed while the centres of mass were offset. The same mass, stiffness and resistance distributions were taken at all stories. The overall behaviour was mainly governed by the formation of plastic hinges at the base of the two walls. Although the results depended strongly on the design criteria and the structural type, a general statement was that torsional response can be efficiently controlled by proper design of yielding strength and that due attention should be given to the redundancy of the system.

Dolce and Ludovici (1992) evaluated the response of 3-storey RC framed structures with eccentric mass designed according to Eurocode 8 and Eurocode 2. The results of the non-linear 3D dynamic analysis showed that, if the strength design is based on 3D dynamic analyses and the Eurocode 8 detailing provisions are met, the inelastic demands in the columns of eccentric structures can be comparable, and sometimes less, than in similar symmetric structures. The strength design for vertical loads, the code detailing provisions and the reinforcement arrangements affected strongly the non-linear response of structural elements. In particular, minimum steel requirements influenced considerably the behaviour of columns while vertical load stresses and detailing rules affected mainly the behaviour of beams. However, it was proved that a correct strength design can reduce storey rotations, but still some significant interactions between structural and non-structural elements can occur in flexible structures.

Duan and Chandler (1993) employed a simple multi-storey asymmetric frame model to re-evaluate certain of the main conclusions of earlier comparable studies on single-storey models. They concluded that unlike elastic studies, single-storey models were not sufficient to investigate the inelastic torsional effects in multi-storey regularly asymmetric buildings and, therefore, multi-storey models should be employed. The inelastic response of the upper columns at the stiff side of the model increased significantly with the building's fundamental uncoupled lateral period and with the magnitude of stiffness eccentricity. Moreover, in equivalent static design, the application of a concentrated top force reduced significantly the inelastic response of the upper columns compared to the modal analysis procedure and the simple linear distribution of the base shear. This top concentrated force was essential to control the inelastic response of the upper-storey columns in medium-period structures and to some extent in short-period structures, but it is not included in certain current building codes (Eurocode 8). Furthermore, the application of the equivalent static torsional provisions to the design of asymmetric buildings could lead to non-conservative estimates of response, particularly for structures with intermediate or large stiffness eccentricity. In these cases, the critical stiff-edge elements were vulnerable to excessive additional ductility demand and suffered significantly more structural damage than in corresponding symmetric buildings. Finally, regularly asymmetric buildings excited well into the inelastic range were not conservatively designed using modal analysis.

De Stefano et al. (1995) investigated the seismic response of 3D RC asymmetric multi-storey buildings designed according to EC8 for high ductility structures and subjected to different earthquake records. The capacity design rules activated the desired strong column-weak beam behaviour, even though the columns of the external frames experienced rather large yielding. The comparison of the maximum top storey displacements and interstorey drifts with well recognised threshold values showed that the considered mass-eccentric buildings did not reach collapse and only the flexible side frames were significantly damaged. However, the input ground motion significantly influenced the results and, as expected, the most critical conditions were found under earthquakes characterised by large amplifications of the ground accelerations in the period range of the structures. Thus, it was concluded that asymmetric buildings designed by Eurocode 8 for high ductility RC structures were able to resist rather severe earthquakes. However, such irregular buildings were provided with a lower degree of safety against collapse compared to that of their symmetric counterparts.

De La Llera and Chopra (1996) studied the inelastic behaviour of asymmetric multi-storey buildings, which showed similar trends to the response of single-storey structures. The strength of the resisting planes, the intensity of ground motion, the stiffness and strength asymmetry affected the inelastic behaviour of the models. Stiffness and strength asymmetry controlled the torsional performance of the structure, with stiffness asymmetry influencing the elastic response and strength asymmetry guiding the inelastic behaviour. The increased strength in the resisting planes orthogonal to the direction of ground motion led to more uniform deformation demands among the resisting planes in the direction of motion. Moreover, the torsional unbalance of very asymmetric systems could be corrected by manipulating strength and stiffness distributions whereas other retrofit situations required the use of stronger orthogonal resisting planes or lumping of strength close to the centre of mass.

Paulay (1996, 1997a, 1997b, 1997c, 1997d, 1998a, 1998b) published many papers on the inelastic seismic response of structures subjected to torsion. In these publications, he proposed a simple approach based on force-displacement relationships to the consideration of the torsional effects in the seismic design of ductile buildings (Paulay (1996, 1997a)). Instead of increasing torsional strength, he emphasised the importance of controlling twist by assuring that some residual torsional stiffness of the system is available. Some well-established parameters, such as yield displacement, element and system stiffness, were redefined to enable the inelastic deformation pattern of rigid diaphragms to be simply quantified. He suggested that in order to eliminate in ductile structures possible adverse effects of excessive twist, some residual torsional restraint should be provided by elastic elements, while overstrengths under imposed inelastic displacements are being developed in other elements. Paulay (1996) concluded that, instead of a relatively involved dynamic analysis to access the ductile torsional response of systems, a procedure based on the equivalent static method and addressing displacement control is adequate.

Paulay (1997b) reviewed the torsional provisions of the current seismic codes and recommended that torsional phenomena be treated for two distinct conditions: elastic response associated with serviceability limit state (SLS) and ductile response relevant to the ultimate limit state (ULS). Existing building codes imply elastic response and they should be considered applicable only to serviceability limit state criteria. The required magnification of the stiffness eccentricity with the use of the accidental eccentricity appeared to be adequate. Predictions of elastic response were much more reliable than those relevant to ductile response and, therefore, it was concluded that there is no need to impose limits on stiffness eccentricities. The designer should be able to ascertain whether torsional response satisfies the deflection and the strength criteria of the serviceability limit state.

Furthermore, Paulay (1997b) noted that the design criteria for the ductile seismic response of the buildings were satisfied when earthquake-induced deformations (system twist) were such as to limit the expected displacement ductility demand on any element to its specified ductility capacity and its interstorey displacements to their acceptable limits. For torsionally restrained systems, having lateral elastic elements transverse to the base shear under consideration, the torque to be resisted was based on the strength eccentricity. From the yield displacement of the ductile translatory elements, the yield displacement of the system at CM could be estimated and limited to ensure that the displacement ductility capacity of the critical elements was not exceeded. In torsionally unrestrained systems, where transverse elements capable of resisting a torque within the elastic domain are absent, ductility relationship between the elements and the whole of the system was based on the assumption that one of the edge elements would not yield. The displacement ductility demand of the system needed to be reduced to values less than those assigned to ductile elements and the total design base shear was based on the reduced ductility capacity of the system.

Paulay (1997c) examined once more the proposed displacement-based design approach to earthquake-induced torsion in ductile structures. The torsional restraint provided by some elastic elements was proved to be a beneficial property of ductile systems. When the transverse elements, providing torsional restraint, responded well within the elastic domain, their displacements and the consequent twist of the system was small compared with the inelastic displacements of the translatory elements. It was thus considered that the effect of torsional inertia on the ductility demands imposed on translatory elements of torsionally restrained systems is likely to be small enough to be neglected in design. In contrast, torsional inertia effects on torsionally unrestrained ductile systems could be very significant. Finally, he concluded that under a skew earthquake attack, all lateral force-resisting elements may enter the inelastic range of response and the torsional restraint of the system may vanish.

Paulay (1997d) examined the seismic torsional response of ductile structural wall systems and noted that with a distinct design rather than analysis-oriented approach, some torsional phenomena arising during the elasto-plastic seismic response of building structures are addressed. The proposed design procedure enabled the designer to apply displacement-based principles without undue restrictions, to impart to the structure the most desirable features of seismic response. The design approach proposed imposed more severe limitations on structures without transverse elements than those embodied in existing seismic code provisions.

Finally, Paulay (1998a, 1998b) presented again the mechanism-based design strategy for the torsional seismic response of ductile buildings, a strategy very different from that embodied in current seismic codes. Two fundamentally different mechanisms were postulated. In the first mechanism, elastic transverse elements were assigned to resist torsion and control twist while translatory elements were subjected to different inelastic displacements (torsionally restrained system). In the other system, preferably to be avoided, elements capable of resisting torsion during inelastic translatory response were absent or inadequate (torsionally unrestrained system). Torsionally restrained mechanisms subjected to inelastic skew degenerated into torsionally unrestrained mechanisms.

2.4 KEY ISSUES IN MODELLING TORSIONAL EFFECTS

2.4.1 Uncoupled Lateral Period

The uncoupled lateral period T_y in the y-direction is a general, yet critical, structural parameter of the torsional response and a measure of the total lateral stiffness K_y of the structure for a given mass m. The fundamental lateral period for a TB model, or the equivalent uncoupled lateral period for a TU structure, is defined as

$$T_y = 2\pi\sqrt{\frac{m}{K_y}} \tag{2.4.1}$$

The uncoupled lateral period describes the oscillatory characteristics of a structure and is usually related empirically to the number of storeys, or the model height, and the structural system employed. Earthquake resistant design codes employ this parameter to limit storey drift and to define drift indices (see Sections A.4, B.10 and C.3.7). Furthermore, based on the lateral period, the strength demand of a structure can be defined by earthquake design spectra and each seismic code provides formulas for the approximate calculation of the structural period (refer to Sections A.5.1, B.4 and C.3.2). Finally, in the elastic range of response, depending on the structural lateral period, the maximum ground acceleration, velocity, or displacement controls the maximum dynamic response of the structure.

2.4.2 Uncoupled Torsional to Lateral Frequency Ratio

The uncoupled torsional to lateral frequency ratio Ω is a system parameter that strongly influences the elastic response of eccentric buildings, but opinions differ regarding its importance in the inelastic range. Tso and Sadek (1985) reported that, unlike the elastic studies, the coincidence of the uncoupled lateral and torsional frequencies (Ω equal to unity) does not lead to abnormally high inelastic response. Furthermore, Bozorgnia and Tso (1986) observed that Ω does not appear to be a critical parameter for estimating ductility demand. On the other hand, Syamal and Pekau (1985) found that the peak ductility demand on the stiff element decreases with increasing Ω while Goel and Chopra (1991b) concluded that the deformation of the stiff element decreases with decreasing Ω.

Finally, Annigeri at al. (1996) reported that the inelastic response of eccentric systems having Ω equal to unity is not critical, and the variation in the ductility ratio depends on the Ω definition, on the structural period, and on the strength distribution among the lateral elements. Furthermore, they observed that the ductility demand on the stiff-edge element is affected by Ω and generally decreases with increasing Ω while the ductility demand on the flexible-edge side generally increases with increasing Ω. The observations made by these researchers are contradictory due to the use of different model types, definitions of Ω, structural periods, and strength distributions.

The uncoupled frequency ratio has been defined in three different ways by various investigators, depending upon the definition of the torsional stiffness K_θ and the mass moment of inertia I_m. For example, Goel and Chopra (1991b) and Irvine and Kountouris (1980) considered K_θ at the centre of stiffness (CS) and I_m at the centre of mass (CM). Syamal and Pekau (1985) and Kan and Chopra (1981a, 1981b) considered both at CM, and Tso and Sadek (1985) and Bozorgnia and Tso (1986) considered both at CS. Evidently, the differences amongst the three alternative Ω definitions increase with eccentricity. Systems with identical Ω values but with different definitions employed, represent different dynamic systems, and their results are not in agreement. For parametric studies, the Ω definition with K_θ calculated at CS and I_m calculated at CM has the advantage, because, for a given value of Ω, it is possible to vary the eccentricity without changing any of the physical parameters. The stiffness distribution and the structural system remain intact while CM is shifted. Obviously, the value of Ω depends upon the definition employed and it is generally described by

$$\Omega = \frac{\omega_\theta}{\omega_y} \tag{2.4.2}$$

where ω_y represents the fundamental lateral frequency of the system defined as

$$\omega_y = \sqrt{\frac{K_y}{m}} \tag{2.4.3}$$

and ω_θ represents the fundamental torsional frequency of the system defined as

$$\omega_\theta = \sqrt{\frac{K_\theta}{I_m}} \tag{2.4.4}$$

K_y is the total lateral stiffness of the system, K_θ is the total torsional stiffness at CS, m is the mass of the system, and I_m is the mass moment of inertia at CM given by

$$I_m = m\left(\frac{a^2 + b^2}{12}\right) \tag{2.4.5}$$

where a and b are the floor plan dimensions. Thus, Ω may also be defined as

$$\Omega = \sqrt{\frac{K_\theta\, m}{K_y\, I_m}} \tag{2.4.6}$$

and based on the formula for the radius of gyration of stiffness about CS

$$r_k = \sqrt{\frac{K_\theta}{K_y}} \tag{2.4.7}$$

and on the formula for the radius of gyration of mass about CM

$$r_m = \sqrt{\frac{I_m}{m}} \tag{2.4.8}$$

Ω may also be defined as a function of r_m and r_k, or as a function of ρ_m and ρ_k

$$\Omega = \frac{r_k}{r_m} = \frac{\rho_k}{\rho_m} \tag{2.4.9}$$

where ρ_m is the normalised mass radius of gyration and ρ_k is the normalised stiffness radius of gyration (both normalised to the floor plan dimension b).

2.4.3 Normalised Static Eccentricity

The application of the torsional code provisions is based on the determination of the static eccentricity, which, for multi-storey buildings, depends on the horizontal and vertical stiffness distribution, together with the horizontal and vertical distribution of the design earthquake forces. The concept of the equivalent static torsional moment is employed by all major codes to specify the design forces arising from structural asymmetry. By this approach, the dynamic torsional effects are simulated by appropriately amplifying the static eccentricity of each floor or storey level (see also Section 4.2.2), prior to calculating the design torsional moments.

Although the static eccentricity e_s is a very important parameter in the application of seismic design provisions, there is an apparent lack of consistency about its definition by codes (Tso, 1990) and by various research investigators (Cheung and Tso, 1986a, 1986b; Kan and Chopra, 1977c; Stafford, Smith and Vezina, 1985). For single-storey structures, e_s is defined as the distance between CS and the resultant of the lateral loading at the floor level which, for the equivalent static analysis, is assumed to act through CM. The reference position for the determination of e_s may also be CR, the point where the application of the lateral load will not cause any rotation, and it can also be identified as the centre of twist remaining stationary when the structure is subjected to a purely torsional loading. For this special case, the centre of rigidity and the shear centre coincide. Finally, the same reference position is the centre of element strengths in an inelastic analysis, for models with proportional stiffness and strength.

However, it is in the extension of these definitions to multi-storey buildings, that essentially three definitions of static eccentricity arise differing in terms of the reference position from which the eccentricity is measured. Wittrick and Horsington (1979) defined as reference point the **storey shear centre,** the point where the application of the storey shear produces no relative rotation between the floors immediately above and below the reference storey. Cheung and Tso (1986a, 1986b) adopted as reference point the **centre of rigidity,** the point located at any floor level such that, when the vertical distribution of lateral loads passes through them, there is no rotation of the floors about the vertical axis. Finally, Humar (1984) defined the reference point for a particular floor as the **centre of resistance,** the point through which the lateral earthquake force at that floor should pass without the floor undergoing any rotation relative to the base, but other floors may rotate.

The first two definitions have been adopted implicitly in the application of the torsional design provisions (Tso, 1990). Jiang et al. (1993) made an explicit evaluation and comparison of the three static eccentricity definitions for multi-storey buildings and found that the third approach was highly sensitive to changes in floor geometry and it was not recommended for practical design. Both the first and the second approaches appeared to be relatively insensitive to the form of eccentricity and gave similar magnitudes and distributions of static eccentricity. The second approach was more sensitive to geometric, or stiffness, irregularities and it was affected more by changes in the vertical distribution of the lateral design loading. These conclusions were in close agreement with those found by Tso (1990).

The advantages of the first approach are that, firstly, the calculation of storey eccentricities and of the design torsional moments is the most straightforward of the three methods. Secondly, this method is the least sensitive to changes in lateral load distribution and structural asymmetry. Thirdly, the positions of shear centres are independent of the lateral load distribution. Fourthly, the storey eccentricities can be regarded as a measure of the relative rotation between floors reflecting directly the influence of asymmetry on the load demand for resisting elements. This latter quantity is of direct concern to structural engineers for the purpose of the seismic design.

2.4.4 Accidental Eccentricity

The inclusion, or otherwise, of the accidental eccentricity provisions of the seismic codes when determining the design strengths of the lateral resisting elements is a difficult topic. The problem lies in the fact that the accidental eccentricity e_a is included to account mainly for possible rotational ground motions and unavoidable differences between designed and as built distributions of stiffness and mass. The accidental eccentricity does not account for the effect of the static eccentricity e_s, the parameter explicitly considered in the structural analysis. In linear analysis, these two eccentricities are separable and the results of the dynamic analysis can be easily compared with those of the equivalent static analysis, with or without e_a. In inelastic analysis, the inclusion of e_a results in different strength distributions and increased total strength, and hence, the choice is critical to the results obtained.

Since the effect of the accidental eccentricity on strength is intended to account for factors that cannot be included in the analysis, some investigators do not include the accidental eccentricity for the strength calculation of the structural elements (Chandler and Duan, 1991b). The argument against this approach is that the model studied is not the one that is actually designed. In other words, the exclusion of these effects from the parametric studies is a deficiency of the techniques presently used by investigators. There are also some practical problems when applying the UBC torsional provisions (Correnza et al., 1995) since their approach to redress the effects of large torsional flexibility is formulated through the inclusion of an increased accidental eccentricity and, therefore, it is not appropriate to totally exclude it.

2.4.5 Overstrength Ratio

In the context of torsional studies, overstrength arises when the total lateral design strength of a code-designed TU system exceeds the nominal lateral strength of its reference system. The overstrength ratio O_s is a result of assessing different values of design eccentricity and it is a characteristic of code-designed structures. Some investigators (Rutenberg et al., 1992a) used normalised strength, that is the total strength of the TU system normalised to that of the TB system in order to separate the effect of the increased total strength from the effect of strength distribution amongst the elements. Thus, the **normalised strength** is more appropriate for optimising strength distribution among the elements, whereas the use of the **as-computed** (non-normalised) **strength** is suitable for comparing the response of asymmetric systems with their reference models, in order to assess the relative protection they offer.

2.4.6 Earthquake Ground Motion

The results of inelastic torsional analyses are highly affected by the frequency content of a record, the pattern of pulses, the number of records used, and the normalised intensity of the records. All these factors have been a major source of variance among published results. The way in which several time-histories are used for this purpose varies among investigators. One way is to subject both symmetric and asymmetric systems to a number of records, to compute their responses for each record, and then to evaluate the mean response values.

The alternative approach is to select a design spectrum-compatible ensemble of records. In this case, the spectrum of the records selected is still spiky whereas the design spectrum is smooth. Therefore, for any given period, the design spectral acceleration may either be larger or smaller than the dynamic spectra value. Since different investigators use small and different ensembles, discrepancies among results occur. The selection of the earthquake records employed in this study, their characteristics, and normalisation procedure followed is described in Section 4.4.

2.5 CONCLUSIONS

The seismic vulnerability of asymmetric plan-eccentric structures has been repeatedly demonstrated in strong earthquakes and a considerable amount of research effort has been applied to this problem over the past 20 years, both in linear and non-linear inelastic ranges of response. However, there is no universal agreement among the researchers as to satisfactory criteria for deriving a solution to the design problem posed by torsion due to the complex effects of parameters governing the post-yield seismic response of eccentric structures.

The influence of these parameters on the deformation and ductility demand of the critical elements has still to be fully defined, and clear relationships have not been provided between the inelastic torsional response and the structural parameters of an eccentric system. Differing definitions regarding the stiffness distribution, the accidental eccentricity, the reference models, the transverse elements, the uncoupled torsional to lateral frequency ratio, the overstrength due to code torsional provisions and the earthquake ground motions have resulted in lack of agreement between the conclusions of various research studies. Therefore, the key aspects of structural modelling that require further investigation are:

1. The inclusion of transverse structural elements oriented perpendicular in the uni-directional systems.
2. The inclusion of codified torsional provisions into the strength design of structural models considered for the inelastic analysis of torsional effects.
3. The development of dynamic eccentricity coefficients for design that are more effective and economical.
4. The investigation of multi-storey regularly asymmetric building models designed according to different seismic code procedures.
5. The investigation of different types of multi-storey asymmetric building models (frame and frame-wall structures).
6. The effect of design code provisions, i.e. minimum reinforcement requirements, capacity designs, drift and damage ratios etc.

CHAPTER 3

STRUCTURAL PARAMETRIC MODELLING

3.1 INTRODUCTION

In Chapter 2, the review of past research studies examining the torsional effects of asymmetric single-storey and multi-storey buildings indicated that the inelastic dynamic response of the structures is highly complex and dependent on numerous fundamental structural parameters and on their definitions. In the majority of the studies investigating the inelastic torsional response of structures subjected to severe earthquake ground motions, many critical structural parameters have been defined. A fundamental cause for the several conflicting results and conclusions drawn by different researchers is the variety of definitions employed for the structural parameters and the configuration of the structural models investigated.

This chapter concerns the structural parametric modelling assumptions of the thesis and introduces the definitions of the basic structural parameters influencing the inelastic torsional response of the structures. The key issues presented have been identified as areas of concern, where the use of different definitions, or the making of divergent assumptions, has resulted in basic lack of agreement between the results of various research studies. Details of different approaches employed in these areas are described, and the two types of reference models widely employed in the past are explained. The configuration of the typical 3-element single-storey model is presented and the necessity for the investigation of multi-storey building models is justified and discussed. Finally, the configuration of the structural models employed in this study is presented.

3.2 SINGLE-STOREY ASYMMETRIC MODELS

3.2.1 The 3-Element Single-Storey Model

A structural model employed for the investigation of torsion should incorporate key properties and dynamic characteristics of real structures and it should be able to provide valuable information on torsional effects. The most commonly adopted structural model for torsional studies has been the single-storey 3-element model with a perfectly rigid floor diaphragm supported by moment-resisting frames, or other forms of lateral load-resisting elements. As also indicated from the literature review presented in Chapter 2 (Section 2.2.2), the number of elements selected to support the floor diaphragm influences the inelastic response of the structure.

Duan (1991) concluded that a two-element model underestimates the inelastic torsional response compared to models with more elements. However, employing more than three elements in the direction of the earthquake ground motion results in a model with a more complex system definition, and, for this reason, the 3-element model was considered adequate. As a result, the 3-element model consists of three elements oriented parallel to the earthquake direction, herein defined as the lateral y-direction (Figure 3.2.1), and it has been considered simple but sufficient to predict the inelastic response of TU structures. Apart from the three load-resisting elements located in the direction of excitation, sometimes two or three perpendicular elements are also included and these are called transverse elements (Figure 3.2.2).

3.2.2 Fundamental Torsional Definitions

For single-storey models, the **centre of rigidity or resistance CR** coincides with the **centre of stiffness CS** and it is determined as the point where the sum of the first moment of stiffnesses of all resisting elements is zero. Therefore, the location of CR is calculated as

$$x_{CR} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \tag{3.2.1}$$

$$y_{CR} = \frac{\sum k_{ix} y_i}{\sum k_{ix}} \tag{3.2.2}$$

where x_{CR} and y_{CR} are the Cartesian co-ordinates of CR, k_{ix} and k_{iy} are the stiffnesses of the elements oriented in the x- and y-direction, respectively, and x_i and y_i are the normal distances from CR to the elements under consideration. The **stiffness** or **static eccentricity** e_s is defined as the distance between CR and CM while the **geometric eccentricity** e_g is defined as the distance between CR and the **geometric centre GC** (Figure 3.2.1). The static eccentricity inducing torsional effects is attributable to changes in the mass or stiffness distributions relative to the reference model.

In torsional response studies, the element stiffness distribution is asymmetric resulting in CR offset from GC while the mass is distributed uniformly over the floor diaphragm (CM located at GC and offset from CR by the static eccentricity e_s). Thus, a model is termed **torsionally unbalanced TU** when both e_s and e_g are larger than zero, **torsionally balanced TB** when e_s is zero and e_g is larger than zero and **symmetric SM** when both e_s and e_g are zero (Figure 3.2.1). When the mass distribution in TU models is uniform, the system is called **stiffness-eccentric** or **stiffness-asymmetric** and it is representative of many buildings exhibiting torsional response. In other cases, the mass distribution is non-uniform, the models are called **mass-eccentric,** and the static eccentricity may be a combination of stiffness and mass asymmetry (Figure 3.2.1). These two definitions of TU systems produce significantly different inelastic response (Tso and Ying, 1992) and both situations have been examined in this study.

3.3 REFERENCE MODELS

The choice of the reference system depends on the purpose for comparing the response of asymmetric systems with the reference model. One approach treats the reference model mainly as a normalising factor in order to permit simple comparison among different designs of asymmetric systems. Such a reference system is usually taken as a SDOF oscillator having the same natural period and total yield strength equal to the code static base shear. In the alternative approach, the reference system is either symmetric SM or torsionally balanced TB (Section 2.2.3) and it is designed based on the static lateral force provisions of the codes. There is no effective difference between SM and TB models provided the accidental eccentricity e_a

(Section 2.4.4) is not considered. When e_a is taken into account (see also Section 5.4.3), the TB model becomes a TU system in the post-elastic range.

In view of this lack of uniformity in the reference models employed, it may be difficult to compare the relative performance of different asymmetric models. On the other hand, it is possible to design a given asymmetric system to perform no worse than its closest reference system, and hence, permit an evaluation of individual code torsional provisions. Some researchers believe that including e_a results in TB systems at service load, which become TU systems in a strong earthquake and, therefore, it should not be encouraged (Correnza et al., 1992a, 1992b). If this view is accepted, then the use of such systems as reference models is not warranted. The inelastic seismic response of the reference models is analytically investigated in Chapter 5.

3.3.1 The Symmetric Reference Model

A model configuration is termed a symmetric (SM) reference model when both stiffness and mass are distributed evenly about GC and the position of CM and CR are identical and coincident with GC (Figure 3.2.1). The resisting elements are located symmetrically from GC and their position may be variable in order to vary other structural parameters. This model is the simplest example of a system exhibiting no torsional response in both the elastic and inelastic ranges and its response may be compared with the response of systems exhibiting torsion.

There are three important drawbacks in using the SM model as a reference case. Firstly, in studies where the element spacing is assumed to be variable (Goel and Chopra, 1990a, 1990b; Chandler and Duan, 1990, 1991a, 1991b), the lever arm which is a measure of the individual contribution of a given element to the structure's torsional stiffness, is variable. Therefore, whilst varying the element spacing to achieve specific values of structural parameters, such as Ω, the influence of torsion on element deformation cannot be determined consistently. Secondly, varying the element spacing is unrealistic since, in real buildings, the element locations are invariably fixed, and the outer frame elements are usually situated at the periphery of the floor diaphragm. Thirdly, in order to introduce eccentricity to the TU model, the stiffness distribution of the elements has to vary from the reference model, and the comparison of the two systems is difficult.

3.3.2 The Torsionally Balanced Reference Model

A system overcoming the drawbacks of the SM model has been developed (Tso and Zhu, 1992a, 1992b; Zhu and Tso, 1992) and it is called torsionally balanced (TB) reference system (Figure 3.2.1). In real structures, the static eccentricity leading to torsional effects frequently arises from uneven distribution of stiffness and, hence, it is important to develop a reference model that maintains the same stiffness distribution. Furthermore, the TB model should respond purely in translation in both the elastic and inelastic ranges and, therefore, it must satisfy certain conditions. The static eccentricity of the TB system should be zero and the mass should be distributed such that CM is coincident with CR while the structural eccentricity is defined by the stiffness distribution. In the inelastic range, no torsion is introduced when the yield strength of the elements is proportional to the distribution of stiffness.

In contrast with the SM reference model, the TB reference model has the outer elements fixed at the periphery of the floor diaphragm and, as a consequence, the frequency ratio Ω depends only on geometric eccentricity and on the structure's plan aspect ratio. In order to maintain a constant Ω value for a range of static eccentricities, the strength distribution between the elements must be varied. However, comparing systems with different eccentricities may be unreliable due to the variable element stiffness distributions resulting from this approach.

In this study, both SM and TB reference models are employed. The SM models are adopted as reference models for mass-eccentric structures while the TB models are employed for stiffness-eccentric structures. Thus, a consistency is always maintained between the strength distribution of each TU model examined and its reference model since there are no differences in their stiffness distribution (refer also to Section 5.2).

3.4 MULTI-STOREY ASYMMETRIC MODELS

3.4.1 The Necessity for Multi-Storey Models

As seen from the literature review presented in Chapter 2, most previous studies on the elastic and inelastic earthquake response of structures were based on single-storey asymmetric models. A single-storey asymmetric model was proved to be sufficient to investigate the effects of torsional coupling of multi-storey regularly asymmetric buildings, in which the centres of mass and the centres of rigidity lie on two vertical lines (Section 2.2.1). In the elastic range, the maximum response of a regularly asymmetric multi-storey model can be determined by combining the response of the corresponding torsionally uncoupled multi-storey model and the response of an associated torsionally coupled single-storey model (Kan and Chopra, 1977b; Hejal and Chopra, 1989a, 1989b). Furthermore, in regularly asymmetric buildings, the stiffness eccentricity is the same at all floor levels and the torsional to translational frequency ratios associated with all modal pairs are identical (Hejal and Chopra, 1989a, 1989b).

Nonetheless, the above procedure cannot be applied to the inelastic earthquake response of regularly asymmetric multi-storey buildings since the structure's response is non-linear and inelastic and its vibration mode shapes change with time. Hence, the principle of modal superposition, upon which the above procedure is based, is only valid for a very short period of time in which the structure's dynamic properties can be assumed constant, but it is not valid to predict the structure's inelastic response in the entire response history. A single-storey asymmetric building model is sufficient for studying the inelastic earthquake response of regularly asymmetric multi-storey buildings only if its response is dominated by the first modal pair of vibration, although periods and mode shapes of the first modal pair change with time. However, in a multi-storey building, the contribution of higher modes to the building's response increases because of the period elongation caused by the element yielding.

In explaining the numerous upper storey collapses during the 1985 Mexico City earthquake, Villaverde (1991) found that due to the development of plastic hinges at the bottom of the first-storey columns and at most of the beam ends, the natural periods of typical structures elongate. Consequently, during some time intervals, the structure's instantaneous third mode period is in resonance with the dominant period of the earthquake ground motion, and the structure's vibration is predominated by this mode. A failure mechanism involving the collapse of upper storeys is generated due to storey shears that have exceeded the design values. Therefore, to adequately account for the contribution of higher modes, a multi-storey model is needed to study the inelastic seismic response and the effective design of regularly asymmetric buildings.

Duan and Chandler (1993) employed a multi-storey model to re-evaluate the conclusions of earlier studies based on single-storey models and concluded that, unlike the elastic studies, a single-storey model is not on its own sufficient to investigate the inelastic torsional effects of multi-storey regularly asymmetric buildings (Section 2.3.2). Nevertheless, in the TU models employed by Duan and Chandler (1993), the flexural stiffness of the beams was very high relative to the flexural stiffness of the columns. In that way, each frame was considered as a "shear beam" for computational purposes, but resulted in unrealistic beam dimensions and models. Furthermore, they assumed that the moment-curvature relationship of all structural elements was bilinear hysteretic, without any stiffness degradation. The interaction between bending moment and axial force in columns was neglected and the configuration of the models was rather simple including only three planar frames parallel to the seismic direction. Similar assumptions and approximations in existent inelastic multi-storey studies resulted in simple forms of structural representations and made very crucial the need to further examine the response of multi-storey asymmetric buildings.

3.4.2 Idealised Multi-storey Regularly Asymmetric Models

The idealised multi-storey regularly asymmetric models employed in this study are assumed to have the following properties:

1. The models are multi-storey and mono-symmetric. The distribution of their mass, stiffness and strength is always symmetric about the x-axis, but it may be asymmetric about the y-axis.

2. The floor levels are rectangular, rigid in their own plane, with a typical plan aspect ratio equal to 2/5.
3. The columns are assumed to be massless and axially inextensible.
4. The geometric centres, the centres of mass and the centres of stiffness of the floors lie on three vertical lines, which means that all floor levels have the same mass and stiffness distributions.
5. The torsional inertia of the individual load-resisting elements and the translational stiffness perpendicular to their own principal plane are negligible and, consequently, they have been ignored in the analyses.
6. No soil-structure interaction is considered and, therefore, the models are displaced with rigid base translation only.
7. Damping is assumed to be viscous, and the damping ratio ξ is constant throughout the dynamic seismic loading and unloading of the structure.
8. Yielding of beams is defined in terms of the yielding moments in pure bending at the ends of beams while axial force-bending moment interaction diagrams are included for the columns.
9. Considering members with rigid ends simulates the behaviour of beam-column joints.

3.4.3 Structural Configuration of the Models Employed

The multi-storey asymmetric frame building models considered in this study consist of 6, 12 and 24-storey RC three-dimensional frames with 2 and 5 bays of 6.00 m span in the y-direction and in the x-direction, respectively. The height of all storeys has been considered constant and equal to 3.2 m and, at any floor level, the lateral stiffness matrices of the frames are proportional to each other by the same amount (termed as "proportional framing"). As a result, the vertical line passing through the centres of rigidity is separated from the vertical line passing through the centres of mass by a distance equal to the stiffness eccentricity e_s. CM vertical line is located either at the same position with GC vertical line, or eccentrically from it, and the position of CR vertical line varies depending on the stiffness eccentricity of the model. Consequently, the dimensions of the column elements and beam elements depend on the stiffness eccentricity that has to be obtained in each model.

In this study, the response of asymmetric plan configurations is investigated by varying the mass and/or stiffness distribution throughout the floor levels of different models. Although the static eccentricity in real structures is usually produced from stiffness eccentricity and CM is located approximately at GC, the first models examined are mass-eccentric (Figure 3.4.1) with a static eccentricity of 0.15*b* (model S15) and 0.30*b* (model S30). The symmetric model S is used as a reference model of the mass-eccentric models S15 and S30 and it has the same stiffness distribution with these TU models (model type S). The understanding of the inelastic seismic response of mass-eccentric models is the basis for the understanding of the response of more complicated stiffness-eccentric models, which may also have a mass eccentricity. Moreover, another objective for examining mass-eccentric models is that their response can be compared with similar mass-eccentric multi-storey models examined in previous inelastic studies (De Stefano et al., 1995; Dolce and Ludovici, 1992).

The second category of TU models investigated includes stiffness-eccentric models with or without mass eccentricity (Figures 3.4.2 and 3.4.3) and their reference models are the TB models A and B. Due to the fact that CR is shifted from GC, the stiffness distribution of the frames depends on the amount of static eccentricity that has to be achieved in each model. Figure 3.4.2 presents the set of models having CR shifted from GC by a distance equal to 0.15*b* (model type A) while, in Figure 3.4.3, the models have CR shifted from GC by a distance equal to 0.30*b* (model type B). In the stiffness-eccentric model A15, CM is coincident with GC and the geometric eccentricity e_g is zero. Model A30 is stiffness/mass-eccentric with a static eccentricity equal to 0.30*b* and both CM and CR are eccentrically located (shifted at a distance 0.15*b* from GC in opposite directions). The second set of TU stiffness-eccentric models (model type B) includes model B30 with CM coincident to GC and e_s equal to 0.30*b* while models B15 and B45 are stiffness/mass-eccentric with e_s equal to 0.15*b* and 0.45*b*, respectively, and CM eccentrically located from GC.

From the presentation of the reference and TU models adopted in this study, it can be observed that the notation of each model depends on its structural characteristics. The letter used for each model indicates its stiffness distribution and, consequently, the location of CR (model type S, A and B) while the two numbers that follow indicate the amount of static eccentricity. For each model configuration, models with different heights are investigated (6, 12 or 24 storeys) and the number of floor levels is also indicated in the notation of the model. For example, model 12B45 is the model that consists of 12 floor levels, its static eccentricity is equal to 0.45b, and CR is found at a distance equal to 0.30*b* (model type B).

For models without transverse elements, there are no lateral resisting elements in the x-direction and the earthquake ground motion is applied only in the y-direction. A model having resisting elements along only one direction and subjected to uni-directional ground motion leads to results generally in good agreement with those obtained by considering resisting elements in both directions and using bi-directional excitation (Correnza at al., 1994). The effect of the bi-directional loading and of the inclusion of transverse structural elements is analytically examined in Chapter 8.

3.4.4 Factors Influencing the Configuration of the Models

As explained in the previous section, there are three types of reference models (termed S, A and B) and each one corresponds to different TU systems. Although each reference model consists of different frames and structural elements, they are all consistent by having the same lateral and torsional stiffness. Consequently, all models with the same number of storeys also have the same lateral and torsional period and result in equal values of Ω selected to be equal to unity, which is the threshold that defines torsionally flexible and torsionally stiff models.

The selection of the structural dimensions for the six frames of each model type depends on many factors. The most important factor is that all models with identical height should be consistent by having the same lateral and torsional stiffness, and the same Ω value equal to unity. This requirement enables the comparison of the inelastic response of models with different structural configurations and static eccentricities. In each model type, all the internal frames (frames 2 – 5) are identical while the two external frames (frame 1 and 6) differ. In the stiffness-eccentric model types A and B, frame 1 is the stiff frame while frame 6 is the flexible one, and they are located at the stiff and at the flexible side of the structure, respectively. In that way, the 6-element models investigated are consistent with the simplified 3-element model, which consists of one internal frame located at GC and two external frames located at the stiff and flexible sides of the model. In the mass-eccentric model type S, the two external frames are identical since CR should be located at GC.

The minimum cross-sectional requirements implied by the seismic codes is also another important factor that influences the selection of the structural dimensions. The minimum column dimension is 30 cm and the minimum beam dimension is 20 cm for highly ductile structures (see Sections 4.3.2 and 4.3.3). The flexible frame 6 of model type A is the frame with the lowest stiffness required when compared with the frames of the other model types. Thus, after applying the minimum possible element dimensions in frame 6 of model A and aiming to have the same total lateral and torsional stiffness in all models, the dimensions of the structural elements are defined.

When the structural element dimensions are determined, the equivalent static forces are calculated and they are applied to each model. An elastic static analysis is performed and it is checked whether the selected structural dimensions satisfy the interstorey drift limitations imposed by the seismic codes (refer to Sections A.4.1, B.10.2 and C.3.7). The amount of reinforcement needed for every section of the structural elements is calculated and it is confirmed whether the reinforcement ratios required are lower than the maximum reinforcement ratios permitted by the codes. Therefore, it is verified whether the dimensions of the structural elements selected are large enough for the amount of reinforcement needed (Sections 4.3.2 and 4.3.3).

Another major consideration when dimensioning the column cross-sections is the limitation of the normalised axial force $v_d=N/(A_c f_{cd})$ imposed by the seismic codes to ensure adequate ductility, where N is the axial load, A_c is the cross-sectional area of the concrete section, and f_{cd} is the design value of the concrete cylinder compressive strength (see also Section 4.3.3). In the EC8 code, this limitation is stricter than the relevant limitations in the NZS and UBC codes and it depends on the ductility category of the structure under consideration (v_d=0.55 for high ductility structures). High axial loading leads to excessive demands regarding confinement reinforcement and this is one more reason for selecting appropriately large cross-sectional dimensions for the columns (Penelis and Kappos, 1997). Thus, the selection of the structural elements is an iterative procedure taking into account various factors.

3.4.5 Dimensions of the Structural Elements

The dimensions of the columns and beams are selected based on the factors presented in the previous section (Section 3.4.4) and they are given in centimetres (cm) in Tables 3.4.1 – 3.4.4. The beam dimensions are uniform over the height while the column elements are tapered over the height of the structure and they have a different column section every 6 floors. The column dimensions are selected in such way that the centres of rigidity of all floor levels are located on the same vertical line.

Frame 6 of model type A is the frame with the smallest stiffness and the minimum column and beam dimensions imposed by the seismic codes are applied at that frame while the stiffest frame is frame 1 in model type B. Therefore, the largest column cross-sections are found at the lower storeys of the 24-storey model type B due to the normalised axial force limitation, which increases the column dimensions depending on the maximum axial load applied.

The selected dimensions of the structural elements satisfy the maximum reinforcement limit permitted by the building codes. The models are tested for the most unfavourable loading condition and they are designed for the seismic code producing the largest equivalent static forces. The most conservative seismic code is EC8 code and its base shear calculation is always the largest when compared with the base shear calculation of the NZS and UBC codes. The beam and column dimensions of the models are tested for all the eccentricity cases and when both code-specified design eccentricities are included in the static analysis. Therefore, the selected structural dimensions are tested for the most unfavourable design cases.

It should be mentioned at this point that in reality the dimensions used for the structural elements are usually in multiples of 5cm. In this study, in order to achieve the static eccentricities required, the dimensions of the structural elements chosen are in multiples of 1 cm.

BEAMS OF ALL MODELS			
Model Type	Frame (1)	Frames (2-5)	Frame (6)
S	30.9 x 61.8	35.3 x 70.5	30.9 x 61.8
A	40.0 x 80.0	33.9 x 67.8	20.0 x 40.0
B	47.6 x 95.1	28.3 x 56.6	23.8 x 47.6

Table 3.4.1 Dimensions of the beam elements of all the models (cm).

COLUMNS OF THE 6-STOREY MODELS			
Model Type	Frame (1)	Frames (2-5)	Frame (6)
S	46.4	52.9	46.4
A	60.0	50.8	30.0
B	71.4	42.4	35.7

Table 3.4.2 Dimensions of the column elements of the 6-storey models (cm).

COLUMNS OF THE 12-STOREY MODELS						
Model Type	Frame (1)		Frames (2-5)		Frame (6)	
	Floor Levels		Floor Levels		Floor Levels	
	1-6	7-12	1-6	7-12	1-6	7-12
S	57.2	46.4	65.2	52.9	57.2	46.4
A	74.0	60.0	62.7	50.8	37.0	30.0
B	88.0	71.4	52.3	42.4	44.0	35.7

Table 3.4.3 Dimensions of the column elements of the 12-storey models (cm).

COLUMNS OF THE 24-STOREY MODELS												
	Frame (1)				Frames (2-5)				Frame (6)			
	Floor Levels				Floor Levels				Floor Levels			
	1-6	7-12	13-18	19-24	1-6	7-12	13-18	19-24	1-6	7-12	13-18	19-24
S	78.8	68.0	57.2	46.4	89.9	77.6	65.2	52.9	78.8	68.0	57.2	46.4
A	102.0	88.0	74.0	60.0	86.4	74.6	62.7	50.8	51.0	44.0	37.0	30.0
B	121.3	104.7	88.0	71.4	72.1	62.2	52.3	42.4	60.7	52.3	44.0	35.7

Table 3.4.4 Dimensions of the column elements of the 24-storey models (cm).

3.4.6 Models with Transverse Elements

An extension of the model with only lateral load-resisting elements is to include elements spanning in the transverse direction as well, as in prototype structures. Systems with transverse structural elements but without introducing the corresponding earthquake component have been utilised by Goel and Chopra (1994) and this has been the source of some controversy (Correnza et al., 1994). However, De La Llera and Chopra (1994a) suggested that the transverse elements should indeed be excluded to account for the absence in analysis of the ground motion component in the x-direction.

However, in some circumstances, the presence of transverse elements was found not to be very significant (De Stefano et al., 1993a) provided that the linear properties of the system were kept intact. Nevertheless, the transverse elements make some difference since they affect the torsional strength, a parameter that is not considered by most investigators. Correnza et al. (1994) indicated that for short to medium period structures, it is essential to include the transverse earthquake excitation for the accurate comparison of bi-directional systems to uni-directional.

However, it is still widely considered that the results of studies employing structural models having structural elements in only one direction and subjected to uni-directional ground motion excitations are sufficiently accurate for continued studies of inelastic torsional effects. This accords with the intentions of building codes, such as EC8, which for regularly asymmetric systems permit planar models to be employed in designing for seismic load effects in the two principal directions. The inclusion of such transverse elements requires additional modelling assumptions to be made in the structural definition, such as the distribution of the total transverse stiffness and strength, the location of the transverse elements and the inclusion of the transverse earthquake component. The structural configuration and the inelastic response of TU models including transverse elements has been analytically investigated in Chapter 8.

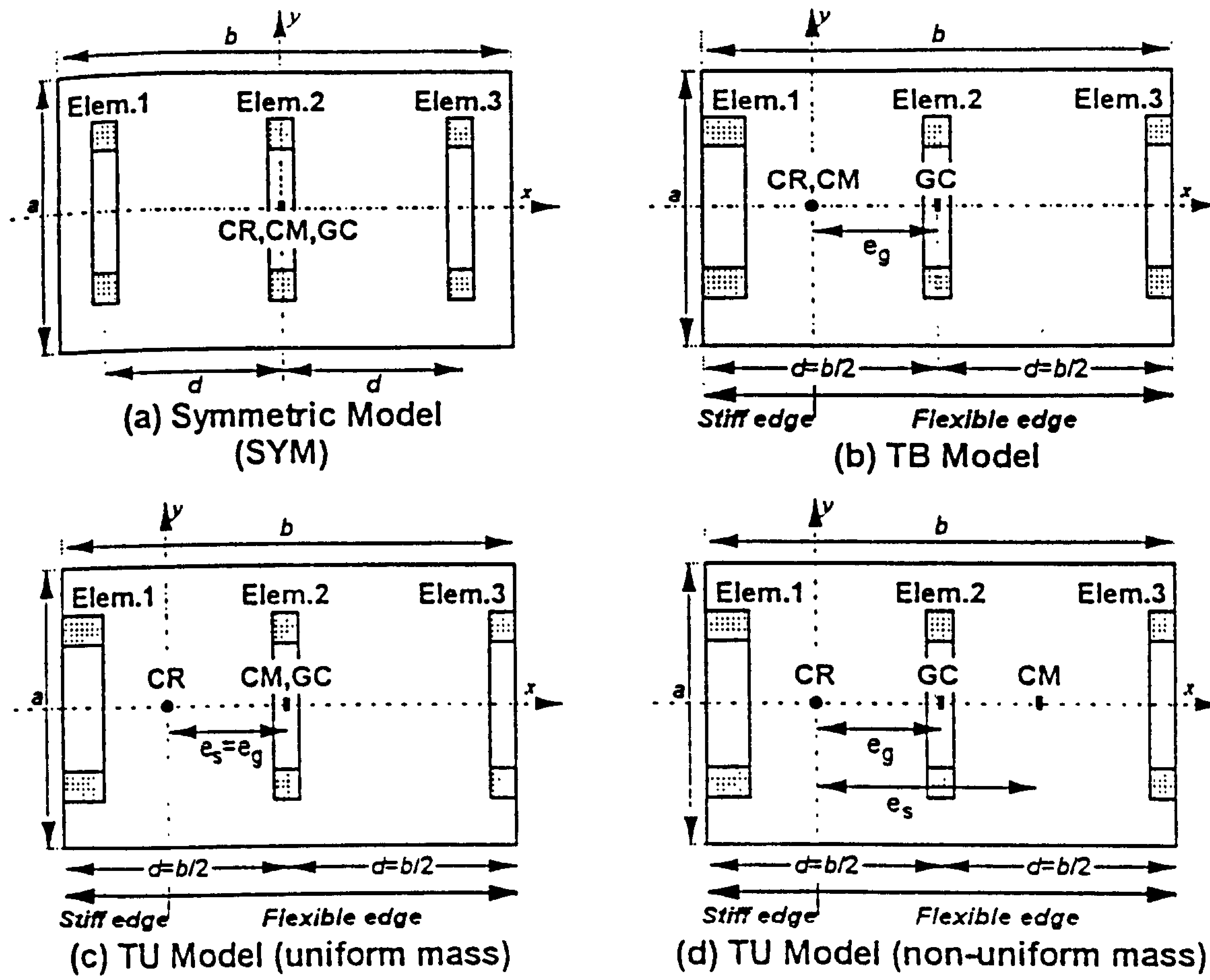


Figure 3.2.1 Structural plan configurations of typical 3-element models (Correnza, 1994).

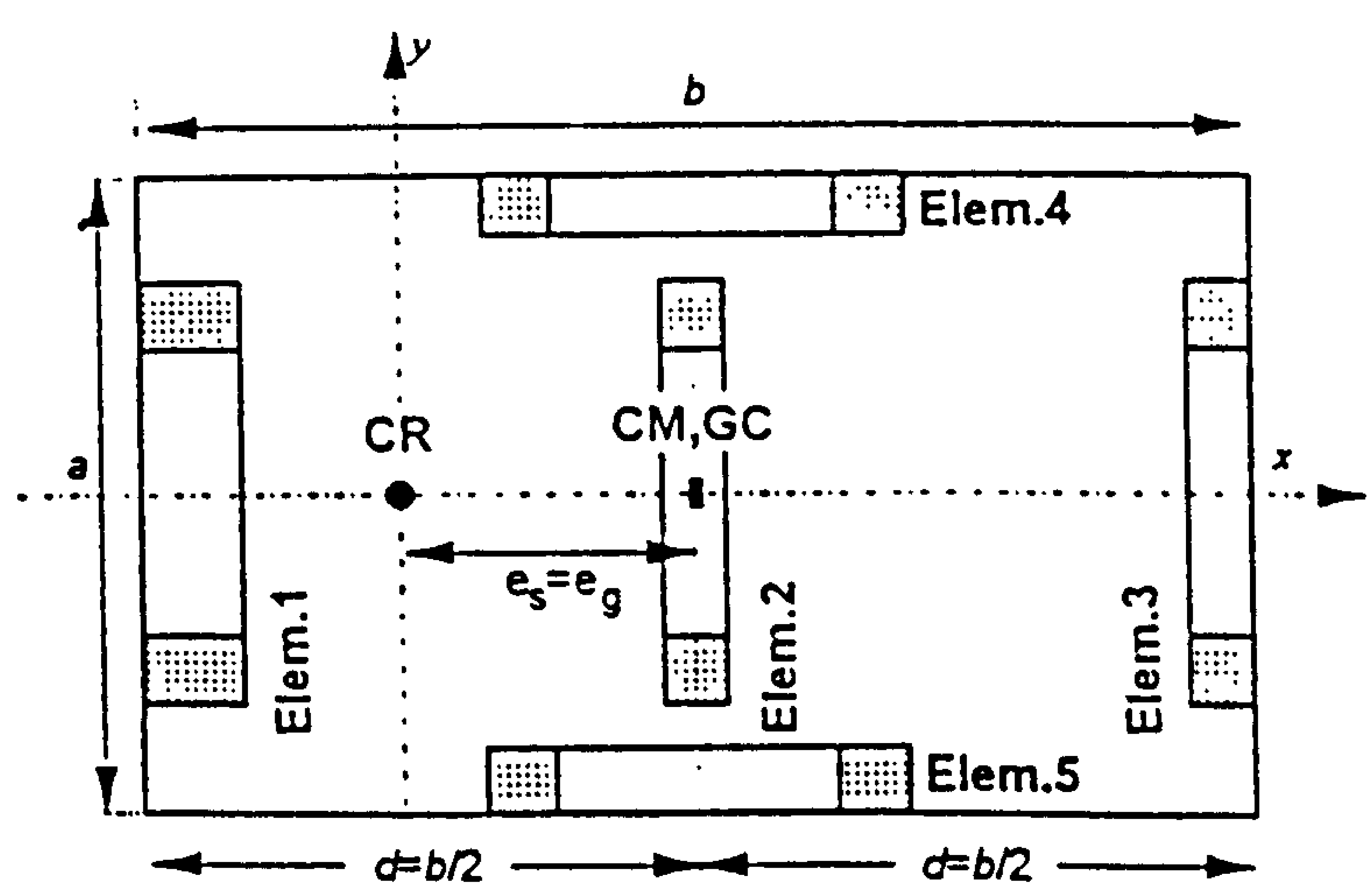


Figure 3.2.2 Structural plan configuration of a typical 3-element model with transverse elements (Correnza, 1994).

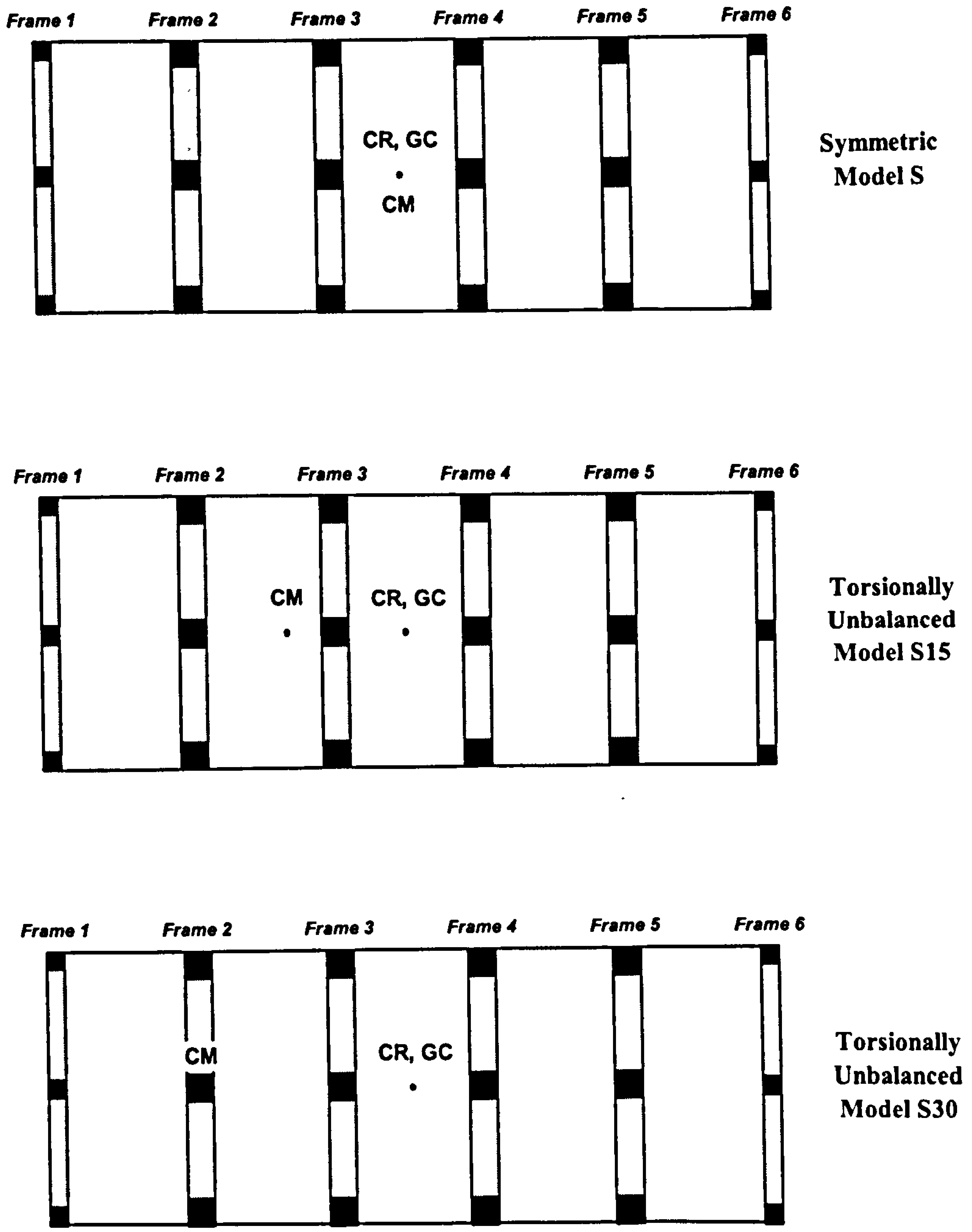


Figure 3.4.1 Plan configuration of the SM reference model S and of the TU mass-eccentric models S15 and S30.

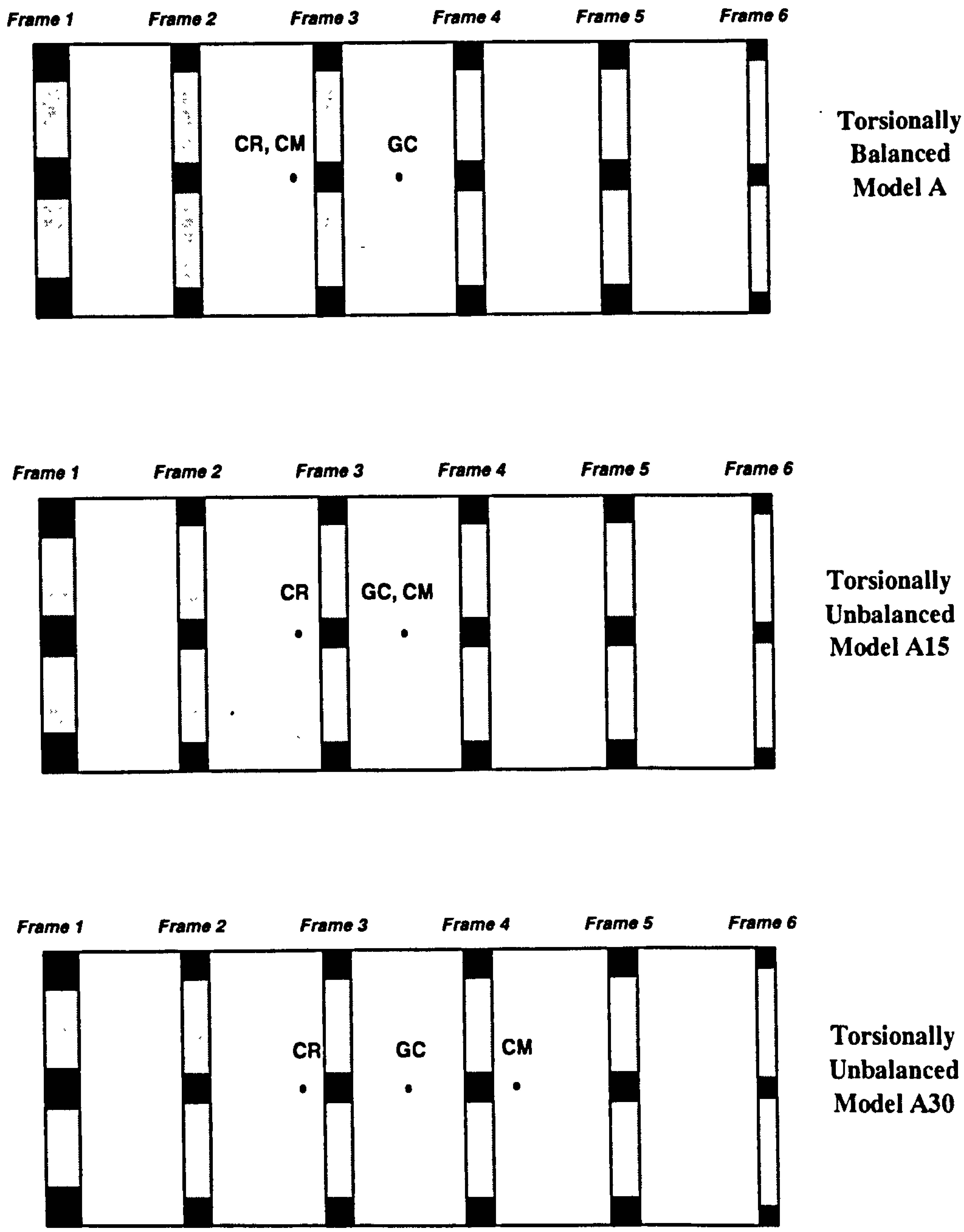


Figure 3.4.2 Plan configuration of the TB reference model A and of the TU models A15 (stiffness-eccentric) and A30 (stiffness/mass-eccentric).

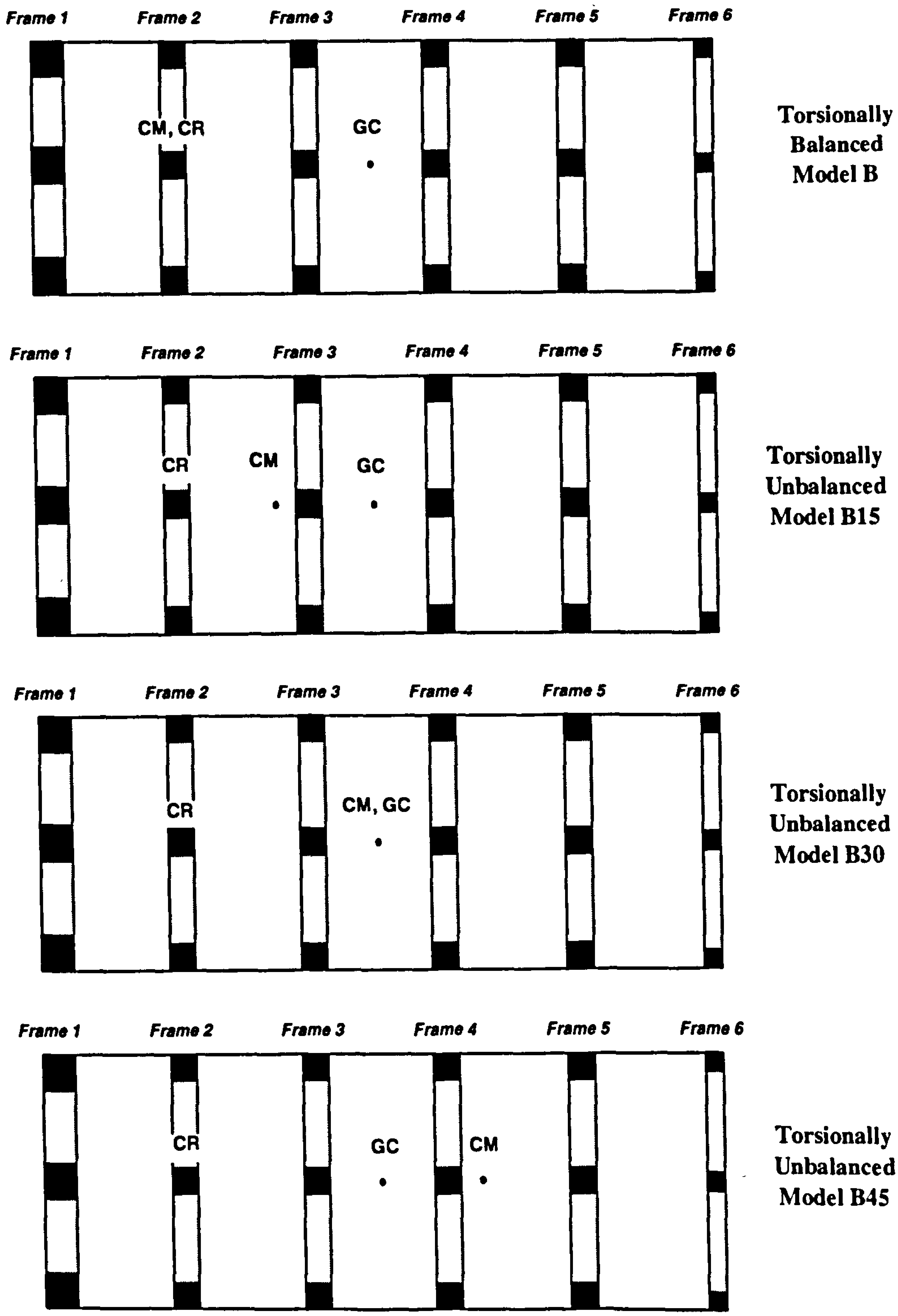


Figure 3.4.3 Plan configuration of the TB reference model B and of the TU models B15 (stiffness/mass-eccentric), B30 (stiffness-eccentric) and B45 (stiffness/mass-eccentric).

CHAPTER 4

DESIGN ANALYSIS PROCEDURE

4.1 INTRODUCTION

This Chapter is concerned with the dynamic parametric modelling decisions and assumptions of the thesis, and it presents the design analysis procedure followed for the models investigated. Information regarding the elastic analysis procedure and the static torsional provisions of the seismic codes is given while details about the element's structural design are included. Moreover, information about the non-linear dynamic analysis is presented concerning the selection of the earthquake records employed, the computer program specifications, and the response parameters adopted for the presentation of the inelastic torsional response of the models. The procedure to determine the structural model configuration and to carry out the analysis and design of the models could be summarised in five basic steps (Figure 4.1.1):

1. The structural configuration and geometry of the models is defined and information is given regarding the structural element dimensions (Sections 3.4.3 and 3.4.5).
2. The linear elastic analyses of the models are performed with the inclusion of the seismic code torsional provisions. The maximum design stresses of the structural elements are evaluated (Section 4.2) and they are adopted for their design.
3. The amount of reinforcement of the structural elements is calculated based on the results of the elastic analyses and on the code design provisions (Section 4.3). The design moments calculated by the elastic analysis are utilised as input data for the

inelastic analyses of the models carried out in the following step and the ground motion accelerograms needed for the inelastic time-history analyses are also chosen (Section 4.4).

4. The response of the structures is evaluated by subjecting them to different selected earthquake accelerograms through non-linear step-by-step analyses (Section 4.5).
5. The results of the non-linear analyses are processed to evaluate local and global damage indices (Sections 4.6 and 4.7).

4.2 ELASTIC ANALYSIS AND TORSIONAL PROVISIONS

4.2.1 Code Approach for Seismic Torsional Design

The three codes employed in this study comprise the New Zealand Standard code, 1992 edition (NZS), the European Committee for Standardisation, Eurocode 8, 1993 edition (EC8) and the United States Uniform Building Code, 1994 edition (UBC). The design torsional provisions and the overall seismic design methodologies of these codes are presented in detail in the relevant Appendices of the thesis while, in the following sections, only their torsional design provisions are described and discussed.

Torsional coupling in asymmetric buildings results in additional torque applied simultaneously with the seismic lateral load at each floor level leading to increased deformation and strength demand in certain resisting elements. The value of the additional torque can be roughly estimated by simply multiplying the seismic lateral load by the static eccentricity e_s. Unlike the case of single-storey buildings, the seismic design and analysis of multi-storey structures involves consideration of the vertical distribution of the earthquake lateral load in addition to its horizontal distribution due to torsional coupling. To account for the torsional effects arising in asymmetric structures, most of the current seismic codes include torsional design provisions and provide two alternative methods to be employed in design practice: the equivalent static force analysis and the linear elastic modal analysis.

4.2.2 Equivalent Static Force Analysis

The equivalent static force procedure is applicable to regular buildings in which the centres of floor mass and the centres of rigidity at floor levels lie approximately on two vertical lines, and the distributions of mass and stiffness are nearly uniform over the height. Each seismic code has specific criteria that have to be satisfied (Sections A.1, B.1, C.1) in order to use this method, which is simple to apply and requires little computational effort. The fundamental lateral period T_y of the building is first estimated using the empirical formulae specified in codes (Equations (A.5.4), (B.4.1), (B.4.2), (C.3.3), (C.3.4)) and the design base shear is then calculated from the appropriate design spectrum given in each code.

The basic assumption made for the static equivalent force procedure is that the building's earthquake response is dominated by its fundamental vibration mode. Additionally, in order to calculate the base shear, the building's total mass is used, instead of the effective mass corresponding to the first mode, which is approximately 60-80% of the total mass (Clough and Penzien, 1993). Therefore, the static procedure generally results in a conservative estimate of the base shear when compared with modal analysis due to these assumptions. Furthermore, most building codes assume that the fundamental mode shape of the structure is a straight line, based on the observation that the fundamental mode shape is generally close to a straight line for a large number of regular buildings. As a result, the seismic lateral load distribution along the height of a structure is linear.

EC8 adopts the approach of a linear seismic lateral load distribution but it fails to account in quantitative terms for the contribution of higher vibration modes, which influence significantly the response of the upper storeys. Moreover, for tall buildings, which are relatively flexible and have their fundamental periods in the long-period region, the fundamental mode significantly deviates from a straight line and lies between a straight line and a parabola with a vertex at the base (Chopra and Newmark, 1980; Gupta, 1990). In these buildings, the influence of higher mode shapes on the total response of the upper storeys is more significant than in short-period structures and the linear load distribution underestimates their response. To account for the effect of higher modes and longer fundamental periods, UBC and NZS seismic codes require a concentrated force to be applied at the top of the building (Equations (C.3.6) and (A.5.6)) and the remaining force to be distributed linearly along the height (Equations (C.3.7) and (A.5.5)). NZS code requires a constant concentrated top force equal to 8% of the base shear in order to encourage simplicity while UBC indicates a concentrated force proportional to the structural period.

Having determined the vertical distribution of the earthquake lateral load acting at floor levels, the storey shears can be calculated by simply summing up the lateral forces acting at the floor levels above the storey under consideration. The design moment in a specific storey is calculated as the product of the storey shear force and the appropriate design eccentricity. Codes specify two design eccentricities called the first design eccentricity e_{D1} and the second design eccentricity e_{D2} to account for the increased, or decreased, strength demand in certain elements. The design eccentricities define the locations through which the design lateral load must be applied to induce the design torque about a vertical axis through CR of the specific floor. Both design eccentricities are implemented to determine the strength of the structural elements and the case resulting in the highest elemental strength is adopted. Most current seismic codes specify the design eccentricities as

$$e_{D1} = e_{d1} + e_a \tag{4.2.1}$$

$$e_{D2} = e_{d2} - e_a \tag{4.2.2}$$

where e_{d1} and e_{d2} are called dynamic eccentricities and have the form of $e_d = a\,e_s$, where e_s is the static eccentricity of the floor under consideration and e_α is the accidental eccentricity (Figure 4.2.1).

Thus, the design eccentricities take into account a dynamic amplification of the static eccentricity and the accidental torsional effects that could occur in all buildings due to the torsional earthquake component, geometric irregularities, and differences between actual and assumed distributions of mass, stiffness and strength. The accidental eccentricity usually takes the form of $e_\alpha = \beta\, b$, where b is the dimension of the building measured perpendicular to the earthquake direction, and β is a coefficient defining the accidental eccentricity as a proportion of the b dimension. A summary of the design eccentricities of the codes employed is given in Table 4.2.1.

UBC is the only seismic code not specifying the accidental factor β as a constant. A torsional regularity provision is defined in the UBC standard, whereby the minimum accidental eccentricity of $0.05b$ is modified by the factor A_x that is dependent on the rotational deformation of the building under a defined static lateral load and should not exceed 3.0 (Equation (C.3.8)).

The NZS seismic regulations indirectly specify the design eccentricities by stipulating that the design seismic force must be applied along a horizontal axis in the direction of the earthquake ground motion, at a distance of $\pm 0.1b$ from CM. The advantage in defining the design eccentricity in this manner is that there is no need to determine the location of CR and, therefore, the process for the strength calculation of the structural elements is considerably simplified (Section A.3.1).

EC8 is the only major earthquake resistant design code that explicitly considers the plan dimensions of the structure (indirectly the plan aspect ratio of the structure) and the torsional to translational stiffness ratio in its torsional provisions (Section B.3). The first design eccentricity of the EC8 code includes an additional eccentricity component e_1 introduced to account for the dynamic, or coupling, effects resulting from simultaneous torsional and translational oscillations. The additional eccentricity is defined as the lower of the two values calculated from Equations (B.3.1) and (B.3.2). By including the additional eccentricity, the EC8 first design eccentricity is dependent on structural parameters, such as static eccentricity, torsional and lateral stiffness and floor plan dimensions of the building under consideration.

Code	First Design Eccentricity	Second Design Eccentricity
NZS-92	$1.0\ e_s + 0.10b$	$1.0\ e_s - 0.10b$
EC8-93	$1.0\ e_s + 0.05b + e_1$	$1.0\ e_s - 0.05b$
UBC-94	$1.0\ e_s + 0.05A_x b$	$1.0\ e_s - 0.05\ A_x\ b\ (e_s \leq 0.05A_x b)$ $0.0\ e_s - 0.05\ A_x\ b\ (e_s > 0.05A_x b)$

Table 4.2.1 Design eccentricities of the three seismic codes employed in this study.

In the case of multi-storey asymmetric buildings, there are two alternative approaches employed by the codes in determining the torsional moments: the floor eccentricity and the storey eccentricity approach (Tso, 1990).

In the **floor eccentricity approach** (adopted by the NZS code), the torque acting at a floor level is calculated as the product of the lateral force acting at floor level and the design eccentricity (Figure 4.2.2). The design floor eccentricity is a function of the floor stiffness eccentricity, which is the horizontal distance between CM and CR. The centres of rigidity are defined as the set of points at each floor level through which the lateral forces induce only translation of the floor diaphragms. In general, the locations of CR at floor levels depend on the vertical and horizontal stiffness distribution and on the vertical distribution of the applied lateral load. Having determined the lateral forces and torques acting at floor levels, the internal forces of the individual structural elements can be calculated.

In the **storey eccentricity approach** (adopted by the EC8 and UBC codes), a cut is made at each storey level and the storey torque is equal to the product of the storey shear and the design storey eccentricity (Figure 4.2.3). The design storey eccentricity is a function of the storey stiffness eccentricity, which is the horizontal distance between the location of the resultant of the lateral forces acting above the storey being considered and the storey shear centre of the storey. The storey shear centres are defined as the points at each storey level through which the resultant of the element shears pass when the lateral forces pass through CR and give no rotation of the floor diaphragms. The storey shear centres are also dependent on the horizontal and vertical stiffness distributions and on the vertical distribution of the lateral load.

Unlike the case of single-storey buildings, the centres of rigidity at floor levels and the storey shear centres are not always the same set of points. However, Tso (1990) has shown that the above two approaches are equivalent if the above floor and storey stiffness eccentricity concepts are employed.

4.2.3 Linear Elastic Modal Analysis

The modal analysis procedure, also called the dynamic analysis procedure in some codes, is specified to be generally applicable to both regular and irregular structures. For irregularly asymmetric buildings, the use of the modal analysis rather than the equivalent static force procedure is required by the codes. In modal analysis, the actual vibration periods and the mode shapes of asymmetric buildings are calculated by solving the eigenvalue problem

$$[K]\{\phi_i\} = \omega_i^2[M]\{\phi_i\} \qquad (4.2.3)$$

where $[K]$ and $[M]$ are, respectively, the global stiffness and mass matrices, ω_i is the ith natural frequency, and $\{\phi_i\}$ is the ith vibration mode of the building. First the normal modes and the natural periods of the system are determined. Then, for each mode, the maximum accelerations are found from the design spectrum, the effective modal masses are determined and the maximum inertia forces are calculated followed by the maximum response parameters (moments, shears, displacements etc.)

The modal responses are combined employing either the Square Root of the Sum of Squares (SRSS) procedure or the Complete Quadratic Combination (CQC) procedure to obtain an approximate value of total response. The SRSS method is based on the concept that all modes do not reach their maximum value simultaneously and that the response of both translational and torsional modes may be considered as independent from each other. Thus, their most probable maximum values result through SRSS. When the concept of the mode independence is not fulfilled, and which according to the EC8 code is that for any two successive modes of vibration the lower period is smaller than the 90% of the higher period, the CQC method must be adopted.

The total lateral force, or strength, at the building's base obtained from the modal analysis is usually lower than the base shear determined from the static force procedure. Hence, some codes require that the strength of all structural elements to be scaled up such that the total strength at the building's base is equal to (NZS) or at least 90% (UBC) of the base shear determined by the static force procedure. However, EC8 code does not require such a scaling procedure.

4.2.4 The Necessity to Improve Code Torsional Provisions

Current code analysis procedures for earthquake resistant design are based largely on the linear elastic theory and can provide satisfactory estimates of the strength demands of elastically responding elements. The present philosophy of earthquake resistant design allows buildings to be excited well into the inelastic range, by utilising the structural ductility and energy dissipation capacity, without requiring a non-linear inelastic analysis. The non-linear hysteretic behaviour of the resisting elements due to yielding influences the torsionally coupled response of asymmetric buildings and triggers behaviour different from that predicted by linear elastic theory. Lessons from past strong earthquakes have demonstrated that some of these code torsional provisions are inadequate and need to be evaluated and improved.

Seismic building codes recommend the elastic modal analysis procedure and the equivalent static analysis to account for the torsional effects arising in TU buildings. Previous studies (Duan and Chandler, 1993) have shown that modal analysis is inadequate for the design of regularly asymmetric buildings excited well into the inelastic range, even if the total strength obtained by modal analysis has been scaled up to be the same as in corresponding symmetric structures. The equivalent static force procedure as recommended by codes may also be deficient in accounting for additional ductility demand in critical elements.

Chandler and Duan (1993) concluded that a solution for achieving satisfactory inelastic performance of asymmetric buildings without carrying out inelastic dynamic analysis could best be achieved by improving the static force procedure rather than relying on linear elastic modal analysis. Based on results obtained for both single-storey and multi-storey models, modified equivalent static force procedures for torsional design have been developed (see Chapter 9). Due to the simplicity of the equivalent static method and the satisfactory results produced by modified equivalent static procedures (Chandler and Duan, 1993; Moghadam and Tso, 1996), these methods should be further examined with a view of making refinements to accommodate the different design criteria for elastic and inelastic systems. In this study, the equivalent static method has been employed for the design of multi-storey regularly asymmetric models (Chapters 5 – 9). Chapter 9 presents the conclusions drawn on the equivalent static analysis from the inelastic analyses of the previous chapters and attempts to check the effectiveness of a modified equivalent static method.

4.3 STRUCTURAL DESIGN PROVISIONS

4.3.1 Reinforcement Design Procedure

Although load factors for dead and live loads are quite similar in all codes, a substantial discrepancy exists in the treatment of the factors applicable to earthquake forces. In the UBC code, factors of approximately 1.4 are adopted, while, in the NZS and EC8 codes, the appropriate factor is unity. The original intention of load factors implemented in the strength design of the structural elements in the 1960's was to avoid the full development of the resistance capacity of the elements under the maximum loads likely to occur. Therefore, with a seismic design philosophy based on ductility this approach is inappropriate, since development of strength is expected under the design-level ground shaking. Applying load factors to a force level that has already been reduced from the elastic response level merely implies a reduction to the expected ductility requirement and obscures the true level of ductility required since, in UBC code, the real force reduction factor is approximated by R/1.4.

The section design provisions of codes from the United States, New Zealand and European Community indicate similar procedures for the calculation of the reinforcement needed for the structural elements. Therefore, for simplicity reasons, it was considered appropriate to calculate the amount of reinforcement of a section using the design provisions of Eurocode 2 and Eurocode 8. Small differences in the reinforcement requirements do not change the overall response of the models and it is not among the objectives of this study to examine the effectiveness of the reinforcement provisions of existing building codes. On the other hand, employing different reinforcement procedures would further complicate the parametric analyses and it would make harder to distinguish and isolate the effect of code torsional provisions on the seismic response of the structures.

The design procedure employed for gravity loading in Eurocode 2 is also used in Eurocode 8 for seismic loading, based on the fact that the material safety factors for concrete and steel remain unchanged and the 0.85 factor in the stress block is retained to account for cyclic loading. The design procedure for the reinforcement calculation of the structural elements is formulated automatically by using a created computer program that includes the design provisions for both columns and beams. For a fully automated design procedure, analytical approximations of the moment-axial force (M-N) interaction curves are also incorporated, and the flexural reinforcement needed is calculated based on the results of the elastic analyses carried out by SUPER-ETABS computer program (Maison and Neuss, 1983).

4.3.2 Seismic Design Procedure for Beams

Beams are the structural elements that dissipate most of the seismic energy in a structure through flexural yield mechanisms, and they are designed and detailed to be highly ductile. The minimum beam width should be at least equal to 20 cm for practical reasons and to reduce the slenderness of the beams. To avoid the possibility of lateral instability of the web in potential plastic hinge regions, the width to height ratio of the web must be higher than 0.4 for highly ductile structures.

The seismic code provisions for the design of the beams apply provided that

$$N > -0.1 A_c f_{cd} \tag{4.3.1}$$

where N is the axial load, f_{cd} is the design value of concrete cylinder compressive strength, and A_c is the total cross-sectional area of the concrete section under consideration. Otherwise, the seismic code provisions for the columns have to be used for the design of the beams. The design bending moments of the beams are obtained from the elastic analyses of the models and the maximum bending moments computed are used for the calculation of the reinforcement needed. At each storey level, the interior ends of two beams are designed for the same design moment to avoid termination and anchorage of beam bars at the interior beam-column joints, where reinforcement congestion could create construction difficulties.

The limitations about the longitudinal steel ratio of the beams $\rho = A_s / (bd)$ (where A_s is the steel area within the tension zone, b is the width and d is the depth of the section) concern a minimum and a maximum value defined as

$$\rho_{min} = 0.5 \frac{f_{ctm}}{f_y} \tag{4.3.2}$$

$$\rho_{max} = 0.35 \frac{f_{cd}}{f_{yd}} \frac{\rho_2}{\rho_1} + 0.0015 \tag{4.3.3}$$

where f_{ctm} is the mean value of axial tensile strength of concrete, f_y is the yield strength of the reinforcement, f_{yd} is the design yield strength of the reinforcement, ρ_2 is the compression reinforcement ratio, and ρ_1 is the tensile reinforcement ratio, limited by ρ_{min} and ρ_{max}. Equation (4.3.3) is for a high ductility capacity of the beams and aims to ensure adequate ductility without reinforcement congestion.

Beams need an additional reinforcement at their supports and, consequently, compression reinforcement equal to 50% of the corresponding tension reinforcement is applied to ensure an adequate ductility level. These steel bars are appropriately anchored in concrete, so that they can operate as tension reinforcement in case of moment reversal. However, in order to ensure this behaviour, the structural element has to be secured against premature shear failure and, as a result, beams can carry much larger moments than the design moments. The 50% lower bound of compression reinforcement intends not only to recognise the cyclic nature of the seismic response, but also to include the beneficial effect of the compressive reinforcement on the inelastic deformation capacity of the beam under large negative moment (Penelis and Kappos, 1997)

4.3.3 Seismic Design Procedure for Columns

The seismic design provisions for columns apply when $N < -0.1A_c f_{cd}$, otherwise the seismic design provisions for beams are employed. For structures located in areas of high seismic risk, the limitations on section dimensions include a minimum column width equal to 30 cm for highly ductile structures. If the interstorey drift sensitivity coefficient θ (Equation (B.10.1)) is higher than 0.1, the slenderness effect has to be limited (Section B.10.1) and the column dimensions have to be smaller than a certain fraction of the larger distance l_0 between the inflection point (M = 0) and the end column section $(b / l_0 \geq 1/8)$.

The maximum and minimum design axial forces for the columns are determined from the elastic static analyses of the frames under different load combinations. The columns are designed based on the most critical combination of design bending moment and axial force. The limitations on longitudinal reinforcement include a minimum steel ratio ρ_{min}=0.01 and a maximum value ρ_{max}=0.04. The minimum value aims to ensure appropriate constitution of cracked concrete without yielding and the maximum value avoids congestion and high shear demands. At least three bars at each column side should be applied to enhance shear resistance of a beam-column joint.

At each joint, the column section immediately above, or below, the joint is designed for its own critical combination of design bending moment and axial force, and the larger amount of reinforcement obtained is used for both sections. The use of the same reinforcement above and below a joint avoids the termination of the anchorage of the column longitudinal steel bars within the joint. All column steel bar splices occur near mid-column height where the bending moment is relatively small.

After deriving the column strengths for the most unfavourable axial force, the capacity design criterion is applied based on the equilibrium of moments at beam-column joints. If necessary, the column reinforcement is appropriately increased, in order that the plastic hinge formation in columns is avoided. The exceptions from the EC8 capacity design procedure include the base of the ground storey columns, the top storey columns in multi-storey structures, buildings with one or two storeys, and one column out of every four columns. The flexural strength of the columns relative to the beams framing into a specific joint (strong column-weak beam concept), is defined as

$$\sum M_c \geq 1.35 \sum M_b \qquad (4.3.4)$$

for a structure designed for high ductility capacity, where $\sum M_c$ is the sum of the design flexural strengths of the columns framing into the joint, and $\sum M_b$ is the sum of the design flexural strengths of the beams framing into the joint.

Although the equilibrium condition at beam-column joints controls the column design moments and takes into account the actual moments of the beams, plastic hinges may still form at some columns. This occurs due to the variation of the column strength with axial force, the shift of the point of contraflexure in columns, the effect of the bi-axial seismic loading, the stain-hardening effect in steel, and the contribution of slab reinforcement to beam strength.

4.4 SELECTION OF EARTHQUAKE GROUND MOTION

4.4.1 Influence of A/V Ratio on Structural Response

In dynamic seismic analysis, if the site of the structure under consideration is known, suitable earthquake ground motions may be selected from records reflecting the local seismological and geotechnical conditions. Otherwise, when the response of a structure is being investigated to general earthquake loading, it is necessary to consider a wider range of earthquake loads because forces, displacements and ductility demands are highly dependent on the ground motion. Furthermore, it is necessary to ensure that all ground motions employed possess similar intensities, so that a comparison of the structural response can be made.

The peak ground acceleration to peak ground velocity ratio A/V is a simple, yet meaningful, parameter to identify the characteristics of individual accelerograms (Zhu at al., 1988a, 1988b). The peak ground acceleration (in units of g) is associated with high frequency waves while the peak ground velocity (in units of m/sec), obtained from the integration of the ground acceleration, is associated with moderate to low frequency waves. Hence, ground motions with large amplitude and high-frequency oscillations possess high A/V ratios while records with few long-duration acceleration pulses have low A/V ratios. Similarly, due to the frequency-dependent attenuation of seismic waves, the attenuation of velocity with distance is slower than the attenuation of acceleration, causing the accelerograms recorded near the earthquake source to possess high A/V ratios. In addition, the filtering effect of the ground medium causes long duration records associated with high A/V ratios while structures on rock and firm soils experience relatively shorter duration and higher frequency base excitation.

Sawada et al. (1992) performed a statistical study on a selection of Japanese earthquake records from which it was concluded that lower A/V ratios are exhibited by earthquakes with lower predominant frequencies, broader response spectra, longer duration, increased magnitude, and increased epicentral distance. These conclusions have been confirmed for Californian records (Zhu et al., 1988a, 1988b) and North American and Eurasian accelerograms (Chandler, 1991; Tso et al., 1992) indicating that a range of A/V ratios include all the seismological features likely to influence the response of a structure. The A/V ratio reflects the frequency content of an accelerogram and earthquake records possessing high A/V ratios are more critical for stiffer structures whereas low A/V ratios are more critical for flexible ones. Design codes incorporate this feature by prescribing different design spectra for structures on various soil types usually expressed in terms of peak ground acceleration.

Depending on the A/V ratio, ground motions are classified in three groups having records with low $(A/V < 0.8)$, normal $(0.8 \le A/V \le 1.2)$, and high $(1.2 < A/V)$ ratios. These provisions lead to more uniform ductility demands (Zhu et al., 1988a, 1988b; Tso et al., 1992) and a closer correlation is achieved between strength supply and demand. The implied variety of ductility demands experienced by structures designed to different codes places increased emphasis on the need to include a meaningful range of accelerograms in any analysis procedure.

4.4.2 Effect of Ground Motion Scaling on Seismic Response

All the important features of seismic response, such as strength and ductility demand, are highly dependent on the ground motion intensity and, therefore, when considering seismic response due to a number of earthquake motions, it is important to ensure that their intensities are similar. In this manner, the effects of other features such as the frequency content and duration of the loading can be assessed. Moreover, when evaluating the performance of a structure designed to resist code-prescribed seismic loads through the inspection of its response to actual ground motions, the seismic energy imparted into the structure by the imposed base accelerations should be equal to that implied in the code design spectrum. The recorded accelerograms display wide variations in intensity and they are scaled to a common intensity level to ensure that each one imposes similar levels of demand. The loads imposed on a structure during an earthquake are proportional to its instantaneous acceleration due to the base motions. On account of this, recorded ground motions are scaled to a peak acceleration value that agrees with the methods through which codes normally define seismic loads.

It is possible to identify three regions of structural periods in which the structural response is dependent on the values of ground motion acceleration, velocity or displacement and can be respectively defined as $T_y < 0.5$ sec, 0.5 sec $< T_y < 3.0$ sec and T_y > 3.0 sec. Consequently, it would seem appropriate to scale earthquake records in a manner that reflects the periodicity of the structure under consideration, as confirmed by a number of studies on large selections of earthquake records (Zhu et al., 1988a, 1988b; Chandler, 1991). In these studies, the degree of spectral dispersion is reduced in the low-period range when acceleration scaling is applied while a similar reduction is observed in intermediate- and long-period ranges with velocity scaling.

4.4.3 Spectrum Intensity of Earthquake Records

In addition to the peak acceleration, velocity and displacement, many other parameters have been proposed to measure the intensity of the earthquake ground shaking. Some of these parameters are directly based on the ground motion data and others on the response quantities of linearly elastic oscillators. A parameter derived directly from the response of linearly elastic oscillators is the spectrum intensity. Housner (1970) defined the spectrum intensity as the area under a pseudo-velocity spectrum curve between the periods 0.1 and 2.5 seconds

$$SI(\beta) = \int_{0.1}^{2.5} S_v(T,\beta)dT \tag{4.4.1}$$

where S_v is the pseudo spectral velocity, T is the response period and β is the fraction of critical damping. The pseudo-spectral velocity S_v is obtained from the integration with respect to period of the acceleration response spectrum

$$S_v(T_i,\beta) = \int_{0.0}^{T_i} S_a(T,\beta)dT \tag{4.4.2}$$

where S_a is the spectral acceleration. Thus, S_v may be considered to be the area under the acceleration response spectrum and the spectrum intensity considered to be the area under the pseudo-spectral velocity characteristic between the limits indicated.

The spectrum intensity can be interpreted as an average measure of the severity of ground shaking regarding its effect on the elastic response of structures in the sense that it is related to elastic vibrational energy and covers a period range of engineering interest. The effect of scaling to equal spectrum intensities is to ensure that earthquake records possess equal energy contents between the periods 0.1 and 2.5 seconds. Nau and Hall (1984) have shown that such a procedure significantly reduces response spectral dispersion in the range 0.5 - 3.0 seconds, producing a more consistent level of displacement ductility demand. Zhu (1989) concluded that peak ground velocity correlates well with the spectrum intensity over earthquake records having drastically different A/V ratios, whereas the relationship between peak ground acceleration and spectrum intensity strongly depends on the A/V ratio. Therefore, the peak ground velocity was found to be a superior parameter to describe the intensity of ground shaking at a site for building design, as compared to peak ground acceleration.

4.4.4 Selection of Earthquake Records

The inelastic time-history analyses carried out in this study are based on six earthquake events. Adopting an ensemble of earthquakes reduces the effects of frequency content and of particular characteristics of the individual earthquake ground motions. The key characteristics of the records selected are summarised in Table 4.4.1 and their ground acceleration time-histories are presented in Figures 4.4.1 and 4.4.2. For bi-directional seismic loading conditions applied in models with structural elements in both orthogonal directions (Chapter 8), both horizontal components of the Imperial Valley and Kern County seismic events are applied (see also Table 4.4.1).

GROUND MOTION RECORDS SELECTED					
Earthquake	**Date**	**M_L**	**Site**	**Epicentral Distance (km)**	**Component**
Imperial Valley, California	18/5/1940	6.6	El Centro	8	S00E & S90E
Kern County, California	21/7/1952	7.6	Taft Lincoln	56	S69E & N21E
Mexico Earthquake	19/9/1985	8.1	El Suchil	230	S00E
Montenegro, Yugoslavia	15/4/1979	7.0	Albatros Hotel	17	N00E
San Fernando, California	9/2/1971	6.4	3407 6th St.	39	S00W
Mexico Earthquake	19/9/1985	8.1	La Union	84	N00E

Table 4.4.1 Characteristics of the earthquake ground motions selected for the inelastic time-history analyses carried out in this study.

To minimise site effects, the earthquake ground motions selected were recorded on stiff soil, or rock, and earthquake records from different parts of the world are included to cover a broad range of seismological and geological conditions. The local (Richter) magnitude M_L of the earthquake events range from 6.4 to 8.1 (Table 4.4.1). Their epicentral distances to the recording sites vary between 8 and 230 km and, therefore, both 'near-field' and 'far-field' seismic events are included. The strong motion duration of each earthquake accelerogram is 20 seconds, so that several cycles of inelastic response are undergone, even for long-period structures. The ground motions selected belong to the intermediate A/V ratio range and their A/V ratios vary from 0.82 to 0.99 (Table 4.4.1).

4.4.5 Scaling of the Selected Ground Motions

The multi-storey frame models investigated in this study have fundamental periods that fall into the velocity-controlled range of the response spectrum. Therefore, the lateral component of each earthquake record, applied to models with structural elements only in the lateral direction, is scaled to a common peak ground velocity, which is equal to 0.35 m/sec (Table 4.4.2) and represents a severe earthquake loading that can generate inelastic response. Since all records selected have similar A/V ratios, with a mean value equal to 0.87, the peak accelerations of the scaled accelerograms are similar as well and approximately equal to 0.30g. After scaling the lateral component of the selected earthquake records at a peak velocity equal to 0.35 m/sec, the design spectrum intensity (SI) of each record is calculated using Equation (4.4.1). The design spectrum intensities of all the accelerograms don't differ from each other more than $\pm 20\%$ (Table 4.4.2) and no further scaling to a common SI level is deemed necessary.

For the bi-directional response analyses carried out in Chapter 8, the ground motion recordings in both orthogonal horizontal directions are employed. The earthquake component with the larger maximum ground velocity acts in the lateral direction and it is termed lateral earthquake component (El Centro S00E and Taft Lincoln S69E) (Table 4.4.3). The other component, termed transverse component, acts in the x-direction (El Centro S90E and Taft Lincoln N21E). The transverse earthquake component is not normalised to a common peak velocity (*V*=0.35 m/sec), but it is scaled by the same factor (A_v/A) applied to the lateral component in order to maintain the integrity of the seismic event (Tables 4.4.2 and 4.4.3). As a result, the scaling applied does not alter the ratios of peak ground acceleration, velocity and displacement for the lateral to transverse components of the earthquake events.

SCALING OF THE LATERAL COMPONENT OF THE EARTHQUAKE RECORDS								
Earthquake Record		**A/V**	**V (m/s)**	**A (g)**	**A_V (g) for V=0.35 m/s**	**Factor A_V/A**	**SI**	**SI x (A_V/A)**
El Centro	S00E	0.93	0.375	0.348	0.326	0.94	140.55	131.41
El Suchil	S00E	0.91	0.116	0.105	0.319	3.03	51.83	157.05
Taft Lincoln	S69E	0.90	0.198	0.179	0.316	1.77	72.60	128.28
Albatros	N00E	0.85	0.202	0.171	0.298	1.74	81.61	141.67
3407 6th St.	S00W	0.83	0.193	0.161	0.292	1.81	68.07	123.20
La Union	N00E	0.82	0.203	0.166	0.287	1.73	86.46	149.32
Mean, μ		***0.87***	***0.214***	***0.188***	***0.306***	-	***83.52***	***138.49***
St. dev., σ		***0.04***	***0.008***	***0.075***	***0.015***	-	***27.77***	***11.97***
σ/μ		***0.05***	***0.364***	***0.400***	***0.048***	-	***0.33***	***0.09***

Table 4.4.2 Scaling of the lateral component of the earthquake ground motion accelerograms to a common peak ground velocity (V = 0.35 m/sec).

SCALING OF THE TRANSVERSE COMPONENT OF THE EARTHQUAKE RECORDS						
Earthquake Record		**A/V**	**V (m/s)**	**A (g)**	**A_V (g) for V=0.35 m/s**	**Factor A_V / A**
El Centro	S00E	0.93	0.375	0.348	0.326	0.94
	S90E	0.67	0.324	0.217	0.204	0.94
Taft Lincoln	S69E	0.90	0.198	0.179	0.316	1.77
	N21E	0.82	0.190	0.156	0.276	1.77

Table 4.4.3 Scaling of the transverse component of the earthquake accelerograms.

4.5 NON-LINEAR DYNAMIC ANALYSIS

4.5.1 The Necessity of 3D Non-linear Dynamic Analysis

In recent years, extensive experimental and analytical research has been devoted to study RC frames under seismic loading and the conclusions drawn from studies of planar frames can be directly extended to real 3D buildings provided that they are symmetric in plan. In presence of plan irregularity due to the asymmetric mass, stiffness or strength, the seismic response of 3D buildings is characterised by lateral-torsional coupling, which results in non-uniform plan-wise patterns of plastic demands. Therefore, each frame is subjected to a different cyclic loading history when compared to the loading history it would undergo if located in a symmetric scheme.

In the past several years, studies have been devoted to evaluate the effects of torsional coupling in order to generate design guidelines for 3D asymmetric structures. However, almost all results have been derived from parametric analysis of simplified single-storey models and very simple bilinear force-deformation relationships have been assumed for the resisting elements (Goel and Chopra, 1991a, 1991b; De Stefano et al., 1993a, 1993b; De La Llera and Chopra, 1994a, 1994b, 1994c). Although such investigations have led to the identification of the main parameters affecting the torsional behaviour of structures, further studies are needed to verify their applicability to realistic 3D RC buildings. In fact, single-storey models are not able to account for the modifications in the response of the frames arising due to the activation of different dissipation mechanisms. Additionally, the force-deformation hysteretic behaviour of RC members is more complex than that represented by simple bi-linear models, since significant decay in the element properties is produced under cyclic loading.

4.5.2 Computer Program for 3D Dynamic Analysis

The 3D dynamic inelastic analysis of multi-storey systems is performed using step-by-step integration for small successive time steps. Within every integration time interval, the response of the inelastic system is considered linear and the value of the stiffness is equal to the slope of the local tangent to the load-deflection curve. Thus, the response of the non-linear system is considered to be the response of successive linear systems with different stiffnesses while yielding occurs in some members and the stiffness of the structure changes. Hence, a considerable amount of computational time is required for the dynamic inelastic analysis of multi-storey structures. Although there are some reservations about this method, mainly related to the uncertainties regarding the stiffness and the damping introduced in the model, dynamic non-linear analysis constitutes a powerful tool for the study of the response of the structures.

For the inelastic dynamic analyses carried out in this study, the structural dynamics computer program DRAIN-TABS (Guendelman-Israel and Powell, 1977) has been employed to determine the effects of intense seismic loading on the structural response. The program analyses 3D structures subject to earthquake ground motions and combines the features of the planar 2D structural program DRAIN-2D (Kanaan and Powell, 1973) and of the 3D elastic analysis program TABS (Wilson and Dovey, 1972). However, the solution procedure is substantially more complicated than in DRAIN-2D due to of the use of substructure techniques to consider coupling of the frames through the diaphragms.

Each frame of a structure is idealised as a finite number of elements (members) that can deform connected to a finite number of nodes (joints). The nodes are located in an H, Z co-ordinate system, where H lies in the global X, Y plane and Z is vertical. Nodes are commonly considered as points and the elements may be one or two-dimensional. Loads may be applied to the frame directly at the nodes, or through the elements, in terms of fixed-end forces. Lumped masses for consideration of vertical and rotational inertia effects may be placed at individual nodes. However, masses for consideration of horizontal inertia effects should preferably be lumped into translational and rotational inertia of the diaphragms.

For any frame, the degrees of freedom are typically the two translational and one rotational displacement at each node. The relationship between the potential displacements of the nodes and the actual degrees of freedom is defined by an identifying array. The numbering of the degrees of freedom is affected by the constraints of nodes having zero displacements, identical displacements, or displacements connected to diaphragms. The inelastic behaviour of RC members is taken into consideration with the adoption of appropriate models for the moment-rotation diagram at their ends (Section 4.5.4).

The input needed by the program for the analysis of the structures include the geometry of the system, the strength of the structural members, the masses of the frames, the viscous damping ratio ξ (Section 4.5.3), and the digitised time history of acceleration at the base of the structure (Section 4.4). Using these input data, the program produces the moment-rotation diagrams of the structural elements, the mass matrices, the stiffness matrices, the damping matrices and the excitation vector. The mass matrix is the only matrix that remains constant during the integration procedure. After assembling the mass, stiffness, damping and load vector matrices of the individual frames, the relevant matrices for the whole building are assembled and the modified equilibrium equations are solved.

4.5.3 Damping Calculation

For non-linear response analyses, the response is not obtained by superposition of the uncoupled modal responses and, hence, the damping cannot be expressed by the damping ratios, instead an explicit damping matrix is needed. Thus, it is appropriate to define the proportional damping matrix for the initial elastic state of the system, before non-linear deformations occur. Furthermore, it is assumed that this damping property remains constant during the response, even though the stiffness may be changing and causing hysteretic energy losses in addition to the viscous damping losses. The simplest way to formulate a proportional damping matrix is to make it proportional to either the mass, or the stiffness, matrix because the undamped mode shapes are orthogonal with respect to each of these. Thus, different damping types may be specified:

Mass-proportional damping $\mathbf{C} = \alpha \mathbf{M}$ is assumed, where **C** is the viscous damping matrix, **M** is the mass matrix, and

$$a = 4\pi\xi / T_1 \tag{4.5.1}$$

where T_1 is the fundamental period of the structure and ξ is the damping ratio that for inelastic analysis of RC structures with a typical value 2 – 5%. The value used in this study is equal to 5%.

Stiffness-proportional damping $\mathbf{C} = \beta\mathbf{K}_T$ or $\mathbf{C} = \beta_0\mathbf{K}_0$ is assumed, where $\mathbf{K}_T$ is the current tangent stiffness matrix at any time, $\mathbf{K}_0$ is the original elastic stiffness matrix at any time, and

$$\beta_0 = \beta = \xi T_1 / \pi \tag{4.5.2}$$

The above expressions show that, for mass-proportional damping, the damping ratio is directly in proportion with the period while, for stiffness-proportional damping, it is inversely proportional to the period. It is important to note that the dynamic response generally includes contributions from all modes, even though only a limited number of modes are included in the uncoupled equations of motion. Thus, neither of these types of damping matrix is suitable for use with an MDOF system in which the frequencies of the significant modes span a wide range because the relative amplitudes of the different modes are seriously distorted by inappropriate damping ratios. An obvious improvement results if the damping is assumed to be proportional to a combination of mass and stiffness matrices, and this type of damping has been employed for the inelastic analyses carried out in this study.

Stiffness/mass-proportional damping $\mathbf{C} = a\mathbf{M} + \beta\mathbf{K}_T$ or $\mathbf{C} = a\mathbf{M} + \beta_0\mathbf{K}_0$ called Rayleigh damping. Because detailed information about the variation of damping ratio with frequency seldom is available, it is usually assumed that the same damping ratio applies to both control frequencies and, in this case, the proportionality factors are given by the following simplified equations.

$$a = \frac{4\pi}{(T_1 + T_2)}\xi \tag{4.5.3}$$

$$\beta = \beta_0 = \frac{T_1 T_2}{\pi(T_1 + T_2)}\xi \tag{4.5.4}$$

4.5.4 Reinforced Concrete Element Models

The inelastic behaviour of RC members is taken into consideration with the adoption of appropriate models for the moment-rotation diagram at their ends. For columns, a **beam-column element** is used (type 2 element) with elastoplastic bilinear behaviour (Figure 4.5.1) and the yield moment-axial force interaction is controlled by idealised yield surfaces. For beams, the **RC beam element** (type 6 element) is used, which is a modification of the Takeda model and consists of an elastic beam with inelastic springs at its ends). This model is characterised by stiffness degradation and represents with adequate reliability the inelastic response of RC members with predominately flexural deformation to cyclic loading (Figure 4.5.2).

For a quantitative estimation of the model parameters, the cross-sectional dimensions, the reinforcement, and the characteristics of the construction materials are needed for every structural element. Most computer programs for seismic analysis of buildings use point hinge models to represent the inelastic behaviour of RC members. In such models, inelasticity is permitted only at predetermined sections, which, in the case of seismic loading, are the member ends. The rest of the member remains in the elastic range and an appropriate value of stiffness has to be specified which should represent the average flexural stiffness of the member. Following recommendations from previous studies (Kappos, 1986b), upper and lower bounds are considered for the stiffnesses of the elastic part of the point hinge elements, especially for relatively low earthquake intensities. In this study, the stiffness is equal to 40% of the stiffness gross-section *EI* of the beams and 80% of the stiffness gross-section of the columns.

4.6 RESPONSE PARAMETERS

4.6.1 Overall Response Parameters

The overall response parameters are either related to the response of a frame at storey levels or associated to the response of a frame or structure. The **storey level response parameters** include the maximum storey displacement, the interstorey drift, shear and moment. The **global response parameters** of a frame, or structure, consist of the maximum displacement and the displacement ductility. The response parameters essentially employed in earthquake resistant design are the maximum deformation δ_{max}, the displacement ductility ratio μ_δ and the interstorey drift Δ_i.

Deformability or maximum deformation δ_{max} is the capability of a material, structural element, frame or structure to deform before ultimate failure or rupture.

Ductility is the capability of a material, structural component, element or structure to undergo deformation after its initial yield without any significant reduction in yield strength and it is defined as

$$\mu = \delta_{max} - \delta_{yield} \tag{4.6.1}$$

Displacement ductility ratio is the ratio of the maximum structural or element deformation undergone without significant loss of yield strength to the initial yield deformation and it is determined as

$$\mu_\delta = \delta_{max} / \delta_{yield} \tag{4.6.2}$$

Interstorey drift ratio is the ratio of the maximum interstorey displacement of a floor level *i* divided by the storey height

$$\Delta = (\delta_i - \delta_{i-1}) / h_i \tag{4.6.3}$$

where δ_i is the maximum displacement of a floor level *i* and h_i is the storey height.

4.6.2 Local Response Parameters

The deformation response parameters for structural members include member rotations, plastic hinge lengths, rotation ductility, curvature ductility and plastic hinge rotation. Ductility factors for flexural members may be defined in terms of rotations or in terms of curvatures. Curvature ductility ratio would be a good measure of inelastic flexural deformations, but the results from typical inelastic analyses are rotations, rather than curvatures, since they are computed on the basis of plastic hinge idealisations. Reliable predictions of plastic curvatures require refined plasticity formulation that have seldomly been attempted due to high computational requirements. Hence, for members in structures undergoing seismic motion, the definition of a rotation ductility factor becomes difficult, because the bending moment and the yield rotation at a member end depends on the time-varying rotations.

The **rotational ductility factor** is a useful index for quantitatively expressing the ductility of an element and it is defined as

$$\mu_\theta = \frac{\theta_{max}}{\theta_y} \tag{4.6.4}$$

where θ_{max} is the maximum total rotation at the end of a member and θ_y is the yield rotation, which in the case of beams usually corresponds to the yielding of tension steel. In order to estimate θ_y, assumption needs to be made regarding the shape of the member at first yielding. The rotational ductility factor may also be written as

$$\mu_\theta = 1 + \frac{\theta_p}{\theta_y} \tag{4.6.5}$$

and this formula is commonly used for an inelastic dynamic analysis, where θ_p is the plastic hinge rotation of the members' ends and is calculated by the inelastic analysis.

The main advantage of the rotational ductility demand μ_θ is that it can be measured experimentally, but it depends on the loading pattern of the element that influences the calculation of the yield rotation. For the beam elements, the effect of varying axial force on the flexural yield strength is not considered and θ_y is a constant, while, for the column elements, the yield moment varies with axial force and, consequently, the yield rotation θ_y varies as well. When a member yields in anti-symmetric bending, the yield rotation is given by

$$\theta_y = \frac{M_y l}{6EI} \tag{4.6.6}$$

where l is the member length, EI is the member stiffness, and M_y is the yield moment of the member. A more general formula for the yield rotation θ_y is given by the following relationship (Kappos, 1986a)

$$\theta_{yk} = \frac{M_{yk} l}{\lambda_k EI} \quad (k = i, j) \tag{4.6.7}$$

where

$$\lambda_k = \frac{6\beta}{2\beta - 1} \quad (\beta = M_i / M_j) \tag{4.6.8}$$

and i, j are the member ends, and θ_{yk} depends on the ratio of M_i / M_j at yield.

In the case of the column elements, the rotation of a section is not sufficient to characterise by itself the severity of plastic deformations since an axial force increases, or decreases, plastic strains without affecting the rotations. This becomes more apparent in the case of column yielding due to an axial force at zero moment and, consequently, at zero rotation. In practice, the rotational ductility factor defined by Equation (4.6.5) has also been used for columns, with a 'yield rotation' corresponding to the variable yield moment, as determined from the interaction diagram at the applicable level of axial force.

Considering also the experimental evidence suggesting that rotational ductilities are better measures of flexural damage than curvature ductilities, the rotational ductility factor, despite its limitations, is the most appropriate simple index to characterise the severity of inelastic flexural deformations. However, it should be kept in mind that the available ductility of a column is drastically reduced as the axial force increases and this is one of the main reasons for which columns should be designed to remain elastic. Selection of the most appropriate ductility factor for a beam, or beam-column, element should be made on the basis of available correlation with experimentally derived failure criteria for cyclic loading. Unfortunately, despite the large amount of experimental data, such correlation is not yet well established due to the many parameters affecting the problem and due to the difficulty in defining failure for an element subjected to cyclic loading.

A simple ductility factor does not obviously account for cyclic damage effects or, in the case of RC members, strength reduction due to shear, slippage of reinforcement etc. However, other indices proposed to account for such effects have not met wide acceptance, either because their potential advantages have not been well established or due to lack of experimental substantiation. Thus, the rotational ductility factor is still the simplest and the most widely used index for characterising the severity of inelastic flexural deformations in buildings due to strong motion earthquakes.

4.7 FAILURE CRITERIA

The performance criteria implicit in most earthquake code provisions require that a structure should be able to resist a minor earthquake without damage, a moderate earthquake with minor repairable damage, and a major earthquake without collapse (see also Section 1.3.2). While no clear quantitative definition of the earthquake intensity ranges mentioned above is given, these criteria provide a general indication of what should be expected in terms of the overall performance of structures designed in accordance with the modern seismic code provisions. However, the seismic codes contain no specific requirements for the analytical prediction of collapse, and a structure is considered to be able of surviving a major earthquake when its structural elements are dimensioned and detailed in accordance to the code provisions and when the strength and stability requirements are satisfied.

The definition of collapse of a structural member, or structure, is quite subjective and depends more on engineering judgement. Kappos (1991a) suggested criteria for the analytical prediction of collapse in RC buildings subjected to earthquake excitation referring either to local (member) failure or to overall (storey) failure. These criteria give a reasonably conservative prediction of the ultimate limit state without implying that a structure would actually collapse whenever one of them is violated. The failure criteria employed in this study are presented in the following sections.

4.7.1 Global Failure Criteria

The local failure of a member does not necessarily indicate collapse of the storey, or even more of the whole structure. Thus, it is considered essential to include global and local failure criteria in the analytical procedure for assessing the seismic performance of RC buildings. In the following criteria presented, it is conservatively assumed that the failure of a single storey is equivalent to an effective overall failure of the building, although this is not always the case, especially in structures with a 'soft' first storey. The basic global failure criteria employed by most investigators include the interstorey drift, the storey stability index, and the column sidesway collapse mechanism.

(a) Interstorey Drift Ratio

The amplitude of the vibrations experienced by a building during its seismic response is restricted in order to limit both structural and non-structural damage and to guard against the structural instability. The most useful parameter for assessing these amplitudes is the interstorey drift ratio (Equation (4.6.3)), being the relative horizontal displacement of two adjacent storeys, normalised by the storey height. A limit is placed on the maximum value of interstorey drift ratio permitted in order to control P-Δ effects and the amount of structural damage.

An interstorey drift ratio equal to 1% is considered to be a conservative threshold to limit ductility demand, even though such value is usually assumed in monotonic non-linear analyses. Thus, many investigators suggested that an interstorey drift ratio of 2% can be set as the collapse limit for most RC buildings (Kappos, 1991a), and this criterion is usually combined with the formation of a column sidesway mechanism. However, even when a column sidesway mechanism has been formed, a building is assumed to have failed if the maximum interstorey drift ratio exceeds 3%. For such a value of drift ratio, all non-structural elements are severely damaged and the repair of the structure is no longer cost-effective (Kappos, 1991a).

(b) Storey Stability

At the ultimate limit state, analysis for the P-Δ effects must be carried out, unless the storey stability condition of the code is satisfied. The stability condition is usually expressed as a ratio of the interstorey displacement to the storey height, which according to the seismic codes, should not exceed a specified value (refer to Sections A.4.2, B.10.1 and C.3.8). The effect of second-order forces is estimated using the index given by all seismic codes representing the ratio of the secondary moment resulting from the storey drift to the primary moment due to seismic lateral loading

$$\theta_i = \frac{\Delta x_i W_i}{V_i h_i} \tag{4.7.1}$$

where $\Delta x_i/h_i$ is the relative interstorey drift, W_i is the total vertical force, and V_i is the total shear force at the specific storey. The total shear force corresponds to yield moments developing at the ends of the vertical members when carrying the gravity loads and it is considered as a reasonable average value for seismic loading, which changes continuously the shear values.

