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Seismic design and analysis of unbonded post-tensioned precast wall systems

Sriram R. Aaleti
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Seismic design and analysis of unbonded post-tensioned precast wall systems

by

Sriram R. Aaleti

A thesis submitted to the graduate faculty
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

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Program of Study Committee:
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Iowa State University
Ames, Iowa
2005
This is to certify that the master's thesis of

Sriram R. Aaleti

has met the thesis requirements of Iowa State University
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ABSTRACT

Following the satisfactory response of the unbonded post-tensioned precast concrete jointed wall system tested for seismic performance as part of the PREcast Seismic Structural Systems (PRESSS) test building, a set of design guidelines was published. Based on these guidelines, Thomas & Sritharan developed a procedure to analyze the unbonded jointed wall systems. The primary objective of this research is to improve this analysis procedure so that it can be applied to analyze both unbonded post-tensioned single walls and jointed wall systems. Using the experimental data from PRESSS test building, ATLSS research center single wall tests, the accuracy of this analysis procedure and that based on the monolithic beam analogy (MBA) are examined. It was found that both the analysis methods predicted the moment resistance of the walls adequately at the given base rotation. Based on these analysis procedures, revised set of design guidelines are proposed for design of precast jointed wall systems with unbonded post-tensioning steel. A detailed investigation on the influence of several wall parameters on the lateral load behavior of jointed wall system is conducted and a new jointed wall concept refer to as the “jointed wall-column (JWC) system” is proposed. It is shown by analysis that, the JWC system will be more economical than that of an equivalent jointed wall system tested in the PRESSS building.
LIST OF SYMBOLS

\begin{itemize}
\item \textit{A} \quad \text{Area of wall base}
\item \textit{A}_p \quad \text{Area of post-tensioning steel}
\item \textit{B} \quad \text{Width of one wall panel}
\item \textit{b} \quad \text{Width of UFP connector}
\item \textit{B} \quad \text{Entire wall system length}
\item \textit{b}_{sc} \quad \text{Width of UFP connector}
\item \textit{c} \quad \text{Neutral axis depth}
\item \textit{C} \quad \text{Compressive reaction on one wall panel}
\item \textit{C}_0 \quad \text{Compressive reaction on one wall panel at zero drift}
\item \textit{C}_c \quad \text{Compression capacity of one wall panel}
\item \textit{C}_{conf} \quad \text{Compressive reaction obtained utilizing confinement model}
\item \textit{C}_{des} \quad \text{Compressive reaction on one wall panel at design limit state}
\item \textit{E}_c \quad \text{Modulus of elasticity of concrete}
\item \textit{E}_p \quad \text{Modulus of elasticity of post-tensioning}
\item \textit{E}_{scc} \quad \text{Secant modulus of confined concrete at peak stress}
\item \textit{F} \quad \text{Vertical interface shear force}
\item \textit{f}_c \quad \text{Concrete stress}
\item \textit{f}_c' \quad \text{Concrete strength}
\item \textit{f}_{rc} \quad \text{Peak confined concrete strength}
\item \textit{F}_{decomp} \quad \text{Horizontal force on the wall causing decompression at wall end}
\item \textit{f}_g' \quad \text{Grout strength}
\item \textit{f}_{lk} \quad \text{Effective lateral confining stress in the 'x' direction}
\end{itemize}
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
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<tr>
<td>$f_{ty}$</td>
<td>Effective lateral confining stress in the 'y' direction</td>
</tr>
<tr>
<td>$f_p$</td>
<td>Total stress in the post-tensioning tendon</td>
</tr>
<tr>
<td>$f_{p,des}$</td>
<td>Stress in post-tensioning tendon at design limit state</td>
</tr>
<tr>
<td>$f_{p0}$</td>
<td>Initial stress in the post-tensioning tendon</td>
</tr>
<tr>
<td>$f_{pi}$</td>
<td>Initial stress in the post-tensioning tendon</td>
</tr>
<tr>
<td>$f_{py}$</td>
<td>Yield stress of post-tensioning tendon</td>
</tr>
<tr>
<td>$F_{sc}$</td>
<td>Total force of all shear connectors along a vertical joint</td>
</tr>
<tr>
<td>$F_{sc,left}$</td>
<td>Total force of all shear connectors in joint to left of panel</td>
</tr>
<tr>
<td>$F_{sc,net}$</td>
<td>Net vertical force on one panel from all shear connectors</td>
</tr>
<tr>
<td>$F_{sc,right}$</td>
<td>Total force of all shear connectors in joint to right of panel</td>
</tr>
<tr>
<td>$F_{sc0}$</td>
<td>Total force in one shear connector</td>
</tr>
<tr>
<td>$f_{yh}$</td>
<td>Yield strength of transverse reinforcement</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>$h_{eff}$</td>
<td>Height above foundation of lateral load resultant on wall</td>
</tr>
<tr>
<td>$H_{story}$</td>
<td>Height per story</td>
</tr>
<tr>
<td>$H_t$</td>
<td>Wall height</td>
</tr>
<tr>
<td>$h_a$</td>
<td>Unbonded length of post-tensioning tendon</td>
</tr>
<tr>
<td>$h_w$</td>
<td>Wall height</td>
</tr>
<tr>
<td>$I$</td>
<td>Moment of inertia</td>
</tr>
<tr>
<td>$I_{eff}$</td>
<td>Effective moment of inertia</td>
</tr>
<tr>
<td>$k_1$</td>
<td>Uniform stress in Whitney rectangular stress block divided by $f_g$'</td>
</tr>
<tr>
<td>$K_c$</td>
<td>Confinement effectiveness coefficient</td>
</tr>
<tr>
<td>$L_p$</td>
<td>Equivalent plastic hinge length</td>
</tr>
<tr>
<td>$l_w$</td>
<td>Length of one wall panel</td>
</tr>
<tr>
<td>$l_{w,tot}$</td>
<td>Total wall length</td>
</tr>
<tr>
<td>$M$</td>
<td>Moment resistance of wall</td>
</tr>
<tr>
<td>$M_{wall}$</td>
<td>Moment capacity of one wall</td>
</tr>
</tbody>
</table>
$M_{wall, system}$  Total moment capacity of wall system

$M_{decomp}$  Moment at decompression point

$M_{des}$  Moment demand at design limit state

$M_{lo}$  Lift-off moment of wall

$M_n$  Nominal moment of wall

$M_{ot}$  Overturning moment

$M_{r, net}$  Net righting moment of wall

$M_u$  Factored moment

$n$  Number of wall panels

$N$  Total tension force on one wall panel

$N_0$  Total tension force on one wall panel from gravity plus post-tensioning at zero drift

$N_{des}$  Total tension force on one wall panel from gravity plus post-tensioning at design limit state

$n_{wc}$  Number of shear connectors required per vertical joint

$n_{story}$  Number of stories

$n_{con}$  Number of UFPs along a vertical joint

$P$  Total force in the post-tensioning tendon

$P_0$  Initial Force in the post-tensioning tendon

$P_{des}$  Force in the post-tensioning tendon at design limit state

$P_{wt}$  Wall panel post-tensioning force

$Q_{des}$  Base shear demand for the design level ground motion

$R$  Response modification factor

$t_w$  Wall thickness

$t_{er}$  Thickness of wall panel effective in resisting compressive force
$V$  Base shear
$V_b$  Base shear
$V_{csc}$  Base shear capacity at the failure state
$V_{dec}$  Base shear capacity at decompression state
$V_{des}$  Base shear demand under design level ground motion
$V_{des}$  Design base shear
$V_{ell}$  Base shear capacity at the softening state
$V_i$  Force applied to $i$th floor by the actuator
$V_{lip}$  Base shear capacity at the yielding state
$V_{\text{max}}$  Expected maximum base shear demand under survival level ground motion
$v_{ua}$  Factored shear stress
$W$  Wall panel self weight
$W$  Total gravity load from all floors on one panel
$W_{\text{panel}}$  Self-weight of one panel
$W_{\text{story}}$  Building weight per floor
$\alpha$  Distance from the compression face of the member to the center of the compression force divided by the member depth
$\alpha_0$  Distance from the compression face of the member to the center of the compression force divided by the member depth at zero drift
$\alpha_{des}$  Distance from the compression face of the member to the center of the compression force divided by the member depth at design limit state
$\beta$  Moment arm reduction coefficient
$\beta_1$  Depth of equivalent stress block divided by the neutral axis depth
$\Delta$  Roof drift
$\delta_{\text{all}}$  Allowable story drift defined by NEHRP
$\Delta_{csc}$  Roof drift capacity at the failure state
\( \Delta_{cfc} \)  Roof drift capacity corresponding to the crushing of the concrete inside the wire mesh

\( \Delta_{dec} \)  Roof drift capacity at decompression state

\( \delta_{des} \)  Maximum story drift demand under design level ground motion

\( \Delta_{des} \)  Expected maximum roof drift demand under design level ground motion

\( \Delta_e \)  Elastic displacement component of a monolithic wall

\( \Delta_{ell} \)  Roof drift capacity at softening state

\( \Delta_{endlift} \)  Wall end uplift

\( \Delta f_p \)  Increase in stress in post-tensioning tendon

\( \Delta f_{p\infty} \)  Increase in stress in post-tensioning tendon between zero and design drift when concrete and grout strength are infinite

\( \Delta_j \)  Top floor displacement of jointed precast wall

\( \Delta_{llp} \)  Roof drift capacity at the yielding state

\( \Delta m \)  Top floor displacement of monolithic wall

\( \Delta_p \)  Elongation of post-tensioning tendon

\( \Delta p \)  Plastic displacement component of a monolithic wall

\( \Delta_{sur} \)  Expected maximum roof drift demand under survival level ground motion

\( \Delta_{sur} \)  Maximum roof drift demand under the survival level ground motion

\( \Delta_{top,decomp} \)  Top floor displacement at decompression point

\( \varepsilon_c \)  Concrete strain

\( \varepsilon_{c,ext} \)  Extreme fiber concrete strain

\( \varepsilon_{cc} \)  Strain corresponding to fcc'

\( \phi_c \)  Elastic curvature

\( \phi^* \)  Elastic curvature at the base of a precast jointed wall

\( \phi_f \)  Reduction factor
\[ \phi_p \] Plastic curvature
\[ \phi_s \] Capacity reduction factor
\[ \phi_u \] Ultimate curvature
\[ \phi_y \] Yield curvature
\[ \gamma_c \] Density of concrete
\[ \eta \] distance from the compression face to the neutral axis depth divided by member depth
\[ \eta_0 \] distance from the compression face to the neutral axis depth divided by member depth at zero drift
\[ \eta_{des} \] distance from the compression face to the neutral axis depth divided by member depth at design limit state
\[ \kappa_0 \] Ratio of design strength of shear connectors in one vertical joint to the vertical load on one panel
\[ \theta_{des} \] Base rotation
\[ \theta \] Base rotation at design limit state
\[ \rho_{fP0} \] stress ratio to ensure that post-tensioning tendon does not yield at maximum drift
\[ \rho_{ROC} \] demand/capacity ratio for overturning moment on panel
\[ \rho_{MOM} \] Force ratio to ensure that the panel slides rather than rocks
\[ \rho_{UPL} \] Ratio of uplift force to hold down force on one panel
\[ \rho_x \] Transverse reinforcement area in ‘x’ direction
\[ \rho_y \] Transverse reinforcement area in ‘y’ direction
\[ \rho_{ZRD} \] Parameter ratio controlling the residual drift
\[ \sigma_0 \] Initial stress at the base of the wall
\[ \zeta \] Damping of the system
\[ \zeta_{friction} \] Damping of the system using frictional inter-panel connectors
\[ \zeta_{ufp} \] Damping of the system using UFPs as inter-panel connectors
CHAPTER 1. INTRODUCTION

1.1 General

Concrete structural walls provide a cost effective means to resist seismic lateral loads and thus they are frequently used as the primary lateral load resisting system in reinforced concrete buildings. Structural walls with high flexural stiffness typically assists in limiting the inter-story drift and consequently causes limited structural damage during seismic events. Superior performance of buildings that consisted of structural walls was evident in several past seismic events [2]. The concrete structural walls can be of cast-in-place concrete or of precast concrete. With the added benefits of prefabrication, precast walls make an excellent choice for resisting lateral loads in concrete buildings. However, the application of precast systems is in general limited in seismic regions due to the lack of research information, which in turn, has imposed constraints in the current design codes. This chapter presents an introductory discussion on the performance of the structural walls in past earthquakes as well as on the concept of precast unbonded jointed wall systems for seismic regions.

1.2 Past Performance of Precast Structures with Structural Walls

Significant structural damage to concrete frame buildings and precast structures has been observed in moderate to large earthquakes that have occurred from 1960 to 1999. Fintel, who examined the structural damage of buildings after several of these earthquakes, reported that there was not a single concrete building with structural walls that experienced any significant damage [5]. A detailed literature review was conducted by Thomas & Sritharan [18] on the
seismic performance of precast structures with structural walls during the seismic events that occurred from 1960 to 1990. The most damaging recent earthquakes, which alerted the engineering community to closely examine the seismic behavior of precast structures, were the 1994 Northridge earthquake in California, the 1995 Kobe earthquake in Japan, and the 1999 Kocaeli earthquake in Turkey.

In the 1994 Northridge earthquake, several precast concrete parking structures performed poorly, causing significant structural damage. The primary cause for this damage was not any inherent deficiency in precast concrete elements, but was the use of poor connection details between precast elements and the violation of deformation compatibility (i.e. gravity framing system in buildings should deform along with earthquake-force-resisting-system and maintain its gravity load carrying capacity) expected in design codes by the parking structures. An investigation of the structural damage after the seismic event revealed that the lateral load resisting precast shear walls remained uncracked, while precast concrete elements of the floor system collapsed [19]. The positive aspect of all the devastation caused by the 1995 Kobe earthquake was good performance of several precast and prestressed concrete structures. Apartment buildings in Japan are typically two-to-five stories in height, and some of these buildings also include precast concrete walls as the primary elements to resist the gravity and lateral loads. None of these buildings that included the precast walls experienced any damage in the Kobe earthquake [see Figure 1.1], while cracking of concrete members was observed in cast-in-place concrete buildings. In the 1999 Kocaeli earthquake, a few apartment buildings with large precast wall panels connected in vertical and horizontal directions were found to have performed more than adequately amidst a lot of devastation [see Figure 1.2].

1.2.1 Limitations of Precast Concrete Application in Seismic Regions

There are several limitations that restrict the use of precast concrete in seismic regions. The primary limitation stems from poor performance of precast concrete frame buildings in the past seismic events. Although the poor performance of buildings was largely attributed to the use of substandard materials, poor construction practices, and insufficient design of
Figure 1.1 Precast concrete structures that experienced no damage during the 1995 Kobe earthquake in Japan [19].

connections, it had contributed to the decline of designers’ confidence in the use of precast concrete in seismic design [20].

Stringent provisions in the model building codes of the United States (e.g. Uniform Building Code (1997), NEHRP (1997), and International Building Code 2003 (2002)), also limit the precast concrete applications in seismic regions [21]. Typically these building codes require that the precast seismic systems be shown by analysis and tests to have lateral load resisting characteristics that are equal or superior to those of monolithic cast-in-place reinforced concrete systems. This requirement has led to the development of a design concept known as the ‘cast-in-place emulation’ [11, 21, 22]. To develop precast systems using the cast-in-place emulation, the current building codes propose two alternative designs: 1) structural systems that use “wet joints”; and 2) structural systems based on “dry joints”. In precast structural systems with wet joints, the connections are established using in-situ concrete to achieve the cast-in-place emulation [6]. However, these systems do not have all of the economical advantages of precast concrete technology because of the use of in-situ concrete. Furthermore, precast concrete systems that emulate the cast-in-place concrete systems have joints that are typically proportioned with sufficient strength to avoid inelastic deformations within these joints. Plastic hinges in these systems are forced to develop in the precast members, which does not lead
to an economical design. Dry joints in precast buildings are typically established through bolting, welding, or by other mechanical means. The behavior of precast concrete systems with dry joints differs from that of the emulation systems because the dry joints create natural discontinuities in the structure. The dry joints are often inherently less stiff than the precast members, and thus the deformations tend to concentrate at these joints.

These above described limitations present an opportunity for the development of innovative precast concrete seismic structural systems that may be quite different from the emulation types in term of concept and behavior[1]. Also, it is clear that a new structural system with an established set of design techniques will promote the confidence in the designers to use the precast concrete option for seismic design.

1.3 Unbonded Precast Post-Tensioned Wall Systems

In response to the recognized need to overcome the limitations for the use of precast concrete in seismic regions, the PRESSS (PREcast Seismic Structural Systems) program was initiated in the early 1990’s in the United States. Through this program, researchers envisioned to fulfill two primary objectives: (1) to develop comprehensive and rational design recommendations based on fundamental and basic research data which will emphasize the viability of
precast construction in various seismic zones, and (2) to develop new materials, concepts and technologies for precast construction suitable for seismic application [1].

As part of the PRESSS program, several tests were conducted at the University of California, San Diego (UCSD), and Lehigh University on unbonded precast wall systems to: (1) validate a rational design procedure for precast seismic structural systems, (2) provide acceptance of prestressing/post-tensioning of precast seismic systems, (3) provide experimental proof of overall building performance under seismic excitation, and (4) establish a consistent set of design recommendations for precast seismic structural systems [23].

The post-tensioned structural wall systems investigated as a part of the PRESSS program were the unbonded post-tensioned single walls and unbonded post-tensioned jointed wall systems. A jointed wall system was included in the PRESSS test building that was tested at UCSD [7, 9, 23]. At Lehigh University, several unbonded post-tensioned precast single walls with horizontal joints were tested [19]. Although the focus of this thesis is on jointed wall systems, analysis and design of single walls with unbonded post-tensioning tendons are also addressed.

1.3.1 Jointed Wall System

In a jointed wall system, two or more unbonded single precast walls are connected to each other with the help of special connectors along the vertical joints, as shown in Figure 1.3. Unbonded post-tensioned steel is distributed symmetrically about the center of each wall. The basic concept of the wall system is that it allows the wall to rock individually at the base when the wall system is subjected to lateral loads and return to its vertical position after the event has concluded [9].

The post-tensioned steel is typically designed to remain elastic under the design-level earthquake loading. As a result, the post-tensioning steel provides the restoring force for the jointed wall when the applied lateral load is removed. This restoring force helps to minimize the residual displacements of the wall when the lateral load is removed. The restoring capacity of the jointed wall depends on the amount of post-tensioning steel, the number of vertical
Figure 1.3 Details of a precast concrete jointed wall system.

connectors, initial prestressing force, and the cyclic behavior of the vertical connector. The vertical connectors dissipate energy by experiencing inelastic deformations under the applied earthquake loads. The shear transfer from the wall to the foundation at the base utilizes a friction mechanism. For these reasons, jointed wall systems have the ability to dissipate energy with minimal damage and little residual drift.

1.4 Research Scope

At various stages of the precast seismic systems development, guidelines were proposed for the design of jointed walls under seismic lateral loads. As used for monolithic walls, a section level analysis can not be easily performed at the base of a jointed wall due to the strain incompatibility induced by the usage of unbonded post-tensioned reinforcement. This has led to
design and analysis methods that approximate the strength of confined concrete and use of the equivalent rectangular stress block for predicting the neutral axis depth for design calculations and the structural behavior of the jointed wall. Using the experimental results available from the PRESSS building test and those from the single wall tests at Lehigh University, the study presented in this thesis is to (1) modify the guidelines for jointed wall design proposed by Thomas & Sritharan, (2) revise the seismic design guidelines for the designing of precast jointed walls, (3) study the behavior of the multi-panel (more than two wall panels) jointed wall, and (4) study the various factors influencing the jointed wall capacity and develop an efficient design to resist lateral loading.

The accuracy of the modified analysis is examined by comparing the analysis results with the experimental data. Also as a part of this study, a software tool has been developed in Visual Basic to predict the moment rotation behavior of jointed precast wall systems, with unbonded post-tensioned steel.

