Numerical and experimental study of seismic retrofitting for one-bay single-storey reinforced concrete (R/C) frames with an encased R/C panel

G. C. Manos, V. Soulis, K. Katakalos & G. Koidis
Laboratory of Strength of Materials and Structures, Department of Civil Engineering, Aristotle University of Thessaloniki, Greece

Abstract

The in-plane behaviour of one-bay single-storey reinforced concrete (R/C) frames, retrofitted by jacketing of their columns together with a cast-in-place encased R/C panel, was studied numerically when subjected to cyclic seismic-type horizontal loading. The influence of such an encased R/C panel is examined, when the panel is connected with the surrounding frame with or without steel ties. From the preliminary numerical results it is concluded that an encased panel results in a considerable increase of the stiffness and the bearing capacity of such a system, especially when steel ties are present at the interface. The placement of steel ties moderates the amplitude of the forces that are transferred at the narrow column-to-beam joint regions in a direction normal to the interface through the contact/gap mechanism, and consequently, mitigates the possibility of crushing the encased panel and/or parts of the columns or beams at these regions. Thus, it can be concluded that the encased R/C panel connected by the appropriate steel ties with the surrounding R/C frame, has a beneficial effect on the seismic behaviour of this type of structural system. Experiments were performed in order to quantify the behaviour of such steel tie connections at the interface under a stress field that is expected to develop at this part of the encasement during seismic type loading. Such measured behaviour is in agreement with the assumptions made concerning the behavioural characteristics of these steel ties in the preliminary numerical analysis
Keywords: retrofitting, reinforced concrete frames, encasement R/C panels, jacketing, soft ground floor.

WIT Transactions on Modelling and Simulation, Vol 55,
www.witpress.com, ISSN 1743-355X (on-line)
doi:10.2495/CMEM130331

1 Introduction

Many multi-storey reinforced concrete (R/C) structures, built in seismic regions, have their ground floor designed to function as a parking space. Therefore, the bays of the R/C frames at this level are left without masonry infills whereas all the stories above have their corresponding bays of the R/C frames infilled with external masonry walls or with internal masonry partitions. It was demonstrated by extensive past research that the dynamic and earthquake behaviour of such structures, having a relatively flexible (soft) ground floor and stiff upper stories, results in increased demands on the structural elements of the ground floor, due to the interaction of the masonry infills with the surrounding R/C frames. This, in turn, leads to structural damage, unless the structural R/C elements at the ground floor are properly designed [1–5]. As this behaviour was not well understood in the past, there are many structures with such a soft storey that were designed and constructed with their R/C structural elements now in need of upgrading their capacity.

The performance of a retrofitting scheme is studied here that can be easily applied to such a soft ground floor of multi-storey R/C structures [6–9]. A common retrofitting scheme usually consists of:

- R/C jacketing of the existing R/C columns at the ground floor level.
- R/C jacketing of the existing R/C beams at the ground floor level.

With the jacketing of the ground floor structural elements, a certain increase in their strength and ductility is expected to be achieved. The current Greek guidelines for retrofitting reinforced concrete buildings [7], also provides for the addition of an R/C panel that can be added as an encasement, filling the space between the jacketed columns and beams. In the relevant provisions of these guidelines [7] the designer is provided with a number of distinct choices. In the present investigation the following choices will be studied:

a) The encased R/C panel is not connected to the surrounding R/C structural elements within (columns or beam). Alternatively, a limited connection is described between the R/C panel and the upper/lower horizontal frame interface.

b) The encased R/C panel is constructed together with a connection with the surrounding R/C structural elements strengthened by jacketing within a bay (columns or beam), utilizing steel ties. In this case, the thickness of the encasing R/C panel is smaller than the width of the beams that form the encased bay.

