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EXPERIMENTAL INVESTIGATIONS OF 1/3-SCALE R/C FRAME WITH INFILL WALLS BUILDING STRUCTURES

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SUMMARY

This paper presents experimental dynamic shaking table tests on 1/3-scale specimens of RC frame structures with infill walls under real earthquake excitations, with the aim of verifying the validity and applicability of the proposed retrofitting technique. Taking into account the similitude requirements, the artificial mass simulation has been applied as a method for model design of the specimens. Accordingly, the time scale and the intensity of earthquake records have been altered to ensure that the shaking table motion will produce the required inelastic behavior of the model. The first test specimen was a pure 3D R/C frame structure with infill walls. The second specimen had additional CFRP strips on the inner and outer faces of the infill walls applied using epoxy resin, for the purpose of demonstrating the proposed retrofitting technique. Longitudinal deformations, lateral deformations, shear deformations of the infill walls and displacements at the column bases were measured. Accelerometers were mounted to measure the accelerations in both longitudinal and lateral directions. The general conclusion is that the CFRP strips put on the infill walls can significantly improve the RC frame behavior under strong seismic excitation.

INTRODUCTION

The research reported herein has been carried out within the NATO Science for Peace Project SfP 977231 "Seismic Assessment and Rehabilitation of Existing Buildings". It represents a common project realized by the Department of Civil Engineering, Middle East Technical University, Ankara, Turkey in cooperation with The Institute of Earthquake Engineering and Engineering Seismology-IZIIS, University "Ss. Cyril and Methodius", Skopje, Republic of Macedonia, Department of Civil Engineering, University of Texas at Austin, USA, Center of Research and Technology-CERTH, Thessaloniki, Greece, and FORTH/ICE-HT, Patras, Greece.

The initiative for this project aroused from the severe damage that resulted from the last major earthquakes in Turkey. The experience has shown that inadequate lateral stiffness and unsuitable design and construction often lead to substantial damage when the structure is subjected to high lateral loads. Hence

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proper repair and strengthening of such buildings, as well as seismic retrofit is required in order to avoid catastrophic losses and have safer and stronger buildings.

Generally, repair and strengthening of RC frames damaged by earthquakes, as well as seismic retrofit of existing undamaged RC frames can be realized through *system* and *member* behavior improvement. The system behavior improvement approach involves a new lateral load resisting system, which will increase the lateral strength and stiffness of the existing system. One of the most widely used techniques based on this principle is construction of RC walls by infilling some bays of the existing frames. Repair and strengthening or seismic retrofit of some reinforced concrete elements can be performed by epoxy injection, by jacketing or fiber reinforced plastics (FRP) or carbon fiber (CF) wrapping.

When constructing a new RC wall in an existing building the occupants need to be evacuated, which makes this rehabilitation technique complicated and not very practical. Therefore the research task has been focused on developing simple methodologies and practical and economical rehabilitation techniques for seismically vulnerable buildings without disturbing the occupants. Within this paper, the experiments performed to demonstrate the proposed strengthening/retrofit technique by using Carbon Fiber Reinforced Polymers (CFRP) for increasing the capacity and ductility of the masonry infill, are discussed.

DESCRIPTION OF THE SHAKING TABLE AND THE TESTING EQUIPMENT

The two 1/3 scale models (2 story R/C frame specimens) of a 4 story R/C frame building prototype have been designed, constructed and tested on the shaking-table at the IZIIS Laboratory, Skopje, Macedonia. The basic idea was that some of the common irregularities and deficiencies detected in these types of buildings during past earthquakes in Turkey be taken into account during the shaking-table tests. Of course, model simulation of consequences from these deficiencies (poor lateral strength, inadequate lap splice length, unconfined joints, inadequately confined member end zones, poor detailing of the ties, improper detailing of the beam bottom reinforcement, poor concrete quality, see [1]) i.e. simulation of the real behavior of the structures, is generally a very difficult task, especially because of the strict similitude requirements and size effects arising in these RC non-linear models. Also, the dynamic conditions have made the simulation more difficult.

