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SHAKE TABLE TESTS ON FRAMES MADE WITH NORMAL AND FRP-CONFINED RUBBERISED CONCRETE

Imad EL KHOURI[1], Reyes GARCIA[2], Petru MIHAI[3], Mihai BUDESCU[4], Nicolae TARANU[5], Ionut Ovidiu TOMA[6], Maurizio GUADAGNINI[7], David ESCOLANO MARGARIT[8], Ioana SORINA ENTUC[9], Gabriel OPRISAN[10], Iman HAJIRASOULIHA[11], Kypros PILAKOUTAS[12]

ABSTRACT

This paper investigates the behaviour of two one-bay one-storey regular reinforced concrete frame buildings tested on a shake table. The 1/3-scale buildings were specifically designed to have “short columns”. A control frame cast with conventional reinforced concrete (RC) was initially subjected to a set of earthquakes using increasing Peak Ground Acceleration (PGA) levels and failed abruptly at 1.6g. To assess the effectiveness of fibre reinforced polymer (FRP) confined rubberised reinforced concrete (CRuRC) at improving structural deformability, a second frame cast with CRuC was also tested up to a PGA of 1.81g when the limits of the shake-table were reached. The results indicate that, compared to its RC counterpart, the FRP CRuC building attained three times more drift at the same level of excitation. The results demonstrate that confined rubberized concrete can help in improving the deformability and seismic capacity of a reinforced concrete structure.

Keywords: Shake-table test, Confined rubberised concrete, Drift, Short columns

1. INTRODUCTION

1.1 Background to the problem

Population increase leads to increased demand for housing and expansion of urban centres, increasing exposure and the risk of damage and fatalities during seismic events. Conventional reinforced concrete (RC) has a relatively high material stiffness which can be undesirable during earthquakes as the stiffness of structural elements is proportional to the actions they attract. Hence, more ductile structures are needed to dissipate energy and control deflections to avoid non-structural damage. Eurocode 8 (European Committee for Standardization 2004) takes this into consideration and aims to design for high energy dissipation capacity and overall ductile behaviour that spreads widely to different elements of all storeys.

Devices like viscous fluid dampers and friction dampers enhance energy dissipation, however they are fairly expensive and complicated to setup, and require maintenance and monitoring. A candidate for increasing ductility in reinforced concrete members is rubberised concrete (RuC), where scrap and crumb rubber particles, derived from tyre shredding, replace a specific ratio of the aggregates in the mix.

[1] PhD, University of Sheffield, Sheffield, United Kingdom, imadelkhouri@gmail.com
[2] Post Doc researcher, University of Sheffield, Sheffield, United Kingdom, r.garcia@sheffield.ac.uk
[3] Associate Professor, Gheorghe Asachi Technical University of Iași, petru.mihai@tuiasi.ro
[4] Professor, Gheorghe Asachi Technical University of Iași, mbudescu@ce.tuiasi.ro
[5] Professor, Gheorghe Asachi Technical University of Iași, taranu@tuiasi.ro
[6] Professor, Gheorghe Asachi Technical University of Iași, iotoma@tuiasi.ro
[7] Associate Professor, University of Sheffield, United Kingdom, m.guadagnini@sheffield.ac.uk
[8] Post Doc researcher, University of Sheffield, United Kingdom, d.escolano@sheffield.ac.uk
[9] Professor, Gheorghe Asachi Technical University of Iași, entuc@ce.tuiasi.ro
[10] Professor, Gheorghe Asachi Technical University of Iași, oprisan@ce.tuiasi.ro
[11] Associate Professor, University of Sheffield, United Kingdom, i.hajirasouliha@sheffield.ac.uk
[12] Professor, University of Sheffield, Sheffield, United Kingdom, k.pilakoutas@sheffield.ac.uk

The EU Waste Framework Directive discourages the incineration of tyres as a disposal method, thus it is desired that as many tyres and/or their constituents are re-used in preferably high value applications because of their excellent intrinsic physical properties. In January 2014, the Anagennisi project -funded under the 7th Framework Programme of the European Commission- kicked off with the aim to develop solutions to reuse all tyre components in innovative concrete applications. One of the main hypothesis of Anagennisi is that rubber crumb can partially replace aggregate to lead to a more deformable concrete, owing to the relative low elastic modulus of the rubber.

