

**ANNEX**  
**SPECIFICATION FOR STRUCTURES TO BE BUILT IN DISASTER AREAS**  
**CHAPTER I - GENERAL PRINCIPLES**

**1.1. SCOPE**

**1.1.1** - Requirements of this Specification shall be applicable to newly constructed buildings as well as to existing buildings.

**1.1.2** - Provisions to be applied for the prevailing buildings of which the intended use and / or carrier system shall be changed, the performance shall be evaluated before or after the earthquake and buildings to be strengthened were given in **Section - 7**.

**1.1.3** - Requirements of this Specification shall be applicable to reinforced concrete (cast-in-situ and pre - stressed or non - pre - stressed prefabricated) buildings, structural steel buildings and building-like structures, and timber, masonry and adobe buildings.

**1.1.4** - Minimum conditions and rules to be implied to wooden buildings and such kind of buildings shall be determined by the Ministry of Public Works and Settlement until the provisions of relevant specification shall be put into effect and projects of them shall be arranged according to those basis.

**1.1.5** - In addition to buildings and building-like structures, non-building structures permitted to be designed in accordance with the requirements of this Specification are limited with those specified in 2.12 of Chapter 2. In this context bridges, dams, harbor structures, tunnels, pipelines, power transmission lines, nuclear power plants, natural gas storage facilities, underground structures and other structures designed with analysis and safety rules that are different than those for buildings are outside the scope of this Specification.

**1.1.6** - Buildings equipped with special system and equipment between foundation and soil for the purpose of isolation of building structural system from the earthquake motion, and buildings incorporating other active and passive control systems are out of the scope of this Specification.

**1.1.7** - Rules to be applied to structures which are outside the scope shall be specifically determined by the Ministries supervising the constructions and such structures shall be designed to those rules until their own special specifications are prepared.

The objective of this Part of the Specification is to define the minimum requirements for the earthquake resistant design and construction of buildings and building-like of structures or their parts subjected to earthquake ground motion.

**1.2. GENERAL PRINCIPLES**

**1.2.1** - The general principle of earthquake resistant design to this Specification is to prevent structural and non-structural elements of buildings from any damage in low-intensity earthquakes; to limit the damage in structural and non-structural elements to repairable levels in medium-intensity earthquakes, and to prevent the overall or partial collapse of buildings in high-intensity earthquakes in order to avoid the loss of life. Performance criteria based on the evaluation and reinforcement of the existing buildings is stated in **Chapter 7**.

**1.2.2** - The design earthquake considered in this Specification, corresponds to *high-intensity* earthquake defined in **1.2.1** above. For buildings with Building Importance Factor of **I=1** in accordance with **Chapter 2, Table 2.3**, the probability of exceedance of the design earthquake within a period of 50 years is 10 %. Earthquakes with different the probability of exceedance are defined in **Chapter 7** in order to be considered in the evaluation and reinforcement of the existing buildings.

**1.2.3** - Seismic zones cited in this Specification are the first, second, third and fourth seismic zones depicted in *Seismic Zoning Map of Turkey* prepared by the Ministry of Public Works and Settlement and issued by the decree of the Council of Ministers dated 18/04/1996 and numbered 96/8109.

**1.2.4** - Buildings to be built in the seismic zones according to this Specification shall be in accordance with Turkish Standards and “General Technical Specification” of the Ministry of Public Works and Settlement in terms of materials and labor.

## CHAPTER 2 - ANALYSIS REQUIREMENTS FOR EARTHQUAKE RESISTANT BUILDINGS

### 2.0. NOTATION

|              |   |   |
|--------------|---|---|
| $A(T)$       | = | Spectral Acceleration Coefficient   |
| $A_o$        | = | Effective Ground Acceleration Coefficient   |
| $B_a$        | = | Design internal force component of a structural element in the direction of its principal axis a  |
| $B_{ax}$     | = | Internal force component of a structural element in the direction of its principal axis a due to earthquake in x direction  |
| $B_{ay}$     | = | Internal force component of a structural element in the direction of its principal axis a due to earthquake in y direction perpendicular to x direction   |
| $B_b$        | = | Design internal force component of a structural element in the direction of its principal axis b  |
| $B_{bx}$     | = | Internal force component of a structural element in the direction of its principal axis b due to earthquake in x direction  |
| $B_{by}$     | = | Internal force component of a structural element in the direction of its principal axis b due to earthquake in y direction perpendicular to x direction   |
| $B_B$        | = | Any size calculated with the combination of mode contributions in the Mode-Superposition Method   |
| $B_D$        | = | Amplified value of $B_B$  |
| $D_i$        | = | Amplification factor to be applied in Equivalent Seismic Load Method to $\pm 5\%$ additional eccentricity at i'th storey of a torsionally irregular building  |
| $d_{fi}$     | = | Displacement calculated at i'th storey of building under fictitious loads $F_{fi}$  |
| $d_i$        | = | Displacement calculated at i'th storey of building under design seismic loads   |
| $F_{fi}$     | = | Fictitious load acting at i'th storey in the determination of fundamental natural vibration period  |
| $F_i$        | = | Design seismic load acting at i'th storey in Equivalent Seismic Load Method   |
| $f_e$        | = | Equivalent seismic load acting at the mass centre of the mechanical and electrical equipment  |
| $g$          | = | Acceleration of gravity ( $9.81 \text{ m/s}^2$ )  |
| $g_i$        | = | Total dead load at i'th storey of building  |
| $H_i$        | = | Height of i'th storey of building measured from the top foundation level (In buildings with rigid peripheral basement walls, height of i'th storey of building measured from the top of ground floor level) |
| $H_N$        | = | Total height of building measured from the top foundation level (In buildings with rigid peripheral basement walls, total height of building measured from the top of the ground floor level)               |
| $H_w$        | = | Total height of partition measured from under the foundation or from the ground floor   |
| $h_i$        | = | Height of i'th storey of building [m]   |
| $I$          | = | Building Importance Factor  |
| $\ell_w$     | = | Length of partition or piece of strap partition on plan   |
| $M_n$        | = | Modal mass of the n'th natural vibration mode   |
| $M_{xn}$     | = | Effective participating mass of the n'th natural vibration mode of building in the x earthquake direction considered  |
| $M_{yn}$     | = | Effective participating mass of the n'th natural vibration mode of building in the y earthquake direction considered  |
| $m_i$        | = | i'th storey mass of building ( $m_i = w_i / g$ )  |
| $m_{\theta}$ | = | In the case where floors are modeled as rigid diaphragms, mass moment of inertia around vertical axis passing through unshifted mass centre of i'th storey of building                                      |
| $N$          | = | Total number of stories of building from the foundation level (In buildings with rigid peripheral basement walls, total number of stories from the ground floor level)                                      |

|                    |   |  |
|--------------------|---|--|
| $n$                | = | Live Load Participation Factor   |
| $q_i$              | = | Total live load at i'th storey of building   |
| $R$                | = | Structural Behavior Factor   |
| $R_{down}, R_{up}$ | = | In case single storey frames with columns hinged at the top are used as top floor (roof top) in cast – in – situ reinforced concrete, prefabricated and structural steel buildings, R factors defined for lower stories and the top floor, respectively. |
| $R_{NC}$           | = | Structural Behavior Factor defined in <b>Table 2.5</b> for the case where entire seismic loads are carried by frames of nominal ductility level  |
| $R_{YP}$           | = | Structural Behavior Factor defined in <b>Table 2.5</b> for the case where entire seismic loads are carried by walls of high ductility level  |
| $R_a(T)$           | = | Seismic Load Reduction Factor  |
| $S(T)$             | = | Spectrum Coefficient   |
| $S_{ae}(T)$        | = | Elasticity spectrum ordinate [ $m/s^2$ ]   |
| $S_{aR}(T_r)$      | = | Acceleration spectrum ordinate for the r'th natural vibration mode [ $m/s^2$ ]   |
| $T$                | = | Building natural vibration period [s]  |
| $T_1$              | = | First natural vibration period of building [s]   |
| $T_A, T_B$         | = | Spectrum Characteristic Periods [s]  |
| $T_m, T_n$         | = | m'th and n'th natural vibration periods of building [s]  |
| $V_i$              | = | Storey shear at i'th storey of building in the earthquake direction considered   |
| $V_t$              | = | In the Equivalent Seismic Load Method, total equivalent seismic load acting on the building (base shear) in the earthquake direction considered  |
| $V_{tB}$           | = | In the Mode-Superposition Method, total design seismic load acting on the building (base shear) obtained by modal combination in the earthquake direction considered   |
| $W$                | = | Total weight of building calculated by considering Live Load Participation Factor  |
| $w_e$              | = | Weight of mechanical or electrical equipment   |
| $w_i$              | = | Weight of i'th storey of building by considering Live Load Participation Factor  |
| $Y$                | = | Sufficient number of natural vibration modes taken into account in the Mode-Superposition Method   |
| $\alpha$           | = | Coefficient used for determining the gap size of a seismic joint   |
| $\alpha_s$         | = | Ratio of sum of shear forces developed at the bases of structural walls of high ductility level to total shear force developed at the bases for the entire building  |
| $\beta$            | = | Coefficient used to determine lower limits of response quantities calculated by Mode-Superposition Method  |
| $\Delta_i$         | = | Storey drift of i'th storey of building  |
| $(\Delta_i)_{ave}$ | = | Average storey drift of i'th storey of building  |
| $\Delta F_N$       | = | Additional equivalent seismic load acting on the N'th storey (top) of building   |
| $\delta_i$         | = | Effective storey drift of i'th storey of building  |
| $(\delta_i)_{mak}$ | = | Maximum effective storey drift of i'th storey of building  |
| $\eta_{bi}$        | = | Torsionally Irregularity Factor defined at i'th storey of building   |
| $\eta_{ci}$        | = | Strength Irregularity Factor defined at i'th storey of building  |
| $\eta_{ki}$        | = | Stiffness Irregularity Factor defined at i'th storey of building   |
| $\Phi_{xin}$       | = | In buildings with floors modeled as rigid diaphragms, horizontal component of n'th mode shape in the x direction at i'th storey of building  |
| $\Phi_{yin}$       | = | In buildings with floors modeled as rigid diaphragms, horizontal component of n'th mode shape in the y direction at i'th storey of building  |
| $\Phi_{\theta in}$ | = | In buildings with floors modeled as rigid diaphragms, rotational component of n'th mode shape around the vertical axis at i'th storey of building  |
| $\theta_i$         | = | Second Order Effect Indicator defined at i'th storey of building   |

## 2.1. SCOPE

**2.1.1** - Seismic loads and analysis requirements to be applied to the earthquake resistant design of all cast-in-site and prefabricated reinforced concrete buildings, structural steel buildings and building-like structures to be built in seismic zones defined in **1.2.3** are specified in this chapter. Rules for masonry buildings are specified in **Chapter 5**, respectively.

**2.1.2** - Rules for the analysis of building foundations and soil retaining structures are specified in **Chapter 6**.

**2.1.3** - Non-building structures which are permitted to be analyzed in accordance with the requirements of this chapter shall be limited to those given in **Chapter.12**.

**2.1.4** – Design loads to be applied for strengthening and evaluating earthquake performances of existing buildings are specified in **Chapter 7**.

## 2.2. GENERAL GUIDELINES AND RULES

### 2.2.1. General Guidelines for Building Structural Systems

**2.2.1.1** – The building structural system resisting seismic loads as a whole as well as each structural element of the system shall be provided with sufficient *stiffness*, *stability* and *strength* to ensure an uninterrupted and safe transfer of seismic loads down to the foundation soil.

**2.2.1.2** - It is essential that floor systems possess sufficient stiffness and strength to ensure the safe transfer of lateral seismic loads between the elements of the structural system. In insufficient cases, appropriate transfer elements shall be arranged on floors.

**2.2.1.3** - In order to dissipate a significant part of the seismic energy fed into the building by ductile behavior of structural system, *ductile design* principles specified in **Chapter 3** and in **Chapter 4** of this Specification should be followed.

**2.2.1.4** - Design and construction of irregular buildings defined in **2.3.1** below should be avoided. Structural system should be arranged symmetrical or nearly symmetrical in plan and torsional irregularity defined as type **A1** irregularity in **Table 2.1** should preferably be avoided. In this respect, it is essential that stiff structural elements such as structural walls should be placed so as to increase the torsional stiffness of the building. On the other hand, vertical irregularities defined as types **B1** and **B2** in **Table 2.1** leading to *weak storey* or *soft storey* at any storey should be avoided.

**2.2.1.5** - Effects of rotations of column and in particular wall supporting foundations on soils classified as group (C) and (D) in **Table 6.1** of **Chapter 6** should be taken into account by appropriate methods of structural modeling.

### 2.2.2. General Rules for Seismic Loads

**2.2.2.1** - Unless specified otherwise in this chapter, in order to determine seismic loads acting on buildings, *Spectral Acceleration Coefficient* specified in **2.4** and *Seismic Load Reduction Factor* specified in **2.5** shall be based on.

**2.2.2.2** - Unless specified otherwise in this Specification, seismic loads shall be assumed to act non-simultaneously along the two perpendicular axes of the building in the horizontal plane. Provisions concerning combined effect of earthquakes in considered axes are given in **2.7.5**.

**2.2.2.3** - Unless specified otherwise in this Specification, load factors to be used to determine design internal forces under the combined effects of seismic loads and other loads according to *ultimate strength theory* shall be taken from the relevant structural specifications.

**2.2.2.4** - It shall be assumed that the wind loads and seismic loads act non-simultaneously, and the most unfavorable response quantity due to wind or earthquake shall be considered for the design of each structural element. However, even if the quantities due to wind govern are more unfavorable, rules given in this Specification shall be applied for dimensioning and detailing of structural elements and their joints.

## **2.3. IRREGULAR BUILDINGS**

### **2.3.1. Definition of Irregular Buildings**

Regarding the definition of irregular buildings whose design and construction should be avoided because of their unfavorable seismic behavior, types of irregularities in plan and in elevation are given in **Table 2.1** and relevant conditions are given in **2.3.2** below.

### **2.3.2. Conditions for Irregular Buildings**

Conditions related to irregularities defined in **Table 2.1** are given below:

**2.3.2.1** - Irregularity types **A1** and **B2** are irregularities that govern the selection of the method of seismic analysis as specified in **2.6** below.

**2.3.2.2** - In buildings with irregularity types **A2** and **A3**, it shall be verified by calculation in the first and second seismic zones that the floor systems are capable of safe transfer of seismic loads between vertical structural elements.

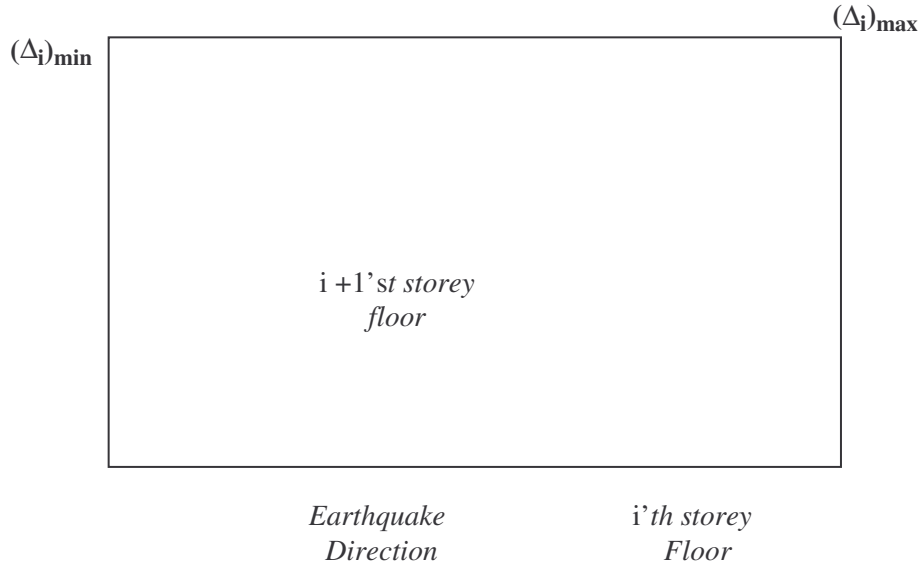
**2.3.2.3** - In buildings with irregularity type **B1**, if total infill wall area at  $i$ 'th storey is greater than that of the storey immediately above, then infill walls shall not be taken into account in the determination of  $\eta_{ci}$ . In the range  $0.60 \leq (\eta_{ci})_{\min} < 0.80$ , structural behavior factor given in **Table 2.5** shall be multiplied by **1.25**  $(\eta_{ci})_{\min}$  and applied to the entire building in both earthquake directions. In no case, however,  $\eta_{ci} < 0.60$  shall be permitted. Otherwise strength and stiffness of the weak storey shall be increased and the seismic analysis shall be repeated.

**2.3.2.4** – In order to be applied in all seismic regions, conditions related to buildings with irregularity type **B3** are specified below:

(a) In all seismic zones, columns at any storey of the building shall in no case be permitted to rest on the cantilever beams or on top of or at the tip of gussets provided in the columns underneath.

**TABLE 2.1 - IRREGULAR BUILDINGS**

| <b>A – IRREGULARITIES IN PLAN</b>   | <b>Related Items</b> |
|---|----------------------|
| <p><b><u>A1 – Torsional Irregularity :</u></b><br/> The case where <i>Torsional Irregularity Factor</i> <math>\eta_{bi}</math>, which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum relative storey drift at any storey to the average relative storey drift at the same storey in the same direction, is greater than 1.2 (<b>Figure 2.1</b>). <math>[\eta_{bi} = (\Delta_i)_{\max} / (\Delta_i)_{\text{avr}} &gt; 1.2]</math><br/> <i>Relative storey drifts shall be calculated in accordance with 2.7, by considering the effects of <math>\pm</math> %5 additional eccentricities.</i></p>  | <b>2.3.2.1</b>       |
| <p><b><u>A2 – Floor Discontinuities :</u></b><br/> In any floor (<b>Figure 2.2</b>);<br/> <b>I</b> - The case where the total area of the openings including those of stairs and elevator shafts exceeds 1 / 3 of the gross floor area,<br/> <b>II</b> – The case where local floor openings which make the safe transfer of seismic loads difficult to vertical structural elements,<br/> <b>III</b> – The cases of abrupt reductions in the in-plane stiffness and strength of floors.</p>  | <b>2.3.2.2</b>       |
| <p><b><u>A3 – Projections in Plan :</u></b><br/> The cases where dimensions of projections in both of the two perpendicular directions in plan exceed the total plan dimensions of that storey of the building in the respective directions by more than 20% (<b>Figure 2.3</b>).</p>   | <b>2.3.2.2</b>       |
| <b>B – IRREGULARITIES IN ELEVATION</b>  | <b>Related Items</b> |
| <p><b><u>B1 – Interstorey Strength Irregularity (Weak Storey) :</u></b><br/> In reinforced concrete buildings, the case where in each of the orthogonal earthquake directions, <i>Strength Irregularity Factor</i> <math>\eta_{ci}</math> which is defined as the ratio of the <i>effective shear area</i> of any storey to the <i>effective shear area</i> of the storey immediately above, is less than <b>0.80</b>. <math>[\eta_{ci} = (\sum A_e)_i / (\sum A_e)_{i+1} &lt; 0.80]</math><br/> <i>Definition of effective shear area in any storey :</i><br/> <math>\sum A_e = \sum A_w + \sum A_g + 0.15 \sum A_k</math> (See 3.0 for notations)</p>   | <b>2.3.2.3</b>       |
| <p><b><u>B2 – Interstorey Stiffness Irregularity (Soft Storey) :</u></b><br/> The case where in each of the two orthogonal earthquake directions, <i>Stiffness Irregularity Factor</i> <math>\eta_{ki}</math>, which is defined as the ratio of the average relative storey drift at any i'th storey to the average relative storey drift at the storey immediately above or below, is greater than 2.0. <math>[\eta_{ki} = (\Delta_i / h_i)_{\text{avr}} / (\Delta_{i+1} / h_{i+1})_{\text{avr}} &gt; 2.0</math> or <math>\eta_{ki} = (\Delta_i / h_i)_{\text{avr}} / (\Delta_{i-1} / h_{i-1})_{\text{avr}} &gt; 2.0]</math><br/> <i>Relative storey drifts shall be calculated in accordance with 2.7, by considering the effects of <math>\pm</math> %5 additional eccentricities.</i></p> | <b>2.3.2.1</b>       |
| <p><b><u>B3 - Discontinuity of Vertical Structural Elements :</u></b><br/> The cases where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath (<b>Figure 2.4</b>).</p>   | <b>2.3.2.4</b>       |



*In the case where floors behave as rigid diaphragms*

*In their own planes:*

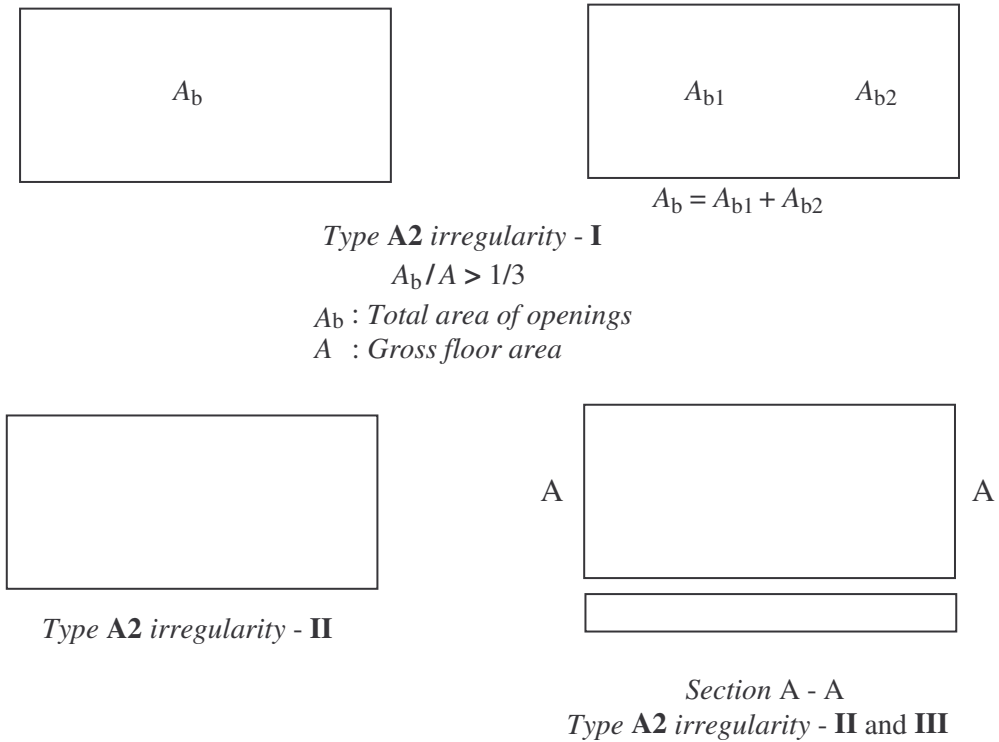
$$(\Delta_i)_{\text{avr}} = 1/2 [(\Delta_i)_{\max} + (\Delta_i)_{\min}]$$

*Torsional irregularity factor :*

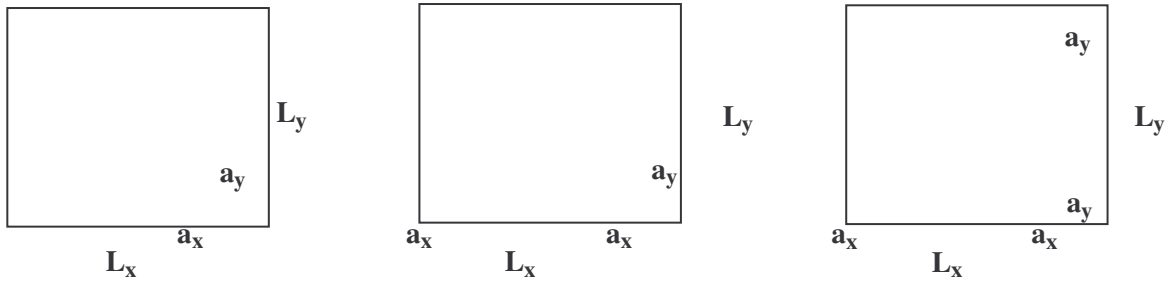
$$\eta_{bi} = (\Delta_i)_{\max} / (\Delta_i)_{\text{ort}}$$

*Torsional irregularity:  $\eta_{bi} > 1.2$*

**Figure 2.1**

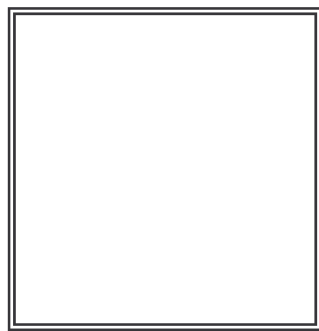


**Figure 2.2**

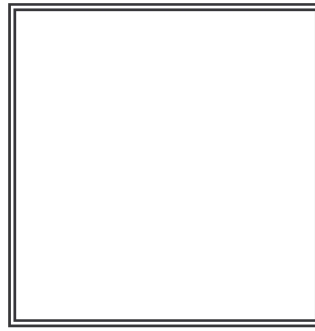


*Type A3 irregularity:*  
 $a_x > 0.2 L_x$  and at the same time  $a_y > 0.2 L_y$

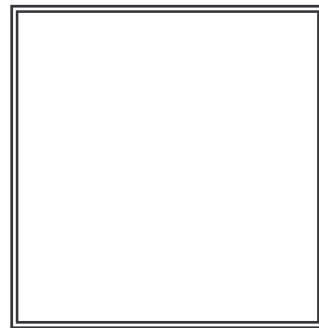
**Figure 2.3**



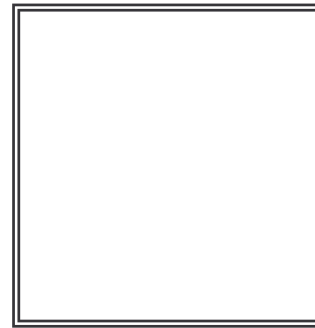
*See 2.3.2.4 (a)*



*See 2.3.2.4 (b)*



*See 2.3.2.4 (c)*



*See 2.3.2.4 (d)*

**Figure 2.4**

**(b)** In the case where a column rests on a beam which is supported at both ends, all internal force components induced by the combined effect of vertical loads and seismic loads shall be increased by 50 % at all sections of the beam and at all sections of the other beams and columns adjoining to the beam in the earthquake direction considered.

**(c)** Structural walls at upper stories shall in no case be permitted to rest on the columns below.

**(d)** Structural walls shall in no case be permitted in their own plane to rest on the beam span at any storey of the building.

## 2.4. DEFINITION OF ELASTIC SEISMIC LOADS: SPECTRAL ACCELERATION COEFFICIENT

The *Spectral Acceleration Coefficient*,  $A(T)$ , which shall be considered as the basis for the determination of seismic loads is given by **Equation (2.1)**. *Elastic Spectral Acceleration*,  $S_{ae}(T)$  which is the ordinate of *Elastic Acceleration Spectrum* defined for 5 % damped rate is derived by multiplying Spectral Acceleration Coefficient with gravity,  $g$ .

$$A(T) = A_o I S(T) \quad (2.1)$$

$$S_{ae}(T) = A(T) g$$

### 2.4.1. Effective Ground Acceleration Coefficient

The *Effective Ground Acceleration Coefficient*,  $A_o$ , appearing in **Equation (2.1)** is specified in **Table 2.2**.

**TABLE 2.2 - EFFECTIVE GROUND ACCELERATION COEFFICIENT ( $A_o$ )**

| <i>Seismic Zone</i> | $A_o$ |
|---------------------|-------|
| 1                   | 0.40  |
| 2                   | 0.30  |
| 3                   | 0.20  |
| 4                   | 0.10  |

### 2.4.2. Building Importance Factor

The *Building Importance Factor*,  $I$ , appearing in **Equation (2.1)** is specified in **Table 2.3**.

**TABLE 2.3 - BUILDING IMPORTANCE FACTOR ( $I$ )**

| <i>Purpose of Occupancy or Type of Building</i>   | <i>Importance Factor (<math>I</math>)</i> |
|---|---|
| <b><u>1. Buildings required to be utilized after the earthquake and buildings containing hazardous materials</u></b><br>a) Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations)<br>b) Buildings containing or storing toxic, explosive and flammable materials, etc. | 1.5                                       |
| <b><u>2. Intensively and long-term occupied buildings and buildings preserving valuable goods</u></b><br>a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc.<br>b) Museums  | 1.4                                       |
| <b><u>3. Intensively but short-term occupied buildings</u></b><br>Sport facilities, cinema, theatre and concert halls, etc.   | 1.2                                       |
| <b><u>4. Other buildings</u></b><br>Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)   | 1.0                                       |

### 2.4.3. Spectrum Coefficient

**2.4.3.1** - The *Spectrum Coefficient*,  $S(T)$ , appearing in **Equation (2.1)** shall be determined by **Equation (2.2)**, depending on the local site conditions and the building natural period,  $T$  (**Figure 2.5**):

$$\begin{aligned} S(T) &= 1 + 1.5 \frac{T}{T_A} & (0 \leq T \leq T_A) \\ S(T) &= 2.5 & (T_A < T \leq T_B) \\ S(T) &= 2.5 \left( \frac{T_B}{T} \right)^{0.8} & (T_B < T) \end{aligned} \quad (2.2)$$

*Spectrum Characteristic Periods*,  $T_A$  and  $T_B$ , appearing in **Equation (2.2)** are specified in **Table 2.4**, depending on *Local Site Classes* defined in **Table 6.2** of **Chapter 6**.

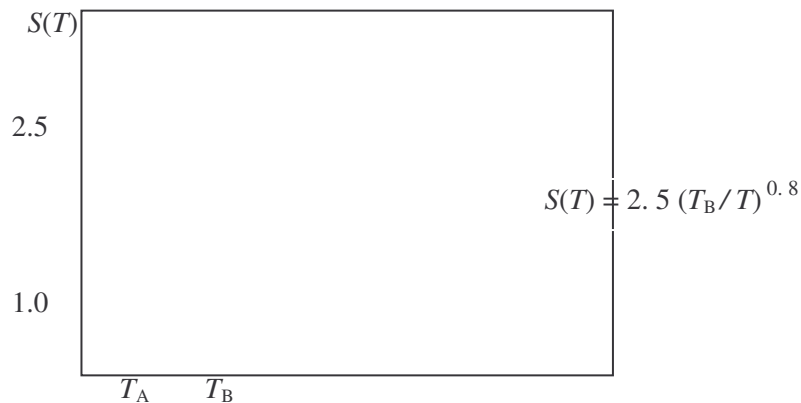
**TABLE 2.4 - SPECTRUM CHARACTERISTIC PERIODS ( $T_A$ ,  $T_B$ )**

| <i>Local Site Class<br/>according to Table 6.2</i> | $T_A$<br>(second) | $T_B$<br>(second) |
|--|-------------------|-------------------|
| Z1   | 0.10              | 0.30              |
| Z2   | 0.15              | 0.40              |
| Z3   | 0.15              | 0.60              |
| Z4   | 0.20              | 0.90              |

**2.4.3.2** - In case where the requirements specified in **6.2.1.2** and **6.2.1.3** of **Chapter 6** are not met, spectrum characteristic periods defined in **Table 2.4** for local site class Z4 shall be used.

### 2.4.4. Special Design Acceleration Spectra

In required cases, elastic acceleration spectrum may be determined through special investigations by considering local seismic and site conditions. However spectral acceleration coefficients corresponding to so obtained acceleration spectrum ordinates shall in no case be less than those determined by **Equation (2.1)** based on relevant characteristic periods specified in **Table 2.4**.



**Figure 2.5**

## 2.5. REDUCTION OF ELASTIC SEISMIC LOADS: SEISMIC LOAD REDUCTION FACTOR

In order to consider the specific nonlinear behavior of the structural system during earthquake, elastic seismic loads to be determined in terms of spectral acceleration coefficient defined in 2.4 shall be divided to below defined *Seismic Load Reduction Factor* to account for. *Seismic Load Reduction Factor*, shall be determined by **Equation (2.3)** in terms of *Structural System Behavior Factor*,  $R$ , defined in **Table 2.5** for various structural systems, and the natural vibration period  $T$ .

$$\begin{aligned} R_a(T) &= 1.5 + (R - 1.5) \frac{T}{T_A} & (0 \leq T \leq T_A) \\ R_a(T) &= R & (T_A < T) \end{aligned} \quad (2.3)$$

### 2.5.1. General Conditions on Ductility Levels of Structural Systems

**2.5.1.1** - Definitions of and requirements to be fulfilled for *structural systems of high ductility level* and *structural systems of nominal ductility level* whose Structural System Behavior Factors are given in **Table 2.5**, are given in **Chapter 3** for reinforced concrete buildings and in **Chapter 4** for structural steel buildings.

**2.5.1.2** - In structural systems denoted as being *high ductility level* in **Table 2.5**, ductility levels shall be high in both lateral earthquake directions. Systems which have high or combined ductility level in one earthquake direction and nominal ductility level in the perpendicular earthquake direction shall be deemed to be *structural systems of nominal ductility level* in both directions.

**2.5.1.3** – Systems which have the same ductility level in both directions or high in one direction and mixed in other direction, different  $R$  coefficients can be used in different directions.

**2.5.1.4** – Bare or infilled joist and waffle slab systems whose columns and beams do not satisfy the requirements given in 3.3, 3.4 and 3.5, and reinforced concrete flat slab systems without structural walls shall be treated as *systems of nominal ductility level*.

**2.5.1.5** – In the first and second seismic zones;

(a) Except paragraph (b) below, *structural systems of high ductility level* shall be used for the reinforced concrete buildings with structural systems comprised of frames only.

(b) In reinforced concrete buildings with Building Importance Factor  $I = 1.2$  and  $I = 1.0$  according to the **Table 2.3**, structural systems comprised of frames of nominal ductility level can only be built on the condition that  $H_N \leq 16$  m.

(c) In all buildings with Building Importance Factor of  $I = 1.5$  and  $I = 1.4$  according to **Table 2.3**, *structural systems of high ductility level* or *structural systems with mixed ductility level* defined in 2.5.4.1 shall be used.

**2.5.1.6** – *Structural systems of nominal ductility level* without structural walls can only be permitted to be built in the third and fourth seismic zones with the following conditions:

(a) Reinforced concrete buildings defined in 2.5.1.4, can be built on the condition that  $H_N \leq 13$  m.

(b) –Excluding the systems indicated in 2.5.1.4, reinforced concrete and structural steel buildings comprised only of frames of nominal ductility level are permitted to be built on the condition that  $H_N \leq 25$  m.

**TABLE 2.5 - STRUCTURAL SYSTEM BEHAVIOUR FACTORS (R)**

| <b>BUILDING STRUCTURAL SYSTEM</b>  | <b>Systems of Nominal Ductility Level</b> | <b>Systems of High Ductility Level</b> |
|--|---|--|
| <b><u>(1) CAST-IN-SITE REINFORCED CONCRETE BUILDINGS</u></b>   |   |  |
| (1.1) Buildings in which seismic loads are fully resisted by frames.....   | 4   | 8                                      |
| (1.2) Buildings in which seismic loads are fully resisted by coupled structural walls.....   | 4   | 7                                      |
| (1.3) Buildings in which seismic loads are fully resisted by solid structural walls.....   | 4   | 6                                      |
| (1.4) Buildings in which seismic loads are jointly resisted by frames and solid and / or coupled structural walls.....   | 4   | 7                                      |
| <b><u>(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS</u></b>  |   |  |
| (2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer   | 3   | 7                                      |
| (2.2) Single-storey buildings in which seismic loads are fully resisted by columns with hinged upper connections   | —   | 3                                      |
| (2.3) Prefabricated buildings with hinged frame connections in which seismic loads are fully resisted by prefabricated or cast – in – situ solid structural walls and / or coupled structural walls. | —   | 5                                      |
| (2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and / or coupled structural walls                    | 3   | 6                                      |
| <b><u>(3) STRUCTURAL STEEL BUILDINGS</u></b>   |   |  |
| (3.1) Buildings in which seismic loads are fully resisted by frames.....   | 5   | 8                                      |
| (3.2) Single – storey buildings in which seismic loads are fully resisted by columns with connections hinged at the top.....   | —   | 4                                      |
| (3.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls  |   |  |
| (a) Centrally braced frames .....  | 4   | 5                                      |
| (b) Eccentrically braced frames .....  | —   | 7                                      |
| (c) Reinforced concrete structural walls.....  | 4   | 6                                      |
| (3.4) Buildings in which seismic loads are jointly resisted by structural steel braced frames or cast-in-situ reinforced concrete structural walls   |   |  |
| (a) Centrally braced frames..... (b)   | 5   | 6                                      |
| Eccentrically braced frames..... (c)   | —   | 8                                      |
| Reinforced concrete structural walls.....  | 4   | 7                                      |

### 2.5.2. Conditions for Reinforced Concrete Solid Structural Wall - Frame Systems of High Ductility Level

Requirements for buildings where seismic loads are jointly resisted by reinforced concrete solid structural walls of *high ductility level* and reinforced concrete or structural steel frames of *high ductility level* are given below:

**2.5.2.1** – In order to use  $R = 7$  for cast – in – situ reinforced concrete and structural steel frame systems or  $R = 6$  for prefabricated reinforced concrete frame systems as it is given in **Table 2.5**, sum of shear forces developed at the bases of solid structural walls under seismic loads shall not exceed 75% of the total shear force developed at the bases for the entire building ( $\alpha_s \leq 0.75$ ).

**2.5.2.2** – In the case where the requirement **2.5.2.1** cannot be satisfied, coefficient  $R$  to be used in the range  $0.75 < \alpha_s \leq 1.0$  shall be calculated by the expression  $R = 10 - 4 \alpha_s$  for cast – in – situ reinforced concrete and structural steel frame systems and by  $R = 9 - 4 \alpha_s$  for prefabricated reinforced concrete frame systems.

**2.5.2.3** – In structural walls in which  $H_w / \ell_w \leq 2.0$ , internal forces calculated according to  $R$  coefficients defined above shall be increased by being multiplied with  $[3 / (1 + H_w / \ell_w)]$ . However, this coefficient is not taken more than 2.

### 2.5.3. Conditions on Mandatory Use of Structural Walls in Certain Systems of Nominal Ductility Level

Systems of nominal ductility level defined in paragraphs (a) and (b) of **2.5.1.6** can be built in all seismic regions and over the height limit defined in the same paragraphs. But in this case, it is mandatory to use solid or coupled reinforced concrete structural walls of nominal or high ductility level in reinforced concrete buildings through the full height of the building with the following conditions fulfilled and centrally or eccentrically braced frames of nominal or high ductility level in structural steel buildings.

**2.5.3.1** – When structural walls of *nominal ductility level* are used in the structural system, sum of shear forces developed at the bases of structural walls according to seismic loads shall be more than 75% of the total shear force developed for the entire building in each earthquake direction.

**2.5.3.2** When structural walls of *high ductility level* are used in the structural system, requirements specified in **2.5.4.1** below for mixed structural systems shall be applied.

### 2.5.4. Conditions for Mixed Structural Systems in Terms of Ductility Level

**2.5.4.1** – Systems of nominal ductility level defined in paragraphs (a) and (b) of **2.5.1.6** are permitted to be mixed with structural walls of *high ductility level*. In so obtained *systems of mixed ductility level*, solid or coupled reinforced concrete structural walls of high ductility or for steel buildings structural steel eccentric or centric braced frames may be used provided that the following conditions are met.

(a) In the seismic analysis of such mixed systems, frames and structural walls (or braced frames) shall be jointly considered, however in each earthquake direction it shall be  $\alpha_S \geq 0.40$ .

(b) In the case where  $\alpha_S \geq 2/3$  in both earthquake directions,  $R$  factor defined in **Table 2.5** for the case where seismic loads are fully resisted by structural of *high ductility level* ( $R = R_{YP}$ ), may be used for the entire structural system.

(c) In the range  $0.40 < \alpha_S < 2/3$ , the expression  $R = R_{NC} + 1.5 \alpha_S (R_{YP} - R_{NC})$  shall be applied to the entire structural system in both earthquake directions.

**2.5.4.2** – Reinforced concrete rigid peripheral walls used in basements of buildings shall not be taken into consideration as parts of structural wall systems or structural wall-frame systems appearing in **Table 2.5**. Rules to be applied in analysis of such buildings are given in **2.7.2.4** and **2.8.3.2**.

### **2.5.5. Conditions for Systems with Hinged Connections**

**2.5.5.1** – In reinforced concrete buildings made of single – storey frames with columns hinged at the top;

(a) In case cast – in – situ reinforced concrete columns are used,  $R$  factor defined in item (2.2) of **Table 2.5** shall be used for prefabricated buildings.

(b) Conditions concerning reinforced concrete prefabricated and structural steel buildings, whose  $R$  factors are given in items (2.2) and (3.2) of **Table 2.5**, are given in **2.5.5.2**. Conditions relating to usage of such frames as top floor (roof top) in cast – in – situ reinforced concrete, prefabricated and structural steel buildings are defined in **2.5.5.3**.

**2.5.5.2** – A single mezzanine floor can be constructed inside such buildings provided that it is not larger than 25 % of place area of the building. Structural system of mezzanine floor shall be taken into account in the seismic analysis of such buildings together with the main structural frames. In this case, combined system shall be arranged as the system of high ductility level in prefabricated reinforced concrete buildings. In combined system, existence of torsional irregularity shall be controlled and if it exists, it shall be taken into account in analysis. Connections of mezzanine floor with main structural frames can be hinged or monolithic.

**2.5.5.3** – In case single storey frames with columns hinged at the top are used as top floor (roof top) in cast – in – situ reinforced concrete, prefabricated and structural steel buildings, for the top floor,  $R$  factor ( $R_{up}$ ) defined in items (2.2) or (3.2) of **Table 2.5** and for the lower floors,  $R$  factor ( $R_{down}$ ) which may be defined differently shall be used with together provided that following conditions are met.

(a) At the beginning, seismic analysis shall be made for entire building according to **2.7** or **2.8** by taking  $R = R_{down}$ . Reduced and effective relative storey drifts defined in **2.10.1** shall be obtained for entire building from this analysis.

(b) Internal forces of the top floor shall be obtained by multiplying internal forces calculated in (a) with ( $R_{down} / R_{up}$ ) ratio.

(c) On the other hand, internal forces of lower floors shall be composed of sum of two sides. First side is the internal forces calculated in (a). Second side shall be calculated by acting structural system of lower floors by multiplying the forces calculated in (b) as support reactions of top floor columns with  $(1 - R_{up} / R_{down})$ .

## 2.6. SELECTION OF ANALYSIS METHOD

### 2.6.1. Analysis Methods

Methods to be used for the seismic analysis of buildings and building-like structures are, *Equivalent Seismic Load Method* given in 2.7, *Mode – Superposition Method* given in 2.8 and *Analysis Methods in the Time Domain* given in 2.9. Methods given in 2.8 and 2.9 may be used for the seismic analysis of all buildings and building-like structures.

### 2.6.2. Application Limits of Equivalent Seismic Load Method

Buildings for which *Equivalent Seismic Load Method* given in 2.7 is applicable are summarized in **Table 2.6**. Methods given in 2.8 or 2.9 shall be used for the seismic analysis of buildings outside the scope of **Table 2.6**.

**TABLE 2.6 - BUILDINGS FOR WHICH EQUIVALENT SEISMIC LOAD METHOD IS APPLICABLE**

| <i>Seismic Zone</i> | <i>Type of Building</i>   | <i>Total Height Limit</i> |
|---------------------|---|---------------------------|
| 1, 2                | Buildings in which torsional irregularity coefficient satisfies the condition $\eta_{bi} \leq 2.0$ at every storey  | $H_N \leq 25$ m           |
| 1, 2                | Buildings in which torsional irregularity coefficient satisfies the condition $\eta_{bi} \leq 2.0$ at every storey and at the same time without type <b>B2</b> irregularity | $H_N \leq 40$ m           |
| 3, 4                | All buildings   | $H_N \leq 40$ m           |

## 2.7. EQUIVALENT SEISMIC LOAD METHOD

### 2.7.1. Determination of Total Equivalent Seismic Load

**2.7.1.1** – *Total Equivalent Seismic Load* (base shear),  $V_t$ , acting on the entire building in the earthquake direction considered shall be determined by **Equation (2.4)**.

$$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10 A_0 I W \quad (2.4)$$

The first natural vibration period of the building,  $T_1$ , shall be calculated in accordance with 2.7.4.

**2.7.1.2** – Total building weight,  $W$ , to be used in **Equation (2.4)** as the seismic weight shall be determined by **Equation (2.5)**.

$$W = \sum_{i=1}^N w_i \quad (2.5)$$

Storey weights  $w_i$  of **Equation (2.5)** shall be calculated by **Equation (2.6)**.

$$w_i = g_i + n q_i \quad (2.6)$$

*Live Load Participation Factor*,  $n$ , appearing in **Equation (2.6)** is given in **Table 2.7**. In industrial buildings,  $n = 1$  shall be taken for fixed equipment weights while crane payloads shall not be taken into account in the calculation of storey weights 30 % of snow loads shall be considered in calculation of weight of roof top to be used in determination of seismic loads.

**TABLE 2.7 - LIVE LOAD PARTICIPATION FACTOR ( $n$ )**

| <i>Purpose of Occupancy of Building</i>  | <i>n</i> |
|--|----------|
| Depot, warehouse, etc.   | 0.80     |
| School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc. | 0.60     |
| Residence, office, hotel, hospital, etc.   | 0.30     |

**2.7.2. Determination of Design Seismic Loads Acting at Storey Levels**

**2.7.2.1** – Total equivalent seismic load determined by **Equation (2.4)** is expressed by **Equation (2.7)** as the sum of equivalent seismic loads acting at storey levels (**Fig. 2.6a**):

$$V_t = \Delta F_N + \sum_{i=1}^N F_i \quad (2.7)$$

**2.7.2.2** – *Additional equivalent seismic load,  $\Delta F_N$* , acting at the  $N$ 'th storey (top) of the building shall be determined by **Equation (2.8)**.

$$\Delta F_N = 0.0075 N V_t \quad (2.8)$$

**2.7.2.3** – Excluding  $\Delta F_N$ , remaining part of the total equivalent seismic load shall be distributed to stories of the building (including  $N$ 'th storey) in accordance with **Equation (2.9)**.

$$F_i = (V_t - \Delta F_N) \frac{w_i H_i}{\sum_{j=1}^N w_j H_j} \quad (2.9)$$

**2.7.2.4** – In buildings with reinforced concrete peripheral walls at their basements being very rigid relative to upper stories and basement floors behaving as rigid diaphragms in horizontal planes, equivalent seismic loads acting on the basement stories and on the upper stories shall be calculated independently as in the following. These loads shall be applied together with structural system composed of combination of upper and lower stories.

(a) In determining the total equivalent seismic load and equivalent storey seismic loads in accordance with **2.7.1.1**, **2.7.2.2** and **2.7.2.3**, appropriate  $R$  factor shall be selected from **Table 2.5** without considering the rigid peripheral basement walls and seismic weights of the upper stories only shall be taken into account. In this case, foundation top level appearing in the relevant definitions and expressions shall be replaced by the ground floor level. Fictitious loads used for the calculation of the first natural vibration period in accordance with **2.7.4.1** shall also be based on seismic weights of the upper stories only (**Fig. 2.6b**).

(b) In calculating equivalent seismic loads acting on rigid basement stories, seismic weights of basements only shall be taken into account and *Spectrum Coefficient* shall be taken as  $S(T) = 1$ . In determining equivalent seismic loads acting on each basement storey, spectral acceleration obtained from **Equation (2.1)** shall be multiplied directly with the respective weight of the storey and resulting elastic loads shall be reduced by dividing them to  $R_a(T) = 1.5$  (**Fig. 2.6c**).

(c) Strength of flooring system of ground storey, in its self – plane, surrounded by multi rigid walls and located at the passage from upper stories to sub – basement shall be controlled according to internal forces obtained in this analysis.

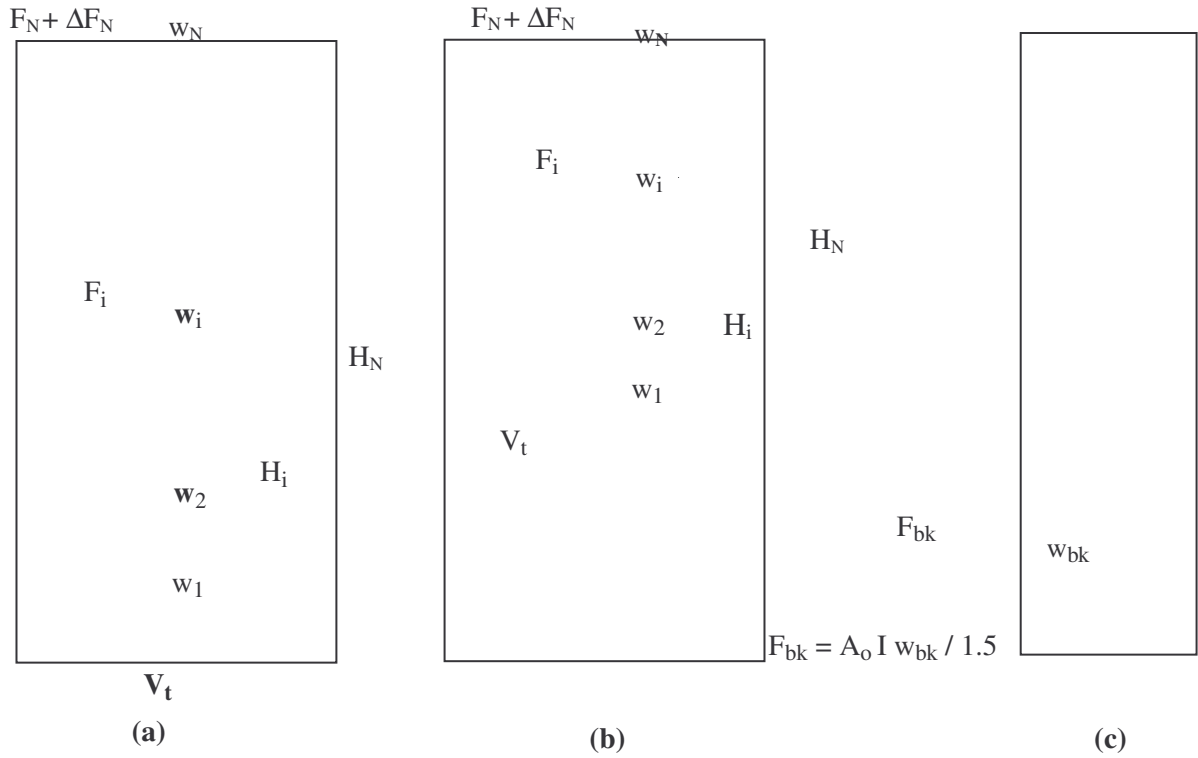


Figure 2.6

### 2.7.3. Displacement Components to be Considered and Application Points of Seismic Loads

**2.7.3.1** – In buildings where floors behave as rigid horizontal diaphragms, two lateral displacement components and the rotation around the vertical axis shall be taken into account at each floor as independent static displacement components. In order to consider eccentricity effect, at each floor, equivalent seismic loads determined in accordance with 2.7.2 shall be applied on the points obtained by shifting the actual mass centre by + 5% and – 5% times the floor length in the perpendicular direction to the earthquake direction considered as well as to storey mass center (**Fig. 2.7**).

**2.7.3.2** - In buildings where type **A2** irregularity (defined in **Table 2.1**) exists and floors do not behave as rigid horizontal diaphragms, sufficient number of independent static displacement components shall be considered to account for the in-plane deformation of floors. In order to consider additional eccentricity effects, each of the individual masses distributed over each floor shall be shifted by + 5% and – 5% times the floor length in perpendicular direction to the earthquake direction considered (**Fig. 2.8**).

**2.7.3.3** - In the case where type **A1** irregularity defined in **Table 2.1** exists at any  $i$ 'th storey such that the condition  $1.2 < \eta_{bi} \leq 2.0$  is satisfied,  $\pm 5\%$  additional eccentricity applied to this floor according to 2.7.3.1 and / or 2.7.3.2 shall be amplified by multiplying with coefficient  $D_i$  given by **Equation (2.10)** for both earthquake directions.

$$D_i = \left( \frac{\eta_{bi}}{1.2} \right)^2 \quad (2.10)$$

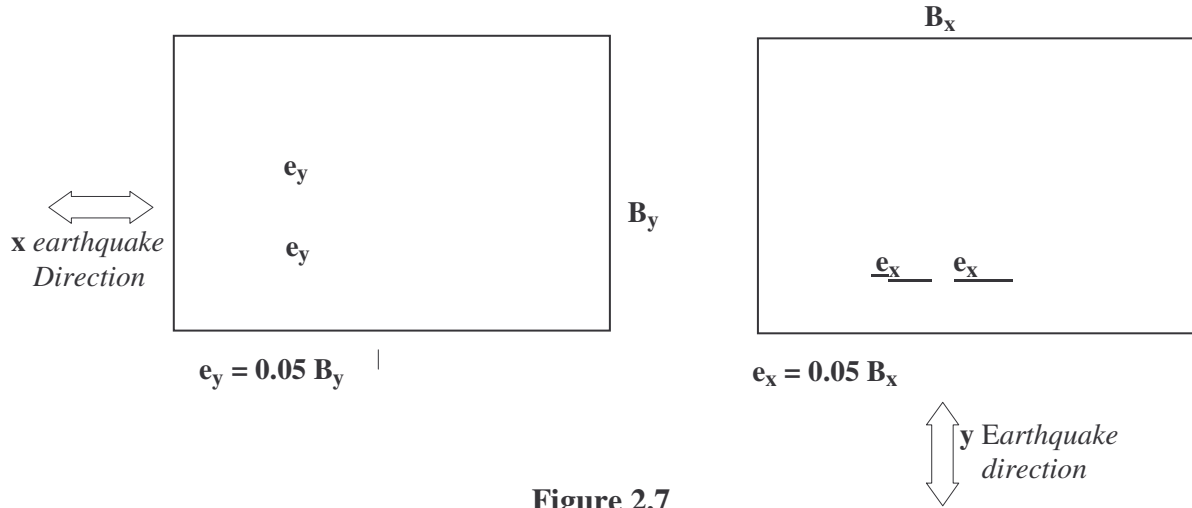


Figure 2.7

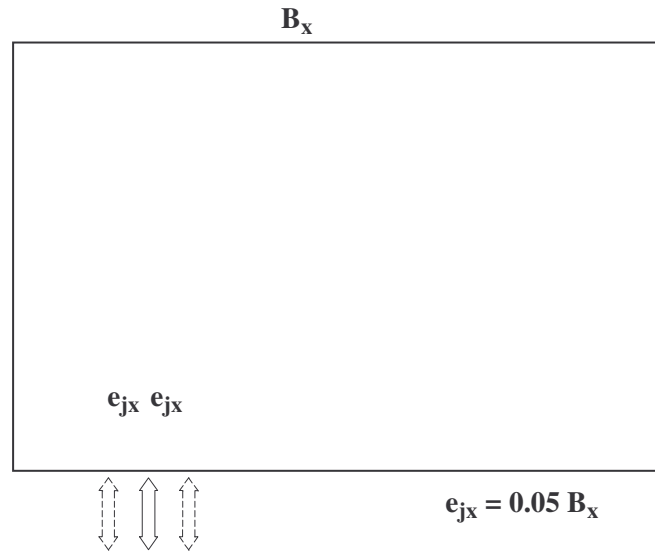


Figure 2.8

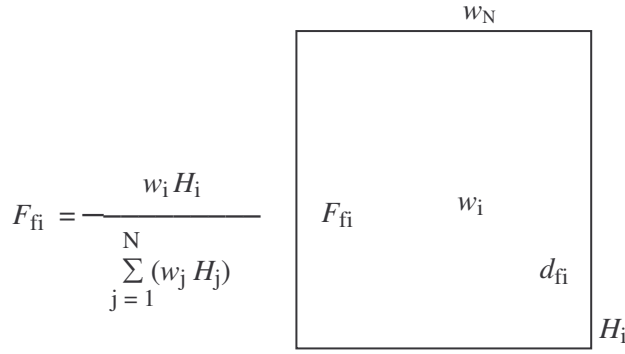
#### 2.7.4. Determination of First Natural Vibration Period of Building

**2.7.4.1** – In the case Equivalent Seismic Load Method is applied, the first natural vibration period of building in the direction of earthquake is not taken more than the value calculated by **Equation (2.11)**.

$$T_1 = 2\pi \left( \frac{\sum_{i=1}^N m_i d_{fi}^2}{\sum_{i=1}^N F_{fi} d_{fi}} \right)^{1/2} \quad (2.11)$$

$F_{fi}$  referring to the fictitious load acting on the  $i$ 'th storey shall be obtained from **Equation(6.9)** by substituting any value (*for example unit value*) in place of  $(V_t - \Delta F_N)$  (**Figure 2.9**).

**2.7.4.2** – In buildings in which  $N > 13$  excluding basement(s), natural period is not taken more than  $0.1 N$  as being independent from the value calculated in **Equation (2.11)**.

