STRUCTURAL BEHAVIOR OF STRAND REINFORCED CONCRETE MEMBERS WITHOUT PRESTRESSING

by
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Thesis Title: Structural Behavior of Strand Reinforced Concrete Members without Prestressing

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Dedication

To my beloved parents,

Without whom I wouldn’t make it up to here.
Abstract

One way to reduce the large amount of steel present in reinforced concrete members that are subjected to high loads is to use high strength materials. In this study, use of high strength steel, seven-wire strands with tensile strength of 1860 MPa instead of mild steel rebars, is explored as longitudinal reinforcement in concrete beams and columns without prestressing. The use of such reinforcement is not addressed in design codes since the idea is new. Thus, appropriate strength reduction factors with high strength reinforcement is not established. The stress-strain relationship for the high strength steel strands does not have a clear yield point and lacks adequate ductility compared with conventional mild steel with yield strength less than 520 MPa. The study addresses the issue through both experimental tests and computational studies. The experimental program considers eight flexural tests of 2.2 m long beams subjected to 2-point loading, another six flexural tests of 1.7 m short beams subjected to a single point load, and four axial compression tests of 1 m tall columns. The variables considered in the study are the steel reinforcement ratio, compressive strength of the concrete, and loading condition. The experimental results showed that the load-deflection relationship for strand reinforced beams is much different than that for rebar reinforced beams. Both the stiffness prior to steel yielding and the ductility in the high strength steel beams are lower than in the mild steel beams. These findings were closely predicted by the theoretical part of the study that involved modeling the concrete by the Thornfeldt model and the steel by an elasto-plastic relationship. The correlation between steel strain and curvature ductility for strand reinforced sections is used to predict new strain limits for compression-controlled and tension-controlled regions to achieve similar ductility as the mild steel reinforced sections. The new strain limits for Grade 1860 MPa steel for compression-controlled and tension-controlled sections are 0.01 and 0.02, respectively. The limited tests conducted on columns showed that the strength of strand reinforced columns is about 10% lower than the theoretical prediction for ideal conditions by the code, with no appreciable effect on ductility.

Search terms: Beams, columns, curvature ductility, deflection ductility, reinforced concrete, rebars, high strength strands.
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Chapter 1: Introduction

In this chapter, background information on the subject is provided, as well as the problem statement, objectives and scope of the study. Also, the thesis framework and organization plan is included.

1.1 Introduction and Research Significance

Steel is among of the most commonly used solid materials after concrete in the construction industry. Prestressing steel is one type of reinforcement that comes in the form of wires, strands, or deformed bars. Those forms of reinforcements were influenced by the need for high strength steel reinforcement in order to improve the load-carrying capacity and serviceability of long span concrete elements. Seven wires steel strands are mainly manufactured in three sizes: 9.5, 12.7 and 15.2 mm. The steel strands have a yield strength that is more than 3 times larger than that of conventional reinforcement. Figure 1 shows a typical prestressing stand.

![Commonly used prestressing strand](image)

**Figure 1: Commonly used prestressing strand [1].**

This type of steel can potentially be used without prestressing as a replacement of the traditional reinforcement in different structural elements. The superior mechanical properties of such reinforcement as well as the economical characteristics in terms of load carrying capacity to reinforcement volume make it a good possible alternative to conventional reinforcement. Indeed, using unstretched steel strands as a conventional reinforcement in concrete members has not been investigated much in the past, except for the recent work of Baran and Arsava [2]. If proven feasible, this study may open the door for commercial use of such reinforcement in concrete
members. Unlike the current study, the previous work of Baran and Arsava on the subject has dealt with pure flexural behavior of beams, with no investigation of the cases of concurrent shear and flexure, as well as axial compressive load.

It is known that conventional rebars made with billet, axle or low alloy steels have distinct yield point on their stress-strain relationships; while prestressing strands do not have that, as shown in Figure 2. In the study completed by Baran and Arsava [2], displacement deformability ratio was proposed as a measure of ductility in flexural beams with unstretched strands. This ductility measure is defined as the ratio of ultimate deflection to service deflection. In order to suggest design recommendations for such a type of reinforcement in concrete members, a comprehensive study on the load-deflection relationship, strength, and ductility needs to be carried out. Concrete members reinforced with unstretched steel strands and subjected to flexure, shear and axial compression are not currently supported by structural design codes because they have not been extensively investigated in the past. By researching the steel strand’s use in various concrete members subjected to different actions, the applicability of such reinforcement can be evaluated and design recommendations can be proposed in accordance with the structural design codes.

![Stress-strain relationships for typical reinforcing bar and strand.](image)

**Figure 2**: Stress-strain relationships for typical reinforcing bar and strand.

### 1.2 Problem Statement

Concrete members are often subjected to large magnitude of loads resulting in high bending moment, shear or axial compression. Examples of such cases include girders, transfer slabs, and columns in high-rise buildings. The use of conventionally
reinforced elements when the load is high and the cross-section dimensions are limited can result in congested steel cages with little room for concrete to infiltrate between the rebars. Therefore, there is a need to investigate the use of high strength steel in order to reduce the amount of steel reinforcement in such cases. This could solve the problem by reducing the demand for larger volumes of steel and increasing the spacing between the reinforcement. Extensive research has been recently conducted on the use of high strength bars with yield strength in the order of 550-700 MPa. Steel reinforcement with higher strength than 700 MPa has not been addressed. However, there is only one published study about the application of unstretched steel strands with ultimate tensile strength of 1860 MPa as conventional reinforcement in flexural concrete beams [2]. In that study, it was shown that lack of adequate ductility of concrete beams conventionally reinforced with high strength strands was a major concern. Since there is no distinct yield point in the stress-strain curve of high strength steel, experimental and numerical assessment are needed to assess the ductility for concrete sections with this type of steel, which is not covered in most structural design codes. Moreover, the study completed by Baran and Arsava [2] was not comprehensive since only one concrete compressive strength was used, the reinforcement amount was within a narrow range, and flexure was considered without shear. Hence, the findings of the study might not be applicable in the cases where the strands reinforcement area falls outside of the considered range and the concrete compressive strength is different from what was used. Also, the use of strands as longitudinal compression reinforcement in tied columns was not investigated. This study attempts to investigate the potential of this type of reinforcement for various structural applications, including beams and columns. In particular, it addresses both strength and ductility considerations, and proposes relevant design recommendations for flexure, combined flexure and shear and pure axial compression.

1.3 Theoretical Background

To design any reinforced concrete member, the structural design code provides simple steps to determine the strength of members subjected to various load effects, such as bending moment, shear and axial loads. In the ACI 318-14, flexural strength of singly-reinforced rectangular sections at ultimate can be predicted as [3]:

$$M_{n} = A_s f_y \left(d - \frac{a}{2}\right)$$  \hspace{1cm} (1)
where:

- \( f_y \) = steel yield strength (MPa),
- \( A_s \) = longitudinal steel area (mm\(^2\)),
- \( d \) = effective depth of reinforcement from the extreme compressive fibers (mm),
- \( a \) = equivalent rectangular stress block depth (mm), taken equal to \( \beta_1 c \),
- \( c \) = the depth of the neutral axis from the extreme compression fibers (mm),

and

- \( \beta_1 \) = variable that is related to the shape of the concrete stress-strain curve at ultimate.

In a flexural member, equilibrium between tension in the steel and compression in the concrete gives the depth of the equivalent rectangular stress block as follows:

\[
a = \frac{A_s f_y}{0.85 f'_c d} \tag{2}
\]

Figure 3 shows a singly-reinforced section in flexure, together with its strain and stress diagrams at ultimate.

![Diagram of a singly-reinforced section in flexure](image)

**Figure 3: Strain distribution diagram for single reinforced section**

In order to ensure a minimum ductile behavior of newly designed flexural members, the ACI 318-14 code requires the strain in the extreme layer of the tension steel \( \epsilon_t \), to have a minimum value equal to 0.004. Using similar triangles, the strain in the extreme steel layer, \( \epsilon_t \), is obtained from the strain diagram:

\[
\epsilon_t = 0.003 \frac{d - c}{c} \tag{3}
\]
where:

d_t = the distance from the bottom longitudinal reinforcement layer to the extreme compressive fibers of the concrete (mm).

This equation does not account for unusual types of reinforcement like high strength steel, such as unstretched prestressing strands. Since steel strands do not have a distinct yield point, a new minimum limit on \( \epsilon_t \) is needed to ensure enough ductility in strands reinforced concrete beams in flexure that is equivalent to conventionally reinforced members. Also, To ensure certain moment capacity prior to crack initiation in concrete sections, ACI 318-14 [3] imposes a lower limit on the amount of reinforcement which is:

\[
\rho_{\text{min}} = \frac{\sqrt{f'_{c}}}{4 f_y} \leq \frac{1.4}{f_y}
\]  

(4)

The nominal axial capacity of concrete members under ideal concentric loading is determined using the equation provided by ACI 318-14 code [3]:

\[
P'_{n} = 0.85 f'_{c} \left( A_g - A_{st} \right) + f_y A_{st}
\]  

(5)

where:

\( A_g = \) gross area of the column (mm\(^2\))

\( A_{st} = \) Total longitudinal reinforcement area (mm\(^2\))

To account for minimum accidental eccentricity of the applied load, the above equation is reduced by 20 % for tied columns. Therefore,

\[
P_n = 0.8 P'_{n}
\]  

(6)

Under pure axial compression, the longitudinal reinforcement plays a role where yield strength \( f_y \) is used to determine the contribution of steel in compression. While there is no distinct yield point of unstretched reinforced sections, the axial nominal capacity of columns needs to be reevaluated when strands are used instead of conventional reinforcement. Also, strands loaded axially may fail in a different manner to rebars because the buckling strength of strands between the ties at high loads is lower than corresponding rebars following spalling of the concrete cover.
To ensure adequate ductility in a concrete section subjected to axial compression, ACI 318 [3] provides a limit for the maximum reinforcement in columns as follows:

\[ A_{st(max)} = 0.08 A_g \] (7)

This ratio needs to be reevaluated, since the yield strength of strands is more than three times higher than the yield strength of conventional steel. The AASHTO’s LRFD Bridge Design Specification [4] provides maximum steel percentage equation in columns that takes into account the added capacity due to pre-stressing strands:

\[ \frac{A_s}{A_g} + \frac{A_{ps}f_{pu}}{A_gf_y} \leq 0.08 \] (8)

where:

\[ A_{ps} = \text{The longitudinal strands reinforcement area (mm}^2) \]

\[ f_{pu} = \text{Ultimate tensile strength of the strands (N - mm}^2) \]

This expression will provide guidance on how to convert the maximum mild steel reinforcement limit in columns to high strength steel.

1.4 Objectives of the Study

The objectives of the study are to:

- Investigate experimentally the flexural and axial compression behaviors of concrete members reinforced with unstretched prestressing steel strands.
- Compare the bending moment and axial compression behaviors of members reinforced with unstretched strands with corresponding members conventionally reinforced with rebars.
- Study the cracking pattern, strength, and ductility in strand reinforced concrete members subjected to bending moment, shear and axial load, and compare the behavior to that of conventionally reinforced members.
- Perform deformation capacity assessment on high strength strands in concrete members using curvature and displacement ductility evaluation with consideration of a wide range of reinforcement ratios, compressive strengths and loading conditions.
1.5 Scope of the Study

The scope of this study includes investigating the bending moment in beams as well as the axial compression capacity in non-slender columns that are reinforced with unstretched high strength seven-wire strands. This study involves experimental testing and numerical studies on a number of flexural beams and axially loaded columns in compression. The experimental study involves testing of 2200 mm simply supported flexural beams with a 200 mm by 300 mm rectangular cross-section. It also includes testing 1700 mm long simply supported beams in different loading conditions resulting in combined flexure and shear. Further, 1000 mm long columns with a 200 mm square cross section are tested under concentric loading conditions. The concrete specimens are tested in the AUS structural laboratory by a Universal Test machine under displacement controlled conditions. For each type of experimental testing, equivalent specimen of conventionally reinforced section are prepared, fabricated and tested as well. Two percentages of steel reinforcement are used in the beam tests, equal to 0.773% and 1.377% in the conventionally reinforced beams and 0.212% and 0.380% in the unstretched strands reinforced ones. Also, two different longitudinal steel percentages are utilized in columns, (2.01% and 4.02% for conventionally reinforced columns and 0.55% and 1.10% for strands reinforced members). Two concrete compressive strengths are considered in the bending moment experimental testing part, (40 and 60 MPa). The shear span to depth ratio for the beams that are subjected to both flexure and shear is equal to 1.0. The experimental program in this study involves instrumentation in the form of strain gauges and Linear Variable Differential Transformers (LVDT). The strain gauges will be installed on the longitudinal steel in beams and columns to detect the strain changes with loading. The LVDTs will be attached in the short beams diagonally across the expected diagonal cracks in the region of maximum shear to detect the average crack width. Two LVDT’s are placed on two opposite faces of axially loaded columns to monitor axial strain during the tests. In the ductility assessment part, the deformation capacity of beams loaded with the two loading conditions is measured by conducting curvature and displacement ductility analyses in order to model the behavior of the steel strands ductility at different strain limits. This is used to investigate the minimum reinforcement strain limits in beams and maximum reinforcement in columns, and propose appropriate strength reduction factors.
Practical design recommendations and guidelines in line with the ACI 318 code are proposed in this study.

1.6 Thesis Framework

The thesis is organized into seven chapters. Chapter 1 introduces the subject, explains the problem, specify the objectives of the study and set the work plan of the thesis. It provides information about the high strength strands, their uses, their mechanical performance, ductility challenges and suggests a suitable methodology. Chapter 2 gives background about the past published literature performed in this area of study. It provides information on high strength strands properties, structural performance and historical events. Chapter 3 demonstrates the experimental program, which includes the test setup, material properties, samples preparation, casting procedures and testing details. Chapter 4 presents the experimental results in terms of load-deflection relationships and load-strain graphs as well as observations related to the failure modes, load deflection behavior, and stress-strain remarks. Chapter 5 provides the deformation capacity assessment using curvature ductility and displacement ductility analyses and correlations between them. In Chapter 6, the results are discussed, analyzed and compared with their theoretical counterparts using different approaches. It includes calculations for the considered members regarding stiffness at service load levels, cracking moment, curvature and displacement ductility, ultimate capacity and reinforcement efficiency for both reinforcements using experimental and theoretical approaches. Chapter 6 also includes design recommendations for flexural and axially loaded members in code format. In Chapter 7, the research work is evaluated and summarized. Also, the main remarks and conclusions for the theoretical and experimental parts of the study are added.
Chapter 2: Background and Literature Review

This chapter provides background information on high strength steel strands, as well as past studies conducted on reinforced concrete members utilizing high strength steel and high strength strands. Also, included are the literature findings with consideration of the current study.

2.1 Background

High strength steel strands are type of reinforcement that comes in the form of steel wires that are twisted laterally on each other to provide superior mechanical performance in tension. Those strands are usually used in prestressing applications in beams and slabs to give an advantage in terms of the higher cracking loads, improved flexural capacity, reduced cross section dimensions, and lower deflection at service load levels. Prestressing strands are manufactured as stress-relieved or low-relaxation. The low relaxation seven wires steel strands are mainly manufactured in three sizes: 9.5, 12.7 and 15.2 mm with an ultimate strength of 1725 MPa for ASTM Grade 250 and 1860 MPa for ASTM Grade 270. 1860 MPa strands strength is based on ASTM A416 specification, as shown in Table 1. The stress-strain relationship of such strands differs from that of conventional mild steel in the sense it does not have a well-defined yield point, as shown in Figure 4.

Figure 4: Stress-strain curve for different grades of high strength steel.
Table 1: ASTM A416 Grade 1860 MPa.

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Tensile strength (MPa)</th>
<th>Cross sectional area (mm²)</th>
<th>Mass per meter (g/m)</th>
<th>Minimum breaking strength (KN)</th>
<th>Minimum Load at 1% Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.53</td>
<td>1860</td>
<td>54.84</td>
<td>432</td>
<td>102.3</td>
<td>92.1</td>
</tr>
<tr>
<td>12.7</td>
<td>1860</td>
<td>98.71</td>
<td>775</td>
<td>183.7</td>
<td>165.3</td>
</tr>
<tr>
<td>15.2</td>
<td>1860</td>
<td>140</td>
<td>1102</td>
<td>260.7</td>
<td>234.6</td>
</tr>
</tbody>
</table>

ACI 318-14 suggests the use of 0.2% offset method to determine the yield strength of the conventional mild steel by plotting a straight-line with an offset of 0.002 parallel to the stress strain line within the elastic region, where the intersection with curve gives the yield strength of the steel [3]. However, this method cannot be used in the case of high strength steel because there is no defined yield plateau on its stress-strain curve. NCHRP project suggests two different approaches to capture the yield strength in HSS bars. The first approach is the 0.35 % extension under load (EUL) method and the second approach is the 0.5 EUL method. Figure 5 below explains how the yield stress is captured by the three methods [5].

![Figure 5: HSS yield strength evaluation methods][15]

The lack of well-defined yield stress in high strength steel makes it difficult for designers to predict the flexural capacity of reinforced concrete sections at ultimate using the strain compatibility approach. The ACI 318-14 code [3] utilizes an equivalent stress block for simulating compression in the concrete at ultimate and an elastic-plastic model for the stress-strain behavior of mild steel. For high strength steel, this is not possible due to the distinct shape of the stress-strain relationship of the material. If experimental testing is conducted on a strand, then the yield strength

---

[15]: Image reference for Figure 5.
can be assumed to be the stress at a strain equal to 1%, as shown in Figure 6. Design codes suggest that the yield strength is about 85% of the ultimate strength for stress-relieved steel and 90% of the ultimate strength for low-relaxation steel.

![Stress-strain curve](image)

**Figure 6: Low relation strands Grade 275 stress-strain curve [4].**

The use of high strength steel in flexural application involving conventionally reinforced concrete was not reported by the ACI 318 and AASHTO codes until recently. Before 2012, the upper limit for the steel yield strength in flexural reinforcement was 515 MPa. The National Cooperative Highway Research Program (NCHRP) sponsored a project in 2007 on the use of high strength steel in structural applications addressing the challenges related to the steel strain limits, cracking pattern and serviceability limits [5]. Based on the work of Shahrooz et al. [6], new HSS provisions were added to the AASHTO’s bridge design specifications to allow the use of steel with a yield strength up to 690 MPa in flexural applications. However, the provisions do not cover high strength strands, as they were not investigated at that time. Under the new provisions, AASHTO (2013) recommends adoption of an elastic-perfectly plastic analysis for high strength steel rebars, considering the stress value corresponding to a strain equal to 0.35% or 0.5% [4]. The use of high strength steel rebars using a stress corresponding to 0.35% was restricted under two conditions, which are a steel reinforcement ratio exceeding 2.6% and concrete compressive strength larger than 69 MPa, to ensure conservative prediction of the design moment capacity [4].
In order to evaluate the use of steel with a yield strength exceeding 689 MPa, Baran and Arsava (2012) conducted a study on high strength steel strands employed in concrete beams using experimental and analytical approaches [2]. The use of elastic-perfectly plastic steel model was shown to be conservative in flexural capacity prediction, where the non-linear actual stress strain model resulted in higher moment capacity when steel reinforcement ratios exceeded 0.35%. It was concluded that the use of high strength steel strands is applicable with an adjusted formula related to the maximum reinforcement ratio as a function of the reinforcement ratio at balanced conditions, based on the ACI 318 code prior to 2002. This was done using the so called “displacement deformability factor”, by equating the displacement deformability of the high strength strands with the displacement deformability of the mild steel and determining the corresponding reinforcement ratio for a wide range of different concrete sections [2]. The formulation predicted by Baran and Arsava (2012) does not cover the new provisions of the ACI 318 code after 2002 regarding the steel strain limits in flexural design applications. Their limited study does not cover the use of strands in combined flexure and shear or axial compression. Also, it does not provide information about the use of strands in a wide range of concrete strengths. This leaves a wide space for the researchers to dig into the subject to understand and evaluate the use of unstretched steel strands in different design applications.

