RC MEMBERS STRENGTHENING BY LATERAL POST-TENSIONING OF EXTERNAL METAL STRIPS

H.MOGHADDAM [1], M.SAMADI [2], S.MOHEBBI[3]

***Mail Address*:** [1-2-3]Department of Civil Engineering, Sharif University of Technology,Tehran, Iran.
***E-mail*:** [1]moghaddam@sharif.edu, [2]*maysamsamadi@yahoo.com*,
[3]saiidmohebbi@civil.sharif.edu

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ABSTRACT The majority of concrete buildings in the regions of high seismicity in Iran do not meet seismic code requirement, and many of these buildings are vulnerable in severe earthquakes. Concerns for seismic strengthening of concrete grew considerably in the recent years, and has been accelerated since the Bam earthquake 2004. This paper presents the results of experimental and analytical study on the application of strapping technique for retrofitting of concrete columns. Experimental program included axial compressive tests on cylindrical and prismatic small-scale columns which were actively confined by pre-stressed metal strips. Test Results showed significant increase in strength and ductility of columns due to active confinement with metal strips. The effect of various parameters such as pretensioning force in the strip, number of strip layers wrapped around the specimens and spacing of confining strips on strength of concrete is studied. Nonlinear Finite element models of tested specimens were also made and analyzed. The observed stress-strain behaviors of columns with different levels of confinement are compared to those obtained from finite element method.

INTRODUCTION

Recent earthquakes have revealed an urgent need to develop retrofit techniques for the existing buildings and bridges designed in accordance with old seismic codes so as to meet the requirements of current seismic design standards. Some of the common problems revealed by earthquakes such as Kobe (Japan 1995), Athens (Greece 1999) and Kocaeli (Turkey 1999) include inadequate confinement of concrete, leading to shear, anchorage and splice failures.

It is well known and proven that lateral confinement improves the strength and ductility of concrete (Dilger et al., 1989). Confinement reinforcement is generally applied to compressive members as lateral reinforcement with the aim of increasing their strength and ductility. In addition, lateral confinement prevents slippage and buckling of the longitudinal reinforcement (Saadatmanesh et al., 1994). Lateral reinforcement can be provided by using circular hoops, rectangular ties, jacketing by steel , FRP, ferrocement, etc.

Because the total cost of replacement of the vulnerable structures is so overwhelming, the development of innovative rehabilitation and strengthening techniques is required to extend the life expectancy of many existing buildings and bridges. A number of repair and strengthening techniques are currently in use for reinforced concrete structures. Unfortunately, the majority of them is very expensive, time consuming and require the interruption of use of the structure whilst works are carried out. Hence, there is a pressing need for the development of improved, low cost, less disruptive techniques, which will make necessary interventions in many structures economically viable. It should be borne in mind that the cost of retrofitting buildings is the primary factor which deters many private owners from executing essential works. (Frangou & Pilakoutas 1995)

In this paper, an easy technique of retrofit of concrete, which was firstly presented and used by professor Pilakoutas at the University of Sheffield, is presented. The main aim of this research was quantification of the enhancement of concrete strength and ductility by the application of the technique. The results of experimental and analytical studies on performance of the technique are discussed. This study focused on application of this technique for high strength concrete.

The technique used for strengthening concrete columns in this study, involves post-tensioning high-strength packaging straps around the column (by using standard strapping machines used in the packaging industry) and subsequently locking their ends in metal clip.

Commercially available strapping tensioners and sealers make it easy to pretension the strip and fix the strip ends in the clamps. The available straps have widths of 10 to 50 mm and thicknesses of 0.5 to 1.12 mm. In terms of strength, high strength strips in excess of 10000 kg/cm^2, are available in the market. The strips are tensioned to 30 percent of their yield stress. Hence, an effective lateral stress is applied on the column prior to loading. This has many benefits such as full utilization of the strip capacity and prevention from premature crushing of the confined concrete, as would be the case with not properly tightened strips.

