

EFFECTS OF UNBONDED MILD STEEL ON SEISMIC PERFORMANCE OF UNBONDED POST-TENSIONED PRECAST CONCRETE SHEAR WALLS

A Thesis

by

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Submitted to the

Graduate School of Sciences and Engineering
In Partial Fulfillment of the Requirements for
the Degree of

Master of Science

in the
Department of Civil Engineering

Özyeğin University
May 2022

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**EFFECTS OF UNBONDED MILD STEEL ON SEISMIC
PERFORMANCE OF UNBONDED POST-TENSIONED
PRECAST CONCRETE SHEAR WALLS**

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Dedicated To
My Wife Havva & Parents

“Nations that have made a habit of living comfortably without working, learning or getting tired are doomed to lose their dignity first, then their freedom and then their future.”

M. Kemal Atatürk

ABSTRACT

Seismic energy dissipation of precast concrete lateral force-resisting systems constructed based on non-emulative or jointed connections is very limited since these systems experience little to no damage during seismic events. One of the additional seismic energy dissipation mechanisms for unbonded post-tensioned precast concrete shear walls, which are constructed based on jointed connections, is using unbonded mild steel reinforcement over the wall joints, where joint gap opening occurs under lateral loads. Seismic energy dissipation is achieved by cyclic yielding of unbonded mild steel reinforcement both in tension and compression. Post-tensioned steel tendons are provided to resist lateral forces and minimize permanent drifts under seismic loads. Effects of unbonded mild steel bars on the system energy dissipation capacity and permanent drifts are not documented comprehensively for non-emulative precast concrete lateral force resisting systems. The main objective of this study is to investigate the effects of the mild steel design parameters on the seismic behavior of non-emulative unbonded post-tensioned precast concrete shear wall systems.

In this study, a developed numerical finite element (FE) model is verified for a jointed post-tensioned precast concrete shear wall system with experimental data. The experimentally verified modeling approach is used to develop FE models for several 5-story unbonded post-tensioned shear walls to investigate the effects of unbonded mild steel design parameters such as steel amount, location along the joint, and unbonded length on wall lateral behavior including its self-centering, energy dissipation capacity, and permanent prestress force losses for the unbonded tendons. The nonlinear structural behavior of the walls was determined by performing monotonic and cyclic pushover

analyses, and nonlinear time history analysis with the OpenSees software. The finding of this study provides new and valuable contributions to the existing literature to further improve the seismic design of non-emulative precast lateral force resisting systems and provide a better understanding of the influence of unbonded mild steel on the seismic performance of jointed precast concrete walls.



ÖZETÇE

Prefabrik duvar panellerinin birleşim yerlerinde ıslak beton kullanılmadan oluşturulan kuru bağlantılı aderanssız ard germeli prefabrik (AAP) duvar sistemleri yanal deprem kuvvetleri etkisinde sırasında çok az veya hiç yapısal hasar görmediğinden bu sistemlerin deprem enerji sönümleme kapasitesi çok sınırlıdır ve bundan dolayı görece kat ötelemeleri genelde büyüktür. Bu sistemlerin deprem enerjisini sönümleme kapasitesini artırmak ve kat ötelemelerini sınırlamak için ek sismik enerji sönümleme mekanizmalarından biri, deprem yükleri altında kat duvar panellerin birleşim derzlerinde oluşan açılmadan faydalanarak bu derzler üzerinde aderanssız yumuşak çelik donatı kullanılmasıdır. Sismik enerji sönümlenmesi hem çekme hem de basma kuvvetleri altında aderanssız yumuşak çelik donatının döngüsel akması ile sağlanır. Literatürdeki çalışmalarda, kuru bağlantılı prefabrik sistemlerin deprem yükleri gibi yanal kuvvetler altında aderanssız yumuşak çelik çubuklar kullanılarak sağlanabilecek enerji sönümleme kapasiteleri, bu çubukların detaylı tasarım ilkeleri ve duvarlardan oluşan taşıma sisteminin sahip olduğu kendinden merkezlenme (ihmal edilebilir kalıcı ötelenme) özelliği üzerindeki etkileri kapsamlı bir şekilde incelenmemiştir. Bu çalışmanın temel amacı, aderanssız yumuşak çelik donatının tasarım parametrelerinin belirlenmesi ve bu parametrelerin AAP duvar sistemlerinin sismik davranışı üzerindeki etkilerini araştırmaktır.

Bu çalışmada, kuru bağlantılı aderanssız ard germeli prefabrik duvar sistemleri için geliştirilmiş bir sayısal sonlu eleman (FE) modeli deneysel verilerle doğrulanmıştır. Bir dizi beş katlı aderanssız ard germeli perde duvar için sonlu eleman modelleri oluşturulmuştur. Bu duvarlar sistemleri için aderanssız yumuşak çelik donatı miktarı,

duvarların birleşim derzi boyunca bu donatının konumu ve donatının aderanssız bırakılan boyu gibi bu donatının tasarım parametrelerinin, duvar sisteminin kendi kendine merkezlenme, enerji sönümleme kapasitesi ve deprem sonrası aderanssız ard germe çubuklarında oluşan ard germe kuvvetinde oluşan germe kayıpları gibi duvar sisteminin deprem davranışı üzerindeki etkileri araştırılmıştır.

Sismik tasarımı yapılan duvarların deprem yükleri altındaki yapısal davranışları, OpenSees analiz programı kullanılarak yapılan doğrusal olmayan itme analizleri (pushover), döngüsel (tersinir) kuvvet-yer değiştirme (cyclic) ve doğrusal olmayan zaman tanım alanında dinamik analizlerle belirlenmiştir. Bu çalışmadan elde edilen veriler, kuru bağlantılı aderanssız ard germeli prefabrik duvar sistemlerinin deprem tasarımını geliştirmek, deprem enerjisini sönümlemek için derzler boyunca duvar panelleri arasında kullanılan aderanssız yumuşak çeliğin tasarım ilkelerini ortaya koymak ve bu enerji sönümleme donatısının bu prefabrik duvar sistemlerinin sismik performansı üzerindeki etkilerinin daha iyi anlaşılmasını sağlamak için mevcut literatüre yeni ve değerli katkılar sağlamaktadır.

ACKNOWLEDGMENTS

I would first and foremost like to express my endless gratitude to my thesis advisor Dr. Bülent Erkmén for his worthwhile supervision, patience, toleration, and for giving me an opportunity to defeat my ill fate. This chance might be small for the world, but words are not enough to tell what it means for me. I could not have done it without him.

I would also like to acknowledge thesis committee members Asst. Prof. Taner Yılmaz and Assoc. Prof. Dr. Ali Sarı for their valuable feedback and kind comments.

I am profoundly grateful to Özyeğin University for providing me with financial support and an awesome research environment. I also wish to thank Gizem Bakır for her kindness and helpfulness to me and my colleagues during my education.

I would like to extend my deepest gratitude to esteemed professors Prof. Dr. Atilla Ansal, Prof. Dr. Feyza Çinicioğlu, Prof. Dr. Gülay Altay, Dr. Derya Deniz, and Dr. Taner Yılmaz for their invaluable contribution to my engineering knowledge. Being acquainted with you, learning something from you, and even having a conversation with you are an honor for me.

Many thanks should go to my best friends Buğra Ceyhan, Levent Coşar, Onur Cem Aygün, Yasmin Moaiyedfar, Züleyha Kanpara Cıvaş, Efekan Kor, Eylül Mina Aydın, and Gülşah Sağbaş for their help, support, motivation. Working and spending time during the last three years was pretty pleasant for me.

Finally, I would like to express special thanks to my mum, dad, siblings, and wife for providing encouragement and infallible spiritual support during my studies. This accomplishment would not have been possible without you and your love.



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CHAPTER I

INTRODUCTION

1.1 Motivation

Reinforced concrete shear walls are lateral load-resisting systems used to improve the seismic performance of structures. They also function as vertical load-bearing elements whose thickness in the plan is quite large compared to their height. The shear walls transfer the horizontal and vertical loads from the floors and beams to the ground and limit the horizontal drift of the structures, especially under influence of lateral loads such as earthquakes and winds. In general, columns provide sufficient strength for the vertical loads, i.e., the weight of the structure and live loads originating from building occupants, appliances, etc. Shear walls, which have larger cross-sections compared to the columns, greatly increase the resistance of the structures against vertical loads while the main purpose of using shear walls is to increase the strength and rigidity of structures against lateral loads and minimize damage to other structural and non-structural elements. In other words, shear walls are structural elements sacrificed during seismic events to minimize damage to other parts of the structure under lateral loads.

Extensive experimental and analytical studies have been conducted to advance the seismic performance of precast concrete systems and develop their design philosophy [1], [2], [3], [4], [5], [6], [7]. Regarding the design recommendations, precast concrete shear walls could be designed with a wet connection in which concrete panels are erected by using cast-in-place concrete and emulating monolithic cast-in-place walls. However,

using wet concrete and steel for connections between precast concrete members requires experienced personnel and does not provide some of the precast concrete advantages such as construction speed, good quality control, and minimized cost. Therefore, the studies are focused on precast walls with dry connections, which are more convenient in terms of benefits associated such as cost-benefit, quality control, and workload.

Unbonded post-tensioned connections, which are dry connections, became the focus of research and industry, especially after the PRESSS Research Program [8]. The PRESSS research program greatly expanded the database for seismic performance of precast concrete structures [5]. However, debonding the flexural steels from the adjacent concrete was first introduced by Priestly and Tao [9]. In this system, because the unbonded steel tendons are only connected to the adjacent concrete at the tendons ends, they remain elastic, residual deformations are minimized due to post-tensioning force, and concrete crushing due to bond stress is eliminated due to lack of bond and strain compatibility between concrete and steel. The nonlinear behavior of the system is provided by gap opening between the precast panel joints, nonlinear compressional response of concrete, and through supplementary energy dissipation mechanism if provided. Kurama studied the flexural behavior of unbonded post-tensioned precast concrete shear walls analytically [3] and [4]. The results indicated unbonded post-tensioned (PT) steel can remain elastic for properly designed unbonded post-tensioned shear walls, which are subjected to large non-linear displacement. Furthermore, using unbonded post-tensioned tendons provide superior self-centering ability and flexural ductility to the wall without significant permanent drift and damage.

Conventional cast-in-place concrete and precast concrete shear walls designed based on emulative construction methodologies deform at their plastic hinge regions due

to horizontal concrete cracking and yielding of longitudinal flexural reinforcing bars. Under seismic loads, concrete cracks and these cracks propagate during cyclic opening and closing under seismic excitation thus providing a source to dissipate seismic energy. In addition, flexural reinforcement yielding and, in some cases, fracturing also provide another source to dissipate seismic energy. On the other hand, these two seismic energy dissipation mechanisms are not available for unbonded post-tensioned precast concrete shear walls. Because negligible concrete damage occurs only at wall toe or boundary regions due to the rocking motion of the wall, and the unbonded post-tensioned tendons remain in the elastic range. Therefore, the seismic energy dissipating capacity of unbonded post-tensioned precast concrete walls is considerably low [10], [11], [12], [13], and supplemental energy dissipation sources are needed for precast systems to minimize wall maximum lateral drift. Viscous fluid dampers, friction dampers, mechanical connectors, mild steel bars are one of the additional external energy dissipators and were studied experimentally and analytically by several researchers [10], [12], [14], [15], [16], [17], [18], [19], [20], [22], [23], [24], [25], [26], [27], [28].

Several energy dissipation mechanisms have been proposed to improve the seismic performance of unbonded post-tensioned precast concrete shear walls. These include supplemental viscous dampers [19], vertical energy dissipating connections [16], and steel O-connector hysteretic dampers [23]. Another common mechanism commonly used to dissipate seismic energy for the walls is utilizing mild partially unbonded steel bars between the wall base panel and foundation also known as hybrid precast concrete wall systems. A hybrid system has good energy dissipation characteristics without severe damage and possesses self-centering ability. Holden et al. [10] described the hysteresis response of monolithic, unbonded post-tensioned, and the hybrid system as shown in

Figure 1-1. Although there are design guidelines and a large number of studies, still there is no specific and closed form formulation for seismic design of unbonded post-tensioned precast walls with unbonded mild steel. There is a need for guidelines to develop simple equations to determine design parameters for unbonded mild steel needed to maximize energy dissipation. Some important design parameters include maximum steel area, steel location and distribution along the wall joint, and the required unbonded length of mild steel.

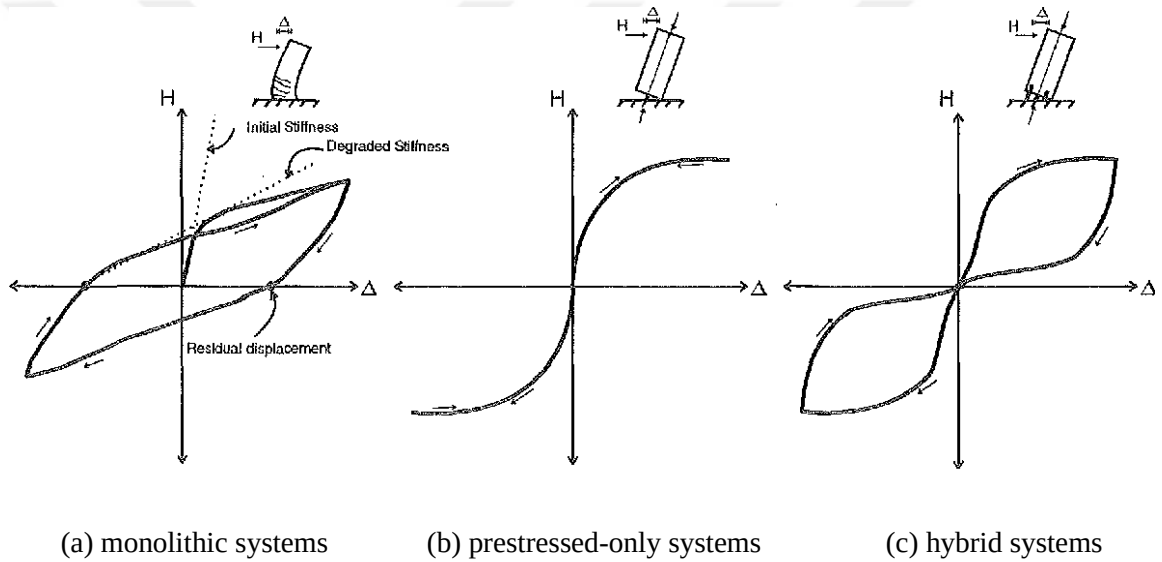


Figure 1-1: Hysteretic behavior of different wall systems [10]

In the current study, influence of design parameters of unbonded mild steel bars including steel area, distribution of bars along the joint, and unbonded length on wall lateral load cyclic behavior, energy dissipation capacity, recentering ability, wall joint gap opening, and loss of post-tensioning force is investigated. Also, simple design guidelines are developed for determination of unbonded mild steel design parameters.

1.2 Research Objective

The main objective of this study is to investigate the effect of mild steel reinforcement design parameters on lateral load resisting behavior and seismic performance of unbonded post-tensioned precast shear walls. Most of the related literature focus on the effects of post-tensioning force, addition supplementary energy dissipation mechanism, and walls self-centering behavior. However, effects of unbonded mild steel, specifically steel area, unbonded length, and its location on wall seismic behavior are not yet sufficiently covered. In addition, there is a need for engineering guidelines on unbonded mild steel design to optimize wall energy dissipation while ensuring wall characteristic behavior including gap opening and its superior self-centering characteristics. This study aims to provide valuable information to enhance seismic behavior of jointed precast lateral load resisting systems with the following main objectives:

1. Identify main design parameters for unbonded mild steel bars that affect seismic performance of the walls.
2. Develop design guidelines to determine unbonded mild steel design parameters for given seismic levels.
3. Determine effects of unbonded mild steel design parameters on wall seismic performance by performing nonlinear pushover and time history analysis of prototype walls.
4. Determine optimum design parameters such as steel area, steel location and distribution, and unbonded length for unbonded mild steel to maximize wall

seismic performance while preserving its gap opening over foundation level joints and its superior self-centering characteristic.

1.3 Organization of the Thesis

This thesis is comprised of six chapters. Here in this chapter, a short overview of precast structural systems and development of hybrid precast concrete wall systems and objectives of this thesis are presented.

Chapter 2 introduces a literature review about the seismic behavior of precast concrete structural systems, hybrid wall systems, and supplemental energy dissipation mechanisms.

Chapter 3 presents the design of a number of prototype walls in accordance with the recommendation of ASCE 7-16, ACI ITG-5.1, and ACI ITG-5.2 that will be used in analytical studies in Chapter 4.

Chapter 4 presents a modeling approach for prototype walls, development of an OpenSees analytical model for the prototype walls and validation of the modeling approach.

Chapter 5 introduces the conducted analysis and the analysis results including a comprehensive discussion on behavior of prototype walls under monotonic pushover loads, cyclic pushover loads, and nonlinear dynamic loads.

Chapter 6 presents a summary of the thesis and overall outcomes of the research.

CHAPTER II

LITERATURE REVIEW

In this chapter, the current literature on seismic performance of precast concrete structures including the studies on unbonded post-tensioned precast concrete structures and supplemental energy dissipating systems is reviewed. This section is structured into three separate sections: In the first section, the history of precast concrete structures is presented with a focus on PRESSS research program; In the second section, research on unbonded post-tensioned concrete structures is covered and studies on a different type of unbonded post-tensioned concrete structures e.g., structural walls, frames subassemblies and bridge piers are discussed; and in the third section, supplemental energy dissipating systems are examined and studies on different energy dissipating system namely, viscous fluid dampers, viscous friction dampers, connectors, and mild steel bars are presented.

2.1 PRESSS Research Program

On the purpose of developing effective and comprehensive design recommendations based on research data and introducing innovative concepts and technologies for precast concrete structures by extensive experimental and analytical studies, a research program called Precast Seismic Structural System (PRESSS) research program was initiated under United States – Japan coordination. Within the scope of the research program, seismic behavior of precast concrete structural systems e.g., precast frames, precast wall panels, etc. was investigated.

The program consisted of three phases. In phase I, structural concepts that are in demand for innovative design of precast concrete building systems in moderate to high seismicity regions were identified and their cyclic behavior was evaluated. Then, a concept of design recommendation for precast frames and shear walls was developed. In phase II, extensive experimental studies for precast concrete structures and subassemblies were performed and analytical models were developed and tested for verification of the expected seismic response of structures. In the last phase, phase III, several large-scale multi-story precast concrete buildings were tested both experimentally and analytically. Eventually, seismic design recommendations for precast concrete structural systems were developed based on program findings. The research program mainly comprised of four categories namely, concept development, experimental program, analytical studies, and design recommendations. The PRESSS research program was supported by the National Science Foundation (NSF) and industry. Besides, several researchers were also involved from different institutions e.g., the University of California, National Institute for Standards and Technology, Lehigh University, and the University of Auckland, etc.

Schultz and Magana [29] studied the behavior of the vertical and horizontal connection between concrete panels in precast shear walls within the scope of the second phase of the PRESSS research program in the NIST Building and Fire Research Laboratory (BFRL) with the aim of enhancing the jointed construction philosophy in precast concrete shear walls. A prototype wall of a six-story office building, which comprised of four panels of which two panels were placed side by side and two more panels stacked on them via vertical and horizontal joints, was studied. In the experimental program, seven vertical joint specimens and four horizontal joint specimens were tested. The selected vertical joint specimens comprise of welded plate connection details, bolting

or combination of bolting and welded connection namely NSP, SFP, IFB, PTS, VJF, UFP, and XAP. The concrete panels were constructed using typical construction materials including concrete, reinforcements, and anchors and tested in the NIST Tri-directional Test Facility using reversed-cyclic drift histories. Results showed that the response of the vertical connection specimens for the imposed cyclic loading was positive regarding stiffness, ductility, and energy dissipation capability. Furthermore, NSP and IFB connections, which are welded loose plates, performed well under applied loads, and could be used in the region of moderate seismic risk while VJF and UFP connections exhibit outstanding seismic force-resisting and could be used in the region of high seismic risk. Researchers also concluded that yielding plate design in various configurations is possible for precast concrete shear wall panels using current design methods; however, more conservative design approaches for anchorage are needed.

Details of design, testing, and results for the experimental program on the horizontal connection details are given in Schultz et al. [30] as a continuation of former research on vertical joint connection details. In this study, four horizontal joint connections namely Grouted Splice Sleeve (GSS), Post-Tensioned Tendon (PTT), Precast Vertical Joint (PVJ), and Debonded Smooth Bar (DSB) connections were studied. The testing setup used for testing the performance of different horizontal connections was the same as that used for vertical connections as well as the geometric and material properties used for construction of the panels. Also, the same test protocol was used for the panels with horizontal connections. Based on the test results presented, the GSS specimen responded with moderately wide hysteresis loops, and flexural capacity of the wall decayed gradually after the peak load was achieved. The response of the PTT specimen exhibited moderate energy dissipation, and significant stiffness reduction was

not observed until the post-tensioned tendons were loaded beyond their proportional limit. Although tendon post-tensioning stresses were lost, PTT wall peak strength was maintained through loading. The response of PVJ specimen was similar to that of GSS specimen but despite reaching the predicted vertical shear capacity, no cracks occurred in the precast vertical joints. Due to fracture of the debonded bars in DSB connection, the testing was not completed as planned however the estimated behavior with long debonded length of mild steel bar was to exhibit well cyclic behavior with higher energy dissipation capacity. The study concluded that the four connection details for horizontal joints could satisfy structural performance requirements for precast concrete walls.

Priestly et al. [31] conducted an experimental study on a 5-story precast concrete building within the scope of the third phase of the PRESSS research program at the University of California, San Diego (UCSD). The building, which was detailed by Nakaki et al. [5] was constructed at 60% scale and comprises four different precast frames in one direction and precast hybrid walls in the orthogonal direction. Figure 2-1 shows the elevation view of the jointed wall structure. Four RC panels each with a height equal to half of the building height were used to build a shear wall system in which two panels were connected via unbonded post-tensioned tendons in the vertical direction and another two panels were placed on them by using energy dissipating U-shaped flexural plates in the horizontal direction. The structure was subjected to an earthquake load that was 50% higher than that required for Uniform Building Code (UBC) Zone 4 [32]. At the end of the test, minor concrete crushing at the wall base and minor cracking at the base of the columns were observed. The observed and measured performance of the walls and frames was excellent due to characteristics of unbonded post-tensioned systems despite the fact that the building was subjected to a higher earthquake load than the design earthquake.

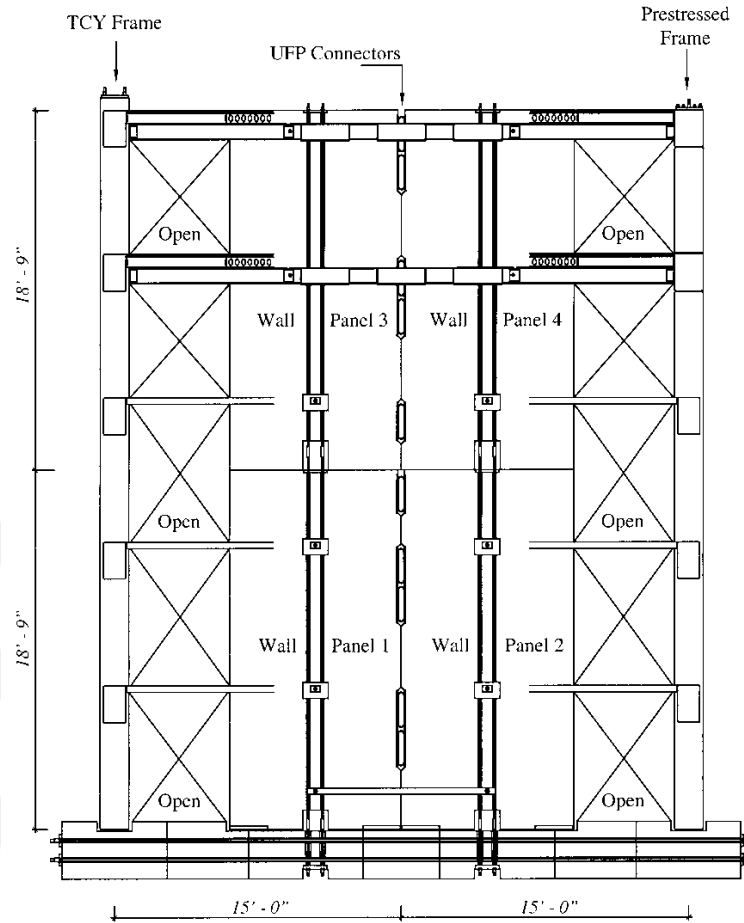


Figure 2-1: Elevation of the jointed wall [31]

2.2 Unbonded Post-tensioned Precast Concrete Shear Walls and other Applications

Unbonded post-tensioned precast concrete shear walls with horizontal connections were studied analytically by Kurama et al. [3] [4] with the intention of evaluating seismic behavior for different seismic zones and different soil types and developing a design approach for precast concrete walls as the primary lateral load resisting system. An analytical model of a prototype wall which incorporates fiber beam-column element for concrete panel and truss element for unbonded post tensioned strands based on the initial phase of the PRESSS research program was created after verification

of the fiber model by using experimental results for the 1/3-scale three-story precast wall by Mueller [33] and Armouti [34]. Influence of different analytical model parameters (e.g., number of fiber element and post-tensioning bar modeling) and eleven structural design parameters (e.g., amount of spiral reinforcement, the strength of plan concrete, initial stress in concrete, and post-tensioning bars) on the lateral load response of the prototype walls were investigated. Four main behavior states are defined on a monotonic pushover curve for unbonded post-tensioned walls under the combination of lateral and axial load. A description of these states is given below, and the states are shown in Figure 2-2.

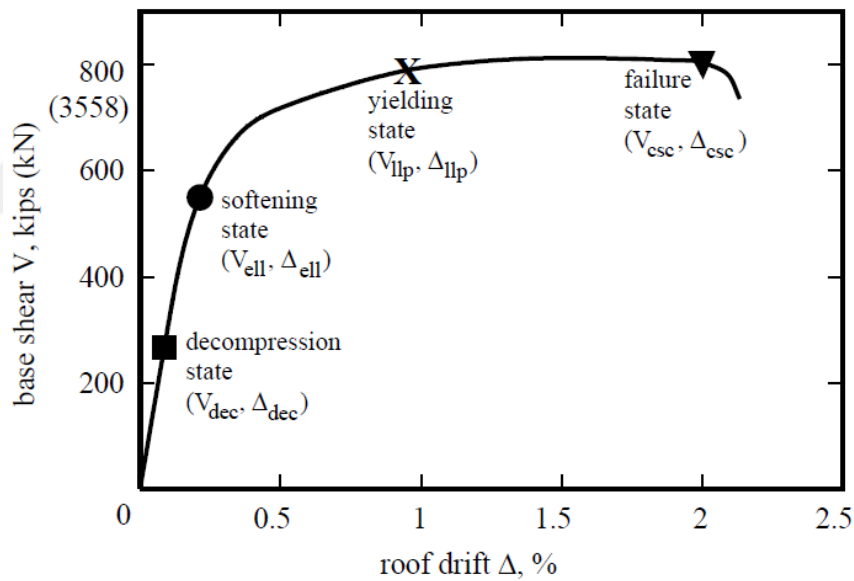


Figure 2-2: Four states of unbonded post-tensioned wall behavior [4]

Decompression state – is the instant that stress on the concrete due to combination of the axial and prestressing loads is just overcome at the edge of the panel. In other words, this state is initiation of gap opening primarily between the base panel and foundation.