2.2 Literature Review

In this section, background on past studies related to flexural, shear and axial load behavior of concrete members reinforced with unusual steel reinforcement will be discussed and analyzed. In recent years, there have been studies conducted to improve the reinforcement behavior in flexural, shear, and axial members using different kinds of reinforcement.

In this context, a study by Dawood et al. in 2004 was carried out to analyze the flexural behavior of concrete beams reinforced with Multi Criteria Formable Reinforcing Steel (MMFX) rebars [7]. This type of steel reinforcement was used because it is two times stronger than the conventional reinforcing steel rebars. Flexural tests have been implemented for several beams to predict the load deformation relationships in related members. A ductility numerical assessment was done to model the behavior of concrete beams reinforced with MMFX rebars. MMFX
section ductility was determined using cracked section analysis dividing the beam cross section into many layers to predict the strain profile at different stress conditions. For a given reinforcement ratio and concrete compressive strength the strain at the extreme compression fibers as well as depth of equilibrium were determined at yield and ultimate states. Design guidelines for the flexural design of MMFX reinforced beams were predicted using the proposed material model. Resistance factors were determined to be as 0.65 for compression controlled sections with a strain limit of 0.006 and 0.9 for tension controlled sections with a strain limit of 0.015. A design chart was developed in this study to determine the moment capacity at different steel ratio and compressive strength values as shown in Figure 7:

![Design chart for MMFX reinforced flexural members](image)

**Figure 7:** Design chart for MMFX reinforced flexural members [7].

Another research was done to improve the flexural behavior in beams using high strength steel reinforcement. In this study, a moment curvature-analysis for eight beams was completed by Mast et. al. (2009) [8] to provide design guidelines for the acceptable strain limits in such members. This was done to predict the resistance factors for different members under compression and tension-controlled sections depending on the strain limits. Three of the beams were designed in this study according to the ACI 318 code using conventional steel reinforcement (f_y = 420 MPa) with a concrete compressive strength of 34 MPa. The other five beams were designed using high strength steel reinforcement with a concrete compressive strength in the range between 28 and 69 MPA. The experimental results showed that for an acceptable performance, a resistance factor of 0.65 is needed in members with
reinforcement strain limits less than or equal to 0.004. However, a resistance factor of 0.9 is reasonable when the reinforcement strain limit is greater than or equal to 0.009. For the beams with transition sections, a linear interpolation is used to get the approximated resistance factors. A simplified model for the flexural design of high strength steel was predicted by Florida Department of Transportation (DOT) and University of North Florida (UNF), considering a plastic perfectly plastic stress-strain relationship with an elastic modulus of 200 GPa followed with a yielding plateau at the stress 690 MPa. The simplified design was found to be conservative in nominal moment capacity prediction process when the reinforcement ratio is less than 2% as shown in Figure 8:

![Figure 8: Florida DOT design chart for high strength steel in flexure [8].](image)

Baran and Arsava (2012) proposed a new idea regarding using the prestressing strands as a regular reinforcement in concrete beams [2]. This was done to provide superior performance in concrete sections, since the beams reinforced with strands have much higher load carrying capacity than the ones reinforced with conventional rebars. In the study, thirteen concrete beams were prepared and tested for flexure, seven by strands and six by conventional rebars. The load-deflection relationships for the tested beams were plotted for both cases as shown in Figure 9:
Figure 9: Load deflection behavior in conventional and strand reinforcement [2].

The ductility was evaluated in both types of reinforcement by using the displacement deformability ratio, which is the ratio of ultimate to service deflection, where the strain at the compression zone is determined using the strain compatibility behavior. Moment capacity assessment was done on the high strength strands reinforced sections using two different approaches. One approach using sectional analysis with the actual nonlinear material model and another approach using the procedures outlined in ACI-318 code assuming elastic perfectly plastic material stress strain behavior with an elastic modulus of 200 GPa and yield strength of 1700 MPa. The moment capacity assessment results showed that for the same reinforcement ratio, strands reinforced beams provide greater carrying capacity but slightly smaller service stiffness than the conventionally reinforced counterparts. A numerical analysis was done in this study to determine the deformation capacity of the beams to put some reinforcement limits for the high strength steel strands to ensure enough ductility in such cross-sections [2].

A recent study was implemented by Shahrooz et al. in 2014 to investigate the applicability of using steel rebars of more than 552 MPA in yield strength which is the maximum reinforcement limit specified by the ACI 318 (ACI 2011) code [6]. This was examined in flexural beams to improve the flexural capacity, reduce material quantity and avoid reinforcement congestion in concrete sections. ASTM A1035 Grade 689 steel has been used because it has no discernible yield plateau in addition to its very high strength under flexure. In the experimental program of this study, six beams of 6.1 m simple spans under four points of loading were designed,
prepared and tested. Crack width was recorded in the region of constant moment and the service load stress was adjusted over the LRFD AASHTO specifications to be less than 0.6 the yield strength to limit the crack width in concrete [6].

![Graph of Flexural Strength Reduction Factors](image)

Figure 10: Flexural strength reduction factors for 414 and 689 steel grades [6].

A new revisions to LRFD AASHTO 2013 [4] specifications regarding the resistance factors were investigated for high strength steel reinforced sections. A comparison graph for the reduction factors of steel grade 414 and grade 689 is shown in Figure 10. It was recommended that the reinforcement strain limit must be changed to ensure enough curvature ductility compared to conventional Grade 414 steel. The strain limit was recommended to increase to 0.004 for compression controlled sections and 0.008 for tension controlled sections.

In 2011, Lee et al. suggested using high strength steel stirrups in concrete beams instead of using conventional stirrups for better shear behavior [9]. Experimental testing for the cracking behavior was needed in this case, since ACI 318-08 code does not allow using shear reinforcement with yielding strength of more than 420 MPA to limit the cracking width in beams. Experimental testing was directed toward evaluating the shear behavior in beams with high strength shear reinforcement in a multiple concrete compressive strength beams. This was done to investigate the effect of the stirrups yielding strength and concrete compressive strength on shear capacity. Thirty-two beams were fabricated and tested for shear behavior. All the beams failed in shear after yielding of the stirrups, even though, the beams were designed to have greater stirrups yield strength than what is required by
ACI 318 code [9]. Figure 11 illustrates the strain rates for stirrups in different concrete compressive strength:

![Figure 11: Strain rates for different concrete compressive strengths [9].](image)

It was concluded that the yield strength of shear reinforcement does not affect the cracking width in beams at all. This was realized since the experimental and analytical results of diagonal crack width were almost constant at different load levels regardless of the stirrups yield strength in beams. It was shown also, that the maximum stirrups yield strength in concrete beams of more than 60 MPA of compressive strength must be reevaluated again in the ACI 318 code provisions.

Ward and Shahrooz in 2009 evaluated the performance of multiple concrete members with high strength steel reinforcement [10]. The study aimed to determine the tension and compression controlled steel strain limits for high strength reinforced beams as well as investigating the related flexural and shear cracking behavior. Also, it intended to evaluate the columns capacity using high strength steel confinement by conducting an experimental and analytical moment axial relationships. It was proposed that analyzing stress strain diagram of high strength steel reinforcement as elastic-perfectly plastic model does not provide good estimates in terms of flexural capacities. Curvature ductility analysis was done for both, the conventional steel and the high strength steel proposed in this study, which was determined by dividing the ultimate curvature with the curvature at service at different steel ratios. The strain limits of high strength reinforcement ranges between A615 to A1035 were adjusted to be 0.004 for compression controlled sections and 0.008 for tension controlled ones. The experimental results showed that that the diagonal crack stiffness gets lower when the spacing between high strength stirrups of A1035 becomes larger [10].
Moreover, from a flexural point of view, it has been noticed that the high strength steel reinforcement results in lower curvature ductility and lesser initial stiffness than the conventional steel reinforcement A615 as shown in Figure 12.

![Figure 12: Moment-curvature diagram for A615 and A1035](image)

For columns confinement, the study also ascertained that ACI 318 code provisions for columns spiral spacing needs to be adjusted in high strength confined columns for better design behavior. It was concluded at the end that the deflection requirements and fatigue behavior need to be investigated and reevaluated again in high strength steel reinforced sections [10].

Another comprehensive study on the use of high strength steel as a flexural reinforcement in structural members was done by NCHRP project [5]. In this study, analytical assessment was conducted to evaluate the steel strain limits for the high strength steel that will match the same ductility at the strain limits specified for conventional steel in the ACI-318 code. In order to estimate the curvature ductility of the beams, a sectional analysis was performed using the strain compatibility and equilibrium principles considering the actual stress strain relationship of the high strength steel bars using Ramberg-Osgood (R-O) equation. The curvature ductility was determined as the ratio of ultimate to service curvature, where the curvature at service is at 0.6 $f_y$, Where $f_y$ is the yield stress specified by the steel stress strain model in both reinforcements. The steel strain limits for high strength steel bars were predicted using the strain-curvature ductility relationship predicted in Figure 13. NCHRP Project considered different parameters in their study such as the concrete...
compressive strength, tension longitudinal reinforcement and compression longitudinal reinforcement. For each of the cases, the steel strain in ASTM A1035 Grade 689 steel was determined at the same curvature ductility of ASTM A615 Grade 414 steel at the steel strain limits of 0.002 and 0.005. Based on the afore-mentioned analytical assessment, the steel strain was adjusted to 0.008 and 0.004 for the tension and compression controlled limits in high strength steel reinforcement [5].

![Curvature Ductility vs Steel Strain Graph]

*Figure 13: Strain-ductility relationships for A 615 and A 1035 grades steel* [5].

In this project, the flexural crack width was assessed in concrete girders with reinforcing bars with a yield stresses of 248, 414 and 496 MPa. The crack width was compared with the AASHTO de facto limits of class 1 and class 2 exposures based on the assumption that the service loads do not exceed $0.6 f_y$. All the measured crack width met the AASHTO de facto limits for both exposure classes, which indicates the conservativeness of ACI and AASHTO in flexural crack width estimation, which can work for both; the conventional and high strength steel [5].
Chapter 3: Experimental Program

This chapter introduces the experimental program of the research study, which includes work plan, samples dimensions, equipment used, testing setup, and material properties of the specimens.

3.1 Laboratory Work

In this study laboratory work is needed to understand the behavior of strands reinforced members in different structural applications. The theoretical work that will be introduced later was calibrated based on the experimental results. To get good results and develop proper design recommendations, a well-planned experimental program must be set. Since the use of un-stretched pre-stressing steel strands in concrete is not common today in the construction field and published studies in this field are very limited, it was important to generalize the experimental program to consider for flexure, shear, and axial with emphasis on strength and ductility of such an application.

The experimental work included fourteen half-scale beams and four columns which were prepared, fabricated and tested per the study needs. In this context, eight 2200 mm long beams of 200 by 300 mm cross section were tested in flexure on simple a span of 1900 mm. Six 1700 mm long beams of the same cross section were tested in shear on a simple span of 1240 mm. Moreover, four 1000 mm tall columns of 200 mm square cross section were tested in concentric compression. A pair of similar specimens was used when investigating a chosen variable, one specimen with conventional rebar reinforcement and another with un-stretched steel strands. A summary of the framework of the experimental program is illustrated in Figure 14. In Table 2, details of the lateral steel reinforcement within the instrumented region are provided. The number and size of conventional reinforcement and the corresponding strand reinforcement for each pair of specimens was carefully selected in accordance with the experimental program objective. That is to achieve approximately the same tensile force in the reinforcement in beams and the same compressive force in the reinforcement in columns at ultimate. By doing so, the strength and ductility, in terms of the load-deformation behavior, can be objectively compared and judged.
Figure 14: Experimental program framework.
Table 2: Experimental program labeling details summary

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Designation</th>
<th>b x h (mm x mm)</th>
<th>Longitudinal Reinforcement</th>
<th>Stirrups/Ties within Instrumented Region</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure</td>
<td>F-RL-40</td>
<td>200x300</td>
<td>2 No. 16</td>
<td>No stirrups</td>
</tr>
<tr>
<td></td>
<td>F-SL-40</td>
<td></td>
<td>2 No. 9.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-RH-40</td>
<td></td>
<td>2 No. 16 + 1 No. 20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-SH-40</td>
<td></td>
<td>2 No. 12.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-RL-60</td>
<td></td>
<td>2 No. 16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-SL-60</td>
<td></td>
<td>2 No. 9.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-RH-60</td>
<td></td>
<td>2 No. 16 + 1 No. 20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-SH-60</td>
<td></td>
<td>2 No. 12.7</td>
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</tr>
<tr>
<td>Combined Flexure-Shear</td>
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<td>200x300</td>
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<td>No stirrups</td>
</tr>
<tr>
<td></td>
<td>S-SL-NST</td>
<td></td>
<td>2 No. 9.5</td>
<td>No stirrups</td>
</tr>
<tr>
<td></td>
<td>S-RL-100</td>
<td></td>
<td>2 No. 16</td>
<td>No. 8 @ 100 mm</td>
</tr>
<tr>
<td></td>
<td>S-SL-100</td>
<td></td>
<td>2 No. 9.5</td>
<td>No. 8 @ 100 mm</td>
</tr>
<tr>
<td></td>
<td>S-RH-NST</td>
<td></td>
<td>2 No. 16 + 1 No. 20</td>
<td>No stirrups</td>
</tr>
<tr>
<td></td>
<td>S-SH-NST</td>
<td></td>
<td>2 No. 12.7</td>
<td>No stirrups</td>
</tr>
<tr>
<td>Axial Compression</td>
<td>C-RL-100</td>
<td>200x200</td>
<td>4 No. 16</td>
<td>No. 8 @ 100 mm</td>
</tr>
<tr>
<td></td>
<td>C-SL-100</td>
<td></td>
<td>4 No. 9.5</td>
<td>No. 8 @ 100 mm</td>
</tr>
<tr>
<td></td>
<td>C-RH-100</td>
<td></td>
<td>8 No. 16</td>
<td>No. 8 @ 100 mm</td>
</tr>
<tr>
<td></td>
<td>C-SH-100</td>
<td></td>
<td>8 No. 9.5</td>
<td>No. 8 @ 100 mm</td>
</tr>
</tbody>
</table>

3.2 Samples Steel Detailing

The eight beams in the first group were tested in pure flexure under a pair of loads near mid-span. They had four different reinforcement types, as summarized in Table 2. In the conventionally reinforced specimens, two No. 16 bars (ρ= 0.773%) and two No. 16 bars and one No. 20 bar (ρ=1.377%) at the bottom were used. On the other hand, in strands reinforced specimens, two 9.5 mm diameter strands were used at the bottom (ρ=0.212%) and two 12.7 mm diameter strands (ρ=0.380%) were utilized at the bottom. Two No. 10 bars was used as a top reinforcement in all regions of the specimens where stirrups are used. The specimens were considered one time with 40 MPA concrete and another time with 60 MPA concrete, with the same reinforcement detailing. The beams included rectilinear stirrups made from No. 8 bars.
and placed at 100 mm spacing within a distance of 800 mm from both ends of the beams. The middle 600 mm region was free from lateral reinforcement in order to study flexure due to longitudinal bottom reinforcement without considering the effect of the stirrups, since the applied loading consisted of two loads at 650 mm from the two supports. The purpose of the stirrups that were provided outside the middle region was to protect the beams from any premature shear failure. Drawings for the reinforcement detailing of a sample beam F-RL-40 are shown in the Figure 15.

![Figure 15: Structural drawings for the flexural sample specimen F-CLR-40.](image)

The six combined flexure-shear beams were reinforced with conventional rebars and strands such that the effective steel reinforcement ratios were the same as in the flexure beams. They were subjected to a single point load near the support resulting in a span-to-depth ratio equal to 1.0. Stirrups of size No. 8 bars were used in the beams at 100 mm spacing in the low shear region to protect the beams from any premature failure in the un-instrumented region. Within the region between the near support and applied load in which the shear is most critical, some specimens contained stirrups whereas others were lacking. Structural drawings showing the reinforcement detailing of a sample beam S-RL-NST are presented in the Figure 16.

![Figure 16: Structural drawings for the shear sample specimen S-CLR-NST.](image)

For the short column specimens, two different steel reinforcements were considered. In the conventionally reinforced specimens, four No. 16 bars were used in
one specimen and eight No. 16 bars were employed in the other, resulting in gross reinforcement ratios of 2.01% and 4.02%, respectively. On the other hand, in strands reinforced specimens, four 9.5 mm diameter strands were used in one specimen and eight 9.5 mm diameter strands were utilized in the other, resulting in gross reinforcement ratios of 0.55% and 1.1%, respectively. Ties No. 8 bars were used at 100 mm spacing in the central region in addition to 150 mm steel jacketing in the top and bottom ends in order to prevent premature failure due to concentration of stresses at the supports. Structural drawings for the reinforcement detailing of a sample column C-RL-100 are shown in the Figure 17.

![Figure 17: Structural drawings for the axial sample specimen C-RL-100.](image)

### 3.3 Test Setup and Instrumentations

The 18 specimens were tested in the Construction Materials and Structures laboratory at AUS inside the 2500 kN Instron Universal Test Machine (UTM). The eight flexural beams were subjected to two points of load that are 700 mm apart from each other in the central region of the beams. Those points were subjected to displacement controlled loading condition at a rate of 1 mm per minute. The six flexural-shear specimens were loaded at one point at a distance of 300 mm from one support under a displacement controlled rate of 0.5 mm. The four short columns were tested vertically under a concentric displacement controlled environment at a rate of 0.3 mm per minute.

The experimental program in this study involved instrumentation in the form of strain gauges and Linear Variable Differential Transformers (LVDTs). Two 60 mm strain gauges were installed on the concrete surface on compression side of the long and short beams to detect the concrete strain with loading. Moreover, two 10 mm strain gauges were installed on the rebars in the beams at the bottom to measure
tensile strains in the longitudinal steel reinforcement. Strain in the longitudinal steel rebars in the columns were detected with the help of 10 mm strain gauges installed on two diagonally opposite bars within the instrumented region to measure the compression carried by the steel. An LVDT was attached in flexure-shear beams diagonally across the 45° expected crack in the region of maximum shear to detect the average crack width with loading. Another LVDT was attached to the soffit of the long beams at midspan and to the soffit of the short beams under the point load to record the deflection in the vertical direction. Two LVDT’s were placed vertically on opposite faces of the column specimens to measure the average displacement between two specified points on the columns with the increase in loading. The test setup and instrumentations for the three parts of the experimental program are demonstrated in Figures 18, 19, and 20.

