Retrofitting of Square Reinforced Concrete Columns with Steel Jackets

By

Khair Al-Deen Isam Khair Al-Deen Bsisu

Supervisor

Prof. Samih Shaker Qaqish

Submitted in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy in Civil Engineering

Faculty of Graduate Studies
University of Jordan

August, 2002

Committee Decision

This thesis was successfully defended and approved on: 22 / 8 / 2002

Examination Committee:	Signature
Dr. Samih S. Qaqish, Chairman Prof. of Civil Engineering	Samih Qaqish
Dr. Bassam N. Abu-Ghazaleh, Member Prof. of Civil Engineering	
Dr. Abdelqader S. Najmi, Member Prof. of Civil Engineering	
Dr. Nazzal S. Armouti, Member Assistant Prof. of Civil Engineering	
Dr. Khaled Z. Kahhaleh, Member Director, Building Research Center, Royal Scientific Society, Jordan	Khaled Kahhaleh

To

My mother, my wife, and my children

ACKNOWLEDGMENT

The guidance, encouragement and assistance of Prof. Sameh Qaqish in conducting this research are deeply appreciated.

Special thanks are also expressed to members of the examining committee, Prof. Bassam Abu-Ghazaleh, Prof. Abdelqader Najmi, Dr. Nazzal Armouti and Dr. Khaled Kahhaleh for their valuable reviews.

Thanks to Mr. Zuhair Shaker for his help and assistance while performing the experimental work.

Table of Contents

LIST OF TABLES

Retrofitting of Square Reinforced Concrete Columns with Steel Jackets

By
Khair Al-Deen Isam Bsisu

Supervisor
Prof. Samih Shaker Qaqish

ABSTRACT

Strengthening reinforced concrete columns to resist increased loads by retrofitting with steel jackets is common engineering practice for strengthening and repair of columns.

Not only steel jacketing is inexpensive, the technique dose not require highly trained labor, it is unobtrusive, does not reduce space, easy to inspect and can be applied whilst the structure is still in use. Steel jacketing can be used to repair existing structures, to prolong their design life, and to upgrade structures for alternative use and additional loading.

In this desertation 39 square reinforced concrete columns are tested to investigate retrofitting with steel jackets technique and design procedures. This will provide Jordanian engineers with theoretical and experimental verification of the technique and allow them to develop retrofitting designs with ease and confidence.

Confinement models from previous researchers conducted between 1928 and 2001 were compared with experimental results. Conclusions were drawn toward the best model to be used for estimating the strength of confined square reinforced concrete columns with steel jackets not extending to the full height of the column. An equation to determine the thickness of the steel jacket to effectively confine concrete was derived.

Tests showed that retrofitting square reinforced concrete columns with full-height steel jackets enhanced the strength of the tested columns such that the retrofitted specimens behaved in a similar manner to the concrete filled steel tube columns. The ACI-318M-99 and LRFD design equations for concrete filled steel tubes were compared with experimental results of retrofitted square reinforced concrete columns with full-height steel jackets and with concrete filled steel tube columns to verify the use of this equation for design of retrofitting systems. A computer model utilizing the SAP-2000 program was also devised, the results of which were agreeable with the experimental results.

Eccentrically loaded columns confined with steel jackets not extending to the full height of these columns had better ductility than the original reinforced concrete columns and limited increase in the load carrying capacity, while the columns retrofitted with full-height jackets had a higher load carrying capacity as well as increased ductility.

Notation

A_c cross sectional area of concrete

A_j cross sectional area of steel tube (jacket).

A_m cross sectional area of model

A_p cross sectional area of prototype

A_s cross sectional area of reinforcing steel bars.

A_{sp} area of spiral bar

b_c core dimension to centerline of perimeter hoop ($b_c \le d_c$)

D the diameter of the column

d_c core dimension to centerline of perimeter hoop ($b_c \le d_c$)

d_s diameter of spirals between the centers, Figure 2.3.

E_c modulus of elasticity of concrete

E_j modulus of elasticity of jacket material

E_m modulus of elasticity of model

E_p modulus of elasticity of prototype

E_s steel modulus of elasticity of the steel jacket

E_{sj} modulus of Elasticity of steel tube (jacket).

f_c axial stress in concrete

f_c' 28 day cylinder crushing strength of concrete

f_{cc}' confined concrete stress

f_{co}' unconfined concrete stress

f_j the stress induced in the jacket.

f_{jy} yield strength of steel tube (jacket).

f_l confining pressure

f_o reference plastic stress at the intercept of the second slope with the stress axis in the Mirmiran stress-strain relationship.

f_y yield strength of reinforcing steel bars.

f_{yh} yield strength of transverse hoop reinforcement

K model to prototype scale factor

k_1 confinement coefficient (Richart Model)

k_2 empirical coefficient = $5 \cdot k_1$ (Richart Model)

K_e confinement effectiveness coefficient (Mander Model)

k_f effectiveness of confinement = $\frac{r_f}{25}$ (Proposed equation for steel jacket thickness)

K_l effective length factor

l laterally unbraced length of member

L_m length of model

L_p length of prototype

n a curve shaped parameter

P_n nominal axial strength of column

P_o maximum nominal axial strength of column.

P_u ultimate axial strength of column

r governing radius of gyration about the axis of buckling

R radius of the steel tube (jacket).

r_f radius of fillet of steel jacket

s' clear spacing between spirals or hoop bars, Figure 2.4.

s spiral spacing or pitch

t_j the steel jacket (tube) thickness

w_i the *i*-th clear distance between adjacent longitudinal bars.

α effectiveness of confinement ratio

β an empirical coefficient that depends on the slenderness ratio

ε_c axial strain in concrete.

ε_{cc} confined concrete stress

ε_{co} unconfined concrete stress

ε_x stress in the *x* direction

ε_x strain in the *x* direction

ε_y strain in the *y* direction

ε_y stress in the *y* direction

ε_z strain in the *z* direction

ε_z stress in the *z* direction

λ_c column slenderness parameter

ν Poisson's ratio

ρ_c ratio of area of longitudinal reinforcement to area of core concrete.

ρ_{cc} the ratio of area of longitudinal reinforcement to area of core concrete

ρ_s ratio of volume of transverse confining steel to volume of confined concrete core

σ normal stresses

σ_m stresses in model

σ_p stresses in prototype

τ shearing stresses

Φ_c resistance factor for axially loaded composit columns (LRFD)

ω_w volumetric mechanical ratio of the confining material to the volume of concrete

1. INTRODUCTION

Strengthening reinforced concrete columns to resist increased loads by retrofitting with steel jackets is common engineering practice for strengthening and repair of these columns. Ordinary reinforced concrete columns are brittle elements, their failure is sudden and catastrophic, therefore columns in reinforced concrete construction should be treated with extreme care to prevent any change in loading imposed and to make certain that materials used reach their design strengths. Consequently, retrofitting of any column should be considered at the smallest evidence of anticipated insufficient strength of materials or change in imposed loads.

Occasionally there have been reports about collapse of multi-story reinforced concrete buildings in Egypt resulting in the unfortunate loss of life in addition to the loss of properties (Kholusi, 1992), Figure 1.1.

Figure 1.1. The collapse of a building in Mansoura 132 km north of Cairo, Al-Ahram Weekly On-line issue number 576, 7-13 March 2002

Recently there had been limited incidents of such failures in Beirut, fortunately with no casualties, Figure 1.2, as well as reports about a couple of buildings under construction in Amman.

Figure 1.2 A multi-storey building collapsed in a crowded commercial district Beirut, BBC on-line, March 23, 2002.

More and more structural engineers are being forced to look at structural columns from a maintenance point of view to deal with cases of strengthening existing columns in old as well as new buildings for the following reasons:

1- New structures that may include unsafe columns due to bad workmanship or due to errors in modeling and design. Such cases, although not very frequent, have to be dealt with taking into consideration the need to preserve the shape and size of the column without altering the intended functional use of the structure and at the same time without compromise to the structural integrity and safety of the structure.

2- The need to place additional loads on columns due to the change in building regulations, this includes either the permission to add more floors, or the change of the allowed occupational use of the structure. Such changes are known to happen, especially in largely populated area. As an example; in Cairo, Egypt, some building owners added several stories to their existing buildings while no effective strengthening of columns at the lower stories were applied, which led to the failure of few buildings, while others still poise as an imminent danger. Recently, in Amman, there has been a tendency towards increasing the number of stories, or the occupational use of existing buildings, such cases should be studied to ensure structural safety of load bearing elements.

3- Aging of old structures due to deterioration of concrete, corrosion of reinforcing steel bars or both, which leads to the loss of strength of columns and the inability to carry design loads. These structures may be of historical or monumental values and could be considered as part of our heritage, or they could be ordinary structures that simply cost less to repair and maintain than to demolish and reconstruct.

4- Occasionally some structures, or part of them, are subjected to accidents, such as fire or a car collision with one or more of the columns in a car park or a highway bridge, which leads to reduction of column carrying capacity. Therefore we are faced by the choice between repair and demolition, this decision might depend on the scale of the damage, and the effectiveness of the strengthening technique.

This led to the search for an established, reliable, economical, and easy to apply method. Retrofitting of circular columns is being recommended for strengthening of such columns in some design manuals, while square and rectangular columns are being considered on a case by case basis. Since most of the columns in residential and office buildings in Jordan are of square or rectangular shapes, it could be concluded that there is a need for such a strengthening technique of non-circular columns.

There is more than one method of retrofitting reinforced concrete columns (UNDP/UNIDO, 1983), some of the methods are:

1. Using reinforced concrete jackets, Figure 1.3, this is achieved by increasing the section of the reinforced concrete column through adding additional reinforcement in the longitudinal as well as the transverse directions, making proper connection between the old and new reinforcement, then, placing of formwork, and finally casting of concrete.

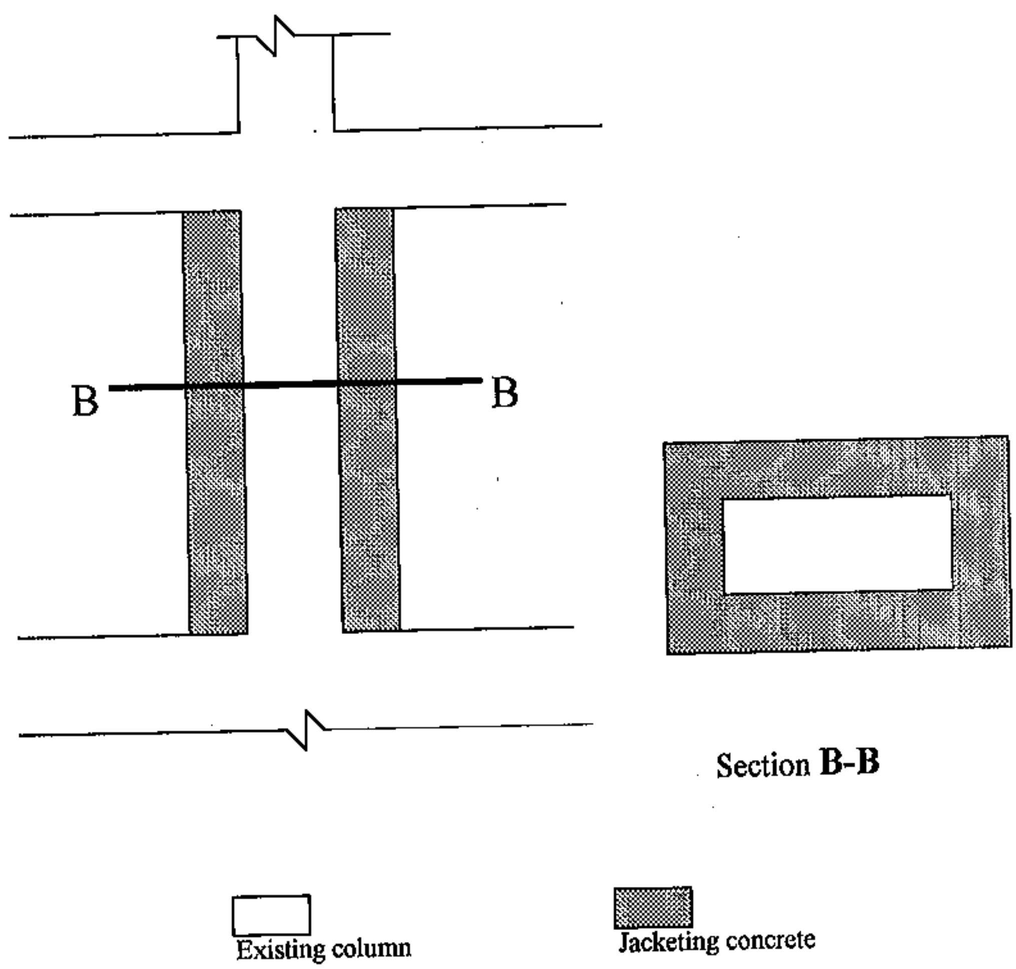


Figure 1.3. Jacketing existing reinforced concrete columns with reinforced concrete jacket.

This method is effective, and economical, and has been applied successfully to several cases, still it will consume a lot of space and will in most cases need extensive labor, and could require the evacuation of the structure.

2. Steel jacketing, either full or partial, Figure 1.4, where the steel jacket is manufactured at the workshop, and applied in a fast and almost clean procedure of welding and grouting. The steel jacket must be protected against fire hazard by applying a special layer that will provide for at least 2-hours of protection against fire.

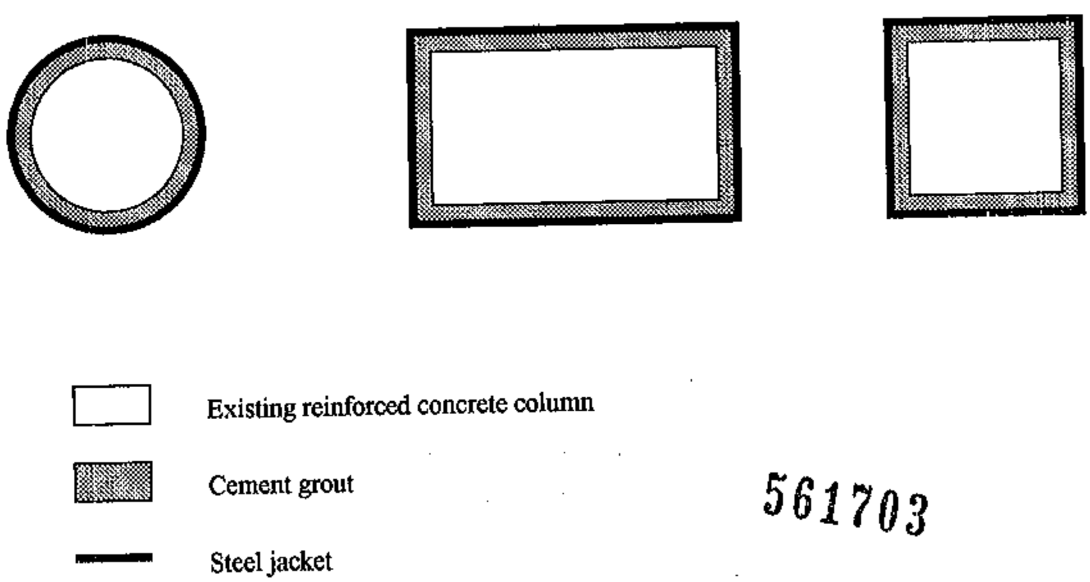


Figure 1.4. Steel Jacketing of Reinforced Concrete Columns

3. Wrapping the columns with fiber reinforced plastics (FRP). This fairly new method which is still under development, is effective and does not consume space, but it is expensive and require the use of

special chemicals, such as bonding epoxies, and trained labor. FRP possess very high tensile strength, but it's a brittle material, which means that although the FRP wrap will increase the strength of the retrofitted columns through confinement, it will contribute less to their ductility. Moreover some FRP materials are still undergoing experimental study and evaluation (Uomoto, 2001).

Retrofitting columns using steel jackets, seemed to be the solution to several cases of repair, maintenance and strengthening of square building columns for the following reasons:

1- The steel jacket does not obstruct or consume a lot of space, since the steel jacket itself is thin and should be as close as possible to the concrete column to effect confinement.

2- Steel jacketing can be applied while the building is functional, no need for partial or full evacuation.

3- Steel jacketing does not require highly trained labor; it could be applied using normal skilled and unskilled labor with no additional training or expertise.

4- Steel jacketing does not require the use of special equipment for wrapping or placement, just the welding machine available at every construction location.

5- Steel jacketing is cheap and cost effective, materials used are ordinary building materials that are available in the local market and could be purchased at regular market price.

Retrofitting columns using steel jacketing have been shown to effectively strengthen existing circular reinforce concrete columns by confining the concrete. However jacketing is less effective in confining concrete within square or rectangular shaped columns, where it has been dealt with on a case by case basis since there is no established guidelines for their design although several models for the prediction of the confined concrete strength were proposed.

Interaction between the steel jacket and the reinforced concrete column is studied and the retrofitted column ultimate carrying capacity is evaluated. It is suggested that the bond between the steel jacket and the reinforced concrete column developed by the cement grout is sufficient to form a composite action between the two materials similar to that of the concrete filled steel tubes.

To investigate this retrofitting technique thirty nine specimens were studied. Ten specimens of reinforced concrete columns retrofitted with full steel jackets to test the full steel jacket retrofitting system. Ten specimens were jacketed to test the retrofitting by confinement with steel jackets system. Five specimens were jacketed and tested to determine the effectiveness of the cement grout to develop bond between the reinforced concrete column and the steel jacket. Ten specimens were tested without retrofitting to determine the strength of unretrofitted columns. And four concrete filled steel tube columns were tested to verify that the behavior of the full steel jacket system is the same as composite concrete filled steel tube columns.

Confinement models from previous research conducted between 1928 and 2001 were compared with experimental results, and conclusions drawn toward the best model to be used for estimating the strength of confined square reinforced concrete columns with steel jackets.

The available equation for the thickness of the steel jacket (tube) in the literature was not convincing to several researchers, hence, an equation to determine the thickness of the steel jacket to effectively confine concrete was derived analytically. Tests as well as empirical formula found in a previous research showed that this derived equation produced acceptable results.

Tests showed that retrofitting square reinforced concrete columns with full-height steel jackets enhanced the strength of the tested specimens such that the retrofitted specimens behaved in a similar manner to the concrete filled steel tube columns. The ACI-318M-99 and LRFD design equations for concrete filled steel tubes were compared with experimental results of retrofitted square reinforced concrete columns with full-height steel jackets and with concrete filled steel tube columns to verify the use of this equation for design of retrofitting systems for square reinforced concrete columns.

Eccentrically loaded columns confined with steel jackets not extending to the full height of the columns had better ductility than the original reinforced concrete columns and limited increase in the load carrying capacity, while the columns retrofitted with full-height jackets had a higher load carrying capacity as well as increased ductility.

2. Literature Review

When designing a structural element, the applied forces are first calculated, then the dimensions and material properties are specified. In the case of retrofitting a structural column the designer is limited by the dimensions and strength of the existing concrete column, which has insufficient strength to resist applied loads, the strengthening technique proposed requires the addition of lateral confinement force to increase the load carrying capacity of the element.

Retrofitting and strengthening of reinforced concrete columns is based on the well-established fact that lateral confinement of concrete can substantially enhance its axial compressive strength. Theoretical and experimental studies have been carried out yielding reliable models for the prediction of the axial compressive strength of confined concrete. Much of the research on the retrofitting of reinforced concrete columns has concentrated on steel jacketing and proved it can effectively confine the concrete of the circular shaped columns. However, steel jacketing of rectangular and square shaped columns has not been thoroughly investigated and attempts have been made to extend the same retrofitting technique of circular columns.

that plane. The limiting values of the normal stresses σ and shearing stresses τ are interdependent; thus, the limiting value of the shearing stress, above which failure occurs, is a function of the simultaneous value of the normal stress on the same plane, as shown in Figure 2.1.

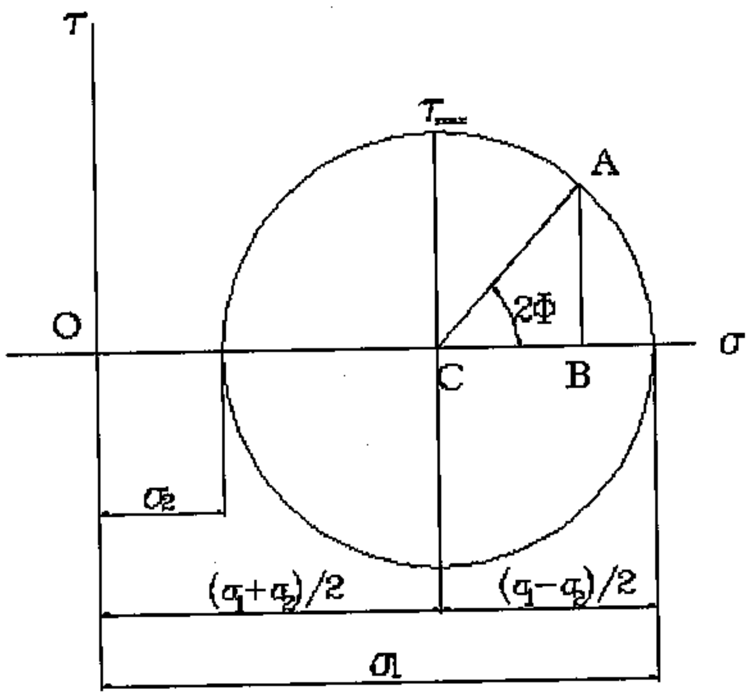


Figure 2.1. Construction of Mohr's Circle

For other values of σ_1 and σ_2 similar circles could be drawn, as shown in Figure 2.2, and it is evident that the curve representing $f(\sigma)$ must be tangent to all of these circles. This curve represents a limiting curve above which no state of stress for the material can exist.

Richart *et al.* (1928) developed a theory of failure of a material composed of non-isotropic elements using Böker analysis as basis. The

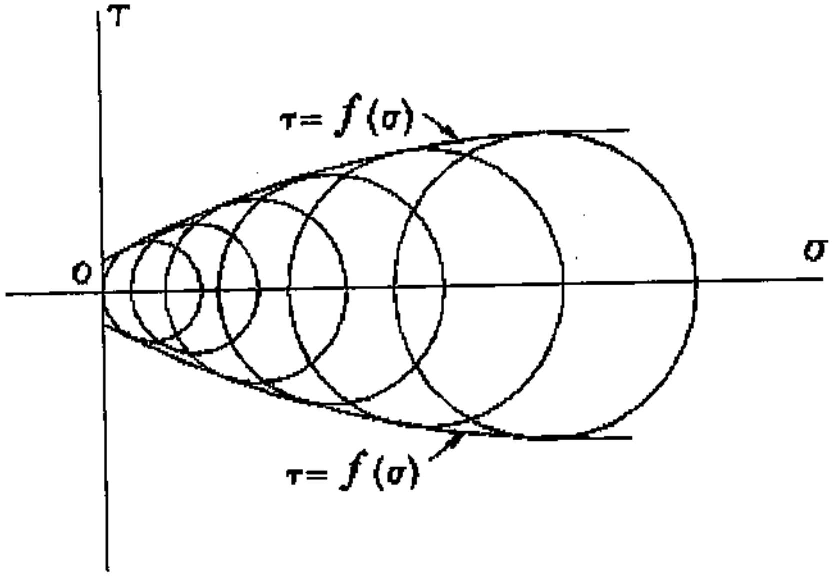


Figure 2.2. Relation of Limiting Curve to Mohr's Circle

hypothetical anisotropic material is assumed to be a continuous perfectly elastic mass made up of small elements, each element has a plane of weakness or susceptibility to sliding failure in a particular direction. These "directions of weakness" are assumed to vary arbitrarily from one element to another. With the application of simple compression to the assumed material, the intensity of stress is uniform throughout all elements and all deform elastically. When increasing the load, few elements having directions of weakness most favorable to the development of a sliding failure undergo a plastic sliding deformation, these elements are arbitrarily scattered throughout an otherwise elastic material, and their deformation is governed by that of the adjoining elements. As the load and the consequent shearing stress increase, other slightly less favorably situated elements undergo plastic deformation, though as long as the material remains continuous all elements must deform equally. As plasticity spreads throughout the material with increased load, the material reaches a stage of rapid breaking down, and a general sliding may occur in the direction of the angle of internal friction of concrete.

2.3 Confined Concrete Models:

Several researchers underwent experimental and theoretical studies in order to develop methods to estimate the strength of confined reinforced concrete, some of the findings of these researches are presented herewith:

2.3.1 The Richart Model:

Richart *et al.* (1928) established a model for predicting the strength of confined concrete, they performed tests on ninety six concrete cylindrical specimens 100x550mm and sixty four cylindrical specimens 100x200mm loaded axially to failure while subjected to two and three-dimensional confining fluid pressure. They found out that the presence of lateral pressure added to the strength of the specimens and proposed the following relations:

$$f'_{cc} = f'_{co} + k_1 \cdot f_l \qquad \text{.....................(2-4)}$$

where; f'_{cc} = confined concrete stress

f'_{co} = unconfined concrete stress

f_l = confining pressure

k_1 = confinement coefficient = 4.1 (Richart et al.)

And; $$\varepsilon_{cc} = \varepsilon_{co}\left(1 + k_2 \cdot \frac{f_l}{f'_{co}}\right) \qquad \text{..................(2-5)}$$

where; ε_{cc} = confined concrete strain

ε_{co} = unconfined concrete strain

$k_2 = 5 \cdot k_1$

Researchers later on investigated the Richart model and some of them suggested different values for k_1. According to Park and Pauly (1975) tests by Balmer (1944) have given values for k_1 in the range between 4.5 and 7 with an average value of 5.6.

