ANALYTICAL MODELING OF DIAGONALLY REINFORCED CONCRETE
COUPLING BEAMS UNDER LATERAL LOADS

A Thesis

Submitted to the Graduate School
of the University of Notre Dame
in Partial Fulfillment of the Requirements
for the Degree of

Master of Science

by

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Notre Dame, Indiana
July 2009
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Ali İrmak Özdağlı
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Abstract

by

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This thesis describes an analytical model to investigate the nonlinear behavior of diagonally reinforced concrete coupling beams under reversed-cyclic lateral loading. The analytical model decomposes a coupling beam into two main components: (1) the concrete beam and any horizontal reinforcement that it contains; and (2) the diagonal reinforcement. The diagonal reinforcement is modeled using nonlinear truss elements in the diagonal direction whereas the concrete beam is modeled using nonlinear fiber beam-column elements in the horizontal direction. The diagonal elements are slaved to the horizontal elements at intermediate and end nodes along the span of the beam.

The analytical model is verified by comparing the measured behavior of coupling beam test specimens from previous research with analytical estimations. It is shown that as compared to prior analytical models, the proposed model introduces a significant improvement on the modeling of diagonally reinforced coupling beams.
To My Beloved Brother,

and Lonely İstanbul
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ACKNOWLEDGMENTS

This research was supported by the Department of Civil Engineering and Geological Sciences at the University of Notre Dame. The support of the University of Notre Dame is gratefully acknowledged.

The author thanks his research advisor, Dr. Yahya C. Kurama for providing excellent technical guidance. This work could not have been possible without his support and advice, motivation and encouragement and eternal patience.

The opinions, findings, and conclusions expressed in this thesis are those of the author and do not necessarily reflect the views of the University of Notre Dame.
# LIST OF SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>$A_c$</td>
<td>Concrete cross-sectional area of the coupling beam resisting shear</td>
</tr>
<tr>
<td>$A_{i}$</td>
<td>Cross-sectional area of $i^{th}$ truss bar component</td>
</tr>
<tr>
<td>$A_d$</td>
<td>Area of one diagonal reinforcement group in one direction</td>
</tr>
<tr>
<td>$A_v$</td>
<td>Area of shear reinforcement</td>
</tr>
<tr>
<td>$b_w$</td>
<td>Web width of coupling beam</td>
</tr>
<tr>
<td>$C_u$</td>
<td>Ultimate compression force developing in a diagonal bar</td>
</tr>
<tr>
<td>$d$</td>
<td>Distance from extreme compression fiber of coupling beam to centroid of longitudinal tension reinforcement</td>
</tr>
<tr>
<td>$d'$</td>
<td>Thickness of concrete cover</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Elastic modulus of concrete</td>
</tr>
<tr>
<td>$E_{cti}$</td>
<td>Modulus of concrete intension after cracking</td>
</tr>
<tr>
<td>$E_{sec}$</td>
<td>Secant modulus of elasticity of concrete at peak stress</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>Compressive strength of concrete</td>
</tr>
<tr>
<td>$f_{ct}$</td>
<td>Tension strength of concrete</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Yield strength of diagonal reinforcement</td>
</tr>
<tr>
<td>$h$</td>
<td>Height of beam</td>
</tr>
<tr>
<td>$l$</td>
<td>Distance from the centroid of compression wall to the centroid of tension wall</td>
</tr>
<tr>
<td>$l_n$</td>
<td>Span length of coupling beam</td>
</tr>
<tr>
<td>$M_0$</td>
<td>Overturning moment generated at any level by external lateral load on a coupled shear wall</td>
</tr>
<tr>
<td>$M_1$</td>
<td>Moment induced in Wall 1 (tension wall) in a coupled wall system</td>
</tr>
<tr>
<td>$M_2$</td>
<td>Moment induced in Wall 2 (compression wall) in a coupled wall system</td>
</tr>
<tr>
<td>$P_L$</td>
<td>Lateral push-over force</td>
</tr>
<tr>
<td>$P_T$</td>
<td>Axial force developed in a prestressed tendon</td>
</tr>
<tr>
<td>$P_u$</td>
<td>Lateral load capacity of coupled shear wall system</td>
</tr>
<tr>
<td>$s$</td>
<td>Center-to-center spacing of transverse reinforcement</td>
</tr>
<tr>
<td>$T$</td>
<td>Axial force generated in each of the walls in a coupled wall system</td>
</tr>
<tr>
<td>$T_u$</td>
<td>Ultimate tension force developing in a diagonal bar</td>
</tr>
<tr>
<td>$V_u$</td>
<td>Ultimate shear strength of coupling beam</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Angle of orientation of diagonal reinforcement</td>
</tr>
<tr>
<td>$\varepsilon_{cc}$</td>
<td>Compressive strain of confined concrete at peak stress</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Modification factor for lightweight concrete reflecting its reduced mechanical properties</td>
</tr>
</tbody>
</table>
\sigma_{m,i} \quad \text{Yield stress of } i^{\text{th}} \text{ truss bar component}
CHAPTER 1:
INTRODUCTION

1.1 Overview

Concrete structural walls (SWs) are one of the most preferable systems for mid- and high-rise building structures designed to resist large lateral loads. Especially, by exhibiting an efficient energy dissipation capability and providing a large amount of structural strength, SWs become an indispensable unique solution for many structures in seismic regions.

The continuity of SWs along the building height is often disturbed due to presence of openings such as doors, windows and corridors. In such cases where both architectural functionality and use of SWs are must, coupled structural walls (CSW) are preferred as the primary lateral support system (Kwan et al., 2007).

A regular CSW consists of two or more SWs interconnected by beams (coupling beams) which are formed between openings (Figure 1.1). Due to the presence of high shear forces, coupling beams are the most critical structural elements in the seismic design of CSWs and need a more careful design consideration than ordinary beams (Hindi et al. 2004). In 1974, Paulay and Binney conducted a series of coupling beam specimen tests in search for improving the ductility of coupling beams (Paulay and Binney, 1974). In this research, they introduced a new reinforcement layout distinct from the conventional rectilinear reinforcement layout with longitudinal bars. According to the
new reinforcement layout, horizontal bars are replaced with diagonal bars. Paulay and Binney (1974) found out that diagonal bars significantly improve the cyclic response of coupling beams having shear span to effective depth ratio below 2.0. In Figure 1.2, conventional and diagonally reinforced coupling beams are shown.

Figure 1.1: A regular coupled structural wall system
In addition, Barney et al. (1980) demonstrated that diagonally reinforced coupling beams with short spans maintain their load capacity at larger displacements better than conventionally reinforced beams. For larger span-to-depth ratios (from 2.5 to 5), the improvement was not as significant.

Figure 1.2: Reinforcement layout of (A) conventionally reinforced beam (B) diagonally reinforced beam (Barney et al. 1980)
In later years, Tassios et al. (1996), Galano et al. (2003), Hindi (2003), and Fortney et al. (2008) conducted new experiments on diagonally reinforced coupling beams focusing on their specific design parameters such as transverse reinforcement ratio and layout as well as boundary conditions. Additionally, Santhakumar (1974) tested two different multi-story coupled shear walls, one having conventionally reinforced coupling beams and the other having diagonally reinforced coupling beams. Santhakumar demonstrated that diagonally reinforced beams offer improved ductility and energy dissipation. Some of the experimental test results mentioned above is discussed in Chapter 2.

1.2 Previous Analytical Representations of Diagonally Reinforced Beams

The ultimate aim of this thesis is to demonstrate a new analytical model to represent the behavior of diagonally reinforced coupling beams under reversed cyclic loads. The previous research on the modeling of diagonally reinforced beams has focused primarily on truss models, which have limited capability to capture the cyclic behavior of coupling beams.

The first truss model and a closed form solution to estimate the ultimate load capacity of diagonally reinforced coupling beams was developed by Paulay and Binney (1974). Hindi (2003) proposed another truss model based on Paulay’s model to predict the monotonic lateral force-displacement envelope of diagonally reinforced coupling beams. Park et al. (2007) demonstrated that diagonally reinforced coupling beams can be modeled as struts and ties. These previous analytical models are discussed more in detail in Chapter 2.
1.3 Research Significance

The early analytical models developed by previous researchers are not capable to predict the hysteretic behavior of diagonally reinforced coupling beams. Moreover, they fail to provide estimations of local behavior in the beams and also ignore strength and stiffness loss due to nonlinear shear effects. The model proposed in this thesis represents a new method to determine the structural behavior of diagonally reinforced coupling beams. The model can estimate the cyclic behavior of the coupling beams including local parameters such as the diagonal steel strains.

1.4 Objectives

The objectives of this research are:

1) To develop a new analytical model to estimate the behavior of diagonally reinforced coupling beams under reversed cyclic lateral loads

2) To verify the analytical model based on previous experimental results.

3) To conduct parametric sensitivity analyses by varying the model properties to help future researchers develop their own analytical models in the absence of experimental data.

1.5 Summary of Approach

The structural analysis program, DRAIN-2DX (Prakash et al., 1993) is used as the main platform for the research. The proposed analytical model is verified by comparing the results with two coupling beam and one coupled shear wall system test results.
Furthermore, the effects of various model parameters, such as the element length and material property on the behavior of the coupling beams are investigated.

To achieve the objectives stated Section 1.4, the following approach is taken:

1) A literature survey of the previous analytical and experimental research on the lateral load behavior of diagonally reinforced coupling beams and coupled walls is performed.

2) A structural analysis model using DRAIN-2DX is developed to represent the hysteretic behavior of diagonally reinforced coupling beams under lateral loads. The model is verified by comparing the analytical results with test results from previous research.

3) The effects of several modeling parameters on the analytical results are examined.

1.6 Organization of Thesis

The remainder of this thesis is organized into five chapters (Chapters 2-6). Chapter 2 discusses the previous research on diagonally reinforced coupling beams and coupled walls. Chapter 3 describes the proposed analytical models. Chapter 4 discusses the validation of the proposed model with experimental results. Chapter 5 presents parametric sensitivity analyses to investigate the effects of changes in material and element properties on the model results. Finally, Chapter 6 presents a summary and conclusions from the research and recommendations for future research.
CHAPTER 2:
BACKGROUND

This chapter provides results of the literature review conducted as part of this thesis. An overview of the previous research on the seismic design and behavior of diagonally reinforced concrete coupling beams is presented. In addition, available analytical models are discussed.