![Figure 18: Test setup and installation for moment sample beams.](image)

![Figure 19: Test setup and installation for shear sample beams.](image)
The test outcomes were captured at the specified load increment on each of the tests. The results include the applied load, actuator extension, vertical deflection and concrete and steel strains. Relationships from the obtained data in the form of: (1) load-deflection graphs for beams and columns, (2) moment-curvature curves for beams, (3) load-strain in longitudinal steel relationships for beams and columns, and (4) load-strain plots in concrete for beams. These relationships were used in the analysis part to determine the strength, ductility, stiffness, failure mode and residual strength beyond ultimate capacity. The test results were studied, analyzed and used to develop practical recommendations.

3.4 Concrete Properties

Two different concrete mixes were prepared and casted at the same time for all the specimens by a concrete precast company located in Ras Al-Khaimah, UAE. ASTM Ordinary Portland Cement was used in the concrete mixes. Two types of natural coarse aggregate were utilized in the mixes, which include sizes of 10 mm and the 20 mm. Also, two types of fine aggregate were employed, which are dune sand and washed sand. DERASIM water-reducing admixture (ASTM Type A) was used in the 40 MPa target mix, while ADVA high-range water-reducing admixture (ASTM Type F) was used in 60 MPa target mix. Further, the weight ratio of 10mm to 20mm coarse aggregate in the 40 MPa mix was 40:60. The corresponding weight ratio in the 60 MPa mix was 44:56. The water/cement ratio was 38.5% and 34% in the 40 MPa
and 60 MPa mixes, respectively. More details on the properties of the aggregates are shown in Tables 3 and 4 below. Also, the mix design details are shown in Table 5.

Table 3: Fineness modulus of aggregate.

<table>
<thead>
<tr>
<th></th>
<th>Dune Sand</th>
<th>Washed Sand</th>
<th>Combined Dune &amp; Washed Sand</th>
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</thead>
<tbody>
<tr>
<td>F.M</td>
<td>2.65</td>
<td>3.65</td>
<td>3.15</td>
</tr>
</tbody>
</table>

Table 4: Coarse aggregate properties.

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Size (mm)</th>
<th>Bulk S.G</th>
<th>Apparent S.G</th>
<th>Absorption (%)</th>
<th>Moisture (%)</th>
<th>Abrasion Loss (%)</th>
<th>Crushing Value (%)</th>
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<td>Coarse</td>
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<td>2.69</td>
<td>2.72</td>
<td>0.6</td>
<td>0.37</td>
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<td>19.0</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>2.68</td>
<td>2.71</td>
<td>0.6</td>
<td>0.37</td>
<td>25.0</td>
<td>22.0</td>
</tr>
</tbody>
</table>

Table 5: Concrete mix design details.

<table>
<thead>
<tr>
<th>Target Strength (MPa)</th>
<th>Coarse Aggregate (Kg/m³)</th>
<th>Fine Aggregate (Kg/m³)</th>
<th>Cement (Kg/m³)</th>
<th>Water (Kg/m³)</th>
<th>Admixture (Kg/m³)</th>
<th>Total weight (Kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 mm</td>
<td>360</td>
<td>680</td>
<td>240</td>
<td>630</td>
<td>400</td>
<td>154</td>
</tr>
<tr>
<td>20 mm</td>
<td>390</td>
<td>630</td>
<td>250</td>
<td>610</td>
<td>440</td>
<td>150</td>
</tr>
</tbody>
</table>

For each concrete mix, three cylinders (150mm by 300mm) and six cubes (150mm) were prepared and casted at the day of testing as shown in Figure 21.

Figure 21: Preparation and casting of the cubes and cylinders.

The concrete cube and cylinder specimens were tested in compression at the age of 7 days, 14 days and also at the day of the large beam/column tests. Typical failures modes of the concrete cube and cylinder are presented in Figure 22.
The stress strain curve for one typical sample is plotted as in Figure 23:

Figure 23: Typical stress strain diagram of a cylinder.

Table 6 demonstrates the compressive strength results for all the samples at 7, 14 days and at the day of testing. Figure 24 shows the strength gain with time for the different strength concretes.

Figure 24: Average concrete strength gain with time for both.
Table 6: Compressive strength results of the cubes and cylinders.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strength at 3 days (MPa)</th>
<th>Strength at 7 days (MPa)</th>
<th>Strength at testing day (MPa)</th>
<th>Avg. Cyl/Cube ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 MPa samples</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cubes</td>
<td>31.96</td>
<td>36.57</td>
<td>53.52</td>
<td>87 %</td>
</tr>
<tr>
<td></td>
<td>32.41</td>
<td>35.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>31.52</td>
<td>37.75</td>
<td>51.89</td>
<td></td>
</tr>
<tr>
<td></td>
<td>31.34</td>
<td>36.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cylinders</td>
<td>-</td>
<td>-</td>
<td>44.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-</td>
<td>46.6</td>
<td></td>
</tr>
<tr>
<td>60 MPa samples</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cubes</td>
<td>48.75</td>
<td>52.57</td>
<td>71.73</td>
<td>91 %</td>
</tr>
<tr>
<td></td>
<td>49.24</td>
<td>52.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>47.58</td>
<td>51.86</td>
<td>71.53</td>
<td></td>
</tr>
<tr>
<td></td>
<td>49.24</td>
<td>56.22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cylinders</td>
<td>-</td>
<td>-</td>
<td>66.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-</td>
<td>64.3</td>
<td></td>
</tr>
</tbody>
</table>

3.5 Steel Properties

Different steel reinforcement bar sizes were used in the specimens, including 8mm, 10mm, 16mm, and 20mm. Further, seven wires prestressing strands of 9.53mm and 12.7mm diameters were utilized. Steel samples were tested in tension to detect the actual elastic modulus, yielding strength and tensile capacity for the different steel sizes and types. Figure 25 shows the stress-strain plot for 11 different samples. The average yield strength and average elastic modulus were respectively 525 MPa and 199.7 GPa for the conventional steel bars, and 1670 MPa and 197.8 GPa for the strands, as shown in Figure 26. The yield strength of the prestressing strands was obtained as the stress corresponding to 1% strain.

3.6 Specimens Fabrication

The beam and columns specimens were prepared, fabricated and casted in the concrete precast company. After concrete has cured and hardened, all samples were transported to and tested in the AUS laboratory. Steel cages for the two types of steel reinforcement for the flexure beams, flexure-shear beams, and short columns are shown in Figures 27, 28 and 29.
Figure 25: Stress-strain curves for the steel specimens.

Figure 26: Average stress strain curves for rebars and strands.

Figure 27: Steel cages for both reinforcement in flexure specimens.
Figure 28: Steel cages for both reinforcement in combined loaded specimens.

Figure 29: Steel cages for both reinforcement in column specimens.

The 10 mm strain gauges were installed on the longitudinal steel rebars and strands in the beams and columns to measure the strain in the reinforcement during loading, as shown in Figure 30.

(a) Flexure specimen  (b) Flexure-shear specimen
(c) Axial compression specimen

Figure 30: strain gauges fixing at the instrumented regions.
Figure 31 shows installation of the strain gauges on the steel strands and bars after grinding.

![Figure 31: Installation process of the strain gauges.](image)

The concrete was prepared in the precast factory as per the mix design described earlier. The fresh concrete was tested for slump, density and temperature prior to casting in the formwork, as shown in Figure 32. The steel cages were fixed inside the formwork prior to concrete casting, as shown in Figure 33. Each formwork contained spaces for two similar beams. Concrete mixing was done in the batching facility and the concrete was transported to the casting place in a big truck for each of the considered mixes. The formwork was fixed above a mechanical table vibrator to ensure adequate compaction and avoid honeycombs in the finished specimens, as shown in Figure 34. Figure 35 shows concrete pouring into the formwork.

![Figure 32: Fresh concrete test; slump and temperature.](image)
Figure 33: Formworks preparation.

Figure 34: Mechanical vibration of the fresh concrete.

Figure 35: Concrete casting of the specimens.
Figure 36 shows the finished fresh concrete long beams, short beams, and columns just after concrete pouring and compaction.

![Figure 36: Finished fresh concrete samples.](image1)

The casted specimens were cured using a special chemical curing powder to accelerate the curing process without the need for continuous moisture. They were stored at an ambient temperature in the factory until the date of transporting them to the structural laboratory at AUS, as shown in Figure 37.

![Figure 37: Finished concrete specimens.](image2)

For the beams, the 60 mm strain gauges were later installed at the top of concrete at 3mm and 6mm on center from the extreme top fibers shortly before the day of testing. One LVDT was attached to the bottom of the flexure beams at
midspan and at the point of loadings for the flexure-shear beams to record the vertical deflection during the application of the load, as demonstrated in Figure 38. For the flexure-shear beam specimens, an additional LVDT is installed within the region of high shear as shown in Figure 39 to measure the average crack width within that region. For the columns, two LVDT’s were placed on opposite faces of the specimens to measure the average vertical displacement between two specified points within the instrumented region where the failure is expected, as shown in Figure 40.

Figure 38: Experimental setup of the flexural samples.

Figure 39: Experimental setup of the combined loaded samples.
Figure 40: Experimental setup of the column specimens.
Chapter 4: Experimental Results

The experimental work was divided as explained earlier into three main groups, which are intended to examine flexure, flexure combined with shear, and concentric axial compression. For each specimen that is reinforced with high strength steel strands, there is an equivalent specimen that is conventionally reinforced with steel rebars. This equivalency is related to the force at yield level carried by the longitudinal reinforcement (tension in the beams and compression in the columns). Different ratios were used to determine the longitudinal steel effect in each type of reinforcement on the structural response of the specimen. The recorded flexural and axial compression capacity and ductility in the beams and columns from the experiments are presented in this chapter. The experimental results in this chapter are compared later with the theoretical equations in the analysis part of the study. Table 7 shows summary of the tested specimens in the experimental program.

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Designation</th>
<th>Reinforcement Type</th>
<th>Longitudinal Reinforcement</th>
<th>Stirrups or Ties?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure (b=200mm, h=300 mm)</td>
<td>F-RL-40</td>
<td>Bars</td>
<td>2No.16</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>F-SL-40</td>
<td>Strands</td>
<td>2x9.5 diam.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-RH-40</td>
<td>Bars</td>
<td>2No.16 &amp; 1No.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-SH-40</td>
<td>Strands</td>
<td>2x12.7 diam.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-RL-60</td>
<td>Bars</td>
<td>2No.16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-SL-60</td>
<td>Strands</td>
<td>2x9.5 diam.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-RH-60</td>
<td>Bars</td>
<td>2No.16 &amp; 1No.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>F-SH-60</td>
<td>Strands</td>
<td>2x12.7 diam.</td>
<td></td>
</tr>
<tr>
<td>Combined Flexure-Shear (b=200mm, h=300 mm)</td>
<td>S-RL-NST</td>
<td>Bars</td>
<td>2No.16</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>S-SL-NST</td>
<td>Strands</td>
<td>2x9.5 diam.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S-RL-100</td>
<td>Bars</td>
<td>2No.16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S-SL-100</td>
<td>Strands</td>
<td>2x9.5 diam.</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>S-RH-NST</td>
<td>Bars</td>
<td>2No.16 &amp; 1No.20</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>S-SH-NST</td>
<td>Strands</td>
<td>2x12.7 diam.</td>
<td></td>
</tr>
<tr>
<td>Axial Compression (b=200mm, h=200 mm)</td>
<td>C-RL-100</td>
<td>Bars</td>
<td>4No.16</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>C-SL-100</td>
<td>Strands</td>
<td>4x9.5 diam.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C-RH-100</td>
<td>Bars</td>
<td>8No.16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C-SH-100</td>
<td>Strands</td>
<td>8x9.5 diam.</td>
<td></td>
</tr>
</tbody>
</table>
4.1 Load-Deflection Relationships

The 18 scaled models were subjected to displacement-controlled loading at a rate of 1.0, 0.5, and 0.3 mm per minute for the flexural beams, combined flexural-shear beams, and axial compression columns, respectively. The load was applied on each specimen using the test setup explained in the previous chapter. The applied load, deflection, strain in concrete and strain in tension steel reinforcement were logged at each increment of loading throughout the test. The load-deflection relationships were recorded for all specimens and are presented in Figures 41 to 58. Photos of each specimen are also included at pre-cracking, post cracking and failure stages within the figures. As expected, the specimen load-deflection curves showed reduction in stiffness in the post cracking region. Crack initiation was different for the various specimens, depending on the load effect, concrete compressive strength, amount of reinforcement, and type of reinforcement. For instance, cracks started to form earlier in the specimens that were subjected to pure flexure than those exposed to combined flexure-shear, and much earlier than those in pure axial compression. Further, the cracks started quicker in the specimens with lower reinforcement ratios and weak compressive strengths. All beams were failed in flexure even the ones with combined flexure-shear loading, except for one beam (S-RH-NST) which was highly reinforced with mild flexural reinforcement and no transverse reinforcement. The findings of the flexural ultimate capacity were consistent, where the samples with high reinforcement and strong compressive strength showed higher capacity, as expected. The columns specimens failed in compression where the post-peak capacity cracks formed randomly on the specimen surfaces along the height. The compression capacity results of the columns were consistent in all the samples with adequate ductility, irrespective of the type of reinforcement.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 41: Experimental details of specimen F-RL-40.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 42: Experimental details of specimen F-SL-40.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 43: Experimental details of specimen F-RH-40.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 44: Experimental details of specimen F-SH-40.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 45: Experimental details of specimen F-RL-60.
Figure 46: Experimental details of specimen F-SL-60.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 47: Experimental details of specimen F-RH-60.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 48: Experimental details of specimen F-SH-60.
Figure 49: Experimental details of specimen S-RL-NST.
(a) Load-deflection curve

(b) Pre-cracking stage

(c) Post-cracking stage

(d) At ultimate

Figure 50: Experimental details of specimen S-SL-NST.
Figure 51: Experimental details of specimen S-RL-100.
Figure 52: Experimental details of specimen S-SL-100.
Figure 53: Experimental details of specimen S-RH-NST.
Figure 54: Experimental details of specimen S-RH-NST.
Figure 55: Experimental details of specimen C-RL-100.
Figure 56: Experimental details of specimen C-SL-100.
Figure 57: Experimental details of specimen C-RH-100.
Figure 58: Experimental details of specimen C-SH-100.
4.2 Specimens Load-Carrying Capacity

The load-deflection relationships presented earlier are obtained directly from the UTM machine as the load exerted by the swivel head. For the flexural specimens, this load is split equally at two locations at 650 mm from each support, making the distance between the loads equal to 600 mm. On the other hand, the load applied by the machine on the combined flexure-shear specimens is concentrated at one point located at 300 mm from one of the two supports, resulting in a shear span-to-depth ratio equal to 1.0. In order to examine the flexural behavior of the considered specimens under different conditions, the loading must be converted to bending moment along the span. Figure 59 shows the shear force and bending moment diagrams of the long flexural beams as a function of the applied load. The corresponding diagrams for the combined flexure-shear loading on the short beams are presented in Figure 60. The extended length of the beam beyond the support reaction on the right side is due to limited space inside the UTM machine for the intended shear span-to-depth ratio. As for the specimens that were tested under concentric compression, the capacity of the columns was simply taken as the maximum recorded applied load by the machine, without any further analysis.

The maximum load that was exerted on the tested beams and columns was recorded by the UTM machine and shown in Table 8. As the failure mode in all beams (except for one specimen) was that of flexural failure, the moment capacity at ultimate in the beams was determined using the moment diagram in Figures 59 and 60, depending on the applied loading. Even for the single specimen that failed in shear, subsequent analysis demonstrated that the shear and flexural capacities were very close to each other. The first cracking load was observed during the tests in the beams and recorded in Table 8. Often the load-deflection relationships showed clear indication of the cracking load, by a sudden reduction in the stiffness, which matched the visual observations.

4.3 Strain Results

Strain results were obtained from both the concrete and the steel used in all the specimens. To understand and analyze the stress and corresponding strain behavior as well as the failure mode of the tested beams, it was important to install strain gauges at different locations along the specimens. Two 10 mm strain gauges were installed
internally on the longitudinal steel within the critical instrumented region. In addition, another two 60 mm strain gauges were installed externally on the surface of concrete in compression, one at 3 mm and another at 6 mm from top, for the beam specimens.

Figure 59: Shear and Moment diagram for flexure specimens.

Figure 60: Shear and Moment diagram for flexure-shear samples.
Table 8: Summary of the experimental results.

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Designation</th>
<th>Maximum load (KN)</th>
<th>Maximum Moment (KN-m)</th>
<th>Cracking Load (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure</td>
<td>F-RL40</td>
<td>163.39</td>
<td>53.10</td>
<td>42.5</td>
</tr>
<tr>
<td></td>
<td>F-SL40</td>
<td>153.06</td>
<td>49.75</td>
<td>35.7</td>
</tr>
<tr>
<td></td>
<td>F-RH40</td>
<td>274.62</td>
<td>89.25</td>
<td>57.2</td>
</tr>
<tr>
<td></td>
<td>F-SH40</td>
<td>211.62</td>
<td>68.78</td>
<td>38.1</td>
</tr>
<tr>
<td></td>
<td>F-RL60</td>
<td>169.18</td>
<td>54.98</td>
<td>37.8</td>
</tr>
<tr>
<td></td>
<td>F-SL60</td>
<td>154.34</td>
<td>50.16</td>
<td>41.5</td>
</tr>
<tr>
<td></td>
<td>F-RH60</td>
<td>282.44</td>
<td>91.79</td>
<td>49.2</td>
</tr>
<tr>
<td></td>
<td>F-SH60</td>
<td>232.01</td>
<td>75.40</td>
<td>60.7</td>
</tr>
<tr>
<td>Combined Flexure-Shear</td>
<td>S-RL-NST</td>
<td>246.78</td>
<td>56.12</td>
<td>51.2</td>
</tr>
<tr>
<td></td>
<td>S-SL-NST</td>
<td>218.60</td>
<td>49.71</td>
<td>61.0</td>
</tr>
<tr>
<td></td>
<td>S-RL-100</td>
<td>357.93</td>
<td>81.39</td>
<td>52.4</td>
</tr>
<tr>
<td></td>
<td>S-SL-100</td>
<td>288.18</td>
<td>65.53</td>
<td>47.7</td>
</tr>
<tr>
<td></td>
<td>S-RH-NST</td>
<td>244.06</td>
<td>55.50</td>
<td>70.9</td>
</tr>
<tr>
<td></td>
<td>S-SH-NST</td>
<td>231.76</td>
<td>52.70</td>
<td>62.8</td>
</tr>
<tr>
<td>Compression</td>
<td>C-RL-100</td>
<td>1975.96</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>C-SL-100</td>
<td>2291</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>C-RH-100</td>
<td>1729.90</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>C-SH-100</td>
<td>2076.06</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The four strain gauges were targeting mid-span in the flexure specimens and the location of the applied load in the combined flexure-shear specimens. The concrete strain gauges are intended to measure the compressive strain in the concrete. As often the case, some of the strain gauges did not work at all throughout the tests due to mechanical failure, inadequate waterproofing, etc. Some other strain gauges started working well at the beginning of the test, but either stopped reporting results later or reported illogical results due to crack propagation in their vicinity. The results of the properly working strain gauges were averaged and plotted for each beam. From the obtained data, relationships involving strains in the steel and concrete were plotted versus the load, as shown in Figure 61 to Figure 74. All the results for the beams confirm the visual observations of flexural failure events, where the longitudinal steel reinforcement has yielded in all samples (except S-SH-NST) prior to concrete crushing at the end. For the 4 tested columns, plots for the strain in the longitudinal reinforcement versus the applied load are presented in Figure 75. They show that the
mild steel indeed reached yielding before concrete crushing. As for the strand reinforced columns, the strain in the longitudinal reinforcement did not yield since the concrete crushes at much lower strain (about 0.003) than the yield strain of the high strength steel (about 0.01).

