SEISMIC DAMAGE OBSERVED ON PREFABRICATED INDUSTRIAL STRUCTURES AFTER 1999 EARTHQUAKES IN TURKEY AND PROTECTING MEASURES

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ABSTRACT :

Field investigations after the 1999 Kocaeli and Düzce Earthquakes revealed that the damage in such structures was primarily due to the improper detailing in beam-column connections and lack of required lateral stiffness. In this study, extensive, serious damage and failures in prefabricated reinforced concrete industrial structures in the 1999 Marmara Earthquakes have been examined. To investigate the causes of the damage beside the assessment of practical imperfections, a comparision between the sets of criteria encompassed in the TEC-98, UBC-97 and EC8-98 Codes especially on the criteria of base shear force, displacement and loads acting on the nodes/connections has been made. In order to compare the above-mentioned codes a sample prefabricated structure is analysed. From this study, it is observed that the capacity dictated from TEC-98 is lower than the capacity of the other two codes. This deficiency is removed in TEC-2007.

KEYWORDS : Earthquake, damage, prefabricated structures, seismic codes, measure

1. INTRODUCTION

The fundamental reasons lying beneath the use of prefabricated construction can be enumerated as: -reduction of construction period, -improved quality-control of member fabrication in the factory environment, -increased rate of production and economy compared to conventional construction techniques. Recently, the prefabricated frame and large panel systems are used in Turkey (Fig.1). Contrary to the advantages that prefabricated technology provides, failures and damages observed in the structures erected using this technique in destructive earthquakes that hit the country especially in the last 10 years (such as 1992 Erzincan *Mw 6.8*, 1996 Adana-Ceyhan *Mw 6.3*, 1999 Adapazari-Izmit *Mw 7.4*, 1999 Düzce *Mw 7.2*), showed the need for the re-examination of the criteria in the Turkish Earthquake Code (TEC) and revision of the code deficiencies relative to the criteria in UBC and EC-8 (the codes in the United States and the European Union, respectively).

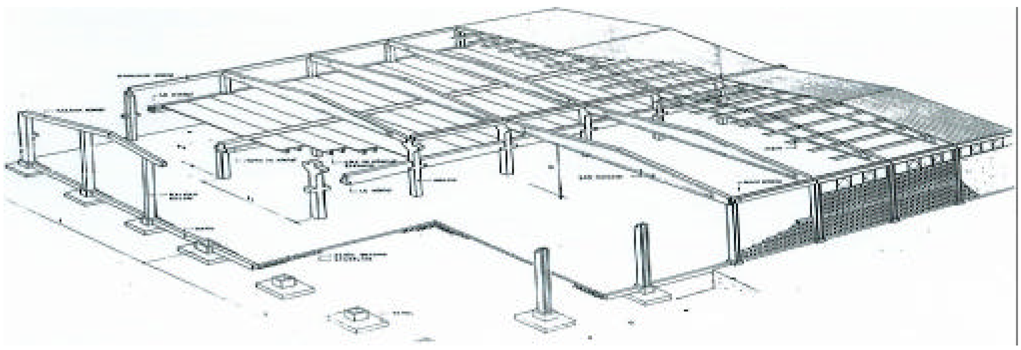

Figure 1 Prefabricated frame system

This study focusses on the analyses of single-storey prefabricated industrial structures, according to the criteria deficiencies in the Turkish Code which are apt to cause damage. In order to compare the abovementioned codes a sample prefabricated structure, which totally failed after 1999 Marmara Earthquake is analysed. Moreover, damage and failure types observed in the prefabricated reinforced concrete buildings after the earthquakes that hit Turkey in recent years will be examined and recommendations for strengthening will be put forward.

