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SEISMIC DESIGN OF STEEL STRUCTURES

by

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SEISMIC DESIGN OF STEEL STRUCTURES

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ABSTRACT

One of the most important disasters that humanity suffered throughout the history is earthquake. Although it is impossible to overcome earthquake, protection from its harms is possible by taking some precautions. Structural steel behaves the best for earthquake resistance among all structural materials, but considerable care is needed in design and detailing of framing systems.

The collapse mechanism influences the energy dissipation capacity of the structure. Therefore, it plays a very important role in seismic design of steel structures. The case in which a framed structure fails according to global type mechanism can be adopted as the reference case, because it is considered to exhibit enough ductility to withstand severe earthquakes. In the thesis, a computer program is developed and verified based on the method of global mechanism in ECCS recommendations. The program is used on the structures which are preliminary designed by elastic or plastic theories. It ensures the structures to collapse at global mechanism. In the program, if the collapse mechanism of the system is different from global mechanism, the plastic moments of columns are modified by an amplification factor. Consequently, new column sections satisfying global mechanism are determined according to the increased plastic moments of columns.

ÖZET

Tarih boyunca insanlığın en çok acı çektiği felaketlerden biri depremdir. Depremi önlemek mümkün değildir ama alınacak bir takım tedbirlerle zararlarından korunulabilir. Yapı türleri içinde, çelik yapılar depreme karşı en iyi davranışı gösterenleridir ancak yine de bir takım önlemler almak zorunludur.

Göçme mekanizması, yapıların enerji emme kapasitesini etkiler. Dolayısıyla da çelik yapıların depreme karşı projelendirilmesinde büyük rol oynar. Kiriş uçlarında ve en alt kolonların alt uçlarında plastik mafsal oluşması (global mekanizma) şeklinde göçme öngörülen çerçeve binalar referans olarak ele alınabilir. Çünkü, bu yapılar büyük deprem yükü altında bile yeterli sünekliği gösterebilmektedir. Bu çalışmada, ECCS'nin tavsiyeleri arasında yer alan global mekanizma metodu esas alınarak bir bilgisayar programı geliştirilmiş ve çalışırlığı gösterilmiştir. Bu program, elastik veya plastik teoriye göre ön boyutlandırması yapılmış yapılar üzerinde kullanılabilir. Söz konusu program, yapılarda global mekanizmanın gerçekleşmesine imkan vermektedir. Eğer sistemin göçme mekanizması global mekanizmadan farklı ise, kolonların plastik momentleri programın hesap ettiği çarpan kullanılarak yeniden belirlenir. Sonuç olarak, belirlenen kolon plastik momentlerine göre hesaplanmış ve global mekanizmayı sağlayan yeni kolon boyutları seçilir.

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LIST OF SYMBOLS

F_k	Horizontal force acting at k'th storey
h_k	Elevation of k'th storey
$M_{pb,jk}$	Plastic moment of j'th beam of k'th storey
$M^{(r)}_{p,ii_m}$	Plastic moment of i'th column of i_m'th storey
$M_{R.b}$	Resisting moments of the beams
$M_{R.c}$	Resisting moments of the columns
n_b	Number of bays
n_c	Number of columns
n_p	Number of storeys
W_{px}	Plastic modulus
Z	Plastic section modulus
α	Amplification coefficient
α_{cr}	Critical elastic multiplier of vertical loads
α_u	Kinematically admissible multiplier for global type mechanism
γ	Slope of softening branch
σ_y	Yield stress in pure tension
τ_y	Yield stress in pure shear

I. INTRODUCTION

Designing for earthquake resistance is difficult, because the fundamental concept of earthquake resistance is different from the design for other loadings, such as wind pressure or gravity loads, since it involves a number of cyclic reversals. Earthquake resistant design of steel structures has been developing in the last years by means of analytical and experimental researches. However, the codes do not include many of the aspects of these research results. The main objective of this thesis is to present the basic principles for the design and construction of seismic resistant steel structures and apply the design method of Mazzolani and Piluso which is based on plastic analysis and global ductility.

Seismic action results from the vibration of the soil transmitted to the structure during the earthquake. When the mass of the structure responds to the earthquake, inertia forces are induced in the structure. Therefore the response of the structure to earthquakes depends on both the ground motion and on the characteristics of the structure. The ground motion is dependent on many factors, involving the seismicity of the area. The main characteristics of the structure which influence its response to seismic action are; its mass and regularity. Regular structures in both plan and elevation behaves better during earthquakes.

Although structural steel is in many ways an ideal material for earthquake resistance, care should be taken in design and detailing of framing systems and connections. In addition, different structural systems are used in order to absorb dissipated energy during earthquake which are, concentrically braced frames, moment resisting frames and eccentrically braced frames.

Elastic analysis of structures is important to study its performance. However, if the load is increased until yielding occurs, at some locations the structure goes elastic-plastic deformations, and on further increase a fully plastic condition is reached. Some design methods based on global ductility of steel and plastic analysis developed by different researchers and engineers for earthquake resistance of steel structures. One of these methods is developed by Mazzolani and Piluso.

According to this method, the collapse mechanism plays a very important role in seismic design of structures. In order to obtain a collapse mechanism of global type in framed structures, flexural strength of columns has to be greater than flexural strength of beams. Therefore, an algorithm is developed to ensure that the mechanism prescribed other than the global mechanism can be eliminated. At the end of the thesis, a worked example with the method of Mazzolani and Piluso on a moment resisting frame is given.

II. THE LESSONS LEARNED FROM PAST EARTHQUAKES

2.1. DAMAGE STUDIES

The knowledge in seismic design has developed as a result of analytical and experimental research and experience gained from past earthquakes. Lessons learned from past earthquakes have been the most important source among all others, because earthquakes perform the most realistic laboratory tests on the buildings.

Engineers are most accustomed to static loads. One of the most important lessons learned from damage surveys is the difference in failure patterns between static loads applied in a single direction and those due to cyclic loading. There are important differences in the way that crack patterns develop between the two.

Analysis on the past earthquakes show that the damages can be attributed to one of the following causes or combination of these:

a) Mistakes made in the building configuration or the structural system chosen.
b) Inadequate detailing and proportioning or errors made in detailing.
c) Poor construction quality caused by inadequate supervision.

In order to be able to produce a seismic resistant building, care should be taken at each stage listed above. For example, an excellent design will not mean much, unless the building is constructed properly under good supervision.

2.2. GROUND BEHAVIOUR

The effects of violent shaking on the ground are temporarily to increase lateral and vertical forces, to disturb the intergranular stability of non-cohesive soils and to impose strains directly on surface material where the fault plane reaches the surface.

The disturbance of the granular structure of soils by shaking leads to consolidation of both dry and saturated material, due to the closer packing of grains. For saturated sands the pore pressure may be increased by shaking to the point where it exceeds the confining soil pressure, resulting in temporary liquefaction.

Shear movements in the ground may be at the surface or entirely below it. Where the earthquake fault reaches the surface permanent movements of considerable magnitude may occur in meters. Surface shear failures can occur in weaker strata, leading to damage of embedded or buried structures. Sub-surface shear failures can also reduce the transmission of ground motion to the surface, effectively putting an upper bound on the surface motion.

2.3. IMPORTANT CATEGORIES OF DAMAGE

Damage can initiate at any level of earthquake and have different reasons. Most frequent observed types of these damages can be summarised as following:

a) Brittle failure of bolts in shear or tension.
b) Brittle failure of welds, particularly fillet welds, in shear or tension.
c) Member buckling, including torsional buckling.
d) Local web and flange buckling.
e) Uplift of braced frames.
f) Local failure of connection elements such as cleats and T's.
g) Bolt slip.

h) High deflection in unbraced frames.

i) Failure of connections between steel members and other building elements, such as floors.

In order to prevent such damages, following precautions should be taken into consideration:

a) All elements of the seismic force resisting structure should be designed to be capable of ductile response.

b) All forms of brittle failure, such as bolt fracture in tension or shear, and member buckling must be avoided, even in response to a major earthquake.

c) Joints should be provided at discontinuities with adequate provision for movement so that pounding damage can not occur.

d) Non-structural elements should be attached in such a way that they can accommodate displacements that will occur in a major earthquake.

e) Failure mechanisms should provide maximum redundancy; the possibility of failure by local collapse, such as would occur if a column failed, should be avoided.

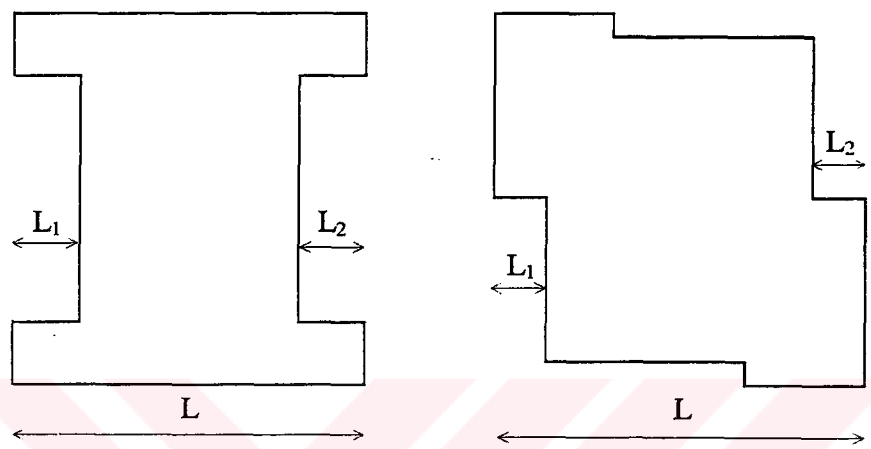
f) All portions of the building should be well tied together.

2.4 CONCEPTUAL DESIGN

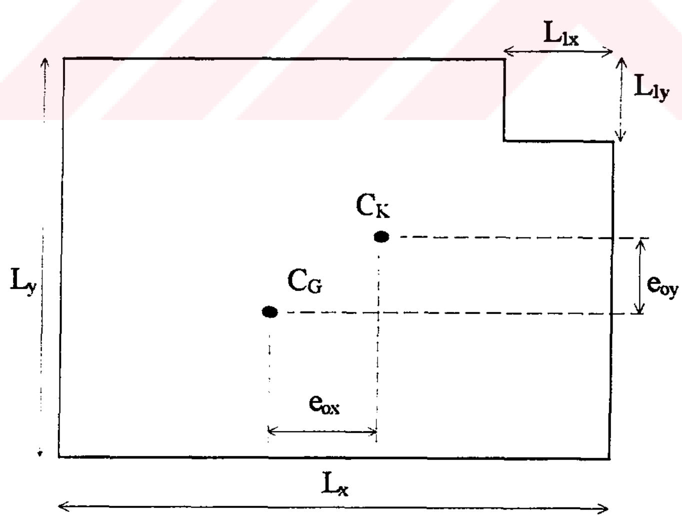
Nothing within the power of a structural engineer can make a badly conceived building into a good earthquake resistant structure. Decisions made at the conceptual stage are difficult to modify so that it is essential that their full consequences are taken into consideration in terms of performance.

One of the main objectives in early planning is to establish the optimum locations for service cores and for stiff structural elements that will be continuous to the foundation. It is usual to find that structural and architectural requirements are in conflict at the concept planning stage but it is essential that a satisfactory compromise is reached at this time.

The desirable aspects of building form are simplicity, regularity and symmetry in both plan and elevation. These properties all contribute to a more even and more predictable distribution of earthquake forces in the structural system. Any irregularity in the distribution of stiffness or mass is likely to lead to an increased dynamic response.



$$\frac{L_1 + L_2}{L} \leq 0.25 \qquad \frac{L_1 + L_2}{L} \leq 0.25$$



$$\frac{L_{1x}}{L_x} \leq 0.25 \quad \frac{e_{ox}}{r_x} \leq 0.15 \quad \frac{L_{1y}}{L_y} \leq .025 \quad \frac{e_{oy}}{r_y} \leq 0.15$$

Figure 2.1. *Plan Configuration* [1]

2.5. STRUCTURAL REGULARITY

The experience of the past earthquakes demonstrate that regular structures behave much better than non-regular ones. A building is referred to as regular, when the following conditions are satisfied.

2.5.1. Plan Configuration

The building has a significant symmetry of structure and mass with regard to at least two orthogonal axes. When re-entrant corners or recesses exists, their dimension does not exceed 25 per cent of the external size of the building (figure 2.1.) in the corresponding direction.

The distance as measured perpendicularly to the direction of the seismic action between the centre of gravity of the mass and the centre of stiffness does not exceed, at each floor, 15% of the torsional stiffness distance, defined as the square foot of the ratio between the torsional stiffness and the translational stiffness of the building at the considered floor [1].

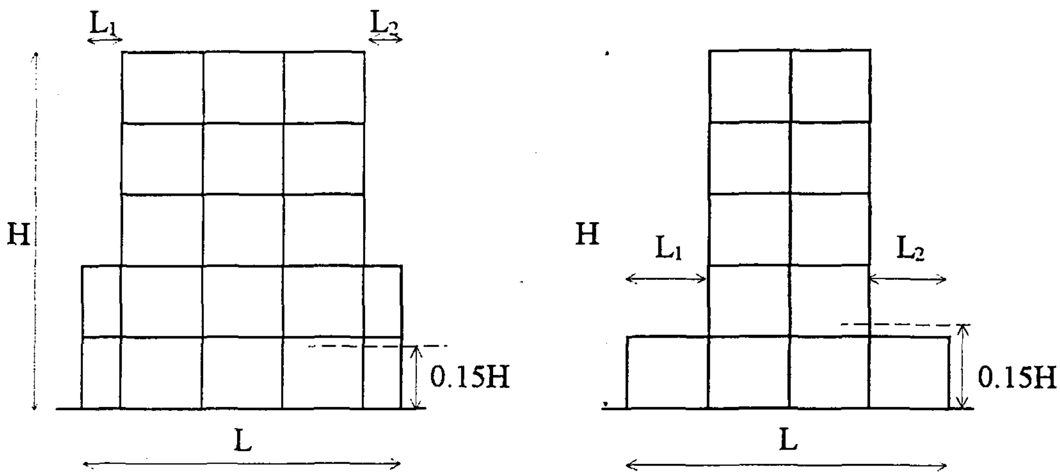
2.5.2. Vertical Configuration

The stiffness and mass properties must be approximately uniform along the building height.

As it is seen in figure 2.2., in case of gradually tapering buildings with symmetry about the vertical axis, the extend of setback at each floor must not exceed 20% of the previous plan dimension.

The above limit may exceed up to 50%, if the setback stops at a level 15% below the top of the building.

In case of only one tapered facade, the setback at each floor must not exceed 10% and overall setback must not be greater than 30% of the plan dimension at the first floor.

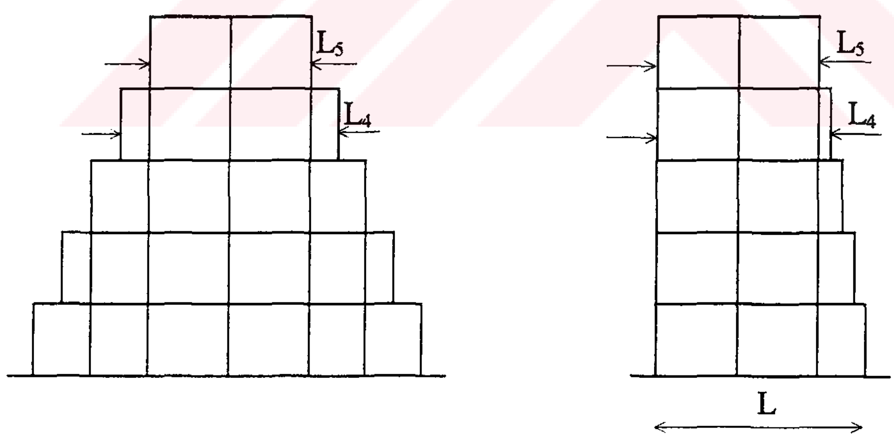


$$\frac{L_1 + L_2}{L} \leq 0.20$$

(Set back occurs above 0.15H)

$$\frac{L_1 + L_2}{L} \leq 0.50$$

(Set back occurs below 0.15H)



$$\frac{L_4 + L_5}{L_4} \leq 0.20$$

$$\frac{L - L_5}{L} \leq 0.30$$

$$\frac{L_4 - L_5}{L_4} \leq 0.10$$

Figure 2.2. *Vertical Configuration* [1]

III. STRUCTURAL STEELWORK DESIGN

3.1. GENERAL ABOUT STEELWORK DESIGN

Structural steel is in many ways an ideal material for earthquake resistance. In comparison with reinforced concrete, steel structures are more flexible. However, they have higher secondary stresses from P-δ effects and accompanying larger displacements may lead to higher levels of damage to non-structural components.

