Experimental Investigation of the Seismic Behaviour of Prefabricated RC Structures

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EXTENDED ABSTRACT

Prefabricated RC structures often suffer severe damage during strong earthquakes, especially to beam-to-column connections. Several systems are applied world-wide, the most common being skeletal frames. In such systems, of primary concern are the beam-to-column connections, which are usually formed by dowels that fasten the beams to the columns and in situ cast concrete that fills any gaps. In most cases, however, the in situ cast concrete is of small cross section and lightly reinforced and it does not contribute significantly to the resistance of the joint.

In this paper, the preliminary experimental results on the seismic response of single-storey, precast RC frame structures are presented. The tests were performed on the shaking table facility of the Laboratory for Earthquake Engineering at the National Technical University of Athens, Greece, in the framework of the project "Precast EC8: Seismic behaviour of precast concrete structures with respect to Eurocode 8 (Co-Normative Research)" under the programme "Competitive and Sustainable Growth" of the European Commission. The objectives of the tests were to:

- Investigate the response of the beam-to-column connections and detect possible drawbacks of the specific systems applied to the specimens.
- Investigate the integrity and the overall response of precast structures to strong earthquake motions which bring them close to collapse.
- Provide experimental data for the validation and calibration of analytical methods of design.

Five series of experiments were performed. In each series, the seismic response of a different specimen was examined. Each test structure was subjected to a number of biaxial base excitations, which were based on a real earthquake record that was scaled to several values of the peak acceleration.

All specimens were made by four precast columns and precast beams. The slab was formed by precast girders and in situ cast concrete, which covered the entire area or part of it, depending on the system. The differences among the five test structures concerned the dimensions of the columns and the beams, the height of the models, the construction of the top slab and the column-to-beam connections. In one case, a precast panel wall was mounted on one side of the structure.

Two different construction systems were examined. Their difference concerned mainly the beam-to-column connections, which were weak for the one (1Ø20 dowel at each beam end) and strong (2Ø32 dowels at each beam end) for the other. A less important difference concerned the construction of the top plate, which was more flexible at the second system.

During the experiments, the deformation of the structure was measured by accelerometers and displacement transducers. The accelerations were measured on the roof (two in each direction), at the top of the columns (two in each direction) and on the shaking table (4 horizontal and 4 vertical). The displacement transducers were measuring the roof displacement (two in each direction), the column-to-beam relative sliding and rotation and the column uplift. In order to measure the joint rotations, diagonal instruments were placed at the joints.

The experimental results showed that the largest accelerations recorded were almost 2.5 times stronger than the base ones. In the case of the specimen with the wall, torsional vibrations were induced to the structure, which caused the acceleration at the side opposite to the wall to be of the same order of magnitude with the one of the similar specimen without wall, although the overall response in the direction of the wall was reduced.

The measured displacements showed that the weaker joint connections resulted in residual opening of the joints and permanent dislocations of the beams. In the case of the strong connections, such displacements were observed only for very strong base excitations.
The time histories of the joint rotations showed the significant permanent distortion of the joints in the case of the weak connections. This was caused by the inclination of the columns, due to the severe bending and distortion of the joint dowels, which forced the joints to open. In the case of the strong connections, no residual distortion occurred, although the amplitude of the joint rotations during the seismic motion was larger.
Concerning the damage, the weak connections performed poorly for strong seismic motions. The damage observed included:

- Opening of the joints due to permanent dislocations and torsional rotations of the beams. Such residual displacements show that the dowels bent during the seismic response;
- Damage to the beam edges due to the impact that occurred between beams and columns. This damage was caused mainly by the joint rotations, which also caused a smaller damage to the column edge. It should be noted that the beams seated directly on the columns; if some kind of bearings was used, this damage would be reduced;
- Damage to the concrete surrounding the dowels. This type of damage shows that high stresses were developed around the dowels. For the strong connections, such damage was not observed;
- Damage to the in situ cast concrete at the connections. The experiments showed that the in situ cast connections did not have enough strength to resist strong seismic excitation and cracked severely;
- Inclination of the columns, as a result of the opening of the joints.

In the case of the specimens with strong connections, the observed damage was much smaller. However, dislocation of the beams occurred in this case too, but for very strong base excitations, around 0.8g. In general, these structures could sustain very large displacements without significant structural damage to the joints. For the specific specimens of the tests, the column-to-footing connections suffered considerable damage due to the relatively wide gap between the columns and the footing sockets. Because of this damage, the columns behaved as partially fixed at their base and, thus, large displacements developed at the roof level, which, however, did not cause severe damage to the beam-to-column joints.

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INTRODUCTION

The use of precast technology in building construction is spreading every day due to the considerably smaller time required for the erection and the smaller labour cost compared to in situ cast concrete. Several systems are applied worldwide, the most common being skeletal frames. In such systems, of primary concern are the beam-to-column connections and several techniques have been proposed on this topic, for example pinned, semi-rigid and monolithic connections, application of prestress etc.

Although the seismic behaviour of precast structures is of crucial importance in regions of high seismicity, little research has been presented on this subject. Elliott [1] presents the recent developments on this topic. The most notable effort has been the PRESS (Precast Seismic Structural Systems) project [2-5] carried out in U.S.A. and Japan during a 10-year period. In the framework of this project, a 60% scale, 30ft×30ft, five-storey building with several types of connections was tested at the University of California, San Diego.

The important role of the stiffness of the connections has been investigated by Imai et al [5] and Watanabe [6]. Semi-rigid connections were examined extensively within the European project COST C1 Action [7], in which 25 countries and more than 125 research centres were involved. A wide range of materials and geometries were studied; the results, however, showed significant diversion. A number of researchers [8-13] have also examined this subject.

In this paper, the preliminary experimental results on the seismic response of single-storey precast RC frame structures are presented. The research was conducted in the framework of the project "Precast EC8: Seismic behaviour of precast concrete structures with respect to Eurocode 8 (Co-Normative Research)" under the programme "Competitive and Sustainable Growth" of the European Commission. Research centres from Italy, Greece, Portugal, Slovenia and China participate in the project, which is coordinated by Professor G. Toniolo of the Polytechnic of Milan, Italy. The experiments were performed using the shaking table facility of the Laboratory for Earthquake Engineering at the National Technical University of Athens, Greece, in the period from October 2004 to August 2005. The objectives of the tests were to:

- Investigate the response of the beam-to-column connections and detect possible drawbacks of the specific systems applied to the specimens.
- Investigate the integrity and the overall response of precast structures to strong earthquake motions which bring them close to collapse.
- Provide experimental data for the validation and calibration of analytical methods of design.

