An Evaluation of Seismic Design Guidelines Proposed for Precast Jointed Wall Systems

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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. The primary objective of the research presented in this report is to evaluate the adequacy of the design guidelines and make appropriate recommendations so that the guidelines can be adopted for design of jointed precast walls in seismic regions. The test data to date on such systems are those collected during the wall direction testing of the PRESSS test building. Hence, this data set has been employed in the validation process.

Disciplines
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D. J. Thomas, S. Sritharan

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REPORT

IOWA STATE UNIVERSITY
OF SCIENCE AND TECHNOLOGY

Department of Civil, Construction and Environmental Engineering
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Proposed for Precast Jointed Wall Systems

by

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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. The primary objective of the research presented in this report is to evaluate the adequacy of the design guidelines and make appropriate recommendations so that the guidelines can be adopted for design of jointed precast walls in seismic regions. The test data to date on such systems are those collected during the wall direction testing of the PRESSS test building. Hence, this data set has been employed in the validation process.

Furthermore, in order to validate the design guidelines over a range of lateral displacements, an analytical procedure was first developed by reversing the suggested guidelines. Additionally, because of the shortcomings associated with the use of the equivalent stress block concept in design, an alternative analysis method was also considered, which was based on the monolithic beam analogy (MBA) originally developed for jointed frame systems. The analytical results from monotonic loading were compared to experimental response of the jointed wall established from the PRESSS test data.

It was found that the analysis method based on the PRESSS guidelines underestimated the lateral load resistance of the jointed wall in the PRESSS test building by up to 22% at large lateral displacements and overestimated by greater percentages at small displacements. When
the force transferred through the shear connectors was based on the measured force-displacement response, the PRESSS guidelines and MBA underestimated the moment response of the jointed wall by 12% and 5%, respectively, at the maximum measured lateral displacement. However, utilizing the PRESSS guidelines with the measured force in the shear connectors, it was found that the neutral axis depth was overestimated by over 100% and the post-tensioning elongation was underestimated by 26%. It was also revealed that the framing action contributed to the moment resistance in the wall direction of the PRESSS building by as much as 25%. Several recommendations are provided for improving the PRESSS design guidelines, which will simplify the design calculations while providing efficient design details for the precast jointed wall systems. With improvements, the PRESSS guidelines are shown to predict the observed performance of the jointed wall in the PRESSS test building satisfactorily.
ACKNOWLEDGEMENTS

The research described in this report was funded by the Precast/Prestressed Concrete Manufacturers Association of California (PCMAC), which is greatly acknowledged. Additionally, the authors would like to thank Professor John F. Stanton of the University of Washington and Suzanne D. Nakaki of the Nakaki Bashaw Group, Inc. for their assistance during this project.

Conclusions, opinions and recommendations expressed in this report are those of the authors alone, and should not be construed as being endorsed by the financial sponsor.
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<td>E_p</td>
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<td>E_{sec}</td>
<td>Secant modulus of confined concrete at peak stress</td>
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<td>F</td>
<td>Vertical interface shear force</td>
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<td>f_c</td>
<td>Concrete stress</td>
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<tr>
<td>f'_c</td>
<td>Concrete compressive strength</td>
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<td>f_{cc}</td>
<td>Confined concrete compressive strength</td>
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<td>F_{decomp}</td>
<td>Horizontal force on the wall that causes decompression at wall base</td>
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<td>$f'_l$</td>
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<td>$f_{p,des}$</td>
<td>Stress in post-tensioning tendon at design limit state</td>
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<td>$f_{p0}$</td>
<td>Initial stress in the post-tensioning tendon</td>
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<tr>
<td>$f_{pi}$</td>
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<td>$f_{py}$</td>
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<td>$F_{sc0}$</td>
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<td>$F_u$</td>
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<td>$F_V$</td>
<td>Vertical force induced by actuator</td>
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<td>$f_{ye}$</td>
<td>Expected yield strength</td>
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<td>$f_{yh}$</td>
<td>Yield strength of transverse reinforcement</td>
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<td>$G$</td>
<td>Shear modulus</td>
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<tr>
<td>$h_{\text{eff}}$</td>
<td>Height above foundation of lateral load resultant on wall</td>
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<tr>
<td>$H_{\text{story}}$</td>
<td>Height per story</td>
</tr>
<tr>
<td>$H_t$</td>
<td>Wall height</td>
</tr>
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</table>
\( h_u \) Unbonded length of post-tensioning tendon

\( h_w \) Wall height

\( I \) Moment of inertia

\( I_{eff} \) Effective moment of inertia

\( k_1 \) Uniform stress in Whitney rectangular stress block divided by \( f_g' \)

\( K_e \) Confinement effectiveness coefficient

\( L_p \) Equivalent plastic hinge length

\( l_w \) Length of one wall panel

\( l_w,_{tot} \) Total wall length

\( M \) Moment resistance of wall

\( M_{cap,panel} \) Moment capacity of one panel

\( M_{cap,wall} \) Total moment capacity of wall system

\( M_{decomp} \) Base 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

\( M_{sc} \) Plastic moment strength of one UFP connector

\( M_u \) Factored moment

\( n \) Number of wall panels

\( N \) Total tension force on one wall panel from gravity plus post-tensioning
$N_0$ 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

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

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

$n_{story}$ Number of stories

$N_{u fp}$ Number of UFP connectors 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

$P_{p0}$ Wall panel post-tensioning force

$P_u$ Factored axial load

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

$R$ Response modification factor

$R$ Average radius of the UFP

$S$ Section modulus

$t$ Wall thickness

$t$ Plate thickness in UFP connector

$t_{sc}$ Plate thickness in UFP connector

$t_w$ Thickness of wall panel

$t_{w,eff}$ Thickness of wall panel effective in resisting compressive force
V  Base shear
V  Force in each UFP connector
V_1  Force applied to 1st floor by the actuator
V_2  Force applied to 2nd floor by the actuator
V_3  Force applied to 3rd floor by the actuator
V_4  Force applied to 4th floor by the actuator
V_5  Force applied to 5th floor by the actuator
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\textsuperscript{th} floor by the actuator
V_llp  Base shear capacity at the yielding state
V_max  Expected maximum base shear demand under survival level ground motion
V_sc  Shear strength of one UFP shear connector
V_ss  Shear slip capacity
v_u  Factored shear stress
W  Self weight of wall panel
W  Total gravity load from all floors on one panel
W_{panel}  Self-weight of one panel
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<td>$W_{\text{story}}$</td>
<td>Building weight per floor</td>
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<td>$\alpha$</td>
<td>distance from the compression face of the member to the center of the</td>
</tr>
<tr>
<td></td>
<td>compression force divided by the member depth</td>
</tr>
<tr>
<td>$\alpha_0$</td>
<td>distance from the compression face of the member to the center of the</td>
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<tr>
<td></td>
<td>compression force divided by the member depth at zero drift</td>
</tr>
<tr>
<td>$\alpha_{\text{des}}$</td>
<td>distance from the compression face of the member to the center of the</td>
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<tr>
<td></td>
<td>compression force divided by the member depth at design limit state</td>
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<td>$\beta$</td>
<td>Moment arm reduction coefficient</td>
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<td>$\beta_1$</td>
<td>Depth of equivalent stress block divided by the neutral axis depth</td>
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<td>$\Delta$</td>
<td>Roof drift</td>
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<td>$\delta_{\text{all}}$</td>
<td>Allowable story drift defined by NEHRP</td>
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<td>$\Delta_{\text{csc}}$</td>
<td>Roof drift capacity at the failure state</td>
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<td>$\Delta_{\text{ctc}}$</td>
<td>Roof drift capacity corresponding to the crushing of the concrete inside the</td>
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<td>Roof drift capacity at decompression state</td>
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<td>$\Delta_{\text{ell}}$</td>
<td>Roof drift capacity at softening state</td>
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<td>$\Delta_{\text{enduplift}}$</td>
<td>Wall end uplift</td>
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<tr>
<td>$\Delta_g$</td>
<td>Maximum roof drift that can be sustained by gravity load system</td>
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<td>$\Delta f_p$</td>
<td>Increase in stress in post-tensioning tendon</td>
</tr>
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\( \Delta f_{p,\infty} \) 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

\( \Delta_p^p \) 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 \( f_{cc}^* \)

\( \varepsilon_{fracture} \) Minimum guaranteed fracture strain in a UFP connector

\( \varepsilon_{max} \) Maximum strain to which the UFP will be subjected

\( \varepsilon_{sc,des} \) Strain in the UFP connector plate at the design limit state

\( \varepsilon_{sc,max} \) Maximum permissible strain in UFP connector under cyclic loading

\( \phi_e \) Elastic curvature

\( \phi_{e^*} \) elastic curvature at the base of a precast jointed wall

\( \phi_f \) Reduction factor

\( \phi_p \) Plastic curvature

\( \phi_s \) Capacity reduction factor
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<td>Ultimate curvature</td>
</tr>
<tr>
<td>$\phi_y$</td>
<td>Yield curvature</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td>Density of concrete</td>
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<td>$\eta$</td>
<td>Distance from the compression face to the neutral axis depth divided by member depth</td>
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<tr>
<td>$\eta_0$</td>
<td>Distance from the compression face to the neutral axis depth divided by member depth at zero drift</td>
</tr>
<tr>
<td>$\eta_{des}$</td>
<td>Distance from the compression face to the neutral axis depth divided by member depth at design limit state</td>
</tr>
<tr>
<td>$\kappa_0$</td>
<td>Ratio of design strength of shear connectors in one vertical joint to the vertical load on one panel</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Base rotation</td>
</tr>
<tr>
<td>$\theta_{des}$</td>
<td>Base rotation at design limit state</td>
</tr>
<tr>
<td>$\rho_{fp0}$</td>
<td>Stress ratio to ensure that post-tensioning tendon does not yield at maximum drift</td>
</tr>
<tr>
<td>$\rho_{MOM}$</td>
<td>Demand/capacity ratio for overturning moment on panel</td>
</tr>
<tr>
<td>$\rho_{ROC}$</td>
<td>Force ratio to ensure that the panel rocks rather than slides</td>
</tr>
<tr>
<td>$\rho_{UPL}$</td>
<td>Ratio of uplift force to hold down force on one panel</td>
</tr>
<tr>
<td>$\rho_x$</td>
<td>Transverse reinforcement ratio in 'x' direction</td>
</tr>
<tr>
<td>$\rho_y$</td>
<td>Transverse reinforcement ratio in 'y' direction</td>
</tr>
<tr>
<td>$\rho_{ZRD}$</td>
<td>Parameter ratio controlling the residual drift</td>
</tr>
</tbody>
</table>
\( \sigma_0 \) Initial stress at the base of the wall

\( \zeta \) Damping of the system

\( \zeta_{\text{friction}} \) Damping of the system using frictional inter-panel connectors

\( \zeta_{\text{ufp}} \) Damping of the system using UFPs as inter-panel connectors
Chapter 1: Introduction

1.1 Overview

Concrete walls have many advantages, particularly in seismic prone regions, and with the added benefits of prefabrication, precast concrete wall systems could be an excellent choice for designing earthquake resistant buildings. However, the application of precast systems in general is limited in seismic prone regions due to the lack of research information and constraints imposed by current design codes. This chapter discusses the lead up to the research of unbonded post-tensioned jointed precast concrete walls in seismic regions, including past performance of shear walls, benefits and limitations of precast concrete, as well as a brief overview of the PREcast Seismic Structural Systems (PRESSS) research program.

1.2 Past Performance of Shear Walls in Seismic Events

Shear walls have long been known for their triple function to support gravity loads, provide lateral resistance and function as a wall, along with their exceptional performance in seismic events [1]. Fintel, author of Shearwalls—An Answer for Seismic Resistance?, has investigated and reported on many earthquakes between 1960 and 1990 and reports being unaware of a single concrete building containing shear walls that experienced earthquake collapse [2]. Although there were cases of cracking in various degrees, no lives are known to have been lost in such buildings.
In the 1960 Chile earthquake, registering a 9.5 on the Richter scale [3], the report states, “...the Chilean experience confirms the efficiency of concrete shear walls in controlling structural and nonstructural damage in severe earthquakes. There were instances of cracking of shear walls, but this did not affect the overall performance...the walls continued to function after damage had occurred...” [2].

The epicenter of the 1963 earthquake in Skopje, Yugoslavia, was located directly under the city with a magnitude of 6.3 on the Richter scale [3]. Following the event, unreinforced concrete walls were found to have no damage due to interstory drifts imposed by the earthquake; however, several frame buildings collapsed and many were damaged severely by this earthquake [2]. For example, the 14-story Party Headquarters Building, which uses structural wall-frame interaction for lateral resistance, swayed significantly pushing desks from one end of the building to the other, but the building remained standing without any structural or non-structural (i.e. broken windows) damage [3].

In the Caracas, Venezuela, earthquake of 1967, a Richter scale magnitude of 7.0 was recorded, with the epicenter located off the coast in the Caribbean Sea [3]. A primary example from this event is the Plaza One Building (see Figure 1.1), the only building containing structural walls, which survived the event without any damage. Several frame buildings surrounding it collapsed while others experienced excessive damage [2]. According to Fintel, the most significant observation following this event was that buildings containing shear walls outperformed the buildings made from flexible frames [3].
During the 1971 San Fernando California earthquake, having a magnitude of 6.8 on the Richter Scale [3], a six-story building (Figure 1.2) with shear walls located in an area of severe damage endured the event with minor cracks, while an eight-story neighboring concrete frame structure (Figure 1.3) was damaged so severely that it was eventually demolished [2]. The resulting conclusion by Fintel was that shear wall-type structures show a superior response to earthquakes, compared with frame-type structures, by limiting the interstory drifts [3].
Figure 1.2 Six-story Indian Hill Medical Center with shear walls [4].

(a) View of Holy Cross Hospital  (b) Examples of damage to the building

Figure 1.3 Damaged concrete frame building in the 1971 San Fernando earthquake [4].
A similar case existed during the 1972 Managua, Nicaragua, earthquake, 6 on the Richter scale [3], where two buildings were located across the street from one another in an area that experienced severe damage and thus were subjected to the same intensity of ground motion. The first building, a 14-story concrete framed structure, was demolished after enduring extreme damage caused by the earthquake, while the nearby second building, an 18-story structure containing concrete walls, suffered no significant structural or non-structural damage aside from one coupling beam that failed in shear [2]. The superior performance of the 18-story concrete wall structure in addition to the experience from past earthquakes, began a movement towards changing the U.S. codes to incorporate shear wall buildings and shear wall-frame interactive systems into American seismic codes [3].

In 1977 an earthquake measuring 7.2 on the Richter scale occurred 130 miles north of Bucharest, Romania [3]. This event caused 35 buildings to collapse, while hundreds of buildings containing concrete wall systems withstood the earthquake, the majority with no damage [2]. Similarly, in the 1985 Mexico City earthquakes (measuring 8.1 and 7.5 on the Richter scale and located 250 miles from the epicenter [3]) 280 buildings collapsed and in the 1988 Armenia earthquake (measuring 6.9 on the Richter scale and located near the city Spitak [3]) 72 buildings collapsed and 149 buildings were severely damaged, none of which contained structural walls [2]. However, 21 buildings containing shear walls have been reported to have experienced the earthquake undamaged in the Armenia earthquake.
In the 1985 Chile earthquake very little damage was evidenced even though the magnitude of the event was similar to that of the Mexico City earthquake. Fintel attributed the relatively low damage to the common practice of using concrete shear walls to control lateral drift in Chile, and suggested that the use of structural walls could minimize the damage to structures during a seismic event [2].

The observations summarized above from several earthquakes demonstrate the exceptional performance of buildings with shear walls in seismic regions. For this reason, Fintel postulates that controlling damage to concrete buildings in severe earthquakes is difficult without incorporating structural walls, and that it is the responsibility of the engineering profession to ensure that at least residential buildings in seismic regions are constructed using structural walls [2].

1.3 Benefits of Precast Walls

The use of precast concrete in the construction of structures is beneficial in many ways. Since precast concrete is cast in a factory-like setting, the quality of construction is much higher than cast-in-place construction in which the concrete is cast at the job site. This is primarily attributed to the controlled setting in which the construction and quality control takes place. The setting also increases the ability of the construction workers to accurately follow the design specifications with a supervisor present, who is more readily available than an on-site inspector may be, to assist with quality control. The manufacturing process used for precast concrete also makes prestressing and placement of post-tensioning ducts more convenient. The equipment necessary for prestressing in precast plants is already set up for this type of
operation to be performed easily and accurately while placement of the post-tensioning ducts is further simplified by casting walls horizontally and due to the improved quality in the construction of the reinforcement cages.

