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Performance of precast concrete structures in October 2011 Van earthquake, Turkey

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A destructive earthquake, magnitude of $M_w = 7{\cdot}2$ (Richter scale), hit the city of Van, located in eastern Turkey, on 23 October 2011 and another major earthquake with a magnitude of $M_w = 5{\cdot}6$ occurred on 9 November 2011. Significant damage was observed in all types of civil engineering structures in the city centre and nearby. This paper presents the field observations on the seismic performance of precast concrete structures during the earthquakes. Possible damaging factors were discussed in detail after a comprehensive site survey. The majority of the investigated structures were industrial precast concrete structures located in the organised industrial zone of Van. In addition to industrial precast concrete structures, a precast multi-storey residential building located in the city centre was also examined. The findings from the site investigations were compared with the seismic behaviour of similar precast concrete structures during the former devastating earthquake in north-western Turkey in 1999. The effects of improper design and detailing of precast connections during the construction of the precast concrete structures in the high-seismicity regions are reported.

Introduction

Two devastating earthquakes having magnitudes of $M_W = 7{\cdot}2$ and $M_W = 5{\cdot}6$ struck the eastern part of Turkey on 23 October and 19 November 2011. The earthquakes caused almost 650 casualties and more than 4000 people were wounded according to the Van governorship's report (see http://www.van.gov.tr/default_B0.aspx?id=1809). The major structural damage and the life loss were mainly localised around the city of Van and the town of Ercis (Figure 1). The region is classified as seismic zones I and II according to the Turkish seismic zoning map, where the lateral peak ground accelerations are $0{\cdot}4\boldsymbol{g}$ and $0{\cdot}3\boldsymbol{g}$ respectively with a 10% probability of exceedence in 50 years. The epicentre of the October 2011 earthquake was 30 km north of the city of Van (near Tabanli village). The peak ground acceleration values of the first shock were reported as 178·5 gal (1·785 m/s^2) in the north–south direction, 168·5 gal in the east–west direction and 75·5 gal in the vertical direction (reported by the Earthquake Department of the Disaster Emergency Management Presidency (AFAD, 2011)). The epicentre of the second shock was in Lake Van close to the coast of town Edremit. The measured peak ground accelerations for north–south, east–west and vertical directions were 148·1 gal, 245·9 gal and 150·54 gal respectively. The comparison of response spectrums of north–south and east–west components of the first shock with the 5% damped elastic design spectrum according to the Turkish Earthquake Code (TEC) 2007 (TEC, 2007) is given in Figure 2. The elastic design spectrum of the TEC was not exceeded by the response spectra of the seismic action. TEC defined the 5% damped elastic design spectrum in a similar fashion to Eurocode 8 (BSI, 2005) by considering the effects of soil condition, national seismic zone map and building importance factors. However, the effect of the vertical component of the ground motion is neglected in design of structures, contrary to Eurocode 8 (BSI, 2005).

Turkey is partly located on the Anatolian Peninsula that is surrounded by the world's major tectonic plates, such as the Arabian, Eurasian and African plates. Relative displacements of those mega plates lead to frequent and hazardous seismic activities in Turkey. Destructive earthquakes mainly occur on two main faults, namely the north Anatolian fault and the east Anatolian fault. The city of Van is located at the junction of those two faults. The surrounding area of the city of Van is composed of lake, river and land sediments and has layers of loose sand, gravel and clay. The groundwater table is high, especially for the areas close to Lake Van. There are well-known volcanoes such as Nemrut, Suphan and Tendurek in the hinterland of Van (Kocaeli University Reconnaissance Report, Ozden *et al.*, 2011), and several major earthquakes having magnitudes of $M_W = 5$ and higher, up to $M_W = 7{\cdot}2$, have been reported in the region.

