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A COMPARATIVE ANALYSIS OF SEISMIC RETROFIT TECHNIQUES FOR REINFORCED CONCRETE BRIDGE COLUMNS

By

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ABSTRACT

Reinforced concrete bridge columns built prior to 1971 cannot withstand earthquake forces due to insufficient transverse reinforcement, poor detailing, and lap-splices in the plastic hinge area. In order to correct this deficiency, these piers must be reinforced by confinement in the axis perpendicular to the longitudinal bars. Experimental research as well as field experience has shown that there are a significant number of technologies that are effective column retrofit solutions for the thousands of bridges worldwide that are in need of upgrade.

There is a significant amount of documented research dealing with the technical effectiveness of individual techniques, however, very little research has compared these techniques for cost effectiveness and functionality. This dissertation addresses this issue by analyzing and conducting feasibility assessments of existing technologies, providing detailed retrofit designs for 6 circular and 6 square columns using four different approaches: steel jackets, FRP wrapping, concrete jackets and external pre-stressing and by pricing all of these designs. From this data, conclusions are drawn concerning the cost effectiveness of the four technologies. This study also analyses bridge assessment techniques.

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CONTENTS

LIST OF TABLES

LIST OF FIGURES

NOTATIONS

A_c	Effective shear area
Ag	Gross section area
A_g	Gross area of concrete section
A_{ps}	Nominal area of the prestressing strands
A_{sh}	Area of hoop
A_{sp}	Bar area of spiral reinforcement
B_j	Short principal diameter of elliptical jacket
c	Distance from neutral axis to extreme compression fiber of the section
cc	Concrete cover to the main longitudinal reinforcement.
C_j	Coefficient depending on the details of longitudinal reinforcement
D	Column diameter or outside dimension of the column
D_j	Diameter of steel jacket
E_{sj}	Modulus of steel jacket
f'_c	Cylinder strength
f'_c	Compressive strength of unconfined concrete
f'_{cc}	Compressive strength of confined concrete
f_{ps}	Stress level applied on prestressing strands
f_{pu}	Ultimate strain of prestressing strands
f_{pu}	Ultimate strength of prestressing strand
f_w	Wrap confinement force
F_{yh}	Hoop yield stress
f_{yj}	Yield strength of steel jacket
K	Moment enhancement ratio
L_h	Hinge length
$M_{i,ACI}$	Ideal flexural strength calculated by the ACI method
M_u	Ultimate moment
n	Number of spliced bars
P	Applied axial load
P	Perimeter line in the column cross section along the lap-spliced bar locations

s_p	Spacing between prestressing strands
s_{ps}	Spacing between prestressing strands
ϕ	Strength reduction factor
t_j	Thickness of jacket
V_c	Concrete shear
V_{cf}	shear contribution of carbon-fiber plastic sheet
V_{ci}	Initial concrete shear
V_i	Initial total shear
V_p	Shear due to axial load
V_s	Steel shear
V_{si}	Initial steel shear
α	Aspect ratio factor
α	Factor accounting for column aspect ratio
β	Modifier for longitudinal steel ratio
γ	Factor for loss of ductility
ε_{cu}	Ultimate compression strain in concrete
ε_{sm}	Strain at maximum stress in confining jacket
θ	Crack angle
θ_p	Plastic hinge rotation
λ_{cf}	Parameter representing ratio of FRP to concrete
μ	Displacement ductility ratio
ρ_s	Volumetric ratio of transverse reinforcement
ψ	Mechanical reinforcement ratio

Chapter 1

INTRODUCTION

1-1 GENERAL

Reinforced concrete bridges, especially those designed prior to 1971 have not performed as well as might be expected when subjected to seismic excitations. Recent earthquakes in California, Japan, Central and South America have shown that bridges suffer under the effect of earthquakes. The lack of redundancy and simplistic designs contribute to weakness and make bridges more susceptible to damage than buildings. Two span single column bridges are especially vulnerable.

Experiences since 1971 have brought about major changes to the design philosophies and codes, and as such, these newer bridges have generally performed well under seismic attack. However, pre-1971 structures did not perform well as evidenced by recent earthquakes in California and Japan. For instance in the 1989 Loma Pieta (California) earthquake, more than 80 bridges were damaged resulting in 40 deaths, $ 1.8 billion ($USD) in damage and severe economic disruption due to loss of major transportation routes. (Priestley 1996)

The marginal cost of providing higher levels of safety in new construction is relatively low, but the cost of retrofitting existing structures to bring them up to existing code levels can be very high. The number of deficient bridges is quite high and the funds are limited so that there is a need for a method of deciding which ones require upgrading and which technology is the most economical and best suited to the task. Codes and standards are poorly suited to this task. It is therefore important to develop procedures based on consideration of seismic risk, cost of retrofit, and the consequences of earthquakes with and without retrofit.

Worldwide, in seismically active zones, there are a significant number of bridges that have been built prior to 1971 that are vulnerable to severe damage. For instance, on the west coast of the USA (California, Oregon, Washington), there are approximately 4000 bridges in this category.(National Bridge Inventory). In British Columbia Canada, roughly 300 bridges are in this category and an estimated $250 million is needed to correct the problem (BC Ministry Transport 1997).

Older bridges typically had insufficient transverse reinforcement where #3(9.5mm) or #4(12.7mm) hoops were placed at 12"(300mm) centers regardless of the cross-section. The hoops were also simply lapped thus giving no anchorage. The large spacing of the hoops did not provide enough confinement for developing sufficient deformability or preventing the buckling of longitudinal reinforcement. Current design practices require approximately 8 times the transverse reinforcement of the pre-1971 designs. Inadequate detailing at the base of the column also contributed to poor seismic design. Often the starter bars extended a length of 20 times the longitudinal bar diameter from the foundation, which did not allow the column to develop its full flexural strength.

Under-designed reinforced concrete bridge columns pose a safety and economic threat to the general public and this threat needs to be addressed. Wholesale replacement of bridge columns is obviously not a viable solution so that a technically and economically feasible retrofit technique is required. Considerable effort in the form of research and field applications has been directed towards developing a technical solution to the problem of retrofitting reinforced concrete bridge columns.

1-2 PREVIOUS RESEARCH

For the past 25 years, extensive research has been conducted to address the technical issues surrounding seismically insufficient concrete columns. The focus of the research has been on the steel jacketing, concrete jacketing, FRP (fiber reinforced polymer), and post-tensioning steel cable (known as Retro-Belt) techniques.

Some of the early development work was done initially at the University of Canterbury, New Zealand and subsequently at the University of California at San Diego by Priestley, Park, Pauley, and Seible (Ang et al 1989) and focused on the steel jacketing solution. This technology proved to be quite successful and has been used extensively in California where thousands of columns have been retrofitted with steel jackets. The addition of a relatively thick concrete jacket is also effective in enhancing flexural strength, ductility and shear strength of columns. This technique is better suited for building columns than for bridge columns, but it has been used in some Japanese bridge retrofits.

Variations of the steel and concrete jacket techniques have been the subject of the more recent research. The University of Ottawa (Saatcioglu 2002) has developed a retrofitting technique using external pre-stressing strands or high steel straps that has been successfully tested in the lab showing that this method is effective in improving both strength and deformability of columns. The use of relatively thin steel jackets reinforced with various steel stiffeners in critical sections, have also been researched and proven to be efficient. Other research has demonstrated that a jacket of cement mortar and closely bound fine wire mesh will improve the shear strength of reinforced concrete columns.

The largest research effort in recent times is on the use of advanced composite column-jacketing systems. These systems range from hand lay-up of glass or carbon fabrics to pre-manufactured layered glass or carbon shell systems. The need to come up with a lightweight, non-corrosive, easily installed jacket that provides the same design strength benefits as steel or concrete is driving this research. Research has shown that FRP systems are a viable solution to column retrofit problem.

1-3 RESEARCH NEED

The focus of the research to date has been to demonstrate and prove the technical feasibility of individual retrofitting techniques for reinforced concrete columns. No literature was found addressing cost comparison of the different technologies. As such, the following needed to be researched:

- Inventory of retrofit technologies
- Detailed design of typical pre-1971 bridge columns using different technologies
- Detailed cost evaluation of these designs

- Recommendations on assessment strategies for existing bridges
- Presentation of results

1-4 OBJECTIVES AND SCOPE

The objective of this research project is to provide cost comparisons of retrofitting techniques in order to determine the most economical alternative. The comparison is also intended to reflect differences in material and labour needs, as well as complexities in construction processes.

The scope of the project is as follows:

- Description and analysis of 14 retrofit schemes that are steel, concrete, mortar, and FRP based.
- Evaluation of the 14 schemes against 12 cost/durability related characteristics. The purpose of this evaluation was to determine which ones should be evaluated in greater detail.
- Detailed design of 12 columns retrofitted with steel jackets, FRP, concrete jackets, and external pre-stressing. These design followed established procedures and used the program RcSection 1.3 for moment curvature analysis
- Detailed pricing of the above noted options
- Development of an assessment strategy for existing structures
- Presentation of results

CHAPTER 2

LITERATURE REVIEW

2-1 GENERAL

This Chapter provides a summary of the literature reviewed on available seismic retrofit techniques for concrete columns. The review is compiled chronologically, by technology and subject matter, and presented in the following sections.

2-2 LITERATURE ON ASSESMENT OF SEISMIC PERFORMANCE OF BRIDGE COLUMNS

Priestley and Park (1987) conducted wide-ranging research at the University of Canterbury, New Zealand, aimed at improving the understanding of the seismic performance of bridge structures. They summarized the work done from 1971 to 1987 on strength and ductility of concrete columns under simulated seismic loading. The research involved two stages of testing; i) axial load testing to investigate the stress-strain characteristics of confined concrete, and ii) flexural loading to determine the strength and ductility of columns with various cross-sectional shapes. The primary purpose of the research was to verify the methods used at the time for predicting flexural strength of reinforced concrete columns. As a result of this research, a new design method for confined concrete columns was proposed

Columns with various cross-sections were tested including: square, octagonal, diagonally loaded square, hollow circular, and hollow square. Two different testing configurations were used to allow for different aspect ratios. Variables tested included cross section shapes, aspect ratio, axial load level, the amount, and configuration of confining reinforcement and the influence of lapping longitudinal reinforcement in the plastic range. The lateral loading was done on a standard test pattern. The first cycle was used to determine the flexural strength and yield displacement. Subsequent loadings consisted of two cycles at ductility factors of $\mu = \pm 2$ to ± 6. Axial load was kept constant during testing. The theoretical ideal moment capacity was calculated using the ACI rectangular stress block for concrete in compression. The design of the transverse reinforcement for the test columns was according to the New Zealand Code of Practice for the Design of Concrete Structures in effect at the time of the study.

In conducting the experimental tests, the authors were primarily concerned with:

1) the available displacement-ductility factor

2) the stability of the load displacement hysteresis loops

3) the peak compression strain on the confined concrete

4) the length of the plastic hinge region

The result of the tests indicated the following:

- Recorded hysteresis loops at ductility levels of up to $\mu = 6$ were very stable
- Measured flexural strength was substantially above the predicted level, particularly for columns with high axial loads

- Squat columns (L/D = 2) performed better than slender (L/D = 4) columns
- Column ductility was not significantly influenced by the transverse reinforcement steel grade.
- Hollow circular columns with thin walls did not represent a viable design option
- Lapped splices in the plastic region was found not to be recommended
- The data did not indicate any dependence of the plastic hinge length on the axial load ratio, longitudinal reinforcement ratio, and yield strength of longitudinal reinforcement.

The tests conducted in this study indicated that the flexural strength calculated by the ACI concrete compressive stress block was conservative. As such the authors proposed an alternate design equation $M_u = \phi * K * M_{i,ACI}$ where $M_{i,ACI}$ is the ideal flexural strength calculated by the ACI method, K is the moment enhancement ratio and ϕ is a strength reduction factor. This revised formula is less conservative than the existing ACI method and therefore would result in a reduction of longitudinal reinforcement and make the structure more economical. The authors also suggested a design sequence as follow:

1. Select a global displacement ductility factor for the entire structure
2. Calculate displacement ductility factor as Δ_{max}/Δ_y
3. Calculate ρ_s / ρ_{scode}
4. Calculate ρ_s

Where Δ_{max}/Δ_y = ratio represents maximum horizontal displacement to yield displacement, ρ_s / ρ_{scode} ratio represents the volume of transverse reinforcement to volume of concrete core for the proposal and the New Zealand code. As part of their analysis of the displacement ductility capacity, the authors also suggested that the ductility could be calculated as follows:

$$\mu = 1 + 3(1 + 5.4\alpha)\frac{L_p}{L}(2 - \frac{L_p}{L})$$

Where $\alpha = \rho_s / \rho_{scode}$

The authors concluded that the suggested design equations for flexural strength and ductility are simple, easy to use and would result in substantial economies in flexural design.

Ang, Priestley, Paulay (1989) researched the shear strength of circular reinforced concrete columns by testing 25 columns under axial loads and cyclic reversals of lateral loads. They concluded that at low flexural ductilities, the superposition of concrete contribution plus a 45-degree truss mechanism described shear behavior well. However, existing U.S and New Zealand design equations for the concrete contribution were found to be very conservative and inconsistent in their shear design procedure. Based on the above, the authors suggested revised design equations.

A total of 25, 400 mm diameter columns, were tested with various axial loads, longitudinal and transverse reinforcement, and aspect ratios. The loading pattern used was similar to that of Prietley and Park (1987) with μ= 1.5,2,4,6,8 unless premature failure occurred. As a result of the tests, the columns were categorized according to their hysterectic behavior as either ductile flexural (μ>6), moderately ductile with shear failure (μ = 4-6), limited ductile with shear failure (μ = 2-4) and brittle shear failure (μ <2). The strength of concrete as a shear resisting mechanism was evaluated both at the onset of cracking and at maximum shear load. The results of tests were compared with computed values obtained from design equations of both the New Zealand code and ACI code. Flaws were identified in both codes in so much as the codes were found to be conservative. A similar evaluation was done to establish the influence of shear reinforcement on the ultimate shear strength of columns. And finally, the degradation of shear strength with ductility was examined.

As a result of the experience gained from the laboratory tests and the identification of shortcomings of the ACI and New Zealand codes, the authors made some design recommendations. Tentative proposals for the degraded shear strength, in the form of shear strength versus flexural ductility relationship, were developed but required further testing. However, specific recommendations were made for initial shear strength and final shear strength as follows:

Initial shear strength can be calculated using additive principle with 45° truss Mechanism $V_i = V_{ci} + V_{si}$

$$V_i = 0.37\alpha(1 + 3P / f'_c A)\sqrt{f'_c} \cdot A_e + \frac{\pi}{2} A_{sh} f_{yh} \frac{D'}{s}$$

Final shear strength reflects a reduction in concrete shear-resisting mechanism with increasing ductility and an increase in the truss mechanism strength due to steeper shear plane $V_f = V_{cf} + V_{si}$

$$V_{cf} = 18.5\rho_s \sqrt{f'_c} \cdot A_e \leq 0.185\sqrt{f'_c} \cdot A_e$$

$$V_{sf} = \frac{\pi A_{sh} f_{yh} D'}{2s} \cdot \frac{\sqrt{1-\psi}}{\psi} \leq \frac{2.15\pi A_{sh} f_{sh} D'}{2s}$$

α = aspect ratio factor | A_g = gross section area

P = axial force | f'_c = cylinder strength

A_c = effective shear area | A_{sh} = area of hoop

F_{yh} = hoop yield stress ψ= mechanical reinforcement ratio

Ang et al. concluded from the study that the shear strength was dependent on the axial load, the column aspect ratio, the amount of transverse reinforcement, and the flexural displacement ductility factor. At low ductility, the additive principle described shear behavior well and that the ACI and New Zealand codes were conservative.

Mitchell, Bruneau, Williams, Anderson, Saatcioglu, and Sexsmith (1995) presented their findings on the performance of bridges in the 1994 Northridge earthquake. For the seven bridges that suffered collapse, the authors described the damage and explained the probable causes of this damage. Of the seven bridges, six suffered column failure and the seventh suffered damage but did not collapse. These bridges had either been built or designed prior to 1971 when changes to seismic codes were introduced. Essentially all the damage to bridges in this earthquake occurred in older structures that had been constructed to outdated standards, which had not been retrofitted. The collapses could have been predicted and as such prevented by utilizing the accepted retrofitting techniques such as steel jackets. This indicated that seismic retrofitting was effective.

The field analysis in this study confirms work done in previous studies that identified seismic column deficiencies as inadequate lateral reinforcement and concrete confinement resulting in shear or combined shear and flexure failures. For instance, the columns in some of the bridges had No. 4 (12.7 mm diameter) or No 5 (16 mm diameter) transverse reinforcement spaced at 12 inches (300 mm), which was far below modern code requirements for lateral reinforcement, and did not provide sufficient confinement and shear capacity.

This paper also noted that short columns, which are heavily reinforced to carry vertical or lateral loads, do not perform well if the lateral confinement and shear capacity are not adequate to resist shear forces that develop when plastic hinges form in the column. This was found to be especially important if the rotation was high enough to reduce the shear capacity of concrete.

Williams and Sexsmith (1996) discussed a requirement to have a simple and objective method of evaluating structures in need of seismic retrofit, especially when the candidates were numerous. This method should address the assessment technique that allows the designer to make decisions between competing structures and competing retrofit schemes. The process can be enhanced by the use of inelastic analysis programs and seismic damage indices. The authors state that these tools are not widely used because engineers do have complete confidence in them. However, the research described in this paper validates that inelastic analysis procedures, together with damage indices, can provide accurate and robust predictions of structural capacity.

Most of the analysis methods are based on the traditional cost/benefit approach and are done on a case-by-case basis. An evaluation is done for each bridge and a ranking is assigned according to the cost/benefit results. Funds are then allocated to the structure according to their ranking. However, this procedure does have some difficulties in that:

1. It is necessary to define and quantify structural damage by using a damage index, which is a non-dimensional parameter calculated from the measured or predicted hysteresis response of the structure.
2. Structural damage probabilities must be estimated
3. Structural damage must be related to the consequence costs

Williams and Sexsmith contend that inelastic analysis procedures are a key component in these procedures, but only if they are reliable and easy to use. Similarly, damage indices must provide consistent indications of damage levels for a variety of structures. In this study, they assessed the extent to which these requirements are met with existing tools.

The inelastic damage analysis was done on a program called IDARC, which is based on a tri-linear moment-curvature model. The accuracy of the program was evaluated by comparing its results to that of a series of laboratory tests conducted on models of twin-column support bents of two bridges in Vancouver. The results of the program correlated quite well with the lab tests indicating that the non-linear analysis was accurate as long as care is taken in selecting some of the key program parameters such as the deformation-related strength loss parameter and the energy-related strength loss parameter. Damage indexes, which aim to give consistent numerical indications of the level of damage, were also evaluated against the lab results. The modified Park and Ang index was calibrated against test results and then compared to other forms of the index (for example, Stone and Taylor index). The Park and Ang index was found to give good correlation with observed damage when the structure was appropriately calibrated. The authors suggest index thresholds for the assessment of non-ductile structures, as: i) D>0.1 for repairable, ii) D>0.4 for non-repairable, and iii) D>1.0 for collapse. The authors concluded that a risk analysis approach is useful in the allocation of retrofit funds and that the key to the effectiveness of this approach is the use of clearly defined damage states, reliable nonlinear analysis techniques and well calibrated damage indices. They found that the program IDARC was easy to use and accurate in predicting the structural non-linear performance of bridge structures except when the structure is very poorly detailed. Finally, they concluded that the Park and Ang damage index demonstrated good correlation with observed results.

Jaradat, Mclean and Marsh (1998) conducted a two part study on the performance of bridge columns under seismic loads. The first study considered the flexural and shear performance of older columns. Experimental results and observed behavior of 8 specimens constructed to pre-1971 standards were presented. The second part of the study took the column test results and compared them to various theoretical models of shear and flexural behavior.

The primary purpose of the first study was to explore the performance of older columns, particularly with regard to the residual strength in degraded hinge sections. The variables of the study were shear span-to-depth ratio, longitudinal steel ratio, lap splice length, and retrofitting detail. Two of the columns were retrofitted according to CALTRANS specifications for steel jacketing. The lateral loading sequence was similar to that used by Priestley and Park, which was described in an earlier part of this document. The displacement ductility ratios applied were ±1 ±2 ±3 ±4 ±5 ±6. The results of tests were tabulated and comparisons were made on the basis of; i) longitudinal steel ratio, ii) M/VD ratio, iii) lap splice length, and iv) retrofit scheme. The results of Part 1 of the study can be summarized as follows:

- Flexural strength degraded very quickly if both poor confinement and lap splices were present in the plastic hinge region. However, if the plastic region had no splices but poor confinement, moderate ductility was experienced.

- The hinge region at the base of the column that contained lap splices continued to carry shear and axial loads despite loss of flexural strength. This confirmed the retrofit design technique.
- Caltrans steel jacket retrofit technique at the base of the column improved performance in the splice region, but increased the strength of the column such that shear failure occurred in the top region.

The second part of this study focuses on comparing the experimental results of the first part of the study with various analytical models of column shear and flexural behavior. The predictive models that were used for comparison were; i) the ACI/AASHTO model, ii) the University of California at San Diego (UCSD) proposed model, iii) the University of California at Berkeley (UCB) proposed model, iv) Caltrans model and iv) the Priestley and Seible flexural strength model. Based on the results from the first part of this study, as well as the results of other studies, the authors highlighted the comparison of lab/design shear strength. From the analysis it was evident that the ACI/AASHTO model was the most conservative, which was to be expected as this was used for design purposes. It was also clear that the UCSD model provided the best fit to the test data. In further analysis, which included displacement ductility, shear span-to-depth ratio, axial load ratio, and transverse steel ratio, the ACI/AASHTO model was not very good in predicting the effects of these factors on shear strength. The best performance, with all of these factors considered, was with the UCSD model. When flexural strength was considered in more detail, the Priestley and Seible model closely predicted the peak moment strength of the column in the plastic-hinge area with poor confinement and no lap splice. This model was also good in predicting the moment capacity, the onset point and rate of degradation, and the eventual residual moment strength.

Kowalsky and Priestley (2000) proposed an improved analytical model for shear strength of circular reinforced columns in seismic regions. Revisions were suggested for the three-component equation of the University of California of San Diego [UCSD] model. The revisions suggested accounted for; i) the effect of concrete compression zone or the neutral axis depth, ii) aspect ratio, iii) displacement ductility and iv) longitudinal steel ratio. The study described other design equations, such as ATC-32, discussed the original UCSD equation, proposed changes to the UCSD model, and validated the changes by comparison to information in a 47-column database.

Most shear design equations included only two components, V_c, the resisting shear of concrete, and, V_s the resisting shear of steel. The UCSD model has a third component, V_p, which recognized the shear strength enhancement provided by the axial load. The original UCSD shear model components were; i) the concrete component, which degraded with increasing ductility due to widening of cracks, ii) the axial load component, and iii) the truss component. The design equation was expressed as

$$V_a = V_s + V_p + V_c$$

where:

$$V_c = 0.8 A_g \gamma \sqrt{f'_c}$$

$$V_p = P \frac{(D - c)}{2L}$$

$$V_s = \left(\frac{\pi A_{sp}}{2} f_y \frac{D'}{s}\right) \cot(\theta) .$$

Through this research, Kowalsky et al. proposed that the truss mechanism equations needed to be changed because the assumption that the diagonal crack in a column was able to mobilize transverse reinforcement along the full length was incorrect. The authors argued that the mobilization did not occur in the compression zone and that it was more appropriate to consider a reduced length that could be computed as: (D-c-cov). The effective spiral area was a function of the neutral axis depth and the shear would be accurately calculated by an integral function. A good approximation for design purposes was

$$V_{sga} = \frac{\pi}{2} A_{sp} f_y \frac{D - c - \mathrm{cov}}{s} \cot(\theta)$$

In looking at the concrete mechanism for column shear, it is logical to assume that the shear strength be greater for columns with smaller aspect ratios as confinement effect of the adjacent members is greater in these situations. It is also reasonable to assume that a smaller longitudinal steel ratio will result in a decrease in the strength of the concrete shear resisting mechanism. The paper notes that these two factors were not considered in the original UCSD model and proposes the following modifications:

$$V_c = \alpha \cdot \beta \cdot \gamma \sqrt{f'_c}(0.8 A_g)$$

where α accounts for the column aspect, β is a modifier for the longitudinal steel ratio, and γ factors the loss of strength due to ductility.

As a final step, the researchers compared all the design equations, including the proposed modified UCSD model, to data available on the strength of 47 columns. The statistical test results for the shear strength evaluation displaying the average strength ratio ($V_{exp.}/V_{des}$) and its standard deviation for brittle and ductile failures are shown in Table 2.1. Although the average value of the strength ratio of the modified UCSD shear model and the original model are very similar, the standard deviation of the revised model is much smaller, hence the revised model displays much better correlation between the design and actual value. It is therefore a better strength predictor than the original model.

Table2.1 Comparison of design equations

	Original UCSD-D	Revised UCSD-D	Memo 20-4	ATC-32	ATC-40	Original UCSD-A	Revised UCSD-A
Brittle Shear	1.18(ave) ±0.20(sd)	1.19 ±0.10	1.54 ±0.70	2.68 ±2.10	2.61 ±1.23	1.00 ±0.17	1.01 ±0.08
Ductile Shear	1.21 ±0.14	1.27 ±0.06	1.74 ±0.67	2.00 ±0.40	2.59 ±0.82	1.03 ±0.12	1.08 ±0.05

2-3 LITERATURE ON STEEL JACKETING TECHNIQUE

Chai, Priestley and Seible (1991) were the early researchers in developing steel jacketing techniques for the seismic retrofit of deficient reinforced concrete columns. This was a significant project since it became the basis for Caltrans column retrofit program. They demonstrated that retrofitted columns exhibited ductility that was as high as those designed to modern codes and that the jackets also inhibited bond failures in lap splices of longitudinal reinforcement in the plastic region.

Structural inadequacies of older bridge columns were identified as follows:

- Flexural strength was inadequate. Typically, the lateral force coefficient used was only 10% of the gravity weight.
- Because of insufficient transverse reinforcement, the columns had inadequate flexural ductility.

- The lap length at the base of the column was insufficient to develop yield strength in longitudinal bars.
- Inadequate shear strength.

This research took the idea that closely spaced lateral confinement was effective in retrofitting columns, so that steel jackets, which are easier to construct and more aesthetically pleasing should also be effective. Six large columns were constructed and tested and loaded both axially and laterally. The lateral loads simulated seismic action and were applied in steps, up to displacement ductility levels of $\mu = \pm 6$. The axial load was equal to 17.7% $A_g f_c'$. A reference column was used as the baseline for comparison of the retrofit results. It was shown that steel jackets were quite effective as a retrofit scheme. Hysteresis loops were stable at displacement ductility levels of $\mu = 7$ corresponding to a drift ratio of 5.3 %. Failure occurred as a result of low-cycle fatigue of the longitudinal reinforcement. No bond failures occurred. Steel jacketing was also successful in repairing a column that exhibited bond failure of the spiced reinforcement in the plastic hinge area. After the application of the jacket, strength and ductility was maintained to drift ratios greater than 4%. Since this project focused on the flexural strength of columns, the authors identified a need to investigate the effect of steel jackets on shear strength, which was the topic for subsequent research.

Prietley, Seible, Xiao, and Verma (1994), in a two-part study, discussed the effect of steel jacketing on the shear strength of reinforced concrete bridge columns. Part 1 dealt with theoretical considerations in assessing shear strength, comparisons to design approaches and design models for steel retrofitting. Part 2 reported on laboratory tests of retrofitted circular and rectangular columns and comparisons with design theory. The investigation was conducted to study the shear failure mode of columns designed prior to 1971 and to test the effectiveness of full steel jackets. The research conducted in these papers formed the basis for the retrofit assessment and design of squat shear-dominated bridge columns in California.

In the first part of the study, Priestley et al. evaluated the then current design approaches, specifically, the ACI 318-89 method, the Ang, Priestley, and Paulay method, and the modified version of the latter. As described earlier in this document, the modified Ang, Priestley, and Paulay method which considers not only the concrete shear contribution, and the shear carried by the transverse steel, but also the shear capacity provided by axial load through arching action. This method has proven to accurately predict the shear behavior of columns. Steel jacketing retrofitting techniques for both circular and rectangular columns were then evaluated. Design recommendations to determine the shear contribution of steel jackets are as follows:

For circular columns, the shear enhancement is:

$$V_{sj} = .0865\pi t_j f_{yj}(D_j - t_j)$$

For rectangular columns;

in strong direction: $V_{sj} = 3.46 f_{yj} t_j D_j - t_j[1-(1-\frac{\pi}{4})B_j / D_j]$

in weak direction: $V_{sj} = 3.46 f_{yj} t_j (B_j - t_j)[1-(1-\frac{\pi}{4})D_j / B_j]$

Where t_j = jacket thickness, f_{yj} = yield strength of jacket, D_j = diameter of jacket, B_j = short principal diameter of elliptical jacket.

In this first part of the study, Priestley et al. concluded that the so called refined ACI approach be withdrawn from use because it was more conservative than the traditional

ACI method and that it was excessively sensitive to axial loads. The paper recommended that the modified Ang et al. approach be adopted for design purposes. It further concluded that the design approach presented for steel retrofitting accurately reflected field and laboratory conditions.

The second paper presented the results from testing 14 large-scale columns under cyclic reversals and compared the retrofitted structures to the as-built ones, which were designed to pre-1971 standards. The results were compared with the predictive models of the first paper and to a simplified bilinear force-deformation model.

From this study the authors concluded that the as-built columns were very brittle and exhibited rapid loss of strength and stiffness after their shear failures. In some cases they were unable to support their axial load after shear failure. There was an observed correlation between shear strength and ductility and the shear strength was on average more than 100% higher than that calculated by ACI equations. On the other hand, the Ang, Priestley and Paulay method was very accurate. The force displacement hysteretic response for steel jacket retrofitted columns was very stable with a capacity of $\mu_{\Delta}= 8$ and drift angles of 4% or greater. The critical patterns for the as-built columns versus the retrofitted columns changed from shear deformation to flexural deformation dominant. Steel jackets increased the stiffness of the columns by 30% for circular and 64% for rectangular samples. Also, the flexural overstrength of the retrofitted columns was on average 29% above the ideal strength. As such, the authors concluded that steel jacketing of circular and rectangular columns was very effective in enhancing shear strength and flexural ductility of shear deficient columns.