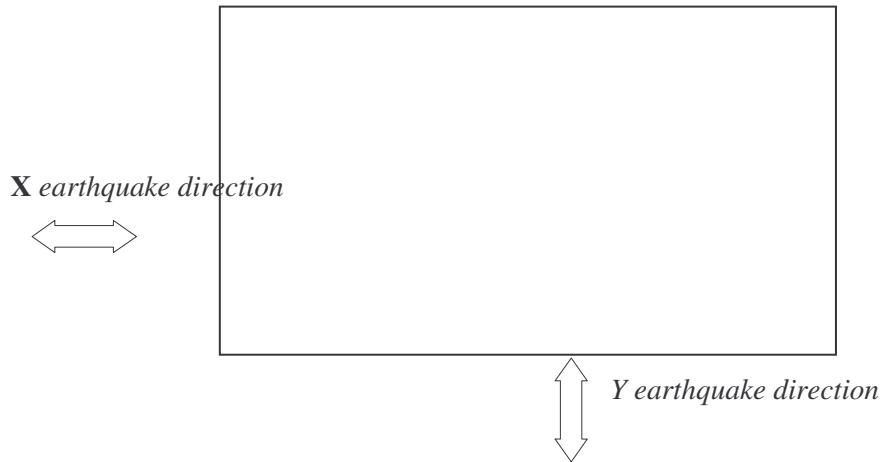


**Figure 2.9**

### 2.7.5. Internal Forces in the Directions of Elements at Principal Axis

Under combined effect of the earthquakes in the directions of x and y acting on structural system respectively, internal forces in the directions of principal axes a and b of elements of structural system shall be obtained by **Equation (2.12)** so as to yield the most unfavorable results (**Figure 2.10**).

$$\begin{aligned} B_a &= \pm B_{ax} \pm 0.30 B_{ay} & \text{veya} & B_a = \pm 0.30 B_{ax} \pm B_{ay} \\ B_b &= \pm B_{bx} \pm 0.30 B_{by} & \text{veya} & B_b = \pm 0.30 B_{bx} \pm B_{by} \end{aligned} \quad (2.12)$$



**Figure 2.10**

## 2.8. MODE SUPERPOSITION METHOD

In this method, maximum internal forces and displacements are determined by the statistical combination of maximum contributions obtained from each of the sufficient number of natural vibration modes considered.

### 2.8.1. Acceleration Spectrum

*Reduced acceleration spectrum* ordinate to be taken into account in any n'th vibration mode shall be determined by **Equation (2.13)**.

$$S_{aR}(T_n) = \frac{S_{ae}(T_n)}{R_a(T_n)} \quad (2.13)$$

In the case where elastic design acceleration spectrum is determined through special investigations in accordance with **2.4.4**, relevant spectrum ordinate shall be considered in **Equation (2.13)** in lieu of  $S_{ae}(T_n)$ .

## 2.8.2. Dynamic Degrees of Freedom to be considered

**2.8.2.1** – In buildings where floors behave as rigid horizontal diaphragms, two horizontal degrees of freedom in perpendicular directions and a rotational degree of freedom with respect to the vertical axis passing through the mass center shall be considered at each storey. Modal seismic loads shall be calculated for those degrees of freedom at each storey, but in order to consider additional eccentricity effects, they shall be applied to the points obtained by shifting the actual mass center by +5 % and –5 % of the floor length in perpendicular direction to the earthquake direction and to the mass center as an additional load (**Figure 2.7**).

**2.8.2.2** – In buildings where floor discontinuities defined under the title of **A2** irregularity exists and floors do not behave as rigid horizontal diaphragms, sufficient number of dynamic degrees of freedom shall be considered to account for the in-plane deformation of floors. In order to consider additional eccentricity effects, each of the modal seismic loads acting on the individual masses distributed over each floor shall be shifted by +5 % and –5 % of the floor length in perpendicular direction to the earthquake direction (**Fig. 2.8**). In such buildings, internal force and displacement quantities due to additional eccentricity effects alone may also be calculated in accordance with **2.7**. Such quantities shall be directly added to those combined in accordance with below given **2.8.4** without taking into account additional eccentricity effects.

## 2.8.3. Sufficient Number of Vibration Modes to be considered

**2.8.3.1** - *Sufficient number of vibration modes*,  $Y$ , to be taken into account in the analysis shall be determined to the criterion that the sum of effective participating masses calculated for each mode in each of the given  $x$  and  $y$  lateral earthquake directions perpendicular to each other shall in no case be less than 90 % of the total building mass.

$$\begin{aligned} \sum_{n=1}^Y M_{xn} &= \sum_{n=1}^Y \frac{L_{xn}^2}{M_n} \geq 0.90 \sum_{i=1}^N m_i \\ \sum_{n=1}^Y M_{yn} &= \sum_{n=1}^Y \frac{L_{yn}^2}{M_n} \geq 0.90 \sum_{i=1}^N m_i \end{aligned} \quad (2.14)$$

The expressions of  $L_{xn}$ ,  $L_{yn}$  and modal mass  $M_n$  appearing in **Equation (2.14)** are given below for buildings where floors behave as rigid diaphragms:

$$\begin{aligned} L_{xn} &= \sum_{i=1}^N m_i \Phi_{xin} \quad ; \quad L_{yn} = \sum_{i=1}^N m_i \Phi_{yin} \\ M_n &= \sum_{i=1}^N (m_i \Phi_{xin}^2 + m_i \Phi_{yin}^2 + m_{\theta i} \Phi_{\theta in}^2) \end{aligned} \quad (2.15)$$

**2.8.3.2** - In analysis of buildings with reinforced concrete peripheral walls at their basements being very rigid relative to upper stories and basement floors behaving as rigid diaphragms in horizontal planes, it may be sufficed with the consideration of vibration modes which are effective in the upper stories only. In this case, in the analysis performed by the Mode Superposition Method which corresponds to the analysis by Equivalent Seismic Load Method as given in Paragraph (a) of **2.7.2.4**, the coefficient  $R$  selected from **Table 2.5** shall be used without considering the rigid peripheral basement walls whereas the upper storey masses only shall be taken into account. Paragraphs (b) and (c) of **2.7.2.4** shall be applied as they are given.

#### **2.8.4. Combination of Modal Contributions**

Rules to be applied for the statistical combination of non-simultaneous maximum contributions of response quantities calculated for each vibration mode, such as base shear, storey shear, internal force components, displacement and storey drift, are specified in the following *provided that they are applied independently for each response quantity*:

**2.8.4.1** – In the cases where natural periods of any two vibration mode with  $T_m < T_n$  always satisfy the condition  $T_m / T_n < 0.80$ , *Square Root of Sum of Squares (SRSS) Rule* may be applied for the combination of maximum modal contributions.

**2.8.4.2** - In the cases where the above given condition is not satisfied, *Complete Quadratic Combination (CQC) Rule* shall be applied for the combination of maximum modal contributions. In the calculation of *cross correlation coefficients* to be used in the application of the rule, modal damping factors shall be taken as 5 % for all modes.

#### **2.8.5. Lower Limits of Response Quantities**

In the case where the ratio of the base shear in the given earthquake direction,  $V_{IB}$ , which is obtained through modal combination according to **2.8.4**, to the base shear,  $V_t$ , obtained by *Equivalent Seismic Load Method* through **Equation 2.4** is less than the below given value of  $\beta$  ( $V_{IB} < \beta V_t$ ), all internal force and displacement quantities determined by *Mode Superposition Method* shall be amplified in accordance with **Equation (2.16)**.

$$B_D = \frac{\beta V_t}{V_{IB}} B_B \quad (2.16)$$

In the case where at least one of the irregularities of type **A1**, **B2** or **B3** defined in **Table 2.1** exists in a building  $\beta = 0.90$ , whereas none of them exists  $\beta = 0.80$  shall be used in **Equation (2.16)**.

#### **2.8.6. Internal Forces in the Directions of Elements at Principal Axis**

Under combined effect of the earthquakes in the directions of  $x$  and  $y$  acting on structural system respectively, directional combination rule given in **2.7.5** shall be applied to the internal forces obtained by combining elements of structural system in accordance with **2.8.4** and in the direction of  $a$  and  $b$  principal axes.

## 2.9. ANALYSIS METHODS IN TIME DOMAIN

Artificially generated, previously recorded or simulated ground motions may be used for the linear or nonlinear elastic analysis of buildings and building-like structures in the time domain.

### 2.9.1. Artificially Generated Seismic Ground Motions

In order to use artificially generated ground motions, at least three seismic ground motions shall be generated in accordance with the following properties.

(a) The duration of strong motion part of the acceleration shall neither be less than 5 times of the first natural vibration period of the building nor less than 15 seconds.

(b) Average of spectral acceleration values of simulated seismic ground motion corresponding to period zero shall not be less than  $A_0g$ .

(c) Average of spectral acceleration values to be recalculated for each simulated acceleration record with 5 % damping ratio shall not be less than 90 % of elastic spectral accelerations  $S_{ae}(T)$  defined in 2.4 for the periods between  $0.2T_1$  and  $2T_1$  according to first period  $T_1$  in the direction of earthquakes considered. In the case where *nonlinear elastic analysis* is performed in the time domain, spectral acceleration values to be based on in order to obtain reduced seismic ground motion shall be calculated by **Equation (2.13)**.

### 2.9.2. Recorded or Simulated Seismic Ground Motions

Recorded earthquakes or physically simulated ground motions to source and wave propagation characteristics may be used. While generating such ground motions, floor conditions should be taken into account concordantly. In case of using recorded or simulated seismic ground motions at least three seismic ground motions shall be generated and those motions shall satisfy all the conditions given in 2.9.1.

### 2.9.3. Analysis in Time Domain

In case of making nonlinear elastic analysis in time domain, internal force – transformation cohesions representing dynamic behavior of elements of structural system under cycling loads shall be defined by utilizing related literature provided that their theoretical and experimental validities are proven. In linear and nonlinear analysis, in case of using three ground motions the maximum of results, and in case of using at least seven ground motions average of results shall be taken for the design.

## 2.10. LIMITATION OF RELATIVE STOREY DRIFTS, SECOND ORDER EFFECTS AND SEISMIC JOINTS

### 2.10.1. Calculation and Limitation of Effective Relative Storey Drifts

**2.10.1.1** – *Reduced relative storey drift*,  $\Delta_i$ , of any column or structural wall shall be determined by **Equation (2.17)** as the difference of displacements between the two consecutive stories.

$$\Delta_i = d_i - d_{i-1} \quad (2.17)$$

In **Equation (2.17)**  $d_i$  and  $d_{i-1}$  represent lateral displacements obtained from the analysis according to reduced seismic loads at the ends of any column or structural wall at stories  $i$  and  $(i - 1)$ . However, the condition in **2.7.4.2** and the condition of minimum equivalent seismic load defined in **Equation (2.4)** **may not be considered in calculation of  $d_i$  and  $\Delta_i$ .**

**2.10.1.2** – Effective relative storey drift,  $\delta_i$ , for columns and structural walls of the  $i$ 'th storey of a building for each earthquake direction shall be obtained from **Equation (2.18)**.

$$\delta_i = R \Delta_i \quad (2.18)$$

**2.10.1.3** – The maximum value of effective relative storey drifts,  $\delta_i$ , within a storey,  $(\delta_i)_{\max}$ , calculated by **Equation (2.18)** for columns and structural walls of the  $i$ 'th storey of a building for each earthquake direction shall satisfy the condition given by **Equation (2.19)**:

$$\frac{(\delta_i)_{\max}}{h_i} \leq 0.02 \quad (2.19)$$

This limit can be increased maximum of 50 % in single – storey buildings in which seismic loads are fully resisted by structural steel frames with connections capable of cyclic moment transfer.

**2.10.1.4** – In the case where the condition specified by **Equation (2.19)** is not satisfied at any storey, the earthquake analysis shall be repeated by increasing the stiffness of the structural system. The serviceability of nonstructural brittle elements (such as facade elements) under effective relative storey drifts shall be verified by calculation even if the above given conditions satisfied.

## 2.10.2. Second – Order Effects

Unless a more refined analysis considering the nonlinear elastic behavior of structural system is performed, second-order effects may be taken into account as follows.

**2.10.2.1** – In the case where *Second – Order Effect Indicator*,  $\theta_i$ , satisfies the condition given by **Equation (2.20)** for the earthquake direction considered at each storey, second – order effects shall be evaluated in accordance with currently enforced specifications of reinforced concrete or structural steel design.

$$\theta_i = \frac{(\Delta_i)_{avr} \sum_{j=i}^N w_j}{V_i h_i} \leq 0.12 \quad (2.20)$$

Here  $(\Delta_i)_{avr}$  shall be determined in accordance with **2.10.1.1** as the average value of reduced relative storey drifts calculated for  $i$ 'th storey columns and structural walls within the storey.

**2.10.2.2** – In the case where the condition given by **Equation (2.20)** is not satisfied, seismic analysis shall be repeated by sufficiently increasing the stiffness of the structural system.

## 2.10.3. Seismic Joints

Excluding the effects of differential settlements and rotations of foundations and the effects of temperature change, the conditions are indicated concerning sizes of gaps to be retained in the seismic joints between building blocks or between the old and newly constructed buildings:

**2.10.3.1** - Unless a more unfavorable value is obtained in accordance with **2.10.3.2** below, sizes of gaps shall not be less than the value found as result of multiplication of coefficient  $\alpha$  specified below with square root of sum of squares of displacements obtained in adjacent buildings or blocks for each storey. Storey displacements to be considered shall be the average values of reduced displacements  $d_i$  calculated within a storey at the column or structural wall joints. In case the seismic analysis is not possible for the existing old building, the storey displacements of old building shall not be assumed to be less than those obtained for the new building at the same stories.

(a)  $\alpha = R / 4$  if all floor levels of adjacent buildings or building blocks are the same at all stories.

(b)  $\alpha = R / 2$  shall be valid for entire building if any of the floor levels of adjacent buildings or building blocks are not the same.

**2.10.3.2** – Minimum size of gaps shall be at least 30 mm up to 6 m height and from thereon a minimum 10 mm shall be added for each 3 m height increment.

**2.10.3.3** - Seismic joints shall be arranged so as to allow the independent movement of building blocks in all earthquake directions.

## **2.11. SEISMIC LOADS APPLIED TO STRUCTURAL APPENDAGES, ARCHITECTURAL ELEMENTS, MECHANICAL AND ELECTRICAL EQUIPMENT**

**2.11.1** – Equivalent seismic loads to be applied to structural appendages which are connected to structural system but serving independently such as balconies, parapets, chimneys, etc. and to all non – structural architectural elements such as facade and partition panels, etc.; to be used in analysis of mechanical and electrical equipments and also connections of these to structural system of building are given by **Equation (2.21)**.

$$f_e = 0.5 A_o I w_e \left( 1 + 2 \frac{H_i}{H_N} \right) \quad (2.21)$$

The seismic load calculated shall be acted to center of gravity of related element so as to give the most unfavorable internal forces in the horizontal direction. Half of equivalent seismic load calculated in **Equation (2.21)** shall be acted vertically to the elements which are not in the vertical direction.

**2.11.2** – In the case where the sum of weights of mechanical or electrical equipment indicated by  $w_e$  in **Equation (2.21)** at any  $i$ 'th storey is greater than  $0.2w_i$ , equipment weights and stiffness properties of their connections shall be taken into account in the structural analysis.

**2.11.3** – **Equation(2.21)** may not be applied in industrial buildings where *floor acceleration spectrum* is determined by appropriate methods defining the peak acceleration at the floor where mechanical or electrical equipment are located.

**2.11.4** - Twice of the seismic load obtained from **Equation (2.21)** shall be considered in the analysis of emergency electric back – up and fire fighting systems, for all equipment attached to the infill walls and for their connections.

## 6.12. NON – BUILDING STRUCTURES

Non – building structures permitted to be analyzed in accordance with the requirements of this chapter and the corresponding *Structural Behavior Factors*, ( $R$ ), to be applied to such structures are given in **Table 2.8**. Applicable seismic load reduction factors shall be determined in accordance with **Equation (2.3)**. Where applicable, *Building Importance Factors* specified in **Table 2.3** shall be used for non – building structures. However *Live Load Participation Factors* specified in **Table 2.7** shall not be applied. Except snow loads and crane payloads, unreduced weights of all solid and liquid materials stored and mechanical equipment shall be used.

**TABLE 2.8 - STRUCTURAL SYSTEM BEHAVIOUR FACTORS FOR NON-BUILDING STRUCTURES**

| <b><i>TYPE OF STRUCTURE</i></b>   | <b><i>R</i></b> |
|---|-----------------|
| Elevated liquid tanks, pressurized tanks, bunkers, vessels carried by frames of high ductility level or steel eccentric braced frames                       | 4               |
| Elevated liquid tanks, pressurized tanks, bunkers, vessels carried by frames of nominal ductility level or steel centric braced frames                      | 2               |
| Cast – in – situ reinforced concrete silos, industrial chimneys and suchlike structural systems with uniformly distributed mass along height <sup>(*)</sup> | 3               |
| Reinforced concrete cooling towers <sup>(*)</sup>   | 3               |
| Space truss steel towers, steel silos and industrial chimneys with uniformly distributed mass along height <sup>(*)</sup>                                   | 4               |
| Guyed steel high posts and guyed steel  | 2               |
| Inverted pendulum type structures carried by a single structural element with mass concentrated at the top  | 2               |
| Industrial type steel storage racks   | 4               |

<sup>(\*)</sup>*Seismic analysis of such structures shall be performed in accordance with 2.8 or 2.9 by considering discrete dynamic degrees of freedom defining the structure sufficiently.*

## 2.13. REQUIREMENTS FOR SEISMIC ANALYSIS REPORTS

The following requirements shall be applied for the analysis reports that include seismic analysis of buildings:

**2.13.1** – Types of irregularities specified in **Table 2.1** shall be evaluated in detail for the building to be designed and, if any, existing irregularities shall be identified.

**2.13.2** – The selected structural system of high or nominal ductility level shall be clearly defined with respect to the requirements of **Chapter 3** or **Chapter 4**, and the selection of the applicable  $R$  factor from **Table 2.5** shall be explained.

**2.13.3** – The selection of the applicable analysis method in accordance with **2.6** shall be clearly explained by considering the seismic zone in which the building exists, building height and structural irregularities involved.

**2.13.4** – The following rules shall be applied in the cases where the analysis is performed by computer:

(a) Analysis report shall include three-dimensional illustrations of structural system by indicating the joint and element numbering.

(b) All input data as well as output data including internal forces and displacements shall be included in the analysis report in an easily understandable format. When requested by the approval authority, all computer files shall be delivered electronically.

(c) The title, author and the version of the computer software used in the analysis shall be clearly indicated.

(d) When requested by the approval authority, theory manual and user's guide of the computer software shall be included in the analysis report.

## **2.14. INSTALLATION OF ACCELEROMETERS**

Upon endorsement by the Ministry of Public Works and Settlement, strong motion accelerometer shall be permitted to be installed by the ministry or university institutions on the public, private or corporate buildings and other structures for the purpose of recording the strong earthquake motions, and owners or operators of buildings or structures shall be responsible from the safety of such instruments.

## CHAPTER 3 - EARTHQUAKE RESISTANT DESIGN REQUIREMENTS FOR REINFORCED CONCRETE BUILDINGS

### 2.0. NOTATIONS

Dimensioned expressions used in this chapter with the following notations are in *Newton* [N] for forces, *millimeter* [mm] for lengths and *Mega Pascal* [MPa] = [N/mm<sup>2</sup>] for stresses.

|            |   |
|------------|---|
| $A_c$      | = Gross section area of column or wall end zone   |
| $A_{ch}$   | = Gross section area of a solid wall, wall segment of a coupled wall, a floor or a floor segment of a perforated floor  |
| $A_{ck}$   | = Concrete core area within outer edges of confinement reinforcement  |
| $\sum A_e$ | = Effective shear area at any storey for the earthquake direction considered  |
| $\sum A_g$ | = Sum of section areas of structural elements at any storey behaving as structural walls in the direction parallel to the earthquake direction considered   |
| $\sum A_k$ | = Sum of masonry infill wall areas (excluding door and window openings) at any storey in the direction parallel to the earthquake direction considered  |
| $A_{os}$   | = Section area of spiral reinforcement  |
| $\sum A_p$ | = Sum of plan areas of all stories of building  |
| $A_{s1}$   | = Total area of tension reinforcement placed on one side of the beam-column loop at the top to resist the negative beam moment  |
| $A_{s2}$   | = Total area of tension reinforcement placed on the other side of the beam – column loop with respect to $A_{s1}$ at the bottom to resist negative beam moment  |
| $A_{sd}$   | = Total reinforcement area of each of the cross rebar bundles in coupling beam  |
| $A_{sh}$   | = Along the height corresponding to transverse reinforcement spacing $s$ , sum of projections of cross section areas of all legs of hoops and crossties of columns or wall end zones in the direction perpendicular to $b_k$ considered   |
| $A_w$      | = Effective web area of column cross section (excluding protrusions in the direction perpendicular to the earthquake direction)   |
| $\sum A_w$ | = Sum of effective web areas of column cross sections, $A_w$ 's at any storey   |
| $a$        | = Lateral distance between legs of hoops and / or crossties of columns or wall end regions  |
| $b_j$      | = In the earthquake direction considered, column width in case the beam into the joint has the same width as column or expands in both sides of the column, otherwise twice the smaller of the distances measured from the vertical centerline of beam to the edges of column (It shall not exceed beam width plus joint depth) |
| $b_k$      | = For each of the orthogonal lateral directions, cross section dimension of concrete core of column or wall end zone (distance between the centers or outermost rebars)   |
| $b_w$      | = Width of beam web, thickness of wall web  |
| $D$        | = Concrete core diameter of circular column (distance between the centers of spiral reinforcement)  |
| $d$        | = Effective beam height   |
| $f_{cd}$   | = Design compressive strength of concrete   |
| $f_{ck}$   | = Characteristic compressive cylinder strength of concrete  |
| $f_{ctd}$  | = Design tensile strength of concrete   |
| $f_{yd}$   | = Design yield strength of longitudinal reinforcement   |
| $f_{yk}$   | = Characteristic yield strength of longitudinal reinforcement   |
| $f_{ywk}$  | = Characteristic yield strength of transverse reinforcement   |
| $H_{cr}$   | = Critical wall height  |

|            |   |
|------------|---|
| $H_w$      | = Total structural wall height measured from top foundation level or ground floor level   |
| $h$        | = Column cross section dimension in the earthquake direction considered   |
| $h_k$      | = Beam height   |
| $\ell_b$   | = Development length of tensile reinforcement as given in TS-500  |
| $\ell_n$   | = Clear height of column between beams, clear span of beam between column or wall faces   |
| $\ell_w$   | = Length of wall or segment of coupled wall in plan   |
| $M_a$      | = Moment at the bottom of column clear height which is used for the calculation of column shear force   |
| $(M_a)_t$  | = Moment calculated under the combined effect of seismic loads and vertical loads multiplied with load coefficients at the bottom section of structural wall. |
| $M_{pa}$   | = Moment capacity calculated at the bottom of column clear height by considering $f_{ck}$ , $f_{yk}$ and strain hardening of steel                            |
| $M_{pi}$   | = Positive or negative moment capacity calculated at column face on left end i of a beam by considering $f_{ck}$ , $f_{yk}$ and strain hardening of steel     |
| $M_{pj}$   | = Negative or positive moment capacity calculated at column face on right end j of a beam by considering $f_{ck}$ , $f_{yk}$ and strain hardening of steel    |
| $\sum M_p$ | = Sum of moment capacities of beams framing into a loop   |
| $M_{pü}$   | = Ultimate moment capacity calculated at the top of column clear height by considering $f_{ck}$ , $f_{yk}$ and strain hardening of steel                      |
| $(M_a)_t$  | = Moment capacity calculated at the bottom section of wall by considering $f_{ck}$ , $f_{yk}$ and strain hardening of steel                                   |
| $M_{ra}$   | = Ultimate moment resistance calculated at the bottom of column or wall clear height by considering $f_{cd}$ and $f_{yd}$                                     |
| $M_{ri}$   | = Positive or negative ultimate moment resistance calculated at column or wall face on left end i of a beam by considering $f_{cd}$ and $f_{yd}$              |
| $M_{rj}$   | = Negative or positive ultimate moment resistance calculated at column or wall face on right end j of a beam by considering $f_{cd}$ and $f_{yd}$             |
| $(M_r)_t$  | = Ultimate moment resistance calculated at bottom section of wall by considering $f_{cd}$ and $f_{yd}$  |
| $M_{rü}$   | = Ultimate moment resistance calculated at the top of column or wall clear height by considering $f_{cd}$ and $f_{yd}$  |
| $M_{ü}$    | = Moment at the top of column clear height which is used for the calculation of column shear force  |
| $N_d$      | = Axial force calculated under combined effect of seismic loads and vertical loads multiplied with load coefficients  |
| $N_{dm}$   | = Greater of the axial pressure forces calculated under combined effect of seismic loads and vertical loads   |
| $s$        | = Spacing of transverse reinforcement, step of spiral reinforcement   |
| $V_c$      | = Contribution of concrete to shear strength  |
| $V_d$      | = Shear force calculated under combined effect of seismic loads and vertical loads multiplied with load coefficients  |
| $V_{dy}$   | = Simple beam – shear force developed at any section of the beam due to vertical loads  |
| $V_e$      | = Shear force taken into account for the calculation of transverse reinforcement of column, beam or wall  |
| $V_{ik}$   | = Sum of shear forces calculated in the earthquake direction considered in accordance with <b>Chapter 2</b> at all columns of the i'th storey                 |

|             |  |
|-------------|--|
| $V_{is}$    | = Sum of shear forces calculated in the earthquake direction considered in accordance with <b>Chapter 2</b> at the i'th storey columns where <b>Equation (3.3)</b> is satisfied at both bottom and top loops |
| $V_{kol}$   | = Smaller of the shear forces at above and below the loop calculated in accordance with <b>Chapter 2</b>   |
| $V_r$       | = Shear strength of a section of column, beam or wall  |
| $V_t$       | = Total seismic load acting on a building (base shear) according to <b>Chapter 2</b>   |
| $\alpha_i$  | = Ratio of $V_{is} / V_{ik}$ calculated for any i'th storey  |
| $\emptyset$ | = Reinforcement diameter   |
| $\gamma$    | = Angle of cross rebar bundle used in coupling beam with the horizontal axis   |
| $\rho$      | = Tension reinforcement ratio at the top and bottom of beam support section  |
| $\rho_s$    | = Volumetric ratio of spiral reinforcement of column [ $\rho_s = 4 A_{os} / (D s)$ ]   |
| $\rho_{sh}$ | = Volumetric ratio of horizontal web reinforcement of wall [ $(\rho_{sh})_{min} = 0.0025$ ]  |

### 3.1. SCOPE

**7.1.1** - Dimensioning and reinforcing of all structural elements of reinforced concrete buildings to be built in seismic zones shall be performed, along with currently enforced relevant standards and codes, primarily in accordance with the requirements of this chapter. Requirements for reinforced concrete building foundations are given in **Chapter 6**.

**3.1.2** - Requirements and rules specified in this chapter are applicable to cast - in - situ monolithic reinforced concrete buildings, and unless otherwise stated, prefabricated buildings whose structural systems are comprised of reinforced and / or pre - stressed concrete elements.

**3.1.3** - Lateral load carrying systems of reinforced concrete buildings covered in this chapter may be comprised of frames only, of walls only or of combination of frames and walls.

**3.1.4** - Reinforced concrete buildings with concrete strength exceeding that of **C 50** and buildings where steel profiles are used as reinforcement in structural elements are outside the scope of this chapter.

### 3.2. GENERAL RULES

#### 3.2.1. Classification of Reinforced Concrete Structural Systems

Lateral load resisting structural systems of reinforced concrete buildings shall be classified with respect to their seismic behavior into two classes defined below. Special cases and requirements regarding the mixed use of such classes of systems are given in **2.5.4** of **Chapter 2**.

**3.2.1.1** - Reinforced concrete structural systems given below are defined as *Systems of High Ductility Level*:

(a) Frame type structural systems comprised of columns and beams dimensioned and reinforced in accordance with the requirements of **3.3**, **3.4** and **3.5**.

(b) Structural systems comprised of solid or coupled structural walls dimensioned and reinforced in accordance with the requirements of **3.6**.

(c) Frame - wall structural systems made of combining two systems defined above.

**3.2.1.2** - Reinforced concrete structural systems given below are defined as *Systems of Nominal Ductility Level*:

(a) Frame type structural systems comprised of columns and beams dimensioned and reinforced in accordance with the requirements of **3.7**, **3.8** and **3.9**.

(b) Structural systems comprised of solid or coupled structural walls dimensioned and reinforced in accordance with the requirements of **3.10**.

(c) Frame - wall structural systems made of combining two systems defined above.

### 3.2.2. Relevant Standards

Cast - in - situ and prefabricated reinforced concrete structural systems shall be designed, along with the requirements of this chapter, according to the seismic loads and analysis requirements given in **Chapter 2**, to other loads specified in TS - 498 and TS - 9967; and to the requirements as well as material and load factors specified in TS - 500, TS - 708, TS - 3233 and TS - 9967.

### 3.2.3. Section Stiffnesses to be used in Structural Analysis

Section stiffnesses for uncracked sections shall be used in the structural analysis to be performed with the methods given in **Chapter 2**. However respective values for cracked sections may be used for beams framing into walls in their own plane and for coupling beams of the coupled structural walls.

### 3.2.4. Method to be used in Section Design

The use of the *Ultimate Strength Method* given in TS - 500 is mandatory in earthquake resistant dimensioning and reinforcement calculations of reinforced concrete structural elements in all seismic zones.

### 3.2.5. Material

**3.2.5.1** - In all buildings to be built in seismic zones, concrete with strength less than **C 20** shall not be used.

**3.2.5.2** - In all seismic zones, it is necessary to use concrete produced with concrete quality control requirements specified in TS - 500 and concrete placed with using vibrators. However, in case of the usage of self - placed concretes, placing concrete with vibrator is not required.

**3.2.5.3** - Unribbed reinforcement steel can not be used exempt hoops and crossties with flooring reinforcement. With the exception of elements mentioned in **3.2.5.4** below, reinforcing steel with strength exceeding that of **S 420** shall not be used reinforced concrete structural elements. The rupture strain of reinforcement to be used shall not be less than 10 %. Experimentally obtained average yield strength of reinforcing steel shall not be more than 1.3 times the characteristic yield strength specified in the relevant steel standard. In addition, experimentally obtained average rupture strength shall not be less than 1.25 times the average yield strength obtained as well from the experiment.

**3.2.5.4** - Reinforcing steel with strength exceeding that of **S 420** may be used in flat slabs, in the slabs of joist floors, in peripheral external walls of basements, in the webs of structural walls of buildings in which entire seismic loads are resisted by such walls of full building height satisfying both of the conditions given by **Equation (3.14)** in **3.6.1.2**, and as pre - stressing steel in prefabricated buildings.

### 3.2.6. Development Length of Tensile Reinforcement

Unless stated otherwise in this chapter, development lengths of tensile rebars with and without hooks shall be determined in accordance with TS - 500.

### 3.2.7. Welded Splices and Mechanically Connected Reinforcement

**3.2.7.1** - Welded lap splices of longitudinal reinforcement shall be made by certified welders. Butt weld splices shall not be permitted. Carbon equivalency of the reinforcement steel to be welded shall not exceed the limit value given in TS - 500.

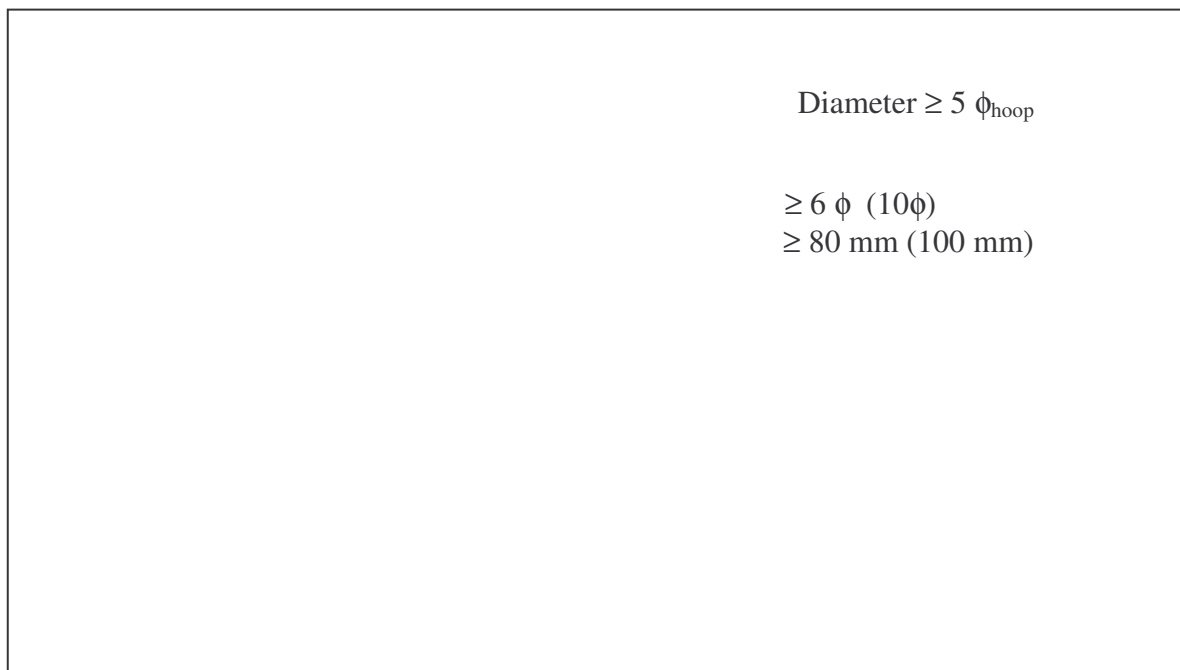
**3.2.7.2** - Tension test shall be applied to at least 2 % of welded splices and mechanical connections of longitudinal reinforcement provided that number of tests shall not be less than 5. Experimentally determined tensile strength of the connection shall not be less than the rupture strength given in TS - 500.

**3.2.7.3** - Transverse reinforcement shall not be permitted to be welded to longitudinal reinforcement.

**3.2.7.4** - Frames of steel windows and doors, anchors, connection plates, elements of plumbing system, machinery and equipment shall not be permitted to be welded to longitudinal and transverse reinforcement.

### 3.2.8. Special Seismic Hoops and Crossties

Hoops and crossties used in columns, beam - column joints, wall end zones and beam confinement zones of all reinforced concrete systems of high ductility level or normal ductility level in all seismic zones shall be *special seismic hoops* and *special seismic crossties* for which requirements are given below (**Fig.3.1**):



**Figure 3.1**

**3.2.8.1** - Special seismic hoops shall always have *135 degree hooks* at both ends. However, 90 degree hook may be made at one end of the special seismic crossties. In this case, crossties with 135 degree and 90 degree hooks shall be placed on one face of a column or wall in a staggered form in both horizontal and vertical directions. 135 degree hooks shall be bent around a circle with at least  $5\varnothing$  diameter where  $\varnothing$  denotes the diameter of transverse reinforcing bar. Lengths of hooks measured from tangent point shall not be less than  $10\varnothing$  and 100 mm for plain bars,  $6\varnothing$  and 80 mm for ribbed bars.

**3.2.8.2** - Special seismic hoops shall engage the longitudinal reinforcement from outside with hooks closed around the same rebar. Diameter and spacing of special seismic crossties shall be the same as those of hoops. Crossties shall be connected to longitudinal reinforcement always at both ends. Hoops and crossties shall be firmly tied such that they shall not move during concrete pouring.

### **3.3. COLUMNS OF HIGH DUCTILITY LEVEL**

#### **3.3.1. Cross-section Requirements**

**3.3.1.1** - Shorter dimension of columns with rectangular section shall not be less than 250 mm and section area shall not be less than  $75000 \text{ mm}^2$ . Diameter of circular columns shall be at least 300 mm.

**3.3.1.2** - In order the gross section area of column to be the biggest one of axial pressure strengths calculated under the combined effect of  $N_{dm}$  vertical loads and seismic loads, gross section area of column shall satisfy the condition  $A_c \geq N_{dmax} / (0.50 f_{ck})$ .

#### **3.3.2. Longitudinal Reinforcement Requirements**

**3.3.2.1** - Longitudinal column reinforcement shall not be less than 1 %, nor shall it be more than 4 % of gross section area. Minimum number of rebars shall be  $4\varnothing 16$  or  $6\varnothing 14$  for rectangular columns and  $6\varnothing 14$  for circular columns.

**3.3.2.2** - Longitudinal reinforcement ratio shall not exceed 6 % at lap spliced sections.

#### **3.3.3. Arrangement of Longitudinal Reinforcement**

**3.3.3.1** - Lap splices of column longitudinal reinforcement should be made, as much as possible, within the column central zone defined in **3.3.4.2**. In this case the splice length shall be equal to the development length  $\ell_b$  given in TS - 500 for tension bars.

**3.3.3.2** - In the case where lap splices of column longitudinal reinforcement are made at the bottom end of the column, the following requirements shall be met:

(a) In the case where 50 % of longitudinal reinforcement or less is spliced at the bottom end of column, lap splice length shall be at least 1.25 times  $\ell_b$ .

(b) In the case where more than 50 % of longitudinal reinforcement is spliced at the bottom end of column, lap splice length shall be at least 1.5 times  $\ell_b$ . The same condition shall apply to starter bars protruding from the foundation.

(c) In both cases given above, minimum transverse reinforcement defined in **3.3.4.1** shall be used along the length of the lap splice.

**3.3.3.3** - In the case where the column cross-section changes between consecutive stories, slope of the longitudinal reinforcement within the beam - column joint shall not be more than 1 / 6 with respect to the vertical. When the change in cross section is more or in the case of top storey columns; development length of the column longitudinal reinforcement within the other side of the beam above shall not be less than 1.5 times the development length  $\ell_b$  given in TS - 500 for tension reinforcement, nor shall it be less than **40  $\phi$** . In the case of no beam present on the other side, development shall be achieved, if necessary, by downward bending of rebar along the far face of the column. Length of 90 degree bent horizontal hook or downward bent vertical hook shall be at least **12  $\phi$**  (**Fig. 3.2**).

**3.3.3.4** - Longitudinal distance between mechanical or welded connections on adjacent longitudinal rebars shall not be less than 600 mm.



$$\begin{aligned}(a + b) &\geq 1.5 \ell_b \\ (a + b) &\geq 40 \phi \\ b &\geq 12 \phi\end{aligned}$$

$$\begin{aligned}e &\geq 1.5 \ell_b \\ e &\geq 40 \phi\end{aligned}$$

$$\begin{aligned}(a + b + c) &\geq 1.5 \ell_b \\ (a + b + c) &\geq 40 \phi \\ c &\geq 12 \phi\end{aligned}$$

**Figure 3.2**

### **3.3.4. Transverse Reinforcement Requirements**

Unless a more unfavorable situation governs in accordance with below given **3.3.7.6**, the minimum transverse reinforcement requirements of columns are those specified in **3.3.4.1** for *column confinement zones* and in **3.3.4.2** for the *column central zone* (**Fig.3.3**). *Special seismic hoops* and *special seismic crossties* defined in **3.2.8** shall be used along the full length of the column.

**3.3.4.1** - Special confinement zones shall be arranged at the bottom and top ends of each column. Length of each of the confinement zones shall not be less than smaller of column cross section dimensions (diameter in circular columns), 1 / 6 the clear height of column measured upward from floor level or downward from the bottom face of the deepest beam framing into the column, and 500 mm. Such reinforcement shall be extended into the foundation for a length equal to at least twice the smaller of column cross section dimensions. Requirements for transverse reinforcement to be used in confinement zones are given below. Those reinforcements shall be continued through the length not less than 25 times of the diameter of biggest longitudinal reinforcement and not less than 300 mm inside the foundation. However, on the column which are supported to pot foundations, longitudinal reinforcements in the confinement zones shall be continued through the length of the pot.

(a) Transverse reinforcement with a diameter less than  $\varnothing 8$  shall not be used in confinement zones. Along the column, spacing of hoops and crossties shall not be more than  $1/3$  the smaller cross section dimension and 100 mm, nor shall it be less than 50 mm. Lateral distance between legs of hoops and crossties,  $a$ , shall not be more than 25 times the hoop diameter. Pitch of the continuous spirals shall not be more than  $1/5$  the core diameter and 80 mm.

(b) In the case where  $N_d > 0.20 A_c f_{ck}$  in columns with hoops, minimum total area of transverse reinforcement to be used in confinement zones shall be calculated to satisfy the more unfavorable of the requirements given in **Equation (3.1)**. In this calculation, core diameter of column,  $b_k$ , shall be considered separately for each direction (**Fig. 3.3**).

$$\begin{aligned} A_{sh} &\geq 0.30 s b_k [(A_c / A_{ck}) - 1] (f_{ck} / f_{ywk}) \\ A_{sh} &\geq 0.075 s b_k (f_{ck} / f_{ywk}) \end{aligned} \quad (3.1)$$

(c) In the case where  $N_d > 0.20 A_c f_{ck}$  in columns with spirals, minimum volumetric ratio of transverse reinforcement to be used in confinement zones shall be calculated to satisfy the more unfavorable of the requirements given in **Equation (3.2)**.

$$\begin{aligned} \rho_s &\geq 0.45 [(A_c / A_{ck}) - 1] (f_{ck} / f_{ywk}) \\ \rho_s &\geq 0.12 (f_{ck} / f_{ywk}) \end{aligned} \quad (3.2)$$

(d) In the case where  $N_d \leq 0.20 A_c f_{ck}$ , at least  $2/3$  the transverse reinforcement given by **Equation (3.1)** and **Equation (3.2)** shall be used as a minimum transverse reinforcement in column confinement zones.

**3.3.4.2 - Column central zone** is the region between the confinement zones defined at the bottom and top ends of the column (**Fig. 3.3**). Transverse reinforcement with a diameter less than  $\varnothing 8$  shall not be used along the column central zone. Along this zone, spacing of hoops and crossties shall not be more than half the smaller cross section dimension and 200 mm. Lateral distance between the legs of hoops and / or crossties,  $a$ , shall not be more than 25 times the hoop diameter.

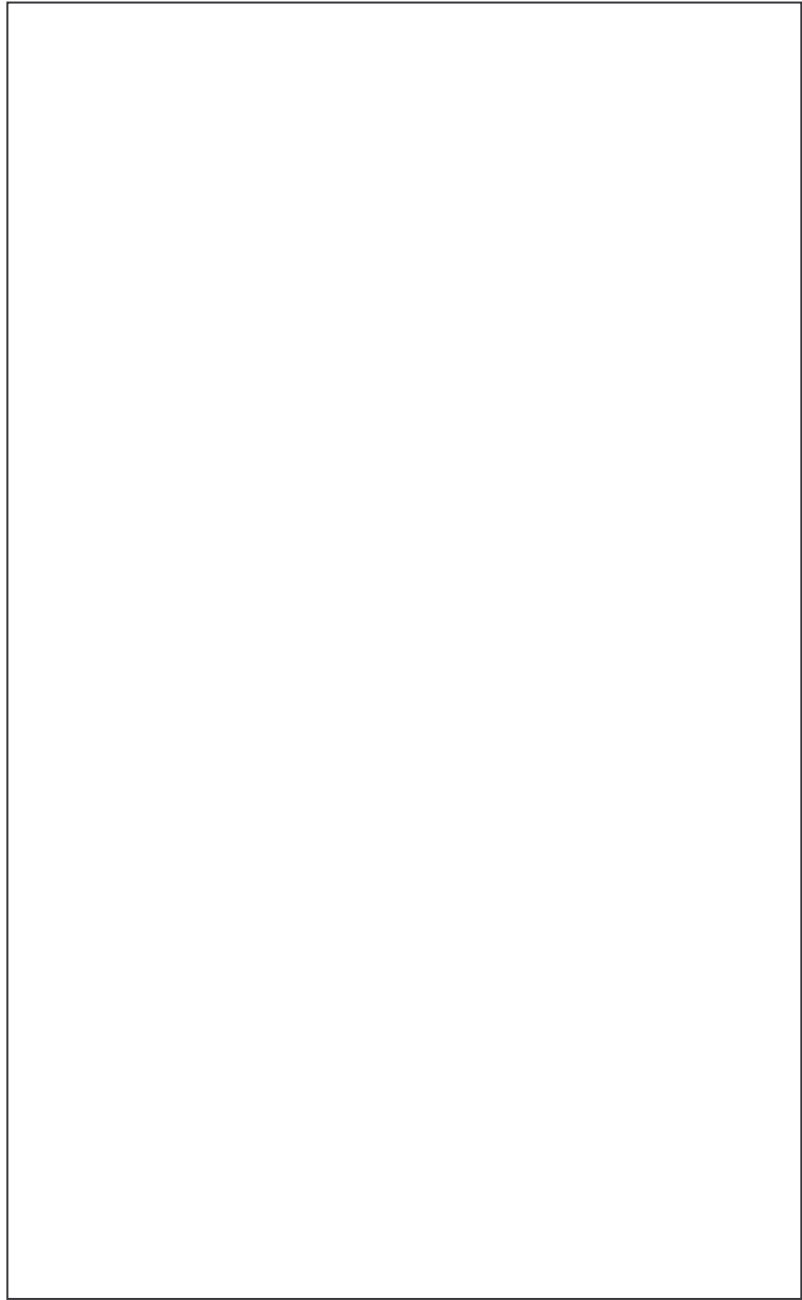
### 3.3.5. Requirement of Having Columns Stronger Than Beams

**3.3.5.1** - In structural systems comprised of frames only or of combination of frames and walls, sum of ultimate moment resistances of columns framing into a beam-column joint shall be at least 20% more than the sum of ultimate moment resistances of beams framing into the same joint (**Fig. 3.4**):

$$(M_{ra} + M_{r\ddot{u}}) \geq 1.2 (M_{ri} + M_{rj}) \quad (3.3)$$

**3.3.5.2** - In order that **Equation (3.3)** is applied, beams framing into the joint shall satisfy the dimensional requirements given in **3.4.1.1**.

**3.3.5.3** - **Eq (3.3)** shall be applied separately for both earthquake directions and senses to yield the most unfavorable result (**Fig. 3.4**). In calculating the column ultimate moment resistances, axial forces  $N_d$  shall be taken to yield the minimum moments consistent with the sense of earthquake direction.



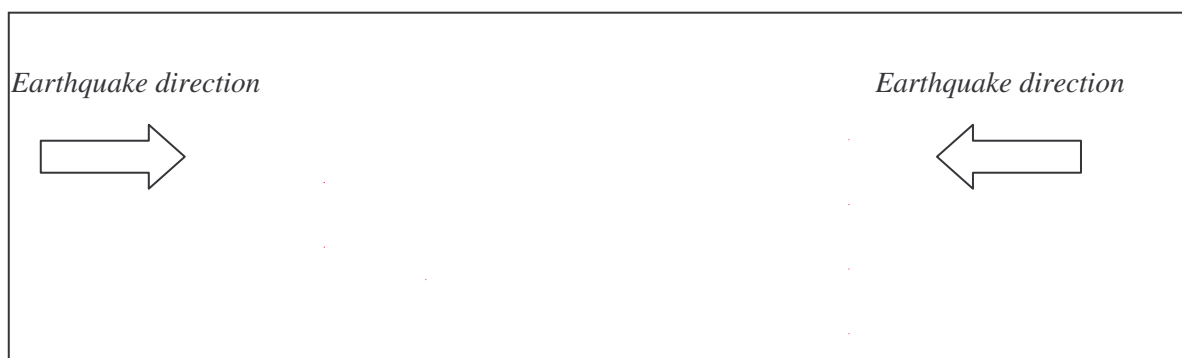
**Figure 3.3**

**3.3.5.4** – Special situations regarding the application of Equation (7.3) are described in the following:

(a) **Equation (3.3)** need not to be applied in the case where  $N_d \leq 0.10 A_c f_{ck}$  in both columns framing into the joint.

(b) **Equation (3.3)** need not to be checked in single storey buildings and in joints of topmost storey of multi-storey buildings.

(b) **Equation (3.3)** need not to be checked in the case where a wall connected by beams works like a column in its weak direction.



**Figure 3.4**

### **3.3.6. The Case Where Some Columns Cannot Satisfy the Requirement of Having Columns Stronger Than Beams**

**3.3.6.1** – In structural systems comprised of frames only or of combination of walls and frames, **Equation (3.3)** may be permitted not to be satisfied in a given earthquake direction at some joints at the bottom and / or top of an  $i$ 'th storey, provided that **Equation (3.4)** given below is satisfied.

$$\alpha_i = V_{is} / V_{ik} \geq 0.70 \quad (3.4)$$

Columns satisfying the condition of  $N_d \leq 0.10 A_c f_{ck}$  may be taken into account in the calculation of  $V_{is}$  even if they do not satisfy **Equation (3.3)**.

**3.3.6.2** – In the case where **Equation (3.4)** is satisfied, bending moments and shear forces of columns satisfying **Equation (3.3)** at both bottom and top joints shall be amplified by multiplying with the ratio  $(1/\alpha_i)$  within the range of  $0.70 < \alpha_i < 1.00$ .

**3.3.6.3** – In the case where **Equation (3.4)** is not satisfied at any storey, all frames of structural systems which may be comprised of frames only or of combination of walls and frames shall be considered as *frames of nominal ductility level*, and the analysis shall be repeated by changing the Structural Behavior Factor according to **Table 2.5**. As it is mentioned in **2.5.4.1** in **Chapter 2**, it is possible, however, to combine frames of nominal ductility level with structural walls of high ductility level.

### 3.3.7. Shear Safety of Columns

**3.3.7.1** – Shear force,  $V_e$ , to be taken into account for the design of column transverse reinforcement shall be calculated by **Equation (3.5)**.

$$V_e = (M_a + M_{ii}) / \ell_n \quad (3.5)$$

In determining  $M_a$  and  $M_{ii}$  of **Equation (3.5)**, below given **3.3.7.2** shall be applied for the case where **Equation (3.3)** is satisfied at both bottom and top joints of the column, whereas **3.3.7.3** shall be applied otherwise (**Fig.3.5**).

**3.3.7.2** – Sum of ultimate moment capacities,  $\sum M_p$ , at the ends of beams framing into the joint where **Equation (3.3)** is satisfied shall be calculated:

$$\sum M_p = M_{pi} + M_{pj} \quad (3.6)$$

In the case where a more rigorous analysis is not performed, it may be assumed to be  $M_{pi} \cong 1.4 M_{ri}$  and  $M_{pj} \cong 1.4 M_{rj}$ . The moment  $\sum M_p$  shall be distributed to columns in proportion to the moments obtained in accordance with **Chapter 2** at column ends framing into the joint, and such distributed moments obtained at the bottom or top end of the column shall be considered in **Equation (3.5)** as  $M_a$  or  $M_{ii}$ , respectively. **Equation (3.6)** shall be applied separately for both senses of earthquake direction and the largest value of  $\sum M_p$  shall be considered in the distribution.

Even if **Equation (3.3)** is satisfied, calculation of  $M_a$  and  $M_{ii}$  of **Equation (3.5)** may be performed conservatively in accordance with below given **3.3.7.3**.

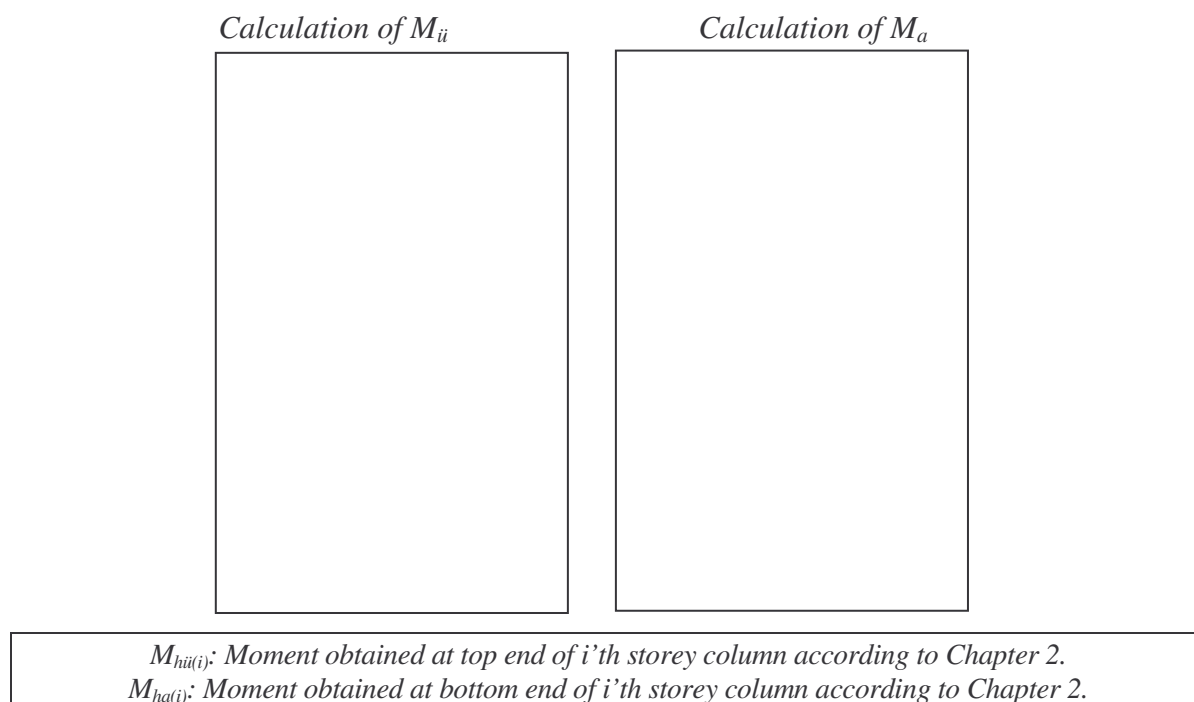
**3.3.7.3** – End moments of columns framing into the joint where **Equation (3.3)** is not satisfied shall be calculated as the ultimate moment capacities and shall be substituted into **Equation (3.5)** as  $M_a$  and / or  $M_{ii}$ . In the case where a more rigorous analysis is not performed, it may be assumed to be  $M_{pa} \cong 1.4 M_{ra}$  and  $M_{pi} \cong 1.4 M_{ri}$ . In the calculation of moments  $M_{pa}$  and  $M_{pi}$ , axial force  $N_d$  shall be taken into account so as to maximize those moments, being consistent with the sense of earthquake direction.

**3.3.7.4** – The moment  $M_a$  at the bottom of a column framing into the foundation shall also be calculated as the ultimate moment capacity in accordance with **3.3.7.3**.

**3.3.7.5** – Shear force,  $V_e$ , obtained according to **Equation (3.5)**, shall not be less than the shear force calculated under factored gravity and seismic loads combined, and in addition it shall satisfy the conditions given by **Equation (3.7)** below. In the case where the condition given by **Equation (3.7b)** is not satisfied, cross section dimensions shall be increased as required and the seismic analysis shall be repeated.

$$\begin{aligned} V_e &\leq V_r \\ V_e &\leq 0.22 A_w f_{cd} \end{aligned} \quad (3.7)$$

**3.3.7.6** – In calculating the column transverse reinforcement for shear force,  $V_e$ , contribution of concrete to the shear strength of the section,  $V_c$ , shall be determined in accordance with TS - 500. However, in the calculation of transverse reinforcement along the column confinement zones specified in **3.3.4.1**, the coefficient of shearing force of concrete shall be taken as  $V_c = 0$  in case shearing force made of only seismic loads is bigger than the half of the total shearing force and at the same time provided the condition  $N_d \leq 0.05 A_c f_{ck}$ .



**Figure 3.5**

### 3.3.8. Conditions Related to Short Columns

Short columns may be developed due to structural arrangements or due to openings provided in infill walls between columns (**Fig. 3.6**). In cases where short columns cannot be avoided, shear force for transverse reinforcement shall be calculated by **Equation (3.5)**. The moments in **Equation (3.5)** shall be calculated at bottom and top ends of the short column as  $M_a \cong 1.4 M_{ra}$  and  $M_{ii} \cong 1.4 M_{ri}$  with  $\ell_n$  being the length of the short column. In addition, calculated shear force shall satisfy the conditions given by **Equation (3.7)**. The minimum transverse reinforcement requirements and conditions of arrangement defined in **3.3.4.1** for column confinement zones shall be applied along the length of the short column. Transverse reinforcement shall be extended along the full storey length of columns which are transformed into short columns in between infill walls (**Fig. 3.6**)



**Figure 3.6**

### 3.4. BEAMS OF HIGH DUCTILITY LEVEL

#### 3.4.1. Cross - section Requirements

**3.4.1.1** – Dimensional requirements of cross - section of beams forming frames together with columns, or of beams connected to structural walls in their own planes are given below:

(a) Width of the beam web shall be at least 250 mm. Web width shall not exceed the sum of the beam height and the width of the supporting column in the perpendicular direction to the beam axis.

(b) Beam height shall not be less than 3 times the thickness of floor slab and 300 mm, nor shall it more than 3.5 times the beam web width.

(c) Beam height should not be more than 1 / 4 the clear span. Otherwise below given **3.4.2.5** shall be applied.

(d) Limitations specified above in relation to beam width and heights are not applicable to reinforced concrete or pre - stressed / prefabricated beams with hinge connections to columns, to coupling beams of coupled structural walls, and to the secondary beams which are connected to frame beams outside the beam-column joints.

**3.4.1.2** - It is essential that design axial force satisfies the condition  $N_d \leq 0.1 A_c f_{ck}$  in order that any structural element be sized and reinforced as a beam. Otherwise such elements shall be sized and reinforced as a column in accordance with **3.3**.

