Investigation of the effectiveness of two styles of R/C jackets in pre-earthquake retrofitting of columns and b/c joints of R/C structures

A. G. Tsonos\textsuperscript{1} & I. P. Rentzeperis\textsuperscript{2}

\textsuperscript{1}Department of Civil Engineering, Aristotle University of Thessaloniki, Greece
\textsuperscript{2}Thessaloniki, Greece

Abstract

This paper presents the findings of an experimental study to evaluate methods of retrofit which addresses particular weaknesses that are often found in reinforced concrete structures, especially older structures, namely the lack of the required flexural and shear reinforcement within the columns and the lack of the required shear reinforcement within the joints. Thus, the use of two styles of four-sided reinforced concrete jackets for the pre-earthquake retrofitting case of columns and beam-column joints was investigated experimentally. In this paper the effectiveness of the two jacket styles was also compared.

Keywords: beam-column frames, beams, columns, joints, cyclic loads, earthquake resistant structures, hinges (structural), reinforced concrete, repairs.

1 Introduction

Damage caused by earthquakes through the years, indicated that some reinforced concrete buildings designed and constructed in the 1960s and 1970s were found to have serious structural deficiencies. These deficiencies are mainly a consequence of a lack of capacity design approach and/or poor detailing of reinforcement. As a result, lateral strength and ductility of these structures were minimal.

The feasibility and technical effectiveness of two styles of four-sided reinforced concrete jacket systems in pre-earthquake retrofitting case of columns and beam-column joints was investigated in this paper. Thus, three identical reinforced concrete exterior beam – column – joint – slab – transverse beam
subassemblages (O₁, P₁ and L₁) were constructed with non-optimal design parameters: flexural strength ratio, joint shear stress, with less column transverse reinforcement than that required by the modern Codes [1,2] and without joint transverse reinforcement, representing the common construction practice of columns and beam-column joints of older structures built in the 1960s and 1970s.

The subassemblage O₁ was subjected to cyclic lateral load history so as to provide the equivalent of severe earthquake damage and was used as “reference specimen”. The subassemblages P₁ and L₁ represent parts of an old frame structure, which was upgraded to resist future strong earthquakes. These two subassemblages were tested only after strengthening by four-sided reinforced concrete jackets. These jackets were applied in the columns and b/c joint regions of the subassemblages P₁ and L₁, which after the interventions were named SP₁ and GSL₁. A premixed, non-shrink, rheoplastic, flowable and non-segregating mortar of high-strength was used for the construction of the cement grout jacket of specimen SP₁. Shotcrete was used in lieu of conventional concrete in the construction of the cement grout of specimen GSL₁.

A direct comparison of the load-deflection curves of the original subassemblage O₁ and the retrofitted subassemblages SP₁ and GSL₁, was provided in this paper. The effectiveness of the two jacket styles was also compared.

2 Description of the specimens

2.1 Original test specimens

Three identical test specimens O₁, P₁ and L₁ were constructed using normal weight concrete and deformed reinforcement. Both specimens were typical of existing older structures built in the 1960s and 1970s. ACI-ASCE Committee “Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-1985)” specifies the maximum allowable joint shear stresses in the form of \( \gamma \sqrt{f_c'} \) MPa, where joint shear stress factor \( \gamma \) is a function of the joint type (i.e. interior, exterior, etc.) and of the severity of the loading, and \( f_c' \) is the concrete compressive strength. Lower limits of the flexural strength ratio \( M_R \) and joint transverse reinforcement are also confirmed by this Committee. Thus, for the beam-column connections examined in this investigation, the lower limits of \( M_R \) and \( \gamma \) are 1.40 and 1.00 respectively [2].

As seen in fig. 1, all the specimens O₁, P₁ and L₁ had less column transverse reinforcement than that required by the new Greek Code for the Design of Reinforced Concrete Structures [1], did not have joint transverse reinforcement (often ties in the joint region were simply omitted in the construction process in the past because of the extreme difficulty they created in the placing of reinforcement), whereas the values of flexural strength ratio were less than 1.40, and those of the joint shear stress were greater than \( 1.0 \sqrt{f_c'} \) MPa for all the specimens O₁, P₁ and L₁. Thus, the beam-column connections of the original
specimens can be expected to fail in shear. The dimensions of the test specimens were primarily dictated by the availability of formwork and laboratory testing capacities, resulting in a beam-to-column subassemblage model of approximately 1:2 scale. The concrete compressive strengths of specimens O1, P1 and L1 were 16.20MPa, 16.00MPa and 16.40MPa respectively.