Softening state – is the initiation of notable lateral stiffness degradation of the wall due to nonlinear concrete behavior in compression and gap opening at the horizontal joints.

Yielding state – is the elastic limit of post-tensioning steel. Up to this point, the nonlinear behavior of the wall is due to mainly the gap opening and nonlinear response of concrete under compression.

Failure state – is the failure of the wall because of the crushing of confined concrete at the edge of the concrete wall panel.

In the study, a performance-based design approach for buildings with unbonded post-tensioned precast concrete walls was proposed based on required performance under design level ground and survivability level ground motions. The design is based on following performance requirements:

- Response of wall is nearly nonlinear elastic and nonlinear behavior is due to a gap opening predominantly, post-tensioned bars stay in the elastic limit, severe cracking at edge of the base concrete panel is prevented, and shear slip between concrete panel is prevented under design level ground motion and
- Axial-flexural compression failure in wall panel is prevented, the nonlinear strain on post-tensioning bar is small after yielding owing to debonding and it loses prestress due to this nonlinear strain, while lateral stiffness decrease, base shear capacity does not, permanent drift after the seismic load is small due to prestressing and shear slip between concrete panel is prevented under survivability level ground motion.

Using this design approach and as per NEHRP [35] provisions, six prototype wall models representing a part of a six-story office building were designed for moderate and high seismic sites and three different soil types (i.e., loose, medium, stiff). Detailed geometric and material properties of the walls are given by Kurama [3] and shown in Figure 2-3.

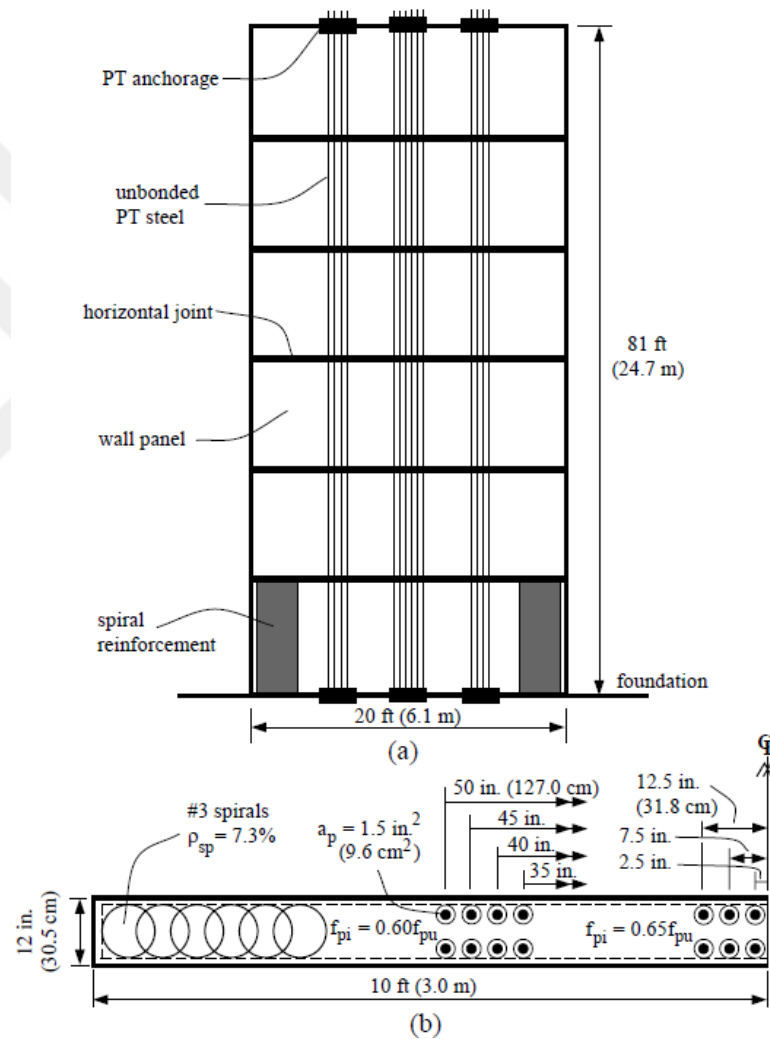


Figure 2-3: Elevation and cross-section near the base [3]

Nonlinear time history analyses of the walls were conducted using thirty ground motion records scaled to design and survival levels. The seismic response of the wall under selected ground motions, feasibility of design approach, effects of design

parameters on the dynamic response of the wall was investigated. In the below, preliminary results from the design approach, analytical model, static monotonic and cyclic load test, and dynamic load test are presented.

- Debonding the post-tensioned bars, large nonlinear displacements, and large gap opening can be achieved without yielding of bars and slip friction between the concrete panel.
- Post-tensioning force and axial force ensure self-centering behavior and restoring force which closes the gap opening and provides less permanent drift.
- Response for lateral load is nearly nonlinear elastic; however, energy dissipation capability is low.
- Due to rocking, large compressive strain occurs on the edges of the concrete panel near the base therefore, spiral reinforcement is needed at this region to confine concrete and achieve large nonlinear drift without crushing of concrete.
- Analytical models with fiber elements were successively created and used to conduct static and dynamic analysis.
- Unbonded post-tensioned precast walls can be designed under design ground motion (DGM) with negligible damage, and under maximum considered earthquake (MCE) with damage but no failure. However, the estimation of maximum roof drift for design should be improved.

- Maximum roof drift of unbonded post-tensioned concrete walls is greater than that of cast-in-place monolithic concrete walls.
- Effect of higher modes on the base shear demand is larger for unbonded post-tensioned concrete walls than the monolithic cast-in-place walls.
- Maximum base shear demand is larger for unbonded post-tensioned concrete walls than the monolithic cast-in-place walls.
- Reduction in post-tensioning force due to yielding causes a reduction in shear slip capacity and affects hysteretic behavior.
- Confinement with spiral reinforcement is needed only for the base panel.
- Ground motion intensity does not affect the maximum length of gap opening between foundation and base panel under both considered level ground motion strongly.
- Amount of axial load due to dead and live load during an earthquake has no significant effect on seismic behavior.

Perez et al. [36] conducted an experimental study at Lehigh University to verify the results of Kurama's analytical study on seismic behavior and design of unbonded post-tensioned concrete walls. In the study, experimentally and analytically observed wall behaviors were evaluated by testing 5/12-scale 6-story walls under static monotonic and cyclic loads. Effects of steel area, initial post-tensioning stress, initial concrete stress, post-tensioning steel eccentricity, and confinement in the concrete panel on flexural behavior of walls were studied. Base shear versus drift curves of the wall obtained from the experiment and analytical study were in good agreement. Under cyclic loading, the

wall exhibited good self-centering ability. Observation on the wall behavior indicated that the wall presents nearly nonlinear elastic behavior with small energy dissipation capacity and such nonlinear behavior resulted from gap opening between concrete panels as proposed by Kurama [4]. The behavior of the wall determined experimentally was very similar to that defined by Kurama, and the four states defined in Figure 2-2 were also experimentally observed. However, the failure mode was slightly different than that analytically predicted. Under cyclic loading, diagonal cracks were observed in the vicinity of the edge of the base panel along its height, and these cracks caused the confined region to be exposed and spiral reinforcement to fracture. It was concluded that reinforcing detail needs to be redesigned to control cracks and avoid such failure modes. One of the fundamental results from Kurama et al. [4] was that the energy dissipation response of unbonded post-tensioned concrete walls is small while the wall undergoes large drift values under seismic loads. This is a significant disadvantage of such wall systems since low energy dissipation indicates large wall drifts under lateral loads.

In order to increase wall energy dissipation capacity, Kurama [19] proposed using supplementary viscous dampers, which supplies energy dissipation and reduces wall drift under lateral loads. Linear viscous fluid dampers are attached diagonally to each side of the wall at every floor level as shown in Figure 2-4. To determine effects of dampers, the walls studied in previous research by Kurama [4] were used with the dampers. Viscous dampers were modeled with truss elements using stiffness proportional damping. Axial deformation and flexibility of the bracing elements modeling dampers were ignored, and braced columns were enslaved in the lateral direction with a rigid floor system. Seven ground motions were selected and scaled to a constant maximum incremental velocity on ground motion demand spectra. Afterward, nonlinear time history analysis was conducted

to evaluate effectiveness of the proposed energy dissipation system and design approach employed. Analysis results showed that maximum roof drift is successfully reduced to the target roof drift without significant damage in the walls using the supplemental viscous dampers.

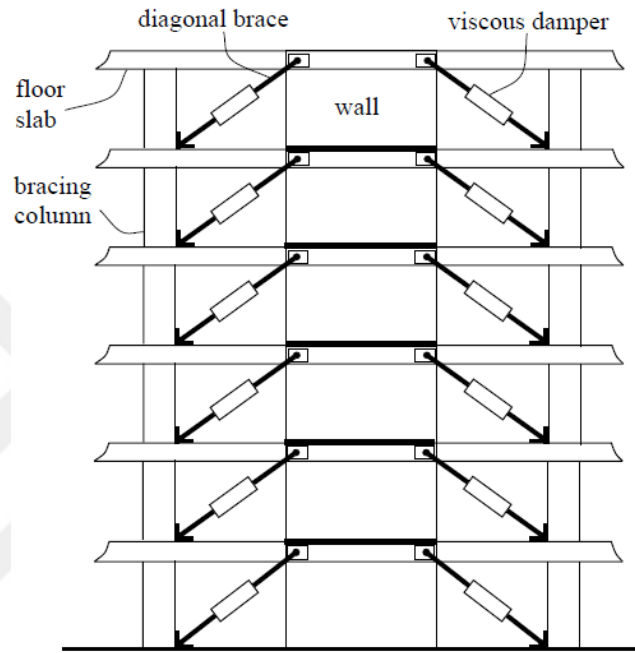


Figure 2-4: Proposed supplemental energy dissipation by Kurama [19]

Because using viscous dampers is not economical, Kurama [11] proposed a new and simple energy dissipating system as an alternative which is also known as a hybrid precast wall system. In a hybrid system, bonded mild steel bars, which are unbonded over a specific length, are added to the wall along the gap opening joint. Initially, twelve prototype walls (four walls with a different number of stories and different sites, four emulative walls, and four hybrid walls) with different amounts of mild steel were tested analytically to investigate the effect of analytical parameters and effects of mild steel on wall seismic performance. Time history analyses were conducted to examine the effect

of mild steel bars on the predicted behavior of hybrid walls under earthquake load.

According to the study, the following conclusions were made:

- The emulative walls exhibited more energy dissipation through cyclic loading, but the walls had more permanent drift under lateral loads.
- Adding unbonded mild steel bars to unbonded post-tensioned walls decreased maximum lateral displacement and energy dissipation significantly. However, the unbonded mild steel was less effective than the viscous dampers.
- Because maximum wall drift and number of large drifts decrease, unbonded mild steel bars can be used in high seismicity regions, however, there was not much difference in the response between the dry connection precast wall system including and excluding mild steel bars for moderate seismicity regions.
- Self-centering behavior of unbonded post-tensioned walls was not affected with the use of unbonded mild unbonded steel.
- The amount of unbonded mild steel needed depends on maximum lateral drift allowed for the walls.
- Maximum roof acceleration of the wall decreased with the increase of mild steel added to the walls.
- The unbonded mild steel is more effective to reduce wall lateral drift for short-period structures compared to medium and long-period structures.

Holden et al. [10] [37] conducted experimental research on two types of precast concrete wall namely emulative wall and hybrid wall in order to investigate and compare their seismic performance. Two half-scale precast concrete cantilever walls with the same geometry were constructed using similar construction materials. The first wall was designed as conventional monolithic ductile walls per Standards New Zealand (SNZ) while the second wall was designed as a hybrid wall that incorporated energy dissipation mild bars, steel fiber reinforcements, and carbon fiber prestressed tendons. The experimental setup and wall elevation are presented in Figure 2-5. Both wall specimens were tested under reversed cyclic loading. Experimental results showed that the emulative wall had a ductile behavior with developing plastic hinges at the base of the wall and thus having a good energy dissipation capacity under lateral loads. The emulative wall, which was also designed with damage avoidance design philosophy, exhibited good self-centering behavior during the whole test, no significant residual drift, and no visible cracks even though some design deficiencies were detected, and failure of the panel was prevented successfully by using steel base plates. The specific findings from comparison of two walls seismic performance, the following conclusions were made:

- Emulative wall failure occurred at approximately 2.5% drift while the hybrid wall failure occurred at 3% drift level without visible damage.
- Equivalent viscous damping or energy dissipation capacity of the emulative wall was bigger than the hybrid wall.
- Dissipated energy was due to damage of concrete in the emulative wall while energy dissipation was provided by the mild steel bars for the hybrid wall.

- The emulative wall had residual drifts while the hybrid wall response showed excellent self-centering or no significant permanent drift.

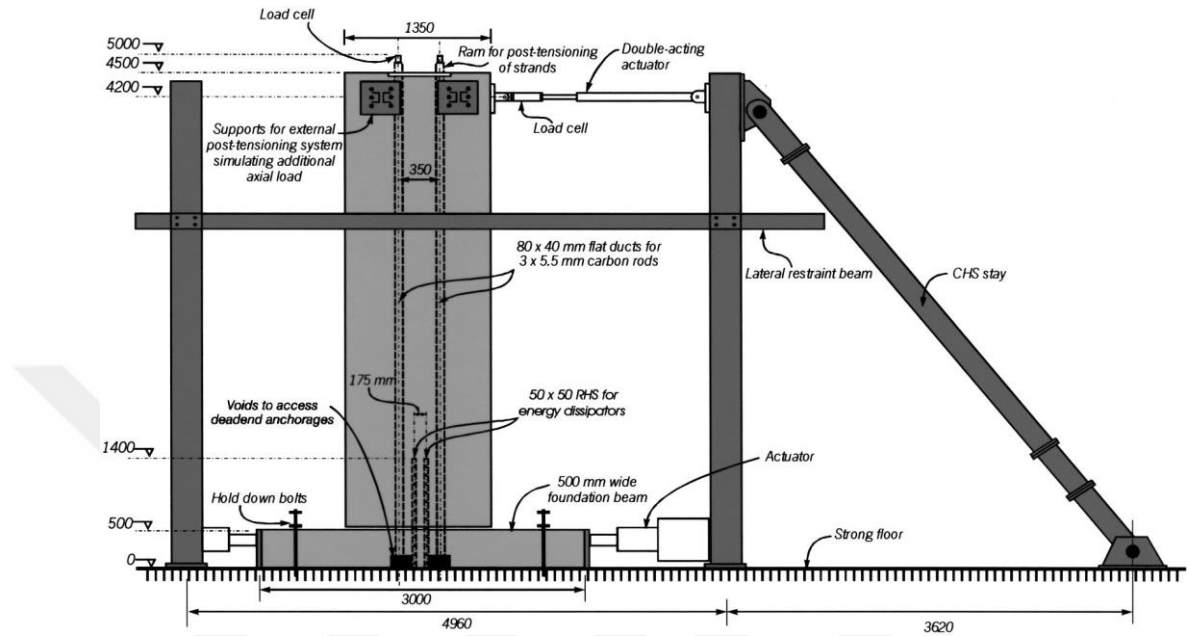


Figure 2-5: Experiment setup of Holden [37]

Restrepo and Rahman [12] conducted an experimental program on self-centering walls with energy dissipators. Three half-scale 4-story walls with the same geometry were designed. The difference between the use of energy dissipators and axial load applied to the walls. Dog-bone mild steel bars were used as energy dissipators for Unit 2 and 3 in oversized corrugated ducts while Unit 1 had only consisted of prestressed tendons. These walls were tested under quasi-static reversed cyclic loading. Response of Unit 1 wall was nonlinear elastic rocking motion without any residual drift due to gap openings. Unit 2 wall exhibited fat hysteresis loop on base shear-drift curve with excellent self-centering and energy dissipation capability. The response of Unit 3 wall was similar to that of Unit 2. The maximum drifts of the walls were 2.8%, 3%, and 4.1%, for Unit 1, 2, and 3, respectively. Based on the test results, Unit 1 presented no significant energy dissipation,

while Unit 2 and 3 had efficient energy dissipation capacity, which was equivalent to a 14% equivalent viscous damping ratio.

Perez et al. [38] proposed an analytical model as in Figure 2-6 based on beam fiber elements to predict nonlinear behavior of unbonded post-tensioned precast concrete wall and compared the proposed model with experimental results from Perez et al. [36].

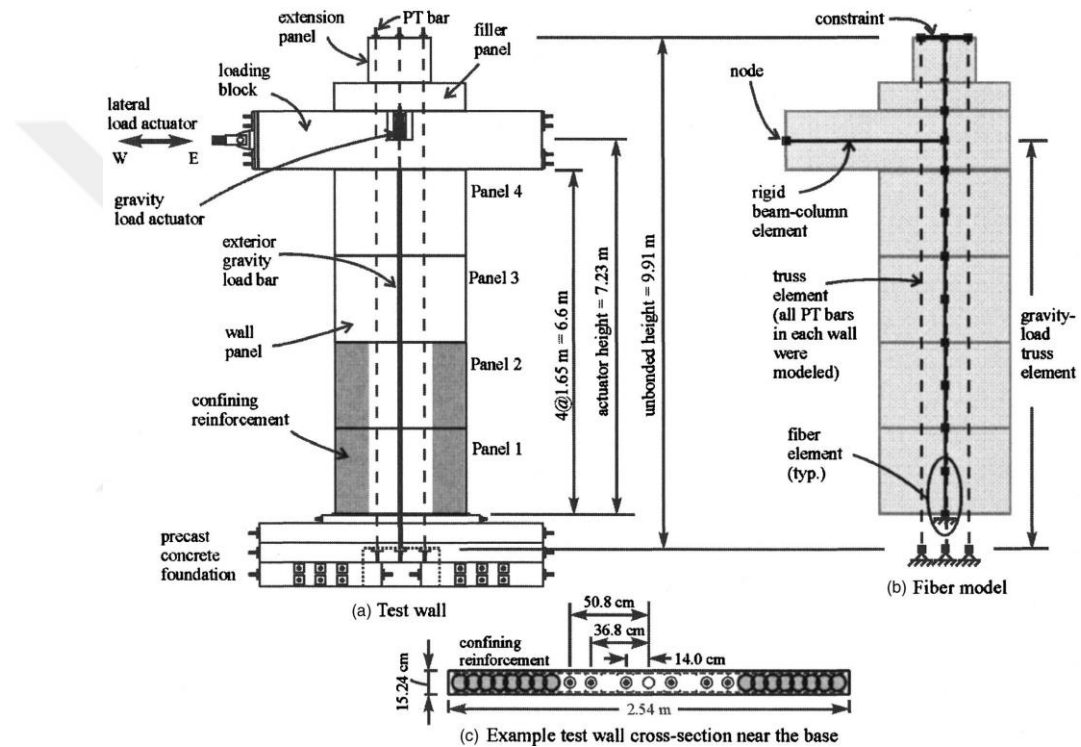


Figure 2-6: Experiment setup and fiber model [38]

In addition, a simple model composed of several closed-form expressions to predict unbonded post-tensioned wall lateral behavior as a tri-linear idealization was proposed by Perez et al. [38]. The results from the simple model were compared with those from the fiber model to evaluate its accuracy. The purpose of the simple model is to develop closed form expression to be used for the seismic design of unbonded post-tensioned concrete walls. For this purpose, five half-scale wall models which represent the first four-story of the six-story unbonded post-tensioned precast concrete wall were

modeled with the simple model and fiber model approaches, and the computed responses were compared with the experimental results for the walls. A parametric study of 16 walls showed that the lateral load behavior, which is characterized by base shear and roof drift, of the walls predicted both with fiber and simple models are in good agreement, and the simple model can predict the walls' limit states accurately. The parametric study performed for effects of area of post-tensioned steel, initial post-tensioning steel stress level, and gravity loads indicated that both modeling approaches show similar trends for each parameter variation. In conclusion, it was shown that the simple model can be used for seismic design of unbonded post-tensioned walls.

Previous studies have suggested that using unbonded post-tensioned tendons is the main source of these wall superior self-centering behavior. However, loss of post-tensioning force beyond the elastic limit of the bars was not investigated. Erkmen and Schultz [84] conducted an analytical study to investigate the self-centering mechanism and effects of several design parameters including post-tensioning force on self-centering behavior. An analytical fiber model developed for an unbonded post-tensioned wall called PTT wall, which was described by Schultz et al. [30], was created and verified with the experimental cyclic loading results. The developed model was used to investigate the effect of design parameters (e.g., tendon end-anchorage detail, initial stress of tendons, the layout of tendons, and magnitude of axial load) on the self-centering behavior of the PTT wall. Test results showed that self-centering ability ensured with not only post-tensioned tendons but also axial load due external load and weight, hysteretic behavior of the wall affected when end-anchorage detail of the tendons allowed take compression. However, the initial post-tensioning force and layout of the tendons had a negligible effect on self-centering behavior. To investigate effects of these parameters on self-centering

behavior of the walls under seismic load, five prototype walls representing a 5-story precast concrete parking building were developed and studied under ground motion records. Figure 2-7 shows the elevational and cross-section of one of the prototype walls.

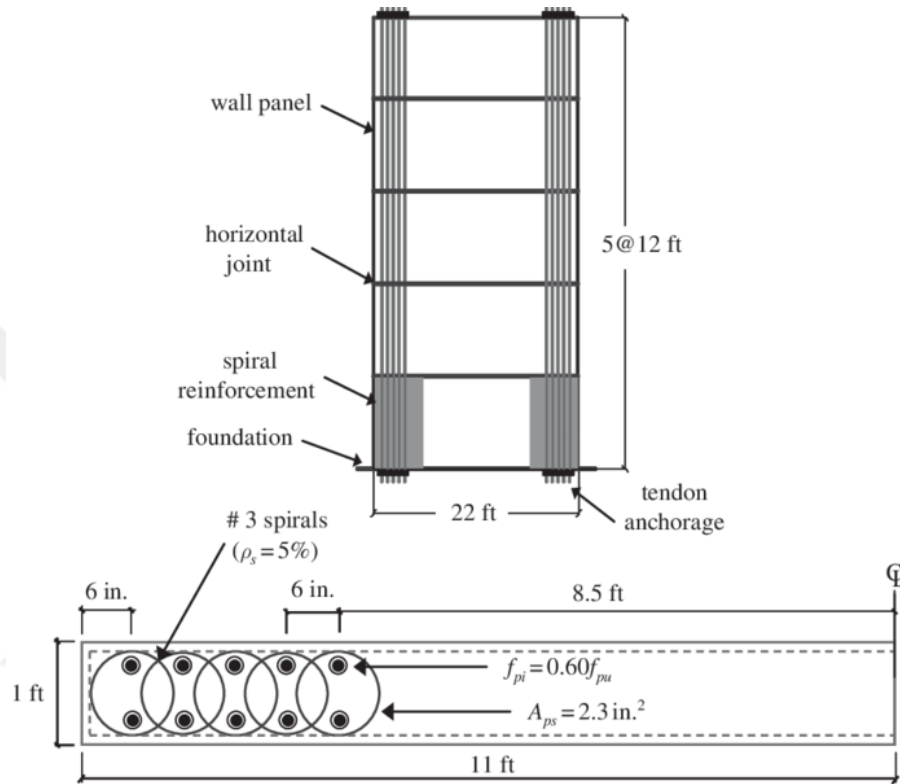


Figure 2-7: Elevation and cross-section of one of the prototype walls [84]

Based on the analysis results, the following conclusions were made:

- Placing the unbonded tendons at the edges of the wall instead of wall vertical centerline decreased maximum roof drift due to early yield and more energy dissipation due to tendons however permanent drift was not affected with the distribution of tendons.
- The layout of the unbonded tendons affected the shape of hysteresis loops, lateral stiffness, and load resisting capacity of the wall.
- Effects of post-tensioning level on wall residual drift was negligible.

- When the end-anchorage of post-tensioned tendons was detailed as compression-allowed to allow the tendons to undergo compression, post-tensioning stress was completely lost, and significantly large permanent wall drifts were observed.

Smith et al. [39] tested a 0.4-scale 4-story hybrid precast concrete wall under reversed cyclic loading and compared the results with the analytical model result. The wall behavior was as expected from the design, and the experimental response was successfully captured using the analytical model. However, it was concluded that a more accurate fiber-element model is needed for better predicting the experimental behavior. One disadvantage of the fiber beam model is the inherent assumption that plane sections remain plane and limitations to model discrete gap opening of wall joints.

Smith et al. [13] conducted another experimental study to compare seismic performance of emulative and hybrid walls. For this purpose, two 0.4-scale hybrid precast and one emulative precast concrete shear walls with identical geometry and similar construction materials were tested. For the emulative wall post-tensioned tendons were not used. For the hybrid walls the same post-tensioned tendons (area and post-tensioning force) were used but different mild steel bars (spliced and continuous) were used. For the walls, two different analytical models were developed namely the fiber element model using Drain-2DX and the finite element model using ABAQUS package. Based on the findings from the tests and simulations, the finite element model can be used to predict the overall response of the hybrid walls including gap opening, base-shear vs drift response, and post-tensioned tendon stress variation, and wall energy dissipation. Absence of post-tensioning force led to insufficient restoring force and high strength degradation. Continuous mild steel bars were recommended as energy dissipating bars

instead of spliced bars due to pull out of the bars from splice. During the rigid body motion, slight cracking occurred at the base joint while more and longer cracks through the panels occurred on the base panel to upper panels of the emulative walls.

Gu et al. [40] conducted an experimental and numerical study on self-centering hybrid walls. Two half-scale test specimens were tested under cyclic loading to evaluate their self-centering behavior of the wall. A multi-truss element model was developed using OpenSees [41] and verified to perform a parametric study. Effects of parameters including aspect ratio, initial post-tensioning stress, the total area of post-tensioning and mild steel bars, and yield strength of mild steel bars on wall self-centering behavior were evaluated using the analytical model. Results showed that monolithic walls exhibited a larger damping ratio and energy dissipation due to plastic hinge formation, while the hybrid walls had good self-centering ability and less permanent drifts. Parametric analysis results also showed that energy dissipation capacity of hybrid walls can be increased with increasing mild steel area, and wall aspect ratio and initial vertical stress can significantly affect the wall lateral response.

2.3 Moment Resisting Frames and Bridge Piers with Unbonded Steel

In response to the recognized need for innovative precast systems for seismic regions, application of using unbonded prestressing tendons was also studied for precast moment resisting systems and bridge piers. Figure 2-8 shows a typical application of unbonded post-tensioned steel for such a beam-column connection and bridge pier. For unbonded beam-column connection, the post-tensioned steel tendons are left unbonded over the column and a beam length on both sides of the column. To prevent immature concrete failure at the top and bottom regions of the beams, spiral reinforcement is used

to confine the concrete and increase its ductility. The same concept and design philosophy is used for the bridge piers to improve their seismic behavior and self-centering under lateral loads.

