Monotonic, cyclic and shaking table tests on pinned beam – column connections

I.N. Psycharis & H.P. Mouzakis
School of Civil Engineering, National Technical University, Athens, Greece

15 WCEE
LISBOA 2012

SUMMARY:
The results of an extensive experimental investigation on pinned beam – column connections are reported. Two types of tests were performed: (i) Static, monotonic and cyclic and (ii) dynamic shaking table tests on half-frames under seismic excitation. The research was focused on important design aspects and the effect of several parameters on the shear resistance of the connection. The results show that the resistance of dowel connections depends mainly on the cross section of the dowels, provided that the cover concrete is thick enough to prevent early spalling. The response of the connections under dynamic excitations is, in general, compatible with the one for cyclic loading, but the damage is generally less; though, repetition of the same earthquake increases the damage significantly. The vertical component of the earthquake does not seem to be important. Based on the experimental results, a new formula is proposed for the estimation of the resistance of pinned connections.

Keywords: design of dowels; pinned connections; precast; cyclic response; shaking table

1. INTRODUCTION

Precast frame structures have been widely applied around the world especially for single-storey or low-rise, mainly industrial, buildings. In such systems, the seismic performance greatly depends on the type of the connections, especially the beam-to-column ones. In many countries (Italy, Greece, Spain, Portugal, Slovenia, Turkey, Armenia etc.), simple pinned connections made of dowels are traditionally used in frame type buildings.

Although the safety of precast constructions during earthquakes has been recognized as one of the most important issues in the design, surprisingly enough only limited research has been conducted on the experimental investigation of the resistance of the connections, especially of pinned connections materialized by dowels. Among others, the investigation of the dowel mechanism by Vintzeleou and Tassios (1985, 1987) and Tsoukantas and Tassios (1989) is mentioned because it is widely used in the design.

Recently, significant experimental and numerical research on the seismic behaviour of precast structures with pinned connections was conducted in the framework of the European FP7 project "SAFECAST: Performance of innovative mechanical connections in precast building structures under seismic conditions", which focused on the experimental investigation and the analytical modelling of typical mechanical, dry connections that are used in precast structures. The experimental investigation on pinned beam – column connections that was carried out at the Laboratory for Earthquake Engineering of the National Technical University of Athens, Greece in the framework of SAFECAST is presented herein. The research was aiming to the investigation of the shear resistance of dowel connections to monotonic, cyclic and dynamic loading and to recommendations concerning their design towards the improvement of their seismic response. Only indicative results are reported in this paper; more details can be found in SAFECAST (2012a & b) and Psycharis & Mouzakis (2012).

2. PURE SHEAR MONOTONIC AND CYCLIC TESTS

2.1. Specimens and experimental programme

With the static tests, the effect of various design parameters on the resistance of pinned beam-to-column connections under pure shear monotonic and cyclic loading was investigated. The specimens represented a typical pinned connection of linear precast members and consisted of the end parts of the beam and the column that were connected by steel dowels (Figure 1). Each specimen was subjected to monotonic or cyclic, displacement-controlled loading, applied at the rear end of the beam, while the column was securely fastened to the strong floor of the Laboratory. For the tests, a 500 kN capacity MTS hydraulic actuator with displacement range of ± 200 mm, connected to the reaction wall of the Laboratory, was used. The driving force was applied exactly at the level of the joint, in order to achieve pure shear conditions. To prevent any out-of-plane motion, a special device was used, which allowed only uniaxial motion of the beam without any rotation at the joint. It should be noted, however, that in real structures there might be rotation at the joints, caused by the deflection of the columns, which might reduce the overall shear resistance of the connections, as observed in the dynamic tests on frames, reported in section 3. No extra vertical load was applied at the joint.

The cross sections of the column and beam elements were orthogonal with dimensions similar to the ones usually applied in precast single-storey buildings (physical scale). All specimens were identical in dimensions. The columns and the beams were constructed of high strength concrete with measured mean compression strength of 30 to 35 MPa, while the steel of the dowels and the reinforcement was of grade B500C. For the assembly of the specimens, Ø65 waiting ducts were placed in the beams for the passage of the dowels during mounting. The ducts were grouted after assemblage with non-shrinking grout and the dowels were bolted at their top. Most experiments were performed 24 h after grouting when the mean compression strength of the grout was about 23 MPa, considerably less than the strength of the concrete. However, two experiments were performed with high strength grout, about 44 MPa. Horizontal hooks, fully anchored in the body of the beam, were placed in front of the dowels, specifically: 5Ø12/5 at the lower 0.30 m of the section and 3Ø12/10 at the rest of the section.