Another more obvious advantage of precast concrete is the reduction in site formwork and site labor, caused by off-site pouring, which in turn increases the speed of on-site construction. Also, the availability of cranes with increased capacity, new construction techniques and off-site fabrication has made it easy for contractors to adapt to precast construction [5]. There are also many less obvious advantages of precast construction such as a decrease in traffic congestion, air pollution and noise pollution [6]. Traffic is reduced near the job site due to the decrease in required concrete trucks and construction workers. This, and reduced traffic congestion, reduce the air and noise pollution at the construction site.

Precast construction has also been described as the building process of the future [7]. This is because (1) the materials are relatively inexpensive, (2) the method of construction, involving factory manufacture of components and rapid construction, lends itself to innovation in design and construction, and (3) advanced technology including robotics and use of computer aided manufacturing can be easily adaptable to increase efficiency of the construction practice and erection procedures, ultimately reducing building costs.

1.4 Limitations of Precast Concrete in Seismic Regions

There are several limitations that lead many designers to consider precast concrete as inferior to cast-in-place concrete in seismic regions, and thus discard it as a possible design option.
The primary limitation originates from the poor performance of precast concrete structures in past seismic events. This includes poor performance of many framed structures consisted of precast components, which are primarily due to the use of deficient connection details, ineffective load paths in building design, and poor construction practices [5,8,9]. For example, several precast structures in the 1988 Armenian earthquake performed poorly. Most of the poor performance of precast structures was attributed to substandard materials and construction practices as well as the use of insufficient connection details [7,9]. However, it is conceivable that the poor performance of these structures has generally contributed towards decreasing designers’ confidence in the precast construction practice in seismic regions.

There is also a lack of design procedures and engineers with seismic design experience in precast concrete. For example, the ACI 318 standard, through its 1999 edition, does not contain seismic design provisions for precast concrete, but permit the design of precast structures that are equivalent to monolithic concrete structures [10]. Similarly, in the most recent edition, ACI 318-02 [11], emulation is required by Sections 21.6 and 21.8 for precast frames and precast walls, respectively, but special precast frame systems are permitted in seismic regions with the penalty of requiring experimental verification.

The current precast design provision requiring engineers to use reinforced concrete emulation does not fully utilize the advantages of the material strengths and unique properties of precast construction [7,12]. Following past earthquakes, it has been observed that under large lateral displacements, significant damage occurred in precast concrete structures that were designed...
using cast-in-place concrete emulation [13]. These incidences could have been likely due to use of poor connection details. However, they contribute towards designers developing uncertainties about the seismic performance of precast concrete structures.

Therefore, a reliable design procedure for precast concrete must be developed in order for this technology to be widely accepted in seismic regions. Economical and effective means have been under investigation for joining precast concrete elements together to resist seismic actions [7], along with establishing design and construction techniques, and promoting confidence in the use of precast concrete as an option for seismic design around the world.

1.5 Unbonded Precast Wall Systems

In considering the exceptional performance of structural walls in past earthquakes (Section 1.2), the benefits of precast concrete (Section 1.3) and the limitations that must be overcome (Section 1.4), the PREcast Seismic Structural Systems (PRESSS) research program was initiated in the United States in the early 1990s. This program 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 the various seismic zones, and (2) To develop new materials, concepts and technologies for precast construction in the various seismic zones [7].

A concept development workshop was held in April 1991 to obtain input from precast concrete producers, design engineers and contractors on the concept developments and connection classification projects of PRESSS [14]. A 6-story hotel with a structural wall
system was one of the design concepts discussed at the workshop. The structural wall concept was split into two designs: the reinforcing bar structural wall system and the post-tensioned structural wall system. The reinforcing bar structural wall system consists of one-story panels connected at each floor with welded or sleeved reinforcing bars which resist overturning and panel-to-panel structural forces. The post-tensioned structural wall system, the system selected for the PRESSS test building [12,14,15,16] and the focus of the research presented herein, also consists of one-story panels stacked on top of one another, but uses vertical unbonded post-tensioning through the panels.

Upon completion of the concept development workshop, and various testing and analytical models in the first two phases, a 60% scale five-story precast concrete test building was designed, built and tested for seismic resistance at the University of California at San Diego (UCSD) in Phase III of the PRESSS program. Through tests of this large-scale building, the goal of the PRESSS researchers envisioned to meet the following four objectives: (1) validate a rational design procedure for precast seismic structural systems, (2) provide acceptance of precast prestressed/post-tensioned seismic structural 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 [12].

The test building consisted of two types of systems. In one direction, the frame direction, the building consisted of two seismic frames based on four different frame connections, while a jointed precast wall system was used in the perpendicular direction [12] (see Figure 1.4).
jointed structural wall system selected included two walls and was based on the unbonded post-tensioned structural wall concept. The walls were made from a total of four precast panels, each 2 ½ stories tall (18.75-ft) by 9-ft wide by 8-in thick (Figures 1.5 and 1.6). The panels were joined vertically to form two walls separated by a small gap. Each wall was secured to the foundation using four unbonded post-tensioning bars. They were then connected horizontally by 20 U-shaped flexural plates (also referred to as UFP connectors, see Figures 1.5 and 1.7) placed along the vertical joint between the walls.

The basic concept of the wall system was to allow the walls to rock individually at the base for a ground excitation of significant magnitude and return to its vertical position after the event has concluded. The vertical post-tensioning contributed to overturning moment resistance and also enabled transfer of shear forces between the walls and the foundation through a friction mechanism. The U-plates, which were selected for their ability to maintain their force capacity through large displacements, were used as vertical joint connectors, as well as damping devices for the wall system that was achieved by flexural yielding of the plates [12].
Figure 1.4 Floor plans of the PRESSS test building [12].
**Figure 1.5** Elevation view of the jointed wall system in the PRESSS test building [12].
Figure 1.6 The PRESSS building after erecting the wall system [15].
In order to test the building, lateral forces and displacements were applied by two actuators connected to the floors at each level. With displacement control, this test setup enabled the actuator loads at each level to be equally distributed to each bay of the building during the wall direction testing. The PRESSS building was tested in the wall direction first followed by the frame direction [16]. The primary test method of the building was pseudodynamic testing, which used modified segments of recorded accelerograms. These segments were chosen to match the target acceleration spectra corresponding to 5% damping, which represented four limit states recommended for performance based design by the Structural Engineers Association of California (SEAOC) [17]. The input motions at the four levels...
correlated with 33% (EQ-I), 50% (EQ-II), 100% (EQ-III) and 150% (1.5EQ-III) of the design-level earthquake. The other two types of tests conducted included the stiffness measurement test and inverse triangular load test [16]. The inverse triangular load test, which exercised the building through a displacement profile similar to that expected for the fundamental mode response, was applied in the wall direction testing at the end of the pseudodynamic tests to EQ-I, EQ-II and modified EQ-III input motions. They were referred to as IT-I through IT-III tests.

As depicted for a generalized wall system with three walls in Figure 1.8, forces (V₁ to V₅) applied to the floor levels by the actuators transmit inertia effects to the wall system, as expected during a seismic event. The overturning moment caused by these forces (V₁ to V₅) is resisted at the base of the walls by the tension force in the post-tensioning tendon plus the self weight of the wall (N₁des), the compressive force in the concrete (C₁des) and the force introduced to the wall by the UFP connectors (Fsc) (Figure 1.8). By design, the post-tensioning tendons remain elastic, and the walls are expected to return to their initial position upon removal of the lateral forces, resulting in minimal damage and little residual drift, leaving the structure prepared for the next seismic event.

The observed base moment-top floor lateral displacement of the PRESSS building in the wall direction is shown in Figure 1.9. The wall direction response was reported to have exceeded the expectations under the design level or EQ-III earthquake [15]. The expected displacement at the roof level was 9-in, while the recorded peak displacement was 8% lower at 8.3-in. When subjected to 150% of the design-level event (i.e. 1.5EQ-III), the top floor experienced
a maximum displacement of 11.5-in with minimal damage to the wall, where spalling of some cover concrete occurred at the wall toes, requiring minor repair. As a result of the minor damage and low residual drift (0.06%), it was clear that at 150% of the design-level earthquake, the building had responded within the serviceability level of performance [15].

![Diagram of forces acting on a jointed wall system](image)

**Figure 1.8** Forces acting on a jointed wall system, adopted from [18].

Upon the good response of the PRESSS building in the wall and frame directions, Stanton and Nakaki published a set of design recommendations for the precast concrete frame and wall systems incorporated in the PRESSS test building [18]. The recommendations proposed for the wall system have not been evaluated using experimental results. With details of the
recommended guidelines in Section 2.5.2, the study presented in this report examines the validity of the proposed guidelines.

![Graph showing Fifth floor displacement (mm) vs Base moment (kip-in) and Fifth floor displacement (in) vs Base moment (kN-m).]

**Figure 1.9** Overall response of the PRESSS building in the wall direction.

### 1.6 Scope of Research

The main objective of this report is to validate the design guidelines reported by Stanton and Nakaki [18]. The experimental data available to date for precast jointed systems are those collected during the wall direction testing of the five-story PRESSS test building. Hence, this data set is included in the validation process. The validation of the experimental data is performed using two analytical procedures, one of which is based on the design guidelines proposed by Stanton and Nakaki as part of the PRESSS research. In the validation process,
overall moment response, post-tensioning steel elongation and neutral axis depth are compared, and appropriate recommendations are suggested to improve the proposed PRESSS guidelines for seismic design of jointed precast wall systems.

1.7 Report Layout

This report contains five chapters including the introduction presented in this chapter. The following chapter contains a literature review, which includes a brief summary of previous investigations on the analysis and design of precast seismic wall systems. This is followed by a chapter entitled Formulation of Validation Procedure. An analytical procedure developed to validate the PRESSS design guidelines and an alternative analysis procedure that may be used to establish an alternative design method are presented in this chapter. Next, an experimental verification chapter compares results from the analytical models with the experimental data and concludes with recommendations to improve design guidelines proposed for jointed precast wall systems by Stanton and Nakaki [18]. Finally, the report closes with a final chapter, which contains an overview of the report, as well as conclusions of the research results and recommendations for future research.

1.8 Wall Definitions

A ‘wall system’ in this report defines all of the components, including the walls and connector elements as shown in Figure 1.8. In the direction of lateral loading, the first wall that leads the wall system response is defined as the ‘leading wall’. Similarly, the last wall in the loading direction is referred to as the ‘trailing wall’, and the ‘intermediate walls’ are those located between the first and last walls. At the toes of the wall, the maximum and minimum
compressive forces are expected in the leading and trailing walls, respectively, as shown in
Figure 1.8. Each wall may consist of several ‘wall panels’ stacked vertically as shown in
Figures 1.5 and 1.6. In accordance with these definitions, the wall system of the PRESSS
building consisted of a leading and trailing wall with no intermediate walls. These definitions
will be used throughout this report.
Chapter 2: Literature Review

2.1 Precast Seismic Wall Systems

The benefits of employing shear walls as the main system for resisting lateral forces in design of buildings have been long realized, but until recently the potential use of precast wall systems to resist seismic forces has not been extensively researched. Many researchers have recently advocated the advantages of precast concrete walls, noting additional benefits when detailing them with unbonded post-tensioning. This includes improved performance of the wall system under seismic loading by reducing damage and residual displacements. In order to adopt these advantages, the design methods and suitable analysis techniques for precast concrete wall systems must be established. In this chapter a literature review pertaining to the analysis and design of precast seismic wall systems is summarized.

2.2 Precast Seismic Design Considerations

The use of precast concrete in seismic regions is primarily hindered by the lack of established design concepts that can fully benefit the precast concrete technology and qualified precast designers. Considerations to improve the use of precast concrete in seismic regions and design issues at the conceptual level are addressed within this section.

Englekirk [19]

According to Englekirk the primary constraint facing the precast concrete industry is the requirement that it must comply with a technology developed for cast-in-place concrete (i.e.
a precast concrete system is required to emulate behavior of a comparable cast-in-place system as discussed further in Section 2.3). Therefore, the precast industry must develop a seismic design technology that permits intelligent use of precast components with suitable connections. Englekirk suggested that four questions must be considered to increase precast concrete use in seismic regions. They are:

1. How must precast concrete buildings be erected if they are to be competitive?
2. What precast concrete products are most versatile and competitive?
3. What production techniques could be used to manufacture these precast concrete products?
4. What connector concepts streamline production and erection?

These questions were addressed by the National Institute of Standards and Technology (NIST [20]), who developed a program which defined the major processes necessary for precast to be more successful in seismic regions. In response to the first question, it was concluded that there is a need for high rise precast concrete buildings whose seismic resistance should be provided entirely by ductile frames. Along with this, erection needs must be developed to mimic those of a structural steel frame. Essential elements for this procedure include: a two floor erection process, alignment capabilities and early integration of the final bracing system. In terms of product versatility, the use of rectangular components for beams and columns was strongly encouraged. In addition to this, prestressing was recommended to improve handling, control member deflection and to reduce cost. Addressing connector concept criteria, the following list was developed:

- Avoid extensive welding and the associated embedment hardware,
• Incorporate adequate tolerances,
• Avoid large formed wet joints, and
• Design joints that minimize crane time.

In addition to the items necessary for precast to excel in seismic regions, Englekirk investigated code restrictions for precast concrete shear walls in the United States. Englekirk discovered that due to the design criteria available in 1990, high rise construction of precast walls in seismic regions was not possible. The principal obstacles, which were identified by Englekirk, were:

• A boundary element requirement for walls when: \( \frac{P_u}{A} + \frac{M_u}{S} \geq 0.2f'_c \), where \( P_u \) is the factored axial load, \( M_u \) is the factored design moment, \( A \) is the cross sectional area of the wall, \( S \) is the section modulus of the wall and \( f'_c \) is the concrete strength.

• A requirement of two curtains of steel in walls when: \( v_u \geq 2\sqrt{f'_c} \),

where, \( v_u \) is the factored shear stress.

Through investigation of overseas practice, Englekirk found that seismic design of precast concrete structures has been successfully completed particularly in Japan and New Zealand. Precast concrete construction has not only been used but also promoted in Japan. Japanese design criterion appears to accept a prescribed level of subassembly ductility as proof that sufficient building ductility exists. Therefore, Japanese contractors typically test full-scale subassemblies to a standard loading sequence to confirm that the component ductility is greater than that required by the design criteria. Alternatively, New Zealand requires certain
precast concrete bracing systems to be designed to a higher yield level (1.5 times) than comparable cast-in-place systems.

The design criteria restrictions in the United States, along with the successful methods used in Japan and New Zealand, suggests that the United States precast concrete industry must promote a performance type design philosophy if it is at all interested in the structural system market in regions where seismicity is a consideration.

2.3 Precast Concrete Emulation

Current design codes generally require emulation when designing precast concrete structures (e.g., UBC 1997 [21] and ACI 318-02 [11]). Emulation is defined as designing connections between precast elements such that the seismic performance of the precast structure is equivalent to that of a conventionally designed cast-in-place monolithic concrete structure. As discussed in Chapter 1, this design concept does not take advantage of the strengths and unique properties of precast construction. Despite not completely benefiting the technology, the emulation concept has been adopted for design of precast systems in the United States and more widely overseas. Therefore, this section briefly addresses the current emulation methodology for precast concrete structures.

*ACI Committee 550 [22]*

ACI Committee 550 recently published a document describing various concepts of emulating cast-in-place concrete systems using precast concrete. With options of using different
methods for connecting precast concrete elements, the emulation process requires three general steps:

1. Select the desired structural system for resisting gravity and lateral loads. This can be done by using either a moment-resisting frame or a combination of a gravity-load-resisting frame with lateral-load-resisting shear walls.

2. Design and detail the structure to meet the requirements of a building constructed of monolithic cast-in-place reinforced concrete, noting that the structural elements must be suitable for plant fabrication, must be capable of being transported and must be erected by cranes.

3. Arrange the structure using typical precast elements of appropriate sizes and shapes to meet the above criteria. Then design the connections between elements to allow them to emulate behavior of a comparable monolithic system.