The second order effects need not to be considered if the interstorey stability index is less than 0.10 for the EC8 and UBC codes (Equations (B.10.1)) and (C.3.10)) or less than 0.13 for the NZS code (Equation (A.4.1)). In the UBC code, this index must never exceed 0.3 while, for index values higher than 0.1 and less than 0.2, the effects can be taken into account by increasing the seismic action by a factor equal to $1/(1-\theta)$. Moreover, design codes employ the simplification that the total inelastic displacements are the product of the force reduction factor and the displacements given by linear elastic analysis using the reduced design forces. However, this method is neither universally accurate, nor conservative, and the actual peak displacements are also a function of the frequency content of the ground motions.

(c) Column Hinging Collapse Mechanism

The condition of the interstorey drift ratio limits the allowable deformation in each storey of a structure, while the condition of stability index imposes a similar limit on deformation in terms of the structure's resistance and the gravity loading. To ensure that the strength demands imposed on a structure are not excessive, the formation of plastic hinges is also monitored. The formation of a column hinging collapse mechanism implied by the simultaneous formation of plastic hinges, at the upper and lower ends of each column within a storey, indicates failure. The occurrence of a plastic hinge is identified when the yield strain of the reinforcement is exceeded. Kappos and Tassios (1987) indicated that the column hinging collapse criterion is rather conservative because, at the time a hinge forms at a certain member end, another member may enter the unloading stage and respond with a stiffness equal to, or slightly lower than, the elastic one. Thus, the column sidesway mechanism criterion should be always combined with the interstorey drift ratio limit in order to predict collapse.

4.7.2 Local Failure Criteria

According to the capacity design philosophy employed by modern seismic codes, beams are designed and appropriately reinforced to dissipate the seismic energy while columns are provided with adequate reserve strength to avoid the formation of column sidesway mechanisms. Therefore, the development of plastic hinges in the columns during an earthquake ground motion should be avoided, in order to make sure that the seismic energy is dissipated by the beams only (Park, 1986). Due to axial compression, columns have less available ductility than beams and, for the same frame ductility expressed in terms of displacements, much larger plastic column rotations are required than beam rotations (Penelis and Kappos, 1997).

Concerning the local failure criteria, the rotational ductility factor (Equations (4.6.4) and (4.6.5)) is the simplest and the most widely used index for characterising the severity of inelastic flexural deformations in structures subjected to earthquake ground motions. Therefore, it is important to establish the relationship between the displacement ductility of the structure (global ductility) with the rotational ductility factor of the structural elements (local ductility) since the displacement ductility factor is related to the design ductility factors of the seismic codes. If a structure undergoes a beam sidesway mechanism (with hinges at the ends of the beams and the lower part of the base columns), then the displacement profile is nearly linear over the height of the structure. In this case, the global displacement ductility of the structure is identical with the interstorey displacement ductility of each storey.

However, when a column sidesway mechanism forms in one storey, the displacement ductility of the structure is not the same with the displacement ductility of the specific storey. In this case, the storey displacement ductility is calculated and it is checked whether it exceeds the value of the ductility factor employed for the design of the structure. Thus, the rotational ductility demand of the structural elements can be also related to the design ductility of the codes. A local ductility demand higher than the permitted value does not necessarily imply collapse. The local collapse criteria should be always checked in relation to the global collapse criteria, which concern the maximum interstorey drift, the storey stability index, and the formation of a column sidesway mechanism.

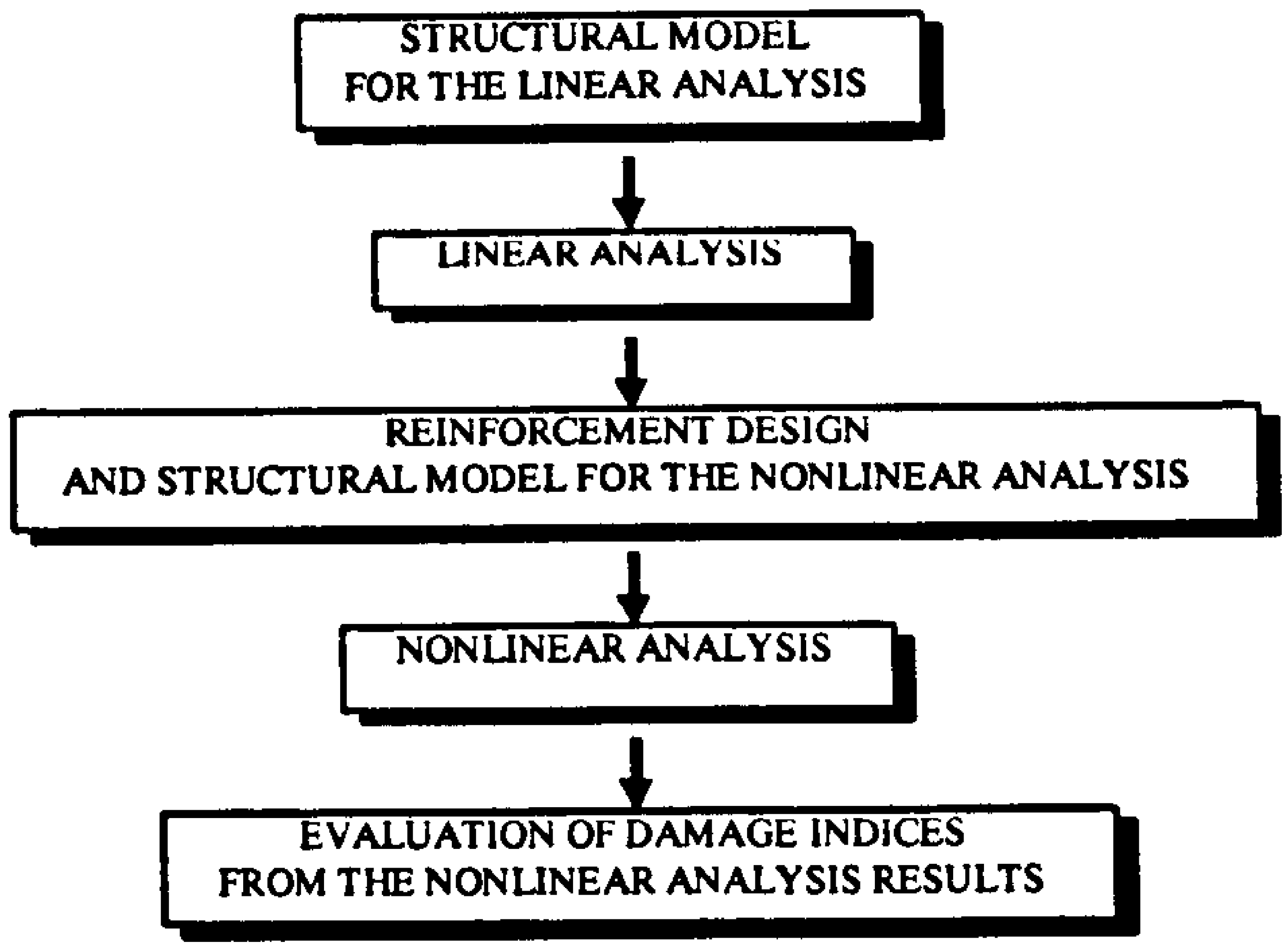


Figure 4.1.1 Flow-chart regarding the design analysis

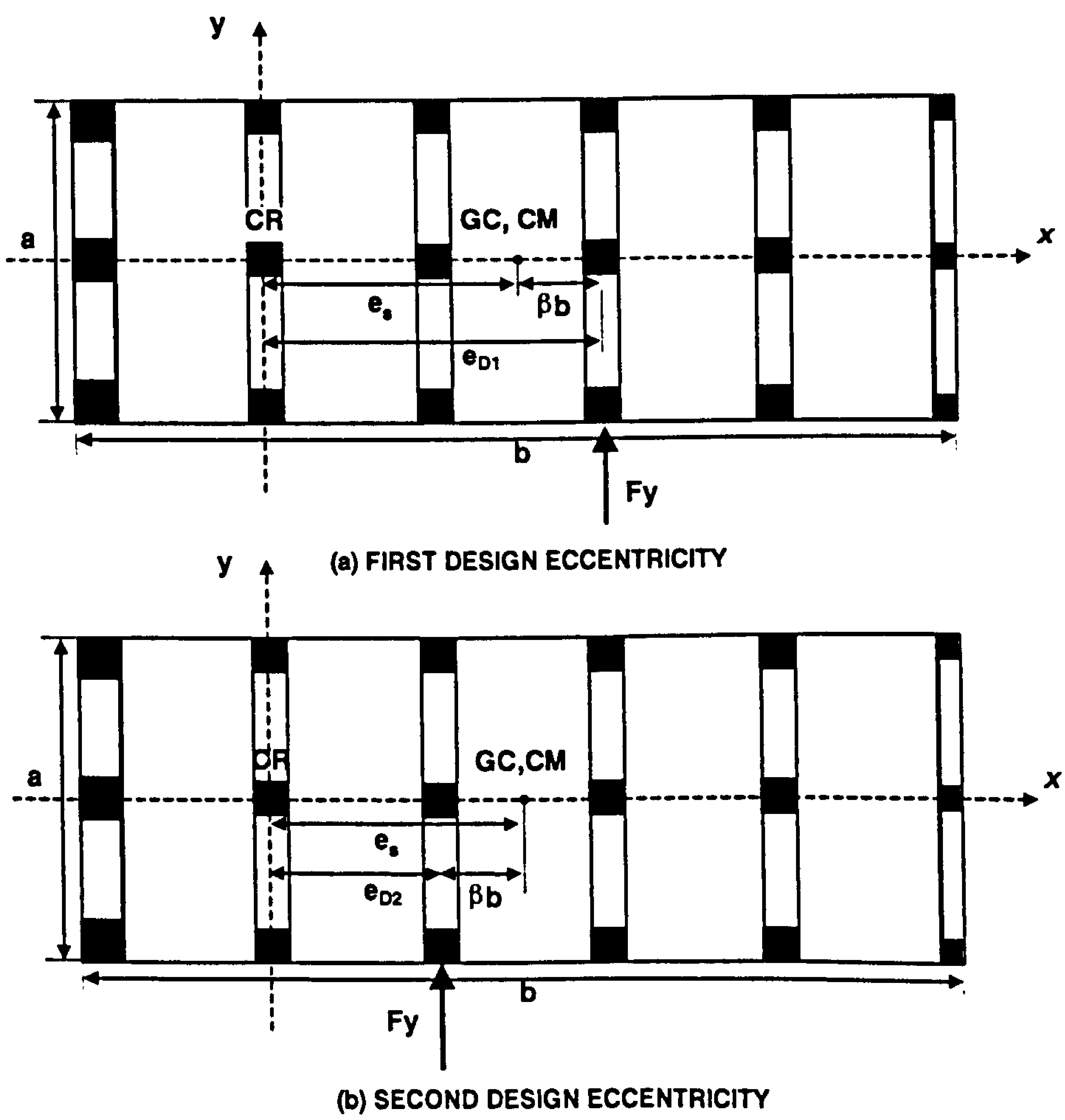


Figure 4.2.1 Definition of the design eccentricities.

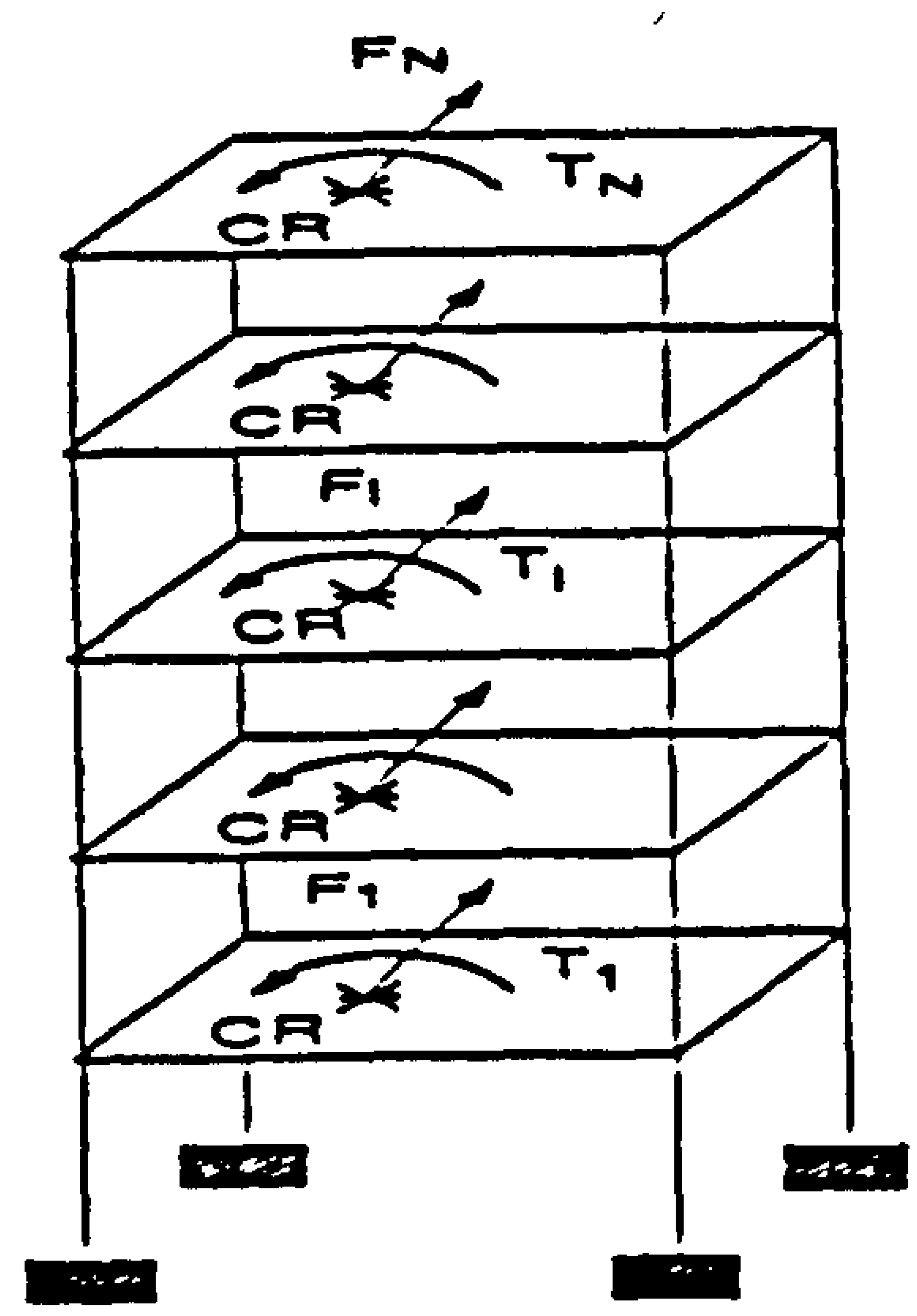


Figure 4.2.2 Floor torques and lateral forces acting at floor levels (Duan, 1991).

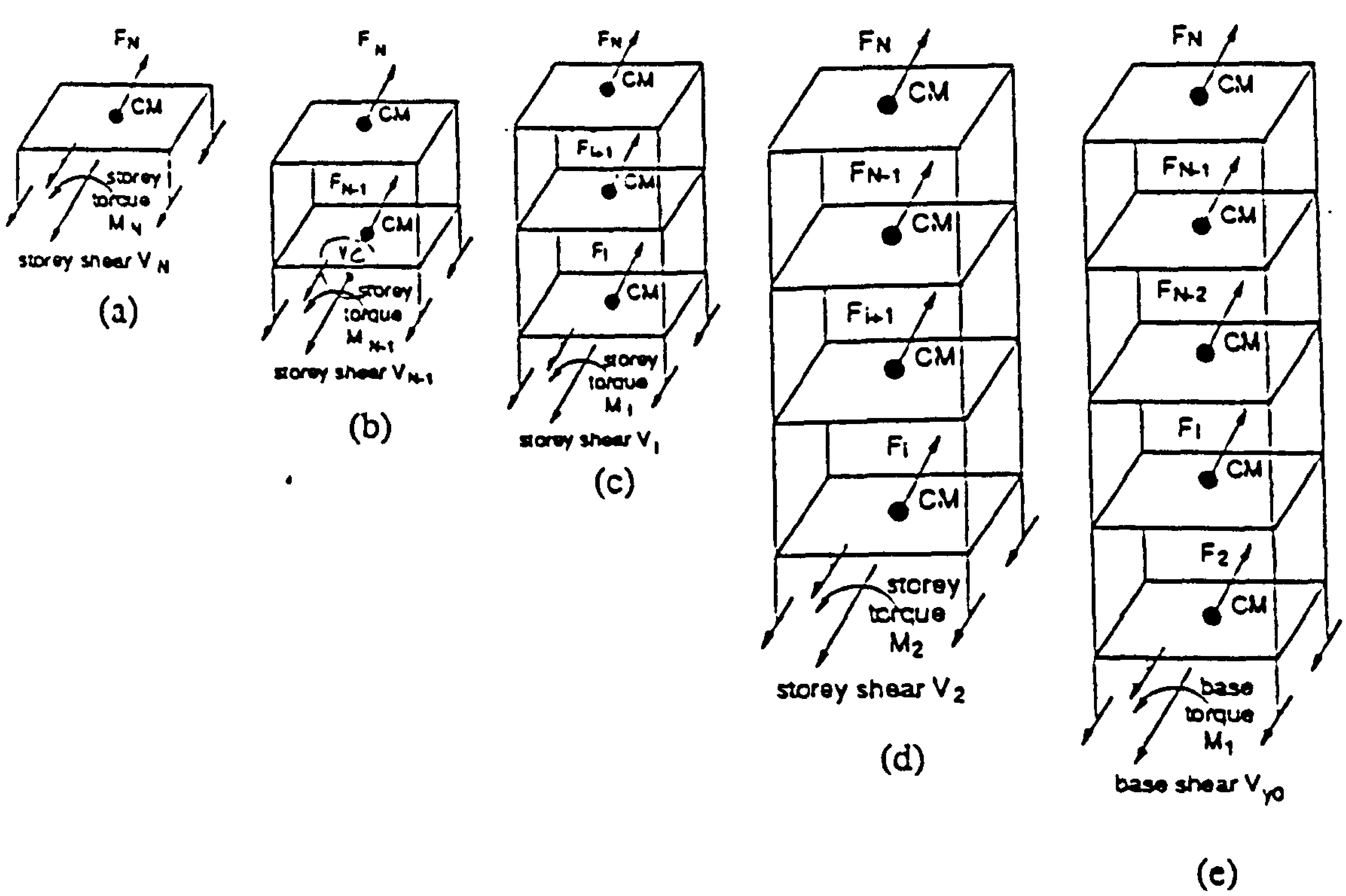


Figure 4.2.3 Storey shears and storey torques (Duan, 1991).

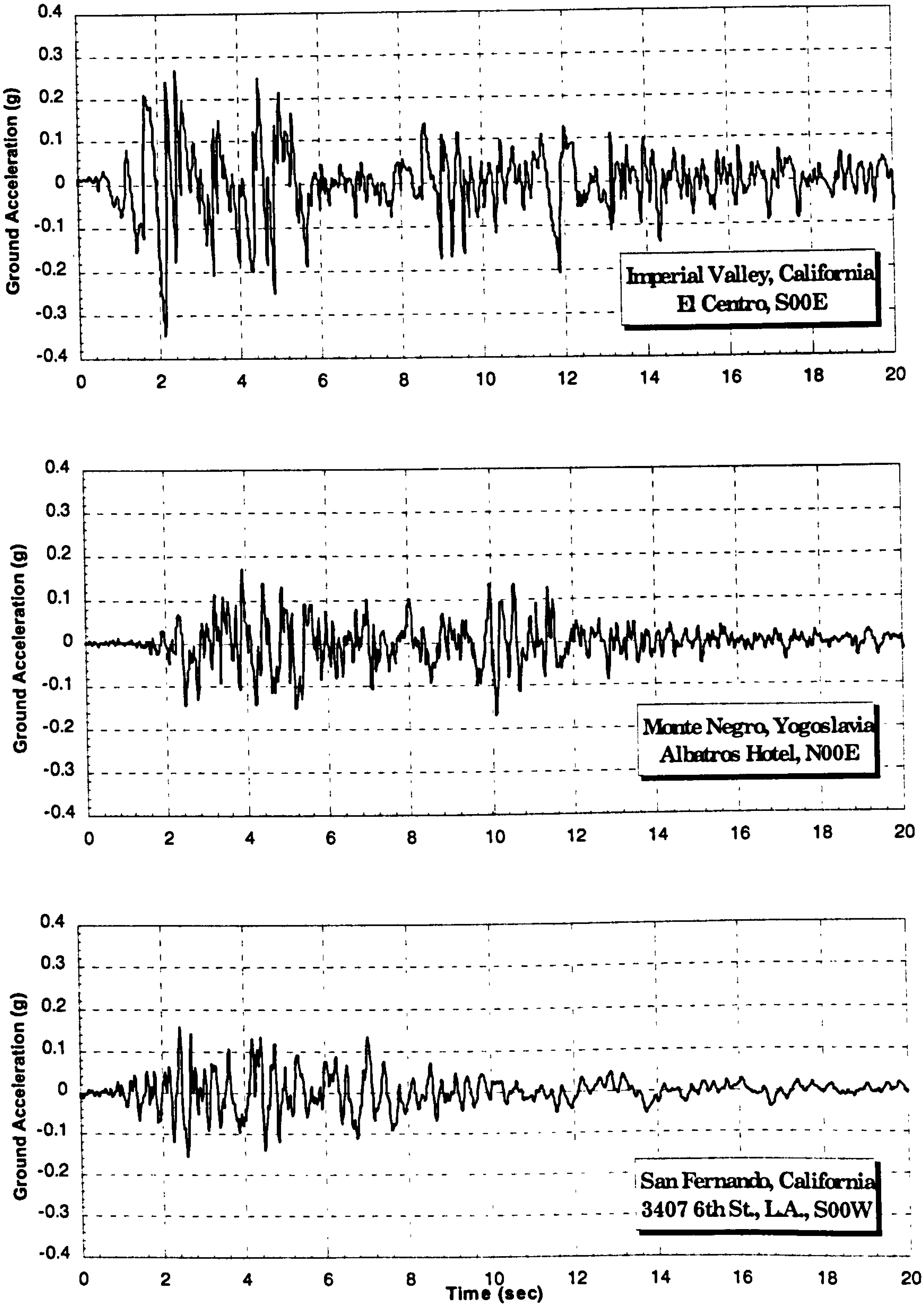


Figure 4.4.1 Time-history acceleration of the lateral component of the earthquake records selected.

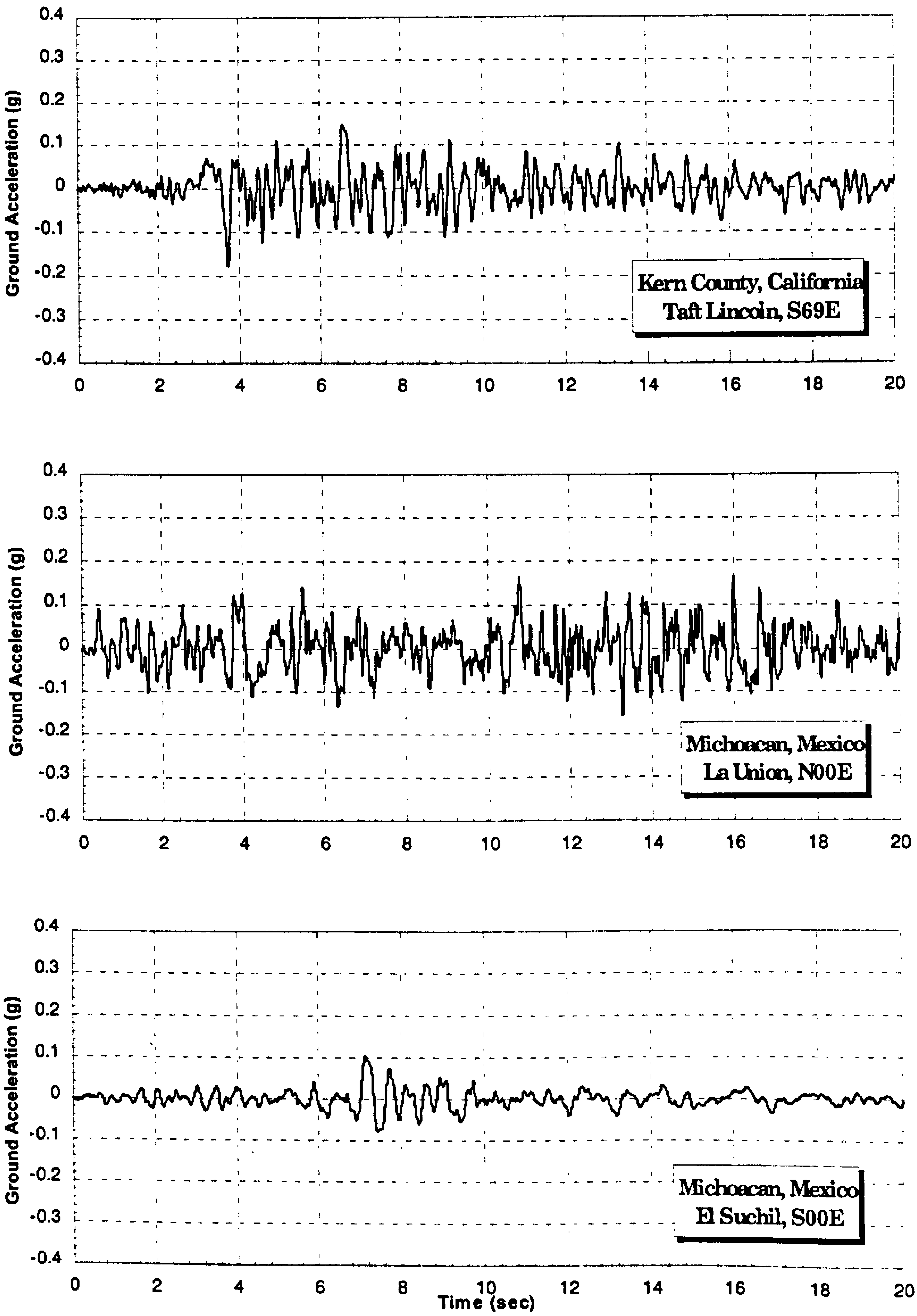


Figure 4.4.2 Time-history acceleration of the lateral component of the earthquake records selected.

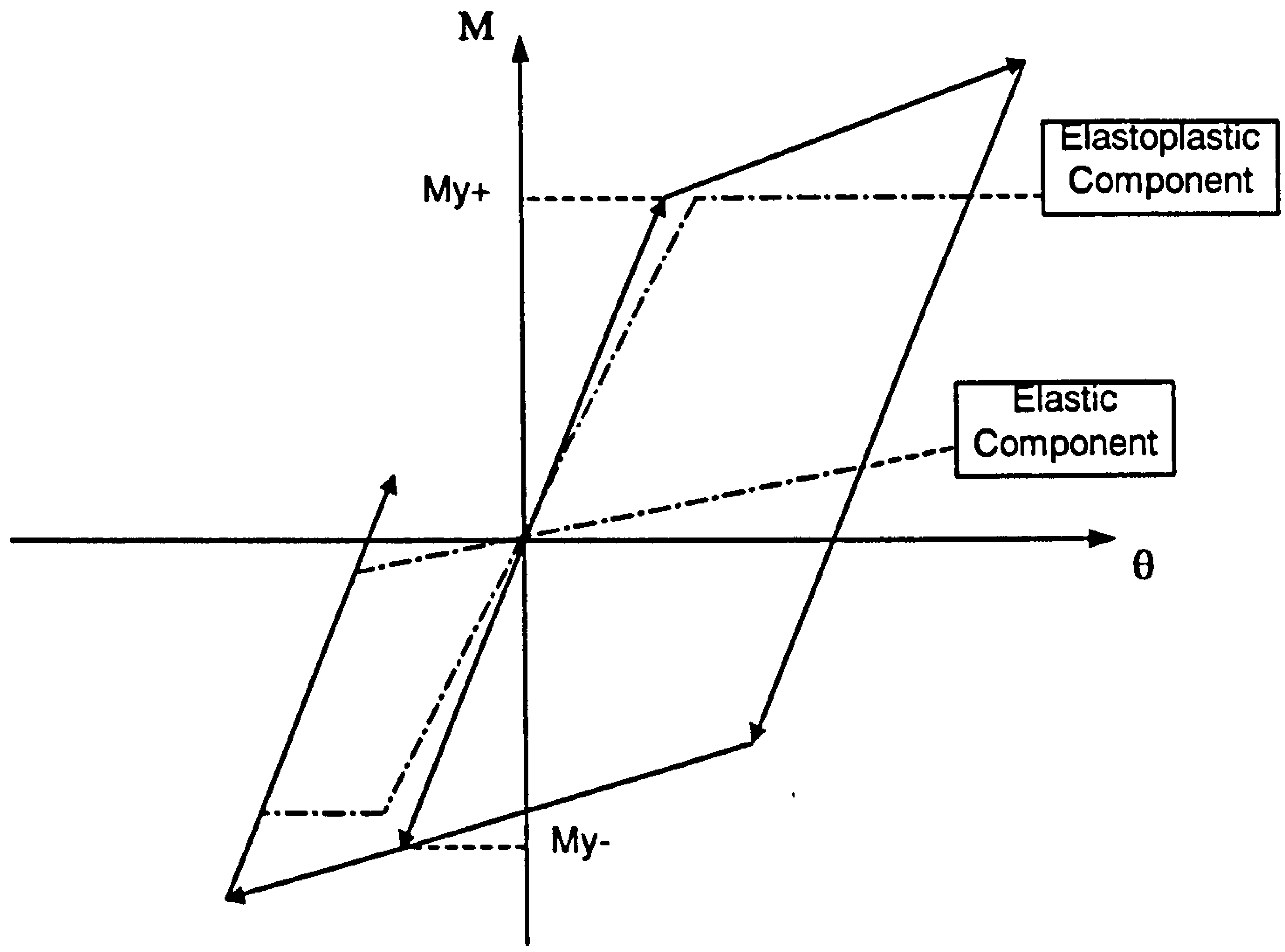


Figure 4.5.1 Moment – rotation relationship for the elastoplastic beam-column element (element type 2).

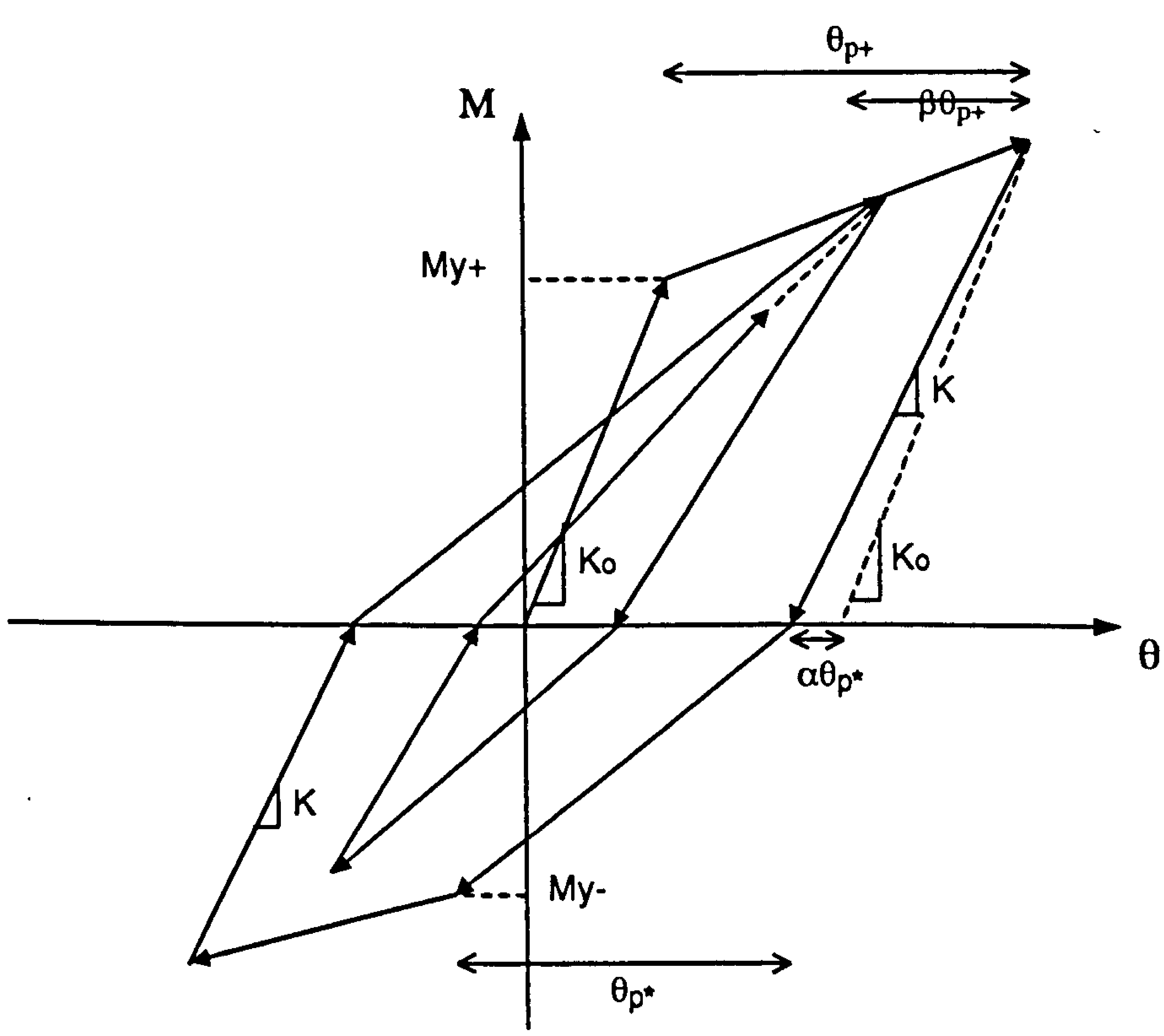


Figure 4.5.2 Moment – rotation relationship for the stiffness degrading beam element (element type 6).

CHAPTER 5

DESIGN AND INELASTIC SEISMIC RESPONSE OF THE REFERENCE MODELS

5.1 INTRODUCTION

From previous investigations and the discussion presented in the literature review of Chapter 2, it has been apparent that the conclusions derived from the results of studies investigating the inelastic torsional effects are highly dependent on the definition and selection of the reference models. Therefore, the lack of uniformity in the results and conclusions of different torsional studies is mainly due to the lack of uniformity in the definition of the reference models adopted. The aim of this chapter is to present the inelastic seismic response of different types of reference models and to investigate the factors influencing their inelastic dynamic behaviour.

The reference models are needed to investigate the critical effects of inelastic torsional response in parametrically defined torsional systems by comparing their behaviour to the behaviour of the torsionally unbalanced (TU) models. Both symmetric (SM) and torsionally balanced (TB) reference models are examined (as also seen in Section 3.3) and their inelastic response is analysed and compared. Furthermore, it is demonstrated that the model configuration and the structural design procedure of the reference models influence very significantly their inelastic response and, consequently, alter the conclusions concerning the inelastic torsional behaviour of the TU models. A clear understanding of the inelastic seismic response of the reference systems leads to a better understanding of the inelastic response of the TU structures, and the factors influencing the TB models affect the dynamic behaviour of the TU models as well.

In Section 5.2, the configuration of the reference models employed is presented while the influence of the seismic code provisions is investigated in Section 5.3 by presenting the regularity requirements, the methods of analysis allowed to be used, and the torsional design provisions. Section 5.4 deals with the influence of various factors on the inelastic dynamic behaviour of the reference models and the effect of the structural design provisions on their inelastic response is one of the main factors analysed. The minimum steel ratios implied by the codes for the design of the structural elements change the strength distribution of the frames and, consequently, alter the total strength of the structure. The importance of a strength distribution proportional to the stiffness distribution of the frames and its influence in the inelastic dynamic response of a structure is another significant issue examined. The factors investigated influencing the strength distribution of the models are the vertical uniform load distribution, the minimum reinforcement requirements of the codes, and the accidental eccentricity provisions. In Section 5.5, the inelastic response of SM models is compared with the response of similar TB models, the influence of the number of storeys is examined, and the torsional design provisions of different seismic codes are compared. The conclusions of the investigations carried out in this chapter are summarised in Section 5.6.

5.2 THE REFERENCE MODELS ADOPTED

In order to access the effect that torsion induces on a structure, a reference model is usually employed and its inelastic response is compared with the response of the TU model. The reference model has the same fundamental properties as the corresponding TU model, such as structural period, total lateral stiffness and total torsional stiffness (see also Section 3.3). The choice of the reference model depends on the purpose of the study and on the configuration of the asymmetric structures examined. Typically, a reference model incorporates no static eccentricity such that the model is stiffness and mass proportional and the centre of stiffness is coincident with the centre of mass. On the other hand, some researchers believe that the actual response of a reference model is critical in evaluating the performance of the code-designed TU systems, and thus, the reference system should be designed with respect to the seismic code provisions. The variations in the definition and configuration of the reference models are numerous and they result in varying structural responses and contrasting conclusions from apparently similar investigations.

The strength distribution of the reference systems can be determined in two ways. In the first approach, the accidental eccentricity component is not incorporated in their design and the strength distribution of the frames is proportional to their stiffness distribution. In the second approach, the element design strength distribution is determined by employing the design eccentricities of the seismic codes and, as a result, the strength distribution of the reference models is not proportional to the stiffness distribution.

From the above two approaches of strength distribution, it is apparent that there is no effective difference between SM and TB models, provided the accidental eccentricity is not considered and the models are appropriately designed (Section 5.4.3). Furthermore, if a reference system is defined for a behavioural type of study, the fundamental objective is to further understand the inelastic behaviour of the TU systems. However, for a code-based (code-evaluation) investigation, the aim is to examine the response of TU structures designed to the static torsional provisions and to assess the performance and adequacy of the torsional seismic code requirements. Distributing the strength of the reference models by incorporating, or not, the accidental provisions is one of the main issues investigated (Section 5.4.3).

Three different types of reference models are employed in this study: the SM model S (Figure 3.4.1) and the TB models A and B (Figures 3.4.2 and 3.4.3). In each category of these three reference models, structures with 6, 12 and 24 floor levels are included corresponding to respective TU systems. All models are designed based on the torsional seismic provisions of the EC8, NZS and UBC codes and the influence of their design provisions is investigated. Finally, reference models including transverse elements are also examined and their inelastic response is compared with the response of the reference models consisting of structural elements located only in one direction, parallel to the seismic loading (refer to Chapter 8).

5.3 INFLUENCE OF THE SEISMIC CODE PROVISIONS

5.3.1 Regularity Requirements and Method of Analysis

The equivalent static method of analysis is a simple procedure applied in regular buildings and it requires little computational effort (Section 4.2.2). The buildings analysed by this method are not expected to have any essential contribution from higher modes of vibration. Each seismic code applies specific regularity criteria that have to be satisfied in order that the static method of analysis and the static torsional provisions may be employed. The regularity criteria of each seismic code employed in this study are presented analytically in the relevant Appendices (Sections A.2, B.1 and C.2). The reference models have no static eccentricity and respond purely in translation in both the elastic and inelastic ranges. Thus, the regularity requirements are obviously satisfied, and the static method of analysis may be employed (Sections A.5, B.3 and C.3).

5.3.2 Base Shear and Equivalent Static Lateral Forces

Irrespective of the stiffness distribution and the static eccentricity of each model investigated, all models having the same height are designed for identical equivalent static forces since they also have identical fundamental structural properties. The formulas for the base shear calculation and the distribution of the design lateral forces are analytically presented in the relevant Appendices (refer to Sections A.5, B.3 and C.3). The results from the base shear calculation and the distribution of the equivalent forces over the height of the models are presented in Tables 5.3.1 – 5.3.3. The equivalent horizontal forces of the models are calculated for an effective peak ground acceleration normalised by the acceleration of gravity equal to 0.30 while the soil is considered stiff. All models belong to the ordinary structure category and they are designed for the highest ductility factor for RC frames of each seismic code. For the UBC seismic code, the 1.4 factor applied to the seismic forces is already included in the base shear calculation.

BASE SHEAR CALCULATION FOR THE 6-STOREY MODELS			
LEVEL	**EC8**	**NZS**	**UBC**
1	83.04	63.40	44.75
2	166.08	126.80	89.50
3	249.11	190.20	134.25
4	332.15	253.60	179.00
5	415.19	317.01	223.74
6	498.23	496.18	268.49
Total	**1743.79**	**1447.20**	**939.73**

Table 5.3.1 Equivalent horizontal forces and base shear calculation for the 6-storey models (kN).

BASE SHEAR CALCULATION FOR THE 12-STOREY MODELS			
LEVEL	**EC8**	**NZS**	**UBC**
1	25.44	12.72	12.60
2	50.88	25.43	25.19
3	76.33	38.15	37.79
4	101.77	50.87	50.39
5	127.21	63.58	62.99
6	152.65	76.30	75.58
7	178.10	89.02	88.18
8	203.54	101.73	100.78
9	228.98	114.45	113.38
10	254.42	127.17	125.97
11	279.86	139.88	138.57
12	305.31	238.85	238.01
Total	**1984.49**	**1078.16**	**1069.44**

Table 5.3.2 Equivalent horizontal forces and base shear calculation for the 12-storey models (kN).

BASE SHEAR CALCULATION FOR THE 24-STOREY MODELS			
LEVEL	**EC8**	**NZS**	**UBC**
1	10.80	4.97	4.90
2	21.60	9.94	9.79
3	32.40	14.90	14.69
4	43.20	19.87	19.58
5	54.00	24.84	24.48
6	64.80	29.81	29.38
7	75.60	34.78	34.27
8	86.40	39.74	39.17
9	97.20	44.71	44.06
10	108.00	49.68	48.96
11	118.80	54.65	53.86
12	129.60	59.62	58.75
13	140.40	64.58	63.65
14	151.20	69.55	68.54
15	162.00	74.52	73.44
16	172.80	79.49	78.34
17	183.60	84.46	83.23
18	194.40	89.42	88.13
19	205.20	94.39	93.02
20	216.00	99.36	97.92
21	226.80	104.33	102.82
22	237.60	109.3	107.71
23	248.40	114.26	112.61
24	259.20	248.83	349.69
Total	**3240.00**	**1620.00**	**1701.00**

Table 5.3.3 Equivalent horizontal forces and base shear calculation for the 24-storey models (kN).

5.3.3 Accidental Eccentricity and Torsional Effects

After calculating the base shear of a model and having determined the vertical distribution of the horizontal seismic forces, the design storey shear of each floor can be calculated by simply summing up the lateral forces acting at floor levels above the storey under consideration. The design storey shear is distributed to the various elements of the vertical lateral force-resisting system in proportion to their rigidities. To account for the uncertainties in the locations of loads, the mass at each level is assumed to be displaced from the calculated centre of mass in each direction by a distance specified in the seismic codes.

As seen in Section 4.2.2, codes specify two design eccentricities to account for the increased, or decreased, strength demand in certain elements. Usually for the reference models, the design storey shears are applied at CM and the accidental eccentricity requirements of the seismic code are not taken into account. In this chapter, where the response of the reference models is examined and the factors influencing their inelastic response are investigated, the influence of the accidental eccentricity is also studied. The design eccentricities of the three seismic codes employed were presented in Table 4.2.1.

5.3.4 Interstorey Drift Limitation and P-Delta Effects

Storey drift is the relative displacement between consecutive floor levels produced by the design lateral forces and includes calculated translational and torsional deflections. All three seismic codes employed in this study impose an interstorey drift limitation (see also Sections A.4.1, B10.2 and C.3.7) and the calculated storey drift must not exceed specific limit values. The P-delta effects refer to the additional moment produced by the vertical loads and the lateral displacements of columns or other resisting elements. These second-order effects need not to be considered when the ratio of the secondary moment resulting from the storey drift to the primary moment due to the seismic forces does not exceed 0.1 for any floor level (refer also to Sections A.4.2, B.10.1 and C.3.8).

5.4 FACTORS INFLUENCING THE STRENGTH DISTRIBUTION OF THE REFERENCE MODELS

The aim of this section is to investigate the factors that influence the strength distribution of the reference models and, consequently, affect their inelastic behaviour. The strength distribution of the reference models should ideally be proportional to their stiffness distribution. When a model is stiffness and strength proportional (SSP), the centre of strength (CP) of the structural elements coincides with their centre of rigidity (CR) and, as a result, the model responds purely in translation with identical inelastic seismic response for all the frames. The importance of the strength distribution in the response of the reference models is more pronounced in TB models, where CR is not located at GC due to the stiffness distribution of the frames. Thus, a strength distribution that is not proportional to the stiffness distribution shifts CP from CR causing torsional response and non-uniform damage distribution of the frames.

The most important factor influencing the strength distribution of the models is the inclusion of the minimum reinforcement requirements imposed by the codes for the design of the structural elements (Section 5.4.1). A design method producing an SSP reference model is tested, and the importance of a strength distribution proportional to the stiffness distribution is proved, especially for the inelastic seismic response of the TB reference models (Section 5.4.1). Additionally, the distribution of the vertical uniform load (Section 5.4.2) and the inclusion of the accidental eccentricity provisions (Section 5.4.3) are also examined in this chapter since they influence the strength distribution of the structural elements.

In the figures presenting the results of the parametric analysis carried out in all the chapters of this thesis, information regarding the structural model analysed is given in each graph. The typical information included concerns the model analysed, the seismic code adopted, the earthquake record applied and the inclusion, or not, of the accidental eccentricity provisions and of the minimum steel ratios. Thus, for example, "UBC - No ea - No min" means that the design seismic forces are calculated according to the UBC seismic code regulations, without the inclusion of the accidental eccentricity provisions ("No ea") and that the strength of the elements is calculated without the minimum reinforcement requirements ("No min").

Furthermore, in the figures presenting the maximum rotational ductility demand of the structural elements versus the floor levels, the ductility demand value represents the maximum value per floor level. Thus, the column ductility demand value demonstrated represents the maximum value of the top and bottom sections of the three columns in each floor, and the beam rotational ductility demand is the maximum value of the edge sections of the two beams. In the graphs presenting the column rotational ductility demand of each floor level, the value of the ground floor (first floor level) does not include the rotational ductility demand of the bottom sections of the first storey columns. This is due to the fact that the formation of plastic hinges in the bottom sections of these columns is permitted by all seismic codes and their plastic hinge rotations are always much higher than the rotations of the other columns.

5.4.1 Minimum Reinforcement Provisions

Based on the results of the elastic static analyses, the models are designed and the amount of reinforcement needed for the structural elements is calculated using the structural design provisions presented in Section 4.3. These provisions influence the strength distribution of the frames and change the inelastic response of the models. Especially in the case of column elements, their strength distribution changes drastically due to the high amount of minimum reinforcement ratio required (equal to 1%). From the results of the elastic static analysis, the strength distribution of both columns and beams is proportional to their stiffness distribution while by including the minimum reinforcement requirements, their strength distribution changes and it is no longer proportional to the stiffness distribution. Three different design procedures are tested in this section in order to quantify the influence of the minimum steel ratios and prove the importance of a stiffness and strength proportional (SSP) model.

Dolce and Ludovici (1992) studied the influence of the minimum reinforcement provisions on the inelastic torsional response of asymmetric structures by carrying out two different design analyses. In the first analysis, the minimum reinforcement ratio for the columns was equal to 1% and, in the second analysis, a reduced minimum reinforcement ratio was incorporated, equal to 0.3%. The strength of the column elements was strongly modified by the minimum reinforcement requirements, and even the 0.3% steel ratio controlled the amount of reinforcement needed in a few column sections. The minimum steel requirements attenuated the differences between the inelastic response of mass-eccentric and symmetric structures. Thus, the need to investigate different design procedures is justified by the fact that the minimum steel ratios over-design the element cross-sections and alter drastically the strength distribution of the frames and their inelastic behaviour.

Three different design procedures are investigated in this part of the study. In the first design procedure, the minimum steel ratios for both columns and beams are not included while the tensile reinforcement of the beams is checked to be at least half the compressive reinforcement. This provision is incorporated in the design of the beams because, in many cases, their tensile moment is found to be zero from the results of the elastic analysis. The capacity check is also incorporated in order to increase the strength of the columns relevant to the strength of the beams while their strength distribution still remains proportional to their stiffness distribution. In the second design procedure, the minimum compressive reinforcement ratios of the beams are included while the minimum steel ratios of the columns are not incorporated. The capacity check increases the strength of the columns, and the minimum tensile reinforcement provision for the beams is also included. This design method alters the strength distribution of the models and results in a strength distribution non-proportional to the stiffness distribution. Finally, the third design method includes the minimum steel ratios for both columns and beams and, therefore, the strength of the columns is considerably increased. In this case, the capacity check doesn't significantly influence the strength of the columns due to the large amount of steel already induced by the minimum reinforcement provisions.

The importance of the strength distribution in the inelastic response of the reference models is investigated by examining the TB model 6B designed according to all design methods presented above. The response of model 6B is compared with the response of the other two reference models with the same number of storeys, the SM model 6S and the TB model 6A. Model 6B is selected because it is simple and, at the same time, its response represents the response of all stiffness-eccentric models of any height. The accidental eccentricity is not included in the elastic analysis carried out in this section, because this factor would change the strength distribution of the frames and, consequently, it would further complicate the inelastic response of the models. The design seismic forces are calculated based on the UBC code regulations since this code is less conservative than the other two codes examined and its design forces are the smallest ones.

Frame	BEAMS			COLUMNS			
	Not modified	Method 1	Methods 2 & 3	Not modified	Method 1	Method 2	Method 3
1	16.0	16.0	9.4	16.0	16.0	9.4	9.0
2	2.0	2.0	1.7	2.0	2.0	1.7	1.7
3	2.0	2.0	1.7	2.0	2.0	1.7	1.7
4	2.0	2.0	1.7	2.0	2.0	1.7	1.7
5	2.0	2.0	1.7	2.0	2.0	1.7	1.7
6	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Table 5.4.1 Strength distribution for the frames of model 6B designed to different methods and normalised to the strength of frame 6.

Frame	BEAMS (-)		BEAMS (+)		COLUMNS		
	Method 1	Methods 2 & 3	Method 1	Method 2 & 3	Method 1	Method 2	Method 3
1	1.0	1.2	5.9	6.5	2.0	2.2	2.3
2	1.0	1.7	6.5	10.2	2.0	3.2	3.6
3	1.0	1.7	6.8	10.8	2.0	3.3	3.7
4	1.0	1.7	7.2	11.5	2.0	3.4	3.8
5	1.0	1.7	7.6	12.3	2.0	3.5	3.9
6	1.0	2.0	7.8	14.7	2.0	4.0	4.5

Table 5.4.2 Strength increase for the frames of model 6B designed to different methods and normalised to the strength found from the elastic analysis.

The strength distribution of the frames of model 6B is presented in Table 5.4.1, and the influence of each design procedure is demonstrated. In this table, the total strength of each frame is presented as a ratio of the total strength of frame 6, which is the frame with the lowest stiffness and strength. In model 6B, the stiffness of frame 1 is 16 times the stiffness of frame 6, and the stiffness of frames 2-5 is twice the stiffness of frame 6. As a result, the stiffness ratio of frame 1 is 16 and the stiffness ratio of frames 2-5 is 2, when normalised to the stiffness of frame 6. Therefore, the strength ratios of model 6B produced from the results of the elastic analysis ("Not modified") are equal to the stiffness ratios of the frames (Table 5.4.1). For the first design procedure, the strength distribution of both columns and beams remains proportional to their stiffness distribution, because the minimum steel ratios are not included in the design of the models. For the other two methods, the incorporation of the minimum steel ratios changes the strength distribution of the model examined.

Table 5.4.2 presents the increase of strength that each design method induces in the structural elements of model 6B. The total strength of each frame designed to each procedure is normalised to the strength required for the same frame from the results of the elastic analysis. The minimum steel ratios increase the positive beam moments considerably due to the fact that, in many floor levels, the positive beam moments are close to zero. The column strength increase caused by the inclusion of the minimum reinforcement requirements is the highest (method 3) and it is higher than the increase that only the capacity design induces in the columns (method 2).

The influence of the strength distribution in the inelastic seismic response of a TB model is demonstrated by the results of the inelastic analyses of model 6B designed to the three design procedures. Figures 5.4.1 and 5.4.2 indicate that the first design method produces a strength distribution proportional to the stiffness distribution and results in the same inelastic response for all frames. Contrary to that, methods 2 and 3 result in a non-proportional strength distribution and produce a different inelastic response with each frame responding differently. Similar to the response of model 6B, when models 6S and 6A are designed according to methods 2 and 3, the inelastic response of their frames varies significantly. Moreover, when the first design method is applied and the strength distribution of all the reference models is proportional to their stiffness distribution, their response is identical. Thus, when the first design method is employed, the response of all the reference models with the same number of levels is the same, indicating the importance of employing the first design method for the investigation of their inelastic behaviour.

The above remark that different types of reference models with the same height and identical fundamental properties can respond in the same way is very important. The first design method and, hence, a strength distribution proportional to the stiffness distribution result in a uniform ductility demand and damage distribution for different reference models. This is very important for the discussion regarding the differences between TB and SM models and their applicability for the investigation of the torsional behaviour of the TU models. A design method without minimum steel ratios proves that the response of a SM and of a TB model may be the same when they are carefully designed. Thus, the differences in the conclusions of previous studies, which employed different types of reference models, are due to the varying strength distribution of the reference models rather than their model type (TB or SM models).

From the above investigation, the need to employ two different design methods for the inelastic analyses of this study is justified. The design methods that will be used in the following sections are the first design method that produces a proportional strength distribution and the third design method that is consistent with the code provisions. These two design methods will be termed in the following sections first and second design method, respectively. The first method aims to develop a better understanding of the response of the reference models while the second method examines the influence and the efficiency of the design provisions (capacity design and minimum reinforcement ratios) of the structural codes.

5.4.2 Vertical Load Distribution

Another factor affecting the strength distribution of a model is the distribution of the vertical (gravity) uniformly distributed load since it influences the moment distribution of the structural elements. In order to investigate the influence of the vertical load distribution on the lateral strength distribution of the frames, two different cases of vertical load distribution are tested. The first load case is a vertical load distribution proportional to the stiffness distribution of the frames. The ratio of the vertical load applied on each frame is equal to the stiffness ratio of that frame relative to the flexible frame 6. In the second load case, a uniform load distribution is examined while the total vertical load applied on each floor is the same for both cases.

Table 5.4.3 displays the strength distribution of model 6B analysed for both vertical load distributions. The first column of each load case presents the unmodified strength distribution from the results of the elastic analysis. The second column illustrates the strength distribution of the structural elements designed to the first design method, i.e. without the minimum reinforcement requirements. Finally, the third column presents the strength distribution of the model after designing the structural elements to the second method, i.e. with the minimum steel ratios.

STRENGTH DISTRIBUTION OF MODEL 6B						
Frame	**Proportional Vertical Load**			**Uniform Vertical Load**		
	Not modified	**Design 1**	**Design 2**	**Not modified**	**Design 1**	**Design 2**
1	16.0	16.0	9.0	6.0	6.0	7.3
2	2.0	2.0	1.7	1.2	1.2	1.4
3	2.0	2.0	1.7	1.2	1.2	1.4
4	2.0	2.0	1.7	1.2	1.2	1.4
5	2.0	2.0	1.7	1.2	1.2	1.4
6	1.0	1.0	1.0	1.0	1.0	1.0

Table 5.4.3 Strength distribution for model 6B analysed for two different vertical load distributions and normalised to the strength of frame 6.

The importance of a proportional strength distribution in the inelastic response of the reference models was indicated in Section 5.4.1, and the vertical load distribution that results in a moment distribution proportional to the stiffness distribution is the proportional vertical load case. The second design method, which includes the minimum steel ratios of the codes, changes the strength distribution of the frames for both vertical load cases. Figure 5.4.3 shows the time-history displacement of the top storey of frames 1 and 6 of model 6B when designed according to the first design procedure for both vertical load cases. The displacement of frame 1 is identical to the displacement of frame 6 only for the proportional vertical load case. The time-history displacements of frames 1 and 6 differ when a uniform vertical load is applied due to the non-proportional strength distribution of the model (Table 5.4.3). Thus, the vertical load distribution should be proportional to the stiffness distribution of each reference model and, especially, in stiffness-eccentric (TB) reference models where shifting the centre of strength away from the centre of rigidity would result in a torsional response. Consequently, for all the models examined, their vertical load distribution has been taken to be proportional to their stiffness distribution, although this is not a realistic vertical load distribution.

5.4.3 Accidental Eccentricity Provisions

A reference model may be designed with or without the code provisions for the effect of torsion. Usually the reference models incorporate no static eccentricity such that their stiffness and mass distributions are identical, and their centre of stiffness is coincident with their centre of mass. Since the effect of the accidental eccentricity on strength is intended to account for factors that by their nature cannot be included in the analysis, some investigators do not include the accidental eccentricity (Chandler and Duan, 1991b). On the other hand, some others believe that a reference system should be designed with respect to the seismic code provisions, and the accidental eccentricity should be included. Moreover, the approach of the current UBC provisions to redress the effects of large torsional flexibility is formulated through an increase of the accidental eccentricity and, therefore, it is not appropriate to totally exclude it (Correnza et al., 1995). Finally, depending on whether a reference system is defined for a behavioural type of study, or a code-based investigation, the accidental eccentricity is excluded, or included, respectively. In the first case, the fundamental objective is to understand further the behaviour of the TU systems while, in the second case, the aim is to assess the performance and adequacy of the torsional provisions of the codes.

The effect of distributing the strength of a reference model by incorporating, or not, the accidental torsional provisions is examined in this section by determining the strength distribution of model 6B in two ways. In the first approach, the strength distribution of the model is taken to be proportional to its stiffness distribution and the elastic analysis is carried out without the accidental eccentricity. In the second approach, strictly adhering to the seismic code provisions, the element design strength is determined by employing the design eccentricities. When the accidental eccentricity is included in the elastic static analysis of a model, its strength distribution is not proportional to its stiffness distribution, irrespective of the design method adopted for the strength calculation of the structural elements (Section 5.4.1).

Therefore, the inelastic behaviour of model 6B designed according to the accidental eccentricity provisions of the UBC code is no longer the same as the response illustrated in Figures 5.4.1 and 5.4.2. When the accidental eccentricity component is included in the design of the model, each frame responds in a different way (Figure 5.4.4) and its inelastic response is reduced depending on its strength increase due to the accidental eccentricity. The strength of frame 2, located at the centre of rigidity, is not modified by the accidental eccentricity while the strength increase of the rest of the frames depends on the distance of each frame from the location of the design seismic forces. Thus, the strength increase of frames 1 and 3 is the same while the strength increase of frame 6 is the highest since it is located the furthest away from the seismic forces. The influence of the accidental eccentricity component is further investigated and discussed in Section 5.5.3, where the seismic code provisions of different seismic codes are examined.

5.5 INELASTIC RESPONSE OF THE REFERENCE MODELS

5.5.1 Different Types of Reference Models

The inelastic seismic response of the reference models is highly dependent on the structural configuration of the models and on their strength distribution. In Section 5.4, the factors influencing the strength distribution of the reference models were investigated while, in this section, the inelastic response of different types of reference models is examined. The reference models employed in this study are: the symmetric model S and the torsionally balanced models A and B, each one consisting of three different numbers of floor levels (6, 12 and 24 storeys). The influence of the configuration of each reference model is investigated in this section by examining the inelastic response of models 24S, 24A and 24B designed according to both design procedures adopted in this study (see also Section 5.4.1).

The first design method excludes the minimum reinforcement provisions for the design of the structural elements while the second method is consistent with the design provisions of the structural codes. The influence of the design method adopted is demonstrated once more and, when models 24S, 24A and 24B are designed according to the first design procedure and without the inclusion of the accidental eccentricity, the response of all their frames is identical (Figure 5.5.1). Consequently, there is no distinction in the inelastic response of different reference models having the same height, identical structural properties, and a SSP stiffness distribution.

When the second design procedure is employed for the design of the structural elements, the strength distribution of the models is considerably different, and the inelastic response of each reference model is no longer the same. The columns of the 24-storey reference models are over-designed due to the inclusion of the minimum reinforcement requirements and, apart from the bottom sections of the first-storey columns, no other plastic hinges are formed. Therefore, only the inelastic response of the beam elements is presented in Figures 5.5.2 – 5.5.4 and, as expected, the response of each model differs. The rotational ductility demand of the frames of model 24S is similar (Figure 5.5.2) due to the fact that its centre of strength is coincident with its centre of rigidity while the inelastic response of each frame of models 24A and 24B is different (Figures 5.5.3 and 5.5.4).

Hence, the inelastic response of different reference models is the same only when their strength distribution is proportional to their stiffness distribution (Figure 5.5.1). The design method adopted influences their strength distribution and their response, especially when they are stiffness-eccentric (TB models). The adoption of the first design procedure is very important, because it proves that the behaviour of different reference models can be the same when they are designed without the minimum steel ratios. Consequently, the differences in the inelastic seismic response of different reference models are mainly attributed to the differences in their strength distributions, and the comparison of the response of a TU model with a SM or TB model is effectively the same, when the reference models are appropriately designed.

5.5.2 Reference Models with Different Numbers of Floor Levels

The influence of the number of levels in the inelastic seismic response of the reference models is investigated in this section and the dynamic behaviour of models 6B, 12B and 24B is monitored. The design seismic forces of these models are calculated based on the UBC seismic code regulations, no accidental eccentricity provisions are incorporated, and the strength of the structural elements is calculated based on the first design method (without the minimum steel ratios). The inelastic response of the same type of reference model having different numbers of floor levels is highly dependent on the ground motion characteristics. In Figure 5.5.5, the inelastic response of the column elements of models 6B, 12B and 24B is presented when subjected to the ground motion accelerograms selected (Table 4.4.1). The mean response values of the models are also presented, and the inelastic response of the 6-storey and 12-storey reference models is in the same range while the inelastic response of the 24-storey model is lower. The rotational ductility demand of the beams is also presented in Figure 5.5.6 and it is higher than the ductility demand of the column elements due to the inclusion of the capacity design requirements.

In models 6B and 12B, the maximum values of rotational ductility demand are mainly produced by the El Suchil earthquake record while, in model 24B, different earthquake records at each floor level produce the maximum response values. In model 12B, the maximum value of rotational ductility demand is formed at the 3rd floor level and it is approximately equal to 7.5 for the El Suchil record. For the 3407 6th St. record, the column ductility demand at the same floor is 5.4 times less the maximum value, showing a large variation of the inelastic seismic response of the models depending on the earthquake record characteristics. From the results of the above analyses, it can be concluded that each earthquake record produces a different response pattern in models with identical structural configuration and varying heights. The same record may produce the maximum response value at one floor level while, at another floor level, it may produce the minimum response value. The response of the 24B model due to different records applied is more uniform while the response of the 12B model showed the highest variation.

5.5.3 Reference Models Designed to Different Seismic Codes

The influence of the static provisions of different seismic codes in the inelastic response of the reference models is examined in this section by monitoring the dynamic behaviour of model 24B, analysed according to the EC8, NZS and UBC seismic code provisions and designed without the minimum reinforcement requirements. The inclusion of the minimum steel ratios increases significantly the strength of the models and the influence of the calculation of the design forces with different seismic codes cannot then be identified. Therefore, model 24B is analysed for different base shears, with and without the accidental eccentricity, and by employing the first design method for the design of the structural elements.

The base shear calculation for each model based on different seismic code regulations was discussed in Section 5.3.2 and the equivalent static forces for each design code are given in Tables 5.3.1 – 5.3.3. The design base shear of the EC8 code is considerably higher than the base shear values of the other two codes and, as a result, the total strength of the model is higher when analysed for the EC8 regulations. Since model 24B is designed without the inclusion of the minimum steel ratios, its inelastic response is identical to the response of all 24-storey reference models, when the accidental eccentricity provisions are not included (Section 5.4.3). Hence, the inelastic response of both columns and beams of all 24-storey reference models is higher for the UBC and the NZS seismic code regulations while it is considerably lower for the EC8 regulations (Figure 5.5.7). The influence of the accidental eccentricity provisions of different seismic codes is also examined without including the minimum reinforcement provisions for the design of the structural elements (Figure 5.5.8). When the accidental eccentricity is included, the response of each frame of model 24B is different while the maximum values of ductility demand are considerably reduced. The results for the NZS and UBC seismic code regulations are similar while the results for the EC8 provisions are different due to the higher base shear. The influence of the static torsional provisions of different seismic codes is analytically examined in Chapters 6 – 8, where the inelastic torsional behaviour of different types of TU models is investigated.

5.6 CONCLUSIONS

The aim of this chapter was to present the inelastic seismic response of the reference models employed in this study and to investigate the factors influencing their inelastic behaviour. Both symmetric (SM) and the torsionally balanced (TB) reference models were examined and their inelastic behaviour was compared. The factors influencing the strength distribution of the models were examined in Section 5.4 while, in Section 5.5, the inelastic response of different types of reference models was presented. The basic conclusions concerning the inelastic response of the reference models can be summarised as:

1. The most important factor influencing the inelastic behaviour of the reference models is their **strength distribution.** A strength distribution proportional to the stiffness distribution of the structural elements results in a reference model that responds purely in translation, with all frames responding in the same way.
2. The **factors influencing the strength distribution** of the reference models are the minimum reinforcement provisions in the design of the structural elements, the distribution of the vertical uniform load over the frames, and the accidental eccentricity provisions. The vertical load distribution of the reference models must be proportional to their lateral stiffness distribution, especially in TB models, where only this load distribution results in a proportional strength distribution. Moreover, the inclusion of the accidental eccentricity requirements alters the strength distribution of the frames, and results in a strength distribution non-proportional to the stiffness distribution. Finally, the inclusion of the minimum reinforcement provisions in the design of the models affects significantly their strength distribution and results in a different inelastic response for each frame.
3. **Different design procedures** have been tested and the importance of employing two design procedures for the strength calculation of the models is indicated. The first design method excludes the minimum steel ratios of the codes for the design of the structural elements and results in a strength distribution proportional to the stiffness distribution of the models. The second design method is consistent with the design code requirements and includes the minimum steel ratios for the strength calculation of the models. The first method is employed to better understand the behaviour of both reference and TU models while the second design method examines the efficiency of the seismic code provisions.
4. The inelastic response of **different types of reference models** (SM and TB models) having the same height and fundamental properties (fundamental lateral period and stiffness) is the same when the structural elements are designed without the accidental eccentricity provisions and the minimum reinforcement

requirements. Including the minimum steel ratios in the design of the structural elements results in a varying inelastic response for each reference model.

5. The response of **reference models with varying height** and identical structural configuration is different and depends on the model period and on the earthquake record characteristics. Each earthquake record affects differently each reference model and its maximum inelastic response at different storeys is produced from different accelerograms. Mid-rise structures (12-storey models) are significantly affected by the earthquake characteristics while high-rise structures (24-storey models) result in a more uniform response for different earthquake records.
6. When the **reference models are designed to different seismic code provisions,** the inelastic response of the same model varies and depends on the seismic code regulations. The NZS and UBC design seismic forces have similar values and, as a result, the inelastic response of a model designed for the NZS and UBC provisions is similar as well. The EC8 code is more conservative than the other two codes and its equivalent horizontal forces are higher while the inelastic response of the model is considerably lower.

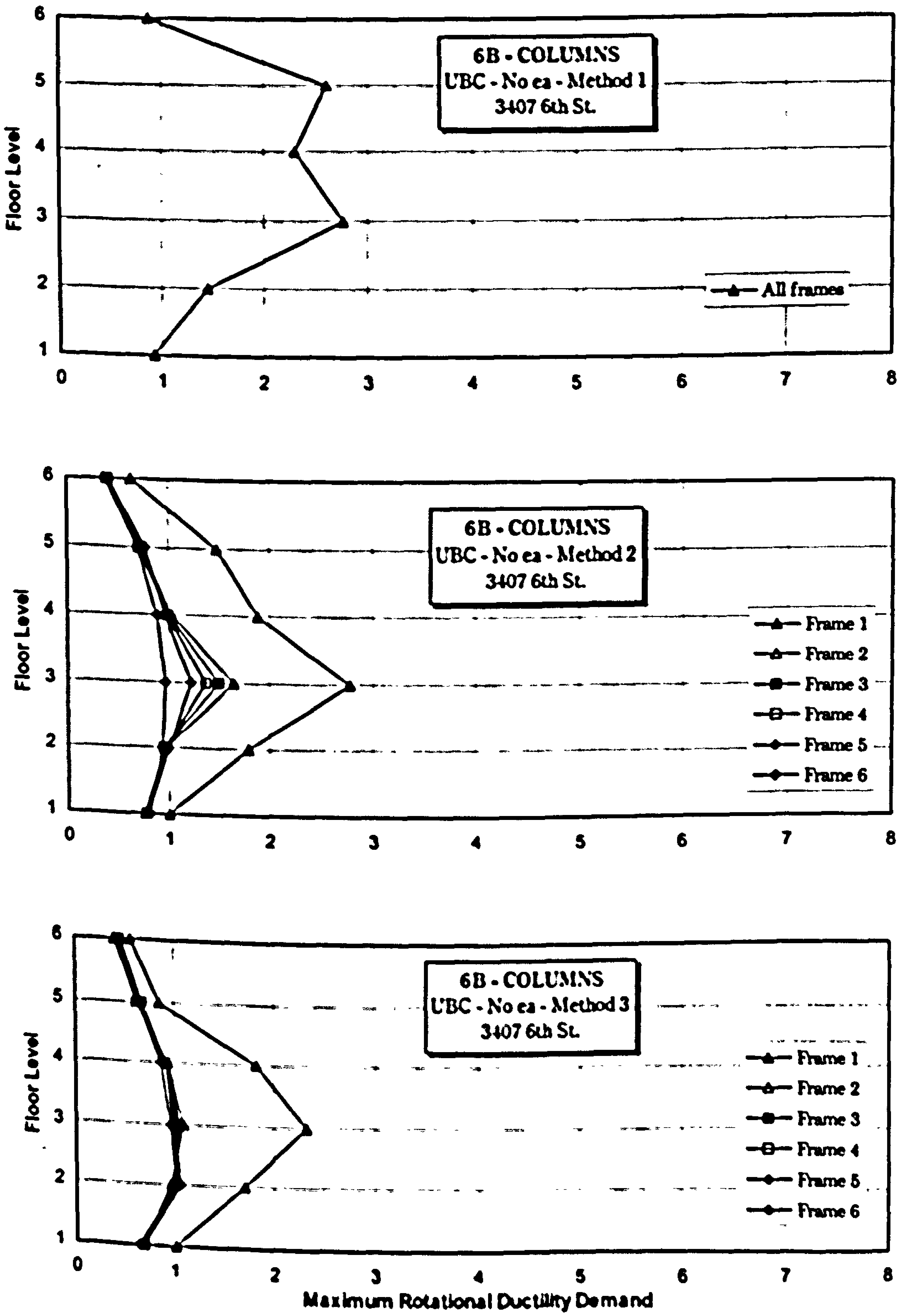


Figure 5.4.1 Maximum rotational ductility demand vs. floor levels for the columns of model 6B, designed according to different methods.

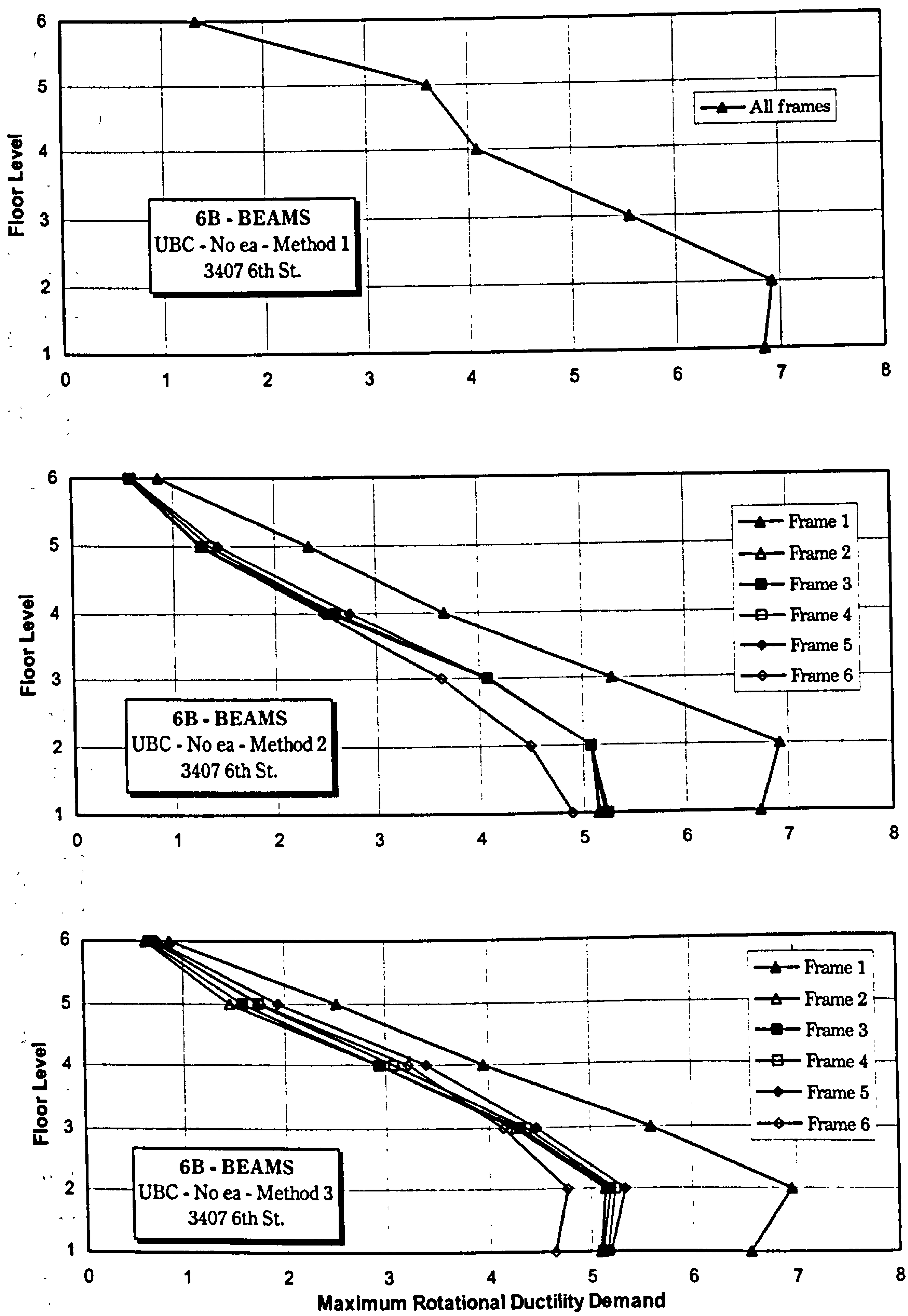
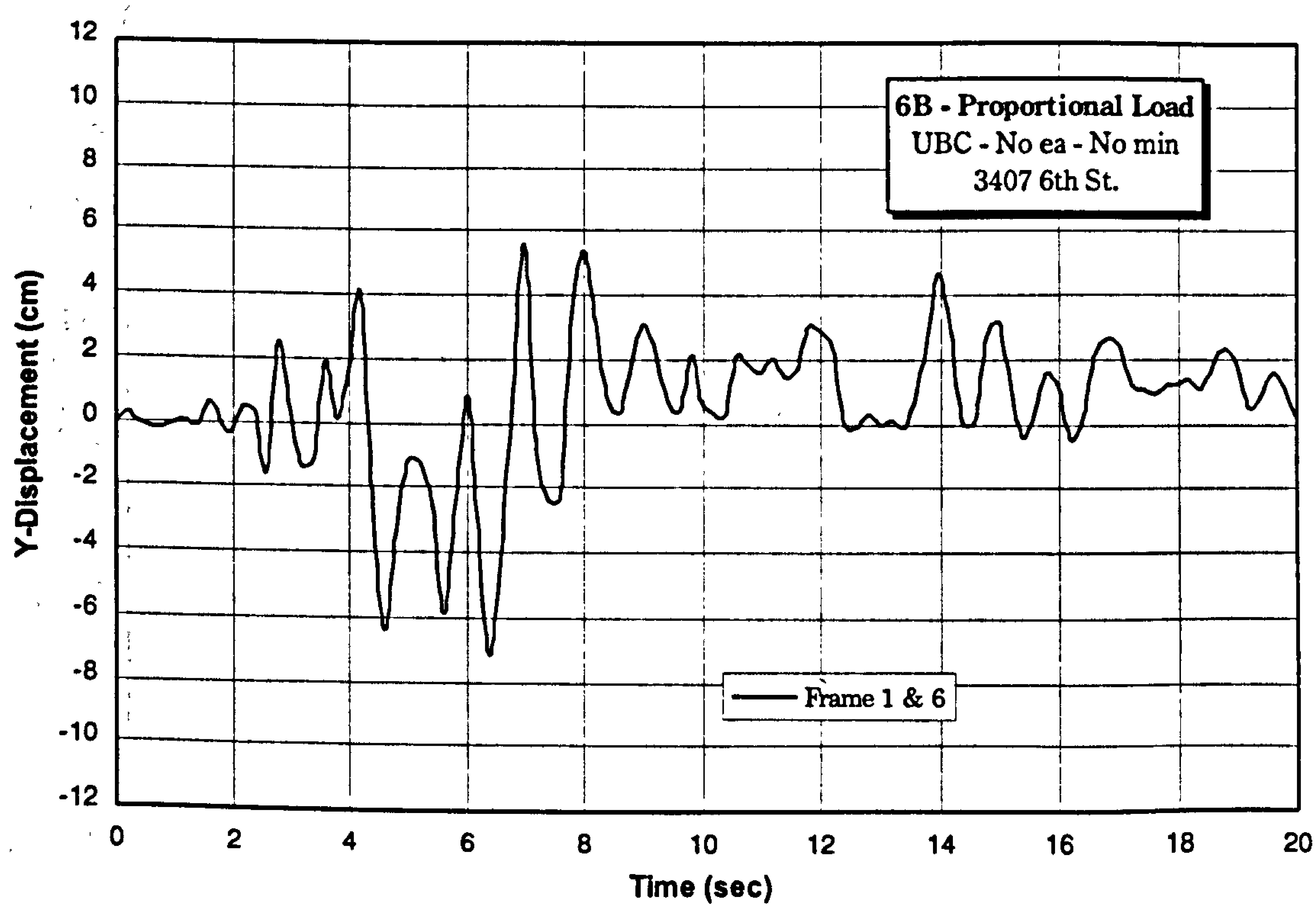


Figure 5.4.2 Maximum rotational ductility demand vs. floor levels for the beams of model 6B, designed according to different methods.



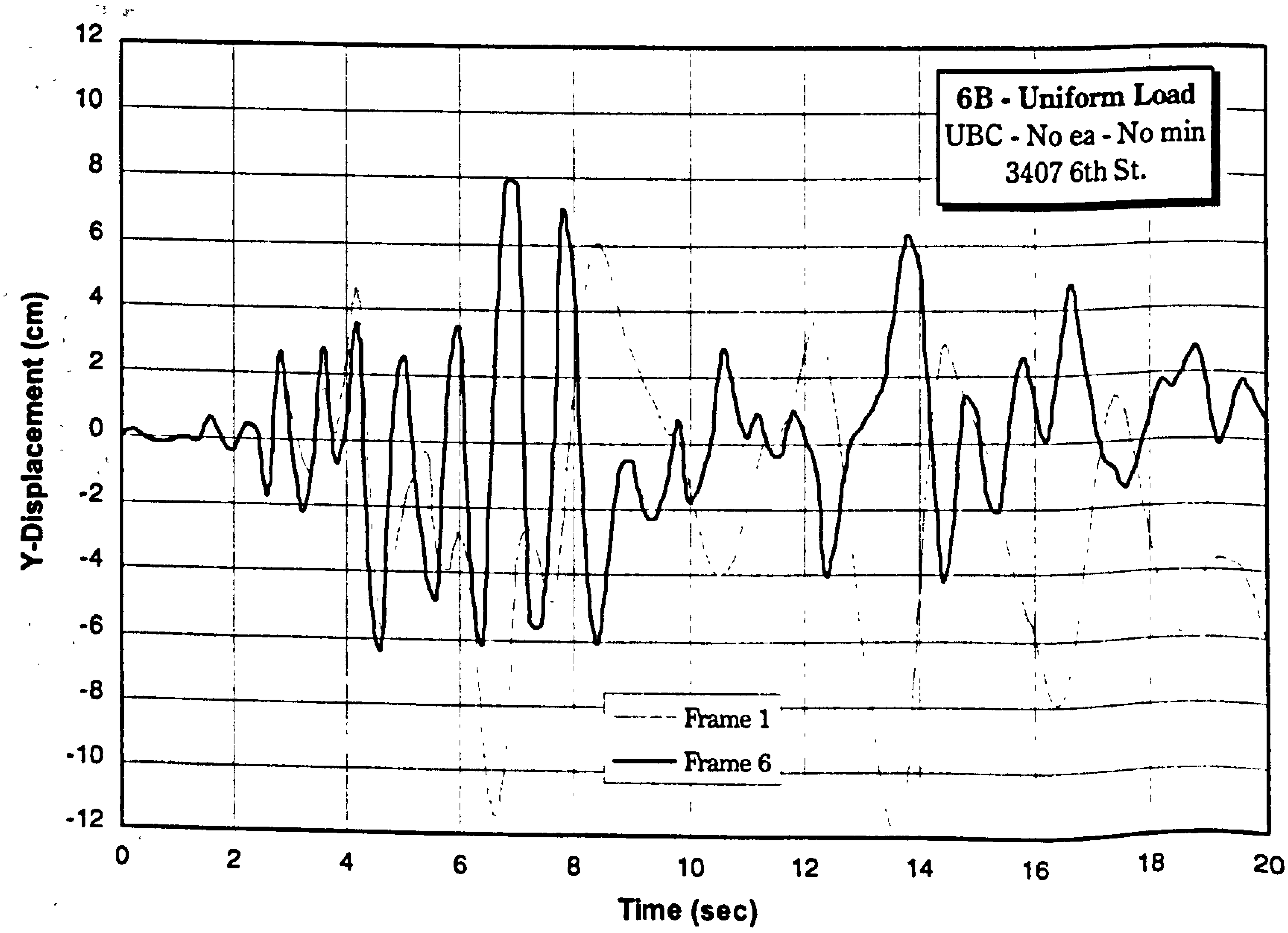
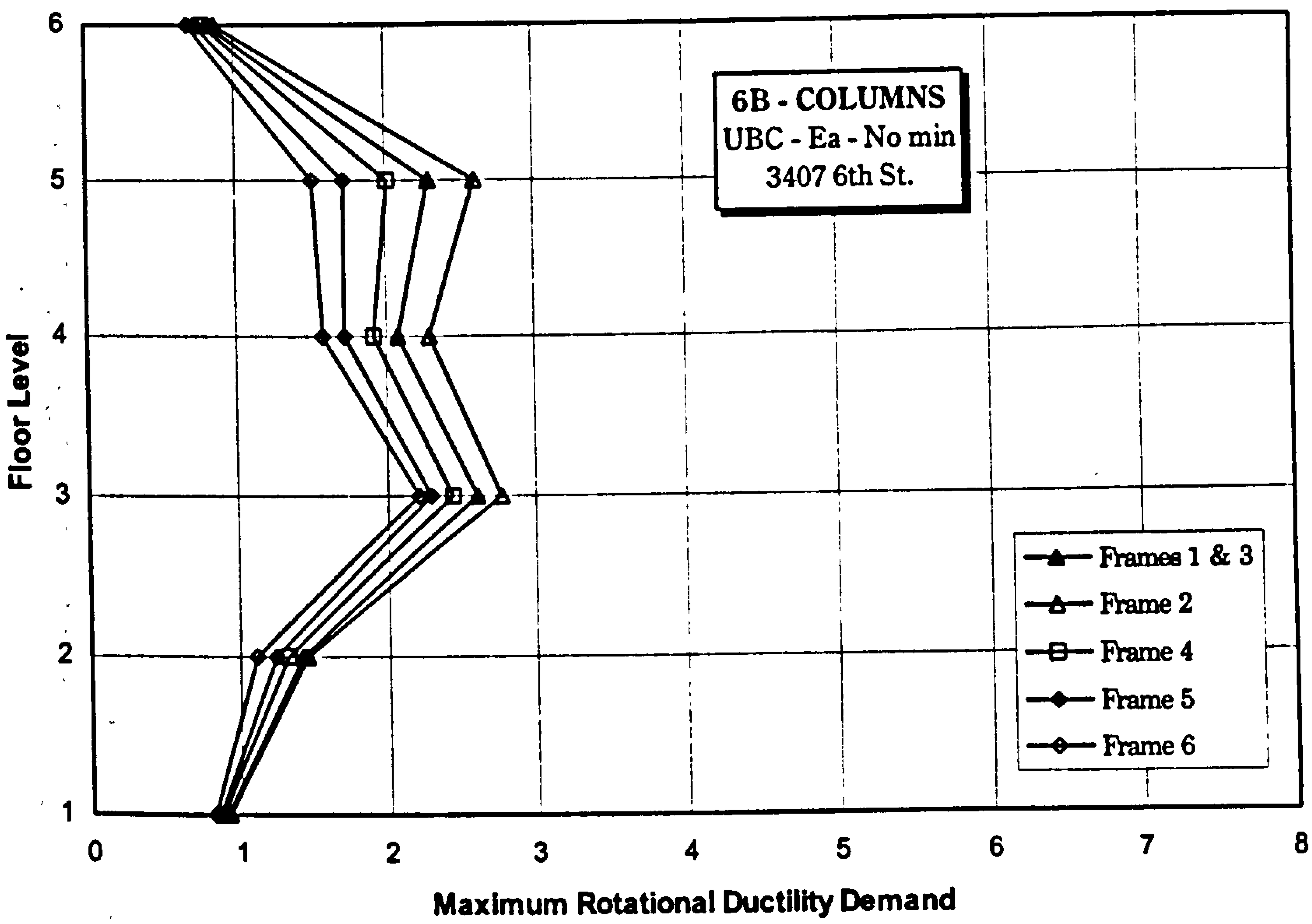


Figure 5.4.3 Time-history displacement of the top floor level of the external frames (frames 1 and 6) of model 6B, analysed for different vertical load distributions.



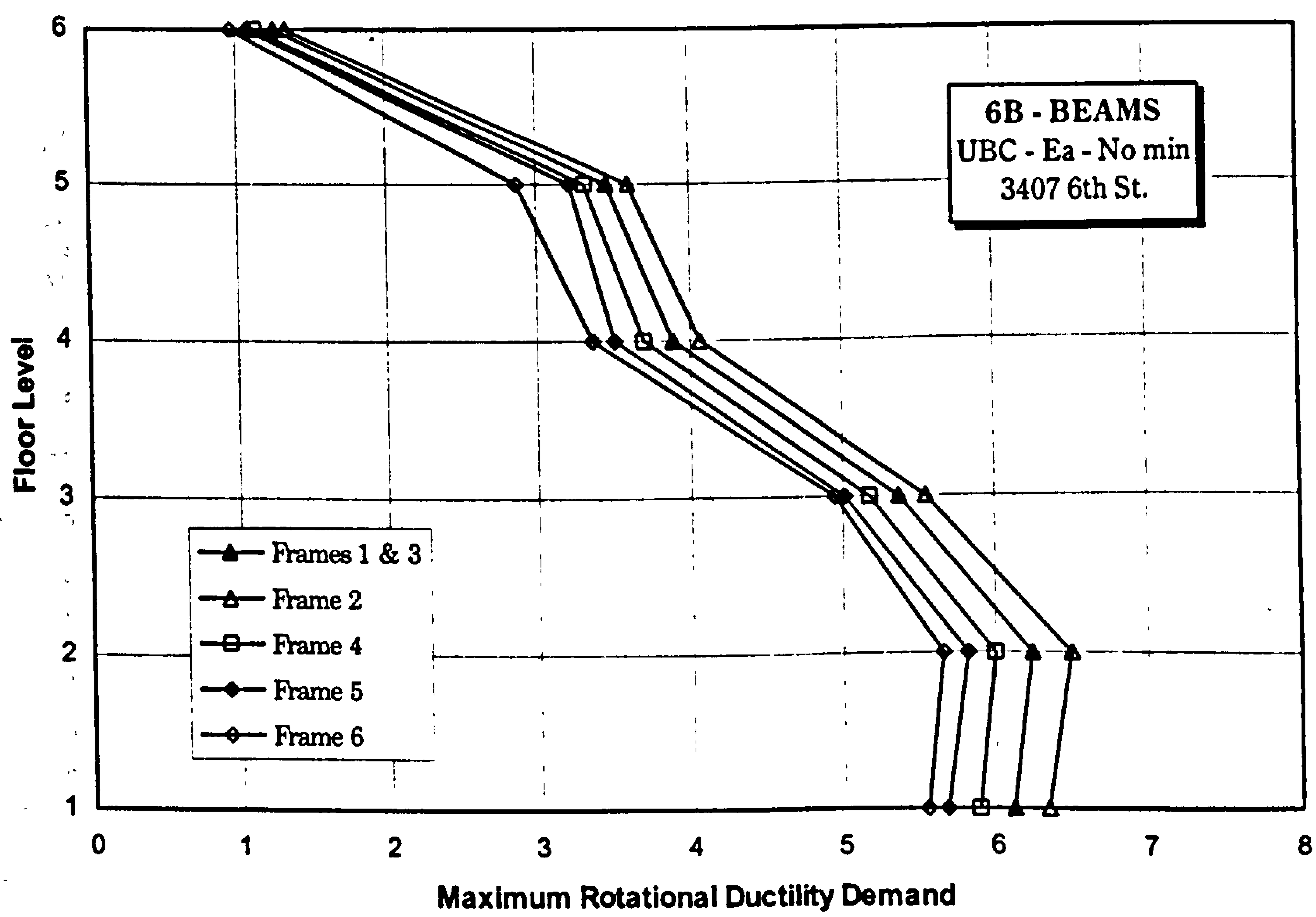


Figure 5.4.4 Maximum rotational ductility demand vs. floor levels for model 6B designed by including the accidental eccentricity provisions.

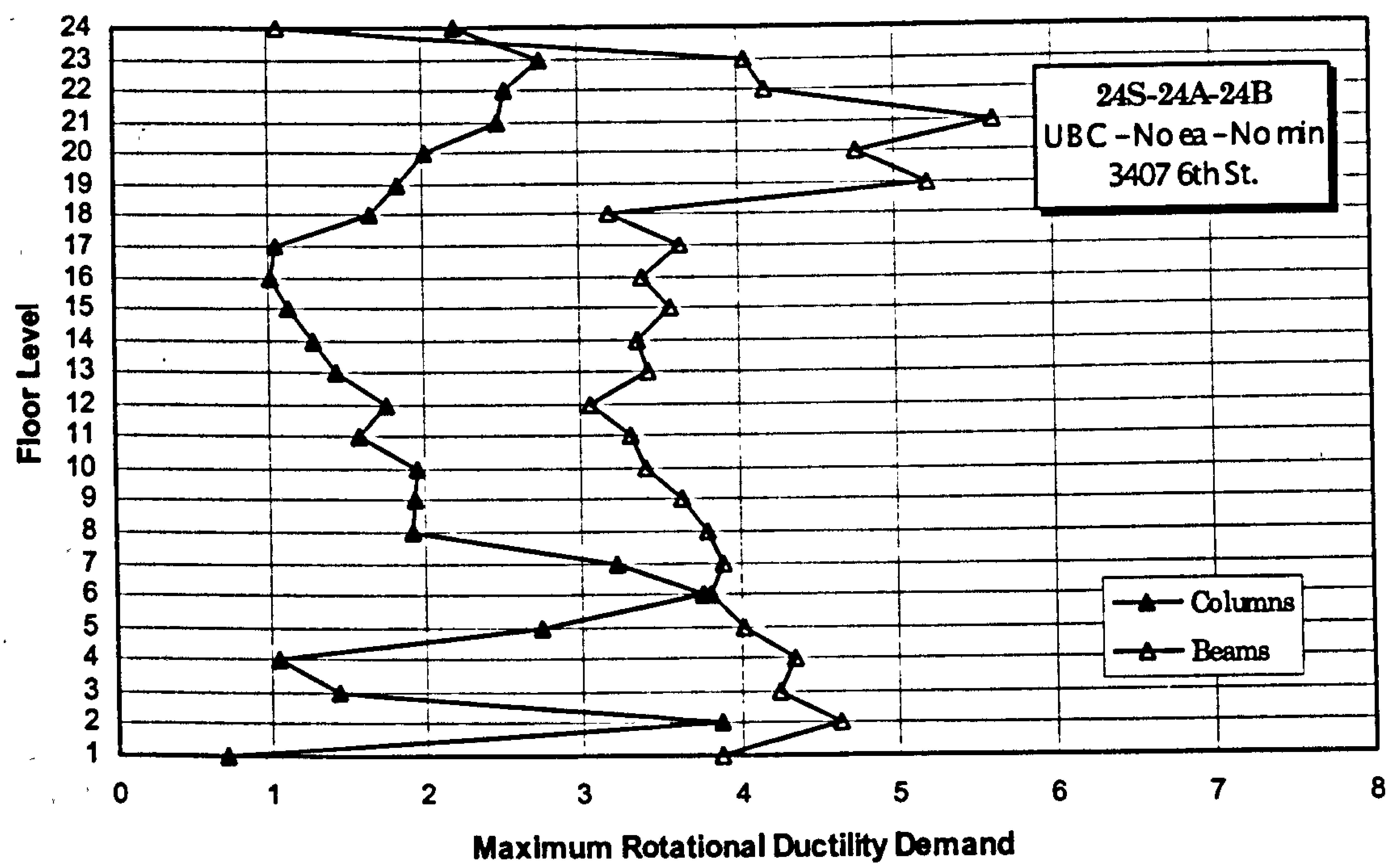


Figure 5.5.1 Maximum rotational ductility demand vs. floor levels for the 24-storey reference models designed for the 1st design method.

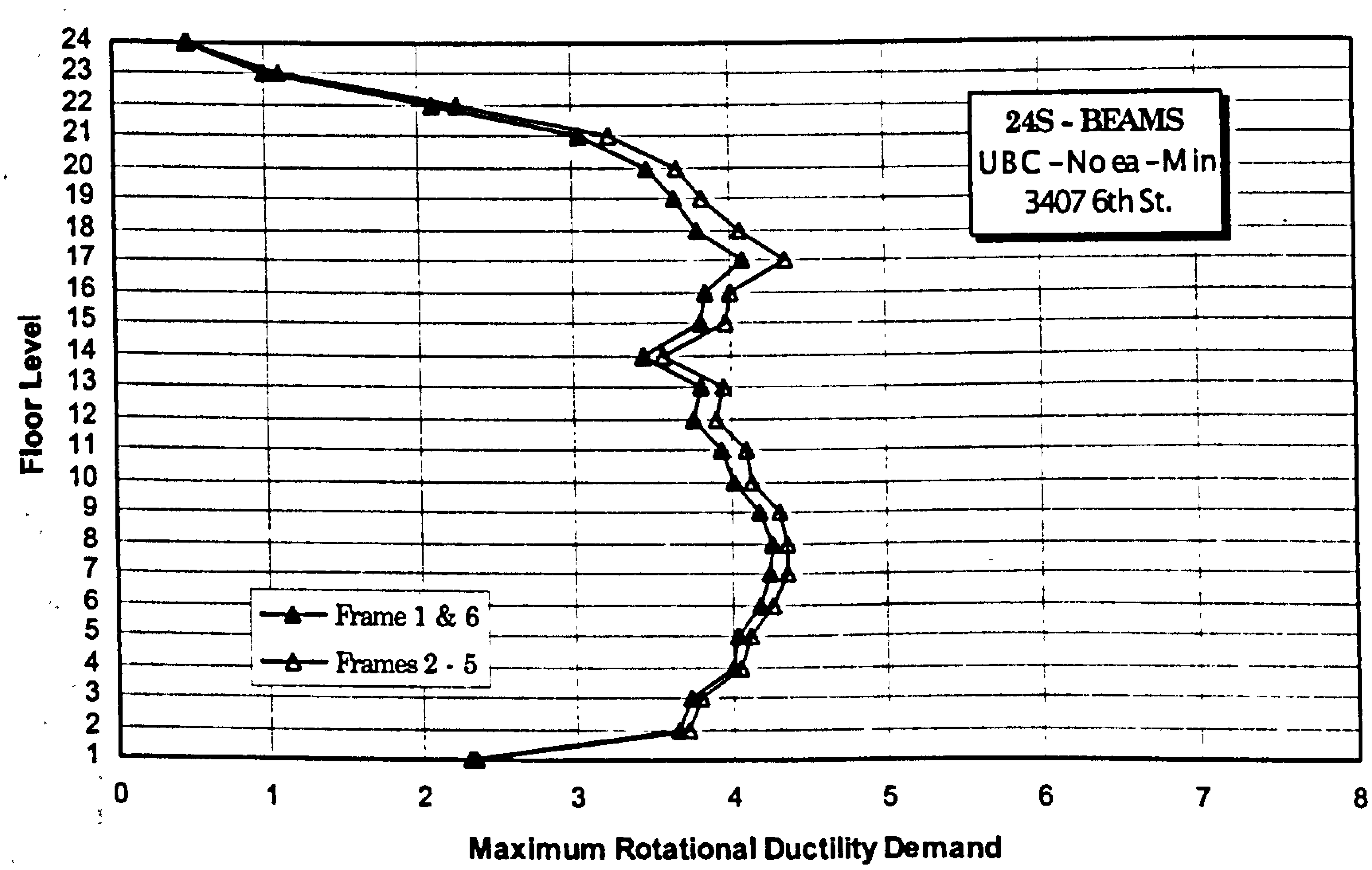


Figure 5.5.2 Maximum rotational ductility demand vs. floor levels for the beams of model 24S, designed for the 2nd design method.

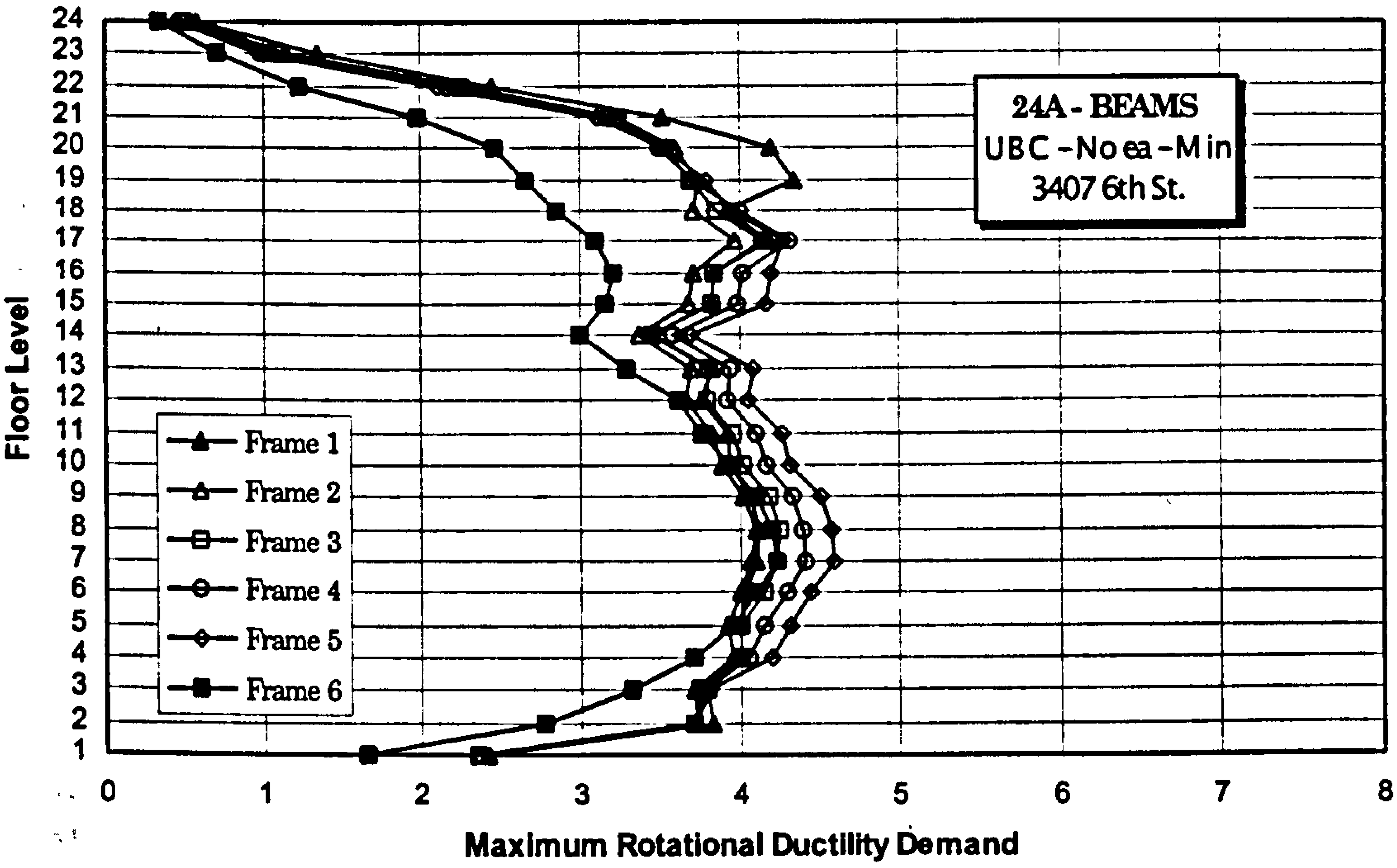


Figure 5.5.3 Maximum rotational ductility demand vs. floor levels for the beams of model 24A, designed with the minimum reinforcement requirements (2nd design method).

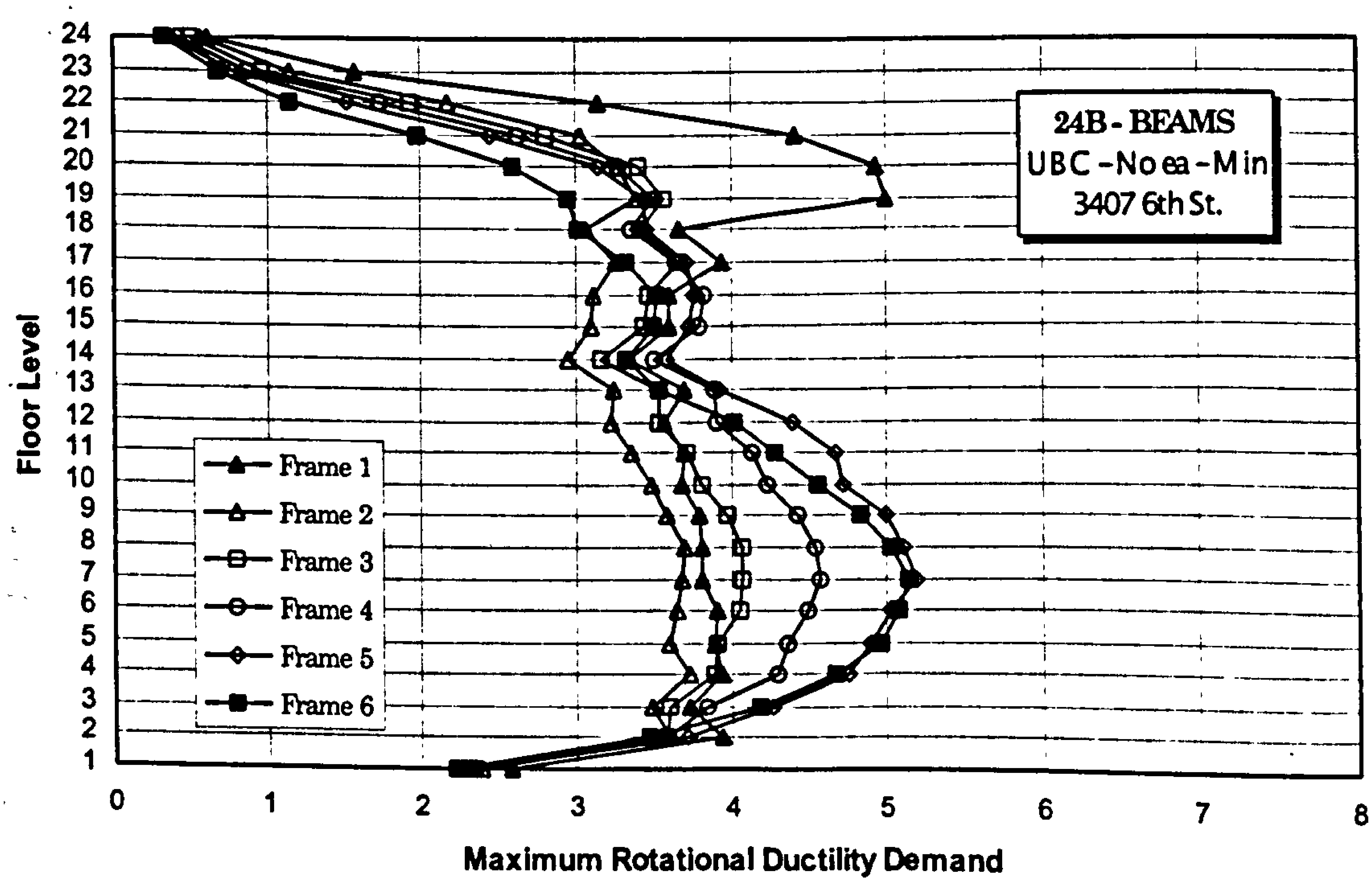


Figure 5.5.4 Maximum rotational ductility demand vs. floor levels for the beams of model 24B, designed with the minimum reinforcement requirements (2nd design method).

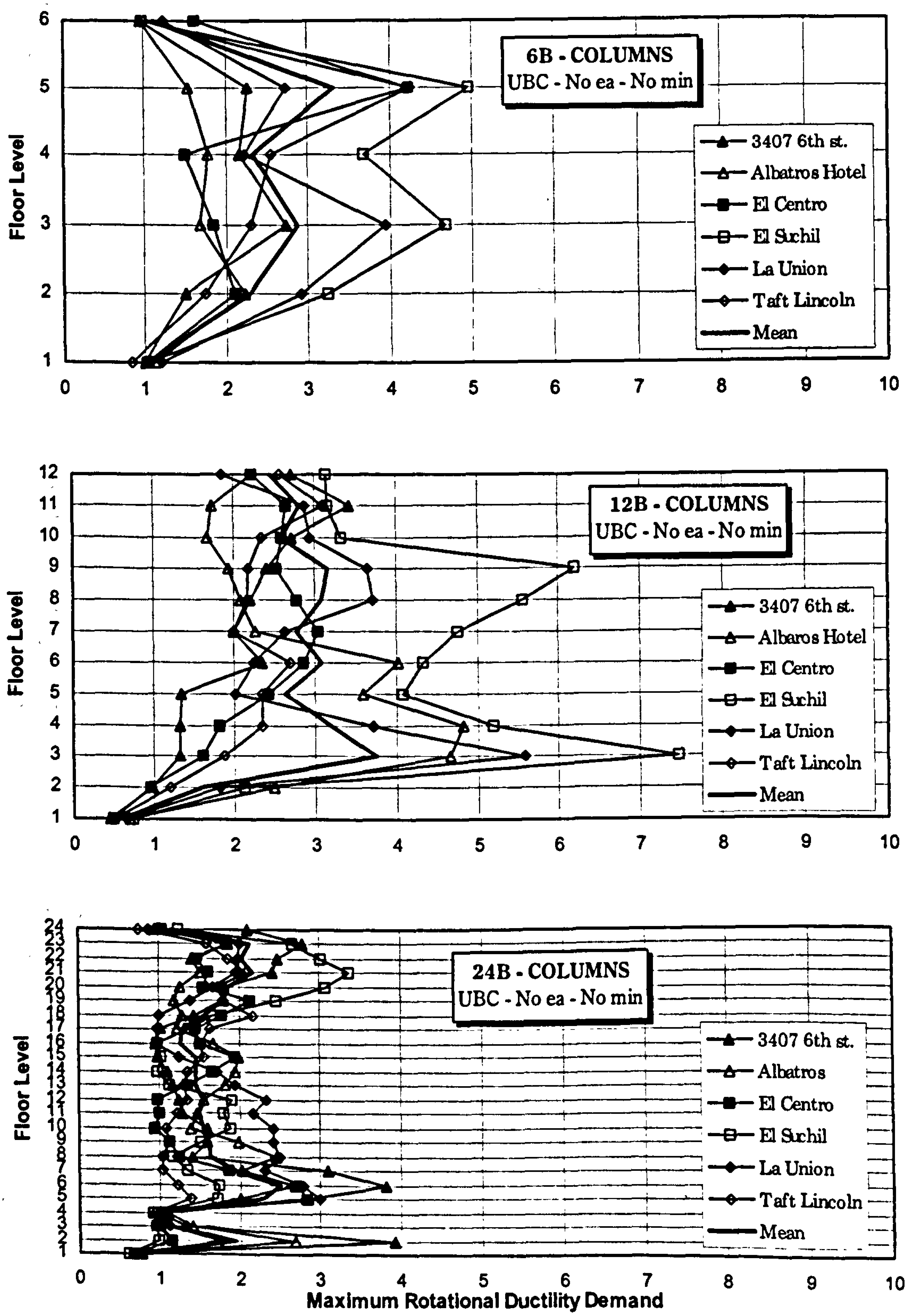


Figure 5.5.5 Maximum rotational ductility demand vs. floor levels for the columns of models 6B, 12B and 24B, subjected to different earthquake ground motions.