1.2 Scientific background

Compared to conventional concrete (CC), RuC has been reported to have larger deformation capacity (Hooton et al. 1998) and higher vibration damping (Liu et al. 2013; Najim and Hall 2012; Xue and Shinozuka 2013). However, RuC has lower compressive strength, tensile strength and stiffness than CC. The compressive strength of RuC can be up to 90% lower than that of CC at 100% rubber content (Batayneh et al. 2008; Khatib and Bayomy 1999; Sukontasukkul and Chaikaew 2006; Toutanji 1996).

To recover the strength and exploit the potential deformation capacity that RuC can offer, recent research has examined the use of Fibre Reinforced Polymer (FRP) confinement to produce FRP confined rubberised concrete (CRuC). Li et al. (2011) tested CRuC cylinders cast in prefabricated Glass FRP (GFRP) pipes (Li et al. 2011). Relatively low aggregate replacement volumes were used for fine (15% and 30% of crumb rubber) and coarse (15% of fibre rubber) aggregates. Whilst GFRP CRuC was up to 5.25 times stronger than unconfined RuC specimens, relatively low compressive strengths of 16.3 to 22.9 MPa were achieved. Moreover, the ultimate axial strain at failure of the CRuC cylinders was only 1.2%, which is similar to the ultimate strain of conventional FRP confined concrete. Youssf et al. (2014) tested CRuC cylinders cast in preformed Carbon FRP (CFRP) tubes (Youssf et al. 2014). Low fine aggregate replacement volumes of 0%, 5%, 10% and 20% of crumb tyre rubber were used in the mixes. The compressive strength of CFRP CRuC cylinders ranged from 61.7 MPa (for 1 CFRP layer) and up to 112.5 MPa (for 3 CFRP layers), thus being suitable for structural applications. However, the stress-strain behaviour of the CFRP CRuC cylinders and CFRP-confined conventional concrete cylinders was similar, and therefore the deformability potential of rubber was not realised. More recently, Duarte et al. (2016) tested "short" RuC columns confined with cold formed steel tubes (Duarte et al. 2016). In this case, only the coarse aggregate was replaced with rubber (5% or 15% volume replacement). Whilst the columns' ductility was increased by up to 50%, the use of steel tubes promoted local buckling and limited the capacity of the specimens.

Based on these results, it is evident that the use of small volumes of rubber aggregate replacement (below 20-30%) is insufficient to achieve a highly deformable CRuC that can be used in structural applications. However, whilst large volumes of aggregate replacement are expected to promote a better use of the properties of rubber, several technological challenges needed to be solved to mitigate the negative effect of rubber on the fresh and hardened properties of CRuC. As part of Anagennisi, Raffoul et. al. (2016, 2017) have addressed this challenge and produced a CRuC suitable for structural applications that can be used to produce concrete achieving up to 8% strain at failure (Raffoul et al. 2016, 2017). Figure 1 shows the stress strain behaviour of confined and unconfined RuC cylinders under uniaxial compression.