Precast concrete structures, as well as other types of structures, built in the region experienced light to severe damage during the

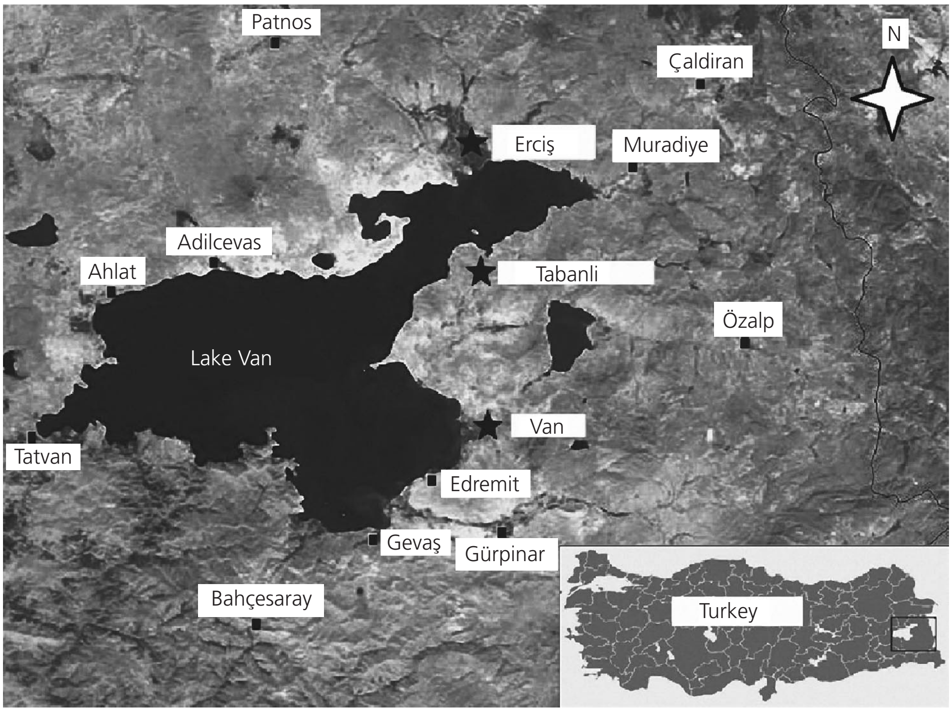

Figure 1. The city of Van and the earthquake region

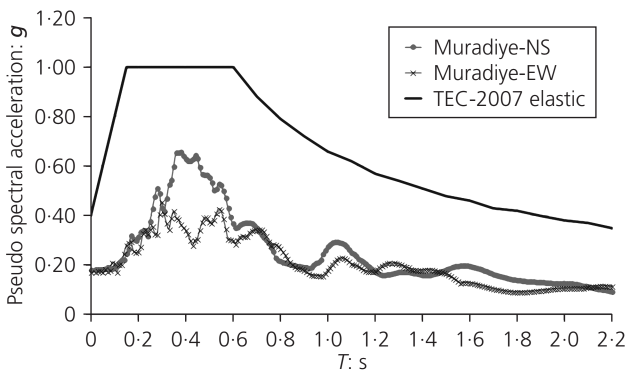

Figure 2. Comparison of response acceleration of N–S, E–W components of 23 October 2011 Van earthquake with the elastic design spectra of TEC

2011 Van earthquake. The susceptibility of such structures to seismic actions is closely related to the type of connection they use. The use of non-moment-resisting connections, in most cases, leads to residual damage in the joint or total collapse in the structure.

This paper mainly describes the behaviour and performance levels of the precast concrete structures after 23 October and 19 November 2011 earthquakes around the city of Van. Most of the precast concrete structures, especially the industrial buildings, were damaged during the previous devastating earthquake in 1999 in Turkey owing to factors such as lack of lateral stiffness and shear transfer deficiency in connection. There are several studies in the literature that evaluate the performance of precast structures during the Kocaeli earthquake in 1999 (Ataköy, 1999; Bruneau, 2002; Ozden and Meydanlı, 2003; Saatcioglu *et al.*, 2001; Sezen *et al.*, 2000; Sezen and Whittaker, 2006; Wood, 2003). Since the affected region in the 2011 Van earthquake was newly developed, almost all of the investigated precast concrete structures were built after 1999. Accordingly, this situation provides the opportunity to examine the lessons learned from the 1999 Kocaeli earthquake.

Performance of precast concrete structures

The TEC has three recent versions established in the years of 1975, 1998 and 2007. Critical revisions were applied for each version considering the shortcomings of the previous version according to experienced earthquakes as well as conducted researches. Furthermore, Eurocode 8 (BSI, 2005) provisions were also taken into account for the modifications in establishing the 1998 and 2007 versions of TEC. Thus, the current version of the TEC represents conceptual accordance with Eurocode 8 (BSI, 2005), especially in terms of modelling, selection of analysis method, sensibility to plan and the vertical irregularities of the structures for design purposes.