2 Preliminary numerical simulation

The numerical study was limited to examining a single-storey one-bay R/C frame (figure 1) formed by two columns (left and right) and two beams (top and bottom). The over-all frame dimensions, chosen, arbitrarily, were: The length between the mid axes of the two columns equal to 6m. The height between the mid-axes of the top and bottom beams equal to 3m. The cross section of the columns 340mm x 340mm; that of the top beam 300mm x 600mm and that of the bottom beam with large flexural stiffness representing a rather stiff foundation. The numerical model included three non-linear springs located at all

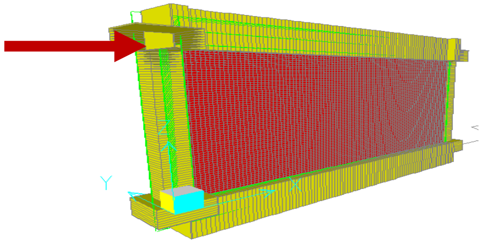


Figure 1: Single-storey one bay R/C frame with the encased R/C panel.

the joints between the columns and the beams either at the top and bottom of each column or at the left and right side of each beam, whereas the region representing the actual beam-column joint was assumed non-deformable. These non-linear springs were provided with tri-linear moment-rotation proper-ties thus representing the possibility of plastic hinges forming there. The properties of these moment-rotation springs were derived considering typical reinforcing details for the columns and beams for such structural elements at the ground floor; an axial force level for the columns equal to 510 KN, representing the axial force for a building with three more stories above the ground floor where the examined single-storey one bay R/C sub-assembly is assumed to be located.

In this way, only the flexural non-linear behaviour scenario, by the formation of the plastic hinges is considered, excluding any shear non-linear behaviour of the R/C structural members. In this preliminary numerical simulation, the behaviour of the encased R/C panel itself was considered to be elastic. A subsequent numerical simulation considered the possibility of non-linear behaviour of the encased R/C panel itself. The properties of these moment-rotation springs were derived considering typical reinforcing details for the columns and beams for such structural elements at the ground floor as well as an axial force level for the columns equal to 510 KN, representing the axial force level for a building with three more stories above the ground floor where the examined single-storey one bay R/C sub-assembly is assumed to be located. The thickness of the encased R/C panel was assumed to be equal to 150mm constructed with a material having Young's modulus equal to E=7GPa to account for a level of pre-cracking.

2.1 Numerical models

This first model was that without any encasement.

All the models, which were studied and are described briefly below, apart from the model without the encased panel designated as a *bare frame model*, included an encased R/C panel (with a 150mm thickness) which was connected to the surrounding frame in a variety of ways, as will be explained below. The connection of the encased R/C panel with the surrounding frame was numerically simulated with two distinct types of non-linear link elements each one representing a distinct force transfer mechanism [1, 6]. The first type of link represents the contact/gap mechanism between the R/C panel and the surrounding frame. The second type of connection between the R/C encasement panel and the surrounding frame represents an additional force transfer mechanism that simulates the presence of a steel metal tie. This second type of link was assumed to have an elasto-plastic behaviour in its longitudinal direction, representative of a steel reinforcing bar with a diameter of 12mm and a yield stress of 500MPa. A similar elasto-plastic behaviour was also assumed in the tangential direction representing such tangential behaviour of a 12mm diameter tie. Towards the quantification of such tangential behaviour of a steel tie, a sequence of tests was conducted that will be briefly described in section 4. The behaviour of the following three distinct models were studied that included an encasement of an R/C panel within the one-storey one bay R/C frame.

Encased model a. This model was provided with only contact/gap links all around the interface of the R/C encased panel with the surrounding frame..
Encased model b. In this model, the previous contact/gap transfer mechanism was retained. In addition, the second type of links were added only at the interface of the encased R/C panel with the top and bottom beams.
Encased model c. In this model, the contact/gap transfer mechanism was again retained together with the second type of links all around the interface.