Figure 1: Dimensions and cross-sectional details of original specimens O1, P1 and L1 (dimensions in cm).

2.2 Strengthening technique, specimens SP1 and GSL1

The subassemblages P1 and G1 represent parts of an old frame structure, which was upgraded to resist future strong earthquakes. Thus, both specimens P1 and G1 were tested only after strengthening by reinforced concrete jacketing and are designated hereafter as specimens SP1 and GSL1 respectively.

The original as-built specimens O1 had experienced brittle shear failure at the joint region. Strengthening of both specimens SP1 and GSL1 involved encasing the original beam-column joints and the columns of P1 and G1 with a four-sided cement grout jacket reinforced with additional collar stirrups in the joint region and additional ties in the columns [3, 4, 5, 6, 7, 8].

A premixed, non-shrink, rheoplastic, flowable and non-segregating mortar of high strength with 0.95cm maximum size of aggregate was used for the construction of the cement grout jacket of specimen SP1. Shotcrete was used in lieu of conventional concrete in the construction of the cement grout jacket of
specimen GSL1. In fig. 2(a) is shown the application of dry-mix shotcrete during strengthening procedure of subassemblage GSL1. Experience of the nozzle operator being crucial to ensuring a good quality installation, all shotcrete was placed by a shotcrete firm with over 35 years of experience. To ensure quality placement one 800×800mm test panel was shot vertically to 120mm thickness [9]. The panel was cored to check compressive strength and visual placement quality [10]. As shown in fig. 2(b) specimens SP1 and GSL1 had a four-sided cement grout jacket, plus Ø14 longitudinal bars at each corner of the column connected by Ø8 supplementary ties at 7cm. All longitudinal bars in the jackets extended through the joint region of the subassemblages.

Collar stirrups were used in the joint of the strengthened specimens SP1 and GSL1 to increase its shear strength [3]. These collar stirrups were inclined bars Ø14 bent diagonally across the joint core of SP1 and GSL1 as shown in figures 2(a), 2(b).

The columns of both the strengthened specimens SP1 and GSL1 satisfied all the requirements of the new Greek Code for the Design of Reinforced Concrete Structures [1] and the b/c joint region of these specimens satisfied all the requirements of the ACI-ASCE Committee 352 [2]. The subassemblages SP1 and GSL1 could therefore be expected to develop flexural hinges in the beams without severe damage concentration in the joint region.

The concrete compressive strengths of the jackets of SP1 and GSL1 were 41.00MPa and 39.50MPa respectively. In order to compare the effectiveness of the two jacket-styles, the corresponding structural members of both the strengthened subassemblages SP1 and GSL1 must have the same strength. For this reason the concrete compressive strengths of both the shotcrete jacket and cast-in-place jacket were almost the same.

The original specimens O1, P1 and L1 and the strengthened ones SP1 and GSL1 were constructed using deformed reinforcement (NOTE: Ø8, Ø14 = bar with diameter 8mm, 14mm). Approximately 10 electrical-resistance strain gages were bonded in the reinforcing bars of each specimen.

3 Test setup: loading sequence

A testing frame in the Laboratory of Reinforced Concrete Structures at the Aristotle University of Thessaloniki was used to apply cyclic displacements to the beam, while maintaining a constant axial load (150kN) in the column of all the specimens, fig. 3(a). The specimens O1, SP1 and GSL1 were loaded transversely according to the load history shown in fig. 3(b).

4 Test results

The connection of the as-built reference subassemblage O1 as expected, exhibited premature shear failure during the early stages of seismic loading. Damage occurred both in the joint area and in the columns’ critical regions. The beam of specimen O1 remained intact at the conclusion of the tests. Failure mode of specimens SP1 and GSL1 involved, as expected, the formation of a plastic
hinge in the beam near the column juncture. Damage of these specimens (SP₁ and GSL₁) occurred both in the beam’s critical region and in the joint area.

Figure 2: (a) Applying dry-mix shotcrete during strengthening procedure of subassemblage GSL₁, (b) Jacketing of column and beam-column connection of subassemblages SP₁ and GSL₁.
Figure 3:  (a) Test setup (dimensions in mm), (b) Lateral displacement history.
Figure 4: Plots of applied shear-versus-drift angle for specimens O₁, SP₁ and GSL₁.