For the proper sitting of the beams on the columns, elastomeric (neoprene) pads of about 20 mm thickness were used, as typically done in practice. It is noted that the elastomeric pads affect the shear resistance of the connections, because they increase the distance of the plastic hinges that are developed in the dowels for large displacements.

The experimental investigation was focused on important design parameters that affect the shear resistance of the joint, and on possible improvements and recommendations for a more efficient design of pinned connections. The parameters that were investigated include:

- The diameter of the dowels. Most of the tests were performed on connections made of 2Ø25 dowels because, in that case, we could bring the specimens to failure or close to it before exceeding the limits of the hydraulic actuator. However, some tests were performed on connections made of 1Ø32 and 2Ø16 dowels. It is noted that connections with Ø16 dowels are rather unusual in practice, but were used in this investigation for comparison reasons.
- The number of dowels. Most tests were performed on connections with two dowels. However, some tests were performed on connections with one dowel in order to investigate probable interaction phenomena between adjacent dowels.
- The distance d of the axis of the dowels from the beam edge in the loading direction (see Figure 1), which is used as a measure of the thickness of the cover concrete. Due to lack of space, it is usually desirable in practice to reduce the distance d as much as possible. Most experiments were performed with specimens with d = 0.10 m, but specimens with d = 0.15 m and d = 0.20 m were also tested.
- The strength of the grout, specifically its effect on the overall resistance of the connection.

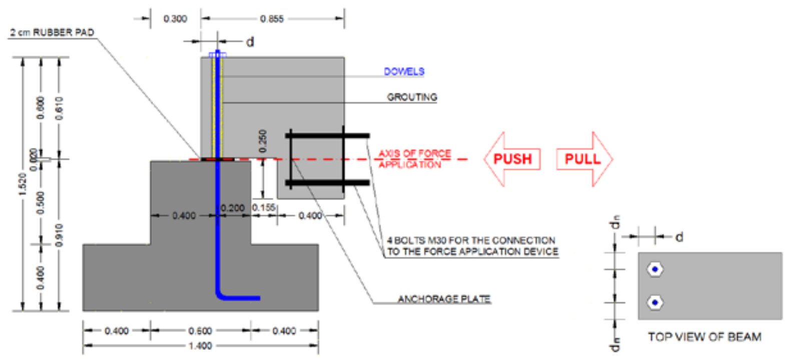


Figure 1. General layout of the specimens used in monotonic and cyclic tests.

In total, 22 tests were performed. The monotonic tests were executed with the driving force applied in the pull or the push direction up to the failure of the dowels or when the maximum capacity of the actuator was reached. The cyclic tests were displacement-controlled and were performed at a rate of 0.2 mm/sec. Most of the cyclic tests started in the pull direction, but in some cases two cyclic tests were performed: one starting in the pull direction and one in the push. Three cycles were performed at each displacement level, which was increasing in steps of the yield displacement, d_y; the latter was determined from the force-displacement curves for monotonic pull and monotonic push loading.

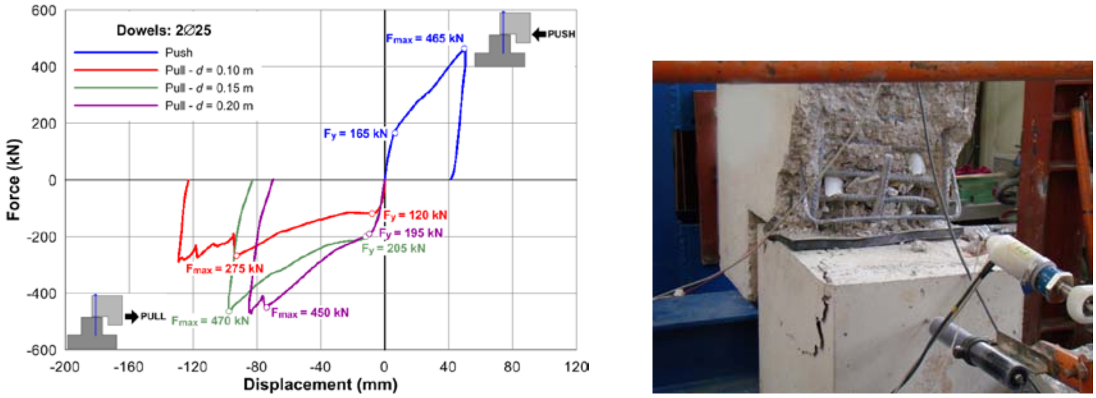
2.2. Experimental results

For large displacements, the dowels were considerably bent around the joint (Psycharis and Mouzakis 2012). Thus, although the tests were performed under pure shear conditions by allowing the beam to move only horizontally, in the strength of the connection contributed not only the shear resistance of the dowels but also their strength in tension. The highly deformed part of the dowels extended a few centimetres in each side of the joint and plastic hinges were developed at its ends. As the shear displacement was increasing, the shear force of the dowels was decreasing and their tension was increasing. Breaking of the dowels, whenever occurred, took place at the place of plastic hinges, inside the beam or the column. Thus, a portion of the broken dowels extruded from the column or the beam inside the opposite element and continued to pose significant resistance against horizontal movement. For this reason, it was not always possible to determine whether the dowels had failed or not during a test. Due to this phenomenon, in some tests the dowels broke at both plastic hinges.