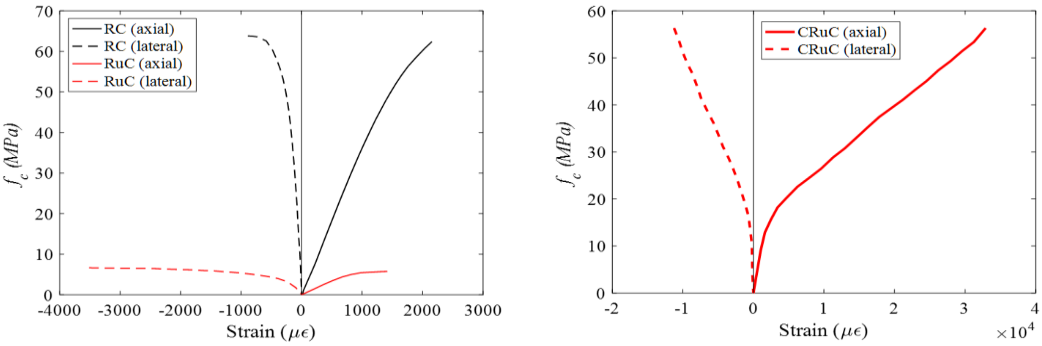

Figure 1: Stress-strain behavior (left) Unconfined RuC and (Right) Confined RuC (Raffoul et al. 2017)

1.3 Structural element testing

Several researchers tested rubberised reinforced concrete (RuRC) with various rubber contents.

- Son et al. (2011) explored RuRC rectangular columns with 15% rubber replacement by volume of fine aggregates under pure axial compressive load. The compressive load-carrying capacity of the specimens decreased with increasing the rubber content. However, what was notable is that the RuRC columns were able to undergo twice the lateral deformations as the regular concrete columns. This provided higher energy dissipation capacity and ductility (Son et al. 2011).

- Youssf et al. (2015) investigated CFRP RuRC circular columns under axial and reverse cyclic loading. The rubber replacement by volume of fine aggregates was 20%. Damping, snap-back tests, and finally a cyclic test until failure was performed on three columns; two conventional concrete and one RuRC. The RuRC concrete column increased the hysteric damping ratio by 13% and the energy dissipation by 150% as compared to the RC specimens. The ultimate lateral strength and deformability of the 3 columns was similar, but that is attributed to a relatively low rubber percent replacement (Youssf et al. 2015).

- Ismail and Hassan (2016) investigated the performance of 12 self-consolidating RuRC beams in flexure. The rubber replacement by volume of fine aggregates varied from 0-50%. As expected the added rubber deteriorated the fresh and mechanical properties of the concrete, as well as decreased the flexural stiffness of the beams. It was reported that using up to 10% fine-rubber replacement improved the beam’s deformation capacity, ductility and toughness, without affecting the ultimate flexural load. Up to 20% the same pattern continued, however with a slight decrease in the ultimate flexural load. Anything more that 20% replacement had adverse effects on the moment capacity of the beam (Ismail and Hassan 2016).

- As part of the Anagennisi project, Elghazouli et al. (2017) looked at the behavior of circular reinforced concrete column specimens with conventional concrete, unconfined and confined rubberized concrete (CRuRC) under lateral cyclic loading, with sustained axial load. Both rubberized concrete specimen had a 60% rubber replacement by volume of fine and course aggregates, however only one column was confined with three layers of Aramid FRP (AFRP). The authors concluded that high rubber content enhanced the ductility of the columns, while maintaining an acceptable lateral bending resistance (Elghazouli et al. 2017).

The aforementioned experiments illustrate the potential of confined rubberised reinforced concrete in acting as a highly ductile structural element. However, only isolated structural components were tested, whereas full scale structures, as well as the pseudo-dynamic tests give an estimate to the capacity of structural elements, while shake-table tests are more suited to simulate real life seismic excitations and rate of loading. This papers discusses the results obtained in terms of natural frequencies, maximum drift in the short column, and observed damage.

2. EXPERIMENTAL PROGRAM

The Anagennisi project tested two reinforced concrete frames with 300 mm short columns, one made with conventional concrete, and the other with rubberised concrete with 60% replacement of each coarse and fine aggregates.

The frame was designed to have shear failure in the short column span, to simulate cases where a column is restrained along its height. The frames were tested using uniaxial shaking, with Peak Ground Accelerations (PGA) starting at 0.14g and incrementally increased depending on the response of the shake table and the strain measurement on the longitudinal rebar in the column.