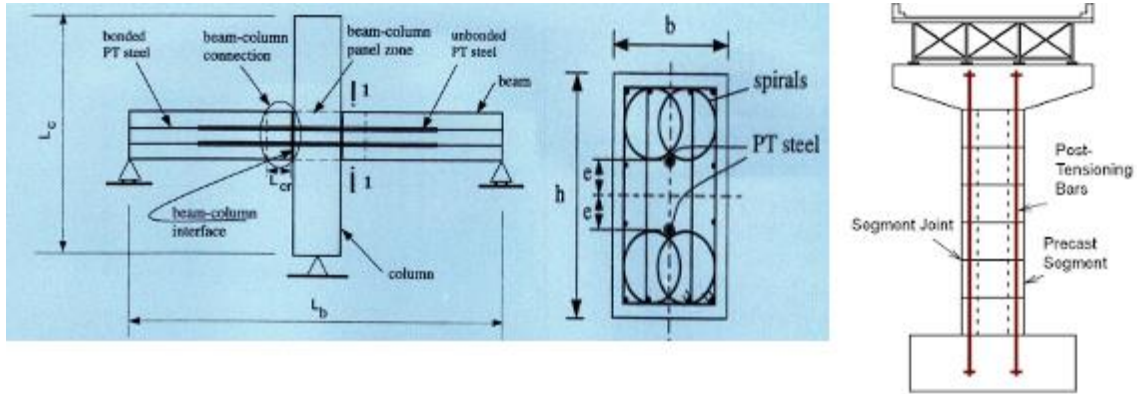


Figure 2-8: Unbonded post-tensioned beam-column and bridge pier

As a part of the second phase of the PRESSS research program, Cheok et al. [42] experimentally investigated lateral load performance of beam-column connection of precast concrete frames with a grouted post-tensioned tendon. The objective of the study was to develop an economic moment resistant precast concrete connection. The testing program included two precast post-tensioned concrete beam-to-column connections. The results show that the prestressed connection is strong and as ductile as the monolithic connections, but the energy dissipation capacity is limited.

Priestly and Tao [9] proposed using partially debonded post-tensioned tendons for precast prestressed concrete frames. For this connection, a part of the post-tensioned bar in precast beams was wrapped and bonded through the joint for some distance on either side of the column. The results of dynamic inelastic analyses showed that ductility demands for connection with partially debonded tendons will not be greater than that with

fully bonded tendons. In addition, the proposed design is expected to reduce residual displacements and joint shear stress.

The proposed innovative concept for precast concrete frames was tested by Cheok et al. [1] in the National Institute of Standards and Technology (NIST) laboratories. Four test specimens with different post-tensioning bars (e.g., bonded or unbonded) were tested under cyclic load until the failure point. Preliminary results from the experiment showed that earthquake performance of non-emulative post-tensioned frames is comparable to that of cast-in-place frames, and frames with similar flexural strength of conventional frame can be achieved with hybrid design. The hybrid frames presented good self-centering capability with no permanent deformation and large drift capacity. There were no visible cracks on the hybrid frame while the conventional frame suffered considerable damage. It was also concluded that the hybrid system provided higher shear resistance than the conventionally designed frames with no shear strength degradation.

El-Sheik et al. [2] studied unbonded post-tensioned precast concrete frames analytically and proposed a design approach. Prototype frames representing the lateral load resisting system of a six-story office building were selected. Two analytical models namely a fiber model and spring model were developed to study seismic behavior of the frame using nonlinear pushover static analysis and time-history dynamic analyses. The beam-column connections were modeled using fiber beam-column elements and zero-length rotational spring elements for the fiber and spring model, respectively. Both models were verified using results from an experimental study in NIST and provided good agreement of hysteretic behavior for the connection. The moment-rotation behavior of the connection was described using trilinear idealization namely linear limit state, yield limit state, and ultimate limit state. In addition, they proposed some closed form

expressions for these limit states as design guidelines. It was concluded that the strength, ductility, and self-centering capability of the unbonded post-tensioned precast frames is more than adequate for severe earthquake loading.

Priestly and MacRae [7] tested 2/3-scale two unbonded post-tensioned precast beam-column joints representing interior and exterior joints of a frame under reversed cyclic loading to investigate design assumption of hysteretic behavior. The test units were prepared as mid-span to mid-span of both columns and beams and subjected to lateral displacement. Test results were evaluated based on hysteresis response, plastic hinge region behavior, force transfer mechanism, performance of beam end region concrete confinement. According to the results, exterior and interior joints reached approximately 40% and 100% larger inter-story drifts than the design level drifts, respectively. No joint failure was observed, and strength degradation was negligible. The overall response of the joints was convincing even low-level reinforcement was used for frames. When the frame performance was compared with that of conventional design, it was observed that energy dissipation was less, but large residual drift and severe damage were prevented with the proposed design concept. It was concluded that more conservative confinement reinforcement in the plastic hinge region for joint design is necessary. Thus, further research is needed to investigate the shear design of beams.

Alongside the research summarized above, several experimental and analytical studies on seismic performance of unbonded precast frames and coupling beams for hybrid walls were conducted with the initiation of the PRESSS research program such as Pampanin et al.[43], Morgan and Kurama [44], Ajrab et al. [45], Speith et al. [46], Kurama and Shen [47], Lu et al. [48], Huang et al. [49], Qian et al. [50], Guo et al. [51], Hwang et al. [52], Guerrero et al. [53], and Kim et al. [54], etc.

The viability of using unbonded post-tensioned prestressing steel for structural systems subject to lateral loads was also studied for bridge piers. Mander and Cheng [55] proposed a design philosophy called Damage Avoidance Design (DAD) to prevent damage at the column beam connection and bridge pier toes due to rocking action. Also, an energy-based method for estimating equivalent viscous damping was introduced and validated with the result of an experimental study on precast concrete rocking bridge piers.

Zatar [56] tested several bridge pier specimens under reversal cyclic and pseudo-dynamic loading as part of the Precast Concrete Pier Research Program. A restoring force model was proposed, and this model was verified with experimental results. It was concluded that using precast concrete piers with unbonded post-tensioned steel provided excellent restoring behavior, small energy dissipation capacity, and less residual drift, and fewer cracks compared to conventional RC piers. Also, the proposed model could replicate the response successfully with better prediction of residual drift after earthquakes and consideration of unbonded post-tensioned tendons application with various parameters.

The hybrid design concept, where mild steel is also used to dissipate some energy, was applied to bridge piers by Palermo et al. [57] and fundamental design parameters which affect the cyclic response of bridge pier connection were investigated with the analytical study. It was emphasized that a parameter describing self-centering to energy dissipation in terms of moment contribution had a role for hybrid connection. No significant difference was observed based on maximum displacement according to time history analysis results. However, conventionally designed monolithic connections

resulted in residual drift at the order of 20% of maximum displacement while residual drift for the hybrid designed connection was negligible.

Hewes and Priestly [58] described an analytical model, and this model was used to verify with experimental study of four large-scaled unbonded precast columns which aims to investigate lateral load-deformation response. In the experimental program, the influence of aspect ratio, confinement level, and tendon initial stress was evaluated alongside the effectiveness of the analytical model for capturing load-deformation behavior. At the end of the test, large displacements were achieved without significant damage therefore such a system was regarded as implementable in bridge design for high seismicity regions. It was indicated that the initial stress level of the tendon had a crucial impact on behavior and should be carefully determined.

In addition to summarized research projects above, a number of research projects have initiated to develop a better understanding and better design guidelines for unbonded post-tension bridge piers such as Bilington and Yoon [59], Shim et al. [60], Kim et al. [61], Sideris et al. [62], etc. Moreover, an unbonded post-tensioned bridge pier with additional energy dissipators was studied by Cohagen et al. [63], Ou et al. [64], Marriot et al. [65] and Guerrini et al. [53], etc.

Erkmen [67] investigated effects of distribution of unbonded post-tensioned tendons on the seismic response of unbonded post-tensioned precast concrete walls. Three prototype shear walls were designed as the only lateral load resisting system for a 5-story precast concrete parking garage. The prototype walls were 16 m in height with 7 m by 0.3 m cross-sectional dimensions. Typical construction materials were used for each wall. Both end regions of the concrete base panel were confined with spiral reinforcement to

avoid premature concrete failure due to wall rocking. Each wall had 20 unbonded tendons, making a total cross-sectional steel area of 14.5 cm². Different unbonded steel distribution schemes along the horizontal joints were considered to distinguish effects of unbonded steel layout on wall lateral performance. For unbonded steel tendons three cases were investigated. The steel tendons were placed in the vicinity of wall vertical centerline, wall two ends, and distributed along the wall joint with an equal center to center distance. Unbonded tendons were post-tensioned to 75% of their yield stress. Mild steel bars as an energy-dissipating system were not modeled, instead 5% mass and stiffness proportional critical damping was presumed for the analytical model to distinguish effects of unbonded steel layout on wall seismic performance. The wall model was developed using Drain-2DX, which has been extensively used for modeling the walls. Nonlinear static pushover, cyclic pushover and nonlinear dynamic analysis were performed. Roof drift, base shear force, self-centering capability, and change in the post-tensioning force were monitored.

Results of monotonic pushover analysis showed that initial lateral stiffness, fundamental period, and maximum load resisting capacity of the walls were not significantly affected with unbonded tendons layout. When the tendons were placed in the wall vertical center line, wall drift at yielding was increased, but base shear force at the softening stage increased and yielding drift decreased when the unbonded tendons were placed at the ends or distributed along the wall joint. Cyclic analysis results showed that the shape of the hysteresis was considerably affected with unbonded tendon layout. Placing unbonded tendons in the vicinity of the wall vertical centerline decreased total post-tensioning forces and increased the amount of dissipated energy. Nonlinear dynamic analysis results showed that placing the unbonded tendons at wall ends or distributing

along the joint has limited effects on wall maximum roof drift under design and maximum considered earthquakes. In addition, placing the tendons at all ends or uniformly distributing resulted in higher prestress force losses and therefore it was recommended to place the tendons in the vicinity of the wall vertical centerline.

2.4 Previous Studies on Additional Energy Dissipation Mechanism

The analytical and experimental research performed on seismic behavior of unbonded post-tensioned precast shear walls, beam-column connections, and bridge piers has showed that the energy dissipation capability of the unbonded post-tensioned precast systems is considerably less than that of conventionally designed structure due to the fact that inelastic behavior is provided by gap opening without any significant damage on concrete where plastic hinges are expected to occur. To prevent large displacement or drifts under lateral loads and obtain larger energy dissipation through cyclic loading, it has been shown through research that additional energy dissipation mechanisms are needed for such systems to limit their displacement/drift demands under lateral loads such as seismic loads. In the following section, previous research on different mechanisms proposed in the literature to increase such systems energy dissipation capacity are summarized. Such mechanisms include using mild steel bars, viscous fluid dampers, and connectors as supplemental energy dissipators.

2.5 Viscous Fluid and Friction Dampers to Dissipate Energy

Viscous fluid dampers or friction devices can be placed to precast concrete shear wall connections, beam-column joints, or column toe regions that incidence of gap opening happens for the purpose of increasing the amount of dissipated energy. Kurama

[19] used the viscous fluid dampers as a bracing column which connected to a precast concrete wall at one end and the successive columns at another end diagonally. Kurama [20] introduced a friction damper that is used in the vertical connection joints where slip displacement between precast walls takes place. Bora et al. [15] studied a slotted-bolt connection in which a slip occurred at the connection between wall base and foundation and energy dissipation was achieved due to friction. Morgen and Kurama [44] presented a friction damper for precast RC frames with debonded post-tensioned strands and conducted an experimental study on frame connection specimen where two friction dampers were placed on both sides of column-beam joints to investigate the nonlinear response of precast frame joints with dampers. Marriot et al. [68] conducted shake table tests for precast concrete walls with viscous dampers only, mild steel dampers only, a combination of dampers and without dampers to investigate the performance of post-tensioned walls with supplemental energy dissipators. Guo et al. [17] proposed a friction device that is distributed along with each floor level where beam-column connections intersect with precast shear walls. Negro and Tornaghi [69] studied a friction-based bolted connection, like a slotted bolt connection, between the vertical joint connection of successive precast concrete walls. Xu et al. [26] proposed a disc spring device that placed symmetrically at the bottom corners of the precast shear wall. Only disc spring devices were tested experimentally to investigate design parameters that affect the response of the device. Xiao et al. [25] conducted a low cyclic reversal loading test on two conventional walls and two self-centering precast shear walls with disc spring devices with the intent of analyzing hysteretic and self-centering behavior of the wall. Xu and Li [28] presented horizontal friction devices between the wall panel and foundation, vertical friction devices between successive precast sandwich walls. The behavior of walls with friction

devices was studied experimentally. The result from the experiment and theoretic analysis were compared with the result of finite element simulation for validation. Li et al. [22] introduced a yield-based and friction-based energy dissipating system for unbonded post-tensioned precast walls and conducted a large-scale experimental and analytical study. Bedrinana et al. [14] proposed the design of unbonded post-tensioned precast walls with external replicable hysteretic dampers that consist of a small-diameter mild steel bar and a round steel tube around the bar. Both dampers and wall specimens with dampers were tested under cyclic load to investigate deformation capacity and flexural strength. Also, a numerical model was developed to validate experimental results.

2.6 Connectors between Precast Elements to Improve Joint Behavior

In the jointed connection precast shear walls, overall strength, ductility, stiffness, and energy dissipation capacity is mainly controlled by vertical joints, joint gap opening, and slip friction between the panels. Because the deformation of the system is mainly controlled by the gap opening between the members, different types of mechanisms have been proposed, studied, tested, and evaluated by researchers to employ this gap opening to dissipate seismic energy. Soudki et al. [70] proposed using a shear key in the dry pack grouted connections to increase shear resistance and prevent slip displacement. Schultz and Magana [29] conducted an experimental study on behavior vertical connections between precast shear wall panels. Seven different vertical connection types, namely Notched Shear Plates (NSP), Slotted Flexure Plates (SFP), Inclined Flat Bars (IFB), X-shaped Axial Plates (XAP), Pinned Tension Struts (PTS), Vertical Joint Friction (VJF), and U-shaped Flexure Plates (UFP) have been evaluated in terms of yield parameters, ultimate parameters, and energy dissipation parameters. Perez [71] studied the seismic

response and design of vertical joint and ductile connectors between precast concrete panels. In the study, an analytical model was developed and tested under monotonic and cyclic loading to investigate the effect of different components besides the vertical ductile connectors on the energy dissipation behavior of the wall. Priestly et al. [31] presented stainless-steel energy dissipating U-shaped flexure plated connectors (UFP) to be used for connection between the vertical joints of precast wall panels to dissipate energy. Sritharan et al. [72] and [73] presented a precast load resisting system called Precast Wall with End Columns (PreWEC), in which a precast concrete shear wall is placed between concrete columns, with connectors to dissipate energy. As an energy-dissipating connector, special replicable connectors (e.g., J-connector) were used. Analytical models of the walls with special connectors were developed using OpenSees software and tested under monotonic and cyclic load. Henry et al. [18] studied the design and behavior of special connectors for PreWEC wall systems. In order to evaluate self-centering behavior, finite element analysis of different connector types namely SFP, flexural plates with holes, J, and O-connectors were conducted. Twigden and Henry [23] presented an experimental study on the unbonded post-tensioned rocking wall with O-connectors. Nine tests under cyclic loading were conducted by using different materials, profiles and cutting processes of O-connectors with the intent of evaluating the effectiveness and suitability of connectors as energy dissipators. Nazari conducted several large-scale experimental tests on Single Rocking Walls (SRW) and Precast Walls with End Columns (PreWEC) [74] and [75]. In the program, four SRWs and four PreWECs specimens were subjected to quasi-static load with different intensities to evaluate the lateral response of the walls in terms of the number and location of O-connectors. Obtained results were validated with the developed analytical model using OpenSees software. Like the

previous study, Twigden et al. [24] also performed an experimental study on the hysteretic behavior of SRWs and PreWECs. Two SRWs and two PreWECs with different numbers of O-connectors, but the same properties were tested under cyclic lateral loading.

2.7 Unbonded Mild Steel Bar to Dissipate Energy

Using bonded mild steel bars along the horizontal connections for the jointed precast concrete wall systems is another effective way to increase the energy dissipation capacity of the systems due to the yielding of mild steel bars in tension and compression under cyclic loading. Kurama [11] studied the effect of mild steel bars on the behavior of unbonded post-tensioned walls and investigated important parameters for mild steel bars. In addition to the unbonded post-tensioned prestressed steel partially unbonded mild steel bars, which are unbonded over the joint gap opening, are used to increase energy dissipation capacity of the walls. The unbonded mild steel is dissipating energy through steel hysteretic behavior under tension and compression with gap opening. A prototype wall with partially unbonded mild steel studied by Kurama [11] is shown in Figure 2-9 below.

To maximize energy dissipation through the unbonded mild steel, the bars are placed in the vicinity wall vertical centerline. The bars are designed to yield at large displacements. However, the bars are difficult to install for such a precast system, and replacing the bars is not easy after a severe earthquake.

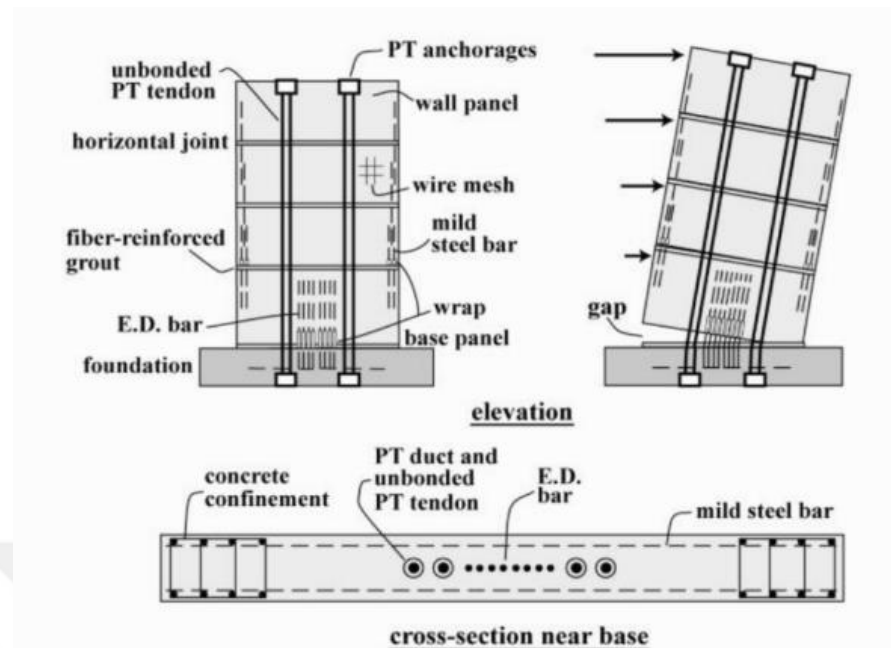


Figure 2-9: Unbonded post-tensioned wall with partially unbonded mild steel

Rahman and Restrepo [12] conducted an experimental study on hybrid walls where dog-bone mild steel bars were placed between wall base and foundation. Results of the test under quasi-static reversed cyclic loading showed that flag shape hysteresis response with 14% equivalent viscous damping ratio achieved. Holden [37] [76] tested conventionally designed and hybrid designed geometrically identical walls under reversed cyclic loading for the purpose of comparing cyclic behavior of walls. Similarly, Smith and Kurama [13] conducted an experimental study on two 0.4-scale hybrid walls and one emulative wall to evaluate lateral load behavior of two design approaches for wall design. The wall with continuous mild steel bars had good energy dissipation, self-centering, and ductile behavior over large lateral displacements.

CHAPTER III

DESIGN OF PROTOTYPE STRUCTURE AND SHEAR WALLS

This chapter describes structural properties and design of a prototype parking garage structure for seismic loads. The structure preliminary lateral load resisting system is composed of unbonded post-tensioned precast shear walls. Design details for prototype shear walls are given. The required steel amounts for both unbonded tendons and mild steel bars were estimated for the given seismic loads. Design guidelines for the walls with different mild steel layouts are given. Eventually, several hybrid walls are designed for seismic assessment of the prototype structure and effects of unbonded mild steel.

3.1 Prototype Parking Garage Structure

General details of the prototype precast concrete parking garage structure are given by Erkmen [77]. The building floor and elevation views are given in Figure 3-1 and Figure 3-2, respectively. The prototype structure is a 5-story precast concrete parking garage with a rectangular floor footprint of 81 m by 54 m. The building is 17 m in height with a 3.2 m story height. The building has 1 m perimeter walls on the roof. The lateral load resisting system of the parking garage is composed of four unbonded post-tensioned concrete shear walls in the N-S direction and bearing walls in the E-W direction.

Four unbonded post-tensioned precast shear walls are used as the only lateral load resisting system in the N-S direction, and 30 load-bearing walls are used for the lateral load resisting system in the E-W direction. Each shear wall comprises five precast

concrete wall panels along the building height. Bearing walls, which have 205 mm thickness and 2.3 m length, are spaced at approximately 3.7 m center to center. In total, 24 individual columns are used as the gravity load-bearing systems alongside the bearing walls and shear walls. At floor, 75-cm deep and 3.7 m wide pre-topped double tees are simply supported at their ends to carry the floor gravity loads. Doubled tees are supported by the bearing walls and load-bearing spandrels in the longitudinal direction. For bays at both ends, double tees are supported by the spandrels and four 9.1 m x 1.0 m inverted T-beams. Two sloping ramps are located at the center of the building. The bays between the inverted T-beams connect the ramped bay ends at successive floor levels. The unbonded post-tensioned precast shear walls are located at each end of the ramps and support the inverted T-beams. Along the inner side of the spandrels, 5 cm x 75 cm, washes are used to achieve drainage purposes and resist water ponding. Four elevators and four staircases are placed at building corners for pedestrians' access to each floor.

In this study, only the lateral seismic behavior of the prototype precast parking garage building in the N-S direction is considered. Because the only horizontal load resisting system in this direction is the unbonded post-tensioned precast concrete shear walls, the detailed design of other building structural elements is not included, and the interaction between the walls and other building elements is ignored for simplicity. The details of structural elements other than the shear walls such as precast load bearing and non-load bearing spandrels, washes, pre-topped double tees, inverted T-beams, and columns are used to determine the required gravity and seismic loads that shear walls need to be designed to resist. The gravity loads of these structural elements are taken from Erkmen [77].

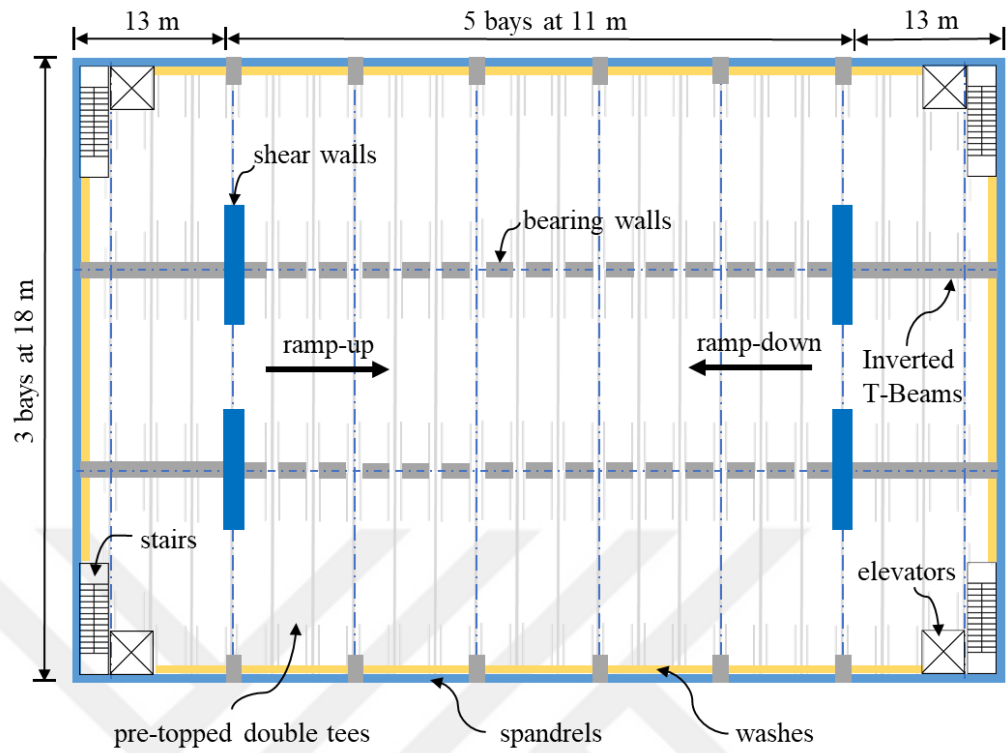


Figure 3-1: Prototype building plan view [77]

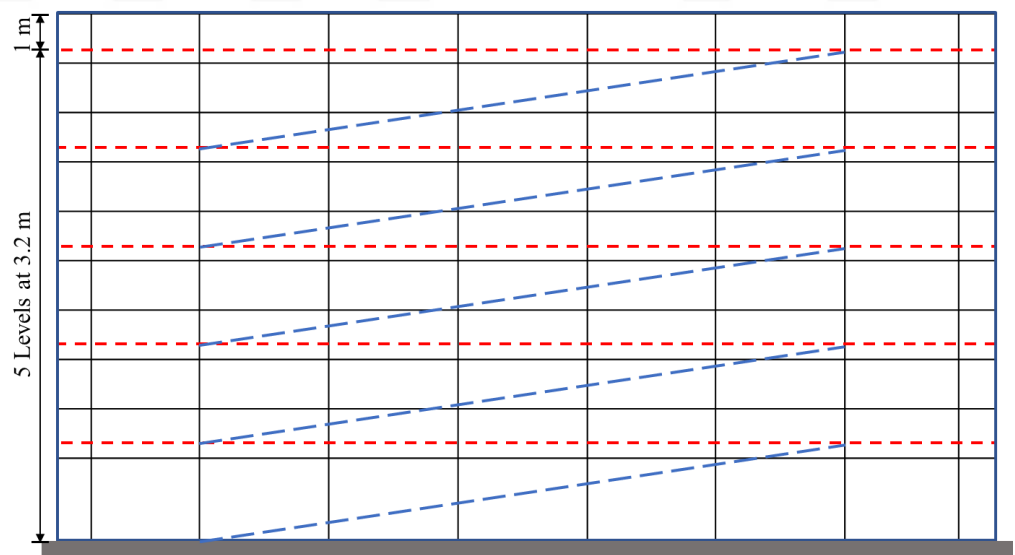


Figure 3-2: Prototype building elevation view [77]

3.2 Gravity Loads

The gravity loads are computed based on the tributary area method for shear walls based on two load combinations namely $1.2D+1.0E+1.0L$ and $0.9D+1.0E$, where D , E , and L indicate the dead load from the total weight of the structure and structural elements, seismic load, and live load, individually. The contribution of the live loads was neglected for the lateral load combination. Live loads are only considered to control the sufficiency of the boundary elements. Since the structure is a public use parking garage, unreduced live loads are used.

The structure's dead load includes self-weight of the precast concrete elements such as pre-topped double tees, spandrels, inverted girders, columns, shear walls, and washes, and other permanent structural elements like ceiling, electrical equipment, ducts, pipes, insulations, finishes, etc. Live loads are obtained from ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures [78]. Loads from the elevators and staircases are not considered. In Table 3-1, the self-weights of the structural members that are located on each floor and total self-weight are listed.

