DEVELOPMENT OF A SEISMIC DISSIPATING MECHANISM
FOR PRECAST CONCRETE CLADDING PANELS

A Thesis in
Architectural Engineering

by

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ABSTRACT

Modern earthquake-resistant design aims to isolate architectural precast concrete panels from the structural system so as to reduce the interaction with the supporting structure and hence minimize damage. The present study seeks to maximize the cladding-structure interaction by developing an energy dissipating cladding system (EDCS) that is capable of functioning both as a structural brace, as well as a source of energy dissipation. The EDCS is designed to provide added stiffness and damping to buildings with steel moment resisting frames with the goal of favorably modifying the building response to earthquake-induced forces without demanding any inelastic action and ductility from the basic lateral force resisting system. Because many modern building facades typically have continuous and large openings on top of the precast cladding panels at each floor level for window system, the present study focuses on spandrel type precast concrete cladding panel. The research work was divided broadly into four main phases: literature study; preliminary study; component level study; and building level study. The preliminary design of the EDCS was based on existing guidelines and research data on architectural precast concrete cladding and supplemental energy dissipation devices. In the component-level study, the preliminary design was validated and further refined based on the results of nonlinear finite element analyses. The stiffness and strength characteristics of the EDCS were established from a series of nonlinear finite element analyses. Using elastic theory, simple expressions were derived to approximate the lateral stiffness of the EDCS. In addition, a simple mathematical model of the EDCS was developed to facilitate the preliminary design of buildings incorporating EDCS. From the results of the nonlinear time history analyses performed on a 9-story and a 20-story moment frames, the EDCS was found to be effective in reducing interstory drift and member forces through the combined action of added stiffness and energy dissipation.
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CHAPTER 1
INTRODUCTION

1.1 PROBLEM STATEMENT

1.1.1 Nonstructural Damage From Earthquake

The role of nonstructural elements in building seismic performance became apparent in the 1964 Alaska earthquake and was seen even more clearly in the 1971 San Fernando and 1972 Managua, Nicaragua earthquakes [Arnold et al., 1987]. Extensive damage to nonstructural components was common to these major earthquakes. Arnold et al. [1987] attempted to quantify the cost of the building damage resulting from several major earthquakes prior to 1985. The study revealed that nonstructural elements contributed 30% of the overall damage in which as much as 40% was attributed to damage from exterior claddings. Significant cladding damage has also been reported in the 1978 Miyagiken-Oki, 1987 Whittier Narrows and the 1995 Hyogoken-Nambu earthquakes [McMullin, 2000].

As discussed by several authors [Oliveto, 2002], one of the most challenging issues from a traditional earthquake resistance design standpoint is that of reducing nonstructural and building content damage. This aspect is often overlooked and when addressed, can be very costly to incorporate as part of the traditional earthquake design. There are two primary mechanisms causing nonstructural damage. The first is through direct vibration (acceleration), transmitted into a nonstructural element through a building structure that has been excited by ground motion. The second is through the significant drift or yielding of the structural element, which in turn causes damage to an attached or adjoining nonstructural component. For the first mechanism, the nature of the structural system determines the extent to which accelerations originating in ground movement are damped or amplified as they are transmitted through the structure. In the latter, although structural interaction is the immediate cause of the damage, the main cause is drift, yielding or damage to the building structure.

As a result, there are two different and contradicting design philosophies that are debated within the structural engineering profession concern with minimizing nonstructural damage. The
first concept is to reduce interstory drift and therefore, the building should be designed to be as stiff as possible. The drawback of this concept is the development of high floor accelerations. The opposing idea is flexible building design that tends to attract less force and lower floor accelerations. On the other hand, flexible buildings generate much higher interstory drifts and damage is more likely to deformation-sensitive components. A design solution resulting in reduction of both interstory drift and floor accelerations is preferred.

1.1.2 Effect of Architectural Cladding on Building Performance

Architectural precast concrete cladding systems are considered as a non-load bearing wall system and are designed primarily to transfer their self-weight and out-of-plane (wind and earthquake) lateral loads to the supporting building structure. The contribution of the cladding system to the lateral stiffness of the building is often ignored in the structural design.

Studies have shown that these architectural components can contribute significantly to the lateral stiffness of the structure and that the panels can be subjected to significant in-plane forces. Experiments conducted on actual buildings have confirmed that structural frames clad with precast concrete panels can be much stiffer laterally than the stiffness calculated for the frames alone [Ellis 1980, Goodno and Will 1978, Wiss and Curth 1970].

Modern earthquake-resistant design requires that these cladding panels be isolated from the lateral load resisting system with the aim of preventing the panel from participating in the building response, hence minimizing damage on the cladding panels. Flexible connections or connections that allow relative movements at the attachment points as suggested by PCI [1989], are all conceived to minimize the cladding-structure interaction. However, based on post-earthquake observations, the use of such cladding connection details would still induce some form of interaction effect between cladding and structural system [Pinelli et al., 1990] and could restrain the racking deformation of the structural frame, and as a result stiffen it significantly against any lateral loading [Gaiotti and Smith, 1992].

Rather than emphasizing the need to minimize cladding-structure interaction, some research is progressing towards the opposite direction that seeks ways to maximize the benefits of cladding-structure interaction while attempting to minimize the damaging effect on the cladding panels. This cladding-structure interaction is further exploited by some researchers through the development of an energy dissipating mechanism in the cladding connections. An energy
dissipating system is an innovative approach of improving the control of building seismic response, which includes reducing lateral floor acceleration and interstory drift, based on the addition of special mechanical devices to the structural system. The concept of this approach is controlling energy demands and supplies in the energy balance equation which is explained in greater detail in Chapter 2. Hence, by using energy dissipating or protective devices, it is possible to control the interstory drift and to lower the accelerations, subsequently not only to control the damage to the primary structural members but also to reduce the damage to the nonstructural components.

Modern building facades typically have continuous and large openings on top of the spandrel beams or precast cladding panels at each story for the installation of windows or ventilation requirement (e.g., for open parking structure) as shown in Figure 1.1. However, existing studies on the special cladding system were confined to story high or floor-to-floor type cladding panels that span from one floor to an adjacent floor. Such systems would require modifications and necessitate further study for buildings requiring significant openings in the building facade.

![Figure 1.1: Spandrel Cladding Panels Used as the Building Facade.](image-url)
1.2 OBJECTIVES AND SCOPE OF RESEARCH

This research program is aimed at enhancing the performance of the building seismic response by using the implicit advantage of the building enclosure integrated in the lateral load resisting system of the building. The primary objective of this research is to develop an added energy dissipating system which can be applicable specifically to spandrel type precast concrete cladding panels, although the concept can also be applied to floor-to-floor high panels. This spandrel type Energy Dissipating Cladding System (EDCS) would be able to function as a structural brace while providing some form of controlled energy dissipation. In an earthquake event, the EDCS is designed to improve the building’s vibration control characteristics and, more importantly, to significantly reduce interstory drift. The design objective is to ensure that no damage will be incurred on the cladding panel and its connections as a result of the cladding-structure interaction, and to limit all irreversible inelastic deformation in the EDCS without demanding any inelastic action and ductility from the basic Lateral Force Resisting System (LFRS). More importantly, unlike existing solutions that focus primarily on added energy dissipation rather than providing lateral load resistance, the present research seeks to implement the EDCS as a form of structural brace.

The current research is limited to rectangular-shaped architectural precast concrete spandrel panels without openings. The performance of the EDCS was evaluated only on structural steel moment frames though it is expected that the same concept could be applied to other structural systems.

The research work was divided broadly into four phases: literature study; preliminary study; component level study; and building level studies. The objectives and the tasks specific to each phase are described below.

Phase 1: Literature Review

An extensive literature research was carried out with the aim of better understanding the state-of-the-art works in areas related to the present study. These areas include seismic analysis and design requirement, precast concrete cladding system, vibration control with emphasis on the application of passive energy dissipation system for wind or seismic loading in buildings, and structural dynamic analysis and modeling techniques. In addition, information (e.g., technical specifications, test reports, etc.) on commercially available precast cladding products and energy
dissipation systems (e.g., Taylor Devices, Inc, Pall Dynamics Ltd, etc) were obtained as part of the literature search.

**Phase 2: Feasibility Study**

A series of linear and nonlinear analyses were carried out to evaluate the effectiveness of the different form of spandrel type bracing system and supplemental energy dissipating devices. The specific tasks were to:

- Investigate the effectiveness of different forms of one-third height spandrel bracing on a three-dimensional model of a 4-story moment frame building to withstand lateral loading using code-specified equivalent linear procedure; and
- Investigate the effect of adding spandrel type bracing and different supplemental energy dissipating devices on a two-dimensional 3-story shear frame model using a nonlinear time history method.

**Phase 3: Component Level Studies**

Under this phase, a detailed design of the EDCS was developed based on existing design guidelines. The behavior of the EDCS and its major components were studied using nonlinear finite element techniques with the aim of establishing the stiffness and strength characteristics of the EDCS. In addition, the finite element results were used to validate a simplified mathematical model of the EDCS. The specific tasks were to:

- Develop the operational concept behind the EDCS;
- Design the various components of the EDCS based on available design guidelines;
- Review existing literature on finite element modeling of reinforced concrete members with ANSYS;
- Identify and calibrate critical parameters defining the constitutive models of reinforced concrete and to verify the suitability of the finite modeling approach through the analysis of a well-understood reinforced concrete element and correlating the numerical results with available test results;
- Validate the appropriateness of the finite element modeling strategy for concrete anchoring systems by comparing the numerical results with existing design guidelines;
- Model and analyze the EDCS by extending the finite element modeling strategies from the calibration studies;
Investigate the stiffness and strength characteristics of the EDCS;
Develop and refine reinforcement and connection details for the precast concrete panel developed at the beginning of the component study; and
Develop simple and accurate mathematical representation of the in-plane load-deformation characteristics of the EDCS for subsequent use in building level analyses.

Phase 4: Building Level Studies
Under Phase 4, the performance of the EDCS on the building response was evaluated and quantified. The tasks under this phase were to:

- Perform a detailed review of the characteristics of the test structures of Aiken and Kelly [1990];
- Develop accurate analytical models of the test structures that are capable of capturing the observed dynamic response characteristics of the test structures;
- Develop analytical models of the test structures with EDCS using the simplified mathematical model of the EDCS developed in Phase 3;
- Perform a comparative study to evaluate the performance of the EDCS in relation to conventional moment-resisting frame with and without supplemental energy dissipating devices; and
- Demonstrate the earthquake-resistant design of a contemporary building incorporating EDCS and to compare the design results with that based on traditional design approach.

1.3 THESIS ORGANIZATION

The thesis is organized into nine chapters corresponding to the different phases of the research program outlined above. A brief description of each chapter is presented here.

Following the Introduction, Chapter 2 summarizes the concept, design philosophy and current research in three areas most relevant to the current study: precast concrete cladding, energy dissipating system and cladding system with energy dissipation mechanism.

The results of the feasibility studies are presented in Chapters 3. Chapter 3 summarizes the preliminary analytical studies that were performed to investigate the effect of adding perimeter spandrel bracing and supplemental energy dissipation devices on the static and dynamic response of typical ordinary moment resisting frame buildings. In the first part of the chapter, the
effect of one-third height perimeter spandrel bracing on the lateral force resistance of a three-
dimensional model for a typical 4-story building was investigated using the equivalent static force
procedure. In the second part, nonlinear time history analysis was employed to study the effect of
added spandrel bracing system and various energy dissipation systems on the dynamic response
of a two-dimensional model of a 3-story building subjected to actual earthquake motion.

In Chapter 4, the concept and the detailed design of the various components of the EDCS
are presented. The function and design of the EDCS components are described. The detailed
design example is presented in Appendix A.

Chapter 5 covers the nonlinear finite element modeling strategy for EDCS using a
commercial finite element analysis package. Firstly, a review of the constitutive models and
element types suitable for modeling the complex nonlinear behavior of reinforced concrete was
performed. This is followed by a discussion of the current strategies adopted by researchers for
finite element modeling of reinforced concrete beams. To verify the suitability of the modeling
strategy and to calibrate the parameters defining the various constitutive models, a well-
understood reinforced concrete element (i.e., deep beam) was analyzed and the numerical results
were correlated with available experimental data. In addition, the behavior of headed concrete
anchors, typically used as an effective anchorage system for precast concrete cladding, was
modeled for different loading conditions and the results were compared with the nominal values
estimated from prevailing design code. The modeling strategies were extended to the nonlinear
finite element analysis of the EDCS. The EDCS was modeled with different reinforcement and
connection details. The numerical results provided valuable information on the stiffness and
strength characteristics of the EDCS and were also used to improve the detailing of the EDCS.

Chapter 6 extends the numerical results of the finite element study in Chapter 5 to the
development of a simplified mathematical model for the EDCS. The simple mathematical
formulation was intended to be readily added to the mathematical model of any buildings or
structures under consideration, while correctly capturing the in-plane stiffness and strength
characteristics of the EDCS. The in-plane load-deformation characteristics of the EDCS was
broken down into its major components, and rational methods of estimating the stiffness for each
component were presented. The approximate closed-form solutions were correlated with the
numerical results from the finite element analysis.

In Chapter 7, the EDCS was incorporated into the analytical models the representative
(prototype) building to evaluate its performance in terms of lateral stiffness and energy
dissipation. Rather than considering a fictitious building for the purpose of the comparative study,
the EDCS was added to the analytical models of the structures tested by Aiken and Kelly [1990].

Accurate mathematical models of the different test structures used in the experiment were
developed and correlated with available experimental results. In the comparative study, the EDCS
was added to the calibrated building model and its performance was evaluated and presented in
Chapter 7.

Chapter 8 represents an effort to illustrate the design of the EDCS for a typical 20-story
building and to compare the results with the traditional method of earthquake-resistant design.
The seismic design criteria are first presented followed by the use of linear static and time-history
procedures for the design of the ordinary moment resisting frame. The design of the moment
frame incorporating EDCS using nonlinear time history methods are presented next. The
influence of the slip load on the building response and selection of the appropriate slip load are
also discussed.

The report concludes with a summary of the major findings from the research program.
Other important considerations (e.g., architectural and construction, etc.) are also discussed.
Recommendations for future research on the subject matter are proposed and included in Chapter
9.
CHAPTER 2
LITERATURE RESEARCH

2.1 CHAPTER OVERVIEW

The present research is concerned with the development of an innovative architectural precast cladding system capable of functioning as a structural bracing system with energy dissipation capability at partial height configuration. To achieve this, a literature search was first carried out to evaluate the state-of-the-art information in a number of important areas. There is a need to review the latest knowledge in the area of architectural precast concrete cladding system, more especially in the area of earthquake-resistant design. Because of the need to incorporate some form of energy dissipating mechanism into the cladding system, a thorough review of the different passive energy dissipation systems used in building application must be performed. More importantly, it would be necessary to obtain information about research programs that have focused the use of energy dissipation concept in architectural precast cladding system. These are the objectives of the literature research and the findings are summarized in the current chapter.

Following the chapter overview, the current knowledge in the design and application of conventional architectural cladding systems are presented. Analytical and experimental studies conducted by other researchers to better quantify the behavior of the cladding systems are also mentioned. The different forms of passive energy dissipating systems currently available for structural application are evaluated here. The basic underlying principles and the mathematical modeling approaches behind each system are discussed. Last but not least, current research that focus on the application of energy dissipation concept on architectural cladding system are reviewed.
2.2 ARCHITECTURAL CLADDING SYSTEM

2.2.1 Background

The exterior facade of a building serves three main purposes [Das, 1986]: as a major element in defining the aesthetics image; separating the building interior environment from the exterior, and controlling the amount of heat, cold, sunlight, moisture, wind, and noise into the building; and resisting all types of external forces applied to the building. Exterior facade may be divided into three primary groups based on the way they are assembled and installed and the type of structural subframing. These are unit assemblies, grid assemblies, and built-up assemblies. The details are presented in the followings:

- **Unit Assemblies** – Figure 2.1 illustrates some commonly used curtain wall units (i.e., spandrel type or strip precast panel, floor-to-floor type precast panel, multi-floor type precast panel). Generally these units are precast concrete or prefabricated metal panels.
- **Grid Assemblies** – Grid assemblies consist of vertical, horizontal, or a combination of vertical and horizontal mullions which are designed to carry the weight of infill panels and lateral loads. The examples of grid assemblies are shown in Figure 2.2.
- **Built-up Assemblies** – Built-up assemblies can be either built-in-place or prefabricated. Exterior facades with brick or stone facing masonry block or stud backup, are common examples of built-up walls as shown in Figure 2.3.

![Figure 2.1: Unit Assemblies [Das, 1986].](image-url)
Figure 2.2: Grid Assemblies [Das, 1986].

Figure 2.3: Built-up Assemblies [Das, 1986].
Spandrel type cladding, one form of unit assemblies, is a long horizontal element that spans from column to column at each floor level. This type of panel is fully supported by a perimeter beam at each floor level though it is not uncommon for the cladding to be attached to the columns for out-of-plane stability. On the other hand, a floor-to-floor type cladding spans from one floor and is attached to the adjacent floor while a multi-floor type cladding spans two or more floors. The different arrangement of precast cladding panels is shown in Figure 2.4.

Figure 2.4: Typical Arrangements of Precast Cladding Panels [Arnold et al., 1987].
The number and location of connection points is controlled by the panel type and size. Typically, an economical design aims to achieve the largest possible size of panel with the least number of connections. Recommendations on the locations and restraint conditions for various panel sizes are included in the PCI Manual for Structural Design of Architectural Precast Concrete [PCI, 1989] as shown in Figure 2.5.

Figure 2.6: Typical Connection Arrangement recommended by PCI [PCI, 1989].
Generally, cladding panel is attached to the building structural system at four points. In floor-to-floor type cladding panels, two bearing-type connections are usually placed at the bottom of the panel at one floor level. These connections are designed to take the full weight of the panel. Two tie-back connections are placed at the upper part of the panel to the upper floor level to resist out-of-plane loads. An alternative design is to place the load-bearing connections at the top and tie-back connections at the bottom; this is typical for spandrels. However, the connections for spandrel type panels vary greatly depending on the detail of the perimeter of the floor system. For tall, thin panels such as column covers, lateral support at an intermediate level is often necessary. The terminology used in describing precast concrete unit (floor-to-floor type) is illustrated in Figure 2.7.

![Figure 2.7: Terminology for Precast Concrete Units [PCI, 1989]](image)

For the connections of concrete cladding panel, PCI [PCI, 1989] defined “bearing (direct and eccentric) connections” as connections intended to transfer vertical loads to the supporting structure or foundations or rigid supports where movements are negligible. Eccentric bearing connections are usually used for panels above the first support level where movement of the support system is likely. “Tie-back (lateral) connections” are defined as connections intended to keep the precast concrete panel in a plumb or other desired position and transfer out-of-panel wind and seismic loads. Some typical examples of these connections are shown in Figure 2.8.
2.2.2 Research on Cladding Panel and its Connections

Nonstructural precast concrete claddings are typically designed to support their own weight and transfer both gravity and lateral loads to the primary building structure. Due to the deflection of the supporting elements, PCI [1989] recommended to support the entire weight of the panel at one level. If supported by more than one floor, any differential deflection of supporting building frame members may cause the weight distribution to be indeterminate. Figure 2.9 illustrates the basic design principles.

Figure 2.8: Examples of Connections for Precast Concrete Panel [PCI, 1989].
In cladding design, the simplest and most direct load transfer paths through the connections and ductility are preferred. The number of load transfer points should also be kept to a practical minimum. These measures are to reduce the sensitivity of the connection to deflection, and also to minimize the need to accurately calculate loads from, for example, volume changes and building frame distortions. To keep the load path determinate, there should not be more than two bearing connections per panel to transfer gravity loads and the bearing points should be placed at the same level (PCI, 1989).

In the earthquake-resistant design, exterior cladding systems must be designed to accommodate the anticipated story drift. This is normally achieved by allowing the panel to rock or sway as shown in Figure 2.10. In the sway mechanism, the top or bottom edge connections are fixed, while the connections at the opposing edge are designed to accommodate the story drift by bending of rods or plates or through the use of bolted connections and slotted holes. On the other
hand, the rocking mechanism is attained through connections with vertical slots or oversized holes allowing a rocking motion. The bearing anchors are detailed not to restrain uplift in this system.

Figure 2.10: Mechanisms for Accommodating In-plane Story Drift [Wang, 1987].

For spandrel type panels, no rocking or sway will occur within the unit. Hence, the entire story drift for a given floor must be tolerated by elements (e.g., glazing) attached to the panel. Window systems installed between spandrel panels may be forced to accommodate the full story drift in half the story height or less, as illustrated in Figure 2.11.

Figure 2.11: Effect of Story Drift on Glazing between Spandrels [PCI, 1989].

For tall narrow panels (e.g., column covers, multi-floor wall units) incorporating the sway mechanism, the lateral connections designed to transfer in-plane forces should be placed as near to the center of gravity of the panel as possible to minimize overturning effects. Alternatively, these panels could be designed using the rocking mechanism.
A number of studies have been carried out on the influence of cladding on the response of building structures. Glogau [1977] suggested preventing the effect on the structural performance to separate the cladding from the main structure. Sack and Perry [1981] investigated the interaction of curtain wall and the structural frame through the response of high-rise buildings subjected to earthquake. Both full-scale experimental and analytical studies were utilized. A number of typical buildings, mostly in San Francisco, were studied. Static stiffness properties and a limited amount of low cycle fatigue data were experimentally obtained from various combinations of connector body types. Tested precast concrete panels were attached to integrative (bearing) connections at the bottom, and dissociate (tie-back) connections, both slotted angle and flexible rod types, at the top. The study presented that slotted angles are extremely effective for isolating the panel from the structure as long as the bolts can travel smoothly through the slots. However, the proper installation of the bolts to the slotted angle is required to prevent any damage to the cladding panel and also structural elements. Moreover, the dynamic response of the structures must be predicted accurately to ensure that the slotted connection can accommodate the interstory drift occurred in the actual event. This phenomenon indicates that the slotted angle is not as simple and reliable answer to the connection design problem as the flexible bar-insert system. Contradictory, there might be situations when the desired connection flexibility is more than can be obtained from a bar-insert connection, due to geometric constraints or excessive normal loads due to wind. In these cases, the use of slotted connections may be the better alternatives. The normal loads on the panels can be easily sustained with structural angles as indicated in the experimental results of the study. The interstory drift can be accommodated by sizing the slots such that frame movement is less than half of the slot length. The interstory drift may be alleviated by the use of horizontal slots while vertical displacement due to the drift may be accommodated by vertically oriented slots. According to Arnold et al. [1987], the slotted angle connection does not provide a mechanism for energy dissipation while the flexible bar-insert connection does.

Wang [1987] tested a number of precast concrete cladding elements on a six story steel frame structure in Tsukuba, Japan. Configurations included story height panels, column covers and corner conditions. Static loading tests were conducted up to a 1/40 story drift ratio. Tests were performed on two types of bearing connections which are tube and angle sections, and two types of ductile connections, which are long and short rods in slotted holes. The observations showed that the levels of performance of most elements were as generally predicted, however, the failure of some components sometimes occurred unexpectedly, which could be from the erection.
The study emphasized that although the simplicity of static testing clarified the cause and effect of observed results the drawback of the method was presented. These experimental results must be conservatively interpreted due to the limitations of the static test. Consequently, the worst potential effects of earthquake with comparable displacements were not revealed. On the other hand, in addition to the gravity and lateral loads precast concrete panels experience thermal movements, long term shrinkage due to curing of concrete, and movement of supporting structure from applied loads and thermal effects. Connection configurations that will restrain these movements should be avoided. Bearing supports are provided with Teflon-coated plates or neoprene pads to allow thermal expansion without much friction, while tie-backs are detailed to resist lateral force in the direction normal to the panel. Slotted inserts in the panel and long tie-backs allow relatively free movement of the panel vertically and longitudinally. Also, determinate force resisting systems are preferred. If indeterminate support configuration cannot be avoided, such as multiple out-of-plane lateral supports for long spandrel panels, then the design of the connections and panel reinforcement should include consideration of forces due to thermal bowing, length changes, shrinkage, and forces induced by primary structure deformations.

2.2.3 Research On Cladding Panel Resisting Lateral Load

Although cladding is regarded as nonstructural, the effect of cladding on the behavior of structures is significant. According to Arnold [1989], there are four levels of potential interaction between the structure and the cladding system, which are the followings:

- Detachment: The assumption is that the cladding is completely detached from the structure and will not strengthen or stiffen the building. This is often based upon the use of a push-pull type connection and while complete detachment is probably impossible, the cladding does behave rather independent from the structure;
- Accidental Participation: Although expected to be detached, the cladding actually plays a significant role in strengthening or stiffening the structure;
- Controlled Stiffening or Damping: By design, the cladding is expected to stiffen or dampen the motion of the structure. This is often engineered using special connections that posses damping characteristics;
- Full Structural Participation: The cladding is expected to fully participate in the load carrying capability of the structure.
These varying design and behavior aspects have been investigated by a number of researchers.

2.2.3.1 Analytical Studies

The study performed by Miller [1972] showed that measured deflections of tall buildings with cladding enclosure tend to be smaller than computed deflection. Kulka et al. [1975] suggested an idea of integration of exterior wall panels into the load resisting walls to carry both vertical and horizontal loads such as those due to earthquake and wind. It is claimed that to disregard exterior concrete wall panels as load carrying elements, particularly for horizontal loads, was impossible. The action of exterior panels should be included from the beginning of design resulting in great benefit to the overall structure behavior. The research by Sherwood [1975] and Gram [1976] have also shown that cladding may contribute 30% or more to the lateral stiffness of the primary structure. Their literature review also referenced research by Dubas [1972] and Oppenheim [1973] as reaching the same conclusion.

Henry and Roll [1986] studied the effect of cladding on reinforced concrete building for the case of panels connected to the columns. The effects of shear deformation were included in the mathematical model of cladding since the cladding functioned as a shear diaphragm subjected to lateral loads. The stiffness matrices for the connectors also included the finite joint effects resulting from the eccentricity of the connector with respect to the column centerline. A nine story, three bay building was selected for the study. The model was a cast-in-place, moment-resisting, concrete structural frame enclosed with precast concrete panels. The typical bay model is illustrated in Figure 2.12. Both linear elastic, static and dynamics analyses were performed. Parameters included in the analyses were the ratios of bay width and panel height to story height and also panel material (i.e., light weight and normal weight concrete). It had been found that the redistribution of the member forces occurred. There was a drastic reduction in the magnitude of the shear forces and bending moments in the floor beams, in some cases, even a change in the direction for the bending moments. Their studies showed that it may not be conservative to ignore the lateral stiffening effect of cladding and proper design should be made to accommodate large forces generated at the connections.
Figure 2.12: Typical Bay Model [Henry and Roll, 1986].

Goodno et al. [1983] used a combination of analytical and experimental methods to study the influence of heavy cladding on building frequencies and linear seismic response of high-rise buildings. Analytical models were used to study the influence of cladding on the dynamic properties of the model, which were altered by 15-30% for translational modes and by up to 65% for torsional modes. Finite element studies showed that interstory shear stiffness is heavily dependent on cladding connection details and panel support conditions while experimental studies found that cladding measurably stiffened the structure and shortened the natural period. It can be concluded that the study confirmed the literature research suggesting that exterior façade is a participating structural element, in spite of design assumptions to the contrary. Moreover, the research has shown that it may not always be conservative to neglect the additional stiffening contribution of heavyweight cladding connection systems, because dynamic characteristics of the overall structure model can be altered to such a degree by the added stiffness that sensitivity of the overall structure to certain earthquake loadings may be increased substantially. This conclusion is supported by the studies from McCue et al. [1978] which showed that the stiffening effect of the cladding could result in a shift of vibration frequencies of the building toward a more critical earthquake ground motion frequency range, resulting in higher seismic response. Moreover, it has also been found that connection forces and interstory shear stiffness values studied for a localized panel response model were affected significantly by the presence of oversized bolt holes, slots in connection angles, and initial friction in cladding connection attachments. However, the load bearing bottom connections were observed to exceed their ultimate vertical shear capacity at relatively low interstory displacement levels in all cases except when both top panel connections were slotted horizontally. PCI [1989] recommended procedures
for attaching precast concrete cladding panels to exterior building frame members for the purpose of isolating the brittle panels from potentially damaging interstory drift motions. This was found to be less than fully effective in accomplishing this objective [Arnold et al., 1987].

Palsson et al. [1984] studied the influence of precast concrete panels on lateral and torsional stiffness of a 24-story building. The cladding system included two precast panels per bay, each supported by four clip angles. Connections attached to wedge inserts embedded in the panel. The clip angles were welded to spandrels in the lightweight exterior frames which supported the cladding. Details of the cladding system are illustrated in Figure 2.13. The effect of cladding on dynamic properties and linear seismic response was explored by varying panel stiffness. Cladding stiffness was added to the bare frame model until analytical frequency values matched vibration test results. Then, using the cladding stiffness values obtained, an accidental eccentricity between centers of mass and rigidity at each floor level was imposed and linear seismic response was computed. It had been found that torsional response effects were increased substantially.

![Figure 2.13: Typical Exterior Bay of Cladding System [Palsson et al., 1984].](image)

Studies involving modeling technique for cladding system had been reported. Gjelsvik [1974] performed elasto-plastic analyses to determine the interaction between precast panels and structural frame. The study focused on the floor-to-floor type panels which were bolted on to a steel frame using connecting bolts embedded in the panels. Some of typical force displacement relationships were experimentally obtained for various length over bolt diameter ratios. The tests showed that the longer the bolt the closer its behavior is to the assumed idealized elastic perfectly plastic behavior. The collapse mechanism appeared to vary depending on the strength of the
bolted connections. Will et al. [1979] and Leboeuf [1981] used finite elements to model a precast concrete panel and its clip-angle connections for the same prototype structures as used on the later study by Palsson et al. [1984]. Goodno et al. [1988] developed a localized response model for the same system (Figure 2.14a). The panels were assumed to be flat and rigid, and elastic spring elements were employed at connection points to represent the clip angles and the wedge inserts. To further refine the localized response model, a super element model (Figure 2.14b) was assembled to represent one half of a typical bay of cladding and supporting frame.

![a) Localized Response Model](image1)

![b) Super Element Model](image2)

Figure 2.14: Cladding Models [Goodno et al. 1988].

To represent the structural aspects of cladding panels, Henry et al. [1989] developed a box frame mathematical model consisting of beam elements to incorporate cladding in structural analysis. The box frame model was composed of four box-like frames. The four identical panel-boxes were attached at adjacent corners to represent a single precast concrete cladding panel. Each panel-box was composed of four beam elements that were rigidly connected. The panel box horizontal beams were used primarily to model the panel flexural characteristics and the vertical beams model the panel shear characteristics. The resulting forces obtained from the box frame model analysis were compared to the results from finite element analysis. Figure 2.15 illustrates the models used in box frame and finite element analyses.
El-Gazairly and Goodno [1989;1990] studied the effect of cladding on the fundamental frequencies, mode shapes, and seismic response of a twelve story reinforced concrete frame structure damaged in the 1985 Mexico earthquake. The structure was analyzed as a tier building model where the lateral stiffness of the frames, shear walls, and cladding panels were generated and combined for the completed model. Finite elements were used to model the column cladding panels with cracks at the location of the plate inserts in the panel during earthquake. Linear spring with different stiffness were employed to model the panel connections. It had been found that cladding stiffness resulted in an increase of 30-49% in the lower frequencies, and maximum reductions of 93% and 94% in translational and rotational displacements. Forces generated at cladding column location appeared to exceed the estimated cladding connection capacity. Hence, the assumption for the connection linearity might not be applicable. In subsequent analyses, El-Gazairly et al. [1992] and Goodno et al. [1992] developed GT-IDARC program to model the three dimensional nonlinear behavior of reinforced concrete frame including cladding system and masonry infill walls.

For a better visualization of the cladding and structure interaction, Gaiotti and Smith [1989; 1992] proposed an analogous spring model to illustrate the stiffness contribution to the building structure. The model (Figure 2.16) included the springs representing the flexibilities of the three major components which are the panel with its connections ($f_{p}$, $f_{hc}$, $f_{vc}$), the columns ($f_{c}$), and the beam comprising three components ($f_{b1}$, $f_{b2}$, $f_{b3}$). In addition, single story, single bay
frame with and without cladding panels were modeled in the way to represent the actual behavior of a typical single story, single bay segment of a multistory, multi-bay frame having similarly clad stories and bays all around. The panel-to-frame connections were represented in the model by short vertical and horizontal links, either singly or in combination, with axial stiffnesses equal to calculated vertical and horizontal stiffnesses of the connections. Figure 2.17 illustrates the mathematical model of panel-clad frame. The deformations of the interacting frame and panels from each analysis were obtained. The study had also presented the modified racking stiffnesses of the basic complete cladding frame system resulting from varying the stiffnesses of the various components. Since the stiffness of cladding frame system is quite sensitive to the connection stiffness, the authors suggested that it is very important to know connection stiffness fairly accurately to estimate the stiffness of cladding frame system precisely. Moreover, the study had shown that the widths of the columns at the frame’s joints have a significant effect on the racking stiffness of the cladding structure, and they must be accounted for in the analysis.

Figure 2.16: Analogous Spring Models [Smith and Gaiotti, 1989].
2.2.3.2 Experimental Studies

There have been a number of research projects studies reported on testing of precast concrete cladding systems. To investigate the effect of cladding panels on the dynamic response of the structure, Uchida et al. [1973] have conducted both free and forced vibration tests on a two story, two bay steel frame with precast concrete panels. The results showed that cladding helped increase lateral stiffness and damping of the system. Meyyappa et al. [1981] measured the ambient response of the same 24 story steel frame office building studied by Palsson et al. [1984] to determine the lightweight cladding panels on frequencies and damping of different modes. Data was collected at different stages of construction. It was found that the frequencies of the second and third mode increased while the fundamental frequency was not affected during
construction. The study also showed that cladding had an increasing effect on damping especially for torsional modes. Subsequently the connection tests conducted by Goodno et al. [1988] and Meyyappa et al. [1988] showed that the inserts (failed in brittle mode with catastrophic fracture of concrete in pull out tests). To provide more ductility to the anchor, the authors recommended the use of longer reinforcement for the inserts and the tying of the inserts to the panel flexural steel.

Sack et al. [1989] conducted full-scale tests of cladding connection assemblies mounted in reinforced concrete blocks. The load displacement characteristics and energy dissipation capabilities of the various connection details were obtained from experiments. These mechanical properties were compared to the values predicted by classical structural mechanics methods and finite element analysis. Subsequently, a one story, one bay steel frame with two precast concrete panels was tested (Figure 2.18). Each panel consisted of two flexible connections (tie-rod) at the top and two bearing connections (clip angles) at the bottom. The top connectors were found to be highly stressed in horizontal bending and the rods were susceptible to low cycle fatigue when subjected to repeated seismic loading. Also, the flexibilities and natural frequencies of the cladding panels were experimentally obtained and were compared to the values obtained from finite element analysis which incorporated mechanical properties from experiments.

Figure 2.18: Test Configuration [Sack et al., 1989].
Rihal [1989] tested cyclic in-plane racking tests of precast concrete cladding panels with bearing connections at the bottom and threaded-rod connections at the top. The load capacity of the threaded-rod specimen was found to decrease with increasing length. Moreover, the results also showed that failure occurred in the threaded-rods at loading end of top lateral connection in all cyclic tests, which implied that the in-plane resistance of the panels was controlled by the binding resistance of the threaded-rod connections.

2.3 ENERGY DISSIPATING SYSTEMS

2.3.1 Background

Traditional seismic design approach or ductility-based approach has been performed by relying on the application of force reduction factors (or behavior factors) to approximately account for the inelasticity, the energy dissipation capacity of the structure as well as the deformation capacity of the structure. As illustrated in Figure 2.19, this traditional approach implies that some damage may occur, possibly to the level that the structure is no longer repairable. Therefore, ductility-based design aims at ensuring that the seismic ductility demand does not exceed the ductility supply.

Alternatively, energy-based design methods have been proposed. In general, these methods are based on the assumption that the energy demand during an earthquake can be predicted and that the energy supply of a structure can also be assessed. Following this approach, a satisfactory design implies that the energy supply should be larger than the energy demand. This innovative approach is also known as protective approach [Oliveto, 2002] or structural control [Chaidez, 2003]. The protective structural systems studied by the structural control are classified into three major groups, which are passive, active and semi-active, and hybrid controls.

The passive control approach consists of incorporating passive devices to the structure to control the structure motion and to modify its dynamic parameters, which are basically damping and stiffness, to reduce the structural response. The passive response control system can be broadly divided into three following subgroups according to the approaches employed to manage the input earthquake energy.
Seismic isolation system:
The system partially reflects and partially absorbs some of the earthquake input energy before this energy can be transmitted to the structure. The net effect is a reduction of energy dissipation demand on the structural system. As shown in Figure 2.20, mostly isolators are placed at the foundation of a structure and used to increase the horizontal flexibility or less frequently, to increase the rocking stability. Subsequently, a new mode of vibration is added to the superstructure, causing the lengthening of its fundamental period and keeping this period apart from the main period contents of the input. These schemes are suitable for a large class of structures that are short to medium height, and whose dominant modes are within a certain frequency range. The examples of seismic isolation system are elastomeric bearings, lead rubber bearings, and sliding friction pendulum. It should be noted that the seismic isolation is effective when the vibration is transmitted through the ground. Therefore, the structure subjected to the vibration induced by wind is not recommended using the isolation system.

Passive Energy dissipation system
The basic function of passive energy dissipation devices when incorporated into a structure is to absorb or consume a portion of the input energy, thereby reducing energy dissipation demand on primary structural members and minimizing possible structural damage. They are generally located between the main structure and the bracing system.

Figure 2.19: Conventional Design of Seismic Resistant Building Structure [Moreschi, 2000].
as illustrated in Figure 2.21. In general, the aim of including energy dissipation devices in a seismic structure is to concentrate the dissipation of hysteretic energy in specially designed and detailed regions of the structure and to avoid inelastic behavior in members of the LFRS, except perhaps under a catastrophic event. [Aiken et al., 1992] However, in the case of the redesign of an existing structure the onset of inelastic behavior in members of the LFRS may be unavoidable. Hence, for seismic redesign the incorporation of energy dissipation devices aim at minimizing the inelastic behavior of the existing LFRS. Unlike seismic isolation, however, these passive energy dissipation devices can be effective against wind induced motions as well as those due to earthquakes [Soong and Dargush, 1997]. The examples of passive energy dissipation system are metallic dampers, friction dampers, viscoelastic dampers, and viscous fluid dampers.

- **Mass dampers**

The concept of mass dampers is the use of the relative motion between massive elements and the structure so the big inertia forces involved partially cancel the external forces on the structure. Mass dampers are usually more effective when installed on the top of the structure to control the first mode (Figure 2.22). However, more than one mass damper can be installed on different levels to control several modes of vibration [Chaidez, 2003]. Different types of mass dampers are illustrated in Figure 2.23.

According to Chaidez [2003], the comparison of these three passive response control systems is shown in Table 2.1.

<table>
<thead>
<tr>
<th>Item</th>
<th>Desirable Features</th>
<th>Isolation system</th>
<th>Energy dissipators</th>
<th>Mass dampers</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Low initial cost</td>
<td></td>
<td></td>
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<tr>
<td>2</td>
<td>Effective to resist wind forces</td>
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<td>Low cost of replacement</td>
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<td>7</td>
<td>Effective against severe earthquakes</td>
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Table 2.1: Comparison Between the Passive Control Systems [Chaidez, 2003].
Figure 2.20: Building Equipped with Base Isolation System [Chaidez, 2003].

Figure 2.21: Building Incorporated With Energy Dissipation Devices [Chaidez, 2003].

Figure 2.22: Mass Damper Installed on the Top of the Building [Moreschi, 2000].
Active structural control is an area of structural protection in which the motion of a structure is controlled or modified by means of the action of a control system through some external energy sources. This energy is supplied to actuators in the system to be able to push the structure to counteract the input forces, while sensors are installed to monitor the status of the structure in real time. The received information is processed by a controller (i.e., computer), then sent back to actuators to properly minimize the response. The basic scheme of the active structural control and the analogy of the system with human body are shown in Figure 2.24 and Figure 2.25, respectively. The examples of active structural control system are active bracing systems, active mass dampers (Figure 2.26), variable stiffness or damping systems, and smart materials.

On the other hand, semi-active systems require only nominal amounts of energy to adjust their mechanical properties and, unlike fully active systems, they cannot add energy to the structure. That means that the semi-active systems lack of the capacity to push the structure,
rather only capable to stop its motion. These systems are simpler, more robust, more reliable and more economical than the active ones and its efficiency is only slightly lower [Chaidez, 2003].

Hybrid systems consists of a series or parallel combination of an active (or semi-active) system with a passive one. An example of an actuator plus a base isolation system is shown in Figure 2.27a. Another example for hybrid system is a two-axis hybrid-type mass damper (Figure 2.27b). However, only passive energy dissipation systems are the focus of the present study.

Figure 2.24: Schematic Details of Active Structural Control System
[Barbat and Canet, 1994; Soong, 1990].

Figure 2.25: Analogy of an Active Control System With a Human Body (Servio Model).
2.3.2 Design Philosophy

For years, serious efforts have been undertaken to develop the concept of energy dissipation, or supplemental damping, into a workable technology. After the pioneering work of Housner [Housner, 1956], many authors have investigated the possibility of developing design procedures based on an energy balance approach. Although significant insight has been gained, a satisfactory design procedure based purely on energy concepts has not been achieved. This is mainly because energy dissipation capacity cannot be used as univocal measure of damage [Serino, 1995]. Nevertheless, for the study of the seismic response of structural systems it is
becoming customary to analyze the balance of the energy input and to use energy considerations to improve seismic design. This is particularly important in the case of structural systems incorporating energy dissipation devices, where a comparison of the energy balance of the structure with and without devices provides a way of evaluating the efficiency of both systems in terms of the energy dissipated by the primary members of the LFRS.

Popov et al. [Popov et al., 1993] proposed an innovative idea called “source-sink analogies” to understand the way the seismic input energy is dissipated, as illustrated in Figure 2.27. These models explain the different energy paths encountered in conventional structures and those incorporating a base isolation system. For the conventional system, the seismic input energy, \( E_i \), beyond the sum of elastic strain energy, \( E_s \), kinetic energy, \( E_k \), and viscous damping energy, \( E_\zeta \), is entirely dissipated as hysteretic energy, \( E_h \). In contrast, for a base isolation system, the \( E_k \) of the first rigid body mode is damped out by the isolator. The analogy in Figure 2.28 shows how the seismic damage energy is shared by the energy dissipated by the structure \( E_h \) and that dissipated by the devices \( E_{hd} \).

![Source-sink Analogies](image)

**Figure 2.28:** Source-sink Analogies for Seismic Input Energy Dissipation

[Martinez-Rueda, 2002].
2.3.3 YIELDING/ METALLIC DAMPER

2.3.3.1 Introduction

One of the most effective mechanisms available for the dissipation of energy is through the inelastic deformation of metallic devices. Traditional seismic design approach depends on the inelastic action of structural members to provide the energy dissipation. However, the idea of using separate metallic hysteretic dampers within a structure to absorb the energy began with the conceptual and experimental work by Kelly et al. [1972] and Skinner et al. [1975]. Because of the inherent minimum maintenance requirements, the development of simple devices involving either steel plasticity or lead plasticity has been favored. In particular, research efforts have been directed towards the development of hysteretic devices of solid cross section with stable behavior at high levels of plastic strain. Because of its composition, mild steel was found to be the most suitable material, preferably heat-treated for five hours at 620 degree Celsius after fabrication. In design, welding is kept well away from highly strained zones; otherwise rapid failure can take place [Martinez-Rueda, 2002]. Figure 2.29 shows several yielding dampers included torsional beam, flexural beam, and U-strip dampers.

![Figure 2.29: Metallic Damper Geometry (Skinner et al., 1975).](image)

For the U-shaped strips (Figure 2.29c), the strip is initially in a semicircular form with two equal straight sections on either side. When one side is moved relative to the other, the
semicircular portion rolls along the strip and work is done at the two points where the radius of curvature is changed from straight to the radius of the semicircle and then from this radius to straight again. Thus at any instant the energy dissipation is concentrated at two transverse surfaces, but these two surfaces move along the strip. In general, this device is comparatively flexible in the elastic range and can be operated with very large displacements in the inelastic range. Tests carried out on a U-shaped strip device under reversed cyclic loading showed that the mode of failure is characterized by a localized kinking of the strip followed rapidly by complete transverse fracture. Although the use of rollers and stainless steel led to lifetimes longer than those obtained with mild steel, it was concluded that the expense of the rollers and stainless steel is the major disadvantage in comparison to mild steel. Properly designed mild steel U-shaped strips can reliably produce lifetimes in excess of 100 cycles.

Aguirre and Sanchez [1992] adapted the U-shaped yielding device studied by Kelly et al. [1972] to develop a yield device for tension-compression steel bracing. The device, referred to as oval element, is fabricated from mild steel strips, schematically shown in Figure 2.30. Several cyclic tests to determine optimum device dimensions as well as fatigue life have been conducted in the device. Test results showed that it is possible to select the dimensions of the device to guarantee 100 cycles for a desired maximum displacement. The analysis of the oval elements is rather complex due to the complex interaction between yielding and rolling mechanisms experienced by the steel strips.

![Figure 2.30: Application of Oval Element as Yield Device for Bracing](Aguirre and Sanchez, 1992).

Kelly et al. [1972] also studied a base-isolation system for which energy dissipation capacity is achieved by the yielding of bent mild steel bars, as shown in Figure 2.31. This yielding device was inspired by field observations of heavily damaged columns due to severe earthquake loading. The observed ability of ordinary reinforcing bars in continuing to resist earthquake loading after concrete has spalled away suggested that plain mild steel round bars could be used to provide damping in a base isolation system, provided a bend is introduced into
the bar, to allow for extension without premature tensile failure, during excursions in the horizontal plane. In general, a combination of inelastic bending and torsion occurs in bars with this shape. Test results of bars of various diameters and for varying directions of imposed horizontal displacement indicate that the energy dissipation of this device is primarily associated with bending at the fixities region. Martinez-Rueda [2002] suggested that the positive attribute of the bent bars is the occurrence of a progressive locking-up mechanism as horizontal deflection increases and the bars straighten. In addition, in the event of uplift under a catastrophic event, energy will still be dissipated as the bars straighten vertically, particularly at the corners of buildings.

Another steel-plate energy dissipation devices used in the intersection of chevron braces are the Added DAMping and Stiffness (ADAS) elements. These elements may be considered as an extended application of the tapered yielding plates that have been successfully applied in base-isolation systems. Figure 2.32 shows a moment-resisting frame incorporating ADAS elements in the connection between chevron braces and floor system. When properly designed and implemented, ADAS elements can increase the strength, stiffness and energy dissipation capacity of moment-resisting frames. In the case of concentric-braced frames, the introduction of ADAS elements results in a substantial increase in the energy dissipation capacity per unit-story drift. Commonly, ADAS elements consist of mild steel plates of X shape or triangular. The tapered section of the devices allows a uniform distribution of plastic deformations along the height of the devices. This results in maximum curvatures and strains in the plates that will be significantly smaller than those in a rectangular plate for similar lateral displacements.

Figure 2.31: Bent Mild Steel Bars as Energy Dissipation Device in Base Isolation

[Kelly et al., 1972].
Whittaker et al. [1991] conducted a research program to assess the suitability of ADAS elements for upgrading moment-resisting frames. Several four-plate, six-plate, and seven-plate ADAS elements were tested under monotonic and reversed cyclic loading. The mechanical characteristics that most significantly affected the response of an ADAS element were found to be the elastic stiffness, yield strength and yield displacement, $\sigma_y$. The appropriate design displacement of the ADAS element, compatible with the demands imposed by the design earthquake, was found to be $3\sigma_y$. For a region of high seismic risk, it was proposed that ADAS elements should be able to sustain their mechanical characteristics for at least 15 to 20 cycles. All of the ADAS elements tested exhibited stable hysteretic behavior up to displacement amplitude of $3\sigma_y$. A seven-plate ADAS element was subjected to a large number of yielding cycles with increasing amplitude between $\sigma_y$, and $14\sigma_y$. The strength and stiffness of this element did not degrade and satisfied the maximum credible earthquake limit state requirements. It was suggested that the susceptibility of ADAS elements to failure by low-cycle fatigue is negligible unless their lateral strength is so low that they are subjected to more than 100 cycles at a displacement level exceeding $10\sigma_y$.

The seismic performance of the ADAS elements was also investigated through a series of shake table tests of a 3-story steel moment-resisting frame. The frame was tested both without braces and with ADAS elements in the intersection of chevron braces. Results indicated that the incorporation of ADAS elements improved the behavior of the bare frame. The testing program also demonstrated the three major advantages of braced frames with ADAS elements, which are

- inelastic deformations and hysteretic energy dissipation can be confined to a limited number of predetermined, easily replaceable elements,

Figure 2.32: Moment-Resisting Frame with ADAS Elements [Whittaker et al., 1991].
• for minor and moderate levels of earthquake shaking, yielding of the ADAS elements can result in a significant reduction in interstory deformations, and

• stable hysteretic behavior of the bracing system can be maintained throughout a severe earthquake.

A variation of the ADAS elements is the Triangular-plate Added Damping and Stiffness (TADAS) devices developed by Tsai et al. [1992; 1993]. As shown in Figure 2.33, TADAS elements consist of a series of triangular steel plates that connect chevron braces to beams of steel frames. In contrast with the ADAS elements where the X-shaped plates are bolted together through two ends of each plate, the triangular plates of the TADAS elements are connected to a base plate using a welded connection as indicated in Figure 2.33. Experimental results have shown that despite the closeness of light fillet welds to the region of maximum stress in the triangular plates, the TADAS elements exhibit stable hysteretic behavior under cyclically increasing load, with no signs of stiffness or strength degradation for rotation amplitudes of more than 0.29 radians (Figure 2.34). Following the appropriate cyclic behavior observed in the tests of TADAS elements, a 2-story large-scale steel frame was tested under seismic loading following a pseudo-dynamic procedure. As expected, the observed behavior was very similar to that of steel frames with ADAS elements. Also, analytical predictions using nonlinear analysis techniques agreed well with experimental results.

Tsai [1995] indicated that the most attractive features in the proposed TADAS element is that the effects of gravity load in the frame can be completely separated from the device by using the slotted holes in the connection details shown in Figure 2.33. Under large deformations of the device, the vertical displacements at the end of the triangular plate can be easily accommodated. Therefore, the plasticity within the triangular plate is generated by bending only, and the inelastic response of the proposed TADAS device is highly predictable. However, it appears that the construction procedure of the TADAS elements is far more elaborate than that required to fabricate ADAS elements. Furthermore, the dimensions of the ADAS elements may be selected in such a way that the behavior of the X-shaped plates is primarily controlled by flexure. It is also noted that if the need for replacing plates arises after a damaging event, this can be done easier in the case of ADAS elements for which the plates are fixed by means of a bolted connection.
Pocanschi et al. [1990] proposed an enhanced bracing system that combines the economy of tension-only cross bracing with the good hysteretic behavior of ADAS elements. The proposed system is shown in Figure 2.35 and consists of a closed cross-bracing system working only in tension, fixed at the bottom of the columns, and capable of moving at the top corners of the frame. A yielding device very similar to an ADAS element is connected between the frame and the cable bracing to control their relative motion. Because of the continuity of the bracing mechanism, the increase in length of a crossbar corresponds to the same length reduction of the other crossbar. So if the cable remains elastic, under cyclic loading both cross braces are permanently under tension.

The closed circuit-bracing system as proposed by Pocanschi et al. ignores the advantages of using X-shaped steel plates such as those used in ADAS elements to optimize energy dissipation and deformation capacity of the plates. Furthermore, as discussed earlier, to achieve stable hysteretic behavior, welded connections should be detailed in such a way that welding occurs far from the regions with anticipated maximum ductility demands.
Clark et al. [1999] suggested the design studies and large-scale tests of tension/compression yielding braces, also called “unbonded braces,” in support of their first application in the U.S. The design procedure is based on the equivalent static force method currently prescribed for eccentric braced frames in the Uniform Building Code. The schematic of the unbonded braces is illustrated in Figure 2.36. The core steel in these braces provided stable energy dissipation by yielding under reversed axial loading, while the surrounding concrete-filled steel tube resisted compression buckling. A slip surface or unbonding layer separated the steel core from the surrounding tube. Moreover, nonlinear pushover analyses of several different braced frame designs corresponding to an eccentric braced frame system, a concentric braced frame system, and an unbonded braced frame system were performed to support the application of the system.

The yielding frame is also one of the techniques used in energy dissipation systems. According to Tyler [1985], the yielding device shown in Figure 2.37a) was first introduced in the late 1970s by David Smith and Robert Henry of Auckland, New Zealand. The device, referred to as yielding frame, achieves energy absorption through yielding in a rectangular frame made of round bars. The yielding frame is geometrically similar to the framework in which it is incorporated and hence the two parts of each diagonal are collinear. Under lateral cyclic loading the device distorts to a parallelogram that promotes a stable cyclic behavior without the progressive slack that normally develops in cross bracing under severe seismic loading. A device of this type will only perform satisfactorily if the braces are loaded by dynamic horizontal forces oscillating about a zero load. If there is a permanent dead load component in the horizontal forces then the effect of an earthquake will be to cause a steady movement in one direction with the consequent locking up of the device in that direction. Cyclic tests of a cross-bracing system

Figure 2.35: Closed Cross-Bracing with Yield Device [Pocanschi et al., 1990].
incorporating the yielding device showed that many hundreds of cycles of loading could be completed without the development of slack in the bracing rods. A locking-up effect occurs for very large deformations but these will normally be outside the range contemplated in the design. As an example, consider a bracing system tested by Tyler that performed well for up to 200 cycles at 1 Hz for lateral displacements equivalent to a drift of 3.5% with no development of slack.

Figure 2.36: Schematic of Buckling-Resistant Unbonded Braces [Clark et al., 1999].

Figure 2.37: Yielding Frames as Devices for Cross-Bracing.
One of the major limitations of the yield devices tested by Tyler is that the transverse section of the yielding frame is constant, thus the device favors high concentration of ductility demands around the corners of the device only. Hence, a small volume of the device contributes to energy dissipation. Furthermore, the study of Tyler is also limited in the sense that no tests were performed under earthquake loading.

Ciampi and Samuelli-Ferretti [1990] proposed several yielding devices for cross bracing. As shown in Figure 2.37b) and Figure 2.38a), the devices consist of a yielding inner frame fabricated from steel plates. It can be shown that when forces are applied along the diagonals, the bending moment diagram in the device varies linearly, with maximum values at nodes and zero values at the center. Hence, to promote a condition of uniform plastification in bending, the member sections of the yielding frame should vary accordingly. This can be accomplished by varying the width or the depth of the section, in a linear or parabolic fashion, respectively, as in the case of devices shown in Figure 2.37b) and Figure 2.38a), respectively. To ensure that plastification occurs primarily due to bending and also to resist shear and axial force, a transition region of constant section is used in the central part of the device members. The device shown in Figure 2.38a) is lighter and requires a simple fabrication procedure. In fact, this device can be constructed by cutting a single plate, avoiding a welding process. The design of the devices is done assuming a maximum conventional steel strain of 3%. The devices proposed by Ciampi and Samuelli-Ferretti have been tested under cyclic loading. The applied displacement history consisted of uniform cyclic displacements corresponding to a maximum steel strain equal to 3%. Experimental results for the device with variable width (Figure 2.37b) showed that the effect of geometric nonlinearities produce an apparent strain hardening in the response of the device. After 30 cycles of stable behavior, the response progressively degraded until the device broke down in the 51st cycle. The failure was attributed to excessive closeness of the welded connections to zones of maximum plastic deformations and to local brittleness by thermal shock due to welding. Tests on a device with variable depth (Figure 2.38a) showed undesirable cyclic behavior associated with the accumulation of axial plastic deformation due to tension. These deformations resulted in the expansion of the size of the device. In real applications, the expansion of the device may compromise the behavior of the entire bracing system. It was suggested that the performance of the device could be easily improved using pinned steel plates parallel to the sides of the device as shown in Figure 2.38b). Such complementary plates work under axial force only and are designed in such a way that they remain elastic, assuring that plastic deformations in the device occur primarily in bending. Cyclic tests performed on the modified device have shown
that the elongation of one diagonal returns to zero as the other one is put back to zero tensile load. This results in stable hysteretic behavior, proving the effectiveness of the modification of the device.

Nonlinear numerical analyses of steel-braced frames incorporating the devices mentioned above have shown good agreement with experimental results. To predict the actual behavior of the devices, the design of a real structure incorporating the modified yield device has been conducted. The structure is a steel braced frame to be used in one of the buildings of the Laboratories of the National Research Council in Frascati, Italy. A series of nonlinear time-history analyses using an artificial record compatible with a design spectrum were conducted for different PGA levels. A comparison between conventional bracing and the proposed dissipative bracing showed the advantages of the latter ones, in terms of reduction of top displacements, interstory drifts, base shears, and number of excursions of large amplitude.

According to Martinez-Rueda [2002], the main contribution of Ciampi and Samuelli-Ferretti for the design of dissipative cross bracing using yielding devices is the attention they paid to the detailing of the devices. The device of Ciampi and Samuelli-Ferretti is a better option when compared with that of Tyler [1985], because it optimizes the sources of energy dissipation and has redundancy. Even with the eventual fracture of the yielding frame, the complementary pinned plates guarantee that a bracing system would still be available under the action of a catastrophic event.
To effectively include these devices in the design of an actual structure, one must be able to characterize their expected nonlinear force-displacement behavior under arbitrary cyclic loads. Ozdemir [1976] was the first to consider this modeling problem. He utilized analogies with existing elastoplastic and viscoplastic constitutive theories to develop appropriate forms for the force-displacement relationships. Additionally, Ozdemir detailed efficient numerical algorithms for computing the response of structures with metallic dampers subjected to general time-dependent loading, such as that caused by an earthquake. Shortly thereafter, Bhatti et al. [1978] employed that methodology to study the response of structures that utilized torsion bar dampers in conjunction with a seismic base isolation system.

### 2.3.3.2 Basic Principles

A significant dissipative mechanism for the yielding type damper is mainly from the inelastic deformation of metal. Usually that metal is mild steel, although sometimes lead or metal alloys are employed. Typical stress-strain curves are shown in Figure 2.39. Normally the mechanical properties of structural steel are consistent and stable at room temperature. Although a significant portion of the dissipated energy is converted into heat, it is not expected that the temperature increase from the heat will significantly change the mechanical properties of the device [Dargush and Soong, 1997].

![Nominal Stress – Conventional Strain Diagrams](Dargush and Soong, 1997)

Figure 2.39: Nominal Stress – Conventional Strain Diagrams [Dargush and Soong, 1997].
2.3.3.3 Mathematical Modeling

Numerous mathematical models have been developed to idealize the stress-strain curves. The first two simple elasto-plastic stress-strain curves and the rheological models are shown in Figure 2.40. An elastic, perfectly plastic stress-strain relationship is flat beyond yielding and the strain is the sum of elastic and plastic parts, which is

\[ \varepsilon = \varepsilon_e + \varepsilon_p = \frac{\sigma}{E} + \varepsilon_p \left( \varepsilon > \frac{\sigma_0}{E} \right) \]  
\hspace{2cm} (2.1)

In the rheological model, the elastic strain, \( \varepsilon_e \), is analogous to the deflection of the linear spring of stiffness \( E \), and the plastic strain, \( \varepsilon_p \), is analogous to the movement of the frictional slider.