2.1 Geometry of the building

2.1.1 Conventional reinforced concrete frame (RC)

The building was a one-bay one-storey frame, with a storey height of 1360 mm and a rectangular plan of 2550×1950 mm (see Figure 2). The slab was 60 mm thick, with a clear span of 2250×1650 mm. The columns had a cross-section of 150×150 mm, reinforced with four 14 mm bars as longitudinal reinforcement, and 4 mm stirrups spaced at 100 mm as shear reinforcement. The beams in the X and Y direction had a cross-section of 150×260 mm, with longitudinal reinforcement of four 12 mm and four 10 mm bars respectively, and 6 mm stirrups spaced at 100 mm in the shear spans and at 200 mm in the middle for both beams. The design flexural capacities of the columns and beams were 14.3 and 21.6 kNm, respectively. This gave a Column-to-Beam strength ratio of 0.66, which ensures the failure to occur in the area of focus which was the short column span. The column had an unfactored shear capacity of 30 kN; the required shear force to develop the flexural moment over the short column span is 61.5 kN, thus still ensuring a shear failure of the short column. The slab was reinforced with 4 mm bars spaced at 100 mm in both directions.

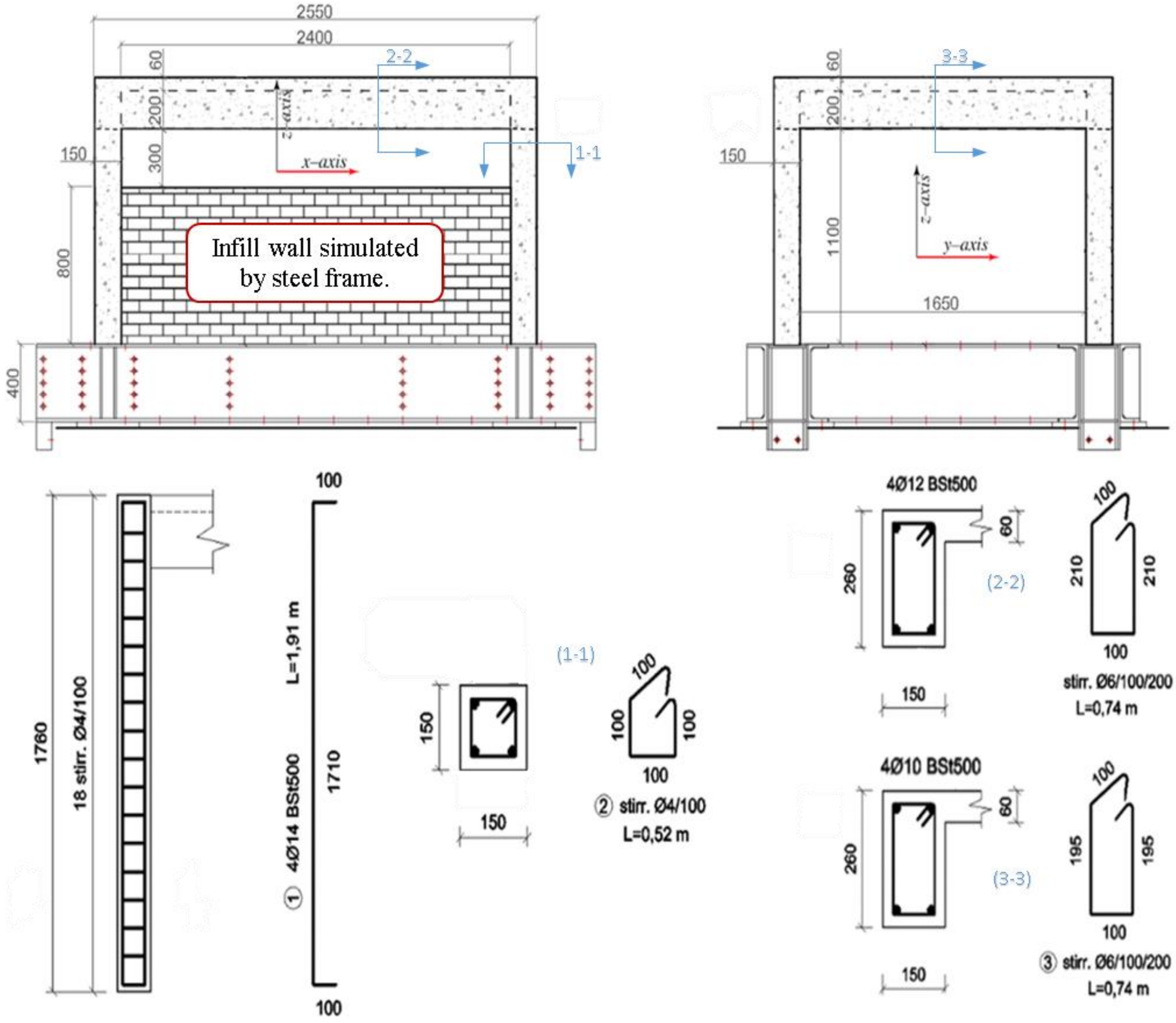

Figure 2: Geometry of the building

2.1.2 Confined rubberised reinforced concrete frame (CRuRC)

The CRuRC frame had the same geometry and reinforcement as the one of the conventional concrete, with the difference of the concrete being rubberised concrete with 60% replacement by volume of coarse and fine aggregates. The short column span was confined/wrapped with 3 layers of AFRP, and the beam column joint was confined with 2 layers of Carbon Fibre Reinforced Polymer, using resin as the binder in both cases. An extra 150 mm span of the column under the short column was also wrapped with three layers of AFRP to avoid crushing outside the area of interest. The edges of the column were rounded and smoothened for proper application and efficacy of the AFRP. For consistency, although no external confinement was applied, the columns' edges of the conventional concrete were also rounded. The design flexural capacities of the columns and beams were 13.8 and 20.7 kNm respectively. The column had an unfactored shear capacity of 283 kN. The same gravity load as in the conventional concrete frame was applied.