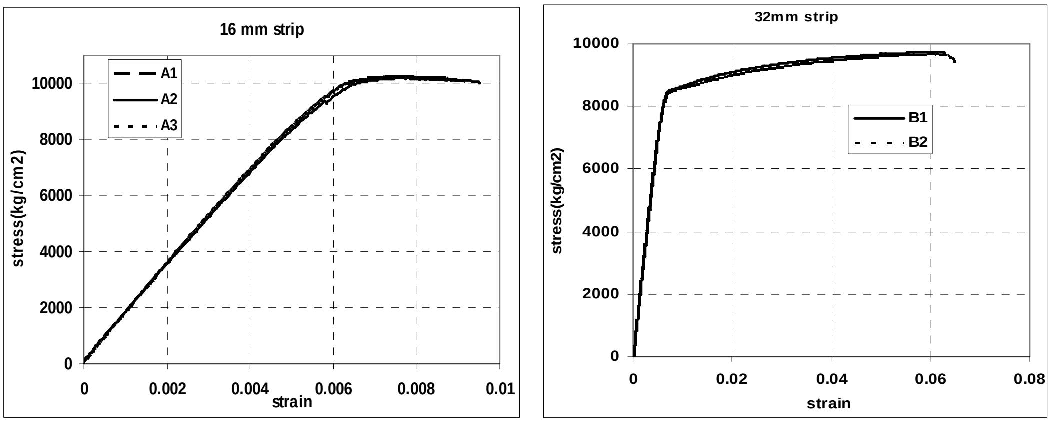
The low cost of strip and speed and ease of application of the strapping technique make this method efficient for use as a repair and strengthening technique for RC structural members. An RC column would normally require six man days' work to be jacketed whilst a maximum of two days' work is required for external strapping, which clearly demonstrates the cost saving when using the proposed technique. (Frangou & Pilakoutas 1995)

EXPERIMENTAL PROGRAM AND OBSERVATION

The concrete specimens were fabricated in the structure and concrete Laboratory at the building and housing research center. The material used for the concrete specimens included type I portland cement, local sand and gravel. The maximum size of the gravel was 12 mm. No additive was used in any of the mixes.

Experiments included 24 cylindrical and 14 prismatic concrete specimens. The column models were made of a relatively high-strength concrete with no air-entrainment. The concrete reached an average uniaxial compressive strength f'_c of about 50 MPa. The specimens were removed from the forms after 2 days and put into water to be continuously moist cured. The cylindrical and prismatic specimens were tested after 428 days after casting.

Two sizes of metal strips were used for strengthening of the specimens. The specimens were strengthened by using 16* 0.5 mm and 32*0.8 mm strips. in addition to the difference in width and thickness, the material behavior of the strips were also dissimilar. In Figure 1, the stress-strain behavior of the used strips, which were obtained from standard tensile tests, are shown. As can be seen in these figures, although both strips have similar strengths, the elongation of the strips, that is an important characteristic of the confining elements, is quite different. The 32mm wide strip has larger ductility making it more suitable for application as a confining element for concrete.



(a) (b)

Figure-1. Stress-strain relationship of used confining strips a) 16mm strip, b) 32mm strip

One of the important parameters in this study was to compare the active and passive external lateral confinements by this technique. In order to do so, some of the cylindrical specimens were tensioned only to 40 kg (which will be called passively confined specimens hereafter) while a tensioning force of 250 kg was applied in pretensioning the other specimens (which will be called actively confined specimens hereafter). In fact, the metal strips of the latter specimens are tensioned to 0.31 of their yield strain. Two pneumatic tensioners were used to strap the two strips.

The tensioning force in the strips was calibrated by means of special setups. Once the air pressure before the tensioner was set to a certain value, the tensioning force in the strip was monitored by means of a tensile load cell and a data logger. The following figures show the relationship between air pressure and the tensioning force in the 16mm and 32mm strips and the fitted regression lines. It can be seen that a linear relation exists between these parameters.