For elastic, linear hardening behavior, the relationship requires an additional constant, \( \delta \), the reduction factor for the slope following yielding, the slope before yielding being the elastic modulus \( \delta E \), and that after yielding being \( \delta E \). The relationship can be written

\[ \varepsilon = \frac{\sigma_0}{E} + \frac{(\sigma - \sigma_0)}{\delta E} \quad (\sigma \geq \sigma_0) \]  
\hspace{2cm} (2.2)

where, in rheological model,

\[ E = E_1, \quad \delta E = \frac{E_1E_2}{E_1 + E_2} \]  
\hspace{2cm} (2.3)

The slope \( \delta E \) corresponds to the stiffness of the two springs \( E_1 \) and \( E_2 \) in series.

![Figure 2.40: Stress-Strain Curves and Rheological Models [Dowling, 1999].](image)

a) elastic, perfectly plastic behavior       b) elastic, linear hardening behavior
In Ramberg-Osgood model (Figure 2.41), elastic and plastic strains, $\varepsilon_e$ and $\varepsilon_p$, are considered separately and summed.

\[ \varepsilon = \frac{\sigma}{E} + \left( \frac{\sigma}{H} \right)^{\frac{1}{n}} \]  

(2.4)

where $n$ is called a strain hardening exponent, which can be calculated from the slope on the log-log plot of stress versus plastic strain if the logarithmic decades in the two directions are of equal length. The constant H is the value of $\sigma$ at $\varepsilon_p = 1$.

Figure 2.41: Ramberg-Osgood Model [Dowling, 1999].

Under monotonic loading, the deformation behavior follows the relationship explained above. If the direction of straining is reversed into the compression range after yielding, yielding will again occur prior to the stress reaching the yield strength for monotonic compression, dependent upon the prior amount of strain hardening (Figure 2.42). This early yielding behavior is called the Bauschinger effect. The rheological model used which is consistent with the behavior is kinematic hardening, the alternative choice of isotropic hardening not being employed as it is a poor model for real materials [Dowling, 1999].

However, in plasticity concept, all time dependency in the formulation of the constitutive model was ignored. Plastic flow was assumed to occur instantaneously compared to the time variation of the applied load. This assumption is applicable for steel deforming under moderate strain rates, but not for lead under similar conditions nor for steel under very strain rates or at high temperature. Hence, viscoplasticity model is introduced to the analysis, in which creep and relaxation are considered. Concepts and details of viscoplastic model can be found from the references [Soong and Dargush, 1997; Dowling, 1999].
Constitutive models explained earlier do not normally include failure criteria. Typically metals subjected to cyclic loads with the inelastic range of deformation fail due to low-cycle fatigue [Dargush and Soong, 1997]. Since metallic damper is subjected to variable strain amplitude cycling, Palmgren-Minor cumulative damage rule, a rain flow counting method, and Continuum Damage Mechanics (CDM) are alternatives for estimating fatigue life [Dowling, 1999].

2.3.4 Friction Damper

2.3.4.1 Introduction

Friction damper, also called friction dissipator, is a device that utilizes the mechanism of friction to provide the energy dissipation. The friction damper device is mostly assembled with bracing system as shown in Figure 2.43. Several friction damper types have been proposed which aim to improve the seismic response to new and retrofitted buildings. Based primarily upon an analogy to the automotive brake, Pall et al. [Pall et al., 1980] began the development of passive frictional dampers to slow down the motion of buildings by ‘braking rather than breaking’ [Pall and Marsh, 1982]. The Limited Slip Bolted (LSB), Figure 2.44, incorporated brake lining pads between steel plates to provide a consistent force-displacement response. The static and dynamic tests were conducted to study the hysteretic behavior of the LSB device. The load-displacement

![Figure 2.42: Cyclic stress-strain response with Bauschinger Effect [Dowling, 1999].](image)

Figure 2.42: Cyclic stress-strain response with Bauschinger Effect [Dowling, 1999].
response with different sliding surface materials under monotonic loading obtained for the tests is illustrated in Figure 2.45 while Figure 2.46 shows hysteresis loops under constant amplitude displacement-controlled cyclic loading. During the tests, contact was maintained between the faying surfaces by pretensioning high strength bolts.

![Diagram of friction dissipation](image1)

(a) Frame with the dissipator located between the braces and the upper slab  
(b) Frame with the dissipator located between the brace end and the beam-column connection  
(c) Frame with the dissipator located in the middle of the cross bracing  
(d) Frame with the dissipator located in the middle of the diagonal brace  

Figure 2.43: Locations of Friction Damper [Chaidez, 2003].

![Diagram of limited slip joint](image2)

Figure 2.44: Limited Slip Bolted Joint Detail [Pall et al., 1980].
Figure 2.45: Load-Displacement Response of LSB Joints [Pall et al., 1980].

Figure 2.46: Hysteresis Loops of LSB Joints [Pall et al., 1980].
An alternative design illustrated in Figure 2.47 has also been proposed by Pall and Marsh [Pall and Marsh, 1982] for application in conjunction with cross-bracing in framed structures. It consists of rigid diagonal bars with friction hinges at intersection points connected together by means of horizontal and vertical elements and brake lining pads are utilized for the sliding surfaces. When tension in one of the braces forces the joint to slip, it activates the four links that force the joint in the other brace to slip simultaneously. Pall [Pall, 1983] also proposed several more friction devices for tension-only and tension-compression bracing systems as shown in Figure 2.48. If the braces are designed not to buckle in compression then a simple slotted friction connection can be used to slip in tension and compression. These devices are designed not to slip under normal service loads and moderate earthquakes. During a severe earthquake, the device slips at a predetermined load, before yielding occurs in the other structural elements of the frame.

![Figure 2.47: Pall Dampers in Conjunction with Cross Bracing](image1)

![Figure 2.48: Alternatives of Pall Dampers in Bracing System](image2)
A simple elastoplastic model was used to represent the behavior of X-braced friction damper. However, Filiatrault and Cherry [1987] determined that this is only valid if the device slips during every cycle, and that the slippage is always sufficient to completely straighten any buckled braces. Otherwise the determined energy dissipation is overestimated. Therefore, Filiatrault and Cherry [1987] proposed a more detailed model for Pall damper (Figure 2.49). Each member of the system is represented by elements reflecting its individual axial and bending characteristics. Consequently, structural braces are assumed to yield in tension but buckle elastically in compression, while the links are allowed to yield in both tension and compression. The brake lining pads are represented by a hysteretic model corresponding to the test results obtained by Pall et al. [1980]. Moreover, the hysteresis loops of the X-braced damper with heavy duty asbestos brake lining pads, subjected to cyclic displacement-controlled loading along one diagonal, are determined in the study (Figure 2.50).

Figure 2.49: Refined Model for X-Braced Pall Damper [Filiatrault and Cherry, 1987].

Figure 2.50: Hysteresis Loops for X-Braced Pall Damper [Filiatrault and Cherry, 1987].
Anagnostides et al. [1989; 1990] proposed a new type of friction device for tension-only cross bracing. Two variants of the proposed device are shown in Figure 2.51. In contrast with the friction device proposed by Pall [1983] in which the friction joints slip following a linear trajectory, the proposed device presents a simpler design based on the use of rotational friction joints. These joints consist of frictional washers bolted to steel plates and distribution washers by high-strength bolts. The strength of the device depends upon the material and dimensions of the washers and the pressure applied by the bolts. Several friction materials were tested under cyclic friction using a rotational frictional joint. Materials tested included cast iron, Nitroy 40B, stainless steel, ground flat stock, FF Ferrodo friction material, and 3501 F friction material. In terms of hysteretic behavior 3701 F Ferrodo was found to be the best from all the friction materials tested, showing predictable and constant slipping load for a number of cycles expected during earthquake excitation. However, 3701 F Ferrodo friction material experiences creep under permanent compressive stress and, hence, long-term performance of friction devices has to be carefully considered in the design. It was found that by retightening the devices after two periods of 14 days, creep was eliminated as a potential problem.

Martinez-Rueda [2002] stated that the main contribution of Anagnostides et al. for the further development of friction-damped bracing systems was the adoption of rotational friction as opposed to translational friction used by previous authors. It is expected that rotational friction devices be easier to construct and, consequently, cheaper. In addition, more consistent hysteretic friction behavior may be achieved in rotational friction devices because the geometry of the frictional sources favors the application of a more uniform clamping pressure on the frictional material.
An uniaxial friction device shown in Figure 2.52 is a Sumitomo friction damper that has found application in Japan with the aim of reducing the structural response due to soil vibrations and small or moderate earthquakes [Aiken and Kelly, 1990]. The copper alloy friction pads, with pieces of graphite inserted, slide along the inner surface of the cylindrical steel casing. The required normal force is provided through the action of the spring against the inner and outer wedges. This device was originally conceived as a shock absorber in railway rolling stock and it was later applied as a friction damper for chevron bracing system (Figure 2.53). Large-scale tests of friction-damped braced frames incorporating Sumitomo devices have been reported by Aiken et al. [Aiken et al., 1992]. Preliminary tests on individual dampers showed that the device is independent of loading frequency, amplitude, number of loading cycles, and temperature. The Sumitomo device showed regular and repeatable hysteretic behavior with no variation in slip load during earthquake motion as shown in Figure 2.54. Analytical predictions for the response of the friction-damped frame agreed well with experimental results. This confirmed the fact that the stable hysteretic behavior of the friction devices makes them particularly amenable for accurate modeling.

Figure 2.55 presents the somewhat more sophisticated Energy Dissipating Restraint (EDR) developed by Fluor Daniel, Inc. Initially, the EDR was developed as a restrictive seismic device for the support of piping systems in nuclear plants. Dissipation occurs on the interface between bronze friction wedges and the steel cylinder wall. The combination of wedges, stops, and internal spring produces a frictional force proportional to the relative displacement of the device ends. The outstanding features of the device are its self-centering capability and that the frictional force is proportional to the displacement. In fact, this self-centering behavior would tend to reduce permanent offsets if the structure were deformed inelastically. In the EDR, two types of behavior are combined, which are linear stiffness and friction. Different combinations in between are possible, leading to different hysteresis loops, as shown in Figure 2.56. Figure 2.57 shows a structure equipped with this device.

Moreover, Nims et al. [1993] has performed the scaled model test and also suggested that bracing systems incorporating energy-dissipating struts are more effective in reducing the response of structures to relatively harmonic excitations than for impulsive excitations.
Figure 2.52: Sumitomo’s Friction Damper [Aiken and Kelly, 1990].

Figure 2.53: Sumitomo’s Friction Dampers Installed in the Model [Aiken and Kelly, 1990].
Figure 2.54: Hysteresis Loops of the Sumitomo's Friction Damper [Aiken and Kelly, 1990].

Figure 2.55: The Energy Dissipating Restraint (EDR) [Nims et al., 1993].

Figure 2.56: Hysteresis Loops of the EDR [Nims et al., 1993].
Meanwhile, an improved and simpler type of friction device, Slotted Bolted Connection (SBC), intended for application in concentrically braced frames. Baktash and Marsh [1986] proposed a simple SBC for bracing system. As shown in Figure 2.58, the braces are connected to the structure by bolting through steel gusset plates with slotted holes. Brake lining pads are inserted at both sides of the plates. The pressure to control the slip force in the friction joints is provided by a system of spring plates. A 4-story large-scale steel frame with the proposed friction devices was tested under sinusoidal excitation on a shake table. Different slip forces were considered in the study. The condition of optimum slip force was found to be that when the shear force is shared equally between the columns and the braces, thus the story shear force causing the braces to slip is equal to the shear force causing the rigid frame to yield.

Figure 2.57: Frame with Equipped with EDRs in Each Floor [Nims et al., 1993].

Figure 2.58: Friction Devices for Bracing System [Baktash and Marsh, 1986].
FitzGerald et al. [1989] employ all structural steel components (Figure 2.59), while Grigorian et al. [1993] advocate inclusion of brass insert plates (Figure 2.60). In both cases, Belleville washers are used to maintain initial bolt tensions. Upon tightening of the bolts, the main plate is compressed directly between the brass insert plates. The holes in such plates and in the steel outer plates are of conventional size. When the tensile or compressive force applied to the connection exceeds the friction forces developed between the frictional surfaces, the main plate slides relatively with respect to the brass insert plates. Energy is dissipated by means of friction between the sliding surfaces. However, Martinez-Rueda [2002] pointed that past experience on the design of SBCs has shown that under large cyclic displacements the lack of Belleville washers results in a quick loss of bolt tension, which in turn triggers a quick degeneration of the device hysteretic behavior.

Tests of SBCs with brass shims under reversed cyclic loading demonstrated the repeatable and reliable hysteretic behavior of these friction devices. The rectangular shape of the hysteresis loops (Figure 2.61), coupled with the reasonably constant slip force, indicates that the assumption of elastic-perfectly plastic behavior for SBCs with brass insert plates is valid. An example of a structure equipped with this type of damper is shown in Figure 2.62.

Figure 2.59: Detail of Slotted Bolted Connection (SBC) [FitzGerald et al., 1989].
Figure 2.60: Detail of Slotted Bolted Connection (SBC) [Grigorian et al., 1993].

Figure 2.61: Hysteresis Loops of SBC [Grigorian et al., 1993].

Figure 2.62: Single Story Frame with SBC [Grigorian et al., 1993].
More recently, Filiatrault et al. [2000] have proposed a new version of a dissipative strut referred by the authors as a friction-based ring spring damper, Figure 2.63. This device consists of a friction spring created by the assembly of cylindrical wedges or rings connected in series. As the assembly is loaded in compression the axial displacement is accompanied by sliding friction between the conical contact surfaces of the rings. The assembly is retained at both ends by cylindrical cups. Under compression, the left cup compresses the rings while the tie-bar-head slides in the slot of the right cup. Under tensile load reversal, the right cup is pulled by the tie-bar-head and the friction springs are subjected to compression again. This complex interaction between the elements of the device produces a stable symmetrical flag-shaped hysteretic behavior similar to that shown in Figure 2.64.

![Friction-based Ring Spring Damper](image)

**Figure 2.63:** Friction-based Ring Spring Damper [Filiatrault et al., 2000].

![Hysteresis Loops](image)

**Figure 2.64:** Hysteresis Loops for Different Adjustments of the Dissipating Strut [Aiken et al., 1992].

In Denmark, Damptech company provided damper devices used in structures including buildings, bridges, elevated highways, towers, offshore structures, industrial buildings, houses and prefabricated panels (metal or wood). The device is illustrated in Figure 2.65 and the position of this device installed in the building is also presented in Figure 2.66.
The use of friction device is also well developed in Japan. The Takenaka Company in Japan called such a device as “Super sliding Bearing, SSB.” SSB consists of upper and lower steel plates, natural rubber pad, sliding pad (i.e., Telfon) and coated stainless steel plate. The configuration of this device is illustrated in Figure 2.67.
2.3.4.2 Basic Principles

To maximize the energy dissipation, the contacting surfaces are generally intended to remain dry during operation without introducing a hydrodynamic lubricating layer on the interface. Hence, the following principles will refer to dry friction (Coulomb’s theory) including surface conditions and environmental effects.

Referred to Figure 2.68, the static frictional force, \( F \), can be defined as the minimum force required to maintain equilibrium or to prevent relative motion between bodies while the ratio of the absolute value of the maximum static frictional force, \( F_{\text{max}} \), to magnitude of the normal force between the two surfaces, \( N \), is referred to as the coefficient of static friction, \( \mu \).

Theoretically, the frictional force is proportional to the normal load, independent of the area of contact and independent of the velocity. However, it should be emphasized that frictional processes are seldom that simple. According to Chaidez [2003], the following additions and modifications to friction principles should be made.

- For extremely low pressures and for pressures high enough to produce excessive deformation, the coefficient of static friction increases somewhat.
- For extremely low relative velocities, the coefficient of kinetic friction increases and apparently becomes equal to the coefficient of static friction without any mathematical discontinuity.
- For very high velocities, the coefficient of kinetic friction decreases appreciably.

![Diagram](image)

Figure 2.68: Relationship between frictional force and applied force [Chaidez, 2003].
The modern theory of solid dry friction focuses on identification of true contact area, the mechanism involved in interfacial bonding, and the localized inelastic deformation that occurs in the contact region [Soong and Dargush, 1997]. On detailed examination, it is possible to find out that both natural and engineered surfaces are not smooth at the microscopic level, but rather they contain waviness and roughness (Figure 2.69). As a result of this initial topography, true contact occurs only through the interaction of surface roughness. Researchers have found that a variety of topographical models produce true contact areas roughly proportional to the normal force, which is in general agreement with Coulomb theory [Buckley, 1981].

When true contact does occur directly between metals, adhesive bonds from across the interface often producing coefficients of friction, \( \mu > 1 \). However, adhesion provides a significant contribution primarily for the contact of clean metals. Typically, surface films and debris particles may also be present at the interface. In particular, oxide layers readily form under atmospheric conditions and generally prevent the development of adhesive bonds [Buckley, 1981; Ludema, 1996].

During slippage, localized heating of the contact materials will definitely occur as energy is dissipated along the interface. These thermal effects alter the frictional response by causing material softening or by promoting oxidation. However, for the type of sliding systems typically encountered in friction dampers, it is unlikely that system response will be sensitive to relatively small variations in ambient temperature that can be anticipated at any site [Soong and Dargush, 1997].
2.3.4.3 Mathematical Modeling

A mechanical model of a single friction damper located between girder and braces is proposed by Chaidez [2003] as shown in Figure 2.70. In this figure, $x$ and $x'$, represent the horizontal displacements of the main frame and of the dissipation device, respectively. The coefficient $K_N$ is the stiffness of the bracing system holding friction damper. In the contact surface, the limit condition for the unidirectional constitutive model based on Coulomb’s law is

$$f(F, u_N) = g(F, u_N) = |F| - \mu N = |F| - \mu K_N u_N \leq 0$$

(2.5)

where $f(F, u_N)$ and $g(F, u_N)$ are the plastic yielding limit function and the plastic potential, respectively [Mase, 1977]. $F$ is the friction force between the friction damper and the structure, $\mu$ is the coefficient of static dry friction ($\mu = \tan \phi^\text{fric}$ where $\phi^\text{fric}$ is the roughness angle). $N$ is the normal force given by $N = K_N u_N$ where $K_N$ is the penetration stiffness and $u_N$ is the penetration displacement. It should be noted that if the condition of Equation 2.1 is not satisfied, i.e., if $|F| > \mu N$, it means that sliding does occur ($\dot{x} \neq \dot{x'}$).

![Simplified Model of a Friction Damper](image)

**Figure 2.70:** Simplified Model of a Friction Damper [Chaidez, 2003].

Figure 2.71 illustrates a typical single story building equipped with a friction damper. The dashed line represents the original position of the structure, the lighter dashed line represents the assumed undeformed new position of the structure, and the solid line represents the real deformed structure. The coordinate $x$ and $x'$ are the horizontal displacements of the main frame and of the friction damper relative to the ground displacement. The sliding displacement is equal to $x - x'$. 
A mathematical model of the system is shown in Figure 2.72a while Figure 2.72b) shows the free-body diagram of the blocks corresponding to the main structure and to the damper. The equations of motion of the system in Figure 2.72 are

\[ m\dddot{x} + c\ddot{x} + kx = -m\dddot{x}_g(t) + P(t) - F \quad (2.6) \]

\[ m'\dddot{x}' + c'\ddot{x}' + k'x' = -m'\dddot{x}_g(t) + F \quad (2.7) \]

where \( m, c, \text{ and } k \) are mass, damping, and stiffness of the main structure (the lower block in the model), while \( m', c', \text{ and } k' \) are mass, damping, and stiffness of the combination of damper and bracing system (the upper block in the model). \( x, \dot{x}, \ddot{x} \) and \( \dddot{x} \) are displacement, velocity, and acceleration of mass \( m \) relative to the ground while \( x', \dot{x}', \ddot{x}' \), and \( \dddot{x}' \) are the corresponding quantities for the damped bracing system. \( P(t) \) is the applied force function of time \( t \) acting on \( m \), and \( F \) is the friction force between the damed bracing system and main structure. \( \dddot{x}_g(t) \) represents the ground acceleration. Also, the stiffness \( k' \) can be obtained by means of the expression

\[ k' = \frac{2EAL^2}{\left(4H^2 + L^2\right)^{\frac{3}{2}}} \quad (2.8) \]
where $E$ is the Young’s Modulus, $A$ is the cross section of tensile brace, $H$ is the column height, and $L$ is the girder length.

Equations 2.6 and 2.7 can be rewritten in the following matrix form,

$$
\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = -\mathbf{M}\ddot{\mathbf{r}}(t) + \mathbf{P}(t) + \mathbf{F}
$$

(2.9)

where mass matrix, $\mathbf{M} = \begin{bmatrix} m & 0 \\ 0 & m' \end{bmatrix}$, damping matrix, $\mathbf{C} = \begin{bmatrix} c & 0 \\ 0 & c' \end{bmatrix}$, stiffness matrix, $\mathbf{K} = \begin{bmatrix} k & 0 \\ 0 & k' \end{bmatrix}$, displacement vector, $\mathbf{x} = \begin{bmatrix} x \\ x' \end{bmatrix}$, unit vector, $\mathbf{r} = \begin{bmatrix} 1 \\ 1 \end{bmatrix}$, applied force vector, $\mathbf{P}(t) = \begin{bmatrix} P(t) \\ 0 \end{bmatrix}$, and friction force vector, $\mathbf{F} = \begin{bmatrix} -F \\ F \end{bmatrix}$.

**2.3.5 Viscoelastic Damper**

**2.3.5.1 Introduction**

Viscoelastic materials used in structural application are typically copolymers or glassy substances, which dissipate energy when subjected to shear deformation. A typical viscoelastic damper is shown in Figure 2.73, which consists of viscoelastic layers bonded with steel plates.
When mounted in a structure, shear deformation and hence energy dissipation takes place when the structural vibration induces relative motion between the outer steel flanges and the center plate [Soong and Dargush, 1997]. Figure 2.74 shows several installation options of viscoelastic damper in building structure.

![Diagram of viscoelastic damper configuration](image)

**Figure 2.73:** Typical Viscoelastic Damper Configuration [Soong and Dargush, 1997].

![Diagram of installation options for viscoelastic dampers](image)

**Figure 2.74:** Installation Options for Viscoelastic Dampers [Soong and Dargush, 1997].

The use of viscoelastic dampers in reducing wind and earthquake-induced motion of tall buildings structures was demonstrated by Samali and Kwok [Samali and Kwok, 1995]. The different damper configurations used in various buildings as discussed in the study are presented in Figure 2.75, Figure 2.76, and Figure 2.77.
Figure 2.75: Typical Configuration of World Trade Center Damper [Samali and Kwok, 1995].

Figure 2.76: Dampers Locations in Columbia Sea First Building in Seattle [Samali and Kwok, 1995].

Figure 2.77: Damper Configuration in Santa Clara County Building in San Hose [Soong and Dargush, 1997].
2.3.5.2 Basic Principles

Viscoelastic damping is exhibited strongly in polymeric and glassy materials, which depends upon temperature and frequency but is linear with respect to vibration amplitude up to a certain limit. Polymeric materials are made up of long molecular chains as shown in Figure 2.78. Damping arises from relaxation and recovery of the polymer network after deformation. A strong dependence exists between frequency effects and temperature effects because of the direct relationship between material temperature and molecular motion [Nashif et al., 1985]. If the polymers are homogeneous and isotropic, the shear, Young’s and bulk moduli are closely related to each other. Some polymers are not homogeneous or isotropic, and in this case, the modulus and damping properties are more complicated functions of direction and position within the material [Jones, 2001].

On the other hand, glasses are characterized not by long networks as in polymer, but by short-term order and long-term disorder (Figure 2.78). Damping also arises from relaxation processes after deformation of glass, recovery being due not to the original distribution of short networks but to other conditions of thermodynamic equilibrium. Creep can occur in glass material, but not in a cross-linked polymer. Since the static stiffness of cross-linked polymer can be quite high, creep will not occur [Nashif et al., 1985].

![Typical Polymeric Structure Network](image1)
![Typical Glass Structure](image2)

Figure 2.78: Polymer and Glass Structures [Nashif et al., 1985].

The complex modulus properties of viscoelastic polymer, represented by the moduli $G$ and $E$, vary strongly with temperature and frequency. The modulus decrease with the increase of temperature while the effect of frequency is the inverse of the effect of temperature, as depicted in Figure 2.79.
2.3.5.3 Mathematical Modeling

For an ideal elastic material, the stress-strain relationships are linear and simple as shown in the following Equations,

\[ \tau = G \gamma \]  \hspace{2cm} (2.10) \\
\[ \sigma = E \varepsilon \]  \hspace{2cm} (2.11) \\
\[ E = 2G(1 + \nu) \]  \hspace{2cm} (2.12)

where \( E \) is Young’s modulus, \( G \) is the shear modulus, \( \sigma \) is normal stress, \( \tau \) is shear stress, \( \varepsilon \) is normal strain, \( \gamma \) is shear strain, and \( \nu \) is Poisson’s ratio. For most metals at temperature well below the melting point, the shear and Young’s moduli do not vary strongly with temperature and they also do not vary rapidly with frequency for harmonic excitation, unless the strain becomes very large.

On the other hand, the stress-strain relationship for viscoelastic material are more complicated since there is a time or phase lag between the strain and the corresponding stress under harmonic excitation (Figure 2.80). The phase lag implies that a velocity dependent term exists in the stress-strain relationship. Therefore, the stress-strain relationships is given by
where $\omega$ is excitation frequency. $E'$ and $G'$ are defined as the storage modulus and shear storage modulus, which is a measure of the energy stored and recovered per cycle. $E''$ and $G''$ are defined as the loss modulus and shear loss modulus, which is a measure of the energy dissipated per cycle. $\eta$ is the loss factor, which is also often used as a measure of energy dissipation capacity, defined by $\eta = \frac{E''}{E'} = \frac{G''}{G'}$. The relationship of the loss factor, $\eta$, and the damping ratio, $\zeta$, is $\zeta = \frac{\eta}{2}$. A range of values of $\eta$ for some common engineering materials is given in Table 2.2.

Equations 2.13 and 2.14 can be rewritten as:

\[
\sigma = E'(1 + i\eta)\epsilon = E'^*\epsilon \quad \text{(2.15)}
\]
\[
\tau = G'(1 + i\eta)\gamma = G'^*\gamma \quad \text{(2.16)}
\]
\[
E' = 2G'(1 + \nu) \quad \text{(2.17)}
\]

Figure 2.80: Harmonic Excitation and Response for Elastic and Viscoelastic Materials [Jones, 2001].
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Table 2.2: Loss factors [Beards, 1996].

<table>
<thead>
<tr>
<th>Material</th>
<th>Loss Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminium-pure</td>
<td>0.00002-0.002</td>
</tr>
<tr>
<td>Aluminium alloy-dural</td>
<td>0.0004-0.001</td>
</tr>
<tr>
<td>Steel</td>
<td>0.001-0.008</td>
</tr>
<tr>
<td>Lead</td>
<td>0.008-0.014</td>
</tr>
<tr>
<td>Cast Iron</td>
<td>0.003-0.03</td>
</tr>
<tr>
<td>Manganese copper alloy</td>
<td>0.05-0.1</td>
</tr>
<tr>
<td>Rubber-natural</td>
<td>0.1-0.3</td>
</tr>
<tr>
<td>Rubber-hard</td>
<td>1.0</td>
</tr>
<tr>
<td>Glass</td>
<td>0.0006-0.002</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.01-0.06</td>
</tr>
</tbody>
</table>

\[
E^* = \frac{\sigma_0}{\varepsilon_0} = \sqrt{E'^2 + E''^2} \quad (2.18)
\]

\[
G^* = \frac{\tau_0}{\gamma_0} = \sqrt{G'^2 + G''^2} \quad (2.19)
\]

where \(E^*\) and \(G^*\) are complex moduli. \(\sigma_0\) and \(\tau_0\) are maximum normal and shear stresses while \(\varepsilon_0\) and \(\gamma_0\) are maximum normal and shear strains.

Another type of stress-strain relationship is obtained from the elliptical or near-elliptical hysteresis loop for linear viscoelastic material as illustrated in Figure 2.81. The stresses are given by

\[
\sigma = E'\varepsilon \pm E'' \sqrt{\varepsilon_0^2 - \varepsilon^2} \quad (2.20)
\]

\[
\tau = G'\gamma \pm G'' \sqrt{\gamma_0^2 - \gamma^2} \quad (2.21)
\]

It should be noted that the shape of the ellipse does not change as the maximum strain, \(\varepsilon_0\), changes. However, the shape does change, as the loss factor, \(\eta\), changes [Nashif et al., 1985].

The energy dissipated by the viscoelastic material per unit volume and per cycle of oscillation is given by an area of the ellipse, which can be written as
Since the maximum stored strain energy, \( U \), is \( \frac{1}{2} E' \varepsilon_0^2 \), it follows that \( \eta = D/2\pi U \) is an important measure of the damping capability of the material; the larger the value of \( D \), the larger is \( \eta \) and the thicker is the hysteresis loop [Nashif et al., 1985].

\[
 D = \oint \sigma \, d\varepsilon = \int_0^{2\pi/\omega} \sigma \left( \frac{d\varepsilon}{dt} \right) dt = \pi \eta E' \varepsilon_0^2 \tag{2.22}
\]

\[
 D = \oint \tau \, d\gamma = \int_0^{2\pi/\omega} \tau \left( \frac{d\gamma}{dt} \right) dt = \pi \eta G' \gamma_0^2 \tag{2.23}
\]

In reality, it is not easy to separate the stiffness and damping effects in structural systems because they are inherent properties which are often coupled [Beards, 1996]. Hence, the mathematical models of structures require these stiffness and damping properties to be considered together in the form of complex stiffness. The equation of free motion for a single degree of freedom system with hysteretic damping is therefore \( m\ddot{x} + k^* x = 0 \), that is, the combined effect of the elastic and hysteretic resistance to motion can be represented as a complex stiffness, \( k^* \). The complex stiffness, \( k^* \), is equal to \( k(1+i\eta) \), where \( k \) is the static stiffness.

There have been a number of classical rheological models being developed to represent the viscoelastic behavior. Apparently, many models have been based on combinations of elastic (spring) and viscous (dashpot) elements. The basic discrete models were presented, such as the Maxwell and Kelvin-Voigt models as illustrated in Figure 2.82a) and Figure 2.82b). When only a very few elements are involved in the model, calculations are relatively simple, but the agreement with observed behavior is usually poor, since the constant dashpot coefficient used implied far
too rapid a variation of complex modulus properties with frequency [Jones, 2001; Osinski, 1998]. Hence, the standard element model (Figure 2.82c), which is a combination of both Maxwell and Kelvin-Voigt models, and multiple standard element model (Figure 2.82d) have been presented.

The complex stiffness of the Maxwell, Kelvin-Voigt, standard element, multiple standard element models are presented in Equations 2.24, 2.25, 2.26 and 2.27 respectively.

\[ k' = k(1 + i\eta) = \frac{i\omega c_1}{k_1 + i\omega c_1} \quad (2.24) \]

\[ k' = k(1 + i\eta) = k_2 + i\omega c_2 \quad (2.25) \]

\[ k' = k(1 + i\eta) = \frac{i\omega c_1}{k_1 + i\omega c_1} + k_2 + i\omega c_2 \quad (2.26) \]

\[ k' = k_0 + \frac{i\omega c_1}{k_1 + i\omega c_1} + \frac{i\omega c_2}{k_2 + i\omega c_2} + \ldots \quad (2.27) \]
where \( k_i = k_{i1} + k_{i2} + \ldots \) is the summation of the Kelvin-Voigt stiffness elements. This can be separated into real and imaginary components as followed,

\[
k^* = \left( k_0 + \frac{\omega^2 k_1 c_1^2}{k_1^2 + \omega^2 c_1^2} + \frac{\omega^2 k_2 c_2^2}{k_2^2 + \omega^2 c_2^2} + \ldots \right) + i \left( \frac{\omega \gamma k_1 c_1^2}{k_1^2 + \omega^2 c_1^2} + \frac{\omega \gamma k_2 c_2^2}{k_2^2 + \omega^2 c_2^2} + \ldots \right) \quad (2.28)
\]

The proposed rheological models shown above are simple but do not represent the significant variation of material behavior with respect to temperature and frequency. To characterize the frequency- and temperature-dependent properties for viscoelastic material, the fractional derivative model was introduced. Concepts and details of the fractional derivative model can be found from the references [Soong and Dargush, 1997; Jones, 2001; Nashif et al., 1985].

### 2.3.6 Viscous Damper

#### 2.3.6.1 Introduction

In viscous damper, dissipation occurs via conversion of mechanical energy to heat as a piston deforms a thick, highly viscous substance, such as a silicon gel. Figure 2.83a) depicts a particular damper manufactured by GERB Vibration Control, which can be designed to provide vibration control in piping networks [Schwahn and Delinic, 1988] or for use as components in seismic base isolation systems [Huffmann, 1985; Makris and Constantinou, 1990]. The axisymmetric configuration provides motion, and hence energy dissipation, in all six degrees of freedom. An alternative, perhaps more effective, design concept involves the development of the Viscous Damping Wall (VDW) illustrated in Figure 2.83b) [Arima et al., 1988]. In this design, developed by Sumitomo Construction Company, the piston is simply a steel plate constrained to move in its plane within a narrow rectangular steel container filled with a viscous fluid. For typical installation in a frame bay, the piston is attached to the upper floor, while the container is fixed to the lower floor. Relative interstory motion shears the fluid and thus provides energy dissipation. By incorporating a sufficient number of VDW panels within the structural frame, a significant increase in the damping level can be achieved [Miyazaki and Mitsusaka, 1992]. Typical hysteresis loops of GERB and VDW dampers are shown in Figure 2.84.
a) Cylindrical Pot GERB Damper  b) Viscous Damping Wall

Figure 2.83: Viscous Liquid Dampers
[Makris and Constantinou, 1991; Miyazaki and Mitsusaka, 1992].

a) GERB Damper [Makris et al., 1995]  b) VDW Damper [Arima et al., 1988]

Figure 2.84: Damper Hysteresis Loops.
To maximize the energy dissipation density of these devices, one must employ materials with large viscosities. Typically, this leads to the selection of materials that exhibit both frequency and temperature dependent behavior. However, there is another class of fluid dampers that rely instead upon the flow of fluids within a closed container. In these designs, the piston acts now, not simply to deform the fluid locally, but rather, to force the fluid to pass through small orifices. Consequently, extremely high levels of energy dissipation density are possible. However, a correspondingly high level of sophistication is required for proper internal design of the damper unit.

A typical Taylor Devices fluid damper for seismic application is illustrated in Figure 2.85a [Constantinou et al., 1993]. This cylindrical device contains compressible silicone oil, which is forced to flow via the action of a stainless steel piston rod with a bronze head. The head includes a state-of-the-art fluidic control orifice design with a passive bimetallic thermostat to compensate for temperature changes. In addition, an accumulator is provided to compensate for the change in volume due to rod positioning. High strength seals are required to maintain closure. These uniaxial devices, which were originally developed for military and harsh industrial environments, have recently found application in seismic base isolation systems as well as for supplemental damping during seismic and wind-induced vibration. Typical experimentally measured hysteresis loops from Constantinou and Syman [1993] are illustrated in Figure 2.86. Another fluid damper featuring orifice flow to achieve energy dissipation is shown in Figure 2.85b. This device, manufactured by Jarret, utilizes a pressurized compressible silicone-based elastomer to provide additional structural stiffness and damping. Typical hysteresis loops obtained experimentally for the device are shown in Figure 2.87 [Pekcan et al., 1995].
a) Taylor Devices Fluid Damper  

b) Jarret Elastomeric Spring Damper

Figure 2.85: Viscous Liquid Dampers [Constantinou et al., 1993; Pekcan et al., 1995].

Figure 2.86: Tylor Device Hysteresis Loops [Constantinou and Symans, 1993].
2.3.6.2 Basic Principles

In this study, only Taylor type damper (Figure 2.84a) is of interest. The force generated by the fluid damper is due to a pressure differential across the piston head [Constantinou and Symans, 1992]. When the device is subjected to compression force, the piston moves from left to right. Subsequently, Therefore, the damping force is proportional to the pressure differential in these two chambers. Although, the fluid volume is reduced from compression, the development of a restoring (spring like) force is prevented by the use of accumulator. Basically the devices provide additional viscous damping to the fundamental mode of the structure (typically with a frequency less than the specified cutoff frequency) and additional damping and stiffness to the higher modes. As a result, this may completely suppress the contribution of the higher modes of vibration.

The force in the viscous damper is given by

\[ P = b\Delta p \]  \hspace{1cm} (2.29)
where $\Delta p$ is the pressure differential in two chambers. $b$ is an arbitrary constant, which is function of the piston head area ($A_p$), piston rod area ($A_r$), orifice area ($A_1$), number of orifices ($n$), area of control valves ($A_2$), and the discharge coefficient of the orifice ($C_{d1}$) and control valve ($C_{d2}$).

According to Bernoulli’s equation, the pressure differential across the piston for cylindrical orifices can be express as

$$\Delta p = \frac{\rho}{2n^2C^2_{d1}} \left( \frac{A_p}{A_1} \right)^2 \dot{x}^2 \text{sgn}(\dot{x})$$

(2.30)

where $\rho$ is the fluid density and $\dot{x}$ is the velocity of the piston with respect to the cylinder. In cylindrical-shaped orifices, the pressure differential is proportional to the piston velocity squared. Such orifices are termed square law or Bernoullian orifices [Constantinou and Symans, 1992].

### 2.3.6.3 Mathematical Modeling

The equation of motion for the single story structure with supplemental dampers (Figure 2.88) can be derived as

$$m\ddot{x} + c\dot{x} + kx + \eta P_d = -m\ddot{x}_g$$

(2.31)

where $m$ is the mass of the structure, $k$ is the stiffness of the undamped structure, $c$ is the damping constant of the structure without dampers, $\eta$ is the numbers of dampers, $P_d$ is the horizontal component of force in a single damper, and $\ddot{x}_g$ is the ground acceleration. $\ddot{x}$, $\dot{x}$ and $x$ are the relative acceleration, velocity, and displacement, respectively.

For a damper inclined at an angle $\theta$ with respect to the horizontal axis, the constitutive equation describing the damper force $P_d$ in the horizontal direction is

$$P_d + \dot{\lambda} P_d = C_o \dot{x} \cos^2 \theta$$

(2.32)

where $\dot{\lambda}$ is the relaxation time, and $C_o$ is the damping constant at zero frequency.
2.4 ARCHITECTURAL CLADDING WITH ENERGY DISSIPATING DEVICES

Conventional design of connections addresses the effects of interstory drift effect through a mechanism that isolates the panel from the main structure, hence ignoring any interaction between the cladding panel and the building structural system. However, the cladding panel connection can also be designed to engage in interaction with the structural system. Craig et al. [1992] proposed the concept of the advanced connections to take advantage of this interaction to dissipate energy, therefore reducing the response of the main structure. There are generally three main parts in the advanced connection of a precast cladding system as illustrated in Figure 2.89, which are: first, the attachment points built into the precast panel, which typically consist of steel inserts embedded in the concrete panel; second, the connector body which forms the structural connection between the cladding panel and the main structure; third, the attachment to the main building structure, which can be another steel insert embedded in the concrete frame.

From the analysis in a test program conducted by Pinelli [1990], it was suggested that the attachments or inserts are not by themselves capable of providing the levels of ductility and damping required from an advanced connection without loss of strength and integrity. Therefore in an advanced connection the energy dissipation must occur in the connector body if the integrity of the concrete panels is to be maintained.
Pinelly et al. [1992] summarized several ideas for advanced connector bodies in Figure 2.90. Ductility and damping can be developed through a number of different passive processes including: extrusion; inelastic connector action initiated through torsional or flexural effects in the connection element; friction effect developed in slip processes for connectors designed with layered materials and fastened with bolts in oversized holes; use of composite system manufactured with materials selected for strength and ductility; and use of viscoelastic materials.

2.4.1 Energy Dissipating Connectors – Yielding Concept

According to Pinelly et al. [1996], many absorbing devices based on the plastification of steel take advantage of the fact mild steel that can reliably provide high stiffness in the elastic
range as well as absorb energy with moderate strain hardening when deformed beyond the elastic limit. In addition, steel is relatively easy to manufacture in different shapes, and the stiffness and damping properties of the devices can be improved with judicious choice of geometries. Steel is also economical, and widely used in construction, and therefore it is a material trusted by practitioners.

An attractive plastic deformation mechanism for steel connections is torsion. Torsional devices have some advantages: they have better energy absorption qualities because the uniform distribution of the torsional moment results in a better use of the material and exhibit progressive ductile failure distributed over the length of the device. However, the attachment is somewhat complicated if trying to achieve pure torsion in a device between surfaces moving parallel to each other, although the design can be simplified if torsion and bending are combined.

Kelly et al. [1972] presented the connector, which is a ductile close loop, made of mild steel with semi-circular ends (Figure 2.91). The flexural action in the rolling and unrolling of the loop ends will provide energy dissipation during a moderate or strong earthquake. The advantage of the loop is its symmetry, which makes it suitable for cyclic loading. Also, in a loop, the strain depends on the ratio of thickness to radius and is independent of the displacement.

![Ductile Loop Connection](image)

**Figure 2.91**: Ductile Loop Connection [Kelly et al., 1972].

The research on the enhancement of infilled frame systems has been conducted. Newmark and Rosenblueth [1971] described a system of peripheral metal bands to protect infill partitions from earthquake damage. This system was proposed by Guerrero [1965] and is shown in Figure 2.92a). If the shearing and normal forces required to make the band yield are chosen properly, it is possible to have a passive control on the lateral forces that the structure will transmit to the infill partition and to make use of the energy absorption capacity of the metal bands.

Although the above proposal of Guerrero [1965] may be effective in protecting infill wall partitions and in making a significant contribution to the energy dissipation capacity of the structure, the implementation in practice of this system appears to be rather cumbersome and
hence expensive. A very large number of connections between metal bands and concrete members are required. This would imply a very expensive construction procedure that includes the provision of steel inserts in the columns, beams, and infill wall, according to Martinez-Rueda [2002].

Muto [1969] introduced a nonconventional seismic design approach that has been used in Japan, where a number of tall buildings have been designed with reinforced concrete infill panels. As shown in Figure 2.92b), each panel has a set of vertical slits as a result of which the panel acts as a series of RC columns. When a panel is deformed by interstory deflection, plastic hinges are formed at the top and bottom of each effective column, thus absorbing energy.

According to Martinez-Rueda [2002], the slitted RC wall of Muto appears to be the first documented application in built structures of energy-dissipating elements based on inelastic behavior. These elements can be sacrificed during a major event and, in terms of their limited ductility and repairability, they may be considered as poor energy dissipation devices for today standards. In fact, while constituting an advance in seismic design, the use of slitted walls may result in some disadvantages particularly if the panels are used in the redesign of an existing building. The slitted walls add substantial weight to the structure and consequently to the inertial forces generated by the earthquake motion. Moreover, reinforced concrete suffers rapid deterioration under cyclic plastic deformation. It would seem more logical to use reinforced concrete to carry vertical loads and use plastically deforming steel to dissipate energy. Additionally, in the event of a damaging earthquake, the removal and replacement of failed slitted walls will certainly involve significant construction work affecting considerably the functioning of the building.

Figure 2.92: Yielding Devices into Infilled Wall [Martinez-Rueda, 2002].
In Wanganui, New Zealand, a 6-story building with the facades cladding panels included braces with energy dissipating steel inserts was designed by Matthewson and Davey [1979]. Details and location of precast concrete cladding panels are shown in Figure 2.93 and Figure 2.94. Separate steel plates were cast into the panels and into the columns, and are butt welded together to transfer the vertical component of the diagonal forces. Inertia loads were applied to the panel top chords by the floor diaphragms. The steel inserts in the panel diagonals were mild steel rolled hollow sections. The steel was stress relieved to maximize available ductility as the only obtainable form was cold rolled. The yielding rolled hollow section was separated from the surrounding concrete by another rolled hollow section, which gave approximately 1/16” clearance. Lateral translation of one end of an insert relative to the other was limited by a circumferential plate cut to a close tolerance fit and placed at the lower end of the diagonal. Inserts were galvanized after fabrication.

The cladding panels in the building were part of the LFRS, and truss action of the cross bracing dominated the structural behavior. Frame action became significant only after yielding of the inserts. The preliminary estimate gave the building system using precast concrete panels as structural elements cost approximately 25% less per square meter than a six story conventional reinforced concrete frame building of similar size and use.

Figure 2.93: Building elevation [Matthewson and Davey, 1979].
Figure 2.94: Braced Cladding Panel and Steel Insert Detail [Matthewson and Davey, 1979].
Pinelly et al. [1993] presented connector consisting of a section of square tube, cut away as shown in Figure 2.95a) to create two narrow flexural elements whose widths are tapered to initiate plastification over a greater portion of material. The two tapered beams in flexure have a smaller cut away width through the cut-away than the fixed untapered elements to ensure that they deformed with double curvature. The connector should be placed between a panel and the supporting structure through a bolted attachment as shown in Figure 2.95b).

An experimental test program was also conducted to study the behavior of the different components of a connection system. Analytical models (Figure 2.96 and Figure 2.97) of the connection were incorporated into a two dimensional model of a 6-story building with cladding. The response of the structure to earthquake excitation was traced using time histories of the energy demand and supply to the building, both with and without cladding. Results showed that properly designed energy dissipative connector elements could be responsible for the total hysteretic energy dissipated in the structural system. A design criterion for the connection, formulated in terms of energy, provides the optimal balance of stiffness and strength to be added to the structural system by the dissipators, which results in maximum energy dissipation in the connections, no plastification in the structural members, and reduced structural response.
Cohen [1994] discussed about the study by Pinelly et al. [1993] in order for this research to lead to practical applications. Several comments have been made and summarized.

- Some types of cladding addressed by Pinelly et al. [1993] have not appeared in recent work described in U.S. architectural magazines, including Architecture: The AIA Journal, Architectural Record, Progressive Architecture, etc. Therefore, Cohen [1994] recommended that the authors develop research collaboration with an architect, preferably one who is well versed in both design and technology.

- For architectural detailing, it seemed that the dimension between the cladding panels and the exterior surface of the structural frame needed to be minimized.
For structural detailing, it was possible to simplify the connection behavior and uncouple structural function. The locations of the connections should be along the sides of the panel rather than the corners. In doing this, there was a clear distinction between supporting the weight of the panels and transmitting shear forces from the panel to the frame. Also, if the demand was such that multiple metallic yielding devices were needed, several could be installed along the panel edges. In addition, the connection could be more easily detailed not to transmit compression forces to the panels.

The analytical model of the energy-dissipating connections seemed to be very complicated, considering the uncertainties in the fabrication and construction process. Also, description was not clear on how to accomplish the design process.

In this discussion, Cohen [1994] also referred to structural detailing scheme expressed by Cohen and Powell [1991; 1993] as follows:

a) Horizontal shears are transferred between the spandrel beams and panels through the connections along the horizontal bottom and top edges of the panels. At the bottom edges of the panels, the connections are designed to be elastic. At the top edges of the panels, the connections are designed to be inelastic, and hence energy dissipating. These connections remain elastic for wind loads and mild earthquakes. The bottom and top connection are flexible in the vertical direction. This flexibility eliminates compression forces in the panels from column shortening, beam flexure, and differential thermal movement. The connections are assumed to have no rotational stiffness in the analytical model.

b) Vertical shear is transferred between the columns and panels through the connections along the vertical edges of the panels. The connections are attached at mid-height of the columns. They also support the gravity load of the cladding. These connections are (vertically) short and fin-like, so that column shortening and differential thermal movement do not compress the panel. The connections are designed to remain elastic. They are analytically flexible in the horizontal direction and have no rotational stiffness.

c) At each horizontal edge, separate connections are assumed for the panels in the stories above and below (i.e., an elastic connection for the panel above, and an inelastic connection for the panel below). There is no direct panel-to-panel connection.

d) At each vertical edge, a single elastic fin connection is assumed, connected to both the left panel and the right panel. This provides for force transfer from panel to column and also directly from panel to panel.
2.4.2 Energy Dissipating Connectors – Frictional Concept

A number of studies on the use of friction devices for wall systems have also been undertaken for a few decades. The pioneer work of friction devices for the wall elements was presented by Tyler [1977a]. He proposed a method to reduce damage in infill panels while providing the building with an acceptable level of damping. As shown in Figure 2.98, the system introduced sliding elements of Polytetrafluoroethylene (PTFE) (PTFE is more commonly known under the trademark Teflon) that joined infill panels with frame members. The infill panels were then allowed to take load within their capacity when the joints slip within a known force range. In this way, it was possible to provide supplemental damping and at the same time make a reasonable allowance for the effect of secondary elements in the design of the main structure. PTFE sliding elements consisted of layers of PTFE joint by a high-strength bolt to steel plates and infill panels. Teflon was selected as the friction material because of the good energy absorption characteristics as indicated by a near-rectangular hysteresis loop observed during tests of the joints under harmonic excitation [Tyler, 1977b].

According to Martinez-Rueda [2002], although the application of friction devices proposed by Tyler appeared very promising, he did not undertake studies of his proposal using analytical or experimental models of a framed building under seismic loading. Consequently, no guidance was provided with respect to how the proposed devices should be calibrated to achieve acceptable building performance.

![Figure 2.98: PTFE Sliding Elements [Tyler, 1977a].](image-url)
Pall et al. [1980] developed friction devices for the passive seismic control of precast and cast-in-place concrete walls. Figure 2.99 shows possible locations of friction joints in shear walls, as well as typical details for the friction joints. The slipping friction joints consist of heavy-duty brake lining pads (Ferrodo) inserted between sliding steel plates jointed by high-strength bolts. The friction joints must be carefully engineered to approach ideal elastoplastic behavior. The slip load of the joint is determined by the coefficient of friction and the clamping force on the joint. The desired clamping force is provided using high-strength bolts. Experimental results have shown [Pall and Marsh, 1981] that the hysteretic behavior of the slipping friction joints is in fact reliable and repeatable, and approaches a rectangular hysteretic loop with negligible degradation over many more cycles than encountered in successive earthquakes. By creating vertical joint lines inside concrete walls and using carefully engineered friction joints to couple them together, the following beneficial characteristics of the proposed system emerge:

- The walls act monolithically during service load conditions including wind and moderate earthquakes.
- The flexibility of the system increases as the friction joints slip during major earthquakes resulting in effective period elongation which in general results in reduced seismic accelerations.
- The friction joints dissipate a large portion of the seismic input and delay the possible onset of inelasticity in the LFRS.

Figure 2.99: Friction Joints in Concrete Wall [Pall and Marsh, 1981].
Pall and Marsh [1981] performed a series of nonlinear time-history analyses of a concrete wall of an apartment building of 20 stories for several earthquake intensities. The earthquake record of El Centro 1940 NS component was used in the study. The analyses showed that the friction joints should be tuned to optimize seismic performance. Because of the very low displacements encountered in wall systems, the criterion that defines the optimum device tuning was the minimization of stresses at the base of the walls. Comparisons of the building performance assuming full monolithic wall action (i.e., equivalent to the case of friction joints with infinite strength) and with slipping friction joints showed that these joints reduced effectively maximum values of shear, bending, deflections and overturning moments of the walls in as much as 25%, 30%, 40%, and 20%, respectively.

Another research of friction damping device was investigated by Popov and Youssef [1998]. They studied the use of reinforced of concrete cladding on walls and domes to resist seismic force with added friction connection (Figure 2.100). The results presented that the cladding became an integral part of the structural system. Preliminary experimental evidence on the excellent behavior under cyclic load and selected cases of simulated seismic motion of a full-size panel with simple energy dissipators was presented. Based on experiments, it would appear that this system of using the friction devices described was very suitable for practical applications. It was suggested that this system should be particularly effective on retrofit and for precast low-rise buildings where energy dissipation capability was important. Moreover, the cost of these dissipators was low and installation was simple.

Figure 2.100: Cross Section of the Cladding Connection [Popov and Youssef, 1998].
The applications of the friction-based energy dissipators for cladding and wall system are not limited only in U.S. and Canada, but also spread all over the world. An example of the application in UK was presented by Petkovski and Waldron [1995]. A system consisting of heavy precast concrete panels connected to a reinforced concrete frame building by clamped slip connections is presented in Figure 2.101 and a mathematical model is shown in Figure 2.102.

Some of the aspects relevant for the design of the frame were studied and the performance of the system was analyzed by the means of inelastic seismic response. The analyses showed that with a standard design for the frame and relatively simple criteria for defining the characteristics of the friction connections, the seismic behavior of the structure could be...
significantly improved. A comparison of the responses of the reference frames and the clad
structures showed that, with a relatively modest strength demand on the control system,
displacements were substantially reduced, thereby significantly reducing damage to the main
structure in a major earthquake.

Figure 2.102: Cladding Panel System and the Mathematical Model
[Petkovski and Waldron, 1995].

2.5 SUMMARY

A literature research was conducted to review the state-of-the-art knowledge in
architectural precast concrete cladding system, passive energy dissipation systems, and the
application of energy dissipation concept in architectural cladding system. It was found that
current guidelines on the earthquake-resistant architectural precast cladding system are aimed at
minimizing the cladding-structure interaction and damage arising from it. However, studies have
shown that it is not possible to completely eliminate such interaction effects and that the installed
cladding panels have significant effect on the lateral stiffness and dynamic characteristics of a
building. A review of the energy dissipating systems in building application revealed that much
research has been performed for more than a decade on the use of such system to improve the
seismic resistant of building structures. The concept of energy dissipation has also been extended
to architectural precast concrete cladding system through the development of advanced cladding
connections.
2.6 CONCLUSION

Modern building facades typically have continuous and large openings on top of the spandrel beams or precast cladding panels at each story for the installation of windows or ventilation requirement (e.g., for open parking structure). This aspect appeared to be ignored in the development of energy dissipating cladding panels that span from one floor to an adjacent floor. Furthermore, existing studies on advanced cladding system generally focus on the development of unique mechanism for energy dissipation with little or no attention on the added lateral stiffness provided by the cladding panel itself. It was therefore concluded that the development of an added energy dissipating system which can be applicable specifically to spandrel type precast concrete cladding panels should be pursued.

Another conclusion of the study is that energy dissipating elements that are separate from the building structural frame and will be added to the system as dampers are becoming more and more accepted. Development of many different varieties of such systems points to this direction and it encourages research for their further development to be used in different parts of buildings.
CHAPTER 3
FEASIBILITY STUDY

3.1 CHAPTER OVERVIEW

One justification for including an energy dissipating system in cladding panels is the possibility of using such an energy dissipating panel as a bracing member. This bracing system can potentially substitute conventional bracing systems or complement them. In particular, since the cladding panels are usually used on all perimeter bays in a given side, such a substitution can provide the opportunity of doing away with visible conventional bracing systems. Since published information related to the idea of adding partial-height bracing as part of spandrel type cladding panels is scarce, it is first necessary to develop a comparison between the performance of this partial high bracing system and the conventional full-height bracing system under seismic loading. To demonstrate the influence of the spandrel-type cladding as the new bracing system on the seismic response of a building system, multi-story buildings with added special braces (at one-third height) will be modeled in this step of the study.

This chapter covers two analytical studies that were carried out to investigate the effect of perimeter spandrel bracing on the static and dynamic behavior of a multi-story building and frame. The purposes of the studies include:

- To compare the stiffness of one-third height perimeter spandrel bracing with conventional full-height LFRS’s on a typical 4-story ordinary moment resisting frame building using the same equivalent static seismic forces.
- To investigate the effect of added spandrel bracing system and energy dissipation system on the dynamic response of a typical 3-story shear frame subjected to an actual earthquake motion.
The preliminary analytical studies are presented in two parts. Part I describes the static analysis of a three-dimensional 4-story moment resisting frame building for equivalent static earthquake loading in accordance to IBC2006 [ICC, 2006]. The second part covers a series of time history analyses of a two-dimensional, 3-story shear frame with energy dissipating systems attached and one-third height spandrel bracing.

3.2 ANALYSIS METHODOLOGY

Existing guidelines and codes [SEAOC, 1999; ICC, 2006] generally allow the designer to use any of the following analysis methods: Linear Static Procedure (LSP), Nonlinear Static Procedure (Pushover Analysis) (NSP), Linear Dynamic Procedure (LDP), and Nonlinear Dynamic Procedure (NDP). The availability of economical and high-speed computers and finite element structural analysis software has made time-history analyses, which has greater applicability, particularly attractive. Static analysis may still be required in certain cases. For example, the result from LSP is often used to scale the dynamic analysis results to meet code specific design spectra. Generally, when energy dissipation systems are implemented, LSP is of very limited applications. Here, NDP is used to capture the nonlinear hysteretic response of the EDS. For the purpose of this report, LSP is used for linear structures without energy dissipation system whereas NDP is employed when EDS are incorporated. A well-established finite element structural analysis program ETABS Nonlinear Version 8.2.6 was selected due to its availability and ease of use. It should be pointed out that ETABS was available in two versions: Plus and Nonlinear. Analysis of buildings incorporating base isolators or supplemental damping devices requires the nonlinear version. A number of researchers investigating structures with supplemental EDS have utilized ETABS or SAP software in their analytical studies (Table 3.1). It should be pointed out that SAP (current version is known as SAP2000), based on the same numerical techniques and solution algorithms as ETABS, could have been used since it is a more general purpose program (intended for civil structures such as bridges, dams, stadiums, as well as buildings). ETABS was chosen primarily due to its ability to account for the unique properties inherent (and terminology used) in a mathematical model of a building [CSI, 2002]. This greatly facilitates the construction of any multi-story building model.
PART 1 – THREE-DIMENSIONAL STATIC ANALYSIS

The purpose of this part of the analytical study is to investigate the influence of one-third story height spandrel bracing on lateral load resistance of a typical 4-story ordinary moment resisting frame building against equivalent static seismic forces.

3.3.1 Model Description

3.3.1.1 Ordinary Moment resisting Frame (OMF) Building

The structure considered is a L-shaped 4-story steel Ordinary Moment Resisting Frame (OMF) building (Figure 3.1). All stories are 3.7 m (12 ft) high. All bays are 7.3 m (24 ft) wide in the two orthogonal (X and Y) directions. The composite floor consists of 76.2 mm (3 in.) of composite deck. The simply-supported secondary (infill) beams are specified as composite beams.
whereas the girders connecting to the columns are non-composite. The building is modeled as simply-supported at the base. Ten different types of LFRS are investigated and a brief description of the systems is covered next.

3.3.1.2 Lateral Force Resisting Systems (LFRS)

The Lateral Force Resisting Systems (LFRS) studied are summarized in Table 3.2 and in Figure 3.2. The first three cases or systems, 1OR, 2SW and 3XB, represent conventional LFRS construction while the remaining incorporate various configurations of the proposed one-third height spandrel steel bracing. Except for system 2SW in which the LFRS (i.e., shear core wall) is placed within the building envelope (Figure 3.2b), the braces are pinned-connected to the perimeter frames for all other systems. The following should be noted.

- All models have the same basic moment frame (i.e., OMF). Different LFRS is added to the basic moment frame for each system.
- Where spandrel bracing system is used in a story, the same system is applied for the entire perimeter of the building for that story.

Figure 3.1: 4-story Steel OMF Building.
• All spandrel braces are pin-connected to the columns at one-third height, either below or above the story. For 4SB1 and 5SB2, the braces are connected to the column-beam connections. For 6SB3, the braces are connected to the columns only.