Ben-Zvi *et al.* (1966) suggested that the confining pressure is only activated when the concrete column expands laterally under axial load, which they labeled: "passive stress". This "passive stress" is limited by the strain capacity of concrete. Accordingly they suggested applying a prestressing force to the jacket in order to induce the desired confinement force. Still they used the Richart model with a slight modification by introducing a new factor β, an empirical coefficient that depends on the slenderness ratio. The Richart model becomes according to Ben-Zvi *et al.*:

$$f'_{cc} = f'_{co} + \beta \cdot k_1 \cdot f_l \qquad \text{.................(2-6)}$$

where, $0 \leq \beta \leq 1.0$

Lam and Teng (2001) reviewed existing models for axial strength of confined circular concrete columns and found out that there is a clear overall linear relationship between the compressive strength and the lateral confining pressure. Therefore they concluded that the additional complexities in representing this relationship in the existing models appears to be unnecessary. They restated the Richart model with the confinement coefficient $k_l = 2.0$.

The Richart model was adopted by several reinforced concrete textbooks, such as; Ferguson (1967); Park and Pauly (1975); and Nilson and Winter (1991).

2.3.2 The Mander Model

Mander *et al.* (1988) have suggested another model for confined concrete strength based on the effectiveness coefficient. The Seismic Retrofitting Manual for Highway Bridges later adopted this model for circular columns. Although the model has been developed for both circular and rectangular shaped columns, the manual only adopted the part that dealt with circular columns, and suggested the use of an elliptical shaped steel jacket for rectangular columns, therefore the same design equation for both circular and rectangular columns.

Mander *et al.* (1988) tested thirty-one circular, square and rectangular reinforced concrete columns, the circular sections contained longitudinal and spiral reinforcement, the square sections contained longitudinal and spiral reinforcement, and the rectangular sections contained longitudinal reinforcement and rectangular hoops. Based on the results of his tests he arrived at the following model:

$$f'_{cc} = f'_{co}\left(-1.254 + 2.254\cdot\sqrt{1+\frac{7.94\cdot f'_l}{f'_{co}}} - 2\cdot\frac{f'_l}{f'_{co}}\right) \quad \text{..........(2-7)}$$

where; f'_{cc} = confined concrete compressive strength

f'_{co} = unconfined concrete compressive strength

$$f_l' = \frac{1}{2} \cdot K_e \cdot \rho_s \cdot f_{yh} \qquad \text{.................(2-8)}$$

$$\rho_s = \frac{4 \cdot A_{sp}}{d_s \cdot s} \qquad \text{..................(2-9)}$$

f_{yh} = yield strength of transverse hoop reinforcement

A_{sp} = area of spiral bar

s = spiral spacing or pitch, Figure 2.3

d_s = diameter of spirals between the centers, Figure 2.3

The effectively confined core for circular columns is shown in Figure 2.3, and the confinement effectiveness coefficient K_e is defined for circular hoops as follows; $K_e = \dfrac{\left(1 - \dfrac{s'}{2d_s}\right)^2}{1 - \rho_{cc}}$ ....(2-10)

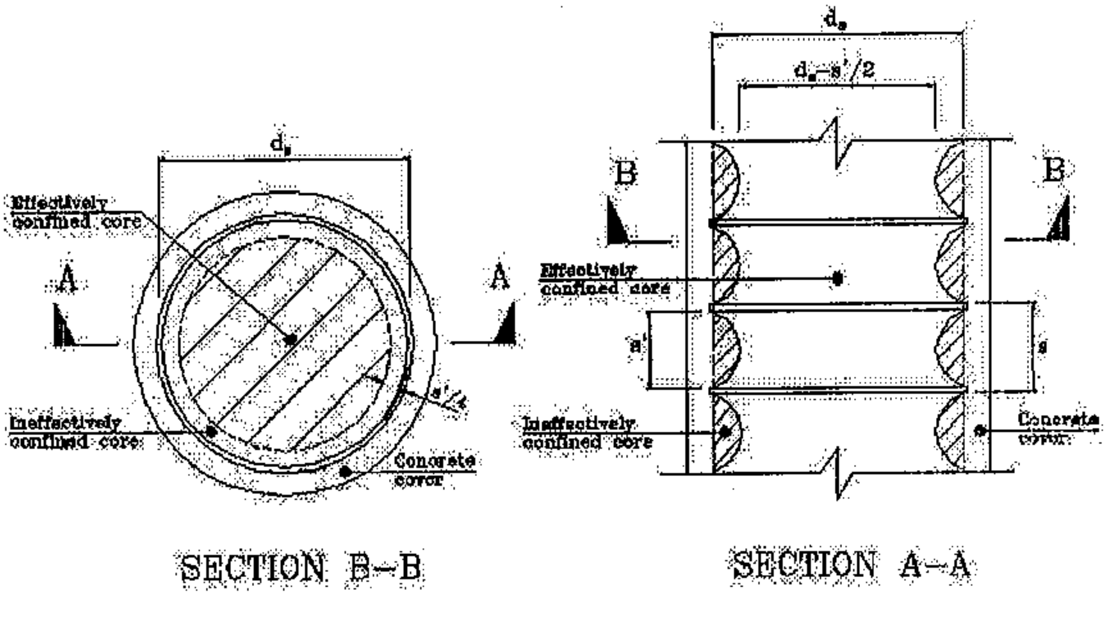


Figure 2.3. Effectively confined core for circular reinforced concrete columns

Similarly, for circular spirals; $K_e = \dfrac{1 - \dfrac{s'}{2 \cdot d_s}}{1 - \rho_{cc}}$ ....(2-11)

Where; s' = clear spacing between spirals or hoop bars, Figure 2.3.

d_s = diameter of spirals between the centers, Figure 2.3.

ρ_{cc} = the ratio of area of longitudinal reinforcement to area of core concrete.

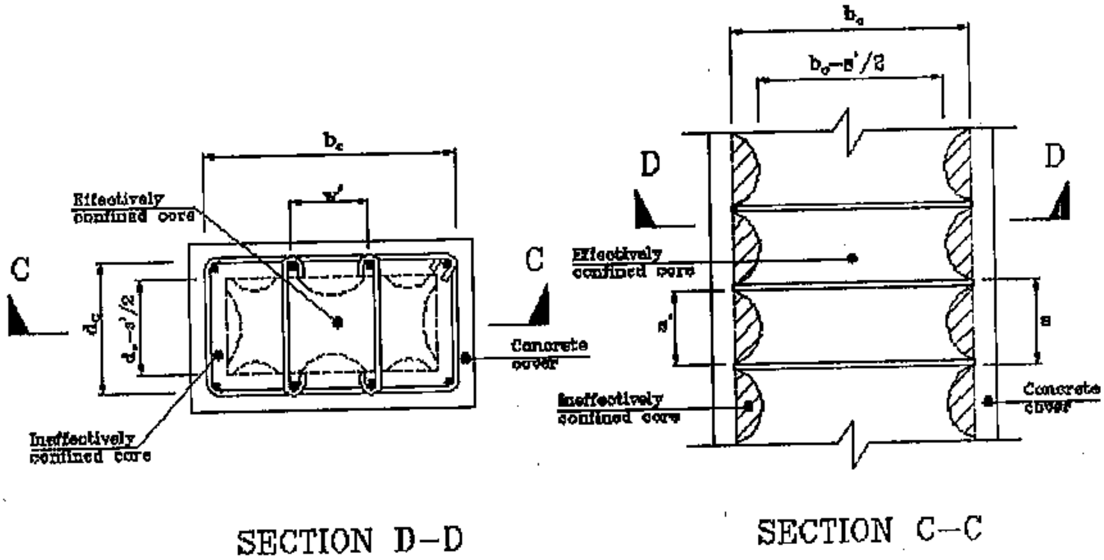


Figure 2.4. Effectively confined core for rectangular shaped reinforced concrete column.

The effectively confined core for rectangular columns is shown in Figure 2.4 and the confinement effectiveness coefficient is defined as;

$$K_e = \frac{\left(1 - \sum_{i=1}^{n} \frac{(w_i')^2}{6 \cdot b_c \cdot d_c}\right) \cdot \left(1 - \frac{s'}{2 \cdot b_c}\right) \cdot \left(1 - \frac{s'}{2 \cdot d_c}\right)}{(1 - \rho_{cc})} \qquad \text{...........(2-12)}$$

where;

ρ_{cc} = the ratio of area of longitudinal reinforcement to area of core concrete.

$w_i =$ the i-th clear distance between adjacent longitudinal bars.

b_c and $d_c =$ core dimensions to centerline of perimeter hoop in x and y directions ($b_c \le d_c$)

2.3.3 The Seismic Retrofitting Manual for Highway Bridges

As mentioned earlier, in 1995 the Seismic Retrofitting Manual for Highway Bridges partially adopted the Mander model describer above, which was originally developed for hoop confinement, to determine the confined strength of concrete.

One of the manual's techniques for retrofitting reinforced concrete columns is steel jacketing. This is achieved by assembling two half-shells of steel plate rolled to the column radius with a 13 to 25 mm clearance, then site welded along the vertical seems. The gap between the jacket and the columns is filled with cement grout. The jacket is then considered to be equivalent to continuous hoop reinforcement, Figure 2.5, and the confinement stress is found by:

$$f_l \cdot D = 2t_j \cdot f_j \qquad \text{............(2-13)}$$

where, $D =$ the diameter of the column

$f_l =$ confining pressure

t_j = the steel jacket thickness

f_j = the stress induced in the jacket.

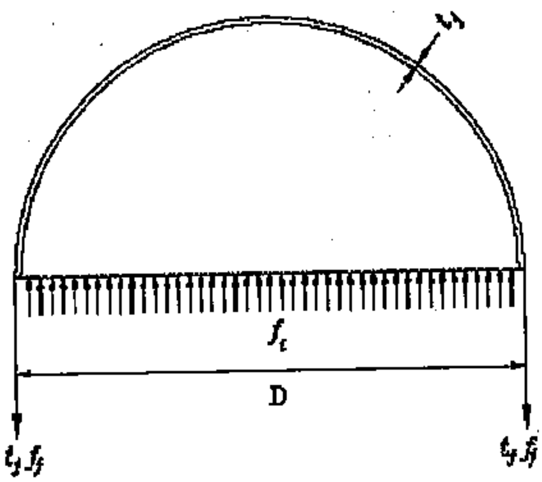


Figure 2.5. Confinement of Circular Columns

Assuming the steel modulus of elasticity of the steel jacket is E_s=200 GPa, then at concrete dilation strain $\varepsilon_d = 0.001, f_j = 200$ MPa, then Eq. 2-13 becomes:

$$t_j = \frac{f_l D}{400} \qquad \text{.................(2-14)}$$

and the confined strength of concrete can now be found by Eq. 2-7 which is the basic Mander model equation:

$$f'_{cc} = f'_{co}\left(-1.254 + 2.254\cdot\sqrt{1+\frac{7.94\cdot f'_l}{f'_{co}}} - 2\cdot\frac{f'_l}{f'_{co}}\right) \qquad \text{..........(2-7)}$$

The manual gives an approximate value for the confined strength of concrete for low-to-moderate confinement ratios, which is:

$$f'_{co} = 1.5 f'_c \qquad \text{.....................(2-15)}$$

The design criteria for rectangular columns consists of equation for the elliptical circumference of the steel jacket shown in Figure 2.6 are:

$$\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1 \qquad \text{.....................(2-16)}$$

The extreme radii in the two principal directions are:

$$R_1 = \frac{b^2}{a} \quad , \quad R_2 = \frac{a^2}{b} \qquad \text{.................(2-17)}$$

and a reasonable approximation for the jacket radius is suggested to be taken as:

$$R = \frac{R_1 + R_2}{2} \qquad \text{.................(2-18)}$$

The manual then suggested that the equations for the circular column be used, substituting $D = 2R$.

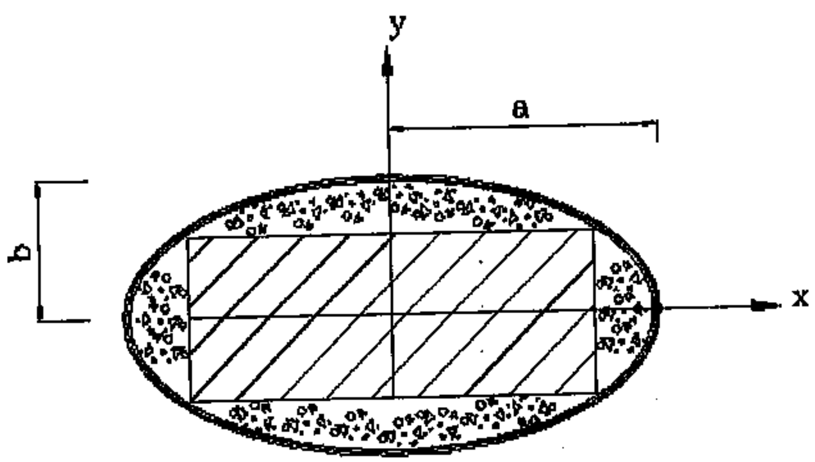


Figure 2.6. Elliptical jacket retrofit for rectangular columns

Hipley (1997) based on his personal experience, reported that elliptical steel casing of rectangular reinforced concrete columns may have annulus gaps in the shell more than four inches (100 mm), therefore pea-gravel and sand should be added to the grout in order to reduce shrinkage. Moreover it is difficult to maintain the elliptical shape under the fluid pressure of the grout, the long side tries to push outwards by the fluid pressure and the short side will come inward.

2.3.4 The Mirmiran Model

Mirmiran *et al.* (1998) suggested a model for confined concrete which is essentially a modified form of the Richart model described above.

A total of 30 - 152.5x305 mm cylindrical specimens were tested under uni-axial compression. Three concrete batches and three jacket thicknesses were used. The confining jacket used was from fiber composites.

The ultimate strength of confined concrete is taken as;

$$f'_{cc} = f'_{co} + 6 \cdot f_l^{0.7} \qquad \text{.............(2-19)}$$

where; f'_{cc} = the ultimate strength of confined concrete

f'_{co} = peak strength of unconfined concrete

f_l = confining pressure = $\frac{2 \cdot f_j \cdot t_j}{D}$ (Equation 2-13 of the Seismic Retrofitting Manual for Highway Bridges)

f_j = hoop strength of jacket

t_j = jacket thickness

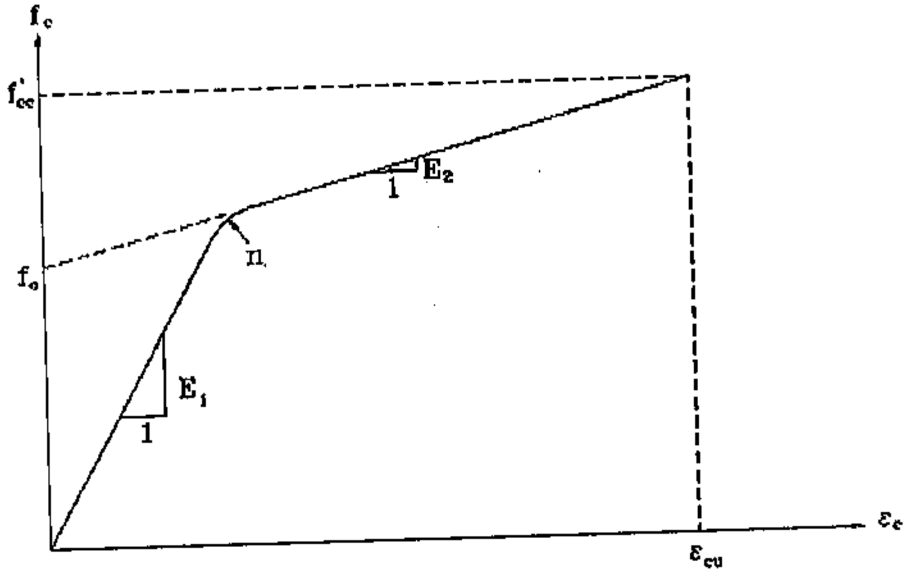


Figure 2.7, Parameters of Mirmiran Stress-Strain Relationship

The stress-strain relation in the Mirmiran model is presented graphically in Figure 2.7, and numerically by the equation:

$$f_c = \frac{(E_1 - E_2)\varepsilon_c}{\left[1 + \left\{\frac{(E_1 - E_2)\varepsilon_c}{f_o}\right\}^n\right]^{\frac{1}{n}}} \qquad \text{.............(2-20)}$$

where; f_c = axial stress in concrete

ε_c = axial strain in concrete.

f_o = reference plastic stress at the intercept of the second

slope with the stress axis.

$$f_o = 0.872 \cdot f'_{co} + 0.371 \cdot f_l + 6.258 \qquad \text{...............(2-21)}$$

$$E_1 = 3950 \cdot \sqrt{f'_{co}} \qquad \text{...............(2-22)}$$

$$E_2 = 245.61 \cdot f_{co}^{'0.2} + 1.3456 \cdot \frac{E_j \cdot t_j}{D} \qquad \text{...............(2-23)}$$

n = a curve shaped parameter ≈ 1.5

E_j = modulus of elasticity of jacket material.

2.3.5 The Vintzileou Model

Vintzileou (2001) developed a new model for the confined concrete strength based on the assessment of previous models and experimental results. He also concluded that the confined concrete strength does not seem to depend on the type of confining material or its modulus of elasticity.

The Vitzileou model depends on two factors, α and ω_w, where, ω_w denotes the volumetric mechanical ratio of the confining material and α denotes the effectiveness of confinement (α is less or equal to 1.0). In the case of cylindrical elements confined by tubes, then α = 1.0. The following expressions are derived for confined concrete strength:

1. cylinders:

$$f'_{cc} = (1 + 1.15\alpha\omega_w)(1.15 - 0.0025 f'_{co}) f'_{co} \quad \text{for } \alpha\omega_w \le 2.0 \quad \text{.........(2-24)}$$

$$f'_{cc} = (3.2 + 0.5\alpha\omega_w)(1.15 - 0.0025 f'_{co}) f'_{co} \quad \text{for } \alpha\omega_w \ge 2.0 \quad \text{........(2-25)}$$

2. 2- prisms:

$$f'_{cc} = (1 + 0.6\alpha\omega_w)(1.15 - 0.0025 f_{co}) f_{co} \quad \text{............(2-26)}$$

where, ; f'_{cc} = confined concrete compressive strength

f'_{co} = unconfined concrete compressive strength

ω_w = volumetric mechanical ratio of the confining material to the volume of concrete

$= \dfrac{4t_j}{D}$, for cylindrical shapes with diameter D ....(2-27a)

$= \dfrac{4t_j}{B}$, for square sections with width B ....(2-27b)

t_j = jacket thickness

and, α = effectiveness of confinement ratio ≤ 1.0

2.4. Concrete Filled Tubes (CFT)

Shams and Saadeghvaziri (1997) presented a paper on the State of the Art Concrete-Filled Steel Tubular Columns, they dedicated part of their work to steel jacket retrofitting. For this purpose test were performed on two reinforced concrete columns, each 610mm in diameter and 3810mm in length. One column was retrofitted with a 4.8mm steel jacket. A 6.3mm gap was provided between the steel jacket and the column, which was filled with cement grout. They concluded that this technique resulted in a system that is more or less similar to concrete filled tube columns.

Park *et al.* (1983) studied steel encased reinforced concrete columns and concluded that the requirements of minimum thickness for steel tubes by both the ACI and AISC-LRFD are conservative although values of D/t_j greater than 72 could perform less favorably.

Boyd *et al.* (1995) supported the above statement by Park et. al. and further added that Equation 2-29a and Equation 2-29b are based on achieving yield stress in an empty shell under monotonic longitudinal compressive load before local buckling occurs. Their tests showed that concrete filled tube columns with higher D/t_j ratios had greater ductility and contributed that to the more ductile material characteristics in the thinner steel shell.

Furthermore Boyd *et al.* (1995) sited several papers investigating buildings constructed with concrete filled tube columns that utilized very thin steel shells concluding that the requirement for minimum thickness seemed to be unnecessarily restrictive for concrete filled tubes.

Shams *et al.* (1997) and Boyd *et al.* (1995) noted that neither the ACI-318 requirements nor the LRFD specifications accounted for any possible enhancement in strength due to confinement of concrete by steel tubes. Park et. al. suggested that the steel casing of concrete filled tubes had a confining effect that could enable an increase of the axial load capacity of these columns.

2.4.1 ACI 318M-99 Requirements for CFT

According to the ACI 318M-99 a concrete filled tube consists of a concrete column reinforced with a structural steel shape (tube) in addition to the reinforcing bars, i.e.,

$$P_o = 0.85(A_s f_y + A_j f_{jy} + 0.85 A_c f_c') \qquad \text{.................(2-28)}$$

where, P_o = maximum nominal axial strength of column.

A_s = cross sectional area of reinforcing steel bars.

f_y = yield strength of reinforcing steel bars.

A_j = cross sectional area of steel tube.

f_{jy} = yield strength of steel tube.

A_c = cross sectional area of concrete.

f_c' = unconfined crushing strength of concrete.

To prevent local buckling, thickness of the steel tube should be greater than:

$t_j \geq D\sqrt{\dfrac{f_{jy}}{8E_{sj}}}$, for circular sections with diameter D ....(2-29a)

$t_j \geq b\sqrt{\dfrac{f_{jy}}{3E_{sj}}}$, for each face of width b of rectangular sections ....(2-29b)

where, t_j = thickness of steel tube.

f_{jy} = yield strength of steel tube.

E_{sj} = modulus of Elasticity of steel tube.

2.4.2 AISC-LRFD Specifications (93)

According to the AISC-LRFD specifications a concrete filled tube is basically a steel tube with a reinforced concrete part inside, therefore the design strength of a concrete filled tube is $\phi_c P_n$, where:

$\phi_c = 0.85$ = resistance factor for axially loaded composite columns

$P_n = A_j F_{cr}$ = nominal axial strength ....(2-30)

$F_{cr} = (0.658^{\lambda_c^2})F_{my}$, for $\lambda_c \leq 1.5$ ....(2-31a)

$F_{cr} = \left[\dfrac{0.877}{\lambda_c^2}\right]F_{my}$, for $\lambda_c \geq 1.5$ ....(2-31b)

$$\lambda_c = \frac{K_l l}{r\pi}\sqrt{\frac{F_{my}}{E_m}} \qquad \text{..................(2-32)}$$

A_j = gross cross sectional area of steel tube.

K_l = effective length factor

l = laterally unbraced length of member

r = governing radius of gyration about the axis of buckling

$$F_{my} = f_{jy} + f_y \frac{A_s}{A_j} + 0.85 f_c' \frac{A_c}{A_j} \qquad \text{..................(2-33)}$$

$$E_m = E_{sj} + 0.4 E_c \frac{A_c}{A_j} \qquad \text{..................(2-34)}$$

A_s = cross sectional area of reinforcing steel bars.

f_y = yield strength of reinforcing steel bars.

f_{jy} = yield strength of steel tube.

A_c = cross sectional area of concrete.

f_c' = crushing strength of concrete.

The limitation for the minimum wall thickness of the steel tube is the same as specified in the ACI 318M-99 requirements stated above, Equation 29a and Equation 29b.

When λ_c (column slenderness parameter) approaches zero and multiplying Equation 2-33 by A_j the resulting equation is the similar to Equation 2-28:

$$\phi_c P_n = 0.85(A_j f_{jy} + A_s f_y + 0.85 A_c f_c') \qquad \text{..................(2-35)}$$

Hence, since most reinforced concrete columns are short both the ACI and the LRFD design criteria's are the same.

2.4.3 Unbonded Steel Tubes Filled with Concrete

Orito *et al.* (1987) tested bonded and unbonded concrete filled steel tubes, Figure 2.8. The inner surface of the steel tubes for the unbounded case was covered with antifrictional material (asphalt). The load was applied to both the steel tube and the concrete core in the bonded case and only to the concrete core in the unbonded case.

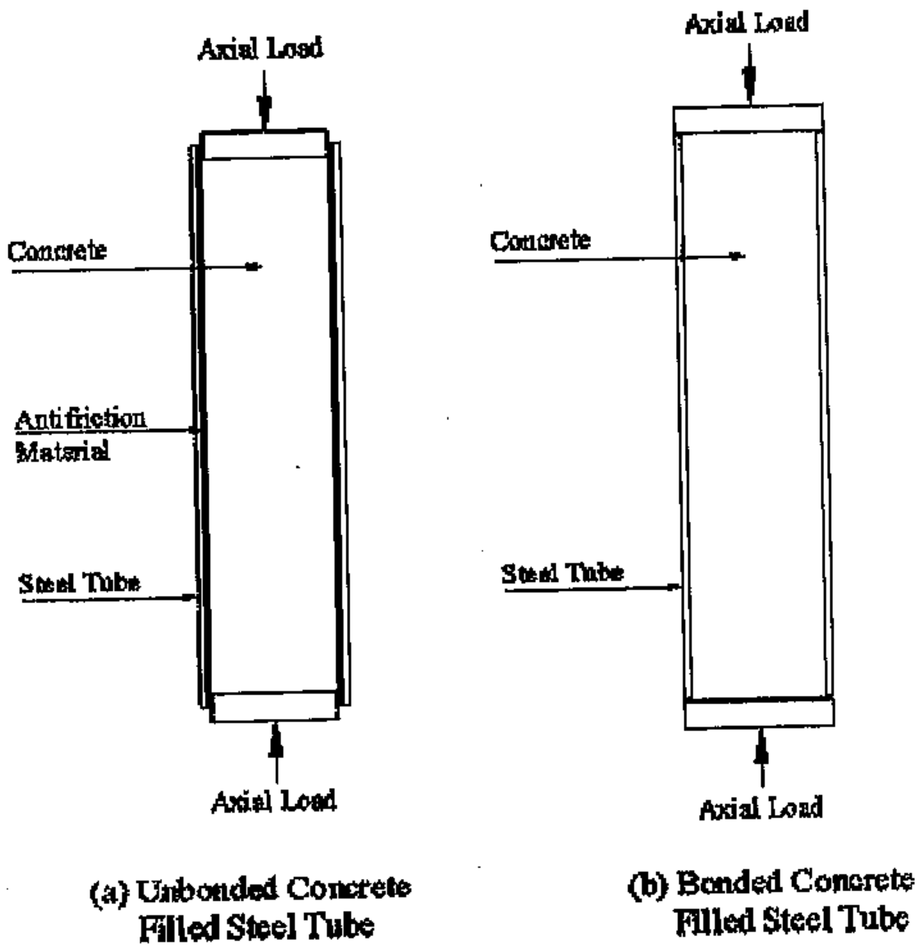


figure 2.8. the bonded and unbonded concrete filled steel tubes

Their conclusions with regard to the strength of the bonded and unbonded concrete filled steel tubes could be summarized as follows:

- In the circular bonded concrete filled steel tubes, the tubes directly share the axial load with the core concrete. Failure is due to the crushing of the core concrete, the compressive yielding (or local buckling) of the steel tube.