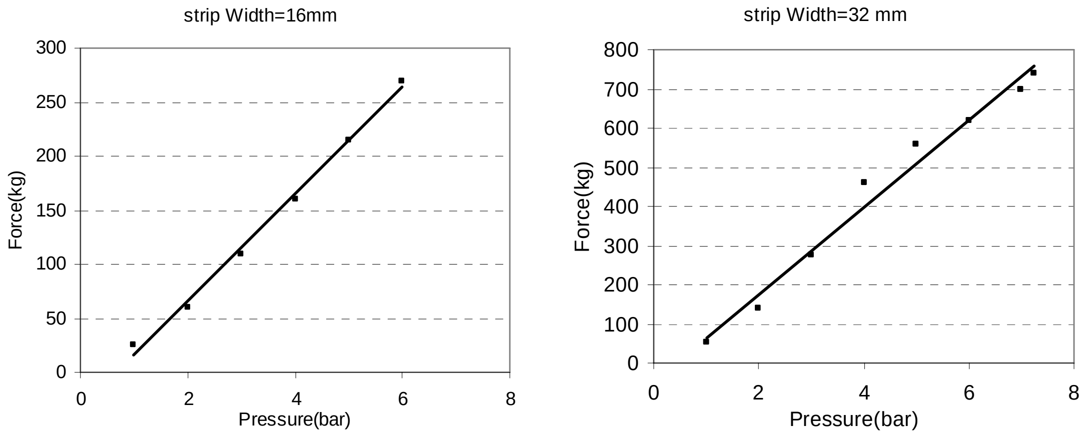


Figure-2. Results of calibration tests, the relation between tensioning force and air pressure

After tensioning the strip, the two ends of the strips were fixed together by sealing the clamp. This results in some loss in pretensioning force. The loss in tensioning force due to the sealing process was also monitored for some strips. The average percentage of losses was 19% and 31% for 16 mm and 32 mm strips, respectively.

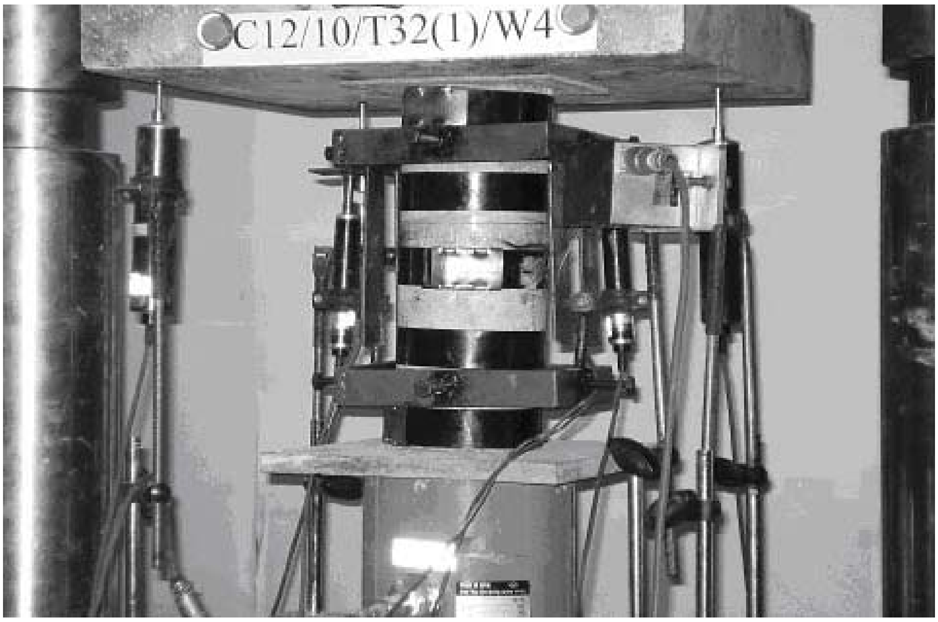


Figure-3. Setup of axial test of a cylindrical specimen

After the concrete column models had been cured, the metal strips were strapped around the specimens. Axial compression tests were conducted using a Tinius Olson testing machine with a capacity of 1,780 KN (400 kips) in the Concrete Laboratory of building and housing research center. The load was increased until significant strength decay was recorded, which indicates failure of the specimens.

A total of six displacement transducers from TML Company were made to obtain the longitudinal and transverse strains. Three 50 mm CDP displacement transducers were mounted on steel rods between top and bottom plates to measure the axial displacement of the column specimen top surface. In addition a special setup with two 25 mm CDP displacement transducers was made to measure the relative displacement over the middle 3/4 height of the specimens. Also a DP tape measure type displacement transducer was used to measure the circumferential strain of the specimens. Figure 3 illustrates the test setup for a typical loaded column specimen.