Figure 61: Variation of strain with increase in load for F-RL-40.

Figure 62: Variation of strain with increase in load for F-SL-40
Figure 63: Variation of strain with increase in load for F-RH-40.

Figure 64: Variation of strain with increase in load for F-SH-40.
Figure 65: Variation of strain with increase in load for F-RL-60.

Figure 66: Variation of strain with increase in load for F-SL-60.
Figure 67: Variation of strain with increase in load for F-RH-60.

Figure 68: Variation of strain with increase in load for F-SH-60.
Figure 69: Variation of strain with increase in load for S-RL-NST.

Figure 70: Variation of strain with increase in load for S-SL-NST.
Figure 71: Variation of strain with increase in load for S-RL-100.

Figure 72: Variation of strain with increase in load for S-SL-100.
Figure 73: Variation of strain with increase in load for S-RH-NST.

(a) Concrete  (b) Longitudinal steel

Figure 74: Variation of strain with increase in load for S-SH-NST.

(a) Concrete  (b) Longitudinal steel
4.4 Observations from the Experimental Tests

4.4.1 Cracking behavior. Observations from the experimental tests showed that the concrete deformation in strands reinforced beams was always concentrated in a few number of cracks around the loading points. However, the cracks were spread more uniformly over the beam length in the conventional steel reinforced beams; thus, enhancing the ductility of the members. Furthermore, more uniformly distributed small cracks were detected in specimens reinforced with small amount of steel than in specimens with large amount of steel. For instance, in the 60 MPa specimens, four major cracks were observed in specimen (F-SL-60) with low strand reinforcement, while only two major cracks have been found in specimen (F-SH-60) with high strand reinforcement. In the 40 MPa samples, although both specimens (F-SL-40) and (F-SH-40) experienced three major cracks, specimen (F-SL-40) had an additional horizontal crack sandwiched between two vertical cracks, resulting in an arching action. Specimen (F-SL-40) failed quicker than its (F-SH-40) counterpart because the three vertical cracks and horizontal crack were joined; thus, causing a reduction in the depth of the beam. The experimental test results also showed that Specimens (F-RH-40) and (F-RL-40) had less inherentductility compared with (F-RH-60) and (F-RL-60), which is logical because the ductility reduces with an increase in the concrete compressive strength. This pattern was not always observed in the strand reinforced flexural specimens due to the complex nature of the behavior and unusual cracking.

Figure 75: Longitudinal steel strain versus load for the compression specimens.
formation. As expected, concrete crushing at ultimate was observed within the mid-span region between the applied loads where the bending moment was largest in the flexural beam specimens.

In the combined flexure-shear specimens, an almost similar behavior was observed in terms of the concrete and reinforcement effects, but with different crack formation and failure pattern. For instance, one major crack was detected in the specimens (S-SL-NST) and (S-SH-NST), but two major cracks were observed in (S-SL-100). This is expected since there were stirrups within the instrumented region in this beam, which helped distributing the cracks and improving the ductility behavior. In fact, the flexural capacity of beams (S-SL-NST) and (S-SL-100) was almost the same, but the ductility behavior was distinctively different. The flexural cracks appeared first at lower load levels followed by shear crack later during the test. Flexural cracks were observed in (S-SL-NST) and (S-SH-NST), but both flexural and shear cracks were found in (S-SL-100). However, all the previously mentioned beams failed due to flexure, as evidenced by eventual concrete crushing prior to collapse, where good ductility was attained following steel yielding. As for the specimens that were longitudinally reinforced with conventional rebars, the cracks were distributed in a good manner, where many cracks appeared around the loading point. In such beams, both flexural near the applied load and shear cracks in the high shear region were observed. In this group, the two specimens with low rebars ratio (S-RL-NST) and (S-RL-100) failed in flexure. However, the other specimen (S-RH-NST) failed in shear, where the dominant crack was inclined and concrete crushing in the compression zone was not evident at failure. The reason behind that is the higher steel reinforcement ratio compared to (S-RL-NST), which resulted in higher flexural capacity, while the shear capacity is same. It should be noted that the load that causes (S-RH-NST) to fail in flexure is quite close to the one that leads to shear failure. There was inadequate ductility observed in this beam because the failure mode was shear with little deformation capability beyond the ultimate load. In all combined flexure-shear samples the crushing event was observed at the concrete top under the loading point as expected except in specimen (S-RH-NST).

The cracking behavior in similar columns with different reinforcement was clearly different. For instance, gradual cracking failure was observed in steel reinforced columns with a considerable deformation capacity after ultimate. However,
in strands reinforced columns, the specimens failed suddenly after ultimate with a drastic drop in load with no enough warning before failure. On the other hand, there was no clear difference in ductility with respect to the reinforcement ratio, whether in strands or rebars. Few cracks appeared at the upper and lower portions in steel reinforced columns and started widening with the load till failure. However, one major crack was detected in strands reinforced sections at a specific area along the column depth. This is because the reinforcement diameter in the case of strands is much less.

4.4.2 Failure mode. For the 14 considered beams, the mode of failure was observed to be flexural, even for the beams that were subjected to flexure plus significant shear. In these beams the failure mode included vertical cracks on the tension side of the beams at the location of maximum moment, followed by concrete crushing just prior to collapse. However, only one beam (S-RH-NST) failed in shear in a brittle fashion, with no signs of final concrete crushing at ultimate conditions. After detailed analysis of this beam, it was found that it almost attained its maximum flexural capacity and the longitudinal steel was on the verge of yielding.

As the load increased on the beams, vertical flexural cracks started to appear at lower load values due to tensile stresses in the concrete exceeding the modulus of rupture. At the beginning of the tests, few short cracks started forming at the location of maximum bending moment, then as time passed more cracks appeared along the beam length from both sides. While most of the cracks were flexural in nature, some of the cracks that formed near the supports in the regions of high shear were flexural-shear cracks. As expected, initiation of flexural cracks in the short combined flexure-shear specimens was at higher load values than the corresponding loads levels for pure flexural specimens. In the flexural-shear specimens, more inclined hairline cracks were observed at angles of around 45 degrees in the region between the load and adjacent support. Those cracks propagated upwards towards the location of the applied load, but their width was much smaller than the dominant flexural cracks that were vertical.

In the four axial compression specimens, the failure mode was a purely compression since the axial load was concentric on the columns. At the beginning of the test, cracks started to appear at both ends of the columns due to clear cover
spalling off the steel tubing that was installed to eliminate premature failure of the specimens. Further cracks formed mostly within the instrumented regions; they were very narrow when the load was low and propagated towards the edges of the column as the load increased. At ultimate and following spalling of the clear cover off the core in the strand reinforced columns, buckling was observed in the strands between the ties showing separation of the seven wires from each other and causing the reinforcement to lose some of its strength. This phenomenon was not depicted in the conventionally reinforced columns.

4.4.3 Load-deflection behavior. The load deflection behavior in the steel reinforced beams was different than that in the strands reinforced ones in terms of yielding, elastic behavior, ductility and service stiffness. For instance, the yielding point was clear in all steel reinforced beams, which is not the case in strands reinforced ones. The load deflection curve was always steeper in the strands reinforced beams, but the displacement the beam handles after ultimate point is less, which reflects the lower ductility in the case of strands. The stiffness in higher reinforced beams was higher than in lower reinforced ones, where the load deflection curve became milder. This was observed in steel and strands reinforced sections. The ultimate displacement was higher in general in the purely flexural loaded samples, but the deformation capacity was a little bit less, because the displacement at service load condition is more. Relatively, the same behavior in terms of the difference between steel rebars and strands was observed in axial compression samples, but with no yielding of the steel in the case of strands. This is because the ultimate strain in the case of strands is always less than the yielding strain of the steel reinforcement.

4.4.4 Steel strain response. It was observed that the strain behavior of the steel reinforcement was similar to the load-deflection relationship in the case of beams. The steel reinforcement type, volumetric ratio and concrete compressive strength effects on the ductility of the tested beams were clear from the load-strain graphs. All the results confirm the observations of the flexural failure events in which the longitudinal steel has yielded in all beams prior to concrete crushing except for the specimen S-SH-NST, as presented in Figure 61 to Figure 74 earlier. The steel strain at yield was in the range of 0.0024-0.0028 in the rebars reinforced samples and 0.01-0.012 in strands reinforced ones. The strain at maximum load in S-SH-NST was around 0.0023, which is on the verge of yielding, but did not actually yield. The
elastic modulus was almost the same up to yielding in the rebars reinforced samples, but it was reducing with the increase in loading in the strands reinforced sections.

As explained before, the concrete failed due to top fibers crushing in all beam specimens except S-SH-NST. The concrete ultimate strains were detected to be in the range of 0.0009-0.0014 in the strain gauges at 30 mm from the top of the beams and 0.00025-0.0006 in the strain gauges at 60 mm from the top of the beams at the location of maximum bending moment. As expected, the upper strain gauge readings were better and more logical than the lower strain gauges at ultimate condition. Thus, the readings of the lower strain gauges are neglected in the analysis. The concrete strains at crushing in the 60 MPa samples were a bit higher than those of the 40 MPa samples, which is similar to what is typically observed in concrete cylinder testing.

The difference in the load-strain relationships of the four tested columns was not significant. However, it was observed that the load-strain curves in the strands reinforced specimens were a little bit steeper than the corresponding curves in the rebars reinforced specimens. Strain at ultimate was found to be in the range of 0.00149-0.00294, with the heavily reinforced columns having higher strains in the reinforcement at ultimate.
Chapter 5: Theoretical Studies

5.1 General Background

In this chapter, the flexural ductility performance of different cross-sections reinforced with high strength steel strands are investigated and compared with corresponding sections conventionally reinforced with mild steel rebars. The parameters that are considered in the analysis include the material properties, longitudinal steel reinforcement ratio, and loading conditions. The variations in bending moment capacity and steel reinforcement efficiency are captured with consideration of the aforementioned parameters. The chapter also proposes relevant design recommendations for minimum ductility in flexural members based on the strain limits that conform to the ACI 318 code.

To begin with, ductility allows the member to continue carrying high loads when deformation under plastic condition takes place. On the material level, steel strands possess less ductility than normal rebars made from mild steel. This is because the ratio of the strain at ultimate to that at yield is much smaller than the equivalent ratio for mild steel. Also, the yield point in high strength steel strands is not clearly defined on the stress-strain diagram as in mild steel. If high strength steel strands are to be used in reinforced concrete members, ductility becomes a major issue; hence, it must be addressed since most current structural design codes do not have provisions for steel with yield strength higher than 515 MPa.

In general, flexural ductility can be measured using the ratio of curvature or deflection at ultimate to that at yield. Information on geometry, reinforcement and material properties is sufficient to compute the curvature ductility. However, additional information related to the beam length and loading are required to calculate the deflection ductility. While the moment-curvature relationship is distinct for a given cross-section, the load-deflection relation is not unique because it depends on the span length, support conditions and loading state. Since high strength steel strands do not have a well-defined yield point, curvature and deflection ductility of concrete members reinforced with such steel cannot be accurately defined. Following the approach used by past researchers on the subject [9, 12], the curvature ductility index, \( \mu_c \), and displacement ductility index, \( \mu_d \), will be calculated in this study as the ratio of
ultimate to service deformations. The deformations at service are taken as the curvature in the section or deflection of the beam when the steel reaches 60% of the steel yield strength. The value of 60% of the yield point is in line with the basic assumption in the ACI code when considering service deflection calculations and crack-control limitations. Figure 76 shows the two conditions that correspond to flexural beams at service and ultimate.

![Figure 76](attachment:image.png)

**Figure 76:** Stress and strain relationships at service and ultimate conditions.

### 5.2 Material Stress-Strain Models

In order to compute the curvature ductility index for reinforced concrete sections in flexure, information on the full stress-strain relationship of concrete is required. This is because Whitney block cannot be used to determine the compression in the concrete due to bending at extreme strain levels less than 0.003. In this study, Thornfeldt et al. (1987). [11], concrete stress-strain model is utilized for the purpose of predicting the corresponding internal compressive stresses in the concrete based on the strain diagram at both ultimate and service levels.

Thornfeldt’s stress-strain model is based on the following mathematical equation:

\[
f_c = \frac{n \left( \frac{\varepsilon_c}{\varepsilon_{co}} \right)}{n - 1 + \left( \frac{\varepsilon_c}{\varepsilon_{co}} \right)^nk}
\]  

(9)
In the above, the strain corresponding to $f'_c$ (MPa), $\varepsilon_{co}$, is determined by:

$$\varepsilon_{co} = \left(\frac{f'_c}{E_c}\right) \left(\frac{n}{n-1}\right)$$  \hspace{1cm} (10)

Also, the modulus of elasticity of the concrete, $E_c$ (MPa), is calculated by either the ACI 318 expression or the following equation by Carrasquillo et al. [16]:

$$E_c = 3300 \sqrt{f'_c} + 6900$$  \hspace{1cm} (11)

and the constant “n” is computed from:

$$n = \frac{f'_c}{17} + 0.8$$  \hspace{1cm} (12)

The decay factor in the stress-strain relationship is given by:

$$k = 0.67 + \frac{f'_c}{62} \geq 1$$  \hspace{1cm} (13)

Using the above information, typical stress-strain relationships for concrete with different compressive strengths are developed using Thornfeldt et al. model [11] and shown in Figure 77.

![Figure 77: Typical concrete stress-strain relationships using Thornfeldt model.](image-url)
As for the stress-strain relationship for the steel reinforcement in tension, the study considers an idealized elastic-perfectly plastic behavior involving two straight lines joining at yield for both the mild steel rebars and high strength strands, as shown in Figure 78. This approach is practical and conservative as it ignores the strain hardening in mild steel and the increase in strength beyond yield in high strength steel.

![Figure 78: Idealized elastic plastic relationship of the modeled reinforcements.](image)

### 5.3 Curvature Ductility Analysis

Curvature ductility, moment capacity and reinforcement efficiency at ultimate condition are determined for different concrete cross-sections that are reinforced with high strength steel strands. Those sections are then compared to corresponding sections reinforced with mild steel reinforcement. The parametric study aims to predict the variation in deformation capacity and reinforcement efficiency with the steel reinforcement ratio for different concrete compressive strengths. The considered parameters are predicted in each case using an effective tension reinforcement steel ratio for up to 10% when using mild steel and around 1.5% when utilizing high strength steel. Beyond these limits, the curvature ductility becomes negligibly small, as the curvature at ultimate becomes smaller than the curvature corresponding to service condition. Two types of steel were considered in this study, a mild steel with yield strength of 414 MPa (ASTM A615-Billet steel, ASTM A617-Axle steel, or ASTM A706-Low Alloy) and high strength steel strands with ultimate tensile strength of 1860 MPa (ASTM A416). The concrete compressive strength ranged between 25 MPa and 80 MPa for both types of steel. Rectangular cross-sections with thickness-to-width ratios ranging between 0.25 and 4.0 were addressed. Although flanged sections were not considered, the ductility behavior of such sections is not believed to be
different from that of rectangular shapes. Results of the analysis are used to adjust the reinforcement strain limits for tensioned-controlled and compression-controlled cross-sections reinforced strands, to match the inherent ductility in conventionally reinforced sections required by the ACI 318 [3]. As the increase in yield strength reduces the ductility in reinforced concrete sections, it is expected that the strain limits of $\varepsilon_y$ (yield strain) and 0.005 at the boundaries of the transition-controlled region for sections reinforced with mild steel rebars will get reduced in sections reinforced with high strength steel strands.

As stated earlier, to determine the curvature ductility index of a given cross-section, one needs to compute the curvature at ultimate and divide it by the curvature when the steel reinforcement reaches a stress equal to 0.6 times the yield strength. Note that the ultimate condition corresponds to crushing of concrete at the extreme compressive fibers, assumed to be at a strain equal to 0.003.

At service, the compressive strain at the extreme fibers of the concrete is determined by a trial-and-error procedure. First, the depth of the neutral axis for this condition is assumed, and the corresponding extreme strain in the concrete, $\varepsilon_c$, is obtained from the linear strain diagram by similar triangles. The neutral access depth, $c$, is then divided into a number of thin layers (e.g. “n” layers each of thickness $c/n$) and the corresponding stress at the mid-depth of each layer is computed using Thernfeldt et al. [11] model. The strength of concrete in tension just below the neutral axis is negligibly small and is neglected. Next, the compressive forces, $P_i$, are determined for each layer by multiplying the stress by the layer’s area. The compression forces in the concrete are summed up and compared with the total tensile force in the steel reinforcement, in this case equal to the area of reinforcement times 60% of the yield strength. If the two forces are very close to each other, then the assumed depth of the neutral axis is correct and is then used to calculate the curvature, $\Phi_s$, (in the units of radians) from $\varepsilon_c/c$. However, if the two forces do not match, then a new neutral axis is assumed and the procedure continues as before until the final tension and compression converge. If the corresponding moment at service is desired, then the nominal moment, $M_n$, of all the compressive forces are summed about the location of the steel reinforcement.
At ultimate condition, the method of finding the curvature, $\phi_u$, of the cross-section when the extreme compressive strain in the concrete reaches 0.003 is similar to the one of service load condition. Here, the strain in the steel is unknown, but is larger than the yield strain if the cross-section is under-reinforced. A neutral axis is assumed and the concrete part of the cross-section is sub-divided to many thin layers. The strain at the center of each layer is determined from similar triangles, the corresponding stress is obtained using Thornfeldt model, and the compressive force $P_i$, in each layer is computed. The total compressive force in the concrete must be equal to the tensile force at yield; otherwise, a new neutral axis is assumed and the procedure is repeated until both forces become equal. At that instant, the curvature at ultimate is equal to 0.003/c. The above explained approach is carried out in this study on mild steel rebar reinforced sections and corresponding sections employing high strength steel strands.

A summary of the steps required to calculate the curvature at service, $\phi_s$, and also at ultimate, $\phi_u$, is shown below in reference to Figures 79 and 80, respectively. Using these values, the curvature ductility index is obtained from:

$$\mu_c = \frac{\phi_u}{\phi_s}$$

(14)

Figure 79: Internal strain, stress and force diagrams at service.