2.1 Design of Diagonally Reinforced Concrete Coupling Beams

Since 1977, ACI 318 (2008) has been providing provisions on the design of diagonally reinforced concrete coupling beams. ACI 318 (2008) assumes that coupling beams having a span-to-depth ratio \(l_n/h\) lower than 2 are considered as deep beams and dominated by shear forces. ACI 318 (2008) requires coupling beams with \(l_n/h < 2\) and \(V_u\) exceeding \(4\lambda\sqrt{f'_c A_c}\) (where, \(\lambda\) = modification factor for lightweight concrete reflecting its reduced mechanical properties; \(f'_c\) = compressive strength of concrete; \(A_c\) = concrete cross-sectional area of the coupling beam resisting shear) be reinforced 'with two intersecting groups of diagonally placed bars symmetrical about the midspan' due to their ability to provide adequate resistance in deep coupling beams. Tests conducted by Paulay et al. (1974) and Barney et al. (1980) have shown that diagonal reinforcement is not effective for coupling beams with span-to-depth ratio \(l_n/h\) greater than 5.
ACI 318-08 proposes Equation 2.1 for the shear strength of a diagonally reinforced coupling beam, $V_u$. This equation was first proposed by Paulay et al. (1974). The upper limit on the equation was proposed by Barney et al. (1980).

$$V_u = 2A_s f_y \sin \alpha \leq 10\sqrt{f'_c} A_c$$

(2.1)

where, $A_s$ is the total area of reinforcement in each group of diagonal bars, $f_y$ is the yield strength of the diagonal reinforcement, $\alpha$ is the angle of orientation of the diagonal reinforcement, $f'_c$ is the compressive strength of concrete, and $A_c$ is the concrete cross-sectional area of the coupling beam resisting shear.

In ACI 318 (2008), two confinement options are presented. In the first option, transverse reinforcement is placed around each group of diagonal bars, prohibiting buckling and providing confinement of concrete (Figure 2.1(A)). In the second option, the entire beam cross section is confined (Figure 2.1(B)). The previous ACI design provisions, which dictated only the first option, are reviewed and discussed by Harries et al. (2005). Harries et al. stated that the ACI 318 design provision in Figure 2.1(A) causes difficulties in practice. Particularly, the congestion of the transverse reinforcement around the diagonal bars at the beam mid-span requires careful detailing for the placement of the bars. Therefore, Harries et al. suggested that the confinement of the entire coupling beam as shown in Figure 2.1(B) allows for more versatility in design and still provides adequate resistance against lateral loads.
Figure 2.1: Detailing of diagonally reinforced coupling beams from ACI 318-08:
(A) Option 1 (B) Option 2
Wallace (2008) investigated the cyclic force versus deformation behavior of coupling beams specimens with different aspect ratios \((l_n/h)\) and the two reinforcement layouts given in ACI 318-08. It was experimentally confirmed that the detailing arrangement in Figure 2.1(B) is as effective as the layout in Figure 2.1(A).

2.2 Advantages of Diagonal Reinforcement Layout over Traditional Reinforcement Layout

The previous research on conventionally reinforced coupling beams conducted by Paulay (1971a, 1971b) showed that beams with traditional reinforcement layouts (i.e., horizontal bars in the top and bottom of the beam) cannot satisfy the seismic ductility demands of coupled shear wall structures. Paulay found out that during the application of stepwise increasing reversed cyclic lateral loads, the length of the beam increases creating tension in the steel. Once the reinforcement starts to yield in tension, vertical cracks over the full depth of the member become larger at the beam ends. Since the residual tensile stresses in the reinforcing steel are never recovered, after subsequent reversed loadings, the cracks remain open and continue to grow. When the aggregate interlock mechanism of concrete, which provides additional shear resistance to the beam, fails due to the large cracks caused by large displacements, sliding shear failure occurs. This is followed by significant stiffness degradation and loss of lateral load resistance. The typical lateral load-rotation response of a conventionally reinforced coupling beam tested by Paulay is depicted in Figure 2.2. In the first four cycles, the beam behaved almost elastic. After cycle 6, sliding shear failure dominated the response and consequently, at the end of cycle 7, the beam had almost no resistance.
Upon these findings, Paulay and Binney (1974) introduced a new reinforcement layout based on the diagonal placement of steel bars. Three diagonally reinforced coupling beams with similar reinforcement arrangement were tested. Figure 2.3 shows the response of one of the beams with the same geometric dimensions as the conventionally reinforced coupling beam in Figure 2.2. The beam behavior demonstrates the characteristics of the steel reinforcement with Bauschinger effect and considerable energy dissipation with no loss of strength. In addition, this time, a much larger ultimate rotation is reached by the specimen, showing the increased ductility capacity of diagonally reinforced coupling beams. The sliding shear failure observed in conventionally reinforced coupling beams is prevented as a result of the use of the diagonal reinforcement. The vertical and diagonal cracks were resisted mainly by the
tension stresses in the diagonal bars. Paulay and Binney recommended that diagonal reinforcement should be anchored to the shear walls and tied to prevent buckling.

Based on their test data, Paulay and Binney proposed a closed form procedure to estimate the ultimate load capacity of diagonally reinforced coupling beams. This model is discussed in Section 2.4.

![Diagram showing lateral load versus rotation behavior of a diagonally reinforced coupling beam](image)

Figure 2.3: Lateral load versus rotation behavior of a diagonally reinforced coupling beam (Paulay and Binney, 1974)

As part of a program to develop design criteria for reinforced concrete structural walls, Barney et al. (1980) tested eight reinforced concrete coupling beams to evaluate their behavior under cyclic loads. Three specimens had horizontal reinforcement and two specimens had full length diagonal bars. The rest were reinforced diagonally only in the plastic hinge end regions and horizontally in the whole span.
It was found out that the performance of conventionally reinforced coupling beams is limited by the deterioration of the shear resisting mechanism in the hinge regions. The deterioration of concrete was associated with the pinching of the lateral load versus deflection relationship as shown in Figure 2.4.

The beams with full-length diagonal bars tested by Barney et al. had shear span to effective depth ratios of 1.4 and 2.8 (corresponding to full span to depth ratios of 2.5 and 5, respectively). It was shown that the full length diagonals increased the lateral load resistance and energy dissipation capacity of these coupling beams. No loss of stiffness or strength was observed over large lateral displacements. As demonstrated in Figure 2.5, the lateral load versus displacement response curve of the diagonally reinforced beams did not exhibit any pinching.
2.3 Coupled Wall Experiments

As stated in Chapter 1, shear wall systems with diagonally reinforced coupling beams are one of the most preferred seismic lateral load resisting systems where architectural functionality and form are a major concern. The overturning moment, $M_0$ in a coupled shear wall system exposed to lateral loads is resisted by three internal actions. These are the bending moment developing in Wall 1, $M_1$; the bending moment developing in Wall 2, $M_2$ and the axial forces, $T$ developing in both walls, as shown in Figure 2.6. The total overturning moment, $M_0$ can be then defined as:

$$M_0 = M_1 + M_2 + lT$$  \hspace{1cm} (2.2)

where, $l$ is the distance from the centroid of Wall 1 to the centroid of Wall 2.
Santhakumar (1974) tested two reinforced concrete coupled shear wall systems with differently reinforced coupling beams under reversed cyclic lateral loading. As shown in Figure 2.7, System Wall A had conventionally reinforced concrete beams whereas System Wall B had diagonally reinforced beams. Both structures had the same reinforcement in the wall piers.
Figure 2.8: Lateral load versus roof deflection behavior for Wall A (Santhakumar, 1974)

Figure 2.9: Lateral load versus roof deflection behavior for Wall B (Santhakumar, 1974)
Santhakumar found that the shear capacity of the wall pier carrying axial tension was dramatically reduced as evidenced by the reduced inclination of the diagonal cracks after progressive reversed cyclic loading. The compression wall pier carried a much larger share of the total lateral load. The total energy dissipated by the reinforcement in the coupling beams of System Wall B was much greater than that of System Wall A. In Figures 2.8 and 2.9, the measured lateral load versus roof deflection relationships of both wall systems are given. It is evident that the difference in the behaviors of the two wall systems is due to reinforcement layout in the beams. The pinching in the hysteresis loops of Wall A results from the inferior performance of conventionally reinforced coupling beams. On the other hand, the diagonal bars in beams of Wall B enabled an increase in the energy dissipation capacity. The hysteresis loops of Wall B not only show large ductility with no loss of strength but also possess stable characteristics similar to a ductile steel structure.

2.4 Previous Analytical Studies

Paulay (1974) presented a simple truss model to estimate the ultimate lateral load capacity of diagonally reinforced coupling beams. According to Paulay's model, it is assumed that after first yielding, the diagonal bars carry all of the forces in tension and compression developing in the beam.

As shown in Figure 2.10, during a load cycle, one of the diagonals goes into compression whereas the other goes into tension. The ultimate compression force is represented as $C_u$ and the ultimate tension force as $T_u$. Both forces are equal to the yield force of the steel and calculated as:
where, $A_s$ is the area of each diagonal reinforcement and $f_y$ is the yield strength of the steel.

The resisting shear force, $V_u$ and the resisting moment, $M_u$ at the beam ends can be calculated as:

\[ V_u = 2T_u \sin \alpha = 2C_u \sin \alpha \]  \hspace{1cm} (2.4)

\[ M_u = V_u l_n/2 = (h - 2d')T_u \cos \alpha = (h - 2d')C_u \cos \alpha \]  \hspace{1cm} (2.5)

where, $\alpha$ is the angle of inclination for the diagonal reinforcement, $l_n$ is the length of the beam, $h$ is the height of the beam, and $d'$ is the concrete cover.

Figure 2.10: Truss model proposed by Paulay et al. (1974)
In Figure 2.11, the lateral load versus rotation relationship of a beam tested by Paulay et al. (1974) and its theoretical ultimate shear capacity are shown. Based on this verification, Paulay's model can be used to predict the shear capacity of a diagonally reinforced beam with acceptable precision.

However, Paulay's model has two drawbacks.

- The model does not include the contribution of the concrete to the ultimate strength of the beam.
- The model is not capable to predict the behavior of the beam under cyclic loads.

Figure 2.11: Lateral load versus rotation behavior of a diagonally reinforced coupling beam tested by Paulay et al. (1974)
To eliminate the first drawback, an analytical model predicting the monotonic force-displacement envelope of a diagonally reinforced coupling beam based on Paulay's model was developed by Hindi (2003). In this model, it is assumed that the lateral forces are resisted by diagonal tension and compression. The compressive diagonal force is carried by the diagonal reinforcement and the concrete core, whereas the tensile diagonal force is carried only by the diagonal reinforcement as shown in Figure 2.12.

The tensile and compressive forces are calculated as:

\[ T = A_s f_y \]  \hspace{1cm} (2.6)  

\[ C = A_s f_s + A_c f_c \]  \hspace{1cm} (2.7)
where, $A_c$ is the diagonal concrete area and $f_s$ and $f_c$ are the steel and concrete stresses, respectively. Contribution of concrete in compression is included as $A_c f_c$ into Equation 2.7. Then, the resisting shear force can be calculated using Equation 2.8 as:

$$V = (T + C) \sin \alpha$$  \hspace{1cm} (2.8)

Figure 2.13 compares the results of a diagonally reinforced coupling beam tested by Tassios et al. (1996) with Paulay's and Hindi's proposed models. It can be seen that the monotonic force-displacement behavior derived from Hindi's model predicts the ultimate capacity of the coupling beam relatively better than Paulay's model.

