RETROFITTED ONE-BAY SINGLE-STORY R/C FRAME WITH ENCASED R/C PANEL UNDER SEISMIC-TYPE LOADING

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ABSTRACT

The in-plane behavior of one-bay single-story 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 it is connected to the surrounding frame with or with-out steel ties. From preliminary numerical results that were concluded the encased panel results in a considerable increase of both stiffness and bearing capacity, especially when steel ties are present at the interface. The presence of steel ties moderates the amplitude of the forces transferred at the narrow column-to-beam joint regions in a direction normal to the interface through the contact/gap mechanism; thus it, mitigates the possibility of crushing the encased panel and/or parts of the columns or beams at these regions. Thus, an encased R/C panel connected by the appropriate steel ties with the surrounding R/C frame, has a beneficial effect on the seismic behavior of this type of structural system. Experiments were performed to quantify the behavior of such steel tie connections at the interface under a stress field similar to the one that develops at this part of the encasement during seismic type loading. Furthermore, advanced numerical simulations of the tested connection detail are also conducted to validate numerical tools that can quantify such a transfer of forces for connection details that can be used in design. It was concluded that the described advanced numerical simulation is a quite valid numerical approach.

Keywords: RC frames; Jacketing; Steel ties; Realistic advanced numerical simulations; Seismic loadings;

1. INTRODUCTION

Many multi-storey reinforced concrete (R/C) structures, built in seismic regions, have their ground and low-level floors designed to function as open spaces, like parking spaces. Therefore, the bays of the R/C frames at these levels 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 (Manos, 2012) that the dynamic and earthquake behavior of such structures, having a relatively flexible ground floor (soft story) and stiff upper stories, results in increased demands on the structural elements of the ground floor. This is due to the interaction of the masonry infills with the surrounding R/C frames that contributes to a substantial increase of the story stiffness of these upper stories compared to the story stiffness of the ground floor (Manos, 2012, 2011, 2011). This, in turn, leads to structural damage, unless the structural R/C elements at the ground floor are properly designed (Manos, 2012, 2011, 2011, OASP, 2001, 2003). As this behavior 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. These temporary shoring schemes serve primarily to secure the transfer of the gravitational forces at the “soft

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story” level despite the potential damaged columns or shear walls thus safeguarding against partial or total collapse.

Such temporary shoring schemes should be deployed as soon as possible as their contribution can be critical in ensuring the structural stability of such damaged building in the aftermath of a strong earthquake main shock that is usually followed by a series of quite strong aftershocks. This contribution of the depicted in figures 1b and 1d temporary shoring schemes can be enhanced even further by retrofitting schemes, which will replace the temporary shoring and will increase the stiffness and the resistance of the “soft story” becoming a permanent part of the structural system. Such a retrofitting scheme is studied here. It can be easily applied to such a soft ground floor of multi-storey R/C structures (Manos 2011, 2011, 2013, 2012, OASP, 2012). Initially, a retrofitting effort usually consists of repairing and strengthening the structural elements at the soft ground floor by:

A. R/C jacketing of the existing R/C columns at the ground floor level (figure 1a).
B. R/C jacketing of the existing R/C beams at the ground floor level.

With the jacketing of these 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 (OASP 2012) 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 (figure 1b).

In the relevant provisions of these guidelines (OASP 2012) 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 structurally 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 encased R/C panel is smaller than the width of the beams that form the encased bay.

2. PRELIMINARY NUMERICAL SIMULATION

The numerical study that is presented in this section was limited to examining a single-storey one-bay R/C frame (figure 2) formed by two columns (left and right) and two beams (top and bottom). The over-all frame dimensions, chosen arbitrarily, were: 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 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 each column 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 behavior scenario, by the formation of the plastic hinges is considered, excluding any shear non-linear behavior of the R/C structural members. In this preliminary numerical simulation, the behavior of the encased R/C panel itself was considered to be elastic. A subsequent numerical simulation considered the possibility of non-linear behavior 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
The following three alternative numerical models are examined (Manos 2013, 2012), using the software SAP2000:

a) The first numerical model was that without any encasement (“bare model”).

b) In the previous “bare model” an encased R/C panel is added having a 150mm thickness. This panel was connected to the surrounding frame with non-linear link elements, which represent the contact/gap mechanism between the encased panel and the surrounding frame. Through this mechanism only forces normal to the interface are transferred between the encased panel and the surrounding frame. This is denoted as “encased model b”.