- In the circular unbonded concrete filled steel columns, the tube do not directly share the axial load and is an element used only to confine the concrete core, resulting in an increase in the concrete compressive strength.

Orito et. al. has derived an expression for the confined concrete strength of circular unbonded concrete filled steel columns:

$$f'_{cc} = f'_{co} + 3\frac{t_j}{R} f_{jy} \qquad \text{......................(2-36)}$$

where, ; f'_{cc} = confined concrete compressive strength

f'_{co} = unconfined concrete compressive strength

t_j = thickness of the steel tube (jacket).

R = radius of the steel tube (jacket).

f_{jy} = yield strength of steel tube (jacket).

The second term of the right side of the above equation can be rewritten as:

$$3\frac{t_j}{R}f_{jy} = k_1 f_l$$

which reduces Equation 2-36 to the Richart Model Equation 2-4 with $k_1 = 3$ and

$$f_l = \frac{2t_j f_{jy}}{D} \qquad \text{.....................(2-37)}$$

where, D = the diameter of the steel tube (jacket).

3. Experimental Setup and Materials

Experimental-analytical approach is typical of concrete research to support existing analysis and design procedures of concrete elements or suggest alternative ones, therefore, thirty-nine tests on square columns were carried out to investigate the technique of retrofitting square reinforced concrete columns with steel jackets.

The main objective of the experiment was to determine the axial compressive strength of retrofitted square columns with steel jackets and compare that with the axial compressive strength of: non-retrofitted columns, confined columns with partial steel jackets and concrete filled tubular columns. Bond between the steel jacket, grout and reinforced concrete column was also investigated.

3.1 Testing Machine

The testing machine used was a universal testing machine: Toni-MFL model DZN-40 of the year 1984 with a total capacity of 400 kN. All specimens were prepared and placed under the applied load with a high degree of accuracy to ensure the load application to the required position. The loading system was a pin ended cylindrical hydraulic jack. The built-in load cells measured the load carrying capacity of the tested column and total axial shortening. Schematic general view of the testing machine is shown in Figure 3.1.

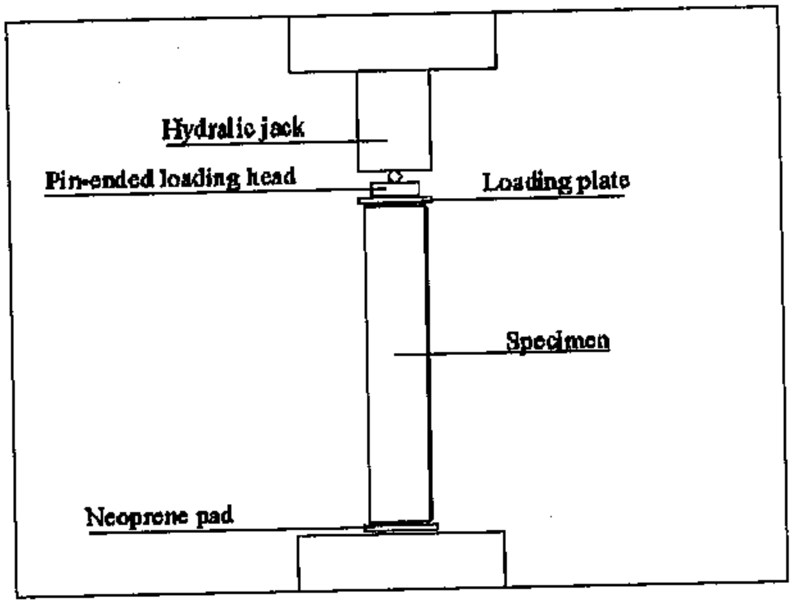


Figure 3.1. Schematic view of the testing machine

3.2 Structural Similitude

Similitude between two objects may exist with regard to any one of their physical characteristics. Three kinds of similitude exist in general, the geometric similitude, kinematic similitude and the dynamic similitude. In the present investigation only the geometric similitude is required (Sabins *et al.* 1983).

To satisfy the geometric similitude, dimensions of prototype and model must be in some constant ratio. Direct model analysis requires similitude between the prototype and model in mechanical properties so that when the model is loaded in the same manner as the prototype all effects such as stresses, strains, and deformations in the model are directly proportional to those of the prototype.

The scale factor K defines the length relationship between prototype and model:

$$K = \frac{L_p}{L_m}, \text{ and } K^2 = \frac{A_p}{A_m} \qquad \text{(3-1)}$$

where, L_p and A_p are length and cross sectional area of prototype,

and, L_m and A_m are length and cross sectional area of model.

Other scale relations can be derived from these two basic primary relations such as:

$$\frac{\sigma_p}{\sigma_m} = \frac{E_p}{E_m} \qquad \text{(3-2)}$$

where, σ_p and E_p are stress and modulus of elasticity for prototype

and, σ_m and E_m are stress and modulus of elasticity for model.

Since the same materials are used in the model and the prototype, which means that $E_p = E_m$, Equation 3-2 can be rewritten as:

$$\frac{\sigma_p}{\sigma_m} = 1 \qquad \text{(3-2')}$$

It follows that:

$$\frac{(\text{Load})_{\text{prototype}}}{(\text{Load})_{\text{model}}} = \frac{A_p}{A_m} = K^2 \qquad \text{(3-3)}$$

3.3 Test Specimens

In order to investigate rectangular reinforced concrete columns that are usually found in buildings a 400×400mm column was considered. A steel jacket 4-mm thick having an exterior area of 480×480mm was selected.

A scale factor of $K=\frac{1}{4}$ was used to scale down all specimens. Table (3.1) presents the dimension of both model and prototype columns, the specimens were divided into five groups as follows:

Table 3.1. Dimensions of specimens

Group	Prototype Column Size	Model Column Size	Prototype Reinforcement	Model Reinforcement	Prototype Jacket	Model Jacket
1	400x400	100x100	4Φ32mm Φ10@200mm	4Φ8mm Φ2.5@50mm	480x480 t=4mm	120x120 t=1mm
2	400x400	100x100	4Φ32mm Φ10@200mm	4Φ8mm Φ2.5@50mm	480x480 t=4mm	120x120 t=1mm
3	400x400	100x100	4Φ32mm Φ10@200mm	4Φ8mm Φ2.5@50mm	N/A	N/A
4	400x400	100x100	4Φ32mm Φ10@200mm	4Φ8mm Φ2.5@50mm	480x480 t=4mm	120x120 t=1mm
5	472x472	118x118	4Φ32mm Φ10@200mm	4Φ8mm Φ2.5@50mm	480x480 t=4mm	120x120 t=1mm

1. Group 1: this group consisted of 10 column specimens, shown in Figure 3.2, intended to test the strength of reinforced concrete columns retrofitted with full steel jackets under both concentric and eccentric axial loading.

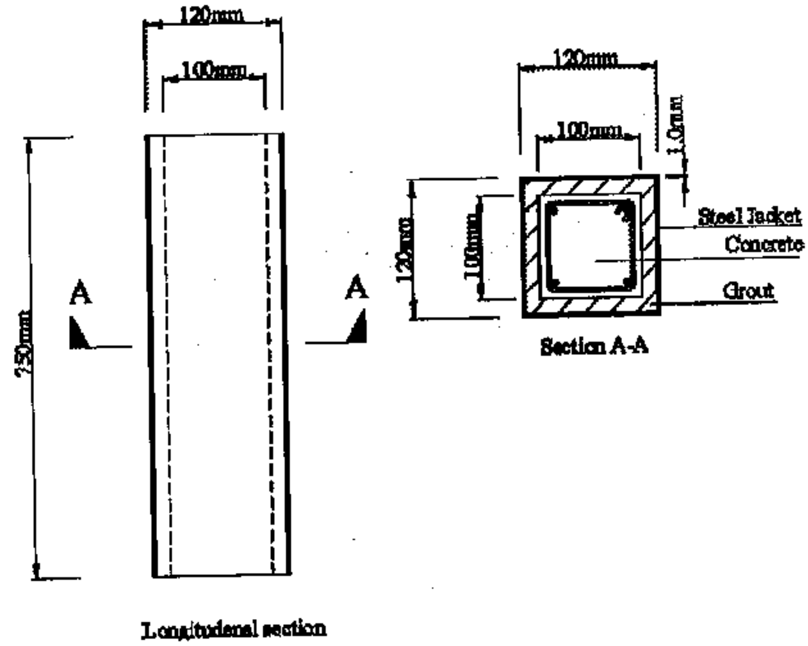


Figure 3.2. Sectional details of group 1

2. Group 2: this group consisted of 10 column specimens, shown in Figure 3.3, intended to test the strength of confined concrete columns with steel jackets under both concentric and eccentric axial loading.
3. Group 3: this group consisted of 10 specimens of reinforced

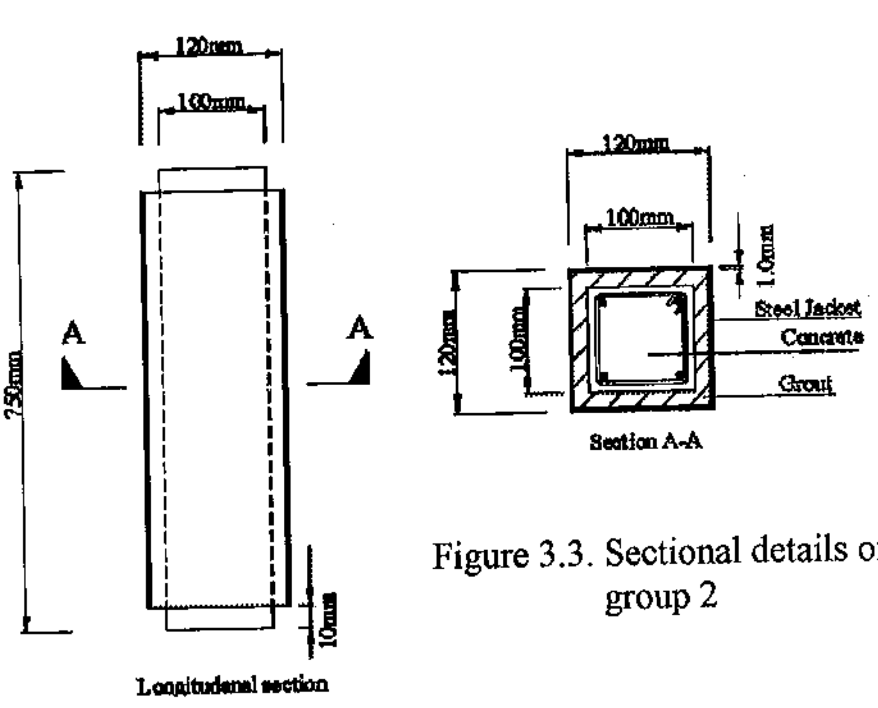


Figure 3.3. Sectional details of group 2

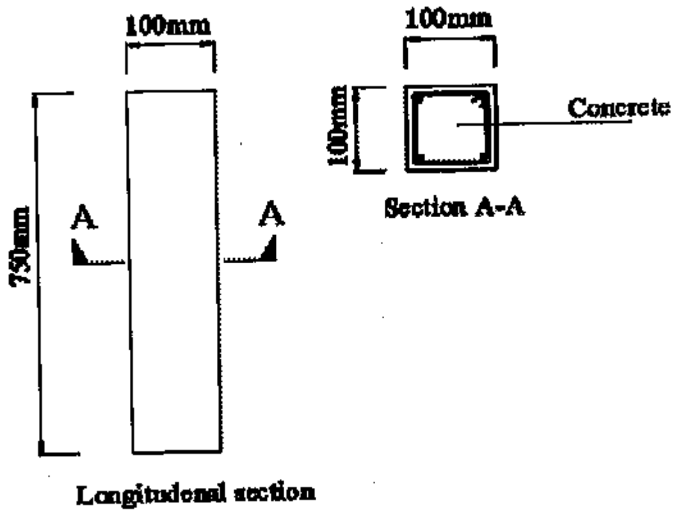


Figure 3.4. Sectional details of group 3

4. Group 4: this group consisted of 5 specimens of reinforced concrete column specimens, shown in Figure 3.2. The purpose of this test is to cause the concrete column to slip when the bond stress between the concrete column and the grout from one side and the steel jacket and the grout from the other side reach their ultimate values. This was achieved by testing specimens under axial load applied to the concrete column only by means of a 100x100X2 mm steel plate while the specimen rested on a specially manufactured plate with a hole in the middle 100x100X2 mm as shown in Figure 4.11, page 57.
5. Group 5: this group consisted of 4 specimens of tubular column specimens, shown in Figure 3.5, tested under concentric and eccentric axial loading.

All specimens were cast at the laboratories of the Faculty of Engineering and Technology at the University of Jordan and left to cure in controlled conditions for at least 28 days before testing.

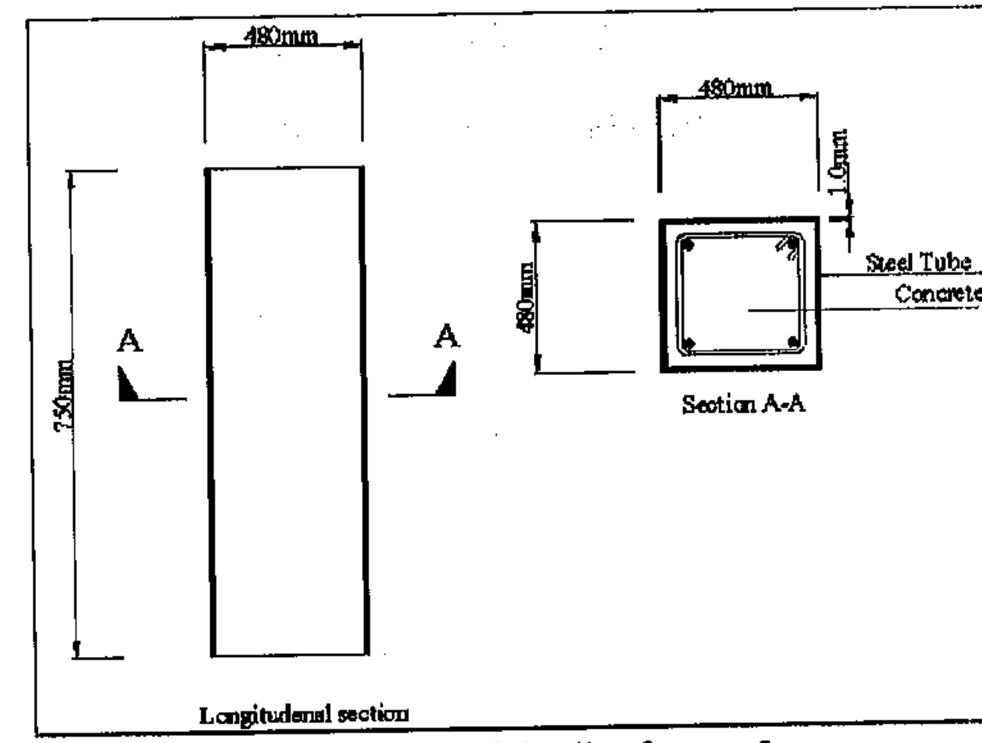


Figure 3.5. Sectional details of group 5

3.4 Materials

Concrete mix design is described in Appendix A. Cubes taken from the concrete used had a mean strength of 22.45 N/mm^2.

Reinforcing steel for longitudinal and transverse reinforcement had a yield strength $f_y = 200$ N/mm^2.

Grout between the reinforced concrete columns and steel jackets consisted of cement, sand passing sieve number 30 and water with proportions of 1(cement): 1(sand): 0.75(water). Cubes taken from the grout mix and tested had a mean compressive strength of 24 N/mm^2.

Steel sheets used to form the steel jackets had a yield strength f_y = 400 N/mm^2, the jackets were formed by bending the sheets into u-shaped section, then assembling the two u-sections to form a square by welding both sections together, as shown in Figure 3.6.

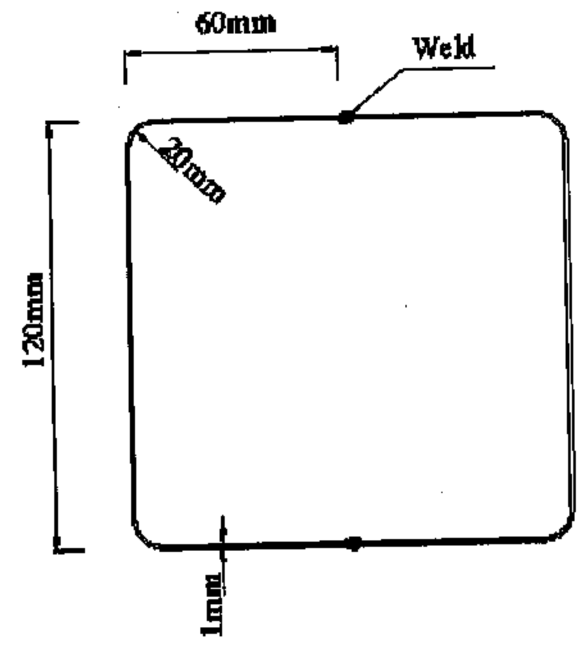


Figure 3.6. Steel jacket details

Figure 3.7 shows the reinforcement cage before casting the reinforced concrete column. Figure 3.8 shows the casting of reinforced concrete columns. Figure 3.9 shows the steel jacket after welding. Figure 3.10 shows the reinforced concrete placed inside the steel jacket

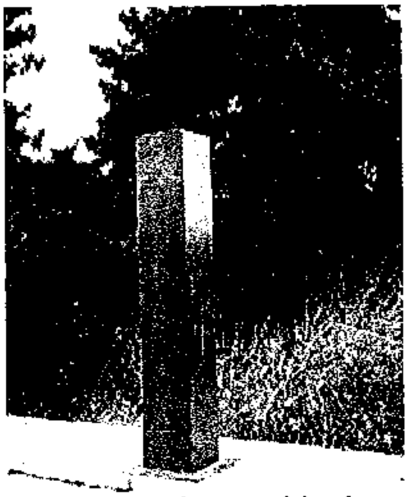


Figure 3.9. The steel jacket

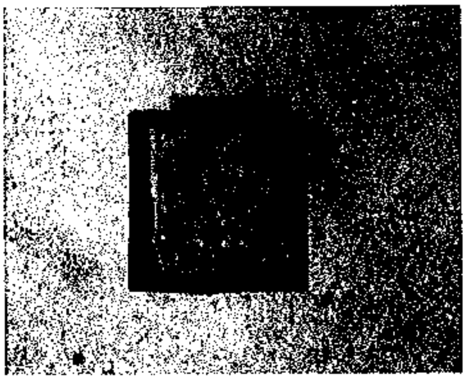


Figure 3.10. The reinforced column inside the steel jacket before grouting

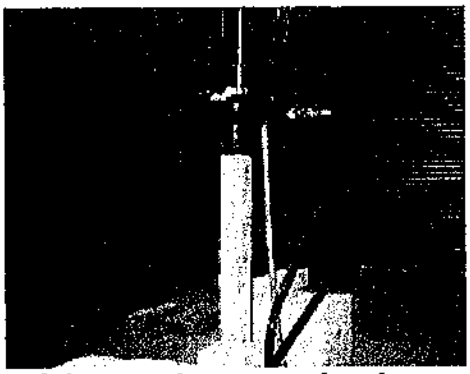


Figure 3.11. One of the specimens under the testing machine

4. EXPERIMENTAL RESULTS AND DISCUSSION

The results and the behavior of column specimens under load, based on the recorded data and observations during testing are presented in this chapter. Illustrations of the columns are introduced together with curves showing relationship between each group of specimens. Discussions and comments of these relations and other findings are included were needed.

4.1 Experimental Results

Behavior of specimens under testing is an important part of the experimental results. Therefore observations during testing along with the recorded data are presented.

4.1.1. Group 1: Reinforced Concrete Columns with Full Steel Jacket:

Failure of Group 1 members occurred due to the local buckling of the steel jacket accompanied by yielding of the jacket in compression and subsequently crushing of the concrete core as shown in Figure 4.1.

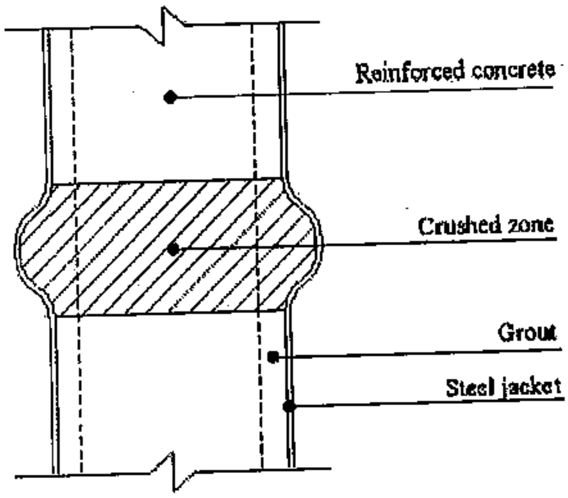
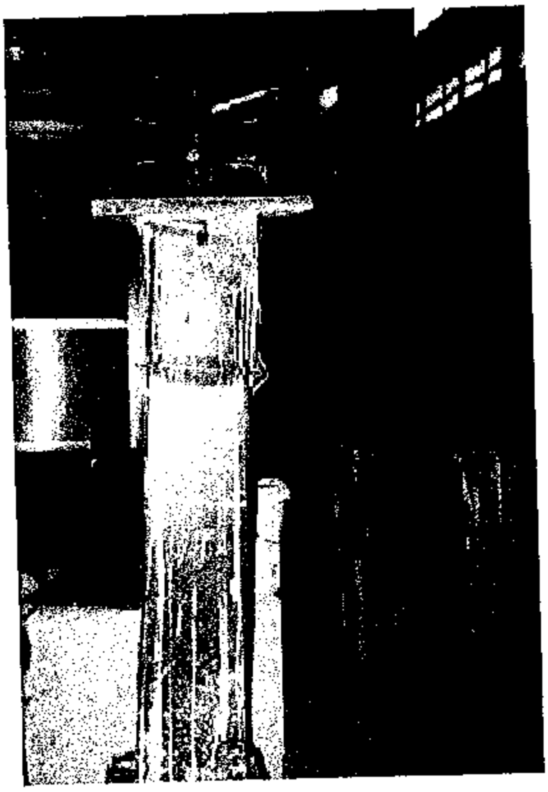
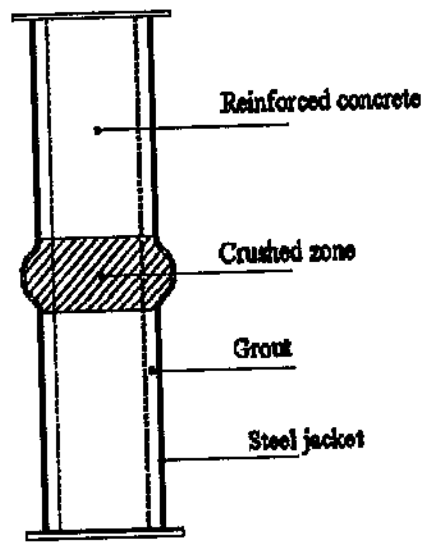


Figure 4.1. Longitudinal cross section showing the typical failure of "Group 1" specimens.

Specimens showed a large amount of ductility before failure and Figure 4.2 presents the typical behavior of specimens in this group.



(a) Typical failure



(b) Longitudinal Section

Figure 4.2.Typical Behavior of "Group 1" Specimens.

This group consisted of two sets of data, one for each type of loading, these data are:

(a) **Group 1-a**: five specimens were loaded in an axially concentric manner to investigate the maximum axial capacity of this type of retrofitting. The results are shown in Table 4.1. These results can be represented as an average best-fit curve typical of this group of specimens as shown in Figure 4.3.