The shaking table on which the structural models were installed in order to be subjected to a biaxial earthquake motion is a pre-stressed reinforced concrete plate 5.0x5.0 m in plan (see [2]). Four vertical hydraulic actuators located at four corners, at a distance of 3.5 m in both orthogonal directions, with total force capacity of 888 kN, support the table. The total weight of the shaking table is 330 kN. The natural frequency of the shaking table is 48 Hz for maximum loading mass placed in the center of the table. The maximum applied accelerations are: vertical 0.50g and horizontal 0.70g with maximum displacement in vertical direction ±0.050m and in horizontal direction ±0.125m. The frequency range is 0-80 Hz. In order to provide the required power of the actuators three inter-connected hydraulic pumps with maximum flow of 1.250 l/min and a maximum pressure of 350x105 Pa are used. The gravity load due to the table and the model mass is sustained by a special system, located in the lower part of each of the four vertical actuators, with static supports which utilize nitrogen. The total bearing capacity for static loads is 720 kN. The horizontal and vertical actuators of the table are supported by reinforced concrete rigid structure with a total mass of 12.000 kN. The shaking system controls five degrees of freedom of the table, two translations and three rotations. The analog control system controls displacements, velocity, differential pressure and acceleration of the six actuators. Reverse control is provided by three-variable servo control system, which is capable of controlling displacements, velocities and acceleration simultaneously. A complete package of computer programs for control and acquisition is also used.

The disposition of the testing instruments is shown in Fig. 1.

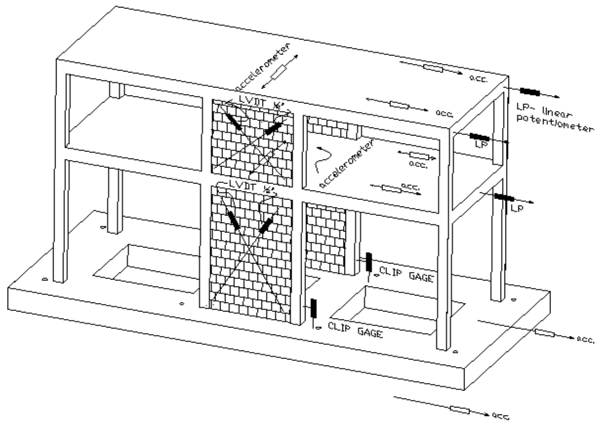

Fig. 1. View of the instrumentation

Three LPs (Linear Potentiometers) were used to measure longitudinal displacements. Two LPs were placed on the second floor slab of the frame model, in each corner, and the third LP on the first floor slab in the transversal mid span. Eight LVDTs (Linear Variable Displacement Transducer) for measuring shear deformations of the infill walls and two clip gages for measuring the curvature of the column bases were used. Eight accelerometers were utilized in the dynamic tests: an accelerometer was placed directly on the shaking table to check the input motion, another accelerometer was placed on the foundation plate to see whether slip motion between the shaking table and the test specimen exists, that is whether the test specimen was ideally fixed, three accelerometers were mounted on each floor slab-two for longitudinal motion and one for transversal motion. The intention was to check whether any torsion appears. Twenty-eight steel reinforcement strain gages were placed (sixteen on the reinforcement of the column bases, two in each column base and twelve in the longitudinal beams-mid spans and on the beam-column joints).

MODEL DESIGN OF SPECIMENS 1 AND 2

The design of the specimens for shaking table test poses a problem due to the difficulty of satisfying all the similitude requirements for the structural models. In particular, it is difficult to find suitable substitutes for construction materials that satisfy similitude requirements in the inelastic range of behavior. In most cases prototype materials must be used in the construction of the model and ballast is added to increase the density of the model to satisfy similitude requirements for the materials. Therefore most models used in shaking table experiments are not true replicas but are adequate for predicting the prototype behavior provided that the requirements for geometrical similitude and earthquake scaling are satisfied. As the scale of the model is reduced, its fabrication becomes more difficult. Similarity between a prototype structure and a small-scale model is maintained by proper scaling of significant physical quantities that govern structural behavior. The design of a model is initiated by performing a dimensional analysis. In a dimensional analysis, the first step is to identify the significant variables that affect the structure presented in Table 1.