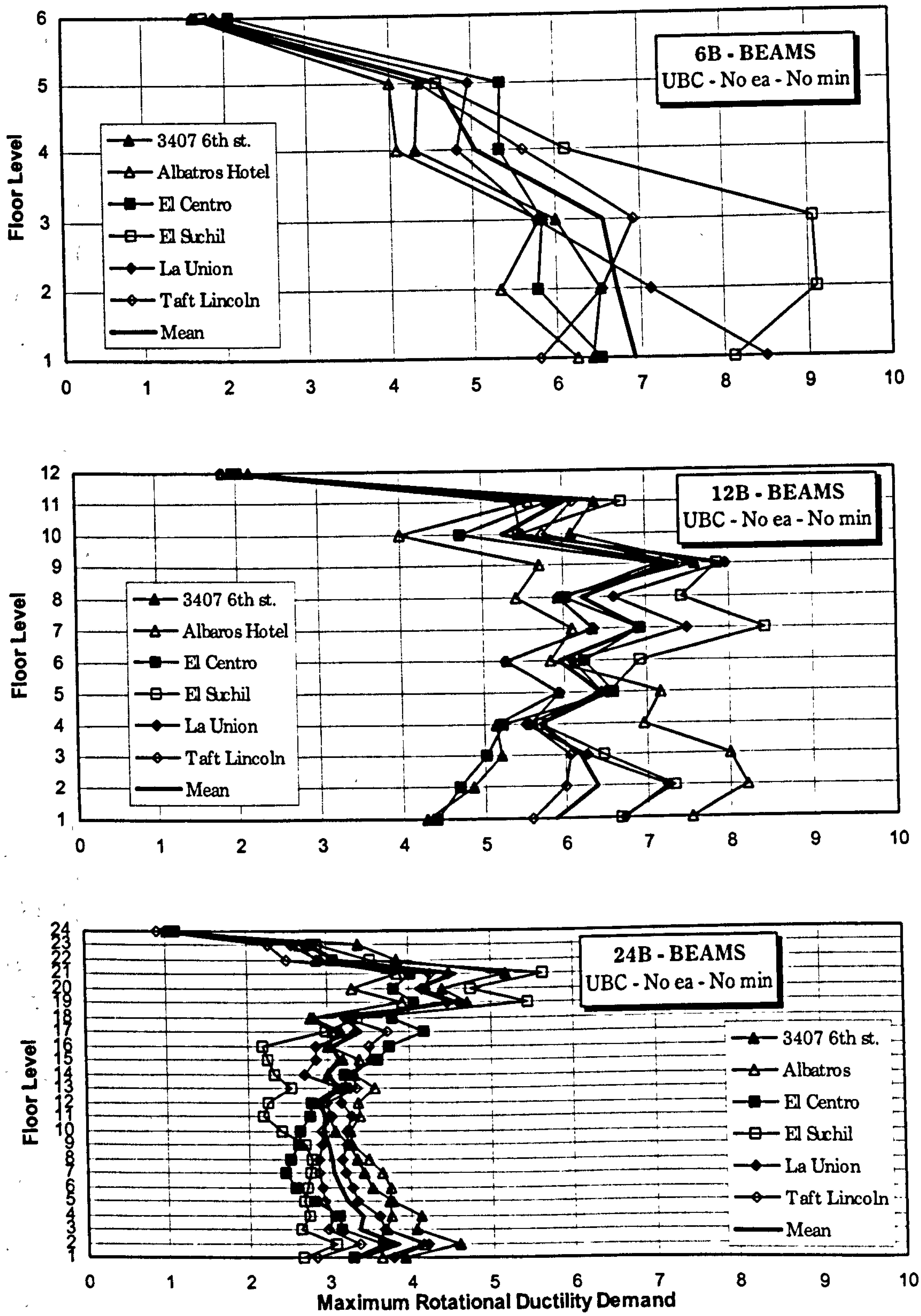
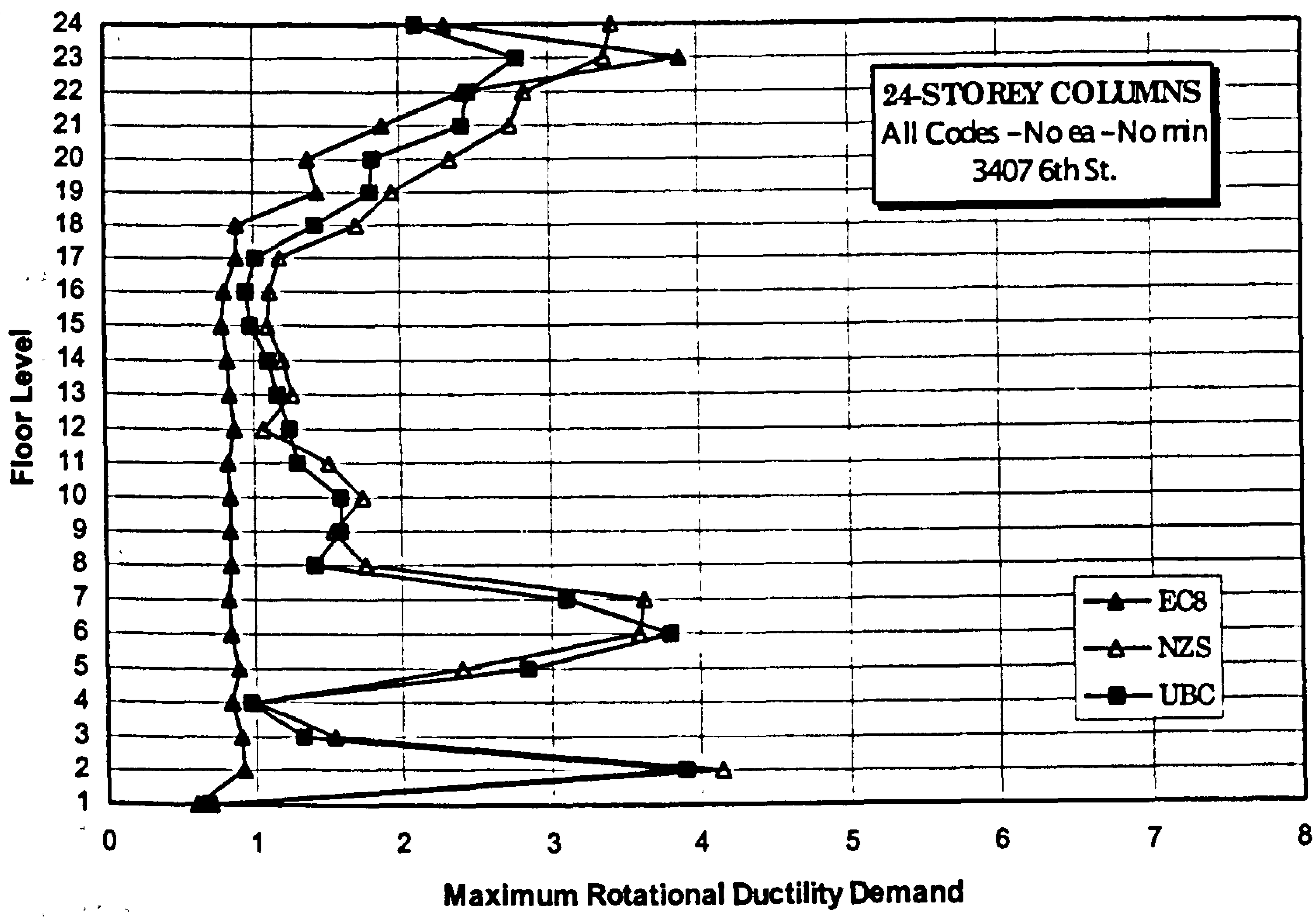


Figure 5.5.6 Maximum rotational ductility demand vs. floor levels for the beams of models 6B, 12B and 24B, subjected to different earthquake ground motions.



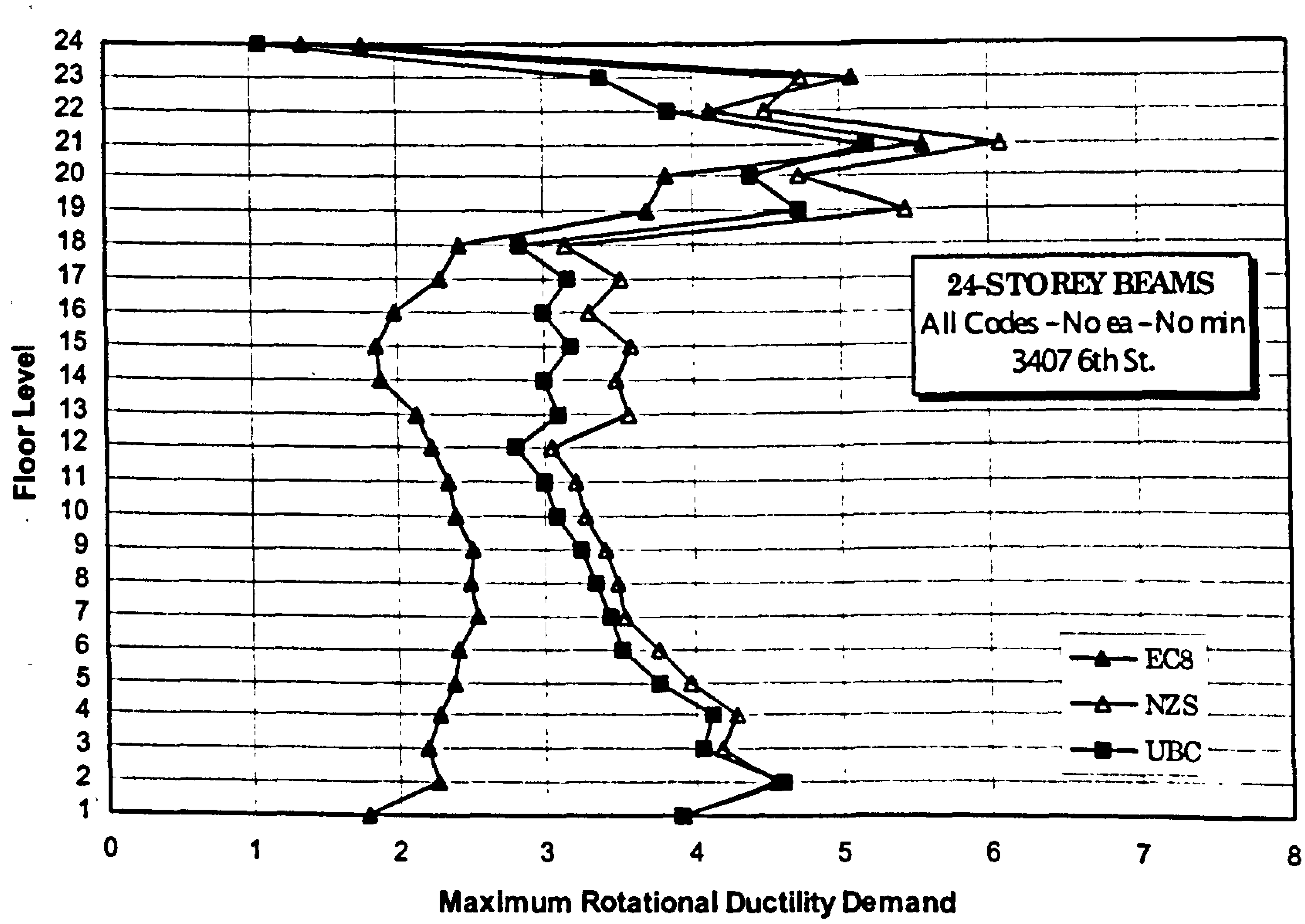


Figure 5.5.7 Maximum rotational ductility demand vs. floor levels for the 24-storey reference models designed to different seismic codes, without accidental eccentricity and with the 1st method.

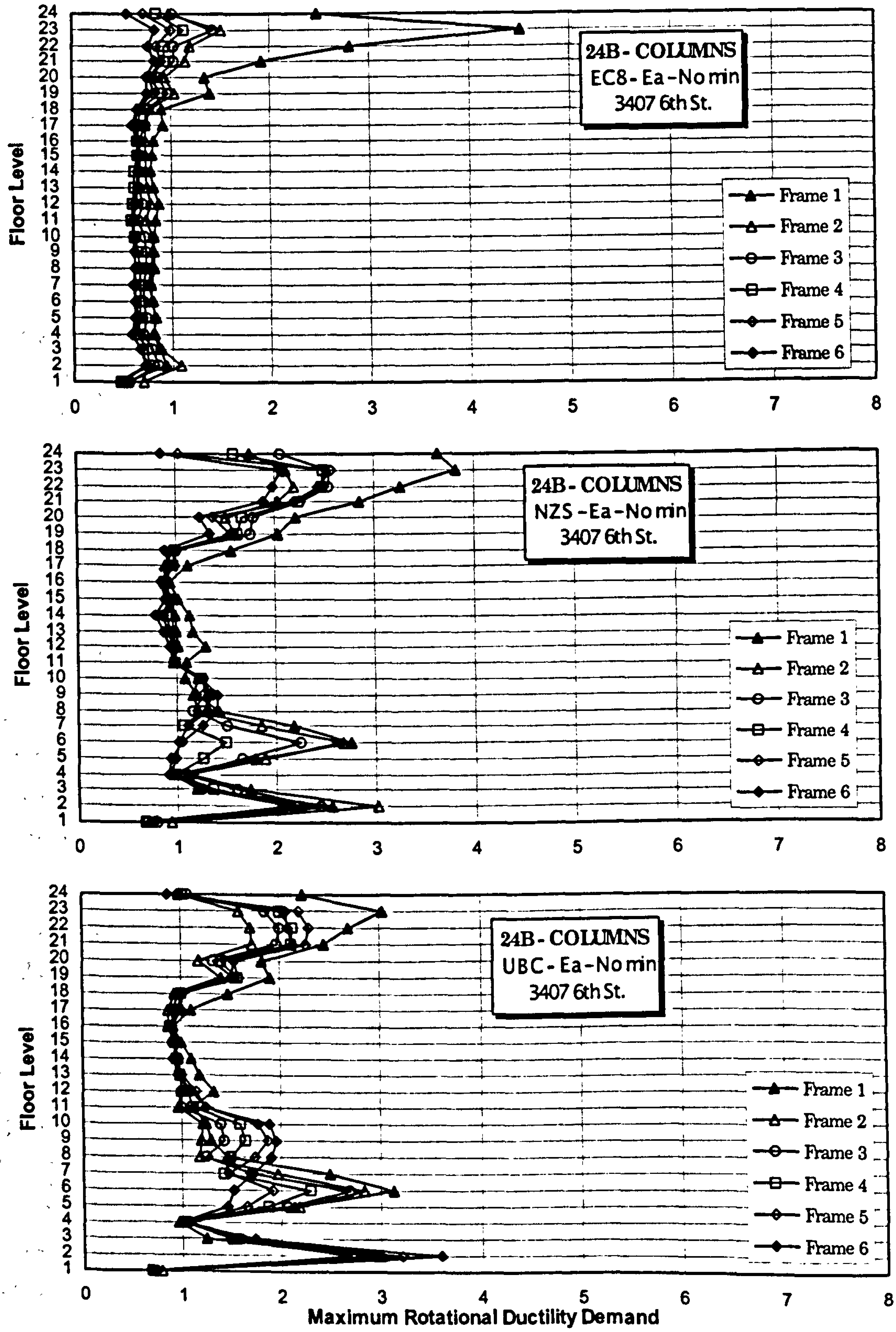


Figure 5.5.8 Maximum rotational ductility demand vs. floor levels for model 24B, designed to different seismic codes, by employing the accidental eccentricity provisions and the 1st design method.

CHAPTER 6

INELASTIC SEISMIC RESPONSE OF MULTI-STOREY REGULARLY ASYMMETRIC MODELS

6.1 INTRODUCTION

From the investigations and the results of the detailed inelastic analyses carried out in Chapter 5, it is apparent that the evaluation of the inelastic response of the torsionally unbalanced (TU) models is highly dependent on the definition of the reference models employed. The definition adopted for the symmetric (SM), or the torsionally balanced (TB) models, by which the torsional effects of the TU models are quantified and compared is a major factor contributing to the inconsistent conclusions of previous studies. The factors that influence the inelastic seismic behaviour of the reference models and, consequently, affect the interpretation of the inelastic seismic response of the TU models have been analytically investigated in Sections 5.4 and 5.5.

The strength distribution of the models has a major influence on the maximum ductility demand and deformation of the lateral load-resisting elements of structures subjected to severe earthquake loading. The most important factors influencing the strength distribution of the models is the design procedure adopted for the calculation of the reinforcement needed for the structural elements (Section 5.4.1) and the inclusion of the accidental eccentricity provisions of the seismic codes (Section 5.4.3). Thus, in order to cover a variety of cases, the TU models examined are defined in four different ways concerning the inclusion, or not, of the above two factors (Section 6.3). Moreover, the comparison between the response of TU and reference models is also based on different cases concerning the incorporation of these factors (Section 7.2).

The following section of this chapter (Section 6.2) deals with the influence of the static code torsional provisions in the design of TU models. The regularity requirements of different seismic codes are examined and it is checked whether the TU models analysed satisfy these requirements. The design eccentricity provisions of the three seismic codes adopted are presented, and the approach of each code to calculate the accidental eccentricity is assessed. The values of the first and second (alternative) design eccentricities are illustrated for models with different static eccentricities. In Section 6.3, the different definitions employed for the models examined are explained, and the effect of the alternative design methods is demonstrated (see also Section 5.4.1).

In Sections 6.4 – 6.7, the inelastic dynamic response of different multi-storey regularly asymmetric (TU) frame models is investigated, and the influence of various factors on their inelastic dynamic behaviour is systematically examined. The factors analysed include the amount of the static eccentricity (Section 6.4), the model type adopted (mass-eccentric and/or stiffness-eccentric) (Section 6.5), the model height (Section 6.6), and the torsional provisions of different seismic codes (Section 6.7). The influence of the inclusion of the accidental eccentricity provisions and the incorporation of the minimum reinforcement requirements imposed by the design codes for the design of the structural elements is examined in combination with the factors presented above.

The factors influencing the inelastic seismic response of the TU models, investigated in Chapter 6, are re-examined in Chapter 7 by comparing the seismic response of the same set of TU models with the seismic response of their reference models (Sections 7.3 – 7.6). Different definition cases regarding the inclusion of the accidental eccentricity provisions and the incorporation of the minimum steel ratios are employed for the comparison of the models and they are presented in Section 7.2. Therefore, the final conclusions regarding each factor influencing the inelastic dynamic behaviour of the TU models are based on the inelastic analyses carried out in both Chapters 6 and 7 and they are summarised in the relevant sections of Chapter 7. The conclusions reached from this study are also compared with the conclusions reached by previous studies.

6.2 TORSIONAL PROVISIONS OF DIFFERENT SEISMIC CODES

6.2.1 Regularity Requirements and Method of Analysis

The studies examining the inelastic seismic response of code-designed TU models subjected to severe earthquake loading aim to quantify the additional damage-inducing response of the models arising from the inelastic torsional effects, relative to the behaviour of the reference models. Such torsional effects are accounted for in earthquake resistant building regulations by specifying an additional strength increase to the vulnerable structural elements. The equivalent static method (Section 4.2.2) is a simple procedure widely employed by all existing seismic codes, which endeavours to account primarily for the effects of the static eccentricity resulting from differences between the mass and stiffness distribution on the torsional response of the structures. This approach accounts for all other non-quantifiable sources of torsion by the use of an accidental eccentricity component applicable even in cases of structural symmetry.

Each seismic code applies specific regularity criteria that have to be satisfied in order to utilise the equivalent static method of analysis and the static torsional provisions (see also Sections A.1, A2, B.1, B.3, C.1 and C.2). The reference models obviously satisfy the regularity criteria (see also Section 5.3.1) while, for the TU models, it should be checked whether they satisfy the regularity criteria of the seismic codes. The fundamental characteristics of the models investigated are shown in Table 6.2.1, where the notation of each model is explained and the locations of the centres of mass and rigidity are given relatively to their distance from frame 1 (see also Figures 3.4.1 – 3.4.3).

For all model types shown in Table 6.2.1, systems with various heights are examined and, therefore, the models analysed consist of 6, 12, or 24 floor levels. Therefore, when a model is termed as 12A30, it has 12 storeys, a static eccentricity equal to 0.30*b*, and its model type is A (Section 3.4.3). Thus, the first figure(s) of the notation of each model indicates the number of floor levels, the following letter represents the model type, and the final figures correspond to the amount of static eccentricity. The model type indicates the structural configuration of each model and the locations of their centres of rigidity and mass (Table 6.2.1).

NOTATION AND CHARACTERISTICS OF THE MODELS				
Notation	**Model Type**	**Static Eccentricity**	**CR**	**CM**
S	SM reference model	-	1500	1500
S15	TU mass-eccentric model	0.15*b* (450)	1500	1050
S30	TU mass-eccentric model	0.30*b* (900)	1500	600
A	TB reference model	-	1050	1050
A15	TU stiffness-eccentric model	0.15*b* (450)	1050	1500
A30	TU stiffness/mass-eccentric model	0.30*b* (900)	1050	1950
B	TB reference model	-	600	600
B15	TU stiffness/mass-eccentric model	0.15*b* (450)	600	1050
B30	TU stiffness-eccentric model	0.30*b* (900)	600	1500
B45	TU stiffness/mass-eccentric model	0.45*b* (1350)	600	1950

Table 6.2.1 Notation and characteristics of the models investigated. (All values are in centimetres and the locations of CM and CR are given relatively to their distance from frame 1).

(a) Regularity requirements of the NZS seismic code

In the NZS (1992) Standard code, the equivalent static method is employed where at least one of the three regularity criteria of the code is satisfied (Sections A.1 and A.2). All the models of this study satisfy the third regularity criterion, which requires the vertical and the horizontal regularity requirements to be satisfied by systems with a fundamental period less than 2 seconds. The vertical regularity criterion requires that the lateral displacement at each floor level should be reasonably proportional to the level height. The models investigated satisfy this criterion since they are regularly asymmetric and their centres of mass and rigidity are located on two vertical lines. Furthermore, the horizontal regularity provisions require that the models contain no abrupt variations in stiffness and no re-entrant corners, which is also satisfied by all models. In addition, one of the following two horizontal regularity criteria should be satisfied.

The first horizontal regularity criterion stipulates that the offset between CM and CR at any floor level should be less than 0.3 times the plan dimension perpendicular to the earthquake loading direction. This criterion is satisfied for the models investigated, except for models with static eccentricity equal to 0.45*b* (models termed as 6B45, 12B45 and 24B45). Although these systems violate both horizontal regularity provisions, they are employed in order to examine extreme cases of TU models with high static eccentricities. The second horizontal regularity criterion is a limitation on the structural torsional stiffness and requires that, under the action of the equivalent static forces, the horizontal displacement ratio measured at the ends of a system on an axis transverse to the direction of the lateral forces should be within the range of 3/7 to 7/3. This criterion is dependent on the torsional stiffness of the model and requires the lateral forces to be applied at a distance $\pm 0.10b$ from CM. This regularity requirement is more restrictive than the first one and none of the TU models investigated satisfy it. Table 6.2.2 presents the horizontal displacement ratios at the ends of models 6S, 6A, 6S15 and 6A15 (Figures 3.4.1 and 3.4.2) transverse to the earthquake direction. These models are chosen because they represent the basic model types investigated with the lowest static eccentricity.

HORIZONTAL REGULARITY CRITERION OF THE NZS (1992) SEISMIC CODE						
Model	**CR**	**CM**	**Accidental Eccentricity**	**Point of Application of the Design Forces**	**Ratio of the Horizontal Edge Displacements (0.43 – 2.33)**	**Regular**
6S	1500	1500	No	1500	1.00	Yes
			Yes	1800 1200	2.90 0.34	No
6S15	1500	1050	No	1050	0.15	No
			Yes	1350 750	0.60 0.10	No
6A	1050	1050	No	1050	1.00	Yes
			Yes	1350 750	2.44 0.25	No
6A15	1050	1500	No	1500	3.99	No
			Yes	1800 1200	18.4 1.56	No

Table 6.2.2 Horizontal regularity criterion of the NZS (1992) seismic code. (All values are in centimetres and the locations of CM and CR are given relatively to their distance from frame 1).

The second horizontal regularity requirement of the NZS code requires the lateral forces to be applied at a distance $\pm 0.1b$ from CM but, in this section, the ratios of the edge displacements of the models are calculated with and without the accidental eccentricity (Table 6.2.2). Even for the reference models, this criterion is not satisfied when the accidental eccentricity is included while, for the TU models, it is never satisfied irrespective of the inclusion, or not, of the accidental eccentricity. Hence, most of the TU models of this study satisfy only the first horizontal regularity criterion while the second criterion is quite strict. All the TU models will be analysed based on the equivalent static method in order to prove whether this second regularity requirement is restrictive, or not, and to assess the effectiveness of the equivalent static method for the seismic design of asymmetric (TU) models.

(b) Regularity requirements of the EC8 seismic code

Based on the EC8 (1993) seismic code regulations, all structures are distinguished as regular, or non-regular, for the purpose of the seismic design. The consequences of the structural regularity of the models on their seismic design are summarised in Table B.1.1 (Section B.1). In the EC8 code, the equivalent static method can be applied if the models satisfy the regularity requirements or one of the two sets of conditions given in Section B.3. The TU models of this study satisfy only the second set of conditions, which requires a rigid floor diaphragm behaviour and the centres of stiffness and mass to be located on two vertical lines along the height.

Regarding the regularity conditions of the EC8 code, the models employed satisfy the conditions for regularity in elevation while they do not satisfy all the conditions for plan regularity. The horizontal regularity conditions requiring a symmetric lateral stiffness and mass, a total dimension of re-entrant corners less than the 25% of the overall plan dimension, and a large in-plane stiffness of the floors in comparison with the stiffness of the vertical elements, are satisfied by all models. However, the models do not satisfy the plan regularity condition requiring the maximum storey displacement in the direction of the seismic forces not to exceed the average storey displacement by more than 20% when the seismic forces are applied with the accidental eccentricity.

In Table 6.2.3, this last plan regularity provision is examined by presenting the maximum storey displacement of various models compared with their average storey displacement. The models examined are the same models presented in Table 6.2.2, where the horizontal regularity criterion of the NZS code was investigated. Although this regularity requirement of the EC8 code requires the accidental eccentricity to be applied, the ratios of the edge displacements of all models are calculated with and without the inclusion of the accidental eccentricity.

HORIZONTAL REGULARITY CRITERION OF THE EC8 (1993) SEISMIC CODE						
Model	**CR**	**CM**	**Accidental Eccentricity**	**Point of Application of the Design Forces**	**Max / Avg Displacement (< 1.2)**	**Regular**
6S	1500	1500	No	1500	**1.00**	**Yes**
			Yes	1650 1350	1.24 1.24	**No**
6S15	1500	1050	No	1050	1.73	**No**
			Yes	1620 900	**1.19** 1.98	**No**
6A	1050	1050	No	1050	**1.00**	**Yes**
			Yes	1200 900	1.22 1.28	**No**
6A15	1050	1500	No	1500	1.60	**No**
			Yes	2070 1350	2.12 1.42	**No**

Table 6.2.3 Horizontal regularity criterion of the EC8 (1993) seismic code. (All values are in centimetres, and the locations of CM and CR are given relatively to their distance from frame 1).

Similar to NZS regularity criterion (Table 6.2.2), none of the models investigated satisfy this EC8 horizontal regularity criterion when the accidental eccentricity is included while, for TU models, this criterion is never satisfied either with or without the accidental eccentricity. However, for SM and TB models, this provision is satisfied only when the accidental eccentricity is excluded while, by including the accidental eccentricity, the ratio of the maximum over the average displacement is higher, but close to, the 1.2 limit value. Therefore, this criterion can also be considered strict since it is not even satisfied by the reference models, when analysed with the accidental eccentricity.

(c) Regularity requirements of the UBC seismic code

In the UBC (1994) code, the static method of analysis can be applied to regular structures under 73m in height (Section C.1) and the highest (24-storey) models examined in this study are close to that limit (77m). Based on the UBC provisions, regular structures are those that satisfy the configuration requirements for vertical and plan regularity. All the models employed in this study satisfy the vertical regularity requirements (Section C.2.1) since all floor levels of the models have the same stiffness (stiffness regularity), the same mass (mass regularity), and the same horizontal dimension of the lateral force-resisting system (vertical geometric regularity).

HORIZONTAL REGULARITY CRITERION OF THE UBC (1994) SEISMIC CODE						
Model	CR	CM	Accidental Eccentricity	Point of Application of the Design Forces	Max / Avg Displacement (< 1.2)	Regular
6S	1500	1500	No	1500	1.00	Yes
			Yes	1650 1350	1.24 1.24	No
6S15	1500	1050	No	1050	1.73	No
			Yes	1500 900	1.00 1.98	No
6A	1050	1050	No	1050	1.00	Yes
			Yes	1200 900	1.22 1.28	No
6A15	1050	1500	No	1500	1.60	No
			Yes	1650 1050	1.76 1.01	No

Table 6.2.4 Horizontal regularity criterion of the UBC (1994) seismic code. (All values are in centimetres, and the locations of CM and CR are given relatively to their distance from frame 1).

The models employed have no re-entrant corners, no variations in stiffness and no out-of-plane offsets of the vertical elements (see Section C.2.2). However, the torsional regularity requirement requiring that the maximum storey drift at one end of the structure computed by including accidental torsion should be less than 1.2 times the average of the storey drifts of the two ends of the structure is not satisfied. This condition is identical to the EC8 plan regularity condition (Table 6.2.3) and only the reference models without the inclusion of the accidental eccentricity satisfy it (Table 6.2.4).

Consequently, the static provisions stipulated by all seismic codes investigated are generally applicable for almost symmetric structures with respect to the mass distribution, stiffness distribution and floor plan layout. Nevertheless, the equivalent static method and the static torsional provisions are employed for all TU models employed. In this manner, the performance of the static torsional provisions, the efficiency of the equivalent static method, and the validity of such strict regularity conditions will be investigated.

6.2.2 Accidental Eccentricity Provisions

The strength distribution of the models may be determined by two alternative methods. In the first method, the accidental eccentricity component is ignored while, in the second method, by strictly adhering to the seismic code provisions, the accidental eccentricity is included. As seen in Section 5.4.3, the accidental eccentricity alters significantly the strength distribution of the models and changes their inelastic seismic response. Each seismic code adopts a different procedure to calculate the first and the second (alternative) design eccentricities (Table 6.2.5).

In the UBC code, accounting for the effect of asymmetry is a two-step procedure. In the first step, uncertainties concerning the location of the horizontal forces are accounted for by displacing their line of action from CM in each direction by an accidental eccentricity equal to 5% of the building dimension b perpendicular to the direction of excitation. The second step checks for torsional irregularity and the accidental amplifier A_x is calculated based on the results of the elastic analysis of the first step (refer to Section 4.2.2). The factor A_x is dependent on the rotational structural deformation under the application of the equivalent static forces (Section C.3.5). When A_x

is higher than unity, torsional irregularity exists and the accidental eccentricity component is equal to $0.05A_xb$. The interpretation of the second design eccentricity is based on the general UBC design requirement, which requires that the forces applied on the structural elements should not be reduced due to torsional effects (Table 4.2.1).

FIRST & SECOND DESIGN ECCENTRICITY FOR ALL SEISMIC CODES								
Model	CR	CM	EC8		NZS		UBC ($A_x = 1$)	
S	1500	1500	1650	1350	1800	1200	1650	1350
S15	1500	1050	1620	900	1350	750	1500	900
S30	1500	600	1170	450	900	300	1500	450
A	1050	1050	1200	900	1350	750	1200	900
A15	1050	1500	2070	1350	1800	1200	1650	1050
A30	1050	1950	2520	1800	2250	1650	2100	1050
B	600	600	750	450	900	300	750	450
B15	600	1050	1620	900	1350	750	1200	600
B30	600	1500	2070	1350	1800	1200	1650	600
B45	600	1950	2360	1800	2250	1650	2100	600

Table 6.2.5 First and second design eccentricity calculated for all seismic codes. (All the values are in centimetres and the locations of the design eccentricities are given relatively to frame 1).

For the resisting elements located at the flexible side of stiffness-eccentric models, the '+' sign governs and the first design eccentricity produces the largest force on the flexible side elements, which is higher than the force applied on the same elements in the TB model. Whereas, for the stiff side elements, the '-' sign controls and gives the highest force on the elements under consideration. For static eccentricities exceeding the accidental eccentricity component, the strength of the stiff-edge element is less than the strength obtained in the TB model. For these cases, the design eccentricity is assumed to be zero, the lateral forces are applied at CR, and the forces on the stiff side are equal to those calculated for the reference models. Therefore, in contrast to the EC8 and NZS codes, the UBC standard does not permit any strength reduction for the structural elements of the TU models due to the favourable effects of torsion. This restriction applies also to the strength of the elements computed for A_x equal to unity.

The static provisions of the NZS and EC8 codes stipulate a constant accidental eccentricity, which in the NZS code is equal to $0.1b$ while in the EC8 code is equal to $0.05b$. The NZS code specifies the design forces to be applied through a horizontal axis in the direction of the earthquake ground motion, at a distance $\pm 0.1b$ from CM (Table 6.2.5). The advantage in defining the design eccentricity in this manner is that there is no need to determine the location of the centre of stiffness and the procedure for the strength calculation of each element is greatly simplified.

The first design eccentricity of the EC8 code includes an additional eccentricity e_1 to account for the dynamic or coupling effects resulting from simultaneous torsional and translational structural oscillations. The eccentricity e_1 is defined as the lower of the two values calculated by the Equations (B.3.1) and (B.3.2). By incorporating this additional eccentricity, the first design eccentricity of the EC8 code is dependent on critical structural parameters, such as static eccentricity, torsional and lateral stiffness, and floor plan dimensions, and hence, is dependent on the structural configuration of each model analysed (Table 6.2.5).

The variation of the design eccentricities of the three seismic codes (normalised to the floor plan dimension b of the model) vs. the static eccentricity is presented in Figure 6.2.1. The accidental eccentricity component is incorporated and, therefore, the design eccentricities are calculated by strictly adhering to the static code provisions. As expected, the design eccentricity is lowest for structures with low static eccentricity. The first design eccentricity of the NZS and UBC codes (for A_x=1) are proportional to the static eccentricity and they are represented with two straight lines. The EC8 first design eccentricity is not a straight line due to the inclusion of the additional eccentricity component and it depends on the static eccentricity and on the model characteristics. The highest first design eccentricity is calculated based on the EC8 code and the lowest is the UBC first design eccentricity calculated for A_x equal to unity.

The second design eccentricities of the EC8 and NZS codes are proportional to the static eccentricity of the models. The second design eccentricity of the UBC code is the same as the second design eccentricity of the EC8 code for static eccentricities lower than 0.05. For static eccentricities greater than 0.05, the second design eccentricity of the UBC code is always zero and the lateral forces are applied at CR. Thus, the design forces applied on the structural elements of the TU models are at least equal, or greater than, the design forces applied on the corresponding structural elements of the reference models and no strength reduction is permitted due to the favourable effects of torsion.

In Figure 6.2.1, the first and second design eccentricities of the UBC code are calculated without including the amplification factor A_x and they depend only on the model static eccentricity. The amplification factor depends on the torsional resistance of the structure and, thus, each model type results in a different amplification factor.

6.3 DIFFERENT DEFINITIONS APPLIED TO THE MODELS

The contradictory conclusions of previous studies examining the inelastic seismic response of TU models are mainly attributed to the differences in the definition of the models. The basic factors giving rise to inconsistencies is the inclusion, or not, of the accidental eccentricity component in the design of the structures (Section 5.4.3) and the design procedure adopted for the element strength calculation (Section 5.4.1). For the inelastic analyses of this study, both the reference and the TU models are designed based on the following four definition cases. In the first case, the models are designed by excluding the minimum reinforcement ratios and the accidental eccentricity provisions of the seismic codes ("No min – No ea"). In the second case, the accidental eccentricity is incorporated in the design of the models while the minimum reinforcement provisions are still excluded ("No min – Ea"). In the third case, the minimum steel ratios are included while the accidental eccentricity is excluded ("Min – No ea"). Finally, in the fourth case, both the minimum steel ratios and accidental eccentricity requirements are incorporated in the design of the models ("Min – Ea").

The influence of different definition cases in the inelastic seismic response of the reference models is investigated by examining the inelastic response of the TB model 6B.

In this model, both CM and CR are located at a distance equal to 6.0 metres from frame 1 (Table 6.2.1) and e_s is equal to 0.30*b*. A TB reference model is employed since the response of these models is highly influenced by the strength distribution and the design method employed (refer to Section 5.5.1). The model is analysed to the NZS regulations since the NZS base shear is lower than the EC8 base shear and higher than the UBC base shear (Table 5.3.1). Moreover, the NZS torsional provisions are the simplest to apply, without incorporating an additional eccentricity component e_I, as in the EC8 code, or an amplification factor A_x, as in the UBC code (Table 4.2.1). For these reasons, the NZS code has been employed for most of the parametric analyses carried out in this chapter.

STRENGTH DISTRIBUTION OF MODEL 6B				
Frame	**No min – No ea**	**No min – Ea**	**Min – No ea**	**Min – Ea**
1	1.00	1.13	1.05	1.16
2	1.00	1.00	1.32	1.32
3	1.00	1.13	1.34	1.37
4	1.00	1.26	1.36	1.44
5	1.00	1.40	1.38	1.53
6	1.00	1.55	1.61	1.75
Total	**1.00**	**1.17**	**1.17**	**1.26**

Table 6.3.1 Strength distribution of model 6B designed to the NZS seismic code regulations for all the definition cases.

Table 6.3.1 presents the strength distribution of model 6B designed to all the definition cases and normalised to its strength when designed without the accidental eccentricity and the minimum reinforcement ratios ("No min – No ea"). Hence, the values correspond to the strength increase of each frame introduced by the inclusion of the accidental eccentricity and/or the inclusion of the minimum steel ratios. The strength increase of each frame results in a proportional reduction of its inelastic seismic response, and the strength ratios presented in Table 6.3.1 help in a better understanding of the inelastic dynamic behaviour of code-designed reference models.

The inelastic response of all the frames of model 6B is identical for the "No min – No ea" case (Figure 6.3.1) since the strength distribution of the model is proportional to its lateral stiffness distribution (see also Section 5.4.1). The inclusion of the accidental eccentricity ("No min – Ea") results in a different inelastic response for each frame, and their ductility demand is reduced proportionally to their strength increase introduced by the accidental eccentricity (Figure 6.3.1). The strength ratios presented in the second column of Table 6.3.1 indicate that the accidental eccentricity provisions do not influence the strength of frame 2 (located at CR) while the strength of the other frames is increased proportionally to their distance from the design forces. The strength increase of each frame results in a proportional reduction of its inelastic response (Figure 6.3.1) while the inelastic response of frame 2 remains the same. The strength increase of frame 6 due to the accidental eccentricity is the highest (equal to 55%) and, hence, the reduction of its inelastic response is the highest as well.

The influence of the inclusion of the minimum steel ratios in the design of the models (2nd design method) is quantified by comparing the inelastic response of model 6B designed with both methods (Figures 6.3.1 and 6.3.2). In Figure 6.3.2, the inelastic response of frame 1 is the highest due to the low strength increase caused by the minimum steel ratios (3rd and 4th columns of Table 6.3.1). The response of the frames is reduced proportionally to their strength increase while the response of the upper columns is reduced more. This is due to the fact that each frame of the 6-storey models has a uniform column section over all floor levels and the lowest element strengths from the results of the elastic analyses are located at the higher storeys. Thus, the inclusion of the minimum steel ratios increases most the strength of the upper columns.

The strength increase of each frame of model 6B due to the accidental eccentricity requirements can be observed more clearly when the 1st design method is employed (Table 6.3.1). The strength increase of the structural elements introduced by the minimum steel ratios reduces the influence of the accidental eccentricity. For example, for the 1st method, the strength increase of frame 6 due to the accidental eccentricity is equal to 55% (2nd column of Table 6.3.1) while, for the 2nd method, the strength increase of the same frame is only equal to 8% (3rd and 4th columns of Table 6.3.1).

STRENGTH DISTRIBUTION OF MODEL 6B45				
Frame	**No min – No ea**	**No min – Ea**	**Min – No ea**	**Min – Ea**
1	1.00	1.25	1.90	1.91
2	1.00	1.00	1.35	1.35
3	1.00	1.09	1.04	1.11
4	1.00	1.13	1.00	1.13
5	1.00	1.15	1.00	1.15
6	1.00	1.17	1.00	1.17
Total	**1.00**	**1.16**	**1.27**	**1.36**

Table 6.3.2 Strength distribution of model 6B45 designed to the NZS seismic code regulations for all the definition cases.

Similar to the reference models, there are four definitions that can be adopted for the TU models and their influence is examined by analysing model 6B45 to the NZS regulations. Table 6.3.2 presents the strength of each frame of model 6B45 designed for all definition cases and normalised to the strength of the same frame designed for "No min – No ea". Figure 6.3.3 illustrates the inelastic response of model 6B45 designed with the 1st design method and, in contrast to the TB case, even when the accidental eccentricity is excluded, each frame responds differently. Frame 1 experiences the highest inelastic response, with a response pattern completely different from the response patterns of the other frames. However, when the accidental eccentricity is included (Figure 6.3.3), the response of the frames is reduced, and especially frame 1 with the highest strength increase due to the accidental eccentricity (Table 6.3.2). As also noticed by Correnza et al. (1992a), the proportional reduction of the ductility demand of the frames due to the inclusion of the accidental eccentricity (equal to 50% for frame 1) is higher than the strength increase induced by the accidental eccentricity (equal to 25% for frame 1). This indicates that the dynamically responding systems are more efficient in dissipating earthquake-induced vibrations than a straightforward linear relation between ductility demand and strength would indicate.

The influence of the inclusion of the minimum steel ratios in the design of model 6B45 is illustrated in Figure 6.3.4 and its inelastic response is compared with the response of the model when designed without the minimum reinforcement ratios (Figure

6.3.3). When the 2nd design method is adopted, the response of the model is made much more uniform across the frames and the ductility demand of frames 1 and 2 is significantly reduced due to the high strength increase induced by the minimum steel ratios (Table 6.3.2). The inclusion of the accidental eccentricity ("Min – Ea") reduces the response of frames 3 – 6 while frames 1 and 2 are not influenced due to the significant strength increase already induced by the minimum steel ratios.

All these remarks regarding the influence of the accidental eccentricity and of the minimum reinforcement requirements will be also noticed in the following sections of this chapter were the influence of different factors in the inelastic seismic response of the TU models is analytically investigated. Furthermore, since the influence of these factors results in similar changes in the response of both columns and beams, the inelastic behaviour of the column elements is mainly presented in the figures of this study.

6.4 INELASTIC SEISMIC RESPONSE OF TU MODELS WITH DIFFERENT STATIC ECCENTRICITIES

The amount of the static eccentricity provides the most fundamental influence on the inelastic response of the TU models and it is examined in this section by presenting the torsional behaviour of TU models with the same structural configuration but with different static eccentricities. The systems investigated are the stiffness-eccentric model 6B30 and the stiffness/mass-eccentric models 6B15 and 6B45 (see also Table 6.2.1 and Figure 3.4.3), designed to the NZS code regulations for the same reasons indicated in Section 6.3.1. Both design methods (with and without the minimum steel ratios) are employed and the effect of the accidental eccentricity is examined as well.

The strength distribution of the TU models investigated is given in Table 6.4.1 in order to better understand and explain their inelastic seismic response. The strength ratios presented in the first column of each definition case correspond to the strength of each frame normalised to the strength of frame 6 of model 6S15 designed for "No min – No ea" since this frame has the lowest strength. These strength ratios provide information regarding the influence of the amount of the static eccentricity. The strength ratios given in the second column of each case correspond to the strength of each frame normalised to the strength of the same frame designed for "No min – No ea". These ratios indicate the strength increase of the frames due to the inclusion of the minimum steel ratios and/or the incorporation of the accidental eccentricity provisions.

STRENGTH DISTRIBUTION OF MODEL 6B15								
Frame	**No min – No ea**		**No min – Ea**		**Min – No ea**		**Min – Ea**	
1	6.95	1.00	8.07	1.16	7.83	1.13	8.57	1.23
2	1.10	1.00	1.11	1.00	1.47	1.34	1.47	1.34
3	1.31	1.00	1.45	1.11	1.53	1.17	1.61	1.23
4	1.52	1.00	1.84	1.21	1.66	1.09	1.90	1.25
5	1.76	1.00	2.25	1.28	1.83	1.04	2.26	1.29
6	1.00	1.00	1.33	1.33	1.05	1.05	1.34	1.34
Total	**13.65**	**1.00**	**16.05**	**1.18**	**15.38**	**1.13**	**17.15**	**1.26**
STRENGTH DISTRIBUTION OF MODEL 6B30								
Frame	**No min – No ea**		**No min – Ea**		**Min – No ea**		**Min – Ea**	
1	5.32	1.00	6.39	1.20	7.32	1.38	7.58	1.42
2	1.10	1.00	1.10	1.00	1.48	1.35	1.48	1.35
3	1.53	1.00	1.69	1.10	1.66	1.09	1.78	1.16
4	2.01	1.00	2.34	1.16	2.04	1.01	2.35	1.17
5	2.50	1.00	2.99	1.20	2.50	1.00	2.99	1.20
6	1.50	1.00	1.84	1.22	1.50	1.00	1.84	1.22
Total	**13.97**	**1.00**	**16.35**	**1.17**	**16.51**	**1.18**	**18.00**	**1.29**
STRENGTH DISTRIBUTION OF MODEL 6B45								
Frame	**No min – No ea**		**No min – Ea**		**Min – No ea**		**Min – Ea**	
1	3.84	1.00	4.80	1.25	7.30	1.90	7.31	1.91
2	1.09	1.00	1.10	1.00	1.48	1.35	1.48	1.35
3	1.77	1.00	1.94	1.09	1.84	1.04	1.98	1.11
4	2.52	1.00	2.84	1.13	2.52	1.00	2.84	1.13
5	3.26	1.00	3.75	1.15	3.26	1.00	3.75	1.15
6	2.01	1.00	2.35	1.17	2.01	1.00	2.35	1.17
Total	**14.49**	**1.00**	**16.78**	**1.16**	**18.41**	**1.27**	**19.72**	**1.36**

Table 6.4.1 Strength distribution of models 6B15, 6B30, 6B45 designed to the NZS seismic code regulations for all the definition cases.

Figure 6.4.1 presents the rotational ductility demand of the columns of models 6B15, 6B30 and 6B45 designed for the "No min – No ea" case. The inelastic response of frame 1 is usually the highest for all models examined, with values of ductility demand exceeding unity in all floor levels. The increase of the static eccentricity increases the ductility demand of frame 1, which is justified from the fact that the total strength of frame 1 in model 6B15 is the highest while the strength of frame 1 in model 6B45 is the lowest (Table 6.4.1). Contrary to the inelastic response of frame 1, the response of the other frames is reduced when the static eccentricity is increased. The strength ratios of frames 3 – 6 increase in models with higher static eccentricities since the design forces are located at CM, closer to these frames (Table 6.2.5). Furthermore, the highest ductility demand of frame 1 is always found at the lower storeys while the highest ductility demand of the rest of the frames is located at the upper storeys. The difference between the inelastic response of frame 1 and the rest of the frames increases in models with higher static eccentricities since the difference between the strength of frame 1 and the other frames increases as well. Therefore, the frame with the highest ductility demand is frame 1 of model 6B45 and the frame with the lowest ductility demand is frame 2 of the same model. The response of model 6B15 is more uniform than the response of the other two models since the design seismic forces are found closer to the stiff edge of this models and they result in a considerably higher strength for frame 1 (Table 6.4.1).

The inclusion of the accidental eccentricity ("No min – Ea") reduces the response of all the frames and, in particular, the response of frame 1 in model 6B45 (Figure 6.4.2) due to the high strength increase induced by the accidental eccentricity (Table 6.4.1). The accidental eccentricity influences in a different way each model depending on the points of application of the design forces. Similar to the results presented in Figure 6.4.1, the difference between the response of frame 1 and the rest of the frames increases for higher static eccentricities and the inelastic response of model 6B15 is more uniform. All the remarks made above regarding the "No min – No Ea" case still apply for the "No min – Ea" case while the maximum ductility demand values are reduced.

The inclusion of the minimum steel ratios reduces the response of all the frames and, in particular, the response of frame 1 in model 6B45 due to its high strength increase, which is equal to 90%. The strength ratios of Table 6.4.1 indicate that the minimum steel ratios influence each model differently depending on the existing strength distribution of the frames. Hence, the minimum steel ratios influence all the frames of model 6B15, frames 1 – 4 of model 6B30, and frames 1 – 3 of model 6B45 and they result in a similar total strength of frame 1 in all models. The ductility demand of frame 1 is now the highest only in the lower storeys (Figures 6.4.3 and 6.4.4) while, in the upper storeys, the response of frames 3 – 6 is higher than the response of frame 1. Finally, the "Min – Ea" case results in the highest strength increase (Table 6.4.1) and, consequently, in the lowest ductility demand values (Figure 6.4.4).

When the minimum steel ratios are excluded from the design of the structural elements, the total strength of all the models is similar while, by including the minimum steel ratios, the total strength of models with higher static eccentricities is higher as well (Table 6.4.1). The increase of the static eccentricity decreases the response of the frames, except the upper storeys of frames 3 – 6 (Figures 6.4.3 and 6.4.4). Thus, although the strength of frame 1 is the highest in model 6B15, its inelastic response is the highest since the total strength of this model is the lowest when the minimum steel ratios are included (Table 6.4.1). Consequently, TU models with high values of static eccentricity result in a low inelastic response due to the high strength increase induced by the minimum steel ratios. The highest value of ductility demand is found in frame 1 of model 6B15 while the lowest value of ductility demand is located in frame 2 of model 6B45. The reduced response of the upper storeys of frames 4 – 6 in model 6B15 designed for "Min – No ea" (Figure 6.4.3) is justified from the fact that only in this model the inclusion of the minimum steel ratios increases the strength of frames 4 – 6 (Table 6.4.1).

The inclusion of the accidental eccentricity reduces the response of all the frames, except the response of frame 2 located at CR. When the minimum steel ratios are excluded, the accidental eccentricity increases significantly the strength of the frames depending on the point of application of the seismic forces. However, when the minimum steel ratios are included in the design of the structural elements, the strength of frames is not significantly increased by the accidental eccentricity since the minimum steel ratios have already increased their strength. Thus, the influence of the accidental eccentricity can be better observed when the minimum steel ratios are not incorporated. Moreover, irrespective of the design method adopted and the inclusion, or not, of the accidental eccentricity, the maximum ductility demand of frame 1 is located in the lower storeys while, in frames 3 – 6, it is found in the upper storeys.

Figure 6.4.5 presents the time-history displacement of the models excited by the "La Union" earthquake and designed for "Min – Ea". There are only minor differences between the time-history displacement of a model when designed to different definition cases while the "Min – Ea" case is chosen since it is adopted for the design of real structures. In Figure 6.4.5, the response of the two external frames of all the models is presented and it can be noticed that the time-history displacement of frame 6 is always higher since frame 1 is stiffer and stronger. The highest peak values of lateral displacement are found in frame 6 of model 6B45 due to the higher rotation induced in this model with the highest static eccentricity. The response of frame 1 decreases with the increase of the static eccentricity while the response of frame 6 increases and, therefore, the difference between the response of frame 1 and 6 increases as well.

Figure 6.4.6 presents the maximum lateral displacement of the models excited by the earthquake records selected and designed for "Min – Ea". This lateral displacement corresponds to the maximum response value of all the frames of each model excited by each earthquake record. The highest lateral displacement is found in frame 6 of each model (Figure 6.4.5) and, therefore, the maximum displacement of each storey represents the displacement of frame 6 (Figure 6.4.6). Each model is influenced differently by each earthquake record, and different earthquake records result in the maximum and minimum values of lateral displacement for each model investigated. The model with the highest lateral displacement is model 6B45, the model with the higher static eccentricity.

Similar conclusions are reached by examining the maximum interstorey drift ratio of the models excited by the ensemble of the earthquake records (Figure 6.4.7) and designed for "Min – Ea". The values of the maximum drift ratio are similar for all the models investigated (especially in the lower storeys) and they represent the interstorey drift ratios of frame 6 of the models. For all the models examined, the maximum drift ratios are significantly lower than the 2% limit ratio, which has been set as the collapse limit of about three-quarters of RC buildings (refer to Section 4.7.1).

6.5 INELASTIC SEISMIC RESPONSE OF TU MODELS WITH DIFFERENT STIFFNESS DISTRIBUTIONS

The influence of the stiffness distribution of different model types is examined by presenting the inelastic response of models with the same static eccentricity and different structural configurations. The models analysed are the stiffness-eccentric model 6B30, the stiffness/mass-eccentric model 6A30, and the mass-eccentric model 6S30 (Figures 3.4.1 – 3.4.3). All models are designed to the NZS code and both design methods are adopted for their strength calculation (Table 6.5.1). The strength ratios of the first column of each case represent the total strength of each frame normalised to the strength of frame 6 of model 6A30 designed for "No min – No ea" since this frame has the lowest strength. The strength ratios of the second column of each case represent the total strength of each frame normalised to the strength of the same frame designed for "No min – No ea".

Figure 6.5.1 indicates that each model type designed for "No min – No ea" responds differently depending on its stiffness and strength distribution. In the mass-eccentric model, frames 5 and 6, located the furthest away from CM, result in the highest inelastic response while, in the stiffness-eccentric models, the response of frame 1 is the highest. The response of frame 1 of model 6A30 is higher than the response of frame 1 of model 6B30 since the latter frame is stronger (2.8 times) and stiffer (Table 3.4.2) and it is located closer to the design seismic forces (Table 6.2.5).

The inclusion of the accidental eccentricity ("No min – Ea") reduces the response of all frames (Figure 6.5.2) and especially the response of frame 1 of model 6A30 due to its high strength increase (equal to 40%) (Table 6.5.1). The inelastic response of each model is different from the response of the other two models and depends on its stiffness and strength distribution. In model 6S30, frames 5 and 6 are found in the inelastic range while, in model 6A30, frames 1 and 2 respond in the inelastic range and, in model 6B30, only frame 1. The inelastic response of the internal frame 5 of model 6S30 is the highest while, in the other models, the inelastic response of the external frame 1 is the highest. This is due to the difference between the structural configuration of the mass-eccentric model and of the stiffness-eccentric models. In the mass-eccentric model, all internal frames are stiffer than the two external frames while, in the stiffness-eccentric models, frame 1 is much stiffer than frame 6. Furthermore, the total strength increase is the same in all models (equal to 17%) while the maximum strength increase in each model is located in different frames (Table 6.5.1).

STRENGTH DISTRIBUTION OF MODEL 6S30								
Frame	No min – No ea		No min – Ea		Min – No ea		Min – Ea	
1	4.82	1.00	5.69	1.18	4.82	1.00	5.69	1.18
2	6.36	1.00	7.23	1.14	6.39	1.01	7.23	1.14
3	4.65	1.00	4.92	1.06	4.96	1.06	5.17	1.11
4	3.12	1.00	3.37	1.08	4.33	1.39	4.36	1.40
5	1.68	1.00	2.36	1.41	4.25	2.54	4.29	2.56
6	1.53	1.00	2.29	1.49	2.89	1.88	2.92	1.90
Total	**22.17**	**1.00**	**25.86**	**1.17**	**27.64**	**1.25**	**29.66**	**1.34**
STRENGTH DISTRIBUTION OF MODEL 6A30								
Frame	No min – No ea		No min – Ea		Min – No ea		Min – Ea	
1	2.80	1.00	3.92	1.40	6.84	2.44	6.84	2.44
2	2.34	1.00	2.67	1.14	3.82	1.63	3.83	1.64
3	3.66	1.00	3.77	1.03	4.10	1.12	4.16	1.14
4	5.07	1.00	5.70	1.12	5.18	1.02	5.73	1.13
5	6.59	1.00	7.72	1.17	6.59	1.00	7.72	1.17
6	1.00	1.00	1.21	1.21	1.02	1.02	1.21	1.21
Total	**21.46**	**1.00**	**24.99**	**1.16**	**27.55**	**1.28**	**29.50**	**1.37**
STRENGTH DISTRIBUTION OF MODEL 6B30								
Frame	No min – No ea		No min – Ea		Min – No ea		Min – Ea	
1	7.90	1.00	9.48	1.20	10.87	1.38	11.25	1.42
2	1.63	1.00	1.64	1.00	2.19	1.35	2.19	1.35
3	2.27	1.00	2.50	1.10	2.47	1.09	2.64	1.16
4	2.99	1.00	3.47	1.16	3.03	1.01	3.48	1.17
5	3.71	1.00	4.44	1.20	3.71	1.00	4.44	1.20
6	2.23	1.00	2.72	1.22	2.23	1.00	2.72	1.22
Total	**20.73**	**1.00**	**24.26**	**1.17**	**24.50**	**1.18**	**26.73**	**1.29**

Table 6.5.1 Strength distribution of models 6S30, 6A30 and 6B30 designed to the NZS seismic code regulations for all the definition cases.

The inclusion of the minimum steel ratios reduces the response of all the frames in different models, which now result in similar response values (Figures 6.5.3 and 6.5.4). Frame 1 of model 6A30 presents the largest reduction of ductility demand since the strength increase of this frame due to the inclusion of the minimum steel ratios is the largest as well (2.44 times) (Table 6.5.1). In the stiffness-eccentric models, the frames influenced more by the minimum steel ratios ("Min – No ea") are frames 1 and 2 while, in the mass-eccentric model, frames 5 and 6 present the highest strength increase. When the minimum steel ratios are incorporated in the design of the models ("Min – Ea"), the accidental eccentricity provisions increase the strength of frames less influenced by the minimum steel ratios. Therefore, the strength of frames 1 and 2 in mass-eccentric models and the strength of frames 4 – 6 in stiffness-eccentric models present the highest increase. Furthermore, the inclusion of the minimum steel ratios (with or without the accidental eccentricity) results in a different total strength increase for each model (Table 6.5.1).

Figures 6.5.3 and 6.5.4 indicate that mass-eccentric models respond differently than stiffness-eccentric models. In models 6A30 and 6B30, frame 1 results in the highest inelastic response while, in model 6S30, frame 6 presents the highest inelastic behaviour since it has the lowest strength (Table 6.5.1). In the mass-eccentric model, the external frames have the same stiffness distribution and the design forces are located closer to frame 1, resulting in a higher strength increase for frame 1. Similar to the remarks made in Section 6.4.1, in stiffness-eccentric models, the upper storeys of the flexible side present higher ductility demand values while the maximum response values of the stiff frame are located in the lower storeys. A similar inelastic response presents the mass-eccentric model, which results in higher ductility demand values in the upper storeys of frames 1 and 2 and in the lower storeys of frames 5 and 6 (the frames with the highest inelastic response).

Figure 6.5.5 indicates that the displacement of frame 6 is always higher than the displacement of frame 1 in both stiffness-eccentric models. In the mass-eccentric model, the displacement of the two external frames is similar and the peak values of lateral displacement are located in either frame 1 or frame 6 for different earthquake records. The highest values of lateral displacement are found in frame 6 of model 6B30 while the lowest values are found in frame 1 of the same model. Consequently, the difference between the displacement of frames 1 and 6 is the highest in model 6B30. The maximum lateral displacement of the models, when excited by the ensemble of the earthquake records selected (Figure 6.5.6), indicates that the lateral displacement of each storey is the highest in the stiffness-eccentric model 6B30. Each model is influenced differently by each earthquake record, and different earthquake records result in the maximum and minimum values of lateral displacement for each model. Finally, the maximum values of interstorey drift ratio for all models are always lower than the 2% limit ratio (Figure 6.5.7) while their mean values are below 1% indicating no potential for collapse.

6.6 INELASTIC SEISMIC RESPONSE OF TU MODELS WITH DIFFERENT NUMBERS OF FLOOR LEVELS

The effect of the structural period in the response of TU models is examined by presenting the behaviour of three models with the same stiffness and mass distributions and different heights. The models studied are the stiffness/mass-eccentric models 6B45, 12B45 and 24B45 (e_s equal to 0.45b) consisting of 6, 12, and 24 storeys, respectively. The models are designed to the NZS code and their strength distribution is presented in Table 6.6.1, where the strength of each frame is normalised to its strength when designed for "No min – No ea". The inelastic response of each model is highly dependent on its fundamental period and the highest model results in the highest inelastic response, when designed for "No min – No ea" (Figure 6.6.1). The response of frame 1 is the highest and the response of frames 1 – 4 increases for higher models. The highest response values are found in frame 1 of model 24B45 with ductility demand values exceeding 25 at several floor levels. The excessive ductility demand values would not generally arise in practice since the 1st design method is only employed to observe better the influence of the accidental eccentricity and of the minimum steel ratios. Furthermore, the NZS base shear is the lowest for the 24-storey models (Table 3.4.4) and the excessive inelastic response of the 24-storey model can also be justified by the low base shear (see also Section 6.7).

Identical conclusions regarding the influence of the model height are also reached for the "No min – Ea" case (Figure 6.6.2). Although, the ductility demand values are considerably decreased (from 43 to 16 in model 24B45), the inelastic response of the models still increases for higher models, and it is mainly the ductility demand of frame 1 that exceeds unity. The influence of the accidental eccentricity in the design of TU models with different numbers of floor levels is similar since the total strength increase is the same for all models (equal to 15%) (Table 6.6.1). Furthermore, the strength increase of each frame due to the inclusion of the accidental eccentricity is also identical for all models while the strength increase of frame 1 is the highest (equal to 25%) resulting in the highest reduction of the rotational ductility demand.

STRENGTH DISTRIBUTION OF MODEL 6B45				
Frame	**No min – No ea**	**No min – Ea**	**Min – No ea**	**Min - Ea**
1	1.00	1.25	1.90	1.91
2	1.00	1.00	1.35	1.35
3	1.00	1.09	1.04	1.11
4	1.00	1.13	1.00	1.13
5	1.00	1.15	1.00	1.15
6	1.00	1.17	1.00	1.17
Total	**1.00**	**1.16**	**1.27**	**1.36**
STRENGTH DISTRIBUTION OF MODEL 12B45				
Frame	**No min – No ea**	**No min – Ea**	**Min – No ea**	**Min - Ea**
1	1.00	1.24	2.27	2.29
2	1.00	1.00	1.96	1.96
3	1.00	1.08	1.37	1.40
4	1.00	1.12	1.13	1.21
5	1.00	1.15	1.05	1.17
6	1.00	1.16	1.04	1.18
Total	**1.00**	**1.14**	**1.56**	**1.63**
STRENGTH DISTRIBUTION OF MODEL 24B45				
Frame	**No min – No ea**	**No min – Ea**	**Min – No ea**	**Min - Ea**
1	1.00	1.23	3.22	3.27
2	1.00	1.00	2.39	2.39
3	1.00	1.08	1.65	1.69
4	1.00	1.12	1.33	1.40
5	1.00	1.15	1.17	1.27
6	1.00	1.16	1.13	1.25
Total	**1.00**	**1.15**	**1.91**	**1.98**

Table 6.6.1 Strength distribution of models 6B45, 12B45 and 24B45 designed to the NZS (1992) seismic code regulations and for different definition cases.

The inclusion of the minimum reinforcement requirements reduces drastically the response of all TU models, with or without the accidental eccentricity provisions (Figures 6.6.3 and 6.6.4). Therefore, model 24B45, which resulted in the highest ductility demand, now presents the highest total strength increase due to the minimum steel ratios (Table 6.6.1) and responds similar to the other two models. Frame 1 of the highest model (model 24B45) presents the highest reduction of ductility demand due to the high strength increase of this frame (3.22 times) (Table 6.6.1). The ductility demand of frame 1 is no longer the highest of all the frames, and the flexible side of the models results in a higher inelastic response, especially in models 12B45 and 24B45. The columns with the highest ductility demand are mainly the upper columns of frames 4 – 6 since they present the lowest strength increase due to the inclusion of the minimum steel ratios (Table 6.6.1).

Figures 6.6.3 and 6.6.4 indicate that, in higher structures (models 12B45 and 24B45), the columns with ductility demand values exceeding unity are only the upper columns located in frames 4 – 6. In model 6B45, except the upper floor levels of frames 4 –6, the lower floor levels of frame 1 also exceed unity. The increase of the model height does not seem to influence significantly the ductility demand of the upper storeys of frames 4 – 6 while it reduces the ductility demand of the lower storeys of all frames. Thus, the conclusions reached in Sections 6.4 and 6.5 regarding the inelastic response of various 6-storeys models indicating that the lower storeys of the stiff frame also result in ductility demand values higher than unity exist only for 6-storey models. However, the high ductility demand values at the upper floor levels of the flexible side of stiffness-eccentric models exist in all TU models with different heights.

The time-history displacement of the TU models investigated having different numbers of floor levels (Figure 6.6.5) indicates that the 24-storey model results in the highest values of lateral displacement, while the lowest values correspond to the 6-storey model. The maximum lateral displacement for each storey of the models is illustrated in Figure 6.6.6 for the ensemble of the earthquake ground motions selected and it corresponds to the maximum lateral displacement of the flexible frame (frame 6). In model 6B45, the maximum lateral displacement of the top storey is equal to 20 cm, in model 12B45, it is approximately equal to 25 cm, while, in model 24B45, it reaches 57 cm when excited by the "3407 6th St." earthquake record. Each earthquake record influences the response of each model in a different way, and the maximum lateral displacement is caused in each model by a different record.

Finally, all the TU models examined having the same structural configuration, but different numbers of storeys, result in a maximum interstorey drift ratio lower than the 2% limit ratio (Figure 6.6.7), which in combination with the formation of a sidesway mechanism has been set as a collapse limit. The interstorey drift ratios for the 24-storey model are more spread due to the influence of the earthquake records selected but they are always less than 1%, with a mean interstorey drift value generally of around 0.4%.

6.7 INELASTIC SEISMIC RESPONSE OF TU MODELS DESIGNED TO DIFFERENT TORSIONAL PROVISIONS

The static torsional provisions of three different seismic codes (EC8, NZS and UBC codes) are incorporated in the design of the models analysed in this study. The overall seismic design methodologies of these three codes have been presented in detail in the relevant Appendices of the thesis while their torsional provisions were discussed in Sections 4.2, 5.3 and 6.2. The effect of the static torsional provisions of the seismic codes exhibited in the inelastic dynamic behaviour of the TU models is the issue examined in this section by investigating models with the same static eccentricity but different structural configurations (models 6S30, 6A30 and 6B30) (Table 6.2.1).

When the accidental eccentricity is not incorporated in the design of the structural models, their strength distribution depends only on the base shear value. For the 6-storey models, the NZS base shear is 1.4 times the UBC value, and the EC8 base shear is 1.85 times the UBC base shear (see also Table 5.3.1). Consequently, the strength of the 6-storey models is higher for the EC8 regulations and lower for the UBC regulations. By including the accidental eccentricity provisions, the strength of each frame depends not only on the values of the design seismic forces, but also on their points of application. Therefore, before examining the static torsional provisions of the codes, the influence of their design eccentricities in the strength distribution of the models is examined irrespective of the values of the design seismic forces calculated with each code.

INFLUENCE OF DIFFERENT DESIGN ECCENTRICITIES IN THE STRENGTH DISTRIBUTION OF MODEL 6S30

Frame	EC8		NZS		UBC	
	450	**1170**	**300**	**900**	**150**	**1500**
1	1.08	1.00	1.17	1.00	**1.25**	1.00
2	1.06	1.00	1.12	1.00	**1.18**	1.00
3	1.03	1.00	1.05	1.00	**1.08**	1.00
4	1.00	1.15	1.00	1.08	1.00	**1.24**
5	1.00	1.89	1.08	1.47	1.31	**2.41**
6	1.26	1.01	1.52	1.00	**1.78**	1.57
Total	1.15		1.16		1.32	

MODEL 6A30

Frame	EC8		NZS		UBC	
	1800	**2520**	**1650**	**2250**	**1050**	**2400**
1	1.18	2.16	1.49	1.61	**2.72**	1.92
2	1.07	1.00	1.14	1.00	**1.43**	1.00
3	1.00	**1.06**	1.00	1.03	1.00	1.04
4	1.00	**1.21**	1.00	1.11	1.00	1.17
5	1.00	**1.30**	1.00	1.16	1.00	1.24
6	1.00	**1.37**	1.00	1.19	1.00	1.29
Total	1.31		1.18		1.38	

MODEL 6B30

Frame	EC8		NZS		UBC	
	1350	**2070**	**1200**	**1800**	**600**	**1950**
1	1.14	1.00	1.27	1.00	**1.82**	1.00
2	1.00	1.00	1.01	1.00	**1.02**	1.00
3	1.00	**1.20**	1.00	1.10	1.00	1.16
4	1.00	**1.31**	1.00	1.16	1.00	1.24
5	1.00	**1.37**	1.00	1.20	1.00	1.29
6	1.00	**1.42**	1.00	1.22	1.00	1.33
Total	1.23		1.19		1.44	

Table 6.7.1 Strength increase of models 6S30, 6A30 and 6B30 due to different design eccentricities and identical seismic forces. (The locations of the design eccentricities are given relative to their distance from frame 1 (in cm)).

Table 6.7.1 shows the strength increase of the models investigated when the first and the second design eccentricities of each seismic code are adopted and when the design seismic forces are identical for all codes. Consequently, the strength ratios of Table 6.7.1 correspond to the strength of each frame calculated for different design eccentricities and normalised to its strength when no design eccentricities are applied and when the design seismic forces are identical. The strength distribution of the models analysed according to the design eccentricities and the seismic forces of each code is presented in Tables 6.7.2 – 6.7.4 for different definition cases. Both design eccentricities are applied and the strength of each frame is normalised to its strength when designed to the UBC code since this code produces the lowest strength values.

STRENGTH DISTRIBUTION OF TU MODELS DESIGNED TO DIFFERENT CODES
("No min – No ea")

Frame	Model 6S30			Model 6A30			Model 6B30		
	EC8	NZS	UBC	EC8	NZS	UBC	EC8	NZS	UBC
1	1.73	1.48	1.00	1.03	1.02	1.00	1.25	1.16	1.00
2	1.67	1.43	1.00	1.34	1.22	1.00	1.48	1.31	1.00
3	1.54	1.35	1.00	1.50	1.33	1.00	1.60	1.38	1.00
4	1.39	1.25	1.00	1.64	1.41	1.00	1.70	1.46	1.00
5	1.08	1.04	1.00	1.72	1.47	1.00	1.75	1.49	1.00
6	1.31	1.19	1.00	1.76	1.50	1.00	1.77	1.51	1.00
Total	**1.53**	**1.34**	**1.00**	**1.50**	**1.33**	**1.00**	**1.50**	**1.32**	**1.00**

Table 6.7.2 Strength distribution of models 6S30, 6A30 and 6B30 designed to the torsional provisions of different seismic codes ("No min – No ea").

STRENGTH DISTRIBUTION OF TU MODELS DESIGNED TO DIFFERENT CODES
("No min – Ea")

Frame	Model 6S30			Model 6A30			Model 6B30		
	EC8	NZS	UBC	EC8	NZS	UBC	EC8	NZS	UBC
1	1.51	1.40	1.00	1.18	0.80	1.00	0.97	0.97	1.00
2	1.52	1.39	1.00	1.09	1.05	1.00	1.47	1.31	1.00
3	1.48	1.33	1.00	1.54	1.32	1.00	1.72	1.36	1.00
4	1.36	1.14	1.00	1.77	1.38	1.00	1.84	1.39	1.00
5	1.14	0.80	1.00	1.85	1.39	1.00	1.88	1.40	1.00
6	1.07	1.15	1.00	1.90	1.40	1.00	1.91	1.40	1.00
Total	**1.39**	**1.24**	**1.00**	**1.54**	**1.20**	**1.00**	**1.40**	**1.18**	**1.00**

Table 6.7.3 Strength distribution of models 6S30, 6A30 and 6B30 designed to the torsional provisions of different seismic codes ("No min – Ea").

STRENGTH DISTRIBUTION OF TU MODELS DESIGNED TO DIFFERENT CODES ("Min – Ea")									
Frame	Model 6S30			Model 6A30			Model 6B30		
	EC8	NZS	UBC	EC8	NZS	UBC	EC8	NZS	UBC
1	1.49	1.37	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2	1.45	1.33	1.00	1.00	1.00	1.00	1.00	1.00	1.00
3	1.27	1.15	1.00	1.20	1.07	1.00	1.42	1.16	1.00
4	1.06	1.00	1.00	1.65	1.29	1.00	1.72	1.31	1.00
5	1.00	1.00	1.00	1.83	1.38	1.00	1.86	1.38	1.00
6	1.00	1.00	1.00	1.79	1.33	1.00	1.90	1.38	1.00
Total	**1.23**	**1.15**	**1.00**	**1.35**	**1.15**	**1.00**	**1.31**	**1.12**	**1.00**

Table 6.7.4 Strength distribution of models 6S30, 6A30 and 6B30 designed to the torsional provisions of different seismic codes ("Min – Ea").

The strength ratios of Table 6.7.1 indicate that, for model 6S30, the maximum strength of frames 1 – 3 and 6 is found when the second design eccentricity has the highest value, i.e. when it is located closer to frame 1 (the furthest away from CM). The UBC provision requiring a design eccentricity equal to $0.05bA_x$ (A_x=3) results in the highest second design eccentricity, located at 150 cm from frame 1 (Table 6.7.1). The maximum strength of frames 4 and 5 is calculated when the design forces are located closer to frame 6 (the furthest away from CM). Once more, the UBC provision requiring the strength of each frame to be at least equal to the strength of the corresponding frame of its reference model results in the highest first design eccentricity located at 1500 cm from frame 1 (Table 6.7.1). For this UBC provision, the design seismic forces are applied at CR of the TU model, which is coincident with CM of the reference model.

Although the maximum strength increase of all the frames of model 6S30 is found for the UBC design eccentricities (Table 6.7.1), the low base shear value of this code results in the lowest total strength (Tables 6.7.2 – 6.7.4). The highest strength of model 6S30 is found for the EC8 code while the lowest is calculated for UBC, indicating that the strength of the frames is mainly influenced by the base shear value. Exceptions are the strength of frame 5, which reaches its highest value for the EC8 code and its lowest for NZS, and the strength of frame 6, which is the highest for the NZS regulations and the lowest for UBC (Table 6.7.3). Although the minimum steel requirements increase the total strength of model 6S30, differences in its strength distribution can still be noticed for different static torsional provisions (Table 6.7.4). Each of the frames 4 – 6 of model 6S30 result in the same total strength when designed to any seismic code while the rest of the frames have different strength values for each seismic code employed.

In the stiffness-eccentric models 6A30 and 6B30, the highest strength of frames 1 and 2 is found for the highest value of the second design eccentricity (Table 6.7.1). This value corresponds to the UBC seismic code provision that permits no strength reduction in the TU models and, consequently, the design seismic forces are applied at CR (at 600 cm from frame 1 in model 6B30 and at 1050 in model 6A30). Moreover, the maximum total strength for frames 3 – 6 is calculated for the highest first design eccentricity, which is the first design eccentricity of the EC8 code due to the additional eccentricity component e_1. Once more, although the UBC design eccentricities result in the highest total strength for stiffness-eccentric models (1.38 for model 6A30 and 1.44 for model 6B30) (Table 6.7.1), the high base shear value of the EC8 code influences more their total strength (Tables 6.7.2 – 6.7.4). The inclusion of the minimum steel ratios influences the strength of frames 1 – 4 while the strength of frames 5 and 6 remains identical (Tables 6.7.3 and 6.7.4). The strength of frames 1 and 2 in stiffness-eccentric models is identical for all seismic codes due to the influence of the minimum steel ratios (Table 6.7.4).

When model 6S30 is designed for "No min – No ea" (Figure 6.7.1), its response pattern is similar for all seismic codes while the ductility demand values differ depending on the base shear values (Table 6.7.2). For all seismic codes, frames 5 and 6 result in the highest inelastic response while the ductility demand values increase for lower design forces. Consequently, the inelastic response of model 6S30 is the lowest for the EC8 code and the highest for the UBC code. Only frames 5 and 6 result in ductility demand values higher than unity while, for the UBC code, almost all frames exceed unity. This indicates clearly that, for the "No min – No ea" case, UBC responds worst than the other two codes due to its low base shear value (Table 6.7.2).

The inclusion of the accidental eccentricity reduces the inelastic response of frames 5 and 6 of model 6S30 (Figure 6.7.2) since these frames result in the highest strength increase due to the accidental eccentricity provisions (Table 6.7.1). Although the total strength increase of model 6S30 is the highest for the UBC design eccentricities

(32%) (Table 6.7.1), the low base shear value of this code leads to the lowest total strength of the model (Table 6.7.3) and, consequently, to the highest ductility demand values. The response of the frames is proportional to their strength and, therefore, frame 6 responds best for the NZS regulations and frame 5 for the EC8 regulations (Table 6.7.3). For the rest of the frames, only when they are designed to the UBC regulations, their rotational ductility demand exceeds unity in the 5th floor level. As a general remark, it could be said that without the inclusion of the minimum steel ratios, the rotational ductility demand of all floor levels is higher in the upper floor levels and the UBC torsional provisions result in the highest ductility demand values.

The inclusion of the minimum reinforcement requirements results in a more uniform inelastic response for model 6S30, when designed to any seismic code (Figure 6.7.3) and no major differences are caused by the inclusion of different torsional provisions. The rotational ductility demand of frames 5 and 6 exceeds unity in the lower floor levels (mainly in the 2nd storey) for all codes while frame 1 exceeds unity in the 5th floor level only when designed to the UBC code. These results are justified from the strength ratios presented in Table 6.7.4 where it can be noticed that the strength of frames 5 and 6 is identical for all seismic codes. Furthermore, the higher inelastic response of frame 1 when designed to the UBC code is justified by its low strength (Table 6.7.4).

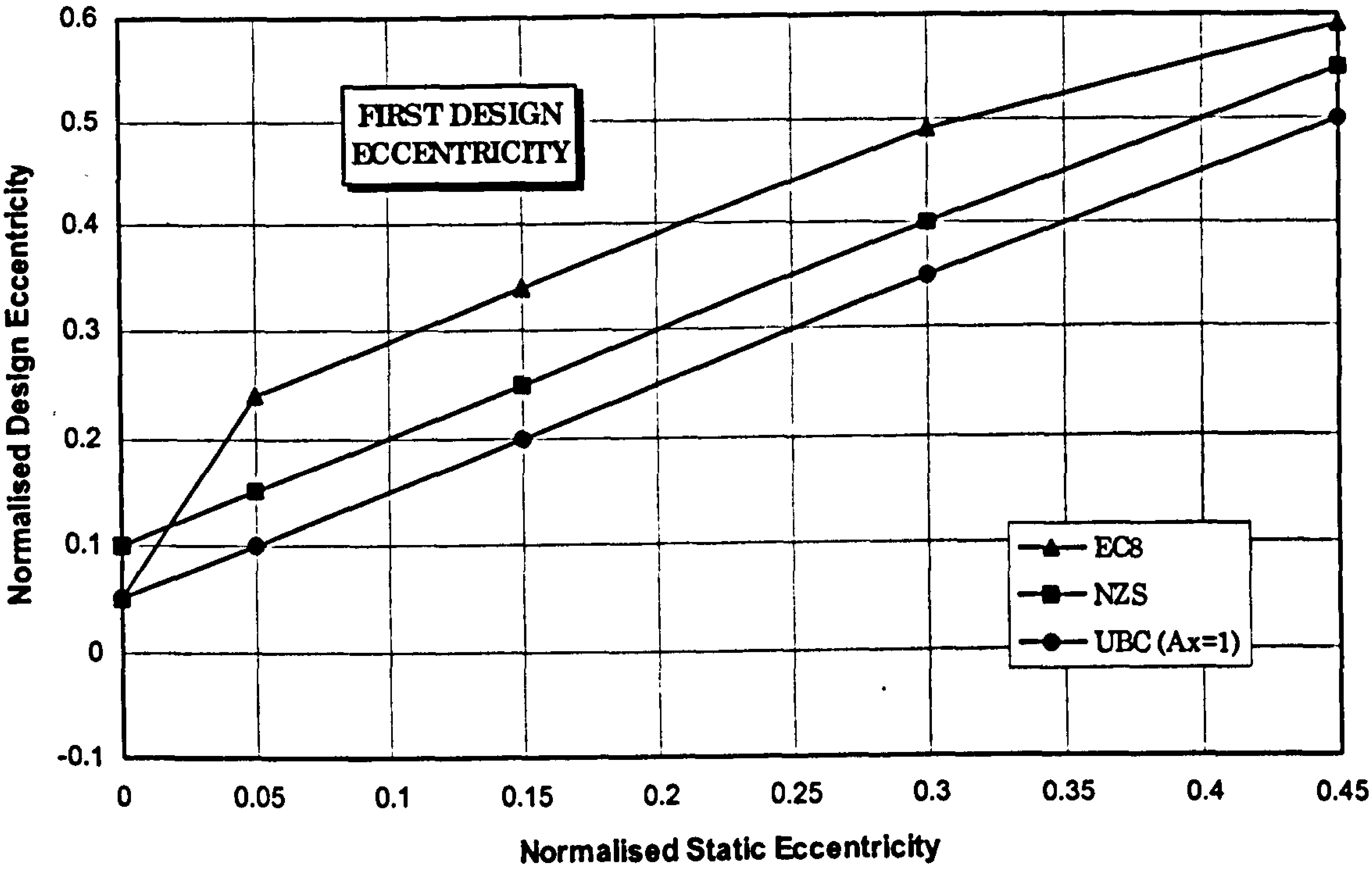
For the "No min – No ea" case, the inelastic response of the stiffness/mass-eccentric model 6A30 depends only on the design seismic forces (Figure 6.7.4) and, consequently, the lowest ductility demand values are found for the EC8 provisions and the highest for the UBC regulations. Including the accidental eccentricity ("No min – Ea") results in significantly decreased ductility demand values in frame 1 (Figure 6.7.5) since this frame presents the highest strength increase (Table 6.7.1). All frames respond best for the EC8 code while, for the UBC and NZS codes, frame 1 responds better for the UBC regulations and the rest of the frames respond better for the NZS provisions (Table 6.7.3). The increased ductility demand values in the upper storeys can also be noticed in this model, especially when designed to the UBC provisions.

The inclusion of the minimum steel requirements ("Min – Ea') results in a similar response of model 6A30 for all torsional provisions (Figure 6.7.6). The maximum response values are considerably reduced and only minor differences can be noticed in the response of the models. The ductility demand values are slightly higher when the model is designed to the UBC code due the low total strength of the model designed to this seismic code (Table 6.7.4). An increased ductility demand is found in the upper storeys of frames 5 and 6 for all seismic codes while the maximum ductility demand of frame 1 is found in the lower storeys. It should be reminded that the opposite was observed in the response of the frames of the mass-eccentric model 6S30 (Figure 6.7.3).

Finally, the inelastic response of the stiffness-eccentric model 6B30 (Figures 6.7.7 – 6.7.9) indicates that the behaviour of the stiff frame is the highest (as also seen in model 6A30). Due to the higher stiffness and strength of frame 1 in model 6B30, its response is much lower than the response of frame 1 in model 6A30 (Figures 6.7.4 and 6.7.7). When the accidental eccentricity is excluded ("No min – No ea"), the model responds best for the EC8 code and worst for UBC (Figure 6.7.7). Excessive ductility demand values are noticed in the upper storeys of frames 3 – 6, when designed to the UBC provisions.

By including the accidental eccentricity ("No min – Ea"), the model responds similar for the NZS and EC8 codes while a higher ductility demand is found for the UBC provisions (Figure 6.7.8). The ductility demand still increases in the upper storeys of frames 3 – 6, especially when the model is designed to the UBC code. The highest strength increase of the stiff frame is observed for UBC (82%) due to the provision that permits no strength reduction in the TU model (Table 6.7.1) while the lowest strength increase is found for the EC8 code (14%). Therefore, although the total strength of the model is the highest for the EC8 code (Table 6.7.3), the strength of frame 1 is the highest for the UBC code and frame 1 responds best when designed to the UBC regulations.

When the minimum reinforcement requirements are included in the design of model 6B30 ("Min – Ea"), the inelastic seismic response of the model presents no major differences for any of the three seismic codes adopted (Figure 6.7.9). The rotational ductility demand of frames 4 – 6 still increases in the upper floor levels, especially when the UBC seismic code is employed. This is also indicated from the strength ratios of Table 6.7.4 where it can be seen that the strength of frames 4 – 6 varies depending on the seismic code employed. The response values of model 6B30 are the highest when the UBC static torsional provisions are adopted due to the low total strength of the model when designed according to the static torsional provisions of the UBC code.



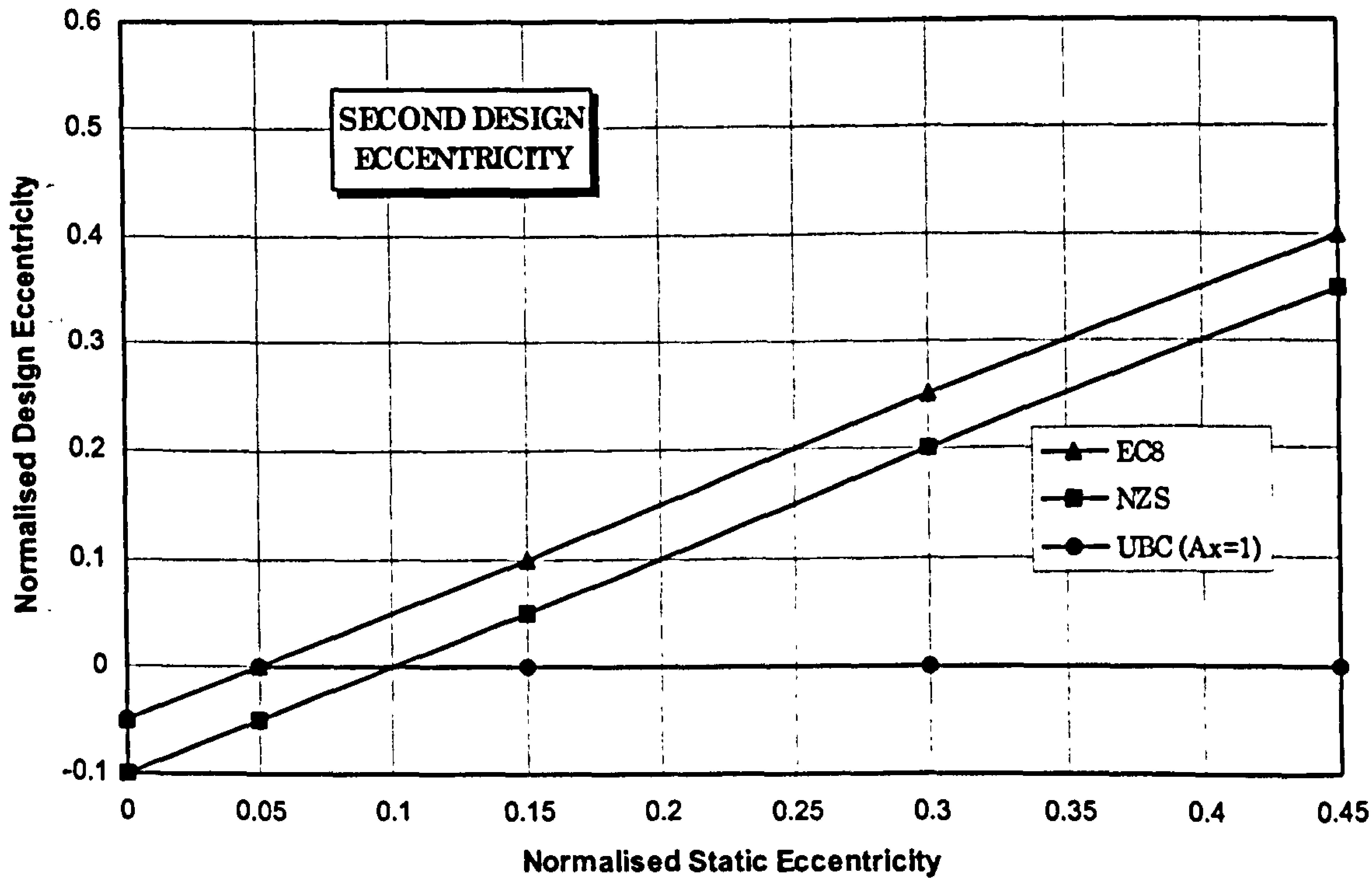
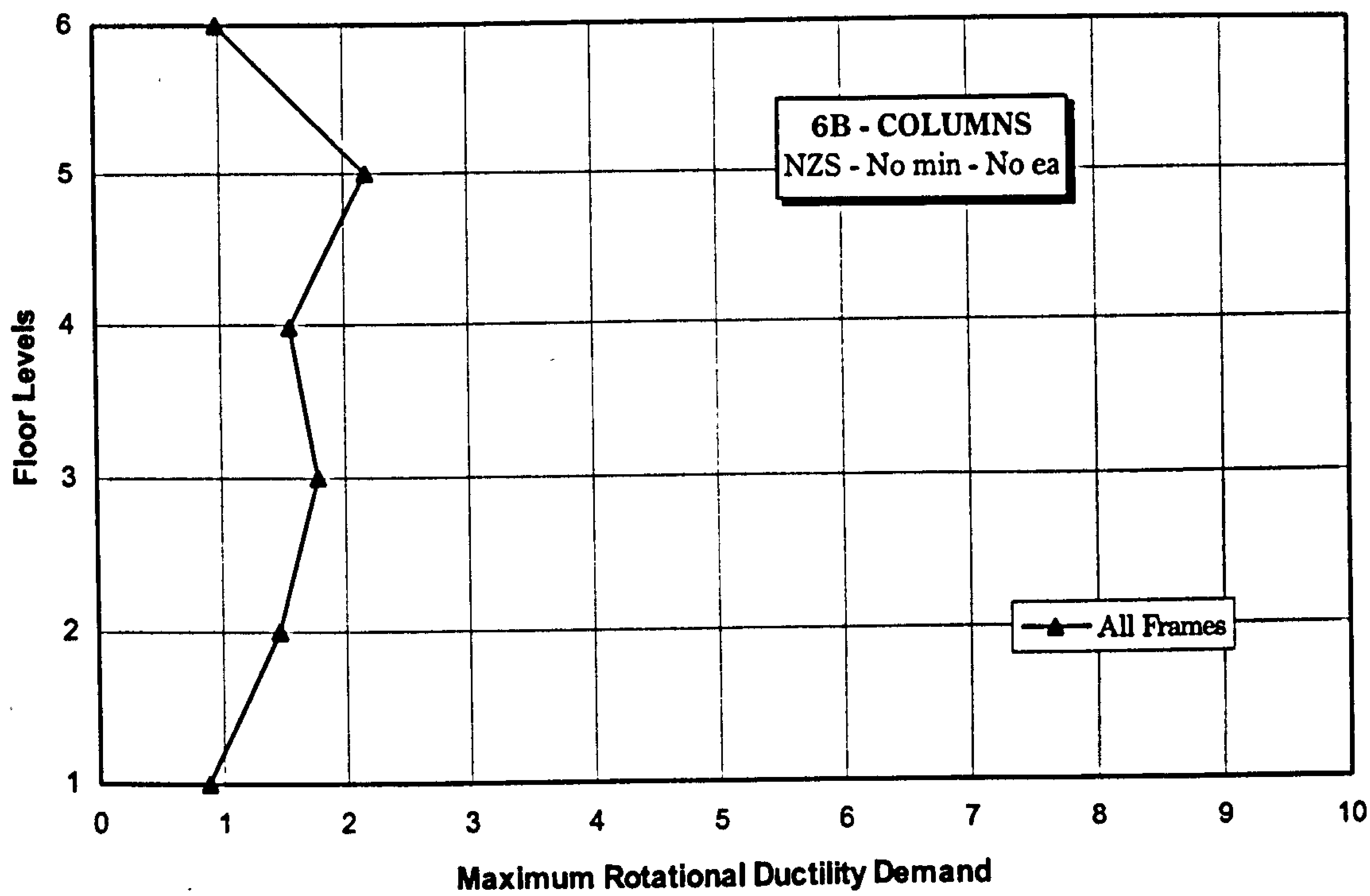


Figure 6.2.1 Normalised first and second design eccentricities calculated for all seismic codes vs. the normalised static eccentricity.



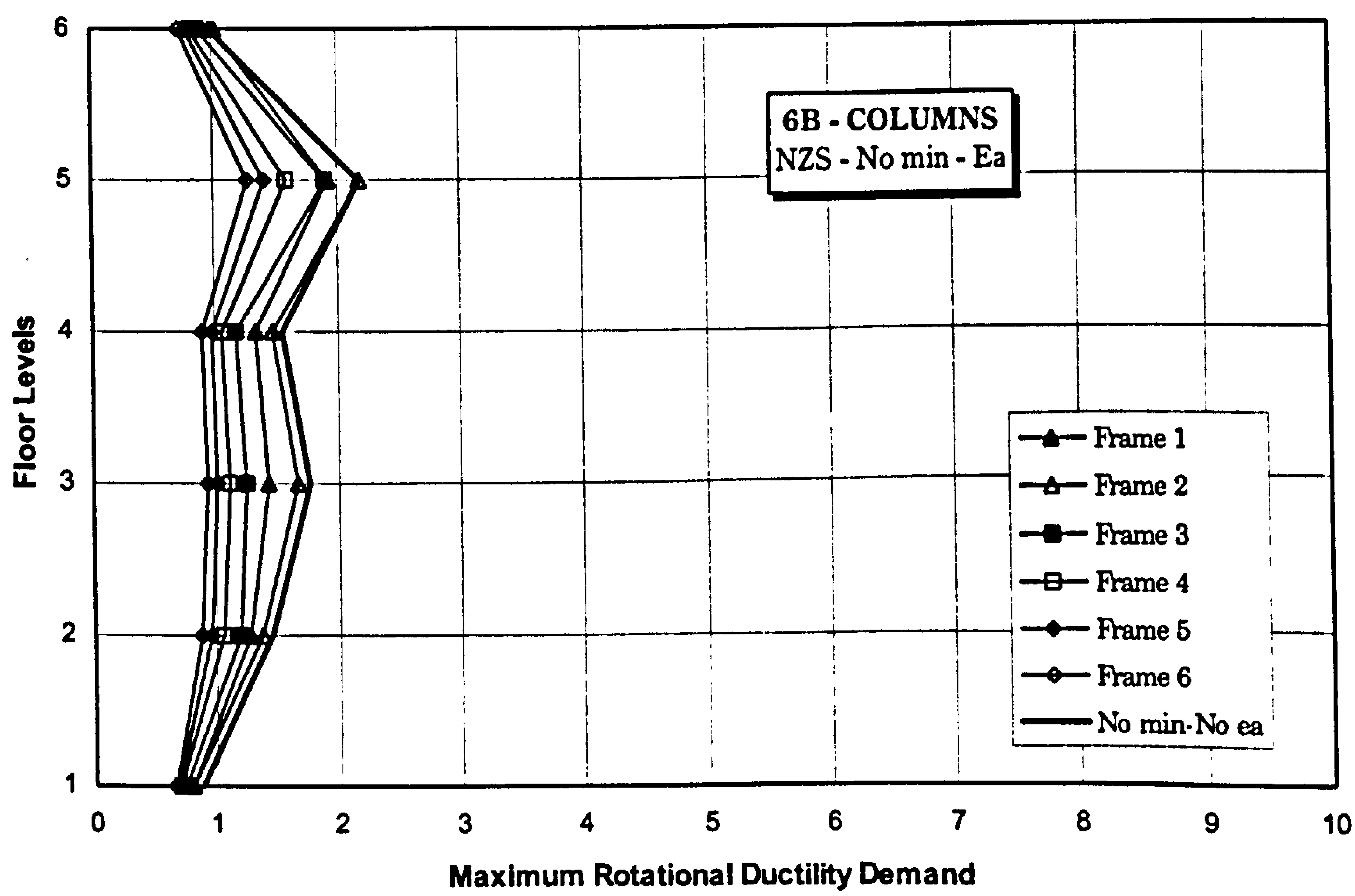
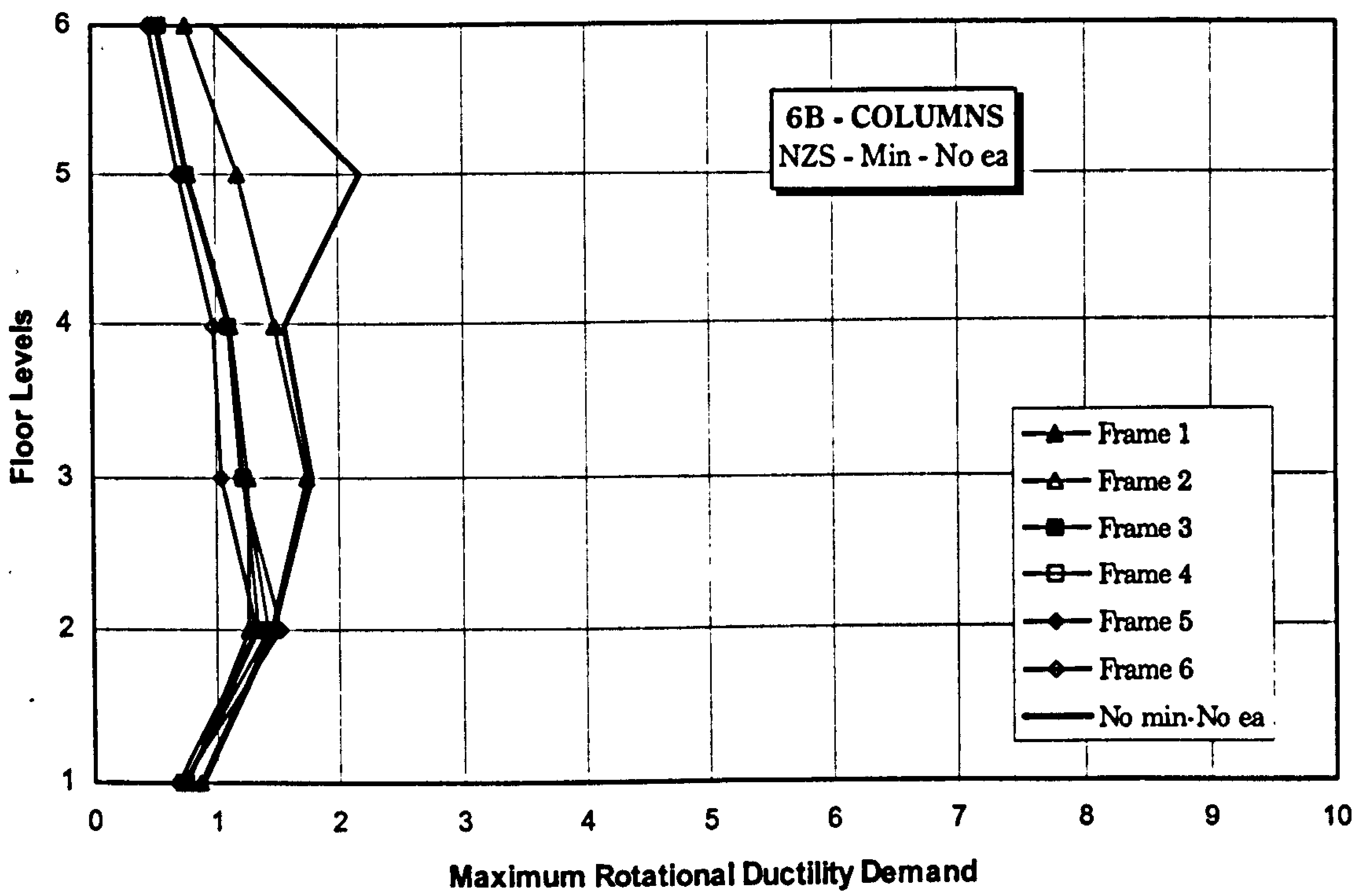


Figure 6.3.1 Rotational ductility demand vs. floor levels for the columns of model 6B designed to the NZS code and the 1st design method.



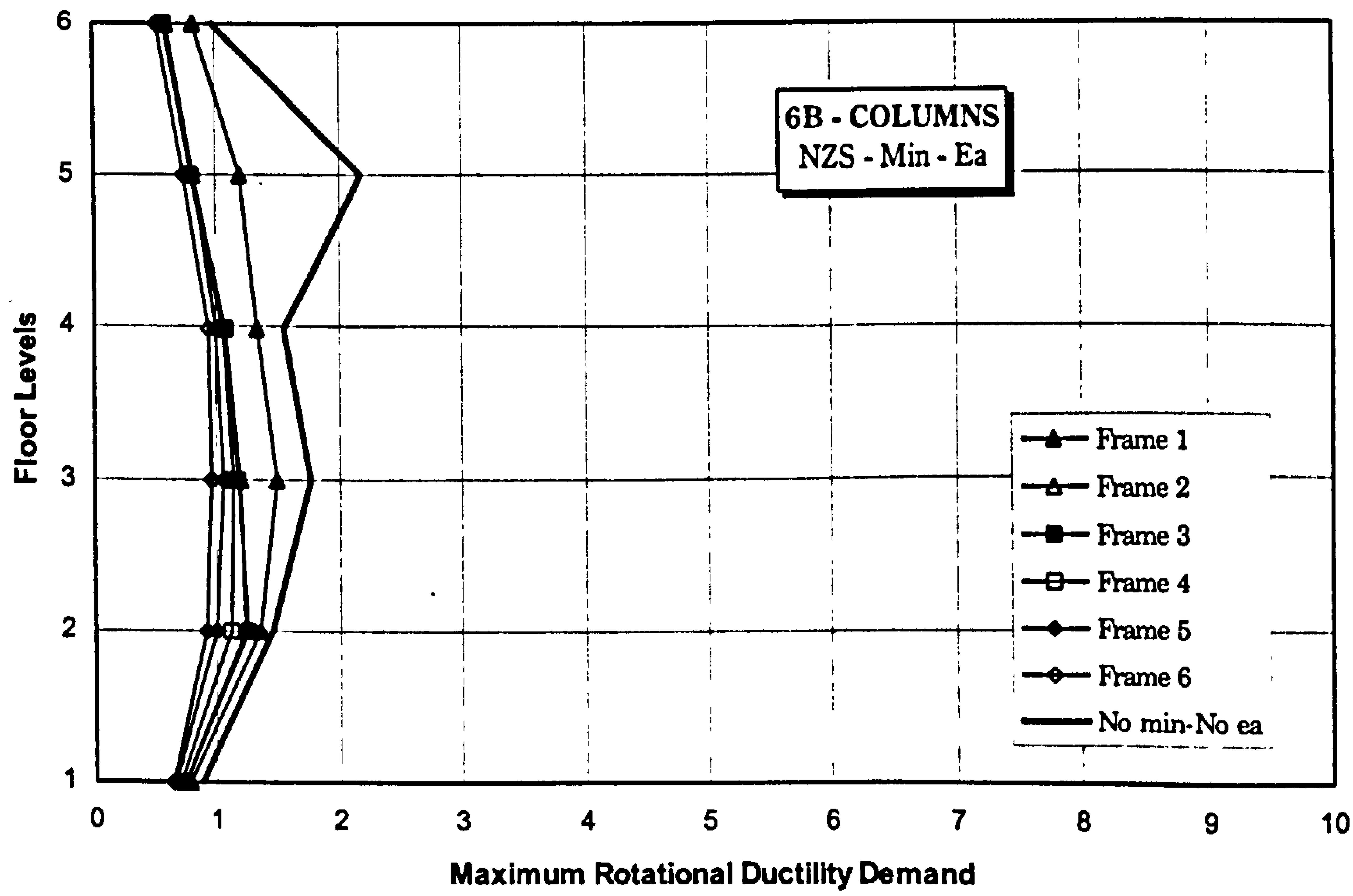
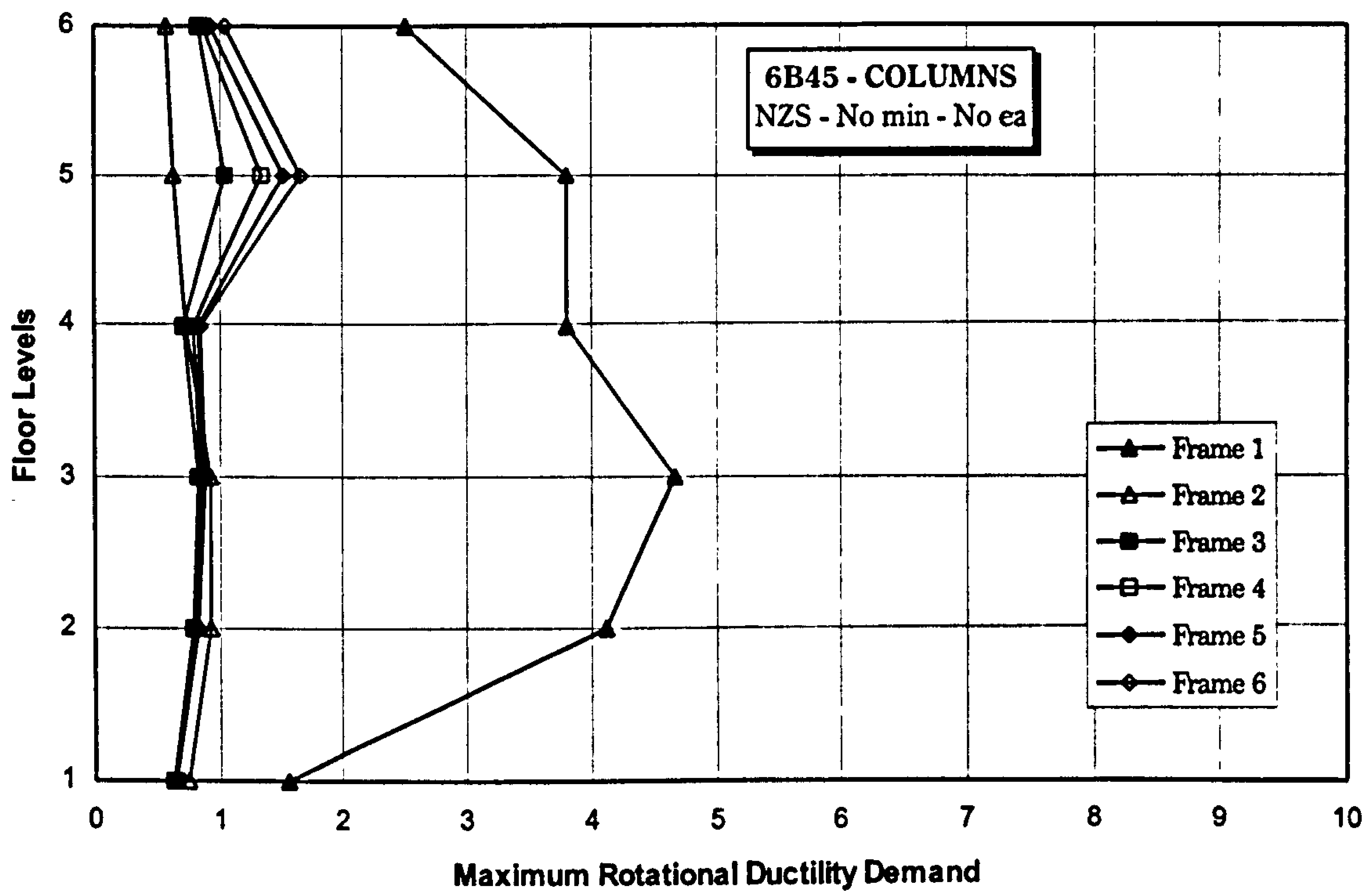


Figure 6.3.2 Rotational ductility demand vs. floor levels for the columns of model 6B designed to the NZS code and the 2nd design method.



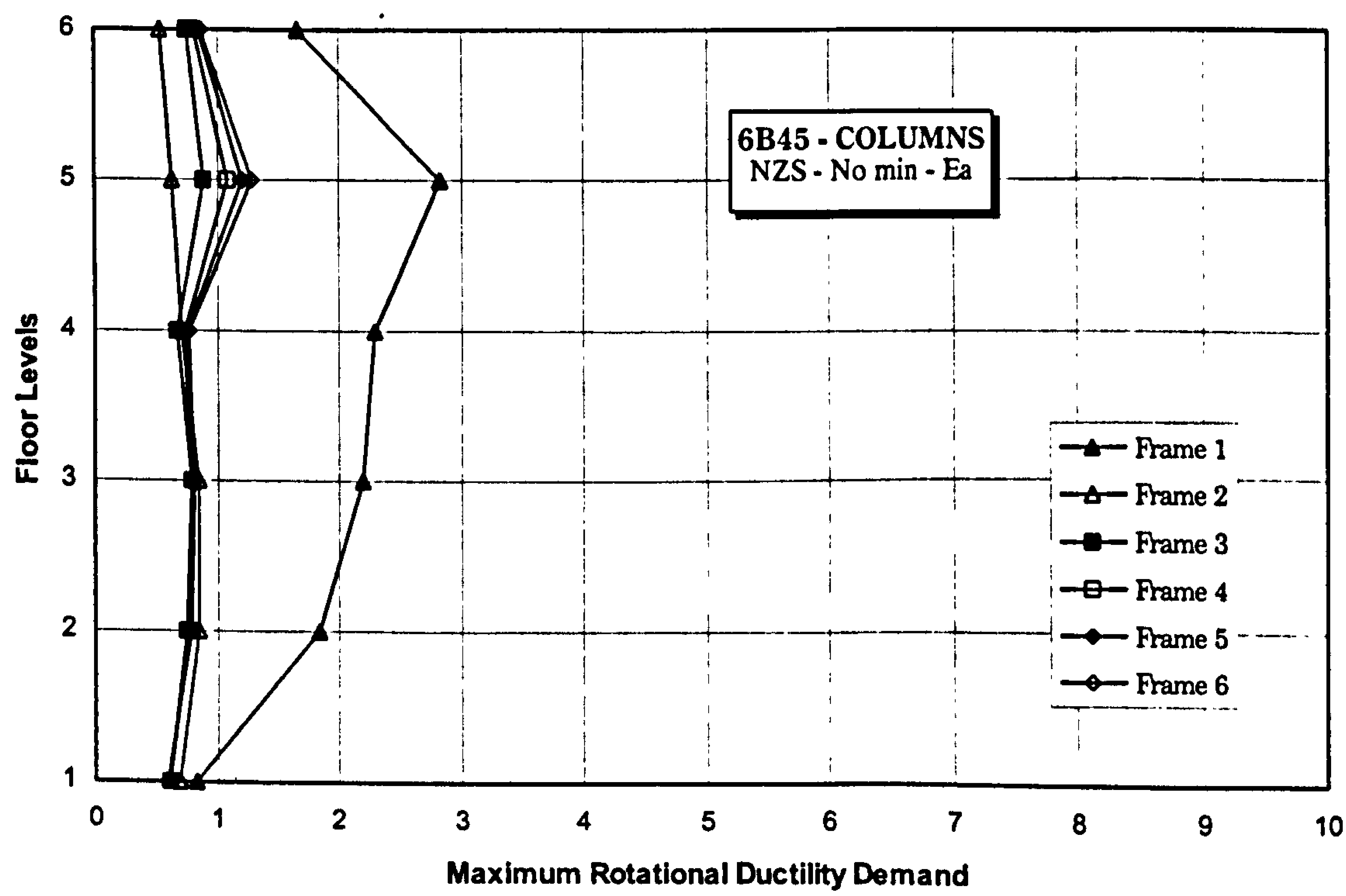
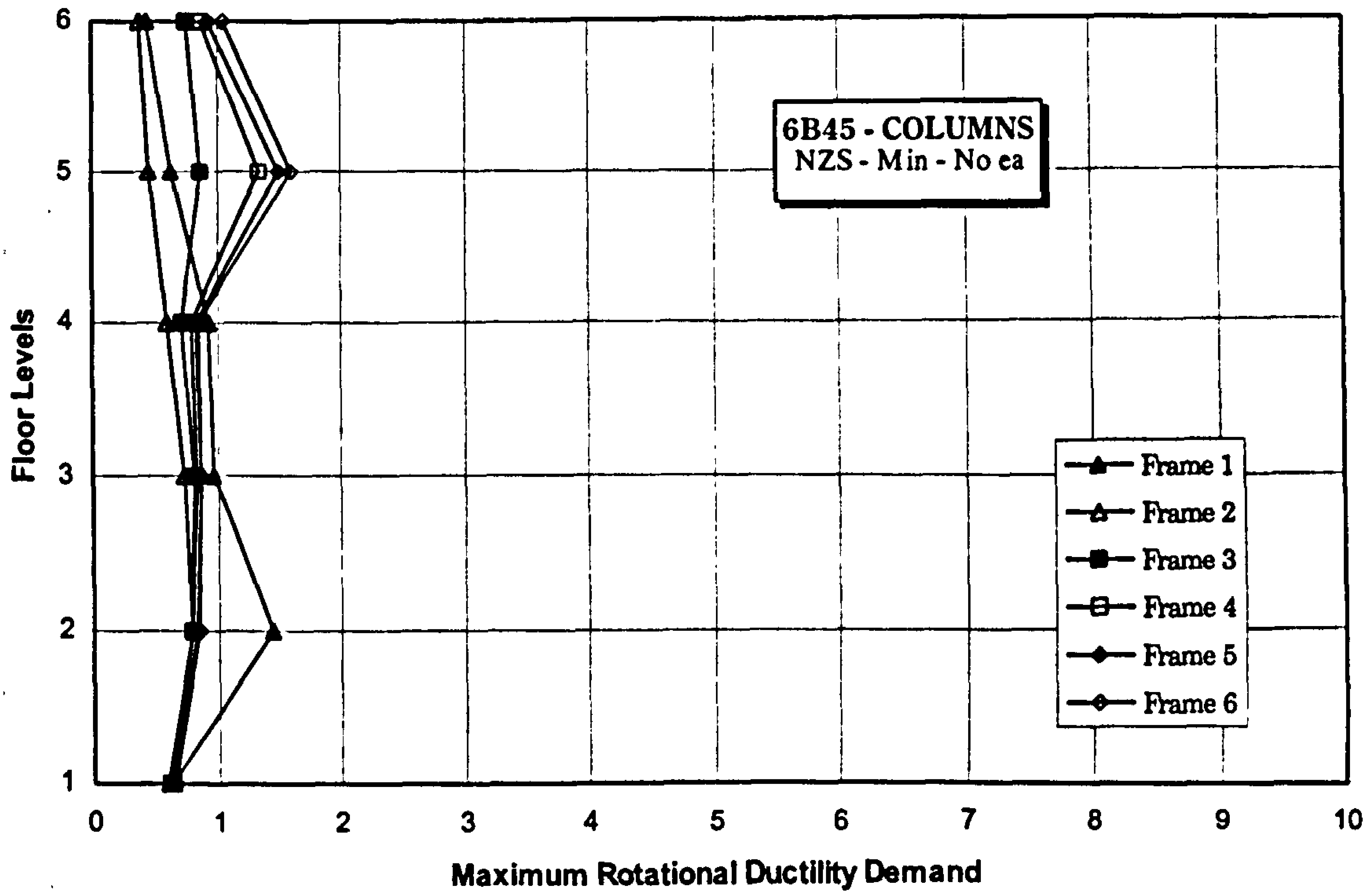


Figure 6.3.3 Rotational ductility demand vs. floor levels for the columns of model 6B45 designed to the NZS code and the 1st design method.