FLA-5-11 Strain gages of TML Company were attached to the strips to obtain the strain of strips during the test. A 200 ton load cell was located at the end of the specimen to measure the load at desired intervals together with other data.

The compressive tests were conducted on the column specimens. The axial load, with a load rate of 178 KN/min (40 kip/min), was increased monotonically until the column specimens failed. This loading rate is equivalent to 0.25 MPa/s (37 psi/s). The ASTM standard loading rate for compressive strength of cylindrical concrete specimens is within the range of 0.14–0.34 MPa/s (20–50 psi/s). Therefore, selected loading rate falls within the range of the ASTM standard.

Four major failure modes were observed at the end of the test of the column specimens, including cone, shear, cone-shear and cone-columnar.

The results obtained from strain gauges as well as displacement transducers were analyzed. It was observed that the axial stress and the confining pressure kept increasing until the value of lateral strain reached the yield strain of the strips in a circumferential direction. The specimens reached their maximum strengths when one or more of the strips yielded. After the peak stress, the strips ruptured one by one resulting in the loss of axial stress. Column specimens with two layers of the metal strips gained larger strengths as well as a larger ultimate axial strain as compared with column specimens with one layer of the metal strip.

The strength enhancement of cylindrical as well as prismatic specimens is drawn in the following figures versus the ratio of strips spacing to concrete diameter.

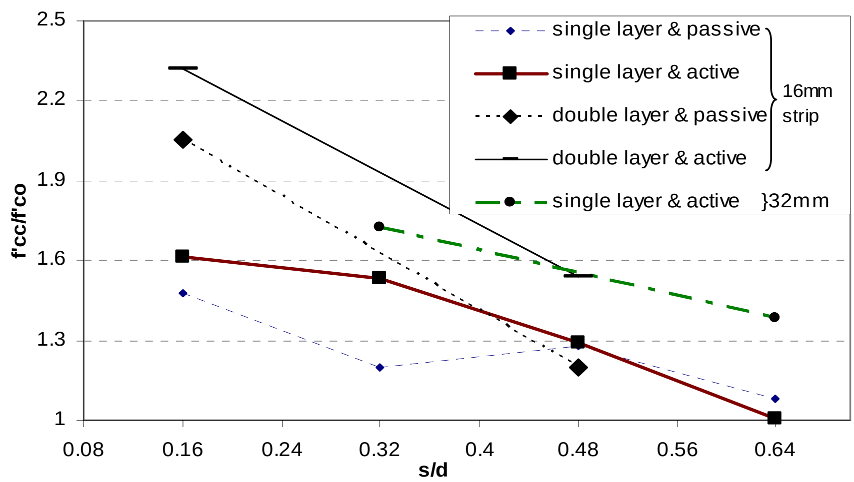


Figure-4. Strength enhancement of Cylinder specimens

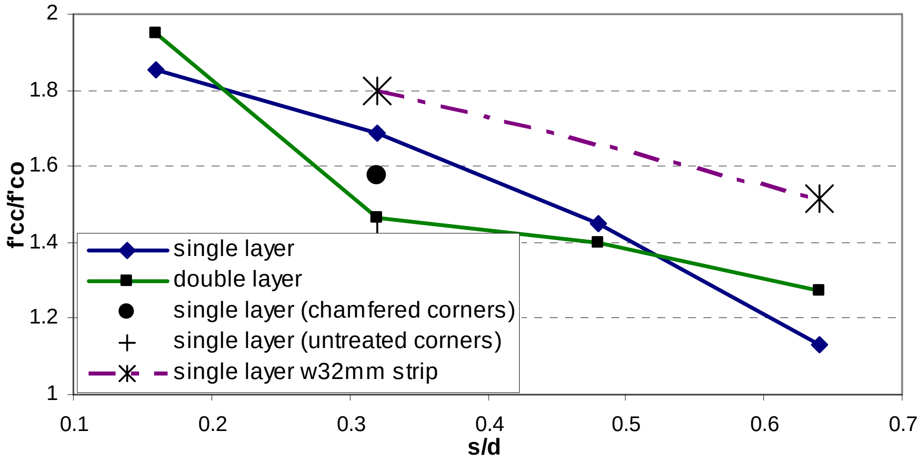


Figure-5. Normalized strength of prismatic specimens

It can be concluded from the figures that:

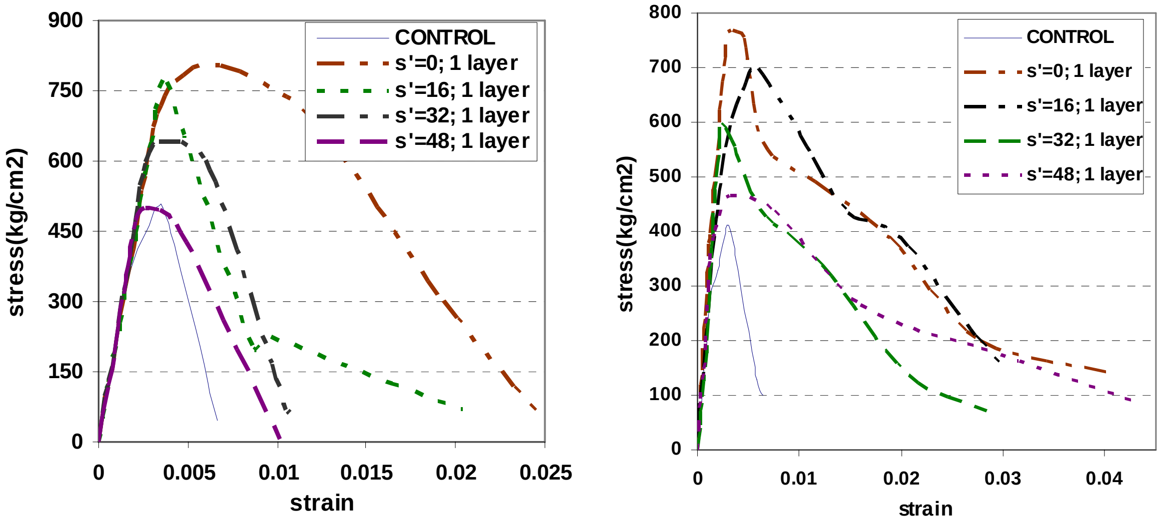
1) This technique has been able to increase the strength of concrete up to 2.3.

2) An increase in the spacing between the strips, has always led to increase in strength of confined concrete.

3) The concrete confined with double layer metal strips has generally shown better enhancement in concrete strength than confinement with single layer.

4) Active confinement resulted in more increase in concrete strength than the passive one this is mainly because whilst the ordinary passive confinement is mainly utilized after the core concrete has dilated (which means that some cracks have occurred in it); the active confinement influences the core concrete even before load application.

The observed stress-strain behavior of some of the test specimens are drawn in the following figures. In these figures, the results of specimens that were actively confined with only one layer of 16mm strip confined with strips with different clear spacings (s') are shown. Figure 6a shows the results of cylindrical specimens while in figure 6b, results of the prismatic specimens are shown.



(a)

(b)

Figure-6. Stress-strain behavior of actively confined concrete specimens; a) cylindrical and b) prismatic

NONLINEAR FINITE ELEMENT MODELING

Nonlinear Finite elemnt models of the tested specimens were made by using eight node solid elements in ABAQUS program. The concrete damaged plasticity model of the program was used for modeling the nonlinear behavior of concrete. This model uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete.
It consists of the combination of non-associated multi-hardening plasticity and scalar (isotropic) damaged elasticity to describe the irreversible damage that occurs during the fracturing process. Concrete damaged plasticity model requires that the elastic behavior of the material be isotropic and linear. The model is a continuum, plasticity-based, damage model for concrete. It assumes that the main two failure mechanisms are tensile

cracking and compressive crushing of the concrete material. The evolution of the yield (or failure) surface is controlled by two hardening variables, ε_t^{pl} and ε_c^{pl}, linked to failure mechanisms under tension and compression loading, respectively. ε_t^{pl} and ε_c^{pl} are tensile and compressive equivalent plastic strains, respectively.