1.5 Report Layout

This report contains five chapters including the introduction presented in this chapter. The following chapter provides a summary of literature review, which includes brief discussion of previous experimental and analytical investigations on the lateral behavior of unbonded precast wall systems. Various design methodologies proposed for seismic design of unbonded precast wall systems are also included in this chapter. This is followed by a chapter entitled "Analysis and Validation of Behavior of Unbonded Precast Wall Systems". An analytical procedure developed by Thomas & Sritharan [1.18] with certain modifications is presented in this chapter. It also consists of a comparison of results from the analytical models with the experimental data for single wall as well as jointed wall systems. Chapter 4, entitled "Design Procedure for Unbonded Post-Tensioned Jointed Wall Systems", presents a revised set of design equations suitable for the jointed wall systems with design examples. Finally, Chapter 5 contains a summary of the report, along with the conclusions drawn from the research results and recommendations for future research. The thesis also includes three appendices, which
contain the derivation of $\beta$ value, the accuracy of the design equation for jointed wall systems presented in Chapter 4, and a detailed parametric study along with an optimum solution for the jointed wall in a paper format.
CHAPTER 2. REVIEW OF LITERATURE

2.1 Introduction

An overview of previous research conducted on precast concrete walls for seismic application is presented in this chapter. The literature review, which primarily focuses on unbonded post-tensioned precast walls, is categorized under three headings: experimental studies, analytical procedures, and design methodologies. In the section under experimental studies, large-scale tests conducted on unbonded precast walls by researchers at University of California at San Diego (UCSD) as part of the PRESSS program, researchers at ATLSS Research center of Lehigh University, and Rahman and Restrepo at the University of Canterbury, New Zealand are summarized. In the analytical studies section, different analytical methods proposed for analyzing the unbonded post-tensioned precast walls with connectors are presented. The monolithic beam analogy concept proposed by Pampanin et al. (2001) and the subsequent investigation by Thomas & Siritaran (2003), the analytical method proposed by Perez et al. (2004) and the analysis procedure established based on the PRESSS design guidelines by Thomas & Siritaran (2003) are described in detail. Finally, the design guidelines proposed by Galusha, Stanton & Nakaki, and Perez, Pessiki & Sause are summarized in the design section.

2.2 Experimental Studies

2.2.1 PRESSS Research Program

As discussed in Chapter 1, the focus of the PRESSS program was to develop comprehensive and rational design recommendations to emphasize the viability of precast concrete structures
in seismic regions. As a part of this program, a 60% scale model of a five story precast building with moment resisting frames in one direction and a precast jointed wall in the perpendicular direction [see Figure 2.2] was tested at UCSD. The main objectives of this large-scale testing were 1) to demonstrate the viability of precast concrete for seismic regions through experimental means, and 2) to develop seismic design guidelines for precast concrete systems, which can be incorporated into the model building codes.

The precast jointed wall used in the PRESSS building consisted of a total of four 8-in thick wall panels, each 2 1/2 stories tall (18.75-ft) by 9-ft wide. The panels were joined vertically to form two walls separated by a small gap between them. Each wall was secured to the foundation using four unbonded post-tensioning bars. These two walls were connected horizontally by 20 U-shaped flexural plates (also referred to as U-plates or UFP connectors), which were placed in the gap between the walls in vertical direction. The U-plates were used as the connector because of their ability to maintain force resistance under large displacements and contribute
Figure 2.2 Plan view of the PRESSS test building showing different seismic systems [9].

to energy dissipation by flexural yielding of the plates.

The test building was subjected to a series of simulated seismic tests, including the pseudodynamic tests involving modified segments of recorded accelerograms. Based on the observed response of the PRESSS building, it was concluded that the structural response of the wall system under different levels of seismic loading was excellent [see Figure 2.3] and damage to the precast jointed wall was noticeably negligible even when subjected to loads that were greater than the design-level earthquakes [Figure 2.3].

The observations reported by Priestley et al. on the behavior of the jointed wall are summarized as follows [8,23].

1. The experimentally measured peak roof displacement at the design level earthquake was 8.3 inches, which was about 8% lower than the target design displacement of 9 inches.

2. When subjected to 150% of the design-level earthquake load, the wall experienced a maximum displacement of 11.5 inches, with damage limited to spalling of cover concrete
at the wall toe regions.

3. After the 150% design-level load was removed, the wall had only 0.06% residual-drift showing the self-centering capability of the jointed wall, which was due to the restoring force provided to the wall by unbonded post-tensioned steel.

![Figure 2.3 The measured response of the PRESSS building in the wall direction [8].](image)

### 2.2.1.1 Behavior of UFP connector

The UFP connector [Figure 2.4] is an energy-dissipating flexible connector which resists the vertical shear force by rolling on a vertical plate and undergoing flexural inelastic action, thereby contributing to energy dissipation. The UFP connector for the PRESSS wall was fabricated with ductile 304 stainless steel to prevent cracking, which might occur in curved regions during the fabrication due to the use of a small radius. As a part of the PRESSS research program, Schultz and Magana investigated the behavior of UFP connector for seismic applications [13]. They found that the stainless steel UFP connector is nearly 2.5 times stronger than it was assumed. Overall, they reported that the UFP proved to be a desirable connector to
resist seismic actions that were simulated using reversed displacement cycles with an equivalent
drift ratios of up to 2%. As a part of the research conducted at Iowa State University, *Thomas*
tested the UFP connectors and established a force vs. displacement envelope for the UFP
connector [see Figure 2.5].

![Figure 2.4](image)

**Figure 2.4** Details of the UFP connector investigated by Schultz and Magana [13].

![Figure 2.5](image)

**Figure 2.5** The force-displacement response envelope established for the UFP connector used in the PRESSS building [18].
2.2.2 Single Wall Tests At The ATLSS Research Center

[19] A set of unbonded post-tensioned precast concrete single walls with the horizontal joints were tested under reversed cyclic loading at the ATLSS research center of Lehigh University in order to investigate their lateral load behavior. The tests were conducted on 5/12 scaled models, which represented a prototype precast wall designed for a six-story office building [Figure 2.6]. This study investigated the effect of total area of prestress steel, initial stress in post-tensioning steel, and confinement of concrete on the lateral load behavior of single precast walls with unbonded post-tensioning. As a part of this research, an analytical model was developed to predict the lateral behavior of this type precast walls.

Unlike the PRESSS wall, the test walls did not contain any vertical joints or ductile connectors, which were the primary energy dissipation source in the PRESSS wall. Also, the post-tensioning steel in these walls was distributed along the wall lengths, whereas the unbonded steel was concentrated at the center of each wall in the PRESSS building. A total of five precast walls, designated as TW1, TW2, TW3, TW4, and TW5, were tested as part of this research. The dimensions and the material properties of the wall specimens are summarized in Table 2.1. The specimens TW1 and TW2 were identical walls and utilized spiral reinforcement for concrete confinement in the bottom panel [Figure 2.6b, Figure 2.6c-1]. The specimens TW3 and TW4 were identical except for the initial stress in prestressing steel, and both walls utilized hoop reinforcement for confinement [Figure 2.6c-2]. Wall TW5 had the same confinement details as the TW3, but had less prestressing steel and initial prestressing stress compared to TW3. TW1 was tested under monotonic lateral loading with a constant gravity loading, while all other wall specimens were tested under cyclic lateral loading with a constant gravity load [Figure 2.6a]. The force-displacement responses of the test units observed under cyclic lateral loading are shown in Figure 2.7 and Figure 2.8, with the summary of the important results in Table 2.2.

Based on the observed experimental responses of the test units, the following conclusions were drawn about the behavior of the precast single walls by Perez et al. [19].

1. Significant gap opening occurred only along the base joint of the unbonded wall after the
### Table 2.1 The properties of single walls tested at the ATLSS research center [19].

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max.Drift (%)</th>
<th>$M_{crp}$</th>
<th>Residual Drift</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>TW1</td>
<td>3.7</td>
<td>157.8</td>
<td>-</td>
<td>crushing of concrete</td>
</tr>
<tr>
<td>TW2</td>
<td>3.7</td>
<td>157.8</td>
<td>0.1</td>
<td>buckling failure</td>
</tr>
<tr>
<td>TW3</td>
<td>3.75</td>
<td>155.8</td>
<td>-0.1</td>
<td>crushing of concrete</td>
</tr>
<tr>
<td>TW4</td>
<td>4.14</td>
<td>148.2</td>
<td>0.03</td>
<td>crushing of concrete</td>
</tr>
<tr>
<td>TW5</td>
<td>6.24</td>
<td>99.7</td>
<td>-0.01</td>
<td>crushing of concrete</td>
</tr>
</tbody>
</table>

### Table 2.2 Key results reported for the unbonded post-tensioned precast walls tested at ATLSS center [19].

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max.Drift (%)</th>
<th>$M_{crp}$</th>
<th>Residual Drift</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>TW1</td>
<td>3.7</td>
<td>157.8</td>
<td>-</td>
<td>crushing of concrete</td>
</tr>
<tr>
<td>TW2</td>
<td>3.7</td>
<td>157.8</td>
<td>0.1</td>
<td>buckling failure</td>
</tr>
<tr>
<td>TW3</td>
<td>3.75</td>
<td>155.8</td>
<td>-0.1</td>
<td>crushing of concrete</td>
</tr>
<tr>
<td>TW4</td>
<td>4.14</td>
<td>148.2</td>
<td>0.03</td>
<td>crushing of concrete</td>
</tr>
<tr>
<td>TW5</td>
<td>6.24</td>
<td>99.7</td>
<td>-0.01</td>
<td>crushing of concrete</td>
</tr>
</tbody>
</table>
decompression limit state and the lateral drifts of the walls were governed by this gap opening behavior along the base.

2. The flexural cracking was concentrated at the base of the walls which resulted in a significantly reduced amounts of energy dissipation.

3. Inelastic strains were developed in the prestressing steel at large drift ratios, causing loss of prestress load upon unloading. Prestress losses in the range of 11-23% were observed when the walls were displaced to higher drift levels.

4. Although the post-tensioned steel yielded during the experiments, the extent of yielding was minimal. The largest strain measured in prestressing steel was 23% of strain capacity, which shows a comfortable factor of safety for the fracture of post-tensioning steel.

5. The test specimens showed a small or no residual displacement upon removal of lateral
loads, which was the result of having enough restoring force in the system that was provided by the unbonded prestressing steel. The observed largest residual displacement during the wall test before failure was 0.1%.

6. From the experimental results it was concluded that a post-tensioned precast concrete wall can be designed to maintain its base shear capacity up to a lateral drift of 6% without failing or suffering significant damage at the ends of the wall base.

Figure 2.7 Experimental response of TW1 under monotonic loading [19].
Figure 2.8 Experimental cyclic load response of walls TW2 and TW3, respectively [19].

Figure 2.9 Experimental cyclic load response of walls TW4 and TW5, respectively [19].

2.2.3 Rahman and Restrepo

Rahman and Restrepo [24] tested three precast concrete shear walls: one connected to the foundation by means of post-tensioning only, and the other two connected to the foundation by post-tensioning steel and mild steel reinforcement. The construction details of units 2 and 3 are shown in Figure 2.10. Units 2 and 3 were identical, except for the confinement length, axial load, and debonded length of the mild steel reinforcement. These walls used a milled bar as the energy dissipater and the milled portion of the bar was embedded into the foundation. The two wall units performed satisfactorily under applied cyclic lateral loading. The force vs.
displacement history of these units are shown in Figure 2.11.

Figure 2.10  The construction details of Units 2 and 3 [24].
2.3 Analytical Studies

2.3.1 KURAMA et al.

The flexural behavior of single unbonded post-tensioned concrete walls with the horizontal joints was investigated analytically at Lehigh University (Kurama et al.) [6, 14, 15]. The analytical study used a beam-column fiber element available in the DRAIN-2DX program (Prakash and Powell 1993) to model the axial flexural behavior of an unbonded post-tensioned precast wall. Based upon the numerous simulations (nearly 98) of the wall model under seismic loading and cyclic loading, Kurama et al. proposed various limit states to represent the lateral behavior of the unbonded post-tensioned walls [see Figure 2.11]. The first of these states is the Decompression State, which is the point when gap opening would initiate in the horizontal joint at the wall base. The second state is the Softening State. From this state on, noticeable reduction to the lateral stiffness of the wall would be observed. This state is followed by the Yielding State, which defines the point when the strain in the post-tensioning steel reaches the yield strain. The final state is the Failure State, a point at which the flexural failure of the wall occurs due to crushing of confined concrete at the wall toe regions. Based on the various simulations, the following conclusions were drawn:
1. The nonlinear elastic behavior of the post-tensioning tendons provides self-centering capability for the post-tensioned precast walls.

2. The nonlinear displacements occur primarily due to gap opening along the horizontal joints.

3. The lateral load behavior of the unbonded post-tensioned walls can be represented by a tri-linear curve, joining the various wall limit states defined above.

4. From the analysis, it was found that the unbonded post-tensioned wall had larger displacements under seismic loading when compared to a normal monolithic concrete wall, but the post-tensioned wall had small residual displacement.

5. The unbonded post-tensioned walls can be analyzed by using the fiber beam-column element available in the DRAIN-2DX program.

---

Figure 2.12  Wall limit states proposed by Kurama et al. [14, 15]
2.3.2 PRESSS Analysis Procedure

Based on the performance of the PRESSS building, Stanton and Nakaki established a design procedure for jointed precast walls [10]. By reversing this design procedure, the PRESSS analysis procedure was derived by Thomas (2003). The steps involved in this analysis procedure can be summarized as follows:

**Step 1: Define wall dimensions and material properties.** This includes: grout strength \( f'_g \), concrete strength \( f'_c \), concrete density \( \gamma_c \), modulus of elasticity of post-tensioning steel \( E_p \), yield strength of post-tensioning steel \( f_{py} \), area of post-tensioning steel \( A_p \), initial stress in post-tensioning steel \( f_{po} \), unbonded length of post-tensioning steel \( h_u \), number of panels \( n \), height of each wall \( h_w \), length of wall panel \( L_w \), thickness of wall \( t_w \), and connector force-displacement relationship to determine the force in connector \( F_{sc} \).

**Step 2: Select a base rotation \( \theta \)**
Select a value for \( \theta \) between 0 and 0.03.

**Step 3: Determine various parameters:**

- Calculate the increase in stress in the post-tensioning tendon between zero base rotation and the selected base rotation assuming the wall rocked about its corner:

\[
\Delta f_{pc} = 0.5E_p \theta \frac{L_w}{h_u}
\]  

(2.1)

- Calculate the self-weight of each wall:

\[
W_{panel} = L_w h_w t_w \gamma_c
\]  

(2.2)

- Determine the total gravity load on one wall panel:

\[
W = W_{panel} + L_w W_{floor}
\]  

(2.3)

where, \( W_{floor} \) is the distributed vertical load on the wall from all floors.
• Calculate the compression capacity of one wall panel:

\[ C_c = L_w t_w (k_1 f_g') \]  \hspace{1cm} (2.4)

where, \( k_1 \) is the uniform stress in the equivalent rectangular stress block divided by \( f_g' \).

**Step 4:** Assume force \( P \) in the post-tensioning tendon at the selected base rotation.

**Step 5:** Determine forces at base rotation (\( \theta \)), as illustrated in Figure 2.13.

• Calculate the total tension force:

\[ N = P + W \]  \hspace{1cm} (2.5)

• Calculate the compressive force:

\[ C = N \pm F_{sc} \]  \hspace{1cm} (2.6)

where, \( F_{sc} \) is the force in the UFP connectors determined using a suitable force-displacement response as discussed above. A value for \( F_{sc} \) should be assumed for the first iteration and the value from the previous iteration based on the wall end uplift may be used as a satisfactory initial value for the subsequent iterations.

• Calculate the distance from the compression face to the center of the compression force divided by the length of the wall \( (L_w) \):

\[ \alpha = 0.5 \frac{C}{C_c} \]  \hspace{1cm} (2.7)

\( (\alpha L_w \) defines the distance from the edge of the wall to the resultant compression force.)

• Calculate the distance from the compression face to neutral axis depth divided by the length of the wall \( (L_w) \):

\[ \eta = \frac{f_g}{f_g'} \]  \hspace{1cm} (2.8)

\( (\eta L_w \) defines the neutral axis depth)

• Calculate the wall end uplift:

\[ \Delta_{udlift} = \theta L_w (1 - \eta) \]  \hspace{1cm} (2.9)
• Determine a new value for $F_{sc}$ based on the force-displacement curve for the connector.

• Calculate the elongation of the post-tensioning tendon:

$$\Delta_p = \theta L_w(0.5 - \eta)$$  \hspace{1cm} (2.10)

• Calculate the increase in stress in the post-tensioning tendon:

$$\Delta f_p = E_p \frac{\Delta p}{h_{tu}}$$  \hspace{1cm} (2.11)

• Calculate the total stress in the post-tensioning tendon:

$$f_p = f_{po} + \Delta f_p \leq f_{py}$$  \hspace{1cm} (2.12)

• Recalculate the total post-tensioning force:

$$P = A_p f_p$$  \hspace{1cm} (2.13)

Iterate Step 5 until $P$ converges.

**Step 6:** Compute the resisting moment of the wall panel:

$$M_{wall} = L_w(C(0.5 - \alpha) + 0.5F_{sc})$$  \hspace{1cm} (2.14)

Steps 3 through 6 should then be repeated for each additional wall.

**Step 7:** Compute the resisting moment of the entire wall system:

$$M_{wall,system} = \sum_{i=1}^{n} M_{wall}$$  \hspace{1cm} (2.15)
2.3.3 Analysis Procedure Based on MBA

2.3.3.1 Background

To overcome the strain incompatibility at the section level, Pampanin et al. (2001) [25] proposed the monolithic beam analogy concept to analyze precast frame systems with jointed connections having unbonded reinforcement. This method was extended and investigated for the jointed wall systems by Thomas & Sridharan (2003) [18]. In this method, a simple relationship between the extreme concrete fiber strain, neutral axis depth (c), and the base rotation (θ) was established by setting the total displacement of the jointed precast wall equal to the total displacement of an equivalent monolithic wall. Accordingly,

\[
\theta = (\phi_u - \phi_c)L_p = \left(\frac{\varepsilon_{c,ext}}{c} - \phi_c\right)L_p
\]  

(2.16)
where, \( \varepsilon_{c,ext} \) is the extreme fiber concrete strain at the critical section. Thomas & Sritharan found that the plastic hinge length \( (L_p) \) equal to 0.06 \( h_w \) gave good prediction of the observed base moment vs. lateral displacement response and the elongation of the prestressing steel as a function of the lateral displacement for the PRESSS wall system [18]. Hence, the above equation can be expressed in the following form.

\[
\varepsilon_{c,ext} = c(\phi_e + \frac{\theta}{0.06h_w}) \quad \text{where} \quad \phi_e = \frac{M}{E_c I_{eff}}
\]  

(2.17)

where \( \phi_e \) is the wall elastic curvature and \( M \) is the base moment at base rotation \( \theta \).

### 2.3.3.2 Analysis Procedure

The steps involved in the MBA analysis procedure as suggested by Thomas & Sritharan can be summarized as follows [18]:

**Step 1:** Define wall dimensions and material properties. This includes: grout strength \( (f_g') \), concrete strength \( (f_c') \), concrete density \( (\gamma_c) \), modulus of elasticity of post-tensioning steel \( (E_p) \), yield strength of post-tensioning steel \( (f_{py}) \), area of post-tensioning steel \( (A_p) \), initial stress in post-tensioning steel \( (f_{pc}) \), unbonded length of post-tensioning steel \( (h_u) \), number of panels \( (n) \), height of each wall \( (h_w) \), length of wall panel \( (L_w) \), thickness of wall \( (t_w) \), and connector force-displacement relationship to determine the force in connector \( (F_{sc}) \)

**Step 2:** Calculate moment resistance of each wall at the decompression point:

\[
M_{dec} = \frac{(P_o + W) I_g}{0.5 t_w L_w^2}
\]

(2.18)

where,

\[
W = \gamma_c L_w t_w h_w + L_w W_{floor}.
\]

\[
I_g = I_{eff} = \frac{t_w L_w^3}{12}
\]

**Step 3:** Select a base rotation \( \theta \), where

\[ 0 \leq \theta \leq 0.03 \]
Step 4: Assume a neutral axis depth \((c)\) for a selected base rotation \(\theta\).