Table 3-1: Self-weight of structural components

Components	Self-Weight
DTs (366 cm x 76 cm, pre-topped)	3.88 kN/m ²
Wash (5 cm x 75 cm at the perimeter)	0.048 kN/m ²
IT Beams (4 at 1.49 kN/m)	0.192 kN/m ²
Spandrels (Load Bearing)	0.48 kN/m ²
Spandrels (Non-load Bearing)	0.287 kN/m ²
Columns (24 at 0.82 kN/m)	0.144 kN/m ²
Walls (36 at 8.7 kN/m)	0.192 kN/m ²
Shear Walls (4)	0.120 kN/m ²
Total Self Weight	5.339 kN/m ²

3.3 Seismic Design Forces

In order to design post-tensioned precast shear walls, seismic forces on the walls including base shear force, base moment, and axial load due to self-weight, dead loads, and live loads are needed to be determined. Axial load on each floor is estimated by considering load combinations and tributary floor area of each wall. Design base shear force may be determined by using any of the procedures as per ASCE 7-16 [78]. Equivalent Lateral Force (ELF) procedures are used to determine the design base shear force for the walls.

Seismic weight and the fundamental period of the structure are essential parameters that are needed to be estimated to design precast concrete shear walls. In addition, design spectral acceleration parameters, response modification factor, seismic importance factor, and deflection amplification factor are also required parameters to estimate design base shear force. In this part, the design base shear force with seismic weight and fundamental period is calculated. Assumed seismic design parameters are given in Table 3.2. The prototype precast parking garage is located in a highly seismic region and stiff soil in which design spectrum with 5% damping has $0.8g$ spectral design parameters in a short period (S_{DS}) and $0.41g$ and $0.47g$ acceleration parameters at period of 1 sec (S_{D1}) for C and D site classes, respectively. The response modification factor R and deflection amplification factor C_d are obtained from ASCE 7-16 [78]. Because the shear walls are unbonded post-tensioned precast concrete shear wall and classified in the category of the special reinforced concrete shear wall, R and C_d factors are selected as 6 and 5, respectively. The seismic importance factor I is obtained from ASCE 7-16 [78] as 1.0 for risk category I.

Table 3-2: Wall seismic design parameters

Design Parameters	Values
S_{DS}	0.8g
S_{D1}	0.41g & 0.47g (Site Class C & D)
R	6.0
I	1.0
C_d	5

3.3.1 Seismic Weight and Fundamental Period

ASCE 7-16 recommends that all structural components that are physically tied to the structure and permanent equipment should be considered as contributing to the effective seismic weight of the structure, but mass due to live loads can be ignored. Hence, only the dead load and self-weight including shear walls, double tees, washes, inverted girders, spandrels, and columns are considered. The effective seismic weight per story is estimated by multiplying the self-weight of one floor with the area of that floor as

$$W_s = w_{total} A_{floor} = 5.339 \frac{kN}{m^2} \times 4415 m^2 \cong 23575 kN$$

Because there are four unbonded walls, $\frac{1}{4}$ of the total seismic mass is considered for each wall. Building fundamental period is required to determine seismic design forces. The fundamental period of the prototype parking garage in the direction of the shear walls can be approximately determined in accordance with the following equations

$$T_a = C_t h_n^x \quad (3-1)$$

where h_n is the building height and the coefficients C_t and x are obtained from Table 12.8-2 in ASCE 7-16 [78]. Building fundamental period was predicted to be 0.48 seconds.

3.3.2 Base Shear Force, V

The seismic base shear force of the structure in the direction under consideration can be calculated in accordance with the following equation:

$$V_d = C_s W \quad (3-2)$$

where W is the total effective seismic weight of the structure and C_s is the seismic response coefficient which can be determined with Equation 3-3. C_s value should be smaller than the value computed in Eq. 3-4 and not less than the value computed in Equation 3-5:

$$C_s = \frac{S_{DS}}{\frac{R}{I}} \quad (3-3)$$

where S_{DS} is the design spectral response parameter in a short period, R is the response modification factor, and I is the seismic importance factor,

$$C_s = \frac{S_{D1}}{T \left(\frac{R}{I} \right)} \quad (3-4)$$

where S_{D1} is the design spectral response parameter at period 1.0 s, and T is the fundamental period that was determined in the previous part.

$$C_s = 0.044 S_{DS} I \geq 0.01 \quad (3-5)$$

The seismic response coefficient C_s is obtained as 0.1334 by using the equations given above. Thereby, the design base shear force for the prototype building is estimated at approximately 15717 kN that is nearly 3930 kN for each unbonded precast shear wall.

3.4 Vertical Distribution of Seismic Forces

Vertical distribution of seismic design forces is needed to determine base moment. Equivalent lateral force procedure in ASCE 7-16 [78] is used to determine vertical distribution of seismic loads. The total seismic design force is distributed to each floor level along the building height in proportion to the seismic weight of the corresponding floor. The lateral seismic force on each floor level is evaluated in accordance with the following equations:

$$F_x = C_{vx}V \quad (3-6)$$

where F_x indicates lateral force induced to stories, x indicated floor level, V is the total design base shear force at the base of the building, C_{vx} is a vertical distribution factor that is determined in accordance with the following equation:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (3-7)$$

where w_i and w_x are the effective seismic mass of the building that appointed to the level i or x , h_i and h_x are the height of the building at the level i or x from the foundation, and k is a power value related to the period of the structure ($k=1$ for building that has a period less than 0.5 s).

Table 3-3 summarizes the seismic weights, vertical distribution factors, induced seismic force, total base moments at each floor level, and total reaction that occurred at the base of the building. Since the primarily considered lateral load resisting system comprised of four unbonded post-tensioned precast concrete shear walls, lateral seismic

forces and moments at each floor level and base of the structure are divided by four as equal to each shear wall. Overall cumulative shear force, moment, and gravity load acting on each floor level and foundation are listed in Table 3-4.

Table 3-3: Lateral force distribution

Floor Level	Height, h (m)	Weight, w (kN)	$w_i h_i^k$ (kN.m)	C_{vx}	F_x (kN)	M (kN.m)
1	3.2	23575	75440	0.333	1048	3353
2	6.4	23575	105880	0.267	2096	13412
3	9.6	23575	226320	0.2	3143	30178
4	12.8	23575	301760	0.133	4191	53649
5	16.0	23575	377200	0.067	5239	83826
	Total (Σ)	117875	1131600	1.0	15717	184418

Table 3-4: Design force demands per wall

Floor Level	Shear, V_{wd} (kN)	Moment, M_{wd} (kN.m)	Gravity Load, N_{wd} (kN)
4	1310	20960	632
3	2358	34374	1264
2	3144	41920	1896
1	3668	45274	2528
Foundation	3930	46112	3160

3.5 Prototype Unbonded Precast Shear Walls

The unbonded post-tensioned concrete walls comprises of five 3.2 m height precast concrete panels resulting in a total wall height of 16 m. The cross-sectional wall dimensions are 7 m in length and 0.3 m thickness, therefore the aspect ratio for walls is approximately 2.3. The unbonded post-tensioned tendons are located in the vicinity of the wall vertical centerline and anchored at their ends to the foundation and roof. The tendons

are placed in the vicinity of the center line to minimize the prestress losses due to wall drift as recommended by design guidelines. The tendons are enclosed by oversized ducts in order to prevent friction between tendons and concrete.

During the rocking motion of the wall, large compressive strains generate at the boundary region of concrete wall panels. Thus, these regions are confined with the spiral reinforcement to lateral confine the concrete and prevent its premature failure. Wall dimensions, distribution of seismic loads, floor gravity loads, and base shear and moment are given in Figure 3-3.

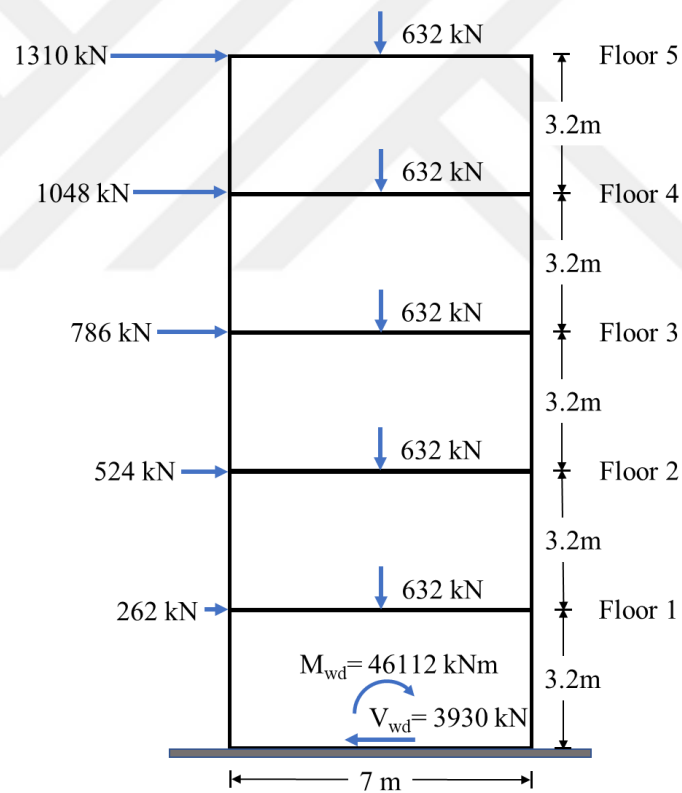


Figure 3-3: Seismic forces for unbonded post-tensioned shear wall

3.6 Wall Construction Material

Typical construction materials are assumed for the precast concrete walls. The stress-strain profiles of concrete and steel materials assumed for the walls are given in Figures 3-4 and 3-5, respectively. The unconfined compressive strength of concrete for the prototype wall, f'_{c0} is assumed to be 40 MPa. Confined concrete properties are estimated by using the nonlinear stress-strain relationships proposed by Mander et al. [79]. The confined concrete strength f'_{cc} is estimated as 75 MPa, and it is used for wall ends where spiral steel is used to prevent concrete premature failure under compression.

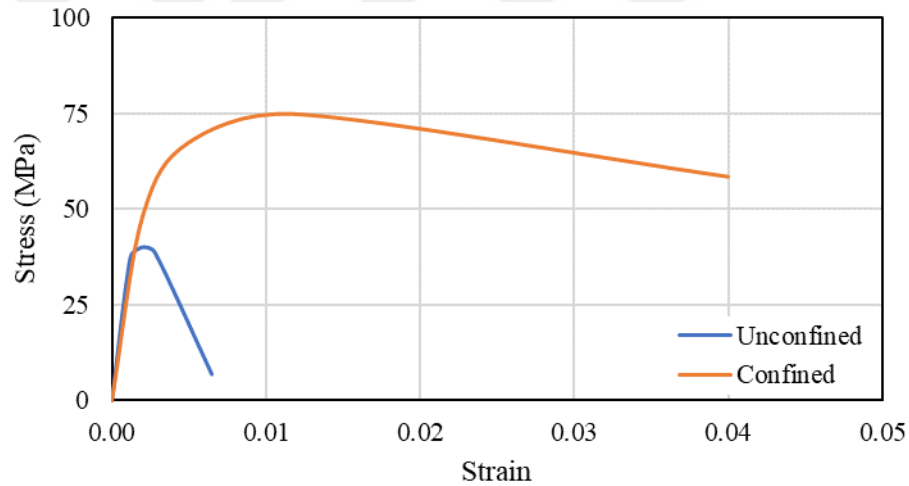


Figure 3-4: Stress-strain relationship of unconfined and confined concrete [79]

The properties of the unbonded post-tensioned steel tendons were obtained from the stress-strain relationship, which was used for the PTT wall specimen tested at NIST Experimental Program [30]. The average yield strength of the PT steel, f_{py} is 903 MPa at a strain, ϵ_{py} of 0.0046 mm/mm, and the average ultimate strength, f_{pu} , is 1125 MPa at a strain, ϵ_{pu} of 0.075 mm/mm. The strain value at fracture, ϵ_{pf} is 0.085 mm/mm. Therefore, the corresponding Young's Modulus of PT steel, E_p is 195,880 MPa. Each PT steel is left unbonded and post-tensioned to an effective stress of 75% of its ultimate strength, f_{pu} .

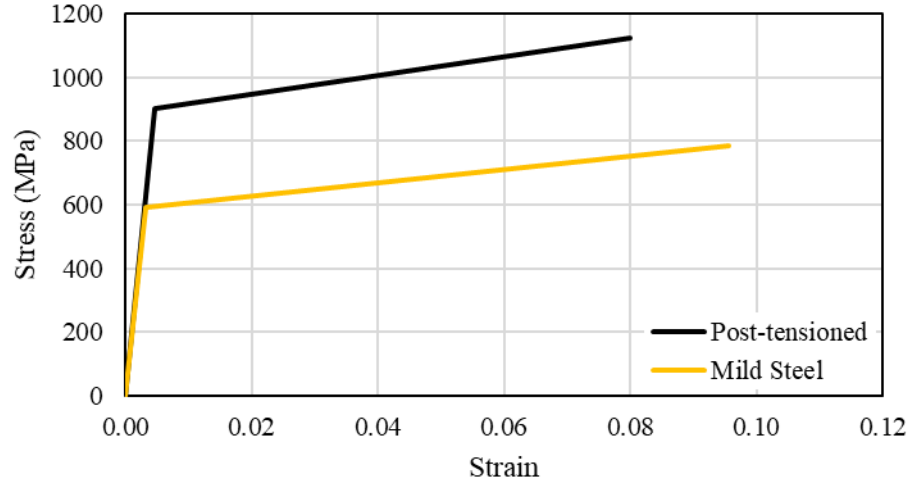


Figure 3-5: Stress-strain relationship of post-tensioned and mild steels [30]

The properties of the energy dissipating mild steel bars are obtained from the stress-strain relationship that was used for the GSS wall specimen tested at NIST Experimental Program [30]. The average yield strength of the PT steel, f_{sy} is 415 MPa at a strain, ϵ_{sy} of 0.0046 mm/mm, and the average ultimate strength, f_{su} , is 615 MPa at a strain, ϵ_{su} of 0.12 mm/mm. Therefore, the corresponding Young's Modulus of PT steel E_s is 193,053 MPa. Mild steel bars used for the wall are left unbonded at a predetermined length and no post-tensioning is applied.

Elastic modulus, E_c and shear modulus, G_c are the mechanical properties of the concrete that are needed for linear elastic analysis models for the determination of effective flexural stiffness and effective shear stiffness, respectively. These parameters shall be determined in accordance with the following Eq. 3-8 and 3-9:

$$E_c = 4730 \sqrt{f'_c} \text{ (MPa)} \quad (3-8)$$

where f'_c is the design compressive strength in MPa of unconfined concrete and

$$G_c = \frac{E_c}{2(1 + \nu)} (MPa) \quad (3-9)$$

where ν is the Poisson ratio for the concrete and is assumed as 0.18. The values of modulus of elasticity and shear modulus were predicted to be 29915 MPa and 12676 MPa, respectively.

3.7 Wall Drift Demands under Seismic Loads

The gap opening at the wall base joint is needed to properly design the unbonded length required for the mild steel. If the unbonded length is too short, then the bars will yield at small deformations and may fracture when the maximum gap opening occurs. On the other hand, if mild steel unbonded length is too big then the bars will not experience significant yielding and the energy dissipation due to mild steel yielding will be limited. The wall gap opening depends on wall drift values.

Wall drift is defined as the ratio of lateral deformation at the top of the structure to the height of the structure from foundation to roof. As recommended by ACI ITG-5.1 [80], three types of lateral deformation can be observed on a precast concrete shear wall that are flexural deformation, shear deformation, and shear slip between panels. However, deformation due to shear slip is not considered in the calculation of drift demand since the design provision requires that substantial shear slip between precast concrete wall panels should be prevented. Total lateral top displacement of the wall due to both flexure and shear can be determined by using deflection equations by assuming the shear wall as a solid cantilever beam. Figure 3-6 exhibits the deflected shape of the solid cantilever shear wall where δ_w is the lateral top displacement, V_{wd} is the design base shear force,

H_{wall} is the total height of the wall, and h_{wall} is the height that design base shear force acts on as a point load.

The height can be determined with equilibrium equations or as in the following equation:

$$h_{wall} = \frac{M_d}{V_{wd}} \quad (3-10)$$

Overall lateral displacement of the wall is the summation of the flexural displacement, $\delta_{w,flex}$ and shear displacement, $\delta_{w,sh}$ in which shall be estimated in accordance with Eq. 3-12 and 3-13, respectively:

$$\delta_w = \delta_{w,flex} + \delta_{w,shear} \quad (3-11)$$

$$\delta_{w,flex} = \frac{V_{wd} h_{wall}^2}{6E_c I_e} (3H_{wall} - h_{wall}) \quad (3-12)$$

$$\delta_{w,sh} = \frac{V_{wd} H_{wall}}{G_c A_{sh}} \quad (3-13)$$

where E_c is the young's modulus, I_e is the effective second moment of area that can be determined by using Eq. 3-14, G_c is the shear modulus, and A_{sh} is the effective shear area of the cross-section which can be estimated with Eq. 3-16:

$$I_e = 0.5 I_{gross} \quad (3-14)$$

where:

$$I_{gross} = \frac{1}{12} t_w L_w^2 \quad (3-15)$$

$$A_{sh} = 0.8 A_{gross} \quad (3-16)$$

where A_{gross} is the gross area of the wall cross-section. The gross and effective moment of inertia was obtained to be 8.6 m^4 and 4.3 m^4 , respectively. Effective shear area was estimated to be 1.7 m^2 . Also, flexural displacement and shear displacement were obtained to be 0.02548 m and 0.00295 m , respectively corresponding to 0.02843 m overall displacement. Once the total lateral displacement is obtained, the linear elastic drift Δ_{we} shall be obtained using Eq. 3-17:

$$\Delta_{we} = \frac{\delta_w}{H_{wall}} 100\% \quad (3-17)$$

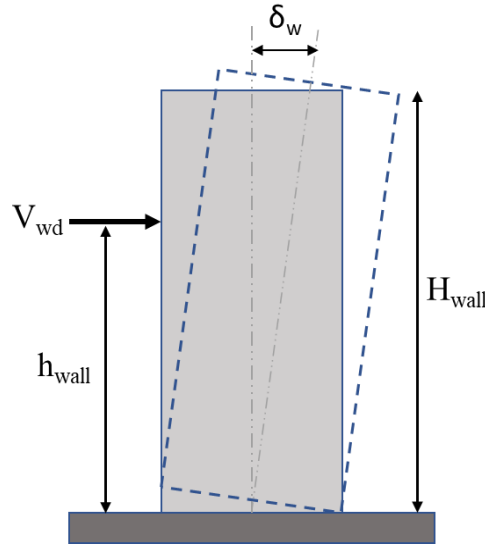


Figure 3-6: The deflected shape of the shear wall

3.8 Design and Maximum Level Drift

As per ASCE 7-16 [78], the design level drift, Δ_{wd} under design ground motion (DGM) can be determined simply by multiplying the linear elastic drift with deflection amplification factor or in accordance with Eq. 3-18 and 3-19:

$$\delta_{wd} = \frac{C_d \delta_w}{I_e} \quad (3-18)$$

$$\Delta_{wd} = \frac{\delta_{wd}}{H_{wall}} 100\% \quad (3-19)$$

where C_d is the deflection amplification factor that can be chosen from Table 12.2-1 in ASCE 7-16, δ_w is the linear elastic deflection determined in elastic analysis, and I_e is the importance factor that can be selected from Table 1.5-2 in ASCE 7-16. Because the considered structural element is the unbonded post-tensioned concrete shear wall and it is classified as building frame systems with “special reinforced concrete shear wall”, the corresponding C_d and I_e values are found to be 5.0 and 1.0, respectively.

In fact, nonlinear dynamic analysis under appropriate ground motion histories can be used to determine both design level drift and maximum level drift. Besides, the maximum level drift shall be taken as equal to the validation level drift value in Eq. 3-20 that was proposed by ACI ITG-5.1 [80].

$$\Delta_{wc} = 0.9\% \leq \frac{H_w}{L_w} 0.8\% + 0.5\% \leq 3.0\% \quad (3-20)$$

where H_w and L_w are the height and length of the shear wall giving the aspect ratio. In this study, the maximum level drift is assumed as in Eq. 3-21 like in the design guidelines by Smith and Kurama [81]:

$$\Delta_{wm} = 0.95 \Delta_{wc} \quad (3-21)$$

The estimated drift values are 0.18%, 0.89%, and 2.2% for linear elastic, design, and maximum drift levels, respectively.

3.9 Wall Design

The unbonded-post tensioned precast concrete shear walls are designed to have substantial gap opening under seismic loads between the base panel and foundation per requirements in Section 4-4-6 in ACI ITG-5.2 [82]. Therefore, only the base joint of the wall is considered in this section for gap opening. In other words, the gap opening on above wall joints is neglected. The wall dimensions are already defined, and the required steel area for the unbonded post-tension tendons will be estimated by using equilibrium and constitutive relationship. In the determination of the steel area, the design guidelines by Smith and Kurama [81], ACI-318 [83], and ACI ITG-5.2 [82] are used.

The free-body diagram for the base joint of the unbonded post-tensioned concrete shear wall is given in Figure 3-7. The unbonded PT steels are placed at both sides of the wall centerline with eccentricities e_p . As shown in the figure, the gap between the base panel and foundation opens as much as the wall drifts under design level ground motion and the PT steel elongates in the amount of δ_{pd} . During this rocking motion, a compressive stress block develops at the edge where high compressive strain occurs along the contact length, and therefore the wall centerline shifts down to the tension side.

Post-tensioning force is an essential factor in seismic behavior of the unbonded post-tensioned precast concrete walls in that it interlocks the precast panels and resist

Required steel area can be estimated by obtaining elongation of PT steel at drift Δ_{wd} . Since the base panel pulls away from the foundation in the amount of $L_w - c_d$, the contact length is needed to determine the elongation. The contact length, c_d shall be obtained by solving the Eq. 3-22 and 3-23:

$$\frac{M_{wd}}{\phi_f} = C \left(\frac{L_w}{2} - \frac{a_d}{2} \right) \quad (3-23)$$

where C is the resultant compressive stress at the base of the wall, f_c' is the unconfined concrete compressive strength, t_w is the wall thickness, M_{wd} is the design base moment, Φ_f is the axial flexural capacity reduction factor as per ACI 318, L_w is the length of the wall, and the a_d is the length of the compressive stress block that can be determined by multiplying the contact length with a factor of β_1 as per ACI 318-11. Solving the equations with $f_c' = 40$ MPa, $t_w = 0.3$ m, and $\beta_1 = 0.75$, the contact length c_d and resultant compressive stress C are calculated as 2.687 m and 20555.5 MPa, respectively.

Elongation of the post-tensioned steels at design level drift Δ_{wd} shall be obtained in accordance with Equation 3-24:

$$\delta_{pd} = \Delta_{wd} \left(\frac{L_w}{2} - c_d \pm e_p \right) \quad (3-24)$$

where Δ_{wd} is the drift at design level ground motion, c_d is the contact length, and $e_p = 0.5$ m is the eccentricities of the post-tensioned steels lumped at each side of the wall centerline. The minimum and maximum elongations of PT steel for each side of the centerline are calculated as 0.002785 m and 0.011685 m, respectively. The corresponding PT tendons stress f_{pd} at elongation δ_{pd} shall be obtained with the following Eq. 3-25:

$$f_{pd} = f_{pi} + E_{pi} \left(\frac{\delta_{pd}}{L_{pu}} \right) \quad (3-25)$$

where f_{pi} is the initial post-tensioning force which is equal to 75% of the yield stress of the tendons, E_{pi} is the initial Modulus of Elasticity equals to 195,880 MPa, and $L_{pu} = 16$ m is the total unbonded length of post-tensioned tendons. The minimum and

maximum tendon stresses are calculated as 576 MPa and 685.3 MPa, respectively with an average of 630.66 MPa.

Eventually, the total area of unbonded post-tensioned tendons can be obtained from the fundamental equilibrium equations for the base joint of the shear walls as in Eq. 3-26:

$$C = A_{pd}f_{pd} + N_{wd} \quad (3-26)$$

where A_{pd} is the total PT steel area, f_{pd} is the tendon stress at drift Δ_{wd} , and N_{wd} is the total axial load at the base joint of the shear wall. The minimum total steel area is calculated as 0.02767 m².

As emphasized before, the unbonded post-tensioned tendon stresses should be in the elastic range under both design level and maximum level ground motions. Thus, the stress level of the post-tensioned tendons at maximum drift needs to be checked. At the maximum level drift, the elongation of the post-tensioned tendons shall be determined with Eq. 3-27:

$$\delta_{pm} = \Delta_{wm} \left(\frac{L_w}{2} - c_m \pm e_p \right) \quad (3-27)$$

where Δ_{wm} is the drift at maximum level ground motion, c_m is the contact length that is assumed to be equal to $1.05 c_d$, and e_p is the eccentricities of the post-tensioned steels lumped at each side of the wall centerline. The minimum and maximum elongation of post-tensioned steel for each side of the centerline is calculated as 12.34 mm and 33.54 mm, respectively. The corresponding post-tensioned tendons stress f_{pm} at elongation δ_{pm} can be estimated with the following:

$$f_{pm} = f_{pi} + E_{pi} \left(\frac{\delta_{pm}}{L_{pu}} \right) \quad (3-28)$$

The minimum and maximum tendon stresses are calculated as 693.3 MPa and 953.7 MPa, respectively with an average of 823.5 MPa. According to the stress-strain relationship for post-tensioned steel in Figure 3-6, the result shows that the tendon stresses are in the elastic region at both design level and maximum level ground motions.

Consequently, the design of the unbonded post-tensioned precast shear wall is completed by specifying the post-tensioned steel area. The designed shear wall elevation and cross-sectional views at the base panel are given in Figures 3-8 and 3-9. The prototype walls used in the parametric study have the same geometry, material properties, and initial post-tensioning forces to investigate effects of mild steel on wall performance. The walls are 30 cm in thickness, 7.0 m in length, 16.0 m in total height with a story height of 3.2 m. The base wall panel concrete is confined at both ends to prevent concrete premature failure under high concrete compressive stresses in these regions. Total 20 post-tensioning tendons with 290 cm² total steel area are placed in the vicinity of the wall centerline in two rows with 0.15 m center-to-center displacement as shown in Figure 3-8. The tendons are placed in oversized ducts to ensure tendon debonding, and they are anchored to the structure at only foundation and roof levels. The unbonded tendons are post-tensioned to 19445 kN which is equal to 75% of its yield stress.