2. SEISMIC DAMAGE OBSERVED ON PREFABRICATED STRUCTURES

Depending on the supervision of the concrete fabricated and reinforcement qualities of the prefabricated members, no material-based deficiency has been observed in the prefabricated structures (Aydogan, 2002; Arslan, 2006). Satisfaction of sufficient strength, lateral rigidity and ductility, is a crucial concern for such structures. Previous research demonstrated that the most vital complication in prefabricated structures is sustaining the lateral loads exerted during earthquakes (Tankut, 2000). The industrial-type structures especially with their fixed-base and crown-hinged (reverse-pendulum type) design tend to convert the entire lateral load to displacement by Eqn. 2.1. In Fig. 2a, a system possessing monolithic joints and in Fig. 2b, a crown-hinged system are presented. The displacement in the latter system far exceeds that of the former. Another significant distinction among the systems is the variation in the location of the potential plastic hinge formation. Whereas in Fig. 2a hinging occurs at both tips of the columns with a moment having a value of u, in Fig. 2b, it forms at the bottom ends of the columns with a moment reaching a value of U, where U is twice u. The weakness of the crown-hinged prefabricated system regarding both lateral rigidity and ductility can clearly be seen (U>u, M>m, $d_2>d_1$). In the light of the above explanation, the pictures of failed prefabricated structures are worth examining (Fig. 2).

$$d = F.h_c^3 / (3E_cI_c) \tag{2.1}$$

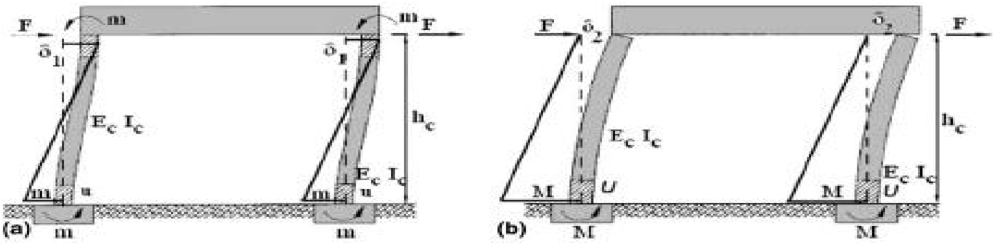

Figure 2 Monolithic and hinged prefabricated systems

TEC-98 states that the structural behaviour coefficient R may be assumed as 5 for hinged connection frames. When this value is compared with that of in situ cast frames, i.e., 8, it corresponds to a 60% increase in lateral loads for the former. The inter-storey drift is limited to a maximum value of 0.0035. Furthermore, while the code allows only 1 storey construction for crown-hinged structures, it also insists on the requirement of in situ cast shear wall application for higher storey construction. Following the earthquakes, numerous totally failed and many highly damaged prefabricated buildings were reported (Zorbozan, 1998; Ersoy, 2000). An inventory illustrating the damage status of the buildings constructed in the region by the Turkish Prefabricated Construction Association member firms is given as follows: total number of precast building 481, heavily damaged 17, partially damaged 14 and non-damaged 450. Even though the number of damaged structures seems low, tens of failed buildings were excluded from these records. In Fig. 3, the post-earthquake view of a structure is portrayed.

Figure 3 Overall collapse of the prefabricated systems

Common damages and their types encountered in single-storey hinged-connection prefabricated frames are itemized below (Arslan, 2006).

1) Insufficiencies in detailing in the regions where inclined roof-beams meet with columns: -The connection between columns and beams is a moment-free hinged-connection. The friction force holding the beam on the support, in other words, is dependent on the weight of the beam, friction coefficient between the supporting surfaces, and shear resistance provided by the transverse section. -Inclined roof-beams rest on the corbels attached to the columns. A pair of 12–20mm diameter reinforcements extended outside each corbel crossing the holes at the head of the beam and finally the remaining ring-like gaps in the holes are filled with mortar. Roof-beams are connected perpendicularly to each other only via secondary concrete beams extending in the longitudinal direction of the structure. These beams rest on 10–12mm reinforcement extensions on the inclined roof-beams with a simple detailing. These reinforcement bars are covered and the remaining voids are filled with mortar upon finishing. The seismic lateral inertia forces acting perpendicular to the inclined roof-beams could not be transferred to the column tops. The reason for that is inadequacy of the supports. As a result these beams overturned and leaned upon each other like dominoes.