In past earthquakes steel structures have usually performed well because of having a high level of material ductility and energy absorption. However, designing in steel does not mean definite survival. In order to make good use of the inherent ductility of steel, considerable care is needed in design and detailing of framing systems and connections.

3.2. MATERIALS AND WORKMANSHIP

Care should be taken in the selection of steel quality. For ductile elements, steel should be low carbon and weldable steel with good notch ductility. Where lamellar tearing is a consideration, a low sulphur content, 0.02 or less, is desirable. The yield strength should not exceed 360 N/mm^2, and the ratio of ultimate strength to yield strength should exceed 1.4. The upper limit of yield strength should be stipulated, generally not more than 15% greater than the specified value.

Lamellar tearing can occur where butt or fillet welds of 20mm or over are made on plates at least 30mm thick, where there is a high degree of restraint . Tearing can take place in planes parallel to the direction of rolling. The solutions to this problem lie to a limited degree in the selection of the steel (low sulphur content) and in good detailing of welds.

The detailing and fabrication of ductile portions of the structure should consider the possibility of low cycle fatigue - structures responding to earthquakes rarely go through more than 20 cycles of response. Fatigue failure can initiate at notches and cracks which run at right angles to the direction of stress. Welding should follow the best standards of quality. Bolt holes should be drilled and not punched or reamed [2].

3.3. STRUCTURAL TYPOLOGIES

The type of seismic resistant structure should be chosen such that fulfils the required stiffness and strength criteria and is robust enough and that any singular local damage or failure does not induce global collapse. As the horizontal bracings have to be designed in order to elastically resist to the maximum horizontal forces, it means that, during earthquakes, they do not provide any dissipative action. Consequently, the dissipation of the earthquake input energy is exclusively entrusted to the vertical bracing system. Therefore, from the seismic behaviour point of view, the different structural typologies are identified by means of the different typologies of vertical bracing system.

For steel buildings in seismic regions, three families of structural systems are introduced: concentrically braced frames, moment resisting frames and eccentrically braced frames.

3.3.1 Concentrically Braced Frames

Concentrically braced frames are among the most common steel structural systems for resisting lateral forces which develop due to wind, earthquake or other actions. The relative economy of their design and construction along with their good performance in terms of stiffness makes concentrically braced frames an attractive choice of designers.

Diagonal bracing elements with coincident centerlines form vertical cantilever trusses. Horizontal forces are mainly resisted by these elements subjected to axial action. In these structures the dissipative zones are mainly located in the tensile diagonals, because it is assumed that compression diagonals are buckled. Different behaviours are performed according to the type bracing, which can be classified into three categories: diagonal bracings, V-bracings, and K-bracings.

The diagonal bracing (figure 3.1) dissipates energy by means of the plastification of both compression and tension diagonals. Horizontal external loads are resisted by the tension diagonals only, neglecting the compression diagonals.

In V-bracings (figure 3.2) horizontal external loads can be resisted by considering both tension and compression diagonals. Compression diagonals dissipates energy, whereas the tension diagonal remains elastic. The intersection point of these diagonals must lie on a member connected to the horizontal diaphragm.

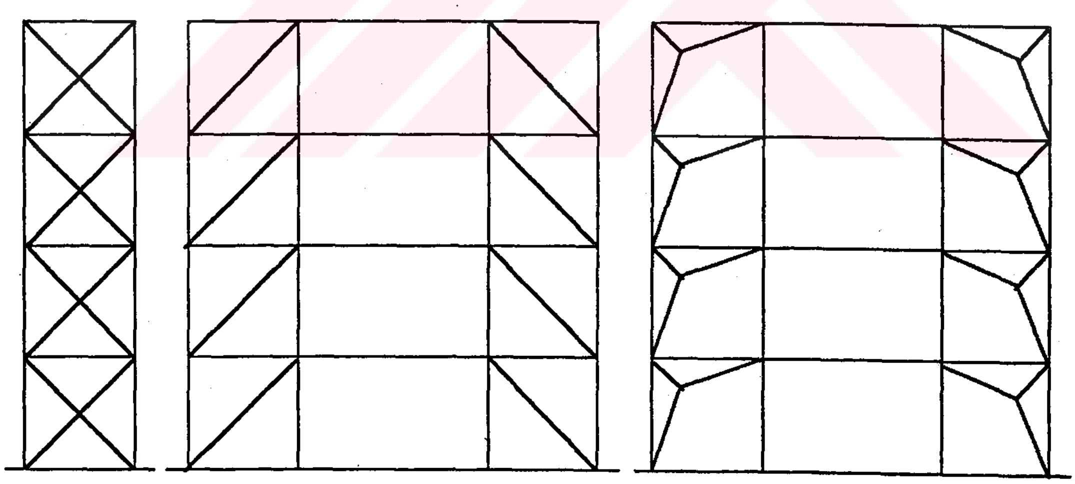


Figure 3.1. *Diagonal Bracings*

The K-bracings (figure 3.3) cannot be considered as dissipative because the diagonals intersect the column in an intermediate point.

Because one diagonal of an opposing pair is always in tension, the possibility of a brittle type failure is present. An additional drawback in the use of concentrically braced frames is that there is effectively no way in which access can be gained through the panel, which places a major restriction on the areas where they can be used.[3]

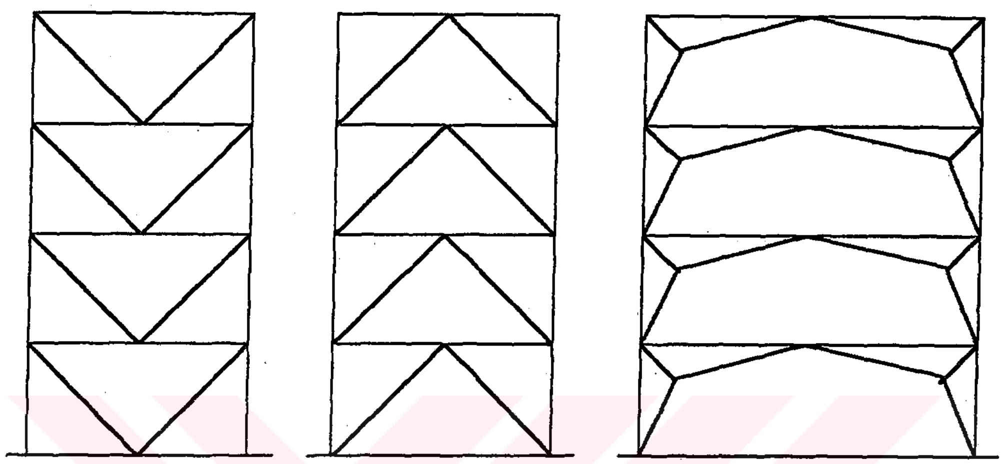


Figure 3.2. *V-Bracings*

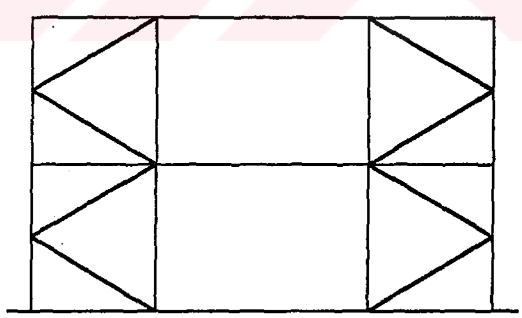


Figure 3.3. *K-Bracings*

3.3.2. Moment Resisting Frames

In regions of high seismicity, moment resisting frames are widely used as lateral force resisting systems especially for low-rise steel structures. Their stable hysteretic behaviour and superior ability to dissipate earthquake input energy have tended to favour them over other structural systems. Good performance of moment resisting frames was observed during past major earthquakes.

The earthquake input energy is allowed to be dissipated in moment resisting frames through inelastic deformation in one or more of two elements meeting at the beam to column joint, i.e. in girders or in columns. Consequently, in addition to the conventional design concept of strong column-weak beam, another design philosophy weak column-strong beam may also be considered acceptable in certain situations.[4]

3.3.2.1. Strong Column-Weak Beam Design

In this design philosophy, the beams are detailed to be weaker than the adjoining column and are designed to be the critical elements that undergo inelastic deformations. This criterion is satisfied by ensuring that the strength of the columns at each joint is higher than the overstrength of the adjoining beams. In order to avoid the plastic hinge formation in column, the design value for bending moments in columns are increased by multiplying with an amplification factor.

3.3.2.2. Weak Column-Strong Beam Design

In this design philosophy, the columns are considered the critical elements that dissipate the earthquake input energy. This situation arises in low-rise buildings, where the beam's design is dominated by gravity loads while that of column is governed by earthquake forces.

3.3.3. Eccentric Braced Frames

A suitable harmonization between the lateral rigidity of bracings and the ductility of frames can be obtained in the hybrid framing system of eccentrically braced frames. In this case the horizontal forces are mainly absorbed by bars giving an axial reaction and eccentricity of the layout allows the energy dissipation by means of a cyclic bending or shearing behaviour of the beams.

The level of energy absorption of these frames is similar to the moment resisting frame systems. In addition, the eccentrically braced frame system has advantages in terms of drift control and represents an economic solution also in the range of medium and high-rise buildings. The active link is the main energy dissipator in the structural system. It must be designed in order to obtain that its bending and shear limit strength precedes the attainment of the tension and compression limit strength of other bars.

3.3.3.1. Elastic Behaviour

Whether or not a braced frame acts in conjunction with a moment frame, its stiffness is of great importance. Acting with a frame its stiffness will affect the distribution of force between the moment frame and the braced frame. In either case the braced frame will have a major effect on the overall stiffness of the system.

The elastic design parameters of an eccentric braced frame (EBF) can be characterised in the way illustrated in figure 3.4 for a simple system. The length assigned to the link, or 'active link', beam is its clear span. The levels over which stiffness can be varied

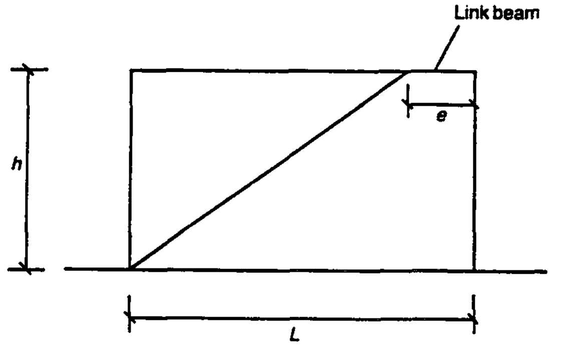


Figure 3.4. *Eccentric Braced Frame Parameters*

for more complex arrangements can be illustrated with reference to this frame, using the parameter e/L as the control variable. As e/L is varied the system changes from a moment frame with $e/L = 1$ to a concentric braced frame with $e/L = 0$ [2].

Figures 3.5-3.7 show the influence of other frame parameters on the elastic stiffness. Figure 3.5 varies the frame geometry for a fixed set of section properties. Figure 3.6 deals with a greater width-to-height ratio where it would be impractical to use a single brace. Figure 3.7 deals with variations in section properties for a fixed framing geometry. Each relationship shows considerable variation in stiffness that is possible, the sensitivity being great within the area of most practical configurations with e/L between 0.05 and 0.25.

For e/L <0.5 the shear stiffness of the link beam plays a significant role in the elastic stiffness of the frame. It is clear from the kinematics of the system that shear forces are concentrated in the link beam, and figure 3.8 shows the significance of the variations in the link beam shear stiffness.

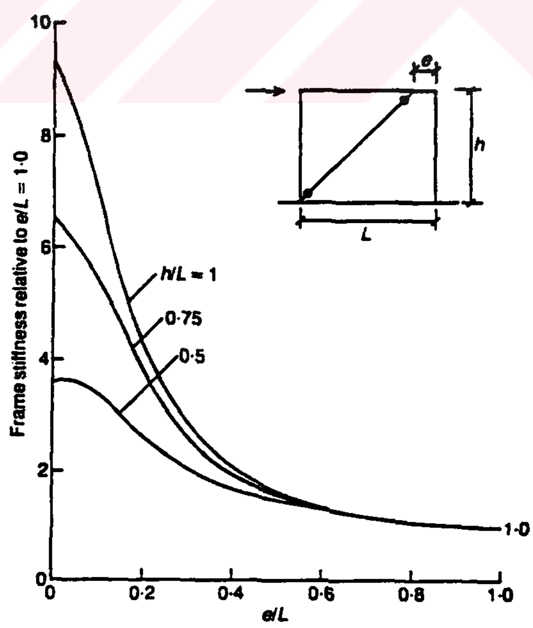


Figure 3.5. *Eccentrically Braced Frames Stiffness for Varying Geometric Arrangements* [2]

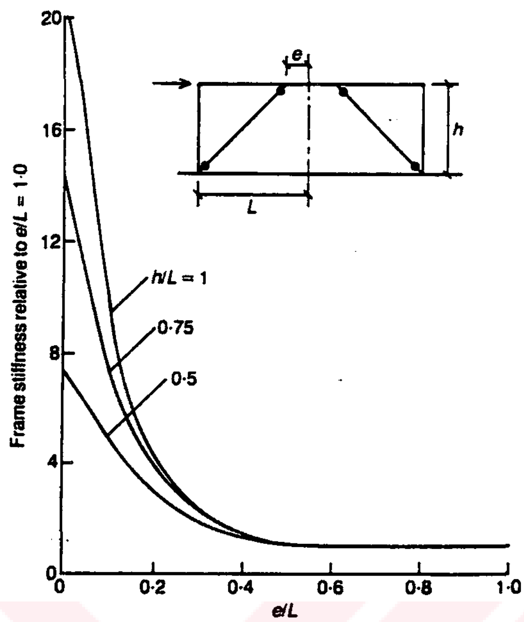


Figure 3.6. *Eccentrically Braced Frames Stiffness for Varying Geometric Arrangements* [2]

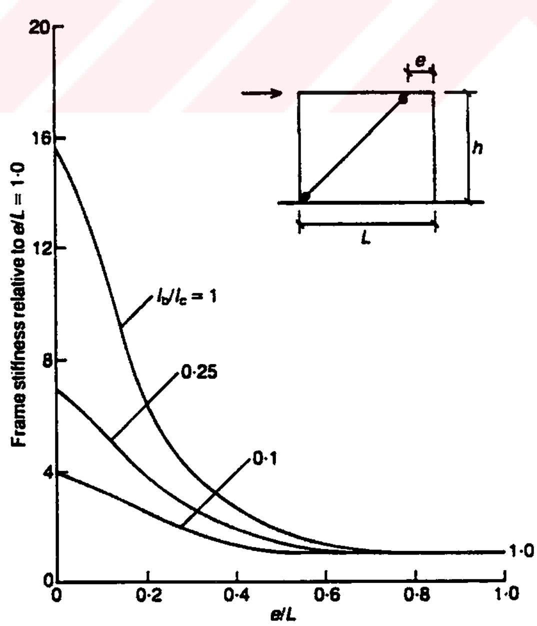


Figure 3.7. *Eccentrically Braced Frames Stiffness for Varying Member Properties* [2]

($I_b/A_{br}L^2$=0.001, EI_b/A_bL^2=0.01, h/L=0.75)