The remainder of this section focuses on emulation of wall systems. The critical section in walls is generally the connection interface between the precast panels and the foundation systems, since this is the location of the maximum moment and shear caused by lateral loads. Such horizontal connections are usually detailed with grout and vertically lap spliced reinforcing bars, similar in concept to the floor slab to wall panel connection shown in Figure 2.1. The grout provides continuity for compressive forces across the joint while the reinforcing bars do the same for tensile forces.

Additionally, it was recommended by the ACI committee that for the design of shear walls with aspect ratios less than 3-to-1 (i.e., the ratio between the height and length of wall), the
effects of shear deformations should be considered. An additional 5% eccentricity for accidental torsion effects was suggested for design of buildings with precast walls in addition to the eccentricity calculated using the distance from the center of mass to the center of stiffness as well as any code requirements. It was also recommended that precast walls be designed as cantilevers from the foundation, even when floors are connected to the walls at intermediate locations.

Figure 2.1  A lapped splice connection suggested between precast wall panel and precast floor slab [22].
2.4 Unbonded Post-Tensioned Precast Walls

In consideration of the potential benefits of a non-emulative precast wall alternative, a concept for an unbonded post-tensioned precast concrete wall has been investigated. This was based on the concept suggested by Priestley and Tao [23] for precast building frames with the idea that the unbonded post-tensioning would provide an improved restoring force to the lateral load resisting systems. Kurama et al. [13, 24, 25] have extensively studied unbonded precast walls, which consist of separate panels stacked vertically. The behavioral and analytical findings of their study as well as their design recommendations are discussed in this section.

2.4.1 Behavior and Analysis

Kurama et al. [13, 24]

The performance of unbonded post-tensioned precast concrete walls under lateral loads is controlled by the behavior along the horizontal joints, specifically, gap opening and shear slip (Figure 2.2). In gap opening, the post-tensioning force and the axial force due to gravity load provides a restoring force that tends to close the gaps between the panels upon unloading of the lateral loads. In shear slip, there is no mechanism available to provide the restoring force required for reversing the slip as in the gap opening case; therefore, shear slip is difficult to control under a seismic event and thus the wall should be designed to prevent such behavior. By ensuring adequate post-tensioning force, the precast walls can be designed to respond in a manner illustrated in Figure 2.2a without experiencing shear slip as shown in Figure 2.2.b.
To describe the seismic performance, Kurama et al. specified four states for the lateral force-displacement response of the unbonded post-tensioned precast wall system, as detailed in Figure 2.3. The first of these states is the Decompression State, which is the point where gap opening is initiated at the horizontal joint between the base of the wall and the foundation. The next state is the Softening State. This state is identified by the beginning of a significant reduction in the lateral stiffness of the wall due to gap opening along the horizontal joints and non-linear behavior of the concrete in compression. The third state is the Yielding State, the point when the strain in the post-tensioning steel first reaches the limit of proportionality. The final state is the Failure State, which occurs when the flexural failure of the wall occurs, or in other words, when concrete crushing occurs to the confined concrete at the wall toes.
Figure 2.3 Lateral load behavior of a precast wall with unbonded post-tensioning [13, 24].

The behavior of precast walls using unbonded post-tensioning was noted for having the following advantages under cyclic lateral loading:

1. Limiting inelastic response of the post-tensioning tendons introduces a self-centering capability.
2. Degradation in the initial stiffness of the wall is small.
3. Inelastic straining of the post-tensioning steel can be limited. Thus, the reduction in prestress expected during load cycles beyond the yielding state can be controlled.

The major disadvantage of the unbonded post-tensioned precast walls is that their response is characterized more towards non-linear elastic behavior producing very little inelastic energy dissipation, as observed by the “slender” hysteresis loops shown in Figure 2.4.
In order to study the behavior of the precast post-tensioned wall, six prototype walls were subjected to more than 200 non-linear dynamic time-histories, using 15 design-level and 15 survival-level ground motion records, by means of the DRAIN-2DX computer program [26]. The primary precast wall example used in this report was referred to as wall WH1, designed for regions of high seismicity assuming a medium profile for the foundation soil. The wall was 20-ft wide and 81-ft tall, consisting of six panels stacked vertically. There were a total of 28 unbonded post-tensioned tendons in the wall, each having an area of 1.5 in.$^2$, and the wall contained #3-spiral confinement reinforcement at each end along the length within the bottom panel. Under the Hollister ground motion, considered as a design-level earthquake,
wall WH1 was predicted to have a maximum base rotation of just over 3% and a maximum base moment of just under 800,000 kip-in., as can be seen in Figure 2.5.

![Graph showing predicted base moment-rotation response of an unbonded precast wall.](image)

**Figure 2.5** Predicted base moment-rotation response of an unbonded precast wall [13, 24].

A cast-in-place reinforced concrete wall with the same strength, initial stiffness, drift capacity, initial fundamental period and viscous damping as Wall WH1 was also analyzed to compare the two systems. One difference between the walls was their hysteretic behavior expected under lateral cyclic loads. The inelastic energy dissipation of the cast-in-place wall was approximately twice that of Wall WH1, however, the cast-in-place wall did not have self-centering capabilities. At the completion of the ground motion the cast-in-place wall was found to have approximately 3% residual drift while the post-tensioned wall had nearly zero as shown in Figure 2.6.
Figure 2.6 Comparison of roof drifts obtained from dynamic analysis of walls [13, 24].

Other major observations made when comparing the responses of the unbonded post-tensioned precast wall (WH1) and the cast-in-place reinforced concrete wall were:

1. The maximum roof drift of Wall WH1 was larger than that of the cast-in-place wall. On average, Wall WH1 produced 38% and 14% larger roof drifts than those obtained for the cast-in-place wall under design-level ground motions and survival-level ground motions, respectively.

2. The seismic response of Wall WH1 decayed less rapidly resulting in a large number of drift cycles.

3. Wall WH1 oscillated around the zero-drift position, whereas, the cast-in-place wall accumulates a significant residual drift; an example is shown in Figure 2.6.
Another important aspect investigated was the reduction in the prestress force, a result of post-tensioning steel yielding when the unbonded walls are subjected to large inelastic lateral displacements. Two significant observations were reported in respect to this:

1. Reduction in prestress force changes the hysteretic behavior of the wall, and
2. Reduction in prestress force significantly reduces the shear slip resistance of the wall.

Therefore, the authors emphasized that it is extremely important to consider the reduction of the prestress force in the design of walls at the survival-level ground motions.

**Kurama et al. [25]**

In addition to the behavior study on precast wall behavior, Kurama et al. carried out a parametric study using five walls designed for a set of six-story office buildings, four of which were designed for regions of high seismicity and one for regions of moderate seismicity, denoted by ‘H’ and ‘M’ in the second letter of the wall designations used in Figure 2.7. As shown in this figure, the walls are of different dimensions, different areas of post-tensioning tendons, different locations of post-tensioning tendons as well as different amounts of confinement spiral reinforcement. Additionally, walls WH1, WH2, WH3 and WM2 were designed for a site with a soil profile of medium density, while wall WH4 was designed for a site with a soft soil profile. The dynamic analyses of the systems were conducted under seven natural and four generated ground motion records and were conducted on wall models developed using the DRAIN-2DX program.
Figure 2.7 Unbonded precast prestressed walls studied by Kurama et al.: (a) typical wall, and (b) through (f) prototype walls chosen for the study [25].
The analysis output was compared with design criteria established for a proposed design approach (described later in Section 2.4.2). First, considering site characteristics, the maximum drifts obtained from the analysis for walls WH1, WH4 and WM2 were found to exceed the drifts selected for the design- and survival-level states. For the design-level motion the walls exceeded the design-level drift predictions by 15% to 49% whereas for the survival-level ground motion the selected design drifts were exceeded by 14% to 158%. Therefore, it was concluded by Kurama et al. that improved methods for accommodating target drifts in the design are needed and that the observed large drift demands may necessitate the use of supplemental energy dissipation in the design of unbonded post-tensioned walls. Overall, the difference between the analytical results and the design criteria drift demands was found to be greater for:

1. Survival-level ground motions than for design-level ground motions.
2. Walls designed for sites with high seismicity than for sites with moderate seismicity.

It was further noted that the design criteria suggested for the maximum story drift provided slightly better match to the analytical results than the similar observation made for the maximum roof drift demand. Additionally, the base shear demands obtained by analysis were found to be below those estimated by the design procedure. Therefore, Kurama et al. concluded that the base shear demand calculation developed for cast-in-place concrete walls can be applied to unbonded post-tensioned precast walls. It was also concluded that there was a reduction in the total post-tensioning force as the analysis proceeded beyond the yield strength, but shear slip did not occur in the walls under the ground motions studied.
The dynamic analysis results were also compared to evaluate the effects due to the differences in initial prestress and eccentricity of the post-tensioning steel using walls WH1, WH2 and WH3. The first observation was that the initial prestress and eccentricity have little effect on the maximum drift demands of the walls, which were characterized by 1.05, 1.14 and 1.07 percent maximum base rotations for the design-level and 2.72, 3.02 and 2.74 percent maximum base rotations for the survival-level earthquakes, representing WH1, WH2 and WH3, respectively.

The increase in the drift demand for wall WH2 was attributed to the smaller base shear resistance of the wall resulting from the reduced initial prestress. The decrease in the drift for WH3, as compared with WH2, was reported to be a result of larger post-tensing stiffness of WH3. Wall WH1, with an initial prestress of 0.6 times the ultimate stress, had a maximum base shear demand of 1631-kips. Walls WH2 and WH3, with an initial prestress of 0.3 times the ultimate stress, had maximum base shear demands of 1410-kips and 1408-kips, respectively.

The shear slip capacity of the walls was reduced with a decrease in initial prestress and an increase in eccentricity. Wall WH1, with the largest initial prestress and smallest eccentricity, had the greatest shear slip resistance of 1751-kips. Wall WH3, with the smallest initial prestress and largest eccentricity, had the smallest shear slip resistance of 825-kips. Wall WH2, which fell in the middle with the same eccentricity as wall WH1 and the same initial prestress as wall WH3, had a resistance of 1345-kips. Kurama et al. also observed that walls
with larger initial prestressing resulted in smaller gap openings at the horizontal interfaces between the precast panels.

2.4.2 Design

Kurama et al. [13, 24]

In evaluation of the results from the parametric study, discussed above, and the need for a non-emulative design approach, a performance-based seismic design approach was proposed in order to allow the designer to specify and predict the performance of a building consisted of unbonded post-tensioned precast walls (see Figure 2.8) 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.

Seismic Performance Levels

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

1. “Immediate Occupancy” — Post-earthquake damage state in which the building has only limited structural and non-structural damage.
2. “Life Safety” — Post-earthquake damage state in which the building has significant damage, but some margin against total or partial structural collapse remains.
3. “Collapse Prevention” — Post-earthquake damage state in which the building is on the verge of partial or total collapse.
Figure 2.8  Unbonded post-tensioned precast wall system: (a) elevation, and (b) cross section near base [13, 24].
Building Limit States and Design Capacities

The building limit states for the unbonded post-tensioned precast concrete walls were identified as follows:

1. Decompression at the base, which denotes initial gap opening at this location.
2. Decrease in lateral stiffness due to gap opening at the base.
3. Spalling of cover concrete near the base.
5. Attainment of the base moment capacity.
6. Reduction in the prestress due to inelastic straining of the post-tensioning steel.
7. Crushing of the concrete confined by spirals.
8. Reduction in the lateral load resistance.
9. Reduction in the gravity load resistance.
10. Shear slip between precast panels along the horizontal joints.
11. Crushing of the concrete outside the spiral confinement region, but inside the region reinforced with wire mesh.

The design capacities of the wall system associated with these limit states are determined by a non-linear static push over analysis. The building limit states 1, 2, 4 and 7 listed above are identified in the lateral load response of an unbonded post-tensioned wall shown in Figure 2.3.

Seismic Input Levels and Structural Demands

The design approach utilizes two seismic input levels along with associated structural demands to adequately satisfy the various design limit states:
1. Design-level ground motion
   a. design base shear demand, $V_{des}$
   b. maximum roof drift demand, $\Delta_{des}$
   c. maximum story drift demand, $\delta_{des}$

2. Survival-level ground motion
   a. maximum roof drift demand, $\Delta_{sur}$
   b. maximum base shear demand, $V_{max}$

The design objectives proposed by Kurama et al. are based on achieving the immediate occupancy performance level under the design-level ground motion and achieving the collapse prevention performance level under the survival-level ground motion.

**Wall Design Criteria**

The recommended seismic design of the unbonded post-tensioned precast concrete wall system also has several design criteria that compare estimated structure demands with structure design capacities. They are as follows:

1. Criterion for the base shear capacity at the yielding state, $V_{llp}$
   \[ \phi_f V_{llp} \geq V_{des} = \frac{Q_{des}}{R} \]
   where $\phi_f$ is the reduction factor as defined by a code standard (e.g., ACI 318-02 [11]), $Q_{des}$ is the base shear demand for the design-level ground motion, and $R$ is the response modification factor.

2. Criterion for the base shear capacity at the softening state, $V_{ell}$
   \[ V_{ell} \geq V_{des} = \frac{Q_{des}}{R} \]
3. Criterion for the roof drift capacity at the yielding state, Δ_{llp}

\[ Δ_{llp} ≥ Δ_{des} \]

where Δ_{des} is the expected maximum roof drift demand under the design-level ground motion.

4. Criterion for the maximum story drift under the design-level ground motion, δ_{des}

\[ δ_{des} ≤ δ_{all} \]

where δ_{all} is the allowable story drift as defined by NEHRP [27].

5. Criterion for the roof drift capacity at the failure state, Δ_{csc}

\[ Δ_{csc} ≥ Δ_{sur} \]

where Δ_{sur} is the expected maximum roof drift demand under the survival-level ground motion

6. Criterion for the length and height of the spiral confined region near the base

\[ Δ_{ctc} ≥ Δ_{csc} \]

where Δ_{ctc} is the roof drift capacity corresponding to the crushing of the concrete inside the wire mesh, and Δ_{csc} is the roof drift capacity at the failure state.

7. Criterion for the shear slip capacity, V_{ss}

\[ φ_s V_{ss} ≥ V_{max} \]

where φ_s is the shear capacity reduction factor as defined by a code standard (e.g., ACI 318-02 [11]) and V_{max} is the expected maximum base shear demand under the survival-level ground motion.

8. Criterion for the maximum roof drift under the survival-level ground motion, Δ_{sur}

\[ Δ_{sur} ≤ Δ_g = 2.5\% \]
where $\Delta_g$ is the maximum roof drift that can be sustained by gravity load system.

**Design Steps**

Utilizing the above criteria, a parametric investigation was performed (discussed previously in Section 2.4.1) to determine how the design capacities are affected by changes to several critical parameters of the wall. From this investigation, the following design procedure was finalized with an objective of meeting all of the wall design criteria listed above:

1. Select trial wall dimensions (wall height, length, width, etc.).
2. Set the initial stress in the post-tensioning ($f_{pi}$) to a desired value, generally 55% to 65% of the ultimate strength of the post-tensioning steel.
3. Determine the area of post-tensioning ($A_p$) by satisfying design criterion 2, discussed above.
4. Check design criterion 1 to ensure that the selected area of post-tensioning and length of wall are sufficiently large.
5. Check design criteria 4 and 8 to ensure that $\delta_{des} \leq 1.5$ percent and $\Delta_{sur} \leq 2.5$ percent, where both $\delta_{des}$ and $\Delta_{sur}$ are functions of the length ($l_w$) and thickness ($t_w$) of the wall.
6. Check design criterion 3 to ensure that the initial post-tensioning stress and location of post-tensioning steel are appropriate.
7. Check design criterion 7 to satisfy the shear slip condition.
8. Check design criteria 5 and 6 to ensure sufficient spiral reinforcement (Kurama et al. recommended that the spiral reinforcement be provided at least one-fourth of the wall length at each corner and to a height of at least one-story).
2.5 Precast Jointed Wall Systems

In addition to the single wall systems discussed in Section 2.4, researchers have investigated the use of unbonded post-tensioned precast jointed wall systems in buildings as the primary lateral load resisting elements. The basic setup and behavior of the jointed wall concept is the same as the single wall concept with the exception that the jointed wall system contains two or more walls which are connected horizontally (see Figures 1.6 and 1.8). The connection between walls is performed along the height of the wall using appropriate connectors. The connectors are supposed to significantly contribute to energy dissipation, and thus have potential to reduce lateral drift and large amplitude dynamic cycles of the jointed wall systems when compared to the single precast walls discussed in Section 2.4.