2.2 Material Properties

2.2.1 Conventional reinforced concrete frame

The concrete compressive strength and elastic modulus determined from compressive tests on three standard cylinders (150×300 mm), were f_c=37 MPa and E_c=32 GPa. The properties of the longitudinal reinforcement used in the beams and columns were obtained from direct tensile testing. The average yield and ultimate strengths were f_y=513 MPa and f_u=626 MPa, respectively. The stirrups used, as well as the mesh in the slab, were Grade-250 mild steel reinforcement.

2.2.2 Confined rubberised reinforced concrete frame

The average concrete compressive strength and elastic modulus of the rubberized concrete (150×300 mm), were f_c=8.2 MPa and E_c=11.5 GPa, correspondingly. The reinforcement properties were the same as in the RC frame. The AFRP sheets had a thickness of 0.2 mm, with a 300 mm sheet width which allowed for a continuous roll around the column. The sheet's mean elastic modulus was 120 GPa, tensile strength of 2900 N/mm^2, and an elongation at rupture of 2.5%. The Carbon Fibre Reinforced Polymer had a thickness of 0.164 mm, mean elastic modulus was 252 GPa, tensile strength of 4900 N/mm^2, and an elongation at rupture of 2%.

2.3 Test set-up, instrumentation and ground motion record

The frame columns were restrained in the X direction, at 1100 mm from the ground, leaving a short column span of 300 mm. A prefabricated steel frame was fitted between the columns to simulate an infill wall (see Figure 3 e and d). Soaked wooden planks were placed at the contact points between the concrete and steel frame, to fill the gap so as to avoid local crushing from pounding. Gravity load was provided via 15 lead blocks bolted to the top of the slab, transferring a total of 4 tonnes, or 10 kN per column. The bare RC frame had an estimated self-weight of 1.6 tonnes, compared to 1.2 tonnes for the CRuC frame.