Step 5: Calculate forces corresponding to base rotation \((\theta)\) and neutral axis depth \((c)\) assuring that the equilibrium condition is met. Utilizing the wall geometry,

- Calculate the tendon elongation:

\[
\Delta_p = \theta(0.5L_w - c)
\]  

(2.19)

- Calculate the increase in tendon stress:

\[
\Delta f_p = \frac{E_p \Delta p}{h_w}
\]  

(2.20)

- Calculate the total post-tensioning force \((P)\) and the total tension force \((N)\) under the current base rotation and assumed neutral axis depth:

\[
P = P_o + A_p \Delta f_p
\]  

(2.21)

\[
N = P + W
\]  

(2.22)

Step 6: Determine the force contribution of the connectors using the force-displacement curve. In this approach, the relative vertical displacement between the two adjacent walls is approximated to the wall-end uplift, which is estimated using the geometry of the wall:

\[
\Delta_{endlift} = \theta(L_w - c)
\]  

(2.23)

For a given wall end uplift, assume the force in the connector is \(F_{sc}\). The compressive force \((C)\) can be determined from the equilibrium condition at the wall base in the vertical direction:

For extreme wall panels,

\[
C = N \pm F_{sc}
\]  

(2.24)

For intermediate wall panels,

\[
C = N + F_{sc1} - F_{sc2}
\]
where \( F_{sc} = N_{con} F_{sc0} \), \( N_{con} \) is the total number of connectors in the vertical joint.

**Step 7:** Determine the extreme fiber concrete strain for the assumed neutral axis depth \( c \):

\[
\varepsilon_{c,ext} = c \left( \frac{M}{E_e I_{eff}} + \frac{\theta}{0.06 h_w} \right)
\]
(2.25)

where \( M \) is the base moment resistance of the wall panel, \( E_e \) is the modulus of elasticity of concrete, and \( I_{eff} \) is the effective moment of inertia of the wall.

**Step 8:** Using a confined concrete model, calculate the resultant compression force and its location. The confined concrete model suggested by Mander et al (1988), was selected by Thomas & Sritharan to define the stress-strain curves for confined and unconfined concrete. According to this model, the stress-strain relationship of the confined concrete is as follows:

\[
f_c = \frac{f'_{c,eff}}{r - 1 + x^r}
\]
(2.26)

\[
f'_{cc} = f'_c (2.254 \sqrt{1 + \frac{7.94 f''_{f}}{f'_c} - 2 \frac{f''_{f}}{f'_c} - 1.254})
\]
(2.27)

\[
\varepsilon_{cc} = 0.002 (1 + 5 \frac{f'_{cc}}{f'_c} - 1))
\]
(2.28)

where

\[
x = \frac{\varepsilon_c}{\varepsilon_{cc}} \quad r = \frac{E_c}{E_c - E_{sec}} \quad \text{and} \quad E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}
\]
(2.29)

\( f'_{cc} \) is the peak confined concrete strength

\( \varepsilon_{cc} \) strain corresponding to \( f'_{cc} \) and

\( f'_{f} \) is the effective lateral confining stress. While assumed equal to zero for unconfined regions, \( f'_{f} \) is the sum of \( f'_{lx} = k_f \rho_x f_{yh} \) and \( f'_{ly} = k_f \rho_y f_{yh} \) for rectangular confined regions. \( \rho_x \) and \( \rho_y \) are the transverse reinforcement area ratios in the principal directions.

\( K_f \) is the confinement effectiveness coefficient and 0.6 is recommended for rectangular wall sections.
By integrating the stress-strain profile (using the numerical techniques), determine the area \((A_{conc})\) under the stress-strain curve up to \(\varepsilon_{c,ext}\). The resultant compression force can be found by: \(C_{conf} = t_w A_{conc}\). In this procedure, the location of the resultant force \((C_{yi})\) can be found out. If the resultant compressive force \((C_{conf})\) is not equal to the compressive force established by equilibrium \((C)\), then the neutral axis depth is changed and Steps 5 through 7 are repeated until the two forces converge.

**Step 9:** Compute the resisting moment of each wall by taking moment about the corner of each wall. The process should be iterated once more to ensure that the MBA (Step 6) utilizes an accurate moment in computing the extreme fiber concrete strain.

For intermediate walls,

\[
M_{wall} = C(0.5L_w - C_y) + 0.5(F_{sc,r} + F_{sc,r+1})L_w
\]  

(2.30)

For end walls,

\[
M_{wall} = C(0.5L_w - C_y) + 0.5F_{sc,r}L_w
\]

**Step 10:** Compute the total moment capacity of the wall system:

\[
M_{rap,wall} = \sum_{i=1}^{n} M_{rap,panel}
\]  

(2.31)
2.3.4 Modified PRESSS Analysis Procedure

By evaluating the PRESSS analysis procedure, Thomas & Sritharan found that the analysis procedure based on the PRESSS guidelines predicted the base moment-displacement response of the jointed wall in the PRESSS building satisfactorily. However, this procedure significantly underestimated the prestressing steel elongation in the leading wall by overestimating the neutral axis depth. To address this shortcoming, Thomas & Sritharan proposed modifications to the PRESSS analysis procedure, which can accurately predict the prestressing steel elongation as well as the lateral load behavior of the wall system.

The modifications proposed for the analysis procedure are as follows [18]:

- Use a constant neutral axis depth (determined at 2% base rotation using PRESSS analysis) for analysis at all base rotations from 0 to 3%.

- The connector forces on either side of the intermediate walls should not be assumed to be equal.

- To take the confinement effect into consideration, the concrete strength ($f'_c$) is multiplied by a factor of 1.6 along with a $\beta$ value of 0.85. No reduction for $\beta$ was suggested for high strength concrete.

By incorporating the proposed modifications into the PRESSS analysis procedure, the following procedure was presented.

**Step 1:** Define wall dimensions and material properties. This includes: grout strength ($f'_g$), concrete strength ($f'_c$), concrete density ($\gamma_c$), modulus of elasticity of post-tensioning steel ($E_p$), yield strength of post-tensioning steel ($f_{py}$), area of post-tensioning steel ($A_p$), initial stress in post-tensioning steel ($f_{po}$), unbonded length of post-tensioning steel ($h_u$), number of panels ($n$), height of each wall ($h_w$), length of wall panel ($L_w$), thickness of wall ($t_w$), and connector force-displacement relationship to determine the force in connector ($F_{wc}$).

**Step 2:** Determine parameters:
- Calculate the increase in stress in the post-tensioning tendon between zero base rotation and the selected base rotation assuming the wall rocked about its corner:

\[ \Delta f_{poc} = 0.5E_p \theta \frac{L_w}{h_u} \]  

(2.32)

- Calculate the self-weight of each wall:

\[ W_{panel} = L_w h_w t_w \gamma_c \]  

(2.33)

- Determine the total gravity load on one wall panel:

\[ W = W_{panel} + L_w W_{floor} \]  

(2.34)

where, \( W_{floor} \) is the distributed vertical load on the wall from all floors.

- Calculate the compression capacity of one wall panel:

\[ C_v = L_w t_w (k_1 f'_g) \]  

(2.35)

where, \( k_1 \) is the uniform stress in the equivalent rectangular stress block divided by \( f'_g \).

**Step 3:** Determine forces at base rotation \( \theta = 0.02 \), as illustrated in Figure 2.13.

- Assume force \( P \) in the prestressing steel.

- Calculate the total tension force:

\[ N = P + W \]  

(2.36)

- Calculate the compressive force:

\[ C = N \pm F_{sc} \]  

(2.37)

where, \( F_{sc} \) is the force in the UFP connectors determined using a suitable force-displacement response as discussed above. A value for \( F_{sc} \) should be assumed for the first iteration and the value from the previous iteration based on the wall end uplift may be used as a satisfactory initial value for the subsequent iterations.
• Calculate the distance from the compression face to the center of the compression force divided by the length of the wall \( L_w \):

\[
\alpha = 0.5 \frac{C}{C_c}
\]  

(\( \alpha L_w \) defines the distance from the edge of the wall to the resultant compression force.)

• Calculate the distance from the compression face to neutral axis depth divided by the length of the wall \( L_w \):

\[
\eta = 2 \frac{\alpha}{\beta_1}
\]  

(\( \eta L_w \) defines the neutral axis depth)

• Calculate the wall end uplift:

\[
\Delta_{endlift} = \theta L_w (1 - \eta)
\]  

• Determine a new value for \( F_{sc} \) based on the force-displacement curve for the connector.

• Calculate the elongation of the post-tensioning tendon:

\[
\Delta_p = \theta L_w (0.5 - \eta)
\]  

• Calculate the increase in stress in the post-tensioning tendon:

\[
\Delta f_p = E_p \frac{\Delta_p}{\beta_n}
\]  

• Calculate the total stress in the post-tensioning tendon:

\[
f_p = f_{po} + \Delta f_p \leq f_{pj}
\]  

• Recalculate the total post-tensioning force:

\[
P = A_p f_p
\]  

Iterate all sub steps presented above until the post-tensioned force \( P \) is converged.

• Calculate the neutral axis depth \( c : L_w \eta \)
Step 4: Select a base rotation $\theta$ between 0 and 0.03. Determine forces at base rotation ($\theta$), as illustrated in Figure 2.12.

- Calculate the elongation of the post-tensioning tendon:
  \[
  \Delta_p = \theta (0.5L_w - c)
  \]  
  (2.45)

- Calculate the increase in stress in the post-tensioning tendon:
  \[
  \Delta f_p = E_p \frac{\Delta_p}{h_u}
  \]  
  (2.46)

- Calculate the total stress in the post-tensioning tendon:
  \[
  f_p = f_{po} + \Delta f_p
  \]  
  (2.47)

- Total post-tensioning force:
  \[
  P = A_p f_p
  \]  
  (2.48)

- Calculate the total tension force:
  \[
  N \approx P + W
  \]  
  (2.49)

- Calculate the compressive force:
  For the leading wall,
  \[
  C = N + F_{sc}
  \]  
  (2.50)

For the trailing wall,
  \[
  C = N - F_{sc}
  \]

where, $F_{sc}$ is the force in the UFP connectors determined based on the wall end uplift.

Wall end uplift is given by
\[
\Delta_{endlift} = \theta (L_w - c)
\]  
(2.51)
**Step 5:** Compute the resisting moment of the wall by taking moment about the center of each wall.

For intermediate walls,

\[ M_{wall} = C(0.5L_w - 0.425c) + 0.5(F_{sc,r} + F_{sc,r+1})L_w \]  \hspace{1cm} (2.52)

For end walls,

\[ M_{wall} = C(0.5L_w - 0.425c) + 0.5F_{sc,r}L_w \]

**Step 6:** Compute the total moment capacity of wall system:

\[ M_{wall, system} = \sum_{1}^{n} M_{wall} \]  \hspace{1cm} (2.53)
2.3.5 Perez et al.

Perez et al. [26] proposed a tri-linear idealization for the lateral load response of an unbonded post-tensioned precast concrete wall with vertical joints and ductile connectors. This analysis procedure provides the close form solution to estimate the wall resistance at the three limit states defined by Kurama et al. [see section 2.3.1]. The following assumptions are made for arriving at this procedure:

- Wall panels undergo only in-plane flexural, shear, and axial deformations. Torsion and out-of-plane deformations are not considered.
- Seismic forces at each floor and at the roof level are transferred to wall panels by floor and the rigid roof diaphragms.
- All wall panels undergo the same displacements at the floor and roof levels due to the rigid floor and roof diaphragm assumption.
- Vertical joint connectors behave in elastic-perfectly plastic manner, with sufficient ductility to be functional during seismic events.
- Walls are braced adequately from them experiencing out-of-plane buckling.

**Base Shear Capacity at Softening State, $V_{dl}$:**

- It has been shown that, unbonded post-tensioned single walls reach the effective linear limit state when the base moment is between $2M_{dec}$ and $3M_{dec}$, where $M_{dec}$ is the base moment capacity at the decompression state.

- Assuming Softening State occurs when the moment is $2.5M_{dec}$, and also, assuming a linear stress distribution along the compression region of the single wall, the length of the compression region will be $0.25L_x$.

- Assuming the lengths of compression regions of the middle panels and the average length of compression regions of exterior panels are $0.25L_x$. 
• Taking the moments about point $O$ in Figure 2.13, base shear capacity at the Softening Point is given by,

$$V_{ell} = \frac{A_{ell} + B_{ell} + \sum_{k=2}^{n-1} C_{ell,k}}{H_w (\sum_{i=1}^{r} r_{HF_i})}$$  \hspace{1cm} (2.54)

$$A_{ell} = -T_1 \left( \frac{L_x}{2} - e_p \right) - T_2 \left( \frac{L_x}{2} + e_p \right) - N_1 (L_x - e_{N1}) + C_1 (L_x - \frac{C_1}{3})$$  \hspace{1cm} (2.55)

$$B_{ell} = -T_1 \left( \frac{L_x}{2} - e_p + (n-1)L_x \right) - T_2 \left( \frac{L_x}{2} + e_p + (n-1)L_x \right) - N_n (nL_x - e_{Nn}) - C_n (nL_x - \frac{C_n}{3})$$  \hspace{1cm} (2.56)

$$C_{ell} = -T_1 \left( \frac{L_x}{2} - e_p + (k-1)L_x \right) - T_2 \left( \frac{L_x}{2} + e_p + (k-1)L_x \right) - N_n (kL_x - e_{Nn}) - C_n (kL_x - \frac{C_n}{3})$$  \hspace{1cm} (2.57)

$$c_1 = \frac{L_x \sqrt{C_1}}{2(\sqrt{C_1} + \sqrt{C_n})}, c_n = \frac{L_x \sqrt{C_n}}{2(\sqrt{C_1} + \sqrt{C_n})}, c_k = \frac{L_x \sqrt{C_k}}{2(\sqrt{C_1} + \sqrt{C_n})}$$  \hspace{1cm} (2.58)

And the corresponding roof displacement is given by, $\Delta_{ell} = \Delta_{Fr} + \Delta_{Sr} + \Delta_{Nr} + \Delta_{Pr}$, where,

$$\Delta_{Sr} = \sum_{i=1}^{r} \frac{1}{E_{c}A_w} r_{F_i} A_{ell} r_{HF_i} h_{h}$$  \hspace{1cm} (2.59)

$$\Delta_{Fr} = \sum_{i=1}^{r} \frac{1}{2E_{c}A_w} r_{F_i} V_{ell} r_{HF_i} h_{h}^2 \left[ r_{HF_i} - \frac{r_{HF_i}}{3} \right]$$

$$\Delta_{Nr} = \sum_{i=1}^{r} \frac{1}{E_{c}A_w} M_{w,i} r_{HF_i} h_{h}^2 \left[ r_{HF_i} - \frac{r_{HF_i}}{3} \right]$$

$$\Delta_{Pr} = \frac{n \rho [T_2 - T_1] h_{h}^2}{2E_{c}A_w}$$

**Base Shear Capacity at Yielding State, $V_{lyp}$:**

• It was found from the fiber model analysis, the compression force acts at a distance equal to $\frac{L_x}{30}$ from the edge of the wall panel.

• It is assumed that the further most post-tensioning tendon reaches yield.

• Thus, the base shear capacity and the corresponding roof displacement are given by,

$$V_{lyp} = \frac{A_{lyp} + B_{lyp} + \sum_{k=2}^{n-1} C_{lyp,k}}{H_w (\sum_{i=1}^{r} r_{HF_i})}$$  \hspace{1cm} (2.60)

where,

$$A_{lyp} = -T_1 \left( \frac{L_x}{2} - e_p \right) - T_2 \left( \frac{L_x}{2} + e_p \right) - N_1 (L_x - e_{N1}) + C_1 (L_x - \frac{L_x}{30})$$  \hspace{1cm} (2.61)

$$B_{lyp} = -T_1 \left( \frac{L_x}{2} - e_p + (n-1)L_x \right) - T_2 \left( \frac{L_x}{2} + e_p + (n-1)L_x \right) - N_n (nL_x - e_{Nn}) + C_n (nL_x - \frac{L_x}{30})$$  \hspace{1cm} (2.62)

$$C_{lyp} = -T_1 \left( \frac{L_x}{2} - e_p + (k-1)L_x \right) - T_2 \left( \frac{L_x}{2} + e_p + (k-1)L_x \right) - N_n (kL_x - e_{Nn}) + C_n (kL_x - \frac{L_x}{30})$$  \hspace{1cm} (2.63)

Corresponding roof displacement is given by $\Delta_{lyp} = \Delta_{ll} + \Delta_{gy}$, where

$$\Delta_{gy} = \frac{2H_n^2 (f_{pl} - f_{py})}{2(L_x + 2e_p)}$$  \hspace{1cm} (2.64)

$\Delta_{ll}$ is the roof displacement at Softening Point.
2.4 Design Methods

2.4.1 Galusha

*Galusha* presented the procedure that was used for the design of the jointed wall system in the PRESSS test building [12]. A summary of this design procedure is as follows:

**Step 1:**

- Select the wall system configuration: wall height \(H_w\), entire wall system length \(B\), and number of panels \(n\).
- Select a value for \(\alpha\), the re-centering coefficient, within the suggested range of 1.0-1.2
- Establish material properties: i.e. modulus of elasticity of post-tensioning steel \(E_p\), yield stress of post-tensioning steel \(f_{py}\), and concrete compressive strength \(f'_c\).
- Calculate the width of each wall panel, \(L_w = \frac{B}{n}\).
Step 2:

- Specify the building data: number of stories \( n_{\text{story}} \), height per story \( H_{\text{story}} \), building weight per floor \( W_{\text{story}} \), and design rotation \( \theta_{\text{design}} \) in order to carry out the displacement-based design (DBD). Note that the PRESSS wall system was designed using DBD.

- Use DBD to establish the seismic design loads and thus calculate the design base shear \( V_b \) and design overturning moment \( M_{\text{ot}} \).

Step 3:

- Select an estimate for the moment arm reduction coefficient \( \beta \), within the suggested range of 0.9-1.0. (This accounts for the fact that the rocking of wall does not occur at the corner but rather some distance in from the corner due to the crushing of the concrete at the corner and the underlying grout.)

- Calculate the initial post-tensioning stress:
  \[
  f_{po} = f_{pu} - \left( \frac{b_3}{2} \theta_{\text{design}} \right) \frac{E_p}{H_t} \tag{2.65}
  \]

- Assume a wall thickness \( t \) in order to calculate the panel self weight \( W \).

Step 4:

- Calculate the initial post-tensioning force in the wall:
  \[
  P_{po} = \frac{M_{\text{ot}} - \frac{nWb_3(1+\alpha)}{2n}}{\frac{n\theta(1+\alpha)}{2n} + \frac{nE_p}{f_{pu}H_t} \left( \frac{b_3}{2} \right)^2 \theta_{\text{design}}} \tag{2.66}
  \]

- Determine the area of post-tensioning steel:
  \[
  A_p = \frac{P_{po}}{f_{po}} \tag{2.67}
  \]

- Calculate the interface shear force anticipated between the walls in the vertical direction:
  \[
  F = \frac{nP_{\text{tot}} \beta}{\alpha(2n - 2)} \tag{2.68}
  \]

Step 5:
• Calculate the lift-off moment \( (M_{io}) \), net righting moment \( (M_{r, net}) \) and the nominal moment \( (M_n) \):

\[
M_{io} = \frac{n P_{tot} b \beta}{2} + (n - 1)bF
\]

Note that in lift-off moment, the connector forces are coming into picture, as it was assumed in the procedure that the connectors have rigid-plastic behavior.

\[
M_{r, net} = \frac{n P_{tot} b \beta}{2} - (n - 1)bF \tag{2.70}
\]

\[
M_n = M_{ot} \tag{2.71}
\]

• Calculate the damping of the system \( (\zeta) \) and compare it with the estimate used in the DBD method.

If the inter-panel connectors are frictional devices:

\[
\zeta_{friction} = \left(\frac{2}{\pi}\right) \frac{2 \theta_{desig}(M_{io} - M_{r, net})}{4 \theta_{desig} M_n} = \frac{(M_{io} - M_{r, net})}{\pi M_n} \tag{2.72}
\]

If the inter-panel connectors are U-shaped flexure plates as used in the PRESSS building:

\[
\zeta_{UPP} = 0.625 \zeta_{friction} \tag{2.73}
\]

**Step 6:**

• Check the wall thickness for shear and other code requirements. Iteration may be necessary with the assumed thickness in Step 3.

• Estimate the \( \beta \) value from the analysis and check if \( \beta \) is equal to that assumed in Step 3. Iterate with \( \beta \) value, if necessary.