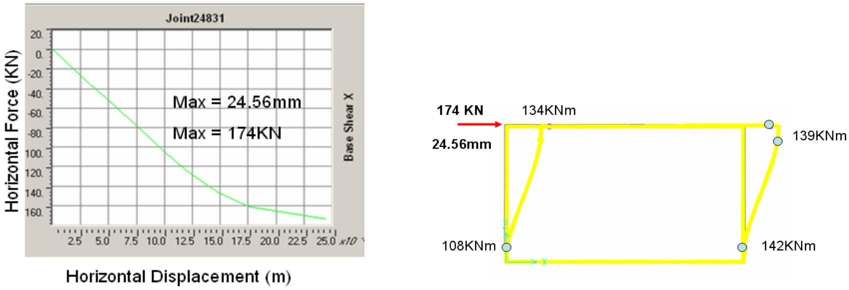
3 Numerical results

All numerical models were subjected to a horizontal incremental force in a direction coinciding with the mid-axis of the top beam (figure 1) in a "push-over" type of loading. The following non-linear mechanisms were considered:
-The possibility of developing plastic hinges at the top and bottom of the columns as well as at the left and right edge of the top beam.
-The possibility to triggering the contact/gap mechanism at the interface between the encased panel and the surrounding R/C frame
-The possibility of the steel ties connecting the encased panel with the top and bottom beam and the left and right column behaving in an elasto-plastic way both in a direction normal as well as tangential to the interface.

These non-linear mechanisms were not extended to include the R/C panel itself at this preliminary numerical analysis. Such a non-linear behaviour was included in a subsequent simulation not presented here. The numerical results include: a) The variation of the applied horizontal force against the corresponding displacement. b) The deformed shape of the single-storey one bay frame with or without the encasement at the maximum deformation level at the end of the "push-over" loading sequence. c) The variation of the forces that developed at the interface between the encased panel and the surrounding R/C frame.

3.1 Behaviour of the "bare frame' model

The non-linearity in the curve representing the variation of the applied horizontal force at the axis of the top beam against the corresponding displacement, as depicted in figure 2(a), is quite evident when the horizontal displacement exceeds the value of 15mm. When the top beam horizontal displacement level reaches the maximum amplitude of 24.5mm, plastic hinges develop at the critical locations of the beams and columns, as depicted in figure 2(b). The maximum amplitude of the horizontal force at that level is 174KN, which represents the bearing capacity of the bare frame whereas its initial stiffness is approximately 10KN/mm.

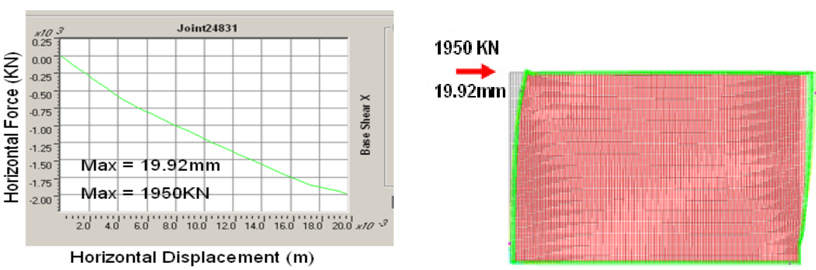


(a) (b)

Figure 2: (a) Horizontal force-horizontal displacement “bare frame response. (b) The studied “bare frame” single-storey one bay R/C frame without the encased R/C panel.