Twelve precast concrete structures were visited and investigated in detail by the reconnaissance team. The investigated precast concrete structures in the city of Van have different types of connections, either with moment-resisting post-tensioned, welded, cast-in-place or with non-moment-resisting details.

The precast concrete structure with post-tensioned connections was a moment-resisting residential multi-storey building. Another was a two-storey sport hall, having non-moment-resisting connections with cast-in-situ reinforced concrete shear walls, at the city centre. The rest of the investigated structures were precast concrete industrial facilities, with columns fixed at the base and pinned at the top, located in the city industrial zone (CIZ). It was declared by industrial zone authorities that three of the 66 industrial buildings suffered heavy damage, as testified by the reconnaissance team, and those damaged structures having non-moment-resisting connections, were in the process of construction at the time of the earthquake. All the investigated buildings were designed and constructed under the guidance of TEC (2007). The Turkish Precast Concrete Association (TPCA) declared that only six precast concrete buildings in the region were designed and constructed under the guidance and consultancy of their members and those buildings were reported to be undamaged.

Multi-storey precast concrete building with moment-resisting post-tensioned connections

The residential building with moment-resisting post-tensioned connections has seven storeys with an approximate foot-print area of 380 m^2. The plan geometry was rectangular: two bays by two bays. The bays were $L_x = 12{\cdot}30$ m and $L_y = 8{\cdot}00$ m in two orthogonal directions. The flooring system in the structure was hollow core slab. Hollow core slab segments were supported by the 12·30 m span, spanning in the short direction. The beam dimensions spanning in the short direction were 50×70 cm, whereas those spanning in the long direction were 60×80 cm. All the columns had an identical cross-section of 75×70 cm. The columns of the building were each constructed in a single seven-storey long (approximately 22 m) piece and transported to the construction site. Cast-in-place socket type foundations were used and designed as fixed supports. Beams with tapered ends were seated on square corbels of the columns and post-tensioning was used for the connection continuity. Figure 3(a) displays a view of the construction stage before the earthquake.

Post-earthquake damage investigation of that specific building revealed no structural damage of either the precast members or the post-tensioned connections (Figure 3(c)). The column-to-foundation connections were carefully investigated and no flexural or shear cracks were observed. No visible damage was detected on the structural frame components of the building after the Van earthquake of October 2011. On the other hand, 45° diagonal tension cracks were observed on the dry partition walls of the building (Figure 3(d)). This type of cosmetic damage reveals that there was incompatibility of deformation capacities between the non-structural walls and the frame. The frame remained relatively elastic, whereas the walls exceeded their failure displacement limit. The performance level of the structure was immediate occupancy, and the same performance level can be assigned to the exterior and partition walls by reducing the ultimate design inter-storey drift level.

Precast concrete sport hall with reinforced concrete shear walls

The one-bay precast concrete sport hall has two storeys with a plan dimension of 19×51 m, as shown in Figure 4(a). Each bay width in the longitudinal direction was measured as 5·1 m. The structure was constructed on a mat foundation and built with cast-in-situ shear walls in the first storey on both orthogonal directions (Figure 4(a)). The ratio of shear wall areas to the total plan area (shear wall index) in the longitudinal direction of the building was equal to 1·2% and this value was equal to 0·6% in the transverse direction. All of the beam to column connections in the structure consisted of two-pinned shear connections. Although reinforced concrete structures in the vicinity of the structure had significant damage, this sports hall was almost completely free of damage (Figures 4(b) and 4(c)). It is believed that such a successful earthquake performance may be attributed to the high cast-in-place shear wall index (greater than 0·5%) and to the existence of a rigid diaphragm at the first floor level.

