SEISMIC PERFORMANCE OF PRECAST INDUSTRIAL BUILDINGS IN TURKEY

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ABSTRACT

Precast frame buildings are used throughout Turkey for industrial facilities. One-story warehouses are the most common structural configuration; however, low-rise commercial and manufacturing facilities are also constructed using precast concrete members. These structural systems are economical to construct and provide large open areas needed for manufacturing. Many precast industrial building collapsed during the recent earthquakes in Turkey. This paper summarizes relationships between the observed damage in one-story warehouse structures and the stiffness of the lateral-load resisting system.

Introduction

Approximately 90% of the warehouse and light industrial facilities constructed in Turkey during the 1990s used precast members (Karaesmen, 2001). The most common structural system for these facilities (Fig. 1) is based on a structural configuration that was developed in Western Europe to carry gravity loads only (Ersoy et al. 1999). Turkish engineers modified the connection details so that the precast buildings have the capacity to resist lateral loads. However, each producer of precast elements has developed a unique set of connection and reinforcement details, and the details vary appreciably from producer to producer.

Structural damage and collapse (Fig. 2) of precast buildings was widely reported throughout the epicentral regions of the 1999 earthquakes (Ataköy, 1999; EERI, 2000). The objective of this investigation was to document the observed damage and determine the likely causes. Researchers from the University of Texas, Kocaeli University, Boğaziçi University, Middle East Technical University, Purdue University, and the University of Minnesota visited more than 60 precast industrial buildings in the epicentral regions of the August 1999 Kocaeli and November 1999 Düzce earthquakes. Their observations, and the results of a parametric study to identify the causes of the observed structural damage, are summarized in this paper.

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Figure 1 Interior of an industrial building near Düzce

Figure 2 Collapsed industrial building near Adapazarı

Structural Characteristics of Precast Industrial Buildings

One-story industrial buildings represent the most common form of precast construction in the northwest and central Turkey and the overwhelming majority of the structures that sustained damage during the 1999 earthquakes. Two types of structural damage were frequently observed in the precast buildings: flexural hinges at the base of the columns (Fig. 3) and pounding of the precast elements at the roof level (Fig. 4). In addition, buildings under construction were susceptible to collapse when the roof girders rotated off their supports (foreground in Fig. 2).

Figure 3 Flexural hinge at the base of a precast column

Figure 4 Impact damage near the roof level

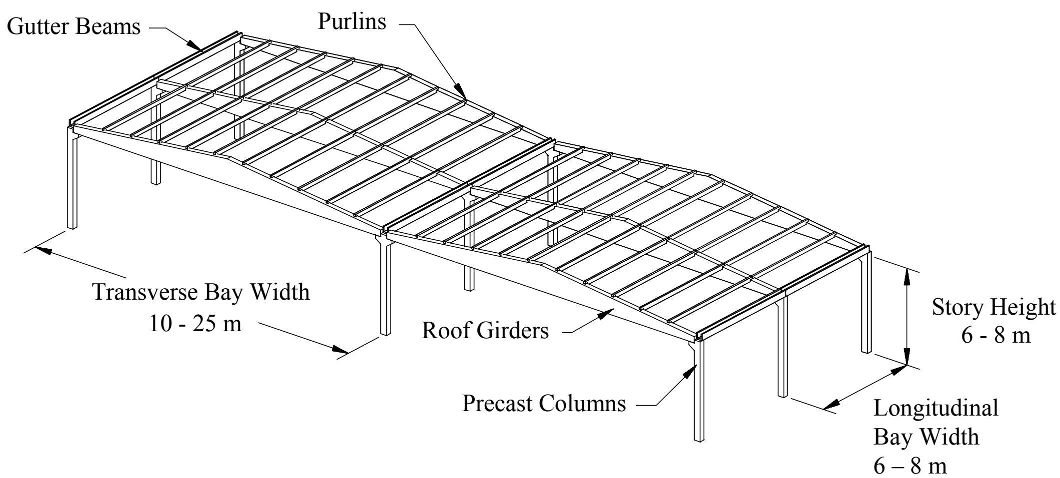

Figure 5 Typical configuration of one-story industrial buildings

The single-story industrial buildings were characterized by long-span roof girders which provided large open areas needed for manufacturing (Fig. 5). The buildings tended to be rectangular in plan with one to four bays in the transverse direction and ten to thirty bays in the longitudinal direction. Transverse bay widths ranged from 10 to 25 m, and longitudinal bay widths ranged from 6 to 8 m. Story heights also ranged from 6 and 8 m.