3.2 Behaviour of the “encased model a”

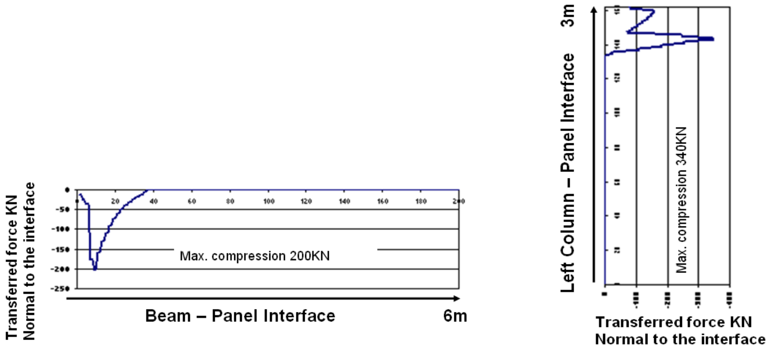
The non-linear trend in the curve representing the variation of the a applied horizontal force at the axis of the top beam against the corresponding displacement, as depicted in figure 3(a), is quite evident when the horizontal displacement level exceeds the value of 10mm. These non-linear trends are less pronounced here than for the bare frame model. When the top beam horizontal displacement level reaches the maximum amplitude of 19.92mm, the corresponding maximum amplitude of the horizontal force at that level is 1950KN (figure 3(b)). If this force level is compared to the bearing capacity of



(a) (b)

Figure 3: (a) Horizontal force-displacement “encased model a” response. (b) “encased model a” A single-storey one bay R/C frame with the encased R/C panel.

the bare frame it, represents an increase of 800%. A similar increase can also be observed in the stiffness that reaches the level of 150KN/mm. Figures 4(a) and 4(b) represent the transfer of forces at the interface between the encased panel and the surrounding frame through the contact/gap mechanism alone [1, 6]. As shown, this transfer takes place at the corners of the encased panel where it contacts the columns and the beam near the region of column-to-beam joints whereas a large part of the interface is free of forces due to the gap that forms at the interface at these locations. It can also be seen that these contact forces in a direction normal to the interface reach a relatively large amplitude in these narrow column-to-beam joints regions. Such high amplitude forces are expected to introduce additional non-linear mechanisms such as crushing of the encased panel and/or parts of the columns or beams at these regions. These additional mechanisms are not included in this preliminary numerical simulation.

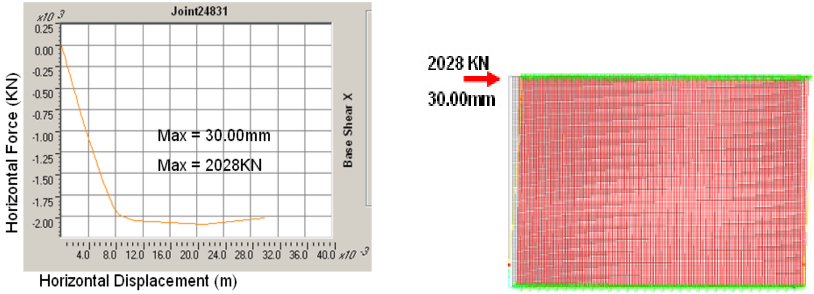


(a) (b)

Figure 4: Transfer of forces at the interface (a) between the encased panel and the top beam; (b) between the encased panel and the left column.

3.3 Behaviour of the "encased model b"

The horizontal force-displacement non-linearity, depicted in figure 5(a), is quite evident when the horizontal displacement level exceeds the value of 8mm. When the top beam horizontal displacement level reaches the maximum amplitude of 30.00mm, the corresponding maximum amplitude of the horizontal force at that level is 2028KN (figures 5(a) and 5(b)). The presence of steel ties between the encased panel and the surrounding frame retained the increase in the bearing capacity and the stiffness that was observed in the encased model a before. The stiffness reaches the value of 250KN/mm. Figure 6 shows the transfer of forces at the interface between the encased panel and the surrounding frame through the steel ties. As can be seen, the transfer takes place partly through the steel ties that are located at the interface between the encased panel and the top beam in the tangential direction. This transfer mechanism also results in moderating the amplitude of the forces that are transferred at the narrow column-to-beam joints regions in a direction normal to the interface through the contact/gap



(a) (b)

Figure 5: (a) Horizontal force-displacement "encased model b" response. (b) "encased model b" single-storey one bay R/C frame with the encased R/C panel.