2.2.1. Monotonic loading

Figure 2a shows the force-displacement curves obtained for specimens with 2Ø25 dowels and various values of the distance, *d*, of the axis of the dowels from the beam edge. In the push direction, the test stopped when the driving force was close to the capacity of the actuator, thus the actual resistance of the connection was somehow larger. In the pull direction, the maximum attained force was determined by the failure of the dowels.

Significantly reduced yield force and ultimate resistance were attained in the pull direction for d = 0.10 m (d/D = 4) compared to those in the push direction. Compared with the corresponding resistance for d = 0.20 m (d/D = 8), reduced values by 60% were attained. For the intermediate case of d = 0.15 m (d/D = 6), the yield force and the ultimate resistance were similar to the ones for d = 0.20 m but they were attained at larger displacements. This significant reduction in the resistance in the pull direction for small values of d/D is attributed to the spalling of the cover concrete of the dowels (Figure 2b), which started early in the test and resulted in a temporary loss of stiffness, as evident from the "plateau" that is observed on the force-displacement diagram for d = 0.10 m in Figure 2a. Such extensive damage was not observed for thicker covers, where only minor cracking occurred at the base of the beam. Even for d = 0.10 m, however, the dowels were restrained for large displacements by the horizontal stirrup reinforcement of the beam and the resistance was increasing again, up to the point where the dowels broke in tension. The horizontal stirrup reinforcement did not break at any test, but, in many cases, the lower bar was evidently bent due to the pushing of the dowels.



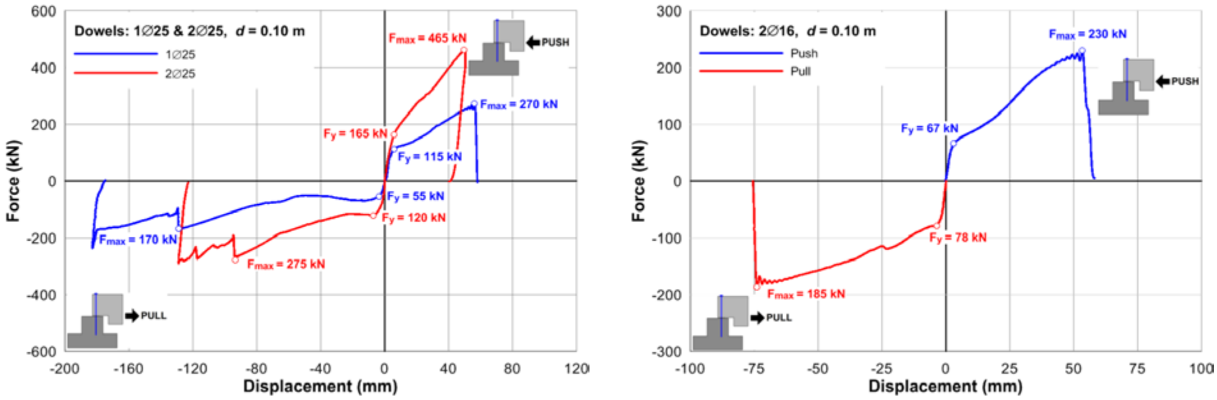
(a) (b)

Figure 2. (a) Force-displacement relation for monotonic push and monotonic pull tests of connections with 2Ø25 dowels and various values of the cover d; (b) Damage to the specimen with d = 0.10 m, observed at the end of the monotonic pull test.

Damage was also observed during the monotonic push test. In this case, splitting of the concrete in the normal to the loading direction occurred in the beam and the column and also splitting occurred at the top of the beam, in the region around the dowels, due to the high pressure applied by the bolts.