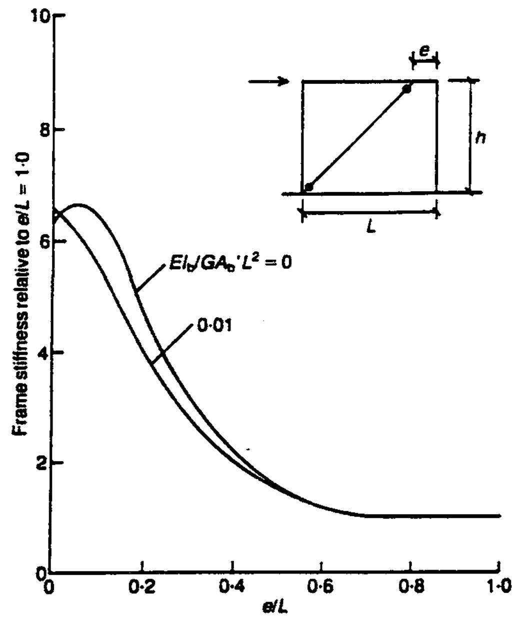


Figure 3.8. *Eccentrically Braced Frames: Influence of Shear Deformation on Frame* [2] *Stiffness* ($I_b/A_{br}L^2$ = 0.001, I_b/I_c =0.25, h/L=0.75)

3.3.3.2. Inelastic behaviour

Behaviour in the inelastic range is dominated by the link members, which may have very high ductility requirements. The inelastic capacity of the shear links themselves depends on the moment shear relationship, as illustrated in figure 3.9, the controlling parameters being given by [2]

$$V_p^* = \tau_y (d - t_f) t_w \tag{3.1}$$

$$M_p^* = \sigma_y (b - t_w)(d - t_f) t_f \tag{3.2}$$

$$M_p = \sigma_y Z \tag{3.3}$$

where τ_y is the yield stress in pure shear, σ_y is the yield stress in pure tension, $\sigma_y = \tau_y \sqrt{3}$ for the von Mises yield criterion, V_p* is plastic shear, M_p* is plastic moment, Z is the plastic section modulus and d, t_f, b and t_w are defined in figure 3.9. The interaction curve in the figure may be approximated as

$$\left(\frac{|M| - M_p^*}{M_p - M_p^*}\right)^2 + \left(\frac{V}{V_p^*}\right)^2 = 1 \qquad M_p^* \leq |M| \leq M_p \qquad (3.4)$$

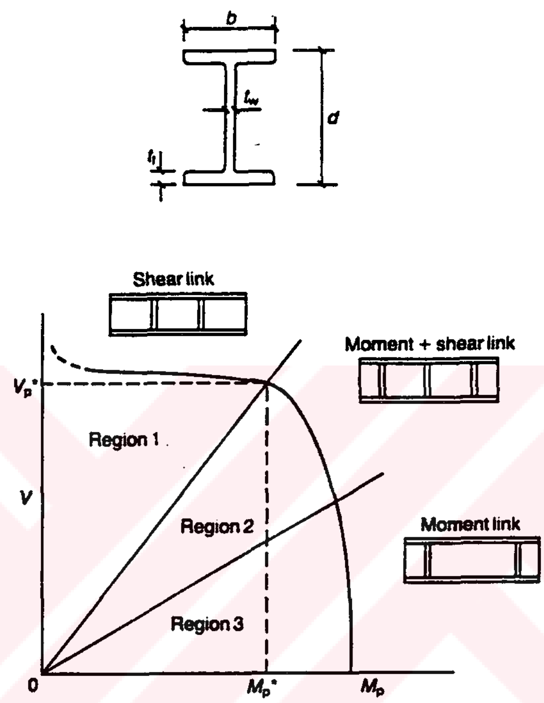


Figure 3.9. *Moment-Shear Interaction Diagram and Appropriate Link Stiffening Details* [2]

Referring to figure 3.9, three kinds of link can be identified. In region 1 the link is a shear link and if provided with suitable stiffeners to prevent web buckling will provide excellent ductility under cyclic loading. In region 2 and 3 the links are moment links, the difference between the two being that in region 2 the shears are still substantial and stiffeners are required for both web shear and flange buckling. In region 3 stiffeners are required for flange buckling only.

3.4. BEHAVIOUR FACTOR

Structures are usually designed so that some of the energy input during severe earthquakes is dissipated through plastic deformations. In order to prevent collapse, the value of these plastic deformations must be limited according to the available local and global ductility of the structure.

The behaviour factor q is introduced in order to account for the energy dissipation capacity and post-elastic resistance of the structure. The correct evaluation of the q-factor, which can also be defined as the minimum ratio between the acceleration leading to collapse and the one corresponding to the achievements of first yielding, requires several dynamic analysis for different ground motions. The proposed methods for evaluating q-factor can be grouped in three categories;

1. Methods based on the theory of the ductility factor.
2. Methods based on the extension of the results on the dynamic inelastic response of simple degree of freedom systems.
3. Methods based on the energy approaches.

Table 3.1. *Recommended Behaviour Factor Values*

Structural System	q
Structures Mainly Reacting In Bending	$5\,\alpha_u/\alpha_1 < 8$
Frame Structures	
Eccentric Truss Bracings	
Braced Frame Structures	
Concentric Truss Bracings	
Diagonal Bracings	$4\,\alpha_u/\alpha_1$
V-Bracings	$2\,\alpha_u/\alpha_1$
Cantilever Structures	$2\,\alpha_u/\alpha_1$
Mixed Structures	2

In general, the values of behaviour factor recommended by the European Convention for Constructional Steelwork (ECCS) are given in table 3.1, where α_1 is the multiplier of the horizontal seismic actions, by keeping constant the other design loads, which corresponds to the point, where the structure reaches its elastic limit in one section and α_u is the multiplier of the horizontal seismic actions by keeping constant the other design loads, which corresponds to the point, where the structure reaches the maximum load bearing capacity due to the formation of plastic hinges in the assumed dissipative zones sufficient to transform the structure in a mechanism.

The approximate values of the ratio α_u/α_1 are given in table 3.2. for different characteristics of structures [1].

Table 3.2. Approximate Values of α_u/α_1

Structural System	Approximate Values α_u/α_1
Multistorey System	1.2
One Storey Frames or Eccentric Truss Bracings	1.1
Concentric Truss Bracings	1.0
Cantilever Structures	shape factor of the cross section

3.5. DESIGN OF BEAMS

For moment frames it is customary to design on the basis of strong columns and weak beams, so that columns behave elastically while the latter is inelastic. Therefore beams are required to provide energy absorption and adequate rotation capacity at certain points. For suitable steels, stable hysteretic yield behaviour can be provided by normal universal beam or wide flange sections as long as buckling can be avoided or controlled.

Lateral or torsional buckling of compression members may lead to sudden collapse and cannot normally be tolerated. However, local buckling of plates is less serious because they retain a substantial post-buckling strength, and this is relevant to beam design as the webs and flanges behave as plates in their buckling performance. Nevertheless buckling will generally tend to pinch the hysteresis loop, reducing the energy absorbed, and this is obviously undesirable. Increased rotations may also causing distress in some adjacent portion of the structure.

Research has clearly demonstrated that the buckling strength of members subjected to cyclic loading reduces with successive cycles so that the rules adopted in design for cyclic loads need to be considerably more conservative, in comparison with those for statically applied monolithic loads.

In a beam the compression forces are small enough to assure that the full elastic bending capacity of the cross-section can develop. The maximum shear must be limited in order to not reduce the bending resistance of the cross-section.[5]

3.6. DESIGN OF COLUMNS

Columns should be designed such that hinges form in the beams before yielding in the columns occurs and that the shear in the columns does not hinder full plastic hinges to develop.

Combined compression and bending checks on the column must be made by considering the worst case combination of axial force N and bending moment M_x and M_y.

The design values for bending moments are given by the sum of the bending moments $M_{c,s}$ due to horizontal seismic actions multiply by the amplification factor α, which imposes that sum of the bending moment in the column should be greater than the sum of the resisting moments in the beams, plus the bending moments $M_{c,o}$ due to the other loads; cut paste.[6]

3.7. CONNECTION DESIGN AND JOINT BEHAVIOUR

The design of connections in seismically resistant structures differs from non-seismic design because of the necessity to accommodate inelastic response in the members. Although studies of inelastic behaviour in connections have shown that some energy absorption is possible, they are designed to remain elastic.

The forces exerted on connections should be those occurring when there is plastic hingeing in the connecting members with a capacity design approach, and allowing for

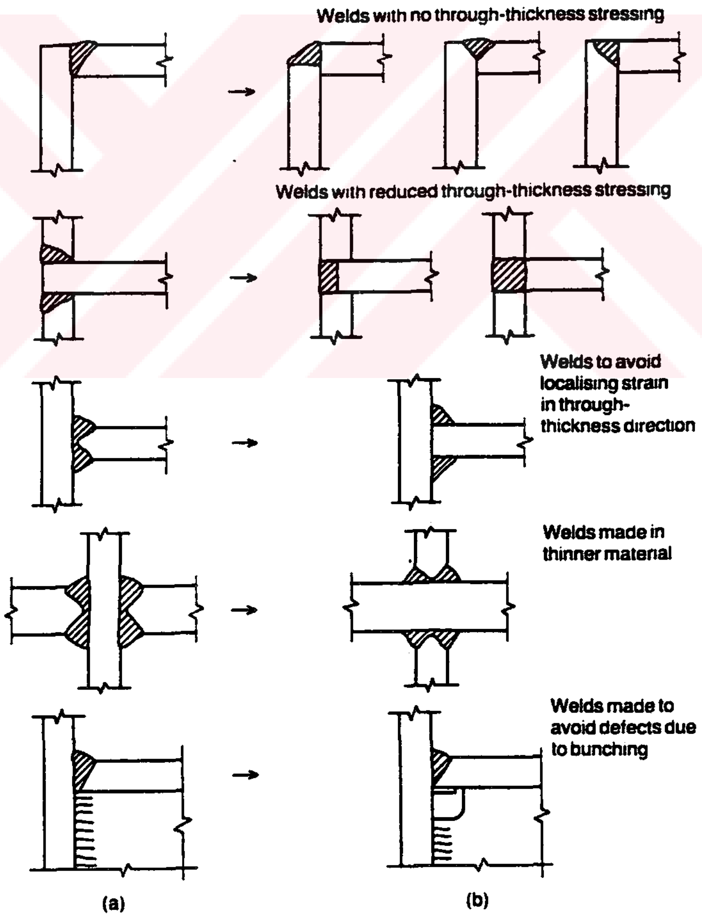


Figure 3.10. *Weld Details to Avoid Lamellar Tearing (a) Not Recommended (b) Recommended* [2]

overstrength and strain hardening. Because of uncertainty in the manner in which a structure will respond to the violent shaking of an earthquake, following minimum forces for connections are recommended [2].

a) For moment connections, a moment of 1.5 times the connecting member moment based on the nominal yield stress.
b) For non-moment connections, one-third of the moment capacity of the connecting member based on the nominal yield stress.
c) For all connections one-half of the strength of the member in tension or compression, based on the nominal yield stress.
d) For all connections 15 per cent of the member strength in shear, based on the nominal shear strength.

3.7.1. Welding

Normal good practice will apply in welding, but the highest standard is needed because of the possibility of low cycle fatique The avoidance of lamellar tearing is assisted by detailing to avoid cross-plate stressing by welds under conditions of constraint. Figure 3.10 shows some recommended detailing practice.

Connections made by means of butt welds or full-penetration groove welds do not require any checking. Fillet welds are acceptable if the following rules are observed.

a) Intermittent welding should be minimised as the ends of runs are stress raising discontinuities.
b) The throat thickness should not be less than half the plate thickness.
c) Tearing stresses in the parent metal should be checked where high strength electrodes are used, and the leg length of the weld is small.

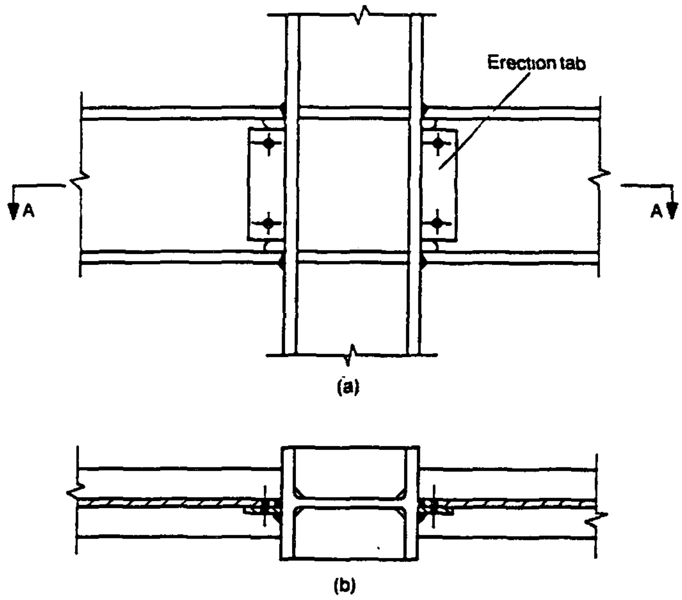


Figure 3.11. *Welded Beam-Column Connection: (a) Elevation; (b) Plan on A-A*

3.7.2. Beam-Column Joints

Beam and column splice joints will normally be designed in a similar way to those used in non-seismic design. Their location should generally be in zones of low stress.

Some types of beam-column joints are shown in figures 3.11~3.14. The choice of type will depend more on economics, the available skills, quality control and fabrication resources than on design requirements. Each of the types shown can function as a moment connection. The all welded types shown in figures 3.11 and 3.12 can be provided with bolted web shear connectors in place of the welded web shown. Where connections are bolted to column as in figures 3.13 and 3.14, care needs to be taken over the effects of bending distortion on the end plate or the head of the Tee connector.

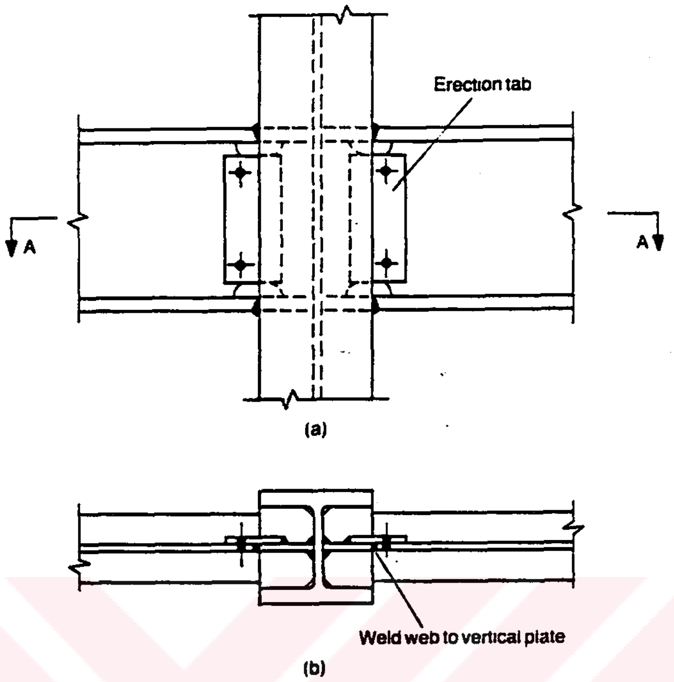


Figure 3.12. *Welded Beam-Column Connection-Weak Axis: (a) Elevation; (b) Plan on A-A*