© 2005 WIT Press

WIT Transactions on The Built Environment, Vol 81, © 2005 WIT Press
www.witpress.com, ISSN 1743-3509 (on-line)
The performance of the test specimens is presented herein and discussed in terms of applied shear-versus-drift angle relations. Drift angle $R$, which is plotted in figures which follow, is defined as the beam tip displacement $\Delta$ divided by the beam half span $L$ and expressed as a percentage (see the inset on figure 4).

Plots of applied shear-versus-drift angle for all the specimens $O_1$, $SP_1$ and $GSL_1$ are shown in figure 4. The original subassemblage $O_1$ showed stable hysteretic behavior up to drift angle $R$ ratio of 2.0 percent. It showed a considerable loss of strength, stiffness and unstable degrading hysteresis beyond drift angle $R$ ratio of 2.0 percent (fig. 4). Strengthened specimens $SP_1$ and $GSL_1$ exhibited stable hysteresis up to the 5th cycle of drift angle $R$, of 3.5 percent and up to the 4th cycle of drift angle $R$, of 3.0 percent respectively. Both specimens showed a considerable loss of strength, stiffness and unstable degrading hysteresis beyond drift angle $R$ ratios of 5.0 percent (fig. 4).

A comparison of the performance of strengthened specimens $SP_1$ and $GSL_1$ with that of the original specimen $O_1$ respectively indicated that the strengthened specimens achieved significantly increased strength, stiffness and energy dissipating capacities compared to the original specimens, even in the large displacement amplitude cycles of drift angle $R$ ratios between 3 percent and 4.5 percent (fig. 4).

![Specimens GSL1 / SP1](image)

**Figure 5:** Stiffness comparison between strengthened specimens $GSL_1$ and $SP_1$.

The principal objective of this research project was to test and validate the performance of shotcrete jacketed reinforced concrete columns and joints of frame structures under reversed, cyclic loading. An important part of this investigation was to compare shotcrete jacket performance with cast-in-place concrete jacketing. For comparison of the effectiveness between the two jacket styles it is worth comparing the peak-to-peak stiffness, the energy dissipated and
the peak strength observed for every load cycle of the specimen SP$_1$ (strengthened by cast-in-place jacket) with that of specimen GSL$_1$ (strengthened by shotcrete jacket). The comparison of the peak-to-peak stiffness for every load cycle is illustrated in fig. 5 while the energy dissipated of each specimen SP$_1$ and GSL$_1$ is shown in fig. 6. Figure 7 compares the peak strength observed throughout the tests. The comparison is made by observing the ratio of the peak strengths of SP$_1$ to that of GSL$_1$. From these diagrams it is clearly seen that specimen SP$_1$ achieved a slight increase in strength, stiffness and energy dissipating capacities as compared with those of specimen GSL$_1$.

![Specimens GSL1 / SP1](image)

**Figure 6:** Energy dissipation comparison between strengthened specimens GSL$_1$ and SP$_1$.

## 5 Conclusions

Based on the results obtained in this paper, the following conclusions can be drawn.

1. Original as-built reference subassemblage O$_1$ representing an existing beam-column subassemblage, performed poorly under reversed cyclic lateral deformations. The connection of this subassemblage exhibited premature shear failure during the early stages of seismic loading, and damage to the subassemblage was concentrated in the joint region.

2. The performance of the jacketed subassemblages SP$_1$ and GSL$_1$ represented a vast improvement over the non-ductile response of the as-built reference subassemblage O$_1$. The tests also showed that both the employed strengthening techniques were effective in transforming the brittle joint shear failure mode of the reference specimen O$_1$, into a more ductile failure mode of strengthened specimens, which developed flexural hinges in their beams. Damage to the strengthened specimens SP$_1$ and GSL$_1$ was concentrated both in the beam critical region and in the joint area.
Figure 7: Strength ratio of specimen GSL\textsubscript{1} to specimen SP\textsubscript{1}.

3. The performance of both the shotcrete jacket and cast-in-place jacket in strengthening of columns and beam-column joints of existing old frame structures was satisfactory.

4. It was also demonstrated that the behaviour of the shotcrete jacketed subassemblage GSL\textsubscript{1} was on par with the cast-in-place jacketed subassemblage SP\textsubscript{1}.
References