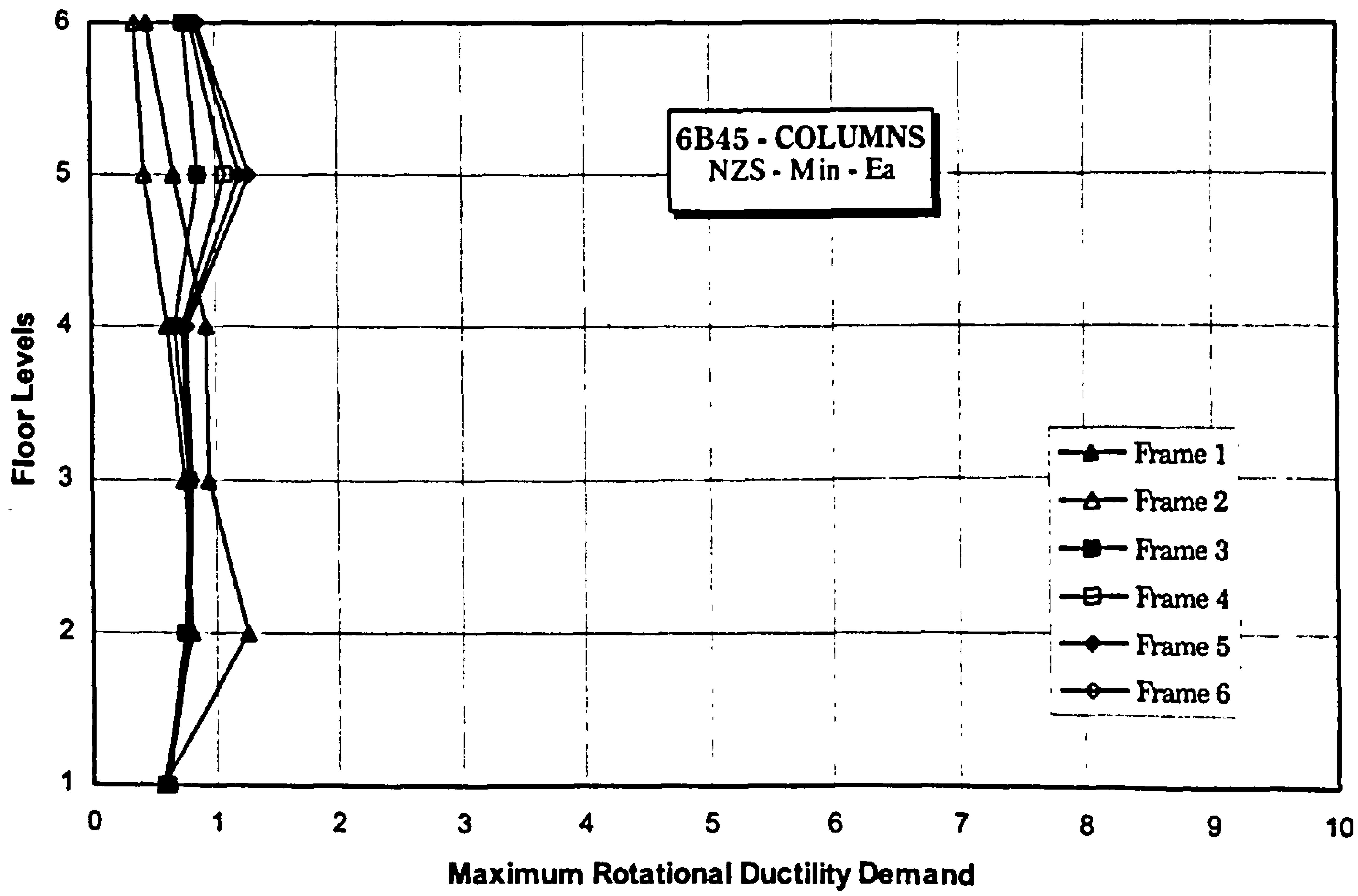


Figure 6.3.4 Rotational ductility demand vs. floor levels for the columns of model 6B45 designed to the NZS code and the 2nd design method.

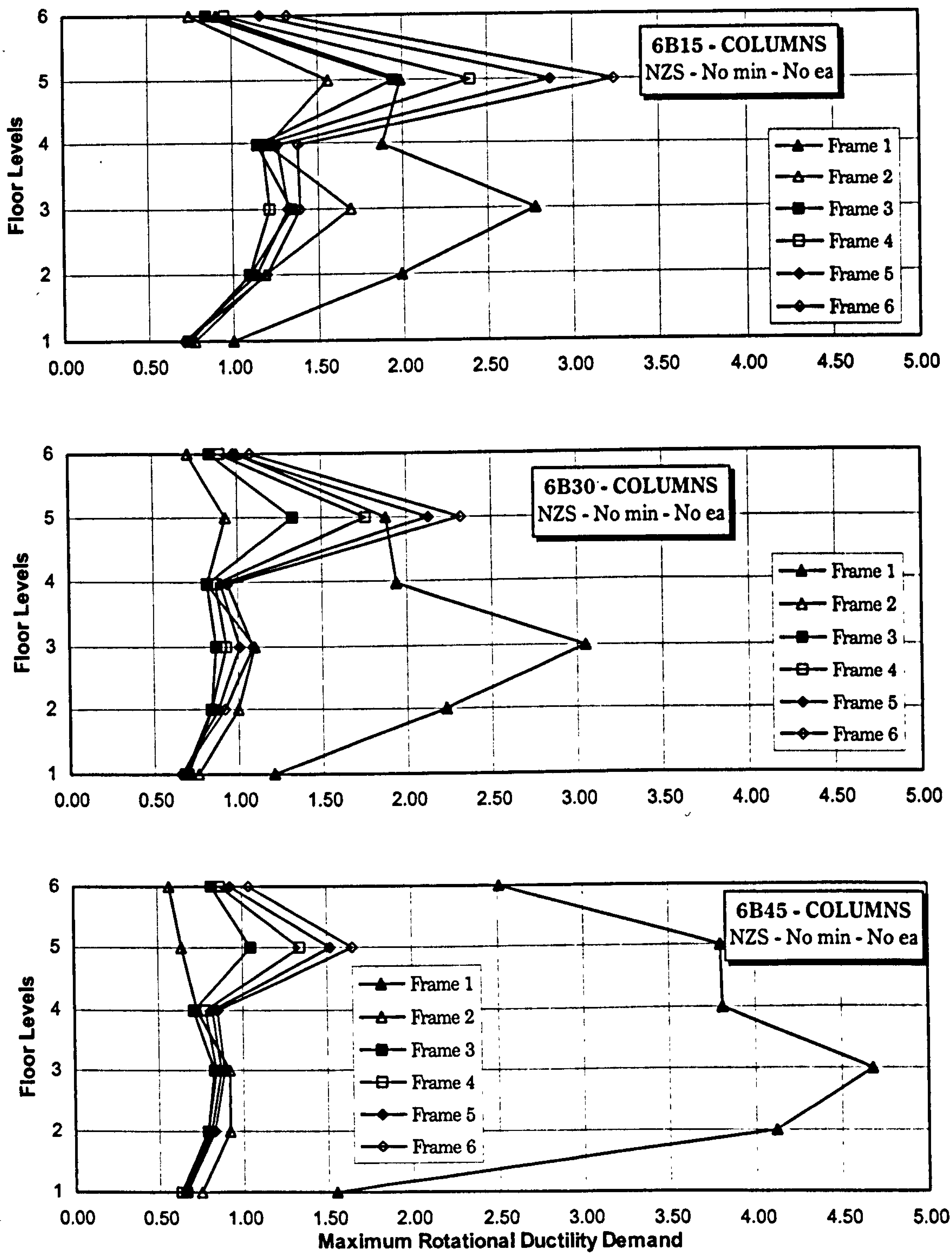


Figure 6.4.1 Rotational ductility demand vs. floor levels for the columns of models with different static eccentricities designed to the NZS code for "No min – No ea".

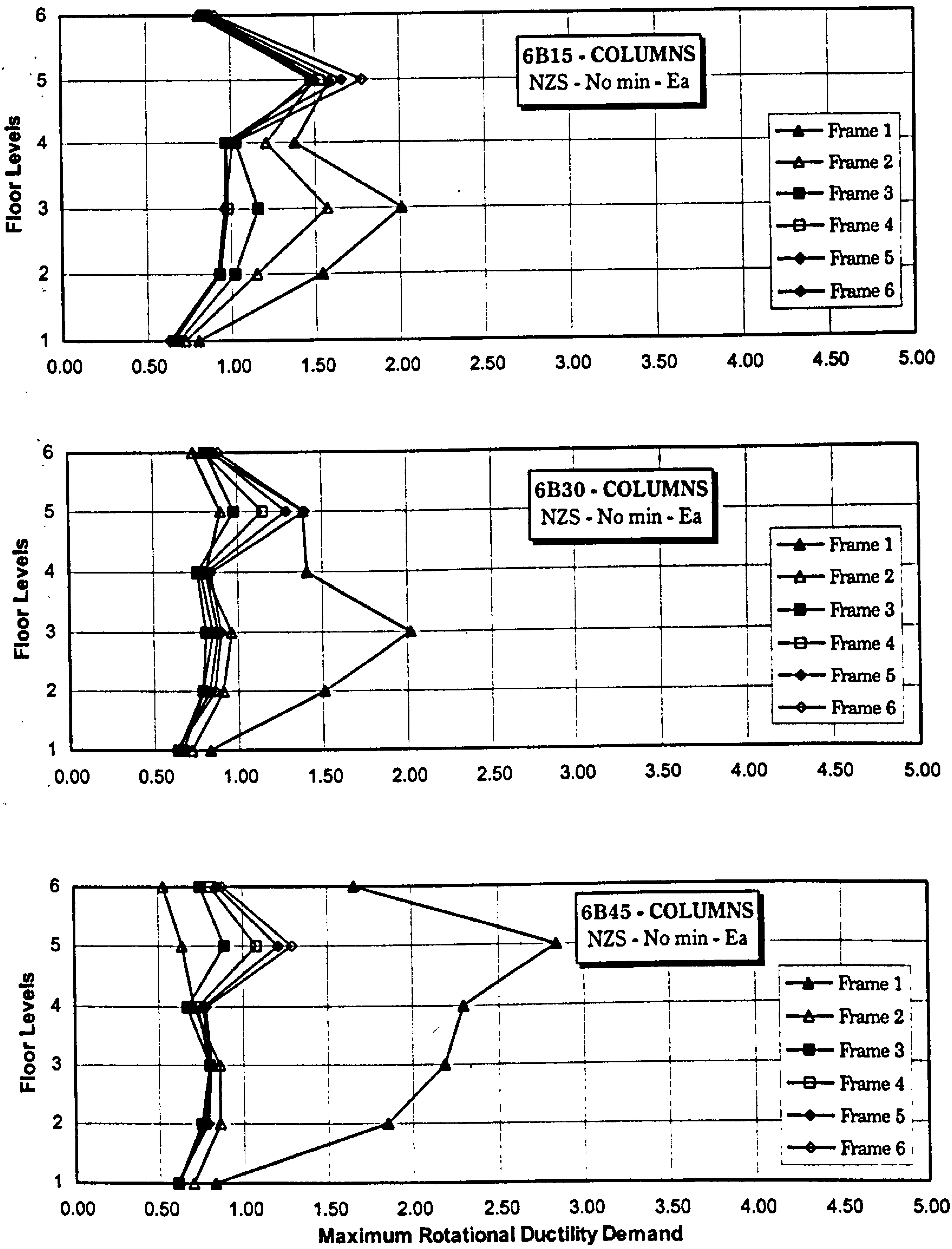


Figure 6.4.2 Rotational ductility demand vs. floor levels for the columns of models with different static eccentricities designed to the NZS code for "No min – Ea".

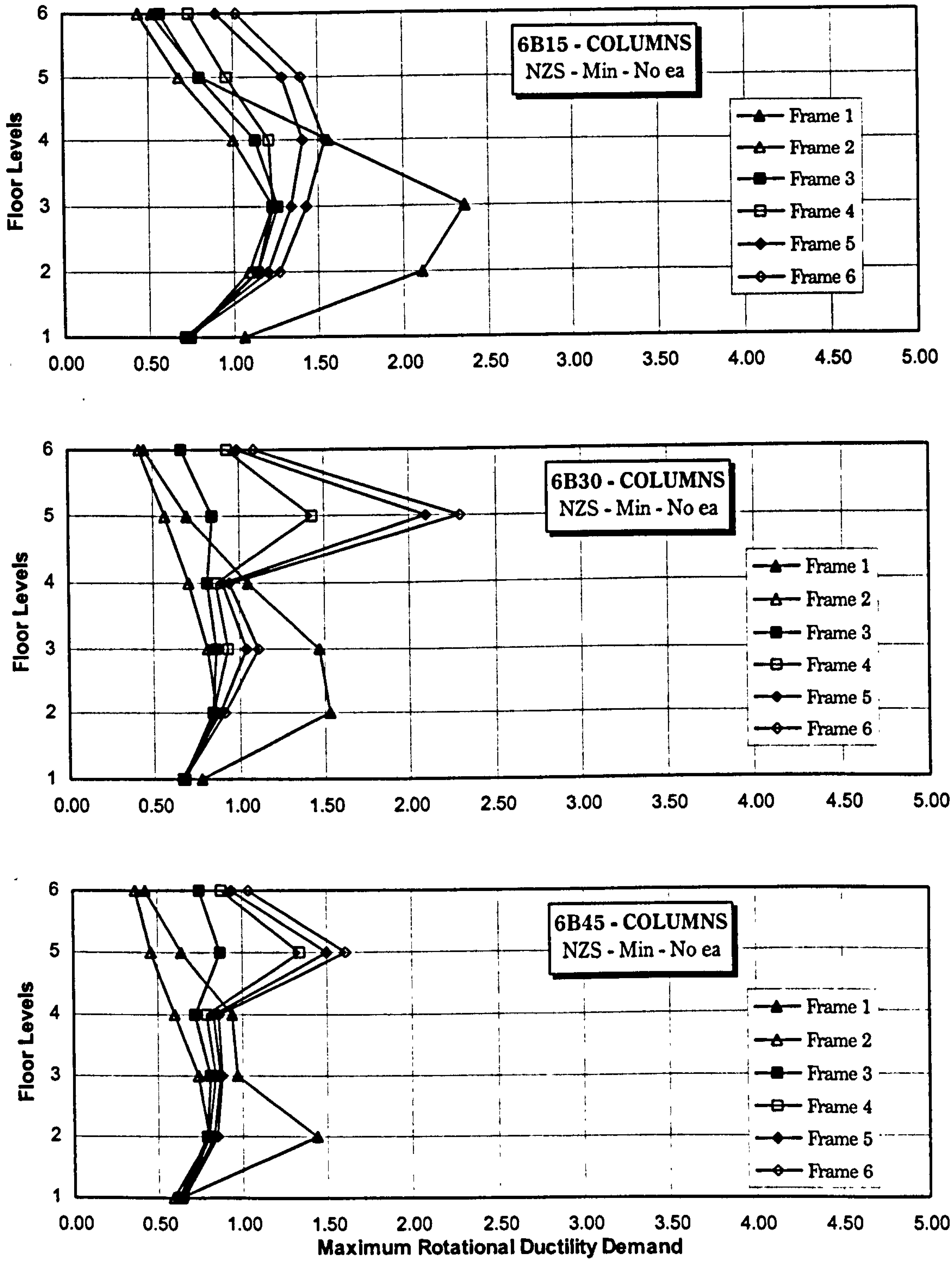


Figure 6.4.3 Rotational ductility demand vs. floor levels for the columns of models with different static eccentricities designed to the NZS code for "Min – No ea".

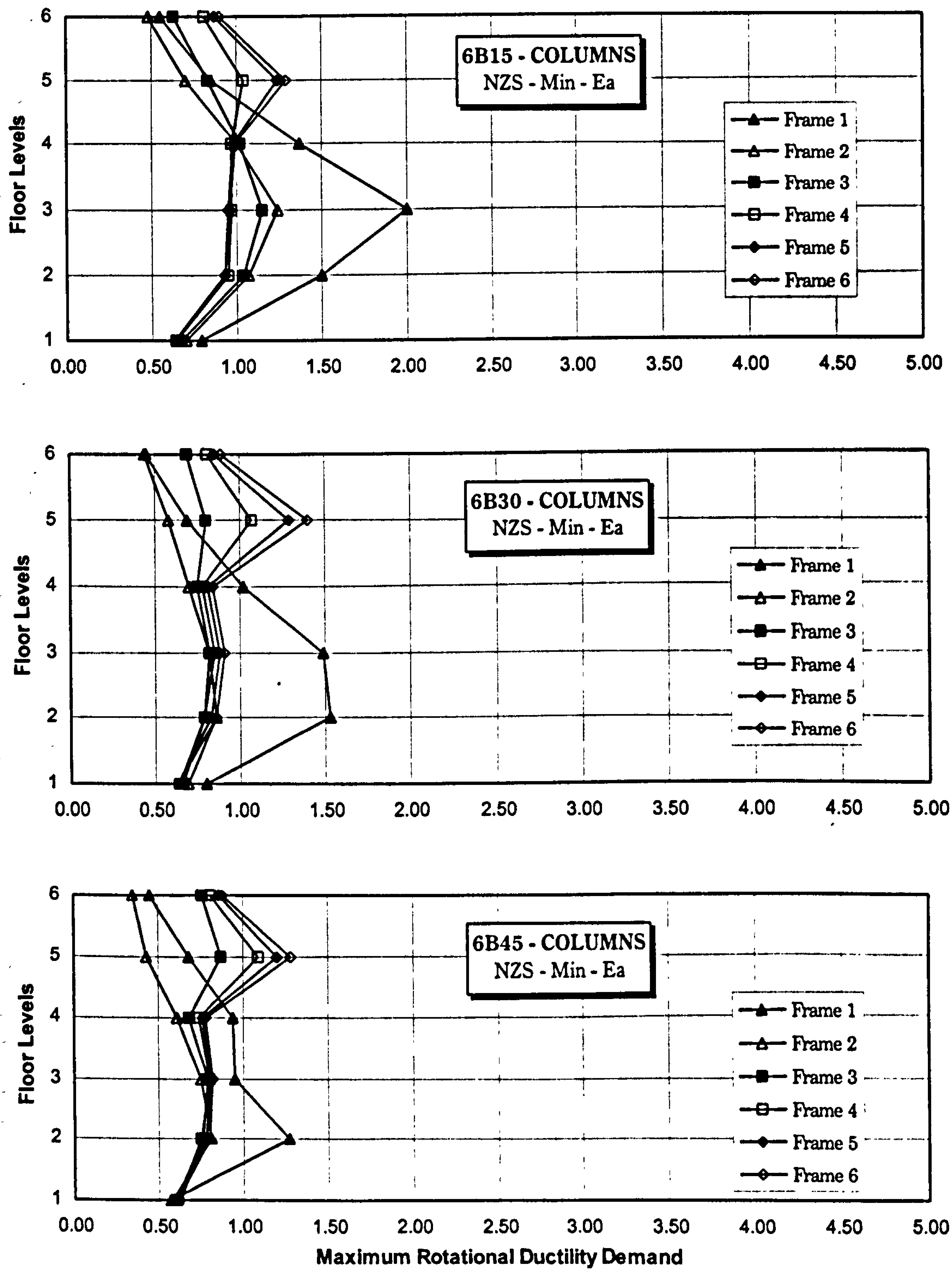


Figure 6.4.4 Rotational ductility demand vs. floor levels for the columns of models with different static eccentricities designed to the NZS code for "Min – Ea".

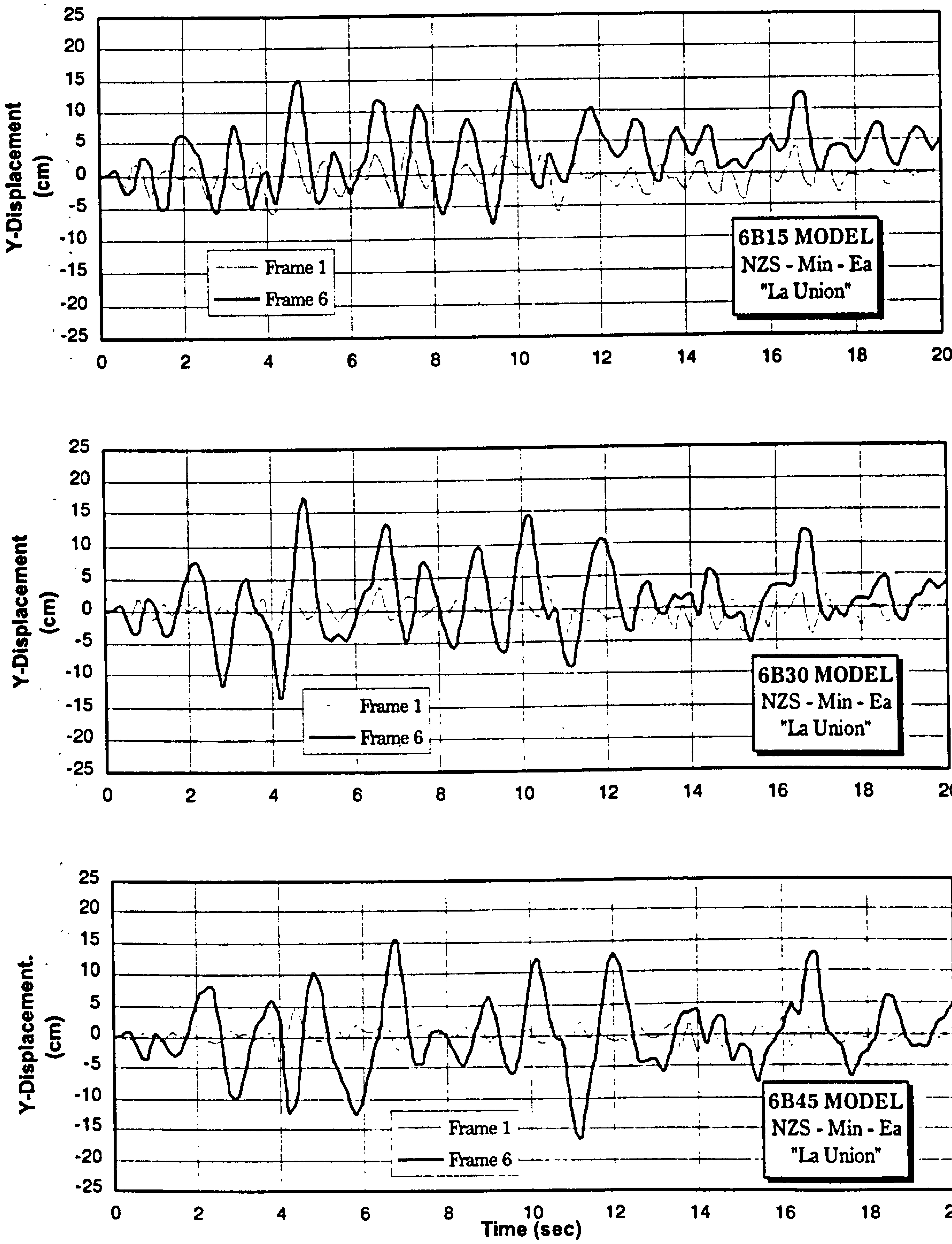


Figure 6.4.5 Time-history displacement of TU models with different static eccentricities excited by the "La Union" earthquake record and designed to the NZS code for "Min – Ea".

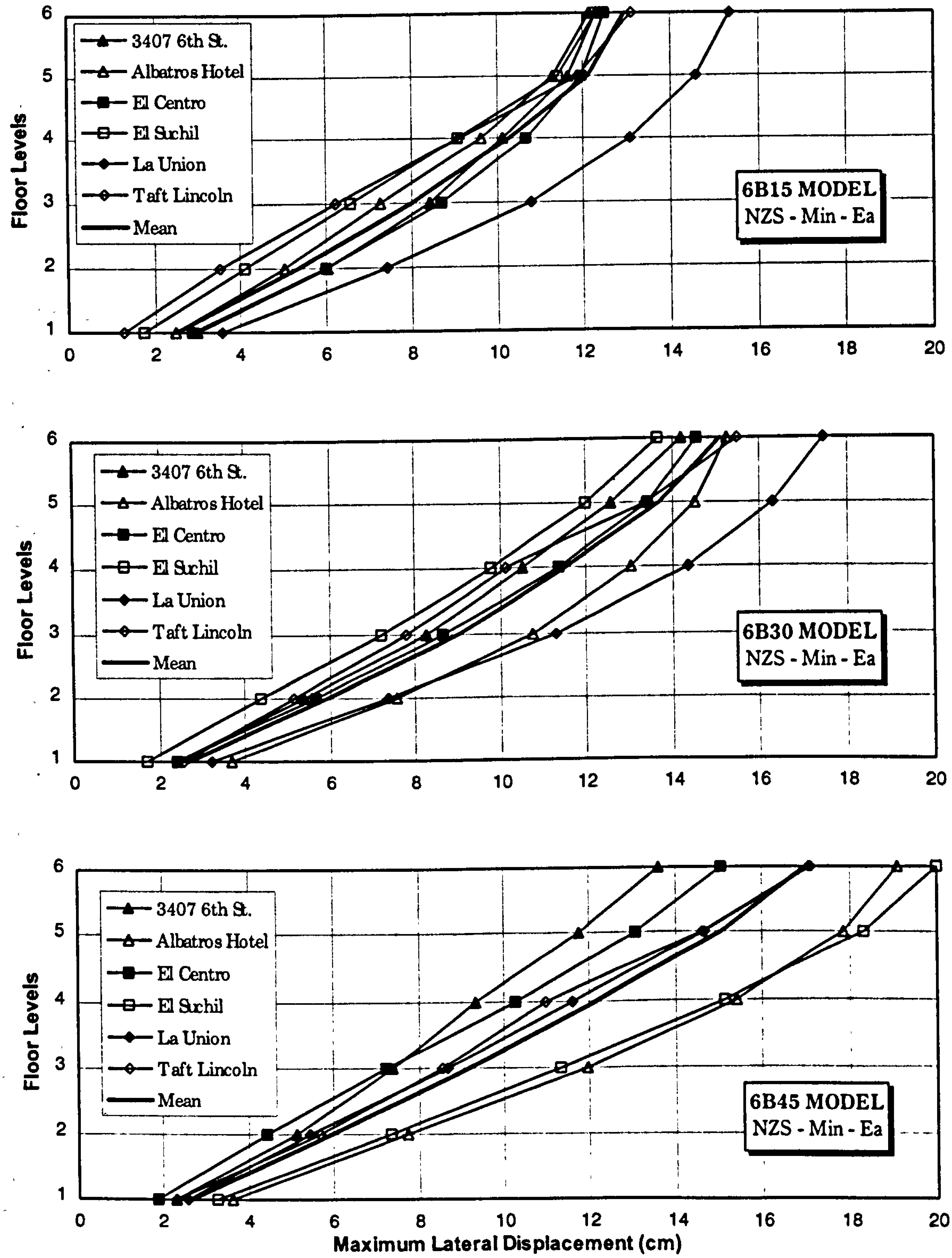


Figure 6.4.6 Maximum lateral displacement (frame 6) of models with different static eccentricities excited by all the earthquake records selected and designed to the NZS code for "Min – Ea".

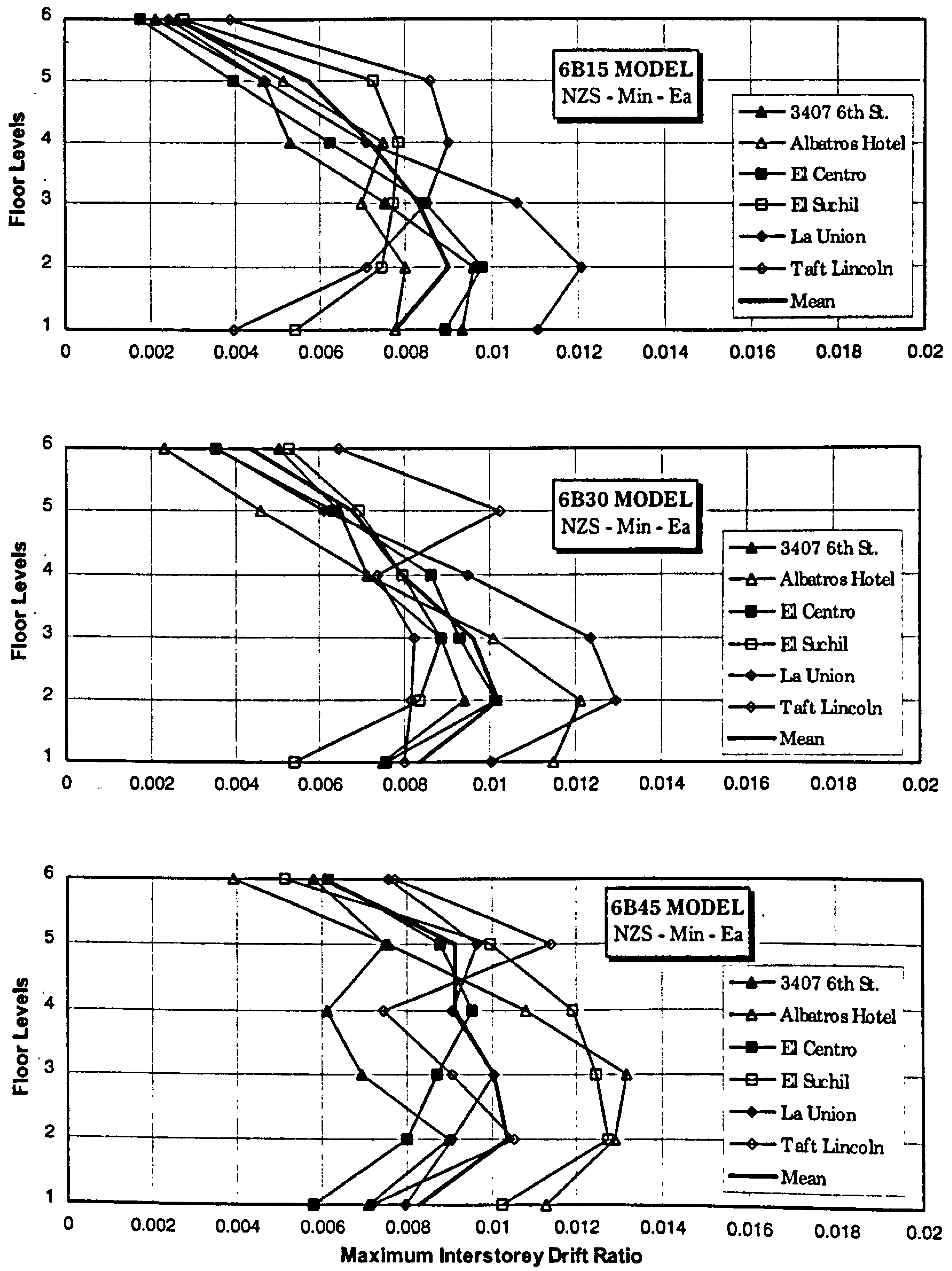


Figure 6.4.7 Maximum interstorey drift ratio of models with different static eccentricities excited by all the earthquake records selected and designed to the NZS code for "Min – Ea".

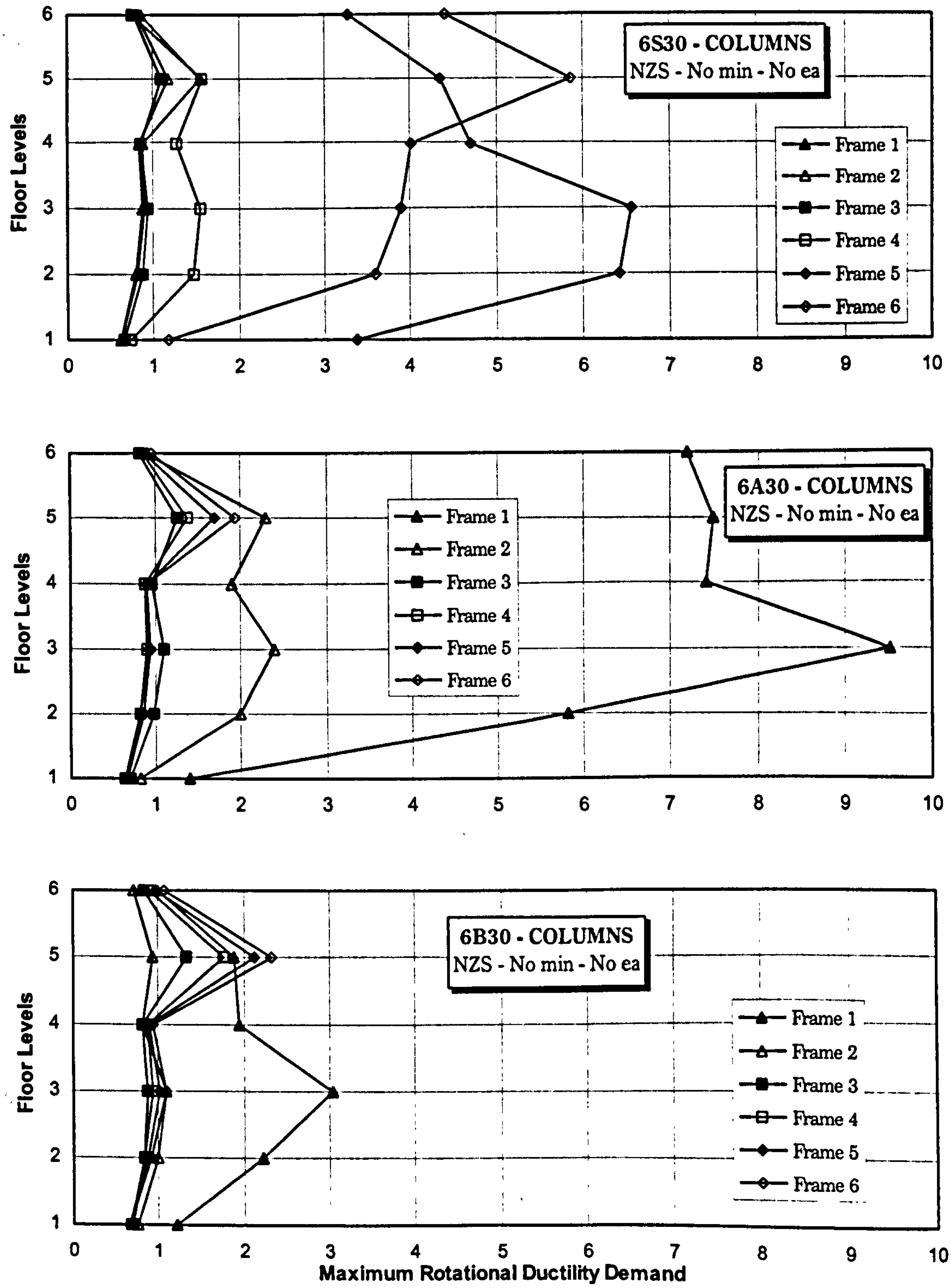


Figure 6.5.1 Rotational ductility demand vs. floor levels for the columns of different model types with the same static eccentricity designed to the NZS code for "No min – No ea".

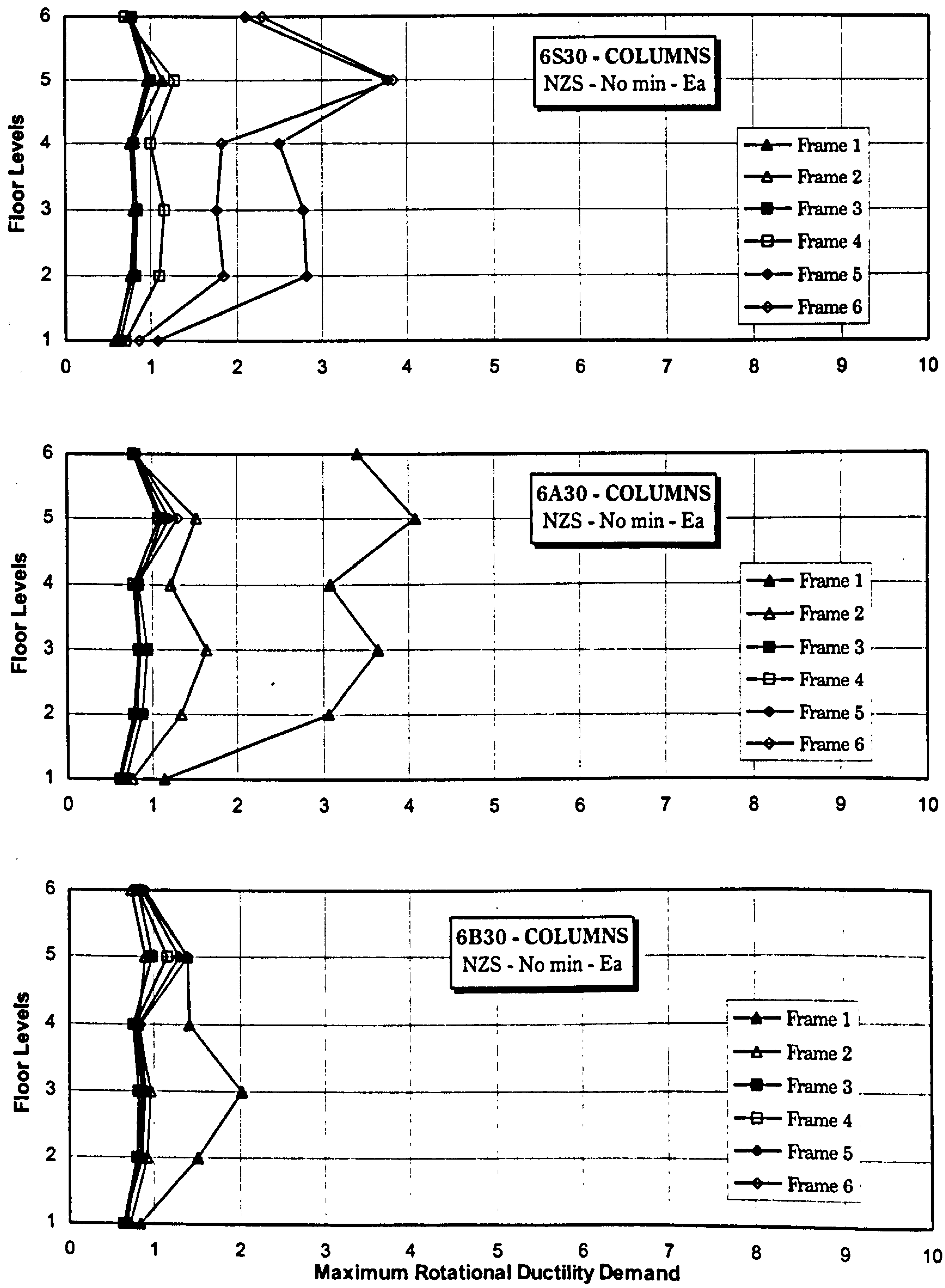


Figure 6.5.2 Rotational ductility demand vs. floor levels for the columns of different model types with the same static eccentricity designed to the NZS code for "No min – Ea".

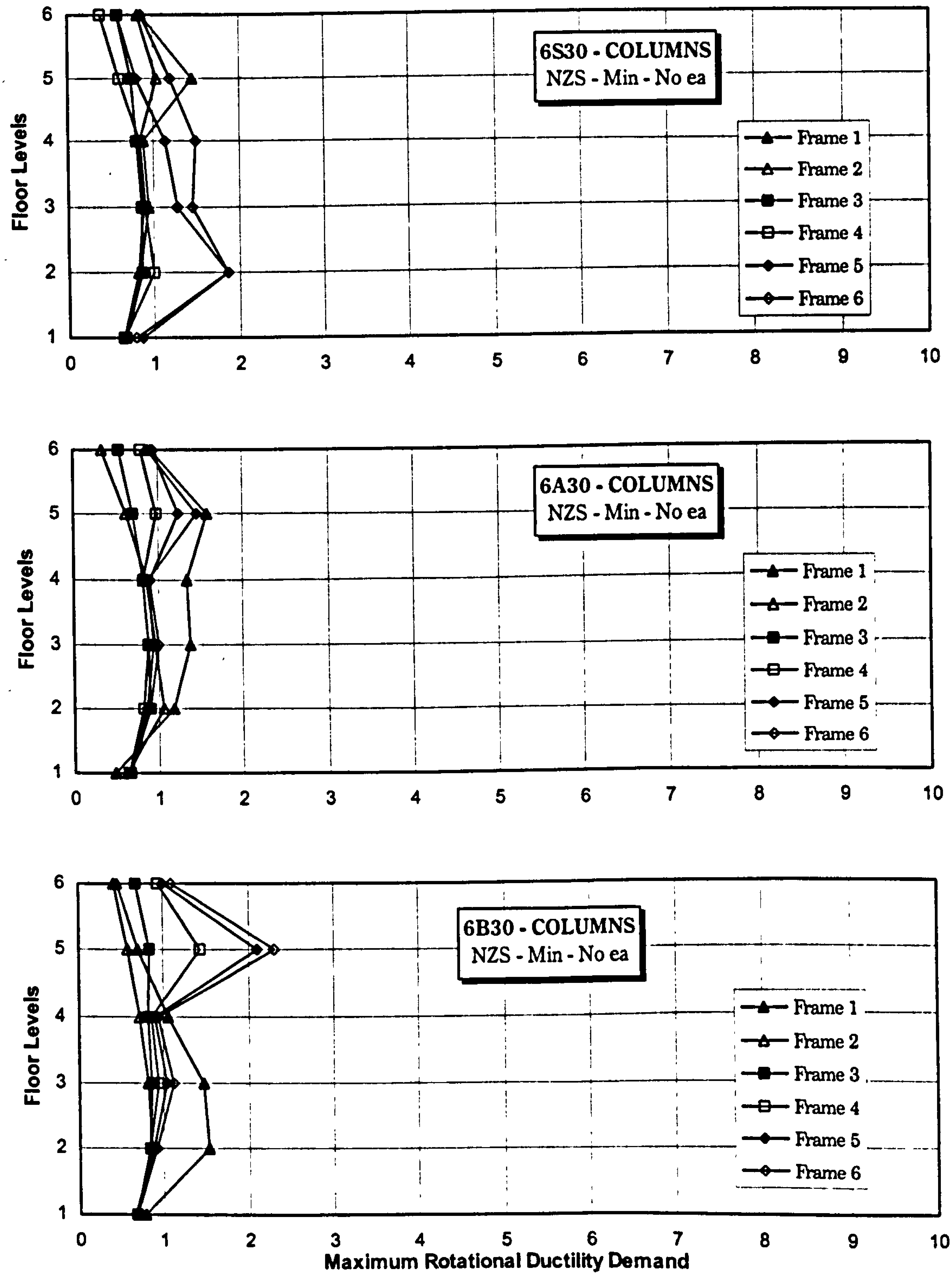


Figure 6.5.3 Rotational ductility demand vs. floor levels for the columns of different model types with the same static eccentricity designed to the NZS code for "Min – No ea".

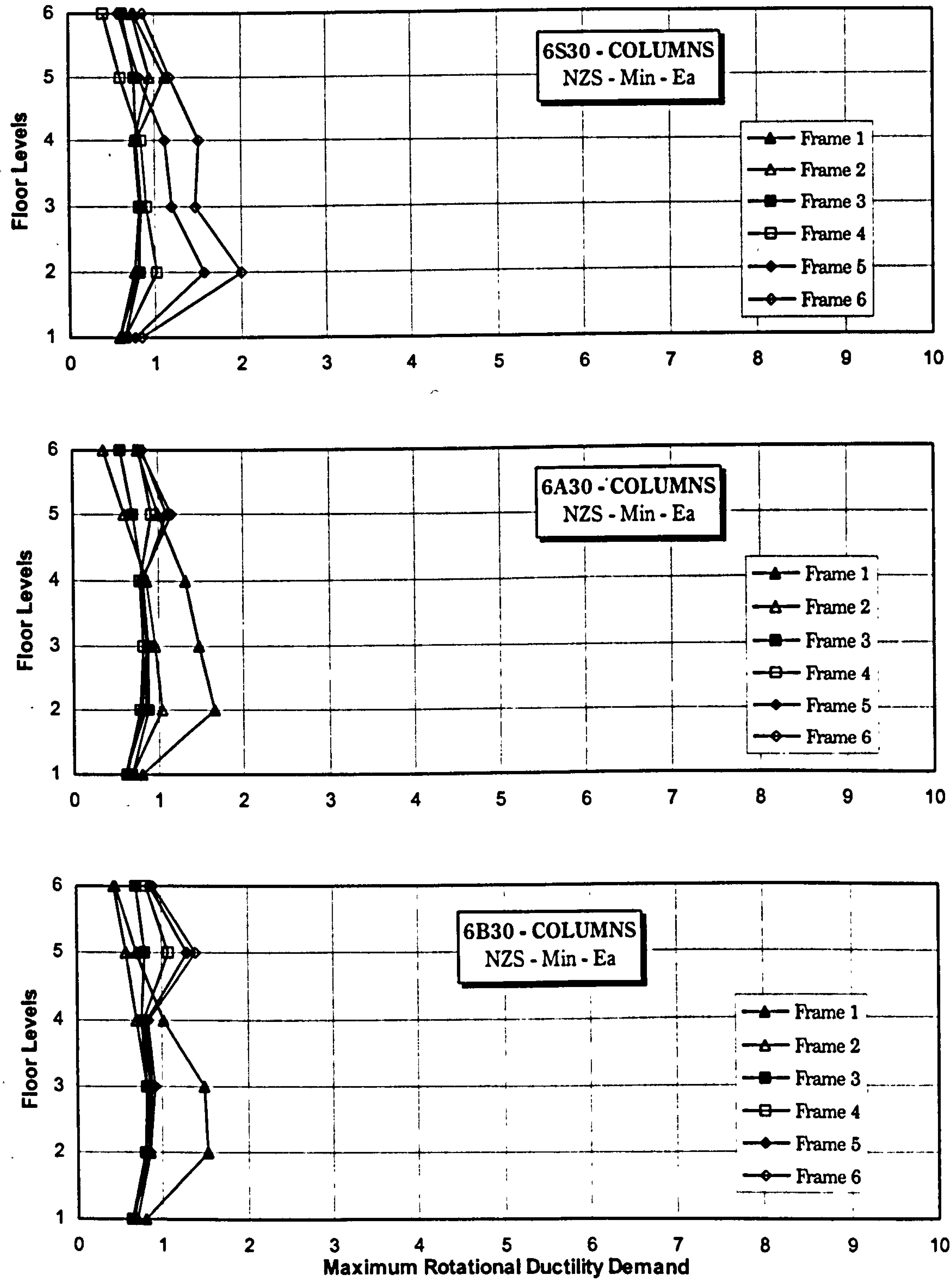


Figure 6.5.4 Rotational ductility demand vs. floor levels for the columns of different model types with the same static eccentricity designed to the NZS code for "Min – Ea".

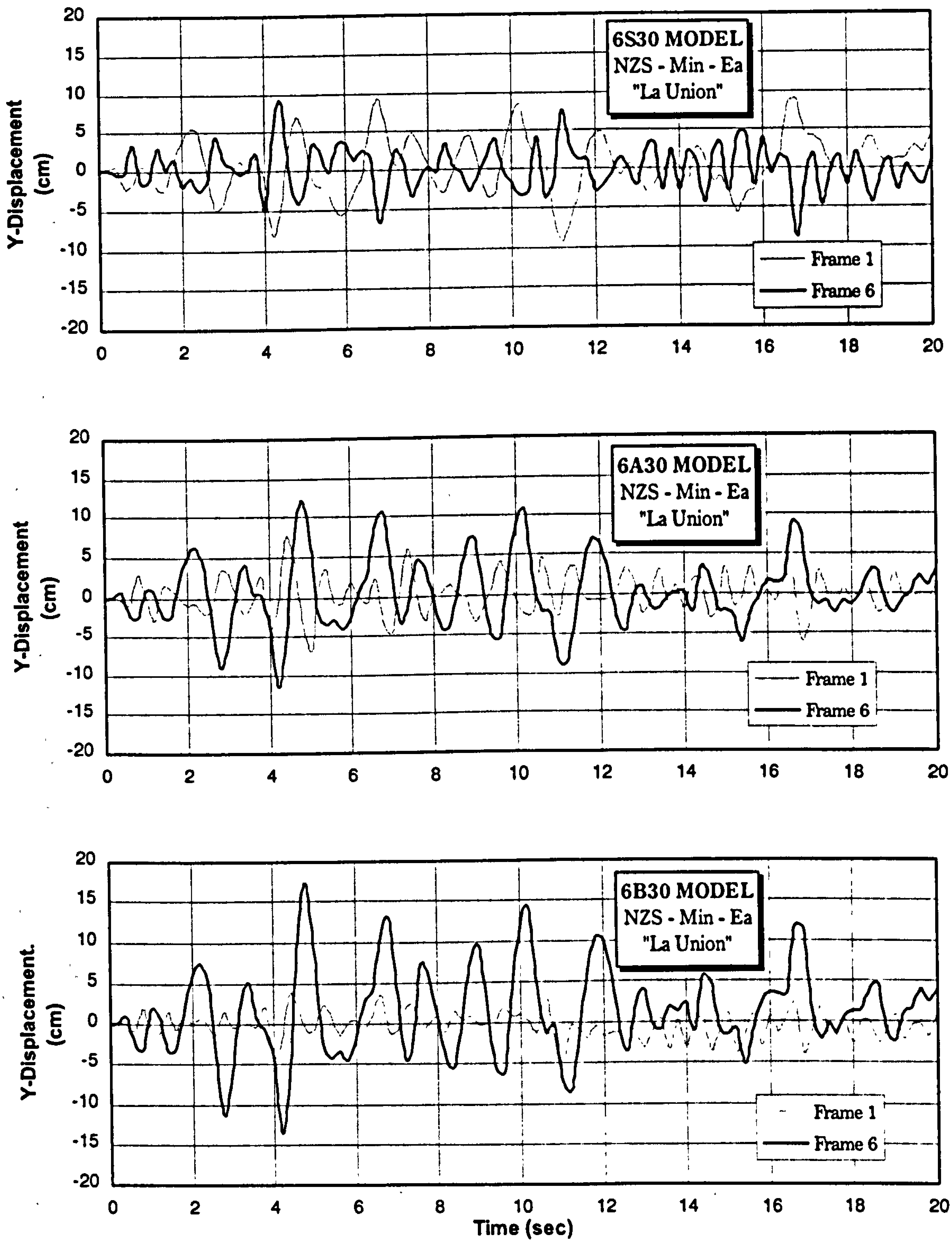


Figure 6.5.5 Time-history displacement of different model types with the same static eccentricity excited to the "La Union" earthquake record and designed to the NZS code for "Min – Ea".

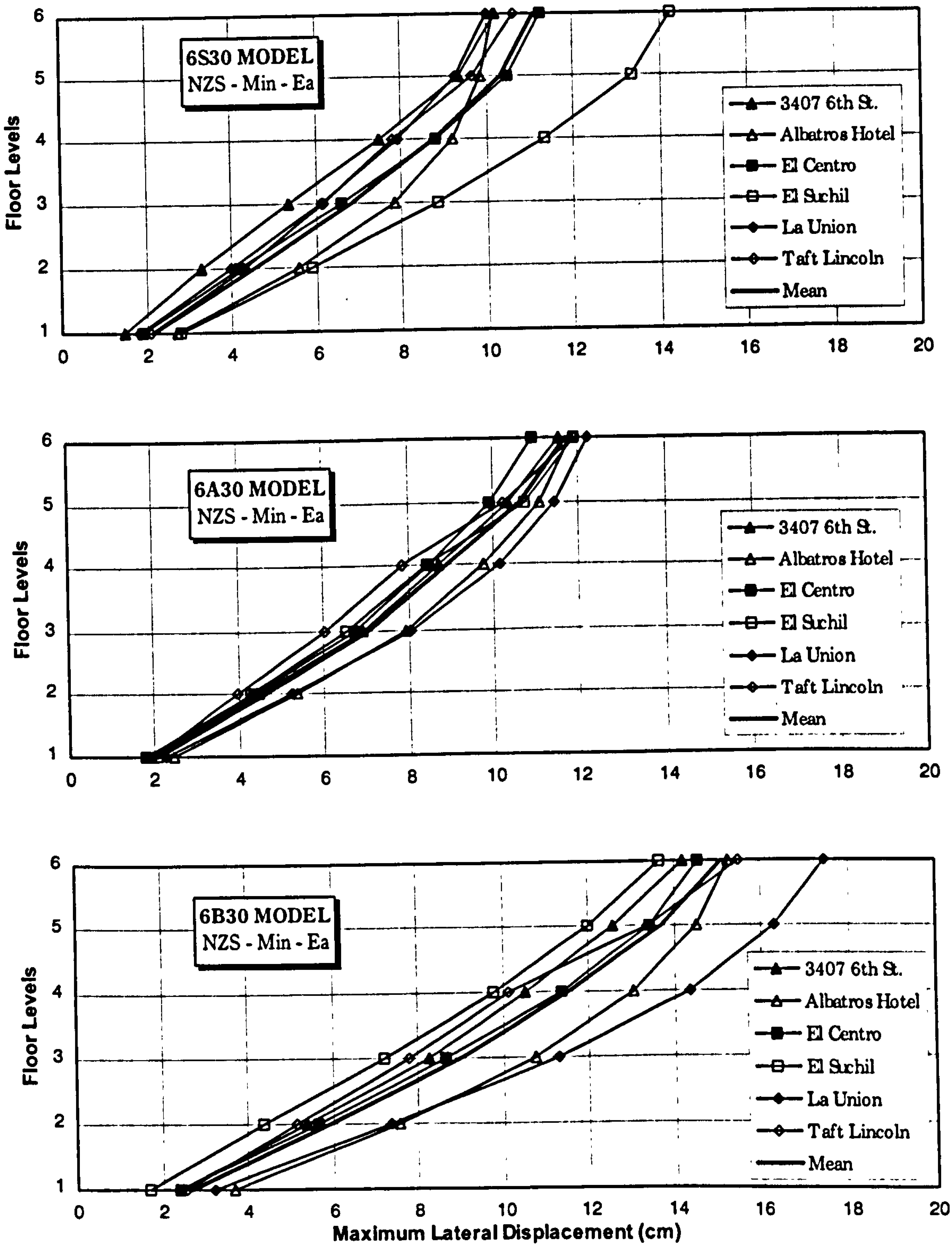


Figure 6.5.6 Maximum lateral displacement of different model types with the same static eccentricity excited to all the earthquake records and designed to the NZS code for "Min – Ea".

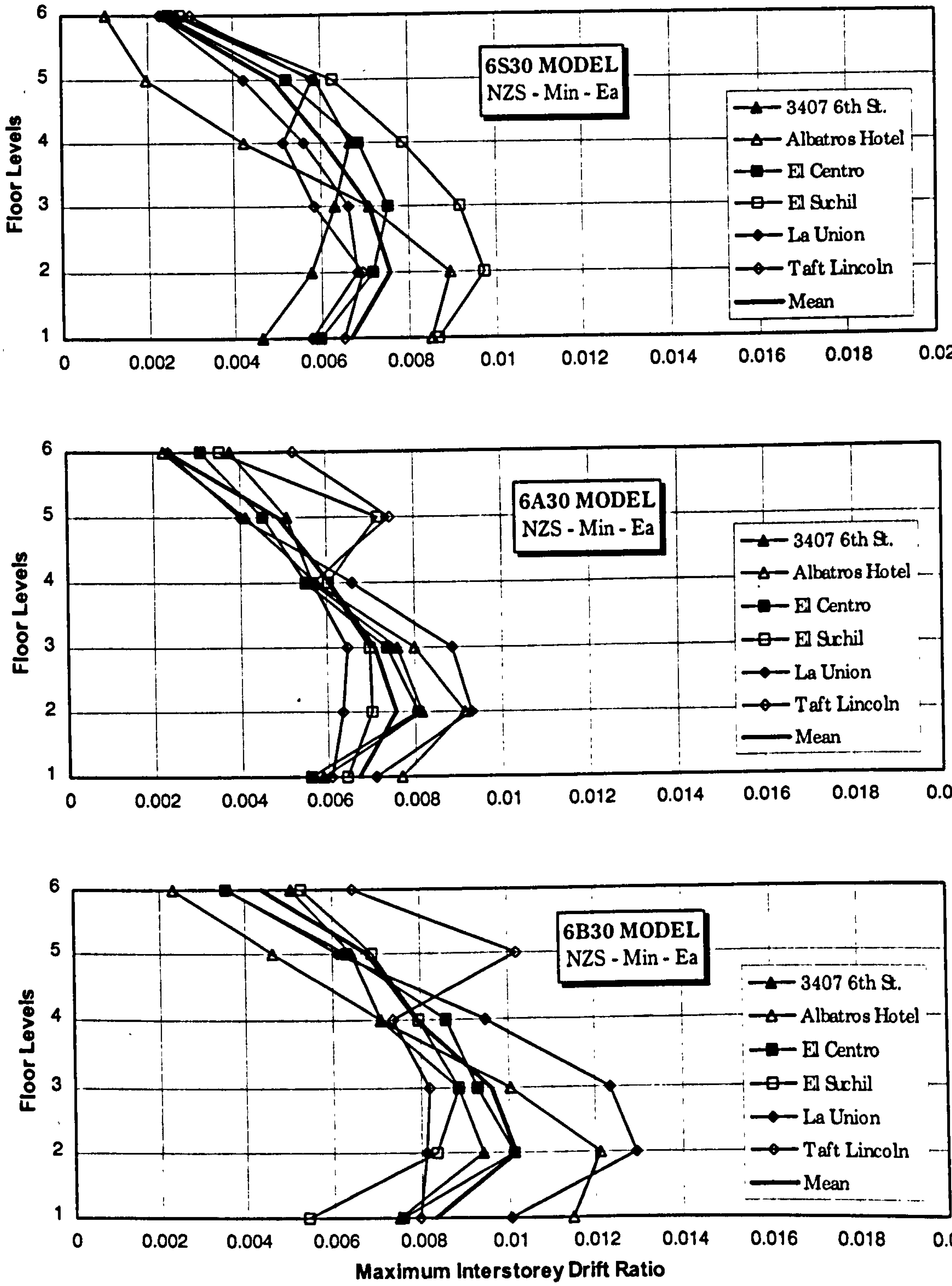


Figure 6.5.7 Maximum interstorey drift ratio of different model types with the same static eccentricity excited to all the earthquake records and designed to NZS code for "Min – Ea".

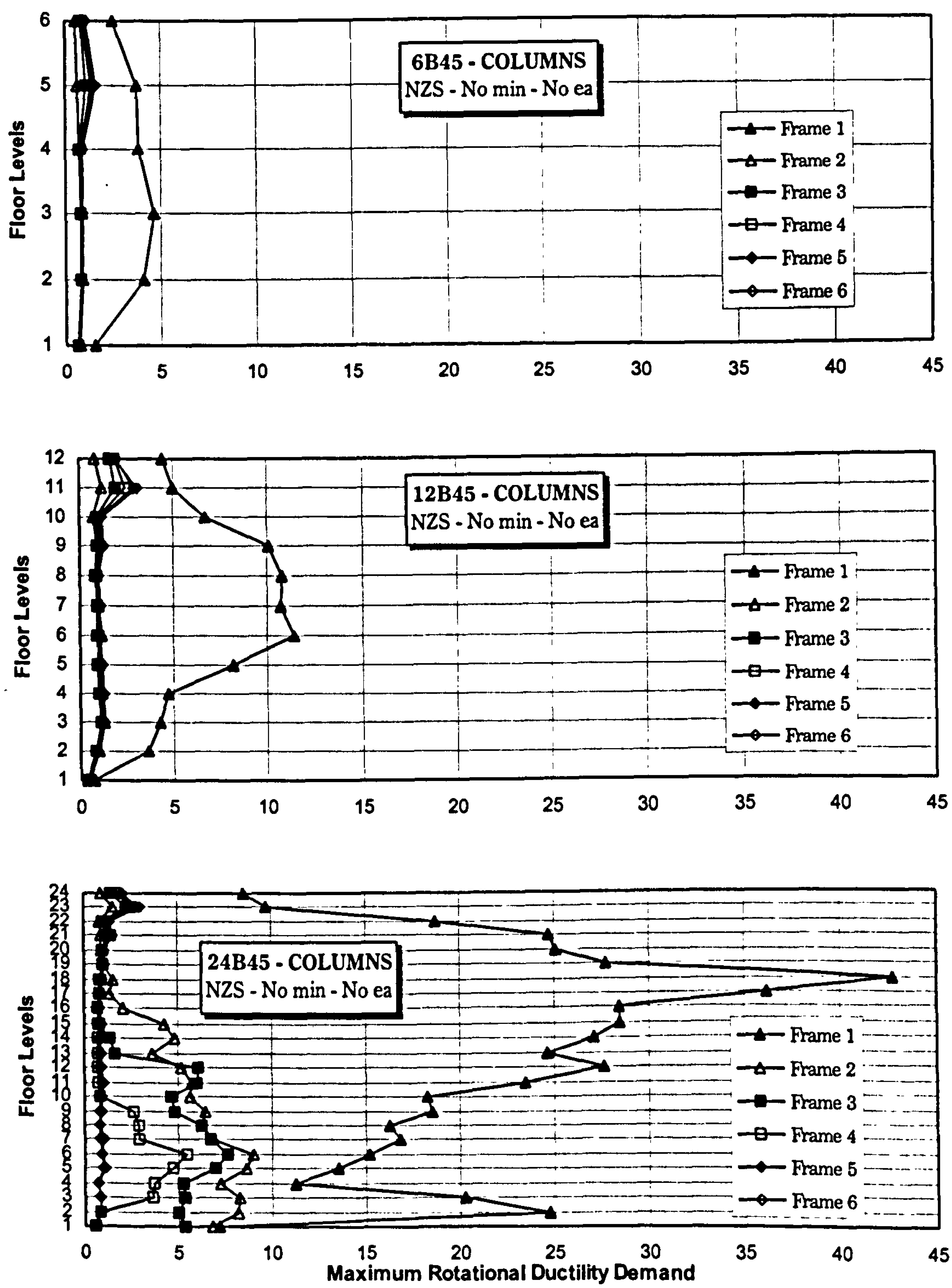


Figure 6.6.1 Rotational ductility demand vs. floor levels for the columns of models with different numbers of floor levels designed to the NZS code for "No min – No ea".

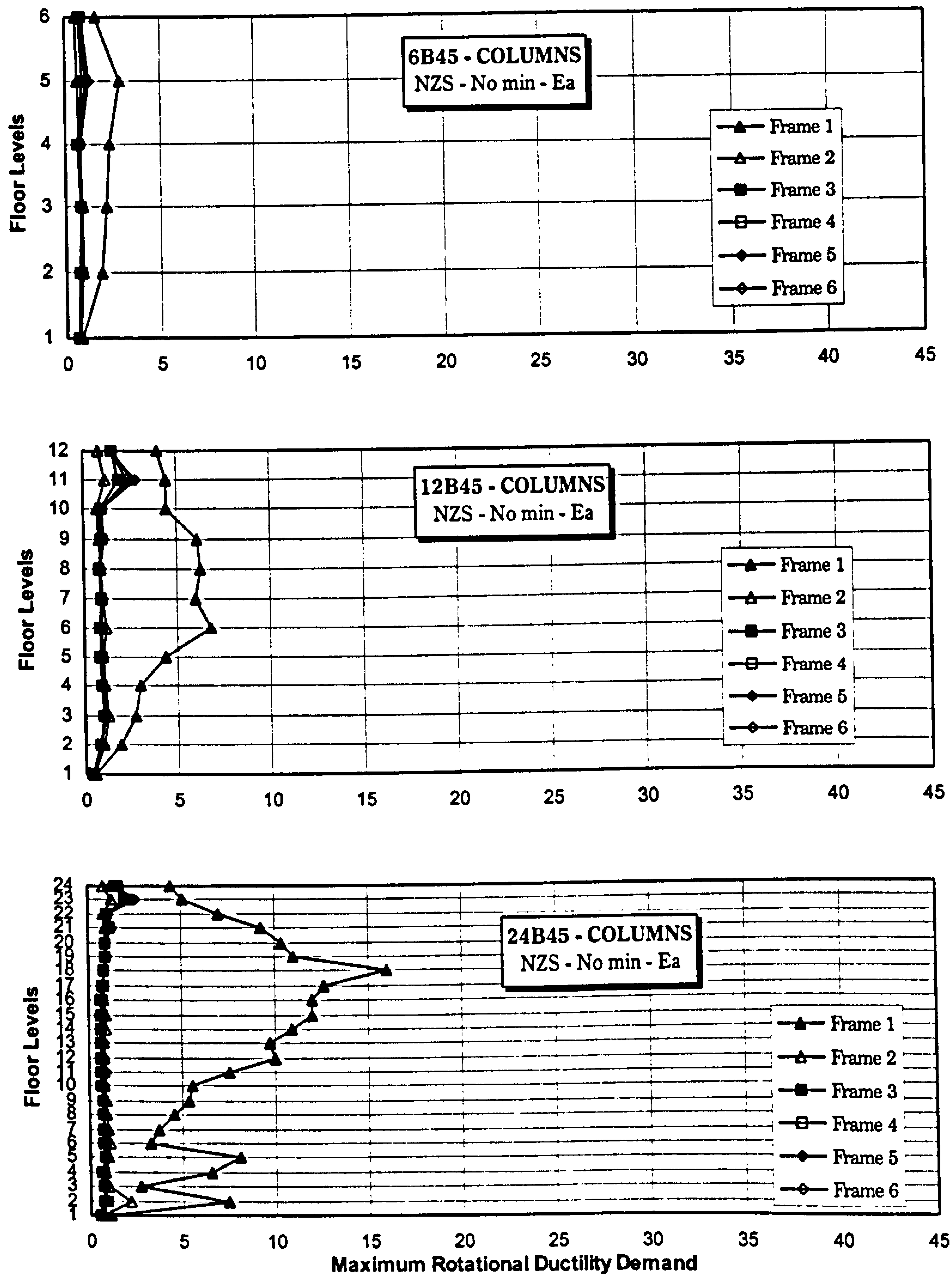


Figure 6.6.2 Rotational ductility demand vs. floor levels for the columns of models with different numbers of floor levels designed to the NZS code for "No min – Ea".

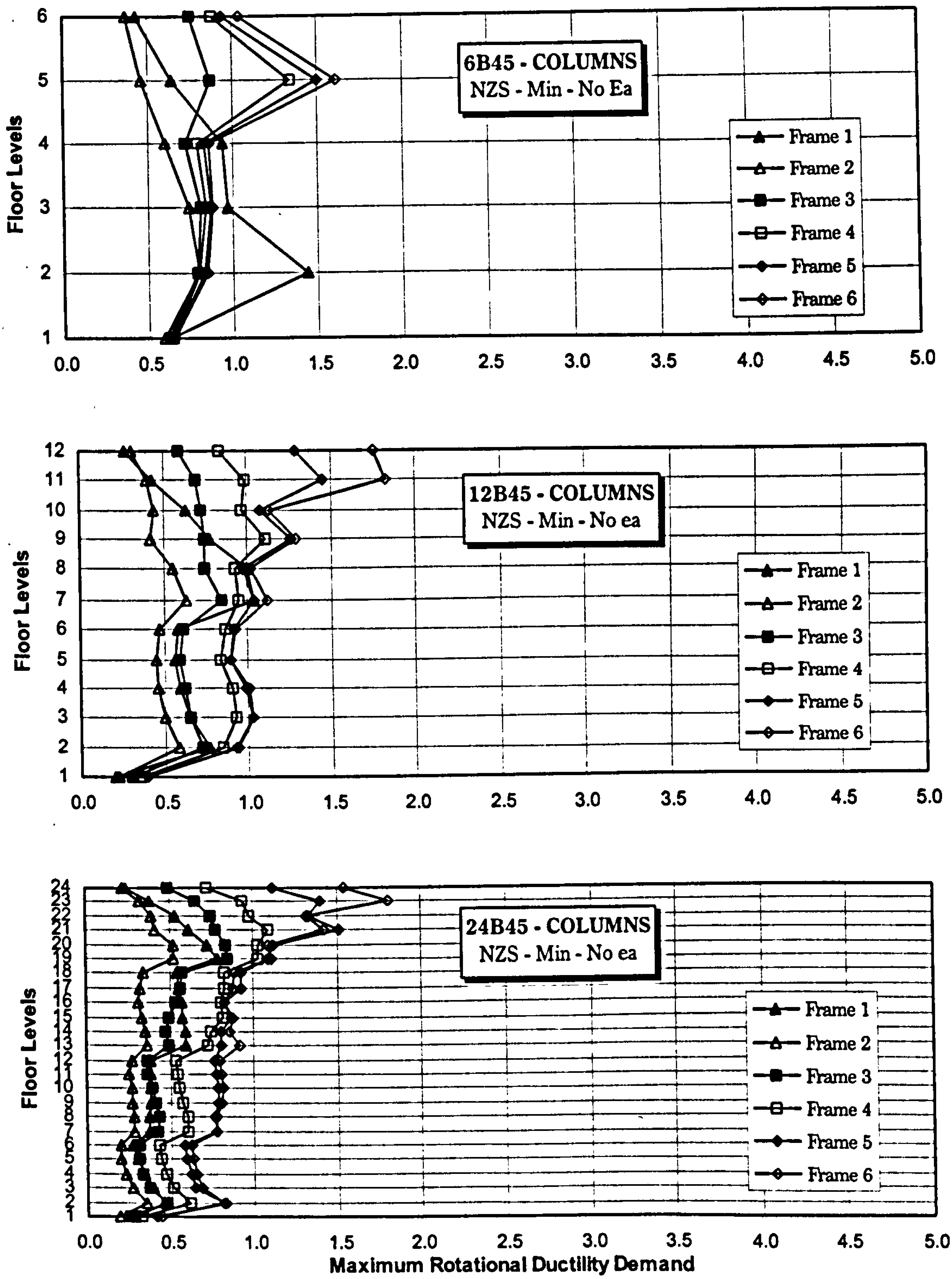


Figure 6.6.3 Rotational ductility demand vs. floor levels for the columns of models with different numbers of floor levels designed to the NZS code for "Min – No ea".

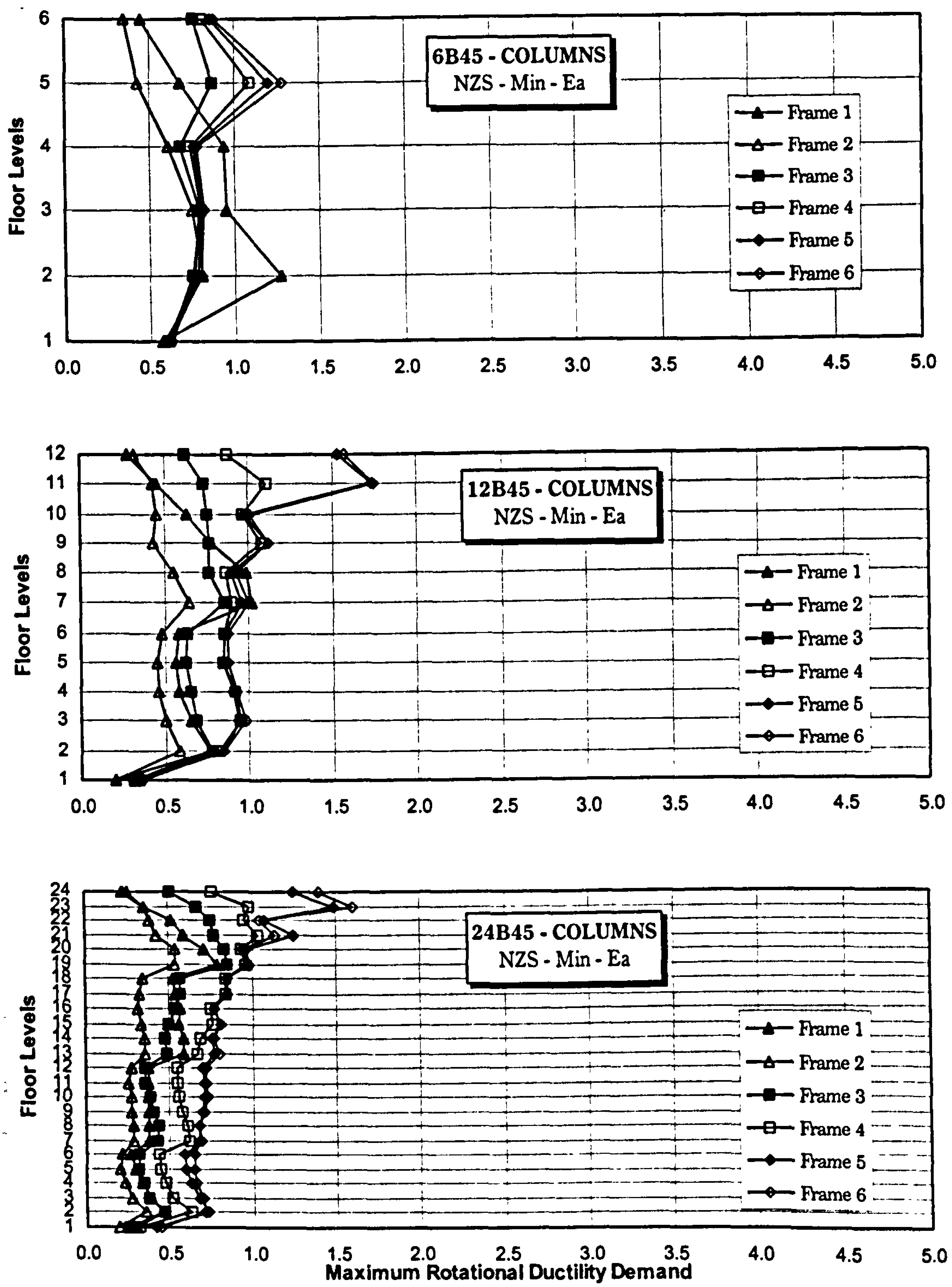


Figure 6.6.4 Rotational ductility demand vs. floor levels for the columns of models with different numbers of floor levels designed to the NZS code for "Min – Ea".

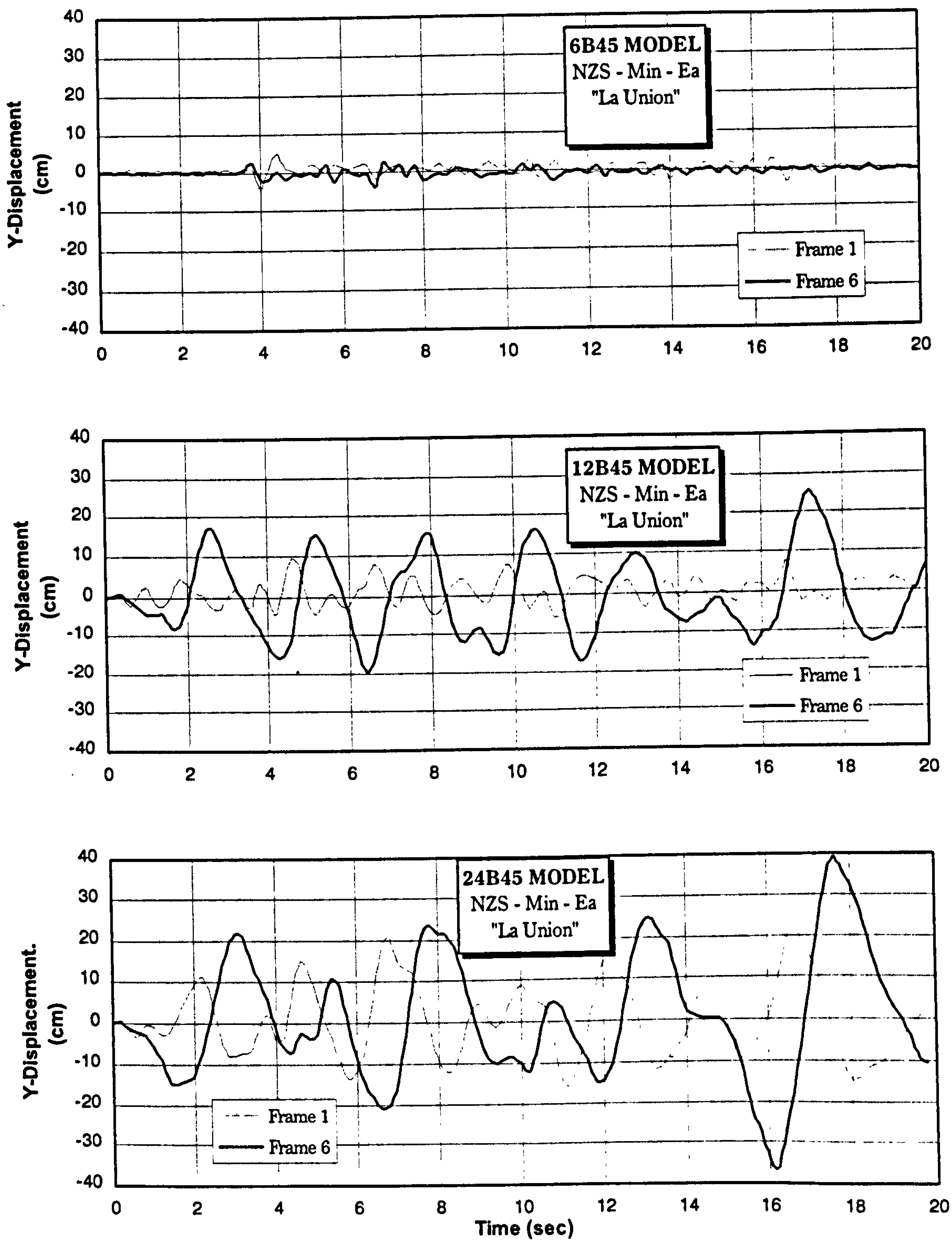


Figure 6.6.5 Time-history displacement of models with different numbers of floor levels excited by the "La Union" earthquake record and designed to the NZS code for "Min – Ea".

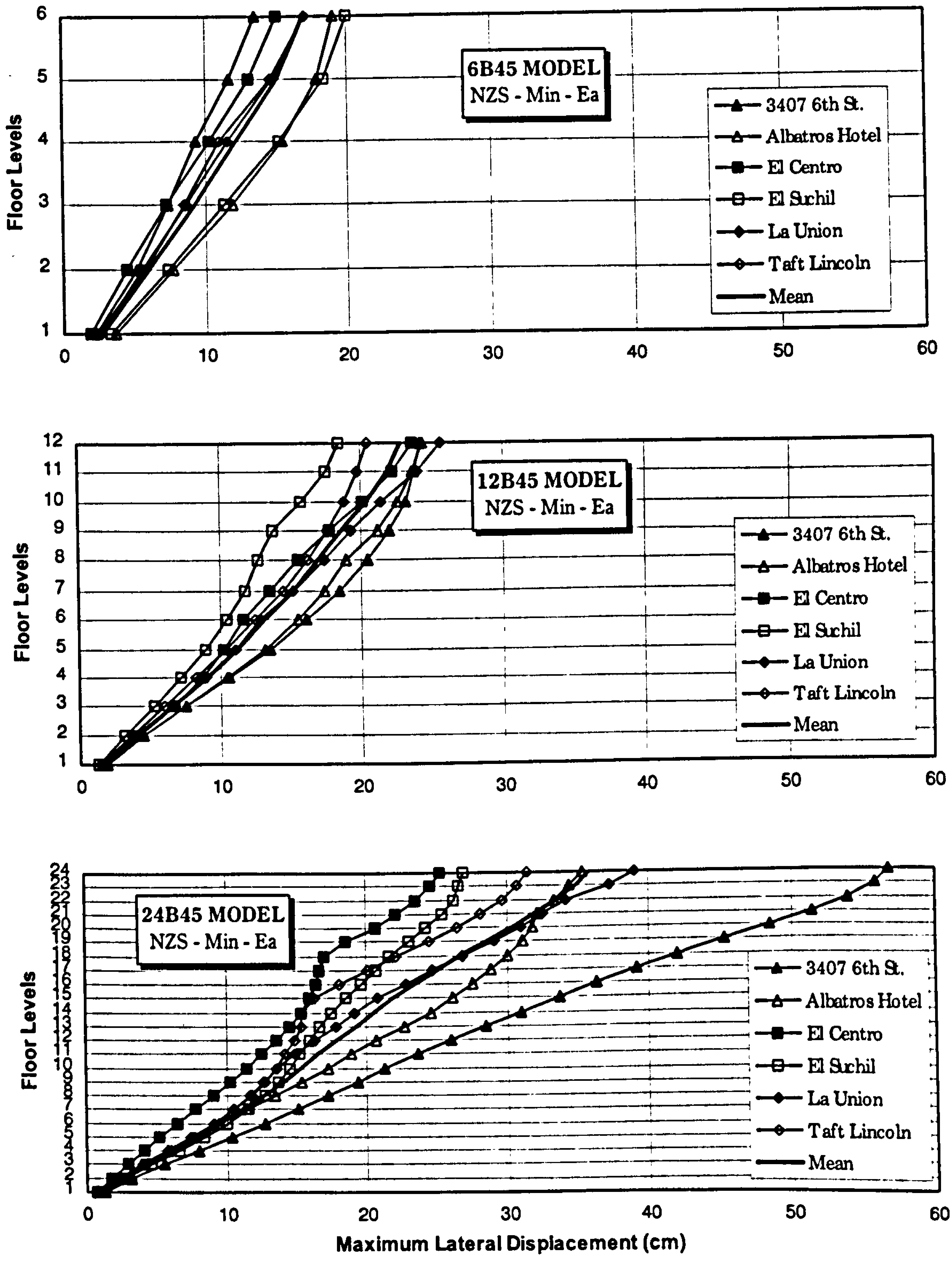


Figure 6.6.6 Maximum lateral displacement of models with different numbers of floor levels excited by all the earthquake records selected and designed to the NZS code for "Min – Ea".

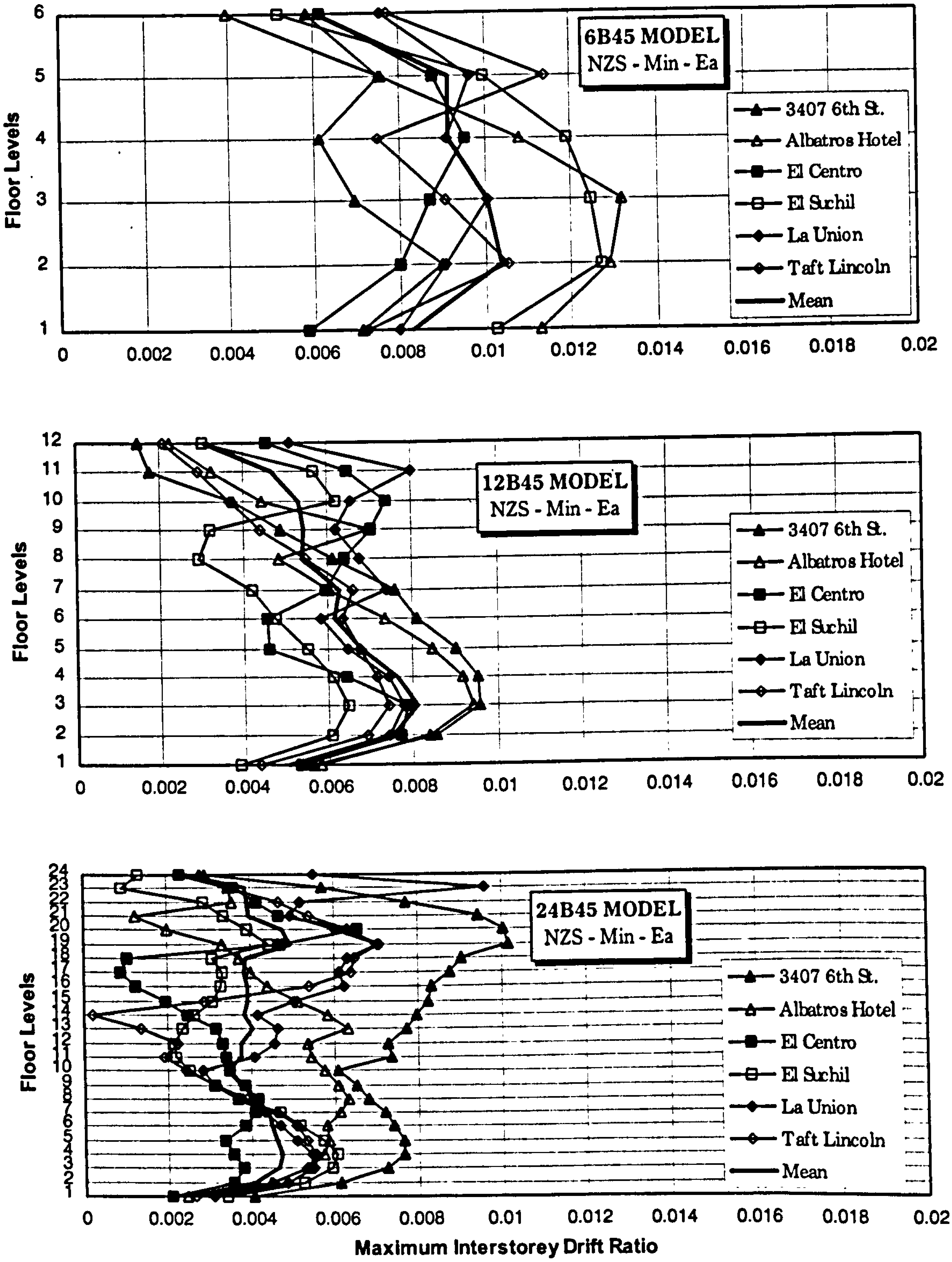


Figure 6.6.7 Maximum interstorey drift ratios of models with different numbers of floor levels excited by all the earthquake records selected and designed to NZS and "Min – Ea".

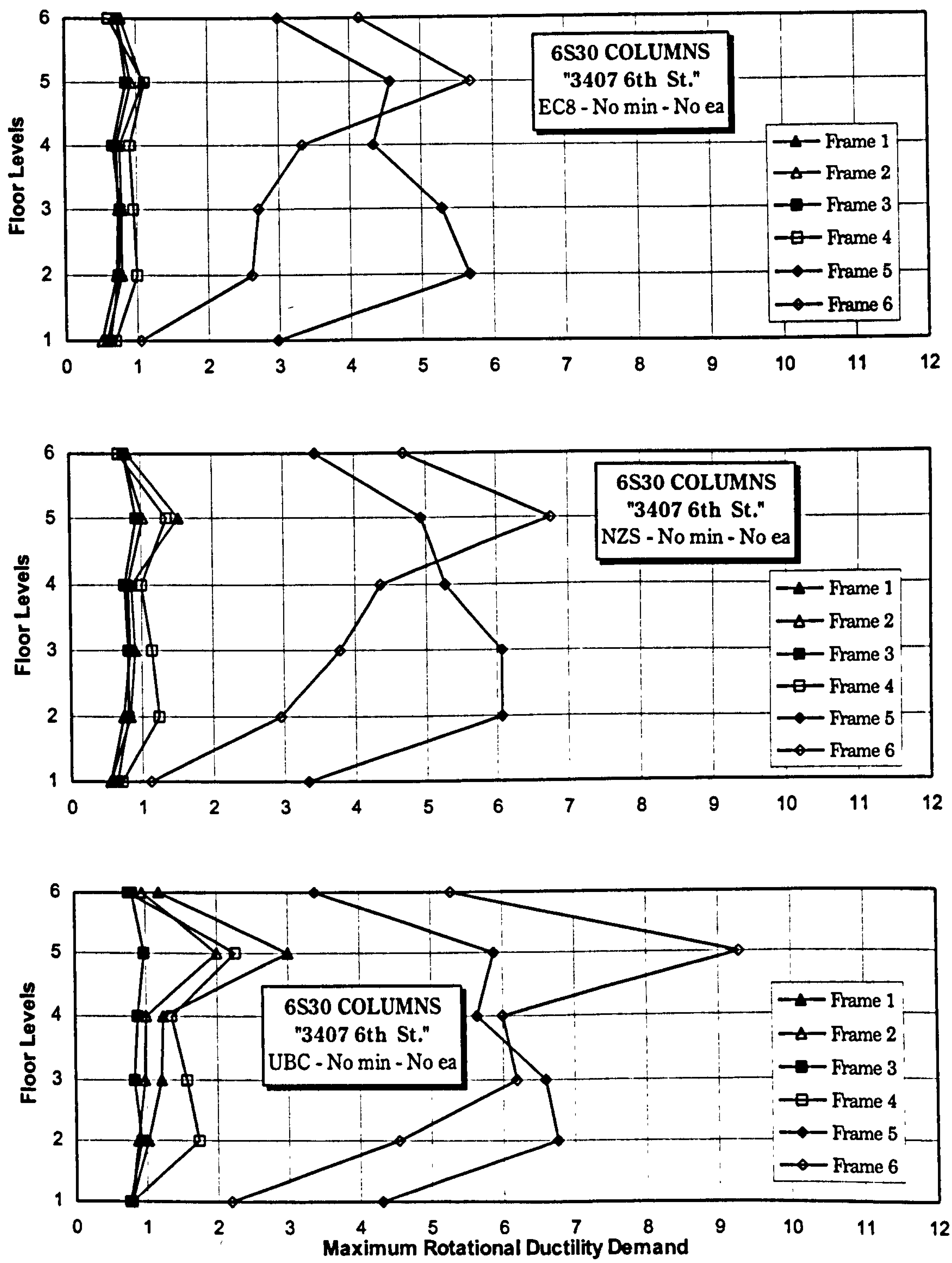


Figure 6.7.1 Rotational ductility demand vs. floor levels for the columns of model 6S30 analysed to different torsional provisions, designed for "No min – No ea" and excited by the "3407" 6th St" record.

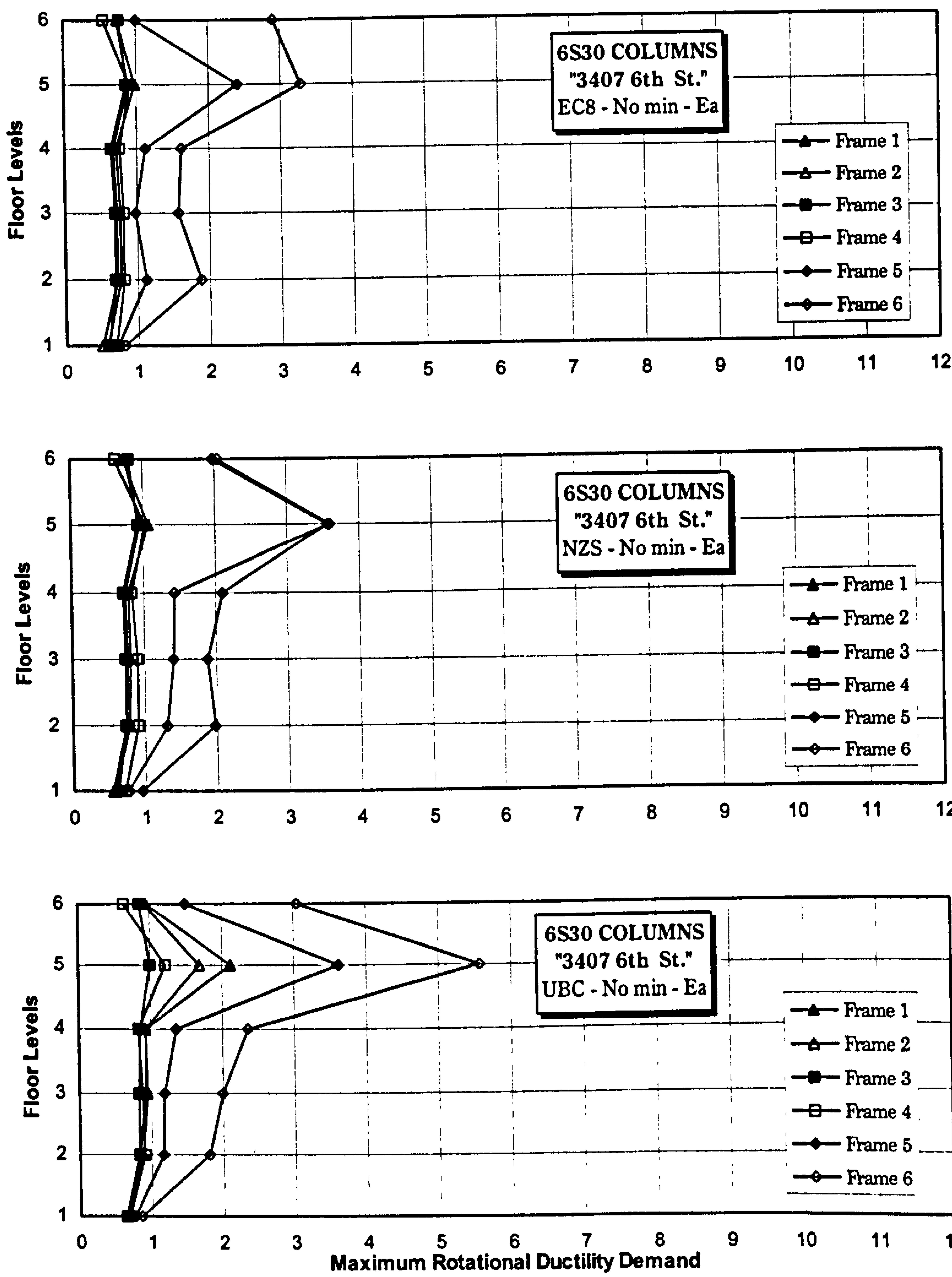


Figure 6.7.2 Rotational ductility demand vs. floor levels for the columns of model 6S30 analysed to different torsional provisions, designed for "No min – Ea" and excited by the "3407" 6^{th} St" record.

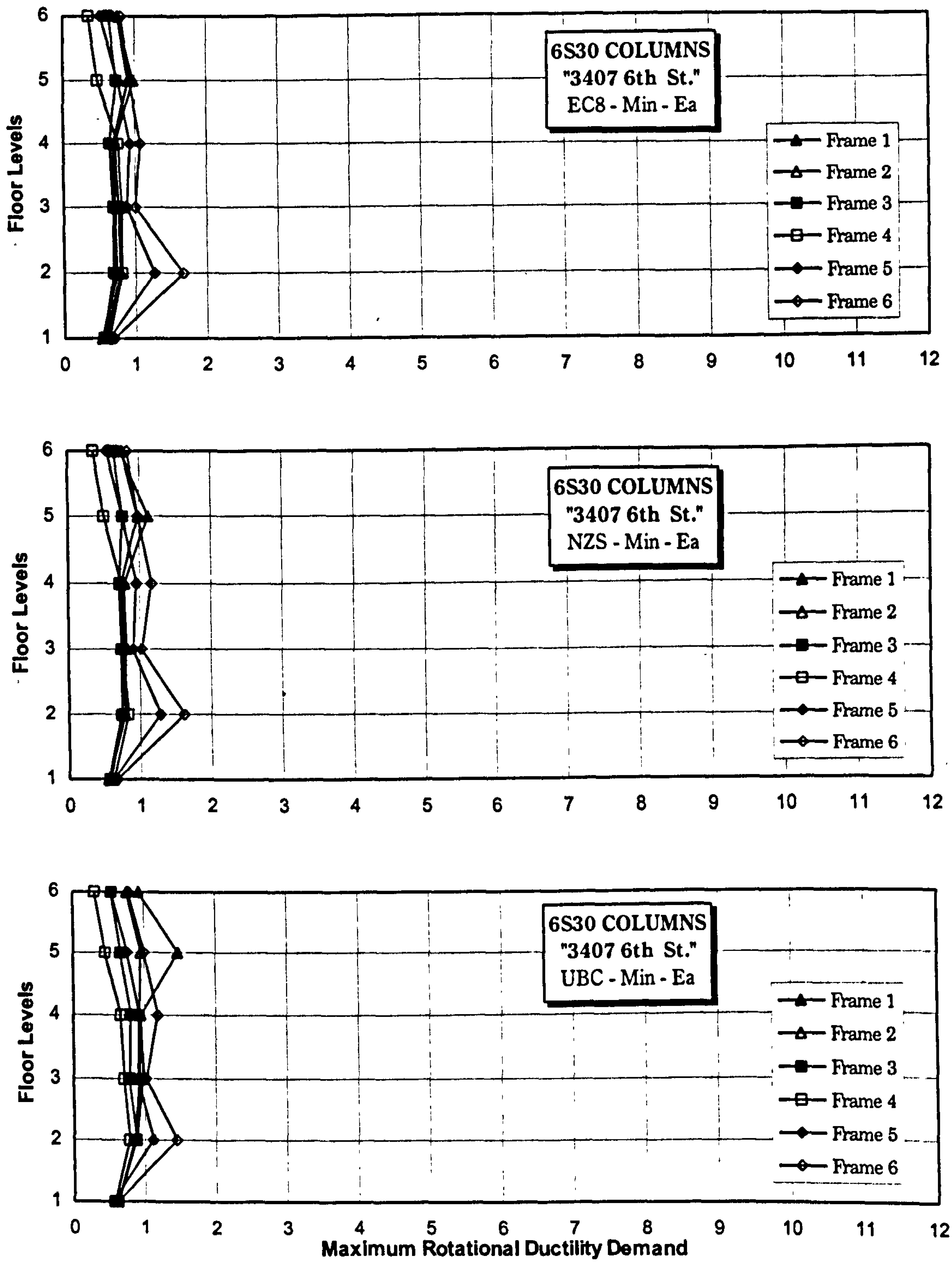


Figure 6.7.3 Rotational ductility demand vs. floor levels for the columns of model 6S30 analysed to different torsional provisions, designed for "Min – Ea" and excited by the "3407" 6th St" record.

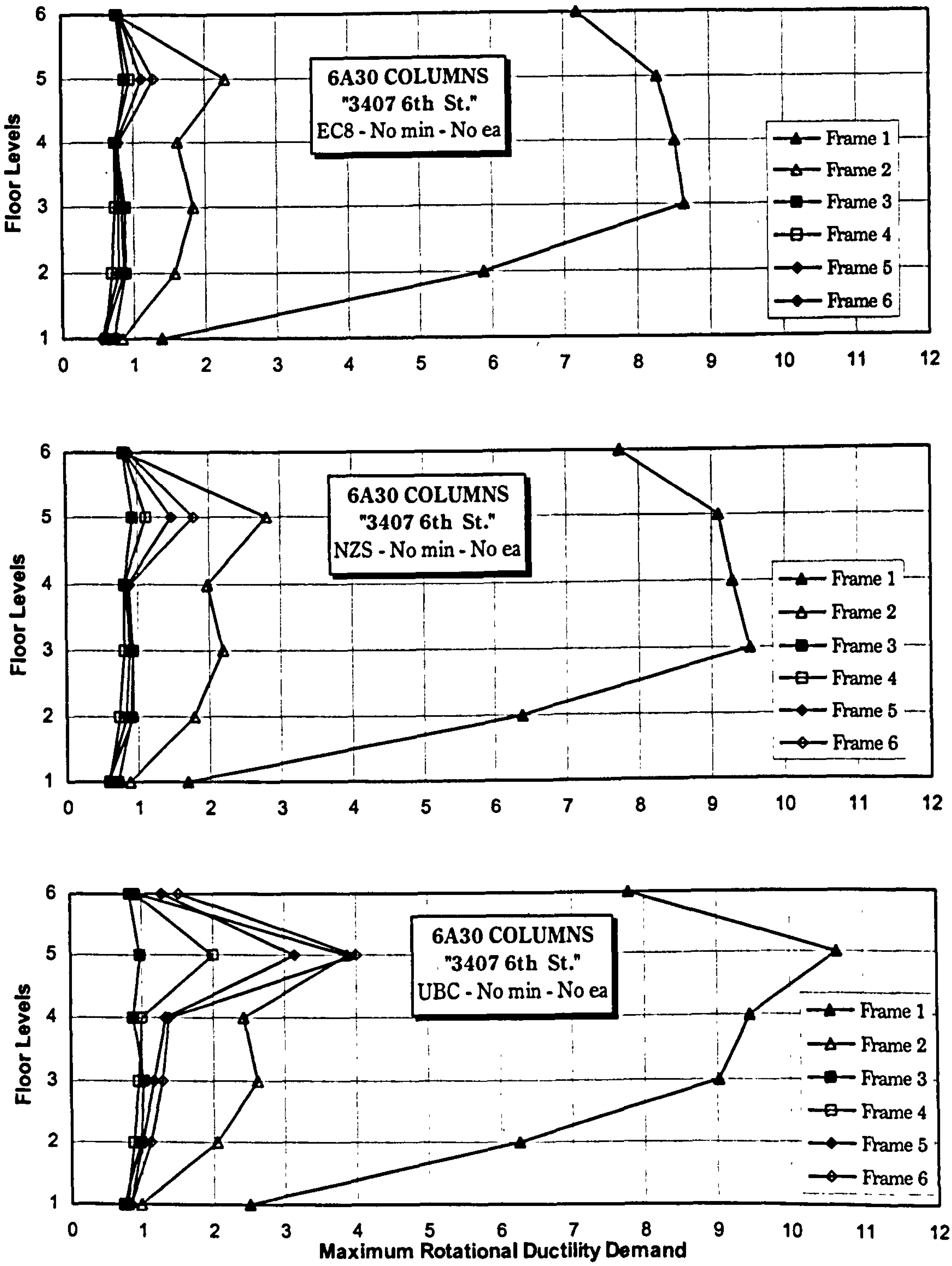


Figure 6.7.4 Rotational ductility demand vs. floor levels for the columns of model 6A30 analysed to different torsional provisions, designed for "No min – No ea" and excited by the "3407" 6^{th} St" record.

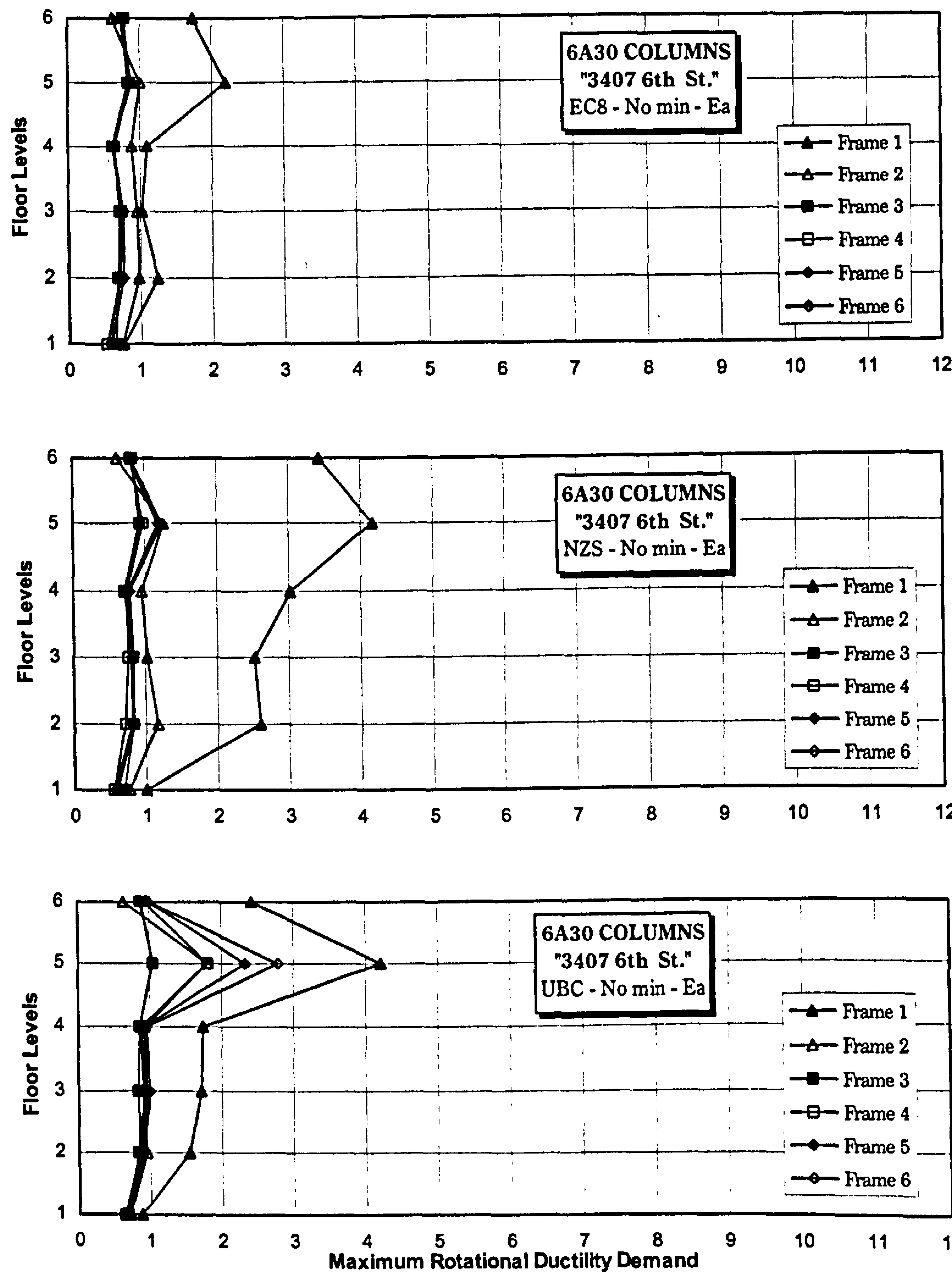


Figure 6.7.5 Rotational ductility demand vs. floor levels for the columns of model 6A30 analysed to different torsional provisions, designed for "No min – Ea" and excited by the "3407" 6th St" record.

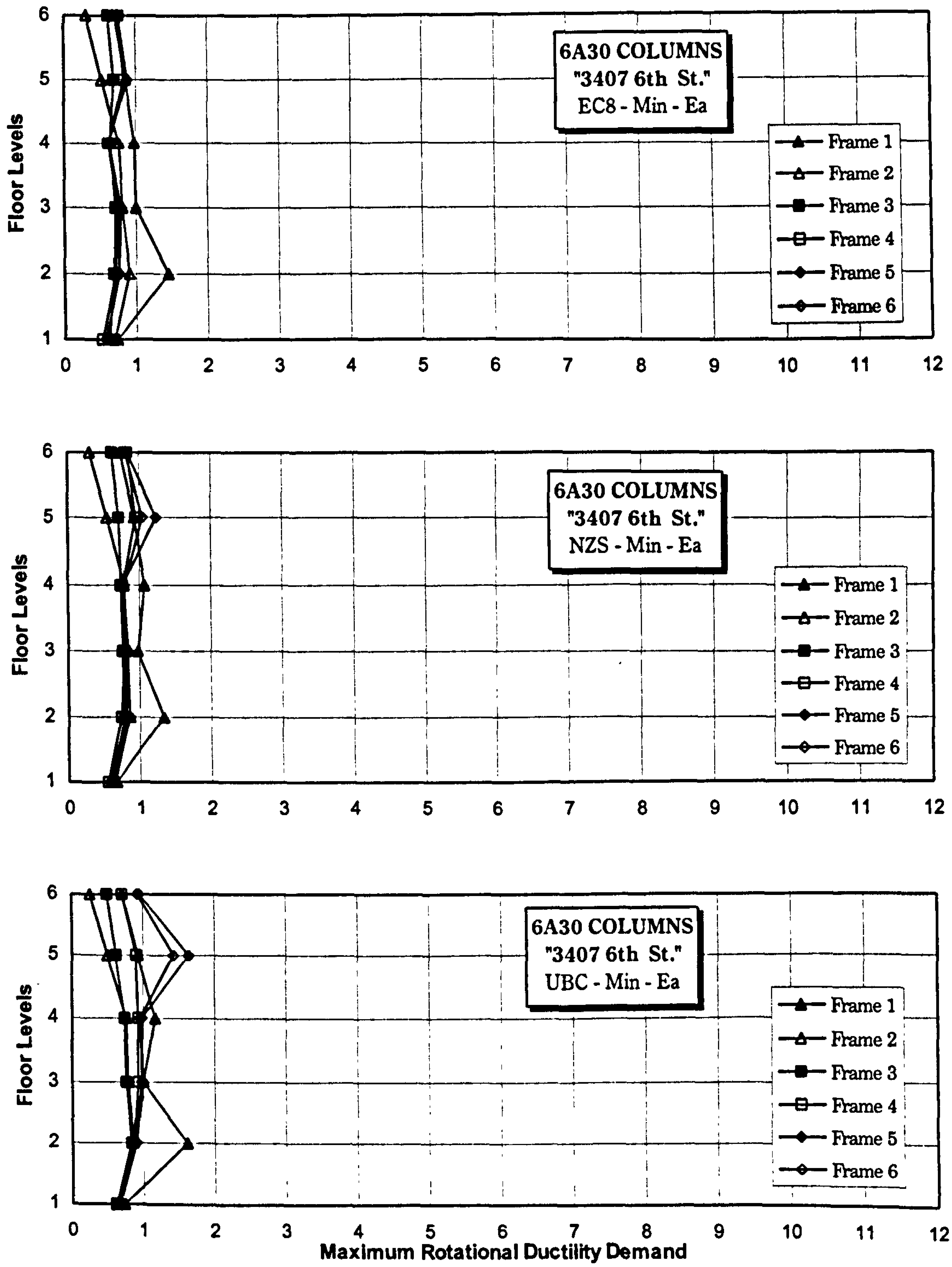


Figure 6.7.6 Rotational ductility demand vs. floor levels for the columns of model 6A30 analysed to different torsional provisions, designed for "Min – Ea" and excited by the "3407" 6th St" record.