Figure 2.13: Lateral load versus behavior of a diagonally reinforced coupling beam tested by Tassios et al. (1996) (adapted from Hindi, 2003)
However, the following significant drawbacks restrict the use of Hindi's model:

- The model is limited to axially restrained beams.
- The effect of the concrete tension strength on the ultimate capacity of the coupling beam is omitted.
- The model cannot capture the hysteretic behavior of diagonally reinforced coupling beams under cyclic loading.

Park et al. (2007) presented a nonlinear strut-tie model to represent the behavior of reinforced concrete members subjected to cyclic loads. In this model, a diagonally reinforced coupling beam is idealized into longitudinal, transverse, and diagonal truss elements. Each element is modeled as a composite element of concrete and reinforcing steel. Figure 2.14 shows the reinforcement details of a diagonally reinforced coupling beam specimen and its idealization.

Figure 2.14: Truss model proposed by Park et al. (2003)
Figure 2.15 compares the measured behavior of a diagonally reinforced coupling beam (dashed line) with analytical results from Park et al. (solid line). It can be seen that the model demonstrates a satisfactory performance in estimating the hysteretic behavior of the beam.

The primary limitations of this model are:

- Since the model ignores the shear resistance of concrete, it underestimates the load carrying capacity of beams with low transverse reinforcement ratio.

- The model has limited capability to estimate local parameters such as the diagonal reinforcement steel strains.

Figure 2.15: Lateral load versus displacement behavior of a diagonally reinforced coupling beam tested by Galano et al. (1996)
CHAPTER 3:
PROPOSED ANALYTICAL MODELS

This chapter discusses analytical models for diagonally reinforced coupling beams and coupled wall systems with diagonally reinforced beams under cyclic loading. In the first part of the chapter, the structural analysis program, DRAIN-2DX, used as the structural analytical platform is described briefly. Later, analytical models of horizontally reinforced coupling beams are explained. Subsequently, the geometric and material features of the proposed model for diagonally reinforced coupling beams are presented and discussed in detail. Furthermore, the analytical modeling of flexural walls is addressed. Finally, the analytical modeling of coupled wall systems is discussed.

3.1 Introduction to DRAIN-2DX

DRAIN-2DX is a nonlinear structural analysis computer program widely used for reinforced concrete, steel, and composite structures (Prakash et al., 1993). Because of its open-source nature, DRAIN-2DX is very adaptable to research development. Various element types have been predefined in the program source-code. These are the Inelastic Bar Element (Element 01), Plastic Hinge Element (Element 02), Spring Element (Element 04), Elastic Panel Element (Element 06), Compression/Tension Link Element
(Element 09) and Fiber Beam-Column Element (Element 15). For the purpose of this research, Element 01 and a modified version of Element 15 are used.

3.1.1 Fiber Element Discretization

The fiber beam-column element (Element 15) in DRAIN-2DX is an inelastic element developed for the analysis of structures including reinforced concrete, steel, or composite members. As seen in Figure 3.1, each fiber element is divided into a series of segments. To each segment, a group of fibers are assigned. These fibers are located at the mid-length cross section of the segment called the "slice" and govern the behavior of the segment. A uniaxial material stress-strain relationship is assigned to each fiber. The fiber stress-strain curve is idealized as a series of points rather than a function (i.e., the stress-strain relationship is a multi-linear idealization of the actual smooth material behavior). Each fiber follows a pre-defined hysteresis rule for steel or concrete. The accuracy of the fiber element depends on the number of slices and fibers used. A finer discretization over the element length and cross-section improves the accuracy of the model but also yields longer analysis times.

The DRAIN-2DX fiber element is chosen for the modeling of diagonally reinforced coupling beams because it can account for many nonlinear effects such as the crushing and cracking of concrete, yielding of steel as well as post-yield, crush and crack behaviors (Prakash et al., 1993). The main limitation of the fiber element is that the element does not consider nonlinear shear deformations. It is designed to deform elastically in shear. The modeling of nonlinear shear effects is discussed next.
3.1.2 Modeling of Nonlinear Shear Effects

Coupling beams subjected to lateral loads undergo large deformations and are exposed to large shear forces. It is inevitable that the beams suffer from shear nonlinearities due to shear distortion. However, the basic fiber element (Element 15) in DRAIN-2DX does not consider shear nonlinearities and the shear modulus is taken as the linear-elastic modulus. To allow for the modeling of nonlinear behavior in shear, a new fiber element (Element 16) was developed by Jiang (2009). A comparison between the results obtained from coupling beam models using Element 15 and Element 16 can be found in Chapter 5.
3.1.3 Inelastic Bar Element

The inelastic bar element (Element 01) in DRAIN-2DX is mainly used to model truss members and other simple structural members. It can transmit axial load only. The Young's modulus, yield stress in tension and compression, and optionally, a strain hardening ratio are assigned as material variables. Buckling in compression can also be included in the element behavior. Since the main deformation mechanism of diagonal reinforcing bars is axial tension and compression, they can be represented using truss bar elements. Implementing truss bar elements into the geometry of the proposed model by the modeling of the diagonal bars is discussed in Section 3.2.2.

3.2 Material Models

In the DRAIN-2DX fiber element, material behavior is defined as a series of stress-strain points. Thus, to obtain accurate analytical results, each material model should represent the actual stress-strain properties for the concrete and reinforcing steel in the best possible way.

3.2.1 Concrete

A multi-linear concrete fiber (C-type fiber) is used to simulate the concrete behavior. The stress-strain points for the fiber are selected from the envelope of a smooth concrete stress-strain curve in compression as given in Figure 3.2. The concrete fiber input is limited to 5 points in compression and 5 points in tension, however by changing the source-code this limit can be increased if needed.
For most of the available coupling beam experiments, the concrete compressive properties were either not given or only the unconfined concrete strength was presented. To develop the entire behavior of concrete in compression from its strength, a model developed by Mander et al. (1988) is used. Basically, Mander et al. developed a stress-strain model for concrete confined with transverse reinforcement. The model depends on several variables such as the unconfined concrete strength and strain corresponding to peak stress, the yield strength and strain of the transverse reinforcement, and the amount of transverse reinforcement, as given in Figure 3.3. The concrete stress-strain \((f_c - \varepsilon_c)\) relationship is given by:

\[
f_c = \frac{f_{cc}}{r^{-1.0+x}r}
\]  

where, \(f_{cc}\) is the compressive strength of confined concrete. Variables \(x\) and \(r\) are given by Equations 3.2 and 3.3, respectively.
\[ x = \frac{\epsilon_c}{\epsilon_{cc}} \]  

(3.2)

where, \( \epsilon_{cc} \) is the compressive strain of confined concrete at peak stress.

\[ r = \frac{E_c}{E_c - E_{sec}} \]  

(3.3)

\[ E_{sec} = \frac{f_{cc}}{\epsilon_{cc}} \]  

(3.4)

In Equations 3.3 and 3.4, \( E_c \) and \( E_{sec} \) are given as the elastic modulus of concrete and the secant modulus of elasticity at peak stress. As recommended by ACI 318-08, \( E_c \) can be determined by:

\[ E_c = 57000\sqrt{f_c'}, \text{ in psi units} \]  

(3.5)

Figure 3.3: Stress-strain model of confined and unconfined concrete proposed by Mander et al. (1988)

Further details on the model can be found in Mander et al. (1988). In Figure 3.4, a sample cyclic confined concrete stress-strain relationship in compression is presented.

The cyclic behavior of the concrete fiber in DRAIN-2DX is compared with the
monotonic Mander et al. model. It can be seen that the DRAIN-2DX model starts to deviate from the Mander et al. model after the peak stress is reached. The overestimation of the post-peak stiffness is done intentionally, since a sharp drop in the stiffness of the material can cause numerical problems and early termination of the analysis.

![Cyclic stress-strain relationship of concrete in compression](image)

Figure 3.4: Cyclic stress-strain relationship of concrete in compression

Modeling of concrete behavior in tension is more challenging. The tension strength of concrete is uncertain; therefore, the widely accepted Equation 3.6 from ACI 318-08 is used. Elastic modulus in tension is assumed to be equal to the elastic modulus in compression, which is calculated using Equation 3.5.

\[
f_{ct} = 7.5\sqrt{f'_c}
\]  

(3.6)

where, \( f_{ct} \) is the tension strength of concrete and \( f'_c \) is the compression strength of unconfined concrete.
During the modeling procedure, the tensile strength of concrete and its behavior after cracking require a very time consuming calibration process. In other words, the tensile strength from Equation 3.6 is taken as a starting point. The effect of the concrete tensile strength on the analytical behavior of diagonally reinforced coupling beams is discussed in Chapter 5.

3.2.2 Reinforcing Steel

There are three types of steel reinforcement used in a diagonally reinforced coupling beam. These are: (1) diagonal bars, which constitute the main resisting reinforcement in the beam, (2) horizontal bars, which are usually used to hold the transverse reinforcement, and (3) transverse reinforcement, which provides confinement and shear resistance to the beam. Each one of these reinforcement types is represented differently in the analytical model as summarized in Table 3.1.

*Modeling of Diagonal Reinforcement*

Modeling of the diagonal bars in a diagonally reinforced coupling beam is very important because the strength and ductility of the beam mainly comes from this reinforcement. The diagonal cracks developing in the beam during loading are limited by the diagonal bars.
<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Analytical Representation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diagonal Reinforcement</td>
<td>Inelastic Truss Bar</td>
</tr>
<tr>
<td>Horizontal Reinforcement</td>
<td>S-type Steel Fiber defined in Element 16</td>
</tr>
<tr>
<td>Transverse Reinforcement</td>
<td>Transverse Tributary Reinforcement Fiber defined in Element 16</td>
</tr>
</tbody>
</table>

When steel reinforcement is subjected to inelastic cyclic loads during an earthquake event, the yield plateau of the reinforcement is suppressed and the stress-strain curve exhibits a Bauschinger effect (i.e., upon load reversal, the stress-strain curve becomes nonlinear before reaching the yield level). The typical stress-strain relationship of a steel bar under cyclic loading is given in Figure 3.5.

As stated previously in Section 3.1.3, the diagonal reinforcing bars are modeled using inelastic truss bar elements. Since the inelastic truss bar in DRAIN-2DX is limited to bilinear behavior, it cannot simulate the typical stress-strain relationship in Figure 3.5 accurately. Consequently, this effect is reproduced by dividing the bar element into a number of parallel elastic-perfectly-plastic components. For each component, the user inputs the cross-sectional area, the yield stress, and the Young's modulus. The strain hardening in each component is taken as zero and the hysteretic behavior under cyclic loading follows symmetric elastic-perfectly-plastic behavior. As seen in Figure 3.6, each component has the same Young's modulus but a different yield stress and cross-sectional area. The degree of Bauschinger effect is governed by the cross-sectional area and yield stress of the individual components. The total cross-sectional area of the components
should be equal to the area of the diagonal bar \((A = A_1 + A_2 + \cdots + A_i)\). Upon empirical observation, five components were found to be adequate to represent a steel cyclic behavior with Bauschinger effect.