• For 7SB4, 8SB5 and 9SB6, each brace is connected to the beam at a quarter-length. For system 10SB7, each brace is connected to the beam mid-span.

• X-bracing is used to avoid a soft story effect at the 1st story. Where spandrel braces exist below the 1st story, the upper end of the X-bracing is connected to the spandrel bracing-column connection.

Table 3.2: LFRS Schedule.

<table>
<thead>
<tr>
<th>No.</th>
<th>Model/System</th>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>1OR</td>
<td>3.1</td>
<td>OMF</td>
</tr>
<tr>
<td>2.</td>
<td>2SW</td>
<td>3.2 a), b)</td>
<td>OMF + reinforced-concrete shear core wall</td>
</tr>
<tr>
<td>3.</td>
<td>3XB</td>
<td>3.2 c)</td>
<td>OMF + X-bracing tower placed at perimeters A3-4, F1-2, 5B-C and 1C-D (Figure 3.1b).</td>
</tr>
<tr>
<td>4.</td>
<td>4SB1</td>
<td>3.2 d)</td>
<td>OMF + Spandrel-height X-bracing at all stories, except roof level.</td>
</tr>
<tr>
<td>5.</td>
<td>5SB2</td>
<td>3.2 e)</td>
<td>OMF + X-bracing at first story + Spandrel-height X-bracing at all other stories, excluding roof level.</td>
</tr>
<tr>
<td>6.</td>
<td>6SB3</td>
<td>3.2 f)</td>
<td>OMF + X-bracing at first story + Spandrel-height interstory X-bracing at all other stories, excluding roof level</td>
</tr>
<tr>
<td>7.</td>
<td>7SB4</td>
<td>3.2 g)</td>
<td>OMF + X-bracing at first story + Spandrel-height interstory knee bracing at all other stories, excluding roof level</td>
</tr>
<tr>
<td>8.</td>
<td>8SB5</td>
<td>3.2 h)</td>
<td>7SB4 + Spandrel-height corner bracing at roof level</td>
</tr>
<tr>
<td>9.</td>
<td>9SB6</td>
<td>3.2 i)</td>
<td>OMF + X-bracing at first story + Spandrel-height eccentric X-bracing at all other stories except roof level</td>
</tr>
<tr>
<td>10.</td>
<td>10SB7</td>
<td>3.2 j)</td>
<td>OMF + X-bracing at first story + Spandrel height V-bracing at all other stories except roof level</td>
</tr>
</tbody>
</table>
Figure 3.2: Lateral Force Resisting Systems.
3.3.2 Analysis Procedures

Though the building possesses a plan irregularity (i.e., a reentrant corner), it is under 19.8 m (65 ft) in height. IBC2006 [ICC, 2006] allows the building under consideration to be analyzed with the Code’s Equivalent Lateral Force (ELF) method. As required by the Code, however, an increase in design forces of 25% for connection of diaphragms to vertical elements and to collectors, and connection of collectors to vertical elements must be provided for [ICC, 2006]. However, this requirement is not crucial for the present study. All structural elements were sized by the ETABS steel design module based on AISC-LRFD [1999] provisions. Eigenvector modal analysis was also performed to establish the dynamic properties of the each system.

3.3.3 Loadings

The gravity loads used consist of the dead and live loads acting on the building and the details are given in Table 3.3. All earthquake loads were generated and distributed automatically by ETABS for both N-S and E-W directions, in accordance to the IBC2006’s ELF procedure [ICC, 2006]. The earthquake parameters and loading criteria used in this study (Table 3.4) represent a zone of high seismicity, with $S_{DS}$ and $S_{DL}$ greater than 0.5g and 0.2g, respectively [SEAOC, 1999]. It should be pointed out that different response modification factors $\mu$, building periods ($T$) and seismic response coefficient ($C_s$) could have been computed for different LFRS’s used here. However, the primary objective of the study is to compare the stiffness of the spandrel type bracing with the other LFRS’s. Hence to facilitate comparison, the same equivalent seismic lateral load was used by specifying a single value for these parameters. Here, these parameters were based on the basic moment frame (i.e., OMF).

In accordance to IBC2006 [ICC, 2006], the load combination used in analysis was $(1.2D + 1.0E + 0.5L)$ for each (X and Y) direction of earthquake component. In this study, simple addition was used for the load combination. It should be pointed out that any load combination could be used since only the relative performance of the various systems is important. The effect of the initial P-$\Delta$ analysis was also included in all static load cases.
Table 3.3: Gravity Loadings.

<table>
<thead>
<tr>
<th>No.</th>
<th>Type</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td></td>
<td>Self weight of building structure</td>
<td>automatically calculated</td>
</tr>
<tr>
<td>2.</td>
<td>Dead</td>
<td>Additional weight for partitions, false ceiling,</td>
<td>1.7 kN/m² (35 psf)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>electrical and mechanical services</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td></td>
<td>Cladding self weight at the perimeter of the building</td>
<td>3.65 kN/m (250 psf)</td>
</tr>
<tr>
<td>4.</td>
<td>Live</td>
<td>Live load reduction not used in the analysis</td>
<td>4.8 kN/m² (100 psf)</td>
</tr>
</tbody>
</table>

Table 3.4: Seismic Design Parameters.

<table>
<thead>
<tr>
<th>No.</th>
<th>Design Parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Site class</td>
<td>SC</td>
<td>B</td>
</tr>
<tr>
<td>2.</td>
<td>Seismic use group</td>
<td>SUG</td>
<td>I</td>
</tr>
<tr>
<td>3.</td>
<td>Occupancy importance factor</td>
<td>I</td>
<td>1.0</td>
</tr>
<tr>
<td>4.</td>
<td>Mapped 0.2 s spectral acceleration</td>
<td>S_s</td>
<td>1.00g</td>
</tr>
<tr>
<td>5.</td>
<td>Mapped 1.0 s spectral acceleration</td>
<td>S_j</td>
<td>0.39g</td>
</tr>
<tr>
<td>6.</td>
<td>Site class factor for 0.2s acceleration</td>
<td>F_a</td>
<td>1.0</td>
</tr>
<tr>
<td>7.</td>
<td>Site class factor for 1.0s acceleration</td>
<td>F_v</td>
<td>1.0</td>
</tr>
<tr>
<td>8.</td>
<td>Design short period ground acceleration</td>
<td>S_Ds</td>
<td>0.67 g</td>
</tr>
<tr>
<td>9.</td>
<td>Design long period ground acceleration</td>
<td>S_D1</td>
<td>0.26 g</td>
</tr>
<tr>
<td>10.</td>
<td>Fundamental period for OMF</td>
<td>T</td>
<td>0.766 sec</td>
</tr>
<tr>
<td>11.</td>
<td>Accidental eccentricity ratio</td>
<td>e</td>
<td>5%</td>
</tr>
<tr>
<td>12.</td>
<td>Seismic response coefficient</td>
<td>C_s</td>
<td>~ 0.098</td>
</tr>
<tr>
<td>13.</td>
<td>Response Modification Factor</td>
<td>R</td>
<td>3.5</td>
</tr>
</tbody>
</table>
3.3.4 Analytical Results

3.3.4.1 Member Sizes

The final member sizes used in the models are tabulated in Table 3.5. The secondary beam (in-filled beam), column and girder in case 1OR met the strength check in accordance to IBC2006 [ICC, 2006] load combination and were thus used for all building cases.

Table 3.5: Size of Members.

<table>
<thead>
<tr>
<th>Case</th>
<th>Secondary beam</th>
<th>Column</th>
<th>Girder</th>
<th>Bracing</th>
<th>Shear wall thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1OR</td>
<td>W12x16</td>
<td>W12x96</td>
<td>W18x50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2SW</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td>6”</td>
</tr>
<tr>
<td>3XB</td>
<td></td>
<td></td>
<td></td>
<td>L 4x4x(\frac{3}{4})</td>
<td>-</td>
</tr>
<tr>
<td>All other cases</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.3.4.2 Fundamental Periods

A total of twelve modes were determined for the linear modal analysis of each system. The period of the dominant mode (based on the highest mass participation factor) for each system is shown in Table 3.6 and it is immediately noted that the frequencies for the building with steel bracing is close to that (0.766 sec) estimated based on IBC2006 equation [ICC, 2006]. As expected, the periods are lower in the X-direction due to greater stiffness from greater number of bays. The shear wall provides the greatest additional stiffness to the OMF while the added stiffness from each spandrel bracing system is close (but lower than) to that from the X-bracing.
3.3.4.3 Maximum Displacements and Interstory Drifts

The maximum displacements and story drifts are summarized in Table 3.7. The corresponding envelopes are shown in Figures 3.3 and 3.4. The results indicate that the various configurations of perimeter spandrel bracing systems are effective in reducing the story displacements and interstory drift though the total lateral stiffnesses of the perimeter spandrel braces are slightly lower than that of the system using conventional X-bracing tower. It appears that system 10SB7 is the most effective in reducing lateral response, compared to other spandrel bracing systems.

Table 3.6: Fundamental Periods.

<table>
<thead>
<tr>
<th>System</th>
<th>Fundamental (dominant) mode</th>
<th>X-direction</th>
<th>Y-direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mode</td>
<td>Period (sec)</td>
<td>Mass participation factor</td>
</tr>
<tr>
<td>1OR</td>
<td>2</td>
<td>1.222</td>
<td>93.2%</td>
</tr>
<tr>
<td>2SW</td>
<td>3</td>
<td>0.207</td>
<td>74.6%</td>
</tr>
<tr>
<td>3XB</td>
<td>2</td>
<td>0.743</td>
<td>86.9%</td>
</tr>
<tr>
<td>4SB1</td>
<td>2</td>
<td>0.828</td>
<td>91.1%</td>
</tr>
<tr>
<td>5SB2</td>
<td>2</td>
<td>0.790</td>
<td>88.8%</td>
</tr>
<tr>
<td>6SB3</td>
<td>2</td>
<td>0.819</td>
<td>88.0%</td>
</tr>
<tr>
<td>7SB4</td>
<td>2</td>
<td>0.839</td>
<td>87.6%</td>
</tr>
<tr>
<td>8SB5</td>
<td>2</td>
<td>0.837</td>
<td>87.8%</td>
</tr>
<tr>
<td>9SB6</td>
<td>2</td>
<td>0.772</td>
<td>89.4%</td>
</tr>
<tr>
<td>10SB7</td>
<td>2</td>
<td>0.760</td>
<td>89.8%</td>
</tr>
</tbody>
</table>
Table 3.7: Maximum Displacements and Story Drifts.

<table>
<thead>
<tr>
<th>System</th>
<th>Max 4th story displacement</th>
<th>Max story drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X-direction</td>
<td>Y-direction</td>
</tr>
<tr>
<td></td>
<td>(mm)</td>
<td>(in.)</td>
</tr>
<tr>
<td>1OR</td>
<td>49.9</td>
<td>1.965</td>
</tr>
<tr>
<td>2SW</td>
<td>5.3</td>
<td>0.210</td>
</tr>
<tr>
<td>3XB</td>
<td>21.2</td>
<td>0.836</td>
</tr>
<tr>
<td>4SB1</td>
<td>25.1</td>
<td>0.989</td>
</tr>
<tr>
<td>5SB2</td>
<td>23.1</td>
<td>0.911</td>
</tr>
<tr>
<td>6SB3</td>
<td>25.3</td>
<td>0.997</td>
</tr>
<tr>
<td>7SB4</td>
<td>26.4</td>
<td>1.040</td>
</tr>
<tr>
<td>8SB5</td>
<td>26.8</td>
<td>1.055</td>
</tr>
<tr>
<td>9SB6</td>
<td>22.0</td>
<td>0.868</td>
</tr>
<tr>
<td>10SB7</td>
<td>21.4</td>
<td>0.842</td>
</tr>
</tbody>
</table>

![Graphs showing maximum story displacement envelopes for different systems.](image)

**Note:** 1 in. = 25.4 mm

Figure 3.3: Maximum Story Displacement Envelopes.
3.3.4.4 Building Forces

From Table 3.8, with the exception of system 2SW, the maximum story shear and maximum torque values in both directions are identical since the weight of the building (hence the total design seismic base shear) is almost the same for these cases; the weight difference due to different bracing configurations is negligible. 2SW has larger (about 36%) story shears due to the heavier RC shear wall. With spandrel bracing, the total axial force in the building is about 11% lower than for the building with conventional X-bracing, indicating an advantage of using perimeter bracing. This reduction is due to the distribution of the axial forces over a greater number of (perimeter frames) columns.
As seen from Table 3.9, the maximum column axial forces follow the same trend as the building axial force. However, the Combined Stress Index (CSI) based on AISC-LRFD [1999] interaction equation is different for each case. It should be pointed out that the maximum CSI for case 1OR is nearly unity because the sizes of the structural elements were optimized by ETABS design module for the OMF.

The addition of shear wall or X-bracing reduces both bending moments and axial forces in the columns. On the other hand, the spandrel bracing tends to increase the column bending moments in the x and y-directions by 7% to 15% and 1% to 6%, respectively. However the maximum axial force and CSI values are generally lower (as much as 10%) than the values for the building with the conventional X-bracing.

### Table 3.8: Building Forces.

<table>
<thead>
<tr>
<th>System</th>
<th>Total axial force, P</th>
<th>Maximum story shear</th>
<th>Maximum torque</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(MN)</td>
<td>(kips)</td>
<td>(kN)</td>
</tr>
<tr>
<td>1OR</td>
<td>27.5</td>
<td>6,176</td>
<td>965</td>
</tr>
<tr>
<td>2SW</td>
<td>28.1</td>
<td>6,328</td>
<td>1,334</td>
</tr>
<tr>
<td>3XB</td>
<td>27.5</td>
<td>6,182</td>
<td>996</td>
</tr>
<tr>
<td>4SB1</td>
<td>24.3</td>
<td>5,462</td>
<td>974</td>
</tr>
<tr>
<td>5SB2</td>
<td>24.3</td>
<td>5,456</td>
<td>974</td>
</tr>
<tr>
<td>6SB3</td>
<td>24.3</td>
<td>5,456</td>
<td>974</td>
</tr>
<tr>
<td>7SB4</td>
<td>24.2</td>
<td>5,446</td>
<td>970</td>
</tr>
<tr>
<td>8SB5</td>
<td>24.2</td>
<td>5,448</td>
<td>970</td>
</tr>
<tr>
<td>9SB6</td>
<td>24.3</td>
<td>5,453</td>
<td>974</td>
</tr>
<tr>
<td>10SB7</td>
<td>24.2</td>
<td>5,446</td>
<td>979</td>
</tr>
</tbody>
</table>

### 3.3.4.5 Column Forces

As seen from Table 3.9, the maximum column axial forces follow the same trend as the building axial force. However, the Combined Stress Index (CSI) based on AISC-LRFD [1999] interaction equation is different for each case. It should be pointed out that the maximum CSI for case 1OR is nearly unity because the sizes of the structural elements were optimized by ETABS design module for the OMF.

The addition of shear wall or X-bracing reduces both bending moments and axial forces in the columns. On the other hand, the spandrel bracing tends to increase the column bending moments in the x and y-directions by 7% to 15% and 1% to 6%, respectively. However the maximum axial force and CSI values are generally lower (as much as 10%) than the values for the building with the conventional X-bracing.
3.3.4.6 Maximum Brace Forces

The maximum brace forces are summarized in Table 3.10. It should be pointed out that, with the exception of system 4SB1 that does not have X-bracing, the maximum brace force for building with spandrel bracing systems will occur in the braces at the first story. Therefore, there is a need to differentiate the maximum brace force for the 1st story from the other stories as shown in Table 3.10. The maximum tensile and compressive axial forces in the braces of each spandrel bracing system are lower than those found in the 3XB, indicating that a lighter (and more economical) section could be designed for the spandrel braces. This further supports the implementation of perimeter spandrel type bracing. For system 9SB6, both maximum tensile and compressive axial forces in the spandrel braces are more than 60% lower than that based on the conventional X-bracing.

Table 3.9: Column Forces.

<table>
<thead>
<tr>
<th>System</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axial Force, $P$</td>
</tr>
<tr>
<td></td>
<td>(kN)</td>
</tr>
<tr>
<td>1OR</td>
<td>1,926</td>
</tr>
<tr>
<td>2SW</td>
<td>1,922</td>
</tr>
<tr>
<td>3XB</td>
<td>1,926</td>
</tr>
<tr>
<td>4SB1</td>
<td>1,673</td>
</tr>
<tr>
<td>5SB2</td>
<td>1,673</td>
</tr>
<tr>
<td>6SB3</td>
<td>1,673</td>
</tr>
<tr>
<td>7SB4</td>
<td>1,673</td>
</tr>
<tr>
<td>8SB5</td>
<td>1,673</td>
</tr>
<tr>
<td>9SB6</td>
<td>1,673</td>
</tr>
<tr>
<td>10SB7</td>
<td>1,673</td>
</tr>
</tbody>
</table>

Note: Within each system, the maximum values may not occur simultaneously at the same column. The location of the maximum values also tends to vary from one system to another.
PART 2 – TWO-DIMENSIONAL DYNAMIC ANALYSIS

The purpose of this part of the analytical study is to investigate the effect of added one-third height spandrel bracing system and Energy Dissipation System (EDS) on the dynamic response of a typical 3-story shear frame subjected to an actual earthquake motion. In addition, the finite element program employed for the modeling and time history analysis of nonlinear hysteretic behavior of various types of EDS is evaluated.

3.4.1 Model Description

3.4.1.1 Shear Frame

As shown in Figure 3.5, the structure is a two-dimensional (global Z-X plane), 3-story shear steel frame. A single rigid diaphragm connects the ends of the beams at each story. Because of the rigid diaphragms, no axial force is present in the beam members. The columns are modeled as pure bending element and due to the rigid diaphragm, possess only translational degree-of-

<table>
<thead>
<tr>
<th>System</th>
<th>1st Story Tension</th>
<th>1st Story Compression</th>
<th>All other Story Tension</th>
<th>All other Story Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>3XB</td>
<td>193 (kN) 43.4 (kips)</td>
<td>238 (kN) 53.5 (kips)</td>
<td>193 (kN) 43.4 (kips)</td>
<td>238 (kN) 53.5 (kips)</td>
</tr>
<tr>
<td>4SB1</td>
<td>117 (kN) 26.2 (kips)</td>
<td>110 (kN) 24.7 (kips)</td>
<td>117 (kN) 26.2 (kips)</td>
<td>110 (kN) 24.7 (kips)</td>
</tr>
<tr>
<td>5SB2</td>
<td>214 (kN) 48.1 (kips)</td>
<td>241 (kN) 54.1 (kips)</td>
<td>68 (kN) 15.3 (kips)</td>
<td>70 (kN) 15.8 (kips)</td>
</tr>
<tr>
<td>6SB3</td>
<td>205 (kN) 46.1 (kips)</td>
<td>233 (kN) 52.3 (kips)</td>
<td>117 (kN) 26.2 (kips)</td>
<td>99 (kN) 22.2 (kips)</td>
</tr>
<tr>
<td>7SB4</td>
<td>208 (kN) 46.8 (kips)</td>
<td>235 (kN) 52.9 (kips)</td>
<td>138 (kN) 31 (kips)</td>
<td>122 (kN) 27.4 (kips)</td>
</tr>
<tr>
<td>8SB5</td>
<td>209 (kN) 46.9 (kips)</td>
<td>235 (kN) 52.9 (kips)</td>
<td>159 (kN) 35.8 (kips)</td>
<td>126 (kN) 28.4 (kips)</td>
</tr>
<tr>
<td>9SB6</td>
<td>210 (kN) 47.3 (kips)</td>
<td>237 (kN) 53.2 (kips)</td>
<td>67 (kN) 15 (kips)</td>
<td>77 (kN) 17.4 (kips)</td>
</tr>
<tr>
<td>10SB7</td>
<td>210 (kN) 47.1 (kips)</td>
<td>235 (kN) 52.9 (kips)</td>
<td>58 (kN) 13 (kips)</td>
<td>145 (kN) 32.7 (kips)</td>
</tr>
</tbody>
</table>
freedom at the beam-join connections. The model, labeled as SF, thus behaves as a shear frame with a single degree-of-freedom at each story. The ETABS properties of the framing elements are summarized in Table 3.11.

Lumped masses of 29.8 kN-sec²/m (0.17 kips-sec²/in.), 43.8 kN-sec²/m (0.25 kips-sec²/in.) and 43.8 kN-sec²/m (0.25 kips-sec²/in.), each representing the weight at each floor level, including tributary weight from beams and columns, are assigned to beam-column joints at the 1st, 2nd and 3rd story, respectively; the weight of the shear frame is 1151 kN (259 kips). In addition, small masses, 0.035 kN-sec²/m (2 x 10⁻⁴ kips-sec²/in.) are assigned to the EDS elements to help the nonlinear time history analyses solutions converge [CSI, 2002]. Various bracing and EDS configurations are incorporated into the shear frame and are discussed next.

![Three Story Shear Frame](image)

**Figure 3.5: Three Story Shear Frame.**

### 3.4.1.2 Supplemental Bracing and EDS

Diagonal braces connect the supplemental EDS to the shear frame. The braces were pin-connected to the shear frame and were made sufficiently stiff (about ten times more than the EDS) to ensure that deformation occurred only in the EDS and not in the braces. The EDS (e.g., viscous, yielding and friction) were incorporated at selected brace-beam joints. The different bracing configurations studied are shown in Figure 3.6
The chevron (or Inverted V) bracing represents one of the common bracing configurations for attaching EDS to a structural frame. Two proposed one-third height spandrel bracing configurations were studied and shown in Figure 3.6b and 3.6c). The proposed spandrel (single) diagonal bracing in Figure 3.6b is simple but it would result in uneven lateral load distribution for the pair of columns at each story. In the spandrel chevron configuration, a sufficiently stiff beam (or truss) element was (pinned) attached to the column at one-third height so that the lateral resistance from both columns was activated. The spandrel bracing systems investigated in Section 3.3 (Figure 3.2) are not suitable for EDS implementation because the configuration tends to limit the interstory drift significantly thus rendering the EDS ineffective.

The EDS elements were modeled in ETABS by assigning a panel zone to the brace-beam joint with nonlinear link properties (Figure 3.7) that effectively permit only relative shear deformation (Figure 3.7c) between the braces and diaphragm in the global horizontal direction (rather than along the axis of the braces). This configuration was chosen because, for a given interstory drift of a shear frame, the horizontal movement of the upper end of the brace would be greater than the deformation along the axis of the braces. The characteristics of the various EDS

---

**Table 3.11: Frame Element Properties.**

<table>
<thead>
<tr>
<th>Element</th>
<th>Properties</th>
<th>Symbol</th>
<th>Location</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam¹</td>
<td>Cross-sectional Area</td>
<td>A</td>
<td>All</td>
<td>6.5x10⁴ mm² (1x10² in.²)</td>
</tr>
<tr>
<td></td>
<td>Moment of Inertia</td>
<td>Iₓ</td>
<td>All</td>
<td>4.2x10¹¹ mm⁴ (1x10⁸ in.⁴)</td>
</tr>
<tr>
<td>Column²</td>
<td>Cross-sectional Area</td>
<td>A</td>
<td>All</td>
<td>6.5x10⁶ mm² (1x10⁴ in.²)</td>
</tr>
<tr>
<td></td>
<td>Moment of Inertia³</td>
<td>Iᵧ</td>
<td>1ˢᵗ Story</td>
<td>1.7x10⁶ mm⁴ (404.1 in.⁴)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2ⁿᵈ Story</td>
<td>1.3x10⁶ mm⁴ (323.3 in.⁴)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3ʳᵈ Story</td>
<td>1.0x10⁶ mm⁴ (242.5 in.⁴)</td>
</tr>
<tr>
<td>Braces</td>
<td>Cross-sectional Area</td>
<td>A</td>
<td>All</td>
<td>6.5x10⁴ mm² (1x10² in.²)</td>
</tr>
</tbody>
</table>

Notes:

¹ The beam section properties did not affect the results since they were attached to a rigid diaphragm.
² Unless specified otherwise, all other ETABS properties for the frame elements not shown in the above table were not used.

The calculated elastic stiffness (or spring constant) of each column was calculated (based on 12EI/L³) as 7.3 kN/mm (41.7 kips/in.), 5.8 kN/mm 33.3 kips/in. and 4.4 kN/mm (25 kips/in.) for the 1ˢᵗ, 2ⁿᵈ and 3ʳᵈ story, respectively. Material properties used: Modulus of elasticity, E = 200 GPa (29,000 kips/in.²), Poisson ratio, ν = 0.3.
modeled and the corresponding nonlinear link properties are discussed further in the following sections.

Figure 3.6: Supplemental Bracing and EDS Configurations.

Figure 3.7: Typical Panel Zone with Nonlinear Link Properties.


3.4.1.3 Fluid Viscous-based EDS

Fluid viscous damper elements of the type described in Hanson [1993] were used to connect the braces to the frame. ETABS nonlinear DAMPER property [CSI, 2002], that is based on Maxwell’s model of viscoelasticity, was specified for the links in the panel zone. To model pure damping behavior, the effect of the spring was made negligible by making it sufficiently stiff. ETABS [CSI, 2002] recommends that the spring stiffness be large enough so that the characteristic time (i.e., ratio of stiffness to damping coefficient) of the (Maxwell’s) spring-dashpot model (Figure 3.7a) is two orders of magnitude smaller than the size of the load steps. The details of the DAMPER property for the viscous dampers are summarized in Table 3.12.

Table 3.12: ETABS DAMPER Property for Viscous Dampers.

<table>
<thead>
<tr>
<th>Bracing Type</th>
<th>Story</th>
<th>Local Axis</th>
<th>Type</th>
<th>Properties</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB &amp; SC</td>
<td>All</td>
<td>1-1</td>
<td>Linear</td>
<td>Effective stiffness</td>
<td>$K_e$</td>
<td>$1.8 \times 10^6$ kN/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>($1 \times 10^7$ kips/in.)</td>
</tr>
<tr>
<td>CB &amp; SC</td>
<td>All</td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Damping exponent</td>
<td>$\alpha$</td>
<td>1.0</td>
</tr>
<tr>
<td>CB</td>
<td>1st</td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>174 kN/mm (995 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Damping coefficient</td>
<td>$C$</td>
<td>0.17 kN-sec/mm (0.995 kips-sec/in.)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>139 kN/mm (796 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Damping coefficient</td>
<td>$C$</td>
<td>0.14 kN-sec/mm (0.796 kips-sec/in.)</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>105 kN/mm (597 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Damping coefficient</td>
<td>$C$</td>
<td>0.10 kN-sec/mm (0.597 kips-sec/in.)</td>
</tr>
<tr>
<td>SC</td>
<td>All</td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>876 c kN/mm (5000 c kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Damping coefficient</td>
<td>$C$</td>
<td>Varies: 0.04 to 1.05 kN-sec/mm (0.25 to 6 kips-sec/in.)</td>
</tr>
</tbody>
</table>

1 All fluid viscous dampers are assumed to exhibit linear force-velocity relationship i.e., $F = CV^{1.0}$.
2 The nonlinear stiffness and damping values for the chevron braced shear frame were obtained from ETABS software verification Example 11 [CSI, 2003].
3 The damping coefficient dependent stiffness produced a characteristics time (of the Maxwell’s system) that was exactly two orders of magnitude smaller than the size of the time-step ($\Delta t = 0.02$ for all analysis).

Notes: Unless otherwise specified, zero values were specified for all other parameters for the DAMPER property not shown in the table.
3.4.1.4 Yielding-based EDS

Steel yielding elements that absorb energy through hysteresis (Bechtel X-shaped Added Damping and Stiffness (ADAS) steel plates as described in Scholl [1993] and Tsai, et al. [1993]) were used to connect the chevron braces to the frame. The yielding elements were modeled in the panel zone using ETABS PLASTIC1 property [CSI, 2002], that is based on the hysteretic uniaxial plasticity behavior proposed by Wen [1976] (Figure 3.8).

It should be pointed out that the present model is not able to model the fatigue characteristic of yielding damper. The details of the PLASTIC1 property for the yielding dampers are summarized in Table 3.13.

![Figure 3.8: Bouc Wen’s Plasticity Model.](image)

3.4.1.5 Friction-based EDS

Based on the approach of several researchers [Butterworth, 1999; Shao and Miyamoto, 1999], the Wen’s plasticity model appears to be appropriate to model friction EDS. A high elastic spring stiffness, yielding exponent and inelastic ratio were specified to generate the rectangular hysteresis loops characteristics of Coulomb friction. The slip load was specified as the yield strength in the Wen’s model. The details of the PLASTIC1 property for the friction dampers are summarized in Table 3.14.
Table 3.13: ETABS PLASTIC1 Property for Yielding Dampers.

<table>
<thead>
<tr>
<th>Bracing Type</th>
<th>Story</th>
<th>Local Axis</th>
<th>Type</th>
<th>Properties</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>All</td>
<td>1-1</td>
<td>Linear</td>
<td>Effective stiffness</td>
<td>$K_e$</td>
<td>$1.8 \times 10^6$ kN/mm (1x10^7 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Post yield stiffness ratio</td>
<td>$K_y$</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Yielding exponent</td>
<td>$\alpha$</td>
<td>2.0</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt;</td>
<td></td>
<td>2-2</td>
<td>Linear</td>
<td>Effective stiffness</td>
<td>$K_e$</td>
<td>14.6 kN/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>(83.3 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yield force</td>
<td>$F_y$</td>
<td>48.7 kN (10.9 kips)</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt;</td>
<td></td>
<td></td>
<td>Linear</td>
<td>Effective stiffness</td>
<td>$K_e$</td>
<td>11.7 kN/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>(66.6 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yield force</td>
<td>$F_y$</td>
<td>36.6 kN (8.2 kips)</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td></td>
<td></td>
<td>Linear</td>
<td>Effective stiffness</td>
<td>$K_e$</td>
<td>8.8 kN/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>(50.0 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yield force</td>
<td>$F_y$</td>
<td>19.2 kN (4.3 kips)</td>
</tr>
</tbody>
</table>

Notes: The nonlinear properties are obtained from ETABS software verification Example 10 [CSI, 2003]. Unless otherwise specified, zero values are specified for all other parameters for the PLASTIC1 property not shown in the table.

Table 3.14: ETABS PLASTIC1 Property for Friction Dampers.

<table>
<thead>
<tr>
<th>Bracing Type</th>
<th>Story</th>
<th>Local Axis</th>
<th>Type</th>
<th>Properties</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD &amp; SC</td>
<td>1&lt;sup&gt;st&lt;/sup&gt;, 2&lt;sup&gt;nd&lt;/sup&gt;</td>
<td>1-1</td>
<td>Linear</td>
<td>Effective stiffness</td>
<td>$K_e$</td>
<td>$1.8 \times 10^6$ kN/mm (1x10^7 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2-2</td>
<td>Nonlinear</td>
<td>Post yield stiffness ratio</td>
<td>$K_y$</td>
<td>$10^{-6}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yielding exponent</td>
<td>$\alpha$</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Linear</td>
<td>Effective stiffness</td>
<td>$K_e$</td>
<td>$1.8 \times 10^4$ kN/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nonlinear</td>
<td>Stiffness</td>
<td>$K$</td>
<td>(1x10^5 kips/in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yield strength</td>
<td>$F_y$</td>
<td>Varies: 44 – 1557 kN (10 – 350 kips)</td>
</tr>
</tbody>
</table>

Note: Unless otherwise specified, zero values are specified for all other parameters for the PLASTIC1 property not shown in the table.
3.4.1.6 Viscoelastic-based EDS

Viscoelastic EDS is not considered for the present study though they are expected to perform in a similar manner to velocity-dependent fluid viscous EDS except for the effect of added stiffness. The same ETABS nonlinear DAMPER property would be used to model the elliptical hysteretic behavior of viscoelastic EDS for future studies.

3.4.2 Analysis Procedures

The simple shear frame structure could be readily analyzed using LSP (method proposed by SEAOC [SEAOC, 1999]), LDP (either response spectrum or time-history) or NDP. As mentioned earlier, the need to model the nonlinear (hysteretic) behavior of EDS and the relatively short analysis time (less than a minute for the problem at hand on a INTEL Pentium IV 2.4GHz processor with 512MB memory) made the nonlinear time history analysis a more appropriate and attractive choice for the feasibility study.

3.4.2.1 Modal Analysis

As strongly recommended by ETABS [CSI, 2002], all nonlinear time history analyses were performed using Ritz vectors, instead of basing on natural mode shapes from eigenvector analysis. It has been demonstrated [Wilson et al., 1982] that dynamic analyses based on load-dependent Ritz vectors yield more accurate results than the use of the same number of natural mode shapes. Ritz vectors yield excellent results because they are generated by taking into account the spatial distribution of the dynamic loading, whereas the direct use of the natural mode shapes neglects this very important information [CSI, 2002].

The ground acceleration (in the global X-axis) and all link element nonlinear degrees of freedom were used to generate the Ritz vectors. A total of 20 modes were requested for each analysis (though ETABS automatically reduces to the applicable number of modes).
3.4.2.2 Earthquake Record

The unscaled N-S component (El Centro station) of the 1940, May 19 Imperial Valley earthquake was used for all time history analysis (Figure 3.9). The digitized accelerogram includes 960 data points at equal time intervals of 0.02 sec; a total duration of 19.2 sec. The peak ground acceleration (PGA) is 0.35g at 2.12 sec. The frequency content is strongest between 1 Hz (1 sec) to 2 Hz (0.5 sec), and most of the energy is concentrated between 1.5 and 5.5 sec into the signal. Only one earthquake motion was used for the preliminary study. Earthquake motions with different frequency content and energy content would be used for future analytical studies.

The excitation was applied to the shear frame in the global X-direction. It should be pointed out that additional modal damping was not justified for the analysis cases that included dampers since the actual energy dissipating mechanism (via the dampers) were modeled [Wilson et al., 2002]. Table 3.15 summarizes the analysis cases that were performed as part of the analytical study. The case CBYDHIST and CBVDHIST are identical to ETABS software verification Examples 10 and 11 [CSI, 2003], respectively. They have been included for comparison purposes.

3.4.3 Results and Discussions

3.4.3.1 Modal Analysis

The natural periods for the analysis cases are summarized in Table 3.16. Generally, the supplemental bracing increases the frame stiffness and thus reducing the period for all modes. As evident from the larger period, the one-third height spandrel bracing (cases SDFDHIST and SCFDHIST) is less stiff than the conventional full-height chevron (in CBFDHIST).

For case CBVDHIST, the natural periods agreed almost exactly with the values for the unbraced shear frame (SFMODAL) because pure viscous damping was assumed in the finite element model (linear effective stiffness in local 2-2 axis of the Maxwell’s viscoelastic model is zero for both cases) and that the full-height bracing did not affect the mode shapes of the original unbraced frame. However, the natural modes (and frequencies) of case with spandrel chevron bracing (SCVDHIST) are lower than the unbraced frame due to the added first-story stiffness from chevron bracing, and more importantly the change in fundamental modal shapes due to one-
third height spandrel bracing. It should be noted that the frequencies of the fundamental modes for all cases are close to the spectral peak of the earthquake motion and hence represents severe excitation.

Table 3.15: Analysis Cases.

<table>
<thead>
<tr>
<th>No.</th>
<th>Analysis Case Label</th>
<th>Bracing Type</th>
<th>Dampers Used</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>SFMODAL0 – SFMODAL40</td>
<td>None</td>
<td>None</td>
<td>Modal analysis cases for ritz vectors. The number on the label indicates the modal damping ratio used. The same damping ratio was applied to all modes and the ratios used are 0%, 1%, 5%, 10%, 20%, 30% and 40%.</td>
</tr>
<tr>
<td>2.</td>
<td>CBVDHIST</td>
<td>CB</td>
<td>Viscous</td>
<td>Nonlinear time history analysis. Modal analysis was automatically performed by ETABS to obtain the Ritz vectors. No modal damping was applied. This case is identical to ETABS software verification Example 11 [CSI, 2003].</td>
</tr>
<tr>
<td>3.</td>
<td>CBYDHIST</td>
<td>CB</td>
<td>Yielding</td>
<td>Nonlinear time history analysis. Modal analysis was automatically performed by ETABS to obtain the Ritz vectors. No modal damping was applied. This case is identical to ETABS software verification Example 10 [Computers and Structures, Inc, 2003].</td>
</tr>
<tr>
<td>4.</td>
<td>SDFDHIST10 – SDFDHIST200</td>
<td>SD</td>
<td>Friction</td>
<td>Nonlinear time history analyses. Modal analysis was automatically performed by ETABS to obtain the Ritz vectors. No modal damping was applied. The number on the label indicates the slip load (in kips) for the friction dampers. Constant slip load was used at all stories.</td>
</tr>
<tr>
<td>5.</td>
<td>SCVDHIST0 – SCVDHIST6</td>
<td>SC</td>
<td>Viscous</td>
<td>Nonlinear time history analyses. Modal analysis was automatically performed by ETABS to obtain the Ritz vectors. No modal damping was applied. The number on the label indicates the pure fluid viscous damping (in kips-sec/in.) for the viscous dampers.</td>
</tr>
<tr>
<td>6.</td>
<td>SCFDHIST10 – SCFDHIST200</td>
<td>SC</td>
<td>Friction</td>
<td>Nonlinear time history analyses. Modal analysis was automatically performed by ETABS to obtain the Ritz vectors. No modal damping was applied. The number on the label indicates the slip load (in kips) for the friction dampers. Constant slip load was used at all stories.</td>
</tr>
</tbody>
</table>
The use of classical damping or linear modal (viscous) damping, as a percentage of critical damping, has often been used to approximate the nonlinear behavior of structure. The equivalent modal damping has little theoretical or experimental justification [Wilson, 2002]. However, it is a convenient method to quantify viscous energy dissipation in real structures and it greatly facilitates dynamic analysis. The following discusses the effect of modal damping on the response of the 3-story shear frame and the results would be useful for quantifying, in terms of modal damping, the amount of energy dissipation provided by the various supplemental EDS.

Table 3.16: Natural Periods.

<table>
<thead>
<tr>
<th>Analysis Case</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>Mode 4</th>
<th>Mode 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFMODAL</td>
<td>0.74066</td>
<td>0.29638</td>
<td>0.21317</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CBYDHIST</td>
<td>0.52624</td>
<td>0.21058</td>
<td>0.15143</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CBVDHIST</td>
<td>0.74064</td>
<td>0.29634</td>
<td>0.21307</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SDFDHIST(^1,2)</td>
<td>0.42216</td>
<td>0.18644</td>
<td>0.00163</td>
<td>0.0016</td>
<td>0.0016</td>
</tr>
<tr>
<td>SCVDHIST</td>
<td>0.46820</td>
<td>0.22240</td>
<td>0.00271</td>
<td>0.0016</td>
<td>0.0016</td>
</tr>
<tr>
<td>SCFDHIST(^1)</td>
<td>0.35711</td>
<td>0.15731</td>
<td>0.00271</td>
<td>0.0016</td>
<td>0.0016</td>
</tr>
</tbody>
</table>

1 For each case, the natural frequencies are not affected by changes in slip load since the same friction damper effective stiffness (10\(^5\) Kips/in) is used for the linear modal analysis.

2 The two additional modes account for the deformation of the braced column.

3.4.3.2 Effect of Modal Damping

The use of classical damping or linear modal (viscous) damping, as a percentage of critical damping, has often been used to approximate the nonlinear behavior of structure. The equivalent modal damping has little theoretical or experimental justification [Wilson, 2002]. However, it is a convenient method to quantify viscous energy dissipation in real structures and it greatly facilitates dynamic analysis. The following discusses the effect of modal damping on the response of the 3-story shear frame and the results would be useful for quantifying, in terms of modal damping, the amount of energy dissipation provided by the various supplemental EDS.

The effect of modal damping on the maximum story displacement envelopes, normalized (with respect to weight of the shear frame) maximum story shear envelopes and normalized (with respect to peak ground acceleration) maximum story accelerations for the unbraced shear frame (SFOMODAL cases) are shown in Figures 3.9, 3.10, and 3.11, respectively. The same modal damping is applied to all (three) modes. Figures 3.12, 3.13 and 3.14 show the influence of modal damping on the maximum story displacement, base shear factor (i.e., base shear over weight of frame) and normalized (with respect to the peak ground acceleration) floor accelerations of the shear frame. The above plots could be used to evaluate the amount of equivalent modal damping.
present in the shear frame with various configurations of supplemental EDS and bracing system. The maximum response values for the undamped and unbraced shear frame are summarized in Table 3.17.

![Graph of Maximum Story Displacement Envelope with Modal Damping](image1)

**Figure 3.9:** Variation of Maximum Story Displacement Envelope with Modal Damping.

![Graph of Normalized Maximum Story Shear Envelope with Modal Damping](image2)

**Weight of shear frame = 1,151 kN (258.8 kips)**

![Graph of Normalized Maximum Story Shear Envelope with Modal Damping](image3)

**Figure 3.10:** Variation of Normalized Maximum Story Shear Envelope with Modal Damping.
Figure 3.11: Variation of Normalized Maximum Story Acceleration Envelope with Modal Damping.

Figure 3.12: Variation of Maximum Story Displacement with Modal Damping.
Figure 3.13: Variation of Normalized Base Shear with Modal Damping.

Figure 3.14: Variation of Normalized Story Acceleration with Modal Damping.
As indicated earlier in Table 3.15, cases CBYDHIST and CBVDHIST are examples from ETABS software verification examples [CSI, 2003] and represent the typical implementation strategy for yielding and viscous EDS in a multi-story building frame. The results from these two cases are used to evaluate the performance of the proposed spandrel bracing and dampers system. ETABS [CSI, 2003] reported that the supplemental EDS produce an equivalent modal damping of 5% in the fundamental mode.

The maximum story displacement envelopes and maximum story shear envelopes are shown in Figure 3.15 and Figure 3.16, respectively. The displacement envelopes indicate that equivalent modal damping ratios of 5% and 10% of critical could be achieved through the use of the supplemental EDS with yielding and viscous dampers, respectively. Based on story shear (Figure 3.16) and story acceleration (Figure 3.17) criteria, however, the equivalent damping ratios appeared to be lower, about 1% and 5% of critical, respectively. These observations indicate the difficulty in establishing an equivalent modal damping for system under consideration. Better correlation may be possible using different damping ratio for each mode.

### Table 3.17: Maximum Response Values of Unbraced Undamped Shear Frame.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1&lt;sup&gt;st&lt;/sup&gt;</th>
<th>2&lt;sup&gt;nd&lt;/sup&gt;</th>
<th>3&lt;sup&gt;rd&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum displacement</td>
<td>55.6 mm (2.20 in.)</td>
<td>102.3 mm (4.03 in.)</td>
<td>140.7 mm (5.54 in.)</td>
</tr>
<tr>
<td>Maximum normalized story factor</td>
<td>18.3 mm (0.72 in.)</td>
<td>14.0 mm (0.55 in.)</td>
<td>9.4 mm (0.37 in.)</td>
</tr>
<tr>
<td>Maximum story acceleration</td>
<td>72.6 mm (2.86 in.)</td>
<td>75.9 mm (2.99 in.)</td>
<td>109.7 mm (4.32 in.)</td>
</tr>
</tbody>
</table>

### 3.4.3.3 Performance of Viscous and Yielding EDS with Chevron Bracing (CB)

As indicated earlier in Table 3.15, cases CBYDHIST and CBVDHIST are examples from ETABS software verification examples [CSI, 2003] and represent the typical implementation strategy for yielding and viscous EDS in a multi-story building frame. The results from these two cases are used to evaluate the performance of the proposed spandrel bracing and dampers system. ETABS [CSI, 2003] reported that the supplemental EDS produce an equivalent modal damping of 5% in the fundamental mode.
Figure 3.15: Maximum Story Displacement Envelopes for Chevron Braced Frames.

Figure 3.16: Normalized Maximum Story Shears Envelopes for Chevron Braced Frames.
The hysteresis loops of the modeled fluid viscous dampers for case CBVDHIST are presented in Figure 3.18. The zero slopes of the major axis of the elliptical-shaped loops indicate that the damper is approximating pure viscous behavior with negligible stiffness. The maximum forces developed in the dampers are 48 kN (10.8 kips), 48 kN (10.8 kips) and 22 kN (5.0 kips), and the maximum shear deformations are 33 mm (1.3 in.), 38 mm (1.5 in.) and 28 mm (1.1 in.) for the 1st, 2nd and 3rd story, respectively. Substantial energy dissipation has occurred in the 1st and 2nd story dampers as evident from the relative areas enclosed by the loops.

Figure 3.19 shows the hysteresis loops of the yielding (ADAS) dampers for case CBYDHISt. The maximum forces developed in the dampers are 76 kN (17.0 kips), 62 kN (14.0 kips) and 32 kN (7.3 kips) and the maximum shear deformations are 41 mm (1.6 in.), 46 mm (1.8 in.) and 33 mm (1.3 in.) for the 1st, 2nd and 3rd story, respectively. It should be noted that degeneration of hysteresis looping that is characteristic of yielding dampers, cannot be modeled by the ETABS PLASTIC1 link property at the present moment.
Figure 3.18: Hysteresis Lops for Case CBVDHIST.

Note:
1 kips = 4.448 kN
1 in. = 25.4 mm
$K = 14.6 \text{ kN/mm (83.3 kips/in.)}$
$F_y = 48.7 \text{ kN (10.94 Kips)}$

$K = 11.7 \text{ kN/mm (66.6 kips/in.)}$
$F_y = 36.6 \text{ kN (8.2 Kips)}$

$K = 8.8 \text{ kN/mm (50.0 kips/in.)}$
$F_y = 19.2 \text{ kN (4.3 kips)}$

Figure 3.19: Hysteresis Loops for Case CBYDHIST.
The energy time history profile of the structure is a very useful measure of the effectiveness of the supplemental EDS. Figure 3.20 presents the time variation of the total input energy, nonlinear damping energy and kinetic energy (potential energy not shown for better clarity) of the structure throughout the duration of the earthquake motion. It should be pointed out that the input energy is not the ground energy that is imparted to the structure; it represents the total internal energy (which is the sum of potential, kinetic and damping energies) produced in the structure as a result of its response to the ground excitation and is given by,

\[ I.E. = \int_{\text{Over full structure}} F(t)x(t)dt = \int_{\text{Over full structure}} m\ddot{x}(t)x(t)dt \]  

(3.1)

where \( I.E. \) is the input energy, \( F(t) \) is the inertia force and \( m, x(t) \) and \( \dot{x}(t) \) are the mass, displacement and acceleration, respectively, associated to each degree of freedom defined for the structure. The goal in vibration control is to increase the dissipated energy, so that for a given input energy, the potential (or elastic strain) energy in the structure is minimized.

Figure 3.21 shows the energy time history for case CBYDHIST. The energy profiles are similar to the case with viscous dampers. Again significant amount of energy is dissipated by the hysteretic yielding dampers as evident by the small difference between the input energy and dissipated energy histories. By differentiating the energy time history, the power time history is obtained which may be useful for evaluating the effectiveness of the dampers throughout the duration of the earthquake motion.

From the above dynamic analyses, it was evident that the supplemental EDS’s are effective in reducing the dynamic response of the shear frame to the earthquake excitation. These EDS are installed to the frame using the conventional (full-height) chevron bracing configuration. The following section covers the performance of supplemental EDS with the proposed one-third height spandrel bracing.
Note: Potential energy not shown

Figure 3.20: Energy Time History for Case CBVDHIST.

Note: Potential energy not shown

Figure 3.21: Energy Time History for Case CBYDHIST.

Note:
1 kips-in = 1.352 kN-m
The variation of maximum story displacements with slip load is shown in Figure 3.22. The first story displacements are negligible due to the chevron bracing and thus are not shown. Generally, the third story displacement is higher than those for the 2nd story. The horizontal dashed lines, representing the response of the undamped and unbraced shear frame (i.e., SFMODAL0), have been included for comparison purposes. It is shown that the combined supplemental diagonal bracing and EDS have beneficial vibration control characteristics. For the spandrel diagonal braced configuration, there exists an optimum slip load, between 222 kN (50 kips) to 267 kN (60 kips), that would result in minimum floor displacements. The reason for this is that at relatively low slip load, the hysteresis looping, hence energy dissipation and damping provided by the friction damper would be low. On the other hand, a relatively high slip load would increase the frame stiffness significantly and thus limit the interstory drift. Again the hysteresis looping (hence damping) would be low.

The presence of an optimum slip load (and minimum floor displacement) is not clear with the spandrel chevron braced configuration. Figure 3.23 shows the influence of normalized base shear with slip load. The minimum base shear factors are about 0.45 and 0.40 for the SDFDHIST and SCFDHIST cases, respectively. With the spandrel chevron bracing, varying the slip loads between 334 kN (75 kips) and 667 kN (150 kips) did not significantly change the base shear. Compared with the case of the unbraced and undamped shear frame (SFMODAL0), the friction damped shear frame generally has lower base shear.

The variation of normalized maximum story acceleration with slip load (Figures 3.24 and 3.25) is characterized by some exceptionally high peaks. Generally, the maximum story accelerations are high. The origin of the high peaks were not well understood though the acceleration time histories indicated that these high peak accelerations are isolated to only one or at most three cycles. These peaks may be due to the resonance effects as the dominating frequencies of the frame approach the frequency content of the ground motion. This may indicate a limitation of the spandrel bracing configuration for short buildings. In contrast to maximum displacement, the 2nd story maximum accelerations are generally higher than the 3rd story. The 2nd story acceleration for the unbraced and undamped shear frame (horizontal dashed lines) is generally lower than those for the friction damped shear frame, except at slip loads between 178 kN (40 kips) and 267 kN (60 kips).
Figure 3.22: Variation of Maximum Story Displacement with Slip Load for Cases with Spandrel Bracing.

Figure 3.23: Variation of Normalized Base Shear with Slip Load for Cases with Spandrel Bracing.

Note: 
1 kips = 4.448 kN
1 in. = 25.4 mm

Weight of shear frame = 1 151 kN (258.8 kips)
Figure 3.24: Variation of Normalized Maximum Story Acceleration with Slip Load for Cases with Spandrel Diagonal Bracing.

Figure 3.25: Variation of Normalized Maximum Story Acceleration with Slip Load for Cases with Spandrel Chevron Bracing.
The maximum story displacement, maximum story shear and maximum story acceleration envelopes, for three cases of the spandrel diagonal bracing configuration, are shown in Figures 3.26 through 3.28. It is evident that it is difficult to quantify the performance of the friction damped structure in terms of equivalent modal damping.

The above analyses show that friction dampers may be incorporated into the proposed spandrel bracing configurations for earthquake vibration control. Due to the smaller displacement at one-third height configuration, the EDS, though effective in energy dissipation and vibration control, may not be as effective as that compared to using conventional bracing. However, the system proposed in this study comprises of many of such spandrel type bracing (with or without supplemental EDS) incorporated into the architectural cladding system, and distributed uniformly around the building perimeter.

Figure 3.29 shows the typical rectangular hysteresis loops of the simulated friction damper (at the 3rd story) for three different slip loads. The friction damper at the 1st story for all SDFD cases did not participate in energy dissipation probably due to the presence of the chevron bracing between the base and the 1st story. The Wen’s plasticity model appears to be appropriate for modeling the hysteretic behavior of the friction damper. The maximum inelastic deformation (i.e., maximum slip) reduces with increasing slip load. Here, only the analysis cases with the diagonal bracing are reported. The other friction damped cases with spandrel bracing have similar characteristics.

![Figure 3.26: Maximum Story Displacement Envelopes for Spandrel Diagonal Braced Cases.](image)
Figure 3.27: Normalized Maximum Story Shear Envelopes for Spandrel Diagonal Braced Frames.

Figure 3.28: Normalized Maximum Story Acceleration Envelopes for Spandrel Diagonal Braced Frames.
To evaluate the performance of the friction dampers over the entire duration of the earthquake motion, the damper force time history is normalized with respect to the slip load and plotted in Figure 3.30 for two slip loads. A less than unity for the peak at each cycle indicates that no inelastic deformation and hence energy dissipation has occurred in the dampers. The friction damper with the higher slip load (Figure 3.30b) is much less effective over the entire duration of the earthquake motion. In particular, energy dissipation is negligible between 5 and 10 sec into the excitation and the braced shear frame is undergoing undamped forced vibration.

Figures 3.31 through 3.33 show the effect of slip load on the input, kinetic and dissipated energy time histories of the structure. As the slip load increases from 89 kN (20 kips) to 245 kN (55 kips), more energy is dissipated by the friction dampers and the elastic potential energy of the structure decreases. For the case with a slip load of 534 kN (120 kips), the energy dissipated in the dampers did not increase between 5 and 10 sec, indicating that no energy dissipation has occurred during this period. This is consistent with earlier observations on the normalized friction force time history.

Figure 3.29: Typical Hysteresis Loops for Case SDFDHIST.
Figure 3.30: Normalized Friction Damper Forces Time History.
Figure 3.31: Energy Time History for Case SDFD20.

Figure 3.32: Energy Time History for Case SDFD55.
As the slip is increased, the total input energy of the system decreases as the frame deformation decreases from the higher damper-bracing stiffness. However, a reduction in input energy does not necessarily indicate better EDS performance. In the above cases, it merely indicates that increasing slip load increases the stiffness of the shear frame and reduces the deformation of the structure, thus reducing the potential and kinetic energies. It may be more meaningful to compare the energy profiles that are normalized with respect to the input energy. The profile will indicate the percentage of input energy that is removed from the shear frame at any time step.

Figure 3.34 shows the normalized link energy (i.e., energy dissipated by the friction damper) time histories for the three cases. The total dissipated energy in the dampers, at each time step, is divided by the input energy at the same time step. The lower slip loads, 89 kN (20 kips) to 245 kN (55 kips), seem to be more effective in energy dissipation. The average energy dissipation over the entire 19.2 sec of the excitation are 76%, 73% and 48% for 89 kN (20 kips), 245 kN (55 kips) and 534 kN (120 kips), respectively. The normalized time histories also indicate that noticeable energy dissipation only commences at about 1 sec, 1.8 sec and 2.8 sec into the excitation, for slip loads of 89 kN (20 kips), 245 kN (55 kips) and 534 kN (120 kips), respectively. This is expected since the displacement-dependent friction dampers would not slip (i.e., undergo inelastic deformation) at relatively low building displacement, in particular at the beginning of the ground motion.
3.4.3.4 Performance of Fluid Viscous Dampers with Spandrel Chevron Bracing (SC)

The fluid viscous dampers are attached to the shear frame with spandrel chevron bracing at the 1st and 2nd stories. Figure 3.35 shows that as the viscous damping coefficient increases, the maximum story displacements, for both 3rd and 2nd stories, are reduced. However, the damping coefficient must exceed about 350 kN-sec/m (2 kips-sec/in.) to reduce the displacement results below that of the unbraced, undamped shear frame case (i.e., SDMODAL0). The influence of the viscous damping coefficient on normalized base shear is shown in Figure 3.36. Similar to the maximum displacement envelope, the base shear reduces with damping coefficient though damping must now exceed about 525 kN-sec/m (3 kips-sec/in.) to reduce the base shear below that of basic shear frame. Figure 3.37 represents the variation of acceleration normalized maximum story acceleration with slip load. Except for the 1st story, damping must exceed about 525 kN-sec/m (3 kips-sec/in.) before any noticeable reduction in maximum floor accelerations could be achieved for the higher stories.

The maximum story displacement, maximum story shear and maximum story acceleration envelopes for three cases with different damping coefficients are shown in Figures 3.38 through 3.40. It can be seen that the profile of these envelopes do not match with
any of those obtained for the unbraced shear frame with linear modal damping (i.e., SDMODAL). This may indicate difficulty in establishing an equivalent modal damping for the proposed EDS system for the 3-story shear frame.

**Figure 3.35**: Variation of Maximum Story Displacement with Damping for Cases with Spandrel Chevron Bracing.

**Figure 3.36**: Variation of Normalized Maximum Base Shear Factor with Damping for Cases with Spandrel Chevron Bracing.
Figure 3.37: Variation of Normalized Maximum Story Acceleration with Damping for Cases with Spandrel Chevron Bracing.

Figure 3.38: Maximum Story Displacement Envelopes for Spandrel Chevron Braced Cases.
Figure 3.39: Normalized Maximum Story Shear Envelopes for Spandrel Chevron Braced Cases.

Figure 3.40: Normalized Maximum Story Acceleration Envelopes for Spandrel Chevron Braced Cases.

Figure 3.41 shows the typical hysteresis loops of the modeled fluid viscous dampers for one of the cases with a damping coefficient of 350 kN⋅sec/m (2 kips⋅sec/in.). Figure 3.42 is a plot of the normalized link energy (i.e., energy dissipated by the viscous damper) time histories for three different viscous damping coefficients. As expected, energy dissipation increases with damping coefficient. The average energy dissipation over the entire 19.2 sec of the excitation are
37%, 68% and 82% for 88 kN-sec/m (0.5 kips-sec/in.), 350 kN-sec/m (2 kips-sec/in.) and 1051 kN-sec/m (6 kips-sec/in.), respectively.

Figure 3.41: Hysteresis Loops for Case CBVDHIST2.
3.5 SUMMARY

Two analytical studies were carried out to investigate the effect of perimeter spandrel bracing and supplement energy dissipating system on the static and dynamic behavior of a multi-story building and frame. In the first part of the study, the stiffness of one-third height perimeter spandrel bracing was compared against conventional story-high LFRS’s. A total of seven different spandrel type bracing systems and two conventional LFRS’s were added separately onto a three-dimensional, 4-story ordinary moment resisting frame building. The building was subjected to the same equivalent static lateral load. The analytical results indicate that perimeter spandrel bracing system can effectively provide a lateral stiffness that is close to that offered by conventional X-bracing LFRS. The results also show that despite the lateral stiffness of the individual one-third height spandrel bracing being lower than that of the full-height X-bracing, perimeter spandrel bracing system can provide a lateral stiffness comparable to conventional X-bracing system by virtue of its numbers. The perimeter spandrel bracing system also has an advantage of reducing the building axial forces and building torque. The spandrel bracing system can effectively reduce the column axial forces due to better distribution of building axial load over the larger number of columns. Because the spandrel bracing is connected at one-third height of the column, there was concern that high moments in column may occur at the bracing-column...
column connections. However, the results did not reveal significant increase in the column moment. Furthermore, the CSI for the building with spandrel bracing system was comparable to the building with conventional X-bracing. In the second study, the effect of added spandrel bracing system and energy dissipation system on the dynamic response of a 3-story shear frame subjected to an actual earthquake motion was investigated. The time history analyses show that it is possible to model the nonlinear behavior of velocity-dependent and displacement-dependent EDS using finite element software ETABS (nonlinear version). The supplemental EDS with the proposed one-third height spandrel bracing has beneficial vibration control characteristics for the 3-story shear frame considered. However, the suitability of the different dampers depends on the bracing configurations used. The preliminary results indicate that supplemental friction dampers could significantly reduce the interstory drift and base shear though it could also lead to higher story acceleration. On the other hand, it was found that a large damping coefficient must be specified for fluid-viscous damper due to small interstory drift at one-third story height. It was also found that there was difficulty in quantifying the amount of energy dissipation provided by the nonlinear EDS using equivalent modal damping, though better correlation may still be possible if the modal damping was specified for each mode of vibration.