Figure 80: Internal strain, stress and force diagrams at ultimate.
I- Curvature at service:
1. Assume a neutral axis location from the extreme compressive fibers: \( c \)
2. Determine the strain at the extreme compressive fibers of the concrete:
   \[ \varepsilon_{cs} = \frac{0.6f_y}{E_s} \frac{d}{d - c} \]  
   (15)
3. Subdivide the concrete depth in compression into \( n \) equal layers, and find the strain
   at the center of each layer \( i \) using similar triangles:
   \[ \varepsilon_{ci} = \frac{c_i \varepsilon_{cs}}{c} \]  
   (16)
   where \( c_i \) is the distance from the neutral axis to the mid-depth of layer \( i \).
4. Find the compressive force in each thin layer of concrete: \( P_i \), where \( f_{ci} \) is the stress at
   the center of the layer corresponding to strain \( \varepsilon_{ci} \), obtained from Thornfeldt model.
   \[ P_i = \frac{1}{n} \times c \times b \times f_{ci} \]  
   (17)
5. Sum up the total compressive forces in all the \( n \) concrete layers, \( \Sigma P_i \), and compare
   the resultant to the tensile force in the steel reinforcement, equal to \((0.6f_yA_s)\). If the
   two values are equal then continue; otherwise, go back to Step 1 and assume a
   different value of \( c \).
6. The curvature ductility at service is equal to:
   \[ \phi_s = \frac{\varepsilon_{cs}}{c} \]  
   (18)
II- Curvature at ultimate:
1. Assume a neutral axis location from the extreme compressive fibers: \( c \)
2. Determine the strain at the tensile steel reinforcement:
   \[ \varepsilon_s = \frac{0.003(d - c)}{c} \]  
   (19)
3. Subdivide the concrete depth in compression into \( n \) equal layers, and find the strain
   at the center of each layer \( i \) using similar triangles:
   \[ \varepsilon_{ci} = \frac{0.003c_i}{c} \]  
   (20)
4. Find the compressive force in each thin layer of concrete: \( P_i \), where \( f_{ci} \) is the stress at
   the center of the layer corresponding to strain \( \varepsilon_{ci} \), obtained from Thornfeldt model.
   \[ P_i = \frac{1}{n} \times c \times b \times f_{ci} \]  
   (21)
5. Sum up the total compressive forces in all the n concrete layers, $\sum P_i$, and compare the resultant to the tensile force in the steel reinforcement, equal to $(f_y A_s)$. If the two values are equal then continue; otherwise, go back to Step 1 and assume a different value of $c$.

6. The curvature ductility at ultimate is equal to:

$$\phi_u = \frac{0.003}{c}$$ (22)

Using extensive analysis in Matlab, the steel reinforcement strain versus curvature ductility relationship is simulated for tens of thousands of singly-reinforced rectangular cross-sections involving different cross-section aspect ratio, concrete compressive strength, and effective steel reinforcement ratios. The analysis considers reinforcement in the form of mild steel rebars ($f_y = 414$MPa) and high strength strands ($f_{pu} = 1860$ MPa), as shown in Figure 81. For practical considerations, only reinforced sections with curvature ductility over 1.0, in which the curvature at ultimate is higher than that at service, are included in the analysis.

![Figure 81: Strain-ductility relationship for various RC sections.](image)

Figure 81 shows that the relationship between the ultimate steel strain and ductility is significantly different among the two considered types of reinforcement. Also, the relationship between the two parameters is almost linear for a given concrete
compressive strength and steel yield strength. As expected, the strands reinforced sections possessed lower curvature ductility than corresponding rebar reinforced sections at the same steel strain at ultimate due to their much high strength. On average, the ratio of the curvature ductility of the mild-to-high strength steel RC sections is equal to 3.1 when the strain in the steel at ultimate is 0.5%, 3.25 when it is 1%, 3.44 when it is 1.5%, and 3.67 when it is 2%. Although it is known that an increase in the concrete compressive strength increases the flexural ductility of a given cross-section, Figure 81 shows that for a given strain in the steel reinforcement the ductility of a reinforced concrete cross-section decreases with an increase in the concrete strength. This is because the reinforcement ratio that corresponds to a specific strain in the steel at ultimate is much higher when using high strength concrete than when using lower strength concrete; hence, its impact on ductility is more critical than the effect of higher concrete strength. The compressive strength effect on curvature ductility in the case of strands is not as significant as in the case of rebars.

The “moment capacity ratio” is defined as the nominal flexural capacity at ultimate divided by \( bd^2 \), i.e. \( \frac{M_n}{bd^2} \). This expression results in a constant value for any section having the same effective reinforcement ratio, yield strength and concrete compressive strength. In other words, this ratio filters out the effect of the cross-section dimensions from the moment capacity. For a singly reinforced rectangular section, the moment capacity ratio is given by the following expression if Whitney block of constant stress equal to 85% of the concrete compressive strength is used to model the actual concrete stress in compression at ultimate condition:

\[
\frac{M_n}{bd^2} = \rho f_y \left( 1 - \frac{0.59 \rho f_y}{f'_c} \right)
\]  

(22)

In this study, the nominal moment capacity is obtained with the use of Thornfeldt concrete model instead of Whitney’s model. This is done in order to be consistent with the previous calculations of the curvature and to provide better accuracy of the results. Figure 82 provides relationships between the effective steel reinforcement ratio and the moment capacity ratio for the same RC sections that were considered in Figure 81. In this figure, only reinforced sections with curvature ductility over 1.0 are included.
Figure 82: Steel ratio versus moment capacity relationship by Thornfeldt et al.

From Figure 82, the moment capacity ratio for the two considered steel strengths at a given steel reinforcement ratio in strands reinforced sections is more than three times the corresponding moment capacity ratio in rebars reinforced sections. This is logical because the tension force in the reinforcement at ultimate is proportional to the yield strength of the steel. The effect of the concrete compressive strength on the relationship between the effective steel reinforcement ratio and moment capacity ratio is more predominant in mild steel reinforced sections than in equivalent high strength steel reinforced sections.

The “steel reinforcement efficiency” is an important parameter that gives an idea about how effective the reinforcement is within a given cross-section. It is defined as the ratio of the moment capacity ratio divided by the effective steel reinforcement ratio, i.e. \( \frac{M_n}{\rho bd^2} \). The reinforcement efficiency is simulated for different reinforced concrete cross-sections having a wide range of steel reinforcement ratios, where the results are presented in Figure 82. In the figure, mild steel as well as high strength steel are addressed for a wide range of concrete compressive strengths. The results show that the reinforcement efficiency in reinforced concrete sections decreases with an increase in the effective steel reinforcement ratios. This is because the increase in moment capacity (associated with higher amount of steel) is not at the same rate of increase in the steel reinforcement ratio. This trend is more dominant in the case of low concrete compressive strength
than in high concrete compressive strength. The results also demonstrate that the decrease in reinforcement efficiency in the sections with high strength steel is at a much higher rate compare to the decrease in the sections with mild steel.

![Figure 83: Steel ratio versus reinforcement efficiency relationship by Thornfeldt.](image)

The ACI 318 code relates the strength reduction factor in flexure to the strain in the extreme layer of steel closest to the tension side. It subdivides the flexural behavior into 3 regions: (1) tension-controlled, (2) transition, and (3) compression-controlled. Cross-sections within the tension-controlled region have their strain in the extreme tension steel layer at least equal to 0.005. Sections within the compression-controlled region have their strain in the extreme tension steel layer at most equal to the yield strain (taken 0.002 for grade 414 steel). Steel for sections in the transition region has a strain value that lies between $\varepsilon_y$ and 0.005.

Following the approach used by Shahrooz et al. [6] to determine the strain limits for high strength bars in concrete sections is used in this study on grade 1860 strands. This approach involves determining the corresponding strain limits of high strength strands from those of mild steel rebars such that the curvature ductility index for both is the same. To determine the new strain limits for reinforced concrete section having unstretched grade 1860 steel 7-wire strands, we consider the relationship between curvature ductility index and steel strain presented earlier in Figure 81 and reproduced again in Figure 84. To determine the new boundary
between compression and transition controlled sections, as well as the boundary between the transition and tension-controlled sections, the corresponding curvature ductility index is determined in mild steel reinforced sections for an upper bound and lower bound concrete compressive strength. From the graph, the ductility values corresponding to the yield strain in the rebars are 1.92 and 1.76 for concrete strengths equal to 25 and 80 MPa. At these curvature ductility indices, the corresponding strain in the sections employing high steel strands can be determined for the two extreme compressive strengths, equal in this case to 0.0099 and 0.0087. These new strain limits for all concrete strengths are shown in Table 9. Also, presented in the table are the new strain limits corresponding to 0.004 and 0.005 in mild steel. The additional new strain limit corresponding to 0.004 in mild steel section is the minimum strain specified by the ACI 318 code for the flexural design of new members.

![Figure 84: New strain limits prediction graph.](image)

Table 9: Strain limits at tension, compression and transition regions.

<table>
<thead>
<tr>
<th>Old strain limits based on mild steel Grade 414 MPa</th>
<th>New strain limits for Grade 1860 steel strands</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f'_c = 20$ MPa</td>
</tr>
<tr>
<td>0.002</td>
<td>0.0099</td>
</tr>
<tr>
<td>0.005</td>
<td>0.0198</td>
</tr>
<tr>
<td>0.004</td>
<td>0.0149</td>
</tr>
</tbody>
</table>
Although there is a range in the new strain limits corresponding to the different concrete compressive strengths addressed in this study, practically, one value needs to be considered irrespective of the concrete strength. Therefore, new strain limits corresponding to the lower concrete strength $f'_c = 20$ MPa are recommended, equal to 0.01 and 0.02 after rounding, as shown in Figure 85.

![Figure 85: Predicted reduction factors graph for design of flexure.](image)

Based on the above, the recommended strength reduction factors for flexural design with unstretched Grade 1860 steel strands are:

$$\varepsilon_t \leq 0.01: \quad \phi = 0.65$$

$$\varepsilon_t \geq 0.02: \quad \phi = 0.90$$

$$0.01 < \varepsilon_t < 0.02: \quad \phi = 0.65 + 25(\varepsilon_t - 0.01)$$  \hspace{1cm} (24)

Figure 86 presents strand reinforced section design control graph using the previous analysis, showing the strain limits for each of the considered concrete compressive strengths. The figure shows that the transition region gets narrower as the compressive strengths get lower. Also, the effective steel reinforcement ratios at the stress limits are reduced with the decrease in the concrete compressive strength. Such a graph can be used to compute the design flexural capacity of singly-reinforced rectangular sections utilizing high strength strands.
5.4 Deflection Ductility Analysis

While the previous analysis on curvature ductility is useful; it has some shortcomings because it is a function of just the cross-section dimensions and material properties, but not the beam length, boundary conditions and nature of loading. An alternative to the curvature ductility index involves the use of the deflection ductility index, defined as the ratio of the beam deflection at ultimate to that at yield. However, as explained earlier, since the stress-strain relationship of high strength steel does not show a clear yield point, the deflection at service (when the longitudinal steel reaches 60% of its yield strength) can replace that at yield. To find the deflection of an RC beam under a given loading condition using the conjugate beam method, the moment-curvature relationship of the reinforced concrete cross-section can be utilized. This approach does not require information about the cracked moment of inertia, which is variable along the length of the beam due to changes in the bending moment diagram. To simplify the analysis, an idealized moment-curvature relationship in the form of elastic-perfectly plastic correlation is used in this study, as shown in Figure 87.
Figure 87: Idealization of the moment-curvature relationship.

Application of high concentrated loads on RC beams causes formation of plastic hinges at the location of the concentrated loads at ultimate condition. The size of a plastic hinge affects the calculation of the beam deflection at ultimate. There has been extensive research on this subject, of which the important studies have been summarized by Zhao et al. [17]. The results of these past studies have provided plastic hinge width, $W_{P-H}$, expressions for beams as a function of the diameter of the rebars, $d_b$, effective depth of steel reinforcement, $d$, yield strength of the flexural reinforcement, $f_y$, and distance from critical section to the point of contra-flexure (zero-moment), as shown below:

- Sawyer (1964) [12]:
  \[ W_{P-H} = 0.25d + 0.075z \]  

- Mattock (1967) [13]:
  \[ W_{P-H} = 0.5d + 0.05z \]  

- Paulay and Priestley (1992) [14]:
  \[ W_{P-H} = 0.08z + 0.022d_b f_y \]  

- Panagiotakos and Fardis (2001) [15]:
  \[ W_{P-H} = 0.18z + 0.021d_b f_y \]  

The plastic hinge is assumed to spread on each side of the applied concentrated load equally, as shown in Figure 88. Due to its large rotational capability, the curvature within the whole plastic hinge width is taken equal to $\phi_u$.

In the following discussion, mid-span deflection calculations at service and ultimate are derived for simply-supported RC beams subjected to two point loads equidistant from the supports and to a single concentrated load at some distance away from a support. The corresponding deflection ductility indices for these two cases are also determined.
Figure 88: Assumed spread of the plastic hinge.

Using the conjugate beam method, the vertical deflection at any point can be determined by first drawing the bending moment diagram along the beam’s length, then dividing the moment by $EI$, placing the result as loading along the conjugate beam, and finally computing the bending moment at the desired point due to this loading. For a beam under two-point loading, the bending moment diagram at service and the corresponding conjugate beam are shown in Figure 89.

Figure 89: Actual and conjugate beam at service for 2 loads case.

Figure 90: Actual and conjugate beam at service for 2 loads case.

Note: $a' = a - W_{P,H}/2$ and $\phi'_y = \phi_y (a'/a)$
The corresponding actual and conjugate beams at ultimate are presented in Figure 90. Note that the influence of the plastic hinge width on the curvature diagram in the conjugate beam figure is accounted for by widening the $\phi$-region by $W_{P,H}/2$ on both sides.

The mid-span deflection of the beam at service is determined using Figure 69 by finding the support reactions of the conjugate beam first, taking a cut at mid-span, and calculating the resultant moment at that location, as follows:

$$
\Delta_s = \left(\frac{\phi_s a}{2} + \phi_y \left(\frac{L - 2a}{2}\right)\right) \frac{L}{2} - \left(\frac{\phi_s a}{2}\right) \left(\frac{L}{2} - \frac{2a}{3}\right) - \phi_s \left(\frac{L - 2a}{2}\right) \left(\frac{L - 2a}{4}\right)
$$

which simplifies to the following expression:

$$
\Delta_s = \frac{\phi_s L^2}{8} - \frac{\phi_s a^2}{6}
$$

Likewise, the mid-span deflection of the beam at ultimate is determined using Figure 91 by taking a cut at mid-span of the conjugate beam as follows:

$$
\Delta_u = \left(\frac{\phi'_y a'}{2} + \phi'_y \left(\frac{L - 2a'}{2}\right)\right) \frac{L}{2} - \left(\frac{\phi'_y a'}{2}\right) \left(\frac{L}{2} - \frac{2a'}{3}\right)
$$

$$
- \phi'_y \left(\frac{L - 2a'}{2}\right) \left(\frac{L - 2a'}{4}\right) + \left(\phi_u - \phi'_y\right) \left(\frac{L}{2} - a'\right) \left(\frac{L}{2}\right)
$$

$$
- \left(\phi_u - \phi'_y\right) \left(\frac{L}{2} - a'^2\right)
$$

which reduces to:

$$
\Delta_u = \frac{\phi'_y a'^2}{3} - \frac{\phi_u a'^2}{2} + \frac{\phi_u L^2}{8}
$$

After substituting the $(a-d/2)$ for $(a')$ and $(\phi_y a'/a)$ for $(\phi'_y)$, we get:

$$
\Delta_u = \frac{\phi_y (a - \frac{W_{P,H} - H}{2})^3}{3a} - \phi_u \left(a - \frac{W_{P,H}}{2}\right)^2 + \frac{\phi_u L^2}{8}
$$

The service and ultimate deflections are derived as well for simply-supported RC beams subjected to one point loading at a specific distance from the support of a flexural loaded beam. The corresponding deflection ductility indices for these two
cases are also determined. Using the conjugate beam method, the vertical deflection at any point can be determined similarly by the conjugate beam method by computing the bending moment of the conjugate beam at the desired point due to this loading. For a beam under one-point loading, the bending moment diagram at service and the corresponding conjugate beam are shown in Figure 71.

**Figure 71: Actual and conjugate beam at service for 1 load case.**

In this case, the corresponding actual and conjugate beams at ultimate are presented in Figure 92. Note that the influence of the plastic hinge width on the curvature diagram in the conjugate beam figure is accounted for by stretching the \( \phi_u \) region by \( W_{P-H}/2 \) on both sides of the loading point.

**Figure 92: Actual and conjugate beam at ultimate for 1 load case.**

The service deflection in this case is determined using Figure 91 by taking a cut in the conjugate beam at the location of the load and calculating the resultant moment at that point:
\[
\Delta_s = \frac{\phi_s (2L - a) a}{6} - \frac{\phi_s a (\frac{a}{3})}{2} = \frac{\phi_s (L - a) a}{3} \tag{33}
\]

The corresponding deflection at ultimate is determined with the help of Figure 92 by taking a cut in the conjugate beam at the location of the load and calculating the resultant moment at that point:

\[
\Delta_u = \sum M = \frac{a}{L} \left[ \frac{\phi''_y (L - 2a + a')^2}{3} + \phi_u W_{p-H} (L - a) + \frac{\phi'_y a'}{2} (L - \frac{2a'}{3}) \right] - \frac{\phi'_y a'}{2} \left( \frac{a'}{3} + \frac{W_{p-H}}{2} \right) - \phi_u a'^2 \frac{2}{8} \tag{34}
\]

Figure 93 shows the relationship between deflection ductility index and curvature ductility index for the case of a single load on a simple span for a specific example. The relationships are presented for the case of L=6000mm, \(\phi_s=1.0 \times 10^{-5}\) 1/mm, \(\phi_y=1.67 \times 10^{-5}\) 1/mm, and a constant \(W_{p-H}=500\)mm, which covers a curvature ductility index ranging between 2 and 7, in which the RC beam is under-reinforced. The relationship is provided for two shear span-to-length ratios: \(a/L=0.1\) and 0.5. The analysis showed that the relationship between the two indices is always linear and is insensitive to the location of the load(s) on the span (i.e. \(a/L\)). In general, the deflection ductility index is always smaller than the corresponding curvature ductility index, and the difference between the two indices slightly increases for beams possessing high ductility.

![Figure 93: Deflection to curvature ductility indices in the single loads case.](image-url)
The corresponding relationships between the deflection ductility and curvature ductility indices for the case of two loads on a simple span for \(a/L=0.1, 0.3\) and \(0.5\) are given in Figure 94. Similar to the case of a single load, the analysis of double loads on the span showed that the relationship between the two indices is also linear. However, the shear span-to-length ratio significantly affects this relationship, since for a given curvature ductility, the corresponding deflection ductility index when \(a/L\) is small has a much larger value than that when it is large. Although the deflection ductility index for the considered cases is always smaller than the corresponding curvature ductility index, both indices approach each other when the ratio \(a/L\) becomes very small. The analysis also shows that the effect of the shear span-to-beam length ratio on the relationship between curvature ductility and deflection ductility is minimal for beams possessing low ductility compared to highly ductile beams.

Figure 94: Deflection to curvature ductility indices in the double loads case.

The analysis on the previous example is extended to investigate the effect of the span length on the relationship between curvature ductility index and deflection ductility index. In the example, all parameters are kept the same, including \(a/L\) ratio, except the span length, which is taken one time 3000 mm and another time 6000 mm, as shown in Figure 95. The results indicate that the span length greatly affects the results when there is a single load on the span, but mildly impacts the findings when there are two loads. For a given curvature ductility index, the corresponding deflection ductility index is always larger when the span is smaller. The difference between the two indices for the two considered spans for the case of a single load is
much more pronounced when the curvature ductility is high. However, the effect of the span length on the relationship for the case of double loads is not significant. The results also show that the effect of the number of loads on the span is much more pronounced than the effect of the span length. For example, for a given curvature ductility of 4, the deflection ductility for the case of single load is 2.79 for the 3 m span and 2.24 for the 6 m span. The corresponding deflection ductility for the two loads condition is 3.86 for the 3 m span and 3.53 for the 6 m span; both of which larger than the previous case of a single load.