Table 4.1.Experimental Data for (Group 1-a) Specimens

Specimen number									
1		2		3		4		5	
Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)
0	0	0		0		0		0	
25	0.4	25	0.3	25	0.4	25	0.4	25	0.3
50	1.2	50	0.8	50	1.1	50	0.9	50	1.1
75	1.4	75	1.2	75	1.5	75	1.3	75	1.5
100	1.5	100	1.8	100	2.2	100	1.8	100	1.9
125	2.3	125	2.4	125	2.5	125	2.4	125	2.2
150	2.7	150	2.8	150	2.9	150	3	150	2.9
175	3	175	3.1	175	3.3	175	3.4	175	3.5
200	3.5	200	3.4	200	3.6	200	3.7	200	3.9
225	3.9	225	3.6	225	3.9	225	4	225	4.3
250	4.4	250	3.9	250	4.3	250	4.4	250	4.8
275	4.9	275	4.2	275	4.5	275	4.7	275	5
280	5.3	280	4.6	280	4.8	280	4.9	280	5.2
285	5.5	285	5	285	5	285	5.2	285	5.4
290	5.7	290	5.4	290	5.5	290	5.6	290	5.5
295	5.9	295	5.7	295	5.8	295	5.9	295	5.8
300	6.1	300	6.2	300	6	300	6.1	300	6.2
305	6.3	305	6.5	305	6.2	305	6.4	305	6.8
310	6.5	310	6.8	310	6.5	310	6.6	309(F)	7.3
315	6.7	313(F)	7.1	315	6.8	315	6.9		
320	6.9			320	7.1	320	7.3		
325	7.1			325	7.5	321(F)	7.5		
330	7.3			327(F)	7.8				
335	7.7								
340	8.1								
347(F)	8.5								

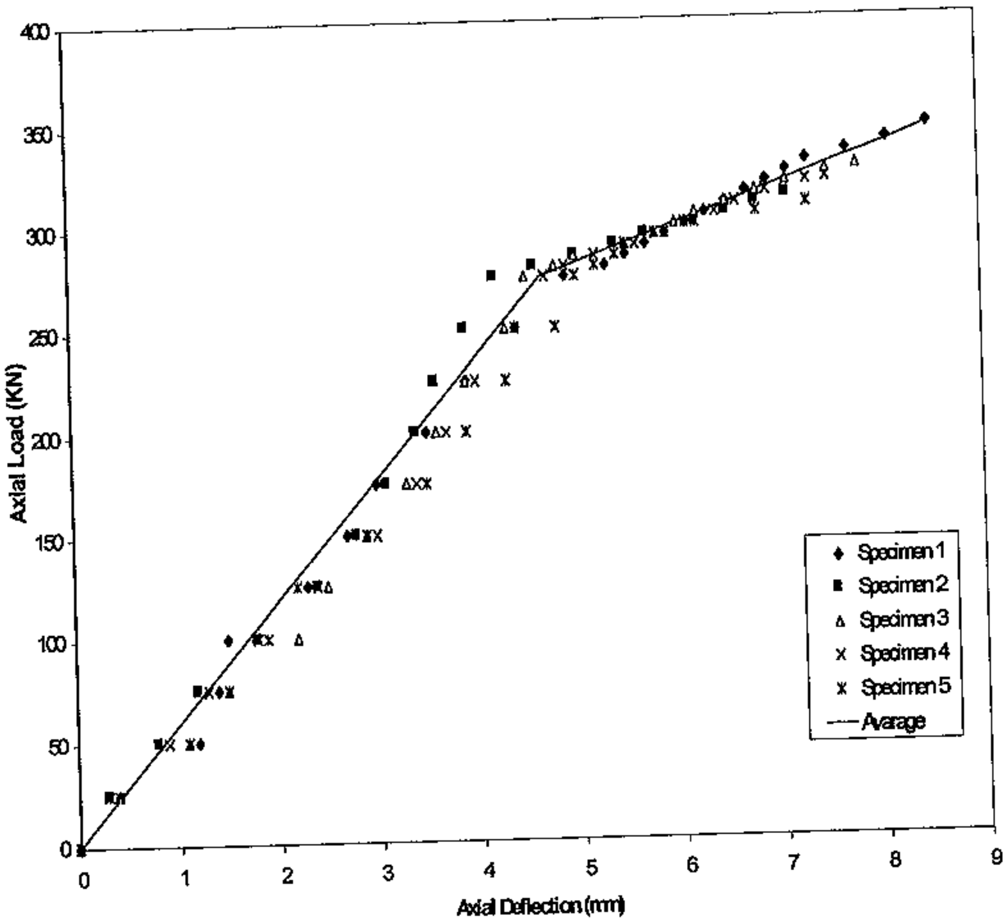


Figure 4.3. Graphical Representation of Data of Group 1-a

(b) **Group 1-b**: Five specimens were loaded in an axially eccentric manner, to investigate the maximum axial capacity of this type of retrofitting under an axial eccentricity $e = 7.5$ mm. The results are shown in Table 4.2. These results can be represented as an average best-fit curve typical of this group of specimens as shown in Figure 4.3. Members of this sub-group followed the same behavior of the Group1-a, the local buckling of the steel jacket was initiated at the compression zone, and immediately spread to all four sides accompanied by crushing of the concrete core.

Table 4.2, Experimental Data for (Group 1-b) Specimens

Specimen Number									
6		7		8		9		10	
Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)
0	0	0	0	0	0	0	0	0	0
25	0.4	25	0.3	25	0.4	25	0.4	25	0.5
50	1.3	50	1.2	50	1.2	50	1.3	50	1.5
75	1.6	75	1.5	75	1.7	75	1.6	75	1.8
100	2.1	100	2	100	2.2	100	2.2	100	2.3
125	2.7	125	2.6	125	2.5	125	2.7	125	2.9
150	3.5	150	3.4	150	3.2	150	3.3	150	3.5
175	4.4	175	4.5	175	4.3	175	4.6	175	4.7
200	5.3	200	5.2	200	5.3	200	5.1	200	5.3
210	5.7	210	5.8	210	5.9	210	5.6	210	5.9
220	6.1	220	6	220	6.2	220	5.9	218(F)	6.3
230	6.3	227(F)	6.2	223(F)	6.5	230	6.2		
235(F)	6.5					233(F)	6.6		

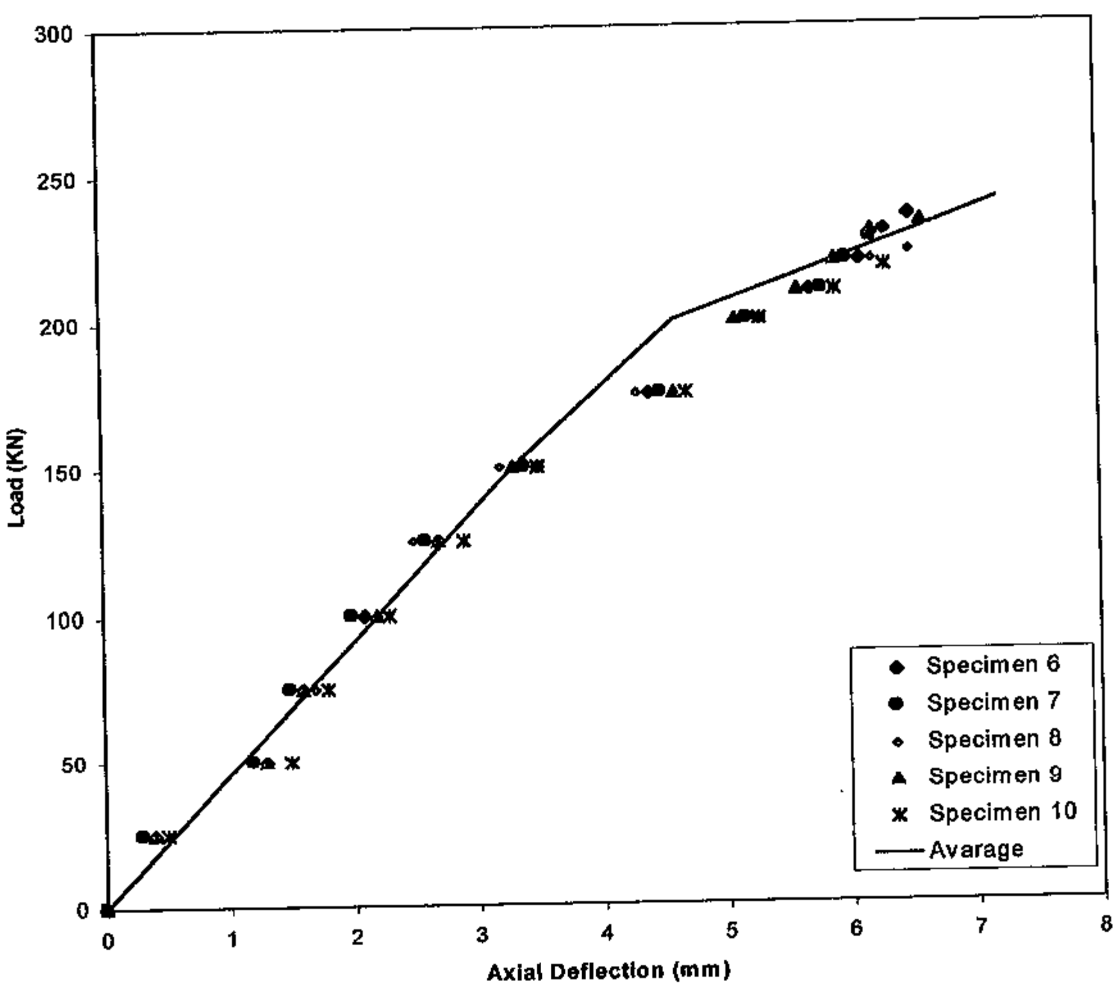
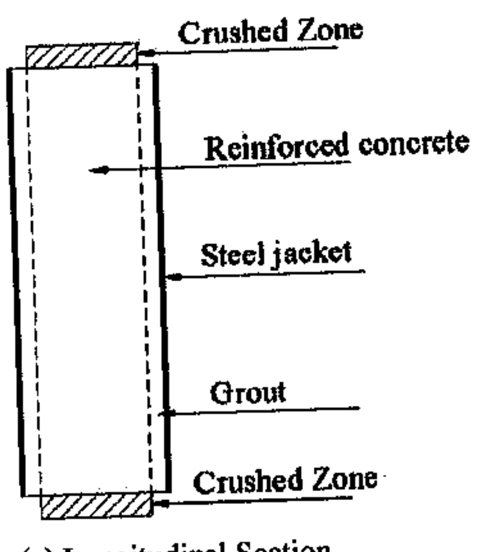


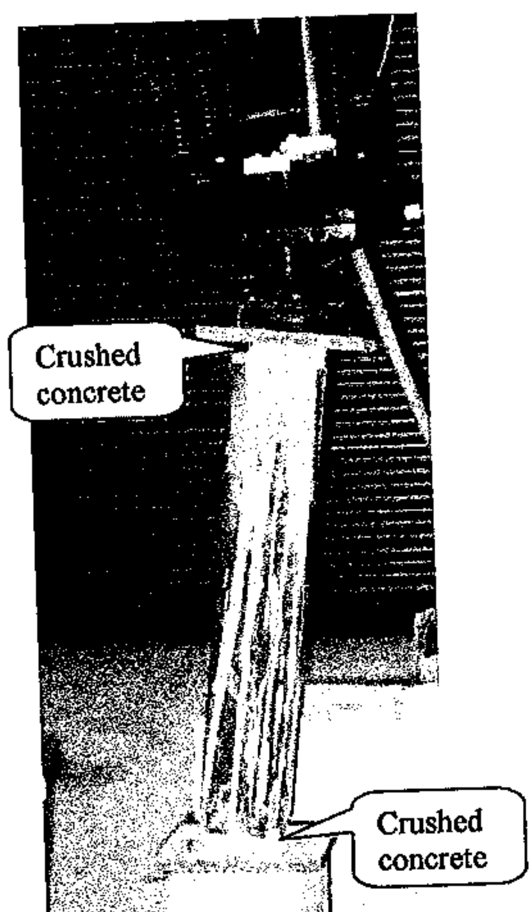
Figure 4.4, Graphical Representation of Data of Group 1-b

4.1.2. Group 2: Confined Reinforced Concrete Columns with Steel Jackets:

It was observed that the tensile capacity of the steel jacket was not reached, i.e., the steel jacket did not yield, and failure occurred when the concrete part on either the top or bottom of the specimen crushed under the applied load. The specimens of this group showed less ductility than Group 1 specimens and Figure 4.5 presents the typical behavior of specimens in this group. The increase of strength due to confinement is expected, but the type of failure could imply that the full confinement strength of the steel jacket was not reached.



(a) Longitudinal Section



(b) Typical Failure

Figure 4.5. Typical behavior of group 2

This group consisted of two sets of data for each type of loading, these data are:

a) **Group 2-a**: five specimens were loaded in an axially concentric manner to investigate the maximum axial capacity of this type of retrofitting. The results are shown in Table 4.3.

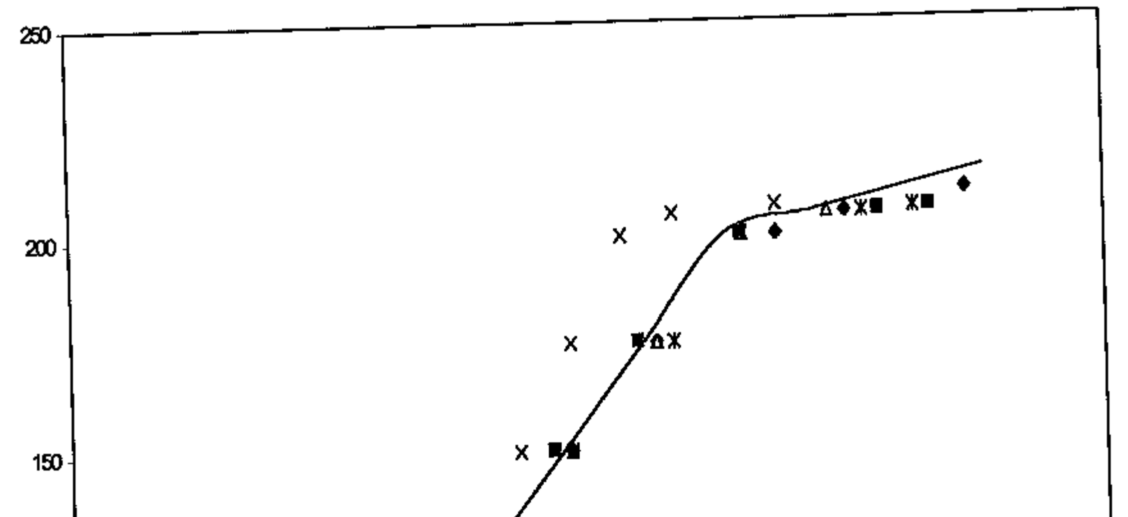


Figure 4.6.Graphical Representation of Data of Group 2-a

These results can be represented as an average best-fit curve typical of this group of specimens as shown in Figure 4.6.

Table 4.3. Experimental Data for (Group 2-a) Specimens

Specimen Number									
11		12		13		14		15	
Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)	Load	Axial Deflection (mm)
0	0	0		0		0		0	
25	0.4	25	0.3	25	0.4	25	0.4	25	0.3
50	1.4	50	1.1	50	1.1	50	0.9	50	1.2
75	1.7	75	1.5	75	1.4	75	1.5	75	1.5
100	2.1	100	1.9	100	2.1	100	2	100	1.9
125	2.5	125	2.2	125	2.4	125	2.3	125	2.3
150	2.9	150	2.8	150	2.9	150	2.6	150	2.9
175	3.4	175	3.3	175	3.4	175	2.9	175	3.5
200	4.1	200	3.9	200	3.9	200	3.2	200	3.9
205	4.5	205	4.7	205(F)	4.4	205	3.5	205	4.6
210(F)	5.2	205.8(F)	5			207(F)	4.1	206(F)	4.9

b) **Group 2-b**: Five specimens were loaded in an axially eccentric manner, to investigate the maximum axial capacity of this type of retrofitting under an axial eccentricity $e = 7.5$ mm. The results are shown in Table 4.4. These results can be represented as an average best-fit curve typical of this group of specimens as shown in Figure 4.7.

Table 4.4. Experimental Data for (Group 2-b) Specimens

Specimen Number									
16		17		18		19		20	
Load	Delta(mm)	Load	Delta	Load	Delta	Load	Delta	Load	Delta
0	0	0	0	0	0	0	0	0	0
25	0.4	25	0.4	25	0.4	25	0.4	25	0.3
35	1.4	35	1.3	35	1.2	35	1.4	35	1.3
45	1.7	45	1.6	45	1.7	45	1.8	45	1.7
55	2.1	55	2	55	2.1	55	2.2	55	2.1
65	2.5	65	2.6	65	2.5	65	2.7	65	2.5
75	3.3	75	3.5	75	3.6	72	3.9	75	3.4
85	4.5	77	4.8	79	4.4			82	4
89	5.2								

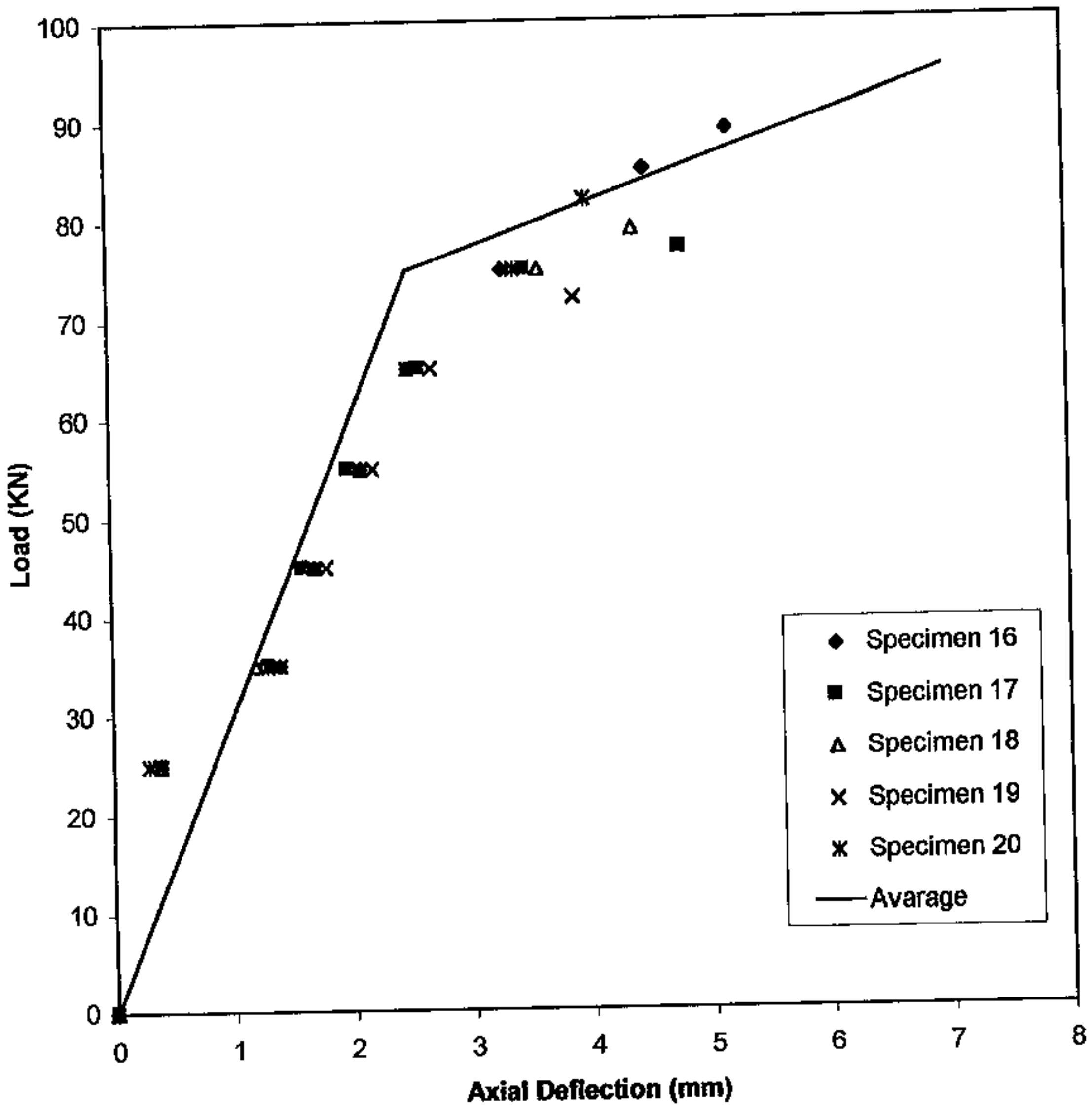


Figure 4.7.Graphical Representation of Data of Group 2-b

4.1.3. Group 3: Reinforced Concrete Sections without Retrofitting:

The reinforced concrete column specimens behaved as expected. The specimens had a brittle failure and the ultimate load recorded was close to expected ultimate load. Failure was initiated by spalling of the concrete cover followed by local buckling of the reinforcing steel and immediately crushing of concrete core, Figure 4.8 presents the typical behavior of specimens in this group.

4.1.4. Group 4: Specimens of reinforced concrete columns retrofitted with Steel jackets and tested for bond

These specimens were tested under axial load applied to the concrete column only by means of a 100x100x2 mm steel plate while the specimen rested on a specially manufactured plate with a hole in the middle 100x100x2 mm as shown in Figure 4.11. The purpose was to

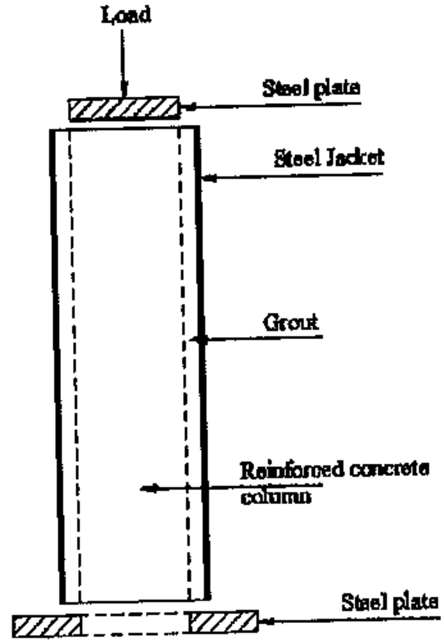


Figure 4.11.Load Applied on Specimens of Group 4 to Determine the Bond Strength

cause the concrete column to slip when the bond stress between the concrete column and the grout form one side and the steel jacket and the grout from the other side reaches its ultimate value.

This test was described in the work of Chen, J.F., Yang, Z.I., Pan, X.M., and Holt, G.D., as a standard procedure used by previous researchers and is called "The Double Shear Pushing Test". Here it is utilized to determine the amount of stress transferred by bond from the reinforced concrete column to the steel jacket by means of the grout substrata. The specimens of this group failed when the grout crushed under bearing pressure, as shown in Figure 4.12, therefore it is safe to assume that the value of the ultimate bond strength developed is larger than the recorded ultimate values for this group. The values of the failure loads of the specimens, Table 4.7, were large enough to imply that the retrofitted columns might behave as concrete filled steel tubes, which speared the introduction of the next group of specimens to further investigate this assumption. These data are presented graphically in Figure 4.13.