3.6 CONCLUSION

The linear static analysis of a 3-D, 4-story ordinary moment frame indicated that despite the lower lateral stiffness of the individual partial height spandrel bracing (as compared to full-height X-bracing), the perimeter spandrel bracing system can provide a lateral stiffness comparable to conventional X-bracing system by virtue of its numbers and its location in the building perimeter. It was therefore concluded that the EDCS, designed as a spandrel type structural brace, would be effective as a structural brace. The nonlinear time history analyses of a 2-D, 3-story shear frame indicated that due to small interstory drift at one-third story height and space limitation, the viscous damper may not be suitable for the spandrel type bracing. Furthermore, the proprietary viscous damper was expected to come with a cost premium. On the other hand, viscoelastic dampers are more suited for wind application [Hanson and Soong, 2001] while yielding-based dampers are expected to be replaced after the design earthquake event. It was concluded that the friction-based damper would be most appropriate EDS for the EDCS.
CHAPTER 4

COMPONENT LEVEL STUDY – EDCS CONCEPT AND DESIGN

4.1 CHAPTER OVERVIEW

Partial height spandrel type bracing is a departure from conventional full-height bracing systems (e.g., concentrically-braced, eccentrically-braced). The analytical results presented in the previous chapter have shown that such bracing system on a moment resisting frame can be an effective LFRS system by virtue of its numbers and its location in the building envelope. The preliminary studies have also shown that friction damper would be appropriate for the spandrel type bracing system to provide additional form of energy dissipation. The next logical step would be to develop the concept and design of the EDCS unit that would meet the dual function of a structural brace and a mechanism of energy dissipation. The objective of the present chapter is to develop the concepts of the EDCS leading to a practical design that would achieve the above-mentioned functions.

The chapter begins with a discussion of the working concepts of the EDCS. Two practical schemes that would achieve the intended functions are presented and compared, finally leading to the selection of one of the approaches. The various components of the EDCS and their functions are discussed next. Finally, the design of the individual component is presented.

4.2 DESIGN CONCEPTS

The design of the EDCS followed as close as possible to that of conventional precast concrete cladding system so as to improve constructability and to minimize production cost. A brief discussion of the connections details used in conventional spandrel type bracing is presented next.

A typical spandrel type cladding panel has two bearing connections for supporting its own weight and at least two flexible tie-back connections to transfer out-of-plane loads while accommodating the in-plane movement; the use of four tie-backs in the field are also common to
facilitate installation. As discussed in Chapter 2, the conventional earthquake-resistant design of precast concrete panel suggested by PCI [1989] is directed at isolating the cladding panel from the structural frame thereby minimizing damage resulted from cladding-structure interaction. Spandrel cladding panel with tie-back connections attached to the same floor beam as bearing connections is not affected by story drift since the entire set of connections move together with the floor beam. For spandrel panel with tie-back connections attached to the columns, the flexible tie-back connections provide the necessary isolation and prevent the seismic lateral force from transmitting from structure to the panel. In effect, the panel can be designed as a non-structural element and it is not necessary to consider the panel interaction in the dynamic response of the structure (but it should be remembered that the heavy panels have a contribution to the effective seismic weight of the building). Other researchers [Craig et al., 1992; Pinelly et al., 1993; Goodno and Craig, 1998] have designed more advanced type of connections that behave as hysteretic energy dissipation mechanisms.

In order for the EDCS to function both as a structural brace and a source of energy dissipation, the following factors must be considered:

1. The EDCS must be capable of transferring the developed in-plane force in an elastic manner without deterioration of strength or stiffness under cyclic loading.
2. Based on the analytical studies in the previous chapter, friction-based energy dissipating system was found be the most appropriate for the EDCS. The device must be incorporated into the bracing system of the EDCS.
3. At least one of the support connections of the EDCS must be restrained in both vertical and lateral directions to transfer the load from the supporting beam to the precast concrete panel. The other bearing connections must be capable of resisting uplift due to the action of the lateral load (refer to Figure 4.15 on source of uplift). And, they must be detailed in such a manner that volume changes in the panel are not restricted.
4. While tie-back connections can still be used to transfer out-of-plane loads, some form of connection must be provided that could transfer the high in-plane force from the column to the EDCS.

With these considerations in mind, two possible concepts (A and B) were developed and presented next.
4.2.1 Concept A

For concept A, a diagonal brace with a friction damper is mounted inside the cladding panel as shown in Figure 4.1. In this configuration, both ends of the brace are pin connected to the beams and columns to allow free rotation of the brace due to the lateral movement. At one-third height, the horizontal drift would be less than 30% (assuming columns deflected in double-curvature) of the interstory drift. Because the friction damper is inclined, the horizontal drift would translate to even smaller deformation. The friction damper was chosen to overcome this limitation since it works well even at very low displacement. For low intensity earthquake motions, the force \( F \) developed in the brace would be well below the design slip force \( F_s \) of the friction damper. In this situation, the steel brace would function as a simple brace between the column and beam. Under moderate or high intensity seismic motions, the force in the brace would reach the slip load, thereby activating the friction damper.

Figure 4.2 illustrates a possible design based on the above concept. The connection details are shown in Figure 4.3. A bracing subsystem, comprising of a steel brace, a friction damper and bearing supports, are added to a conventional precast panel. The concrete panel itself is isolated from the bracing system to prevent damage as a result of the action of the brace and the friction damper. In other words, the EDCS, as a whole system, behaves as a structural brace but the concrete panel does not interact with the structure. Since the load path is from the top connection to the brace to the bearing connections without going through the concrete panel. The bracing subsystem can be exposed or embedded inside the panel. If the bracing subsystem is embedded, adequate clearance must be provided to allow unrestricted movement of the brace and friction damper inside the panel. The design and production of a hollow precast panel is expected to be complex and costly. Furthermore, embedding the bracing and friction damper may pose a problem when it comes to (regular) inspection or when replacement is required. Special openings must then be incorporated onto the panel for such purposes.
Figure 4.1: EDCS Design Concept A.
Figure 4.2: Possible EDCS Configuration Based on Design Concept A.
Figure 4.3: Connection Details Based on Design Concept A.
4.2.2 Concept B

Instead of adding a bracing subsystem, Concept B proposes that the precast concrete panel of the EDCS be designed to carry the required lateral load as shown in Figure 4.4. Instead of embedding the friction damper inside the panel, it is installed on the concrete panel and is designed as part of the connection between column and the top of the cladding panel. Furthermore, the friction damper is designed to slip horizontally, thus maximizing the drift effect. Since the friction damper is not embedded and is installed onto the concrete panel, inspection or replacement of the damper should be relatively simple. The precast concrete panel would function as a rigid brace during low intensity earthquake motions. Under moderate or high intensity seismic motions, the force developed in the cladding panel would reach the slip load of the friction damper, causing it to slip and dissipate part of the building input energy. In this regard, the friction damper has an important function of limiting the maximum load that can be transferred to the concrete panel.

A possible design based on the Concept B is shown in Figure 4.5 with the details shown in Figure 4.6. As mentioned in the design considerations of the EDCS, one of the support bearing connections is bolted (and welded) to supporting beam while the other bearing connection is restrained from uplift but is designed (through the use of slotted connection) to translate laterally to accommodate volume changes in the panel. The friction damper is incorporated as part of the connection between the column and the cladding panel.

Concept B was selected because it maximizes the cladding-structure interaction. For Concept A, the concrete panel is effectively isolated from the bracing subsystem. While Concept A requires a separate bracing subsystem, the concrete panel is designed to function as the structural brace for Concept B. By eliminating the need for separate bracing subsystem, this would reduce material cost and simplify the fabrication process. The crucial aspect of the design is the ability of the precast concrete panel to carry the high in-plane loads and hence must be investigated properly. Furthermore, by maximizing the use of the connection details typical of conventional precast cladding system, production and installation is expected to be simpler and likely to cost less than that for Concept A. Consequently, Design Concept B was selected over Concept A because of its simplicity. The detailed description of the design is present in the followings.
Figure 4.4: Conceptual Design of EDCS – Alternative B.
Figure 4.5: Possible EDCS Configuration Based on Design Concept B.
Figure 4.6: Connection Details Based on Design Concept B.
4.3 ENERGY DISSIPATION CLADDING SYSTEM (EDCS)

4.3.1 EDCS Component Description

The EDCS can be divided into four major components: precast concrete panel, bearing connections, tie-back connections and a friction damper unit. The design considerations of each component are discussed in the following: According to PCI [2004], the stresses that occur during manufacturing, handling, and erection would usually govern the precast panel design, while seismic forces could be critical for the connection details.

4.3.1.1 Precast Concrete Cladding Panel

The spandrel type precast concrete cladding panel should be reinforced with non-prestressed steel bars. PCI [1989] suggested the maximum slenderness ratio (i.e., minimum thickness over unsupported length) for non-prestressed flat panel ranges from 0.05 to 0.02. The maximum unsupported lengths for different panel thickness are given in Table 4.1. Because the panel is designed to withstand significant in-plane force, the unsupported length should be chosen closer to the lower bound.

Table 4.1: Maximum Unsupported Length of Non-prestressed Concrete Flat Panel.

<table>
<thead>
<tr>
<th>Panel thickness (mm)</th>
<th>Maximum unsupported panel length (m)</th>
<th>(ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>101.6</td>
<td>2.0 – 5.1</td>
<td>6.7 – 16.7</td>
</tr>
<tr>
<td>152.4</td>
<td>3.0 – 7.6</td>
<td>10 - 25</td>
</tr>
<tr>
<td>203.2</td>
<td>4.1 – 10.2</td>
<td>13.3 – 33.3</td>
</tr>
<tr>
<td>254.0</td>
<td>5.1 – 12.7</td>
<td>16.7 – 41.7</td>
</tr>
<tr>
<td>304.8</td>
<td>6.1 – 15.2</td>
<td>20 – 50</td>
</tr>
</tbody>
</table>

For non-prestressed cladding panel, a minimum amount of reinforcement must be designed to withstand the stresses induced during handling and for shrinkage and crack control. The reinforcement can be either welded wire fabric or steel rebar. The most common sizes of
welded wire fabric used by precast manufacturers range from WWR4x4–W4x4 to WWR6x6–W2.5x2.5; for steel rebars, No. 3 through No. 6 bars [PCI, 2004].

Despite the fact that high in-plane stress would develop in the panel, it is expected that the connections would govern the maximum lateral load rather than the panel itself. For example, with a 4 in. thick by 80 in. high panel and assuming concrete tensile strength of 400 psi, it would take a load (assuming uniformly applied to the panel) more than 100 kips to split the panel. The concrete anchors or connections would have failed well below this load. In any case, the friction damper is designed to limit the force transferred into the concrete panel. And based on the analytical study in the previous chapter, it is expected that the optimum slip load for the EDCS would not exceed 50 kips. However, extra reinforcement is likely required around the loading point and support connections to better confine concrete anchoring systems and minimize localized concrete cracking.

4.3.1.2 Bearing Connections

Two bearing connections are used to support the entire weight of the cladding panel. Typically, lateral restraint is not provided at these bearing connections because of the in-plane volume change. However, to withstand the lateral force and to prevent uplift, one bearing connection (e.g., connection A in Figure 4.6) must be at least pin supported to the (supporting) beam. Due to the eccentricity of the applied load, the bearing connection may be required to be unusually large to withstand both seismic and gravity loads. Alternatively, separate shear plate, welded to the supporting beam, could be added to take (most of) the seismic load at the panel midspan (Figure 4.7). The shear plate could also be installed side-by-side with load bearing connection (Figure 4.8). In this configuration, the thin shear plate is significantly more flexible than the rigid bearing connection and hence would carry no or very little vertical load. On the other hand, the bearing connection simply sits on the shim plates and would not provide any significant lateral resistance. To accommodate volume changes in the panel, the other bearing connections must be allowed to translate in the in-plane direction but held down. This could be achieved as shown in the details for Connection B in Figure 4.6. Alternatively, instead of using a rigid tube, vertical bars (or plates) welded to the supporting beam could be used as bearing connections. The vertical plate would resist the vertical loads while exhibiting enough flexibility to accommodate the lateral movement.
4.3.1.3 Tie-Back Connections

Tie-back connection used in the EDCS serve two main functions. It is used to keep the panel in place and to resist out-of-plane seismic load while allowing lateral movement. For the EDCS, only one is required (Connection C) but as mentioned earlier, up to four may be placed to facilitate installation (one in each corner). The movements in both in-plane vertical and horizontal directions are allowed through these connections. Typically, commercially produced slotted
connectors such as Corewall or PSA type inserts (as shown in Figure 4.9) are used in tie-back connections. The hardware would be installed in accordance to the manufacturer’s guidelines.

![Tie-Back Connection with PSA Slotted Insert](image)

Figure 4.9: Tie-Back Connection with PSA Slotted Insert.

### 4.3.1.4 Friction Damper

From the feasibility study in Chapter 3, the friction damper appeared to be the most suitable energy dissipation device to integrate with the one-third height cladding configuration compared to viscous, viscoelastic, and yielding dampers. The friction damper serves two important functions: a source of energy dissipation and a “fuse” that effectively controls the amount of in-plane force that would be transferred into the concrete panel.

The friction damper is adopted from the slotted bolted connection (SBC) design proposed by Grigorian et al. [1993]. The SBC was tested at the University of California, Berkeley, and was found to exhibit remarkable stability and uniformity in the hysteresis looping. The friction damper used in the EDCS is shown in Figure 4.10, where the brass shims (or brake lining) are attached (using adhesive) to one leg of each angle to create two symmetrical faying surfaces. A center (mill-scale cleaned) plate with slotted hole is sandwiched between the two angle sections with friction interfaces. This plate is attached to the supporting column. To develop the required force, stacks of belleville spring washers are used together with the high-strength bolts and nuts.
The correct bolt tension can be set by means of load indicating washers (also known as direct tension indicators) typically used for slip-critical connections.

The slip load can be estimated from Equation 4.1 as follows.

\[ F_s = 2n\mu N_b \]  \hspace{1cm} (4.1)

Where \( n \) is the number of bolts, \( N_b \) is the tension force on each bolt and \( \mu \) is the coefficient of kinematic friction. A factor of two is used because of two interfaces. In Popov et al. [1995] design, the shims were reported to be half hard cartridge brass with kinematic friction coefficient varying from 0.27 to 0.31. Based on experimental test results [Pall et al., 1980; Aiken and Kelly, 1990; Grigorian et al., 1993; Popov et al., 1995] the careful selection of the friction material (e.g., brass shims, brake lining pad) can effectively reduce the difference between the static and kinematic coefficient of friction. In effect, smooth slippage without the stick-slip phenomenon
can be achieved and the characteristics rectangular shaped hysteresis loops are produced (Figure 2.46).

In quantifying the stiffness of a friction damper or any elastic perfectly-plastic material, the term secant stiffness is more appropriate than tangent stiffness. The tangent stiffness becomes irrelevant when the damper slips (i.e., in the plastic regime). The secant stiffness is defined by the slope of the line from the origin to the current point of interest on a load-deformation curve. Before the friction damper slips, the secant and tangent stiffnesses are equal and constant. However, once the damper has slipped, tangent stiffness becomes zero but this does not imply that the resistance force provided by the friction damper is zero. Rather, the friction force (or slip force) becomes constant with increasing slip. In other words, the secant stiffness is not zero but decreases with increasing slip.

It should be pointed out that the inelastic portion of the load-deformation curve of does not extend infinitely, but rather it is limited by the length of slotted holes as shown in Figure 2.45. For the friction damper developed for the EDCS, the bolt that connects the friction damper to the supporting column is designed as a secondary safety “fuse” that limits the maximum lateral load exerted on both EDCS and the column in the event that the friction damper fails to activate at the design slip load.

The selection of the optimum design slip load would be based on the building type and the performance objectives of the seismic design. This is further discussed in the building studies presented in Chapters 7 and 8.

4.3.2 EDCS Component Design Approach

The structural design of the EDCS components follows that for conventional precast concrete cladding panels and is based on existing design guidelines (e.g., AISC-LRFD, PCI Design Handbook and ACI 318). An example of the detailed structural design is attached in Appendix A. In addition to the seismic forces generated from the weight of the panel, the calculation of the design forces acting on each component must be factored in the slip load from the friction damper. This is discussed next.

Figure 4.11 shows a typical cladding panel with the associated dimensions. The total weight of panel is calculated from
Where \( L, H, t \) are the panel length, height, and thickness, respectively. With normal-weight concrete, \( w \) can be taken to be 23.6 kN/m\(^3\) (150 lbs/ft\(^3\)). The estimated in-plane and out-of-plane seismic forces generated from the panel self weight are discussed next.

### 4.3.2.1 Horizontal Seismic Force (\( E_h \) or \( F_p \))

Based on IBC2006 (ICC, 2006), the horizontal seismic force, \( F_p \), can be estimated by

\[
(0.3S_{ds} I_p)W_p \geq F_p = \left[ \frac{a_p}{R_p} \right] 0.4S_{ds} \left( 1 + 2 \frac{z}{h} \right) I_p \geq (1.6S_{ds} I_p)W_p
\]  

\[(4.3)\]
$S_{DS}$ is the design short period spectral response acceleration, $I_p$ is the component importance factor, $z$ is the height in structure of point of attachment of component with respect to the base of the building, and $h$ is the average roof height with respect to the base. The ratio of $z/h$ should not exceed unity. $a_p$ and $R_p$ represent the component amplification factor and component response modification factor for architectural cladding, respectively, which are as follows:

For cladding panel and connector body: \[
\frac{a_p}{R_p} = \frac{1.0}{2.5} = 0.4 \quad (4.4)
\]

For connector fastener: \[
\frac{a_p}{R_p} = \frac{1.25}{1.0} = 1.25 \quad (4.5)
\]

Assuming $z/h = 1.0$ (conservatively for cladding panel below building average roof height), the lateral seismic force, $F_p$. Substituting expressions for connection components in cladding panel are calculated from

For cladding panel and connector body:

\[
F_p = \left[(0.4)0.4S_{DS} \left(1 + 2(1.0)\right)I_p \right]W_p = \left(0.48S_{DS}I_p \right)W_p \quad (4.6)
\]

For connector fastener:

\[
F_p = \left[(1.25)0.4S_{DS} \left(1 + 2(1.0)\right)I_p \right]W_p = \left(1.5S_{DS}I_p \right)W_p \quad (4.7)
\]

### 4.3.2.2 Vertical Seismic Force ($E_v$ or $F_p$)

The vertical seismic force, $E_v$, is obtained from the following expression [IBCO, 2006]:

\[
E_v = (0.2S_{DS})W_p \quad (4.8)
\]
4.3.2.3 Free Body Diagrams

To facilitate the analysis of the forces exerted on the connections of the EDCS, a series of simple free-body diagrams (FBD) are presented from Figure 4.12 through 4.15. These FBD’s show the forces developed in the connections as a result of the application of unit force in different directions and at different locations. The detailed calculation is shown in Appendix A. The degree-of-freedoms provided at the connections are shown in Figure 4.16.

Application: Gravity and vertical seismic load

Figure 4.12: FBD of Cladding Panel Subjected to Vertical Force.
Figure 4.13: FBD of Cladding Panel Subjected to Horizontal In-plane Force.

Application: In-plane horizontal seismic load

Figure 4.14: FBD of Cladding Panel Subjected to Horizontal Out-of-plane Force.

Application: Out-of-plane horizontal seismic load
Application: Brace force/slip load from friction damper

Figure 4.15: FBD of Cladding Panel Subjected to Slip Force.

Figure 4.16: Degree of Freedoms at Connection Points.
4.4  SUMMARY

Two different design approaches were conceived with the same objective of developing an energy dissipating cladding system (EDCS) capable of performing as a structural brace and a source of energy dissipation. Concept B, which requires the precast concrete panel to function as a structural brace, was much simpler than Concept A. Concept A incorporates an embedded bracing subsystem, together with the friction damper, Concept B was selected as the basis behind the EDCS developed in the present study. The major components of the EDCS were identified and their preliminary design was discussed.

4.5  CONCLUSION

Concept B aims to follow the design of conventional architectural precast cladding system as closely as possible. As a result, the preliminary design of the EDCS components could be based on existing design guidelines on the conventional cladding system. However, finite element analysis should be used to validate the preliminary design and if necessary further refine it. This is discussed in the next chapter.
CHAPTER 5

COMPONENT LEVEL STUDY - FINITE ELEMENT ANALYSES

5.1 CHAPTER OVERVIEW

A comprehensive search and review of published documents revealed that there is inadequate literature on the finite element analysis (FEA) of precast concrete panels. The scarcity of research work in this area is probably due to the fact that precast concrete panels, especially architectural precast cladding panels, are not designed for significant structural forces. Where FEA is employed in a study for precast concrete panel, the panel is often idealized as uncracked and infinitely stiff. The panel connections to the structural frame remain the focus of some research interest since the connections could turn out to be the weakest link in the event of a catastrophic failure. For the present research, the precast concrete panel, as part of the EDCS, is designed to participate in the building LFRS. It is therefore important to better understand the strength and stiffness characteristics of the concrete panels subjected to significant in-plane loading.

For the present study, FEA is used in lieu of costly (and time-consuming) experimental testing. Due to the lack of experimental data on seismic performance of precast concrete panels, it would be inappropriate to perform the FEA analysis of the panel without first validating the constitutive models and modeling approach with a well understood reinforced concrete element (e.g., reinforced concrete beams). The objectives of this phase of the study are:

- Review existing literature on FEA techniques for modeling of a typical reinforced concrete structural member using ANSYS;
- Identify and quantify the critical parameters defining the constitutive models of reinforced concrete members in ANSYS;
- Calibrate the critical parameters used in the constitutive models and verify the suitability of the present modeling approach through the FEA of a well-understood reinforced concrete member, and by comparing analytical results with available test results;
- Validate the appropriateness of the FE modeling strategy for headed stud anchors, by comparing the results with existing design guidelines;
- Analyze the EDCS by extending the numerical results and modeling strategies from the calibration study; and
- Investigate the stiffness and strength characteristics of the EDCS and develop reinforcement and connection details for the precast concrete panel.

The chapter begins with a review of the constitutive models for reinforced concrete and a brief discussion of the elements types in ANSYS element library relevant to the modeling of reinforced concrete. This is followed by a review of current strategies adopted by a number of researchers for modeling reinforced concrete beams using ANSYS. Next, the results of two separate case studies performed using ANSYS are discussed. The chapter ends with a discussion on how the results from these case studies are used for the FEA of the EDCS.

5.2 BACKGROUND

Reinforced concrete structures are commonly designed to satisfy both serviceability and safety criteria. To ensure the serviceability requirement, prediction of cracking and estimation of deflection under service loads need to be considered. To meet the safety or strength requirement, an accurate estimation of the ultimate load is essential but it is also desirable to predict load-deformation characteristics of the structure.

Because of the complexities associated with the development of rational analytical procedures for reinforced concrete, many design methods still rely on the empirical approach, using the test results from a large number of experiments. Nowadays, with the availability of inexpensive and high performance computers and well-developed FEA software, FEA is now a powerful and general analytical tool to model the behavior of structural concrete. Through FEA, important parameters like stress-strain relationships, cracking model, etc., that have significant influence on the structural concrete behavior can be conveniently and systematically investigated. However, the need for some form of experimental research still continues to provide a firm basis for design equations. Experimental data also supply the much needed information, e.g. material property, to validate the mathematical models for FEA. On the other hand, reliable FE models can
considerably cut down the number of experiments required, hence reducing both time and cost of solving a given problem.

Several recent studies (discussed in Section 5.5) have focused on a commercial FE package, ANSYS, for modeling reinforced concrete beams and prestressed beams. These studies have reported good correlations between the analytical results and experimental data. This is largely attributed to the availability of a rather sophisticated element, known as SOLID65, from ANSYS element library. This element was developed primarily to model the complex nonlinear behavior of brittle materials, especially plain concrete and reinforced concrete. In particular, the complex cracking phenomenon of concrete can be modeled with its built-in cracking models. The availability of this exclusive concrete element, together with good correlations reported by these studies, has made ANSYS (Version 9) a preferred FE software for modeling precast concrete cladding panels for the present research.

5.3 CONSTITUTIVE MODEL

Reinforced concrete is a composite material that is made up of two materials with significantly different physical and mechanical behaviors. Concrete is a heterogeneous material exhibiting complex nonlinear behavior while steel is considered as homogeneous material with well-defined material properties. The typical load-deflection response of reinforced concrete beam is illustrated in Figure 5.1. This nonlinear behavior can be divided into three main stages, which are the uncracked elastic range, the crack propagation, and the plastic (yielding or crushing) stage. The major causes of nonlinearity are cracking of concrete, yielding of reinforcement and concrete crushing. The interaction of the constituents of reinforced concrete, e.g., concrete, aggregate interlock at crack, dowel action of the reinforcing steel crossing a crack, and bond slip between reinforcing steel, also has an effect on nonlinear behavior of reinforced concrete. The time-dependent effects such as creep, shrinkage, and temperature change also contribute to the nonlinear response as well. However, only the time-independent material nonlinearities of reinforced concrete are considered in the present study.
5.3.1 Concrete

5.3.1.1 Concrete Stress-Strain Relationships

A typical uniaxial stress-strain curve for plain concrete is shown in Figure 5.2. The relationship illustrates the elastic range in region OA, where the proportional limit or linear elastic limit \( f_c \) is approximately 30% of the maximum compressive strength \( f_{cm} \). In the elastic region, concrete behavior can be defined by two variables, which are the modulus of elasticity \( E_0 \) and Poisson’s ratio \( \nu \). The elastic material stiffness matrix \([D]\) for uncracked concrete is given by
The modulus of elasticity ($E_c$) can be estimated from the maximum compressive strength ($f_{cm}^c$) through the following expression [ACI, 2005]

$$E_c = \frac{57,000 \sqrt{f_{cm}^c}}{1 + \nu}$$

It should be pointed out that the modulus of elasticity in the elastic range is assumed to be the same, in both compressive and tensile regions. The Poisson’s ratio for concrete under uniaxial compressive loading ranges from 0.15 to 0.22. It has been observed experimentally that the Poisson’s ratio of concrete remains constant until nearly 80% of the maximum compressive strength ($f_{cm}^c$) [Chen, 1982]. In the present study, a constant Poisson’s ratio of 0.2 is used.
The transition from elasticity to plasticity in compressive region is defined by $f_c$ (point A), after which concrete is weakened by internal microcraking up to the end of plastic flow region $CD$ at point $D$. Figure 5.3 illustrates the loading surfaces of concrete in biaxial planes. The initial yield surface is defined as the limiting surface of elasticity regime, similar but at a certain distance away from the failure surface. The motion of subsequent loading (or yield) surfaces is defined by the hardening rule (e.g., work or isotropic hardening, kinematic hardening). It should be noted that to generate the stress-strain relationship in this plastic region (based on the flow theory of plasticity), three assumptions must be made: the shape of an initial yield surface; the evolution of subsequent loading surfaces (i.e., hardening rule); and the formulation of appropriate flow rule (e.g., associated flow rule).

In the tensile region, concrete is assumed to crack when its stress has reached the maximum tensile strength, defined by modulus of rupture of concrete ($f_t$), which is given by ACI code [ACI, 2005],

$$f_r = 7.5 \sqrt{f'_c} \quad \text{(5.3)}$$

![Figure 5.3: Loading Surfaces of Concrete in Biaxial Planes [Chen, 1982].](image-url)
5.3.1.2 Failure Criteria

The strength of concrete under multiaxial stresses can be predicted by the interaction of simple tensile, compressive, and shearing stress together. Generally concrete fails in either tensile or compressive modes. In tensile failure, major cracks develop, and the concrete loses the tensile strength perpendicular to crack direction. Compressive failure is typically defined by the formation of many small cracks and loss of its strength through concrete crushing. Several failure models or criteria have been developed to predict the failure of concrete under different multiaxial stress conditions.

The simplest failure model used is the one-parameter models that include Rankine yield criterion, Tresca yield criterion and the Von Mises yield criterion. Under tensile and small compressive stresses, concrete fails by brittle fracture with very little plastic flow before failure. The maximum-tensile-stress criterion of Rankine was proposed to determine whether a tensile or compressive type of failure has taken place. For this criterion, brittle fracture of concrete occurs when the maximum principal stress reaches the concrete tensile strength \( f' \), regardless of the normal or shearing stresses occurring simultaneously on the other planes. Under high hydrostatic pressure, concrete can yield and flow like a ductile material on the yield surface. The shearing-stress criterion was introduced for concrete in high pressure range. In the Tresca yield criterion, the maximum shearing stress is the key variable, while the octahedral stress is used in the Von Mises yield criterion.

The one-parameter model is easy to apply but is limited to simple stress states. Many complex failure models have been developed for better prediction under other multi-stress conditions. Such models include the two-parameter models (i.e., Mohr-Coulomb criterion, Drucker-Prager criterion), the three-parameter models (i.e., Bresler-Pister criterion, William-Warnke criterion), the four-parameter models (i.e., Ottosen criterion, Reimann criterion, Hsieh-Ting-Chen criterion) and the five-parameter model (i.e., William-Warnke criterion). The five-parameter William and Warnke [1975] model is incorporated into ANSYS “concrete element” and hence will be discussed next.

The schematic description in principal stress space of the failure model is illustrated in Figure 5.4. Typically the failure surface is defined in terms of hydrostatic and deviatoric sections. The hydrostatic sections are the intersection curves between failure surface and the meridian plane, containing the hydrostatic axis with constant angle \( \theta \) (also known as angle of similarity). The hydrostatic section is composed of two second-order parabolas, called tensile and
compressive meridians at $\theta = 0^\circ$ and $\theta = 60^\circ$, respectively. Both parabolas have a common apex at the hydrostatic axis, corresponding to hydrostatic tensile yield. On the other hand, the deviatoric section is the intersection between the failure surface and the deviatoric plane, perpendicular to the hydrostatic axis. Because of isotropic property, the deviatoric section has three-fold symmetry, consisting of three elliptic sections with smooth transitions.

The William-Warnke three-parameter model has straight meridians in hydrostatic section and noncircular deviatoric cross sections, as shown in Figure 5.5. The three parameters are identified by the three simple tests, which are the uniaxial tension test ($\sigma_1 = f'_c$, $\theta = 0^\circ$), the uniaxial compression test ($\sigma_3 = -f'_c$, $\theta = 60^\circ$) and the equal biaxial compression test ($\sigma_2 = \sigma_3 = -f'_c$, $\theta = 0^\circ$). Chen [1982] presented a comparison between the William-Warnke model and the experimental data reported by Launay and Gachon [1972], as shown in Figure 5.6. Closed agreement of the analytical predictions and experimental results was observed in the low compression and tension zone. However, significant disparity was observed in the high compression zone. From this comparison, Chen [1982] suggested that William-Warnke three-parameter model be limited to concrete failure in tension and low compression regime.

The William-Warnke three-parameter model can be further refined by adding two more parameters or points for describing curved meridians to match the experimental data at the high compression zone. As illustrated in Figure 5.7, these extra points are the high compressive stress point on the tensile meridian ($\theta = 0^\circ$, $\xi > 0$) and the high compressive stress point on the compressive meridian ($\theta = 60^\circ$, $\xi_2 > 0$). Figure 5.8 shows good agreement between the William-Warnke five-parameter model and experimental data reported by Launay and Gachon [1972].

Figure 5.4: William-Warnke Failure Surface in Principal Stress Space
[William and Warnke, 1975].
a) hydrostatic section  b) deviatoric section

Figure 5.5: William-Warnke Three-Parameter Model [Chen, 1982].

a) hydrostatic section  b) deviatoric section

Figure 5.6: Comparison of William-Warnke Three-Parameter Model and Experimental Data with $f_{bc} = 1.8, f'_c = 0.15$ [Chen, 1982].

a) hydrostatic section  b) deviatoric section

Figure 5.7: Five-Parameter Model [Fuschi, et al., 1994].
5.3.1.3 Cracking and Crushing Phenomenon

In the present model, the onset of cracking or crushing of concrete is determined by the following criterion due to a multiaxial stress state [SAS, 2003]

\[
\frac{F}{f'_c} - S \geq 0
\]  

where \( F \) is a function of principal stresses \( (\sigma_1, \sigma_2, \sigma_3) \), \( S \) is the failure surface expressed by the William-Warnke five-parameter model, and \( f'_c = \) uniaxial compressive strength.

The William-Warnke failure surface is defined by five parameters, which are ultimate uniaxial tensile strength \( (f'_t) \), ultimate uniaxial compressive strength \( (f'_c) \), ultimate biaxial compressive strength \( (f'_{bc}) \), ultimate compressive strength for a state of biaxial compression superimposed on hydrostatic stress state \( (f_b) \), and ultimate compressive strength for a state of uniaxial compression superimposed on hydrostatic stress state \( (f_1) \). The minimum parameters required by ANSYS are the ultimate uniaxial tensile strength \( (f'_t) \) and ultimate uniaxial compressive strength \( (f'_c) \). In the absence of experimental data, the other three parameters default to the values proposed by William-Warnke [1975] as follows,

Figure 5.8: Comparison of William-Warnke Five-Parameter Model and Experimental Data with \( f'_{bc} = 1.8, f'_t = 0.15, \xi = 3.67, \bar{r}_1 = 1.59, \bar{r}_2 = 1.94 \) [Chen, 1982].
However, these default values are valid only for stress states with a low hydrostatic stress ($\sigma_h$) condition, which is

$$|\sigma_h| \leq \sqrt{3} f'_c \quad (5.8)$$

The nature of concrete failure (cracking or crushing, direction, etc.) is further categorized into four domains [SAS, 2003], depending on the relationships of the three principal stresses. The four domains are as follows:

1$^{\text{st}}$ Domain: $0 \geq \sigma_1 \geq \sigma_2 \geq \sigma_3$ (compression – compression – compression)
- If the failure criterion is satisfied, the material is assumed to crush.

2$^{\text{nd}}$ Domain: $\sigma_1 \geq 0 \geq \sigma_2 \geq \sigma_3$ (tension – compression – compression)
- If the failure criterion is satisfied, cracking occurs in the plane perpendicular to principal stress $\sigma_1$.
- Crushing can also occur.

3$^{\text{rd}}$ Domain: $\sigma_1 \geq \sigma_2 \geq 0 \geq \sigma_3$ (tension – tension – compression)
- If the failure criteria for both principal stresses $\sigma_1$ and $\sigma_2$ are satisfied, cracking occurs in the plane perpendicular to principal stresses $\sigma_1$ and $\sigma_2$.
- If only the failure criterion for principal stress $\sigma_1$ is satisfied, cracking occurs only in the plane perpendicular to principal stress $\sigma_1$.
- Crushing can also occur.

4$^{\text{th}}$ Domain: $\sigma_1 \geq \sigma_2 \geq \sigma_3 \geq 0$ (tension – tension – tension)
- If the failure criteria for all principal stresses are satisfied, cracking occurs in the plane perpendicular to principal stresses $\sigma_1$, $\sigma_2$ and $\sigma_3$.
- If the failure criteria for both principal stresses $\sigma_1$ and $\sigma_2$ are satisfied, cracking occurs in the plane perpendicular to principal stresses $\sigma_1$ and $\sigma_2$. 

\begin{align*}
  f'_{bc} &= 1.2 f'_c \quad (5.5) \\
  f_1 &= 1.45 f'_c \quad (5.6) \\
  f_2 &= 1.725 f'_c \quad (5.7)
\end{align*}
If the failure criterion only for principal stress $\sigma_i$ is satisfied, cracking occurs only in the plane perpendicular to principal stress $\sigma_i$.

In ANSYS, cracking is modeled through an adjustment of material properties which effectively treats cracking as a smeared band of cracks, rather than discrete cracks (Figure 5.9). As pointed out by Chen [1982], the discrete approach, though it can give a better representation of the crack pattern, it is complex and time-consuming since the location and orientation of the cracks must be known in advance. In ANSYS, any plasticity specified (e.g., Drucker-Prager) is computed before the cracking and crushing checks [SAS, 2003].

![Cracking Models](image)

Figure 5.9: Cracking Models [Kwak and Filippou, 1990].

5.3.1.4 Modeling of Cracked and Crushed Concrete

When cracking or crushing behavior has occurred, the stress-strain relations (i.e., stiffness matrices) are adjusted. Under crushing condition, the crushed concrete element is assumed to be deteriorated completely with zero stiffness [SAS, 2003]. This simple approach appeared to be reasonable but often present significant difficulties in the analysis due to non-convergence of the solution.

For cracking behavior, cracks are assumed to form in planes perpendicular to the direction of maximum principal tensile stress when it reaches the specified tensile strength of concrete. Here, the elastic modulus is usually assumed to be zero (Figure 5.10a) for unreinforced concrete. For reinforced concrete, since some tension can transfer through the bond between the concrete and reinforcing steel, tensile stresses are assumed to be carried by concrete between the cracks. The concrete hangs on the reinforcing bars and contributes to overall stiffness of the
structure, so called *stiffness effect* or *tension stiffening effect* (Figure 5.10). In Figure 5.10, $f'_{t}$ or $f_{t}$ is the specified concrete tensile strength, $T_{c}$ is the multiplier for the amount of tensile stress relaxation (default to 0.6 in ANSYS), and $R_{t}$ is the slope or secant modulus of the cracked concrete. Hence, the material stiffness matrix $[D^{c}]$ of cracked concrete is modified from Equation 5.1 and is now redefined in terms of modulus of elasticity ($E$), Poisson’s ratio ($\nu$), secant modulus ($R_{t}$), shear transfer coefficients for open ($\beta_{l}$) and closed cracks ($\beta_{c}$), depending on the cracking arrangement (i.e., crack directions, crack closed and open conditions). It should be noted that the shear transfer coefficients range from 0.0 (smooth crack – complete loss of shear transfer) to 1.0 (rough crack – no loss of shear transfer). The modified stiffness matrices of cracked concrete element are discussed in great detail in ANSYS Help [SAS, 2003] and will not be covered here.

5.3.2 Reinforcing Steel

5.3.2.1 Behavior of Reinforcing Steel

Unlike concrete, the properties of reinforcing steel are generally not dependent on environmental conditions or time. Since reinforcing steel is comparatively thin, it is assumed to be one dimensional element, capable of transmitting axial force only. Hence, the specification of uniaxial constitutive relationship is sufficient to define the steel property in the analysis [Nilson, 1982; Chen, 1982; Kwak and Fillipou, 1990]. Three different idealizations, illustrated in Figure 5.11, are commonly used depending on the desired accuracy level.
In the first idealization (Figure 5.11a) the reinforcing steel is modeled as linear elastic perfectly plastic approximation, neglecting the increase of strength due to strain hardening. This approximation underlies the ACI design equations and gives satisfactory results when the strain at the onset of strain hardening is much larger than the yield strain, which is true for low-carbon steels with low yield strength. On the other hand, the linear elastic with strain hardening models (Figure 5.11b and Figure 5.11c) represent more accurate behavior when steel stress at high strains is required, particularly in seismic design.

However, it was suggested by Kwak and Fillipou [1990] that use of the strain-hardening steel model when the structure is subjected to monotonic bending moments has the advantage in improving the numerical stability of the solution compared to the elastic perfectly-plastic model. Hence, a linear elastic with linear strain hardening model is used. This model requires stresses and strains at the onset of yielding, and at the onset of strain hardening. Moreover, a perfect bond between steel and concrete is assumed.

![Stress-Strain Curves for Reinforcing Steel](image)

Figure 5.11: Stress-Strain Curves for Reinforcing Steel [Park and Paulay, 1975].
5.3.2.2 Modeling Approaches for Reinforcing Steel

Three different approaches, as illustrated in Figure 5.12, can be used in the modeling of reinforcing steel in a reinforced concrete structure: the discrete model; the embedded model; and the distributed or smeared model [Nilson, 1982; Tavarez, 2001; Wolanski, 2004].

A discrete model with one dimensional reinforcement elements (bar or beam elements) has been most widely used. The rebar elements are superimposed in the concrete model. The rebar and the concrete mesh share the same nodes and concrete occupies the same regions occupied by the rebar. An advantage of the discrete model is that it is able to account for possible displacement of the rebar with respect to the surrounding concrete. However, a drawback of this model is that the concrete mesh is restricted by the location of the rebar and also the rebar volume is not deducted from the volume of concrete.

In an embedded model, one dimensional reinforcement elements are built into the isoparametric concrete elements such that reinforcement displacements are compatible with the surrounding concrete. Higher order isoparametric concrete elements may be used in this model, therefore, increasing the number of nodes and degrees of freedom in the model. However, since the reinforcement stiffness matrix is evaluated separately from the concrete element, this model is very advantageous when the reinforcement pattern is complex.

For a distributed or smear model, the reinforcement is assumed to be uniformly distributed over the concrete elements with a particular orientation angle. The composite concrete-reinforcement stiffness matrix is used in this model. Therefore, perfect bond between reinforcing steel and concrete is assumed. This model is used for large-scale models where the reinforcement does not significantly contribute to the overall response. However, it was found that the optimum modeling strategy is to use the discrete reinforcement model [Fanning, 2001]. For the present study, the discrete model is used for the steel reinforcement.
5.4 ANSYS ELEMENT TYPE

5.4.1 Concrete

The SOLID65 “concrete element” (Figure 5.13) from ANSYS element library is defined by eight nodes having three degrees of freedom at each node (translations in the nodal x, y, and z directions). As mentioned earlier, this element incorporates William-Warnke five-parameter failure criteria and is capable of modeling cracking and crushing phenomenon.

To implement the William-Warnke model in the SOLID65 concrete element, nine constants must be specified [SAS, 2003]. They have been discussed earlier and are listed below.
1. Shear transfer coefficients for an open crack ($\beta_o$)
2. Shear transfer coefficients for a closed crack ($\beta_c$)
3. Uniaxial tensile cracking stress ($f_t'$)
4. Uniaxial crushing stress ($f_c'$)
5. Biaxial crushing stress ($f_{bc}'$)
6. Ambient hydrostatic stress state for use with constants 7 and 8 ($\sigma_h$)
7. Biaxial crushing stress under the ambient hydrostatic stress state ($f_{1c}$)
8. Uniaxial crushing stress under the ambient hydrostatic stress state ($f_{2c}$)
9. Stiffness multiplier for cracked tensile condition ($T_c$)

Kachlakev, et al. [2001] reported that the convergence problems occurred when the shear transfer coefficient for an open crack dropped below 0.2. Wolanski [2004] suggested that a shear transfer coefficient of 0.3 for an open crack and 1.0 for a closed crack would be appropriate for his FEA of reinforced concrete beams. Moreover, Kachlakev, et al. [2001] and Wolanski [2004] both recommended that the crushing capability of the SOLID65 element could be deactivated to avoid non-convergence of solution. Unless indicated otherwise, the crushing capability of SOLID65 element is generally not used in the FEA for the present study due to the same reasons cited by the other researchers.

![Figure 5.13: SOLID65 Element](image-url)
5.4.2 Reinforcing Steel

The LINK8 element (Figure 5.14) was used to model reinforcing steel. This 3-D spar element is an uniaxial tension-compression element with three translational degrees of freedom at each node (nodal x, y, and z directions). This element is also capable of plastic deformation.

![Figure 5.14: LINK8 Element [SAS, 2003.]](image)

5.4.3 Shim Plate and Anchored Plate

The three-dimensional SOLID45 element (Figure 5.15) was used to model shim plate and anchored plate. The element is defined by eight nodes having three degrees of freedom at each node (translations in the nodal x, y, and z directions). The element is capable of modeling plasticity, creep, swelling, stress stiffening, large deflection, and large strain conditions.

![Figure 5.15: SOLID45 Element [SAS, 2003.]](image)
5.4.4 Tie-Rod, Headed Stud and Bearing Connection (HSS section)

The BEAM188 element (Figure 5.16) was used to model the tie-rod, headed stud, bearing connection (HSS section). The three dimensional element can be either a linear (2-node) or a quadratic beam element, based on Timoshenko beam theory with shear deformation effects. The element has six degrees of freedom (translations in the x, y, and z directions and rotations about the x, y, and z directions) at each node. The warping degree of freedom at the node can be added. In this study, a quadratic beam option was chosen without warping degree of freedom. The beam sectional properties can be defined in this element type. The shapes of the available cross-section are shown in Figure 5.17.

Figure 5.16: BEAM188 Element [SAS, 2003].

Figure 5.17: BEAM188 Section Types [SAS, 2003].
5.5 CALIBRATION STUDY

5.5.1 Literature Review on FEA of Reinforced Concrete Beams Using ANSYS

5.5.1.1 Analytical Study of Barbosa and Ribeiro [Idelsohn, et al., 1998]

Barbosa and Ribeiro [Idelsohn, et al., 1998] studied the numerical modeling technique of a simply supported reinforced concrete beam (Figure 5.18) using ANSYS Version 5.3. The singly-reinforced beam studied was 200 mm (7.87 in.) wide by 350 mm (13.7 in.) deep and 4 m (13.1 ft) long. Only longitudinal flexure reinforcement, with an area of 1142 mm$^2$ (1.77 in$^2$) and yield stress of 500 MPa (72.5 ksi), was considered. The concrete compressive and tensile stresses were specified as 30 MPa (4,351 psi) and 2.5 MPa (363 psi), respectively. The beam was subjected to uniformly distributed loading on the top surface over its entire length.

![Analytical Reinforced Concrete Beam](image)

Figure 5.18: Analytical Reinforced Concrete Beam [Idelsohn, et al., 1998].

In the study, different uniaxial stress-strain relationship (i.e., linear elastic, elastic perfectly-plastic, and multilinear work hardening) for the SOLID65 concrete element were investigated. In addition, the study compared the smeared and discrete models for defining reinforcing steel and the influence of crushing model on the analytical solution. All models employed the built-in smeared crack model. The numerical load versus deflection plot at midspan from the FEA was compared to the analytical solution derived from classical reinforced concrete beam theory. The study showed that the use of the crushing capability in the SOLID65 concrete element led to solution non-convergence, regardless of the reinforcement modeling approach.
used. Only the models with multi-linear hardening stress-strain relationship in the absence of crushing were able to converge to the theoretical failure load; the other models encountered numerical instability. Parvin and Granata [2000] reported that non-convergence occurs when a crack extends completely through the SOLID65 concrete element. In such case, the element has no stiffness and nodal displacements are not restrained. It was suggested that use of “dummy” elements (PIPE16 element) to control nodal displacements could eliminate this problem.

5.5.1.2 Externally CFRP Strengthened Reinforced Concrete Beams of Fanning and Kelly [2000]

Fanning and Kelly [2001] conducted a test programme to study the strength enhancement to simply supported reinforced concrete beams offered by externally applied carbon fibre reinforced polymer (CFRP) composite material plates. The full size beams measured 155 mm (6.1 in.) wide by 240 mm (9.4 in.) deep, 3 m (9.8 ft) long and with a clear span of 2.8 m (9.2 ft). Tensile and compression reinforcement comprised of three and two bars of high yield (460 MPa or 65.4 ksi) 12 mm (0.47 in.) diameter steel bars, respectively. Ten 6 mm (0.24 in.) diameter shear links (i.e., vertical stirrups) were placed at 125 mm (4.9 in.) apart within the shear spans; the yield strength was 250 MPa (35.6 ksi). The reported concrete compressive and tensile stresses were 80 MPa (11.6 ksi) and 5 MPa (725 psi), respectively. Eight out of the ten beams were strengthened by application of CFRP to the bottom surface of the beam. The test results showed that with adequate anchorage, the added CFRP increased the beam original flexure capacity by as much as 70%. However, there was no observable increase in shear strength and strengthening scheme led to a non-ductile failure.

Using ANSYS version 5.4, Fanning [2001] modeled the tested systems with a combination of SOLID65 (concrete medium and adhesive layer), LINK8 (reinforcement) and SHELL63 (strengthening plates) elements as shown in Figure 5.19. A linear elastic stress strain relationship was used for the SOLID65 and SHELL63 elements while an elastic perfectly-plastic rule was specified for the LINK8 elements. The cracking model was activated for the concrete elements but the concrete crushing behavior was not modeled. The discrete presentation of the reinforcing steel with LINK8 element was used. The FEA results (Figure 5.20) showed that the crack model in ANSYS SOLID65 element was able to capture the progressive ductile failure mechanism of unstrengthened (i.e., ordinarily reinforced) beams, correlating very well with the
experimentally observed stiffness and strength characteristics. However, for modeling the externally strengthened beams, the crack model in ANSYS SOLID65 element should be limited to the cases with properly anchorage CFRP plates and no peel-off modes of failure.

In a separate study by Fanning [2001], different strategies for modeling reinforced concrete beam and prestressed concrete beam in ANSYS (Version 5.5) were investigated. It was found that the discrete representation of the reinforcing steel was appropriate for ordinarily reinforced concrete beam, both in terms of controlling element size and precise placement of the reinforcement. For prestressed concrete beam, the best modeling strategy suggested was to model the prestressed tendons discretely with all other reinforcing steel represented by the smeared model. Furthermore, Fanning [2001] concluded that the concrete properties estimated from prevailing design codes were appropriate for defining the analytical model.

Figure 5.19: FE Mesh [Fanning and Kelly, 2000].

Figure 5.20: Numerical and Experimental Deflection Comparisons [Fanning and Kelly, 2000].
5.5.1.3 Reinforced Concrete beam and Prestressed Concrete Beam Models of Wolanski [2004]

Wolanski [2004] performed the FEA of reinforced and prestressed concrete beams using ANSYS Version 7.1. Using the experimental data from a separate research study by Buckhouse [1997], Wolanski [2004] developed FE models (in ANSYS) to capture the stiffness and strength characteristics of the beams.

In Buckhouse [1997] experiment, three under-reinforced control concrete beams were tested as part of a study to investigate the effectiveness of reinforcing concrete beam for flexure using external structural steel channels. The control beam was 254 mm (10 in.) wide, 457.2 mm (18 in) deep, and 4.72 m (15.5 ft) long and with a clear span of 4.57 m (15 ft). The Grade 60 reinforcing bars consisted of three 15.875 mm (0.625 in.) for main bars and 9.5 mm (0.375 in.) U-stirrups placed at varying spacing according to the shear force. The reported 28-day concrete strength was 32.9 MPa (4,770 psi). These beams were loaded at third points along the length. The beams were loaded to failure as shown in Figure 5.21. The failure mode was characterized by crushing of the concrete with significant yielding of the steel reinforcement in the constant moment region.

In his study, Wolanski [2004] developed a FE model for one of the two control beams tested as shown in Figure 5.22. SOLID65 element was used to model concrete medium while the steel plate bearing supports were modeled with SOLID45 elements. The flexure and shear reinforcement were modeled in the discrete fashion using LINK8 element. The multilinear isotropic property was specified for SOLID65 concrete element along with the William and Warnke five-parameters failure criterion. In addition, the compressive uniaxial stress-strain relationship was defined by a parabolic stress-strain relationship similar to the one proposed by

![Figure 5.21: Tested Beam of Buckhouse [1997].](image-url)
Hognestad [1951]. Cracking model of the SOLID65 element was activated but the crushing model was not used to avoid non-convergence issues.

A comparison of the load (midspan) deflection curves obtained from the FE analyses using ANSYS, experimental data and fundamental reinforced concrete theory, as reported by Wolanski (2004), is shown in Figure 5.23. The deflection at failure predicted by the FE model was within 2% of the test result. The results obtained from the FEA study appeared to be in good agreement with each other.

Figure 5.22: FE Model for Reinforced Concrete Beam [Wolanski, 2004].

Figure 5.23: Comparison of Load-Deflection Response [Wolanski, 2004].
5.5.2 Case Study 1 – Reinforced Concrete Deep Beam

5.5.2.1 Introduction

Based on the existing literature review presented, it appeared that ANSYS is capable of modeling the complex nonlinear behavior of structural concrete through the availability of the sophisticated SOLID65 concrete element. However, before any modeling of the precast concrete panel was performed, it was first necessary to attempt to model a well-understood reinforced concrete element and correlate the analytical results with available experimental data. It was important to gain experience in the use of ANSYS while verifying the suitability of the modeling strategy and the values assumed for the many parameters (e.g., stress-strain relationships, element types, etc.) used in defining the constitutive models of reinforced concrete.

The studies reported earlier focused primarily on conventional reinforced concrete beams which are “structurally” different from the precast concrete panel. The behavior of reinforced concrete deep beam, with similar depth-to-width (or thickness) ratio to precast concrete panel, appeared to be a more suitable choice. It was decided that the calibration (or verification) exercise would focus on the FEA analysis of a deep beam. Leonhardt and Walther [1966] carried out a well-documented research program to test several simply supported and continuous reinforced concrete deep beams. The test results have been extensively referenced by several researchers [Reineck et al., 2002; Lourenco et al., 2005], especially in the area of disturbed regions (or D-regions) modeling. One of the simply-supported deep beam specimens, WT2, was selected for modeling using ANSYS. This is discussed next.

5.5.2.2 Specimen Description

The reinforced concrete deep beam is a 1.6 m (5.25 ft) wide by 1.6 m (5.25 ft) deep by 100 mm (3.9 in.) thick (Figure 5.24). This deep beam was supported on 160 mm (6.3 in.) long bearing plates with a center-to-center clear span of 1.44 m (4.72 ft). The beam was loaded uniformly on the top with a uniform load applied over the clear span distance of 1.28 m (4.20 ft). The main (bottom) flexure reinforcement consist of four 8 mm (0.32 in.) diameter reinforcing bars having a yield stress of 427.5 MPa (62 ksi). The vertical faces of the deep beam were reinforced with 5 mm (0.2 in.) diameter reinforcing bars at 259 mm (10.2 in.) in both directions.
The reported concrete strength at the time of testing was 30.2 MPa (4,380 psi). The deep beam failed at a total load of 1,170 kN (263 kips).

**Figure 5.25** shows the (expanded) three-dimensional FE model for the concrete and reinforcing steel cage developed using ANSYS. Only half of the beam was modeled due to symmetry at midspan of the beam. A total of 2,888 SOLID65 elements were used to model the concrete mass whereas a total of 566 LINK8 elements were used to assemble the reinforcing cage. 101.6 mm (4 in.) by 101.6 mm (4 in.) concrete elements were used except for the surface elements where 50.8 mm (2 in.) by 101.6 mm (4 in.) elements were specified to match the required concrete cover. The A36 steel bearing plates at the supports were modeled using 60 SOLID45 elements. Earlier preliminary FEA suggested that the element sizes were suitable in terms of solution accuracy and computation effort. The left support plate was pinned at mid-span of its lower surface while the other plate was restrained only in the horizontal direction (i.e., a roller support). Placement of the reinforcing bars proved to be one of the most challenging tasks.

**5.5.2.3 ANSYS FE Model**

Figure 5.24: Deep Beam of Leonhardt and Walther [1966].

![Deep Beam Diagram]
in the modeling. Special algorithm had to be coded into the ANSYS input file to correctly position the LINK8 elements within the concrete mass while ensuring compatibility of nodes with the SOLID65 elements.

![FE Model of Deep Beam](image)

Figure 5.25: FE Model of Deep Beam.

**Stress-Strain Relationships For Concrete**

Many uniaxial stress-strain relationships have been proposed for describing the nonlinear concrete behavior. Many of these relationships are very similar, differing only in the mathematical function that defines each relationship. Here, four types were investigated.

1. Hognestad [1951]
2. Modified Hognestad [1951]
3. “Equivalent” elastic perfectly plastic
4. Linear elastic

The stress-strain curves are plotted in Figure 5.26. The following points are made with regards to the concrete stress-strain relationships.

- All stress-strain curves have the same ultimate compressive strength ($f'_c = 30.2$ MPa or 4,380 psi) and (initial) tangent modulus of elasticity ($E'_c = 2.60 \times 10^4$ MPa or $3.77 \times 10^6$ psi).
• The nonlinear ascending portion of Hognestad and modified Hognestad stress-strain curves are identical. They are defined by the same parabolic function. The parabolic function is given by

\[ \sigma_c = f_c \left[ \frac{2\varepsilon_c}{\varepsilon_0} - \left( \frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right] \tag{5.9} \]

where \( \sigma_c \) is the stress at a given strain \( \varepsilon_c \). \( \varepsilon_0 \) is the strain at maximum stress and is given by

\[ \varepsilon_0 = \frac{1.8 f_c}{E_c} \tag{5.10} \]

Based on the specified concrete strength, \( \varepsilon_0 \) is equal to 0.00209.

• The Hognestad stress-strain relationship is defined by the above parabola followed by a horizontal line terminating at a limiting strain of 0.003. The modified Hognestad stress-strain curve has a descending segment terminating at a limiting strain of 0.0038 at 0.85 \( f_c' \).

• The “equivalent” elastic perfectly plastic curve is defined by the same ultimate compressive strength and tangent modulus of elasticity, with the limiting strain of 0.003. The yield strain is estimated to be 0.00116.

• The linear elastic stress-strain curve is based on the assumption of no plasticity in concrete.

Analysis Cases

A series of FEA cases was carried out to investigate the suitability of constitutive models for concrete. Table 5.1 summarizes the analysis cases performed. On the average, each case took more than eight hours to analyze on a computer with Pentium 4, 2.4 GHz processor and 1GB memory. Cases dbh1 to dbh5, are not reported here because they were used in the preliminary study to establish the optimum load step and mesh size. These cases show exactly the same load-deflection characteristics as db6 but were not able to reach the expected failure load due to the solution not converging. This problem was solved by specifying a sufficiently small load step and using a finer mesh, finally resulting in case dbh6. Cases db7, db13 and db14 were created to investigate the effect of changing the support conditions. But since the results were not directly relevant to the present study, they are not reported here.
Figure 5.26: Concrete Stress-Strain Curves.

Table 5.1: Deep Beam Analysis Cases.

<table>
<thead>
<tr>
<th>Case</th>
<th>Label</th>
<th>Stress-strain relationship</th>
<th>Plasticity Rule</th>
<th>Concrete Cracking Model</th>
<th>Concrete Crushing Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>db6</td>
<td>Hognestad</td>
<td>MISO</td>
<td>Active</td>
<td>Inactive</td>
</tr>
<tr>
<td>2</td>
<td>db8</td>
<td>Modified Hognestad</td>
<td>MISO</td>
<td>Active</td>
<td>Inactive</td>
</tr>
<tr>
<td>3</td>
<td>db9</td>
<td>Hognestad</td>
<td>MISO</td>
<td>Active</td>
<td>Active</td>
</tr>
<tr>
<td>4</td>
<td>db10</td>
<td>Linear Elastic</td>
<td>None</td>
<td>Active</td>
<td>Active</td>
</tr>
<tr>
<td>5</td>
<td>db11</td>
<td>Modified Hognestad</td>
<td>MISO</td>
<td>Inactive</td>
<td>Inactive</td>
</tr>
<tr>
<td>6</td>
<td>db12</td>
<td>Hognestad</td>
<td>KINH</td>
<td>Active</td>
<td>Inactive</td>
</tr>
<tr>
<td>7</td>
<td>db15</td>
<td>Elastic Perfectly plastic</td>
<td>BISO</td>
<td>Active</td>
<td>Inactive</td>
</tr>
</tbody>
</table>

Note:  
a. MISO – Multilinear isotropic hardening  
b. BISO – Bilinear isotropic hardening  
c. KINH – Multilinear kinematic hardening
5.5.2.4 FEA Results

Influence of Concrete Stress-Strain Curve

The plots of load versus mid-span deflection for cases db6, db8, db10 and db15 are compared with the experiment data in Figure 5.27. The models for all cases exhibited almost identical response up to 667.2 kN (150 kips). The load-deflection curve for case dh6 matches very well with the experimental result up to 889.6 kN (200 kips), beyond which the analytical model exhibited a stiffer response than the actual test beam. The difference between the predicted and reported failure load is less than 0.4%. The result was very encouraging because no calibration was made to the concrete mode; only the load steps were changed in earlier cases. It should be pointed out that with the exception of case dhb6 where the solution stopped due to excessive element deformation (an indication of material yielding), the other analysis cases (dbh8, dbh10 and dbh15) encountered premature termination due to non-convergence of solution arising from numerical instability. Thus, the last point on the load-deflection curve for each of these cases may not represent the failure condition.