Table 1. Model Similitude requirments

Scaling Parameters	Model Type Scale Factors		
	True Replica Models	Artificial Mass Simulation	Gravity Forces Neglected
Length, l_r	l_r	l_r	l_r
Time, t_r	$l_r^{1/2}$	$l_r^{1/2}$	$l_r(E/\rho)_r^{1/2}$
Frequency, f_r	$l_r^{-1/2}$	$l_r^{-1/2}$	$l_r^{-1}(E/\rho)_r^{1/2}$
Velocity, v_r	$l_r^{1/2}$	$l_r^{1/2}$	$(E/\rho)_r^{1/2}$
Acceleration, a_r	1	1	$l_r^{-1}(E/\rho)_r$
Strain, ε_r	1	1	1
Stress, σ_r	E_r	E_r	E_r
Modulus of Elasticity, E_r	E_r	E_r	E_r
Displacement, δ_r	l_r	l_r	l_r

The selection and production of materials is probably the most difficult step in a successful research investigation by using models. Exact duplication of prototype material properties is required if the model is to simulate elastic and nonlinear, inelastic behavior of structural system up to failure. Since the mass characteristics of the model are an important parameter, using artificial mass simulation can minimize the distortion of the mass density. In artificial mass simulation, mass is added to the structure in such a manner that will not appreciably change its stiffness. Care must be taken to mount adequately the additional mass without affecting the structural system. The added mass should be distributed over locations where it would normally act. The proper distribution is often difficult as geometrical restriction often dictates where the additional mass can be placed. For this project, the artificial mass simulation type was used following the project idea to use original prototype structure's materials (Fig.2).

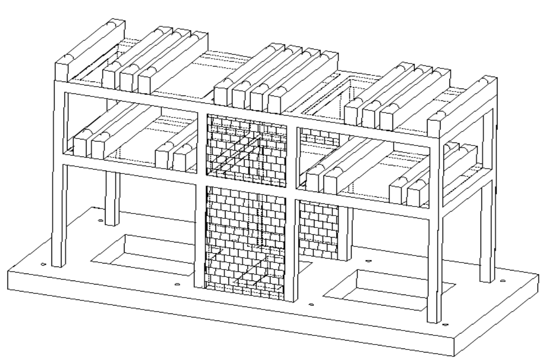

Fig. 2. 3D view of the applied additional mass to the model

The dimensions of the test specimen in longitudinal direction were: 1600 mm width of the side bays, 1000 mm width of the middle bay; the height of the first story was 1500 mm and the second story 1000 mm. In transversal direction, the frames were 1400 mm apart. The columns were 110x110 mm and beams were

110x150 mm in size. The model was resting on a 250 mm thick foundation plate with 5000x2200 mm in plane.

In the beams and columns 4ϕ8 mm plain bars were used as longitudinal reinforcement. The beam top reinforcement was extended into the exterior column and bent 90° downward and the bottom reinforcement were extended into the external column and its ends were hooked inward. The middle bay of the frame had 9 stirrups and each side bay had 15 stirrups. The length of transversal beam was 1400 mm with 4ϕ8 mm longitudinal reinforcement and 13 stirrups. The column reinforcement was spliced at floor levels with splice length of 20ϕ (160 mm). The longitudinal reinforcement of all the columns was spliced at the foundation level. Four dowels (4ϕ8 mm) were cast with the foundation and had a length of 40ϕ (320 mm). Each column had 4ϕ8 mm longitudinal plain reinforcing bars and 14 stirrups with ϕ4 mm. The stirrup spacing was 100 mm for both columns and beams. The stirrup ends had 90° hooks. The slabs (50 mm thick) were reinforced with ϕ6/100 mm top and bottom reinforcement in both directions. The reinforcing details of the frames are presented in Figures 3 and 4. In Fig. 5 a display of the both specimens 1 and 2 during their construction is given.