2.4.2 Stanton & Nakaki

Upon completion of seismic testing of the PRESSS building (see Section 2.2.2), Stanton and Nakaki published a set of design guidelines for the design of jointed walls [10], which were based on the design procedure presented by Galusha. The proposed guidelines used the following assumptions:

• The design forces and drift limits are known, which are usually selected to satisfy the code requirements.

• The total wall length \( (L_{w,tot}) \), wall height \( (h_w) \) and wall thickness \( (t_w) \) are known, which are generally obtained from architectural drawings and preliminary calculations.
• The shear connectors are assumed to have a rigid-plastic behavior.

• The wall panels are assumed to be identical and behave in a rigid manner.

• The post-tensioning steel reaches the yield strain at the design drift.

The design guidelines can be summarized as follows:

**Step 1:**
Establish the following material properties: strength ($f_{py}$) and modulus of elasticity of post-tensioning steel ($E_p$), strength of shear connectors, strength of concrete ($f'_c$), and strength of grout ($f'_g$).

**Step 2:**
Using either the Displacement-Based Design (DBD) or the Force-Based Design (FBD) method, determine the design base shear ($V_{des}$) and design drift ($\theta_{des}$).

**Step 3:**
Select the number of panels ($n$) using the following considerations: the wall panel aspect ratio ($\frac{L_w}{h_w}$), the post-tensioning tendon elongation, the lateral strength, and the damping ratio ($\zeta$).

**Step 4:**
Establish the following constants:

- Length of each wall:
  \[ L_w = \frac{L_{w,\text{tot}}}{n} \]  

  (2.74)

- Increase in prestressing in the post-tensioning tendon between zero drift and design drift,
  \[ \Delta f_{ps} = 0.5E_p\theta_{des} \frac{L_w}{h_n} \]  

  (2.75)

  where, $h_n$ is the unbonded length of the post-tensioning tendon.

- Design moment ($M_{des}$) is equal to $V_{des}h_{eff}$, where $V_{des}$ is the design base shear and $h_{eff}$ is the height above the foundation that the lateral load resultant acts on the wall.

- Panel weight ($W_{wall}$) is equal to $L_wh_w\gamma_c$, where $\gamma_c$ is the density of concrete.

- Total weight $W = W_{wall} + L_ww_{floor}$, where $w_{floor}$ is the vertically distributed weight of the floors on the wall panel.
• Calculate the compression capacity of wall: 
\[ C_c = L_w t_{w, eff} (k_1 f_y) \]
where \( k_1 \) is the uniform stress in the equivalent rectangular stress block divided by \( f_y \).

• Calculate the force in shear connectors,
\[ F_{sc, net} = F_{sc, left} - F_{sc, right} \]
(2.76)

where \( F_{sc, left} \) and \( F_{sc, right} \) are the total yield force of all shear connectors in the vertical joints on the left and right side of the wall panel, respectively. Note that \( F_{sc, net} = 0 \) is suggested for the intermediate walls.

**Step 5:**
Select the tendon reinforcement area \( (A_p) \) and initial prestressing stress \( (f_{po}) \).

**Step 6:**
Establish the condition which corresponds to the base of the wall starts to lift off (This condition is also referred to as the decompression point):

• initial force in the prestressing tendon: \( P_0 = A_p f_{po} \)
• total axial force on each wall: \( N_0 = P_0 + W \)
• compressive reaction on each wall: \( C_0 = N_0 + F_{sc, net} \)
• distance from the compression face of the wall to the compression force: \( (\alpha_0 l_w) \), where \( \alpha_0 = 0.5 \frac{f_y}{f_{po}} \)
• neutral axis depth \( (\eta_0 l_w) \), \( \eta_0 = 2 \frac{\alpha_0}{\beta_1} \), where \( \beta_1 \) is the depth of the equivalent stress block divided by the neutral axis depth.
• ratio of the design strength of the shear connectors to the vertical load \( \kappa_0 = \frac{F_{sc}}{N_0} \), where \( F_{sc} \) is the total yield force of all shear connectors in one vertical joint.

**Step 7:**
These same conditions can then be determined at the design drift \( (\theta_{des}) \) (see Figure 2.14) using an iterative method. Note that the difference between equations in Step 6 and equations in this step is the drift endured by the system and is denoted by '0' for zero drift and 'des' for the design drift. Assume a value \( P_{des} \) for the post-tensioning steel force at the design drift.

\[ N_{des} = P_{des} + W \]  
(2.77)
\[ C_{des} = N_{des} + F_{nc,sel} \]
\[ \alpha_{des} = 0.5 \frac{C_{des}}{C} \]
\[ \eta_{des} = 2 \frac{\alpha_{des}}{\beta_4} \]

- The post-tensioning elongation (\( \Delta_p \)) at the design drift is given by

\[ \Delta_p = \theta_{des} \{ 0.5 - \eta_{des} \} \]  
(2.78)

- Increase in stress between zero drift and the design drift (\( \Delta f_p \)) is obtained from

\[ E_p \frac{\Delta_p}{h_u} = \Delta f_p \{ 1 - 2\eta_{des} \} \]  
(2.79)

- Therefore, the total stress (\( f_{p,des} \)) can be determined from

\[ f_{p,des} = f_{p0} + \Delta f_p \leq f_y \]  
(2.80)

where, \( f_y \) is the yield strength of the post-tensioning tendon. This condition is needed to assure the recentering of the wall.

- The force in the post-tensioning tendon at the design drift can then be determined from

\[ P_{des} = A_p f_{p,des} \]  
(2.81)

This step (7) should be iterated until \( P_{des} \) converges.

**Step 8:**

Using the design level conditions compute the moment capacity for an individual wall (\( M_{cap,panel} \)):

\[ M_{cap,panel} = L_w (C(0.5 - \alpha) + 0.5 F_{sc}) \]  
(2.82)

- Calculate the total moment resistance of the wall system (\( M_{cap,wall} \)). Each wall must be designed using steps one through seven and then the moment capacities of the walls can be summed together to develop the total moment capacity which should be greater than \( \frac{M_{cap,wall}}{M_{cap,wall}} \). Hence,

\[ M_{cap,wall} = \sum_{i=1}^{n} M_{cap,panel} \]  
(2.83)

**Step 9:** Finally, ensure that the system meets the following additional criteria:

- Check the demand/capacity ratio for overturning moment on the panel.

\[ \rho_{MOM} = \frac{M_{des}}{M_{cap,wall}} \leq 1.0 \]  
(2.84)
Figure 2.15 Locations of forces in an unbonded post-tensioned jointed wall system at design drift [10]

- Check the stress ratio to ensure that the prestressing tendon does not yield at the maximum drift

\[ \rho_{f_{po}} = \frac{f_{po}}{f_{yp} - \Delta f_p} \leq 1.0 \]  

(2.85)

- Check the ratio of uplift force to the hold down force on each wall to prevent the uplifting of the wall system.

\[ \rho_{UPL} = \kappa_o \leq 1.0 \]  

(2.86)

- Check the parameter ratio controlling the residual drift

\[ \rho_{ZRD} = \frac{\kappa_o (n - 1 + 2\alpha_{a,ave}, \kappa_o)}{n(0.5 - \alpha_{a,ave})} \leq 1.0 \]  

(2.87)

- Check the force ratio to ensure that the panel slides rather than rocks

\[ \rho_{ROC} = \frac{\kappa_o L_{w}}{\mu h_{eff}} \left( (0.5 - \alpha_{a,ave}) + \frac{n - 1 + 2\alpha_{a,ave}}{n} \right) \leq 1.0 \]  

(2.88)
2.4.3 Design Procedure by Perez et al.

Perez et al. [21, 26] proposed a performance-based seismic design procedure which allows a designer to specify and predict the performance of an unbonded post-tensioned precast wall system under a selected seismic force. Consequently, this procedure requires the identification of seismic performance levels, building limit states and capacities, seismic input levels, and structure demand prior to conducting the wall design. Unlike the procedures discussed before, this procedure does not provide any equations to estimate the required post-tensioning steel area or the required prestressing force.

Seismic Performance Levels

Two seismic performance levels were identified to ensure satisfactory behavior of walls under seismic loading:

- **Immediate Occupancy**: post-earthquake damage state that ensures the building had suffered only limited structural and non-structural damage. The structure responds to the ground motion in an elastic manner with limited cracking and limited yielding of the structural members.

- **Collapse Prevention**: post-earthquake damage state that indicates the building is on the verge of partial or total collapse, but has not collapsed.

Structure limit states and capacities: These limit states describe the damage in various structural and non-structural elements of the building. The limit states for unbonded precast wall systems with vertical connectors were suggested to be the same as those proposed by Kurama et al. for unbonded single precast walls (see Section 2.3.1, Figure 2.12). To control the damage to non-structural members, a 2% inter-story limit is adopted according to the NEHRP recommended provision. The structural performance levels and structure limit states are presented graphically in Figure 2.16.

- The immediate occupancy performance level is assumed to have been reached when yielding of the post-tension steel occurs. That is, if the displacement response of the earthquake exceeds \( \Delta_{th} \), then the structure is likely to require repair before the building can be occupied.

- The collapse prevention level is assumed to have been reached when the crushing of the confined concrete occurs at the wall base.

Seismic Design Criteria

The recommended seismic design of the unbonded post-tensioned precast concrete jointed wall systems also has several design criteria that compare the estimated structural demands with the structural de-
sign capacities. These are described below.

**Criterion 1, Softening:**

This criterion controls the softening of an unbonded post-tensioned precast wall under lateral load, preventing premature reduction in the lateral stiffness of the wall. This is achieved by satisfying the following equation.

\[
V_{cll} \geq \alpha_d V_{des} = \alpha_d \frac{Q_{des}}{R}
\]  

where,

\( V_{cll} \) is the base shear capacity at softening limit state (which can be estimated using the analytical procedure by Perez et al.);

\( Q_{des} \) is the base shear demand for the design level ground motion;

\( R \) is the response modification factor; and

\( \alpha_d \) is the modification factor (a value of 0.65 is recommended for \( \alpha_d \) based on dynamic analysis results of the post-tensioned precast walls by Kurama et al.).

**Criterion 2, Base moment Capacity**

This criterion controls the base moment capacity of the wall. To satisfy this criterion,

\[
\phi_f V_{lp} \geq V_{des} = \frac{Q_{des}}{R}
\]  

Figure 2.16 The structural performance levels and the structure limit states [21, 26].
where \( \phi_f \) is the flexural reduction factor as defined by ACI 318.

**Criterion 3, Yielding of the post-tensioning steel**

This criterion controls the yielding of the post-tensioning steel and requires the following equation to be satisfied.

\[
\Delta_{\text{t}} \geq \Delta_{\text{d,e,s}}
\]

where

\( \Delta_{\text{t}} \) is the roof displacement at the yielding limit state, which can be estimated using the analysis procedure by Perez et al. [2.3.5], and

\( \Delta_{\text{d,e,s}} \) is the expected maximum roof displacement demand under the design level ground motion.

**Criterion 4, Gap closure at the base**

This design criterion controls the initial prestress of post-tensioning steel in the walls, to ensure that the gaps open at the wall bases close when the applied lateral loads are removed. The following equation was proposed to satisfy this criterion.

\[
\Phi_{\text{ge}}[T_1[0.5l_x - c_p] + T_2[0.5l_x + c_p]] + N[l_x - c_N] \geq P_{ij}l_x
\]

where, \( \Phi_{\text{ge}} \) is the initial prestress reduction factor.

**Criterion 5, Inter-story drift**

This design criterion controls the maximum story drift under design level ground motion to control the lateral stiffness of the walls.

\[
\Delta_{\text{all}} \geq \Delta_{\text{d,e,s}}
\]

where, \( \Delta_{\text{all}} \) is the maximum allowable story drift.

**Criterion 6, Crushing of confined concrete**

This design criterion controls the axial-flexural compression failure of the walls. The following equation may be used to satisfy this criterion.

\[
\Delta_{\text{rc}} \geq \Delta_{\text{max}}
\]

where, \( \Delta_{\text{rc}} \) is the displacement capacity at the roof level that corresponds to the crushing of the confined concrete,

\( \Delta_{\text{max}} \) is the displacement demand at roof level which is obtained for the maximum considered ground
motion, and

\[ \Delta_{ccc} \] can be obtained by performing nonlinear static push-over analysis of a fiber-based model of the wall system using the DRAIN-2DX program.

**Criterion 7, Fracture of the post-tensioning steel**

This criterion ensures that the fracture of the post-tensioning steel does not occur. Hence,

\[ \Delta_{fp} \geq \Delta_{ccc} \quad (2.94) \]

where \( \Delta_{fp} \) is the roof displacement corresponding to the fracture of the post-tensioning steel. **Other criteria**

It is required to prevent shear slip along the wall-to.foundation connections under the action of earthquake lateral loads by ensuring

\[ \phi_s V_{ss} \geq V_{max} \quad (2.95) \]

where, \( \phi_s \) is the shear capacity reduction factor as defined by the ACI 318 code [11], and

\( V_{max} \) is the expected maximum base shear demand under the survival-level ground motion.

This procedure is an conceptual design procedure and does not provide with any means to evaluate the various design parameters of jointed wall system to resist lateral loads. So, this procedure is not further considered in this report.
CHAPTER 3. ANALYSIS AND VALIDATION OF BEHAVIOR OF UNBONDED PRECAST WALL SYSTEMS

3.1 Introduction

The primary purpose of this study is to further modify the PRESSS analysis procedure (presented in section 2.3.2) so that the behavior of the unbonded precast wall systems can be predicted accurately. A successful effort to modify the PRESSS analysis procedure was made by Thomas & Sritharan (2003) by incorporating a constant neutral axis depth regardless of the base rotation and using an effective concrete confinement factor of 1.6 for defining the concrete strength [18]. This analysis procedure was validated against the jointed wall test data, from the PRESSS test building. Although the analysis procedure predicted the behavior of the jointed wall accurately, this procedure does not allow the variation of confined concrete strength to be modeled as a function of the amounts of confinement reinforcement. The proposed analysis in section 3.2 makes an attempt to address this problem. Furthermore, a trilinear approximation is used to account for the variation in neutral axis depth as a function of the wall base rotation. This concept was shown to be adequate for jointed hybrid frame system by Celik & Sritharan (2004) [27]. With these modifications, an analysis procedure for quantifying the behavior of single and jointed wall systems is presented in Section 3.2. Hereafter this procedure will be referred to as the simplified analysis procedure. In Section 3.3, validation of this analysis procedure is presented using test data from the jointed precast wall used in the PRESSS building and several single unbonded wall tests conducted at the ATLSS Research Center. An alternative analysis method based on “monolithic beam analogy (MBA)” (section 2.3.3), which uses a global displacement condition to overcome the strain incompatibility at a section level, is also examined using the same test data. Results from these two analysis procedures are combined to improve the design guidelines proposed for jointed wall systems in Chapter 4.
3.2 Simplified Analysis Procedure

To analyze a jointed wall system or a single wall with unbonded post-tensioning steel, an iterative procedure is suggested to find the neutral axis depth, for a given base rotation (2%). The force equilibrium and geometric compatibility conditions are used in the iteration process, which also leads to an estimate of the post-tension steel elongation. According to the design guidelines presented by Stanton & Nakaki, the following assumptions are made in the analysis procedure [10].

1. The walls are provided with adequate out-of-plane bracing, preventing them from experiencing torsional and out-of-plane deformations.

2. The dimensions and material properties of the walls and connectors are known.

3. The fiber grout pad located at the interface between the wall and the foundation does not experience any strength degradation.

4. All walls will undergo the same lateral deformation at every floor level due to the rigid floor assumption.

5. The wall base has enough friction resistance, such that the wall will not undergo any lateral movement at the base.

6. The connectors and the post-tension steel anchors remain fully effective for the entire analysis.

7. All vertical joints in a jointed wall system have the same number of connectors.

The analysis procedure is described in following steps.

Step 1: Define wall system dimensions, reinforcement details, and material properties.

The following variables are defined in this step.

Wall System Dimensions:

- $h_w =$ height of the wall system,
- $t_w =$ thickness of the wall system,
- $t_{cr} =$ confinement area thickness,
- $l_{wall} =$ length of the wall system,
- $n =$ number of walls in the jointed system, and
- $l_w (= \frac{l_{wall}}{n}) =$ length of each wall.

Prestressing/Post-tensioning Steel Details:

- $A_{pl} =$ area of a post-tensioning tendon,
$n_{pt} =$ number of post-tensioning tendons,

$h_u =$ unbonded length of the post-tensioning tendon,

$x_{pt,i} =$ location of the post-tension tendon from the rocking edge of the wall,

$E_p =$ modulus of elasticity of the post-tensioning tendon,

$f_{p0} =$ initial stress in the post-tensioning tendon,

$f_{py} =$ yield strength of the post-tensioning tendon, and

$A_{pt,\text{total}} (= n_{pt} A_{pt}) =$ total post-tensioning steel area in each wall.

**Confinement Details:**

$A_s =$ area of confinement steel,

$s =$ spacing of the confinement steel,

$l_{cr} =$ length of confinement area, and

$f_y =$ mild steel yield strength.

**Concrete Properties:**

$f'_c =$ concrete compressive strength,

$f_g =$ compressive strength of interface grout,

$f'_c =$ confined concrete strength,

$\varepsilon_{cc} =$ concrete strain at fcc,

$\varepsilon_{cu} =$ concrete crushing strain, and

$\rho_c =$ concrete density.

**Connector Details:**

$n_{con} =$ number of connectors per joint, and

$F_{sc} =$ force in the connector corresponding to a displacement (use an experimentally established force-displacement response of the connector to determine $F_{sc}$ as recommended by Thomas & Sritharan).

**Step 2: Decompression point**

In this step, the decompression point is found, which defines the beginning of a gap opening at the wall base, which corresponds to the condition when the stress in the extreme concrete fiber farthest from the rocking edge reaches zero. Assuming a linear strain distribution at the critical section, due to the moment induced by the decompression force $F_{decomp}$, the following equations are used to determine the corresponding moment resistance. The decompression moment ($M_{decomp}$) is calculated from the elastic flexural formula $\sigma = \frac{M}{I}$, substituting appropriate values for the neutral axis depth ($c$) and the moment of inertia ($I$):

$$M_{decomp} = \frac{\sigma I}{c}$$

(3.1)
\[ \sigma_i = \frac{\sum_{i=1}^{n_p} f_{pi} A_{pt}}{t_w l_w} \]  
\[ I = \frac{t_w l_w^3}{12} \]  
\[ c = \frac{l_w}{2} \]  

**Step 3: Neutral axis depth at 2\% base rotation**

The neutral axis depth that satisfies the vertical direction force equilibrium at the wall base is found for each wall through iteration with an assumed neutral axis depth as the initial value. The following sub-steps are used in this process.

1. Assume a neutral axis depth (c) for the wall.

2. Determine the total gravity load on each wall (N):

\[ N = \gamma_t l_w t_w h_w + w_{floor} l_w \]

where, \( w_{floor} \) is the uniform dead load acting on the wall.

3. Determine the stresses and strains in the post-tensioning tendon:

   - The elongation of the post-tensioning tendon:

\[ \Delta_{p,i} = 0.02 (x_{pi} - c) \]  

4. Determine the forces in the vertical connector

   - Calculate the wall end uplift from following equation:

\[ \Delta_{endlift} = 0.02 (l_w - c) \]  

It is assumed that the connector deformation is equal to \( \Delta_{endlift} \).
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- Determine the corresponding force in the vertical connector \( F_{sc} \) from the force-displacement response of the connector.

5. Determine the new neutral axis depth

Assuming a uniform compressive stress acting at the wall base over a length of \( \beta c \), where \( c \) is the neutral axis depth, the resultant compressive force

\[
C = \beta c f_{cv} t_{cr}
\]

(3.9)

where, \( \beta \) is a constant and varies from 0.8-0.9 depending on the concrete strength and the amount of confinement steel reinforcement provided. For this procedure an average value of 0.85 is assumed for \( \beta \).

- Calculate the resultant compressive force from equilibrium of forces

\[
C = P + N + n_{con} F_{sc}, \text{ for the leading wall}
\]

\[
= P + N + n_{con} F_{sc} - n_{con} F'_{sc}, \text{ for the intermediate wall, and}
\]

\[
= P + N - n_{con} F'_{sc}, \text{ for the trailing wall.}
\]

\( F'_{sc} \) is the force in the connector corresponding to the previous wall iteration at 2% base rotation.