Multi-span precast concrete industrial facilities

The CIZ of Van is located between the city of Van and the epicentre (Tabanli village), by the Lake Van, and near the Van university campus. The geotechnical report for this area, which was prepared by the authorities of CIZ, mostly concludes that sandy and clayey soil layers exist up to a depth of 20 m. Almost all of the precast concrete industrial facilities in the CIZ were single-storey, multi-span buildings, with pin connections (non-moment resisting). Socket type foundations were used for the column bases, whereas the cantilever columns were free to rotate at their top levels. The precast pre-stressed triangular roof beams were seated on the corbels of the columns and connection was solely through the rebars, mostly four protruding from the corbels and passing through the holes at the ends of the roof beams (during the field investigations after the 1999 earthquake, it was observed that the number of rebars was equal to one or two in the cities of Kocaeli and Sakarya). The re-bars were embedded in the holes of the beams by grout injection.

The level of damage was low for the facilities that were in use during the earthquake. The existence of facade walls and roof cover plays an important role in reducing the damage level by maintaining extra lateral stiffness. Three of the precast structures graded as immediate occupancy are represented in Figure 5. Moreover, two facilities were in full production just a couple of days after the first earthquake. In some facilities, local settlement and heaving of the floor cover was observed on the opposite faces of the precast columns. In addition, owing to the deformation incompatibility, almost all of the precast concrete structures had cracks between structural and non-structural elements such as walls (Figure 5(a)). There was cracking and crushing at the corners of the structural elements in the joint regions because no

(a) (b) (c) (d)

Figure 3. Multi-storey precast concrete building with post-tensioned connections

elastomeric bearings were used in the beam to column connections at the connection interface (Figure 5(c)).

As the roof beams were seated on corbels and purlins were simply supported on beams, no diaphragm effect was provided at the roof level through structural members for the structures under construction. Accordingly, the damage or partial collapses were mostly observed in the structures in which the peripheral walls and roof covers were not yet installed, in other words for the structures under construction. Similar failure types were also reported after the 1999 Kocaeli earthquake (Wood, 2003). It is believed that the existence of peripheral walls reduces the inter-storey drift, whereas the metallic roof cover results in a sort of diaphragm action. Three industrial buildings that were under construction and were heavily damaged after the 2011 Van earthquake are shown in Figure 6. The failure of these industrial buildings was mainly attributable to the falling of the roof beams. Although the falling of the roof beams might easily be attributed to the lack of grouting (as the building was under construction the producer claims that the earthquake happened just before grouting) at the pin supports at the connections, there were failures observed of connections where the grouting had recently been completed. It was also detected that the grouting mixture was not prepared appropriately (Figure 7(a)). Although some of