Keywords: precast, concrete, seismic response, shaking table

EXPERIMENTAL PROCEDURE

Since one of the objectives of the experimental investigation was to examine the performance of the beam-to-column connections, it was decided not to use scaled-down specimens in which the relation between the shear force, friction and resistance of the steel dowels could be altered compared to the prototype. The limitations, however, posed by the shaking table dimensions (4.00m×4.00m) and its force/moment capacity did not allow the use of large specimens. To overcome this problem, it was decided to use single-storey test structures with reduced overall plan dimensions, but with columns and beams with cross sections in physical

scale. Since the test structures were significantly smaller in plan than the usual structures encountered in practice, additional mass was placed on the top, in order to simulate the inertia forces that would develop to a structure with larger spans, which would correspond to a medium-size prototype, almost 50% larger in plan than the specimen.
Five series of experiments were performed. In each series, the seismic response of a different specimen was examined. Each test structure was subjected to a number of base excitations, scaled to several values of the peak acceleration. All specimens were made by four precast columns and precast beams. The slab was formed by precast girders and in situ cast concrete, which covered the entire area or part of it, depending on the precast system. The differences among the five test structures concerned the dimensions of the columns and the beams, the height of the specimens, the construction of the top slab and the column-to-beam connections, as described in detail in the following. In one case, a precast panel wall was mounted on one side of the structure.
All precast members were constructed using C25/30 concrete and S500s steel. The assembly of the specimens was performed on the shaking table. After the mounting of the precast elements, the in situ cast concrete was poured. High strength concrete was used for this purpose, in order to obtain adequate strength within a week, time when the experiments started. All specimens were provided by PROET S.A., Greece.

Specimens

1st Series of Experiments

The test structure used in the 1st series of experiments is shown in Fig. 1. The overall dimensions were 2.80m×3.40m and the total height was 5.30m. The prefabricated columns had a height of 4.50m and a cross section 0.40m×0.40m. Beams of two types of cross section were used: orthogonal and L-shape. Precast T-shape girders were sitting on the L-shape beams, on top of which a lightly reinforced slab of 0.10m width was poured in situ. Details on the cross sections of the beams and the girders are given in Fig. 2a. The additional weight in this series of experiments was 60.0 KN, while the self weight of the beams, the girders and the slab was 91.8 KN and the weight of the columns and footings was 110.4 kN.
At each end of the beams, one shear dowel of 20mm diameter bolted on the top was used to fasten them with the columns (Fig. 2b). From the outer corner of the columns flanges extended up to the height of the beams, in order to form a nest of dimensions 0.15m×0.15m, which was filled with in situ cast concrete. This area was reinforced by an extra dowel extended from the column and steel hooks provided at the beam ends, as shown in Fig. 2b.
The footings were of 0.80m height and had conical sockets for the insertion of the columns. The gap between the column and the footing was filled by concrete. The footings were securely fastened to the shaking table by 8 dowels.
The test structure was designed according to Eurocodes 2 and 8 [14, 15] for peak ground acceleration 0.16g, soil type D and behaviour factor q=3.50, assuming pinned joints at the beam ends. According to this analysis, the columns were reinforced by 8 Ø16 rebar and Ø8/10 stirrups. The longitudinal reinforcement corresponds to the minimum required according to the codes.

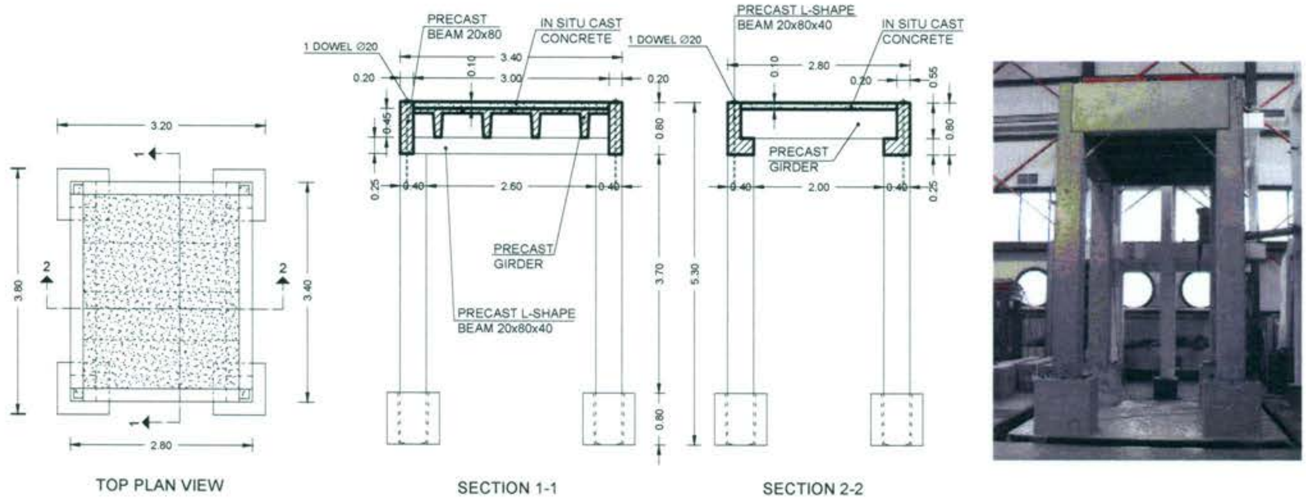


Fig. 1. Top view, vertical cross sections and photo of the specimen used in the 1st series of experiments.

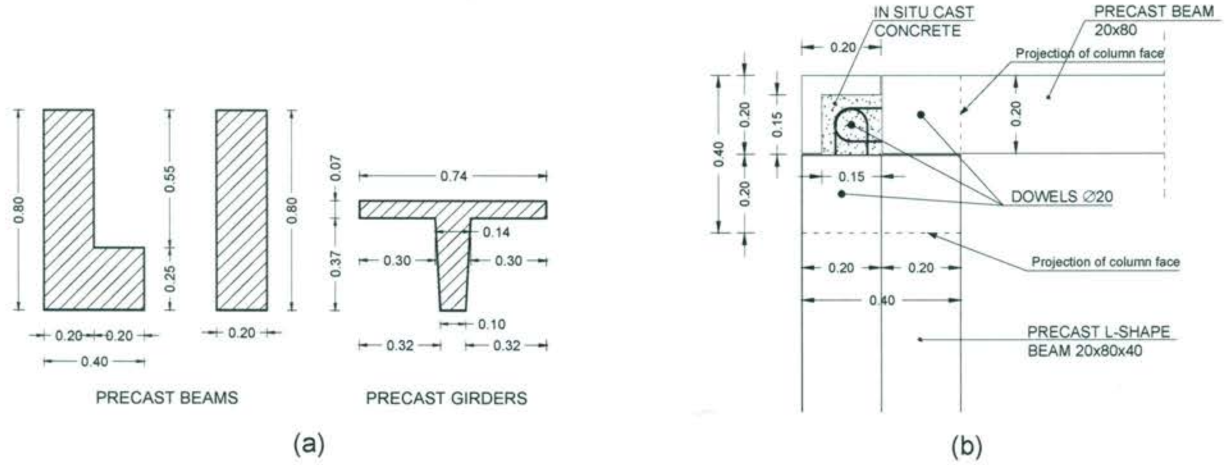


Fig. 2. Specimens 1, 2 and 3: (a) cross sections of the precast beams and girders; (b) detail of the column-to-beam connection (plan view).