Chai (1996) investigated two aspects of seismic behavior. The first being the influence of steel jacketing on the structural characteristics of circular bridge columns, specifically the increase in lateral stiffness and the ductility capacity as a result of retrofitting, and the second being the analysis of the damage to steel-jacketed columns during the 1989 Loma Prieta earthquake. As part of his study of the influence of steel jackets on column behavior, the topics investigated were ultimate compressive strain, moment-curvature response, and column lateral stiffness. Steel jackets provided the required confinement to concrete columns and through the dilation process enhanced the ultimate compressive strain of concrete. This study used Caltrans design of jacket thickness $t_j = C_j \frac{D}{200} \geq 6.4mm$ and developed relationships between the thickness ratio t_j/D and the ultimate compressive strain of concrete. From the study, it was shown that large increase in compressive strain could be achieved with the then current design thickness. For example, a retrofitted column with a ratio of t_j/D = 0.005 would provide a 7.2 times increase in ultimate compressive strain for concrete with f'_c = 30MPa. The author, using a program to simulate the moment-curvature response, developed a relationship between curvature and moment for concrete columns confined with jackets of various thicknesses. The analysis of the moment-curvature response indicated that at lower lateral loads, the relationship was unaffected, but at higher loads, there was a large increase in curvature ductility for columns with steel jackets. For jackets of intermediate thickness, the decisive conditions were governed by the ultimate strain of concrete, whereas, at greater thickness, the ultimate condition was limited by the ultimate tensile strain of longitudinal steel. It was clear from the study that there was a large increase in curvature ductility due to steel jacketing. It was shown that the column lateral stiffness to is affected by jacket thickness and length, as well as the bond strength between the jacket and the column. Again, through a theoretical analysis, the author concluded that with typical steel jacket thicknesses used in practice, one could expect an increase in stiffness of 35 to 60% for columns with aspect ratios of $3 \leq L/D \leq 9$. Chai then verified the performance of steel-jacketed bridge columns through a series of inelastic dynamic analyses using the Loma Prieta earthquake ground motions records. A bilinear stiffness model was used and a program called COLRET computed the load deformation curve. The study confirmed that the steel jacketing design used by Caltrans provided adequate protection against damage by the ground motion recorded during this earthquake.

Aboutaha, Engelhart, Jirsa, Kreger (1999) researched seismic rehabilitation of shear critical columns by the use of rectangular steel jackets. Although this paper dealt primarily with building columns, some of the work was applicable to rectangular bridge columns, which forms the focus of this review.

Eleven large-scale columns were tested with thin rectangular jackets in order to test the effectiveness of retrofitting for inadequate shear strength. Full jackets, steel collars, and partial jackets were used in the tests. Partial jackets are more suitable to buildings, so they are not described here. The full jackets were built using ¼ inch steel plates and angle iron. The collars were comprised of angle iron bolted in the corners. In all tests, one inch of grout was used between the steel and the concrete. Lateral loads were applied to both the as-built, and retrofitted models. The as-built models were designed to pre-1971 specifications.

When tested, the columns that were not retrofitted exhibited shear failure. In one case the specimen exhibited limited inelastic deformation followed by dramatic loss of strength and stiffness at a drift ratio of 2 percent. In all cases, the columns did not develop their full flexural capacity prior to shear failure. The retrofitted columns with full steel jackets showed evidence of a very ductile flexural response. For instance, the column with a ¼ inch plate maintained its full flexural strength through drift ratios of 4 to 6 percent. Hysteresis loops were stable at displacement ductility levels of $\mu = 7$. Steel collars, however, did not prove to be as effective as the solid steel jackets. Aboutaha et al., in their analysis of the results, observed that large diagonal shear cracks developed on both the as-built and retrofitted columns at about the same lateral loads. From this observation they suggested that the steel jackets were passive and that they provided no shear resistance until significant column deformation took place. The steel jacket did not prevent the initial cracking but did prevent the widening of the cracks. The authors further suggested that the steel jacket should remain elastic to prevent any widening of the cracks and major loss of lateral strength.

Design recommendations and a simple model were developed to predict the shear strength of the jacketed column. The design expression is given below:

$$V_n = V_c + V_{st} + V_{sj}$$

Where;

V_n = nominal shear strength at section (kips).

V_c = nominal shear strength provided by the concrete (kips).

$$= 2.0\sqrt{f_c} \cdot b_w \cdot d$$

V_{st} = nominal shear strength of transverse ties (kips).

$$= A_{st} \cdot F_{yt} \cdot d / s$$

V_{sj} = nominal shear strength provided by the steel jacket (kips).

$$= A_{sj} \cdot F_{ysj} / 2 \cdot (d_{sj} / s_{sj})$$

From this study, it was concluded that large rectangular columns could be successfully retrofitted with thin, solid steel jackets. For instance, a 36-inch wide column with a shear to span ratio of 1.33 was successfully strengthened with a ¼ inch jacket.

Daudey and Filiatrault (2000) by building scale models of five columns with lap splices of an existing Montreal bridge in Canada tested the effectiveness of steel jackets as a retrofit strategy. The paper was unique in that it specifically looked at a test case in Eastern Canada and that the column cross-sections were rectangular but complex. The steel jackets used for the tests were both of circular and elliptical cross-section which was the preferred technology in use in California and Japan. The columns were built prior to 1971, and the detailing was typical of that era. An axial load of 4-½% $A_g f'_c$ and a cyclic load of $\mu = \pm 2, 4, 6, 7, 8, 9, 10$ were applied. A concrete mix was developed to simulate the in-place conditions.

The authors described observed failure modes of piers during earthquakes, specifically, flexure failure, lack of flexural ductility, and shear failure. Seismic retrofit strategies of circular and elliptical grout filled steel jackets were also described in detail. These were the ones developed by Priestley et al. as described previously in this Chapter. Quite predictably, the column that was not retrofitted failed at low ductility levels. The failure was due to the slip between the longitudinal bars of the pier and the dowel bars of the foundation. Vertical cracking over the lap region appeared early as did spalling of the concrete cover. Both vertical and horizontal cracks were present at $\mu = 1$. All retrofitted specimen behaved in a similar manner. They maintained their flexural strength and the hysteretic loops were stable up to a ductility ratio of $\mu = 6$. Beyond this level, slipping occurred with a slow degradation of lateral strength. When looking at the plastic hinge, good correlation was observed in the lengths observed in the lab and those proposed by previous studies. From this study, Daudey and Filiatrault concluded that the retrofitting technique utilizing circular and elliptical steel jackets was very effective and that both of these shapes were equally valuable in reinforcing complex rectangular shapes. Furthermore, the industry standard of using a 50 mm gap between the bottom of the casing and the footing was adequate and finally, the use of expansive grout instead of concrete between the jacket and the column did not improve the performance of the pier.

Xiao and Wu (2003) researched the use of thin steel jackets on square and rectangular columns as a retrofit technique for reinforced concrete columns. These jackets were further stiffened in the plastic hinge region with, thick plates, or angle iron, or square pipes. The concept put forth was that the partially stiffened jacket would rely on the beam action of the stiffeners to develop the required transverse confinement to the concrete section. The test procedures followed were typical of other studies. An axial load of 30% of the gross sectional capacity was applied and a cyclical load applied to reach peak drift ratios of Δ/l = 1 to 8 percent. Five specimens were tested, one as-built, one retrofitted with a thin rectangular jacket with no stiffeners, one with a thick plate stiffener, one with angle stiffeners and the last with square pipe stiffeners.

The as-built column suffered brittle shear failure at a drift ratio of 1.5% after which it could not carry the axial load. Degradation occurred at large displacements in the column retrofitted with a thin steel jacket. The jacket provided enough shear strength to enable the development of flexural capacity and limited ductility, but failure occurred as a result of the jacket bulging at the ends of the column. All of the columns retrofitted with stiffeners in the ductile regions, performed very well in the tests. No damage was exhibited at drift ratios of 8% and the hysteretic curve was stable. Load carrying capacities were increased as a result of sufficient confinement and strain hardening of the longitudinal steel.

Xiao and Wu concluded from the study that columns designed to pre-1971 codes suffered brittle shear failure, that the retrofit method of combining a relatively thin steel jacket for shear enhancement reinforced with partial stiffeners for confinement was very effective in improving the seismic behavior of columns. They also concluded that stiffeners could be designed using their suggested method, which is one of developing equivalent confining pressures based on ACI 318-99 code.

2-4 CONCRETE JACKETING

Rodriguez and Park (1994) conducted research on square building columns to test the effectiveness of concrete jackets as a seismic enhancement technology. The aim of the research was to investigate the increase in strength, stiffness, and ductility as a result of both a damaged and undamaged column being encased with a reinforced concrete jacket.

Four 16"x16" (406 mm x 406 mm) prototypes were built that would have been typical of a 1950 design, two of which were tested, repaired, strengthened and tested again and the other two were strengthened then tested. The jackets for two of the units had 100 - mm (3.94 in) thick concrete containing eight longitudinal bars bundled in the corners and square hoops spaced at 92-mm for transverse reinforcement. The other two units had twelve longitudinal bars spaced around the jacket with additional transverse reinforcement. The axial compression load ratio $P/A_g f'_c$ was equal to 0.2 for the as built specimen and 0.1

for the jacketed specimen. The applied cyclic loads applied in the inelastic range, as expressed by displacement ductility factor, were $\mu_n=\pm1, \pm2, \pm3, \pm4, \pm5, \pm6$.

The columns that were not retrofitted failed at low ductility, as expected. The retrofitted columns that were repaired, as well as the ones that were retrofitted without repair, performed very well under seismic action as demonstrated by the stable hysteretic curves to levels of $\mu=6$.

From this study, the authors concluded that columns designed to the 1950 codes have low available ductility and that bond failure occurred between the concrete and the plain longitudinal bars. They also concluded that both layouts of concrete jackets were very effective in improving the stiffness, strength, and ductility of reinforced concrete columns and hence could be used as a seismic retrofitting technology. And finally, they noted that this technique was very labor intensive.

Priestley, Seible and Calvi (1996) in their textbook briefly describe the concrete jacketing technique for both circular and rectangular reinforced concrete bridge columns. The process involves adding a relatively thick layer of reinforced concrete around the column and doweling the longitudinal reinforcement into the footings with sufficient length to develop the reinforcement strength. In most cases, the footings must also be enhanced to increase their flexural and shear strength.

The authors state that the construction and effectiveness of these jackets is relatively easy for circular columns by using closely spaced hoops or small-pitched spirals. However, it is difficult to achieve the proper amount of confinement with rectangular columns because the longitudinal bars in the mid-region of each face is likely to buckle and only the concrete in the corners are confined.

2-5 FIBER REINFORCED POLYMER

Saadatmanesh, Ehsani, and Li (1994) conducted an analytical study of FRP retrofitting reinforced concrete bridge columns. The investigated the use of fiber composite straps, both in the form of continuous spirals and discontinuous rings. Previously developed stress-strain models for confined concrete were used in the analysis of circular and rectangular columns. These models were included in a computer program specifically develop for this study. The program was used to predict the ultimate moment and curvature at failure of the columns as the loads varied from pure compression to pure bending. Research included two different types of fiber composites, consisting of E-glass, and carbon fiber. The E-glass had a tensile strength of 1103 MPa and a modulus of elasticity of 48.2 GPa. Although there were many different types of carbon fibers, the authors chose one that was classified as an intermediate modulus fiber with a tensile strength of 2.862 GPa and a modulus of elasticity of 172 GPa. The variables of the study were concrete compressive strength, strap thickness, and strap spacing. The data was analyzed and summarized according to the variables of the study and included axial load-moment curvature diagrams, ductility factor versus axial load ratio, strap thickness and clear spacing and moment ratio versus strap thickness.

The results of the study indicated that fiber reinforcement was equally beneficial in both circular and rectangular columns and that the rate of increase in the ultimate axial load, ductility and maximum moment capacity decreased with increasing concrete compressive strength. It was further noted that as the strap thickness increased the ductility factor increased linearly, and that the rate of increase in ductility factor decreased with increasing strap spacing. Another observation was that the maximum moment capacity was less than the maximum axial load and ductility factor, but that this characteristic was desirable because the column would fail in flexure rather than brittle shear. The study also indicated that E-glass had larger elongation at failure but carbon had larger energy-absorbing capacity, resulting in higher axial load and ductility levels for fiber of equal volume.

Saadatmanesh, Ehsani and Jin (1996) in an experimental study of seismic behavior of reinforced concrete columns evaluated five columns fitted with glass straps in the hinging zone. The samples were tested under inelastic load reversals and constant axial load. The results indicated that the FRP straps were very effective in improving seismic resistance of structures. Both passive and active retrofit schemes were tested. The passive scheme involved wrapped fibers around the column in the region of lap splices. In this case, as the concrete expanded outwards, the tensile stresses developed gradually in the straps. In the active method the composite straps were somewhat oversized and the gap between the column and the straps was injected with a pressurized grout infill. This results in a tensile stress being induced in the straps. A total of five columns were tested, two were control specimen and the other three were retrofitted, one with lapped spliced bars and one with continuous longitudinal bars. The piers were designed and built to pre-1971 specifications. The composite straps were 0.8-mm thick and built-up to a total thickness of 5-mm in the potential hinge region. The stress-strain relationship of the composite strap was linear-elastic to failure. An axial load of 445 kN was applied as well as reversed lateral loading to displacement ductility levels of $\mu = \pm 7$. The results indicated that the retrofitted columns performed very well under seismic loading. This was described by load-versus-displacement curves, which were stable up to a ductility level of ± 6. Table 2.2 provides a summary of measured and calculated strengths of columns tested.

Table 2.2 Measured versus calculated strength of columns

Specimen	Calculated lateral load kN (nonretrofitted)	Measured maximum lateral load kN (retrofitted)	Increase in strength resulting from retrofitting
1	50.7	58.3	Control
2	50.7	81.4	40 percent
3	50.7	89.4	53 percent
4	50.7	71.6	Control
5	50.7	87.2	22 percent

From this experiment, it was concluded by Saadatmanesh et al. that columns built prior to 1971 failed at low ductility levels because of bond failure in the lap splice region, and that the use of continuous reinforcement improved the ductility only moderately. Further, they concluded that external wrapping of the columns with FRP in the plastic hinge region substantially improved their strength and that improvements resulting from the active confining method was not substantial enough to justify their extra costs.

Seible, Priestley, Hegemier and Innamorato (1997) developed, validated and provided implementation recommendations on the use of continuous carbon fiber prepreg tows wound in an automated fashion as a retrofit technique. The fiber was applied and tested on both circular and rectangular columns. The jackets were designed using established design approaches. The specimens were built from these designs and then tested in the laboratory. The study found that carbon fiber was as effective as steel in seismic retrofit of reinforced concrete columns.

The authors described the three different failure modes resulting from the improper detailing in pre-1971 columns. They were; i) brittle shear failure, ii) confinement failure of the flexural plastic hinge region and iii) lap splice debonding. The study specifically set out to test the effectiveness of carbon jackets in preventing these failures.

Seible et al. proposed design equations to determine the required thickness of fibers. These were based on previous studies and are summarized as follows:

For shear: $$t_j^v \sim \frac{1}{E_j \cdot D} \cdot C_v$$

Where; E_j: Modulus of elasticity of fibers.

D: Diameter of column.

C_v: Shear retrofit coefficient.

For hinge confinement: $$t_j^c \sim \frac{D}{f_{ju}\varepsilon_{ju}} \bullet C_c$$

Where; f_{ju}: Tensile strength of jacket.

ε_{ju}: Tensile strength of jacket.

C_c : Flexural-hinge retrofit coefficient.

For lap-splice clamping; $$t_j^s \sim \frac{D}{E_j} \bullet C_s$$

Where; D and E_j as above and C_s: Clamping coefficient.

Using the above equations, the required fiber thickness for three design examples were computed; i) a rectangular column in double bending, ii) a rectangular cantilever column, and iii) a circular cantilever column. The columns were retrofitted to attain an objective level of ductility, i.e., displacement ductility ratio $\mu = 6$ to 8. The columns were designed, built and retrofitted using continuous pre-impregnated carbon/epoxy tows wound by an automated system and tested with the application of lateral loads to reach the desired displacements. It was demonstrated experimentally that advanced composite jacket systems can be as effective as steel jacketing in retrofitting seismically deficient reinforced concrete bridge columns. It was also shown that the established design models accurately predicted the shear, plastic-hinge confinement, and lap-splice de-bonding behavior of both rectangular and circular modified columns with different reinforcement ratios. Further, Seible et al. concluded that the concepts and design parameters outlined in this study were ready to be implemented in field applications provided that reduction factors are used for differences in material properties and construction.

Saadatmanesh, Ehsani, EERI, and Jin (1997) focused on retrofitting rectangular bridge columns with both rectangular and oval shaped composite straps. Five rectangular columns with different reinforcement details were built and tested under cyclic loading. Two of the columns were used as control specimens, two were retrofitted with rectangular straps, and the other was retrofitted with oval straps. Both the oval and rectangular systems were very effective in improving the ductility and energy absorption of columns. As with most studies, the columns were built to pre-1971 specifications and then retrofitted with E-glass composite straps. Two strap systems were used; one was rectangular and the other oval shape. The latter was shaped with fast curing cement before the application of fibers. The two control cases were different in that one was built with lap-splice reinforcing bars, and the other built with continuous bars. The details of the specimen are SHOWN IN Table 2.3.

Table 2.3 Specimen description of FRP retrofitted columns

Specimen	Retrofit scheme	Longitudinal steel details	Longitudinal steel ratio	Confining strap configuration
1	Control (not retrofitted)	Starter bars	2.70%	None
2	Active	Starter bars	2.70%	Rectangular
3	Control (not retrofitted)	Continuous bars	5.45%	None
4	Passive	Continuous bars	5.45%	Rectangular
5	Passive	Continuous bars	5.45%	Oval

The test procedure followed was very similar to that described in the study conducted by Saadatmanesh et al. described above. The results are tabulated in Table 2.4.

Table 2.4 Strength comparisons of retrofitted columns

Specimen	Calculated lateral strength using ACI(kN) (Not retrofitted)	Measured maximum lateral load (kN) (Retrofitted)	Increase in strength resulting from retrofitting
1	102.7	96.5	Control
2	102.7	138.8	44%
3	132.5	161.5	Control
4	132.5	21.4	32%
5	132.5	226.8	40%

The authors concluded that columns designed to the pre-1971 code failed at low ductility levels due to bond failure of the lapped starter bars and that this failure was brittle. As well, rectangular columns with high ratio continuous longitudinal reinforcement failed in shear due to the lack of sufficient transverse reinforcement. FRP composites significantly improved the strength and ductility of columns as shown by their stable hysteresis curves up to levels of $\mu = \pm 6$. And finally, it was concluded that the use of both rectangular and oval shaped straps were equally effective retrofitting techniques.

Xiao and Ma (1997) conducted experimental and theoretical studies on the use of prefabricated composite jackets as a retrofitting technique for circular reinforced concrete columns with poor lap-splice details. By building three columns, one as-built and two retrofitted, the study demonstrated both theoretically and experimentally that prefabricated composite jackets were quite effective in improving the seismic performance of columns. The authors chose to test pre-fabricated composite systems because of improved quality control and speed of application associated with such systems. These shells were 3.2 mm (1/8 in) thick with a modulus of elasticity of 48,300 MPa and an ultimate strength of 552 MPa. They were bonded together with high strength adhesive. The as-built column was constructed to pre-1971 design standards and the retrofitted columns were enhanced by the addition of four and five shells of composites. After failure, the as-built column was repaired and re-tested. The tests were conducted under constant axial compression of 5% of column concentric capacity. This was indicated to be representative of California bridge columns. Lateral load was applied as incrementally increasing deformation reversals, reaching up to displacement ductility ratios of $\mu = \pm 8$.

As with other studies, the as-built specimen failed in the lap-splice region at a low level of horizontal displacement. The retrofitted columns performed in a consistently stable fashion throughout testing. The 4-layer jacket resulted in stable performance up to a ductility factor of 6. The 5-layer jacket resulted in stable response up to $\mu = 8$. The repaired column developed improved hysteric performance as compared with the as-built column, but the loops were pinched and the axial load reduced beyond μ=4.

The analytical models used were those established from previous work and included hinge length analysis, confined concrete stress-strain models and bond link design equations. The researchers proposed a modification to the hinge length calculations that had been proposed by Priestley and Seible(1991). Accordingly;

For as-built columns: $L_h = 0.08h + 0.022d_{lb}f_s$

Where; h : height of column,

d_{lb}: diameter of longitudinal bar,

f_s : steel stress in extreme critical tensile reinforcement.

For retrofitted columns: $L_h = g + 0.044\, d_{lb}f_s$

Where; g : gap at the bottom of the jacket.

The authors concluded that the analytical approach proposed accurately predicted the behavior of columns with lap-spliced longitudinal reinforcement. They proposed that the method be used as a good tool for seismic assessment and retrofit of columns. Again, as with previous research, it was shown that columns with poor lap-splice details would fail by brittle shear and that retrofitting with prefabricated composite jackets significantly improved their seismic performance.

Purba and Mufti (1999) investigated the behavior of circular concrete columns reinforced with carbon fiber reinforced polymer jackets. Although this study did not specifically address seismic retrofit of older columns, it did provide an understanding of the confinement effects of columns with polymer jackets, which was helpful in the current research project. Specimens were constructed with and without fiber confinement and tested under static compressive loads. The results indicated a significant improvement in the ultimate load carrying capacity of the columns wrapped in carbon fiber polymers.

The experimental program consisted of testing three round columns under static concentric load. One of the columns had no lateral reinforcement, and the other two were strengthened by the addition of external carbon fibers, which were applied by hand and considered to be a passive type system. Axial and compressive strains were measured and analyzed. A theoretical stress-strain model proposed by Saadatmanesh et al. (1994) was used to verify the experimental results. A summary of the experimental results is provided in Table 2.5. The theoretical load versus axial strain relationships were also established and compared with those obtained by experiments. The comparisons showed good correlations.

Table 2.5 Increase in load capacity of FRP columns

Specimen	Ultimate load (kN)	Increase in load (%)	Increase in axial strain (%)
As built	772		
Retrofitted	1537	99	190
Retrofitted	1416	83	151

The researchers concluded that the increase in ductility and strength of columns confined with carbon fiber reinforced polymers was substantial, and that there was good agreement between the theoretical and experimental results. However, it was also concluded that the previous assumption of attaining ultimate tensile strength of fiber was not correct and further research was needed in the area.

Ma, Xiao and EERI, (1999) focused their activities on both the retrofit and repair of circular bridge columns with poor lap splice details. The retrofit was done with both continuous and segmented prefabricated glass fiber reinforced polymer shells and the repair was done with epoxy injection initially, and then modified by the addition of polymer jackets. Because of economics, the authors felt that it was important to look at the feasibility of repairing columns instead of replacing them after earthquake damage. The study demonstrated that columns could be repaired effectively after damage and that both continuous and segmented prefabricated fiber jackets could be used for this purpose.

Seven tests were conducted, consisting of i) as-built column, ii) repaired previously tested as-built column with epoxy injection, iii) repaired previously repaired and tested as-built column with fiber jackets, iv) a column retrofitted with individual fiber glass shells, v) a column retrofitted with a continuous fiber glass shell, and vi) two repaired specimens which had been previously retrofitted and tested. The procedure followed was typical of earlier tests, in that, a constant axial load and a cyclical lateral load was applied. The level of axial load was 10% of the nominal capacity. Lateral load reversals continued up to a displacement ductility ratio of $\mu = \pm 8$.

The results indicated that the as-built column began to fail at a ductility factor level of 2, and that at a level of 4, the column lost about 60% of its capacity due to bond failure. When epoxy was injected into the cracks in order to repair the damaged specimen, the maximum strength was reached at $\mu = \pm 4$. In general the repaired column developed relatively stable hysteretic loops before it reached ultimate displacement. The same column was repaired again, by the addition of five fiber jackets and it demonstrated even better performance reaching an ultimate capacity at $\mu = \pm 6$. The column that was retrofitted with five layers of individual prefabricated composite cylindrical shells was able to carry 84% of the flexural strength at $\mu = \pm 8$. The column that was wrapped with a 5 layer continuous shell also performed very well reaching a flexural strength of 81% of capacity at $\mu = \pm 8$. This last jacketed column was repaired by injecting epoxy into its damaged plastic hinge and re-tested. The results indicated that the specimen partially restored its load capacity and ductility.

Ma et al. concluded from this experimental study that retrofitting lap-splice deficient columns with either continuous or segmented prefabricated composite jackets can significantly improve their seismic performance, and that both types of jackets are equally effective. They further concluded that damaged columns could be effectively repaired with a combination of both epoxy injection into the cracks and wrapping with jackets.

Ye, Yue, Zhao and Li (2002) studied the shear strength of concrete columns strengthened with carbon fiber reinforced plastic sheets. By testing seven columns under different load and physical material conditions they determined that through a simple superposition method, the effect of fiber reinforcement on the ultimate shear strength of columns could be calculated using a shear coefficient. The study was conducted under the design umbrella of the bridge codes in use in the Republic of China where for instance, the upper value of the axial force ratio is 0.34. This ratio is defined as $n = N/f_c bh$ where N is the axial load, f_c is the compressive strength of concrete, and b and h are the dimensions of the column. The lateral load was applied in a typical seismic simulation fashion. The main test variables were; the shear span/depth ratio, the axial force ratio, and the amount of carbon fiber utilized. An as-built specimen was used as reference. Since one of the primary objectives of the study was to study the shear contribution provided by carbon sheets, strain gauges were installed along the circumference to record the performance of FRP sheets. One of the significant observations was that the strain on FRP sheets was small prior to shear cracking, and increased quickly after cracking.

The shear resistance calculation method proposed in this paper suggests that the total shear capacity of the column is the summation of the capacity of reinforced concrete column and fiber sheets. The capacity of the concrete is calculated in the traditional manner and the fiber capacity is calculated as follows:

$$V_{cf} = \upsilon \bullet \lambda_{cf} f_t b h_o$$

Where;

$$\upsilon = \frac{1.639(0.403 - 1.053n + 0.176a/h)}{\sqrt{\lambda_{cf}} + 1.207},$$

$$\lambda_{cf} = \frac{2A_{cf}}{b}$$

A_{cf} is the total area of fibers, a/h is the shear span/depth ratio, and b is the column cross-sectional depth. From this study, the authors concluded that the shear strength of reinforced concrete columns can be increased with carbon fiber reinforced sheets, that the shear effect of the sheets is only effective after the concrete cracks, and that the shear effect of carbon fibers is almost the same as reinforcement hoops. Further, they concluded that the shear strength of columns could be calculated by the proposed superposition method with the shear strengthening coefficient. However, this coefficient has a maximum value, limiting strength increase in column due to fiber reinforcement.

Harmon, Gould, Ramakrishnan and Wang (2002), in the first part of a two-part study, looked at analytical models of FRP confined concrete columns subjected to axial load, cyclic shear and cyclic flexure. Although the research is aimed at design issues of new structures, the paper is relevant to the current research project in that it establishes methods that help in predicting the behavior of columns wrapped in FRP. It was indicated that the design procedures adopted by the Federal Highway Administration were based on data obtained from full-scale tests of columns and not on fundamental theories. The reason the authors pursued this research was because of the lack of theory that predicts circumferential strains in the wrap material and the stress-strain behavior of confined concrete.

The detailed method addressed by researchers was based on the crack path model, which defines the stress-strain relationship and also predicts the average strain in the confinement material. From this same model, the moment-curvature relationship for columns subjected to axial load and bending was developed. From the moment-curvature relationship, the force displacement relationship was established. To complete the detailed analysis, the average wrap strain was needed, and this was done using the diagonal crack model. From these analytical detailed models, the warp confinement force, $f_{w,}$ was determined, as indicated below.

$$f_w = \frac{V\sin\theta}{A_c\cos\theta} + \frac{P\sin\theta(\cos\theta - \mu\sin\theta)}{A_c\cos\theta(\sin\theta - \mu\cos\theta)} - \frac{\mu f_t}{(\sin\theta + \mu\cos\theta)\cos\theta}$$

Where;

V = shear in the column

P = total compressive force taken by the concrete

A_c = area of concrete cross section

μ = Friction coefficient

θ = crack angle

Harmon et al. concluded that the above noted detailed method, although accurate for many cases, was not accurate for all of them. Furthermore, this method was very cumbersome. They therefore suggested a simplified method by assuming that the tensile strength of concrete is entirely lost. The reduced equation becomes:

$$f_w = \frac{V}{A_c \cot\theta} + \frac{P}{A_c \cot\theta} \cdot \frac{\cot\theta - \mu}{1 - \mu\cot\theta}$$

The simplified model is intended for conservative design when high levels of ductility are desired and not as a predictive model.

The authors concluded that the detailed models were good in predicting the average wrap strains, curvatures, and, displacement of confined concrete columns when shear was not present. When shear was present, the accuracy declined. They also concluded that uniaxial compression models for confined concrete are somewhat accurate for calculating deflections and curvatures, but not accurate for calculating wrap strains. Lastly, they observed that increased curvature increases wrap strain.

Gould and Harmon (2002), in the second part of previous research conducted experimental studies. A total of twelve columns were tested to assess the validity of their analytical models. The specimens were built by filling fiber-reinforced polymer (FRP) tubes with concrete. The variables of the study were axial load, fiber volume ratio, and column length. Various failure modes were observed including fracture of the FRP, fracture of the steel reinforcement, shear sliding at the base and column instability. The test results were categorized in terms of moment curvature behavior, average circumferential strain versus curvature, force displacement, moment versus average circumferential strain, the variation of wrap strain with column height and axial load, and failure modes.

The findings in this study that are of particular interest are that wrap strains due to shear, flexure, and axial load are all inter-dependent and that jacket strains increase with increased curvature even when the axial load, shear, and moment are constant. Also of note is that low-cycle fatigue of axial reinforcement is a significant concern for confined concrete columns and typically limits ductility.

2-6 PRESTRESSED EXTERNAL HOOPS

Coffman, Marsh and Brown (1993) studied the use of pre-stressed steel hoops as a retrofitting scheme for circular reinforced bridge columns. Using results from their laboratory test, they demonstrated that the use of these hoops was effective in retrofitting long columns where shear is not a significant failure feature. Their approach was very similar to other studies in that they showed that columns built prior to 1971 were seismically deficient primarily due to the insufficient transverse reinforcement which did not provide enough confinement of the longitudinal lap splices in the plastic regions. The researchers then set out to reinforce the columns by using prestressed external hoops around the columns. The hoops were two semicircular pieces of steel attached together by swaged couplers and then prestressed to approximately 350 MPa. The specimens were four round columns, one of which was as built, and the other three retrofitted. The as built column was designed to pre-1971 standards. The axial load was in the order of 20% $A_g f'_c$. The cyclical lateral loading was based on the criteria established by Park and adopted in the New Zealand design code which states that "the structure should withstand at least four times that at first yield without the horizontal load-capacity being reduced by more that 20 percent."

The test results indicated that the as-built column exhibited loss of strength after the first cycle at $\mu = 4$ thus failing the above noted criteria. The retrofitted columns completed 12 to 16 cycles at $\mu = 4$ thus exceeding the benchmark level. The authors concluded that the retrofitting techniques did not change the column stiffness or increase its strength but were effective for long columns that were not shear dominant.