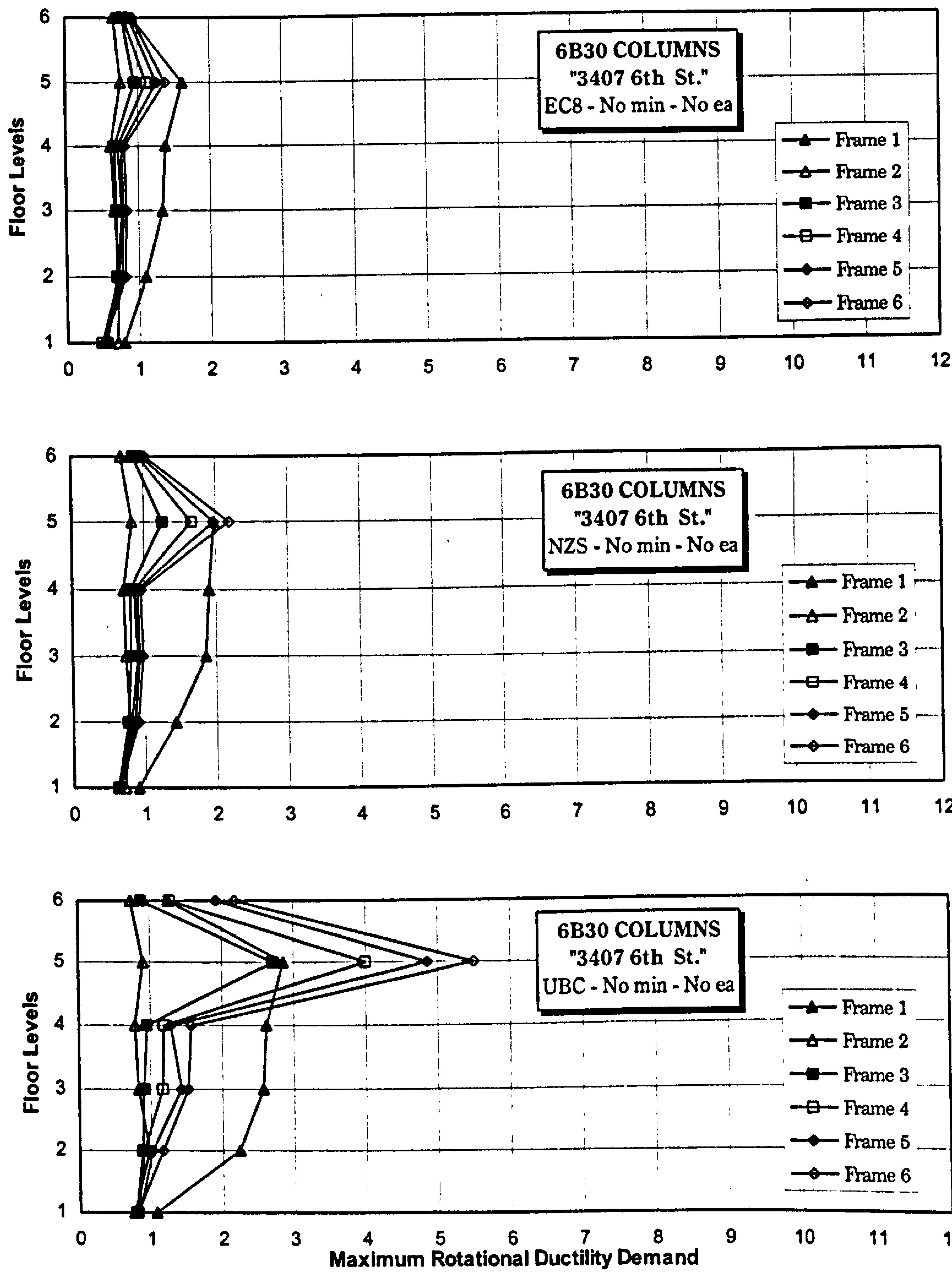
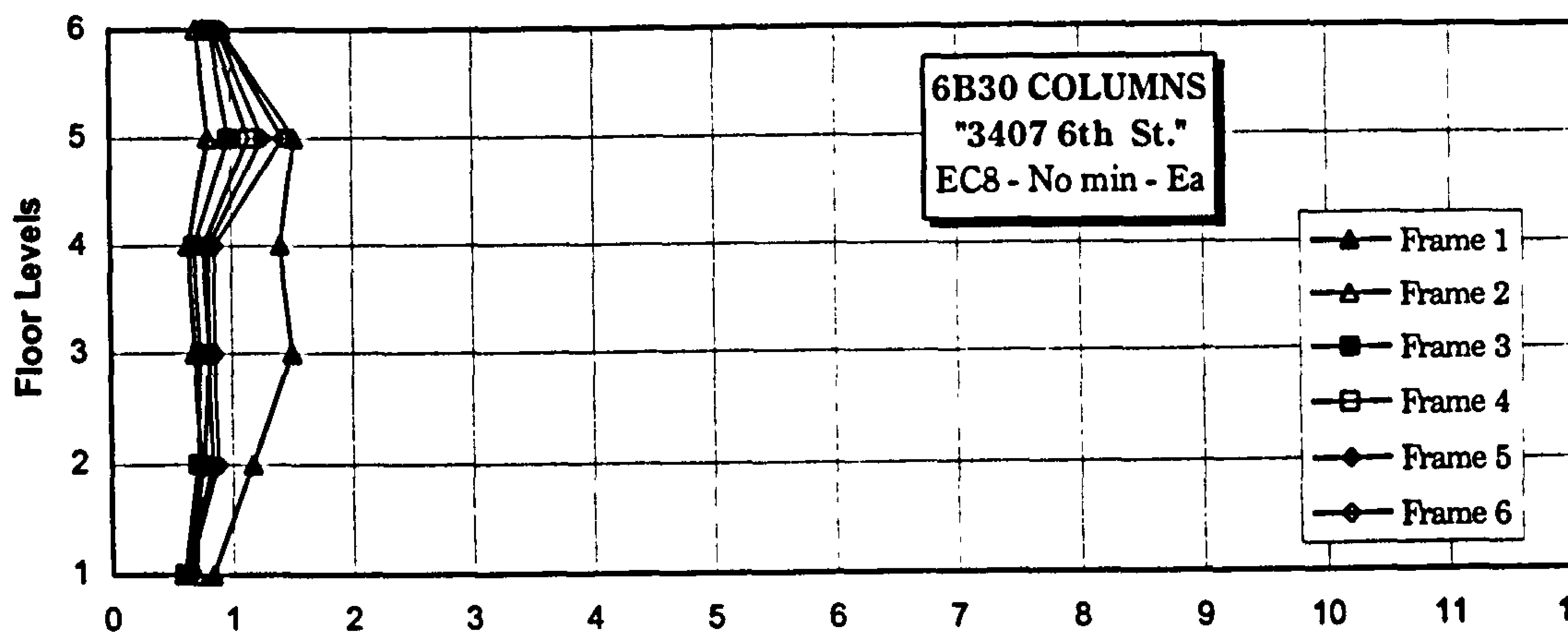
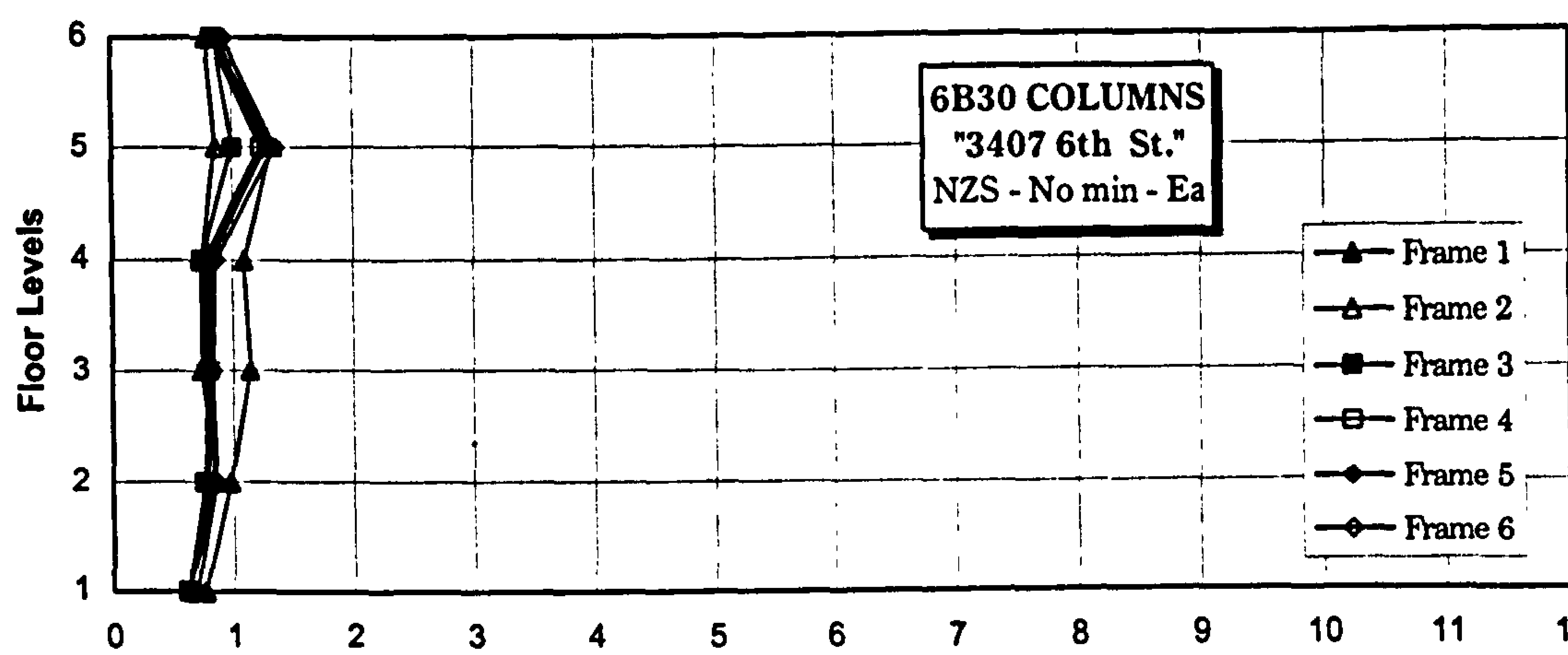


Figure 6.7.7 Rotational ductility demand vs. floor levels for the columns of model 6B30 analysed to different torsional provisions, designed for "No min – No ea" and excited by the "3407" 6th St" record.





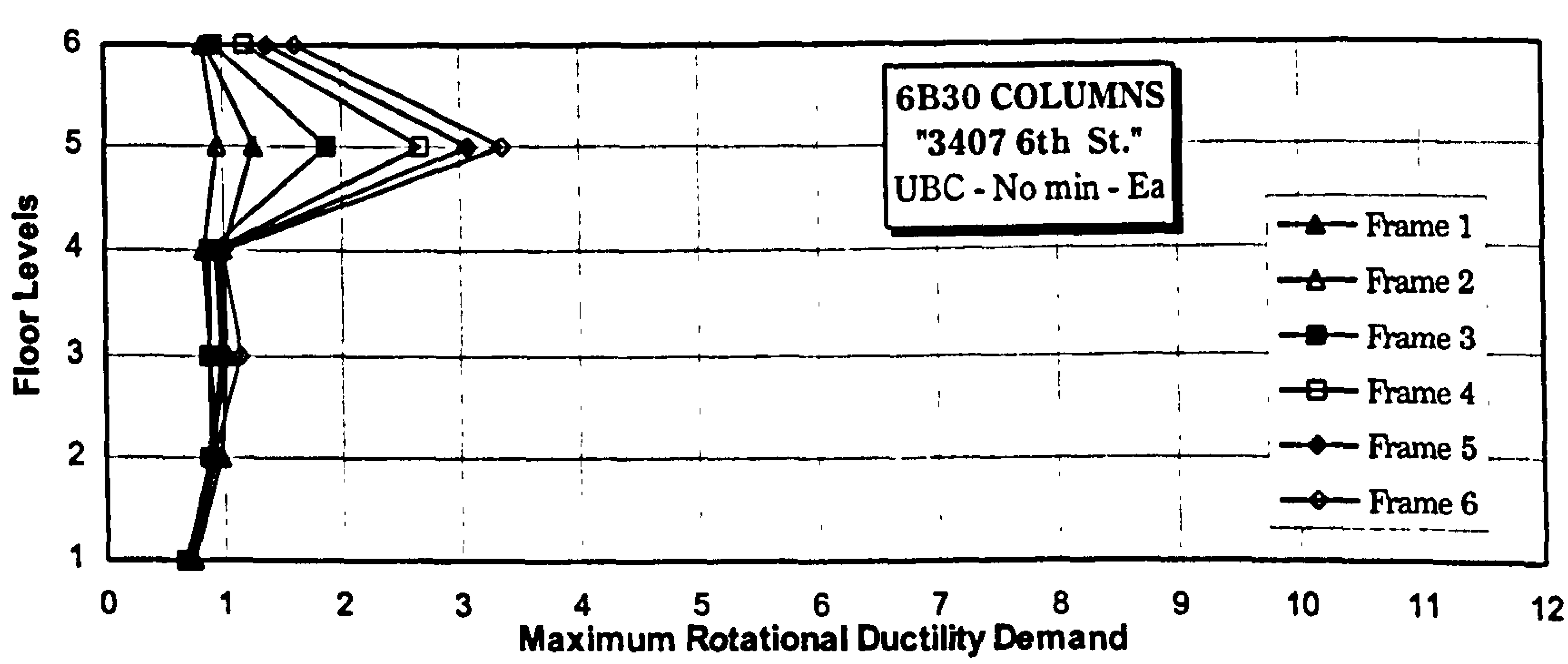


Figure 6.7.8 Rotational ductility demand vs. floor levels for the columns of model 6B30 analysed to different torsional provisions, designed for "No min – Ea" and excited by the "3407" 6th St" record.

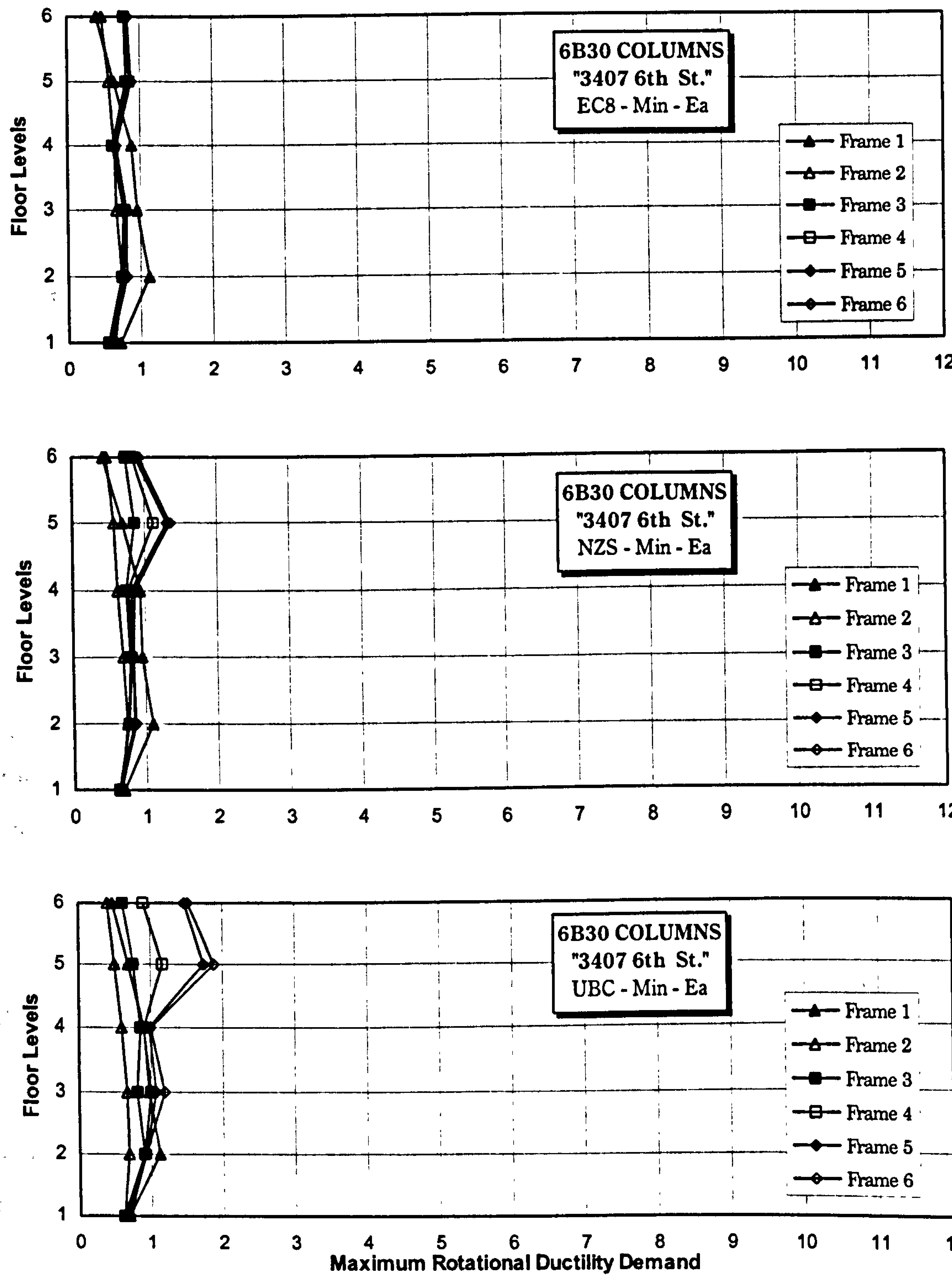


Figure 6.7.9 Rotational ductility demand vs. floor levels for the columns of model 6B30 analysed to different torsional provisions, designed for "Min – Ea" and excited by the "3407" 6th St" record.

CHAPTER 7

COMPARISON OF THE INELASTIC SEISMIC RESPONSE OF MULTI-STOREY REGULARLY ASYMMETRIC MODELS TO THE RESPONSE OF THEIR REFERENCE MODELS

7.1 INTRODUCTION

From the results of the parametric analyses carried out in Chapter 6, it has been shown that the inelastic seismic response of TU models is highly dependent on various factors (Sections 6.3 – 6.7). The major influence of the element strength distribution on the maximum ductility demand and deformation of the lateral load-resisting elements of TU models subjected to severe loading has been evaluated based on the results of the inelastic analyses carried out. Among the most important factors influencing the strength distribution of the models, and consequently their inelastic behaviour, is the design method adopted for the reinforcement calculation of the structural elements and the inclusion, or otherwise, of the accidental eccentricity.

A comprehensive assessment of the code torsional provisions can now be carried out, provided that a consistency is maintained in their application in both TU and reference models. The effect of the accidental eccentricity on the inelastic dynamic response of the models has been examined in Section 6.3 while, in this chapter, the response of a TU model is compared to the response of its reference model by employing three definition cases. In the first case, neither the TB nor the TU model incorporates the accidental eccentricity (case I), in the second case, only the TU model includes the accidental eccentricity (case II) while, in the final case, both TB and TU models incorporate the accidental eccentricity (case III). The effect of these definition cases is investigated in Section 7.2 and the influence of the design method adopted for the strength calculation of the models is also examined.

In Sections 7.3 – 7.6, the inelastic dynamic response of the same set of multi-storey regularly asymmetric models investigated in Chapter 6 is re-examined. Their inelastic response is now compared to the response of their reference models by using different definitions and by employing two design methods (Section 5.4.1). The factors analysed include the static eccentricity (Section 7.3), the model type (stiffness- and/or mass-eccentric) (Section 7.4), the structural period of the model (Section 7.5) and the torsional provisions of the seismic codes (Section 7.6). The influence of these factors on the inelastic dynamic behaviour of the TU models is evaluated based on the results of both Chapters 6 and 7. The most important factors influencing the inelastic seismic response of the TU models will be further examined in Chapter 8 by analysing TU models with transverse elements excited to both earthquake components.

7.2 INFLUENCE OF THE DIFFERENT DEFINITION CASES ADOPTED

7.2.1 Evaluation of the Inelastic Seismic Response of TU Models by Employing Different Definition Cases

The actual response of the reference models is critical in evaluating the performance and the effectiveness of code-designed TU models. Based on the definitions employed for the investigation of reference and TU models (Section 6.3), three different definitions are adopted for the comparison of the response of TU models to the response of their reference models. Depending on the inclusion, or not, of the accidental eccentricity, the ratio of the response of a TU model over the response of its reference model is firstly presented when neither the reference nor the TU model includes the accidental eccentricity (case I). In the second case, only the TU model includes the accidental eccentricity (case II) while, in the third case, both the reference and the TU models incorporate the accidental eccentricity (case III). The influence of the definition cases employed for the comparison of the response of the TU models to the response of their reference models is examined in this section. For this purpose, the response of model 6B45 (examined in Section 6.3.2) is compared to the response of its reference model 6B (investigated in Section 6.3.1), designed to the NZS code.

Figure 7.2.1 illustrates the ductility demand of the columns of model 6B45 over the ductility demand of the columns of model 6B, designed without the minimum steel ratios. Frame 1 results in the highest ductility demand ratios with averaging values around 2.3 and it is the only frame with response ratios exceeding unity in all storeys (for all cases) indicating that the ductility demand of frame 1 is higher in the TU model. The opposite occurs with the rest of the frames, which result in response ratios lower than unity, and frame 2 presents the lowest response ratios (averaging around 0.5). Case I results in the highest response ratios for frame 1, case II produces the lowest response ratios for all the frames while case III reduces the response ratios of frame 1 and increases the response ratios of the other frames (compared with case I). The inclusion of the accidental eccentricity only in the TU model (case II) results in low response ratios that could lead to wrong conclusions. Thus, a consistency should be maintained in the application of the torsional provisions in both TU and reference models (cases III and I).

Similar to the results presented in Section 6.3, the inclusion of the minimum steel ratios changes significantly the strength distribution of the models and leads to different response patterns (Figure 7.2.2). The ductility demand ratios of most of the columns are similar for all the cases while there are minor differences in the ductility demand ratios of the upper columns of frames 3 – 6. The normalised ductility demand of the upper columns of frames 3 – 6 always exceeds unity while frame 1 responds better in the TU case and it may even be conservatively designed at the upper storeys.

The higher response of the upper columns of frames 3 – 6 in the TU model is justified from their low strength increase due to the minimum steel ratios. The minimum reinforcement provisions increase more the strength of frames 2 – 6 in model 6B (Table 6.3.1) and the strength of frames 1 and 2 in model 6B45 (Table 6.3.2). Moreover, the upper columns are influenced more since they result in lower strength values from the results of the elastic static analyses. The higher inelastic response of the frames located at the flexible side is consistent with the conclusions reached by other researchers. De Stefano et al (1995) found that the structural damage of multi-storey asymmetric structures was located on the flexible side of the models. Furthermore, Chandler et al (1994) indicated that there is a significant additional ductility demand at the flexible side of short/medium-period structures designed to the UBC and NZS codes while the ductility demand ratios at the stiff side were always less than unity.

Consequently, the minimum steel ratios change significantly the response ratios of the frames while the definition cases concerning the inclusion of the accidental eccentricity affect only their maximum response values. De La Llera and Chopra (1994c) also concluded that the accidental eccentricity requirement is a refinement with small influence on the sizing and detailing of the models. Thus, it is mainly the inclusion of the minimum steel ratios that changes the strength distribution of the models and, as noticed by De Stefano et al. (1995), the capacity check and the minimum steel ratios of the codes govern the strength checks of the structural elements. Furthermore, the strength calculation of the structural elements based on two methods is consistent with the approach followed by Dolce and Ludovici (1992). They indicated the importance of the minimum reinforcement requirements in the design of the columns by employing the 1% minimum steel ratio of the codes and a lower steel ratio equal to 0.3%.

Similar conclusions regarding the definition cases employed for the comparison of the TU models with their reference models are reached when the ductility demand ratios of the beams are presented (Figures 7.2.3 and 7.2.4). The response patterns of the beams are similar to the response patterns of the columns, when the same design method is adopted. These remarks are also consistent with the conclusion reached by Dolce and Ludovici (1992) that the trends of the inelastic response of the beams are similar to the trends of the columns. Thus, in the following sections, the response ratios of the columns are mainly presented since their inelastic torsional response is of higher importance.

7.2.2 Conclusions regarding the Influence of Different Definition Cases

In Section 6.3.1, the different definitions used of the presentation for the inelastic response of the reference and TU models were investigated while, in Section 7.2.1, the definitions adopted for the comparison of the behaviour of the TU models with the behaviour of their reference models were examined. In these two sections, the effect of the accidental eccentricity on the inelastic response of the models was proved and the influence of the design method adopted was also analysed. All the points discussed in this section regarding the inclusion of the accidental eccentricity and the design method adopted are also noticed in the results of the inelastic analyses carried out in Sections 7.3 – 7.6. Thus, the conclusions regarding the effect of the different definition cases employed based on the inelastic analyses carried out in both Chapters 6 and 7 can be summarised as follows:

1. The importance of employing two design methods for the strength calculation of the models is justified from the results of the inelastic analyses. The minimum steel ratios alter significantly the strength distribution and the inelastic response of the models. The response pattern (degree of plastic hinging) of a TU model designed to a different method is quite variable and leads to contradictory conclusions regarding the inelastic behaviour of each frame. Employing two design methods helps in a better understanding of the response of TU models and indicates the significant influence of the minimum steel ratios. Thus, all the factors influencing the response of the TU models are investigated by employing both design methods.
2. The influence of the accidental eccentricity provisions is also indicated in relation to the design method. The minimum steel ratios increase the strength of the models and the influence of the accidental eccentricity cannot be clearly identified. Only without the incorporation of the minimum steel ratios, the influence of the accidental eccentricity can be observed. Therefore, the importance of employing two design methods for the strength calculation of the models is demonstrated once again.
3. The accidental eccentricity alters the strength distribution of the models and, hence, changes their response, especially when the minimum steel ratios are not included. Contrary to the minimum steel ratios, the accidental eccentricity reduces the response values of the frames without greatly altering their response patterns.
4. The differences in the response of TU models due to the minimum steel ratios and the accidental eccentricity are justified from the strength increase that each of these factors induces to the structural elements. The strength increase of each frame caused by the accidental eccentricity is dependent on its distance from the point the design

seismic forces are applied. The strength increase of the frames due to the minimum steel ratios depends on the dimensions of the structural elements and on the strength increase already induced by the accidental eccentricity.

5. The reduction of the response of each frame due to the accidental eccentricity and the minimum steel ratios is dependent on the strength increase that each one produces to the elements. The proportional reduction of the ductility demand of each frame is higher than its strength increase, indicating that the dynamically responding systems are more efficient in dissipating earthquake-induced vibrations than would be suggested by a linear relationship between ductility demand and strength.
6. Each frame in both reference and TU models results in a different cyclic loading history and, hence, in a different response pattern for each definition case. Only the reference models can result in identical ductility demand values for all their frames designed without the minimum steel ratios and the accidental eccentricity, indicating that only stiffness and strength proportional models respond purely in translation.
7. When the stiffness-eccentric models are designed without the minimum steel ratios, excessive ductility demand values are found in the stiff frame. By employing the 2nd design method, these excessive values are considerably reduced, indicating once more the great influence of the minimum steel ratios. The 2nd design method leads to more uniform ductility demand values for all the frames.
8. Similar to the ductility demand of the TU models, the normalised ductility demand is significantly affected by the minimum steel ratios. Different patterns of normalised response ratios are found for different design methods, and different conclusions are drawn regarding the response of the TU models.
9. The definition cases regarding the inclusion of the accidental eccentricity in the design of the models influence the maximum response values of the frames without altering their response patterns. Case II, which includes the accidental eccentricity only in the TU model, results in the lowest response ratios indicating that a consistency should be maintained in the application of the accidental eccentricity requirements in both reference and TU models.

10. The trends found regarding the response of the columns are always similar to the trends found for the beams. The inclusion of the accidental eccentricity and the design method influence in a similar way the torsional response of both columns and beams.

7.3 INFLUENCE OF THE STATIC ECCENTRICITY

7.3.1 Evaluation of the Inelastic Seismic Response of TU Models with Different Static Eccentricities

The influence of the static eccentricity was investigated in Section 6.4.1 by presenting the inelastic response of TU models with identical structural configurations but with different static eccentricities. In this section, the effect of the static eccentricity in the torsional behaviour of TU models is further investigated by comparing the response of the same TU models examined in Section 6.4.1 (models 6B15, 6B30 and 6B45) to the response of their reference model 6B. All systems are designed to the NZS regulations and both design methods are employed for the strength calculation of the models.

When the 1st design method is employed, frame 1 results in the highest values of normalised ductility demand and exceeds unity in almost all floor levels (Figures 7.3.1 – 7.3.3). Only in the upper floor levels of models 6B15 and 6B30, the normalised ductility demand of frame 1 is lower than the ductility demand of other frames since the rotational ductility of frame 1 is lower than the ductility demand of these frames (see also Figures 6.4.1 and 6.4.2). Similar to the results of Section 6.4.1, the pattern of the normalised ductility demand of frame 1 is always different from the patterns of the other frames, which are similar to each other. Moreover, the maximum values of normalised ductility demand of frame 1 are mainly located in the lower storeys while, the maximum values of the rest of the frames are found in the upper storeys. This is consistent with the results of Section 6.4.1, where excessive ductility demand values were located in the upper storeys of frames 2 – 6 and in the lower storeys of frame 1 (Figures 6.4.1 and 6.4.2).

The increase of the static eccentricity decreases the ductility demand ratios of frames 2 – 6 while it increases the ratios of frame 1 and the difference in the response ratios of frame 1 and the other frames. The same conclusion was also reached from the results of the inelastic analyses carried out in Section 6.4.1, where the ductility demand of frame 1 increased for higher static eccentricities while the ductility demand of the rest of the frames decreased. Thus, for the 1st design method, both the normalised (Figures 7.3.1 – 7.3.3) and the maximum ductility demand (Figures 6.4.1 and 6.4.2) lead to similar conclusions. Case II produces the lowest normalised ductility demand values since the torsional provisions are included only in the TU models (Figure 7.3.2). Cases III and I maintain a consistency in the application of the torsional provisions in TU and reference models and result in higher values of ductility demand ratios (Figures 7.3.1 and 7.3.3). Combining the results of the analyses carried out in both Chapters 6 and 7 indicates that the frame with ductility demand values higher than unity and with values of normalised ductility demand also higher than unity is mainly frame 1 (Figures 6.4.2 and 7.3.3).

For the 2nd design method, the patterns of the normalised ductility demand change completely and the columns with response ratios exceeding unity are mainly the upper columns of frames 3 – 6 (Figures 7.3.4 – 7.3.6). Case I produces the highest response ratios while the results of cases II and III are similar. Increasing the static eccentricity does not seem to affect significantly the normalised ductility demand. The response ratios of frames 1 and 2 are usually lower than unity and, in some cases, they are even reduced in the upper storeys. Therefore, the upper columns located at the flexible side of stiffness-eccentric models result in higher ductility demand values than the reference models while the opposite occurs to the upper columns of the stiff side. This is due to the fact that the stiff side of the TU models is influenced by the minimum steel ratios (Table 6.4.1) while the stiff side of the reference model (model 6B) is not affected at all (Table 6.3.1). Only in the lower storeys of TU models with low static eccentricities, the normalised ductility demand of frame 1 is higher than unity.

Consequently, both the normalised ductility demand (Figures 7.3.4 – 7.3.6) and the maximum ductility demand (Figures 6.4.4 and 6.4.5) lead to similar conclusions regarding the effect of the static eccentricity on the response of the TU models. However, in many cases, specific floor levels that result in a ductility demand lower than unity may respond worst than their reference model while floor levels responding better may result in a ductility demand higher than unity. Therefore, it is very important to examine the normalised response ratios in combination with the maximum response values. For the "Min – Ea" case, the ductility demand of the 5th floor level of frames 4 – 6 always exceeds unity irrespective of the static eccentricity and it is higher than the ductility of the reference model (Figures 6.4.4 and 7.3.6). In models with lower static eccentricities (models 6B15 and 6B30), the lower storeys of frame 1 exceed unity and respond worst than the TB model. The accidental eccentricity mainly reduces the maximum response values of the upper storeys and influences more the response of the model with the lowest static eccentricity (Figures 6.4.3 – 6.4.4 and Figures 7.3.4 – 7.3.6). Therefore, although the increase of the static eccentricity increases the ductility demand values for the 1st design method, including the minimum steel ratios results in higher response values for models with lower static eccentricities.

The normalised displacement of the external frames of the models investigated is illustrated (Figures 7.3.7 and 7.3.8) for all the earthquake records and the ground motion characteristics affect mainly the normalised deformation of the flexible side (frame 6). The wide range of response for the flexible side indicates that, as concluded by Chandler and Hutchinson (1992), the torsional response of the models is insensitive to the *A/V* ratio. All the earthquake records have similar values of *A/V* ratio, which are almost equal to unity, but they result in a varying inelastic response, especially in frame 6. The normalised displacement of frame 1 is usually lower than unity (Figure 7.3.7) while the normalised displacement of frame 6 is higher than unity (Figure 7.3.8). Therefore, the flexible side of the TU models exhibits higher lateral displacement than the flexible side of their reference model. This remark is consistent with the conclusion reached by Rutenberg (1992) that the flexible side results in higher values of lateral displacement. Chandler et al. (1995) also concluded that the normalised displacement is the most significant parameter for the flexible edge in stiffness-eccentric models and observed that the maximum displacement of a TU model could be 3 times the displacement of its reference model for high static eccentricities.

The mean normalised lateral displacement of frame 1 reduces for higher static eccentricities while the opposite occurs with frame 6, which results in higher lateral displacement in models with higher static eccentricities (see also Figure 6.4.6). Therefore, the torsional deformation of inelastic systems increases for higher static eccentricities, as also concluded by Goel and Chopra (1991a). Rutenberg et al. (1992a) noticed that the maximum lateral deformation is larger in TU systems while the peak ductility demand

and the maximum lateral displacement do not usually occur in the same element (also consistent with the results of this study). Finally, Tso and Wong (1995) concluded that the normalised edge displacement of a TU model depends on its static eccentricity, and Tso and Zhu (1992a) stated that the additional displacement demand at the flexible side always depends on the static eccentricity.

7.3.2 Conclusions regarding the Influence of the Static Eccentricity

In Section 6.4.1, the influence of the static eccentricity was investigated by presenting the response of TU models with the same structural configuration but with different static eccentricities. In Section 7.3.1, the normalised response values of the same set of models were presented and their response was compared to the response of their reference models. Therefore, the conclusions regarding the effect of the static eccentricity based on the analyses carried out in both Chapters 6 and 7 can be summarised as follows:

1. The inelastic response of the TU models is influenced by the static eccentricity in combination with the inclusion, or not, of the accidental eccentricity provisions and the minimum steel ratios. Models with identical structural configurations, but with different static eccentricities, are influenced differently by the accidental eccentricity since the design forces are applied at different locations. The minimum steel ratios increase the strength of each frame depending on its strength already induced by the accidental eccentricity. Therefore, TU models with different static eccentricities result in different strength distributions and, consequently, in different inelastic response.
2. The response pattern of the stiff frame in stiffness-eccentric models is always different from the response patterns of the rest of the frames, irrespective of the design method adopted and the inclusion, or not, of the accidental eccentricity. Contrary to the response of the stiff frame, the response of the other frames is similar.
3. When the first design method is employed for the design of the structural elements, the maximum ductility demand values are mainly located at the stiff frame. The increase of the static eccentricity increases the response of frame 1 while it reduces the response of the other frames and the difference between the response of frame 1 and the rest of the frames. The response of all frames in models with low static

eccentricities is similar while the response of frame 1 differs in models with higher static eccentricities. The maximum ductility demand values are usually located in the lower storeys of frame 1 and in the upper storeys of the other frames. The comparison of the response of TU models to the response of their reference models indicates that it is mainly frame 1 that responds worst than the corresponding frame of the reference model and models with higher static eccentricities result in higher ductility demand.

4. When the minimum steel ratios are included in the design, the ductility demand of the elements is considerably reduced. The response of the stiff frame is influenced more and it no longer results in the highest response values. Contrary to that, models with higher static eccentricities respond better when the minimum steel ratios are included due to their higher total strength increase. Thus, not only the strength distribution of the frames is important but also the total strength of the model. Furthermore, although the response of the frames reduces for models with higher static eccentricities, the upper storeys of the flexible side (frames 4 – 6) result in similar ductility demand values, higher than unity. Therefore, the frames with ductility demand higher than unity and higher than the ductility demand of the reference model are the upper floor levels of frames 4 – 6 and the lower floor levels of frame 1 in models with lower static eccentricities. Consequently, models with higher static eccentricities respond better due to the higher strength increase induced by the minimum steel requirements.
5. Irrespective of the design method adopted for the models, the maximum ductility demand values of frame 1 are mainly located in the lower storeys while the maximum values of frames 2 – 6 are found in the upper storeys. Furthermore, the normalised ductility demand increases in the upper storeys of frames 2 – 6 and in the lower storeys of frame 1 indicating that their total strength is lower in the TU models.
6. The flexible frame 6 results in the highest lateral displacement, increasing for models with higher static eccentricities. The difference between the displacement of frames 1 and 6 increase for models with higher static eccentricities indicating that the torsional deformation increases for higher static eccentricities.
7. The normalised lateral displacement of frame 1 is lower than unity while the normalised displacement of frame 6 exceeds unity indicating that the flexible side responds worst than the reference model. Hence, the normalised displacement is a

very significant parameter for the flexible edge of stiffness-eccentric models and the lateral deformation is higher in TU models rather than in their reference models.

8. The ground motion characteristics affect the normalised lateral displacement of the flexible side. Although the ground motions selected have similar A/V ratios, they result in a considerably different response and indicate that the inelastic torsional behaviour is insensitive to the A/V ratio.
9. The maximum interstorey drift ratios are lower than the 2% drift limit for all the TU models examined, indicating that there is no potential for collapse. The highest ductility demand values reach the value of 2.0, which is acceptable for models designed for the ultimate limit state.

7.4 INFLUENCE OF DIFFERENT MODEL TYPES

7.4.1 Evaluation of the Inelastic Seismic Response of TU Models with Different Stiffness Distributions

The influence of the lateral stiffness distribution on the response of different models having the same static eccentricity was presented in Section 6.5.1. The effect of the stiffness distribution on the inelastic torsional behaviour of the TU models is further examined in this section by comparing the behaviour of the same models investigated in Section 6.5.1 to the behaviour of their reference models. Therefore, the response ratios of models 6S30, 6A30 and 6B30 are presented when designed to both design methods.

When the models are designed by excluding the minimum steel ratios, the mass-eccentric model 6S30 results in ductility demand ratios higher than unity in frames 5 and 6 while, in the stiffness-eccentric models 6A30 and 6B30, it is mainly frame 1 that exceeds unity (Figures 7.4.1 – 7.4.3). Thus, for the 1st method, mass-eccentric models respond differently than stiffness-eccentric while a similar conclusion was reached in Section 6.5.1 and the frames that resulted in the highest ductility demand values are the frames that also result in values of normalised ductility demand higher than unity. Moreover, the stiffness-eccentric models 6A30 and 6B30 result in similar response while their response values differ significantly due to the varying stiffness and strength distributions (Table 6.5.1). The normalised ductility demand of frame 1 of model 6A30 (with values as high as 8) is higher than the ductility demand of frame 1 of model 6B30 due to the higher ductility demand of the former frame (Figures 6.5.1 and 6.5.2).

Employing the 2nd method (Figures 7.4.4 – 7.4.6) results in significantly different patterns of the normalised ductility demand with considerably lower values. For the stiffness-eccentric models, the columns with response ratios exceeding unity are mainly the upper columns of frames 3 – 6 and the lower columns of frame 1 (case III). For the mass-eccentric model 6S30, the columns with response ratios higher than unity are the columns of frame 6, the lower columns of frame 5 and the upper columns of frames 1 and 2. Therefore, although the maximum response ratios of the models are found in the same range, the frames with normalised ductility demand exceeding unity differ depending on the stiffness distribution.

It should be also noticed that the frames with normalised ductility demand values higher than unity are the same frames that resulted in the highest ductility demand in Section 6.5.1. The frames with both the ductility demand and the normalised ductility higher than unity are the 2nd storey of frame 1 and the 5th storey of frames 4 – 6 in model 6B30. In model 6A30, the frames exceeding unity are the 2nd – 4th storeys of frame 1 and the 5th storey of frames 5 and 6. Finally, in model 6S30, the frames with ductility demand and normalised ductility higher than unity are the 2nd – 5th floor levels of frame 6, the 2nd floor level of frame 5 and the 5th floor level of frames 1 and 2.

However, when the minimum steel ratios are incorporated, the response ratios increase in the upper storeys of frames 3 – 6 of stiffness-eccentric models and in the upper storeys of frames 1 and 2 of mass-eccentric models. The increase in the normalised ductility demand caused by the minimum steel ratios is attributed to the different strength increase of a frame when located in the reference model and in the TU model. The ductility demand of the upper storeys of frames 3 – 6 in stiffness-eccentric models and of the upper storeys of frames 1 and 2 in the mass-eccentric model are always higher than unity for both design methods (Figures 6.5.1 – 6.5.4). However, the normalised ductility demand of these frames exceeds unity only for the 2nd method (Figures 7.4.1 – 7.4.6) indicating that the minimum steel ratios increase more the strength of these frames when located in the reference models.

Figure 7.4.7 presents the normalised lateral displacement of the edge elements of the models investigated, excited to the "3407 6th St." earthquake record and designed to the 2nd method. In stiffness-eccentric models, the maximum normalised deformation is found at the flexible side (frame 6) while, in the mass-eccentric model, the normalised deformation of frames 1 and 6 is similar. In models 6A30 and 6B30, frame 1 results in a lateral displacement lower than the displacement of the reference model. Frame 6 of model 6B30 that resulted in the highest lateral displacement (Figure 6.5.6), now has the highest normalised displacement while frame 1 has the lowest. The mass-eccentric model 6S30 has similar values of normalised displacement for both frames 1 and 6 while the most vulnerable side is the flexible side of stiffness-eccentric models.

Goel and Chopra (1990a), who examined the effects of stiffness and strength distribution in the inelastic torsional behaviour of single-storey asymmetric models, also concluded that mass-eccentric and stiffness-eccentric systems with identical structural periods and static eccentricities result in a different inelastic response. Furthermore, similar to the results of this study, Tso and Ying (1992) indicated that there is an additional ductility demand at the flexible side of stiffness-eccentric models while, in mass-eccentric models, the additional ductility demand is located at the other side of the models. Finally, they concluded that the effects of torsion might result in a substantial additional ductility demand at the flexible-edge element of stiffness-eccentric models.

7.4.2 Conclusions regarding the Influence of the Stiffness Distribution

In Section 6.5.1, the influence of the stiffness distribution was investigated by presenting the response of different model types having the same static eccentricity but different stiffness distributions. In Section 7.4.1, the normalised response values of the same set of models (models 6S30, 6A30 and 6B30) were presented and their response was compared to the response of the reference models. Therefore, the conclusions regarding the effect of the different model types are based on the inelastic analyses carried out in both Chapters 6 and 7, and they can be summarised as follows:

1. Models with the same static eccentricity but with different structural configurations respond differently and their ductility demand values differ in both magnitude and

distribution. Mass-eccentric models result in different response patterns than stiffness-eccentric models due to their structural configuration and the influence of the accidental eccentricity and of the minimum steel ratios.

2. For the 1st design method, the strength of the models depends on their stiffness distribution and on the point of application of the design seismic forces. In stiffness-eccentric models, the maximum ductility demand is located in the stiff frame while, in mass-eccentric models, the maximum inelastic response is found in frames 5 and 6. The ductility demand values differ in each model and models 6A30 and 6S30 result in a higher inelastic response than model 6B30.
3. The minimum steel ratios increase the strength of frames 1 – 3 in stiffness-eccentric models and of frames 4 – 6 in mass-eccentric models. When the accidental eccentricity provisions are also included, the frames influenced more by each factor depend on the model type. When the strength of a frame is considerably increased by one of these two factors, then it is less influenced by other factor. Therefore, the accidental eccentricity increases the strength of frames 1 – 3 in mass-eccentric models and the strength of frames 4 – 6 in stiffness-eccentric models.
4. For the 2nd design method, the response of the frames is considerably reduced and different models result in similar ductility demand. For stiffness-eccentric models, the columns with response ratios higher than unity are the upper columns of frames 3 – 6 and the lower columns of frame 1. For mass-eccentric models, the columns exceeding unity are the lower columns of frames 5 and 6 and the upper columns of frame 1.
5. The inclusion of the minimum steel ratios results in lower ductility demand and normalised ductility demand values. However, for specific frames, it might result in higher normalised ductility demand due to the different way that they influence the same frame when located in the TU or in the TB model. The strength increase of a frame might be higher in the reference model and, hence, the normalised response ratios result in higher values. Thus, the normalised response values should be always examined together with the response values of the TU and reference models.
6. The frame with the highest lateral displacement is the flexible frame in stiffness-eccentric models. In mass-eccentric models, both external frames have the same stiffness and the maximum lateral displacement can be located in either of them

depending on the ground motion characteristics. The normalised displacement of frames 1 and 6 in mass-eccentric models is similar while the normalised displacement of frame 6 is the highest in stiffness-eccentric models.

7. The maximum interstorey drift ratios for different model types are always lower than the 2% collapse limit, and they are even below 1% indicating no potential for collapse. Furthermore, the maximum ductility demand does not exceed the value of 2.0, which is considered acceptable for models designed for the ultimate limit state.

7.5 INFLUENCE OF THE NUMBER OF FLOOR LEVELS

7.5.1 Evaluation of the Inelastic Seismic Response of TU Models with Different Numbers of Floor Levels

In Section 6.6.1, the influence of the number of floor levels (building height) on the inelastic response of the TU models was examined by illustrating the response of TU models with the same stiffness and mass distributions but with various numbers of floor levels. From the results of the inelastic analyses carried out in Chapter 6, it was concluded that the model height influences significantly the response of the models only when the minimum steel ratios are not included in their design. For the 1st design method, the ductility demand of the models increased for higher buildings while, for the 2nd design method, the inelastic response range of TU models with varying heights was similar. The importance of the fundamental structural period is further investigated in this section by comparing the behaviour of the same set of TU models investigated in Section 6.6.1 (models 6B45, 12B45 and 24B45) to the behaviour of their reference models. All models are designed to the NZS code for all the definition cases and for both design methods.

When the models are designed without the minimum steel ratios, the response ratios of case I are clearly the highest (reaching the value of 25 in model 24B45) while the response ratios of cases II and III are similar (reaching the value of 10 in model 24B45). The ductility demand ratios exceeding unity are mainly the ratios of frame 1 since the ductility demand of this frame is by far the highest for the 1st design method (Figures 6.6.1 and 6.6.2). As expected, the stiff frame of the highest model (model 24B45) presents the highest normalised ductility demand due to its excessive values of ductility demand when designed without the minimum steel ratios. Moreover, for case I, frames 2 – 4 of model 24B45 also result in normalised ductility demand values higher than unity, while the normalised ductility demand of the stiff frame is by far the highest (Figure 7.5.1). Consequently, without the minimum steel ratios, TU models with higher structural periods result in higher ductility demand values (Figures 6.6.1 and 6.6.2) and the stiff frame is always the most vulnerable.

By employing the 2nd design method (Figures 7.5.4 – 7.5.6), the patterns of normalised ductility demand change completely since the inelastic response of the models also changes by including the minimum steel ratios (Figures 6.6.3 and 6.6.4). For model 6B45, the columns with response ratios exceeding unity are the upper columns of frames 4 – 6 while, for models 12B45 and 24B45, the response ratios of frames 4 – 6 exceed unity in all floor levels. Furthermore, no major differences are encountered for the three definition cases and the increase of the model height increases the normalised response values indicating that the difference between the response of the TU and the reference models is higher for higher structures. Only the upper storeys of the flexible elements result in ductility demand higher than unity, higher than the ductility demand of the reference models (Figures 6.6.4 and 7.5.6). Thus, the upper columns of frames 4 – 6 are vulnerable when designed to the 2nd method and result in similar ductility demand values.

Consequently, the height of the TU models influences the response of the structural elements only when they are designed without the minimum steel ratios (Figures 6.6.1 and 6.6.2) while, when the minimum reinforcement requirements are included, no major differences are encountered in their response (Figures 6.6.3 and 6.6.4). The same conclusion was also reached by Goel and Chopra (1991b) who noticed that the effects of the plan asymmetry in elastic systems depend significantly on the structural period, while the period dependence is less pronounced in inelastic systems.

The conclusion reached by Duan and Chandler (1993) that the upper columns of the stiff-edge increase their inelastic response with the increase of the structural period is consistent with the results of this study only when the minimum steel ratios are not incorporated. The strength increase of the stiff frame caused by the minimum steel ratios leads to a ductility demand of the stiff-edge elements lower than unity, which reduces for higher structures (Figures 6.6.3 and 6.6.4). Rutenberg (1992) noticed that, although the peak ductility demand is higher in asymmetric systems rather than in symmetric, the peak ductility demand reduces for increasing model height. Rutenberg et al (1992a) concluded that the normalised ductility demand of eccentric systems is not dependent on their structural period while Figures 7.5.3 and 7.5.6 indicated that the normalised ductility demand increases for higher structures, irrespective of the design method employed.

As also indicated in Figure 6.6.6, Rutenberg (1992) observed that the maximum lateral displacement of TU models increases with increasing period while Rutenberg et al. (1992b) noticed that the period dependence of the peak ductility demand depends on the ground motion characteristics. Figure 6.6.6 indicated that the response of the higher models 12B45 and 24B45 is influenced in a similar way by each earthquake record while the response of the lower model 6B45 is different. For example, the maximum lateral displacement in models 12B45 and 24B45 is caused by the "3407 6th St." earthquake record, which produces the lowest lateral displacement in model 6B45.

Tso and Ying (1992) found that the torsional effects result in additional inelastic displacement for the flexible side that can be substantial, especially in stiffness-eccentric models irrespective of the model height. Figure 7.5.7 indicates that the normalised lateral displacement of TU models with varying heights is highly dependent on the earthquake characteristics. Chandler and Hutchinson (1992) concluded that the effect of torsion on the edge displacement is more pronounced in short-period structures, as seen in Figure 7.5.7 where the model with the lowest period results in higher values of normalised displacement. Goel and Chopra (1991a) also reached to the same conclusion and noticed that short-period inelastic structures result in a larger increase in deformation.

Finally, Correnza (1994) indicated that the additional deformation demand of the flexible edge elements is significantly high for short-period systems (Figure 7.5.7). He also concluded that significant additional ductility demand arises in the flexible side of short/medium-period systems, when designed to static torsional provisions that do not amplify the static eccentricity (UBC and NZS codes). Additional ductility demand arises in the flexible edge elements of the models examined in this section and designed to the NZS code while the torsional provisions of different seismic codes are analytically investigated in Section 7.6. Correnza (1994) noticed that all code provisions are overly conservative for systems with long lateral periods and, in Figure 6.6.4, it was also noticed that the ductility demand of the highest model is the lowest.

7.5.2 Conclusions regarding the Influence of the Number of Floor Levels

In Section 6.6.1, the influence of the lateral period of the models was investigated by presenting the response of TU models having the same stiffness and mass distributions but different numbers of floor levels. In Section 7.5.1, the normalised response values of the same models (models 6B45, 12B45 and 24B45) were presented and their response was compared to the response of the reference models. Hence, the conclusions regarding the effect of the number of storeys in the inelastic response of the TU models are based on the analyses carried out in both Chapters 6 and 7 and can be summarised as follows:

1. For the 1st design method, the stiff frame results in excessive ductility demand values while the response of the other frames is much lower. The highest ductility demand values are found at the stiff frame of the tallest model and the increase of the model height increases the response of the models. Excessive values of ductility demand are found even when the accidental eccentricity is incorporated and they are attributed to the exclusion of the minimum steel ratios from the design of the models. Thus, these ductility demand values would not generally arise in practice, and they only indicate the effect of the minimum steel ratios on the design of the models.
2. For the 2nd design method, the ductility demand values are significantly reduced and the stiff frame is no longer the most vulnerable, having ductility demand values lower than unity and similar for TU models with varying heights. The upper storeys of frames 4 – 6, located at the flexible side, result in higher response values. Generally, the response of frames 2 – 6 is always higher in the upper storeys, irrespective of the design method. Therefore, for the 2nd method, the increase of the model height does not influence significantly the response of the models while the ductility demand slightly decreases for higher structures.
3. The flexible frame of stiffness-eccentric models results in the highest lateral displacement while the increase of the structural period increases the response values. Torsional effects result in additional inelastic displacement of the flexible side, which

can be substantial irrespective of the model height and highly dependent on the earthquake characteristics. The effect of torsion on the edge displacement is more pronounced in short-period structures, resulting in higher normalised displacements.

4. The maximum interstorey drift ratios of all models are always lower than the 2% collapse limit and the highest value of interstorey drift ratio is found in the lowest model (the 6-storey model). The low interstorey drift ratios and the ductility demand values lower than 2.0 indicate no potential for collapse for structures designed for the ultimate limit state.

7.6 INFLUENCE OF DIFFERENT TORSIONAL PROVISIONS

7.6.1 Evaluation of the Inelastic Seismic Response of TU Models Designed to Different Torsional Provisions

The influence of different torsional provisions in the design of TU models was investigated in Section 6.7.1 by presenting the inelastic torsional behaviour of different model types (models 6S30, 6A30 and 6B30) designed to the EC8, NZS and UBC regulations. The influence of the design eccentricities of these codes on the strength distribution of the models was presented and the effect of the base shear values calculated with each seismic code was also indicated. In Section 6.7.1, the models were excited to only one earthquake record in order to observe more clearly the influence of the torsional provisions. In this section, the average response value of the TU models is presented when subjected to all the earthquake records selected. The influence of different static torsional provisions will be further examined in Chapter 8 where the behaviour of TU models with transverse elements will be presented for the three seismic codes.

Due to the inclusion of the minimum reinforcement requirements, no major differences are noticed in the response of model 6S30 designed to different torsional provisions ("Min – Ea") (Figure 7.6.1). The frames resulting in ductility demand values higher than unity are mainly frames 5 and 6 (especially in the lower storeys) and frame 1 in the 5th floor level (as also indicated in Section 6.7.1). Generally, the ductility demand of frames 4 – 6 increases in the lower storeys while the ductility demand of frames 1 and 2 increases in the upper storeys. The maximum response values of each frame are similar for any seismic code while the ductility demand of frame 1 increases for the UBC code. This is justified from the fact that frames 1 and 2 result in considerably different strength values when designed to different codes and the lowest strength value is calculated for UBC (Table 6.7.4). The rest of the frames (frames 4 – 6) have similar strength values due to the inclusion of the minimum steel ratios (Tables 6.7.2 – 6.7.4).

The columns with normalised ductility demand higher than unity are the columns of frame 6, the lower columns of frame 5 (for EC8 and NZS) and the upper columns of frames 1 and 2 (Figure 7.6.2). The normalised ductility demand values of frames 1 and 2 are higher for UBC while the normalised ductility demand values of frames 5 and 6 are higher for EC8. Therefore, for the EC8 and NZS codes, the highest normalised ductility demand is found in the 2nd floor level of frame 6 while, for the UBC code, the 5th floor level of frame 1 results in the highest ductility demand ratio. The normalised ductility also increases in the upper storeys of frames 1 – 3 and in the lower storeys of frames 5 and 6. Therefore, the columns with a ductility demand higher than unity and an inelastic response higher than the response of the reference model are mainly the columns of frame 6 for all the seismic codes adopted. Although the number of columns with ductility demand higher than unity is higher for the UBC code, the columns with a normalised ductility demand and a rotational ductility demand higher than unity are less for this code. This is due to the fact that the normalised ductility demand depends on the response of the reference model, which is higher for UBC and, therefore, results in higher values of normalised ductility demand.

For the stiffness/mass-eccentric model 6A30, the highest ductility demand values are always found in the 2nd floor level of frame 1 for all the seismic codes employed (Figure 7.6.3). Apart from frame 1, which always exceeds unity in almost all floor levels, the upper floor levels of frames 5 and 6 also exceed unity when designed to the NZS and UBC codes. Therefore, the main difference in the response of model 6A30 designed to different torsional provisions is the significant increase of ductility demand in the upper storeys of frames 5 and 6 for the UBC code. Frames 5 and 6 result in considerably different strength values when designed to different codes while frames 1 and 2 are mainly influenced by the minimum steel ratios and result in the same total strength (and consequently similar inelastic response) for any seismic code adopted (Table 6.7.4).

The normalised ductility demand values of model 6A30 indicate differences in the response of the model when designed to a different code (Figure 7.6.4). For the EC8 and NZS codes, the lower storeys of frame 1 and the upper storeys of frames 5 and 6 exceed unity while, for the UBC code, only the upper storeys of frames 5 and 6 exceed unity. The maximum normalised ductility demand is found for the UBC code and reaches the value of 2.5 at the 5^{th} and 6^{th} floor levels of frame 6. The columns with both a ductility demand and a normalised ductility demand higher than unity are the lower columns of frame 1 for EC8 and NZS, and the upper columns of frames 5 and 6 for UBC (Figures 7.6.3 and 7.6.4). Thus, only for the UBC, the ductility demand of frames 5 and 6 in the TU model exceeds unity and is higher than the ductility demand of the reference model.

The response of the stiffness-eccentric model 6B30 is similar to the response of the stiffness/mass-eccentric model 6A30 (Figures 7.6.5 and 7.6.6). The ductility demand values of the lower columns of frame 1 are always higher than unity for any code while the ductility demand of the upper storeys of frames 4 – 6 exceeds unity only for NZS and UBC. The response values of frame 1 are similar for all codes while the response values of frames 4 – 6 increase for the UBC code. The response of model 6B30 is justified from the fact that the strength of frames 1 and 2 is identical for any code (Table 6.7.4). The other frames result in different strength values when designed to different code provisions and the most significant differences are observed in the strength of frames 4 – 6.

The normalised ductility demand (Figure 7.6.6) indicates an increased ductility demand in the upper storeys of frames 3 – 6 for all codes and an increased ductility demand in the lower storeys of frame 1 for EC8 and NZS. The maximum normalised ductility demand values are found in the 5^{th} and 6^{th} storeys of frames 5 and 6 when designed for UBC (almost reaching the value of 3.5). For the NZS code, the normalised ductility reaches the value of 2.0 in the 5^{th} storey of frame 6 and, for the EC8 code, the 6^{th} storey of frame 6 reaches 1.5. Thus, the most vulnerable storeys are the 2^{nd} and 3^{rd} floor levels of frame 1 for EC8 code, the 5^{th} storey of frames 4 – 6 and the 2^{nd} storey of frame 1 for NZS, and the 3^{rd} – 6^{th} floor levels of frames 5 and 6 for UBC. Generally, the stiffness-eccentric model 6B30 responds best for the EC8 code and worst for the UBC code.

Therefore, the inelastic time-history analyses (Sections 6.7.1 and 7.6.1) indicated that the influence of different torsional provisions in the behaviour of TU models depends on many factors. These factors include the design eccentricities, the design seismic forces, the incorporation of the minimum steel ratios in the design of the models, and the model type. Similar to the results of this study, Correnza et al. (1994) concluded that there is an additional ductility demand at the flexible side of short/medium-period stiffness-eccentric structures designed to NZS and UBC codes. Including an additional eccentricity in the EC8 first design eccentricity seemed to be necessary to control the ductility demand of the flexible side. Figures 7.6.3 and 7.6.5 showed that the ductility demand of the flexible side is the lowest for EC8 and the highest for UBC. Correnza et al. (1994) proved that the normalised ductility of the stiff side is lower than unity for any code whereas this study indicated that there is an additional ductility when designed to a code that permits the strength reduction of the stiff side (EC8 and NZS codes).

Chandler and Duan (1991b) also noticed that the second design eccentricity of the EC8 and NZS codes are inadequate for the stiff side while Chandler et al. (1993) observed that EC8 offers no consistent protection in the inelastic range. Duan and Chandler (1993) found that EC8 and NZS provisions are non-conservative particularly for intermediate/large static eccentricities while no additional ductility demand of the stiff side is found for the UBC code. Goel and Chopra (1992) stated that the seismic codes allowing design forces in asymmetric models smaller than in the symmetric ones result in additional ductility demand at the stiff side. Tso and Wong (1995) concluded that, when the lateral strength is distributed based on the EC8 static torsional provisions, there is a large ductility demand at the stiff side. By examining the torsional provisions of the UBC code, Wong and Tso (1995) observed that this code provides a substantial strength increase in the stiff side and reduces its additional ductility demand.

7.6.2 Conclusions regarding the Influence of Different Torsional Provisions

In Section 6.7.1, the influence of different torsional provisions was investigated by presenting the response of various 6-storey TU models designed to the EC8, NZS and UBC torsional provisions. In Section 7.6.1, the normalised response values of the same set of TU models (models 6S30, 6A30 and 6B30) were presented and their response was compared to the response of the reference models. Therefore, the conclusions regarding the effect of different torsional provisions based on the inelastic analyses carried out in both Chapters 6 and 7 can be summarised as follows:

1. When the accidental eccentricity and the minimum steel ratios are excluded from the design of the models, the total strength of each model depends only on the base shear value while their strength distribution depends on the stiffness distribution of the models. Consequently, the inelastic response patterns are identical for all codes while their response values differ depending on the design seismic forces. Hence, all model types respond better for the EC8 regulations and worst for the UBC regulations.
2. When the accidental eccentricity is included while the minimum steel ratios are still excluded from the design of the elements, the inelastic behaviour of the models depends not only on the design seismic forces but also on their points of application. The torsional provisions of different seismic codes influence each model type differently due to the varying points of application of the seismic forces. The strength of the models is mainly influenced by the base shear value and, although the UBC design eccentricities result in the highest total strength for all models, the low base shear value of this code results in the lowest total strength.
3. In mass-eccentric models, the UBC provisions requiring an amplification factor for the calculation of the first design eccentricity result in the highest strength for frames 1 – 3 and frame 6. The maximum strength of frames 4 and 5 is produced by the second design eccentricity of the same code that requires the strength of the TU model to be at least equal to the strength of the reference model. In stiffness-eccentric models, the EC8 first design eccentricity that includes an additional eccentricity component influences the strength of frames 3 – 6 while the strength of frames 1 and 2 is influenced by the UBC second design eccentricity.
4. When the minimum steel ratios are included, the response values of different model types designed to various seismic codes present no major differences. The minimum steel ratios increase the strength of the elements and the effect of different torsional provisions cannot be clearly observed in the torsional behaviour of the models. Only minor differences in the response of the same model designed to different torsional

provisions can be noticed while the response values for the UBC code are slightly higher due to the lower base shear value of this seismic code.

5. In stiffness-eccentric models, the frames responding in the inelastic range are the lower storeys of frame 1 (for all codes) and the upper storeys of frames 5 and 6 (for NZS and UBC). EC8 code requiring an additional eccentricity for the first design eccentricity is the only code that results in no ductility demand at the flexible side while the highest ductility demand for these frames is found for the UBC code. The frames that respond worst than their reference models are the upper storeys of frames 3 – 6 (for all codes) and the lower storeys of frame 1 (for EC8 and NZS codes). The UBC provision permitting no strength reduction in the TU model results in no additional ductility demand in the stiff side. The normalised ductility demand of frame 1 is higher for EC8 code while the normalised ductility demand of frames 5 and 6 is higher for UBC. Consequently, the EC8 code controls better the ductility demand of the flexible while the response of the stiff side is better controlled by UBC.
6. In mass-eccentric models, the frames responding in the inelastic range are the lower floor levels of frames 5 and 6 (for all codes) and the upper floor levels of frame 1 (for NZS and UBC). Contrary to the inelastic seismic response of stiffness-eccentric models, the inelastic response of mass-eccentric models is similar of all seismic codes adopted and no major differences are encountered.
7. The response of mass-eccentric models is worst than their reference models in frame 6 and in the upper storeys of frames 1 and 2 (for all codes) while the normalised ductility demand of the lower floor levels of frame 5 also exceeds unity for EC8 and NZS. The UBC provision permitting no strength reduction in the TU model results in no additional ductility demand in frame 5. The normalised ductility demand of frames 5 and 6 is higher for EC8 code while the normalised ductility demand of frames 1 and 2 is higher for UBC.

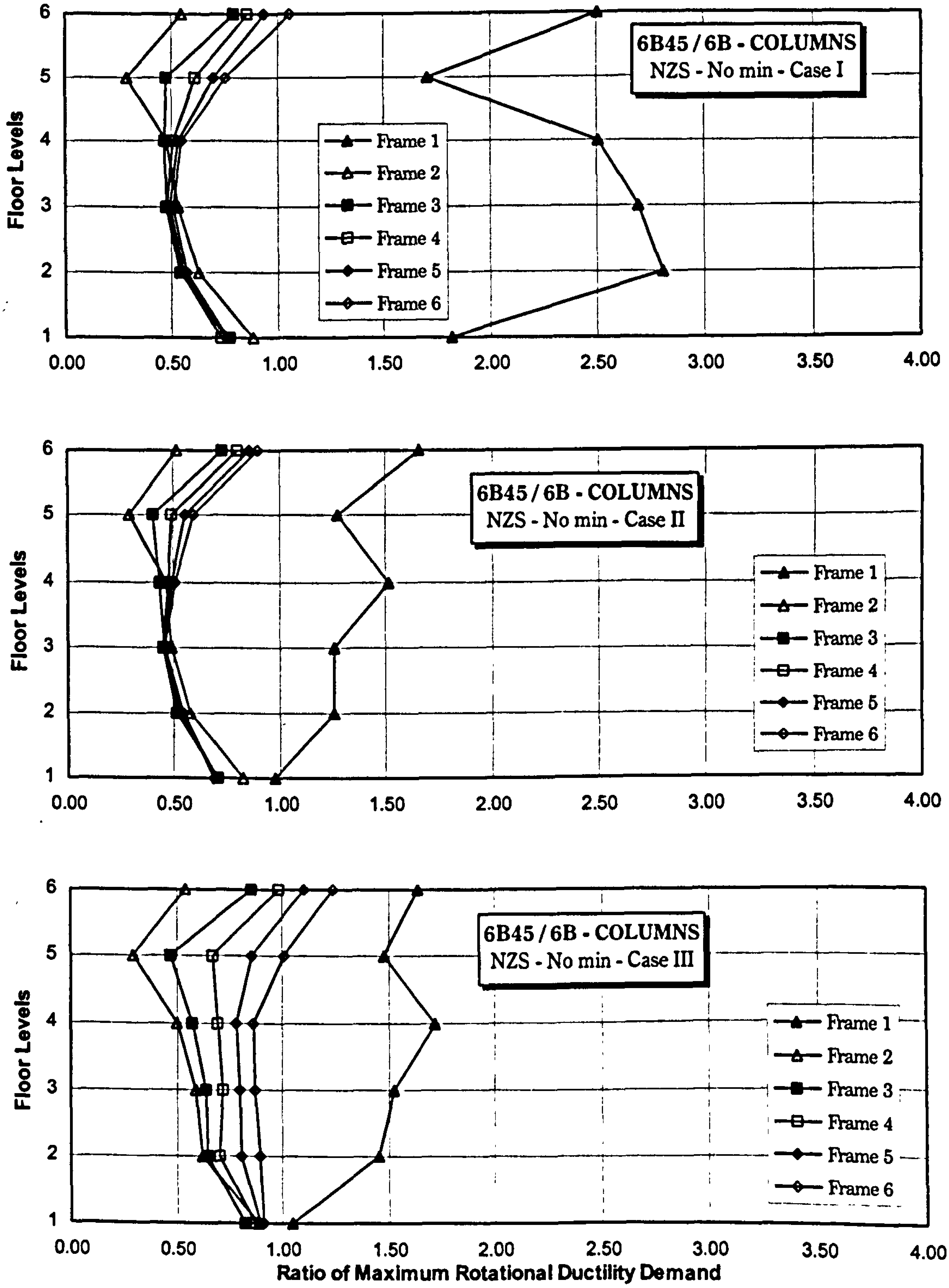


Figure 7.2.1 Ratios of rotational ductility demand vs. floor levels for the columns of model 6B45/6B designed to the NZS code and the 1st design method.

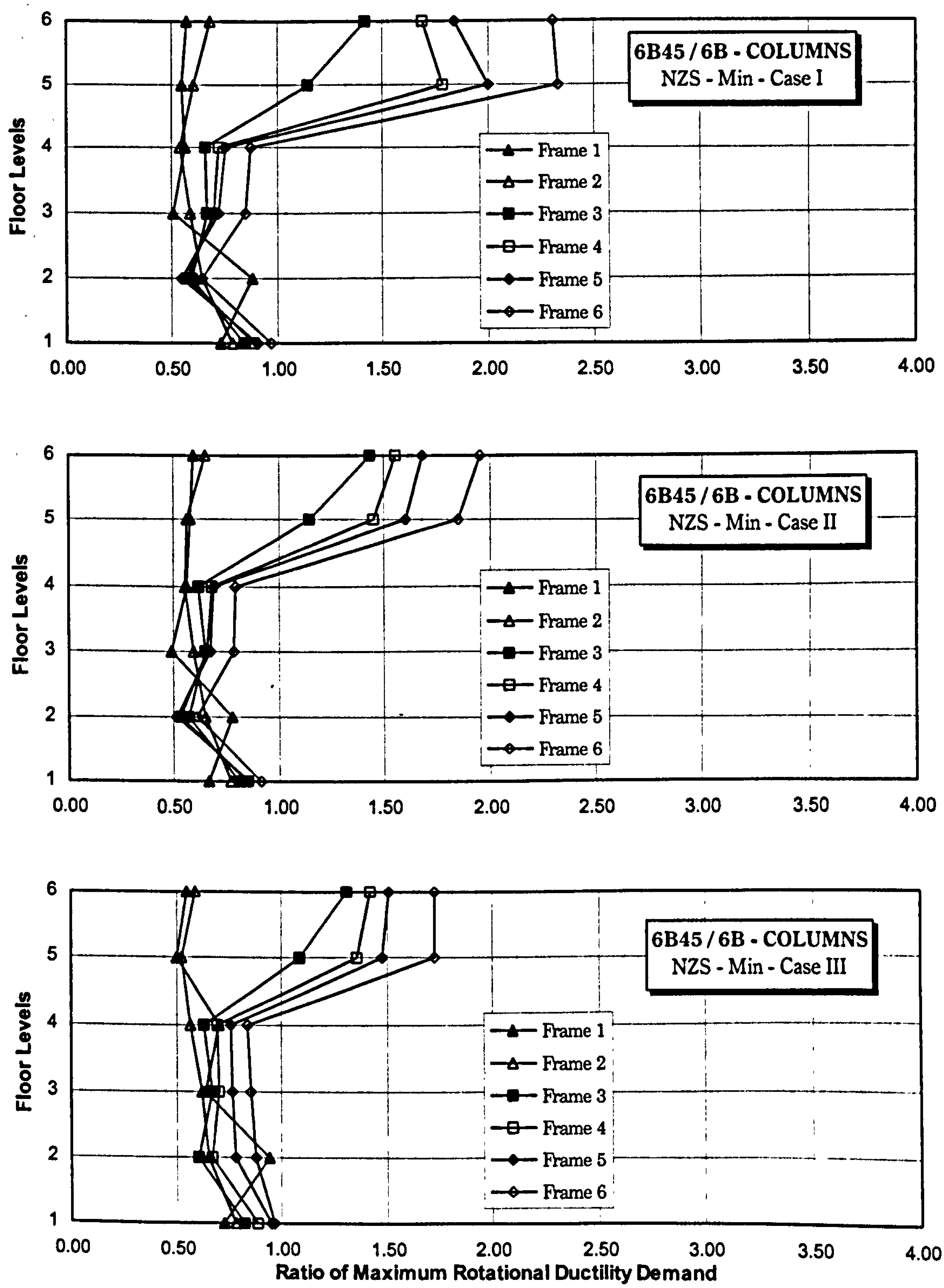


Figure 7.2.2 Ratios of rotational ductility demand vs. floor levels for the columns of model 6B45/6B designed to the NZS code and the 2[nd] design method.

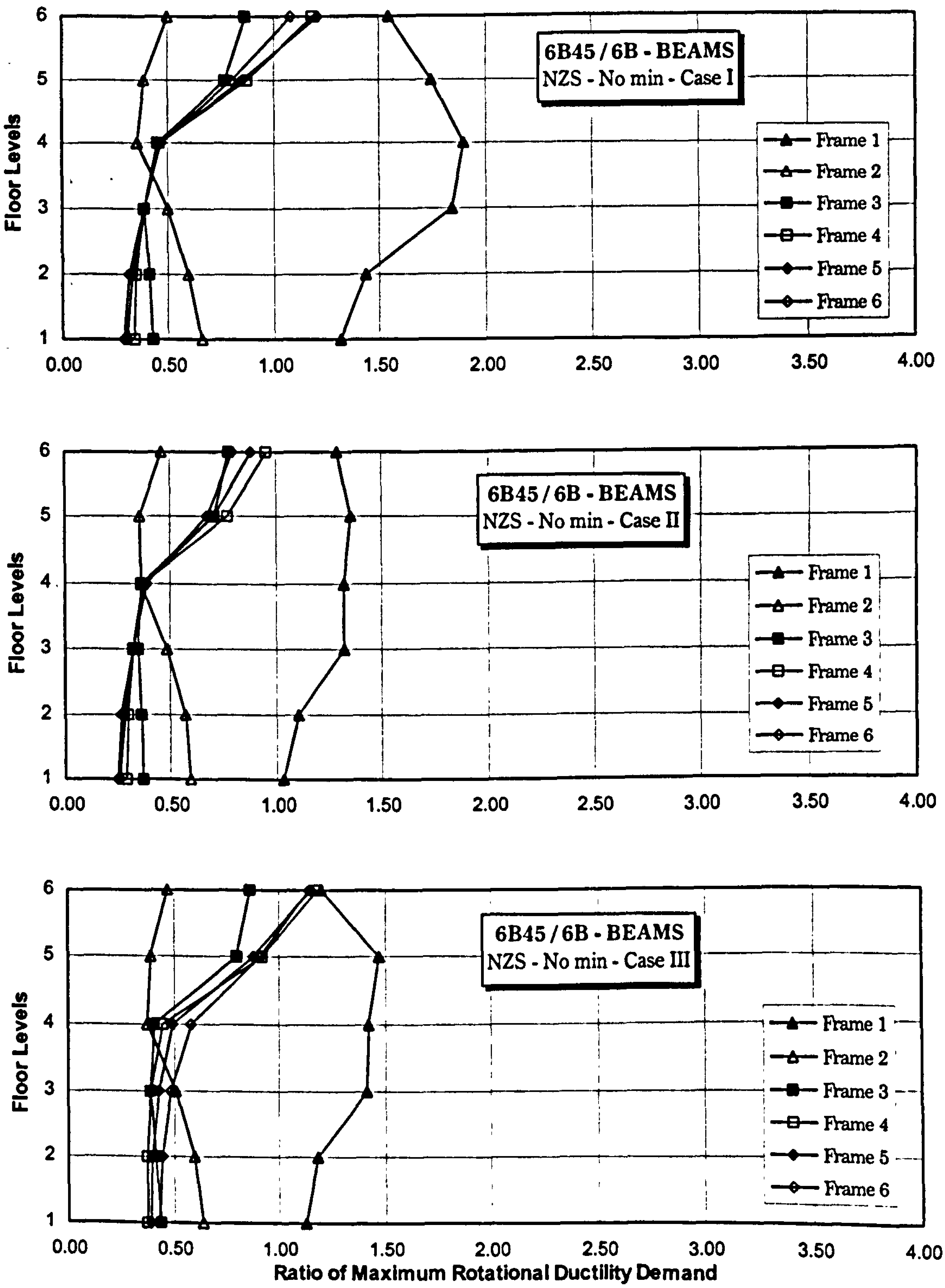


Figure 7.2.3 Ratios of rotational ductility demand vs. floor levels for the beams of model 6B45/6B designed to the NZS code and the 1st design method.

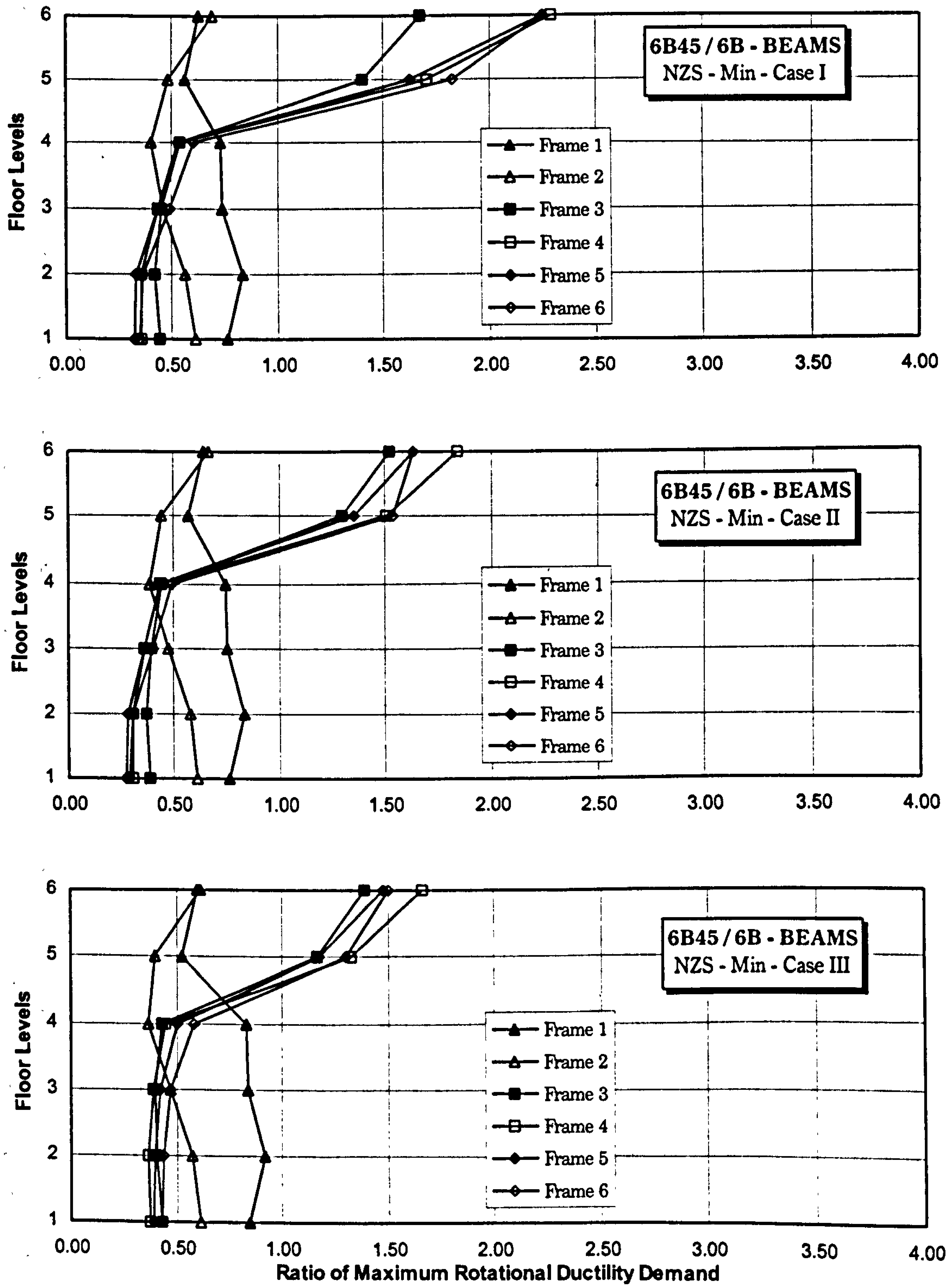


Figure 7.2.4 Ratios of rotational ductility demand vs. floor levels for the beams of model 6B45/6B designed to the NZS code and the 2nd design method.

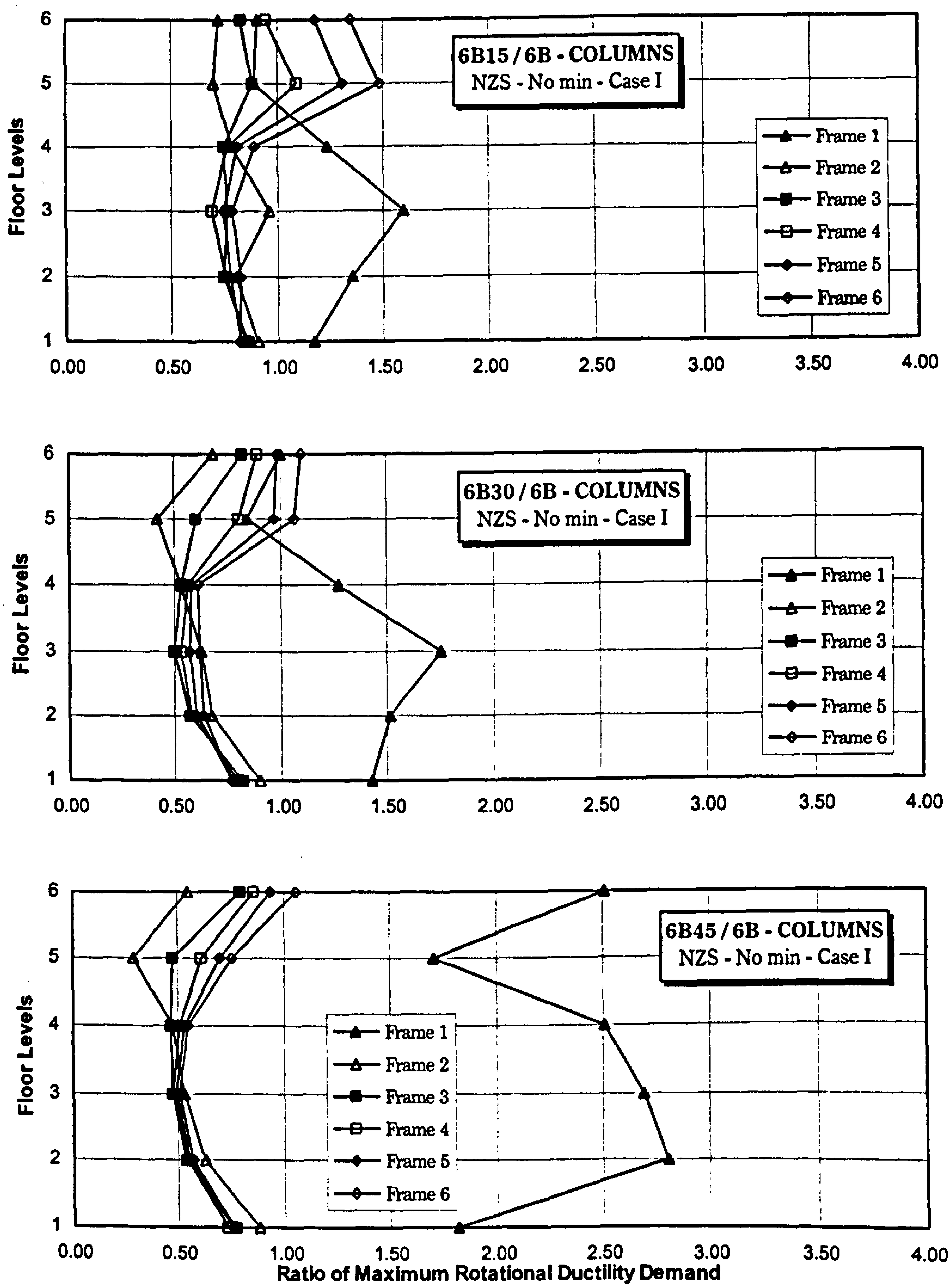


Figure 7.3.1 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different values of static eccentricity designed to the NZS code, the 1st design method and case I.

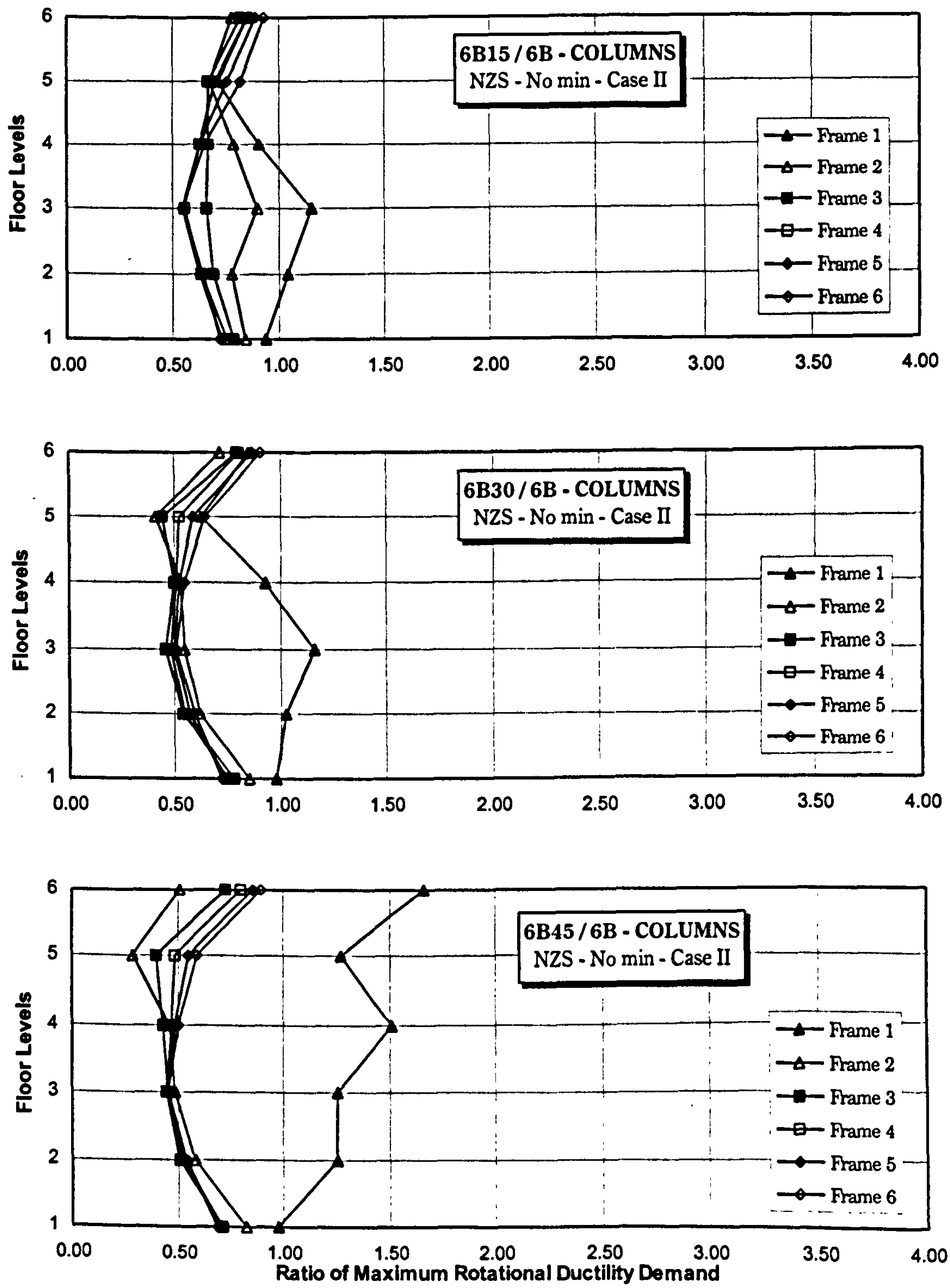


Figure 7.3.2 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different values of static eccentricity designed to the NZS code, the 1st design method and case II.

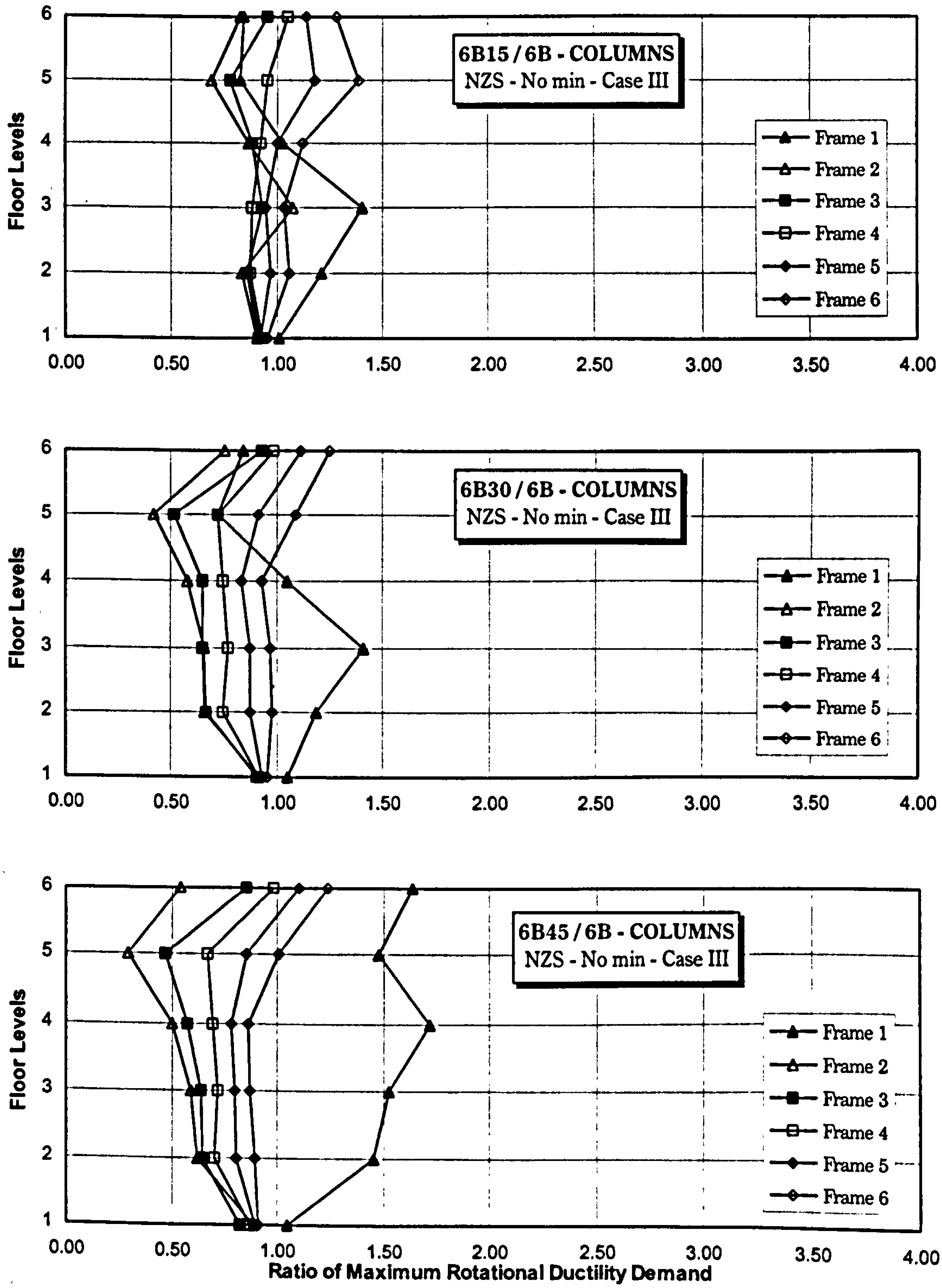


Figure 7.3.3 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different values of static eccentricity designed to the NZS code, the 1st design method and case III.

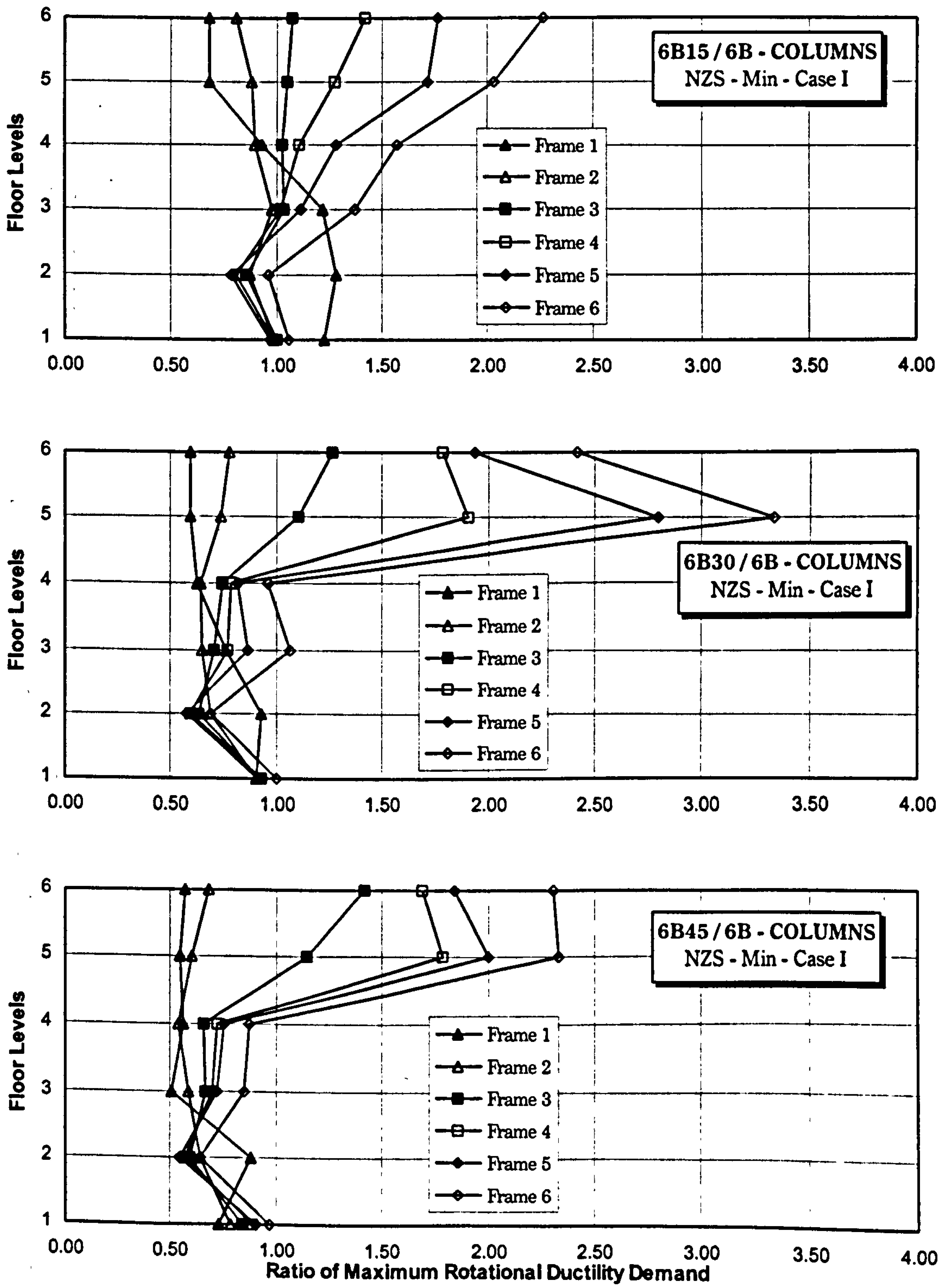


Figure 7.3.4 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different values of static eccentricity designed to the NZS code, the 2nd design method and case I.

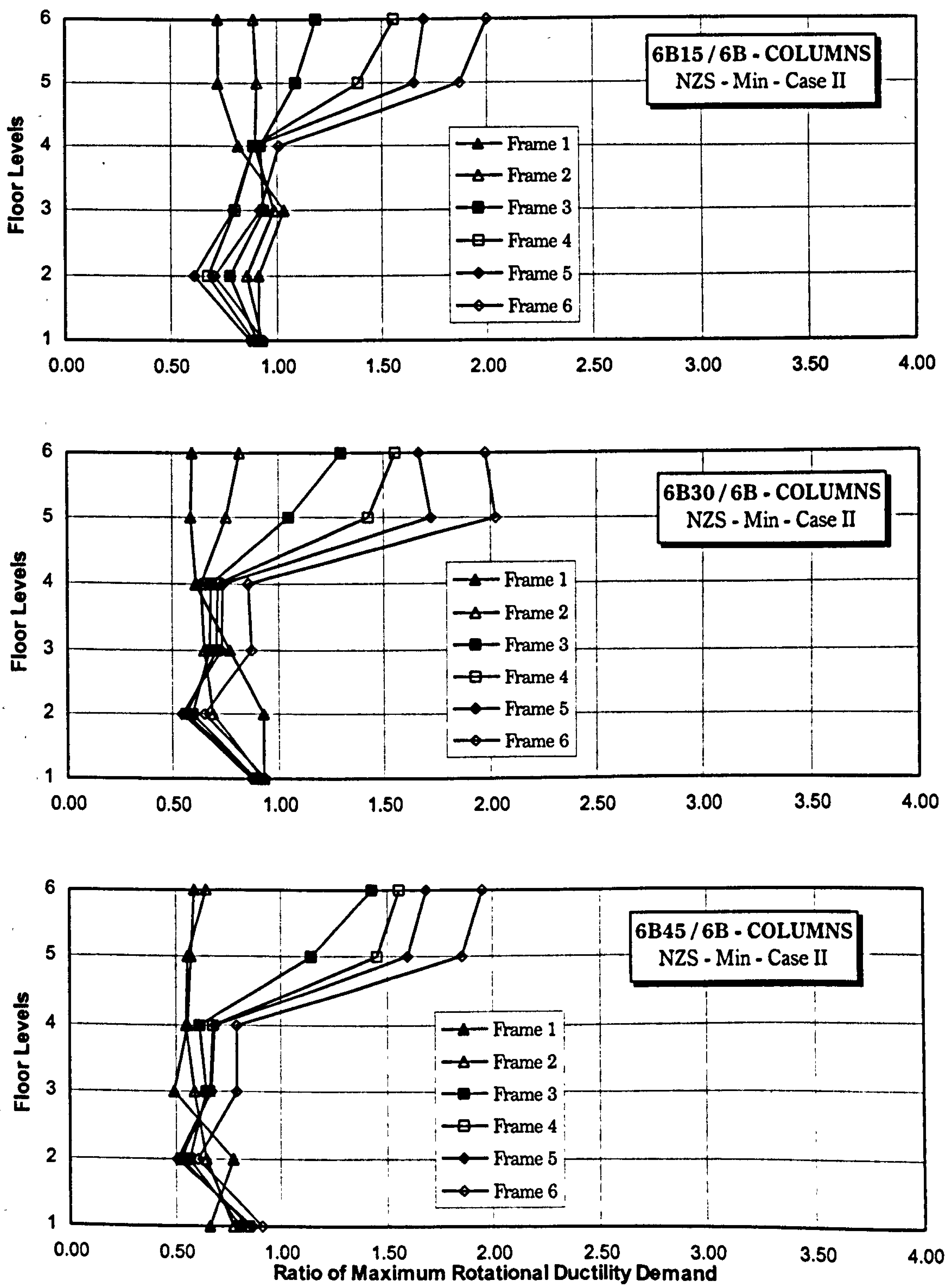


Figure 7.3.5 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different values of static eccentricity designed to the NZS code, the 2nd design method and case II.

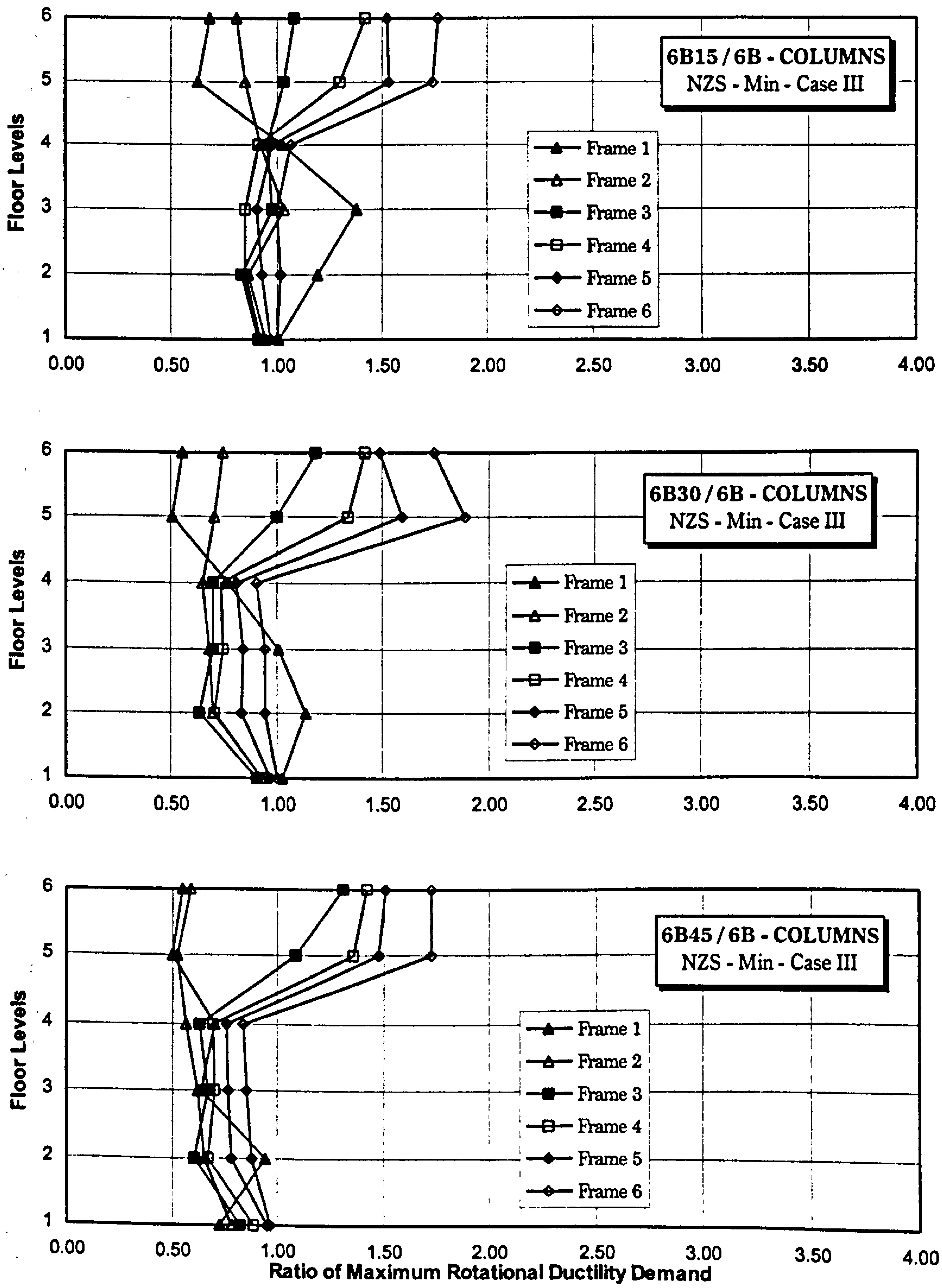


Figure 7.3.6 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different values of static eccentricity designed to the NZS code, the 2nd design method and case III.

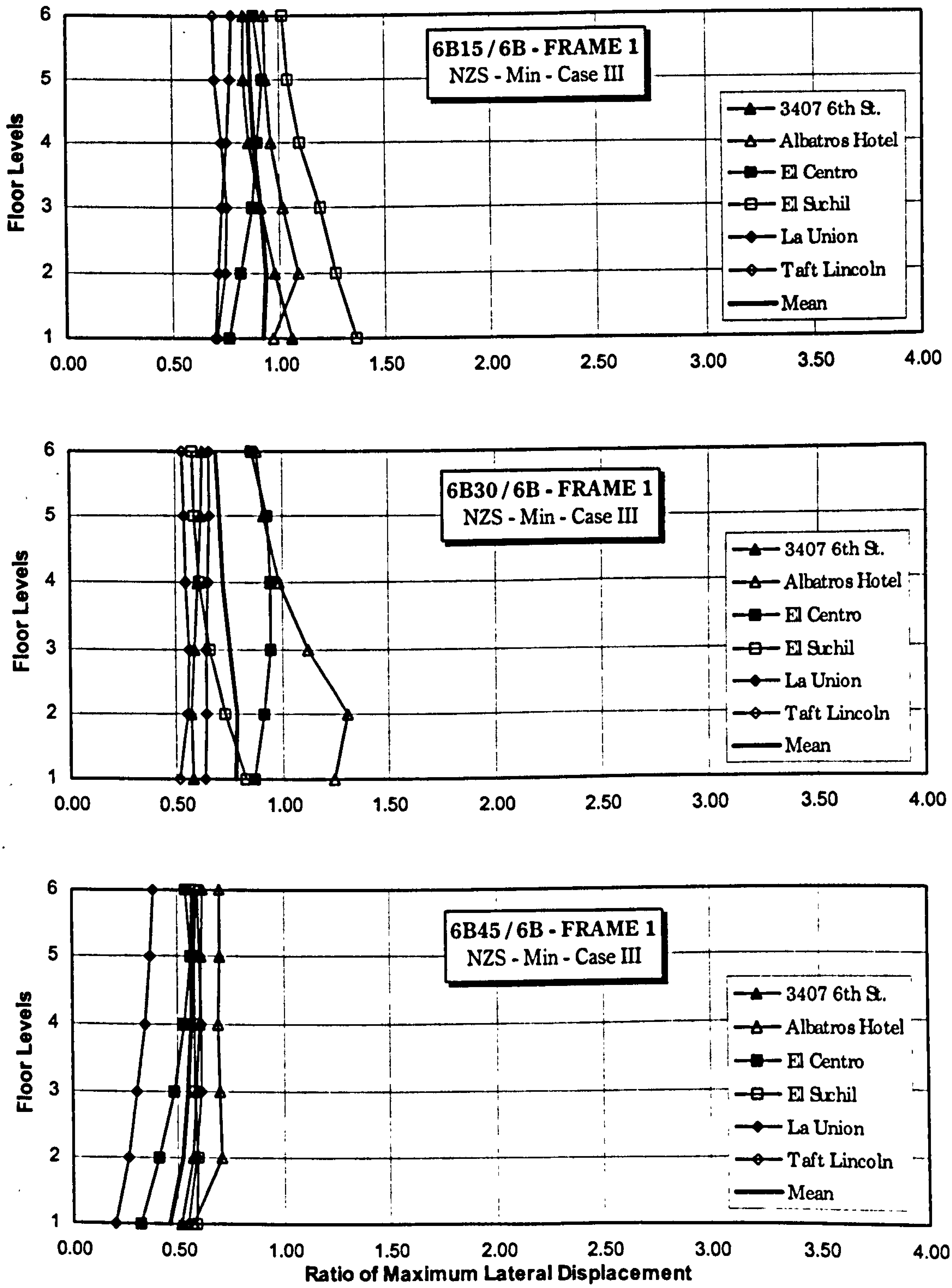


Figure 7.3.7 Ratios of maximum lateral displacement vs. floor levels for frame 1 of TU models with different values of static eccentricity designed to the NZS code, the 2nd design method and case III.

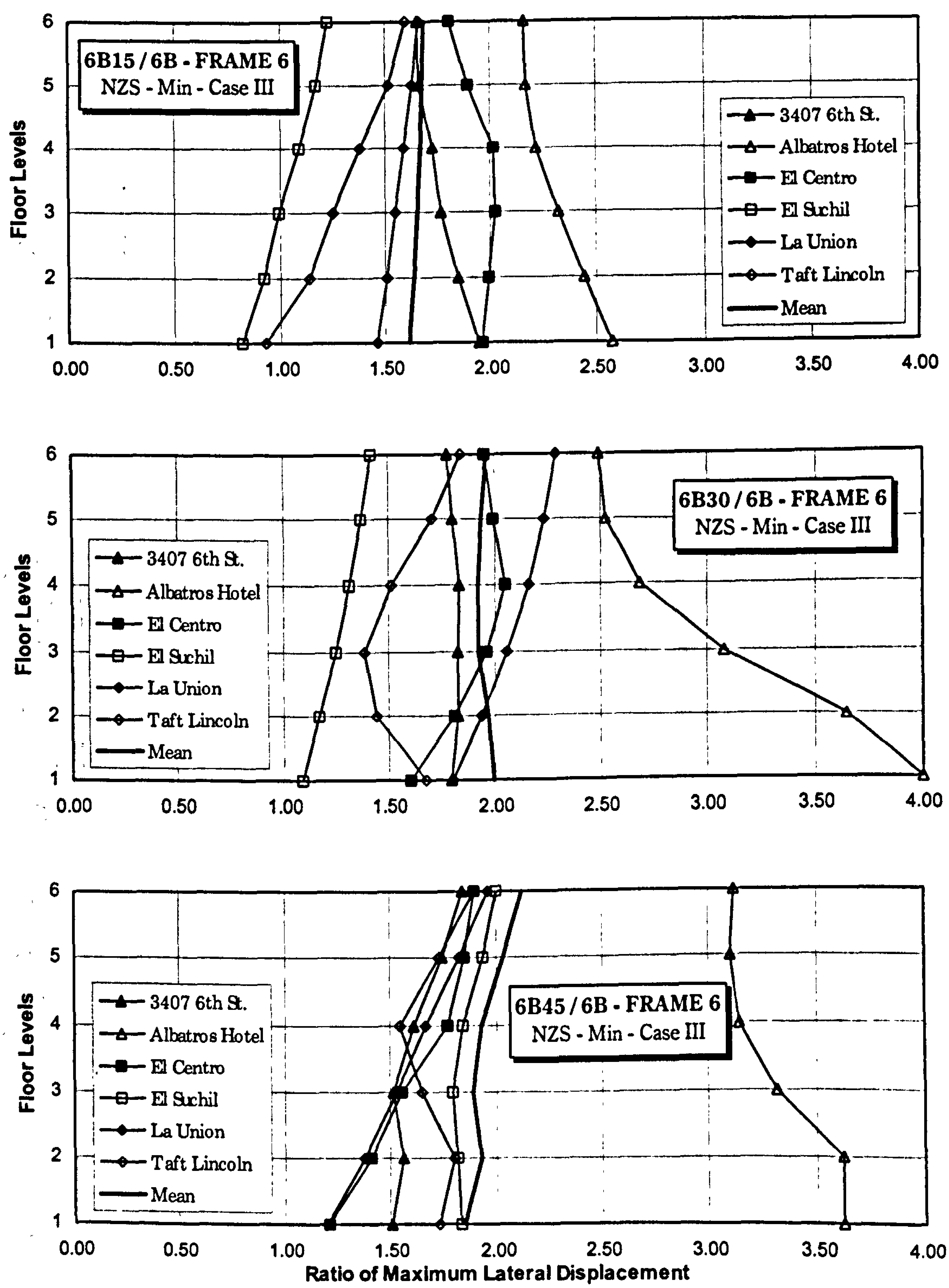


Figure 7.3.8 Ratios of maximum lateral displacement vs. floor levels for frame 6 of TU models with different values of static eccentricity designed to the NZS code, the 2nd design method and case III.

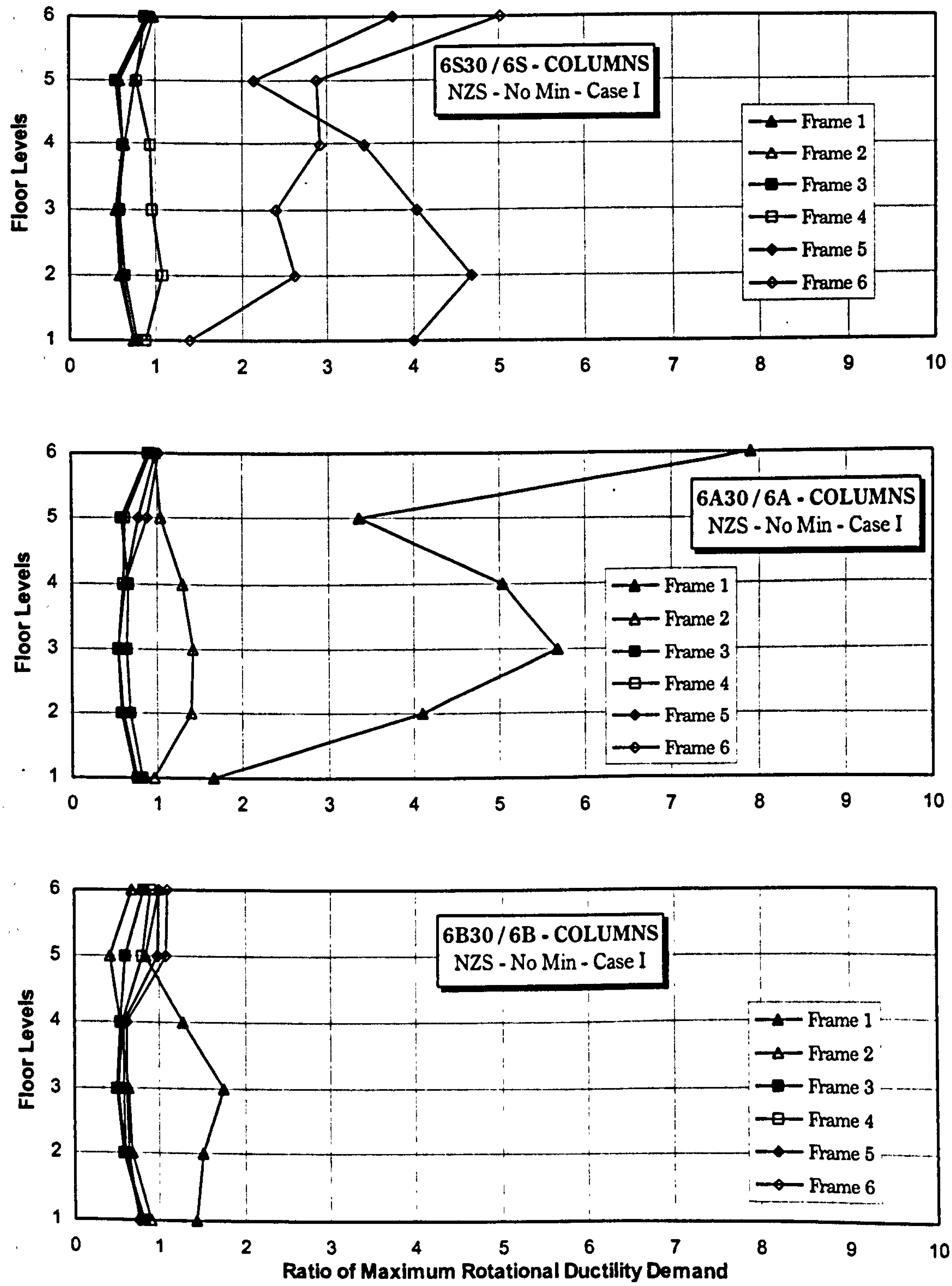


Figure 7.4.1 Ratios of rotational ductility demand vs. floor levels for the columns of different types of TU models designed according to the NZS code, the 1st design method and case I.