The above deflection ductility analysis in Figure 95 is applicable for both steel and strands reinforced concrete beams, as long as information is available with regard to the beam length, number of loads, location of loads, and moment curvature relationships. Note that this analysis does not account for the effect of shear on the deflection ductility. Hence, the accuracy of the results is expected to reduce for RC beams subjected to high shear values and low bending moments.
Chapter 6: Analysis of Results

In this chapter, the experimental results of all specimens are analyzed and compared with theoretical studies using different approaches with consideration of the material properties and loading conditions. Elastic stiffness and cracking moment are calculated and compared with the experimental findings. Moreover, load-carrying capacity and reinforcement efficiency of each specimen are calculated using experimental and theoretical analysis. The concrete compressive strength, reinforcement ratio and loading pattern effect on the strength and ductility of the beam and column specimens are analyzed and discussed. For the beams, both curvature and deflection ductility indices are used to evaluate the deformation capacity of each of the considered types of steel reinforcement.

6.1 Flexural Specimens

6.1.1 Service stiffness. In this section, the effects of the steel reinforcement ratio and concrete compressive strength on the stiffness of the beams at service are determined for eight of the 14 tested beams. The other six tested beams are not considered in the analysis because they have very short shear span-to-length ratio; thus, their behavior is more like a deep beam than a flexural beam. The service stiffness is defined as the slope of the experimental load-deflection curve within the region after concrete cracking but before the onset of steel yielding, as shown in Figure 96.

![Figure 96: Determination of service stiffness from typical load deflection graphs.](image)

In general, the service stiffness increases with an increase in the steel reinforcement ratio and concrete compressive strength. This is because the
transformed moment of inertia is affected by the amount of steel within the section, and the modulus of elasticity of concrete is related to the concrete compressive strength. The variation of service stiffness among the considered steel reinforcement ratios and concrete compressive strengths are presented in Figure 97. Note that the magnitudes of the service stiffness shown in the figure represents average values of the slope within the post-cracking and pre-yielding region. The observed results show that there is a slight improvement of up to 7% in the stiffness with 50% increase in the concrete compressive strength for both rebars and strands. However, there is significant increase in the obtained stiffness at service when comparing the high reinforced beams with the low reinforced beams, especially when using mild steel rebars. When comparing the stiffness at service for the rebar reinforced beams with the corresponding strand reinforced beams, the former is always larger. This is because the amount of reinforcement presents in the beams with rebars is higher than in the beams with strands, which contribute to the stiffness. Also, the steel in the strands is in the form of seven small wires, instead of one solid bar, which further contribute to lower stiffness in such beams.

Figure 97: Service stiffness results of the flexural samples.

The stiffness of the same beams is again calculated using a different approach. The previous analysis gives an idea about the beams stiffness, but does not provide a quantity in the form of “EI”, where $E$ is the modulus of elasticity and $I$ is the moment of inertia of the beam’s cross-section. This approach requires determining the load on
the beam and deflection at mid-span from experimental findings when the strain in the longitudinal steel (obtained from strain gauge readings) reaches 60% of the yield strength. The results are then used in the deflection formula of a simple beam subjected to two concentrated loads to obtain an equivalent stiffness, \((EI)_{\text{equiv}}\), as shown in Figure 98. Note that the obtained value represents an equivalent stiffness of the beam since the actual \(EI\) along the beam is not constant. Substituting the service deflection and load from the experimental results in the equation provided in Figure 98, one can compute the equivalent \(EI\) of the eight beams analyzed earlier and determine the effect of \(f'_c\) and steel reinforcement ratio on the considered parameter.

\[
\Delta_x = \frac{P_s \times 1250 \times 1900 \left(1900^2 - 1250^2 - 1900^2\right)}{6 \times (EI)_{\text{equiv}} \times 1900}
\]

**Figure 98: Relationship between load, deflection and stiffness**

The results of the new analysis of the eight beams, together with the deflection and load experimental results at service are provided in Table 10 and in Figure 99. These values can be compared with the gross moment of inertia times the modulus of elasticity of the concrete used in the beams to get an idea about the extent of cracking. The results are consistent with those provided earlier in Figure 98. They agree that as the concrete compressive strength and amount of steel reinforcement increase the stiffness of the beam also increases. They also confirm that the effect of the steel reinforcement ratio is much more significant with regard to the stiffness at service than the concrete compressive strength.

**Table 10: Equivalent stiffness along the tested beams at service.**

<table>
<thead>
<tr>
<th>Designation</th>
<th>(\Delta_x) (mm)</th>
<th>(P_s) (kN)</th>
<th>((EI)_{\text{equiv at Service}}) (N-mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>4.5</td>
<td>95.8</td>
<td>2.63x10^{12}</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>8.7</td>
<td>71.8</td>
<td>1.02x10^{12}</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>4.72</td>
<td>142.3</td>
<td>3.73x10^{12}</td>
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<tr>
<td>F-SH-40</td>
<td>8.45</td>
<td>86.4</td>
<td>1.27x10^{12}</td>
</tr>
<tr>
<td>F-RL-60</td>
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<td>99.3</td>
<td>2.90x10^{12}</td>
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<td>8.29</td>
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<td>1.08x10^{12}</td>
</tr>
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<td>F-RH-60</td>
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<td>8.09</td>
<td>111.7</td>
<td>1.71x10^{12}</td>
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</table>
6.1.2 Cracking moment. Cracking load was detected for each of the beam specimens experimentally and the corresponding cracking moment was computed based on the relationship between the applied loads and bending moment. The cracking moment was also computed theoretically using the cross-section details, transformed area method, and tensile strength of the concrete. The calculations showed that the moment of inertia of strands reinforced sections was slightly less than that of rebars reinforced sections because the area of strands is much smaller than the area of rebars area in equivalent sections. The cracking moment was calculated for each reinforced section, in reference to Figure 100, as follows:

\[ M_{cr} = \frac{f_r \cdot I}{c_b} \]  

(35)

Where,

\[ I = \frac{bh^3}{12} + bh\left(\frac{h}{2} - c_b\right)^2 + (n - 1)A_s(c_b - h + d)^2 \]  

(36)

\[ c_b = \frac{bh^2}{2} + (n - 1)A_s(h - d) \]  

(37)

\[ n = \frac{E_s}{E_c} \]  

(38)

\[ E_c = 4700\sqrt{f'_c} \]  

(39)
\[ f_r = 0.62 \sqrt{f'_c} \]  

(40)

where \( b \) = width of cross-section (mm), \( h \) = height of cross-section (mm), \( d \) = effective depth of steel reinforcement from extreme compression fibers (mm), \( c_b \) = centroid of transformed section from bottom fibers (mm), \( E_s \) = modulus of elasticity of steel (MPa), \( E_c \) = modulus of elasticity of concrete (MPa), \( f'_c \) = concrete compressive strength (MPa), \( n \) = modular ratio, and \( f_r \) = modulus of rupture of concrete (MPa).

Figure 100: Transformed section properties.

Table 11 shows the results that lead to determination of the centroid and moment of inertia of the transformed section for the 14 beam specimens. Table 12 presents the results of the cracking moment obtained using the theoretical approach, together with the experimental values. Note that the experimental values do not consider the extra load exerted on the beam due to the self-weight, which is negligibly small (less than 0.65 kN-m).

Before discussing the results, one should take into consideration that past experience has shown that the modulus of rupture of the concrete is highly volatile and has a wide scatter in relation to the concrete compressive strength. The ACI expression of the modulus of rupture represents a simple expression for practical use. Comparison of the results showed that the experimental cracking moments were close to the theoretical results in the eight beam specimens that were subjected to two loads near mid-span; with an average value for the ratio equal to 1.00. However, the experimentally obtained cracking moments were much higher than the theoretical values for the six beams that were subjected to a single load at 300 mm from one of the supports, with an average value for the ratio equal to 1.40. This is expected because for such beams traditional flexure theory does not apply because arching
action in the beam between the applied load and near support delays the formation of cracks. The structural design code takes this into consideration in the design of such beams and recommends the use of strut-and-tie model for their evaluation and design.

Table 11: Centroid and moment of inertia calculation summary.

<table>
<thead>
<tr>
<th>Label</th>
<th>$A_s$ (mm$^2$)</th>
<th>$f'_c$ (MPa)</th>
<th>$E_c$ (MPa)</th>
<th>$E_s$ (MPa)</th>
<th>$n=E_s/E_c$</th>
<th>$c_b$ (mm)</th>
<th>$I$ (mm$^4$)</th>
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<td>45.65</td>
<td>31755</td>
<td>197834</td>
<td>6.23</td>
<td>149</td>
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<td>31755</td>
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Table 12: Cracking moment calculation summary

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<th>$(M_{cr})_{exp}$ (kN-m)</th>
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6.1.3 Flexural strength. In this section, the flexural capacity of the beams at ultimate is determined from the experimental results, detailed theoretical modelling,
and ACI 318 code approach. The first procedure involves determining the highest load that the specimen can carry from the experimental load-deflection relationships and converting the load to bending moment based on the locations of the loads and boundary conditions. The second method is basically a sectional analysis based on the assumption that concrete crushes in compression at a strain 0.003, the stress-strain relationship of the concrete follows Thornfeldt model, and the stress in the steel at ultimate is equal to the yield strength for under-reinforced sections, as explained in Chapter 5. The last approach is based on the nominal capacity in the ACI 318 code which also assumes concrete to crush at a strain equal to 0.003 and steel reaches yielding if the section is under-reinforced, but approximates the compression in concrete by a Whitney block of intensity equal to 85% of the concrete compressive strength. The nominal bending moment capacity in the ACI 318 code is computed using the following equation:

\[ M_n = A_s f_y \left( d - \frac{a}{2} \right) \]  \hspace{1cm} (41)

The depth of Whitney stress block, \( a \), is obtained from the following expression after equating compression in the concrete to tension in the steel reinforcement [3]:

\[ a = \frac{A_s f_y}{0.85 f'_c b} \]  \hspace{1cm} (42)

where:
- \( f_y \) = flexural steel yield strength (MPa),
- \( A_s \) = flexural steel bars area (mm\(^2\)),
- \( f'_c \) = concrete compressive strength (MPa),
- \( d \) = effective depth of longitudinal reinforcement from the extreme fibers (mm),
- \( a \) = equivalent rectangular Whitney stress block depth, which is taken equal to \( \beta_1 c \)
- \( c \) = the depth of the neutral axis from the extreme compression fibers, and
- \( \beta_1 \) = variable that is related to the shape of the concrete stress-strain curve at ultimate.

In order to filter out the effect of the cross-section dimensions from the moment capacity, the “moment capacity ratio” concept is used in all of the analyses.
This ratio is obtained by dividing the flexural capacity by the $(bd^2)$, where $b$ is the width of the cross-section and $d$ is effective depth of the steel reinforcement from the extreme compression fibers. Results of the analysis using the three considered approaches are summarized in Tables 13, 14 and 15 for the experimental, theoretical and ACI 318 methods. Tables 14 and 15 also show the ratio of the experimental capacity to computed value. Note that 13 out of the tested beams exhibited flexural failure, culminating in crushing of concrete in the compression zone at the location of maximum bending moment. Only beam S-RH-NST, which was reinforced with mild steel rebars and subjected to a single concentrated load at 300 mm from the support, failed in shear after a major diagonal tension crack formed and opened up. Collapse of this beam occurred suddenly in a brittle fashion, without showing any sign of crushing of concrete.

As expected, the theoretical moment capacity values based on Thorfeldt’s model and Whitney block agree closely with each other. Moreover, both methods give moment capacities that simulate the experimental findings reasonably well. The ratio $(M_{exp}/M_{Theo})$ ranged between 0.82 and 1.14, whereas the ratio $(M_{exp}/M_{ACI})$ ranged between 0.83 and 1.16. Only 4 beams, of which 3 of them were reinforced with strands, exhibited lower moment capacity at ultimate by experimental testing than by calculations. The only beam that was reinforced with mild steel, in which the analytical methods under-estimated its flexural strength by 8-9% was S-RH-NST, which failed in shear. The mean value of the ratio $(M_{exp}/M_{Theo})$ for the 14 tested beams is 1.006, with a coefficient of variation equal to 0.0916. The corresponding mean and coefficient of variation of the ratio $(M_{exp}/M_{ACI})$ are equal to 1.015 and 0.0919, respectively.

The experimental results showed that the flexural capacity of the tested specimens is greatly affected by the amount of steel reinforcement, but only slightly affected by the concrete compressive strength and the loading condition, as shown in Figure 101 for the two-point loading and Figure 102 for the 1-point loading. When the effective mild steel reinforcement ratio increased by 78%, the corresponding bending moment capacity in the 2-points loading case increased by about 68% for both specimens made with $f'_c = 40$ MPa and $f'_c = 60$ MPa. For the same loading condition, when the effective high strength steel reinforcement ratio increased by 79%, the corresponding bending moment capacity increased by about 38% for the specimen.
made with $f'_c = 40$ MPa and 50% for the specimen made with $f'_c = 60$ MPa. The corresponding increase in the flexural capacity in the specimens that were subjected to a single load near the support is 45% when mild steel was utilized and 32% when high strength steel was employed.

Table 13: Experimental moment capacity results at ultimate.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>$P_{exp}$ (KN)</th>
<th>$M_{exp}$ (kN.m)</th>
<th>$M_{exp}/(bd^2)$ (MPa)</th>
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<td>F-SL-40</td>
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<td>6.60</td>
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<td>282.44</td>
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<td>6.79</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>232.01</td>
<td>75.40</td>
<td>5.58</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>246.78</td>
<td>56.12</td>
<td>4.15</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>218.60</td>
<td>49.71</td>
<td>3.68</td>
</tr>
<tr>
<td>S-RH-NST*</td>
<td>357.93</td>
<td>81.39</td>
<td>6.02</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>288.18</td>
<td>65.53</td>
<td>4.85</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>244.06</td>
<td>55.50</td>
<td>4.11</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>231.76</td>
<td>52.70</td>
<td>3.90</td>
</tr>
</tbody>
</table>

* This beam did not exhibit flexural failure.

Table 14: Theoretical moment capacity results.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>$M_{Theo}$ (kN-m)</th>
<th>$M_{Theo}/(bd^2)$ (MPa)</th>
<th>$M_{exp}/M_{Theo}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>52.12</td>
<td>3.855</td>
<td>1.02</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>45.80</td>
<td>3.387</td>
<td>1.09</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>88.89</td>
<td>6.574</td>
<td>1.00</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>78.94</td>
<td>5.838</td>
<td>0.87</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>53.11</td>
<td>3.928</td>
<td>1.04</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>46.49</td>
<td>3.438</td>
<td>1.08</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>91.64</td>
<td>6.778</td>
<td>1.00</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>81.05</td>
<td>5.994</td>
<td>0.93</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>52.12</td>
<td>3.855</td>
<td>1.08</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>45.80</td>
<td>3.387</td>
<td>1.09</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>88.89</td>
<td>6.574</td>
<td>0.92</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>78.94</td>
<td>5.838</td>
<td>0.83</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>52.12</td>
<td>3.855</td>
<td>1.06</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>45.80</td>
<td>3.387</td>
<td>1.15</td>
</tr>
</tbody>
</table>
Table 15: ACI-based moment capacity results.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>$M_{ACI}$ (kN-m)</th>
<th>$M_{ACI}/(bd^2)$ (MPa)</th>
<th>$M_{exp}/M_{ACI}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>52.0</td>
<td>3.85</td>
<td>1.02</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>45.6</td>
<td>3.37</td>
<td>1.09</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>88.63</td>
<td>6.56</td>
<td>1.01</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>78.72</td>
<td>5.82</td>
<td>0.87</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>52.87</td>
<td>3.91</td>
<td>1.04</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>46.26</td>
<td>3.42</td>
<td>1.08</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>91.36</td>
<td>6.76</td>
<td>1.00</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>79.45</td>
<td>5.88</td>
<td>0.95</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>52.0</td>
<td>3.85</td>
<td>1.08</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>45.6</td>
<td>3.37</td>
<td>1.09</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>88.63</td>
<td>6.56</td>
<td>0.92</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>78.72</td>
<td>5.82</td>
<td>0.83</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>52.0</td>
<td>3.85</td>
<td>1.07</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>45.6</td>
<td>3.37</td>
<td>1.16</td>
</tr>
</tbody>
</table>

Figure 101: Nominal flexural capacities for the flexural loaded samples.

Figure 102: Nominal flexural capacities for the combined loaded samples.
6.1.4 Flexural behavior of beams. The flexural behavior of the tested beams is best judged based on the obtained load-deflection relationships. Such a relationship clearly points out the stiffness, strength and ductility of the structural member. To determine the effect of an increase in the steel reinforcement ratio on the load-deflection curve, one needs to test two similar beams in all aspects except in the amount of steel reinforcement. The load-deflection curves were plotted for the pair of samples with similar concrete compressive strength but different amount of steel for the specimens subjected to two-point loading, as shown in Figure 103 and Figure 104. The corresponding curves for the specimens subjected to a 1-point loading are presented in Figure 105. Note that for the latter case, two relationships are provided for the specimens with low steel reinforcement, one for specimens with stirrups at 100 mm spacing (labeled 100 in the graph) and the other without stirrups (denoted by NST).

Figure 103: Load-deflection relationships Group-1 beams with $f'_c = 40$ MPa.

Figure 104: Load-deflection relationships for Group-1 beams with $f'_c = 60$ MPa.
The experimental results show significant differences in the flexural behavior between mild steel rebar reinforced specimens and high steel strand reinforced specimens. In general, the ductility of the specimens with mild steel is higher than the ductility in the corresponding specimens with high strength steel for both conditions of loading. Also, when considering one type of steel, the increase in the amount of reinforcement causes an increase in the displacement at yield and also a decrease in the displacement at ultimate. These two findings contribute to the reduction in the ductility of the specimen with higher reinforcement. As expected, the ductility of the specimens with higher strength concrete is larger than the corresponding ductility of the specimens with lower strength concrete due to the smaller distance between the neutral axis and extreme compression fibers in the former specimens. Moreover, the stiffness, which is represented by the slope of the ascending branch of the curve, of the beams with larger steel reinforcement ratio is higher for both types of reinforcement. This is because the load at yield in the specimens with higher reinforcement is much larger than their counterparts with lower reinforcement, while the deformation at yield is almost the same. The stiffness of the beam specimens decreases continuously with loading in strands reinforced sections, whereas it is almost the same up to yielding in steel reinforced section. This is due to the distinct stress-strain relationships of the two steel materials. At the onset of yielding, the stiffness of the strands reinforced specimens is much lower than the corresponding stiffness of the rebar reinforced specimens. The findings observed in the 2-point load condition are valid for the 1-point case, except for specimen S-RH-NST which failed in shear in a brittle fashion due to its insufficient flexural reinforcement. Figure 105 shows that the presence of stirrups helps in slightly increasing the ductility by confining the concrete core, but without impacting the strength.

![Load-deflection relationships](image-url)

Figure 105: Load-deflection relationships for 1 points loaded samples.
6.1.5 Loading condition effect on beams. The effect of loading condition on the bending moment capacity of the tested specimens was evaluated for beams with similar reinforcement ratios and concrete compressive strengths but different condition of loading, as shown in Figure 106. Note that the specimens designated first with the latter F are 1900 mm long between the supports and have a region of large constant bending moment and zero shear between the two applied loads. On the other hand, the specimens designated first with the latter S have 1240 mm span and are subjected to large bending moment together with large shear force at the critical location under the applied load. The results indicate higher ductility in the F-specimens than in the corresponding S-specimens. From Chapter 5, it was concluded that although the deflection ductility is higher for a short beam than for a long beam, the effect of the presence of two loads on the span, compared with one load, is much more pronounced; hence, the higher ductility of the F-specimens compared with the S-specimens as depicted by the experimental results in Figure 80. The load-deflection curves of the short S-specimens showed higher stiffness than their long F-counterparts because short beams subjected to a load exerted near the support deflect much less than equivalent long beams subjected to two loads applied near mid-span.