2nd Series of Experiments

Several problems were encountered during the first series of experiments, which imposed specific actions to be taken in order to improve the experimental set-up. The problems raised and the corresponding improvements were:

(a) Large overturning moments were developed, causing undesirable response of the shaking table. It was decided to lower the height of the structure to 4.00m by reducing the height of the columns to 3.20m. Also, the additional weight at the top was reduced to 40 KN.

(b) Significant uplift occurred at the base, due to inadequate fastening of the columns to the footings. Uplift occurred because tensile forces developed at the columns, caused by the large overturning moment in conjunction with the small size of the specimen. In real structures, in which the distance between the columns is much larger, such tensile forces are not expected to be developed. It should be noted that the uplift changed significantly the seismic response of the structure, acting as base isolation. Therefore, in order to simulate better the behaviour of real structures it was decided to fasten the columns to the footings in the following series of experiments. This was done by 8 (2 at each side) horizontal steel dowels at each footing, connecting it with the column.

(c) The cross section of the columns was rather large, compared to the size of the test structure, resulting in a quite stiff model. The above-mentioned reduction of the height would make the structure even stiffer. For this reason, it was decided to reduce the cross section of the columns to 0.30m×0.30m, leaving, however, the top and bottom part unchanged (0.40m×0.40m), in order not to affect the beam-to-column and the column-to-footing joints. In addition, the height of the footings was reduced to 0.40m in order to increase the effective height of the columns and make them more flexible.

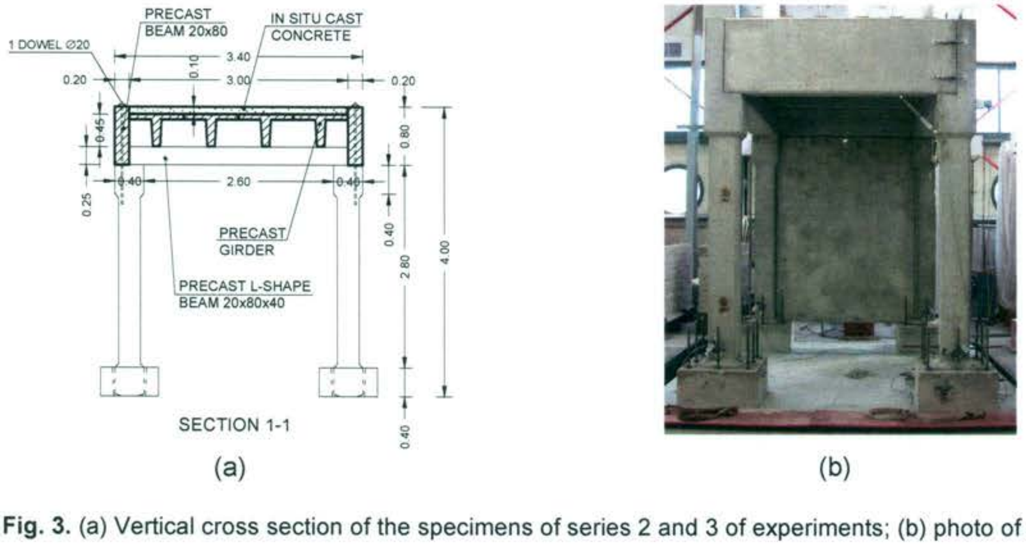


Fig. 3. (a) Vertical cross section of the specimens of series 2 and 3 of experiments; (b) photo of specimen of series 3.

The structure used in the 2nd series of experiments was similar to the one of the 1st series, as far as the construction system and the plan dimensions are concerned, but the above-mentioned changes were applied. In Fig. 3a, the corresponding vertical section is shown. The specimen was designed for the same assumptions used for specimen 1 and the columns were reinforced by 8 Ø14 rebar and Ø8/11 stirrups.

3rd Series of Experiments

In the 3rd series of experiments, an identical structure with the one of the 2nd series was used, except of a precast panel wall, which was mounted on the north side (Fig. 3b). The wall was tied to the columns only. The addition of the wall increased significantly the stiffness of the structure, especially in the transverse direction, but its eccentric position induced torsional vibrations.

4th and 5th Series of Experiments

In the 4th and 5th series of experiments, a different precasting system was tested. In these specimens, the slab consisted only from the girders, without in situ cast concrete, except of narrow strips, 0.30m wide, between the girders (Fig. 4a). The girders sit on the transverse beams on a flange (Fig. 4b) and their height was smaller than that of specimens 1, 2 and 3 (0.25m instead of 0.44m). In this way, a more flexible slab was formed. In the contrary, the beam-to-column connections were stronger, since two dowels Ø32 were used at each beam end. Also, rubber sheets were used at the joints between beams and columns in order to reduce the damage caused by impact, which was observed in the previous experiments.
In order to accommodate for the wider beams used in these experiments, the cross section of the top cushion of the columns was increased to 0.50m×0.50m. Except of that and the lack of the bottom bump, the columns were similar to those of specimens 2 and 3 and reinforced in the same way. The deletion of the bottom bump was made for construction simplicity and, theoretically, should not affect the response, since this part of the column was clamped to the footing by in situ cast concrete that filled the gap. However, it was observed during the experiments that the columns were not completely fixed at their base. This happened because the gap between the column and the footing was relatively wide and the infill concrete cracked after a few cycles of intense shaking, providing only partial fixation. In series 5, additional reinforcement was placed in the gap, in order to improve the behaviour of the concrete, with limited success though.
Specimens 4 and 5 had small changes in their dimensions, compared to specimen 2, specifically: the slab was 3.40m×2.90m at specimen 4 and 3.40m×3.50m at specimen 5; the column height was 3.40m and the total height was 4.02m.
The two specimens were similar, except of the longitudinal beams, which were mounted only on specimen 5 (see Figg. 5 and 6). Similarly to the transverse beams, these beams were fastened to the columns by two Ø32 dowels at each end. The presence of these beams resulted in a significant increase of the stiffness of the structure.