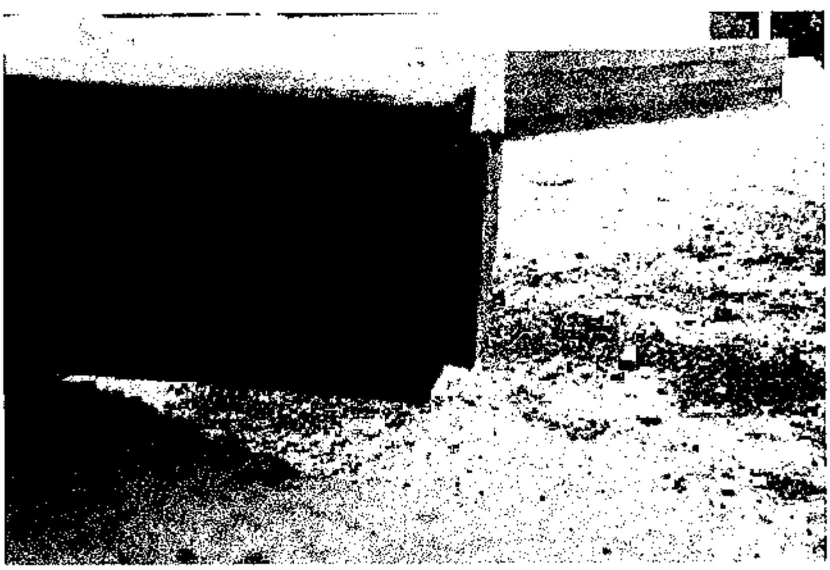


Figure 4.12.Typical Behavior of Group 4 Specimens

Table 4.7. Experimental Data for (Group 4) Specimens

Specimen Number									
31		32		33		34		35	
Load	Axial Slip (mm)	Load	Axial Slip (mm)	Load	Axial Slip (mm)	Load	Axial Slip (mm)	Load	Axial Slip (mm)
0	0	0		0		0		0	
25	0.4	25	0.3	25	0.4	25	0.4	25	0.3
50	1.3	50	0.9	50	1.2	50	1.3	50	1.2
75	1.6	75	1.9	75	1.8	75	1.9	75	1.7
100	2.1	100	2.7	100	2.4	80	2.9	100	2.2
105	2.5	103(F)	7.5	105	2.4	85	3.1	105	2.5
108(F)	5.6			110(F)	4.8	90(F)	4.1	110(F)	4.6

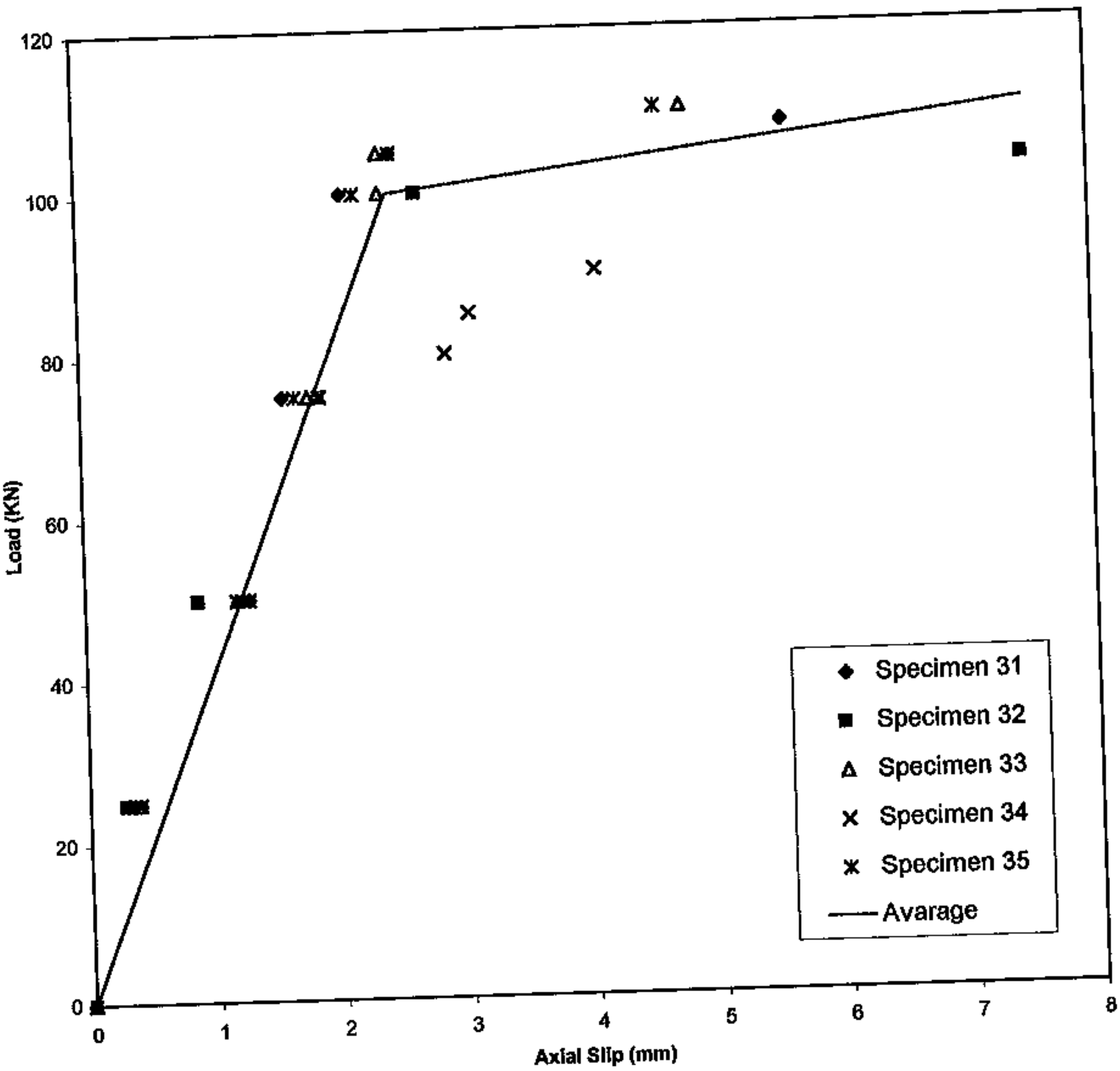


Figure 4.13. Graphical Representation for Data of Group 4

4.1.5. Group 5, Concrete Filled Steel Tubes

This group of columns was included to determine the ultimate strength of composite columns when cast monolithically with the steel tubes and compare it with the strength of the retrofitted columns of groups 1 and 2. The key factor affecting the behavior and reliability of such an assumption is the bond strength between the steel jacket and the concrete inside.

Failure of Group 5 members occurred due to the local buckling of the steel jacket accompanied by yielding of the jacket in compression and subsequently crushing of the concrete core as shown in Figure 4.14.

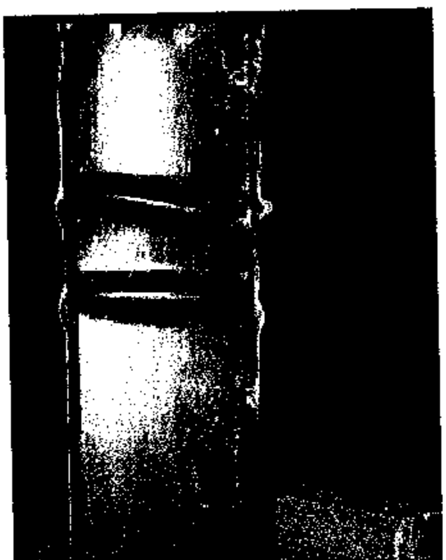


Figure 4.14.Typical Behavior of Group 5 Specimens

This group consisted of two sets of data for each type of loading, these data are:

a) **Group 5-a**: Two specimens were loaded in an axially concentric manner to investigate the maximum axial capacity of the concrete filled tubes. The results are shown in Table 4.8. These results can be represented as an average best-fit curve typical of this group of specimens as shown in Figure 4.15.

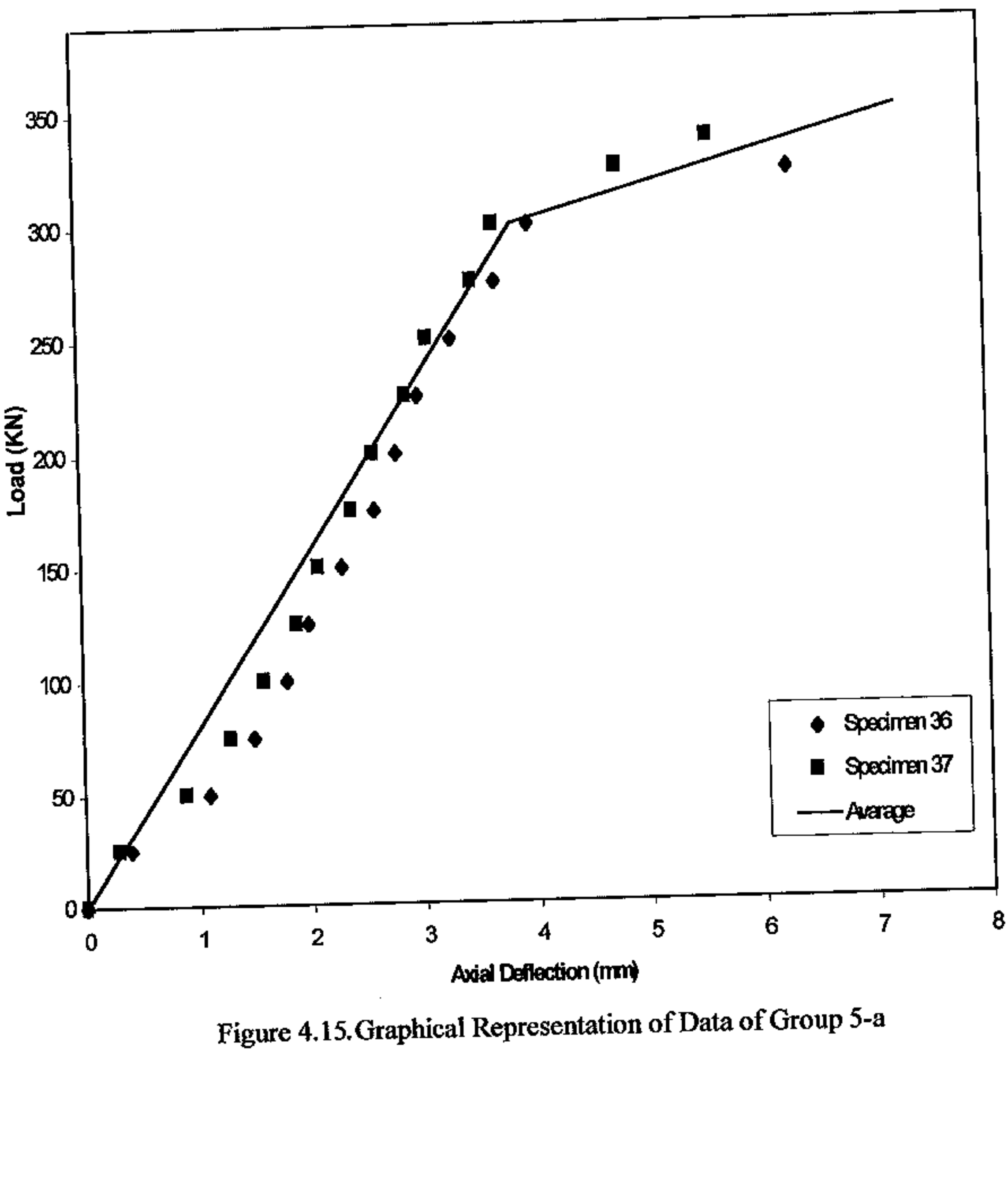


Figure 4.15. Graphical Representation of Data of Group 5-a

Table 4.8.Experimental Data for (Group 5-a) Specimens

Specimen Number			
36		37	
Load	Axial Deflection (mm)	Load	Axial Deflection (mm)
0	0	0	0
25	0.4	25	0.3
50	1.1	50	0.9
75	1.5	75	1.3
100	1.8	100	1.6
125	2	125	1.9
150	2.3	150	2.1
175	2.6	175	2.4
200	2.8	200	2.6
225	3	225	2.9
250	3.3	250	3.1
275	3.7	275	3.5
300	4	300	3.7
323	6.3	325	4.8
		337	5.6

Table 4.9 Experimental Data for (Group 5-b) Specimens

Specimen Number			
38		39	
Load	Axial Deflection (mm)	Load	Axial Deflection (mm)
0		0	
25	0.3	25	0.4
50	1.1	50	1.3
75	1.5	75	1.7
100	1.9	100	2.3
125	2.5	125	2.8
150	3.3	150	3.4
175	4.2	175	4.4
200	5	200	5.1
210	5.5	210	5.8
220	6.1	220	6.3
230	6.3	222	6.7
235	6.5		

The reason is that when the moment is applied to Group 2-b specimens, the extreme fibers at the top and bottom (Figure 4.5-a) reach their ultimate strength and subsequently fail with less ductility than the Group 1-b specimens that shares the axial stresses with the steel jacket, while the steel jacket in Group 2-b has not developed its full confinement capacity.

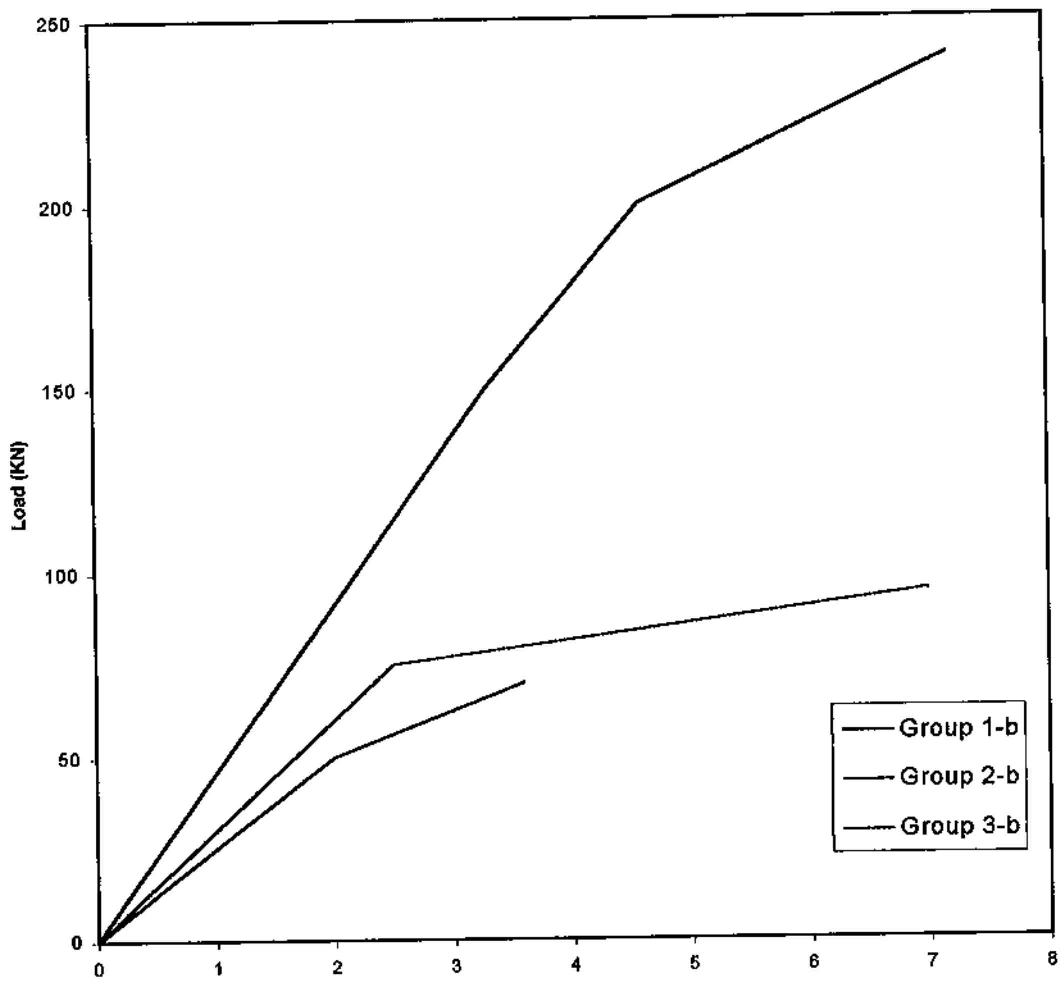


Figure 4.20, Comparison of Group 1-b, Group 2-b and Group 3-b

5. DESIGN CONSIDERATIONS

In this chapter the findings and conclusions of the previous chapters will be reflected into meaningful terms to assist the designer of a retrofitting system for a square reinforced concrete column in achieving reliable and economical design.

5.1 Confinement Pressure

Richart et. al. applied a fluid pressure to his specimens; therefore the easiest part of his formula for the confined concrete strength, Equation (2-4), was the determination of the value of the lateral confining pressure since this term was pre-defined by the nature of the experiment. Ben-Zvi et. al. suggested applying a prestressing force to the steel jacket so that the confining pressure will be pre-determined. The Seismic Retrofitting Manual for Highway Bridges specified that the thickness of the steel jacket be calculated as in Equation (2-14) which can be rewritten as;

$$f_l = \frac{400t_j}{D} \qquad \text{.............................(5-1)}$$

where, D = the diameter of the column.

t_j = the steel jacket thickness

f_l = confining pressure

f_j = the stress induced in the jacket.

The above equation can be misleading since it suggests that when the thickness of the steel jacket is increased the confining pressure increase linearly, which is certainly not exact. Obviously the confinement pressure is a function of the concrete strength as well as the strength of the steel jacket.

In an effort to achieve a more comprehensive approach for determining the confinement pressure and steel jacket thickness, first the following approximation is borrowed from the Seismic Retrofitting Manual for Highway Bridges:

$$f'_{cc} = 1.5 f'_c \qquad \text{.....................(2-15)}$$

where; f'_{cc} = confined concrete compressive strength

f'_{co} = unconfined concrete compressive strength

Using the Richart model for confinement with k_l = 2.0 as suggested by Lam and Teng, then:

$$1.5 f'_{co} = f'_{co} + 2 f_l \qquad \text{.......................(5-2)}$$

or, $f_l = \frac{f'_{co}}{4}$ ....(5-3)

Referring to Equation (2-13)

$$f_l \cdot D = 2t_j \cdot f_j \qquad \text{................(2-13)}$$

where, D = the diameter of the column.

t_j = the steel jacket thickness

f_l = confining pressure

f_j = the stress induced in the jacket.

and assuming that when the concrete reaches its ultimate confined strength, the steel jacket reaches its yield stress, then Equation 2-13 becomes:

$$f_l D = 2t_j f_{jy} \qquad \text{....................(5-4)}$$

where, f_{jy} = yield strength of steel jacket.

substituting the value of f_l determined in Equation 5-3;

$$\frac{f'_{co}}{4} D = 2t_j f_{jy} \qquad \text{.....................(5-5)}$$

rearranging we get;

$$t_j = \frac{f'_{co}}{8 f_{jy}} D \qquad \text{.....................(5-6)}$$

and the confining pressure is calculated by:

$f_l = \frac{2t_j f_{jy}}{D}$, for circular jacket with diameter D ....(5-7)

It is interesting to notice the resemblance between Equation 5-7 derived analytically and Equation 2-37 that was derived empirically by

Orito et. al. for confined concrete with unbonded steel tubes.

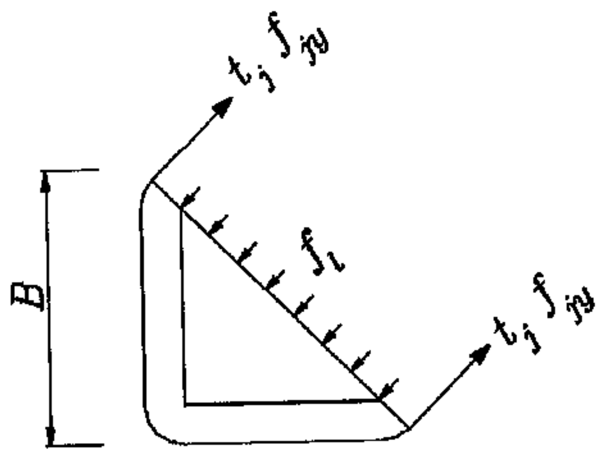


Figure 5.1 Square Shaped Steel Jacket

For square columns the largest splitting force develops along the diagonal of the square jacket with face B, as shown in Figure 5.1, thus;

$$f_l B\sqrt{2} = 2t_j f_{jy} \qquad \text{.....................(5-8)}$$

Suter and Pinzelli, 2001, tested square columns with jacket fillet size ranging from 5 to 25 mm. The effectiveness of confinement varied linearly, and reached its maximum when the fillet size was 25mm, consequently the thickness of a square shaped steel jacket becomes,

$$t_j = \frac{f'_{co}}{f_{jy}} \cdot \frac{B\sqrt{2}}{8 \cdot k_f} \qquad \text{.....................(5-9)}$$

where, B = dimension of the face of the steel jacket.

$k_f = \frac{r_f}{25}$ = effectiveness of confinement.

r_f = radius of steel jacket corner fillet; $5mm \le r_f \le 25mm$

and, $f_l = \frac{2t_j f_{jy}}{B\sqrt{2}}$, for square jackets ....(5-10)

5.2 Load Carrying Capacity of Retrofitted Column:

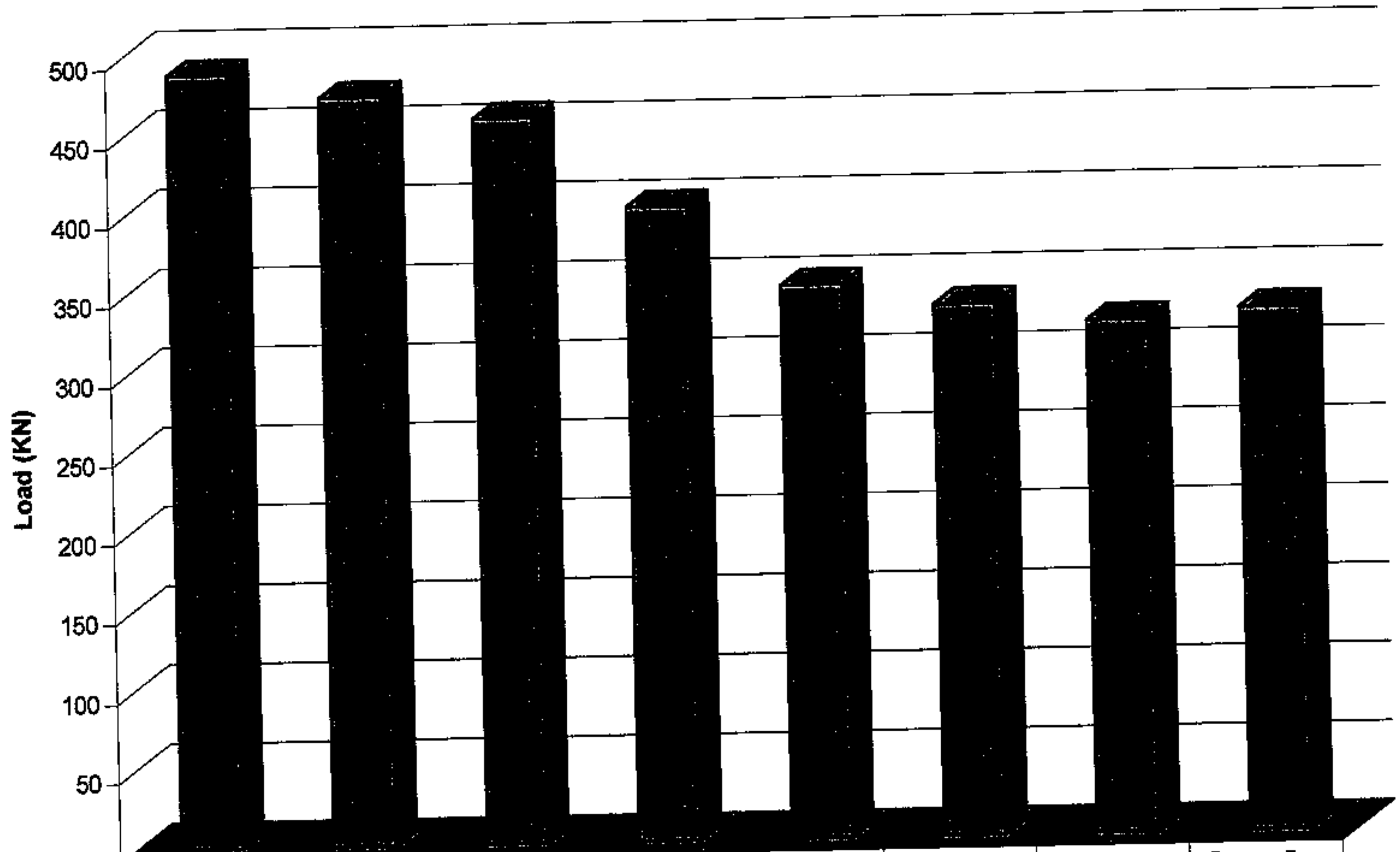
Test results described earlier as well as other previous researches have shown an increase in strength of retrofitted columns, this increase is primarily contributed to the confinement effect of the steel jacket. Different techniques were investigated in this research, the appropriate load carrying capacity of both of them is compared here to existing design procedures, and where appropriate, suggestions are made to reach a better solution to the problem.

5.2.1 Retrofitted Columns with Full Jacket (Group 1)

As discussed in the previous chapter, a retrofitted column with full steel jacket is similar in behavior to the concrete filled tube, and they both had the same ultimate strength. Using the ACI 318M-99 building code requirements for concrete filled tubes, Equation 2-25, first it has to be established whether to use confined strength of concrete as suggested by some researchers or unconfined strength of concrete as required by the ACI and agreed with the results of some researchers as well.

The confined concrete strength according to several models is calculated in Appendix C. These values are substituted in the following Equation 5-11; which is the ACI formula (Equation 2-28) without the safety factor (0.85); then results are compared in Figure 5.2.

$$P_u = 0.85 A_c f_c + A_s f_y + A_j f_{jy} \qquad \text{.....................(5-11)}$$



	Richart Model	Mander Model	Mirmiran Model	Lam & Teng Model	Vintzileou Model	Unconfined strength	Group 1-a (avarage)	Group 5-a (avarage)
Load (KN)	488.122	472.699	457.502	400.131	349.191	335.275	323.5	330

Figure 5.2 . Comparison between Several Confined Concrete Models and Unconfined Concrete Strength using ACI Equation 5-11 with Experimental Results of Retrofitted Column and Concrete Filled Tubes.

where, P_u = ultimate axial strength of column.

A_s = cross sectional area of reinforcing steel bars.

f_y = yield strength of reinforcing steel bars.

A_j = cross sectional area of steel tube (jacket).

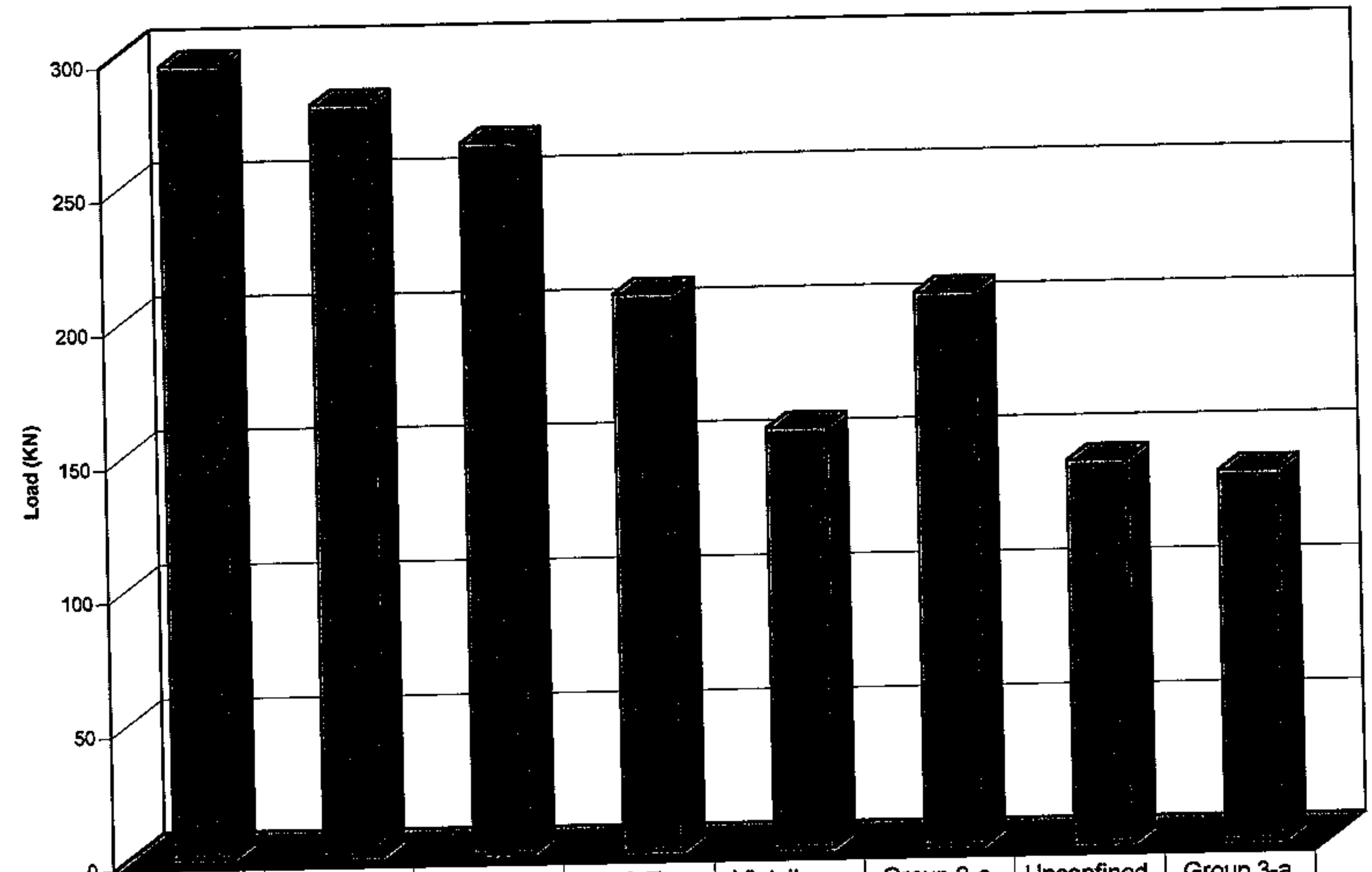
f_{jy} = yield strength of steel tube (jacket).