![Figure 106: Effect of loading condition on load deflection behavior.](image-url)
6.1.6 Reinforcement efficiency. Reinforcement efficiency, $R_E$, is used to determine how efficient the cross-section dimensions and longitudinal steel reinforcement in contributing to the flexural capacity. As stated earlier in Chapter 5, $R_E$ is equal to the bending moment capacity ratio, $M/(bd^2)$, divided by the effective flexural steel reinforcement ratio, $\rho$. Table 16 shows the calculated reinforcement efficiency for all 14 beams based on the experimental results, theoretical approach using Thornfeldt concrete model, and the ACI 318 code. The findings of all beams except those containing stirrups (S-RL-100 and S-SL-100) are presented graphically in Figure 107, Figure 108 and Figure 109. Since each pair of rebars and strands reinforced beams were designed to carry almost the same bending moment at ultimate by proportioning the area of the strands according to the ratio of the yield strengths of the steel used, the reinforcement efficiency of the strand reinforced beams was 2.79-3.42 times that of rebar reinforced beams. This is expected since the steel reinforced ratios for the strands reinforced beams were smaller than the corresponding rebars reinforced beams by almost the same ratio.

<table>
<thead>
<tr>
<th>Designation</th>
<th>$\rho$ (%)</th>
<th>$(R_E)_{exp}$</th>
<th>$(R_E)_{theo}$</th>
<th>$(R_E)_{ACI}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>0.77</td>
<td>508.04</td>
<td>498.67</td>
<td>497.51</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>0.21</td>
<td>1738.41</td>
<td>1600.49</td>
<td>1593.54</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>1.38</td>
<td>479.44</td>
<td>477.47</td>
<td>476.10</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>0.38</td>
<td>1339.89</td>
<td>1537.90</td>
<td>1533.63</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>0.77</td>
<td>526.05</td>
<td>508.14</td>
<td>505.84</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>0.21</td>
<td>1752.88</td>
<td>1624.77</td>
<td>1616.60</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>1.38</td>
<td>493.09</td>
<td>492.29</td>
<td>490.76</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>0.38</td>
<td>1469.04</td>
<td>1579.12</td>
<td>1547.85</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>0.77</td>
<td>536.91</td>
<td>498.67</td>
<td>497.51</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>0.21</td>
<td>1737.14</td>
<td>1600.49</td>
<td>1593.54</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>1.38</td>
<td>437.22</td>
<td>477.47</td>
<td>476.10</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>0.38</td>
<td>1276.70</td>
<td>1537.90</td>
<td>1533.63</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>0.77</td>
<td>531.00</td>
<td>498.67</td>
<td>497.51</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>0.21</td>
<td>1841.77</td>
<td>1600.49</td>
<td>1593.54</td>
</tr>
</tbody>
</table>

The results in Figure 107, Figure 108 and Figure 109 show that the reinforcement efficiency in conventionally reinforced beams is almost independent of the reinforcement ratio, varying within a narrow range of 437-537 MPa experimentally, 477-508 MPa theoretically, and 476-506 MPa using the ACI 318.
code. On the other hand, the reinforcement efficiency obtained experimentally for strands reinforced beams greatly fluctuated with the reinforcement ratio, averaging 1742 MPa for the beams with low reinforcement and 1362 MPa for the beams with high reinforcement. This pattern was not observed in the two analytical approaches where both the theoretical and ACI 318 methods yielded similar uniform results averaging around 1575 MPa.

Figure 107: Experimental results of the steel reinforcement efficiency.

Figure 108: Theoretical results of the steel reinforcement efficiency.
6.1.7 Curvature ductility analysis. The curvature ductility index, $\mu_c$, is determined experimentally for each beam using the recorded strain values in the concrete and steel reinforcement at service and ultimate conditions. Service condition in this case is defined when the flexural steel reinforcement reached 60% of the yield stress. It is chosen in this study instead of the deformation at yield because the yield point of high strength strands is not clearly identified on the stress-strain curve of the material. To determine the curvature at service, the compressive strain in the concrete was obtained when the tensile strain in the steel reaches $0.6f_y$, in which $f_y$ is 525 MPa for the rebars and 1670 for the strands. Once the strain in the concrete is obtained at that instant, a linear strain diagram can be assumed and a similar triangles approach allows for determining the location of the neutral axis (i.e. zero strain) from the extreme compressive fibers of the concrete. Note that the strain gauge on the concrete was placed at 30 mm from the top surface; therefore, similar triangles is used to find the compressive strain in the concrete at the extreme fibers. The curvature is finally computed by dividing the strain in the extreme concrete fibers by the neutral axis distance from that point, as shown in Figure 110. Table 17 shows the results of the strain values in the steel and at the extreme top fibers of the concrete at service. Also, included in the table are the load applied on the beams at that instant, together with the location of the neutral axis, $c$, and resulting curvature.

![Figure 109: ACI-based results of the steel reinforcement efficiency.](image)

<table>
<thead>
<tr>
<th>Reinforcement Efficiency (MPa)</th>
<th>Low steel</th>
<th>Low strands</th>
<th>High steel</th>
<th>High strands</th>
<th>Low steel</th>
<th>Low strands</th>
<th>High steel</th>
<th>High strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 MPA</td>
<td>497.51</td>
<td>476.10</td>
<td>505.84</td>
<td>490.76</td>
<td>497.51</td>
<td>476.10</td>
<td>505.84</td>
<td>490.76</td>
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<tr>
<td>60 MPA</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Combined loaded (40 Mpa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Using the experimental results to determine the curvature at ultimate requires determining the compressive strain in the concrete and tensile strain in the steel at peak load. Once again, a linear strain diagram was assumed and similar triangles were used to find the location of the neutral axis after converting the concrete strain gauge reading to the extreme compressive fibers of the concrete. Table 18 shows the strains in the steel and concrete at ultimate, together with the location of the neutral axis, c, and resulting curvature. Also included in the table is the curvature ductility index. Note that specimen S-RH-NST failed in shear in a brittle fashion because its flexural
strength was higher than its shear capacity; hence, no values are provided in Table 67 for this beam.

The curvature ductility analysis that is based on the experimental results in Table 18 shows indices between 2.99 and 12.74. It confirms that the beams with low steel reinforcement ratio exhibit more ductile behavior than their counterpart with high steel reinforcement ratio mainly due to the differences in strain values in the steel. The strands reinforced beams consistently exhibited less curvature ductility than the equivalent rebar reinforced beams that have about the same capacity.

Table 18: Experimental evaluation of curvature ductility at ultimate.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load at ultimate (KN)</th>
<th>Steel strain (µ strain)</th>
<th>Top concrete strain (µ strain)</th>
<th>c (mm)</th>
<th>$\phi_u$ (10^-6/mm)</th>
<th>$(\mu_c)^{Exp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>163.4</td>
<td>20395</td>
<td>3762</td>
<td>40.49</td>
<td>92.91</td>
<td>10.65</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>153.1</td>
<td>18412</td>
<td>3752</td>
<td>44.01</td>
<td>85.24</td>
<td>3.63</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>274.6</td>
<td>10173</td>
<td>2577</td>
<td>52.55</td>
<td>49.04</td>
<td>5.66</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>211.6</td>
<td>15418</td>
<td>3064</td>
<td>43.10</td>
<td>71.08</td>
<td>2.99</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>169.2</td>
<td>25976</td>
<td>4894</td>
<td>41.22</td>
<td>118.73</td>
<td>12.74</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>154.3</td>
<td>25407</td>
<td>4704</td>
<td>40.62</td>
<td>115.81</td>
<td>4.87</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>282.4</td>
<td>13643</td>
<td>3199</td>
<td>49.39</td>
<td>64.78</td>
<td>7.49</td>
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<td>F-SH-60</td>
<td>232.0</td>
<td>15126</td>
<td>3688</td>
<td>50.97</td>
<td>72.36</td>
<td>3.06</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>246.8</td>
<td>19324</td>
<td>3900</td>
<td>43.66</td>
<td>89.32</td>
<td>11.25</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>218.6</td>
<td>15196</td>
<td>3524</td>
<td>48.94</td>
<td>72.00</td>
<td>3.04</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>244.1</td>
<td>18584</td>
<td>3949</td>
<td>45.57</td>
<td>86.67</td>
<td>11.34</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>231.7</td>
<td>16144</td>
<td>3710</td>
<td>48.59</td>
<td>76.36</td>
<td>3.11</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>357.9</td>
<td>2348</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>-</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>288.2</td>
<td>15627</td>
<td>3480</td>
<td>47.36</td>
<td>73.49</td>
<td>3.03</td>
</tr>
</tbody>
</table>

The curvature ductility indices are next determined for the same 14 beams using the theoretical approach involving Thornfeldt concrete model and an elastoplastic steel model based on yield, as shown in Figure 111. Full details of this analysis are included in Table 19, including the strain at top of concrete beam at service, the depth of neutral axis from compression fibers at service, the strain in the steel reinforcement at ultimate and the location of neutral axis at ultimate. Strain in steel at service is not provided because it is equal to 60% of the yield strain, and strain in the concrete at ultimate is not given since it is equal to 0.003 for all specimens.
Calculations of the theoretical values of the curvature at service and ultimate are accomplished following an iteration process programmed in Matlab. For the most part, the theoretical results agree very well with the experimental ones.

![Graph showing concrete and steel stress-strain curves](image)

**Figure 111: Material models used in the theoretical curvature analysis.**

**Table 19: Curvature analysis details by the theoretical approach.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete strain at service (µ strain)</th>
<th>c at service (mm)</th>
<th>$\phi_s$ \left(10^{-6}/\text{mm}\right)$</th>
<th>Steel strain at ultimate (µ strain)</th>
<th>c at ultimate (mm)</th>
<th>$\phi_u$ \left(10^{-6}/\text{mm}\right)$</th>
<th>($\mu_c$)$_{\text{Theo}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>0.00060</td>
<td>72</td>
<td>8.39</td>
<td>0.0203</td>
<td>33</td>
<td>8.98</td>
<td>10.71</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>0.00093</td>
<td>40.5</td>
<td>23.1</td>
<td>0.0238</td>
<td>29.1</td>
<td>103</td>
<td>4.47</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>0.00085</td>
<td>91.1</td>
<td>9.34</td>
<td>0.0101</td>
<td>59.4</td>
<td>50.5</td>
<td>5.41</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>0.00129</td>
<td>52.9</td>
<td>24.5</td>
<td>0.0120</td>
<td>52</td>
<td>57.7</td>
<td>2.36</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>0.00056</td>
<td>67.9</td>
<td>8.21</td>
<td>0.0283</td>
<td>24.9</td>
<td>120</td>
<td>14.67</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>0.00087</td>
<td>38</td>
<td>22.8</td>
<td>0.0329</td>
<td>21.7</td>
<td>138</td>
<td>6.06</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>0.00078</td>
<td>86.1</td>
<td>9.07</td>
<td>0.0146</td>
<td>44.3</td>
<td>67.7</td>
<td>7.47</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>0.00119</td>
<td>49.5</td>
<td>24.1</td>
<td>0.0171</td>
<td>38.3</td>
<td>77.3</td>
<td>3.21</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>0.00060</td>
<td>72</td>
<td>8.39</td>
<td>0.0204</td>
<td>33</td>
<td>89.8</td>
<td>10.71</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>0.00093</td>
<td>40.5</td>
<td>23.1</td>
<td>0.0238</td>
<td>29.1</td>
<td>103</td>
<td>4.47</td>
</tr>
<tr>
<td>S-RL-100*</td>
<td>0.00060</td>
<td>72</td>
<td>8.39</td>
<td>0.0204</td>
<td>33</td>
<td>89.8</td>
<td>10.71</td>
</tr>
<tr>
<td>S-SL-100*</td>
<td>0.00093</td>
<td>40.5</td>
<td>23.1</td>
<td>0.0238</td>
<td>29.1</td>
<td>103</td>
<td>4.47</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>0.00085</td>
<td>91.1</td>
<td>9.34</td>
<td>0.0101</td>
<td>59.4</td>
<td>50.5</td>
<td>5.41</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>0.00129</td>
<td>52.9</td>
<td>24.5</td>
<td>0.0120</td>
<td>52</td>
<td>57.7</td>
<td>2.36</td>
</tr>
</tbody>
</table>

* The theoretical analysis of these beams neglects the effect of stirrups

Table 20 below shows a summary of the experimental and theoretical curvature ductility indices and the difference between them. The ratio of experimental
to theoretical curvature indices for the 13 relevant specimens’ averages about 1.077, with a coefficient of variation of 20.3%. The theoretical results are very close to the experimental findings, considering the assumptions made in the theoretical analysis with regard to material properties and linearity of the strain diagram. As expected, the accuracy is higher with the mild steel than with the high strength steel strands. In general, the experimental results provided much lower values than their counterparts of the theoretical results in the case of 1 point loaded beams.

**Table 20: Summary of the experimental and theoretical ductility results.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>((\mu_c)_{\text{Theo}})</th>
<th>((\mu_c)_{\text{Exp}})</th>
<th>((\mu_c)<em>{\text{Theo}} / (\mu_c)</em>{\text{Exp}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>10.71</td>
<td>10.65</td>
<td>1.01</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>4.47</td>
<td>3.63</td>
<td>1.23</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>5.41</td>
<td>5.66</td>
<td>0.96</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>2.36</td>
<td>2.99</td>
<td>0.79</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>14.67</td>
<td>12.74</td>
<td>1.15</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>6.06</td>
<td>4.87</td>
<td>1.24</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>7.47</td>
<td>7.49</td>
<td>1.00</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>3.21</td>
<td>3.06</td>
<td>1.05</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>10.71</td>
<td>11.25</td>
<td>0.95</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>4.47</td>
<td>3.04</td>
<td>1.47</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>10.71</td>
<td>11.34</td>
<td>0.94</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>4.47</td>
<td>3.11</td>
<td>1.44</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>5.41</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>2.36</td>
<td>3.03</td>
<td>0.78</td>
</tr>
</tbody>
</table>

**6.1.8 Deflection ductility analysis.** Deflection ductility is another measure of the deformation capacity of beams under flexural loads. From the experimental testing, the deflection ductility was determined directly from the load-deflection curve by dividing the deflection at ultimate by the corresponding deflection at service (when the strain gauge reading for the steel reaches 60% of yield). Table 21 shows details of the experimental deflection ductility analysis for the tested specimens. The theoretical computations of the deflection ductility are much more involved in chapter 5, because they require information about moment curvature relationships, plastic hinge size, loading condition, and beam length. Using the theoretical curvature ductility information in the previous section, one can determine the theoretical deflection ductility index. Table 22 shows details of the theoretical deflection ductility analysis.
Table 21: Details of the experimental deflection ductility analysis.

<table>
<thead>
<tr>
<th>Designation</th>
<th>Load at service (KN)</th>
<th>Load at ultimate (KN)</th>
<th>Defl. at service (mm)</th>
<th>Defl. at ultimate (mm)</th>
<th>( \mu_d(\text{Exp}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>95.8</td>
<td>163.4</td>
<td>4.49</td>
<td>34.53</td>
<td>7.69</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>71.8</td>
<td>153.1</td>
<td>8.69</td>
<td>33.25</td>
<td>3.83</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>142.3</td>
<td>274.6</td>
<td>4.72</td>
<td>15.60</td>
<td>3.31</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>86.4</td>
<td>211.6</td>
<td>8.45</td>
<td>26.84</td>
<td>3.18</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>99.3</td>
<td>169.2</td>
<td>4.24</td>
<td>26.55</td>
<td>6.26</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>65.42</td>
<td>154.3</td>
<td>8.29</td>
<td>31.25</td>
<td>3.77</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>123.4</td>
<td>282.4</td>
<td>3.75</td>
<td>15.26</td>
<td>4.07</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>111.7</td>
<td>232.0</td>
<td>8.09</td>
<td>27.33</td>
<td>3.38</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>135.3</td>
<td>246.8</td>
<td>1.31</td>
<td>10.02</td>
<td>7.65</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>151.1</td>
<td>218.6</td>
<td>3.33</td>
<td>11.30</td>
<td>3.39</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>121.9</td>
<td>244.1</td>
<td>1.07</td>
<td>8.27</td>
<td>7.73</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>158.7</td>
<td>231.7</td>
<td>4.16</td>
<td>19.17</td>
<td>4.61</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>267.1</td>
<td>357.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>184.0</td>
<td>288.2</td>
<td>3.37</td>
<td>10.67</td>
<td>3.17</td>
</tr>
</tbody>
</table>

Table 22: Details of the theoretical deflection ductility analysis.

<table>
<thead>
<tr>
<th>Designation</th>
<th>Deflection at service (mm)</th>
<th>Deflection at ultimate (mm)</th>
<th>((\mu_d)_{\text{Theo}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-RL-40</td>
<td>3.20</td>
<td>29.40</td>
<td>9.20</td>
</tr>
<tr>
<td>F-SL-40</td>
<td>8.79</td>
<td>35.36</td>
<td>4.02</td>
</tr>
<tr>
<td>F-RH-40</td>
<td>3.56</td>
<td>17.08</td>
<td>4.80</td>
</tr>
<tr>
<td>F-SH-40</td>
<td>9.31</td>
<td>21.17</td>
<td>2.27</td>
</tr>
<tr>
<td>F-RL-60</td>
<td>3.13</td>
<td>39.07</td>
<td>12.49</td>
</tr>
<tr>
<td>F-SL-60</td>
<td>8.69</td>
<td>46.44</td>
<td>5.34</td>
</tr>
<tr>
<td>F-RH-60</td>
<td>3.45</td>
<td>22.49</td>
<td>6.51</td>
</tr>
<tr>
<td>F-SH-60</td>
<td>9.16</td>
<td>27.33</td>
<td>2.98</td>
</tr>
<tr>
<td>S-RL-NST</td>
<td>0.79</td>
<td>5.25</td>
<td>6.65</td>
</tr>
<tr>
<td>S-SL-NST</td>
<td>2.17</td>
<td>7.14</td>
<td>3.29</td>
</tr>
<tr>
<td>S-RL-100</td>
<td>0.79</td>
<td>5.25</td>
<td>6.65</td>
</tr>
<tr>
<td>S-SL-100</td>
<td>2.17</td>
<td>7.14</td>
<td>3.29</td>
</tr>
<tr>
<td>S-RH-NST</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S-SH-NST</td>
<td>2.30</td>
<td>4.95</td>
<td>2.15</td>
</tr>
</tbody>
</table>
The relationship between curvature and deflection ductility indices is shown in Figure 112 for the 8 specimens subjected to 2-point loading and Figure 113 for the remaining 5 specimens subjected to a point. The type of loading affects the structural behavior of beams because 2-point loading results in a constant moment region within the central part of the beam with zero-shear, whereas 1-point loading results in a region between the load and near support subjected to large shear together with moment. Note that the specimen that was subject to a point load and failed in shear, not in flexure, is omitted from the analysis. The figures include both experimental and theoretical results, and they show the same linear ascending trend for the experimental results and theoretical findings. With regard to the specimens that were subjected to 2-point loading, the theoretical results generally overpredicted the deflection ductility when compared to the experimental results, although the two gave close results when the curvature ductility was in the range of 2-6. The opposite is true for the beams that were loaded with a single concentrated load at 300 mm from near support; in this case the theoretical results underpredicted the deflection ductility when compared with the experimental results. This could be due to the interaction between shear and moment in these specimens, which was not accounted for in the theoretical approach.