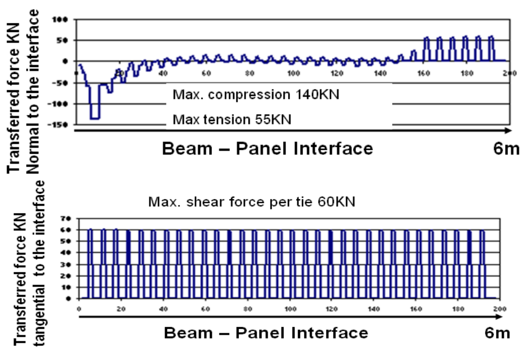
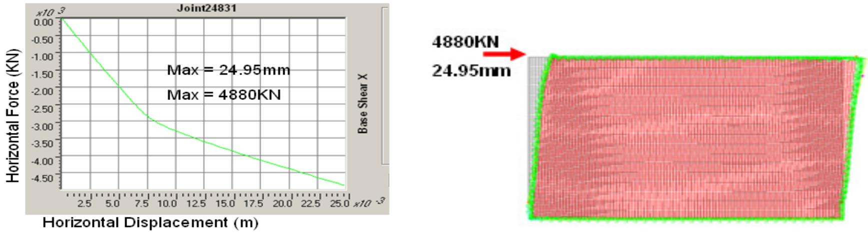


Figure 6: Transfer of forces at the interface for the encased model b.

mechanism. Such transfer of forces at the interface will mitigate the possibility of crushing of the encased panel and/or parts of the columns or beams at these regions.

3.4 Behaviour of the "encased model c"

The non-linear trend in the curve representing the variation of the a applied horizontal force at the axis of the top beam against the corresponding displacement, as depicted in figure 7(a), is quite evident when the horizontal displacement level exceeds the value of 7.5mm. When the top beam horizontal displacement level reaches the maximum amplitude of 24.95mm the corresponding maximum amplitude of the horizontal force at that level is 4880KN. The presence of steel ties between the encased panel and the surrounding frame both at the top and bottom beam as well as the left and right columns further augments the increase in the bearing capacity and the stiffness that was observed in the encased model b before (figures 7(a) and 7(b)). The stiffness reaches the value of 300KN/mm. Figures 8 represent the transfer of forces at the interface between the encased panel and the surrounding frame



(a) (b)

Figure 7: (a) Horizontal force-displacement "encased model c". (b) The "encased model c" single-storey one bay R/C frame with the encased R/C panel.

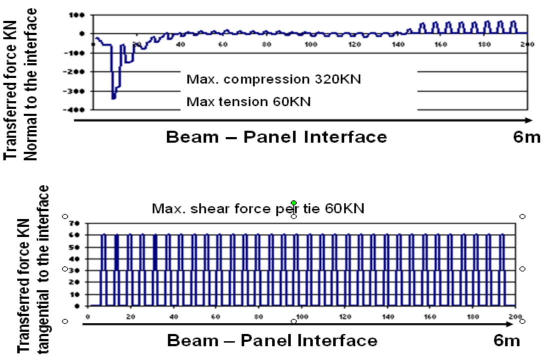


Figure 8: Transfer of forces at the interface between the encased panel and the top beam from the simulation of the encased model c.

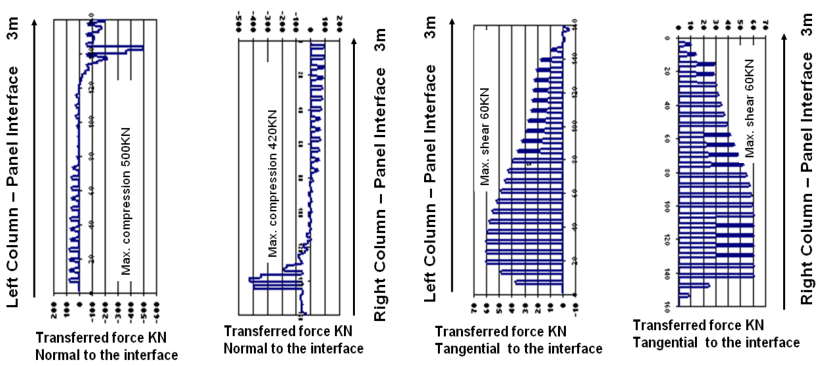


Figure 9: Transfer of forces at the interface between the encased panel and the left and right columns from the simulation of the encased model c.