- Calculate the neutral axis depth:

\[
c = \frac{C}{\beta f_{cv} t_{cr}}
\]

(3.10)

Iterate the above five sub-steps until \( c \) converges.

**Step 4:** Select a base rotation \((\theta)\). \((0 \leq \theta \leq \theta_{\text{ultimate}})\).

Choose any \( \theta \) value in the range of \( \theta \) and \( \theta_{\text{ultimate}} \).

**Step 5:** Determine the forces acting on the wall at base rotation \( \theta \) \( \text{[Figure B.2]} \)

1. Determine the neutral axis depth \( c_0 \) corresponding to base rotation \( \theta \):

Based on the experimental results, it was found that the neutral axis depth does not significantly vary for interface rotations above 0.5%, as illustrated in Figure 3.1. Consistent with this observation, the neutral axis depth in this analysis procedure is calculated at 2% base rotation (Step 3), which is then utilized for all rotations above 0.5%. To model the neutral axis depth more accurately at small \( \theta \), a trilinear variation of neutral axis depth is assumed as a function of base rotation as demonstrated in Figure 3.1. In this Figure, point 1 corresponds to the wall length
at 0% base rotation, where as points 2 and 3 are defined at base rotations of 0.1% and 0.5%, respectively. The neutral axis depth \( c \) at point 3 is found from Step 3 equal to neutral axis depth at 2% rotation, and at point 2 is approximated to 2\( c \).

![Diagram of neutral axis variation](image)

Figure 3.1 A trilinear idealization for the neutral axis depth as a function of base rotation for analysis of walls with unbonded tendons.

2. Determine stresses and strains in the post-tensioning steel:

- The elongation of the post-tensioning tendon:
  \[
  \Delta_{\mu,i} = 0.02(x_{\mu,t} - c)
  \]

- The total strain in the post-tensioning tendon:
  \[
  \frac{\Delta_{\mu,i}}{h_{\mu}} + \frac{f_{\mu t}}{E_p}
  \]

- Calculate the stress in each post-tensioning tendon \( f_{\mu} \), from the stress-strain curve of the post-tensioning steel.

  The above steps should be repeated to determine the stress in all post-tensioning tendons.

- The total post-tensioning force:
  \[
  P = \sum_{i=1}^{n_{\mu}} A_{\mu} f_{\mu}
  \]
Figure 3.2 Various forces acting on the jointed wall system at base rotation $\theta$. 
• Calculate the location of the resultant post-tensioning force from the rocking edge:

\[ X_{pt} = \sum_{p=1}^{n_p} \frac{A_{ptj} f_{pt,j}}{P} \]  

(3.14)

3. Determine the forces in the vertical connector

• Calculate the wall end uplift using following equation:

\[ \Delta_{endlift} = 0.02(l_w - \theta) \]  

(3.15)

It is assumed that the connector deformation is equal to \( \Delta_{endlift} \).

• Determine the corresponding force in the vertical connector (\( F_{sc} \)) from the force-displacement response of the connector.

4. Calculate the resultant compressive force from equilibrium of forces

\[ C = P + N + n_{con} F_{sc}, \]  

for the leading wall,

\[ C = P + N + n_{con} F_{sc} - n_{con} F'_{sc}, \]  

for the intermediate wall, and

\[ C = P + N - n_{con} F'_{sc}, \]  

for the trailing wall.

\( F'_{sc} \) is the force in the connector corresponding to the previous wall calculation at base rotation \( \theta \).

5. Calculate the location of the resultant compressive force at the maximum drift:

The resultant concrete compressive force acts at a distance of \( \alpha_{lever} \theta \) from the rocking edge of the wall. The lever arm factor at the maximum drift \( \alpha_{lever,max} \) depends on the concrete strain (\( \varepsilon_c \)) at the drift.

\[ \alpha_{lever,max} = 0.418 + 0.064 \ln \frac{\varepsilon_c}{\varepsilon_{cc}} \]  

(3.16)

\[ \varepsilon_c = \frac{1.05\varepsilon_{ultmax}}{0.06l_w} \]

where \( c \) is the neutral axis depth at 2% base rotation.

**Step 6:** Compute the resisting moment of the wall

Taking the moment about the center of the wall.

\[ M_{wall} = 0.5l_w F_{sc} + P(X_{pt} - 0.5l_w) + C(0.5l_w - \alpha_{lever} \theta) \]  

for the leading wall,

\[ = 0.5l_w (F_{sc} + F'_{sc}) + P(X_{pt} - 0.5l_w) + C(0.5l_w - \alpha_{lever} \theta) \]  

for the intermediate walls, and

\[ = 0.5l_w F'_{sc} + P(X_{pt} - 0.5l_w) + C(0.5l_w - \alpha_{lever} \theta) \]  

for the trailing wall.

It is assumed that the value of \( \alpha_{lever} \) varies linearly from 0.33 to \( \alpha_{lever,max} \) as the base rotation varies.
from zero to $\theta_{max}$.

Steps 2 through 6 should be repeated for each wall before going to step 7.

**Step 7:** Compute the resisting moment of the entire wall system

The moment capacity of the wall system is obtained by summing all the individual wall capacities. Thus,

$$M_{\text{wall system}} = \sum_{i=1}^{n} M_{\text{wall}}$$  \hspace{1cm} (3.17)

Though this procedure is developed for the jointed wall systems, it can also be used to predict the lateral load behavior of unbonded post-tensioned precast single walls. For the single wall analysis, the connector forces will be set equal to zero. This procedure can also be used to analyze jointed walls with different wall lengths. In this case, appropriate wall lengths should be used to estimate the neutral axis depth, connector forces, and wall base moments.

### 3.3 Validation of Analysis Procedures

This section compares the results obtained using the analytical method described in the previous section and those obtained using the MBA method (see Section 2.3.3) with experimental data from wall direction response of the PRESSS test building and test data reported by Perez et al. (2004) from their tests on single unbonded precast walls. For comparison purposes, the base moment vs. the top floor lateral displacement response envelopes, post-tensioning steel elongations, and the neutral axis depths at the wall base are used.

#### 3.3.1 PRESSS Jointed Wall

Following the PRESSS building testing, Thomas & Sridaran have conducted a thorough investigation to quantify the actual contribution of the jointed wall [18]. During this investigation, they found that the wall direction response of the PRESSS building was significantly influenced by framing action resulting from the seismic columns and precast floors in the bottom three stories. By isolating the framing action contribution, they arrived at the experimental base moment vs. displacement response envelope for the wall system using data points at selected measured lateral displacements. Six displacement transducers measured the vertical displacements of the leading and trailing walls at the base with respect to the wall foundation during testing. Using the data from these devices, the neutral axis depths ($c$) and the post-tensioning tendons elongations ($\Delta_p$) were also established for the leading and trailing walls at the selected lateral displacements. Also estimated from the displacement devices was
the wall end uplift ($\Delta_{enduplift}$) for the leading wall, which was assumed to be the same as the relative vertical displacement between the walls for the analysis procedure presented in previous section. By assuming a linear profile for the gap opening at the base, Thomas & Sritharan had presented change in post-tensioning force, post-tensioning elongation, the UFP connector displacement, and the neutral axis depth as a function of the top floor displacement for the PRESSS wall system. This information, along with the dimensions and properties reported by these researchers is used to verify the applicability of the simplified analysis method for the jointed wall.

3.3.1.1 Base Moment Resistance

Figure 3.3 compares the base moment vs the top lateral displacement established for the jointed wall with those calculated from the simplified analysis procedure and MBA. The figure also shows the base moment response predicted by the PRESSS analysis procedure with and without the modifications suggested by Thomas & Sritharan. It is seen that the simplified analysis procedure and the MBA provide a good estimate for the base moment vs lateral displacement response envelope. At the top floor displacement of 11.5 inch, the simplified analysis method is only 2.3% below and MBA is 5.7% below the experimental value. At the design drift of 2%, the simplified analysis procedure and MBA underestimated the moment resistance of the jointed wall by 2.2% and 5% respectively.

3.3.1.2 Neutral Axis Depth

The neutral axis depths calculated using the analytical methods for the PRESSS jointed wall system are compared with experimental data in Figures 3.4 and 3.5. It is seen from these figures that the simplified analysis procedure provides a good estimate for neutral axis depth for both leading and trailing walls. The difference between the analytical and extracted neutral axis depths exists, essentially cause the discrepancies observed between the calculated and experimental elongations of the post-tensioning tendons in Figures 3.6 and 3.7. More accurate estimation of the neutral axis depths will more accurately estimate the elongations in the post-tensioning tendons.

3.3.1.3 Elongation of Post-tensioning Steel

Figures 3.6 and 3.7 compare the calculated post-tensioning steel elongation with the experimental data for both the leading and trailing walls. The elongation at the maximum top floor displacement of 11.5-in (or 2.56% drift) in the leading wall is underestimated by 6.42% using the simplified procedure
Figure 3.3  Base moment vs top floor displacement response envelopes for the jointed wall in the PRESSS building.
Figure 3.4 Neutral axis depth for the leading wall of the PRESSS jointed wall system.

Figure 3.5 Neutral axis depth for the trailing wall of the PRESSS jointed wall system.
and by 9.9% using MBA method. Likewise, in the trailing wall the elongation is underestimated by 6% by the simplified procedure and 8.3% by MBA method.

![Graph showing Top Floor Displacement (mm) vs. Post-Tensioning Elongation (in)](image)

Figure 3.6 Post-tensioning steel elongation in the leading wall of the PRESSS jointed wall system.

### 3.3.2 Post-Tensioned Unbonded Single Walls (Perez et al.)

The test specimens of interest in this study are TW1, TW3, and TW5, which are described in detail in section 2.2.2 [21]. Recall that, TW1 was tested under monotonic loading while the other two walls were subjected to the full reversed cyclic lateral displacements. The load sequence of TW3 consisted of three cycles each at 0.05%, 0.1%, 0.25%, 1%, 1.5%, 2%, and 3%. The TW5 wall test consisted of the same load cycles as that for TW3, except that a few more cycles at 3.5%, 4%, 5%, 6% were added. The cyclic lateral displacement was applied using a hydraulic actuator connected to the top of the wall. A constant axial load was applied in the vertical direction through the center of the walls in all cases, subjecting the wall to a uniform compressive stress to simulate the gravity effects.

The lateral displacement of the loading block (which corresponded to the actual wall displacement), was measured using four displacement devices: string pots (LB SP-N, LB SP-S) and two LVDTs (LB LVDT-N, LB LVDT-S) were attached to the loading block on the north side and south side of the test wall, respectively. The rotation at the wall base was measured using a rotation meter (RMB), which
was positioned 9 inches above the base of the wall. A series of displacement instruments mounted 5 inches above the wall base measured the opening of the gap at wall-to-foundation interface.

Confined concrete strain gages were attached to No.4 bars running vertically within the confined portions of the bottom wall panel. Assuming strain compatibility between concrete and steel, the measured strain was considered to represent the confined concrete strains. Strain gauges were also attached to the post-tensioning steel to monitor the initial prestress as well as the variation resulting from the applied lateral load. The instrumentation for TW3 and TW5 was the same as described above.

The following section presents the comparisons between the experimental results and the analytical results obtained from the simplified analysis procedure and MBA. Included in this chapter are comparisons of the global response of the walls, such as the base shear vs. lateral drift response envelope and neutral axis depth variations with lateral drift. In addition, comparisons between the experimental and analytical estimates of strains in the post-tensioning bars are made to evaluate the analysis procedures. In all the cases, the wall analysis was performed using the properties summarized in Table 2.1.
3.3.2.1 Base Shear Response

The lateral load responses of walls TW1, TW3, and TW5 are presented by plotting the base shear vs. the lateral drift in Figures 3.8, 3.9, and 3.10, respectively. From these figures, it is clear that the simplified analysis procedure and MBA captured the wall response envelopes satisfactorily. In all cases, the initial stiffness and the ultimate shear capacity of the walls were calculated accurately. Table 3.1 summarizes the predicted and experimental base shear capacities of the walls by various procedures at the maximum drift. It is seen that the analysis methods overestimate the base shear capacities of walls TW3 and TW5 by 5.5% and 3.27% respectively. This discrepancy is caused by the slight overestimation of the stress in the post-tensioning steel [e.g. see Figure 3.16].

<table>
<thead>
<tr>
<th></th>
<th>TW1</th>
<th>TW3</th>
<th>TW5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>161 kip</td>
<td>154.33 kip</td>
<td>101.98 kip</td>
</tr>
<tr>
<td>Simplified analysis</td>
<td>161.65 kip</td>
<td>162.93 kip</td>
<td>105.32 kip</td>
</tr>
<tr>
<td>MBA method</td>
<td>159.2 kip</td>
<td>157.4 kip</td>
<td>104 kip</td>
</tr>
</tbody>
</table>

Table 3.1 Comparison of base shear capacities calculated for single precast walls with unbonded post-tensioning.

3.3.2.2 Neutral Axis Depth

The neutral axis depths calculated by the simplified analysis method and MBA are compared with the experimental data in Figures 3.11, 3.12, and 3.13 for the three walls. The MBA analysis underestimated the neutral axis depth at low drift ratios, which can be seen in Figure 3.11. Although only a few experimental data points were available for TW3 and TW4, these figures show that the analysis methods accurately estimates the neutral axis depths, especially at larger lateral drifts. Table 3.2 summarizes the experimental and predicted neutral axis depths at 2% drift. In all the cases, the maximum difference between the analysis and experimental results is about 15%. Also, note that only few data points were reported by Perez et al. for TW3, TW5.

3.3.2.3 Post-tensioning Steel Response

Figures 3.14, 3.15 and 3.16 compare the calculated post-tensioning steel stress variation with the experimental data. Overall, both analysis methods estimate the variation in stress of the post-tensioning


<table>
<thead>
<tr>
<th></th>
<th>TW1</th>
<th>TW3</th>
<th>TW5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>20.37 in</td>
<td>18.95 in</td>
<td>14.02 in</td>
</tr>
<tr>
<td>Simplified analysis</td>
<td>22.63 in</td>
<td>21.91 in</td>
<td>13.04 in</td>
</tr>
<tr>
<td>MBA method</td>
<td>21.35 in</td>
<td>19.58 in</td>
<td>12.96 in</td>
</tr>
</tbody>
</table>

Table 3.2 The neutral axis depth of the single precast walls tested at ATLSS center.

steel satisfactorily, including the yielding of the tendons that occurred near 1.5% lateral drift. The most significant discrepancy is seen for the PT2 in TW5, which is believed to be either faulty strain gauge reading or the anchorage slip.

### 3.3.2.4 Concrete strain

One advantage of the MBA analysis is that it estimates the concrete strain at the base of the wall. Figure 3.17 shows the comparison between the calculated concrete strain by MBA with the experimental strain gauge data for TW1. For comparison, a strain gauge (CE1) located at 4.5 inch from west end and 5 inch from the wall base is considered. Strain in the concrete is calculated at this location from the concrete strain at the base obtained from MBA. The concrete strain is underestimated by 8.7% and 8.3% using MBA analysis at 2% and 3% drift, respectively.

![Figure 3.8 Base shear vs. lateral drift response envelope for TW1.](image-url)
Figure 3.9  Base shear vs. lateral drift response envelope for TW3.

Figure 3.10  Base shear vs. lateral drift response envelope for TW5.
Figure 3.11  The neutral axis depth of TW1 as a function of lateral drift.

Figure 3.12  The neutral axis depth of TW3 as a function of lateral drift.
Figure 3.13 The neutral axis depth of TW5 as a function of lateral drift.

Figure 3.14 Variation in stress of various post-tensioning steel in TW1 as a function of lateral drift.
Figure 3.15 Variation in stress of various post-tensioning steel in TW3 as a function of lateral drift.

Figure 3.16 Variation in stress of various post-tensioning steel in TW5 as a function of lateral drift.
Figure 3.17  Concrete strains near the base of TW1.
CHAPTER 4. DESIGN PROCEDURE FOR UNBONDED POST-TENSIONED JOINTED PRECAST WALL SYSTEMS

4.1 Introduction

This chapter presents a seismic design procedure for unbonded post-tensioned jointed wall systems that have multiple walls with the same wall length. In general, the procedure follows that proposed by Stanton & Nakaki [10] and suggested changes by Thomas & Srinatharan (2003) [18]. A unique contribution of this thesis is that it addresses how to estimate the design moments for the critical walls (i.e., the leading wall in a two-wall jointed wall system and for an intermediate wall in a multi-wall jointed wall system). It is also shown through two design examples that the design can be simplified by designing the critical wall and applying the same details for all other walls in the jointed system.

4.2 Jointed Wall System Design

In a jointed wall system, multiple unbonded single precast walls are connected to each other with the help of special connectors along the vertical joints, as shown in Figure 4.1. Unbonded post-tensioned steel is distributed symmetrically about the center of each wall. The basic concept of the wall system is that it allows the wall to rock individually at the base when the wall system is subjected to lateral loads and return to its original vertical position after the event has concluded.

The post-tensioned steel is typically designed to remain elastic under the design-level earthquake loading. As a result, the post-tensioning steel provides the restoring force for the jointed wall even when the vertical connectors experience inelastic action due to the earthquake load. This restoring force helps to minimize the residual displacements of the wall when the lateral load is removed. The restoring capacity of the jointed wall depends on the amount of post-tensioning steel, the number of vertical connectors, initial prestressing force, and the cyclic behavior of the vertical connector. The
shear transfer from the wall to the foundation at the base utilizes a friction mechanism.

Using the simplified analysis procedure presented in Section 3.2, a parametric study of two-wall, three-wall and four-wall systems is conducted to understand the effects of various design parameters on the behavior of the jointed wall systems (see Appendix B). From this study, the following conclusions are drawn:

1. In a two-wall jointed system, the leading wall provides about $\frac{2}{3}$ of the total lateral force resistance. However, in a jointed wall system having more than two walls, the intermediate wall provides the larger moment resistance than the leading or the trailing wall. The percentage contribution of the intermediate wall will obviously depend on the number of walls in the system.

2. As suggested by Thomas & Srinath (2003), the post-tensioning steel in the trailing wall would first reach the yield limit state in a jointed wall system, which should dictate the initial design stress in the post-tensioning steel. However, the area of prestressing steel should be determined
by the wall providing the largest moment resistance.

These findings are included when establishing the design guidelines presented below, which follows the assumptions and outline proposed by *Stanton & Nakaki* [10].

**Design Assumptions**

The following assumptions are made for the design of jointed wall systems.

- The wall will undergo in-plane deformations only. Torsion and out of plane deformations are prevented by providing adequate out-of-plane bracing.
- All individual walls are assumed to have identical dimensions, reinforcement details, and the initial prestressing force.
- All the vertical joints contain the equal number of identical connectors, and a dependable force vs. displacement response envelope is available for the connector.
- All walls undergo the same lateral displacement at the floor and roof levels due to the rigid floor assumption.
- The post-tensioned steel is located at the center of each wall.
- The post-tensioning steel reaches the yield strain at the design drift \( \theta_{d,\text{a}} \), which is typically taken as 2%.

**Design Steps**

The design of the jointed wall can be performed using the following steps.

*Step 1:*

- **Establish the material properties:**
  - Prestressing steel: Modulus of elasticity \( (E_p) \) and yield strength \( (f_{pu}) \).
  - Concrete: Unconfined concrete strength \( (f_c) \) and coefficient of friction against concrete \( (\mu) \).
  - Connector: Load vs. displacement response.

- **Establish the wall dimensions:** total length of the wall system \( (L_{\text{wall}}) \), wall height \( (H_w) \), wall thickness \( (t_w) \), and the number of the walls \( (n) \). The height and length of each wall can be determined from the architectural drawings or from the preliminary calculations.

  While deciding at the number of walls, the \( \frac{L_{\text{wall}}}{H_w} \) ratio should be taken into consideration. *Stanton*
& Nakaki suggest that $\frac{L_{wall}}{h_{w}}$ should be more than 2.0 to ensure flexural dominant behavior for the wall. Hence, the length of each wall is

$$L_w = \frac{L_{wall}}{n}$$

- **Thickness of wall**: The following guidance may be used to establish an initial value for the wall thickness.
  1. The thickness of the wall can be assumed to be in the range of $h_{\text{min}}$ to $h_{\text{max}}$ [24].
  2. The wall thickness should be sufficient to limit the shear stress in the wall to what is specified in design codes.
  3. The wall thickness should be such that the wall is constructible with the required confinement reinforcement.