Figure 3.5: Cyclic straining of reinforcing steel (adapted from Paulay and Priestley, 1992)

Figure 3.6: Reproduction of Bauschinger effect using inelastic bar elements
Generating a complete stress-strain curve from a number of elastic-perfectly plastic truss elements requires the development of an algorithm. In Figure 3.7, this process is summarized as a flowchart. According to the algorithm, every truss element has a different area and yield strength but the same Young's modulus. The combination of these unique truss elements produces the predefined stress-strain points.

![Figure 3.7: Algorithm for simulation of Bauschinger effect](image)

As given in the flowchart, in the first phase, the total area of the diagonal reinforcing steel is determined and expected Bauschinger behavior is defined using an adequate number of stress-strain points \((\sigma_l - \epsilon_l)\). In other words, the Bauschinger behavior of the steel is multilinearized using a series of stress-strain points.
In phase 2, by multiplying every yield strain point, $\varepsilon_i$ with the Young’s modulus of the diagonal steel, $E$ (Equation 3.7), the yield stress matrix, $\sigma_m$ is constructed as presented in Equation 3.8.

$$\sigma_{m,i} = E \cdot \varepsilon_i \quad (3.7)$$

$$\sigma_m = \begin{bmatrix}
\sigma_{m,1} & \sigma_{m,1} & \sigma_{m,1} & \sigma_{m,1} & \sigma_{m,1} \\
\sigma_{m,1} & \sigma_{m,2} & \sigma_{m,2} & \sigma_{m,2} & \sigma_{m,2} \\
\sigma_{m,1} & \sigma_{m,2} & \sigma_{m,3} & \sigma_{m,3} & \sigma_{m,3} \\
\sigma_{m,1} & \sigma_{m,2} & \sigma_{m,3} & \sigma_{m,4} & \sigma_{m,4} \\
\sigma_{m,1} & \sigma_{m,2} & \sigma_{m,3} & \sigma_{m,4} & \sigma_{m,5}
\end{bmatrix} \quad (3.8)$$

Likewise, the force matrix, $F$ given in Equation 3.9 is determined by multiplying each predefined stress point, $\sigma_i$ with the total area of the steel, $A$ using Equation 3.7.

$$F_i = A \cdot \sigma_i \quad (3.9)$$

$$F = \begin{bmatrix} F_1 \\ F_2 \\ F_3 \\ F_4 \\ F_5 \end{bmatrix} \quad (3.10)$$

In phase 3, the area for each individual truss member, $A_i$ is calculated using Equation 3.11.

$$A_m = \frac{F}{\sigma_m} = \begin{bmatrix} A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \end{bmatrix} \quad (3.11)$$
Figure 3.8: Cyclic stress-strain relationship: (A) single truss bar element and (B) multiple bar elements

The yield stress of each component is given in the last column of $\sigma_m$ matrix. Finally, the $i^{th}$ truss bar will have a unique area, $A_i$ and a unique yield stress, $\sigma_{m,i}$ but same Young's modulus, $E$ as the other components.
It should be noted that all of the equations above are based on the assumption that the Bauschinger behavior is defined by five stress-strain points. Matrix sizes can change accordingly depending on the number of stress-strain points.

Figure 3.8 shows the cyclic stress-strain relationship of a single truss bar and of multiple bar elements modeling the Bauschinger effect. Even though the single truss bar model represents a reasonable bilinear idealization of actual steel behavior, the comparison exhibits clearly the significance of the Bauschinger effect. The importance of this effect on the overall behavior of coupling beams is discussed in Chapter 5.

Note that the diagonal reinforcement should be confined properly in order to avoid buckling of the bars due to large deformations in compression, as required in ACI 318-08. Since the coupling beam test specimens used in the development of the proposed analytical model in this thesis did not suffer from buckling of reinforcement, modeling of bar buckling is omitted.

**Modeling of Horizontal Reinforcement**

The horizontal steel reinforcement is modeled using the S-type steel fiber in the fiber beam-column elements representing the coupling beam. DRAIN-2DX assumes that there is full bond between the C-type concrete fibers and the adjacent S-type steel fibers. A uniaxial multi-linear stress-strain curve up to 5 points can be assigned to the S-type fiber. Figure 3.9 shows the typical stress-strain relationship of a horizontal reinforcement exhibiting Bauschinger effect.

It should be noted that the total cross-sectional area of developed horizontal bars in a diagonally reinforced coupling beam is often very small and the effect of this
reinforcement on the overall strength of the beam is negligible. Therefore, in the modeling of the diagonally reinforced coupling beams in this thesis, the horizontal bars are ignored.

Figure 3.9: Cyclic stress-strain relationship of the horizontal steel fiber

*Modeling of Transverse Reinforcement*

In Element 16, the transverse steel bars are modeled as tributary reinforcement within the concrete fiber formulation. The tributary transverse reinforcement is determined by dividing the total transverse reinforcement cross-section area in the fiber with the product of the fiber length and fiber width (Jiang, 2009).

A multi-linear stress-strain relationship is used for the tributary transverse reinforcement and the behavior of this reinforcement under cyclic loading follows the same hysteresis rules as the S-type fiber in DRAIN-2DX (see Figure 3.9).

Based on the study described in Barney et al. (1980), since the transverse reinforcement strains during reversed cyclic loading of diagonally reinforced coupling...
beam are very small and no yielding of this steel is observed, the transverse steel stress-strain relationship can be modeled as linear-elastic.

Note that the use of tributary transverse reinforcement in Element 16 may alter the predefined C-type fiber stress-strain behavior by increasing the compression strength and ultimate strain due to increased confinement effects. Therefore, the concrete stress-strain relationship needs to be calibrated accordingly before the analysis.

3.3 Modeling and Analysis Assumptions

In order to simplify the proposed modeling processes, the following assumptions and considerations are made for the two-dimensional analysis and modeling of horizontally and diagonally reinforced coupling beams, isolated flexural walls and coupled wall structures.

1. Torsional or out of plane displacements of these structures are not modeled.
2. Buckling of diagonal reinforcement is not considered and not modeled.
3. Abutment walls on coupling beam test subassemblages are assumed to be fully rigid.
4. Elastic and inelastic deformations that may occur at the base support of coupled walls or abutment walls of coupling beams are not considered.
5. Bond between concrete and diagonal reinforcement is modeled at discrete points as explained in Section 3.5. This "discrete" bonding is assumed to remain always effective during the cyclic response of the structure. Full
bond is assumed between concrete and the horizontal reinforcement.

Slipping of the reinforcement is not considered.

6. Dead load of the concrete structure weight is omitted.

3.4 Modeling of Lateral and Gravity Loads

The lateral strength and cyclic behavior of coupling beams, isolated flexural walls, and coupled wall structures are determined from static nonlinear lateral load (i.e., push-over) analyses. For the analysis of coupling beams, the lateral load is applied at the center of the free abutment wall portion perpendicular to the beam's longitudinal axis as shown in Figure 3.12. For isolated flexural walls and coupled walls, the lateral loads are applied at the floor and roof levels parallel to the ground as given in Figure 3.14 and Figure 3.16. The lateral load is applied as a unit load and a displacement-controlled analysis with respect to the fixed end of the structure (either the midpoint of the fixed abutment wall in a coupling beam or the base in a wall structure) is conducted.

In order to simulate gravity loads for the testing of isolated flexural or coupled walls, axial load is often applied at the top of the structure perpendicular to the ground as shown in Figure 3.14 and Figure 3.16. Opposite to the lateral load modeling using a displacement-controlled analysis, a load-controlled analysis is conducted for gravity loads.

3.5 Modeling of Horizontally Reinforced Coupling Beam Subassemblies

Since Element 16 is a newly developed fiber beam-column element, the capabilities of the element should be well understood before implementing it to
diagonally reinforced coupling beam analytical models. For this reason, three different horizontally reinforced coupling beams tested by Lee et al. (2003) are analytically modeled. The verification of the analytical estimations is discussed in Chapter 4. Since the modeling of horizontally reinforced coupling beams is out of the context of this research, only a brief explanation of the model is given in this section.

Three coupling beam specimens having the same span length and cross-section dimensions shown in Figure 3.10(A) and Figure 3.11(A), (all dimensions given in mm) are tested under lateral loads. The main differences between the three specimens are the concrete compression strength, yielding stress, cross-sectional area and spacing of shear reinforcements.

![Image](image.png)

Figure 3.10: Horizontally reinforced coupling beam specimen tested by Lee et al. (A) and its analytical representation (B)

In Figure 3.10, a horizontally reinforced coupling beam test specimen and its analytical representation is given. Each test specimen is represented by five Element 16
beam-column fiber elements. The abutment wall is represented as a rigid element in the analytical model. One of the beam ends is restrained in the x, y and z directions whereas the other end where the lateral load, $P_L$ is applied is free in the x and y directions but restrained against rotation.

Figure 3.11: Cross-section of a horizontally reinforced coupling beam tested Lee et al. (A) and its simplified analytical representation (B)

In Figure 3.11, the cross-section of a horizontally reinforced coupling beam test specimen and its simplified fiber discretization are shown. Mainly, the cross-section is made of C-type fibers representing the unconfined and confined concrete and S-type fibers representing the horizontal steel reinforcement. The cross-section discretization is explained
3.6 Modeling of Diagonally Reinforced Coupling Beam Subassemblies

Similar to a horizontally reinforced beam, a diagonally reinforced coupling beam test specimen is also monolithically connected to two abutment walls (Figure 3.12(A)).

The analytical model decomposes the coupling beam into the following two main components: (1) the concrete beam and any horizontal reinforcement that it contains; and (2) the diagonal reinforcing bars. All of the nodes at the beam ends are slaved to master nodes located at the middle of the corresponding wall portion, as shown in Figure 3.12(B). The horizontal concrete/steel fiber elements are connected to each other through intermediate nodes. Likewise, the diagonal elements are linked to each other through intermediate nodes, which are slaved to the beam element nodes at the same x coordinates along the beam length.

The concrete beam is modeled using nonlinear fiber beam-column elements (Element 16) in the horizontal direction, with unconfined and confined concrete fibers modeling the compression/tension behavior of the concrete and steel fibers modeling the behavior of the horizontal reinforcement. The typical fiber cross-section of a nonlinear fiber beam-column element contains three types of fibers. These are: (1) confined concrete; (2) unconfined concrete and (3) horizontal steel. The confined concrete fibers have larger compressive peak strength and strain due to the confinement effect provided by the transverse reinforcement. The concrete cover is modeled as unconfined concrete fibers and has a smaller compressive strength due to the lack of transverse reinforcement. Additionally, each horizontal reinforcement layer is represented by a single steel fiber. The material behavior modeling of the concrete and horizontal reinforcements is discussed previously in Sections 3.2.1 and 3.2.2.
Figure 3.12: Reinforcement detail of a diagonally reinforced coupling beam tested by Barney et al. (A) and its analytical representation (B)

In Figure 3.13, the cross-section of a typical diagonally reinforced coupling beam test specimen and its simplified fiber discretization are given. The diagonal reinforcing bars are not shown in the figure, since they are modeled separately using truss elements. As stated in Section 3.2.2, the horizontal reinforcing bars placed in the beam do not have a significant cross-sectional area to provide flexural resistance and are also not anchored to the abutment walls. Thus, as given in Figure 3.13, horizontal steel fibers are not included when the cross-section of the beam is discretized. Since the outer concrete fibers are exposed to large deformations, there will be a lot of nonlinearity in these fibers as compared to the inner fibers. For this reason, to enhance the accuracy of the model, the number of fibers outwards is increased. On the contrary, by decreasing the number of fibers inwards, the analysis time can be reduced.
The diagonal reinforcing bars are modeled using nonlinear truss elements (Element 01) in the diagonal direction. The material modeling of the diagonal reinforcements is discussed previously in Sections 3.1.3 and 3.2.2.