#### 3.4.2. Longitudinal Reinforcement Requirements

**3.4.2.1** – The requirement given by **Equation (3.8)** shall be applied as the minimum ratio of top tension reinforcement at beams supports.

$$\rho \geq 0,8 f_{ctd} / f_{yd} \quad (3.8)$$

**3.4.2.2** – Diameter of longitudinal rebars shall not be less than 12 mm. At least two rebars each at the bottom and top of the beam shall be continuously provided along the full span length of the beam.

**3.4.2.3** – In the first and second seismic zones, bottom reinforcement at a beam support shall not be less than 50 % of the top reinforcement provided at the same support. However this percentage may be decreased to 30 % in the third and fourth seismic zones.

**3.4.2.4** – Ratio of tensile reinforcement along beam spans and at supports shall not be more than the maximum value specified in TS - 500 and 2 %.

**3.4.2.5** – In the special cases where the condition given in paragraph (c) of **3.4.1.1** is not satisfied, web reinforcement shall be provided along the beam height on both sides of the web. Total area of web reinforcement shall not be less than 30 % of the greater of the sum of top and bottom longitudinal reinforcement at right or left support sections. Diameter of web reinforcement shall not be less than 12 mm and spacing shall not be more than 300 mm. For the development of web reinforcement similar to that of longitudinal reinforcement, paragraph (b), and (c) of **3.4.3.1** shall be applied.

### **3.4.3. Arrangement of Longitudinal Reinforcement**

**3.4.3.1** – Requirements for the placement and development of the longitudinal reinforcement are given below (**Fig. 3.7**):

(a) At least 1 / 4 the maximum of the top support reinforcement at the ends of a beam shall be extended continuously along the full span length. The remaining part of the top support reinforcement shall be arranged in accordance with TS - 500.

(b) In cases where beams framing into columns are not extended to the other side of columns, bottom and top beam reinforcement shall be extended up to the face of the other side of the confined core of the column and then shall be bent 90 degrees from inside the hoops. In this case, total length of the horizontal part of the longitudinal rebar inside the column and the 90 degree bent vertical part shall not be less than the straight development length specified in TS - 500. Horizontal part of the 90 degree bent shall not be less than  $0.4\ell_b$  and vertical part shall not be less than 12 Ø. In walls and columns which  $\alpha$  value is more than straight development length  $\ell_b$  and also more than 50 Ø, development of longitudinal reinforcement shall be provided straight without making 90 degree hook.

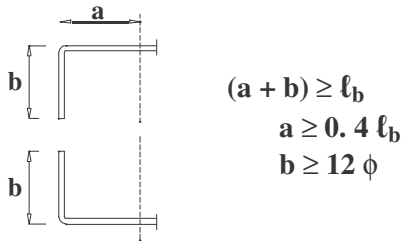
(c) In the case where beams frame into columns from both sides, beam bottom rebars shall be extended to the adjacent span from the column face by at least the development length  $\ell_b$  given in TS - 500. In cases where this is not possible because of reasons such as the depth difference in beams, development shall be achieved in a way similar to paragraph(b) above, i.e., to the case where beam is not extended to the other side of the column.

1 / 4 the max support reinforcement

(See TS - 500 for other rules of arrangement)

Bottom  
reinforcement of  
adjacent span

Bottom  
reinforcement of  
adjacent span



**Figure 3.7**

**3.4.3.2** -Requirements related to splicing of longitudinal reinforcement are given below:

(a) Lap splicing shall not be made along beam confinement zones defined in **3.4.4** below, within beam - column joints, and in regions where reinforcement has a possibility of yielding, such as the mid-span region for bottom reinforcement. In places outside such regions where lap splicing can be made, *special seismic hoops* defined in **3.2.8** shall be used. Spacing of such hoops shall not exceed 1 / 4 of the beam depth and 100 mm. It is not necessary to use special seismic hoops in the span center of the upper assembly rebar.

(b) Mechanical connections or welded lap splices shall only be applied to every alternate bar at a section and longitudinal distance between two consecutive splices shall not be less than 600 mm.

#### **3.4.4. Transverse Reinforcement Requirements**

A region with a length twice the beam depth measured from the column face of a beam support shall be defined as *confinement zone* and *special seismic hoops* defined in **3.2.8** shall be used along this region. In the confinement zone, distance of the first hoop to the column face shall be max. 50 mm. Unless a more unfavorable value is obtained from **3.4.5.3**, hoop spacing shall not exceed 1 / 4 of the beam depth, 8 times the minimum diameter of the longitudinal reinforcement and 150 mm (**Fig.3.8**). Outside the confinement zone, minimum transverse reinforcement requirements specified in TS - 500 shall be applied.

$$\leq 50 \text{ mm}$$

$$s_k$$

|   |  |   |
|---|--|---|
| <i>Beam<br/>Confinement<br/>zone</i><br>$= 2 h_k$ | Beam central zone<br>(Minimum transverse<br>reinforcement according to TS-500) | <i>Beam<br/>Confinement<br/>zone</i><br>$= 2 h_k$ |
|---|--|---|

$$s_k \leq h_k / 4$$

$$s_k \leq 8\phi \text{ } (\phi = \text{min. longitudinal rebar diameter})$$

$$s_k \leq 150 \text{ mm}$$

**Figure 3.8**

### 3.4.5. Shear Safety of Beams

**3.4.5.1** - Shear force,  $V_e$ , to be taken into account for beam transverse reinforcement shall be calculated by **Equation (3.9)** such that the most unfavorable result is obtained by separately considering the cases of earthquake acting from left to right or from right to left (**Fig.3.9**).

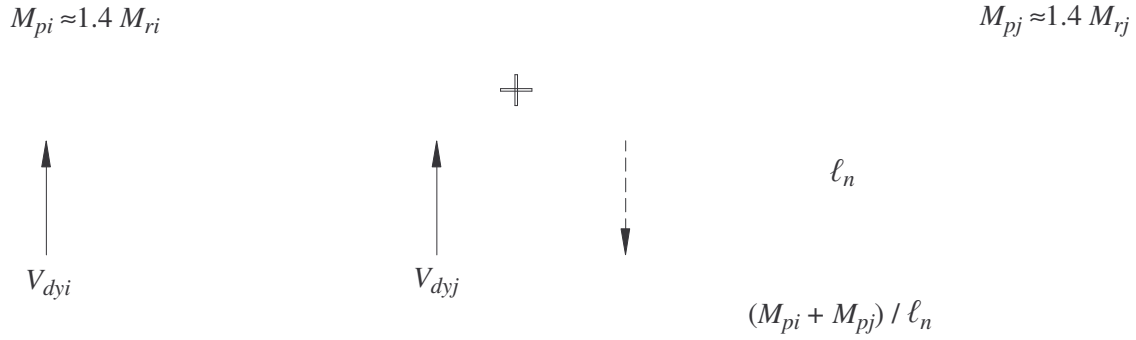
$$V_e = V_{dy} \pm (M_{pi} + M_{pj}) / \ell_n \quad (3.9)$$

Unless a more rigorous analysis is performed, ultimate moment capacities at the beam ends may be taken as  $M_{pi} \cong 1.4 M_{ri}$  and  $M_{pj} \cong 1.4 M_{rj}$ .

**3.4.5.2** – Shear force,  $V_e$ , calculated by **Equation (3.9)** shall satisfy the conditions given below by **Equation (3.10)**. In the case where the condition given by **Equation (3.10b)** is not satisfied, cross-section dimensions shall be increased as required and the seismic analysis shall be repeated.

$$V_e \leq V_r \quad (3.10)$$

$$V_e \leq 0.22 b_w d f_{cd}$$



**Figure 3.9**

**3.4.5.3** - In calculating the beam transverse reinforcement for shear force,  $V_e$ , contribution of concrete to the shear strength of the section,  $V_c$ , shall be determined in accordance with TS - 500. However, in calculating the transverse reinforcement along the beam confinement zones defined in 3.4.4, the coefficient of shearing force of concrete shall be taken as  $V_c = 0$  in case shearing force made of only seismic loads is bigger than the half of the total shearing force. In no case shall the contribution of inclined longitudinal bars to the shear strength be taken into account.

### 3.5. BEAM - COLUMN JOINTS OF FRAME SYSTEMS OF HIGH DUCTILITY LEVEL

#### 3.5.1. Confined and Unconfined Joints

Beam - column joints of frame systems comprised of columns and beams of high ductility level shall be separated into two classes as defined below.

(a) In the case where beams frame into all four sides of a column and where the width of each beam is not less than 3 / 4 the adjoining column width, such a beam-column joint shall be defined as a *confined joint*.

(b) All joints not satisfying the above given conditions shall be defined as *unconfined joint*.

#### 3.5.2. Shear Safety of Beam-Column Joints

**3.5.2.1** – Shear force in beam-column joints along the earthquake direction considered (Fig.3.10) shall be calculated by **Equation (3.11)**.

$$V_e = 1.25 f_{yk} (A_{s1} + A_{s2}) - V_{kol} \quad (3.11)$$

In the case where beam frames into column from only one side and discontinuous on the other side, it shall be  $A_{s2} = 0$ .

**3.5.2.2** – The shear force calculated by **Equation (3.11)** in a joint along the given earthquake direction shall in no case exceed the limits given below (**Fig.3.10**). In the cases where those limits are exceeded, cross-section dimensions of column and/or beam shall be increased and the seismic analysis shall be repeated.

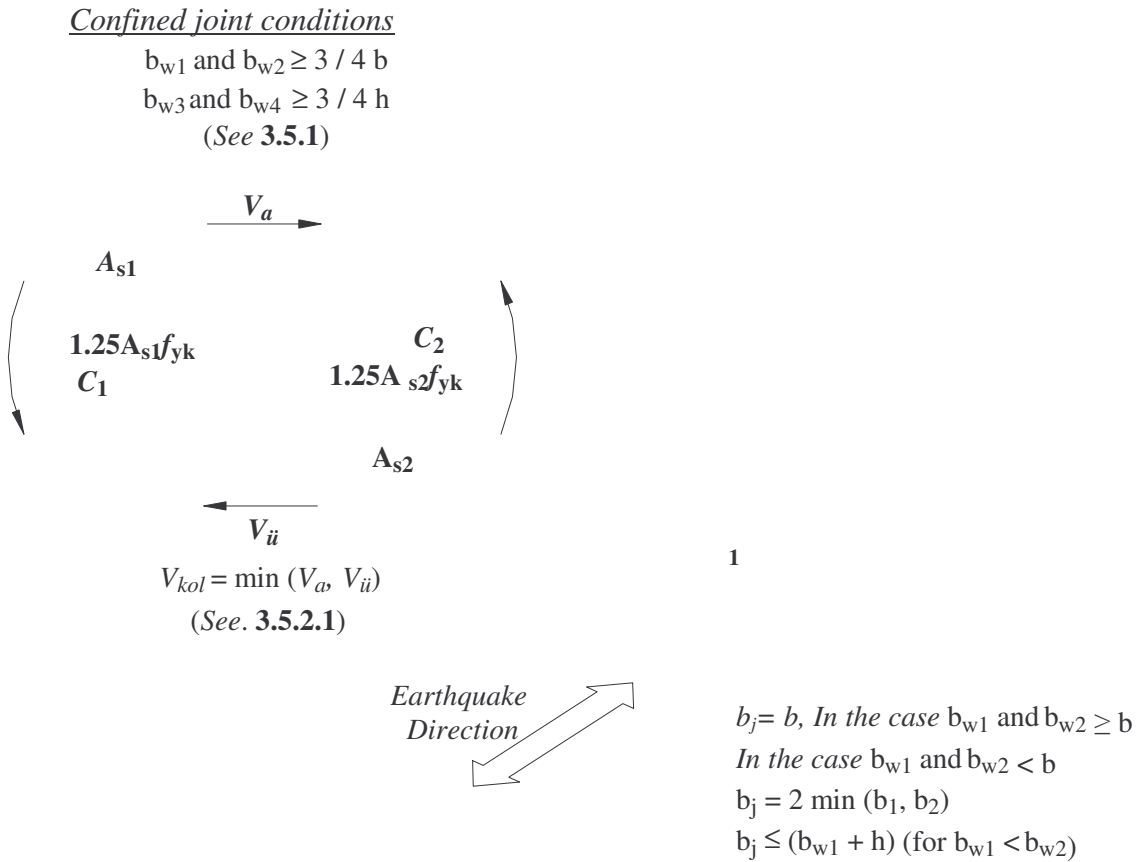
(a) In confined joints:  $V_e \leq 0.60 b_j h f_{cd}$  **(3.12)**

(b) In unconfined joints:  $V_e \leq 0.45 b_j h f_{cd}$  **(3.13)**

**3.5.2.3** – Requirements for minimum transverse reinforcement in beam - column joints are given below (**Fig. 3.3**):

(a) In confined joints, at least 40 % of the amount of transverse reinforcement existing in the confinement zone of the column below shall be provided along the height of the joint. However, diameter of transverse reinforcement shall not be less than 8 mm and its spacing shall not exceed 150 mm.

(b) In unconfined joints, at least 60 % of the amount of transverse reinforcement existing in the confinement zone of the column below shall be provided along the height of the joint. However in this case, diameter of transverse reinforcement shall not be less than 8 mm and its spacing shall not exceed 100 mm.



**Figure 3.10**

### 3.6. STRUCTURAL WALLS OF HIGH DUCTILITY LEVEL

#### 3.6.1. Cross-Section Requirements

**3.6.1.1** – Structural walls are the vertical elements of the structural system where the ratio of length to thickness in plan is equal to at least seven. With the exception of the special case given in **3.6.1.2** and **3.6.1.3** below, wall thickness shall not be less than 1 / 20 the storey height and 200 mm. In those walls, limits of the wall thickness in the end points are given **3.6.2.1** below.

**3.6.1.2** – In buildings where seismic loads are fully carried by structural walls along the full height of building, wall thickness shall not be less than 1 / 20 the highest storey height and 150 mm, provided that both two of the conditions given by **Equation (3.14)** are satisfied.

$$\begin{aligned} \sum A_g / \sum A_p &\geq 0.002 \\ V_t / \sum A_g &\leq 0.5 f_{ctd} \end{aligned} \quad (3.14)$$

**Equation (3.14)** shall be applied at the ground floor level in buildings with stiff peripheral walls in basement stories, whereas it shall be applied at foundation top level for other buildings.

**3.6.1.3** – On the walls situated in lateral direction with the elements that the length is equal to at least to 1 / 5 of the storey length and have storey length bigger than 6 m, wall thickness in the ground may be equal to at least 1 / 20 of horizontal length between the points where its situated in lateral direction. However this thickness should not be less than 300 mm.

#### 3.6.2. Wall End Zones and Critical Wall Height

**3.6.2.1** - *Wall end zones* shall be developed on both ends of walls where  $H_w / \ell_w > 2.0$  (**Fig. 3.11**). With the exemption of buildings specified in **3.6.1.2**, wall thickness shall not be less than 1 / 15 of the storey height and 200 mm. In cases where wall end zones situated to lateral direction with the elements that the length is equal to at least to 1 / 5 of the storey height, wall thickness in the ground may be equal to at least 1 / 20 of horizontal length between the points where its situated in lateral direction. However this thickness should not be less than 1 / 20 of the storey height and 300 mm. Wall end zones may be developed within the wall itself or within an adjoining wall or in an enlarged section at the edge of the wall.

**3.6.2.2** – The *critical wall height* measured from the foundation level shall be determined as to satisfy the unfavorable one of the following conditions given in **Equation (3.15)** provided that it does not exceed  $2\ell_w$ .

$$\begin{aligned} H_{cr} &\geq \ell_w \\ H_{cr} &\geq H_w / 6 \end{aligned} \quad (3.15)$$

Here  $H_w$ , is the wall height measured from level that reduce more than 20 % of length of the wall in plan or from the top of the ground. In buildings where the stiffnesses of reinforced concrete peripheral walls in basement stories are excessive compared to upper stories, and where basement slabs behave as rigid diaphragms in horizontal planes,  $H_w$  and  $H_{cr}$  shall be considered upwards from the ground floor. In such buildings critical wall height shall be extended downwards along the height of first basement storey below the ground floor.

**3.6.2.3** – In structural walls with rectangular cross section, the plan length of each of the end zones along the above defined *critical wall height* shall not be less than 20 % of the total plan length of the wall, nor shall it be two times the wall thickness. The plan length of each of the end zones along the wall section above the *critical wall height* shall not be less than 10 % of the total plan length of the wall, nor shall it be less than the wall thickness (**Fig. 3.11**).

**3.6.2.4** – In the case where wall end zones are arranged within an adjoining walls or at enlarged sections at the edges of the wall, cross section area of each of the wall end zones shall be equal at least to the area defined in **3.6.2.3** for rectangular section walls.

### **3.6.3. Web Reinforcement Requirements**

**3.6.3.1** – Total cross section area of each of the vertical and horizontal web reinforcement on both faces of structural wall shall not be less than 0.0025 of the gross section area of the *wall web* remaining in between the wall end zones. In the case where  $H_w / \ell_w \leq 2.0$ , wall web section shall be considered as the full section of the wall. The spacing of longitudinal and transverse reinforcement in wall web shall not be more than 250 mm (**Fig. 3.11**).

**3.6.3.2** – In buildings where both conditions given by **Equation(3.14)** of **3.6.1.2** above are satisfied, each of the total vertical and horizontal web reinforcement ratios may be decreased to 0.0015. However in such a case, reinforcement spacing shall not exceed 300 mm.

**3.6.3.3** – Excluding the wall end zones, reinforcement mesh on both faces of the wall web shall be tied each other by at least 4 *special seismic crossties* per unit square meter of the wall surface. However, excluding the wall end zones, at least 10 special seismic crossties per unit square meter of the wall surface shall be used along the *critical wall height* defined in **3.6.2.2**. Crosstie diameter shall be at least equal to that of the horizontal reinforcement.

### **3.6.4. Arrangement of Web Reinforcement**

Horizontal web reinforcement may be arranged as explained below in **3.6.4.1** or in **3.6.4.2** (**Fig. 3.11**). Horizontal web reinforcement so arranged may be taken into account in determining the confinement reinforcement to be provided in accordance with below given **3.6.5.2** at wall end zones along the *critical wall height*.

**3.6.4.1** – Horizontal web rebars shall be bent 90 degree at the outer edge of the wall end zone and tied to the vertical corner reinforcement at the other face by a 135 degree hook.

**3.6.4.2** – In the case where horizontal web rebars are terminated at the wall end without 90 degree bent,  $\supset$  shaped horizontal bars with the same diameter of web reinforcement shall be placed at both ends of the wall. Those bars shall be extended inside the wall web by at least the development length measured from the inner boundary of the wall end zone. However, in case development length of the web reinforcement is less than or equal to length of wall end zone,  $\supset$  shaped bars should not be placed. In this case, total area of the transverse in the wall end zones on unit length shall not less than the total area on the unit length of the horizontal reinforcement in the wall web.

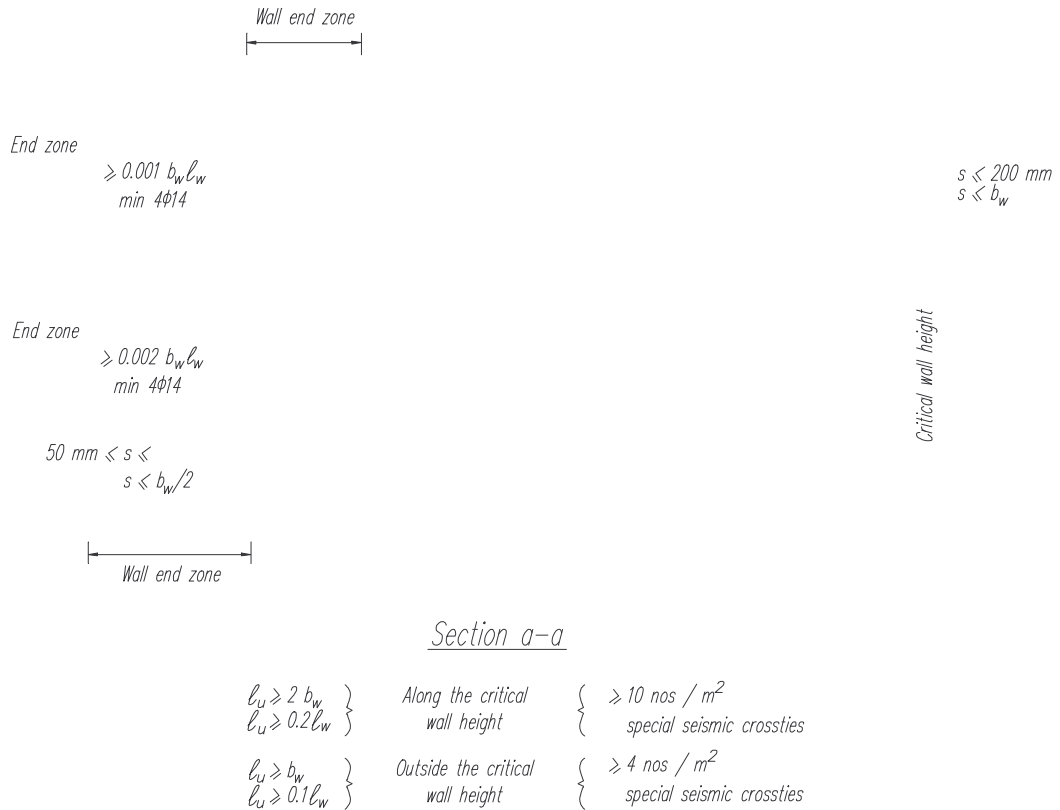
### 3.6.5. Reinforcement Requirements at Wall End Zones

**3.6.5.1** – The ratio of the total area of vertical reinforcement at each wall end zone to the gross wall cross section area shall not be less than 0.001. However this ratio shall be increased to 0.002 along the *critical wall height* defined in 3.6.2.2. Amount of vertical reinforcement at each wall end zone shall not be less than 4Ø14 (**Fig. 3.11**).

**3.6.5.2** – Vertical reinforcement at wall end zones shall be confined as similar to columns, by transverse reinforcement made of hoops and crossties, in accordance with the below given rules.

(a) Diameter of transverse reinforcement to be used at wall end zones shall not be less than 8 mm. Horizontal distance between the legs of hoops and / or crossties,  $a$ , shall not be more than 25 times the diameter of hoops or crossties.

(b) At least 2 / 3 of the transverse reinforcement determined by **Equation(3.1)** in 3.3.4.1 for the confinement zones of columns shall be provided at wall end zones along the *critical wall height* defined in 3.6.2.2. Vertical spacing of hoops and / or crossties shall not be more than half the wall thickness and 100 mm, nor shall it be less than 50 mm (**Fig. 3.11**). Such reinforcement shall be extended into the foundation by at least a height equal to twice the wall thickness.

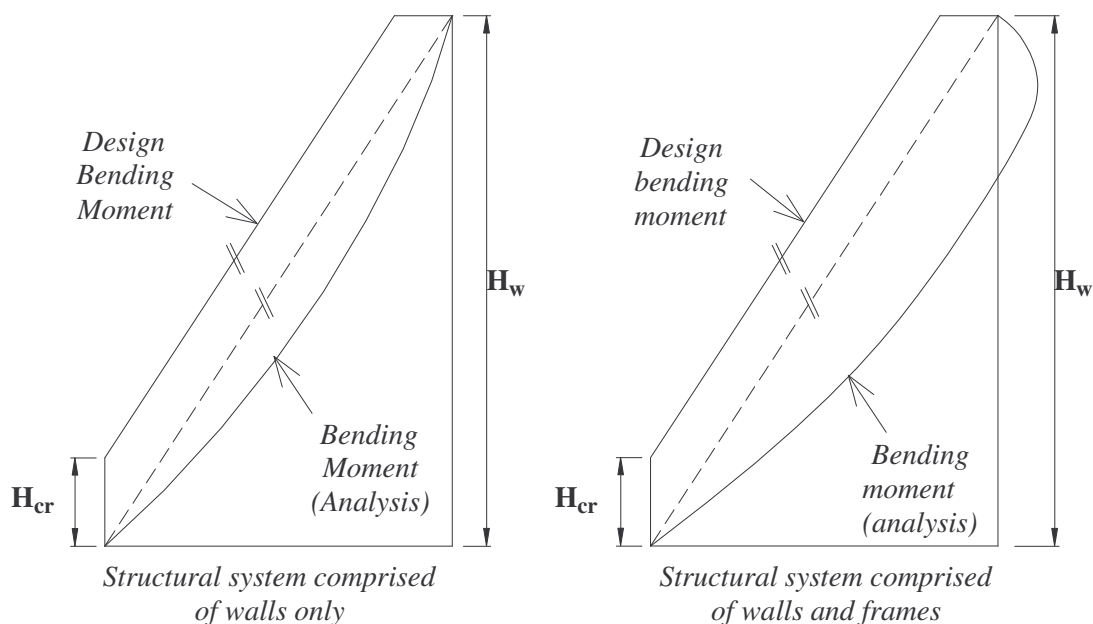


**Figure 3.11**

(c) Vertical spacing of hoops and / or crossties at wall end zones outside the critical wall height shall not be more than the wall thickness and 200 mm (**Fig. 3.11**).

### 3.6.6. Design Bending Moments

**3.6.6.1** - In walls satisfying the condition  $H_w / \ell_w > 2.0$ , design bending moments along the critical wall height determined according to **3.6.2.2** shall be taken as a constant value being equal to the bending moment calculated at the wall base in accordance with **Chapter 2**. Above the critical wall height, a linear bending moment diagram shall be applicable which is parallel to the line connecting the moments calculated at the base and at the top of the wall (**Fig. 3.12**). In buildings with rigid peripheral walls at basements, the constant wall bending moment shall be considered along the critical wall height defined in **3.6.2.2**. In all sections of the walls satisfying  $H_w / \ell_w \leq 2.0$ , design bending moments shall be taken as equal to bending moments calculated according to **Chapter 2**.



**Figure 3.12**

**3.6.6.2** – In the case where  $H_w / \ell_w > 2.0$ , ultimate moment resistances of wall cross sections at each storey shall satisfy in their strong direction the condition given by **Equation(3.3)** for columns. Otherwise, seismic analysis shall be repeated by increasing the wall dimensions and / or reinforcement.

**3.6.6.3** – In walls satisfying the condition  $H_w / \ell_w > 2.0$ , design shear force based on the calculation of any considered transverse reinforcement in section,  $V_e$ , shall be calculated with **Equation (3.16)**.

$$V_e = \beta_v \frac{(M_p)_t}{(M_d)_t} V_d$$

**(3.16)**

Dynamic magnification coefficient of shear force placed in this correlation shall be taken as  $\beta_v = 1.5$ . In the case where a more rigorous analysis is not performed,  $(M_p)_t \cong 1.4 (M_r)_t$  can be taken as the hardening moment capacity in the ground of wall. Design shear forces in all section of the wall satisfying  $H_w / \ell_w \leq 2.0$ , shall be taken as equal to shear forces calculated according to **Chapter 2**.

### 3.6.7. Shear Safety of Structural Walls

**3.6.7.1** – Shear strength of wall cross sections,  $V_r$ , shall be calculated by **Equation (3.17)**.

$$V_r = A_{ch} (0.65 f_{ctd} + \rho_{sh} f_{yd}) \quad (3.17)$$

The shear force  $V_d$  shall satisfy the conditions defined below:

$$\begin{aligned} V_d &\leq V_r \\ V_d &\leq 0.22 A_{ch} f_{cd} \end{aligned} \quad (3.18)$$

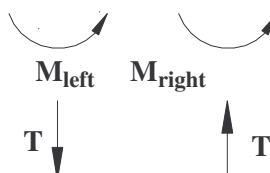
Otherwise transverse reinforcement of the wall and / or wall cross sections shall be increased so as to satisfy the above conditions.

**3.6.7.2** - By considering the shear force transferred to that section, vertical reinforcement at construction joints at the foundation level and at the above stories, shall be controlled with shear friction design defined in TS-500.

### 3.6.8. Rules and Requirements for Coupled Structural Walls

**3.6.8.1** – All rules and requirements given above for structural walls are equally valid for each of the wall segments forming the coupled structural walls.

**3.6.8.2** – Sum of the base moments developed along the given earthquake direction at the wall segments forming a coupled structural wall system shall not be more than 2 / 3 the total overturning moment developed in the coupled structural wall system due to seismic loads (**Fig. 3.13**). In the case where this condition is not satisfied, each of the wall segments forming the coupled structural wall shall be treated as a solid structural wall, and **R** factor taken from **Table 2.5** of **Chapter 2** shall be changed accordingly.



$$(M_a + M_b) \leq 2/3 \sum F_{wi} H_i$$

$F_{wi}$ : Seismic load acting at  $i$ 'th storey on coupled structural wall system

**Figure 3.13**

**3.6.8.3** – In the reinforcement design of wall segments forming the coupled structural wall, it may be allowed to transfer at most 30 % of the moment of the wall segment in tension to the wall segment in compression (*redistribution*).

**3.6.8.4** – Rules related to the shear reinforcement of coupling beams are given below:

(a) In the case where any of the conditions below is satisfied, calculation of shear reinforcement of coupling beams shall be performed in accordance with **3.4.5**.

$$\begin{aligned} \ell_n &> 3 h_k \\ V_d &\leq 1.5 b_w d f_{ctd} \end{aligned} \quad (3.19)$$

(b) In the case where none of the conditions given by **Equation (3.19)** is satisfied, the special shear reinforcement to be provided for the coupling beam shall be determined by methods whose validity are proven by tests, or cross rebars shall be used to resist the shear of the coupling beam (**Fig. 3.14**). Total reinforcement area of each bundle of cross rebars shall be determined by **Equation (3.20)**.

$$A_{sd} = V_d / (2 f_{yd} \sin \gamma) \quad (3.20)$$

There shall be at least four rebars in each bundle of cross rebars and they shall be extended into the wall segments by at least  $1.5 \ell_b$ . Bundles shall be confined with special seismic hoops whose diameter shall not be more than 8 mm and their spacing shall not be more than 8 times the cross rebar diameter and 100 mm. In addition to cross rebars, coupling beam shall be provided with the minimum amount of hoops and horizontal web reinforcement specified in TS-500 (**Fig. 3.14**).



**Figure 3.14**

### 3.7. COLUMNS OF NOMINAL DUCTILITY LEVEL

#### 3.7.1. Cross - section Requirements

Cross - section requirements given in **3.3.1** for columns of high ductility level are equally applicable to columns of nominal ductility level.

#### 3.7.2. Longitudinal Reinforcement Requirements

Longitudinal reinforcement requirements given in **3.3.2** for columns of high ductility level are equally applicable to columns of nominal ductility level.

#### 3.7.3. Arrangement of Longitudinal Reinforcement

Requirements related to the arrangement of longitudinal reinforcement given in **3.3.3** for columns of high ductility level are equally applicable to columns of nominal ductility level.

In the case where reinforcement lap splices are made at the bottom end of the column, minimum transverse reinforcement defined in **3.7.4.1** shall be used along the length of the lap splice.

#### 3.7.4. Transverse Reinforcement Requirements

The minimum transverse reinforcement requirements of columns are specified in **3.7.4.1** for *column confinement zones* and in **3.7.4.2** for the *column central zone*. *Special seismic hoops* and *special seismic crossties* defined in **3.2.8** shall be used along all regions of column.

**3.7.4.1** – The definition given in **3.3.4.1** for the length of each of the confinement zones is equally applicable to columns of nominal ductility level. In columns of nominal ductility level, spacing of transverse reinforcement along the confinement zones shall not be more than  $1/3$  of the cross - section size, 8 times of the diameter of minimum longitudinal reinforcement and 150mm.

**3.7.4.2** – In relation to the column central zone, definition and the minimum transverse reinforcement requirements given in **3.3.4.2**, as well as the conditions given in **3.3.4.3** are equally applicable to columns of nominal ductility level. Transverse reinforcement in column central zone shall be determined in accordance with **3.7.5.3**.

#### 3.7.5. Shear Safety of Columns

**3.7.5.1** – In columns of nominal ductility level, the shear force,  $V_d$ , obtained under the combined effect of gravity loads and seismic loads given in **Chapter 2** shall be taken into account in the determination of transverse reinforcement.

**3.7.5.2** – In relation to the upper bound of shear force, the condition given by **Equation (3.7)** for columns of high ductility level is equally applicable to columns of nominal ductility level, provided that  $V_d$  shall be considered in lieu of  $V_e$ .

**3.7.5.3** – In the determination of transverse reinforcement according to the shear force defined in **3.7.5.1**, the contribution of concrete to the shear strength of the section,  $V_c$ , shall be determined in accordance with TS - 500 by taking into account the minimum axial force,  $N_d$ , calculated under gravity loads combined with seismic loads.

#### 3.7.6. Conditions Related to Short Columns

In relation to short columns, conditions given in **3.3.8** for columns of high ductility level are equally applicable to columns of nominal ductility level.

### 3.8. BEAMS OF NOMINAL DUCTILITY LEVEL

#### 3.8.1. Cross-section Requirements

Cross-section requirements given in **3.4.1.1** for beams of high ductility level are equally applicable to beams of nominal ductility level.

#### 3.8.2. Longitudinal Reinforcement Requirements

Longitudinal reinforcement requirements given in **3.4.2** for beams of high ductility level are equally applicable to beams of nominal ductility level.

#### 3.8.3. Arrangement of Longitudinal Reinforcement

Requirements related to the arrangement of longitudinal reinforcement given in **3.4.3** for columns of high ductility level are equally applicable to columns of nominal ductility level.

#### 3.8.4. Transverse Reinforcement Requirements

A region with a length twice the beam depth measured from the column face of a beam support shall be defined as *confinement zone* and *special seismic hoops* defined in **3.2.8** shall be used along this region. In the confinement zone, distance of the first hoop to the column face shall be max. 50 mm. Unless a more unfavorable value is obtained from the below given **3.8.5**, hoop spacing shall not exceed  $1/3$  of the beam depth, 10 times the minimum diameter of the longitudinal reinforcement and 200 mm. Outside the confinement zone, minimum transverse reinforcement requirements specified in TS - 500 shall be applied.

#### 3.8.5. Shear Safety of Beams

**3.8.5.1** – In beams of nominal ductility level, the shear force,  $V_d$ , obtained under the combined effect of gravity loads and seismic loads given in **Chapter 2** shall be taken into account in the determination of transverse reinforcement.

**3.8.5.2** – In relation to the upper bound of shear force, the condition given by **Equation (3.10)** for beams of high ductility level is equally applicable to beams of nominal ductility level, provided that  $V_d$  shall be considered in lieu of  $V_e$ .

**3.8.5.3** – In the determination of transverse reinforcement according to the shear force defined in **3.8.5.1**, the contribution of concrete to the shear strength of the section,  $V_c$ , shall be determined in accordance with TS - 500. In no case shall the contribution of inclined longitudinal bars to the shear strength be taken into account.

### 3.9. BEAM-COLUMN JOINTS OF FRAME SYSTEMS OF NOMINAL DUCTILITY LEVEL

Rules and conditions given in **3.5** in relation to beam - column joints of frame systems formed by columns and beams of high ductility level are equally applicable to beam - column joints of systems of nominal ductility level with the exception of **3.5.2.1** and **3.5.2.2**.

### 3.10. STRUCTURAL WALLS OF NOMINAL DUCTILITY LEVEL

Structural walls of nominal ductility level shall be dimensioned and reinforced in accordance with the internal forces developed under the combined effect of gravity loads and seismic loads. With the exception of rules and conditions given in **3.6.6**, **3.6.8.2**, **3.6.8.3** as well as the definitions and conditions given in relation to the *critical wall height*, rules and conditions given in **3.6** for structural walls of high ductility level are equally applicable to structural walls of nominal ductility level. However in **3.6.7.1**,  $1.5 V_d$  shall be taken instead of  $V_e$ .

### 3.11. SLABS

**3.11.1** – Slabs shall possess sufficient stiffness and strength to enable the safe distribution of seismic loads acting on storey masses to the vertical elements of the structural system.

**3.11.2** – In all seismic zones, slab thickness of all cast-in-situ or prefabricated, bare or infilled joist floor systems shall not be less than 50 mm. However it is mandatory that *shear connectors* be made between joists and the slab, and their adequacy be proven by calculation to insure the safe transfer of in-plane shear forces developed under gravity loads and seismic loads. Requirements given in TS - 500 for the slab thicknesses of other floor systems are applicable.

**3.11.3** – In relation to the shear strength of slab systems, conditions given in **3.6.7** for the shear strength of structural walls of high ductility level are equally applicable except **3.6.7.1**.

### 3.12. SPECIAL REQUIREMENTS FOR PREFABRICATED BUILDINGS

Prefabricated buildings which are constructed through assembling the structural elements manufactured in factory conditions shall comply with the following special requirements in addition to the other requirements specified in this regulation.

#### 3.12.1. Frames with Hinge Connections

**3.12.1.1** – Hinge connections made of weld, shall possess sufficient strength to resist at least 2 times of the connection strength to be occurred from the earthquake according to **Chapter 2** and other hinged connections shall also possess sufficient strength to resist at least 1.5 of it. In confinement calculations safety tensions shall be increased maximum 15 %.

#### 3.12.2. Moment Resisting Frames

**3.12.2.1** – It shall be proven through analytical methods with appropriate references from the literature or tests that moment resisting connections of prefabricated building frames possess strength and ductility that are equivalent to the monolithic behavior under cyclic and repeated loading due to earthquakes.

**3.12.2.2** – Connections shall possess sufficient strength to transfer moments, shear forces and axial forces to be developed at the ultimate strength level without any reduction in strength and ductility. In welded connections and other type of connections, 1.5 times and 1.2 times the seismic connection forces, respectively, obtained according to **Chapter 2** shall be taken into account.

**3.12.2.3** – Connections must be arranged in sufficient distance from the potential plastic hinges that can develop within the elements connected.

### **3.12.3. Conditions Concerning Pre - Stressing Elements**

With the exception of floor elements and beam type elements with hinge connections to the columns, full pre - stressing shall not be permitted in prefabricated structural elements to be used in seismic zones. Limited pre - stressing application, in addition to pre - stressing steel, can be provided by means of using pre - stressing steel in an amount to be obtained sufficient ductility in elements or stressing of a pre - stressing steel with a low tensile force. Tensile of the pre -stressing steel under the effects of earthquake shall not exceed the value calculated with dividing the elastic limit to safety coefficient of material.

## **3.13. REQUIREMENTS FOR REINFORCED CONCRETE APPLICATION DESIGN DRAWINGS**

### **3.13.1. General Requirements**

**3.13.1.1** – Quality classes of concrete and reinforcing steel to be used in the building shall be indicated on all drawings.

**3.13.1.2** – The *Effective Ground Acceleration Coefficient*, the *Building Importance Factor* considered in the design, the *Local Site Class* selected from **Table 6.2** and the *Structural Behavior Factor* determined from **Table 2.5** shall be indicated on all floor framing plan drawings.

**3.13.1.3** – Hook bending detail of special seismic hoops and special seismic crossties defined in **3.2.8 (Fig. 3.1)** shall be shown on all detail drawings of columns, beams and structural walls.

### **3.13.2. Column and Beam Details**

**3.13.2.1** – Position, diameter and number of vertical rebars within the cross - section shall be shown in detail on column application drawings. Further, horizontal sections shall be taken at each beam - column joint showing in plan the rebars extended upwards from the column below and longitudinal rebars of all beams framing into the column. Hence it shall be clearly shown that column and beam rebars are arranged in such a way that they shall not hinder the proper pouring of concrete into the joint. Ore reinforcement of wall and column from the dig shall be indicated in the drawing with number, diameter and spacing with expansions of transverse reinforcement concerning those.

**3.13.2.2** – Vertical sections shall be taken with vertical rebar detailing clearly shown for each type of column with fully identical longitudinal and transverse reinforcement. Vertical columns sections shall include rebar splicing regions, lap lengths, beam - column joint at the top of the column. In this respect, standard details valid for all beams - columns joints of the building shall not be accepted.

**3.13.2.3** – Lengths of column confinement zones as well as diameters, numbers, spacing and cross-sectional details of transverse reinforcement provided along those regions, along the column central zone and within the top beam - column joint shall be clearly indicated on the drawings, separately for each column type.

**3.13.2.4** – In the application drawings of structural walls; positions, diameters and numbers of vertical bars in the web and wall end zones shall be indicated. In addition, vertical sections shall be taken for each wall type showing the vertical layout of rebars. Critical wall height shall be clearly indicated on the vertical section. Diameters, numbers, spacing and cross - sectional details of transverse reinforcement provided along the critical wall height and other sections of the wall shall be clearly indicated on the drawings.

### **3.13.3. Beam Details**

In beam details, lengths of confinement zones at beam supports as well as diameters, numbers, spacing and cross-sectional details of transverse reinforcement provided along those regions and along the beam central zone shall be clearly indicated on the drawings, separately for each beam.

## CHAPTER 4 - EARTHQUAKE RESISTANT DESIGN REQUIREMENTS FOR STRUCTURAL STEEL BUILDINGS

Dimensioned expressions used in this chapter with the following notations are in *Newton* [N] for forces, *millimeter* [mm] for lengths and *MegaPascal* [MPa] = [N/mm<sup>2</sup>] for stresses.

### 4.0. NOTATION

|           |   |
|-----------|---|
| $A$       | = Area of Cross - Section   |
| $A_k$     | = Area of shearing  |
| $A_n$     | = Useful cross - section area   |
| $b$       | = Width   |
| $b_{cf}$  | = Flange width of column section  |
| $b_{bf}$  | = Flange width of beam section  |
| $D$       | = Outside diameter in circular ring sections  |
| $D_a$     | = Increase coefficient of yield stress  |
| $d_b$     | = Length of cross - section beam  |
| $d_c$     | = Length of cross - section column  |
| $E$       | = Notation for seismic load   |
| $E_s$     | = Elasticity modulus of structural steel  |
| $e$       | = Length of bond beam   |
| $G$       | = Notation for dead load  |
| $H_{avr}$ | = Average storey heights on the up and down of the loop point   |
| $h$       | = Height of web plate   |
| $h_i$     | = Storey height of the $i$ 'th floor of the building  |
| $\ell_b$  | = Distance between the points where the beam is supported in lateral direction  |
| $\ell_n$  | = Distance between the possible plastic joint points in the edges of the beams  |
| $M_d$     | = Bending moment calculated under the combined effects of seismic loads and gravity loads   |
| $M_p$     | = Bending moment capacity calculated at the bottom end of the column  |
| $M_{pa}$  | = Moment calculated at the bottom end of column   |
| $M_{pi}$  | = Positive or negative plastic moment calculated at the left end $i$ of the beam  |
| $M_{pj}$  | = Negative or positive plastic moment calculated at the right end $j$ of the beam   |
| $M_{pn}$  | = Reduced moment capacity   |
| $M_{pü}$  | = Moment calculated at the top end of column  |
| $M_{vi}$  | = Additional bending moment occurred on the surface of the column due to shearing force in the possible plastic joints at the left end $i$ of the beam  |
| $M_{vj}$  | = Additional bending moment occurred on the surface of the column due to shearing force in the possible plastic joints at the right end $j$ of the beam |
| $N_{bp}$  | = Axial pressure capacity   |
| $N_{cp}$  | = Axial tensile capacity  |
| $N_d$     | = Factored axial force calculated under simultaneous action of vertical loads and seismic loads   |
| $Q$       | = Notation for live load  |
| $R$       | = Behavioral coefficient of structural system   |
| $r_y$     | = Radius inertia in the lateral direction of the 1 / 3 of the part that is under the compressive strength of the beam flange and web                    |
| $t$       | = Thickness   |
| $t_{bf}$  | = Flange thickness of the beam section  |
| $t_{cf}$  | = Flange thickness of the column section  |
| $t_{min}$ | = Minimum plate thickness in the sliding zone   |
| $t_p$     | = Including the reinforcing plates, total plate thickness in the sliding zone   |

|                |   |
|----------------|---|
| $t_t$          | = Thickness of the reinforcing plate  |
| $t_w$          | = Web thickness   |
| $u$            | = Length of the periphery of reinforcing plate  |
| $V_d$          | = Shearing force calculated under the combined effect of gravity loads and seismic loads  |
| $V_{dy}$       | = Simple beam shearing force occurred due to gravity loads in the surface of the beam which combines with column  |
| $V_e$          | = Necessary shearing strength of the beam - column confinement zone   |
| $V_{ke}$       | = Necessary shearing strength of the sliding zone   |
| $V_{ik}$       | = Sum of shear forces calculated in the earthquake direction considered in accordance with <b>Chapter 2</b> at all columns of the i'th storey in the peripheries of framed or framed - wall systems   |
| $V_{is}$       | = Sum of shear forces calculated in the earthquake direction considered in accordance with <b>Chapter 2</b> at the i'th storey columns where <b>Equation (4.3)</b> is satisfied at both bottom and top joints in the peripheries of framed or framed - wall systems |
| $V_p$          | = Shearing force capacity   |
| $V_{pn}$       | = Reduced shearing force capacity   |
| $W_p$          | = Plastic moment of resistance  |
| $\alpha_i$     | = Ratio of $V_{is} / V_{ik}$ calculated for any i'th storey   |
| $\Delta_i$     | = Relative storey drift at the i'th floor of the building   |
| $\gamma_p$     | = Turning angle of the bond beam  |
| $\Omega_o$     | = Increase coefficient  |
| $\sigma_a$     | = Yield stress of the structural steel  |
| $\sigma_{bem}$ | = Depending on the slenderness of the element, pressure safety stress calculated according to TS - 648  |
| $\sigma_{em}$  | = Safety stress   |
| $\theta_p$     | = Relative floor drift angle  |

## 4.1. SCOPE

**4.1.1** - Dimensioning of all structural elements of structural steel buildings to be built in seismic zones and design of their joints shall be performed, along with currently enforced relevant standards and codes, primarily in accordance with the requirements of this chapter.

**4.1.2** - Lateral load carrying systems of structural steel buildings covered in this chapter may be comprised of steel frames only, of steel braced frames only or of combination of frames with steel braced frames or reinforced concrete structural walls. Steel carrying systems of armoured concrete floorings that work compositely with the steel beams are also under the scope of this Chapter.

**4.1.3** - Requirements for structural steel building foundations are given in **Chapter 6**.

## 4.2. GENERAL RULES

### 4.2.1. Classification of Steel Structural Systems

Lateral load carrying systems of structural steel buildings shall be classified with respect to their seismic behavior into two classes defined in **4.2.1.1** and **4.2.1.2** below. Special cases and requirements regarding the mixed use of such classes of systems are given in **2.5.4** of **Chapter 2** and also in **4.2.1.3** and **4.2.1.4**. In the case where reinforced concrete structural walls are used as part of the structural system, rules given in **3.6** or **3.10** of **Chapter 3** shall be applied.

**4.2.1.1** - Structural steel systems given below are defined as *Systems of High Ductility Level*:

(a) Frame type structural systems complying with the requirements of **4.3**.

(b) Lateral carrying systems comprised of centric steel braced frames that provides the conditions specified in **4.6** and centric steel braced frames complying with the requirements of **4.8**.

(c) Steel braced wall - frame systems made of combining the two systems defined in paragraph (a) and (b) above.

**4.2.1.2** – Structural steel systems given below are defined as *Systems of Nominal Ductility Level*:

(a) Frame type structural systems complying with the requirements of **4.4**.

(b) Structural systems comprised of concentric steel braced frames complying with the requirements of **4.7**.

(c) Steel braced wall - frame systems made of combining the two systems defined above.

**4.2.1.3** – In case the lateral load carrying systems specified above are differ from each other in both two lateral seismic direction, conditions of R coefficients are given in **2.5.1.2** and **2.5.1.3** and also conditions concerning R coefficients to be applied in mixed use in any direction are given in **2.5.4**.

**4.2.1.4** – Conditions concerning steel or armored concrete - steel combined buildings consist maximum two different lateral carrying system in vertical direction and R coefficients to be applied to those are given in **2.5.5.2**.

#### **4.2.2. Relevant Standards**

**4.2.2.1** - Design of structural steel systems covered in the scope of this chapter shall be performed according to the seismic loads and analysis requirements given in **Chapter 2** of this Specification, to other loads specified in TS - 498, to the requirements of TS - 648 based on allowable stress method. In special cases where the rules in relevant standards are different, rules in this Chapter shall be taken as basis.

**4.2.2.1** - For the matters outside the rules given in this Chapter, rules in TS - 3357 and TS - 648 shall be complied. based on allowable stress method and to those of TS - 4561 based on ultimate strength design method. For the matters outside the scope of those standards and hereby Regulation, then it can be benefited from the internationally accepted standards and regulations.

#### **4.2.3. Material Conditions and Material Safety Factors**

**4.2.3.1** – Within the scope of this Regulation, all structural steels that have weld ability and defined in the other internally accepted standards or in TS - 648 can be used. On the plates that have at least 50 mm thickness on the rolling profiles with 40 mm walled thickness of flanges and artificial profiles made of those plates, minimum value of Charpy - V - Notch (CVN) (*Notch Resistance*) shall be 27 Nm (27 J) in 218 C on the tests carried out in accordance with ASTM A673 or equivalent standards.

**4.2.3.2** – Bolts to be used in the combination and joints of the elements under the effect of seismic load shall be ISO 8.8, 10.9 or high quality. These bolts shall be pre - stressed with all pre - stress strength applicable to them in the joints that transmit moment, whereas on the other joints those shall be pre - stressed with half of it. ISO 4.6 and 5.6 quality bolts can be used in the combinations and joints of the elements that are not under the effect of seismic loads and in the details of foundation connections.

**4.2.3.3** – On the welded joints, electrodes shall be used in accordance with the steel material and welded method and yield strength of the electrode will not be less than the yield strength of the combined materials. On the welded column - beam joints of the frames that transmit moment, full penetration butt weld or butt wells seams shall be used. On the minimum value of electrodes of Charpy - V - Notch (CVN) (*Notch Resistance*) shall be 27 Nm (27 J) in - 298 C in those welds.

**4.2.3.4** – On the elements that are under the effect of seismic load, welded and bolted joints can not be used together in the same joint points.

**4.2.3.5** – On the section calculations made according to *Safety Stress Method* under the combined effect of gravity loads and seismic loads, safety stresses shall be increased utmost 33 %. On the design of combinations and joints based on the safety stresses, this increment will not exceed 15 %. Also combinations and joints shall be controlled according to element capacities or increased seismic effects in such a way that specified in the relevant articles of this Chapter.

**4.2.3.6** – As prescribed in the articles **4.3.2.1**, **4.3.4.1**, **4.8.6** and **4.9.1** of this Chapter, on the calculation of required capacities of structural steel elements and joint details, values of  $D_a \sigma_a$  increased yield stress shall be used instead of the value of yield stress  $\sigma_a$ .  $D_a$  factors to be used in the calculation of increased yield stress are given in the **Table 4.1** depending on the class and element type of the structural steel.

**TABLE 4.1 –  $D_a$  INCREASE COEFFICIENTS**

| Class and Element Type of Structural Steel       | $D_a$ |
|--|-------|
| Rolling profiles made of Fe 37 steel             | 1.2   |
| Rolling profiles made of other structural steels | 1.1   |
| Plates made of all structural steels             | 1.1   |

#### 4.2.4. Increased Seismic Effects

On the required points of the articles **4.3.1.2**, **4.3.5.3**, **4.4.2.1**, **4.4.2.3**, **4.6.3.1**, **4.6.5.2**, **4.7.2.1**, **4.8.6.4** and **4.9.1** of this Chapter, on the design of structural steel elements and combination details, increased seismic effects given below are taken into consideration. Loads that give increased seismic effects are defined as:

$$1.0 G + 1.0 Q \pm \Omega_o E \quad (4.1a)$$

Or in case of unfavorable results

$$0.9 G \pm \Omega_o E \quad (4.1b)$$

Values of *Increase Coefficient*,  $\Omega_o$ , to be applied to internal forces formed of seismic loads calculated according to **Chapter 2**, are given in the **Table 4.2** depending on the types of structural steel systems.

**TABLE 4.2 – INCREASE COEFFICIENTS**

| Type of Structural Systems   | $\Omega_o$ |
|--|------------|
| Frames Of High Ductility Level   | 2.5        |
| Frames Of Normal Ductility Level                                       | 2.0        |
| Centric Steel Braced Frames (Frames of High or Normal Ductility Level) | 2.0        |
| Eccentric Steel Braced Frames  | 2.5        |

#### 4.2.5. Inner Force Capacities and Boundary Values of Tensile

In order to use in necessary cases, tensile boundary values of inner force capacities and boundary values of tensile of the structural elements are defined as follows:

Inner force capacities of structural elements:

$$\text{Bending moment capacity} : M_p = W_p \sigma_a \quad (4.2a)$$

$$\text{Shearing force capacity} : V_p = 0.60 \sigma_a A_k \quad (4.2b)$$

$$\text{Axial pressure capacity} : N_{bp} = 1.7 \sigma_{bem} A \quad (4.2c)$$

$$\text{Axial tensile capacity} : N_{cp} = \sigma_a A_{net} \quad (4.2d)$$

Tensile Boundary Values of Combination Elements:

|                               |                     |
|-------------------------------|---------------------|
| Full penetration weld         | : $\sigma_a$        |
| Partial penetration butt weld |                     |
| Or filled weld                | : $1.7 \sigma_{em}$ |
| Bolted combinations           | : $1.7 \sigma_{em}$ |

Here,  $\sigma_{em}$  indicates safety stresses of relevant combination element (normal stress, sliding and crushing stress).

### 4.3. FRAMES OF HIGH DUCTILITY LEVEL

Rules to be respected in dimensioning of the frames of high ductility level are given below.

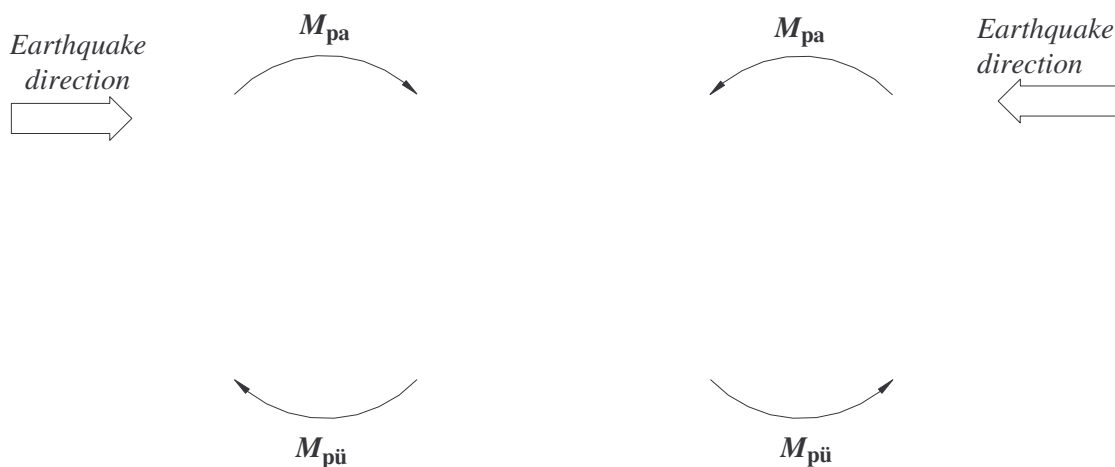
#### 4.3.1. Cross-section Requirements

**4.3.1.1** - Requirements concerning *flange width / thickness* ratio and *web depth / thickness* ratio of the frames of high ductility level in the beams - columns are given in **Table 4.3**.

**4.3.1.2** - Besides providing necessary tensile controls under the axial force and bending moments occur due to the combined effect of gravity loads and seismic, columns shall also have the sufficient resistance capacity under the axial pressure and tensile force (without considering bending moments) occur from the condition of increased load according to **Equation (4.1a)** and **Equation (4.1b)** in the first and second seismic zones. Axial pressure and tensile capacities of column cross - sections shall be calculated with **Equation (4.2c)** and **Equation (4.2d)**.