Yalcin (1999) in his research at the University of Ottawa conducted both an analytical and experimental study of the capacity, demand and seismic retrofit of bridge columns. This study consisted of four different stages. In the first stage, an inventory of bridges in Canada was conducted in order to determine the bridge types, designs, and column details. This information was required in order to ascertain the seismic drift demand and capacities of these structures. During the second stage, a computer program was written to conduct analysis of members in the deformation inelastic range. Form this program and experimental data, force-displacement relationships were developed for a large number of columns with different design parameters. The data was tabulated in order to determine column drift capacities. Stage three was done in order to establish the predicted demand on concrete bridge columns. An inelastic response history analyses was conducted in order to establish drift demands during earthquakes. The parameters studied included earthquake records for Eastern and Western Canada, relevant structural components and bridge support conditions. The results were tabulated in terms of fundamental periods, geographic location, support conditions, and earthquake intensities.

The final stage was the experimental stage of testing external steel hoops as a retrofitting technique.

For the experiments, seven columns were tested, two square and five round. They were designed and built to pre-1971 specifications to exhibit shear dominant behavior. The columns were retrofitted with high strength steel packaging straps or with prestressing strands. Both the circular and square columns were retrofitted with strands at 150 mm spacing and prestressed to either 5% f_{pu} or 25% $f_{pu.}$ The axial load applied was 15% of the concentric capacity and the loading program consisted of three full cycles at various drift ratios up to failure.

From the study, the author observed that a high percentage of bridges in Canada were built prior to 1970 and as such is seismically deficient because of the lack of proper reinforcement detailing. Other conclusions were that inelastic drift capacities depend on the shear span-to-depth ratio, axial load ratio, longitudinal reinforcement ratio, material strength, and concrete confinement. An important observation was the fact that concrete bridge columns in eastern Canada are not vulnerable to seismic damage, but that the reverse may be true for western Canada, specifically British Columbia. And finally, Yalcin concluded that both the prestressed hoops and the high-strength steel straps offered a lot of potential as a retrofitting technique, but the prestressing external hoops were the most effective. The straps could be effective if they are stronger and an improved anchorage system is developed. The spacing of these hoops was also found to be an important factor with the optimum being 150mm at a stress level of 25% $f_{pu.}$

Mes (1999) in an associated study describes his research which was a continuation of the work begun by Yalcin and was focused on the retrofitting of columns with different aspect ratios. The test approach was similar to other studies with a representative axial load and various reversed cyclic loads. A total of six columns were built and tested, four at a height of 2 meters and two at a height of 1.55 meters. The as-built design was that of a typical pre-1971 column. The retrofitting technique was the same as that used by Yalcin, which was, prestressing strands applied at a spacing of 150 mm and stressed to a maximum of 25 % of the ultimate strength. The shorter column was further enhanced by the application of a 25 mm fiber-reinforced concrete protective shell around the strands. This shell could help protect the strands against corrosion and improve the aesthetics of the structure.

From this study, Mes concluded that the retrofitting technique, consisting of external prestressing by high strength steel strands, which was developed by the University of Ottawa, was very effective in improving strength and ductility of bridge columns. This technique works best when the strands are spaced at 150 mm with an initial prestress of 25% of strand ultimate strength. This spacing was equally effective in controlling shear cracking and improving diagonal tension in shear-dominant columns, as it was effective in providing confinement in flexure-dominated columns. Mes further concluded that the technique was equally effective on square columns as round columns and that with the aid of raiser disks the strands were able to evenly distribute the lateral pressure on the square column. And finally, he observed that the performance of the additional concrete external jacket was excellent.

Beausejour (2000) in this third study of the University of Ottawa technique examined the effectiveness of the method on columns that have lap splices of the longitudinal reinforcement in the potential plastic hinge region. The study approach was similar to the others. A total of six columns were built and tested under a constant axial load and simulated seismic loading. Three columns were retrofitted and compared with the companion three non-retrofitted columns. The test data indicated that the method provided sufficient confinement to increase ductility to seismic requirements and that this confinement needed to be increased in the splicing region to avoid early slippage of the spliced bars. Once retrofitted, the columns developed significant ductility.

As part of the study, the author made some design recommendations to determine the amount of prestressing required. The equation suggested to determine the parameters is:

$$f_t = \frac{2A_{ps}f_{ps}}{Ds_{ps}} = 0.7MPa$$

where D = Column diameter or outside dimension of the column.

A_{ps}= Nominal area of the prestressing strand.

f_{ps}= Stress level applied on prestressing strands.

s_{ps}= Spacing between prestressing strands.

The suggested maximum spacing of the strands is the lesser of D/4 or 150 mm. It is also suggested that the confinements be increased in the lap-splice region so that the lateral pressure is 2.0 MPa.

From the study, Beausejour concluded columns with poor lap-splice details retrofitted with external prestressing strands will withstand seismic loads as long as the strands are stressed to 50% f_{pu} and spaced at 100 mm in the weak splice area.

2-7 SUMMARY OF LITERATURE REVIEW

The majority of the research done to date on the topic of seismic weaknesses of reinforced concrete bridge columns and the correction of these weaknesses by some form of retrofitting technique has been focused on demonstrating the structural flaws of older columns and subsequently proving that a particular solution is effective. However, very little research has been done comparing these technologies.

Summary:

- Prior to 1971, seismic design and detailing provisions were not required for improved ductility and energy absorption capacity of concrete columns. Specifically, the concept of confinement of the potential plastic hinge regions of bridge columns was not implemented.
- Four deficiencies exist in these pre-1971 reinforced bridge columns
    - Inadequate flexural strength as a result of the low lateral force levels used in design to characterize the effects of seismic loads. Typically, values of 6% to 10% of gravity weights were utilized. This level of seismic forces is substantially lower than those used in current seismic design practice.
    - Inadequate flexural ductility as a result of insufficient transverse reinforcement. Typically, columns were built with No.4 (12.7mm) bars spaced at 12 inches (305mm) regardless of column size. As well, this reinforcement was only lap-spliced which meant that the bars would unravel at low ductility levels of 2 to 3 whereas levels of 6 to 8 are required during an earthquake.
    - Undependable flexural capacity because of the lap splices at the base of the column. In many structures, the longitudinal reinforcement was extended from the footing by a distance of only 20 times the bar diameter. This length

is too short for full development of the yield strength of steel, resulting in quick degradation of flexural strength.

- Inadequate shear strength where actual flexural strength exceeded actual shear strength, leading to premature shear failure. This is especially a problem in shear-dominant shorter columns where failure can be brittle.

- World wide, there are a large number of bridges that require seismic retrofitting and the cost to complete this work is very high.
- The solution to the issue of seismically weak reinforced concrete columns is to confine them on the exterior with a strong material. There are many different retrofitting schemes that are technically viable:
  - Solid steel preformed half shell circular jackets, welded on site with the gap between the column and the steel filled with grout.
  - Solid steel elliptical jackets similar to the above but used for square or rectangular columns with the void being filled with either concrete or grout.
  - Rectangular steel jackets made up of plates and angle iron, field welded or corner bolted.
  - Solid steel thin rectangular jackets reinforced with steel stiffeners of various cross sections, including thick plates, angle iron or square pipes.
  - Layers of fine steel wire mesh closely bound together and encased in mortar cement.
  - Pre-stressed external steel hoops or bands which can be either two semi-circular hoops tightened with swaged couplers or a single hoop tightened with Dywidag twisted ring anchors

- Reinforced concrete jackets constructed on site and reinforced with both longitudinal and transverse bars.
- Various combinations of externally bonded fiber reinforced polymer systems. The two main material components of these systems are the resins and the fibers. The resins are epoxies, vinyl esters, or polymers. The fibers used are mainly carbon, continuous glass, or aramid. Over and above the different materials that can be used, there are various methods of application. Mainly:
  - Wet lay-up systems, which consist of dry unidirectional or multidirectional fiber sheets or fabrics impregnated with resin on-site. This system can be applied either mechanically or by hand.
  - Prepreg systems, which consist of uncured unidirectional or multidirectional fiber sheets or fabrics that are pre-impregnated with resin at the manufacturer's premises. Again these can be applied either mechanically or by hand.
  - Pre-cured systems, which comprise a wide variety of composite shapes. For seismic retrofitting, they usually consist of shells which are cut, placed around the column and glued in place.

CHAPTER 3

COMPERATIVE INVESTIGATION

3-1 GENERAL

The main objective of current research is to compare existing seismic retrofit strategies for reinforced concrete columns and provide structural and economic assessment of their appropriateness for a given application. This Chapter provides a brief presentation of all techniques considered, followed by a subjective evaluation to eliminate least feasible techniques as viable alternatives for future applications. Actual retrofit design of selected bridge columns is presented with cost analysis for each case. Finally, comparisons are made between the viable retrofit approaches selected. The process that has lead to practical conclusions is illustrated in Figure 3.1 in the form of a flow chart.

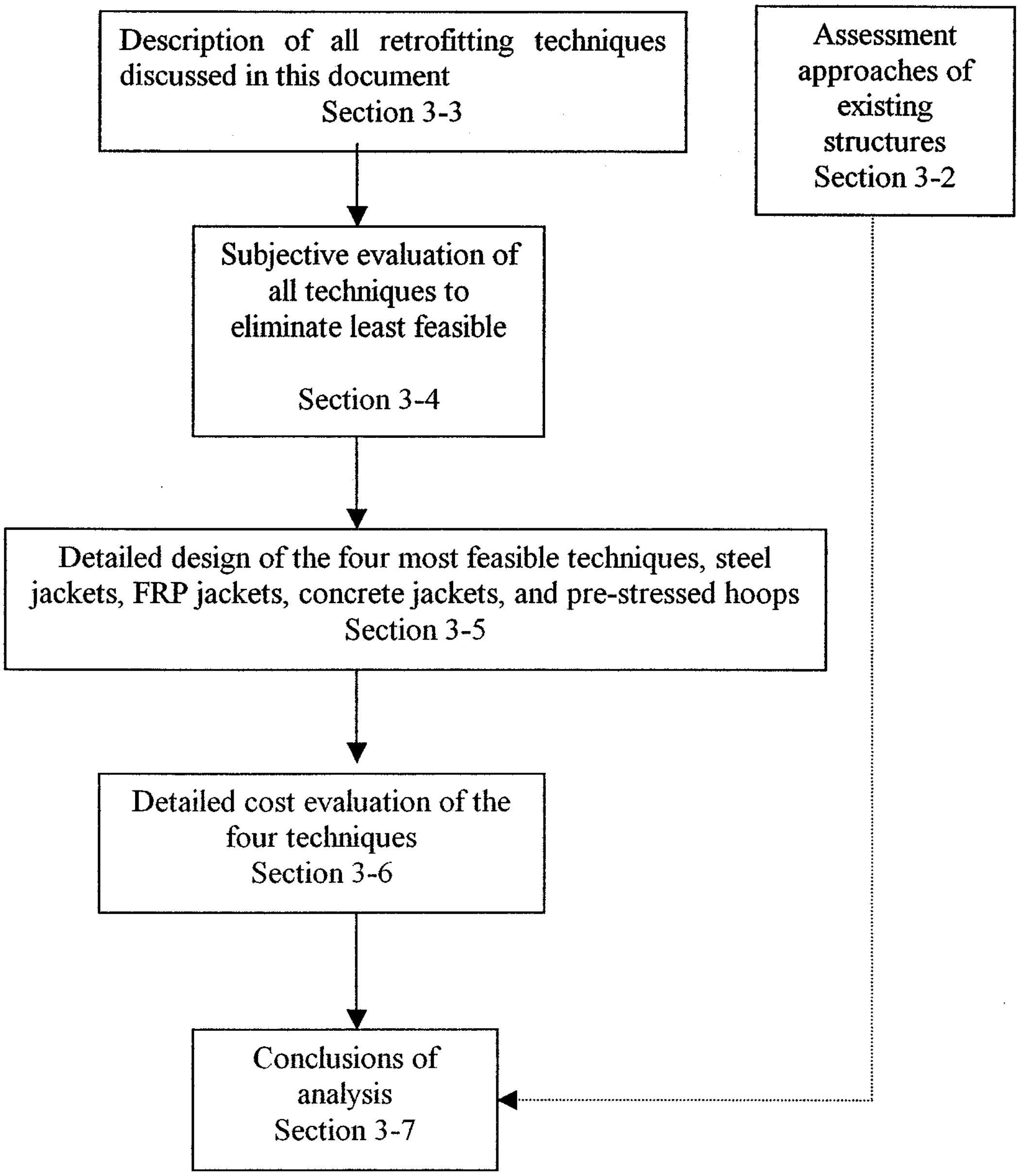


Figure 3.1 Flow chart illustrating the comparative research process

3-2 SEISMIC ASSESSMENT OF EXISTING STRUCTURES

3-2-1 GENERAL

The prime purpose of this research to provide a comparative analysis of comparing retrofitting techniques, however, an important first step in choosing a technique and developing a retrofitting strategy, is to understand the process of risk assessment of existing structures. The assessment tactic establishes a framework for retrofitting decisions and hence is presented at this time, by first describing the pertinent data, discussing some of the current schemes used, and recommending an approach on the basis of the information gathered.

3-2-2 Pertinent Data and Approach

The goal of seismic retrofit, and to a certain extent, seismic design of new structures, is not to make the structure "earthquake proof" but to minimize the likelihood of a collapse. When dealing with new construction, the incremental cost of building a structure that will likely withstand a strong earthquake is relatively low in comparison to upgrading an existing structure to withstand the same earthquake. Due to the high costs involved in retrofitting, the uncertainties of both the magnitude and location of the next earthquake, as well as the uncertainties of the losses and damages to existing structures, a detailed assessment of an upgrading plan becomes a challenge.

The assessment plans in use today are based on two levels of evaluation. The first level involves an overall screening and prioritization to determine the degree of seismic risk of each bridge and is based on general indicators such as type of structure, soil conditions, strategic importance in the road network, and the amount of traffic carried. The second level of assessment considers a detailed structural evaluation of the bridge.

The prioritization schemes include many different factors, and these factors can be classed into three major categories; i) seismic risk, ii) structural risk, and iii) importance of the structure. The seismic risk is the site-specific probability of an earthquake and its expected ground motion. This information is usually available from appropriate building codes or government agencies. These are described as the acceleration ratio, soil type, and liquefaction potential. The structural risk or vulnerability is related to the structural characteristics of the bridge. Some of the relevant factors in this analysis include; whether the structure is continuous or single span; whether the span is continuous over internal supports; whether bents are single columns or multicolumn; whether the abutments are right angle or skewed. When specifically concerned with columns, the age of the bridge is important in order to determine if the column has insufficient transverse reinforcement, lap slices in the potential plastic hinge region or insufficient confinement. Some jurisdictions include analysis of the measure of damage likely to occur on columns by describing a "damage index" for the pier(Sexsmith 1994).

The final factor in the prioritization analysis is the importance factor, which relates to the consequence of failure or collapse. The usual considerations for this portion of the analysis are; i) traffic volume, ii) total length, iii) skew angle, iv) length of detour if the bridge were to be closed, and v) whether or not the bridge is on a route that is classed as lifeline or emergency route. Lifeline routes are the ones that are of highest importance and need to be kept open to all traffic immediately after a "design earthquake" (i.e. 10% probability of exceedance in 50 years) has occurred. Lifeline bridges must also be useable by emergency vehicles and for security/defense purposes immediately after a strong earthquake e.g. a 1000-year return period event. Emergency-route bridges are those that should be kept open to emergency vehicles immediately after a design earthquake.

The second level of prioritization assessment is the detailed analysis of the structure. There are various approaches that can be taken in this analysis, some of which are; i) capacity/demand ratio, ii) plastic collapse mechanism (pushover), and iii) inelastic time-history analysis.

The capacity/demand ratio analysis was developed in the 1980's and was used for some time afterwards. In its simplest form, this ratio is the comparison of the elastic demand of a bridge structure to its strength (capacity). The implication is that a ratio greater than 1 implies failure, but in its more complete form, the demand/capacity ratio can exceed 1 if ductile response is assured. As research in seismic behavior evolved in the 90's and 2000's this methodology has been re-examined to reveal some basic flaws and is less popular today (Priestley, 1996).

The plastic collapse mechanism analysis, which is today's preferred analysis tool, is done on independent stand-alone frames separated from the other frames at the joints. The first step of the analysis is to perform collapse analyses of individual bents and tracking its characteristics such as displacement and plastic rotation. This individual bent information is then assembled into the frame information and compared to the rigidity and stiffness characteristics of the frame. From this, critical bents and mode of failure can be identified. A manual analysis of the plastic collapse of structures has its limitations, but special purpose computer programs have been developed to address these shortcomings.

Inelastic time-history analysis is the most sophisticated method available for checking the performance of a bridge. A detailed explanation of this approach is beyond the scope of this research, but in simplistic terms, this tool is one that utilizes as input a particular earthquake record and from this record evaluates the bridge response. There are libraries of individual member force-deformation relationships that have been developed to represent bar, beam or column behavior. A large amount of data and computations are required to perform a detailed analysis of this type.

During the second level of prioritization, the characteristics of the member strength and deformation should also be determined. For the case of columns, the flexural strength of the sections with lap-splices would be determined as well as the deformation capacity, and the shear strength.

3-2-3 Assessment Schemes Presently in Use

Various government agencies and research organizations have developed prioritization approaches. Caltrans evaluation process includes 12 risk components each with a different weighting factor.(Priestley 1996) They are:

Year of construction (13%)	Height (7%)
Acceleration (11%)	Skew (7%)
Soil (12%)	Facilities crossed (6%)
Hinges (11%)	Route type (5%)
Single-column (10%)	Detour (5%)
Traffic volume (8%)	Abutment (4%)

The Ministry of Transportation and Highways of British Columbia (BC Ministry of Transportation 1997) has an extensive evaluation process that considers the classification of bridges and the level of retrofitting. The bridges are classified as lifeline structures, emergency route bridges, or other bridges. The criteria for a structure to be classed as lifeline has three components; i) summer average daily traffic, ii) length, and iii) detour length if the bridge were to collapse. There are three levels of retrofitting; i) superstructure retrofitting, ii) safety retrofitting, and iii) functional retrofitting. The first level simply involves the use of restrainer cables and shear keys; the second level prevents the collapse during the design earthquakes and the third level is a general one where retrofitting the bridge will allow it to remain open to all traffic. Their prioritization criterion has three components: seismicity, importance, and structural vulnerability. A pre-weight score is assigned to each criterion; each bridge is evaluated according to these measures and then order ranked.

Baker and Miller(1999) studied the economics of bridge seismic retrofitting for the city of Seattle road network. They looked at the cost-benefit of these improvements by comparing the cost of the upgrades to the benefits of uninterrupted travel and avoidance of the costs associated with additional delays, operating costs and eliminated trips. The traffic forecasting models for the city as well as the retrofit improvement and bridge replacement costs was used in a traditional economic study model. The study demonstrated that for high priority lifeline bridges, the costs of retrofitting were economically feasible in that the net present value and rate of return were positive.

3-2-4 Recommended Approach

There are a number of decisions that need to be made at the start of a seismic retrofitting program. The first is whether or not one should retrofit, and the second is how much of the structure should be retrofitted. In principle, the first decision is made by: establishing a prioritization scheme that allows the designers and portfolio managers to rank those structures that are in the greatest need of upgrade and then allocating the affordable resources to this need. The second decision is also one of prioritization and affordability where the lowest cost modifications that provide the greatest risk reduction are completed initially. For instance, by extending the seat widths at movement joints, seismic risk can be reduced considerably so that these modifications would be completed before a complete retrofit of the columns and footings.

Ideally, the above noted decisions should be make on well established data where the prioritization clearly establishes the priorities on a bridge by bridge basis and that the decision to complete a total retrofit of a particular bridge is done on sound economic studies. However, the data available to conduct prioritization studies and cost-benefit studies is not always readily available and can vary substantially. For instance, the approach of conducting detailed traffic studies where: the traffic delay costs, accident costs, and emission costs as a result of an event that is difficult to predict (i.e. earthquake) are compared to the retrofitting costs, involves many variables that if incorrect, could skew the results. The challenge of estimating the travel related costs are as difficult as forecasting earthquakes.

Based on the research conducted as part of this study, the conclusion that can be drawn is that the seismic assessment phase should focus on a prioritization process that has as an end product a portfolio of projects that provides the relative position of each bridge in order of priority. This list of bridges that are in need of retrofit and ranked according to their need of upgrade becomes the basis for the allocation of funds.

This process can be approached in one of two ways; i) a detailed cost-benefit analysis with extensive traffic forecasting and earthquake prediction models, or ii) an exercise of route classification, and structure prioritization based on general risk parameters. The detailed cost-benefit study is not a practical approach for a couple of reasons:

1. Any detailed study is largely based on forecasts that are very uncertain. There are not a lot of sophisticated models that have been proven to be accurate. (i.e. models that are able to forecast the next earthquake)
2. The general public and policy makers are risk averse. What this means is that they are conservative when faced with a public safety decision and that the "gaming" theories used in most forecasting models are not valid in these decisions.

In order to meet the needs of balancing public safety requirements and the economic use of government funds, the following approach is suggested:

- Establish a route prioritization system that identifies the most important links in the road network. These could be classed, in decreasing order of importance, as lifeline, emergency route, and other routes. The analysis required to establish these classifications should be done by careful examination of the traffic flow models as well as socio-economic factors.
- Immediately provide the easy low costs retrofits, such as extending seat widths, to the lifeline bridges.

- Establish prioritization criteria, with weighing factors, for the superstructure-retrofitting program that incorporates seismicity (acceleration ratio, soil conditions), importance (traffic, length, height, skew) and structural vulnerability (number of columns, seat width, rocker bearing). The values used in this part of the evaluation are not absolute numbers but ranges of values. For instance, for bridges that are 0-100 meters long, the score is 0, for bridges 100-300 meters long the score is 0.50 and for bridges that are greater than 300 meters long the score is 1.00.
- Establish a priority rating for each structure and list in descending order of priority.
- Based on the available funds, establish a long-term program that incorporates the above noted priorities ensuring that the lifeline and emergency route structures are retrofitted early in the program.
- When conducting detailed evaluations of specific bridges, the plastic collapse mechanism analysis method should be utilized. This method provides the proper balance of accuracy and ease of use. The capacity/demand ratio method has been shown to be inaccurate and the inelastic time-history analysis method requires a very high level of computational effort giving doubtful results.

3-3 DESCRIPTIONS OF RETROFITTING TECHNIQUES

3-3-1 Steel Based Systems

Solid steel preformed shell circular jackets is a system where two half shells of steel plate are rolled into a radius that is usually 20 to 30 mm larger than the column. These two sections are then welded at the seams in the field. A gap of about 50mm is left between the jacket and any supporting member (footing) to prevent bearing of the steel on the supporting member. If this bearing occurred, the steel jacket would act as compression reinforcement and hence add excessive strength enhancement in the plastic hinge region. Figure 3.2 illustrates a circular steel jacket.

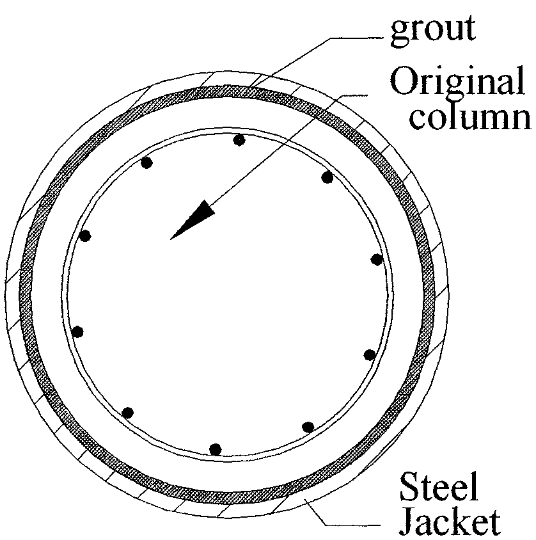


Figure 3.2 Circular column retrofit with steel jacket

The most common steel casings are manufactured from A36 steel and are anywhere from 8 mm to 25 mm in thickness. The gap between the jacket and the column is filled with grout. The grout is pumped from the bottom in order to remove any air gaps. The grout must be pumped slowly in order to avoid pushing the tube to one side or the other. Small bars are welded inside the jacket to maintain proper internal spacing. After the grout has cured, the top of the casing should be hand packed with mortar to form a slope for drainage.

Solid steel elliptical jackets are used to retrofit rectangular and round jackets for square columns. The technique is similar to that used for circular columns. However, because of the larger gaps that exist between the column and the steel jacket, more effort is required in the placement of the grout or in-fill concrete. Figure 3.3 shows a rectangular column with an elliptical steel jacket.

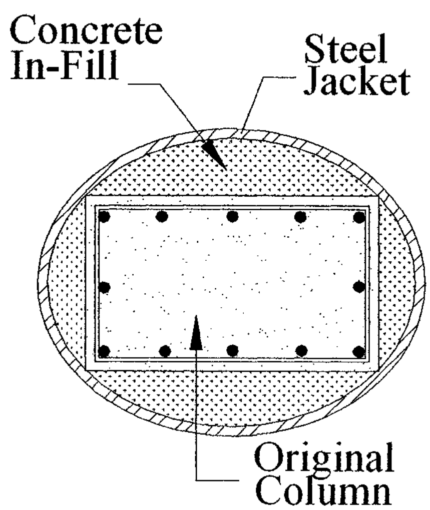


Figure 3.3 Rectangular column with an oval steel jacket

If the gap between the column and the steel jacket is more than 100 mm, concrete is used instead of grout. The elliptical shape tends to change under concrete pressure to become a circle. This may be prevented either by a bracing system or the height of the daily pours need to be restricted, usually to about 3 meters. The method of sealing the bottom of the gap also requires special detailing. In most cases, special formwork resolves this problem. Alternatively, a seal is placed at the bottom of the column to prevent the escape of the concrete or grout mix.

Rectangular steel jackets are made up of plates and angle irons that are field welded. This is a solid configuration where the plates are extended throughout the plastic hinge region. The jackets can be partially pre-fabricated into two L sections with the final assembly competed in the field. The weld size is usually ¼"(6.4mm) and extended through the full length of plates. The 1"(25mm) gap between the steel jacket and the concrete column is filled with non-shrink grout. As with other steel jackets, there is a requirement for a 50 mm space between the jacket and the footing to avoid possible bearing of the steel against footing. Figure 3.4 illustrates a rectangular solid steel jacket.

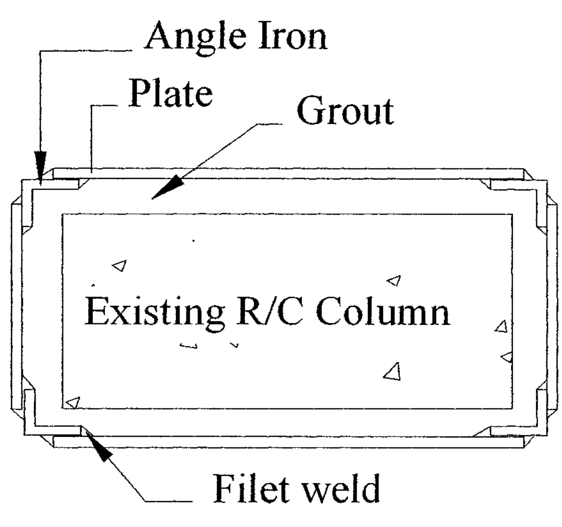


Figure 3.4 Rectangular welded solid steel jacket

Solid steel thin rectangular jackets are also used for rectangular or square columns, as illustrated in Fig. 3.5. The plates are reinforced with steel stiffeners of various cross sections, including thick plates, angle iron, or square pipes. The jackets are relatively thin, for instance 1/8" (3.15 mm), and shop welded into two L brackets and completed in the field by welding of the two sections together. The stiffeners are also welded on site. There have not been any field applications of this method, but the configurations that were tested in the laboratory were plates of 3.15 mm, reinforced with either a thick plate, or with 6 stiffener hoops, 3 at the top and 3 at the bottom. The thick plate stiffeners used were 15.9 mm, the angle stiffeners were 31.8 mm x 31.8 mm x 6.4 mm and the square stiffeners were 31.8 mm x 31.8 mm x 6.4 mm spaced at 51 mm. As with other steel jacketing techniques, the space between the jacket and the reinforced concrete column is filled with grout.

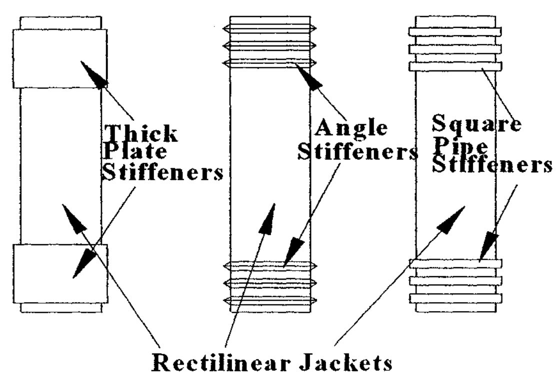


Figure 3.5 Thin steel jackets with stiffeners

Ferro cement jackets are made of cement mortar and layers of fine wire mesh, closely bound together to create a high performance material. (With regard to cracking, tensile strength, ductility, and impact resistance). As in the case of previous two applications, this technique has not been used in the field. The description herein is that of experimental columns. Square or rectangular columns are suited to this procedure as the columns can be wrapped with a flexible mesh to form a circular jacket. The jacketing is done by wrapping the column with individual layers of wire mesh, and depending on the amount of reinforcement needed, additional mesh layers are applied. Each of the layers is tied together with the same diameter wire as the mesh. An overlap of 100 mm is provided on each layer. A steel mold is then placed over the structure. The cement slurry is injected into holes in the mold to fill the void between the mold and the original concrete column. In laboratory columns, the jackets extended the full length of the column except for a 25 mm gap at the footing and upper support. The molds were removed after 7 days.

Pre-stressed external steel hoops are relatively easy to construct. They usually are made in two ways, one is made up of two semi-circular hoops that are tightened with swaged couplers, and the other is made up of a single hoop stressed with special anchors. For circular columns, the strands are applied directly to the concrete. For square or rectangular columns, raisers are placed between the strands and the column in order to improve the uniformity of lateral confinement pressure. The maximum spacing of strands is usually D/4 or 150 mm. The strands are stressed to a maximum of 25% of their ultimate strength and should be designed to provide a confining pressure of 0.7 MPa, except when inadequate splicing exists in the base, in which case the pressure should be at least 2.0 MPa. A protective layer of concrete (shotcrete/gunite) can be applied over the strands, which also improves aesthetics.

3-3-2 Concrete Based Systems

Concrete jacketing involves the addition of a thick layer of reinforced concrete around the column. This technique is more suitable for circular columns than for rectangular columns. The new longitudinal bars must be dowelled into the footing with sufficient anchorage depth so that full enhancement of flexural strength can be attained. In most cases the footing must also be retrofitted to enhance footing flexural and shear strength to ensure that plastic hinging develops in the column. The use of closely spaced hoops or a spiral of small pitch can easily achieve the enhancement of circular columns. However, unless the jacket is elliptical or circular, the retrofitting of rectangular columns is difficult. The construction of new reinforced concrete jacket is similar to that used in new construction. The longitudinal bars are installed, and then the transverse bars are placed, followed by the forms. The concrete is then poured in the forms. Figure 3.6 illustrates a reinforced concrete jacket on a circular column.