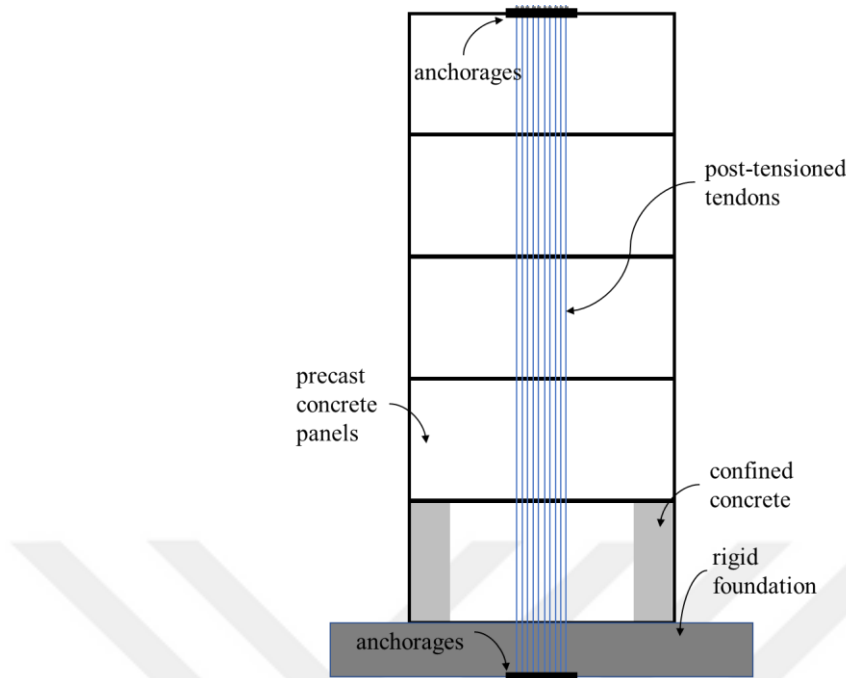


Figure 3-8: Elevation view of the unbonded post-tensioned precast concrete wall

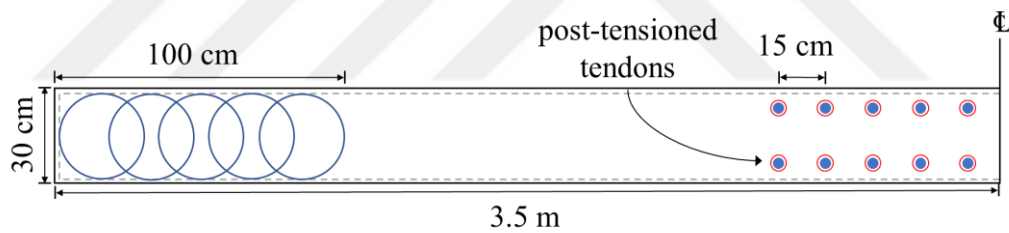


Figure 3-9: Cross-sectional view of the base panel

3.10 Estimation of Wall Joint Gap Opening

The objective of this section is to develop closed form equations to predict wall gap opening as a function of wall lateral displacement. The developed approach is based on assumptions that wall shear deformation is negligible and wall gap opening occurring at wall joints above the base is negligible. This is a conservative approach for mild steel design. Predicting the gap opening as a function of wall lateral displacement is important for proper design of unbonded mild steel. The gap opening is used to compute mild steel unbonded length needed to ensure the mild steel yield and dissipate energy and while

doing that maximum strain demand is small enough to ensure that mild steel does not rupture.

3.11 Drift

In this section, equations for lateral displacement and drift are proposed based on the original shape and deflected shape of a shear wall under lateral load as given in Figure 3-10. It is assumed that the wall rotates by θ and is displaced laterally by δ_x . When the wall is displaced under lateral load, the contact length of the wall panel at the base is assumed to be m , and the gap angle for wall panel gap opening is shown as β . Points A and B are the leftmost and mid-point of the wall top, respectively while Point A' and B' are the locations of points A and B after at the wall lateral top displacement of δ_x .

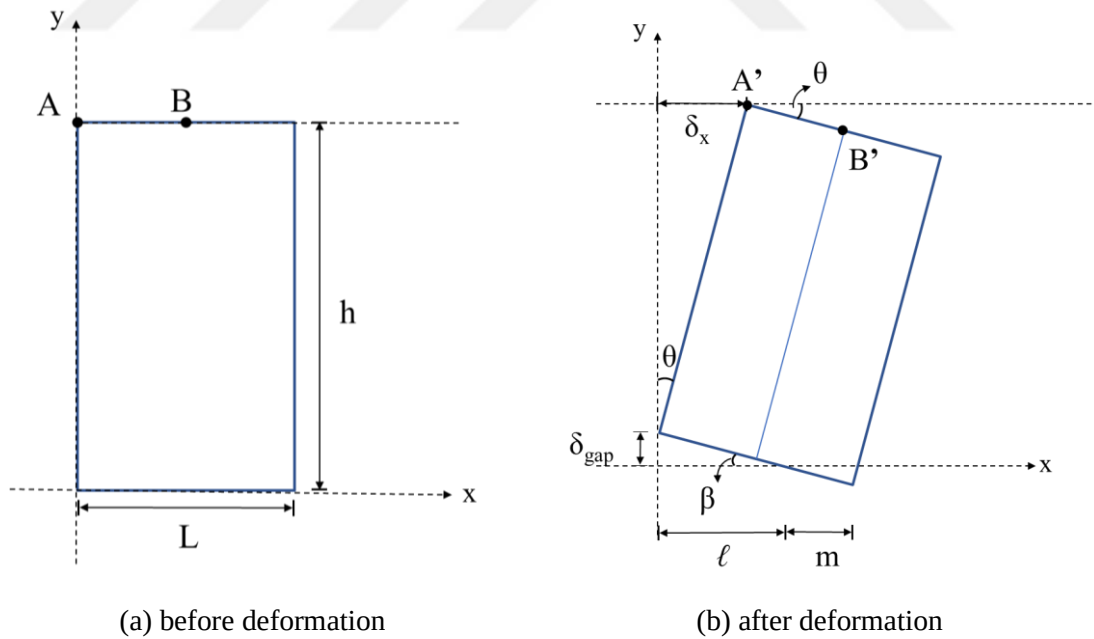


Figure 3-10: Original and deflected shape of wall under lateral load

The coordinates of points A and B shown in Figure 3-10 before and after wall lateral displacement are given as:

$$A(0, h) \rightarrow A'(\sin \theta \times h, \tan \beta \times \ell + h \times \cos \theta) \quad (3-29)$$

$$B\left(\frac{L}{2}, h\right) \rightarrow B'\left(\sin \theta \times h + \frac{L}{2} \times \cos \theta, \tan \beta \times \ell + h \times \cos \theta - \frac{L}{2} \times \sin \theta\right)$$

Wall lateral displacement, δ_x is estimated as relative horizontal displacement of wall top midpoint given as:

$$\delta_x = B'_x - B_x = \sin \theta \times h + \frac{L}{2} \times \cos \theta - \frac{L}{2} \quad (3-30)$$

The expression can be simplified as follows:

$$\delta_x = h \sin \theta + \frac{L}{2} (\cos \theta - 1) \quad (3-31)$$

3.12 Concrete Compression Depth

Under lateral loads, large compressive stress occurs at ends of the wall base panel, where the base panel is supported by the foundation, as shown in Figure 3-11. The part of the wall panel that is still in contact with the foundation and carrying the compression force is called concrete compressive depth and shown as with symbol m . This depth, which is also called contact length, is a significant performance parameter controlling wall gap opening and optimum unbonded length of both mild and post-tensioned steels. As shown in Figure 3-11, the compressive stress integral over the contact area should be equal to the total axial load due to dead load, live load, self-weight, and post-tensioning force. In other words, due to vertical force equilibrium for the wall panel, the summation of all vertical loads should be equal to the integral of compressive stress over the wall contact area.

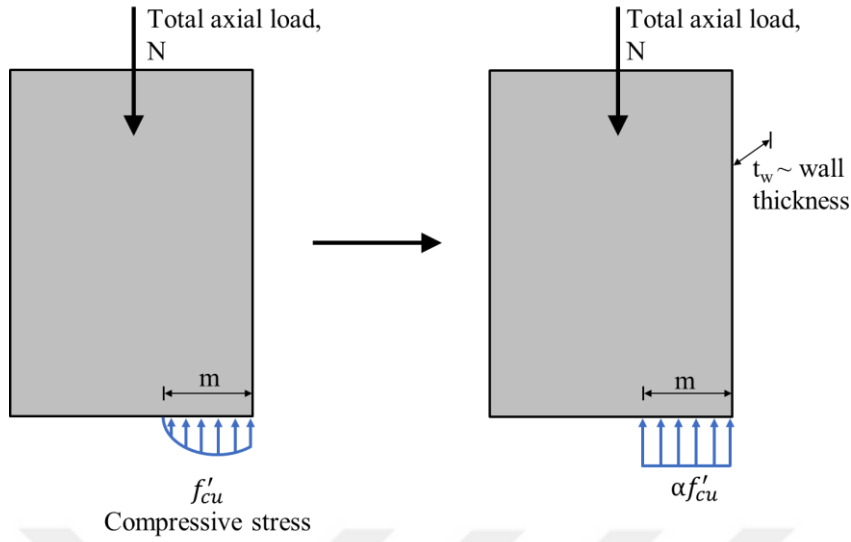


Figure 3-11: Idealized shape of the compressive block stress

The concrete compression depth, m can be estimated by converting the concrete actual nonlinear stress-strain curve to an equal rectangular block with the same compression depth m using the following expression:

$$m = \frac{N}{\alpha f_{cu} t_w} \quad (3-32)$$

where f_{cu} is the concrete compressive strength, t_w is the wall thickness, N is the total axial load due to dead load, live load, self-weight, and post-tensioning force, and α is a factor related to the length of the concrete block stress. The coefficient α shall be determined by creating an equivalent concrete compressive stress block using concrete stress-strain relationship. As given in Figures 3-12, an equivalent stress block can be created if the areas under stress-strain curves A_1 and A_2 are equal. The coefficient, α can be calculated as a function of concrete strain as shown in Figure 3-12. For the confined concrete assumed in this study, the coefficient α is converging to 0.9 as shown in Figure 3-12c.

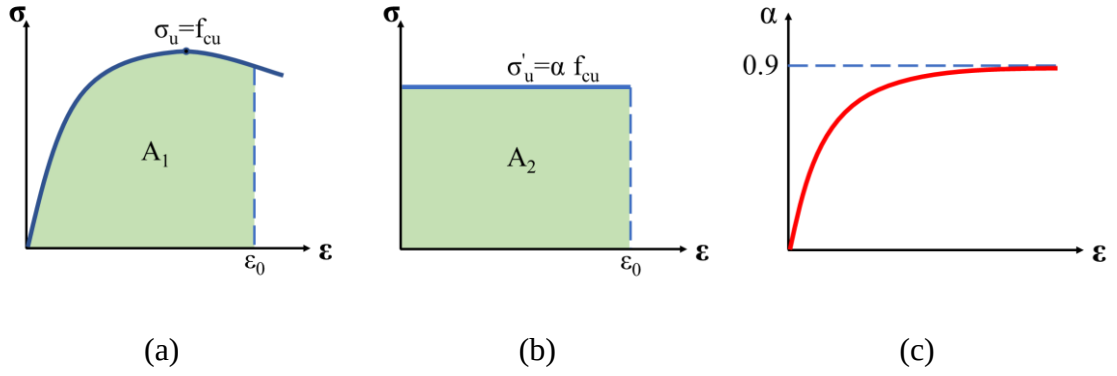


Figure 3-12: Equivalent compressive stress block

3.13 Gap Opening

The gap opening at any location along the wall joint can be predicted using the gap opening angle β and wall contact length m between the wall panel and foundation. The gap opening at the wall end δ_{gap} between the base panel and wall foundation can be predicted as:

$$\delta_{gap} = \ell \times \tan \beta, \text{ if the opening angle is small, } \delta_{gap} = \ell \times \beta \quad (3-33)$$

where $\ell = L - m$ and equal to the distance between the neutral axis and wall end edge. The gap opening at wall end can also be expressed as follow by substituting the expression for ℓ and m :

$$\delta_{gap} = \left(L - \frac{N}{\alpha f_{cu} t_w} \right) \times \tan \beta$$

If the walls are assumed to be rigid, the gap opening angle β and wall drift angle θ will be equal. However, the walls are not rigid, and the predicted gap opening will be overestimated if the wall is assumed to be rigid. The wall shear deformation increases as the lateral load increases. However, once the lateral load remains constant (i.e., shear

deformation) the shear deformation can be predicted, and a relationship between the wall drift and gap opening angles can be established. In this study, the relationship was established using deformations computed using the finite element model results. The computed relationship between the two angles is given by

$$\beta = 0.93 \theta$$

where, 0.93 is the proposed correction factor due to shear deformation, and it is equal to ratio of gap opening angle to wall drift angle. The gap opening angle is typically small, and the gap opening can be expressed using the following relationships as a function of wall drift.

$$\delta_{gap} = \left(L - \frac{N}{\alpha f_{cu} t_w} \right) \times 0.93 \times \theta \quad (3-34)$$

$$\delta_{gap} = \left(L - \frac{N}{\alpha f_{cu} t_w} \right) \times 0.93 \times \frac{\delta_x}{h} \quad (3-35)$$

It should be noted that the above relationships are valid for any lateral displacement if the appropriate values are used for the correction factors α and β . The proposed values of 0.9 and 0.93 for α and β , respectively are for the ultimate case where ultimate lateral force capacity is reached. The corresponding ultimate gap opening can be predicted as a function of wall lateral displacement or drift to design unbonded post-tensioned steel and mild steel bars.

3.14 Design of Energy Dissipating Mild Steel Bars

Mild steel bars are used for unbonded post-tensioned precast shear walls for energy dissipation purposes. This can be provided by ensuring yielding of bars in tension and compression by design during the excitation. Another wall seismic performance criterion is to have a wall gap opening only between the base wall panel and the foundation. If the unbonded mild steel amount placed over the joint between the wall panel and the foundation is excessive then the gap opening will not occur between the base wall panel and foundation but over upper wall joints. In such cases, the partially unbonded mild steel will not yield and not dissipate any energy under seismic loads due to lack of gap opening over the joint that mild steel is placed. Therefore, the amount of mild steel to be placed between the base wall panel and the foundation should be estimated in such a way that it provides maximum energy dissipation while keeping the joint gap opening between the foundation and wall base panel.

A design recommendation is proposed here to ensure gap opening at the base joint while maximizing mild steel amount and associated energy dissipation over the joint. The developed closed form expressions are based on wall deformation and seismic load distribution consistent with the wall first fundamental mode shape. Lateral loads acting on the building creates an overturning moment at each floor level while the post-tensioning and gravity loads create a restoring moment at the same level. Because the mild steel bars are only placed between the base panel and the foundation, the resisting overturning moments for the upper levels are controlled by total axial load (i.e., gravity and live loads) and post tensioning load only. The maximum mild steel area can be estimated by determining the gap opening moment and resisting moments at both

foundation and first floor levels. Figure 3-13 shows the distribution of the total lateral load, V with respect to wall 1st mode shape.

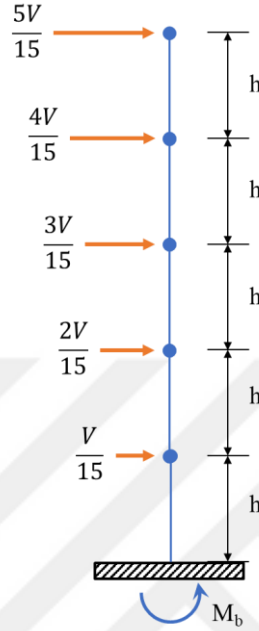


Figure 3-13: Seismic load distribution based on the 1st natural mode shape

The total overturning moment occurs at the base of the wall, M_b can be expressed as in Eq. 3-36:

$$M_b = \frac{5V}{15} \times 5h + \frac{4V}{15} \times 4h + \frac{3V}{15} \times 3h + \frac{2V}{15} \times 2h + \frac{V}{15} \times h = \frac{11Vh}{3} \quad (3-36)$$

Similarly, the total overturning moment at the first-floor level, M_1 is given by Eq. 3-37:

$$M_1 = \frac{5V}{15} \times 4h + \frac{4V}{15} \times 3h + \frac{3V}{15} \times 2h + \frac{2V}{15} \times h = \frac{8Vh}{3} \quad (3-37)$$

M_b and M_1 are the overturning moments causing the gap opening at foundation level and first floor level. On the other hand, a resisting moment resulting from the total axial loads, post-tensioning force, and mild steel bar resists closing the gap opening.

These resistance forces are given in Figure 3-14. The loads P_1 , P_2 , P_3 , P_4 , and P_5 are the cumulative total vertical loads at corresponding floor level due to self-weight of the structure, other dead loads, and live loads acting on wall tributary area and carried by the wall. The P_p is the total post-tensioning force which is assumed to be lumped at the wall vertical centerline. The load P_m is the axial load due to unbonded mild steel bars placed L' from its centroid to the wall rocking end.

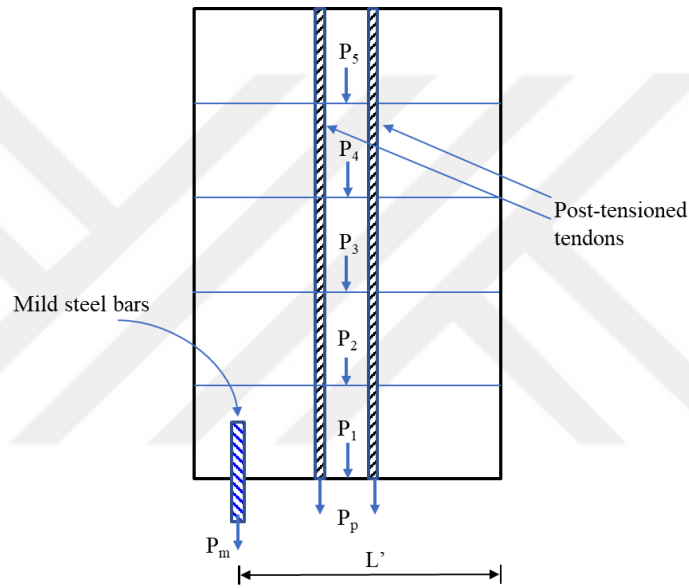


Figure 3-14: Vertical loads acting on a hybrid wall

Total resisting moment at wall base, M_{br} and at first floor level, M_{1r} about the point that resultant concrete compressive force is acting can be expressed as given in Eq. 3-38 and Eq. 3-39. The resultant concrete compressive force is located in the middle of the compressive stress block depth m , and this depth is assumed to be equal for both base and upper panels.

$$M_{br} = P_p \left(\frac{L}{2} - \frac{m}{2} \right) + P_1 \left(\frac{L}{2} - \frac{m}{2} \right) + P_m \left(L' - \frac{m}{2} \right) \quad (3-38)$$

$$M_{1r} = P_p \left(\frac{L}{2} - \frac{m}{2} \right) + P_2 \left(\frac{L}{2} - \frac{m}{2} \right) \quad (3-39)$$

For the wall joint at the base to open but the joint at the first floor not to open, the following condition should be satisfied for overturning and resisting moments:

$$M_b \geq M_{br} \quad \& \quad M_1 < M_{1r} \quad (3-40)$$

The gap opening moment should be greater than the resisting moment at the base of the wall while the resisting moment should be greater than the gap opening or overturning moment at the first-floor level. This condition can be also expressed using the following expression, where the closed form relationships developed for the moments are used:

$$\begin{aligned} \frac{11Vh}{3} &\geq P_p \left(\frac{L}{2} - \frac{m}{2} \right) + P_1 \left(\frac{L}{2} - \frac{m}{2} \right) + P_m \left(L' - \frac{m}{2} \right) \\ \frac{8Vh}{3} &< P_p \left(\frac{L}{2} - \frac{m}{2} \right) + P_2 \left(\frac{L}{2} - \frac{m}{2} \right) \end{aligned} \quad (3-41)$$

The above two equations can be combined and rearranged to obtain the following relationship for mild steel maximum axial force to ensure that gap opening occurs at the foundation level while the other joints remain closed under lateral loads:

$$P_m \left(L' - \frac{m}{2} \right) \leq \frac{11}{8} \left[P_p \left(\frac{L}{2} - \frac{m}{2} \right) + P_2 \left(\frac{L}{2} - \frac{m}{2} \right) \right] - P_1 \left(\frac{L}{2} - \frac{m}{2} \right) - P_p \left(\frac{L}{2} - \frac{m}{2} \right) \quad (3-42)$$

If the mild steel bars are placed in the vicinity of wall vertical centerline like post-tensioning bars, L' will be equal to $L/2$ and a much simpler expression will be obtained for mild steel total force as given by Eq. 3-43.

$$P_m \leq \frac{3}{8}P_p + \frac{11}{8}P_2 - P_1 \quad (3-43)$$

If the total axial loads due to dead load and live load are not significantly different at the floor and foundation levels, P_1 and P_2 can be assumed to be equal, then the disequilibrium can be written as in Eq. 3-44

$$P_m \leq \frac{3}{8}P_p + \frac{3}{8}P_1 = \frac{3}{8}(P_p + P_1) \quad (3-44)$$

3.15 Mild Steel Area

The amount of the mild steel bars placed in the vicinity of the wall vertical centerline can be estimated in accordance with Eq. 3-43 or Eq. 3-44 to allow gap opening to occur only between the base panel and the foundation. These equations need to be satisfied for all stress levels for both mild steel and post-tensioned steel for wall seismic performance levels. However, performance levels control the maximum area of mild steel (i.e., A_s). These are as follows:

1. Mild steel bars at yielding stress level and post-tensioned tendons at initial effective stress level.
2. Mild steel bars at ultimate stress level and post-tensioned tendons at yielding stress level.

$$(A_s f_{sy}) \leq \frac{3}{8}(A_p f_{pi}) + \frac{11}{8}P_2 - P_1 \quad (3-45)$$

$$(A_s f_{su}) \leq \frac{3}{8} (A_p f_{py}) + \frac{11}{8} P_2 - P_1 \quad (3-46)$$

The mild steel area determined using these equations ensures that wall gap opening will occur at foundation level for seismic loads with vertical distribution based on the wall first fundamental period of vibration. Because steel area of the post-tensioning tendons and materials stress-strain relationship are known for both post-tensioned and mild steels, the maximum cross-section area of mild steel, which is located in the vicinity of wall vertical centerline over the wall-foundation joint is estimated as 16150 mm² by solving the above equations. The maximum area of the mild steel bars is reduced to approximately 90% of the calculated amount to ensure gap opening at the foundation level when higher modes effects are considered. Five different prototype walls with mild steel bars placed in the vicinity of the walls' vertical centerline are designed to further investigate the effect of mild steel amounts on the seismic behavior of the wall. Five individual prototype walls namely Wall-C1, Wall-C2, Wall-C3, Wall-C4, and Wall-C5 are identical except for the amount of mild steel. The mild steel is 30%, 45%, 60%, 75%, and 90% of the maximum computed mild steel area (i.e., 16150 mm²) for Wall-C1, Wall-C2, Wall-C3, Wall-C4, and Wall-C5, respectively.

In addition, three prototype walls with mild steel bars placed in the vicinity of wall edges are designed to investigate the effect of mild steel location on wall seismic behavior. Because the mild steel bars move from the wall vertical centerline to its edges, wall resisting moment to overturning will increase. Therefore, the steel area of the mild steel bars needs to be reduced. The total steel area of the bars is adjusted 6000 mm² to have similar lateral load capacity for walls with mild steel at the edge and Wall-C3. For this purpose, three prototype walls namely Wall-E1, Wall-E2, and Wall-E3 having

different unbonded lengths are designed. Figure 3-15 shows wall elevation and cross-section views with mild steel bars placed in the vicinity of wall edges.

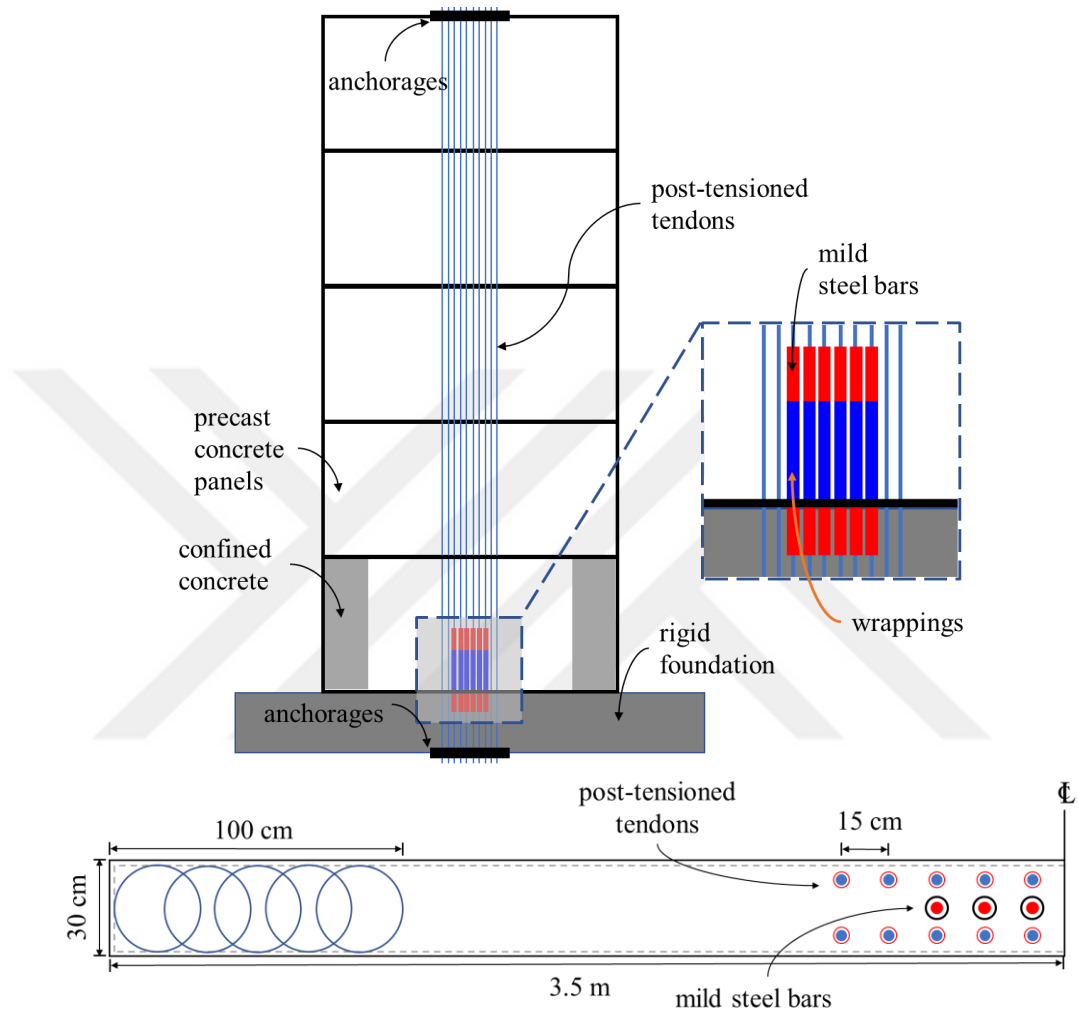


Figure 3-15: Wall with mild steel bars in the vicinity of wall vertical centerline

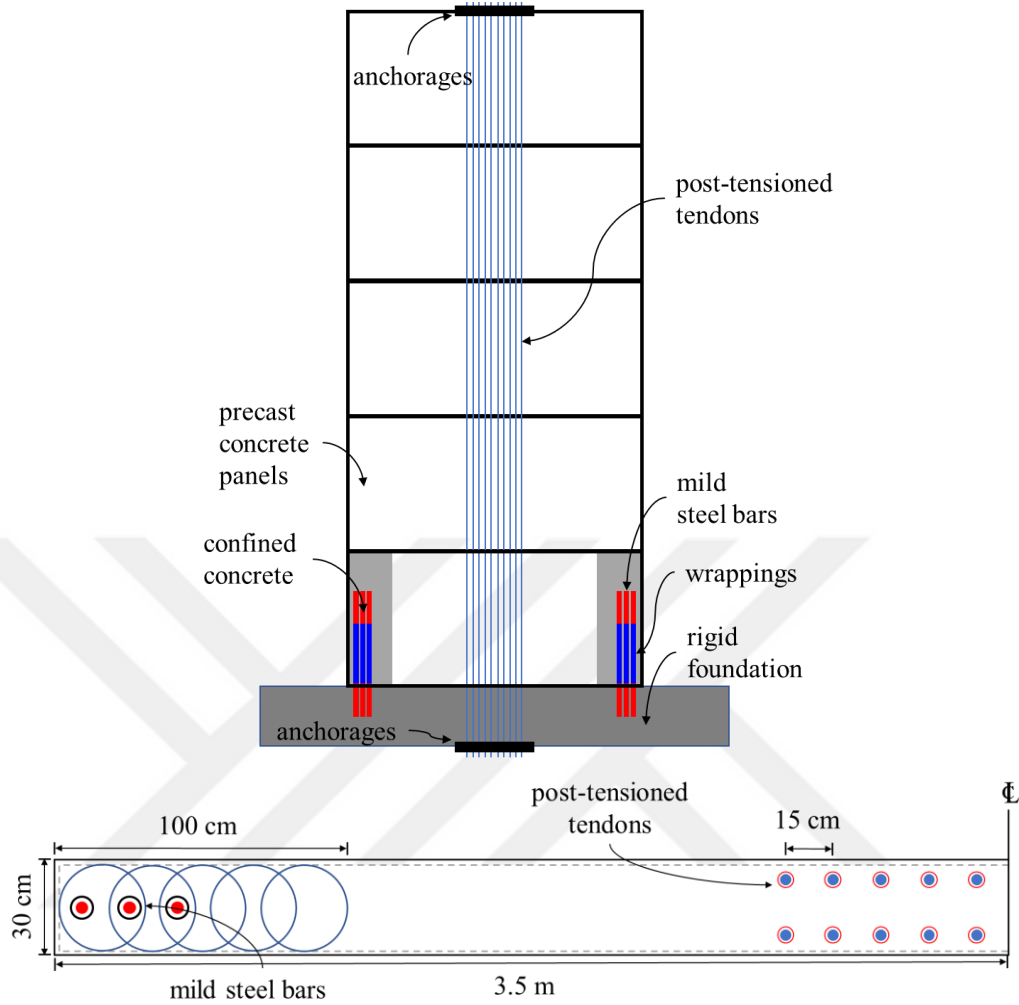


Figure 3-16: Wall with mild steel bars in the vicinity of wall edge

3.16 Unbonded Length of Mild Steel

The unbonded length of the mild steel bar is a significant design parameter on the expected behavior of the wall. As per ACI ITG-5.2 [82], the unbonded length of the mild steel bars needs to be short enough to assure desired wall behavior such as yielding of mild steel bars to dissipate energy and long enough to ensure steel maximum strain is less than 85% of its ultimate strain to prevent bars rupture. The maximum strain of the bars can be estimated using the elongation of the bars since they are left unbonded. For this purpose, the bar elongation is assumed to be equal to gap opening at the same location as shown in Figure 3-17.