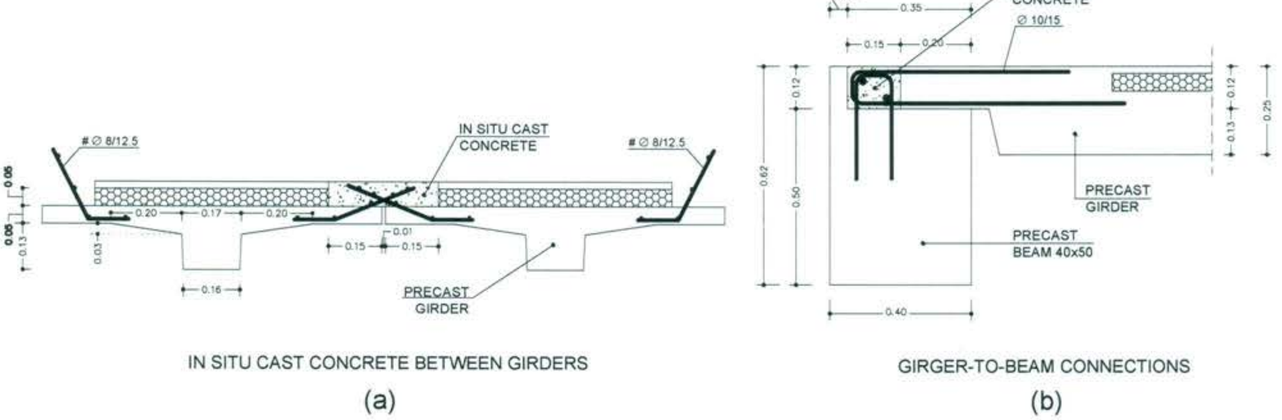


Fig. 4. Specimens 4 and 5: (a) detail of the in situ cast concrete between girders (vertical section); (b) detail of the girder-to-beam connection (vertical section).

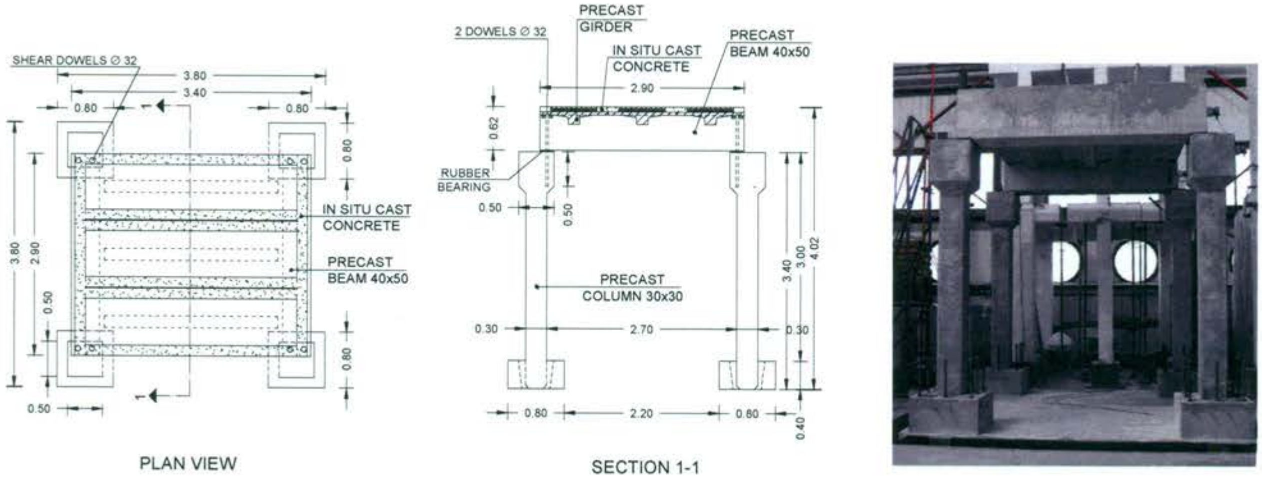


Fig. 5. Top view, vertical cross section and photo of the specimen used in the 4th series of experiments.

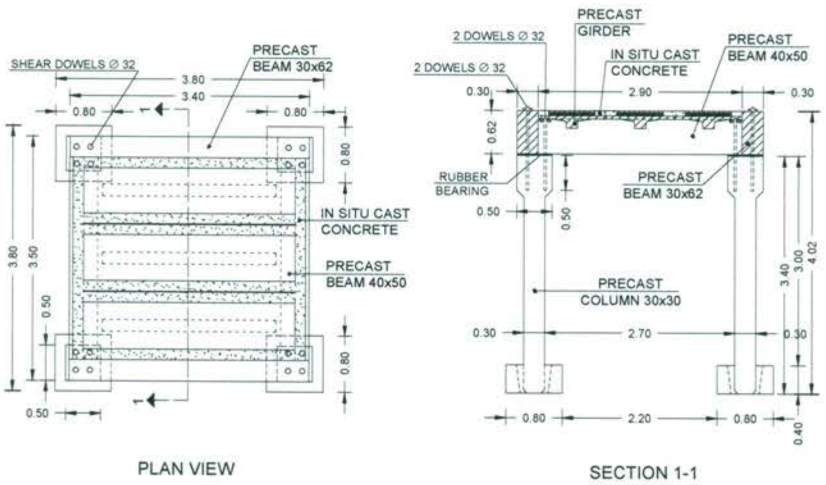


Fig. 6. Top view and vertical cross section of the specimen used in the 5th series of experiments.

Experimental Set-up

The deformation of the structure during the experiments was measured by accelerometers and displacement transducers. In Fig. 7 the instrumentation used in the first series of experiments is shown; similar was the set-up for the other experiments.

The accelerations were measured on the roof (two in each direction), at the top of the columns (two in each direction) and on the shaking table (4 horizontal and 4 vertical). The displacement transducers were measuring the roof displacement (two in each direction), the column-to-beam relative sliding and rotation and the column uplift. In order to measure the joint rotations, diagonal instruments were placed at the joints, as shown in Fig. 7. The rotation of the joint was then calculated (refer to Fig. 8) from the recording of the diagonal instrument D_d and the horizontal instrument D_h, which was measuring the opening of the joint, using the formula:

$$\Delta\varphi(t) = cos^{-1}\left[\frac{L^2 + [L + D_h(t)]^2 - [R + D_d(t)]^2}{2 \cdot L \cdot [L + D_h(t)]}\right] - \frac{\pi}{2} \tag{1}$$

For all experiments, L was equal to 370 mm.