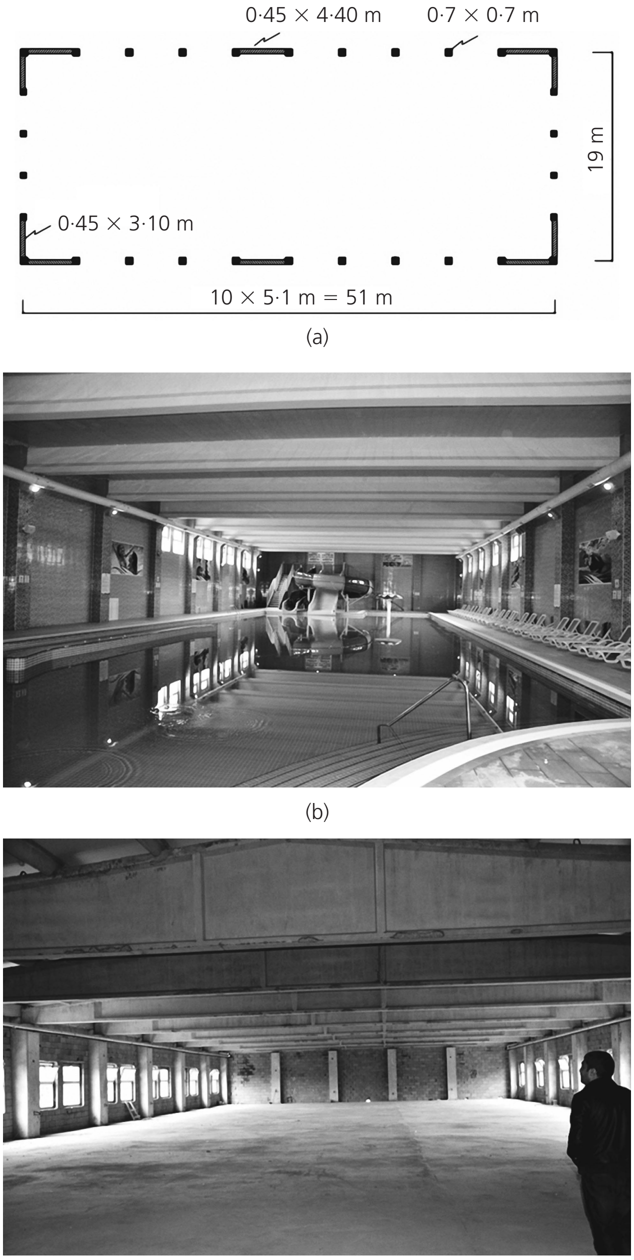

Figure 4. Precast concrete sport complex in the city centre of Van (courtesy of Serif Guner)

the re-bars protruding from the corbels have threaded ends connecting the column and the roof beam, no nuts and washers were used during the assembly process (Figure 7(b)). In the spans where the roof beam to column connections worked well on one end and failed on the other, the columns not only underwent inertia forces but also the gravity forces and extra moment created by the falling of the roof beams, resulting in flexural hinging at the column bases (Figure 7(c)). This situation might have been caused by the possibility that some of the holes were grouted and some of them were not (while one end of a beam could have easily demounted, the other end was still connected to the column tip causing extra force and moment on the column base and extra displacement demand). The longitudinal reinforcement in such columns experienced yielding, even strain hardening and rupture, resulting in total collapse of the columns (Figure 7(d)). Surprisingly, it was observed that the column longitudinal reinforcement ratios for the collapsed precast frames ranged between 0·60% and 0·75%, which is very low compared to design calculations. The plastic hinge length for the columns with such low reinforcement ratios was approximately 1 to 1·5 times the column cross-section. TPCA also conducted detailed investigations on the quality of reinforcing steel. For this purpose, samples were taken from the field and mechanical tests were performed in an accredited laboratory for the determination of reinforcement quality with respect to allowable limits proposed by the TEC (2007) and Eurocode 2 (BSI, 2004) provisions. Table 1 denotes the test results of the samples (supplied by TPCA). It is apparent from the results that the re-bar diameter, yield and rupture strength values are not satisfying the limits defined by the TEC (2007) and Eurocode 2 (BSI, 2004). Especially for the first and second samples the yield and rupture strengths are almost equivalent and the elongation at rupture values are below the limits leading to brittle behaviour. The bending angles at the ends of the cross-ties (not the peripheral hoops) were allowed to be 90° on one end and 135° on the other, provided that the consecutive two cross-ties did not have the same angle on the same sides according to the TEC (2007). A similar recommendation is also outlined in Eurocode 8 (BSI, 2005). This alternative use of cross-tie ends was not observed in the failed column cross-sections. Compressive crushing and tensile cracking took place at nearly all the beam column connections, no matter whether the frame was failed or not. It was apparent that no elastomeric pads were used in either failed or non-failed pre-cast concrete structures. In some collapsed frames, it was observed that the radial or confining reinforcement around re-bar holes for the beam to column connection was missing (Figure 7(e)). In some structures, the number of holes was inconsistent (Figure 7(f)). Such observations are good examples for the low construction quality and assembly practice in the local area. The similar deficiencies observed in 1998 Adana-Ceyhan earthquake and 1999 Kocaeli earthquake, caused the failure of the precast industrial frames in the October 2011 Van earthquake.

Simple performance evaluation of a damaged one-storey industrial building

One of the partially collapsed precast concrete structures in the Van CIZ area is considered for performance evaluation (Figure 6(b), Figure 8(a)). The building was in the construction phase and no proper beam-to-column connection was supplied; it was clearly stated that structural integrity and planned framing action were not accomplished. Therefore, individual column performance was investigated by applying non-linear static analysis and response spectrum analysis. Cross-sectional dimensions of the column, the corresponding physical achievements of which after earthquake are shown in Figures 7(b)–7(d), are represented in

(a)

(b)

(c)

Figure 5. Examples of precast concrete facilities having light damage in the city industrial zone (CIZ) of Van city