Figure 5.27: Influence of Stress-Strain Relationship on Load-Deflection Curves.
For case dbh8 where the modified Hognestad stress-strain curve was implemented, the analytical result appeared to better correlate with the experimental data for loads above 889.6 kN (200 kips). It is believed that above 889.6 kN (200 kips), concrete crushing (at the top extreme concrete fiber) has occurred, leading to greater reduction in the beam stiffness. A descending segment for the stress-strain curve seems to be able to model the crushing phenomenon. However, the problem of non-convergence of solution prevented the attainment of the expected failure load, despite the use of very small (0.04 N or 0.01 lbs) load step in the analysis. For cases dbh10 and dbh15, the solutions did not converge at low applied load and there is no clear indication that the use of a linear elastic or elastic perfectly-plastic stress-strain curves, respectively, led to comparable results. From these results, it appeared that the simple Hognestad stress-strain relationship is appropriate up to 70% of the failure load.

**Effect of Crushing and Cracking Model**

Case dbh9 was identical to dbh6 except that the crushing model of the SOLID65 concrete elements was used (Figure 5.28). As expected, the solution failed to converge even at low applied load. This problem is not uncommon as reported by other researchers. In fact, several researchers recommended deactivating the crushing capability of SOLID65 concrete element. One way to solve this problem might be the use of an explicit solver (e.g., ANSYS/LS-DYNA). For subsequent analyses, the crushing capability of the concrete element was deactivated.

Figure 5.29 shows the result of discarding both the cracking and crushing models. Cases db8 and db11 are identical (with modified Hognestad stress-strain curve) except in the latter case, both cracking and crushing models were not activated; the crushing model to avoid solution non-convergence. Without any concrete cracking (and crushing), the analytical model in db11 exhibited a significantly stiffer response from a low load of 222.4 kN (50 kips). This deviation clearly indicates that the cracking phenomenon played a significant role in determining the analytical load-deflection characteristics. This might also suggest that the parabolic Hognestad (or modified Hognestad) uniaxial stress-strain curve cannot completely account for the reduction of the beam stiffness due to significant concrete cracking, and an appropriate cracking model must be implemented to correctly correlate with the observed test results.
Without an explicit concrete crack model, the analytical model for case db11 failed at 1,241 kN (279 kips); an over prediction of 6%. The benefit of omitting a concrete crack model is the significant reduction (more than 100%) in computation time and excellent solution convergence. However, the stress-strain curve may need to be slightly modified to correlate with the experimentally observed stiffness and strength characteristics.

The other advantage of employing the cracking model is the ability to plot the crack patterns in ANSYS at any given load step. The analytical crack patterns would be a valuable parameter for analytical-experimental comparison since recording of crack pattern is almost always performed in the testing of reinforced concrete structures. Figure 5.30 shows the sequence of cracks formation, at roughly equal load increments, from the onset of cracking to the failure load (db6). The analytical crack pattern at failure load correlates well with the test result.

Figure 5.28: Influence of Crushing Model on Load-Deflection Curves.

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Influence of Hardening Rule

Chen [1982] reported that the combined isotropic (work) and kinematic hardening rule is appropriate for the behavior of concrete subjected to load reversal. However, there was not enough information at the time of the study to specify a mixed hardening rule in ANSYS. Here the isotropic and kinematic hardening rules were applied separately for case db6 and db12, respectively. As seen from Figure 5.31, the kinematic hardening rule appeared to give a slightly better correlation; this is not sufficient to justify its suitability since only monotonic loading condition is examined. However, Chen [1982] suggested that the kinematic hardening is more appropriate than the isotropic rule for modeling concrete.

Figure 5.29: Influence of Cracking Model on Load-Deflection Curves.
Figure 5.30: Crack Development in Deep Beam.
Redistribution of Stresses

It is well-known that significant redistribution of stresses can occur in reinforced concrete structures due to concrete cracking. Figure 5.32 shows the redistribution of stress in the D-region of the analytical beam (db6). Even at less than half the failure load, there was significant redistribution of the normal stresses along the depth of the beam after the onset of cracking at about 249 kN (55.9 kips). The nonlinear elastic stress distribution at 222 kN (50.0 kips) agreed well with the analytical results from other researcher [MacLeod, 2005].

Figure 5.31: Effect of Hardening Rule.
5.5.3 Case Study 2 – Headed Concrete Anchors

5.5.3.1 Introduction

Headed concrete anchors (HCA) or studs are specified in the proposed EDCS as an effective concrete anchorage system to distribute the high in-plane loads and reactions from the bearing connections to the concrete. It was decided that the HCA should be explicitly modeled as part of the FEA of the precast concrete panel. However, literature on modeling of HCA using ANSYS was lacking at the time of the present study. Hence it was necessary to conduct preliminary FEA to give confidence to the suitability of the modeling technique adopted for the HCA. This section summarizes the preliminary modeling and analyses of HCA with ANSYS. The numerical results were compared with design values obtained from the prevailing design code.

Figure 5.32: Redistribution of Normal Stress in Deep Beam.
5.5.3.2 Specimen Description

The HCA consisted of a 152.4 mm (6 in.) by 152.4 mm (6 in.) by 25.4 mm (1 in.) thick anchorage plate with four Type B 12.7 mm (½ in.) diameter by 152.4 mm (6 in.) long headed studs (Figure 5.33). The (shear) load application on the HCA was specified as concentric. Three different locations of the HCA were investigated here. These locations correspond to three distinct loading conditions, namely front-edge loading, corner-edge loading and side-edge loading. The loading condition is dictated by the ratio of the side-edge distance to the back-edge distance. The unreinforced concrete panel measured 2.13 m (7 ft) and 4.26 m (14 ft) for side-edge loading condition by 2.44 m (8 ft) by 203.2 mm (8 in.) thick with a concrete strength of 34.5 MPa (5,000 psi). The right edge of the panel was restrained from translations and rotation in all directions.

Figure 5.33: Locations of HCA.
5.5.3.3 Nominal Design Values

New design guidelines for the anchorage to concrete and the design of cast-in-place headed studs or bolts, based on the “Concrete Capacity Design” (CCD) approach, were recently introduced as Appendix D, “Anchoring of Concrete” in ACI 318-02 [ACI, 2003]. Appendix D in the 2005 edition of ACI 318 underwent significant changes to reflect new test data; more changes were expected for the 2008 edition. In response to the new ACI provisions and new test data, PCI sponsored a comprehensive research program [Anderson et al., 2000] to study design criteria for headed stud groups loaded in shear and the combined effects of shear and tension, as used in precast construction. The result of that study was a set of design guidelines that was incorporated into the 6th Edition of the PCI Design Handbook [PCI, 2004]. Based on the new guidelines, the (unfactored) nominal design values were calculated for the three loading conditions and summarized in Table 5.2. The detailed calculations were presented previously in Chapter 4.

Table 5.2: Nominal Design Values of HCA Groups.

<table>
<thead>
<tr>
<th>Loading condition</th>
<th>Side Edge Distance (SED), mm (in.)</th>
<th>Back Edge Distance (BED), mm (in.)</th>
<th>SED/BED ratio</th>
<th>Nominal design values(^2), kN (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front-edge</td>
<td>1320 (52)</td>
<td>406.4 (16)</td>
<td>3.25</td>
<td>102.7 (23.1)</td>
</tr>
<tr>
<td>Corner</td>
<td>558.8 (22)</td>
<td>406.4 (16)</td>
<td>1.375</td>
<td>86.7 (19.5)</td>
</tr>
<tr>
<td>Side-edge</td>
<td>254 (10)</td>
<td>1930.4 (76)</td>
<td>0.13</td>
<td>167.2 (37.6)</td>
</tr>
</tbody>
</table>

1 For SED/BED < 0.2, side-edge loading condition generally controls; 0.2 < SED/BED < 3, a corner condition exists. Front-edge condition with SED/BED > 3 [PCI, 2004].

2 Based on concrete failure (steel failure at 231.3 kN or 52 kips for all cases)

5.5.3.4 ANSYS FE Model

The unreinforced concrete panel was modeled with 25.4 mm (1 in.) by 50.8 mm (2 in.) and 50.8 mm (2 in.) by 50.8 mm (2 in.) SOLID65 elements. The Hognestad stress-strain relationship with multi-linear kinematic hardening rule was specified for concrete. Other material properties for the concrete were generally the same as those used for the deep beam except for some that were adjusted for a concrete strength of 34.5 MPa (5,000 psi). The concrete cracking model was employed but the crushing model was deactivated. Figure 5.34 shows the 3-D FE
model of the HCA assembly. The studs were modeled with BEAM188 and a single layer of SOLID45 elements for the anchorage plate. The BEAM188 elements were embedded into the SOLID45 elements to achieve a near fixed condition at the stud-plate interface.

5.5.3.5 FEA Results

Table 5.3 summarizes the analytical crack patterns at the maximum load at which the FEA terminated. These maximum loads are not the failure loads due to non-convergence of solution. However, these maximum values are close to the nominal design values. More importantly, the predictions show the correct order of strength with the front-edge having the highest resistance. Each analytical model exhibited three unique crack patterns corresponding to the three different loading conditions. Based on the limited comparison, the simple modeling strategy adopted appeared to be appropriate.

5.6 FEA of EDCS

5.6.1 Model Description

5.6.1.1 Precast Concrete Cladding Panel

The spandrel panel studied is shown in the Figure 5.35. The panel is 2.13 m (7 ft) high (based on one-third height configuration). The panel height takes into account the depth of supporting beam, concrete floor slab, raise-floor requirement (in typical office), finishes, etc. and also height to openings (i.e., window). The panel is 7.32 m (24 ft) long and 203.2 mm (8 in.) thick. This width is a typical bay width (e.g., L-shaped building analyzed earlier in Chapter 3).
Figure 5.34: FE Model of HCA.

Table 5.3: Analytical Crack Patterns.

<table>
<thead>
<tr>
<th>Loading conditions</th>
<th>Crack pattern at max. load</th>
<th>Max. shear load</th>
<th>Nominal design value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front-edge</td>
<td><img src="image" alt="Front-edge Crack Pattern" /></td>
<td>115.2 kN (25.9 kips)</td>
<td>102.7 kN (23.1 kips)</td>
</tr>
<tr>
<td>Corner</td>
<td><img src="image" alt="Corner Crack Pattern" /></td>
<td>94.7 kN (21.3 kips)</td>
<td>86.7 kN (19.5 kips)</td>
</tr>
<tr>
<td>Side-edge</td>
<td><img src="image" alt="Side-edge Crack Pattern" /></td>
<td>173.0 kN (38.9 kips)</td>
<td>167.2 kN (37.6 kips)</td>
</tr>
</tbody>
</table>
According to PCI Design Handbook [PCI, 2004] Sec. 7.3.6.1, structural welded wire reinforcement (WWR) is commonly used to satisfy the serviceability requirements (i.e., shrinkage and cracking control) due to ease of placement. The wire mesh is supplemented with deformed reinforcing bars (or ties) for strength requirement. WWR can be placed in one or more layers, though one layer on each face is typical. ACI 318-02’s minimum reinforcement requirement of 0.1% is usually adequate for most precast panels. Common sizes used by precast manufacturers are WWR 4x4-W4xW4 to 6x6-W2.5xW2.5. With a 203.2 mm (8 in.) thick panel, the minimum reinforcement (0.1% of the gross area) is calculated to be 203.2 mm²/m (0.096 in²/ft). Thus, WWR 4x4-W4xW4 with steel area of 254 mm²/m (0.120 in²/ft) is adequate.

Conventional steel rebar could be used instead of WWR. It is recommended that rebar sizes should be kept reasonably small (#3 through #6) for ease of placement and for better crack and shrinkage control. The recommended maximum spacing of reinforcement in panels exposed to the environment is 152.4 mm (6 in.) for WWR and three times the panel thickness, with a maximum of 457.2 mm (18 in.), for reinforcing bars.

To transfer the in-plane applied force through the panel, different schemes of placing the extra reinforcing bars were employed. The supplementary reinforcement also helps to confine the concrete, hence preventing the development of localized cracking, especially at the highly stressed connection anchoring points.

Figure 5.35: Dimensions of Spandrel Precast Concrete Panel.
5.6.1.2 Connections

The bearing and tie-back connections are located as shown in Figure 5.35. As discussed in Chapter 4, the connections were designed for a maximum lateral load of 177.92 kN (40 kips). This load correspond to a designed slip load of 88.96 kN (20 kips) for the friction damper. This is to ensure that the friction damper would slip and operate well below the failure load of the connections. HSS6x6x1/2 was used for the bearing connections while 25.4 mm (1 in.) diameter rod was specified for the tie-back. Two configurations were used to anchor the connections to the concrete panel. In the early stages of FEA, the HSS was assumed to extend into the concrete, hence providing the required anchorage. Subsequent analyses, discussed below, would reveal generation of high stress concentration with that detail option. The result of this understanding was the use of HCA to better distribute the load.

The HCA used in the EDCS comprised of a 508 mm (20 in.) long by 304.8 mm (12 in.) wide by 50.8 mm (2 in.) thick anchorage plate with eight 22.2 mm (7/8 in.) diameter by 152.4 mm (6 in.) headed studs. Based on PCI design guidelines [PCI, 2004], the nominal design tensile and shear capacities of the HCA group were calculated as 282 kN (63.4 kips) and 94.3 kN (21.2 kips) for corner condition, respectively. Additional bars (with adequate development length) are attached to plates in two orthogonal directions to increase the shear capacity (in excess of 266.9 kN or 60 kips). A smaller HCA assembly could be used to further optimize the design.

5.6.2 ANSYS FE Model

A total of 6,003 SOLID65 elements were used for the concrete panel. The Hognestad stress-strain relationship with multi-linear kinematic hardening rule was used. The cracking model in the SOLID65 element was employed. All relevant material properties for the concrete were specified for a concrete strength of 34.48 MPa (5,000 psi). The basic FE models for the concrete panel are shown in Figure 5.36. Depending on the analysis case, different additional reinforcing steel were added.

The HSS section (for the bearing supports) was modeled with BEAM188 elements. Where anchorage plate was used (e.g., for HCA), the BEAM188 elements were embedded into the SOLID45 (steel plate) elements. This was required to prevent unnecessary rotations at the plate-beam interface. It should be pointed out that the “stick” BEAM188 element has practically
zero “foot-print” that could lead to unrealistically high stress concentration occurring in the adjacent SOLID45 plate elements leading to excessive flexure of the plate. In reality, the (welded) base of the actual HSS covered a finite area of the plate and this portion of the plate could be considered as infinitely stiff; any significant plate bending would occur outside this area. Thus, it would not be inaccurate to specify a high rigidity for the SOLID45 plate elements confined within this area. Except for the size, the FE model for the HCA was similar to that reported in Section 5.5.3.

The threaded-rod for the tie-back connection was modeled with BEAM188 element. It could have been defined with the simpler LINK8 spar element since its response was mainly uniaxial tension-compression. The LINK8 element was specified for all reinforcing bars in the concrete and placed in accordance to the layout specified for each analysis case (as discussed in the next section). A concrete cover of 25.4 mm (1 in.) was used.

The nonlinear behavior of the friction damper is generally well-understood and could be modeled using ANSYS COMBI165 spring-damper element. Adding the friction damper in the FE

Figure 5.36: Basic FE Models of EDCS.
model would effectively limit the maximum load of the EDCS to the damper slip load. In this case, the EDCS components (i.e., concrete panel and connections) would merely respond elastically as designed. Although not critical, it would be more useful to obtain information about the inelastic response of these components, for example the onset of inelastic response. Thus, the friction damper was excluded from the FE model to achieve higher loads and inelastic component response.

A concentrated load and moment were simultaneously concentrically applied to the surface of the anchorage plate. The moment accounted for the out-of-plane load eccentricity of 254 mm (10 in.) For the bearing supports, the free end of the BEAM188 element was specified as pinned or fixed for support A, depending on the analysis case, and roller (horizontal direction) condition for support B. The tie-back is capable of resisting out-of-plane force only.

5.6.3 Analysis Cases

The FEA cases carried out are summarized in Table 5.4. There were a total of 8 cases corresponding to different reinforcing layout and connection details.

5.6.4 Analysis Results

5.6.4.1 Load-Deformation Characteristics

The load-deformation plots are shown in Figure 5.37; the deflection was measured at the loading point. From these plots, the initial tangent stiffness, maximum load, the elastic-limit load (i.e., load at the onset of nonlinear response) for each case are summarized in Table 5.5. The numerical results clearly showed that the extent of cracking at the connections controlled the maximum load at which each analysis could reach. The analytical crack patterns for each model are presented in Appendix B. All analysis cases did not achieve failure due to non-convergence of solution. A summary of the results is provided below.
Table 5.4: Precast Panel Analysis Cases.

<table>
<thead>
<tr>
<th>Label</th>
<th>Model details</th>
</tr>
</thead>
<tbody>
<tr>
<td>cp1</td>
<td>Concrete (SOLID65) (\rightarrow) WWR (LINK8)</td>
</tr>
<tr>
<td>cp2</td>
<td>Concrete (SOLID65) (\rightarrow) WWR (LINK8) (\rightarrow) 4-7 diagonal bars (LINK8)</td>
</tr>
<tr>
<td>cp3</td>
<td>3 layers of #6@4&quot; (LINK8) (\rightarrow) Concrete (SOLID65) (\rightarrow) 2 layers of #3@4&quot; (LINK8) (\rightarrow) 3 layers of #6@4&quot; (LINK8)</td>
</tr>
<tr>
<td>cp4</td>
<td>3 layers of #6@4&quot; (LINK8) (\rightarrow) Concrete (SOLID65) (\rightarrow) 2 layers of #3@4&quot; (LINK8) (\rightarrow) 3 layers of #6@4&quot; (LINK8) (\rightarrow) HCA (SOLID45 and BEAM188)</td>
</tr>
</tbody>
</table>
Table 5.4: Precast Panel Analysis Cases (Cont’d).

<table>
<thead>
<tr>
<th>Label</th>
<th>Model details</th>
</tr>
</thead>
<tbody>
<tr>
<td>cp5</td>
<td>Identical to cp4 except that no HSS bearing support was defined at support A. Boundary conditions was applied directly to the anchoring plates of HCA.</td>
</tr>
<tr>
<td>cp6</td>
<td>Identical to cp4 except that the end condition for HSS bearing support A was fixed.</td>
</tr>
<tr>
<td>cp7</td>
<td>Identical to cp5 except for all mid-layers of reinforcing bars removed.</td>
</tr>
<tr>
<td>cp8</td>
<td>Identical to cp6 except for all mid-layers of reinforcing bars removed.</td>
</tr>
</tbody>
</table>

Figure 5.37: Load-Deformation Curves for Precast Panels.
1. cp1 was modeled to represent conventional architectural precast cladding panel. It was specified with minimum reinforcement with no additional steel reinforcing bars near the connections. The FEA for cp1 terminated at a very low load (9.2 kips) due to extensive cracking around the loading point. Although this load may not necessary be the ultimate lateral load capacity of the panel, the stiffness of the panel was deteriorating rapidly beyond this load level. This was not desirable since the concrete panel was expected to behave essentially elastic up to about 40 kips. The analytical crack pattern (Appendix C) observed was consistent with a corner failure.

2. It was initially thought that diagonal bars (or sections) connected directly between the pinned support and the loading point would provide a direct load path, hence considerably improving the lateral load carrying capacity. As observed from case cp2, the increase in lateral stiffness was insignificant. Based on the stress results in the diagonals, it was found that the diagonal bars or struts would only be effective if the bars were debonded from the concrete. With the bars bonded to the concrete mass, the bar force would be completely transferred to the concrete mass along the developed length of bar. The developed length for #7 Grade 60 bars and 34.48 MPa (5,000 psi) concrete is only 1.22 m (4 ft), compared to the panel length of 7.32 m (24 ft).

3. In cp3, by adding reinforcing bars at the loaded regions, the FE model was able to reach higher load prior to termination of the analysis due to a lesser extent of cracking. The

<table>
<thead>
<tr>
<th>Label</th>
<th>Initial tangent stiffness</th>
<th>Elastic-limit load</th>
<th>Maximum load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN/mm</td>
<td>kips/in</td>
<td>kN</td>
</tr>
<tr>
<td>cp1</td>
<td>43</td>
<td>243</td>
<td>20.9</td>
</tr>
<tr>
<td>cp2</td>
<td>45</td>
<td>257</td>
<td>20.9</td>
</tr>
<tr>
<td>cp3</td>
<td>51</td>
<td>292</td>
<td>46.7</td>
</tr>
<tr>
<td>cp4</td>
<td>40</td>
<td>226</td>
<td>82.3</td>
</tr>
<tr>
<td>cp5</td>
<td>489</td>
<td>2,792</td>
<td>177.9</td>
</tr>
<tr>
<td>cp6</td>
<td>191</td>
<td>1,089</td>
<td>180.1</td>
</tr>
<tr>
<td>cp7</td>
<td>41</td>
<td>233</td>
<td>87.2</td>
</tr>
<tr>
<td>cp8</td>
<td>191</td>
<td>1,088</td>
<td>180.1</td>
</tr>
</tbody>
</table>

Table 5.5: Analytical Initial Tangent Stiffness, Elastic-limit Load and Maximum Load.
elastic-limit load had increased by more than two times though it only improved the initial stiffness marginally by 20%.

4. Embedding the HSS into the concrete resulted in high stress concentration and early reduction in lateral stiffness as observed in cases cp1 to cp3. The introduction of the HCA in cp4, to better distribute the anchoring force, significantly increased the elastic-limit load by as much as 76%. A slight reduction (23%) in overall stiffness was not expected since the HSS exhibited higher flexure stiffness than the headed stud anchor.

5. Changing the support condition of the cantilevered bearing supports from pinned (cp4) to fixed (cp6) led to the greatest increase (more than five times) in the lateral stiffness. This significant reduction is not unreasonable since the stiffness of a cantilever beam bending in double-curvature is at four times that of a beam bending in simple curvature. Removing the (right) cantilevered bearing support, in cp5, resulted in a two times increase in stiffness, compared to cp6. In practice, the connection detail (pinned bearing support with welded shear plates) shown in Figure 5.38 (b) could be used to achieve a high out-of-plane fixity.

6. An additional mid-layer of reinforcing bars was added with the intention of confining the concrete around the loaded points. Comparing cp4 and cp6 with cp7 and cp8, respectively, the results indicate no significant reduction in the in-plane stiffness and strength as the result of omitting the center layer. The conventional two layers, one on each face, seems adequate.

![Figure 5.38: Bearing Support with Shear Plates.](image)
For cp8, at 40 kips (i.e., twice the designed slip load), only minor cracks formed in the surface concrete elements adjacent to the loaded HCA as shown in Figure 5.39. The depth of these cracks was confined to the thickness of the concrete cover and hence would not cause any stiffness or strength degradation. Thus the reinforcement and connection details specified in cp8 would allow the EDCS (except for the friction damper) to behave elastically as required. It should be pointed out that cp8 represents only one of the several possible solutions. Further optimization of the EDCS, especially the connection and reinforcement details, is possible but is beyond the scope of the present study.

Figure 5.39: Concrete Cracking at 40 kips for cp8.

Figure 5.40 shows the displacement contours of cp8 in the global x-, y- and z-directions at 40 kips. The observed twisting effect of the panel was due to the eccentric loading and the flexibility of the bearing supports. The largest deflection of 2.0 mm (0.0792 in.) was observed in the z-direction at the top edge of the panel.
5.6.4.2 Principal Stresses

The stress trajectory plots of the principal stresses for model cp8 are presented in Figure 5.41. The arrows indicate the general direction of the vector in the principal direction 1. Moving away from the loaded connections, the stress trajectories became uniform and predominately uniaxial suggesting the presence of an undisturbed or B-region. On the other hand, the trajectories in the vicinity (within a distance of eight times the panel thickness) of the connections were highly directional. For short spandrel panel (with length less than two times the height) or panels with openings, the principal stress plots would provide valuable information about the load path within the panel and possibly help in developing Strut-and-Tie models for determining the reinforcement requirements.

Figure 5.42 shows the maximum and minimum principal stress contours of the panel at 40 kips. The contours show that the panel between the connections is predominately under uniaxial tension. The plots also show the extent of stress concentration around each connection.

5.7 SUMMARY

Existing literature suggested that ANSYS (Version 9) with its SOLID65 concrete element is capable of modeling the complex behavior of structural concrete, subjected to a number of limitations. The results of the present FE study reaffirmed its suitability. The main challenge of using ANSYS for analyzing reinforced concrete is solution non-convergence which appeared to be the result of the use of crack model in the concrete element. The stress-strain relationship and the cracking model are important parameters in modeling reinforced concrete structures. Based on the FEA of a deep beam and HCA, the proposed constitutive models for reinforced concrete and the modeling strategies appeared to be adequate. The results from the calibration study were extended to the analysis of proposed EDCS. The stiffness and strength characteristics of the EDCS for different reinforcement and connection details were investigated and a feasible EDCS design was found. In the next chapter, rational methods of estimating the stiffness and strength of the EDCS are presented.
Figure 5.40: Panel Displacement Contours for Model cp8.
Figure 5.41: Principal Stresses Vector Plot.
Figure 5.42: Maximum and Minimum Principal Stresses Contours.
5.8 CONCLUSION

The generally good correlations between the analytical and experimental results indicates that the concrete properties estimated from existing design guidelines (e.g., ACI 318 [ACI 318, 2006]) could be used for the FE model without any modification. Without an explicit concrete crack model, even with a nonlinear uniaxial stress-strain relationship, the analytical models for the simply-supported deep beam tend to exhibit a significantly stiffer response than the test beam. The crack model in SOLID65 concrete element appeared to be satisfactory. The specification of a descending segment in the modified Hognestad stress-strain relationship for concrete softening appeared to give slightly better correlations with the test result, especially near the failure load. But due to non-convergence problem, it was not used for subsequent FEA. The crushing model in SOLID65 element may improve the correlations, especially at high load levels, but persistent non-convergence issue would deter its use. For the FEA of precast concrete panels, since the focus is mainly on the elastic response region, significant localized crushing at high loading need not be considered for the present study.

The present FE modeling approach for the HCA is simple and appears to be appropriate. Although none of the analysis cases reached the failure, the maximum load at which the analysis stopped gave good indication of the relative strength of different loading conditions. The analytical crack patterns were generally consistent with the loading conditions. As discussed in Chapter 4, except for the friction damper, all components (i.e., concrete panel, bearing connections, etc.) of the EDCS are designed to perform elastically at the maximum lateral load dictated by the designed slip load of the damper. Thus an accurate prediction of the failure load of the HCA is not crucial for subsequent FEA on the EDCS.

A number of FE models for the proposed EDCS were created and analyzed using ANSYS. These models were identical except for the different connections and reinforcement details used. The nonlinear FEA showed that a precast concrete panel can be designed to remain essentially elastic up to the slip load of the friction damper if adequate reinforcement and proper detailing of the connections are provided. It was found that the diagonal bars or brace placed between the support and the load application would only be effective if the bars were debonded from the concrete. With the bars bonded to the concrete mass, the bar force would be completely transferred to the concrete mass along the developed length of bar due to the length of the spandrel panel. The results from the FEA showed that embedding the HSS into the concrete resulted in high stress concentration and early reduction in lateral stiffness. The HCA would
provide better distribution of anchoring force than embedded HSS section, significantly increased the elastic-limit load by as much as 76%. Changing the support condition of the cantilevered bearing supports from pinned to fixed led to as much as five times increase in the lateral stiffness of the panel. The additional center layer of reinforcing bars was not necessary because the results indicate no significant reduction in the in-plane stiffness and strength as the result of omitting the center layer; the conventional two layers, one on each face, appears adequate. The principal stress trajectory plots and contours showed that a significant portion of the spandrel panel was under uniform stress distribution. This suggests the presence of an undisturbed or B-region. On the other hand, the trajectories in the vicinity (within a distance of eight times the panel thickness) of the connections were highly directional. From the FEA, one possible EDCS design was identified that would allow the proposed system to behave elastically with no or negligible concrete cracking.
CHAPTER 6
COMPONENT LEVEL STUDY – MATHEMATICAL MODEL

6.1 CHAPTER OVERVIEW

In the previous chapter, the FE technique was used in the design and detailing of the EDCS. The numerical results were used to refine the connection details and reinforcement schemes that would meet both the stiffness and strength aspects. While the results supported the EDCS concept as capable of functioning both as a structural brace and an energy absorbing device, the performance of the EDCS as a passive seismic protection system must be evaluated at the building level. Despite the fact that it is possible to model the EDCS’s and a representative building or structure using FEA techniques discussed earlier, it would be more logical (and efficient) to first reduce the FE model of the EDCS to a simpler mathematical formulation. The proposed mathematical model must be sufficiently detailed to accurately capture the in-plane load-deformation characteristics of the EDCS, yet simple enough to allow it to be easily added to the mathematical model of any buildings or structures. The objective of this stage is to develop a simple and accurate mathematical representation of the in-plane load-deformation characteristics of the EDCS for subsequent use in building level analyses (discussed in Chapter 7).

Following the chapter overview, the simplified mathematical model of the EDCS is discussed. Next, the load-deformation characteristic of the EDCS is broken down into its major contributing components. The elastic stress distribution in the reinforced concrete panel is then investigated next. This leads to the development of an approximate method to predict the panel stiffness. A series of FEA simulations were performed to verify the accuracy of the proposed approximate methods and to establish its range of applicability. Finally, the stiffness characteristics of the bearing support and its estimation is reported.
6.2 SIMPLIFIED EDCS MODEL

To achieve numerical accuracy, a single 7.9 m (26 ft) long by 2.1 m (7 ft) high by 203 mm (8 in.) thick EDCS unit was modeled with more than 10,000 three-dimensional elements and took more than eight hours for a single analysis on a medium performance desktop computer. A multi-story building with multiple bays incorporating the EDCS would easily require hundred of units to clad the entire building envelope. It would be almost computationally impossible to model the individual EDCS using the approach outlined in Chapter 5. Instead, the strategy adopted here is to represent the in-plane behavior of the EDCS as a two dimensional assemblage of truss and spring elements as shown in Figure 6.1. The (infinitely) rigid truss elements only serve to transmit the axial forces developed in the EDCS to the supporting beam and column. The axial deformations in these elements are made negligible and are oriented as shown to correctly replicate the direction of the forces at the connections. The entire load-deformation characteristic of the EDCS is presented here by a single elastic perfectly-plastic spring element. The properties of the bilinear spring are discussed next.

Figure 6.1: Simplifying the Mathematical Model of EDCS.
6.3 COMPONENTS OF EDCS IN-PLANE STIFFNESS

The overall lateral stiffness ($K_{EDCS}$) of EDCS can be considered as comprising of a series of springs, each capturing the load-deformation of a major component of the system. As shown in Figure 6.2, the major components contributing to the lateral stiffness of the assembly are the reinforced concrete panel ($K_{concrete}$), the bearing support ($K_{bearing}$), the headed stud assemblies ($K_{stud}$), and the damper assembly ($K_{damper}$). While the stiffnesses of the concrete panel and the (cantilevered) bearing supports may be approximated by theory of elasticity (discussed later in Section 6.5), the load-deformation characteristic of the concrete anchorage (i.e., headed studs and plate) is more complex as it involved the interaction between the embedded studs and concrete matrix. One way to overcome this difficulty is to treat the concrete panel and headed stud assemblies as a single component ($K_{panel}$) in the stiffness calculation.

![Diagram of EDCS components](image)

Figure 6.2: Components of EDCS Stiffness.

The individual springs act together in series based on the load path. The overall lateral stiffness of the EDCS ($K_{EDCS}$) is related to the individual stiffnesses ($k_i$) by

$$K_{EDCS} = \frac{1}{\sum_{All} \frac{1}{k_i}}$$  \hspace{1cm} (6.1)
The component stiffnesses discussed so far are plotted in Figure 6.3. Although the ultimate failure load of each major component of the EDCS could be found (from prevailing design codes), it is not necessary to construct the entire load-deformation curve since the lateral load in the EDCS is capped at the design slip load of friction damper as illustrated. All other components are designed to respond elastically throughout the operating range of the damper. The estimation of the stiffness of the individual component is discussed next. The load-deformation of the friction damper is well documented in literature and hence would not be covered here.

![Figure 6.3: Load-Deformation of EDCS Components.](image)

### 6.4 CONCRETE PANEL

#### 6.4.1 Normal Stress Distribution

Before developing a rational method to estimate the in-plane stiffness of the concrete panel, it would be first necessary to understand the elastic stress distribution of the panel. The
strain and the stiffness characteristics ($K_{\text{concrete}}$) of the panel can then be found from fundamental theory of elasticity.

### 6.4.1.1 Theoretical Solution for Elastic Stress Distribution

With sufficient distance (e.g., panel depth) away from the disturbed regions (i.e., supports and loading point), from Saint-Venant’s principle, the (elastic) normal stress distribution at any cross-section along the panel, between supports A and D (Figure 6.4), is approximately linear over the panel height. The equations describing the elastic stress distribution can be found from static equilibrium.

![Figure 6.4: Free-Body Diagram of Rectangular Panel.](image)

From static equilibrium of the whole panel, the vertical reaction at support A, $F_{A,y}$, is equal to

$$F_{A,y} = \frac{(H - a_y - a_y)}{(L - 2b_y)} P = \beta P \quad (\text{6.2})$$

where $H$, $L$, $a_y$, $a_y$, and $b_y$ are panel dimensions: panel height, panel length, vertical edge distance of loading point, vertical edge distance of bearing support, and horizontal edge distance of bearing support, respectively. $P$ is the applied horizontal load, and $\beta$ is defined as the slope of the line connecting supports A and D. Consider only the free-body diagram of the left segment of the panel. Now $\Sigma F_x = 0$ gives
where \( f_1 \) and \( f_2 \) are the normal stresses at top and bottom of the panel, respectively; \( t \) is the panel thickness. And \( \Sigma F_y = 0 \) gives

\[
V = F_{A,y} = \beta P \quad (6.4)
\]

where \( V \) is the shear force along the cross-section XX. The shear stress is assumed to be uniformly distributed over the panel height. \( \Sigma M_z = 0 \) at support A gives

\[
V_x = 0.5(f_1 + f_2)Ht\left( \bar{y} - a_b \right) \quad (6.5)
\]

where \( \bar{y} \) is the height to the centroid of the trapezoidal concrete stress block and is given by

\[
\bar{y} = \frac{H(2f_1 + f_2)}{3(f_1 + f_2)} \quad (6.6)
\]

and \( x \) is the distance from support A to the cross-section XX. Substituting Equations 6.4 and 6.6, Equation 6.5 can be rewritten as,

\[
\beta Px = 0.5(f_1 + f_2)Ht\left[ \frac{H(2f_1 + f_2)}{3(f_1 + f_2)} - a_b \right] \quad (6.7)
\]

Rearranging,

\[
\frac{6\beta Px}{Ht} = \left[ H(2f_1 + f_2) - 3a_b(f_1 + f_2) \right] \quad (6.8)
\]

\[
\frac{6\beta Px}{Ht} = f_1(2H - 3a_b) + f_2(H - 3a_b) \quad (6.9)
\]

Substituting Equation 6.3 and eliminating \( f_1 \), we have

\[
\frac{6\beta Px}{Ht} = \left( \frac{2P}{Ht} - f_2 \right)(2H - 3a_b) + f_2(H - 3a_b) \quad (6.10)
\]
Simplifying gives

\[ f_1 = \frac{2P_x}{H^2 t} \left[ 3(a_b + \beta x) - H \right] \] (6.11)

\[ f_2 = \frac{2P_x}{H^2 t} \left[ 2H - 3(a_b + \beta x) \right] \] (6.12)

The normal stress distributions calculated from Equations 6.11 and 6.12 are plotted along the panel as shown in Figure 6.5. It should be pointed out that this simple stress distribution may not apply to the disturbed regions (D-region) where localized stress concentration would exist. For short panel (e.g., length is less than two times the depth), the entire panel would be in the D-region and the deep beam analogy may be more appropriate. However, for most spandrel cladding, the length to depth ratio generally exceeds four times.

![Analytical Elastic Normal Stress Distribution](image)

**Figure 6.5**: Analytical Elastic Normal Stress Distribution.

### 6.4.1.2 Comparison with FEA Result

To check the applicability of Equations 6.11 and 6.12, the analytical result for the FE model cp5 (in Chapter 5) of a typical reinforced concrete panel (Figure 6.6) was analyzed. Model cp5 was modeled without the two bearing connections (i.e., HSS section) to remove the effect of
the cantilevered bearing connections. The loading and boundary conditions were assigned directly to the anchoring plate of the HCA’s. The dimensions for this panel are summarized in Table 6.1.

![Diagram of concrete elements](image)

**Figure 6.6**: FE Model CP5.

<table>
<thead>
<tr>
<th>No.</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(mm)</td>
</tr>
<tr>
<td>1</td>
<td>Panel height</td>
<td>$H$</td>
<td>2133.6</td>
</tr>
<tr>
<td>2</td>
<td>Panel length</td>
<td>$L$</td>
<td>7315.2</td>
</tr>
<tr>
<td>3</td>
<td>Panel thickness</td>
<td>$t$</td>
<td>203.2</td>
</tr>
<tr>
<td>4</td>
<td>Vertical edge distance of loading point</td>
<td>$a_t$</td>
<td>355.6</td>
</tr>
<tr>
<td>5</td>
<td>Vertical edge distance of bearing support</td>
<td>$a_b$</td>
<td>660.4</td>
</tr>
<tr>
<td>6</td>
<td>Horizontal edge distance of loading point</td>
<td>$b_t$</td>
<td>304.8</td>
</tr>
<tr>
<td>7</td>
<td>Horizontal edge distance of bearing support</td>
<td>$b_b$</td>
<td>965.2</td>
</tr>
<tr>
<td>8</td>
<td>Slope</td>
<td>$\beta$</td>
<td>0.2075</td>
</tr>
</tbody>
</table>

The lateral load-deformation curve of model cp5 is shown in Figure 6.7. As mentioned earlier, the maximum load of 275.3 kN (61.9 kips) is not the failure load; it is the load at which the FEA terminated due to non-convergence of solution. As a result of substantial concrete cracking beyond 177.9 kN (40 kips), the stiffness of the panel was reduced by 47%, from 489 kN/mm (2,792 kips/in.) to 260 kN/mm (1,485 kips/in.) The panel stress distributions were
investigated at two loads, 35.6 kN (8 kips) and 275.5 kN (61.9 kips). With the elastic-limit of the panel at about 160.1 kN (36 kips), the stress distribution at the two loads should show noticeable differences, especially at the D-regions, due to redistribution of stress from concrete cracking.

Figure 6.8 shows the normal stress distributed based on FEA for two load levels; while consistent with Saint-Venant’s principle, there exists an undisturbed B-region where the elastic stress distributions were nearly linear. The (shaded) disturbed or D-regions extended about eight times the thickness of the panel away from the loaded points. Therefore, the assumption that the D-region covered a distance equal to the overall height of the panel, \( H \), would be conservative. The extent of the D-region did not change appreciably at the higher load level. However, at 275.5 kN (61.9 kips), the nonlinear stress distribution profiles in the D-regions have changed significantly due to stress redistribution from concrete cracking.

Figure 6.9 compares the stress distribution at selected cross-sections in the B-regions predicted by Equations 6.11 and 6.12 with that based on the FEA results. The closed-form solutions appear to correlate well with the numerical results for the distribution in the B-region. In
the D-regions between the supports A and D, the normal stress distribution from the FEA generally followed the linear profiles described by Equations 6.11 and 6.12, except for small localized region near the HCA. This suggests that the in-plane load-deformation of the panel can be approximated by the elastic beam theory.

Figure 6.8: Normal Stress Distribution.
Figure 6.9: Correlation of Stress Distributions in the B-Region.
### 6.4.2 Lateral Stiffness Estimation

The approach is to idealize the concrete panel as a cantilevered beam. The overall in-plane response of the concrete panel can be separated into three distinct modes, namely axial, shear and flexure as shown in the Figure 6.10. An axial component exists because, for the present study, one of the bearing supports was designed as a roller.

![Concrete Panel Response Modes](image.png)

**Figure 6.10**: Concrete Panel Response Modes.

The stiffnesses for the three modes of responses can be represented by

**(Axial)**

\[
K_{c, \text{axial}} = \frac{EA'}{l'}
\]  \hspace{1cm} (6.13)

**(Shear)**

\[
K_{c, \text{shear}} = \frac{2GA'}{3h'}
\]  \hspace{1cm} (6.14)

**(Flexure)**

\[
K_{c, \text{flexure}} = \frac{3EI'}{h'^2}
\]  \hspace{1cm} (6.15)

$h', l', A', A'_{1}, A'_{2}$ and $I'$ are the “effective” panel height, length, cross-section areas and moment of inertia, respectively. $E$ and $G$ are the modulus of elasticity of reinforced concrete in tension and shear, respectively. The above equations are based on elementary theory of elasticity.
assumptions of small deflections and plane sections remaining plane after bending [Timoshenko and Goodier, 1970]. From the approximate linear stress distribution, it seems appropriate to use the dimensions between the supports (i.e., point A and D in Figure 6.4) for the above equations, although it is also not unreasonable to use the overall dimensions of the panels in the above stiffnesses formulation. The overall lateral stiffness of the concrete plane can be determined from the following expression:

\[ K_{\text{Concrete}} = \frac{1}{\frac{1}{K_{C, \text{axial}}} + \frac{1}{K_{C, \text{shear}}} + \frac{1}{K_{C, \text{flexure}}}} \]  \hspace{1cm} (6.16)

### 6.4.3 Correlation with FEA

#### 6.4.3.1 Model Description

To investigate the validity and applicability of the above equations, a new series (pk) of FEA was performed. A total of thirteen models were created in ANSYS. Each of these models was similar to cp5 (with respect to reinforcement layout and connection details) except for the panel dimensions, concrete compressive strength and locations of the HCA’s. These parameters were varied so that each model gave a slightly different lateral stiffness value using the equations discussed above. Table 6.2 summarizes the parameter that was changed for each model. pk1 was taken as the control case for this series.
Table 6.2: FE Models of Cladding Panel.

<table>
<thead>
<tr>
<th>Model</th>
<th>Front View/ Comments</th>
<th>Side</th>
</tr>
</thead>
<tbody>
<tr>
<td>pk1</td>
<td>Control L: 7315.2 mm (288”); H: 2133.6 mm (84”); t: 203.2 mm (8”)</td>
<td></td>
</tr>
<tr>
<td>pk2</td>
<td>Thickness reduced from 203.2 mm (8”) to 154.2 mm (6”)</td>
<td></td>
</tr>
<tr>
<td>pk3</td>
<td>Thickness increased from 203.2 mm (8”) to 254.0 mm (10”)</td>
<td></td>
</tr>
<tr>
<td>pk4</td>
<td>Height increased from 2133.6 mm (84”) to 3352.8 mm (132”)</td>
<td></td>
</tr>
</tbody>
</table>
### Table 6.2: FE Models of Cladding Panel (Cont’d).

<table>
<thead>
<tr>
<th>Model</th>
<th>Front View/ Comments</th>
<th>Side</th>
</tr>
</thead>
<tbody>
<tr>
<td>pk5</td>
<td><img src="image" alt="Model pk5" /></td>
<td><img src="image" alt="Side View pk5" /></td>
</tr>
<tr>
<td></td>
<td>Length reduced from 7315.2 mm (288&quot;) to 4572.0 mm (180&quot;)</td>
<td></td>
</tr>
<tr>
<td>pk6</td>
<td><img src="image" alt="Model pk6" /></td>
<td><img src="image" alt="Side View pk6" /></td>
</tr>
<tr>
<td></td>
<td>Concrete strength reduced from 34.5 MPa (5 ksi) to 27.6 MPa (4 ksi)</td>
<td></td>
</tr>
<tr>
<td>pk7</td>
<td><img src="image" alt="Model pk7" /></td>
<td><img src="image" alt="Side View pk7" /></td>
</tr>
<tr>
<td></td>
<td>Concrete strength increased from 34.5 MPa (5 ksi) to 41.4 MPa (6 ksi)</td>
<td></td>
</tr>
<tr>
<td>pk8</td>
<td><img src="image" alt="Model pk8" /></td>
<td><img src="image" alt="Side View pk8" /></td>
</tr>
<tr>
<td></td>
<td>(Bottom) bearing supports moved towards bottom edge by 203.2 mm (8&quot;)</td>
<td></td>
</tr>
<tr>
<td>pk9</td>
<td><img src="image" alt="Model pk9" /></td>
<td><img src="image" alt="Side View pk9" /></td>
</tr>
<tr>
<td></td>
<td>(Bottom) bearing supports moved towards side edges by 304.8 mm (12&quot;)</td>
<td></td>
</tr>
</tbody>
</table>
Table 6.2: FE Models (Cont’d).

<table>
<thead>
<tr>
<th>Model</th>
<th>Front View/ Comments</th>
<th>Side</th>
</tr>
</thead>
<tbody>
<tr>
<td>pk10</td>
<td>(Top) loading point moved away from top edge by 203.2 mm (8&quot;)</td>
<td></td>
</tr>
<tr>
<td>pk11</td>
<td>(Top) loading point moved away from edge by 203.2 mm (8&quot;)</td>
<td></td>
</tr>
<tr>
<td>pk12</td>
<td>Height increased from 2133.6 mm (84&quot;) to 2743.2 mm (108&quot;)</td>
<td></td>
</tr>
<tr>
<td>pk13</td>
<td>Length reduced from 7315.2 mm (288&quot;) to 6096.0 mm (240&quot;)</td>
<td></td>
</tr>
</tbody>
</table>

6.4.3.2 FEA Results

Load-deformation Characteristics

Figure 6.11 shows the lateral load-deformation curves of all the thirteen FE models. With the exception of pk6, all curves are linear up to 155.7 kN (35 kips). pk6 was specified a low concrete compressive strength which probably led to significant concrete cracking and reduction of stiffness at a lower load. With the maximum lateral force in the EDCS limited to the 89.0 kN (20 kips) slip load of the friction damper, the EDCS would be expected to remain elastic. In any case, the elastic-limit could be increased by welding additional tail bars to HCA and refining the confinement details. Table 6.3 summarizes the initial tangent stiffness, maximum load, the elastic-limit load for each model.
Concrete Panel Stiffness

By applying regression techniques on the numerical stiffness results, the “effective” dimensions for Equations 6.13 through 6.15 could be identified. For the axial stiffness, the area ($A_1$) was taken as the panel gross cross-sectional area. This is reasonable since linear stress distribution extended from the top edge to the bottom edge of the panel. The horizontal distance between left pin support A and load application point D was used as the “effective” length ($l'$).

For shear response, the shear modulus of elasticity ($G$) was taken to be 40% of the Young’s modulus [ACI 318, 2005]. The shear area ($A_2$) was taken as the entire plan area of the panel. The “effective” height ($h'$) for both shear and flexural response was taken to be the vertical distance

Figure 6.11: PK Series Load-Deformation Curves.
between points A and D. After substituting the “effective” dimensions, the following equations were obtained

\[
K_{c, \text{ axial}} = \frac{EA'}{l'} = \frac{E(Ht)}{(L - b_b - b_i)} \quad (6.17)
\]

\[
K_{c, \text{ shear}} = \frac{2GA'}{3h'} = \frac{2(0.4E)[L]}{3(H - a_b - a_i)} \quad (6.18)
\]

\[
K_{c, \text{ flexure}} = \frac{3EI'}{h'^3} = \frac{3E(L't/12)}{(H - a_b - a_i)^3} \quad (6.19)
\]

**Table 6.3:** Analytical Initial Tangent Stiffness, Elastic-limit Load and Maximum Load.

<table>
<thead>
<tr>
<th>Model</th>
<th>Initial tangent stiffness, $K_{\text{panel}}$</th>
<th>Elastic-limit load</th>
<th>Maximum load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN/mm</td>
<td>kips/in</td>
<td>kN</td>
</tr>
<tr>
<td>pk1</td>
<td>387</td>
<td>2,212</td>
<td>160.1</td>
</tr>
<tr>
<td>pk2</td>
<td>309</td>
<td>1,762</td>
<td>133.4</td>
</tr>
<tr>
<td>pk3</td>
<td>457</td>
<td>2,611</td>
<td>177.9</td>
</tr>
<tr>
<td>pk4</td>
<td>443</td>
<td>2,530</td>
<td>169.0</td>
</tr>
<tr>
<td>pk5</td>
<td>572</td>
<td>3,269</td>
<td>160.1</td>
</tr>
<tr>
<td>pk6</td>
<td>355</td>
<td>2,027</td>
<td>62.3</td>
</tr>
<tr>
<td>pk7</td>
<td>419</td>
<td>2,395</td>
<td>173.5</td>
</tr>
<tr>
<td>pk8</td>
<td>380</td>
<td>2,168</td>
<td>160.1</td>
</tr>
<tr>
<td>pk9</td>
<td>375</td>
<td>2,141</td>
<td>160.1</td>
</tr>
<tr>
<td>pk10</td>
<td>411</td>
<td>2,347</td>
<td>173.5</td>
</tr>
<tr>
<td>pk11</td>
<td>394</td>
<td>2,252</td>
<td>155.7</td>
</tr>
<tr>
<td>pk12</td>
<td>426</td>
<td>2,435</td>
<td>164.6</td>
</tr>
<tr>
<td>pk13</td>
<td>456</td>
<td>2,604</td>
<td>160.1</td>
</tr>
</tbody>
</table>

The edge distances for the headed studs anchors (HCA), $a_b$, $a_i$, $b_b$, and $b_i$ were previously defined in Figure 6.4. $E$ was calculated from Equation 5.2. The overall lateral stiffness of the concrete panel alone (i.e., excluding the deformation of the HCA) was determined from Equation 6.16.
The calculated stiffness values for each model are summarized in Table 6.4. The flexural stiffnesses are about two orders of magnitude higher than the axial and shear values. It should be pointed out that numerical stiffness values (from ANSYS) in Table 6.4 did not take into account the deformation of the HCA’s while the initial tangent stiffnesses reported in Table 6.3 (and Table 5.5) included the HCA’s stiffness.

Table 6.4: In-plane Stiffness of Concrete Panels.

<table>
<thead>
<tr>
<th>Model</th>
<th>$K_{C,\text{axial}}$</th>
<th>$K_{C,\text{shear}}$</th>
<th>$K_{C,\text{flexure}}$</th>
<th>$K_{\text{Concrete}}$</th>
<th>ANSYS Result</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$x 10^3$ kN/mm</td>
<td>$x 10^4$ kips/in</td>
<td>$x 10^3$ kN/mm</td>
<td>$x 10^4$ kips/in</td>
<td>$x 10^3$ kN/mm</td>
</tr>
<tr>
<td>pk1</td>
<td>2.05</td>
<td>1.17</td>
<td>1.62</td>
<td>5.63</td>
<td>5.27</td>
</tr>
<tr>
<td>pk2</td>
<td>1.54</td>
<td>0.88</td>
<td>7.39</td>
<td>4.22</td>
<td>2.98</td>
</tr>
<tr>
<td>pk3</td>
<td>2.56</td>
<td>1.46</td>
<td>12.33</td>
<td>7.04</td>
<td>4.96</td>
</tr>
<tr>
<td>pk4</td>
<td>3.20</td>
<td>1.83</td>
<td>4.71</td>
<td>2.69</td>
<td>0.44</td>
</tr>
<tr>
<td>pk5</td>
<td>3.82</td>
<td>2.18</td>
<td>6.16</td>
<td>3.52</td>
<td>0.96</td>
</tr>
<tr>
<td>pk6</td>
<td>1.82</td>
<td>1.04</td>
<td>8.81</td>
<td>5.03</td>
<td>3.54</td>
</tr>
<tr>
<td>pk7</td>
<td>2.24</td>
<td>1.28</td>
<td>10.81</td>
<td>6.17</td>
<td>4.34</td>
</tr>
<tr>
<td>pk8</td>
<td>2.05</td>
<td>1.17</td>
<td>8.34</td>
<td>4.76</td>
<td>2.40</td>
</tr>
<tr>
<td>pk9</td>
<td>1.94</td>
<td>1.11</td>
<td>9.86</td>
<td>5.63</td>
<td>3.96</td>
</tr>
<tr>
<td>pk10</td>
<td>2.05</td>
<td>1.17</td>
<td>12.05</td>
<td>6.88</td>
<td>7.23</td>
</tr>
<tr>
<td>pk11</td>
<td>2.14</td>
<td>1.22</td>
<td>9.86</td>
<td>5.63</td>
<td>3.96</td>
</tr>
<tr>
<td>pk12</td>
<td>2.63</td>
<td>1.50</td>
<td>6.38</td>
<td>3.644</td>
<td>1.07</td>
</tr>
<tr>
<td>pk13</td>
<td>2.57</td>
<td>1.47</td>
<td>8.21</td>
<td>4.69</td>
<td>2.29</td>
</tr>
</tbody>
</table>

The results in Table 6.4 are plotted in Figure 6.12. The approximate beam theory appeared to correlate well with the numerical results. Models pk4 and pk12 gave the highest difference of -10.5% and -11.6%, respectively. Because the overall height of models pk4 and pk12 were more than that of the control model (pk1), the extent of the D-regions in these models increased, resulting in the panel behaving less of a cantilevered beam.
Influence of HCA

Taking into account the flexibility of the HCA, the overall lateral stiffness of the panel was significantly reduced by more than 80%, as shown in Figure 6.13. The correlation between the numerical results and beam theory predictions improved slightly as evident from the higher coefficient of determination (i.e., $R^2$ value). This reduction was not unexpected since the designed HCA’s was only about one-quarter stiffer than the panel. Due to lack of experimental data and as a conservative estimate, the in-plane stiffness of the panel with HCA was taken to be 20% of that of the concrete panel without HCA (Equation 6.20).

$$K_{\text{panel}} = 0.2K_{\text{concrete}} \quad (6.20)$$

It should be pointed out that the expressions are applicable for the range of parameters shown in Table 6.5.
The cantilevered bearing supports for conventional architectural precast cladding system are typically not designed for lateral load; friction force developed between the cantilevered section and shim is assumed to be sufficient to resist any unanticipated lateral load. For the EDCS, the bearing support A, must be specifically designed for the slip load of the friction damper.

**Figure 6.13:** Comparison of Concrete Panel Stiffness with HCA.

**Table 6.5:** Applicable Range of Panel Parameters.

<table>
<thead>
<tr>
<th>No.</th>
<th>Parameter</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Length</td>
<td>2.7 m (108” or 9’) – 7.3 m (288” or 24’)</td>
</tr>
<tr>
<td>2</td>
<td>Height</td>
<td>2.1 m (84” or 7’) – 4.0 m (156” or 13’)</td>
</tr>
<tr>
<td>3</td>
<td>Thickness</td>
<td>154.2 mm (6”) – 254.0 mm (10”)</td>
</tr>
<tr>
<td>4</td>
<td>Concrete Compressive Strength</td>
<td>27.6 MPa (4 ksi) – 41.1 MPa (6 ksi)</td>
</tr>
</tbody>
</table>

**6.5 BEARING SUPPORT**

The cantilevered bearing supports for conventional architectural precast cladding system...
The stiffness of the bearing support is a function of the relative out-of-plane flexibility of the bearing support and the concrete panel. If the panel is infinitely stiff with respect to the bearing support, the following equations for the out-of-plane stiffness would apply

\begin{align*}
(Pinned \ support) \ K_{\text{bearing}} &= \frac{3(EI)_{\text{bearing}}}{L_{\text{bearing}}} \quad (6.21) \\
(Fixed \ support) \ K_{\text{bearing}} &= \frac{12(EI)_{\text{bearing}}}{L_{\text{bearing}}^3} \quad (6.22)
\end{align*}

$E_{\text{bearing}}$ and $I_{\text{bearing}}$ are the Young modulus of elasticity and moment of inertia of the bearing support, respectively; $L_{\text{bearing}}$ is the “effective” cantilevered length. If, however the concrete panel is considerably more flexible than the bearing support, the above equations would grossly overestimate the bearing support stiffness and alternative formulations that take into account the bending of the panel are required. The following section outlined the derivation of the stiffness of the bearing support taking into account the out-of-plane flexibility of the concrete panel.

Figure 6.14 shows the simplified model representing the out-of-plane response of the concrete panel and the bearing support. Here, the eccentrically applied force has been replaced by an equivalent force and couple applied directly at the anchoring plate. Here, the modulus of elasticity of the concrete panel ($E_{\text{panel}}$) is taken as concrete modulus of elasticity from Equation 5.2. The moment of inertia for the panel is given by

\begin{equation}
I_{\text{panel}} = \frac{Ht^3}{12} \quad (6.23)
\end{equation}

The structural analysis of the frame was separated into two parts. In the first part, the horizontal deflection due to the horizontal force ($F$) was found. Next, the side sway due to the couple ($M$) was determined. The two deflections were later superimposed to give the overall stiffness of the bearing support. The determination of the stiffness of the frame is summarized in the following subsections.
6.5.1 Deflection Due to Lateral Force

The bending moment diagram of the two-member frame subjected to a horizontal load is shown in Figure 6.15. The following derivation assumes fixed support condition at A. If support A is pinned, the structure becomes determinate and the stiffness was derived in a slightly different way. Based on indeterminate structure analysis (e.g., moment distribution, slope-deflection, etc.), the resulting member end-moments, \( M_A \) and \( M_B \) can be shown to be

\[
M_A = \frac{(k_{R,1} + 2k_{R,2})}{2(k_{R,1} + k_{R,2})} M_{\Delta F} \quad (6.24)
\]

\[
M_B = \frac{k_{R,2}}{(k_{R,1} + k_{R,2})} M_{\Delta F} \quad (6.25)
\]

\( k_{R,1} \) and \( k_{R,2} \) are the relative stiffness factors of the bearing support and panel, respectively and are given by Equations 6.26 and 6.27. \( M_A \) is the fixed-end moment due to an end displacement \( \Delta_F \) (caused by the applied force \( F \)) and is given by Equation 6.28.
The shear force at support A can be computed using the end-moments as

\[ V_A = \frac{M_A + M_B}{L_{bearing}} \]  \hspace{1cm} (6.29)

Substituting Equations 6.24 and 6.25 into Equation 6.29 and recognizing (from static equilibrium) that the shear force \( V_A \) must be equal to the applied force \( F \), the equivalent stiffness is equal to the following expression:

\[ k_{F, fixed} = \frac{F}{\Delta_F} = \frac{3(EI)_{bearing}}{L_{bearing}^3} \left( \frac{k_{R,1} + 4k_{R,2}}{k_{R,1} + k_{R,2}} \right) \]  \hspace{1cm} (6.30)

By defining a relative stiffness ratio \( \alpha \) as shown in the following Equation 6.31, Equation 6.30 can be further simplified as Equation 6.32.
For pinned condition at support A, the lateral stiffness of the statistically determinate frame can be found from classical geometrical or energy methods. It can be shown that the lateral stiffness is given by the following expression:

\[
\alpha = \frac{(EI)}{L} \text{bearing} \left( \frac{(EI)}{L} \text{panel} \right)
\]

(6.31)

\[
k_{F,\text{fixed}} = \frac{3(EI)_{\text{bearing}}}{L_{\text{bearing}}} \left( \frac{12 + 4\alpha}{3 + 4\alpha} \right)
\]

(6.32)

For pinned condition at support A, the lateral stiffness of the statistically determinate frame can be found from classical geometrical or energy methods. It can be shown that the lateral stiffness is given by the following expression:

\[
k_{F,\text{pinned}} = \frac{3(EI)_{\text{bearing}}}{L_{\text{bearing}}} \left[ \frac{1}{1 + \alpha} \right]
\]

(6.33)

### 6.5.2 Deflection Due to Couple

To facilitate the analysis, the frame was divided into the non-sway and sway cases. The bending moment diagrams (for fixed support A) are shown in Figure 6.16. The member end-moments are given by Equations 6.34 through 6.38.

\[
M_{A,1} = \frac{k_{R,1}}{4(k_{R,1} + k_{R,2})} M
\]

(6.34)

\[
M_{B,1} = \frac{k_{R,1}}{2(k_{R,1} + k_{R,2})} M
\]

(6.35)
In the absence of a horizontal force, the end-moments $M_A$ and $M_B$ must be equal in magnitude and sense and the following equation applies:

$$M_{B,1} - M_{B,2} = M_{A,2} - M_{A,1}$$  \hfill (6.39)
Substituting Equations 6.34 through 6.38 into the above expression, we have the equivalent stiffness for the couple equal to

$$k_{M, \text{fixed}} = \frac{M}{\Delta_M} = \frac{4(EI)_{\text{bearing}}}{L_{\text{bearing}}} \left( 1 + \frac{k_{R,2}}{k_{R,3}} \right)$$  \hspace{1cm} (6.40)

Substituting the relative stiffness ratio from Equation 6.31, we have

$$k_{M, \text{fixed}} = \frac{4(EI)_{\text{bearing}}}{L_{\text{bearing}}} \left( 1 + \frac{4}{3\alpha} \right)$$  \hspace{1cm} (6.41)

For pinned support condition at A, the lateral stiffness can be easily shown (e.g., using virtual work method) to be expressed as follows:

$$k_{M, \text{pinned}} = \frac{6(EI)_{\text{bearing}}}{L_{\text{bearing}}}$$  \hspace{1cm} (6.42)

### 6.5.3 Overall Lateral Stiffness

For fixed condition at support A, Equations 6.32 and 6.41 can be combined to give the overall out-of-plane stiffness of the bearing support as in Equation 6.43.

$$k_{\text{fixed}} = \frac{12(EI)_{\text{bearing}}}{L_{\text{bearing}}^3} \left( \frac{\alpha + 3}{7\alpha + 3} \right)$$  \hspace{1cm} (6.43)

For pinned support condition, combination of Equations 6.33 and 6.42 gives the following expression:

$$k_{F, \text{pinned}} = \frac{3(EI)_{\text{bearing}}}{L_{\text{bearing}}^3} \left[ \frac{2}{2 + 3\alpha} \right]$$  \hspace{1cm} (6.44)
For infinitely stiff concrete panel, the relative stiffness ratio ($\alpha$) approaches zero and Equations 6.43 and 6.44 are reduced to Equations 6.21 and 6.22, respectively.