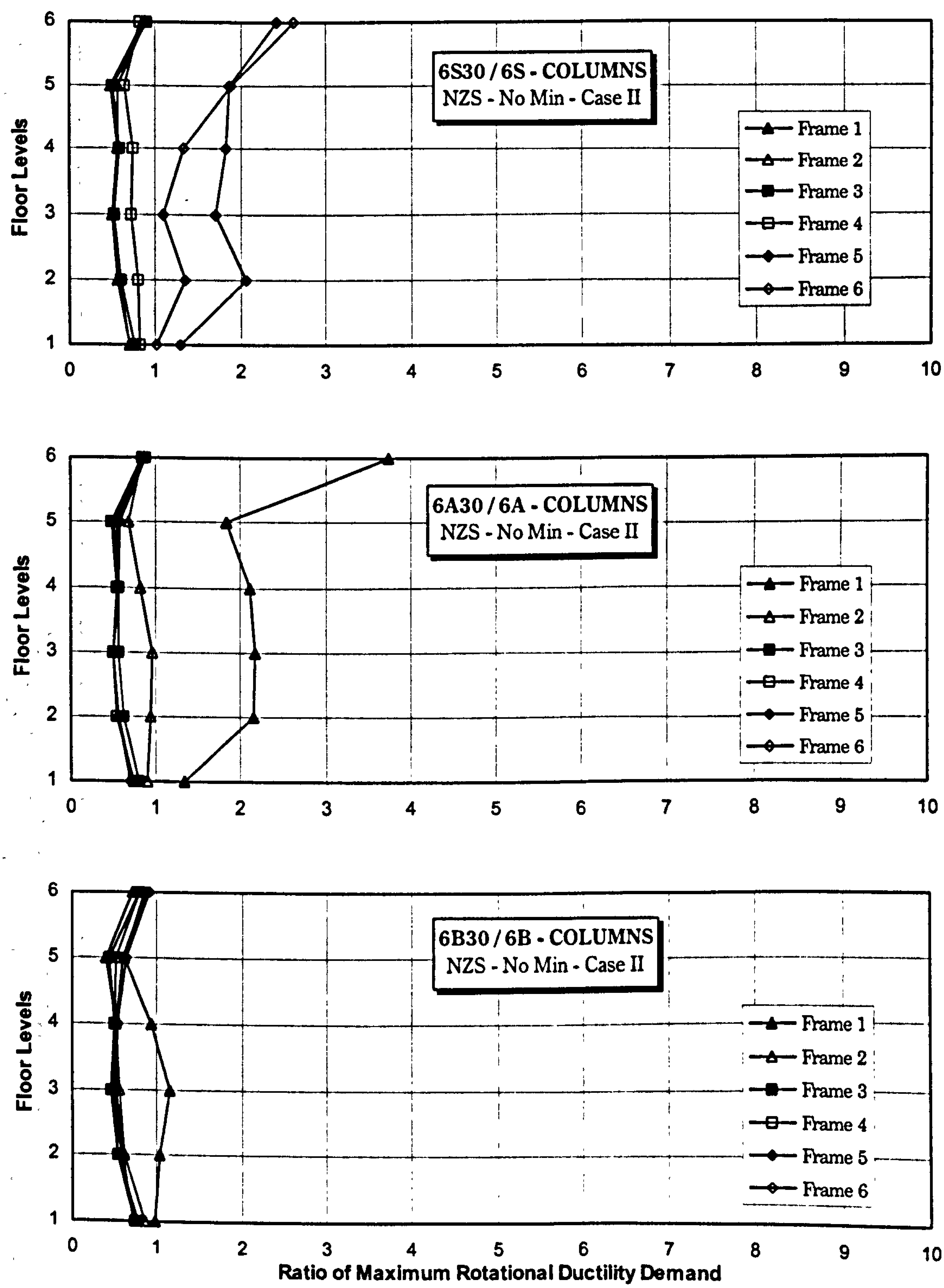


Figure 7.4.2 Ratios of rotational ductility demand vs. floor levels for the columns of different types of TU models designed to the NZS code, the 1st design method and case II.

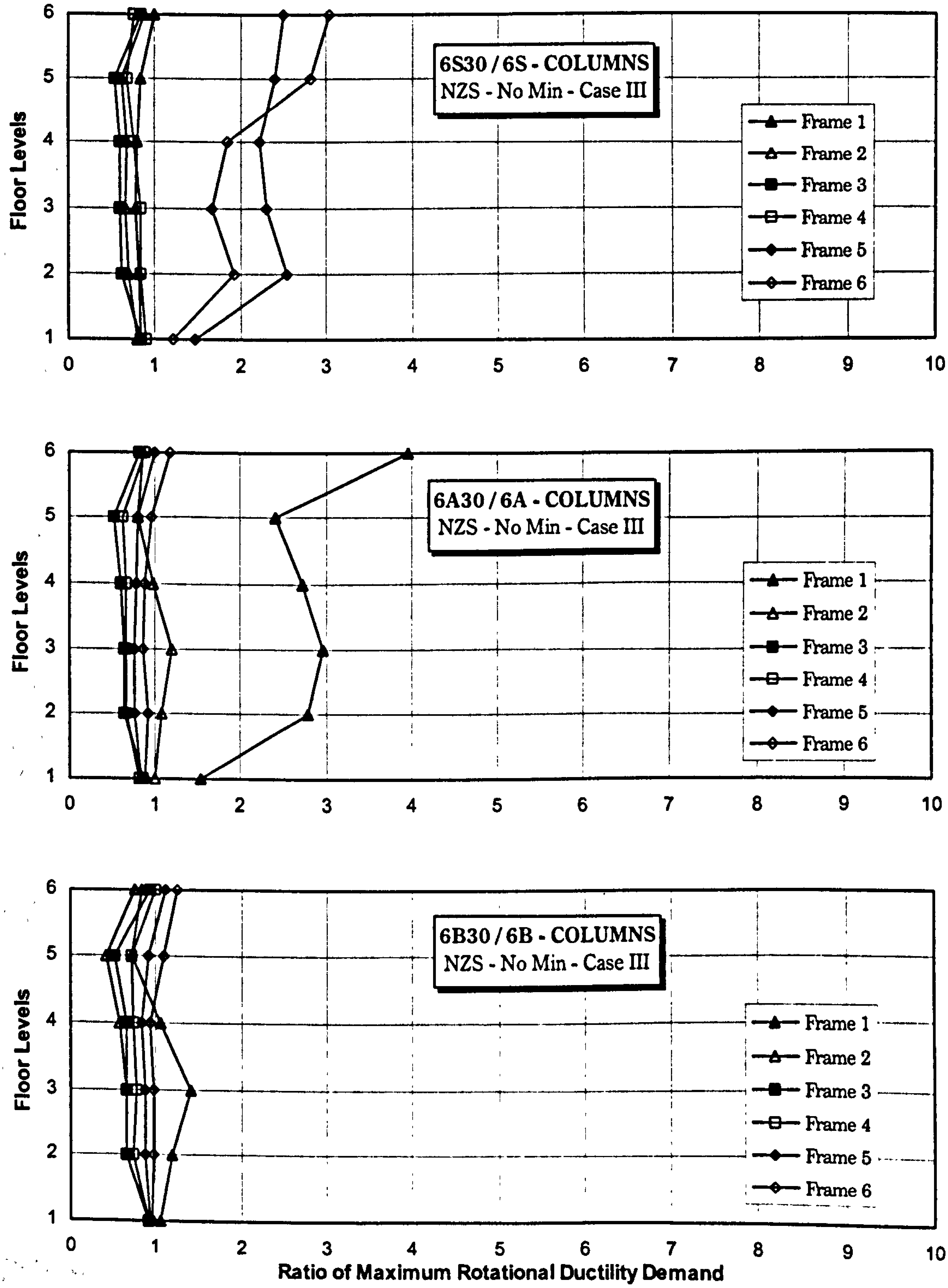


Figure 7.4.3 Ratios of rotational ductility demand vs. floor levels for the columns of different types of TU models designed to the NZS code, the 1st design method and case III.

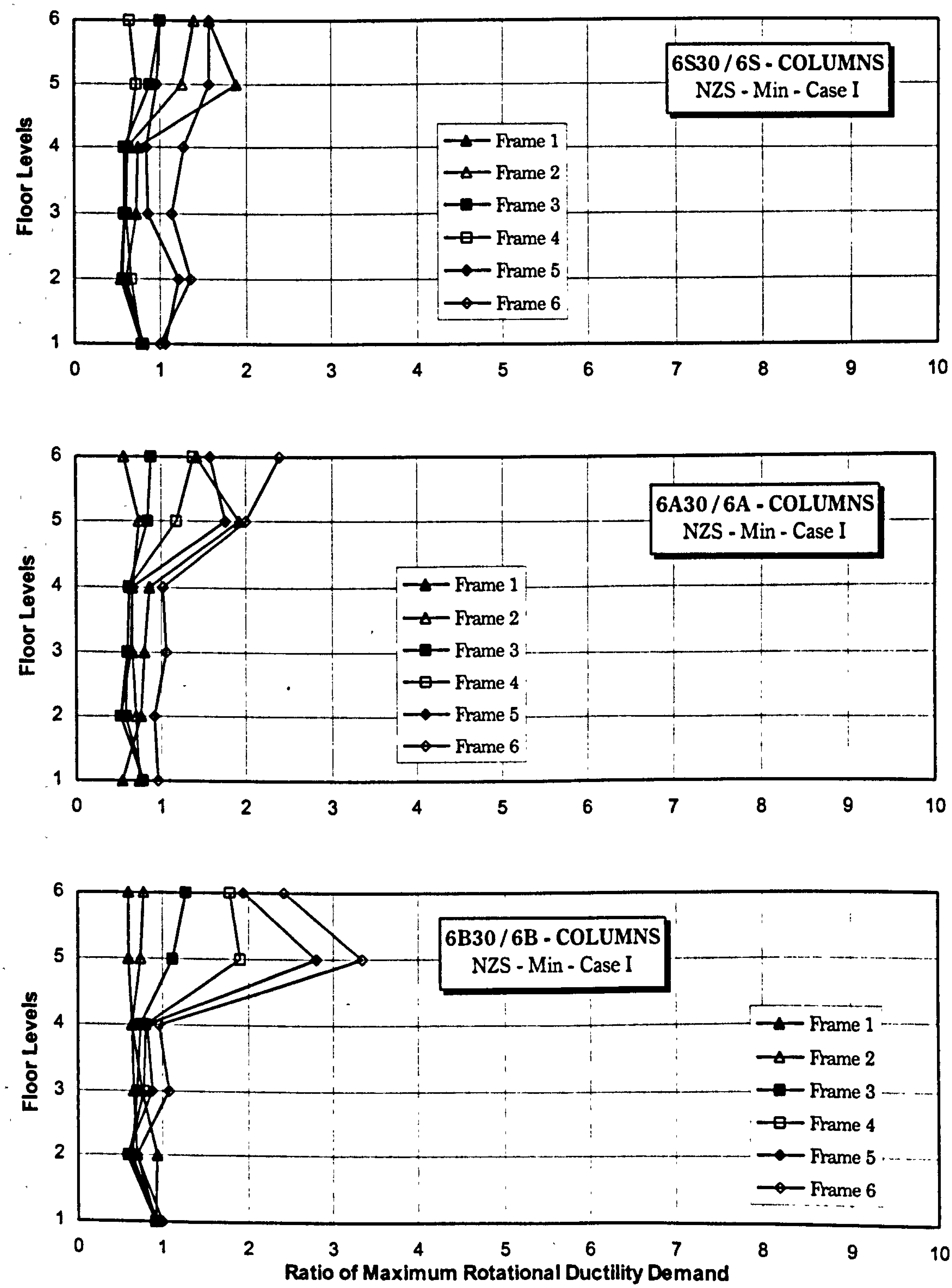


Figure 7.4.4 Ratios of rotational ductility demand vs. floor levels for the columns of different types of TU models designed to the NZS code, the 2nd design method and case I.

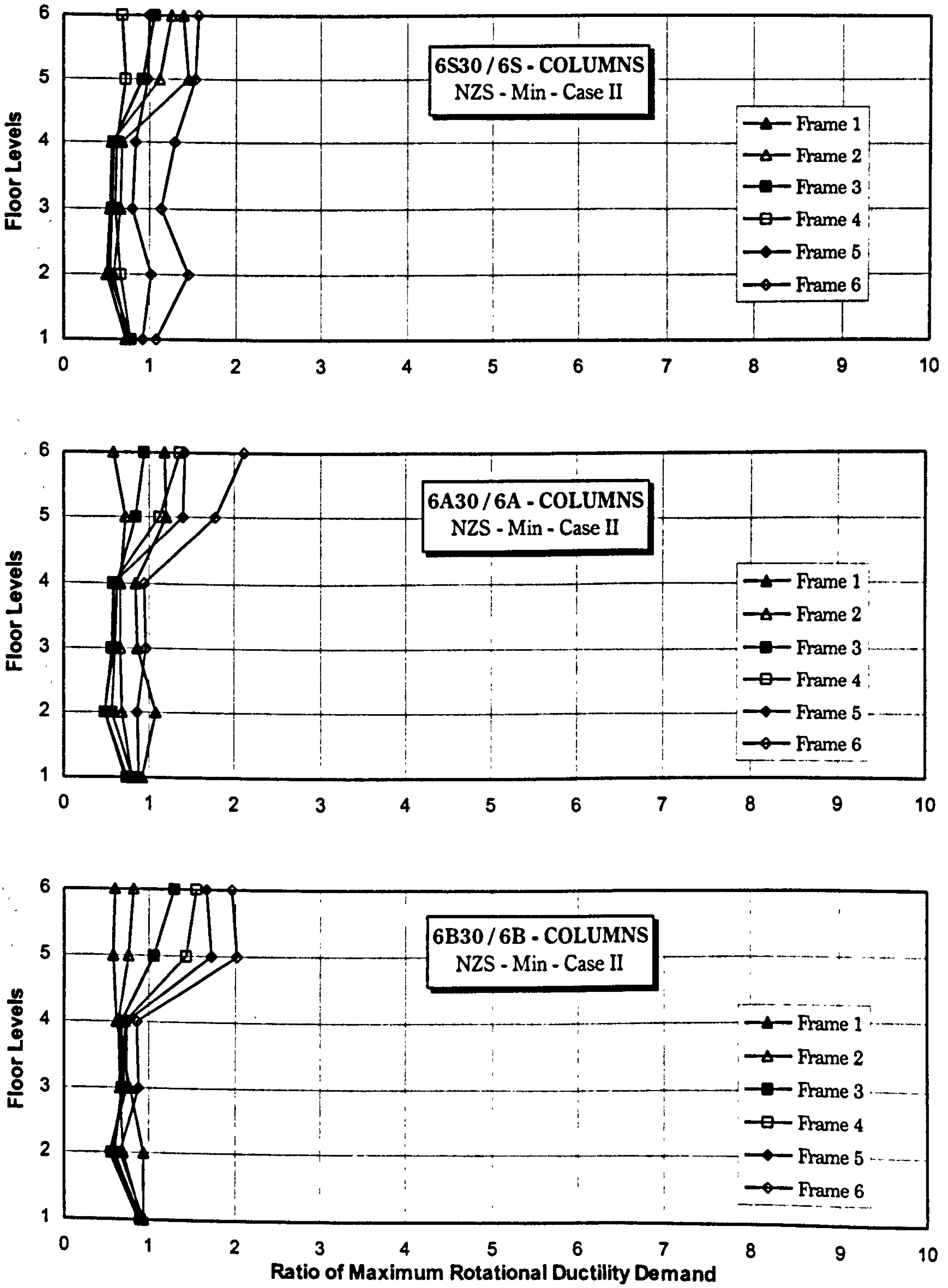


Figure 7.4.5 Ratios of rotational ductility demand vs. floor levels for the columns of different types of TU models designed to the NZS code, the 2nd design method and case II.

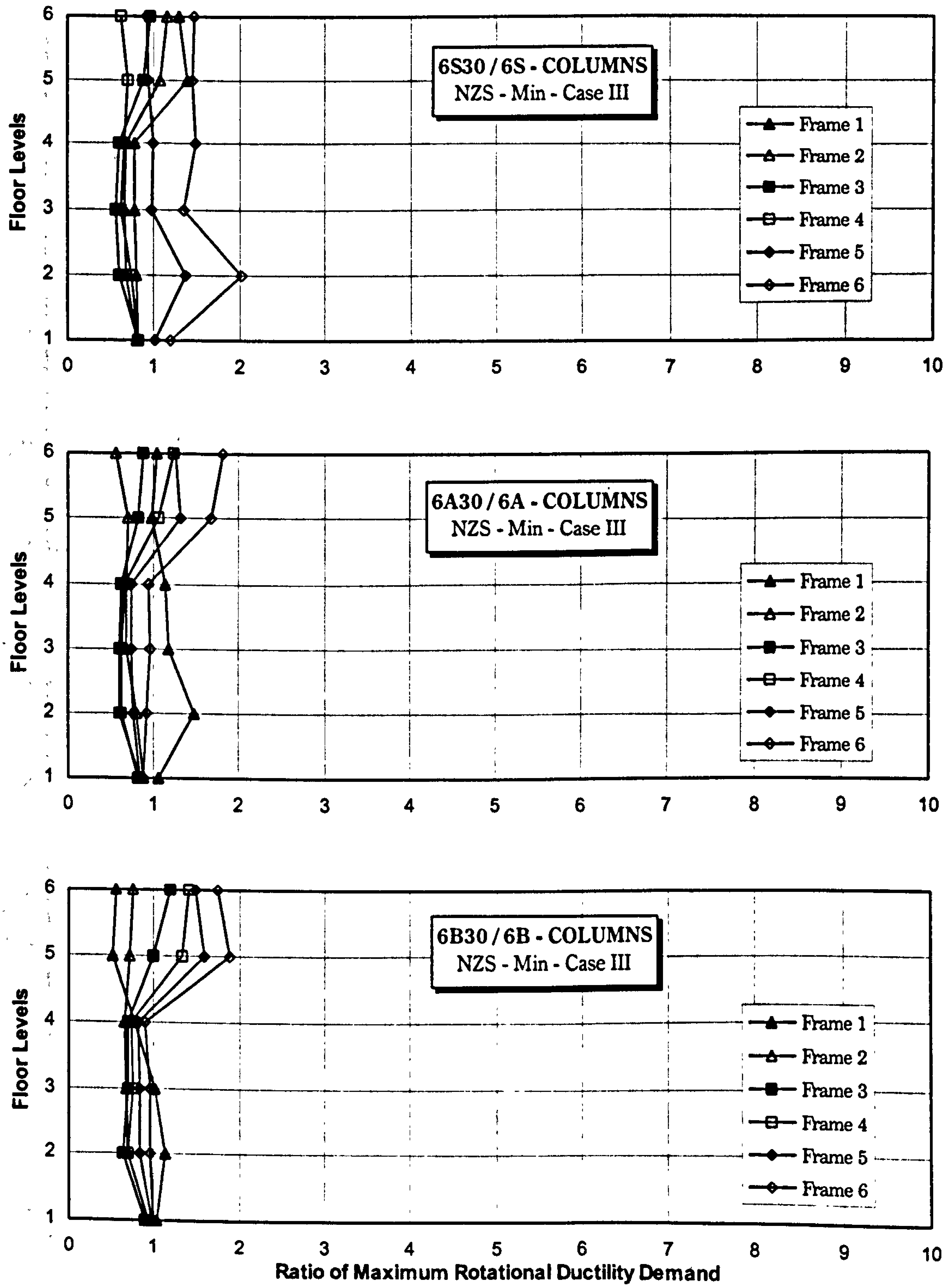
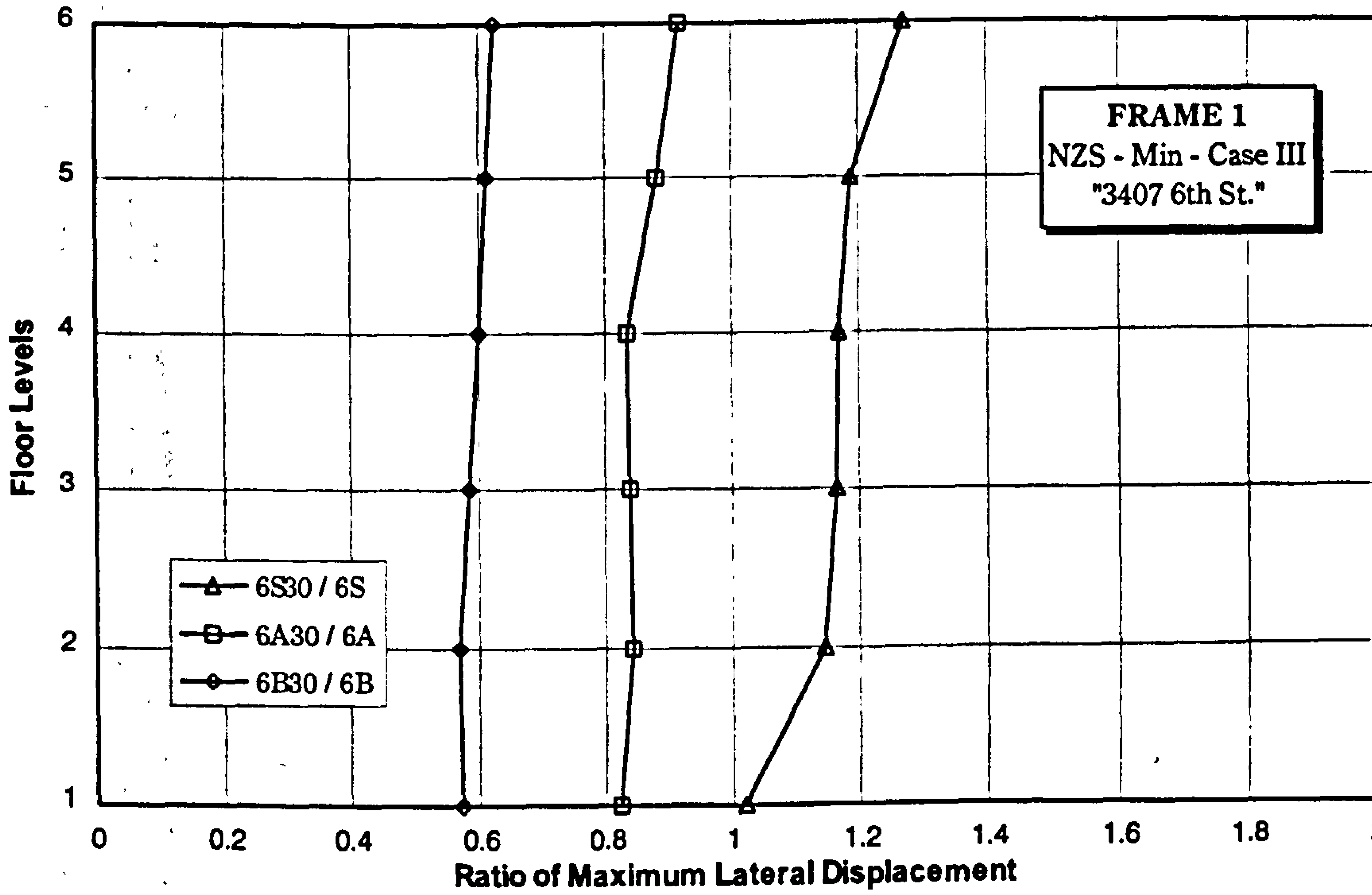


Figure 7.4.6 Ratios of rotational ductility demand vs. floor levels for the columns of different types of TU models designed to the NZS code, the 2nd design method and case III.



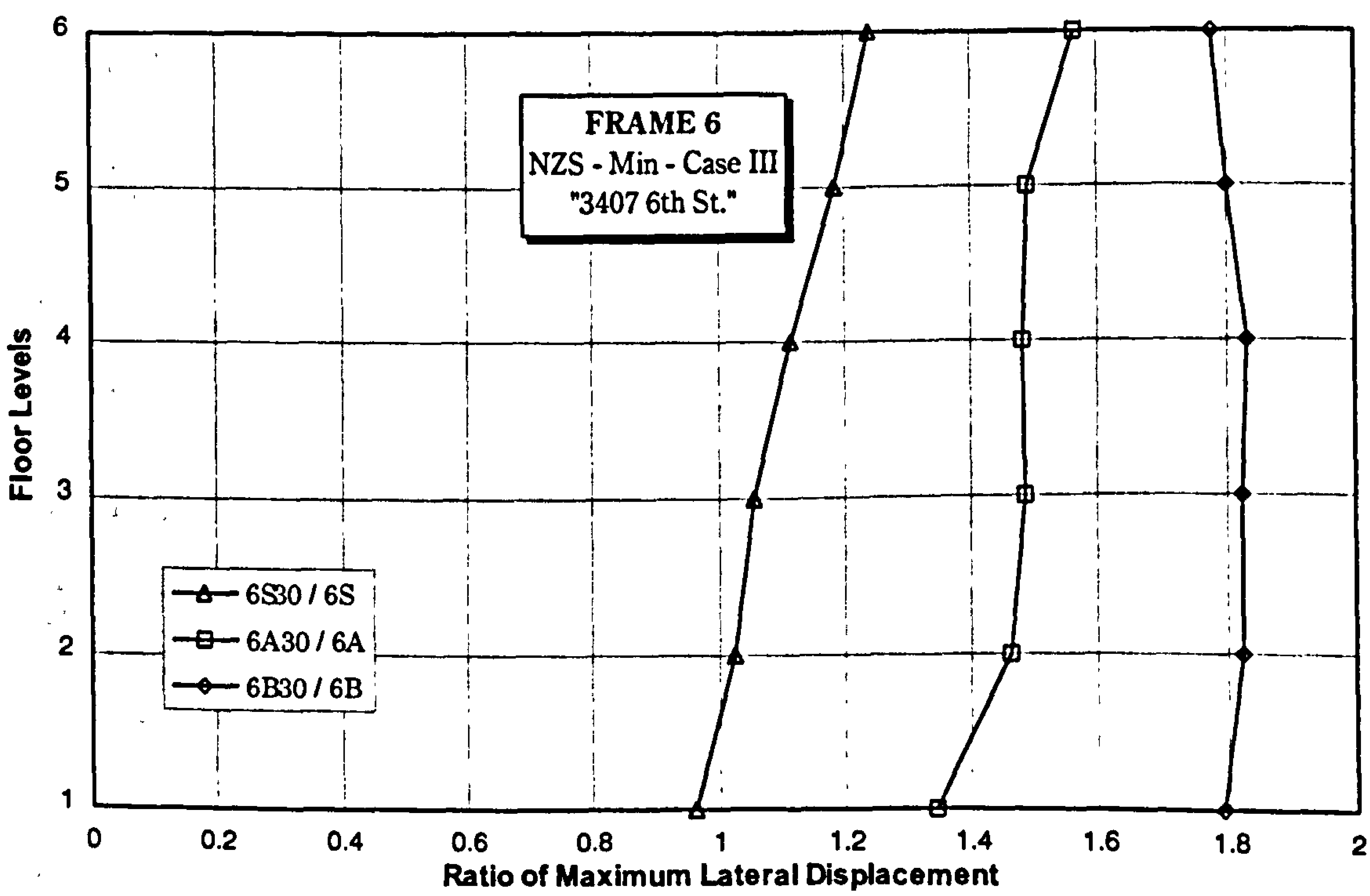


Figure 7.4.7 Ratios of maximum lateral displacement vs. floor levels for frames 1 and 6 of different types of TU models designed to the NZS code, the 2nd design method and case III.

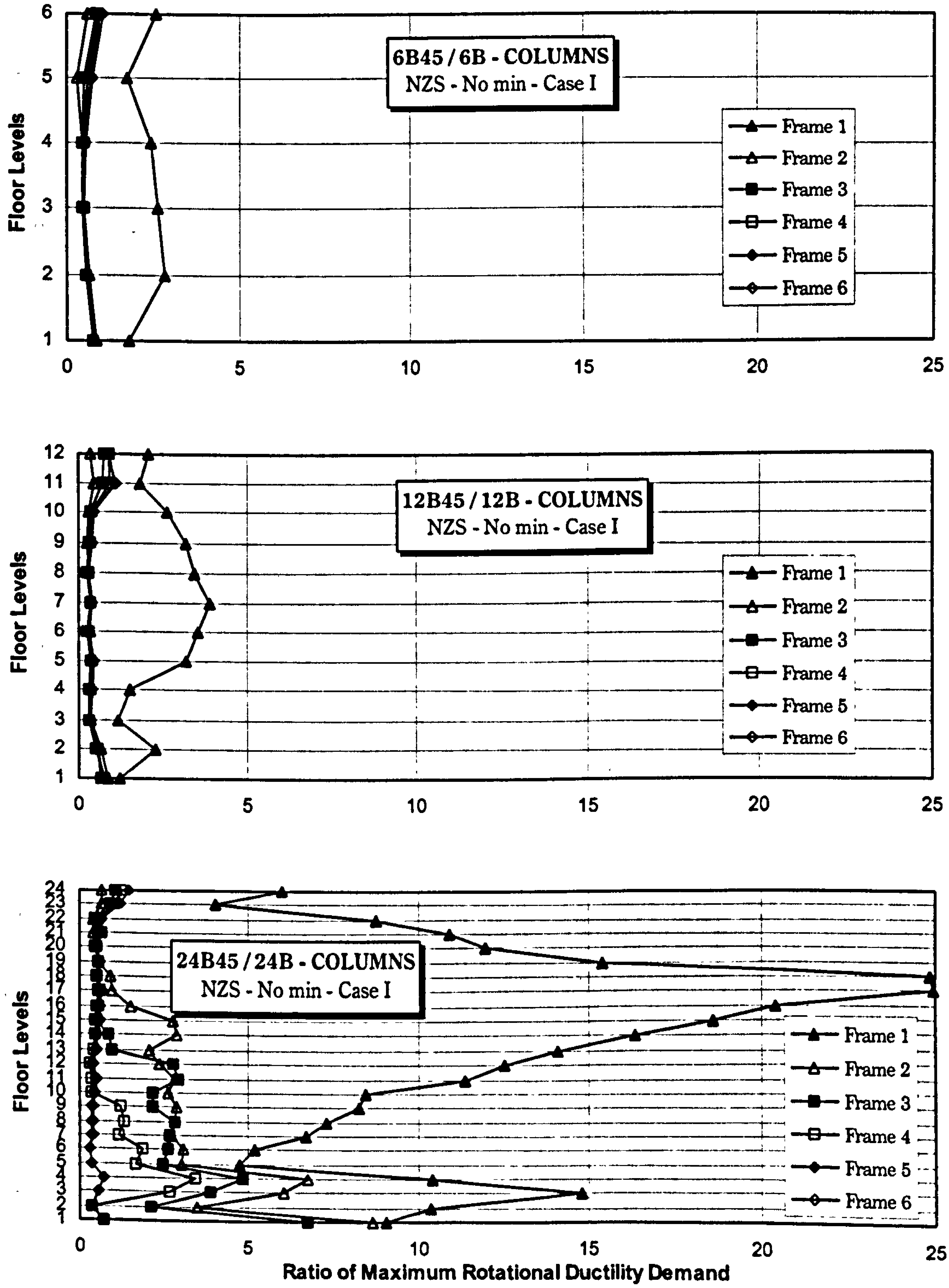


Figure 7.5.1 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different numbers of floor levels designed to the NZS code, the 1st design method and case I.

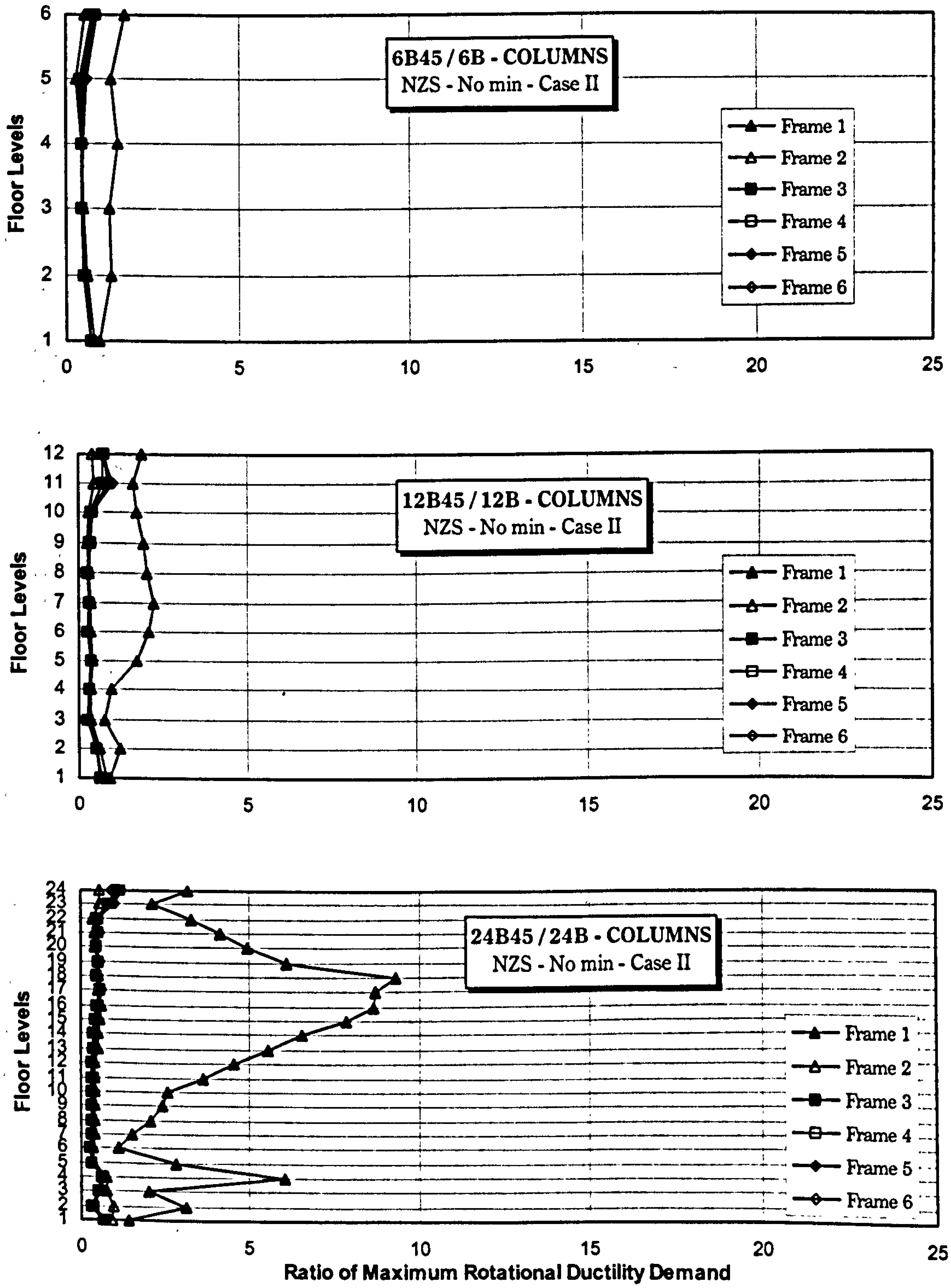


Figure 7.5.2 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different numbers of floor levels designed to the NZS code, the 1st design method and case II.

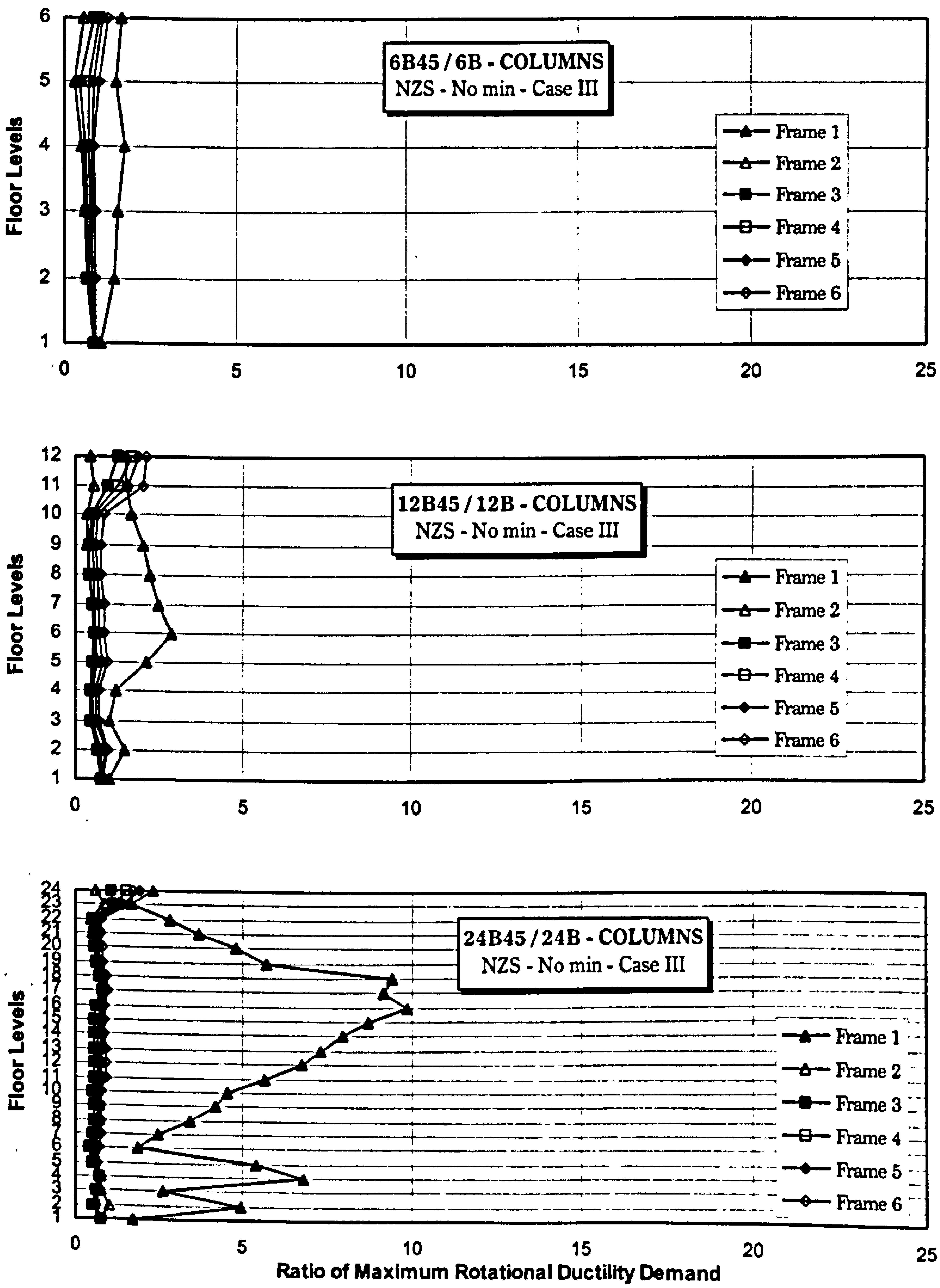


Figure 7.5.3 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different numbers of floor levels designed to the NZS code, the 1st design method and case III.

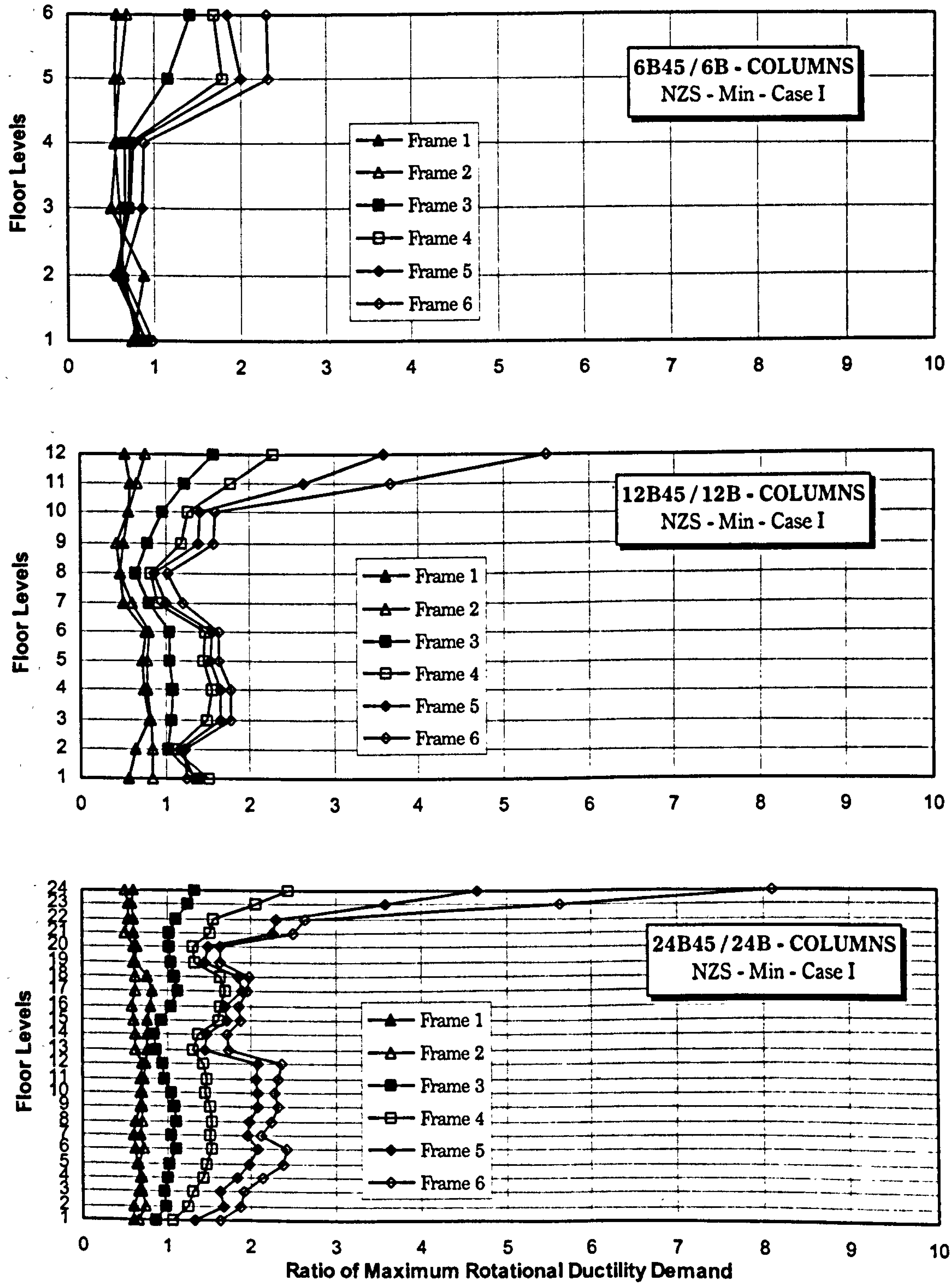


Figure 7.5.4 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different numbers of floor levels designed to the NZS code, the 2nd design method and case I.

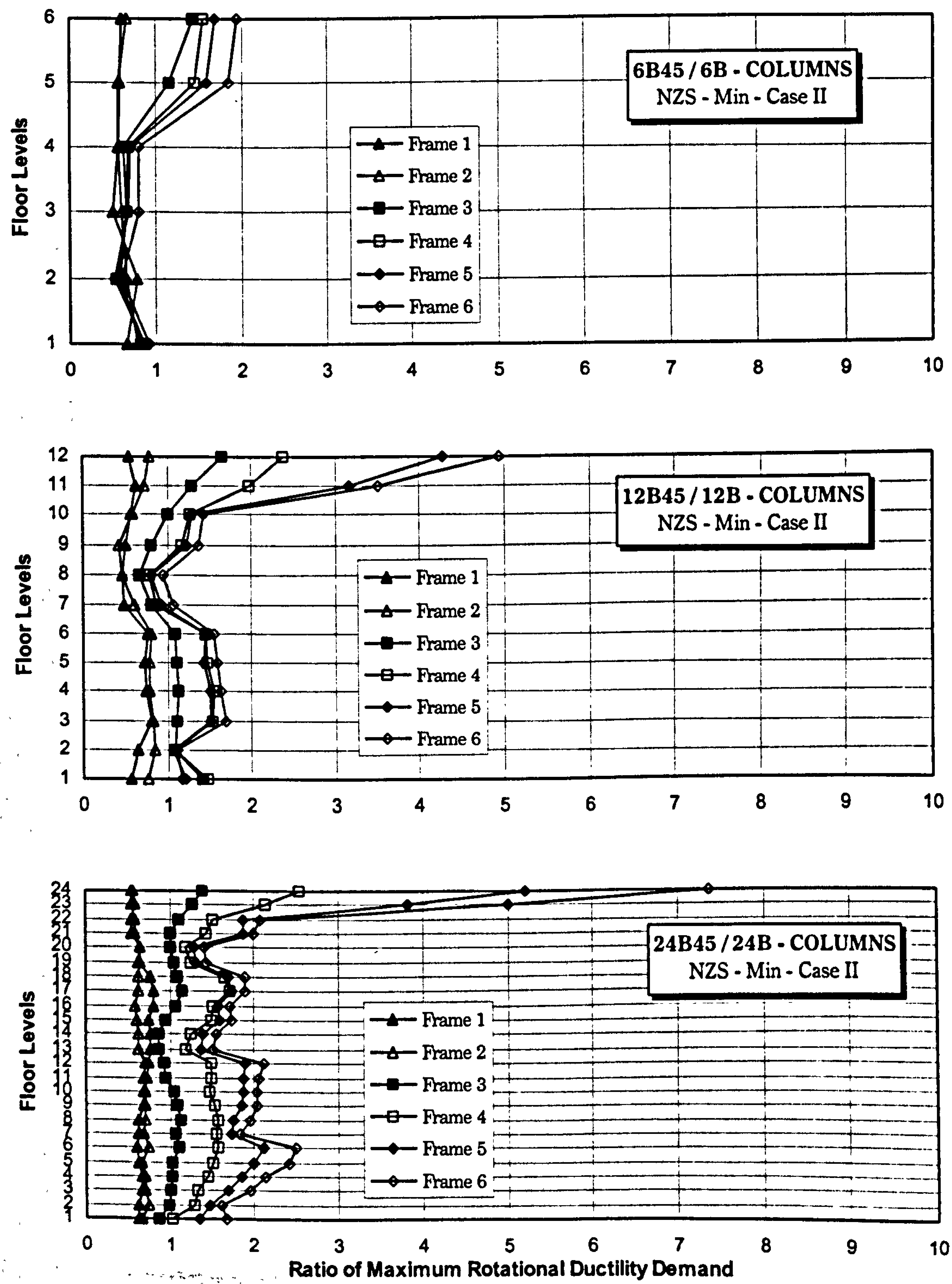


Figure 7.5.5 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different numbers of floor levels designed to the NZS code, the 2nd design method and case II.

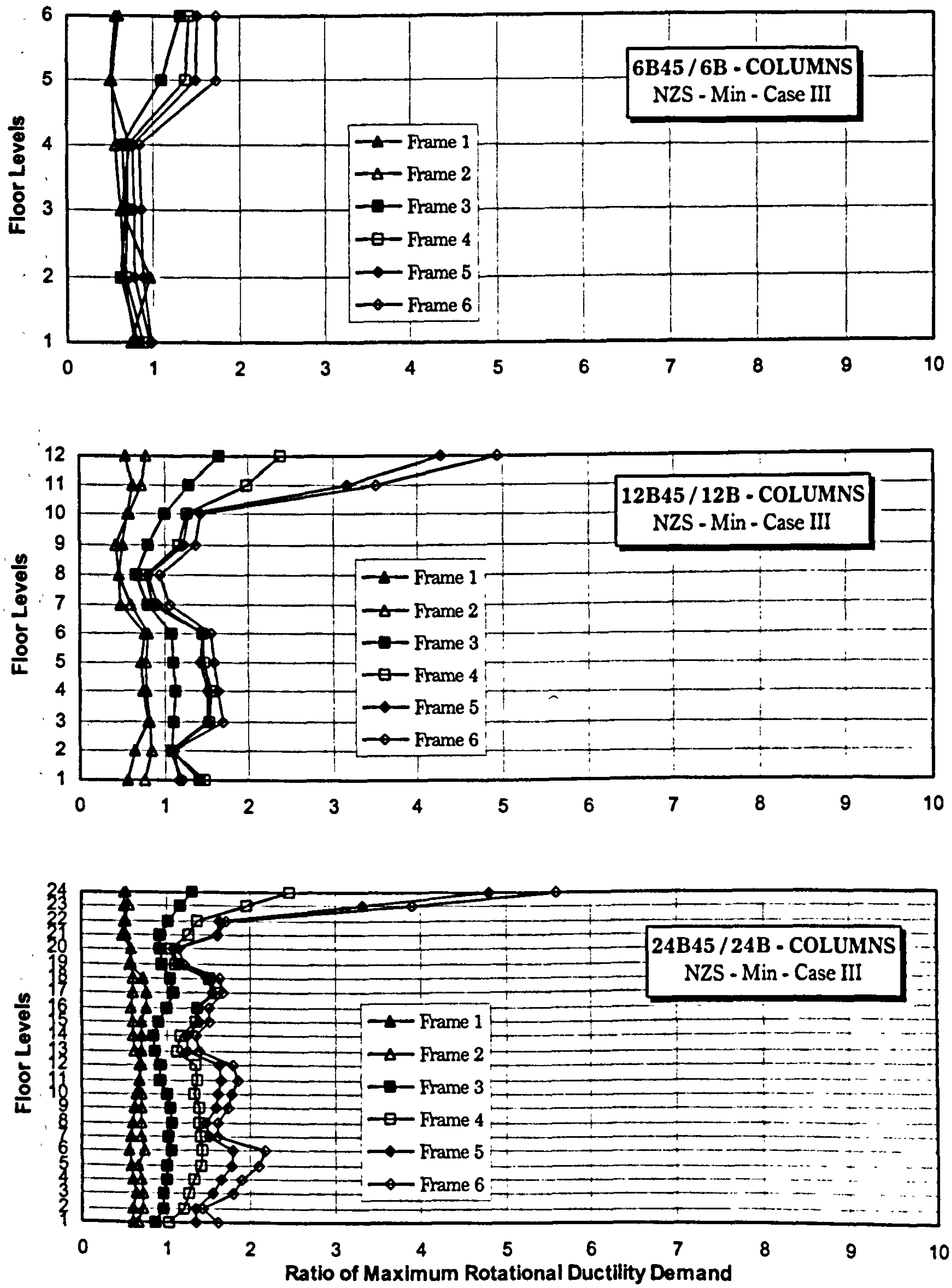


Figure 7.5.6 Ratios of rotational ductility demand vs. floor levels for the columns of TU models with different numbers of floor levels designed to the NZS code, the 2nd design method and case III.

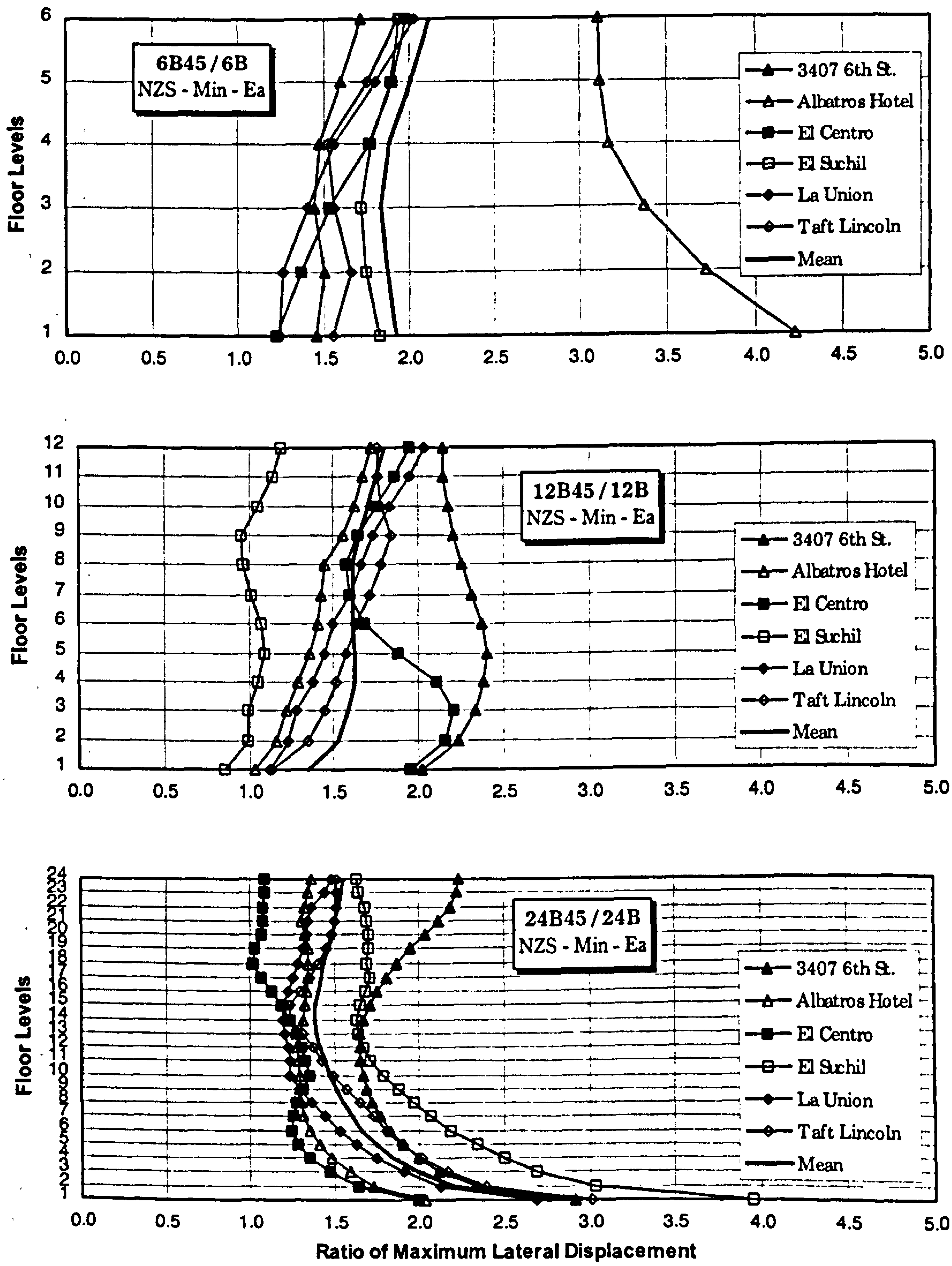


Figure 7.5.7 Ratios of maximum lateral displacement vs. floor levels for frame 6 of TU models with different numbers of floor levels designed to the NZS code, the 2nd design method and case III.

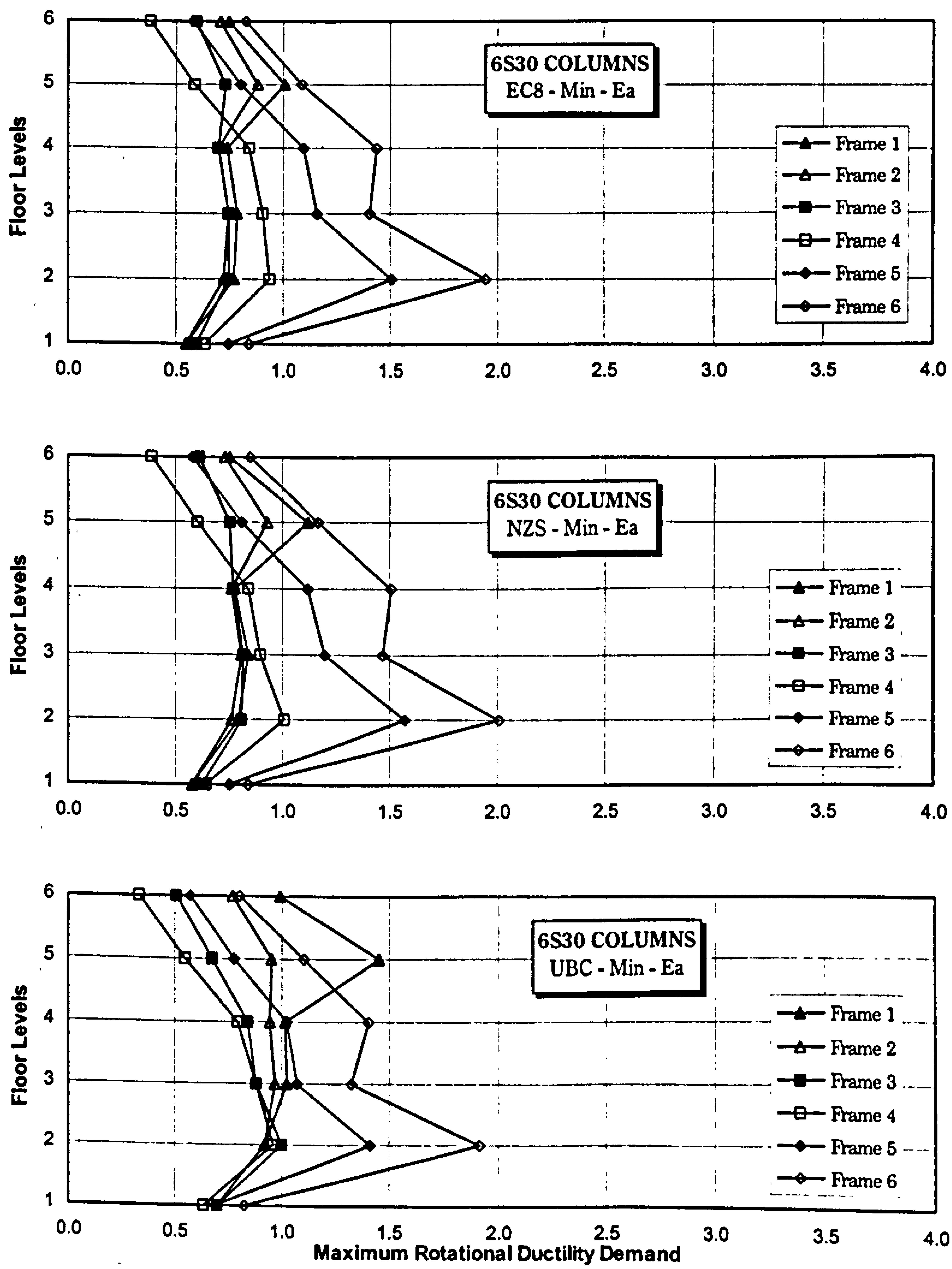


Figure 7.6.1 Maximum rotational ductility demand vs. floor levels for the columns of model 6S30 designed with different torsional provisions, the 2nd method and with the accidental eccentricity.

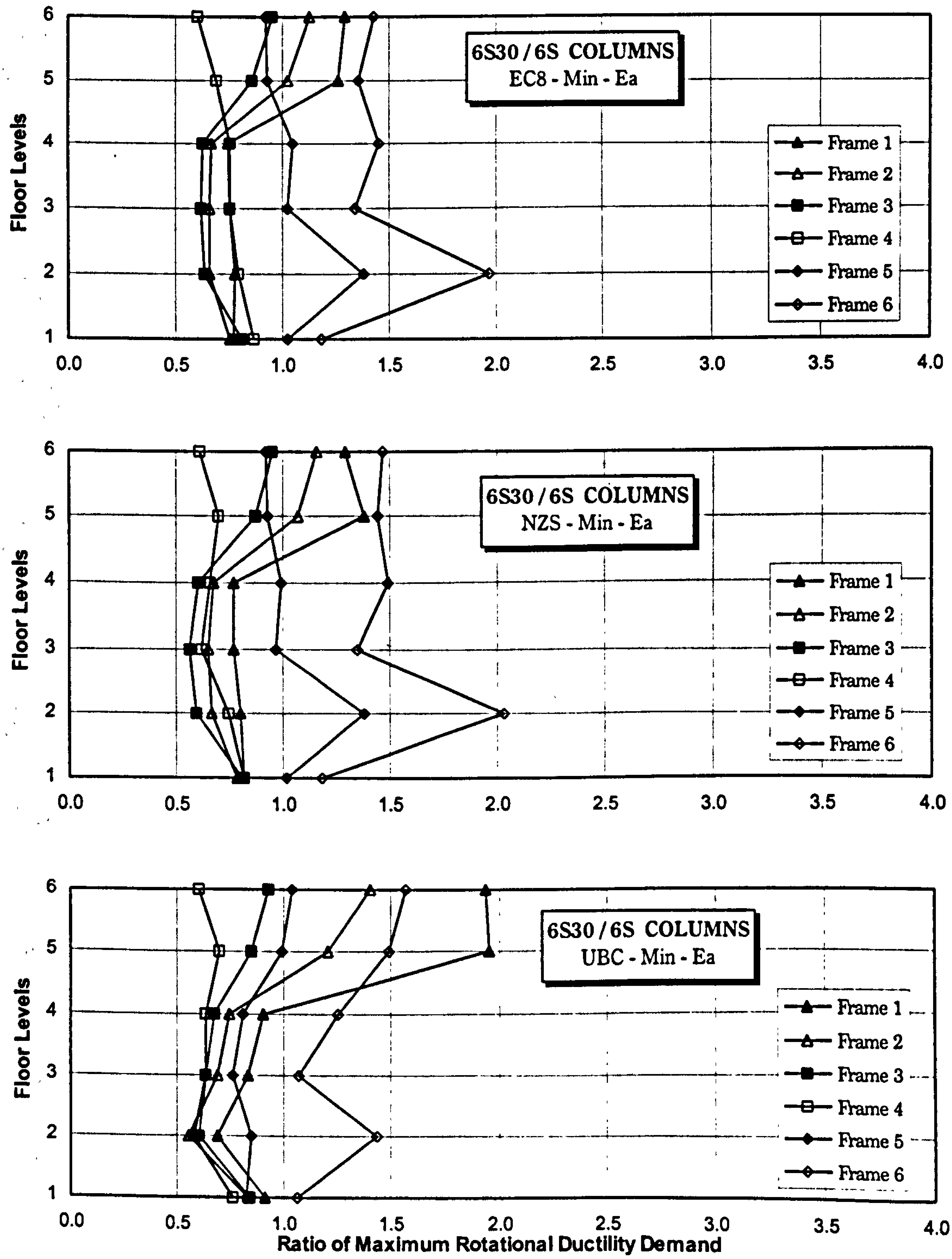


Figure 7.6.2 Ratios of rotational ductility demand vs. floor levels for the columns of model 6S30/6S designed to different torsional provisions, the 2nd method and with the accidental eccentricity.

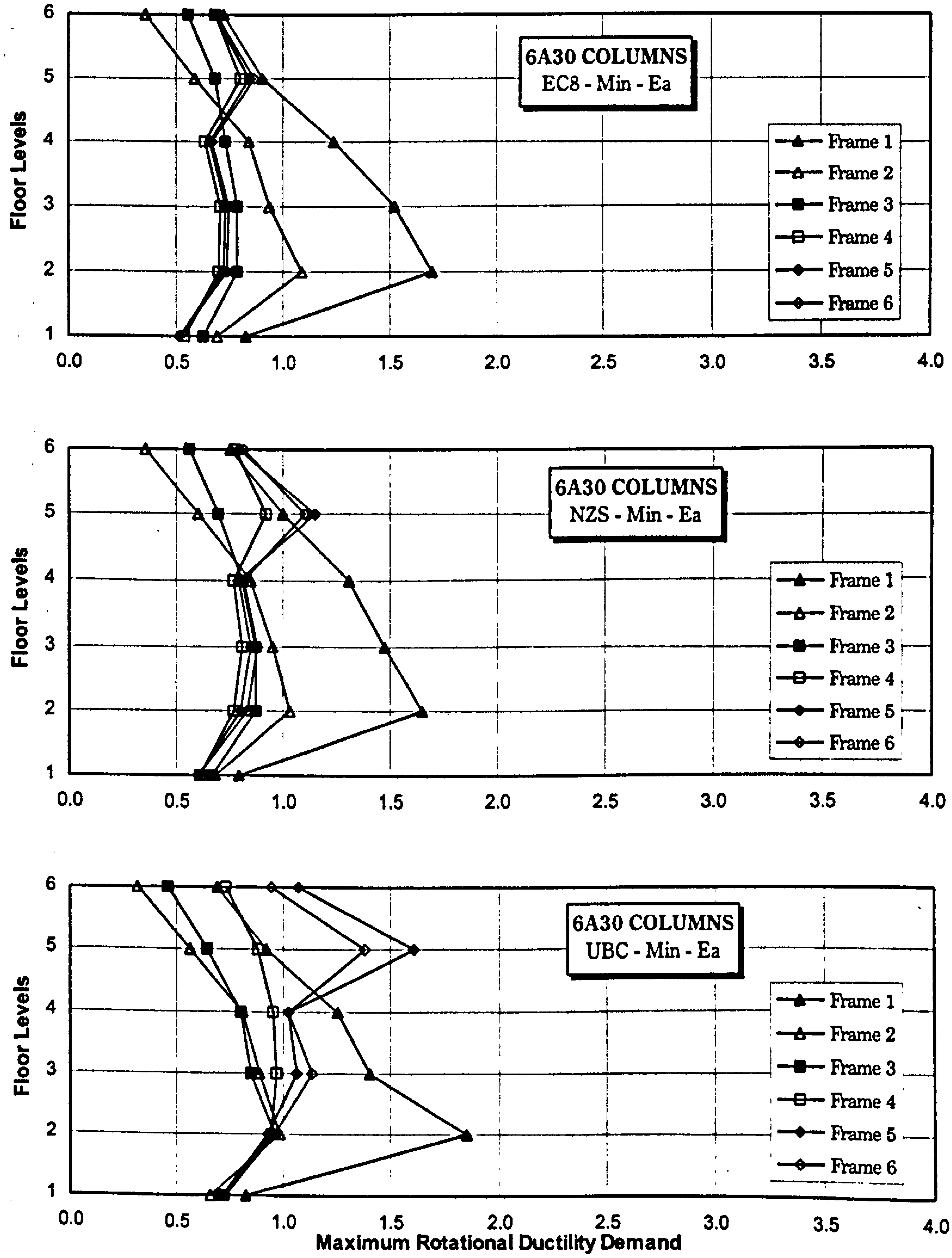


Figure 7.6.3 Maximum rotational ductility demand vs. floor levels for the columns of model 6A30 designed with different torsional provisions, the 2[nd] method and with the accidental eccentricity.

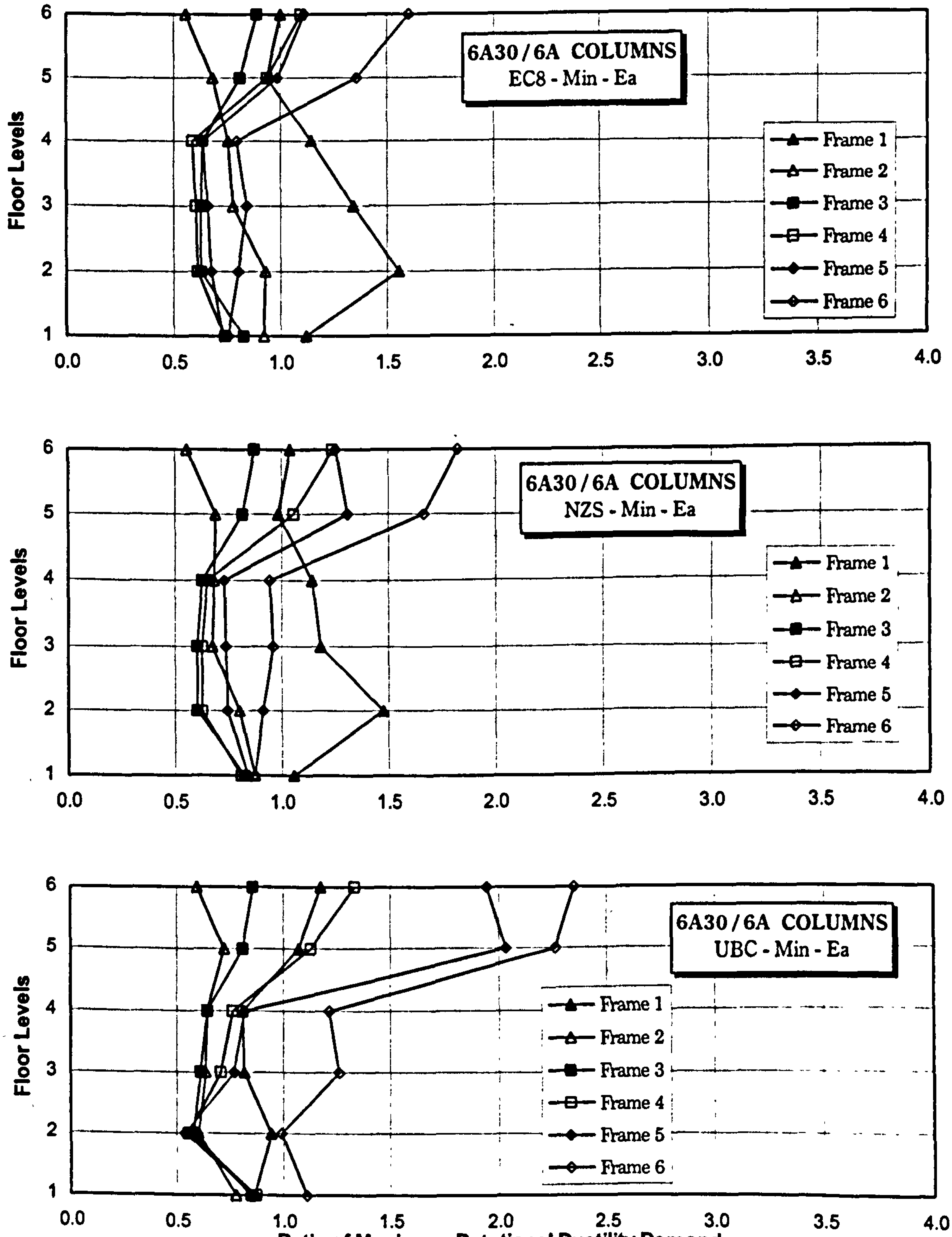


Figure 7.6.4 Ratios of rotational ductility demand vs. floor levels for the columns of model 6A30/6A designed to different torsional provisions, the 2[nd] method and with the accidental eccentricity.

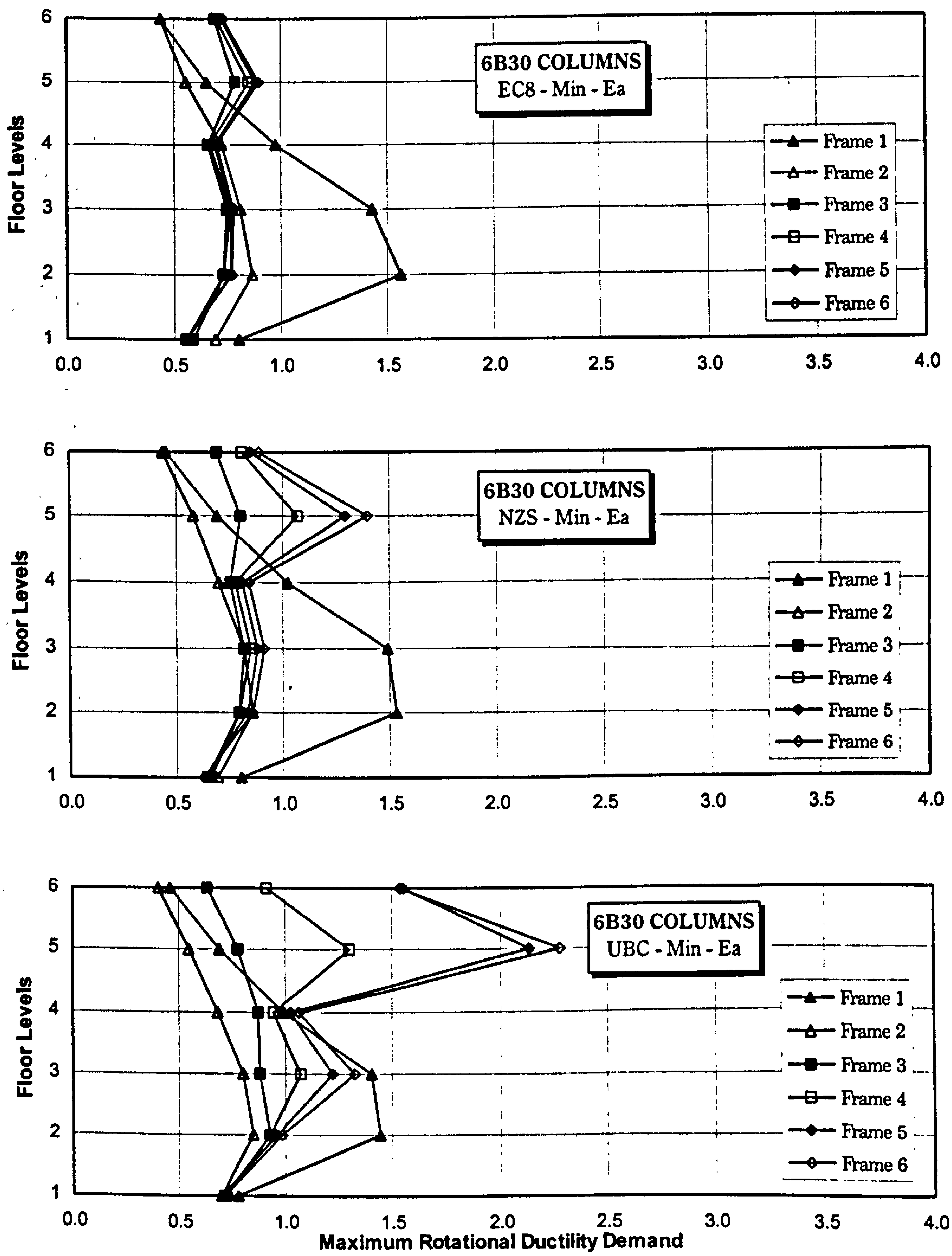


Figure 7.6.5 Maximum rotational ductility demand vs. floor levels for the columns of model 6B30 designed with different torsional provisions, the 2nd method and with the accidental eccentricity.

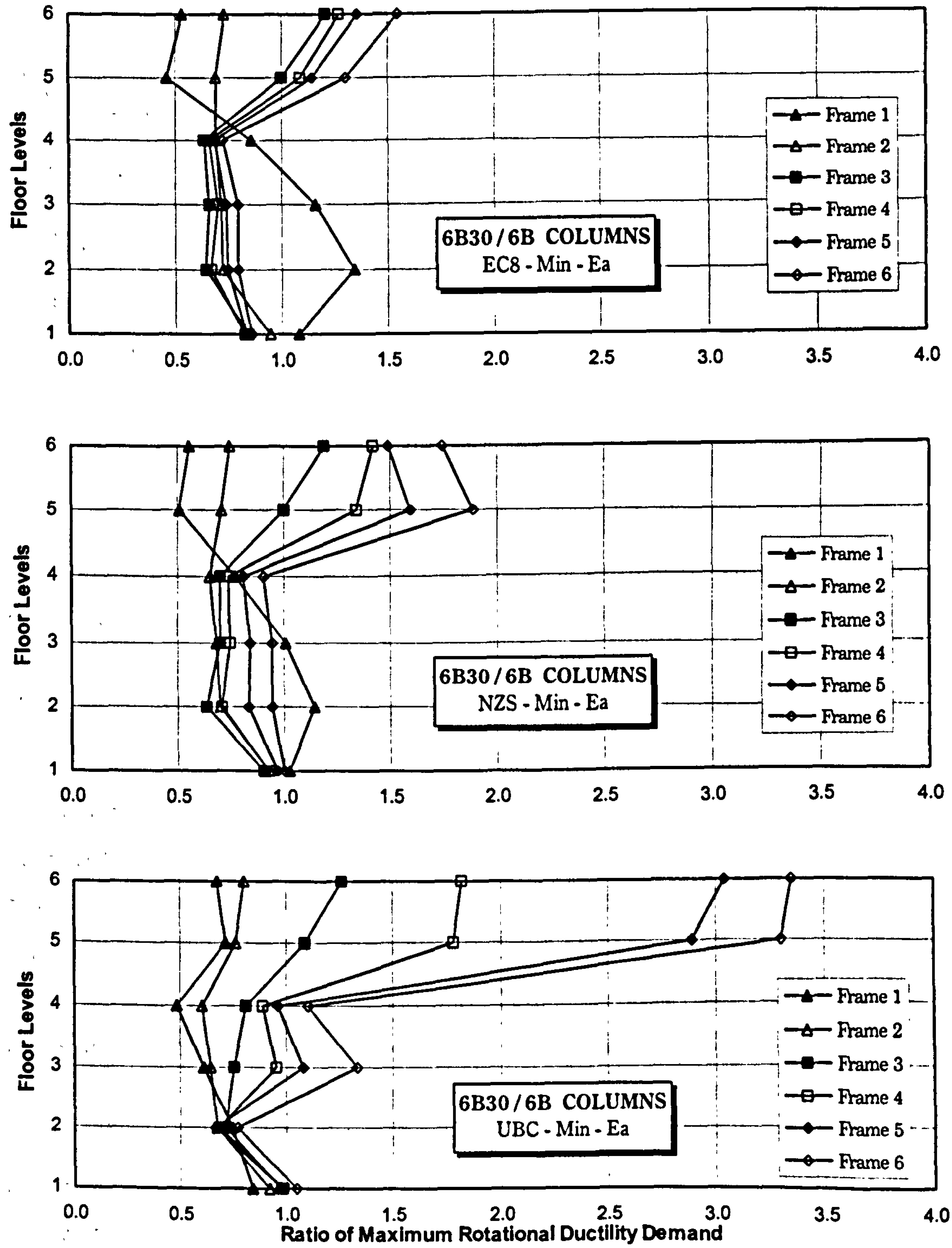


Figure 7.6.6 Ratios of rotational ductility demand vs. floor levels for the columns of model 6B30/6B designed to different torsional provisions, the 2nd method and with the accidental eccentricity.

CHAPTER 8

INELASTIC SEISMIC RESPONSE OF MULTI-STOREY REGULARLY ASYMMETRIC BUILDING MODELS WITH TRANSVERSE FRAME ELEMENTS

8.1 INTRODUCTION

From previous studies examining the inelastic seismic response of asymmetric (TU) systems, it has been concluded that the 3-element models, widely employed by many investigators, are adequate to assess the global (structure) and local (element) response of real structures subjected to uni-directional (lateral) loading with acceptable accuracy (Duan, 1991). However, some investigators (Goel and Chopra, 1990a, 1990b, 1991a, 1991b) questioned the ability of a 3-element model to represent adequately the torsional behaviour of real TU structures incorporating load-resisting elements oriented in both principal directions (Section 3.2.1). This is due to the fact that the structural elements oriented perpendicular to the earthquake direction may contribute significantly towards the torsional stiffness of the structure. Moreover, most real structures consist of load resisting elements aligned parallel to both orthogonal axes. Therefore, in order to ensure widely applicable results, the response of multi-storey regularly asymmetric structures with transverse elements subjected to both uni-directional and bi-directional seismic loading has also been investigated in this chapter.

Seismic code provisions stipulate that the two sets of orthogonal elements in a structure are designed independently by considering each normal principal direction separately. Therefore, the 3-element structural model represents a system where only the lateral direction is considered. However, when coupling arises between the torsional and translational oscillations of a structure, a more general and representative system should be employed. In the latter case, both sets of orthogonal elements are included and the model is loaded by both horizontal earthquake components (bi-directional loading).

When the lateral elements (parallel to the earthquake direction) are loaded into the inelastic range, the transverse elements (perpendicular to the earthquake loading) remain essentially elastic and contribute to the torsional structural resistance (Goel and Chopra, 1991b). Therefore, although the lateral elements do not contribute to the torsional stiffness, the structure continues to have considerable torsional stiffness due to the transverse elements. Furthermore, Goel and Chopra (1990a) concluded that the inclusion of the transverse elements affects the response of systems in the short-period region by considerably reducing their torsional deformation. In the medium/long-period range, they noticed that the inclusion of transverse structural elements has little effect on the system's torsional response due to the reduced yielding levels in structures having longer periods.

Duan (1991) observed that real structures undergo seismic loading along both principal directions and both horizontal earthquake components excite them when subjected to real earthquake ground motions. Thus, systems with transverse elements should be loaded by both earthquake components in order to understand better the effect of the transverse elements on the inelastic torsional response of asymmetric systems. The inelastic response of asymmetric models including transverse elements and subjected to bi-directional loading should be compared to the response of the same models subjected to uni-directional earthquake loading in order to indicate the influence of the inclusion of the second earthquake component.

Therefore, the multi-storey TU systems with transverse elements investigated in this chapter are subjected to both a bi-directional and a uni-directional seismic loading and their inelastic torsional behaviour is compared to the behaviour of similar TU models without transverse elements. In that way, the applicability of the conclusions of previous studies examining the inelastic torsional response of TU models without transverse elements is examined and the influence of the transverse elements on the response of asymmetric models is investigated. The response of TU models incorporating transverse elements is compared with the inelastic seismic response of TU models with structural elements only in the lateral direction, as examined in Chapters 6 and 7, and the differences between their inelastic responses are assessed.

8.2 STRUCTURAL CONFIGURATION OF TU MODELS WITH TRANSVERSE ELEMENTS

Correnza (1994) investigated the effect of including transverse elements on the inelastic torsional response of TU models by adopting two different types of single-storey 5-element models. These models were defined with respect to the 3-element models also investigated by Correnza (1994) which consisted of structural elements located only in the lateral direction. Two separate and alternative criteria were applied for the modification of the 3-element models and two types of 5-element models were studied. The first 5-element model included transverse elements without increasing the torsional stiffness of the model, but with a consequent modification of the lateral stiffness distribution. This was achieved by compensating for the torsional stiffness contribution of the transverse elements, by imposing equal stiffness reductions in the edge elements and increasing the stiffness of the middle element to maintain the same lateral stiffness. The second 5-element model was defined without changing the lateral stiffness distribution of the 3-element model, leading to some increase of the torsional stiffness of the structure.

In this study, the TU models with transverse elements are determined based on the second definition (Correnza, 1994), that is by adding transverse elements directly to the models analysed in Chapters 5 – 7. Therefore, the TU models with transverse elements maintain the same lateral stiffness distribution as the TU models without transverse elements. The columns and beams oriented in the lateral direction remain the same while beam elements are added in the transverse direction. The systems investigated are the stiffness-eccentric models 6B30 and 6BT30 and the mass-eccentric models 6S30 and 6ST30, with static eccentricities equal to 0.30*b* (Figures 8.2.1 and 8.2.2). All models consist of six storeys, model 6ST30 is similar to model 6S30 and model 6BT30 is similar to model 6B30, except for the inclusion of beam elements in the transverse direction.

The models without transverse elements examined in the previous chapters have different stiffness distributions, but they always have the same total lateral stiffness (Section 5.2). Each model with transverse elements maintains the same stiffness distribution in the lateral direction while identical beam elements are added to both model types in order to have the same total stiffness in the transverse direction. Therefore, the fundamental period in the transverse direction is the same as the fundamental period in the lateral direction for all the systems investigated. The dimensions of the structural elements of each model analysed in this chapter are given in Tables 8.2.1 and 8.2.2 and the frames added in the transverse direction are termed frames 7 – 9 (Figure 8.2.2).

BEAM DIMENSIONS FOR THE MODELS WITH TRANSVERSE ELEMENTS				
Model Type	Frame (1)	Frames (2 – 5)	Frame (6)	Frames (7 – 9)
ST	30.9 x 61.8	35.3 x 70.5	30.9 x 61.8	30.9 x 61.8
BT	47.6 x 95.1	28.3 x 56.6	23.8 x 47.6	30.9 x 61.8

Table 8.2.1 Beam dimensions for the models with transverse elements (in cm).

COLUMN DIMENSIONS FOR THE MODELS WITH TRANSVERSE ELEMENTS			
Model Type	Frame (1)	Frames (2 – 5)	Frame (6)
ST	46.4	52.9	46.4
BT	71.4	42.4	35.7

Table 8.2.2 Column dimensions for the models with transverse elements (in cm).

8.3 INFLUENCE OF DIFFERENT SEISMIC LOADING CONDITIONS

8.3.1 Inelastic Response of Reference Models with Transverse Elements Subjected to Different Loading Conditions

The inelastic behaviour of various reference models without transverse structural elements was analytically examined in Chapter 5 and the factors influencing their response were discussed in Sections 5.4 and 5.5. In this chapter, reference models including transverse elements are employed and they are termed models 6ST and 6BT (see Figure 8.2.2). Model 6ST has the same structural configuration as model 6S in the lateral direction and model 6BT consists of identical lateral structural elements as model 6B. Transverse beam elements have been added in both models 6ST and 6BT.

A comparison between the response of reference models with and without transverse elements is carried out in this section and the differences between the response of SM and TB models are observed. The inclusion of one or two components of the earthquake ground motion (uni-directional or bi-directional seismic loading) is also discussed and three different cases are investigated. In the first case (case I), the reference models do not include transverse elements (models 6S and 6B) and they are subjected to only the lateral earthquake component. In the second case (case II), reference models with identical lateral configurations and beams in the transverse direction (models 6ST and 6BT) are subjected to uni-directional seismic loading. Finally, in the third case (case III), the reference models 6ST and 6BT with transverse elements are subjected to bi-directional earthquake loading. The strength of the elements is calculated by including the minimum steel ratios and the accidental eccentricity provisions while the NZS code is employed for the representative design of the models. For the bi-directional loading, ground motion recordings in both orthogonal directions are applied, and the earthquake component with the lower maximum ground velocity acts in the transverse direction (see also Sections 4.4.1 and 4.4.2). The earthquake records applied are shown in Table 4.3.1 while, in order to maintain the integrity of the earthquake event, both earthquake components are scaled by the same factor (Table 4.3.3).

Figure 8.3.1 presents the maximum ductility demand for the SM models 6S and 6ST subjected to both a uni-directional and a bi-directional loading. The response of the lateral elements presents no differences for the different cases examined and the response of the lateral frames 1 – 6 is not influenced by the transverse elements or by the second earthquake component. Due to the symmetric model configuration, the response of frame 1 is identical to the response of frame 6, the response of frame 2 is identical to frame 5, and the response of frame 3 is identical to the response of frame 4 (for both columns and beams). Contrary to the lateral frames, the transverse frames 7 – 9 present some ductility demand only when the second earthquake component is incorporated (case III) (Figure 8.3.1). Moreover, only the transverse columns located at the 5^{th} floor level result in a ductility demand higher than unity, which is higher than the ductility demand of the lateral columns of the same floor level. The response values of the transverse beams are similar or lower than the response values of the lateral beams, and the transverse beams result in identical response values with an increased ductility demand at the 5th storey.

Contrary to the SM models, the response of the lateral frames in TB models is influenced by the transverse elements while it is not influenced by the earthquake loading condition (uni-directional or bi-directional loading) (Figures 8.3.2 and 8.3.3). Therefore, cases II and III result in identical response values for both columns and beams, and no differences are observed in the behaviour of the lateral elements. When the transverse elements are included, the ductility demand of the columns of frame 1 is increased while the rest of the columns present slightly reduced ductility demand values. No major differences are presented in the response of the beams, which result in similar ductility demand values for all the cases (Figure 8.3.3). Similar to model 6ST, the transverse frames of model 6BT result in non-zero ductility demand, only when subjected to bi-directional seismic loading (case III). The inelastic response of all transverse frames is similar (Figures 8.3.2 and 8.3.3) while, for both columns and beams, an increased ductility demand is observed at the 5th storey, as indicated in SM models (Figure 8.3.1).

8.3.2 Inelastic Response of TU Models with Transverse Elements Subjected to Different Loading Conditions

The TU models examined are the mass-eccentric models 6S30 and 6ST30 and the stiffness-eccentric models 6B30 and 6BT30. All models are designed by including the minimum steel ratios and the NZS torsional provisions. The response of the lateral elements is influenced significantly by the inclusion of transverse frames (Figures 8.3.4 – 8.3.7) while the inclusion of the second earthquake component does not change their behaviour and cases II and III result in identical ductility demand. The lateral frames are influenced in a different way by the transverse elements, depending on whether the model is stiffness-eccentric or mass-eccentric. In mass-eccentric models without transverse elements, frames 5 and 6 result in the highest ductility demand values while, when the transverse frames are included, the upper storeys of frames 1 and 2 respond less favourably than the other frames (Figures 8.3.4 and 8.3.5). Therefore, the inclusion of the transverse elements increases the ductility demand of the upper floors of frames 1 – 3 while it reduces the response of frames 5 and 6.

In stiffness-eccentric models, the incorporation of transverse elements increases the ductility demand of the upper storeys of frames 3 – 6 while it reduces the response of frame 1. Consequently, when the transverse elements are not included, frame 1 presents the highest response while, by including transverse elements, the most vulnerable frames are the upper storeys of frames 3 – 6 (Figures 8.3.6 and 8.3.7). Furthermore, in both stiffness- and mass-eccentric models, the transverse frames result in ductility demand values lower than unity for systems subjected to uni-directional loading (case II). However, for a bi-directional loading (case III), their ductility demand increases significantly, especially at the 5th floor level (as also indicated in the reference models).

The differences in the response of TU models subjected to different loading conditions, with or without transverse elements, can be also observed in their plastic hinge formation. For the mass-eccentric models 6S30 and 6ST30, the plastic hinge formation of frames 1, 6 and 7 is presented in Figures 8.3.8 and 8.3.9. The increased ductility demand in the upper floors of frame 1 due to the inclusion of the transverse elements is clearly indicated in its plastic hinge formation (Figure 8.3.8). Figure 8.3.8 also indicates an increased hinge formation in the upper storeys of frame 6 and a reduced hinge formation in the lower storeys. The increased ductility demand of the transverse frames due to the second earthquake component is indicated in Figure 8.3.9, which presents clearly an increased plastic hinge formation at the upper storeys of frame 7.

Generally, the differences observed in the response of the lateral frames due to the inclusion of the transverse frames are caused by the varying stiffness distribution of the models that result in different time-history displacements. The time-history displacement of frames 1 and 6 for models 6S30 and 6ST30 indicates that the transverse elements change the lateral displacement of TU models whilst cases II and III result in an identical lateral displacement (Figure 8.3.10). The same conclusion is also reached by presenting the maximum lateral displacement of models 6S, 6ST, 6S30 and 6ST30 (Figure 8.3.11). The lateral displacement of the reference models is identical for all three cases while the lateral displacement of the TU models is reduced by the inclusion of the transverse elements. The maximum transverse displacement of TU models with transverse elements is considerably increased for a bi-directional excitation, indicating the great influence of the second earthquake component in the response of the transverse frames.

8.4 INFLUENCE OF DIFFERENT SEISMIC CODE PROVISIONS

8.4.1 Inelastic Response of Mass-Eccentric TU Models Designed to Different Torsional Provisions

The influence of different torsional provisions in the inelastic response of various TU models was analytically examined in Section 6.7 and the behaviour of the same models was compared to the behaviour of their reference models in Section 7.6. In this chapter, the influence of different torsional provisions is further analysed by employing TU models that incorporate elements in both horizontal directions, subjected to bi-directional earthquake loading (case III). The models without transverse elements (presented for comparison reasons) are subjected to uni-directional loading (case I) and the strength of the elements is calculated by including the minimum steel ratios and the accidental eccentricity provisions. In this section, the mass-eccentric models 6S30 and 6ST30 are analysed and designed to the torsional provisions of all three seismic codes.

The results presented in Figures 8.4.1 and 8.4.2 indicate that, as also concluded in Section 7.6.2, the frames resulting in the highest values of rotational ductility demand in mass-eccentric models are frames 5 and 6. The inclusion of transverse elements (model 6ST30) results in a completely different behaviour of the lateral frames (Figures 8.4.3 and 8.4.4) since it reduces the response of frames 5 and 6 and increases the response of frames 1 – 3. Both the ductility demand and the normalised ductility demand follow similar response patterns, showing that the upper storeys of frames 1 – 3 and 6 produce ductility demand values higher than unity and higher than the ductility demand of the reference model. No major differences are encountered when different seismic code regulations are adopted (Figure 8.4.3). Especially for the EC8 and NZS codes, the ductility demand values are identical while the UBC code results in higher response values in the top storey of frames 1 and 2. Furthermore, the normalised ductility demand is similar for EC8 and NZS and higher for UBC (Figure 8.4.4). It should be also noticed that, while the maximum ductility demand in model 6S30 is equal to 2.0, the maximum ductility demand in model 6ST30 reaches 5.0, at the top floor level of frame 1.

The inelastic response of the transverse frames indicates that only the ductility demand of the 5^{th} floor level of frames 7 – 9 exceeds unity (Figures 8.4.5 and 8.4.6). The maximum response values are located in frame 7 while no major differences in the ductility demand values of the transverse frames are found when designed to different codes. The increase of the ductility demand in the TU model is similar for all the floor levels of the transverse frames. Therefore, the excessive ductility demand found in the 5th storey of the frames of model 6ST30 is similar to the ductility demand of the same storey of model 6ST (Figure 8.3.1). Hence, no major differences are found in the response of the transverse frames due to the fact that, in the transverse direction, there is no static eccentricity and the stiffness and mass distributions of the TU model are identical to the corresponding distributions of the reference model. The maximum interstorey drift ratios of models 6S30 and 6ST30 (Figure 8.4.7) indicate that frame 6 of model 6S30 and frame 1 of model 6ST30 result in the highest ratios since these frames result in the highest response values (Figures 8.4.1 – 8.4.4). The interstorey drift ratios of both lateral and transverse frames are always much lower than the 2% collapse limit, which in combination with a column sidesway mechanism would indicate likely collapse.

Furthermore, no major differences arise in the plastic hinge formation of frames 1, 6 and 7 of model 6ST30 designed to any seismic code. The response of frames 1 and 7 is worst for the UBC regulations while the response of frame 6 is similar for all seismic codes. In the transverse frame 7, there is an increased formation of column plastic hinges in the 5th floor level (see also Figure 8.4.5), but there is no formation of a column sidesway mechanism. The 5th floor level of frame 1 is the only storey that presents the formation of a column sidesway mechanism, with plastic hinges formed at both the top and the bottom of all vertical members.

Previous studies (Kappos, 1991a) indicated that the above sidesway criterion is conservative because at the time that a plastic hinge forms at a certain member, another member may enter in the unloading stage and respond with a stiffness equal or lower than the elastic (see also Section 4.7). Therefore, a combined criterion is adopted involving both the formation of a column sidesway mechanism and the occurrence of an interstorey drift ratio in excess of 2%. As indicated in Figure 8.4.7, all frames of model 6ST30 result in interstorey drift ratios much lower than the 2% limit, indicating no collapse even for the 5th floor level of frame 1. Similarly, none of the rest of the frames reaches collapse while an increased caution should be paid for the upper floor levels of frames 1 – 4, where higher ductility demand values are found.

8.4.2 Inelastic Response of Stiffness-Eccentric TU Models Designed to Different Torsional Provisions

In Sections 6.5 and 7.4, the influence of the structural configuration in the inelastic response of different TU models was proved and, in Sections 6.7 and 7.6, the effect of the torsional provisions in different model types was examined. The varying response of mass-eccentric and stiffness-eccentric models is further investigated in this section by examining the torsional behaviour of the stiffness-eccentric models 6B30 and 6BT30 designed to the torsional provisions of different codes. Model 6BT30 having transverse elements is subjected to bi-directional earthquake loading while model 6B30 without transverse elements is subjected to uni-directional loading.

As also concluded in Section 7.6.2, in stiffness-eccentric models without transverse elements, the frames responding in the inelastic range are the lower storeys of frame 1 (for all codes) and frames 5 and 6 (for NZS and UBC) (Figure 8.4.11). EC8 code requiring an additional eccentricity for the first design eccentricity is the only code that results in no ductility demand at the flexible side while their highest ductility demand is found for the UBC code. Furthermore, the frames that respond worst than their reference models are the upper storeys of frames 4 – 6 (for all codes) and the lower storeys of frame 1 (for EC8 and NZS) (Figure 8.4.12). The UBC provision permitting no strength reduction in the TU model results in no additional ductility demand in the stiff side (frame 1). Moreover, the normalised ductility demand of frame 1 is higher for the EC8 code while the normalised ductility demand of frames 5 and 6 is higher for the UBC code. Consequently, the EC8 code controls better the ductility demand at the flexible edge while the response of the stiff side is better controlled by UBC code.

The inclusion of transverse elements (model 6BT30) results in a different behaviour of the lateral frames (Figures 8.4.13 and 8.4.14). Both the ductility demand and the normalised ductility demand of the frames follow similar response patterns, indicating that mainly the upper storeys of frames 3 – 6 produce a ductility demand higher than unity and, at the same time, higher than the ductility demand of the reference model. When the response of model 6BT30 is compared with the response of model 6B30, it can be noticed that the response of frame 1 is reduced while the response of the rest of the frames is increased. Differences in the ductility demand of model 6BT30 are encountered when designed to a different code (Figure 8.4.13). The lowest response values are found for the EC8 code and the highest for UBC while the normalised ductility demand values are similar for EC8 and NZS and higher for UBC (Figure 8.4.14). As also indicated in the mass-eccentric models, the maximum ductility demand in model 6B30 is equal to 2.0 while the maximum ductility demand in model 6BT30 reaches the value of 6.0 (for UBC). Similar to the inelastic response of models without transverse elements (Section 7.6.2), in models with structural elements in both directions, UBC code results in the highest response values for the flexible side while it is the only code that results in no rotational ductility demand at the stiff side.

The inelastic seismic response of the transverse frames is presented in Figures 8.4.15 and 8.4.16 indicating that, as in the case of mass-eccentric models, it is mainly the ductility demand of the 5th floor level of frames 7 – 9 that exceeds unity. Similar response values are found for all codes and the highest ductility demand is found for frame 9 (almost equal to 4.0) while, for the mass-eccentric model 6ST30, the response of frame 7 is the highest (Figure 8.4.5). The normalised ductility demand shows that there is only a small increase in the ductility demand of frames 7 and 8 while the ductility demand of frame 9 can be almost 4 times higher, when designed for the EC8 code (Figure 8.4.16). Therefore, the excessive ductility demand found in the 5th storey of the transverse frames of model 6ST30 is similar to the ductility demand found in model 6ST (Figure 8.3.1).

For both models 6B30 and 6BT30, the maximum interstorey drift ratios are found in frame 6 for the lateral direction and frame 9 for the transverse direction (Figure 8.4.17). The interstorey drift ratios of all the frames are always much lower than the 2% collapse limit, which in combination with a column sidesway mechanism would indicate collapse. The interstorey drift ratios have similar response values, almost reaching 1.2% in the lower floor levels of model 6B30 and in the upper floor levels of model 6BT30.

The plastic hinge formation of frames 1, 6 and 9 of model 6BT30 is presented in Figures 8.4.18 – 8.4.20 and the inelastic response of frames 6 and 9 is worst for the UBC regulations while the response of frame 1 is worst for the EC8 code. In the transverse frame 9, increased column hinges are observed in the 5th floor level, as also indicated in Figure 8.4.15, but no frame results in a column sidesway mechanism. Differences in the inelastic response of the frames when designed to different seismic codes can be also noticed in Figures 8.4.18 – 8.4.20. Frame 1 presents more plastic hinges for the EC8 code while frames 6 and 9 results in more plastic hinges for UBC. Generally, it could be said that a stiffness-eccentric model with transverse elements responds best for the EC8 code.

Similar to the results of this study, Correnza (1994) concluded that there is a significant inelastic response of the transverse elements for highly eccentric short-period systems, even when subjected to uni-directional loading, which increases with the inclusion of the transverse earthquake component. Furthermore, he observed that including transverse elements and a bi-directional loading increases the response of the flexible side by up to 100% in short-period structures, indicating that previous studies without transverse elements are not accurate for the flexible side of stiffness-eccentric models. De Stefano and Faella (1996) concluded that, in short-period systems, there is larger damage due to the bi-axial interaction effects, as also indicated from the results of this chapter. Furthermore, Jiang et al (1996) observed that the inelastic response of models without transverse elements is smaller than the response of models with transverse elements when excited in the inelastic range. Rutenberg et al (1992) concluded that the peak ductility demand of the transverse elements is lower than the peak ductility demand of the lateral elements (see also Figures 8.4.13 and 8.4.15).

Finally, it can be said that the inclusion of transverse elements excited by a bi-directional earthquake loading considerably changes the nature and the magnitude of the inelastic response of the models. Wong and Tso (1994) and Duan (1991) concluded that the results based on models without transverse elements subjected to uni-directional loading are adequate to give information regarding the inelastic seismic response of TU models including transverse elements and subjected to bi-directional loading.

Contrary to these conclusions, the results of this study indicated that the inclusion of transverse elements influences significantly the inelastic torsional behaviour of the models by considerably increasing the maximum ductility demands of specific lateral load-resisting structural elements. When the transverse frames are included, the torsional stiffness of the models is considerably increased and this results in lower maximum values of lateral displacements (Figure 8.3.11). The inclusion of the transverse beams changes the inelastic response of the model and alters the lateral displacement of each frame. Figure 8.3.10 indicates that the displacement of frame 6 is reduced while the displacement of frame 1 is increased. Moreover, the maximum lateral displacement presented in Figure 8.3.11 corresponds to the maximum displacement of frame 6 when no transverse elements are included while when the transverse elements are included, it corresponds to the maximum displacement of frame 1. Although the values of the lateral displacement are reduced due to the inclusion of the transverse frames (higher torsional stiffness), the maximum values of ductility demand are increased. This is explained by the results presented in Figures 8.4.7 and 8.3.4. Figure 8.4.7 indicates that although the maximum interstorey drift ratios of model 6ST30 are reduced, the interstorey drift ratios of frame 1 are increased while the drift ratios of frame 6 are reduced. Moreover, the interstorey drift ratio of frame 2 (where CM is located) is also increased with the inclusion of the transverse elements.

Therefore, the changes in the lateral displacements are consistent with the changes in the rotational ductility demand and, as a result, the ductility demand of frame 6 is reduced while the ductility demand of frame 1 is increased (Figure 8.3.4). This increase of the lateral displacement and of the ductility demand of frame 1 causes the increased response values of the model and, although its maximum lateral displacement is reduced by the inclusion of the transverse elements, its maximum ductility demand is increased. Therefore, an increased torsional stiffness does not necessarily imply that the ductility demand of the model will be reduced since the response of specific frames (frame 1 in this case) could control its inelastic response.

8.5 CONCLUSIONS

In this chapter, the inelastic seismic response of TU models incorporating transverse elements was examined by subjecting the models to both uni-directional and bi-directional seismic loading conditions. The inelastic torsional behaviour of these models was compared with the behaviour of TU models without transverse elements and the effect of the transverse elements was evaluated in combination with the earthquake loading. The conclusions reached in this chapter can be summarised as follows:

1. In symmetric reference models, the inclusion of transverse structural elements and the application of different seismic loading conditions (bi-directional or uni-directional earthquake loading) do not influence the inelastic seismic response of the lateral frames, which result in identical values of rotational ductility demand.
2. Contrary to the SM models, the response of the lateral frames in TB models is influenced by the inclusion of the transverse elements, resulting in different values of rotational ductility demand.
3. In TB models with transverse elements, the inelastic seismic response of the lateral frames is identical for both a uni-directional and bi-directional seismic loading.
4. The transverse elements of both SM and TB models respond in the inelastic range only when subjected to bi-directional seismic loading, indicating the significance of the second earthquake component. Therefore, only the inelastic seismic response of the transverse frames is influenced by the inclusion of the second earthquake component while the response of lateral frames remains identical.
5. Due to the symmetric configuration of the models in the transverse direction, the response of all transverse frames is similar in both SM and TB models while they present an increased ductility demand in the 5^{th} floor level.
6. Similar to the TB reference models, the inelastic response of the lateral frames in TU models is influenced by the inclusion of transverse elements while they are not influenced by the seismic loading condition (uni-directional or bi-directional loading).
7. The lateral frames in different types of TU models (mass-eccentric or stiffness-eccentric) are influenced in a different way by the transverse elements. In mass-eccentric models, the response of the upper floor levels in frames 5 and 6 is reduced (the most vulnerable frames without the inclusion of the transverse elements) while the response of the upper floor levels of frames 1 – 3 is increased, resulting in the highest ductility demand. In stiffness-eccentric models, the transverse elements increase the ductility demand of the upper floor levels in frames 3 – 6 while they reduce the response of frame 1 (the most vulnerable frame without the inclusion of the transverse elements).
8. Similar to the response of the transverse elements in the reference models, the inelastic seismic response of the transverse frames in TU models is considerably

influenced by the second earthquake component. Especially the ductility demand of the 5th floor level of the columns is significantly increased while the increase in the ductility demand of the beams is more uniform over the floors.

9. The lateral displacement of the reference models is not influenced by the transverse elements while the lateral displacement of TU models is reduced by the inclusion of the transverse frames. The lateral displacement of reference and TU models is not influenced by the seismic loading condition, resulting in identical values of lateral displacement for both seismic loading conditions applied. Furthermore, the transverse displacement of the reference models is identical for uni-directional or bi-directional seismic loading while the transverse displacement of the TU models is increased when the second earthquake component is included.
10. The torsional provisions of different seismic codes influence significantly the inelastic behaviour of the lateral frames in TU models, depending on the model type (mass-eccentric or stiffness-eccentric). Contrary to the lateral frames, the inelastic response of the transverse frames is not significantly influenced by the torsional provisions due to their symmetric configuration and the fact that there is no static eccentricity in this direction. Therefore, the transverse structural elements result in similar values of ductility demand when designed to different seismic codes.
11. The rotational ductility demand of the structural elements increases considerably in the lateral frames of both mass-eccentric and stiffness-eccentric models due to the inclusion of the transverse frames. Although the increased torsional stiffness of the system results in reduced values of lateral displacement, and the maximum ductility demand of the model is increased since it is controlled by the ductility demand of specific frames. Although the maximum lateral displacement of the system is reduced, some frames result in higher values of lateral displacement when the transverse elements are included (frame 1 in model 6ST30) and also result in higher ductility demand values. Therefore, the response of specific frames can affect significantly the inelastic response of the models.
12. In mass-eccentric models, the response values of the lateral frames are similar for the EC8 and NZS codes while differences are observed in the response values of the UBC code. The rotational ductility demand of the upper floor levels of frames 1 and 2

(the frames resulting in the highest inelastic response) is increased for the UBC code while the response of the upper floor levels of frames 3 and 6 is reduced for this code. Although the interstorey drift ratios of all frames are much lower than the 2% collapse limit, an increased caution should be paid in the upper floor levels of frames 1 – 3 and 6, which result in higher values of ductility demand (almost reaching 5.0). It should be reminded that similar TU models without transverse elements result in considerably lower values of ductility demand (reaching the value of 2.0).

13. In stiffness-eccentric models, the lowest response values of the lateral frames are found for the EC8 code while the UBC code results in the highest response values. The rotational ductility demand of the upper floor levels of frames 4 – 6 (the frames resulting in the highest inelastic response) is increased for the UBC code while the response of the stiff frame 1 is higher for the EC8 code. Although the interstorey drift ratios of all frames are much lower than the 2% collapse limit, an increased caution should be paid in the upper storeys of frames 3 – 6, which result in the highest ductility demand, reaching the value of 6.0. The maximum ductility demand found in similar TU models without transverse elements is 3 times lower than the ductility demand found in these models.
14. In stiffness-eccentric models, the EC8 torsional provisions control better the ductility demand of the flexible side due to the inclusion of the additional eccentricity component. The ductility demand of the stiff side is better controlled by the UBC code due to the provision permitting no strength reduction in the TU models compared with the strength of their reference models. A similar conclusion regarding the influence of the different seismic code provisions is also reached from the investigation of TU models without transverse elements, although the maximum values of ductility demand are lower and are located in different frames.
15. In mass-eccentric models, no major differences are found in the inelastic torsional response of the frames due to the inclusion of different seismic code regulations. EC8 and NZS codes seem to control better the inelastic response of frames 1 and 2 while UBC code controls better the response of frames 3 and 6. Similar remarks are made in the inelastic response of TU models without transverse elements although the maximum response values of the frames are lower and are located in different frames.

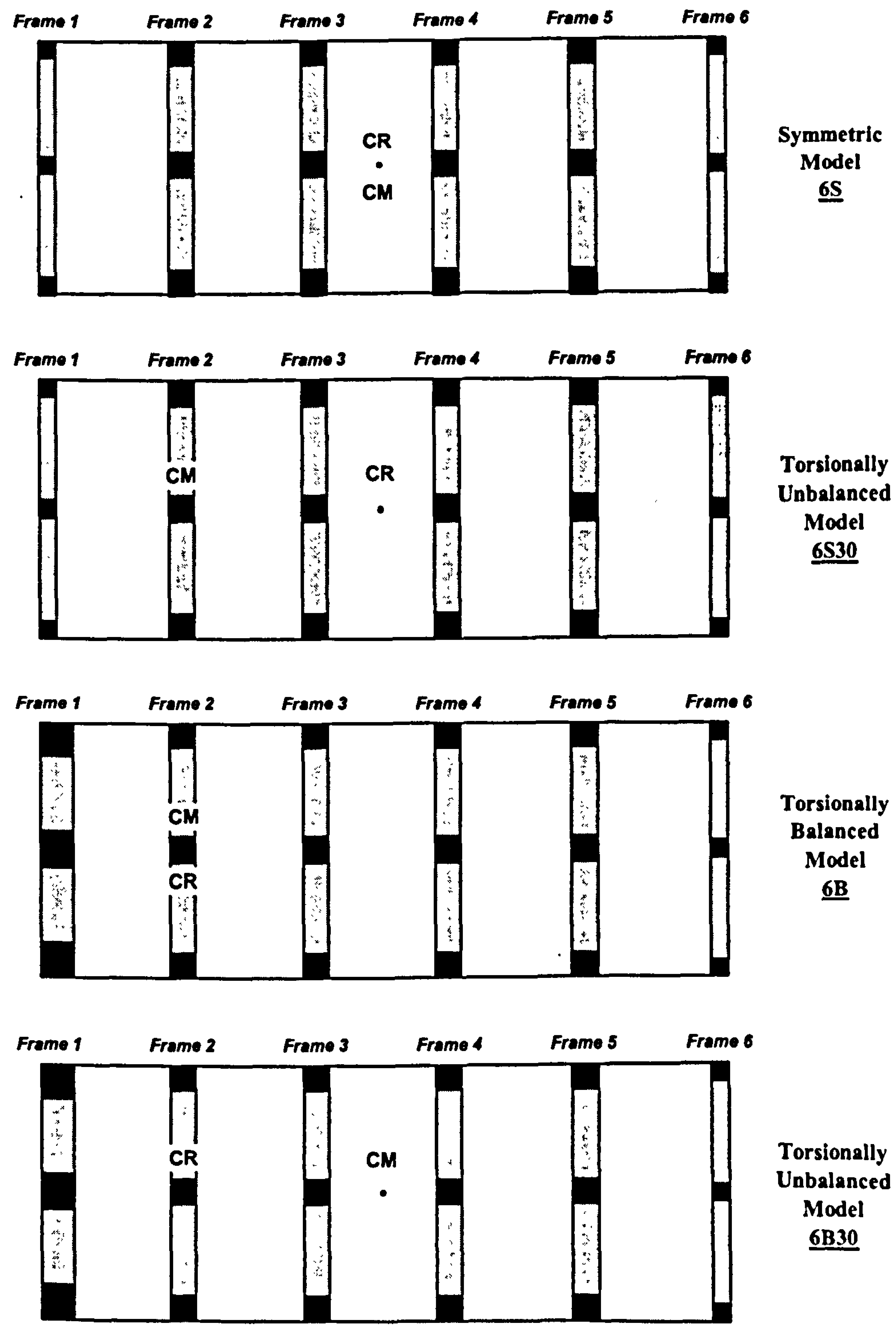


Figure 8.2.1 Structural configuration for the reference and the TU models without transverse elements examined in Chapter 8.