The conventional concrete frame was instrumented with three accelerometers in the X direction, two in the Y direction and one in the vertical direction. Eight displacement transducers fixed on the table platform, recorded the relative displacement in the X direction of the top of each column and at the level of the short column restraint. Three wire transducers, connected to a fixed outside frame, measured the absolute displacement of the shake table and the top of both sides of the frame. Four inclined LVDTs were used to measure the beam-column joint opening in the X direction. Sixteen strain gauges were fixed on the reinforcement; two on the outside longitudinal reinforcement of the short column and two on the inside ones, one on each of the three stirrups in the short column span, and one on the bottom longitudinal bar of the beam 50 mm away from the column. The same strain gauge distribution was replicated on the opposite column in the X direction. The rubber concrete frame had the same instrumentation with the addition of six strain gauges on the external AFRP, three on each side. The sampling frequency for the first test was set to 200 Hz, however it was increased to 500 Hz in the rubber concrete test.

Figure 3: (a) Reinforcement cage in place with strain gauges connected, (b) Strain gauges on the AFRP, (c) Accelerometers and inclined LVDT, (d) RC frame prior to testing, and (e) RuC frame prior to testing

The shake table input consisted of a unidirectional horizontal record with increased level of Peak Ground Accelerations (PGAs). The record was an artificial earthquake based on the Eurocode 8 (EC8) using soil type C spectrum (European Committee for Standardization 2004) (see Figure 4). The total duration of the record was 30 seconds. Every shaking test was preceded and followed by a white noise of 0.013g amplitude to assess the natural frequency of the structure. Later on during the data processing phase, the recorded accelerations from the main shaking tests provided a better means of finding the natural frequencies. It was noted from preliminary analytical stiffness calculations of the frames that the periods of interest were between 0.05 to 0.25 s (this inherent stiffness is because of the stiff nature of the short columns).

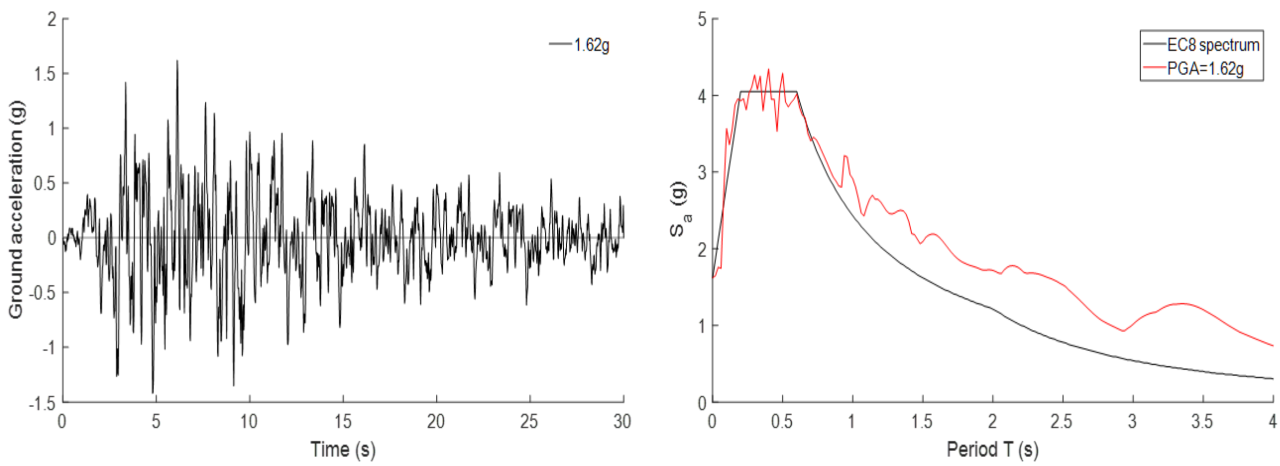

Figure 4: (left) Artificial input record, and (right) EC8 spectrum matching for damping=5%.

3. TEST RESULTS

3.1.1 Conventional Reinforced concrete frame

The structure was subjected to PGA levels ranging from 0.14g to 1.62g at which the structure failed in a brittle shear manner at the level of the short column. Table 1 presents the natural frequencies and period of the structure with each test, as well as the normalized period against the undamaged one. Some of the frequencies were obtained from the dynamic identification tests performed between tests.