The base of each precast column was grouted in a socket footing (typically precast) to form a fixed connection. Long-span roof girders were oriented along the transverse axis of the building and were supported on column corbels. The depth of these girders often varied along their length, forming a triangular shape. Beams with U-shaped cross sections were oriented along the longitudinal axis of the building. These beams function as gutters to collect water from the roof. Purlins span between the roof girders at regular intervals. Typically five to eight purlins ran between adjacent roof girders.

The precast roof girders, gutter beams, and purlins were pinned at both ends. Vertical dowels extended up from the supporting member and the horizontal elements were cast with vertical holes near their ends to accommodate these dowels. The holes were filled with grout in most buildings. In some cases the dowels were threaded, and nuts were installed before grouting.

Typically, lightweight materials, such as metal decking or asbestos panels, were used to form the roof. Clay tile infill was used in most cases for the interior and exterior walls (Fig. 1), but precast concrete wall panels were also used in a few buildings. The typical, one-story industrial building depends entirely on the cantilevered columns for lateral strength and stiffness. Even when precast wall panels were used for cladding, the connection details were developed such that the wall panels did not contribute to the lateral stiffness of the building.

Influence of Column Stiffness on Structural Performance

A parametric study was conducted to relate the behavior of one-story precast industrial buildings to the column stiffness. An 80 by 200 m building in Adapazarı, which sustained light damage during the Kocaeli earthquake, was selected as the prototype structure for this study. The transverse bay widths were 20 m, the longitudinal bay widths were 7.5 m, and the story height was 7 m.

A linear model of the framing system in the transverse direction of the building was developed. The base of each column was fixed and the connections between the columns and roof girders were pinned such that vertical loads and shear were resisted, but the flexural resistance at the ends of the beams was negligible.

For the purpose of the parametric study, column dimensions were varied from 40 by 40 cm to 80 by 80 cm. It should be noted that the overwhelming majority of the columns in one-story industrial buildings were 50 by 40 cm or smaller. Column sizes up to 60 by 60 cm were common in multi-story precast buildings, and the largest precast columns observed were 80 by 75 cm at an automotive manufacturing plant that was under construction near Gölcük. The cross-sectional dimensions and mass of the roof girders, gutter beams, purlins, roofing materials, and cladding in the prototype building were used in all analyses. The variation of the calculated fundamental period with the assumed column dimensions is given in Table 1.

Table 1 Column Sizes Considered in Parametric Study

Column Dimensions		Calculated
Depth	Width	Period*
cm	cm	sec
40	40	1.56
40	45	1.48
45	40	1.32
45	45	1.24
50	40	1.13
50	45	1.07
50	50	1.03
50	55	0.99
55	50	0.91
55	55	0.86
60	55	0.76
60	60	0.74
65	65	0.64
70	70	0.57
80	80	0.44

* Calculated period corresponds to cracked cross-sectional properties.

Table 2: Ground Motions Considered in the Parametric Study

Station	Component	Peak Acceleration g	Epicentral Distance km	Soil Conditions
Düzce (DZC)	180 270	0.41 0.51	10**	Soft Soil
İzmit (IZT)	090 180	0.23 0.17	12*	Rock
Yarimca (YPT)	240 330	0.30 0.32	22*	Soft Soil
Sakarya (SKR)	090	0.41	35*	Stiff Soil
Bolu (BOL)	000 090	0.74 0.81	42**	Soft Soil
Gebze (GBZ)	000 270	0.27 0.14	50*	Stiff Soil
Arcelik (ARC)	000 090	0.21 0.13	60*	Stiff Soil
Düzce (DZC)	180 270	0.32 0.37	110*	Soft Soil

* Approximate distance to epicenter of Kocaeli earthquake.
** Approximate distance to epicenter of Düzce earthquake.