**Step 2:**

Based on a force-based design procedure (FBD) or displacement-based design (DBD), arrive at the required moment resistance for the wall system ($M_{design}$). Hence, the precast wall system should be designed such that

$$\phi M_{n,wall} \geq M_{design}$$

where $\phi$ is the flexural reduction factor and $M_{n,wall}$ is the nominal moment capacity of the wall system.

**Step 3:**

- Assuming a vertical relative displacement between walls to be $0.9L_w\theta_{des}$, determine the force in the connector ($F_{con}$) at the design drift from the connector force-displacement curve.

- For systems with the same properties in each direction, the equivalent viscous damping is given by the following equation [28].

$$\zeta_{eq} = \frac{2A_{loop}}{\pi A_{rec}}$$

where, $A_{loop}$ = area enclosed by the hysteresis loop, and $A_{rec}$ = area of the rectangle circumscribing the hysteresis loop.

- If the UFP connectors are used, then based on the required damping/energy dissipation, establish the number of connectors in each vertical joint ($N_{con}$) [12].

$$N_{con} = \frac{\pi \zeta_{eq} M_n}{1.25(n-1)F_{con} L_w}$$

(4.1)
where, $\zeta_{eq}$ is the equivalent damping.

**Step 4: Design Area of the post-tensioning steel**

- Design $A_p$ for the wall resisting the maximum moment:

$$M_{\text{design,wall}} = \Omega \frac{M_{\text{design}}}{n \phi} \quad (4.2)$$

$$\Omega = 1 + \frac{\alpha \phi N_{\text{con}} F_{\text{con}} L_w}{M_{\text{design}}} \quad (4.3)$$

where, $n$ is the number of walls,

$\Omega$ is the moment contribution factor, and

$\alpha$ is a constant.

- For a two-wall system (i.e. when $n=2$), determine the required area of steel $A_p$ using the moment equilibrium of forces acting on the leading wall, as given by Equation 4.4.

$$M_{\text{design,lead}} = (P_D + 0.95 A_p F_{py})(L_w/2) - \frac{P_D + 0.95 A_p F_{py} + N_{\text{con}} F_{\text{con}}}{2 f_c t_w} + \frac{N_{\text{con}} F_{\text{con}} (L_w - \frac{P_D + 0.95 A_p F_{py} + N_{\text{con}} F_{\text{con}}}{3.2 f_c t_w})}{3.2 f_c t_w} \quad (4.4)$$

where, $P_D$ is equal to self weight + super imposed live load = $\gamma_c L_w t_w H_w + w_{\text{floor}} L_w$.

$A_p$ is the required area of prestress steel.

$\gamma_c$ is concrete unit weight.

$w_{\text{floor}}$ is superimposed live load.

Equation 4.4 will lead to a quadratic equation in $A_p$ and use the small positive root as the design solution.

- For a multi-wall system, determine the required area of steel $A_p$ using the moment equilibrium of the forces acting on the intermediate wall, as given by Equation 4.5.

$$M_{\text{design,inter}} = (P_D + 0.95 A_p F_{py})(0.5 L_{\text{wall}}) - \frac{P_D + 0.95 A_p F_{py}}{3.2 f_c t_w} + N_{\text{con}} F_{\text{con}} L_w \quad (4.5)$$
Equation 4.5 will lead to a quadratic equation in $A_v$ and use the small positive root as the design solution.

**Step 5: Design initial stress in the post-tensioning steel**

- Estimate the neutral axis depth in the trailing wall using equation 4.6:

$$c_{\text{trail}} = \frac{P_D + A_p F_{py} - N_{\text{con}} F_{\text{con}}}{(1.6 * 0.85 f'_c t_w)}$$

(4.6)

- The maximum permissible initial stress for post-tensioning steel is thus:

$$F_{pi} = F_{py} - \frac{(0.5 L_w - c_{\text{trail}}) \cdot \theta_{\text{des}} \cdot E_p}{H_w}.$$

(4.7)

**Step 6: Estimate the moment capacity**

Use the area of post-tensioning steel, initial prestress, and the connector details designed above for all walls appropriately. Using the simplified analysis procedure, show that the moment capacity of the wall system with the designed prestress and connector details is greater than the design moment.

**Step 7: Confinement design**

Estimate the required strain demand in the concrete from equation 4.8. From the MBA analysis procedure,

$$\varepsilon_{\text{conc}} = c(M_{\text{max}} \frac{M_{\text{base}}}{E_c I_{eff}} + \frac{\theta_{\text{max}}}{0.06 H_w})$$

(4.8)

where, $M_{\text{max}}$ is the base moment resistance of the leading wall at maximum drift in consideration ($\theta_{\text{max}}$),

$E_c$ is the modulus of elasticity of concrete, which may be taken as $57000 \sqrt{f_c}$ (psi),

$I_{eff}$ is the effective moment of inertia of the wall and is equal to $\frac{1}{12} f_c I_c$,

and $c$ is the neutral axis depth of the leading wall at $\theta_{\text{max}}$, which can be obtained from the simplified analysis.

Determine the confinement steel required ($\rho_s$) based on the confined concrete model proposed by Mander et al. (1988) (as shown below) [17], and check with the minimum confinement requirements of ACI code.

$$\rho_s = \frac{(\varepsilon_{\text{conc}} - 0.004)(f'_{\text{su}})}{1.4 f'_c \varepsilon_{\text{su}}}$$

where, $\rho_s$ is the volumetric ratio of the required confinement steel,

$f'_{\text{yh}}$ is the yield strength of the transverse reinforcement,

$\varepsilon_{\text{su}}$ is the ultimate strain capacity of the transverse reinforcement,

$f'_{\text{su}} = 1.6 f'_c$, and

$$\rho_s = \frac{\rho_e + \rho_y}{2},$$
4.3 Design Examples

Presented below are two set of design examples, which show that the design method presented in Section 4.2 will be satisfactory for designing jointed precast wall systems.

4.3.1 Example 1

Design a two-wall, three-wall, and four-wall jointed system to resist a base moment of 75780 kip-in. For this example, assume the wall length and total number of UFP connectors to be the same as those used in the PRESSS test building. In addition, use the same material properties established for the PRESSS wall, except for the concrete strength, which is taken as 6 ksi.

Design solution:

**Step 1:**

Material Properties:
Concrete strength ($f'_c$) = 6 ksi.
Concrete density ($\gamma_c$) = 150 pcf.
Yield strength of post-tensioning bar ($f_{py}$) = 140 ksi.
Young’s Modulus for post-tensioning bar ($E_p$) = 27000 ksi.
The force-displacement response of the UFP connector shown in Figure 2.5 is used.

Wall dimensions:
Length of a single wall ($L_w$) = 108 in.
Thickness of the wall ($t_w$) = 8 in.
Height of the wall ($H_w$) = 450 in.

**Step 2:**

Given that, the design moment ($M_{design}$) = 75780 kip-in.

**Step 3:**

Assume the design drift ($\theta_{ds}$) = 0.02.

Force transmitted through each connector at $\theta_{ds}$, $F_{con}$ = 11.65 kip.

In order to have the same damping as the PRESSS building, the total number of connectors for each wall system is taken as that used in the PRESSS building.

Hence, the total number of connectors in the wall system = 20. Table 4.1 shows the number of connectors per vertical joint.
<table>
<thead>
<tr>
<th>Wall system</th>
<th>( N_{con} )</th>
<th>No. of connectors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Three-wall</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Four-wall</td>
<td>7</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.1 Number of connectors in each vertical joint between walls

**Step 4: Find** \( M_{design,wall} \) and \( A_p \)

From Equation 4.2,
\[
M_{design,wall} = \Omega \frac{M_{design}}{n\phi}
\]
\[
\Omega = 1 + \frac{\alpha N_{con} F_{con} L_w}{M_{design}}
\]

In the above equation, \( \phi = 0.9 \) should be used as per the current design practice. However, \( \phi = 1 \) is used in this example to demonstrate the accuracy of the design procedure. Table 4.2 demonstrates the estimation of \( M_{design,wall} \) for the various wall systems.

Now solving Equation 4.4 and Equation 4.5 of the design procedure, \( A_p \) values shown in Table 4.3 are obtained. The smallest value is taken as the required steel area.

**Step 5: Design of initial stress in post-tension steel, \( F_{p,i} \)**

From Equation 4.6 and Equation 4.7 of the design procedure, design the initial post-tensioning steel stress. The details are shown below in Table 4.4.

**Step 6:**

By using the simplified analysis procedure for the wall systems with the designed values of \( A_p \) and \( F_{p,i} \), the actual moment capacities of the wall systems are obtained. The results are shown in the Table 4.5.

The calculated capacities of individual walls and the wall systems in all cases at the design drift of 2%
## Table 4.3 Area of required post-tensioning steel for various wall systems

<table>
<thead>
<tr>
<th></th>
<th>roots of quadratic</th>
<th>design solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall system</td>
<td>$A_p_1$: 3.598 sq.in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_p_2$: 54.75 sq.in.</td>
<td></td>
</tr>
<tr>
<td>Three-wall system</td>
<td>$A_p_1$: 2.21 sq.in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_p_2$: 59.64 sq.in.</td>
<td></td>
</tr>
<tr>
<td>Four-wall system</td>
<td>$A_p_1$: 1.524 sq.in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_p_2$: 60.33 sq.in.</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.4 Design values for the initial stress in the post-tensioning tendons.

<table>
<thead>
<tr>
<th></th>
<th>Neutral axis depth of trailing wall (in.)</th>
<th>Initial post-tensioning steel stress ($F_p$)(ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall system</td>
<td>4.67</td>
<td>79.26</td>
</tr>
<tr>
<td>Three-wall system</td>
<td>3.47</td>
<td>77.79</td>
</tr>
<tr>
<td>Four-wall system</td>
<td>2.54</td>
<td>76.64</td>
</tr>
</tbody>
</table>

Table 4.5 Actual and Design moments for the wall systems.

<table>
<thead>
<tr>
<th></th>
<th>Actual capacity (kip-in)</th>
<th>Design moment (kip-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall system</td>
<td>$M_{lead,calculated}$: 49608.96 kip-in</td>
<td>$M_{lead,design}$: 49212.65 kip-in</td>
</tr>
<tr>
<td></td>
<td>$M_{total,calculated}$: 78688.18 kip-in</td>
<td>$M_{total,design}$: 75780 kip-in</td>
</tr>
<tr>
<td>Three-wall system</td>
<td>$M_{inter,calculated}$: 30298.45 kip-in</td>
<td>$M_{inter,design}$: 29579.38 kip-in</td>
</tr>
<tr>
<td></td>
<td>$M_{total,calculated}$: 78329.31 kip-in</td>
<td>$M_{total,design}$: 75780 kip-in</td>
</tr>
<tr>
<td>Four-wall system</td>
<td>$M_{inter,calculated}$: 21692.90 kip-in</td>
<td>$M_{inter,design}$: 21212.675 kip-in</td>
</tr>
<tr>
<td></td>
<td>$M_{total,calculated}$: 77995.11 kip-in</td>
<td>$M_{total,design}$: 75780 kip-in</td>
</tr>
</tbody>
</table>
are greater than the design values. (Figure 4.2 and Figure 4.3).

**Step 7: Confinement design**

For confinement steel design, $\theta_{max}$ is taken as 0.03 and the corresponding moment capacities ($M_{max}$) are taken from the simplified analysis that lead to the various capacities in Table 4.6. The $M_{max}$ and the required concrete strain capacity values are shown below in Table 4.6. Note that $\rho_s$ required is less than $\rho_{s\_min}$, which is equal to $\frac{0.12f'_c}{f_y}\approx 0.012$. So, the minimum amount of confinement should be provided.

<table>
<thead>
<tr>
<th>Wall system</th>
<th>$M_{max}$(kip-in)</th>
<th>N.Axix depth (in.)</th>
<th>$\varepsilon_{concre}$</th>
<th>$\rho_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall system</td>
<td>55764.77</td>
<td>11.128</td>
<td>0.0126</td>
<td>0.0081</td>
</tr>
<tr>
<td>Three-wall system</td>
<td>33540.16</td>
<td>6.838</td>
<td>0.00765</td>
<td>0.0034</td>
</tr>
<tr>
<td>Four-wall system</td>
<td>24227.37</td>
<td>4.926</td>
<td>0.0055</td>
<td>0.0014</td>
</tr>
</tbody>
</table>

Table 4.6  Design of the confinement steel ratios for the wall systems.
4.3.2 Example 2

Design a two-wall, three-wall, and four-wall jointed system to resist a base moment of 75780 kip-in. For this example, assume the length of the total wall system and number of UFP connectors to be the same as those used in the PRESSS test building. In addition, use the same material properties established for the PRESSS wall, except for the concrete strength which is taken as 6 ksi.

Design solution:

**Step 1:**

*Material Properties:*

Concrete strength \((f_c') = 6\) ksi.

Concrete density \((\gamma_c) = 150\) pcf.

Yield Strength of post-tensioning bar \((f_{pt}) = 140\) ksi.

Young’s Modulus for post-tensioning bar \((E_p) = 27700\) ksi.

The force-displacement response of the UFP connector is taken as same as the PRESSS building test.

*Wall dimensions:*

Length of a single wall \((L_w) = 108\) in.

Thickness of the wall \((t_w) = 8\) in.
Height of the wall \( (H_w) = 450 \text{ in.} \)

**Step 2:**

<table>
<thead>
<tr>
<th>S.No</th>
<th>Wall system</th>
<th>Length of single wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Two-wall</td>
<td>108</td>
</tr>
<tr>
<td>2</td>
<td>Three-wall</td>
<td>72</td>
</tr>
<tr>
<td>3</td>
<td>Four-wall</td>
<td>54</td>
</tr>
</tbody>
</table>

Table 4.7 Length of the wall in wall systems.

Given that, the design moment \( (M_{design}) = 75780 \text{ kip-in.} \)

**Step 3:**

Assume the design drift \( (\theta_{des}) = 0.02. \)

Force transmitted through each connector at \( \theta_{des} \), \( F_{con} \) is shown below in Table 4.8.

In order to have the same damping as the PRESSS building, the total number of connectors in the wall system is taken same as the PRESSS building.

Hence, the total number of connectors in the wall system = 20. Table 4.8 shows the number of connectors per vertical joint.

<table>
<thead>
<tr>
<th>S. No</th>
<th>Wall system</th>
<th>( N_{con}, \text{No. of connectors} )</th>
<th>( F_{con} ) (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Two-wall</td>
<td>20</td>
<td>11.6</td>
</tr>
<tr>
<td>2</td>
<td>Three-wall</td>
<td>10</td>
<td>10.5</td>
</tr>
<tr>
<td>3</td>
<td>Four-wall</td>
<td>7</td>
<td>9.8</td>
</tr>
</tbody>
</table>

Table 4.8 Number of connectors per vertical joint and connector forces.

**Step 4: Find** \( M_{design} \) **and** \( A_p \)

From Equation 4.2,

\[
M_{design,wall} = \Omega \frac{M_{design}}{n\phi}
\]

\[
\Omega = 1 + \frac{\alpha \phi N_{con} F_{con} L_w}{M_{design}}
\]

In the above equation, \( \phi = 0.9 \) should be used as per the current design practice. However, \( \phi = 1 \) is used in this example to demonstrate the accuracy of the design procedure. Table 4.9 demonstrates the estimation of \( M_{design,wall} \) for the various wall systems.

Now solving Equation 4.4 and Equation 4.5 of the design procedure, the following \( A_p \) values (Table 4.10) are obtained. The smallest value is taken as the required area of post-tensioning steel.
### Table 4.9 Estimation of $M_{design,wall}$ for various wall systems.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Two-wall system</th>
<th>Three-wall system</th>
<th>Four-wall system</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n$</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>$\phi$</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>0.9</td>
<td>1.03</td>
<td>1.03</td>
</tr>
<tr>
<td>$L_w$</td>
<td>108</td>
<td>72</td>
<td>54</td>
</tr>
<tr>
<td>$F_{con}$</td>
<td>11.65 kip</td>
<td>10.5 kip</td>
<td>9.8 kip</td>
</tr>
<tr>
<td>$M_{design}$</td>
<td>75780 kip-in</td>
<td>75780 kip-in</td>
<td>75780 kip-in</td>
</tr>
<tr>
<td>$\Omega$</td>
<td>1.3</td>
<td>1.10</td>
<td>1.05</td>
</tr>
<tr>
<td>$M_{design,inter}$</td>
<td></td>
<td>27855.6 kip-in</td>
<td>19898.2 kip-in</td>
</tr>
<tr>
<td>$M_{design,lead}$</td>
<td>49212.65 kip-in</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 4.10 Area of required post-tensioning steel for various wall systems.

<table>
<thead>
<tr>
<th></th>
<th>roots of quadratic</th>
<th>design solution</th>
</tr>
</thead>
</table>
| Two-wall system | $A_{p1}: 3.598 \text{ sq.in.}$  
               | $A_{p2}: 54.75 \text{ sq.in.}$  | $A_p: 3.598 \text{ sq.in.}$ |
| Three-wall system | $A_{p1}: 4.61 \text{ sq.in.}$  
                   | $A_{p2}: 36.61 \text{ sq.in.}$  | $A_p: 4.61 \text{ sq.in.}$ |
| Four-wall system  | $A_{p1}: 5.34 \text{ sq.in.}$  
                   | $A_{p2}: 25.58 \text{ sq.in.}$  | $A_p: 5.34 \text{ sq.in.}$ |

### Step 5: Design of initial stress in post-tension steel, $F_p$

Now from Equation 4.6 and Equation 4.7 of the design procedure, design the initial post-tensioning steel stress required. The details are shown below in Table 4.11.

### Step 6:

<table>
<thead>
<tr>
<th></th>
<th>Neutral axis depth of the trailing wall (in.)</th>
<th>Initial post-tensioning steel stress ($F_p$)(ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall system</td>
<td>4.67</td>
<td>79.26</td>
</tr>
<tr>
<td>Three-wall system</td>
<td>8.65</td>
<td>106.32</td>
</tr>
<tr>
<td>Four-wall system</td>
<td>10.66</td>
<td>119.88</td>
</tr>
</tbody>
</table>

### Table 4.11 Design values for the initial stress in post-tensioning tendons

By using the simplified analysis procedure for the wall systems with the designed values of $A_p$ and $F_p$, the actual moment capacities of the wall systems are obtained. The results are shown in Table 4.12.

The calculated capacities of individual walls and the wall systems in all cases at the design drift of 2% are greater than the design values. (Figure 4.4 and Figure 4.5).
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<table>
<thead>
<tr>
<th>Wall system</th>
<th>Actual capacity (kip-in)</th>
<th>Design moment (kip-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall system</td>
<td>$M_{\text{int}, \text{calculated}}$: 49608.96 kip-in</td>
<td>$M_{\text{int}, \text{design}}$: 49212.65 kip-in</td>
</tr>
<tr>
<td></td>
<td>$M_{\text{total}, \text{calculated}}$: 78688.18 kip-in</td>
<td>$M_{\text{total}, \text{design}}$: 75780 kip-in</td>
</tr>
<tr>
<td>Three-wall system</td>
<td>$M_{\text{int}, \text{calculated}}$: 27937.97 kip-in</td>
<td>$M_{\text{int}, \text{design}}$: 27855.6 kip-in</td>
</tr>
<tr>
<td></td>
<td>$M_{\text{total}, \text{calculated}}$: 76097.31 kip-in</td>
<td>$M_{\text{total}, \text{design}}$: 75780 kip-in</td>
</tr>
<tr>
<td>Four-wall system</td>
<td>$M_{\text{int}, \text{calculated}}$: 18980.17 kip-in</td>
<td>$M_{\text{int}, \text{design}}$: 19898 kip-in</td>
</tr>
<tr>
<td></td>
<td>$M_{\text{total}, \text{calculated}}$: 75688.78 kip-in</td>
<td>$M_{\text{total}, \text{design}}$: 75780 kip-in</td>
</tr>
</tbody>
</table>

Table 4.12 Actual and Design moments for the wall systems.