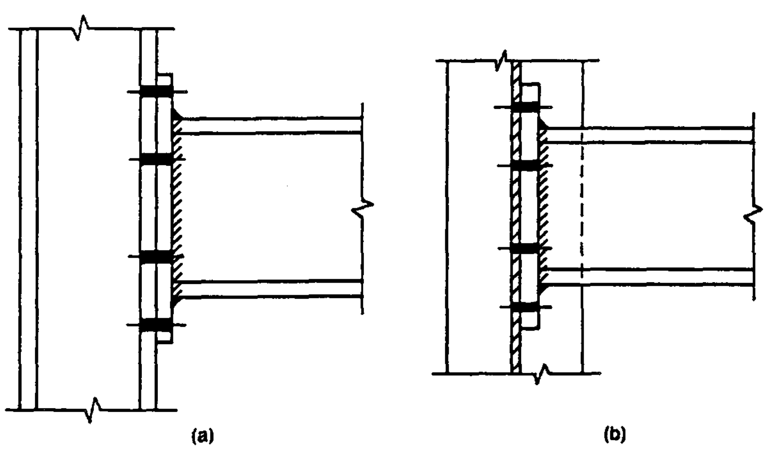


Figure 3.13. *Bolted Beam-Column Connections: (a) Strong Axis; (b) Weak Axis*

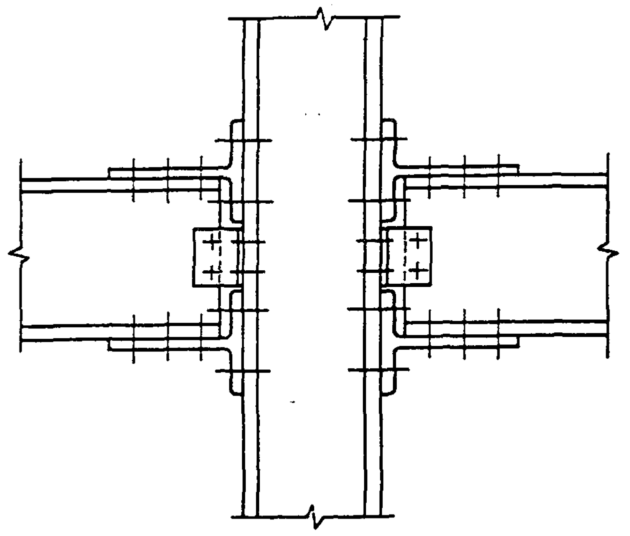


Figure 3.14. *Bolted Beam-Column Connection Using Tees and High Strength Friction Grip Bolts*

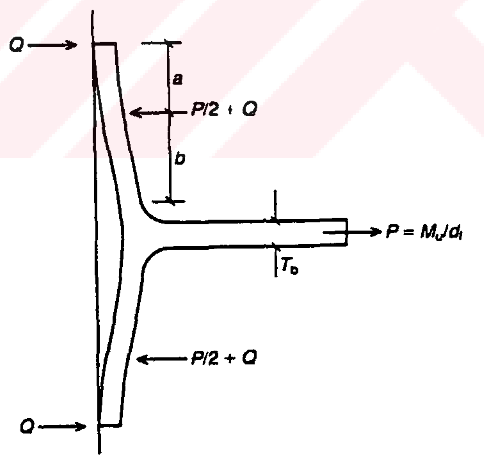


Figure 3.15. *Effect of Prying on Bolt Forces in Tee Connectors*

Figure 3.15 shows the effect of prying on the Tee connection, and a similar effect may occur with the welded end plate. For normal section the value of Q, the prying force, does not increase the bolt force by more than 10 per cent and only exceeds this, where the Tee flange is unusually thin or the bolts are spaced further from the root than necessity.

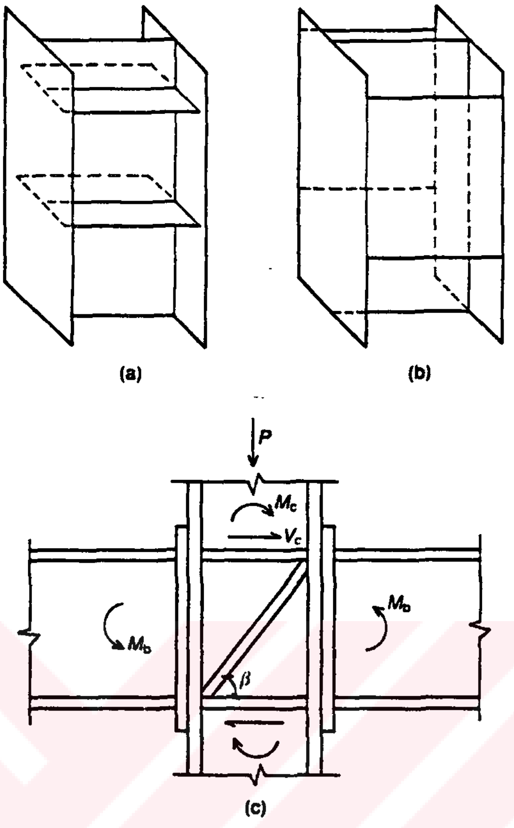


Figure 3.16. *Column Stiffeners in the Panel Zone: (a) Web Stiffeners; (b) Doubler Plates; (c) Diagonal Stiffeners*

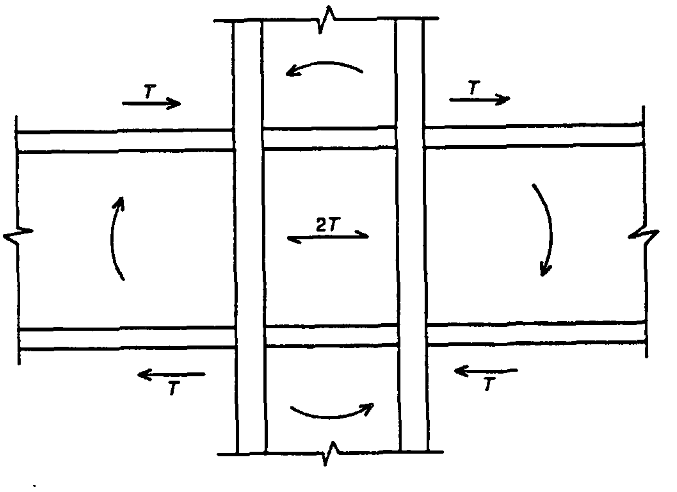


Figure 3.17. *Shear Forces in the Panel Zone*

Types of stiffener used in the panel zone are shown in figure 3.16. The behaviour of the flange will depend on the support which it derives from stiffeners and is normally analysed on the basis of yield line theory.

Figure 3.17 shows the shear forces in the panel zone due to lateral loading. It can be seen from the figure that the panel zone shear is 2T and, a point which is often not realised by designers, the total shear taken by the weld between the stiffener and web is also 2T.

IV. PLASTIC ANALYSIS

4.1. GENERAL ABOUT PLASTIC ANALYSIS

An elastic analysis of a structure is important to study its performance, especially with regard to serviceability, under the loading for which the structure is designed. However, if the load is increased until yielding occurs at some locations, the structure undergoes elastic-plastic deformations, and on further increase a fully plastic condition is reached, at which a sufficient number of plastic hinges are formed to transform the structure into a mechanism. This mechanism would collapse under any additional loading. A study of the mechanism of failure and the knowledge of the magnitude of the collapse load are necessary to determine the load factor in analysis.

Alternatively, if the load factor is specified, the structure can be designed so that its collapse load is equal to, or higher than, the product of the load factor and the service loading.[7]

4.2 CONCEPTS AND ASSUMPTIONS

The important concepts and assumptions with regard to the plastic behaviour of structures according to the plastic theory are as follows;

1) The structures and the loads one all in the same plane, and each member has at least one axis of symmetry lying in that plane.
2) The material is ductile and has the capacity of undergoing large plastic deformation without fracture.

3) The cross-sections of each member has a maximum strength-generally the plastic moment M_p- which is developed through plastic yielding of the entire cross section.
4) Because of the ductility of steal, rotation at relatively constant moment will occur through a considerable angle; i.e. a plastic hinge will form.
5) Connections proportioned for full continuity will transmit the calculated plastic moment.
6) Plastic hinges will first form at sections where the moments under elastic condition reach M_p. With these sections rotating at constant moment, additional loading will be accompanied by a redistribution of moments in the structure, so that plastic hinges will appear at some other locations where the moments under elastic conditions were less than M_p.
7) The deformations are small, and therefore the equilibrium equations can be formulated for the undeformed structure, as in the case of elastic analysis. Similarly, virtual-work expressions for mechanism displacement are based on small deflections.
8) No instability will occur before the attainment of the plastic limit load.
9) The loading in proportional; the ratios between various loads remain constant during loading.[8]

4.3. THE DUCTILITY OF STEEL

Steel possesses ductility -a unique property that no other structural material exhibits in quite the same way. Through ductility, structural steel is able to absorb large deformations beyond the elastic limit without the danger of fracture. It is this characteristic of structural steel that makes possible the application of plastic analysis to structural design.

The ductility is evident on a stress-strain curve obtained from a simple tension or compression test (figure 4.1.). Below the yield stress level, the material in elastic. Above it the strain increases greatly without any further increase of the stress (8 to 15 times the elastic strain). After this plateau, some increase in strength is observed as the material strain hardens.

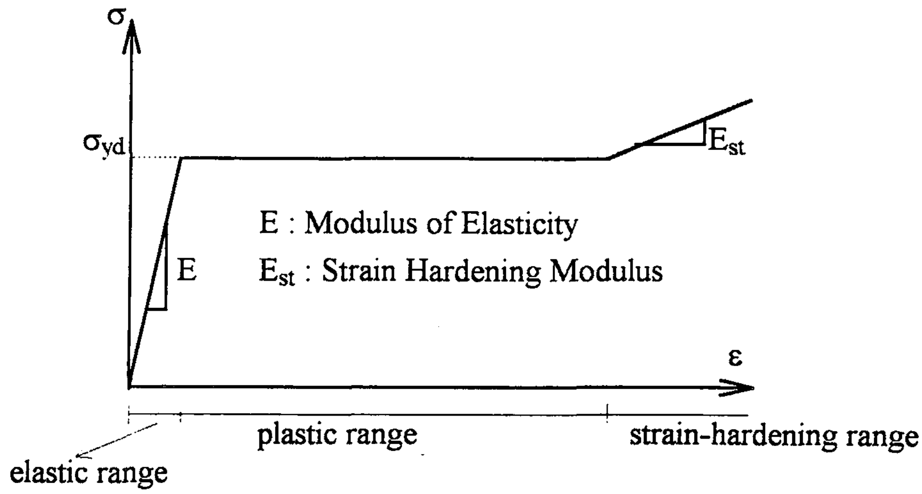


Figure 4.1. *Idealised Stress-Strain Curve of Structural Steel*

From all of these, it follows that the initial attainment of a maximum fiber stress which is equal to the yield stress does not result in failure of a structure.

The strain hardening range is also necessary to achieve moment redistribution in a structure. A member fabricated of a material which exhibits very limited strain hardening will show very high local strains at the critical sections, and this will lead to early instability or, if this is prevented, to fracture of the material.

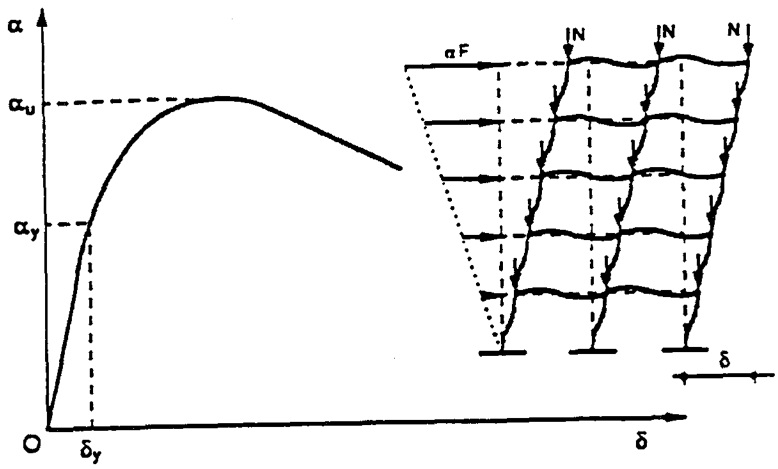
V. GLOBAL DUCTILITY

5.1. GENERAL DEFINITION

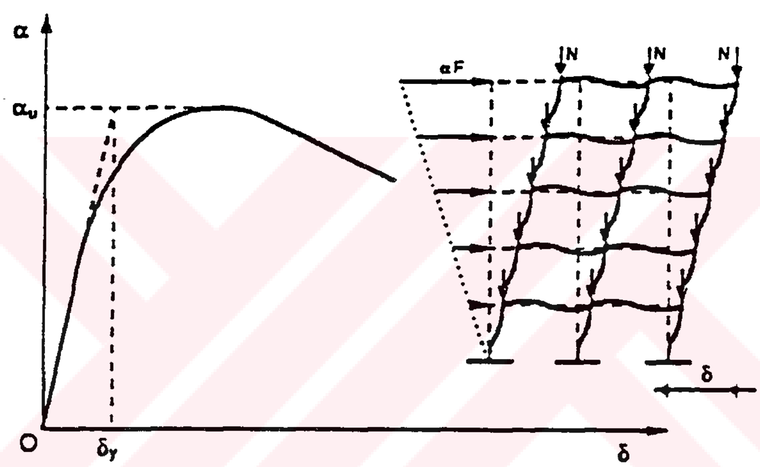
Ductility of structures plays a fundamental role in seismic design. In fact structures are usually designed so that some of the energy input during severe earthquakes is dissipated through inelastic deformations. Therefore, seismic analysis of structures has been usually divided into two fundamental steps; the first step requires the evaluation of the available ductility, while the second one is based on the computation of the ductility required by the severest design earthquake. It is clear that available ductility should be greater than the required one in order to prevent collapse.

For a given structure the global ductility is defined on the ratio between the values of a given displacement evaluated at the ultimate and yield condition. In other words it is the ability of a structural system to sustain deformations beyond its yield point without significant loss of strength. In spite of the fact that the physical meaning of the ductility concept is universally recognised, its quantitative definition is not clearly established.

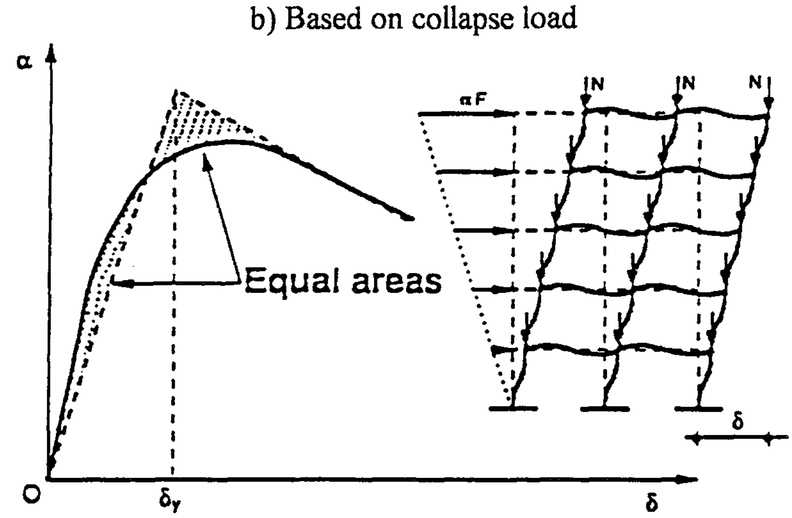
First of all, attention must be paid to the definition of the external actions. With reference to seismic design, the problem concerns the choice of the shape of an horizontal force system, which intensity is defined by using an unique multiplier α. To this scope different distribution patterns can be adopted depending upon the considered number of vibration modes. Once the distribution pattern of horizontal forces has been chosen, the global ductility can be evaluated as the ratio between the two top sway displacement δ, at ultimate and yield states. As different options can be adopted for the definition of ultimate and yield states, the definition of ductility ratio is not universally accepted.