The element length is determined empirically depending on the amount of data and precision retrieved from the model. The horizontal length of each fiber/truss element on the beam is taken either as $1s$ or $2s$, where $s$ is center-to-center spacing of the transverse reinforcement.

At early stages of the proposed model, the bond effect between the concrete and the diagonal bars was omitted. The analysis results showed that the predicted strains in the diagonal bars were much smaller than the measured strains. Consequently, there was also a considerable strength reduction compared to the measured strength. With the aim of increasing the strains in the diagonal bars, a discrete bonding effect between the concrete and the bars was introduced by slaving the diagonal truss elements to the
horizontal fiber elements at intermediate nodes along the span of the beam with rigid links, as shown Figure 3.12(B). The bonding effect is discussed further in Chapter 5. In addition, to model the anchorage of the diagonal bars into the abutment walls, the end nodes of the diagonals are slaved to the midpoint of the wall abutments.

3.7 Modeling of Isolated Flexural Walls

The modeling of multi-story coupled wall structures also requires the modeling of the isolated concrete structural walls being coupled. Below, an analytical model of isolated walls is explained for this purpose. For shear dominated walls, Element 16 is imposed whereas for isolated flexural walls Element 15 is used. During the analytical modeling process, each story in the wall is defined with an element. Elements are connected to each other by intermediate nodes located at the floor levels. The first story element, $l_1$ has two segments having different lengths. The first segment corresponds to the plastic hinge length, which was taken empirically as two thirds of the first element length. Other story elements are modeled with a single segment.

Every element contains concrete and longitudinal reinforcement fibers. For inadequately confined concrete cores, the confinement effect can be ignored. For modeling of longitudinal bars, horizontal reinforcement modeling explained in Section 3.2.2. is followed. Bauschinger effect of steel is taken into account as described previously.

The wall foundation is assumed to be rigid. Therefore, the wall base node is kinematically constrained against displacements and rotation. An isolated four story
isolated flexural wall tested by Thomsen (1995) and its analytical model are given in Figure 3.14.

![Diagram of load assembly and wall specimen](image)

Figure 3.14: Tested wall specimen tested by Thomsen (A) and its analytical representation (B)

Cyclic lateral displacements were applied to the top of the specimen through a hydraulic actuator mounted horizontally. In Figure 3.14, this lateral load is represented by $P_L$. In addition, an axial load was applied at the top of the wall by hydraulic jacks mounted on the top of the load transfer assembly. This axial load is represented by $P_t$ (see Figure 3.14(B)).
In Figure 3.15, the cross-section of a typical flexural wall test specimen and its simplified fiber discretization are given. Compared to diagonally reinforced coupling beams (Figure 3.13), the longitudinal reinforcement used in the wall provides most of the flexural resistance. Therefore, as given in Figure 3.15, it is necessary to include longitudinal steel fibers when the cross-section of the wall is discretized.

Figure 3.15: Cross-section detail of wall structure tested by Thomsen (A) and its simplified analytical representation (B)

3.8 Modeling of Coupled Wall Systems with Diagonally Reinforced Concrete Beams

The analytical model of a coupled wall system is basically decomposed into the isolated structural walls and the diagonally reinforced coupling beams. When modeling the wall portions, the procedure given in Section 3.7 is followed. The modeling of the coupling beams is explained in Section 3.6. At each floor level, the beam end nodes and
the diagonal bar end nodes are slaved to corresponding nodes at the mid-point of the walls. A coupled wall system with diagonally reinforced coupling beams tested by Santakumar (1974) and its analytical model are given in Figure 3.16. In Santakumar's experiment, cyclic lateral displacements were applied to the specimen through the mid-point of three floors. In Figure 3.16(B), the lateral load at \( i \text{th} \) floor is represented by \( P_{Li} \). In addition, the axial load applied to each wall is represented by \( P_t \) (see Figure 3.16(B)). Since each coupling beam in the test specimen possesses only three stirrups, the coupling beam fiber element length is taken as \( 1s \).

As will be discussed later in Chapter 4, the wall system was relatively slender and the model was not exposed to cyclic reversed loading in plastic phase, shear distortion was not significant. Therefore, to reduce the analysis time, Element 15 was used as the main fiber-beam column element in the modeling of this coupled wall structure.
Figure 3.16: Reinforcement detail of coupled wall system tested by Santakumar (A) and its analytical representation (B)
In this chapter, the analytical models proposed in Chapter 3 are verified by comparing the measured behavior of the test specimens under lateral loads with analytical estimations. The comparisons include global response parameters such as the shear force versus lateral displacement behavior as well as local parameters such as the reinforcing steel strains. Following this validation, Chapter 5 presents a parametric analytical study to investigate the effects of several model parameters on the analytical results.

4.1 Verification Process

The verification process of the proposed models consists of four steps: (1) model the tested specimen using the corresponding proposed model; (2) determine the predicted response of the model by analyzing it under the given loading history; (3) compare the analytical response of the specimen with the measured response and; (4) calibrate the material behavior according to the measured response.

Following the process given above, three shear dominated coupling beams tested by Lee et al. (2003), one flexure dominated and one shear dominated coupling beams tested by Barney et al. (1980), one flexure dominated isolated structural wall tested by
Thomsen (1995) and finally, a multi-story coupled wall system tested by Santhakumar (1974) are modeled and analyzed.

4.2 Verification of Element 16 by Comparison of Horizontally Reinforced Coupling Beams with Analytical Models

As stated previously in Chapter 3, Element 16 is a new fiber beam-column element and in order to evaluate its capabilities, it should be implemented to simple shear dominated beams and analytical estimations should be verified by test results.

Figure 4.1, Figure 4.2 and Figure 4.3 present the load versus deflection behavior comparisons of shear dominated coupling beams tested by Lee et al. (2003) and the DRAIN-2DX models. The rotation of the beam is represented by the x-axis whereas the lateral force by the y-axis.

Specimen 1 and Specimen 3 results estimated by the analytical model match the measured results reasonably well. The pinching effect due to high shear distortion during cyclic loading is captured successfully. However, in Specimen 2, the pinching effect is overestimated and therefore the strength envelope is not fully captured, especially in the negative rotation direction. Generally, based on comparisons of Specimens 1, 2 and 3, Element 16 is able to compute the behavior of shear dominated beams effectively.
Figure 4.1: Comparison of lateral load versus deflection behaviors for Specimen 1: (A) measured behavior from Lee et al. and (B) the analytical model.

Figure 4.2: Comparison of lateral load versus deflection behaviors for Specimen 2: (A) measured behavior from Lee et al. and (B) the analytical model.
4.3 Comparison of Tested Diagonally Reinforced Coupling Beams with Analytical Models

For the verification of the proposed model, two diagonally reinforced coupling beams tested by Barney et al. are used. One of the beams has a span length of 33 in. and a depth of 6.7 in. as given in Figure 3.13. Due to its large span to depth ratio, it can be defined as a flexure dominated beam. The other beam shares the same cross-section properties as the first one but has a 16 in. long span and is defined as a shear dominated beam.

Figure 4.4 shows the comparison of the lateral load versus deflection behavior of the flexure dominated coupling beam test specimen (Beam 1, with $l_n/d = 5$) tested by Barney et al. (1980) versus the DRAIN-2DX model behavior. The relative lateral displacement between the beam the ends and the shear force developing in the beam are represented by the x- and y-axes, respectively. Figure 4.4 indicates that there is a
satisfactory agreement between the analytical model and the experimentally measured behaviors. The analytical model is able to capture the load envelope and the stiffness degradation adequately. Since the amount of steel used in the diagonals were not equal, the response of the tested beam was not symmetric. The asymmetry in the overall strength is captured by the analytical model successfully. Furthermore, the pinching effect due to the asymmetric diagonal reinforcing area is observable in both the test result and the model estimation. The analysis was stopped where the shear resistance of the tested specimen started to drop. In Table 4.1, the measured and analytically estimated maximum and minimum strengths are tabularized. The calculated error is within 6%.

Figure 4.4: Comparison of lateral load versus deflection behavior for Beam 1: (A) measured behavior from Barney et al. and (B) the analytical model

Note that there are discrepancies between the predicted and measured behaviors during unloading. For example, it can be seen that the analytical model underestimates the measured shear force at zero deflection (Figure 4.4). These differences are due to the modeling of the Bauschinger effect in the diagonal reinforcing bars explained in Chapter
3. The effects of the Bauschinger model on the overall behavior of diagonally reinforced coupling beams is studied and discussed in Chapter 5.

**TABLE 4.1**

MAXIMUM AND MINIMUM STRENGTH COMPARISONS – BEAM 1

<table>
<thead>
<tr>
<th>Measured Maximum Strength (kips)</th>
<th>Estimated Maximum Strength (kips)</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.3</td>
<td>8.5</td>
<td>3.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Measured Minimum Strength (kips)</th>
<th>Estimated Minimum Strength (kips)</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>-6.4</td>
<td>-6.8</td>
<td>5.5</td>
</tr>
</tbody>
</table>

Figure 4.5 and Figure 4.6 show the steel strains of the diagonal reinforcing bars at the beam-wall juncture (see Figure 4.7 for strain gauge locations) with respect to the shear force developed in the tested beam and the analytical model. In both comparisons, the corresponding loading cycles are numbered.

Given in Figure 4.5(A), the diagonal bar (Diagonal 1) is in tension when the deflection of the beam is positive. At the end of each loading phase, Diagonal 1 reaches its peak strain. As soon as the beam is loaded in the reverse direction, the bar goes into compression and its strain decreases. Likewise, as seen in Figure 4.6(A), the other diagonal bar (Diagonal 2) goes into tension when the beam is deflected in the negative direction. It should be noted that in Figure 4.6(A), there are some unnumbered intermediate cycles. These cycles refer to later stages of the test when the strain gauges were reported as damaged, and thus, are not included in this comparison.
Figure 4.5: Comparison of lateral load versus diagonal steel strain relationship of Diagonal 1 in Beam 1: (A) measured behavior from Barney et al. and (B) the analytical model

In both comparisons (Figure 4.5 and Figure 4.6), steel strains from the analytical model and the tested beam are generally agreeable, especially considering the inherent variability in individual strain gauge readings in reinforced concrete structures. However, during the first cycle, the diagonals in the tested beam remain essentially in the linear-elastic range, whereas in the analytical model, yielding of the steel is observed. This difference in the initial cycle is caused by the early yielding of the analytically modeled steel as a result of the Bauschinger model implemented.