### 6.5.4 Correlation with FEA Results

The stiffness coefficient values calculated using Equations 6.43 and 6.44 were compared to the numerical results from FE model cp7 (pinned support condition) and cp8 (fixed support) in Table 6.6. In deriving the closed-form equations, the bearing-panel joint was assumed to be infinitely rigid. Because the HCA exhibited some degree of rotational flexibility, the “effective” bearing length would vary with different support conditions. Three “effective” cantilevered lengths of the bearing support were investigated here. The shortest length of 330 mm (13 in.) was the distance between the bearing end supports to the center of the stud groups. The distance from the tip of the studs to the fixed end support was 406 mm (16 in.) while the distance to the center of the panel was 356 mm (14 in.).

Table 6.6: Comparison of Bearing Stiffness.

<table>
<thead>
<tr>
<th>Source</th>
<th>$L_{bearing}$</th>
<th>Bearin</th>
<th>Definition of bearing length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(mm)</td>
<td>(in)</td>
<td>(kN/mm) (kips/in) (kN/mm) (kips/in)</td>
</tr>
<tr>
<td>ANSYS (cp7)</td>
<td>55</td>
<td>312</td>
<td>453</td>
</tr>
<tr>
<td>ANSYS (cp8)</td>
<td>330 13</td>
<td>93 529</td>
<td>418 2,387</td>
</tr>
<tr>
<td>Eq. 6.43 and 6.44</td>
<td>356 14</td>
<td>78 447</td>
<td>346 1,974</td>
</tr>
<tr>
<td></td>
<td>406 16</td>
<td>57 328</td>
<td>246 1,403</td>
</tr>
</tbody>
</table>

$E_{bearing} = 2 \times 10^5$ MPa (29 x 10$^6$ psi), $E_{panel} = 2.8 \times 10^4$ MPa (4 x 10$^6$ psi),
$I_{bearing} = 2.0 \times 10^7$ mm$^4$ (48.3 in$^4$), $I_{panel} = 5.87 \times 10^8$ mm$^4$ (3,584 in$^4$)
$L_{panel} = 5.88$ m (232 in.)
6.6 SUMMARY

A mathematical representation of the in-plane load-deformation characteristics of the EDCS was presented and the key components of the EDCS stiffness were identified. The closed-form equations describing the linear elastic stress distribution in the panel was derived from static equilibrium relationship. It was found to correlate well with the FEA results in the region between the lateral supports. This suggests that the panel could, within the limits of applicability, be analyzed as an elastic beam to obtain the in-plane load-deformation characteristics. Using fundamental elastic beam theory and classical structural analysis methods, a rational method for estimating the stiffness coefficient for these components was suggested. The approximate solutions were found to correlate well with the numerical results from the FEA study.

6.7 CONCLUSION

In the modeling of architectural cladding panels, to facilitate analysis, the panels are often assumed to be uncracked and rigid [Petkovski and Waldron, 1995; Goodno et al., 1998]. As shown in the present study, this may not be the case as the overall lateral stiffness of the panel is sensitive to the stiffness of its contributing components. Figure 6.17 summarizes the estimated contribution of each component in the EDCS in terms of flexibility coefficients (i.e., reciprocal of the stiffness); \( f_{\text{EDCS}} \), \( f_{\text{bearings}} \), \( f_{\text{stud}} \), \( f_{\text{concrete}} \) and \( f_{\text{panel}} \) are the flexibility coefficients of the EDCS, (fixed) cantilevered bearing support, headed stud assembly, concrete panel and the panel with
HCA, respectively. Clearly, the assumption of an infinitely stiff precast concrete panel would not be appropriate here.

![Diagram showing flexibility coefficients of EDCS components.]

Figure 6.17: Flexibility Coefficients of EDCS Components.
CHAPTER 7
BUILDING LEVEL STUDY - CORRELATION AND COMPARISON

7.1 CHAPTER OVERVIEW

The stiffness and strength characteristics of the EDCS have been investigated in previous chapters through FEA technique and classical structural analysis method. The result is a simplified mathematical model that can be incorporated into the analytical model of a building under consideration. The next logical step is to evaluate the characteristics and effectiveness of the EDCS at the building level. Instead of analyzing an as-built or a generic building, the approach adopted here is to add the EDCS to an existing structure or building that has been extensively tested and with well understood structural response under earthquake excitation. Because friction damper is part of the EDCS, it would be useful for the structure under consideration to have been tested with similar supplemental energy-absorbing devices. In addition, the test results should be well-documented for proper correlation and comparison purposes. For the present study, the experimental data from the research program by Aiken and Kelly [1990] was used.

The objective of the research program by Aiken and Kelly [1990] was to perform earthquake simulator tests of a large-scale, multistory steel structure incorporating two different types of energy absorbers, a viscoelastic device and a friction device. Through the tests, Aiken and Kelly [1990] were able to evaluate the characteristics and effectiveness of the supplemental dampers. In addition, the study also investigated analytical methods appropriate for the mathematical modeling of the structure incorporating the supplemental devices. The test report contains comprehensive data, in particular, the dynamic responses of the test structures, with and without energy dissipating devices, subjected to earthquake motions. The extensive experimental data presented an excellent opportunity for meaningful comparison with analytical results obtained for the EDCS designed in present study. The purposes of the this phase of the study were to

- Review the characteristics of the test structures of Aiken and Kelly [1990];
• Develop analytical models to accurately capture the dynamic response characteristics of the test structures through a correlation study; and
• Perform a comparative study to evaluate the performance of the EDCS in relation to conventional moment-resisting frame with and without supplemental energy dissipating devices.

The chapter begins with a brief discussion of the research program by Aiken and Kelly [1990] with emphasis on the physical characteristics of the test structures. This is followed by a correlation study which compares the results from the present analytical models with the experimental data. In the final section, a comparative study is performed to assess the performance of the present EDCS with two test structures from Aiken and Kelly [1990] study.

7.2 RESEARCH PROGRAM OF AIKEN AND KELLY [1990]

7.2.1 Description of Test Structures

The test structure was a 3-bay, 9-story steel moment-resisting frame tested by Aiken and Kelly [1990] in the Earthquake Simulator Laboratory (ESL) at the University of California, Berkeley’s Earthquake Engineering Research Center (EERC). The test structure was originally designed and built in 1976 – 1977 for studies of the behavior of a building with column uplift permitted at the foundation level [Huckelbridge, 1977]. Subsequently, the same test structure was adapted for a number of different research programs [Yang, 1982; Griffith et al., 1988]. The experimental model was designed to present a fairly realistic section in the weak direction of a typical steel-frame building (i.e., prototype) at approximately one-quarter scale.

The first story was 1.2 m (4 ft) high with the upper stories at 0.9 m (3 ft) high as shown in Figure 7.1a. The bays were 1.52 m (5 ft) and 1.98 m (6.5 ft) wide at the center and ends, respectively. The structure was approximately 8.53 m (28 ft) high by 5.49 m (18 ft) wide and with an aspect ratio of 1.56. Girders and columns were W6 x 8.5 and W4 x 13 sections, respectively. The structure had three bays in the direction of testing and one bay of 1.83 m (6 ft) wide in the lateral direction. Additional bracings were provided in the out-of-plane direction to increase the
lateral and torsional stiffnesses since the out-of-plane response was not of any interest to the authors. All primary connections were welded while the connections for the out-of-plane bracing were bolted. To ensure the moment-frame can develop the anticipated moments, the exterior face of the beam-column joint panel zones for the OMF were reinforced with 15.9 mm (5/8 in.) and 4.8 mm (3/16 in.) double-plates for interior and exterior columns, respectively. This bare frame was referred to as OMF. The test program of Aiken and Kelly [1990] also included the study of the concentrically-braced configuration of the test structure (Figure 7.1b). Chevron bracings were also added to central bay of the OMF. The bracing members were 1½ x 1½ x ¼ double-angles at the bottom level and 1 x 1 x ¼ double-angles at the remaining levels (i.e., 2 – 9). The concentrically-braced frame was labeled as CBF.

Figure 7.1: MRF and CBF Configurations of Test Model [Aiken and Kelly, 1990].

To satisfy similitude requirements (discussed below), additional masses of 44.5 kN (10 kips) per floor, in the form of concrete blocks and lead ballast were added to the model in the test. The combined weight of the OMF and the additional masses were about 413.7 kN (93 kips).

Two different types of support conditions were investigated by Aiken and Kelly [1990], one representing conventional fixed end conditions and the second to model the interaction between the structure and the shake-table.
7.2.2 Energy Dissipation Devices

In Aiken and Kelly [1990] study, two different types of energy-dissipating devices, a viscoelastic device and a friction device, were added separately to the MRF for performance testing. For the purpose of the present study, only the frame with added friction devices or dampers, labeled as FD, would be used for the correlation and comparative studies. The FD test frame is shown in Figure 7.2. The friction devices were cylindrical friction dampers designed and developed by Sumitomo Metal industries, Ltd., Japan. The device was originally developed as a shock absorber in railway application. In these devices, friction force is generated through the sliding of copper alloy friction pads with the inner surface of the steel cylinder.

![FD Configurations of Test Model](image)

Figure 7.2: FD Configurations of Test Model [Aiken and Kelly, 1990].

7.2.3 Similitude Requirements

Since the models of Aiken and Kelly [1990] were one-quarter scale, scaling rules were used (e.g., for earthquake motion). Based on the laws of similitude and using dimensionless analysis, scaling factors for different geometric and material properties can be easily derived. Table 7.1 lists the scaling factors for some basic parameters.
As mentioned earlier, the above factors were used to establish the correct magnitude for the test parameters (e.g. floor weight, time-base) at one-quarter scale configuration. Likewise, they can be used to extend the one-quarter scale test model to prototype results. For example, the prototype girder would have a depth of 609.6 mm (24 in.) and a moment of inertia of 62\times10^6 \text{mm}^3 (3,789 \text{in}^3), the prototype column has a depth of 406.4 mm (16 in.), an area of 3.9\times10^4 \text{mm}^2 (61 \text{in}^2) and a moment of inertia of 47\times10^6 \text{mm}^3 (2,893 \text{in}^3). The equivalent prototype structure would be 34.1 m (112 ft) high, comprising of W24 x 131 girders and W14 x 211 columns, with a floor loading of 95 psf and a fundamental period of about one second. These parameters match fairly close to those of similar category steel moment frames.

### Table 7.1: Similitude Scaling Factors.

<table>
<thead>
<tr>
<th>No.</th>
<th>Parameters</th>
<th>Basic dimensions</th>
<th>Scaling</th>
<th>Scaling factor = Prototype / ¼-scale model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Length</td>
<td>L</td>
<td>L</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>Time</td>
<td>T</td>
<td>\sqrt{L}</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>Mass</td>
<td>M</td>
<td>L^2</td>
<td>16</td>
</tr>
<tr>
<td>4</td>
<td>Displacement</td>
<td>L</td>
<td>L</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Area</td>
<td>L^2</td>
<td>L^2</td>
<td>16</td>
</tr>
<tr>
<td>6</td>
<td>Acceleration</td>
<td>L T^-2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>7</td>
<td>Stress</td>
<td>M L^{-1} T^{-2}</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>Strain</td>
<td>-</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>Stiffness</td>
<td>M T^2</td>
<td>L</td>
<td>4</td>
</tr>
<tr>
<td>10</td>
<td>Force</td>
<td>M L T^{-2}</td>
<td>L^2</td>
<td>16</td>
</tr>
<tr>
<td>11</td>
<td>Moment of Inertia</td>
<td>L^4</td>
<td>L^4</td>
<td>256</td>
</tr>
</tbody>
</table>

7.2.4 Excitation Signals

For the earthquake tests, each test structure (MRF, CBF and FD) was subjected to several ground motions generated from eleven historical earthquake records with varying magnitudes and frequency contents. The signal was processed based on operating limits of the ESL shake-table.
To satisfy similitude relationships, the time-base (or time-step) of the excitation signals was halved. This effectively halved the duration of the input signals and doubled the frequency content.

7.3 CORRELATION STUDY

7.3.1 Analytical Models for MRF and CBF

The conventional bare frame (i.e., MRF) and braced frame (i.e., CBF) were modeled (in ETABS) as two dimensional planar frames. All frame elements were modeled as continuous with classical beam-column elements. The section and material properties used are summarized in Table 7.2. ETABS allows the modeling of double plates at the beam-column panel zone joints simply by specifying the doubler plate thickness. Rigid-end offsets were applied to all the frame elements to account for the reduction of length due to the finite depth of the beam-column joints. It was found, through modal frequency correlations, that a rigid factor of 0.25 for the rigid-end offset was appropriate. Lump masses of 44.5 kN (10 kips) weight were distributed to the four beam-column joints at each story as required.

7.3.2 Analytical Models for FD

Supplemental friction devices and braces were added to the MRF analytical model as shown in Figure 7.3. A rigid truss element, that is only capable of resisting axial load and deformation, connects the apex of the braces to the column. By specifying a large cross-sectional area (6.4x10^6 mm^2 or 10^4 in^2) for the rigid truss element, horizontal deformation is restricted to the Sumitomo friction dampers, modeled here by the ETABS LINK elements with PLASTIC1 property.
Table 7.2: Properties of Frame Elements.

<table>
<thead>
<tr>
<th>Element</th>
<th>Section</th>
<th>Overall Depth (mm (in))</th>
<th>Overall Width (mm (in))</th>
<th>Area $\text{mm}^2$ (in$^2$)</th>
<th>Moment of Inertia $\times 10^5$ mm (in$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girders</td>
<td>W6 x 8.5</td>
<td>148.1 (5.83)</td>
<td>100.1 (3.94)</td>
<td>1619 (2.51)</td>
<td>24.25 (14.8)</td>
</tr>
<tr>
<td>Columns</td>
<td>W4 x 13</td>
<td>105.7 (4.16)</td>
<td>103.1 (4.06)</td>
<td>2471 (3.83)</td>
<td>18.52 (11.3)</td>
</tr>
<tr>
<td>Braces for CB</td>
<td>2L1½ x 1½ x ¼</td>
<td>38.1 (1.5)</td>
<td>79.8 (3.14)</td>
<td>910 (1.41)</td>
<td>1.08 (0.658)</td>
</tr>
<tr>
<td>(Ground level)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Braces for CBF</td>
<td>2L1 x 1 x ¼</td>
<td>25.4 (1.0)</td>
<td>57.4 (2.26)</td>
<td>606 (0.94)</td>
<td>0.41 (0.248)</td>
</tr>
<tr>
<td>(1st - 9th)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Braces for FD</td>
<td>HSS2¼ x 2¼ x 3/16</td>
<td>57.2 (2.25)</td>
<td>57.2 (2.25)</td>
<td>884 (1.37)</td>
<td>1.56 (0.953)</td>
</tr>
<tr>
<td>(All story)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. Unless specified otherwise, all other ETABS properties for the frame elements not shown in the above table were not used. Material properties specified: Steel modulus of elasticity, $E_s = 200$ GPa (29,000 kips/in$^2$), Poisson ratio, $\nu = 0.3$.

As mentioned earlier in Chapter 3, the PLASTIC1 property is based on Bouc Wen’s elasto-plastic model. Except for the slip load, ETABS LINK elements have the same properties (e.g., high stiffness) as those used for the nonlinear time history analysis of the 3-story shear building in Chapter 3. The same distribution of the slip loads used in the experiment was specified in the analytical model and is shown in Figure 7.3.

### 7.3.3 Analytical Model for Shake Table

Aiken and Kelly [1990] reported that the effects of the shake-table interaction (due to table pitching) were significant, especially at large signal inputs. The experimental results revealed that the interaction effect could contribute up to 25% of the total displacement of the 9-story structure. In particular, the shake-table interaction has considerable effect on the time history correlation between the experimental results and analytical predictions. The mathematical
model to represent the foundation rotational flexibility as proposed by the Aiken and Kelly [1990] are shown in Figure 7.4.

![Figure 7.3: FD Analytical Model.](image)

<table>
<thead>
<tr>
<th>Floor</th>
<th>Slip load, kN (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>21.4 (4.8)</td>
</tr>
<tr>
<td>8</td>
<td>21.4 (4.8)</td>
</tr>
<tr>
<td>7</td>
<td>23.1 (5.2)</td>
</tr>
<tr>
<td>6</td>
<td>43.6 (9.8)</td>
</tr>
<tr>
<td>5</td>
<td>44.5 (10.0)</td>
</tr>
<tr>
<td>4</td>
<td>42.7 (9.6)</td>
</tr>
<tr>
<td>3</td>
<td>42.7 (9.6)</td>
</tr>
<tr>
<td>2</td>
<td>43.6 (9.8)</td>
</tr>
<tr>
<td>1</td>
<td>51.6 (11.6)</td>
</tr>
</tbody>
</table>
The slab was modeled as a (infinitely) rigid beam pinned about its midpoint. A “rotational mass” or mass moment of inertia of 70.6 kN·m·sec\(^2\) (625 kips-inches-sec\(^2\)) was specified at this pinned support. The ends of the beam are supported through two linear springs with stiffness, \(K_p\). From their study, it was found, through trial-and-error, that a spring constant of 400 kips/inch for \(K_p\) was recommended for MRF and 44 kN/mm (250 kips/in) for the FD model.

![Figure 7.4: Table-Structure Interaction Model.](image)

### 7.3.4 Ground Motion Selections

Since the objective of the present analytical study is to evaluate the performance of the energy dissipating devices, rather than for building design, the selection of the ground motions was not site-specific but rather was influenced by the dynamic properties (e.g., first and second mode periods) of the structures. Three historical strong ground motions were selected as shown in Table 7.3. The first two ground motions were employed by Aiken and Kelly [1990] in their earthquake simulator tests whereas the 1994 Northridge earthquake was chosen to reflect a more modern record.

All ground motion data were obtained from Pacific Earthquake Engineering Research Center (PEER) Strong Motion Database. The database contained 1,557 records from 143 earthquakes from tectonically active regions. It should be pointed out that, for a given earthquake and station, the digitized accelerograms provided by PEER differs slightly from the shake-table signals used in Aiken and Kelly’s [1990] experiment due to different correction filters used by the providers of these data. The filters include bandpass filtering (removing noise contamination) and
instrument correction (to remove the effects of frequency-dependent instrument response). A short description of each ground motion is presented below. It should be pointed out that the ground motion described below were time-scaled down by a factor of two to satisfy similitude requirement, as previously discussed.

Table 7.3: Recorded Strong Ground Motions.

<table>
<thead>
<tr>
<th>Earthquake Name</th>
<th>Station Name</th>
<th>Comp.</th>
<th>Mag.</th>
<th>Peak Ground Acceleration (g)</th>
<th>Geology</th>
<th>Time History Label</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imperial Valley</td>
<td>El Centro Array #9</td>
<td>S00E</td>
<td>6.95</td>
<td>0.313</td>
<td>100 ft of stiff clay over volcanic rock</td>
<td>ELCEN</td>
</tr>
<tr>
<td>05/18/1940</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kern County</td>
<td>Taft Lincoln School</td>
<td>S69E</td>
<td>7.36</td>
<td>0.178</td>
<td>River alluvium</td>
<td>TAFT</td>
</tr>
<tr>
<td>07/21/1952</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Northridge</td>
<td>LA, Century City CC North</td>
<td>N90E</td>
<td>6.05</td>
<td>0.26</td>
<td>Pleistocene medium alluvium</td>
<td>LACC</td>
</tr>
<tr>
<td>01/17/1994</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

7.3.4.1 ELCEN Time History

The El Centro 1940 S00E or North-South horizontal component was one of the earliest recorded strong motion accelerograms and it formed the basis for the development of many seismic design guidelines and codes (e.g., UBC, FEMA-273, etc.). The (time-scaled) digitized accelerogram shown in Figure 7.5 includes 7,999 data points at equal time intervals of 0.005 sec, giving a total duration of 19.2 sec. The peak ground acceleration (PGA) is 0.313g (3.07 m/sec² or 120.94 in./sec²) at 1.08 sec. The FFT of the accelerogram is shown in Figure 7.6. It can be seen that the frequency content is strongest between 1 Hz (1 sec) to 4 Hz (0.25 sec), and most of the energy is concentrated between 1.5 and 5.5 sec into the signal. The linear elastic response spectra for different damping ratios are plotted in Figure 7.7.
7.3.4.2 TAFT Time History

The PEER (time-scaled) time history (Figure 7.8) has a total of 5,416 data points, and a duration of about 27 sec. The peak ground acceleration (PGA) is 0.178g (1.74 m/sec$^2$) or 68.7
in./sec$^3$) at 1.875 sec into the signal. From Figure 7.9, the frequency content is spread over a wider range (1 and 6 Hz) than the ELCEN signal. The excitation energy is mostly contained between 1.5 and 5 sec. The elastic response spectra are plotted in Figure 7.10.

![Figure 7.7: Design Spectra of ELCEN Time History.](image)

![Figure 7.8: TAFT Time History.](image)
Figure 7.9: FFT of TAFT Time History.

Figure 7.10: Design Spectra of TAFT Time History.
7.3.4.3 LACC Time History

The 1994 Northridge earthquake, centered in the San Fernando Valley, was approximately equal in magnitude to the 1971 San Fernando event. Ground shaking was very strong, with amplitudes among the highest ever recorded but the recorded ground motion intensity was consistent with both theoretical and empirical based predictions of ground motion for large magnitude earthquakes [SEAOC, 1999]. The event provided a significant test for many buildings designed in accordance with the seismic design codes current at that time and had significant effect on the subsequent development of seismic-hazards maps for California.

The PEER time history (Figure 7.11) has a total of 2,000 data points, and a duration of about 10 sec. The peak ground acceleration (PGA) is 0.26g (2.51 m/sec^2 or 98.8 in./sec^2) at 1.705 sec into the signal. As seen from the transform function (Figure 7.12), the frequency content is spread over a wide range. The excitation energy is mostly contained between 1.5 and 3.5 sec. The elastic response spectra are plotted in Figure 7.13.

![Figure 7.11: LACC Time History.](image)
Figure 7.12: FFT of LACC Time History.

Figure 7.13: Design Spectra of LACC Time History.
7.3.5 Results

7.3.5.1 Dynamic Properties

The suitability of the mathematical model for MRF, CBF and FD model were first investigated by a series of modal analyses. The natural frequencies of the three models are summarized in Table 7.4. Only the first three modes were necessary because the combined modal mass participation of the first three modes are more than 95% of the total mass of the model. The analytical predictions agreed well with the experiment values, generally within less than 10%. As expected, the introduction of the foundation rotational flexibility due to interaction effects slightly increases the natural periods of the analytical models. The good correlations of the natural frequencies gave some confidence of the correctness of the present ETABS analytical models.

The equivalent modal damping ratios determined from the experiment and the values used in the present correlation study are summarized in Table 7.5. For the FD model, Aiken and Kelly [1990] reported that a value of 4% was found to be appropriate for their analytical model. The unusually high modal damping ratios included damping from the shake-table mechanism. This is further discussed in the time history analysis below. No modal damping was specified for higher modes.

7.3.5.2 Time History Analysis

Different intensity excitation signals were generated from ELCEN and TAFT ground motions and summarized in Table 7.6. The numbers (i.e., 50, 200, 300, etc.) in the abbreviation follows the convention used by Aiken and Kelly [1990]. It represents the maximum horizontal displacement (or span) of the shake-table used for that signal and are included here to facilitate cross referencing. The time scale factor was used to satisfy similitude relationship while the acceleration factor was required to match the PGA of the shake-table excitation signal.
Table 7.4: Natural Frequencies of MRF, CBF and FD.

<table>
<thead>
<tr>
<th>Model</th>
<th>Mode</th>
<th>Experiment [Aiken and Kelly, 1990]</th>
<th>ETABS analytical model</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Frequency</td>
<td>Period (sec)</td>
<td>Frequency</td>
</tr>
<tr>
<td>MRF (fixed support)</td>
<td>1</td>
<td>1.95</td>
<td>0.52</td>
<td>1.96</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.60</td>
<td>0.15</td>
<td>6.05</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>11.11</td>
<td>0.09</td>
<td>10.72</td>
</tr>
<tr>
<td>MRF (with table-interaction)</td>
<td>1</td>
<td>1.89</td>
<td>0.53</td>
<td>1.81</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.60</td>
<td>0.15</td>
<td>6.07</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>11.26</td>
<td>0.09</td>
<td>10.76</td>
</tr>
<tr>
<td>CBF (fixed support)</td>
<td>1</td>
<td>Not available</td>
<td></td>
<td>3.86</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Not available</td>
<td></td>
<td>12.35</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Not available</td>
<td></td>
<td>22.94</td>
</tr>
<tr>
<td>CBF (with table-interaction)</td>
<td>1</td>
<td>2.95</td>
<td>0.34</td>
<td>2.96</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>11.39</td>
<td>0.09</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Not available</td>
<td></td>
<td>21.66</td>
</tr>
<tr>
<td>FD (fixed support)</td>
<td>1</td>
<td>3.00</td>
<td>0.33</td>
<td>3.37</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>10.10</td>
<td>0.10</td>
<td>10.51</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Not available</td>
<td></td>
<td>19.23</td>
</tr>
<tr>
<td>FD (with table-interaction)</td>
<td>1</td>
<td>2.60</td>
<td>0.38</td>
<td>2.46</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>9.70</td>
<td>0.10</td>
<td>10.50</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Not available</td>
<td></td>
<td>18.21</td>
</tr>
</tbody>
</table>

Notes:

1 The average experimental values were determined from a series of diagnostic tests (i.e., free-vibration, random excitation and pulse free vibration) with fixed support conditions with no or negligible shake-stable interaction.

2 The experimental values were obtained from earthquake tests that included the interaction from the shake-table. \( K_p = 70 \text{kN/mm (400 kips/in.)} \).

3 Aiken and Kelly [1990] did not report a \( K_p \) value for CBF. A value of 70 kN/mm (400 kips/in.) was initially assumed. However, from time history analyses, a value of 35 kN/mm (400 kips/in.) was found to give the best peak roof displacements and accelerations correlations. A lower \( K_p \) value is reasonable since CBF is the stiffest among the test models.
### Table 7.5: Experimental and Analytical Modal Damping Ratios.

<table>
<thead>
<tr>
<th>Model</th>
<th>Mode</th>
<th>Experimental (%)</th>
<th>Value used for comparative study (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF (fixed support)</td>
<td>1</td>
<td>2.9 – 3.3</td>
<td>3.10 (Average)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.3 – 0.4</td>
<td>0.35 (Average)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.9 – 1.2</td>
<td>1.05 (Average)</td>
</tr>
<tr>
<td>MRF (with table-interaction)</td>
<td>1</td>
<td>4.1</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>CBF (with table-interaction)</td>
<td>1</td>
<td>4.9</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>FD (with table-interaction)</td>
<td>1</td>
<td>4.8</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>3.3</td>
<td>4.0</td>
</tr>
</tbody>
</table>

### Table 7.6: Time History Cases.

<table>
<thead>
<tr>
<th>Time history case</th>
<th>Time history source</th>
<th>Shake-table PGA (g)</th>
<th>Model to which the time history case is applied</th>
<th>Scaling factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Time</td>
</tr>
<tr>
<td>EC50MRF</td>
<td></td>
<td>0.141</td>
<td>MRF</td>
<td>0.5</td>
</tr>
<tr>
<td>EC200MRF</td>
<td></td>
<td>0.461</td>
<td>MRF</td>
<td>0.5</td>
</tr>
<tr>
<td>EC300MRF</td>
<td></td>
<td>0.604</td>
<td>MRF</td>
<td>0.5</td>
</tr>
<tr>
<td>EC400MRF</td>
<td></td>
<td>0.806</td>
<td>MRF</td>
<td>0.5</td>
</tr>
<tr>
<td>EC50CBF</td>
<td></td>
<td>0.091</td>
<td>CBF</td>
<td>0.5</td>
</tr>
<tr>
<td>EC100CBF</td>
<td></td>
<td>0.184</td>
<td>CBF</td>
<td>0.5</td>
</tr>
<tr>
<td>EC50FD</td>
<td></td>
<td>0.134</td>
<td>FD</td>
<td>0.5</td>
</tr>
<tr>
<td>EC200FD</td>
<td></td>
<td>0.394</td>
<td>FD</td>
<td>0.5</td>
</tr>
<tr>
<td>EC250FD</td>
<td></td>
<td>0.476</td>
<td>FD</td>
<td>0.5</td>
</tr>
<tr>
<td>EC300FD</td>
<td></td>
<td>0.555</td>
<td>FD</td>
<td>0.5</td>
</tr>
<tr>
<td>EC400FD</td>
<td></td>
<td>0.712</td>
<td>FD</td>
<td>0.5</td>
</tr>
<tr>
<td>TF50MRF</td>
<td>ELCEN</td>
<td>0.091</td>
<td>MRF</td>
<td>0.5</td>
</tr>
<tr>
<td>TF200MRF</td>
<td>TAFT</td>
<td>0.362</td>
<td>MRF</td>
<td>0.5</td>
</tr>
<tr>
<td>TF50FD</td>
<td></td>
<td>0.089</td>
<td>FD</td>
<td>0.5</td>
</tr>
<tr>
<td>TF400FD</td>
<td></td>
<td>0.839</td>
<td>FD</td>
<td>0.5</td>
</tr>
</tbody>
</table>
MRF and CBF Analytical Models

The peak roof acceleration (PRA) and the peak roof displacement (PRD) are presented in Table 7.7. With the table-interaction included in the mathematical model, the correlations are significantly improved. Except for the TAFT ground motions, the analytical predictions for the PRD closely matched those from the experiment. As mentioned earlier, the difference between PEER ground motion data and the one used in Aiken and Kelly [1990]’s shake-table experiment is likely to be the reason for the disparities between analytical and experimental results for the TAFT signal.

From Table 7.6, the introduction of the support springs to account for the shake-table interaction only increases the periods of the mathematical model, by at most 10%. However, as clearly seen from the roof displacement time histories in Figure 7.14, the effect of the table-structure interaction on the time history correlation is significant.

<table>
<thead>
<tr>
<th>Model</th>
<th>Time history case</th>
<th>Experiment</th>
<th>Analytical</th>
<th>Difference</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PGA (g)</td>
<td>PRA (g)</td>
<td>PRD (mm)</td>
<td></td>
</tr>
<tr>
<td>MRF (fixed support)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EC50MRF</td>
<td>0.141</td>
<td>0.474</td>
<td>17.7</td>
<td>0.70</td>
<td>0.535</td>
</tr>
<tr>
<td>EC200MRF</td>
<td>0.461</td>
<td>1.417</td>
<td>66.9</td>
<td>2.64</td>
<td>1.751</td>
</tr>
<tr>
<td>EC300MRF</td>
<td>0.604</td>
<td>1.480</td>
<td>78.8</td>
<td>3.10</td>
<td>2.294</td>
</tr>
<tr>
<td>EC400MRF</td>
<td>0.806</td>
<td>1.607</td>
<td>83.9</td>
<td>3.30</td>
<td>1.607</td>
</tr>
<tr>
<td>TF50MRF</td>
<td>0.091</td>
<td>0.182</td>
<td>6.73</td>
<td>0.26</td>
<td>0.398</td>
</tr>
<tr>
<td>TF200MRF</td>
<td>0.362</td>
<td>0.791</td>
<td>28.0</td>
<td>1.10</td>
<td>1.583</td>
</tr>
<tr>
<td>CBF (with table-interaction)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EC50CBF</td>
<td>0.091</td>
<td>0.353</td>
<td>8.9</td>
<td>0.35</td>
<td>0.385</td>
</tr>
<tr>
<td>EC100CBF</td>
<td>0.184</td>
<td>0.635</td>
<td>18.3</td>
<td>0.72</td>
<td>0.779</td>
</tr>
</tbody>
</table>

Note: The modal damping ratio used for each mode is shown in Table 7.5
The correlations for the PRA were not as satisfactory as the PRD, especially at higher ground motion intensities. Even with table-structure interaction accounted for, the difference was still as high as 46%. Unfortunately, the PRA correlations were not discussed by Aiken and Kelly [1990]. But bearing in mind that the PEER accelerogram used in the present study differed slightly from the excitation signal used in the Aiken and Kelly [1990] shake-table experiment, the generally good correlation is indeed very encouraging and this provided much confidence on the accuracy of the mathematical models to represent the actual test structures and conditions.

FD Analytical Model

For the FD model, the time history analyses were performed with table-structure interaction. As mentioned earlier, Aiken and Kelly [1990] reported that 4% modal damping was appropriate for their analytical FD model. It was not clear from their report whether this value was applied to the relevant (i.e., first and second) modes. To investigate this, a series of analyses, at different damping ratios (2%, 2.5%, 3% and 5%) were performed. It was found that based on roof displacement time history correlations, the damping ratio should not be lower than 2.5% and that lower damping ratios gave slightly better correlation for PRD. However, in terms of modal and dissipated energy time histories, 4% damping was found to give the best correlation; a lower damping resulted in dissipated energy higher than those observed experimentally. As a result, a modal damping of 4% was applied to the first and second modes for the FD model.

The PRA and PRD values are summarized in Table 7.8. The PRD correlations are not as good as for MRF, with the analytical model under-predicting. The analytical results for the PRA were generally not satisfactory with differences as high as 191%. However, the analytical results are conservative. As pointed out by Aiken and Kelly [1990], the differences between the analytical and experimental results are partly due to the method used to model the table-structure interaction effects, and better results, especially for high intensity excitations, could be obtained from a properly calibrated nonlinear interaction model. On the other hand, the analytical roof displacement time histories (Figures 7.15 and 7.16) show excellent phase agreement with the experimental results, indicating that the present analytical model is appropriate.
Figure 7.14: Experimental and Analytical Roof Time Histories for EC400MRF.
Table 7.8: FD Peak Roof Displacements and Accelerations.

<table>
<thead>
<tr>
<th>Model</th>
<th>Time history</th>
<th>Experiment</th>
<th>Analytical</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA</td>
<td>PRA</td>
<td>PRD</td>
<td>PGA</td>
</tr>
<tr>
<td></td>
<td>(g)</td>
<td>(g)</td>
<td>(mm)</td>
<td>(in)</td>
</tr>
<tr>
<td>EC50FD</td>
<td>0.134</td>
<td>0.326</td>
<td>9.9</td>
<td>0.39</td>
</tr>
<tr>
<td>EC200FD</td>
<td>0.394</td>
<td>0.866</td>
<td>35.1</td>
<td>1.38</td>
</tr>
<tr>
<td>EC250FD</td>
<td>0.476</td>
<td>1.021</td>
<td>44.5</td>
<td>1.75</td>
</tr>
<tr>
<td>EC300FD</td>
<td>0.555</td>
<td>1.105</td>
<td>54.3</td>
<td>2.14</td>
</tr>
<tr>
<td>EC400FD</td>
<td>0.712</td>
<td>1.578</td>
<td>69.4</td>
<td>2.73</td>
</tr>
<tr>
<td>TF50FD</td>
<td>0.089</td>
<td>0.248</td>
<td>8.6</td>
<td>0.34</td>
</tr>
<tr>
<td>TF400FD</td>
<td>0.839</td>
<td>1.408</td>
<td>43.8</td>
<td>1.72</td>
</tr>
</tbody>
</table>

Note: The modal damping ratio used is shown in Table 7.5.

Figure 7.15: Experimental and Analytical Roof Time Histories for EC400FD.
Figure 7.17 and 7.18 compare the peak story displacement and the peak story shear for the two time history cases. For the EC400FD case, the analytical peak story displacement compared well with Aiken and Kelly [1990] analytical model. However, the same cannot be said for the peak story shear where the present analytical model tend to significantly under-predict the peak story shear at the lower floors. Reducing the modal damping from 4% to 3% did not improve the correlations. On the other hand, the TF400FD case shows much better correlations as shown in Figure 7.18.

From the above discussion, it is shown that the peak response parameters obtained from both Aiken and Kelly [1990] analytical model and ETABS model did not match very closely with the experimental results. As mentioned above, a linear spring with a constant $K_p$ may not be adequate to capture the nonlinearity. However, the excellent time history correlations suggest that the disparities in the peak response values are not an indication of poor analytical-experimental correlation.

Figure 7.16: Experimental and Analytical Roof Time Histories for TF400FD.
Figure 7.17: Experimental and Analytical Peak Story Displacement and Shear for EC400FD.

Figure 7.18: Experimental and Analytical Peak Story Displacement and Shear for TF400FD.
7.4 COMPARATIVE STUDY

7.4.1 Introduction

The MRF and FD were selected for this phase of the study for performance comparison with the EDCS. Fixed support conditions were assumed for all the models in the comparative study. This assumption permits a common basis for comparison without introducing complication arising from the table-structure interaction. The spring stiffness ($K_p$) for the table-structure interaction is a function of the characteristics of both the test structure and the test conditions and values have been suggested only for the MRF, CBF and FD models. Hence in the absence of experimental data, there appears to be no clear basis for selecting a suitable $K_p$ value for the EDCS system. However, the assumption of a fixed support condition is not unreasonable in building design problems.

The other necessary change that was made to the analytical models was the amount of modal damping for the first two fundamental modes. As discussed by Aiken and Kelly [1990], the modal damping ratios (Table 7.5) determined for the test structures included the damping effect from the shake-table. In fact, at least half of the first-mode damping was attributed to the shake-table mechanism. For fixed base condition, it was suggested that a ratio of less than 1% was appropriate for the bare MRF and about 2% for both CBF and FD to account for the bracing and significant increase in bolted connections attached to the structural frame. For the present study, 1% damping was used for the MRF and the EDCS models and 2% for FD. A higher modal damping of 2% could have been specified for the EDCS model to reflect greater number of connections but a conservative approach might be more suitable.

7.4.2 Analytical Model for Structure with EDCS

The EDCS (at one-quarter scale) was added to the MRF model as shown in Figure 7.19. The modeling approach for the EDCS is similar to that for the Sumitomo friction damper in the FD model. Each EDCS was modeled with three rigid truss elements and a nonlinear LINK element. The truss elements were oriented as shown to model the correct directions of the forces exerted by the EDCS connections on the frame elements. These elements were also specified with a high axial rigidity to limit horizontal deformation to the LINK element only. Except for the
stiffness and slip load (i.e., yield strength), the specification of the ETABS LINK element is identical to that used for the Sumitomo friction dampers.

The distribution of the EDCS stiffness in bays and slip load over the height are summarized in Figure 7.19. The stiffness values are based on the study in Chapter 6 and scaled down to reflect the one-quarter scale of the test model. The stiffness value and slip load for each bay are computed based on two panels, consistent with the configuration of the test models of Aiken and Kelly [1990]. The same stiffness value was used for the same bay width while the same slip load was applied to all three bays at the same floor level. It should be noted that for the FD model, each Sumitomo friction damper was modeled with an infinitely high stiffness (1.8 x 10^5 kN/mm or 10^6 kips/in) even at one-quarter scale. On the other hand, the flexible cantilevered bearing supports considerably reduce the stiffness of the EDCS as discussed in Chapter 6; the similitude requirement further reduced the stiffnesses to the low values shown. The stiffness of the EDCS at the outer bays is about three-and-a-half times that of the interior bay. The slip loads shown were obtained by dividing the actual experiment slip load by three bays at each floor. This may not represent the optimum solution for the EDCS but it provides a common basis for comparison with the FD model. A total of twelve LINK properties were necessary to cover the different combinations of stiffnesses and slip loads.

### 7.4.3 Ground Motions

From each of the three earthquake motions (i.e., ELCEN, TAFT and LACC), three levels of intensity ground motions were generated corresponding to high, moderate and low zones of seismicity. This was accomplished by amplitude-scaling the original earthquake signals using the ASCE7 [2005] procedure for two dimensional seismic response history analysis.
Figure 7.19: Analytical Model of EDCS.

### Stiffness, kN/mm (kips/in)

<table>
<thead>
<tr>
<th>Bay 1</th>
<th>Bay 2</th>
<th>Bay 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>26 (149)</td>
<td>91 (520)</td>
<td>26 (149)</td>
</tr>
</tbody>
</table>

### Slip load, kN (kips)

<table>
<thead>
<tr>
<th>Floor</th>
<th>Slip load, kN (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>7.1 (1.60)</td>
</tr>
<tr>
<td>8</td>
<td>7.1 (1.60)</td>
</tr>
<tr>
<td>7</td>
<td>7.7 (1.73)</td>
</tr>
<tr>
<td>6</td>
<td>14.5 (3.27)</td>
</tr>
<tr>
<td>5</td>
<td>14.8 (3.33)</td>
</tr>
<tr>
<td>4</td>
<td>14.2 (3.20)</td>
</tr>
<tr>
<td>3</td>
<td>14.2 (3.20)</td>
</tr>
<tr>
<td>2</td>
<td>14.5 (3.27)</td>
</tr>
<tr>
<td>1</td>
<td>17.2 (3.87)</td>
</tr>
</tbody>
</table>
The criterion requires the average 5% damped response spectra for the suite of earthquake motions used not fall below the 5% damped design response spectrum for periods between $0.2T_s$ and $1.5T_s$ where $T_s$ is the first-mode period of the structure. The 5% damped design response spectra were generated based on the procedures outlined in Section 11.4.5 of ASCE7 [2005] and are plotted in Figure 7.20. Considering the range of natural periods (0.30 sec and 0.5 sec) of the three test structures, the limiting periods were found to be about 0.06 sec and 0.8 sec.

Here, a single amplitude scale factor was applied to the suite of three earthquake motions. In Figure 7.21, the scaled 5% damped average response spectra were plotted together with the design spectra. The scale factors shown were the minimum values satisfying the above-mentioned criterion. It can be seen that the scale factors were controlled by the descending tail of the response spectra between 0.6 and 0.8 sec. The average response spectra tend to deviate significantly from the target spectra at the periods lower than 0.5 sec. The use of more earthquake ground motions would produce smoother average spectra that would better match the design spectra. The resulting scaled ground motions used for this phase of study are summarized in Table 7.9. These ground motions are applied separately to the three models.
Figure 7.21: Scaled 5% Average Response Spectra.

Table 7.9: Time History Cases.

<table>
<thead>
<tr>
<th>Time history case</th>
<th>Time history source</th>
<th>PGA (g)</th>
<th>Scaling factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Time</td>
<td>Acceleration</td>
</tr>
<tr>
<td>EC1</td>
<td>ELCEN</td>
<td>0.297</td>
<td>0.5</td>
</tr>
<tr>
<td>EC2</td>
<td></td>
<td>0.588</td>
<td>0.5</td>
</tr>
<tr>
<td>EC3</td>
<td></td>
<td>0.882</td>
<td>0.5</td>
</tr>
<tr>
<td>TF1</td>
<td>TAFT</td>
<td>0.169</td>
<td>0.5</td>
</tr>
<tr>
<td>TF2</td>
<td></td>
<td>0.334</td>
<td>0.5</td>
</tr>
<tr>
<td>TF3</td>
<td></td>
<td>0.501</td>
<td>0.5</td>
</tr>
<tr>
<td>LA1</td>
<td>LACC</td>
<td>0.243</td>
<td>0.5</td>
</tr>
<tr>
<td>LA2</td>
<td></td>
<td>0.481</td>
<td>0.5</td>
</tr>
<tr>
<td>LA3</td>
<td></td>
<td>0.721</td>
<td>0.5</td>
</tr>
</tbody>
</table>
It should be pointed out that different scale factors could have been used for each earthquake motion; this approach appears to be suggested by the code. However, it was found that this generally led to the average spectra departing even further away from the target spectra at the short-duration periods and resulting in very high PGA ground motions. The process of minimizing the scale factors became more complex as other judgment-based criteria (e.g., limiting values for the scale factors) have to be specified. Furthermore, the scaled ground motions obtained would have considerably different PGA even at the same intensity level.

### 7.4.4 Analytical Results

#### 7.4.4.1 Natural Periods

The natural periods for the first three modes for each model are summarized in Table 7.10. Since all the models have the same total mass and mass distribution, the relative stiffnesses of FD and EDCS with respect to MRF were calculated by Equations 7.1 and 7.2.

\[
\frac{K_{FD}}{K_{MRF}} = \left( \frac{f_{1,FD}}{f_{1,MRF}} \right)^2
\]  

\[
\frac{K_{EDCS}}{K_{MRF}} = \left( \frac{f_{1,EDCS}}{f_{1,MRF}} \right)^2
\]

Where \( K_{MRF}, K_{FD} \) and \( K_{EDCS} \) are the stiffnesses of the MRF, FD and EDCS models, respectively; \( f_{1,MRF}, f_{1,FD} \) and \( f_{1,EDCS} \) are the first-mode frequencies of the respective frames. As expected, the one-third height of the EDCS does not increase the lateral stiffness of the MRF frame as much as the full-height concentric bracing in the FD model.

It should be pointed out that the modal frequencies for structures with supplemental damping device generally vary with excitations. The periods shown in Table 7.10 were computed based on the assumption of a linear elastic (or effective) stiffness for each friction damper, independent of the degree of slip in the dampers. When structures with supplemental damping device are subjected to ground excitation, the stiffness characteristics change with time.
Generally, for low intensity ground motion, there is little or no slip in the friction dampers and the structure behaves essentially as conventional braced frame. With high-level excitation, significant nonlinear deformation or slip occurs in the dampers resulting in a change in stiffness and hence a shift in the frequencies. Figure 7.22 shows the roof acceleration transform functions for MRF and EDCS. These plots show that while the frequencies for the first three modes of MRF are clearly defined even at high level of ground motion, it was impossible to identify the first-mode frequency from the transform functions for the FD model at the high level of excitation. Once significant slip occurred in the friction dampers, the structure responds in a band of frequencies, in addition to the natural frequencies.

Table 7.10: Natural Periods and Relative Stiffnesses.

<table>
<thead>
<tr>
<th>Model</th>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
<th>Cumulative mass participation factor</th>
<th>Relative stiffness ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF</td>
<td>1</td>
<td>1.96</td>
<td>0.51</td>
<td>84.6</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.05</td>
<td>0.17</td>
<td>94.1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.72</td>
<td>0.09</td>
<td>97.2</td>
<td>-</td>
</tr>
<tr>
<td>FD</td>
<td>1</td>
<td>3.37</td>
<td>0.30</td>
<td>85.0</td>
<td>2.96</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>10.51</td>
<td>0.10</td>
<td>94.4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>19.23</td>
<td>0.05</td>
<td>97.4</td>
<td>-</td>
</tr>
<tr>
<td>EDCS</td>
<td>1</td>
<td>2.28</td>
<td>0.44</td>
<td>80.1</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.99</td>
<td>0.14</td>
<td>93.7</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>12.20</td>
<td>0.08</td>
<td>97.3</td>
<td>-</td>
</tr>
</tbody>
</table>

7.4.4.2 Time History Analysis

The analyses of nine different time history cases and three frames resulted in twenty-seven sets of results. Table 7.11 summarizes the peak response parameters for the three structures. The peak response parameter profiles are included in Appendix C. The only peak displacement was taken at the roof level since the peak values of other response parameters occurred at other floor levels. For the MRF and EDCS, the peak story shear (PSS) corresponds to the base shear while the PSS for the FD occurred at one of the first three levels. The location of
the peak story acceleration (PSA) varies while the peak interstory drift (PID) generally occurred at the second, third or fourth floor level.

Figure 7.22: FFT of Roof Accelerations for MRF, FD and EDCS.
Table 7.11: Peak Response Parameters for MRF, FD and EDCS.

<table>
<thead>
<tr>
<th>Model</th>
<th>Time history</th>
<th>PGA</th>
<th>PRD</th>
<th>PID</th>
<th>PSS</th>
<th>PSA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(g)</td>
<td>(mm)</td>
<td>(in)</td>
<td>(kN)</td>
<td>(kips)</td>
<td>(g)</td>
</tr>
<tr>
<td>MRF</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EC1</td>
<td>0.297</td>
<td>15.9</td>
<td>0.626</td>
<td>0.00297</td>
<td>77.2</td>
<td>17.36</td>
</tr>
<tr>
<td>EC2</td>
<td>0.588</td>
<td>31.5</td>
<td>1.240</td>
<td>0.00588</td>
<td>152.8</td>
<td>34.36</td>
</tr>
<tr>
<td>EC3</td>
<td>0.882</td>
<td>47.2</td>
<td>1.860</td>
<td>0.00881</td>
<td>229.2</td>
<td>51.54</td>
</tr>
<tr>
<td>TF1</td>
<td>0.169</td>
<td>3.9</td>
<td>0.153</td>
<td>0.00070</td>
<td>18.9</td>
<td>4.25</td>
</tr>
<tr>
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</tr>
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</tr>
<tr>
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<td></td>
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<td>0.422</td>
<td>0.00187</td>
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</table>
To facilitate comparison, each peak response parameter for FD and EDCS was divided by the value of the same parameter for MRF (i.e., the bare frame is taken as the control system). The resulting ratios are summarized in Table 7.12. The dissipated energy ratio (DER) shown in the table is defined as the ratio of the total energy dissipated by the friction dampers to the total input energy of the structure; the input energy is the sum of the potential energy, kinetic energy and dissipated energies (through modal damping or supplemental damping device). The DER does not directly quantify the performance of the dampers, nevertheless it does indicate whether the dampers are working (i.e., dissipating energy) properly. For example, a zero DER (e.g., FD under TF1 and LA1) is a sign that no dampers have been activated during the ground motion. Furthermore, maximizing the energy dissipation is typically one of the goals in the design of supplemental dissipating devices.

Table 7.12: Peak Response Parameter Ratios.

<table>
<thead>
<tr>
<th>Model</th>
<th>Time history case</th>
<th>PGA (g)</th>
<th>ΔPRD (%)</th>
<th>ΔPSS (%)</th>
<th>ΔPSA (%)</th>
<th>ΔPID (%)</th>
<th>DER (%)</th>
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<td>38</td>
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<td>-1</td>
<td>-64</td>
<td>71</td>
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<tr>
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<td>EC3</td>
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<td>79</td>
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<tr>
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<td>TF1</td>
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<tr>
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<td>-14</td>
<td>-31</td>
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<tr>
<td></td>
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<td>41</td>
<td>80</td>
<td>-12</td>
<td>59</td>
<td>30</td>
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<td>LA2</td>
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<td>8</td>
<td>35</td>
<td>29</td>
<td>27</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>LA3</td>
<td>0.721</td>
<td>3</td>
<td>13</td>
<td>43</td>
<td>9</td>
<td>65</td>
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</table>
Peak Roof Displacement and Peak Interstory Drift

The peak roof displacement ratios \( \Delta_{\text{PRD}} \) from Table 7.12 are plotted against the PGA in Figure 7.23. For the El Centro (i.e., EC) based ground motions, the presence of supplemental dampers in FD and EDCS generally led to a reduction in PRD for any given PGA. As expected, FD has significantly larger (as much as 68%) reduction than EDCS. The reduction in PRD for the EDCS did not exceed 30%.

With the Kern County (i.e., TF) based ground motions, FD gave significant reduction (between 6% - 57%) in PRD. However, based on the very low dissipated energy (Table 7.12), the observed reduction was due to added stiffness of the chevron bracing rather than energy dissipation. For the EDCS, appreciable reduction (16%) was only observed at the highest intensity ground motion (i.e., TF3). At the lowest PGA of TF1, the PRD for the EDCS actually increased by 11%. However, this increase does not necessarily imply poor performance since the maximum story displacements induced by the Kern County ground motions are very small (less than 0.39 in.) even at the highest considered PGA.

Figure 7.23: Peak Roof Displacement Ratio Versus Peak Ground Acceleration.
With the Northridge (LA) series, the PRDs for the EDCS model are consistently higher (3% to 41%) than those for bare frame. Again, all the displacements are very small with the highest PRD at maximum considered PGA equal to 0.42 in. As the PGA increases, more energy was dissipated by the friction damper, reducing the PRD to a level of the bare frame (i.e., MRF). The PID ratios ($\Delta_{PID}$) versus PGA plotted in Figure 7.24 also show similar trends for all frames.

The main reason that the El Centro gave the greatest reduction in PRD is because the dominating frequencies (3 – 4 Hz) of the El Centro earthquake coincide with the resonant frequencies of the models, resulting in large interstory displacement. The friction dampers, being displacement-controlled, rely on interstory displacement to dissipate energy. Thus with the large interstory displacement produced by the El Centro ground motions, the friction dampers were able to dissipate large amount of energy.

The significantly different results for the suite of three different earthquakes supported the requirement that two or more earthquakes with different frequency content should be used in the performance study of seismic resistant system. This is to account for the randomness of earthquake motions.
Peak Story Shear

The peak story shear ratios ($\Delta_{PSS}$) versus PGA from Table 7.12 are plotted in Figure 7.25. As the intensity of the ground excitation increases, the peak story shear generally increases. For the EC series of ground motions, the PSS for FD and EDCS are lower than the MRF for the PGAs considered. FD has significantly larger (as much as 57%) peak base shear reduction than EDCS at any given PGA. The maximum reduction for the EDCS appeared to cap at 21%.

![Figure 7.25: Peak Story Shear Ratio Versus Peak Ground Acceleration.](image)

For the Kern County (TF) and Northridge (LA) series of ground motions, both FD and EDCS gave significantly higher peak base shear than the MRF. The reason is that the frequency content of these two earthquakes is spread over a greater range than the El Centro ground motion. Even at high PGAs, the activated friction dampers did not slip sufficiently to dissipate the input energy. As a result, the models behaved essentially as (chevron) braced frame configurations with higher fundamental frequencies and story shear (but smaller drift as discussed above). Generally, EDCS exhibited a lower PSS than FD for any given PGA because of the better energy dissipation (discussed later). This clearly shows that comparison with a separate friction damped system (i.e.,
FD) was necessary in obtaining a proper performance evaluation of the EDCS, not just with the controlled bare frame (i.e., MRF).

**Peak Story Acceleration**

The variation of PSA ratios ($\Delta_{PSA}$) with PGA for each ground motions is plotted in Figure 7.26. Except for the El Centro based ground motions, the PSA for the other ground motions were significantly higher than the bare frame due to the added stiffness from the supplemental devices.

![Figure 7.26: Peak Story Acceleration Ratio Versus Peak Ground Acceleration.](image)

**Column Forces and Moments**

One concern for bracing the columns at one-third height in the EDCS is the possible increase in the column forces, in particular the column shear. The peak axial force, peak shear and peak moments for all columns in the EDCS model were checked against the bare frame for each ground motion. Only the plots for the high intensity earthquakes (i.e., EC3, TF3 and LA3) are shown in Figure 7.27; the results for other ground motions are included in Appendix C. The axial forces in the perimeter columns are higher (due to cantilever effect) than the interior columns while the shear forces and bending moments are higher for the interior columns.
Figure 7.27: Peak Column Forces for EDCS and MRF.
As expected, the El Centro based ground motions generally resulted in lower peak column forces and moments, while the ground motions generated from Kern County and Northridge based ground motions generally led to an increase in column forces and moments. However, this increase is not crucial considering that the magnitude of these forces and moments are significantly lower than those generated from El Centro motions. For structural design, it is clear that the El Centro based ground motions would dictate the size of the frame elements despite the fact that other ground motions resulted in an increase in peak response parameters over the bare frame.

Energy Dissipation by Friction Dampers

The DER and associated PGA from Table 7.12 are plotted in Figure 7.28. With increase in PGA, the amount of energy dissipated in the friction dampers generally increases. What is unexpected is that for the same intensity ground motion, the EDCS generally dissipated a greater amount of the input energy (23 – 73%) than the conventional FD (0% – 79%). The distribution of energy dissipation for each model is investigated next.

Figure 7.28: Dissipated Energy Ratio Versus Peak Ground Acceleration.
Because ETABS only reports the total dissipated energy in all the dampers for each model, it was necessary to manually extract load-deformation (i.e., the hysteresis curve) data for each friction damper and numerically integrate it to obtain the dissipated energy. As illustrated in Figure 7.29, at any consecutive time-steps, the shaded area under the load-deformation curve, representing the incremental work done, was computed by numerical integration. The incremental deformation was deemed small enough that the simple trapezoidal rule was adequate. The summation of all the incremental work done for the entire duration of the ground motion is equal to the dissipated or hysteresis energy. Table 7.13 summarizes the amount of energy dissipated by each damper in the FD model for each ground motion. The load-deformation plots are included in Appendix C. The total dissipated energy computed based on the aforementioned procedure agreed well with ETABS values.

![Figure 7.29: Determination of Dissipated Energy in Friction Dampers.](image)

From the Table 7.13, it can be seen that the (Sumitomo) damper on the last floor level did not activate for any of the given ground motions; this was also observed in Aiken and Kelly [1990] experiment. The dampers on the first three floors did not slip for the Kern County and Northridge based ground motions, even at high PGAs. This phenomenon was not observed in the experiment probably because of the table-structure interaction.
The distributions of the energy dissipation over the structure height presented earlier in Table 7.13 are also plotted in Figure 7.30. The plots show distinctive different energy distribution between the El Centro based ground motions and ground motions resulting from the other two earthquakes. For the El Centro based ground motions, the lower floor dampers dissipated the most friction energy. The dampers on middle floors were the most effective for the Kern County and Northridge based ground motions. As mentioned earlier, this was probably due to the different frequency content of the earthquakes. From the dissipated energy characteristics, it is clear that further improvement of the EDCS is possible by changing the slip loads and the slip load variation.