**Step 7: Confinement design**

For confinement steel design, $\theta_{\text{max}}$ is taken as 0.03 and the corresponding moment capacities ($M_{\text{max}}$) are taken from the simplified analysis that leads to the various capacities in Table 4.13. The $M_{\text{max}}$ and the required concrete strain capacity values are shown below in Table 4.13. Note that $\rho_s$ required is

<table>
<thead>
<tr>
<th>Wall system</th>
<th>$M_{\text{max}}$ (kip-in)</th>
<th>N.Ax depth (c)</th>
<th>$\varepsilon_{\text{concr}}$ (reqd.)</th>
<th>$\rho_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-wall system</td>
<td>55764.77</td>
<td>11.128</td>
<td>0.0126</td>
<td>0.0081</td>
</tr>
<tr>
<td>Three-wall system</td>
<td>28225.76</td>
<td>11.11</td>
<td>0.0128</td>
<td>0.0082</td>
</tr>
<tr>
<td>Four-wall system</td>
<td>19687.07</td>
<td>12.01</td>
<td>0.0133</td>
<td>0.009</td>
</tr>
</tbody>
</table>

Table 4.13 Required confinement steel for the wall systems.

less than $\rho_{s, \text{min}}$, which is equal to $\frac{0.12f_c}{f_{ub}} = 0.012$. So, the minimum amount of confinement should be provided.
Figure 4.4 Comparison of $M_{\text{design}}$ against $M_{\text{calculated}}$ for the single wall that was selected for the design of $A_p$ in each wall system.

Figure 4.5 Comparison of design moment of the wall system with that calculated using the design details.
CHAPTER 5. CONCLUSIONS

5.1 Overview

Concrete structural walls provide a cost-effective means to resist seismic lateral loads and, thus, they are frequently used as the primary lateral load resisting system in reinforced concrete buildings. Structural walls have performed very well in past seismic events. With the added benefits of precast concrete, unbonded post-tensioned precast jointed walls are an excellent system for resisting lateral forces. The primary limitation for using precast systems in seismic regions of the United States is the code specification that requires the design of precast concrete structures to emulate the behavior of monolithic cast-in-place concrete structures. In response to a need to overcome this limitation, as well as to utilize the benefits of the precast concrete, and promote the precast concrete as a preferred alternative for seismic design, the PRESSS (PREcast Seismic Structural Systems) program was initiated in the early 1990s in the United States. As part of the PRESSS research, a five-story precast test building incorporating an unbonded post-tensioned precast concrete jointed wall system was constructed and tested at the University of California at San Diego (UCSD). During the testing it was found that the jointed wall system performed well above the expectations of the researchers. With an objective of implementing this system in practice, a set of design guidelines was established for unbonded post-tensioned jointed walls by Stanton & Nakaki. Utilizing the previous work done by Thomas & Sridharan (2003) and Onur & Sridharan (2004), in this thesis, a simple analysis procedure is developed and validated against the experimental response of the jointed wall systems and the precast single walls. A further validation of analysis procedure developed by Thomas & Sridharan (2003) based on the “monolithic beam analogy (MBA)” is also conducted. Based on this analysis procedure, a design method is developed for designing the jointed wall systems with equal wall lengths.
5.2 Conclusions

The following conclusions have been drawn as a result of the analytical investigation based on both the simplified analysis and that based on the monolithic beam analogy (MBA):

- The simplified analysis method and the analysis procedure based on the monolithic beam analogy (MBA method) adequately predicted the moment resistance of the PRESSS jointed wall and the single precast walls tested by ATLSS research center.

- Both the analysis procedures predicted the post-tensioning steel elongations satisfactorily.

- In a jointed wall system, leading wall contributes to the maximum moment resistance in two-wall system and intermediate wall contributes more in multi-wall system.

- The MBA method was found to be satisfactory in predicting the concrete strain near the wall base in the single precast wall tested at ATLSS research center.

- Increasing the number of walls in a jointed system is found to be inefficient. In other words, for a given length of the jointed wall system, two wall system is more efficient than three wall system.

- Moment contribution of the wall providing the maximum moment resistance can be determined from

  \[ M_{\text{design,n}} = \Omega \frac{M_{\text{design}}}{n} \]
  \[ \Omega = 1 + \frac{\alpha N_{\text{con}} F_{\text{con}} L_w}{M_{\text{design}}} \]

  where, \( n \) is the number of walls,
  \( \Omega \) is the moment contribution factor, and
  \( \alpha \) is a constant.

  when \( n=2 \), \( M_{\text{design,wall}} = M_{\text{design,lead}} \) and \( \alpha = 0.9 \)

  when \( n \geq 3 \), \( M_{\text{design,wall}} = M_{\text{design,inter}} \) and \( \alpha = 1.03 \)

  The \( \alpha \) values were arrived from the study on various wall systems behavior as stated before.

- For a given length of the wall system and the amount of post-tensioning steel, jointed walls with unequal wall lengths are found to be more efficient compared to jointed walls with equal wall lengths.

- The design method developed in Chapter 4 is found to be satisfactory in designing the jointed wall systems with equal wall lengths.
• In a jointed wall system, the design can be simplified by designing only the wall resisting the maximum moment resistance.

5.3 Future Research

• Further investigation of an accurate equivalent plastic hinge length (Lp) for unbonded post-tensioned precast systems should be performed to improve the suggested MBA method.

• Testing of unbonded post-tensioned precast jointed wall systems with different wall lengths should be performed to further validate the simplified analysis procedure, the MBA method and the design procedure.

• More testing of the jointed walls with different aspect ratios are necessary to validate the accuracy of various analytical and design methods.

• Testing of jointed wall systems with end columns are required to prove their better performance over the jointed wall systems proposed in the PRESSS project.
APPENDIX A. ESTIMATION OF $\beta$ AND $\alpha_{lever}$ VALUES

Introduction

The compressive strength and strain capacity of the concrete increase as the confinement reinforcement increases. The stress-strain behavior of confined concrete can be described using the model proposed by Mander et al. [17]. Using this model, an equivalent rectangular block representation is examined for the confined concrete in this appendix. It is shown that, the effective concrete strength ($\beta f'_{ce}$) and the location of the resultant compression force ($\alpha_{lever}$) varies with the unconfined concrete strength and the amount of confinement reinforcement.

Confinement Model

The stress-strain behavior of confined concrete may be described as (see Figure A.1).

\[ f_c = \frac{f_{ce} x_r}{1 + x^r} \quad (A.1) \]

\[ f'_{ce} = f'_c (2.254 \sqrt{1 + \frac{7.94 f'_c}{f_c}} - 2 \frac{f'_c}{f_c} - 1.254) \quad (A.2) \]

\[ \varepsilon_{ve} = 0.002 (1 + 5 (\frac{f'_c}{f_c} - 1)) \quad (A.3) \]

\[ \varepsilon_{ea} = 0.04 + \frac{1.4 \rho_s f_{sb} \varepsilon_{sa}}{f'_{ce}} \quad (A.4) \]

\[ x = \frac{\varepsilon_c}{\varepsilon_{ce}} \quad r = \frac{E_c}{E_c - E_{sec}} \quad \text{and} \quad E_{sec} = \frac{f'_{ce}}{\varepsilon_{ce}} \quad (A.5) \]

where, \(f_c\) is stress in concrete at strain of \(\varepsilon_{ce}\).

\(f'_{ce}\) is the peak confined concrete strength.

\(E_c\) is the young's modulus of concrete.

\(E_{sec}\) is the secant modulus of concrete.
\[ \varepsilon_{cc} \] is the strain corresponding to \( f'_{cc} \).

\[ \varepsilon_{cu} \] is the strain corresponding to crushing of concrete.

\( f_{yh} \) is the yield strength of hoop reinforcement.

\( \varepsilon_{su} \) is the ultimate strain capacity of hoop reinforcement, and

\( f'_{l} \) is the effective lateral confining stress and \( f'_{t} \) is the sum of \( f'_{tx} = k_x \rho_x f_{yh} \) and \( f'_{ty} = k_y \rho_y f_{yh} \) for rectangular confined regions. \( \rho_x \) and \( \rho_y \) are the transverse reinforcement area ratios in the principal directions. \( k_x \) is the confinement effectiveness coefficient. A value of 0.6 is recommended for rectangular wall sections.

**Estimation of \( \beta \) value**

For a given value of \( \varepsilon_d \) in the range of 0 and \( \varepsilon_{cu} \), \( \beta \) is defined such that the area under the stress-strain curve up to \( \varepsilon_d \) will be equal to the area of the rectangle with a constant stress of \( \beta f'_{cc} \). Hence,

\[ \beta = \frac{\int_0^{\varepsilon_d} f_c dz}{f'_{cc} \varepsilon_d} \]  \hspace{1cm} (A.6)
Estimation of $\alpha_{lever}$ value

The location of the resultant compressive force in Figure A.1 is defined as $\alpha_{lever}\varepsilon_d$ from the origin. The moment of area under the stress-strain:

$$\int_0^{\varepsilon_d} f_c \varepsilon d\varepsilon$$

Hence,

$$\alpha_{lever} = \frac{\int_0^{\varepsilon_d} f_c \varepsilon d\varepsilon}{\varepsilon_d \int_0^{\varepsilon_d} f_c d\varepsilon}$$

Variation of $\alpha_{lever}$

To arrive at the variation of $\alpha_{lever}$ and $\beta$ with $f'_c$, the amount of confinement reinforcement provided, an investigation was conducted. In this study, the confinement pressure was varied from 500 psi to 1800 psi, while the unconfined concrete strength was varied from 5 ksi to 9 ksi. The corresponding $\alpha_{lever}$ and $\beta$ values were obtained using the Equations A.5 and A.7. The values of $\alpha_{lever}$ obtained for different concrete strengths and confinements are plotted against $\frac{\varepsilon_d}{\varepsilon_{ce}}$ in Figure A.2. By curve fitting the following variation for $\alpha_{lever}$ is established:

$$\alpha_{lever,max} = 0.418 + 0.064 \ln \left(\frac{\varepsilon_d}{\varepsilon_{ce}}\right)$$

![Figure A.2](image-url)  
Variation of $\alpha_{lever}$ with $\frac{\varepsilon_d}{\varepsilon_{ce}}$. 
APPENDIX B. ANALYTICAL INVESTIGATION OF UNBONDED POST-TENSIONED PRECAST JOINTED WALLS

Introduction

In the present day design world, the common practice is to design a structure for the reduced lateral forces and accept damage in potential plastic hinge regions, which are designed specifically for ductility of the structure. The past earthquake events demonstrated the superior performance of buildings incorporating the structural walls as the primary lateral load resisting system. Although the structural walls are a common and cost-effective way to resist the lateral loads induced by earthquakes, it is necessary to make them more economical for them to be the preferred option over other lateral load resisting systems. Traditionally, cast-in-place reinforced concrete is used for constructing the structural walls. However, the main disadvantage of this practice is the significant damage in plastic hinge regions with large residual drifts and wide residual cracks, leading to increased cost for repairing the structures after a seismic event. The ideal solution for this problem is to uncouple the energy dissipation mechanism from the structure and provide it through other means [29].

The recent trend in the construction industry is to use precast concrete for buildings due to its better quality control and rapid construction. The primary limitations of using precast concrete for lateral load resisting systems in the United States are the code restriction of designing precast concrete structures to emulate the behavior of monolithic cast-in-place concrete structures and the lack of design procedures for precast structures in seismic regions. In response to the recognized need to overcome these limitations of precast concrete walls and to promote the precast concrete as a preferred alternative for seismic design, the PRESSS (PREcast Seismic Structural Systems) program was initiated in the early 1990's in the United States. As part of this program, researchers arrived at an innovative structural system called "jointed wall system" for resisting the lateral loads induced by a seismic event [1, 23].

The jointed wall system consists of multiple unbonded single precast walls connected to each other
with the help of special connectors along the vertical joints, as shown in Figure 4.1. Unbonded post-tensioned steel is located symmetrically across the cross-section of each wall. The unbonded steel yields at larger member deformation compared to bonded steel, allowing the unbonded jointed wall system to undergo a larger nonlinear lateral displacement. The basic concept of the wall system is that it allows the wall to rock individually at the base when the wall system is subjected to lateral loads and return to its vertical position after the event has concluded. The post-tensioned steel is typically designed to remain elastic under the design-level earthquake loading. As a result, the post-tensioning steel provides the restoring force for the jointed wall even when the vertical connectors experience inelastic action due to the earthquake load. This restoring force helps to minimize the residual displacements of the wall when the lateral load is removed. While the yielding of connectors in vertical joints contributes to the major part of total energy dissipated, a small portion of the total energy is dissipated by the inelastic action in the concrete at the wall system base [8, 9].

As a part of the PRESSS program, a 60% scale model of a five story precast building with moment resisting frames in one direction and a precast jointed wall in the perpendicular direction was tested at University of California, San Diego (UCSD). The detailed investigation of the test results revealed that the jointed wall suffered only minor spalling at the base when the building was subjected to an earthquake 50 percent higher than the design-level earthquake [8]. The UFP connectors (the vertical connectors used in the test building) provided a considerable energy dissipation and a small residual drift was observed after the design-level earthquake was removed.

Following the PRESSS building testing, design guidelines were developed for jointed walls by Stauton and Nakaki [10] and was then followed a detailed investigation of the jointed wall by Thomas and Sritharan [18] to validate the proposed design guidelines. This investigation led to the development of an analysis procedure to analyze the behavior of a jointed wall system.

The present paper presents an analysis procedure used to estimate the jointed wall behavior and the study of various wall parameters' influence on jointed wall system behavior. This paper also presents a new jointed wall system configuration, which is arrived at by optimizing various wall parameters based on the analytical study conducted.

### Simplified Analysis Procedure

The performance of unbonded post-tensioned precast concrete walls is controlled by the gap opening at the horizontal joints. To analyze a jointed wall system or a single wall with unbonded post-tensioning steel, an iterative procedure is suggested to find the neutral axis depth, for a given base rotation (2%).
The force equilibrium and geometric compatibility conditions are used in the iteration process, which also leads to an estimate of the post-tension steel elongation. According to the design guidelines presented by Stanton & Nakaki, the following assumptions are made in the analysis procedure.

1. The walls are provided with adequate out-of-plane bracing, preventing them from experiencing torsional and out-of-plane deformations.

2. The dimensions and material properties of the walls and connectors are known.

3. The fiber grout pad located at the interface between the wall and the foundation does not experience any strength degradation.

4. All walls will undergo the same lateral deformation at every floor level due to the rigid floor assumption.

5. The wall base has enough friction resistance, so that the wall will not undergo any lateral movement at the base.

6. The connectors and the post-tension steel anchorages remain fully effective for the entire analysis.

7. All vertical joints in a jointed wall system have the same number of connectors.

The analysis procedure is described in the following steps.

**Step 1:** Define wall system dimensions, reinforcement details, and material properties.

The following variables are defined in this step.

**Wall System Dimensions:**
- \( h_w \) = height of the wall system,
- \( t_w \) = thickness of the wall system,
- \( t_{cr} \) = confinement area thickness,
- \( l_{wall} \) = length of the wall system,
- \( n \) = number of walls in the jointed system, and
- \( l_w = \frac{l_{wall}}{n} \) = length of each wall.

**Prestressing/Post-tensioning Steel Details:**
- \( A_{pt} \) = area of a post-tensioning tendon,
- \( n_{pt} \) = number of post-tensioning tendons,
- \( h_u \) = unbonded length of the post-tensioning tendon,
- \( x_{pt,j} \) = location of the post-tension tendon from the rocking edge of the wall,
- \( E_p \) = modulus of elasticity of the post-tensioning tendon.
$f_{pt} = \text{initial stress in the post-tensioning tendon,}$

$f_{py} = \text{yield strength of the post-tensioning tendon, and}$

$A_{p,\text{total}} (= n_{p}A_{pt}) = \text{total post-tensioning steel area in each wall.}$

Confinement Details:

$A_{s} = \text{area of confinement steel,}$

$s = \text{spacing of the confinement steel,}$

$l_{cr} = \text{length of confinement area, and}$

$f_{y} = \text{mild steel yield strength.}$

Concrete Properties:

$f'_{c} = \text{concrete compressive strength,}$

$f'_{n} = \text{compressive strength of interface grout,}$

$f'_{cc} = \text{confined concrete strength,}$

$\varepsilon_{cc} = \text{concrete strain at fcc,}$

$\varepsilon_{cu} = \text{concrete crushing strain, and}$

$\rho_{c} = \text{concrete density.}$

Connector Details:

$n_{con} = \text{number of connectors per joint,}$

$F_{sc} = \text{force in the connector corresponding to a displacement (use an experimentally established force-displacement response of the connector to determine $F_{sc}$, as recommended by Thomas & Sridharan).}$

**Step 2: Decompression point**

In this step, the decompression point is found, which defines the beginning of a gap opening at the wall base, which corresponds to the condition when the stress in the extreme concrete fiber farthest from the rocking edge reaches zero. Assuming a linear strain distribution at the critical section, due to the moment induced by the decompression force $F_{decomp}$, the following equations are used to determine the corresponding moment resistance. The decompression moment ($M_{decomp}$) is calculated from the elastic flexural formula $\sigma = \frac{M_{c}}{I}$, substituting appropriate values for the neutral axis depth ($c$) and the moment of inertia ($I$):

$$M_{decomp} = \frac{\sigma I}{c} \quad (B.1)$$

$$\sigma_{i} = \frac{\sum_{1}^{n_{p}} f_{pt} A_{pt}}{t_{w} b_{w}} \quad (B.2)$$

$$I = \frac{t_{w} b_{w}^{3}}{12} \quad (B.3)$$

$$c = \frac{b_{w}}{2} \quad (B.4)$$
**Step 3: Neutral axis depth at 2% base rotation**

The neutral axis depth that satisfies the vertical direction force equilibrium at the wall base is found for each wall through iteration with an assumed neutral axis depth as the initial value. The following sub-steps are used in this process.

1. Assume a neutral axis depth \( c \) for the wall.

2. Determine the total gravity load on each wall \( (N) \):

\[
N = \gamma_w l_w t_w h_w + w_{floor} l_w
\]

where, \( w_{floor} \) is the uniform dead load acting on the wall.

3. Determine the stresses and strains in the post-tension tendon:

- The elongation of the post-tensioning tendon:

\[
\Delta_{p,i} = 0.02 (l_{w} - c) \quad \text{(B.5)}
\]

- The strain in each post-tensioning tendon:

\[
\frac{\Delta_{p,i}}{h_a} = \frac{f_p}{E_p}
\]

- Calculate the stress in each post-tensioning tendon \( (f_p) \) from the stress-strain curve of the post-tension steel. The above steps should be repeated to determine the stress in all post-tension tendons.

- The total post-tensioning force:

\[
P = \sum_{i=1}^{n_{ps}} A_{ps} f_p \quad \text{(B.7)}
\]

4. Determine the forces in the vertical connector

- Calculate the wall end uplift from the following equation:

\[
\Delta_{radj} = 0.02 (l_{w} - c) \quad \text{(B.8)}
\]

It is assumed that the connector deformation is equal to \( \Delta_{radj} \).

- Determine the corresponding force in the vertical connector \( (F_{vc}) \) from the force-displacement response of the connector.
5. Determine the new neutral axis depth

Assuming a uniform compressive stress acting at the wall base over a length of \( \beta c \), where \( c \) is the neutral axis depth, the resultant compressive force

\[
C = \beta cf_{ce} A_{ce}
\]  

(B.9)

where, \( \beta \) is a constant and varies from 0.8-0.9, depending on the concrete strength and the amount of confinement steel reinforcement provided. For this procedure an average value of 0.85 is assumed for \( \beta \).

- Calculate the resultant compressive force from equilibrium of forces

\[
C = P + N + n_{con} F_{sc}, \text{ for the leading wall,}
\]

\[
= P + N + n_{con} F_{sc} - n_{con} F'_{sc}, \text{ for the intermediate wall, and}
\]

\[
= P + N - n_{con} F'_{sc}, \text{ for the trailing wall.}
\]

\( F'_{sc} \) is the force in the connector corresponding to the previous wall iteration at 2% base rotation.

- Calculate the neutral axis depth:

\[
c = \frac{C}{\beta f_{ce} A_{ce}}
\]  

(B.10)

Iterate the above five sub steps until \( c \) converges.

**Step 4:** Select a base rotation \( (\theta) \). \((0 \leq \theta \leq \theta_{\text{ultimate}})\).

Choose any \( \theta \) value in the range of 0 and \( \theta_{\text{ultimate}} \).