2) In the design calculation of these connection members, the overturning moments and shear forces generated due to inertia forces acting orthogonal to the frame must also be accounted for and the diameters and anchorage lengths of the reinforcement members designated accordingly. The anchorage fasteners are displaced and the beam is detached from the column.

3) Another type of failure usually occurs due to miscomputations in the roller-support detailing. In the dilatation region the beams must be located so that a definite amount of displacement is released. In doing that, during earthquakes these beams must not slip from the supporting points when the separate parts of the structure are displaced relative to each other. But these details were not applied and were failures observed as a result.

4) Formation of bending cracks at lower sections of the columns is a widespread failure type. This type of failure is the indicator of columns exceeding the ultimate elastic moment bearing strength at lower sections. The inadequacy of the column cross-sections especially in the out-of-plane direction (the asymmetrical approach in column-design) and exceedingly dense arrangement of the reinforcement are the principal reasons for column lower section failures. The bending cracks investigated at the socket level of the columns of a coffee-processing factory erected in 1996, Izmit.

5) Vertical cracks have mostly been observed on the lateral facades of the beams resting on the corbels, in one of the planes cutting the holes, located at the beam-tip and through which the reinforcement extended from the corbels. In some structures, it was seen that one of the tips of the beam extending in between the adjacent columns had fallen down slipping out of the reinforcement bars. Meanwhile, the tip of the beam also bruised the concrete tip of the corbel. Reinforcement bars extending from the corbels slipped out of the beams upon bending.

6) The excessive disproportionate displacements, exceeding the values in the code, of the column tips due to slenderness and consequently the damage caused by the secondary members failing to

generate the displacements in comparable amounts are the sources of a different failure type.
7) The presence of exterior walls besides non-existence of interiors particularly in buildings having two or more spans resulted in differential displacements out-of-plane. In such structures, columns located in the span experience more out-of-plane displacement compared to the column-couples at the sides. In some prefabricated systems, damages concerning this type of failure were observed.

Fig. 4 has been shot from a double-span prefabricated industrial structure constructed in Adapazari Organised Industrial Zone. Columns in the middle were damaged (and failed) significantly compared to the columns in the edges. Fig. 4 shows the post-earthquake state of the interior-column and interior-beam connection section of a triple-span one-storey industrial site constructed in Izmit.

Figure 4 Seismic demage of the system

In the mentioned structures, no rupture, bruising or cracking has been observed in the filling concrete poured between the column and the socket foundation. Only following the 12 November 1999 Düzce Earthquake, in the foundation sockets of a damaged structure, located in Bolu-Kaynasli province, were some cracks detected. In almost none of the damaged structures in Adapazari Organised Industrial Zone has a rupture in sockets observed. What is more is that observing bending failures in the lower sections of the columns embedded in the sockets is another crucial piece of evidence of the fixed-behaviour of the column-foundation connections. In the direction of the material tests conducted in the existing prefabricated structures, some Schmidt Hammer readings had been taken and these tests illustrated that strengths of concrete members tested are in accordance with the limitations given in the code. The strength values for concrete qualities suggested in the design, namely C25 and C30, satisfy the empirically obtained strength values. In mechanical and chemical analyses of the construction steel, it was seen that the carbon content considerably exceeded the upper limit thus giving the reinforced concrete a brittle character. In many prefabricated structures, in beams of which no falling down or toppling occurred, cracks in the beams and also, bruising in the edges of the upper facade of the corbels has been observed. Concentration of the damages in some regions illustrates the significance of both the better designation of the local ground subgroups and intensifying effect of the alluvial soil.

3. SEISMIC CODES RELATED WITH PREFABRICATED STRUCTURES

Records of some seismic codes related with prefabricated structures are given in below.