A_c = cross sectional area of concrete.

f_c = crushing strength of concrete (confined or unconfined as the case may require)

From the above chart, it is obvious that for square concrete columns retrofitted with full jackets and for concrete filled tubes CFT, the unconfined concrete strength generates the nearest prediction of the ultimate strength of the retrofitted column and the concrete filled tubes. This conclusion is consistent with the ACI and LRFD requirements and specifications, and follows the same experimental finding of Orito et. al.

The reason for this behavior is that when the steel jacket is subjected to axial loading, unlike the reinforcing bars embedded inside the concrete mass, the only restraint to buckling of steel jacket is the bond strength between the grout and the steel jacket. When the retrofitted column is subjected to axial loading it starts to deflect axially. At the point where the steel jacket starts to yield in compression and the concrete has reached its maximum compressive unconfined strength the



	Richart Model	Mander Model	Mirmiran Model	Lam & Teng Model	Vintzileou Model	Group 2-a (avarage)	Unconfined strength	Group 3-a (avarage)
Load (KN)	296.122	280.699	265.502	208.131	157.191	206.76	143.275	138.5

Figure 5.3. Comparison between Several Confined Concrete Models and Unconfined Concrete Strength using ACI Equation 5-12 with Experimental Results of Confined Columns with Steel Jackets and Reinforce Concrete columns.

From the above chart it is obvious that the values of the ultimate load derived from the Lam & Teng model are the closest to the experimental values of the confined concrete columns with steel jackets. The unconfined strength also conforms to the experimental results of the unconfined reinforced concrete columns.

Also it is interesting to note that the confined strength of concrete according to Lam & Teng $f'_{cc} = 24.42 N/mm^2$, i.e. its 1.6 larger than f'_{co}. This verifies the approximation made earlier in section 5.1 based on the Seismic Retrofitting Manual for Highway Bridges Equation 2-15.

5.3 Eccentric Loading

When the retrofitted columns are subjected to bending through eccentrically placed loading, they attained the same compressive strength as their concentrically loaded counterparts, their expected eccentric loading was calculated using the unconfined strength. Group 1-b and Group 5-b expected theoretical strength was calculated as a composite member using unconfined strength and taking into account the contribution of the steel jacket (tube). Group 3-b expected theoretical strength was calculated as a reinforced concrete column with unconfined concrete strength. Group 2-b expected theoretical strength was calculated the same as Group 3-b but with confined concrete strength. The results are shown in Figure 5-4 to Figure 5-6.

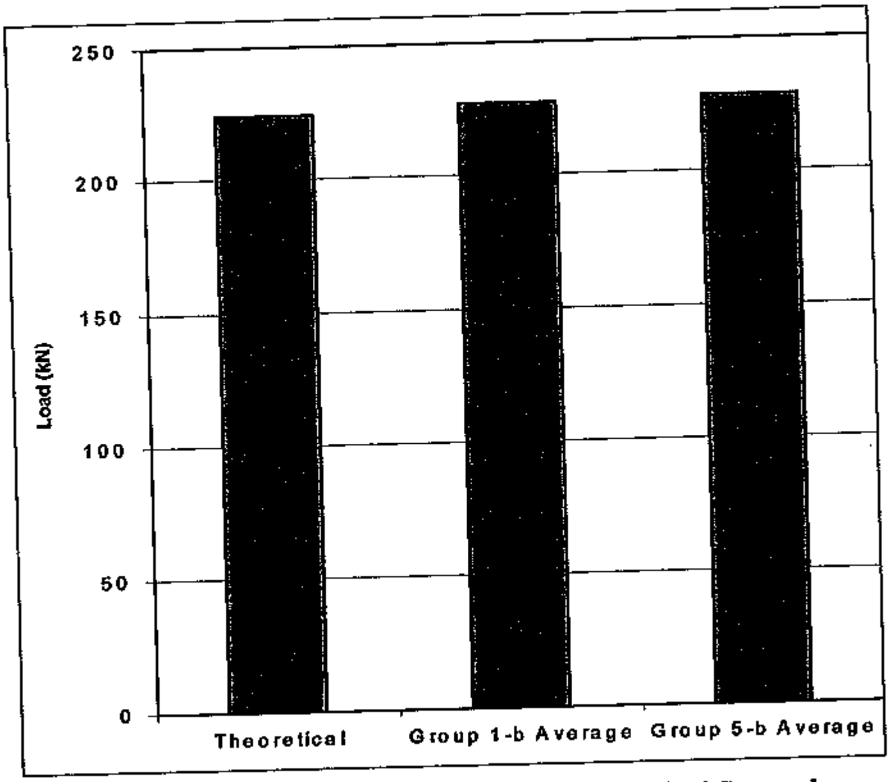


Figure 5.4.Group 1-b and Group 5-b Vs. Theoretical Strength

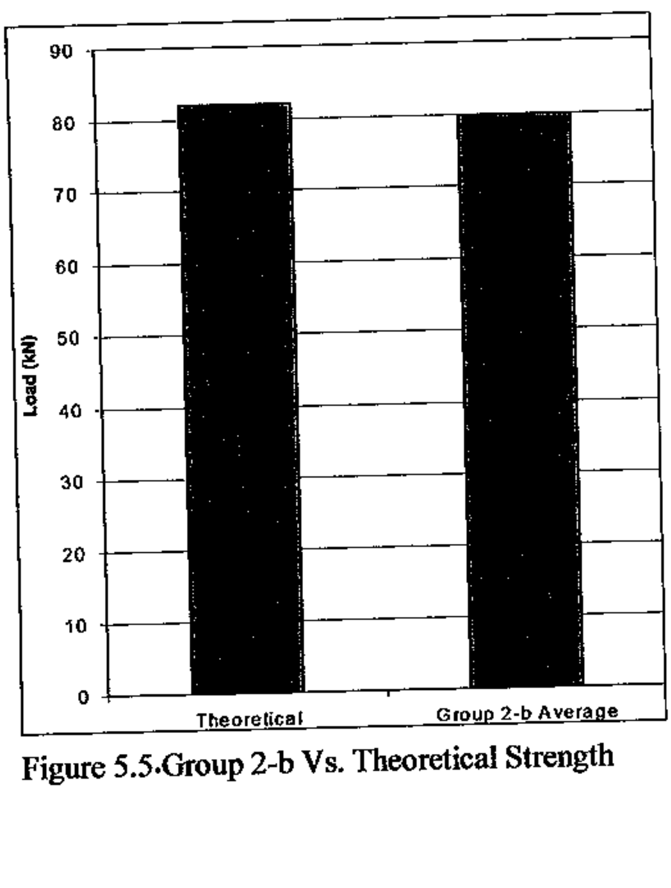


Figure 5.5.Group 2-b Vs. Theoretical Strength

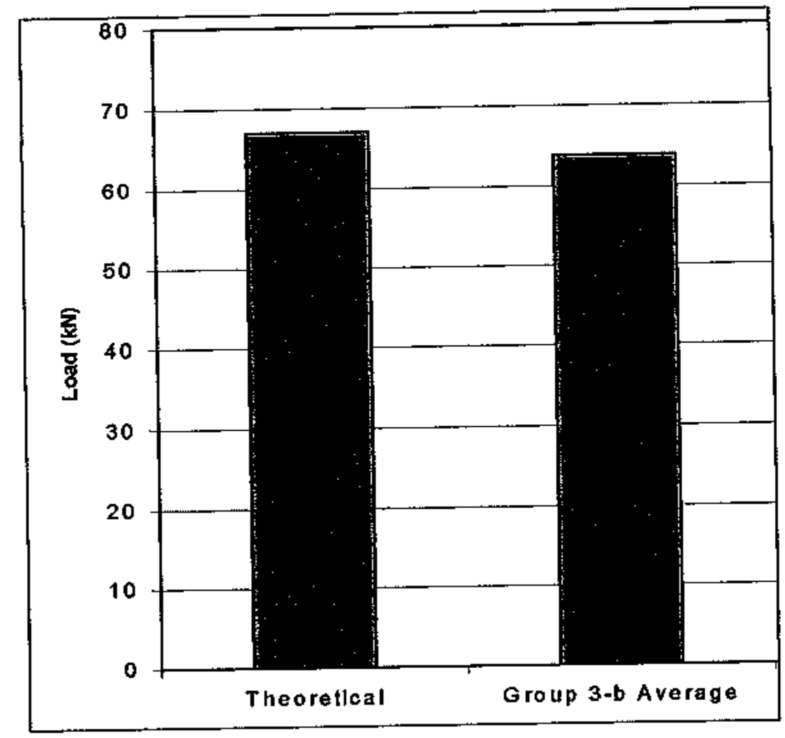


Figure 5.6.Group 3-b Vs. Theoretical Strength

The above charts show that although members retrofitted with full steel jacket and concrete filled steel tube are not expected to utilize confined strength of concrete, they are able to accommodate for higher eccentric loading, because of the composite action between the reinforced concrete and the steel jacket (tube).

This result coupled with a closer look at Figure 4.20 lead to the conclusion that if retrofitting is designed to achieve higher ductility of the retrofitted column, then confinement with steel jackets that does not extend the full length of the column is sufficient. But if the desired outcome is to have an increase in flexural strength as well as ductility of the column, then retrofitting with full steel jacket that extends to the full height of the column might be a better alternative.

6. CONCLUSIONS, RECOMMENDATIONS AND FURTHER RESEARCH

Several researchers have investigated the confined strength of concrete, models for the strength prediction was proposed and tested. Retrofitting of concrete columns depended primarily on the confinement of concrete with the purpose of increasing the compressive strength of the concrete core; therefore it was recommended that the steel jacket should not extend to the full height of the retrofitted reinforced concrete column. This investigation was aimed at:

- Determine the theoretical model that can best predict the confined concrete strength of square reinforced concrete columns.
- Explore the possibility of retrofitting square reinforced concrete columns with full height steel jackets.
- Suggest a design criterion for retrofitting square reinforced concrete columns.

The findings of this investigation are summarized in this chapter, and based on that, recommendations and further research possibilities are suggested.

6.1 Conclusions:

1. Retrofitting square reinforced concrete columns with full steel jackets can enhance the compressive strength of these columns more than double the strength of the original column.

2. The behavior of the retrofitted columns with full steel jackets resembles the behavior of the concrete filled tubes.

3. The ACI formula for concrete filled tubes estimates the strength of these columns using the unconfined concrete strength.

4. The minimum thickness of the steel tube (jacket) recommended by ACI and LRFD is conservative for confinement of concrete. Steel tube (jacket) thickness from Equation 5-9 has been sufficient for the confinement of concrete core of square reinforced concrete columns.

5. The Richart model with k_l=2.0 as put forward by Lam & Teng was found to best predict the confined strength of square reinforced concrete columns retrofitted with steel jackets that do not extend the full height of the column.

6. The confined strength of concrete is approximately 1.5 times the unconfined strength.

7. Confinement of reinforced concrete columns with steel jackets can enhance the ductility of the column, and retrofitting with full steel jacket can enhance ductility as well as the ultimate strength of the column subjected to eccentric axial loading.

8. The Mirmiran stress-strain relation was found to best represent the behavior of concrete core in retrofitted square reinforced concrete columns.

6.2 Recommendations:

1. Determine the thickness of the steel jacket for retrofitting square reinforced concrete columns using the proposed equation:

$$t_j = \frac{f'_{co}}{f_{jy}} \cdot \frac{B\sqrt{2}}{8 \cdot k_f} \qquad \text{....................(5-9)}$$

2. Use confined concrete strength as defined by Richart, Equation 2-4, with k_l=2.0 to calculate the ultimate confined compressive strength of the section when retrofitting square reinforced concrete columns with confining steel jackets that do not extend the full height of the column. The steel jacket will only contribute to the confinement of concrete.

3. Use the ACI 318M-99 design criteria, Equation 2-28, to determine the design strength of retrofitted square reinforced concrete columns with

steel jackets that extend the full height of the column or when designing a new square shaped concrete filled steel tube with unconfined concrete strength.

6.3 Further Research:

1. Investigate the possibility of extending the same retrofitting technique of full steel jacket to rectangular reinforced concrete columns with steel jackets.

2. Investigate the validity of Equation 5-6 derived to determine the thickness of the steel tube (jacket) of circular sections.

3. Explore several connection details between the steel jacket and other members of the structure (slabs / footings) when using full height jacket to ensure load transfer from those elements to the steel jacket, and suggest the best detail for each case of loading.

4. Tests have shown increased ductility of the retrofitted column, therefore this retrofitting system should be investigated under cyclic loading to establish its applicability in seismic areas.

5. This experiment was done with no initial loading on the reinforced concrete columns. Tests done by Ersoy *et al.* (1993) have shown that repaired columns with jackets made under load could carry only 50%

made under load could carry only 50% of the reference specimens. This might be expected when depending on confined strength, the concrete will have dilated laterally when subjected to initial loading, therefore part of the expected "passive stress" would have been wasted. But when using the full-height steel jacket the behavior might be different, since the additional load is shared with the steel jacket itself and confinement of concrete is not required. An experimental program to verify this is suggested.

Appendix A: Concrete Mix Design

The concrete used was designed to have a cylinder crushing strength after 28 days $f_{co}' = 15\ N/mm^2$ (equivalent to 28 day cube strength $f_{cu}' = 18.75\ N/mm^2$). Following is the design proportions used to achieve this strength:

Design mean cylinder strength required = $15 \times 1.16 = 17.4 N/mm^2$

Design equivalent mean cube strength = $\dfrac{17.4}{0.80} = 21.75 N/mm^2$

Water to cement ratio = 0.65

Maximum course aggregate size used: 10 *mm*

Minimum fine aggregate size used: retained on sieve # 200

Aggregate to cement ratio by weight = 4.9 : 1.0

Fine Aggregate = 35% of total aggregate

Course Aggregate = 65% of total aggregate

Slump test value = 80 *mm*

Cube strength results are listed in table A-1

Cube Number	$f_{cu}' = N/mm^2$
1	23.2
2	23.55556
3	20.97778
4	21.2
5	23.33333

Table A-1 Cube Strength

Actual mean Cube strength = 22.45 N/mm^2

Standard deviation = 1.122

Number of cubes below design strength = none

Appendix B: Computer Model

Sample input and output file for the computer model used in determining the ultimate strength of retrofitted square reinforced concrete columns with steel jacket is presented here.

B.1 Input File:

```
; File C:\Khair\Thesis\SAP results\Load=150KN.$2k saved 6/25/02 18:53:51 in N-
mm

SYSTEM
  DOF=UX,UY,UZ,RX,RY,RZ  LENGTH=mm  FORCE=N  PAGE=SECTIONS

JOINT
  1  X=-61  Y=-61  Z=0
  2  X=-61  Y=-61  Z=750
  3  X=-61  Y=61  Z=0
  4  X=-61  Y=61  Z=750
  5  X=61  Y=-61  Z=0
  6  X=61  Y=-61  Z=750
  7  X=61  Y=61  Z=0
  8  X=61  Y=61  Z=750
  9  X=45  Y=45  Z=700
  10  X=61  Y=61  Z=700
  11  X=45  Y=-45  Z=700
  12  X=61  Y=-61  Z=700
  13  X=-61  Y=-61  Z=700
  14  X=-45  Y=-45  Z=700
  15  X=-61  Y=61  Z=700
  16  X=-45  Y=45  Z=700
  17  X=45  Y=45  Z=650
  18  X=61  Y=61  Z=650
  19  X=45  Y=-45  Z=650
  20  X=61  Y=-61  Z=650
  21  X=-61  Y=-61  Z=650
  22  X=-45  Y=-45  Z=650
  23  X=-45  Y=-45  Z=0
  24  X=-45  Y=45  Z=0
  25  X=45  Y=45  Z=0
  26  X=45  Y=-45  Z=0
  27  X=-61  Y=61  Z=650
  28  X=-45  Y=45  Z=650
  29  X=45  Y=45  Z=600
  30  X=61  Y=61  Z=600
  31  X=-45  Y=-45  Z=750
  32  X=-45  Y=45  Z=750
  33  X=45  Y=45  Z=750
  34  X=45  Y=-45  Z=750
  35  X=45  Y=-45  Z=600
```