a) Based on first yield

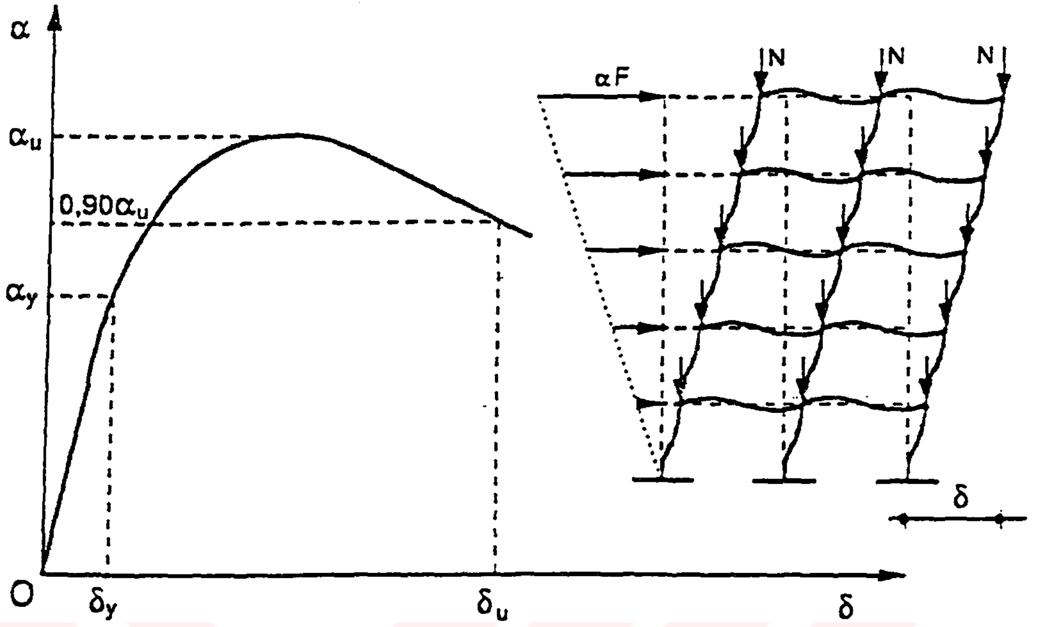


b) Based on collapse load

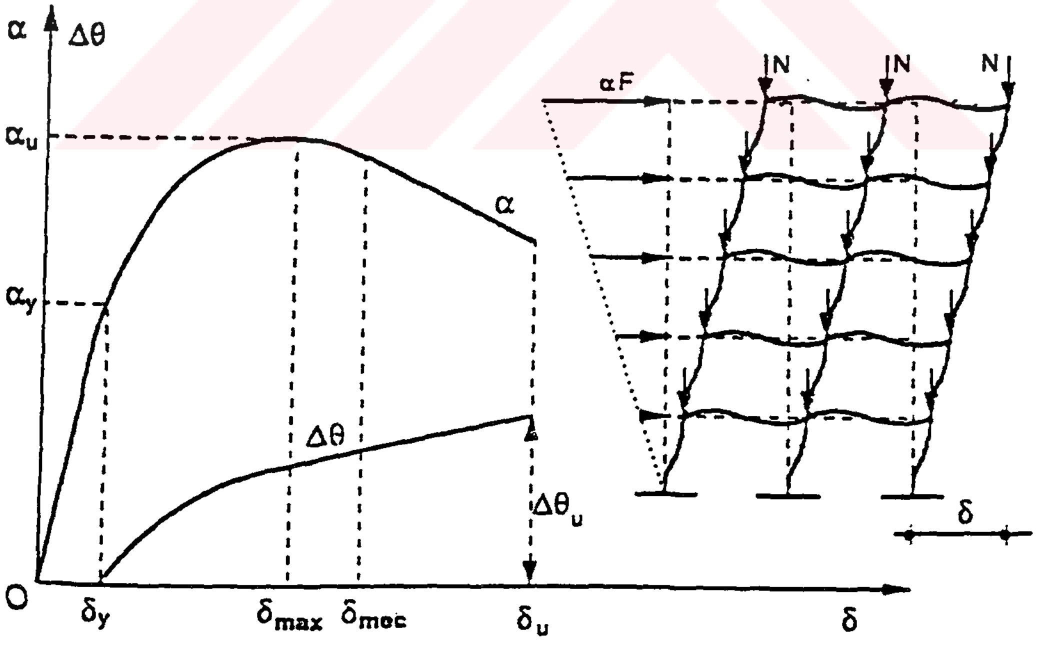


c) Based on energy absorption

Figure 5.1. *Alternative Definitions of the Yield Displacement* [9]



a) Based on a fixed percentage reduction (10% as an example) in load carrying capacity



b) Based on the ultimate plastic rotation $\Delta\theta_u$ of the critical plastic hinge

Figure 5.2. *Alternative Definitions of the Ultimate Displacement* [9]

Alternative ways which have been used to define the yield state are shown in figure 5.1. They assume as reference parameter;

a) the displacement at first yield
b) the elastic displacement under a load equal to the collapse load
c) the yield displacement of an elastic-perfectly plastic system (with geometric non-linearity) with the same energy absorption capacity as the actual structure.

It has been already pointed out that the ductility concept requires that deformations beyond yield point have to be sustained without significant loss of strength. Therefore, it is clear that also different alternatives are possible for the definition the ultimate displacement. In particular the ultimate displacement can be defined as the one corresponding to a fixed percentage reduction in load carrying capacity or as the one corresponding to the attainment of the available rotational capacity R in the critical plastic hinge (figure 5.2).

5.2. RESPONSE UNDER MONOTONIC LOADS

For any given distribution pattern of horizontal forces, the inelastic response of a structure under monotonic horizontal loads is completely described by behavioural curve α-δ which relates the multiplier of horizontal forces α to the top horizontal displacement δ. This behavioural curve is constituted by two branches; an increasing branch and a softening branch.

The increasing branch can be divided into two parts. The first part, which represents the phase of elastic behaviour, extends from the origin until the first yielding is reached. The first yielding multiplier α_y and the corresponding displacement δ_y provide the starting point of the second part of the increasing branch. This part is due to the plastic redistribution capacity of the structure until the values of the ultimate multiplier α_u and of the corresponding displacement δ_{max} are reached.

The softening branch can be divided into two parts as well. The first part, which starts at displacement δ_{max} and ends at δ_{mec}, is characterised by the fact that the structure is still indeterminate and the process of plastic hinges formation is in progress. At the value δ_{mec} of displacement, a collapse mechanism occurs and from this point the last part of the softening branch starts. The shape of this part is strictly related to the collapse mechanism type and to the magnitude of vertical loads. The ultimate displacement is attained when the maximum rotational capacity in the critical plastic hinge is requested.

It has to be pointed out that plastic redistribution produces two effects. On one hand an increase of the load carrying capacity with respect to first yielding is obtained; on the other hand the formation of plastic hinges is not contemporary so that some plastic hinges have to withstand higher inelastic rotations. This phenomenon can lead to a premature failure due to the limitations arising from local ductility of members.

Moreover, as the slope of the softening branch increases as far as the magnitude of vertical loads increases, we can observe that global ductility can be limited by two events; due to the required local ductility when the value of critical elastic multiplier of vertical loads is high, and due to the high slope of softening branch in the opposite case. In both cases the influence of collapse mechanism type is decisive.

5.2.1. The Influence of the Collapse Mechanism

The collapse mechanism plays a very important role in seismic design of structures, because it influences the value of the available global ductility and the energy dissipation capacity of the structure.

The case in which a framed structure fails according to a global type mechanism (figure 5.3.) can be adopted as the reference case, because it is generally considered able to exhibit enough ductility to withstand severe earthquakes.

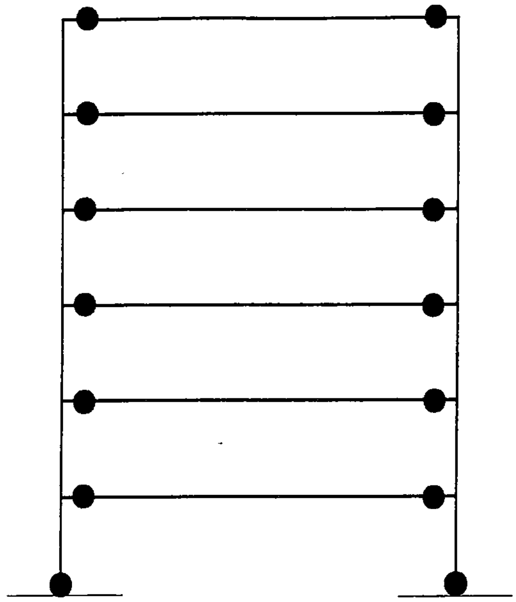


Figure 5.3. *Global Type Mechanism*

The slope of softening branch γ -which is strictly related to collapse mechanism- for framed structures failing according to the global mechanism, can be obtained with good approximation by means of the following relation;

$$\gamma \approx \frac{1}{\alpha_{cr}} \tag{5.1}$$

where α_{cr} is the critical elastic multiplier of vertical loads. In case of a shear type portal frame γ is evaluated by following equation;

$$\gamma = \frac{\pi^2}{12\alpha_{cr}} \tag{5.2}$$

For a given value of the total vertical load, collapse mechanisms different from the global type determine an increase of γ.

5.3. DESIGN REQUIREMENTS

In order to obtain a collapse mechanism of global type in framed structures, flexural strength of columns has to be greater than flexural strength of beams. Therefore, at each beam-to-column joint the following condition has to be satisfied;

$$\Sigma M_{R,c} > \Sigma M_{R,b} \tag{5.3}$$

where $\Sigma M_{R,c}$ is the sum of resisting moments of the columns and $\Sigma M_{R,b}$ is the sum of the resisting moments of the beams connected to the joint. Therefore the amplification of the bending resistance of columns is advised in modern seismic codes.

The amplification coefficient α has to be computed by means of the following condition

$$\Sigma M_{c,o} + \alpha \Sigma M_{c,s} = \Sigma M_{R,b} \tag{5.4}$$

where $M_{c,o}$ is the column bending moment due to non seismic loads and $M_{c,s}$ is the column bending moment due to horizontal seismic forces.

Therefore, the value of the amplification coefficient is obtained;

$$\alpha = \frac{\Sigma M_{R,b} - \Sigma M_{c,o}}{\Sigma M_{c,s}} \tag{5.5}$$

and the design value of the column bending moment is given by;

$$M_{c,d} = M_{c,o} + \alpha M_{c,s} \tag{5.6}$$

5.4. DESIGN METHOD

There are two approaches to the problem of calculation of collapse loads, which can be conveniently called the "equilibrium" and the "geometry" approaches respectively. In the equilibrium approach, by satisfying the equilibrium equations and yield conditions, evaluation of the collapse load is possible without considering the mode of deformation. On the contrary, collapse load is evaluated by considering the mode of deformation. Collapse load predicted by the equilibrium method is always on the low side of the exact collapse load, while the one predicted by the geometry approach is always on the high side.

The equilibrium method is based on the lower-bound theorem, which states that if a stress distribution throughout the structure can be found so that it is everywhere internally in equilibrium and balances certain external loads and at the same time does not violate the yield conditions, then those loads will be carried safely by the structure. The corresponding load multiplier is called statically admissible multiplier. As a consequence, the collapse multiplier can be found as the maximum statically admissible multiplier.

On the contrary, the geometry approach is based on the upper-bound theorem, which states that if an estimate of the plastic collapse load or the corresponding multiplier is made by equating internal rate of dissipation of energy to the rate at which external forces do work in any postulated kinematically admissible mechanism of deformation, then the estimate will be either high or correct. As a consequence, the collapse multiplier can be found as the minimum kinematically admissible multiplier.

The structural design oriented to the control of the failure mode is a relatively recent problem arised from seismic design needs, which has been mainly faced through simplified rules provided in seismic codes. Those rules have not been derived by means of the limit design approach and in most cases they do not allow to attain the requested failure mechanism.

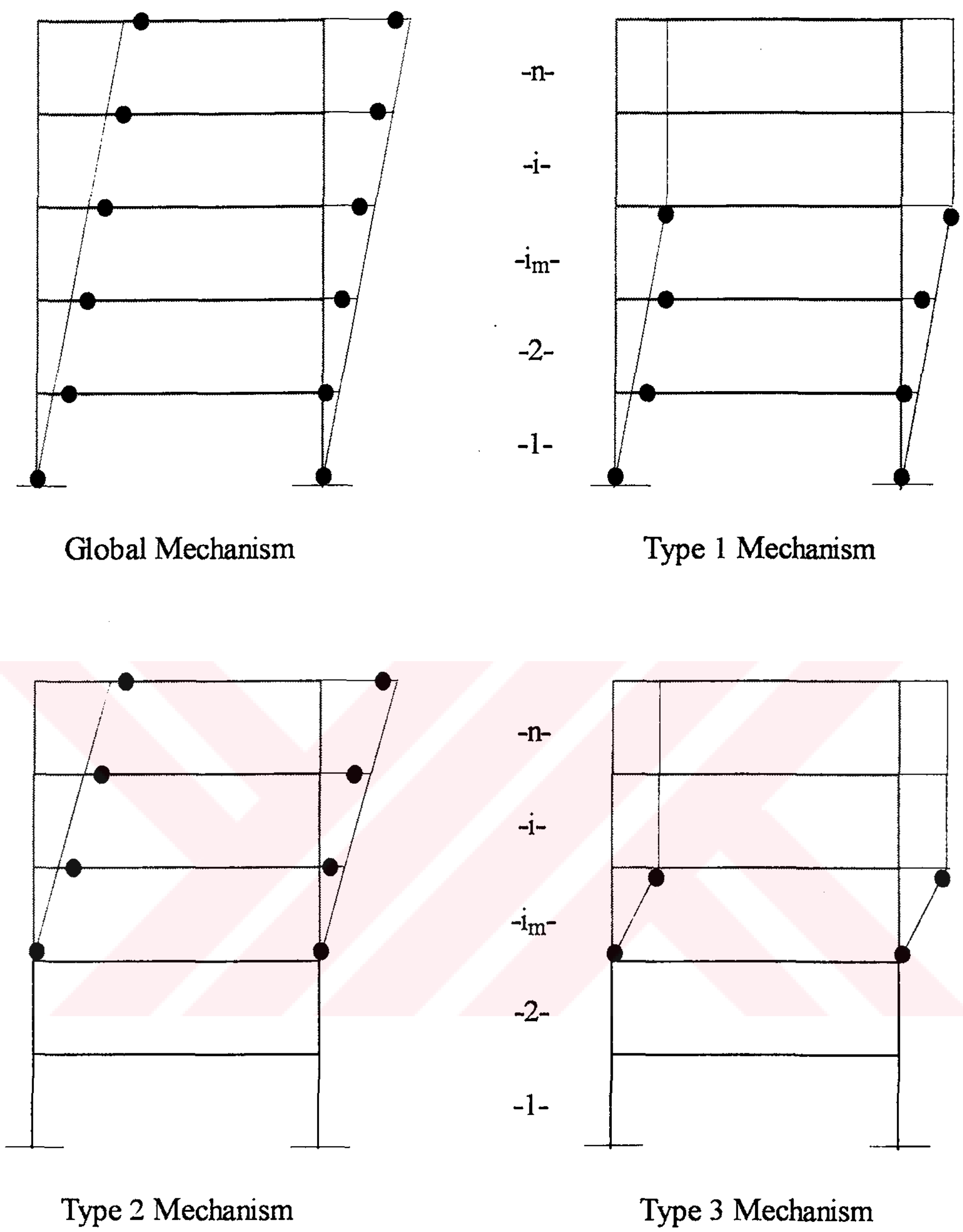


Figure 5.4. *The Collapse Mechanism Typologies in the Design Method of Mazzolani and Piluso* [9]

Starting from these considerations, a new design method has been proposed by Mazzolani and Piluso [9] for the control of the failure mode of seismic resistant steel frames. It is based on the observation that the collapse mechanisms of frames under horizontal forces can be considered belonging to three main typologies.

The collapse mechanism of the global type is a particular case of the type two mechanism. As a consequence, the control of the failure mode can performed through the analysis of $3n_p$ mechanisms (being n_p the number of storeys). Beam sections are assumed to be designed in order to resist vertical loads. Therefore, the values of plastic section modulus of columns have to be defined in such a way that the kinematically admissible multiplier of the horizontal forces corresponding to the global mechanism is less than the ones corresponding to the other $3n_p$-1 kinematically admissible mechanism. It means that, according to the upper-bound theorem, it is the true collapse multiplier and the true collapse mechanism is represented by the global failure mode.[9]

5.4.1. Kinematically Admissible Multipliers

For any given kinematically admissible mechanism, the corresponding kinematically admissible multiplier of the horizontal forces can be found by means of the principle of virtual works;

$$\alpha \ \{F\}^T \ d\{s\} = \{M_p\}^T \ d\{\theta\} \tag{5.7}$$

where $d\{s\}$ is the virtual displacements corresponding to the given mechanism, $\{M_p\}$ is the vector of the plastic moments of the plastic hinges of the given mechanism, $\{F\}^T$ is the vector of horizontal forces acting on the given mechanism and $d\{\theta\}$ is the virtual rotations of the plastic hinges of the given mechanism.