In Figure 3a, comparison of the response of connections made of 2Ø25 and 1Ø25 dowels is presented. In both cases, the distance d of the dowels from the beam edge was 0.10 m (d/D = 4). It is seen that the resistance of the connections with two dowels was less than two times the corresponding value for the connections with one dowel. This must be attributed to the interaction of the two dowels in the transverse direction and the plastic zone that was developed in the region between them, and to the smaller concrete cover of the dowels in the normal to the loading direction, d_n (see Figure 1): for the specimens with two dowels, d_n = 0.10 m (d_n/D = 4.0), while for the specimens with one dowel, d_n = 0.20 m (d_n/D = 8.0). It is noted that for loading in the pull direction, spalling of the cover concrete occurred also in the specimen with one dowel, similarly to the specimen with two dowels.

The resistance of joints made of 2Ø16 dowels with d = 0.10 m is shown in Figure 3b. In this case, d/D = 6.25 and it is seen that the yield force and the ultimate resistance in the pull direction were close to the ones in the push direction, though the response was not symmetric (the maximum force was attained at larger displacement). Similar behaviour was observed for connections with 1Ø32 dowels – d = 0.10 m and 2Ø25 dowels – d = 0.15 m, for which the ratio d/D was about the same.



(a) (b)

Figure 3. (a) Comparison of force-displacement diagrams for connections with 1Ø25 and 2Ø25 dowels under monotonic push and monotonic pull loading (d = 0.10 m); (b) Force-displacement diagrams for monotonic push and monotonic pull tests of connections with 2Ø16 dowels and d = 0.10 m.

Comparisons between the obtained results show that the shear resistance in the push direction depends mainly on the cross section of the dowels, independently of their number and their diameter, as far as the thickness of the cover concrete in the transverse direction and the distance between the dowels are sufficiently large to prevent secondary effects. In the pull direction, the response is generally better for connections made of dowels of smaller diameter, other critical parameters being the same.

2.2.2. Cyclic loading

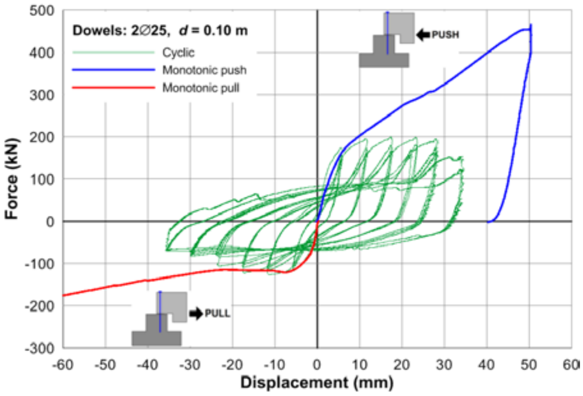
The cyclic response of the connection with 2Ø25 dowels and d = 0.10 m (d/D = 4) is shown in Figure 4a, in which the corresponding curves for monotonic push and monotonic pull are also shown for comparison. It is seen that the cyclic response follows the monotonic one up to displacement at least equal to $2d_y$. After that point, the cyclic resistance remains practically constant for several more cycles, while the monotonic resistance generally increases.

Due to the small thickness of the cover concrete, asymmetric cyclic response was observed, as expected, with the resistance in the push direction being almost double the one in the pull direction. Significant pinching was also observed, especially in the push direction, which was increasing with the number of applied cycles due to the loosening of the dowels caused by the crushing of the concrete around them. The observed damage was similar to the one for monotonic tests: spalling of the cover concrete of the beam during pulling (Figure 4b) and cracks in the column in the transverse direction.

It is interesting to note that no significant strength degradation (larger than 20%) occurred even for quite large shear displacements, up to 30 mm in the push direction and about 20 mm in the pull direction. If these values are considered as ultimate displacements, the corresponding shear ductility is μ_s = 5 and μ_s = 3.5, respectively.

In Figure 5a, the comparison of the behaviour of connections made of 1Ø25 dowel and 2Ø25 dowels is presented. It is seen that the resistance of the connection with one dowel is practically one half the resistance of the connection with two dowels. This shows that the effect of secondary phenomena, as the interaction of the dowels and the influence of the small concrete cover in the normal to the loading direction, is less important for cyclic loading than for monotonic loading where the resistance of the connections with two dowels was less than two times the corresponding value for one dowel (Figure 3a) due to these phenomena.