For the EDCS, the dissipated energy distributions are summarized in Table 7.14. Bay 2 refers to the interior bay, while the bays 1 and 3 refer are the exterior bays. The energy dissipation characteristics for the exterior bays are identical since the dampers specified for both exterior bays at each floor are the same. Because of lower slip loads, all friction dampers were able to slip.

### Table 7.13: Distribution of Dissipated Energy for FD.

<table>
<thead>
<tr>
<th>Time History Case</th>
<th>EC1 (kN-mm) (kips-in.)</th>
<th>EC2 (kN-mm) (kips-in.)</th>
<th>EC3 (kN-mm) (kips-in.)</th>
<th>TF1 (kN-mm) (kips-in.)</th>
<th>TF2 (kN-mm) (kips-in.)</th>
<th>TF3 (kN-mm) (kips-in.)</th>
<th>LA1 (kN-mm) (kips-in.)</th>
<th>LA2 (kN-mm) (kips-in.)</th>
<th>LA3 (kN-mm) (kips-in.)</th>
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</thead>
<tbody>
<tr>
<td>EC1 EC2</td>
<td>182 (1.6)</td>
<td>1507 (13.3)</td>
<td>4094 (36.2)</td>
<td>0.0</td>
<td>278 (2.5)</td>
<td>115 (1.0)</td>
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<td>41 (0.4)</td>
<td>167 (1.5)</td>
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<tr>
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<td>4085 (36.2)</td>
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<td>255 (2.3)</td>
<td>103 (0.9)</td>
<td>0.0</td>
<td>37 (0.3)</td>
<td>145 (1.3)</td>
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<tr>
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<td>4085 (36.2)</td>
<td>0.0</td>
<td>255 (2.3)</td>
<td>103 (0.9)</td>
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<td>145 (1.3)</td>
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<tr>
<td>TF1 TF2</td>
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<td>103 (0.9)</td>
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<td>37 (0.3)</td>
<td>145 (1.3)</td>
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<td>145 (1.3)</td>
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<td>0.0</td>
<td>37 (0.3)</td>
<td>145 (1.3)</td>
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<td>37 (0.3)</td>
<td>145 (1.3)</td>
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<td>37 (0.3)</td>
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<tr>
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<td>37 (0.3)</td>
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<tr>
<td>LA2 LA3</td>
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<td>0.0</td>
<td>0.0</td>
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<td>145 (1.3)</td>
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<td>37 (0.3)</td>
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</table>

<table>
<thead>
<tr>
<th>Floor</th>
<th>Energy dissipated by a damper at each floor (% of the total dissipated energy)</th>
<th>Report total dissipated energy, kN-mm (kips-in.)</th>
<th>Calculated total dissipated energy, kN-mm (kips-in.)</th>
<th>Difference (%)</th>
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and dissipate energy, even under low intensity ground motions. The distributions presented in Table 7.14 are plotted in Figure 7.31.

For the El Centro based ground motions, the distributions of energy dissipation are similar across the bays for all PGAs. For the other two earthquakes, the energy dissipation is the largest at the mid levels and lower levels for the exterior bays and interior bay, respectively. However, because the outer bay dampers did not dissipate as much energy as those in the inner bays, the overall energy distribution (thick continuous lines) for all ground motions were dominated by the trends for the inner dampers, giving the similar triangular distribution profile.

The combined energy dissipated in the outer two bays of dampers accounts for only 16% of the total hysteresis energy for EC1. With higher PGA and interstory drift, this value increases to 32% and 42% for EC2 and EC3, respectively. The friction dampers at the inner bay were dissipating two-and-a-half to ten times more input energy than those in the outer bays. This is because the EDCSs in the inner bays were three-and-a-half times stiffer than those in the outer bays. Given the same interstory displacement, the stiffer EDCSs were more likely to slip and have greater inelastic deformation, resulting in greater amount of energy dissipation.

Figure 7.30: Vertical Distribution of Energy Dissipation for FD.
Table 7.14: Distribution of Dissipated Energy for EDCS.

<table>
<thead>
<tr>
<th>Time history case</th>
<th>EC1</th>
<th>EC2</th>
<th>EC3</th>
<th>TF1</th>
<th>TF2</th>
<th>TF3</th>
<th>LA1</th>
<th>LA2</th>
<th>LA3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reported total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>dissipated energy,</td>
<td>550</td>
<td>1896</td>
<td>3864</td>
<td>20</td>
<td>153</td>
<td>355</td>
<td>34</td>
<td>207</td>
<td>437</td>
</tr>
<tr>
<td>kN-mm (kips-in.)</td>
<td>(4.9)</td>
<td>(16.8)</td>
<td>(34.2)</td>
<td>(0.2)</td>
<td>(1.4)</td>
<td>(3.1)</td>
<td>(0.3)</td>
<td>(1.8)</td>
<td>(3.9)</td>
</tr>
<tr>
<td><strong>Calculated total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>dissipated energy,</td>
<td>551</td>
<td>1899</td>
<td>3876</td>
<td>20</td>
<td>167</td>
<td>356</td>
<td>34</td>
<td>207</td>
<td>438</td>
</tr>
<tr>
<td>kN-mm (kips-in.)</td>
<td>(4.9)</td>
<td>(16.8)</td>
<td>(34.3)</td>
<td>(0.2)</td>
<td>(1.5)</td>
<td>(3.2)</td>
<td>(0.3)</td>
<td>(1.8)</td>
<td>(3.9)</td>
</tr>
<tr>
<td><strong>Difference (%)</strong></td>
<td>0.3</td>
<td>0.2</td>
<td>0.3</td>
<td>0.2</td>
<td>9.6</td>
<td>0.2</td>
<td>-0.4</td>
<td>0.0</td>
<td>-0.2</td>
</tr>
<tr>
<td><strong>Total dissipated</strong></td>
<td>46</td>
<td>298</td>
<td>808</td>
<td>2</td>
<td>11</td>
<td>27</td>
<td>3</td>
<td>15</td>
<td>41</td>
</tr>
<tr>
<td>energy for bay 1 or 3,</td>
<td>(0.4)</td>
<td>(2.6)</td>
<td>(7.2)</td>
<td>(0.0)</td>
<td>(0.1)</td>
<td>(0.2)</td>
<td>(0.0)</td>
<td>(0.1)</td>
<td>(0.4)</td>
</tr>
<tr>
<td>kN-mm (kips-in.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor</th>
<th>Energy dissipated by a damper at each floor (% of total energy for bay 1 or 3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>1 1 1 1 1 1 1 1 2 3</td>
</tr>
<tr>
<td>8</td>
<td>3 3 4 3 3 3 3 3 5 6</td>
</tr>
<tr>
<td>7</td>
<td>8 7 7 7 8 8 7 8 10 13</td>
</tr>
<tr>
<td>6</td>
<td>4 5 7 4 4 4 4 4 5 6</td>
</tr>
<tr>
<td>5</td>
<td>21 14 13 42 30 30 34 24 19</td>
</tr>
<tr>
<td>4</td>
<td>14 15 17 10 12 12 10 11 12</td>
</tr>
<tr>
<td>3</td>
<td>20 22 21 13 16 16 14 15 16</td>
</tr>
<tr>
<td>2</td>
<td>26 24 24 18 23 23 23 24 21</td>
</tr>
<tr>
<td>1</td>
<td>4 5 6 3 3 3 4 4 4 4</td>
</tr>
<tr>
<td>Total dissipated energy for bay 2, kN-mm (kips-in.)</td>
<td>459 (4.1)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor</th>
<th>Energy dissipated by a damper at each floor (% of total energy for bay 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>3 4 4 4 3 4 3 5 6</td>
</tr>
<tr>
<td>8</td>
<td>1 1 2 0 1 1 1 1 3</td>
</tr>
<tr>
<td>7</td>
<td>7 7 7 9 9 9 9 9 10</td>
</tr>
<tr>
<td>6</td>
<td>8 9 10 5 7 7 5 7 9</td>
</tr>
<tr>
<td>5</td>
<td>12 13 13 9 11 11 8 11 12</td>
</tr>
<tr>
<td>4</td>
<td>17 16 16 15 16 16 13 15 15</td>
</tr>
<tr>
<td>3</td>
<td>20 19 18 22 20 20 22 19 18</td>
</tr>
<tr>
<td>2</td>
<td>26 23 22 31 27 27 34 26 22</td>
</tr>
<tr>
<td>1</td>
<td>7 9 9 5 5 5 5 6 6</td>
</tr>
</tbody>
</table>
Figure 7.31: Vertical Distribution of Energy Dissipation for EDCS.
For the Kern County and Northridge based ground motions, the dampers in the two outer bays only account for 14 – 18% of the total energy dissipated. These results strongly suggest that better performance of the EDCS can be achieved by specifying a different slip load distribution for each bay and for each floor. This is discussed next.

### 7.4.4.3 Refinement of Slip Load Distribution for EDCS

As evident from the discussion of the energy dissipation, the slip load distribution based on Aiken and Kelly [1990] may not be optimum because of three factors: different stiffnesses across the bays, smaller slip resulting from one-third height bracing configuration and fixed support conditions assumption. Further reduction of the peak response parameters would be possible. Many methods of optimization have been proposed [Filiatrault and Cherry, 1990; Rao and White, 1996; Moreschi, 2000] which varies from optimizing a relative performance index (based on maximum strain energy and deformation) to the inelastic demand spectrum method. An alternative method of refining the slip load for the EDCS is presented here.

Figure 7.32 shows a typical hysteresis loop for a friction damper with a specified stiffness \( k_{EDCS} \) and slip load \( F_{slip} \). For any given maximum slip \( \Delta_{max} \), the dissipated or hysteresis energy \( E_{EDCS} \) is equal to the area enclosed by the parallelogram which is given

\[
E_{EDCS} = 4F_{slip} \left( \Delta_{max} - \frac{F_{slip}}{k_{EDCS}} \right)
\]  

(7.3)

By setting the derivative of Equation 7.3 to zero, the slip load for maximum dissipated energy can be shown to be

\[
F_{slip} = \frac{k_{EDCS} \Delta_{max}}{2}
\]  

(7.4)

Thus by estimating the maximum absolute slip for each damper, the slip load to achieve maximum energy dissipation could be easily found. It should be pointed out that the slip loads obtained from Equation 7.4 may not be feasible in practice or may be limited by the capacity of the framing elements connected to the EDCS. In such cases, the maximum achievable slip loads can be used.
To illustrate the aforementioned procedure, the slip load distribution was refined for EC3, and the new slip load distribution is summarized in Table 7.15. A (new) lower slip load indicates that the original damper was not dissipating as much energy as it could while a higher load means that the damper could dissipate more energy with higher slip load. The procedure generally resulted in higher slip load for each damper and gave a triangular distribution with the highest slip load at the second story. Except for the top two floors, the slip load for the dampers in the exterior bays are about 28% of those in the interior bay, which corresponds to the 3.5 times difference in damper stiffnesses. Here, the slip load is capped at 3.87 kips and this value replaced the “refined” slip loads in the shaded cells.

Table 7.15: Refined Slip Load Distribution.

<table>
<thead>
<tr>
<th>Bay 1 and 3</th>
<th>Old(^1)</th>
<th>New(^2)</th>
<th>Bay 2</th>
<th>Old(^1)</th>
<th>New(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor 1</td>
<td>17.2</td>
<td>13.5</td>
<td></td>
<td>17.2</td>
<td>47.1</td>
</tr>
<tr>
<td></td>
<td>(3.87)</td>
<td>(3.04)</td>
<td></td>
<td>(3.87)</td>
<td>(10.59)</td>
</tr>
<tr>
<td>Floor 2</td>
<td>14.2</td>
<td>23.4</td>
<td></td>
<td>14.2</td>
<td>82.0</td>
</tr>
<tr>
<td></td>
<td>(3.20)</td>
<td>(5.27)</td>
<td></td>
<td>(3.20)</td>
<td>(18.43)</td>
</tr>
<tr>
<td>Floor 3</td>
<td>14.8</td>
<td>21.4</td>
<td></td>
<td>14.8</td>
<td>75.3</td>
</tr>
<tr>
<td></td>
<td>(3.33)</td>
<td>(4.82)</td>
<td></td>
<td>(3.33)</td>
<td>(16.92)</td>
</tr>
<tr>
<td>Floor 4</td>
<td>14.5</td>
<td>18.7</td>
<td></td>
<td>14.5</td>
<td>65.2</td>
</tr>
<tr>
<td></td>
<td>(3.27)</td>
<td>(4.20)</td>
<td></td>
<td>(3.27)</td>
<td>(14.65)</td>
</tr>
<tr>
<td>Floor 5</td>
<td>7.7</td>
<td>15.7</td>
<td></td>
<td>7.7</td>
<td>55.8</td>
</tr>
<tr>
<td></td>
<td>(1.73)</td>
<td>(3.53)</td>
<td></td>
<td>(1.73)</td>
<td>(12.55)</td>
</tr>
<tr>
<td>Floor 6</td>
<td>7.1</td>
<td>13.0</td>
<td></td>
<td>7.1</td>
<td>46.1</td>
</tr>
<tr>
<td></td>
<td>(1.60)</td>
<td>(2.93)</td>
<td></td>
<td>(1.60)</td>
<td>(10.36)</td>
</tr>
<tr>
<td>Floor 7</td>
<td>7.1</td>
<td>11.2</td>
<td></td>
<td>7.1</td>
<td>39.0</td>
</tr>
<tr>
<td></td>
<td>(1.60)</td>
<td>(2.52)</td>
<td></td>
<td>(1.60)</td>
<td>(8.76)</td>
</tr>
<tr>
<td>Floor 8</td>
<td>5.1</td>
<td>8.0</td>
<td></td>
<td>5.1</td>
<td>16.8</td>
</tr>
<tr>
<td></td>
<td>(1.14)</td>
<td>(1.80)</td>
<td></td>
<td>(1.14)</td>
<td>(3.78)</td>
</tr>
<tr>
<td>Floor 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>27.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(6.26)</td>
</tr>
</tbody>
</table>

\(^1\) Distribution based on Aiken and Kelly [1990].

\(^2\) New distribution.
The plot of the peak response values are shown in Figure 7.33. With the new distribution, it is clear that with the exception of PSA, all other response parameters were further reduced. With the new slip load distribution, the DER is 81% compared to 73% for the old distribution.

Figure 7.33: Effect of Slip Load Distribution on Peak Response Parameters for EC3.
Comparing the EDCS PRD profile with that from the MRF with different amount of viscous damping (Figure 7.34), the equivalent damping offered by the EDCS is about 7.3%. For the present study, with the bare frame at 1.0% viscous damping, the EDCS system added about 6.3% damping to the MRF. This may not appear to be large. However, it should be realized that the structure considered so far has only three bays. The energy dissipating capability and true benefit of the present EDCS would be realized for buildings with more (realistic) numbers of bays. It should be pointed out that only a single iteration was performed for the above example. In the actual optimization process, successive iterations are required to establish the optimum slip load configuration.

![Figure 7.34: Determining Equivalent Modal Damping of EDCS.](image)

### 7.5 SUMMARY

The test structures of Aiken and Kelly [1990] were modeled with ETABS. The good correlations with the experimental data suggested that the analytical representation can
realistically model the dynamic response of the test structures under different levels of earthquake motions. The modeling strategies from the correlation study were extended to the EDCS. It was found that with careful choice of the slip load, the EDCS can reduce the seismic response of 3-bay, 9-story moment resisting frame through the action of increased stiffness and energy absorption. A simple method of improving the slip load distribution of EDCS was presented. In actual buildings with greater numbers of bays, the EDCS is expected to improve the seismic performance of moment-resisting frames much more than what was observed for the 3-bay test models.

7.6 CONCLUSION

The generally good correlations obtained from the large number of nonlinear analyses performed on three different test structures (MRF, CBF and FD) gave much confidence to extend the present analytical models for the performance study of EDCS. These analytical models provide the opportunity to make meaningful evaluation of the performance of EDCS by comparing it with realistic and well-understood conventional moment resisting frames with and without supplemental energy absorbing devices. The correlation studies also confirmed the appropriateness of modeling the friction damper as an elastic-perfectly plastic spring. Based on the simplified assumptions adopted for the analytical study, the comparative study has clearly shown that the EDCS is capable of improving the seismic performance of conventional moment resisting frames through a combination of added stiffness and energy dissipation. By careful selection of the slip load for each damper and the slip load distribution over the structure height and width, energy dissipation and reduction in response parameters could be maximized. For the three bay structure considered in the study, the FD performed significantly better than the EDCS. The EDCS improved the seismic performance of the bare frame but at a lesser extent than FD due to smaller deformation at one-third height configuration. However, as pointed out by Aiken and Kelly [1990], the excellent performance of the proprietary Sumitomo friction dampers comes with a (cost) premium. The present EDCS may perform significantly better as the number of bays (or story) is increased.
CHAPTER 8
BUILDING CASE STUDY

8.1 CHAPTER OVERVIEW

The performance of the EDCS was assessed at one-quarter scale through a series of numerical analysis in the previous chapter. The analytical results revealed that the EDCS was capable of improving the seismic performance of the steel moment frame under strong ground motions. The application of the EDCS on an actual building was investigated and is presented in the present chapter. The objectives of the building case study are to demonstrate the design of the EDCS for an actual building with ordinary moment frame as the LFRS and to compare the design with that obtained from conventional earthquake-resistant design.

Following the chapter overview, a brief discussion is made on the difference in the design philosophies between conventional earthquake-resistance design of ductile moment frames and the design of structures with EDCS. This is followed by a brief description of the building adopted for the present study and the corresponding mathematical models. The loading characteristics and other design parameters are then summarized next. Finally, the resulting design is presented and discussed.

8.2 BACKGROUND

In conventional earthquake-resistant design, the structure response to the design ground motion can be significantly damped through the action of inelastic deformation typically concentrated at the beam-column connections or specially detailed energy absorbing framing elements (e.g., shear link in an eccentrically braced frame). When properly designed and constructed, these energy absorbing elements are expected to sustain many cycles of irreversible inelastic deformation without collapse in the event of an earthquake. In principle, these elements would be replaced at the end of a severe earthquake event because of the presence of large plastic
deformations. However, these elements normally are integrated parts of the structure and replacement may be difficult and very costly. For example, the moderate 1994 Northridge earthquake resulted in the insurers paying out as much as USD11.4 billions for property damage [Butterworth, 1999]. In terms of structural design analysis, the response modification factor or R-factor is intended to simplify the design process by describing the hysteretic energy dissipation and added damping provided by the conventional LFRS with a single number.

The EDCS investigated in the present study belongs to a new category of earthquake-resistant structures known as damage tolerant structures or damage-controlled structures (DCS) [Wada et al., 1992]. DCS also rely on the action of supplemental energy dissipation mechanisms to reduce the dynamic response of the structure. However, unlike the conventional approach of strong-column weak-beam, the DCS does not demand inelastic behavior from the structural frame (i.e., the frame remains essentially elastic). The inelastic demand is limited to the supplemental energy dissipating devices that are added to the structural system but are not an integrated part of the structure [SEAOC, 1999]. As a result, the integrity of the structure or building remains essentially intact even after an earthquake event. Furthermore, these “disposable” supplemental energy dissipating devices could be easily adjusted or replaced.

For the design of DCS, the R-factor is not used. Instead, the damping ratio is usually the primary design parameter for adding dampers to the structure [Hanson and Soong, 2001]. The amount of damping provided by the supplemental energy dissipation devices depends greatly on many factors like building type, damper characteristics, layout of the dampers, etc. that would vary significantly from one building to another. In principle, an R-factor could be established for a specific configuration of the supplemental energy dissipation devices based on the methods outlined for conventional LFRS’s [ATC, 1995; Whittaker et al., 1999; Mitchell et al., 2003; Tsopeles and Husain, 2004]. However, a single-value R-factor would only apply to a single configuration (e.g., single slip load distribution) of the supplemental energy dissipation devices for the given building. This is contrary to the design principles of supplemental energy dissipation devices where the design is varied to achieve the performance target. Furthermore, describing a single-value R-factor would probably necessitate the need for more prescriptive (but restrictive) design requirements.

As suggested by Hanson and Soong [2001], the design of a building with supplemental dampers starts with the determination of the required damping ratio for the given design earthquake and performance requirements. Next, the configuration and locations of the dampers are selected and the damping provided is checked against the required damping. If the desired
high. 140 mm (5.5 in.) thick composite floor decks were installed as part of floor system and
approximately one-third height 114 mm (4.5 in.) thick architectural spandrel type precast concrete
cladding panel covered the entire building facade.

Based on the reported seismic parameters, it was assumed that the building was located in
Oakland, California. The building was constructed almost a decade before the devastating 1989
Loma Prieta earthquake in San Francisco Bay area. In the longitudinal direction, the 13-bay
perimeter frames were designed as SMF and made up the LFRS (Figure 8.2). In the 3-bay wide
transverse direction, combinations of SMF and CBF were designed to resist the lateral loading.
The analytical studies of Goodno et al. [1998] were limited to two-dimensional analyses of the
longitudinal LFRS and information relevant to it was reported. In view of the limited information
available, only the longitudinal LFRS would be investigated here.

The basic moment frame on which the EDCS is implemented could have been OMF,
IMF or SMF. As mentioned earlier, the design approach of DCS assumes that the moment frame
remains essentially elastic and all inelastic actions are provided by the supplemental energy
dissipating devices, in this case, the EDCS. As a result, the original moment frame was designed
as an ordinary moment frame (OMF) which is typically assumed to have negligible inelastic
rotation capacity [AISC, 2002]. In terms of design category, SDC D (or higher) could have been
assumed. However, based on the design under SDC C (discussed later), SDC D or higher would
result in significantly heavier sections that may not be practical; the ductile frames should be
more suited for the higher design categories. As a result, the building was assumed to be located
on a site where SDC C would apply.

![Figure 8.1: Typical Floor Plan.](image)
8.3.1 Mathematical Model

8.3.1.1 Basic Moment Frame (BMF)

The structure was modeled in ETABS as fixed-supported and consisting of 260 beam elements and 280 column elements. The bare moment frame is labeled as BMF. To complement ETABS automated design capability, the columns and beams were divided into a total of thirty-
damping ratio is not achieved, the design is reiterated with a new configuration and locations of the dampers. Similar to other structural design methods, it is an iterative process.

For the present study, a conservative approach is adopted where the moment frame was not expected to contribute to the inelastic demand of the design earthquake. Such an approach may be specified by building owner or building codes for buildings designated as facilities representing a large hazard to the human life in the event of collapse (e.g., school facilities, detention facilities, etc.) or as essential facilities (e.g., police stations, hospitals). The approach could also be a retrofit solution to existing buildings with limited inelastic capacity. The EDCS could be incorporated into special ductile frames (e.g., immediate moment frame or special moment frame) where the inelastic responses and the energy dissipation mechanisms are provided by both the framing elements and the EDCS. In this case, the inelastic actions of the framing elements, in addition to the EDCS, must be explicitly accounted for in the mathematical modeling. This could be achieved with special FEA software like DRAIN-2D or DRAIN-3D that can explicitly model the nonlinear hinges of both steel and concrete elements. This is, however, beyond the scope of the present study. Here, all moment frames were modeled in ETABS to behave elastically under the design ground motions; all inelastic deformation and hysteretic energy dissipation were modeled through the EDCS. The modeling approach is discussed in more detail in the following sections.

.3 BUILDING DESCRIPTION

The building studied is a 20-story steel moment frame building adopted from an office building originally investigated by Goodno et al. [1998]. In their analytical studies, Goodno et al. [1998] investigated the use of advanced cladding connectors to improve the seismic performance of a contemporary building. The study showed that up to 16.8% in reduction in steel weight (as a result of using smaller framing sections), as compared to the baseline building design, could be achieved with the advanced connectors. The office building was reported to be constructed in the early 80’s and was located in the West Coast seismic zone (the building identity and its exact location were not reported). As shown in Figure 8.1, the building measures approximately 81 m (267 ft) by 28 m (92 ft) on plan and is about 76 m (251 ft) high. The building does not possess any form of irregularities on plan or elevation. Except for the ground and last floors, which measured about 4.9 m (16 ft) and 2.9 m (9 ft 6 in.), respectively, each floor is 3.8 m (12 ft 6 in.)
two design groups corresponding to the location of the elements at the start of the design process. The purpose of grouping the elements is to reduce the number of different steel sections required and to avoid odd variation of the steel section either at each floor or over the building height. The grouping was adopted from the design of the as-built building.

Instead of assigning a known section to each framing element, it was necessary to first define a list of steel sections (Table 8.1) from which ETABS would automatically assign to each element as part of its iterative design process. To improve constructability, only W14 sections were specified for the columns, with the heaviest section being W14 x 211. For the beams, the sections were selected based on strong-axis plastic section modulus and weight consideration. The lightest sections (instead of the median) were specified as the starting sections for both columns and beams. The moment frame would be designed as an OMF. All steel design was performed automatically by ETABS in accordance to AISC-LRFD99 [AISC, 1999].

Table 8.1: Specified Section List.

<table>
<thead>
<tr>
<th>Elements</th>
<th>Shape</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>W14</td>
<td>From 22 (starting section) to 233</td>
</tr>
<tr>
<td>Beams</td>
<td>W18</td>
<td>35 (starting section), 40</td>
</tr>
<tr>
<td></td>
<td>W21</td>
<td>44, 50</td>
</tr>
<tr>
<td></td>
<td>W24</td>
<td>55, 62, 68, 76</td>
</tr>
<tr>
<td></td>
<td>W27</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td>W30</td>
<td>90, 108, 116</td>
</tr>
</tbody>
</table>

8.3.1.2 Moment Frame with EDCS

The EDCS was added to the final design of BMF. The new structure, as shown in Figure 8.3, is labeled MFEDCS. The EDCS’s were arranged to maintain the geometrical symmetry of the structure. The MFEDCS was modeled in ETABS the same way as that described previously for the 9-story moment frame in Chapters 3 and 7; the modeling strategies of the friction damper and EDCS in ETABS have been discussed earlier in Sections 3.4.1.2 and 7.4.2, respectively. 114 mm (4.5 in.) thick cladding panels were installed in the as-built building. Based on the numerical results presented in Chapters 5 and 6, thicker panels should be used to avoid substantial flexure
and cracking due to the action of the couple at the loading point. Therefore, 152 mm (6 in.) thick precast concrete cladding panels were specified here. The calculated panel heights were 2.0 m (78 in.), 2.6 m (104 in.), 2.3 m (90 in.) for the 20th floor, 1st floor and all other floors, respectively.

The panel stiffness for different floor height was estimated using the simplified model outlined in Chapter 6. Although three different stiffness values could have been used for three different floor heights, the lowest value of 169 kN/mm (967 kips/in) was conservatively assigned to the elastic-perfectly plastic spring for all EDCS units. The stiffness coefficients of the various components of the EDCS are presented in Table 8.2. Different slip loads of 4.5 kN (1 kips), 22.2 kN (5 kips), 44.5 kN (10 kips), 89.0 kN (20 kips) and 133.5 kN (30 kips) were used.

<table>
<thead>
<tr>
<th>Units</th>
<th>$K_{C_{axial}}$</th>
<th>$K_{C_{shear}}$</th>
<th>$K_{C_{flexure}}$</th>
<th>$K_{Concrete}$</th>
<th>$K_{Panel}$</th>
<th>$K_{Bearing}$</th>
<th>$K_{EDCS}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(kN/mm)</td>
<td>2.10 x 10^3</td>
<td>9.50 x 10^3</td>
<td>2.49 x 10^4</td>
<td>1.71 x 10^3</td>
<td>342</td>
<td>335</td>
<td>169</td>
</tr>
<tr>
<td>(kips/in)</td>
<td>1.20 x 10^4</td>
<td>5.42 x 10^4</td>
<td>1.42 x 10^5</td>
<td>9.77 x 10^5</td>
<td>1.95 x 10^3</td>
<td>1.92 x 10^3</td>
<td>967</td>
</tr>
</tbody>
</table>

Design parameters (All symbols previously defined in Chapter 6):

$H = 1.98$ m (6 ft 12 in.), $L = 5.41$ m (17 ft 9 in.), $t = 152$ mm (6 in.), $\beta = 2.99$

$a_l = 356$ mm (14 in.), $a_b = 660$ mm (26 in.), $b_l = 457$ mm (18 in.), $b_b = 965$ mm (38 in.),

$E_{bearing} = 2 \times 10^5$ MPa (29 x 10^6 psi), $E_{panel} = 2.8 \times 10^4$ MPa (4 x 10^6 psi), $I_{bearing} = 2.0 \times 10^7$ mm^4 (48.3 in^4),

$I_{panel} = 5.84 \times 10^8$ mm^4 (1,404 in^4), $L_{panel} = 5.4$ m (213 in.), $L_{bearing} = 330$ mm (13 in.).

### 8.4 DESIGN APPROACH

Figure 8.4 summarized the design procedure used for the present building study. The steps for the design of the moment frame as an ordinary moment frame followed those of conventional earthquake-resistant design. The framing members were designed automatically by ETABS to satisfy strength requirement. Next, the interstory drift was checked and the sizes of the framing members were manually adjusted (usually increased) to reduce the interstory drift to the allowable limit of 2%. A second analysis was performed using the linear time-history procedure to check the design from the static procedure. For both elastic static and linear time history
analysis, a response modification factor of 3.5 was used to reduce the design earthquake to the design building force.

The MFEDCS was designed at a higher level of performance than the BMF; the frame was designed to remain elastic while achieving interstory drift of not more than 1%. The member sizes determined for the BMF were used for the MFEDCS and were not changed in the design process. Instead, the slip load of the EDCS was adjusted so that no framing member exceeded its design capacity (by checking the combined stress index) while achieving the design objectives. Because the MFEDCS was analyzed using the nonlinear time history procedure that directly accounts for the nonlinear hysteretic behavior of the structure’s components (in this case the EDCS; not the framing members), no response modification factor was applied to the design earthquake ground motions [ICC, 2006].
Figure 8.4: Design Flowchart.
8.5 LOADING CHARACTERISTIC

8.5.1 Gravity Loading

The estimated dead and live loads acting on the moment frame are summarized in Table 8.3. Based on the estimated gravity loads, the equivalent concentrated loads were calculated and applied to the beam-column joints and the mid-span of the beams.

Table 8.3: Gravity Loading.

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(kN/m²)</td>
</tr>
<tr>
<td>Dead (D)</td>
<td>Self-weight of framing element</td>
<td>Automatically calculated by ETABS</td>
</tr>
<tr>
<td></td>
<td>Roof (140 mm (5.5 in.) composite roof deck, waterproofing membrane, insulation, M&amp;E, etc.)</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>Floor (140 mm (5.5 in.) composite floor deck, M&amp;E, ceiling, lighting, partition, etc.)</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>152 mm (6 in.) precast concrete cladding panel</td>
<td>19.0 kN/m height</td>
</tr>
<tr>
<td>Live (L)</td>
<td>Roof</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>Floor (No reduction)</td>
<td>4.8</td>
</tr>
</tbody>
</table>

8.5.2 Seismic Loading

8.5.2.1 Seismic Design Parameters

As explained above, the building was designed for SDC C for this study. The seismic design parameters used are summarized in Table 8.4. The Maximum Credible Earthquake (MCE) ground accelerations were based on the upper limits defined for SDC C in IBC2006 [ICC, 2006].
It was assumed that the office building was not intended for storage purpose and the live load need not be considered in the seismic weight. In addition to the dead load acting on the moment frame, lump masses were assigned to all the beam-column joints to account for the tributary effective seismic weight at each floor. Lump masses of 18,416 kg (0.105 kips-sec\(^2\)/in) and 19,549 kg (0.112 kips-sec\(^2\)/in) were added to the joints at the roof level and every floor level, respectively. The total effective seismic weight of the building with the final design sections was 95,294 kN (21,423 kips). For Goodno et al. [1998] study, the reported weight was 90,241 kN (20,287 kips).

### 8.5.2.3 Equivalent Lateral Force

The bare moment frame was designed based on IBC2006 [ICC, 2006] equivalent lateral force (ELF) (i.e., linear static) procedure. The equivalent static lateral earthquake load (E) was generated based on the seismic parameters shown in Table 8.4. The total design base shear was 2,100 kN (472 kips) based on the minimum seismic response coefficient (\(C_s\)) of 0.0220. The
seismic base shear was distributed over the building height as shown in Figure 8.5. The lateral load was applied onto the moment frame in both opposite directions to achieve the same stresses and hence design results on both sides of the building centerline. It should be pointed out that the approximate ELP was necessary to establish the code-specified minimum base shear.

8.5.2.4 Ground Motion Selection and Scaling

For the MFEDCS, nonlinear time history analysis was necessary to directly capture the nonlinear response of the EDCS. Three ground motions recorded at different stations for the 1989 Loma Prieta earthquake (magnitude of 6.9) were selected for the time history analysis and are summarized in Table 8.5. The time histories and FFT’s are plotted in Figure 8.6. The ground motions are in a list of the earthquake records recommended by ATC-40 [ATC, 1996] for soil sites greater than 10 km (6.25 miles) from the earthquake epicenter. The dominating frequencies of these three ground motions varied a wide range from 0.2 to 2.5 Hz which coincide with the natural frequencies (0.16 to 1.16 Hz) of the structures under consideration. The HOLLISTEW ground motion has the lowest PGA but its dominating frequencies (about 0.2 to 0.5 Hz) matched very closely with the fundamental frequencies of MFEDCS. Hence, this low amplitude ground motion would have the greatest damage potential on the moment frame at any given PGA. All ground motions were obtained from PEER.

The three ground motions were scaled with the average of the three ground motion response spectra exceeding the target design spectrum between the period limits of 0.62 sec and 4.63 sec as shown in Figure 8.7. Instead of a single amplitude scaling factor for the suite of ground motions (as used in the comparative study in Chapter 7), three different factors were used to achieve similar magnitude building base shears from the three ground motions. This also resulted in the average response spectrum matching closer to the design spectrum than compared to the case where a single scaling factor was used. The scaled time history cases are summarized in Table 8.6.
Figure 8.5: Vertical Distribution of Equivalent Earthquake Forces.

Table 8.5: Recorded Strong Ground Motions for 1989 Loma Prieta Earthquake.

<table>
<thead>
<tr>
<th>Station</th>
<th>Comp.</th>
<th>PGA (g)</th>
<th>Number of data points</th>
<th>Time interval (sec)</th>
<th>Time history label</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gilroy Array #2</td>
<td>N00E</td>
<td>0.367</td>
<td>7,990</td>
<td>0.005</td>
<td>GILROY</td>
</tr>
<tr>
<td>Hollister – South &amp; Pine</td>
<td>N00E</td>
<td>0.371</td>
<td>11,991</td>
<td>0.005</td>
<td>HOLLISTNS</td>
</tr>
<tr>
<td>Hollister – South &amp; Pine</td>
<td>N90E</td>
<td>0.177</td>
<td>11,991</td>
<td>0.005</td>
<td>HOLLISTEWEW</td>
</tr>
</tbody>
</table>
Figure 8.6: Time History and FFT of Selected Ground Motions.
Figure 8.7: 5% Damped Design Spectra and Average Response Spectra.

Table 8.6: Scaled Ground Motion Time History Cases.

<table>
<thead>
<tr>
<th>Time history case</th>
<th>Time history source</th>
<th>Acceleration scaling factors</th>
<th>Scaled PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOMAP1</td>
<td>GILROY</td>
<td>1.506</td>
<td>0.553</td>
</tr>
<tr>
<td>LOMAP2</td>
<td>HOLLISTNS</td>
<td>0.569</td>
<td>0.211</td>
</tr>
<tr>
<td>LOMAP3</td>
<td>HOLLISTEW</td>
<td>0.600</td>
<td>0.106</td>
</tr>
</tbody>
</table>
8.5.3 Load Combination

The load combinations used for the steel design were based on IBC2006 [IBCO, 2006] and summarized in Table 8.7. In addition, as required by AISC LRFD99 [AISC, 1999], the design for the columns was checked for the combination of \((1.2D + 0.5L + \Omega_\alpha Q_E)\) where the factored axial capacity exceeds 40% of the design axial load.

Table 8.7: Load Combination.

<table>
<thead>
<tr>
<th>Load combo label</th>
<th>IBC2006 definition</th>
<th>Analysis type</th>
<th>Definition in ETABS</th>
</tr>
</thead>
<tbody>
<tr>
<td>IBC161</td>
<td>1.4D</td>
<td>Static</td>
<td>1.4D</td>
</tr>
<tr>
<td>IBC162</td>
<td>1.2D + 1.6L</td>
<td>Static</td>
<td>1.2D + 1.6L</td>
</tr>
<tr>
<td>IBC165ELP</td>
<td>1.2D + 0.5L + E</td>
<td>Static</td>
<td>1.3D + 0.5L + E</td>
</tr>
<tr>
<td>IBC165LOMAP1</td>
<td>1.2D + 0.5L + E</td>
<td>Time history</td>
<td>1.3D + 0.5L + LOMAP1</td>
</tr>
<tr>
<td>IBC165LOMAP2</td>
<td>1.2D + 0.5L + E</td>
<td>Time history</td>
<td>1.3D + 0.5L + LOMAP2</td>
</tr>
<tr>
<td>IBC165LOMAP3</td>
<td>1.2D + 0.5L + E</td>
<td>Time history</td>
<td>1.3D + 0.5L + LOMAP3</td>
</tr>
</tbody>
</table>

Note: D: Dead; L: Live; Earthquake: \(E = \rho Q_0 + 0.2S_{\phi\alpha}D = Q_E + 0.1D.\)

8.6 ANALYTICAL RESULTS

The two frames (i.e., moment frame without and with EDCS) were analyzed with three different methods (ELP, linear and non-linear time history). All columns and beams were sized using ETABS built-in design capability and in accordance to the same design criteria. The design process was an iterative process involving multiple analyses and design checks to satisfy code-specified strength and serviceability requirement. P-\(\Delta\) effects were included in the linear static analysis of the BMF but were found to be insignificant. As a result, P-\(\Delta\) effects were not considered for the stiffer MFEDCS.
8.6.1 Bare Moment Frame

8.6.1.1 Design Section Based on Equivalent Static Procedure

Table 8.8 summarizes the distribution of W-shapes used for the BMF. In the initial state of the design process, the framing members were designed to satisfy strength requirement. Next, the sizes of the beams and columns were adjusted (usually increased) to reduce the interstory drift to the allowable limit of 2%.

Table 8.8: Distribution of Sections for BMF.

<table>
<thead>
<tr>
<th>Beam</th>
<th>W-shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>Story</td>
<td></td>
</tr>
<tr>
<td>Roof (20th)</td>
<td>W18 x 35</td>
</tr>
<tr>
<td>15th to 19th</td>
<td>W18 x 40</td>
</tr>
<tr>
<td>11th to 14th</td>
<td>W21 x 44</td>
</tr>
<tr>
<td>1st to 10th</td>
<td>W21 x 50</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Column</th>
<th>All other column lines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>Column lines A and N¹</td>
</tr>
<tr>
<td>20th</td>
<td>W14 x 22</td>
</tr>
<tr>
<td>18th to 19th</td>
<td>W14 x 30</td>
</tr>
<tr>
<td>16th to 17th</td>
<td>W14 x 43</td>
</tr>
<tr>
<td>14th to 15th</td>
<td>W14 x 61</td>
</tr>
<tr>
<td>12th to 13th</td>
<td>W14 x 68</td>
</tr>
<tr>
<td>10th to 11th</td>
<td>W14 x 82</td>
</tr>
<tr>
<td>8th to 9th</td>
<td>W14 x 99</td>
</tr>
<tr>
<td>6th to 7th</td>
<td>W14 x 109</td>
</tr>
<tr>
<td>4th to 5th</td>
<td>W14 x 120</td>
</tr>
<tr>
<td>2nd to 3rd</td>
<td>W14 x 132</td>
</tr>
<tr>
<td>1st</td>
<td>W14 x 159</td>
</tr>
</tbody>
</table>

¹ See Figure 8.2 for locations of column lines.
For the final design, a total of four and eighteen different W-shapes were used for the beams and columns, respectively. The total steel weight of the BMF was 2,544 kN (572 kips), which represented only 0.6% of the estimated effective seismic weight. The total weight of the original SMF analyzed by Goodno et al. [1998] was 4,746 kN (1,067 kips) due to higher earthquake loading. Figure 8.8 shows the peak design interstory drift and peak design story shear of the BMF subjected to the equivalent earthquake static force. A deflection amplification factor ($C_D$) of 3.0 for OMF was applied to the elastic interstory drift to bring it to the design (inelastic) level. The columns on the 1st, 18th, 19th and 20th floors were sized in accordance to strength, while the columns on the other floors were sized primarily to satisfy the allowable drift limit.

![Figure 8.8: BMF Peak Interstory Drift and Story Shear from ELP.](image)
8.6.1.2 Design Section Determined from Linear Response Time History Analysis

Linear (i.e., elastic) dynamic analysis was also performed for the structural design of the BMF using the three design ground motions defined in Section 8.5.2.4. 5% damping was specified for the time history analysis consistent with the design spectra. The primary objective is to check the design based on the simplified ELP. The distributions of elastic (and design level) peak interstory drift and story shear are presented in Figures 8.9 and 8.10, respectively. The (scaled) design values for the response parameters were obtained by multiplying the elastic values with the quantities $I/R$ (i.e., ratio of seismic importance factor over the response modification factor). As required by IBC2006 [ICC, 2006], the design values were further scaled up by a factor of 2.25 to satisfy the minimum base shear of 2,100 kN (472 kips) established earlier by the static procedure (i.e., ELP).

![Figure 8.9: BMF Peak Interstory Drift from Linear Time History Analysis.](image)

A low scale factor was used for the HOLLISTEW ground motion (Table 8.6), resulting in a rather low PGA of 0.106g. However, as evident from Figures 8.9 and 8.10, even at this relatively low PGA, the ground motion was still strong enough to induce a building response matching those from the other ground motions with significantly higher PGA’s. The structural
steel design results based on the linear time history analysis was generally consistent with that from the ELP. However, the combined stress index of some framing members at the top floors exceeded unity and hence had to be redesigned. Table 8.9 summarizes the new design sections. The total steel weight of the BMF was increased slightly to 2,567 kN (577 kips). The peak interstory drift and peak story shear profiles did not change significantly from those shown in Figures 8.9 and 8.10, respectively.

![Figure 8.10: BMF Peak Story Shear from Linear Time History Analysis.](image)

Table 8.9: New Framing Member Sizes for BMF.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Story</th>
<th>W-shape</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>17th</td>
<td>W21 x 44</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
</tr>
<tr>
<td>16th to 18th</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
8.6.1.3 Natural Periods

The natural periods for the first five modes are summarized in Table 8.10. Due to significantly lower frame stiffness resulting from smaller framing sections, the periods for BMF were twice as high as those obtained for the frame analyzed by Goodno et al. [1998]. The reported periods for the first three modes were 3.313 sec, 1.235 sec and 0.730 sec.

Table 8.10: Natural Periods of BMF.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
<th>Cumulative mass participation factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.16</td>
<td>6.22</td>
<td>78.9</td>
</tr>
<tr>
<td>2</td>
<td>0.44</td>
<td>2.28</td>
<td>90.0</td>
</tr>
<tr>
<td>3</td>
<td>0.74</td>
<td>1.34</td>
<td>93.9</td>
</tr>
<tr>
<td>4</td>
<td>1.06</td>
<td>0.95</td>
<td>95.9</td>
</tr>
<tr>
<td>5</td>
<td>1.37</td>
<td>0.73</td>
<td>97.1</td>
</tr>
</tbody>
</table>

8.6.2 Moment Frame with EDCS

8.6.2.1 Selection of Slip Load

A series of nonlinear time history analyses were performed on the MFEDCS with varying slip loads of 4.5 kN (1 kips), 22.2 kN (5 kips), 44.5 kN (10 kips), 89.0 kN (20 kips) and 133.5 kN (30 kips) to establish the appropriate slip load level. A single slip load value was found to be adequate for the entire building. With a single slip load, the analytical results revealed that only a number of EDCS’s at the 18th and 19th floors did not develop adequate hysteretic loops (hence low energy dissipation) for one or two of the design ground motions. However, this did not warrant the need to optimize the slip load distribution since actual earthquake ground motions would vary considerably in amplitude and frequency content.

An example of the analytical hysteresis loops of the friction damper is shown Figure 8.11. The deformation is (about three to four times) smaller than that of conventional friction-based systems due to one-third height placement. As a result, the energy dissipation of a
single EDCS unit is smaller but by virtue of the numbers, the overall energy dissipation of the EDCS can be significant. In the example shown in Figure 8.11, the maximum amount of the slip is about 5 mm (0.2in) on the hysteresis loop on the left. The effects of varying the slip load on the peak story shear and peak interstory drift are presented in Figure 8.12 through Figure 8.17.

![Analytical Hysteresis Loops](image)

**Figure 8.11:** Analytical Hysteresis Loops.

![Peak Story Shear](image)

**Figure 8.12:** Peak Story Shear Under LOMAP1.
Figure 8.13: Peak Story Shear Under LOMAP2.

Figure 8.14: Peak Story Shear Under LOMAP3.
Figure 8.15: Peak Interstory Drift Under LOMAP1.

Figure 8.16: Peak Interstory Drift Under LOMAP2.
As expected, increasing the slip load increases the stiffness of the moment frame, thereby attracting more story shear (esp. at the lower stories) while reducing the peak interstory drift. It should be pointed out that the drift profile for the bare frame (i.e., without EDCS) was included for comparison purposes only and does not indicate the actual behavior of the bare frame under the design earthquake.

The selection of the slip load could be based on any one or combinations of the following criteria [Aiken et al., 1988]:

1. The friction dampers in the EDCS should not activate or slip under the design wind load.
2. Each framing element supporting the EDCS should not exceed its capacity
3. The total energy dissipation in the EDCS should be maximized.

Criteria 1 would normally set the lower bound for the slip load. For example, assuming a basic wind speed of 136 km/hr (85 mph) for the west coast area and Exposure B, the maximum equivalent quasi-static design wind load (at the roof level) based on IBC2006 [ICC, 2006] was estimated to be 111 kN (25 kips). If this load was distributed equally to the EDCS, the wind load
exerted on each EDCS should not be more than 9 kN (2 kips). Therefore the minimum slip load
of the EDCS should not be less than this value. Criteria 2, on the other hand, would place a cap
on the design slip load.

The nonlinear time history design results revealed that for slip load between 22 kN (5
kips) to 89 kN (20 kips), the member sizes used for the BMF (Tables 8.8 and 8.9) were generally
adequate. As the slip load reduced to 4.5 kN (1 kips) or increased to 133 kN (30 kips), a number
of beams and columns had exceeded their design capacities and the size of these members had to
be increased to satisfy strength requirement. For a new building, such a change should be easily
accommodated prior to actual construction. However, if the MFEDCS represents a retrofit
solution to an existing building, a change in the as-built framing members would not be a simple
task and could be very costly. In this case, it may be necessary to limit the slip load from 22 kN
(5 kips) to 89.0 kN (20 kips).

Figure 8.18 shows the influence of slip load on the total hysteretic energy dissipation by
the EDCS. It appeared that maximum energy dissipation could be achieved with a slip load
between 89 kN (20 kips) to 133 kN (30 kips). The selection of the slip load could also be based
on the allowable drift imposed by the building code. Here, the MFEDCS was designed as Seismic
Use Group III with a more stringent drift limit of 1% (rather than 2%). In this case, as shown in
Figure 8.19, the slip load must be greater than 22.2 kN (5 kips) to achieve this level of drift.

Figure 8.18: Variation of Energy Dissipation with Slip Load.
The natural periods for the first five modes are summarized in Table 8.11. The stiffness ratios of the MFEDCS relative to the bare frame were computed in the same manner as the 9-story moment frame of Aiken and Kelly [1990] (Section 7.4.4.1).

**Table 8.11: Natural Periods and Relative Stiffnesses of MFEDCS.**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
<th>Cumulative mass participation factor</th>
<th>Relative stiffness ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.25</td>
<td>3.97</td>
<td>78.9</td>
<td>2.45</td>
</tr>
<tr>
<td>2</td>
<td>0.69</td>
<td>1.45</td>
<td>90.6</td>
<td>2.47</td>
</tr>
<tr>
<td>3</td>
<td>1.16</td>
<td>0.86</td>
<td>94.6</td>
<td>2.43</td>
</tr>
</tbody>
</table>

Figure 8.19: Variation of Maximum Interstory Drift with Slip Load.

### 8.6.2.2 Natural Periods

The natural periods for the first five modes are summarized in Table 8.11. The stiffness ratios of the MFEDCS relative to the bare frame were computed in the same manner as the 9-story moment frame of Aiken and Kelly [1990] (Section 7.4.4.1).
8.7 SUMMARY

A 20-story, 13-bay moment frame was designed as an ordinary moment frame for Seismic Use Group I and under SDC C, in accordance to conventional seismic-resistant design procedure. Linear static and elastic time history methods, with a response modification factor of 3.5, were used to design the ordinary moment frame; the approximate ELP was performed to establish the code-specified minimum base shear. The EDCS was incorporated into the ordinary moment frame design to achieve a higher seismic performance level (i.e., Seismic Use Group III). Because the ordinary moment frame with EDCS was analyzed using the nonlinear time history procedure that directly accounts for the nonlinear hysteretic behavior of the structure’s components (i.e., EDCS), no response modification factor was applied to the design earthquake ground motions.

8.8 CONCLUSION

Under the same design earthquake, the building with the ordinary moment frame alone would have suffered some form of structural damage; considerable repairs would be needed to bring the building back to its operational state. On the other hand, the ordinary moment frame with EDCS was designed to remain elastic throughout its response to the design earthquake motion and hence is not expected to experience any permanent structural damage. Damage to the EDCS, typically in the form of permanent slip, would be easily repaired by slacking the bolts in the friction damper of the EDCS and, where necessary, realigning the column lines by jacking [Butterworth, 1999]. Furthermore, the modular EDCS unit could be easily removed and replaced in a fashion similar to normal architectural cladding panels. It should be realized that the design approach adopted for the BMEDCS was rather conservative because moment frame was not required to supply any inelastic action. Goodno et al. [1998] has attempted to combine energy dissipation cladding connections with inelastic actions of special moment frame but acknowledged that this approach is risky and would warrant further extensive study. Furthermore, the OMF would still have considerable reserve strength and redundancy that would provide a backup system to the EDCS in the event of a higher than expected earthquake. The exact amount of reserve strength (and redundancy) available would require further study.
CHAPTER 9

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

9.1 SUMMARY

Modern earthquake-resistant design aims to isolate architectural precast concrete panels from the structural system so as to minimize the interaction with the supporting structure and hence minimizing damage. The present study seeks to maximize the cladding-structure interaction. But unlike existing solutions that focus mainly on energy dissipation rather than providing lateral load resistance, the objective of the present study is to develop an energy dissipating cladding system (EDCS) that is capable of functioning both as a structural brace, as well as a source of energy dissipation. The EDCS is designed to provide added stiffness and damping to buildings with steel moment resisting frames with the goal of favorably modifying the building response (esp. interstory drift) to earthquake-induced forces. Because many modern building facades typically have continuous and large openings on top of the precast cladding panels at each floor level for window system, the present study focuses on spandrel type precast concrete cladding panels. This is a departure from past research by others that are all concerned with floor-to-floor high panels.

The research work was divided broadly into four main phases: literature study; preliminary study; component level study; and building level study. The preliminary design of the EDCS was based on existing guidelines and available research on precast concrete cladding and supplemental energy dissipation devices. In the component level study, the preliminary design was validated and further refined based on the results of nonlinear finite element analysis. The EDCS unit developed in the present study comprises of four major components: bearing supports, precast concrete panel, concrete anchorage system and a damper unit. Instead of incorporating steel bracing into the precast concrete panel, the present study showed that with careful connection details and proper steel reinforcement layout, normal heavy architectural cladding panel can function as a structural brace, transferring high in-plane forces between its connection points without deterioration in stiffness or strength. The friction damper serves two important functions: a source of energy dissipation and a “fuse” that effectively controls the amount of in-
plane force that would be transferred into the concrete panel. Headed stud anchor (HCA) was the preferred concrete anchorage system for the EDCS because it can effectively distribute in-plane forces from the connection hardware to the concrete mass. The stiffness and strength characteristics of the EDCS were established from a series of nonlinear finite element analyses. Using elastic beam theory, simple expressions were derived to approximate the lateral stiffness of the EDCS. In addition, a simple mathematical model of the EDCS was developed, and together with the estimated stiffness coefficients, is intended to facilitate the preliminary design of buildings incorporating EDCS. The performance of the EDCS was evaluated on two representative buildings, a one-quarter scaled, 9-story, 3-bays moment frame and a 20-story, 13-bays contemporary building.

9.2 EDCS ATTRIBUTES

The concept of the EDCS is a departure from conventional earthquake-resistant design. Through a series of analytical studies, the ability of EDCS to improve the earthquake resistance of the buildings has been shown. The key attributes of the EDCS have been discussed throughout the thesis of which some are summarized as follows:

1. Since the EDCS is part of the building envelope, the EDCS is expected to increase the torsional rigidity of a building. This would be beneficial to buildings with plan irregularity. Another example is the case of buildings on main streets where the front facade usually have spandrels and windows, while the side and back faces have solid walls. Furthermore, the EDCS provides a means of better distributing the lateral loads on the building over a larger number of perimeter framing elements, thereby leading to smaller framing forces. By virtue of its numbers, the EDCS can be considered as inherently redundant.

2. The EDCS is designed to suffer no or minimal damage after a design earthquake event. Conventional earthquake resistant structures rely on inelastic action of the LFRS that result in permanent damage to the structural system. As an example, a moment frame originally designed for Seismic Use Group I could be upgraded to III by incorporating EDCS.
3. The developed EDCS can be used to supplement a moment frame to reduce drift to satisfy the code drift requirement. If the frame is already designed to satisfy the allowable drift, it would not require EDCS. On the other hand, if the building design also includes architectural precast cladding panel, it is possible to design the frame more economically (i.e., with less stringent detailing requirements), which may not satisfy the code drift requirement by itself. But when EDCS is added and considered in the design, the drift will reduce to the code acceptable level. As another potential use, the EDCS can also be used as a retrofit solution for existing frames that may not satisfy code drift requirements or is subjected to higher than originally design earthquake, but would become acceptable if the EDCS is used. In any case, these are only potential options. Extensive experimental study is needed to develop possible commercial application.

4. Existing friction-based energy dissipating bracing systems are typically installed in one or two bays at each floor; the slip load level usually varies over the building height. The EDCS offers much more flexibility in specifying different slip load distribution across the building bays. Different slip load distribution could be used for the EDCS installed in different bays. Different groups of the EDCS would activate at different design earthquake levels.

5. Instead of friction-based devices, other forms of energy dissipating devices (e.g., viscoelastic damper) could be incorporated into EDCS for both seismic and wind application. The EDCS concept can also be applied to floor-to-floor type cladding system without much modification.

6. Since the energy dissipating device is placed on the exterior of the EDCS, maintenance and replacement would not pose any major difficulties.

7. When installed on the building facade, the EDCS would look exactly the same as conventional architectural cladding system. The EDCS would be an attractive alternative to conventional full-height bracing systems that tend to obstruct views from windows and openings.

8. The EDCS offers an additional option to designer and architects. It combines seismic resistant design with building facade design. Although the EDCS has its advantages as highlighted above, it may increase the building cost.
9.3 CONCLUSIONS

The present study has developed an innovative design concept that integrates both architectural and structural performance into the EDCS. The EDCS is intended to give the designer an alternative method of improving the earthquake resistance of a building. In the present research, extensive analytical studies were performed to validate the design and performance of the EDCS. Based on the assumptions made in design, modeling, and analyses presented in this study, the following conclusions can be drawn from the study:

1. From the literature reviews it is concluded that extensive cladding damage, which constitutes a significant part of the cost of nonstructural damage, has been frequently observed in past earthquakes [Arnold et al., 1987]. Modern earthquake-resistant design attempts to reduce such earthquake-induced damage through connection details aim at isolating the cladding panels from the supporting structure, thereby minimizing the damaging interaction effects. However, the literature study in Chapter 2 has shown that it is not possible to completely eliminate the cladding-structure interaction.

2. According to the literature research, it is not always necessary to minimize the cladding-structure interaction to prevent damage to the precast concrete cladding. Past research has married the cladding-structure interaction with the energy dissipation concept. However, these studies have been focused primarily on energy dissipation and are limited to floor-to-floor high cladding panels. From the literature reviews and evidence of significant research support on floor-to-floor high energy dissipating panels, it is concluded that the industry would be interested in developments in spandrel type energy dissipating cladding panels.

3. The EDCS developed in the present study introduces an alternative system that can provide significant lateral stiffness and energy dissipation to the building, with the primary objective of reducing the building response to earthquake ground motions. The EDCS was developed as a spandrel type cladding panel since modern building facades typically require continuous strip openings for window installations or ventilation requirements.

4. Based on the result of the study in Chapter 3, despite the lower lateral stiffness of the individual partial height spandrel bracing (as compared to full-height cross-bracing), it is concluded that the perimeter spandrel bracing system can provide a lateral stiffness comparable to conventional cross-bracing system by virtue of its numbers. Because the
spandrel bracings are distributed over the building perimeter, they are effective in distributing the building axial forces and reducing torque.

5. Based on the result of the study in Chapter 3, the friction damper was preferred over other supplemental energy dissipating devices (e.g., yielding, viscous, viscoelastic, etc.) based on relatively low cost, ease of design, production and installation. Furthermore, the friction damper is displacement-based and it will achieve the required slip load at very low displacement. On the other hand, viscous dampers are typically proprietary products that come with a cost premium. The viscous dampers, being velocity-dependent, require considerable stroke (not suitable for the partial height EDCS) to develop the full damping action. Yielding elements are expected to be replaced after the design earthquake event. Viscoelastic materials are more suitable for resisting low amplitude and high frequency wind loading [Hanson and Soong, 2001].

6. Based on the results from the feasibility study (Chapter 3), it was found that there was difficulty in quantifying the amount of energy dissipation provided by the nonlinear energy dissipating devices using equivalent modal damping, though better correlation may still be possible if the modal damping was specified for each mode of vibration.

7. As discussed in Chapter 4, Concept B, which relies on the cladding panel to transfer the in-plane forces, was selected because it fully exploits the cladding-structure interaction. For Concept A, which has an embedded diagonal brace and friction damper, the concrete panel is effectively isolated from the bracing subsystem and a separate bracing subsystem is required. By eliminating the need for separate bracing subsystem in Concept B, reduction in material cost and simplification of the fabrication process would be achieved.

8. Because the design of the EDCS is very similar to that of conventional architectural precast concrete cladding system, the preliminary design of the EDCS components could be based on existing design guidelines on the conventional cladding system. The FEA was employed to check and further refine the design.

9. Existing literature reviewed in Chapter 5 suggested that a commercially available finite element analysis package, ANSYS, was capable of modeling the complex nonlinear behavior of structural concrete through the availability of a concrete element (i.e., SOLID65). From the analysis of a deep beam, the SOLID65 element with its explicit crack model appeared to be suitable. Solution non-convergence was the greatest challenge in the use of SOLID65 element, and it was due to the implementation of the
crack model. Solution convergence requires careful specification of loading criteria (e.g., load step) and solution criteria (e.g., convergence, number of iteration per load step), in addition to appropriate modeling techniques.