through the top beam steel ties whereas figures 9 depict the transfer of forces through the ties that are located at the left and right columns. As can be seen, the transfer takes place partly through the steel ties that are located at the interface between the encased panel and the surrounding frame in the tangential direction.

3.5 Conclusive observations of the influence of the encased R/C panel

From the preceded preliminary numerical analysis, the encasement of the R/C panel within the single-storey one bay R/C frame resulted in a significant increase of the stiffness and the bearing capacity of the studied system. Moreover, the placement of steel ties apart from increasing the stiffness and the bearing capacity also resulted in moderating the amplitude of the forces that are transferred at the narrow column-to-beam joints regions in a direction normal to the interface through the contact/gap mechanism. Such moderation in the amplitude of the transferred forces at the interface will mitigate the possibility of crushing of the encased panel and/or parts of the columns or beams at these regions. It can also be concluded that the presence of steel ties in the interface between the encased R/C panel and the surrounding R/C frame has an overall beneficial effect on the behaviour of this type of structural system to seismic type loading. As shown in the preliminary numerical study when there are steel ties in such an interface these ties will transfer forces in a direction normal and tangential to the interface simultaneously. The level of these forces may vary in amplitude as well as in direction during the loading of the structure in a cyclic seismic type of loading. An experimental investigation was carried out with its main objective to study the mechanism of the transfer of such forces at the interface in such a way that this mechanism can be described both in terms of bearing capacity at a limit-state level linked with failure modes that are expected to appear. A summary of this study is presented in the next section.

4 Summary of the experimental sequence

Figure 10 depicts the dimensions of the studied specimens. As already explained, such a specimen represented a portion of an R/C encased panel connected with a portion of a column of the surrounding frame with the use of steel ties, which are embedded at the mid-plane of the panel and are anchored to the mass of the old concrete of the column, which is also retro-fitted with a jacket. Both the jacket and the R/C encased panel are indicated in a different color in figure 10 in order to signify the new concrete as compared to the existing column (old concrete). If this detail is rotated 90° clockwise it represents a similar connection between part of the R/C panel and the top beam. This specimen was loaded as indicated in figures 11(a) and 11(b). A load was applied normal to the interface (horizontal in figures 11(a) and 11(b)) whereas at the same time an additional load was applied in a direction parallel to the interface (vertical in figures 11(a) and 11(b)).

www.witpress.com, ISSN 1743-355X (on-line)

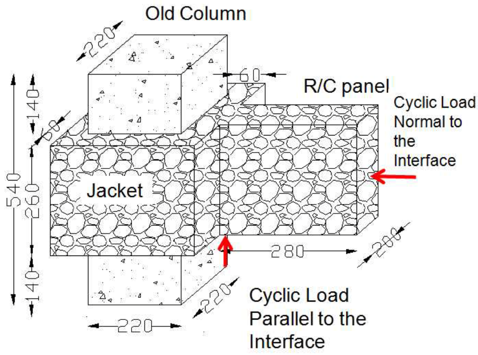
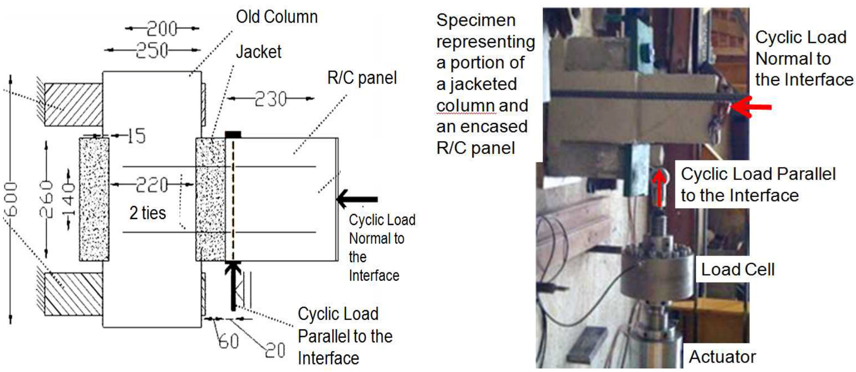