Ground Motion Records

Each of the buildings considered in the parametric study was analyzed using fifteen ground motion records (Table 2). Most of the recording stations were within 50 km of the epicenters of the 1999 earthquakes, and all were within 20 km of the surface trace of the faults (EERI, 2000). The ground motion records were divided into two groups depending on the soil conditions at the recording station. Records from Bolu, Düzce, and Yarimca were used to determine the spectral characteristics for soft soil, while records from Arcelik, Gebze, İzmit, and Sakarya were used to determine the spectral characteristics for stiff soil/rock sites.

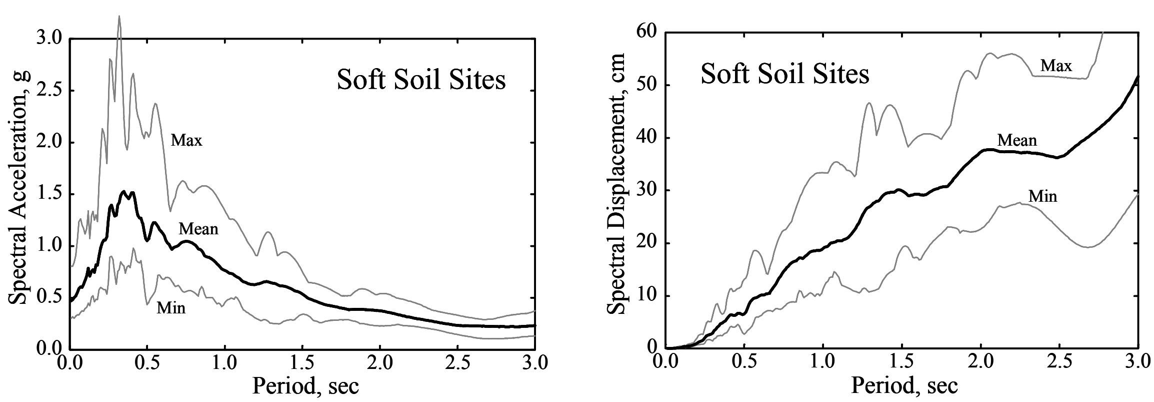

Figure 6 Mean, maximum, and minimum values of elastic response spectra for ground motions recorded on soft soils (damping factor = 0.02)

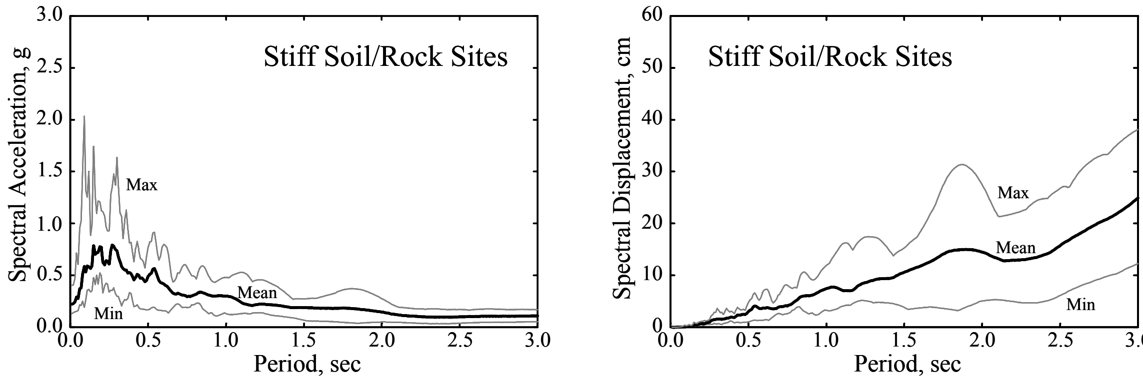

Figure 7 Mean, maximum, and minimum values of elastic response spectra for ground motions recorded on stiff soils (damping factor = 0.02)

Elastic acceleration and displacement response spectra corresponding to a damping ratio of 2% were calculated for each ground motion record. Mean, maximum, and minimum spectra are plotted in Fig. 6 and 7 for the soft soil and stiff soil/rock sites, respectively. Although statistical information is not shown in the plots, the maximum and minimum values were typically less than 1.2 standard deviations from the mean value for the range of periods considered.