10. The generally good correlations between the FEA results and experimental results for the deep beam suggests that the concrete properties estimated from existing design guidelines are appropriate for the FE model without any modification.

11. Without an explicit concrete crack model, even with a nonlinear uniaxial stress-strain relationship, the analytical models for the deep beam tend to exhibit a significantly stiffer response than the test beam. The specification of a descending segment in the modified Hognestad stress-strain relationship for concrete softening appeared to give slightly better correlations with the test result, especially near the failure load. But due to non-convergence problem, the simple Hognestad stress-strain relationship was used for the FEA of the EDCS. The crushing model in SOLID65 element is expected to improve the correlations, especially at high load levels. But persistent non-convergence issue would deter its use. In any case, for the FEA of precast concrete panels, the focus is mainly on the elastic response region and significant localized crushing at high loading was not needed to be considered for the present study.

12. The simple finite element modeling strategy adopted for the headed stud anchors were found to be adequate. The FEA results gave unique crack patterns in accordance to the different loading conditions. Although the FEA was not able to predict the failure loads, the numerical shear capacities appeared to be consistent with values provided by existing design codes.

13. FEA results showed that diagonal bars or brace placed between the support and the load application would only be effective if the bars were debonded from the concrete. With the bars bonded to the concrete mass, the bar force would be completely transferred to the concrete mass along the developed length of bar due to the length of the spandrel panel. The nonlinear FEA showed that precast concrete panels can be designed to remain essentially elastic up to the slip load of the friction damper if adequate reinforcement and proper detailing of the connections are provided.

14. The results from the FEA showed that embedding the HSS into the concrete resulted in high stress concentration and early reduction in lateral stiffness. The HCA would provide better distribution of anchoring force than embedded HSS section, significantly increased the elastic-limit load by as much as 76%.
15. Changing the support condition of the cantilevered bearing supports from pinned to fixed led to as much as five times increase in the lateral stiffness of the panel.

16. The analysis results in Chapter 5 showed that the additional center layer of reinforcing bars was not necessary because the FEA results indicate no significant reduction in the in-plane stiffness and strength as a result of omitting the center layer. Therefore, it is concluded that the conventional two curtains of reinforcement, one on each face, is adequate.

17. The principal stress trajectory plots and contours presented in Chapter 6 showed that a significant portion of the spandrel panel was under uniform stress distribution. This suggests that an undisturbed or B-region is present. On the other hand, the trajectories in the vicinity (within a distance of eight times the panel thickness) of the connections were highly directional, indicating a disturbed region or D-region with nonlinear stress distribution. It is concluded that to accurately describe the stress and stiffness characteristics of the panel, combination of elastic beam theory and stress field based theory (e.g., strut-and-tie method) should be used. However, the stress field based method typically requires substantial engineering judgment [Reineck (Ed.), 2002].

18. Closed-form expressions describing the linear elastic stress distribution in the panel was derived from static equilibrium relationship. Within the range of applicability, these expressions were found to correlate well with the FEA results in the region between the lateral supports. It is therefore concluded that the spandrel panel could be approximated by simple elastic beam theory alone to obtain the in-plane load-deformation characteristics.

19. The in-plane response of the concrete panel could be approximated by three distinct modes: axial, shear and flexure. Closed-form expressions quantifying the contribution of each mode were developed based on the FEA results. From the study it can be concluded that, within the range of applicability, the approximate elastic beam solutions were adequate for the developed EDCS and would be useful for preliminary design purposes.

20. By taking into account the flexibility of the HCA, the FEA results revealed that the overall lateral stiffness of the panel could be significantly reduced by more than 80%. Therefore the stiffness, in addition to the strength, of the concrete anchorage system must be considered in the design for the EDCS.

21. The FEA results revealed that the precast concrete panel with HCA can contribute as much as 58% to the overall lateral flexibility of the EDCS unit. It is therefore concluded
that the (common) assumption of an infinitely stiff precast concrete panel may not be appropriate.

22. From the results of the nonlinear time history analyses performed on 9-story one-quarter scale test structure in Chapter 6 and 20-story full-scale moment frame building in Chapter 7, it is concluded that the EDCS is able to significantly reduce interstory drift and member forces through the combined action of added stiffness and energy dissipation. This was achieved through careful selection of the slip load level and distribution. Therefore, the selection of slip load is a key design parameter for EDCS.

23. The loading condition and stresses developed in the framing members of building with EDCS are expected to be different from conventional moment frame members as a result of the interaction with the EDCS. In particular, columns on which the EDCS is attached are subjected to concentrated load (i.e., slip load) along its height (at about one-third height). Based on the result of the study presented in Chapters 7 and 8, this should not pose an issue because the maximum slip load is typically controlled by the capacities of the supporting framing elements.

24. The study on the example 20-story building demonstrated that by incorporating the EDCS into a moment resisting frame, it is possible to achieve a reduction in building response comparable to that of a code-specified ordinary moment frame (i.e., response modification factor of 3.5). However, the EDCS design in the example shown does not demand any form of ductility, redundancy or ductility from the structural frame itself.

25. The design approach adopted for the building incorporating the EDCS was conservative in the sense that the moment frame remains essentially elastic throughout the design earthquake event. The ordinary moment frame, if designed and detailed in accordance to the conventional seismic code, could reduce (i.e., response modification factor of 3.5) the design earthquake even without the EDCS. Therefore, it is concluded that the moment frame with EDCS could survive a higher than expected earthquake. Furthermore, the EDCS could be retrofitted into existing ordinary moment frame that was not originally designed for higher levels of ground motions.

26. A less conservative approach could be used whereby both the framing members and the EDCS are specifically designed to contribute to the inelastic demand leading to significantly higher reduction in building response because of reduced force demand as a result of inelasticity and ductility. Although this approach could lead to considerable reduction in the size of the structural members and cost saving, it has its challenges. It
would be necessary to model accurately both the inelastic behavior of both framing elements and EDCS. However, the inelastic behavior (e.g., location and numbers of plastic hinges) of the ductile moment frame may have changed as a result of the attached EDCS. Furthermore, it may not be easy to separate the contribution of each system in reducing the building response. More importantly, the failure of any one of the systems may lead to possible collapse. It is therefore concluded that for a more conservative design, the moment frame should be designed to remain elastic with all inelastic demand provided by the EDCS (i.e., as a DCS).

9.4 ARCHITECTURAL AND CONSTRUCTIONAL ASPECTS ON EDCS

The EDCS is suggested to have a dual purpose of being part of the building envelope and as part of the LFRS. Besides the structural design viewpoint, considerations pertaining to the architectural and constructional aspects are equally important and some are discussed below.

- **Protection Against Environmental Effects** – Like typical cladding connections, the friction damper in the EDCS would be located in the air space between the back face of the cladding panel and the insulation system for the building envelope. Although modern wall system design incorporates some sort of ventilation principles to dry the moisture in the air space, high humidity is likely as a result of air stagnation. If the EDCS was installed in an open building (e.g., open parking garage), the damper would be further subjected to rain or snow. In any case, it is necessary to protect the damper unit through special painting and application of sealant. The exterior of the steel components should be primed and painted in accordance to the paint manufacturer’s specification. The interior sliding surfaces should be sealed from the exterior by sealing the holes and gaps (along the edges) of the damper assembly with appropriate sealant (e.g., latex caulk, elastometric sealant). Weathering grade steel, typically used in bridge engineering, is not recommended since a layer of (protective) rust layer will form on the steel surfaces; galvanizing may be more appropriate. For open structure, some form of casing may be required to hide the damper unit and prevent possible vandalism. The EDCS should be included as an item in the facility maintenance plan and inspected regularly.
**Externally Applied Insulation Materials** – Fire and thermal insulation, when required by the architecture design, would be applied in the same manner as for conventional architectural cladding panel. The insulating material adhering to the exterior of the EDCS should not affect the performance of the EDCS, in particular, the operation of the friction damper. Even if the entire damper unit was completely encased by the insulation, the resistance provided by the insulation is insignificant as compared to the design slip load and the friction damper is expected to slip at the design load. Simple tests can be carried out to verify this. It should be pointed out that the developed EDCS does not currently apply to insulated sandwich panels.

**Creep Effects and Relaxation of Bolts** – The friction force in the friction damper is directly proportional to the normal force exerted by the pre-tensioned bolts. Over time, creep or relaxation of bolts may reduce the friction force. According to Pall, et al. [1993], creep in high-strength bolts is only about 7 to 8% over a period of 80 years. To offset the losses due to creep, it is customary to increase the design slip load by 3 to 5% for the actual friction damper [Pall, 1993]. Furthermore, past research [Filiatrault and Cherry, 1987] have shown that a change in the slip load up to 20% of the optimum value does not have a significant effect on the building response. This was also evident in the 20-story building study where a 20% change in the optimum slip load (20 kips) would not have a noticeable effect on the peak interstory drift.

**Use of Other Materials** – In principle, the heavy concrete panel in the EDCS can be replaced by other materials (e.g., light weight concrete panel, metal panel) if it can be demonstrated that the system with this material can transfer the required slip load in an elastic manner without deterioration in strength.

### 9.5 RECOMMENDATIONS FOR FUTURE RESEARCH

The present study has focused primarily on the concept development of a new system in seismic resistant design. Extensive follow-up studies would be required to further expand the idea and to experimentally validate the performance of the EDCS. Based on the findings from the
present research, the following improvement and extended application on the EDCS are suggested:

**Component Level**

1. The material properties assumed in the design of EDCS components used in the finite element modeling were based on the nominal design values recommended by prevailing guidelines and specifications. Experiments for some EDCS components (e.g., friction pad, HCA) should be conducted to obtain more accurate material properties.

2. Finite element analyses were performed to investigate the stiffness and strength characteristics of the EDCS, in particular, crack patterns of the concrete panel. All analytical results should be validated through full-scale testing of a prototype model under both static and cyclic loading.

3. The finite element models of the reinforced concrete panel assume a perfect bond between the steel reinforcement and the concrete medium. Possible improvement in the FEA results could be achieved by considering the bond-slip effect between these two components in the reinforced concrete model.

4. The EDCS would be subjected to different environmental effects from both interior and exterior of the building. Consequently, the sensitive EDCS component (i.e., friction damper) can be affected by the environmental factors (e.g., humidity, temperature, dust). Therefore, a study of the environmental effect on the EDCS should be carried out to ensure that the unit would function properly under any environmental conditions.

5. The precast concrete cladding panel type considered in this study is the spandrel type without any openings. The same concept could be applied to the spandrel or floor-to-floor high panel with or without openings. For the panels with opening, the strut-and-tie method along with the finite element technique could be used to define the load path within the panel and to develop the reinforcement requirement in the concrete panels.

**Building Level**

1. Further optimization can be performed on the various parameters affecting the design of the EDCS. In particular, a mathematical model for optimizing the slip load level and distribution could be developed.
2. To validate the performance of the EDCS and the assumptions associated with its analysis and design, experimental testing should be conducted on a multi-story, multiple bays building either at full-scale or reduced scale. The test structure should have at least three bays and should be at least three stories high.

3. The LFRS investigated in the present study was limited to an ordinary moment resisting frame (OMF). As stated earlier, the EDCS can also be incorporated into special ductile frames such as intermediate moment resisting frames (IMF) or special moment resisting frames (SMF). Extensive studies must be performed to validate the performance of both the EDCS and ductile frame for this dual system.

4. The application of EDCS can be extended to resist wind-induced loading. The main differences between the response from wind load and that produced by earthquake load are smaller deformation and more loading cycles [Hanson and Soong, 2001]. Since wind-induced loading would produce a wider range of movement, it may not be practical to establish an optimum slip load. As a result, the friction damper may not be the best candidate in such applications and the use of viscoelastic damper may be more suitable. To prevent the failure of viscoelastic damper when subjected to the higher than expected load, the viscoelastic damper could be added in series with a friction damper that serves as a fuse to limit the force into the viscoelastic damper [Kelly, 2001]. Again, such applications would warrant further study.

5. The design of the EDCS was kept as simple as possible to minimize production cost. A single EDCS unit is expected to cost more to produce and install as compared to a typical precast concrete cladding panel. But the EDCS may still be an economical solution when compared with other LFRS’s to achieve the same performance criteria (e.g., drift control). An extensive life cycle analysis should be performed on the EDCS that covers different phases of design, manufacture, installation, maintenance and replacement. The life cycle analysis should also be factored in the level of maintenance required between earthquake events (i.e., during periods of inactivity). To facilitate regular inspection and maintenance of the friction damper, access openings in the building interior wall system should be provided. This would require further study.
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APPENDIX A

EDCS DESIGN EXAMPLE

The following spreadsheets contains the sample calculations of EDCS.
# Design Analysis of Energy Dissipation Cladding System

**Title:** Design Analysis of Energy Dissipation Cladding System  
**Designed By:** Bell  
**Checked By:**  

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## 1. Introduction

These calculation sheets outline the preliminary design analysis of Design of Energy Dissipation Cladding System (EDCS) based on the provisions outlined in the 2003 International Building Code (IBC). ASCE7-02 is also referenced in the 2003 IBC.

## 2. Design Parameters

### a. Seismic

- **Location:** SDC C Site
- **Average roof height of structure relative to base:** $h = 1\ ft$
- **Height of point of attachment of component:** $z = 1\ ft$ (Assumed conservatively to equal average roof height)
- **Seismic Use Group:** I
- **Component Importance Factor:** $I_P = 1.50$
- **5% damped, 0.2s spectral acceleration:** $S_a = 0.75\ g$
- **Site Class:** D (Assumed for unknown soil conditions)
- **Site Coefficient for $S_{D1}$:** $F_D = 1.00$
- **Maximum Considered 0.2s spectral acceleration:** $S_{MS} = F_DS_a = 0.75\ g$
- **Design 0.2s spectral acceleration:** $S_{DS} = 2/3S_{MS} = 0.50\ g$
- **Seismic Design Category:** C

### b. Cladding Panel Dimensions

- **Length:** $L = 24\ ft$
- **Height:** $H = 7\ ft$
- **Thickness:** $t = 8\ in.$
  - **Horizontal edge distance of top connections:** $b_t = 1.00\ ft$ (6" minimum edge distance for PSA slotted insert)
  - **Horizontal edge distance of bottom connections:** $b_b = 3.17\ ft$
  - **Vertical edge distance of top connections:** $a_t = 1.17\ ft$ (10" minimum distance for PSA slotted insert)
  - **Vertical edge distance of bottom connections:** $a_b = 2.17\ ft$
  - **Clear distance between panel and support centreline:** $c = 10\ in.$

### c. Material Properties

- **Concrete density:** $w_c = 150\ lb/ft^3$ (Normal weight concrete)
- **Concrete 28-day compressive strength:** $f'_c = 5000\ psi$
- **Yield strength**
  - **Structural steel:** $f_y = 50\ ksi$
  - **Reinforcing steel bar:** $f_y = 60\ ksi$
- **Friction Damper Slip Force:** $F_s = 40.0\ kips$
3. Calculations

a. Geometry
- Horiz. distance between top connections: $L'_t = 22.00$ ft
- Horiz. distance between bottom connections: $L'_b = 17.7$ ft
- Vert. distance between connections: $H' = 3.7$ ft
- Eccentricity: $e = 14.0$ in.

b. Basic Seismic Forces
- Weight of Panel: $W_p = LHtw_c = 16.8$ kips

Component amplification factor:
- Panel and connector body: $a_{p1} = 1.0$
- Fasteners: $a_{p2} = 1.25$

Component response modification factor:
- Panel and connector body: $R_{p1} = 2.5$
- Fasteners: $R_{p2} = 1.00$
### Project Title

**Design Analysis of Energy Dissipation Cladding System**

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#### i. Panel and Connector body

**Eq. 9.6.1.3-1 Horiz. seismic force**

\[ F_p = \left( \frac{a_p}{R_{p1}} \right) \left( 0.4 S_{D1}(1+ 2z/h) \right) W_p \]

- \( F_p \) = 6.048 kips

**Eq. 9.6.1.3-2 Max. seismic force**

\[ F_{p, \text{max.}} = 0.3 S_{D1} W_p \]

- \( F_{p, \text{max.}} \) = 20.16 kips

**Eq. 9.6.1.3-3 Min. seismic force**

\[ F_{p, \text{min.}} = 1.6 S_{D1} W_p \]

- \( F_{p, \text{min.}} \) = 3.78 kips

\[ \therefore \text{use } F_p = 6.05 \text{ kips} \]

#### ii. Connection Fasteners

**Eq. 9.6.1.3-1 Horiz. seismic force**

\[ F_p = \left( \frac{a_p}{R_{p1}} \right) \left( 0.4 S_{D1}(1+ 2z/h) \right) W_p \]

- \( F_p \) = 18.90 kips

**Eq. 9.6.1.3-2 Max. seismic force**

\[ F_{p, \text{max.}} = 0.3 S_{D1} W_p \]

- \( F_{p, \text{max.}} \) = 20.16 kips

**Eq. 9.6.1.3-3 Min. seismic force**

\[ F_{p, \text{min.}} = 1.6 S_{D1} W_p \]

- \( F_{p, \text{min.}} \) = 3.78 kips

\[ \therefore \text{use } F_p = 18.90 \text{ kips} \]

**Vertical seismic force**

\[ F_v = 0.2 S_{D1} W_p \]

- \( F_v \) = 1.68 kips

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### Connector Body

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Vertical</th>
<th>Vertical</th>
<th>Horizontal</th>
<th>Horizontal</th>
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<td>III</td>
<td>IV</td>
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#### Location

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<th>Fz</th>
<th>Fx</th>
<th>Fy</th>
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<th>Fy</th>
<th>Fz</th>
<th>Fx</th>
<th>Fy</th>
<th>Fz</th>
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<td>0.27</td>
<td>0.18</td>
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<td>5.16</td>
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<td>B</td>
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<td>Tie-back</td>
<td>2.67</td>
<td>0.27</td>
<td>0.18</td>
<td>1.512</td>
<td>5.16</td>
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</tr>
<tr>
<td>Tie-back</td>
<td>2.67</td>
<td>0.27</td>
<td>0.18</td>
<td>1.512</td>
<td>5.16</td>
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</tbody>
</table>
## Project Title

**Title:** Design Analysis of Energy Dissipation Cladding System

**Design By:** Bell

**Checked By:**

---

### REF. CALCULATION

<table>
<thead>
<tr>
<th>Location</th>
<th>Load Case</th>
<th>Type</th>
<th>Direction</th>
<th>Load factor</th>
<th>Fx</th>
<th>Fy</th>
<th>Fz</th>
<th>Slip Force</th>
<th>Factored Combined</th>
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<tr>
<td>A</td>
<td>I</td>
<td>In-plane</td>
<td>Vertical</td>
<td>1.2</td>
<td>9.45</td>
<td>8.40</td>
<td>2.67</td>
<td>8.30</td>
<td>19.22</td>
</tr>
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<td>II</td>
<td>In-plane</td>
<td>Vertical</td>
<td>1</td>
<td>40.0</td>
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<td>0.27</td>
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<td>III</td>
<td>Horizontal</td>
<td>In-plane</td>
<td>1</td>
<td>49.45</td>
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<td>0.27</td>
<td>8.76</td>
<td>8.76</td>
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<td>Horizontal</td>
<td>Horizontal</td>
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<td>8.76</td>
<td>0.56</td>
<td>0.27</td>
<td>8.76</td>
<td>8.76</td>
</tr>
</tbody>
</table>

---

**Fastener**

**Date Designed:**

**Date Checked:**
Note: All connectors and fastener shall be designed for load reversal.
LOAD CASE I: (Unit load in direction of panel self-weight, \(W_p\) or in-plane vertical seismic force, \(F_v\))

LOAD CASE II: (Unit load in direction of in-plane horizontal seismic force, \(F_p\))
LOAD CASE III: (Unit load in direction of out-of-plane, $F_p$)

LOAD CASE IV: (Unit load in direction of slip force of friction damper, $F_s$)
1. Introduction


2. Design Parameters

The following forces are obtained from a static equilibrium analysis of a typical spandrel precast concrete cladding.

<table>
<thead>
<tr>
<th>Connector Body Forces (Ductile)</th>
<th>Fastener Forces (Non-ductile)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{Ax}$ 43.02 kips</td>
<td>$F_{Ax}$ 49.45 kips</td>
</tr>
<tr>
<td>$F_{Ay}$ 19.22 kips</td>
<td>$F_{Ay}$ 19.22 kips</td>
</tr>
<tr>
<td>$F_{Az}$ 5.16 kips</td>
<td>$F_{Az}$ 8.76 kips</td>
</tr>
<tr>
<td>$F_{Bx}$ - kips</td>
<td>$F_{Bx}$ - kips</td>
</tr>
<tr>
<td>$F_{By}$ 19.22 kips</td>
<td>$F_{By}$ 19.22 kips</td>
</tr>
<tr>
<td>$F_{Bz}$ 5.16 kips</td>
<td>$F_{Bz}$ 8.76 kips</td>
</tr>
<tr>
<td>$F_{Cx}$ - kips</td>
<td>$F_{Cx}$ - kips</td>
</tr>
<tr>
<td>$F_{Cy}$ - kips</td>
<td>$F_{Cy}$ - kips</td>
</tr>
<tr>
<td>$F_{Cz}$ 5.16 kips</td>
<td>$F_{Cz}$ 8.76 kips</td>
</tr>
<tr>
<td>$F_{Dx}$ 43.02 kips</td>
<td>$F_{Dx}$ 49.45 kips</td>
</tr>
<tr>
<td>$F_{Dy}$ - kips</td>
<td>$F_{Dy}$ - kips</td>
</tr>
<tr>
<td>$F_{Dz}$ 5.16 kips</td>
<td>$F_{Dz}$ 8.76 kips</td>
</tr>
</tbody>
</table>

Legend:
- A – Pinned Bearing connection
- B – Roller Bearing connection
- C – Flexible Tie-back connection
- D – Rigid Tie-back connection

(All connections are designed for load reversal)

Notes on Steel Properties:

<table>
<thead>
<tr>
<th>Description</th>
<th>ASTM</th>
<th>$F_y$ (ksi)</th>
<th>$F_u$ (ksi)</th>
<th>E (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. W-shapes:</td>
<td>A992</td>
<td>50</td>
<td>65</td>
<td>29,000</td>
</tr>
<tr>
<td>2. Angles:</td>
<td>A36</td>
<td>36</td>
<td>58</td>
<td>29,000</td>
</tr>
<tr>
<td>3. Rectangular (&amp; square HSS):</td>
<td>A500 grade B</td>
<td>46</td>
<td>58</td>
<td>29,000</td>
</tr>
<tr>
<td>4. Structural plates and bars (&lt; 8&quot; thick):</td>
<td>A36</td>
<td>36</td>
<td>58</td>
<td>29,000</td>
</tr>
<tr>
<td>5. Threaded rod:</td>
<td>A36</td>
<td>36</td>
<td>58</td>
<td>29,000</td>
</tr>
<tr>
<td>6. Steel reinforcing bars</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3. Design of Tie-Back Connection - Connection C

3.1. Thread Rod Design

Type: Fastener (Non-ductile)

Load Characteristics (Based on page 1)

- \( F_{Cx} \): kips (Slotted hole permits movement in the x-direction)
- \( F_{Cy} \): kips (Slotted hole permits movement in the y-direction)
- \( F_{Cz} \): 8.76 kips

**Internal Forces**

- Axial: T 8.76 kips
- C 8.76 kips

**Section properties**

- Diameter: \( d = 1.000 \text{ in} \)
- Area: \( A_b = \frac{\pi d^2}{4} = 0.785 \text{ in}^2 \)
- Moment of Inertia: \( I_x = I_y = \frac{\pi d^4}{64} = 0.049 \text{ in}^4 \)
- Radius of gyration: \( r = \left(\frac{I_x/A}{4}\right)^{0.5} = 0.250 \text{ in} \)

**Tensile capacity check**

\[ \phi = 0.75 \] (for rupture)

\[ \phi_T \phi_{Tn} = \phi F_{u} (0.75 A_b) = 25.62 \text{ Kips} > 8.76 \text{ (OK)} \]

(Note: 0.75 factor applied to \( A \) to get effective area)

**Compressive capacity check**

Effective length factor: \( K = 2 \) (Assume cantilever)

Column width: \( w_{col} = 12 \text{ in} \) (Assume)

Unsupported length: \( L = 7 \text{ in} \)

Slenderness ratio:

\[ \lambda_c = \frac{(KL/n)(F_{u}/E)^{0.5}}{0.628} = 56.0 \]

Critical buckling stress:

\[ F_{cr} = 30.52 \text{ ksi} \]

\[ \phi_c = 0.85 \] (Threaded rod)

\[ \phi_c P_n = \phi_c F_{cn} A = 20.38 \text{ Kips} > 8.76 \text{ (OK)} \]
3. Design of Tie-Back Connection - Connection C (Cont'd)

3.2. Slotted Angle Design
Type: Connector body (Ductile)

Load Characteristics (Based on page 1)
- $F_{Cx}$: 5.16 kips
- $F_{Cy}$: 5.16 kips
- $F_{Cz}$: 5.16 kips

Internal Forces
- Axial: $T$
- Shear: $V$
- Edge distance: $e_1$

Moment
- $M_u = F_{Cz}(L_1 - e_1) = 30.99$ in-k
  (Min. edge distance per AISC LRFD Table J3.4)

Angle Section properties
- Section: L8 x 6 x 7/8" x 6" long
- Leg size: $L_1 = 8$ in, $L_2 = 4$ in
- Thickness: $t = 0.875$ in
- Width: $W = 6$ in
- Slotted hole length: $L_h = 4.00$ in
- Plastic modulus: $Z = Wt^2/4 = 1.403$ in$^3$
- Elastic modulus: $S = Wt^2/6 = 0.935$ in$^3$

(i) Shear capacity
- $\phi_V = 0.75$
- Net area, $A_{nv} = \frac{1}{2} (W - L_h) = 1.75$ in
- $\phi_Tn = \frac{1}{2} F_{Cy}(0.6A_{nv}) = 45.68$ Kips
  > 5.16 Kips
  (OK)

(ii) Flexural capacity (At critical point A)
- $\phi_f = 0.9$
- $F_{Zf} = 45.46$ in-k
- $1.5F_{Zf} = 68.19$ in-k
- $\phi_fM_n = 40.92$ in-k
- $\phi_fM_n = 30.99$ in-k
  (OK)
3. Design of Tie-Back Connection - Connection C (Cont’d)

3.3. Welded Group Design

Type: Fastener (Non-ductile)

Load Characteristics (Based on page 1)

<table>
<thead>
<tr>
<th>Load</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{Cx}$</td>
<td>- kips</td>
</tr>
<tr>
<td>$F_{Cy}$</td>
<td>- kips</td>
</tr>
<tr>
<td>$F_{Cz}$</td>
<td>8.76 kips</td>
</tr>
</tbody>
</table>

Internal Forces

- Shear: $V = 8.76$ kips
- Eccentricity: $e_2 = 6$ in
- Moment: $M = F_{Cz}e_2 = 52.53$ in-k

Welded Group Properties

- Depth: $L_2 = 4$ in
- Width: $W = 6$ in
- Total length: $L_{total} = 14$ in

Distance to c.g.: $x = L_2^2/(W + 2L_2) = 1.143$ in

Section Modulus:
- $S_{top} = (2WL_2 + L_2^2)/3 = 21.33$ in$^3$/in thick of weld
- $S_{bot} = L_2^2(2W + L_2)/3(W + L_2) = 8.53$ in$^3$/in thick of weld

Elastic Weld Stresses

- Max. shear stress: $f_v = V/L_{weld} = 0.63$ k/in
- Max. bending stress: $f_b = M/S_{top} = 2.46$ k/in
- $f_b = M/S_{bot} = 6.16$ k/in

Combined stress: $f_{max} = (f_v^2 + f_b^2)^{0.5} = 6.19$ k/in

Consider the strength of 1/16” weld size E70XX,

- $f_v = 0.75$
- $R_u = f_v(0.6)(70$ ksi$)(0.70)(1/16$ in$)(1$ in$) = 1.378$ k/in per 1/16” weld thickness
3. Design of Tie-Back Connection - Connection C (Cont'd)

3.3. Welded Group Design (Cont'd)

Required weld thickness \( t_{\text{weld}} = \frac{f_{\text{max}}}{f_R} = 5/16 \text{ in} \)

Weld thickness provided = 8/16 in
Plate thickness \( t = 0.875 \text{ in} \)
Min. fillet weld size = 5/16 < 8/16 (OK)
Max. fillet weld size = 13/16 > 8/16 (OK)

3.4 Slotted Insert
Type: Fastener (Non-ductile)

Load Characteristics (Based on page 1)

<table>
<thead>
<tr>
<th>Load Characteristics</th>
<th>Cx</th>
<th>Cy</th>
<th>Cz</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>kips</td>
<td>F</td>
<td>kips</td>
</tr>
</tbody>
</table>

Internal Forces

Tensile force \( T = 8.76 \text{ kips} \)

Minimum edge distance requirement - The minimum edge distance without the use of additional reinforcing steel is 6” and 10” as shown in the illustration. The edge distance can be reduced if extra reinforcement is installed.

Slotted Insert Used: JVI 6035 PSA

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Length</td>
<td>7.25 in</td>
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<tr>
<td>Width</td>
<td>2 in</td>
</tr>
<tr>
<td>Horizontal edge distance</td>
<td>( b = 0.5L_{\text{PSA}} = 8.375 \text{ in} &gt; 6 \text{ in (min.)} ) (OK)</td>
</tr>
<tr>
<td>Vertical edge distance</td>
<td>( a = 14 \text{ in} &gt; 10 \text{ in (min.)} ) (OK)</td>
</tr>
</tbody>
</table>

Select 6035 PSA slotted insert (by JVI)
The ultimate published pull-out capacity = 18.8 kips > 8.76 kips (OK) (with avail adjustment of 4.375”)
The ultimate published shear capacity = 20 kips (Shear requirement is minimum since slotted hole permits movement in the x and y directions)
3. Design of Tie-Back Connection - Connection C (Cont'd)

3.2b. (ALTERNATIVE) Slotted Plate Design

A slotted plate is used in lieu of the slotted angle if the supporting column flange is parallel to the cladding as shown in the figure on the right.

**Type:** Connector body (Ductile)

**Load Characteristics (Based on page 1)**

<table>
<thead>
<tr>
<th>Load Characteristic</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{Cx}$</td>
<td>(Slotted hole permits movement in the x-direction)</td>
<td>kips</td>
</tr>
<tr>
<td>$F_{Cy}$</td>
<td>(Slotted hole permits movement in the y-direction)</td>
<td>kips</td>
</tr>
<tr>
<td>$F_{Cz}$</td>
<td></td>
<td>5.16 kips</td>
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</table>

**Internal Forces**

<table>
<thead>
<tr>
<th>Force</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T$</td>
<td>- kips</td>
<td></td>
</tr>
<tr>
<td>$C$</td>
<td>- kips</td>
<td></td>
</tr>
<tr>
<td>$V$</td>
<td>5.16 kips</td>
<td></td>
</tr>
</tbody>
</table>

**Column width**

$W_{col}$ = 12 in

**Edge distance**

$e_1 = \frac{1}{2} \text{ in} > 1.75 \text{ in (OK)}$

**Eccentricity**

$e_2 = b + 0.5 - 0.5W_{col} = 6.5 \text{ in (Min. edge distance per AISC LRFD Table J3.4)}$

**Design moment**

$M_u = F_{Cz}e_2 = 33.57 \text{ in-k}$

**Plate Section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>1.125 in</td>
</tr>
<tr>
<td>Width</td>
<td>6 in</td>
</tr>
<tr>
<td>Slotted hole length</td>
<td>$W_{h}$ = 4.00 in</td>
</tr>
<tr>
<td>Plastic modulus:</td>
<td>$Z = Wt^2/4 = 1.591 \text{ in}^3$</td>
</tr>
<tr>
<td>Elastic modulus:</td>
<td>$S = Wt^2/6 = 1.061 \text{ in}^3$</td>
</tr>
</tbody>
</table>

**Shear capacity**

$\phi_v = 0.75$

$A_v = (W - L_h) = 2.25 \text{ in}$

$\phi_vT_n = \phi_vF_{u}(0.6A_v) = 58.73 \text{ Kips} > 5.16 \text{ (OK)}$

![Diagram of Slotted Plate Design](image)
3. Design of Tie-Back Connection - Connection C (Cont’d)

3.2b. (ALTERNATIVE) Slotted Plate Design (Cont’d)

(ii) Flexural capacity (At critical point A)

\[ \phi_b = 0.9 \]

\[ F_{Z_x} = 51.55 \text{ in-k} > 1.5F_{S_x} = 51.55 \text{ in-k} \]

\[ \therefore \phi_b \frac{M_n}{M_{n0}} = 46.39 \text{ in-k} > 33.57 \text{ in-k} \quad (\text{OK}) \]

3.2b. (ALTERNATIVE) Welded Group Design for Slotted Plate

Type: Fastener (Non-ductile)

Load Characteristics (Based on page 1)

<table>
<thead>
<tr>
<th>Load Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_{C_x} )</td>
<td>- kips</td>
</tr>
<tr>
<td>( F_{C_y} )</td>
<td>- kips</td>
</tr>
<tr>
<td>( F_{C_z} )</td>
<td>8.76 kips</td>
</tr>
</tbody>
</table>

Internal Forces

\[ T = (b_t + 0.5) F_{C_z} / 0.5w_{col} = 18.24 \text{ kips} \quad (\text{Assumed pivot point along the centreline of column}) \]

Welded Group Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth ( L_w = W )</td>
<td>6 in</td>
</tr>
</tbody>
</table>

Elastic Weld Stresses

Max. shear stress \( f_s = T/L_w = 3.04 \text{ ksi/in} \)

Consider the strength of 1/16" weld size E70XX,

\[ \phi_v = 0.75 \]

\[ \phi_v R_n = 0.6(70 \text{ ksi})(0.70)(1/16 \text{ in})(1 \text{ in}) = 1.378 \text{ k\text{in per 1/16" weld thickness}} \]

Required weld thickness \( t_{\text{weld}} = f_{\text{max}} / \phi_v R_n = 6/16 \text{ in} \)

Weld thickness provided \( 6/16 \text{ in} \)

Plate thickness \( 1 \text{ in} = 1.125 \text{ in} \)

Min. fillet weld size \( 5/16 < 6/16 \quad (\text{OK}) \)

Max. fillet weld size \( 17/16 > 6/16 \quad (\text{OK}) \)
4. Design of Cantilevered Bearing Connection - Connections A and B

A. Structural Steel HSS Design

<table>
<thead>
<tr>
<th>Load Characteristics (Based on page 1)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{Cx}$</td>
<td>43.02 kips</td>
</tr>
<tr>
<td>$F_{Cy}$</td>
<td>19.22 kips</td>
</tr>
<tr>
<td>$F_{Cz}$</td>
<td>5.16 kips</td>
</tr>
</tbody>
</table>

**Internal Forces**

- **Shear** $V_u = \sqrt{F_{Cx}^2 + F_{Cy}^2} = 47.12$ kips
- **Eccentricity** $e = d - c = 10$ in
- **Moment** $M_{ux} = F_{Cy}c + F_{Cz}e^2 = 207.71$ in-k
  
  $M_{uy} = F_{Cx}c = 430.24$ in-k

**Axial**

$P_u = F_{Cz} = 5.16$ kips

**AISC Manual Section:** HSS6 x 6 x 1/2

<table>
<thead>
<tr>
<th>Width H</th>
<th>6 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness t</td>
<td>0.465 in</td>
</tr>
<tr>
<td>Flange slenderness ratio, b/t</td>
<td>9.90</td>
</tr>
<tr>
<td>Web slenderness ratio, h/t</td>
<td>9.90</td>
</tr>
<tr>
<td>Gross area $A_g$</td>
<td>$9.74$ in$^2$</td>
</tr>
<tr>
<td>Plastic Modulus $S_x$</td>
<td>$16.1$ in$^3$</td>
</tr>
<tr>
<td>Torsional constant $J$</td>
<td>$81.10$ in$^3$</td>
</tr>
<tr>
<td>Radius of gyration $r_y$</td>
<td>$2.23$ in</td>
</tr>
<tr>
<td>(i) Flexural Capacity</td>
<td></td>
</tr>
<tr>
<td>$I_{ub} = 0.9$</td>
<td></td>
</tr>
<tr>
<td>Lateral Torsional Buckling (0.1J)</td>
<td></td>
</tr>
<tr>
<td>$F_{L/T} = 1110.9$ kips</td>
<td></td>
</tr>
</tbody>
</table>

**Check $L_p = 0.13r_yE(JA)^{0.5}/M_p = 259$ in $> c = 10$**
4. Design of Cantilevered Bearing Connection - Connections A and B (Cont'd)

A. Structural Steel HSS Design (Cont'd)

(i) Flexural Capacity (Cont'd)

\[ \phi_b M_{ux} = \phi_r M_p = 820 \text{ in-kips} > 208 \text{ in-kips} \quad \text{(OK)} \]

\[ \phi_b M_{uy} = \phi_r M_p = 820 \text{ in-kips} > 430 \text{ in-kips} \quad \text{(OK)} \]

Check Flange Local Buckling (FLB)

\[ \lambda_p = 1.12 \left( \frac{E}{\sigma_y} \right)^{0.5} = 28.12 > \frac{b}{t} = 9.90 \quad \text{(OK. Range is compact)} \]

Check Web Local Buckling (WLB)

\[ \lambda_p = 3.76 \left( \frac{E}{\sigma_y} \right)^{0.5} = 94.41 > \frac{h}{t} = 9.90 \quad \text{(OK. Web is compact)} \]

(ii) Axial Capacity

Compressive capacity check

Effective length factor: \( K = 2 \) (Assume cantilever)

Unsupported length \( L = c = 10 \) in

Effective length: \( KL = 20 \) in

Slenderness ratio: \( \lambda_c = (KL/r) = 9.0 \)

Critical buckling stress \( F_{cr} = 45.75 \) ksi

(Only compact section shall be used. Q is taken as 1.0)

\[ \phi_c = 0.85 \]

\[ \phi_c P_n = \phi_r F_c A_n = 378.8 \text{ Kips} > 5.16 \quad \text{(OK)} \]

(Agrees with Table 4-6)

Tensile capacity check

\[ \phi_t = 0.9 \] (for yielding)

\[ \phi_t = 0.75 \] (for rupture)

Since the HSS is symmetrically welded to plates on both sides, \( U = 1 \) and \( A_n = A_o = A_g \) i.e. no reduction in area.

\[ \phi_t P_n = \phi_r F_t A_g = 403.2 \text{ Kips} > 5.16 \quad \text{(OK)} \]

\[ \phi_t P_n = \phi_r F_t A_o = 423.7 \text{ Kips} > 5.16 \quad \text{(OK)} \]

(iii) Shear Capacity Check

\[ \phi_v = 0.75 \]

Web area \( A_w = 2H_t = 5.58 \text{ in}^2 \)

Shear stress \( F_v = 27.6 \text{ ksi} \)

\[ \phi_v V_n = \phi_r F_v A_w = 115.506 \text{ Kips} > 47.12 \quad \text{(OK)} \]
4. Design of Cantilevered Bearing Connection - Connections A and B (Cont'd)

A. Structural Steel HSS Design (Cont'd)

(iv) Combined Stress Check

\[ \left[ (P_u / \phi P_D) + (M_u / \phi M_D) + (V_u / \phi V_D) \right]^2 = 0.96 \]

Note: If a shear plate is combined with the bearing support, the HSS would only need to be designed for vertical load; all lateral load would be taken by the shear plate. In this case, the bearing support would be reduced to HSS4x4x3/8.

5. Concrete Anchorage Design - Tube Steel Anchorage in Panel

Type: Fastener (Non-ductile)

Load Characteristics (Based on page 1)

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{Cx}$</td>
<td>49.45 kips</td>
<td></td>
</tr>
<tr>
<td>$F_{Cy}$</td>
<td>19.22 kips</td>
<td></td>
</tr>
<tr>
<td>$F_{Cz}$</td>
<td>8.76 kips</td>
<td></td>
</tr>
</tbody>
</table>

Internal Forces

- Shear: $V_u = (F_{Cx}^2 + F_{Cy}^2)^{0.5} = 53.05$ kips
- Eccentricity: $c = 10$ in
- $\theta_2 = 3$ in
- Moment: $M_{ux} = F_{Cy}c + F_{Cz}\theta_2 = 218.49$ in-k
- $M_{uy} = F_{Cx}c = 494.50$ in-k
- Axial: $P_u = F_{Cz} = 8.76$ kips
- Spacing: $d_v = t - 2$ cover = 5 in

Maximum axial load per vertical bar

\[ F_{1V} = (M_{ux} + V_{uy}(cover + d_v))/2d_v = 54.0 $ kips \]

Required area of steel

- $\phi = 0.9$
- for vertical bars: $A_v = F_{1V}/f_y = 1.00$ in$^2$

Use 4#9 for vertical rebars

For Connection A and D

Maximum axial load per horizontal bar

- Spacing: $d_h = 6$ in
- Maximum axial load per vertical bar

\[ F_{1V} = (M_{uy} + V_{ux}(d_h))/3d_h = 33.9 $ kips \]

Required area of steel

- $\phi = 0.9$
- for vertical bars: $A_v = F_{1V}/f_y = 0.63$ in$^2$

Use 6#8 for horizontal rebars


### 5. (Alternative) Concrete Anchorage Design - Headed Stud Anchors

#### 1. Tensile Capacity

**Material Properties**

- **Headed studs Properties**
  - Shank diameter \( d_0 \) \( = 7/8 \) (See table)
  - Shank area \( A_{se} \) \( = 0.60 \text{ in}^2 \)
  - Head diameter \( d_{hs} \) \( = 1.375 \text{ in} \)
  - Head thickness \( h_{hs} \) \( = 0.3750 \text{ in} \)
  - Bearing area \( A_{brg} \) \( = 0.880 \text{ in}^2 \)
  - Tensile strength \( f_{ut} \) \( = 65 \text{ ksi} \)

- **Plate**
  - Thickness \( t_{up} \) \( = 1/2 \text{ in} \)
  - Min. plate thickness \( t_{up,min} = 0.5d_0 = 0.44 > 0.5 \text{ in.} \) (OK)

**Stud Geometry** (See diagram on right for definition)

- \( d_{el} \) \( = 12 \text{ in} \)
- \( X \) \( = 4 \text{ in} \)
- \( SED = d_{el} + X \) \( = 16 \text{ in} \)
- \( d_{el} \) \( = 10 \text{ in} \)
- \( Y \) \( = 12 \text{ in} \)
- \( BED = d_{el} + Y \) \( = 22 \text{ in} \)
- \( e'_v = 0 \text{ in} \)

**Concrete Breakout Strength**

- Depth of headed stud \( h_{ef} \) \( = 6 \text{ in} \)
- Concrete strength \( f'_c \) \( = 5,000 \text{ lbs/in}^2 \)
- Lightweight concrete factor \( \lambda \) \( = 1.0 \) (normal weight concrete)
- Breakout strength coefficient \( C_{bs} = 3.33(f'_c/h_{ef})^{0.5} \) \( = 96.1 \text{ lbs/in}^2 \)
- Panel thickness \( h \) \( = 8 \text{ in} \)

---

**Dimensions of headed studs (PCI Table 6.5.1.2)**

<table>
<thead>
<tr>
<th>( d_0 )</th>
<th>( A_{se} )</th>
<th>( d_{hs} )</th>
<th>( h_{hs} )</th>
<th>( A_{brg} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 1/4 )</td>
<td>( 0.05 )</td>
<td>( 0.5 )</td>
<td>( 0.1875 )</td>
<td>( 0.15 )</td>
</tr>
<tr>
<td>( 3/8 )</td>
<td>( 0.11 )</td>
<td>( 0.34 )</td>
<td>( 0.28125 )</td>
<td>( 0.33 )</td>
</tr>
<tr>
<td>( 1/2 )</td>
<td>( 0.2 )</td>
<td>( 1 )</td>
<td>( 0.3125 )</td>
<td>( 0.59 )</td>
</tr>
<tr>
<td>( 5/8 )</td>
<td>( 0.31 )</td>
<td>( 1.25 )</td>
<td>( 0.3125 )</td>
<td>( 0.92 )</td>
</tr>
<tr>
<td>( 3/4 )</td>
<td>( 0.44 )</td>
<td>( 1.25 )</td>
<td>( 0.375 )</td>
<td>( 0.79 )</td>
</tr>
<tr>
<td>( 7/8 )</td>
<td>( 0.6 )</td>
<td>( 1.375 )</td>
<td>( 0.375 )</td>
<td>( 0.88 )</td>
</tr>
</tbody>
</table>
5. (Alternative) Concrete Anchorage Design - Headed Stud Anchors (Cont'd)

Concrete Breakout Strength (Cont'd)

Check for edge effect

Front edge distance \(d_{e1}\)  
\(d_{e1} > 1.5h_{ef} = 9\) in  
(No edge effect)

Side edge distance \(d_{e3}\)  
\(d_{e3} < 1.5h_{ef} = 9\) in  
(No edge effect)

:: CASE 1 : Not near a free edge

Projected concrete failure area \(A_N\)  
\(660\) in\(^2\)

Case 1  
\((X + 3h_{ef})(Y + 3h_{ef})\)  
\(660.0\) in\(^2\)

Case 2(a)  
\((X + d_{e1} + 1.5h_{ef})(Y + 3h_{ef})\)  
\(750.0\) in\(^2\)

Case 2(b)  
\((X + 3h_{ef})(Y + d_{e3} + 1.5h_{ef})\)  
\(682.0\) in\(^2\)

Case 4  
\((X + d_{e1} + 1.5h_{ef})(Y + d_{e3} + 1.5h_{ef})\)  
\(775.0\) in\(^2\)

PCI Fig. 6.5.4.2

Mod. factor for edge effect  
\(\Psi_{edN} = (0.7 + 0.3(d_{e,min}/1.5h_{ef}) < 1.0)\)  
1.00

Mod. Factor for cracking  
\(C_{orb}\)  
1.0 (Assumed uncracked concrete region)

Strength reduction factor  
\(\phi\)  
0.75

Factored breakout strength  
\(\phi N_{b,2} = \phi C_{orb} A_N C_{crb} C_{edN}\)  
47.6 kips

Pullout Strength

Strength reduction factor  
\(\phi\)  
0.7

Mod. Factor for cracking  
\(C_{orp}\)  
1.0 (Assumed uncracked concrete region)

Factored breakout strength  
\(\phi N_{b,1} = \phi 11.2 A_{brg} f_{c}' N_{studs}\)  
276.0 kips

Concrete Blownout Strength

\(d_{e,min} = \min(d_{e1}, d_{e3})\)  
10 in  
\(> 0.4h_{ef} = 2.4\) in  
(No side-face blownout)

Steel Tensile Strength

Strength reduction factor  
\(\phi\)  
0.75

Design strength  
\(\phi N_s = \phi (N_{studs} A_{sef})\)  
234 kips

Summary of Tensile Capacity

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete breakout</td>
<td>47.6 kips (Controls)</td>
</tr>
<tr>
<td>2. Concrete pullout</td>
<td>276.0 kips</td>
</tr>
<tr>
<td>3. Steel stud fracture</td>
<td>234.0 kips</td>
</tr>
</tbody>
</table>

:: tensile capacity of HCA is controlled by concrete breakout
5. (Alternative) Concrete Anchorage Design - Headed Stud Anchors (Cont’d)

### Front Edge Strength

- **Single anchor strength**
  \[ V_{od} = 16.5(f'c)^{1/3}(\text{BED})^{1/3} \]
  71.2 kips

- **X-spacing factor**
  \[ C_X = 0.85 + X/8.6E < n_{\text{studs-back}} \]
  0.91

- **Thickness factor**
  \[ h = 8 < 1.5 \text{BED} = 33 \text{ in} \]

- **Eccentricity factor**
  \[ C_{ev} = 1/(1 + 0.67(e'/\text{BED})) \]

- **Cracking factor**
  \[ C_{vcr} = 0.7(\text{SED}/\text{BED})^{1/3} \]

- **Strength reduction factor**
  \[ C_{Cr} = 1 \]

- **Factored shear strength**
  \[ \phi V_{od} = \phi V_{od} C_{X} C_{h} C_{ev} C_{vcr} \]
  29.3 kips

### Corner Edge Strength

- **Corner factor**
  \[ C_{e3} = 0.7(\text{SED}/\text{BED})^{1/3} \]

- **Strength reduction factor**
  \[ \phi = 1 \]

- **Factored shear strength**
  \[ \phi V_{od} = \phi V_{od} C_{X} C_{h} C_{ev} C_{vcr} \]
  20.3 kips

### Side Edge Strength

- **Single anchor strength**
  \[ V_{od1} = 87.3((f'c)^{0.5}(d_{s})^{1.33}(d_{o})^{0.75}) \]
  151.64 kips

- **X-spacing factor**
  \[ n_x = N_x \]

- **Y-spacing factor**
  \[ n_y = N_y \]

- **Eccentricity factor**
  \[ C_{ev1} = 1 - (e'_{1}/4d_{o}) \]

- **Cracking factor**
  \[ C_{vcr} = 1 \]

- **Factored shear strength**
  \[ \phi V_{od1} = \phi V_{od1} C_{X} C_{Y} C_{ev1} C_{vcr} \]
  99.0 kips

### Steel Shear Strength

- **Strength reduction factor**
  \[ \phi = 1 \]

- **Design strength**
  \[ \phi N_s = \phi (N_{\text{studs}})(A_{sef}) \]
  312 kips

**Condition at cracked concrete**
- No edge reinforcement or < No. 4 bar
- Edge reinforcement ≥ No. 4 bar confined within stirrups with a spacing ≤ 4 in.
5. (Alternative) Concrete Anchorage Design - Headed Stud Anchors (Cont'd)

2. Shear Capacity

Summary of Shear Capacity

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Front edge</td>
<td>29.3 kips</td>
</tr>
<tr>
<td>2. Corner edge</td>
<td>20.3 kips</td>
</tr>
<tr>
<td>3. Side edge</td>
<td>99.0 kips</td>
</tr>
<tr>
<td>4. Steel stud fracture</td>
<td>312.0 kips</td>
</tr>
</tbody>
</table>

Check SED/BED = 0.727  
Corner condition control

Type: Fastener (Non-ductile)

Load Characteristics (Based on page 1)

- $F_{Cx} = 49.45$ kips > 20.3 kips
- $F_{Cy} = -$ kips
- $F_{Cz} = 8.76$ kips

Add additional tail bar to anchoring plate

- $F = 29.18$ kips
- Additional capacity required
- $\phi = 0.9$
- Required area of steel

Use 2#5 steel bars as tail bars

END OF CALCULATION
APPENDIX B

ANALYTICAL CRACK PATTERNS OF FINITE ELEMENT MODELS

B.1 Model cp1

The minimum area of steel reinforcement typical of conventional architectural precast concrete panel was specified. Considerable cracking has occurred at the loading point at a very low load. The analysis terminated at 9.2 kips due to non-convergence of solution.

Figure B.1: Crack Pattern of Model cp1.
B.2 Model cp2

By reinforcing the top connection at the loading point, the panel was able to reach higher loads with less cracking compared with cp1. The analysis terminated at 10.1 kips due to non-convergence of solution. The additional diagonal rebars did appear to have a noticeable effect on the panel lateral stiffness and hence it may not be necessary.

Figure B.2: Crack Pattern of Model cp2.
B.3 Model cp3

With significant increased in reinforced bars, the panel was able to sustain higher loads before extensive cracking formed. The additional rebars effectively distribute the applied load to the surrounding concrete mass. The seen in the side view, the characteristic cone-shaped cracking pattern was captured by the model. The analysis terminated at 23.5 kips due to non-convergence of solution. Based on the result, it is expected that localized shear failure of the connection would control the design.

Figure B.3: Crack Pattern of Model cp3.
B.4 Model cp4

In addition to the cone-shaped crack pattern characteristic of a loaded HCA, significant vertical flexure cracks had developed at the left bearing support A due to the bending of the panel strip by the rigid HSS section as it rotate about its pinned end. The numerical results suggested that the use of the embedded HSS section may not be adequate to transfer the high in-plane load.

Figure B.4: Crack Pattern of Model cp4.
B.5 Model cp5

It appeared that the panel performed significantly better without the cantilevered bearing supports. The HCA were able to develop a higher load carrying capacity without appreciable deterioration in stiffness. At higher load, the cracking extends through the back studs of the HCA towards the side edge which appeared to point to a side-edge breakout failure. The result also pointed to the fact that it may be possible to retain the HSS if the HSS is detailed (e.g., stiffener, thicker plate) to better distribute the applied load and moment over the entire plate.

Figure B.5: Crack Pattern of Model cp5.
B.6 Model cp6

Fixing the boundary conditions of the cantilevered bearing support led to the greatest improvement in lateral stiffness and possibly load carrying capacity.

Figure B.6: Crack Pattern of Model cp6.
B.7 Model cp7

Comparing with cp4, the omission of the mid-layer of reinforcing bars did not seem to significantly reduce the stiffness or strength of the panel. This mid-layer was intended to confine and strengthen the concrete around the highly stressed loaded points. It now appeared that the conventional method of placing the steel at each face of the panel is sufficient and a mid-layer will not be necessary.

Figure B.7: Crack Pattern of Model cp7.
Figure B.8: Crack Pattern of Model cp8.
APPENDIX C

RESULTS OF CORRELATION STUDY AND COMPARATIVE STUDY

C.1 PEAK RESPONSE PARAMETER PROFILES

C.1.1 Peak Story Displacement (PSD)

Figure C.1: Peak Story Displacement for EC Series.
Figure C.2: Peak Story Displacement for TF Series.

Figure C.3: Peak Story Displacement for LA Series.
C.1.2 Peak Interstory Drift (PID)

Figure C.4: Peak Interstory Drift for EC Series.

Figure C.5: Peak Interstory Drift for TF Series.
Figure C.6: Peak Interstory Drift for LA Series.
C.1.3 Peak Story Shear (PSS)

Figure C.7: Peak Story Shear for EC Series.

Figure C.8: Peak Story Shear for TF Series.
Figure C.9: Peak Story Shear for LA Series.
C.1.4 Peak Story Acceleration (PSA)

Figure C.10: Peak Story Acceleration for EC Series.

Figure C.11: Peak Story Acceleration for TF Series.
Figure C.12: Peak Story Acceleration for LA Series.
C.2 PEAK COLUMN FORCES AND MOMENTS

C.2.1 EC1

Figure C.13: Peak Column Forces and Moments for EC1.
### Figure C.14

Peak Column Forces and Moments for EC2.
C.2.3 TF1

Figure C.15: Peak Column Forces and Moments for TF1.
Figure C.16: Peak Column Forces and Moments for TF2.
C.2.5 LA1

Figure C.17: Peak Column Forces and Moments for LA1.
C.2.6 LA2

Figure C.18: Peak Column Forces and Moments for LA2.
C.3 HYSTERSIS LOOPS OF FRICTION DAMPER

C.3.1 FD Model

C.3.1.1 EC1 (Calculated total dissipated energy: 1.6 kips-in.)

Figure C.19: Hysteresis Loops of FD for EC1.
**C.3.1.2 EC2** *(Calculated total dissipated energy: 13.3 kips-in.)*

![Hysteresis Loops of FD for EC2](image)

**Figure C.20:** Hysteresis Loops of FD for EC2.
Figure C.21: Hysteresis Loops of FD for EC3.
C.3.1.4 TF1 (Calculated total dissipated energy: 0.0 kips-in.)

Figure 3.22: Hysteresis Loops of FD for TF1.
C.3.1.5 TF2 (Calculated total dissipated energy: 2.3 kips-in.)

Figure 3.23: Hysteresis Loops of FD for TF2.
C.3.1.6 TF3 (Calculated total dissipated energy: 0.9 kips-in.)

Figure 3.24: Hysteresis Loops of FD for TF3.
C.3.1.7 LA1 (Calculated total dissipated energy: 0.0 kips-in.)

Figure 3.25: Hysteresis Loops of FD for LA1.
Figure 3.26: Hysteresis Loops of FD for LA2.
C.3.1.9 LA3 (Calculated total dissipated energy: 1.3 kips-in.)

Figure 3.27: Hysteresis Loops of FD for LA3.
C.3.2 EDCS Model

C.3.2.1 EC1 (Bay1) (Calculated total dissipated energy: 0.4 kips-in.)

Figure 3.28: Hysteresis Loops of EDCS (Bay 1) for EC1.
C.3.2.2 EC1 (Bay2) (Calculated total dissipated energy: 4.1 kips-in.)

Figure 3.29: Hysteresis Loops of EDCS (Bay 2) for EC1.
C.3.2.3 EC2 (Bay1) (Calculated total dissipated energy: 2.6 kips-in.)

Figure 3.30: Hysteresis Loops of EDCS (Bay 1) for EC2.
C.3.2.4 EC2 (Bay2) (Calculated total dissipated energy: 11.5 kips-in.)

Figure 3.31: Hysteresis Loops of EDCS (Bay 2) for EC2.
C.3.2.5 EC3 (Bay1) (Calculated total dissipated energy: 7.2 kips-in.)

Figure 3.32: Hysteresis Loops of EDCS (Bay 1) for EC3.
C.3.2.6 EC3 (Bay2) (Calculated total dissipated energy: 20.0 kips-in.)

Figure 3.33: Hysteresis Loops of EDCS (Bay 2) for EC3.
C.3.2.7 TF1 (Bay1) (Calculated total dissipated energy: 0.0 kips-in.)

Figure 3.34: Hysteresis Loops of EDCS (Bay 1) for TF1.
Figure 3.35: Hysteresis Loops of EDCS (Bay 2) for TF1.
**C.3.2.9 TF2 (Bay1) (Calculated total dissipated energy: 0.1 kips-in.)**

Figure 3.36: Hysteresis Loops of EDCS (Bay 1) for TF2.
**C.3.2.10** TF2 (Bay2) *(Calculated total dissipated energy: 1.2 kips-in.)*

Figure **3.37**: Hysteresis Loops of EDCS (Bay 2) for TF2.
C.3.2.11 TF3 (Bay1) (Calculated total dissipated energy: 0.2 kips-in.)

Figure 3.38: Hysteresis Loops of EDCS (Bay 1) for TF3.
**C.3.2.12** TF3 (Bay2) (Calculated total dissipated energy: 2.7 kips-in.)

Figure 3.39: Hysteresis Loops of EDCS (Bay 2) for TF3.
C.3.2.13 LA1 (Bay1) (Calculated total dissipated energy: 0.0 kips-in.)

Figure 3.40: Hysteresis Loops of EDCS (Bay 1) for LA1.
C.3.2.14 LA1 (Bay2) (Calculated total dissipated energy: 0.2 kips-in.)

Figure 3.41: Hysteresis Loops of EDCS (Bay 2) for LA1.
C.3.2.15 LA2 (Bay1) (Calculated total dissipated energy: 0.1 kips-in.)

Figure 3.42: Hysteresis Loops of EDCS (Bay 1) for LA2.
C.3.2.16 LA2 (Bay2) (Calculated total dissipated energy: 1.6 kips-in.)

Figure 3.43: Hysteresis Loops of EDCS (Bay 2) for LA2.
Figure 3.44: Hysteresis Loops of EDCS (Bay 1) for LA3.
Figure 3.45: Hysteresis Loops of EDCS (Bay 2) for LA3.
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Hathairat Maneetes

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PUBLICATIONS