36	X=61	Y=-61	Z=600
37	X=-61	Y=-61	Z=600
38	X=-45	Y=-45	Z=600
39	X=-61	Y=61	Z=600
40	X=-45	Y=45	Z=600
41	X=45	Y=45	Z=550
42	X=61	Y=61	Z=550
43	X=45	Y=-45	Z=550
44	X=61	Y=-61	Z=550
45	X=-61	Y=-61	Z=550
46	X=-45	Y=-45	Z=550
47	X=-61	Y=61	Z=550
48	X=-45	Y=45	Z=550
49	X=45	Y=45	Z=500
50	X=61	Y=61	Z=500
51	X=45	Y=-45	Z=500
52	X=61	Y=-61	Z=500
53	X=-61	Y=-61	Z=500
54	X=-45	Y=-45	Z=500
55	X=-61	Y=61	Z=500
56	X=-45	Y=45	Z=500
57	X=45	Y=45	Z=450
58	X=61	Y=61	Z=450
59	X=45	Y=-45	Z=450
60	X=61	Y=-61	Z=450
61	X=-61	Y=-61	Z=450
62	X=-45	Y=-45	Z=450
63	X=-61	Y=61	Z=450
64	X=-45	Y=45	Z=450
65	X=45	Y=45	Z=400
66	X=61	Y=61	Z=400
67	X=45	Y=-45	Z=400
68	X=61	Y=-61	Z=400
69	X=-61	Y=-61	Z=400
70	X=-45	Y=-45	Z=400
71	X=-61	Y=61	Z=400
72	X=-45	Y=45	Z=400
73	X=45	Y=45	Z=350
74	X=61	Y=61	Z=350
75	X=45	Y=-45	Z=350
76	X=61	Y=-61	Z=350
77	X=-61	Y=-61	Z=350
78	X=-45	Y=-45	Z=350
79	X=-61	Y=61	Z=350
80	X=-45	Y=45	Z=350
81	X=45	Y=45	Z=300
82	X=61	Y=61	Z=300
83	X=45	Y=-45	Z=300
84	X=61	Y=-61	Z=300
85	X=-61	Y=-61	Z=300
86	X=-45	Y=-45	Z=300
87	X=-61	Y=61	Z=300
88	X=-45	Y=45	Z=300
89	X=45	Y=45	Z=250
90	X=61	Y=61	Z=250
91	X=45	Y=-45	Z=250
92	X=61	Y=-61	Z=250
93	X=-61	Y=-61	Z=250
94	X=-45	Y=-45	Z=250
95	X=-61	Y=61	Z=250
96	X=-45	Y=45	Z=250

```
97  X=45  Y=45  Z=200
98  X=61  Y=61  Z=200
99  X=45  Y=-45  Z=200
100  X=61  Y=-61  Z=200
101  X=-61  Y=-61  Z=200
102  X=-45  Y=-45  Z=200
103  X=-61  Y=61  Z=200
104  X=-45  Y=45  Z=200
105  X=45  Y=45  Z=150
106  X=61  Y=61  Z=150
107  X=45  Y=-45  Z=150
108  X=61  Y=-61  Z=150
109  X=-61  Y=-61  Z=150
110  X=-45  Y=-45  Z=150
111  X=-61  Y=61  Z=150
112  X=-45  Y=45  Z=150
113  X=45  Y=45  Z=100
114  X=61  Y=61  Z=100
115  X=45  Y=-45  Z=100
116  X=61  Y=-61  Z=100
117  X=-61  Y=-61  Z=100
118  X=-45  Y=-45  Z=100
119  X=-61  Y=61  Z=100
120  X=-45  Y=45  Z=100
121  X=45  Y=45  Z=50
122  X=61  Y=61  Z=50
123  X=45  Y=-45  Z=50
124  X=61  Y=-61  Z=50
125  X=-61  Y=-61  Z=50
126  X=-45  Y=-45  Z=50
127  X=-61  Y=61  Z=50
128  X=-45  Y=45  Z=50

RESTRAINT
  ADD=1  DOF=U1,U2,U3,R1,R2,R3
  ADD=3  DOF=U1,U2,U3,R1,R2,R3
  ADD=5  DOF=U1,U2,U3,R1,R2,R3
  ADD=7  DOF=U1,U2,U3,R1,R2,R3
  ADD=23  DOF=U1,U2,U3,R1,R2,R3
  ADD=24  DOF=U1,U2,U3,R1,R2,R3
  ADD=25  DOF=U1,U2,U3,R1,R2,R3
  ADD=26  DOF=U1,U2,U3,R1,R2,R3

PATTERN
  NAME=DEFAULT

MATERIAL
  NAME=STEEL  IDES=S  M=7.827E-09  W=7.682E-05
    T=0  E=200000  U=.3  A=.0000117  FY=400
  NAME=CONC  IDES=C  M=2.401E-09  W=2.356E-05
    T=0  E=15298  U=.2  A=.0000099

FRAME SECTION
  NAME=FSEC2 MAT=CONC SH=R T=22.5,22.5 A=253.125 J=7332.437 I=2669.678,10678.71
AS=210.9375,210.9375
  NAME=FSEC1 MAT=STEEL SH=R T=1,120 A=120 J=39.79 I=10,144000 AS=100,100

SHELL SECTION
  NAME=SSEC1  MAT=CONC   TYPE=Plate,Thin  TH=5

FRAME
```

```
71  J=93,94   SEC=FSEC2  NSEG=4  ANG=0
72  J=95,96   SEC=FSEC2  NSEG=4  ANG=0
73  J=97,98   SEC=FSEC2  NSEG=4  ANG=0
74  J=99,100   SEC=FSEC2  NSEG=4  ANG=0
75  J=101,102  SEC=FSEC2  NSEG=4  ANG=0
76  J=103,104  SEC=FSEC2  NSEG=4  ANG=0
77  J=105,106  SEC=FSEC2  NSEG=4  ANG=0
78  J=107,108  SEC=FSEC2  NSEG=4  ANG=0
79  J=109,110  SEC=FSEC2  NSEG=4  ANG=0
80  J=111,112  SEC=FSEC2  NSEG=4  ANG=0
81  J=113,114  SEC=FSEC2  NSEG=4  ANG=0
82  J=115,116  SEC=FSEC2  NSEG=4  ANG=0
83  J=117,118  SEC=FSEC2  NSEG=4  ANG=0
84  J=119,120  SEC=FSEC2  NSEG=4  ANG=0
85  J=121,122  SEC=FSEC2  NSEG=4  ANG=0
86  J=123,124  SEC=FSEC2  NSEG=4  ANG=0
87  J=125,126  SEC=FSEC2  NSEG=4  ANG=0
88  J=127,128  SEC=FSEC2  NSEG=4  ANG=0
89  J=42,30   SEC=FSEC1  NSEG=2  ANG=0
90  J=30,18   SEC=FSEC1  NSEG=2  ANG=0
91  J=18,10   SEC=FSEC1  NSEG=2  ANG=0
92  J=10,8   SEC=FSEC1  NSEG=2  ANG=0
93  J=13,12   SEC=FSEC1  NSEG=4  ANG=0
94  J=15,10   SEC=FSEC1  NSEG=4  ANG=0
95  J=13,15   SEC=FSEC1  NSEG=4  ANG=0
96  J=12,10   SEC=FSEC1  NSEG=4  ANG=0
97  J=14,16   SEC=FSEC2  NSEG=4  ANG=0
98  J=9,11   SEC=FSEC2  NSEG=4  ANG=0
99  J=16,9   SEC=FSEC2  NSEG=4  ANG=0
100  J=11,14   SEC=FSEC2  NSEG=4  ANG=0
101  J=21,20   SEC=FSEC1  NSEG=4  ANG=0
102  J=27,18   SEC=FSEC1  NSEG=4  ANG=0
103  J=21,27   SEC=FSEC1  NSEG=4  ANG=0
104  J=20,18   SEC=FSEC1  NSEG=4  ANG=0
105  J=22,28   SEC=FSEC2  NSEG=4  ANG=0
106  J=17,19   SEC=FSEC2  NSEG=4  ANG=0
107  J=28,17   SEC=FSEC2  NSEG=4  ANG=0
108  J=19,22   SEC=FSEC2  NSEG=4  ANG=0
109  J=37,36   SEC=FSEC1  NSEG=4  ANG=0
110  J=39,30   SEC=FSEC1  NSEG=4  ANG=0
111  J=37,39   SEC=FSEC1  NSEG=4  ANG=0
112  J=36,30   SEC=FSEC1  NSEG=4  ANG=0
113  J=38,40   SEC=FSEC2  NSEG=4  ANG=0
114  J=29,35   SEC=FSEC2  NSEG=4  ANG=0
115  J=40,29   SEC=FSEC2  NSEG=4  ANG=0
116  J=35,38   SEC=FSEC2  NSEG=4  ANG=0
117  J=45,44   SEC=FSEC1  NSEG=4  ANG=0
118  J=47,42   SEC=FSEC1  NSEG=4  ANG=0
119  J=45,47   SEC=FSEC1  NSEG=4  ANG=0
120  J=44,42   SEC=FSEC1  NSEG=4  ANG=0
121  J=46,48   SEC=FSEC2  NSEG=4  ANG=0
122  J=41,43   SEC=FSEC2  NSEG=4  ANG=0
123  J=48,41   SEC=FSEC2  NSEG=4  ANG=0
124  J=43,46   SEC=FSEC2  NSEG=4  ANG=0
125  J=53,52   SEC=FSEC1  NSEG=4  ANG=0
126  J=55,50   SEC=FSEC1  NSEG=4  ANG=0
127  J=53,55   SEC=FSEC1  NSEG=4  ANG=0
128  J=52,50   SEC=FSEC1  NSEG=4  ANG=0
129  J=54,56   SEC=FSEC2  NSEG=4  ANG=0
130  J=49,51   SEC=FSEC2  NSEG=4  ANG=0
131  J=56,49   SEC=FSEC2  NSEG=4  ANG=0
```

```
132  J=51,54   SEC=FSEC2  NSEG=4  ANG=0
133  J=61,60   SEC=FSEC1  NSEG=4  ANG=0
134  J=63,58   SEC=FSEC1  NSEG=4  ANG=0
135  J=61,63   SEC=FSEC1  NSEG=4  ANG=0
136  J=60,58   SEC=FSEC1  NSEG=4  ANG=0
137  J=62,64   SEC=FSEC2  NSEG=4  ANG=0
138  J=57,59   SEC=FSEC2  NSEG=4  ANG=0
139  J=64,57   SEC=FSEC2  NSEG=4  ANG=0
140  J=59,62   SEC=FSEC2  NSEG=4  ANG=0
141  J=69,68   SEC=FSEC1  NSEG=4  ANG=0
142  J=71,66   SEC=FSEC1  NSEG=4  ANG=0
143  J=69,71   SEC=FSEC1  NSEG=4  ANG=0
144  J=68,66   SEC=FSEC1  NSEG=4  ANG=0
145  J=70,72   SEC=FSEC2  NSEG=4  ANG=0
146  J=65,67   SEC=FSEC2  NSEG=4  ANG=0
147  J=72,65   SEC=FSEC2  NSEG=4  ANG=0
148  J=67,70   SEC=FSEC2  NSEG=4  ANG=0
149  J=77,76   SEC=FSEC1  NSEG=4  ANG=0
150  J=79,74   SEC=FSEC1  NSEG=4  ANG=0
151  J=77,79   SEC=FSEC1  NSEG=4  ANG=0
152  J=76,74   SEC=FSEC1  NSEG=4  ANG=0
153  J=78,80   SEC=FSEC2  NSEG=4  ANG=0
154  J=73,75   SEC=FSEC2  NSEG=4  ANG=0
155  J=80,73   SEC=FSEC2  NSEG=4  ANG=0
156  J=75,78   SEC=FSEC2  NSEG=4  ANG=0
157  J=85,84   SEC=FSEC1  NSEG=4  ANG=0
158  J=87,82   SEC=FSEC1  NSEG=4  ANG=0
159  J=85,87   SEC=FSEC1  NSEG=4  ANG=0
160  J=84,82   SEC=FSEC1  NSEG=4  ANG=0
161  J=86,88   SEC=FSEC2  NSEG=4  ANG=0
162  J=81,83   SEC=FSEC2  NSEG=4  ANG=0
163  J=88,81   SEC=FSEC2  NSEG=4  ANG=0
164  J=83,86   SEC=FSEC2  NSEG=4  ANG=0
165  J=93,92   SEC=FSEC1  NSEG=4  ANG=0
166  J=95,90   SEC=FSEC1  NSEG=4  ANG=0
167  J=93,95   SEC=FSEC1  NSEG=4  ANG=0
168  J=92,90   SEC=FSEC1  NSEG=4  ANG=0
169  J=94,96   SEC=FSEC2  NSEG=4  ANG=0
170  J=89,91   SEC=FSEC2  NSEG=4  ANG=0
171  J=96,89   SEC=FSEC2  NSEG=4  ANG=0
172  J=91,94   SEC=FSEC2  NSEG=4  ANG=0
173  J=101,100   SEC=FSEC1  NSEG=4  ANG=0
174  J=103,98   SEC=FSEC1  NSEG=4  ANG=0
175  J=101,103   SEC=FSEC1  NSEG=4  ANG=0
176  J=100,98   SEC=FSEC1  NSEG=4  ANG=0
177  J=102,104   SEC=FSEC2  NSEG=4  ANG=0
178  J=97,99   SEC=FSEC2  NSEG=4  ANG=0
179  J=104,97   SEC=FSEC2  NSEG=4  ANG=0
180  J=99,102   SEC=FSEC2  NSEG=4  ANG=0
181  J=109,108   SEC=FSEC1  NSEG=4  ANG=0
182  J=111,106   SEC=FSEC1  NSEG=4  ANG=0
183  J=109,111   SEC=FSEC1  NSEG=4  ANG=0
184  J=108,106   SEC=FSEC1  NSEG=4  ANG=0
185  J=110,112   SEC=FSEC2  NSEG=4  ANG=0
186  J=105,107   SEC=FSEC2  NSEG=4  ANG=0
187  J=112,105   SEC=FSEC2  NSEG=4  ANG=0
188  J=107,110   SEC=FSEC2  NSEG=4  ANG=0
189  J=117,116   SEC=FSEC1  NSEG=4  ANG=0
190  J=119,114   SEC=FSEC1  NSEG=4  ANG=0
191  J=117,119   SEC=FSEC1  NSEG=4  ANG=0
192  J=116,114   SEC=FSEC1  NSEG=4  ANG=0
```

```
193  J=118,120  SEC=FSEC2  NSEG=4  ANG=0
194  J=113,115  SEC=FSEC2  NSEG=4  ANG=0
195  J=120,113  SEC=FSEC2  NSEG=4  ANG=0
196  J=115,118  SEC=FSEC2  NSEG=4  ANG=0
197  J=125,124  SEC=FSEC1  NSEG=4  ANG=0
198  J=127,122  SEC=FSEC1  NSEG=4  ANG=0
199  J=125,127  SEC=FSEC1  NSEG=4  ANG=0
200  J=124,122  SEC=FSEC1  NSEG=4  ANG=0
201  J=126,128  SEC=FSEC2  NSEG=4  ANG=0
202  J=121,123  SEC=FSEC2  NSEG=4  ANG=0
203  J=128,121  SEC=FSEC2  NSEG=4  ANG=0
204  J=123,126  SEC=FSEC2  NSEG=4  ANG=0
205  J=3,127  SEC=FSEC1  NSEG=2  ANG=0
206  J=127,119  SEC=FSEC1  NSEG=2  ANG=0
207  J=119,111  SEC=FSEC1  NSEG=2  ANG=0
208  J=111,103  SEC=FSEC1  NSEG=2  ANG=0
209  J=103,95  SEC=FSEC1  NSEG=2  ANG=0
210  J=95,87  SEC=FSEC1  NSEG=2  ANG=0
211  J=87,79  SEC=FSEC1  NSEG=2  ANG=0
212  J=79,71  SEC=FSEC1  NSEG=2  ANG=0
213  J=71,63  SEC=FSEC1  NSEG=2  ANG=0
214  J=63,55  SEC=FSEC1  NSEG=2  ANG=0
215  J=55,47  SEC=FSEC1  NSEG=2  ANG=0
216  J=47,39  SEC=FSEC1  NSEG=2  ANG=0
217  J=39,27  SEC=FSEC1  NSEG=2  ANG=0
218  J=27,15  SEC=FSEC1  NSEG=2  ANG=0
219  J=15,4  SEC=FSEC1  NSEG=2  ANG=0
220  J=5,124  SEC=FSEC1  NSEG=2  ANG=0
221  J=124,116  SEC=FSEC1  NSEG=2  ANG=0
222  J=116,108  SEC=FSEC1  NSEG=2  ANG=0
223  J=108,100  SEC=FSEC1  NSEG=2  ANG=0
224  J=100,92  SEC=FSEC1  NSEG=2  ANG=0
225  J=92,84  SEC=FSEC1  NSEG=2  ANG=0
226  J=84,76  SEC=FSEC1  NSEG=2  ANG=0
227  J=76,68  SEC=FSEC1  NSEG=2  ANG=0
228  J=68,60  SEC=FSEC1  NSEG=2  ANG=0
229  J=60,52  SEC=FSEC1  NSEG=2  ANG=0
230  J=52,44  SEC=FSEC1  NSEG=2  ANG=0
231  J=44,36  SEC=FSEC1  NSEG=2  ANG=0
232  J=36,20  SEC=FSEC1  NSEG=2  ANG=0
233  J=20,12  SEC=FSEC1  NSEG=2  ANG=0
234  J=12,6  SEC=FSEC1  NSEG=2  ANG=0
235  J=1,125  SEC=FSEC1  NSEG=2  ANG=0
236  J=125,117  SEC=FSEC1  NSEG=2  ANG=0
237  J=117,109  SEC=FSEC1  NSEG=2  ANG=0
238  J=109,101  SEC=FSEC1  NSEG=2  ANG=0
239  J=101,93  SEC=FSEC1  NSEG=2  ANG=0
240  J=93,85  SEC=FSEC1  NSEG=2  ANG=0
241  J=85,77  SEC=FSEC1  NSEG=2  ANG=0
242  J=77,69  SEC=FSEC1  NSEG=2  ANG=0
243  J=69,61  SEC=FSEC1  NSEG=2  ANG=0
244  J=61,53  SEC=FSEC1  NSEG=2  ANG=0
245  J=53,45  SEC=FSEC1  NSEG=2  ANG=0
246  J=45,37  SEC=FSEC1  NSEG=2  ANG=0
247  J=37,21  SEC=FSEC1  NSEG=2  ANG=0
248  J=21,13  SEC=FSEC1  NSEG=2  ANG=0
249  J=13,2  SEC=FSEC1  NSEG=2  ANG=0
250  J=25,121  SEC=FSEC2  NSEG=2  ANG=0
251  J=121,113  SEC=FSEC2  NSEG=2  ANG=0
252  J=113,105  SEC=FSEC2  NSEG=2  ANG=0
253  J=105,97  SEC=FSEC2  NSEG=2  ANG=0
```

```
254  J=97,89   SEC=FSEC2  NSEG=2  ANG=0
255  J=89,81   SEC=FSEC2  NSEG=2  ANG=0
256  J=81,73   SEC=FSEC2  NSEG=2  ANG=0
257  J=73,65   SEC=FSEC2  NSEG=2  ANG=0
258  J=65,57   SEC=FSEC2  NSEG=2  ANG=0
259  J=57,49   SEC=FSEC2  NSEG=2  ANG=0
260  J=49,41   SEC=FSEC2  NSEG=2  ANG=0
261  J=41,29   SEC=FSEC2  NSEG=2  ANG=0
262  J=29,17   SEC=FSEC2  NSEG=2  ANG=0
263  J=17,9  SEC=FSEC2  NSEG=2  ANG=0
264  J=9,33  SEC=FSEC2  NSEG=2  ANG=0
265  J=23,126   SEC=FSEC2  NSEG=2  ANG=0
266  J=126,118   SEC=FSEC2  NSEG=2  ANG=0
267  J=118,110   SEC=FSEC2  NSEG=2  ANG=0
268  J=110,102   SEC=FSEC2  NSEG=2  ANG=0
269  J=102,94   SEC=FSEC2  NSEG=2  ANG=0
270  J=94,86   SEC=FSEC2  NSEG=2  ANG=0
271  J=86,78   SEC=FSEC2  NSEG=2  ANG=0
272  J=78,70   SEC=FSEC2  NSEG=2  ANG=0
273  J=70,62   SEC=FSEC2  NSEG=2  ANG=0
274  J=62,54   SEC=FSEC2  NSEG=2  ANG=0
275  J=54,46   SEC=FSEC2  NSEG=2  ANG=0
276  J=46,38   SEC=FSEC2  NSEG=2  ANG=0
277  J=38,22   SEC=FSEC2  NSEG=2  ANG=0
278  J=22,14   SEC=FSEC2  NSEG=2  ANG=0
279  J=14,31   SEC=FSEC2  NSEG=2  ANG=0
280  J=24,128   SEC=FSEC2  NSEG=2  ANG=0
281  J=128,120   SEC=FSEC2  NSEG=2  ANG=0
282  J=120,112   SEC=FSEC2  NSEG=2  ANG=0
283  J=112,104   SEC=FSEC2  NSEG=2  ANG=0
284  J=104,96   SEC=FSEC2  NSEG=2  ANG=0
285  J=96,88   SEC=FSEC2  NSEG=2  ANG=0
286  J=88,80   SEC=FSEC2  NSEG=2  ANG=0
287  J=80,72   SEC=FSEC2  NSEG=2  ANG=0
288  J=72,64   SEC=FSEC2  NSEG=2  ANG=0
289  J=64,56   SEC=FSEC2  NSEG=2  ANG=0
290  J=56,48   SEC=FSEC2  NSEG=2  ANG=0
291  J=48,40   SEC=FSEC2  NSEG=2  ANG=0
292  J=40,28   SEC=FSEC2  NSEG=2  ANG=0
293  J=28,16   SEC=FSEC2  NSEG=2  ANG=0
294  J=16,32   SEC=FSEC2  NSEG=2  ANG=0
295  J=26,123   SEC=FSEC2  NSEG=2  ANG=0
296  J=123,115   SEC=FSEC2  NSEG=2  ANG=0
297  J=115,107   SEC=FSEC2  NSEG=2  ANG=0
298  J=107,99   SEC=FSEC2  NSEG=2  ANG=0
299  J=99,91   SEC=FSEC2  NSEG=2  ANG=0
300  J=91,83   SEC=FSEC2  NSEG=2  ANG=0
301  J=83,75   SEC=FSEC2  NSEG=2  ANG=0
302  J=75,67   SEC=FSEC2  NSEG=2  ANG=0
303  J=67,59   SEC=FSEC2  NSEG=2  ANG=0
304  J=59,51   SEC=FSEC2  NSEG=2  ANG=0
305  J=51,43   SEC=FSEC2  NSEG=2  ANG=0
306  J=43,35   SEC=FSEC2  NSEG=2  ANG=0
307  J=35,19   SEC=FSEC2  NSEG=2  ANG=0
308  J=19,11   SEC=FSEC2  NSEG=2  ANG=0
309  J=11,34   SEC=FSEC2  NSEG=2  ANG=0

SHELL
  1  J=4,32,8,33   SEC=SSEC1
  2  J=33,34,8,6   SEC=SSEC1
  3  J=34,31,6,2   SEC=SSEC1
```

```
4 J=31,32,2,4  SEC=SSEC1
5 J=14,11,16,9  SEC=SSEC1
6 J=22,19,28,17  SEC=SSEC1
7 J=38,35,40,29  SEC=SSEC1
8 J=46,43,48,41  SEC=SSEC1
9 J=31,34,32,33  SEC=SSEC1
10  J=54,51,56,49  SEC=SSEC1
11  J=62,59,64,57  SEC=SSEC1
12  J=70,67,72,65  SEC=SSEC1
13  J=78,75,80,73  SEC=SSEC1
14  J=86,83,88,81  SEC=SSEC1
15  J=94,91,96,89  SEC=SSEC1
16  J=102,99,104,97  SEC=SSEC1
17  J=110,107,112,105  SEC=SSEC1
18  J=118,115,120,113  SEC=SSEC1
19  J=126,123,128,121  SEC=SSEC1
21  J=15,16,10,9  SEC=SSEC1
22  J=9,11,10,12  SEC=SSEC1
23  J=11,14,12,13  SEC=SSEC1
24  J=14,16,13,15  SEC=SSEC1
25  J=27,28,18,17  SEC=SSEC1
26  J=17,19,18,20  SEC=SSEC1
27  J=19,22,20,21  SEC=SSEC1
28  J=22,28,21,27  SEC=SSEC1
29  J=39,40,30,29  SEC=SSEC1
30  J=29,35,30,36  SEC=SSEC1
31  J=35,38,36,37  SEC=SSEC1
32  J=38,40,37,39  SEC=SSEC1
33  J=47,48,42,41  SEC=SSEC1
34  J=41,43,42,44  SEC=SSEC1
35  J=43,46,44,45  SEC=SSEC1
36  J=46,48,45,47  SEC=SSEC1
37  J=55,56,50,49  SEC=SSEC1
38  J=49,51,50,52  SEC=SSEC1
39  J=51,54,52,53  SEC=SSEC1
40  J=54,56,53,55  SEC=SSEC1
41  J=63,64,58,57  SEC=SSEC1
42  J=57,59,58,60  SEC=SSEC1
43  J=59,62,60,61  SEC=SSEC1
44  J=62,64,61,63  SEC=SSEC1
45  J=71,72,66,65  SEC=SSEC1
46  J=65,67,66,68  SEC=SSEC1
47  J=67,70,68,69  SEC=SSEC1
48  J=70,72,69,71  SEC=SSEC1
49  J=79,80,74,73  SEC=SSEC1
50  J=73,75,74,76  SEC=SSEC1
51  J=75,78,76,77  SEC=SSEC1
52  J=78,80,77,79  SEC=SSEC1
53  J=87,88,82,81  SEC=SSEC1
54  J=81,83,82,84  SEC=SSEC1
55  J=83,86,84,85  SEC=SSEC1
56  J=86,88,85,87  SEC=SSEC1
57  J=95,96,90,89  SEC=SSEC1
58  J=89,91,90,92  SEC=SSEC1
59  J=91,94,92,93  SEC=SSEC1
60  J=94,96,93,95  SEC=SSEC1
61  J=103,104,98,97  SEC=SSEC1
62  J=97,99,98,100  SEC=SSEC1
63  J=99,102,100,101  SEC=SSEC1
64  J=102,104,101,103  SEC=SSEC1
65  J=111,112,106,105  SEC=SSEC1
```

```
  66  J=105,107,106,108   SEC=SSEC1
  67  J=107,110,108,109   SEC=SSEC1
  68  J=110,112,109,111   SEC=SSEC1
  69  J=119,120,114,113   SEC=SSEC1
  70  J=113,115,114,116   SEC=SSEC1
  71  J=115,118,116,117   SEC=SSEC1
  72  J=118,120,117,119   SEC=SSEC1
  73  J=127,128,122,121   SEC=SSEC1
  74  J=121,123,122,124   SEC=SSEC1
  75  J=123,126,124,125   SEC=SSEC1
  76  J=126,128,125,127   SEC=SSEC1

LOAD
  NAME=LOAD1  SW=1
    TYPE=UNIFORM
      ADD=1  UZ=-10
      ADD=2  UZ=-10
      ADD=3  UZ=-10
      ADD=4  UZ=-10
      ADD=9  UZ=-10

OUTPUT
  ELEM=JOINT  TYPE=REAC  LOAD=LOAD1
  ELEM=JOINT  TYPE=DISP  LOAD=LOAD1

END

; The following data is used for graphics, design and pushover analysis.
; If changes are made to the analysis data above, then the following data
; should be checked for consistency.
SAP2000 V7.12 SUPPLEMENTAL DATA
  GRID GLOBAL  X "1"  -61
  GRID GLOBAL  X "2"  -45
  GRID GLOBAL  X "3"  45
  GRID GLOBAL  X "4"  61
  GRID GLOBAL  Y "5"  -61
  GRID GLOBAL  Y "6"  -45
  GRID GLOBAL  Y "7"  45
  GRID GLOBAL  Y "8"  61
  GRID GLOBAL  Z "9"  0
  GRID GLOBAL  Z "10"  750
  MATERIAL STEEL FY 400
  MATERIAL CONC FYREBAR 200 FYSHEAR 100 FC 15 FCSHEAR 15
  CONCRETESECTION FSEC2 COLUMN  COVER .5 REBAR RR-3-3
  STATICLOAD LOAD1 TYPE DEAD
END SUPPLEMENTAL DATA
```

B.2 Output File:

```
                         S A P 2 0 0 0 (R)

                    Structural Analysis Programs

                      Nonlinear Version 7.11

                      Copyright (C) 1978-1999
                  COMPUTERS AND STRUCTURES, INC.
                       All rights reserved
           This copy of SAP2000 is for the exclusive use of
                           THE LICENSEE
```

25 Jun 2002 18:53:51

Program SAP2000 Nonlinear Version 7.11
File:LOAD=150KN.OUT

Page

1

D I S P L A C E M E N T   D E G R E E S   O F   F R E E D O M

(A) = Active DOF, equilibrium equation
(-) = Restrained DOF, reaction computed
(+) = Constrained DOF
(>) = External substructure DOF
( ) = Null DOF

JOINTS	UX	UY	UZ	RX	RY	RZ
1	-	-	-	-	-	-
2	A	A	A	A	A	A
3	-	-	-	-	-	-
4	A	A	A	A	A	A
5	-	-	-	-	-	-
6	A	A	A	A	A	A
7	-	-	-	-	-	-
8 TO 22	A	A	A	A	A	A
23 TO 26	-	-	-	-	-	-
27 TO 128	A	A	A	A	A	A

Program SAP2000 Nonlinear Version 7.11
File:LOAD=150KN.OUT

Page

2

A S S E M B L E D   J O I N T   M A S S E S

IN GLOBAL COORDINATES

JOINT	UX	UY	UZ	RX	RY	RZ
1	2.35E-05	2.35E-05	2.35E-05	.000000	.000000	.000000
2	0.000156	0.000156	0.000156	.000000	.000000	.000000
3	2.35E-05	2.35E-05	2.35E-05	.000000	.000000	.000000
4	0.000156	0.000156	0.000156	.000000	.000000	.000000
5	2.35E-05	2.35E-05	2.35E-05	.000000	.000000	.000000
6	0.000156	0.000156	0.000156	.000000	.000000	.000000
7	2.35E-05	2.35E-05	2.35E-05	.000000	.000000	.000000
8	0.000156	0.000156	0.000156	.000000	.000000	.000000
9	0.000126	0.000126	0.000126	.000000	.000000	.000000
10	0.000179	0.000179	0.000179	.000000	.000000	.000000
11	0.000126	0.000126	0.000126	.000000	.000000	.000000
12	0.000179	0.000179	0.000179	.000000	.000000	.000000
13	0.000179	0.000179	0.000179	.000000	.000000	.000000
14	0.000126	0.000126	0.000126	.000000	.000000	.000000
15	0.000179	0.000179	0.000179	.000000	.000000	.000000

16	0.000126	0.000126	0.000126	.000000	.000000	.000000
17	0.000126	0.000126	0.000126	.000000	.000000	.000000
18	0.000179	0.000179	0.000179	.000000	.000000	.000000
19	0.000126	0.000126	0.000126	.000000	.000000	.000000
20	0.000179	0.000179	0.000179	.000000	.000000	.000000
21	0.000179	0.000179	0.000179	.000000	.000000	.000000
22	0.000126	0.000126	0.000126	.000000	.000000	.000000
23	1.52E-05	1.52E-05	1.52E-05	.000000	.000000	.000000
24	1.52E-05	1.52E-05	1.52E-05	.000000	.000000	.000000
25	1.52E-05	1.52E-05	1.52E-05	.000000	.000000	.000000
26	1.52E-05	1.52E-05	1.52E-05	.000000	.000000	.000000
27	0.000179	0.000179	0.000179	.000000	.000000	.000000
28	0.000126	0.000126	0.000126	.000000	.000000	.000000
29	0.000126	0.000126	0.000126	.000000	.000000	.000000
30	0.000179	0.000179	0.000179	.000000	.000000	.000000
31	0.000111	0.000111	0.000111	.000000	.000000	.000000
32	0.000111	0.000111	0.000111	.000000	.000000	.000000
33	0.000111	0.000111	0.000111	.000000	.000000	.000000
34	0.000111	0.000111	0.000111	.000000	.000000	.000000
35	0.000126	0.000126	0.000126	.000000	.000000	.000000
36	0.000179	0.000179	0.000179	.000000	.000000	.000000
37	0.000179	0.000179	0.000179	.000000	.000000	.000000
38	0.000126	0.000126	0.000126	.000000	.000000	.000000
39	0.000179	0.000179	0.000179	.000000	.000000	.000000
40	0.000126	0.000126	0.000126	.000000	.000000	.000000
41	0.000126	0.000126	0.000126	.000000	.000000	.000000
42	0.000179	0.000179	0.000179	.000000	.000000	.000000
43	0.000126	0.000126	0.000126	.000000	.000000	.000000
44	0.000179	0.000179	0.000179	.000000	.000000	.000000
45	0.000179	0.000179	0.000179	.000000	.000000	.000000
46	0.000126	0.000126	0.000126	.000000	.000000	.000000
47	0.000179	0.000179	0.000179	.000000	.000000	.000000
48	0.000126	0.000126	0.000126	.000000	.000000	.000000
49	0.000126	0.000126	0.000126	.000000	.000000	.000000
50	0.000179	0.000179	0.000179	.000000	.000000	.000000
51	0.000126	0.000126	0.000126	.000000	.000000	.000000
52	0.000179	0.000179	0.000179	.000000	.000000	.000000
53	0.000179	0.000179	0.000179	.000000	.000000	.000000
54	0.000126	0.000126	0.000126	.000000	.000000	.000000
55	0.000179	0.000179	0.000179	.000000	.000000	.000000
56	0.000126	0.000126	0.000126	.000000	.000000	.000000
57	0.000126	0.000126	0.000126	.000000	.000000	.000000
58	0.000179	0.000179	0.000179	.000000	.000000	.000000
59	0.000126	0.000126	0.000126	.000000	.000000	.000000
60	0.000179	0.000179	0.000179	.000000	.000000	.000000
61	0.000179	0.000179	0.000179	.000000	.000000	.000000
62	0.000126	0.000126	0.000126	.000000	.000000	.000000
63	0.000179	0.000179	0.000179	.000000	.000000	.000000
64	0.000126	0.000126	0.000126	.000000	.000000	.000000
65	0.000126	0.000126	0.000126	.000000	.000000	.000000
66	0.000179	0.000179	0.000179	.000000	.000000	.000000
67	0.000126	0.000126	0.000126	.000000	.000000	.000000
68	0.000179	0.000179	0.000179	.000000	.000000	.000000
69	0.000179	0.000179	0.000179	.000000	.000000	.000000
70	0.000126	0.000126	0.000126	.000000	.000000	.000000
71	0.000179	0.000179	0.000179	.000000	.000000	.000000
72	0.000126	0.000126	0.000126	.000000	.000000	.000000
73	0.000126	0.000126	0.000126	.000000	.000000	.000000
74	0.000179	0.000179	0.000179	.000000	.000000	.000000
75	0.000126	0.000126	0.000126	.000000	.000000	.000000
76	0.000179	0.000179	0.000179	.000000	.000000	.000000