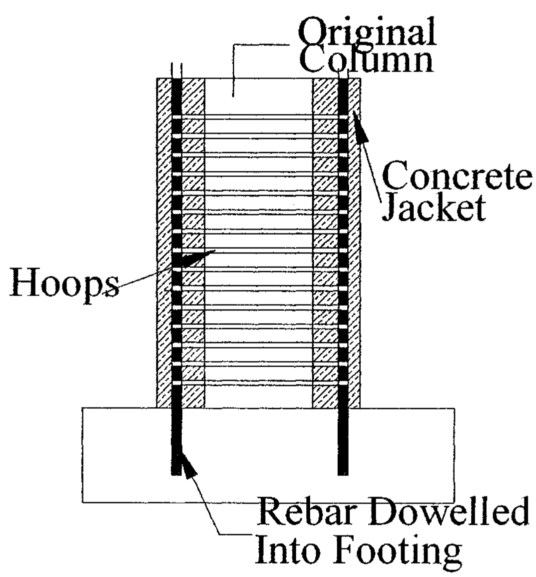


Figure 3.6 Reinforced concrete jacketing

3-3-3 Advanced Composite Materials Systems

The use of advanced composite materials for retrofitting reinforced concrete columns has been the focus of a significant amount of research in the past 10 years. Because of variations in constituent materials, as well as the different application techniques used, there are a significant number of design options. The two main materials that make up components of advanced composites are resins and fibers. The fibers provide high tensile strength and the resins bind these fibers together. The resins also bind the fibers to substrate, in order to develop appropriate mechanical properties of FRP composites.

There are a variety of resins that have been developed for use in FRP systems. The more common types are epoxies, vinyl esters and polyesters. Resins have two primary functions in column retrofit systems; i) adhesive for pre-manufactured shells to concrete and ii) saturate (sometimes referred to as matrix) that binds soft fibers together. The purpose of the adhesive is to provide a shear path between the concrete surface and the composite material. The most common type of structural adhesive used is epoxy, which is obtained by mixing epoxy resin with hardener. The popularity of epoxy may be attributed to its material characteristics, such as pot life and open time, as well as its physical characteristics in hardened form. It has advantages over other adhesives, in terms of shrinkage, creep, and strength. It can also be applied on irregular surfaces.

The saturating resin is used to impregnate reinforcing fibers, fix them in place, and provide a shear load path to effectively transfer load between fibers. When used in a wet-lay-up installation, it also provides adhesion to the substrate. The resin has a strong influence on the mechanical properties of composite, such as transverse modulus and strength, as well as shear and compression properties. The most common saturating resins are epoxy, polyester, and vinyl ester. Epoxies have better mechanical properties than the others, but are more expensive.

There are three common types of fibers used for retrofitting; i) continuous glass, ii) carbon, and iii) aramid. There are also variations of properties within each type of fiber, as indicated in Table 3.1. The fibers can be manufactured in a number of material configurations such as sheets, fabrics, and tows. A sheet is formed from fibers without interlacing, fabrics are formed with interlacing and tows are untwisted bundles of continuous filaments.

FRP systems can be assembled and constructed in a number of ways depending on the materials used and the site conditions. A convenient method of categorizing these systems is to group them as "wet lay-up systems" or "pre-cured systems." The various combinations within these categories are as follows:

Table 3.1 Properties of materials for advanced composites

Material	Modulus(GPa)	Tensile strength (MPa)	Ultimate tensile strain (%)
Carbon			
High strength	215-235	3500-4800	1.4-2.0
Ultra high strength	215-235	3500-6000	1.5-2.3
High modulus	350-500	2500-3100	0.5-0.9
Ultra high modulus	500-700	2100-2400	0.2-0.4
Glass			
E	70	1900-3000	3.0-4.5
S	85-90	3500-4100	4.5-5.5
Aramid			
Low modulus	70-80	3500-4100	4.3-5.0
High modulus	115-130	3500-4000	2.5-3.5

Wet Lay-up systems

- Dry unidirectional or multi directional sheets or fabrics where the installation requires the saturation of fabrics or sheets with resins. The fiber can be applied directly into the resin, which was pre-applied to the column, or, the fiber can be impregnated with resin in a saturator machine and then applied to the substrate.

- Resin pre-impregnated, uncured, unidirectional, or multidirectional sheets or fabric. In theses case, the fibers are soaked in resins at the manufacturer's premises and the installation may be done with or without additional resin.
- Dry fiber tows that are wound mechanically around the column. Resin is applied to the fiber during the winding process.
- Pre-impregnated fiber tows that are wound mechanically.

Prefabricated elements (or pre-cured)

- Pre-manufactured jackets, which are installed through the use of adhesives. These circular casings are either individual or continuous shells. The individual shells are split so that multi-layers of them can be fitted around the column. Continuous shells are wound around the column to the desired thickness.

The installation of FRP systems involves the repair of substrate and surface preparation, the mixing of resins, and the application of materials. In the case of reinforced concrete columns, which are being retrofitted for confinement, the surface preparation should promote continuous intimate contact between the concrete surface and the FRP system. In order to meet this requirement, large voids should be patched with a material compatible with the existing concrete. Resins should be mixed according to the manufacturer's recommended procedures. Important criteria for mixing include temperature, correct mixing ratios and complete mixing of components. The application of constituent materials is dependent on the system. For a wet lay-up system, wet fibers are hand applied onto the structure while making sure that entrapped air is removed. Subsequent layers should be placed before the complete cure of the previous layer. In machine-applied systems, the wrapping machine is place around the column and automatically wraps the tow material around the perimeter of the column while moving up and down. Pre-cured jackets are placed around a column that has been covered with an adhesive and clamped until cured. As a final step, a protective coating is applied to the jacket.

As seen from the above, there are a significant number of variations of materials and methods of application available for FRP systems. Table 3.2 provides a summary of these variables.

Table 3.2 Material and application options for advanced composites

CONSTITUENT MATERIALS	
Resins	**Fibers**
• Epoxy • Vinyl Esters • Polyesters	Carbon • High Strength • Ultra high strength • High modulus • Ultra high modulus Glass • E • S Aramid • Low modulus • High modulus

FIBER STRUCTURE
• Unidirectional sheet • Semi-unidirectional fabric • Multidirectional fabric • Tows

METHODS OF APPLICATION
Wet lay-up by hand • Dry sheet or fabric • Pre-impregnated sheets or fabric Wet lay-up by machine • Dry fiber tows • Pre-impregnated tows Prefabricated elements • Individual shells • Continuous shells

3-4 COMPARITVE EVALUATIONS OF RETROFIT TECHNIQUES

A comparative evaluation of the techniques considered is presented in this section by first describing the advantages and disadvantages of each, and then summarizing them in a tabular form. A subjective rating is then assigned to each system in an effort to identify those that can be eliminated without affecting the outcome of the study, while focusing on more feasible techniques.

3-4-1 Advantages and Disadvantages of each system

<u>Solid steel circular jackets</u>

Of all the retrofitting schemes, this is the one that is most widely used in field applications. It is a technology that the state of California has adopted and used on a large number of bridge columns throughout the state. It is also one of the few, or perhaps the only one that has been proven to be effective during an actual earthquake, since steel jacketed columns in California were subjected to earthquakes without any damage. For example, steel jacketed columns survived the 1994 Northridge earthquake. The high number of installations has also given designers and researchers valuable information concerning this system. Other advantages of the system include moderate cost of materials and pleasing aesthetics, in so much as; the final shape of the column is not changed significantly from the original shape. The durability as expressed in terms of resistance to fire hazards, diesel fuel spills, freeze-thaw cycles, and ultra-violet rays, is very good. However, unless treated and maintained properly, it is not resistant to the effects of salt corrosion. Another disadvantage is the installation difficulties due to the weight of the material, the need for large field welds, and the requirement for pressure injection of the grout. These steel jackets require the use of a hoisting mechanism and special scaffolding because of their size and weight. The welding of two semi-circular sections is critical to the structural performance, thus adding another complexity to the installation.

Solid steel elliptical jackets

Elliptical jackets are similar to circular jackets but are used for rectangular columns. Their advantages are somewhat the same as the round jackets, but because they are limited in numbers they have not been verified extensively in the field. The elliptical systems have all of the disadvantages of the circular system. The construction complexities and related costs are higher because of the requirement for special forms or pour-lift restrictions due to the amount of concrete to fill in between the jacket and the column. Bracing systems are also needed in order to maintain the elliptical shape of steel jacket during concrete pouring. The aesthetics of this application are also poor. The retrofit adds a lot of bulk to the existing column. Furthermore, the pier shape is not symmetrical when applied to the hinge area only.

Rectangular jackets of plates and angle iron

This design is applicable to rectangular columns only, and is specifically intended for shear deficient columns. Because the components are made of steel, some of the advantages and disadvantages are inherent in the characteristics of the material. Durability, such as fire resistance, freeze-thaw resistance, and diesel fuel spill resistance are all very good. However, its durability in salt corrosion environments is weak. What differentiates this design from the other steel designs is that it contemplates the use of standard shapes, which are easier to handle and produce than elliptical shapes. Because of the abundance of exposed welds, its aesthetic qualities are not overly pleasing. However, this kind of jacket maintains the rectangular shape, as opposed to completing column to an ellipse. Because of the number of components that need to be field welded, this design has a high labor component and may be prone to structural weaknesses in the seams

Thin rectangular jackets reinforced with stiffeners

This is another technique that is suited to square or rectangular columns. Being made of steel, it has all of the disadvantages and advantages determined by the material. These have been described in previous sections and need not be repeated here. However, the characteristics particular to this design are important, and will be described. Since research conducted on this technique recommends the use of jackets extending the full length of column, the handling and subsequent field welding of such large panels would be difficult and expensive. The injection of grout between the long jacket and the existing column would also be complicated. The stiffeners that are placed both at the top and bottom of the column are relatively lightweight and easily handled, but would require exacting field welds. The appearance of the retrofitted column would be pleasing due to its shape and symmetry.

Ferro cement jackets

Ferro cement jackets were experimentally tested on square columns weak in shear. This system uses low cost materials, i.e., wire mesh and mortar, but is very labor intensive. Therefore, its application is suitable to developing countries. The installation procedure of wrapping columns with 4 to 5 layers of wire mesh, tying these together, placing a steel mold around the mesh and then pressure injecting the mortar into the mold, require a significant amount of manpower and is its greatest disadvantage. Its advantage is the low material costs. The aesthetics and durability of this approach is questionable. Because a larger round jacket strengthens the square column, a considerable amount of material is added to the structure thus negatively impacting its appearance. The durability of a thick layer of mortar exposed to the elements is also questionable.

Pre-stressed external steel hoops

The greatest advantage of using external steel strands, as a retrofit technique, is its ease of installation and associated low cost. As well, this system can be applied to either round or square columns, making it very flexible. Because the amount of steel used is relatively small, the corrosion issues can be addressed by the use of galvanized or stainless materials. Placing an external concrete cover shell around the strands would add some cost to the process, but would resolve the issues of protection, durability and poor aesthetics of the hoops.

Reinforced concrete jackets

Reinforced concrete jackets, once constructed, offer good durability including better corrosion resistance than steel. The materials used are readily available, relatively inexpensive, and easy to handle. The system is better suited to circular columns but can be used on square and rectangular shapes, if the jacket is circular or elliptical, giving it application flexibility. The greatest disadvantage of the concrete jacket is that it is complicated to construct. First, the existing column must roughened and cleaned, holes must be drilled in the footing to accept the longitudinal bars, forms are then placed around the column and concrete poured into the column. As an alternative to the use of forms, shotcrete can be used in the process.

Advanced composite materials

Advanced composites can be manufactured and used in a variety of way. They may consist of different combinations of constituent materials, fiber structure and methods of applications. Therefore, their evaluation was done on the basis of constituent materials, fiber structure, and method of application.

Constituent materials: Resins are used either as an adhesive or a saturating agent. When used as an adhesive, by far the most common material used is epoxy because of its inherent advantages associated with convenient curing conditions and resulting superior mechanical properties.

As for the choice of fibers, there is little research published on the use of aramid fibers for structural retrofitting. Therefore, limited information is available on its structural properties and durability. The mechanical properties of this fiber are similar to those of glass. Therefore, aramid will not be used in subsequent comparisons of retrofit technologies. Glass is the most common of the materials used in construction industry and has a lower cost that carbon. However, carbon has higher tensile strength and modulus of elasticity. Studies done by Steckel et al. (1998) have shown some differences in the durability of the two materials. The carbon-epoxy based systems showed excellent durability when tested for exposure to 100% humidity, salt water, an alkali solution, diesel fuel, ultraviolet light, elevated temperature and cyclic freeze-thaw cycles. The glass-epoxy based systems demonstrated good durability in many of the tests but they were susceptible to strength reductions of 20% after exposure to moist environments.

The choice of constituent materials thus becomes one of using a carbon-epoxy system that is strong, durable and expensive or of using a glass-epoxy system that is not as strong, not as durable but inexpensive. The application and field conditions may in some cases dictate the proper choice of alternatives.

Fiber structure: Key mechanical properties that are of importance in the design of advanced composite column jackets include the elastic modulus, unidirectional tensile strength, and ultimate unidirectional tensile failure strain. These physical properties, as well as the installation method dictate the type of fiber structure to be used. If hand lay-up is used, a multidirectional fabric is not required so that the use of a unidirectional sheet is most appropriate. If a mechanical lay-up is used, then tows, which are unidirectional and applicable to mechanical installations, are most appropriate.

Method of application: When comparing various methods of application, differences can be encapsulated by evaluating ease of installation, costs of materials and labor, and quality control. The material costs of wet installations (as defined above) are much lower than those of prefabricated installations, but their labor costs are higher. Because they are produced in a manufacturing plant, the quality and long-term durability of prefabricated units are better than the field constructed wet installations. When comparing hand lay-up methods with machine methods, the hand lay-up method is easier but the machine method produces a higher quality product. And finally, the pre-impregnated systems are easier to install and produce a higher quality product but are more expensive than dry systems. Table 3.3 presents salient features of materials, fiber structures and methods of application used for FRP retrofitting.

Advantages and disadvantages of advanced composite materials, as applied to seismic jacketing over steel or reinforced concrete systems, are tabulated in Table 3.4.

Table 3.3 Summary of choices within FRP category

Constituent materials
• Epoxy is the best overall choice • Use Carbon or Glass: Carbon fiber is stronger and more expensive than Glass

Fiber structure
• For hand lay-up, use unidirectional sheets • For machine methods, use tows

Method of application
• Prefabricated systems are expensive but easy to apply and of high quality • Wet systems offer more shape flexibility, lower material cost but higher installation costs • Hand lay-up systems require very little space and scaffolding, but can be of lower quality • Machine systems are of high quality but require construction expertise free space • Pre-impregnated systems are easier to install and of higher quality than the dry systems

3-4-2 Summary of Comparative Evaluation

The review of seismic retrofit techniques discussed in this Chapter allowed a comparative assessment of pros and cons of all the systems considered. This summary is presented in Tables 3.5, 3.6, and 3.7.

Table 3.4 Advantages and disadvantages of FRP jackets

Advantages of FRP jacketing	**Disadvantages of FRP jacketing**
• High tensile strength • Light weight • Durability (except for: glass fiber exposed to water for extended periods, and exposure to high temperatures) • Thin film which is desirable for aesthetics • Low installation costs • Can conform to any shape	• High material costs (carbon is very expensive) • Loss of strength when exposed to water for an extended period • Cannot be applied in very cold temperatures • Design procedures still not fully developed • Cannot be used on square columns

3-4-3 Preliminary Ratings of the Techniques Considered

A preliminary rating scheme is necessary to eliminate some of the techniques that are not feasible so that the study can focus on detailed analysis of the more attractive alternatives. This was accomplished by compiling the characteristic features of each technique and assigning each a weighted importance. This was done with due considerations given to local (Canadian) conditions.

Table 3.5 Summary of the advantages and disadvantages of steel based systems

Technique	Advantages	Disadvantages
Steel based	*Readily available, low material cost, good overall durability*	*Corrosion, material weight*
- Circular jacket	Proven technology	High installation costs, large field welds, difficult to handle materials
- Oval jacket	Some existing field experience	Limited application, high installation costs, large field welds, difficult to handle materials, special forms required, bracing of jacket during construction, corrosion.
- Plates and angles	Standard steel shapes that are readily available and easy to handle.	Abundance of field welds making quality suspect, high installation costs.
- Jacket and stiffeners	Good aesthetics, stiffeners are readily available and easy to handle	Large jackets difficult to handle and field weld, cutting and welding of stiffeners difficult
- Pre-stressed external hoops	Very easy to install, low cost, versatile applications	Poor aesthetics if not covered (but this adds cost)

Table 3.6 Summary of advantages and disadvantages of cement based systems

Technique	Advantages	Disadvantages
Ferro-cement jacket	Very low material costs. Good candidate for developing countries	Labor costs very high, durability questionable, aesthetics poor
Reinforced concrete jacket	Durability, material availability and ease of handling	High labor costs

Table 3.7 Summary of advantages and disadvantages of FRP based systems

Technique	Advantages	Disadvantages
Advanced composites	*Strong, light, durable, thin, low labor costs*	*High material costs, needs moderate temperature for application, lack of design experience*
Carbon/epoxy	Very high strength	High material cost
Glass/epoxy	Moderate costs	Moderate strength (compared to carbon), loss of strength in wet areas
Machine applied	Excellent workmanship, fast installation	Higher costs
Hand applied	Lower costs, can be applied in tight quarters	Quality
Dry sheets	Ease of handling, unlimited choice of sizes	Quality
Pre-impregnated sheets	Ease of installation	Costs, somewhat limited sizes
Prefabricated shells	Very easy to install. Excellent quality control	High material costs, limited shapes

The rating was established by following the criteria outlined below:

For choice of characteristics:

- Include the components that encompass total design criteria including project life cycle principles. For instance; initial first cost-reflected in material cost, ease of installation and ease of handling (as they reflect labor cost).
- Durability, which for a Canadian bridge includes corrosion, water, fire freeze/thaw, and fuel resistance.
- Design maturity and field experience are both related to the efficiency of design and construction and hence accuracy of pricing.

For importance weighting:

- The relative position, as opposed to the absolute figure for each, reflects their importance.
- Because labor costs in Canada are high, ease of installation and ease of handling have a higher weight that material cost.
- Corrosion and water resistance, and freeze/thaw resistance are important due to the Canadian climate.
- Fire and chemical (fuel) resistance are important but because their likelihood of occurrence is low, their weight factor is low.
- Field proof, aesthetics and adaptability to field conditions are important, but not as important as ease of handling and installation

The ratings were assigned based on the information gathered as part of the current investigation. For instance:

- Material costs were rated according to the information gathered in Yardsticks (2002), Walkers (2000), Shi Zang et al. (2000), as well as private correspondence with Don Lamb (Master Builders Technology,2003), Simon Foo (PWGSC,2003), as well as the experience attained by the University of Ottawa, Structures Laboratory.
- Ease of installation and ease of handling were rated according to the installation method and type of materials. For instance, steel jackets weighing 1000 kg and requiring precise seam welding would be difficult to handle and install, whereas steel hoops weighing 2 kg and attached with a connector would be easy to install and handle. Research such as that by Takiguchi (2001) support that ferro-cement jackets are very labor intensive.
- Durability information for FRP systems was obtained from Steckel (1998) and Ming-Hung Teng (2003) and the results of these studies are reflected in the ratings. Information for other technologies was obtained from Priestley (1996).
- The basis of evaluation for aesthetics, adaptability to field conditions and field proof are somewhat self-evident. As an example, FRP sheets are thin and almost invisible when wrapped around a column hence more pleasing than a series of metal plates and stiffeners.

The comparative evaluation indices of steel and cement based systems, as well as advanced composite systems are provided in Tables 3.8 and 3.9.

From the analysis of results, the following observations can be made:

- All of the advanced composite materials techniques scored high. This is due to their good durability, ease of handling, and ease of installation.

- Prestressing hoops have a good score due to their low material costs, ease of handling, and ease of installation and excellent versatility.
- The steel systems that are assembled from components such as plates, angles or tubes did not score well primarily because of the high labor cost and the lack of field experience of this product.
- The Ferro-cement system is not very attractive due to the installation complexities.
- The circular steel jackets scored somewhat lower than the reinforced concrete jackets primarily due to the slight differences in durability.

Table 3.8 Comparative evaluation indices of steel and cement based systems

		Steel										*Mortar/concrete*			
Characteristic	Importance (wt)	Circular		Oval		Plates/angles		Stiffeners/plates		Hoops		Ferro-cement		Reinforced concrete	
		Rating	Total	Rating	Total	Rating	Total	Rating	Total	Rating	Total	Rating	Total	Rating	Total
Material cost	8	8	64	8	64	6	48	7	56	8	64	10	80	8	64
Ease of installation	10	3	30	2	20	3	30	2	20	9	90	1	10	1	10
Ease of handling	9	1	9	1	9	3	27	2	18	10	90	4	36	4	36
Durability -Corrosion resistance	10	1	10	1	10	1	10	1	10	4	40	5	50	6	60
-Water resistance	9	6	54	6	54	6	54	6	54	7	63	6	54	7	63
-Fire resistance	3	2	6	2	6	2	6	2	6	2	6	6	18	7	21
-Freeze/thaw resistance	7	7	49	7	49	7	49	7	49	9	63	5	35	7	49
-Fuel resistance	3	9	27	9	27	9	27	9	27	9	27	7	21	8	24
Design maturity	7	10	70	9	63	1	7	1	7	2	14	1	7	9	63
Field proven	8	10	80	10	80	1	8	1	8	1	8	1	8	8	64
Aesthetics	7	3	21	2	14	1	7	1	7	1	7	2	14	2	14
Adaptability to field conditions	7	2	14	2	14	3	21	3	21	7	49	4	28	4	28
Total rating		434		408		294		284		521		361		496	

Weight Factors… 10 Very Important, 1 Not Very Important
Rating..............10 Low (i.e. inexpensive, easy to install) 1 High (expensive)

Table 3.9 Comparative evaluation indices of advanced composite systems

		Advanced composite materials													
Characteristic	Importance (wt)	Glass Dry hand lay-up		Glass wet hand lay-up		Glass machine applied		Carbon dry hand lay-up		Carbon wet hand lay-up		Carbon machine applied		Prefabricated shells	
		Rating	Total	Rating	Total	Rating	Total	Rating	Total	Rating	Total	Rating	Total	Rating	Total
Material cost	8	4	32	3	24	3	24	3	24	2	16	2	16	1	8
Ease of installation	10	5	50	6	60	7	70	5	50	6	60	7	70	10	100
Ease of handling	9	6	54	8	72	7	63	6	54	8	72	7	63	9	81
Durability -Corrosion resistance	10	9	90	9	90	9	90	9	90	9	90	9	90	10	100
-Water resistance	9	6	54	6	54	6	54	8	72	8	72	8	72	9	81
-Fire resistance	3	1	3	1	3	1	3	2	6	2	6	2	6	2	6
-Freeze/thaw resistance	7	5	35	5	35	5	35	6	42	6	42	6	42	7	49
-Fuel resistance	3	5	15	5	15	5	15	6	18	6	18	6	18	7	21
Design maturity	7	4	28	4	28	4	28	4	28	4	28	4	28	3	21
Field proven	5	3	24	3	24	3	24	3	24	3	24	3	24	3	24
Aesthetics	7	7	49	7	49	7	49	7	49	7	49	7	49	6	42
Adaptability to field conditions	7	8	56	8	56	7	49	8	56	8	56	7	49	7	49
Total rating		492		512		506		513		533		527		582	

Weight Factors… 10 Very Important, 1 Not Very Important
Rating…………..10 Low (i.e. inexpensive, easy to install) 1 High (expensive)

- The results of this evaluation indicate that steel jackets, FRP systems, pre-stressing systems (hoops), and reinforced concrete jackets require detailed design and costs evaluation.

3.5 RETROFIT DESIGNS USING FOUR PRIMARY SYSTEMS

From the preliminary ranking presented in the previous section, it was concluded that four primary systems warrant detailed designs and cost study. These are; i) steel jackets, ii) FRP jackets, iii) external prestressing systems (Retrobelt), and iv) reinforced concrete jackets. A total of 12 bridge columns were selected from the survey of Canadian bridges conducted by Yalcin (1998). These columns had a wide range of cross-sectional dimensions and lengths. Each column was designed 4 times using four different techniques and priced to assess costs involved in each technique.

3.5.1 Selection of columns for further investigation

Yalcin (1998) conducted a survey of Canadian bridge columns. In the survey he reported common use of reinforced concrete bridge columns with cross-sectional dimensions varying between 1.0 m to 2.0 m, and aspect ratios (shear span-to-depth ratios) between 3 and 7. These were primarily circular columns, with some square and rectangular columns as well. From this information it was decided to evaluate the columns shown in Table 3.10.These are representatives of those built in Canada prior to 1971, and requiring retrofit. Figures 3.7 through 3.9 show design details of actual columns built in Ontario and British Columbia, within the geometric ranges considered in the current study, confirming that the selected columns are representative of those used in practice.

Table 3.10 Column geometry selected

Circular Sections (D), mm	Length (L), m	L/D	Square Sections (D), mm	Length (L), m	L/D
500	1.5	3.0	500	1.5	3.0
500	3.0	6.0	500	3.0	6.0
1000	3.0	3.0	1000	3.0	3.0
1000	6.0	6.0	1000	6.0	6.0
2000	6.0	3.0	2000	6.0	3.0
2000	12.0	6.0	2000	12.0	6.0

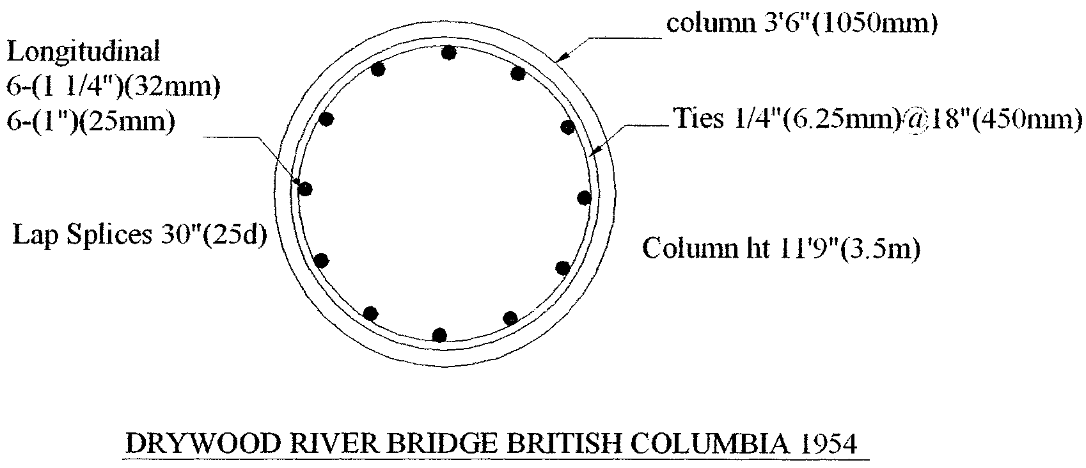


Figure 3.7 Typical column of Drywood River Bridge in British Columbia

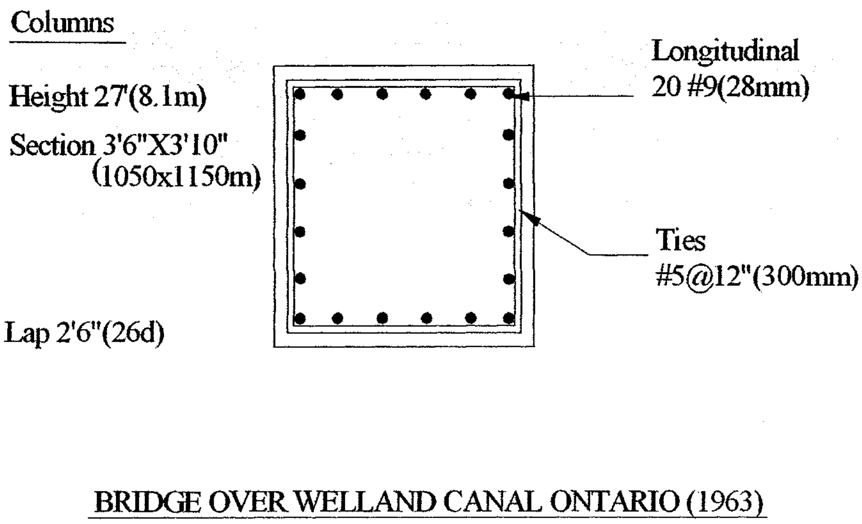


Figure 3.8 Typical column of Welland Canal Bridge in Ontario

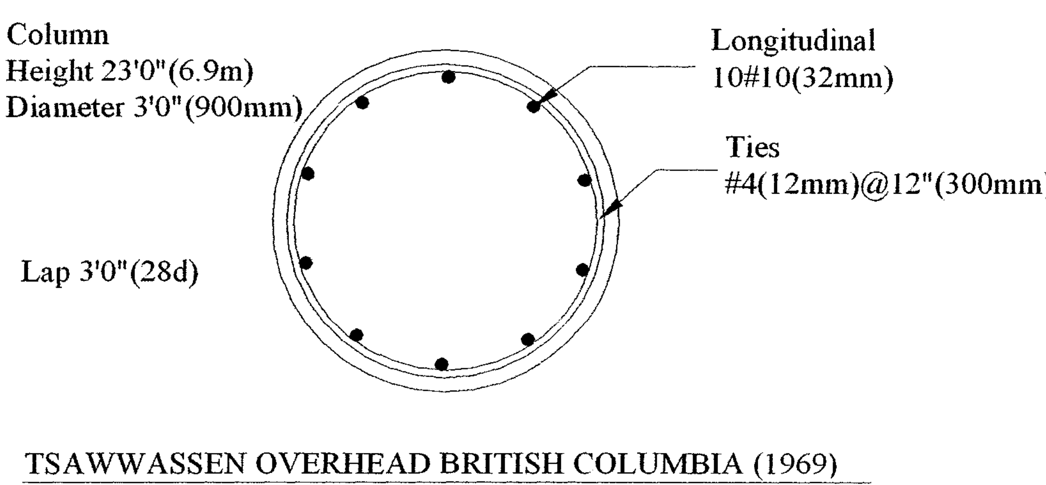


Figure 3.9 Typical column of Tsawwassen Overhead, British Columbia

3.5.2 Design Approach and General Criteria

The column geometry selected are representative of true field conditions and the design assumptions and approaches used in this study is also intended to reflect the conditions of columns in need of seismic retrofit. For instance, the ratio of longitudinal steel reinforcement assumed in the study is 1%. This ratio compares well with those for the Welland Canal Bridge, shown in Fig. 3.8, with 1.1% and the Tsawwassen Bridge, shown in Fig. 3.9, with 1.2% steel ratio. The design process employed for each technology is based on experience gained through the review of previous research/applications, and reflects the Canadian practice to the extend possible.