Figure 10: Specimen of a portion of encased R/C panel and the jacketed column.



(a) (b)

Figure 11: (a) Loading arrangement of a portion of encased R/C panel and the jacketed column. (b) Loading arrangement of a portion of encased R/C panel and the jacketed column.

The load that was applied parallel to the interface was varied in time in a manner consisting of three sinusoidal cycles of constant amplitude with a frequency 0.1Hz. The load that was applied normal to the interface was either kept constant at a predetermined level (tension or compression) or it was also varied in the same way as the load applied parallel to the interface. This type of load was expected to represent the transfer of forces at such an interface with the presence of steel ties, as was found by the preliminary numerical analysis described in a summary form in section 3. The total loading sequence per specimen consisted of a series of such cycles with continuously increasing amplitude till the failure of the specimen. This type of combined cyclic loading is believed to be adequately representative of the stress field that is expected to develop at this part of the encasement from the transfer of forces between the encased R/C panel and the surrounding frame arising from the seismic type of loading of the single-storey one bay frame, as was indicated by the preliminary numerical analysis results. The applied load in a direction parallel to the interface is measured in the ordinates whereas the measured sliding displacement at the interface between the portion of the panel and the jacketed column is measured at the abscissa. The increase in the amplitude in such a gradual cyclic way is consistent with a similar variation of the horizontal force at the level of mid-axis of the top beam (figure 1), representing in this way the variation of a seismic type load. Instrumentation was provided to measure the variation of the applied loads as well as the deformation of the specimen during the loading sequence. Figure 12 depicts such measured response for one of the specimens, with 4 steel ties of 12mm diameter. It was expected that due to the stress field that would arise in the vicinity of the interface when the combined loading was applied, the expected modes of failure would include a shearing pattern for the concrete accompanied by a local deformation of the steel ties. As can be seen in this figure, the measured response reveals three stages in the performance of such a steel-tie connection. Up to a relatively small cyclic displacement level, of the order of 1.0mm, the measured response is almost linear elastic. Then, when the maximum capacity of the connection is reached there are small amplitude plastic deformations. Finally, the plastic deformations increase substantially accompanied by a significant drop in the bearing capacity. Excessive cracking of the interface occurs that reveals the deformed shape of the steel ties, as they are shown in figure 13. The measured response of this steel tie connection at the interface, in a direction parallel to the interface, under a combined loading representative of the stress field that is expected to develop at this part of the encasement, is in agreement with the assumptions made in the numerical

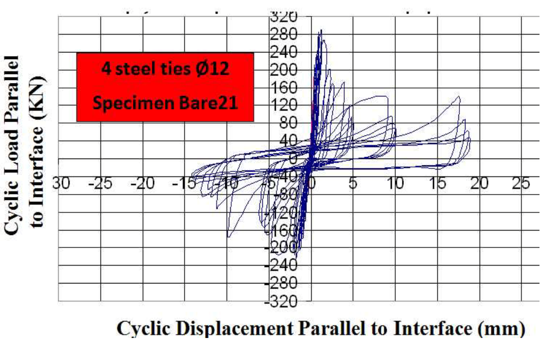


Figure 12: Measured load-displacement cyclic response in direction parallel to the interface.