For global type mechanism, kinematically admissible multiplier is introduced by the equations;

$$\alpha^{(g)} = \frac{\sum_{i=1}^{n_c} M_{p,i1}^{(r)} + \sum_{k=1}^{n_p} \left(\sum_{j=1}^{n_b} 2 \ M_{pb,jk} \right)}{\sum_{k=1}^{n_p} F_k \ h_k} \tag{5.8}$$

being n_c is the number of columns, $M^{(r)}_{p,i1}$ is the plastic moment -reduced for the presence of the axial load- of i'th column of the first storey, n_p is the number of storeys, n_b is the number of bays, $M_{pb,jk}$ is the plastic moment of the j'th beam of the k'th storey, F_k is the horizontal force acting at k'th storey, h_k is the elevation of the k'th storey.

The kinematically admissible multiplier corresponding to the i_m'th mechanism of type 1, type 2 and type 3 are given by equations (5.9), (5.10) and (5.11) respectively.[9]

$$\alpha^{(1)}_{i_m} = \frac{\sum_{i=1}^{n_c} M^{(r)}_{p,i1} + \sum_{k=1}^{i_m-1}\left(\sum_{j=1}^{n_b} 2\, M_{pb,jk}\right) + \sum_{i=1}^{n_c} M^{(r)}_{p,ii_m}}{\sum_{k=1}^{i_m} F_k\, h_k + h_{i_m} \sum_{k=i_m+1}^{n_p} F_k} \tag{5.9}$$

being $M^{(r)}_{p,ii_m}$ is the plastic moment -reduced for the presence of the axial load- of the i'th column of the i_m'th storey.

$$\alpha^{(2)}_{i_m} = \frac{\sum_{i=1}^{n_c} M^{(r)}_{p,ii_m} + \sum_{k=i_m}^{n_p}\left(\sum_{j=1}^{n_b} 2\, M_{pb,jk}\right)}{\sum_{k=i_m}^{n_p} F_k\left(h_k - h_{i_m} - 1\right)} \tag{5.10}$$

$$\alpha^{(3)}_{i_m} = \frac{2 \cdot \sum_{i=1}^{n_c} M^{(r)}_{p,ii_m}}{\sum_{k=i_m}^{n_p} F_k\, \Delta h_{i_m}} \tag{5.11}$$

where Δh_{i_m} is introduced with the notation;

$$\Delta h_{i_m} = h_{i_m} - h_{i_m -1} \tag{5.12}$$

5.5. DESIGN CONDITIONS

In order to design a frame failing in global mode, the cross-sections of columns have to be dimensioned in such a way that, according to the upper-bound theorem, the kinematically admissible horizontal forces multiplier corresponding to the global type mechanism is the minimum among all kinematically admissible multipliers. As a consequence, the following design conditions have to be imposed;

$$\alpha^{(g)} \leq \frac{\alpha^{(1)}_{i_m}}{\nu^{(1)}_{i_m}} \qquad i_m = 1,2,3,\ldots\ldots,n_p \tag{5.13}$$

$$\alpha^{(g)} \leq \frac{\alpha^{(2)}_{i_m}}{\nu^{(2)}_{i_m}} \qquad i_m = 1,2,3,\ldots\ldots,n_p \tag{5.14}$$

$$\alpha^{(g)} \leq \frac{\alpha^{(3)}_{i_m}}{\nu^{(3)}_{i_m}} \qquad i_m = 1,2,3,\ldots\ldots,n_p \tag{5.15}$$

Therefore, there are $3n_p$ design condition to be satisfied in the case of frame having n_p storeys. These conditions, which derive directly from application of the upper-bound theorem, will be integrated by condition related to technological limitations.

The coefficients $\nu^{(i)}_{i_m} > 1$ are introduced in order to take into account influence of second order effects on the value of the kinematically admissible multiplier of the i_m'th mechanism of the i'th type. The value $\nu^{(i)}_{i_m} = 1.15$ suggested with the exception $\nu^{(2)}_{1} = 1$ which corresponds to a mechanism coincident with the global one.

In order to avoid type one mechanisms the following notation is introduced;

$$\theta_1 = \sum_{k=1}^{n_p} \left(\sum_{j=1}^{n_b} 2\ M_{pb,jk} \right) \tag{5.16}$$

$$\xi_{i_m} = \sum_{k=1}^{i_m - 1} \left(\sum_{j=1}^{n_b} 2\ M_{pb,jk} \right) \tag{5.17}$$

$$\lambda_{i_m} = \frac{\sum_{k=1}^{n_p} F_k\ h_k}{\sum_{k=1}^{i_m} F_k\ h_k + h_{i_m} \sum_{k=i_m+1}^{n_p} F_k} \tag{5.18}$$

$$\rho_{i_m} = \frac{\sum_{i=1}^{n_c} M^{(r)}_{p,ii_m}}{\sum_{i=1}^{n_c} M^{(r)}_{p,i1}} \tag{5.19}$$

By means of the above notation, i_m'th condition to be satisfied in order to avoid type one collapse mechanisms can be written in the following form;

$$\rho^{(1)}_{i_m} \geq \frac{\left(v^{(1)}_{i_m} - \lambda_{i_m} \right) \sum_{i=1}^{n_c} M^{(r)}_{p,i1} + v^{(1)}_{i_m} \theta_1 - \lambda_{i_m} \xi_{i_m}}{\lambda_{i_m} \sum_{i=1}^{n_c} M^{(r)}_{p,i1}} \tag{5.20}$$

which has to be applied for i_m=1,2,3,.....,n_p.

In order to avoid type two mechanisms, a new series of parameters is introduced;

$$\theta_{i_m} = \sum_{k=i_m}^{n_p} \left(\sum_{j=1}^{n_b} 2\ M_{pb,jk} \right) \tag{5.21}$$

$$\gamma_{i_m} = \frac{\sum_{k=1}^{n_p} F_k\ h_k}{\sum_{k=1}^{n_p} F_k \left(h_k\ -\ h_{i_m - 1} \right)} \tag{5.22}$$

$$\rho_{i_m}^{(2)} = \frac{\sum_{i=1}^{n_c} M_{p,ii_m}^{(r)}}{\sum_{i=1}^{n_c} M_{p,i1}^{(r)}} \tag{5.23}$$

By means of the above parameters, the i_m'th condition to be satisfied in order to avoid type two collapse mechanisms can be written in the following form;

$$\rho_{i_m}^{(2)} \geq \frac{v_{i_m}^{(2)} \left(\sum_{i=1}^{n_c} M_{p,i1}^{(r)} + \theta_1 \right) - \gamma_{i_m} \theta_{i_m}}{\gamma_{i_m} \sum_{i=1}^{n_c} M_{p,i1}^{(r)}} \tag{5.24}$$

The following parameters are introduced to avoid type three mechanisms;

$$\beta_{i_m} = \frac{\sum_{k=1}^{n_p} F_k\ h_k}{\sum_{k=i_m}^{n_p} F_k\ \Delta h_{i_m}} \tag{5.25}$$

$$\rho_{i_m}^{(3)} = \frac{\sum_{i=1}^{n_c} M_{p,ii_m}^{(r)}}{\sum_{i=1}^{n_c} M_{p,i1}^{(r)}} \tag{5.26}$$

By means of these parameters, the i_m'th condition to satisfy in order to avoid type three collapse mechanisms is expressed by the following relation;

$$\rho_{i_m}^{(3)} \geq \nu_{i_m}^{(3)} \frac{\sum_{i=1}^{n_c} M_{p,i1}^{(r)} + \theta_1}{2\beta_{i_m} \sum_{i=1}^{n_c} M_{p,i1}^{(r)}} \tag{5.27}$$

According to the above formulations, the $3n_p$ design conditions have been derived directly from the application of the upper-bound theorem. In particular, for each storey there are three design conditions to be satisfied because three collapse mechanism typologies have been considered. As, for each storey, these design conditions have to be contemporary satisfied, the ratio;

$$\rho_{i_m} = \frac{\sum_{i=1}^{n_c} M_{p,ii_m}^{(r)}}{\sum_{i=1}^{n_c} M_{p,i1}^{(r)}} \tag{5.28}$$

between the sum of the reduced plastic moments of columns of the i_m'th storey and the same sum corresponding to the first storey columns allows to satisfy the above design conditions if the following relation is verified;

$$\rho_{i_m} = \max\left\{\rho_{i_m}^{(1)},\ \rho_{i_m}^{(2)},\ \rho_{i_m}^{(3)}\right\} \tag{5.29}$$

As the section of columns only decrease along the height of the frame, the values of ρ_{i_m} (with i_m=1,2,.....,n_p) obtained by means of the conditions derived through the application of the upper-bound theorem have to be modified in order to satisfy the following technological limitation;

$$\rho_1 \geq \rho_2 \geq \rho_3 \geq \ldots\ldots \geq \rho_{n_p} \tag{5.30}$$

5.6. DESIGN ALGORITHM

It has been pointed that the upper-bound theorem allows defining condition for each undesired collapse mechanism, regarding the ratio of the sum of the reduced plastic moments of the k'th storey column and at the same sum corresponding to the first storey, to be satisfied in order to avoid the undesired failure mode. As three different collapse mechanism typologies have been considered, there are $3n_p$ design conditions to be satisfied which are provided by relations (5.20), (5.24) and (5.27). These design conditions have to be integrated by the technological condition (5.30). The above mentioned relations can be used in order to design a frame failing in global mode and, therefore, having an ultimate multiplier of the horizontal forces expressed by equation (5.8). The main steps of the algorithm to solve this problem are given below.

1. Computation of the storey functions θ_{i_m}, β_{i_m}, γ_{i_m}, λ_{i_m} and ξ_{i_m}, which are provided by the equations (5.21), (5.25), (5.22), (5.18) and (5.17) respectively.
2. Computation, through equation (5.8) , of a tentative value α_t of the ultimate multiplier corresponding to the global mechanism by imposing that the reduced plastic moment of the first storey columns is not less than the plastic moment of the beams.
3. Computation of the limit values $\rho_{i_m}^{(1)}$, $\rho_{i_m}^{(2)}$ and $\rho_{i_m}^{(3)}$ provided by equations (5.20), (5.24) and (5.27) respectively.
4. Computation of the value of ρ_{i_m} which allows to avoid failure modes corresponding to the three examined collapse mechanism typologies mentioned by the equation (5.29).
5. Modification of the computed values of ρ_{i_m} in order to satisfy the condition (5.30).
6. Computation of the corresponding kinematically admissible multiplier $\alpha_{i_m}^{(1)}$, $\alpha_{i_m}^{(2)}$ and $\alpha_{i_m}^{(3)}$ provided by equations (5.9), (5.10) and (5.11) respectively.
7. Computation of the ultimate multiplier as the minimum among all kinematically admissible multipliers;

$$\alpha_u = \min\left\{\alpha_{i_m}^{(1)}, \alpha_{i_m}^{(2)}, \alpha_{i_m}^{(3)} \text{ with } i_m = 1,2,3,\ldots\ldots, n_p\right\} \tag{5.31}$$

8. If the condition;

$$\left|\alpha_u - \alpha_t\right| > 0.01 \tag{5.32}$$

is verified, then the value of $\sum_{i=1}^{n_c} M_{p,i1}^{(r)}$ corresponding, through equation (5.8) to α_u has to be computed and procedure has to be repeated starting from point three. In the opposite case, it can be assumed $\alpha^{(g)}=\alpha_u$ and the column sections can be derived according to the evaluated plastic moments .[9]

5.7. WORKED EXAMPLES

In order to make introduced method explicit, two worked examples are illustrated. A computer program is provided at Appendix A to make all those computations described at section 5.5 easier. Number of storeys, number of bays, plastic moment of beams and reduced plastic moments of columns are given to the program as data and new plastic moments of columns are computed by the program.

5.7.1. Example 1

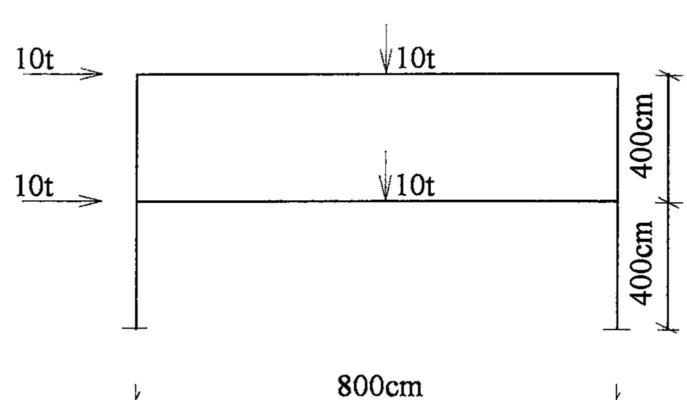


Figure 5.5. *The Analysed Moment Resisting Frame at Example 1*

The aim of the first example is to design a one bay-two storey moment resisting frame in order to obtain a global mechanism in the collapse condition. The characteristics of the frame are; the beams are IPB200 and the columns are IPB220. Horizontal forces exerted at each floor and point loads acting on the beams are 10t.

Firstly, the collapse state of the structure has to be determined. There are two methods in order to determine the collapse state of the structure; mechanism method and step by step method. In this example, mechanism method is preferred.

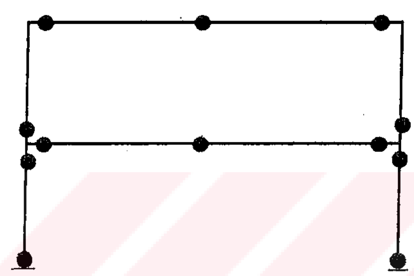


Figure 5.6. *Possible Plastic Hinge Locations*

The objective of the mechanism method is to select from all the possible failure modes of a system, the one that corresponds to the lowest possible plastic limit load. The number of possible independent failure mechanisms of a structure can be expressed by [10]

$$m = p - h \tag{5.33}$$

where m is the number of independent failure mechanism, p is the number of possible plastic hinges and h is the degree of statical indeterminacy. In this example;

$$m = 12 - 6 = 6$$

Two beam, two panel and two joint mechanisms are shown in figure 5.7.

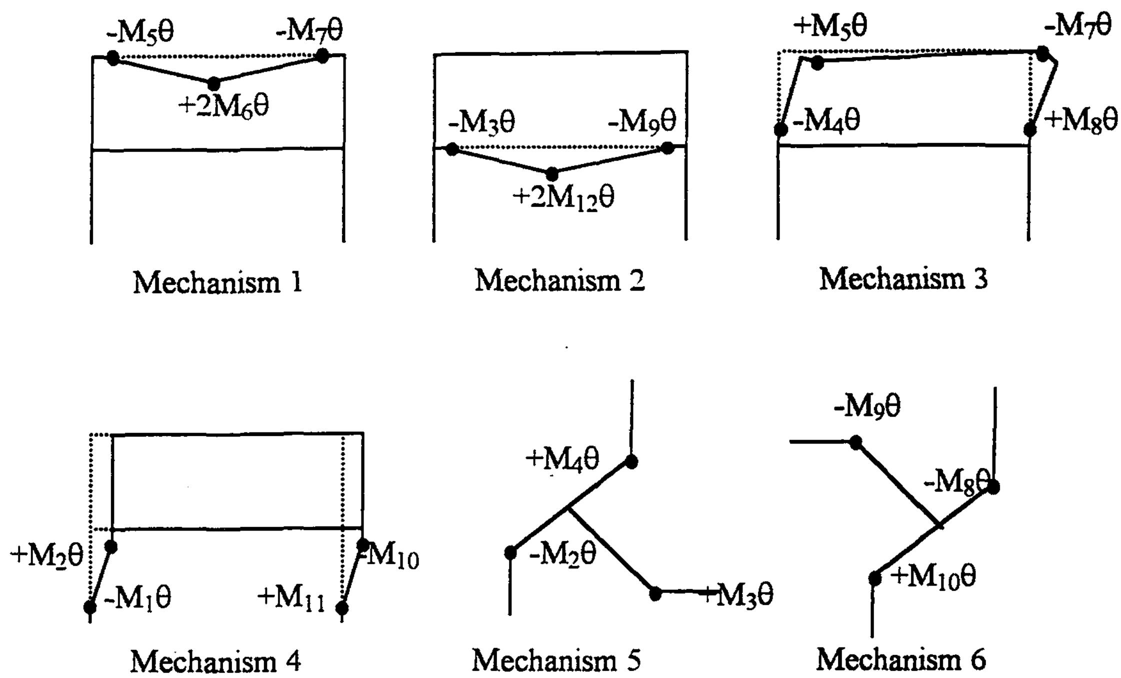


Figure 5.7. *Possible Mechanisms for the Structure*

Plastic moments of column sections are evaluated as following. Firstly plastic modulus of section is calculated as

$$W_{px} = 2\frac{A}{2}\frac{s_x}{2} \tag{5.34}$$

Plastic moment of column is determined by

$$M_{px} = \sigma_{yd}\ W_{px} \tag{5.35}$$

For IPB220;

$$W_{px} = 2\frac{91.1}{2}\frac{19.5}{2} = 888.23\ \text{cm}^3$$

$$M_{px} = 2.4 * 888.23 = 2131.74\ \text{tcm} = 21.31\ \text{tm}$$

Plastic moment of IPB200 is calculated 17.57 tm by the same method. Therefore the plastic moments of M_1, M_2, M_4, M_8, M_{10} and M_{11} are 21.31tm and M_3, M_5, M_6, M_7, M_9 and M_{12} are 17.57tm. According to the mechanism method, minimum admissible multiplier of the system is determined by equating virtual work done by external forces to virtual work absorbed by virtual hinges.