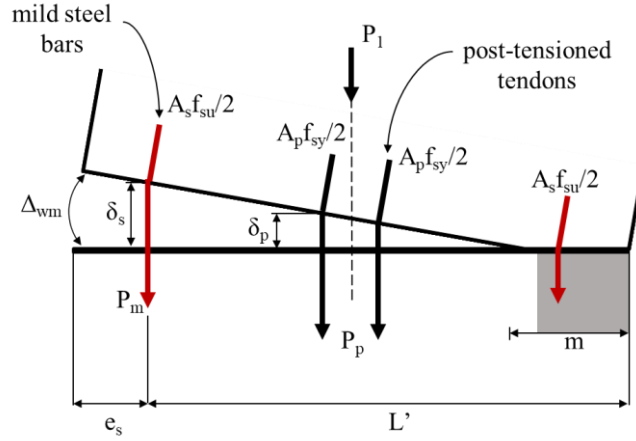


Figure 3-17: Gap opening and free-body-diagram at wall base

Using the expressions developed to predict wall gap opening, the elongation of the bar δ_s can be calculated using the distance from the mild bars to the neutral axis and expected maximum computed wall gap opening at its ends using Eq. 3-47.

$$\delta_s = \Delta_{wm} (L' - m \pm e_s) \quad (3-47)$$

where Δ_{wm} is the wall joint gap opening, L' is the length along the wall joint from mild steel bars venter to the wall rocking edge, m is the concrete compression depth, and e_s is the eccentricity of the mild steel bars from edges of the walls as shown in Figure 3-17. The mild steel bar strain ε_s corresponding to wall gap opening can be determined with the following expression:

$$\varepsilon_s = \frac{\delta_s}{l_s + \alpha_s d_s} \quad (3-48)$$

where δ_s is the elongation of the mild steel bars, l_s is the unbonded length of the mild steel bars, d_s are the diameter of the bar, and α_s is the coefficient for additional wrapping length of the bars that is assumed to be equal to 0 and 2.0 at design level and

maximum level drift, respectively. Using this equation, the unbonded length of the mild steel bars can be properly determined considering allowed maximum bar strain of 85% of bar rupture strain per ACI ITG-5.2 as follow:

$$l_s < \frac{\delta_s}{0.85 \varepsilon_{su}} - \alpha_s d_s \quad (3-49)$$

The final design summary for the mild steel bars is given in Table 3-5. The concrete compression depth is obtained from the analytical investigation of the reference Wall-R. The maximum unbonded lengths of mild steel to ensure ACI ITG-5.2 requirement are determined as 98 cm and 230 cm for bars placed in the vicinity of the wall center and wall edge, respectively. The unbonded length of the mild steel bar at the wall center is assumed 75 cm for all prototype walls. The debonded length of the mild steel reinforcing that is placed at the edge of the wall is assumed 210 cm for Wall-E3. The lengths for the Wall-E1 and E2 are assumed as 150 cm and 180 cm, respectively to investigate the effect of the unbonded length on the seismic performance of the wall.

Table 3-5: Prototype walls and unbonded mild steel used

Wall	Mild Steel Bars		
	Location	Area, A_s (mm ²)	Unbonded Length, l_s (cm)
Wall-C1	center	4800	75
Wall-C2	center	7200	75
Wall-C3	center	9828	75
Wall-C4	center	12000	75
Wall-C5	center	14520	75
Wall-E1	edge	6000	150
Wall-E2	edge	6000	180
Wall-E3	edge	6000	210

CHAPTER IV

ANALYTICAL MODELING AND VALIDATION OF PROTOTYPE SHEAR WALLS

In this chapter, analytical modeling approaches and development of finite element models for the prototype precast concrete hybrid walls that are described in the previous section are presented. For the investigation of the seismic behavior of the prototype precast concrete hybrid walls, the OpenSees program, *The **Open** System for **E**arthquake **E**ngineering **S**imulation* is used to develop a finite element model [41]. The finite element program OpenSees is introduced with its algorithm, and a summary fundamental library for elements, materials, and sections are described. The modeling approach for unbonded post-tensioned precast concrete walls and their structural components is discussed and compared with the studies in the literature. The modeling approach is validated with one experimental and one analytical study. Validation of the precast wall modeling processes and their results are explained individually. Eventually, the developed models for the prototype hybrid walls incorporate unbonded PT steels and mild steels together are presented.

4.1 *OpenSees Finite Element Program*

The OpenSees program is an open-source, C++-based, and object-oriented software for earthquake and geotechnical studies and developed at the University of California Berkeley with funding supported by PEER (Pacific Earthquake Engineering

Research Center), NEES (The Network for Earthquake Engineering Simulations), and NSF (National Science Foundation). The software consists of several classes in which users can create structural or geotechnical models, perform nonlinear analysis, and record the analysis response. The program does not have a user interface, users are able to create an input file that is written in via TCL (Tool Command Language) or Python programming language. However, there are several developed or developing user interfaces like OpenSees Navigator, STKO (Scientific Toolkit for OpenSees) and GiD-OpenSees, etc. enable users to create and monitor models via pre-and post-processors. In Figure 4-1, a basic flow chart for finite element analysis via OpenSees is shown.

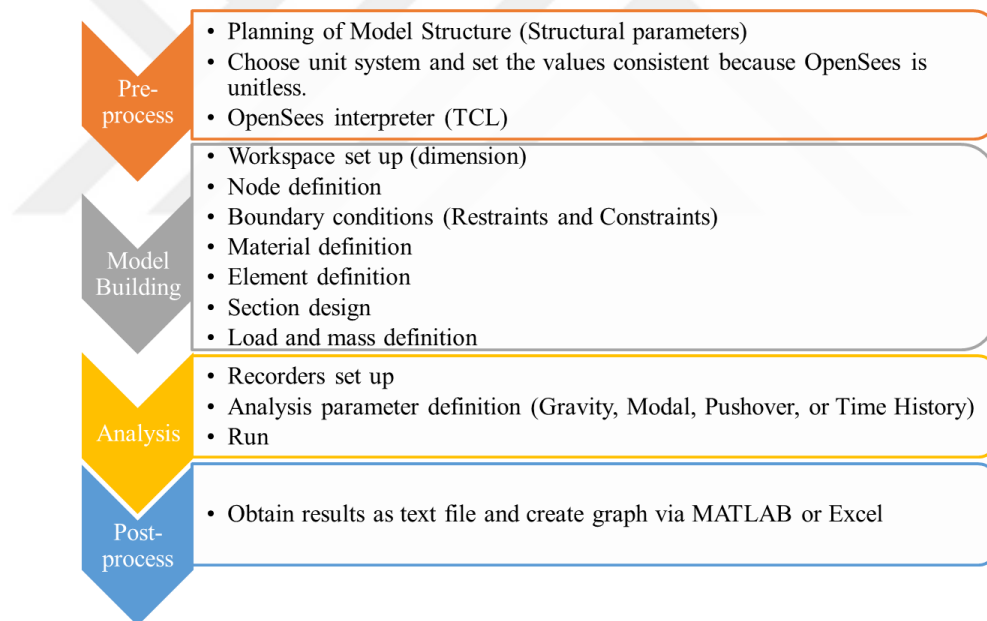


Figure 4-1: A flowchart exhibiting finite element analysis step in OpenSees [41]

OpenSees has been increasingly popular due to its rapidness, cost-effectiveness, being free, and its extensive library compared to other structural analysis programs. The program provides users to create their own functions and contribute or develop the framework. As mentioned before, the C++ programming language is used primarily, but

other programming languages like Fortran, C, or Python are available for development, interference, and contribution to the framework.

4.2 OpenSees Framework

The OpenSees framework comprises of four individual objects such as domain, model builder, analysis, and recorder. In this part, these objects are described with their functions and components briefly.

4.2.1 Domain

This object is the central component of the framework where the data created in the Model Builder object is stored and transferred to analysis and recorder objects. The object keeps the instantaneous case and last defined case of the model at every time increment. The Domain object consists of more than 500 classes and the main classes of the object are presented in Figure 4-2.

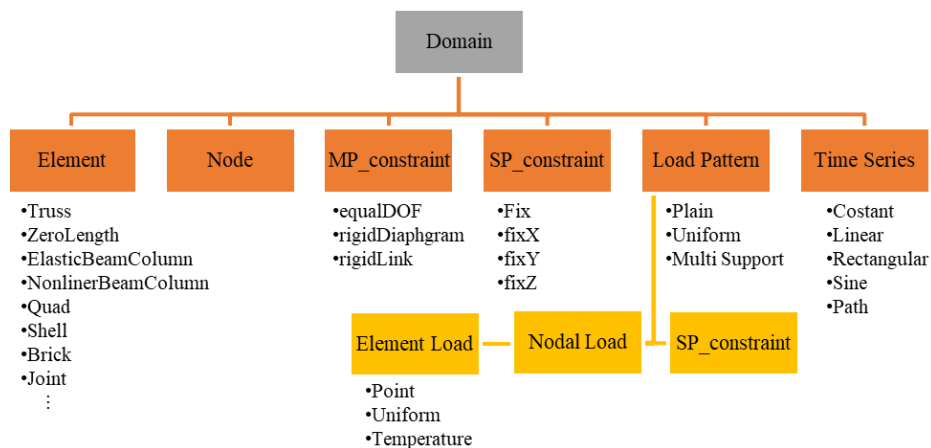


Figure 4-2: Main classes of the domain object

4.2.2 Model Builder

This object is the place where the finite element model of any structure is created with main attributes such as geometry, material, type of element, loads, boundary conditions, transformation, etc. and loaded to the domain. Number of dimensions for workspace is defined herein. It comprises of five main classes and some of the classes are shown in Figure 4-3.

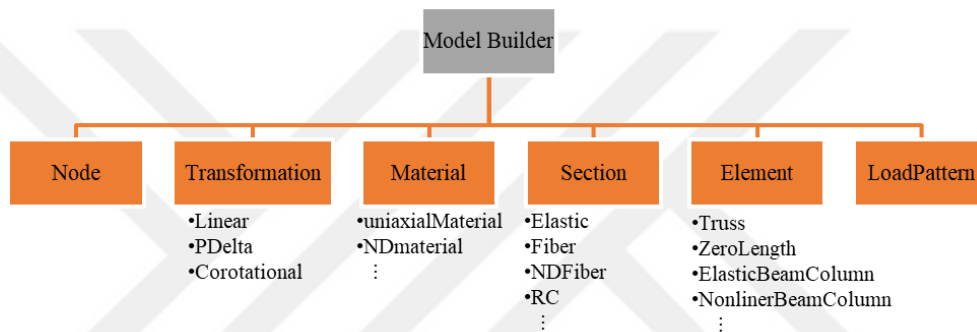


Figure 4-3: Main classes of the model builder object

4.2.3 Analysis

This object is the part of the framework where gravity analysis, eigenvalue analysis, static monotonic and reverse cyclic analysis, and nonlinear time history analysis are performed on the created finite element model. The analysis procedure is established in this object. It iterates the state of the finite element model until it converges by trying the solution method at each time increment. The Analysis object consists of more than 200 classes and the main classes are depicted in Figure 4-4.

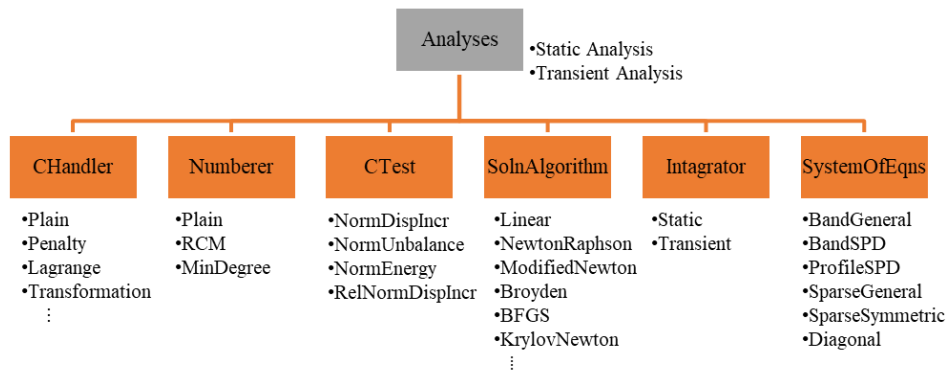


Figure 4-4: Main classes of the analysis object

4.2.4 Recorder

This object is the place where the analysis outputs from the domain can be monitored and stored in a text file. OpenSees does not automatically print output from the analysis, instead, users must request the desired outputs from the program by writing to the interpreter. Then the specified output such as element stress-strain, force-displacement, and nodal displacement history is recorded and written in a text file. Because the OpenSees does not have its own GUI, related envelope curves can be plotted via Excel or MATLAB. The Recorder object contains more than 20 classes in it and some of the main classes are shown in Figure 4-5.

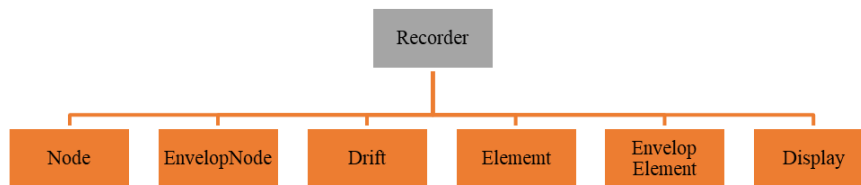


Figure 4-5: Main classes of the recorder object

4.3 Material Library

Material definition is the first step of the finite element modeling of a structure via OpenSees after defining geometry of structure in the workspace. Then, element sections of the model are created and whatever material properties element has, that material properties is assigned to the section. OpenSees material library consists of more than 60 uniaxial materials and more than 30 multi-dimensional materials. In the library, several steel and reinforcing-steel materials, concrete materials, standard uniaxial materials, materials for continuum elements, and some contributed uniaxial and multi-dimensional materials can be found.

Material definition is initiated by constructing *UniaxialMaterial* or *NDMaterial* objects and completed by introducing required arguments which represent stress-strain relationship. Initial modulus of elasticity, yield strength, ultimate strength, yield strain, ultimate strain, and some unique parameters that control the shape and transition of backbone curves are the examples of some of these arguments. Unlike other analysis programs that have a graphical user interface, the skeleton curve for a material stress-strain relationship could not be created specifically. Therefore, users must use either existing material models or the models created via C, C++, or Fortran languages.

4.4 Section Library

Prior to creating elements for models, a cross-section of the element must be defined to associate the element with material properties. Section command is used to create a section force-deformation object which describes the consequent stress-strain relationship at a cross-section of an element. In the OpenSees section library, there are

several types of sections for beam-column elements, fiber elements, membranes, plates, and elastomeric bearings. The types of sections can be briefly described as,

- Elastic Section can be used for linear elastic moment-curvature analysis or development of a complex model as the initial stage. It is created by geometric parameters and material constant such as modulus of elasticity, moment of inertia, and shear modulus.
- Uniaxial Section is created to represent uncoupled section stress-strain response quantities such as axial force-deformation, moment-curvature, shear force-deformation, or torsion force-deformation.
- Fiber Section is constructed to specify a cross-section of beam-column element that has a geometric configuration consisting of regular subregions. The cross-section of elements is equipartitioned into the fibers in which uniaxial or multi-dimensional materials can be identified separately. This type of section makes it possible to combine stress-strain relationships of different materials into a fiber section where P-d – M-k interactions and flexural – axial stiffness are computed by integrating the deformation throughout the section. Section geometry is specified via a series of three commands. If a single fiber is to be created, *fiber* command is used by specifying area, material type, and coordinates in the section considering the local coordinate. In the generation of the object, uniaxial materials which require Bernoulli beam assumption are associated. If a cross-sectional area forming by number of fibers is desired, *patch* command enables users to create rectangle, quadrilateral, or circular cross-section with associated material by specifying location in local coordinate system and number of subdivisions. Like the patch command, layer command is used to create a number of fibers along a

straight or circular line with a single command. However, layer command is more suitable for creating reinforcing bars in cross-section of beam-column elements.

In addition to these sections, *elastic membrane plate section* and *plate fiber section* can be used for shell or plate analysis. A uniaxial elasto-plastic model with linear hardening shall be generalized two-dimensionally using *bidirectional* section. *Isolator2spring* section can be used to present buckling and resultant force-deformation behavior of an elastomeric bearing. In addition, there are contributed special sections for reinforced concrete element and wide flange steel such as *RCSection2d* and *WFSection2d* in which dimension of the section can be obtained from AISC steel manual directly.

OpenSees incorporates a *sectionAggregator* object that enables users to combine existing uniaxial material or previously defined sections into a single section stress-strain relationship by defining a new section recursively. At the beginning, this object has been created to aggregate shear behavior with existing forced-based beam-column elements. However, any desired relationship can be coupled with another material force-deformation behavior or sections. Although its accuracy is not as high as material level coupling, it gives sufficient behavior. Only one pre-defined section can be included. Each material stress-strain relationship can be associated with a particular degree-of-freedom. There is no interaction between the individual degrees-of-freedom.

4.5 Element Library

OpenSees contains many kinds of own and contributed elements that users can use to model various RC, steel, or any kind of structures including frame systems, infill wall systems, cable systems, bridge structures, plate structures, and soil layers. Besides

the main structural elements, there are several elements for more specific structures such as dampers, isolators, etc. and for modeling nontrivial behavior like bar slip, shear friction, etc. In this part, some of the usual element types are briefly discussed.

Zero length elements are the elements that are defined by two individual nodes with the same coordinates. Uniaxial and multi-dimensional material stress-strain relationships can be assigned for each degree of freedom. These type elements are preferred to simulate the interaction behavior between elements or soil and structure i.e., wall to floor connection, element to foundation connection, or reinforcing bars to concrete connection (bar-slip). If a fiber beam-column element connection is modeling, a *zero-length section* element that has the same section and material properties can be used to simulate plastic hinge behavior at the ends. If the *NDmaterial* is used, a *zero-length ND element* must be selected. There are also zero-length elements that can be used to model node to node and node to segment frictional contact using Mohr-Column frictional law.

Truss Elements are the elements that possess force-deformation behavior relationships only in the longitudinal direction. Therefore, only axial loads are resisted, and transverse forces do not affect the element. These element types can be used to simulate bar elements, or damping elements, etc. The element section requires only area parameters, or a pre-created section can be selected. Loads applied as either lumped at the ends or distributed along the element length. Mass and weight can be attributed for eigenvalue and dynamic analysis. Rayleigh damping and consistent mass matrix can be assumed. There are two types of truss elements namely, *truss element* and *corotational truss element* that exclude and include geometric nonlinearities, respectively.

Beam-column Elements can be used for RC and steel structural elements such as columns, beams, shear walls, bridge deck, piers, etc. The elements are based on plane section remain plane assumption. The program has nine different type beam-column elements. *Elastic beam-column elements* are first order elements that use linear elastic theory. In this element type, geometric nonlinearity is not regarded. Though the element captures sufficient behavior for various model, imprecise result can be seen when torsional effect is included. Nonlinear behavior can be supplied by lumped plasticity at both ends of the element with linear or rotational springs. If the element is modeled with stiffness proportional damping, *elastic beam-column element with stiffness modifiers* can be selected. If the shear deformation along the element is considered, then *elastic Timoshenko beam-column element* is the option. The elastic elements do not require material assignment instead, only cross-sectional properties such as area, stiffness, young's modulus, shear modulus, second and torsional moment of inertia.

There are also the nonlinear beam-column elements which consist of mainly force-based and displacement-based beam-column elements. *Force-based beam-column element* is a fiber element that can be modeled with either lumped or distributed plasticity. *Beam with hinges element* is a force-based beam-column element when the plasticity is considered as lumped at the ends. In force-based elements, it is assumed that applied force is constant, and the moment distribution is linear along the element. Force distribution along the element is determined more precisely. The element reposes on the Euler-Bernoulli beam theory. Linear geometry is considered hence, plane section remains plain and perpendicular to the local longitudinal axis under deformation. Some concrete failures such as shear, cracking, and bond slip have no effect on the elements. These elements require a pre-defined fiber section. Shear effect can be included with *section*

aggregators. The elements use default Gauss-Lobatto integration for the determination of element state, but several integration options are also available. It is advised that one story should be modeled with one force-based element with several integration points with finer mesh sizes.

Because OpenSees is based on the displacement-based analysis approach, it is easy to obtain an exact structural response by using a *displacement-based beam-column element*. In displacement-based elements, it is assumed that the axial strain is constant, and the curvature distribution is linear along the element. Shear, cracking, and bond slip effects are considered. Because these are prismatic elements, and the plane section remains plane and perpendicular to the local longitudinal axis under deformation like for the force-based elements. If the shear effect is desired to be included, plastic hinges should be defined separately. The elements use default Gauss-Legendre integration for the determination of element state, but several integration options are also available. Fiber section assignment is required for the element representation. It is suggested that one story should be modeled with more than one displacement-based element with less integration point with finer mesh size to get a more accurate response.

There are also *Contributed Beam-column Elements* especially for shear wall structures. If a shear wall is desired to model with displacement-beam column and flexural and shear interaction is considered, flexure-shear interaction displacement-based beam-column element can be used. The element definition and requirements are similar with displacement-based elements; however, the main differences are section and transformation definition and a new parameter for the center of rotation. Only linear transformation can be defined, and individual fiber section composed of equal length strips along the length of cross-section is required. There also sophisticated shear wall

elements models namely multiple vertical line element model (*MVLEM*) and cyclic shear-flexure interaction model for RC walls (*SFI-MVLEM*) in which axial-flexural behavior of the wall are exhibited by series of uniaxial elements that echeloned by linked with a rigid beam and shear behavior is simulated with a shear spring.

Besides the fundamental elements, the element library incorporates joint elements, link elements, bearing elements, quadrilateral elements like plates and shells, triangular elements, brick elements, tetrahedron elements, contact elements, cable elements, and some complicated isoparametric elements for porous medium, soil layers, and shallow foundation, etc.

4.6 Modeling Approach for Unbonded Post-tensioned Walls

In this section, the finite element modeling approach employed in this study for the seismic performance of prototype walls is discussed. The modeling approach developed is based on models developed for similar walls by Kurama [4] [11], Perez [71] [36] [38], Smith and Kurama [39] [13] and Erkmen [84] [67]. The model developed using OpenSees is given in Figure 4-6. The precast concrete panels are modeled with fiber beam column elements. Both unbonded post-tensioning tendons and unbonded mild steel bars are modeled with truss elements. At the bottom end of truss elements for PT tendons, linear springs are added to prevent the post-tensioned bars from going under compression, which is consistent with the actual behavior. Top end of the tendons and mild steel bars are kinematically constrained to the fiber beam-column element by creating a rigid beam to model bars end anchorage and ensure displacement compatibility between the bars and concrete. The foundation is assumed to be rigid, and fiber elements, mild steel bars, and

linear springs are fixed to the foundation to model their end anchorage. The developed model is based on the following assumptions:

- The foundation is assumed to be fully fixed and soil-structure interactions are ignored.
- Second order effects are not included.
- Masses are lumped at floor levels.
- Shear slip between wall panels is ignored and assumed to be prevented by design.
- At the end anchorages, perfect bonding is assumed, and no bar slip is expected.
- Concrete panel shear behavior is assumed to be elastic.

A detailed description of the modeling approach and element selected for each structural member is discussed in the following sections.

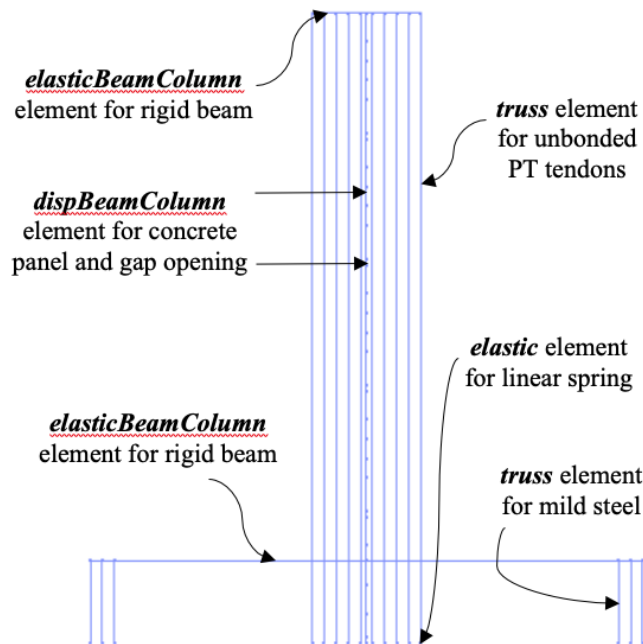


Figure 4-6: Modeling details of a study wall displayed from OpenSees

4.7 Modeling of Concrete Wall Panels

The concrete wall panels are modeled with beam-column elements. The cross-section of each element is composed of several fiber strips along the wall length. Figure 4-7 shows the fiber element cross-section for the base panel. As shown in the figure, finer strips are used for wall ends to obtain more precise stress-strain behavior while relatively larger strips are used in the vicinity of the wall vertical centerline. Neither mild steel bars nor tendons nor wire meshes are incorporated into the fiber cross-section since mild steel bars and PT tendons are unbonded, and they are modeled as separate elements. Also, spiral confinements and wire meshes are not modeled since the confinement effects of spiral steel on concrete behavior is modeled for confined concrete.