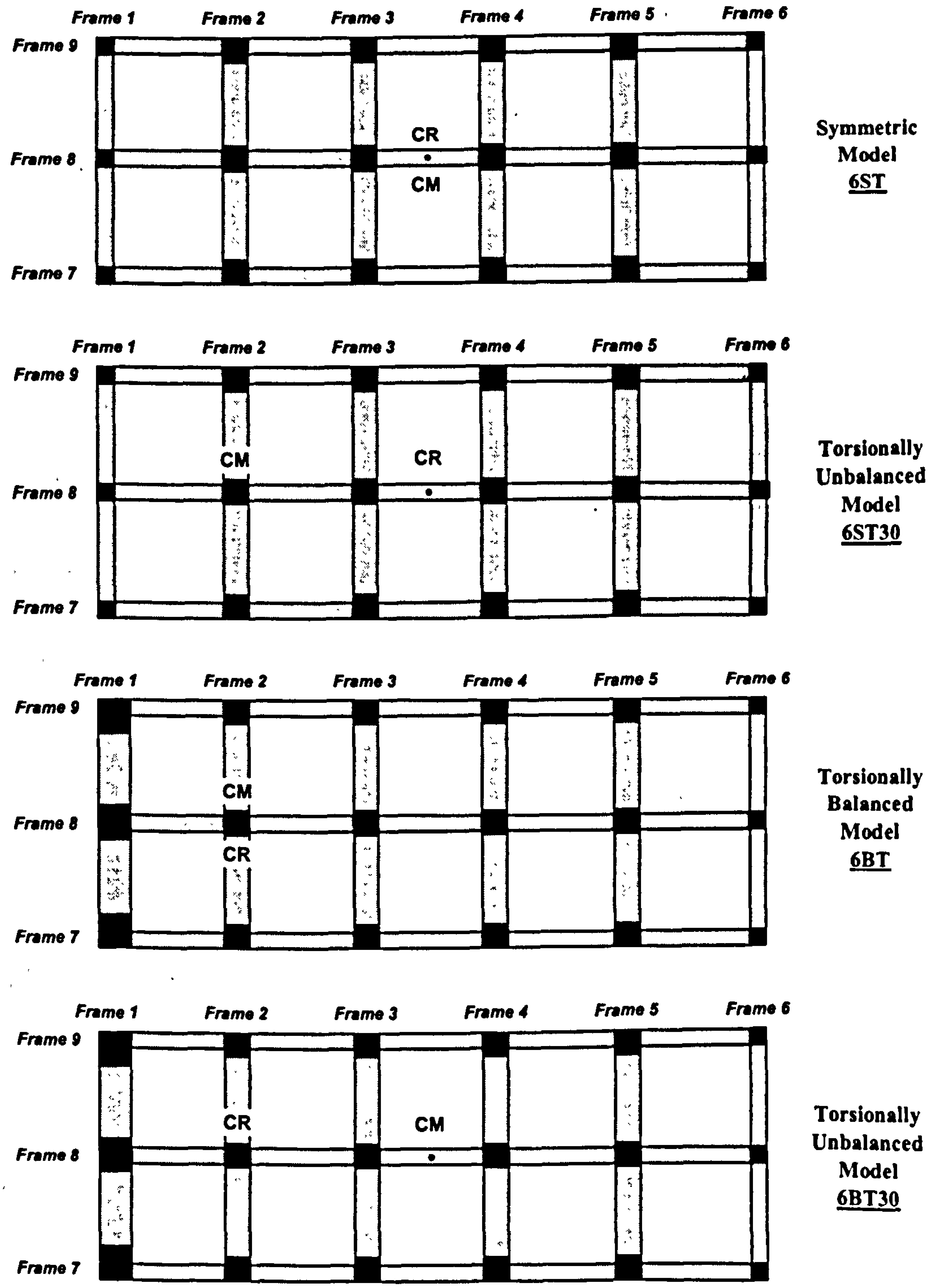
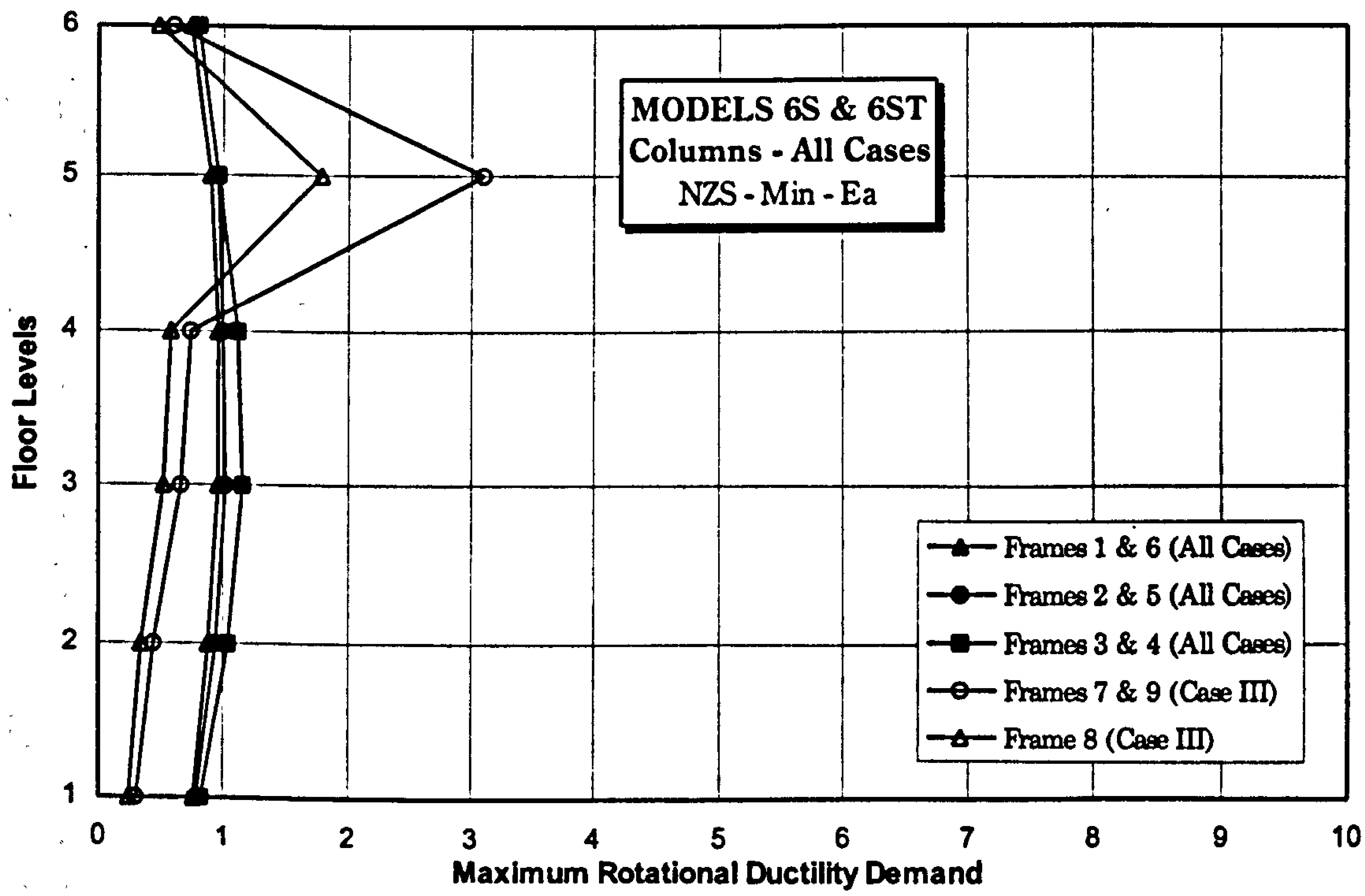


Figure 8.2.2 Structural configuration for the reference and the TU models with transverse elements examined in Chapter 8.



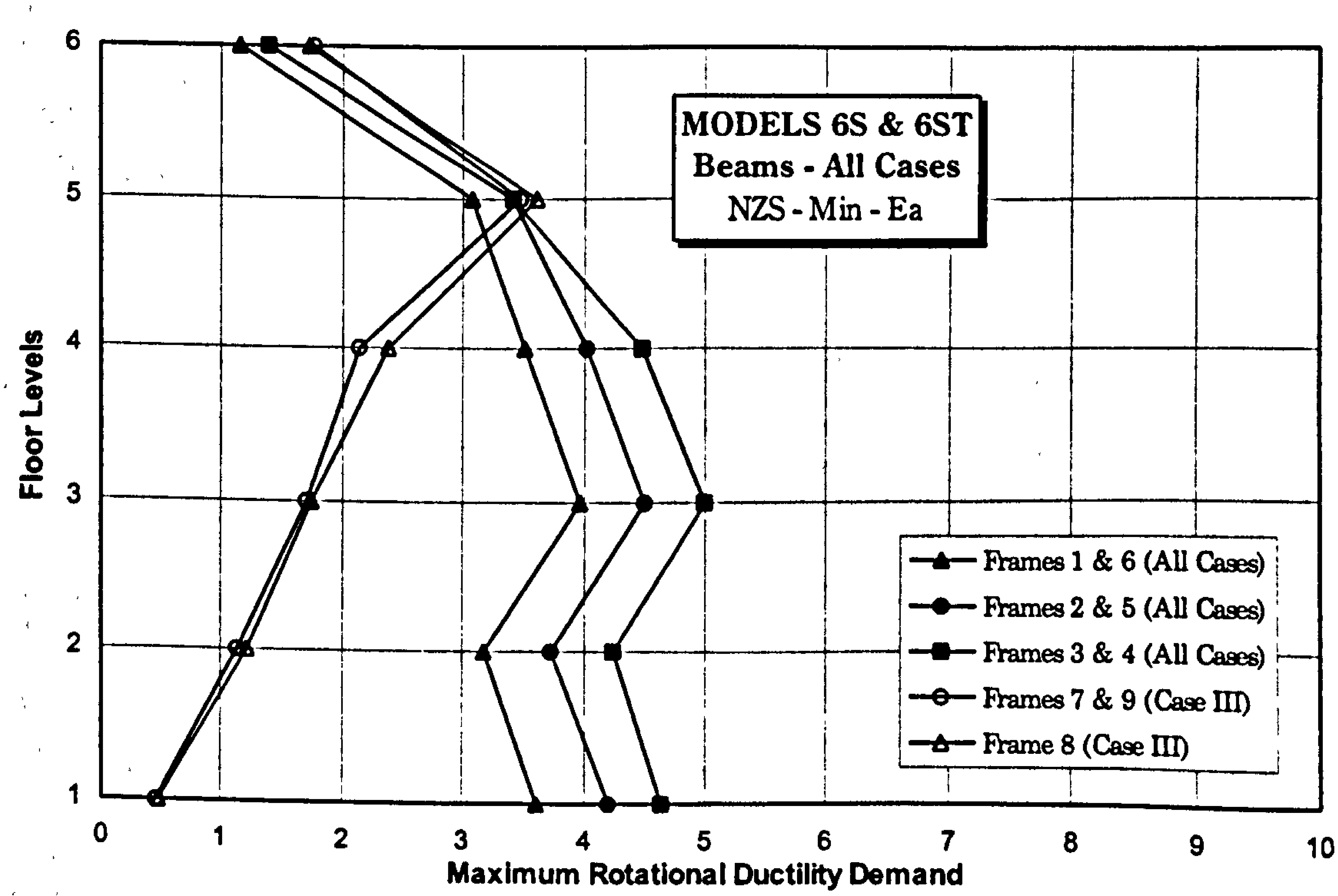


Figure 8.3.1 Maximum rotational ductility demand vs. floor levels for the symmetric reference models 6S and 6ST subjected to different seismic loading conditions.

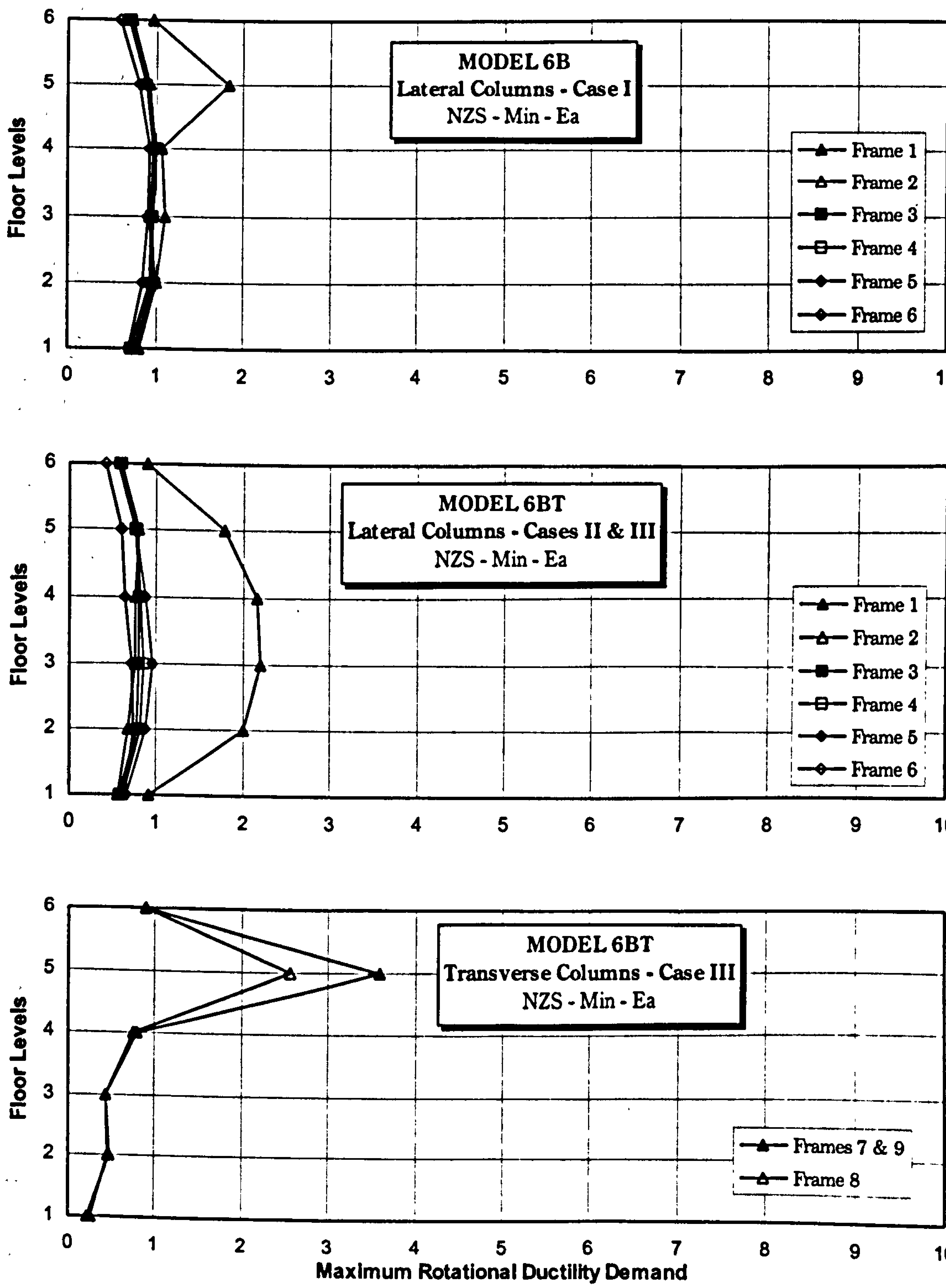


Figure 8.3.2 Maximum rotational ductility demand vs. floor levels for the column elements of the reference models 6B and 6BT subjected to different seismic loading conditions.

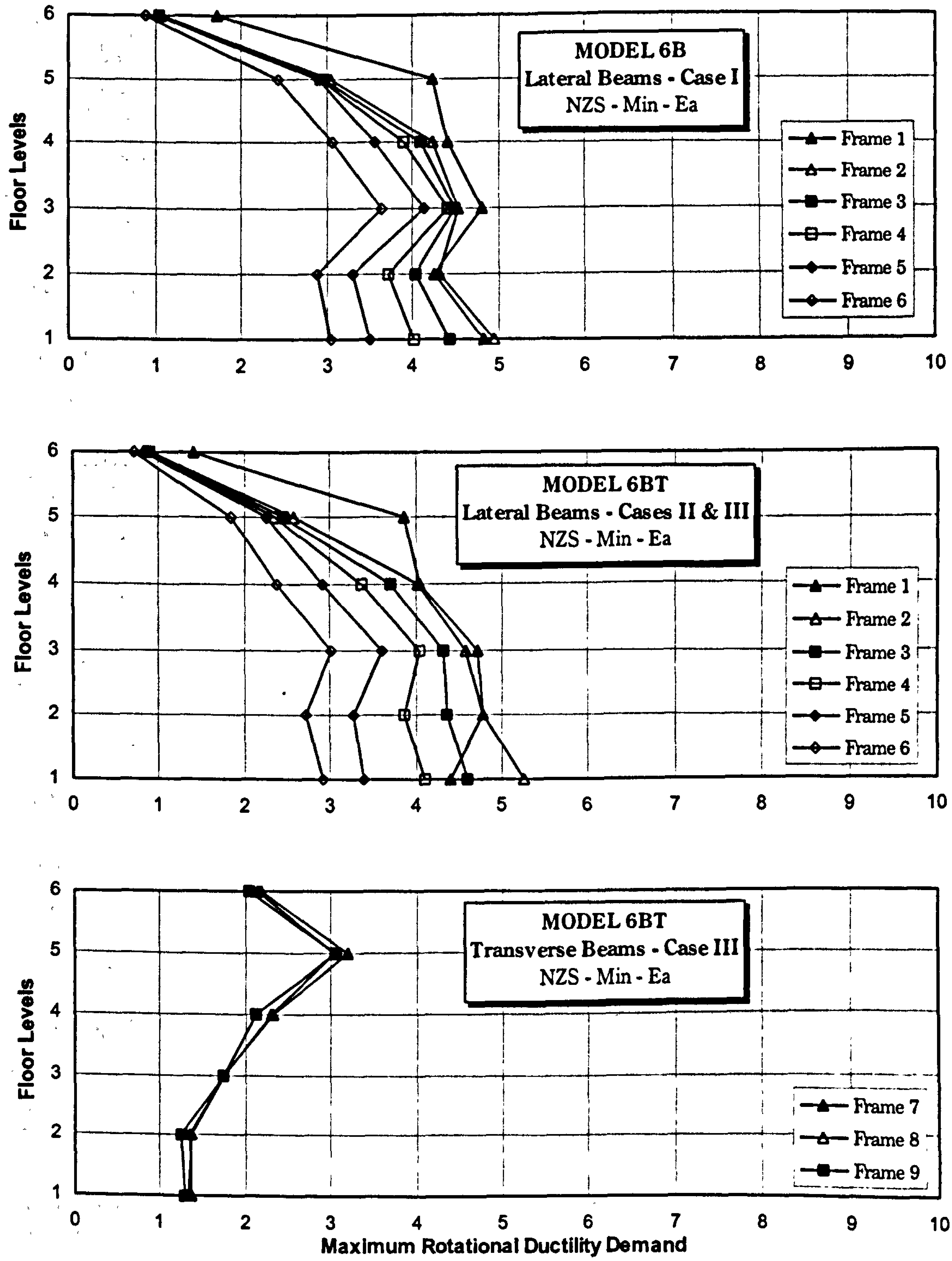


Figure 8.3.3 Maximum rotational ductility demand vs. floor levels for the beam elements of the reference models 6B and 6BT subjected to different seismic loading conditions.

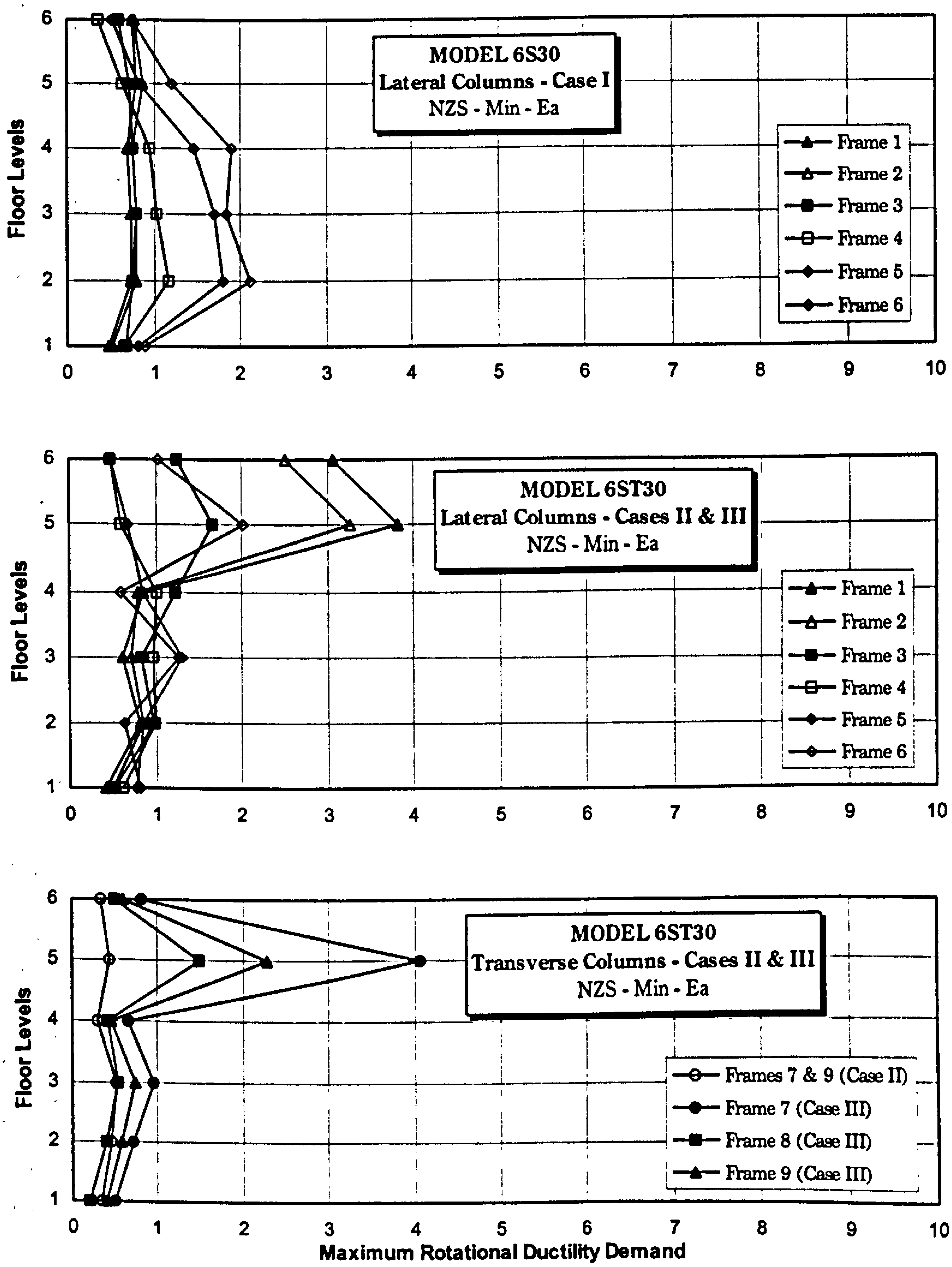


Figure 8.3.4 Maximum rotational ductility demand vs. floor levels for the column elements of the mass-eccentric models 6S30 and 6ST30 subjected to different loading conditions.

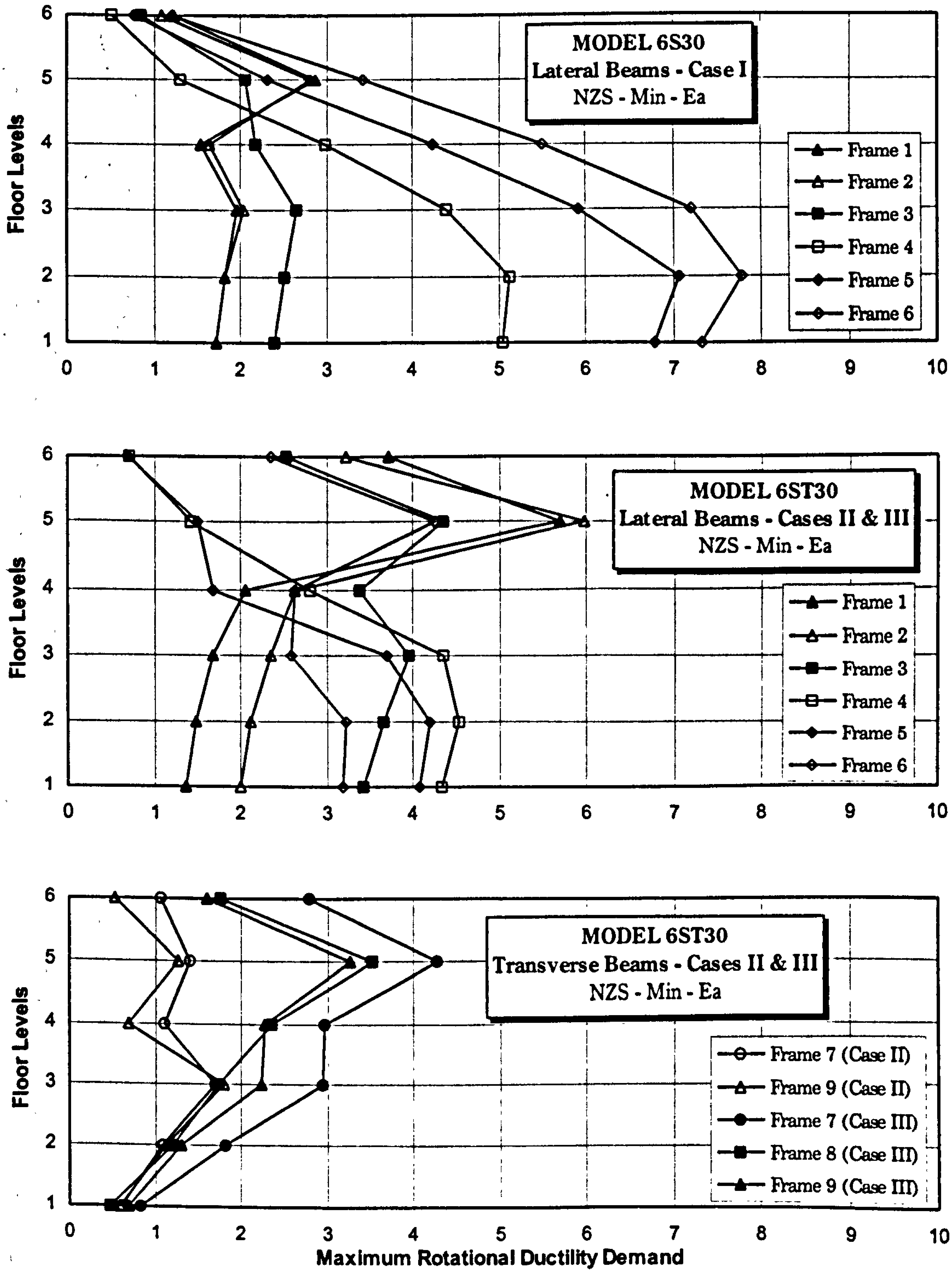


Figure 8.3.5 Maximum rotational ductility demand vs. floor levels for the beam elements of the mass-eccentric models 6S30 and 6ST30 subjected to different loading conditions.

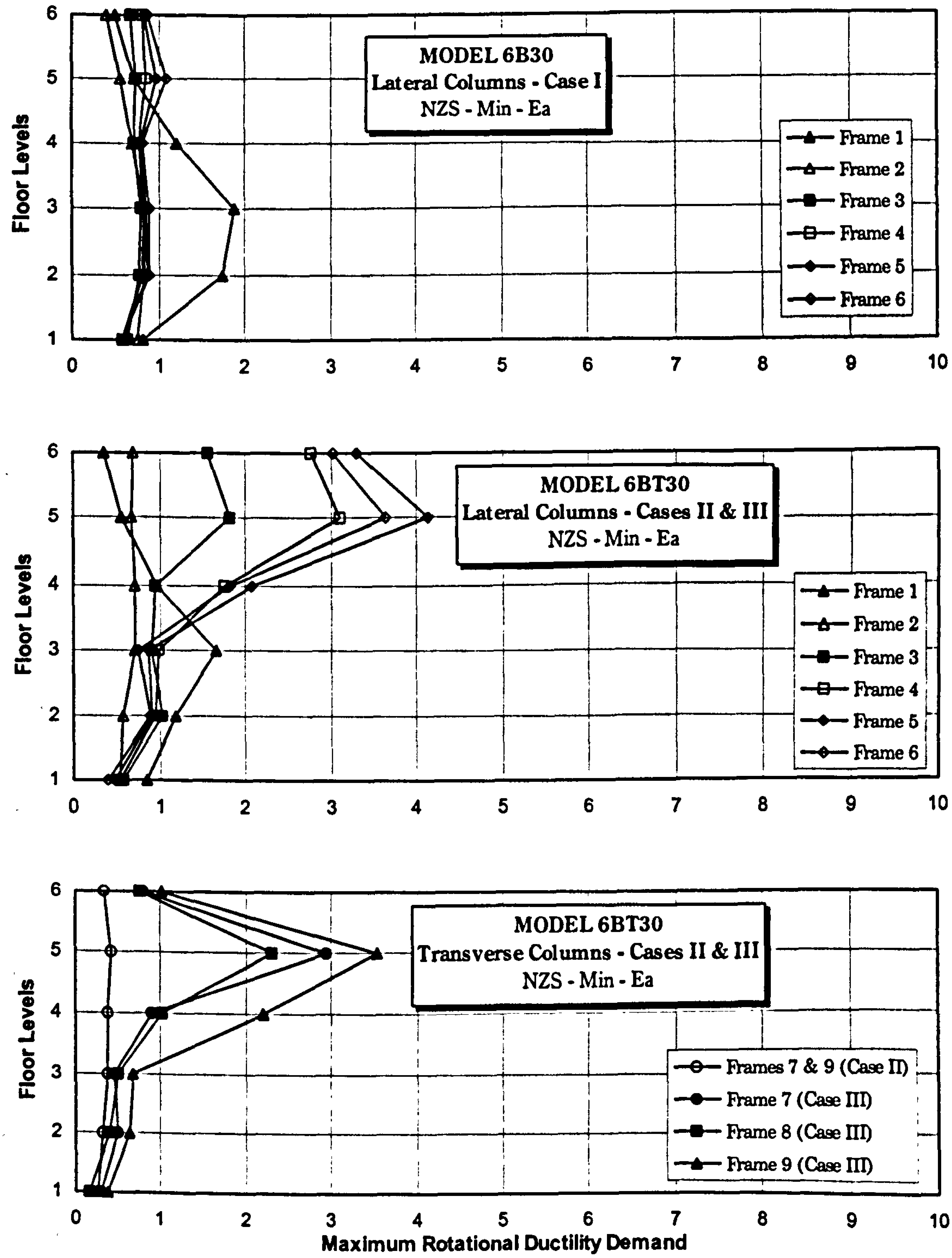


Figure 8.3.6 Maximum rotational ductility demand vs. floor levels for the column elements of the stiffness-eccentric models 6B30 and 6BT30 subjected to different loading conditions.

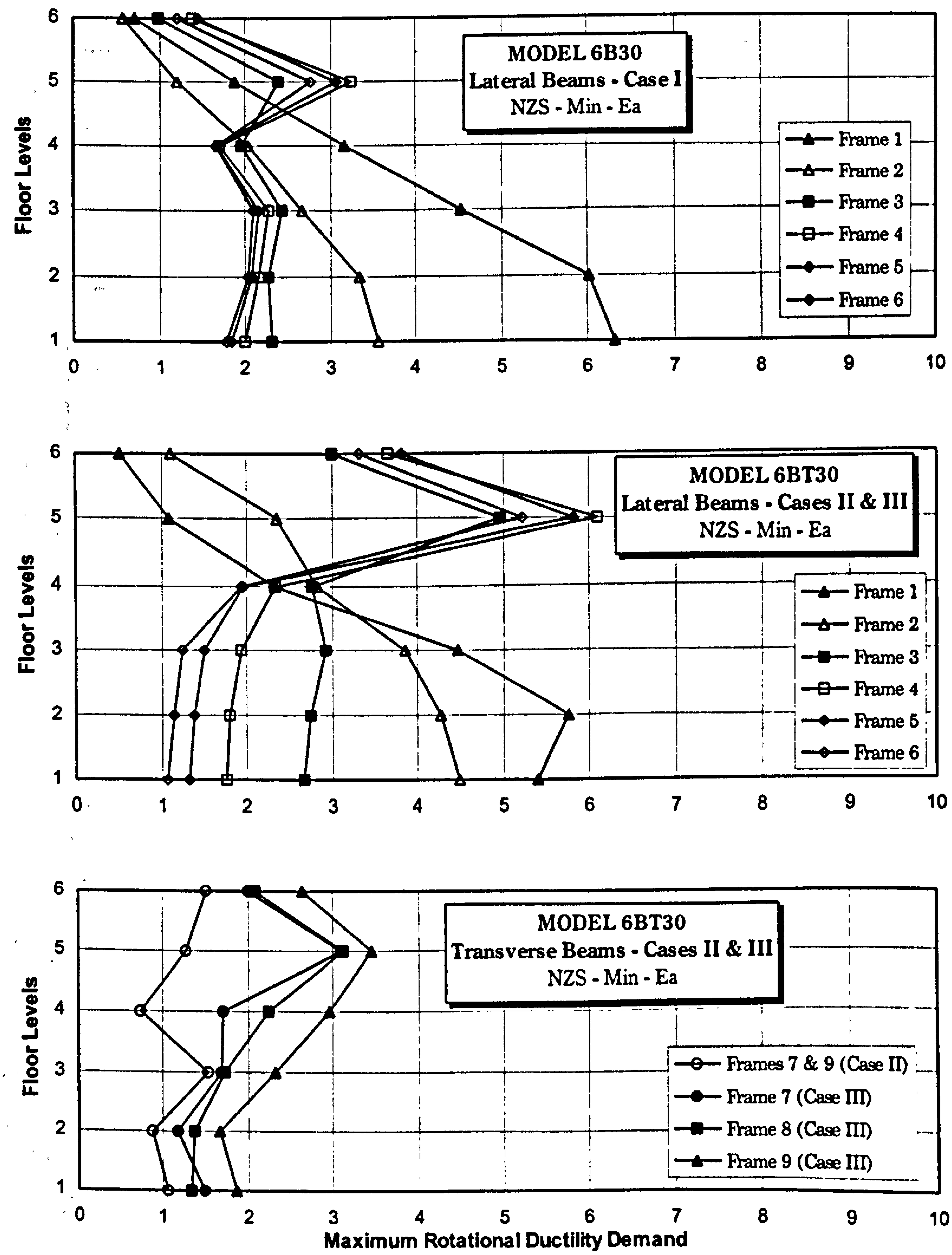


Figure 8.3.7 Maximum rotational ductility demand vs. floor levels for the beam elements of the stiffness-eccentric models 6B30 and 6BT30 subjected to different loading conditions.

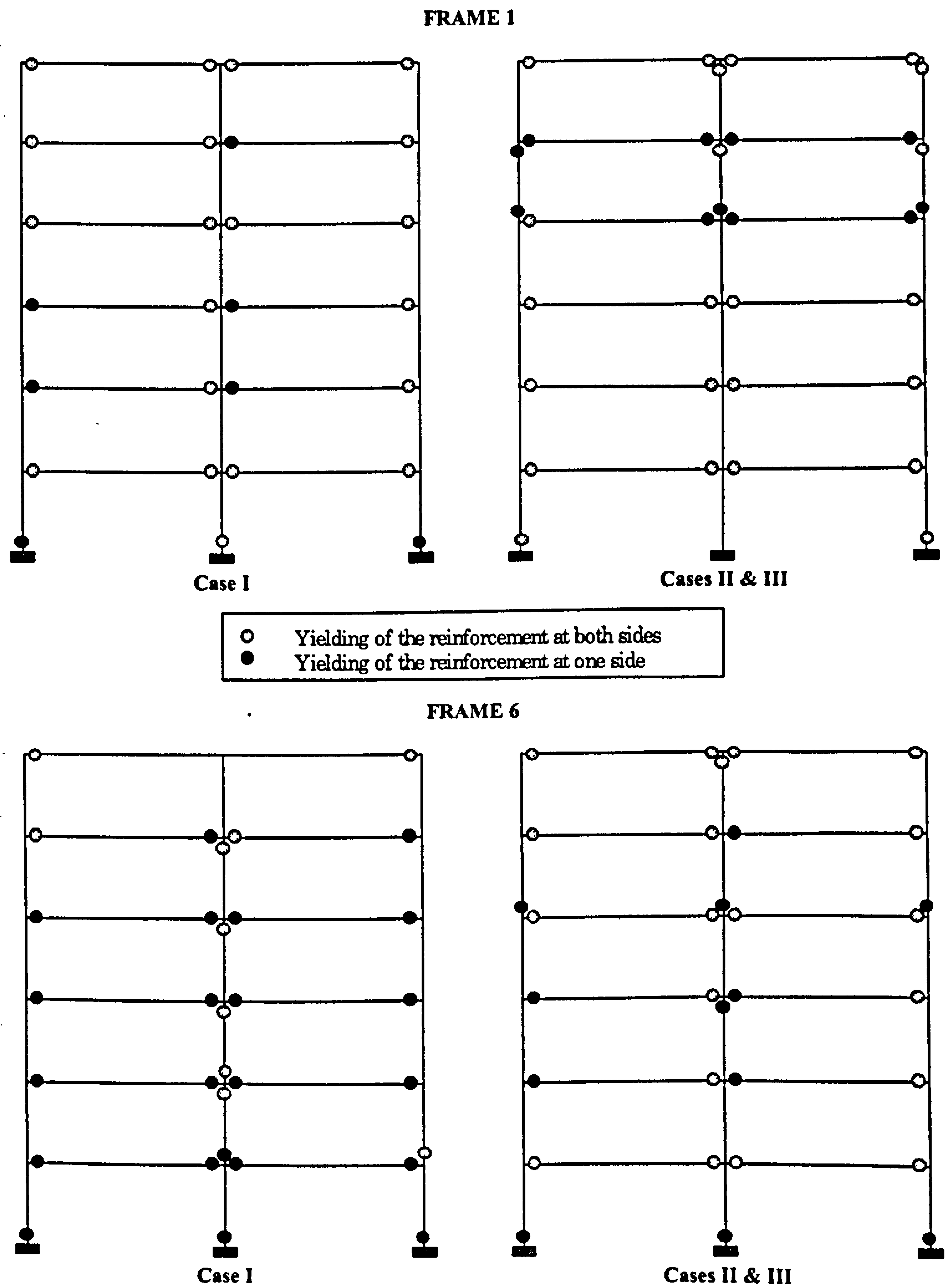
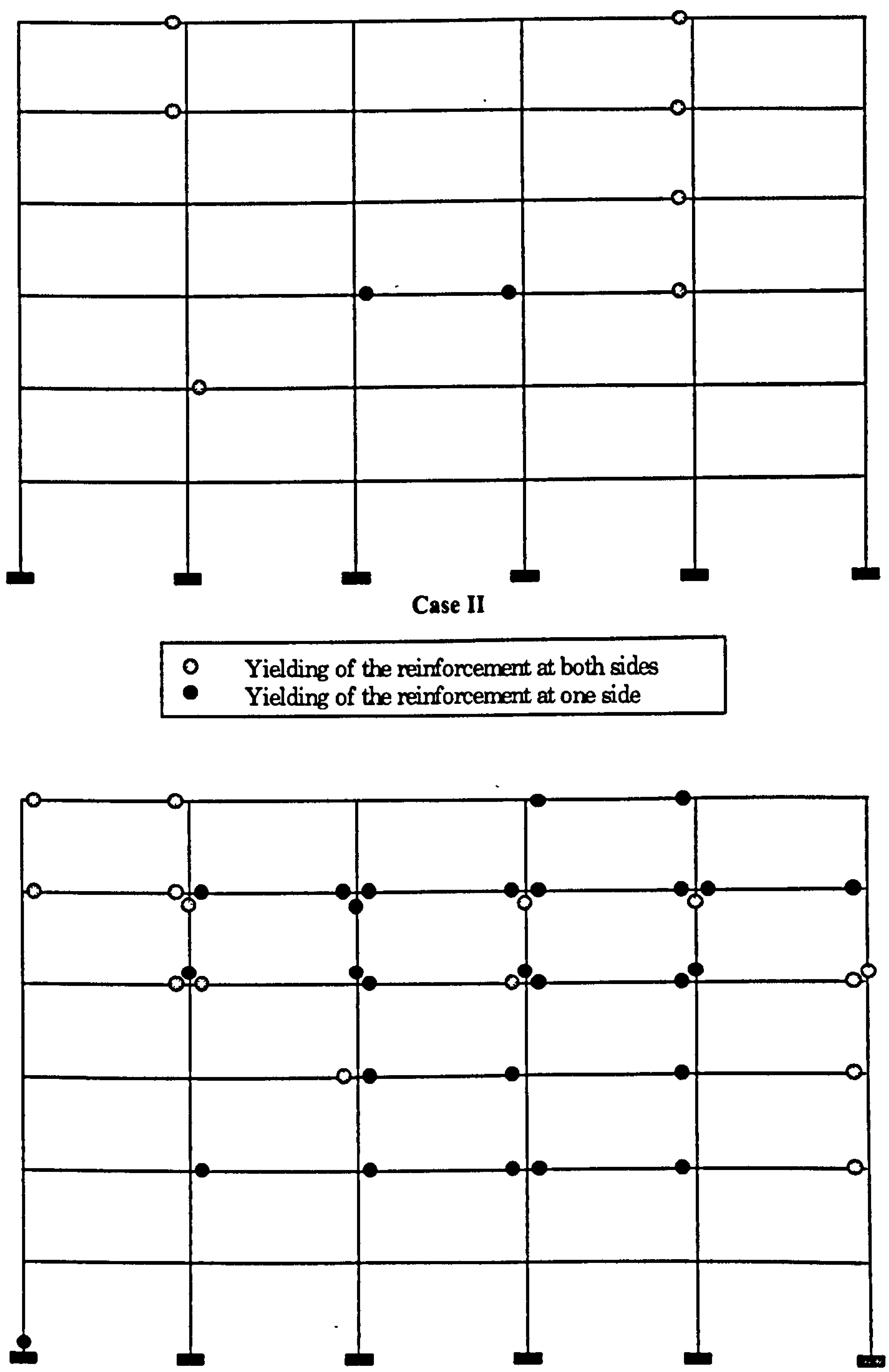


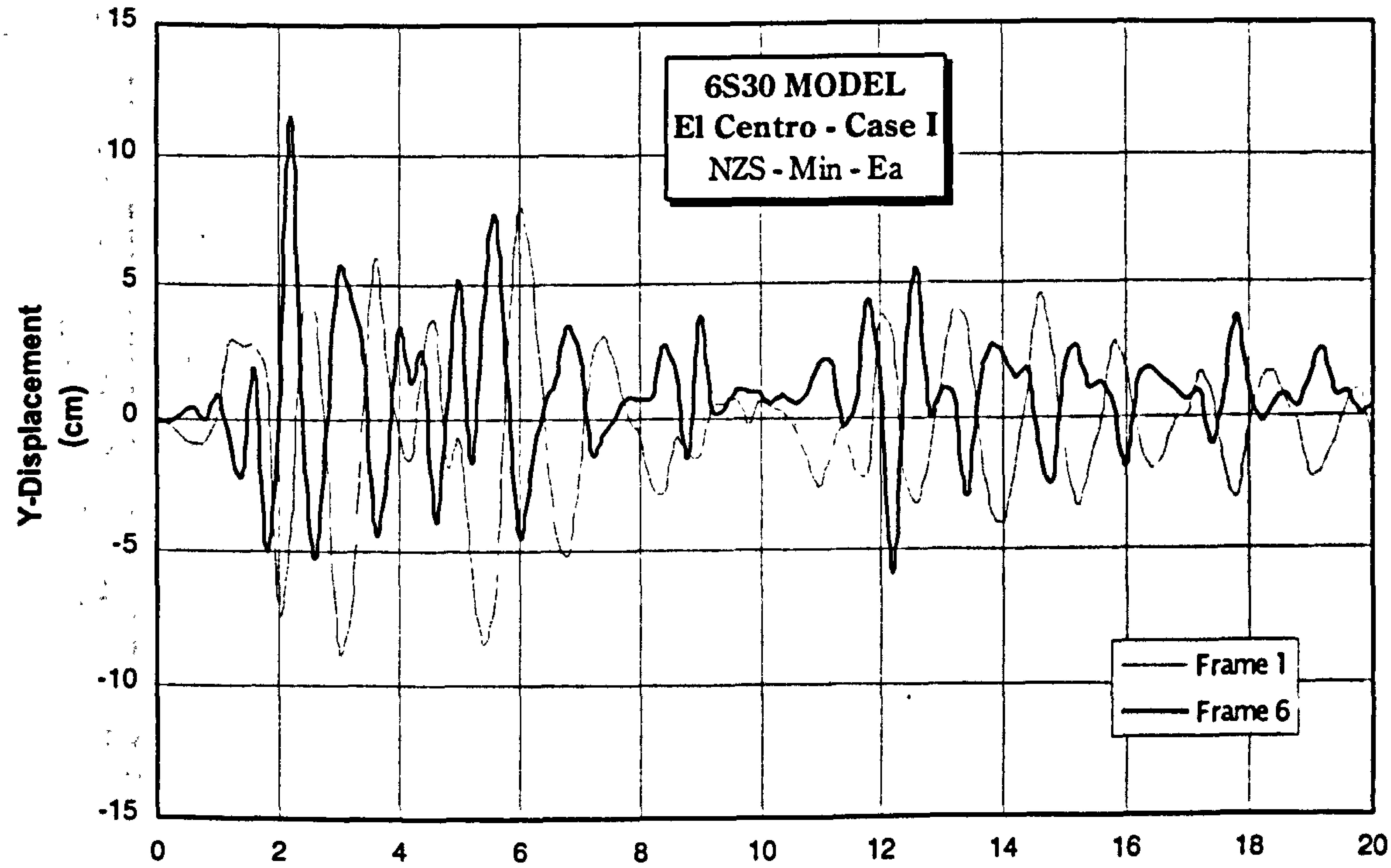
Figure 8.3.8 Influence of the inclusion of transverse elements in the plastic hinge formation of the external lateral frames (frames 1 & 6) of the mass-eccentric model 6S30.

FRAME 7



Case III

Figure 8.3.9 Influence of the inclusion of two earthquake components in the plastic hinge formation of the transverse frame 7 of the mass-eccentric model 6ST30.



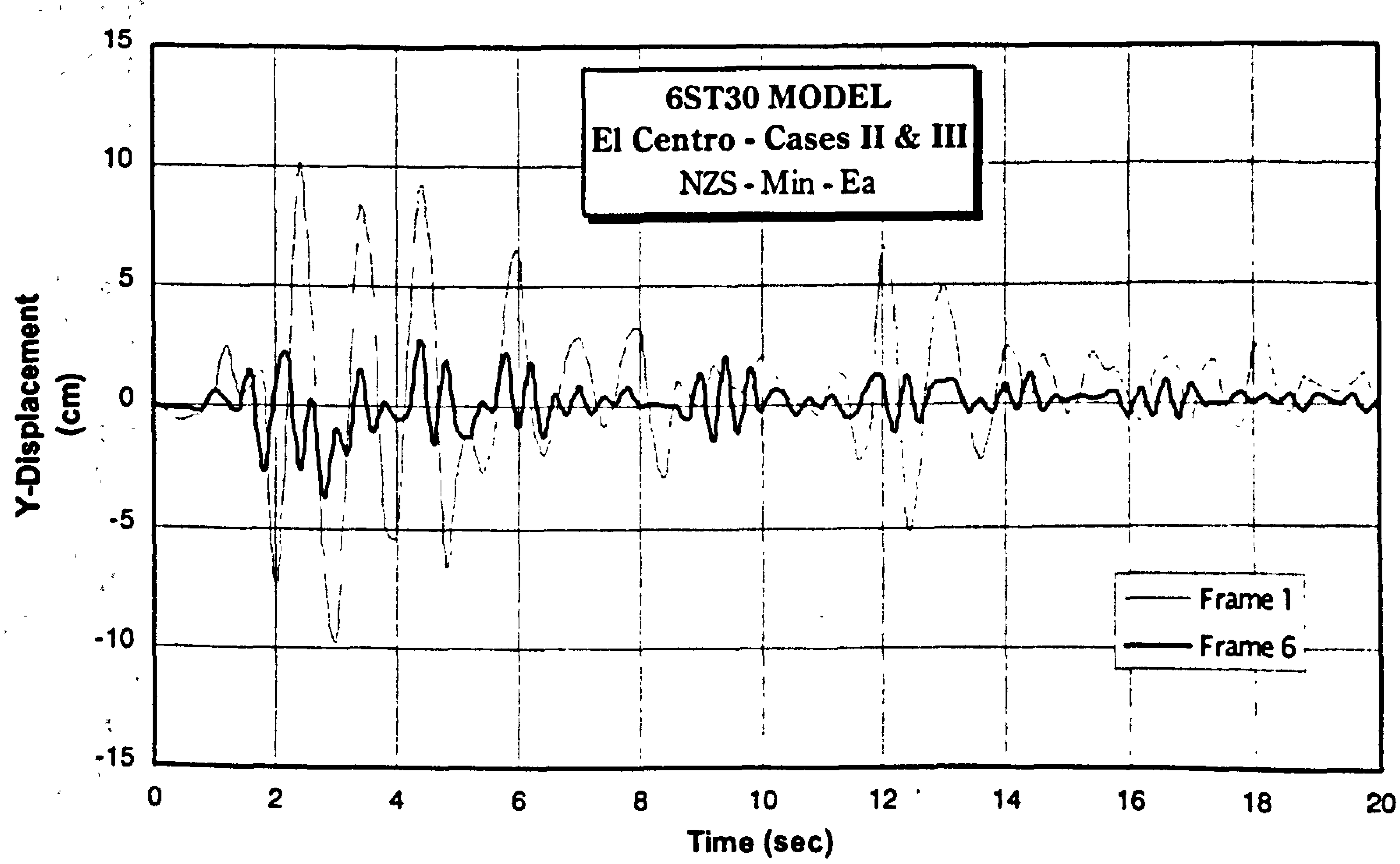


Figure 8.3.10 Influence of the inclusion of transverse elements in the time-history displacement of the external lateral frames (frames 1 & 6) of the mass-eccentric model 6S30.

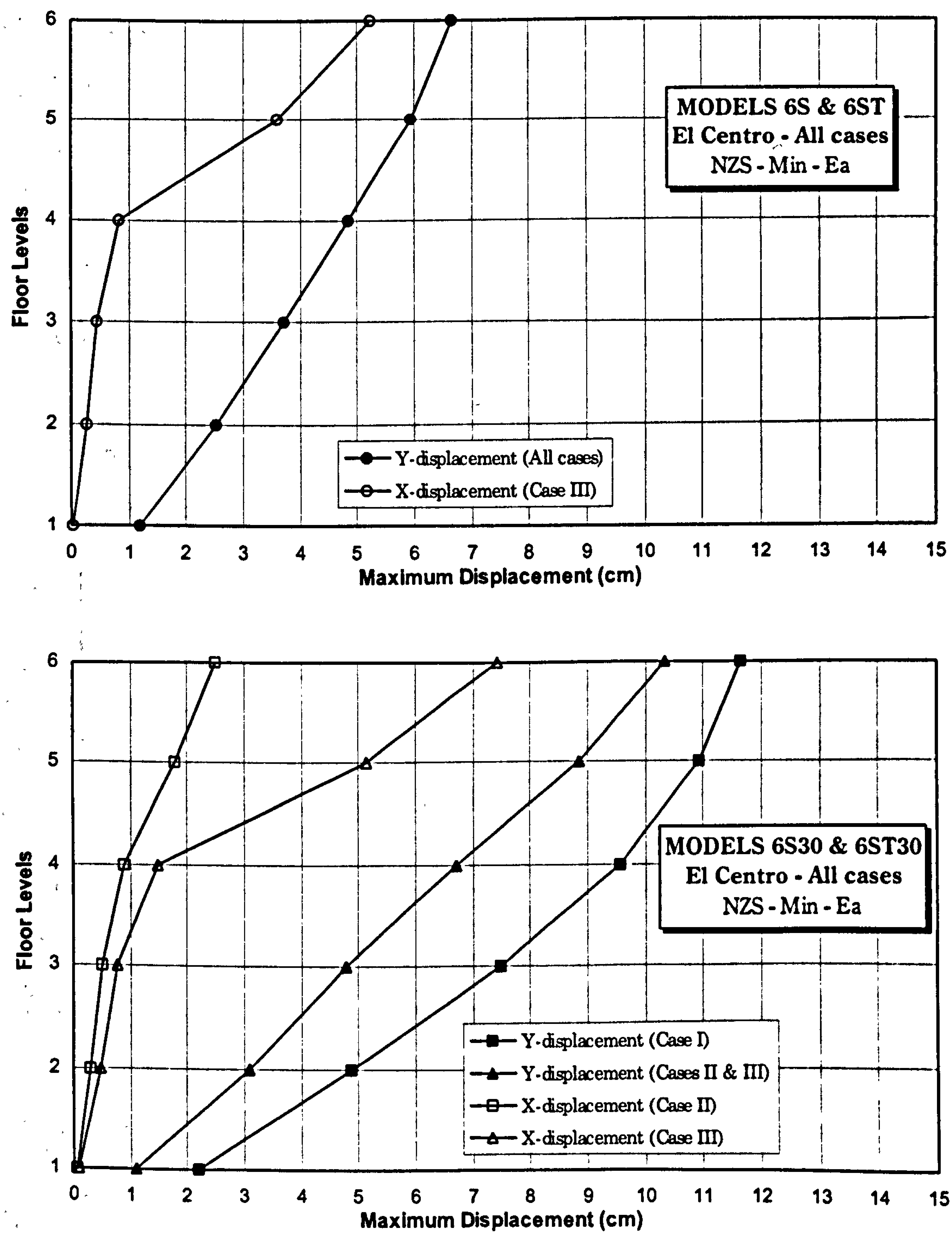


Figure 8.3.11 Maximum lateral and transverse displacement for the reference models 6S & 6ST and for the TU models 6S30 & 6ST30 subjected to different seismic loading conditions.

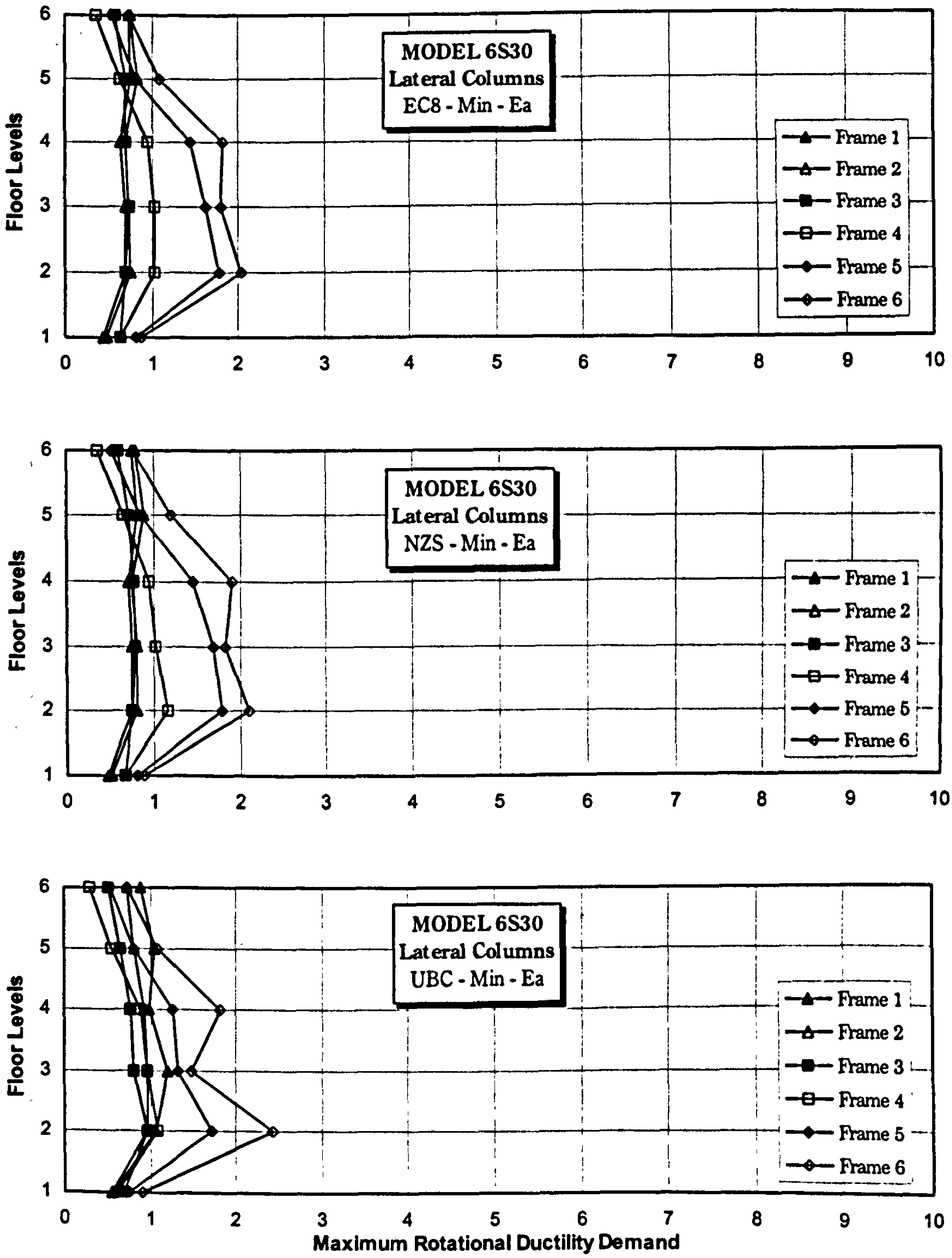


Figure 8.4.1 Maximum rotational ductility demand vs. floor levels for the columns of model 6S30 designed according to the torsional provisions of different seismic codes.

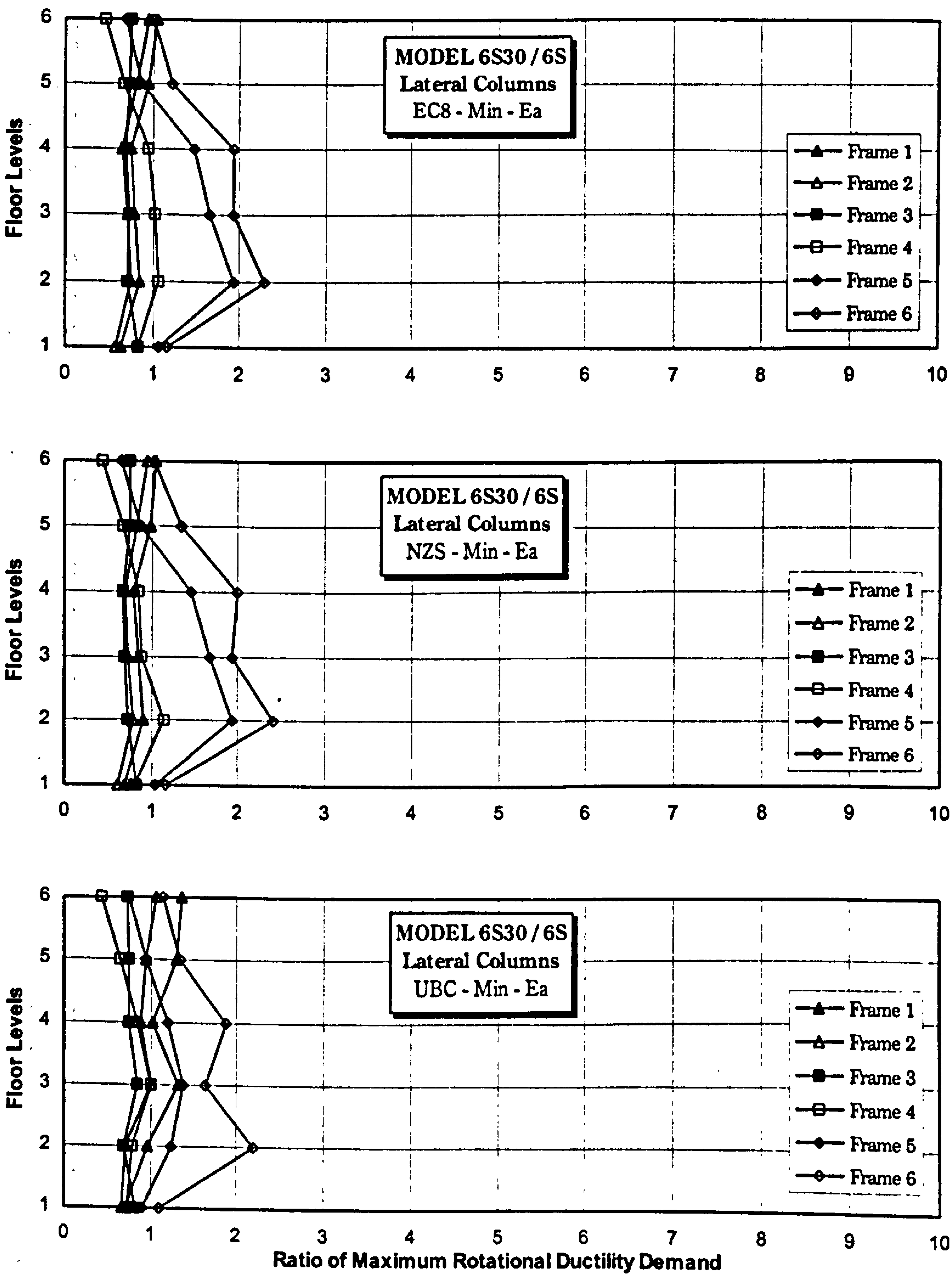


Figure 8.4.2 Ratio of maximum rotational ductility demand vs. floor levels for the columns of model 6S30 / 6S designed according to the torsional provisions of different seismic codes.

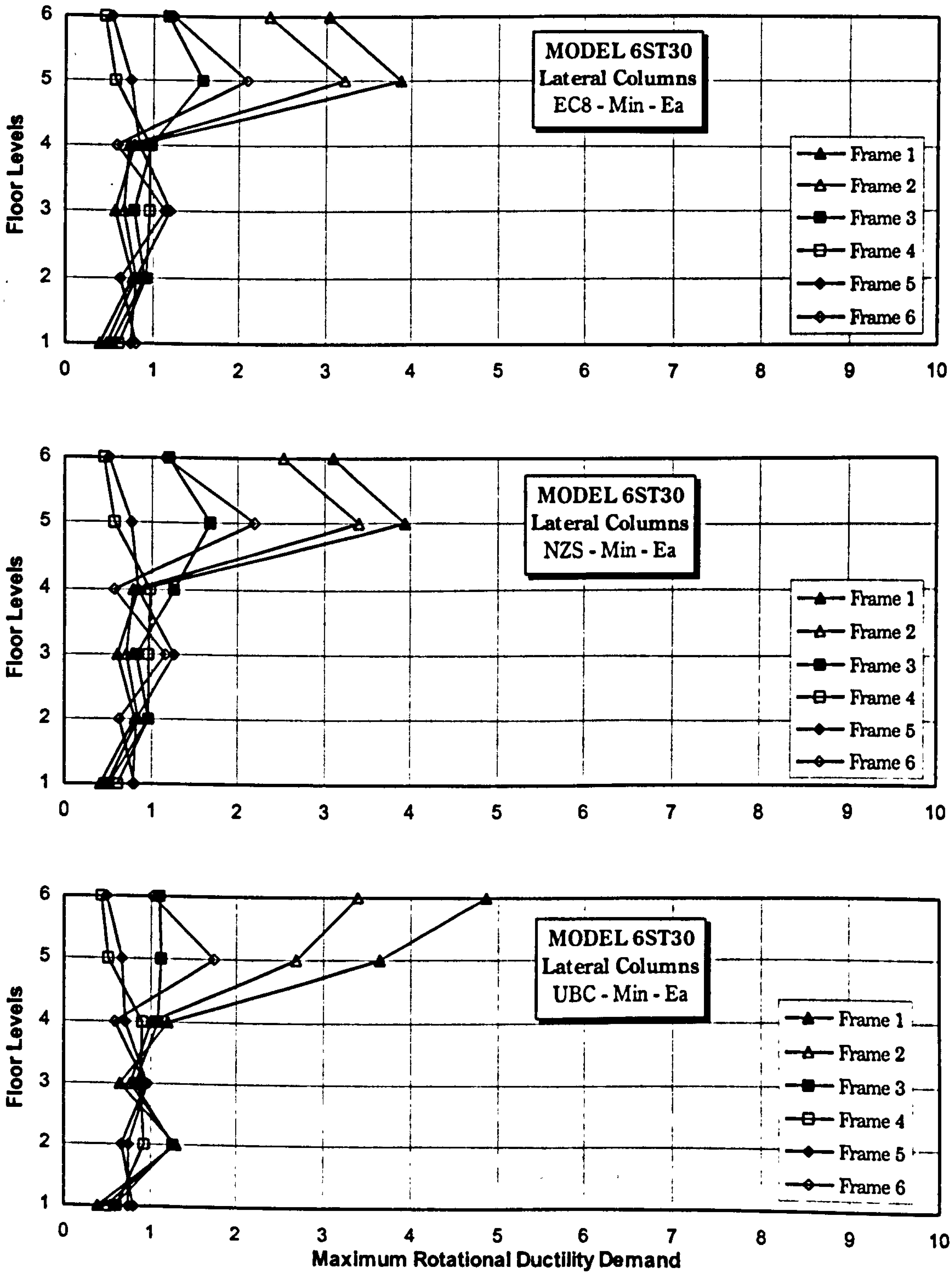


Figure 8.4.3 Maximum rotational ductility demand vs. floor levels for the lateral columns of model 6ST30 designed according to the torsional provisions of different seismic codes.

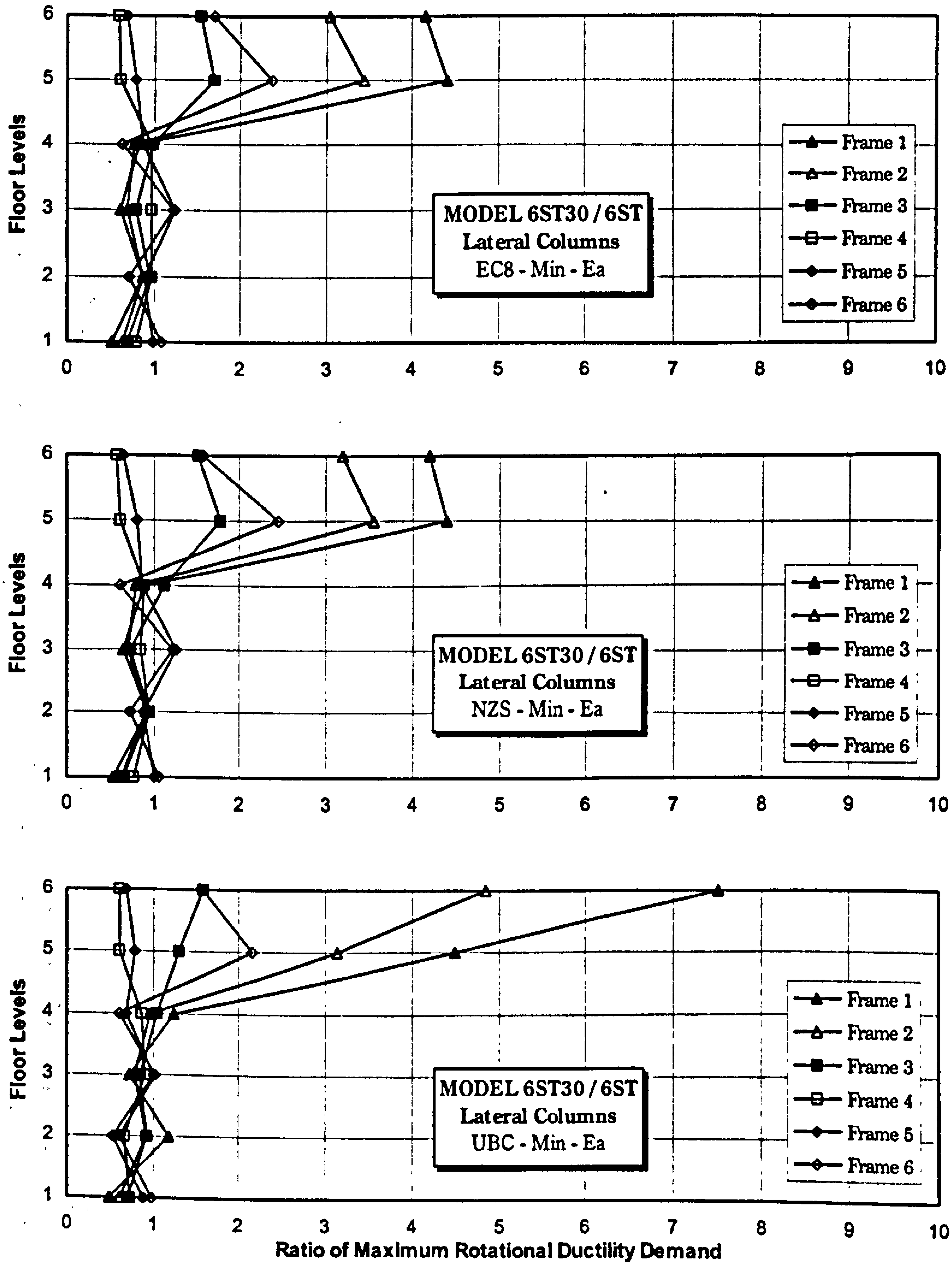


Figure 8.4.4 Ratio of maximum rotational ductility demand vs. floor levels for the lateral columns of model 6ST30 / 6ST designed according to the torsional provisions of different seismic codes.

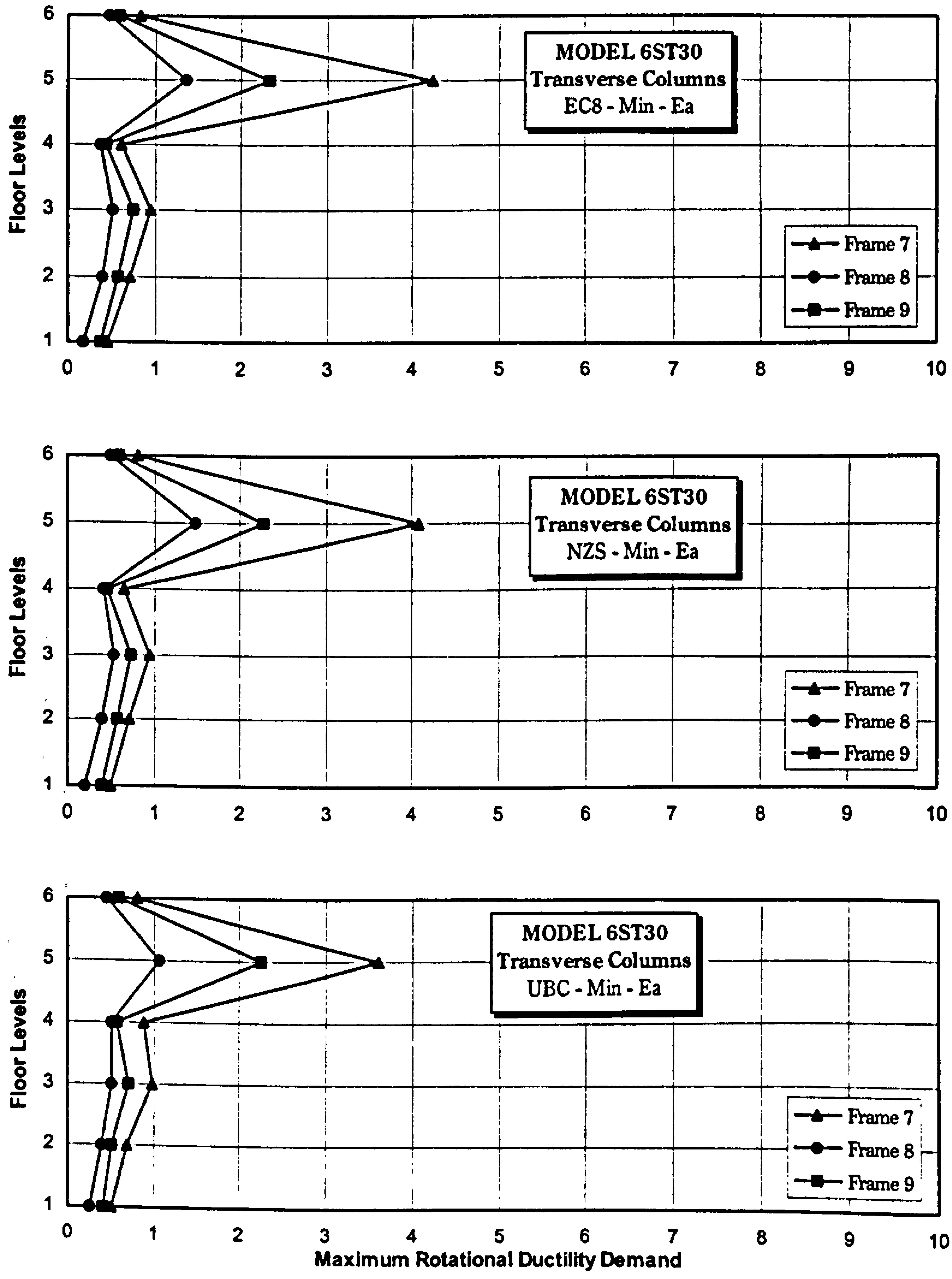


Figure 8.4.5 Maximum rotational ductility demand vs. floor levels for the transverse columns of model 6ST30 designed according to the torsional provisions of different seismic codes.

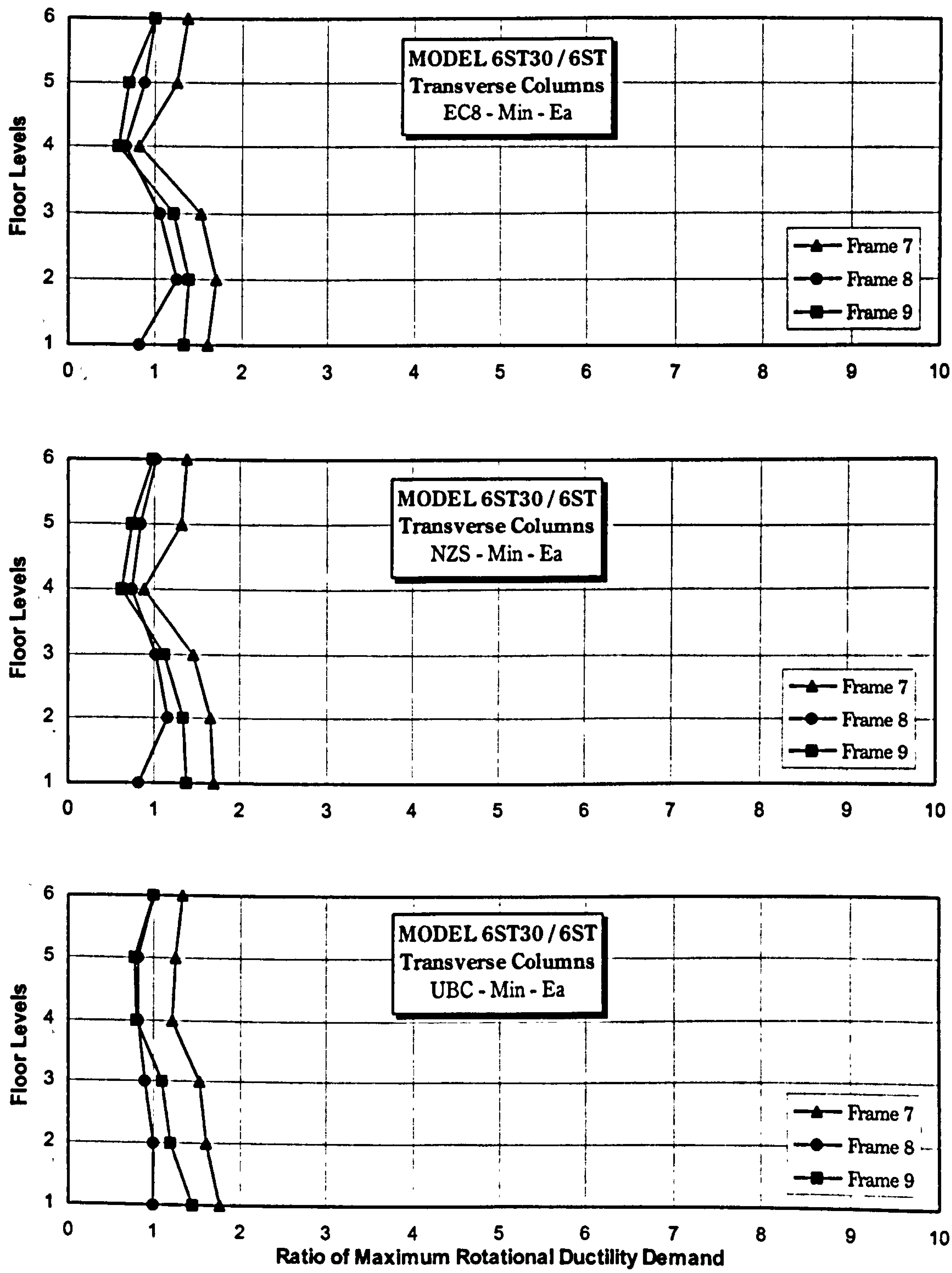


Figure 8.4.6 Ratio of maximum rotational ductility demand vs. floor levels for the transverse columns of model 6ST30 / 6ST designed according to the torsional provisions of different seismic codes.

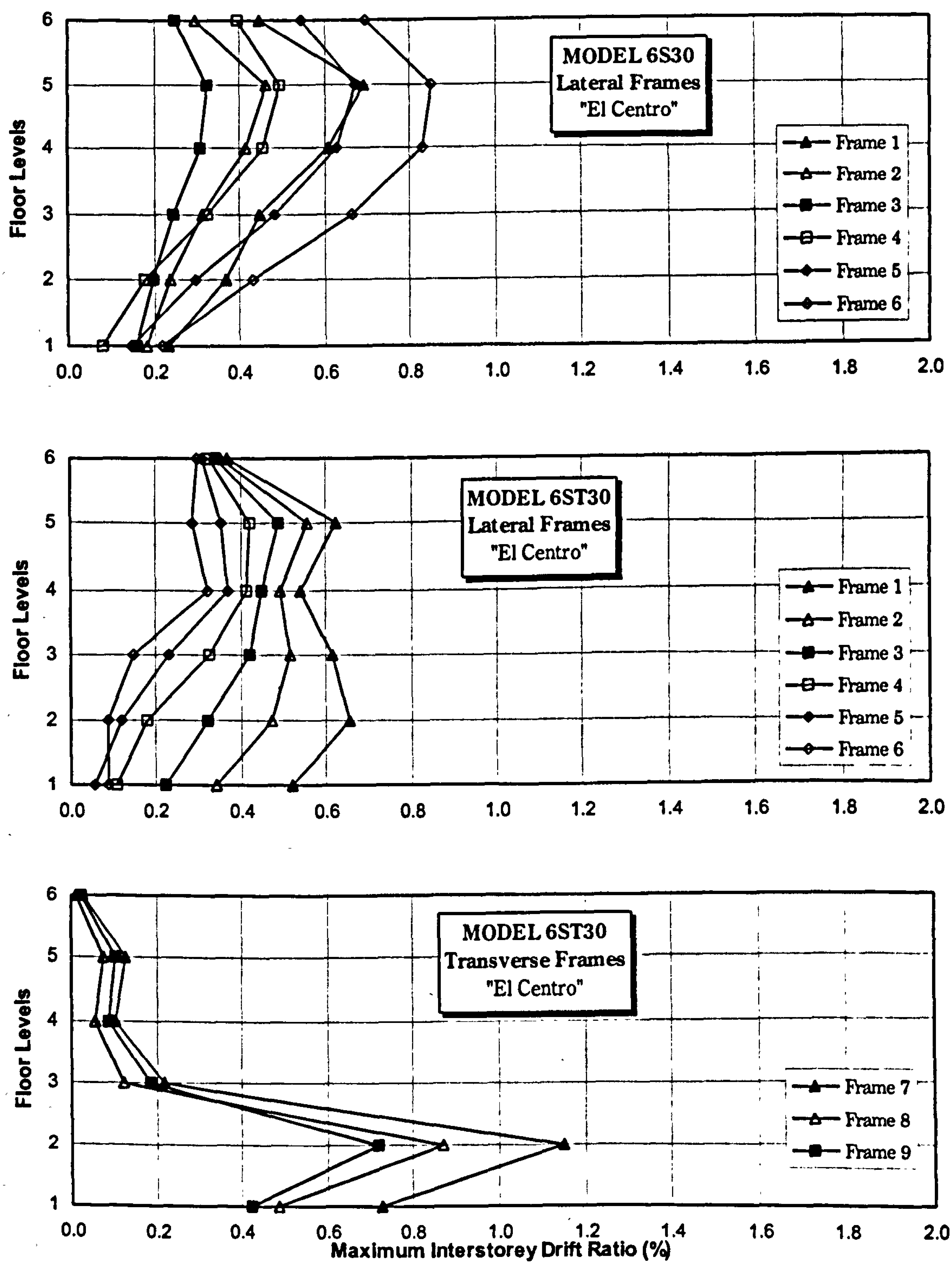
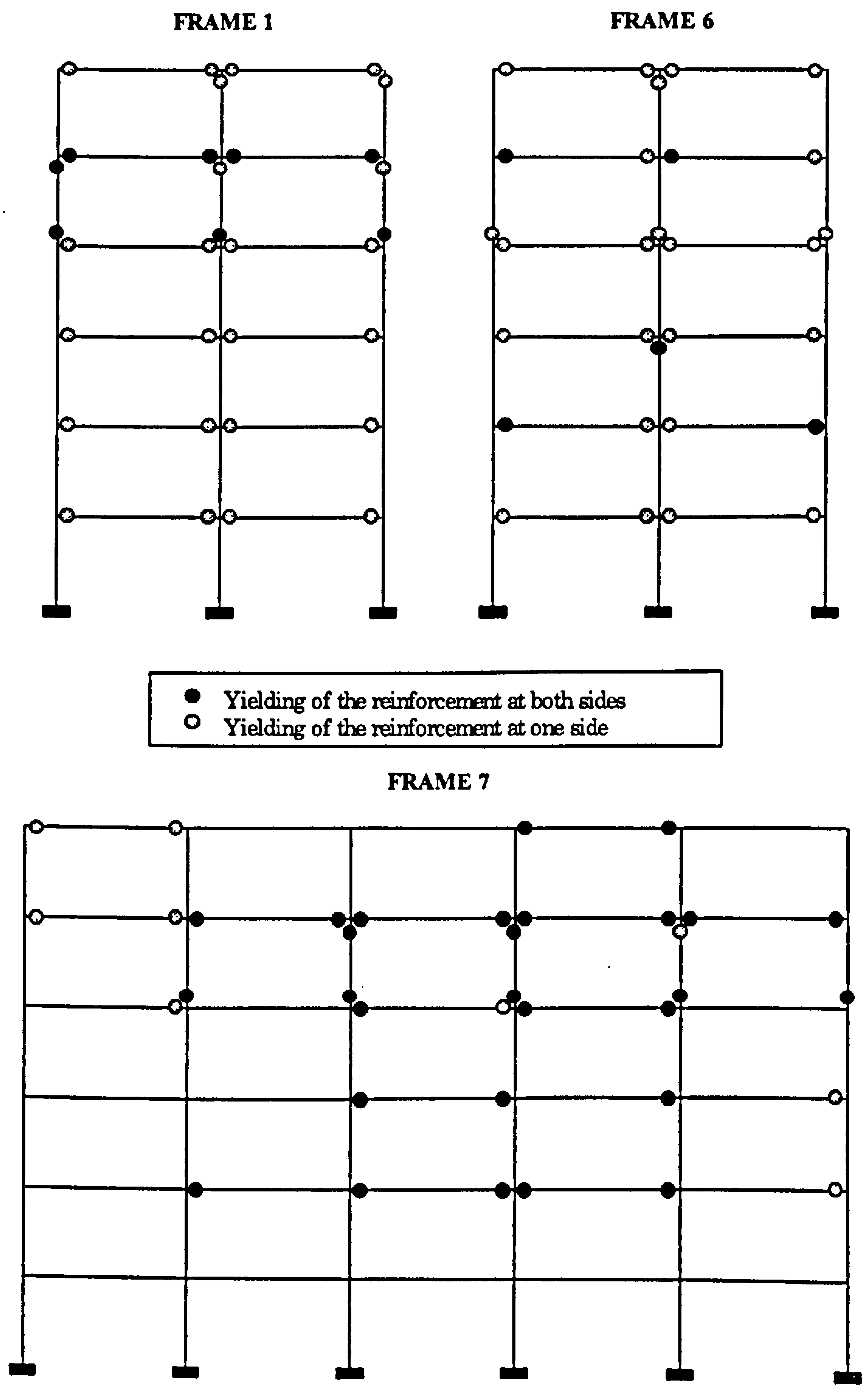


Figure 8.4.7 Maximum interstorey drift ratio for the lateral frames of model 6S30 and for the lateral and transverse frames of model 6ST30 subjected to the El Centro earthquake record.



EC8 Seismic Code

Figure 8.4.8 Plastic hinge formation for the lateral frames 1 and 6 and for the transverse frame 7 of model 6ST30 subjected to the El Centro earthquake record and designed to the EC8 torsional provisions.

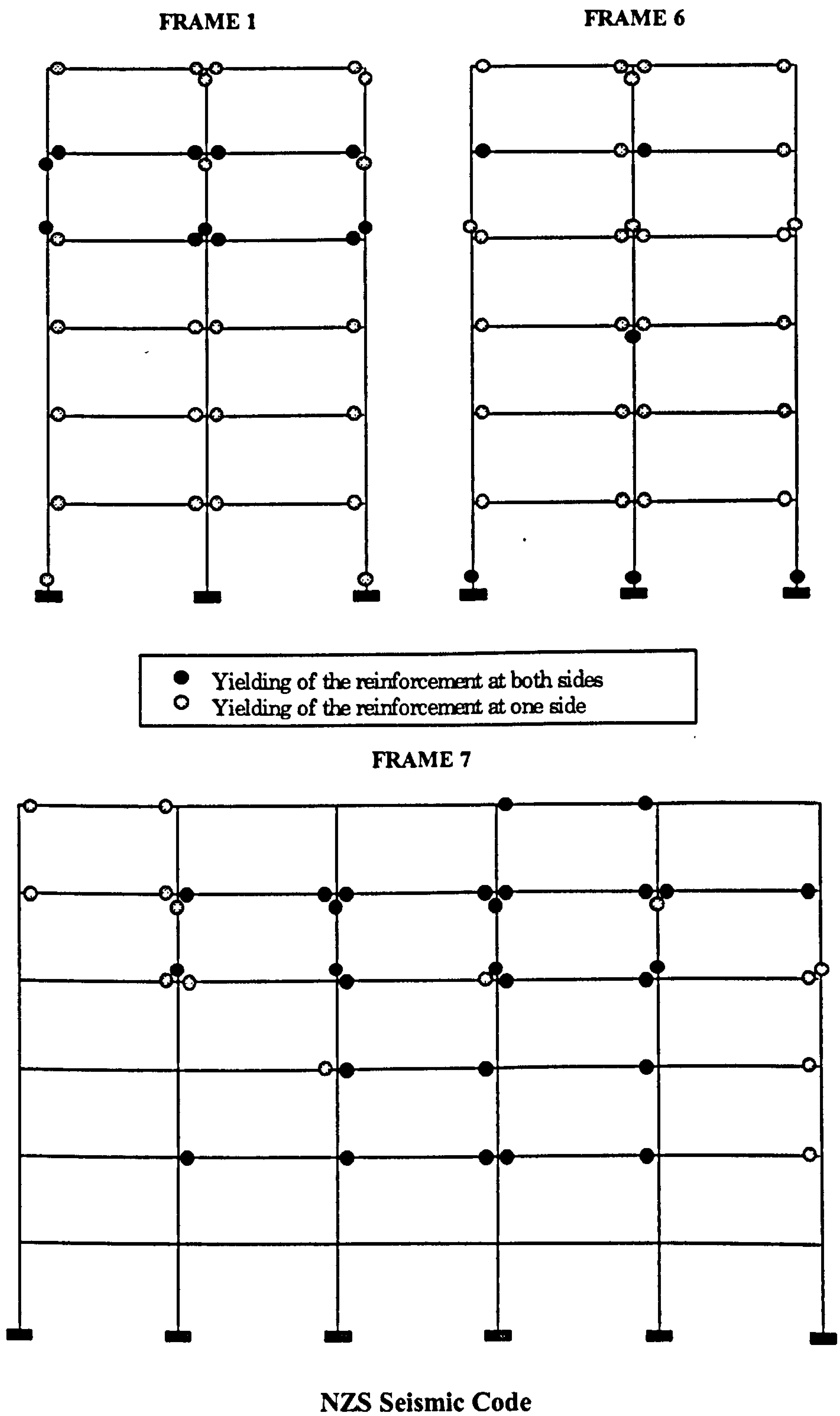
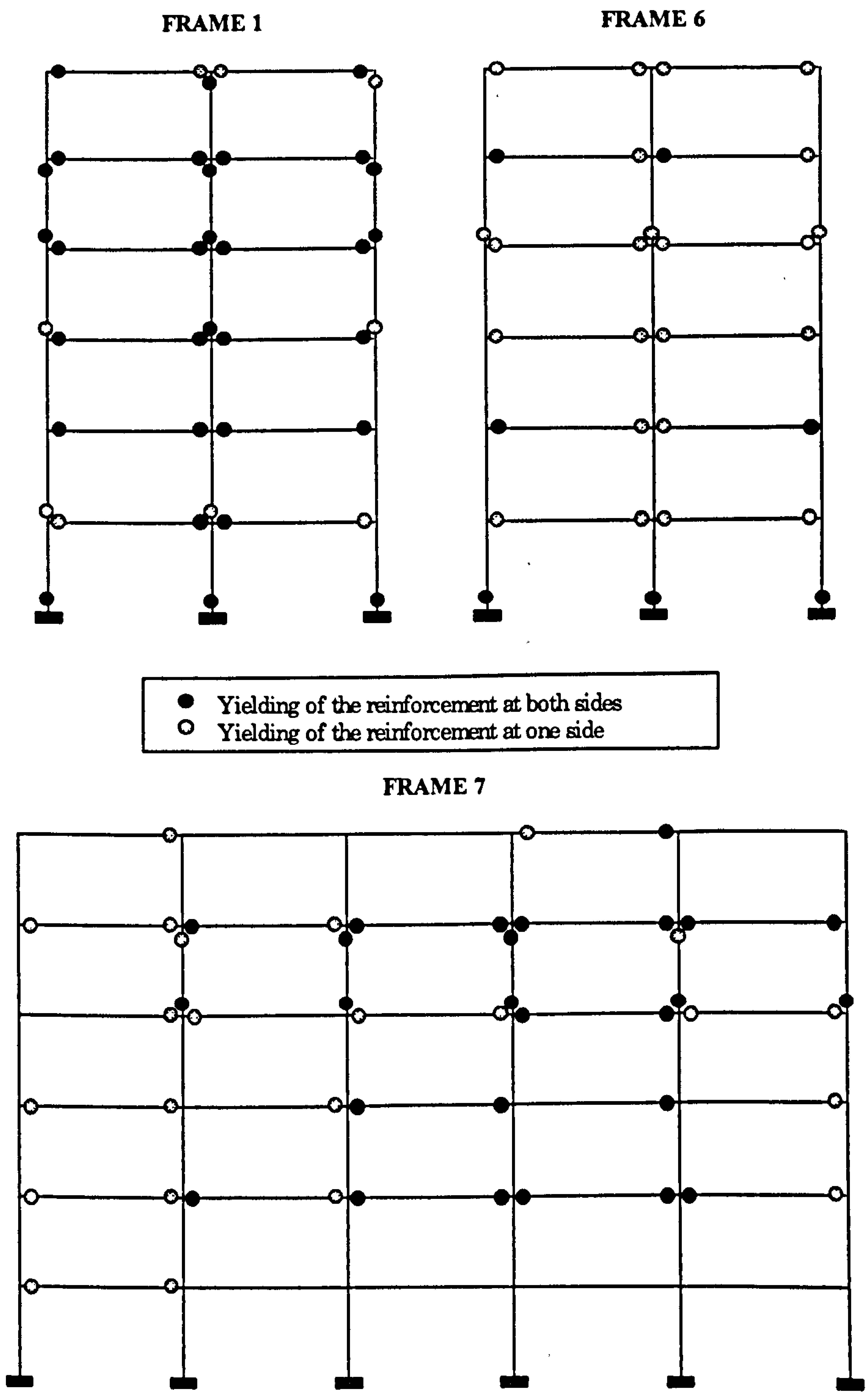


Figure 8.4.9 Plastic hinge formation for the lateral frames 1 and 6 and for the transverse frame 7 of model 6ST30 subjected to the El Centro earthquake record and designed to the NZS torsional provisions.



UBC Seismic Code

Figure 8.4.10 Plastic hinge formation for the lateral frames 1 and 6 and for the transverse frame 7 of model 6ST30 subjected to the El Centro earthquake record and designed to the UBC torsional provisions.

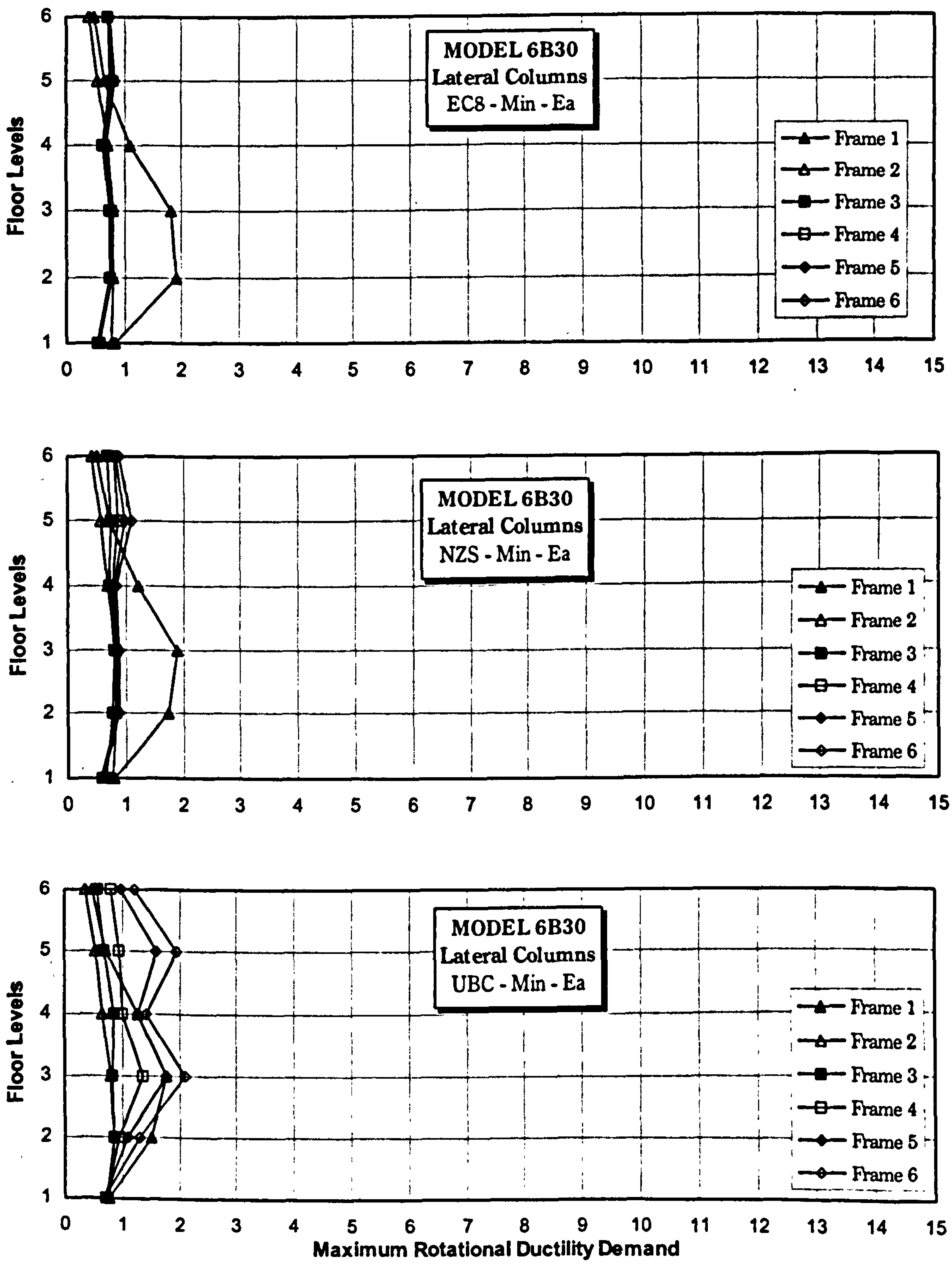


Figure 8.4.11 Maximum rotational ductility demand vs. floor levels for the columns of model 6B30 designed according to the torsional provisions of different seismic codes.

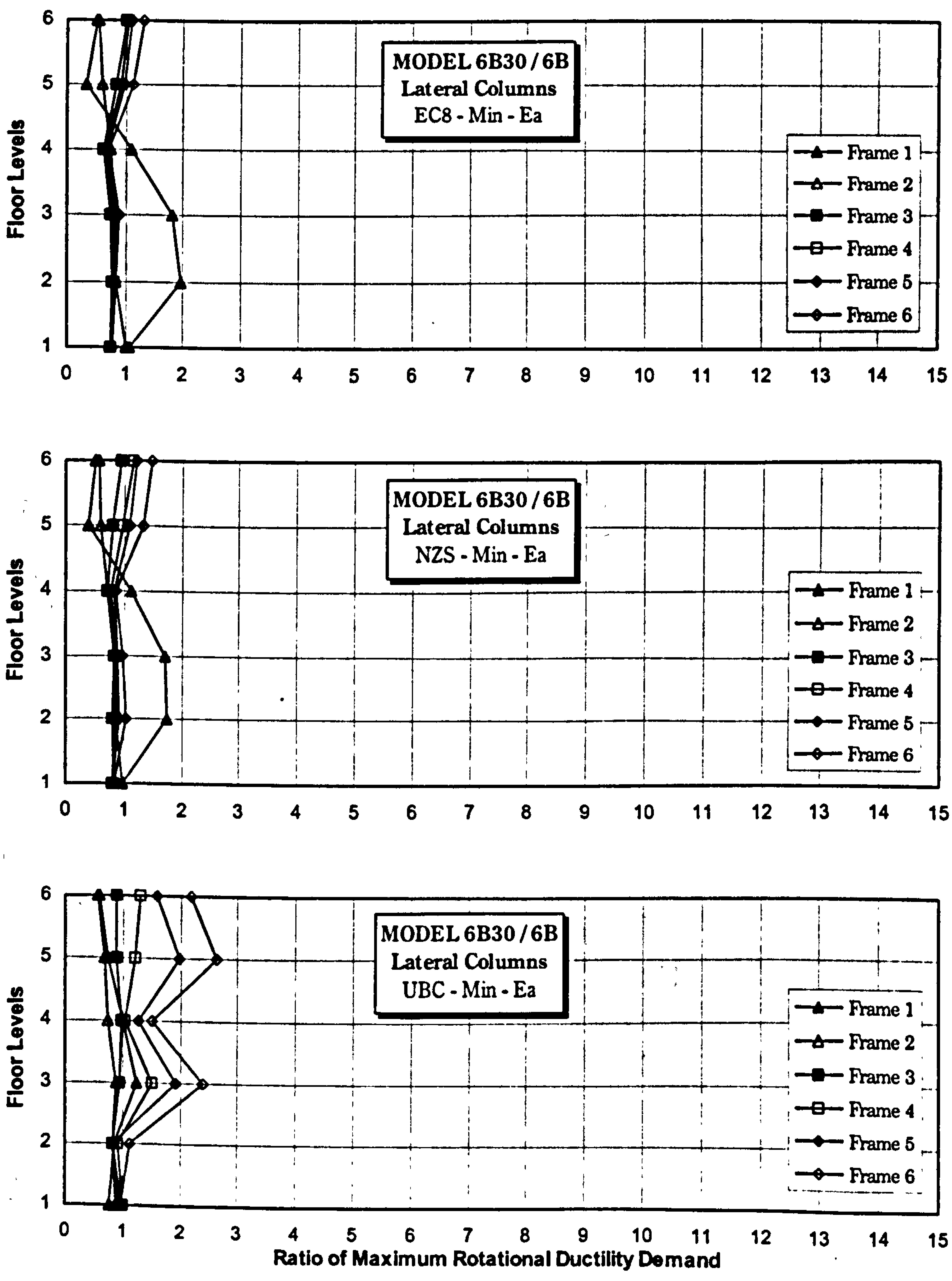


Figure 8.4.12 Ratio of maximum rotational ductility demand vs. floor levels for the columns of model 6B30 / 6B designed according to the torsional provisions of different seismic codes.

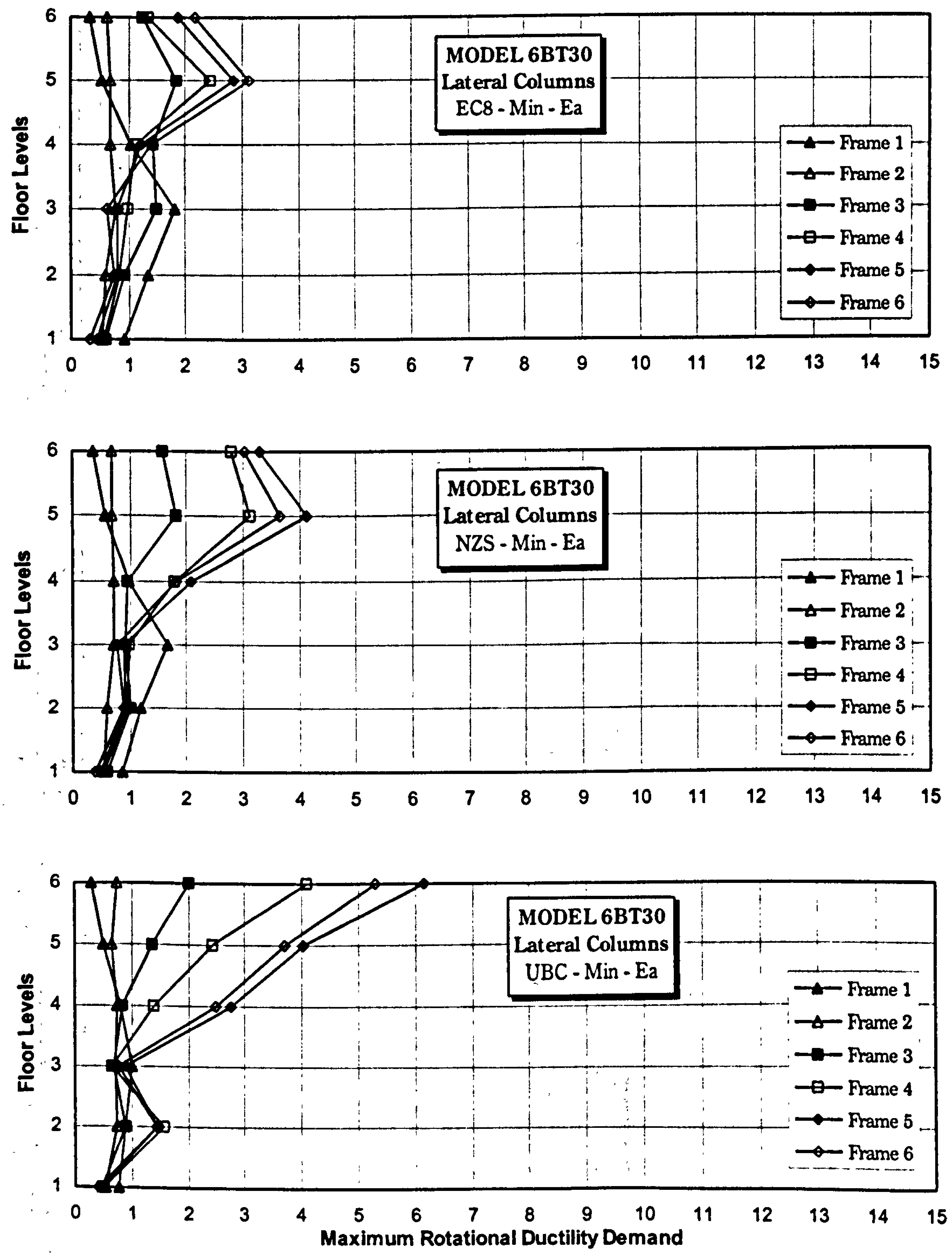


Figure 8.4.13 Maximum rotational ductility demand vs. floor levels for the lateral columns of model 6BT30 designed according to the torsional provisions of different seismic codes.

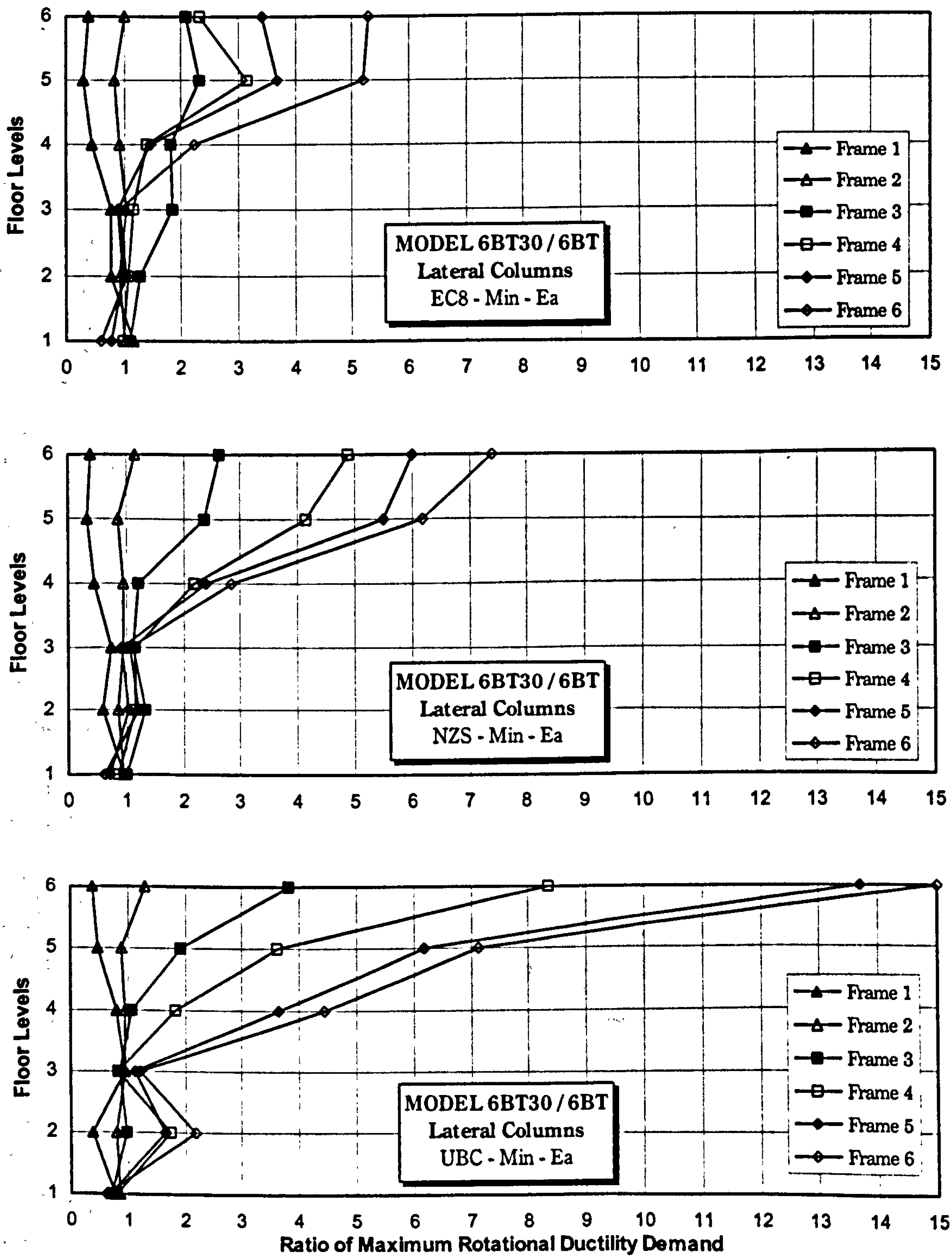


Figure 8.4.14 Ratio of maximum rotational ductility demand vs. floor levels for the lateral columns of model 6BT30 / 6BT designed according to the torsional provisions of different seismic codes.

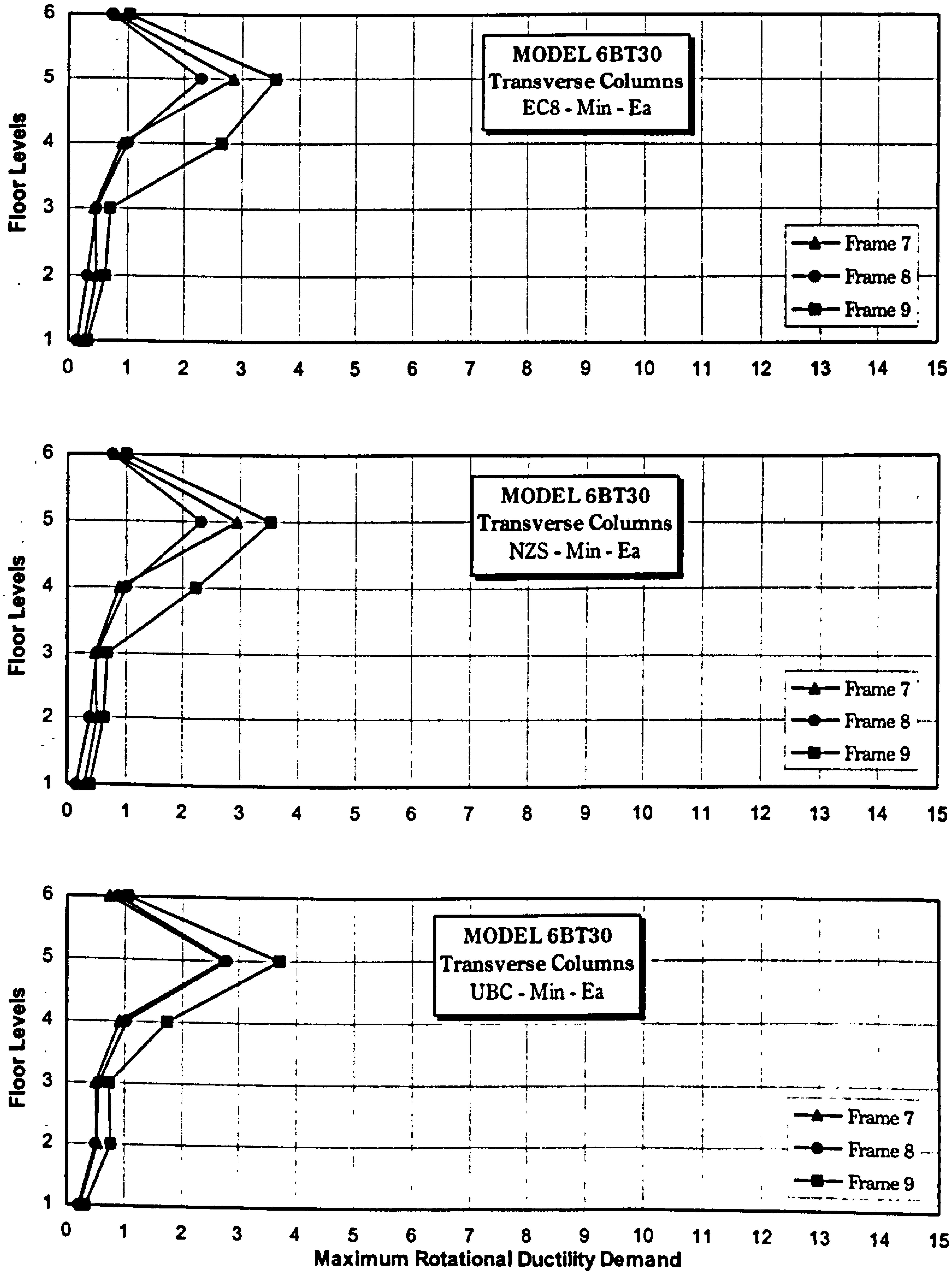


Figure 8.4.15 Maximum rotational ductility demand vs. floor levels for the transverse columns of model 6BT30 designed according to the torsional provisions of different seismic codes.