2.5.1 Behavior and Analysis

*Schultz and Magana [28]*

Schultz and Magana’s primary objective was to investigate the performance of horizontal and vertical joint connectors suitable for use in precast jointed systems. Seven connectors appropriate for vertical joints and four connectors suitable for horizontal joints were studied. The investigation included establishing the performance and developing behavioral models for the connectors. A U-shaped Flexure Plate (UFP) connector (see Figure 2.9), used as a horizontal connector along the vertical joint of the precast wall system in the PRESSS test building, was one of the connectors studied by the researchers.
The UFP connector was proposed as an energy-dissipating flexible connector in which the bending action induced by rolling of the U-plate on a straight steel plate resists vertical shear force. It was originally designed using Grade 36 structural steel, but the material was changed to a more ductile 304 stainless steel when cracks formed across the curved region of the steel U-plate during fabrication. The stainless steel UFP connector proved to be 2.5 times as strong as was assumed and the friction capacity of the interface was exceeded. The connector, which was predicted to develop only 40% of the specified design load, actually attained the entire target design load. Overall, the UFP proved to be a very desirable connector in the design of precast wall systems. Figure 2.10 shows hysteresis response of a UFP Connector that was subjected to reversed displacement cycles to simulate seismic actions in which the maximum displacement of 1.6-in was reported to be equivalent to a drift ratio of 2 percent for the six-story building considered in that study.
Figure 2.10  Force-displacement response reported for a UFP connector by Schultz and Magana [28].

PRESSES Test Building [12, 14 – 16]

As previously described in Section 1.5, the PRESSES test building, which included an unbonded post-tensioned precast wall system consisting of two walls and several UFP connectors along the vertical joint to resist lateral forces in one direction, performed very well during seismic testing conducted in that direction. Under the design-level earthquake, the wall experienced a peak recorded displacement of 8.3 in., just 8% below the target design displacement of 9 in., and accrued a residual drift of only 0.06%. At an event 1.5 times the design-level event, the wall produced a maximum lateral displacement of about 11.5 inches. Corresponding to this maximum displacement, the base moment in the wall direction was approximately 100,000 kip-in. as seen in Figure 2.11, with damage reported to be limited to
minor spalling of cover concrete at the wall toes [15]. No structural damage was observed during the entire wall direction testing. With the negligible damage and low residual drift of the wall system, it was concluded that the building had responded within the serviceability or “immediate occupancy” level of performance for seismic events up to 1.5 times the design-level earthquake.

![Fifth floor displacement (mm)](image)

*Figure 2.11* Overall response of the PRESSS building in the wall direction testing [29].

*Conley et al. [30]*

The wall direction response of the PRESSS test building was investigated analytically by Conley et al. The primary objective of this study was to capture the wall response using a relatively simple 2-D model that could be easily replicated in a design office. The
RUAUMOKO computer program [31] was selected to model the wall direction of the test building. Initially, the wall system was only modeled, but after running the analysis it was discovered that this only accounted for about 75% of the response at the maximum displacement in the wall direction. The remaining moment resistance was suspected to be due to the moment resistance at the base of the eight columns in the test building and thus was added to the model.

First, to model the wall direction response of the PRESSS test building, the two walls from the PRESSS test building were modeled as columns with lumped nodal masses at each floor level as shown in Figure 2.12. The two walls were connected with rigid links using shear springs located at the midpoint to model the behavior of the U-shaped Flexural Plate (UFP) connectors, where each spring represented four UFPs in the test building. An additional column was added to the right of the wall system as in Figure 2.12, with floor displacements slaved to the adjacent wall element to account for the base moment resistance of six seismic and two gravity columns in the test building. Additionally, the location of the compression only springs were fixed at the base of the wall at locations estimated for the resultant compression forces at a drift of 2% for the wall system while the columns were modeled to rotate about their corners. Springs were also used to capture the increase in the elongation of the unbonded post-tensioning tendons at the center of the walls and columns. The accuracy of the model was determined to be adequately validated by comparing the analytical model output with the experimental moment-displacement response from the test building (e.g., see Figure 2.13).
Conley et al. concluded that the primary goal of producing a simple analytical model that can be reproduced by a typical design firm was accomplished and that the displacement-based design (DBD) procedure was further confirmed by the response of the analytical model.

In studying this report, it was observed that the post-tensioning force was overestimated by the RUAUMOKO model which in turn increased the moment resistance of the walls. This is suspected to be due to the location of the compression only springs at the base. First of all, the compression forces were at a fixed location based on a calculation performed at 2% drift for the walls. This overestimated the lever arm between the post-tensioning force and the

**Figure 2.12** The RUAUMOKO model developed by Conley et al. to represent the PRESSS building in the wall direction [30].
compression force for drifts less than 2% since the location of the resultant compression force moves towards the edge of the wall as the drift increases. The spring location at 2% drift was also not exact. This is because the resultant force was based on the equivalent stress block concept, ignoring confinement effects. In addition, the model did not account for the actual contact surface for modeling the wall rotation at the base. The wall rotation in the model was assumed to be about the resultant compression force rather than the neutral axis. This discrepancy also leads to additional post-tensioning elongation in the model and thus increased post-tensioning force and moment resistance. Finally, it was found that the U-plate force contribution assumed at a given base rotation in the RUAUMOKO model of Conley et al. was somewhat higher than the actual value, which also increased the moment resistance of the jointed wall system. More realistic values for the U-plate force contribution are discussed later in the context of Figure 3.3.

![Figure 2.13](image)  
**Figure 2.13**  The base moment-lateral displacement prediction made by Conley et al. [25].
2.5.2 Design

*Galusha [32]*

The jointed wall system in the PRESSS test building was designed by researchers from the University of Washington in collaboration with a consulting engineer from the Nakaki Bashaw Group, Inc. using the design procedure summarized by Galusha [32]. The final design guidelines proposed for the wall system being validated herein used this procedure as the basis, which is summarized in a step-by-step format below.

1. Select the wall system configuration: wall height \( H_t \), entire wall system length \( B \) and number of panels \( n \).
2. Select a value for \( \alpha \), the recentering coefficient, within the suggested range of 1.0-1.2.
3. 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 \( f'_c \).
4. Calculate the width of each wall panel, \( b \).
   \[
   b = \frac{B}{n}
   \]  
   (2.1)
5. 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 \( \theta_{\text{design}} \). A value for \( \theta_{\text{design}} \) is required if the displacement-based design (DBD) is preferred for the wall system as adopted in the design of the PRESSS building.
6. Use DBD to calculate seismic loads and thus calculate the design base shear \( V_b \) and design overturning moment \( M_{ot} \).
7. Select an estimate for \( \beta \), the moment arm reduction coefficient, within the suggested range of 0.9-1.0. (This accounts for the fact that the wall does not rotate on its corner)
8. Calculate the initial post-tensioning stress:

\[
f_{p0} = f_{py} - \left( \frac{b_\beta}{2} \theta_{\text{design}} \right) \frac{E_p}{H_t}
\]  

(2.2)

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

10. Calculate the wall panel post-tensioning:

\[
P_{p0} = \frac{M_{ot} - \frac{nWb\beta(1 + \alpha)}{2\alpha}}{\frac{nb\beta(1 + \alpha)}{2\alpha} + \frac{nE_p}{f_{p0}H_t} \left( \frac{b_\beta}{2} \right)^2 \theta_{\text{design}}}
\]  

(2.3)

11. Determine the area of post-tensioning steel:

\[
A_p = \frac{P_{p0}}{f_{p0}}
\]  

(2.4)

12. Calculate the interface shear force anticipated between the wall panels in the vertical direction:

\[
F = \frac{nP_{tot}\beta}{\alpha(2n - 2)}
\]  

(2.5)

13. Calculate the lift-off moment (\(M_{lo}\)), net righting moment (\(M_{r,\text{net}}\)) and the nominal moment (\(M_n\)):

\[
M_{lo} = \frac{nP_{tot}\beta}{2} + (n - 1)bF
\]  

(2.6)

\[
M_{r,\text{net}} = \frac{nP_{tot}\beta}{2} - (n - 1)b
\]  

(2.7)

\[
M_n = M_{ot}
\]  

(2.8)

14. Calculate the damping of the system (\(\zeta\)) and compare with the estimate used in the DBD.
If the inter-panel connectors are frictional devices:

\[
\zeta_{\text{friction}} = \left(\frac{2}{\pi}\right) \frac{2\theta_{\text{design}} (M_{\text{lo}} - M_{\text{r,net}})}{4\theta_{\text{design}} M_n} = \frac{M_{\text{lo}} - M_{\text{r,net}}}{\pi M_n}
\]  

(2.9)

If the inter-panel connectors are U-Shaped Flexure Plates

\[
\zeta_{\text{ufp}} = 0.625\zeta_{\text{friction}}
\]  

(2.10)

15. Check the wall thickness; iteration may be necessary with assumption made in step 9.

16. Check if \(\beta\) is equal to that assumed in step 7, and iterate if necessary.

A design procedure was also developed for the design of U-shaped Flexural Plates (UFP), which was described earlier in this chapter:

1. Determine the interface shear force (F) that is resisted by the UFPs.

2. Select the number of UFPs \(N_{\text{ufp}}\) located at each interface and determine the force in each UFP \(V\).

\[
V = \frac{F}{N_{\text{ufp}}}
\]  

(2.11)

3. Determine the properties of the proposed material using the minimum guaranteed fracture strain \(\varepsilon_{\text{fracture}}\) and ultimate stress \(F_u\).

4. Select the maximum strain \(\varepsilon_{\text{max}}\) to which the UFP will be subjected. It is recommended to select this value between \(0.25\varepsilon_{\text{fracture}}\) and \(0.5\varepsilon_{\text{fracture}}\) based on a 2-in gage length.

5. Select a UFP width (b).

6. Calculate the UFP thickness.
\[ t \geq \frac{2V}{bF_u \varepsilon_{\text{max}}} \]  

(2.12)

7. Determine the average radius of the UFP.

\[ R = \frac{t}{2\varepsilon_{\text{max}}} \]  

(2.13)

*Stanton and Nakaki [18]*

Upon the completion of the experimental research performed on the PRESSS test building (Sections 1.5 and 2.5.1), Stanton and Nakaki published a set of design recommendations based on Galusha’s design procedure [32] for the design of unbonded post-tensioned precast jointed walls in seismic regions. The design procedure is for wall systems composed of two or more horizontal panels that are separated by vertical joints as shown in Figure 2.14. Additionally, the wall panels are post-tensioned to the foundation and are connected across their vertical joints by shear connectors that can dissipate energy during lateral load response.

To perform the design of the jointed wall system, the following assumptions are made:

1. The design forces and drift limits are known, usually selected to satisfy code criteria.
2. The total wall length \((l_{w,\text{tot}})\), wall height \((h_w)\) and wall thickness \((t_w)\) are known, generally from architectural drawings and preliminary calculations.
3. The wall panels are assumed to be the same size.
4. The shear connectors are treated as rigid-plastic.
5. The post-tensioning steel reaches the yield strain at the design drift.
The design method proposed for unbonded post-tensioned jointed walls by Stanton and Nakaki [18] consisted of 10 steps, which may be summarized as follows:

1. Establish the following material properties: the yield strength ($f_{py}$) and modulus of elasticity ($E_p$) of the post-tensioning material, strength of shear connectors, strength of concrete ($f'_c$), and strength of grout ($f'_g$) that will be placed between the panels and foundation.

2. Using either the Displacement-Based Design (DBD) or the Force-Based Design (FBD) methods, determine the design base shear ($V_{des}$) and design drift ($\theta_{des}$).
3. Select the number of panels (n) using the following considerations: the wall panel aspect ratio, the post-tensioning tendon elongation, the lateral strength, and the damping.

4. Establish the following constants:

   wall panel length \((l_w)\),
   \[
   l_w = \frac{l_{w,\text{tot}}}{n} \quad (2.14)
   \]

   increase in prestressing tendon stress between zero drift and design drift \((\Delta f_{p,e})\),
   \[
   \Delta f_{p,e} = 0.5E_p \theta_{\text{des}} \frac{l_w}{h_u} \quad (2.15)
   \]
   where \(h_u\) is the unbonded length of the post-tensioning tendon.

   design moment \((M_{\text{des}})\),
   \[
   M_{\text{des}} = V_{\text{des}} h_{\text{eff}} \quad (2.16)
   \]
   where \(V_{\text{des}}\) is the design base shear and \(h_{\text{eff}}\) is the height above the foundation that the lateral load resultant acts on the wall.

   panel weight \((W_{\text{panel}})\),
   \[
   W_{\text{panel}} = l_w t_w h_w \gamma_c \quad (2.17)
   \]
   where \(\gamma_c\) is the density of concrete.

   total weight \((W)\),
   \[
   W = W_{\text{panel}} + l_w w_{\text{floor}} \quad (2.18)
   \]
   where \(w_{\text{floor}}\) is the distributed vertical load on the wall at the base from all floors.
compressive capacity of wall panel ($C_c$),

$$C_c = l_w t_{w,\text{eff}} (k_i f'_{g})$$  \hspace{1cm} (2.19)

where $t_{w,\text{eff}}$ is the thickness of the wall panel effective in resisting compressive forces, $k_i$ is the uniform stress in the equivalent stress block divided by $f'_{g}$.

force in shear connectors ($F_{sc,\text{net}}$),

$$F_{sc,\text{net}} = F_{sc,\text{left}} - F_{sc,\text{right}}$$  \hspace{1cm} (2.20)

where $F_{sc,\text{left}}$ and $F_{sc,\text{right}}$ are the total yield force of all shear connectors in the vertical joints on the left and right side of the wall panel, respectively. Furthermore, it is implied that $F_{sc,\text{left}}$ and $F_{sc,\text{right}}$ are equal to $F_{sc}$ and 0 for the leading and trailing walls and intermediate walls, respectively.

5. Select the tendon reinforcement area ($A_p$) and initial prestressing stress ($f_{p0}$).

6. Establish the conditions at the time the base of the wall starts to lift off of the foundation (zero drift, also referred to as the decompression point):

initial force in the prestressing tendon ($P_0$),

$$P_0 = A_p f_{p0}$$  \hspace{1cm} (2.21)

total axial force on one wall panel ($N_0$),

$$N_0 = P_0 + W$$  \hspace{1cm} (2.22)

compressive reaction on one wall panel ($C_0$),

$$C_0 = N_0 + F_{sc,\text{net}}$$  \hspace{1cm} (2.23)

defining distance from the compression face to the compression force as $a_0 l_w$,  

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\[ \alpha_0 = 0.5 \frac{C_0}{C_c} \tag{2.24} \]

defining neutral axis depth as \( \eta_0 l_w \),

\[ \eta_0 = 2 \frac{\alpha_0}{\beta_1} \tag{2.25} \]

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 (\( \kappa_0 \)),

\[ \kappa_0 = \frac{F_{sc}}{N_0} \tag{2.26} \]

where \( F_{sc} \) is the total yield force of all shear connectors in one vertical joint.

7. These conditions established in Step 6 can be determined at the design drift (\( \theta_{\text{des}} \)) using an iterative method (see Figure 2.15 and Eqs. 2.27–2.30). Note that the difference between equations 2.22 through 2.25 and equations 2.27 through 2.30 are the drift endured by the system and is denoted by ‘0’ for zero drift and ‘des’ for the design drift.

\[ N_{\text{des}} = P_{\text{des}} + W \tag{2.27} \]

where an assumed value for \( P_{\text{des}} \), force in the post-tensioning tendon at \( \theta_{\text{des}} \), should be used for the first step iteration process. The value from Eq. 2.34a may be used in subsequent steps.

\[ C_{\text{des}} = N_{\text{des}} + F_{sc,\text{net}} \tag{2.28} \]

\[ \alpha_{\text{des}} = 0.5 \frac{C_{\text{des}}}{C_c} \tag{2.29} \]
\[ \eta_{\text{des}} = 2 \frac{\alpha_{\text{des}}}{\beta_1} \]  

Equation (2.30)

Additionally the post-tensioning elongation \((\Delta_p)\) can be obtained from Eq. 2.31.