1^0) Beam Mechanisms;

Mechanism 1:
$$\frac{10\alpha}{8}\,\frac{1}{2}\,8*4\theta = -M_5\theta + 2M_6\theta - M_7\theta$$

$$20\alpha = 70.28 \;\Rightarrow\; \underline{\alpha = 3.514}$$

Mechanism 2:
$$\frac{10\alpha}{8}\,\frac{1}{2}\,8*4\theta = -M_3\theta + 2M_{12}\theta - M_9\theta$$

$$20\alpha = 70.28 \;\Rightarrow\; \underline{\alpha = 3.514}$$

2^0) Panel Mechanisms;

Mechanism 3:
$$10\alpha * 4\theta = -M_4\theta + M_5\theta - M_7\theta + M_8\theta$$

$$40\alpha = 77.76 \;\Rightarrow\; \underline{\alpha = 1.944}$$

Mechanism 4:
$$10\alpha * 4\theta + 10\alpha * 4\theta = -M_1\theta + M_2\theta - M_{10}\theta + M_{11}\theta$$

$$80\alpha = 85.04 \;\Rightarrow\; \underline{\alpha = 1.063}$$

3^0) Joint Mechanisms;

Mechanism 5:
$$0 = -M_2\theta + M_3\theta + M_4\theta$$

Mechanism 6: $0 = -M_8\theta - M_9\theta + M_{10}\theta$

4^0) Combined Mechanisms;

Mechanisms 1&4: $100\alpha\theta = -M_1\theta + M_2\theta - M_5\theta + 2M_6\theta - M_7\theta - M_{10}\theta + M_{11}\theta$

$$100\alpha = 155.32 \Rightarrow \underline{\alpha = 1.553}$$

Mechanisms 2,4&5: $100\alpha\theta = -M_1\theta + M_4\theta - M_9\theta - M_{10}\theta + M_{11}\theta + 2M_{12}\theta$

$$100\alpha = 137.75 \Rightarrow \underline{\alpha = 1.377}$$

Mechanisms 1,2,3,4,5&6: $160\alpha\theta = -M_1\theta + 2M_6\theta - 2M_7\theta - 2M_9\theta + M_{11}\theta + 2M_{12}\theta$

$$160\alpha = 183.08 \Rightarrow \underline{\alpha = 1.114}$$

As it is seen from above equations the minimum admissible multiplier is achieved at mechanism 4. The values obtained from other mechanism combinations are all bigger than the values given above. Therefore it is concluded that the collapse mechanism of the system is as mechanism 4 and the minimum admissible load multiplier is 1.063. From this point, the introduced method is used to make the structure to collapse at global mechanism.

Table 5.1. *Results Evaluated From the Computer Program*

STOREYS	ρ VALUES	UPDATED PLASTIC MOMENTS (tm)	
1	1.222	26.04	26.04
2	1.222	26.04	26.04

The multipliers of the plastic moments of column sections, which provides the frame collapse at global mechanism, are evaluated as 1.222 for both first and second storeys. As it is seen at table 5.1., the plastic moments of the columns should be at least 26.04tm in order to protect any collapse mechanism other than global one. Therefore the crossections of columns should be designed as IPB240. The plastic moment of IPB240 is;

$$W_{px} = 2\frac{111}{2}\frac{21.3}{2} = 1182.15 \text{ cm}^3$$

$$M_{px} = 2.4 * 1182.15 = 2837.16 \text{ tcm} = 28.37 \text{ tm}$$

In order to control the frame if it collapse at global mechanism or not, mechanism method is used again. Possible mechanisms for the structure are same as figure 5.7. Only difference between the preliminary design is M_1, M_2, M_4, M_8, M_{10} and M_{11} are 28.37tm.

1^0) Beam Mechanisms;

Mechanism 1: $$\frac{10\alpha}{8}\frac{1}{2}8 * 4\theta = -M_5\theta + 2M_6\theta - M_7\theta$$

$$20\alpha = 70.28 \Rightarrow \underline{\alpha = 3.514}$$

Mechanism 2: $$\frac{10\alpha}{8}\frac{1}{2}8 * 4\theta = -M_3\theta + 2M_{12}\theta - M_9\theta$$

$$20\alpha = 70.28 \Rightarrow \underline{\alpha = 3.514}$$

2^0) Panel Mechanisms;

Mechanism 3: $$10\alpha * 4\theta = -M_4\theta + M_5\theta - M_7\theta + M_8\theta$$

$$40\alpha = 91.88 \Rightarrow \underline{\alpha = 2.297}$$

Mechanism 4: $$10\alpha * 4\theta + 10\alpha * 4\theta = -M_1\theta + M_2\theta - M_{10}\theta + M_{11}\theta$$

$$80\alpha = 113.48 \Rightarrow \underline{\alpha = 1.419}$$

3^0) Joint Mechanisms;

Mechanism 5: $0 = -M_2\theta + M_3\theta + M_4\theta$

Mechanism 6: $0 = -M_8\theta - M_9\theta + M_{10}\theta$

4^0) Combined Mechanisms;

Mechanisms 1&4: $100\alpha\theta = -M_1\theta + M_2\theta - M_5\theta + 2M_6\theta - M_7\theta - M_{10}\theta + M_{11}\theta$

$$100\alpha = 183.76 \Rightarrow \underline{\alpha = 1.838}$$

Mechanisms 2,4&5: $100\alpha\theta = -M_1\theta + M_4\theta - M_9\theta - M_{10}\theta + M_{11}\theta + 2M_{12}\theta$

$$100\alpha = 166.19 \Rightarrow \underline{\alpha = 1.662}$$

Mechanisms 1,2,3,4,5&6: $160\alpha\theta = -M_1\theta + 2M_6\theta - 2M_7\theta - 2M_9\theta + M_{11}\theta + 2M_{12}\theta$

$$160\alpha = 197.30 \Rightarrow \underline{\alpha = 1.233}$$

As it is seen from above equations the minimum admissible multiplier is achieved at the combination of mechanisms 1,2,3,4,5 and 6, which is also called the global mechanism. This time the minimum admissible load multiplier is 1.233. Before the introduced method is applied to the frame it was 1.063. The increase in the minimum admissible load multiplier shows that the frame collapses at higher loads.

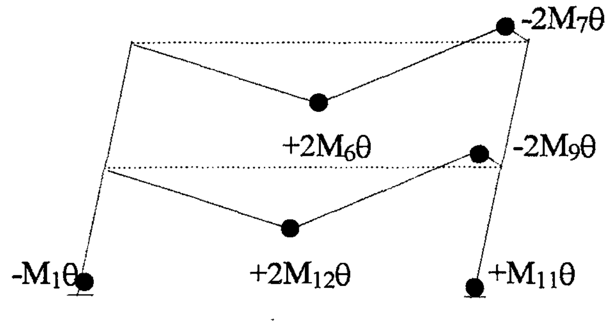


Figure 5.8. *Collapse Mechanism of the Analysed Frame*

5.7.2. Example 2

The computer program is not only for small frames. It can also be applied to multi-storey and multi-bay structures. Hence another moment resisting frame with 3 bay-5 storey, which is represented in figure 5.9., is analysed in order to obtain a global mechanism in the collapse condition. The frame is chosen such that it is preliminary designed according to elastic behaviour. This time the collapse mechanism is not firstly determined, but global collapse mechanism is directly reached by means of the computer program. The cross-sections of the beams of first and second storeys are IPB220, third and fourth storeys are IPB200, and the top storey is IPB180. The crossections of columns of first storey is IPB280, second and third storeys are IPB260 while the others are IPB240. The uniform vertical load acting on the beam is 5 t/m.

Calculation of plastic moment for IPB280;

$$W_{px} = 2\frac{144}{2}\frac{24.9}{2} = 1792.8 \text{ cm}^3$$

$$M_{px} = 2.4 * 1792.8 = 4302.72 \text{ tcm} = 43.03 \text{ tm}$$

Plastic moments of crossections are calculated as 33.69tm for IPB260, 28.37tm for IPB240, 21.31 for IPB220, 17.57 for IPB200 and 12.56 for IPB180.

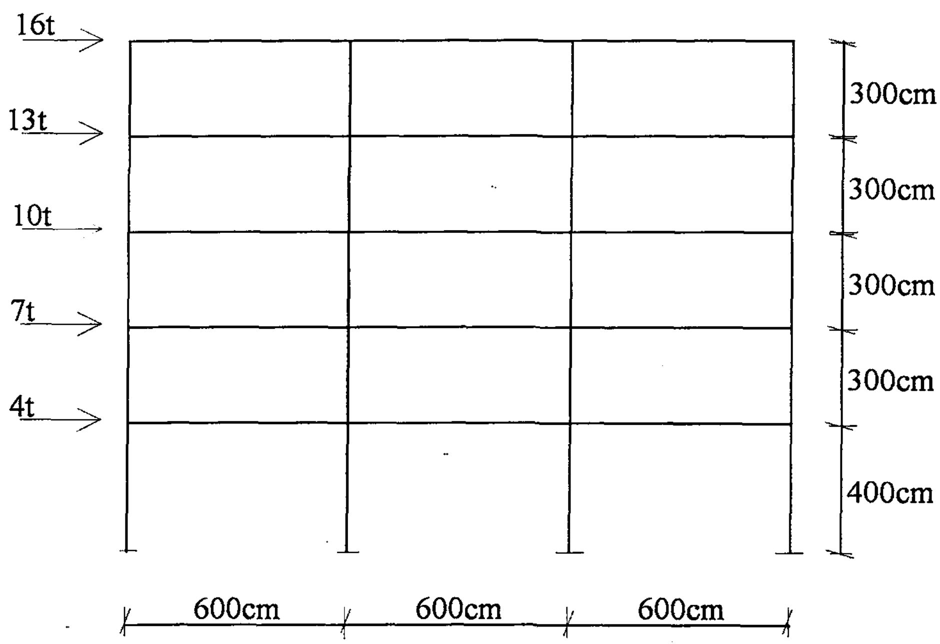


Figure 5.9. *The Analysed Moment Resisting Frame at Example 2*

These values are given to the program at appendix B as data and by means of the program, convergence is achieved after two iterations. The multiplier of the horizontal forces corresponding, as kinematically admissible to the global mechanism is

$$\alpha^{(g)} = 1.293$$

At the end of the program, the plastic moments of the columns are updated to the values given in table 5.2.

Table 5.2. *Updated Plastic Moments of Columns*

STOREY	UPDATED PLASTIC MOMENTS (tm)
1	55.96
2	43.81
3	43.81
4	36.90
5	25.82

New column sections chosen by means of the described method is given in table 5.3.

Table 5.3. *Column Sections Obtained by means of the Described Design Method*

STOREY	COLUMN SECTIONS
1	IPB320
2	IPB320
3	IPB320
4	IPB280
5	IPB240

VI. CONCLUSION

Earthquakes resistant design begins at conceptual stage. It is hard to overcome bad results of any wrong decision at this stage. For example, if the distance between center of gravity and center of stiffness is big, additional dynamic loads are created on the structure during earthquakes. Therefore, the structure should be as symmetric as possible.

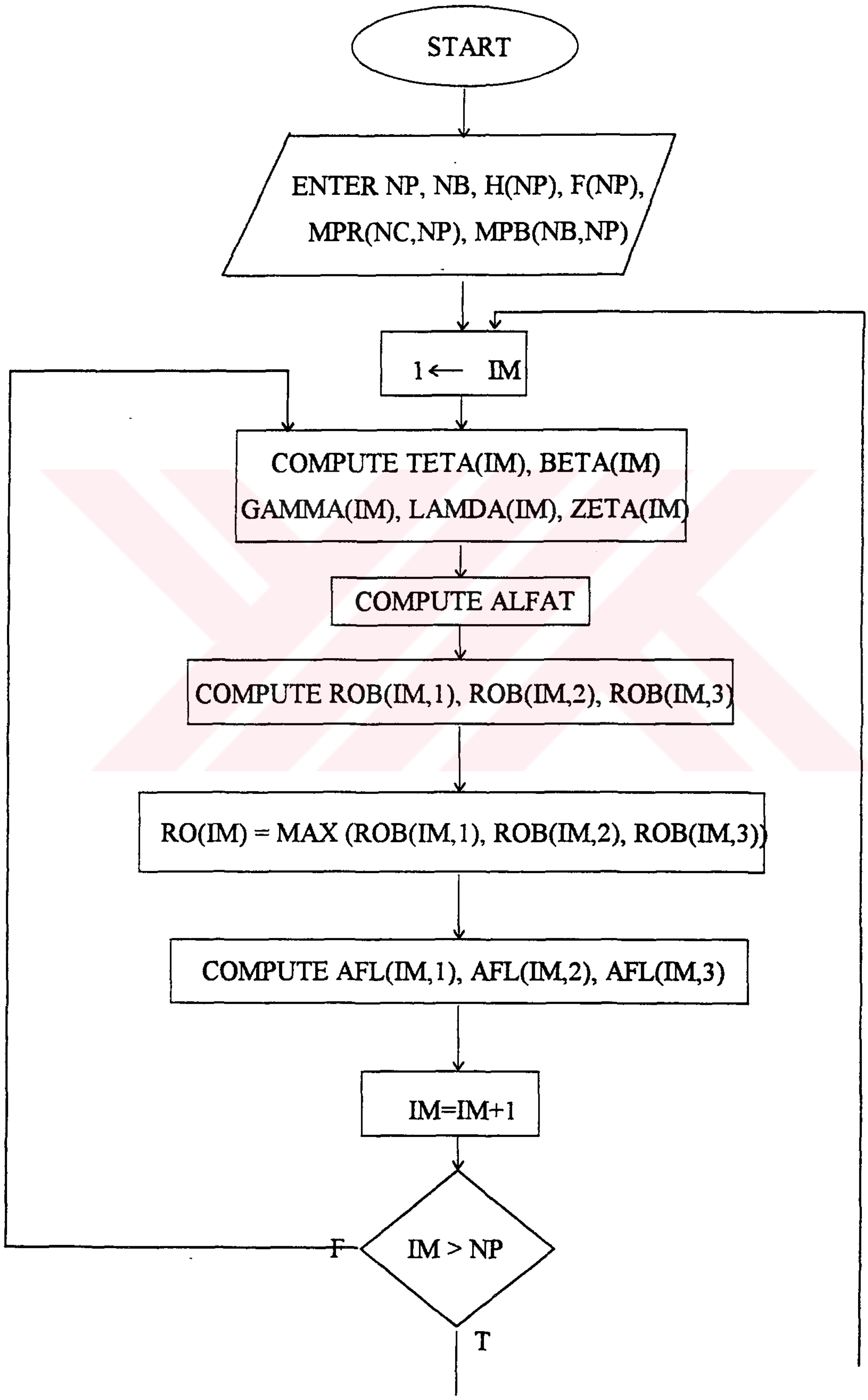
Elastic analysis is important in order to understand the behaviour of the structure. But during earthquakes, dynamic load exceeds elastic limit and the structure begins to deform in elastic-plastic manner and if it continues to increase, fully plastic deformation occurs. Hence, elastic analysis is not sufficient to design earthquake resistant structures. Therefore the method of F.M. Mazzolani and V. Piluso, which is based on plastic analysis and global ductility, is advised to use in seismic design of steel structures. The aim of the method is to control the failure mode of seismic resistant steel frames. It is based on the observation that the collapse mechanisms of frames under horizontal forces can be considered belonging to three main typologies as indicated in figure 5.2. As a consequence, the control of the failure mode can performed through the analysis of $3n_p$ mechanisms.