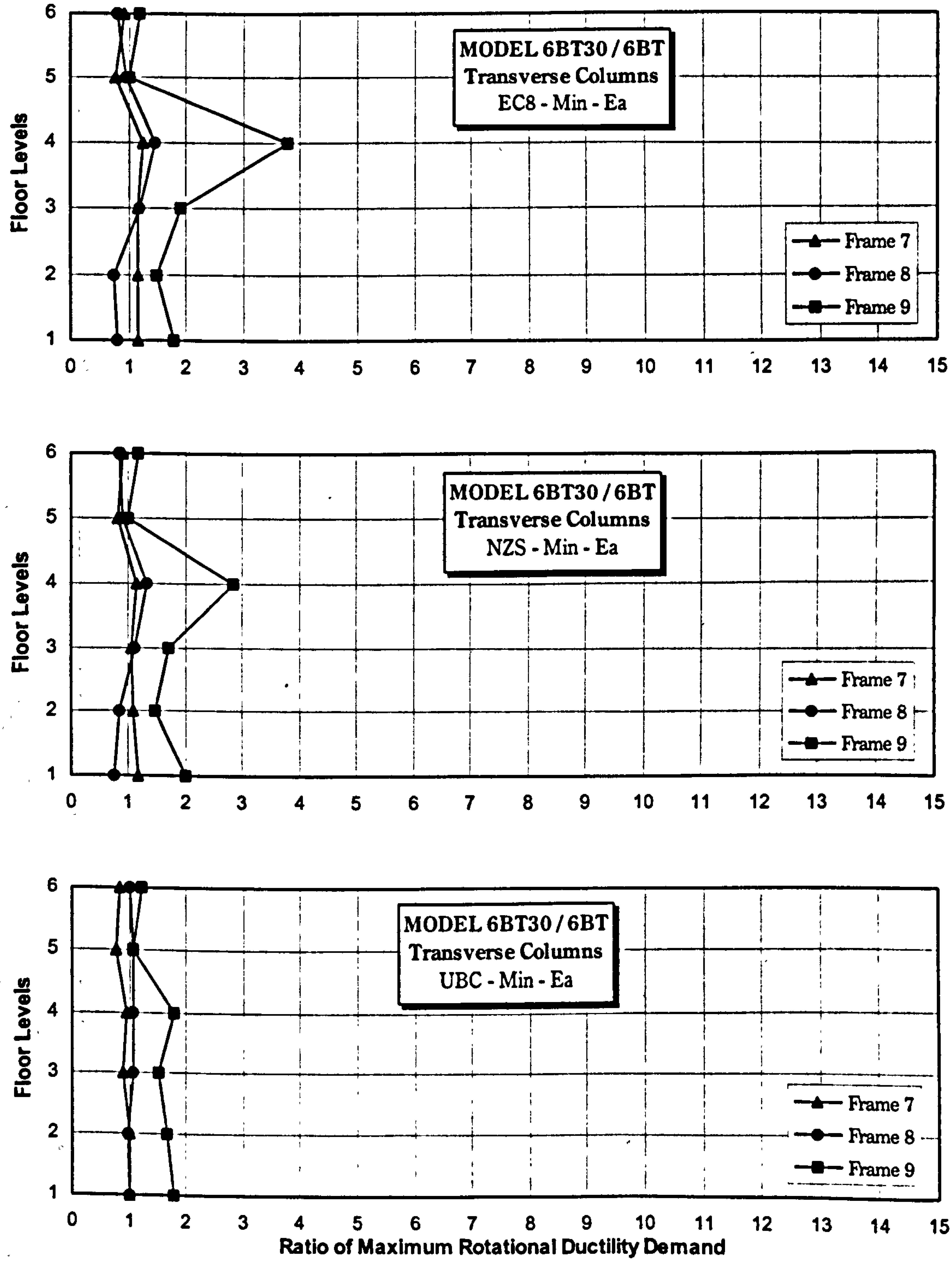


Figure 8.4.16 Ratio of maximum rotational ductility demand vs. floor levels for the transverse columns of model 6BT30 / 6BT designed according to the torsional provisions of different seismic codes.

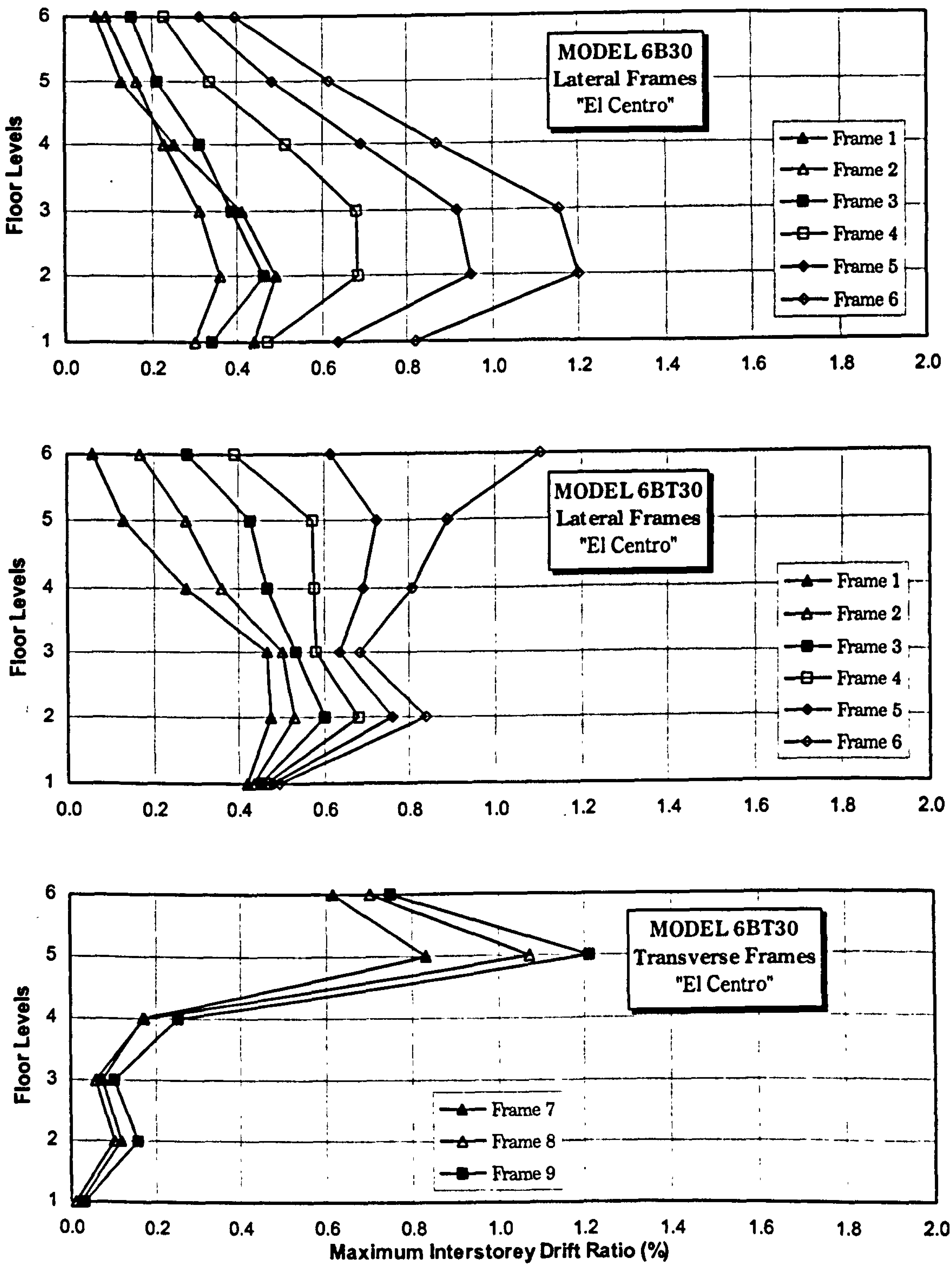
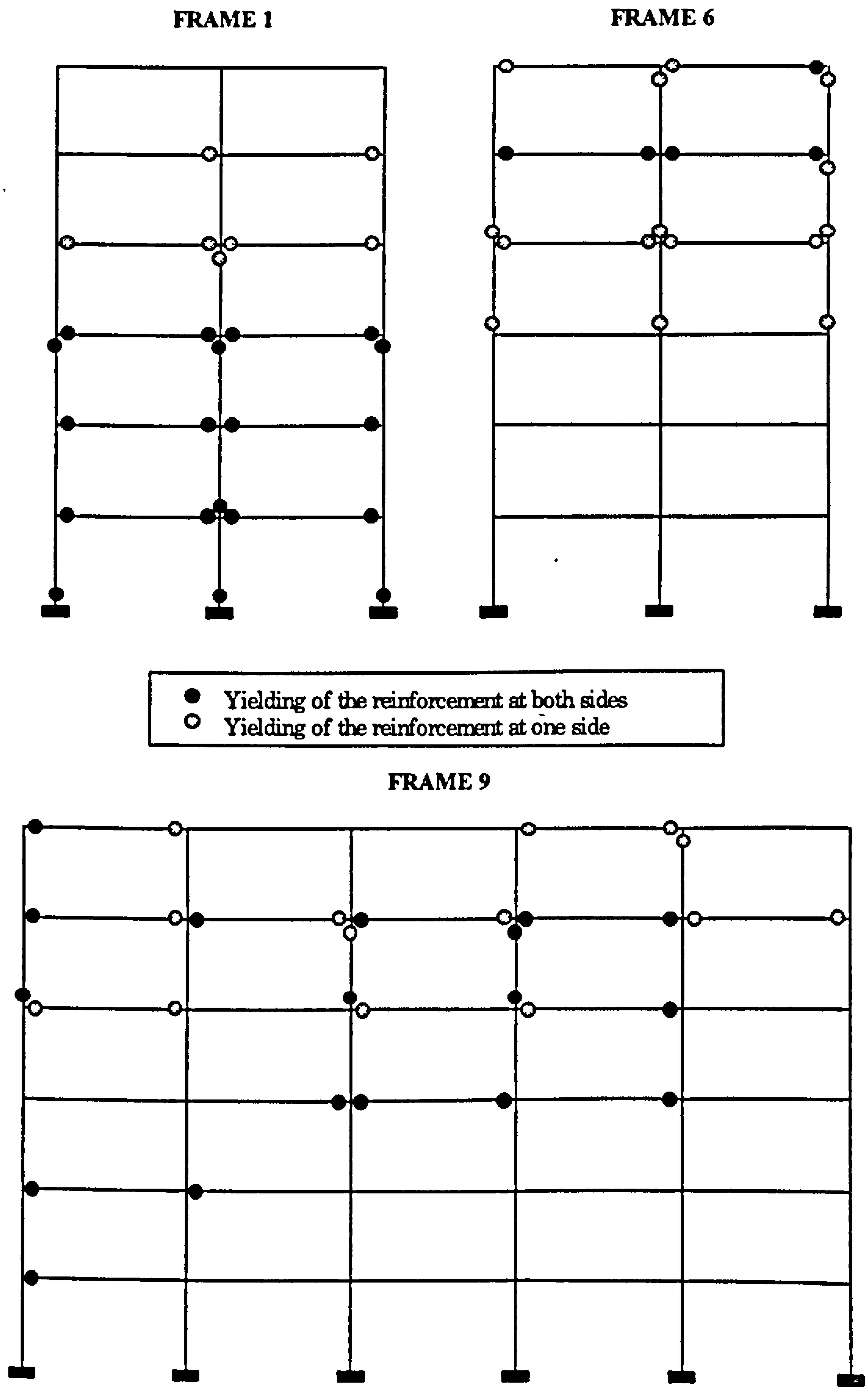
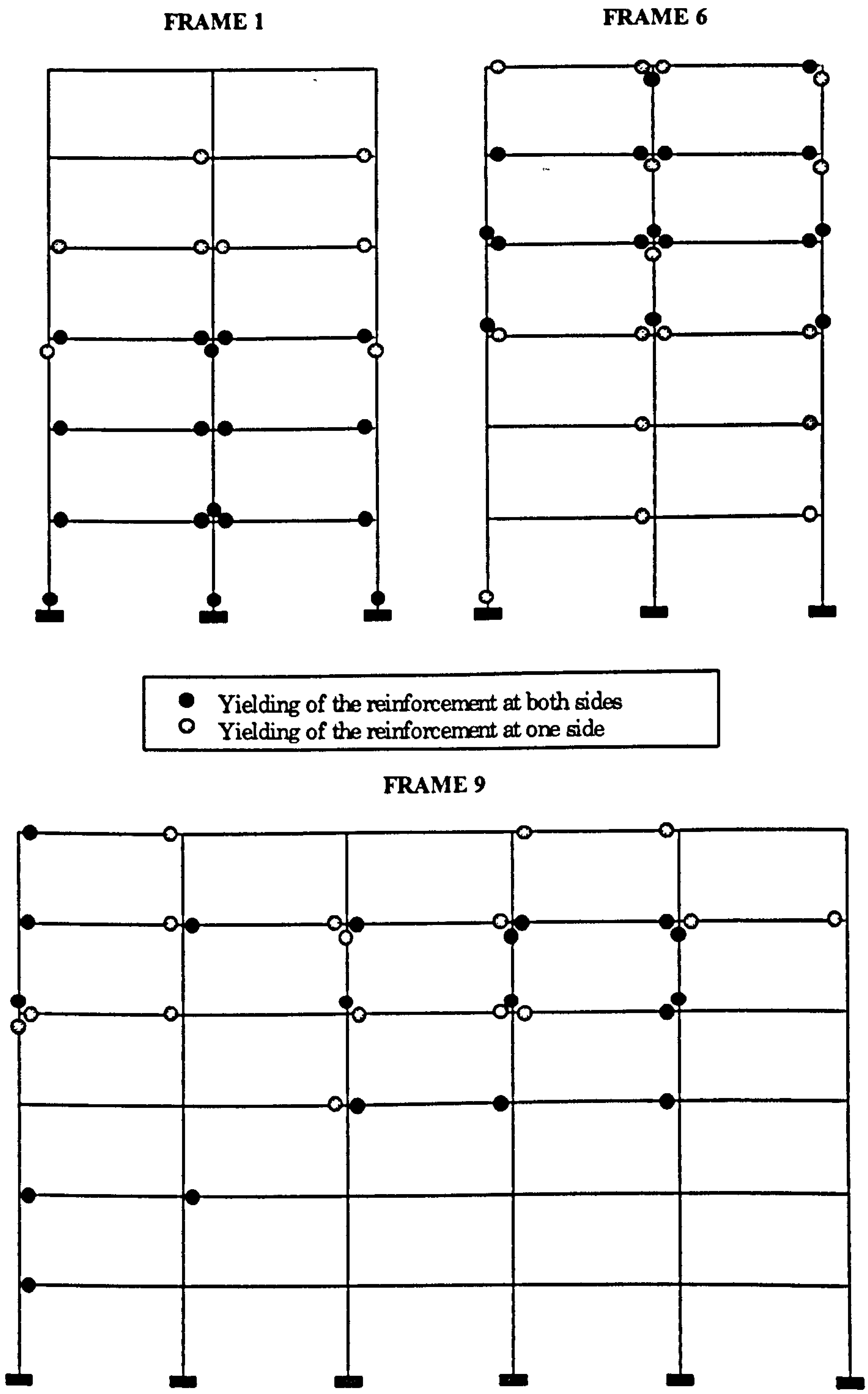


Figure 8.4.17 Maximum interstorey drift ratio for the lateral frames of model 6B30 and for the lateral and transverse frames of model 6BT30 subjected to the El Centro earthquake record.



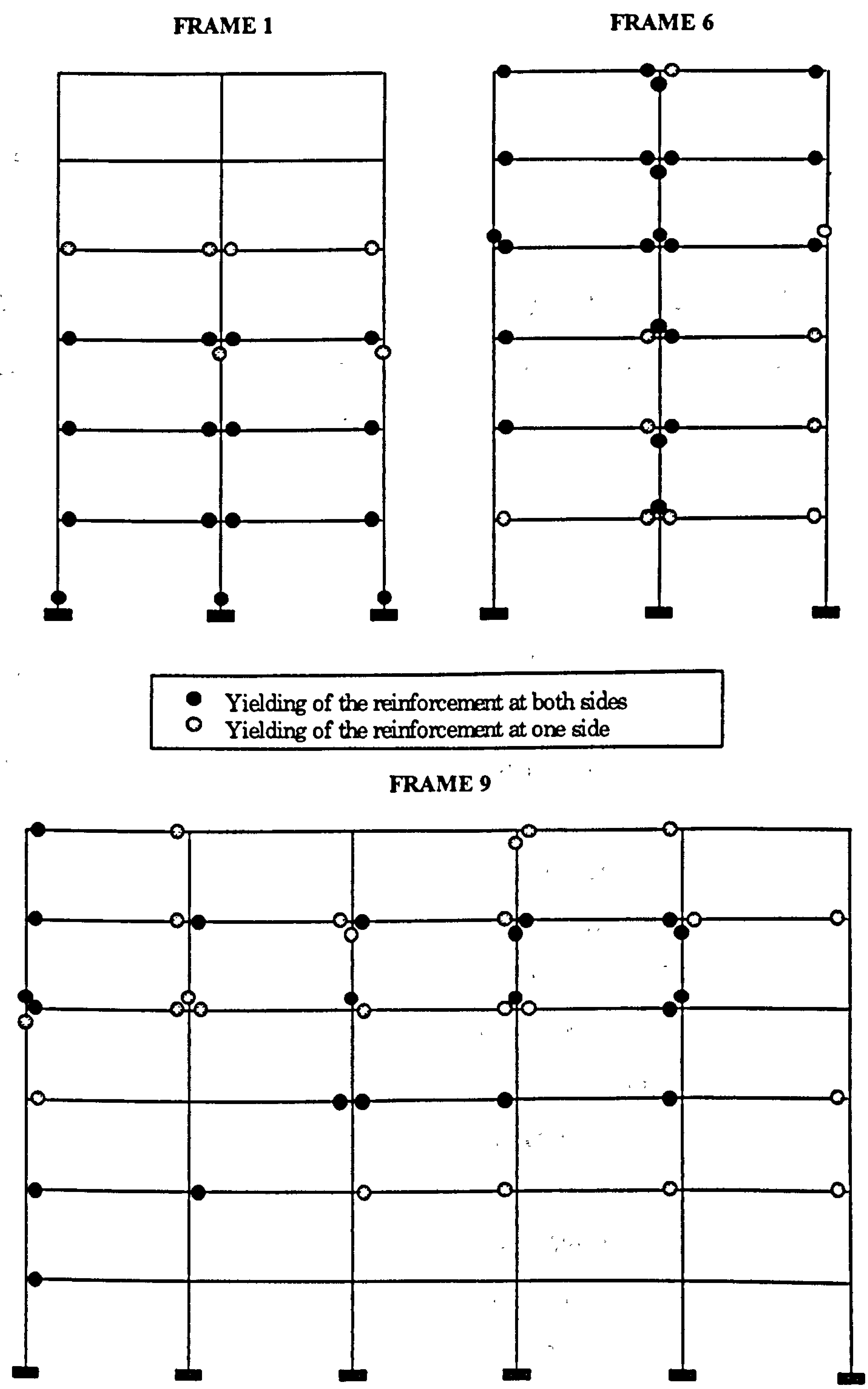
EC8 Seismic Code

Figure 8.4.18 Plastic hinge formation for the lateral frames 1 and 6 and for the transverse frame 7 of model 6BT30 subjected to the El Centro earthquake record and designed to the EC8 torsional provisions.



NZS Seismic Code

Figure 8.4.19 Plastic hinge formation for the lateral frames 1 and 6 and for the transverse frame 7 of model 6BT30 subjected to the El Centro earthquake record and designed to the NZS torsional provisions.



UBC Seismic Code

Figure 8.4.20 Plastic hinge formation for the lateral frames 1 and 6 and for the transverse frame 7 of model 6BT30 subjected to the El Centro earthquake record and designed to the UBC torsional provisions.

CHAPTER 9

EVALUATION OF A RECENTLY PROPOSED EQUIVALENT STATIC FORCE PROCEDURE

9.1 INTRODUCTION

The coupled torsional and translational inelastic dynamic response of asymmetric (TU) structures is responsible for the additional ductility demand and deformation leading to increased damage in critical frames. Many seismic codes endeavour to control and limit the additional seismic response of the structures undergoing rotational motions by the stipulation of static torsional provisions. Throughout the investigations conducted in Chapters 5 – 8, the strength distribution of the various models examined has been determined by adopting the static torsional provisions of the EC8, NZS and UBC seismic codes and the influence of these torsional provisions has been analytically studied.

In the preceding chapters, the design seismic forces for the TU and the reference models analysed were calculated based on the equivalent static method of each seismic code. Presenting the response of various TU models designed to different seismic codes indicated the efficiency of their torsional provisions for appropriately designing various model types. In this chapter, a new equivalent static force procedure suggested by Duan and Chandler (1997) is examined (Section 9.2) and compared with the equivalent static force procedures of the EC8, NZS and UBC codes (Section 9.3). The inelastic response of TU models designed to the new proposed method is compared to the response of the same set of models designed to the existing seismic codes. The relative merits and deficiencies of the seismic code torsional provisions, with respect to their adequacy in controlling the additional ductility demand of the models are then discussed (Section 9.4).

9.2 THE RECENTLY PROPOSED EQUIVALENT STATIC FORCE PROCEDURE

Duan and Chandler (1997) developed an optimised procedure for the design of multi-storey regularly asymmetric buildings, which led to satisfactory inelastic seismic performance for these buildings, by considering both the serviceability limit state (SLS) and the ultimate limit state (ULS). The results of the inelastic analyses carried out by Chandler and Duan (1997) indicated that the new equivalent static method offered a consistent protection to TU models against structural damage. The method retained simplicity for each seismic code implementation and resulted in relatively small, and hence acceptable, increases of the total strength compared with that of the reference models. Hence, this new equivalent static method was regarded as a general guideline for the design of multi-storey regularly asymmetric frame buildings and it was recommended for incorporation into the seismic building codes for design practice.

Although a common seismic design philosophy has been accepted world-wide, its implementation varies and two different approaches have been adopted by national codes. The first approach, termed one-phase design procedure, is to primarily safeguard against structural failure and loss of life, without attempting to limit damage to maintain functioning of buildings when subjected to more frequent minor to moderate earthquakes. Hence, this procedure adopted by the EC8 and UBC codes specifies design spectra only for the ULS. The second approach, adopted by the NZS code and termed a two-phase design procedure, specifies design spectra for both limit states and designers are required to check explicitly that the requirements of both limit states are satisfied.

The torsional provisions in current seismic codes are not dependent on the design limit state concerned and hence not dependent on the force reduction factor R. Chandler and Duan (1997) have shown that the force reduction factor influences the response of the stiff-edge element significantly. When R increases, code torsional provisions become increasingly overconservative in estimating the strength demand of the flexible-edge element and unconservative in estimating the strength demand of the rigid-edge element. In order to overcome these shortcomings in current code torsional provisions, Duan and Chandler (1997) recommended this optimised, two-phase procedure for the design of TU structures for both limit states. In this method, not only the design base shear, but also the design eccentricities, are dependent on the limit state concerned. For each element, the more unfavourable strength demand resulting from the SLS and ULS design is identified and used for sizing and detailing the element considered. The recommended procedure is summarised by the steps below:

Step 1: The TU structure's fundamental uncoupled lateral period T_y is estimated (ignoring torsion) using the simplified, empirical methods suggested in building codes.

Step 2: The seismic base shear forces for both limit states are calculated, according to the design spectra specified in building codes for both SLS and ULS. In the SLS, the base shear force is dependent on T_y (Step 1) while, in the ULS, it is additionally dependent on the force reduction factor R. The base shear forces calculated are already a conservative estimate, since torsion has been ignored at this stage.

Step 3: The two system parameters, the normalised (with respect to the plan dimension b) static eccentricity e and the normalised stiffness radius of gyration ρ_k are calculated. For single-storey TU structures, the procedure of calculating e and ρ_k is straightforward. For multi-storey TU models where the mass centres lie along a vertical line and the stiffness properties of lateral load-resisting elements parallel to the y-axis are proportional to one another, a simple procedure for calculating e and Ω has been developed by Hejal and Chopra (1989a) (Section 3.4.1). The value of ρ_k can then be calculated by using Equations (3.5.8) and (3.5.9).

Step 4: The lateral overstrength factor O_s (dependent on ρ_k) is calculated by

$$O_s = \begin{cases} -3.33\rho_k + 2.33 & 0.25 \le \rho_k \le 0.40 \\ 1.0 & \rho_k > 0.40 \end{cases} \tag{9.2.1}$$

The recommended O_s has its largest value of 1.5 at ρ_k = 0.25, decreasing linearly to unity at ρ_k=0.40 and thereafter remaining constant for larger ρ_k values.

Step 5: The modified base shear forces for both limit states are calculated by multiplying the base shear forces (Step 2) by the overstrength factor O_s (Step 4).

Step 6: The design eccentricity values for both limit states are determined by reading e_d/e from the design eccentricity charts given by Duan and Chandler (1997) (Figure 9.2.1) where e_d is the design eccentricity and e is the normalised static eccentricity. The values of e_d/e depend on ρ_k for both limit states and on the design force reduction factor R for the ULS.

Step 7: For each limit state, the strength demand for each structural element is obtained by applying the corresponding modified base shear force (Step 5) through a point of the floor deck at a distance from CR equal to the corresponding e_d (Step 6), and then carrying out an elastic static analysis.

Step 8: The accidental torsional effects are considered by increasing the strength demand of each element appropriately, for each limit state.

Step 9: For each element, the more unfavourable strength demand resulting from the SLS and the ULS design is identified and used for detailing the elements concerned.

The explicit consideration of the accidental eccentricity effects (Step 8) was beyond the scope of the study presented by Duan and Chandler (1997). Therefore, they recommended a proposed methodology by De La Llera and Chopra (1995) to consider the accidental torsional effects in the SLS while they mentioned that the accidental torsional effects in the inelastic (corresponding to the ULS) response of TU structures remains an issue to be resolved by future studies. Consequently, in order to retain a consistency, the accidental torsional effects are not included in the strength calculation of the models investigated in this section, when designed either according to the EC8, NZS and UBC seismic regulations or according to the new proposed optimised method.

9.3 INELASTIC SEISMIC RESPONSE OF TU MODELS DESIGNED ACCORDING TO THE NEW OPTIMISED METHOD

The investigation of the inelastic dynamic response of various TU models designed to the new optimised method is crucial to assess the efficiency of this procedure. The response of the models investigated is compared to their response when designed to the EC8, NZS and UBC static torsional provisions. The models employed are the mass-eccentric model 6S30 and the stiffness-eccentric model 6B30 (Figure 8.2.1). The models consist of 6 floor levels and their static eccentricity is equal to $0.30b$. Since the aim of this section is to indicate the differences in the inelastic response of TU models designed to the existing seismic code provisions and the new optimised method, only one earthquake record is adopted for the inelastic analyses carried out, namely the "El Centro" record.

Both models 6S30 and 6B30 have the same Ω (=1.0) and ρ_m(=0.31) values (see also Section 3.5.2) and, consequently, they also have an identical ρ_k value equal to 0.31 and an overstrength factor O_s equal to 1.3. Based on the overstrength factors calculated and the design charts given in Figure 9.2.1, the normalised design eccentricities of the models can be found. For both models 6S30 and 6B30, the normalised design eccentricity for the ULS is equal to 0.45 and for the SLS is equal to 0.50. It should be also reminded that only the NZS seismic code specifies design spectra for both limit states and, therefore, the SLS design eccentricities will be used only for this code.

STRENGTH DISTRIBUTION OF MODEL 6B30 (New Method / Existing Provisions)						
	EC8		NZS		UBC	
Frame	No Min	Min	No Min	Min	No Min	Min
1	2.81	1.32	1.70	1.20	1.55	1.04
2	1.27	1.09	1.25	1.04	1.22	1.00
3	0.91	0.92	1.04	1.02	1.03	1.00
4	0.76	0.76	0.93	0.94	0.93	0.97
5	0.67	0.67	0.86	0.86	0.87	0.91
6	0.62	0.62	0.81	0.82	0.82	0.89
Total	**1.23**	**1.08**	**1.30**	**1.05**	**1.20**	**1.00**

Table 9.3.1 Strength distribution of the frames of model 6B30 designed to the new proposed method and normalised to their strength when designed to the existing seismic code provisions.

For the stiffness-eccentric model 6B30, Table 9.3.1 presents the strength of each frame designed to the new proposed method and normalised to the corresponding strength of the same frame designed to the existing seismic code provisions. The first column for each code presents the ("unmodified") strength ratios of the frames (new method / existing seismic code provisions) calculated from the results of the elastic static analyses. The second column for each seismic code presents the strength ratios of the frames when the design building code provisions for the strength calculation of the structural elements are included.

The strength ratios presented in Table 9.3.1 indicate that, for all the seismic codes employed, the new proposed method results in higher strength in frames 1 and 2 (of model 6B30), in slightly lower strength in frames 3 and 4, and in lower strength in frames 5 and 6. The new provisions influence more the strength of frame 1 when previously designed to the EC8 static torsional provisions (2.81 times higher) since, for this code, the design seismic forces are applied the furthest away from frame 1 due to the inclusion of the additional eccentricity component. The total strength increase of model 6B30 is equal to 23% when the design base shear is calculated according to the EC8 code, 30% for the NZS code and 20% for the UBC code.

The inclusion of the minimum reinforcement requirements of the building codes for the design of the structural elements changes the strength ratios of the frames of model 6B30 and results in much lower strength ratios (Table 9.3.1). The strength ratios of the frames are considerably reduced, and the influence of the new design method is not clear anymore. As a result, the effect of the new method is not significant when the minimum steel ratios are included, and for the UBC code, the total strength of the model is identical for both the proposed and the existing UBC torsional provisions. Only for the EC8 and NZS codes, the total strength of the model is slightly higher for the new provisions (8% and 5%, respectively) while differences in the strength distribution can still be noticed. The strength of frames 4 – 6 is always lower for the new procedure while the strength of frame 1 results is higher strength values when designed to the EC8 and NZS codes. In the Figures of this chapter, when the new proposed method is adopted, the code employed for the calculation of the design seismic forces is also indicated.

The above remarks regarding the influence of the new optimised procedure are also evidenced in Figures 9.3.1 and 9.3.2, where the inelastic response of model 6B30 designed with (Figure 9.3.2) or without (Figure 9.3.1) the new provisions is presented. No major differences in the response of the model are encountered when the new optimised method is applied. The maximum ductility demand of frame 1 is reduced when the design seismic forces are calculated according to the EC8 code, the code that generally results in the lowest response values due to its high base shear value. The maximum response values of model 6B30 are generally in the same range, with or without the new optimised requirements, mainly due to the inclusion of the minimum steel ratios for the strength calculation of the structural elements. The results indicate that the new proposed method could result in a better inelastic response of the TU stiffness-eccentric models, provided that the base shear is calculated according to the EC8 code.

Contrary to this conclusion, Figures 9.3.3 and 9.3.4 indicate that the maximum response values of the mass-eccentric model 6S30 are not influenced by the inclusion of the new provisions and they are even slightly increased. With or without the new proposed requirements, the maximum response values are in the same range, and minor differences can be observed. Therefore, as also indicated by the results of the inelastic analyses carried out by Chandler and Duan (1997), this new optimised method needs further modifications of the design of mass-eccentric models.

Therefore, from the results of the inelastic time-history analyses carried out in this section, it can be concluded that the new proposed method can be better applied to stiffness-eccentric models. The maximum response values of mass-eccentric models designed according to the new optimised provisions are slightly increased indicating that further research should be carried out for the applicability of the new method to mass-eccentric models. The maximum response values in all models analysed are generally in the same range, with or without the new proposed requirements. Stiffness-eccentric models result in better inelastic response by employing the new optimised method provided that the base shear value is calculated according to the EC8 seismic code provisions (the highest base shear value). The great influence of the values of the design seismic forces and of the minimum reinforcement requirements in the design of the models is indicated once more. The strength increase of the models due to the new provisions is reduced by the influence of the inclusion of the minimum steel ratios for the design of the models. The simplicity of the method is its most important advantage while further research should be carried out for the incorporation of the accidental eccentricity effects.

9.4 MERITS AND DEFICIENCIES OF THE EXISTING SEISMIC CODE PROVISIONS

The influence of the static torsional provisions of different seismic codes was analytically investigated in this study by examining the inelastic torsional behaviour of mass-eccentric and stiffness-eccentric models, with or without transverse elements. In Section 6.7, the influence of the static torsional provisions of the EC8, NZS and UBC codes was studied by analysing TU models without transverse elements and, in Section 7.6, their inelastic behaviour was compared to the behaviour of their reference models. The response of various TU models including transverse elements and designed to different seismic code regulations was also presented in Section 8.4 and their response was compared to the response of similar TU models without transverse elements. Finally, in Section 9.3, a new proposed equivalent static force procedure was examined and compared with the torsional provisions of the existing seismic codes. Therefore, based on the results of the inelastic dynamic analyses carried out in these sections, the merits and the deficiencies of the seismic codes examined can be summarised as follows:

<u>Eurocode 8 (1993)</u>

1. The static torsional provisions of the EC8 code, in contrast to other more typical code provisions, incorporate an additional eccentricity term dependent on additional structural parameters such as static eccentricity, mass radius of gyration, floor plan aspect ratio and torsional to lateral stiffness ratio. Therefore, structures with a broad range of configurations are accounted for specifically, although the calculation of the EC8 first design eccentricity is complicated due to the multi-parameter dependence of the additional eccentricity component. (Section 4.2).
2. The significant amplification of the first design eccentricity required by the EC8 code results in the highest first design eccentricity, which increases significantly the strength of the flexible side (frames 4 – 6) in stiffness-eccentric models (Table 6.7.1). For mass-eccentric models, the EC8 first design eccentricity also results in the highest strength for frames 4 and 5, when compared to the other codes examined.

3. The EC8 second design eccentricity results in the lowest strength for the stiff side of stiffness-eccentric models and for frames 1 – 3 and 6 of mass-eccentric models (Table 6.7.1).
4. Irrespective of the design eccentricity values, the strength distribution of the models is mainly influenced by the design seismic forces and the inclusion of the minimum steel ratios for the element strength calculation. Therefore, all model types designed to the EC8 code result in the highest strength values due to the high base shear value of this code (Table 6.7.4).
5. No concentrated seismic force is applied at the top floor level of the models when designed according to the EC8 code while the inclusion of a top force could decrease the ductility demand of the upper floor levels.
6. The EC8 code provisions result in the best response for both stiffness-eccentric and mass-eccentric models due to the higher design seismic forces applied to the models.
7. The structural regularity provisions of the EC8 code are highly restrictive since the equivalent static method gives good control even for asymmetric structures with high values of static eccentricity, characterised as non-regular by the code (Section 6.2.1).

New Zealand Standard Code (1992)

1. The NZS first design coefficient is equal to unity and, consequently, there is no need to determine the position of CR since the design seismic forces are applied at $+0.1b$ from CM (Section 4.2). This is particularly convenient and practical for multi-storey structures where CR is more difficult to be defined at each floor level.
2. Since the strength of the structural elements is mainly influenced from the design seismic forces and the minimum reinforcement provisions, all models designed to the EC8 and NZS codes result in similar response values (Sections 6.7, 7.6 and 8.4). Only minor differences in the behaviour of stiffness-eccentric models are encountered at the upper floor levels of the flexible side where the NZS code results in slightly higher response values than the EC8 code. The response of mass-eccentric models is similar for both EC8 and NZS seismic codes.

3. A constant concentrated top force is applied in models designed to the NZS code while the inclusion of a higher top force dependent on the structural period could further decrease the ductility demand of the upper storeys in stiffness-eccentric models.
4. Both stiffness-eccentric and mass-eccentric models designed to the NZS code result in a good seismic performance indicating that the structural regularity provisions of the code are highly restrictive. The equivalent static method gives good results even for structures with high values of static eccentricity, characterised as non-regular (Section 6.2.1).

Uniform Building Code (1994)

1. The UBC accidental eccentricity component is dependent upon the torsional structural stiffness and regularity since the A_x factor is indirectly proportional to the maximum to average displacement ratio of the structure under monotonic load (Section 4.2).
2. The UBC first design eccentricity incorporating the A_x factor results in the highest strength for frames 1 – 3 and 6 of mass-eccentric models.
3. Unlike the other two codes considered, the UBC second design eccentricity allows no strength reduction in TU models (compared with the corresponding strength of their reference models) when the static eccentricity is greater than the accidental eccentricity component (Section 4.2). This UBC provision results in design forces applied at CR and, consequently, in the highest strength for the stiff side of stiffness-eccentric models and for frames 4 and 5 of mass-eccentric models (Table 6.7.1).
4. The strength of the models is mainly influenced from values of the design forces and the minimum steel ratios imposed by the codes and, therefore, all models designed to the UBC code result in the lowest strength values due to the low base shear value of this code (Table 6.7.4).
5. When the structural models are designed to the UBC regulations, a concentrated seismic force is applied at the top floor level of models with a structural period higher

than 0.7 seconds. The inclusion of a top force in TU models with a structural period lower than 0.7 seconds could decrease the ductility demand of the upper floor levels.

6. Although the UBC provisions control better the response of the stiff side in stiffness-eccentric models, they generally result in the worst response for both stiffness-eccentric and mass-eccentric models due to the low design seismic forces.
7. Although the UBC regulations result in the highest ductility demand values, no column sidesway mechanisms are formed in the models and the interstorey drift ratios are always much lower than the 2% limit value, indicating no potential for collapse.
8. The equivalent static method of all seismic codes examined manages to adequately control the inelastic torsional response of the TU models indicating that their structural regularity provisions are highly restrictive. The equivalent static force procedure gives good control even in asymmetric models with high static eccentricities, which are characterised as non-regular (Section 6.2.1), indicating that this method is simple and efficient to design TU structures.

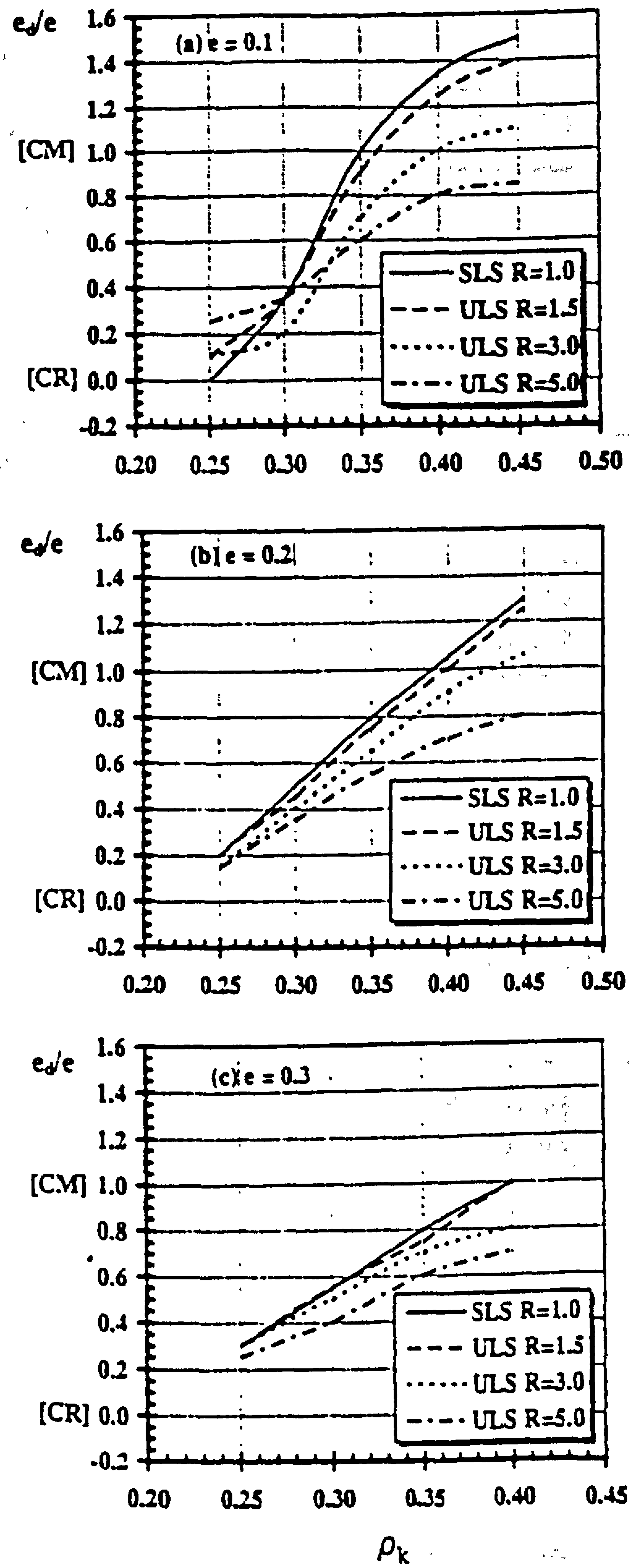


Figure 9.2.1 Design charts for the approximately optimised normalised design eccentricity e_d (Duan and Chandler, 1997).

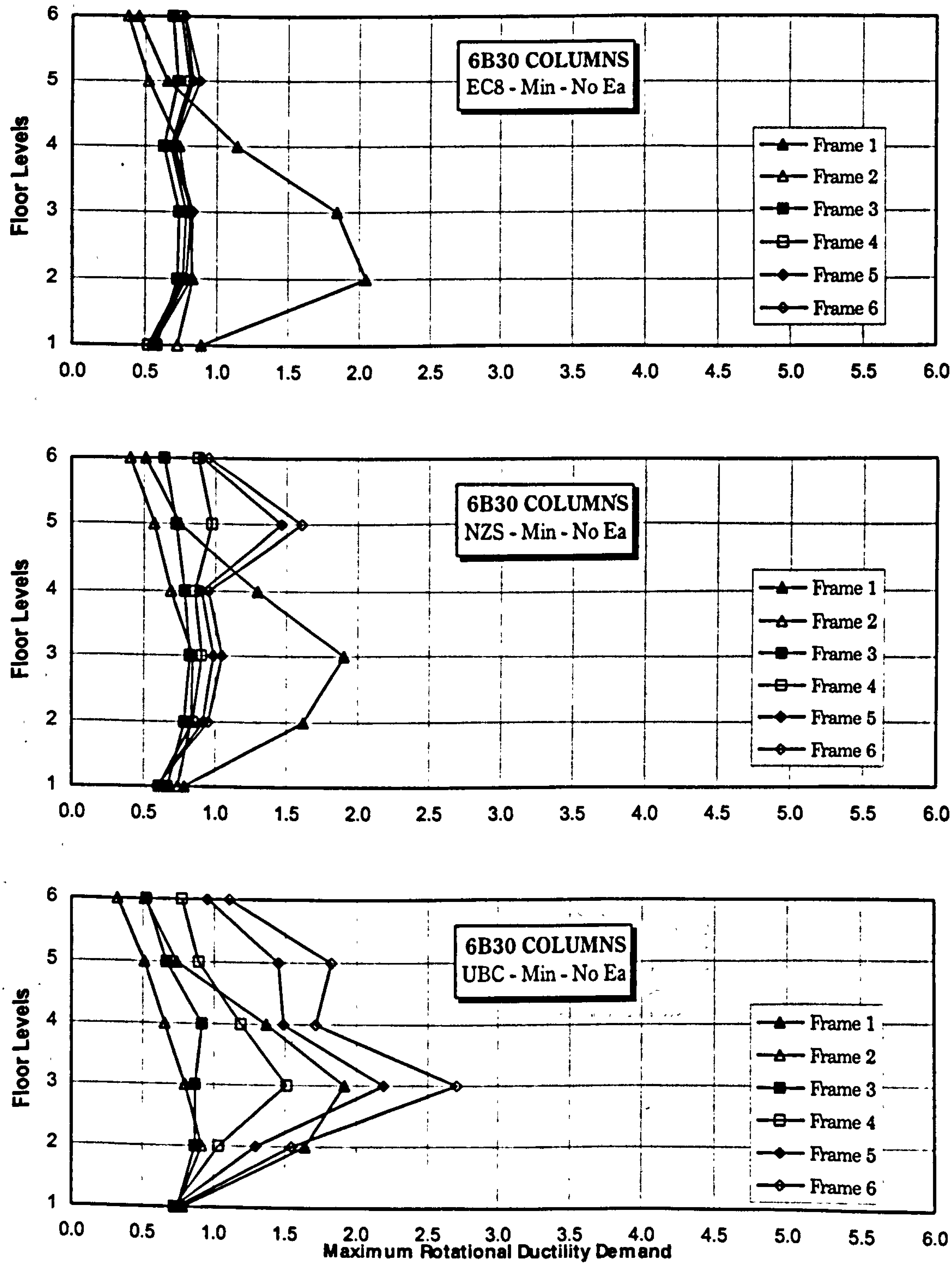


Figure 9.3.1 Maximum rotational ductility demand vs. floor levels for the columns of model 6B30 designed to the EC8, NZS and UBC seismic code provisions.

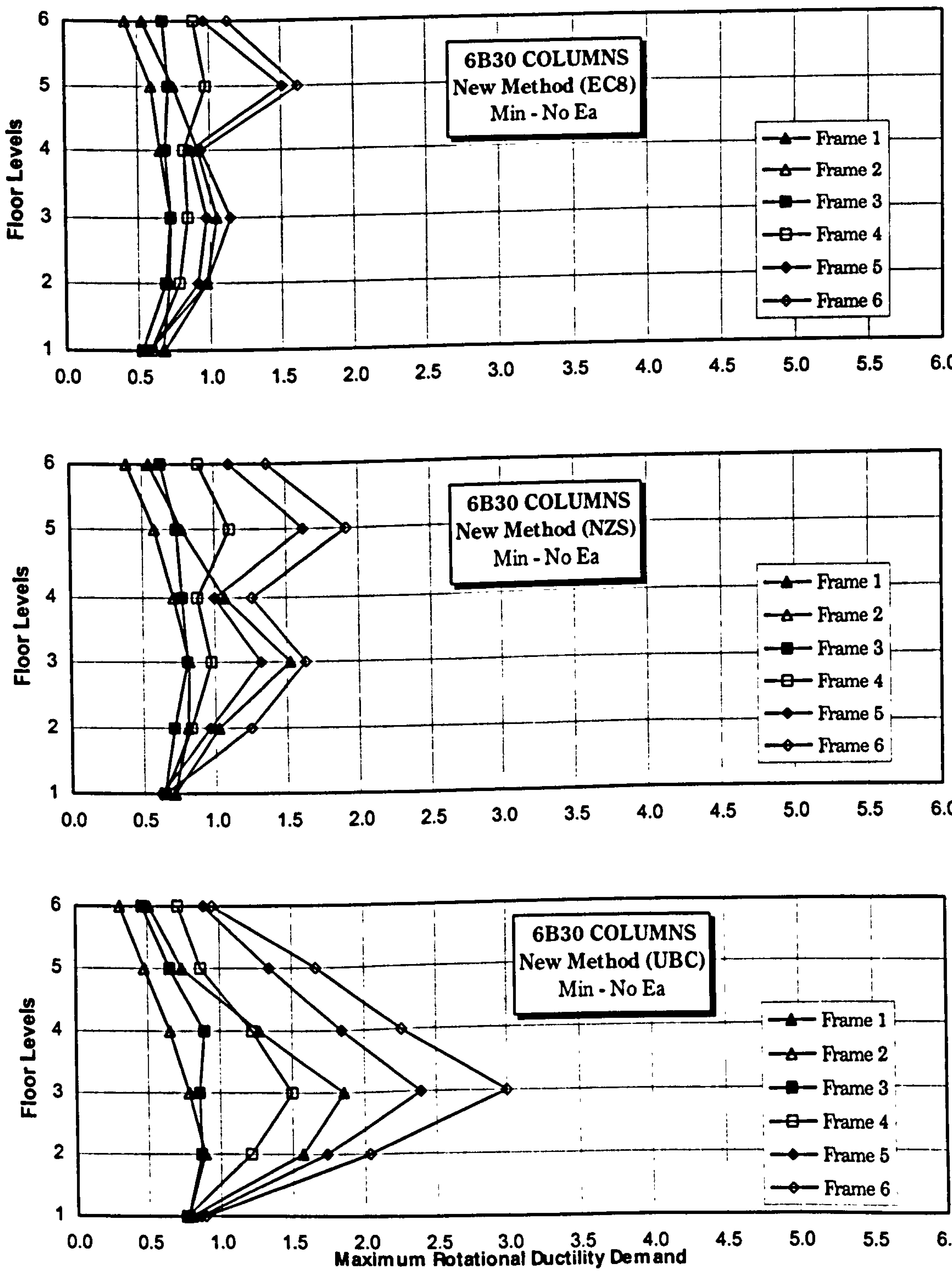


Figure 9.3.2 Maximum rotational ductility demand vs. floor levels for the columns of model 6B30 designed to the new equivalent static force procedure proposed by Duan and Chandler (1997).

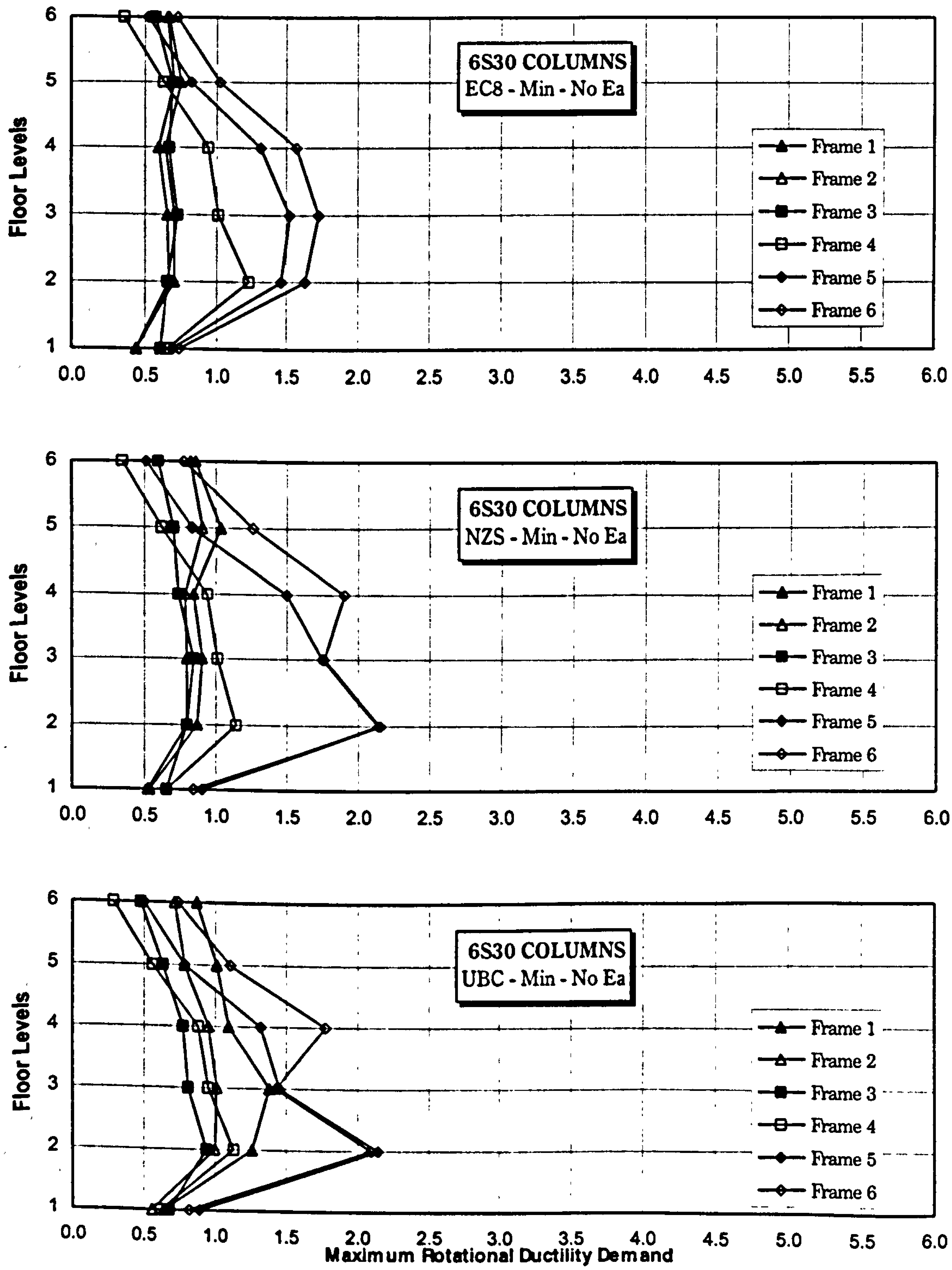


Figure 9.3.3 Maximum rotational ductility demand vs. floor levels for the columns of model 6S30 designed to the EC8, NZS and UBC seismic code provisions.

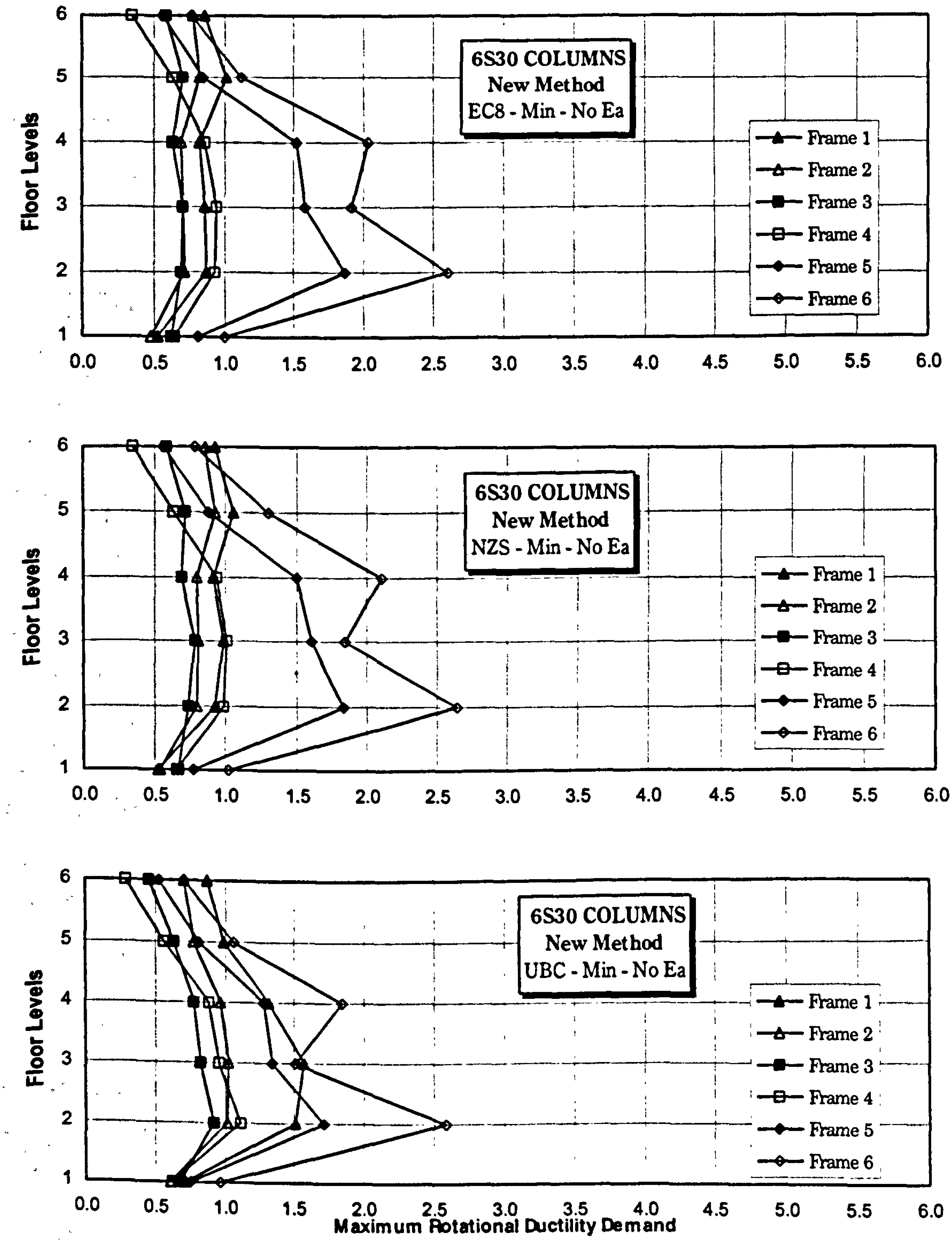


Figure 9.3.4 Maximum rotational ductility demand vs. floor levels for the columns of model 6S30 designed to the new equivalent static force procedure proposed by Duan and Chandler (1997).

CHAPTER 10

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

10.1 FUNDAMENTAL CONCLUSIONS

The aim of this study was to examine the inelastic torsional behaviour of multi-storey asymmetric (TU) buildings since their response to earthquake loading is far more difficult to predict and to design for than the response of symmetric structures. In the first research studies examining the torsional effects of asymmetric buildings, carried out during the 1970s to early 1980s, the attention was focused on the elastic structural behaviour. The main purpose was to achieve a general understanding regarding the torsional effects of the structures by employing simple single-storey models. However, as the response of real structures is mainly inelastic, these studies gave poor information about the torsional behaviour under severe earthquakes and the interest has moved since the mid-1980s towards non-linear studies.

The inelastic seismic response of TU structures is highly dependent on numerous structural parameters and one of the basic objectives of this study was to identify the influence of the key structural parameters by employing realistic structural models. Furthermore, regarding the inelastic response of multi-storey asymmetric buildings, there has been little research carried out and, consequently, the torsional provisions and the recommendations of the existing seismic codes are mainly based on conclusions from single-storey research studies. It was therefore of crucial importance to analyse multi-storey buildings responding in the inelastic range, to compare their response to the response of single-storey structures and to check the efficiency of the existing earthquake resistant codes to design TU multi-storey structures. The effectiveness of the equivalent static force procedure for the seismic design of multi-storey TU structures was investigated and a comparison between the static torsional provisions of different seismic codes (EC8, NZS and UBC codes) was carried out.

This thesis was organised in ten chapters (Section 1.7), which included a general introduction and the objectives of the study (Chapter 1), a literature review about the torsional behaviour of asymmetric structures (Chapter 2) and the structural configuration of the models employed (Chapter 3). In Chapter 4, information regarding the design analysis procedure and the static torsional provisions of the seismic codes employed was presented. Chapter 5 examined the inelastic seismic response of the reference models and investigated the factors that influence their behaviour. The inelastic torsional behaviour of various multi-storey regularly asymmetric frame models was presented in Chapter 6 while their behaviour was compared to the behaviour of appropriate reference models in Chapter 7. TU models incorporating transverse elements and subjected to bi-directional earthquake loading were analytically studied in Chapter 8 while, in Chapter 9, a new proposed equivalent static force procedure was tested and the relative merits and deficiencies of the existing seismic codes were discussed.

At the end of each chapter or major segment of research, the concluding remarks were discussed and, therefore, the reader is referred to these detailed conclusions for categorical discussions on the various subjects examined. In this chapter, the most important conclusions regarding the critical issues analysed are summarised and discussed (Section 10.1) and subjects for further research are recommended based on the results of the inelastic analyses carried out in this study (Section 10.2).

10.1.1 Reference Models (Chapters 5 and 8)

The aim of the investigations carried out in Chapter 5 was to present the inelastic response of different reference models (without transverse frames) and to investigate the factors influencing their inelastic torsional behaviour. The response of reference models with transverse elements was presented in Chapter 8 and the basic conclusions regarding the dynamic behaviour of the reference models can be summarised as follows:

1. All the frames of a reference model respond identically only when their strength distribution is proportional to their stiffness distribution. The torsionally balanced (TB) reference models having an asymmetric stiffness distribution can be **stiffness and strength proportional** (SSP) only when designed without the accidental eccentricity provisions and the minimum reinforcement requirements imposed by the codes for the design of the structural elements. Furthermore, the inelastic response of different types of reference models (SM and TB models) having the same structural properties is identical when designed without the accidental eccentricity provisions and the minimum steel ratios and, therefore, no differences between their inelastic seismic response are encountered.
2. The reference models are particularly important for the comparison between their inelastic response and the response of TU models. Different conclusions could be reached due to the varying strength distribution of the reference models and, for this reason, both the reference and the TU models were designed for **different definition** cases, with and without the incorporation of the minimum steel ratios and the accidental eccentricity provisions.
3. SM models are adopted as reference models for mass-eccentric structures while TB models are the reference models for stiffness-eccentric models. No changes between the stiffness-distribution of the TU models and of their reference models are made and, therefore, **both SM and TB reference models can be considered to provide an appropriate basis** for assessing the significance of the inelastic torsional effects in TU models.
4. The inclusion of transverse structural elements does not influence the inelastic seismic response of the lateral frames in SM models while it affects the response of the lateral frames in TB models. The **transverse structural elements** in both SM and TB reference models respond in the inelastic range only when subjected to bi-directional seismic loading, indicating the great significance of incorporating the second earthquake component. In contrast, the response of the lateral frames remains identical irrespective of the seismic loading condition.

5. The reduction of the inelastic seismic response of each frame of the reference models due to the inclusion of the accidental eccentricity and of the minimum steel ratios is dependent on the strength increase that each of these two factors produces in the structural elements. The reduction of the ductility demand is higher than the strength increase induced in the models indicating that the **dynamically responding systems are more efficient in dissipating earthquake vibrations** than would be suggested by a straightforward linear relationship between ductility demand and strength.

10.1.2 TU Models without Transverse Elements (Chapters 6 and 7)

The inelastic seismic response of various TU models without transverse elements was investigated in Chapter 6 while, in Chapter 7, the response of TU models was compared to the response of their reference models. The most important conclusions regarding the effect of various factors on the inelastic dynamic response of TU models can be summarised as follows:

1. The importance of employing two different design methods (with and without the **minimum steel ratios**) for the strength calculation of the models was justified from the results of the inelastic dynamic analyses. The inclusion of the minimum steel ratios alters significantly the strength of the models and changes their inelastic seismic response leading to contradictory conclusions and indicating the significant influence of the inclusion of the minimum steel ratios. Furthermore, by including the minimum reinforcement requirements in the design of the structural elements, the influence of the accidental eccentricity cannot be clearly identified. Only when the minimum steel ratios are excluded from the design of the models, the effect of the accidental eccentricity provisions can be observed, indicating a further reason for employing two different design methods.
2. The **accidental eccentricity provisions** increase the strength of the models and change their inelastic seismic response, especially when the minimum steel ratios are not incorporated. Contrary to the inclusion of the minimum steel ratios, which alters the response patterns of the frames, the allowance for accidental eccentricity

reduces the maximum response values of the models without greatly altering their patterns of inelastic response.

3. The comparison of the inelastic seismic response of TU models with the response of their reference models was based on **different definition cases** concerning the inclusion of the accidental eccentricity and of the minimum steel ratios. When the accidental eccentricity is included only in the TU model, the normalised response of the models is the highest indicating that a consistency should be maintained in the application of the accidental eccentricity in both reference and TU models.

4. The trends found regarding the inelastic seismic response of the **column elements** are similar to the trends found for the response of the **beam elements**. All the factors examined influence in a similar way the torsional response of both columns and beams while the response values of the beams are always higher than the response values of the columns due to the capacity check.

5. When the minimum steel ratios are not included in the design of stiffness-eccentric models (termed the 1st design method), the increase of the **static eccentricity** increases the response of frame 1 (stiff side) while it reduces the response of frames 2 – 6. Thus, TU models with higher static eccentricities result in higher values of ductility demand and the difference between the response of frame 1 and the rest of the frames increases as well. When the minimum steel ratios are included in the design of the models (termed the 2nd design method), the maximum response values are considerably reduced and models with higher static eccentricities respond better due to the higher total strength increase induced by the minimum steel ratios. Irrespective of the design method adopted, the maximum ductility demand is mainly located in the lower storeys of frame 1 and in the upper storeys of frames 4 – 6. Furthermore, the displacement of frame 6 and the difference between the displacement of frames 1 and 6 increase for models with higher static eccentricities indicating that their torsional deformation is higher. The normalised displacement of frame 1 is lower than unity while it exceeds unity in frame 6 showing that the normalised lateral displacement is a significant parameter for the flexible edge of stiffness-eccentric models and that the lateral deformation is higher in TU models compared to the reference models.

6. When the minimum steel ratios are excluded from the design of the structural elements, the inelastic seismic response of TU models with varying **stiffness distribution** is considerably different while their maximum ductility demand differs in each model. By including the minimum steel ratios, the response of all frames is considerably reduced and different model types result in similar response values. In stiffness-eccentric models, the upper columns of frames 4 – 6 and the lower columns of frame 1 result in high values of ductility demand, for both design methods adopted. The mass-eccentric models result in a completely different inelastic response, and the columns responding in the inelastic range are the lower columns of frames 5 and 6 and the upper columns of frame 1, with or without the inclusion of the minimum steel ratios. The models, which are stiffness- and mass-eccentric, respond similar to the stiffness-eccentric models since their stiffness distribution is not symmetric either.

7. When the minimum steel ratios are excluded from the design of TU models having different numbers of floor levels, the increase of the **model height** increases their inelastic response and excessive ductility demand values are found in the stiff frame of taller stiffness-eccentric models. By including the minimum steel ratios, the ductility demand is significantly reduced and models with different heights result in similar response values. Irrespective of the design method adopted, the upper columns of frames 4 – 6 respond in the inelastic range while the response of the stiff frame is reduced by the inclusion of the minimum steel ratios. The increase of the structural period increases the lateral displacement of the models and the torsional effects result in additional inelastic displacement of the flexible side, which can be substantial, irrespective of the model height, and highly dependent on the characteristics of the earthquake ground motions. The effect of torsion on the edge displacement is more pronounced in short-period structures, which result in larger values of normalised lateral displacement.

8. When the minimum steel ratios are excluded from the design of the models, their response depends on the design forces and on their points of application and, therefore, the **static torsional provisions** of different codes influence each model type differently. The strength of the models is mainly influenced by the base shear

value and although the UBC design eccentricities result in the highest overstrength, the low UBC base shear results in the lowest total strength. The minimum steel ratios increase the strength of the frames and the response of different models designed to various seismic codes presents no major differences.

9. In **stiffness-eccentric models**, the frames responding in the inelastic range are the lower storeys of frame 1 (for all codes) and the upper storeys of frames 5 and 6 (for NZS and UBC). EC8 code requiring an additional eccentricity for the first design eccentricity is the only code that results in no ductility demand (elastic response) at the flexible side while the UBC provision permitting no strength reduction in the TU models results in no additional ductility demand at the stiff side. Thus, the EC8 code controls better the ductility demand at the flexible side while the response of the stiff side is better controlled by the UBC code.

10. In **mass-eccentric models**, the frames responding in the inelastic range are the lower storeys of frames 5 and 6 (for all codes) and the upper storeys of frame 1 (for NZS and UBC). Frame 6 and the upper storeys of frames 1 and 2 respond worst than their reference frames for all codes while the normalised ductility demand of the lower floor levels of frame 5 also exceeds unity for EC8 and NZS. The UBC provision permitting no strength reduction in TU models results in no additional ductility demand in frame 5. Therefore, the normalised ductility demand of frames 5 and 6 is higher for EC8 code while the normalised ductility demand of frames 1 and 2 is higher for UBC.

11. All TU models examined having different structural parameters and designed to various static torsional provisions result in interstorey drift values lower than the nominal 2% collapse limit. This indicates that there is virtually **no potential for collapse** under moderate to severe earthquake loading and that the static torsional provisions manage to control the inelastic response of TU models. Furthermore, the regularity requirements of all codes examined are over-conservative since TU models characterised as non-regular respond very satisfactory when designed with the equivalent static method. Therefore, although the inelastic behaviour of TU models is higher than the behaviour of their reference models, they can resist moderate to severe earthquakes without major structural damage.

10.1.3 TU Models with Transverse Elements (Chapter 8)

The inelastic seismic response of TU models incorporating transverse structural elements was examined in Chapter 8 by subjecting the models to both uni-directional and bi-directional seismic loading. The behaviour of these models was then compared to the behaviour of TU models without transverse elements and the basic conclusions reached can be summarised as follows:

1. The response of the **lateral frames** in TU models is influenced by the inclusion of transverse elements while it is not influenced by the seismic loading condition (uni-directional or bi-directional). The lateral frames in different TU models (mass-eccentric or stiffness-eccentric) are influenced differently from the inclusion of the transverse elements.
2. Contrary to the response of the lateral frames, the inelastic response of the **transverse frames** is influenced by the inclusion of the second earthquake component while it is not significantly affected by the static torsional provisions of the seismic codes adopted due to the symmetric configuration in this direction.
3. The rotational ductility demand of the structural elements increases considerably in the lateral frames of both mass-eccentric and stiffness-eccentric models by the inclusion of the transverse frames. This is justified from the fact that the inclusion of the transverse structural elements increases the torsional stiffness of the structure and the alters the response of the frames. Although the maximum lateral displacement values are reduced, specific frames result in higher lateral displacements and consequently in higher values of ductility demand, which control the response of the models. Thus, increasing the torsional stiffness of a model does not necessarily imply that the response of the model improves. Changes in the stiffness distribution of the models result in significant changes in the inelastic response of specific frames, which may control the response of the whole structure.
4. The results indicated that the inclusion of the transverse frames reduces the maximum values of lateral displacement of the frames. The same conclusion regarding the response of TU models including transverse elements was also

reached by Paulay (1997c), who indicated the **beneficial effects of the transverse elements** by reducing the values of the lateral displacements and, therefore, satisfying the design criteria for ductile structures (limitation of the interstorey displacement values). Again, it should be noted that reduction of the maximum lateral displacement of the model, does not necessarily mean that all frames result in lower values of lateral displacement, and, consequently in a lower ductility demand.

5. In **mass-eccentric** models without transverse structural elements, frames 5 and 6 resulted in the highest inelastic seismic response while, by including transverse frames, the upper floor levels of frames 1 – 3 and 6 respond in the inelastic range. Although the interstorey drift ratios of all floor levels are lower than the 2% collapse limit, an increased caution should be given to the upper storeys of frames 1 – 3 and 6, which result in values of rotational ductility demand approximately equal to 5.0. In similar TU models without transverse structural elements, the ductility demand is found to be much lower, reaching only the value of 2.0. The EC8 and NZS seismic code requirements control better the inelastic response of frames 1 and 2 while the UBC code provisions control better the response of frames 3 and 6. Similar remarks are reached for the inelastic response of TU models without transverse elements although their maximum response values are lower and they are located in different frames.

6. The inclusion of transverse structural frames also influences the inelastic seismic response of **stiffness-eccentric** models by reducing the response values of the stiff frame while the response of the upper storeys of frames 4 – 6 is increased. The frames resulting in higher values of ductility demand are the upper storeys of frames 4 – 6, which respond better for the EC8 seismic code provisions, while frame 1 responds better for the UBC code requirements. Although the interstorey drift ratios are always lower than the 2% collapse limit, the upper storeys of frames 4 – 6 result in higher values of ductility demand, reaching the value of 6.0, while the maximum values of ductility demand in similar TU models without transverse elements is 3 times lower. In stiffness-eccentric models with transverse elements, EC8 code controls better the response of the flexible side due to the

inclusion of the additional eccentricity component while the stiff side is better controlled by the UBC code due to the provision permitting no strength reduction in the TU models. A similar conclusion is also reached from the investigation of TU models without transverse structural elements although the maximum values of ductility demand were lower and they are located in different frames.

10.1.4 A Recently Proposed Optimised Method and Current Seismic Code Provisions (Chapter 9)

In Section 9.3, the efficiency of the equivalent static force procedure proposed by Duan and Chandler (1997) was investigated by examining the inelastic seismic response of various TU models, with and without transverse structural elements. The models were designed to all three seismic codes adopted in this study and their inelastic response was compared to their behaviour when designed by incorporating the new provisions. In Section 9.4, the relevant merits and deficiencies of the existing seismic codes were analytically discussed while the most important conclusions regarding the investigations carried out in Chapter 9 can be summarised as follows:

1. The **recently proposed optimised method by Chandler and Duan (1997)** retains simplicity and generally results in similar response values with the existing seismic code requirements. The inclusion of the minimum reinforcement provisions for the design of the structural elements reduces any possible increases in the total strength of the models due to the new proposed method. The method could be successfully applied to stiffness-eccentric provided that the base shear value is calculated with the EC8 or NZS seismic codes. In mass-eccentric models, the new optimised method seems not to be applicable since the maximum response values are higher when the new method is adopted.
2. Further research regarding the incorporation of the **accidental eccentricity provisions** should be carried out since this method suggests no provisions for the accidental torsional effects in the inelastic range. Since the strength of the structural elements is mainly influenced from the design seismic forces and the minimum reinforcement provisions, TU models designed to different seismic code

provisions, without the incorporation of the accidental eccentricity requirements, result in similar response values.

3. The **equivalent static method** of all seismic codes gives adequate control over the inelastic torsional behaviour of the TU models, indicating that the existing structural **regularity provisions** are highly restrictive. The equivalent static force procedure gives good control even in asymmetric models with high values of static eccentricity, which are characterised as non-regular, indicating that the equivalent static method is a simple and efficient method to design TU structures. The same conclusion was also reached by Paulay (1996), who indicated that, instead of using a relatively involved dynamic analysis to access the ductile torsional response of systems, a procedure based on the equivalent static method addressing displacement control is adequate.

10.2 SUBJECTS RECOMMENDED FOR FURTHER RESEARCH

The investigation of the inelastic seismic response of plan-eccentric structures prone to coupled translational and torsional oscillations is highly specialised and complex. The results of the inelastic dynamic analyses carried out in this study indicated that many factors related to both the structural modelling and the design of the models have a significant influence on the variation observed in their response. Therefore, due to the highly dependent nature of such investigations on numerous parameters, further parametric research studies are required in order to understand better the inelastic seismic response of the TU multi-storey structures. Little research is currently being conducted in relation to many such aspects, which require further detailed study in order to decide about the applicability of earlier simplified studies. Therefore, the subjects requiring further research to be carried out can be summarised as follows:

1. In order to simplify the analysis of the structures, all seismic codes specify that the resisting structural elements oriented in two orthogonal directions can be designed separately by considering only the earthquake ground motion parallel to the elements. Most of the previous studies included structural elements only in the

lateral direction and considered uni-directional earthquake ground motion input. The results of the inelastic dynamic analyses carried out in Chapter 8 indicated that the inelastic seismic response of TU models incorporating transverse structural elements differs from the response of similar TU models without transverse frames. The same conclusion was also reached from the series of research studies carried out by Paulay (1996, 1997a, 1997b, 1997c, 1997d, 1998a, 1998b), which indicated the great difference in the inelastic response of TU models with and without transverse structural elements. Furthermore, the inclusion of the second earthquake component influenced the response of the **transverse frames** indicating the importance of a **bi-directional earthquake excitation.** Therefore, the applicability of employing models without transverse structural elements subjected to uni-directional seismic loading should be further investigated by examining the response of TU models including transverse frames. The results of such a study could improve the above mentioned uni-directional design approach, by modifying the design seismic loads to account for the influence of the transverse frames and of the bi-axial earthquake excitation.

2. Moreover, the research studies carried out by Paulay (1996, 1997a, 1997b, 1997c, 1997d, 1998a, 1998b) indicated that simple approaches based on the **equivalent static method and on force-displacement relationships** could lead to a better consideration of the torsional effects in ductile structures. Based on the results of this study and on the research carried out by Paulay, increasing the torsional strength of the structural elements based on the magnification of the stiffness eccentricity with the use of the accidental eccentricity was proved not to be adequate. Therefore, more research should be carried out in order to prove the importance of a **displacement-based design method.**

3. In this study, the inelastic torsional behaviour of regularly asymmetric models was investigated and the centres of rigidity of the models were located in one vertical line. However, irregular structures arise frequently in practice due to architectural requirements, discontinued shear walls and "soft storeys" from large openings at the ground floor. Although the response of regular structures is easier to predict and their ability to survive a strong earthquake is much better than

irregular structures, designers often compromise structural symmetry for functional and aesthetic needs. Thus, further research should be carried out for **irregular structures** having their centres of rigidity in different locations for each storey. Little research has been carried out on this subject and the guidance provided by current seismic codes for the design of these buildings is not sufficient since the required modal analysis has been proved as inadequate by many researchers (Duan, 1991).

4. Further research could be also carried out for multi-storey TU models having a plan asymmetry in both orthogonal directions. **Bi-eccentric** multi-storey structures can also be found in practice and, therefore, it is of great importance to investigate the inelastic torsional behaviour of TU models incorporating transverse structural elements and having an asymmetric stiffness distribution in both the lateral and the transverse direction. The influence of a bi-axial earthquake ground motion excitation could be further examined in relation to such models.

5. The inelastic seismic response and the effective design of multi-storey regularly asymmetric frame models was investigated in this study while many high-rise buildings include shear wall elements to resist the lateral load. The behaviour of shear walls is different from the behaviour exhibited by moment-resisting frames and the inelastic seismic response of **high-rise regularly asymmetric shear wall buildings** could also be an area for future research.

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APPENDIX A

NEW ZEALAND STANDARD CODE (1992)

A.1 METHODS OF ANALYSIS

The New Zealand Standard code provides three types of analysis for seismic design: the equivalent static method, the modal response spectrum analysis and the numerical integration time-history analysis. The modal response spectrum and the numerical integration time-history analyses are used when the equivalent static method cannot be employed. Where the horizontal regularity criteria (Section A.2.1) are not met, such analyses should be three-dimensional. The equivalent static method may be used only where **at least one** of the following criteria is satisfied:

1. The height between the base and the top of the structure does not exceed 15 m.
2. The fundamental period of the structure does not exceed 0.45 sec.
3. The structure satisfies the horizontal and vertical regularity requirements (Section A.2) and its fundamental period is less than 2 seconds.

A.2 STRUCTURAL REGULARITY

A.2.1 Horizontal Regularity

Two-dimensional modal response spectrum analysis and two-dimensional numerical integration time-history analysis, or the equivalent static analysis, is allowed if the diaphragms do not contain abrupt variations in stiffness or major re-entrant corners, and if **one** of the following criteria is satisfied:

1. The horizontal distance between the CR at any level and the CM of all floors above must not exceed 0.3 times the maximum plan dimension of the structure at

that level, measured perpendicular to the direction of lateral forces, nor change sign over the height.

2. Under the action of equivalent static forces, the ratio of horizontal displacements at the ends of an axis transverse to the direction of the applied lateral forces should be in the range 3/7 to 7/3.

The purpose of these requirements is to use 3D analyses in order to ensure that contributions of torsional dynamic response are not seriously underestimated when eccentricities are significant or when torsional stiffness is inadequate.

A.2.2 Vertical Regularity

The equivalent static method may be used if the lateral displacement at each floor level is reasonably proportional to the height of that level above the base.

A.3 ACCIDENTAL ECCENTRICITY AND TORSIONAL EFFECTS

A.3.1 Accidental Eccentricity

In addition to the eccentricity described in the horizontal regularity requirements (Section A.2.1), an additional accidental eccentricity must be considered for each earthquake direction, as an allowance for uncertainties in the determination of the centre of stiffness, the centre of mass, and the characterisation of the ground motions. The accidental eccentricity is measured from CM and, for forces applied parallel to the principal orthogonal axes of the building, it is taken as ± 0.1 times the building plan dimension at right angles to the loading direction.

A.3.2 Torsional Effects

When the horizontal regularity provisions (Section A.2.1) are satisfied and 2D modal response spectrum analysis has been used to evaluate translational effects, the static lateral method of analysis may be used to estimate torsional effects. In other cases, 3D time-history analysis should be used. When static analysis is used for the torsional effects, the applied torque at each level should be based either on forces calculated by the equivalent static method, or on the combined storey inertial forces obtained from 2D modal spectrum analysis for translation. Torsional effects must be combined with translational effects by direct summation so as to produce the most adverse effect on each member considered.

A.4 SEISMIC DEFLECTIONS AND P-DELTA EFFECTS

A.4.1 Seismic Deflections

The seismic deflections u_i at various floor levels, caused by the equivalent static forces, are determined by elastic analysis. Since the equivalent static method simulates only the first mode of response, it overestimates displacements that may be reduced by a scaling factor of 0.85 for buildings with six or more storeys. When the equivalent static method or the modal response spectrum method is used, lateral displacements and corresponding interstorey displacements (drifts) at the ULS may be estimated by multiplying the elastic displacements by the structural ductility factor μ. This linear scaling of displacement generally is satisfactory for buildings of medium height (6-8 storeys). However, interstorey displacements at the development of the expected maximum ductility are likely to be significantly underestimated in the lower storeys of taller buildings. In such structures, no linear relationship between the ultimate interstorey displacements exists. Where the equivalent static or the modal response spectrum analysis is used, the interstorey deflections must not exceed the following fractions of the corresponding storey height.

(i) 0.020 for $h_n \leq 15\,\text{m}$

(ii) 0.015 for $h_n \geq 30\,\text{m}$

(iii) $0.020\text{-}0.005\dfrac{h_n - 15}{15}$ for $15\,\text{m} < h_n < 30\,\text{m}$

Where the numerical integration time history method incorporating member response is used, interstorey deflections must not exceed 0.025 of the corresponding storey height.

A.4.2 P-Delta Effects

Analysis for P-delta effects at the ULS must be carried out unless at least one of the following criteria is satisfied:

1. The period of the structure does not exceed 0.45 sec.
2. The height of the structure does not exceed 15 m and its period is less than 0.8 sec.
3. The target ductility factor does not exceed 1.5.
4. The ratio of interstorey displacement to the storey height does not exceed the value described in Equation (A.4.1) where u_i and u_{i-1} are the horizontal displacements at levels i and i-1, respectively, V_i is the design storey shear force, and $\sum_{j=i}^{N} W_j$ is the sum of the gravity loads above and including level i.

$$\frac{u_i - u_{i-1}}{h_i - h_{i-1}} \le \frac{V_i}{7.5\sum_{j=i}^{N} W_j} \tag{A.4.1}$$

For multi-storey buildings where capacity design is used to exclude column sway mechanisms, Equation (A.4.1) should apply between the base and the mid-height of the structure, whereas for other buildings should apply over the full height. Equation (A.4.1) emphasises that buildings designed for relatively small lateral force resistance, as quantified by the storey shear force, are more vulnerable to reductions of this resistance due to P-delta moments. With increased flexibility, ductility demand and duration of the seismic shaking, P-delta effects become more critical. When Equation (A.4.1) is not satisfied, a rational method of analysis should be used to increase the shear capacity of the affected storeys to compensate for P-delta moments.

A.5 EQUIVALENT STATIC METHOD

A.5.1 Base Shear Calculation

The structure is designed for a total base shear force V given by

$$V = CW \tag{A.5.1}$$

where W is the weight of the structure and C is the lateral force coefficient given by Equation (A.5.2) for SLS and Equation (A.5.3) for ULS

$$C = C_h(T,1)S_p RZL_s \tag{A.5.2}$$

$$C = C_h(T,\mu)S_p RZL_u \geq 0.03 \tag{A.5.3}$$

S_p is the structural performance factor (=0.67), R is the risk factor (=1.0 for ordinary structures), Z is the zone factor accounting for the seismicity of the area (0.6-1.2), L_s is the SLS factor (=1.0), L_u is the ULS factor (=1/6), and $C_h(T,\mu)$ is the basic seismic acceleration coefficient from the appropriate response spectral chart as a function of the fundamental period T and the ductility factor μ.

Where the equivalent static method is used, the fundamental translational period in the direction under consideration is calculated from the Rayleigh method,

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} W_i u_i^2}{g\sum_{i=1}^{n} F_i u_i}} \tag{A.5.4}$$

where F_i represents any lateral force and u_i are the elastic deflections calculated using the applied forces F_i. The elastic deflections may be calculated ignoring the effect of torsion. Unlike all other codes, the NZS code does not employ the force reduction factor to obtain the inelastic base shear from the elastic design spectrum. Instead, the inelastic design spectra for different levels of global displacement ductility are obtained by adjusting the inelastic base shear relative to the elastic response spectrum, in order to achieve an average displacement ductility demand of a SDOF system close to each target ductility level.

A.5.2 Equivalent Static Lateral Forces

At each level i of the building, the equivalent static force F_i is calculated from

$$F_i = 0.92V\frac{W_i h_i}{\sum_{j=1}^{N} W_j h_j} \tag{A.5.5}$$

where V is the base shear force (Equation (A.5.1)), W_i is the seismic weight, and h_i is the height of the level i, with an additional force F_t at the top of the structure

$$F_t = 0.08V \tag{A.5.6}$$

The equivalent static forces are applied through points eccentric to CM at each level as specified in the accidental eccentricity (Section A.3). The magnitude of the deflections found from the equivalent static analysis can be reduced by the appropriate scale factors. For buildings that have a soft or weak storey, this factor is equal to unity. For buildings with six or more storeys, the factor is equal to 0.85 while, for buildings with less than six storeys, this factor is found by interpolating between 1.0 for a single storey structure and 0.85 for a six-storey structure.

A.6 MODAL RESPONSE SPECTRUM METHOD

A.6.1 General

The modal response spectrum analysis is based on elastic structural behaviour and uses the elastic response spectral charts that provide the basic seismic acceleration coefficient $C_h(T,\mu)$ as a function of the modal period T and the ductility factor μ (=1 for SLS). When the modal response spectrum is used, the design spectrum is given for each mode by Equation (A.6.1) for the SLS and by Equation (A.6.2) for the ULS

$$C(T) = S_m C_h(T,1) S_p R Z L_s \qquad \text{(A.6.1)}$$

$$C(T) = S_m C_h(T,\mu) S_p R Z L_u \qquad \text{(A.6.2)}$$

The response spectrum scaling factor S_m is equal to unity for the SLS and for the ULS the greater of S_{m1} and S_{m2} is used

$$S_{m1} = \frac{C_h(T,\mu)}{C_h(T,1)} \qquad \text{for } T > 0.4 \text{ sec} \qquad \text{(A.6.3)}$$

$$S_{m1} = \frac{C_h(0.4,\mu)}{C_h(0.4,1)} \qquad \text{for } T \leq 0.4 \text{ sec} \qquad \text{(A.6.4)}$$

$$S_{m2} = \frac{K_m C W}{V} \qquad \text{(A.6.5)}$$

where C is given by Equations (A.5.2) and (A.5.3), V is the base shear force for the first mode and K_m=0.8 for regular or elastically responding structures, otherwise K_m=1. The factors R, Z and L_s, L_u are the risk, zone and limit state factors (Section A.5.1).

The modal response spectrum analysis usually leads to smaller base shear than the equivalent static method analysis that gives satisfactory results when restricted to regular structures. The scaling factor S_{m2} is used to ensure that results of the modal response spectrum analysis are not excessively smaller than those derived from the equivalent static method. The scaling is applied to the results calculated in each mode.

When the fundamental period is greater than 1.0 sec and μ=6, the SLS may control the minimum strength level, which may be well less than the strength required to resist wind forces. Such cases should not divert attention from the important detailing requirements to ensure that large ductility can be developed. For these structures, P-delta effects may be significant in ULS. A sufficient number of modes should be included to ensure that at least 90% of the mass is participating.

A.6.2 Three-Dimensional Analysis

For 3D analysis, except for buildings with rigid floor diaphragms, the location and distribution of mass is adjusted to account for the actual and accidental eccentricity in each direction of the seismic forces. For rigid floor diaphragms, the effects of eccentricity for earthquake actions may be estimated by:

1. The general procedure described above with CM adjusted. The rotational inertia of the floor with respect to CM needs not to be modified to account for the altered distribution of mass.
2. The position and distribution of mass need not to be adjusted, but the line of action of the inertia forces must be taken eccentric to CM.

A.7 NUMERICAL INTEGRATION TIME-HISTORY ANALYSIS

This method is used to determine the strength requirements, deflections and forces generated on parts of a structure, to ensure that the ductility demand does not exceed the limits specified, and to verify that the capacity design requirements are satisfied. P-delta effects are either included in the analysis, or additional strength based on interstorey displacements is provided. The chosen earthquake records (at least three) are scaled such that over the period range of the structure, the 5% damped spectrum of the earthquake records does not differ significantly from the code design spectrum. The earthquake records for the ULS should contain at least 15 seconds of strong shaking, or have a strong shaking duration at least five times the fundamental period of the structure, whichever is greater. Scaling factors also are recommended if this method is used for design.

APPENDIX B

EUROCODE 8 (1993)

B.1 STRUCTURAL REGULARITY

For the purpose of seismic design, building structures are distinguished as regular and non-regular. This distinction has implications for the structural model, the method of analysis and the value of the behaviour factor q. The structural model can be either a simplified planar or a spatial one, the method of analysis can be either a simplified modal or a multi-modal one, and the behaviour factor q can be decreased depending on the type of the non-regularity in elevation. The consequences of structural regularity on seismic design are summarised in Table B.1.1. The criteria describing regularity in plan and in elevation are to be considered as necessary conditions (Sections B.1.1 and B.1.2).

PLAN REGULARITY	ELEVATION REGULARITY	SIMPLIFIED MODEL	SIMPLIFIED ANALYSIS	BEHAVIOUR FACTOR
Yes	Yes	Planar	Simplified	Reference
Yes	No	Planar	Multi-modal	Decreased
No	Yes	Spatial	Multi-modal	Reference
No	No	Spatial	Multi-modal	Decreased

Table B.1.1 Consequences of the structural regularity on seismic design.

B.1.1 Criteria for Regularity in Plan

1. Lateral stiffness and mass distribution are symmetrical in plan with respect to two orthogonal directions
2. The plan configuration is compact and the total dimension of re-entrant corners or recesses in one direction does not exceed 25% of the overall external dimension.
3. The in-plane stiffness of the floors is large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor has a small effect on the forces acting on the vertical elements.
4. Under the seismic forces applied with the accidental eccentricity, the maximum storey displacement in the direction of the seismic forces does not exceed the average storey displacement by more than 20%.

B.1.2 Criteria for Regularity in Elevation

1. All lateral load resisting elements run without interruption from their foundations to the top of the building.
2. Both the lateral stiffness and the mass of individual storeys remain constant or reduce gradually, without abrupt changes from the base to the top.
3. In framed buildings, the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys.
4. In case of gradual setbacks preserving axial symmetry, the setback at any floor should not be greater than 20% of the previous plan dimension in the direction of the setback.
5. In case of single setback within the lower 15% of the building height, the setback should not be greater than 50% of the previous plan dimension.
6. In case the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys should not be greater than 30% of the plan dimension of the first storey, and the individual setbacks are not greater than 10% of the previous plan dimension.

B.2 ACCIDENTAL TORSIONAL EFFECTS

In addition to the actual eccentricity, in order to cover uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated CM at each floor level is displaced from its nominal location in each direction by an additional accidental eccentricity equal to 0.05 the floor plan dimension perpendicular to the seismic direction.

B.3 APPROXIMATE ANALYSIS FOR TORSIONAL EFFECTS

For buildings not satisfying the criteria for regularity in plan (Section B.1.1), but fulfilling **one** of the following sets of conditions given as criterion 1 and criterion 2, an approximate analysis for torsional effects can be used.

Criterion 1

1. The building has well distributed and relatively rigid cladding and partitions.
2. The building height does not exceed 10 m.
3. The building aspect ratio (height/length) in both directions does not exceed 0.4.

Criterion 2

1. The in-plane stiffness of the floors is large in comparison with the lateral stiffness of the vertical elements, so that rigid floor diaphragm behaviour may be assumed.
2. The centres of stiffness and mass are each located on a vertical line, if all lateral load resisting elements run without interruption from their foundations to the top of the building and if the deflected shapes of the individual systems under horizontal loads do not differ too much.
3. If the previous condition is met, the common position of the centres of stiffness of all storeys may be calculated as the centre of some quantities, proportional to a system of forces, having the distribution specified in Section B.6.3 and producing a unit displacement at the top of the individual lateral load resisting system.

The analysis can be performed using two planar models, one for each principal direction. The torsional effects are determined separately for those two directions. The horizontal forces F_i are determined according to Section B.6.3 or B.7.2. The horizontal force F_i at storey i is displaced from its nominal location in relation to mass by an additional eccentricity e_1, approximated as the lower of the following values

$$e_1 = 0.1(a+b)\sqrt{\frac{10e_s}{b}} \leq 0.1(a+b) \tag{B.3.1}$$

$$e_1 = \frac{1}{2e_s}\left[r_m^2 - e_s^2 - r_k^2 + \sqrt{(r_m^2 + e_s^2 - r_k^2)^2 + 4e_s^2 r_k^2}\right] \tag{B.3.2}$$

where e_1 is the additional eccentricity taking account of the dynamic effect of simultaneous translational and torsional vibrations, e_s is the actual eccentricity between the centre of stiffness and the centre of mass, r_m^2 is the square of the mass radius of gyration, equal to $(a^2+b^2)/12$, and r_k^2 is the square of the stiffness radius of gyration, the ratio of the storey torsional and lateral stiffnesses. The additional eccentricity e_1 may be neglected, if r_k^2 exceeds $5(r_m^{\ 2}+e_s^2)$. The torsional effects may be set as the envelope of the effects resulting from an analysis for two static loadings, considering the torsional moments M_t due to the two eccentricities

$$M_t = F_i(e_s + e_a + e_1) \tag{B.3.3}$$

$$M_t = F_i(e_s - e_a) \tag{B.3.4}$$

where e_a is the accidental eccentricity of storey mass $(\pm 0.05b)$.

B.4 APPROXIMATE FORMULAE FOR THE FUNDAMENTAL PERIOD

For buildings with height up to 80 m, the fundamental period may be approximated by

$$T = C_t H^{3/4} \tag{B.4.1}$$

where C_t is equal to 0.075 for moment resisting concrete frames, and H is the height of the building in m. Alternatively, the estimation of T can be made by

$$T = 2\sqrt{u} \tag{B.4.2}$$

where u is the top displacement (m) due to the gravity loads applied horizontally.

B.5 METHODS OF ANALYSIS

The reference method for determining the seismic effects is the modal response analysis, using a linear-elastic model of the structure and the design spectrum of the code. Depending on the structural characteristics of the building, the simplified modal response spectrum analysis or the multi-modal response spectrum analysis may be used. Other alternative methods are the power spectrum analysis, the non-linear time-history analysis, and the frequency domain analysis. If a non-linear analysis is used, the amplitudes of the accelerograms derived for the reference return period are multiplied by the importance factor γ_I of the building.

B.6 SIMPLIFIED MODAL RESPONSE SPECTRUM ANALYSIS

B.6.1 General

This method of analysis is applied to buildings that can be analysed by two planar models and whose response is not significantly affected by contributions from higher modes of vibration. These requirements are satisfied by buildings which meet the regularity criteria in plan and in elevation (Sections B.1.1 and B.1.2) or meet the criteria for the approximate torsional analysis (Section B.3). Additionally, these buildings should have fundamental periods of vibration in the two main directions less than the following values

$$T \leq 4T_c \tag{B.6.1}$$

$$T \leq 2.0\text{sec} \tag{B.6.2}$$

where T_c is a parameter describing the elastic response spectrum, depending on the subsoil class (=0.40 for rock soil).

B.6.2 Base Shear Force

The seismic base shear force F_b for each main direction is determined as

$$F_b = S_d(T_1)W \tag{B.6.3}$$

where T_1 is the fundamental translational period for translational motion in the direction considered, $S_d(T_1)$ is the ordinate of the design spectrum at period T_1 and W

is the total building weight. To determine the period of both planar models of the building, approximate expressions based on methods of structural dynamics (e.g. by the Rayleigh method) may be used. For preliminary design purposes, the approximate period expressions given in Section B.4 may be used.

B.6.3 Distribution of Horizontal Seismic Forces

The fundamental mode shapes of both planar models of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the building height. The seismic action effects are determined by applying to the two planar models horizontal forces F_i acting on all storey masses. The horizontal forces F_i are distributed to the lateral load resisting system assuming rigid floors. The forces are determined assuming the entire mass of the structure as a substitute mass of the fundamental mode of vibration, hence

$$F_i = F_b \frac{u_i W_i}{\sum u_j W_j} \tag{B.6.4}$$

where F_i is the horizontal force acting on storey i, F_b is the seismic base shear, u_i, u_j are the displacements of masses m_i, m_j in the fundamental mode shape, and W_i, W_j are the weights of the masses m_i, m_j. When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, F_i are given by

$$F_i = F_b \frac{h_i W_i}{\sum h_j W_j} \tag{B.6.5}$$

where h_i, h_j are the heights of the masses m_i, m_j above the level of application of the seismic action (foundation).

B.6.4 Torsional Effects

For symmetric distribution of lateral stiffness and mass, the accidental torsional effects may be accounted for by amplifying the action effects in the individual load resisting elements with a factor δ given by

$$\delta = 1 + 0.6 \frac{x}{L_e} \tag{B.6.6}$$

where x is the distance of the element under consideration from CM measured perpendicularly to the seismic direction considered and L_e is the distance between the two outermost lateral load-resisting elements. Whenever the conditions given in Section B.3 are met, the approximate analysis of torsional effects can be applied.

B.7 MULTI-MODAL RESPONSE SPECTRUM ANALYSIS

B.7.1 General

This analysis is applied for buildings that cannot be analysed by the simplified response spectrum analysis. For buildings complying with the criteria for plan regularity, or with the regularity criteria of the approximate analysis, the analysis can be performed using two planar models, one in each direction. Buildings not complying with these criteria are analysed using a spatial model. The responses from all modes of vibration contributing significantly to the global response are taken into account by demonstrating that the sum of the effective modal masses for the modes considered amounts to at least 90% of the total mass. Additionally, all modes with effective modal masses greater than 5% of the total mass are considered. If the previous two conditions cannot be satisfied (e.g. in buildings with a significant contribution from torsional modes), the minimum number k of modes to be considered in a spatial analysis should satisfy the conditions

$$k \geq 3\sqrt{n} \tag{B.7.1}$$

$$T_k \leq 0.20\,sec \tag{B.7.2}$$

where n is the number of storeys and T_k is the period of vibration of mode k.

B.7.2 Combination of Modal Responses

The response of two vibration modes i and j (translational and torsional modes) is independent of each other, when their periods T_i and T_j satisfy the condition

$$T_j \leq 0.9T_i \tag{B.7.3}$$

Whenever all relevant modal responses can be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as

$$E_E = \sqrt{\sum E_{Ei}^2} \tag{B.7.4}$$

where E_E is the seismic action effect under consideration (force, displacement, etc.) and E_{Ei} is the value of the seismic action effect due to the vibration mode i. If Equation (B.7.3) is not satisfied, more accurate procedures for the combination of the modal maximum values (e.g. the CQC) have to be applied.

B.7.3 Torsional Effects

Whenever a spatial model is used for the analysis, the accidental torsional effects may be determined as the envelope of the effects resulting from a static loading analysis, consisting of torsional moments M_i about the vertical axis of each storey i

$$M_i = e_{ai} F_i \tag{B.7.5}$$

where e_{1i} is the accidental eccentricity of storey mass ($\pm 0.05b$) and F_i is the horizontal force acting on storey i for all relevant directions. Whenever two separate planar models are used for the analysis, the torsional effects may be accounted for by applying the rules of structural regularity (Section B.1) or the conditions of the simplified torsional analysis (Section B.3) to the action effects computed according to the combination of modal responses.

B.8 ALTERNATIVE METHODS OF ANALYSIS

To use the alternative methods of analysis, the sum of the horizontal shear forces at all supports in each of the two orthogonal directions should not be less than 80% of the corresponding sum obtained by multi-modal analysis. Where the sum in either direction is less than 80% of the value for multi-modal analysis, the computed values of all response variables must be scaled proportionally by the scale factor required to bring the base shear to the value needed.

B.8.1 Power Spectrum Analysis

A linear stochastic analysis can be performed either by using modal analysis or frequency dependent response matrices using as input the acceleration power spectrum. The elastic action effects are defined as the 50% fractile of the probability distribution of the peak response in a time interval equal to the assumed duration of the motion. The design values are determined by dividing these elastic effects by the ratio of the ordinate of the design spectrum corresponding to the fundamental period of the building, multiplied by g.

B.8.2 Time History Analysis

The time dependent response of the structure can be obtained through direct numerical integration of its differential equations of motion, using selected accelerograms to represent the ground motion. The mean values of the response spectra of the selected accelerograms should match closely the response spectra of the code under the specific design conditions.

B.8.3 Frequency Domain Analysis

The seismic action input is the same as in the time-history analysis, but with each accelerogram cast in the form of Fourier summation. The response is obtained by convolving over the frequency domain the harmonic components of the input with their respective frequency response matrices or functions. The elastic action effects are defined as the mean values of the peak responses calculated for each accelerogram. The design values are determined by dividing the elastic effects by the ratio of the ordinate of the elastic response spectrum to the ordinate of the design spectrum corresponding to the fundamental period of the building, multiplied by g.

B.9 DISPLACEMENT ANALYSIS

The displacements induced by the design seismic action are calculated on the basis of the elastic deformation of the system with the following simplified expression

$$d_s = q_d \, d_e \, \gamma_I \tag{B.9.1}$$

where d_s is the displacement of a point of the structural system induced by the design seismic action, q_d is the displacement behaviour factor assumed equal to q unless otherwise specified, d_e is the displacement of the same point as determined by a linear analysis based on the design response spectrum, and γ_I is the importance factor (=1 for ordinary structures). When determining the displacement d_e, the torsional effects of the seismic action must be taken into account.

B.10 SAFETY VERIFICATIONS

B.10.1 Second Order Effects

In the ULS, second P-delta effects need not be considered when the following condition is fulfilled in all storeys:

$$\theta = \frac{W\,\Delta}{V\,h} \leq 0.10 \tag{B.10.1}$$

where θ is the interstorey drift sensitivity coefficient, W_t is the total gravity load at and above the storey considered, in accordance with the assumptions made for the computation of the seismic action effects, Δ is the design interstorey drift evaluated as the difference of the average lateral displacements at the top and bottom of the storey (Section B.9), V is the total seismic storey shear and h is the interstorey height. In cases when $0.1<\theta<0.2$, the second order effects can approximately be taken into account by increasing the seismic action effects by a factor equal to $1/(1-\theta)$. The value of the coefficient θ must not exceed 0.3.

B.10.2 Limitation of Interstorey Drift

In the SLS, for buildings having non-structural elements of brittle materials

$$\frac{\Delta}{v} \leq 0.004h \tag{B.10.2}$$

For buildings with non-structural elements not interfering with structural deformations

$$\frac{\Delta}{v} \leq 0.006h \tag{B.10.3}$$

where h is the storey height, v is the reduction factor to take into account the lower return period of the seismic event associated with the SLS (=2 for ordinary structures) and Δ is the design interstorey drift (Equation (B.9.1)).

APPENDIX C

UNIFORM BUILDING CODE (1994)

C.1 METHODS OF ANALYSIS

The methods of analysis provided are the static lateral force method and the dynamic method. The static lateral force method may be used in the following cases:

1. All structures, regular and irregular, in seismic zone 1.
2. Regular structures under 240 ft (73 m) in height with lateral force resistance provided by a bearing wall system, building frame system, moment-resisting frame system or dual system, except structures located on very soft soil and having a period greater than 0.7 seconds.
3. Irregular structures not more than five storeys or 65 ft (20 m) in height.
4. Structures having a flexible upper portion supported on a rigid lower portion where both portions are regular and the average storey stiffness of the lower portion is at least 10 times the average storey stiffness of the upper portion. The period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

The dynamic method may be used for any structure. However, it must be used for structures over 240 ft (73 m) in height, irregular structures of over 5 storeys or 65 ft (20 m) in height, and structures with dissimilar structural systems located in seismic zones 3 and 4.

C.2 CONFIGURATION REQUIREMENTS

C.2.1 Vertical Structural Irregularities

1. Stiffness irregularity or soft storey: a lateral storey stiffness is less than 70% of that in the storey above, or less than 80% of the average stiffness of the three storeys above.
2. Weight (mass) irregularity: the effective mass of any storey is more than 150% of the effective mass of an adjacent storey.
3. Vertical geometric irregularity: the horizontal dimension of the lateral force-resisting system in any storey is more than 130% of that of an adjacent storey.
4. In-plane discontinuity in vertical lateral force-resisting element: an in-plane offset of the elements greater than their length.
5. Discontinuity in storey strength : the storey strength is less than 80% of the storey above. The storey strength is the total strength of all seismic resisting elements sharing the storey shear.

C.2.2 Plan Structural Irregularities

1. Torsional irregularity (considered for rigid diaphragms): the maximum storey drift, computed including accidental torsion at one end of the structure transverse to an axis is more than 1.2 times the average storey drifts of the two ends of the structure.
2. Re-entrant corners: both projections of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction.
3. Diaphragm discontinuity: diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50% of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50% from one storey to the next.
4. Out-of-plane offsets: discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.
5. Nonparallel systems: the vertical lateral load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting elements.

C.3 STATIC FORCE PROCEDURE

C.3.1 Design Base Shear

The structure is designed for a total base shear force given by

$$V = \frac{ZIC}{R_w} W \qquad (C.3.1)$$

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \qquad (C.3.2)$$

where Z is the seismic zone factor that varies from 0.075 for zone 1 to 0.40 for zone 4, I is the importance factor (=1 for ordinary buildings), S is the site coefficient (=1 for rock and stiff soils), and R_w is a numerical coefficient (=12 for RC moment-resisting frame systems). The value of C may be used for any structure without regard to soil type and structural period. The minimum value for the ratio C/R_w is 0.075.

C.3.2 Structural Period

The structural period T (seconds) may be approximated by

$$T = C_t (h_n)^{3/4} \qquad (C.3.3)$$

where h_n is the total height of the structure and C_t is equal to 0.030 for RC moment-resisting frames when the height is calculated in feet. If the height is calculated in meters, C_t is equal to 0.075.

The fundamental period T may be also calculated using the structural properties and the deformational characteristics of the resisting elements in a properly substantiated analysis. This requirement is satisfied by the following formula

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i u_i^2}{g \sum_{i=1}^{n} f_i u_i}} \qquad (C.3.4)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of the formulas or any other rational distribution. The elastic deflections u_i are calculated using the applied lateral forces f_i.

C.3.3 Vertical Distribution of Force

The total force is distributed over the height of the structure according to

$$V = F_t + \sum_{i=1}^{n} F_i \tag{C.3.5}$$

where the concentrated force F_t at the top is determined by

$$F_t = 0.07TV \leq 0.25V \tag{C.3.6}$$

The T value used for calculating F_t may be the period that corresponds with the design base shear as computed using Equation (C.3.1). F_t need not exceed $0.25V$ and may be considered as zero where T is 0.7 seconds or less. The remaining portion of the base shear is distributed over the height of the structure, including level n, according to

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{C.3.7}$$

At each level designated as x, the force F_x is applied over the building area in accordance with the mass distribution at that level. Stresses in each structural element are calculated as the effect of forces F_x and F_t applied at the appropriate levels.

C.3.4 Horizontal Distribution of Shear

The design storey shear V_x in any storey is the sum of the forces F_t and F_x above that storey. V_x is distributed to the various elements of the vertical lateral force-resisting system in proportion to their rigidities. To account for the uncertainties in locations of loads, the mass at each level is assumed to be displaced from the calculated CM in each direction a distance equal to 5% of the building dimension at the level perpendicular to the direction of the force under consideration. The effect of this displacement on the storey shear distribution is considered.

C.3.5 Horizontal Torsional Moments

Provision is made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. Diaphragms are considered flexible when their maximum lateral deformation is more than two times the average storey drift of the associated storey. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm under lateral load with the storey drift of the adjoining vertical elements under equivalent tributary lateral load.

The torsional design moments at a given storey is the moment resulting from eccentricities between applied design lateral forces at levels above that storey and the vertical resisting elements in that storey plus an accidental torsion. The accidental torsional moment is determined by assuming that the mass is displaced as required by Section C.3.4. Where torsional irregularity exists (Section C.2), the effects are accounted for by increasing the accidental torsion at each level by the factor A_x

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 \leq 3.0 \qquad \text{(C.3.8)}$$

where δ_{max} is the maximum displacement at level x and δ_{avg} is the average of the displacements at the extreme points of the structure at level x. The value of A_x need not exceed 3.0. The more severe loading for each element is considered for the design.

C.3.6 Overturning Moment

The code requires that overturning moments are determined at each level of the structure. The overturning moments are determined using the seismic forces F_x and F_t (Equation (C.3.6) and (C.3.7)) acting on levels above the level under consideration. Hence, the overturning moment M_x at level x of the building is given by

$$M_x = F_t(h_n - h_x) + \sum_{i=x+1}^{n} F_i(h_i - h_x) \qquad \text{(C.3.9)}$$

where $x = 0, 1, 2, \ldots, n\text{-}1$.

C.3.7 Storey Drift Limitation

Storey drift is the relative displacement between consecutive floor levels produced by the design lateral forces and includes calculated translational and torsional deflections. The calculated storey drift must not exceed $0.04/R_w$ or 0.005 times the storey height for structures having a fundamental period of less than 0.7 seconds. For structures having a fundamental period of 0.7 seconds or greater, the calculated storey drift must not exceed $0.03/R_w$ or 0.004 times the storey height. In addition, the drift limitations are not subject to the 80% limit mentioned in the period calculation, nor the limitation imposed on the ratio C/R_w cited in relation with the Equation (C.3.2).

C.3.8 P-Delta Effects

The P-Δ effect refers to the additional moment produced by the vertical loads and the lateral displacements of columns and other resisting elements. The code specifies that the resisting member forces and moments, as well as the storey drift induced by the P- Δ effect, is considered in the evaluation of overall structural frame stability. The P- Δ effect need not to be considered when the ratio of the secondary moment resulting from the storey drift to the primary moment due to the seismic lateral forces, for any storey, does not exceed 0.10. This ratio θ_x may be evaluated as the product of the total gravity loads considered for seismic analysis above the storey times the seismic storey drift, divided by the product of the seismic shear force times the height. The ratio θ_x at level x of the secondary moment M_{xs} resulting from P- Δ effects, and the primary moment M_{xp} due to seismic lateral forces, is calculated from

$$\theta_x = \frac{M_{xs}}{M_{xp}} = \frac{P_x \Delta_x}{V_x h_x} \tag{C.3.10}$$

where P_x is the total seismic weight at level x and above, Δ_x is the drift at storey x, V_x is the shear force at storey x and h_x is the height of the storey x. In seismic zones 3 and 4, P-Δ effects need not be considered when the storey drift does not exceed $0.02/R_w$.

C.4 DYNAMIC LATERAL FORCE PROCEDURE

C.4.1 Ground Motion

The ground motion representation could be one of following:

1. The response spectra given in the code.
2. Elastic design response spectra developed for the specific site.
3. Ground motion time-histories developed for the specific site representative of actual ground motions, which approximate the site design spectrum.

4. The vertical component of ground motion is defined by scaling corresponding horizontal accelerations by a factor of two thirds.

C.4.2 Analysis Procedures

A mathematical model of the physical structure represents the spatial distribution of the mass and stiffness of the structure to an extent, which is adequate for the calculation of its dynamic response. A 3D model is used for the dynamic analysis of structures with highly irregular plan configurations and rigid diaphragms. The code indicates that an elastic response spectrum analysis may be used, in which all the significant maximum modal contributions are combined in a statistical manner to obtain the total structural response. At least 90% of the mass of the structure must be included in the calculation of response of each principal horizontal direction. A time-history response analysis may be also used, in which the dynamic response of the structure to a specific ground motion is obtained at each increment of time.

C.4.3 Scaling of Results

For irregular buildings, if the base shear calculated by dynamic analysis is less than the base shear calculated by the static lateral force procedure, it must be scaled up to match 100% of the static base shear. For regular buildings, the dynamic base shear must be scaled up to match 90% of the static base shear, but it must not be less than 80% of the value obtained when Equation (C.3.3) is used to determine the period. The code also stipulates that the base shear for a given direction, determined using dynamic analysis, need not exceed the value obtained by the lateral static force method. Therefore, in this case, the base shear force and all the response parameters may be scaled down to the value obtained by the static method.

C.4.4 Torsional Effects

The analyses account for torsional effects, including accidental torsional effects as prescribed in Section C.3.5. Where 3D models are used for analysis, the effects of accidental torsion are accounted for by appropriate adjustments in the model such as adjustment of mass locations.