#### 4.3.2. Requirement of Having Columns Stronger Than Beams

**4.3.2.1** - In frame systems or in the frames of frame - wall (braced frame) systems, sum of the plastic moments of columns framing into a beam - column joint in the earthquake direction shall be bigger than  $1.1D_a$  times of the total of bending moment capacities on the surface of the beams which joint in that loop point (**Fig. 4.1**):



**Figure 4.1**

**TABLE 4.3 – CROSS - SECTION CONDITIONS**

| <i>Description of Elements</i>  | <i>Slenderness Ratios</i> | <i>Limit Values</i>  |  |
|---|---------------------------|--|--|
|   |                           | <i>Systems of High Ductility Level</i>   | <i>Systems of Normal Ductility Level</i>   |
| <b>I</b> Sections<br><b>U</b> Sections<br>Under bending effect  | $b/t$                     | $0.3\sqrt{E_s/\sigma_a}$   | $0.4\sqrt{E_s/\sigma_a}$   |
| <b>I</b> Sections<br><b>U</b> Sections<br>Under bending effect  | $h/t_w$                   | $3.2\sqrt{E_s/\sigma_a}$   | $4.0\sqrt{E_s/\sigma_a}$   |
| <b>T</b> Sections<br><b>L</b> Sections<br>Under pressure effect   | $h/t_w$                   | $0.3\sqrt{E_s/\sigma_a}$   | $0.4\sqrt{E_s/\sigma_a}$   |
| <b>I</b> Sections<br><b>U</b> Sections<br>Under bending and pressure effects  | $h/t_w$                   | For, $ N_d/\sigma_a A  \leq 0.10$<br>$3.2\sqrt{E_s/\sigma_a} \left( 1 - 1.7 \left  \frac{N_d}{\sigma_a A} \right  \right)$ | For, $ N_d/\sigma_a A  \leq 0.10$<br>$4.0\sqrt{E_s/\sigma_a} \left( 1 - 1.7 \left  \frac{N_d}{\sigma_a A} \right  \right)$ |
|   |                           | For, $ N_d/\sigma_a A  > 0.10$<br>$1.33\sqrt{E_s/\sigma_a} \left( 2.1 - \left  \frac{N_d}{\sigma_a A} \right  \right)$     | For, $ N_d/\sigma_a A  > 0.10$<br>$1.66\sqrt{E_s/\sigma_a} \left( 2.1 - \left  \frac{N_d}{\sigma_a A} \right  \right)$     |
| Circular ring sections under bending or axial pressure effect (pipes)   | $D/t$                     | $0.05 \frac{E_s}{\sigma_a}$  | $0.07 \frac{E_s}{\sigma_a}$  |
| Rectangular sections under bending or axial pressure effect   | $b/t$<br>or<br>$h/t_w$    | $0.7\sqrt{E_s/\sigma_a}$   | $1.1\sqrt{E_s/\sigma_a}$   |
| <p><b>Descriptions</b></p> <p><math>b</math> : Width of half - flange in <b>I</b> sections<br/><b>U</b> sections and flange width in rectangular sections</p> <p><math>h</math> : <b>I</b> , <b>U</b> , <b>T</b> sections and web length in rectangular sections<br/>long side length in <b>L</b> sections</p> <p><math>D</math> : Outer diameter in the circular ring sections (in pipes)</p> <p><math>t</math> : <b>I</b> , <b>U</b> , <b>T</b> sections and flange thickness in rectangular sections<br/>thickness in the circular sections (pipes)</p> <p><math>t_w</math> : <b>I</b> , <b>U</b> , <b>T</b> , <b>L</b> sections and web thickness in rectangular sections</p> |                           |  |  |

$$(M_{pa} + M_{pi}) \geq 1.1 D_a (M_{pi} + M_{vj} + M_{pj} + M_{vj}) \quad (4.3)$$

In case of using weakened beam sections or forming gussets at the end of beams, terms of  $M_{vj}$  and  $M_{vj}$  in this equation, indicate the additional bending moments occurred on the surface of the column due to shearing forces in the possible plastic joints at the end zones of beam. In case plastic moments form in the beam's sections of the surface of columns, these terms take zero value.

**4.3.2.2 - Equation (4.3)** shall be applied separately for both senses of earthquake direction to yield the most unfavorable result. In calculating the column plastic moments, axial forces shall be considered to yield the minimum moments capacities with the sense of earthquake direction.

**4.3.2.3 - Equation (4.3)** need not to be checked in single storey buildings and in joints of topmost storey of multi - storey buildings.

#### **4.3.3. The Case Where Some Columns Cannot Satisfy the Requirement of Having Columns Stronger Than Beams**

**4.3.3.1** – In structural systems comprised of frames only or of combination of frames and walls (braced frames), **Equation (4.3)** may be permitted not to be satisfied in a given earthquake direction at some joints at the bottom and / or top of an  $i$ 'th storey, provided that **Equation (4.4)** given below is satisfied.

$$\alpha_i = V_{is} / V_{ik} \geq 0.70 \quad (4.4)$$

**4.3.3.2** – In the case where **Equation (4.4)** is satisfied, bending moments and shear forces of columns satisfying **Equation (4.3)** at both bottom and top joints shall be amplified by multiplying with the ratio  $(1 / \alpha_i)$  within the range of  $0.70 < \alpha_i < 1.00$ . In the case where columns is not satisfied **Equation (4.3)**, it shall be calculated under the effects of gravity load and seismic load forming its sections.

**4.3.3.3** – In the case where **Equation (4.4)** is not satisfied at any storey, all frames of structural systems which may be comprised of frames only or of combination of frames and walls shall be considered as *Frames of Nominal Ductility Level*, and the analysis shall be repeated by changing the Structural Behavior Factor according to **Table 2.5**. As it is mentioned in **2.5.4.1** in **Chapter 2**, it is possible, however, to combine frames of nominal ductility level with structural walls of high ductility level.

#### **4.3.4. Confinement Zones of Beam - Column**

**4.3.4.1** – In the beam - column joints that transfer moment of the frames with high ductility level, the following three conditions shall be covered together:

**a)** Confinement shall have the capacity to be provided at least 0.04 radian *Angle of Relative Storey Drift* (relative storey drift / storey height). Therefore, details that availability is proved with scientific and / or analytical methods shall be used. Several samples of bolted and welded confinement detail and application boundaries of them are given in **Information Annex 4A**.

**b)** Necessary bending strength of the joint on the surface of the column will not be less than  $0.80 \times 1.1D_a$  times of the bending moment capacity on the surface of column of combined beam. However, upper limit of this strength shall conform to the biggest bending moment transferred to confinement by joint columns to loop point. Additionally, it will not exceed the bending moment occurred under the combined effects of seismic loads calculated for the  $R = 1.5$  value of the reduced coefficient of gravity loads and seismic loads. In case of using weakened beam cross - sections or forming gussets at the end zones of the beams, capacity of bending moment on the surface of column shall be calculated by adding beam plastic moment with additional bending moments occurred on the surface of the column due to shearing forces in the possible plastic joints at the end zones of beam.

**c)** Shearing force,  $V_e$ , to be based on dimensioning of the joint shall be calculated with **Equation (4.5)**.

$$V_e = V_{dy} \pm 1.1R_a \frac{(M_{pi} + M_{pj})}{\ell_n} \quad (4.5)$$

**4.3.4.2** – On the calculation of combination's bearing capacity, tensile boundary values given in the **4.2.5** shall be used.

**4.3.4.3** – On the calculation of the bearing capacity of the combination, sliding zone limited by column and beam flanges shall be dimensioned provided the following conditions:

**a)** Necessary shearing strength,  $V_{ke}$ , of the sliding zone shall be taken equal to shearing force occurred 0.80 times of the sum of bending moment capacities of beams on the surface of the column combined at the loop point.

$$V_{ke} = 0.8 \sum M_p \left( \frac{1}{d_b} - \frac{1}{H_{ort}} \right) \quad (4.6)$$



**Figure 4.2**

**b)** Shearing force capacity,  $V_p$ , shall be calculated with the following Equation:

$$V_p = 0.6 \sigma_a d_c t_p \left[ 1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_p} \right] \quad (4.7)$$

In order to sliding zone to have necessary shearing strength the following condition has to be provided:

$$V_p \geq V_{ke} \quad (4.8)$$

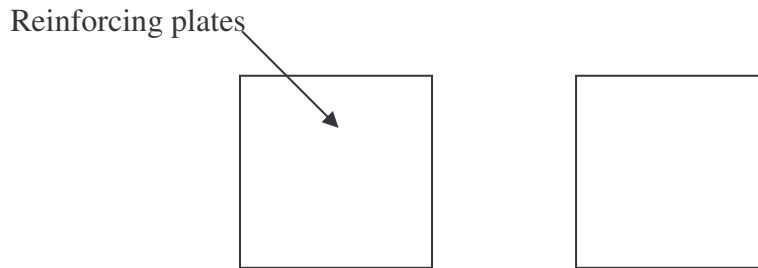
In case this condition is not provided, necessary amount of reinforcing plate shall be used or strutting plates shall be added to sliding zone diagonally.

**c)** Minimum thickness,  $t_{min}$ , of the each web plates of the column and reinforcing plates, if used, shall be provided the below condition (**Figure 4.3**).

$$t_{min} \geq u/180 \quad (4.9)$$

In cases where this condition is not provided, by welding interoperation of reinforcing plates and web plate of the column and it shall be controlled whether the sum of plate thicknesses provide the **Equation (4.9)** or not.

**d)** In cases where reinforcing plates are used in sliding zone, for connecting those plates to cap plates of column, full penetrated butt weld or fillet weld shall be used. These welds shall be controlled in such a way that safely transfer shearing force corresponded by reinforcing plate. On this calculation, welding stress capacities given in (**4.2.5**) shall be used.



**Figure 4.3**

**4.3.4.4** – In details of beam - column joints that transfer moment, by placing ductility plates on both sides of column web in the level of beam flanges, it shall be provided to safely transfer the tensile and pressure strengths on the beam ends to column (and to next beam in the bilateral beam - column joints).

**a)** Thicknesses of ductility plates shall not be less than the cap thickness of the combined beam in the unilateral beam joints where not less than the biggest flange thickness of the combined beams in case there are bilateral beam combinations to column.

**b)** Full penetration butt weld shall be used for the connection of ductility plates to webs and flanges of column. For the connection of the ductility plate to column web, fillet weld can also be used. (**Figure 4.2**). However, this weld should have the thickness and length to

transfer a force equal to shearing force in the plane of ductility plate to column web.

**c)** Ductility plate can not be required if the flange thickness of column provides the following two conditions:

$$t_{cf} \geq 0.54 \sqrt{b_{bf} t_{bf}} \quad (4.10a)$$

and

$$t_{cf} \geq \frac{b_{bf}}{6} \quad (4.10b)$$

### 4.3.5 – Column and Beam Splices

**4.3.5.1** – Column splices made with full penetration butt weld or bolted shall be away from beam - column joint by at least 1 / 3 of the net storey height. Additionally, this distance shall not less than 1.20 m in the splices made of with fillet weld or butt weld without full penetration.

**4.3.5.2** – Beam splices shall be made away from the beam - column connection by a distance at least two times of the beam height.

**4.3.5.3** – Bending capacity of the column - beam splices shall not be less than the bending capacity of the combined element and also shearing force capacity shall not be less than the value given in **Equation (4.5)**. Besides, in the first and second seismic zones, axial force capacity of the column splices shall be also sufficient under the axial pressure and tensile force (without considering bending moments) calculated with **Equation (4.1a)** and **Equation (4.1b)**. On the calculation of bearing capacities of the additional elements, boundary tensile values of weld and bolt given in **(4.2.5)** shall be used.

### 4.3.6 – Supporting the Beam Flanges Laterally

**4.3.6.1** – Up and down flanges of the beams shall be supported laterally.  $\ell_b$  distance between the points where the beams are supported shall provide the following condition.

$$\ell_b \leq 0.086 \frac{r_y E_s}{\sigma_a} \quad (4.11)$$

In addition, points effected by the single loads, points where beam cross - section changes suddenly and points where plastic hinge can occur during the non - linear deformation of the system shall also be supported.

**4.3.6.2** – Necessary pressure and tensile strength of lateral supports shall not be less than 0.02 of the axial tensile capacity of beam flange.

**4.3.6.3** – On the systems where armoured concrete floorings operate compositely with steel beams, it is not necessary to comply above conditions at the flanges of the beams bonded to the armoured concrete flooring.

#### 4.4. FRAMES OF NOMINAL DUCTILITY LEVEL

Rules to be respected on the calculation of frames of nominal ductility level are given below.

##### 4.4.1 – Cross - Section Conditions

**4.4.1.1** - Requirements concerning *flange width / thickness* ratio and *web depth / thickness* ratio of the frames of nominal ductility level in the beams - columns are given in **Table 4.3**. However provided with the control of necessary local buckling, exceeding these boundaries can be permitted in utmost two storey buildings.

**4.4.1.2** - Requirements given above in **4.3.1.2** for the columns of frames of high ductility level are also applicable to frames of nominal ductility level.

**4.4.1.3** - Requirements given above in **4.3.2**, **4.3.3** for frames of high ductility level are not mandatory to be applied for frames of nominal ductility level.

##### 4.4.2 – Confinement Zones of Beam - Column

**4.4.2.1** – Necessary tensile controls shall be carried out under internal forces occurred from the combined effect of gravity loads and seismic loads in the confinements of beam - column of the frames of nominal ductility level. Besides, bearing capacity of the combination shall provide the smallest ones of the internal forces defined below:

**a)** Bending moment capacity of the beam which combined the column calculated in such a way defined in **4.3.4.1 (b)** and necessary shearing force strength calculated with **Equation (4.5)**.

**b)** Bending moment and shear force occurred due to increased load situations given in the **Equation (4.1a)** and **Equation (4.1b)**.

**4.4.2.2** – On the calculation of the confinement's bearing capacity, boundary values of tensile given in **4.2.5** shall be used.

**4.4.2.3** – In the detail of beam column confinement, sliding zone (**Figure 4.2**) which is limited by the flanges of column and beam shall be dimensioned in a way that provides the following conditions.

**a)** On the calculation of necessary shearing force strength of Sliding Zone,  $V_{ke}$ , the shearing force occurred due to increased seismic load given in **Equation (4.1a)** and **Equation (4.1b)** and the smallest one of shearing force calculated with **Equation (4.6)** shall be used.

**b)** Shearing force strength of the sliding zone,  $V_p$ , shall be calculated with **Equation (4.7)**. In order sliding zone to have sufficient shearing strength it is necessary to provided **Equation (4.8)**.

**c)** Rules given in **4.3.4.3 [c]** and **4.3.4.3 (d)** for the calculation of sliding zone of frames of high ductility level are also available for the frames of nominal ductility level.

**4.4.2.4**– Rules given in **4.3.4.4** for the calculation of continuity plates in the frames of high ductility level are also available for the frames of nominal ductility level.

#### 4.4.3 – Column and Beam Joints

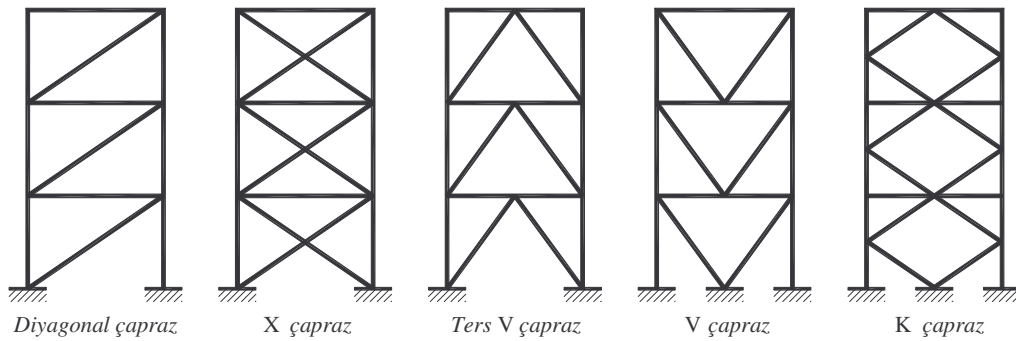
Rules given in **4.3.5** for the joints of column and beam in frames of high ductility level are also available for the frames of nominal ductility level.

#### 4.5 - CENTRIC AND ECCENTRIC STEEL BRACED FRAMES

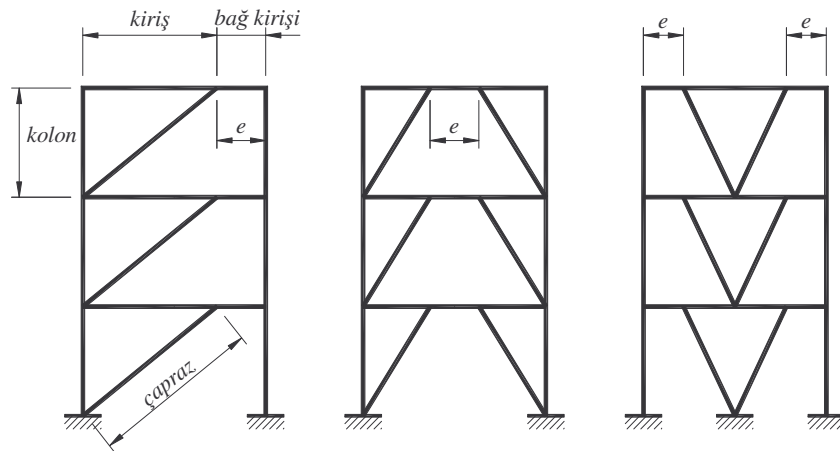
Steel braced frames of high ductility level are defined as lateral load resisting structural elements comprised of hinged joints or frames that transfer moment with braces connected centrically and eccentrically to them. Lateral load bearing capacity of such kind of systems, besides bending strengths, mostly or completely provided with the axial force strengths of the elements. Steel braced frames are divided into two depending on the order of braces:

- a)** Centric Steel Braced Frames (**Figure 4.4**)
- b)** Eccentric Steel Braced Frames (**Figure 4.5**)

Centric Steel Braced Frames in which the braces are connected centrically to loop points of frames can be dimensioned as the systems of high ductility level or the systems of nominal ductility level. On the other hand, Eccentric Steel Braced Frames in which the braces are connected eccentrically to loop points of frames can be dimensioned as the systems of high ductility level.



**Figure 4.4**



**Figure 4.5**

## 4.6. STEEL BRACED FRAMES OF HIGH DUCTILITY LEVEL

Steel braced frames of high ductility level, even in case of buckling of some pressure elements, are dimensioned in a way that avoid significant strength loss in the system. Requirements to be applied in the dimensioning of such systems are as specified below:

### 4.6.1. Conditions of Cross Section

**4.6.1.1** - Requirements concerning *flange width / thickness* ratio and *web depth / thickness* ratio of the frames of high ductility level in the beams - columns and braces are given in **Table 4.3**.

**4.6.1.2** - Slenderness ratio (bar buckling length / radius of inertia) at all pressure elements of roof and vertical plane brace systems shall not be exceed  $4.0\sqrt{E_s / \sigma_a}$

**4.6.1.3** - Spaces of brace plates in multi - partite crosses shall be determined in such a way that slenderness ratio of the single element between two sequential brace plates does not exceed 0.40 times the slenderness ratio of all bar. In case it is shown that buckling of the multi - partite cross does not shearing effect on brace plate, spaces of brace plates shall be determined in such a way that slenderness ratio of the single bar between two brace plates does not exceed 0.75 times the active slenderness ratio of multi - partite bar. Total shearing force capacity of brace plates shall not less than axial the tensile capacity of each bar element. In each bar at least two brace plate shall be used and the plates shall be placed as equal intermittently. Bolt brace plates are not permitted to place on middle three - third of clear space of the bar.

### 4.6.2. Transfer of Horizontal Loads

Vertical centric cross elements on one axis of the building shall be arranged in such a way that at least 30 % and utmost 70 % of horizontal forces affected in the earthquake on the direction of that axis and on each direction of the earthquake cover by crosses which operate when pushed.

### 4.6.3. Joints of Crosses

**4.6.3.1** - On the joint details of crosses, necessary tensile controls shall be made under the internal forces formed of the combined effect of gravity loads and earthquake. Besides, bearing capacity of the joint shall also provide the smallest one of the internal forces define below.

- a)** Axial force (tensile or pressure) capacity of cross
- b)** Depending on the capacities of other elements combined at the loop point, the biggest axial force that can transfer to mentioned cross
- c)** Transverse axial force occurred due to increased load conditions as given in **Equation (4.1.a)** and **Equation (4.1b)**.

**4.6.3.2** - On the calculation of the bearing capacity of joint, boundary values of tensile as given in **4.2.5** shall be used.

**4.6.3.3** - Plates of loop point that tie crosses to columns and / or beams shall also provide the following two conditions:

- a)** Bending capacity in the plane of loop point's plate shall not less than the bending capacity of the cross jointed to loop point.
- b)** In order to prevent buckling of loop point's plate outside the plane, distance of the end of cross to the surface of beam or column shall not more than two times of the thickness of loop plate. In cases where this rule is not followed,

#### **4.6.4 - Additional Conditions for Special Cross Arrangements**

**4.6.4.1** - Additional conditions that have to be provided by **V** - shaped or opposite **V** - shaped cross systems are given below:

- a)** Beams which connect to crosses shall be perpetual.
- b)** Crosses shall be dimensioned under the effect of gravity loads and seismic loads. However, in case the crosses are ignored, beams and end joints which are connected to crosses shall be dimensioned in a way that carries safely the gravity loads on itself.
- c)** Conditions given in **4.3.6** for the beams of frames of high ductility level are also available for the beams which are connected to crosses.

**4.6.4.2 - K** - shaped (where the crosses are connected to mid point of the column) cross order is not permitted for eccentric steel braced frames with high ductility level.

#### **4.6.5 - Joints of Column**

**4.6.5.1** - Column joints shall be made at the center of 1 / 3 zone of column free level.

**4.6.5.2** - Bending strength of column joints shall not less than 50 % of the bending capacity of combined smallest elements whereas shear force strength shall not less than the shearing capacity of the combined smallest elements. Besides, it shall be sufficient under the axial force bearing power of the column joints (without considering bending moments) and tensile and pressure forces formed due to increased seismic loads as given in **Equation (4.1a)** and **Equation (4.1b)** in the first and second seismic zones. On calculation of joint elements, weld and bolt tensile capacities as given in **4.2.5** shall be used.

### **4.7. CENTRIC STEEL BRACED FRAMES WITH NOMINAL DUCTILITY LEVEL**

Rules to be applied on the steel braced frames of nominal ductility level are specified below.

#### **4.7.1. Conditions of Cross - Section**

**4.7.1.1** - Requirements concerning *flange width / thickness* ratio and *web depth / thickness* ratio of the frames of nominal ductility level in the beams - columns are given in **Table 4.3**. However provided with the control of necessary local buckling, exceeding these boundaries can be permitted in utmost two storey buildings.

**4.7.1.2** - Slenderness ratio (bar buckling length / radius of inertia) at all pressure elements of roof and vertical plane brace systems shall not be exceed  $4.0\sqrt{E_s / \sigma_a}$

**4.7.1.3** - Rules concerning brace plates of TS 648 on the multi - partite crosses are applicable. At least two brace plate shall be used in each bar.

**4.7.1.4** - In the case where braces are designed to resist tension only, slenderness ratio of braces shall not exceed 250. However, this rule may not be applied in case the cross elements in utmost two storey buildings are dimensioned in such a way that carries the multiplication of tensile force calculated according to **Chapter 2** and  $\Omega$  coefficient in **Table 4.2**.

#### **4.7.2. Joints of Crosses**

**4.7.2.1** - On the joint details of crosses, necessary tensile controls shall be made under the internal forces formed of the combined effect of gravity loads and earthquake. Besides, bearing capacity of the joint shall also provide the smallest one of the internal forces define below:

- a)** Axial force (tensile or pressure) capacity of cross
- b)** Transverse axial force occurred due to increased load conditions as given in **Equation (4.1.a)** and **Equation (4.1b)**.
- c)** The biggest force to be transferred to the mentioned cross by the other elements combined to loop point.

**4.7.3.2** - On the calculation of the bearing capacity of joint, boundary values of tensile as given in **4.2.5** shall be used.

**4.7.2.3** - Requirements given in **4.6.3.3** for centric steel braced frames of high ductility level are also available for the centric steel braced frames of nominal ductility level.

#### **4.7.3. Additional Conditions for Special Cross Arrangements**

**4.7.3.1** - Requirements given in **4.6.4.1 (a)** and **4.6.4.1 (b)** for centric steel braced frames of high ductility level are also available for the centric steel braced frames of nominal ductility level.

**4.7.3.2** - Conditions given in **4.4.4** for the beams of frames of nominal ductility level are also available for the beams which are connected to crosses.

### **4.8. ECCENTRIC STEEL BRACED FRAMES OF HIGH DUCTILITY LEVEL**

Eccentric steel braced frames of high ductility level are the horizontal structural systems in which brace plates under the seismic effects have the feature of significant non - linear deformation. These systems, during the plastic deformation of brace plates, are dimensioned in a way that provide other beams, except columns, crosses and brace plates, remain in elastic zone. Requirements to be applied in dimensioning of the eccentric steel braced frames of high ductility level are given below.

#### **4.8.1. Conditions of Cross - Section**

**4.8.1.1** - Requirements concerning *flange width / thickness* ratio and *web depth / thickness* ratio of the frames of nominal ductility level in the beams - columns are given in **Table 4.3**. However provided with the control of necessary local buckling, exceeding these boundaries can be permitted in utmost two storey buildings.

**4.8.1.2** - Slenderness ratio (bar buckling length / radius of inertia) at all pressure elements of roof and vertical plane brace systems shall not be exceed  $4.0\sqrt{E_s / \sigma_a}$

**4.8.1.3** - Conditions given in **4.4.4** for the multi - partite crosses are also available for the eccentric steel braced frames.

## **4.8.2. Brace Beams**

**4.8.2.1** - In the eccentric steel braced frames of high ductility level, there shall be one brace beam at least one end of each cross element.

**4.8.2.2** - Length of the brace beam, except the special condition in 4.8.8.1, may be determined as follow.

$$1.0 M_p / V_p \leq e \leq 5.0 M_p / V_p \quad (4.13)$$

$M_p$  bending moment and  $V_p$  shear force capacity in this correlation shall be accounted with **Equation (4.2a)** and **Equation (4.2b)**.

**4.8.2.3** - Brace beams shall be dimensioned under the design internal forces (shear force, bending moment and axial force) formed of seismic effects calculated according to **Chapter 2** and gravity loads.

**4.8.2.4** - Design shear load of brace beam,  $V_d$ , shall provide the both following conditions.

$$V_d \leq V_p \quad (4.14)$$

$$V_d \leq 2M_p / e \quad (4.15)$$

**4.8.2.5** - In case the axial design force of brace beam is,

$$N_d / \sigma_a A > 0.15 \quad (4.16)$$

Following values shall be used in Equation (4.14) and Equation (4.15) instead of  $M_p$ , and  $V_p$

$$M_{pn} = 1.18 M_p \left[ 1 - \frac{N_d}{\sigma_a A} \right]$$

$$V_{pn} = V_p \sqrt{1 - (N_d / \sigma_a A)^2} \quad (4.18)$$

**4.8.2.6** - Web plate of the brace beam shall be single - piece and there won't be any support plate in the web plane. There won't be any spacing on web plate.

## **4.8.3. Lateral Supporting of the Brace Beam**

**4.8.3.1** - Up and bottom flanges of brace beam shall be supported on both ends of the beam whereas in brace beams arranged on the side of column shall be laterally supported on one end of the beam. Necessary strength of lateral supports shall not be less than 0.006 of the axial tensile capacity of beam flange.

**4.8.3.2** - Beside beam section outside the brace beam shall be supported laterally with  $0.45b_{bf}\sqrt{E_s/\sigma_a}$  spaces. Necessary strength of these supports shall not be less than 0.001 of the axial tensile capacity of beam flange.

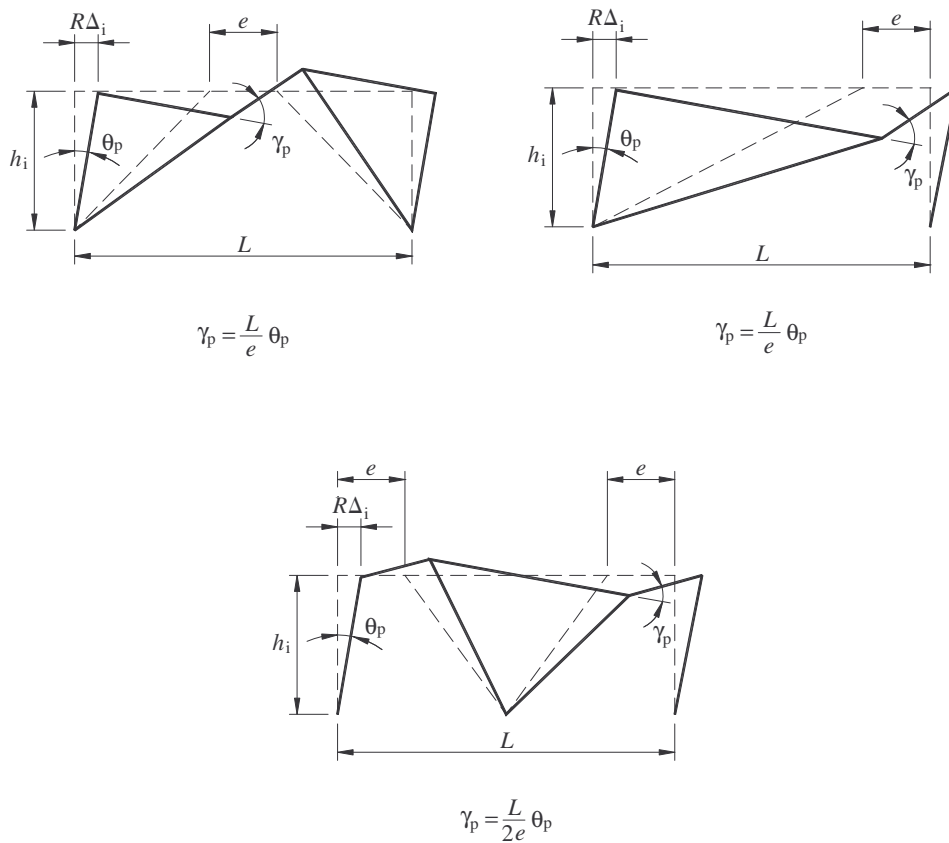
**4.8.3.3** - In the steel structural systems where armoured slabs operate compositely with steel beams it is not necessary to obey the above conditions.

#### 4.8.4. Turning Angle of Brace Beams

Depending on the relative storey drift of i'th storey in which brace beam is located  $\Delta_i$ , defined in **Chapter 2**,

$$\theta_p = R \frac{\Delta_i}{h_i} \quad (4.19)$$

Due to the angle of storey drift determined with above equation,  $\gamma_p$  turning angle of brace beam occurred between brace beam and storey beam of this beam's extension shall not exceed the boundary values as given below (**Figure 4.6**):



**Figure 4.6**

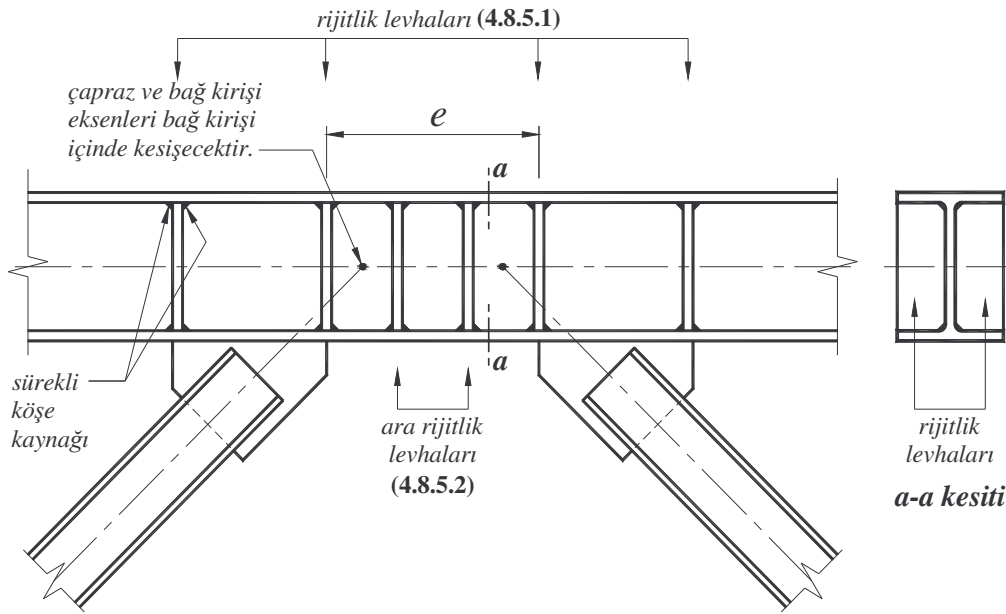
(a) 0.10 radian in case the length of brace beam is equal to or smaller than  $1.6 M_p / V_p$ .

(b) 0.03 radian in case the length of brace beam is equal to or smaller than  $2.6 M_p / V_p$ .

Linear interpolation shall be made in case the length of brace beam is between these two boundary values.

#### 4.8.5. Rigidity (Stiffening) Plates

**4.8.5.1** – *Rigidity plates* shall be placed at the end of cross elements where the cross elements directly transfer load to brace beam and extensions. Rigidity plates unless otherwise stated, shall be placed on both sides of the web plate of brace beam and shall be in the length as web plate and in the width as half flange plate (**Figure 4.7**). Thickness of rigidity plates shall not less than 0.75 of the thickness of web plate and 10 mm. Constant fillet welds that connect rigidity plates to web of the brace beam shall have the capacity to transfer forces formed of the multiplication of cross - section area of rigidity plates and yield stress of material.



**Figure 4.7**

**4.8.5.2** – In addition to the rigidity plates at the end of connection beams inter rigidity plates as defined below shall be placed:

(a) On the brace beams with a length smaller than  $1.6 M_p / V_p$ , inter - spaces of inter rigidity plates shall not be less than  $(30 t_w - d_b / 5)$  incase turning angle of brace beam is 0.10 radian whereas it shall not be less than  $(52 t_w - d_b / 5)$  in case turning angle of brace beam is smaller than 0.03. Linear interpolation shall be made for the inter values of turning angle.

(b) On the brace beams with length taller than  $2.6 M_p / V_p$  and shorter than  $5 M_p / V_p$ , one each rigidity plate shall be placed  $1.5 b_{bf}$  distance from the end of brace beams.

(c) On the brace beams with a length between  $1.6 M_p / V_p$  and  $2.6 M_p / V_p$ , inter rigidity plates as specified in (a) and (b) shall be used together.

(d) On the brace beams with a length taller than  $5 M_p / V_p$ , inter rigidity plates may not be used.

#### 4.8.6. Crosses, Storey Beams and Columns

**4.8.6.1** – Load that causes plasticity of the brace beam shall be determined by means of the multiplication of internal forces formed due to seismic loads and calculated according to **Chapter 2** with the biggest one of the  $M_p$ ,  $M_d$  and  $V_p / V_d$  *Design Increment Factors* calculated as a result of section choose in the brace beam.

**4.8.6.2** – Crosses shall be dimensioned according to internal forces formed of  $1.25D_a$  times of load that causes the plasticity of brace beam.

**4.8.6.3** – Section storey beam located outside the brace beam shall be dimensioned according to internal forces formed of  $1.1D_a$  times of load that causes the plasticity of brace beam.

**4.8.6.4** – Necessary tensile controls shall be made on the columns under internal forces formed of the combined effect of gravity loads and seismic loads. Besides, bearing capacity of the column shall also provide the smallest one of the internal forces define below:

- (a) Internal forces formed of  $1.1D_a$  times of load that causes the plasticity of brace beam.
- (b) Internal forces occurred due to increased load conditions as given in **Equation (4.1.a)** and **Equation (4.1b)**.

**4.8.6.5** – Internal force capacities of the cross - sections of crosses, storey beams and columns shall be calculated with the correlations as given in **Equation (4.2)**.

#### 4.8.7. Joint of Cross and Brace Beam

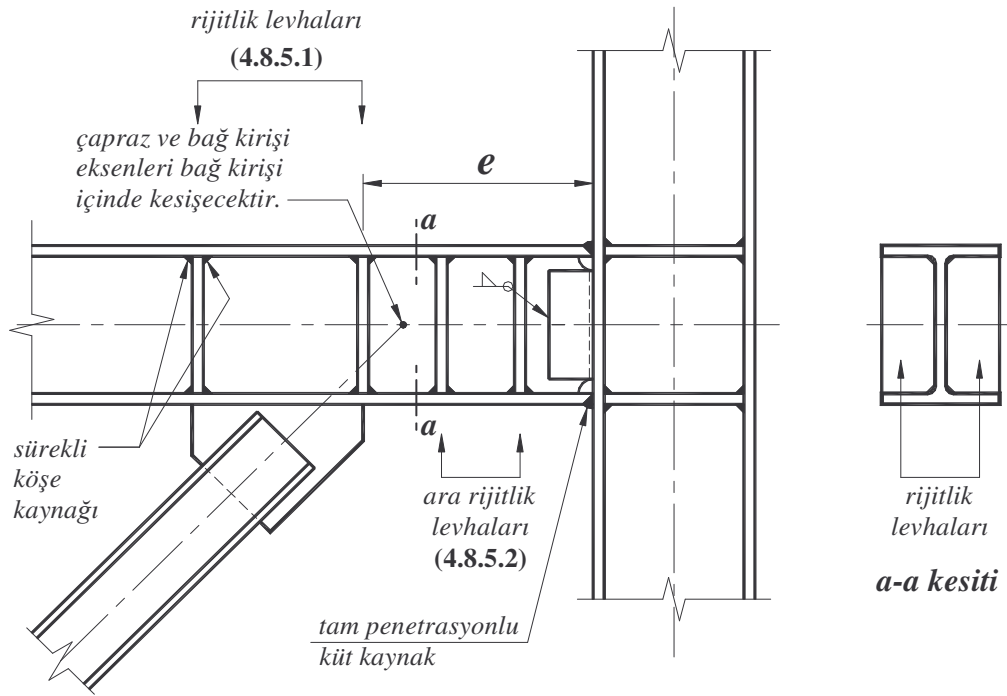
Joint details of crosses with brace beam shall be dimensioned according to internal forces calculated as specified in **4.8.6.2**.

#### 4.8.8. Joint of Brace Beam - Column

**4.8.8.1** – Length of the brace beam that joints to column shall provide the following condition:

$$e \leq 1.6M_p / V_p \quad (4.20)$$

**4.8.8.2** – Necessary bending and shearing strengths of joint on the surface of column shall respectively not be less than the capacity of bending moment of brace beam,  $M_p$ , and shearing force  $V_p$ . Full penetration butt welding shall be applied for the connection of brace beam flanges to column (**Figure 4.8**).



**Figure 4.8**

#### 4.8.9 Beam - Column Joint

Joint details of the part of storey beam with column located outside the brace beam shall be made as hinged inside the beam - web plane. However, this connection shall be dimensioned transversely as equal to 0.01 of the axial tensile capacity of beam flanges and according to buckling moment formed by forces with opposite direction.

#### 4.9. DETAILS OF FOUNDATION CONNECTION

**4.9.1** – On the foundation details of the elements of structural steel system, necessary tensile controls shall be made by predicating the support reactions formed of the combined effect of gravity loads and seismic loads. Besides, bearing capacity of the detail of foundation connection shall also provide smallest ones among the internal forces define below:

- (a) The bending moment formed of  $1.1D_a$  times of the bending moment capacity of column which joints to foundation with total gravity and horizontal forces formed of  $1.1D_a$  times of the axial load capacities of column and crosses which joint to foundation.
- (b) Internal forces occurred due to increased load conditions as given in **Equation (4.1.a)** and **Equation (4.1b)**.

**4.9.2** – Boundary tensile values as given in 4.2.5 shall be used on the calculation of the bearing capacity of connection detail.

## **4.10. RULES CONCERNING THE STATEMENT OF PROJECT AND IMPLEMENTATION PROJECTS**

### **4.10.1. Statement of Project**

**4.10.1.1** – Information specified in **2.13** of **Chapter 2** shall be in the statement of project concerning the statement of earthquake.

**4.10.1.2** – Besides, information listed below shall be in the statement of project:

(a) Material qualities and characteristic strengths of the latten and profile that forms structural system with bolts used in the joints and confinements, type of electrode.

(b) Loads that give the increased seismic effects and load combinations based on design.

**4.10.1.3**– Besides the dimensioning calculations of structural elements and verifications of stability, calculations of combination and joint details with capacity control verifications concerning these details shall be given in detailed in the scope of the project statement.

### **4.10.2. Rules Concerning the Drawings of Steel Implementation Project**

**4.10.2.1** – Following sections shall be in the steel implementation projects:

(a) General construction plans concerning roof slabs and storey slabs

(b) Column implantation (layout) plan

(c) Anchorage plan and details

(d) Front views and sections in sufficient amount

(e) Detailed drawings of columns and beams that composed the structural system with crosses of roof, horizontal plane and vertical plane

(f) Details of all combinations and joints

**4.10.2.2** – Material qualities of profiles and latten used in building with bolt types used in combinations with the type of electrodes to be used shall be specified in all sections.

**4.10.2.3** – *Factor of Effective Ground Acceleration, Building Importance Factor, Local Soil Class* as considered in design and *Load - Bearing System Behavioral Factor* determined according to **Table 2.5** shall be specified in all construction sections.

**4.10.2.4** – In the details of blot combination and joints, type of used bolt, diameters of bolts and drills, features of washer and nuts with pre - stress force to be implied to bolts shall be specified.

**4.10.2.5** – Weld type, thickness of weld and welding length to be implied shall be given in details of welded combinations and joints and also geometrical sizes of welding bent shall be given in butt welds which require to open welding bent.

## INFORMATION ANNEX 4A – BEAM - COLUMN COMBINATION DETAILS IN MOMENT TRANSFERRED FRAMES

### 4A.0. NOTATIONS

$b_{bf}$  = Flange width of beam section  
 $d_b$  = Height of beam cross - section

### 4A.1. SCOPE AND GENERAL MATTERS

**4A.1.1** – As estimated in **4.3.4.1 (a)**, in this Chapter several bolted and welded combination details samples are given which are proved with scientific and / or analytical methods that the samples have the capacity to provide at least 0.04 radian *Relative Storey Drift Angle* (relative storey drift / storey height)<sup>1</sup>.

**4A.1.2** – These details may be used within the limits of implementation concerning moment transferred beam - column combinations of frames of high ductility level.

**4A.1.3** – As for the moment transferred beam - column combinations of frames of nominal ductility level those mentioned details may be used definitively.

**4A.1.4** – Strength calculations and capacity control verifications of combination details shall be carried out in accordance with **4.3.4** and **4.4.2** respectively for the frames of nominal and high ductility level.

### 4A.2. COMBINATION DETAILS OF BEAM - COLUMN

Implementation boundaries that contain combination details of bolted and welded moment transferred beam - column with the usage terms of these details in frames of high ductility level are given below.

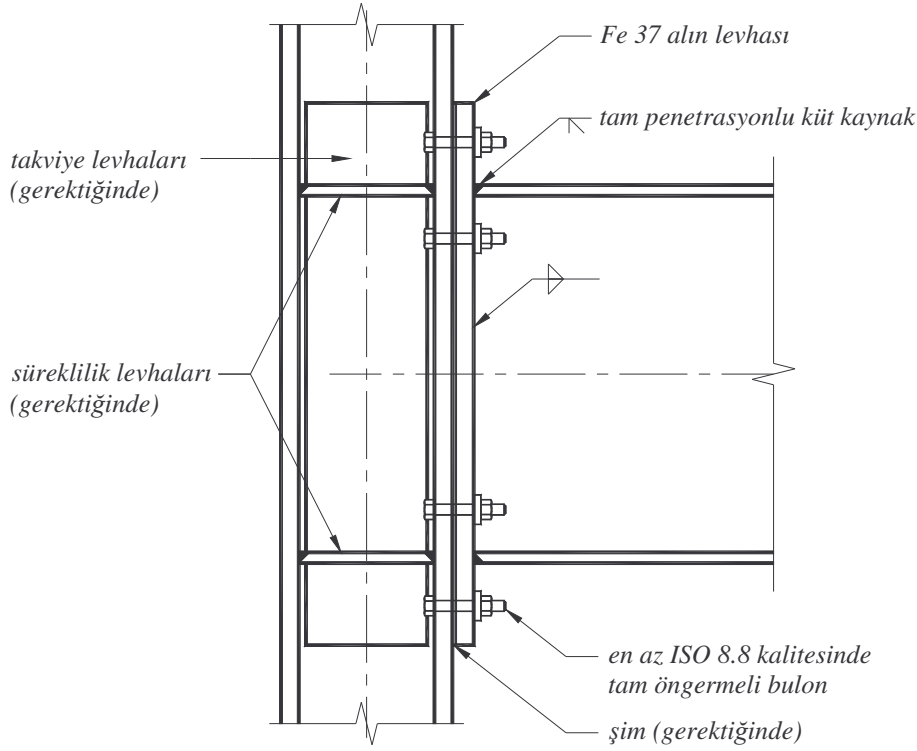
#### 4A.2.1. Combination Details for Bolted Rider Plate

Beam - column combination detail for bolted rider plate is given in **Figure 4A.1**. In the detail, rider plate made of Fe 37 steel combined to flange plates of beam with full penetration butt weld and also combined to web plate with double - sided fillet weld. For the connection of rider plate to column, full pre - stressed bolts with at least ISO 8.8 quality shall be used.

For the implementation of this detail on frames of high ductility level, it is necessary for the parameters of combination detail to provide the implementation boundary given in **Table 4A.1**.

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<sup>1</sup> Federal Emergency Management Agency (FEMA) (2000), Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, FEMA 350, FEMA, Washington, D.C.



**Figure 4A.1**

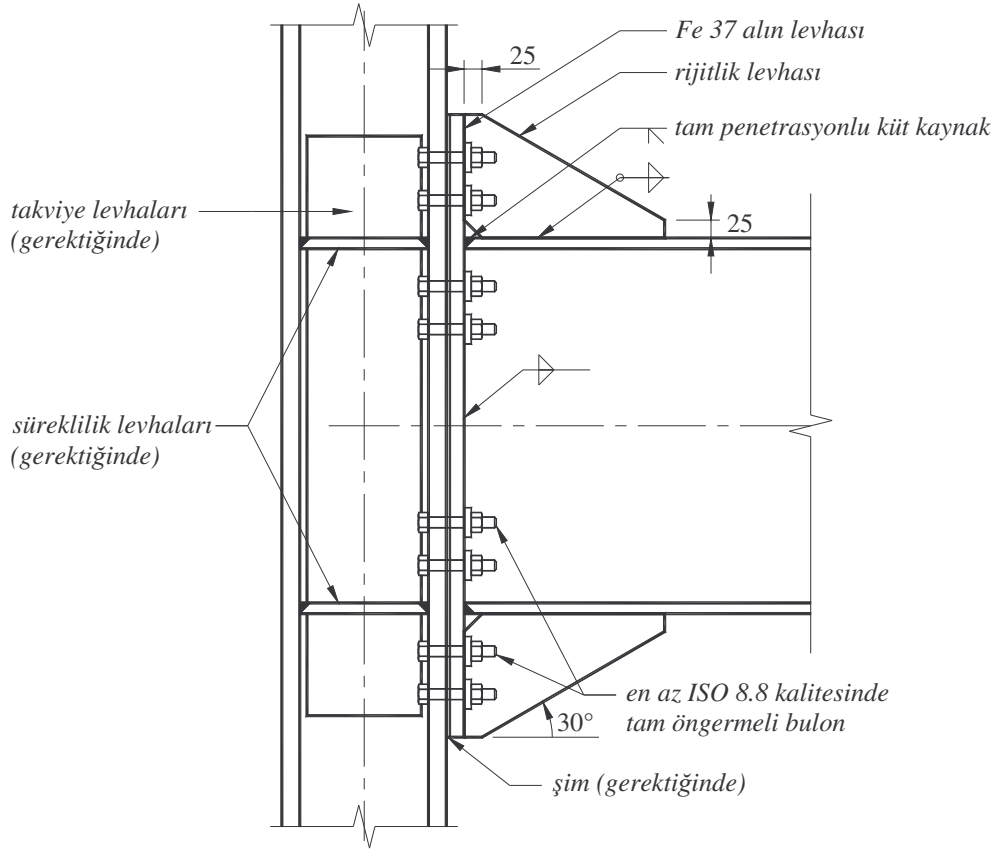
**TABLE 4A.1 – IMPLEMENTATION BOUNDARIES OF THE COMBINATION DETAILSOF BOLTED RIDER PLATE BEAM - COLUMN**

| Parameter of Combination Detail                | Implementation Boundaries  |
|--|----------------------------|
| Cross - section height of beam                 | $\leq 750$ mm              |
| Ratio of beam spacing / cross - section height | $\geq 7$                   |
| Thickness of beam flange                       | $\leq 20$ mm               |
| Cross - section height of column               | $\leq 600$ mm              |
| Class of bolt                                  | 8.8 or 10.9                |
| Conditions of bolted pre - stress              | Full pre - stress          |
| Material class of rider plate                  | Fe 37                      |
| Weld of flange plate                           | Full penetration butt weld |

#### 4A.2.2. Combination Details of Supported Rider Plate Beam - Column

Beam - column combination detail for bolted rider plate supported with rigidity plates is given in **Figure 4A.2**. In the detail, rider plate made of Fe 37 steel combined to flange plates of beam with butt weld and also combined to rigidity plate with double - sided fillet weld. For the connection of rider plate to column, full pre - stressed bolts with at least ISO 8.8 quality shall be used.

For the implementation of this detail on frames of high ductility level, it is necessary for the parameters of combination detail to provide the implementation boundary given in **Table 4A.2**.



**Figure 4A.2**

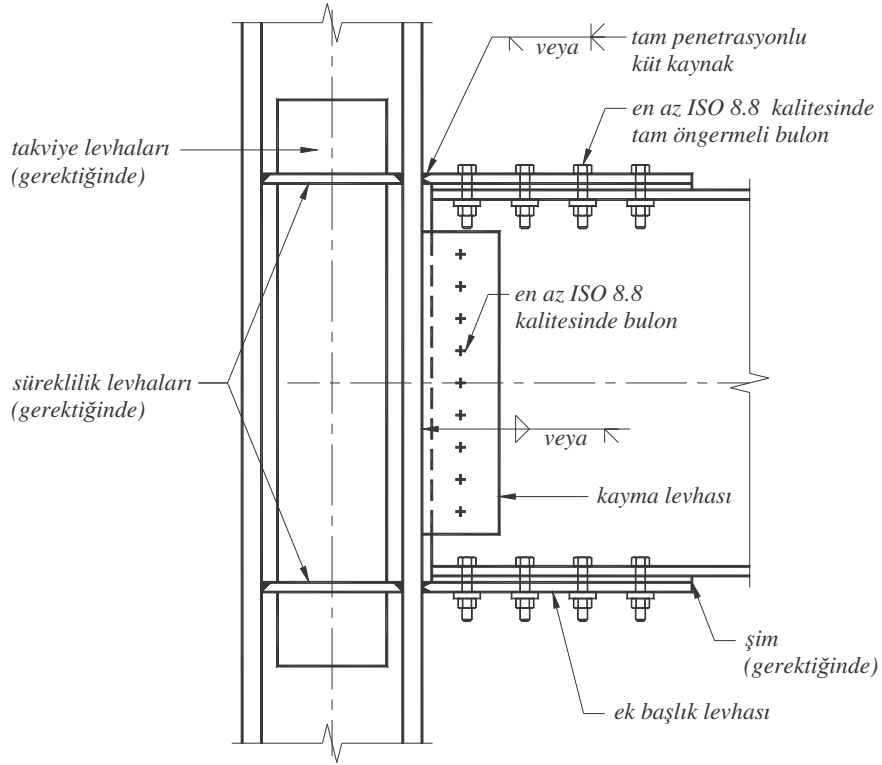
**TABLE 4A.2 – IMPLEMENTATION BOUNDARIES OF THE COMBINATION DETAILSOF BOLTED SUPPORTED RIDER PLATE BEAM - COLUMN**

| Parameter of Combination Detail                | Implementation Boundaries  |
|--|----------------------------|
| Cross - section height of beam                 | $\leq 1000$ mm             |
| Ratio of beam spacing / cross - section height | $\geq 7$                   |
| Thickness of beam flange                       | $\leq 25$ mm               |
| Cross - section height of column               | $\leq 600$ mm              |
| Class of bolt                                  | 8.8 or 10.9                |
| Conditions of bolted pre - stress              | Full pre - stress          |
| Material class of rider plate                  | Fe 37                      |
| Weld of flange plate                           | Full penetration butt weld |

#### 4A.2.3. Combination Details of Bolted Non - Rider Plate

Beam - column combination detail for bolted non- rider plate is given in **Figure 4A.3**. In the detail, connection of beam to column is provided with joint flange plate and sliding plate in web. Joint flange plate combined to column with butt weld and also sliding plate combined with butt weld and fillet weld. For the connection of beam flange and web plates to joint flange plate and sliding plate, bolts with at least ISO 8.8 quality shall be used.

For the implementation of this detail on frames of high ductility level, it is necessary for the parameters of combination detail to provide the implementation boundary given in **Table 4A.3**.



**Figure 4A.3**

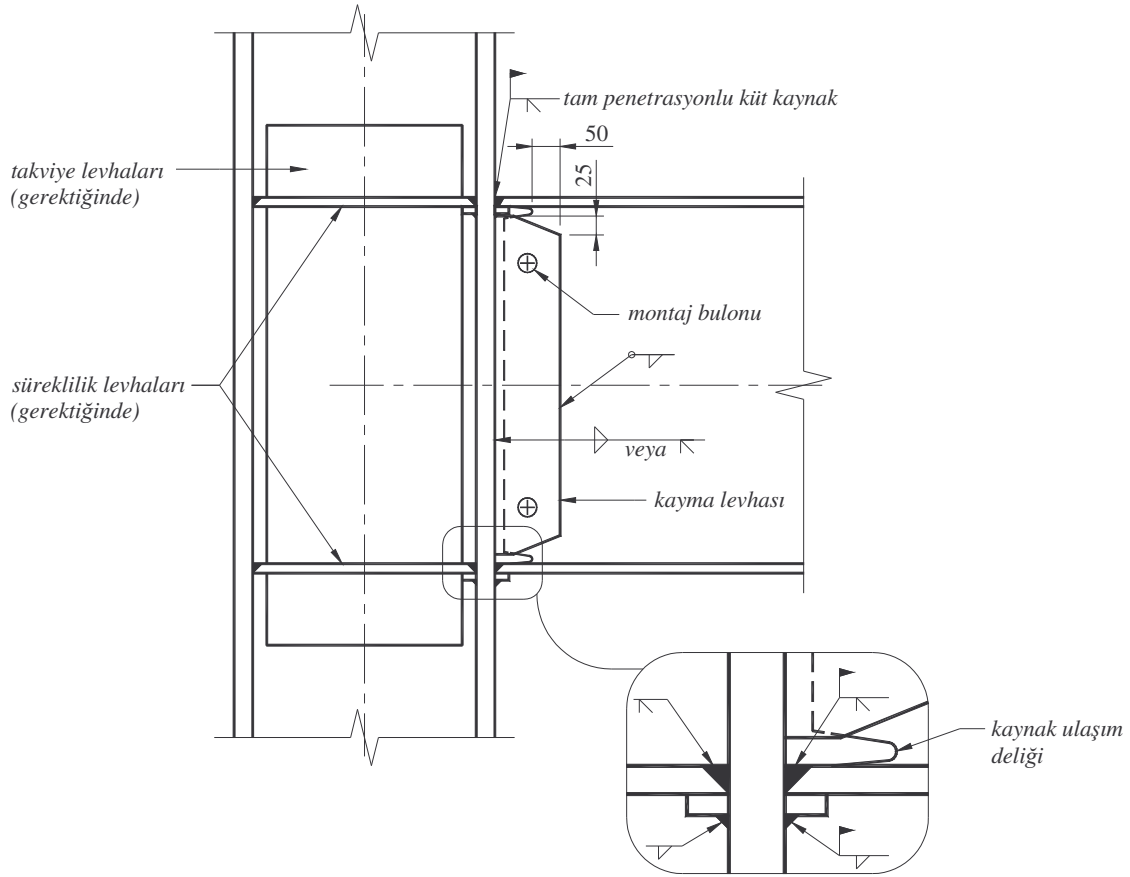
**TABLE 4A.3 – IMPLEMENTATION BOUNDARIES OF THE COMBINATION DETAILSO F BOLTED NON - RIDER PLATE BEAM - COLUMN**

| Parameter of Combination Detail                | Implementation Boundaries  |
|--|----------------------------|
| Cross - section height of beam                 | $\leq 800$ mm              |
| Ratio of beam spacing / cross - section height | $\geq 8$                   |
| Thickness of beam flange                       | $\leq 20$ mm               |
| Cross - section height of column               | $\leq 600$ mm              |
| Class of bolt                                  | 8.8 or 10.9                |
| Biggest bolt size                              | M 30                       |
| Pre - stress Conditions of flange plate bolts  | Full pre - stress          |
| Material class of joint flange plate           | Fe 37, Fe 52               |
| Weld of joint flange plate                     | Full penetration butt weld |

#### 4A.2.4. Welded Combination Detail

Welded combination detail is given in **Figure 4A.4**. In the detail, connection of beam flange plates to column is provided with full penetration butt weld. Web plate of beam combined to column by using sliding plate with butt weld and also fillet weld. As shown in detail, access spaces for weld are necessary for butt welds in beam flanges.

For the implementation of this detail on frames of high ductility level, it is necessary for the parameters of combination detail to provide the implementation boundary given in **Table 4A.4**.