For steel jacket retrofit, the approach taken was that suggested by Priestley (1996), and used by Caltrans (California Transportation Department). Steel jackets have been used extensively in California and the design approach has been proven to be effective. For FRP, CSA S806-02 (May 2002) was used because this was the only Canadian Standard addressing the application. Since the external prestressing system called Retrobelt was developed and fully tested at the University of Ottawa, the design process outlined by Saatcioglu (2003) and Mes (1999) was followed. And finally, the design process followed for concrete jackets was similar to that described by Priestley (1996) but somewhat modified to reflect Canadian Standards.

Because columns built prior to 1971 were deficient in flexural ductility, had lap splices at the base of the pier (within the potential plastic hinge region) and were weak in shear, all of the deficiencies were investigated and retrofitted to meet higher standards. It is *however important to note that* CSA S806-02 (May 2002) does not permit the use of

FRP jacketing for square and rectangular columns as a seismic retrofit technique when deficiency is in lap splice clamping. Because of this restriction, square columns in this study were not verified for lap splice deficiency when FRP jacket was evaluated. Therefore, the FRP results for these structures are somewhat understated since an alternative method, such as the addition of a steel jacket in the lap region, would be required as an additional cost item.

The retrofit design process involved detailed sectional calculations, involving the computation of moment-curvature relationships, maximum moments, and the neutral axis locations. These were calculated using the program RCSECTION, developed by Pikaso Software Inc. of Ottawa.

3-5-3 Design of Steel Jackets

The process followed to design a jacket for flexural ductility enhancement (or confinement) as suggested by Priestley(1996) is as follows:

1. Determine the required plastic rotation θ_p of the plastic hinge.
2. Determine the plastic curvature from $\phi_p = \dfrac{\theta_p}{L_p}$ where the plastic hinge is given by

   $L_p = g + 0.044 f_y d_{bl}$
3. Find the required curvature from $\phi_m = \phi_y + \phi_p$ where the equivalent bi-linear curvature is found from moment-curvature analysis.
4. Compute the maximum compression strain from $\varepsilon_{cm} = \phi_m c$ where c is the neutral axis depth

From the above, the required volumetric ratio of confinement is $\rho_s = \Phi_j(\varepsilon_{cm})$ where Φ_j is describes the relationship between the ultimate compression strain and the volumetric ratio of the jacket confinement. According to Priestley et al. for steel jackets this can be estimated by $\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f'_{cc}}$.

After completion of the steps outlined above, the required thickness of the various available jackets can be calculated. The approach suggested by Priestley et al. is $t_j = \frac{0.18(\varepsilon_{cm} - 0.004)Df'_{cc}}{f_{yj}\varepsilon_{sm}}$ where t_j is the jacket thickness, f_{yj} is the yield stress, f'_{cc} is the compressive strength of confined concrete and ε_{sm} is the strain at maximum stress in the jacket .

The required thickness of a steel jacket to satisfy the lap splice deficiency requirement is $t_j = \frac{2.42 A_b f_y D}{4pl_s(0.0015E_{sj})}$ where A_b is the cross sectional area of the longitudinal bars, D and E_{sj} are the diameter and the elastic modulus of the steel jacket, f_y is the yield strength of the longitudinal reinforcement, l_s is the lap splice length and p is an equivalent perimeter of the cracked concrete around the longitudinal bars.

Priestley contends that when the shear strength $\phi_s(V_c + V_s + V_p)$ is less than the maximum shear force V^o, the additional shear strength required for the retrofit will be $\phi_s V_{sj} \geq V^o - \phi_s(V_c + V_s + V_p)$.

From the above, he suggests that the thickness of a passive steel jacket can be calculated by the following: $t_j \geq \frac{V^o/\phi_s - (V_c + V_s + V_p)}{0.5\pi f_{yj} D \cot\theta} = \frac{V_{sj}}{2.24 f_{yj} D}$ where V_{sj} = shear strength of the steel jacket and f_{yj} = yield strength of the jacket. However, the Canadian Codes do not include the effect of axial loads (V_p) so this portion of the column shear resistance was ignored in the current study.

For 500 mm circular column (12#20 bars)(L=1.5m & 3.0m)

a) Confinement Design:

$$t_j = \frac{0.18(\varepsilon_{cm} - 0.004)Df'_{cc}}{f_{yj}\varepsilon_{sm}} \qquad \varepsilon_{cm} = \phi_m c \qquad \phi_m = \phi_y + \phi_p \quad , \phi_p = \frac{\theta_p}{L_p}$$

$L_p = g + 0.044 f_y d_{bl}$ = 50+ 0.044(270)(20)=292 mm

$\phi_p = \frac{\theta_p}{L_p}$ = 0.04/292 = 136x10^{-6}/mm $\quad \phi_y$ = 5x10^{-6} (from program), ϕ_m=141x10^{-6}/mm

ε_{cm}=141x10^{-6}x176.6 (from program) =0.025

$f'cc$=42 MPa, assuming ρ_s=0.015

$$t_j = \frac{.018(0.025 - 0.004)(500)42}{270(0.15)} = 1.9mm$$

$$check\,\rho_s = 4x\frac{1.9}{500} = 0.015 - close - to - assumption - ok$$

b) Lap Splices Design:

$$\rho_{sj} = \frac{2.42A_b f_{yl}}{pl_s(0.005E_{sj})} > \frac{2.42A_b f_{yl}}{pl_s f_{sj}}$$

$$p = \frac{\pi D'}{2n} + 2(d_b + c) \le 2\sqrt{2}(c + d_b) = \frac{\pi 400}{2x12}(2x70) = 192 > 2\sqrt{2}x70 = 198mm$$

$$l_s = 20d = 400mm$$

$$\rho_{sj} = \frac{2.42x300x270}{192x400(0.0015x200000)} = 0.0084 > \frac{2.42x300x270}{192x400x270} = 0.009$$

$$t_j = \frac{\rho_{sj}D}{4} = \frac{0.009x500}{4} = 1.05mm$$

c) Shear Design:

Use probable moment of 409 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 409x2/1.5=545 kN and for a 3.0 meter column it is 409x2/3=272 kN.

$$V(shear.capacity.of.existing.column) = V_{concrete} + V_{steel}$$
$$V_c = .2\sqrt{f_c'}.8A_g = .2\sqrt{30}x.8x\pi x(250)^2 = 172kN$$

$$V_s = \frac{\pi}{2}x\frac{A_h f_{yh} D'}{s}\cot\theta = \frac{\pi}{2}x\frac{100x270x400}{300}\cot 30 = 97kN$$

For a 3.0-meter column, shear resistance is sufficient

For a 1.5- meter column,

$$t_j \geq \frac{V/\phi_s - (V_c + V_s)}{0.5\pi fD\cot\theta} = \frac{V_{sj}}{2.24 f_{yj} D} = \frac{545/.85 - 269}{2.24x270x500} = 1.2mm$$

According to Priestley(1996) and the Canadian Steel Tables, the minimum available manufactured thickness of plates is 8 mm.

d) Extent of jacket required:

General recommendations:

Confinement---------D or .25 L_b where L_b is the mid-point of a column in double bending

Lap---------------------Full extent of the lap joints

Shear-------------------2D

For the 1.5-meter column, shear requirement is 2(500)=1000 mm for both the top and bottom of the column, hence an 8 mm jacket should be provided for the total length.

For the 3.0-meter column, the requirement for lap is 400mm at the bottom and for confinement, 500 mm at the top and bottom. Hence an 8 mm jacket is required for a length of 500 mm at the top and bottom.

<u>For a 1000mm circular column (12 # 30 bars) (L = 3.0m & 6.0m)</u>

<u>a) Confinement Design:</u>

$$t_j = \frac{0.18(\varepsilon_{cm} - 0.004)Df'_{cc}}{f_{yj}\varepsilon_{sm}}, \varepsilon_{cm} = \phi_m c, \phi_m = \phi_y + \phi_p, \phi_p = \frac{\theta_p}{L_p}$$

$$L_p = g + 0.044 f_y d_{bl} = 50 + 0.044(270)(30) = 406mm$$

$$\phi_p = \frac{\theta_p}{L_p} = \frac{.04}{406} = 98.5x10^{-6} / mm, \phi_y = 6x10^{-6} / mm(program), \phi_m = 104.5x10^{-6} / mm$$

$$\varepsilon_{cm} = 104.5x10^{-6} x301.3(program) = 0.035$$

$$t_j = \frac{.018(0.035 - 0.004)(1000)42}{270(0.15)} = 5.68mm$$

$$check\, \rho_s = 4x\frac{5.68}{1000} = 0.023 - ok.8mm.\min$$

<u>b) Lap Splice Design:</u>

$$\rho_{sj} = \frac{2.42A_b f_{yl}}{pl_s(0.005E_{sj})} > \frac{2.42A_b f_{yl}}{pl_s f_{sj}}$$

$$p = \frac{\pi D'}{2n} + 2(d_b + c) \le 2\sqrt{2}(c + d_b), \frac{\pi 900}{2x12} + (2x80) = 278 > 2\sqrt{2}x80 = 226mm$$

$$l_s = 20d = 600mm$$

$$\rho_{sj} = \frac{2.42x700x270}{278x600(0.0015x200000)} = 0.01 > \frac{2.42x700x270}{278x600x270} = 0.01$$

$$t_j = \frac{\rho_{sj}D}{4} = \frac{0.01x1000}{4} = 2.5mm$$

c) Shear Design:

Use probable moment of 2502 kN.m developed from program so that applied shear for a 3.0 meter column in double bending is 2502x2/3.0=1668 kN and for a 6.0 meter column it is 2502x2/6.0=834 kN.

$$V(shear.capacity.of.existing.column) = V_{concrete} + V_{steel}$$
$$V_c = .2\sqrt{f_c^{'}}.8A_g = .2\sqrt{30}x.8x\pi x(500)^2 = 688kN$$

$$V_s = \frac{\pi}{2}x\frac{A_h f_{yh} D^{'}}{s}\cot\theta = \frac{\pi}{2}x\frac{100x270x900}{300}\cot 30 = 220kN$$

$$t_j \geq \frac{V^0/\phi_s - (V_c + V)_s}{0.5\pi f_{yj} D\cot\theta} = \frac{V_{sj}}{2.24 f_{yj} D}$$

$$3.0m - column, t_j = \frac{1668/.85 - 908}{2.24x270x1000} = 1.7mm$$

$$6.0m - column - ok$$

d) Extent of jacket required:

Again 8 mm thick plate applies (minimum manufactured).

For 3.0 m column, shear dominates (2D=2000 mm,top and bottom), do whole column

For 6.0 m column, confinement dominates (D=1000 mm, top and bottom), so place a 1000 mm jacket at the top, and one on the bottom.

For a 2000mm circular column (32#35bars) (L = 6.0m &12.0m)

a) Confinement Design:

$$L_p = g + 0.044 f_y d_{bl} = 50 + 0.044(270)(35) = 466mm$$
$$\phi_p = \frac{\theta_p}{L_p} = \frac{.04}{466} = 86x10^{-6}/mm, \phi_y = 5x10^{-6}/mm(program), \phi_m = 91x10^{-6}/mm$$
$$\varepsilon_{cm} = 91x10^{-6}x583(program) = 0.05$$

$$t_j = \frac{.018(0.05-0.004)(2000)42}{270(0.15)} = 17.2mm$$

$$check\rho_s = 4x\frac{18.3}{2000} = 0.036 - this.gives.f_c^{'} = 45MPa$$

$0k$

b) Lap Splice Design:

$$\rho_{sj} = \frac{2.42A_b f_{yl}}{pl_s(0.005E_{sj})} > \frac{2.42A_b f_{yl}}{pl_s f_{sj}}$$

$$p = \frac{\pi D^{'}}{2n} + 2(d_b + c) \le 2\sqrt{2}(c+d_b), \frac{\pi 1900}{2x32} + (2x85) = 263 > 2\sqrt{2}x80 = 240mm$$

$$l_s = 20d = 700mm$$

$$\rho_{sj} = \frac{2.42x1000x270}{240x700(0.0015x200000)} = 0.013 > \frac{2.42x1000x270}{240x700x270} = 0.014$$

$$t_j = \frac{\rho_{sj}D}{4} = \frac{0.014x2000}{4} = 7mm$$

c) Shear Design:

Use probable moment of 20125 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 20125x2/6.0=6710 kN and for a 12.0 meter column it is 20125x2/12.0=3355 kN.

$$V(shear.capacity.of.existing.column) = V_{concrete} + V_{steel}$$

$$V_c = .2\sqrt{f_c^{'}}.8A_g = .2\sqrt{30}x.8x\pi x(1000)^2 = 2753kN$$

$$V_s = \frac{\pi}{2}x\frac{A_h f_{yh} D^{'}}{s}\cot\theta = \frac{\pi}{2}x\frac{100x270x1900}{300}\cot 30 = 464kN$$

$$t_j \ge \frac{V^0/\phi_s - (V_c + V)_s}{0.5\pi f_{yj} D\cot\theta} = \frac{V_{sj}}{2.24 f_{yj} D}$$

$$6.0m - column, t_j = \frac{6710/.85 - 3217}{2.24x270x2000} = 3.8mm$$

$$12.0m - column - t_j = 0.6mm$$

d) Extent of jacket required:

Priestley(1996) recommends that when installing steel jackets, the same thickness of plate be used throughout the column. He argues that the material savings of placing two or three different thickness is outweighed by the additional cost of handling different materials. Therefore, use 18 mm plate throughout.

For 6.0m column, shear requirement = 2D = 4000 mm, so do whole column

For 12.0m column, shear requirement = 2D = 4000 mm, so place 4000 mm jacket at top and bottom.

For a 500 mm square column(8#25 bars) (L= 1.5m & 3.0m)

Note: for all square columns evaluated, the retrofit jacket is assumed to be circular

a) Confinement:

$$t_j = \frac{0.18(\varepsilon_{cm} - 0.004)Df'_{cc}}{f_{yj}\varepsilon_{sm}}, \varepsilon_{cm} = \phi_m c, \phi_m = \phi_y + \phi_p, \phi_p = \frac{\theta_p}{L_p}$$

$$L_p = g + 0.044 f_y d_{bl} = 50 + 0.044(270)(25) = 347mm$$

$$\phi_p = \frac{\theta_p}{L_p} = \frac{.04}{347} = 115x10^{-6}/mm, \phi_y = 15x10^{-6}/mm(program), \phi_m = 130x10^{-6}/mm$$

$$\varepsilon_{cm} = 104.5x10^{-6}x138.5(program) = 0.018$$

$$t_j = \frac{.018(0.018 - 0.004)(707)42}{270(0.15)} = 1.9mm..............check\,\rho_s = 4x\frac{1.9}{707} = 0.011 - ok.8mm.\min$$

b) Lap Splice Design:

$$\rho_{sj} = \frac{2.42A_b f_{yl}}{pl_s(0.005E_{sj})} > \frac{2.42A_b f_{yl}}{pl_s f_{sj}}$$

$$p = \frac{s}{2} + 2(d_b + c) \leq 2\sqrt{2}(c + d_b), \frac{200}{2} + (2x75) = 250 > 2\sqrt{2}x75 = 212mm$$

$$\rho_{sj} = \frac{2.42 A_b f_{yl}}{p l_s (0.005 E_{sj})} > \frac{2.42 A_b f_{yl}}{p l_s f_{sj}}$$

$$p = \frac{s}{2} + 2(d_b + c) \leq 2\sqrt{2}(c + d_b), \frac{200}{2} + (2x75) = 250 > 2\sqrt{2}x75 = 212mm$$

$$l_s = 20d = 500mm$$

$$\rho_{sj} = \frac{2.42x500x270}{212x500(0.0015x200000)} = 0.01 > \frac{2.42x500x270}{212x500x270} = 0.011$$

$$t_j = \frac{\rho_{sj} D}{4} = \frac{0.011x707}{4} = 1.9mm$$

c) Shear Design:

Use probable moment of 589.4 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 589.5x2/1.5=786 kN and for a 3.0 meter column it is 589.5x2/3.0=393 kN.

$$V(shear.capacity.of.existing.column) = V_{concrete} + V_{steel}$$

$$V_c = .2\sqrt{f_c'}.8A_g = .2\sqrt{30}x.8x500x500 = 219kN$$

$$V_s = \frac{A_v f_{yh} D'}{s} \cot\theta = \frac{200x270x400}{300} \cot 30 = 125kN$$

$$t_j \geq \frac{V^0 / \phi_s - (V_c + V)_s}{0.5\pi f_{yj} D \cot\theta} = \frac{V_{sj}}{2.24 f_{yj} D}$$

$$1.5m - column, t_j = \frac{786/.85 - 344}{2.24x270x707} = 1.3mm, 3.0m - column - ok$$

d) Extent of jacket required:

For 1.5 m column, shear dominates (2D=1000 mm), so provide a 8 mm circular jacket over the whole column

For the 3.0 m column, confinement (D=500) so provide a 8 mm circular jacket 500 mm long at the top and bottom

For a 1000mm square column(16#30 bars)(L=3.0m & 6.0m)

a) Confinement Design:

$$t_j = \frac{0.18(\varepsilon_{cm} - 0.004)Df'_{cc}}{f_{yj}\varepsilon_{sm}}, \varepsilon_{cm} = \phi_m c, \phi_m = \phi_y + \phi_p, \phi_p = \frac{\theta_p}{L_p}$$

$$L_p = g + 0.044 f_y d_{bl} = 50 + 0.044(270)(30) = 374$$

$$\phi_p = \frac{\theta_p}{L_p} = \frac{.04}{374} = 107x10^{-6}, \phi_y = 7x10^{-6}(program), \phi_m = 114x10^{-6}$$

$$\varepsilon_{\mathrm{cm}} = 114x10^{-6}x240(program) = 0.027$$

$$t_j = \frac{0.18(0.027 - 0.004)(1414)43}{270(0.15)} = 6.21mm$$

$$check\,\rho_s = 4x\frac{6.21}{1414} = 0.018 - ok.8mm.\min$$

b) Lap Splice Design

$$\rho_{sj} = \frac{2.42 A_b f_{yl}}{pl_s(0.005E_{sj})} > \frac{2.42 A_b f_{yl}}{pl_s f_{sj}}$$

$$p = \frac{s}{2} + 2(d_b + c) \leq 2\sqrt{2}(c + d_b), \frac{225}{2} + (2x80) = 285 > 2\sqrt{2}x80 = 226mm$$

$$l_s = 20d = 600mm$$

$$\rho_{sj} = \frac{2.42x700x270}{226x600(0.0015x200000)} = 0.011 > \frac{2.42x700x270}{212x600x270} = 0.012$$

$$t_j = \frac{\rho_{sj}D}{4} = \frac{0.012x1414}{4} = 4.3mm$$

c) Shear Design:

Use probable moment of 3896 kN.m developed from program, so that applied shear for a 3.0 meter column in double bending is 3896x2/3=2597 kN and for a 6.0 meter column it is 3896x2/6=1299 kN.

$$V(shear.capacity.of.existing.column) = V_{concrete} + V_{steel}$$

$$V_c = .2\sqrt{f_c^{'}}.8A_g = .2\sqrt{30}x.8x1000x1000 = 876kN$$

$$V_s = \frac{A_v f_{yh} D^{'}}{s}\cot\theta = \frac{200x270x900}{300}\cot 30 = 281kN$$

$$t_j \geq \frac{V^0/\phi_s - (V_c + V)_s}{0.5\pi f_{yj} D \cot\theta} = \frac{V_{sj}}{2.24 f_{yj} D}$$

$$3.0m - column, t_j = \frac{2597/.85 - 1157}{2.24x270x1414} = 2.2mm$$

$$6.0m - column, t_j = \frac{1299/.85 - 1157}{2.24x270x1414} = 0.43mm$$

d) Extent of jacket required:

Again the minimum jacket thickness of 8 mm applies.

For 3.0 m column, shear dominates (2D=2828 mm top and bottom). Therefore jacket the whole length.

For 6.0 m column, as above, hence do the whole column.

For a column 2000mm square (44#35 bars)(L = 6.0m ,and 12.0m)
a) Confinement Design:

$$t_j = \frac{0.18(\varepsilon_{cm} - 0.004)Df_{cc}^{'}}{f_{yj}\varepsilon_{sm}}, \varepsilon_{cm} = \phi_m c, \phi_m = \phi_y + \phi_p, \phi_p = \frac{\theta_p}{L_p}$$

$$L_p = g + 0.044 f_y d_{bl} = 50 + 0.044(270)(35) = 466mm$$

$$\phi_p = \frac{\theta_p}{L_p} = \frac{.04}{466} = 86x10^{-6}/mm, \phi_y = 4x10^{-6}/mm(program), \phi_m = 90x10^{-6}/mm$$

$$\varepsilon_{cm} = 90x10^{-6}x466(program) = 0.04$$

$$t_j = \frac{.018(0.04 - 0.004)(2828)43}{270(0.15)} = 19.4mm$$

$$check\,\rho_s = 4x\frac{20.6}{1414} = 0.029$$

ok

b) Lap Splice Design:

$$\rho_{sj} = \frac{2.42 A_b f_{yl}}{p l_s (0.005 E_{sj})} > \frac{2.42 A_b f_{yl}}{p l_s f_{sj}}$$

$$p = \frac{s}{2} + 2(d_b + c) \le 2\sqrt{2}(c + d_b), \frac{172}{2} + (2x85) = 256 > 2\sqrt{2}x85 = 240mm$$

$$l_s = 20d = 700mm$$

$$\rho_{sj} = \frac{2.42x1000x270}{256x700(0.0015x200000)} = 0.014 > \frac{2.42x700x270}{256x700x270} = 0.012$$

$$t_j = \frac{\rho_{sj} D}{4} = \frac{0.014x2828}{4} = 10mm$$

c) **Shear Design:**

Use probable moment of 31474 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 31474x2/6=10491 kN and for a 12.0 meter column it is 31474x2/12=5245 kN.

$$V(shear.capacity.of.existing.column) = V_{concrete} + V_{steel}$$

$$V_c = .2\sqrt{f_c'}.8A_g = .2\sqrt{30}x.8x2000x2000 = 3505kN$$

$$V_s = \frac{A_v f_{yh} D'}{s} \cot\theta = \frac{200x270x1900}{300} \cot 30 = 592kN$$

$$t_j \ge \frac{V^0/\phi_s - (V_c + V)_s}{0.5\pi f_{yj} D \cot\theta} = \frac{V_{sj}}{2.24 f_{yj} D}$$

$$6.0m - column, t_j = \frac{10491/.85 - 4097}{2.24x270x2828} = 5mm$$

$$12.0m - column, t_j = \frac{5245/.85 - 4097}{2.24x270x2828} = 1.2mm$$

d) Extent of jacket required:

For the 6.0 m column the requirement for confinement is 25 mm for 2000 mm at the top and bottom. For shear, the requirement is only 8 mm for the whole column. But as previously stated, one thickness should be installed, hence the whole column is done with 20 mm plate. For the 12 m column install 20 mm plate of 4000mm length at the top and bottom.

3-5-4 Design of FRP Jackets

For the purposes of this design study, the FRP selected characteristics are: E_f is 60,000 MPa, ply thickness is 0.9mm, and the stress is 700 MPa.

For 500 mm circular column (12#20 bars) (L=1.5m & 3.0m)

$$P_{ro} = \pm AE_c f_c'(A_a - A_{st}) + AE_s f_y A_{st}$$

$$P_{ro} = 0.79x0.6x30x(\pi x250^2 - 12x300) + 0.85x270x12x300 = 3567kN$$

$$P_f = 800kN, \frac{P_f}{P_{ro}} = 0.22 > 0.2; use 0.22$$

a) Confinement Design:

$$k_c = 1.0 \, for.circular.column$$

$$f_{Fj} = 0.004E_f = (0.004)(60000) = 240 < 0.75(700) = 525MPa$$

$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(500)(\frac{30}{240})(\frac{800}{3567})(\frac{0.04}{1}) = 1.1mm$$

$$Ply.thickness = 0.9mm.\frac{1.1}{.9} = 1.25 .... use..2..ply$$

b) Lap Splice Design:

$$k_c = 1.0 \, for.circular.column$$

$$f_{Fj} = 0.002E_f = (0.002)(60000) = 120 < 0.75(700) = 525MPa$$

$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(500)(\frac{30}{120})(\frac{800}{3567})(\frac{0.04}{1}) = 2.24mm$$

$$Ply.thickness = 0.9mm.\frac{2.24}{.9} = 2.5 .... use..3..ply$$

c) Shear Design: Use probable moment of 409 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 409x2/1.5=545 kN and for a 3.0 meter column it is 409x2/3=272 kN.

$$V_c = 0.2(\lambda)(\phi_c)\sqrt{f_c'}\,b_w d = 0.2(.6)(\sqrt{30})(500)(357) = 117kN$$

$$V_s = \frac{(\phi_s)A_v f_y d}{s} = \frac{0.85(200)(270)(500)}{300} = 76kN$$

$$contribution.of.FRP.jacket = V_f = 2(\phi_f)f_{Fd}t_j D$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{545-117-76}{2(.85)(280)(500)} = 1.5mm/0.9mm = 1.7....use.2.ply.for.1.5m.col$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{272-117-76}{2(.85)(280)(500)} = 0.3mm/0.9mm = 1......use.1.ply.for.3.0m.col$$

d) Extent of jacket required:

Criteria:	Confinement	*L/8 or D/2, (the greater of) for the full requirement
		*L/8 or D/2 for half the full requirement thickness
	Lap	*Full requirement to cover the lap distance
		*L/8 or D/2 for half the full requirement thickness
	Shear	*1.5 D from the point of maximum moment

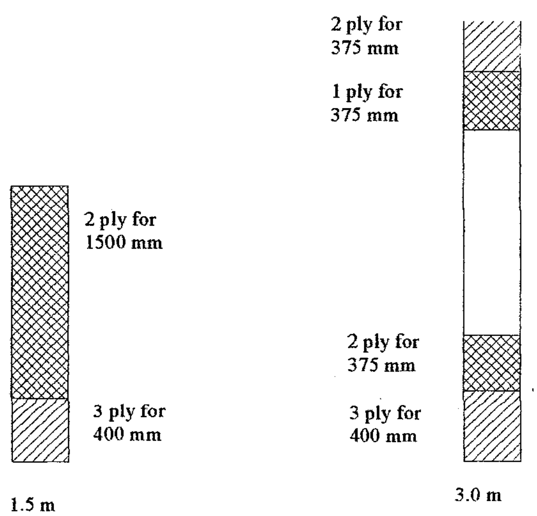


Figure 3.10 Details of FRP jackets for 500 mm circular columns with 1.5 m and 3.0 m lengths

For a 1000mm circular column(12#30 bars) (l= 3.0m & 6.0 m)

$$P_{ro} = \pm \mathrm{AE}_c f_c'(A_a - A_{st}) + \mathrm{AE}_s f_y A_{st}$$
$$P_{ro} = 0.79x0.6x30x(\pi x500^2 - 12x700) + 0.85x270x12x700 = 12976kN$$
$$P_f = 2600kN, \frac{P_f}{P_{ro}} = 0.2 = 0.2; use0.2$$

a) Confinement Design:

$$k_c = 1.0 for.circular.column$$
$$f_{Fj} = 0.004E_f = (0.004)(60000) = 240 < 0.75(700) = 525MPa$$
$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(1000)(\frac{30}{240})(\frac{2600}{12976})(\frac{0.04}{1}) = 2.0mm$$
$$Ply.thickness = 0.9mm.\frac{2.0}{.9} = 2.2..................use..3..ply$$

b) Lap Splice Design:

$$k_c = 1.0 for.circular.column$$
$$f_{Fj} = 0.002E_f = (0.002)(60000) = 120 < 0.75(700) = 525MPa$$
$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(1000)(\frac{30}{120})(\frac{2600}{12976})(\frac{0.04}{1}) = 4.0mm$$
$$Ply.thickness = 0.9mm.\frac{4.0}{.9} = 4.4..................use..5..ply$$

c) Shear Design:

Use probable moment of 2502 kN.m developed from program so that applied shear for a 3.0 meter column in double bending is 2502x2/3.0=1668 kN and for a 6.0 meter column it is 2502x2/6.0=834 kN.

$$V_c = 0.2(\lambda)(\phi_c)\sqrt{f_c'}b_w d = 0.2(.6)(\sqrt{30})(1000)(740) = 486kN$$
$$V_s = \frac{(\phi_s)A_v f_y d}{s} = \frac{0.85(200)(270)(1000)}{300} = 152kN$$
$$contribution.of.FRP.jacket = V_f = 2(\phi_f)f_{Fd}t_j D$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{1668-486-152}{2(.85)(280)(1000)} = 2.1mm/0.9mm = 2.3.....use.3.ply(3.0m.col)$$

$$_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{834-486-152}{2(.85)(280)(1000)} = 0.5mm/0.9mm = .6.....use.1.ply(6.0m.col)$$

d) Extent to retrofit:

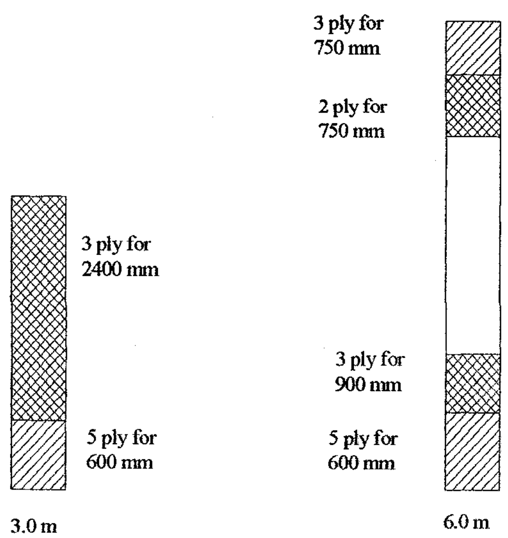


Figure 3.11 Details of FRP jackets for 1000 mm circular columns with 3.0 m and 6.0 m lengths

For a 2000 mm circular column(32#35 bars) (L= 6.0 m & 12.0 m)

$$P_{ro} = \pm \mathrm{AE}_c f_c^{'}(A_a - A_{st}) + \mathrm{AE}_s f_y A_{st}$$

$$P_{ro} = 0.79x0.6x30x(\pi x1000^2 - 32x1000) + 0.85x270x32x1000 = 51562kN$$

$$P_f = 10600kN, \frac{P_f}{P_{ro}} = 0.2 = 0.2; use 0.2$$

a) Confinement Design:

$$k_c = 1.0 for.circular.column$$

$$f_{Fj} = 0.004E_f = (0.004)(60000) = 240 < 0.75(700) = 525MPa$$

$$t_j = 2D\frac{f_c^{'}}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(2000)(\frac{30}{240})(\frac{10600}{51562})(\frac{0.04}{1}) = 4.2mm$$

$$Ply.thickness = 0.9mm.\frac{4.2}{.9} = 4.7..................use..5..ply$$

b) Lap Splice Design:

$$k_c = 1.0 \, for.circular.column$$
$$f_{Fj} = 0.002E_f = (0.002)(60000) = 120 < 0.75(700) = 525MPa$$
$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(2000)(\frac{30}{120})(\frac{10600}{51562})(\frac{0.04}{1}) = 8.2mm$$
$$Ply.thickness = 0.9mm.\frac{8.2}{.9} = 9.1..................use..10..ply$$

c) Shear Design:

Use probable moment of 20125 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 20125x2/6.0=6710 kN and for a 12.0 meter column it is 20125x2/12.0=3355 kN.

$$V_c = 0.2(\lambda)(\phi_c)\sqrt{f_c'}b_w d = 0.2(.6)(\sqrt{30})(2000)(1407) = 1850kN$$

$$V_s = \frac{(\phi_s)A_v f_y d}{s} = \frac{0.85(200)(270)(2000)}{300} = 306kN$$

$$contribution.of.FRP.jacket = V_f = 2(\phi_f)f_{Fd}t_j D$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{6710 - 1850 - 306}{2(.85)(280)(2000)} = 4.8mm / 0.9mm = 5.3.....use.6.ply(6.0m.col)$$

$$_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{3355 - 1850 - 306}{2(.85)(280)(2000)} = 1.26mm / 0.9mm = 1.4.....use.2.ply(12.0m.col)$$

d) Extent of retrofit:

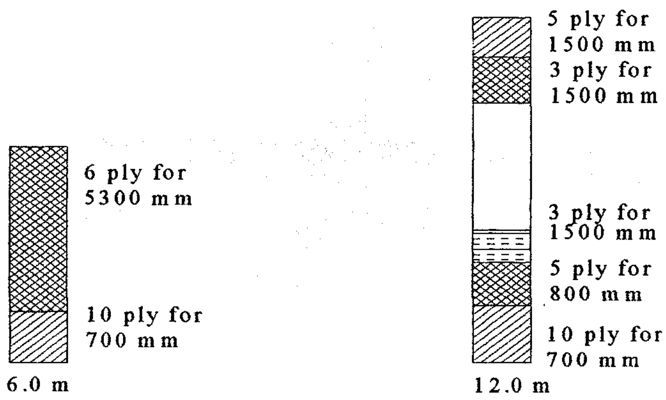


Figure 3.12 Details of FRP jackets for 2000 mm circular columns with 6.0 m and 12.0 m lengths

For 500mm square column(8#25 bars)(L= 1.5 m & 3.0 m)

$$P_{ro} = \pm \mathrm{AE}_c f_c'(A_a - A_{st}) + \mathrm{AE}_s f_y A_{st}$$

$$P_{ro} = 0.79x0.6x30x(500^2 - 8x500) + 0.85x270x8x500 = 4460kN$$

$$P_f = 900kN, \frac{P_f}{P_{ro}} = 0.2 = 0.2; use 0.2$$

a) Confinement Design:

$$k_c = 0.25.for.square.cols$$

$$f_{Fj} = 0.004E_f = (0.004)(60000) = 240 < 0.75(700) = 525MPa$$

$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(500)(\frac{30}{240})(\frac{900}{4460})(\frac{0.04}{\sqrt{.25}}) = 2mm$$

$$Ply.thickness = 0.9mm.\frac{2.0}{.9} = 2.2 .... use..3..ply$$

b) Lap Splice Design:

According to CSA S806-2, lap splice regions in square and rectangular columns shall not be retrofitted with FRP jacketing. For the purposes of this study, it is assumed that lap splice requirements in the existing column are sufficient when designing FRP jackets.