3.1. Turkish Earthquake Code, TEC-98

The Turkish Code for the Structures in Disaster Regions which came into operation in 1975, has become inadequate due to advancements in structural technology and earthquake engineering over the years. The base shear force is computed by using Eqn. 3.1. Here, A_0, denoting the effective ground acceleration coefficient, takes the values 0.10g for the 4th and 0.40g for the 1st degree seismic risk zones, respectively. I, the structural importance factor, takes values varying in between 1 and 1.5 and takes 1 for a conventional reinforced concrete structure. S(T), the spectrum coefficient, is represented by a curve which gives the values of design acceleration spectrum varying with the natural period, T, of the structure. The type of the curve differs depending on the ground characteristics and each curve type gives a value of 2.5 at maximum.

$$V_t = WC = WA(T_1)/R_a(T_1) = 0.10A_0IW \quad , \quad C = A(T_1)/R_a(T_1) \quad , \quad A(T_1) = A_0.I.S(T) \qquad (3.1)$$

$$W = \Sigma g_i + nq_i \quad , \quad R_a(T) = 1.5 + (R-1.5)T/T_A \quad (0 £ T £ T_A) \quad , \quad R_a = R \quad (T > T_A) \qquad (3.2)$$

$$S(T) = 1 + 1.5T/T_A \ (0 £ T £ T_A) \ , \quad S(T) = 2.5 \ (T_A < T £ T_B) \quad S(T) = 2.5(T_B/T)^{0.8} \quad (T > T_B),\ S > 0.1R \qquad (3.3)$$

T_B is the corner period appointed regarding the ground type, R_a defined as the earthquake reduction or behavioural coefficient is the indicator of the structural ductility adopting values varying between 3 and 8. For prefabricated structures with fixed connections at the ground level and hinges at upper ends of the members, the coefficient becomes 5. One of the fundamental changes in the 1998 Code is the dual classification of the structural systems namely those with high ductility and normal ductility. With regulations on detailing given in the Code, structural systems possessing high ductility values can be designed. With higher ductility values, conditions given in the code for detailing, which are directly in correlation with the structural ductility, such as the stirrup spacing, calculations of column-beam intersection zones, arrangement of the compression reinforcement in the cross-section, and concrete quality have been becoming rigorous to satisfy the reduction of the C coefficient in the earthquake load computation. Thus the structure has been rewarded for ductile behaviour. The converse holds true for systems, in which ductility levels are normal. Under intensive seismic motions, due to the elasto-plastic deformation owing to ductility, large lateral displacements form, resulting in the formation of secondary moments. To keep secondary moments at a minimum, inter-storey drifts, in other words, the relative storey displacements, have been limited in the code. Eqn. 3.4 shows TEC-98 displacement criteria. Since the R-value equals 5 in the buildings where all the earthquake loads are met by single-storey frames this limit can be expressed as Eqn. 3.4b. The elastic natural period in single-storey industrial structures can be calculated with Egn.3.5a. m stands for the total mass borne by a single-axis system, h=l is the effective column height and EI is the total elastic rigidity of all the members on the axis in question. For lateral and axial seismic loads acting on the corbel-like members operating in a single floor such as balconies, parapets, and chimneys and in non-structural members such as siding and division panels, the following formula 3.5b is used. Here, W_p refers to the weight of the architectural member.

$$(?_i)_{max} = 0.0035h \quad , \quad (?_i)_{max} = 0.02h/R \quad (a) \qquad (?_i)_{max} = (0.0035h,\ 0.004h)_{min} \quad (b) \qquad (3.4)$$

$$T = 2p[h^3?\ m/(3EI)]^{1/2} \quad , \quad m = W/g \quad (a) \qquad F_p = A_0IW_p \quad (b) \qquad (3.5)$$