**Figure 4A.4**

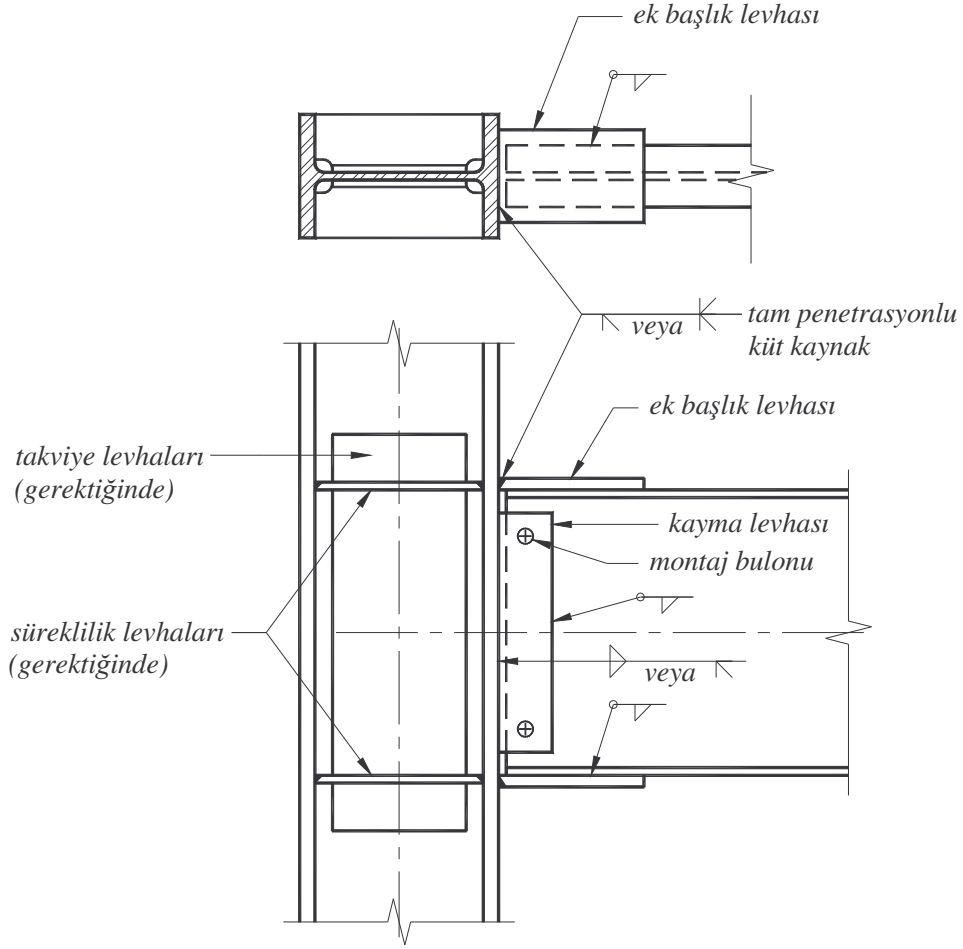
**TABLE 4A.4 – IMPLEMENTATION BOUNDARIES OF THE COMBINATION DETAILSO F WELDED BEAM - COLUMN**

| Parameter of Combination Detail                | Implementation Boundaries  |
|--|----------------------------|
| Cross - section height of beam                 | $\leq 1000$ mm             |
| Ratio of beam spacing / cross - section height | $\geq 7$                   |
| Thickness of beam flange                       | $\leq 25$ mm               |
| Cross - section height of column               | $\leq 600$ mm              |
| Access space for weld                          | Necessary                  |
| Weld of flange plate                           | Full penetration butt weld |

#### 4A.2.5. Combination Detail for Welded Joint Flange Plate

Combination detail for welded joint flange plate is given in **Figure A4.5**. In the detail, combination of joint flange plates to column is provided with full penetration butt weld and combination to beam flange is provided with peripheral fillet weld. Web plate of beam combined to column by using sliding plate with butt weld and also fillet weld. Access spaces for weld are not necessary in this detail.

For the implementation of this detail on frames of high ductility level, it is necessary for the parameters of combination detail to provide the implementation boundary given in **Table 4A.5**.



**Figure 4A.5**

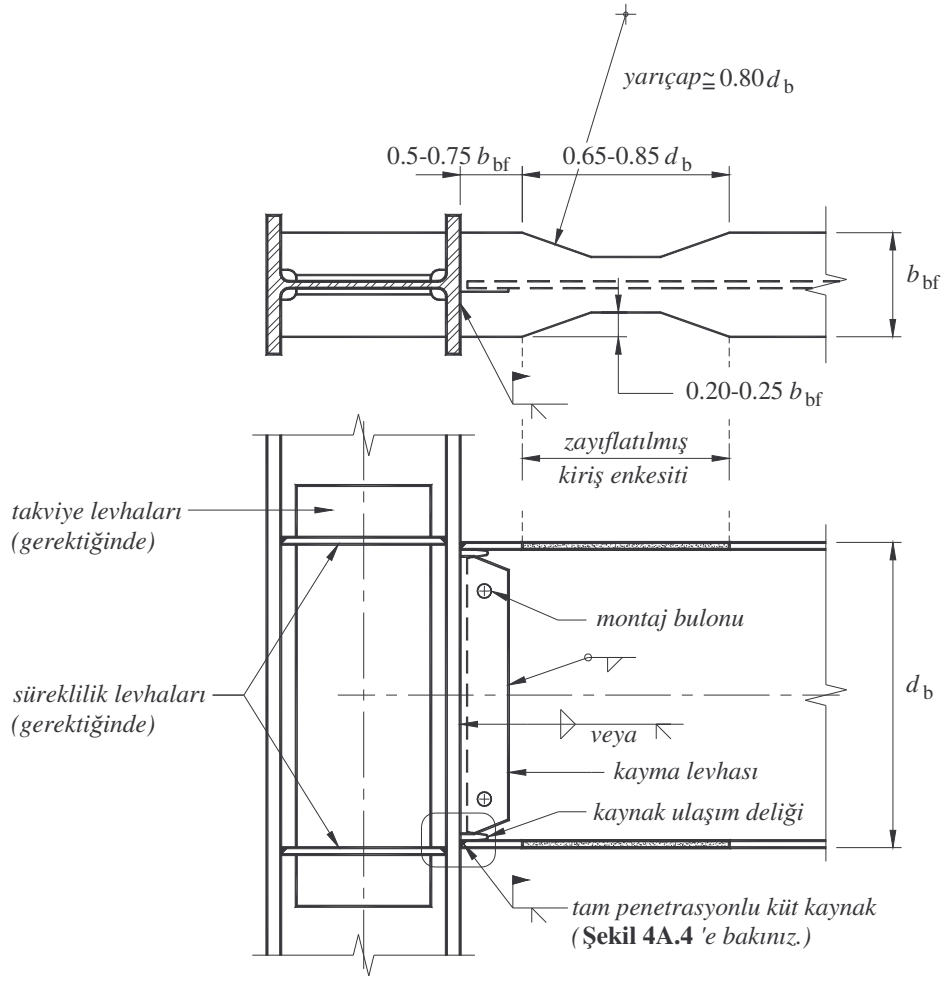
**TABLE 4A.5 – IMPLEMENTATION BOUNDARIES OF THE COMBINATION  
DETAILS OF WELDED BEAM - COLUMN WITH JOINT FLANGE**

| Parameter of Combination Detail                | Implementation Boundaries  |
|--|----------------------------|
| Cross - section height of beam                 | $\leq 1000$ mm             |
| Ratio of beam spacing / cross - section height | $\geq 7$                   |
| Thickness of beam flange                       | $\leq 25$ mm               |
| Cross - section height of column               | $\leq 600$ mm              |
| Material of joint flange plate                 | Fe 52                      |
| Weld of joint flange plate                     | Full penetration butt weld |

#### 4A.2.6. Welded Combination Detail of Weakened Beam Cross - Section

Welded combination detail of weakened beam cross - section is given in **Figure 4A.6**. In the detail which has same features with welded combination detail, additionally weakened cross - section beam is used. Estimated geometrical sizes for weakened beam cross - section are shown in figure.

For the implementation of this detail on frames of high ductility level, it is necessary for the parameters of combination detail to provide the implementation boundary given in **Table 4A.6**.



**Figure 4A.6**

**TABLE 4A.6 – IMPLEMENTATION BOUNDARIES OF THE COMBINATION  
DETAILS OF BEAM - COLUMN WELDED WITH WEAKENED BEAM CROSS -  
SECTION**

| Parameter of Combination Detail                | Implementation Boundaries  |
|--|----------------------------|
| Cross - section height of beam                 | $\leq 1000$ mm             |
| Weight of beam unit length                     | $\leq 450$ kg/m            |
| Ratio of beam spacing / cross - section height | $\geq 7$                   |
| Thickness of beam flange                       | $\leq 45$ mm               |
| Cross - section height of column               | $\leq 600$ mm              |
| Access space for weld                          | Necessary                  |
| Weld of joint flange plate                     | Full penetration butt weld |

## CHAPTER 5 – EARTHQUAKE RESISTANT DESIGN REQUIREMENTS FOR MASONRY BUILDINGS

### 5.1. SCOPE

Dimensioning and reinforcing of masonry buildings and building-like structures to be constructed in seismic zones with load-bearing walls of natural or artificial materials, to resist both vertical and lateral loads shall be performed, along with currently enforced relevant standards and codes, primarily in accordance with the requirements of this chapter. Requirements for masonry building foundations are given in **Chapter 6**.

### 5.2. GENERAL RULES

**5.2.1** – Sliding stress on the walls of building which is generated by seismic loads defined by taking  $S(T_1) = 2.5$  and  $R_a(T_1) = 2.5$  according to **Chapter 2**, shall be calculated and it shall be provided not to exceed permitted limit values.

**5.2.2** – With the exception of the case given in **5.6.2** below, number of stories permitted for masonry buildings is given in **Table 5.1** depending on seismic zones.

**TABLE 10.1 – MAXIMUM NUMBER OF STORIES PERMITTED**

| <i>Seismic Zone</i> | <i>Maximum number of Stories</i> |
|---------------------|----------------------------------|
| 1                   | 2                                |
| 2, 3                | 3                                |
| 4                   | 4                                |

**5.2.3** – Maximum number of stories given in **Table 5.1** correspond to ground storey plus the upper stories. Area of a penthouse built in addition to those stories can not exceed 25 % of gross area of building at foundation level. The penthouse whose storey area is more than 25 % of gross area of the building shall be deemed to be a full storey. In addition, a single basement may be built. In the case where more than a single basement is constructed, maximum number of stories given in **Table 5.1** shall be reduced by one. In all seismic zones, masonry buildings with adobe walls can be constructed utmost single storey without considering basement.

**5.2.4** – Storey height of masonry buildings shall be utmost 3.0 m from one floor top level to the other. In masonry buildings with adobe walls, storey height can not be more than 2.70 m and basement height can not be more than 2.40 m if it exists.

**5.2.5** – Load-bearing walls of masonry buildings shall be arranged in plan, as much as possible, regularly and symmetric or nearly symmetric with respect to the main axes. Construction of partial basement shall be avoided.

**5.2.6** - In plan, load-bearing walls shall be constructed so as to be placed one over the other.

### 5.3. ANALYSIS OF STRESS OF MASONRY WALLS

It shall be indicated that pressure and sliding stresses to be developed under the effect of vertical loads and seismic analysis loads to be calculated by the method given in this section do not exceed pressure and sliding stresses permitted according to type of masonry wall used in walls. If the stresses are exceeded, a new calculation shall be made by increasing structural solid wall areas. Calculation of stress shall not be made for masonry buildings with adobe walls.

**5.3.1.1** – Because shear strength of walls is dependent on vertical stresses existing on the walls, it is required to calculate stress of walls of masonry buildings under vertical loads.

**5.3.1.2** – Comparison of pressure stresses generated on walls with stresses permitted according to type of masonry wall shall be carried out. Loads coming from walls and floorings shall be taken into account in this analysis. The stress to be derived by dividing into cross section of the wall reduced as the cross sections of door and window spaces on the wall shall not be more than the pressure stress permitted according to type of the wall.

### **5.3.2 – Pressure Safety Stress on Walls**

This stress can be calculated by various methods given below:

(a) 0.25 of wall strength calculated by pressure strength tests for wall particles made at equal strength as the pressure strength of masonry unit and mortar to be used in construction of wall is the pressure safety stress.

(b) Wall safety stress can be taken from **Table 5.2** depending on the mortar class used in walls and the average free pressure strength of wall material given in TS – 2510.

(c) If strength test of wall particles is not made, 0.50 of free pressure strength obtained experientially for the block used in wall is pressure strength of wall  $f_d$  and 0.25 of this strength is the pressure safety stress  $f_{em}$  of wall.

(d) If pressure strength of masonry unit used in wall is not given or strength test of wall is not made pressure strength stress for masonry unit used in wall shall be taken from **Table 5.3**.

**5.3.2.1** – Pressure strengths of masonry units and mortar used in wall shall be determined by tests applied in accordance with concerned standards.

**5.3.2.2** – Pressure strength stresses for walls shall be reduced by quantities given in **Table 5.4** according to slenderness rates of walls.

**TABLE 5.2 – PRESSURE SAFETY STRESSES FOR WALLS DEPENDING ON MORTAR CLASS AND FREE PRESSURE STRENGTH OF WALL MATERIAL**

| Average Free Pressure Strength<br>of Wall Material<br>(MPa) | Mortar Class Used in the Wall (MPa) |           |          |          |            |
|---|-------------------------------------|-----------|----------|----------|------------|
|   | A<br>(15)                           | B<br>(11) | C<br>(5) | D<br>(2) | E<br>(0.5) |
| 25  | 1.8                                 | 1.4       | 1.2      | 1.0      | 0.8        |
| 16  | 1.4                                 | 1.2       | 1.0      | 0.8      | 0.7        |
| 11  | 1.0                                 | 0.9       | 0.8      | 0.7      | 0.6        |
| 7   | 0.8                                 | 0.7       | 0.7      | 0.6      | 0.5        |
| 5   | 0.6                                 | 0.5       | 0.5      | 0.4      | 0.4        |

**TABLE 5.3 – PRESSURE SAFETY STRESSES OF WALLS IN WHICH FREE PRESSURE STRENGTH IS UNKNOWN**

| Type of Masonnary Unit and Mortar Used in the Wall  | Pressure Safety Stress of Wall $f_{em}$ (MPa) |
|---|---|
| Vertical hallow block brick (hallow rate is less than 35 %, with lime mortar supported with cement)       | 1.0   |
| Vertical hallow block brick (hallow rate is in between 35 – 45 %, with lime mortar supported with cement) | 0.8   |
| Vertical hallow block brick (hallow rate is more than 45 %, with lime mortar supported with cement)       | 0.5   |
| Filled block brick or clay brick (with lime mortar supported with cement)                                 | 0.8   |
| Stone wall (with lime mortar supported with cement)   | 0.3   |
| Gas concrete (with adhesive)  | 0.6   |
| Filled concrete briquette (with cement mortar)  | 0.8   |

**TABLE 5.4 – REDUCING COEFFICIENTS FOR SAFETY STRESSES ACCORDING TO SLENDERNESS RATE**

| Slenderness rate     | 6   | 8    | 10   | 12   | 14   | 16   | 18   | 20   | 22   | 24   |
|----------------------|-----|------|------|------|------|------|------|------|------|------|
| Reducing coefficient | 1.0 | 0.95 | 0.89 | 0.84 | 0.78 | 0.73 | 0.67 | 0.62 | 0.56 | 0.51 |

### 5.3.3. Calculation of Sliding Stress

Calculation of sliding stresses generated parallel to horizontal joints of walls by seismic design load shall be made as explained in this chapter.

**5.3.3.1** – Relative sliding rigidity of solid wall parts rest among door or window spaces in each axis of masonry building shall be calculated by the expression  $k A / h$ . In here,  $A$  is horizontal cross section of solid wall part and  $h$  is the smallest of heights of spaces at both two sides of solid wall part. If cross section of the wall is rectangular  $k = 1.0$ , if the wall has an end element or there is a grinder or stay wall perpendicular to the wall on the edge of the wall  $k = 1.2$  shall be taken.

**5.3.3.2** – Sliding rigidity of a wall axis is the sum of sliding rigidities of wall parts on this axis. Center of sliding rigidity of building shall be calculated by using sliding rigidity of wall axes.

**5.3.3.3** – Shear force on walls shall be calculated in the direction of both orthogonal axes of the building by considering storey torsion moment as well as storey shear force.

**5.3.3.4** – Sliding stress developed on the wall shall be calculated by dividing seismic force on the wall into horizontal cross section area of the wall and it shall be compared with sliding safety stress of wall  $\tau_{em}$ .

$$\tau_{em} = \tau_o + \mu\sigma \quad (5.1)$$

In this equation  $\tau_{em}$  = sliding safety stress of wall (MPa),  $\tau_o$  = cracking safety stress of wall (MPa),  $\mu$  = coefficient of friction (it can be taken as 0.5) and  $\sigma$  is vertical wall stress (MPa). Cracking safety stress of wall,  $\tau_o$ , shall be taken from **Table 5.5** according to type of masonry unit used in the wall.

**TABLE 5.5 – CRACKING SAFETY STRESS OF WALLS**

| Type of Masonry Unit and Mortar Used in the Wall  | Cracking Safety Stress of Wall $\tau_o$ (MPa ) |
|---|--|
| Vertical hollow block brick (hollow rate is less than 35 %, with lime mortar supported with cement) | 0.25   |
| Vertical hollow block brick (hollow rate is more than 35 %, with lime mortar supported with cement) | 0.12   |
| Solid block brick or clay brick (with lime mortar supported with cement)                            | 0.15   |
| Stone wall (with lime mortar supported with cement)   | 0.10   |
| Gas concrete (with adhesive)  | 0.15   |
| Solid concrete briquette (with cement mortar)   | 0.20   |

#### 5.3.4. Elasticity Module

Elasticity Module ( $E_d$ ) of masonry units used in wall construction shall be calculated by **Equation (5.2)**.

$$E_d = 200 f_d \quad (5.2)$$

### 5.4. LOAD – BEARING WALLS

#### 5.4.1. Materials of Load – Bearing Walls

**5.4.1.1** – Natural stone, solid brick, bricks and block bricks with hole ratios which are not exceeded the maximum void ratios permitted in TS – 2510 and TS EN 771 – 1 as material of load – bearing walls, structural materials and elements of gas concrete, lime sandstone, solid concrete blocks, adobe or similar masonry units may be used as masonry materials in the construction of load – bearing walls in accordance with Turkish Standards.

**5.4.1.2** – Concrete blocks with holes, light aggregated concrete masonry units, bricks and block bricks with hole ratios which are exceeded the maximum void ratios permitted in TS – 2510 and TS – 705 (TS EN 771 – 1) as material of load – bearing walls, other bricks manufactured for infill walls in accordance with TS - 4377 and similarly formed blocks shall never be used as load-bearing wall material.

**5.4.1.3** – Natural stone load-bearing walls shall be used only in the basement and ground stories of masonry buildings.

**5.4.1.4** – Concrete load-bearing walls shall be used only in the basements of masonry buildings.

#### 5.4.2. Strength of Wall Materials

**5.4.2.1** – Strength and other specialties of natural and artificial masonry units used in construction of walls and of mortars connecting those shall be as follows. These conditions are not valid for adobe. Adobe can only be used in adobe buildings.

**5.4.4.2** – According to gross pressure area, minimum pressure strength of natural and artificial masonry units to be used in load – bearing walls shall be 5.0 MPa at least. Pressure strength of natural stones to be used in basement stories shall be 10.0 MPa at least. In the case where concrete walls are constructed in basements, minimum quality of concrete to be used shall be C16.

**5.4.2.3** – Lime mortar supported with cement (cement / lime / sand volumetric ratio = 1 / 2 / 9) or cement mortar (cement/sand volumetric ratio = 1 / 4) shall be used in load – bearing walls or cement mortar.

**5.4.2.4** – Pressure safety stress of walls ( $f_{em}$ ) shall be calculated by using one of the methods given in **5.3.2**.

**5.4.2.5** – Sliding safety stress of walls shall be calculated according to **Equation (5.1)**.

### **5.4.3. Minimum Thickness of Load-Bearing Walls**

The minimum thicknesses of load-bearing walls, excluding plaster thicknesses, are given in **Table 5.6** depending on the number of stories in masonry building. In the case of no basement, minimum wall thicknesses given in the **Table 5.6** shall be valid for ground storey and upper stories. In penthouses permitted in accordance with **5.2.3**, wall thickness specified for the storey below shall be applied.

**TABLE 5.6 – MINIMUM THICKNESSES OF LOAD – BEARING WALLS**

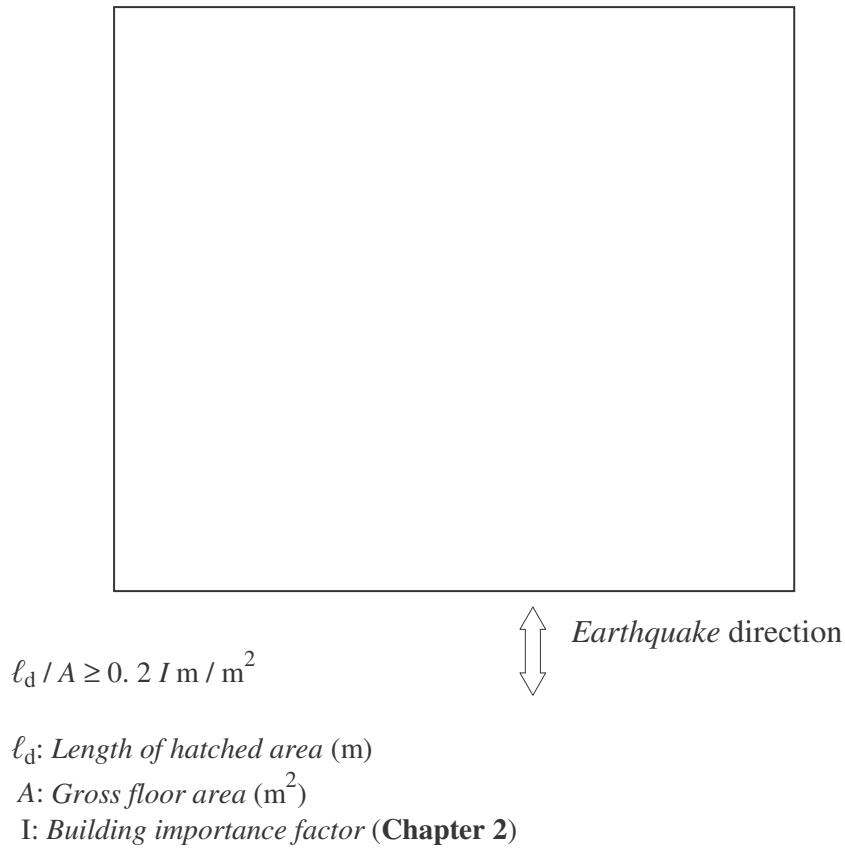
| Seismic Zone         | Permitted Stories | Natural Stone (mm) | Concrete (mm) | Brick and Gas concrete | Others (mm) |
|----------------------|-------------------|--------------------|---------------|------------------------|-------------|
| <b>1, 2, 3 and 4</b> | Basement storey   | 500                | 250           | 1                      | 200         |
|                      | Ground storey     | 500                | -             | 1                      | 200         |
| <b>1, 2, 3 and 4</b> | Basement storey   | 500                | 250           | 1.5                    | 300         |
|                      | Ground storey     | 500                | -             | 1                      | 200         |
|                      | First storey      | -                  | -             | 1                      | 200         |
| <b>2, 3 and 4</b>    | Basement storey   | 500                | 250           | 1.5                    | 300         |
|                      | Ground storey     | 500                | -             | 1.5                    | 300         |
|                      | First storey      | -                  | -             | 1                      | 200         |
|                      | Second storey     | -                  | -             | 1                      | 200         |
| <b>4</b>             | Basement storey   | 500                | 250           | 1.5                    | 300         |
|                      | Ground storey     | 500                | -             | 1.5                    | 300         |
|                      | First storey      | -                  | -             | 1.5                    | 300         |
|                      | Second storey     | -                  | -             | 1                      | 200         |
|                      | Third storey      | -                  | -             | 1                      | 200         |

**5.4.3.1** – In buildings with adobe walls, external load – bearing walls shall be at least 1.5 brick size, whereas internal adobe load-bearing walls shall be at least 1 brick size. Nominal adobe brick dimensions to be used in load – bearing walls shall be 120 x 300 x 400 (main) and 120 x 190 x 400 (lamb) or 120 x 250 x 300 (main) and 120 x 180 x 300 (lamb) in mm's.

#### 5.4.4. Total Length Limit for Load – Bearing Walls

The ratio of the total length of masonry load-bearing walls in each of the orthogonal directions in plan (excluding window and door openings), to gross floor area (excluding

cantilever floors) shall not be less than  $(0.2 I) \text{ m/m}^2$  where  $I$ , represents Building Importance Factor defined in **Chapter 2**.

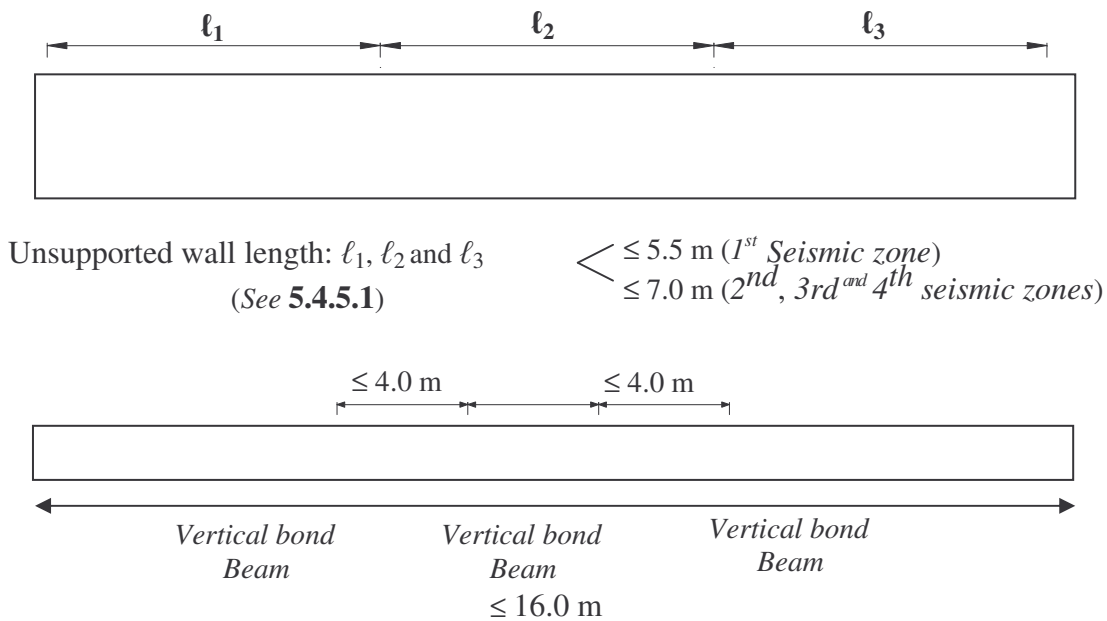


**Figure 5.1**

#### 5.4.5. Maximum Unsupported Length of Load-Bearing Walls

**5.4.5.1** – Unsupported length of any load-bearing wall between the load – bearing wall axes in the perpendicular direction in plan shall not exceed 5.5 m. in the first seismic zone and 7.5 m in other seismic zones. Maximum unsupported length of wall in masonry buildings with adobe walls shall be 4.5 m.

**5.4.5.2** – In the case the condition given in **5.4.5.1** above is not satisfied, reinforced concrete vertical bond beams shall be constructed along the full storey height at the corners of building and in walls with axis to axis spacing in plan not more than 4.0 m. However unsupported length of such walls shall not be more than 16 m (**Figure 5.2**).



**Figure 5.2**

#### 5.4.6. Openings in Load-Bearing Walls

The following rules shall be followed in openings to be provided in load – bearing walls (**Figure 5.3**):

**5.4.6.1** – Plan length of the solid wall segment to be set between the corner of a building and the nearest window or door opening to the corner shall not be less than 1.50 m in the first and second seismic zones and 1.0 m in the third and fourth seismic zones. This quantity shall be at least 1.0 m in buildings with adobe walls in all seismic zones.

**5.4.6.2** – Excluding the corners of buildings, plan lengths of the solid wall segments between the window and door openings shall not be less than 1.0 m in the first and second seismic zones and 0.8 m in the third and fourth seismic zones. This quantity shall be at least 1.0 m in buildings with adobe walls in all seismic zones.

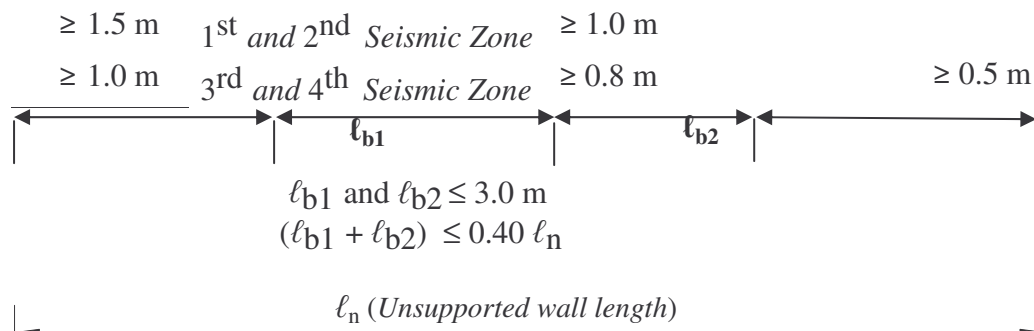
**5.4.6.3** – In the case where reinforced concrete vertical bond beams according to 5.5.3 are made on both sides of the window and door openings, condition of minimum lengths of wall segments given in 5.4.6.1 and 5.4.6.2 may be decreased by 20%. If two timber pillars with section of 0.10 m x 0.10 m are set on both two sides of the window and door openings, solid wall segment between two openings may be 0.80 m in buildings with adobe walls. These timber pillars shall be connected to the timber bond beams of window and door.

**5.4.6.4** – Excluding the corners of buildings, plan length of a solid wall segment between intersection of the walls and the nearest window or door opening to the intersection of the orthogonal walls shall not be less than 0.50 m in the all seismic zones. In case where reinforced concrete vertical bond beams according to 5.5.3 are exist on both sides of the openings along the height of the storey solid wall segment may be less than 0.50 m

**5.4.6.5** – Plan length of each window or door opening shall not be more than 3.0 m. In buildings with adobe walls, door openings shall not be more than 1.0 m in horizontal axis and 1.90 m in vertical axis; window openings shall not be more than 0.90 m in horizontal axis and 1.20 m in vertical axis.

**5.4.6.6** – Total plan lengths of window or door openings along the unsupported length of any wall defined in **5.4.5** shall not be more than 40% of the unsupported wall length.

**5.4.6.7** – In the case where reinforced concrete vertical bond beams according to **5.5.3** are made on both sides of the window or door openings, the maximum length of openings defined in **5.4.6.6** and the maximum ratio of openings defined in **5.4.6.5** may be increased by 20%. This condition is not valid for buildings with adobe walls.



**Figure 5.3**

## **5.5. LINTELS AND BOND BEAMS**

### **5.5.1. Lintels**

**5.5.1.1** – Each of seating lengths of window and door lintels on the walls shall not be less than 15% of lintel clear span and less than 200 mm.

**5.5.1.2** – Cross sections dimensions of lintels as well as longitudinal transverse reinforcement shall not be less than the values given in **5.5.2.1** for horizontal bond beams.

**5.5.1.3** – Timber lintel may be set over and under the window and door in buildings with adobe walls. Timber lintels shall be made with two square timbers with section of 100 mm x 100 mm. Each of seating lengths of timber lintels on the walls shall not be less than 200 mm.

### **5.5.2. Horizontal Bond Beams**

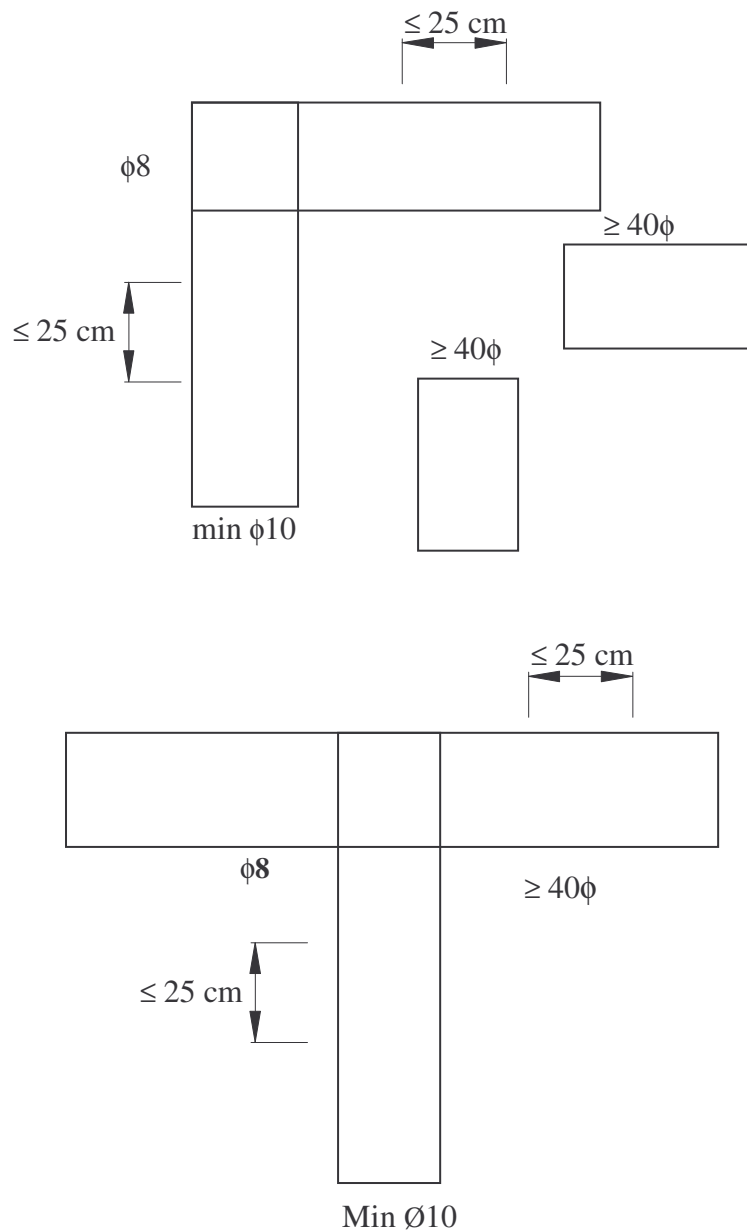
**5.5.2.1** – Reinforced concrete horizontal bond beams satisfying the following conditions shall be made at places where each of the slabs, including stair landings, is supported by structural walls such that they shall be cast (monolithically) with the reinforced concrete slabs.

(a) Width of horizontal bond beams shall be equal to the width of wall, and their height shall not be less than 200 mm.

(b) Concrete quality for bond beams shall be at least C 16,  $\emptyset 8$  hoops with a maximum spacing of 250 mm shall be set with together longitudinal reinforcement at least 6 $\emptyset 10$  on stone walls with three at the bottom and three at the top, and at least 4  $\emptyset 10$  on other load – bearing walls. Longitudinal rebars shall be appropriately overlapped at the corners and intersections to achieve continuity (**Figure 5.4**).

**5.5.2.2** – In rubble stone walls, reinforced concrete bond beams shall be made excluding the slab and stair landing levels in accordance with the rules given in **5.5.2.1** with vertical axis to axis spacing not more than 1.5 m.

**5.5.2.3** – Timber bond beams may be made in adobe masonry walls. Timber bond beams shall be tar emulsified two elements of square sections of 10 cm x 10 cm which are to be placed with outer faces coinciding with the exterior and interior wall surfaces. These pieces shall be tied each other at every 50 cm with nail jointed timber elements of cross section of 5 cm x 10 cm and holes in between shall be filled with stone aggregate.



**Figure 5.4**

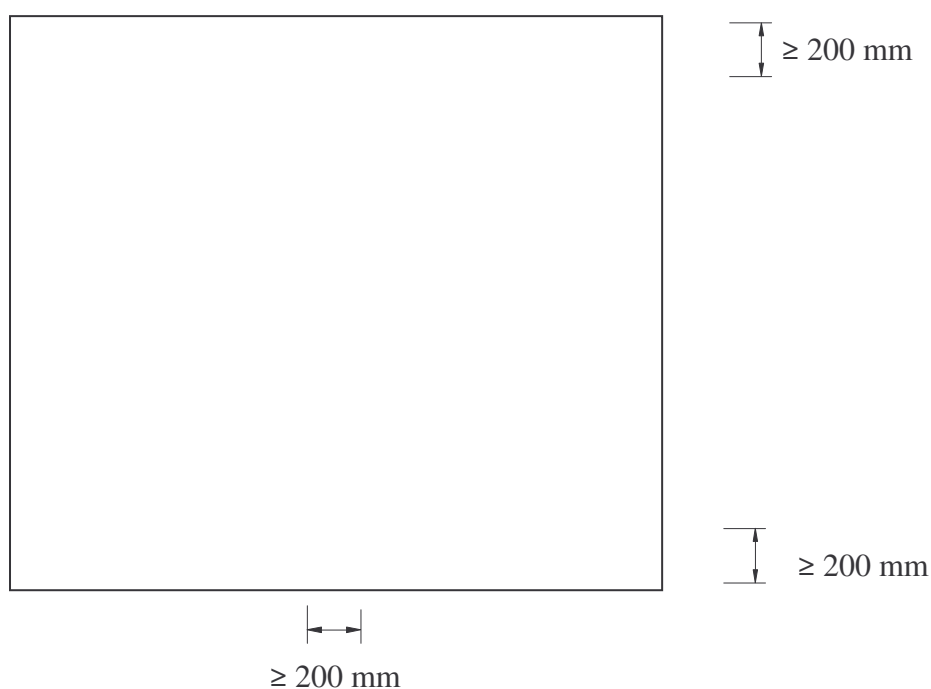
### 5.5.3. Vertical Bond Beams

**5.5.3.1** – In order to enhance the earthquake resistance of masonry buildings, it shall be appropriate to construct reinforced concrete vertical bond beams in full storey height on the corners of buildings, along the vertical intersections of the load – bearing walls and on both sides of the door and window openings.

**5.5.3.2** – Vertical bond beams shall be constructed by reinforcing and concreting the section in between the formworks to be placed parallel to the walls, following the construction of load – bearing walls on both sides (**Figure 5.5**).

**5.5.3.3** – Cross section dimensions of vertical bond beams shall be equal to thicknesses of walls intersecting at corners of the buildings and at the intersections of the load – bearing walls. In vertical bond beams to be constructed on both sides of window and door openings, cross section dimensions of the bond beam perpendicular to the wall shall not be less than the wall thickness, whereas the other cross section dimension shall not be less than 200 mm.

**5.5.3.4** – Concrete quality for vertical bond beams shall be at least C16, Ø8 hoops with a maximum spacing of 200 mm shall be set with together longitudinal reinforcement at least 6Ø12 on stone walls with three parallel to both wall faces and at least 4Ø12 on other load – bearing walls. Longitudinal starter bars shall be provided at the foundation and at the intermediate floors for longitudinal rebars (**Figure 5.5**).



**Figure 5.5**

## 5.6 SLABS

**5.6.1** – Floor slabs of masonry buildings shall be reinforced concrete plate slabs or joist floors whose dimensions and reinforcements are designed in accordance with the requirements of TS-500.

**5.6.2** – Masonry buildings with slabs other than those defined in **5.6.1** above, shall be constructed in all seismic zones with maximum two stories excluding the basement, if any. In such buildings, horizontal bond beams supporting the slabs shall also be constructed in accordance with **5.5.2**. Buildings with adobe walls shall be constructed with maximum one storey excluding basement.

**5.6.3** – Cantilever elements such as balconies, cornices and eaves of roofs shall be made only as an extension of floor slabs and the clear cantilever length shall not be more than 1.5 m. Clear cantilever length of cantilevered stairs shall be at most 1.0 m. This condition is not valid for buildings with adobe walls.

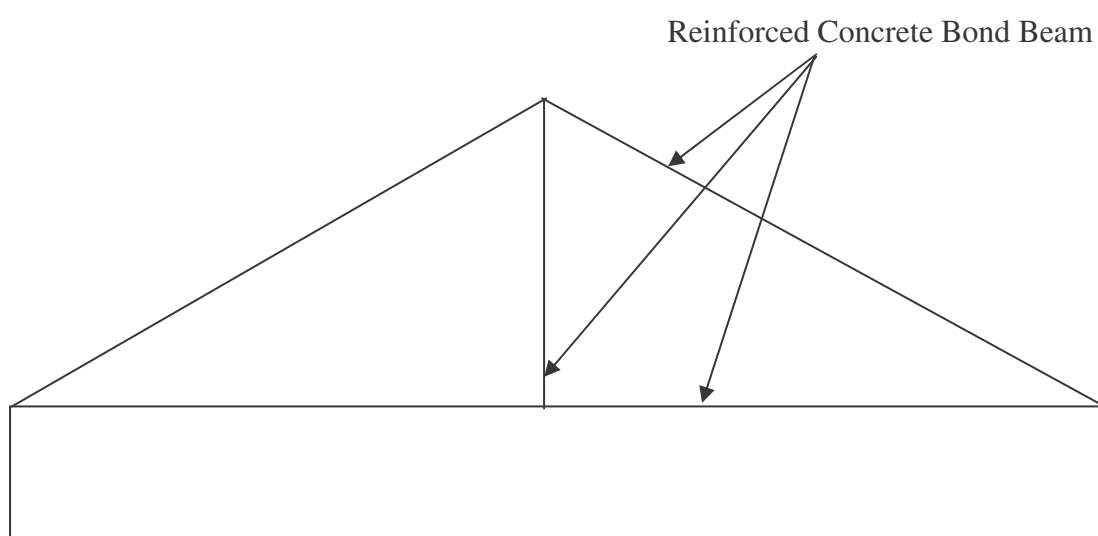
## **5.7. ROOFS**

**5.7.1** – Roofs of the masonry buildings may be constructed as reinforced concrete terrace roof, timber or steel truss roof bearing on roof slab.

**5.7.2** – Connections of the timber roof elements to the roof slabs or to horizontal bond beams on load – bearing walls shall be made in accordance with the rules given in TS – 2510.

**5.7.3** – In the case where the height of the end wall resting on the horizontal bond beam at the top storey exceeds 2.0 m, vertical and inclined bond beams shall be constructed (**Figure 5.6**).

**5.7.4** – Roofs of the buildings with adobe walls shall be constructed in a way not to exceed external walls at most 500 mm and to be as light as possible. Soil roof shall not be made in first and second seismic zones. Soil cover thickness of soil roof shall not be more than 150 mm in third and fourth seismic zones. Roofs of adobe buildings may be constructed as timber spring or reinforced concrete plate.



**Figure 5.6**

## **5.8. NON – BEARING WALLS**

**5.8.1** – Thickness of non-bearing partition walls shall be at least 100 mm. Such walls shall be constructed by connecting to load – bearing walls on both edges along the vertical intersection. At least 10 mm gap shall be allowed between the top of the non – bearing walls and floor bottom of the top slab, however required measures shall be taken in order to prevent the toppling of wall out – of – plane by the effect of seismic loads perpendicular to its plane.

**5.8.2** – Height of parapets on terraces made of masonry wall material shall not be more than 600 mm. Required measures shall be taken in order to prevent toppling of such parapets under the seismic loads.

**5.8.3** – Height of garden walls made of masonry wall material shall not be more than 1.0 m from the pavement level.

## CHAPTER 6 – EARTHQUAKE RESISTANT DESIGN REQUIREMENTS FOR FOUNDATION SOILS AND FOUNDATIONS

### 6.0. NOTATION

|               |  |
|---------------|--|
| $A_o$         | = Effective Ground Acceleration Coefficient defined in <b>Chapter 2</b> .  |
| $C_h$         | = Equivalent lateral seismic coefficient used in calculation of the soil pressure  |
| $C_v$         | = Equivalent vertical seismic coefficient used in calculation of the soil pressure   |
| $H$           | = Total height of uniform soil deposit or sum of layer thicknesses in case of layered soils                                      |
| $h_1$         | = Thickness of the topmost soil layer  |
| $I$           | = Building Importance Factor defined in <b>Chapter 2</b> .   |
| $i$           | = Slope angle which the soil surface on the side of active or passive pressure creates upwards with horizontal axis              |
| $K_{as}$      | = Static active pressure coefficient   |
| $K_{ad}$      | = Dynamic active pressure coefficient  |
| $K_{at}$      | = Total active pressure coefficient  |
| $K_{ps}$      | = Static passive pressure coefficient  |
| $K_{pd}$      | = Dynamic passive pressure coefficient   |
| $K_{pt}$      | = Total passive pressure coefficient   |
| $P_{ad}$      | = Resultant of dynamic active pressure force due to soil mass  |
| $P_{pd}$      | = Resultant of dynamic passive pressure force due to soil mass   |
| $p_{ad}(z)$   | = Variation function of dynamic active pressure due to soil mass with respect to depth   |
| $p_{pd}(z)$   | = Variation function of dynamic passive pressure due to soil mass with respect to depth  |
| $p_v(z)$      | = Variation function of vertical soil pressure with respect to depth   |
| $Q_{ad}$      | = Resultant of dynamic active pressure force due to uniformly distributed external load  |
| $Q_{pd}$      | = Resultant of dynamic passive pressure force due to uniformly distributed external load   |
| $q_{ad}(z)$   | = Variation function of dynamic active pressure due to uniformly distributed external load with respect to depth                 |
| $q_{pd}(z)$   | = Variation function of dynamic passive pressure due to uniformly distributed external load with respect to depth                |
| $q_o$         | = Amplitude of uniformly distributed surcharge   |
| $R_{za}$      | = Reduction factor used in determining the dynamic internal forces applicable to section design of soil retaining walls          |
| $z$           | = Depth measured downwards from free soil surface  |
| $z_{cd}$      | = Depth of the resultant of active or passive pressure force measured downwards from soil surface layer                          |
| $\alpha$      | = Angle of wall – soil interface with vertical towards active or passive pressure  |
| $\delta$      | = Friction angle between soil and wall   |
| $\phi$        | = Internal friction angle of soil  |
| $\varnothing$ | = Rebar diameter   |
| $\gamma$      | = Dry unit volume weight of soil   |
| $\gamma_b$    | = Submerged unit volume weight of soil   |
| $\gamma_s$    | = Saturated unit volume weight of soil   |
| $\lambda$     | = An angle calculated for determining total active and passive pressure coefficients in terms of equivalent seismic coefficients |

## 6.1. SCOPE

Determination of soil conditions of new buildings to be constructed and existing buildings to be enhanced in seismic zones; design of reinforced concrete, structural steel and masonry building foundations and soil retaining structures shall be performed, along with the applicable codes and standards in relevant areas, primarily in accordance with the rules and requirements of this chapter.

## 6.2. DETERMINATION OF SOIL CONDITIONS

### 6.2.1. Soil Groups and Local Site Classes

**6.2.1.1** – Soil groups and local site classes to be considered as the bases of determination of local soil conditions are given in **Table 6.1** and **Table 6.2**, respectively. Values concerning soil parameters in **Table 6.1** are to be considered as standard values given for guidance in determining the soil groups.

**6.2.1.2** – Soil investigations based on required site and laboratory tests are mandatory for below given buildings with related reports prepared and attached to design documents. Soil groups and local site classes defined in accordance with **Table 6.1** and **Table 6.2** shall be clearly indicated in reports.

(a) All buildings with total height exceeding 60 m in the first and second seismic zones,

(b) Irrespective of the building height, buildings in all seismic zones with *Building Importance Factor* of  $I = 1.5$  and  $I = 1.4$  according to **Table 2.3** of **Chapter 2**.

**6.2.1.3** – Regarding the buildings outside the scope of above given **6.2.1.2**, in the first and second seismic zones, it is mandatory to indicate available local information or observation results to be used in determination of soil groups and local site classes in accordance with the definitions in **Table 6.1** and **Table 6.2** or to quote published references relating this subject in the seismic analysis.

**6.2.1.4** – In the first and second seismic zones, determination of horizontal bedding parameters as well as horizontal and axial load carrying capacities of piles under seismic loads in Group (C) and (D) soils according to **Table 6.1** shall be carried out on the basis of soil investigations including in – situ and laboratory tests.

### 6.2.2. Investigation of Liquefaction Potential

In all seismic zones, it is mandatory to investigate whether the *Liquefaction Potential* exists in Group (D) soils according to **Table 6.1**, by using appropriate analysis methods based on in – situ and laboratory tests in the cases the ground water level is less than 10 m from the soil surface and to document these results.

**TABLE 6.1 - SOIL GROUPS**

| <i>Soil Group</i> | <i>Description of Soil Group</i>  | <i>Standard Penetration (N/30)</i> | <i>Relative Density (%)</i> | <i>Unconfined Compressive Strength (kPa)</i> | <i>Drift Wave Velocity (m / s)</i> |
|-------------------|---|------------------------------------|-----------------------------|--|------------------------------------|
| (A)               | 1. Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks                            | —                                  | —                           | > 1000                                       | > 1000                             |
|                   | 2. Very dense sand, gravel...   | > 50                               | 85 – 100                    | —  | > 700                              |
|                   | 3. Hard clay and silty clay...  | > 32                               | —                           | > 400  | > 700                              |
| (B)               | 1. Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity..... | —                                  | —                           | 500 – 1000                                   | 700 – 1000                         |
|                   | 2. Dense sand, gravel.....  | 30 – 50                            | 65 – 85                     | —  | 400 – 700                          |
|                   | 3. Very stiff clay, silty clay...   | 16 – 32                            | —                           | 200 – 400                                    | 300 – 700                          |
| (C)               | 1. Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity                      | —                                  | —                           | < 500  | 400 – 700                          |
|                   | 2. Medium dense sand and gravel.....  | 10 – 30                            | 35 – 65                     | —  | 200 – 400                          |
|                   | 3. Stiff clay and silty clay.....   | 8 – 16                             | —                           | 100 – 200                                    | 200 – 300                          |
| (D)               | 1. Soft, deep alluvial layers with high ground water level  | —                                  | —                           | —  | < 200                              |
|                   | 2. Loose sand.....  | < 10                               | < 35                        | —  | < 200                              |
|                   | 3. Soft clay and silty clay.....  | < 8                                | —                           | < 100  | < 200                              |

**TABLE 6.2 - LOCAL SITE CLASSES**

| <i>Local Site Class</i> | <i>Soil Group according to Table 6.1 and Topmost Soil Layer Thickness (<math>h_I</math>)</i> |
|-------------------------|--|
| Z1                      | Group (A) soils<br>Group (B) soils with $h_I \leq 15$ m                                      |
| Z2                      | Group (B) soils with $h_I > 15$ m<br>Group (C) soils with $h_I \leq 15$ m                    |
| Z3                      | Group (C) soils with $15 \text{ m} < h_I \leq 50$ m<br>Group (D) soils with $h_I \leq 10$ m  |
| Z4                      | Group (C) soils with $h_I > 50$ m<br>Group (D) soils with $h_I > 10$ m                       |

### **NOTES ON TABLE 6.2:**

*(a) In the case where the thickness of the topmost soil layer under the foundation base is less than 3 m, the layer below may be considered as the topmost soil layer indicated in **Table 6.2**.*

*(b) In the case where the foundation system is comprised of vertical piles or piles with a slope of 1 / 6 or less inclined with respect to vertical, the topmost soil layer indicated in **Table 6.2** may be deemed to be the layer at the lower tip of the shortest pile. However in such a case, it is essential to take piles into account as structural elements together with the superstructure in the seismic analysis to be performed in accordance with **Chapter 2**, or to idealize horizontal and vertical pile rigidities with equivalent springs under the pile caps. By considering group effect in the analysis, rigidity and inertia properties of pile caps and tie beams together with horizontal and vertical bedding of piles to the soil (soil – pile interaction).*

*(c) In cases where conditions given in paragraph (b) above are not applied or pile inclination with respect to vertical exceeds 1 / 6 in the third and fourth seismic zones in accordance with **6.3.3.1**, the topmost soil layer indicated in **Table 6.2** shall be taken as the first layer under the pile caps.*

## **6.3. RULES AND REQUIREMENTS FOR FOUNDATIONS**

### **6.3.1. General Rules**

Building foundations shall be constructed on the bases of principles of soil mechanics and foundation construction, by considering the properties of underlying soils such that any damage in the superstructure due to settlements or differential settlements during earthquake is avoided. Rules given in this chapter for foundations are applicable to the foundations of reinforced concrete, structural steel and masonry buildings.

### **6.3.2. Soil Safety Stresses and Safety Ultimate Load of Piles**

**6.3.2.1** – Soil safety stress and safety ultimate load of pile for horizontal and axial loads in foundations with pile specified with respect to static loads may be increased in the case of seismic loading by at most 50 % for foundation soils classified as Group (A), (B) and (C) in **Table 6.1**.

**6.3.2.2** – Soil safety stress and safety ultimate load of piles shall not be increased in the case of seismic loading for foundation soils classified as Group (D) in **Table 6.1**.

### **6.3.3. Requirements for Piled Foundations**

**6.3.3.1** – Inclined piles with more than 1 / 6 inclination with respect to vertical shall not be used in the first and second seismic zones.

**6.3.3.2** – Piled foundations shall be analyzed with respect to earthquake induced lateral loads and effects in addition to axial loads.

**6.3.3.3** – In the first and second seismic zones, longitudinal reinforcement ratio of cast – in – situ cased or uncased bored piles shall not be less than 0.008 within the top 1 / 3 of the pile length under the pile cap, which shall not be less than 3 m. Diameter of spiral reinforcement to be provided within this zone shall not be less than 8 mm and the pitch of spirals shall not be more than 200 mm, however pitch of spirals shall be reduced to 100 mm within a length of at least twice the pile diameter from the top

**6.3.3.4** – Longitudinal reinforcement ratio of reinforced concrete prefabricated driven piles shall not be less than 0.01. In the first and second seismic zones, diameter of transverse reinforcement within the top 1 / 3 of the pile length under the pile cap shall not be less than 8 mm. Hoop spacing or pitch of spirals within this length shall not be more than 200 mm, however hoop spacing pitch of spirals shall be reduced to 100 mm within a length of at least twice the pile diameter top (twice the greatest dimension of piles with rectangular cross-section) from the from the top. Transverse reinforcement requirements shall be equally applicable to pre – stressed prefabricated driven piles.

#### **6.3.4. Foundation Tie Beams**

**6.3.4.1** – In reinforced concrete and structural steel buildings, tie beams shall be provided to connect individual foundations or pile caps in both directions or to connect continuous foundations at column or structural wall axes. Tie beams may be omitted or their numbers may be reduced on foundation soils classified as Group (A) in **Table 6.1**.

**6.3.4.2** – Consistent with the foundation excavation, tie beams may be constructed at any level between the bottom of the foundation and the bottom of the column.

**6.3.4.3** – The minimum requirements to be satisfied by tie beams are given in **Table 6.3** depending on the seismic zone of the building and the soil groups defined in **Table 6.1**.

**TABLE 6.3 – MINIMUM REQUIREMENTS FOR TIE BEAMS**

| <b>DESCRIPTION OF REQUIREMENT</b>                     | <b>Seismic Zone</b> | <b>Soil Group (A)</b> | <b>Soil Group (B)</b> | <b>Soil Group (C)</b> | <b>Soil Group (D)</b> |
|---|---------------------|-----------------------|-----------------------|-----------------------|-----------------------|
| <b>1. Minimum axial force of tie beam (*)</b>         | <b>1, 2</b>         | <b>6 %</b>            | <b>8 %</b>            | <b>10 %</b>           | <b>12 %</b>           |
|   | <b>3, 4</b>         | <b>4 %</b>            | <b>6 %</b>            | <b>8 %</b>            | <b>10 %</b>           |
| <b>2. Minimum cross – section dimension (mm) (**)</b> | <b>1, 2</b>         | <b>250</b>            | <b>250</b>            | <b>300</b>            | <b>300</b>            |
|   | <b>3, 4</b>         | <b>250</b>            | <b>250</b>            | <b>250</b>            | <b>250</b>            |
| <b>3. Minimum cross-section area (mm<sup>2</sup>)</b> | <b>1, 2</b>         | <b>62500</b>          | <b>75000</b>          | <b>90000</b>          | <b>90000</b>          |
|   | <b>3, 4</b>         | <b>62500</b>          | <b>62500</b>          | <b>75000</b>          | <b>75000</b>          |
| <b>4. Minimum longitudinal reinforcement</b>          | <b>1, 2</b>         | <b>4Ø14</b>           | <b>4Ø16</b>           | <b>4Ø16</b>           | <b>4Ø18</b>           |
|   | <b>3, 4</b>         | <b>4Ø14</b>           | <b>4Ø14</b>           | <b>4Ø16</b>           | <b>4Ø16</b>           |

(\*) *As a percentage of the greatest axial force of columns or structural walls tie beams are connected to.*

(\*\*) *The minimum cross – section dimension shall not be less than 1/30 of the clear span of the tie beam.*

**6.3.4.4** – Tie beams shall be considered in the section design as resisting against both pressure and tension forces. In the case of resistance against pressure, buckling effect may be neglected in tie beams confined by soil or floor concrete. In the case of tension, it shall be considered that tension forces are resisted by only reinforcement. Hoop diameter of tie beams shall not be less than 8 mm and their spacing shall not be more than 200 mm.

**6.3.4.5** – Tie beams may be replaced by reinforced concrete slabs. In such a case slab thickness shall not be less than 150 mm. It shall be demonstrated by calculation that the slab and its reinforcement safely transfer the forces equal to those given for tie beams in **Table 6.3**.

### **6.3.5. Under – Wall Foundations of Masonry Buildings**

**6.3.5.1** – Foundations of masonry buildings shall be constructed as reinforced concrete under – wall foundations under the load bearing walls. Depth of an under – wall foundation shall be determined by considering the soil characteristics, ground water level and the local frost depth. In buildings without basement, top level of stone or concrete walls to be constructed on the foundations shall be at least 0.50 m above the pavement level.