c) Again, to the “bare model” an encased R/C panel is added having a 150mm thickness. However, this time the panel was connected to the surrounding frame with two distinct types of non-linear link elements each one representing a distinct force transfer mechanism (Manos 2013, 2012). The first type of link represents again the contact/gap mechanism between the R/C panel and the surrounding frame, as explained before. 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 elastoplastic behavior in its longitudinal direction, representative of a steel reinforcing bar with a diameter of 12mm and a yield stress of 500MPa. Similar elastoplastic behavior was also assumed in the tangential direction representing such tangential behavior of a 12mm diameter tie. Towards the quantification of such tangential behavior of a steel tie, a sequence of tests was conducted that will be briefly described in section 4. The results that are presented here correspond to an encased R/C panel connected with the surrounding columns and beams with both the contact/gap and the steel metal ties mechanisms. The behavior of the following three distinct models was studied that included an en-casement of an R/C
panel within the one-storey one bay R/C frame. This is denoted as “encased model c”.

2.2 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 2) in a “push-over” type of loading. The following non-linear mechanisms were considered:
1. 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.
2. The possibility to triggering the contact/gap mechanism at the interface between the encased panel and the surrounding R/C frame in the direction normal to the interface between the en-cased panel and the surrounding R/C structural elements (either top/bottom beam or lefty/right column).
3. 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 elastoplastic way both in a direction normal as well as tangential to the interface.

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 develop at the interface between the en-cased panel and the surrounding R/C frame.

Figures 3a and 3b depict the numerically predicted behavior for the “bare 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 3a, 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 3b. 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. The above frame performance is dominated by flexure, a fact that presumes that all structural elements are provided with the appropriate shear resistance.

Figures 4a and 4b depict the numerically predicted behavior for the “encased model b”. The non-linear trend 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 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 4b). If this force level is compared to the bearing capacity of the bare frame it, represents an eight (8) times increase. A fifteen (15) times increase, compared to the “bare frame model”, can also be observed studying the variation in the stiffness. For the “encased model b” its stiffness reaches the level of 150KN/mm. Figures 5a, 5b, 6a and 6b represent the transfer of forces at the interface between the encased panel and the surrounding frame through the contact/gap mechanism alone. As shown, this transfer takes place at the corners of the encased panel where it contacts the left/right columns (figures 6a and 6b) and the beam/foundation (figures 5a and 5b) 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 relatively large amplitude in these narrow column-to-beam joints regions. Such high amplitude forces are expected to introduce additional nl 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.

![Figure 5a](image1.png) Transfer of forces normal to the interface between the encased panel and the top beam from the simulation of the encased model b.

![Figure 5b](image2.png) Transfer of forces normal to the interface between the encased panel and the foundation from the simulation of the encased model b.

![Figure 6a](image3.png) Transfer of forces normal to the interface between the encased panel and the left column from the simulation of the encased model b.

![Figure 6b](image4.png) Transfer of forces normal to the interface between the encased panel and the right column from the simulation of the encased model b.

Figures 7a and 7b depict the numerically predicted behavior for the “encased model c”. The non-linear trend 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 7a, 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. Apart from the contact/gap mechanism, 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. The stiffness reaches the value of 300KN/mm, that represents a thirty (30) times increase compared with the stiffness of the “bare mode”. Figures 5a and 5b represent the transfer of forces at
the interface between the encased panel and the surrounding frame through the top beam steel ties whereas figures 8 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 a direction parallel to the interface between the encased panel and the surrounding frame left and right columns (figures 9c and 9d) and the beam/foundation (figures 8b). Due to the non-linear properties assumed for the links representing the steel times the maximum force that can be transferred through each one of this link has an upper limit equal to 60KN.

The contact/gap mechanism when companied with the steel ties mechanism helps also to transfer forces in a direction normal to the interface between the encased panel and the surrounding frame. In figure 8a the forces transferred between the encased panel and the top beam in a direction normal to the interface are depicted. As can be seen the ties help to transfer a limited amount of tensile forces at the right side of the beam; this was not possible when the steel ties were not present (see figure 5a). The compression forces transferred in this case are concentrated again at the left corner of the beam.
This time the compressive zone is again narrow with high amplitude (see figures 8a and 5a). The same can also be observed when examining the transfer of forces between the encased panel and the left and right columns in a direction normal to the interface as can be seen by examining figures 6a and 6b (model only with contact/gap mechanism) and figures 9a and 9b (model with contact/gap and metal ties mechanisms). The relative increase in the maximum amplitude values for the compressive forces between “numerical model b” (only contact/gap mechanism) and “numerical model c” contact/gap and metal ties mechanisms) should be seen taking into account that in the later case the horizontal force amplitude is 2.5 times larger than the corresponding amplitude for the former case.