**Step 5:** Determine the forces acting on the wall at base rotation \( \theta \) [Figure B.2]

1. Determine the neutral axis depth \( c_0 \) corresponding to base rotation \( \theta \):

Based on the experimental results, it was found that the neutral axis depth does not significantly vary for interface rotations above 0.5%, as illustrated in Figure B.1. Consistent with this observation, the neutral axis depth in this analysis procedure is calculated at 2% base rotation (Step 3), which is then utilized for all rotations above 0.5%. To model the neutral axis depth more accurately at small \( \theta \), a trilinear variation of neutral axis depth is assumed as a function of base rotation as demonstrated in Figure B.1. In this Figure, the point 1 corresponds to the wall length at 0% base rotation, where as points 2 and 3 are defined at base rotations of 0.1% and 0.5%, respectively. The neutral axis depth \( (c) \) at point 3 is found from Step 3 is equal to neutral axis depth at 2% rotation, and at point 2 is approximated to 2\( c \).
2. Determine stresses and strains in the post-tensioning steel:

- The elongation of the post-tensioning tendon:
  \[ \Delta_{pt,i} = 0.02(x_{pt} - c) \]  
  (B.11)

- The total strain in the post-tensioning tendon:
  \[ \frac{\Delta_{pt,i}}{h_w} + \frac{f_{pt0}}{E_p} \]  
  (B.12)

- Calculate the stress in each post-tensioning tendon \( (f_p) \), from the stress-strain curve of the post-tensioning steel.

  The above steps should be repeated to determine the stress in all post-tensioning tendons.

- The total post-tensioning force:
  \[ P = \sum_{i} A_{pt} f_p \]  
  (B.13)

- Calculate the location of the resultant post-tensioning force from the rocking edge:
  \[ X_{pt} = \frac{\sum_{i} A_{pt} f_p x_{pt,i}}{P} \]  
  (B.14)
Figure B.2 Various forces acting on the jointed wall system at base rotation $\theta$. 
3. Determine the forces in the vertical connector

- Calculate the wall end uplift using following equation:

$$\Delta_{\text{endlift}} = 0.02(l_w - c_0)$$

(B.15)

It is assumed that the connector deformation is equal to $\Delta_{\text{endlift}}$.

- Determine the corresponding force in the vertical connector ($F_{sc}$) from the force-displacement response of the connector.

4. Calculate the resultant compressive force from equilibrium of forces

$$C = P + N + n_{con} F_{sc}, \text{ for the leading wall,}$$

$$C = P + N + n_{con} F_{sc} - n_{con} F_{sc}', \text{ for the intermediate wall, and}$$

$$C = P + N - n_{con} F_{sc}', \text{ for the trailing wall.}$$

$F_{sc}'$ is the force in the connector corresponding to the previous wall calculation at base rotation $\theta$.

5. Calculate the location of the resultant compressive force at the maximum drift:

The resultant concrete compressive force acts at a distance of $\alpha_{\text{lever}}c_0$ from the rocking edge of the wall. The lever arm factor at the maximum drift $\alpha_{\text{lever, max}}$ depends on the concrete strain ($\varepsilon_c$) at the drift.

$$\alpha_{\text{lever, ultimate}} = 0.418 + 0.064 \ln \frac{\varepsilon_c}{\varepsilon_{cr}}$$

$$\varepsilon_c = \frac{1.05 \epsilon_{\theta, \text{ultimately}}}{0.06h_{nc}}$$

where $c$ is the neutral axis depth at 2% base rotation

**Step 6:** Compute the resisting moment of the wall

Taking the moment about the center of the wall,

$$M_{wall} = 0.5l_w F_{sc} + P(X_{pt} - 0.5l_w) + C(0.5l_w - \alpha_{\text{lever}}c_0) \text{ for the leading wall,}$$

$$= 0.5l_w (F_{sc} + F_{sc}') + P(X_{pt} - 0.5l_w) + C(0.5l_w - \alpha_{\text{lever}}c_0) \text{ for the intermediate walls, and}$$

$$= 0.5l_w F_{sc}' + P(X_{pt} - 0.5l_w) + C(0.5l_w - \alpha_{\text{lever}}c_0) \text{ for the trailing wall.}$$

It is assumed that the value of $\alpha_{\text{lever}}$ varies linearly from 0.33 to $\alpha_{\text{lever, ultimate}}$ as the base rotation varies from zero to $\theta_{\text{max}}$.

Steps 2 through 6 should be repeated for each wall before going to step 7.

**Step 7:** Compute the resisting moment of the entire wall system
The moment capacity of the wall system is obtained by summing all the individual wall capacities. Thus,

\[ M_{\text{wallsystem}} = \sum_{1}^{n} M_{\text{wall}} \]  

Though this procedure is developed for the jointed wall systems, it can also be used to predict the lateral load behavior of unbonded post-tensioned precast single walls. For single wall analysis, the connector forces will be set equal to zero. This procedure can also be used to analyze jointed walls with different wall lengths. In this case appropriate wall lengths should be used to estimate the neutral axis depth, connector forces, and wall base moments.

**Parametric Investigation**

This section presents the analytical parametric investigation of the jointed wall system under pseudo-static lateral loads. The results of the parametric study are obtained from the analysis procedure presented in this paper. This parametric investigation considered 21 unbonded jointed wall systems including: the prototype wall (JW1), which is the jointed wall system used in the PRESSS test building, tested at UCSD as a part of the PRESSS program, 21 other walls (JW2 to JW21) with various wall parameters varied systematically to capture their effect on the jointed wall system behavior. The values of the various wall system parameters in all the walls investigated are shown in following Tables.

**Wall System Parameters**

The influence of the following wall system parameters are investigated on the lateral load behavior of the jointed wall system. The parameters are:

1. Total number of connectors across a vertical joint:

   This wall parameter investigation is to determine the effect of the number of connectors in a vertical joint \( n_{\text{con}} \) on the lateral load response of the jointed wall system, when all other parameters are kept unchanged. Table B.1 shows the various wall parameters’ values for the walls considered.

2. Initial prestressing force with constant post-tensioning steel area:

   The purpose of this investigation is to estimate the effect of initial stress in post-tensioning tendons on the jointed wall behavior, while keeping the total post-tensioning steel area in the wall and the wall system constant. Table B.2 provides the details of the wall systems considered for the investigation.
Table B.1  Parametric investigation of number of connectors per joint.

<table>
<thead>
<tr>
<th>Wall</th>
<th>$A_p$ (sq.in)</th>
<th>$f_{pi}$ (ksi)</th>
<th>$P_t$ (kips)</th>
<th>$W_{live}$ (kips)</th>
<th>$N_{con}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>JW2</td>
<td>3.4</td>
<td>50.65</td>
<td>172.2</td>
<td>31.85</td>
<td>12</td>
</tr>
<tr>
<td>JW3</td>
<td>3.4</td>
<td>50.65</td>
<td>172.2</td>
<td>31.85</td>
<td>16</td>
</tr>
<tr>
<td>JW1</td>
<td>3.4</td>
<td>50.65</td>
<td>172.2</td>
<td>31.85</td>
<td>20</td>
</tr>
<tr>
<td>JW4</td>
<td>3.4</td>
<td>50.65</td>
<td>172.2</td>
<td>31.85</td>
<td>24</td>
</tr>
<tr>
<td>JW5</td>
<td>3.4</td>
<td>50.65</td>
<td>172.2</td>
<td>31.85</td>
<td>28</td>
</tr>
</tbody>
</table>

Table B.2  Parametric investigation of initial stress in post-tensioning steel with constant steel area.

<table>
<thead>
<tr>
<th>Wall</th>
<th>$A_p$ (sq.in)</th>
<th>$f_{pi}$ (ksi)</th>
<th>$P_t$ (kips)</th>
<th>$W_{live}$ (kips)</th>
<th>$N_{con}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>JW6</td>
<td>3.4</td>
<td>45</td>
<td>153</td>
<td>31.85</td>
<td>20</td>
</tr>
<tr>
<td>JW1</td>
<td>3.4</td>
<td>50.65</td>
<td>172.2</td>
<td>31.85</td>
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<td>221</td>
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<td>31.85</td>
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<td>3.4</td>
<td>75</td>
<td>255</td>
<td>31.85</td>
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3. Initial prestressing force with variable post-tensioning steel area:

This parametric investigation is to evaluate the initial stress in post-tensioning steel with variable post-tensioning steel area, while constant initial prestressing force is maintained. Four walls are included in this parametric investigation, as shown in Table B.3.

<table>
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<tr>
<th>Wall</th>
<th>$A_p$ (sq.in)</th>
<th>$f_{pi}$ (ksi)</th>
<th>$P_t$ (kips)</th>
<th>$W_{live}$ (kips)</th>
<th>$N_{con}$</th>
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</thead>
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<td>20</td>
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<td>65</td>
<td>221</td>
<td>31.85</td>
<td>20</td>
</tr>
<tr>
<td>JW12</td>
<td>4.25</td>
<td>52</td>
<td>221</td>
<td>31.85</td>
<td>20</td>
</tr>
<tr>
<td>JW13</td>
<td>5.1</td>
<td>43.3</td>
<td>221</td>
<td>31.85</td>
<td>20</td>
</tr>
</tbody>
</table>

Table B.3  Parametric investigation of initial stress in post-tensioning steel with variable steel area.

4. Post-tensioning steel area:

The purpose of this investigation is to understand the effect of post-tensioning steel area variation in the wall system on its lateral load performance. Table B.4 shows the walls considered for this investigation.

5. Number of walls in a jointed wall system:
Table B.4 Parametric investigation of post-tensioning steel area.

The purpose of this study is to understand the effect of varying the number of walls in a jointed wall system, while keeping the total length of the wall system, total area of post-tensioning steel in the wall system, and the total number of connectors constant. Table B.5 shows the details of the walls considered for this investigation.

Table B.5 Parametric investigation of number of walls in jointed wall system.

In this investigation, base shear vs. top-floor lateral displacement responses are used to understand the effects of wall parameters on jointed wall system lateral load behavior. The base moment vs. lateral displacement responses are obtained from the analysis procedure presented in this paper.

Prototype Wall System

The prototype wall considered for the parametric study in this paper is the jointed wall system used in the PRESSS test building. The precast jointed wall system used in the PRESSS building consisted of four 8-in thick wall panels, each $2\frac{1}{2}$ stories tall (18.75-ft) by 9-ft wide. The panels were joined vertically to form two walls separated by a small gap between them. Each wall was secured to the foundation using four unbonded post-tensioning bars (total area of 3.4 sq. in) located at the center of the walls. These two walls were connected by 20 U-shaped flexural plates (also referred to as U-plates or UFP connectors), which were placed in the gap running in vertical direction between the walls. The U-plates were used as the connector because of their ability to maintain force resistance under large displacements and contribute to energy dissipation by flexural yielding. Confinement of the concrete at the base of each wall is provided with the help of hoop reinforcement, which increases the strain.
capacity of the concrete, allowing the walls to undergo large lateral drifts under applied lateral loading. From here on, in this paper, any mention of the connector will refer to the UFP connector.

Discussion of Results

This section presents the results obtained for the parametric study. First, the calculated lateral load response for the prototype wall is presented along with its experimental response. This will validate the accuracy of the analysis procedure presented for the jointed walls' behavior. Then, the effects of the various wall parameters on the lateral load behavior of the jointed wall system are discussed in detail.

Prototype Wall Behavior

Figure B.3 presents the base moment vs. lateral displacement for the prototype jointed wall calculated from the simplified analysis procedure. The figure also presents the base moment response calculated by various analysis procedures proposed by Thomas & Sritharan. At the top floor displacement of 11.5 inch, the calculated base moment capacity is only 2.3% below the isolated experimental wall capacity.

Figure B.3 Comparison of base moment-top floor displacement response envelopes for jointed wall.
Discussion of Parametric Study Results

1. Effect of number of shear connectors in vertical joint:

Figure B.4 shows the effect of varying the number of vertical connectors (in this case UFP connectors) in a vertical joint. From the Figure it is seen that as the total vertical shear force is increased along the vertical joint, the base moment capacity of the jointed wall is increased correspondingly. It can also be noted from Figure that, as the number of connectors in the joint increases the stiffness of the wall system also increases. However, note that one can’t increase the number of connectors as they wish, as an increase in the number connectors with other parameters intact will increase the residual drifts and the possibility of wall lift off.

![Figure B.4: Effect of number of connectors in vertical joint on the lateral load behavior of jointed wall.](image)

2. Effect of initial stress in post-tensioning steel:

The effect of varying the amount of initial stress in steel, with constant post-tensioning steel area can be seen in Figure B.5. From the Figure it is clear that as the initial stress in post-tensioning steel is increased, the decompression moment increased, and correspondingly, the initial stiffness of the wall system increased. The increase in the initial stress in post-tensioning steel also increases the maximum capacity of the jointed wall. However, increasing the initial stress can not be more than a certain value, as beyond which post-tensioning steel will yield before reaching a required displacement, causing a permanent residual displacement.
3. Effect of the initial stress in post-tensioning steel (with initial post-tensioning force constant):

Figure B.6 presents the effect of varying the amount of initial stress in steel, while the total initial post-tensioning force is kept constant. The decompression moment is unaffected with the increase in the initial stress in post-tensioning steel, as it depends on the initial post-tensioning force only. As the initial stress in post-tensioning steel is increased, the total post-tensioning steel area is decreased, which leads to the decrease in the moment capacity of the wall system.

4. Effect of total post-tensioning steel area:

Figure B.7 presents the effect of varying total post-tensioning steel area on the response of the jointed wall system. As the area of post-tensioning steel increases, the decompression moment also increases, as the total initial post-tensioning force is increased with the increase of the post-tensioning steel area. The moment capacities at higher drift levels also increases as the post-tensioning steel area increase. Note that the overall shape of the response curve is very similar for all the cases.

5. Effect of number of walls in jointed wall system:

Figure B.8 shows the effect of splitting the jointed wall into more walls on the lateral load behavior of the jointed wall system. From the Figure we can observe that, as the numbers of walls increased, the base moment capacity decreases drastically. From the two-wall jointed system to the three-
Figure B.6 Effect of initial stress in post-tensioning steel on the lateral load behavior of jointed wall.

Figure B.7 Effect of area of post-tensioning steel on the lateral load behavior of jointed wall.
wall jointed system, the moment capacity is decreased by nearly 50%. As the number of walls in the jointed system increased, the decompression moment decreased, and correspondingly, the initial stiffness of the jointed wall system decreased.

![Graph showing the effect of splitting of walls on the lateral load behavior of jointed wall system.](image)

Figure B.8 Effect of splitting of walls on the lateral load behavior of jointed wall system

Overall, from the parametric study it is found that the increase in number of connectors in a vertical joint, increase in post-tensioning steel area, and increase in initial stress in post-tensioning steel, increases the lateral load capacity. Whereas the splitting of the wall system (increasing the number of walls) decreases the lateral load capacity of the wall system. The efficiency of the multi-wall (more than two walls) jointed system can be improved by having unequal lengths for walls and different amounts of post-tensioning steel in different walls. That is, for a three-wall jointed system, the intermediate wall length is more than that of trailing and leading wall (which are kept the same length for symmetry), and the amount of post-tensioning steel area in intermediate wall is more than that of the leading and trailing walls. Consider a three-wall jointed system (JW20) with an intermediate wall length of 120 inches, and leading and trailing wall lengths of 48 inches. The post-tensioning steel area in the intermediate wall is 4.2 sq. in. and remaining 2.6 sq.in of steel is provided in the leading and trailing walls. The stresses in the post-tensioning steel in the intermediate, leading, and trailing wall.
are 55 ksi, 95.2 ksi, and 95.2 ksi, respectively.

The calculated response of the JW20 wall system is shown in Figure B.9. From the Figure, it can be seen that the initial stiffness of the wall system JW20 is increased drastically compared to JW18. The moment capacity is also increased by nearly 20% when compared to JW18. Also note that the jointed wall system with unequal wall lengths has better self-centering capability compared to the jointed wall with equal wall lengths. Though the moment capacity is increased by the unequal splitting of the jointed wall system, it does not quite reach the capacity of the two-wall jointed system. In order to achieve that, the length of the intermediate wall has to be further increased, thus still decreasing the leading and trailing wall lengths. This leads to a new concept of jointed system called "jointed wall with end columns."

**Jointed Wall With End Columns**

The jointed wall with end columns system consists of an unbonded single precast wall connected to two end columns with the help of special connectors along the vertical joints, as shown in Figure B.10. Unbonded post-tensioned steel is located symmetrically across the cross-section of the wall. The
precast wall and the columns are tied to the foundation with the help of post-tensioning steel.

Now consider a jointed wall with end columns system (JW21) which has the following properties:

![Diagram of jointed wall with end columns system](image)

**Figure B.10  Jointed wall with end columns system**

length of the precast wall is 168 inches and all other properties are same as the prototype wall (JW1). The end column dimensions are 24 inches by 24 inches. Each end column contains 1 sq. in of post-tensioning steel and the precast wall contains 4.8 sq.in of post-tensioning steel, so that the total amount of post-tensioning steel is same as the prototype wall system. Each vertical gap between single wall and the column consists of 10 connectors. The Figure B.11 shows the lateral load response of JW21.

In the jointed wall with end column system, the connectors used (UFP) for analysis reached their maximum capacity by 2% drift. So, the system response in Figure B.11 is limited to 2% drift. From Figure B.11, one can see that the jointed wall with end columns has the same capacity as the two-wall jointed system. The new system has higher initial stiffness and greater self-centering capability compared to the two-wall jointed system. Thus, this new jointed wall with end columns system is more efficient in resisting lateral loads when compared to the jointed wall system.
Conclusions

Based on the results of the analytical study, the following conclusions can be drawn:

1. The analysis procedure predicts the jointed wall system behavior satisfactorily.

2. The parametric study of 20 walls indicated that the lateral load behavior of the jointed wall can be controlled by controlling various wall system parameters.

3. The splitting of the wall system into more walls is found to be inefficient. The two-wall system is more efficient jointed wall system compared to a multi-wall jointed system with equal wall lengths. So, given a choice, a two-wall jointed system is preferred over a multi-wall jointed system.

4. In the case of a multi-wall jointed system, unequal wall lengths systems are found to be more efficient compared to equal wall length systems.

5. The jointed wall with end columns system is an ideal and efficient system to resist the lateral loads.
APPENDIX C. ACCURACY OF MOMENT CONTRIBUTION FACTOR FOR JOINTED WALLS

Introduction

In Chapter 4, an equation was presented to determine the design moment of the critical wall in a jointed wall system with equal wall lengths. The equation is,

\[ M_{\text{design},w} = \Omega \frac{M_{\text{design}}}{n\phi} \]  \hspace{1cm} (C.1)

\[ \Omega = 1 + \frac{\alpha \phi N_{\text{con}} F_{\text{con}} L_w}{M_{\text{design}}} \]  \hspace{1cm} (C.2)

where, \( n \) is the number of walls,
\( \Omega \) is the moment contribution factor, and
\( \alpha \) is a constant.

When \( n = 2 \), \( M_{\text{design,wall}} = M_{\text{design,lead}} \) and \( \alpha = 0.9 \)

When \( n \geq 3 \), \( M_{\text{design,wall}} = M_{\text{design,inte}} \) and \( \alpha = 1.03 \)

The accuracy of Equation C.1 is examined by considering a three-wall system with various properties (see Tables C.1) and comparing the calculated value of \( \frac{M_{\text{design,wall}}}{M_{\text{design,lead}}} \) for three-wall jointed system with the values obtained from the simplified analysis procedure at a design drift of 2%. The comparison of results are shown in Figure C.1. From this Figure and Table C.1, it is seen that the Equation C.1 estimate the \( \frac{M_{\text{design,wall}}}{M_{\text{design,lead}}} \) value satisfactorily.
<table>
<thead>
<tr>
<th>$L_w$ (inch)</th>
<th>$t_w$ (inch)</th>
<th>$A_{steel}$ (inch$^2$)</th>
<th>$F_{pu}$ (ksi)</th>
<th>$N_{con}$ (kips)</th>
<th>$Fron$ (kip-in)</th>
<th>$Minter$ (kip-in)</th>
<th>$M_{total}$ (kip-in)</th>
<th>$\frac{M_{inter}}{M_{total}}$ (Analysis)</th>
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</table>

Table C.1 A three-wall jointed systems with different design variables.

Figure C.1 Comparison of calculated $\frac{M_{inter}}{M_{total}}$ for a three-wall jointed system with Equation C.1 and the simplified analysis procedure.
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