**6.3.5.2** – Concrete quality of under – wall foundations shall be at least C16. Requirements on the dimensions and the reinforcement of under – wall foundations are given in **Table 6.4** depending on the soil groups defined in **Table 6.1**.

**6.3.5.3** – Foundations may be constructed with steps on sloped ground made of soils classified in **Table 6.1** as Group (A), (B) or (C). Requirements for stepped foundations are also given in **Table 6.4**.

**6.3.5.4** – Lateral spacing between the longitudinal rebars to be placed under – wall foundations shall not exceed 0.30 m at the top and bottom; appropriate overlapping shall be provided to achieve continuity at corners, junctions and in stepped foundations.

**TABLE 6.4 – REQUIREMENTS FOR UNDER – WALL FOUNDATIONS**

| <i><b>DESCRIPTION OF REQUIREMENT</b></i>                        | <i><b>Soil Group (A),(B)</b></i> | <i><b>Soil Group (C)</b></i> | <i><b>Soil Group (D)</b></i> |
|---|----------------------------------|------------------------------|------------------------------|
| Minimum foundation width (mm)                                   | 500                              | 600                          | 700                          |
| Shoe width (from both sides) to be added to wall thickness (mm) | 2 x 150                          | 2 x 200                      | 2 x 250                      |
| Minimum foundation height (mm)                                  | 300                              | 400                          | 400                          |
| Minimum longitudinal reinforcement at the top and bottom        | 3Ø12                             | 3Ø14                         | 4Ø14                         |
| Minimum hoop at foundations                                     | Ø8 / 30                          | Ø8 / 30                      | Ø8 / 30                      |
| Minimum lateral spacing of steps (mm)                           | 1000                             | 1500                         | –                            |
| Minimum step overlapping length (mm)                            | 300                              | 400                          | –                            |
| Maximum step height (mm)  | 300                              | 300                          | –                            |

## 6.4. SEISMIC SOIL PRESSURES AND SOIL RETAINING STRUCTURES

### 6.4.1. Total Active and Passive Pressure Coefficients

**6.4.1.1** – *Total Active Pressure Coefficient,  $K_{at}$ , and Total Passive Pressure Coefficient,  $K_{pt}$* , which shall be used to calculate the sum of static soil pressure and additional dynamic soil pressure induced by earthquake are given by **Equation (6.1)**, by neglecting the soil cohesion in order to remain on the conservative side.

$$K_{at} = \frac{(1 \pm C_v) \cos^2(\varphi - \lambda - \alpha)}{\cos \lambda \cos^2 \alpha \cos(\delta + \alpha + \lambda)} \left[ 1 + \sqrt{\frac{\sin(\varphi + \delta) \sin(\varphi - \lambda - i)}{\cos(\delta + \alpha + \lambda) \cos(i - \alpha)}} \right]^{-2} \quad (6.1a)$$

$$K_{pt} = \frac{(1 \pm C_v) \cos^2(\varphi - \lambda + \alpha)}{\cos \lambda \cos^2 \alpha \cos(\delta - \alpha + \lambda)} \left[ 1 - \sqrt{\frac{\sin(\varphi + \delta) \sin(\varphi - \lambda + i)}{\cos(\delta - \alpha + \lambda) \cos(i - \alpha)}} \right]^{-2} \quad (6.1b)$$

**6.4.1.2** – The angle  $\lambda$  in **Equation (6.1)** is defined by **Equation (6.2)**.

(a) For dry soils,

$$\lambda = \arctan \left[ \frac{C_h}{(1 \pm C_v)} \right] \quad (6.2a)$$

(b) For submerged soils,

$$\lambda = \arctan \left[ \frac{\gamma_s}{\gamma_b} \frac{C_h}{(1 \pm C_v)} \right] \quad (6.2b)$$

**6.4.1.3** – In the case of submerged or saturated soils,  $\delta$  in **Equation (6.1)** shall be replaced by  $\delta/2$ .

**6.4.1.4** – Equivalent lateral seismic coefficient,  $C_h$ , appearing in **Equation (6.2)** is defined by **Equation (6.3)**.

(a) In soil retaining structures behaving as vertical free cantilevers,

$$C_h = 0.2 (I + 1) A_0 \quad (6.3a)$$

(b) In soil retaining structures and elements horizontally supported by building floors or soil anchors,

$$C_h = 0.3 (I + 1) A_0 \quad (6.3b)$$

**6.4.1.5** – Equivalent vertical seismic coefficient,  $C_v$ , appearing in **Equation (6.1)** and **Equation (6.2)** is defined by **Equation (6.4)**. However, it shall be  $C_v = 0$  in basement walls which are horizontally supported by building floors.

$$C_v = \frac{2C_h}{3} \quad (6.4)$$

The cases  $+C_v$  or  $-C_v$  shall be considered as consistent with **Equation (6.2)** to yield more unfavorable lateral soil pressure by **Equation (6.1)**.

#### **6.4.2. Dynamic Active and Passive Soil Pressures**

**6.4.2.1** – Dynamic active pressure coefficient,  $K_{ad}$ , and dynamic passive pressure coefficient,  $K_{pd}$ , induced by earthquake shall be determined by **Equation (6.5)**.

$$K_{ad} = K_{at} - K_{as} \quad (6.5a)$$

$$K_{pd} = K_{pt} - K_{ps} \quad (6.5b)$$

Static active pressure coefficient,  $K_{as}$ , and static passive pressure coefficient,  $K_{ps}$ , appearing in **Equation (6.5)** may be obtained by substituting  $\lambda = 0$  and  $C_v = 0$  in **Equation (6.1)**.

**6.4.2.2** – Variation of dynamic active and passive soil pressures along the depth of soil which is induced by soil mass in addition to static soil pressure in case of earthquake which are induced in addition to static soil pressure by the soil mass during earthquake, is defined by **Equation (6.6)**.

$$p_{ad}(z) = 3 K_{ad} (1 - z/H) p_v(z) \quad (6.6a)$$

$$p_{pd}(z) = 3 K_{pd} (1 - z/H) p_v(z) \quad (6.6b)$$

In the special case of uniform and dry soil, the positive value of the resultant  $P_{ad}$  of dynamic active soil pressure and the negative value of the resultant  $P_{pd}$  of dynamic passive soil pressure which are induced in addition to static soil because of earthquake and  $z_{cd}$  which indicates the depths of such resultants measured from the top soil level, are obtained as given by **Equation (6.7)** and **Equation (6.8)**, respectively, by integrating **Equation (6.6)** along the soil depth by taking  $p_v(z) = \gamma z$ :

$$P_{ad} = 0.5 \gamma K_{ad} H^2 \quad (6.7a)$$

$$P_{pd} = 0.5 \gamma K_{pd} H^2 \quad (6.7b)$$

$$z_{cd} = H/2 \quad (6.8)$$

In the case of submerged soil,  $\gamma_b$  shall be considered in lieu of  $\gamma$  in determining  $p_v(z)$  and hydrodynamic pressure of water shall not be calculated additionally. In the case of saturated soil,  $\gamma_s$  shall be used in lieu of  $\gamma$ .

**6.4.2.3** – Variation of dynamic active and passive pressures along the depth of soil which are induced in addition to static soil pressure by uniformly distributed external loads in case of earthquake, are defined by **Equation (6.9)**.

$$q_{ad}(z) = 2 q_o K_{ad} (1 - z/H) \cos \alpha / \cos (\alpha - i) \quad (6.9a)$$

$$q_{pd}(z) = 2 q_o K_{pd} (1 - z / H) \cos \alpha / \cos (\alpha - i) \quad (6.9b)$$

In the case where soil characteristics are uniform, the resultants  $Q_{ad}$  and  $Q_{pd}$  of active (positive) and passive (negative) soil pressures which are induced in addition to static soil pressure by contribution of earthquake and  $z_{cd}$  which indicates the depths of such resultants measured from the top soil level, are obtained as given by **Equation (6.10)** and **Equation (6.11)**, respectively, by integrating **Equation (6.9)** along the soil depth.

$$Q_{ad} = q_o K_{ad} H \cos \alpha / \cos (\alpha - i) \quad (6.10a)$$

$$Q_{pd} = q_o K_{pd} H \cos \alpha / \cos (\alpha - i) \quad (6.10b)$$

$$z_{cd} = H / 3 \quad (6.11)$$

### 6.4.3. Dynamic Soil Pressures in Layered Soils

Expressions given above by **Equation (6.6)** and **Equation (6.9)** can be applied also for case of layered soils. In such a case, the coefficients  $K_{ad}$  or  $K_{pd}$  pertinent for the layer concerned shall be used and the depth  $z$  shall always be measured downwards from the free soil surface. Resultants of dynamic active or passive pressures of each layer and their depths within the layer may be obtained by integrating **Equation (6.6)** and **Equation (6.9)** along the depth of the relevant layer.

### 6.4.4. Requirements for Soil Retaining Structures

**6.4.4.1** – In the analysis performed by considering dynamic soil pressures given by **Equation (6.6)** and **Equation (6.9)** in addition to static soil pressures and inertia forces of the structure which acts on its self mass, safety factor against sliding shall be taken at least 1.0 and safety factor against overturning shall be taken at least 1.2.

**6.4.4.2** – Internal forces to be taken into account in the section design of reinforced concrete soil retaining walls and reinforced concrete or steel sheet pile walls shall be those obtained by dividing the internal forces calculated according to the dynamic soil pressures given in **Equation (6.6)** and **Equation (6.9)** into the coefficient  $R_{za} = 1.5$ , in addition to internal forces induced by static soil pressures. It may be taken  $R_{za} = 2.5$  for temporary steel sheet pile walls.

## CHAPTER 7 – EVALUATION AND INVIGORATE OF THE EXISTING BUILDINGS

### 7.0. NOTATION

|                    |   |   |
|--------------------|---|---|
| $A_c$              | = | Gross section area of column or wall end zone   |
| $a_1^{(i)}$        | = | Modal acceleration belonging to the first mode made in the end of the (i)'th drive step   |
| $b$                | = | Width of the horizontal plate in steel spiral.  |
| $bw$               | = | The body width of the cross section   |
| $d$                | = | Effective beam and column height  |
| $d_1^{(i)}$        | = | Modal displacement belonging to the first mode obtained at the and end of (i)'th impulse step.  |
| $d_1^{(p)}$        | = | Modal displacement request belonging to the first mode.   |
| $(EI)_e$           | = | Effective deflection rigidity of the cracked cross section.   |
| $(EI)_o$           | = | Deflection rigidity of the cracked cross section  |
| $f_{cm}$           | = | Strength of the existing concrete defined according to 7.2  |
| $f_{ctm}$          | = | Tensile strength of the existing concrete defined according to 7.2  |
| $f_{yw}$           | = | Outflow strength of the steel in steel spiral.  |
| $H_w$              | = | Total height of partition measured from under the foundation or from the ground floor   |
| $h$                | = | Column cross section dimension in the working direction   |
| $h_{wall}$         | = | Height of the filling wall  |
| $h_{ji}$           | = | Storey height of the j'th column or curtain in i'th storey  |
| $h_k$              | = | Length of the column  |
| $L_p$              | = | Size of plastic joint   |
| $\ell_{wall}$      | = | Height of filling wall  |
| $\ell_w$           | = | Length of partition or piece of strap partition on plan   |
| $M_{x1}$           | = | Effective mass belonging to first (prevalent) defined for linear elastic behavior in the x earthquake direction   |
| $N_d$              | = | Factored axial force calculated under simultaneous action of vertical loads and seismic loads   |
| $N_k$              | = | Axial force correspond to moment capacity calculated with the existing reinforcement strength defined according to 7.2.   |
| $R_a$              | = | Inhibition Coefficient of the Power of Earthquake   |
| $r$                | = | Ratio of exposure/capacity  |
| $r_s$              | = | The limit value of ratio of exposure/capacity   |
| $s$                | = | Space between horizontal plates in steel spiral.  |
| $S_{dil}$          | = | Non-linear spectral displacement belonging to first mode  |
| $t_j$              | = | Thickness of the horizontal plates in steel spiral.   |
| $u_{xNI}^{(i)}$    | = | Displacement belonging to first mode obtained from the end of the i'th impulse step in the direction of (x) earthquake at the top of the building (N'th storey) |
| $u_{xNI}^{(p)}$    | = | Top displacement request in the direction of (x) earthquake at the top of the building (N'th storey)  |
| $V_e$              | = | Shear force taken into account for the calculation of transverse reinforcement of column, beam or wall  |
| $V_j$              | = | Additional shearing strength provided with steel spiral.  |
| $V_r$              | = | Shearing strength of the column, beam and curtain cross section   |
| $V_{x1}^{(i)}$     | = | Base shearing force belonging to first mode (prevalent mode) obtained at the end of I'th impulse step in direction of x earthquake                              |
| $\varepsilon_{cg}$ | = | Deformation of concrete pressure unit in the outermost fibrous of the section inside of the lateral reinforcement binders.                                      |

|                 |   |   |
|-----------------|---|---|
| $\epsilon_{cu}$ | = | Deformation of concrete pressure unit in the outermost fibrous of the section of the cross section  |
| $\epsilon_s$    | = | Deformation of reinforcement steel unit   |
| $\phi_p$        | = | Plastically curvature request   |
| $\phi_t$        | = | Total curvature request   |
| $\phi_y$        | = | Mode figure width in the direction of (x) earthquake at the top of the building (N'th storey)   |
| $\Phi_{xN1}$    | = | Coefficient of the Level of Torsion defined in I'th storey  |
| $\Gamma_{x1}$   | = | Additive factor belonging to first mode in the direction of x earthquake  |
| $\eta_{bi}$     | = | Torsionally Irregularity Factor defined at i'th storey of building  |
| $\lambda$       | = | Equivalent Earthquake Power Derogation Factor   |
| $\theta_p$      | = | Plastic drift volition  |
| $\rho$          | = | Tension reinforcement ratio   |
| $\rho_b$        | = | Balanced reinforcement ratio  |
| $\rho_s$        | = | Volumetric ratio of spiral reinforcement which are exist in the cross section and arranged as “special seismic hoops and crossties” according to <b>3.2.8</b> |
| $\rho_{sm}$     | = | Volumetric ratio of the transverse reinforcement necessary to be existed in the cross section according to <b>3.3.4, 3.4.4</b>                                |
| $\rho'$         | = | Pressure reinforcement ratio  |
| $w_e$           | = | Width of polymer ribbon with fiber.   |

## **7.1. SCOPE**

**7.1.1** – Rules of calculation to be used in the assessment of performances of the existing buildings and building - type structures in earthquake zones under the impact of an earthquake, principles to be followed in decisions of strengthening, and principles of design for strengthening for buildings that decision of strengthening is made are defined in this section.

**7.1.2** – Calculation methods and assessment essentials given in this section do not apply to steel and masonry buildings. However, the existing data regarding the existing steel and masonry buildings will also be collected in this section. Calculations and assessment for existing and strengthened steel buildings will be carried out according to the essentials for newly built structures defined in **Sections 2 and 4**. Calculations and assessment for existing and strengthened masonry buildings will be carried out within the frame of the essentials defined in **Section 5**.

**7.1.3** – The existing prefabricated reinforced concrete buildings can be assessed according to the rules given in **Sections 2 and 3**, or else, **7.6** can be used for the determining of the performances of these buildings. However, rules of **3.12** will prevail for the assessment of the confinement zones.

**7.1.4** – Rules given in this section do not apply to non-building-type structures mentioned in **2.12**. Furthermore, assessment and strengthening of registered buildings with historical and cultural value and monuments are outside the scope of this Regulation.

**7.1.5** – Earthquake performance of damaged buildings following an earthquake causing damage in the building cannot be determined with the methods given in this section.

**7.1.6** – To Strengthen a damaged building following an earthquake causing damage in the building, and then to determine the earthquake performance of the strengthened building, essentials given in this section shall be used. The civil engineer responsible for the project will decide in what extent the strength and rigidity of the existing elements of the damaged building will be taken into consideration when strengthening the damaged building.

## **7.2. DATA COLLECTION FROM BUILDINGS**

### **7.2.1. Scope of Data to Be Collected from Buildings**

**7.2.1.1** – Data regarding the details and sizes of the elements to be used in determining the capacities of the elements of the supporting systems of the existing buildings and information regarding the geometry and material characteristics of the supporting systems will be achieved from the projects and reports of such buildings, from observations and measurements to be carried out on the building, and from trials performed on the material samples taken from the building.

**7.2.1.2** – The procedures to be performed in the scope of data collection from buildings are defining of the structural system, determining the geometry, foundation system and ground properties of the building, determining the existing damage, if any, and / or repairs, measuring the dimensions of the elements, determining the material characteristics, and checking the compliance of all these data with the project of the building, if any.

**7.2.1.3** – The procedures of examination, data collection and arrangement, assessment, material sample collection, and trials within the scope of data collection from buildings shall be performed under the responsibility civil engineers.

### **7.2.2. Levels of information**

According to the scope of the data obtained from the examination of the buildings concerning the existing situation, information level for each building type, and consequently, the information level coefficients mentioned in **7.2.16** shall be completed. Information levels shall be classified as limited, medium, and comprehensive, respectively. Information level shall be used for the calculations of capacities of supporting elements.

**7.2.2.1** – In the *limited information level*, there are no projects of the supporting system. Characteristics of the supporting system are determined with measurements performed. Limited information level cannot be applied to “*Buildings that Immediate Use Is required Following an Earthquake*” and “*Buildings Intensely Inhabited by People for Long Periods*” defined in **Table 7.7**.

**7.2.2.2** – In the *limited information level*, in case the project for the supporting system of the building are not present, then more measurements are performed as compared to the case of limited information. If such projects are present, then measurements defined for the limited level are performed to confirm the information in the project.

**7.2.2.3** – In the *comprehensive information level*, the project for the supporting system of the building are present. Measurements sufficient to confirm the information in the project are performed.

### **7.2.3. The Existing Material Strength**

Strength of the materials to be used in the calculation of the capacities of the supporting elements is defined as *the existing materials strength*.

### **7.2.4. Limited Information Level in Reinforced Concrete Buildings**

**7.2.4.1** – *Geometry of the Building*: The measured drawings of the supporting system shall be prepared with field work. In case the architectural projects are present, these can be used as aids to the works for preparation of measured drawings. Information obtained must include the locations, axis openings, heights and dimensions of all the reinforced concrete elements and nonbearing walls, and must be sufficient for the creation of a calculation model for the building. Foundation system shall be determined by digging examination holes of sufficient numbers in- or outside the building. Short columns or similar irregularities in the building shall be entered on the floor plan and sections. Relation of the building with the neighboring buildings (separated, adjoining, jointing present / absent) shall be determined.

**7.2.4.2** – *Details of elements*: Projects or application drawings are not present. It is assumed that the amount of reinforcement in the reinforcement elements and details meet the minimum requirements for reinforcement for the date that the building was constructed. With the purpose of confirming this assumption, or to determine to what extent it is true, reinforcements and lengths of reinforcement laps shall be determined by scraping off the concrete covers of 10 % of bulkheads and columns and 5 % of beams, and at least one in each floor. Such scraping must be performed on the one-thirds of the lengths the columns

and beams in the middle of the opening; however, same must be performed on the overlapping parts of at least three columns with the purpose of determining the length of overlapping. Scraped surfaces shall be covered afterwards with high-strength repair mortar. In addition, number of longitudinal and transverse reinforcement elements of 20 % of the elements that have not been scraped shall be determined using devices for reinforcement-determining devices. The *coefficient of actual reinforcement* expressing the ratio of the amount of reinforcement the actually found in reinforced columns and beams to the minimum reinforcement shall be separately determined for columns and beams. This coefficient shall be applied to all the other elements that reinforcement has not been determined, and the possible amount of reinforcement shall thus be determined.

**7.2.4.3 – Characteristics of Materials:** At least two samples of concrete (borehole sample) shall be collected from columns or bulkheads in each floor according to the conditions stated in TS-10465, and tests shall be performed to determine the lowest pressure strength to be considered as the existing concrete strength. Reinforcement class shall be determined according to the visual examination as explained in the paragraph above, and the characteristic yield strength of the steel in this class shall be taken as the existing steel strength. In this examination, elements showing corrosion in reinforcement shall be marked on the plan, and this condition shall be taken into consideration in element capacity calculations.

## **7.2.5. Medium Level Information in Reinforced Concrete Buildings**

**7.2.5.1 – Geometry of the Building:** In case the reinforcement projects for the building are present, compliance of the geometry to the project shall be checked. In case there are no projects, then measured drawings shall be prepared by field work. Information obtained must include the locations in each floor, openings, heights and dimensions of all the reinforced concrete elements and nonbearing walls, and must be sufficient for the creation of a calculation model for the building. Information regarding the geometry of the building must include the details required for a precise description of the mass of the building. Short columns or similar irregularities in the building shall be entered on the floor plan and sections. Relation of the building with the neighboring buildings (separated, adjoining, jointing present/absent) shall be determined. Foundation system shall be determined by digging examination holes of sufficient numbers in- or outside the building.

**7.2.5.2 – Details of elements:** In case project or manufacturing drawings are not present, then conditions in 7.2.4.2 apply; however, number of bulkheads, columns and beams that concrete covers shall be scraped off for the checking of reinforcement shall not be less than 20 % of the total number of columns in a certain floor, and 10 % of the number of beams, and at least two in each floor. In case project or manufacturing drawings are present, then conditions stated in 7.2.4.2 shall be applied with the numbers stated therein. In addition, number of longitudinal and transverse reinforcement elements of 20 % of the elements that have not been scraped shall be determined using devices for reinforcement-determining devices. The *coefficient of actual reinforcement* expressing the ratio of the amount of reinforcement the actually found in reinforced columns and beams to the minimum reinforcement shall be separately determined for columns and beams. This coefficient used to determine the capacities of elements cannot exceed 1. This coefficient shall be applied to all the other elements that reinforcement has not been determined, and the possible amount of reinforcement shall thus be determined.

**7.2.5.3 – Characteristics of Materials:** One concrete sample (borehole sample) shall be collected from columns or bulkheads in each floor not less than 3 in number, and also not less than 9 from the entire building and one sample from each 400 square meters for the tests to be performed according to the conditions stated in TS - 10465. The mean – standard deviation values found for the samples shall be taken as the *existing concrete strength* when calculating the capacities of the elements. Distribution of the concrete strength throughout the building can be checked with the readings of adjusted concrete test hammer to the borehole sample test results or similar undamaged examination tools. Reinforcement class shall be determined by visual examination on surfaces scraped as described in the paragraph above, and the characteristic strength of the steel of this class shall be taken as the *existing steel strength* in capacity calculations. In this examination, elements showing corrosion in reinforcement shall be marked on the plan, and this condition shall be taken into consideration in element capacity calculations.

## **7.2.6. Comprehensive Information Level in Reinforced Concrete Buildings**

**7.2.6.1 – Geometry of the Building:** Reinforcement projects for the building are present. Compliance with the projects of the actual geometry is checked with the measurements performed in the building. In case the projects show important conflicts with the measurements, then the projects are ignored, and the building is examined according to the rules for medium level information. Short columns or similar irregularities in the building shall be entered on the floor plan and sections. Relation of the building with the neighboring buildings (separated, adjoining, jointing present/absent) shall be determined. Information regarding the geometry of the building must include the required details for a precise description of the mass of the building. Foundation system shall be determined by digging examination holes of sufficient numbers in- or outside the building.

**7.2.6.2 – Details of elements:** Detail projects of reinforcement of the building are present. Procedures stated in 7.2.4.2 for checking of the compliance of reinforcement with the project shall be applied on the same number of reinforcement elements. In addition, locations and numbers of longitudinal and transverse reinforcement elements of 20 % of the elements that have not been scraped shall be determined using devices for reinforcement-determining devices. In case there are any conflicts between the project and the application, then *coefficient of actual reinforcement* expressing the ratio of the amount of reinforcement the actually found in reinforced columns and beams to the minimum reinforcement shall be separately determined for columns and beams. This coefficient used to determine the capacities of elements cannot exceed 1. This coefficient shall be applied to all the other elements that reinforcement has not been determined, and the possible amount of reinforcement shall thus be determined.

**7.2.6.3 – Characteristics of Materials:** One concrete sample (borehole sample) shall be collected from columns or bulkheads in each floor not less than 3 in number, and also not less than 9 from the entire building and one sample from each 200 square meters for the tests to be performed according to the conditions stated in TS - 10465. The mean – standard deviation values found for the samples shall be taken as the *existing concrete strength* when calculating the capacities of the elements. Distribution of the concrete strength throughout the building can be checked with the readings of adjusted concrete test hammer to the borehole sample test results or similar undamaged examination tools. Reinforcement class shall be determined on surfaces scraped as described in the paragraph above, and tests shall be performed on one sample for each steel class (S220, S420, etc.), and yield strength and breaking strength, and deformation characteristics of the steel shall determined and compliance with the project shall be determined. If it complies with the project, the characteristic strength of the steel used in the project shall be taken as the *existing steel*

*strength*. If not, tests shall be performed on at least three more samples, and the least favorable value shall be taken as the *existing steel strength* in the element capacity calculations. In this examination, elements showing corrosion in reinforcement shall be marked on the plan, and this condition shall be taken into consideration in element capacity calculations.

#### **7.2.7. Limited Information Level in Steel Constructions**

Limited information level in steel constructions is not valid.

#### **7.2.8. Medium Information Level in Steel Constructions**

**7.2.8.1 – Geometry of the Building:** Conditions given in **7.2.5.1** are valid as they are, except that the term “*reinforced*” shall be replaced with “*steel*”.

**7.2.8.2 – Details of elements:** In case steel projects or manufacturing drawings are not present, boring controls of all the steel elements and other elements of other types (columns, beams, joints, crosses, flooring) shall be performed, and welding characteristics and joining details shall be established in detail. In case the application projects and manufacturing drawings are present, then the precise dimension controls shall be performed for 20 % of the said elements.

**7.2.8.3 – Characteristics of Materials:** In case steel projects are not present, then one sample shall be cut off from each steel construction type, and tests shall be performed to determine the strength and deformation characteristics. Likewise, a welding sample shall be cut off to perform tests. Such cut off places shall be filled and repaired. One sample bolt shall be taken for tests from bolted connections. The mean strengths obtained from tests shall be taken as *existing steel strength* for the calculation of the strengths of elements. In case steel projects are present, then the characteristic strengths foreseen in the project shall be taken as *existing steel strength*.

#### **7.2.9. Comprehensive Information Level in Steel Constructions**

**7.2.9.1 – Geometry of the Building:** Conditions given **7.2.6.1** are valid as they are, except that the term “*reinforced*” shall be replaced with “*steel*”.

**7.2.9.2 – Details of elements:** Steel detail projects of the building are present. Dimensions of the elements shown in the project and details of joints shall be confirmed by checking in at least 20% of the total numbers of each element and joint types.

**7.2.9.3 – Characteristics of Materials:** Steel class mentioned in the project shall be controlled by cutting a sample from at least steel element. Likewise, a sample shall be cut off from one welded joint shown in the project and tested to control the compliance of steel with the project. Such cut off places shall be filled and repaired. One sample bolt shall be taken for tests from bolted connections. In case compliance with the project is confirmed, then mean strengths obtained from tests shall be taken as *existing steel strength* for the calculation of the strengths of elements. Otherwise, at least three samples shall be collected for tests, and the least favorable value shall be taken as the *existing steel strength*.

### **7.2.10. Limited Information Level in Prefabricated Reinforced Concrete Buildings**

Limited information level in prefabricated reinforced concrete constructions is not valid.

### **7.2.11. Medium Information Level in Prefabricated Reinforced Concrete Buildings**

Conditions mentioned in **7.2.5.1** are valid for the geometry of the building, except that the term “*reinforced*” shall be replaced with “*reinforced concrete*”. Conditions mentioned in **7.2.8.2** are valid for the details of elements, except that the term “*steel*” shall be replaced with “*prefabricated reinforced*”. Although conditions mentioned in **7.2.5.3** are valid for determining the characteristics of materials, number of samples of materials shall be half as much, provided that these samples shall not be less than three in total for each floor, and not less than 9 for the entire building.

### **7.2.12. Comprehensive Information Level in Prefabricated Reinforced Concrete Buildings**

Conditions mentioned in **7.2.6.1** are valid for the geometry of the building, except that the term “*reinforced*” shall be replaced with “*reinforced concrete*”. Conditions mentioned in **7.2.9.2** are valid for the details of elements, except that the term “*steel*” shall be replaced with “*prefabricated reinforced*”. Test shall be performed on at least one sample (borehole sample) collected for each 500 square meters for determining concrete compression strength. Total number of borehole samples collected from the building shall be at least 9. Of the mean ~~concrete~~ concrete compression strength obtained in tests and the value shown in the project for concrete compression strength, one with the smaller value shall be taken as the *existing concrete strength* in the calculation of the capacities of the elements. The reinforcement strength to be used in the calculations of capacities of the elements shall be the characteristic strengths shown in the project for that particular steel class.

### **7.2.13. Limited Information Level in Masonry Buildings**

**7.2.13.1 – Geometry of the Building:** In case the architectural projects are present, compliance of the existing geometry with the project shall be checked with visual examination. In case there are no architectural projects present, then the measured system drawings shall be prepared for the building. Information obtained must include the locations of masonry walls in each floor, their length, thickness, spaces, and heights of floors. Foundation system shall be determined by digging examination holes of sufficient numbers in- or outside the building.

**7.2.13.2 – Details:** Types of the roof and floor, methods of joining to walls, and conditions of girdles and headpieces shall be determined visually.

**7.2.13.3 – Characteristics of Materials:** Types of materials used in walls shall be visually determined by scraping the plaster on one part of the wall. Cutting resistances of walls for each type of wall given in **Section 5** shall be taken as the basis for building strength calculations.

### **7.2.14. Medium Information Level in Masonry Buildings**

In addition to the procedures for limited information level, joints of walls and stability of walls shall be investigated.

### 7.2.15. Comprehensive Information Level in Masonry Buildings

In addition to the procedures for medium information level, at least 3 samples of wall pieces shall be collected from the building to determine the characteristics of the materials used in walls, and the mean values obtained from tests performed on these shall be used for the calculations according to **Section 5**.

### 7.2.16. Information Level Coefficients

- (a) *Information Level Coefficients* to be applied to element capacities according to the level of information obtained from the building examined are given in **Table 7.1**.
- (b) Unless otherwise specified, material strengths shall not be divided by material coefficients given in the relevant design regulations. The *existing material strengths* shall be used in the calculations of element capacities.

**TABLE 7.1 – INFORMATION LEVEL COEFFICIENTS FOR BUILDINGS**

| Information level | Information level coefficient |
|-------------------|-------------------------------|
| Limited           | 0.75                          |
| Medium            | 0.90                          |
| Comprehensive     | 1.00                          |

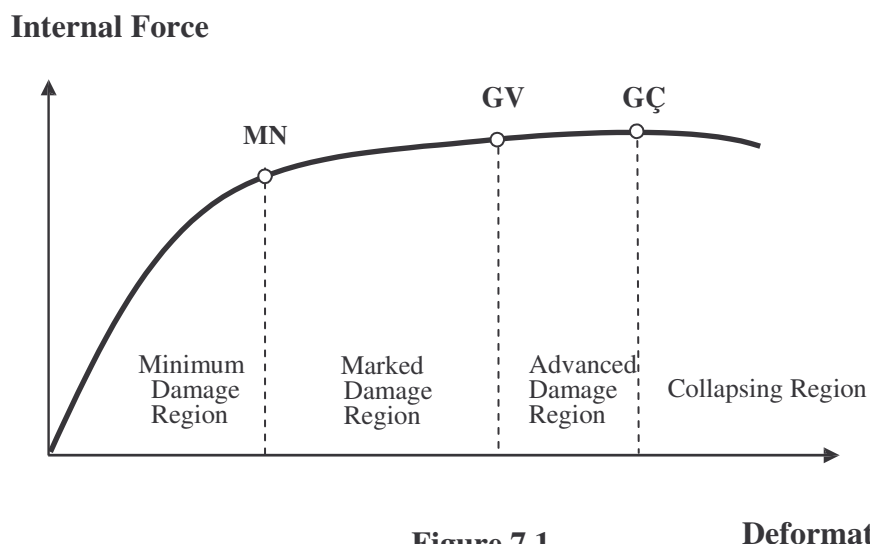
## 7.3. LIMITS OF DAMAGE IN CONSTRUCTION ELEMENTS AND AREAS OF DAMAGE

### 7.3.1. Damage limits in cross sections

Three limit conditions have been defined for ductile elements on the cross section. These are *Minimum Damage Limit* (MN), *Safety Limit* (GV) and *Collapsing Limit* (GÇ). Minimum damage limit defines the beginning of the behavior beyond elasticity, safety limit defines the limit of the behavior beyond elasticity that the section is capable of safely ensuring the strength, and collapsing limit defines the limit of the behavior before collapsing. This classification does not apply to elements damaged in a brittle condition.

### 7.3.2. Sectional Damaged Areas

Elements that the damages with critical sections do not reach MN are within the *Minimum Damage Region*, those in-between MN and GV are within *Marked Damage Region*, those in-between GV and GÇ are in *Advanced Damage Region*, and those going beyond GÇ are within *Collapsing Region* (**Figure 7.1**).



**Figure 7.1**

**Deformation**

### 7.3.3. Definition of Damages in Cross Sections and Elements

Damage regions that cross sections belong to shall be decided according to the comparison of the internal forces and / or deformation calculated using the methods described in 7.5 or 7.6 with the numerical values corresponding to cross section damage limits described in 7.3.1. Damage of the element shall be decided according to the cross section of the element that with greatest damage.

## 7.4. GENERAL PRINCIPLES AND RULES RELATED TO EARTHQUAKE DAMAGE

**7.4.1** – According to this section of the Regulation, objective of the earthquake damage calculation is to determine the earthquake performances of the existing or reinforced buildings. With this purpose, the linear elastic defined in 7.5 or the linear non-elastic calculation methods defined in 7.6 may be used. However, the performance evaluations conducted using these methods that are based on different theoretical perceptions should not be expected to give exactly the same results. The general principles and rules defined below apply for both methods.

**7.4.2** – Elastic (non-reduced) acceleration spectrum given in 2.4 will be used for the definition of the earthquake, however, the modifications introduced on 7.8 will be considered for different exceeding probabilities. Building Importance Factor defined in 2.4.2 will not be applied in the seismic calculation. ( $I=1.0$ ).

**7.4.3** – The seismic performance of the buildings will be evaluated under the collective impacts of the gravity loads and the seismic impacts affecting the building. Live gravity loads will be defined in a way that they will be in conformity with the masses taken into consideration in seismic calculation as per 7.4.7.

**7.4.4** – Seismic forces will be applied to both sides of the building in both directions, individually.

**7.4.5** – The ground parameters that will be used in seismic calculations will be determined according to **Section 6**.

**7.4.6** – Load-bearing system model of the building will be prepared accurately to calculate inner force, translocation and deformations that will arise in the structural components due to the seismic effects and the collective effects of the gravity loads.

**7.4.7** – The floor weights that will be considered in seismic calculation will be calculated according to 2.7.1.2 and the floor masses will be defined in conformity with the floor weights.

**7.4.8** – For the buildings where the slabs act as rigid diaphragms on the horizontal axis, two horizontal translocation per floor and independence levels for the rotations around the horizontal axis will be considered. Independence levels of the floors will be defined for the center of mass of each floor and additional eccentricity will not be applied.

**7.4.9** – The ambiguities regarding the load-bearing systems of the existing buildings will be reflected on the calculation methods via the *information level coefficients* defined in 7.2 in accordance with the scope of the information gathered regarding the building.

**7.4.10** – The columns that are defined as short column as per **3.3.8** will be defined in the load - bearing system model with their actual free lengths.

**7.4.11** – The conditions regarding the definition of the interaction diagrams of the reinforced concrete sections under uniaxial or biaxial bending and axial force effects are given below:

(a) Current strengths of the concrete and reinforcement steels determined in accordance with the information level defined in **7.2** will be taken as the basis in the analysis.

(b) Maximum pressure unit deformation of the concrete and the reinforcement steel can be taken as 0.003 and 0.01, respectively.

(c) Linearizing in a proper way, interaction diagrams can be modeled as multi-line or multi-axis diagrams.

**7.4.12** – While defining the component sizes of the reinforced concrete systems, confinement zones can be considered as infinitely rigid end zones.

**7.4.13** – *Active bending rigidities*  $(EI)_e$  of the cracked section will be used for the reinforced concrete components under bending effect. Unless a more accurate calculation as made, the value given below will be substituted for active bending rigidities:

(a) For beams:  $(EI)_e = 0.40 (EI)_o$

(b) For columns and frames,  $(EI)_e = 0.40 (EI)_o$  if  $N_D / (A_c f_{cm}) \leq 0.10$   
 $(EI)_e = 0.80 (EI)_o$  if  $N_D / (A_c f_{cm}) \geq 0.40$

Linear interpolation can be applied for the intermediate values of the axial pressure force  $N_D$ .  $N_D$  will be determined via a pre-gravity load calculation in which bending rigidities of the non-cracked sections  $(EI)_o$  are used and the total masses used and corresponding loads referred to in the seismic calculations are considered. And the calculation for the gravity load that provides the initial conditions for the seismic calculation will be made using the bending rigidity  $(EI)_e$  determined by the way given above and will be restructured in accordance with the loads in accordance with the masses which the seismic calculations are based on. The same rigidities will also be used in seismic calculation.

**7.4.14** – Platform concrete and the reinforcements in the platform can be included in the calculation of positive and negative plastic momentums of the beams with reinforced concrete platforms.

**7.4.15** – In the section capacity calculation in case of the reinforced concrete components with insufficient coupling or splicing length, yield tensile of the related reinforcement can be diminished by the amount of the shortage in the coupling and splicing length.

**7.4.16** – Ground features will be reflected in the analysis model for the cases where the deformation in the ground may affect the structural behavior.

**7.4.17** – Other modeling principles defined in **Section 2** are valid.

## 7.5. DETERMINING THE BUILDING PERFORMANCE IN EARTHQUAKE WITH LINEAR ELASTIC CALCULATION METHODS

### 7.5.1. Calculation Methods

Linear elastic calculation methods to be used for the determination of seismic performances of buildings are the calculations methods defined in 2.7 and 2.8. Additional rules as stated below shall be applied concerning these methods.

**7.5.1.1** – Equivalent seismic load method shall be implied to buildings not exceed 25 m and 8 storey and also have  $\eta_{bi} < 1.4$  buckling disorder calculated without considering joint eccentricity. On the calculation of total equivalent seismic load (ground shearing force) according to **Equation (2.4)**  $R_a=1$  is taken and right side of the equation is multiplied with  $\lambda$  factor.  $\lambda$  Factor is taken as 1.0 in one and two storey buildings except cellar and 0.85 in others.

**7.5.1.2** – While using the Mod Combination Method,  $R_a=1$  is taken in the **Equation (2.13)**. On the calculation of internal forces and capacities of elements that are adaptable to applied seismic direction and course, internal force directions obtained in the mode that is dominant in this direction shall be based.

### 7.5.2. Determination of Damage Level in the Structural Elements of Armoured Concrete Buildings

**7.5.2.1**– In the description of damage boundaries of ductile elements with linear elastic calculation methods, numerical values figured as ( $r$ ) shall be used in the effect / capacity ratios of beams, column and wall elements and sections of strengthened masonry filled walls.

**7.5.2.2** – Armoured concrete elements are classified as “ductile” if their fracture type is bending and “brittle” if it is shearing.

**a)** In order the beams, columns and walls to be considered as ductile element,  $V_e$  Shearing force calculated in accordance with the bending capacity in the critical sections of those element should not exceed the shearing capacity  $V_r$  calculated according to TS - 500 by using the values of current material strength that are comply with information level as defined in 7.2. Calculation of  $V_e$  for columns shall be made according to 3.3.7, for beams according to 3.4.5 and for walls according to 3.6.6 but in **Equation (3.16)**  $R_a= 1$  is taken. On the calculation of  $V_e$  for columns, beams and walls, bearing force moments shall be used instead of hardening bearing forces. In case the total shearing force calculated with gravity loads by taking  $R_a= 1$  is less than  $V_e$ , then this shearing force shall be used instead of  $V_e$ .

**b)** In order the beams, columns and walls to be considered as ductile element also it is necessary to provide  $H_w/\ell_w > 2.0$  condition.

**c)** Armoured concrete elements that are not provide the conditions for ductile element given in (a) and (b) are defined as *brittle damaged elements*.

**7.5.2.3** – Effect / capacity ratio of ductile beam, column and wall sections is determined by dividing the section moment calculated under seismic load by taking  $R_a= 1$  to over moment capacity. On the calculation of effect / capacity direction of the applied earthquake shall be taken into consideration.

a) Over moment capacity of section is the difference between bending moment capacity of the section and moment effect calculated on the section under gravity loads. Moment effect calculated under gravity loads in the supports of the beam can be reduced maximum 15 % according to retransfer principle.

b) Effect / capacity ratios of column and wall sections can be calculated in such a way as defined in **Information Annex 7A**.

c) Armoured concrete columns that provide the conditions of 3.3.4 in terms of the transverse reinforcement conditions in the coating zone, armoured concrete beams that provide 3.4.4 and armored concrete walls that provide 3.6.5.2 in the end zones are considered as “coated” and those that are not provide the conditions are considered as “un - coated” elements. It is necessary for the elements that are considered as “un - coated” for the coating elements being arranged with “special seismic hoops and crossties” according to 3.2.8 and reinforcement spaces of those should be provide the conditions as defined above.

**7.5.2.4** – Effect / capacity ratio of strengthened filled walls are the shearing force strength of shearing force calculated under the effect of earthquake. Shearing forces formed in the strengthened filled walls which are modeled with diagonal bars shall be taken into consideration as the horizontal concurrent of the axial force of the bar. Calculation of shearing force strength of the strengthened masonry filled walls is given in **Information Annex 7F**.

**7.5.2.5** – It is decided that the elements are located in which damage zone by comparing effect / capacity ratio of beam, column and wall sections and strengthened filled walls ( $r$ ) with boundary values given in **Table 7.2 - 7.5**. Besides, on the determination of damage zones of strengthened filled walls in the armoured concrete buildings boundary ratios of relative storey drift given in **Table 7.5** shall also be taken into consideration. Ratio of relative storey drift shall be obtained by dividing the maximum relative storey drift to storey height. For intermediate - values given in **Table 7.2 - 7.5** linear interpolations shall be applied.

**TABLE 7.2 – EFFECT / CAPACITY RATIOS ( $r$ ) DEFINES THE BOUNDARY OF THE DAMAGE FOR ARMoured CONCRETE BEAMS**

| Ductile Beams                 |               |                                 | Damage Boundary |     |    |
|-------------------------------|---------------|---------------------------------|-----------------|-----|----|
| $\frac{\rho - \rho'}{\rho_b}$ | Coating       | $\frac{V}{b_w d f_{ctm}}^{(1)}$ | MN              | GV  | GÇ |
| $\leq 0.0$                    | Available     | $\leq 0.65$                     | 3               | 7   | 10 |
| $\leq 0.0$                    | Available     | $\geq 1.30$                     | 2.5             | 5   | 8  |
| $\geq 0.5$                    | Available     | $\leq 0.65$                     | 3               | 5   | 7  |
| $\geq 0.5$                    | Available     | $\geq 1.30$                     | 2.5             | 4   | 5  |
| $\leq 0.0$                    | Not available | $\leq 0.65$                     | 2.5             | 4   | 6  |
| $\leq 0.0$                    | Not available | $\geq 1.30$                     | 2               | 3   | 5  |
| $\geq 0.5$                    | Not available | $\leq 0.65$                     | 2.5             | 4   | 6  |
| $\geq 0.5$                    | Not available | $\geq 1.30$                     | 1.5             | 2.5 | 4  |

(1)  $V_e$  shearing force shall be calculated according to 7.5.2.2 (a) in accordance with the direction of earthquake.

**TABLE 7.3 – EFFECT / CAPACITY RATIOS ( $r$ ) DEFINES THE BOUNDARY OF THE DAMAGE FOR ARMOURED CONCRETE COLUMNS**

| Ductile Columns           |               |                                 | Damage Boundary |     |     |
|---------------------------|---------------|---------------------------------|-----------------|-----|-----|
| $\frac{N}{A_c f_c}^{(1)}$ | Coating       | $\frac{V}{b_w d f_{ctm}}^{(2)}$ | MN              | GV  | GÇ  |
| $\leq 0.1$                | Available     | $\leq 0.65$                     | 3               | 6   | 8   |
| $\leq 0.1$                | Available     | $\geq 1.30$                     | 2.5             | 5   | 6   |
| $\geq 0.4$                | Available     | $\leq 0.65$                     | 2               | 4   | 6   |
| $\geq 0.4$                | Available     | $\geq 1.30$                     | 2               | 3   | 5   |
| $\leq 0.1$                | Not available | $\leq 0.65$                     | 2               | 3.5 | 5   |
| $\leq 0.1$                | Not available | $\geq 1.30$                     | 1.5             | 2.5 | 3.5 |
| $\geq 0.4$                | Not available | $\leq 0.65$                     | 1.5             | 2   | 3   |
| $\geq 0.4$                | Not available | $\geq 1.30$                     | 1               | 1.5 | 2   |

(1)  $N_K$  axial force shall be calculated according to **Information Annex 7A**.

(2)  $V_e$  shearing force shall be calculated according to **7.5.2.2 (a)** in accordance with the direction of earthquake.

**TABLE 7.4 – EFFECT / CAPACITY RATIOS ( $r$ ) DEFINES THE BOUNDARY OF THE DAMAGE FOR ARMOURED CONCRETE WALLS**

| Ductile Walls | Damage Boundary |    |    |
|---------------|-----------------|----|----|
| Coating       | MN              | GV | GÇ |
| Available     | 3               | 6  | 8  |
| Not Available | 2               | 4  | 6  |

**TABLE 7.5 – EFFECT / CAPACITY RATIOS ( $r$ ) DEFINES THE BOUNDARY OF THE DAMAGE FOR STRENGTHENED FILLED WALLS AND RATIOS OF RELATIVE STOREY DRIFT**

| Ratio range of $\ell_{wall} / h_{wall}$<br>0.5 - 2.0 | Damage Boundary |        |    |
|--|-----------------|--------|----|
|  | MN              | GV     | GÇ |
| Effect / Capacity Ratios ( $r$ )                     | 1               | 2      | -  |
| Ratios Of Relative Storey Drift                      | 0.0015          | 0.0035 | -  |

**7.5.2.6** – In the joints of armoured column - beam, shearing force to be calculated from **Equation (3.11)** and affects the joint for all boundary conditions should not exceed the shearing forces given **3.5.2.2**. However, in **Equation (3.11)**  $V_e$  calculated without considering hardening according to **3.3.7** shall be used instead of  $V_{kol}$ , whereas in the calculation of strength in **Equation (3.12)** and **Equation (3.13)** *current concrete strength* determined according to information level in **7.2** shall be used instead of  $f_{cd}$ . In case the shearing force of joint exceeds the shearing strength, column - bema confinement zone shall be defines as *brittle damaged element*.

### 7.5.3. Control of Relative Storey Drifts

In the calculation made with linear elastic methods in each earthquake direction, relative storey drifts of column, beam or walls in each storey of the building shall not exceed the value given in **Table 7.6**.  $\delta_{ji}$  indicates the relative storey drift calculated as a replacement difference between bottom and top ends of the  $j$ 'th column or wall in  $i$ 'th storey whereas  $h_{ji}$  indicates the height of the relevant element.

**TABLE 7.6 - BOUNDARIES OF RELATIVE STOREY DRIFT**

| Ratio of Relative Storey Drift | Damage Boundary |      |      |
|--------------------------------|-----------------|------|------|
|                                | MN              | GV   | GC   |
| $\delta_{ji} / h_{ji}$         | 0.01            | 0.03 | 0.04 |

## **7.6. DETERMINING THE SEISMIC PERFORMANCE OF THE BUILDING USING INELASTIC LINEAR METHODS**

### **7.6.1. Definition**

The aim of the inelastic linear measurement methods to be used for determining the structural performances of the buildings under seismic effect and for the strengthening analyses is enabling the measurement of the plastic deformation volitions regarding the ductile behavior and inner force volitions concerning the brittle behavior for a given earthquake. Afterwards, the magnitudes of the mentioned volitions are compared with the deformation and inner force capacities that are defined in this section and structural performance evaluation shall be conducted both at sectional and building level.

### **7.6.2. Scope**

The inelastic linear analysis methods included in the scope of this Regulation are *Incremental Equivalence Seismic Load Method*, *Incremental Mode Combination Method* and *Measurement within the Scope of Time Definition Method*. First two are the methods that shall be used for the *Incremental Repulsion Analysis* that is taken as a basis for determining the non - linear seismic performances and for the strengthening measurements included in this Regulation.

### **7.6.3. The Procedure for the Performance Evaluation via Incremental Repulsion Analysis**

The steps that shall be taken during the inelastic non - linear performance evaluation conducted applying the *Incremental Repulsion Analysis* is summarized below.

- (a) In addition to the general principles and rules defined in **7.4**, the rules defined **7.6.4** shall be followed to idealize the non-linear behavior of the load-bearing system and construct the analysis model.
- (b) Before applying the incremental repulsion analysis, a non-linear static analysis in which the gravity loads that are in accordance with the masses are taken into consideration shall be conducted. The results of this analysis shall be considered as the initial conditions of the incremental repulsion analysis.
- (c) In case the incremental repulsion analysis is conducted via applying the *Incremental Equivalence Seismic Load Method* defined in **7.6.5**, the “modal capacity diagram” belonging to the primary (dominant) mode the coordinates of which are defined as “modal translocation – modal acceleration” shall be derived. The *modal translocation volition* belonging to the primary (dominant) mode shall be set taking the elastic behaviors spectrum defined in **2.4** and the modifications applied on this spectrum in **7.8** for different exceeding probabilities together with the mentioned diagram into consideration. In the final step, the translocation, plastic deformation (plastic rotation) and inner force volitions that corresponds to the modal translocation volition shall be calculated.

(d) In case the incremental repulsion analysis is conducted via applying the *Incremental Mode Combination Method* defined in 7.6.6, modal translocation volitions will as well be derived together with the “*modal capacity diagrams*” for all modes taken into consideration and the translocation, plastic deformation (plastic rotation) and inner force volitions that will arise accordingly in the load-bearing system shall be calculated.

e) The plastic rotation volitions that are calculated out of ductile sections shall be utilized for deriving plastic bending volitions and total bending volitions in accordance with 7.6.8. Afterwards, the unit deformation volitions of reinforced concrete sections, concretes and reinforcement steels shall be calculated accordingly. Performance evaluations regarding the ductile behavior in sectional level shall be conducted via comparing the mentioned values of volition with the unit deformation capacities defined in 7.6.9 for various damage thresholds. Moreover, the deformation volitions calculated in terms of relative storey drifts of the strengthened filled wall shall be compared with the deformation capacities defined in 7.6.10. The shear force volitions derived from the analysis on the other hand shall be compared with the capacities defined in 7.6.11 and performance evaluation regarding the brittle behavior shall be conducted at sectional level.

#### 7.6.4. Idealizing the Inelastic Non - linear Behavior

7.6.4.1 – In order to idealize the inelastic non-linear behavior in respect to the materials, the models that are proved to be valid in the literature can be utilized. However, due to its practicability and extensiveness in engineering practices, *stacked plastic behavior model* is taken as a basis point for the inelastic non-linear analysis applied in the following sections. In this model that corresponds to the *plastic support hypothesis* in case of simple bending, it is assumed that the plastic deformations are formed in an evenly-distributed manner all along the finite-length zones in which the inner forces in the beam, column and frame-type load-bearing components idealized as stick components reach to their plastic capacities. The length of the *plastic deformation zone* referred to as *plastic support length* ( $L_p$ ) shall be taken as the half of the section length ( $h$ ) in the active direction ( $L_p = 0.5 h$ ). For the frames where  $H_w / \ell_w \leq 2.0$ , the plastic deformations under the bending effect will not be taken into consideration.

7.6.4.2 – The length of the plastic deformation zones that undergo plastic deformation only under axial force shall be assumed to be equal to the open length of the related component.

7.6.4.3 – The *plastic section* that represents the stacked plastic deformation should be positioned in the middle of the plastic deformation zone that is theoretically defined in 7.6.4.1. However, the approximate idealizations mentioned below can be allowed for practical applications.

(a) In columns and beams, the plastic sections can be placed right out of the column-beam confinement zone, in other words on the end zones of the net ports of columns or beams. However, the fact that plastic hinges may be formed as well in the beam bays due to the effects of the gravity loads should be taken into consideration.

(b) In reinforced concrete frames, plastic sections may be allowed to be placed on the bottom end zone of the frame zone for each floor. U, T, L or box-section frames should be idealized as a single frame all arms of which works cooperatively. In case there are rigid frames in the basement floors of the buildings, the plastic sections of this frames following on to the upper stairs of the building should be placed starting above from the basement.

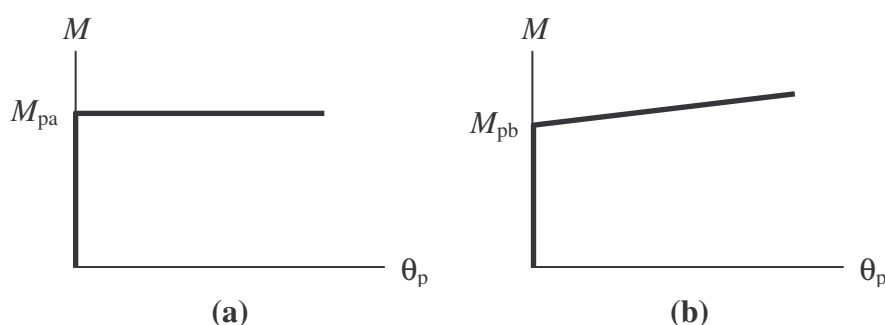
**7.6.4.4** – The interaction diagrams that are defined in accordance with the rules defined in **7.4.11** shall be taken as the yield surfaces for the reinforced concrete sections that are plasticized under the effect of uniaxial or biaxial bending and axial force. When they are linearized in accordance with **7.4.11(c)**, the yield surfaces can be modeled as *yield lines* and *yield planes* in case of two-dimensional and three-dimensional behaviors, respectively.

**7.6.4.5** – The information in the following paragraphs shall be considered regarding the inner force – plastic deformation correlations that shall be used in repulsion analysis model.

(a) The strain hardening effect (rise in the momentum in accordance with the rise in the plastic rotation) in the inner force – plastic deformation correlations can be left approximately (**Figure 7.2a**).

In such a case, the condition of the plastic deformation vector being approximately perpendicular to the yield surface, provided that the inner forces remain over the yield surface, shall be taken into consideration during the steps following the plasticizing of the sections under the impact of the uniaxial or biaxial bending and axial force.

(b) In case the strain hardening effect is taken into consideration (**Figure 7.2b**), the requirements the inner forces and the plastic deformation vector should satisfy during the repulsion steps following the plasticization in the sections under the effect of uniaxial or biaxial bending and axial force shall be defined in accordance with an appropriate strain hardening model to be obtained from the related literature.



**Figure 7.2**

**7.6.4.6** – Any filled wall that is strengthened in accordance with **7.10.4** shall be idealized as per **7.6.4.2** as a two - edge hinged diagonal equivalent pressure and / or draw bar as defined in the **Information Annex 7F**. The axial rigidity and axial yield strength regarding the initial linear elastic behavior of the equivalent stick that shall be modeled as an elasto - plastic (non-hardening) component in the repulsion analysis shall be determined as per the **Information Annex 7F**. The shear strength defined for the wall is the horizontal component of the axial yield strength of the diagonal equivalent pressure stick. When necessary, yield strength of the diagonal equivalent draw bar shall be obtained from **Equation (7F.6)**.

### **7.6.5. Repulsion Analysis with Incremental Equivalent Seismic Load Method**

**7.6.5.1** – The aim of the *Incremental Equivalent Seismic Load Method* is implementing the *non-linear repulsion analysis* under the effect of equivalent seismic loads that are gradually increased in a monotonic way up to the seismic volition threshold provided that it shall be proportional to the vibration mode type of the primary (dominant in the seismic direction). In each step of the repulsion analysis following the gravity load analysis, the translocation, plastic deformation and inner force increments and cumulative values of these increments and finally the maximum values corresponding to the seismic volition shall be calculated.

**7.6.5.2** – To be able to use the Incremental Equivalent Seismic Load Method, the number of floors of the building excluding the basement should not be above 8 and the bending irregularity coefficient that is calculated in accordance with the elastic linear behavior without considering additional eccentricity should meet the condition  $\eta_{bi} < 1.4$  for each floors. Moreover, in accordance with the earthquake taken into consideration, the ratio of the active mass of the primary (dominant) vibration mode calculated taking the linear elastic behavior as a basis point to the total mass of the building (except for the masses of the basement floors covered by the rigid frames) should be above 0.70.

**7.6.5.3** – During incremental repulsion analysis, the distribution of the equivalent seismic load can be assumed to remain constant, independent of the plastic section formations in the load-bearing system. In such a case, load distribution shall be determined in a way that it shall be proportional to the value derived by multiplying the natural vibration mode shape magnitude of the primary (dominant in the seismic direction) that is computed for the linear elastic behavior at the first step of the analysis with the magnitude of the related mass. In the buildings where floor slabs are idealized as rigid diaphragms, two perpendicular horizontal drifts in the center of mass of each floor and the rotation around the vertical axis passing through the center of mass shall be considered as the magnitudes of the primary (dominant) natural vibration mode shapes.