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 behavior 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 parallel 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. It must be underlined that in the preliminary numerical analysis, apart from the non-linear mechanisms that were used to numerically simulate the transfer of forces at the interface between the encased panel and the surrounding frame as well as the simulation of the flexural plastic hinges at predetermined locations at the ends of the columns and beams the possibility of non-linear behavior of the encased panel itself, although very significant, was ignored at present.

3. EXPERIMENTAL SEQUENCE

Summary results will be presented in this session from an extensive experimental sequence which aimed to study the interaction between metal ties at the interface between the encased R/C panel and the surrounding frame (Manos, 2012). As described in the introduction, as a first step the structural members of the R/C frame would be jacketed and then the encased panel will be constructed within the bay of this strengthened R/C frame either with or without metal ties (see section 2). Figure 9a depicts the sketch of such an encased R/C frame with columns and beams having jackets before the placement of the encased R/C panel. It can be easily seen that this detail can also represent the connection between the encased panel with the right column (180° rotation) or with the top and bottom beams (90° or 270° rotation, respectively (figures 10a and 10b). The dimensions shown in figure 9b are the ones selected to be used to construct the specimens to be tested at the laboratory. As already explained, such a specimen represents a portion of an R/C encased panel (200mm thick) connected with a portion of a column/beam of the surrounding frame with or without the use of steel ties. When steel ties are used they are embedded at the mid-plane of the panel and are anchored to the mass of the old concrete of the column (220mm by 220mm), which is also retrofitted with a jacket 60mm thick.

Both the jacket and the R/C encased panel are indicated in a different color in figure 9b in order to differentiate the new concrete from the existing column (old concrete). This specimen was loaded as indicated in figure 10b, 10c and 10d. A load was applied normal to the interface (horizontal in figures 10b, 10c and 10d) whereas at the same time an additional load was applied in a direction parallel to the interface. This connection detail is studied with a specimen having a height of 540mm, with the encasement being 260mm high. This limitation was introduced in order to confine the examination in the connection with a metal tie and control the forces to be transferred.

The load which 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 combined loading was thought 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 2. 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. Such forces will arise from the seismic response of a multi-story R/C structure and it is assumed that will subject an encased R/C frame mainly to this type of in-plane loads, ignoring the forces that would result in the out-of-plane direction of this encased R/C frame.

Figure 10a. Sketch of an encased R/C panel and frame having columns and beams with jackets

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

Figure 10c. Loading arrangement of a portion of encased R/C panel and the jacketed column

Figure 10d. Loading arrangement of a portion of encased R/C panel and the jacketed column

Figure 11a depicts such measured response for one of the specimens, with 4 steel ties of 12mm diameter. In this figure, the load applied in the direction parallel to the interface is plotted in the ordinates whereas the measured resulting sliding displacement at the interface between the portion of the panel and the jacketed column is plotted at the abscissa of this figure. Due to the stress field arising at the vicinity of this interface when the previously described combined loading is applied, the expected modes of failure include a shearing pattern for the concrete volume neighboring the steel ties accompanied by a local deformation of the steel ties themselves. The amplitude of the applied force parallel to the interface was gradually increased in cyclic manner. This is consistent with a variation of the horizontal forces resulting from seismic actions.

As can be seen in figure 11a, 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, increasing the displacement/load amplitude the maximum capacity of the connection is reached. The sliding deformations at the interface are
relatively small including a portion of remaining plastic deformations. Finally, these plastic deformations increase substantially accompanied by a significant drop in the bearing capacity.

Figure 11a. Measured Load-displacement cyclic response in direction parallel to the interface

Figure 11b. Deformed shape of the steel ties at the final stage of loading sequence.

Excessive cracking of the interface occurs that reveals the deformed shape of the steel ties, as they are shown in figure 11b after the end of the test and the destruction of the specimen. 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 up to a point in agreement with the assumptions made in the preliminary numerical simulation presented in section 2.

Figure 12a. Measured Load-displacement cyclic response in direction parallel to the interface

Figure 12b. Broken interface at the end of test for specimen 18 without metal ties

Figure 12c. Broken interface at the end of test for specimen 18 without metal ties

Figure 12a depicts the measured response for one of the specimens without any steel ties. The observed state of the interface at the end of this test is depicted in figures 12b and 12c. By comparing the load-displacement response of figure 12a with the corresponding load-displacement response of figure 11a the contribution of the presence of the steel ties can be clearly identified. The obtained bearing capacity with the four 12mm diameter steel ties is approximately seven (7) times larger than where the ties are not there. This maximum capacity without the ties is observed for very small amplitude sliding displacement. This sliding displacement amplitude is less than 20% of the corresponding displacement amplitude that the maximum capacity was observed when the steel ties were in place. Furthermore, the degradation of the bearing capacity when the sliding displacement amplitude is increased is far more pronounced without the steel ties than with. This simple comparison demonstrates the importance of the presence of the steel ties in transferring the combined forces at the interface between the encased R/C panel and the jacketed columns or beams of the surrounding R/C frame. A next important step is to be able to quantify such a transfer of forces for connection details that can be used in design. This is done in the following section whereby a numerical simulation of the tested connection detail is examined to validate simple numerical tools that can serve such a purpose.
4. NUMERICAL SIMULATION OF THE STUDIED CONNECTION DETAIL