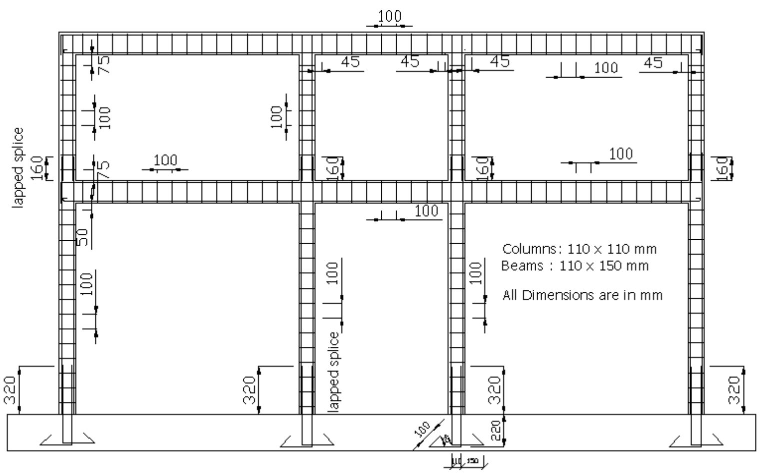

Fig. 3. Reinforcing details of the longitudinal frame

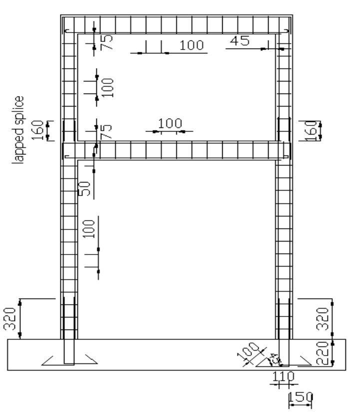

Fig. 4. Reinforcing details of the transversal frame

Fig. 5. Display of the test specimens during construction

The model specimens have been subjected to the Marmara region earthquake in Izmit (August 17, 1999, E-W direction) applied in the longitudinal direction of the model (Fig.6). Because of the similitude requirements, the original record consisting of 6000 points with time step of 0.005sec has been changed to the time step increment of 0.00288 sec. So, the total span of the earthquake applied on the shaking table was 17.32 sec. The acceleration values of the record were not scaled.

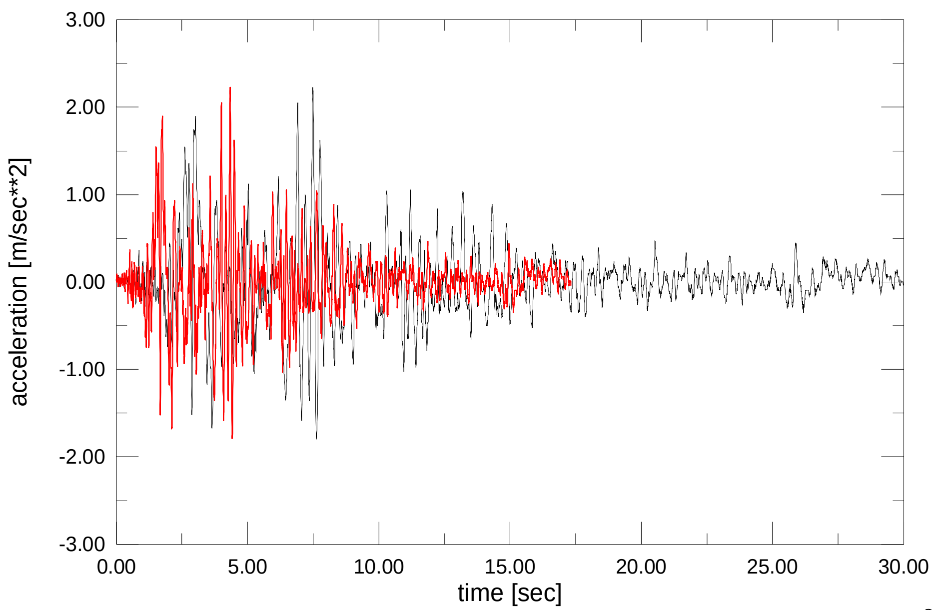

Fig. 6 Original and altered Izmit E-W earthquake record (a_{max}=2.151 m/sec^2)

TEST PROCEDURES

Following the curing period of the concrete, Specimen 1 was placed on the shaking table without infill walls. The first step was to load the model with the ballast (additional mass, see Fig. 2) and excite it with a small shaking table placed on the top slab, in order to measure the mode shapes and frequencies. Checking of the mode shapes has also been performed by using the ambient vibration technique. The same experimental steps were undertaken after construction of the infill walls, to measure the dynamic characteristics of the integral model. The comparison of the natural frequencies has been conducted to establish the influence of the infill walls on the model dynamic characteristics.

The next step was to undertake the shaking table tests in the following order: first a low-level random vibration time history was applied to measure the linear behavior of the model. Following, the scaled Izmit earthquake record (with a_{max}=0.023g) was applied to the test model. Afterwards, input motions were applied on the model with peak ground acceleration up to 0.45g. After each earthquake excitation, low-level random vibration tests were undertaken in order to monitor the change of the natural frequencies due to modal material deterioration.

A similar procedure was performed for Specimen 2. For this case, Carbon Fiber Reinforced Polymer (CFRP) strips were used to strengthen the masonry infill constructed in the middle bay of both stories. The hollow clay tile used for both specimens 1 and 2 was to a scale of 1/3. Specimen 2 was strengthened according to the results and conclusions drawn from the previous studies carried out in the METU

Structural Engineering Laboratory [1]. The most important conclusions from these investigations were that: a) In order to increase the lateral strength, CFRP should be extended and anchored to frame members; b) It is not needed to cover the whole brick panel with CFRP, since diagonally placed CFRP strips are as effective as covering the whole bay; c) A firm anchorage should be provided between CFRP and the panel; d) For the lap splice problems (as in our case for Specimen 1, where brittle failure occurred), that portion of the frame should also be covered by CFRP strips; and e) Anchor dowels should be used to connect CFRP to the infill and to the frame members.