3.2. United States Building Code, UBC-97

The lateral earthquake load coefficient is computed using the formula 3.7. The computed values should not lie outside the maxima and minima. Z refers to the earthquake zone coefficient. The structural behaviour coefficient, R takes the value of 2.2 for prefabricated structures. N_v is the earthquake proximity coefficient. C_v and C_a are the velocity and acceleration spectrum coefficients. The respective elastic displacement ratio among storeys is defined as Eqn. 3.8a and it is kept in between the limit given in Eqn. 3.8b. For the earthquake loads F_p acting on the joints and for the lateral load coefficient Cp, Eqn. 3.9 is used.

$$V = WC = W(C_vI/RT) \quad , \quad C = C_vI/RT \qquad (3.6)$$

$$V < WC_{maks} = W(2.5C_aI/RT) \ , \quad V > WC_{min1} = W(0.11C_aI) \qquad V > WC_{min2} = W(0.8ZN_vI/R) \qquad (3.7)$$

$$s = d_e/h \quad (a) \ , \quad \leq 0.025/(0.7R) \quad (T < 0.7\ sn) \quad , \quad \leq 0.020/(0.7R) \quad (T = 0.7\ sn) \quad (b) \qquad (3.8)$$

$$F_p = W_pC_p = W_p(4C_aI) \leq (0.70C_aI)W_p \quad \text{or} \quad F_p = W_pC_p \leq (a_pC_aI/R_p)(1+3h_x/H)W_p \qquad (3.9)$$

3.3. Eurocode 8-98

In the code, for the computation of the base shear force Eqn. 3.10 is used. Here, W is the total weight of the structure, while C_d stands for the design spectrum coefficient. The design spectrum coefficient C_d is given by Eqn. 3.11. Here, A_0 is the maximum ground acceleration coefficient for design purposes, while S is the ground type coefficient. The coefficient R is obtained by the multiplication of the base value R_0 by the factors presented in Eqn. 3.12a. k_d, given in the equation, refers to the quality level of the structure's ductility. In the design computations of structures possessing high ductility, k_d is taken as 1 while for medium-ductility it equals 0.5k_p, however, is a coefficient related with the locus of joints (connections) and varies between 1.0 and 0.75. The code assumes the value of the structural behaviour coefficient, R_0, as 2.0 for prefabricated structures. EC-8 limits the respective displacements of floors using the relation 3.12b. ?, given in Eqn. 3.12b, is a parameter in relation to the importance factor and varies between 2.0 and 2.5. For lateral earthquake loads acting on the connections at the roof level in the reverse-pendulum type of prefabricated industrial building Eqn. 3.12c is proposed. Being a security parameter, r is taken as 2 in medium and high ductility regions bearing critical loads, and 1.5 in low-ductility locations.

$$V_b = C_d W \quad , \quad C_d = 2.5 A_0 S/R \quad (T_B £ T £ T_C) \tag{3.10}$$

$$C_d = 2.5 S A_0 (T_C/T)^{2/3}/R \quad (T_C £ T £ 3) \qquad C_d = 2.5 S A_0 (T_C/T_D)^{2/3}/(T_D/T)^{2/3}/R \quad (T=3) \tag{3.11}$$

$$R = k_d k_p R_0 \quad (a) \;,\quad s = d_e/h \leq (0.004?)/(k_d R_0 I) \quad (b) \;,\quad F_p = (3 A_0 r I/R_0) W_p \quad (c) \tag{3.12}$$