However, if there is a requirement for lap splice modifications, another technology such as steel jackets must be utilized in the splice region.

c) Shear Design:

Use probable moment of 589.2 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 589.2x2/1.5=786 kN and for a 3.0 meter column it is 589.2x2/3=392 kN.

$$V_c = 0.2\lambda \bullet \phi_c \bullet \sqrt{f_c'} b_w d = 0.2(.6)(\sqrt{30})(500)(370) = 122kN$$

$$V_s = \frac{\phi_s \bullet A_v f_y d}{s} = \frac{0.85(200)(270)(370)}{300} = 56kN$$

$$contribution.of.FRP.jacket = V_f = 2\phi_f \bullet f_{Fd} t_j D$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{786-122-56}{2(.85)(280)(500)} = 2.5mm / 0.9mm = 2.8....use.3.ply.for.1.5m.col$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{392-122-56}{2(.85)(280)(500)} = 0.9mm / 0.9mm = 1......use.1.ply.for.3.0m.col$$

d) Extent of retrofit:

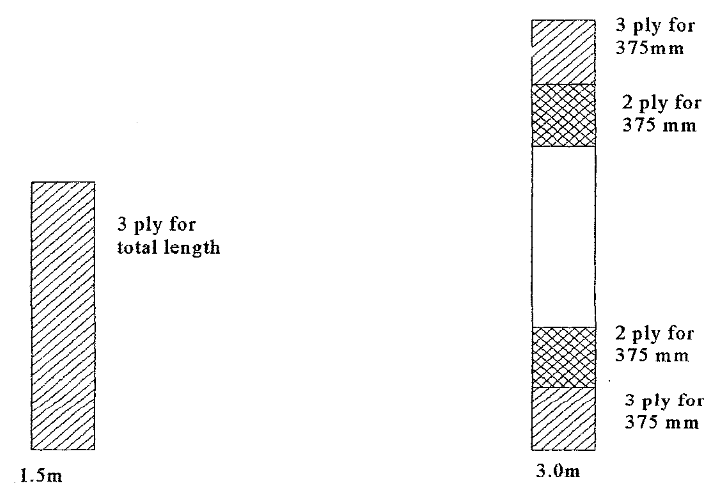


Figure 3.13 Details of FRP jackets for 500 mm square columns with 1.5 m and 3.0 m lengths

For 1000mm square column(16#30 bars)(L = 3.0 m and 6.0 m)

$$P_{ro} = \pm \mathrm{AE}_c f_c'(A_a - A_{st}) + \mathrm{AE}_s f_y A_{st}$$

$$P_{ro} = 0.79x0.6x30x(1000^2 - 16x700) + 0.85x270x16x700 = 16809kN$$

$$P_f = 3400kN, \frac{P_f}{P_{ro}} = 0.2 = 0.2; use 0.2$$

a) Confinement Design:

$$k_c = 0.25.for.square.cols$$

$$f_{Fj} = 0.004E_f = (0.004)(60000) = 240 < 0.75(700) = 525MPa$$

$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(1000)(\frac{30}{240})(\frac{3400}{16809})(\frac{0.04}{\sqrt{.25}}) = 4.0mm$$

$$Ply.thickness = 0.9mm.\frac{4.0}{.9} = 4.4.................use..5..ply$$

b) Lap Splice Design:

According to CSA S806-2, lap splice regions in square and rectangular columns shall not be retrofitted with FRP jacketing. For the purposes of this study, it is assumed that lap splice requirements in the existing column are sufficient when designing FRP jackets. However, if there is a requirement for lap splice modifications, another technology such as steel jackets must be utilized in the splice region.

c) Shear Design:

Use probable moment of 3896 kN.m developed from program so that applied shear for a 3.0 meter column in double bending is 3896x2/3=2597 kN and for a 6.0 meter column it is 3896x2/6=1299 kN.

$$V_c = 0.2\lambda \bullet \phi_c \bullet \sqrt{f_c'} b_w d = 0.2(.6)(\sqrt{30})(1000)(705) = 463kN$$

$$V_s = \frac{\phi_s \bullet A_v f_y d}{s} = \frac{0.85(200)(270)(705)}{300} = 108kN$$

$$contribution.of.FRP.jacket = V_f = 2\phi_f \bullet f_{Fd} t_j D$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{2597-463-108}{2(.85)(280)(1000)} = 4.2mm/0.9mm = 4.7.....use.5.ply.for.3.0m.col$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd} D} = \frac{1299-463-108}{2(.85)(280)(1000)} = 1.5mm/0.9mm = 1.7.....use.2.ply.for.6.0m.col$$

d) Extent of retrofit:

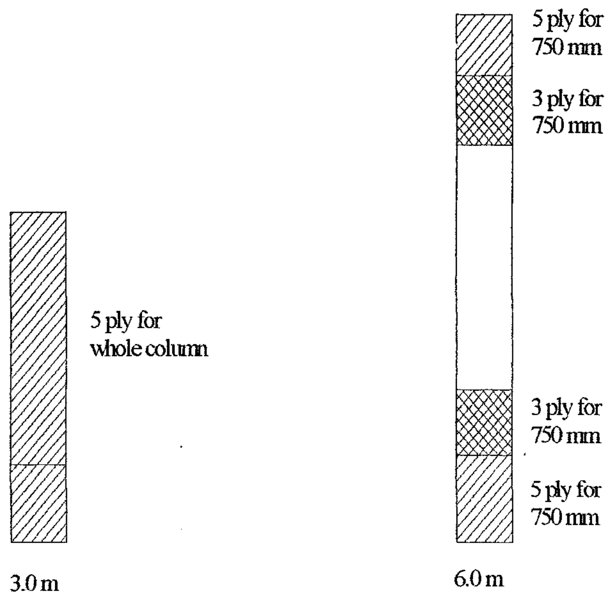


Figure 3.14 Details of FRP jackets for 1000 mm square columns with 3.0 m and 6.0 m lengths

For a 2000 mm square column(44#35 bars)(L= 6.0 m and 12.0 m)

$$P_{ro} = \pm \mathrm{AE}_c f_c^{'} (A_a - A_{st}) + \mathrm{AE}_s f_y A_{st}$$

$$P_{ro} = 0.79x0.6x30x(2000^2 - 44x1000) + 0.85x270x44x1000 = 67074kN$$

$$P_f = 13500kN, \frac{P_f}{P_{ro}} = 0.2 = 0.2; use 0.2$$

a) Confinement Design:

$$k_c = 0.25.for.square.cols$$

$$f_{Fj} = 0.004E_f = (0.004)(60000) = 240 < 0.75(700) = 525MPa$$

$$t_j = 2D\frac{f_c'}{f_{Fj}}\frac{P_f}{P_{ro}}\frac{\delta}{\sqrt{k_c}} = 2(2000)(\frac{30}{240})(\frac{13500}{67074})(\frac{0.04}{\sqrt{.25}}) = 8.0mm$$

$$Ply.thickness = 0.9mm.\frac{8.0}{.9} = 8.9..................use..9..ply$$

b) Lap Splice Design:

According to CSA S806-2, lap splice regions in square and rectangular columns shall not be retrofitted with FRP jacketing. For the purposes of this study, it is assumed that lap splice requirements in the existing column are sufficient when designing FRP jackets. However, if there is a requirement for lap splice modifications, another technology such as steel jackets must be utilized in the splice region.

c) Shear Design:

Use probable moment of 31474 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 31474x2/6=10491 kN and for a 12.0 meter column it is 31474x2/12=5245 kN.

$$V_c = 0.2\lambda\bullet\phi_c\bullet\sqrt{f_c'}b_w d = 0.2(.6)(\sqrt{30})(2000)(1506) = 1980kN$$

$$V_s = \frac{\phi_s\bullet A_v f_y d}{s} = \frac{0.85(200)(270)(1506)}{300} = 230kN$$

$$contribution.of.FRP.jacket = V_f = 2\phi_f\bullet f_{Fd}t_j D$$

$$t_j = \frac{V_f}{2\phi_f f_{Fd}D} = \frac{10491 - 1980 - 230}{2(.85)(280)(2000)} = 8.7mm/0.9mm = 9.7.....use.10.ply.for.6.0m.col$$

$$t_j = \frac{V_f}{2\phi_f.f_{Fd}D} = \frac{5245 - 1980 - 230}{2(.85)(280)(2000)} = 3.1mm/0.9mm = 3.4......use.4.ply.for.12.0m.col$$

d) Extent of retrofit:

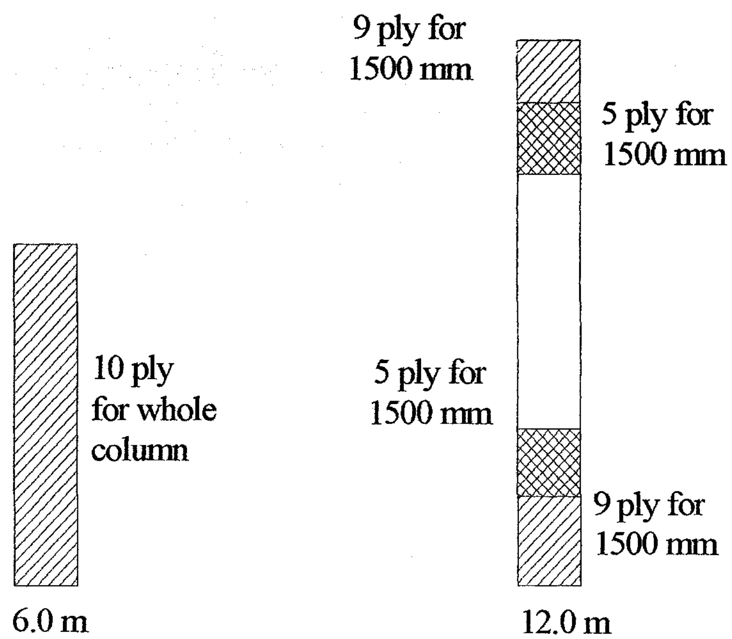


Figure 3.15 Details of FRP jackets for 2000 mm square columns with 6.0 m and 12.0 m lengths

3-5-5 Design of Prestressing Systems (Retro-Belt)

For 500 mm circular column (12#20 bars) L = 1.5 m & 3.0 m)

a) Confinement Design:

$$s_{ps} = (150mm).or.(\frac{h}{4}),............\delta = 0.04$$

$$f_{pe} = f_{pi} + 0.003E_p = 0.5(1500) + 0.003(200,000) = 1350MPa$$

$$A_{ps} = 2\frac{f_c'}{f_{pe}}\frac{P}{\phi \cdot P_o}hs_{ps}\delta = 2\frac{30}{1350}\frac{800}{.85(3567)}(500)(125)(0.04) = 29.3mm^2$$

$$From.table(CISC).use.7.94mm.cable$$

b) Lap Splice Design:

$$f_{pe} = 0.5(1500) + 0.002(200,000) = 1150MPa$$

$$A_{ps} = 2\frac{30}{1150}\frac{800}{0.85(3567)}(500)(125)(0.04) = 34.4mm$$

$$From.tables.use.7.94mm.cable$$

c) Shear Design:

Use probable moment of 409 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 409x2/1.5=545.kN and for a 3.0 meter column it is 409x2/3=272kN.

$$V_c = 0.2\sqrt{f_c^{'}}bd = 0.2\sqrt{30}(500)(357) = 195kN$$

$$V_s = A_v.f_s\frac{d}{s} = 200(270)\frac{500}{300} = 90kN$$

$$50mpa < f_{pi} \leq 0.5f_{pu}$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{545}{.85} - 195 - 90}{2(750 + 0.002(200,000))}\frac{125}{300} = 38.6mm^2$$

$$Use.9.53mm.cable.@125mm.for1.5m.column$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{273}{.85} - 195 - 90}{2(750 + 0.002(200,000))}\frac{125}{300} = 6.5mm^2$$

$$Use.7.94mm.cable.@125mm.for.3.0m.column(same.size.as.confinement)$$

d) Extent of retrofit:

The spacing in this case is h/4 = 125 mm.

For the 1.5m column, strand diameter is 9.53 mm. for the whole length.

For the 3.0m column, strand diameter is 7.94 mm. for a distance of 1000mm at the top and bottom. For middle 1000 mm, spaced @ 250 mm.

For a 1000mm circular column (12 # 30 bars) (L = 3.0 m & 6.0 m)

a) Confinement

$$s_{ps} = (150mm).or.(\frac{h}{4}),............\delta = 0.04$$
$$f_{pe} = f_{pi} + 0.003E_p = 0.5(1500) + 0.003(200,000) = 1350MPa$$
$$A_{ps} = 2\frac{f_c'}{f_{pe}}\frac{P}{\phi \cdot P_o}hs_{ps}\delta = 2\frac{30}{1350}\frac{2600}{.85(12976)}(1000)(150)(0.04) = 62.8mm^2$$
$$From.table(CISC).use.11.11.mm.cable$$

b) Lap Splice Design:

$$f_{pe} = 0.5(1500) + 0.002(200,000) = 1150MPa$$
$$A_{ps} = 2\frac{30}{1150}\frac{2600}{0.85(12976)}(1000)(150)(0.04) = 73.8mm^2$$
$$From.tables.use.12.7mm.cable$$

c) Shear Design:

Use probable moment of 2502 kN.m developed from program so that applied shear for a 3.0 meter column in double bending is 2502x2/3.0=1668 kN and for a 6.0 meter column it is 2502x2/6.0=834 kN.

$$V_c = 0.2\sqrt{f_c'}bd = 0.2\sqrt{30}(1000)(740) = 810kN$$
$$V_s = A_v f_s \frac{d}{s} = 200(270)\frac{1000}{300} = 180kN$$
$$50mpa < f_{pi} \le 0.5f_{pu}$$
$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{1668}{.85} - 810 - 180}{2(750 + 0.002(200,000))}\frac{150}{300} = 63.5mm^2$$
$$Use.11.1mm.cable.@125mm.for3.0m.column$$
$$Shear.for.6.0m.column.ok$$

d) Extent of Retrofit:

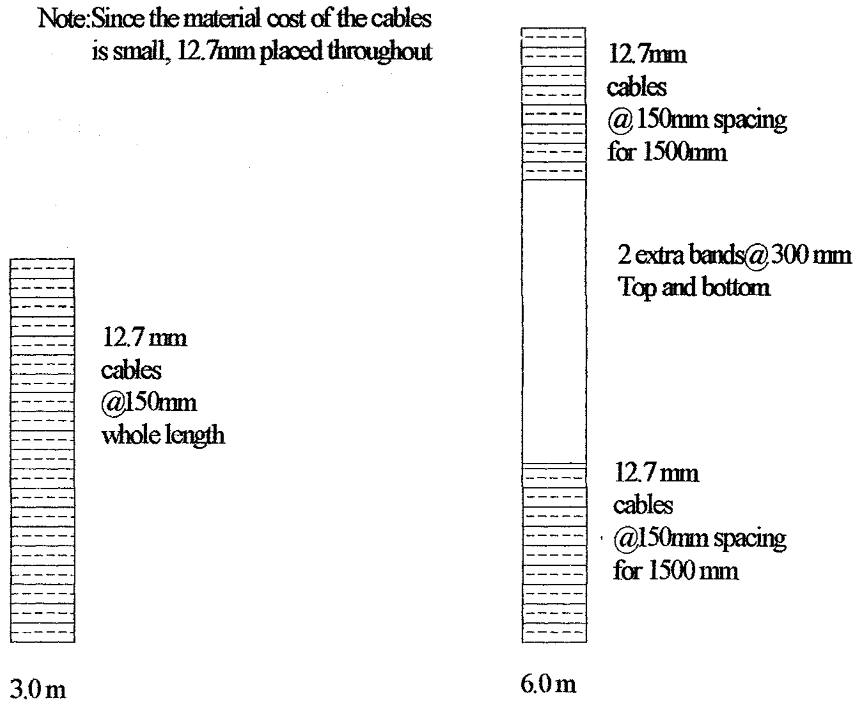


Figure 3.16 Details of Retro-belt design for 1000 mm circular columns with 3.0 m and 6.0 m lengths

For a 2000 mm circular column(32#35 bars) (L= 6.0 m & 12.0 m)

a) Confinement Design:

$$s_{ps} = (150mm).or.(\frac{h}{4}),............\delta = 0.04$$

$$f_{pe} = f_{pi} + 0.003E_p = 0.5(1500) + 0.003(200,000) = 1350MPa$$

$$A_{ps} = 2\frac{f_c^{'}}{f_{pe}}\frac{P}{\phi \cdot P_o}hs_{ps}\delta = 2\frac{30}{1350}\frac{10600}{.85(51562)}(2000)(150)(0.04) = 129mm^2$$

$$From.table(CISC).use.15.2.mm.cable$$

b) Lap Splice Design:

$$f_{pe} = 0.5(1500) + 0.002(200,000) = 1150Mpa$$

$$A_{ps} = 2\frac{30}{1150}\frac{10600}{0.85(51562)}(2000)(150)(0.04) = 151mm^2$$

$$From.tables.use.15.2mm.cable.but.must.be@135mm$$

c) Shear Design:

Use probable moment of 20125 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 20125x2/6.0=6710 kN and for a 12.0 meter column it is 20125x2/12.0=3355 kN.

$$V_c = 0.2\sqrt{f_c'}bd = 0.2\sqrt{30}(2000)(1407) = 3082kN$$

$$V_s = A_v f_s \frac{d}{s} = 200(270)\frac{2000}{300} = 360kN$$

$$50mpa < f_{pi} \le 0.5 f_{pu}$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{6710}{.85} - 3082 - 360}{2(750 + 0.002(200,000))}\frac{150}{300} = 145mm^2$$

$$Use.15.2mm.cable.@140mm.for6.0m.column$$

$$Shear.for.12.0m.column.ok$$

d) Extent of retrofit:

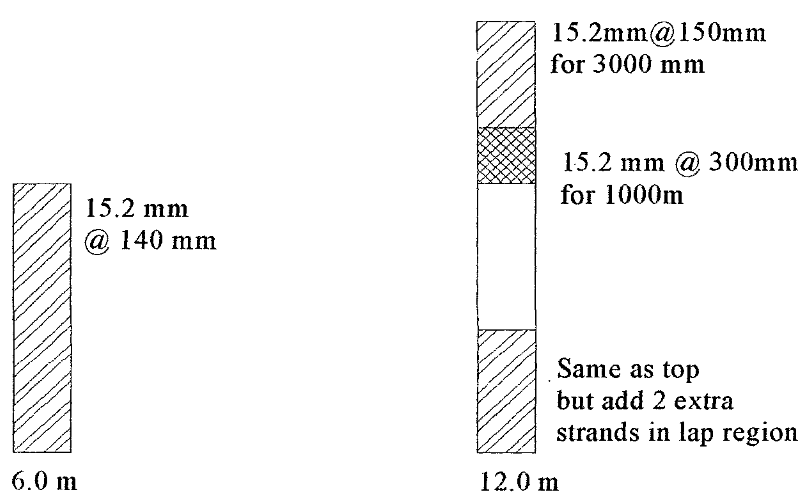


Figure 3.17 Details of Retro-belt design for 2000 mm circular columns with 6.0 m and 12.0 m lengths

<u>For a 500 mm square column(8#25 bars)(L = 1.5 m & 3.0m)</u>

<u>a) Confinement Design:</u>

$$s_{ps} = (150mm).or.(\frac{h}{4}),............\delta = 0.04$$

$$f_{pe} = f_{pi} + 0.003E_p = 0.5(1500) + 0.003(200,000) = 1350MPa$$

$$A_{ps} = 2\frac{f_c'}{f_{pe}}\frac{P}{\phi \cdot P_o}hs_{ps}\delta = 2\frac{30}{1350}\frac{900}{.85(4460)}(500)(125)(0.04) = 26.4mm^2$$

$$From.table(CISC).use.7.94mm.cable@125mm$$

<u>b) Lap Splice Design:</u>

$$f_{pe} = 0.5(1500) + 0.002(200,000) = 1150MPa$$

$$A_{ps} = 2\frac{30}{1150}\frac{900}{0.85(4400)}(500)(150)(0.04) = 31mm^2$$

$$From.tables.use.7.9mm.cable...@125mm$$

<u>c) Shear Design:</u>

Use probable moment of 589.2 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 589.2x2/1.5=786 kN and for a 3.0 meter column it is 589.2x2/3=392 kN.

$$V_c = 0.2\sqrt{f_c'}bd = 0.2\sqrt{30}(500)(370) = 203kN$$

$$V_s = A_v f_s\frac{d}{s} = 200(270)\frac{370}{300} = 67kN$$

$$50mpa < f_{pi} \le 0.5f_{pu}$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{786}{.85} - 203 - 67}{2(750 + 0.002(200,000))}\frac{125}{500} = 71.2mm^2$$

$$Use.12.7mm.cable.@125mm.for1.5m.column$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{392}{.85} - 203 - 67}{2(750 + 0.002(200,000))}\frac{125}{500} = 21mm^2$$

$$Use.6.35mm.cable@125mm.for3.0m.column$$

d) Extent of retrofit:

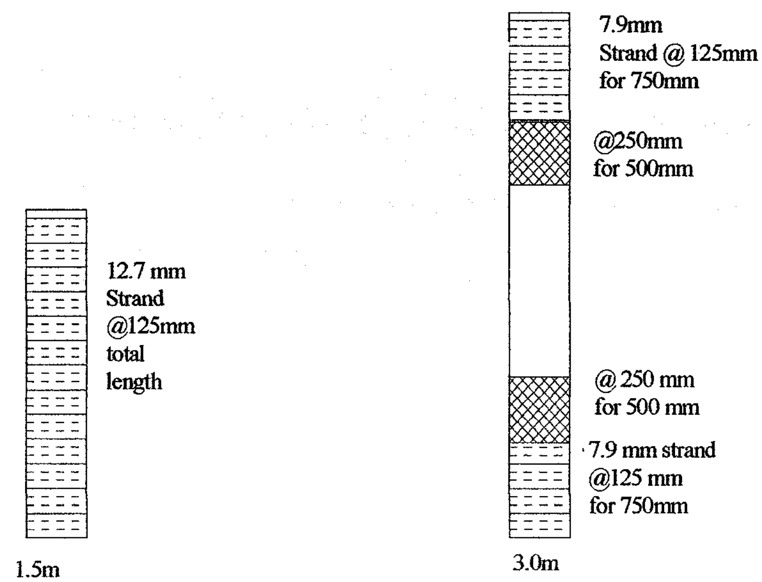


Figure 3.18 Details of Retro-belt design for 500 mm square columns with 1.5 m and 3.0 m lengths

For 1000mm square column(16#30 bars) (L = 3.0 & 6.0 m)

a) Confinement Design:

$$s_{ps} = (150mm).or.(\frac{h}{4}),............\delta = 0.04$$

$$f_{pe} = f_{pi} + 0.003E_p = 0.5(1500) + 0.003(200,000) = 1350MPa$$

$$A_{ps} = 2\frac{f_c'}{f_{pe}}\frac{P}{\phi \cdot P_o}hs_{ps}\delta = 2\frac{30}{1350}\frac{900}{.85(4460)}(1000)(150)(0.04) = 63.5mm^2$$

$$From.table(CISC).use.11.11mm.cable@150mm$$

b) Lap Splice Design:

$$f_{pe} = 0.5(1500) + 0.002(200,000) = 1150MPa$$

$$A_{ps} = 2\frac{30}{1150}\frac{3400}{0.85(16809)}(1000)(150)(0.04) = 74.5mm^2$$

$$From.tables.use.12.7mm.cable...@150mm$$

c) Shear Design:

Use probable moment of 3896 kN.m developed from program so that applied shear for a 3.0 meter column in double bending is 3896x2/3=2597 kN and for a 6.0 meter column it is 3896x2/6=1299 kN.

$$V_c = 0.2\sqrt{f_c'}bd = 0.2\sqrt{30}(1000)(705) = 772kN$$

$$V_s = A_v f_s \frac{d}{s} = 200(270)\frac{705}{300} = 127kN$$

$$50mpa < f_{pi} \leq 0.5 f_{pu}$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)} \frac{s_p}{h} = \frac{\frac{2597}{.85} - 772 - 127}{2(750 + 0.002(200,000))} \frac{150}{1000} = 140mm^2$$

$Use.15.2mm.cable.@150mm.for3.0m.column$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)} \frac{s_p}{h} = \frac{\frac{1299}{.85} - 772 - 127}{2(750 + 0.002(200,000))} \frac{150}{1000} = 41mm^2$$

$Use.9.5mm.cable@150mm.for6.0m.column$

d) Extent of retrofit:

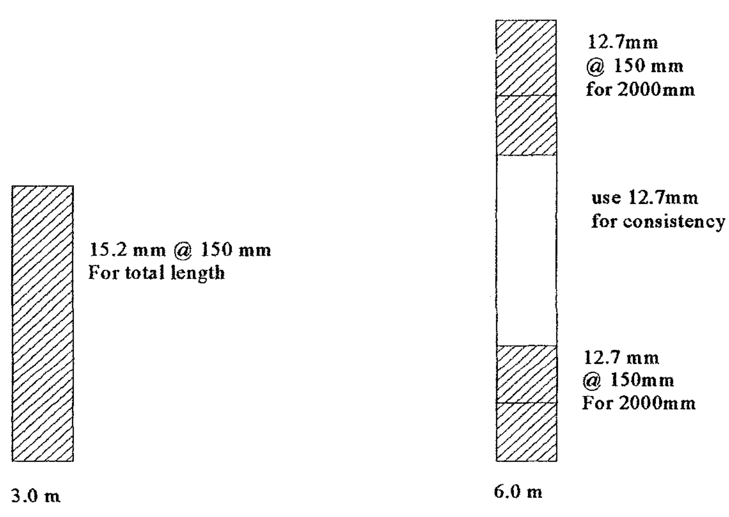


Figure 3.19 Details of Retro-belt design for 1000 mm square columns with 3.0 m and 6.0 m lengths

For a 2000 mm square column(44#35 bars)

a) Confinement Design:

$$s_{ps} = (150mm).or.(\frac{h}{4}),............\delta = 0.04$$

$$f_{pe} = f_{pi} + 0.003E_p = 0.5(1500) + 0.003(200,000) = 1350MPa$$

$$A_{ps} = 2\frac{f_c'}{f_{pe}}\frac{P}{\phi \cdot P_o}hs_{ps}\delta = 2\frac{30}{1350}\frac{13500}{.85(67074)}(2000)(150)(0.04) = 126mm^2$$

$$From.table(CISC).use.15.2mm.cable@150mm$$

b) Lap Splice Design:

$$f_{pe} = 0.5(1500) + 0.002(200,000) = 1150MPa$$

$$A_{ps} = 2\frac{30}{1150}\frac{13500}{0.85(67074)}(2000)(150)(0.04) = 148mm^2$$

$$.15.2mm.cable...@150mm.not.enough - recalculate.to.give.15.2mm@140mm$$

c) Shear Design:

Use probable moment of 31474 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 31474x2/6=10491 kN and for a 12.0 meter column it is 31474x2/12=5245 kN.

$$V_c = 0.2\sqrt{f_c'}bd = 0.2\sqrt{30}(2000)(1506) = 3300kN$$

$$V_s = A_v f_s \frac{d}{s} = 200(270)\frac{1506}{300} = 271kN$$

$$50mpa < f_{pi} \le 0.5f_{pu}$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{10491}{.85} - 3300 - 271}{2(750 + 0.002(200,000))}\frac{150}{2000} = 286mm^2$$

$$.15.2mm.cable.@150mm.for6.0m.column.not.enough - \max.size.cable.available.15.2mm$$
$$recalculate.to..give.15.2mm@75mm.for.6.0m.column$$

$$A_{ps} = \frac{\frac{V_e}{\phi} - V_c - V_s}{2(f_{pi} + 0.002E_p)}\frac{s_p}{h} = \frac{\frac{5245}{.85} - 3300 - 271}{2(750 + 0.002(200,000))}\frac{150}{2000} = 85mm^2$$

$$Use.12.7mm.cable@150mm.for12.0m.column$$

d) Extent of retrofit:

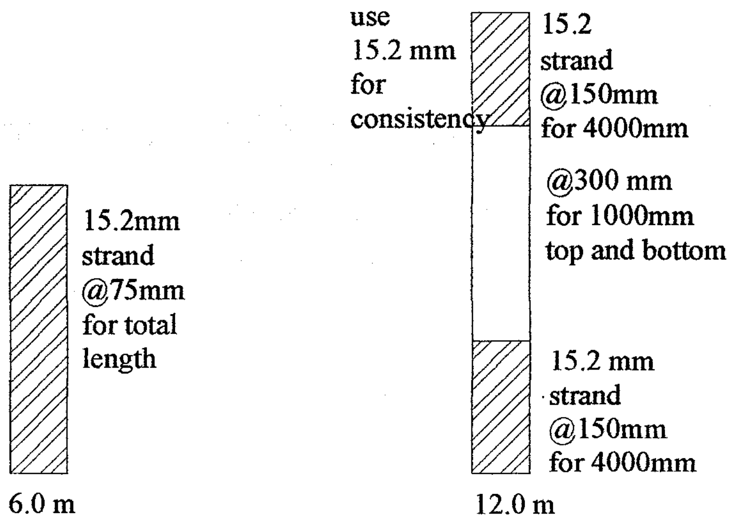


Figure 3.20 Details of Retro-belt design for 2000 mm square columns with 6.0 m and 12.0 m lengths

3-5-6 Design of Reinforced Concrete Jackets

For this design it is assumed that 30 MPa concrete and 400 MPa steel will be used in the jackets. Typically the jacket will have a thickness of 150 mm (100 mm between the existing column and the longitudinal bars and a 50 mm cover) plus the thickness of the longitudinal and transverse bars. The *extent of retrofit* is assumed to be the full length of the existing column because this configuration is easier to construct, cheaper and is more aesthetically pleasing.