77	0.000179	0.000179	0.000179	.000000	.000000	.000000
78	0.000126	0.000126	0.000126	.000000	.000000	.000000
79	0.000179	0.000179	0.000179	.000000	.000000	.000000
80	0.000126	0.000126	0.000126	.000000	.000000	.000000
81	0.000126	0.000126	0.000126	.000000	.000000	.000000
82	0.000179	0.000179	0.000179	.000000	.000000	.000000
83	0.000126	0.000126	0.000126	.000000	.000000	.000000
84	0.000179	0.000179	0.000179	.000000	.000000	.000000
85	0.000179	0.000179	0.000179	.000000	.000000	.000000
86	0.000126	0.000126	0.000126	.000000	.000000	.000000
87	0.000179	0.000179	0.000179	.000000	.000000	.000000
88	0.000126	0.000126	0.000126	.000000	.000000	.000000
89	0.000126	0.000126	0.000126	.000000	.000000	.000000
90	0.000179	0.000179	0.000179	.000000	.000000	.000000
91	0.000126	0.000126	0.000126	.000000	.000000	.000000
92	0.000179	0.000179	0.000179	.000000	.000000	.000000
93	0.000179	0.000179	0.000179	.000000	.000000	.000000
94	0.000126	0.000126	0.000126	.000000	.000000	.000000
95	0.000179	0.000179	0.000179	.000000	.000000	.000000
96	0.000126	0.000126	0.000126	.000000	.000000	.000000
97	0.000126	0.000126	0.000126	.000000	.000000	.000000
98	0.000179	0.000179	0.000179	.000000	.000000	.000000
99	0.000126	0.000126	0.000126	.000000	.000000	.000000
100	0.000179	0.000179	0.000179	.000000	.000000	.000000
101	0.000179	0.000179	0.000179	.000000	.000000	.000000
102	0.000126	0.000126	0.000126	.000000	.000000	.000000
103	0.000179	0.000179	0.000179	.000000	.000000	.000000
104	0.000126	0.000126	0.000126	.000000	.000000	.000000
105	0.000126	0.000126	0.000126	.000000	.000000	.000000
106	0.000179	0.000179	0.000179	.000000	.000000	.000000
107	0.000126	0.000126	0.000126	.000000	.000000	.000000
108	0.000179	0.000179	0.000179	.000000	.000000	.000000
109	0.000179	0.000179	0.000179	.000000	.000000	.000000
110	0.000126	0.000126	0.000126	.000000	.000000	.000000
111	0.000179	0.000179	0.000179	.000000	.000000	.000000
112	0.000126	0.000126	0.000126	.000000	.000000	.000000
113	0.000126	0.000126	0.000126	.000000	.000000	.000000
114	0.000179	0.000179	0.000179	.000000	.000000	.000000
115	0.000126	0.000126	0.000126	.000000	.000000	.000000
116	0.000179	0.000179	0.000179	.000000	.000000	.000000
117	0.000179	0.000179	0.000179	.000000	.000000	.000000
118	0.000126	0.000126	0.000126	.000000	.000000	.000000
119	0.000179	0.000179	0.000179	.000000	.000000	.000000
120	0.000126	0.000126	0.000126	.000000	.000000	.000000
121	0.000126	0.000126	0.000126	.000000	.000000	.000000
122	0.000179	0.000179	0.000179	.000000	.000000	.000000
123	0.000126	0.000126	0.000126	.000000	.000000	.000000
124	0.000179	0.000179	0.000179	.000000	.000000	.000000
125	0.000179	0.000179	0.000179	.000000	.000000	.000000
126	0.000126	0.000126	0.000126	.000000	.000000	.000000
127	0.000179	0.000179	0.000179	.000000	.000000	.000000
128	0.000126	0.000126	0.000126	.000000	.000000	.000000

T O T A L   A S S E M B L E D   J O I N T   M A S S E S

IN GLOBAL COORDINATES

	UX	UY	UZ	RX	RY	RZ
TOTAL	0.018303	0.018303	0.018303	.000000	.000000	.000000

T O T A L   A C C E L E R A T E D   M A S S   A N D   L O C A T I O N

TOTAL MASS ACTIVATED BY ACCELERATION LOADS, IN GLOBAL COORDINATES

	UX	UY	UZ
MASS	0.018149	0.018149	0.018149
X-LOC	-3.69E-16	-3.69E-16	-3.69E-16
Y-LOC	-2.05E-16	-2.05E-16	-2.05E-16
Z-LOC	397.016609	397.016609	397.016609

Program SAP2000 Nonlinear Version 7.11
File:LOAD=150KN.OUT

Page

3

J O I N T   D I S P L A C E M E N T S

TRANSLATIONS AND ROTATIONS, IN GLOBAL COORDINATES

LOAD   LOAD1 ------------------

JOINT	UX	UY	UZ	RX	RY	RZ
1	.000000	.000000	.000000	.000000	.000000	.000000
2	0.003630	0.004446	-0.908900	-0.000267	0.038051	-4.20E-06
3	.000000	.000000	.000000	.000000	.000000	.000000
4	0.003630	-0.004446	-0.908900	0.000267	0.038051	4.20E-06
5	.000000	.000000	.000000	.000000	-0.038051	4.20E-06
6	-0.003630	0.004446	-0.908900	-0.000267	.000000	.000000
7	.000000	.000000	.000000	.000000	.000000	.000000
8	-0.003630	-0.004446	-0.908900	0.000267	-0.038051	-4.20E-06
9	0.007424	0.001349	-1.296850	0.001596	-0.007733	7.31E-05
10	0.001846	0.000708	-0.876781	0.000201	-0.022874	1.94E-06
11	0.007424	-0.001349	-1.296850	-0.001596	-0.007733	-7.31E-05
12	0.001846	-0.000708	-0.876781	-0.000201	-0.022874	-1.94E-06
13	-0.001846	-0.000708	-0.876781	-0.000201	0.022874	1.94E-06
14	-0.007424	-0.001349	-1.296850	-0.001596	0.007733	7.31E-05
15	-0.001846	0.000708	-0.876781	0.000201	0.022874	-1.94E-06
16	-0.007424	0.001349	-1.296850	0.001596	0.007733	-7.31E-05
17	0.002403	0.001244	-1.096608	0.000988	-0.005387	3.17E-05
18	0.000632	0.001189	-0.831557	0.000127	-0.014459	6.83E-07
19	0.002403	-0.001244	-1.096608	-0.000988	-0.005387	-3.17E-05
20	0.000632	-0.001189	-0.831557	-0.000127	-0.014459	-6.83E-07
21	-0.000632	-0.001189	-0.831557	-0.000127	0.014459	6.83E-07
22	-0.002403	-0.001244	-1.096608	-0.000988	0.005387	3.17E-05
23	.000000	.000000	.000000	.000000	.000000	.000000
24	.000000	.000000	.000000	.000000	.000000	.000000
25	.000000	.000000	.000000	.000000	.000000	.000000
26	.000000	.000000	.000000	.000000	.000000	.000000
27	-0.000632	0.001189	-0.831557	0.000127	0.014459	-6.83E-07
28	-0.002403	0.001244	-1.096608	0.000988	0.005387	-3.17E-05
29	0.001771	0.000794	-0.945531	0.000625	-0.003357	2.57E-05
30	0.000429	0.000924	-0.778394	7.81E-05	-0.009118	5.84E-07
31	0.014591	0.004780	-1.578281	-0.003523	0.021118	-0.000175
32	0.014591	-0.004780	-1.578281	0.003523	0.021118	0.000175
33	-0.014591	-0.004780	-1.578281	0.003523	-0.021118	-0.000175
34	-0.014591	0.004780	-1.578281	-0.003523	-0.021118	0.000175
35	0.001771	-0.000794	-0.945531	-0.000625	-0.003357	-2.57E-05
36	0.000429	-0.000924	-0.778394	-7.81E-05	-0.009118	-5.84E-07

37	-0.000429	-0.000924	-0.778394	-7.81E-05	0.009118	5.84E-07
38	-0.001771	-0.000794	-0.945531	-0.000625	0.003357	2.57E-05
39	-0.000429	0.000924	-0.778394	7.81E-05	0.009118	-5.84E-07
40	-0.001771	0.000794	-0.945531	0.000625	0.003357	-2.57E-05
41	0.001105	0.000520	-0.825609	0.000394	-0.002120	1.64E-05
42	0.000267	0.000610	-0.720198	4.77E-05	-0.005752	3.69E-07
43	0.001105	-0.000520	-0.825609	-0.000394	-0.002120	-1.64E-05
44	0.000267	-0.000610	-0.720198	-4.77E-05	-0.005752	-3.69E-07
45	-0.000267	-0.000610	-0.720198	-4.77E-05	0.005752	3.69E-07
46	-0.001105	-0.000520	-0.825609	-0.000394	0.002120	1.64E-05
47	-0.000267	0.000610	-0.720198	4.77E-05	0.005752	-3.69E-07
48	-0.001105	0.000520	-0.825609	0.000394	0.002120	-1.64E-05
49	0.000698	0.000327	-0.725319	0.000248	-0.001337	1.04E-05
50	0.000168	0.000383	-0.658828	2.94E-05	-0.003630	2.33E-07
51	0.000698	-0.000327	-0.725319	-0.000248	-0.001337	-1.04E-05
52	0.000168	-0.000383	-0.658828	-2.94E-05	-0.003630	-2.33E-07
53	-0.000168	-0.000383	-0.658828	-2.94E-05	0.003630	2.33E-07
54	-0.000698	-0.000327	-0.725319	-0.000248	0.001337	1.04E-05
55	-0.000168	0.000383	-0.658828	2.94E-05	0.003630	-2.33E-07
56	-0.000698	0.000327	-0.725319	0.000248	0.001337	-1.04E-05
57	0.000439	0.000204	-0.637406	0.000156	-0.000844	6.50E-06
58	0.000106	0.000238	-0.595455	1.82E-05	-0.002291	1.46E-07
59	0.000439	-0.000204	-0.637406	-0.000156	-0.000844	-6.50E-06
60	0.000106	-0.000238	-0.595455	-1.82E-05	-0.002291	-1.46E-07
61	-0.000106	-0.000238	-0.595455	-1.82E-05	0.002291	1.46E-07
62	-0.000439	-0.000204	-0.637406	-0.000156	0.000844	6.50E-06
63	-0.000106	0.000238	-0.595455	1.82E-05	0.002291	-1.46E-07
64	-0.000439	0.000204	-0.637406	0.000156	0.000844	-6.50E-06
65	0.000277	0.000127	-0.557293	9.87E-05	-0.000533	4.07E-06
66	6.71E-05	0.000147	-0.530817	1.14E-05	-0.001447	9.12E-08
67	0.000277	-0.000127	-0.557293	-9.87E-05	-0.000533	-4.07E-06
68	6.71E-05	-0.000147	-0.530817	-1.14E-05	-0.001447	-9.12E-08
69	-6.71E-05	-0.000147	-0.530817	-1.14E-05	0.001447	9.12E-08
70	-0.000277	-0.000127	-0.557293	-9.87E-05	0.000533	4.07E-06
71	-6.71E-05	0.000147	-0.530817	1.14E-05	0.001447	-9.12E-08
72	-0.000277	0.000127	-0.557293	9.87E-05	0.000533	-4.07E-06
73	0.000174	7.99E-05	-0.482094	6.23E-05	-0.000336	2.55E-06
74	4.24E-05	9.20E-05	-0.465380	7.13E-06	-0.000914	5.72E-08
75	0.000174	-7.99E-05	-0.482094	-6.23E-05	-0.000336	-2.55E-06
76	4.24E-05	-9.20E-05	-0.465380	-7.13E-06	-0.000914	-5.72E-08
77	-4.24E-05	-9.20E-05	-0.465380	-7.13E-06	0.000914	5.72E-08
78	-0.000174	-7.99E-05	-0.482094	-6.23E-05	0.000336	2.55E-06
79	-4.24E-05	9.20E-05	-0.465380	7.13E-06	0.000914	-5.72E-08
80	-0.000174	7.99E-05	-0.482094	6.23E-05	0.000336	-2.55E-06
81	0.000110	5.03E-05	-0.409986	3.93E-05	-0.000212	1.61E-06
82	2.68E-05	5.77E-05	-0.399438	4.49E-06	-0.000578	3.60E-08
83	0.000110	-5.03E-05	-0.409986	-3.93E-05	-0.000212	-1.61E-06
84	2.68E-05	-5.77E-05	-0.399438	-4.49E-06	-0.000578	-3.60E-08
85	-2.68E-05	-5.77E-05	-0.399438	-4.49E-06	0.000578	3.60E-08
86	-0.000110	-5.03E-05	-0.409986	-3.93E-05	0.000212	1.61E-06
87	-2.68E-05	5.77E-05	-0.399438	4.49E-06	0.000578	-3.60E-08
88	-0.000110	5.03E-05	-0.409986	3.93E-05	0.000212	-1.61E-06
89	6.99E-05	3.19E-05	-0.339821	2.47E-05	-0.000134	1.02E-06
90	1.70E-05	3.64E-05	-0.333177	2.82E-06	-0.000365	2.28E-08
91	6.99E-05	-3.19E-05	-0.339821	-2.47E-05	-0.000134	-1.02E-06
92	1.70E-05	-3.64E-05	-0.333177	-2.82E-06	-0.000365	-2.28E-08
93	-1.70E-05	-3.64E-05	-0.333177	-2.82E-06	0.000365	2.28E-08
94	-6.99E-05	-3.19E-05	-0.339821	-2.47E-05	0.000134	1.02E-06
95	-1.70E-05	3.64E-05	-0.333177	2.82E-06	0.000365	-2.28E-08
96	-6.99E-05	3.19E-05	-0.339821	2.47E-05	0.000134	-1.02E-06
97	4.48E-05	2.04E-05	-0.270869	1.54E-05	-8.39E-05	6.52E-07

Appendix C: Calculation of Confined Strength of Concrete

The confined concrete strength according to the different models is calculate here:

Confinement pressure according to Equation 5-10:

$$f_l = \frac{2\times1\times400}{120\times\sqrt{2}} = 4.71N/mm^2$$

Richart Model:

$$f'_{cc} = 15 + 4.1\times4.71 = 37.2N/mm^2$$

Mander Model:

$$f'_{cc} = 15\left(-1.254 + 2.254\sqrt{1+\frac{7.94\times4.71}{15}} - 2\times\frac{4.71}{15}\right) = 34.96\,N/mm^2$$

Mirmiran Model:

$$f'_{cc} = 15 + 6\times(4.71)^{0.7} = 32.75N/mm^2$$

Lam & Teng Model:

$$f'_{cc} = 15 + 2\times4.71 = 24.42N/mm^2$$

Vintzileou Model:

$$\omega_w = \frac{4\times1}{120} = 0.0333$$

$$f'_{cc} = (1+.6\times1\times0.0333)(1.15-.0025\times15)\times15 = 17.02N/mm^2$$

REFERENCES

American Concrete Institute. *1999. Building Code Requirements for Structural Concrete (ACI 318M-99) and Commentary (ACI 318RM-99).* ACI, Michigan, U.S.A.

American Institute of Steel Construction. 1993. *Load and Resistance Factor Design for Structural Steel Buildings.* AISC, Illinois, U.S.A.

Ben-Zvi, E., Muller, G., and Rosenthal, I. 1966. Effect of Active Tri-axial Stress on the Strength of Concrete Elements. *Symposium on Reinforced Concrete Columns, ACI Publication SP-13*: 193-234.

Boyd, P. F., Cofer, W. F., and McLean, D. I. 1995. Seismic Performance of Steel Encased Concrete Columns under Flexural Loading. *ACI Structural Journal*, 92 (3): 355-364.

Chen, J. F., Yang, Z.J., Pan, X. M., and Holt, G. D. 2001. Effect of Test Method on Plate-to-Concrete Bond Strength. *Proceedings of the Fifth International Conference on Fiber-Reinforced Plastics for Reinforced concrete structures*, Cambridge, U.K.: 429-438.

Ersoy U., Tankut A.T., and Suleiman R. 1993. Behavior of Jacketed Columns. *ACI Structural Journal*, 90 (3): 288-293.

Federal Highway Administration, 1995. *Seismic Retrofitting Manual for Highway Bridges.* FHWA, Virginia, U.S.A.

Ferguson, P. M. 1967. *Reinforced Concrete Fundamentals*, Second Edition; John Wiley & Sons, New York, U.S.A.

Hipley P. 1997. Bridge Retrofit Construction Techniques. *Proceedings of the Second National Seismic Conference on Bridges and Highways*. California, U.S.A.

Kholusi, M. M. 1992. New Buildings Failure, a Phenomenon that Started in the Sixties. *Egypt's Engineering Syndicate Magazine*. 48 (433): 28-31.

Kholusi, M. M. 1992. Alexandria Buildings Collapsing on Top of it's Inhabitants. *Egypt's Engineering Syndicate Magazine*. 48 (438): 20-21.

Lam. L., and Teng, J. G. 2001. Strength Model for Circular Concrete Columns Confined by FRP Composites. *Proceedings of the Fifth International Conference on Fiber-Reinforced Plastics for Reinforced concrete structures*, Cambridge, U.K.: 835-844.

Mander, J. B., Priestly, M. J., and Park, R. 1988. Observed Stress-Strain Model of Confined Concrete. *Journal of Structural Engineering, ASCE*, 114 (8): 1827-1847.

Mander, J. B., Priestly, M. J., and Park, R. 1988. Theoretical Stress-Strain Model for Confined Concrete. *Journal of Structural Engineering, ASCE*, 114 (8): 1804-1826.

Mirmiran, A., Shahawy, M., Samman, M., El-Echary, H., Mastrapa, J. C., and Pico, O. 1998. Effect of Columns Parameters on FRP-Confined Concrete. *Journal of Composite Construction, ASCE*, 2 (4): 175-185.

Nilson, A. H., Winter, W. 1991. *Design of Concrete Structures*, Eleventh Edition; McGraw-Hill, Singapore.

Orito, Y., Sato, T., Tanaka, N., and Watanaba, Y. 1987. Study on The Unbonded Steel Tube Structure. *Proceedings, International Conference on Composite Construction in Steel and Concrete, ASCE*: 786-804.

Park, R., and Paulay T. 1975. *Reinforced Concrete Structures*, First Edition, John Wiley & Sons, Singapore.

Park, R. J. T., Priestly, M. J. N., and Walpole, W. R. 1983. Reinforced Concrete Bridge Piles. *Bulletin of the New Zealand National Society for Earthquake Engineering*, 16 (2): 123-140.

Popov, E. P. 1978. *Mechanics of Materials*, Second Edition, Prentice-Hall, N.J., U.S.A.

Richart, F. E., Brandtzaeg, A., and Brown, R. L. 1928. A Study of the Failure of Concrete under Combined Compressive Stresses. *Engineering Experimental Station, Bulletin No. 185*, University of Illinois, Illinois, U.S.A.

Sabnis, G. M., Harris, H. G., White, R. N., and Mirza, M. S. 1983. *Structural Modeling and Experimental Techniques*, First Edition, Prentice-Hall Inc., Englewood Cliffs, N.J.

Samman, M., Mirmiran, A. and Shahawy, M. 1998. Model of Concrete Confined by Fiber Composites. *Journal of Structural Engineering, ASEC*, 124 (9): 1025-1031.

Shams, M., and Saadeghvaziri M. A. 1997. State of the Art of Concrete-Filled Steel Tubular Columns. *ACI Structural Journal*, 94 (5): 558-571.

Shams, M., and Saadeghvaziri, M. A. 1999. Nonlinear Response of Concrete-Filled Steel Tubular Columns Under Axial Loading. *ACI Structural Journal*, 96 (6): 1009-1017.

Suter, R. and Pinzelli, R. 2001. Confinement of Concrete Columns with FRP Sheets. *Proceedings of the Fifth International Conference on Fiber-Reinforced Plastics for Reinforced concrete structures*, Cambridge, U.K. pp. 793-802.

UNDP/UNIDO project PER/79/015. 1983. *Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings*. United Nations Industrial Development Organization, Vienna, Austria.

Uomoto, T. 2001. Durability Considerations for FRP Reinforcements. *Proceedings of the Fifth International Conference on Fiber-Reinforced Plastics for Reinforced concrete structures*, Cambridge, U.K.:17-32.

Vintzileou, E. 2001. An Empirical Model for Predicting the Properties of Concrete Confined by Means of Composite Materials. *Proceedings of the Fifth International Conference on Fiber-Reinforced Plastics for Reinforced concrete structures*, Cambridge, U.K.: 845-853.

تلبيس الاعمده الخرسانيه مربعة الشكل بمقاطع معدنيه

إعداد
خيرالدين عصام خيرالدين بسيسو

المشرف
أ.د. سميح شاكر قاقيش

ملخص

تقوية أعمدة الخرسانه المسلحه لتقاوم أحمال إضافيه بواسطة التلبيس بالمقاطع المعدنيه ممارسه هندسيه معروفه لتقوية و تصليح الاعمده.

إن التلبيس بالمقاطع المعدنيه رخيص التكاليف و التقنيه المستخدمه لا تتطلب عمال مهره إلى حد كبير كما أن هذه المقاطع لا تشكل عائقا أو تخفض الفضاء و من السهل أن تفحص كما يمكن أن تركب بينما المبنى قيد الاستعمال.

يمكن إستخدام التلبيس بالمقاطع المعدنيه لإصلاح أبنيه قائمه و إطالة عمر هذه الابنيه أو لتحويل الابنيه لإستعمال مغاير للإستعمال الحالي و بالتالي تحمل أحمال إضافيه نتيجة ذلك.

من خلال هذه الاطروحه تم دراسة ٣٩ عموداً من الخرسانه المسلحه لدراسة تلبيس الاعمده المربعه بالمقاطع المعدنيه مع مراجعة إجراءات التصميم مما سيزود المهندسين الاردنيين بالإثبات العلمي لهذه التقنيه و يساعد في تطوير تصاميم تلبيس الاعمده الخرسانيه بسهولة و ثقه.

لتحقيق هذا الغرض تم مقارنة المعادلات الموجوده في أبحاث سابقه منذ العام ١٩٢٨ و حتى عام ٢٠٠١ لقوة الخرسانه المحصوره بالنتائج العمليه التي حصلنا عليها في هذا البحث للأعمده الخرسانيه المربعه و الملبسه بمقاطع معدنيه لا تستمر لكامل طول العمود الخرساني و تم إعتماد المعادله المناسبه لهذا النوع من الاعمده. كما تم إشتقاق معادله جديده لتحديد سماكة المقاطع المعدنيه لتحقيق أفضل حصر بواسطة تلبيس الاعمده المربعه بالمقاطع المعدنيه.

نتائج البحث أوضحت بأن الأعمده الخرسانيه المربعة الشكل و الملبسه بمقاطع معدنيـــه تمتـــد لكامل طول العمود الخرساني تحاكي في تصرفاتها الاعمـــده الخرســانيه المصبوبـــه داخـــل المقاطع المعدنيه مربعة الشكل. و قد تم مقارنة النتائج العمليه مع معـادلتي ACI 318M-99 و LRFD لتصميم هذا النوع من الاعمده و ذلك للتأكد من صحة هذه الفرضيه، كما تم عمـــل نموذج بإستخدام الحاسوب بمساعدة برنامج SAP-2000 و كانت النتائج متقاربه.

و تبين أيضاً أن الأعمده الخرسانيه المعرضه لأحامل لامركزيه و المحصوره بمقاطع معدنيـــه لا تمتد لكامل طول العمود الخرساني أصبحت لدنه أكثر من الأعمـــده غـــير الملبســـه إلا أن الأعمده الملبسه بمقاطع معدنيه تمتد لكامل طول العمود الخرساني حصلت على زياده في قـــوة التحمل أكثر بالإضافه للزياده في اللدونه.