**7.6.5.4** – By means of the repulsion analysis conducted in accordance with the constant load distribution defined in **7.6.5.3**, the *repulsion curve* the coordinates of which are “*top translocation – ground shear force*” shall be obtained. Top translocation is the translocation that is calculated in each repulsion step and that takes place in the center of mass of the top floor of the building for the earthquakes in the direction  $x$  that are taken into consideration. And the ground shear force is the sum of the equivalent seismic loads of each step for the earthquake in the direction of  $x$ . With the help of the coordinate transformation applied to the repulsion curve, the *modal capacity diagram* the coordinates of which are “*modal translocation – modal acceleration*” can be obtained via the method defined below:

(a) The modal acceleration for the primary (dominant in the seismic direction) mode for the (i)’ th step,  $a_1^{(i)}$ , is derived with the following method:

$$a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}} \quad (7.1)$$

(b) The following equation can be used to calculate the modal translocation for the primary (dominant in the seismic direction) mode for the (i)’ th step,  $d_1^{(i)}$ :

$$d_1^{(i)} = \frac{u_{xN1}^{(i)}}{\Phi_{xN1} \Gamma_{x1}} \quad (7.2)$$

The contribution coefficient for the primary (dominant in the seismic direction) mode,  $\Gamma_{x1}$ , is computed in the following way, utilizing the values  $L_{x1}$  and  $M_1$  that are defined for the linear elastic behavior of the load-bearing system for the earthquake in the direction  $x$  and is given in **Chapter Equation (2.15)**:

$$\Gamma_{x1} = \frac{L_{x1}}{M_1} \quad (7.3)$$

**7.6.5.5** – As an alternative to **7.6.5.3**, during the incremental repulsion analysis, equivalent seismic load distribution can be considered as *varying* in each repulsion step compared to the previous steps. In such a case, the load distribution shall be defined as proportional to the value obtained from multiplying the magnitudes of the primary (dominant) natural vibration mode shapes that are calculated considering all plastic sections formed in the load-bearing system before each repulsion step with the magnitude of the related mass. In the buildings where floor slabs are idealized as rigid diaphragms, the magnitudes of the primary (dominant) natural vibration mode shapes shall be defined as given in **7.6.5.3**.

**7.6.5.6** – The maximum modal translocation of the primary (dominant) mode, i.e. *modal translocation volition* shall be set taking the elastic behaviors spectrum defined in **2.4** and the modifications applied on this spectrum in **7.8** for different exceeding probabilities together with the modal capacity diagram derived as a result of the repulsion analysis as per the information given in **7.6.5.4**. As a definition, modal translocation volition  $d_1^{(p)}$  is equivalent with the *non - linear spectral translocation*  $S_{di1}$ .

$$d_1^{(p)} = S_{di1} \quad (7.4)$$

The calculations regarding the determination of non-linear spectral translocation  $S_{di1}$  is given in **Information Annex 7C**.

**7.6.5.7** – By substituting the modal translocation volition  $d_1^{(p)}$  that is determined according to **Equation (7.4)** for the final repulsion step  $i = p$  in **Equation (7.2)**', the top the top translocation volition  $u_{xN1}^{(p)}$  for the x seismic direction, shall be obtained.

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} \quad (7.5)$$

All other volition magnitudes (translocation, deformation and inner force volitions) corresponding to this value shall be obtained from the existing repulsion analysis file or shall be computed with a new repulsion analysis that shall be conducted until the top translocation volition is reached.

## **7.6.6. Repulsion Analysis with Incremental Mode Combination Method**

The aim of the *Incremental Mode Combination Method* is incrementally implementing the *Mode Combination Method* taking modal translocations that are gradually and monotonically increased in a way that shall be proportional to the sufficient number of natural vibration mode shapes representing the load-bearing system behavior and that are scaled in a way that they shall be in harmony with each other or taking the modal seismic loads that shall be in harmony with the mentioned modal. Such repulsion analysis method that is based on the “*step by step linear elastic*” behavior in the load - bearing system for each repulsion step between the formations of two sequential plastic sections is explained in **Information Annex 7D**.

## **7.6.7. Calculation with the Non-linear within the Scope of Time Definition Method**

**7.6.7.1** – The aim of the *Calculation with the Non - linear within the Scope of Time Definition Method* is integrating the movement equation of the system step by step taking the non-linear behavior of the load-bearing system. During the analysis, translocation, plastic deformation and inner force that come up in the system in each increment and the maximum values of these magnitudes that correspond to the seismic volition are calculated.

**7.6.7.2** – The *artificial, recorded or simulated* earth movements that shall be used in the analysis to be conducted in the time definition scope shall be determined as per **2.9.1** and **2.9.2** and **2.9.3** shall be taken into consideration through the analysis.

### **7.6.8. Determining the Unit Deformation Volitions**

**7.6.8.1** – The *plastic bending volition* dependent on the  $\theta_p$  plastic rotation volition that shall be obtained in any section as a result of the repulsion analysis conducted as per **7.6.5** or **7.6.6** or obtained as a output information onto the calculation conducted within the scope of time definition as per **7.6.7** shall be calculated as follows:

$$\phi_p = \frac{\theta_p}{L_p} \quad (7.6)$$

**7.6.8.2** – The *total bending volition*  $\phi_t$  of the section shall be obtained adding the  $\phi_y$  *equivalent yield bending* that is defined with the two-line momentum-bending relationship obtained from the analysis conducted under the axial force volition of the section by means of using a reinforcement steel model that as well considers the strain hardening together with a concrete model chosen in accordance with the aim to the  $\phi_p$  plastic bending volition defined with the **Equation (7.6)**:

$$\phi_t = \phi_y + \phi_p \quad (7.7)$$

The unit pressure deformation volition in the reinforced concrete systems and the unit deformation volition in the reinforcement steel shall be calculated applying momentum-bending analysis in accordance with the total bending volition defined with **Equation (7.7)**.

**7.6.8.3** – If any other choice is not introduced, **Information Annex 7B** can be utilized for coated or uncoated concrete and reinforcement steel models.

### **7.6.9. Section Unit Deformation Capacities of Reinforced Concrete Components**

**7.6.9.1** – The seismic volitions obtained in accordance with **7.6.8** in terms of unit deformation volitions for concrete or reinforcement steel shall be compared with the unit deformation capacities given below so as to determine the performance of the load-bearing systems at sectional level.

**7.6.9.2** – The upper bounds (capacity) of deformation for different sectional damage thresholds for the ductile load-bearing system components that undergo plastic deformations are defined below:

The transverse reinforcements that consider the following:

**(a)** For *Minimum Sectional Damage Boundary (MN)*, upper bounds of the concrete unit pressure deformation in the outmost fiber of the section and the reinforcement steel unit deformation volitions:

$$(\epsilon_{cu})_{MN} = 0.0035 \quad ; \quad (\epsilon_s)_{MN} = 0.010 \quad (7.8)$$

**(b)** For *Section Security Bound (GV)*, upper bounds of the concrete unit pressure deformation in the outmost fiber of hoop and the reinforcement steel unit deformation volitions:

$$(\epsilon_{cg})_{GV} = 0.0035 + 0.01 (\rho_s / \rho_{sm}) \leq 0.0135 \quad ; \quad (\epsilon_s)_{GV} = 0.040 \quad (7.9)$$

(c) For Section Collapse Bound (GC), upper bounds of the concrete unit pressure deformation in the outmost fiber of hoop and the reinforcement steel unit deformation volitions:

$$(\varepsilon_{cg})_{GC} = 0.004 + 0.013 (\rho_s / \rho_{sm}) \leq 0.018 \quad ; \quad (\varepsilon_s)_{GC} = 0.060 \quad (7.10)$$

Should be regulated as “*special earthquake hoops and crossties*” as per **3.2.8**.

#### **7.6.10. Deformation Capacities of the Strengthened Filled Walls**

The allowed boundary values (capacities) for the relative floor drifts that are obtained onto the repulsion analysis conducted via modeling the filled walls that are strengthened in accordance with **7.10.4** and **Information Annex 7F** together with the columns and beams surrounding the walls in accordance with **7.6.4.6** are listed in the second row of **Table 7.5**.

#### **7.6.11. Shear Force Capacities of the Reinforced Concrete Load-Bearing System Components**

**7.6.11.1** – The shear strengths that shall be used in brittle bending controls of all reinforced concrete load-bearing components except for the confinements of columns and beams shall be determined in accordance with the TS-500 standard. The *current strength* values that are defined in accordance with the information levels set in **7.2** shall be utilized in shear force calculations. The components the shear force strengths of which are smaller than their shear force volition shall be defined as *brittle damaged components*.

**7.6.11.2** – The shear force volitions that shall be calculated using **Equation (3.11)** for reinforced concrete column-beam confinements should not exceed the shear strength given in **3.5.2.2**. However, in **Equation (3.11)** the shear force volition calculated in the non-linear analysis for the related column shall be used instead of  $V_{kol}$ , and the *current concrete strength* that shall be determined in accordance with the information level defined in **7.2** shall be taken as a basis point in the strength calculation in **Equation (3.12)** or **Equation (3.13)** instead of  $f_{cd}$ . In case the shear force volition exceeds the shear strength, column-beam confinement section shall be defined as *brittle damaged component*.

### **7.7. DETERMINING THE SEISMIC PERFORMANCE OF THE BUILDING**

#### **7.7.1. Seismic Performance of Reinforced Concrete Buildings**

The seismic performance of the buildings are related to the condition of the damages that are expected to come out under the effect of the earthquakes applied and is defined taking four different damage levels as basis. The seismic performance of the buildings are determined applying the calculation methods defined in **7.5** and **7.6** and deciding on the damage areas of the components. The rules to be applied so as to determine the seismic performance of the buildings are given below. The rules given here are related to the reinforced concrete and prefabricated buildings. The rules to be applied for masonry buildings are given in **7.7.6**.

#### **7.7.2. Ready for Use Performance Level**

Onto the results of the calculations regarding all earthquakes applied in any floors, at most 10 % of the beams exceed the *Significant Damage Zone* and all other load-bearing components remain in the *Minimum Damage Zone*. Such buildings can be agreed to be in the *Ready for Use Performance Level* provided that the brittle damaged components, if any, are strengthened.

### 7.7.3. Life Safety Performance Level

The buildings that satisfy the conditions mentioned below can be agreed to be in *Life Safety Performance Level* provided that the brittle damaged components, if any, are strengthened:

- (a) As the result of the calculations made for each earthquake direction applies on each floor, at most 30 % of the beams except for the secondary ones (that does not take place in the horizontal load-bearing system) and at most the proportion of the columns defined in paragraph (b) can exceed the *Advanced Damage Zone*.
- (b) The total contribution of the columns in the *Advanced Damage Zone* to the shear force that is borne by the columns in each floor should not exceed 20 %. For the top floor, the ratio of the total shear forces of the columns in the *Significant Damage Zone* to the total shear forces of all the columns at that floor can be at most 40 %.
- (c) All other load - bearing components are in *Minimum Damage Zone or Significant Damage Zone*. However, the shear forces borne by the columns which exceeds the *Minimum Damage Bound* both in upper and lower sections for any floor should not be more than 30 % of the shear force borne by all columns of the floor (In calculation with linear elastic method, the columns that satisfy **Equation (3.3)** both in upper and lower confinement points are not included in this calculation).

### 7.7.4. Pre - Collapse Performance Level

The buildings that meet the conditions given below are agreed to be in the *Pre - Collapse Performance Level* provided that the fact that all components that are brittle damaged are in the *Collapse Zone*.

- (a) Onto the results of the calculations regarding all earthquakes applied in any floors, at most 20 % of the beams except for the secondary ones (that does not take place in the horizontal load-bearing system) can enter the *Collapse Zone*.
- (b) All other load-bearing components are in *Minimum Damage Zone, Significant Damage Zone or in the Advanced Damage Zone*. However, the shear forces borne by the columns which exceeds the *Minimum Damage Bound* both in upper and lower sections for any floor should not be more than 30 % of the shear force borne by all columns of the floor (In calculation with linear elastic method, the columns that satisfy **Equation (3.3)** both in upper and lower confinement points are not included in this calculation).
- (c) Usage of the building under these circumstances poses threats towards life safety.

### 7.7.5. Collapse Level

If the building does not satisfy the *Pre - Collapse Performance Level*, it is in the *Collapse Level*. Usage of the building under these circumstances poses threats towards life safety.

### 7.7.6. Determining the Seismic Performance of Masonnary Buildings

The performance level of the masonnary buildings shall be determined onto the evaluations made in accordance with 7.2 and the calculations made in accordance with **Section 5**. If the shear strength of all walls of the masonnary building in both two directions is enough to bear the shear forces that form under the effects of the earthquakes applied, the building is decided to satisfy *Ready for Usage Performance Level*. If the contribution of the walls that do not satisfy this condition due to the earthquakes applied in any floor to the floor shear force is below 20 %, the building shall be decided to satisfy the *Life Safety Performance Level*. Only the walls with low performance should be strengthened at least up to the level defined in 7F.2. Except for these situations the buildings are assumed to be in the *Collapse Level*.

## 7.8. TARGETED PERFORMANCE LEVELS FOR THE BUILDINGS

**7.8.1.** The acceleration spectrum defined for to-be-built buildings in 2.4 takes the earthquakes for which the possibility to be exceeded in 50 years is 10 % according to 1.2.2. In addition to this earthquake level, two different seismic given below are defined to evaluate the existing buildings and to be utilized in strengthening designs:

(a) The coordinates of the acceleration spectrum of the earthquakes for which the possibility to be exceeded in 50 years is 50 % shall be taken as approximately the half of the coordinates of the spectrum defined in 2.4.

(b) The coordinates of the acceleration spectrum of the earthquakes for which the possibility to be exceeded in 50 years is 2 % is decided to be taken as approximately 1.5 times of the coordinates of the spectrum defined in 2.4.

**7.8.2.** The earthquakes levels that the seismic performances of existing or to-be-strengthened buildings shall be based on and the minimum performance targets for the buildings at the mentioned earthquakes levels are given in **Table 7.7**.

**TABLE 7.7 – MINIMUM BUILDING PERFORMANCE TARGETS ANTICIPATED FOR DIFFERENT EARTHQUAKE LEVELS**

| <i>The usage purpose and the Type of the Building</i>  | <i>Probability for the Earthquake to be exceeded</i> |                         |                        |
|--|--|-------------------------|------------------------|
|  | <i>50 % in 50 years</i>                              | <i>10 % in 50 years</i> | <i>2 % in 50 years</i> |
| <b>The buildings that should be used after earthquakes:</b> Hospitals, health facilities, fire stations, communications and energy facilities, transportation stations, provincial or district administrative bodies, disaster management centers etc. | –  | <b>RU</b>               | <b>LS</b>              |
| <b>The buildings that people stay in for a long time period:</b> Schools, accommodations, dormitories, pensions, military posts, prisons, museums, etc.  | –  | <b>RU</b>               | <b>LS</b>              |
| <b>The buildings that people visit densely and stay in for a short time period:</b> cinema, theatre and concert halls, culture centers, sports facilities  | <b>RU</b>  | <b>LS</b>               | –                      |
| <b>Buildings containing hazardous materials:</b> The buildings containing toxic, flammable and explosive materials and the buildings in which the mentioned materials are stored.  | –  | <b>RU</b>               | <b>PC</b>              |
| <b>Other buildings:</b> The buildings that does not fit the definitions given above (houses, offices, hotel, tourist facilities, industrial buildings, etc.)   | –  | <b>LS</b>               | –                      |

RU: Ready for Usage; LS: Life Safety; PC: Pre-Collapse (See 7.7)

## 7.9. STRENGTHENING THE BUILDINGS

Strengthening the buildings covers applications such as eliminating the defects that will lead to seismic damages, adding new components that will contribute to the enhancement of earthquake safety, diminishing the mass, improving the seismic behavior of existing components and maintaining the continuity of force distribution.

### **7.9.1. Determining the Seismic Safety of the Buildings Strengthened**

The calculation methods and evaluation basis given in this section shall be used in determining the seismic performance of the buildings and components strengthened.

### **7.9.2. The Design of the Components to be added to the Buildings**

The special rules given in this section together with the rules in **Section 3** and / or **Section 4** and other standards and regulations in effect shall be followed regarding the design of the components to be added to the buildings with strengthening purposes.

### **7.9.3. Strengthening Types**

Strengthening applications shall be evaluated under two different scopes, at component level and building system level, for each load-bearing system type.

**7.9.3.1** – The applications applied to improve the strength and deformation capacities of the seismic load-bearing building components as beam, column, frame and confinement zones are defined as component strengthening.

**7.9.3.2** – The applications introduced with the aims of improving the strength and deformation capacity of the load-bearing system of the building and ensuring the continuity of the inner force distribution, adding new components to the building, strengthening the confinement zones and diminishing the mass of the building to minimize the seismic effects are called *system strengthening*.

## **7.10. STRENGTHENING REINFORCED CONCRETE BUILDINGS**

The component and system strengthening methods given in this section covers techniques frequently used in practice. However, the strengthening methods that are not included in this section can as well be applied provided that the general perception and principles of the section are followed.

### **7.10.1. Coating the Columns**

Following methods can be used to improve shear and pressure strength of the columns to enhance the ductility of the columns and to eliminate the weaknesses of the lap splicing. The bending capacity of the columns can't be improved using these methods.

**7.10.1.1** – *Reinforced Concrete Coating*: It shall be applied via peeling off the rusted area or rumpling the surface of the existing column. Reinforced concrete coating should have the thickness that is enough to be able to install horizontal and vertical reinforcements, pour concrete and sustain the minimum rust area. Minimum coating thickness is 100 mm. Concrete coating should begin from the top of the base floor slab and ends at the bottom of the top floor slab. For the coating activity that shall be held with the aim of improving the axial pressure strength, the rules defined in **3.3.4.2** shall be followed throughout the total height of the column for the transverse reinforcement in the coating concrete. Coated gross sectional dimensions and the design strength of the coating concrete shall be used in calculating the shear and pressure strength of the coated column, but the strengths derived shall be diminished via multiplying the values with 0.9.

**7.10.1.2 – Steel Coating:** Steel coating is constituted by installing four longitudinal angle bracelets to the edges of rectangular reinforced concrete columns and welding the angle bracelets to the horizontal plates positioned within specific spacing. There shouldn't be any gap between the angle bracelets and reinforced concrete surfaces. Horizontal plates should be continuous in all four surfaces. For the steel coating to improve the axial load capacity of the column, the angles should be continuous (the gaps should be eliminated) between the floor and ceiling slabs and the fact that pressure transfer is enabled via the cap plates should be shown in the calculations. If required, pre-loading can be applied to the angle bracelets and the axial pressure load on the existing concrete column section resulting from the gravity load can be diminished. The additional shear strength introduced with the steel coating shall be calculated using **Equation (7.11)**.

$$V_j = \frac{t_j b d}{s} f_{yw} \quad (7.11)$$

In **Equation (7.11)**,  $t_j$ ,  $b$ , and  $s$  represents the thickness, width and the spacing of the horizontal plates and  $d$  represents the useful height of the section. To eliminate the weaknesses of lap splicing using steel coating, the length of the coating should be at least 50 % higher than that of the splicing zone and the steel coating should be tightened with at least 16 mm-diameter bolted anchor that shall be fitted to the reciprocal surfaces of the column in the reinforcement splicing zone. In case the splicing adjunction is applied in the floor edge of the column, at least two rows of bolted anchoring shall be applied in 250 and 500 mm above the slab floor, respectively.

**7.10.1.3 – Coating with fiber polymer (FP):** Coating is enabled via coating and fixating the FP layer around the column provided that the fibers are parallel to the transverse reinforcements. By means of FP coating, ductility capacity, shear and pressure strengths of the reinforced concrete columns and reinforcement coupling strength, when the splicing length of the longitudinal reinforcement is insufficient, are improved. For the FP method, full coating (coating the whole section circle) method should be applied and at least 200 mm splicing should be made after coating. FP shall be applied for the rectangular columns via rolling the edges of the beams at least with 30 mm diameter. FP application should be held in accordance with the method recommended by the producer firm. The calculation methods regarding the improvement in the shear, axial pressure and coupling strengths and ductility of the columns coated with FP are given in **Information Appendix 7E**.

## **7.10.2. Improving the Bending Capacity of the Columns**

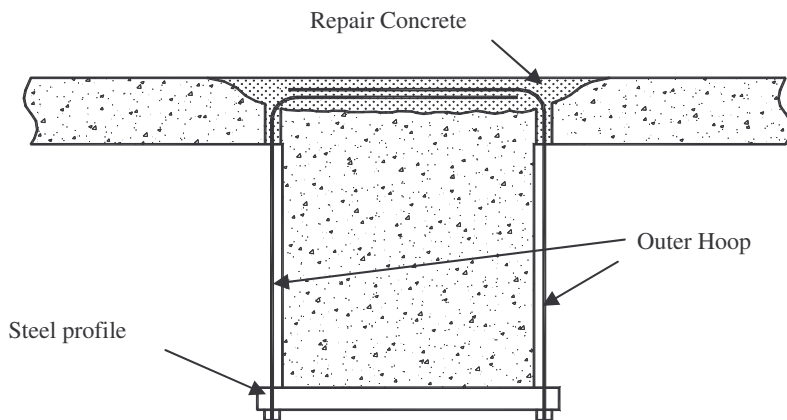
Columns sections can be expanded to improve the bending capacity of the columns. This application as well improves the shear and pressure strengths of the columns. The continuity of the longitudinal reinforcements attached to the column expanded should be ensured. Longitudinal reinforcements will pass through the holes bore on floor slabs and shall be fitted bending into the slot bore in the upper surface of the slab. The necessary horizontal reinforcement shall be fitted via boring holes in beams in the column-beam splicing zones or applying anchoring to the beams. The expanded section of the column shall be coated with transverse reinforcement in accordance with **3.3.4**. The rusting area of the expanded column shall be thick enough to cover the horizontal and vertical reinforcement fitted. In other words, the plaster layer on the surface of the existing column shall be peeled of and concrete surfaces shall be roughened up to ensure the adherence of the old and the new concrete. Coated gross sectional dimensions and the design characteristics of the coating concrete shall be used in calculating the shear and pressure strength of the coated column, but the strengths derived shall be diminished via multiplying the values with 0.9.

### 7.10.3. Coating the Beams

The purpose of coating the reinforced concrete beams is improving the shear strengths and in some cases the ductility capacities of the beams. Bending capacities of the beams can't be improved using the methods given below.

**7.10.3.1 – Adding External Hoop:** In the beam support sections where the shear strength is insufficient, required number of hoops shall be added externally to the both surfaces of the beam, as shown in **Figure 7.3**. The sticks attached to a steel profile, positioned under the beam, with bolts shall be passed through the holes bore on upper slabs and shall be fitted bending into the slot bore in the upper surface of the slab. Afterwards, the holes formed on the concrete shall be filled with concrete. This method can as well be applied implementing different details under the same principles. The shear force of the beams improved by the addition of external hoops shall be calculated as per TS-500 standard. Externally added hoops doesn't have coating effect, they don't improve beam ductility. During the application, profiles and bolts should be protected from the outside effects.

**7.10.3.2 – Coating with fiber polymer (FP):** Full coating (coating the whole section circle) method should be used for improving the beam ductility and shear force with FP coating. The beam shear force strengthened with FP can be calculated using **Equation (7E.1)** given in **Information Annex 7E**. In case non - continuous FP (in form of strings), the spacing between the strings shouldn't exceed the value  $(w_f + d/4)$ . FP shall be applied via rolling the edges of the beams at least with 30 mm diameter. For the FP coating, at least 200 mm splicing should be made after coating. FP application should be held in accordance with the method recommended by the produced firm.



### 7.10.4. Strengthening the Filled Walls

Provided that the method is applied at most for three floors except for the basement, the rigidity and shear strength of the filled wall in the reinforced concrete frame constantly continuous from the top of the grounding up to the top may be improved using the strengthening methods defined in **Information Annex 7F**.

### 7.10.5. Improving the Reinforced Concrete Load-bearing Systems on-site cast Reinforced Concrete Frames

The reinforced concrete load-bearing systems with insufficient lateral rigidity can be strengthened using on-site cast reinforced concrete frames. Reinforced concrete frames can be fixed within the existing frame plane or as a confinement to the frame plane.

**7.10.5.1 – Installing Reinforced Concrete Walls within the Frame Plane:** The frames to be added to the reinforced concrete system shall be designed in the frame axis and shall be continuous from the grounding to the top elevation of the frame. With this purpose, the transverse reinforcements on the end zone of the wall and the longitudinal reinforcements in the wall body when required shall be ensured to be continuous. The walls shall be connected to the frame they are on with anchor sticks and they shall be ensured to work together with the frame. Anchor sticks will have the strength enough to meet the slip tensile that will come out between the existing frame components and the added reinforced concrete wall components due to the seismic load. The distribution of the slip tensile in the interfaces along the frame components shall be calculated in accordance with the common principles of mechanics. Friction shear principles stated in TS - 500 shall be the basis point in designing the anchor sticks. Minimum anchor stick diameter should be 16 mm, minimum anchor depth should be one times of the stick diameter and the widest spacing between sticks should be 40 cm. In case there aren't any existing columns in the end zone of the wall, wall end zone shall be constructed as per **3.6.5**. If there isn't any column in the end zone of the wall, the existing column can be utilized as an end zone. When necessary, end zone of the wall shall be constructed via expanding the existing column in accordance with **7.10.2** or constructing a hidden column inside the wall adjacent to the existing column. In both cases, the vertical reinforcements to be added to the end zones of the wall should be ensured to be continuous between the floors. Grounding shall be laid under the wall in accordance with the rules given in **6.3.1**. Wall grounding shall be sized in a way that will enable the inner forces created in wall floor to be securely transferred to the grounding floor.

So as to diminish the eccentricity possible to be formed in the wall grounding, axial pressure loads of the existing columns can be utilized via expanding the wall grounding in a way that it will cover neighboring columns. Precautions required to enable the wall grounding to work together with the existing grounding system should be taken.

**7.10.5.2 – Attaching Reinforced Concrete Wall Adjacent to the Frame Plane:** The walls to be added to the reinforced concrete system shall be designed to be adjacent to the frame outside the outer frame axis and shall be continuous beginning from the grounding up to the top elevation of the wall. The walls shall be connected to the frame they are adjacent to with anchor sticks to ensure that they work together with the frame. Anchor sticks will have the strength enough to meet the slip tensile that will come out in the interfaces between the existing frame components and the added eccentric wall components due to the seismic load. The rules stated in **7.10.5.1** shall be followed in the design of anchor sticks.

In case there aren't any existing columns in the end zone of the wall, wall end zone shall be constructed as per **3.6.5**. If there isn't any column in the end zone of the wall, the existing column can be utilized as an end zone. When necessary, end zone of the wall shall be constructed via expanding the existing column in accordance with **7.10.2**. Grounding shall be laid under the wall in accordance with the rules given in **6.3.1**. Wall grounding shall be sized in a way that will enable the inner forces created in wall floor to be securely transferred to the grounding floor. So as to diminish the eccentricity possible to be formed in the wall grounding, axial pressure loads of the existing columns can be utilized via expanding the wall grounding in a way that it will cover neighboring columns. Precautions required to enable the wall grounding to work together with the existing grounding system should be taken.

#### **7.10.6. Adding New Frames to the Reinforced Concrete System**

The horizontal loads can be distributed via adding new frames to the exterior of the reinforced concrete system. The grounding of the frames to be added to the system shall be organized together with the grounding of the existing building. To sustain that the new frames will work together with the load-bearing system of the existing building, the frames shall be attached to the slabs in a way that will enable load transfer.

#### **7.10.7. Diminishing the Mass of the Reinforced Concrete System**

Mass reduction is not a structure strengthening method. However, as the gravity loads and seismic forces that affect the building will diminish proportional to the reducing mass the structural safety shall be improved. The closer the mass to be reduced or removed to the top elevations of the building, the more effectiveness on increasing the seismic security shall be. The most effective mass reduction methods are reversing and removing the top floor or floors of the building, replacing the existing roof with a lighter one, taking installation weights on the roof such as water tanks to the ground and replacing heavy balconies, parapets, partition walls and slabs with lighter components.

## INFORMATION ANNEX 7A

### DETERMINATION OF EFFECT / CAPACITY RATIOS OF COLUMNS AND WALLS DOĞRUSAL IN DESIGN WITH LINEAR ELASTICITY METHODS

#### 7A.0. NOTATION

|       |   |  |
|-------|---|--|
| $M_A$ | = | Residual moment capacity   |
| $M_D$ | = | Moment composed from vertical powers   |
| $M_E$ | = | Moment consisted under the earthquake power  |
| $M_K$ | = | Moment capacity calculated in accordance with existing material strength according to <b>7.2</b> . |
| $N_A$ | = | Axial power correspond to momentary moment capacity  |
| $N_D$ | = | Axial power consisted from vertical powers   |
| $N_E$ | = | Axial power consisted under the earthquake power   |
| $N_K$ | = | Axial power correspond to cross section moment capacity  |
| $r$   | = | Ratio of exposure / capacity   |
| $r_s$ | = | The limit value of ratio of exposure / capacity  |

#### 7A.1. Effect / Capacity Ratios in Columns and Walls

On the calculation made with linear elasticity methods, methods to be implemented for determining the effect / capacity ratios in the sections of columns and walls under the effect of moment - axial force are specified in following paragraphs.

**7A.1.1** – Moment - axial force interaction diagram of any column or wall section that linearized according to **7.4.11(c)** is seen in **Figure 7A.1**. Coordinates of point D in Figure corresponds to  $M_D - N_D$  couple formed of gravity loads. Vertical and horizontal projections of second line segment which starts from D point and step out of interaction diagram corresponds to  $M_E - N_E$  couple obtained from seismic design for  $R_a = 1$  and comply with the direction of earthquake (In **Figure 7A.1** two condition where  $M_E$  indicators are different is shown separately). Coordinates of second line segment where intersects point K is  $M_K$  moment capacity of column or wall section or the  $N_K$  axial force that corresponds to it.

**7A.1.2** – According to **7.5.2.3**, redundant moment capacity  $M_A$  and corresponding axial force  $N_A$  defined as follows:

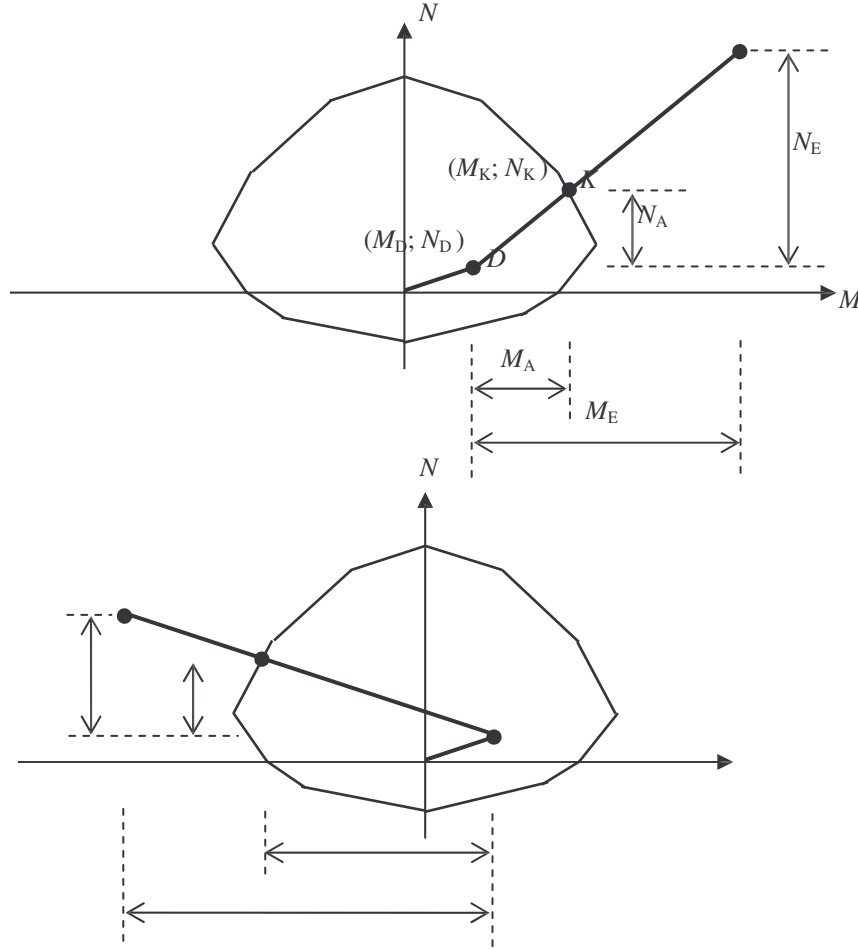
$$M_A = M_K - M_D \quad (7A.1a)$$

$$N_A = N_K - N_D \quad (7A.1b)$$

And effect / capacity ratios of columns and walls may be defined as follows:

$$r = \frac{M_E}{M_A} = \frac{N_E}{N_A} \leq r_s \quad (7A.2)$$

In case  $M_K$  or  $N_K$  as the coordinates of K intersection point in **Figure 7A.1** is obtained geometrically or numerically, then by using **Equation (7A.1)** and **Equation (7A.2)** the effect / capacity ratio of the section under bending and axial force can be directly determined as far as  $M_D$  or  $N_D$  is known from gravity load design and  $M_E$  or  $N_E$  is known from seismic design. The axial force  $N_K$  which correspond to moment capacity of the column section is the axial force to be considered in **Table 7.3** that defines boundary of damage.



**Figure 7A.1**

**7A.1.3** – Effect / capacity ratio of column or wall may also be determined with a successive approach method. For that purpose, estimation is made for  $r$  in the beginning. As  $N_E$  is known from seismic calculation  $N_A$  is calculated from **Equation (7A.2)** and also as  $N_D$  is known  $N_K$  is found from **Equation (7A.1b)**. Accordingly,  $M_K$  moment capacity is obtained from section calculation and by subtracting  $M_D$  from this value  $M_A$  is calculated from **Equation (7A.1a)**. By using  $M_A$  and  $M_E$  new value of  $r$  is determined from **Equation (7A.2)** and by turning back to beginning pass through the forward step of the successive approach. Value of  $r$  in the last step of successive approach which is obtained nearly sufficient to the previous step is defines as the effect / capacity ratio of the section under bending and axial force.  $M_A$  and  $N_A$  values in the last step are put their places in **Equation (7A.1)** and  $M_K$  and  $N_K$  are calculated. The obtained  $N_K$  is the axial force to be considered in **Table 7.3** that defines the damage boundaries.

**7A.1.4** – Effect / capacity ratio specified above for the unidirectional bending / axial force is also implied similarly for bidirectional bending / axial force.

## 7A.2. Special Conditions

**7A.1** can not be implied in case the end of second line in **Figure 7A.1** remains in the interaction diagram. It is obvious that the calculation of the ratio of effect / capacity is not required in this situation as against to  $r < 1$ .

### **7A.3. Upper boundary of axial forces of column and wall**

In the relevant column and columns on it, upper boundary of the  $N_K$  axial force which is calculated in a way as specified above in the pressure or tensile conditions can be defined as the axial force obtained as a result of the transfer of  $V_e$  shear forces calculated in accordance with the direction of earthquake implied according to 3.4.5.1 without considering hardening.

## INFORMATION ANNEX 7B

### TENSILE - DEFORMATION CORELATIONS FOR CONCRETE AND STEEL REINFORCEMENT

#### 7B.0. NOTATION

|                  |   |  |
|------------------|---|--|
| $A_s$            | = | Longitudinal reinforcement area  |
| $a_i$            | = | Distance between vertical reinforcement axis in the cross section bordering                                      |
| $b_o$            | = | Size of the cross section between the axis of lateral reinforcement binders which covers the hub concrete        |
| $E_c$            | = | Elasticity module of concrete  |
| $E_s$            | = | Elasticity module of reinforcement steel   |
| $f_c$            | = | Concrete pressure stress in coated concrete.   |
| $f_{cc}$         | = | Coated concrete strength   |
| $f_{co}$         | = | Concrete pressure stress in non-coated concrete.   |
| $f_e$            | = | Effective winding pressure   |
| $f_s$            | = | Tension in reinforcement steel   |
| $f_{sy}$         | = | Yield strength of the reinforcement steel  |
| $f_{su}$         | = | Breaking strength of the reinforcement steel   |
| $f_{yw}$         | = | Yield strength of the transversely reinforcement   |
| $h_o$            | = | Size of the cross section between the axis of lateral reinforcement binders which covers the hub concrete        |
| $k_e$            | = | Enswathed Effectiveness Coefficient  |
| $s$              | = | Lateral reinforcement binders space  |
| $\rho_s$         | = | Volumetric ratio of total transversely reinforcement (in rectangular cross sections $\rho_s = \rho_x + \rho_y$ ) |
| $\rho_x, \rho_y$ | = | Volumetric ratio of transversely reinforcement in direction represents.  |
| $\epsilon_c$     | = | Deformation of pressure concrete unit  |
| $\epsilon_{cu}$  | = | Deformation of maximum pressure unit in coated concrete  |
| $\epsilon_{sy}$  | = | Yield unit deformation of reinforcement steel  |
| $\epsilon_s$     | = | Yield unit deformation of reinforcement steel in the beginning of strain hardening                               |
| $\epsilon_{su}$  | = | Breaking unit deformation of reinforcement steel   |

#### 7B.1. Coated and Uncoated Concrete Models

On the performance evaluation with *Linear Inelastic Methods* according to 7.6, in order to use in cases where any other model is not chosen, the following tensile - deformation corelations are defined for coated and uncoated concrete (**Figure 7B.1**)\*.

(a) Concrete compressive stress in coated concrete  $f_c$ , is given with the following correlation as the function of compressive unit deformation  $\epsilon_c$ :

$$f_c = \frac{f_{cc} x^r}{r - 1 + x^r} \quad (7B.1)$$

Relation between coated concrete strength  $f_{cc}$  and uncoated concrete strength  $f_{co}$  in this correlation is given below.

$$f_{cc} = \lambda_c f_{co} \quad ; \quad \lambda_c = 2.254 \sqrt{1 + 7.94 \frac{f_e}{f_{co}}} - 2 \frac{f_e}{f_{co}} - 1.254 \quad (7B.2)$$

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\* Mander, J.B., Priestley, M.J.N., Park, R. (1988). Theoretical Stress - Strain Model for Confined Concrete, *Journal of Structural Division (ASCE)*, **114** (8), 1804 -1826

$f_e$  effective coating pressure in here, can be taken as the average of values given below for the two perpendicular directions in rectangular sections:

$$f_{ex} = k_e \rho_x f_{yw} \quad ; \quad f_{ey} = k_e \rho_y f_{yw} \quad (7B.3)$$

In these correlations  $f_{yw}$  yield stress of the transverse reinforcement indicates the volumetric ratios of transverse reinforcements in  $\rho_x$  and  $\rho_y$  relevant directions whereas  $k_e$  indicates coating performance factor as defined below.

$$k_e = \left(1 - \frac{\sum a_i^2}{6b_o h_o}\right) \left(1 - \frac{s}{2b_o}\right) \left(1 - \frac{s}{2h_o}\right) \left(1 - \frac{A_s}{b_o h_o}\right)^{-1} \quad (7B.4)$$

Here  $a_i$  indicates the distance between the axes of vertical reinforcements in the periphery of section,  $b_o$  and  $h_o$  indicates the section sizes remain among the axes of hoops that coats the core concrete,  $s$  indicates the distance between the axes of hoops in vertical direction,  $A_s$  indicates area of longitudinal reinforcement. Correlations concerning  $x$  and  $r$  variable of normalized concrete unit deformation in **Equation (7B.1)** are given below.

$$x = \frac{\epsilon_c}{\epsilon_{cc}} \quad ; \quad \epsilon_{cc} = \epsilon_{co} [1 + 5(\lambda_c - 1)] \quad ; \quad \epsilon_{co} \cong 0.002 \quad (7B.5)$$

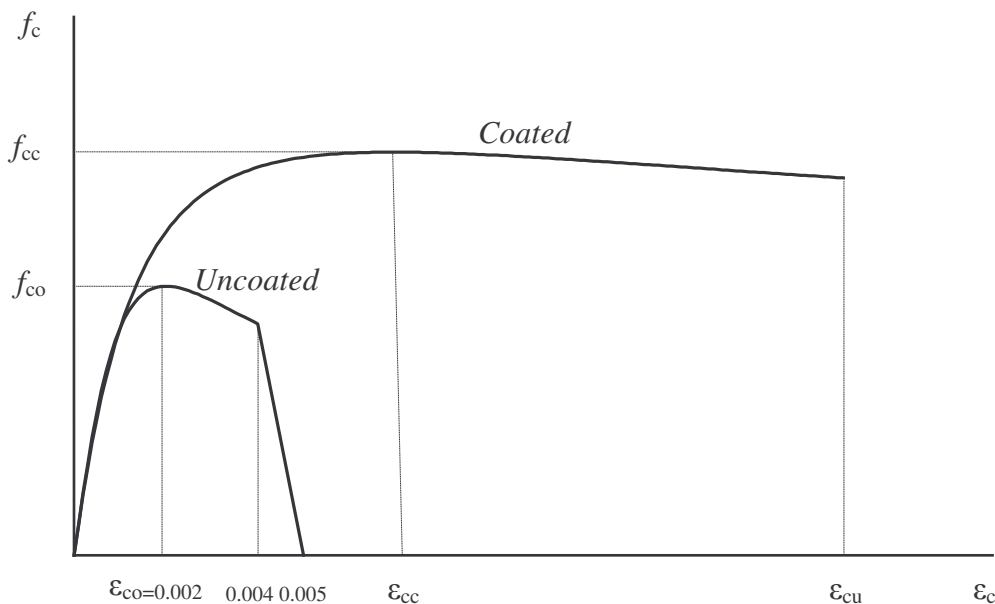
$$r = \frac{E_c}{E_c - E_{sec}} \quad ; \quad E_c \cong 5000 \sqrt{f_{co}} \quad [MPa] \quad ; \quad E_{sec} = \frac{f_{cc}}{\epsilon_{cc}} \quad (7B.6)$$

Maximum compressive unit deformation in coated concrete  $\epsilon_{cu}$  is given below:

$$\epsilon_{cu} = 0.004 + \frac{1.4 \rho_s f_{yw} \epsilon_{su}}{f_{cc}} \quad (7B.7)$$

Here  $\rho_s$  indicates the total volumetric area of transverse reinforcement (in rectangular sections  $\rho_s = \rho_x + \rho_y$ ),  $\epsilon_{su}$  indicates the unit deformation due to strain under maximum tensile in transverse reinforcement steel.

(b) **Equation (7B.1)** as given for coated concrete is also available for uncoated concrete in the zone up to  $\epsilon_c = 0.004$ .  $f_{cc} = f_{co}$  ve  $\epsilon_{cc} = \epsilon_{co}$  is taken in **Equation (7B.5)** and **Equation (7B.6)** as the effective coating pressure  $f_e = 0$  in uncoated concrete and accordingly  $\lambda_c = 1$  from the **Equation (7B.2)**.  $f_c = 0$  is defined in  $\epsilon_c = 0.005$ . In  $0.004 < \epsilon_c \leq 0.005$  range, tensile - deformation relation is linear.



**Figure 7B.1**

## 7B.2. Reinforcement Steel Model

In order to use in the performance evaluation with *Linear Inelastic Methods* according to 7.6, the following tensile - deformation correlations are defined for reinforcement steel. (Figure 7B.2):

$$\begin{aligned}
 f_s &= E_s \epsilon_s & (\epsilon_s \leq \epsilon_{sy}) \\
 f_s &= f_{sy} & (\epsilon_{sy} < \epsilon_s \leq \epsilon_{sh}) \\
 f_s &= f_{su} - (f_{su} - f_{sy}) \frac{(\epsilon_{su} - \epsilon_s)^2}{(\epsilon_{su} - \epsilon_{sh})^2} & (\epsilon_{sh} < \epsilon_s \leq \epsilon_{su})
 \end{aligned} \tag{7B.8}$$

Elasticity modulus of reinforcement steel is  $E_s = 2 \cdot 10^5$  MPa. Information concerning reinforcement steel with quality S220 and S420 can be taken from the following table.

| Quality | $f_{sy}$ (Mpa) | $\epsilon_{sy}$ | $\epsilon_{sh}$ | $\epsilon_{su}$ | $f_{su}$ (Mpa) |
|---------|----------------|-----------------|-----------------|-----------------|----------------|
| S220    | 220            | 0.0011          | 0.011           | 0.16            | 275            |
| S420    | 420            | 0.0021          | 0.008           | 0.10            | 550            |

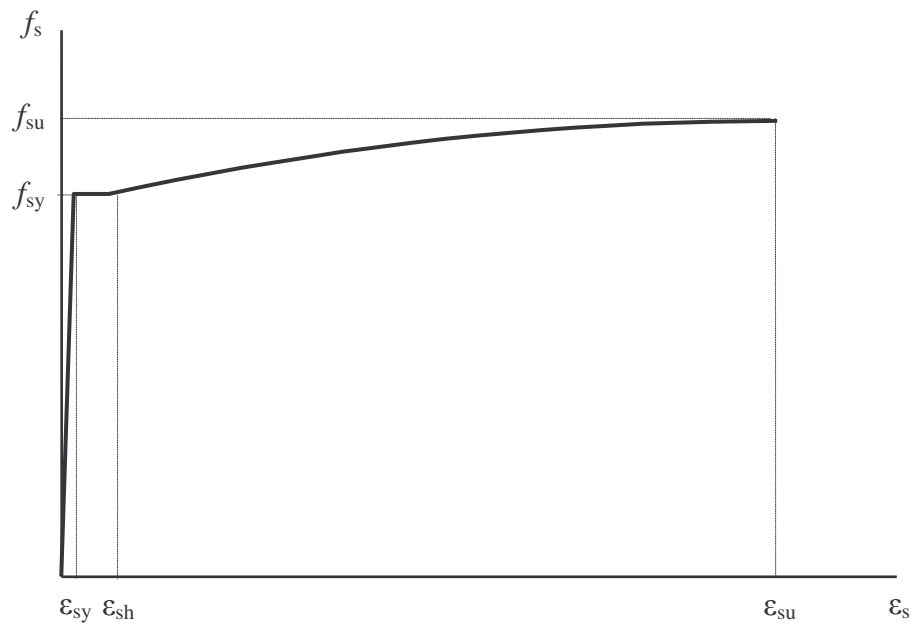


Figure 7B.2

## INFORMATION ANNEX 7C

### DETERMINATION OF NON - LINEAR SPECTRAL REPLACEMENT

#### 7C.0. NOTATION

|                  |   |  |
|------------------|---|--|
| $a_1$            | = | Modal acceleration belonging to first (prevalent) mode   |
| $a_{y1}$         | = | Equivalent yield acceleration belonging to first mode  |
| $C_{R1}$         | = | Spectral displacement ratio belonging to first mode  |
| $d_1$            | = | Modal displacement belonging to first (prevalent) mode   |
| $d_{y1}$         | = | Equivalent yield displacement belonging to first mode  |
| $d^{(p)}_1$      | = | Maximum modal displacement (modal displacement request) belonging to first mode achieved at the end of last p'th impulse step          |
| $R_{y1}$         | = | Strength Decrease Coefficient belonging to first mode  |
| $S^{(1)}_{ac1}$  | = | Elastic spectral acceleration belonging to first mode at the first step of the impulse analyze   |
| $S^{(1)}_{de1}$  | = | Linear elastic spectral displacement belonging to first mode at the first step of the impulse analyze                                  |
| $S_{de1}$        | = | Nonlinear elastic spectral displacement belonging to first mode  |
| $T_B$            | = | Characteristic period in acceleration spectrums defined in 6.4   |
| $T^{(1)}_1$      | = | Natural vibration period belonging to first vibration mode (prevalent in earthquake direction) in impulse step (i=1) at the beginning  |
| $\omega^{(1)}_1$ | = | Natural angular frequency belonging to first vibration mode (prevalent in earthquake direction) in impulse step (i=1) at the beginning |
| $\omega_B$       | = | Natural angular frequency correspond to characteristic period in acceleration spectrum defined in 6.4                                  |

#### 7C.1. Linear and Non - Linear Spectral Replacements

*Non - linear inelastic spectral replacement*,  $S_{di1}$ , in the first step of the drift analysis, shall be obtained with **Equation (7C.1)** depending on the *linear elastic spectral replacement*  $S_{de1}$  which corresponds to  $T_1^{(1)}$  beginning period regarding the first (dominant) mode calculated as based on linear elastic behavior:

$$S_{di1} = C_{R1} S_{de1} \quad (7C.1)$$

*Linear elastic spectral replacement*  $S_{de1}$  is calculated from elastic spectral acceleration  $S_{ae1}$  regarding first mode in the first step of drift analysis:

$$S_{de1} = \frac{S_{ae1}}{(\omega_1^{(1)})^2} \quad (7C.2)$$

#### 7C.2. Ratio of Spectral Replacement

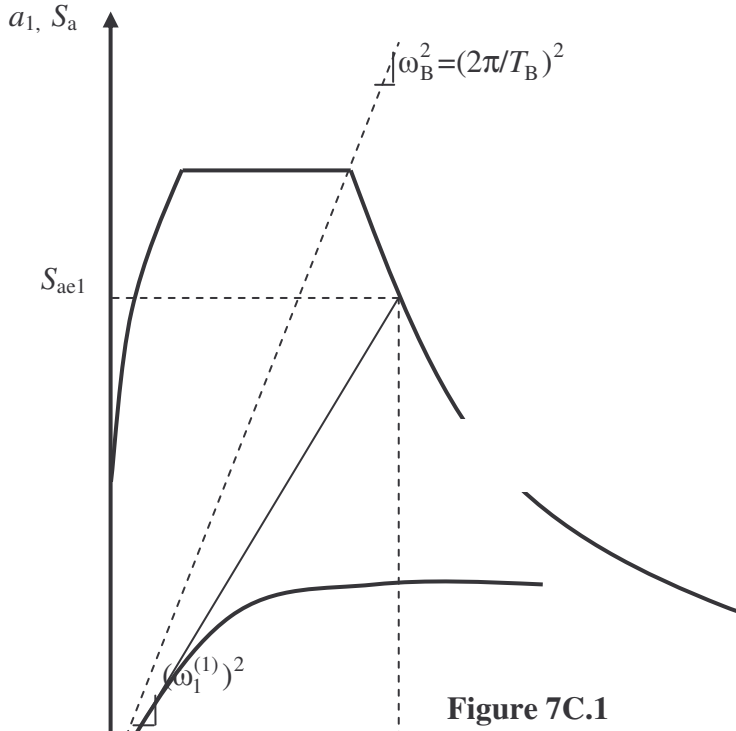
Ratio of spectral replacement  $C_{R1}$ , in **Equation (7C.1)** is determined according to **7C.2.1** or **7C.2.2** depending on the value  $(T_1^{(1)} = 2\pi / \omega_1^{(1)})$  of starting period  $T_1^{(1)}$ .

**7C.2.1** – In case where the starting period  $T_1^{(1)}$  is equal to or more than the *characteristic period*  $T_B$  in the acceleration spectrum defined in 2.4,  $(T_1^{(1)} \geq T_B \text{ or } (\omega_1^{(1)})^2 \leq \omega_B^2)$ , non - linear inelastic spectral replacement  $S_{di1}$  is taken as equal to  $S_{de1}$  *linear elastic spectral replacement regarding also the conjugated linear elastic system* which's natural period is also  $T_1^{(1)}$  in accordance with equal replacement rule. Accordingly, *spectral replacement ratio* in **Equation (7C.1)** is as follows:

$$C_{R1} = 1 \quad (7C.3)$$

In **Figure 7C.1** and following **Figure 7C.2**, *model capacity diagram* that the coordinates are  $(d_1, a_1)$  and belong to first (dominant) vibration mode and behavioral spectrum that the coordinates are “*spectral replacement* ( $S_d$ ) – *spectral acceleration* ( $S_a$ )” are drawn together.

**7C.2.2** – In case where the starting period  $T_1^{(1)}$  is less than the *characteristic period*  $T_B$  in the acceleration spectrum defined in 2.4,  $T_1^{(1)} < T_B$  or  $(\omega_1^{(1)})^2 > \omega_B^2$ , spectral replacement ratio  $C_{R1}$  in **Equation (7C.1)** shall be calculated with successive approach method as follows:



**Figure 7C.1**

(a) Modal capacity diagram obtained as a result of drift analysis, as shown in **Figure 7C.2 (a)**, is turned approximately into a bilinear diagram. Slope of the starting line of this diagram is taken as equal  $(T_1^{(1)} = 2\pi / \omega_1^{(1)})$  with Eigen value  $(\omega_1^{(1)})^2$  pertaining to the first mode that is the slope of the line in the first step ( $i=1$ ) of drift analysis.

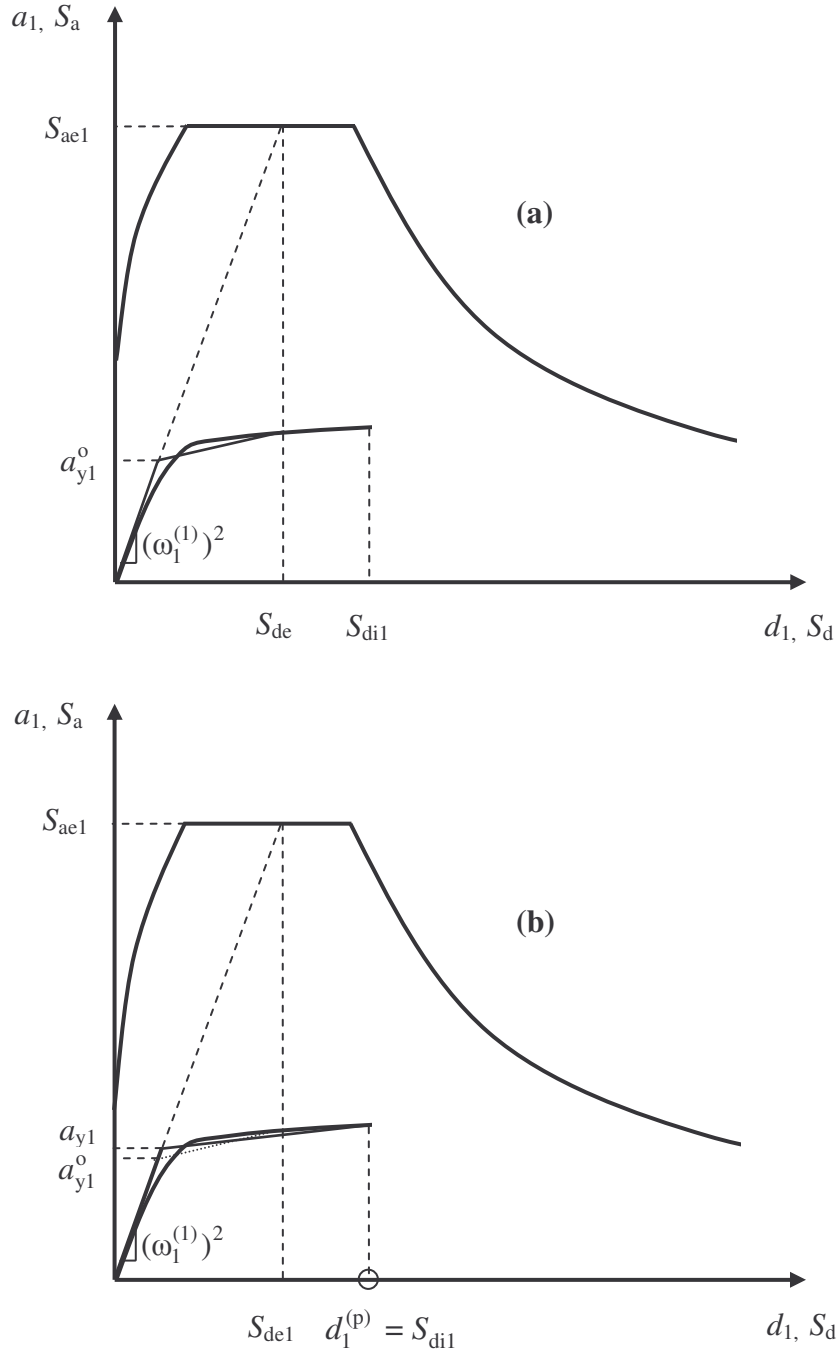
(b) As accepted  $C_{R1} = 1$  in the first step of successive approach, in other words by using **Equation (7C.3)** coordinates of *equivalent yield point* is determined with equal areas rule.  $C_{R1}$  is defined as follows as based  $a_{y1}^o$  seen in **Figure 7C.2 (a)**:

$$C_{R1} = \frac{1 + (R_{y1} - 1) T_B / T_1^{(1)}}{R_{y1}} \geq 1 \quad (7C.4)$$

In this correlation,  $R_{y1}$  indicates the *strength reduce factor* pertaining to first mode:

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \quad (7C.5)$$

(c) By using  $C_{R1}$  in **Equation (7C.4)**, coordinates of *equivalent yield point* is re-determined with equal areas rule by taking  $S_{di1}$  calculated according to **Equation (7C.1)** as basis, as shown in **Figure 7C.2 (b)** and accordingly  $a_{y1}$ ,  $R_{y1}$  and  $C_{R1}$  are calculated again. When the results obtained from successive two steps reasonably close up then the successive approached is concluded.