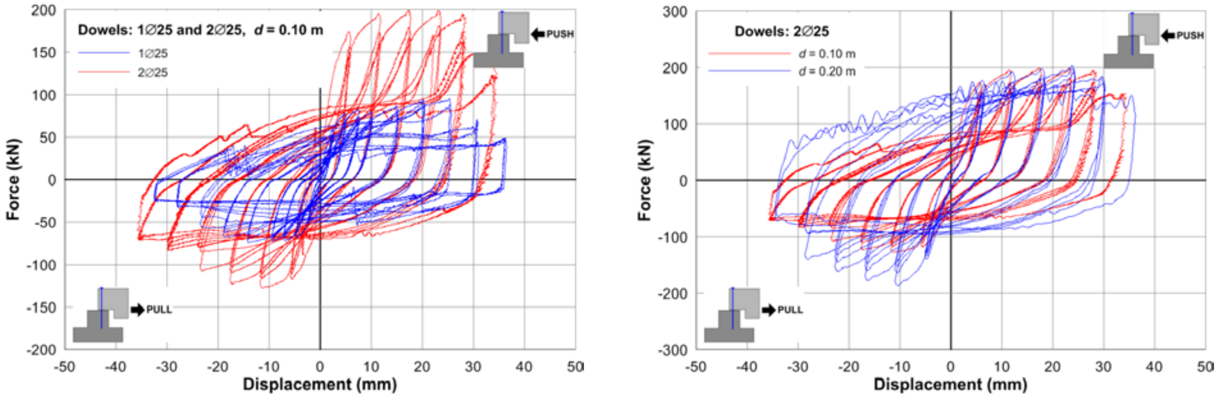
The effect of the distance d of the dowels from the beam edge on the cyclic behaviour is shown in Figure 5b, where the response of the connection made of 2Ø25 dowels with d = 0.10 m (red line) is compared with the one of a similar connection with d = 0.20 m (blue line). As expected, the connection with the thicker cover showed more symmetric response and less pinching. It should be noted that significant spalling occurred in the specimen with d = 0.10 m during pulling, which, however, did not affect the resistance in the push direction.



(a)

(b)

Figure 4. (a) Comparison of the behaviour of connections with 2Ø25 dowels and d = 0.10 m for monotonic and cyclic loading; (b) Damage to the beam at the end of the cyclic test.



(a) (b)

Figure 5. Comparison of the cyclic behaviour of connections made of: (a) 1Ø25 and 2Ø25 dowels with d = 0.10 m; (b) 2Ø25 dowels with d = 0.10 m and d = 0.20 m.

3. SHAKING TABLE TESTS

3.1. Specimens and experimental programme

The shaking table tests were performed on single-storey, one-bay frames made of precast columns and beams connected with pinned connections materialized by dowels. Due to limitations in the span length imposed by the shaking table dimensions, only one half of each frame was modelled (Fig. 6a). At the free end of the beam, which corresponded to the mid-span in the prototype structure, a special device was used that provided free rotation and free sliding of the beam (Figure 6b).

In total, nine specimens were constructed and tested. In all specimens, the cross section at the two ends of the beams was orthogonal, of dimensions 0.40 m × 0.60 m, while in the middle part flanges were provided at the top side forming a T-shaped section, in order to facilitate the placement and the fastening of the extra mass that was added on top (Fig. 6). The columns were fixed at their base and their cross section and reinforcement was varying in order to investigate the following types of frames:

(a) (b)

Figure 6. (a) Experimental setup showing the half-frame specimen; (b) special device for achieving sliding pinned support at the free end of the beam.

- Type FW: Flexible and Weak columns. The columns' cross section was 0.30 m × 0.40 m (the first dimension refers to the direction of shaking) and the reinforcement at the base was less than what would be required by seismic design (under-reinforced columns). In this case, plastic hinges were formed at the base of the columns during the tests and the effective period of the systems was long. Small shear forces were induced to the column – beam joints, not capable to produce any severe damage to the connections despite the large rotations. With these tests, the inelastic behaviour of the columns and the response of the beam – column joints to large rotations were investigated.
- Type SS: Stiff and Strong columns. The columns' cross section was 0.60 m × 0.40 m and the reinforcement at the base was much more than what would be required by seismic design (heavily-reinforced columns). In this case, the columns responded elastically during the tests allowing for large shear forces to be developed at the beam – column joints, capable to produce their failure. The rotations at the joints were very small and, practically, the connections behaved as in pure shear loading.
- Type FS: Flexible and Strong columns. The columns' cross section was 0.30 m × 0.40 m and the reinforcement at the base was more than what would be required by seismic design (over-reinforced columns). In this case, the columns responded practically elastically during the tests, but with significant bending. Large shear forces and rotations were induced to the beam – column connections, which were damaged. With these tests, the response of the joints under simultaneous shear and rotation was investigated.
- Type FN: Flexible and Normal columns. The cross section of the columns was as in type FS and FW, but the reinforcement was according to the seismic analysis (normally reinforced columns). These specimens behaved similarly to the FW specimens, but with less damage at the base of the columns.