The model assumes that the uniaxial tensile and compressive response of concrete is characterized by damaged plasticity.
Under uniaxial tension the stress-strain response follows a linear elastic relationship until the value of the failure stress, σ_{t0}, is reached. The failure stress corresponds to the onset of micro-cracking in the concrete material. Beyond the failure stress the formation of micro-cracks is represented macroscopically with a softening stress-strain response, which induces strain localization in the concrete structure.
Under uniaxial compression the response is linear until the value of initial yield, σ_{c0}. In the plastic regime the response is typically characterized by stress hardening followed by strain softening beyond the ultimate stress, σ_{cu}.

The degradation of the elastic stiffness is characterized by two damage variables, d_c and d_t, which are assumed to be functions of the plastic strains, temperature, and field variables. The damage variables can take values from zero, representing the undamaged material, to one, which represents total loss of strength.
If E_0 is the initial (undamaged) elastic stiffness of the material, the stress-strain relations under uniaxial tension and compression loading are, respectively:

$$\sigma_t = (1 - d_t) E_0 \left(\varepsilon_t - \tilde{\varepsilon}_t^{pl}\right)$$
$$\sigma_c = (1 - d_c) E_0 \left(\varepsilon_c - \tilde{\varepsilon}_c^{pl}\right)$$

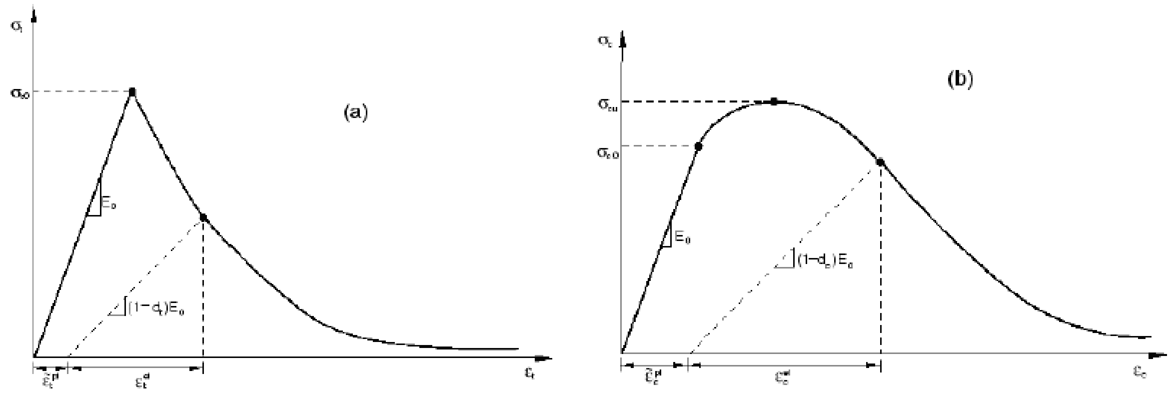


Figure-7. Schematic illustration of the model and its damage variables in Tension(a) and compression (b).

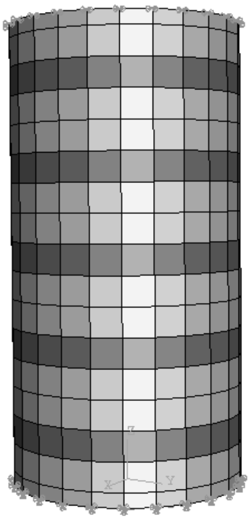
The stress-strain relations for the general three-dimensional multiaxial condition are given by the scalar damage elasticity equation:

$$\sigma = (1 - d) D_0^{el} : \left(\varepsilon - \varepsilon^{pl}\right)$$

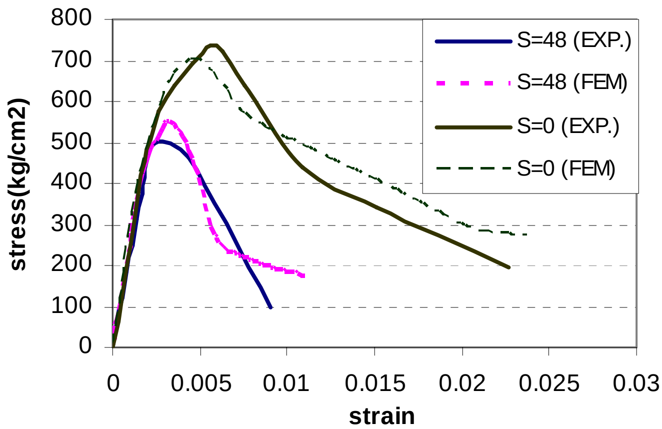
where D_0^{el} is the initial (undamaged) elasticity matrix.