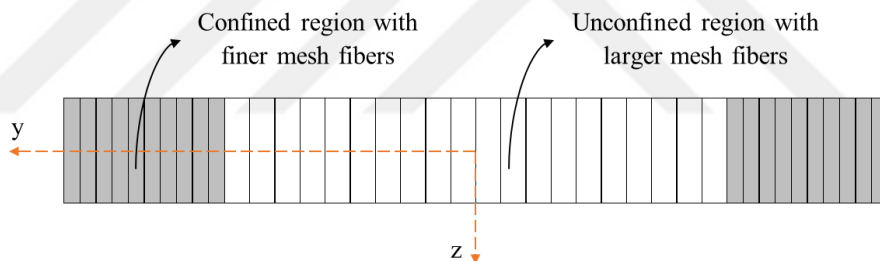


Figure 4-7: Cross-section modeling detail of the shear wall

Each fiber strip is defined with an area and location in the local coordinate system and has a uniaxial material stress-strain relationship assigned. *Concrete04* uniaxial material in which the envelope stress-strain relationship follows the Popovics model [85] is used to model concrete axial-flexural behavior. Spiral confinements are provided only at the base panel ends. Therefore, the confined concrete stress-strain relationship for these regions is determined based on the confined concrete model proposed by Mander et al. [79] as shown in Figure 4-8. Although Mander's concrete model uses the model proposed by Popovics, certain differences are seen between the expected and obtained behavior of the concrete material models. Mander's concrete model can only be created with

concrete04 directly, if the Modulus of elasticity for concrete, E is set to $4700\sqrt{f'_c}$ (in MPa) in the definition of *concrete04* material. By this means, the same backbone curve as in Mander's model can be obtained.

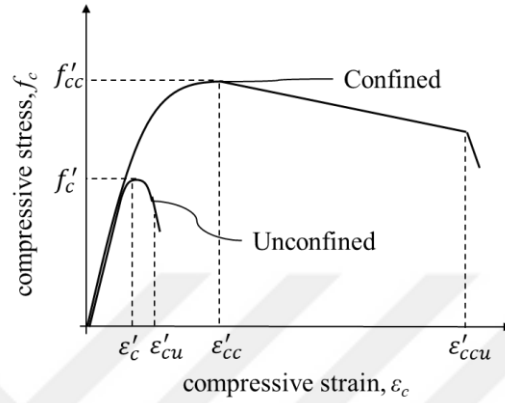


Figure 4-8: Stress-strain relationship of unconfined and confined concrete [79]

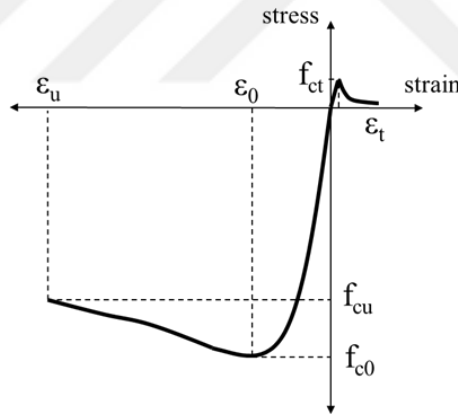


Figure 4-9: Stress-strain curve for concrete panels (Concrete04)

Figure 4-9 shows the stress–strain curve for both confined and unconfined *Concrete04* material in OpenSees. The material is created by defining input parameters like compressive strength, strain, crushing strain, initial stiffness, maximum tensile strength, and strain at that point. The tensile strength of the concrete, f_t is assumed to be $0.62\sqrt{f'_c}$. The tensile strength after the maximum value decreases exponentially in the

model. Also, a *beta* parameter (for exponential curve) can be optionally defined to specify residual stress. *Concrete04* material model uses Karsan-Jirsa behavior model for the determination of slope at any unloading and reloading step [86].

4.8 Modeling of Gap Opening

Properly modeling the wall joint gap opening is an important modeling feature since it controls lateral response of the wall. OpenSees provides a number of elements to model gap opening such as gap materials, zero-length gap elements, or a series of parallel linear compression only springs having concrete material properties in compression can be placed between precast concrete panels. In this study, the gap opening between the wall joints is modeled by using relatively small fiber elements having the same cross-sectional layout but different material properties. Unlike concrete panels, *concrete01* material, which has zero tension, is used to model gap opening. Modeling the gap opening between wall panels with zero tension concrete fibers has been successfully implemented and used by a number of researchers [84] [67] [4] [11] [39] [13].

Figure 4-10 shows the backbone curve for concrete with zero tensile strength material. The *Concrete01* material can be created by inputting only compressive concrete parameters. As in the concrete panel modeling, confined concrete is attributed to fibers at the edges by following Mander's model.

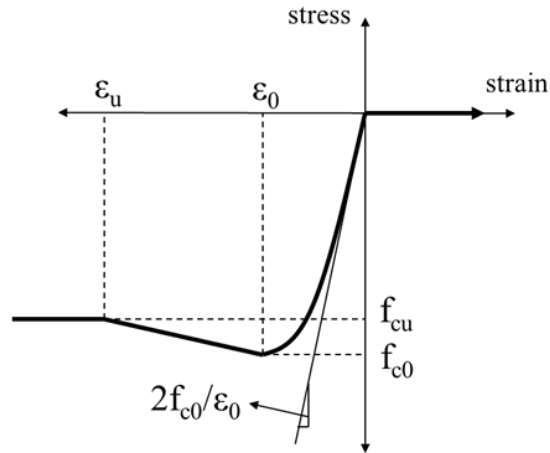


Figure 4-10: Stress-strain curve for concrete without tensile strength (Concrete01)

Significant gap opening is expected to occur between the foundation and base wall panels, and relatively small gap opening can take place between upper panels due to higher modes effects under seismic loads. Therefore, concrete with zero tensile strength material (i.e., *Concrete01*) is also placed between all panels but these fiber elements possess only unconfined concrete material properties. Consequently, four different concrete materials namely unconfined concrete *Concrete04*, confined concrete *Concrete04*, and unconfined concrete with zero tension *Concrete01*, and confined concrete with zero tension *Concrete01* are created and assigned relevant fiber elements. Figure 4-11 exhibits the gap opening behavior under lateral loads and the concrete materials used assigned to concrete fiber beam-column element as well as fiber element for gap opening.

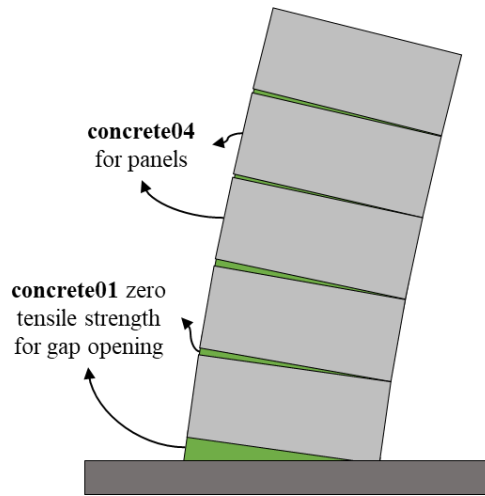


Figure 4-11: Modeling of the concrete materials and gap opening behavior

4.9 Modeling of Unbonded Post-tensioned Tendons

Because the unbonded post-tensioned tendons are unbonded with concrete there are no contact between the tendons and concrete panels, the tendons are modeled using truss elements for each bar. In total, there are 20 tendons in two lines for the prototype walls. Because the developed model is 2D, the tendons located at the same location along the wall are modeled as a single truss element. The truss elements are only connected to the wall fibers at roof level by using an elastic beam to slave the degrees of freedom for the tendons and wall panels at the roof. The end anchorage at the foundation level is modeled by fixing the truss end nodes for all degrees of freedom.

Material property *Steel02* (Giuffre-Menegotto-Pinto steel material) is selected for the truss elements. The material backbone curve is shown in Figure 4-12, and the required parameters are taken from the tendon material stress-strain relationship given in Figure 3-5. The yield strength f_y , ultimate strength f_u , and the linear elastic modulus E_0 are defined as 903 MPa, 1125 MPa, and 195.8 GPa, respectively. The strain value at ultimate

strength is estimated by using a strain hardening ratio b of 0.015. The yield strain ϵ_y and ultimate strain ϵ_u are 0.0046 and 0.08, respectively.

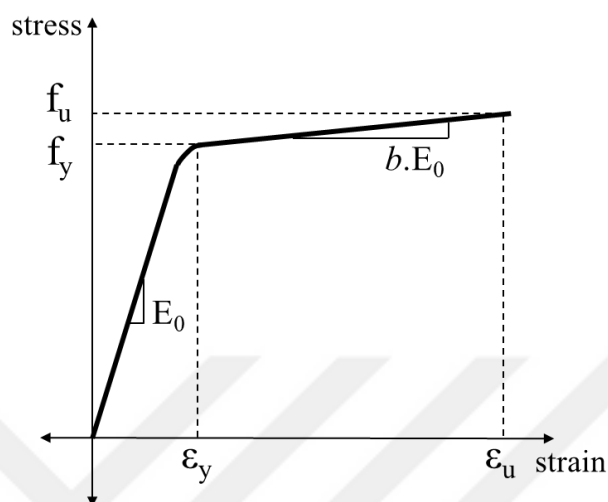


Figure 4-12: Stress-strain curve for PT and mild steel bars (Steel02)

The post-tensioning for tendons is modeled by introducing an initial internal force which is uniformly distributed along the tendon length. The initial stress is predetermined as 75 % of the yield stress, which corresponds to a stress level of 677.25 MPa. However, it is observed that the specified initial post-tensioning stress decreases before the analysis due to shortening of the wall under both gravity force and post-tensioning force. Therefore, the difference on the defined post-tensioning stress level is noted after the gravity analysis and that is added to the initial post-tensioning level to achieve design post-tensioning level for the tendons. For this purpose, the initial post-tensioning stress is increased to 736.4 MPa to achieve an effective post-tensioning stress of 677.25 MPa after gravity loads are applied.

The rigid beam used at the roof level to constrain degrees of freedom for the truss elements and wall is defined with an *ElasticBeamColumn* element. The elastic material

properties for the element are defined so that the element deformation and bending is negligible.

To prevent post-tensioned tendons from carrying any compression force upon unloading and buckling, zero-length bilinear elastic spring elements carrying only tensile force are placed between the tendon ends and the foundation. The nodes that tendons and springs are connected are constrained horizontally and rotationally via the *equalDOF* command to restrain spring to work only in the vertical direction. Figure 4-13 shows the material behavior for the elastic material, which requires only Modulus of Elasticity for tension and compression. Relatively higher tensile modulus of elasticity (i.e., 200 GPa) is selected to minimize spring elongation under the tendon tension forces, and to prevent compression forces the tension modulus of elasticity is taken as zero.

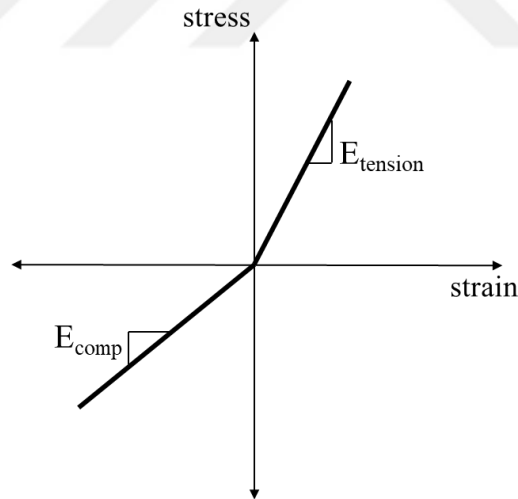


Figure 4-13: Tension only springs (elastic element with $E_{comp}=0$) stress-strain

4.10 Modeling of Mild Steel Bars

The mild steel bars are also unbonded to achieve uniform strain distribution along their length and to delay their yielding. The bars are modeled with truss elements. The bottom end of the bars is fixed to the foundation while the other end of the unbonded bar is embedded into the concrete wall panel. The unbonded length is wrapped with proper material in a way to prevent bond development between the bars and surrounding concrete. To constrain degrees of freedom for the unbonded mild steel and surrounding concrete at the location where the steel is embedded into the concrete, an elastic beam-column element is defined to constrain the degrees of freedom.

The stress-strain relationship for the mild steel bars is also given in Figure 3-5. *Steel02* material with corresponding material properties is assigned to the truss elements representing the mild steel bars as in Figure 4-12. Initial modulus of elasticity E_0 , yield strength, f_y , and ultimate strength, f_u are defined as 138.3 GPa, 415 MPa, and 615 MPa, respectively. The strain hardening ratio b is taken as 0.16. Computed material yield and ultimate strain limits ε_y and ε_u are 0.003 and 0.012, respectively. The same material properties are used for all walls. Differently from the tendons, mild steel bars are modeled to yield under both tension and compression. In other words, no tension-only springs are used for the unbonded mild steel bars.

4.11 Modeling and Validation of PTT Wall from NIST Experiment

The developed analytical modeling approach for precast wall systems with unbonded post tensioned strands is verified with experimental results of a wall specimen called PTT wall, which was tested by Schultz et al. (1998). The wall sample was 3.2 m

by 2.0 m precast concrete panel representing the first two floors of 2/3 scale prototype shear wall. In total six 21.6 mm diameter unbonded tendons, which were uniformly distributed with 0.5 m center-to-center spacing, were used. Spiral reinforcements were placed at wall ends, where high compressive strains are expected to occur due to wall rocking motion. The concrete compressive strength was 35 MPa and post-tensioned steel young's modulus, yield, and ultimate strength of the post-tensioning steel were 196 GPa, 903 MPa and 1123 MPa, respectively.

Figure 4-14 shows a computational model of the PTT wall developed in OpenSees to verify the modeling approach discussed in previous sections. The concrete panel is model with six *dispBeamColumn* fiber elements each having five integration points. The cross-section of the panel is created with *Rectangular Patch* command by defining confined concrete for the edges and unconfined concrete for the middle part. Spiral reinforcement is not modeled explicitly, but Mander's constitutive law for the stress-strain relationship of confined concrete is used instead. While *Concrete04* material is selected for concrete panels, *Concrete01* zero tension concrete material is used for the concrete fiber elements representing the gap opening at the bottom of the wall panel. The post-tensioned tendons are modeled with truss elements using *Steel02* material. The post-tensioning force is defined to the tendon element as internal compressional loads. A rigid beam at the top of the wall is created with an *ElasticBeamColumn* element, then the tendons are anchored to the rigid beam and foundation. Bottom nodes of the tendons and the concrete fiber beam-column element are fixed to the foundation. Finally, *zero length* linear spring elements with bilinear *elastic* material are placed between the foundation and truss ends to prevent tendons from carrying any compression force.

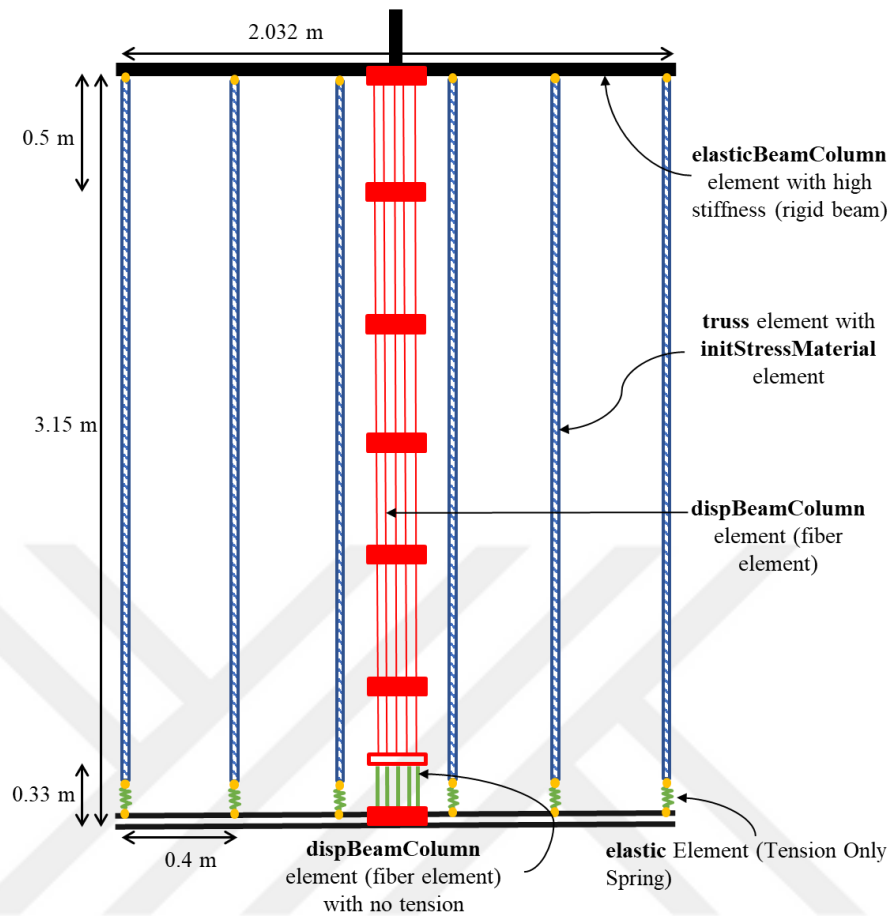


Figure 4-14: Modeling details of the PTT wall tested by Schultz et al. [30]

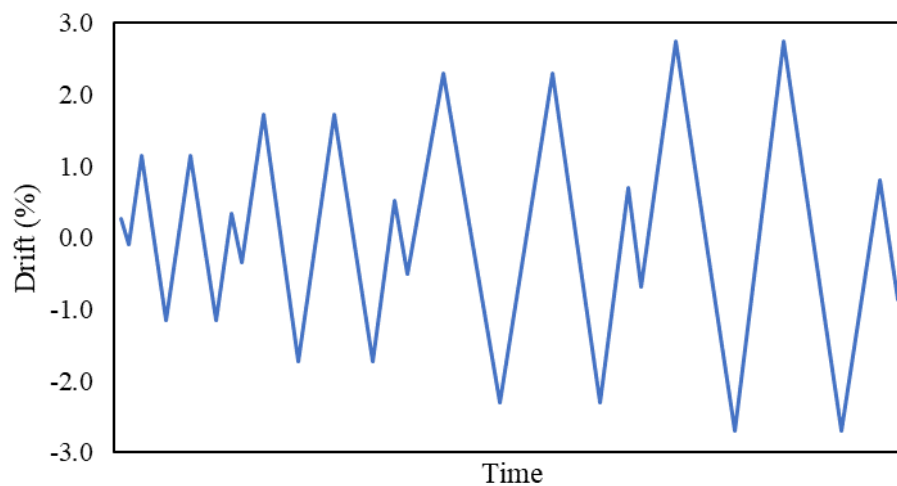


Figure 4-15: Reverse cyclic roof drift history applied to PTT wall

In the experiment, the test wall was subjected to 213.5 kN axial load and quasi static lateral loading by an actuator. Also, an overturning moment was applied to the top

of the wall to represent the continuity moment at the second-floor level. The drift history given in Figure 4-15 was applied to the specimen and lateral force – drift relationship and tendon forces were monitored during the test.

In the developed model, a relatively small rigid beam-column element having nearly half of the actuator beam height is placed at the top of the wall where both axial and horizontal loads will be applied. An overturning moment as much as the wall height and lateral drift history was defined and applied to the top node including 213.5 kN axial load. Base-shear force, tendon forces, and top displacement are recorded during the analysis. Figure 4-16 and 4-17 shows the comparison of the experimental and analysis results for the lateral force-drift relationship of the wall and third bar from the left force-drift relationship. The OpenSees analysis results show that the developed model is adequate to predict wall lateral load resisting capacity, stiffness, cumulative energy dissipation, self-centering, and residual displacements. The stiffness of the first loading loops obtained from the model was slightly higher, but this difference is negligible. It has been observed that the tendon force-drift curve predicted by the model is slightly more rigid than that obtained from the experiment, and more stress occurred in the tendon than that obtained from the experiment. However, the overall behavior of tendon stress computed is in good agreement with that experimentally measured.

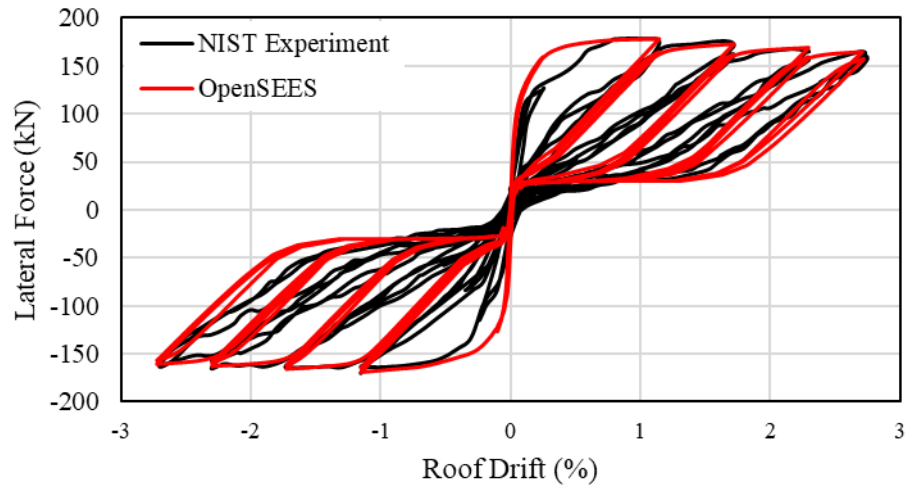


Figure 4-16: Comparison of cyclic force–drift relationship for PTT wall

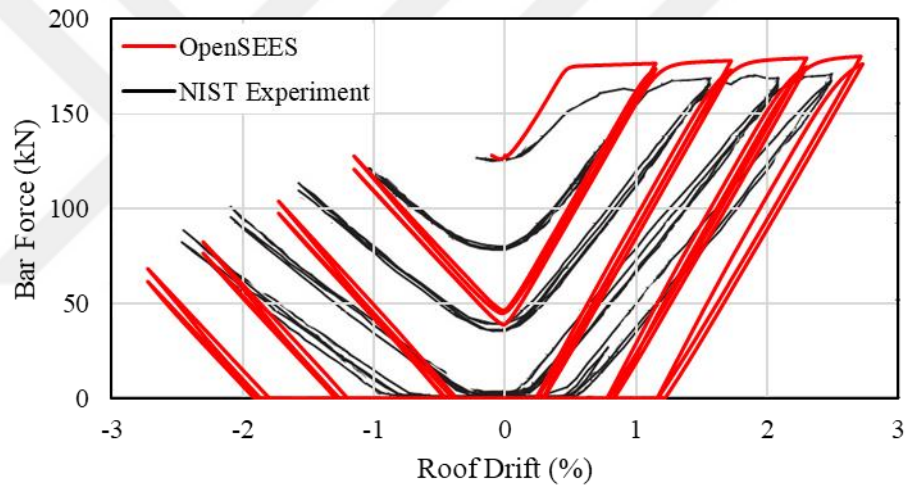


Figure 4-17: Comparison of bar forces for 3rd tendon from left

4.12 Modeling and Validation of 5-Story Unbonded Precast Shear Walls

A second OpenSees modeling approach verification for unbonded post-tensioned wall is done using data published for a 5-story unbonded post-tensioned precast shear wall, namely Wall-C from the numeric study by Erkmen [67]. The prototype Wall-C was 16 m high with a 7 m by 0.3 m cross-section. In total 20 post-tensioned tendons with a total 290 cm² post-tensioned steel area were located in the vicinity of the wall vertical centerline. The tendons were unbonded, and they were only anchored at the foundation

and roof levels. The yield and ultimate strength of the tendons were 903 MPa and 1125 MPa, respectively. The tendons were post-tensioned to 75% of their yield stress, which corresponds to approximately 19445 kN. The unconfined and confined concrete compressive strength for the wall panels were 40 MPa and 75 MPa, respectively. The two end regions of the base panel were confined with spiral reinforcement to prevent concrete premature failure due to wall rocking motion. The analytical model for the prototype wall was created and subjected to static pushover load, reverse cyclic pushover load, and nonlinear dynamic loads using the Drain-2DX program.

For validation of the prototype Wall-C model, the same wall is modeled in OpenSees with the same modeling approach discussed. Figure 4-18 shows the analytical model developed for Wall-C. Truss element is used to model post-tensioned tendons with *Steel02* material model, then post-tensioning stress is applied with *initStress* material. At the bottom of the truss element, *zerolength* elements with elastic material which simulates springs working in tension only are placed to prevent the truss element from exposing to axial load. *ElasticBeamcolumn* element is used to create a rigid beam to anchor the bars to the roof of the wall. The concrete panels are created with the fiber elements, namely *dispbeamcolumn* element, which is a nonlinear beam-column element with *Concrete04* material which uses the Mander's constitutive model for both unconfined and confined concrete material. Relatively small fiber elements are placed at the bottom of each story to model opening gaps along the joints. For the fiber element modeling the gap opening *Concrete01* element is selected for this fiber element, and tension is set to zero to ensure that the fiber element does not carry any tensile stress to model opening gap.

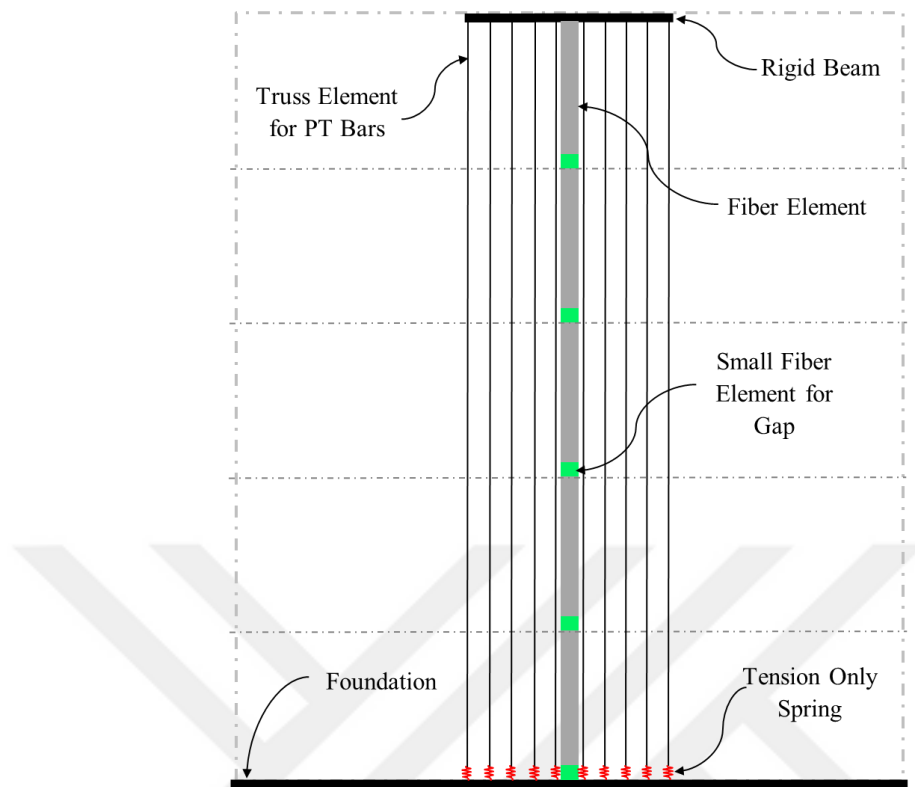


Figure 4-18: Modeling details of the Wall-C by Erkmen [67]

Initially, the created wall model is subjected to nonlinear static pushover load then, reverse cyclic drift history up to 2.5% drift. The base shear forces and top node displacement are monitored during the analysis. At the end of the analysis, monotonic and cyclic base shear force–drift relationships are obtained to compare with the results of Wall-C in terms of lateral load resisting capacity, hysteretic response, total prestressing force, cumulative dissipated energy, and self-centering behavior.