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

Equation (2.31)

Increase in stress between zero drift and the design drift \((\Delta f_p)\) is, therefore,

\[ \Delta f_p = E_p \frac{\Delta e}{h_u} = \Delta f_{p,\text{pc}} (1 - 2\eta_{\text{des}}) \]  

Equation (2.32)

The total stress in the post-tensioning steel \((f_{p,\text{des}})\) can be determined from:

\[ f_{p,\text{des}} = (f_{p,0} + \Delta f_p) \leq f_{py} \]  

Equation (2.33)

where \(f_{py}\) is the yield strength of the post-tensioning tendon and is taken as the maximum value for \(f_{p,\text{des}}\).

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

\[ P_{\text{des}} = A_p f_{p,\text{des}} \]  

Equation (2.34a)

Step 7 should be iterated until \(P_{\text{des}}\) converges to that assumed in Eq. 2.27.

A non-iterative form was also presented for \(P_{\text{des}}\),

\[ P_{\text{des}} = \frac{A_p \left[ f_{p,0} + \Delta f_{p,\text{pc}} \left(1 - \frac{W + F_{\text{sc.net}}}{0.5\beta_1 C_c}\right)\right]}{1 + \frac{A_p \Delta f_{p,\text{pc}}}{0.5\beta_1 C_c}} \leq A_p f_{py} \]  

Equation (2.34b)

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

\[ M_{\text{cap,panel}} = l_w (C_{\text{des}} (0.5 - \alpha_{\text{des}}) + 0.5(F_{\text{sc,left}} + F_{\text{sc,right}})) \]  

Equation (2.35)
9. Calculate the total moment resistance of the wall system ($M_{\text{cap,wall}}$). Each panel must be designed using steps one through eight and then the moment capacities of the panels can be summed together to determine the total moment capacity:

$$M_{\text{cap,wall}} = \Sigma M_{\text{cap,panel}}$$ (2.36)

10. Ensure the wall system meets the following criteria:

$$\rho_{\text{MOM}} = \frac{M_{\text{des}}}{M_{\text{cap,wall}}} \leq 1.0$$ (2.37)

which checks the demand/capacity ratio for overturning moment on the panel.

$$\rho_{\text{fp0}} = \frac{f_{\text{p0}}}{(f_y - \Delta f_p)} \leq 1.0$$ (2.38)
1. Establish Material Properties:
Grout: $f'_{g}, \beta_{t}, \mu$
Concrete: $f'_{c}, \gamma_{c}$
Tendon: $E_{p}, f_{py}$
Connector: force vs. displacement

2. Obtain Design Loads and Drifts:
Use Displacement-Based Design or Force-Based Design to obtain design loads. Compute corresponding design moments ($M_{des}$) and drifts ($\theta_{des}$).

3. Select Number of Panels:
Considerations:
Wall panel aspect ratio
Post-tensioning tendon elongation due to rocking
Lateral strength
Damping

4. Establish Constants:
$L_{w}, \Delta f_{p0}, M_{des}, W_{panel}, W_{c}, C_{f}, F_{ac,net}$

5. Select Reinforcement:
$A_{p}, f_{p0}, F_{ac}$

6. Establish Conditions Immediately after Lift-off at the Base of the Wall:
$P_{0}, N_{0}, C_{0}, \alpha_{0}, \eta_{0}, \kappa_{0}$

7. Establish Conditions at Design Load and Drift:
$N_{des}, C_{des}, \alpha_{des}, \eta_{des}, \Delta p_{f}, f_{p,des}, P_{des}$

8. Compute Resisting Moment of Panel:
$M_{cap, panel}$

9. Compute Moment Resistance of the Wall System:
$M_{cap, wall}$

10. Check Acceptance Criteria:
$\rho_{MOM}, \rho_{P0}, \rho_{UL}, \rho_{ZRD}, \rho_{ROC}$

Figure 2.16 A flow chart representation of the PRESSS recommended design procedure.
which checks the stress ratio to ensure that the prestressing tendon does not yield at maximum drift.

\[ \rho_{\text{UPL}} = \kappa_0 \leq 1.0 \]  (2.39)

which checks the ratio of uplift force to hold down force on one panel to ensure the wall does not uplift.

\[ \rho_{\text{ZRD}} = \kappa_0 \frac{(n - 1 + 2\alpha_{0,\text{ave}}\kappa_0)}{n(0.5 - \alpha_{0,\text{ave}})} \leq 1.0 \]  (2.40)

which is a parameter ratio controlling the residual drift.

\[ \rho_{\text{ROC}} = \frac{\kappa_{0}\mu_{\text{eff}}}{l_{we}} \left( (0.5 - \alpha_{0,\text{ave}}) + \frac{(n - 1 - 2\alpha_{0,\text{ave}}\kappa_0)}{n} \right) \leq 1.0 \]  (2.41)

which is a force ratio to ensure that the panel rocks rather than slides.

The basic outline of the design approach of Stanton and Nakaki described above may be summarized as shown in flowchart form in Figure 2.16.

The guidelines of Stanton and Nakaki also includes steps for the design of U-shaped Flexural Plate (UFP) connector (Figure 2.17), which is one possible choice for the shear connector in the wall system. Any ductile shear connector that has the required strength and deformation capacity may be used to connect the precast wall panels.

Utilizing equilibrium of the shear connector and determining the plastic moment capacity of the shear connector \( M_{\text{sc}} \) by using the connector dimensions and the stress in the plate under
plastic conditions ($f_{sc,des}$, see Figure 2.18), an equation for the shear strength of one UFP connector ($V_{sc}$) can be derived:

$$V_{sc}D_{sc} = 2M_{sc} \quad (2.42)$$

where $D_{sc}$ is the diameter of the UFP connector (depicted in Figure 2.17).

The moment capacity ($M_{sc}$) is taken as:

$$M_{sc} = \left( \frac{b_{sc}t_{sc}^2}{4} \right) f_{sc,des} \quad (2.43)$$

where $b_{sc}$ is the width of the UFP connector and $t_{sc}$ is the thickness of the UFP connector (Figure 2.17).

Additionally, $f_{sc,des}$ can be obtained by using the corresponding UFP strain:

$$\varepsilon_{sc,des} = \frac{t_{sc}}{D_{sc}} \quad (2.44)$$

Figure 2.17 Forces acting on a UFP connector under inelastic conditions [18].
Note that $\varepsilon_{sc,\text{des}}$ is limited to a value of $\varepsilon_{sc,\text{max}}$ (Figure 2.18), which is defined as:

$$\varepsilon_{sc,\text{max}} = \frac{\varepsilon_{sc,u}}{3}$$

(2.45)

From the required $F_{sc}$, the number of shear connectors ($n_{sc}$) can be determined as:

$$n_{sc} = \frac{F_{sc}}{V_{sc}}$$

(2.46)

**Figure 2.18** Stress-strain relation of the UFP material [18].

In the above design procedure, $\varepsilon_{sc,\text{des}}$ for the UFP connector is determined from geometry of the connector without the influence of the design drift. Furthermore, the moment calculation given in Eq. 2.43 implies that the stress-strain response of the UFP material in Figure 2.18 is approximated for an elastic-perfectly plastic response using a yield value of $f_{sc,\text{des}}$. These approximations, along with the fact that hardening of stainless-steel, typically used as the UFP material, depends on previous strain history, can lead to inaccurate estimation of the UFP forces and the base moment resistance of the wall system.


**Monolithic Beam Analogy**

One of the primary issues in the design of the unbonded post-tensioned precast jointed wall systems is the strain incompatibility between the concrete and steel at the section level. The proposed design guidelines by Stanton and Nakaki [18] use an equivalent stress block based on the unconfined concrete compressive strength to obtain the neutral axis depth. This approach inaccurately estimates the neutral axis depth as well as the increase in the post-tensioning force at a given drift. Furthermore, this approach results in an increase in the neutral axis depth as the drift increases, which is expected to be just the opposite of the actual behavior. The neutral axis depth should decrease or possibly remain constant as the wall rotation at the foundation interface increases. This discrepancy was also found in the design of precast hybrid frames [9, 33], but the impact is expected to be greater for the wall system due to large length to thickness ratio of the wall panel. This is discussed in further detail in Chapter 4.

As an alternative to using the equivalent stress block, Pampanin et al. [34] proposed the monolithic beam analogy (MBA) to overcome the strain incompatibility issue and more accurately model the jointed precast frame connections at the section level. The MBA method basically equates the equivalent plastic hinge length of a monolithic system with that of a jointed system unbonded post-tensioned jointed precast wall. Application of this concept to an unbonded post-tensioned jointed precast wall was addressed by Sritharan and Thomas [35] and is detailed in Section 3.3. This approach results in a simple equation to relate the concrete strain in the extreme fiber at the base of each wall panel with the base rotation and neutral axis depth as shown in Eq. 2.47.
\[ \epsilon_{c,ext} = \left[ \frac{\theta}{0.08h_w} + \phi_c \right] c \]  

(2.47)

where \( \epsilon_{c,ext} \) is the concrete strain in the extreme fiber, \( \theta \) is the rotation at the base, \( h_w \) is the height of wall, \( \phi_c \) is the elastic curvature and \( c \) is the neutral axis depth.

Thus, with a given rotation and neutral axis depth an approximate strain can be obtained and section level compatibility can be met. A couple of issues should be noted with the approach based on MBA. First of all, the actual equivalent plastic hinge length of the jointed wall is most likely smaller due to the presence of unbonded post-tensioning and concentrated deformation at the interface. Secondly, this method is not exact, it is only an approximation used to overcome the incompatibility between the concrete and unbonded steel to more accurately portray the behavior of the wall system.
Chapter 3: Formulation of Validation Procedure

3.1 Introduction

The main goal of this study is to validate the PRESSS design guidelines that have been proposed for jointed precast wall systems by Stanton and Nakaki [18]. An effective validation process should encompass a range of values for the wall drift. Also, incorporating the test data available from the PRESSS test building, discussed in Section 2.5.1, in the validation of design guidelines is essential. In consideration of these issues, an analysis method is first formulated based on the PRESSS guidelines summarized in Section 2.5.2. Furthermore, as previously discussed, the use of an equivalent stress block in the design guidelines to overcome the strain incompatibility that exists at the section level causes concerns with the abilities of the analysis method derived from the PRESSS guidelines to accurately estimate the extensions of the post-tensioning bars and the vertical displacements that the UFPs would be subjected to. Consequently, an alternative analysis method is also established in this chapter, which was based on the monolithic beam analogy (MBA) concept described in Section 2.5.2. The predicted response of the wall system based on the two analysis methods are compared with the experimental results in Chapter 4 to validate the PRESSS design guidelines proposed for the jointed wall systems.

The development and presentation of these analysis methods are limited to jointed wall systems with two walls as the wall system included in the PRESSS building [12, 15, 16]. It is noted that the analysis methods presented here may be easily extended to other systems.
consisting of more than two walls. As presented in Chapter 1, the two walls in the system are referred to as the “leading wall” and “trailing wall”. In both directions of loading, the first wall that leads the lateral response of the wall system is referred to as the leading wall while the other wall is identified as the trailing wall. According to this terminology, in wall systems with more than two walls, the walls between the leading and trailing walls will be referred to as the intermediate walls.

3.2 Analysis Method Based on PRESSS Design Guidelines

The PRESSS analysis procedure was developed essentially by reversing the design guidelines suggested by Stanton and Nakaki [18], see Section 2.5.2. Consistent with the guidelines, the walls in the jointed system are analyzed independently assuming that the walls are subjected to identical drifts at all times. Preliminary investigation of the displacement data obtained at the base of the PRESSS wall system supported this assumption.

A flowchart shown in Figure 3.1 summarizes the analysis procedure while the steps involved in the analysis are as follows:

1. **Define wall dimensions and material properties.**
   
   This includes:
   
   \[
   f_g' = \text{grout strength} \quad f_c' = \text{concrete strength}
   \]
   
   \[
   \gamma_c = \text{concrete density} \quad E_p = \text{elastic modulus of post-tensioning steel}
   \]
   
   \[
   A_p = \text{area of post-tensioning steel} \quad f_{py} = \text{yield strength of post-tensioning steel}
   \]
\( f_{p0} = \) initial post-tensioning stress \hspace{1cm} \( h_u = \) unbonded length of post-tensioning steel

\( n = \) number of walls \hspace{1cm} \( h_w = \) height of wall

\( l_w = \) length of wall \hspace{1cm} \( t_w = \) thickness of wall

2. **Select base rotation, \( \theta \).**

Select a value less than 0.03 for \( \theta \).

3. **Determine parameters.**

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

\[
\Delta f_{px} = 0.5E_p \theta \frac{l_w}{h_u}
\]

(3.1)

Calculate the self-weight of the wall:

\[
W_{\text{panel}} = l_w t_w h_w \gamma_c
\]

(3.2)

Determine the total gravity load on the wall:

\[
W = W_{\text{panel}} + l_w w_{\text{floor}}
\]

(3.3)

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

Calculate the compression capacity of one wall:

\[
C_c = l_w t_w \left( k_1 f'_{g} \right)
\]

(3.4)

where \( k_1 \) is the uniform stress in the equivalent rectangular stress block divided by \( f'_{g} \), which is the strength of grout placed between the wall and foundation.
Figure 3.1  Flowchart describing the analysis procedure for the jointed wall system developed based on the PRESSS design guidelines.
When calculating $C_c$, the component $k_1$ should be dependant on the weaker of the two materials: interface grout and concrete in the wall. In the PRESSS building the concrete strength of the wall was found to be weaker than the interface material strength and therefore the concrete strength $f'_c$ instead of $f'_g$ was used in Eq. 3.4.

4. **Assume post-tensioning force, $P$.**

5. **Determine forces at base rotation $\theta$.**

   As illustrated in Figure 3.2, calculate the total tension force:

   $$N = P + W \quad (3.5)$$

   Calculate the compressive force:

   $$C = N + F_{sc} \quad \text{for the leading wall} \quad (3.6a)$$

   $$C = N - F_{sc} \quad \text{for the trailing wall} \quad (3.6b)$$

   where $F_{sc}$ is the force in the UFP connectors.

The PRESSS guidelines implied use of a constant value for $F_{sc}$ in Eq. 3.6 irrespective of the selected $\theta$. As will be shown later, this approach is not satisfactory. An appropriate value for $F_{sc}$ must be expressed as a function of $\theta$ or conveniently as a function of relative vertical displacement between the walls. This relationship suitable for the UFPs used in the PRESSS building has been recently established in a separate study by researchers at Iowa State University. Figure 3.3 shows the UFP force-displacement response envelope for an individual UFP. This response envelope is
used in the validation process in the following manner. Assuming a relative vertical
displacement between the two walls, determine the appropriate $F_{sc}$ directly from
Figure 3.3; the relative vertical displacement found in the previous iteration may be
used as a satisfactory guess. At the beginning of the analysis with the first value for $\theta$,
an initial guess could not be made in this manner and was assumed to be $0.5l_v\theta$. Once
a value for the force in an individual UFP is obtained via Figure 3.3, it should be
multiplied by the number of UFPs along the vertical joint to determine the total force,
$F_{sc}$.

Figure 3.2 Forces acting on a jointed wall system consisting of two walls.
Calculate the distance from the extreme compression fiber to the center of the compression force divided by the length of the wall \((l_w)\):

\[
\alpha = 0.5 \frac{C}{C_c}
\]  
(3.7)

where \(\alpha l_w\) defines the distance from the wall edge to the resultant compression force.

Calculate the distance from the extreme compression fiber to the neutral axis depth divided by the length of the wall \((l_w)\):

\[
\eta = 2 \frac{\alpha}{\beta_1}
\]  
(3.8)

where \(\eta l_w\) defines the neutral axis depth at the wall base.

Calculate the wall end uplift:

\[
\Delta_{\text{enduplift}} = \theta l_w (1 - \eta)
\]  
(3.9)

For the new \(\Delta_{\text{enduplift}}\) determine the corresponding value for \(F_{sc}\) from Figure 3.3.

Calculate the elongation of the post-tensioning tendon:

\[
\Delta_p = \theta l_w (0.5 - \eta)
\]  
(3.10)

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

\[
\Delta f_p = E_p \frac{\Delta_p}{h_u}
\]  
(3.11)

Calculate the total stress in the post-tensioning tendon:

\[
f_p = (f_{p0} + \Delta f_p) \leq f_{py}
\]  
(3.12)

Recalculate the total post-tensioning force:

\[
P = A_p f_p
\]  
(3.13)
Figure 3.3  Force transferred through a UFP connector as a function of relative vertical displacement between the wall panels established from experimentation.