Table 1: Dynamic characteristics of RC frame after each shake-table test.

PGA (g)	*f* (Hz)	*T* (s)	Normalised Period
0 (undamaged)	22.8	0.044	1.00
0.14	22.2	0.045	1.03
0.20	20.8	0.048	1.10
0.29	19.3	0.052	1.18
0.41	17.3	0.058	1.32
0.81	14.8	0.067	1.54
1.62 (failure)	12.1	0.083	1.88

The reduction in the frequency of the structure is a clear sign of damage accumulation. The structure however, being very stiff (≈110 kN/mm), took little damage during the first few tests. At 0.41 and 0.81g few hairline shear cracks began to propagate, decreasing the natural frequency by 24 and 35%, respectively. During the 1.62g test, the shear cracks opened and closed until cracks extended over the full strut zone and total collapse of the short column occurred. The natural frequency in the former test was obtained from windowing the response signal from start to up-to onset of failure, and it showed a drop of 47%. The residual stiffness at that stage during the test was around 28% of the undamaged stiffness of the structure.
Part of the column from the short-column restraint downward remained intact with no visible cracks. The beams in the Y direction showed no cracks or visible damage. However, the beams in the X direction showed flexural cracks in the middle of the span.

Figure 5: (left) Shear crack propagation, captured during the test, (right) Total collapse of the short column and spalling of concrete

3.1.2 Confined rubberised reinforced concrete frame

The structure was subjected to PGA levels ranging from 0.14g to 1.80g, with no signs of failure in the structure. The tests were stopped because the displacement limits of the shake-table were reached. Table 2 presents the natural frequencies with each test, the period of the structure, and the normalized period against the undamaged period. Some of the frequencies were obtained from the dynamic identification tests performed between tests.

Table 2: Dynamic characteristics of RuC frame after each shake-table test.

PGA (g)	f (Hz)	T (s)	Normalised Period
0 (undamaged)	8.8	0.113	1.00
0.14	8.8	0.114	1.00
0.29	8.2	0.122	1.08
0.61	8.1	0.124	1.10
0.72	7.9	0.127	1.12
0.81	7.2	0.139	1.23
1.14	6.6	0.152	1.34
1.36	5.9	0.169	1.49
1.62	5.0	0.200	1.76
1.81	4.3	0.231	2.04

The reduction in the natural frequency of the structure is evident, although not a lot of damage could be observed. Unlike the conventional concrete frame, the more gradual change in the structural period indicate a gradual damage with increasing PGAs. The natural frequency in the last test showed a drop of 51% over the undamaged frequency. The residual stiffness after that test was around 25% of the undamaged stiffness of the structure (≈16 kN/mm).

No damage was observed in the short columns, however it was clear, from the experimental evidence, that there was crushing of the rubber concrete inside the AFRP jacket. The maximum strain measured on the AFRP was 900 με, with no signs of sheet damage. During the 0.61g test, de-bonding of the Carbon Fibre Reinforced Polymer occurred on two diagonally opposing joints, however the edges remained intact and bonded to the concrete. This can be partly due to the extension of those sheets during the testing. The beams showed similar flexural cracks as the ones observed in the RC frame. During the PGA=0.81g test, all joint CFRP sheets de-bonded on the X direction of the beams.