In this numerical simulation effort, a relatively complex model was developed [Manos, 2012] using the FE software ABAQUS. This was a 3-D representation of the problem at hand. The interface was simulated with a contact area and the metal ties with 3-D rods that were embedded in the 3-D finite element mesh in an exact geometric representation of the actual specimen with four (4) steel ties having a diameter of 12mm each. Due to symmetry with respect to the y-z plane only one half (1/2) of the specimen was numerically simulated.

The material law adopted for the steel ties was elastoplastic. This material law was based on the measured behavior if steel specimens identical to the used steel ties which were tested under axial tension. Similar procedure was followed for the reinforcing bars embedded in the R/C encased panel and jacketed column. For the concrete damaged plasticity model was adopted (Hibbit, 2010). An analytical description of this material model is given by Malm (2009) and Mercan et al. 2010.

At the right part of figure 14 the cracked concrete region neighboring the interface is depicted when the applied force results in a sliding displacement equal to 4mm. At the left side of the same figure the observed cracked concrete region near the interface is also shown for similar amplitude of the sliding displacement (see figure 11a). As can be seen in figure 14 the numerical prediction is a realistic simulation of the observed behavior of the concrete volume in the region neighboring the interface.

At the right part of figure 15 the numerically predicted deformation and stress patterns of the steel ties in the region neighboring the interface is depicted when the sliding displacement reached the maximum amplitude (larger than 10mm). At the right part of figure 15 the numerically predicted
deformation and stress patterns of the steel ties in the region neighbouring the interface is depicted when the sliding displacement reached the maximum amplitude (larger than 10mm).

![Figure 15. Observed(a) / numerically predicted (b) deformation and stress patterns of the steel ties](image)

At the left part of the same figure the observed deformed steel ties at the end of the test are also shown (see also figure 11b). As can be seen in figure 15, the predicted steel tie deformation/stress pattern is a realistic simulation of the observed behavior of the steel ties in the region neighboring the interface.

In figure 16 the measured envelope curve of the variation of the applied force parallel to the interface versus the resulting sliding displacement is plotted together with the corresponding numerically predicted force-displacement response. As can be seen in this figure the measured load-displacement response of the examined connection is reasonably well predicted by the described numerical simulation.

![Figure 16. Measured and predicted applied force parallel to the interface versus sliding displacement response](image)

The maximum predicted bearing capacity value is approximately 20% smaller than the corresponding measured value. Moreover, the measured response appears to have a stiffer behavior till it reaches its maximum resistance than the predicted response. Despite this discrepancy, the predicted behavior for both the initial stiff part (for sliding displacement amplitudes smaller than 0.5mm) as well as for the plastic part (for sliding displacement amplitudes larger than 3mm) agree quite well with the measured response of the tested specimen. Based on the above discussion between predicted and observed response it can be concluded that the described in this section numerical simulation is a quite valid numerical approach that can be further utilized towards quantifying the transfer of forces for connections details that are studied in the present work in order to be used in design.

5. CONCLUSIONS
This numerical study examines the influence on the in-plane behavior 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, an encased R/C panel, connected with the appropriate steel ties to the surrounding R/C frame, has an overall beneficial effect on the behavior of this type of structural system to seismic type loading

• An experimental sequence quantified the behavior 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.

• An advanced numerical simulation was also examined that includes the plastification of the steel ties as well as that of the concrete volume utilizing the capabilities of the damaged plasticity model of the commercial software Abaqus. The predicted plastic strain and deformation patterns for the steel ties and the concrete in the regions neighboring the interface were a realistic representation of the corresponding observed behavior. Moreover, the predicted applied force parallel to the interface versus the sliding displacement response agrees reasonably well with the measured response in terms of stiffness, maximum bearing capacity and plastified sliding deformations.

• The described advanced numerical simulation is a quite valid numerical approach that can be further utilized towards quantifying the transfer of forces for connections details that are studied in the present work in order to be used in design.

6. ACKNOWLEDGMENTS
The partial support of the Hellenic Organization of Earthquake Planning and Protection is gratefully acknowledged

7. REFERENCES