As seen in Figure 4.5 and Figure 4.6, the strains of the diagonal bars in compression tend to increase gradually and accumulate plastic tensile strains during the reverse cyclic loading history of the beam. This indicates that the beam axially elongates as it is cycled. Since compared to Diagonal 1, the cross-sectional area of Diagonal 2 is smaller, Diagonal 2 accumulates smaller tensile strains when in compression. Note that
the analytical model cannot capture the behavior of Diagonal 2 in compression since the predicted strains remain in compression with no accumulated tensile strains.

![Graph A](image1.png) ![Graph B](image2.png)

Figure 4.6: Comparison of lateral load versus diagonal steel strain relationship of Diagonal 2 in Beam 1: (A) measured behavior from Barney et al. and (B) the analytical model

![Diagram](image3.png)

Figure 4.7: Strain gauge locations on diagonal reinforcing bars (adapted from Barney et al.)

Tables 4.2 and 4.3 present comparisons between the measured steel strains and the strains estimated by the analytical model. As expressed above, due to the early yielding of the analytically modeled steel, the error in the first cycle reaches up to 110% whereas for the later cycles, the error remains within 20%. As an exception, the errors in
the fifth cycle comparisons for Diagonal 2 are greater than %45. The reason for this increased discrepancy in the later cycles for Diagonal 2 is not known.

TABLE 4.2
PEAK STRAIN COMPARISONS OF DIAGONAL 1 IN BEAM 1

<table>
<thead>
<tr>
<th>Cycle No.</th>
<th>Measured Strain</th>
<th>Estimated Strain</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00296</td>
<td>0.00609</td>
<td>106</td>
</tr>
<tr>
<td>2</td>
<td>0.00782</td>
<td>0.00864</td>
<td>10.5</td>
</tr>
<tr>
<td>3</td>
<td>0.01140</td>
<td>0.01307</td>
<td>14.6</td>
</tr>
<tr>
<td>4</td>
<td>0.01511</td>
<td>0.01733</td>
<td>14.7</td>
</tr>
</tbody>
</table>

TABLE 4.3
PEAK STRAIN COMPARISONS OF DIAGONAL 2 IN BEAM 1

<table>
<thead>
<tr>
<th>Cycle No.</th>
<th>Measured Strain</th>
<th>Estimated Strain</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00233</td>
<td>0.00489</td>
<td>110</td>
</tr>
<tr>
<td>2</td>
<td>0.00547</td>
<td>0.00652</td>
<td>19.3</td>
</tr>
<tr>
<td>3</td>
<td>0.00953</td>
<td>0.01125</td>
<td>18.0</td>
</tr>
<tr>
<td>4</td>
<td>0.01302</td>
<td>0.01915</td>
<td>47.1</td>
</tr>
<tr>
<td>5</td>
<td>0.01677</td>
<td>0.02733</td>
<td>63.0</td>
</tr>
</tbody>
</table>

In Figures 4.8 and 4.9, the tensile strain distributions along Diagonal 1 and Diagonal 2 computed by the analytical model are given for three lateral deflection levels. The diagonal bar element number is shown on the x-axis whereas the strain corresponding to each element is shown on the y-axis. There are in total 13 elements for each of the diagonal bars along the beam span.
Despite the claim made by Barney et al. (1980) that the strain distribution was uniform along the diagonal bars (due to loss of bond with concrete), the proposed model did not establish this behavior. As a result of the kinematic constraints provided between the diagonal bar nodes and the concrete beam nodes (i.e., assuming there is full bond between the diagonals and the concrete at these locations), the estimated strains at the
diagonal ends were maximum whereas the strains in the other bar elements gradually decreased inwards. In the central (midspan) elements, the estimated strains remained very close to zero. Additionally, the strain distribution along the bar elements was symmetric with respect to the central bar element. As expected, the strains in each diagonal bar element increased with increasing beam deflection.

Figure 4.10 compares the lateral load versus deflection behavior of a shear dominated coupling beam specimen (Beam 2 with $l_n/d = 2.5$) tested by Barney et al. (1980) with its analytical model. The strength gain due to the decreased span to depth ratio is captured. Compared to Beam 1 represented in Figure 4.4, the underestimation of the shear resistance at zero displacement is not as significant. Asymmetry in the overall strength due to the different amounts of diagonal bars is more evident in the shear dominated beam. In the later loading cycles, the analytical model starts to overestimate the beam strength, which is mostly noticeable just before the diagonal bar fracture in the beam. Similar to Beam 1, there are discrepancies between the predicted and measured behaviors during unloading caused by the Bauschinger modeling of the diagonal steel bars.
Figure 4.10: Comparison of lateral load versus deflection behavior for Beam 2: (A) measured behavior from Barney et al. and (B) the analytical model

In Table 4.4, the measured and analytically estimated maximum and minimum strengths for Beam 2 are tabularized. The calculated error is within 11%.

<table>
<thead>
<tr>
<th>Measured Maximum Strength (kips)</th>
<th>Estimated Maximum Strength (kips)</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.3</td>
<td>13.5</td>
<td>9.7</td>
</tr>
<tr>
<td>Measured Minimum Strength (kips)</td>
<td>Estimated Minimum Strength (kips)</td>
<td>Error in %</td>
</tr>
<tr>
<td>-14.0</td>
<td>-15.5</td>
<td>11.1</td>
</tr>
</tbody>
</table>

Tables 4.5 and 4.6 compare the steel strains in the diagonal reinforcing bars at the beam-wall juncture. It can be seen that as compared to Beam 1, the estimated steel strains for Beam 2 deviate from the measured strains by a significantly larger amount. The reason for this increased discrepancy in Beam 2 is that during the calibration of the
analytical model, the tensile strength of concrete was intentionally underestimated to provide a better match to the measured beam shear force versus deflection behavior.

TABLE 4.5
PEAK STRAIN COMPARISONS OF DIAGONAL 1 IN BEAM 2

<table>
<thead>
<tr>
<th>Cycle No.</th>
<th>Measured Strain</th>
<th>Estimated Strain</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00453</td>
<td>0.00642</td>
<td>41.8</td>
</tr>
<tr>
<td>2</td>
<td>0.01023</td>
<td>0.01975</td>
<td>93.1</td>
</tr>
<tr>
<td>3</td>
<td>0.01733</td>
<td>0.02976</td>
<td>71.7</td>
</tr>
</tbody>
</table>

TABLE 4.6
PEAK STRAIN COMPARISONS OF DIAGONAL 2 IN BEAM 2

<table>
<thead>
<tr>
<th>Cycle No.</th>
<th>Measured Strain</th>
<th>Estimated Strain</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00191</td>
<td>0.00556</td>
<td>191</td>
</tr>
<tr>
<td>2</td>
<td>0.01242</td>
<td>0.01660</td>
<td>33.7</td>
</tr>
<tr>
<td>3</td>
<td>0.01922</td>
<td>0.02863</td>
<td>49.0</td>
</tr>
<tr>
<td>4</td>
<td>0.02603</td>
<td>0.03449</td>
<td>32.5</td>
</tr>
</tbody>
</table>

In Figures 4.11 and 4.12, the analytical tensile strain distributions along Diagonal 1 and Diagonal 2 in Beam 2 along the beam length are given for three lateral deflection levels. There are in total 6 elements for each diagonal bar. Compared to the results for Beam 1 in Figures 4.5 and 4.6, the strain distribution for the diagonal bars in Beam 2 is not symmetric, especially in the central elements. In addition, the strains in the central elements are not zero, unlike in Beam 1.
Figure 4.11: Tensile strain distribution along Diagonal 1 in Beam 2

Figure 4.12: Tensile strain distribution along Diagonal 2 in Beam 2

In Figure 4.10, a representative moment distribution for Beam 1 and Beam 2, and the locations of the central diagonal bar elements are depicted. Under the applied loading, both beams have no moment in the mid-span. As stated before, unlike Beam 1, the strains in the central bar elements of Beam 2 were not zero. This is because the central diagonal
bar elements in Beam 2 are located to the right and left of the beam midspan, where the bending moment is not zero.

Figure 4.13: Representative moment distribution for Beam 1 and Beam 2

4.4 Comparison of Tested Isolated Walls with Analytical Models

Figure 4.14 compares the lateral load versus roof deflection behavior of a flexure dominated isolated wall tested by Thomsen (1995) versus the DRAIN-2DX model
behavior. The horizontal displacement at the roof level and the lateral force applied at the roof level are shown by the x- and y-axes, respectively. Since shear distortion was not a concern for this wall, the concrete and steel fibers are modeled with the simple fiber beam-column element (Element 15). In general, it can be seen that Element 15 is successful in capturing the behavior of the wall.

Figure 4.14: Comparison of lateral load versus deflection behavior for flexural isolated wall: (A) measured behavior from Thomsen et al. and (B) the analytical model

4.5 Comparison of Coupled Wall Test Results with Analytical Results

In Figure 4.15, the complete lateral load versus roof deflection behavior of a coupled wall specimen with diagonally reinforced coupling beams (CSW) tested by Santhakumar (1974) is given. The horizontal displacement of the structure at the roof level is represented by the x-axis. The y-axis corresponds to the proportion of the base shear calculated by adding the lateral forces applied at the floor levels as depicted in Figure 3.16 to the ultimate lateral load capacity of the wall system, $P_u^*$. The wall is exposed to two different cyclic loading ranges. In the first loading range consisting of
cycles 1 through 8, the general behavior was elastic with only a small amount of yielding in the diagonal bars of the coupling beams. The test results indicate that in the first eight loading cycles, the strength degradation and energy dissipation of the structure are small. In addition, the crack patterns show no shear cracking. However, in the second loading range containing cycles from 9 through 16, a more nonlinear behavior is observed. Particularly, after cycle 9, yielding of the wall flexural reinforcement and coupling beam diagonal reinforcing bars is significant. Shear deterioration and stiffness degradation of the coupled wall system are also noticeable.

Figure 4.15: Complete lateral load versus roof deflection behavior of coupled wall system tested by Santhakumar (1974)
Figure 4.16: Comparison of lateral load versus roof deflection behavior of tested coupled wall system: (A) measured behavior from Santhakumar and (B) the analytical model
Figure 4.16 shows the comparison of the lateral load versus roof deflection behavior of the tested wall system versus the DRAIN-2DX model behavior. The proposed model was able to capture the behavior of the structure up to only 2.5 in. of lateral displacement (35% of the total displacement) including cycles 1 through 9. The analytical model result given in Figure 4.16(B) is able to capture the linear-elastic range successfully. In cycle 9, the test specimen was pushed up to $1.2 P_u^*$, where the structure acquired its maximum strength. The analytical model is able to capture the load envelope and the maximum strength by an error of 10%. After this cycle, during the unloading branch, the analytical model failed to converge due to the development of large compression forces in the beams.