Iterate Step 5 until P converges to the assumed value. Although a non-iterative approach is presented in the guidelines, discussed previously in Section 2.5.2, the iterative approach was used. This was necessary since the UFP force used in the analysis method is not considered constant as in the design guidelines. Note that the non-iterative procedure in the guidelines is expressed in terms of $F_{sc}$ (see Eq. 2.34b).

6.  **Compute resisting moment of wall:**

$$M_{\text{cap,panel}} = l_w \left( C(0.5 - \alpha) + 0.5(F_{sc}) \right)$$  \hspace{1cm} (3.14)

Steps 3 through 6 should now be repeated for the trailing wall.
7. **Compute resisting moment of wall system:**

\[ M_{\text{cap,wall}} = \sum M_{\text{cap,panel}} \]  \hspace{1cm} (3.15)

### 3.3 Alternative Analysis Method

The alternative analysis method is based on the monolithic beam analogy (MBA) concept suggested by Pampanin et al. [34] as discussed in Section 2.5.2. Using this concept, a procedure for performing section analysis of unbonded post-tensioned walls was established [35]. A summary of this procedure and the analysis of the jointed wall using this procedure are described below.

#### 3.3.1 Application of Monolithic Beam Analogy

In order to overcome strain incompatibilities between the unbonded steel and concrete, MBA approximates the total displacement of the jointed precast wall \( \Delta_j \) to the total displacement of an equivalent monolithic wall \( \Delta_m \) as shown in Figure 3.4, and establishes a simplified relationship for the extreme fiber concrete strain \( \varepsilon_{c,\text{ext}} \) as a function of the neutral axis depth \( (c) \) and base rotation \( (\theta) \). Accordingly, assume:

\[ \Delta_j = \Delta_m \]  \hspace{1cm} (3.16)

At a base rotation of \( \theta \) for the precast wall,

\[ \theta h_w + \frac{1}{3} \phi^*_c h_w^2 = \Delta_e + \Delta_p \]  \hspace{1cm} (3.17)

where \( h_w \) is the height of the wall, \( \phi^*_c \) is the elastic curvature at the base of the precast wall and \( \Delta_e \) and \( \Delta_p \) are, respectively, the elastic and plastic displacement components of the monolithic wall.

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For the equivalent monolithic wall,

\[ \Delta_e = \frac{1}{3} \phi_e h_w^2 \]  \hspace{1cm} (3.18)

where \( \phi_e \) is the elastic curvature of the monolithic wall, and

\[ \Delta_p = \phi_p L_p h_w = (\phi_u - \phi_e) L_p h_w \]  \hspace{1cm} (3.19)

where \( \phi_p \) is the plastic curvature of the monolithic wall, \( L_p \) is the equivalent plastic hinge length and \( \phi_u \) is the ultimate curvature of the monolithic wall. Assuming \( \phi_e^* \) is approximately equal to \( \phi_e \) and substituting equations 3.18 and 3.19 into equation 3.17, results in:

\[ \theta = (\phi_u - \phi_e) L_p = \left( \frac{\varepsilon_{c,ext}}{c} - \phi_e \right) L_p \]  \hspace{1cm} (3.20)

where \( \varepsilon_{c,ext} \) is the extreme fiber concrete strain. Thus,

**Figure 3.4** Flexural response and associated curvatures: (a) and (b) corresponds to a precast wall, and (c) and (d) correspond to an equivalent monolithic wall.
\[ \varepsilon_{c,ext} = c \left( \frac{\theta}{L_p} + \phi_c \right) \]  \hspace{1cm} (3.21)

Since there is no mild reinforcing steel connecting the precast wall to the foundation, strain penetration effects are not possible. Thus, the plastic hinge length \((L_p)\) is taken as \(0.08h_w\) as per Paulay and Priestley [36] for the plastic curvature distribution expected along the wall height. This results in the following relationship:

\[ \varepsilon_{c,ext} = c \left( \frac{\theta}{0.08h_w} + \phi_c \right) \]  \hspace{1cm} (3.22)

The elastic curvature may be defined as:

\[ \phi_c = \frac{M}{E_c I_{eff}} \]  \hspace{1cm} (3.23)

where \(M\) is the base moment resistance of the wall at the given neutral axis depth \((c)\) and base rotation \((\theta)\), \(E_c\) is the modulus of elasticity of concrete and \(I_{eff}\) is the effective moment of inertia of the wall.

### 3.3.2 Step-by-Step Procedure

The steps involved in the MBA analysis method for the jointed wall system is summarized in Figure 3.5. As in the PRESSS analysis procedure, the walls are analyzed individually and the total resistance of the wall system is found by adding the resistance of the leading and trailing walls. The steps involved in this approach are as follows:
1. **Define wall dimensions and material properties.**

This includes:

- $f'_c$ = concrete strength
- $\gamma_c$ = concrete density
- $P_0$ = initial post-tensioning force
- $E_p$ = post-tensioning modulus of elasticity
- $f_{py}$ = post-tensioning yield strength
- $A_p$ = area of post-tensioning
- $n$ = number of walls
- $h_w$ = height of wall
- $l_w$ = length of wall panel
- $t_w$ = thickness of wall

Note that $f'_g$ is assumed to be greater than $f'_c$ thus the neutral axis depth at the base of the wall is determined by $f'_c$.

2. **Calculate the decompression point.**

The decompression point is the point at which axial compression stress introduced by post-tensioning and self weight is overcome by the lateral forces on the wall, resulting in a stress of zero at one end of the wall. This is illustrated in Figure 3.6, where Figure 3.6a represents the stress profile at the base of the wall caused by the initial post-tensioning and the gravity loads including the self-weight of the wall. The corresponding stress can be calculated as:

\[
\sigma_0 = \frac{P_0 + W}{l_w t_w}
\]  

(3.24)
Figure 3.5 Flowchart summarizing the main steps involved in the wall system analysis using the MBA concept.
Figure 3.6b represents the effects of the wall due to lateral force $F_{\text{decomp}}$ on the wall panel, which causes an extreme fiber tension stress at the base equal to $\sigma_0$ as defined in Figure 3.6a. Therefore, when these two stress profiles are added together, as in Figure 3.6c, the stress at one end of the wall base is doubled while the stress at the other end is zero, representing the decompression point. At this point, the wall is assumed to be ready to lift off and a gap will develop at the base when the lateral force is increased any further.

The base moment resistance of the wall at the decompression point is obtained using the known flexure formula as follows:

$$M_{\text{decomp}} = \frac{\sigma_0 I_g}{c}$$ (3.25)

where $I_g$ is the moment of inertia of the wall panel based on the gross section and $c$ is the neutral axis depth which is equal to one half of the wall length at this stage (i.e., $l_w/2$, see Figure 3.6b).

Likewise, the top floor displacement may be estimated by calculating the elastic curvature corresponding to the base moment at the decompression point:

$$\phi_e = \frac{M_{\text{decomp}}}{E_c I}$$ (3.26)

The top floor displacement at the decompression point is thus:

$$\Delta_{\text{top,decomp}} = \frac{1}{3} \phi_e h_w^2$$ (3.27)
Figure 3.6 Stress profile at the base of the wall caused by: (a) the initial post-tensioning and gravity loads including the self-weight of the wall, (b) lateral force $F_{\text{decomp}}$ on the wall, and (c) by combination of loads from (a) and (b).

Note that the rotation at the interface between the wall and foundation is assumed to be zero at the decompression point.

Assuming axial stresses due to gravity loads and post-tensioning force are the same for leading and trailing walls, the decompression is expected simultaneously at both wall bases. At this stage, the base moment and top floor displacement of the jointed wall system are defined as:
\[ M_{\text{decomp}} = \frac{2\sigma_0 I_g}{c} \quad (3.28) \]

\[ \Delta_{\text{top,decomp}} = \frac{1}{3} \phi_e h_w^2 \quad (3.29) \]

3. **Select base rotation, \( \theta \).**

The first value of \( \theta \) is taken as \( \theta_{\text{decomp}} + 0.0001 \) and the subsequent values are taken at an increment of 0.0001 until a maximum value of 0.03 is reached, where \( \theta_{\text{decomp}} = 0. \)

4. **Assume a neutral axis dept, \( c \).**

For the selected \( \theta \), assume an appropriate initial value for \( c \) so that the final value for \( c \) may be established using an iterative procedure involving Steps 5 through 7. The final \( c \) value corresponding to the previous step may be taken as the initial \( c \) value in the current step.

5. **From the equilibrium condition calculate forces at base rotation \( \theta \) with the assumed neutral axis depth.**

Utilizing the wall geometry, illustrated in Figure 3.7, calculate the tendon elongation:

\[ \Delta_p = \left( \frac{w_0}{2} - c \right) \theta \quad (3.30) \]

Calculate the increase in tendon stress:

\[ \Delta f_p = E_p \frac{\Delta_p}{h_w} \quad (3.31) \]
Calculate the total post-tensioning force \( (P) \) and the total tension force \( (N) \) under the current base rotation and assumed neutral axis depth:

\[
P = \Delta f_p A_p + P_0 \quad (3.32)
\]

\[
N = P + W \quad (3.33)
\]

The relative vertical displacement between the two adjacent walls is approximated to the wall-end uplift (Figure 3.7b), which is estimated using the geometry of the wall as:

\[
\Delta_{\text{end uplift}} = (l_w - c) \theta \quad (3.34)
\]

![Diagram showing forces acting on a jointed wall system](image)

**Figure 3.7** Forces acting on the jointed wall system at base rotation \( \theta \): (a) full view, and (b) enlarged view at the base of a wall.
For a given value of $\Delta_{\text{enduplift}}$ from equation 3.34, the force transfer through a single UFP ($F_{sc}$) is determined from Figure 3.3, which was presented in Section 3.2. Using the total number of UFPs ($n_{sc}$) located between the wall panels, the force transfer along the vertical joint is calculated from Eq. 3.35.

$$F_{sc} = n_{sc} F_{sc0}$$  \hspace{1cm} (3.35)

Finally, the compressive force ($C$) can be determined from the equilibrium condition of the wall panel in the vertical direction:

$$C = N + F_{sc} \quad \text{for the leading wall} \quad (3.36a)$$

$$C = N - F_{sc} \quad \text{for the trailing wall} \quad (3.36b)$$

6. **Using MBA determine extreme fiber concrete strain.**

As discussed previously in Section 3.3.1 (see Eqs. 3.22 and 3.23), the extreme fiber compression strain may be approximated to:

$$\varepsilon_{c,\text{ext}} = \varepsilon \left( \frac{\theta}{0.08 h_w} + \frac{M}{E_c I_{\text{eff}}} \right)$$  \hspace{1cm} (3.37)

where $E_c$ is the modulus of elasticity for concrete, $I_{\text{eff}}$ is the effective moment of inertia of the wall taken as 100% of $I_g$, where $I_g$ is the moment of inertia of the gross concrete section, and $M$ is the base moment resistance of the wall panel at $\theta$, which is unknown and therefore the moment determined for $\theta$ in the previous step was used.

7. **Using a confined concrete model calculate the compression force and its location.**

The confined concrete model suggested by Mander et al. [37] was selected to
determine the stress profile. According to this model, the stress-strain relationship of confined concrete may be expressed as:

\[ f_e = \frac{f'_{cc}xr}{r - 1 + x'r} \]  \hspace{1cm} (3.38)

where:

\[ x = \frac{\varepsilon_c}{\varepsilon_{cc}} \]  \hspace{1cm} (3.39)

\[ r = \frac{E_c}{E_c - E_{sec}} \]  \hspace{1cm} (3.40)

\[ f'_{cc} = f_c \left( 2.254 \left( 1 + \frac{7.94f'_l}{f'_c} - \frac{2f'_l}{f'_c} - 1.254 \right) \right) \]  \hspace{1cm} (3.41)

\[ \varepsilon_{cc} = 0.002 \left[ 1 + 5 \left( \frac{f'_{cc}}{f'_c} - 1 \right) \right] \]  \hspace{1cm} (3.42)

\[ E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}} \]  \hspace{1cm} (3.43)

where \( \varepsilon_c \) is the concrete strain, \( E_c \) is the modulus of elasticity for concrete, \( f_c \) is the stress at strain \( \varepsilon_c \), \( f'_{cc} \) is the peak confined concrete strength, \( \varepsilon_{cc} \) is the corresponding strain, \( f_l \) is the maximum lateral confining pressure and \( f'_l \) is the effective lateral confining stress. While assumed equal to zero for unconfined regions, \( f'_l \) is the sum of Eqs. 3.44 and 3.45 for rectangular confined regions:

\[ f'_{lx} = K_x \rho_x f_yh \]  \hspace{1cm} (3.44)

\[ f'_{ly} = K_y \rho_y f_yh \]  \hspace{1cm} (3.45)
where \( \rho_x \) and \( \rho_y \) are the transverse reinforcement area ratios in the principal directions, \( f_{yh} \) is the yield stress of the transverse reinforcement and \( K_e \) is the confinement effectiveness coefficient recommended as 0.6 for rectangular wall sections [36].

The stress-strain curve obtained for the walls in the PRESSS building from the above model and measured average unconfined concrete strengths is shown in Figure 3.8. Using these stress profiles to model the confined and unconfined concrete, the resultant compressive force and its location can be obtained. In this study these calculations were performed using the Simpson’s rule by dividing the profile into several rectangular sections as illustrated for a compression region in Figure 3.9. The resultant compressive force \( (C_{\text{conf}}) \) is found by summing the forces represented by the rectangular areas. Similarly, the location of the resultant force is found by summing up the individual rectangular areas multiplied by the distance to their individual centroids from the neutral axis and then dividing this term by the summation of the areas.

The resultant compressive force obtained from the confinement model is compared with the compressive force obtained from the equilibrium condition in Eq. 3.36. If the confined compressive force \( (C_{\text{conf}}) \) is not equal to the compressive force established by equilibrium \( (C) \), then the neutral axis depth is increased and the process (Steps 5 through 7) is repeated until the two forces converge.
Figure 3.8 Confined and unconfined concrete stress-strain relationship established for the PRESSS walls using the Mander’s model and measured concrete strengths.

Figure 3.9 Representation of the compression zone with several rectangular sections to compute the resultant force and its location using the Simpson’s rule.
8. **Compute the resisting moment of the wall:**

\[
M_{\text{cap,panel1}} = C(l_w - y) - N\left(\frac{l_w}{2}\right) \tag{3.46}
\]

\[
M_{\text{cap,panel2}} = -C(y) + N\left(\frac{l_w}{2}\right) \tag{3.47}
\]

where \(y\) is the distance from the edge of the wall to the resultant compression force determined for the stress distribution in Figure 3.9. Steps 5 through 8 should be iterated once more to ensure that the MBA (Step 6) utilizes an accurate moment in computing the extreme fiber concrete strain (see Eq. 3.37).

9. **Compute the resisting moment at the base of the wall system:**

\[
M_{\text{cap,wall}} = M_{\text{cap,panel1}} + M_{\text{cap,panel2}} \tag{3.48}
\]
Chapter 4: Comparison of Experimental and Analytical Results

4.1 Introduction

This chapter examines the adequacy of the analytical methods developed in Chapter 3 using the PRESSS test data obtained in the wall direction testing and makes appropriate recommendations to improve the analytical methods as well as the design guidelines. The wall direction response of the PRESSS building was suspected to have been significantly influenced by the framing action resulting primarily from the seismic columns and precast floors incorporated in the building [30,35]. Furthermore, comparisons between the analytical results and experimental base moment-top floor displacement response envelope representing that due to a monotonic loading are adequate to validate and improve the guidelines, if necessary. Consequently, the response envelope corresponding to the jointed wall system in the PRESSS building was established from the measured experimental data. This envelope, which excludes the framing action of the seismic columns and precast floors, was primarily used in the validation part of the study. The moment contribution due to the framing action in the wall direction testing was subsequently analyzed using a two-dimensional (2-D) model. It is shown that response envelopes established for the jointed wall and frames add up to the total measured response of the PRESSS building in the wall direction.

4.2 Experimental Data

The measured response envelope for the jointed wall was established using the experimental data obtained during the PRESSS test and the details are presented in the following sections.
4.2.1 Test Setup

The test setup adopted for the wall direction testing of the PRESSS building is shown in Figure 4.1 and described in detail in Ref. [16]. Two servo-controlled hydraulic actuators per floor were used for the seismic force simulation. The actuators were positioned eight feet apart horizontally and 14 inches vertically from the top of the floors to the centerline of the actuators. The actuator forces at each location were transmitted to the floor through two pin-connected loading arms. As shown in Figure 4.2, the total base moment resistance of the PRESSS building was obtained by summing the moments induced by the actuator force times the vertical distance from the base of the building to the actuator location.