All foregoing mentioned requirements for proper retrofit (or strengthening) of infill wall members have been applied for construction of Specimen 2. The steps of the incorporation and some details of these CFRP strips are presented in Figs. 7, 8, 9, 10, and 11.

Fig. 7 Specimen 2: step 1(left) - anchorage holes are drilled; step 2(right) –anchor dowels are put

Fig. 8 Specimen 2: Completed model with added carbon fibers

Fig. 9 Specimen 2: Some details of CFRP incorporation

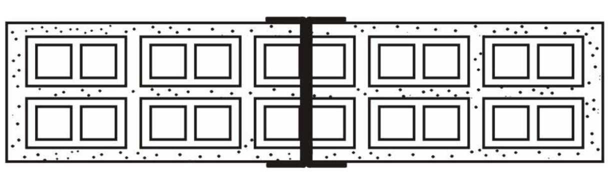

Fig. 10: Application of anchor dowels to infill

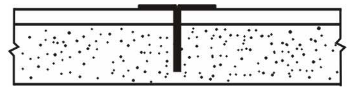

Fig. 11: Application of anchor dowels to frame members

The used CFRP was composed of an epoxy-based matrix and carbon fibers. The material characteristics of the CFRP strips were the following: a) It's a high strength carbon with packaging rolls: width =0.5 m, length=100 and surface = 50 m^2; b) Superficial density, D_s=0.3 kg/m^2; c) Density of fibre ρ=1820 kg/m^3; d) Effective thickness s=0.165 mm, e) Effective area per unit width, A^{eff}= 0.165 mm^2/mm; f) Tensile strength f_{tk}=3420 MPa; g) Tensile strength per unit width, f_{tkl}=565 N/mm; h) Tensile modulus of elasticity E_{tk}=230 GPa; i) Ultimate deformation, e_u=1.5 %; j) Coefficient of thermal expansion, $\alpha=10^{-7}$ 1/K; k) Thermal conductivity, J=17 1/m.s.K; l) Electrical resistivity, W=1.6 x 10^{-5}.

The material characteristics (for concrete and steel) for both specimens were the same (Table 2):

Table 2. Material characteristics for Specimens 1 and 2

Material	Modulus of Elasticity E (kPa)	Poisson ratio v	Density ρ (t/m^3)
Concrete 15MPa	2.55 x 10^7	0.2	2.5
Concrete 30MPa	3.15 x 10^7	0.2	2.5
Brick	0.70 x 10^7	0.2	1.8
Steel 400 MPa	2.1 x 10^8	0.16	7.85

OBTAINED RESULTS FROM THE EXPERIMENTAL TESTS OF SPECIMENS 1 AND 2 AND DISCUSSION

The obtained response from the shaking-table tests of Specimens 1 and 2 subjected to simulated Izmir earthquake, is summarized in Table 3.

Table 3. Selected results from shaking-table tests

Specimen 1 (not retrofitted)					
Span	Max input acceleration	1^{st} floor		2^{nd} floor	
		abs acceler.	rel. displac.	abs acceler.	rel. displac.
035	0.023g	0.028g	0.006 m	0.045g	0.0059 m
090	0.059g	0.078g	0.0175 m	0.122g	0.0175 m
180	0.112g	0.138g	0.0326 m	0.176g	0.0328 m
360	0.201g	0.265g	0.069 m	0.325g	0.071 m
500	0.261g	0.33g	0.105 m	0.396g	0.106 m
750	0.45g	0.39g	0.163 m	0.42g	0.168 m
Specimen 2 (retrofitted)					
Span	Max input acceleration	1^{st} floor		2^{nd} floor	
		abs acceler.	rel. displac.	abs acceler.	rel. displac.
035	0.032g	0.022g	0.0085 m		0.0083 m
070	0.052g	0.039g	0.0168 m		0.0167 m
600	0.41g	0.277g	0.0146 m	0.47g	0.0147 m
601	0.423g	0.74g	0.0113 m	0.42g	0.087 m

As mentioned previously, simulation of the behavior of the 4-story frame structures during the Izmir earthquake taking into account all phenomena and deficiencies of the design and construction, is generally very difficult. During the experimental test of Specimen 1, it was possible first to observe the shear-slip failure of the infill wall due to deterioration of the bond between the columns/beams and the masonry, since these contact zones were simulated as they really were in the real structure (poor mortar connection without anchors), Fig. 12.