The design approach is similar to that suggested by Priestley (1996) with some modifications to reflect the Canadian practice.

For a 500 mm circular column (12#30 Bars) (L=1.5m and 3.0m)

a) Confinement Design:

$$from.CSA...\rho_s = 0.425\left[\frac{A_g}{A_c}-1\right]\frac{f_c^{'}}{f_y} = 0.425\left[\frac{581}{442}-1\right]\frac{30}{400} = 0.01$$

$$\frac{A_h}{s} = \frac{\rho_s D^{'}}{4} = \frac{0.01(750)}{4} = 1.9\,for\,\#10.bars.A_h = 100, s = 53mm..or\,\#15.bars.A_h = 200, s = 105$$

b) Lap Splice Design:

$$\rho_s = 0.009(form.previous.calculations.for.steel.jacket)$$
$$\frac{A_h}{s} = \frac{\rho_s(D^{'})}{4} = 0.009\frac{750}{4} = 1.7.........\#10.bars.@60mm$$

c) Shear Design:

Use probable moment of 409 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 409x2/1.5=545.kN and for a 3.0 meter column it is 409x2/3=272.kN.

$$V_c = 172kN,\quad V_s = 97kN(from.steel.jacket.design)$$
$$V_{required} = 545kN - 269kN = 276kN(for.1.5m.column)$$
$$V_{required} = \frac{\phi_s A_v f_y d}{s}, so, \frac{A_v}{s} = \frac{V_r}{\phi_s f_y d} = \frac{276}{.85(400)(400)} = 2.0$$
$$\#10bars(A_v = 200), s = 100$$
$$3.0m.column.ok.for.shear$$

CSA A23.7.77 limits s to 1/6 of core and between 75 mm and 25mm.

So use #10 bars @ 50mm for both columns.

For a 1000mm circular column (12#30 bars) (L=3.0m and 6.0m)

a) Confinement Design:

$$from CSA...\rho_s = 0.425\left[\frac{A_g}{A_c}-1\right]\frac{f_c'}{f_y} = 0.425\left[\frac{1500}{1250}-1\right]\frac{30}{400} = 0.006$$

$$\frac{A_h}{s} = \frac{\rho_s D'}{4} = \frac{0.0064(1135)}{4} = 1.8......for\#10.bars.A_h = 100, s = 55mm$$

$$for\#15.bars.A_h = 200, s = 110mm$$

b) Lap Splice Design:

$$\rho_s = 0.0125(form.previous.calculations.for.steel.jacket)$$

$$\frac{A_h}{s} = \frac{\rho_s(D')}{4} = 0.0125\frac{1135}{4} = 3.5.................\#15.bars.@60mm$$

c) Shear Design:

$$V_c = 688kN,\quad V_s = 97kN(from.steel.jacket.design)$$

$$V_{required} = 1668kN - 908kN = 760kN(for.3.0m.column)$$

$$V_{required} = \frac{\phi_s A_v f_y d}{s}, so, \frac{A_v}{s} = \frac{V_r}{\phi_s f_y d} = \frac{760}{.85(400)(800)} = 2.7$$

$$\#10bars(A_v = 200), s = 75$$

$$6.0m.column.ok.for.shear$$

For consistency, use #15 bars @ 60 mm for both columns.

For a 2000 mm circular column (32 # 35 bars)(L = 6.0m and 12.0m)

a) Confinement Design:

$$from CSA...\rho_s = 0.425\left[\frac{A_g}{A_c}-1\right]\frac{f_c'}{f_y} = 0.425\left[\frac{4486}{4097}-1\right]\frac{30}{400} = 0.003$$

$$\frac{A_h}{s} = \frac{\rho_s D'}{4} = \frac{0.003(2284)}{4} = 3.4........for\#15.bars.A_h = 200, s = 60mm$$

b) Lap Splice Design:

$\rho_s = 0.01(form.previous.calculations.for.steel.jacket)$

$$\frac{A_h}{s} = \frac{\rho_s(D')}{4} = 0.01\frac{2284}{4} = 5.7....\#15.bars.@35mm......\#20.bars@55mm$$

c) Shear Design:

Use probable moment of 20125 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 20125x2/6.0=6710 kN and for a 12.0 meter column it is 20125x2/12.0=3355 kN.

$$V_c = 2753kN,\ \ V_s = 464kN(from.steel.jacket.design)$$
$$V_{required} = 6710kN - 2753 - 464kN = 3493kN(for.6.0m.column)$$
$$V_{required} = \frac{\phi_s A_v f_y d}{s}, so, \frac{A_v}{s} = \frac{V_r}{\phi_s f_y d} = \frac{3493}{.85(400)(1600)} = 6.4$$
$$\#15bars(A_v = 400), s = 60mm.for.6.0m.column.....12.0m.column.ok.for.shear$$

Use # 15 @ 60mm for all but lap area, here use # 20 @ 55mm

For a 500 mm square column(8#25 bars)(l=1.5m,3.0m)

a) Confinement Design:

$$from.CSA...\frac{A_{sh}}{s} = 0.3h_c\left[\frac{A_g}{A_c} - 1\right]\frac{f_c'}{f_y} = 0.3(760)\left[\frac{880x880}{760x760} - 1\right]\frac{30}{400} = 5.8$$
$$for\ \#15.bars.A_h = 200, s = 60mm$$

b) Lap Splice Design:

$\rho_s = 0.01(form.previous.calculations.for.steel.jacket)$

$$\frac{A_h}{s} = \frac{\rho_s(D')}{4} = 0.01\frac{760}{4} = 1.9..................\#10.bars.@52mm$$

c) Shear Design:

Use probable moment of 589.4 kN.m developed from program so that applied shear for a 1.5 meter column in double bending is 589.5x2/1.5=786 kN and for a 3.0 meter column it is 589.5x2/3.0=393 kN.

$$V_c = 219kN, \quad V_s = 125kN(from.steel.jacket.design)$$
$$V_{required} = 786kN - 219 - 125kN = 442kN(for.1.5m.column)$$
$$V_{required} = \frac{\phi_s A_v f_y d}{s}, so, \frac{A_v}{s} = \frac{V_r}{\phi_s f_y d} = \frac{442}{.85(400)(608)} = 2.1$$
$$\#10bars @ 110mm.but.use.@ 40mm.from.confinement$$
$$3.0m.column.ok.for.shear$$

Use #10 @ 40 mm for both columns

For a 1000 mm square column (16#30 bars)(L= 3.0m & 6.0m)

a) Confinement Design:

$$from.CSA...\frac{A_{sh}}{s} = 0.3h_c\left[\frac{A_g}{A_c} - 1\right]\frac{f'_c}{f_y} = 0.3(1266)\left[\frac{1750^2}{1603^2} - 1\right]\frac{30}{400} = 5.4$$
$$\#15.bars.A_h = 200, s = 74mm.........................\#10bars @ 40mm$$

b) Lap Splice Design:

$$\rho_s = 0.012(form.previous.calculations.for.steel.jacket)$$
$$\frac{A_h}{s} = \frac{\rho_s(D')}{4} = 0.012\frac{1266}{4} = 3.8.....................\#15.bars.@ 50mm$$

c) Shear Design:

Use probable moment of 3896 kN.m developed from program so that applied shear for a 3.0 meter column in double bending is 3896x2/3=2597 kN and for a 6.0 meter column it is 3896x2/6=1299 kN.

$$V_c = 876kN,\quad V_s = 281kN(from.steel.jacket.design)$$
$$V_{required} = 2597kN - 876 - 281kN = 1440kN(for.3.0m.column)$$
$$V_{required} = \frac{\phi_s A_v f_y d}{s}, so, \frac{A_v}{s} = \frac{V_r}{\phi_s f_y d} = \frac{1440}{.85(400)(1012)} = 4.1$$
$$\#15bars@105mm.but.use.@75mm.from.confinement$$
$$6.0m.column.ok.for.shear$$

Use #15 @ 50mm in lap area and #15 @ 75mm in remainder for both columns

For a column 2000mm square (44#35 bars)(L = 6.0m ,and 12.0m)

a) Confinement Design:

$$from.CSA...\frac{A_{sh}}{s} = 0.3h_c\left[\frac{A_g}{A_c} - 1\right]\frac{f_c'}{f_y} = 0.3(2290)\left[\frac{2410^2}{2290^2} - 1\right]\frac{30}{400} = 5.5$$
$$for\,\#15.bars.A_h = 200, s = 70mm$$

b) Lap Splice Design:

$$\rho_s = 0.017(form.previous.calculations.for.steel.jacket)$$
$$\frac{A_h}{s} = \frac{\rho_s(D')}{4} = 0.017\frac{2290}{4} = 9.7..................\#20.bars.@30mm$$

c) Shear Design:

Use probable moment of 31474 kN.m developed from program so that applied shear for a 6.0 meter column in double bending is 31474x2/6=10491 kN and for a 12.0 meter column it is 31474x2/12=5245 kN.

$$V_c = 3505kN,\quad V_s = 592kN(from.steel.jacket.design)$$
$$V_{required} = 10491kN - 3505 - 592kN = 6394kN(for.6.0m.column)$$
$$V_{required} = \frac{\phi_s A_v f_y d}{s}, so, \frac{A_v}{s} = \frac{V_r}{\phi_s f_y d}$$

$$\frac{A_v}{s} = \frac{6394}{.85(400)(1832)} = 10.3.....\#20bars@58mm.for6m.col$$

$$V_{required} = 5245kN - 3505 - 592kN = 1150kN(for.12.0m.column)$$

$$\frac{A_v}{s} = \frac{1150}{.85(400)(1832)} = 1.8....confinement.spacing.ok.for.12mcol$$

Use for 6 m column, #20 @ 30mm in lap region and #20 @ 60mm in remainder

Use for 12 m column, #20 @ 30 mm in lap region and #15 @ 70mm in remainder

3-6 COST ANALYSIS FOR FOUR PRIMARY SYSTEMS

3-6-1 Analysis Approach

A standard approach of cost estimating was utilized in this process, in that the number of units required (i.e. amount of material, number of hours) is calculated and multiplied by the unit cost to obtain the total price. Unit costs were obtained form a number of sources, as indicated below:

a) Yardsticks (2003) provided unit cost information in Canadian dollars for all materials except FRP. It also was the source of installed costs for concrete forms, reinforcing bars, steel plates, and some equipment rental rates.

b) Installed unit costs for FRP were obtained form private correspondence with Don Lamb of Master Builders Technologies Ltd(2003) and Simon Foo at Public Works(2003) and Government Services Canada, reflecting their experiences.

c) Some equipment rental costs were obtained from United Rentals(2003).

d) Walkers(2000) was used to estimate the number of hours required to perform certain operations and applied to a local hourly rate for skilled and unskilled labor.

3-6-2 Steel Jacket Costing

a) Material Estimate

500 mm Circular columns:

Steel for 1.5 m $= \pi dL = \pi .560(1.5) = 2.64m^2.of.8mm.plate.@62.8kg/m^2 = 166kg$

Steel for 3.0 m $\pi dL = \pi .560(0.5+0.5) = 1.6m^2.of.8mm.plate.@62.8kg/m^2 = 100kg$

Grout for 1.5m, $= \frac{\pi}{4}\left[0.550^2 - 0.500^2\right]1.5 = 0.062m^3$

Grout for 3.0 m, $= \frac{\pi}{4}\left[0.550^2 - 0.500^2\right]1.0 = 0.041m^3$

1000 mm Circular columns:

Steel for 3.0 m$= \pi dL = \pi(1.060)(3.0) = 10m^2.of.8mm.plate.@62.8kg/m^2 = 628kg$

Steel for 6.0m$= \pi dL = \pi(1.060)(2.0) = 6.7m^2.of.8mm.plate.@62.8kg/m^2 = 418kg$

Grout for 3.0m$= \frac{\pi}{4}\left[1.05^2 - 1.0^2\right]3.0 = 0.24m^3$

Grout for 6.0m$= \frac{\pi}{4}\left[1.05^2 - 1.0^2\right]2.0 = 0.16m^3$

2000 mm Circular columns:

Steel for 6.0m$= \pi dL = \pi(2.060)(6.0) = 40m^2.of.18mm.plate.@141kg/m^2 = 5640kg$

Steel for12.0m$= \pi dL = \pi(2.060)(8.0) = 54m^2.of.18mm.plate.@141kg/m^{20} = 7614kg$

Grout for 6.0m$= \frac{\pi}{4}\left[2.05^2 - 2.0^2\right]6.0 = 1m^3$

Grout for 12.0m$= \frac{\pi}{4}\left[2.05^2 - 2.0^2\right]8.0 = 1.3m^3$

500 mm Square columns:

Steel for 1.5m = $\pi dL = \pi(.767)(1.5) = 3.6m^2.of.8mm.plate.@62.8kg/m^2 = 227kg$

Steel for 3.0m $\pi dL = \pi(.767)(1.0) = 2.4m^2.of.8mm.plate.@62.8kg/m^2 = 150kg$

Note: for circular jacket on square column, infill is concrete

Concrete for $1.5 = \left[\frac{\pi(0.767)^2}{4} - 0.500^2\right]1.5 = 0.32\,\text{m}^3$

Concrete for $3.0 = \left[\frac{\pi(0.767)^2}{4} - 0.500^2\right]1.0 = 0.21\,\text{m}^3$

1000 mm Square columns:

Steel for 3.0m $\pi dL = \pi(1.474)(3.0) = 13.9m^2.of.8mm.plate.@62.8kg/m^2 = 872kg$

Steel for 6.0m $\pi dL = \pi(1.474)(6.0) = 27.8m^2.of.8mm.plate.@62.8kg/m^2 = 1745kg$

Concrete for $3.0 = \left[\frac{\pi(1.474)^2}{4} - 1.00^2\right]3.0 = 2.2m^3$

Concrete for $6.0 = \left[\frac{\pi(1.474)^2}{4} - 1.0^2\right]6.0 = 4.4m^3$

2000 mm Square columns:

Steel for 6.0 m $\pi dL = \pi(2.888)(6.0) = 54.4m^2.of.20mm.plate.@157kg/m^2 = 8540kg$

Steel for 12.0 m $\pi dL = \pi(2.888)(8.0) = 72.6m^2.of.20mm.plate.@157kg/m^2 = 11400kg$

Concrete for 6.0 m $= \left[\frac{\pi(2.888)^2}{4} - 2.00^2\right]6.0 = 15.3m^3$

Concrete for 12.0 m $=\left[\frac{\pi(2.888)^2}{4}-2.00^2\right]8.0=20.5m^3$

The above material estimates are summarized in Table 3.11.

Table 3.11 Cost of materials for steel jackets

Column	Steel	Grout	Concrete	Total
Circular, 500 mm L=1.5 m	432	140	n.a	572
Circular, 500 mm L=3.0	260	900	n.a	350
Circular, 1000mm L=3.0	1640	530	n.a	2170
Circular, 1000mm L=6.0	1102	350	n.a	1452
Circular, 2000mm L=6.0	14660	2200	n.a	16860
Circular, 2000mm L=12.0	19800	2860	n.a	22660
Square, 500 mm L=1.5	593	n.a	45	638
Square, 500 mm L=3.0	390	n.a	30	420
Square, 1000 mm L=3.0	2267	n.a	300	2567
Square, 1000 mm L=6.0	4540	n.a	580	5120
Square, 2000 mm L=6.0	22200	n.a	2000	24200
Square, 2000 mm L=12.0	29600	n.a	2700	32300

Pricing of materials based on Yardsticks(2002)Steel plate, \$ 2.60 per kg. Grout, \$ 2200 per cubic meter, Concrete, \$ 130 per cubic meter.

b) On site labor

The two classes of labor on this type of job are welders and laborers. The costs were determined by estimating the time required for each activity (based on Walker's 2001) and multiplying by the Canadian hourly rate for this labor class.(Based on Yardsticks 2002).

Welders	500 mm(1.5m) circular, 16 hrs @$35	$560
	500 mm(3.0m) circular, 16 hrs @ $35	$560
	1000mm(3m) circular, 40 hrs @ $35	$1400
	1000mm(6m) circular, 40 hrs @ $35	$1400
	2000mm(6m) circular, 40 hrs @ $35	$1400
	2000mm (12m) circular, 80 hrs @ 35	$2800
	500 mm(1.5m)square, 16 hrs @$35	$560
	500 mm(3.0m) square, 16 hrs @ $35	$560
	1000mm(3m) square, 40 hrs @ $35	$1400
	1000mm(6m) square, 40 hrs @ $35	$1400
	2000mm(6m) square, 40 hrs @ $35	$1400
	2000mm (12m) square, 80 hrs @ 35	$2800

Laborers (for setup, steel handling, clearing site, set-up scaffolding)

	500 mm(1.5m) circular, 80 hrs @$25	$2000
	500 mm(3.0m) circular, 80 hrs @ $25	$2000
	1000mm(3m) circular, 100 hrs @ $25	$2500
	1000mm(6m) circular, 100 hrs @ $25	$2500

2000mm(6m) circular, 120 hrs @ $25	$3000
2000mm (12m) circular, 120 hrs @ 25	$3000
500 mm(1.5m)square, 80 hrs @$25	$2000
500 mm(3.0m) square, 80 hrs @ $25	$2000
1000mm(3m) square, 100 hrs @ $25	$2500
1000mm(6m) square, 100 hrs @ $25	$2500
2000mm(6m) square, 120 hrs @ $25	$3000
2000mm (12m) square, 120 hrs @ 25	$3000

Labor for grout placement (based on unit pricing of $2300 per m^3)

500 mm(1.5m) circular, 2300 @0.062	$140
500 mm(3.0m) circular,2300@0.041	$95
1000mm(3m) circular, 2300@0.24	$550
1000mm(6m) circular, 2300 @0.16	$370
2000mm(6m) circular, 2300 @1.0	$2300
2000mm (12m) circular, 2300 @1.3	$2990

Labor for concrete placement in square columns is assumed to be 2 laborers for 1 day or $25x2x8=$400 for all square column

c) Equipment rentals

Because of the weight and size of the steel jackets, a crane is required for placement. The crane needs to be on site for the duration of the welding operation. The following pricing takes into consideration the duration of the placement operation as well as the minimum costs of cranes for small projects.

A concrete pump is required because of the difficulty of injecting the concrete into the top of the form.

Scaffolding is required during the complete construction period.

Crane Hourly charge is $ 60/ hour
For 500mm columns (circular or square), cost is (16hrs)x($60)=$960

For 1000 mm columns and for the 2000mm columns of length 6.0 m, cost is (40hrs)x($60)=$2400

For the 2000mm column of length 12.0 m, the cost is (80hrs)($60)=$4800

Concrete Pump
Minimum charge is $300

Scaffolding (determined by length of job and height required)
For 500mm and 1000 mm columns, the cost is $500

For 2000mm column, the cost is $1000 for L=6m and $1500 for L= 12

d) Transportation Cost

Because of the weight and size of these steel panels, additional transportation costs are incurred as follows: 500mm cols =$200, 1000mm cols =$400, 2000mm cols=$900

e) Fabrication Cost

The material costs developed above do not include the custom fabrication of the semi-circular jackets. These costs are governed by the size and weight of the panels and are estimated to be $500 for the 500mm, and 1000mm columns and $1000 for the 2000mm columns.

Table 3.12 Summary of costs for steel jackets ($CDN)

Column	Material	Labor	Equip	Transp.	Fab.	*TOTAL*
Circular, 500 mm l=1.5 m	572	2700	1760	200	500	5732
Circular, 500 mm L=3.0	350	2655	1760	200	500	5465
Circular, 1000mm L=3.0	2170	4450	3200	400	500	10720
Circular, 1000mm L=6.0	1452	4270	3200	400	500	9822
Circular, 2000mm L=6.0	16860	6700	3700	900	1000	29160
Circular, 2000mm L=12.0	22660	8790	6600	900	1000	39950
Square, 500 mm L=1.5	638	2960	1760	200	500	6058
Square, 500 mm L=3.0	420	2960	1760	200	500	5840
Square, 1000 mm L=3.0	2567	4300	3200	400	500	10967
Square, 1000 mm L=6.0	5120	4300	3200	400	500	13520
Square, 2000 mm L=6.0	24200	4800	3700	900	1000	34600
Square, 2000 mm L=12.0	32300	6200	6600	900	1000	47000

3-6-3 FRP Jacket Costing

Cost information for FRP application is not contained in Yardsticks(2002) or Walker(2000). The cost of FRP installation was obtained from private correspondence with Simon Foo at Public Works Government Services Canada(2003) and with Don Lamb of Master Builders Technologies Ltd(2003). Information from the purchases of FRP sheets by the University of Ottawa was also used in establishing the costs.

It is important to note that prices can vary substantially with the size of the project and ease of set-up at the site. For small jobs of less than 20 square meters, the set-up would be large in relation to the total job and may include minimum costs. Since most of the materials are manufactured in the USA or overseas, the price can fluctuate with the value of the Canadian dollar.

The fluctuations based on project size are from \$200 to \$650 per m^2 for the first ply and \$80 to \$100 per m^2 for the subsequent plies. The estimated costs that were used for this study are a blended cost and include preparation, all materials, and installation. Preparation includes sand blasting, water blasting, and could include grinding.

The costs used in this study are \$400/sq.m for the first layer and \$90/sq.m for each additional layer.

500 mm Diameter Columns:

For 1.5 m long columns;

$$First\,layer.[\pi d + 0.3(overlap)]1.5 = 2.8m^2$$
$$Other\,layers[\pi d + 0.3(overlap)][1.5 + .4] = 3.6m^2$$

For 3.0 m long columns;

$$First\,layer.[\pi d + 0.3(overlap)][.750 + .775] = 2.8m^2$$
$$Other\,layers[\pi d + 0.3(overlap)][.375 + .375 + 2(.4)] = 3.0m^2$$

1000 mm Diameter Columns:

For 3.0 m long columns;

$$First\,layer.[\pi d + 0.3(overlap)][3.0] = 10.3m^2$$
$$Other\,layers[\pi d + 0.3(overlap)][2(3) + 2(.6)] = 24.8m^2$$

For 6.0 m long columns;

$$First\,layer.[\pi d + 0.3(overlap)][2(1.5)] = 10.3m^2$$
$$Other\,layers[\pi d + 0.3(overlap)][4(.6) + 2(.9) + 3(.75)] = 22.2m^2$$

2000 mm Diameter Columns:

For 6.0 m long columns;

$$First.layer.\left[\pi d + 0.3(overlap)\right]\left[6.0\right] = 39.6m^2$$
$$Other.layers\left[\pi d + 0.3(overlap)\right]\left[4(.7) + 5(6)\right] = 216.5m^2$$

For 12.0 m long columns;

$$First.layer.\left[\pi d + 0.3(overlap)\right]\left[3.0(2)\right] = 39.6m^2$$
$$Other.layers\left[\pi d + 0.3(overlap)\right]\left[4(1.5) + 2(1.5) + 2(1.5) + 4(0.8) + 9(0.7)\right] = 141.9m^2$$

500 mm Square Columns:

For 1.5 m long columns;

$$First.layer.\left[4d + 0.3(overlap)\right]1.5 = 3.5m^2$$
$$Other.layers\left[4d + 0.3(overlap)\right]\left[1.5(2)\right] = 7.0m^2$$

For 3.0 m long columns;

$$First.layer.\left[4d + 0.3(overlap)\right]1.5 = 3.5m^2$$
$$Other.layers\left[4d + 0.3(overlap)\right]\left[0.375(6)\right] = 5.2m^2$$

1000 mm Square Columns:

For 3.0 m long columns;

$$First.layer.\left[4d + 0.3(overlap)\right]3.0 = 12.9m^2$$
$$Other.layers\left[4d + 0.3(overlap)\right]\left[3.0(4)\right] = 51.6m^2$$

For 6.0 m long columns;

$$First.layer.\left[4d + 0.3(overlap)\right]3.0 = 12.9m^2$$
$$Other.layers\left[4d + 0.3(overlap)\right]\left[0.75(12)\right] = 38.7m^2$$

2000 mm Square Columns:

For 6.0 m long columns;

$$First\,layer.\left[4d+0.3(overlap)\right]6.0=49.8m^2 ... Other\,layers\left[4d+0.3(overlap)\right]\left[6.0(9)\right]=448.2m^2$$

For 12.0 m long columns;

$$First\,layer.\left[4d+0.3(overlap)\right]6.0=49.8m^2 ... Other\,layers\left[4d+0.3(overlap)\right]\left[1.5(24)\right]=299.0m^2$$

Table 3.13 Summary of costs for FRP jackets ($CDN)

Column	1st layer Amount M^2	1st layer Cost $(cdn)	Other Layers Amount M^2	Other Layers Cost $(cdn)	Total Cost $(cdn)
Circular, 500 mm L=1.5 m	2.8	1120	3.6	324	1445
Circular, 500 mm L=3.0	2.8	1120	3.0	270	1390
Circular, 1000mm L=3.0	10.3	4120	24.8	2232	6352
Circular, 1000mm L=6.0	10.3	4120	22.2	2000	6120
Circular, 2000mm L=6.0	39.6	15840	216.5	19490	35330
Circular, 2000mm L=12.0	39.6	15840	141.9	12770	28610
Square, 500 mm L=1.5	3.5	1400	7.0	630	2030*
Square, 500 mm L=3.0	3.5	1400	5.2	468	1868*
Square, 1000 mm L=3.0	12.9	5160	51.6	4645	9805*
Square, 1000 mm L=6.0	12.9	5160	38.7	3483	8645*
Square, 2000 mm L=6.0	49.8	19920	448.2	40340	60260*
Square, 2000 mm L=12.0	49.8	19920	299.0	26910	46830*

1st layer $400/sq.m, 2nd layer $90/sq.m
*CSA S806-2 does not permit the retrofit of lap deficient square columns with FRP. Consequently, the above costs may be increased if additional retrofit is done to overcome splice deficiency by another technology.

3-6-4 Costing of External Prestressing Systems

a) Material Estimate for Steel Hardware

The components of this system are prestressing strands which vary in size from 6.35mm to 15.24mm, the Retro-Lock anchors which are used to join the cable ends and maintain the force in strands and the Retro-Bench raisers which are used in square or rectangular columns to distribute lateral pressure evenly on column faces.

500 mm circular columns:

For 1.5 m long column;

9.53 mm strand @125mm, # of loops=1500/125-1=11
length of strand=$[\pi d+0.3](11) = 21m@0.4kg/m = 8.4kg$
$number.of.\mathrm{Re}tro - Lock.(connectors) = 11$

For 3.0 m long column;

7.94 mm strand @125mm, # of loops=2000/125-1=15
..........................@ 250mm, # of loops=1000/250-1=3
length of strand=$[\pi d+0.3](18) = 34m@0.35kg/m = 12kg$
$number.of.\mathrm{Re}tro - Lock.(connectors) = 18$

1000 mm circular columns:

For 3.0 m long column;

12.7 mm strand @150mm, # of loops=3000/150-1=19
length of strand=$[\pi d+0.3](19) = 66m@0.67kg/m = 44kg$
$number.of.\mathrm{Re}tro - Lock.(connectors) = 19$

For 6.0 m long column;

12.7 mm strand @150mm, # of loops=3000/150-1= 19
..........................@300mm,# of loops=1500/300-1= 4
length of strand=$[\pi d+0.3](23) = 79m@0.67kg/m = 53kg$
$number.of.\mathrm{Re}tro - Lock.(connectors) = 23$

2000 mm circular columns:

For 6.0 m long columns;

15.2 mm strand @140mm, # of loops=6000/140-1= 42
length of strand=$[\pi\text{d}+0.3](42) = 276m@0.1.0kg/m = 276kg$
$number.of.\text{Re}\,tro - Lock.(connectors) = 42$

For 12.0 m long columns;

15.2 mm strand @150mm, # of loops=6000/150-1= 42
.........................@300mm,# of loops=2000/300-1= 6
.... extra loops required in lap region=2
length of strand=$[\pi\text{d}+0.3](50) = 330m@1.0kg/m = 330kg$
$number.of.\text{Re}\,tro - Lock.(connectors) = 28$

500 mm square columns:

For 1.5 m long column;

12.7 mm strand @125mm, # of loops=1500/125-1=11
length of strand=$[4\text{d}+0.3](11) = 26m@0.67kg/m = 18kg$
$number.of.\text{Re}\,tro - Lock.(connectors) = 11$
$number.of.\text{Re}\,tro - Bench(raisers) = 11$

For 3.0 m long column;

7.94 mm strand @125mm, # of loops=2000/125-1=15
.........................@ 250mm, # of loops=1000/250-1=3
length of strand=$[4\text{d}+0.3](18) = 42m@0.35kg/m = 15kg$
$number.of.\text{Re}\,tro - Lock.(connectors) = 18$
$number.of.\text{Re}\,tro - Bench(raisers) = 18$

1000 mm square columns: For 3.0 m long column;

15.2 mm strand @150mm, # of loops=3000/150-1=19
length of strand=$[4\text{d}+0.3](19) = 82m@1.0kg/m = 82kg$
$number.of.\text{Re}\,tro - Lock.(connectors) = 19$
$numberof\ \text{Re}\,tro - Bench(raisers) = 19$

For 6.0 m long column;

12.7 mm strand @150mm, # of loops=4000/150-1=26

length of strand=$[4d+0.3](26) = 112m@0.67kg/m = 75kg$

$number.of.Retro - Lock.(connectors) = 26$

$number.of.Retro - Bench(raisers) = 26$

2000 mm square columns:

For 6.0 m long column;

15.2 mm strand @75mm, # of loops=6000/75-1=79

length of strand=$[4d+0.3](79) = 656m@1.0kg/m = 656kg$

$number.of.Retro - Lock.(connectors) = 79$

$numberof\ Retro - Bench(raisers) = 79$

For 12.0 m long column;

15.2 mm strand @150mm, # of loops=8000/150-1=53

15.2 mm strand @ 300mm, # of loops=2000/300-1=6

length of strand=$[4d+0.3](59) = 490m@1.0kg/m = 490kg$

$number.of.Retro - Lock.(connectors) = 59$

$numberof\ Retro - Bench(raisers) = 59$

b) Protective Material:

For this design, it is assumed that shotcreting will be applied to the column to protect the steel hardware against corrosion, while also serving as an esthetically pleasing finish. It is also assumed that the whole column will be finished, not just the part that has been Retro-Belted. The incremental cost savings of partially finishing the column is not warranted.