![Figure 4.1 Test setup in the wall direction testing of the PRESSS building [16].](image-url)
Figure 4.2 Observed response of the PRESSS test building in the jointed wall direction with data points selected for isolating the response envelope of the jointed wall.

The actuator forces would have induced lateral and vertical forces at the floor levels as illustrated in Figure 4.3. Based on the pin connection details adopted in the PRESSS building, these forces would have been transferred to the walls in the upper two floors and were included in the wall analysis. Since the jointed walls were connected to the lower three floors through pins, which were free to move in the vertical direction, the vertical forces induced by the actuators were not transmitted to the jointed wall and would have been resisted by the frame comprised of the seismic columns and precast floors. This frame would have also resisted a small portion of the lateral forces, leaving the rest of the lateral forces to
be resisted by the jointed wall. As a result, separating the jointed wall resistance from the measured response is not straightforward. Therefore, the data acquired by instrumentation attached to the wall system was used to isolate the wall system response envelope independent of the framing action developed during testing of the PRESSS building.

![Diagram](image)

**Figure 4.3** Typical floor forces introduced during testing of the PRESSS building.

### 4.2.2 Selected Data Points

The experimental base moment-displacement response of the wall system was formulated using data points derived at selected measured displacements. As marked with solid circles in Figure 4.2, data corresponding to eight first cycle peaks were utilized to establish the change in the post-tensioning force, the amount of force transferred through the UFP connectors and the neutral axis depth as a function of the top floor displacement. Using this information and equilibrium conditions, the base moment resistance of the wall system was determined. In this process, wall dimensions and properties as defined in Table 4.1 were used. The post-tensioning force reported in this table was taken on 5% above the design level based on the
strain gage data, while the concrete strengths were those measured on the day of testing in the wall direction.

Table 4.1 Dimensions and properties of the walls in the PRESSS building.

<table>
<thead>
<tr>
<th>Property</th>
<th>Wall Panel W1/W2</th>
<th>Wall Panel W1R/W2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Strength (f'_c)</td>
<td>7.31 ksi</td>
<td>7.96 ksi</td>
</tr>
<tr>
<td>Wall Height (h_w)</td>
<td>450 in</td>
<td>450 in</td>
</tr>
<tr>
<td>Wall Length (l_w)</td>
<td>108 in</td>
<td>108 in</td>
</tr>
<tr>
<td>Wall Thickness (t_w)</td>
<td>8 in</td>
<td>8 in</td>
</tr>
<tr>
<td>Dead Load at the Wall Base (W)</td>
<td>65.6 kips (at W1 base)</td>
<td>65.6 kips (at W1R base)</td>
</tr>
<tr>
<td>Initial Post Tensioning Force (P_o)</td>
<td>172.2 kips (4@43.05)</td>
<td>172.2 kips (4@43.05)</td>
</tr>
<tr>
<td>Area of Post-Tensioning Tendons (A_p)</td>
<td>3.4 in^2 (4@0.85 in^2)</td>
<td>3.4 in^2 (4@0.85 in^2)</td>
</tr>
<tr>
<td>Elastic Modulus of Post-tensioning Steel (E_p)</td>
<td>27700 in^2</td>
<td>27700 in^2</td>
</tr>
</tbody>
</table>

4.2.3 Determining Critical Parameters

Six displacement transducers measured the vertical displacements of the walls at the base with respect to the wall foundation during testing. These devices are identified along with their locations in Figure 4.4. Using the data from these devices, the neutral axis depths (c) and the post-tensioning tendons elongations (Δ_p) were determined for the leading and trailing walls at the selected data points. Also estimated from the displacement devices was the wall end uplift (Δ_enduplift) of the leading wall, which was assumed to be the same as the relative vertical displacement between the walls.
**Figure 4.4** Plan view of instruments measuring vertical displacements at the base of the wall system.

Assuming a linear profile at the bottom of the wall as shown in Figure 4.5, the neutral axis depths or the lengths of the contact surface between the wall and the foundation were determined at the selected lateral displacements by interpolating the displacements measured at two locations (i.e., W1VBE and W1VBN as depicted in Figure 4.5). Additionally, this process was used to determine the post-tensioning elongation, end uplift and base rotation.

**Figure 4.5** North profile view of instruments at base of wall panel W1.
The assumption of a linear profile used above is considered satisfactory based on test results reported by Rosenboom [38] from unbonded post-tensioned masonry walls, where the researcher measured displacements between the wall and foundation at multiple locations.

### 4.2.4 Validity of Extracted Data

Additional data obtained during the PRESSS test were used to validate the data extraction as presented in the above section. Of the four wall parameters determined in Section 4.2.3 at selected displacements, independent data measurements were available to verify $\Delta_{\text{enduplift}}$ and $\Delta_p$. The extracted data are compared with the directly measured data below, which confirm that data extraction presented in the previous section is satisfactory.

A displacement transducer, W1VCS, was located close to the wall foundation in the PRESSS building, which measured the relative vertical displacement between the two walls along the vertical joint (Figure 4.6). The measurements from this device included the wall end uplift of the leading wall as well as the small deformation resulting from the trailing wall pushing into the grout pad at the foundation interface. The data extracted from the base devices are compared with the data recorded directly by W1VCS in Figure 4.7. The extracted data is only about 4% less than the displacement recorded by W1VCS at the maximum top floor lateral displacement of 11.5 inches.
Figure 4.6 Location of device W1VCS.

Figure 4.7 Vertical displacement history experienced by the UFP connectors during the PRESSS building test in the wall direction.

Strain gage measurements obtained from the post-tensioning bars facilitated examination of the bar elongation estimated from the base devices. A direct comparison between the data sets was not possible. In Figure 4.5, the effect of post-tensioning was modeled at the
centroidal location of the wall section at a distance of $0.5l_w$ from the wall end. While in fact, each wall in the PRESSS building was anchored with four 1-in. diameter Dywidag bars located at a distance of 4.5-in. on either side of the centroidal axis. Adjusting the $\Delta p$ determined from the base devices to reflect effects of the exact location of the bars containing the strain gages and dividing the adjusted $\Delta p$ values by the unbonded length of the post-tensioning to compute strains, Figure 4.8 compares the directly measured and computed strains from $\Delta p$. At the maximum top floor displacement of 11.5-in, the data extracted from the displacement devices at the base overestimated the strains by 5.8% and 1.1% in leading and trailing walls, respectively.
Figure 4.8 A comparison of experimental strains due to elongation of the post-tensioning tendons.

The discrepancies between the directly measured data and those extracted from the displacement measurements at the base of the walls were small and believed to have been due to (a) neglecting the effects of the trailing wall pushing into the grout pad, and (b) the linear profile assumed for the base rotation as in Figure 4.5. As illustrated in Figure 4.9, the actual profile of the wall end is most likely not perfectly linear. These reasons are consistent with the fact that the data extracted from the displacement measurements underestimated the wall end uplift and overestimated the increase in the post-tensioning bar.

Figure 4.9 Wall base profile comparison.

The representation of a more realistic wall base profile as shown in Figure 4.9 indicates that the actual neutral axis depth of the wall may be somewhat greater than those estimated by the base devices. However, the expected difference in the neutral axis depths, which may be
quantified in future tests using displacement devices at multiple locations along the wall base, had no significant effect on the moment resistance of the wall system.

### 4.2.5 Best Fit Curves

For the extracted data reported in Section 4.2.3, best fit curves were determined so that a continuous curve representing the base moment-lateral load response could be developed for the jointed wall system tested in the PRESSS building. Figures 4.10 and 4.11 show the extracted data and best fit curves for the wall end uplift and elongations of the post-tensioning tendons at the centroids of the wall sections. In both figures, variations of the critical parameters as a function of top floor displacement are represented with straight lines.

![Figure 4.10](image)  
**Figure 4.10** Wall end uplift estimated from test data.
Figure 4.11 Post-tensioning elongation estimated from test data.

Figure 4.12 shows the extracted data for the neutral axis depths and the best fit curves. A second order polynomial curve was used to represent the variation in the neutral axis depth of the trailing wall. A similar approach was adopted for the leading wall up to a lateral displacement of 5.3 inches (or 1.2% drift), beyond which no change in the neutral axis depth was assumed as indicated by the data. The presence of a constant neutral axis depth during the response of the jointed systems in the moderate to large lateral drifts has been supported by analytical and experimental results [9].

4.2.6 Response Envelope

Using the best fit curves presented in Section 4.2.5, the base moment-top floor displacement envelope for the jointed wall system in the PRESSS building was established using the equilibrium conditions at the wall bases. In this procedure, the location of the resultant
compressive force was assumed at a distance of $0.5\beta_1 c$ from the compression face for each wall, where $c$ is the neutral axis depth and $\beta_1$ is the depth of equivalent stress block divided by the neutral axis depth. This assumption was not expected to introduce any significant error as $c$ is based on experimental data as shown in Figure 4.12. The resulting response envelope of the PRESSS wall system is included in Figure 4.13 as the experimental curve.

![Figure 4.12](image_url)

**Figure 4.12** Neutral axis depths estimated from test data.

### 4.3 Validation of Analytical Methods

In this section, predicted results from the two analytical methods developed in Chapter 3 based on the PRESSS guidelines and the MBA concept are compared with experimental results including the base moment-top floor lateral displacement response established using the extracted test data in Section 4.2.
4.3.1 Base Moment Resistance

Figure 4.13 compares the base moment-top floor lateral displacement of the jointed wall system determined from the analysis methods with that extracted from experimental data. There are two curves included for the prediction based on the PRESSS guidelines. In the first one, the force transmitted by the UFP connectors was determined from Figure 3.3, while the second curve was based on the recommended approach for computing the force in UFP connectors as per the PRESSS guidelines. As can be seen in Figure 4.13, the prediction of the PRESSS guidelines assuming a constant force through the UFP connectors was unsatisfactory. This approach underestimated the wall response by as much as 21.8% at large lateral displacements. At small top floor displacements (< 1.5in.), this approach was found to be inappropriate, assuming significantly high UFP forces and resulting in significant overestimation of the base moment resistance of the jointed wall. Therefore, use of a constant UFP force regardless of the base rotation was not further considered.

It is noted that the PRESSS guidelines were probably not meant to be applied at small top floor displacements. However, the validity of the analytical methods established for the jointed wall were examined over a range of lateral displacements to identify the shortcomings of the proposed guidelines and to highlight the benefits of improvements suggested for the design and analytical methods. With validation of the design and analytical methods from small to large lateral displacements, a performance-based design method can be easily established for the precast jointed wall system.
With the appropriate force in the UFP connectors, it can be seen that the analysis methods based on the PRESSS guidelines and the MBA concept provide better predictions of the jointed wall response. At a top floor displacement of 11.5 in., the PRESSS method was 12.2% and the MBA method was 5.0% below the isolated response envelope of the jointed wall. At a design drift of 2% (corresponding to a top floor displacement of 9 in.), assumed for design of the jointed wall, the PRESSS and MBA methods underestimated the moment resistance of the jointed wall by 9.9% and 4.1%, respectively.

4.3.2 Post-tensioning Bar Elongation
The two analysis methods, although producing similar base moment-lateral displacement
envelopes, differed noticeably when critical parameters used in design were compared at the critical sections. Figures 4.14 and 4.15 show the calculated post-tensioning bar elongation for both the leading and trailing walls along with the experimental data extracted previously. The post-tensioning elongation was found to be underestimated by both analysis methods. The elongation at the maximum top floor displacement of 11.5 in. (or 2.56% drift) in the leading wall was underestimated by 26.0% using the PRESSS guidelines and by 15.3% using the MBA method. Likewise, in the trailing wall the elongation was underestimated by 11.4% and 11.8% by the PRESSS guidelines and MBA, respectively. This underestimation, primarily of the PRESSS leading wall, is of a concern since accurate prediction of the post-tensioning force is essential for cost effectively designing an appropriate amount of post-tensioning steel in the walls.

Another interesting observation from Figures 4.14 and 4.15 is that elongation of the post-tensioning is typically higher in the trailing wall than in the leading wall. This observation is supported by both experimental and analytical results. At the maximum lateral displacement of 11.5 in., the elongation obtained for the trailing wall from experimental data was 10.5% higher than that for the leading wall. Given the identical base rotations and different compression forces at the wall toes, it is conceivable that the trailing wall is subjected to reduced contact area, increasing the elongation of the post-tensioning steel more than that experienced by the tendons in the leading wall. A design implication of this observation is that the design of the post-tensioning steel area should account for both the required base moment resistance of the leading wall and expected elongation in the trailing wall.
Figure 4.14 Post-tensioning elongation in leading wall.

Figure 4.15 Post-tensioning elongation in trailing wall.
4.3.3 Neutral Axis Depth

The neutral axis depths calculated by the analytical methods are compared with experimental data in Figures 4.16 and 4.17. Contrary to the expected trend, in which the neutral axis depth at the base of the wall is expected to decrease at the beginning and then possibly remain constant as the top floor displacement increases, PRESSS guidelines generally predicts an increase in the neutral axis depth as the lateral displacement increases. At small displacements, the PRESSS guidelines predict the expected trend for the neutral axis depth in the trailing wall, but the values are significantly smaller than that predicted by the MBA method. The reason for the PRESSS guidelines to show a contrasting trend when predicting the neutral axis depth of the trailing wall at small lateral displacements is due to the interaction of the force in the UFP connector, as shown in Figure 3.3.

The expected trend and the experimentally extracted neutral axis depths were more satisfactorily predicted by the MBA method. Due to the approximate plastic hinge length used in the derivation of the analysis methods based on MBA and the assumption of a linear profile at the wall base used for extracting the data, the actual neutral axis depths in both walls are expected between the MBA prediction and those portrayed by the extracted data. In consideration of these issues, a more realistic prediction of the MBA method is discussed in Section 4.5.2.

The difference between the analytical and extracted neutral axis depths were found to be the cause for discrepancies observed between the predicted and experimental elongations of the
post-tensioning tendons in Figures 4.14 and 4.15. A more accurate estimation of the neutral axis depth will more accurately estimate the elongations in the post-tensioning tendons.

![Graph showing Top Floor Displacement (mm) vs. Neutral Axis Depth (mm)]

**Figure 4.16** Neutral axis depth in leading wall.

### 4.4 Contribution of Framing Action

It was reported in Section 4.1 that framing action resulting from the seismic columns and precast floors was suspected to have provided a significant amount of lateral load resistance in the wall direction testing of the PRESSS building. A small contribution from the base resistance of the two gravity columns was also expected. The moment resistance envelope, which was established for the jointed wall system in Section 4.3.1 from experimental data corresponded to about 80% of the total resistance. The unaccounted 20% base moment resistance supported the hypothesis that framing action played a significant role in the wall
direction testing. Using a 2-D model developed using the computer program RUAUMOKO [31], the contribution of the framing action is estimated in this section.

![Figure 4.17 Neutral axis depth in trailing wall.](image)

**Figure 4.17** Neutral axis depth in trailing wall.

### 4.4.1 2-D Frame Model

As shown in Figure 4.18, a 2-D model consisting of two columns and 5 beams was developed to quantify the framing action. The three seismic columns in each of the seismic frames, which were located on each side of the building perpendicular to the direction of the wall testing (see Figure 1.4), were modeled as one column, while the beam elements modeled the floor systems. The properties of the columns and floors established for the analysis are presented in Table 4.2, in which $E_c$, $G$, $A$ and $I_{eff}$ are, respectively, the elastic modulus, shear modulus, cross-sectional area and effective moment of inertia of the elements.
**Figure 4.18** 2-D RUAUMOKO frame model.