Two Ø25 dowels were used in all but one specimen, in which the connection was made of one Ø25 dowel. The distance of the axis of the dowels from the front side of the beams was d = 0.10 m in all cases, except one in which d = 0.20 m. The grade of the concrete and the steel and the strength of the grout were as in the static tests and the same stirrup reinforcement detail was applied in front of the dowels. For the proper sitting of the beams on the columns, elastomeric (neoprene) pads of 20 mm thickness were again used, as typically done in practice in order to prevent impact between the beam and the column during the seismic response, due to the rotation at the joint.

For each specimen, a series of tests were performed with the base acceleration increasing stepwise up to the point that severe damage was observed or the limit of the shaking table in displacement (±0.10 m) was reached. In the latter case, the base excitation was repeated several times at its maximum intensity, in order to investigate the performance of the connections under repeated earthquakes. Most of the experiments were performed with the base motion applied in the horizontal direction only (longitudinal direction of the beam); however, some experiments were performed for simultaneous application of the horizontal and the vertical components of the ground motion, in order to investigate the effect of the vertical component of the earthquake on the resistance of the joint and the overall behaviour of the structure. In such cases, the same scaling was applied in both components of the original record.

3.2. Experimental results

Indicative results are shown in Figure 7 for the specimen with stiff and strong column (SS) subjected to the stepwise increasing Petrovac record of the Montenegro, 1979, earthquake applied in the horizontal direction. In this case, the bending of the column and the induced rotations at the joint were small, thus the response of the connection was governed by the shear behaviour, similarly to the static cyclic tests.

In general, the dynamic response was close to the cyclic one (also shown in Figure 7), showing similar hysteretic loops and the same maximum resistance in the push direction. In the pull direction, however, the resistance was larger than what was observed in the static tests, because considerably less damage occurred in the beam during the seismic excitations. Indeed, the observed damage (Figure

7d) was quite different compared with that for cyclic loading. In the cyclic tests, spalling of the cover concrete occurred in the beam even for small displacements, leading to 40% reduction in the resistance in the pull direction, while damage in the column was observed only for large displacements. In the dynamic tests, the damage at the joint was evidently smaller for the same level of joint slip, with considerably less spalling occurring in the beam and only for intense shaking. Also, the damage was distributed in both the beams and the columns (Figure 7d). As a result, similar resistance was attained in the pull and in the push directions and the response was close to symmetric. The reduced damage should be attributed to the dynamic effect of the loading and the consecutive increased tensile strength of the concrete.

It should be noted, however, that the damage increased when the base motion with *pga* = 0.65 g was repeated, leading to a reduction in the resistance and to increased displacements, as evident from the comparison of Figures 7b and 7c.

Similar conclusions were drawn from the behaviour of specimens with flexible and strong columns (FS). In this case, significant bending of the column occurred, inducing large rotations at the beam – column connection. Severe damage occurred at the joint when the base motion was scaled to *pga* = 0.60 g, which was further increased when the ground motion was repeated. The damage at the beam – column joint caused a permanent dislocation of the beam with respect to the column; however, even in that case and in the following repetitions of the base motion, the shear force – joint slip diagrams followed the ones for cyclic response. Due to this dislocation, the ultimate resistance in the push direction could not be reached, because the achieved displacements in that direction were small. In the pull direction, the attained resistance was similar to that for cyclic loading.