Circular columns:

$$500.mm.column.(1.5m.col)\frac{\pi}{4}\left[0.7^2 - 0.5^2\right]1.5 = 0.3m^3 (assume \min of 1m^3)$$

$$500.mm.column.(3.0m.col)\frac{\pi}{4}\left[0.7^2 - 0.5^2\right]3.0 = 0.6m^3 (assume \min of 1m^3)$$

$$1000.mm.column(3.0m)\frac{\pi}{4}\left[1.2^2 - 1.0^2\right]3.0 = 1.04m^3$$

$$1000.mm.column(6.0m)\frac{\pi}{4}\left[1.2^2 - 1.0^2\right]6.0 = 2.08m^3$$

$$2000.mm.column(6.0m)\frac{\pi}{4}\left[2.2^2 - 2.0^2\right]6.0 = 4.0m^3$$

$$2000.mm.column(12.0m)\frac{\pi}{4}\left[2.2^2 - 2.0^2\right]12.0 = 8.0m^3$$

Square columns:

$$500.mm.column.(1.5m.col)\left[0.7^2 - 0.5^2\right]1.5 = 0.4m^3 (assume \min of 1m^3)$$

$$500.mm.column.(3.0m.col)\left[0.7^2 - 0.5^2\right]3.0 = 0.8m^3 (assume \min of 1m^3)$$

$$1000.mm.column(3.0m)\left[1.2^2 - 1.0^2\right]3.0 = 1.32m^3$$

$$1000.mm.column(6.0m)\left[1.2^2 - 1.0^2\right]6.0 = 2.64m^3$$

$$2000.mm.column(6.0m)\left[2.2^2 - 2.0^2\right]6.0 = 5.04m^3$$

$$2000.mm.column(12.0m)\left[2.2^2 - 2.0^2\right]12.0 = 10.08m^3$$

Table 3.14 Summary of materials and associated cost for the Retro-belt system

Column	Strand		Connector		Raisers		shotcrete		*TOTAL*
	#M	$	#	$	#	$	M^3	$	$
Circular, 500 mm l=1.5 m	8	**54**	111	**550**	-	-	1	**170**	**774**
Circular, 500 mm L=3.0	12	**80**	18	**900**	-	-	1	**170**	**1150**
Circular, 1000mm L=3.0	44	**295**	19	**950**	-	-	1	**170**	**1415**
Circular, 1000mm L=6.0	53	**355**	23	**1150**	-	-	2	**340**	**1845**
Circular, 2000mm L=6.0	276	**1850**	42	**2100**	-	-	4	**680**	**4630**
Circular, 2000mm L=12.0	330	**2210**	28	**1400**	-	-	8	**1360**	**4970**
Square, 500 mm L=1.5	18	**121**	11	**550**	11	**825**	1	**170**	**1670**
Square, 500 mm L=3.0	15	**100**	18	**900**	18	**1350**	1	**170**	**2520**
Square, 1000 mm L=3.0	82	**550**	19	**950**	19	**1425**	1.4	**240**	**3165**
Square, 1000 mm L=6.0	75	**505**	26	**1300**	26	**1950**	2.7	**460**	**4215**
Square, 2000 mm L=6.0	656	**4395**	79	**3950**	79	**5925**	5.1	**870**	**15410**
Square, 2000 mm L=12.0	490	**3285**	59	**2950**	59	**4425**	10	**1700**	**12360**

Strand cost=\$6.70/kg, Connectors= \$50/unit, Raisers=\$75/unit,Shotcrete=\$170/M^3

c) Labor

The two operations of this system that require labor are the installation of the belts and the finishing process, which in this case is assumed to be shotcreting. This installation is relatively simple and requires only two persons; one to perform the installation (@ \$35/hr) and one helper (@\$25/hr). With setup, the two-man crew could install 20-30 belts on the first day and 30-40 on subsequent days.(8 hr days).

Retro-belt Installation:

For the 500mm and 1000mm columns of any height, as well as for 2000mm diameter circular columns having 12.0 m column height, the number of hours estimated is 8 for each column (8hrs @ \$35 and 8hours @ \$25).

For the 2000 mm diameter circular column having 6.0 m height and 2000mm square columns og 6.0 m and 12.0 m heights, the number of hours of labor is estimated as 16, per column (16 hours @ \$35 and 16 hours @ \$25).

Shotcreting labor:

This process needs to be done in a continuous process and can be accomplished in one day with a 3 person crew. All columns = 3x\$25x8=\$600

d) Equipment Rental

Personal hoisting equipment for Retro-Belt installers:

For 500mm and 1000mm columns of any height, and 2000 mm circular column of 12.0 m height: # Hrs=8 @ \$30.

For the 2000 mm diameter circular columns having 6.0 m column height and 2000 mm square columns of 6.0 m and 12.0 m heights: # Hrs=16 @\$30

Shotcrete Gun:

\$ 320 per day

The labor and equipment cost breakdown for the Retro-belt system is given in Table 3.15. The summary of costs for the same system is give in Table 3.16.

Table 3.15 Labor and equipment cost for Retrobelt System

Column	Retrofit Cost ($Cdn)	Shotcrete Cost ($ Cdn)	Labor Cost ($ Cdn)	Equipment Cost ($ Cdn)
Circular, 500 mm l=1.5 m	480	600	1080	560
Circular, 500 mm L=3.0	480	600	1080	560
Circular, 1000mm L=3.0	480	600	1080	560
Circular, 1000mm L=6.0	480	600	1080	560
Circular, 2000mm L=6.0	960	600	1560	800
Circular, 2000mm L=12.0	480	600	1080	560
Square, 500 mm L=1.5	480	600	1080	560
Square, 500 mm L=3.0	480	600	1080	560
Square, 1000 mm L=3.0	480	600	1080	560
Square, 1000 mm L=6.0	480	600	1080	560
Square, 2000 mm L=6.0	960	600	1560	800
Square, 2000 mm L=12.0	960	600	1560	800

Table 3.16 Summary of costs for Retrobelt System

Column	Material	Labor	Equip	Total.$(cdn)
Circular, 500 mm l=1.5 m	774	1080	560	2414
Circular, 500 mm L=3.0	1150	1080	560	2790
Circular, 1000mm L=3.0	1415	1080	560	3055
Circular, 1000mm L=6.0	1845	1080	560	3485
Circular, 2000mm L=6.0	4630	1560	800	6990
Circular, 2000mm L=12.0	4970	1080	560	6610
Square, 500 mm L=1.5	1670	1080	560	3310
Square, 500 mm L=3.0	2520	1080	560	4160
Square, 1000 mm L=3.0	3165	1080	560	4805
Square, 1000 mm L=6.0	4215	1080	560	5855
Square, 2000 mm L=6.0	15410	1560	800	17770
Square, 2000 mm L=12.0	12360	1560	800	14720

3-6-5 Costing of Concrete Jacketing

a) Materials

Reinforcing bars:

Longitudinal bars are placed around the columns for ease of construction and support for the transverse reinforcement. Four #25 bars per column are needed.

$$500mm(square.or.circular.L = 1.5), steel = 4(1.5)(3.925kg/m) = 23.6kg$$
$$500mm(square.or.circular.L = 3.0), steel = 4(3.0)(3.925kg/m) = 47.2kg$$
$$1000mm(square.or.circular.L = 3.0), steel = 4(3.0)(3.925kg/m) = 47.2kg$$
$$1000mm(square.or.circular.L = 6.0), steel = 4(6.0)(3.925kg/m) = 94.4kg$$
$$2000mm(square.or.circular.L = 6.0), steel = 4(6.0)(3.925kg/m) = 94.4kg$$
$$2000mm(square.or.circular.L = 12.0), steel = 4(12.0)(3.925kg/m) = 188.8kg$$

Transverse reinforcement:

$$500mm(.circular.L = 1.5), steel = \frac{1500}{50} = 30.bars$$
$$......length.of.bars = [\pi(.77) + .1] = 2.52m., Wt = 30(2.52)(0.785) = 59.3kg$$
$$500mm(.circular.L = 3.0), steel = \frac{3000}{50} = 60.bars$$
$$.....length.of.bars = [\pi(.77) + .1] = 2.52m., Wt = 60(2.52)(0.785) = 118.6kg$$
$$1000mm(.circular.L = 3.0), steel = \frac{3000}{60} = 50.bars$$
$$.....length.of.bars = [\pi(1.22) + .1] = 3.93m., Wt = 50(3.93)(1.57) = 308.7kg$$
$$1000mm(.circular.L = 6.0), steel = \frac{6000}{60} = 100.bars$$
$$.....length.of.bars = [\pi(1.22) + .1] = 3.93m., Wt = 100(3.93)(1.57) = 617.5kg$$
$$2000mm(.circular.L = 6.0), steel = \frac{6000}{60} = 100.bars$$
$$.....length.of.bars = [\pi(2.22) + .1] = 7.07m., Wt = 100(7.07)(1.57) = 1110.0kg$$
$$2000mm(.circular.L = 12.0), steel = \frac{12000}{60} = 200.bars$$
$$.....length.of.bars = [\pi(2.22) + .1] = 7.07m., Wt = 200(7.07)(1.57) = 2220.0kg$$

$$500mm(square.L = 1.5), steel = \frac{1500}{40} = 38.bars$$

$$......length.of.bars = [4(.77) + .1] = 3.18m., Wt = 38(3.18)(0.785) = 94.9kg$$

$$500mm(square.L = 3.0), steel = \frac{3000}{40} = 76.bars$$

$$.....length.of.bars = [4(.77) + .1] = 3.18m., Wt = 76(3.18)(0.785) = 189.7kg$$

$$1000mm(square.L = 3.0), steel = \frac{600}{50} = 12.bars$$

$$.. + \frac{2400}{75} = 32.bars$$

$$.....length.of.bars = [4(1.22) + .1] = 4.98m., Wt = 44(4.98)(1.57) = 344.0kg$$

$$1000mm(.square.L = 6.0), steel = \frac{600}{50} = 12.bars$$

$$.. + \frac{5400}{75} = 72.bars$$

$$.....length.of.bars = [4(1.22) + .1] = 4.98m., Wt = 84(4.98)(1.57) = 656.8kg$$

$$2000mm(.square.L = 6.0), steel = \frac{700}{30} = 24.bars$$

$$.. + \frac{5300}{60} = 89.bars$$

$$.....length.of.bars = [4(2.22) + .1] = 8.98m., Wt = 113(8.98)(2.355) = 2389.7kg$$

$$2000mm(square.L = 12.0), steel = \frac{700}{30} = 24.bars(\#20)$$

$$................................ + \frac{11300}{70} = 162(\#15)$$

$$....length.of.bars = [4(2.22) + .1] = 8.98m., Wt = 24(8.98)(2.355) = 507.6kg$$

$$.. = 162(8.98)(1.57) = 2284.0kg$$

Concrete:

$$500mm(circular)L = 1.5, V = \frac{\pi}{4}\left[(0.5+(0.18)2)^2 - 0.5^2\right]1.5 = 0.6m^3(\min.1.m^3)$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots L = 3.0, V = \frac{\pi}{4}\left[(0.5+(0.18)2)^2 - 0.5^2\right]3.0 = 1.2m^3$$

$$1000mm(circular)L = 3.0, V = \frac{\pi}{4}\left[(1.0+(0.18)2)^2 - 1.0^2\right]3.0 = 2.0m^3$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots L = 6.0, V = \frac{\pi}{4}\left[(1.0+(0.18)2)^2 - 1.0^2\right]6.0 = 4.0m^3$$

$$2000mm(circular)L = 6.0, V = \frac{\pi}{4}\left[(2.0+(0.18)2)^2 - 2.0^2\right]6.0 = 7.4m^3$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots L = 12.0, V = \frac{\pi}{4}\left[(2.0+(0.18)2)^2 - 2.0^2\right]12.0 = 14.8m^3$$

$$500mm(square)L = 1.5, V = \left[(0.5+(0.18)2)^2 - 0.5^2\right]1.5 = 0.7m^3(\min.1.m^3)$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots L = 3.0, V = \left[(0.5+(0.18)2)^2 - 0.5^2\right]3.0 = 1.5m^3$$

$$1000mm(square)L = 3.0, V = \left[(1.0+(0.18)2)^2 - 1.0^2\right]3.0 = 2.6m^3$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots L = 6.0, V = \left[(1.0+(0.18)2)^2 - 1.0^2\right]6.0 = 5.1m^3$$

$$2000mm(square)L = 6.0, V = \left[(2.0+(0.18)2)^2 - 2.0^2\right]6.0 = 9.4m^3$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots L = 12.0, V = \left[(2.0+(0.18)2)^2 - 2.0^2\right]12.0 = 18.9m^3$$

Concrete formwork:

Because the concrete jacket is poured around an existing column, a special form is required to perform this operation. The cost for these is estimated at \$63/m^2 which includes the labor for placement and removal.

$500.mm.circular.L = 1.5, A = \pi(.86)1.5 = 4.0m^2$

$500.mm.circular.L = 3.0, A = \pi(.86)3.0 = 8.0m^2$

$1000.mm.circular.L = 3.0, A = \pi(1.36)3.0. = 12.8m^2$

$1000.mm.circular.L = 6.0, A = \pi(1.36)6.0. = 25.6m^2$

$2000.mm.circular.L = 6.0, A = \pi(2.36)6.0. = 44.5m^2$

$2000.mm.circular.L = 12.0, A = \pi(2.36)12.0. = 89.0m^2$

$500.mm.square.L = 1.5, A = 4(.86)1.5 = 5.2m^2$

$500.mm.square.L = 3.0, A = 4(.86)3.0 = 10.4m^2$

$1000.mm.square.L = 3.0, A = 4(1.36)3.0 = 16.3m^2$

$1000.mm.square.L = 6.0, A = 4(1.36)6.0 = 32.6m^2$

$2000.mm.square.L = 6.0, A = 4(2.36)6.0 = 56.6m^2$

$2000.mm.square.L = 6.0, A = 4(2.36)12.0 = 113.2m^2$

Table 3.17 Summary of material costs for reinforced concrete jackets

Column	Rebars ($Cdn)	Concrete ($Cdn)	Forms* ($Cdn)	Total ($Cdn)
Circular, 500 mm l=1.5 m	108	130	252	490
Circular, 500 mm L=3.0	216	160	504	880
Circular, 1000mm L=3.0	463	260	806	1530
Circular, 1000mm L=6.0	926	520	1613	3060
Circular, 2000mm L=6.0	1566	960	2805	5330
Circular, 2000mm L=12.0	3131	1925	5610	10670
Square, 500 mm L=1.5	154	130	330	615
Square, 500 mm L=3.0	308	195	655	1160
Square, 1000 mm L=3.0	510	340	1030	1880
Square, 1000 mm L=6.0	977	665	2055	3700
Square, 2000 mm L=6.0	3230	1225	3570	8025
Square, 2000 mm L=12.0	3215	2460	7130	12805

Rebars \$1.30/kg, Concrete \$130/m^3, *Special Forms \$63/m^2(including labor)

b) Labor

The cost of labor associated with the placement of concrete is usually \$32.50/m^3. Because of the complexities of pouring concrete into a confined area around the existing structure, this rate is \$65.00/m^3.

The cost associated with the placement of reinforcing bars is usually \$1.50/kg. Due to the complexity of this job (i.e placing re-bars around an existing column) the cost is \$2.00/kg for circular columns. The bars that are placed around the square column have an additional complexity in that they need to be bent at a 45° angle near the corners. This brings the cost of rebar placement for square columns to \$2.25/kg.

In order to dowel the longitudinal bars into the footings, the site must be cleared with a backhoe and hand digging, holes drilled into the footing and the bars epoxied into the holes. For each column this would require 3 laborers for one day (3x\$25x8=\$600), plus a backhoe with operator (\$500/day), plus a compressor (\$125/day). The estimated cost of dowelling is then \$1225 per column.

Table 3.18 Summary of labor costs for reinforced concrete jacketing

Columns	Rebars		Concrete		Dowelling	Total
	Wt(kg)	Cost($)	M^3	Cost($)	($)	($)
500c(1.5)	82.9	166	1.0	65	1225	1456
500c(3.0)	165.8	332	1.2	78	1225	1635
1000c(3.0)	356.0	712	2.0	130	1225	2067
1000c(6.0)	712.0	1424	4.0	260	1225	2909
2000c(6.0)	1204.4	2409	7.4	480	1225	4114
2000c(12.0)	2410.8	4822	14.8	962	1225	7009
500s(1.5)	118.5	267	1.0	65	1225	1557
500s(3.0)	236.9	533	1.5	98	1225	1856
1000s(3.0)	391.2	880	2.6	170	1225	2275
1000s(6.0)	751.2	1690	5.1	332	1225	3247
2000s(6.0)	2483.4	5588	9.4	610	1225	7423
2000s(12.0)	2472.8	5564	18.9	1229	1225	8018

Rebar placement \$2.00/kg for circular, \$2.25/kg for square, concrete placement \$65/m^3

c) Equipment (over and above those identified previously)

Scaffolding will be required for a 2-week period to allow for the placement of forms, the pouring of concrete and the removal of forms. This cost, including placement, is approximately $1500/week * 2 weeks = $3000 (may be lower for shorter columns).

A concrete pump will be needed to pump the concrete to the top of the forms. This operation can be done in a day at a cost of $550.

Table 3.19 Summary of costs for reinforced concrete jackets

Column	Material	Labor	Equipment	Total cost
Circular, 500 mm l=1.5 m	490	1456	3550	5496
Circular, 500 mm L=3.0	880	1635	3550	6065
Circular, 1000mm L=3.0	1530	2067	3550	7147
Circular, 1000mm L=6.0	3060	2909	3550	9519
Circular, 2000mm L=6.0	5330	4114	3550	12994
Circular, 2000mm L=12.0	10670	7009	3550	21229
Square, 500 mm L=1.5	615	1557	3550	5722
Square, 500 mm L=3.0	1160	1856	3550	6566
Square, 1000 mm L=3.0	1880	2275	3550	7705
Square, 1000 mm L=6.0	3700	3247	3550	10497
Square, 2000 mm L=6.0	8025	7423	3550	18998
Square, 2000 mm L=12.0	12805	8018	3550	24373

3-7 SUMMARY OF COST ANALYSIS FOR ALL SYSTEMS

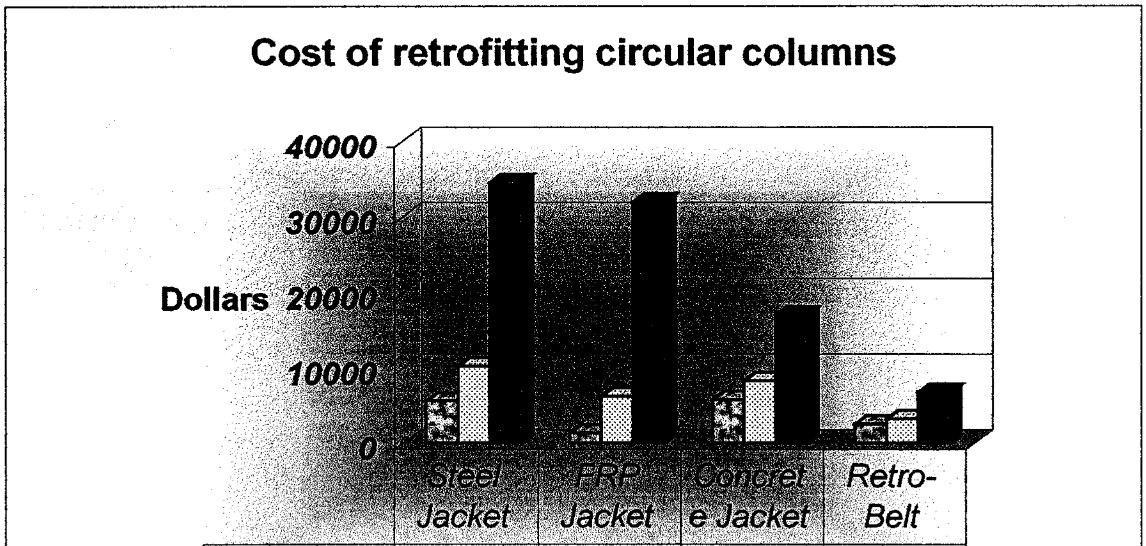
The results of the cost analysis conducted for the four seismic retrofit systems considered are compared in Table 3.20 and Figs. 3.20 and 3.21. Based on these comparisons the following observations can be made:

Table 3.20 Comparison of costs for the four retrofit strategies considered

	Steel jacket	FRP Jackets	Retro-Belt	Concrete jacket
Circular, 500 mm L=1.5 m	5732	1445#	2414	5496
Circular, 500 mm L=3.0	5465	1390#	2790	6065
Circular, 1000mm L=3.0	10720	6352	3055	7147
Circular, 1000mm L=6.0	9822	6120	3485	9519
Circular, 2000mm L=6.0	29160	35330	6990	12994
Circular, 2000mm L=12.0	39950	28610	6610	21229
Square, 500 mm L=1.5	6058	2030*	3310	5722
Square, 500 mm L=3.0	5840	1868*	4160	6566
Square, 1000 mm L=3.0	10967	9805*	4805	7705
Square, 1000 mm L=6.0	13520	8645*	5855	10497
Square, 2000 mm L=6.0	34600	60260*	17770	18998
Square, 2000 mm L=12.0	47000	46830*	14720	24373

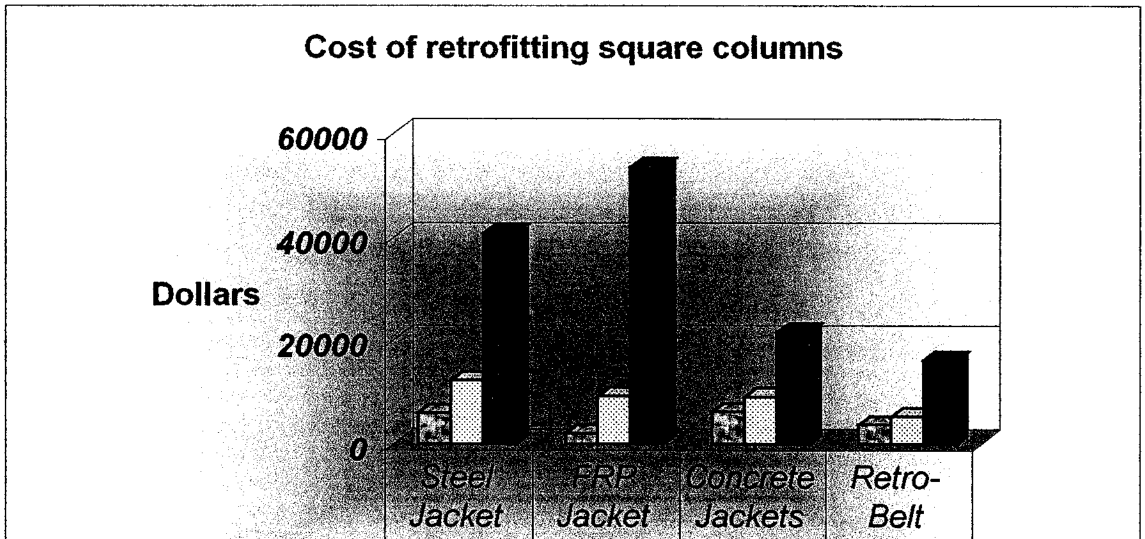
* CSA S806-2 does not permit the retrofit of lap deficient square columns with FRP. Consequently, the above costs may be increased if additional retrofit is done to overcome splice deficiency by another technology.

Minim costs apply because of the small size of job, hence costs may be underestimated.



	Steel Jacket	FRP Jacket	Concrete Jacket	Retro-Belt
500 mm	5600	1420	5780	2602
1000 mm	10270	6236	8333	3270
2000 mm	34555	31970	17111	6800

Figure 3.21 Cost comparisons of circular columns



	Steel Jacket	FRP Jacket	Concrete Jackets	Retro-Belt
500 mm	5950	1950	6144	3735
1000 mm	12240	9225	9101	5330
2000 mm	40800	53545	21685	16245

Figure 3.22 Cost comparisons of square columns

- Retro-Belt is the most economical seismic retrofit technique when compared with the other systems due to its simplicity of design and ease of installation.
- FRP costs used in this study ($400 for the 1st layer and $ 90 for subsequent layers) reflect a reduction in material cost, as compared to earlier applications. This is probably the only material that will come down on price with use.
- Unit prices used in the current study are for the city of Ottawa. It is therefore important to understand the price sensitivities for each system so that local conditions and local pricing can be applied in selecting the most suitable system for a particular location.

<u>Cost Sensitivities of Steel Jackets</u>

* Column size: There is a substantial increase in cost with column size because of the need to use larger and heavier steel plates.

* Skilled labor costs: The welding operation requires skilled labor, and as such, these costs can vary substantially with the time of year and geographical location.

*Crane rental: A heavy crane is required to handle the steel panels and the rental costs can vary by season and location.

<u>Cost Sensitivities of FRP jackets</u>

*Material costs: A large cost component of this technology is the price of the fiber sheets and these change according to demand and availability.

<u>Cost sensitivities of concrete jackets</u>

*Labor costs: This process is very labor intensive and costs could vary substantially by location and season.

Cost sensitivities of Retro-Belt

*Finishing costs: The assumption in this study is that shotcrete would be utilized as a cover over the cables and connectors. On a small job, minimum set-up costs may apply.

- Although not specifically included in this evaluation, the road network economic disruption costs should also be considered. i.e. the closing of a traffic lane on a busy highway for a period of 1-2 weeks would have an economic impact on the system and may force the use of an alternative solution. This issue especially impacts steel jacket and concrete jacket solutions.

CHAPTER 4

SUMMARY AND CONCLUSIONS

4.1 SUMMARY

Recent experience has shown that reinforced concrete bridges designed prior to 1971 cannot resist earthquakes. Their columns do not have sufficient lateral reinforcement to make them ductile or strong enough to absorb these loads. In order to correct these seismic deficiencies, additional external confinement is needed. There are a number of methods available to correct these deficiencies including: steel round and oval jackets, concrete jackets, steel collars built from standard sections, wire mesh and mortar jackets, pre-stressing strands, and advanced composite material jackets. Published literature deals with individual techniques, but none was found dealing with comparisons of these techniques.

This research does compare the techniques by performing the following 5 tasks:

1/ Study available literature and analyze the techniques described in the documents

2/ Evaluate 14 different schemes and eliminate those of least value

3/ Design retrofitting for 12 different columns for 4 different techniques

4/ Estimate the cost of retrofitting the 12 columns

5/ Evaluate and recommend bridge assessment strategies

4.2 CONCLUSIONS

The following conclusions are drawn from this project.

- Recent earthquakes in urban areas such as the one in Northridge in 1994 and in Kobe in 1995 have repeatedly shown the vulnerability of older reinforced concrete columns. Prior to the lessons learned in the 1971 San Fernando earthquake, columns were built with insufficient, inadequately detailed, transverse reinforcement and longitudinal reinforcement that was lapped in the plastic hinge area.
- This inadequate reinforcement and detailing contributes to three different types of column failures: 1/ confinement failure of the flexural plastic hinge region, 2/ bonding loss of the lap-spliced longitudinal reinforcement resulting in the loss of flexural strength 3/ shear failure
- In seismically active areas throughout the world, there are thousands of concrete bridges that could be severely damaged or collapse during an earthquake due to poorly designed columns
- The structural deficiencies of the piers can be corrected by providing external confinement with a number of different materials (steel, concrete, advanced composite materials) used in a variety of configurations (jackets, strands, collars, hybrid jackets/collars)
- An extensive review of literature revealed that very little research has been conducted comparing the advantages, disadvantages, and costs of the individual techniques. Hence the focus of this project.

- There are a wide variety of retrofitting techniques that have been experimentally proven to be effective in that they confine the column to correct the structural deficiencies. What makes these approaches different from one another are the cost, durability, construction, aesthetic, and adaptability factors.
- Based on the assessment of 14 retrofitting alternatives, it was concluded that steel jackets, FRP jackets, pre-stressing strands, and concrete jackets are the most feasible.
- From the detailed cost evaluations, it can be concluded that
  - Retro-Belt (pre-stressing strand) system is very economical compared to the other systems due to its simplicity of design, ease of installation, and low material cost
  - Steel and FRP jacket costs for circular columns are similar, but higher than Retro-Belt and concrete jackets.
  - FRP jacket costs for square columns are the highest of the study and do not include the additional costs of an auxiliary jacket of different material if lap-splice deficiencies exist. (CSA s806 does not permit the use of FRP jackets on square columns). As such, FRP is not a feasible alternative for square columns.
  - Concrete jacket costs in this study are lower than steel and FRP jackets. However, the concrete jacket costs are very labor intensive and were the most difficult to estimate. According to the research literature, this system is not widely utilized due to the high labor. As such, it is concluded that the costs in this study are understated.

- The assessment of a retrofitting program should be one of route classification, and structure prioritization based on general risk parameters. This process should include the following steps: 1/ establishing and categorizing the most important links in the road network, 2/ completing the easiest seismic upgrades, 3/ analyzing each bridge in terms of site seismicity, importance and structural vulnerability, 4/ establish a long term program based on available funds. The plastic collapse mechanism should be used in detailed bridge analysis.

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