![Curvature versus deflection ductility indices for 2 loads beams.](image)
6.2 Axial Compression Specimens

In this section, the strength and ductility of concentrically loaded columns specimens in compression are studied for two specimens longitudinally reinforced with high strength strands and corresponding two specimens with mild steel rebars. The four column specimens have 200 mm square cross-section, 1000 mm length, and made with concrete compressive strength equal to 45.65 MPa. Table 23 shows cross-section and reinforcement details for the 4 tested columns.

Table 23: Cross section and reinforcement details of the tested columns.

<table>
<thead>
<tr>
<th>Label</th>
<th>Long. Steel</th>
<th>$A_g$ (mm$^2$)</th>
<th>Long. steel $\rho_g$ (%)</th>
<th>Ties</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-RL-100</td>
<td>4 No. 16 bars</td>
<td>40,000</td>
<td>2.01</td>
<td>No. 8 @ 100mm</td>
</tr>
<tr>
<td>C-RH-100</td>
<td>8 No 16 bars</td>
<td>40,000</td>
<td>4.02</td>
<td>No. 8 @ 100mm</td>
</tr>
<tr>
<td>C-SL-100</td>
<td>Four 9.5xDiam. strands</td>
<td>40,000</td>
<td>0.55</td>
<td>No. 8 @ 100mm</td>
</tr>
<tr>
<td>C-SH-100</td>
<td>Eight 9.5-Diam. strands</td>
<td>40,000</td>
<td>1.10</td>
<td>No. 8 @ 100mm</td>
</tr>
</tbody>
</table>

6.2.1 Axial capacity. Experimentally, the axial compressive load capacity of the tested columns at ultimate can be detected from the load-deflection curves of the specimens at the peak. The theoretical axial capacity of the columns under ideal concentric loading is determined using the equation provided by ACI 318-14 code as follows:

$$P'_n = 0.85 f'_c (A_g - A_{st}) + f_y A_{st}$$  \hspace{1cm} (43)
where:

\[ A_g = \text{gross area of the column (mm}^2\text{),} \]

\[ f'_c = \text{concrete compressive strength (MPa),} \]

\[ A_{st} = \text{Total longitudinal reinforcement area (mm}^2\text{), and} \]

\[ f_y = \text{steel yield strength (MPa).} \]

In order to account for minimum eccentricity of the axial load on the column, the ACI 318 code reduces the idea capacity computed by Equation 9 by 20%. In the theoretical part of the study, the 20% reduction is not used because it was felt that the columns have levelled surfaces and were centered accurately inside the UTM machine with respect to the actuator head.

Table 24 below shows a summary of the compression capacity results for the tested columns. The experimental and theoretical results were almost the same for the case of rebars reinforced concrete columns; however, the difference between the two results for the case of high strength steel strand reinforced columns was in the order of 8-9%. If the minimum eccentricity factor is considered in the calculation of the nominal axial capacity of the columns, the experimental-to-theoretical capacity ratio for the rebars reinforced sections would have been around 1.25, and for the strands reinforced sections 1.14. The reason why the experimental results of the strands reinforced sections are lower than their corresponding rebars reinforced sections is because the strands are small in size and are made of 7 wires, which makes them flexible and prone to buckling at high loads once the concrete cover spalls off.

**Table 24: Experimental and theoretical results of axial load capacity.**

<table>
<thead>
<tr>
<th>Label</th>
<th>Experimental axial capacity, P&lt;sub&gt;Exp&lt;/sub&gt; (kN)</th>
<th>Theoretical axial capacity, P&lt;sub&gt;Theo&lt;/sub&gt; (kN)</th>
<th>P&lt;sub&gt;Exp&lt;/sub&gt; / P&lt;sub&gt;Theo&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-RL-100</td>
<td>1976</td>
<td>1943</td>
<td>1.02</td>
</tr>
<tr>
<td>C-RH-100</td>
<td>2291</td>
<td>2334</td>
<td>0.98</td>
</tr>
<tr>
<td>C-SL-100</td>
<td>1730</td>
<td>1911</td>
<td>0.91</td>
</tr>
<tr>
<td>C-SH-100</td>
<td>2076</td>
<td>2270</td>
<td>0.92</td>
</tr>
</tbody>
</table>

While utmost care was exercised in order to match the force carried by the longitudinal reinforcement in the rebar reinforced specimens and the strand reinforced specimens, these two forces in the rebar-strand pairs were not equal from the
experimental results. In order to find out the effect of the reinforcement type on the load carrying capacity of the concrete in the columns, the steel carried by the reinforcement must be filtered out. This can easily be done by determining the strain in the longitudinal reinforcement at the instant of maximum capacity, and using the stress-strain relationships of the steel to find out the stress in the steel at that instant. From the stress in the reinforcement, one can compute the force and it can be deducted from the maximum total load on the specimen to give the load on the concrete. Table 25 shows the strains and corresponding stresses in the longitudinal steel reinforcement at ultimate for the 4 tested specimens. Also, included in the table are the force carried by the steel and concrete, separately. As can be seen from the results, the force carried by the concrete in the highly-reinforced columns is lower than that in the lightly-reinforced columns because more of the applied load is supported by the steel. The results also indicate that rebars confine the concrete a little better than strands, due to their greater rigidity and stiffness. On average, the concrete loses 8-12% of its theoretical capacity when strands are used instead of rebars.

### Table 25: Axial compression carried by the concrete at ultimate.

<table>
<thead>
<tr>
<th>Label</th>
<th>Ultimate strain in steel (µ)</th>
<th>Stress in steel (MPa)</th>
<th>Load carried by Steel (kN)</th>
<th>Load Carried by concrete (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-RL-100</td>
<td>1580</td>
<td>556</td>
<td>447</td>
<td>1529</td>
</tr>
<tr>
<td>C-RH-100</td>
<td>1704</td>
<td>563</td>
<td>905</td>
<td>1386</td>
</tr>
<tr>
<td>C-SL-100</td>
<td>1485</td>
<td>1758</td>
<td>387</td>
<td>1343</td>
</tr>
<tr>
<td>C-SH-100</td>
<td>2943</td>
<td>1822</td>
<td>802</td>
<td>1274</td>
</tr>
</tbody>
</table>

### 6.2.2 Reinforcement efficiency.

Reinforcement efficiency was determined in columns as the compression capacity divided by the gross reinforcement ratio and gross cross-sectional area (ρ_bA_g). This is to change the units from force to pressure, as was the case in flexure when the moment capacity was divided by (ρ_bd_l^2). Table 26 below describes the reinforcement efficiency results of the specimens. There was no influence regarding the reinforcement ratio effect on reinforcement efficiency in both types of reinforcement. Selecting lower or higher reinforcement ratio of strands in this case will not have an economical aspect as in the case of beams. The reinforcement efficiency may play a role in the eccentric loaded columns, where ductility of reinforcement becomes important.
Table 26: Results of reinforcement efficiency for axially loaded samples.

<table>
<thead>
<tr>
<th>Label</th>
<th>( \rho_g (%) )</th>
<th>( A_g (\text{mm}^2) )</th>
<th>( (\text{RE})_{\text{Exp}} )</th>
<th>( (\text{RE})_{\text{Theo}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-RL-100</td>
<td>2.01</td>
<td>40,000</td>
<td>49.40</td>
<td>48.58</td>
</tr>
<tr>
<td>C-RH-100</td>
<td>4.02</td>
<td>40,000</td>
<td>57.28</td>
<td>47.78</td>
</tr>
<tr>
<td>C-SL-100</td>
<td>0.55</td>
<td>40,000</td>
<td>43.25</td>
<td>58.35</td>
</tr>
<tr>
<td>C-SH-100</td>
<td>1.10</td>
<td>40,000</td>
<td>51.90</td>
<td>56.76</td>
</tr>
</tbody>
</table>

6.2.3 Deflection ductility. In this section, the ductility of the columns subjected to concentric axial compression is determined experimentally using data from the displacement of the actuator head at the top of each column. The ductility is computed as the ratio of the deflection at the top of the column at ultimate load, to deflection at the top of the column when the load reaches 85% of the peak capacity on the ascending branch of the load-deflection curve. Table 27 shows the important parameters for calculating the ductility of columns. The results show that the deflection ductility for all columns closely averages around 1.46. Also, the ductility is not compromised when using strands instead of rebars, in fact it is slightly better in the higher reinforcement ratio case.

Table 27: Deflection ductility analysis details for columns.

<table>
<thead>
<tr>
<th>Label</th>
<th>Max load (KN)</th>
<th>Deflection Prior to peak (mm)</th>
<th>Deflection at peak (mm)</th>
<th>Ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-RL-100</td>
<td>1975.96</td>
<td>0.324</td>
<td>0.500</td>
<td>1.54</td>
</tr>
<tr>
<td>C-RH-100</td>
<td>2291</td>
<td>0.328</td>
<td>0.425</td>
<td>1.30</td>
</tr>
<tr>
<td>C-SL-100</td>
<td>1729.9</td>
<td>0.498</td>
<td>0.754</td>
<td>1.51</td>
</tr>
<tr>
<td>C-SH-100</td>
<td>2076.06</td>
<td>0.788</td>
<td>1.177</td>
<td>1.49</td>
</tr>
</tbody>
</table>

6.3 Design Recommendations

Based on the results of the theoretical and experimental parts of the study, some design recommendations are provided for concrete sections reinforced with untensioned steel strands. The study showed that nominal bending moment capacity in this case can still be determined for under-reinforced sections in a similar way to mild steel reinforced beams using the ACI 318 equation:

\[
M_n = A_s f_y \left( d - \frac{a}{2} \right)
\]  

(44)
For the sake of determining the strength reduction factor, the strain in the flexural steel reinforcement is also obtained from the same equation that is used for mild steel:

\[ \epsilon_t = 0.003 \frac{d_t - c}{c} \]  \hspace{1cm} (45)

In order to ensure a minimum ductile behavior of newly designed flexural members, the ACI 318-14 code requires the strain in the extreme layer of the tension steel to comply with some limits. The existing limits in the ACI 318 code do not account for unusual types of reinforcement like un-stretched steel strands. Since high strength steel strands with Grade 1860 MPa have a different ductility behavior than mild steel, new strain limits are developed in this study and included in Figure 114:

![Figure 114: Strain-reduction factors graph for the strands reinforced beams.](image)

The behavior of un-tensioned steel strands in concrete columns subjected to pure compression is not that different in terms of ductility. However, the strength of such columns is slightly compromised compared to corresponding conventionally reinforced columns. The nominal axial capacity of columns under ideal concentric loading can still be determined by the ACI 318 code equation but with the inclusion of a reduction factor as follows:

\[ P'_{n} = \alpha [0.85f'c(A_g - A_{st}) + f_y A_{st}] \]  \hspace{1cm} (35)
in which $\alpha$ is a reduction factor equal to 0.9 introduced to the cover the 9% deficiency witnessed in the capacity of strands reinforced concrete columns compared to the theoretical prediction. It should be noted that the study on compression is very limited because it does not cover a wide range of concrete strengths, cross-section dimensions, and reinforcement ratios.
Chapter 7: Summary and Conclusions

7.1 Summary

In this study, high strength steel seven-wire strands with tensile strength of 1860 MPa are used as longitudinal reinforcement in reinforced beams and columns without prestressing. The study uses both experimental laboratory tests and computational studies to check the feasibility of this new type of reinforcement in concrete structures. The experimental program considers 14 flexural tests, of which eight are long subjected to 2-point loading and six short subjected to a single load, and four axial compression tests. The cross-section of the beams is 200 mm wide and 300 mm deep, while the cross-section of the columns is 200 mm square. Other variables that are investigated in the test program include the amount of steel reinforcement, concrete compressive strength, location of the load on the beam specimens, and presence of stirrups. Instrumentation of the specimens is accomplished through strain gauges on the concrete surfaces and steel reinforcement and LVDTs on the members. In all cases, pairs of specimens are reinforced by either steel rebars or unstretched strands in order to compare the responses of concrete members reinforced with both reinforcement types. The computational part of the study involves nonlinear analysis with consideration of concrete modeling by Thornfeldt model and steel by an elasto-plastic behavior. The experimental and the theoretical approaches addressed curvature ductility as well as deflection ductility, and the relationship between the two. Once the theoretical results were confirmed by the experimental findings, the theoretical approach was used to determine the relationship between the strain in the steel reinforcement and curvature ductility for a wide range of steel reinforcement ratios and concrete compressive strength. This relationship made it possible to determine new strain limits for tension-controlled and compression-controlled regions for concrete sections reinforced with high strength strands. As for the tested columns, the results are used to check if the current equation in the code is capable of predicting the strength, and whether the ductility of strand reinforced columns is different from rebar reinforced columns.
7.2 Conclusions

The results of the experimental and theoretical parts of the study lead to the important conclusions regarding the flexural and axial load structural performance of concrete members reinforced with high strength steel strands without prestressing.

7.2.1 Experimental tests.

1. The experimental load-deflection relationships showed that there is a slight improvement in the stiffness at service when the concrete compressive strength is increased for both rebars and strands. However, there is significant increase in the obtained stiffness when comparing the beams having high reinforcement ratio with corresponding beams having low reinforcement ratio, especially when using mild steel rebars. When comparing the stiffness at service for the rebar reinforced beams with the corresponding strand reinforced beams, the former is always larger. This is because the amount of reinforcement present in the beams with rebars is higher than in the beams with strands and individual rebars are much stiffer than individual strands that are made from seven small wires.

2. The experimental results show significant differences in the flexural behavior between mild steel rebar reinforced specimens and high steel strand reinforced specimens. As expected, the flexural capacity of the tested specimens from the experiments is greatly affected by the amount of steel reinforcement, but only slightly affected by the concrete compressive strength and the loading condition. Also, the ductility of the specimens with mild steel is higher than the ductility in the corresponding specimens with high strength steel for both conditions of loading. The increase in the amount of reinforcement causes an increase in the displacement at yield and also a decrease in the displacement at ultimate. These two findings contribute to the reduction in the ductility of the specimen with higher reinforcement. The tests confirmed that the ductility of the specimens with higher strength concrete is larger than the corresponding ductility of the specimens with lower strength concrete due to the smaller distance between the neutral axis and extreme compression fibers in the former specimens.
3. The experimental findings indicate higher ductility in the long beams that were subjected to 2-point loading near midspan than in the corresponding short beams that were loaded with single point load near the support. Moreover, the load-deflection curves of the short specimens showed higher stiffness at service than their long counterparts because short beams subjected to a load exerted near the support deflect much less than equivalent long beams subjected to two loads applied near midspan.

4. The curvature ductility analysis that is based on the experimental results confirmed that the beams with low steel reinforcement ratio exhibit more ductile behavior than their counterpart with high steel reinforcement ratio mainly. The strand reinforced beams consistently exhibited less curvature ductility than the equivalent rebar reinforced beams that have about the same capacity. The experimentally obtained deflection ductility indices are somewhat proportional to the corresponding curvature ductility indices.

5. The experimental test results on the columns showed that current code equations over-estimate the capacity by about 10%, if the minimum eccentricity factor is ignored. The reason why the experimental results of the strand reinforced sections are lower than their corresponding rebar reinforced sections is because the strands are small in size and are made of seven wires, which makes them flexible and prone to buckling at high loads once the concrete cover spalls off. The results also show that the deflection ductility for strand reinforced columns and rebar reinforced columns is about the same.

7.2.2 Theoretical studies.

1. The ductility of a reinforced concrete cross-section for a strain in the steel at ultimate decreases with an increase in the concrete strength. This is because the reinforcement ratio that corresponds to a specific strain in the steel at ultimate is much higher when using high strength concrete than when using lower strength concrete; hence, its impact on ductility is more critical than the effect of higher concrete strength. In general, the compressive strength effect on curvature ductility in the case of strands is not as significant as in the case of rebars.

2. Curvature ductility analysis shows that the reinforcement efficiency in reinforced concrete sections decreases with an increase in the effective steel...
reinforcement ratios. This is because the increase in bending moment capacity, associated with higher amount of steel, is not at the same rate of increase in the steel reinforcement ratio. This trend is more dominant in the case of low concrete compressive strength than in high concrete compressive strength.

3. Using the theoretical approach to match the curvature ductility of high strength steel strand reinforced sections to sections reinforced with mild steel rebar reinforced sections, it was found that the new strain limits for Grade 1860 MPa steel for compression-controlled and tension-controlled sections become 0.01 (instead of 0.002) and 0.02 (instead of 0.005), respectively.

4. From the theoretical moment capacity graphs, it was noticed that the transition region of the new strain limits gets narrower as the compressive strengths decreases. Also, the effective steel reinforcement ratios at the stress limits are reduced with the decrease in the concrete compressive strength.

5. Although the deflection ductility index for the case of one load is always smaller than the corresponding curvature ductility index, both indices approach each other when the shear span-to-beam length ratio becomes very small. The analysis also shows that the effect of the shear span-to-beam length ratio on the relationship between curvature ductility and deflection ductility is minimal for beams possessing low ductility compared to highly ductile beams.

6. For a given curvature ductility index, the corresponding deflection ductility index in the case of two equidistant loads from the midspan is always larger when the span is small. However, the difference between the indices for the case of a single load is much more pronounced when the curvature ductility is high. The results also show that effect of the number of loads on the span is much more pronounced than the effect of the span length.

7. The theoretical results showed that the relationship between the curvature and deflection ductility indices is insensitive to the location of the load(s) on the span. In general, the deflection ductility index is always smaller than the corresponding curvature ductility index, and the difference between the two indices slightly increases for beams possessing high ductility.

8. The shear span-to-length ratio significantly affects the relationship between curvature and deflection ductility indices. For a given curvature ductility, the corresponding deflection ductility index when the shear span-to-length ratio is small has a much larger value than that when it is large. The analysis also
shows that the effect of the shear span-to-beam length ratio on the relationship between curvature ductility and deflection ductility is minimal for beams possessing low ductility compared to highly ductile beams.

9. The ductility analysis also indicates that the span length greatly affect the relationship between curvature and deflection ductility when there is a single load on the span, but mildly impact the findings when there are two loads. For a given curvature ductility index, the corresponding deflection ductility index is always larger when the span is small. However, the difference between the indices for different spans for the case of a single load is much more pronounced when the curvature ductility is high. The results also show that the effect of the number of loads on the span is much more pronounced than the effect of the span length.
References


Vita

Alaa Al Hawarneh was born in Damascus, Syria in April 1993. He received his high school diploma from Al Ahliyah Private School in Sharjah in 2010. He enrolled in the Civil Engineering program at the University of Sharjah, and graduated with Bachelor of Science degree in July 2014. He was awarded the university shield for outstanding students with the first-class honor. He joined the Master’s Civil Engineering program at the American University of Sharjah in September 2014. He was awarded a graduate teaching assistantship, and worked in the meantime as a research assistant. His research interests include structural and computational mechanics, Design of composite materials and structural reliability studies.