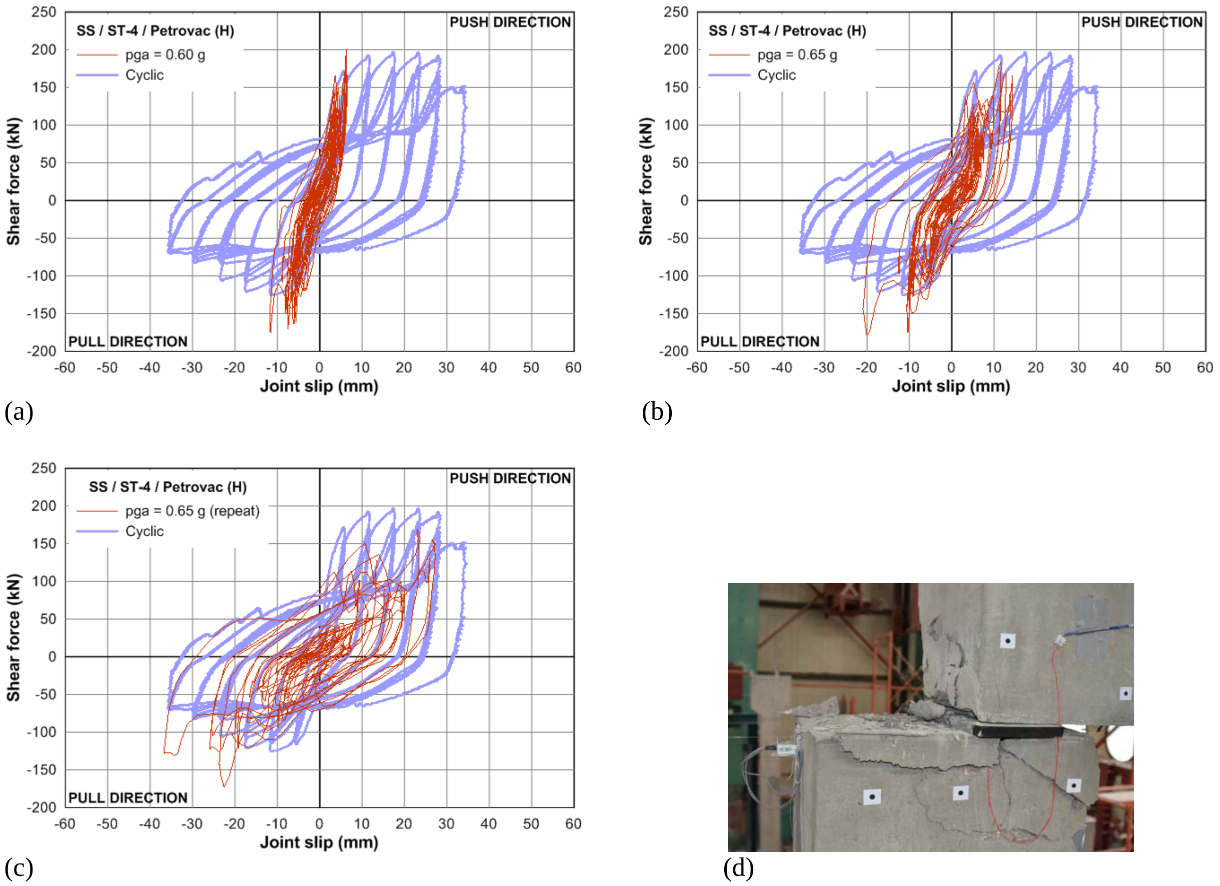


Figure 7. (a), (b), (c) Shear force versus joint slip diagrams for specimen with SS column subjected to Petrovac earthquake in the horizontal direction for stepwise increasing *pga* and comparison with the corresponding cyclic response; (d) Damage at the connection after repetition of base motion with *pga* = 0.65 g.

For the specimens with flexible and weak (FW) or flexible and normal (FN) columns, damage was concentrated at the base of the columns while the connections behaved almost elastically as the induced shear forces were less than their resistance.

4. DESIGN OF PINNED CONNECTIONS

The seismic design of precast structures with pinned beam – column connections is based on the capacity concept, according to which the prevailing energy dissipation mechanism should be through plastic rotations within critical regions of the columns while the connections remain in the elastic region. In Eurocode 8 such connections are termed “overdesigned connections”. Their design is based on the concept that the design shear force E_d which is applied at the connection is derived assuming that the ultimate flexural resistance has been developed at the base of the column. The ability of the connection to sustain the induced seismic loads is verified if the design force E_d is smaller than the shear resistance of the connection, R_d.

Estimation of the shear resistance of the connections is therefore required for seismic design. To this end, a new formula is proposed based on the experimental results for cyclic loading. These results are presented in normalized form in Figure 8 and show that the normalized resistance is practically equal to 1.10 for $d/D > 6$. The corresponding value according to Vintzeleou and Tassios (1987) is much less, equal to 0.65.

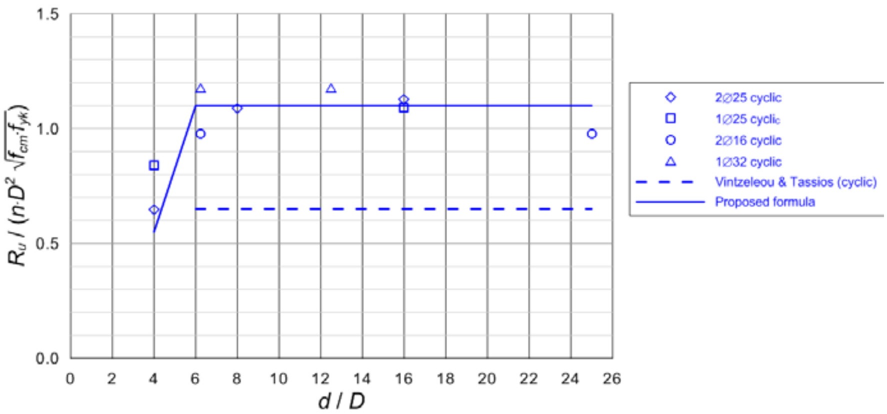


Figure 8. Normalized ultimate strength of pinned connections for cyclic loading (experimental results).