4. COMPARISON OF PROPOSED CODES FOR A MODEL BUILDING

As a model building, an industrial structure designed according to TEC-98 will be analysed. The type of building is known as a reverse-pendulum type, i.e., a reinforced concrete prefabricated production industrial structure possessing a two 20m transverse spacing and 9m and 11m height with connections non-transmitting moments. The system has a six equal span of 12m in the long direction. The enclosed area is 40mx72m. One axis composed of two spacings weighs 200 kN. An 85 kN snow load is added to the overall weight. Having a 30 MPa ultimate strength, the concrete has a modulus of elasticity of 32,000 MPa. The dimensions of the cross-sections of the columns are 65cm x 65cm. The cross-sections of the beams are I shape in short direction and H shape in long direction. The shape of beams in floor plane is the reverse-omega section. All beams have the prestressing. The related factors and loads are taken as follows: A_0=0.4, I=1, R=5, Z_3 (S_D, B), T_A=0.15s, T_B=0.6s, the roof covering load=0.12 kN/m^2, snow load=0.75 kN/m^2. The equivalent static lateral earthquake loads, ratios, and upper limits of the inter-storey drifts computed in an elastic analysis and the loads estimated to act on the connection regions have been, respectively, calculated for TEC-98, UBC-97 and EC8-98 and the results and assumptions are given in tables.

Table 4.1 Comparison of base shear forces calculated concerning different codes

TEC 98; Soil type Z3, A_0=0.40, I=1, T=0.71s , T_B=0.6s., S=2.18 V= **0.174**W , V>0.04W
UBC 97; S_D, C_v=0.64, I=1, R=2.2 , T=0.71s V = **0.41**W , V<0.5W , V>0.05W , V>0.14W
EC8 98; Soil B, S=1, Ro=2.5, T_C=0.6s, T=0.71s, k_d=1, k_p=0.75, R_0=2 R=1.5 V=**0.447**W, V>0.06W

Table 4.2 Comparison of the estimated loads acting on connections

TEC 98 ; Soil Z3, W_p , A_0=0.4, I=1, R=5, T=0.71s, T_B=0.6s, S=2.18, k_0=1.2, F_p= **0.209**W_p
UBC 97; Soil S_D, W_p , C_a=0.44, I_p=1, R_p=3 , a_p=1, h_p=h_x=9.00m, F_p=**1.76**W_p , F_p= **0.59**W_p
EC8 98; B, W_p , A_0=0.3, S=1 , r=2, R_0=2 F_p = **0.89**W_p

Table 4.3 Earthquake design loads

Ratio percent	TEC 98	UBC 97	EC 8-98
Structure	17.4	41.0 (=***2.36*** x 17.4)	44.7 (=***2.57*** x 17.4)
Structural connection	20.9	59 (=***2.82*** x 20.9)	89 (=***4.26*** x 20.9)

Table 4.4 Comparison of displacement amounts

	Elastic disp. d_e	$s=d_e/h$	Upper bound formula s_{maks}		Evaluation	
TEC 98	2.20 cm	0.0024	0.0035,0.02/R=0.0040	(R=5)	$s<s_{maks}$	okey
UBC 97	4.74 cm	0.0053	0.02/(0.7R)=0.013	(R=2.2)	$s<s_{maks}$	okey
EC 8-98	5.50 cm	***0.0061***	(0.004?)/($k_d R_0$I)=***0.0053***	?=2, R_0=2	$s>s_{maks}$	unsuitable

The C-coefficient varies significantly among the codes. These variations are strongly interrelated with R. In the fault-proximity norms given in UBC-97, coefficient values demonstrate variations. One can see from Table 4.1 that TEC-98 possess a much lower lateral load in design calculations with respect to UBC-97 and EC8-98. With the multiplication of the C by the mass, the lateral earthquake load on the structure is calculated. In Table 4.4, the lateral drift generated by this load at the storey level and the ratio obtained from the division of this drift by the storey height for each code are illustrated. The values given under the 's' column stand for the numerical values of a ratio obtained by proportioning the elastic displacement to storey height for each case. In Table 4.2, loads acting on connections (like balconies, parapets, and corbels) are listed for the Codes assessed. In Tables 4.2 and 4.3, TEC-98 underestimates elastic displacement and load acting on nosing members.