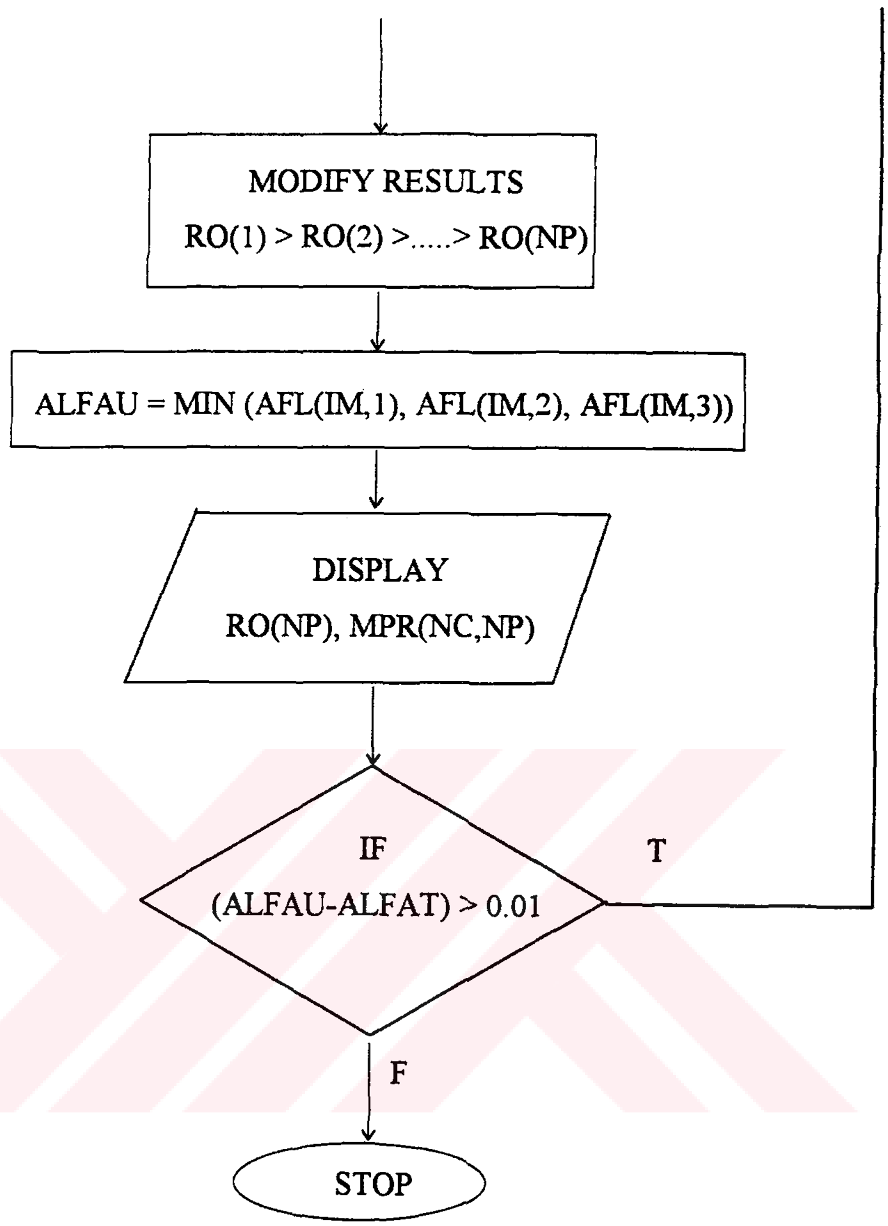
Type of collapse mechanism plays a fundamental role in order to determine the earthquake resistance of structures, because it influences the value of the available global ductility and energy dissipation capacity. The beams and columns should be so designed that plastic hinges will form firstly at beams, then the columns starting from the highest storey to bottom one. Consequently, global mechanism is evaluated and maximum absorption of energy is reached.

In order to design a frame failing in global mode, the cross-sections of columns have to be dimensioned in such a way that, the kinematically admissible horizontal forces multiplier corresponding to the global type mechanism is the minimum among all kinematically admissible multipliers. A computer program is provided at Appendix B which can be used on structures designed using elastic or plastic theories. It ensures the structures to collapse at global mechanism. Kinematically admissible multiplier of global mechanism and other mechanisms are calculated by the program. If the minimum admissible multiplier is different from the multiplier of global mechanism, the plastic moments of columns are increased by an amplification factor. At the end, new column sections satisfying global mechanism are determined according to increased plastic moments of columns. Two worked examples are given at section 5.7 in order to make the method explicit.

APPENDIX A

Flowchart of the Program:




MODIFY RESULTS
RO(1) > RO(2) >.....> RO(NP)
ALFAU = MIN (AFL(IM,1), AFL(IM,2), AFL(IM,3))
DISPLAY
RO(NP), MPR(NC,NP)
IF
(ALFAU-ALFAT) > 0.01
T
F
STOP

APPENDIX B

Computer Program

```
REAL TETA(10),MPB(10,10),F(10),H(10),BETA(10),GAMMA(10),R(10)
REAL LAMDA(10),ZETA(10),MPR(10,10),ROB(10,3),RO(10),AFL(10,3)
REAL FH,DH,FDH,FHF,HF,FHB,FB,MPI,ALFAT,MPM,FDD,ALFAU,S,P(10)
INTEGER IM,M,I,NP,K,J,NB,NC,L
PRINT*, 'ENTER THE NUMBER OF STOREYS'
READ '(I2)',NP
PRINT*, 'ENTER THE NUMBER BAYS'
READ '(I2)',NB
NC=NB+1
OPEN (UNIT=1,FILE='FORCE.DAT')
OPEN (UNIT=3,FILE='MBEAM.DAT')
OPEN (UNIT=5,FILE='MOCOL.DAT')
OPEN (UNIT=6,FILE='MC.OUT')
READ (1,11) (H(I),F(I),I=1,NP)
FORMAT (F4.1,3X,F5.1)
READ (3,13) ((MPB(J,K),J=1,NB),K=1,NP)
FORMAT (3(F5.2,3X))
READ (5,14) ((MPR(J,K),J=1,NC),K=1,NP)
FORMAT (4(F5.2,3X))
WRITE (6,12)
FORMAT ('STOREYS            UPDATED PLASTIC MOMENTS OF COLUMNS')
IM=1
M=IM-1
I=IM+1
TETA(IM)=0
FH=0
IF (M.EQ.0) THEN
  DH=H(IM)
ELSE
  DH=H(IM)-H(M)
ENDIF
FDH=0
FHF=0
DO 5 K=IM,NP
   DO 10 J=1,NB
      TETA(IM)=TETA(IM)+2*MPB(J,K)
   CONTINUE
CONTINUE
DO 15 K=1,NP
   FH=FH+F(K)*H(K)
CONTINUE
DO 20 K=IM,NP
   FDH=FDH+F(K)*DH
CONTINUE
BETA(IM)=FH/FDH
```

```
DO 25 K=1,NP
   IF (M.EQ.0) THEN
     HF=F(K)*H(K)
   ELSE
     HF=F(K)*H(K)-F(K)*H(M)
   ENDIF
   IF (HF.LT.0) HF=0
   FHF=FHF+HF
CONTINUE
GAMMA(IM)=FH/FHF
FHB=0
DO 30 K=1,IM
   FHB=FHB+F(K)*H(K)
CONTINUE
IF (I.GT.NP) THEN
   FB=0
ELSE
   FB=0
   DO 35 K=I,NP
      FB=FB+F(K)
   CONTINUE
ENDIF
LAMDA(IM)=FH/(FHB+H(IM)*FB)
ZETA(IM)=0
IF (IM.EQ.1) GO TO 40
DO 45 K=1,M
   DO 50 J=1,NB
      ZETA(IM)=ZETA(IM)+2*MPB(J,K)
   CONTINUE
CONTINUE
MPI=0
DO 55 K=1,NC
   MPI=MPI+MPR(K,1)
CONTINUE
ALFAT=(MPI+TETA(1))/FH
R(IM)=((1.15-LAMDA(IM))*MPI)+(1.15*TETA(1))
ROB(IM,1)=(R(IM)-(LAMDA(IM)*ZETA(IM)))/(LAMDA(IM)*MPI)
IF (IM.EQ.1) THEN
  P(IM)=(MPI+TETA(1))
ELSE
  P(IM)=1.15*(MPI+TETA(1))
ENDIF
ROB(IM,2)=(P(IM)-(GAMMA(IM)*TETA(IM)))/(GAMMA(IM)*MPI)
ROB(IM,3)=(1.15*(MPI+TETA(1)))/(2*BETA(IM)*MPI)
RO(IM)=ROB(IM,1)
DO 70 K=2,3
   IF (RO(IM).LT.ROB(IM,K)) RO(IM)=ROB(IM,K)
CONTINUE
MPM=0
DO 75 K=1,NC
   MPM=MPM+MPR(K,IM)
CONTINUE
AFL(IM,1)=(MPI+ZETA(IM)+MPM)/(FHB+(H(IM)*FB))
FDD=0
```

```
DO 80 K=IM,NP
   IF (M.EQ.0) THEN
     FDD=FDD+(F(K)*H(K))
   ELSE
     FDD=FDD+(F(K)*H(K))-(F(K)*H(M))
   ENDIF
CONTINUE
AFL(IM,2)=(MPM+TETA(IM))/FDD
AFL(IM,3)=(2*MPM)/FDH
IM=IM+1
IF (IM.LE.NP) GO TO 2
DO 85 L=1,NP
   DO 90 K=L,NP
      IF (RO(L).LT.RO(K)) RO(L)=RO(K)
   CONTINUE
CONTINUE
ALFAU=AFL(1,1)
DO 95 IM=1,NP
   DO 100 K=1,3
      IF (ALFAU.GT.AFL(IM,K)) ALFAU=AFL(IM,K)
   CONTINUE
CONTINUE
IF (RO(1).EQ.1) THEN
  CONTINUE
ELSE
  DO 105 K=1,NP
     DO 110 J=1,NC
        MPR(J,K)=RO(K)*MPR(J,K)
     CONTINUE
  CONTINUE
ENDIF
DO 115 K=1,NP
    WRITE (6,120) K,RO(K),(MPR(J,K),J=1,NC)
    FORMAT (2X,I2,6X,F5.3,3X,4(F6.2,3X))
CONTINUE
WRITE (6,121)
FORMAT (/)
S=ALFAU-ALFAT
IF (ABS(S).GT.0.001) GO TO 1
STOP
END
```

Elevation of Storeys and Horizontal Force Data

```
 4.0     4.0
 7.0     7.0
10.0    10.0
13.0    13.0
16.0    16.0
```

Data of Plastic Moments of Beams

21.31	21.31	21.31
21.31	21.31	21.31
17.57	17.57	17.57
17.57	17.57	17.57
12.56	12.56	12.56

Data of Reduced Plastic Moments of Columns

43.03	43.03	43.03	43.03
33.69	33.69	33.69	33.69
33.69	33.69	33.69	33.69
28.37	28.37	28.37	28.37
28.37	28.37	28.37	28.37

Result of the Computer Program

STOREYS		UPDATED PLASTIC MOMENTS OF COLUMNS			
1	1.284	55.27	55.27	55.27	55.27
2	1.284	43.27	43.27	43.27	43.27
3	1.284	43.27	43.27	43.27	43.27
4	1.284	36.44	36.44	36.44	36.44
5	1.060	30.08	30.08	30.08	30.08
1	1.012	55.96	55.96	55.96	55.96
2	1.012	43.81	43.81	43.81	43.81
3	1.012	43.81	43.81	43.81	43.81
4	1.012	36.90	36.90	36.90	36.90
5	0.859	25.82	25.82	25.82	25.82

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ÖZET

Tarih boyunca insanlığın en çok acı çektiği felaketlerden biri depremdir. Depremi önlemek mümkün değildir ama alınacak bir takım tedbirlerle zararlarından korunulabilir.

Yapı türleri içinde, çelik yapılar depreme karşı en iyi davranışı gösterenleridir ancak yine de bir takım önlemler almak zorunludur. Yapının hem plan hem de yükseklikte düzenli ve simetrik olması depreme karşı daha dayanıklı olmasını sağlar. Ayrıca deprem anında açığa çıkacak enerjiyi emmesi için bir takım elemanlar yapıya ilave edilebilir. Bunların en çok kullanılanı çelik merkezsel veya dışmerkez çapraz sistemleridir.

Göçme mekanizması, yapıların enerji emme kapasitesini etkiler. Dolayısıyla da çelik yapıların depreme karşı projelendirilmesinde büyük rol oynar. Kiriş uçlarında ve en alt kolonların alt uçlarında plastik mafsal oluşması (global mekanizma) şeklinde göçme öngörülen çerçeve binalar referans olarak ele alınabilir. Çünkü, bu yapılar büyük deprem yükü altında bile yeterli sünekliği gösterebilmektedir. Bu çalışmada, ECCS'nin tavsiyeleri arasında yer alan global mekanizma metodu esas alınarak bir bilgisayar programı geliştirilmiş ve çalışırlığı gösterilmiştir. Bu program, elastik veya plastik teoriye göre ön boyutlandırması yapılmış yapılar üzerinde kullanılabilir. Söz konusu program, yapılarda global mekanizmanın gerçekleşmesine imkan vermektedir. Eğer sistemin göçme mekanizması global mekanizmadan farklı ise, kolonların plastik momentleri programın hesap ettiği çarpan kullanılarak yeniden belirlenir. Sonuç olarak, belirlenen kolon plastik momentlerine göre hesaplanmış ve global mekanizmayı sağlayan yeni kolon boyutları seçilir.

İNGİLİZCE ABSTRAKT (en fazla 250 sözcük) :

ABSTRACT

One of the most important disasters that humanity suffered throughout the history is earthquake. Although it is impossible to overcome earthquake, protection from its harms is possible by taking some precautions.

Structural steel behaves the best for earthquake resistance among all structural materials, but considerable care is needed in design and detailing of framing systems. Regular and symmetric structures in both plan and elevation behave much better than non-regular ones. In addition, some structural elements such as concentrically braced frames and eccentrically braced frames are added to structure in order to absorb earthquake energy.

The collapse mechanism influences the energy dissipation capacity of the structure. Therefore, it plays a very important role in seismic design of steel structures. The case in which a framed structure fails according to global type mechanism can be adopted as the reference case, because it is considered to exhibit enough ductility to withstand severe earthquakes. In the thesis, a computer program is developed and verified based on the method of global mechanism in ECCS recommendations. The program is used on the structures which are preliminary designed by elastic or plastic theories. It ensures the structures to collapse at global mechanism. In the program, if the collapse mechanism of the system is different from global mechanism, the plastic moments of columns are modified by an amplification factor. Consequently, new column sections satisfying global mechanism are determined according to the increased plastic moments of columns.

TÜRKÇE ABSTRAKT (en fazla 250 sözcük) :

(TÜBİTAK/TÜRDOK'un Abstrakt Hazırlama Kılavuzunu kullanınız.)