Figure 13: Deformed shape of the steel ties at the final stage of loading of sequence.

www.witpress.com, ISSN 1743-355X (on-line)

simulation concerning these behavioural characteristics. Therefore, the main observations made on the performance of the en-casement, as derived from this preliminary numerical analysis described in section 3, are expected to be in general valid, especially concerning the performance of the interface and of these steel ties.

5 Conclusions

• This numerical study examines the influence on the in-plane behaviour of one-bay single-storey reinforced concrete (R/C) frame arising from the presence of an encased R/C panel with or without steel ties.
• From the preliminary numerical analysis results it can be concluded that such an encasement results in a considerable increase of the stiffness and the bearing capacity of the studied system, especially when steel ties are present at the interface. Moreover, the placement of steel ties also moderates the amplitude of the forces that are transferred at the narrow column-to-beam joints regions in a direction normal to the interface through the contact/gap mechanism, and consequently, mitigate the possibility of crushing of the encased panel and/or parts of the columns or beams at these regions.
• Thus, it can be concluded that an encased R/C panel, connected with the appropriate steel ties to the surrounding R/C frame, has an overall beneficial effect on the behaviour of this type of structural system to seismic type loading
• An experimental sequence was also performed in order to quantify the behaviour of such steel tie connections at the interface under a stress field that is expected to develop at this part of the encasement during seismic type loading. Such measured behaviour, is in agreement with the assumptions made concerning the behavioural characteristics of these steel ties in the preliminary numerical analysis.

References

[1] G. C. Manos, V. J. Soulis, J. Thauampteh “The behaviour of masonry assemblages and masonry-infilled R/C frames subjected to combined vertical and cyclic horizontal seismic-type loading”, J. Advances in Engineering Software, 45 (2012) 213-231.
[2] Organization of Earthquake Planning and Protection of Greece (OASP), Guidelines for Level – An earthquake performance checking of buildings of public occupancy, published by OASP, Athens, 2001.
[3] Provisions of Greek Seismic Code 2000, Organization of Earthquake Planning and Protection of Greece (OASP), Dec. 1999, published by OASP. Revisions of seismic zonation introduced in 2003, Government Gazette, Δ17α /115/9/ΦΝ275, No. 1154, Athens, 12 Aug. 2003.
[4] G. C. Manos, Consequences on the urban environment in Greece related to the recent in-tense earthquake activity, Int. Journal of Civil Eng. and Architecture 5 (12) (2011) 1065-1090.

www.witpress.com, ISSN 1743-355X (on-line)

[5] G. C. Manos, V. J. Soulis and J. Thauampteh. “A Nonlinear Numerical Model and its Utilization in Simulating the In-Plane Behaviour of Multi-Storey R/C frames with Masonry Infills”, The Open Construction and Building Technology Journal, 6, (Suppl. 1-M16) 254-277, 2012.

[6] G. C. Manos (2012) “Cyclic In-plane Behaviour of R/C Frames Retrofitted with Jacketing and an Encased R/C Panel” Report to Hellenic Earthquake Planning and Protection Organization (EPPO) in Greek.

[7] Organization of Earthquake Planning and Protection of Greece (OASP), Guidelines for Retrofitting in Reinforced Concrete Buildings, Government Gazette, Δ17α /04/5/ΦΝ 429-1, No 42, Athens, 20 Jan. 2012.

[8] Klingner R. E. and Bertero V.V. “Earthquake Resistance of Infilled Frames”, ASCE Journal of Structural Engineering. Vol 116, pp. 973-989, 1978.

[9] T. Sugiyama, Y. Matsuzaki, K. Nakano, “Design for Structural Performances of R/C Frame with Cast in Place Non-Structural R/C Walls”, 13th WCEE Vancouver, B.C., Canada August 1-6, 2004, Paper No. 1277.

www.witpress.com, ISSN 1743-355X (on-line)