Figure 8(b). Although the planned column dimensions were $0{\cdot}7 \times 0{\cdot}7$ m in the original blueprints of the structure, the in situ column dimensions were detected as $0{\cdot}6 \times 0{\cdot}6$ m. It was also observed that the reinforcement ratio in the in situ column was about 0·95%, instead of 1·35% as stated in the blueprints. In light of this information and the test results given in Table 1, the yield strength and rupture strength of the reinforcing steel were assumed as 690 MPa and 730 MPa respectively. Elongation at

(a)

(b)

(c)

Figure 6. Heavily damaged precast concrete industrial facilities in the city industrial zone (CIZ)

Specimen	Diameter measured: mm	Yield strength: MPa	Rupture strength: MPa	Elongation: %
1 (∅22)	21·50	691	691	9·8
2 (∅22)	21·08	730	730	8·2
3 (∅18)	17·86	570	657	27·2
4 (∅18)	17·63	626	703	N/A

Table 1. Mechanical test results of reinforcement samples

rupture of the steel was considered as 10%. The longitudinal reinforcing steel ratio in the cross-section was assumed to be 0·95% and the transverse reinforcement having diameter of 8 mm and spacing of 100 mm was considered in the cross-sectional analysis (Figure 8(b)). As the concrete strength was not clear for the structure, two different concrete cylinder strength values, 20 and 25 MPa, were employed for the cross-sectional analysis considering the poor workmanship effect observed in the site. The moment–curvature relationship of a single column obtained by using the aforementioned structural parameters is presented in Figure 8(c). It is clear from the graph that variation of the concrete strength, 20 to 25 MPa, did not cause a significant difference in the moment–curvature relationship. The yield moments of the column for 20 and 25 MPa concrete strength were 477·6 and 496 kNm respectively.

As the structure was under construction during the earthquake, the only axial load carried by the columns was the weight of structural members such as roof beams, U-through and the crane beams. The weight of the roof beam was taken as 10 kN/m and it was considered that only half of the beam acted vertically on the column for load–deflection analysis. Extra weights coming from the other beams were ignored and it was assumed that they had totally fallen at the very beginning steps of shaking. The calculated axial load on the column was about 3% of its total axial load capacity. Hence, only a moment-dependent hinging mechanism was considered. The moment–curvature relationships were used for non-linear static analysis of the column. The plastic hinge length was taken as the width of column that is consistent with the site observations (Figures 7(c) and 7(d)). The lateral load that results in yielding of the column for 20 and 25 MPa concrete strength was computed as 65·22 and 67·73 kN respectively, as given in Figure 8(d). Moreover, the lateral load capacity of the column was 73·26 and 75·37 kN respectively. Response spectrum analysis was also performed for a single column by using the earthquake data measured at Muradiye station (Figure 2) and the consequent moment at the base of the column and shear load were computed as 463·3 kNm and 61·8 kN, respectively. It was believed that one end of the roof beam had fallen easily where it was seated on the column corbel at the middle axis, whereas the other end had not demounted. Therefore, it was considered that the falling beam at one side had also pulled the tip of the column at the other side until the total failure of the connection. It should

(a) (b) (c) (d) (e) (f)