Fig. 12 Specimen 1: Shear-slip failure between the concrete elements and masonry infill wall due to poor connection

Then, the non-ductile failure of the column basis (shear-slip failure and pull-out of the longitudinal reinforcement) has been observed, and consequently formation of plastic hinges on the bottom of the columns (Fig. 13). This prevented collapse of the upper part of the structures, although concrete crushing (Fig. 14) has been observed at some beam-column connections.

Fig. 13 Specimen 1: Brittle shear-slip failure between the columns and foundation due to inadequate stirrup lap-splice-length

Fig. 14 Specimen 1: Crushing and spalling of concrete

It is important to note that the formation of the plastic hinges at the bottom of the columns led to shifting of the basic natural period of vibration of the structure (as it could be seen from the random vibration tests with low magnitude preformed after each loading), so that the structure practically behaved as base-isolated. Although considerable deformations have been measured (maximum displacement of 16.3 cm), the structure did not collapse and any serious damage to the structural elements has not been observed. However, although the experiment did not simulate the spectacular pancake-type failure (as it really happened in the real structure during the Izmit earthquake), the fact is that several important deficiencies have been simulated (the lack of lap splice length of the longitudinal reinforcement from foundations, as well as the poor quality of the concrete, etc.) which resulted in the overall poor behavior of the structure. Anyway, having in mind the difficulties arising during the design and construction of the non-linear shaking-table models (lack of possibilities for scaling and modeling of constitutive relations for materials, size-effect in concrete especially related to fracturing and bond-deteriorating processes), the basic objectives for simulation of a real 4-story building with 2-story 1/3 model have been achieved. The experiments have shown that the proposed retrofit technique by using Carbon Fiber Reinforced Polymers (CFRP) can improve the behavior of this kind of structures, increasing their capacity and ductility, as can be seen from the obtained results in Table 3 for Specimen 2.

CONCLUSIONS

From the performed experimental investigations the following conclusions can be drawn:

1. Two specimens of two-story spatial frame system with 1/3 scale have been constructed: the first one to simulate the behavior of the real four-story frame structures with masonry infill walls, collapsed during the 1999 Izmit earthquake, and the second one retrofitted by CFRP strips to show how this retrofitting technique can significantly improve the seismic response of this kind of structural systems.
2. Shaking-table tests have been performed in order to investigate the influence of the dynamic conditions and real time-history input accelerations (1999 Izmit earthquake) on structural behavior. Although the model design process for this kind of problems can be very uncertain and difficult, having in mind the complexity of the conditions (nonlinear materials, dynamic conditions, complex structural system, etc.), using basic similitude requirements with artificial mass simulation method, rational and logical models

have been designed and constructed, which have successfully served for the research purposes and objectives.

3. The experiments have shown that using the CFRP strip technique for retrofit of these kind of structural systems, the overall behavior under seismic excitation can significantly be improved, as can be seen from the results of these investigations, presented in Table 3, i.e., the maximum displacements dropped from 16.8 cm (non-retrofitted specimen) to 8.7 cm (retrofitted specimen) for the strongest applied earthquake record (0.45g and 0.423g input maximum peak acceleration).
4. From the test of Specimen 1, brittle shear-slip failure at the contact between columns and foundations has been observed due to inadequate lap splice length of the longitudinal reinforcement in columns, which was one of the phenomena observed on real 4-story frame buildings with brick infill walls during the 1999 Izmit earthquake. This deficiency (together with the poor connection between the infill and RC members) has been overcome by applying the proposed retrofitting method, demonstrated on Specimen 2, using CFRP strips and anchor dowels, as shown in Figs. 8, 9, 10 and 11. Generally, significant improvement has been done regarding the member's behavior, as well as the overall structural behavior in terms of increased structural and member's capacity and ductility, as well as lateral strength.
5. As compared to other retrofit and strengthening methods (for example incorporation of a new RC infill), CFRP has proved to be applicable more easily and more rapidly, and also it does not require evacuation of the occupants. Of course, to choose the most appropriate and most optimal retrofitting or strengthening technique, the cost of application must be considered for each separate case and structural system.

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