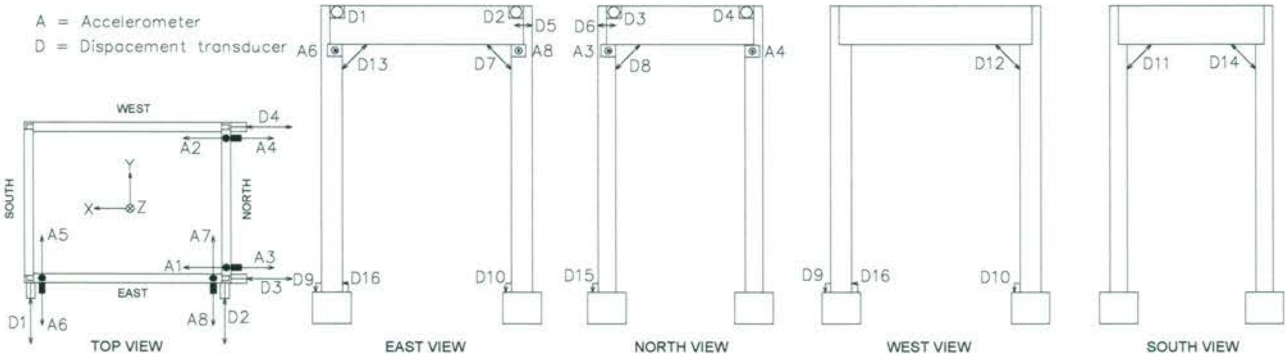


Fig. 7. Instrumentation set-up for specimen 1.

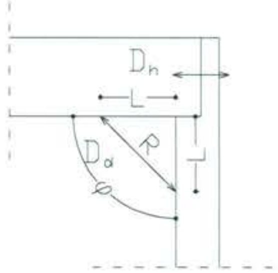


Fig. 8. Instrumentation of joints used to measure the rotation.

Base Excitation

For the base excitation, the two horizontal components of the Griva, Greece, 1990 earthquake were used (Fig. 9). The earthquake had magnitude M=5.9 and was recorded at the city of Edessa, in a distance of 31 km from the epicentre. The records were obtained on soft soil, the influence of which is evident at the accelerograms, which show long duration of the strong shaking, about 6.0s, governed by almost harmonic pulses of period around 0.6s. The peak acceleration was in the order of 0.1g.
Although the peak ground acceleration (pga) is small, this earthquake was chosen because, if amplified to pga values corresponding to strong earthquakes (pga greater than 0.5g), it causes significant damage to flexible structures, as the ones under consideration, due to the long-period pulses that it contains. For each specimen, experiments were performed applying several amplification factors to the records, starting from the original ones (amplification=1, pga=0.1g) and gradually increasing them up to six times (pga=0.6g) or more. In some cases, seismic motions corresponding to nine-time amplification (pga=0.9g) were applied during the final stage of the experiment. Although such high values of amplification correspond to very strong seismic motions, which would probably have different characteristics and thus it is doubtful whether they have physical meaning, this procedure was chosen in order to impose large displacements to the test structures keeping the base motion parameters unchanged, except of the amplitude, in all the experiments.

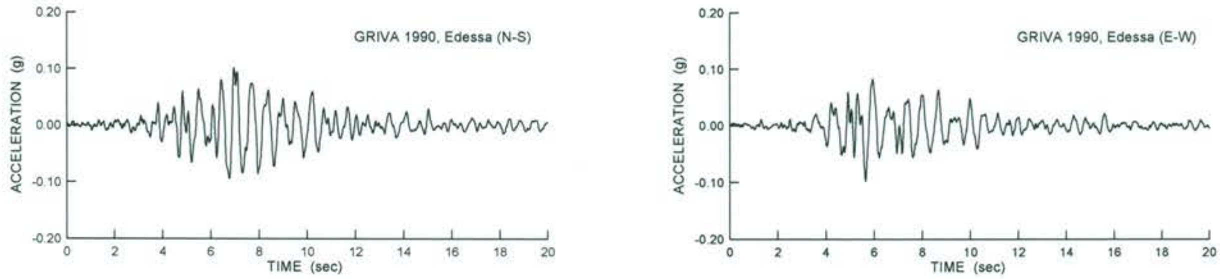


Fig. 9. Horizontal components of the Griva, Greece, 1990 earthquake, recorded at the city of Edessa.

EXPERIMENTAL RESULTS

Dynamic Characteristics of Specimens

In Tab.1, the dynamic characteristics of all the specimens are given, which were derived by sine logarithmic sweep tests in the frequency range 1÷35 Hz. A separate test was performed in each direction. The amplitude of vibration was 0.750 m/s^2 in the X-direction and 1.00 m/s^2 in the Y-direction. The natural periods were determined from the time history of the acceleration response of each specimen, while the damping ratios were calculated applying the half power bandwidth method. The sine sweep rate was one octave per minute and the variation of the exciting frequency f (in Hz) with time t (in sec) was governed by the following rule:

$$f = 1.0 \times 2^{t/60} \tag{2}$$

The high values of damping obtained for the first specimen (8%) should be attributed to the rocking that occurred in this case. The small eigenperiod of specimen 3 in the Y-direction was due to the presence of the panel wall in this direction, which also caused the significant increase in damping. The partial fixation provided at the base of the columns of specimen 4 caused the increase in the natural periods shown in Tab. 1. In the case of specimen 5, additional reinforcement was placed at the column-to-footing gap and for this reason smaller periods were obtained in that case. However, this intervention worked only for weak base motions (including the sweep tests); for input motions amplified more than 4-5 times, the infill concrete cracked and the joints behaved as partially fixed, as mentioned above.

Tab. 1. Weight and dynamic characteristics of the specimens

	Weight (kN)			X-direction			Y-direction		
Specimen	Self	Additional	Total	f (Hz)	T (sec)	ζ (%)	f (Hz)	T (sec)	ζ (%)
1	202.2	60.0	262.2	2.90	0.34	8.00	2.54	0.39	8.00
2	168.1	40.0	208.1	3.16	0.32	1.57	3.34	0.30	2.00
3	158.8	40.0	198.8	4.86	0.21	1.65	6.35	0.16	8.47
4	151.5	40.0	191.5	2.05	0.49	3.20	2.52	0.40	6.90
5	143.2	40.0	183.2	3.88	0.26	5.37	3.56	0.28	5.31

Seismic Response

Representative results of the experimental data are shown in Figg. 10-13. In Fig. 10, the time history of the top acceleration in x-direction is shown for all specimens and for base excitation corresponding to 5-times amplification of the Edessa record (approximately, pga=0.5g in x-direction). The largest accelerations were obtained for specimens 2 and 3, in which the peak value on the roof was almost 2.5 times larger than the base one. The smallest accelerations were obtained for specimens 1 and 4, for which the ratio of the top to the base peak values was approximately 1.5. This may attributed to the uplift that was observed in the case of specimen 1 and the partial fixation at the base of specimen 4.