In general, the minimum spectral displacements from the soft soil sites were approximately equal to the maximum spectral displacements from the stiff soil/rock sites. For periods less than 0.25 sec, the maximum spectral accelerations from the stiff soil/rock sites were nearly the same as the maximum spectral accelerations from the soft soil sites. However, for periods greater than 0.5 sec, the maximum spectral accelerations from the stiff soil/rock sites were approximately equal to the minimum spectral accelerations from the soft soil sites.

Displacement Capacity of Idealized Buildings

The displacement capacity of the idealized columns was determined by first calculating the moment-curvature response of each of the cross sections listed in Table 1. Reinforcement ratios of 1, 2, and 3% were used in the analysis, which are representative of the range of reinforcement ratios observed in the field. The assumed arrangement of the longitudinal reinforcement is shown in Fig. 8.

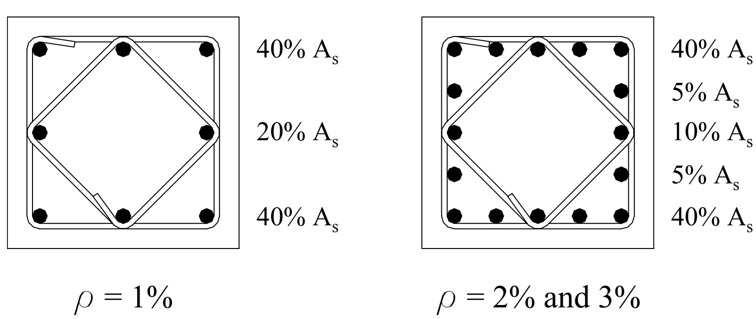

Figure 8 Assumed arrangement of reinforcing bars for parametric study

The behavior of the concrete was modeled using the stress-strain relationship developed by Hognestad (1951), a concrete compressive strength of 30 MPa, an initial modulus of elasticity of 26,000 MPa, and a limiting compressive strain of 0.0035. The stress-strain relationship for steel was assumed to be linear to the yield point. The yield plateau was assumed to extend to a strain of 0.01, and strain hardening was considered for strains between 0.01 and 0.10. The yield stress of the steel was assumed to be 420 MPa, the tensile strength was 500 MPa, and the modulus of elasticity was 204,000 MPa. The contribution of the transverse reinforcement in confining the column core was ignored because ties with 90-degree hooks were used throughout the epicentral region.

Because the cantilevered columns provided all the lateral stiffness for the one-story buildings, the yield displacement and displacement capacity could be calculated using Eq. 1 and 2.

$$\Delta_y = \frac{\phi_y \cdot L^2}{3} \tag{1}$$

$$\Delta_u = \Delta_y + \left(\phi_u - \phi_y\right) \cdot \ell_p \cdot \left(L - \frac{\ell_p}{2}\right) \tag{2}$$

where Δ_y is the yield displacement at the roof, ϕ_y is the calculated yield curvature for the cross section, Δ_u is the displacement capacity at the roof, ϕ_u is the calculated curvature capacity for the cross section, L is the height of the one-story building, and ℓ_p is the height of the equivalent plastic hinge. The plastic hinge length was assumed to be one-half the depth of the cross section for all cases (Moehle, 1992).