**Table 4.2** Properties of elements in the 2-D RUAUMOKO frame model.

<table>
<thead>
<tr>
<th>Member</th>
<th>$E_c$ (ksi)</th>
<th>$G$ (ksi)</th>
<th>$A$ (in$^2$)</th>
<th>$I_{eff}$ (in$^4$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left Column from Base to Floor 1</td>
<td>5500</td>
<td>2300</td>
<td>972</td>
<td>18370.8</td>
</tr>
<tr>
<td>Right Column from Base to Floor 1</td>
<td>5700</td>
<td>2400</td>
<td>972</td>
<td>18370.8</td>
</tr>
<tr>
<td>Left Column from Floor 1 to 5</td>
<td>5500</td>
<td>2300</td>
<td>972</td>
<td>26244</td>
</tr>
<tr>
<td>Right Column from Floor 1 to 5</td>
<td>5700</td>
<td>2400</td>
<td>972</td>
<td>26244</td>
</tr>
<tr>
<td>Floor 1, 2 and 3</td>
<td>3300</td>
<td>1400</td>
<td>2545.5</td>
<td>146250.5</td>
</tr>
<tr>
<td>Floor 4 and 5</td>
<td>3300</td>
<td>1400</td>
<td>2880</td>
<td>15360</td>
</tr>
</tbody>
</table>
The effective moment of inertia ($I_{eff}$) for the seismic columns from the base to the first floor level was taken as $0.70I_g$, where $I_g$ is the gross moment of inertia of the column section. This approximation was made to reflect flexural cracking that the seismic columns experienced below the first floor level during wall direction testing of the PRESSS building [15]. Over the upper four stories of the building, the effective moment of inertia of the seismic columns was taken as $I_g$ since no visible cracking was developed. For the beam elements modeling the floors, the effective moment of inertia was taken as $I_g$ at all levels.

The connections at the column bases and at both ends of the beam elements in the first three floors were expected to have moment resistance and eventual formation of plastic hinges. These connections were represented with rotational springs to model the appropriate behavior as detailed below. In the beam ends connecting the columns at the fourth and fifth floor levels, the moment resistance was not needed to be modeled since vertical components of the actuator forces were designed to be transferred to the walls. The pin connections in the steel channels that supported the precast panels at these floor levels prevented them from contributing to the framing action. Hence, these connections were modeled as perfect pins.

Similar to the wall-to-foundation connection, the six seismic columns in the PRESSS building were connected to the footings using two 1.25-in diameter post-tensioning bars during the wall direction tests [16]. Hence, the analysis developed for the walls based on MBA (see Section 3.3) was directly applied to characterize the base moment-rotation behavior for these columns. After the moment-rotation responses were determined individually at the base of the seismic columns, they were combined together so that the
behavior of the three columns from each seismic frame could be modeled with a single column as shown in Figure 4.18. The combined moment-rotation behavior of the column base resistances are depicted in Figure 4.19, in which the differences in the behavior between leading and trailing columns was due to the small difference in the self-weight of the seismic frames.

![Figure 4.19 Column moment-rotation behavior for the model in Figure 4.18.](image)

The moment-rotation behavior at the base of the gravity columns was also established in a similar manner. The main difference between the seismic and gravity columns was that the latter was connected to the foundation with two post-tensioning bars as well as a 0.75-in. diameter anchor bolt located at each corner. Again the moment resistance calculation followed the MBA concept, with a modification to the definition of the plastic hinge length.
defined in Section 3.3.1. Recognizing that strain penetration into the footing was possible through anchorage of the anchor bolts, the new $L_p$ was taken as:

$$L_p = 0.08h_c + 0.15f_y d_b$$  \hspace{1cm} (4.1)

where $h_c$ is the height of the column, $f_y$ and $d_b$ are, respectively, the yield strength and diameter of the anchor bolt.

It is noted that the resistance of the gravity columns were not included in the 2-D frame model shown earlier in Figure 4.18. However, the base moment-lateral displacement of the columns was determined and is included in Figure 4.21, which shows the total contribution of the two gravity columns designed in the PRESSS building. The contribution of the lateral resistance of the gravity columns were added to the total moment response in Section 4.4.2.

The moment-rotation behavior of connections at Floors 1 through 3 (Figure 4.20) was determined to be dependent on the bearing pad supporting the weight of the double tees. The total weight of the double tees at each floor level was determined to be 80-kips, therefore 40-kips on each side of the building. Since the connection details included a tie down rod, which kept the double tees from lifting off of the pad, and the fact that no slippage was seen during the testing, a coefficient of friction between the steel and bearing pad was estimated at 1.0. Therefore, a yield moment of 800 kip-in. was established along with a yield rotation of 0.02 for the rotational springs at Floors 1 through 3 (see Figure 4.18).
4.4.2 Total Response

After the model was set up to capture the framing action, an inverse triangular push over analysis was performed. This was done by proportionally increasing the load at each floor level, where the force at the fifth floor is five times that of the first, the fourth is four times that of the first, and so on as shown in Figure 4.18. The individual results for the frame model and the gravity columns as well as the combined predicted response using the MBA analysis results are compared with the experimental response in Figure 4.21.

The additional moment resistance included in the combined response is from two sources: the moment resistance at the base of the seismic and gravity columns as well as the resistance produced by the axial forces induced at the column bases in the model shown in Figure 4.18, which created a moment couple. This moment couple, or framing action, accounted for over 8200 kip-in., which corresponded to an axial load of 22.9 kips in each column when they...
reached a top floor displacement of 11.5 inches. With the combined response, the analytical prediction in the wall direction fell only 0.7% below the experimental response at the maximum top floor displacement of 11.5 inches. Although the analysis used a simple frame model, it is satisfactory to confirm the accuracy of the MBA model for the wall system acting independently.

![Graph showing base-moment versus top floor displacement response comparison for the jointed wall system in the PRESSS building.](image)

**Figure 4.21** Base-moment versus top floor displacement response comparison for the jointed wall system in the PRESSS building.

It should be noted that Conley et al. [30] satisfactorily predicted the behavior of the wall direction response without accounting for the framing action. As discussed in Section 2.5.1, it was suspected that the post-tensioning elongation and the force in the UFP connectors were
overestimated for the walls in their analytical model. Furthermore, the force-displacement curve assumed for the UFP connectors was higher than that experimentally produced for the PRESSS wall direction test. The analytical model used by Conley et al. may be improved by incorporating the frame system and more accurately modeling the neutral axis depth and contribution of the UFP connectors.

4.5 Recommendations

After completion of the analytical investigation, several aspects were examined to produce a set of recommendations. The following two sections cover recommendations to improve the PRESSS guidelines and the alternative method based on the MBA concept.

4.5.1 Recommendations for the PRESSS Guidelines

Several improvements are suggested for the use of the PRESSS design guidelines [18]. First of all, the strength of the grout ($f_{g}$) should be required to greater than the concrete strength of wall panels ($f_{c}$). This is important so that the strength of the concrete is used as the basis of the design at the base/foundation interface. This is primarily because the confinement effects on grout are unknown at this time and therefore the behavior of a weaker grout material would be difficult to account for.

Experimental and analytical results confirmed that the uplifts of the leading and trailing walls will be different at a given wall drift. Therefore, when using more than two walls in the system, the intermediate wall(s) should account for the force in the UFP connectors along
both vertical joints on either side of the panel. These two forces should not be considered equal and opposite and thus cancel out. The difference between the UFP forces acting on either side of the intermediate wall(s) will modify to the resultant compression force at the base and increase the overall moment resistance of the wall system.

When designing the post-tensioning tendons, the tendons should be designed for the forces in the leading wall and checked for yielding in the trailing wall. This is primarily because, as shown in Figures 4.14 and 4.15, the elongation in the trailing wall is greater than that in the leading wall. Furthermore, appropriate equations should be developed to quantify the moment contribution of the leading wall so that the post-tensioning steel can be efficiently designed.

Additionally, to take into account the confinement effects of the concrete, it is recommended to multiply the concrete strength ($f'_c$) by a factor of 1.6, along with a $\beta_1$ value of 0.85. Utilizing the increased concrete strength, determine the neutral axis depth using the PRESSS recommendations at a 2% base rotation and use this neutral axis depth for base rotations from 0 to 3%, as illustrated in Figure 4.22. This approach, which will simply the design calculations, has also been found to improve the design and response prediction of precast hybrid frame connections [33]. As reported in this reference, the design and analysis calculations should ideally assume two different concrete strengths for the confined and unconfined concrete strengths. However, a single value of $1.6 f'_c$ is suggested for simplicity.
Along with this, it is recommended to use an experimentally determined force-displacement interaction for the UFP connectors, as done in the analysis performed in this paper. Using these improved PRESSS design recommendations improves the neutral axis depth prediction and, as a result, improves the post-tensioning elongation (Figure 4.23) and moment response predictions (Figure 4.24). The post-tensioning elongation in the leading wall improved from 26% difference to only 5.8% difference at a top floor displacement of 11.5-in and the moment response improved from 12.2% to 3.5% difference at the same displacement.

**Figure 4.22** Neutral axis depth predicted by improved PRESSS recommendations.
Figure 4.23 Post-tensioning elongation predicted by improved PRESSS recommendations.

Figure 4.24 Moment response predicted by improved PRESSS recommendations.
4.5.2 Recommendations for MBA Method

Two improvements could be made to the MBA method: (1) a reduction in the plastic hinge length \( L_p \), and (2) use of an improved confinement model for the high strength concrete. The first improvement is suggested in recognition that a concentrated curvature distribution is expected in the connection region of the jointed systems when compared to the equivalent monolithic systems, which is expected to lead to smaller plastic hinge lengths for the jointed systems than for the monolithic systems. The second improvement is suggested because it was suspected that the confinement model by Mander et al. [37] may not capture the confined behavior of high strength concrete.

In order to test these improvements, an adjustment was made to the previously used data points by adjusting the displacement readings from the compression side to ensure that the neutral axis depth was not underestimated, as illustrated in Figure 4.9. Increasing the displacement measured by the device at the compression end of the walls (i.e. device W1VBN in Figure 4.5) by 1.5 times was found to be satisfactory as this more accurately predicted the post-tensioning strains as illustrated in Figure 4.25 when compared with Figure 4.8, while the wall end uplift was virtually unaffected (Figure 4.7). Consequently, a reduction in \( L_p \) from a value of 0.08\( h_w \), which is for use in monolithic walls, to a value of 0.06\( h_w \) was examined and found to predict the new neutral axis depth more accurately with the assumption that the confinement model does not reduce once the peak stress has been obtained (Figure 4.26). In other words, the confinement model behaves as described previously until a value \( f'_{cc} \) is obtained and then maintains this stress until the maximum strain predicted by the MBA method. This is done because at high strains the concrete stress
estimated by the confinement model of Mander et al. reduces, resulting in a reduction in compressive capacity and therefore an increase in neutral axis depth, but as shown experimentally the neutral axis depth decreases or remains unchanged with increased base rotation.

The benefits of recommendation proposed for the MBA concepts can be further observed in Figures 4.27 and 4.28, where the improved MBA model predicted the post-tensioning elongation and wall moment more accurately. The post-tensioning elongation in the leading wall improved from 15.3% difference to only 4.5% difference at a top floor displacement of 11.5 in. and the moment response improved from 5.0% to 4.4% difference at the same displacement.

![Figure 4.25 Post-tensioning strain by improved MBA method.](image-url)
Figure 4.26 Neutral axis depth predicted by the improved MBA method.

Figure 4.27 Post-tensioning elongation predicted by the improved MBA method.
Figure 4.28 Moment response predicted by the improved MBA method.
Chapter 5: Conclusions and Recommendations

5.1 Overview

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 in the United States is the code restriction of designing precast concrete structures to emulate the behavior of monolithic cast-in-place concrete structures. In consideration of these issues, PRESSS set out to improve the use and design of precast concrete structures in seismic regions. As a part of the PRESSS research, a five-story precast test building incorporating an unbonded post-tensioned precast concrete wall system was constructed and tested at the University of California at San Diego. During the testing it was found that the wall-direction moment response far surpassed all expectations.

With ambitions of implementing this system in practice, a set of design guidelines was established for unbonded post-tensioned jointed walls. Utilizing the PRESSS design guidelines, an analysis method was established (Section 3.2) for the purpose of validation. Additionally, due to concerns of inaccurate tendon elongation resulting from the use of the equivalent stress block in the PRESSS guidelines, an alternative analysis method based on the Monolithic Beam Analogy (MBA) concept (Section 3.3) was also developed to overcome the strain incompatibility issue and accurately represents the concrete stress block at the wall bases.
5.2 Conclusions

The following conclusions were drawn as a result of the analytical investigation based on both the PRESSS design guidelines and the MBA concept:

- Use of an experimentally obtained U-shaped flexural plate (UFP) force-displacement envelope more accurately predicts the moment response in the analysis of the jointed wall system. The representation of the force transmitted through the UFP connectors in the PRESSS guidelines is unsatisfactory and led to underestimation of the jointed wall resistance measured in the PRESSS building by as much as 22%.

- The framing action of the seismic frames and precast floors and the moment resistance of the gravity columns contributed to about 23% in the wall direction response of the PRESSS building. When the framing action is not included and the analysis prediction is based on the UFP force as recommended in the PRESSS guidelines, the moment resistance of the wall was under predicted by 45%. Such a significant discrepancy between the measured and analytical results would not assist with the promotion of the precast jointed wall system nor the PRESSS design guidelines.

- The base moment-top floor lateral displacement response envelope of the PRESSS wall system was satisfactorily predicted by both analysis methods when the measured force-displacement response was used for force in the UFP connectors. At a top floor displacement of 11.5 in., the PRESSS method was found to underestimate the moment response by 12.2% and the method based on MBA was found to be only 5.7% low.
The PRESSS analysis method resulted in an unsatisfactory prediction of the neutral axis depth by over 100% at a top floor displacement of 11.5 inches. As a result, the elongation of the post-tensioning tendon in the walls was underestimated by up to 26%. This is of major concern since accurate estimation of the increase in post-tensioning force is necessary for economically designing the area of the unbonded prestressing steel.

The MBA method was found to be satisfactory in predicting both the neutral axis depth and post-tensioning elongation in the wall system.

5.3 Recommendations

Based on observations made throughout this study presented in this report, it is suggested that the following changes be made to improve the PRESSS guidelines proposed for the jointed wall systems:

- At the wall base-foundation interface, the grout strength should be required to be greater than the concrete strength of the walls. This is primarily due to the unknown stress-strain characteristics of the grout with confinement effects. Requiring the concrete strength to be weaker than grout strength enables the former to govern design at the interface.

- An experimentally obtained force-displacement response envelope for the shear connector along the vertical joint between wall panels should be used to improve the design of wall systems.

- The force contribution from the shear connectors on intermediate walls should not be assumed as zero, since the corresponding force on either side of an intermediate wall
is not necessarily equal. Due to different wall end uplifts, the connector forces acting on either side of an intermediate wall are most likely to be different.

- The post-tensioning steel should be designed for the moment demand of the leading wall and then checked for potential yielding at the design drift using the elongation in the trailing wall. In addition, design equations should be developed to proportion the design moments between the walls.

- The equivalent stress block concept may be satisfactorily used in the design of the wall systems. However, the concrete strength should be increased by a factor of 1.6 to account for the confinement effects and to accurately estimate the neutral axis depths, elongations in the post-tensioning tendons, and force in the shear connectors. Furthermore, the neutral axis depth should be calculated at a base rotation of 2%, and this neutral axis depth may be used for all base rotations up to 3% in the design calculations as needed. As suggested by Celik and Sritharan [33], a tri-linear idealization may be considered for the neutral axis depth variation to improve the analytical prediction of the jointed wall system behavior.

- Once the PRESSS guidelines are updated based on the above recommendations, the equations provided for controlling residual drift, avoiding sliding failure, preventing uplift of wall panel etc. (see Eqs. 2.37 – 2.41) should be appropriately modified and verified against experimental data.

The following recommendations are suggested for improving the analysis method based on MBA:

- Additional improvements in the MBA method may be obtained by decreasing the
equivalent plastic hinge length to a value of 0.06 times the height of the wall ($h_w$). This reduction is suggested since the value of $0.08h_w$ was established for monolithic walls and due to the observed improvement in the prediction of the neutral axis depth, post-tensioning elongation and overall moment response when the plastic hinge length was approximated to 0.06 $h_w$.

- Use of a confinement model suitable for high strength concrete may further improve the MBA method by restricting the neutral axis depth from increasing at higher strains associated with increased base rotation.

5.4 Future Research

- An experimental investigation to verify the suggested plastic hinge length ($L_p$) for unbonded post-tensioned precast systems should be performed to improve the MBA analysis method.

- Testing of unbonded post-tensioned precast jointed wall systems with different aspect ratios and multiple walls should be performed to further validate both the PRESSS guidelines and the MBA method.
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