In Table 4.7, the measured and analytically estimated maximum strength observed at the end of cycle 9 and the initial stiffness during the first cycle are tabularized. The calculated error for strength comparison is within 10%. Initial stiffness comparison shows an error of 28.5%.

<table>
<thead>
<tr>
<th>Measured Maximum Strength, $P/P_u^*$</th>
<th>Estimated Maximum Strength, $P/P_u^*$</th>
<th>Error in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.18</td>
<td>1.10</td>
<td>9.6</td>
</tr>
<tr>
<td>Measured Initial Stiffness, Force/Deflection ($P_u^*/$in.)</td>
<td>Estimated Initial Stiffness, Force/Deflection ($P_u^*/$in.)</td>
<td>Error in %</td>
</tr>
<tr>
<td>1.33</td>
<td>0.95</td>
<td>28.5</td>
</tr>
</tbody>
</table>
From the analytical estimations, the axial forces in the tension wall and the compression wall were found as $1.88 \, P_u^*$ and $-2.76 \, P_u^*$, respectively, at the end of cycle 9. The resisting moment at the tension wall base was $32.7 \, P_u^*$ in. whereas at the compression wall base $38.7 \, P_u^*$ in. As expected, the compression wall moment and axial force were larger than those in the tension wall.

As discussed in Chapter 2, the crack patterns in the walls after the test showed that the compression wall pier carried a much larger share of the total lateral load than the tension wall pier did. From the analytical estimations, the shear forces at the base of the tension wall and the compression wall were found as $0.45 \, P_u^*$ and $0.65 \, P_u^*$, respectively, at the end of cycle 9. Thus, the compression wall pier base shear strength was 30% larger than the tension wall pier strength.
CHAPTER 5:
PARAMETRIC SENSITIVITY STUDIES

As discussed previously in Chapter 3, the proposed analytical model should be calibrated in order to get satisfactory behavior agreeable with the test results. This chapter shows how changes in material behavior and element properties affect the model performance in terms of strength, cyclic behavior, analysis convergence, etc. These results, presented based on several parametric sensitivity analyses, may help future researchers develop an analytical model in the absence of experimental data.

5.1 Parameters Based on Material Properties

Material property is one of the main factors governing structural performance. As stated in Chapter 4, to generate a reasonable match between the proposed model results and the test results, the analytical representation of material behavior requires calibration. In this section, the main material properties such as the tension strength of concrete and the Bauschinger effect of steel are discussed.

5.1.1 Comparison of Linear and Nonlinear Shear Effects

As stated previously in Chapter 3, since coupling beams, particularly, with small span to depth ratio are exposed to large deformations, they suffer from nonlinear shear
distortions. The proposed model considers the nonlinearity in shear deformations by using a beam-column fiber element called Element 16, which is a modified version of Element 15.

The effect of the linear (Element 15) and nonlinear (Element 16) shear deformations on the behavior of coupling beams is investigated in Figure 5.1, which shows the lateral load versus deflection behavior of Specimen 1 (shear dominated horizontally reinforced coupling beam tested by Lee et al.) modeled with Element 16 (solid thin line) and Element 15 (dashed thick line).

![Figure 5.1: Comparison of lateral load versus deflection behavior of Specimen 1 modeled with Element 16 and Element 15](image)

As given in Figure 5.1, Element 15 established a flexural behavior for Specimen 1 rather than a shear dominated behavior. In addition, Element 15 overestimated the shear force of the beam, especially in larger rotations. It also failed to estimate the stiffness loss.
of the beam due to high shear distortion during unloading. This comparison undoubtedly signifies that Element 16 is a very useful fiber beam-column element for the modeling of coupling beams suffering from shear nonlinearity.

5.1.2 Peak Tensile Strength of Concrete

The effect of the tensile strength, $f_{ct}$ of concrete on the behavior of diagonally reinforced coupling beams is investigated in this section. To conduct this parametric investigation, the following assumptions are made:

1. The unconfined cover concrete around the confined concrete core of the beam has no tension strength.
2. The post-cracking stiffness of the confined concrete is zero.
3. The stiffness of concrete in tension before cracking is equal to $E_c$ determined by Equation 3.5 for each analysis.

Figure 5.2 shows the lateral load versus deflection behavior of Beam 1 (flexure dominated coupling beam tested by Barney et al.) modeled using Element 16 with various tensile concrete strengths. It is evident that the increasing tension strength of concrete improves the performance of the beam in terms of the lateral strength. However, no considerable change in the unloading phase is observed. Furthermore, it can be seen that the shear force at zero displacement is not significantly affected by any modification in the tension strength. Both the high and the very low tension strength models failed prematurely due to numerical convergence problems.
Figure 5.2: Effect of peak tension strength of concrete on cyclic behavior: (A) $f_{ct}$ = 0.0 ksi, (B) $f_{ct}$ = 0.2 ksi, (C) $f_{ct}$ = 0.3 ksi, (D) $f_{ct}$ = 0.45 ksi

Table 5.1 presents the maximum and minimum strength readings at various displacements of the models from Figure 5.2. It can be seen that increasing the tension strength by 50% from 0.2 ksi to 0.3 ksi and from 0.3 ksi to 0.45 ksi resulted in an increase in the lateral strength by about 10%.
5.1.3 Behavior of Concrete in Tension after Cracking

The tensile strength of concrete after cracking may affect the performance of diagonally reinforced coupling beams. In this section, the effects of the concrete tension stiffness after cracking on the behavior of diagonally reinforced coupling beams are investigated. To conduct this parametric investigation, the following assumptions are made:

1. The unconfined cover concrete around the confined concrete core of the beam has no tension strength.
2. The peak tension strength and initial uncracked stiffness of concrete are taken as 0.3 ksi and 3100 ksi, respectively, for each analysis.
3. Once concrete reaches the peak tension strength a second stress-strain point is used to define the post-cracking stiffness. For each analysis case, the stress of the second stress-strain point is taken as 0.01 ksi. The
corresponding strain value is determined based on the desired post-cracking stiffness.

Figure 5.3: Behavior of concrete in tension

In Figure 5.3, the behavior of concrete in tension for four analysis cases is shown. Likewise, Table 5.2 lists the concrete tension stiffness after cracking in ksi for each analysis case.

Figure 5.4 presents the lateral load versus deflection behavior of Beam 1 (flexure dominated coupling beam tested by Barney et al.) modeled with Element 16 with the different tensile stiffness representations of concrete after cracking.
TABLE 5.2
STIFFNESS OF CONCRETE IN TENSION AFTER CRACKING

<table>
<thead>
<tr>
<th>Analysis No.</th>
<th>Stiffness after Cracking (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E&lt;sub&gt;ct1&lt;/sub&gt;</td>
<td>0 (flat)</td>
</tr>
<tr>
<td>E&lt;sub&gt;ct2&lt;/sub&gt;</td>
<td>-0.98</td>
</tr>
<tr>
<td>E&lt;sub&gt;ct3&lt;/sub&gt;</td>
<td>-9.86</td>
</tr>
<tr>
<td>E&lt;sub&gt;ct4&lt;/sub&gt;</td>
<td>-101.62</td>
</tr>
</tbody>
</table>

Figure 5.4: Effect of concrete tension stiffness after cracking on cyclic behavior:
(A) E<sub>ct1</sub>, (B) E<sub>ct2</sub>, (C) E<sub>ct3</sub>, (D) E<sub>ct4</sub>
By inspecting Figure 5.4, firstly, it can be seen that increasing the tension stiffness of concrete after cracking improves the numerical stability of the analytical model. The model with stiffness of $E_{ct1}$ was able to complete all hysteresis loops whereas the analysis case with $E_{ct4}$ did not converge after reaching a deflection of 0.5 in. Cases $E_{ct2}$ and $E_{ct3}$ failed at the same displacement; however Case $E_{ct3}$, which has a relatively smaller stiffness generated smaller lateral shear resistance in the early cycles. It can be seen that the shear force at zero displacement is not affected by the concrete stiffness after cracking significantly.

Table 5.3 presents the maximum and minimum lateral strength readings at various displacements of the models from Figure 5.4. It can be seen that when the tension stiffness of concrete after cracking decreases, the lateral strength generated by the beam decreases, but not by a large amount.

<table>
<thead>
<tr>
<th>$E_{cti}$</th>
<th>Deflection = ± 0.35 in.</th>
<th>Deflection = ± 1.75 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Strength (kips)</td>
<td>Minimum Strength (kips)</td>
</tr>
<tr>
<td>1</td>
<td>7.29</td>
<td>-6.02</td>
</tr>
<tr>
<td>2</td>
<td>7.00</td>
<td>-6.13</td>
</tr>
<tr>
<td>3</td>
<td>6.88</td>
<td>-5.76</td>
</tr>
<tr>
<td>4</td>
<td>5.65</td>
<td>-4.18</td>
</tr>
</tbody>
</table>
TABLE 5.4

STRAIN READINGS AT VARIOUS DEFLECTIONS – AFTER CRACKING BEHAVIOR

<table>
<thead>
<tr>
<th>$E_{ei}$</th>
<th>$E$</th>
<th>Diagonal 1 Strain</th>
<th>Diagonal 2 Strain</th>
<th>Diagonal 1 Strain</th>
<th>Diagonal 2 Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0089</td>
<td>0.0092</td>
<td>0.0519</td>
<td>0.0561</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.0082</td>
<td>0.0107</td>
<td>0.0528</td>
<td>0.0593</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.0086</td>
<td>0.0112</td>
<td>0.0538</td>
<td>0.0686</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.0096</td>
<td>0.0115</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.4 gives the average tensile strain readings at both ends of the two diagonal bar groups at various displacements. It is found that as the post-cracking stiffness increases, the strains in the diagonal bars decreases in later cycles.

5.1.4 Bauschinger Effect

The unloading behavior of a coupling beam as well as its stiffness after cracking of the concrete in tension is mainly governed by the diagonal reinforcing bars. As explained previously in Chapter 3, in order to model the loading and unloading performance of diagonally reinforced beams adequately, Bauschinger effect is introduced into the diagonal steel reinforcement models. In addition, as discussed in Chapter 4, Bauschinger effect plays an important role in the shear forces at zero displacement. In this section, a parametric study on the Bauschinger effect is presented.
Figure 5.5: Monotonic stress-strain relationships for Diagonal 1

Figure 5.6: Monotonic stress-strain relationships for Diagonal 2
In Figure 5.5 and Figure 5.6, for each diagonal reinforcement group, three steel stress-strain models with various Baushinger effects as well as the experimental material data are given. The bilinear model is the simplest steel model, which has one yielding point and one strain hardening ratio. Multi-linear stress-strain models with low and high Baushinger effect are designed with multi-truss elements following the algorithm explained in Chapter 3. The model with low Baushinger effect has higher yielding points than the one with high Baushinger effect. Mainly, the different Baushinger models only vary in the strain range of $0 \leq \varepsilon \leq 0.03$. After $\varepsilon = 0.03$, the three steel models merge.