It is interesting to compare the behaviour of specimens 2 and 3, which were identical except of the panel wall mounted on the N side of specimen 3 (y-direction). Although the presence of the wall induced torsional vibrations to the structure, it did not affect the response in the x-direction (Fig. 10). In the contrary, the response in the y-direction was significantly altered by the rotation, as shown in Fig. 11: the roof acceleration at the S side (opposite to the wall) was much larger than the one at the N side (above the wall). Thus, although the overall response in the y-direction was reduced, the acceleration at the side opposite to the wall was similar to the one of specimen 2 (Fig. 11a).

The response of test specimen 4 was quite different from the one of the other ones, as shown in Fig. 10. This happened because the columns were partially fixed at their base in this case, as described above. This resulted in a type of base isolation, leading to significantly smaller top accelerations, compared with the ones recorded at the similar test structure 5.

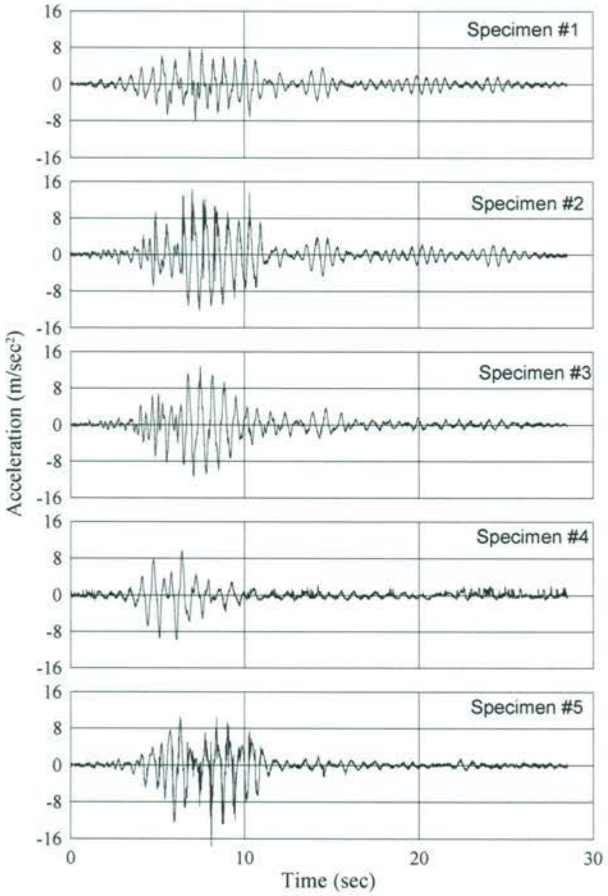


Fig. 10. Horizontal roof acceleration time histories of all specimens in x-direction, for the input motion amplified five times.

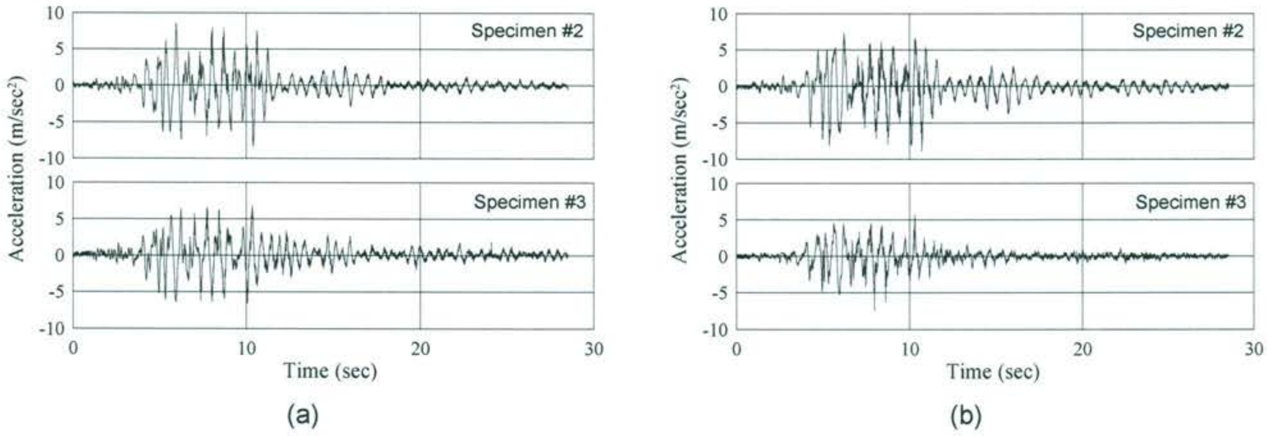
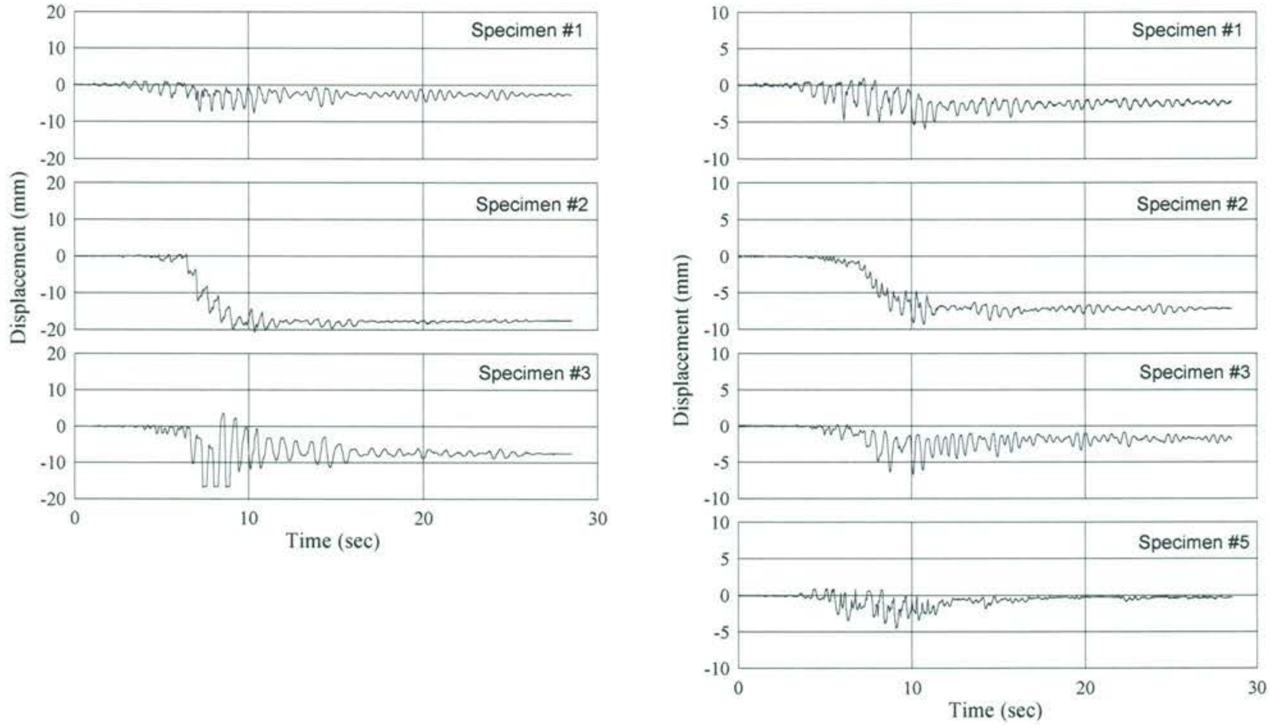