Figure 7. Close-up views of damaged members

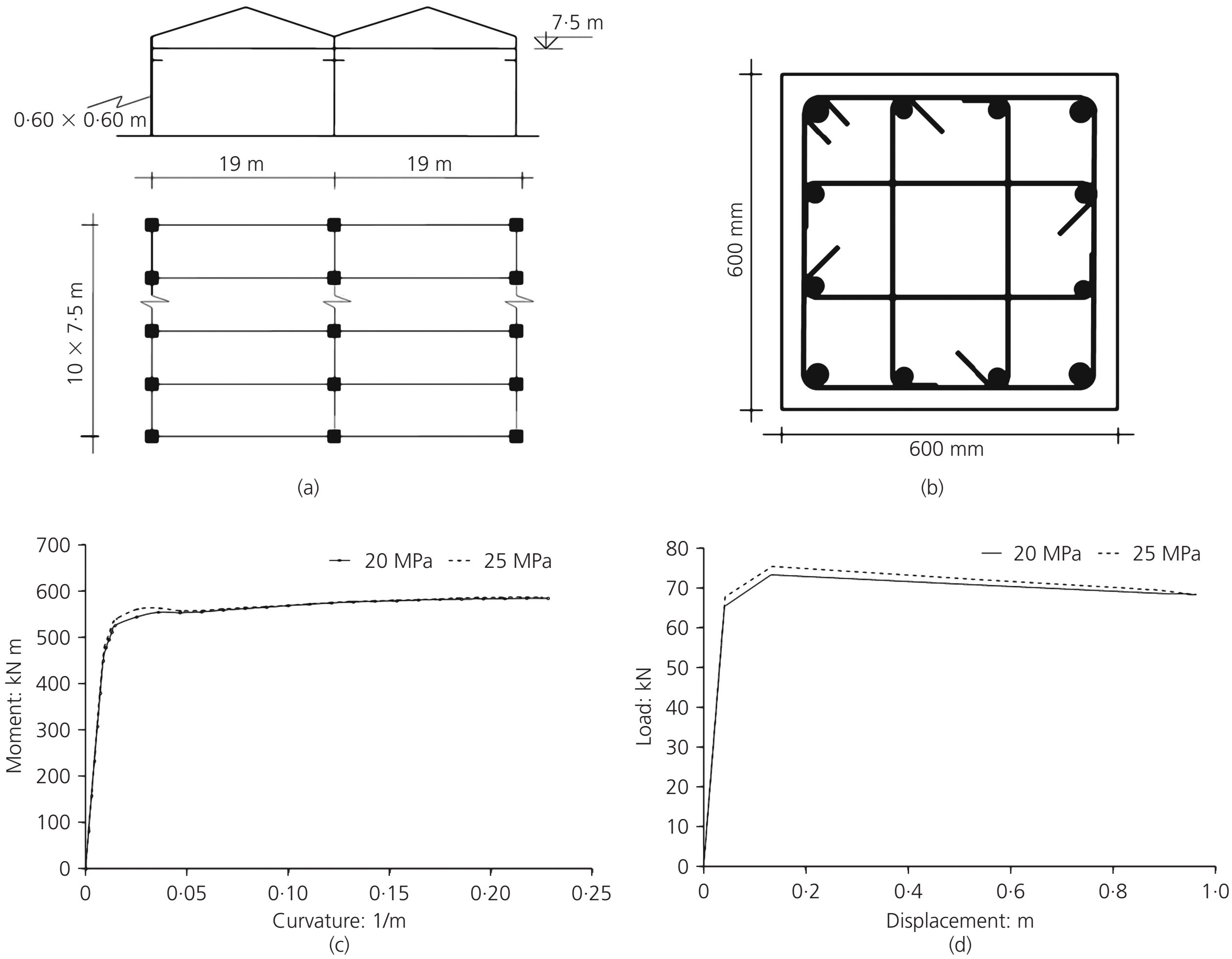

Figure 8. (a) Plan and elevation of partially collapsed structure; (b) cross-section of the column; (c) moment–curvature graph; (d) load–deformation graph of column

be noted that such a pulling effect was not accounted for during the design of this type of building. Obviously, the columns forced to almost their capacity with earthquake excitement collapsed with this diagonal pulling action. This also explains the survival of the columns that had not been subjected to the pulling effect of falling beams.

Summary and conclusions

The post-earthquake field investigations in the region revealed that the performance of precast concrete structures is closely related to their conformity with the current earthquake codes. The performance level for the structures with moment-resisting connections was immediate occupancy for multi-storey buildings. On the other hand, the performance of one-storey industrial buildings was acceptable in the case in which partition and facade walls were constructed and in the case where the metallic roof cover was mounted. However, the precast frames without walls and roof covers experienced various damage levels. The authors believe that the responses of industrial structures are mostly influenced by the lateral drift levels. The reduced drift levels protect the connection from failure and hence attain structural integrity throughout the seismic excitation. In addition, field investigations also revealed that the need for elastomeric bearing pads in the connection region between beam end and corbel is crucial in preventing the formation of cracks and concrete corner crushes at this region. Furthermore, collapsed structures that were under construction during the earthquake indicated that grout injection and securing the re-bars passing through the holes placed at the ends of the beams should be applied in all connections immediately after placing the beams on top of erected columns for providing the complete framing action. Simple and free seating of the beam on the corbels, just relying on the re-bars passing through the holes without grouting and securing processes, should be avoided.

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