It is noted that the cyclic tests were performed for pure shear deformation of the connections, preventing any rotation at the joints. The dynamic shaking table tests and cyclic tests on frames performed at the University of Ljubljana within the SAFECAST project (SAFECAST 2012a) showed that the resistance of pinned connections might be reduced as much as 20% if large rotations occur at the joints. Based on these observations, the following formula is proposed for the estimation of the shear resistance of pinned connections to be used in the seismic design of precast structures:

$$R_d = \frac{C_0}{\bullet_R} \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}} \qquad \text{for } d/D > 6 \qquad (4.1)$$

$$R_d = \frac{C_0}{\bullet_R} \cdot (0.25d/D - 0.50) \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}} \qquad \text{for } 4 \le d/D \le 6 \qquad (4.2)$$

In the above equations, n is the number of the dowels, D is the dowels’ diameter, f_{cd} and f_{yd} are the design compression strength of the concrete and the design yield stress of the steel, respectively and $\bullet_R$ is a general safety factor with suggested value 1.30. For the coefficient C_0, values varying from 0.90 to 1.10 are suggested, depending on the magnitude of the expected joint rotations: for flexible columns, for which large joint rotations may occur, a value of C_0 about 0.90 to 0.95 is suggested, while for stiff columns and walls, for which small joint rotations are expected, this coefficient can be increased. The maximum value of C_0 is 1.10, for practically zero joint rotations. It is not recommended to use values of d/D less than 4.

5. CONCLUSIONS

The following conclusions were drawn from the experimental investigation on pinned beam – column connections under static monotonic and cyclic loading and for seismic excitation:

- The thickness of the cover concrete of the dowels in the direction of the loading plays an important role to the resistance of the connection. Values of $d/D > 6$ are recommended, since the shear resistance in the pull direction might be significantly reduced for smaller values of d/D. It is noted, however, that the reduction in the resistance in the pull direction is generally less for dynamic excitation than for cyclic loading.
- The resistance of the connection for cyclic response is less than one half the monotonic one. The response under dynamic loading is, in general, compatible with the cyclic response while the damage is generally less. However, repetition of the earthquake motion might increase the damage considerably.
- Significant values of shear ductility, about 4 to 6, can be achieved by pinned connections, provided that the cover of the dowels is sufficiently thick. This ductility capacity, however, is not utilized by the seismic design, which is based on the capacity concept and elastic response of the connections.
- Failure of the dowels does not necessarily imply loss of resistance, because broken dowels usually protrude inside the opposite element and resist the horizontal movement. However, disengagement of the dowels might occur for large rotations at the joint.
- The vertical component of the earthquake does not seem to be important.
- Use of high strength grout increases the resistance of the connections and improves their cyclic response by decreasing pinching and increasing ductility.
- Based on the experimental results, a new formula is proposed for the estimation of the shear resistance of pinned connections to be used in seismic design. It is noted though that this formula is valid for connections with similar geometric characteristics with the ones examined in this research.

AKCNOWLEDGEMENT

The research presented herein was conducted in the framework of the European FP7 project “SAFECAST: Performance of innovative mechanical connections in precast building structures under seismic conditions”, Research for SME Associations, Grant Agreement number 218417.

REFERENCES

European Committee for Standardization – CEN (2004). Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. *EN 1998-1*, Brussels.

SAFECAST (2012a). Experimental behaviour of existing connections, *Deliverable 2.1*, FP7 Project No. 218417.

SAFECAST (2012b). Quantification of the effects of dynamic loads, *Deliverable 2.2*, FP7 Project No. 218417.

Psycharis I.N. and Mouzakis H.P. (2012). Shear resistance of pinned connections of precast members to monotonic and cyclic loading. *Eng. Structures* (in press, http://dx.doi.org/10.1016/j.engstruct.2012.03.051).

Tsoukantas, S.G. and Tassios, T.P. (1989). Shear resistance of connections between reinforced concrete linear precast elements. *ACI Structural Journal*, **86 (3):** 242-249.

Vintzeleou E.N. and Tassios T.P. (1985). Mechanics of load transfer along interfaces in reinforced concrete, prediction of shear force vs. shear displacement curves. *Studie de Reserche*, No. 7, Corpo di Perfezionamento per le Construzioni in Cemento Amato, Italcementi Societa per Azioni (S. p:A), Bergamo, Italy, pp. 121-161.

Vintzeleou, E.N. and Tassios, T.P. (1987). Behavior of dowels under cyclic deformations. *ACI Structural Journal*, **84 (1):** 18-30.