Fig. 11. Comparison of the horizontal roof acceleration time histories of specimens 2 and 3 in y-direction for the input motion amplified five times: (a) south side; (b) north side (above the wall of specimen 3).

In Fig. 12, the time histories of the displacement corresponding to the opening of the joints are presented for the base input amplified five times. In the case of specimens 1, 2 and 3, the plots show the recordings of the displacement transducers that were measuring the difference in the movement of the beams and the column flanges, like D5 and D6 in Fig. 7. In the case of specimen 5, the plot shows the gap opening between the two normal beams at the joint (see section 1-1 in Fig. 6); such a measurement was possible only in the y-direction. In specimen 4, only the longitudinal beams were mounted and, therefore, a similar measurement could not be obtained.



(a) (b)

Fig. 12. Comparison of the time histories of the horizontal relative displacement between column and beam (specimens 1, 2 and 3) or between normal beams (specimen 5), for the input motion amplified five times: (a) x-direction; (b) y-direction.

As mentioned above, in the case of specimens 1, 2 and 3, the beam-to-column connection was made by only one Ø20 dowel at each beam end. In the contrary, in the case of specimens 4 and 5, two Ø32 dowels were used. The much weaker connection in the first case resulted in residual opening of the joints, which is evident in the plots of Fig. 12. Especially in the case of specimen 2, a significant permanent dislocation of the beam, approximately 1.8 mm, was recorded in the x-direction. Note that this record was obtained at about the mid-height of the beam; the gap opening at the top of the structure was larger, due to the additional effect of the torsional rotation of the beam that occurred. The corresponding permanent displacements of specimen 3 were quite smaller, due to the additional stiffness introduced by the panel wall (Fig. 12): almost one half and one third in x- and y-direction, respectively. In the case of specimen 5, the strong connections prevented this type of damage.

The time histories of the joint rotations for 5-times amplification of the base motion are shown in Fig. 13 for the roof moving (a) in the x-direction and (b) in the y-direction. In case (a) the plots give the rotation around y-y axis and in case (b) around x-x axis. The rotations were obtained according to eq. (1). In the case of specimens 4 and 5, the relative movement of the beams with respect to the columns was small for this level of excitation; for this reason, D_h was set to zero. No plot is given for specimen 4 in x-direction, because there was no beam in this direction to measure the rotation.

It is interesting to note the significant permanent distortion of the joints in the case of specimen 2, which was caused by the inclination of the columns. In this case, the columns cracked at their base but their residual inclination should be rather attributed to the severe bending and distortion of the joint dowels, which forced the joints to open, and not to the plastic rotation at the base of the columns.

A similar situation occurred for specimen 3, which was similar to specimen 2 except of the presence of the wall, but only in x-direction. In y-direction, the presence of the wall reduced significantly the joint rotations and prevented the permanent distortion. In Fig. 13b, the plot for specimen 3 corresponds to the rotation of the south side of the structure, opposite to the wall. The joint rotations above the wall were not measured but evidently they were smaller.

In the case of specimens 4 and 5, the joints were much stronger and thus no residual distortion occurred, although the amplitude of the joint rotations during the seismic motion was larger than the one for specimens 1-3, due to the partial fixation at the base.