**Figure 7C.2**

## INFORMATION ANNEX 7D

### IMPULSE ANALYZE WITH INCREMENTAL MODE COMBINATION

#### 7D.0. NOTATION

|                         |   |   |
|-------------------------|---|---|
| $a_n^{(i)}$             | = | Modal acceleration belonging to n'th mode at the end of (i)'th impulse step.  |
| $a_{yn}$                | = | Equivalent outflow acceleration belonging to n'th mode  |
| $C_{Rn}$                | = | Spectral displacement ratio belonging to n'th mode  |
| $d_n^{(i)}$             | = | Modal displacement belonging to n'th mode at the end of (i)'th impulse step   |
| $\tilde{F}^{(i)}$       | = | Cumulative scale factor belonging to (i)'th impulse step  |
| $M_{j,x}^{(i)}$         | = | Bending moment formed around the x axis in (j) plastic cross section at the end of (i)'th impulse step,   |
| $\tilde{M}_{j,x}^{(i)}$ | = | Bending moment calculated around the x axis in (j) plastic cross section at the (i)'th impulse step, in the result of linear mode combination analyze with taking $\Delta\tilde{F}^{(i)}=1$ .                                 |
| $M_{j,y}^{(i)}$         | = | Bending moment formed around the y axis in (j) plastic cross section at the end of (i)'th impulse step,   |
| $\tilde{M}_{j,y}^{(i)}$ | = | Bending moment calculated around the y axis in (j) plastic cross section at the (i)'th impulse step, in the result of linear mode combination analyze with taking $\Delta\tilde{F}^{(i)}=1$ .                                 |
| $m_s$                   | = | Mass of any (s) degrees of freedom  |
| $N_j^{(i)}$             | = | Axial force formed in (j) plastic cross section at the end of (i)'th impulse step   |
| $\tilde{N}_j^{(i)}$     | = | Axial force calculated in (j) plastic cross section at the (i)'th impulse step, in the result of linear mode combination analyze with taking $\Delta\tilde{F}^{(i)}=1$  |
| $R_{yn}$                | = | Strength Decrease Coefficient belonging to n'th mode  |
| $r_j^{(i)}$             | = | Typical displacement, plastically deformation or internal force which are formed in any (j) point or cross section at the end of (i)'th impulse step  |
| $\tilde{r}_j^{(i)}$     | = | Typical displacement, plastically deformation or internal force calculated in (j) point or cross section at the (i)'th impulse step, in the result of linear mode combination analyze with taking $\Delta\tilde{F}^{(i)}=1$ . |
| $S_{aen}^{(1)}$         | = | Elastic spectral acceleration belonging to n'th mode in the first step of the impulses analyzes.  |
| $S_{den}^{(1)}$         | = | Linear Elastic spectral displacement belonging to n'th mode in the first step of the impulses analyze   |
| $T_B$                   | = | Characteristic period in acceleration spectrum defined in 6.4' de   |
| $T_n^{(1)}$             | = | Natural vibration period belonging to n'th vibration mode in the impulse step at the beginning (i=1)  |
| $\alpha_{jk,x}$         | = | Coefficient which is defining (k)'th yield platform or line concerning with the moment around the x axis in (j) plastic cross section   |
| $\alpha_{jk,y}$         | = | Coefficient which is defining (k)'th yield platform or line concerning with the moment around the y axis in (j) plastic cross section   |
| $\beta_{jk}$            | = | Coefficient which is defining (k)'th yield platform or line concerning with the axial force in (j) plastic cross section  |

- $\Delta a_n^{(i)}$  = Modal acceleration increase belonging to n'th mode in the (i)'th impulse step  
 $\Delta d_n^{(i)}$  = Modal displacement increase belonging to n'th mode in the (i)'th impulse step  
 $\Delta \tilde{F}^{(i)}$  = Increasing spectrum scale coefficient in the (i)'th impulse step  
 $\Delta f_{sn}^{(i)}$  = Increasing of the displacement effecting any (s) degrees of freedom of the system for n'th natural vibration mode in the (i)'th impulse step  
 $\Delta u_{sn}^{(i)}$  = Increasing of the earthquake load effecting any (s) degrees of freedom of the system for n'th natural vibration mode in the (i)'th impulse step  
 $\Phi_{sn}^{(i)}$  = Amplitude belonging to (s) degrees of freedom of n'th mode figure which was determined by considering plastic cross section configuration in that step at the (i)'th impulse step  
 $\Gamma_{xn}^{(i)}$  = Contribution factor belonging to n'th natural vibration mode for the earthquake in x direction at the (i)'th impulse step  
 $\omega_B$  = Natural angular frequency correspond to  $T_B$  characteristic period in acceleration spectrum defined in 6.4'de  
 $\omega_n^{(i)}$  = Natural angular frequency belonging to (s) degrees of freedom of n'th mode figure this was determined by considering plastic cross section configuration in that step at the (i)'th impulse step  
 $\omega_n^{(1)}$  = Natural angular frequency belonging to n'th vibration mode in the impulse step at the beginning (i=1)  
 $\omega_n^{(p)}$  = At the last impulse step (i=p)

### 7D.1. Introduction

**7D.1.1** – The most significant disadvantage of conducting repulsion analysis using the *Incremental Equivalent Seismic Load Method* explained in 7.6.5 is the assumption proposing that the seismic behavior of the load-bearing system is solely composed of the behavior in the primary (dominant in the seismic direction) natural vibration mode. Due to this reason, the applicability of the method is limited to the few-floor buildings and the buildings that are symmetric or nearly symmetric to the seismic direction according to the plans. Although various repulsion analysis methods that can be applied to the buildings that doesn't satisfy these conditions as they consider multiple vibration modes are proposed, the majority of these deal only with determining the global strength and deformation capacities of the load-bearing system. There are a limited number of methods that aim at deriving the volition values required to make performance evaluation under the effect of a certain earthquake [1–5]\*. By means of the *Incremental Mode Combination Method* explained in this Information Appendix, the contributions of individual modes to the formation of each plastic section can be considered in the repulsion analysis [4, 5], and the plastic rotations and inner force volitions can directly be derived without requiring any additional analysis.

**7D.1.2** – In repulsion analyses conducted under Incremental Mode Combination Method, “step-by-step linear elastic” behavior in each repulsion step between the plastic section formation between two subsequent sections is taken as a basis. Considering the modal translocations monotonically increased via modal scaling, a linear behavior spectrum in each step of which mode combination *rules* are applied is obtained. Utilizing the results of this analysis, the plastic section formed at the end of the step is determined and translocation, plastic deformation and inner force increments together with the incremental values corresponding to these increments and thus the maximum values corresponding to the seismic volition are calculated [4, 5].

## 7D.2. Modal scaling

**7D.2.1** – The translocation increment in an (i)<sup>th</sup> *linear* repulsion step in the formation of two subsequent plastic sections for any independence level (s) for a typical n<sup>th</sup> natural vibration mode can be written as the following:

$$\Delta u_{sn}^{(i)} = \Phi_{sn}^{(i)} \Gamma_{xn}^{(i)} \Delta d_n^{(i)} \quad (7D.1)$$

**7D.2.2** - With adding the  $\Delta d_n^{(i)}$  that represents *modal replacement increase* in the (n)<sup>th</sup> at the (i)<sup>th</sup> drift step and located in **Equation (7D.1)** to the modal replacement at the end of the previous drift step, *cumulative modal replacement* at the end of the (i)<sup>th</sup> step is obtained as follows:

$$d_n^{(i)} = d_n^{(i-1)} + \Delta d_n^{(i)} \quad (7D.2)$$

In order to consider relative contributions of the modes, according to *equal replacement rule* pertaining to single degree of freedom system, *cumulative modal replacement*, is defined proportionally to *elastic spectral replacement*  $S_{den}^{(1)}$  in the first step (i= 1) at the same mode:

$$d_n^{(i)} = S_{den}^{(1)} \tilde{F}^{(i)} \quad (7D.3)$$

Here  $\tilde{F}^{(i)}$  indicates the *cumulative spectrum scale factor* which is considered as constant for all modes in the (i)<sup>th</sup> drift step. As a result of **Equation (7D.2)** and **Equation (7D.3)**, modal replacement increase in the n<sup>th</sup> mode is defined as follows:

$$\Delta d_n^{(i)} = S_{den}^{(1)} \Delta \tilde{F}^{(i)} \quad (7D.4)$$

Here  $\Delta \tilde{F}^{(i)}$  is also indicates the *incremental spectrum scale factor* which is considered as constant for all modes in the (i)<sup>th</sup> drift step. Thus, all modal replacement increases are figured with single parameter. Relation between incremental and cumulative spectrum scale factors are formulated as below:

$$\tilde{F}^{(i)} = \tilde{F}^{(i-1)} + \Delta \tilde{F}^{(i)} \leq 1 \quad (7D.5)$$

$S_{den}^{(1)}$  *Elastic spectral replacement* defined for the first (i= 1) drift step at the above correlations can be obtained from *elastic spectral replacement* defined according to 2.4 for the same step:

$$S_{den}^{(1)} = \frac{S_{aen}^{(1)}}{(\omega_n^{(1)})^2} \quad (7D.6)$$

**7D.2.3** – Modal scaling correlations given with **Equation (7D.3)** and **Equation (7D.4)**, correspond to monolithically increasing elastic spectral replacement during each drift step where a new plastic section formed. In other words, seismic effects in terms of spectral replacements are increased in definite amount in the each drift step starting from zero.

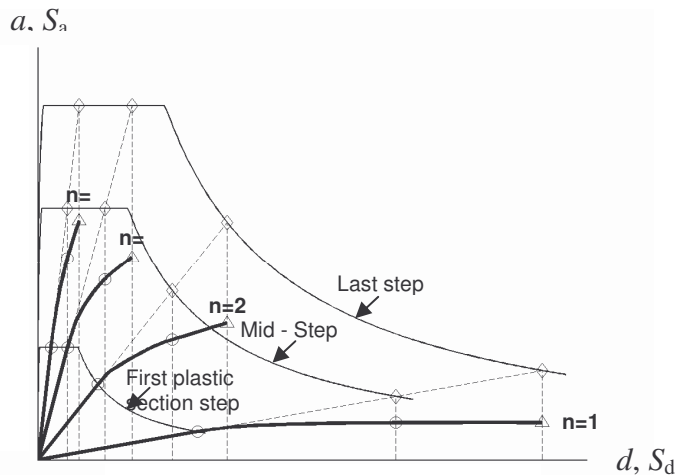


Figure 7D.1

At the end of the linear elastic first step where first plastic section in the system is formed scaled condition of behavioral spectrum ( $\tilde{F}^{(1)} \leq 1$ ) drawn at the coordinates of “spectral replacement ( $S_d$ ) – spectral acceleration ( $S_a$ )” by using **Equation (7D.6)** is shown in **Figure 7D.1**. Scaled condition of spectrum at the end of any following (i)<sup>th</sup> inter - step ( $\tilde{F}^{(i)} \leq 1$ ) is also seen similarly. At the end of the last step of (p)<sup>th</sup> drift, in accordance with the equal replacement rule, elastic behavioral spectrum is obtained ( $\tilde{F}^{(p)} = 1$ ). Also modal capacity diagrams to be obtained below and defined with the coordinates of “Modal replacement ( $d$ ) – modal acceleration ( $a$ )” are schematically shown in **Figure 7D. 1** for the first four mode of a considered typical structural system

### 7D.3. Drift Analysis Algorithm with Incremental Mod Combination Method

By grounding the modal scaling transaction defined above, main steps of the drift analysis to be applied with incremental mode combination method are summarized below:

**7D.3.1** – In the practical implementation of the *Incremental Mode Combination Method*, linear *Mod Combination Analysis* is made by taking  $\Delta\tilde{F}^{(i)} = 1$  in the each (i)<sup>th</sup> drift step. In the analysis, by grounding the axial forces at the end of the previous step, effects of second stage can be taken into consideration. Number of mode to be considered is determined according to **2.8.3** as based on the biggest modals in the first drift step (i=1). In this analysis; (a) According to **Equation (7D.1)** and **Equation (7D.4)**, elastic spectral replacement  $S_{den}^{(1)}$  in the first drift step (i=1) is taken into account as an earthquake data for a typical n<sup>th</sup> mode. This introduction information is used exactly in all drift steps without any change.

(b) For the calculations of all replacements, deformations, modal contributions to internal force magnitudes, *Complete Quadric Combination Rule (CQC)* specified in **2.8.4** is applied. In the implementation of this rule, critical damping ratio is taken as 0.05 in all modes.

**7D.3.2** – At the end of any (i)<sup>th</sup> drift step between sequential two plastic section, *typical magnitude*  $r_j^{(i)}$  represents any replacement, plastic deformation or internal force formed in any (j) point or section of the structural system is formulated in terms of incremental scale factor  $\Delta\tilde{F}^{(i)}$  pertaining only to (i)<sup>th</sup> step as unknown as below:

$$r_j^{(i)} = r_j^{(i-1)} + \tilde{r}_j^{(i)} \Delta\tilde{F}^{(i)} \quad (7D.7)$$

Definitions concerning this correlation are given below:

(a) At the end of a linear mod combination analysis made according to **7D.3.1** in the (i)<sup>th</sup> drift step by taking  $\Delta\tilde{F}^{(i)} = 1$ ,  $\tilde{r}_j^{(i)}$  represents a typical replacement, plastic deformation or internal force calculated in the (j) point or section. Because the indicators became disappeared due to the implementation of Complete Quadric Combination (CQC) modal combination rule; indicator in the mode where the biggest absolute value is obtained from typical replacement, plastic deformation or internal force is taken into account.

(b)  $r_j^{(i)}$  represents the typical magnitude to be obtained according to **7D.3.5** after the calculation of  $\Delta\tilde{F}^{(i)}$  according to **7D.3.4** at the end of (i)<sup>th</sup> drift step. Whereas  $r_j^{(i-1)}$  shows the magnitude obtained at the end of the previous (i - 1)<sup>th</sup> drift step. In this context, step zero (i-1=0), which is before the first drift step (i=1) corresponds to the typical magnitude obtained from the gravity load analysis that is necessarily made before the drift analysis.

**7D.3.3** – General correlation given in **Equation (7D.7)** in the each drift step is specially written for the bending moment in every potential plastic section in the beams, for the internal forces that form the coordinates of yield surface in the columns and walls. For the biaxial bending and axial force in three - dimensional behavior condition:

$$\begin{aligned} M_{j,x}^{(i)} &= M_{j,x}^{(i-1)} + \tilde{M}_{j,x}^{(i)} \Delta \tilde{F}^{(i)} \\ M_{j,y}^{(i)} &= M_{j,y}^{(i-1)} + \tilde{M}_{j,y}^{(i)} \Delta \tilde{F}^{(i)} \\ N_j^{(i)} &= N_j^{(i-1)} + \tilde{N}_j^{(i)} \Delta \tilde{F}^{(i)} \end{aligned} \quad (7D.8)$$

**7D.3.4** – In case of a three - dimensional behavior as considered generally, analytical formulation of the (k)<sup>th</sup> plane part that corresponds any of the yield surfaces linearized according to **7.6.4.5** in the section (j) is formulated as follow:

$$\alpha_{jk,x} M_{j,x} + \alpha_{jk,y} M_{j,y} + \beta_{jk} N_j = 1 \quad (7D.9)$$

With placing the magnitudes in **Equation (7D.8)** to **Equation (7D.9)**, incremental scale factor pertaining to (i)<sup>th</sup> step is calculated without the need of successive approach:

$$(\Delta \tilde{F}^{(i)})_{jk} = \frac{1 - \alpha_{jk,x} M_{j,x}^{(i-1)} - \alpha_{jk,y} M_{j,y}^{(i-1)} - \beta_{jk} N_j^{(i-1)}}{\alpha_{jk,x} \tilde{M}_{j,x}^{(i)} + \alpha_{jk,y} \tilde{M}_{j,y}^{(i)} + \beta_{jk} \tilde{N}_j^{(i)}} \quad (7D.10)$$

In any of (j) potential plastic section, after finding the positive and smallest  $(\Delta \tilde{F}^{(i)})_{jk}$  values obtained for all (k) yield surfaces (lines), the smallest one of them calculated in the structural system is obtained as the  $\Delta \tilde{F}^{(i)}$  incremental scale factor at the end of (i)<sup>th</sup> step. And the section (j) which is corresponding to this value is determined the place of the newly established plastic section in the system.

**7D.3.5** – After obtaining  $\Delta \tilde{F}^{(i)}$  in the (i)<sup>th</sup> drift step;

(a) Cumulative spectrum scale factor  $\tilde{F}^{(i)}$  is calculated from the **Equation (7D.5)**.

(b)  $\tilde{F}_j^{(i)}$ , any typical replacement, plastic deformation or internal force magnitude formed in the any (j) point or section of the structural system is obtained according to **Equation (7D.7)**.

(c) Modal replacement increments pertaining to considered all modes are calculated from **Equation (7D.4)**. Cumulative modal replacements at the end of the (i)<sup>th</sup> drift step are calculated from **Equation (7D.2)** or **Equation (7D.3)**.

**7D.3.6** – Modal acceleration increments pertaining to all modes in the (i)<sup>th</sup> step are calculated with the following correlation:

$$\Delta a_n^{(i)} = (\omega_n^{(i)})^2 \Delta d_n^{(i)} \quad (7D.11)$$

However not used directly from the formulation of the Incremental Mode Combination Method as stated here, as a description, *modal seismic load increment*  $\Delta f_{sn}^{(i)}$  that affects the (s) freedom degree in the (n)<sup>th</sup> mode is given above depending on the modal acceleration increment  $\Delta a_n^{(i)}$ :

$$\Delta f_{sn}^{(i)} = m_s \Phi_{sn}^{(i)} \Gamma_{xn}^{(i)} \Delta a_n^{(i)} \quad (7D.12)$$

Cumulative modal acceleration values at the end of the (i)<sup>th</sup> drift step are calculated with the below formula:

$$a_n^{(i)} = a_n^{(i-1)} + \Delta a_n^{(i)} \quad (7D.13)$$

**7D.3.7** – Typical *modal capacity diagrams* in which modal replacements are indicated in horizontal axis and modal accelerations are indicated in vertical axis is shown in **Figure 7D.1**. As a description, slope of a line section between two sequential plastic sections in the typical capacity diagram pertaining to  $n^{\text{th}}$  mode is equal to the square of the natural angular frequency of the  $n^{\text{th}}$  mode in that step in accordance with the **Equation (7D.11)**  $(\omega_n^{(i)})^2$ , in other words, equals to  $n^{\text{th}}$  Eigen value. As a result of the spread of the plastic replacements, Eigen values of some modes due to the effects of second stage, accordingly the slopes the relevant modal capacity diagrams can take negative values after a definite drift step. It should be taken into consideration that the effects of second stage can change the shapes of modes. Whereas its effects on the modal seismic prompts are generally in allowable levels

**7D.3.8** – After the completion of a drift stage, by considering the plastic section formed at the end of that stage, necessary changes are made in the rigidity matrix of system and transactions are started for the new drift stage. **7.6.4.7** Should be considered in the drift stages that follow the plasticity in the sections under the effects of uniaxial or biaxial bending or axial forces.

#### **7D.4. Determination of Prompt Magnitudes**

**7D.4.1**– Modal replacements in the Incremental Mod Combination Method reaches their maximum values with all other modes. At the end of each drift step, it is controlled whether the cumulative spectrum scale factor which is calculated with **Equation (7D.5)** exceeds the unit value as a maximum value or not. In case of non - exceedance, the analysis continued as defined above. In case of an exceedance;

(a) The obtained drift stage is defined as the last stage and represented by (p) superscript. Taking  $i = p$ , and considering  $\tilde{F}^{(p)} = 1$ , incremental spectrum scale factor concerning the last stage is calculated with **Equation (7D.5)**:

$$\Delta \tilde{F}^{(p)} = 1 - \tilde{F}^{(p-1)} \quad (7D.14)$$

(b) However it is necessary to re - define the modal replacement in the  $n^{\text{th}}$  mode defined with **Equation (7D.4)** at the last drift stage as follows:

$$\Delta d_n^{(p)} = C_{Rn} S_{\text{den}}^{(1)} \Delta \tilde{F}^{(p)} \quad (7D.15)$$

In any of the mode in case  $C_{Rn} > 1$  according to **7D.4.2**,  $C_{Rn} S_{\text{den}}^{(1)}$  is taken as data for earthquake instead of  $S_{\text{den}}^{(1)}$  in **7D.3.1** and with the Method of Mod Combination  $\tilde{r}_j^{(p)}$  value pertaining to typical magnitude is re - calculated.

(c) Maximum value of typical replacement, plastic deformation and internal force, in other words typical prompt magnitude are obtained according to Equation **(7D.7)**:

$$r_j^{(p)} = r_j^{(p-1)} + \tilde{r}_j^{(p)} \Delta \tilde{F}^{(p)} \quad (7D.16)$$

**7D.4.2** –  $C_{Rn}$  spectral replacement ratio regarding any considered  $n^{\text{th}}$  mode is calculated as follows:

(a) In case the condition  $T_n^{(1)} > T_B$  [or  $(\omega_n^{(1)})^2 < \omega_B^2$ ] is covered then  $C_{Rn} = 1$  is taken.

(b) In case where  $T_n^{(1)} < T_B$  [or  $(\omega_n^{(1)})^2 > \omega_B^2$ ] then  $C_{Rn}$  can be approximately determined as follows:

$$\lambda_n^{(p)} = \frac{(\omega_n^{(p)})^2}{(\omega_n^{(1)})^2} ;$$

$$C_{Rn} = \frac{1 + (R_{yn} - 1) T_B / T_n^{(1)}}{R_{yn}} \geq 1 \quad (\lambda_n^{(p)} \leq 0.10) \quad (7D.17)$$

$$C_{Rn} = \frac{1 + (R_{yn} - 1) T_B / T_n^{(1)}}{10 \lambda_n^{(p)} R_{yn}} \geq 1 \quad (\lambda_n^{(p)} > 0.10)$$

In this correlation  $R_{yn}$ , indicates the *strength reduce factor* obtained from the bilinear modal capacity diagram drawn for n'th mode:

$$R_{yn} = \frac{S_{aen}^{(1)}}{a_{yn}} \quad (7D.18)$$

Related to the successive approach concerning bilinear modal capacity diagram, the approach given for the first mode (dominant) and **Information Annex 7C** are useful (See **Figure 7C.2**).

### 7D.5. Special Conditions

**7D.5.1** – If it is assumed that solely the primary (dominant in the earthquake direction) mode has impact on the load-bearing system behavior, all of the equations given below regarding the Incremental Mode Combination Method can only be used for the dominant mode, without introducing any modifications. The repulsion analysis for this special condition is reduced in the single-mode repulsion analysis in which the load distribution in every repulsion step is assumed to be *variable* for the Incremental Equivalent Seismic Load Method according to **7.6.5.5**. For this solution in which the modal scaling is not considered, the magnitudes obtained during all repulsion analysis steps before the final step where  $i = p$  are independent of the earthquake selected.

**7D.5.2** – In case the load-bearing system behavior is linear elastic, Incremental Mode Combination Method is reduced to the linear Mode Combination Method. Since the modal translocation volition will be obtained without any plastic deformation coming out as the yield surfaces are fictitiously expanded, the repulsion analysis in such a case will terminate after a single step and the modal capacity diagrams given in **Figure 7D.1** will consist only of line segments for each.

### 7D.6. References

- [1] Chopra, A.K, Goel, R.K (2002). A modal pushover analysis for estimating seismic demands for buildings, *Earthquake Engineering and Structural Dynamics*; **31**(3): 561-582.
- [2] Goel, R.K, Chopra, A.K, (2004). Evaluation of Modal and FEMA Pushover Analyses: SAC Buildings. *Earthquake Spectra*; **20**(1): 225–254.
- [3] Goel, R.K, Chopra, A.K, (2005). Extension of Modal Pushover Analysis to compute member forces, *Earthquake Spectra*; **21**(1): 125–139.
- [4] Aydinoglu, M. N. (2003). An incremental response spectrum analysis based on inelastic spectral displacements for multi-mode seismic performance evaluation. *Bulletin of Earthquake Engineering*; **1**(1): 3–36.
- [5] Aydinoglu, M. N. (2004). An improved pushover procedure for engineering practice: Incremental Response Spectrum Analysis (IRSA). *International Workshop on Performance - based Seismic Design: Concepts and Implementation*, edited by P. Fajfar and H. Krawinkler, Bled, Slovenia, 28 June – 1 July 2004, Pacific Earthquake Engineering Center, University of California, Berkeley, PEER Report 2004/05: 345–356

## INFORMATION ANNEX 7E

### INCREMENT CALCULATION OF STRENGTH AND DUCTILITY IN THE COULMNS COATED WITH FIBER POLYMER

#### 7E.0. NOTATIONS

|                    |   |   |
|--------------------|---|---|
| $A_s$              | = | Column reinforcement area (for single bar)                                    |
| $b$                | = | Width of the horizontal plates.   |
| $b_w$              | = | Cross section body width  |
| $d$                | = | Effective height of the cross section   |
| $d'$               | = | Thickness of the corrosion allocation.  |
| $E_f$              | = | Elasticity module of the fibrous polymer                                      |
| $f_{cc}$           | = | Compressive strength of the concrete enswathed with fibrous polymer.          |
| $f_{cm}$           | = | Compressive strength of the existing concrete defined according to 7.2        |
| $f_{ym}$           | = | Outflow strength of the existing steel defined according to 7.2               |
| $h$                | = | Dimension of cross section in the direction of                                |
| $f_{hs}$           | = | Tension defiant 0.001 unit lengthening in transversely reinforcement          |
| $f_l$              | = | lateral pressure provided by fibrous polymer                                  |
| $L_s$              | = | Existing overlap length   |
| $n$                | = | Number of overlapped reinforcement  |
| $n_f$              | = | LP winding plate number in one side   |
| $p$                | = | Perimeter of core cross section   |
| $r_c$              | = | Truncation radius made in the corners   |
| $s_f$              | = | Distance of the fibrous polymer band from axis to axis                        |
| $t_f$              | = | Effective thickness for one plate of fibrous polymer                          |
| $V_c$              | = | Contribution of the concrete to the shearing force strength                   |
| $V_f$              | = | Contribution of the fibrous polymer to the shearing force strength            |
| $V_{max}$          | = | Shearing force defined in order to limit main pressure tensions               |
| $V_r$              | = | Shearing strength of the column or cross section                              |
| $V_s$              | = | Contribution of the transversely reinforcement to the shearing power strength |
| $w_f$              | = | Width of the fibrous polymer band   |
| $\varepsilon_{cc}$ | = | Unit shortening correspond to coated concrete pressure strength               |
| $\varepsilon_f$    | = | Effective unit lengthening limit of the fibrous polymer                       |
| $\varepsilon_{fu}$ | = | Breaking unit lengthening of the fibrous polymer                              |
| $\kappa_a$         | = | Cross section figure coefficient of performance                               |
| $\phi$             | = | Diameter of reinforcement   |
| $\rho_f$           | = | Volume ratio of the fibrous polymer   |

#### 7E.1. Increasing the Shearing strength of the Columns

Shearing force strength of column and beams coated with FP is calculated with **Equation (7E.1)**.

$$V_r = V_c + V_s + V_f \leq V_{max} \quad (7E.1)$$

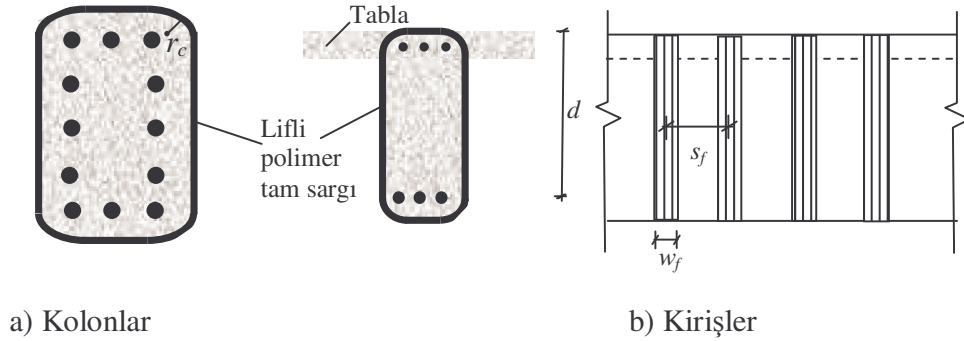
Values of the concrete contribution to shearing force strength  $V_c$ , contribution of transverse reinforcement  $V_s$  and  $V_{max}$  defined in order to limit principle pressure tensile shall be calculated only with *current material strengths* determined according to 7.2 with the Equation suggested by TS - 500. In case the coating realized with strips, contribution of FP coating  $V_f$  to shearing force strength shall be calculated with **Equation (7E.2)**.

$$V_f = \frac{2 n_f t_f w_f E_f \varepsilon_f d}{s_f} \quad (7E.2)$$

In **Equation (7E.2)**,  $n_f$  indicates the number of FP coating layers on one side,  $t_f$  indicates effective thickness for one FP layer,  $w_f$  indicates the width of the FB strip,  $E_f$  indicates the elasticity modulus of FP,  $\varepsilon_f$  indicates the effective unit extension boundary of FP,  $d$  indicates the useful height of element, and  $s_f$  indicates spaces of FP strips from axis to axis (**Figure 7E.1**). In case the coating is made continuously,  $w_f = s_f$  is taken. Effective unit extension value is taken according to **Equation (7E.3)**.

$$\begin{aligned} \varepsilon_f &\leq 0.004 \\ \varepsilon_f &\leq 0.50 \varepsilon_{fu} \end{aligned} \quad (7E.3)$$

In the **Equation (7E.3)**  $\varepsilon_{fu}$  is the unit breaking strain of FP. In case transient FB is used (in strips), spaces of FP strips  $s_f$  shall not exceed  $(w_f + d/4)$  value.



**Figure 7E.1**

## 7E.2. Increasing the Axial Pressure Strength of Columns

In order to increase axial pressure strengths of the columns with FP coating, ratio of the long dimension of column section shall not more than two times the short dimension of the column section. By turning the cross - sections of columns from rectangular to ellipse effectiveness of the FB may be increased. Ratio of long dimension to short dimension in may be maximum three in ellipse sections. While calculating axial load strength of a column coated with FP,  $f_{cc}$  value determined with **Equation (7E.4)** shall be used instead of  $f_{cd}$  for the pressure strength of the concrete.

$$f_{cc} = f_{cm} (1 + 2.4 (f_l / f_{cm})) \geq 1.2 f_{cm} \quad (7E.4)$$

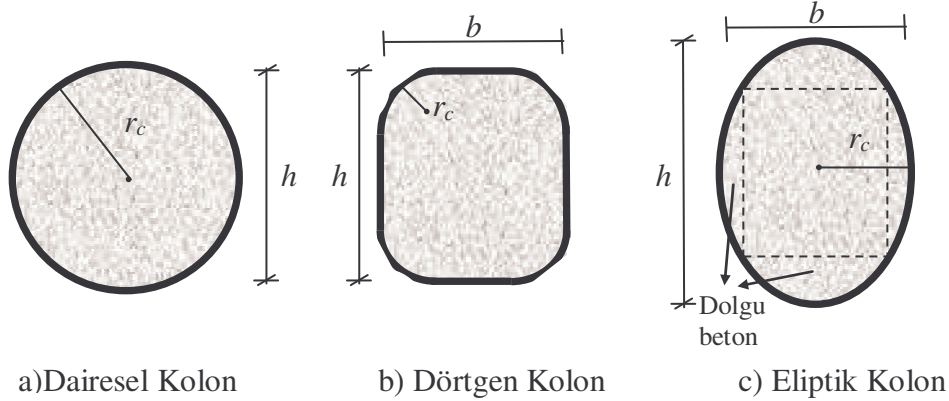
In the **Equation (7E.4)**  $f_{cm}$  is the current pressure strength of un - coated concrete and  $f_l$  is the lateral pressure amount provided by FP coating.  $f_l$  shall be calculated according to **Equation (7E.5)**.

$$f_l = \frac{1}{2} \kappa_a \rho_f \varepsilon_f E_f \quad (7E.5)$$

In the **Equation (7E.5)**  $\varepsilon_f$  shall be calculated with **Equation (7E.3)**. In this Equation  $\kappa_a$  is the effectiveness factor of section shapes, and  $\rho_f$  is the volumetric ratio of Fp.  $\kappa_a$  is given for several sections in **Equation (7E.6)**.

$$\kappa_a = \left\{ \begin{array}{ll} 1 & \text{Dairesel kesit} \\ \left( \frac{b}{h} \right) & \text{Elips kesit} \\ 1 - \frac{(b-2r_c)^2 + (h-2r_c)^2}{3bh} & \text{Dikdörtgen kesit} \end{array} \right\} \quad (7E.6)$$

$B$  and  $h$  shown in **Equation (7E.6)** are the lengths of the short and long sides for rectangular cross sections, relevant dimensions for the short and long sizes of elliptic cross sections and  $r_c$  is the radius of rolling in rectangular cross sections (**Figure 7E.2**).



**Şekil 7E.2**

### 7E.3. Increasing the ductility of columns

In order to increase the ductility of columns via FP coiling, the ratio of the length of the long edge of the column to that of the short edge should not be higher than two. This ratio can not be higher than three in elliptic cross sections. Unit shortening ( $\epsilon_{cc}$ ) that coincides with the concrete pressure tolerance in a column that is coiled via FP can be determined by **Equation (7E.7)**.

$$\epsilon_{cc} = 0.002 \left( 1 + 15 (f_l / f_{cm})^{0.75} \right) \quad (7E.7)$$

$f_l$  in **Equation (7E.7)** shall be calculated by **Equation (7E.5)**. In order to increase the ductility via FP coiling the minimum tolerance increase determined by **Equation (7E.4)** shall be maintained.

(a) While using linear elastic calculation methods, whenever the  $\epsilon_{cc}$  value of any column, as calculated by **Equation (7E.7)**, is greater than 0.018, the column is accepted as coiled; otherwise it is accepted as uncoiled.

(b) For linear inelastic calculation methods, while obtaining the momentum-curvature relation of the FP coiled cross sections, an idealized stretching-deformation relation that shall be composed of two lines may be used for the FP coiled concretes. The stretching and deformation values of this relation can be taken as  $f_c$  (capacity) and 0.002. Values at the last point of the stretching-deformation relation are calculated by **Equation (7E.4)** and **Equation (7E.7)**. In the FP coiled armored concrete carrier system elements, where plastic deformation has occurred, maximum concrete unit shortening value for the cave in the cross section is equal to the value calculated by **Equation (7E.7)**, that for the security limit is 75% of to the value calculated by **Equation (7E.7)** and minimum damage limit shall be taken as 0.004 depending on the performance levels. These values and the unit lengthening values of the equipment steel can not exceed the upper limits set in 7.6.9.

#### 7E.4. Coating for Insufficient Overlapping Length in the Columns

Because coating effect is insufficient for the columns that the section length ratio is more than two or the columns that the longitudinal reinforcements are smooth surface, strengthening of the overlapping zones can not be made with FP coating. Necessary FP thickness for covering insufficiency of overlapping length on the columns of which longitudinal reinforcements are ribbed shall be calculated according to Equation (7E.8).

$$t_f = \frac{500 b_w (f_k - f_{hs})}{E_f} \quad (7E.8)$$

In the **Equation (7E.8)**  $b_w$  is the width of the section,  $f_{hs}$  is the tensile that corresponds 0.001 unit extension in the transverse reinforcement.  $\kappa_a$  factor should be calculated according to **Equation (7E.6)** for different sections. In the **Equation (7E.8)**  $f_k$  value shall be calculated according to **Equation (7E.9)**.

$$f_k = \frac{A_s f_{ym}}{\left[ \frac{p}{2n} + 2(\phi + d') \right] L_s} \quad (7E.9)$$

In the **Equation (7E.9)**  $A_s$  is the area of column reinforcement (for single strip),  $f_{ym}$  is the yield strength of current reinforcement,  $p$  is the periphery of core section,  $n$  is the overlapped reinforcement number,  $\phi$  is the diameter of reinforcement,  $d'$  is the thickness of rust and  $L_s$  is the existing overlapping length.

## INFORMATION ANNEX 7F

### METHODS FOR STRENGTHENING OF THE INFILLED WALLS

#### 7F.0. NOTATIONS

|                         |   |   |
|-------------------------|---|---|
| $A_{\text{wall}}$       | = | Horizontal cross section area of the filling wall   |
| $a_{\text{wall}}$       | = | Width of the equivalent pressure bar (mm)   |
| $E_c$                   | = | Elasticity module of frame concrete   |
| $E_{\text{dp}}$         | = | Elasticity module of the prefabricate concrete wall panel   |
| $E_{\text{wall}}$       | = | Elasticity module of filling wall   |
| $f_{\text{dp}}$         | = | Pressure strength of the prefabricate concrete wall panel   |
| $f_{\text{wall}}$       | = | Pressure strength of filling wall   |
| $f_{\text{yd}}$         | = | Design yield strength of the mesh reinforcement steel   |
| $h_{\text{wall}}$       | = | Height of the filling wall (mm)   |
| $I_k$                   | = | Inertial moment of the column (mm <sup>4</sup> )  |
| $k_{\text{wall}}$       | = | Axial rigidity of the equivalent pressure bar   |
| $k_t$                   | = | Axial rigidity of the wall drawbar which was strengthened with fibrous polymer                                  |
| $l_{\text{min}}$        | = | Minimum depth of anchorage bar  |
| $r_{\text{wall}}$       | = | Diagonal length of filling wall   |
| $s_{\text{max}}$        | = | Maximum distance of anchorage bar   |
| $t_{\text{dp}}$         | = | Thickness of the prefabricate concrete wall panel (mm)  |
| $t_{\text{wall}}$       | = | Thickness of the filling wall   |
| $T_f$                   | = | Tensile strength of the wall drawbar which was strengthened with fibrous polymer                                |
| $t_f$                   | = | Thickness of the fibrous polymer  |
| $V_{\text{wall}}$       | = | Shearing force strength of the filling wall   |
| $\phi_{\text{min}}$     | = | Minimum diameter of anchorage bar   |
| $\lambda_{\text{wall}}$ | = | Coefficient of equivalent pressure bar  |
| $\rho_{\text{sh}}$      | = | Ratio of horizontal body reinforcement in the curtain and wall to the brut cross sectional area of curtain body |
| $\theta_i$              | = | Angle of equivalent pressure bar with horizontal one  |
| $T_{\text{wall}}$       | = | Sliding strength of the filling wall  |
| $T_{\text{dp}}$         | = | Sliding strength of the prefabricate concrete wall panel  |

#### 7F.1. Strengthening the Filled Walls

The rigidity and the shear strength of the filled walls in the reinforced concrete frame that is continuous beginning from the top of grounding up to the top may be improved using the methods given below provided that the methods are applied as per 7.10.4 to at most three-floor buildings excluding the basement floor.

#### 7F.2. Strengthening the Filled Walls using Matted Steel Reinforced Special Plaster

The rigidity and the shear strength of the filled walls can be improved with special-mixture matted steel-reinforced plaster layer to be applied on the wall surface.

(a) The thickness of the plaster layer should be minimum 30 mm and the rust space of the matted reinforcement should be minimum 20 mm. Special-mixture plaster will be made with 4 volume sand, 1 volume cement and 1 volume lime mixture. The pressure strength of the plaster made in this mixture composition will be minimum 5.

(b) The ratio of the diagonal length of the wall to be strengthened to the thickness before the strengthening is applied should be lower than 30. For this type of applications, formation of pressure stick in the existing frame should be ensured and the anchors necessary to maintain the load transfer to the frame should be designed. Because of this requirement, there should a splicing of at least 30 mm deep between the wall surface the application will be made on and the outer surface of the frame components (**Figure 7F.1**). Otherwise, this type of wall strengthening cannot be applied.

(c) The minimum value of the diameter of the frame anchoring stick that will be used between the reinforcement - plaster layer and the frame components should be 12 mm, minimum anchor depth should be ten times of the stick diameter and the widest stick splicing should be 300 mm. Moreover, so as to enable the reinforced plaster layer and the existing filled wall to work together, body anchoring perpendicular to the wall axis should be applied as four anchors per one square meter wall area. The body anchoring sticks to be applied perpendicular to the wall will be buried in the mortar joints of the filled wall provided that the minimum diameter of the stick of 8 mm and the anchoring dept will be at least ten times of the stick diameter. All anchor sticks applied perpendicular or parallel to the wall axis will be implemented in the holes to be born with an epoxy - based material and the edges will be put through the matted reinforcement, bending the edges 90° to become in L shape. The details on the application are shown in **Figure 7F.1**.

(d) To enable the forces arising in the strengthened filled walls to be transferred to the ground, required grounding arrangement should be implemented. The walls strengthened with matted steel reinforcement will be included in the structure model in accordance with the principles given below.

**7F.2.1 – Modeling Principles:** The rigidity and strength features to be used so as for the filled walls strengthened with matted steel reinforcement to be represented in the structure model are defined below. Filled walls that are designed in reinforced concrete frames and the ratio of the diagonal length to the thickness for which is below 30 will be taken into consideration in the structural modeling. The walls that include splicing the ratio of which to wall surface doesn't exceed 10 % may be included in the structure modeling provided that the positions of the splicing don't block the formation of diagonal pressure stick. Filled walls strengthened with matted steel will be represented with equivalent diagonal stick components that receive pressure force in the direction that the earthquake is applied.

(a) *Rigidity:* The thickness of the equivalent pressure stick is equal to the thickness of the strengthened filled wall. The width  $a_{wall}$  will be calculated using **Equation (7F.1)**.

$$a_{wall} = 0.175 (\lambda_{wall} h_k)^{-0.4} r_{wall} \quad (7F.1)$$

In this formulation,  $a_{wall}$  is the width of the stick (mm),  $h_c$  is the length of the column (mm) and wall is the diagonal length of the filled wall (mm).  $\lambda_{wall}$  will be calculated using **Equation (7F.2)**.

$$\lambda_{wall} = \left[ \frac{E_{wall} t_{wall} \sin 20}{4 E_c I_k h_{wall}} \right]^{\frac{1}{4}} \quad (7F.2)$$

In **Equation (7F.2)**,  $E_{wall}$  and  $E_c$  represent the elasticity module of the filled wall and the frame concrete,  $t_{wall}$  and  $h_{wall}$  represents the thickness and the height of the strengthened wall (mm),  $I_k$  represents the inertia momentum of the column (mm<sup>4</sup>) and  $\theta$  represents the angle between the diagonal and the horizontal surface. The axial rigidity of the diagonal pressure stick will be calculated using **Equation (7F.3)**.

$$k_{wall} = \frac{a_{wall} t_{wall} E_{wall}}{r_{wall}} \quad (7F.3)$$

*Shear Strength:* The horizontal component of the equivalent pressure force strength of the diagonal stick will be considered as the shear force of the filled wall strengthened with matted steel reinforcement. The shear strength of the filled wall,  $V_{wall}$ , with horizontal section area  $A_{wall}$ , pressure strength  $f_{wall}$  and shear strength  $\tau_{wall}$  will be calculated with **Equation (7F.4)**.

$$V_{wall} = A_{wall} (\tau_{wall} + f_{yd} \rho_{sh}) \leq 0.22 A_{wall} f_{wall} \quad (7F.4)$$

Here  $f_{yd}$  represents the design yield strength of the matted reinforcement, and  $\rho_{sh}$  represents the ratio of the horizontal body reinforcements of the wall to the gross cross section area. Matted reinforcement should have the same reinforcement area in horizontal and vertical axes.

**7F.2.2 – Material Qualifications:** The recommended values for filled walls built with different types of bricks in accordance with the  $E_{wall}$ ,  $f_{wall}$  and  $\tau_{wall}$  mentioned above are given below. The composite sectional structure of the strengthened wall can be considered in pressure and shear forces of the elasticity module.

Manufactured air brick:

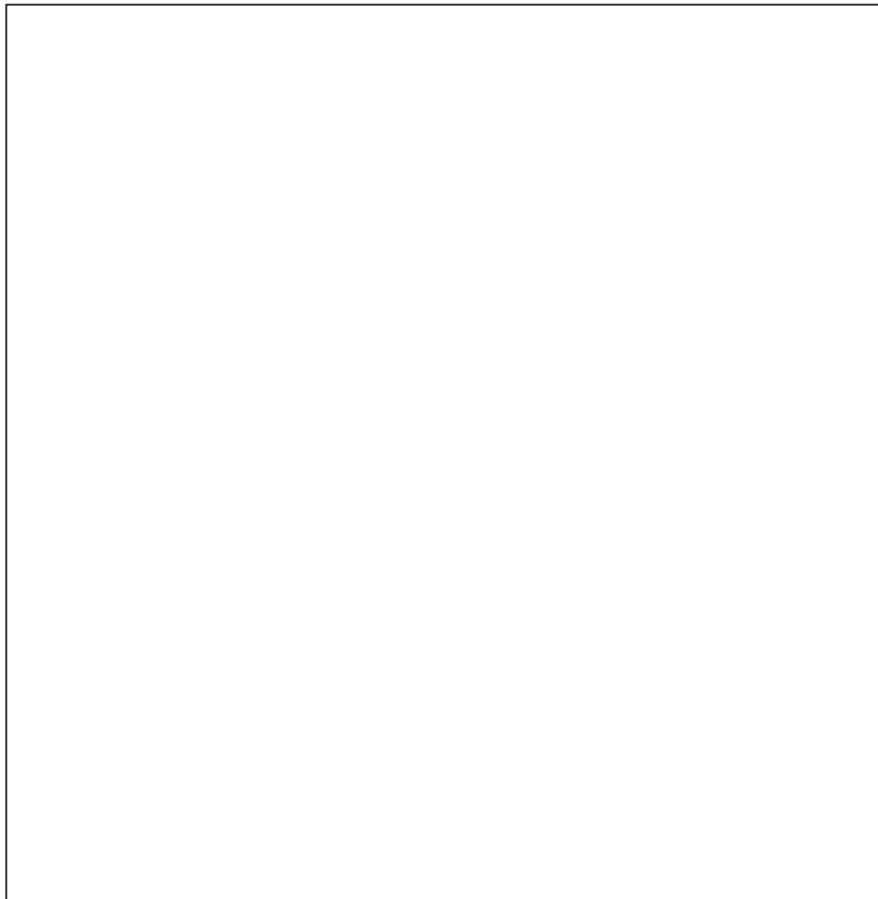
$$E_{wall} = 1000 \text{ MPa}; f_{wall} = 1.0 \text{ MPa}; \tau_{wall} = 0.15 \text{ MPa} \quad (7F.5a)$$

Blend brick:

$$E_{wall} = 1000 \text{ MPa}; f_{wall} = 2.0 \text{ MPa}; \tau_{wall} = 0.25 \text{ MPa} \quad (7F.5b)$$

Gas concrete:

$$E_{wall} = 1000 \text{ MPa}; f_{wall} = 1.5 \text{ MPa}; \tau_{wall} = 0.20 \text{ MPa} \quad (7F.5c)$$



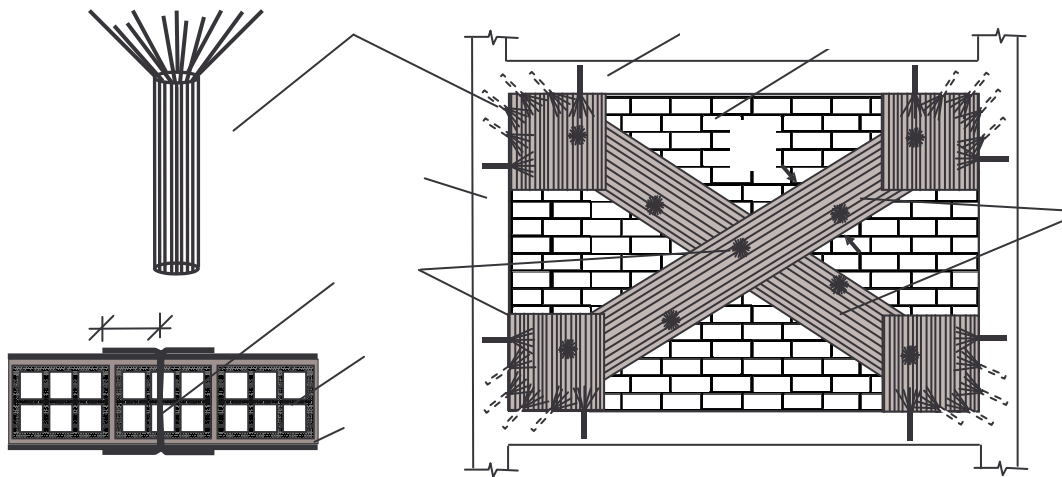
**Figure 7F.1**

### 7F.3. Strengthening Filled Walls with Fiber Polymer

The rigidity and the shear strength of the filled walls with length-height ratio between 0.5 and 2 can be improved by means of the fiber polymer (FP) applied to the wall surface.

(a) For these kinds of applications the formation of the pressure stick in the existing frame should be ensured and anchors required sustain load transfer to the frame should be arranged. For this reason, there should a splicing of at least 30 mm deep between the wall surface the application will be made on and the outer surface of the frame components. Otherwise, this type of wall strengthening cannot be applied.

(b) The details regarding the diagonal-fiber polymer straps are shown in **Figure 7F.2**. Square FP plates the width of which is not lower than the 1.5 times of the strap width will be used to ensure the load distribution in the edge zones and to be able to install sufficient number of anchors between the reinforced concrete frame and the FP straps. Fiber polymer will be applied to both sides of the wall and the FP straps will be fixed to the wall by means of the FP bolts passing through the whole wall thickness. The gap between the FP bolts cannot be over 600 mm and the distance between the belt and the diagonal strap edge cannot be over 150 mm. FP anchors will be used to enable the load transfer between the diagonal FP strap and the frame. FP anchors will be formed via feeding the FP straps with epoxy and rolling them around a silicon stick. The end edges of the FP anchors will be given shape of a fan and minimum 4 anchors will be fitted by injecting epoxy to the dust-free hole, born in the concrete in a manner that the diagonal will be in the direction of the FP strap. The width of the FP rolled around the stick to prepare the anchor should not be below 100 mm. the diameter and the width of the anchor hole will not be below 10 mm and 150 mm, respectively. For the anchor prepared following the way explained, the one smaller in value among 20 kN and 30 % of the shear strength of the FP rolled around the silicon stick will be considered as the shear strength.



**Figure 7F.2**

(c) To enable the forces arising in the strengthened filled walls to be transferred to the ground, required grounding arrangement should be implemented. The walls strengthened with fiber polymers will be included in the structure models as per the principles given below.

**7F.3.1 – Modeling Principles:** The filled walls strengthened with fiber polymer will be represented in the structure model by the couple diagonal pressure and pulling sticks.

(a) *Pressure Sticks:* The rigidity and the shear force of the pressure sticks will be calculated in accordance with **7F.2.1 (a)** and **(b)**.

**(b) Pulling Sticks:** Pulling strength of the pulling stick  $T_f$  will be calculated using **Equation (7F.6)**.

$$T_f = 0.003 E_f w_f t_f \quad (7F.6)$$

Shear strength of the pulling stick will be considered as the horizontal component of the pulling strength. Axial rigidity of the pulling stick will be calculates using **Equation (7F.7)**.

$$k_t = \frac{w_f t_f E_f}{r_{wall}} \quad (7F.7)$$

Here,  $E_f$ ,  $w_f$  and  $t_f$  represent the elasticity module, width and the thickness of the fiber polymer strap, respectively, while  $r_{wall}$  represents the diagonal length of the filled wall. The value taken for  $w_f$  cannot be bigger than the width calculated using **Equation (7F.1)**.

#### 7F.4. Strengthening the Filled Walls using Prefabricated Concrete Panels

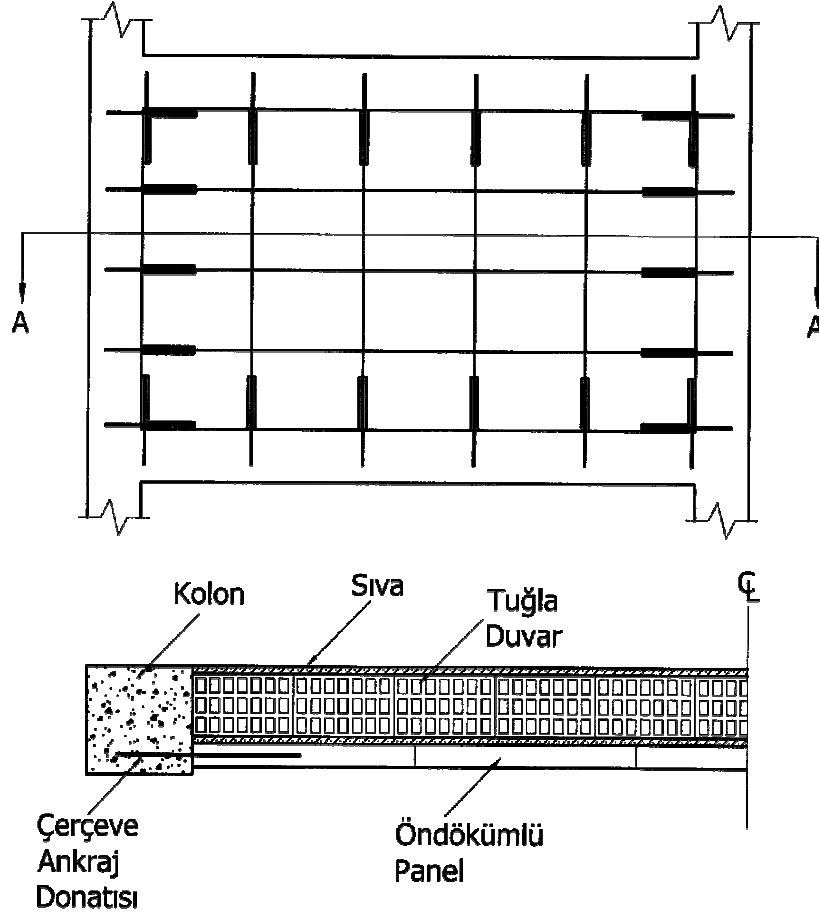
The shear strength and the rigidity of the filled walls can be improved using pre-casting concrete panel components. This type of strengthening should be applied to the walls for which the length-height ratio varies between 0.5 and 2.

**(a)** Pre-casting panels will be fitted in a way that the panel certainly remains within the frame, they won't be fitted eccentrically. For this type of applications, formation of pressure stick in the existing frame should be ensured and the anchors necessary to maintain the load transfer to the frame should be designed. Because of this requirement, there should be number of equal-width splicing that is at least as thick as the wall between the wall surface the application will be made on and the outer surface of the frame components (**Figure 7F.3**). Otherwise, this type of wall strengthening cannot be applied.

**(b)** The ratio of the diagonal length of the walls to be strengthened with prefabricated concrete panels to the pre-strengthening thickness should be smaller than 30. Pressure strength of the pre-casting panel concrete will be minimum 40 MPa. To minimize the shrinkage cracks and the cracks to be possibly formed during the movement, matted reinforcement in single axis will be put in the middle of the panel. Matted reinforcement rate in each axis will not be below 0.001. Minimum and maximum panel thicknesses will be 40 mm and 60 mm, respectively. The panels will be fitted to the walls using an epoxy-based adhesive. The adhesive will as well be applied to the areas between the components to bind the panel components. The strength of concrete adhesion for the epoxy-based adhesive to be used will be minimum 2.5 MPa. The minimum anchor stick diameter that will be applied between the pre-casting panels and the frame components will be 12 mm and the anchor depth will be at least ten times of the stick diameter. The anchors should be applied to all edges of the panels in connection with the frames and will be fitted to the frame using an epoxy-based adhesive. The panel edges in connection with the anchor should be produced in a way that it contains gears leaving space for the anchor stick (**Figure 7F.3**).

**(c)** Prefabricated concrete panels can be in square or square-like rectangular shape provided that its weight enables it to be carried easily by two people or can be produced in form of straps that can take place among the floors of the building. To enable the forces arising in the strengthened filled walls to be transferred to the ground, required grounding arrangement should be implemented. The walls strengthened with prefabricated concrete panels will be included in the structure model in accordance with the principles given below.

**7F.4.1 – Modeling Principles:** Filled walls that are designed in reinforced concrete frames and the ratio of the diagonal length to the thickness for which is below 30 will be taken into consideration in the structural modeling. Filled walls strengthened with prefabricated concrete panels will be represented with equivalent diagonal stick components that receive pressure force in the direction that the earthquake is applied.



**Figure F.3**

(a) *Rigidity:* The rigidity of the equivalent pressure stick will be calculated as per **7F2.1 (a)**. Instead of  $E_{wall}$  and  $t_{wall}$ , elasticity module and the thickness of the prefabricated concrete panel  $E_{dp}$  and  $t_{dp}$  should be substituted in **Equation (7F.3)**. The existing filled wall will not be included in the calculation.

(b) *Shear Strength:* The shear strength of the filled wall strengthened with prefabricated concrete panels will be considered as the horizontal component of the equivalent pressure force strength of the diagonal stick. The reinforcement fitted in the panel components will not be taken into consideration in calculating the shear force of the strengthened filled wall. Moreover, the existing filled wall will as well not be considered in the calculation. Shear strength of the strengthened filled wall,  $V_{wall}$ , for which  $A_{dp}$  represents the horizontal section area,  $f_{dp}$  represents the pressure strength and  $\tau_{dp}$  represents the slipping strength, will be calculated using **Equation (7F.8)**.

$$v_{wall} = A_{dp} \tau_{dp} \leq 0.08 A_{dp} f_{dp} \quad (7F.8)$$