5. REPAIR AND STRENGTHENING OF PREFABRICATED STRUCTURES

In reinforced prefabricated structural systems, the connections are generally the weakest part of the structure for seismic loading and the repair and strengthening details require particular attention to those connections. The measures for repair and strengthening of demages structures can be classified into two groups: -Measures for increasing the total stability of the building against seismic forces, -Measures for repair and increasing the strength of seperate structural elements. The characteristic damage are caused by the flexibility of the basic system. Structural damage usually consist of cracking and distress to the joints and possible failure of members which were not detailed for sufficient ductility. Infill elements generally sustain considerable damage due to their high relative stiffness compared to the structural frame and their brittle characteristics. Joints are repaired by removing loose concrete, injection of cracks with epoxy or grout and patching spalled areas with a suitable grout. Repair of members and joints may actually consist of strengthening those areas by jacketing or other techniques to provide confinement or a new method to resist the appropriate forces. Prefabricated industrial frame buildings generally have damage to bolted or hinged connections. Strengthening of these buildings is often difficult as shear walls can often be added only in the exterior frames of the building and the roof may contain skylights or other discontinuities. If interior shear walls can be added parallel to the frames and not disrupt the functional use of the building, such a solution is the preferred strengthening solution. When shear walls connot be added, then it is necessary to add exterior buttress walls or strengthen the existing column and foundation to provide sufficient strength, stability and ductility to cantilever from the ground and resist seismic forces. For strengthening in the longitudinal direction, it is possible to add new reinforced concrete shear walls or to all new rigid portal frames designed like a shear wall with an opening or adding a new diagonal bracing system of structural steel.

6. CONCLUSIONS AND ASSESSMENT

- Prefabricated structure systems should satisfy the terms of earthquake-resistant construction such as adequate strength, ductility and rigidity. Nevertheless, the recent earthquakes illustrated that even in the cases where one complies with all the criteria given in the code problems might occur.
- Due to the fact that there is intensive demand for the optimum use of space in industrial structures, no spatial partitioning is applied and the use of partition walls or curtain walls is comparatively limited owing to architectural concerns. Hence, it is the task of prefabricated columns to stabilise the

earthquake loads. The base shear force computed from the TEC is less than half of the values computed from the procedures given in the other codes.
- For 1 unit lateral load computed in accordance with the regulations given in TEC-98, UBC-97 and EC8-98 one would estimate 2.36 and 2.57 units, respectively. The structural behaviour coefficient R reaching a value of 5 in TEC-98 varies from 1.5–2.5 in UBC-97 and EC8-98.
- The formula proposed in TEC-98 for the design of connection locations for non-structural members is inadequate compared with the other codes, particularly the EC8-98 (Tezcan, 2003).
- Most intensive damages observed in the recent earthquakes on prefabricated industrial structures are those of the single-storey industrial structures with fixed connections at the ground level and hinges at the upper ends. The incompatibility of the structural systems relative to the selected system's inappropriateness with the limits put forward in the codes regarding lateral rigidity, strength and ductility, raises doubts about the earthquake resistance of these structures.
- Even though prefabricated systems possess the advantages, deficiencies in in-situ mounting and connection of members resulted in increased failures.
- Enquiries conducted on the damaged and collapsed buildings in the earthquake region illustrated that buildings fail especially in the direction orthogonal to the main axis. The reason lying beneath this outcome is the insufficiency of the column lengths of corbel type buildings in this direction.
- The plastic-hinge formation loci would be at the ground level of the column, just above the socket entry for the crown-hinged system. When the potential for column-lower-sections to form plastic hinges is considered, the necessity for increasing the frequency of the stirrups in the lower regions of the columns and the column-length of the stirrup densification emerges.
- It is observed that the capacity dictated from TEC-98 is lower than the capacity of the other two codes. The reason for this is related to the economic situation of the relevant countries.
- The studied structures which failed in the Marmara Earthquake were constructed according to designs imported from non-earthquake countries. Future designs must be performed considering the EQ reality of the region. Also new building and connection types should be developed considering the previous failure and damage pattern of older buildings.

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