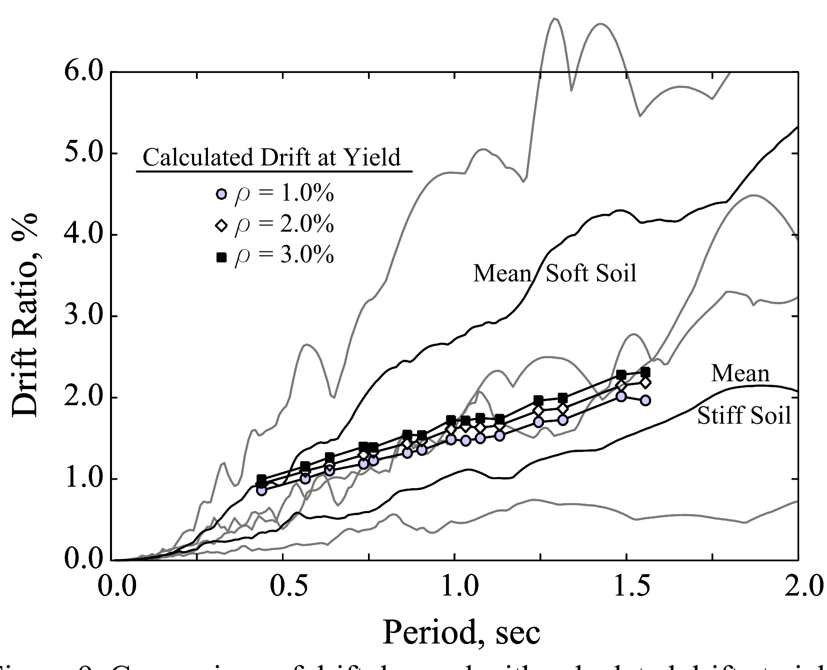

Figure 9 Comparison of drift demand with calculated drift at yield

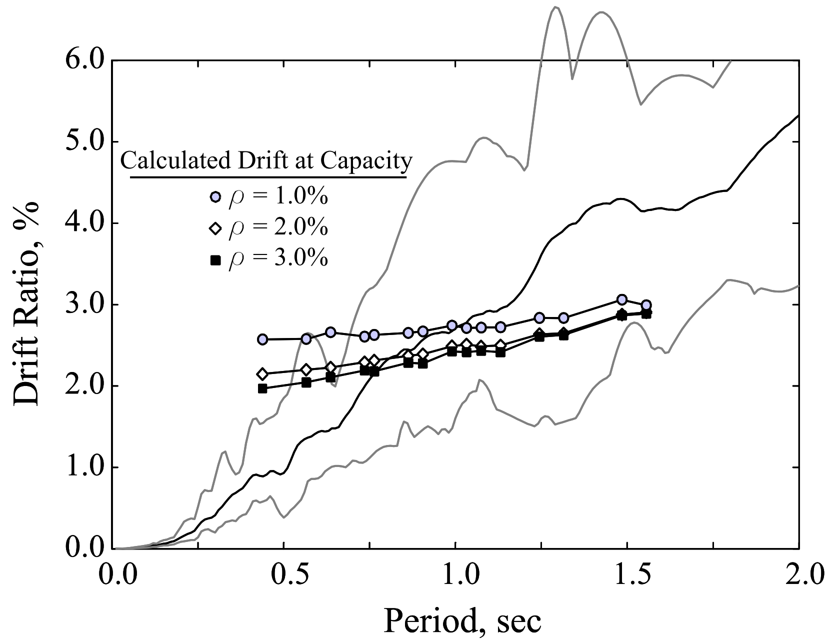

Figure 10 Comparison of drift demand at soft soil sites and calculated drift capacity

Comparison of Drift Demand and Capacity

The inelastic displacement demands in the one-story buildings during the 1999 earthquakes were approximated using the elastic displacement demands calculated using a damping factor of 2% (Shimazaki and Sözen, 1984). The elastic displacement demands from the fifteen ground motion records are compared with the calculated yield displacements in Fig. 9. Similarly to Fig. 6 and 7, mean, maximum, and minimum response spectra are plotted for the two groups of ground motions. All displacements are plotted in terms of the drift ratio: the roof displacement divided by the building height.