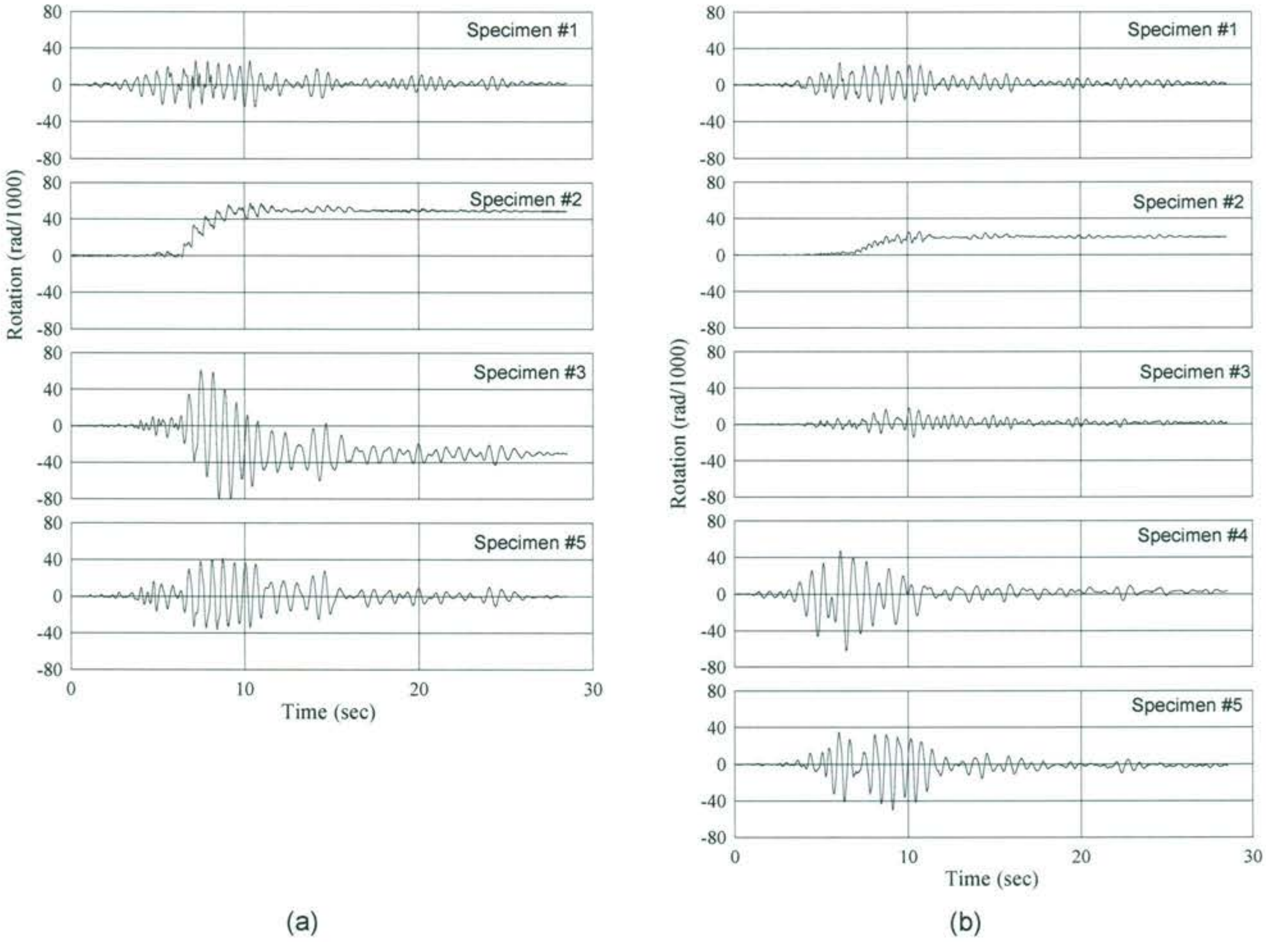


Fig. 13. Joint rotation time histories of all specimens for the input motion amplified five times and for: (a) the roof moving in the x-direction; (b) the roof moving in the y-direction.

Observed Damage

The first three series of experiments concerned the same type of beam-to-column connection, in which the beams were fastened to the columns by one Ø20 dowel at each end. Severe damage was observed at the joints for strong seismic motions, which included:

- Opening of the joints due to permanent dislocations and torsional rotations of the beams (Fig. 14a). Such residual displacements show that the dowels bent during the seismic response;
- Damage to the beam edges due to the impact that occurred between beams and columns (Fig. 14a). This damage was caused mainly by the joint rotations (Fig. 13), which also caused a smaller damage to the column edge (Fig. 14a). It should be noted that the beams seated directly on the columns; if some kind of bearings was used, this damage would be reduced;
- Damage to the concrete surrounding the dowels (Fig. 14b). This type of damage shows that high stresses were developed around the dowels. In the case of specimens 4 and 5, in which two much stronger dowels were used at each connection, such damage was not observed;
- Damage to the column flanges and the in situ cast concrete (Fig. 14c). The experiments showed that the in situ cast connection did not have enough strength to resist strong seismic excitation and cracked severely. The column flanges could not bear the forces that were developed and cracked, failing to confine the in situ cast concrete.

It should be noted that in the case of specimens 2 and 3 (with the reduced cross section of the columns compared to specimen 1) and for excitations with peak acceleration greater than approximately 0.5g, cracks appeared at the base of the columns, showing the development of plastic hinges at these points. Inclination of the columns was also observed as a result of the joint opening.

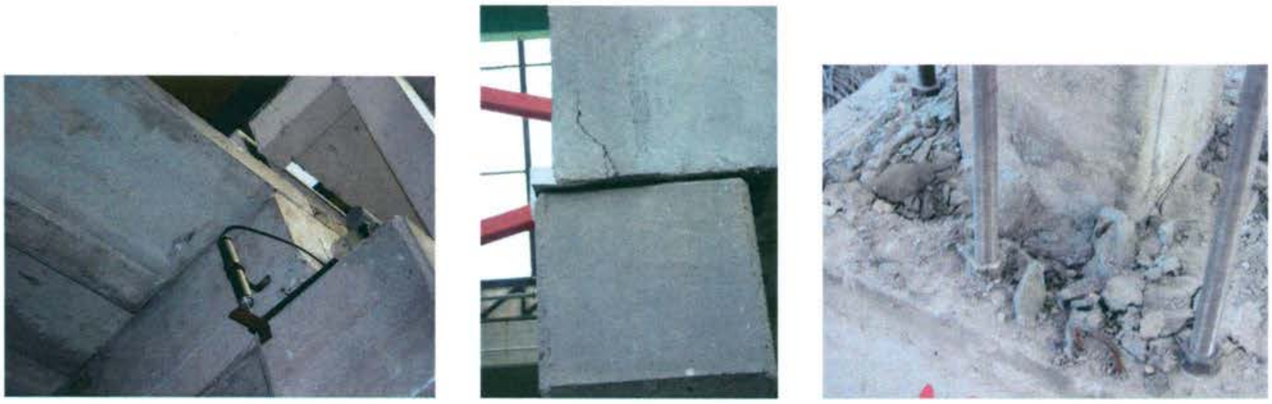
In the case of specimens 4 and 5, two Ø32 dowels were used at each connection. Also, the top plate was more flexible than the one of the previous experiments. The observed damage was much smaller to these structures, although dislocations of the beams occurred in this case too, but for very strong base excitations, around 0.8g (Fig. 15). In general, these structures could sustain very large displacements without significant structural damage to the joints. For strong input motions, the damage was concentrated to the column-to-footing connections, mainly to the infill concrete of the corresponding gap. For this reason, the foundation failed to provide full fixation at the base of the columns leading to large displacements at the roof, which, however, did not affect the integrity of the joints.

CONCLUSIONS

The experimental tests reported in this paper concern the seismic response of five single-storey, precast RC structures tested on the shaking table facility of the Laboratory for Earthquake Engineering at the National Technical University of Athens, Greece. Two types of beam-to-column connections were examined: one weak and one strong. In the latter case, a more flexible top plate was constructed. In the case of the weak connections, severe damage occurred at the joints for strong base motions, requiring repair after the earthquake. Permanent dislocation and torsional rotation of the beams and inclination of the columns was also observed. The strong connections behaved much better and proved that they can sustain severe earthquake motions without significant damage. For the test structures examined, the damage was concentrated mainly at the column-to-footing connections, resulting to partial fixation of the columns and large roof displacements, which, however, did not cause severe damage to the beam-to-column joints.

(a) (b) (c)

Fig. 14. Typical damage to specimens 1, 2 and 3: (a) permanent torsional rotation of beams and damage to the corners due to impact with the column; (b) damage to the concrete surrounding the dowel and the column flanges; (c) damage to the in situ cast concrete. All specimens were subjected to a series of base excitations with pga up to approximately 0.5g.



(a) (b) (c)

Fig. 15. (a) Small damage to beams of specimen 4 due to impact; (b) dislocation and damage to beams of specimen 5; (c) damage to the column-to-footing joints of specimen 5. Both specimens were subjected to a series of base excitations with pga up to approximately 0.8g.

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