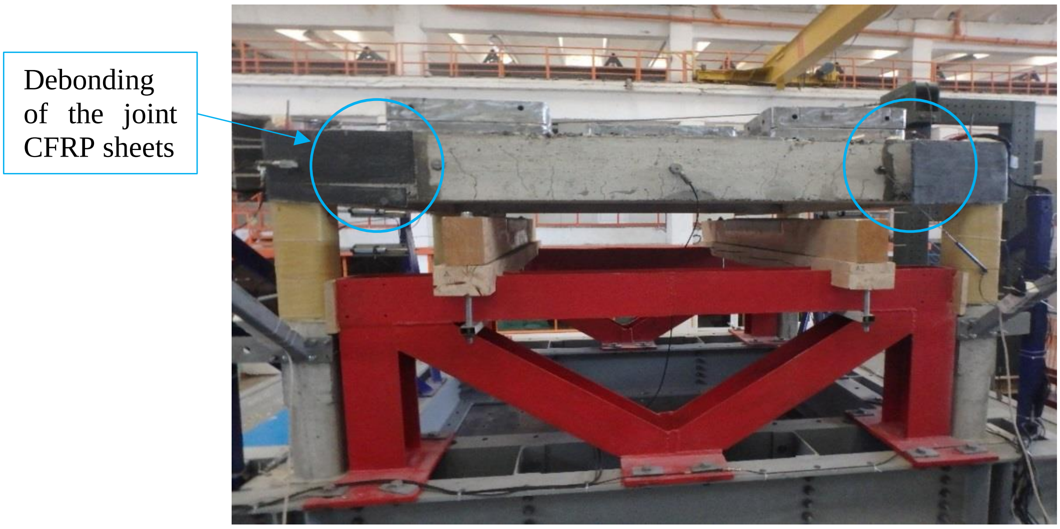

Figure 6: CRuRC frame after testing, showing the de-bonding of CFRP sheets and beam flexural cracks (X direction)

4. COMPARISON OF MAXIMUM DRIFT IN THE SHORT COLUMN

The short column drift indicates the effectiveness of confined rubberized concrete. Table 3 shows the maximum drift measured during the shake table tests of both frames, along with the drift ratio corresponding to the 300 mm shear span of the short column.

Table 3: Maximum drift & drift ratio.

PGA (g)	RC frame		CRuRC frame	
	(mm)	(%)	(mm)	(%)
0.14	0.14	0.05	0.54	0.18
0.29	0.31	0.10	0.82	0.27
0.81	0.76	0.25	1.99	0.66
1.62	5.8	1.93	4.40	1.47

The more flexible RuC allowed for longer drift in the short column with an average of 3 times more than the RC counterpart. The drift at 1.62g PGA test was estimated to be 5.8 mm right before collapse. Comparatively, during the PGA=1.62g test, the CRuC short columns underwent a transient 4.4 mm drift, and in the last test (PGA=1.81g) up to 7.5 mm drift (2.5%). The results indicate that the applied external confinement onto rubberised concrete is very effective in controlling the propagation of damage, as well as increasing the local ductility of structural elements.

5. CONCLUSIONS

This paper presented preliminary results from two shake-table tests. The effectiveness of Reinforced Confined Rubberised Concrete was investigated experimentally through seismic testing of a one-storey one-bay frame with 300 mm short columns made with CRuC. Initially, a benchmark frame was tested with the same geometry but made with conventional concrete. Based on the results of the study, the following conclusions are drawn:

1. The first few shake-table tests on the conventional concrete frame produced little damage on the structure due to its high stiffness. After the tests at PGA=0.41 and 0.81g, shear cracks developed and the natural frequency of the structure decreased noticeably, leading to a brittle failure during the 1.62g test. On the contrary, the frame with confined rubberised concrete short columns had a better distribution of damage (less frequency decay) between tests.

2. The results from the shake-table tests demonstrated that the CRuC was very effective at increasing the seismic capacity of the structure. Whilst the conventional concrete frame failed at PGA=1.62g, the confined rubberised concrete sustained a PGA=1.81g without visible damage.

3. Shear cracks developed in the short column of the conventional concrete frame at a drift ratio of 0.15 to 0.25%. No damage was observed in the CRuRC short column even at a 2.5% drift.

4. The confined rubberised concrete increased the deformation capacity of the short column by an average of 3 times more than the conventional concrete. The drift ratios are relatively small, due to the shear dominated deformation of the short column.

Whilst the above conclusions cannot be generalized to other types of shear dominated structures, CRuC is deemed as an attractive solution to increase deformation capacity of structural components. Consequently, future research should examine, experimentally and analytically, the use of CRuC in other types of elements.

6. ACKNOWLEDGEMENTS

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