Figure 5.7 presents the lateral load versus deflection behavior of Beam 1 (flexure dominated diagonally reinforced coupling beam) with various Baushinger effects. The test result for this beam is given in Figure 4.4 and discussed in Chapters 2 and 4.

Figure 5.7 demonstrates that the steel behavior can radically affect the model, especially during unloading. The bilinear model has linear loading and unloading branches whereas the other two models have smoother curves. Even though the load-deflection envelope of the bilinear model is similar to the other models, due to the relatively sharp unloading behavior, the bilinear model generates higher shear forces at zero deflection as compared to the other two models. The model with low Baushinger effect generates higher shear force at zero deflection than the model with high Baushinger effect as expected. In addition, the high Baushinger model has softer curves during unloading and reloading. All three models generate similar force envelope after a deflection of $\pm 0.8$ in.
Figure 5.7: Bauschinger effect: (A) bilinear model, (B) model with low Bauschinger effect and (C) model with high Bauschinger effect

Figure 5.7 shows that the strength in early load cycles, strength at zero displacement and the general unloading characteristics of the model can be modified by adjusting the Bauschinger effect in the diagonal bars. Additionally, the analysis results show that increasing the Bauschinger effect provides better numerical stability to the model.
Figure 5.8: Cyclic behavior of Diagonal 1 (D1) and Diagonal 2 (D2) at various deflections: (A) ±0.35 in./D1, (B) ±0.35 in./D2, (C) ±0.70 in./D1, (D) ±70 in./D2, (E) ±1.20 in./D1, (F) ±1.20 in./D2
In Figure 5.8, the cyclic stress-strain behaviors of the diagonal bars at various deflections are given. Figure 5.8(A), (C) and (E) shows the Diagonal 1 stress-strain behavior whereas Figure 5.8(B), (D) and (F) show the Diagonal 2 behavior. It can be seen that the models with Bauschinger effect underestimate the steel stresses and strains at each given deflection, especially during the unloading and reloading phases. Due to the smaller forces developing in the bars in these phases, the shear force of the beam at zero deflection is underestimated.

Based on these findings, in order to reduce the underestimation of the beam shear force at zero deflection, a new steel behavior simulating Bauschinger effect may need to be developed. In this new, the material should follow a bilinear path during loading and a multi-linear path during unloading as given in Figure 5.9.

Figure 5.9: Suggested Bauschinger model
5.2 Parameters Based on Element Properties

Element property is one of the factors affecting structural performance. In this section, the effects of fiber element and truss element length on the behavior of diagonally reinforced coupling beams are discussed.

5.2.1 Fiber Element Length

Figure 5.10 shows lateral load versus deflection behavior of Beam 1 (flexure dominated coupling beam tested by Barney et al.) modeled with various fiber and truss element lengths. Each curve in Figure 5.10 has the same material properties but the lengths of the concrete fiber column-beam elements as well as the horizontal lengths of the diagonal truss bar elements are different. The element length is varied between one to four times the transverse reinforcement spacing. All diagonal truss bar elements are slaved to the concrete fiber elements at intermediate nodes.

It can be observed that the element length does not significantly affect the general behavior of the beam in terms of the shear strength capacity, considerably. However, in some hysteresis loops, the shear force in model 2s and 4s show slight variations. Even if the model with element length of 1s failed prematurely due to numerical convergence problems, its lateral force versus deflection behavior coincide with other model estimations and shows promises that it will follow a similar load-deflection behavior.
Figure 5.10: Effect of element length on the on cyclic behavior: (A) 2s, (B) 4s, (C) 1s

In Figure 5.11 and Figure 5.12, the average tensile strain distributions of Diagonal 1 and Diagonal 2 along the span length are given at various deflections of the beam.
It can be seen that at a deflection of ±0.35 in. where all of the three models have roughly the same shear strength, the average end strains of the diagonals from 1s model are the largest. At the same time, since each element of the 4s model is as long as four elements from 1s model or two end elements from 2s model, the 4s model basically shows the average strains generated by the other two models over the respective length. At ±2.00 in. deflection, the 4s model again averages the strains of the 2s model.
5.2.2 Truss Element Length

As stated previously in Chapter 3, in order to simulate a discrete bond effect between concrete and the diagonal reinforcing steel, the inelastic truss bar elements are slaved to the concrete fiber elements at intermediate nodes along the beam length. This section investigates the effect of this model on the behavior of diagonally reinforced coupling beams and on the strains in the diagonal bars.

Figure 5.13: Effect of bondage on cyclic behavior

Figure 5.13 shows the lateral load versus deflection behavior of Beam 1 (flexure dominated coupling beam tested by Barney et al.) modeled with a discrete bond effect (dashed line) and no bond effect (solid line). In the model with discrete bond effect, the horizontal length of each diagonal truss bar element is equal to two times the transverse...
reinforcement spacing. In the model with no bond effect, the horizontal length of the diagonal bar element is equal to the span length of the beam. It is assumed that the concrete stiffness after cracking is equal to zero.

As shown in Figure 5.13, since the steel strains in the model with no bond are less than the strains in the one with discrete bond, the lateral strength generated by the beam is smaller. In addition, the model with discrete bond effect has better numerical stability.

**TABLE 5.5**

**STRAIN READINGS AT VARIOUS DEFLECTIONS – BOND EFFECT**

<table>
<thead>
<tr>
<th>Bauschinger Effect</th>
<th>Deflection = ± 0.25 in.</th>
<th>Deflection = ± 0.5 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diagonal 1 Strain</td>
<td>Diagonal 2 Strain</td>
</tr>
<tr>
<td>Discrete Bond</td>
<td>0.00591</td>
<td>0.01221</td>
</tr>
<tr>
<td>No Bond</td>
<td>0.00158</td>
<td>0.00361</td>
</tr>
</tbody>
</table>

Table 5.5 gives the average tensile strain readings at the ends of the two diagonal bar groups at various displacements. The diagonal steel bars of the model with discrete bond generated three times more strain than the model with no bond. This fact shows that implementing a discrete bond model in the analysis increases the strength of the beam by increasing the diagonal steel strains.
CHAPTER 6:  
SUMMARY, CONCLUSIONS AND FUTURE WORK

In this chapter, the findings from the research presented in this thesis are summarized and recommendations for future research are suggested. In Section 6.1, a brief summary of the research is given by emphasizing the most important findings. Section 6.2 presents the conclusions resulting from the verification of the proposed model and the parametric studies. Finally, Section 6.3 suggests how the current proposed model can be improved based on the findings.

6.1 Summary

This thesis describes the analytical modeling of diagonally reinforced concrete coupling beams and multi-story coupled wall systems with diagonally reinforced coupling beams under lateral reversed cyclic loads. The analytical model is verified using the test results of a flexure dominated and a shear dominated coupling beam and a relatively coupled wall system based on the lateral load versus deflection behavior and the lateral load versus diagonal steel strain relationship. The comparisons yielded generally satisfactory results, but some measurements such as the shear strength at zero deflection or unloading after cyclic loading could not be estimated accurately due to inadequacies in the Bauschinger model implemented for the diagonal reinforcing steel.
The thesis is intended to further the understanding of the behavior of the proposed model by changing several important parameters to identify the effects of material and element properties on the overall behavior of the beam as well as local parameters such as the steel strain distribution along the diagonal bars. This parametric study will help future researchers develop their own analytical models in the absence of experimental data.

6.2 Conclusions

The research presented in this thesis has produced a number of conclusions. The main conclusions are presented below.

1) This thesis proposes an analytical model to estimate the behavior of diagonally reinforced concrete coupling beams and coupled wall systems under reversed cyclic lateral loading. The analytical model has the capability to take account of the nonlinear shear distortions in the beams.

2) Element 16 is used as the main fiber beam-column element in coupling beam models. The effectiveness of Element 16 is verified by three shear dominated horizontally reinforced coupling beam models.

3) The proposed model is verified by comparing the measured behavior of three test specimens under cyclic lateral loads with analytical estimations. Two of the tests were diagonally reinforced coupling beam subassemblies, one flexure dominated and the other shear dominated. The remaining one test was a multi-story coupled wall system with diagonally reinforced coupling beams. The comparisons indicate that there is satisfactory
agreement between the analytical models and the experimental results. The models were able to capture satisfactorily the shear strength, pinching effect and the asymmetric load-deflection behavior of the beams due to the disproportionate amounts of steel used in the diagonals.

4) The following conclusions can be made based on parametric sensitivity study conducted:

- **Effect of nonlinear shear deformations:** Since coupling beams are relatively short, it is inevitable that they suffer from nonlinear shear distortions. The parametric analysis showed that Element 16 is capable of capturing the behavior of coupling beams suffering from shear nonlinearity.

- **Peak tensile strength of concrete:** The study shows that the peak concrete tension strength plays an important role in shear strength and numerical stability of the proposed model.

- **Behavior of concrete in tension after cracking:** Concrete tension stiffness after cracking is one of the very important aspects governing the lateral strength, numerical stability, unloading branch and diagonal steel strains of the model.

- **Bauschinger effect of the diagonal bars:** The analytical estimations show that the shear force at zero displacement and the general unloading characteristics of the model can be modified by adjusting the Bauschinger modeling of diagonal bars.
- Bond effect: Modeling of discrete bond along the diagonals causes increased strains in the diagonals, which results in an increase in the axial tension and compression forces in the bars, and consequently, an increase in the lateral strength of the beam. It also improves the numerical stability of the model as well.

- Element length and number: The analytical estimations indicate that the number and length of the horizontal fiber elements and the diagonal truss elements do not change the load-deflection behavior of the coupling beams significantly. However, the models with longer element length underestimate the steel strains.

6.3 Future Work

Currently, the proposed model requires a long and troublesome calibration process. A method should be developed to shorten this process. Through this method, the Bauschinger effect, tensile strength of the concrete before and after cracking should be quantified in the absence of experimental results.

This model does not consider the buckling of the diagonal bars under large deformations. Buckling effect should be implemented into the steel material models and the modified models should be verified using results of previously tested beams with buckling of diagonals.

Using the proposed model, a parametric analytical study should be conducted on the effects of several structural design properties (such as the beam dimensions, material properties and reinforcement areas) on the lateral strength, stiffness, and cyclic behavior...
of diagonally reinforced coupling beams. These results can then be used to develop design-oriented, closed-form procedures to estimate the lateral load behavior of the beams based on the fundamental engineering principles of equilibrium, kinematics, and constitutive relationships.

The most common problems encountered during the analyses using the proposed model were stability and enormous time consumption of the analysis. Other analytical platforms such as finite element models, should be investigated to overcome these problems.

In order to prevent the underestimation of the shear force at zero deflection, a new steel model that better simulates the Bauschinger effect should be developed. In this model, material should follow a bilinear path during loading and a multi-linear path during unloading.
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