The concrete damaged plasticity model assumes nonassociated potential plastic flow. The flow potential G used for this model is the Drucker-Prager hyperbolic function. The model makes use of the yield function of Lubliner et. al. (1989), with the modifications proposed by Lee and Fenves (1998) to account for different evolution of strength under tension and compression.

The FE models of tested cylindrical as well as prismatic specimens were made in ABAQUS. These models consisted of solid and shell elements for modelling concrete and strips, respectively. The abovementioned plasticity model was defined and used for solid elements . The observed stress-strain behavior of strips in tensile tests were defined as the material behavior of shell elements. The bottom surface of models are restrained and the load was applied by incrementally increasing the displacement of nodes of the top surface. Figure 8 shows the mesh details and results of nonlinear finite element models of some of tested cylindrical specimens compared with observed stress-strain behavior.



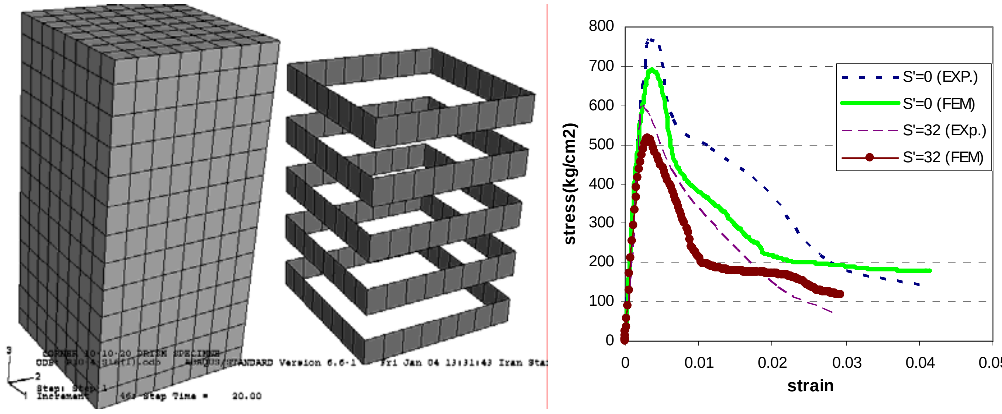
(A)



(B)

Figure-8. (A) Mesh details of a cylindrical model (B) Comparison of FEM &experiments

In Figure 8 analytical and experimental results for two columns with strips spacings of 48 and 0 mm are compared. As can be seen in this figure, there is relatively good agreement between analytical and experimental results. However, the NFEM has underestimated the experimental results. It should be mentioned that the pretensioning force in the strips in actively confined specimens have been applied in the model before application of the incremental displacement.



(A) (B)

Figure-9. (A) Elements of a prismatic model (B) Comparison of FEM &experiments

Similarly in figure 9, mesha details for steel and concrete elements and the results of nonlinear finite element models of some of the prismatic specimens are shown. A comparison between analytical and experimental results show that the nonlinear finite element method has underestimated the post-peak part of stress-strain behavior of confined specimens.

CONCLUSIONS

The applied technique for strengthening of concrete columns could increase strength, ductility of concrete considerably. The technique was able to increase the peak strength and its corresponding strain of concrete up to 230 percent. The gain in ductility of confined concrete was very sensitive to the ductility of the metal strip used. Active confinement resulted in better enhancement of strength and ductility of confined concrete than passive confinement. The efficiency of confinement in cylindrical specimens, i.e. the gain in strength and ductility, was greater than that of prismatic ones. The damaged plasticity model was capable to estimate the behavior of confined concrete with reasonable accuracy. However it underestimates the results of both actively and passively confined concrete.

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