The data in Fig. 9 indicate that essentially all of the buildings considered in the parametric study had the ability to resist the displacements induced by the composite maximum response spectrum for the stiff soil/rock sites without yielding. However, the data in Fig. 9 also indicate that idealized buildings with calculated periods larger than 0.75 sec would yield when subjected to the composite minimum response spectrum for the soft soil sites. These results were not sensitive to the amount of longitudinal reinforcement in the columns.

The calculated drift capacities of the idealized buildings are compared with the response spectra for soft soil sites in Fig. 10. Idealized buildings with periods greater than 0.8 sec are likely to be pushed beyond their displacement capacity by the mean ground motion at the soft soil sites. Only buildings with periods less than 0.5 sec are likely to survive the maximum composite response spectrum without reaching their displacement capacity.

As indicated in Table 1, a period of 0.8 sec corresponds to a building with columns larger than 55 by 45 cm. As noted previously, the overwhelming majority of one-story industrial buildings visited in the epicentral region were constructed with columns this size or smaller. A period of 0.5 sec corresponds to a building with columns larger than 70 by 70 cm. Less than 5% of the single-story buildings visited had columns larger than this size.

Potential Impact Damage

The preceding discussion focused on the overall drift response of one-story industrial buildings and the observed damage at the base of the columns (Fig. 3). The impact damage near the roof level (Fig. 4) may be evaluated by considering the geometry of the structural members (Fig. 11). Because the inelastic action in the columns was concentrated at the base, the top portion of the columns, the roof beams, and the gutter beams may be considered as rigid bodies. The top of the column rotates due to the hinge at the base, which causes the top corner of the roof girder to hit the gutter beam (Fig. 11) or the bottom of the roof girder to hit the top of the corbel.

The column rotation necessary to cause impact of the precast members was compared with the rotations at the top of the column corresponding to yielding of the longitudinal reinforcement and the flexural capacity of the column cross section. For the 40 by 40-cm and 45 by 40-cm cross sections, the precast members were expected to hit before the longitudinal reinforcement yielded, and for cross sections up to 55 by 55 cm, the precast members were expected to hit after the longitudinal reinforcement yielded, but before the column reached its flexural capacity. Columns larger than 55 by 60 cm and larger were expected to reach their flexural capacity before the roof members touched.

These calculations are consistent with observations in the field, because damage at the base of the columns was observed in nearly all the structures that experienced impact damage at the roof level. Again, the data indicate that larger columns are required to control damage in this type of structural system.

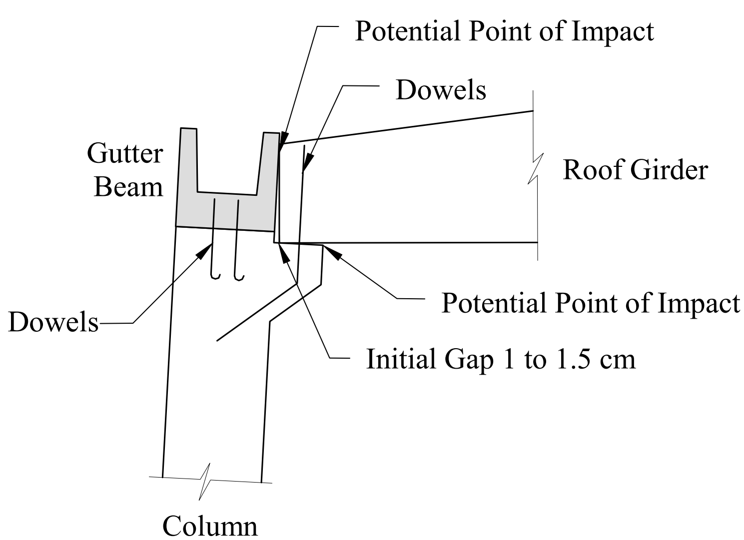

Figure 11 Typical connection details at top of column

Conclusions

This paper has evaluated the flexural response of precast industrial buildings in Turkey. Column dimensions are considered to have a critical influence on the performance of this type of structure. The analyses indicated that buildings that experienced satisfactory performance when founded on stiff soil were likely to collapse when founded on soft soil deposits. Only by increasing the size of the columns can the probability of damage be reduced in future earthquakes.

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