Figures 4-19, through Figure 4-21 show the comparison of the pushover curves, cyclic base shear force–drift behavior, and total post-tensioning forces, respectively. Pushover curves obtained from OpenSees modeling match well with the results reported. Hysteresis loops display similar shape, but minor discrepancies exist between the two modeling approaches. The differences are likely due to small differences in cyclic drift history applied to the wall. In addition, total post-tensioning force obtained from

OpenSees is very similar to that obtained for Wall-C until drift of 1%, but at large drifts the results diverge slightly. However, the results obtained with OpenSees are in good agreement with the results published in terms of base shear force–drift, dissipated energy, lateral load capacity, total post-tensioning force, self-centering behavior, and permanent displacement. In summary, the presented results show that the unbonded post tensioned precast wall can be successfully modeled with OpenSees using the modeling approach discussed.

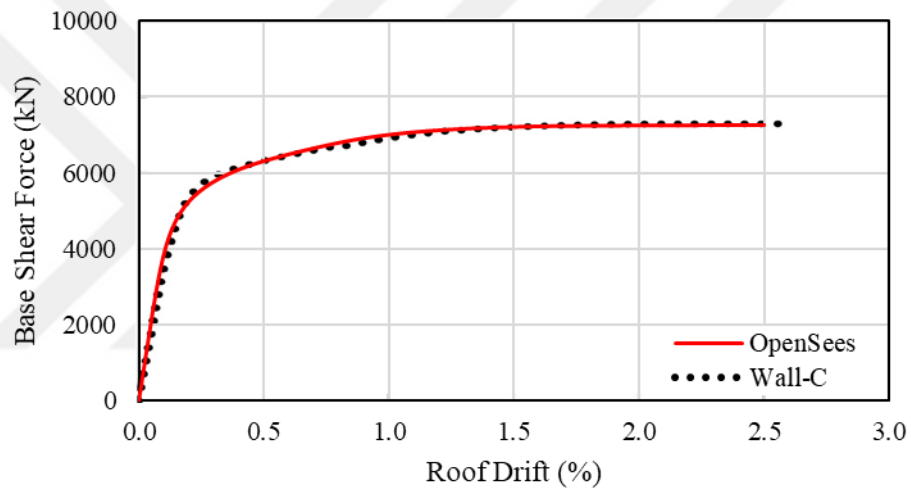


Figure 4-19: Pushover curves validation for Wall-C model

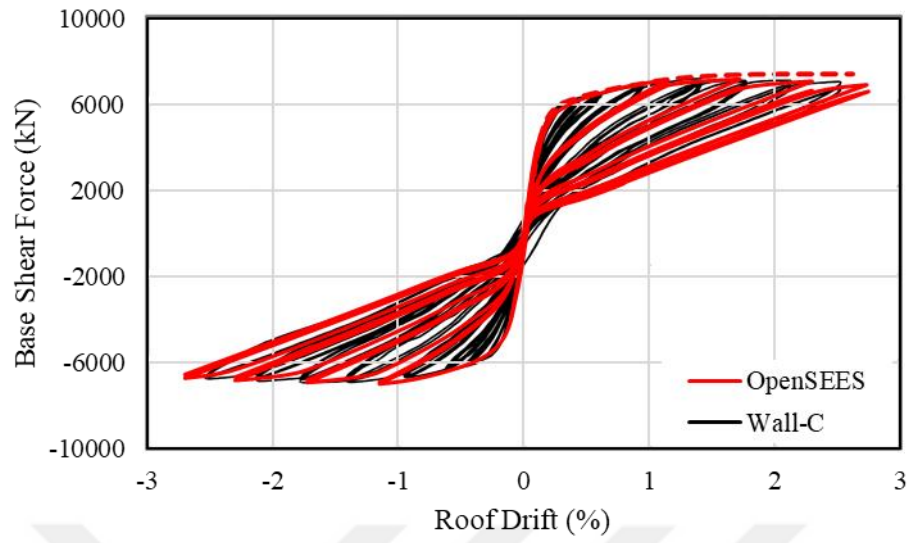


Figure 4-20: Cyclic base shear–drift response validation for Wall-C model

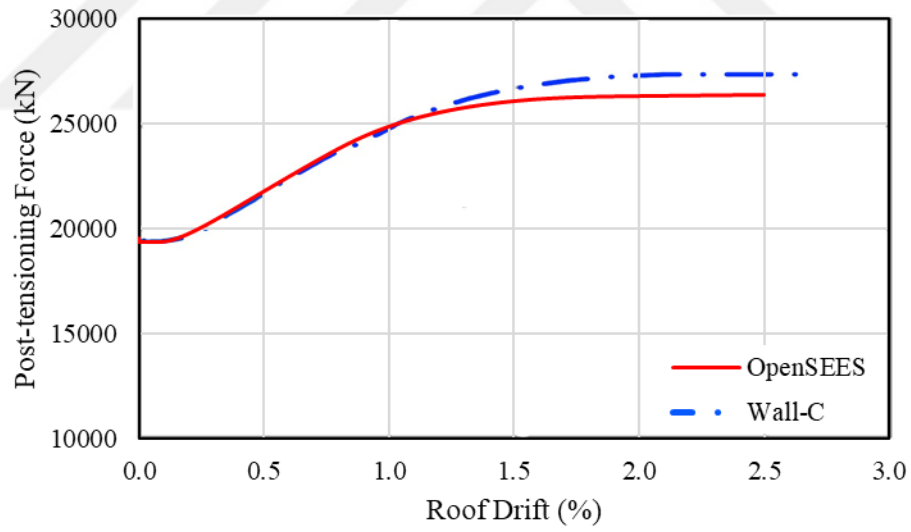


Figure 4-21: Total post-tensioning force–drift validation for Wall-C model

4.13 Modeling of Prototype Hybrid Walls

The verified OpenSees modeling approach is used to develop finite element models for five story prototype building. Models are developed for the prototype unbonded post-tensioned Wall-R and several hybrid walls to investigate effects of unbonded mild steel on walls seismic performance. Mild steel bars are placed in the wall centerline to study effect of mild steel amount, and in the wall edges to study effect of mild steel layout on the seismic behavior of precast concrete shear walls.

Figures 4-22 and Figure 4-23 show the developed wall models in which mild steel bars located in the vicinity of wall vertical centerline (Wall-C1, Wall-C2, Wall-C3, Wall-C4, and Wall-C5) and at the wall ends (Wall-E1, Wall-E2, and Wall-E3), respectively. The developed finite element models for all walls are very similar. The main difference between the walls is mild steel layout, amount, and unbonded length. Both mild steel and unbonded tendons are modeled using truss elements with *Steel02* material stress-strain relationship. At the end of the unbonded length of mild steel, an *ElasticBeamColumn* element is used and connected to the wall node to satisfy displacement compatibility between the wall panel and mild steel. The bonded mild steel length embedded into concrete and provided beyond unbonded length to ensure bar development is not explicitly modeled. Instead, the deformation of embedded bar length is neglected. In other words, it is assumed that embedded length of mild steel is long enough and larger than bar development length to ensure that the bars yield without slip. In addition, any slip or bond stress degradation of mild steel bars under cyclic load is neglected.

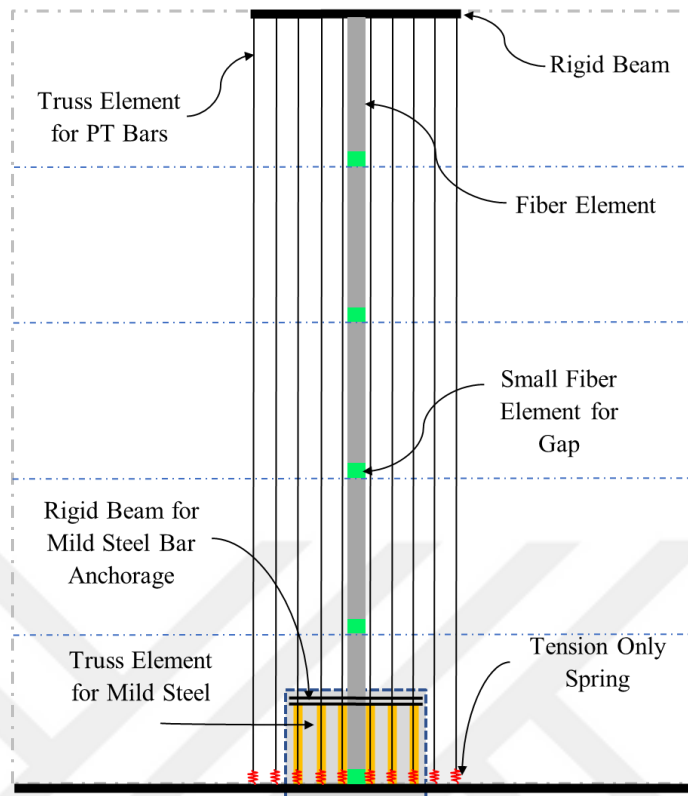


Figure 4-22: Model for prototype wall with mild steel at wall center

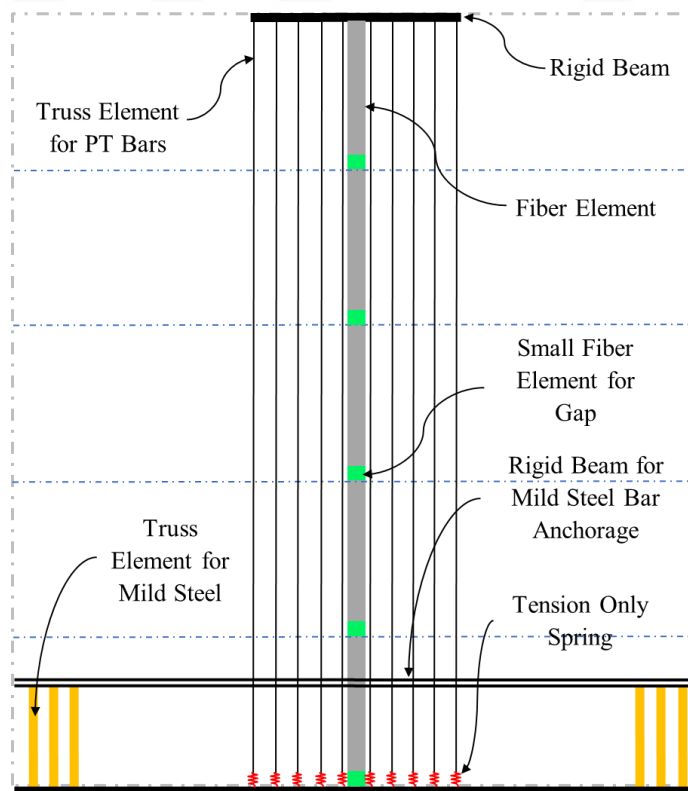


Figure 4-23: Model for prototype wall with mild steel at wall ends

CHAPTER V

SEISMIC BEHAVIOR OF PROTOTYPE WALLS

In this chapter seismic performance of the prototype walls is evaluated using nonlinear static analysis, cyclic reversal pushover analysis, eigenvalue analysis, and time history analysis. The pushover and cyclic pushover analyses are important to develop a better understanding of wall performance and compare different walls performance. The time history analyses are also important to verify walls performance under actual recorded earthquake loads. The behavior of walls is presented in terms of wall lateral force versus roof drift, dissipated energy, and change in post-tensioning force.

For both static and dynamic analyses vertical loads are applied as initial loads at the static analysis step. Vertical loads include dead loads, live loads, and post-tensioning forces. Dead and live loads are introduced to the program as plain load pattern where the whole gravity loads are allocated equally per each floor and assigned as point load to the nodes that represent the floor levels in the direction of the gravity. The post-tensioning loads are applied as internal loads for the tendons.

The lateral loads are applied in the following analysis step. In the case of pushover analyses, the lateral loads are defined as an inverted triangular load pattern which is consistent with the first fundamental mode shape of the structure. The lateral loads are assigned to the floor level nodes in the horizontal direction.

For dynamic time history analysis, the seismic masses for each floor are defined by dividing the total seismic weight of each story to the gravitational acceleration

constant. Each mass is assigned as a lumped mass in the horizontal direction to the top nodes of the fiber element representing floor level. The ground acceleration is applied to the system and the response of the structure is computed. A detailed description of these analyses is given below.

5.1 Analysis

5.1.1 Gravity Analysis

Gravity analysis is the initial analysis step for nonlinear static analysis such as monotonic or cyclic pushover and nonlinear dynamic analysis such as time history or response spectrum analysis. In this analysis, reaction of the structure at the foundation, individual element reactions, and load flow from upper levels to the foundation under gravity loads are determined. In the program, the gravity loads are step by step executed with load control loading as a predetermined load pattern linear time series. Typical analysis features are selected. Static analysis with *Newton* algorithm is used. *NormDispIncr* vector is preferred for the convergence test with predefined tolerance. *BandGeneral* solver is used, and *Plain* numberer and constraints are used as handler. The gravity loads are executed in 100 increments of steps.

5.1.2 Monotonic Pushover Analysis

The nonlinear static analysis, in other words, pushover analysis is a simplified numerical analysis method that determines the behavior and performance of structures in which the change in the load resisting capacity under the lateral loads increasing monotonically is estimated by considering the inelastic behavior of the structural components i.e., plastic hinges. There are two types of pushover analysis namely

displacement control and load control analysis. If the displacement control analysis is selected, a target displacement for the roof level of the structure is stated, and the structure is pushed in a number of steps till the target displacement is reached. For load control, a target base shear force is specified, and the structure is pushed until the target force is achieved. Predetermined horizontal load patterns such as uniform, inverted triangular, or curvature are defined along the structure height and capacity force-displacement curve is obtained by conduction elastic analysis every consecutive displacement and load increment.

Typically, displacement control is selected for convergence purposes especially for the structure load capacity decreases after some drift values. In this study, displacement control push over analysis is preferred. Inverted triangular load pattern consistent with the first fundamental mode shape of the wall is defined. Target roof displacement is specified as 0.4 m, which corresponds to a roof drift value of 2.5 %. In the program, a simple proc is written where target displacement, number of increments, and control node is defined first, and successive series displacement is applied in a loop. Similar analysis features with gravity analysis are selected since it is also a static analysis. Only, integrator type is switched to *dispcntrl* and numberer handler is set to *reverse Cuthill-Mckee* to minimize band size of the matrix.

5.1.3 Reverse Cyclic Pushover Analysis

The cyclic behavior of the prototype walls is useful to determine walls self-centering, strength degradation, and energy dissipation capacity. Nonlinear lateral reverse cyclic pushover analyses are performed under superimposed lateral loads, gravity loads, and internal loads such as post-tensioning forces. The cyclic drift history, which is given

in Figure 5-1, is specified in accordance with the ACI ITG-5.1 recommendations and includes three fully reversed cycles at each drift value. The specific purpose of reverse cyclic pushover analysis is to estimate the influence of unbonded mild steel bars on the wall's energy dissipation capacity, change in prestressing forces, and wall's superior self-centering characteristic.

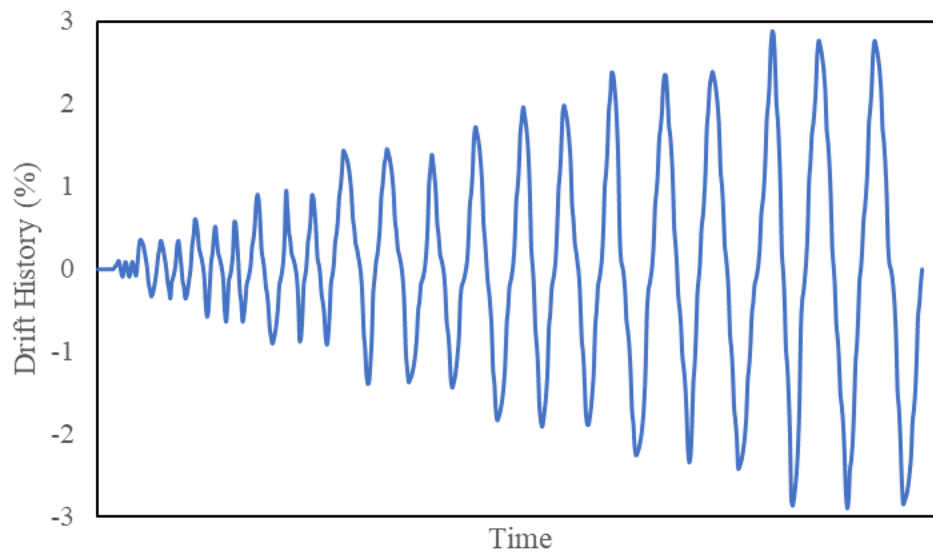


Figure 5-1: Reverse cyclic drift history

5.1.4 Modal Analysis

Modal analysis, in other word, eigenvalue analysis is a procedure in which the dynamic properties of a structure can be obtained such as natural modes, nature frequency, or mass participation ratio. Nodal masses take part in the calculation of the eigen parameters since the acceleration is involved in the dynamic analysis rather than static analysis. Therefore, nodal masses are assigned to the floor level node coordinates only in the horizontal direction. The eigenvalue, λ_n and eigenvector, Φ_n are calculated from the equation of motion for free vibration. Once the eigenvalue, λ_n is obtained, modal frequency and thus modal periods can be calculated.

The fundamental periods for each mode are obtained through the *eigen* command and some additional computation with eigenvalues in the program. The fundamental periods of the first two modes for each prototype wall are documented in Table 5-1. As shown in the table, the periods slightly decreased when mild steel bars were placed in the vicinity of the wall centerline. However, the change in fundamental periods is not significant. The mode shapes corresponding to the first two fundamental periods of the prototype hybrid walls are shown in Figure 5-2.

Table 5-1: Modal periods for all prototype walls.

Prototype Wall ID	Fundamental Periods (sec)	
	1 st Mode	2 nd Mode
Wall-R	0.500	0.076
Wall-C1	0.485	0.075
Wall-C2	0.485	0.075
Wall-C3	0.485	0.075
Wall-C4	0.485	0.075
Wall-C5	0.485	0.075
Wall-E1	0.503	0.076
Wall-E2	0.503	0.076
Wall-E3	0.502	0.076

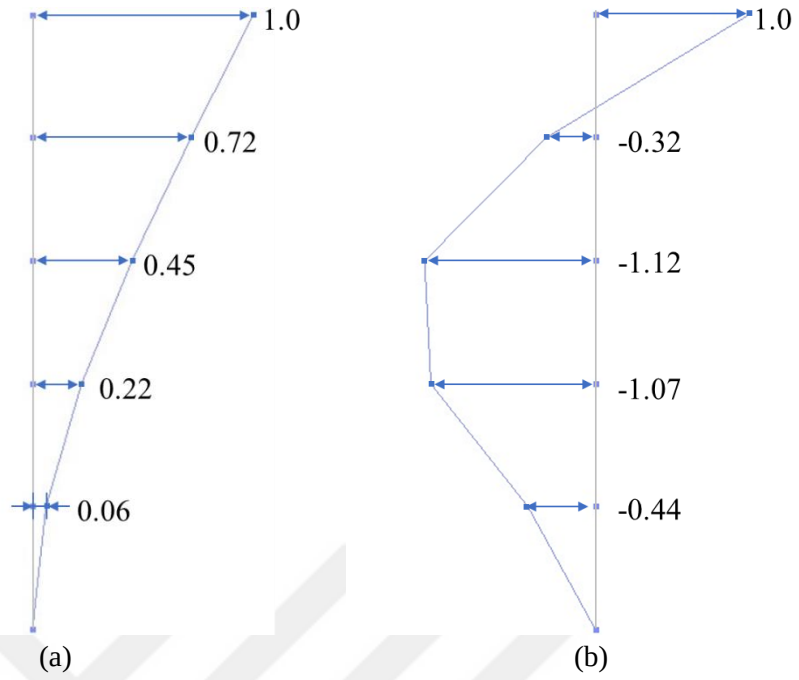


Figure 5-2: Typical (a) first and (b) second mode shapes of prototype walls

5.1.5 Nonlinear Time History Analysis

Nonlinear time history analysis is a dynamic analysis where earthquake loads are exposed to a structure with ground acceleration history. Initially, the building is transformed to a multi-degree of freedom system by lumping each story mass at corresponding story level. Then, mass, stiffness, and damping matrix for the building is developed. As given in Eq. 5-1, the dynamic equilibrium equation is formulated

$$m\ddot{u} + c\dot{u} + ku = F(t) = m\ddot{u}_g \quad (5-1)$$

where, m , c , and k symbolize mass, damping, and stiffness matrices for the building and $\ddot{u}$, $\dot{u}$, u , and $\ddot{u}_g$ represent the acceleration, velocity, displacement matrixes of the building and acceleration of ground motion, respectively. The force $F(t)$ is the external dynamic force which is a function of time. The equation of motion for each mode is

derived from the equilibrium equation. These equations are solved for each mode by step-by-step numerical methods such as Newmark's Method or Duhamel Integration, etc., and time history response of the structural system for applied ground motion acceleration is obtained. OpenSees finite element analysis program is ahead of other programs in terms of easy usability, speed performance, and low computational workload in case of nonlinear analysis. The acceleration history is applied to the structure as a uniform excitation load pattern.

In this study, inelastic dynamic analysis is conducted to investigate the effect of mild steel amount and its layout on seismic response of unbonded post-tensioned walls. Analysis features are modified to transient analysis with Newmark integrator, which is considered the most proper numerical method for dynamic analysis. New load pattern is created for acceleration time series where each property of individual ground motion records is updated manually before passing to another. For the structure a 5% mass proportional Rayleigh damping is specified to fiber elements via region command. Dynamic histories such as top node displacement, base shear demand, stress-strain demand for post-tensioned tendons and gap opening for base panel are monitored. In total, ninety nonlinear time history analysis (NLTHA) are performed for ten ground motion (GM) records. Table 5-2 shows the GM records that are obtained from PEER NGA-West2 database [87]. The ground motion data set is taken from Erkmén [67].

The ground motions are selected for both soft rock and stiff soil profiles (i.e., ASCE 7-16 site classes C and D). The ground motions are scaled to design spectra representing the design basis earthquakes (DBE). Figure 5-3 shows the acceleration response spectra for the scaled ground motions as well as the average spectra. At the end of each time history analysis, recorded outputs such as roof displacement, tendon force,

and base shear force are obtained to get maximum roof drift versus time history behavior, base shear force versus time history behavior, total dissipated energy versus time history behavior, and tendon stress loss versus time history behavior.

Table 5-2: Properties of ground motions

ID	EQ Name	Year	Component	M	R _{rup}	Scale	T (s)
CE1	Chi-Chi Taiwan	1999	Chichi/Chy079-N	7.6	55	11.3	90.0
CE2	Imperial Valley	1979	Impvall/HCPPE237	6.5	26.5	2.5	63.8
CE3	Kern County	1952	Kern/Taf111	7.4	41	2.8	54.4
CE4	Kocaeli, Turkey	1999	Kocaeli/Arc090	7.4	17	4.7	30.0
DE1	Erzincan	1992	Erzincan/Erz-EW	6.9	4.4	1.2	20.8
DE2	Chi-Chi Taiwan	1999	Chichi/Tcu072-N	7.6	7.1	0.8	90.0
DE3	Imp. Valley	1940	Impvall/I-Elc270	7	6.1	1.8	53.5
DE4	Northridge	1994	Northhr/Ro3090	6.7	10.1	1.3	30.3
DE5	Big Bear	1992	Bigbear/Hos180	6.5	35.2	5.1	100.0
DE6	Supers Hills	1987	Superst/B-Icc000	6.5	18.2	1.8	59.9

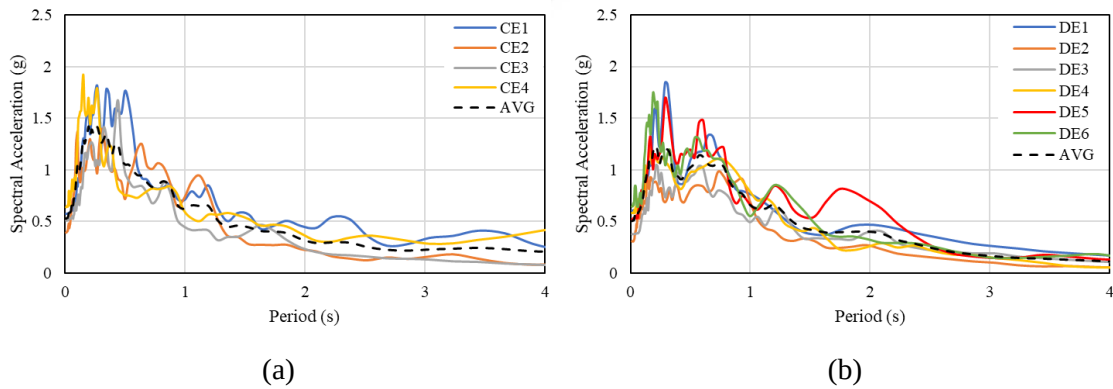


Figure 5-3: Scaled ground motion response spectra

5.2 Behavior of Walls Under Static Pushover Loads

Under pushover loading the walls are displaced up to 3% wall drift, and capacity curves for walls are obtained. Figure 5-4 and Figure 5-5 show the pushover curves for the prototype walls with mild steel bars at the center and at the edges, respectively as well as

for the reference wall Wall-R. Overall wall responses for walls with mild steel are similar. The walls exhibited similar lateral force-drift behavior up to approximately 0.25% drift. However, walls ultimate lateral load carrying capacity increased 9.7% for Wall-C1, 13.8% for Wall-C2, 17.4% for Wall-C3, 20.4% for Wall-C4, 22.8% for Wall-C5, 20.8% for Wall-E1, 19.9% for Wall-E2 and 19.3% for Wall-E3 due to contribution of mild steel when compared with Wall-R reference wall. The results show that presence of the mild steel bars and increase in mild steel bar total area enhance the wall's ultimate lateral load carrying capacity. The results also indicate that ultimate load resisting capacity of the Wall-E1, Wall-E2, and Wall-E3 are almost the same. However, the load resisting capacity slightly decreased with increase of the unbonded length of the mild steel bars. This is expected since effects of unbonded mild steel length has negligible effects on walls overall lateral stiffness for the considered unbonded lengths.

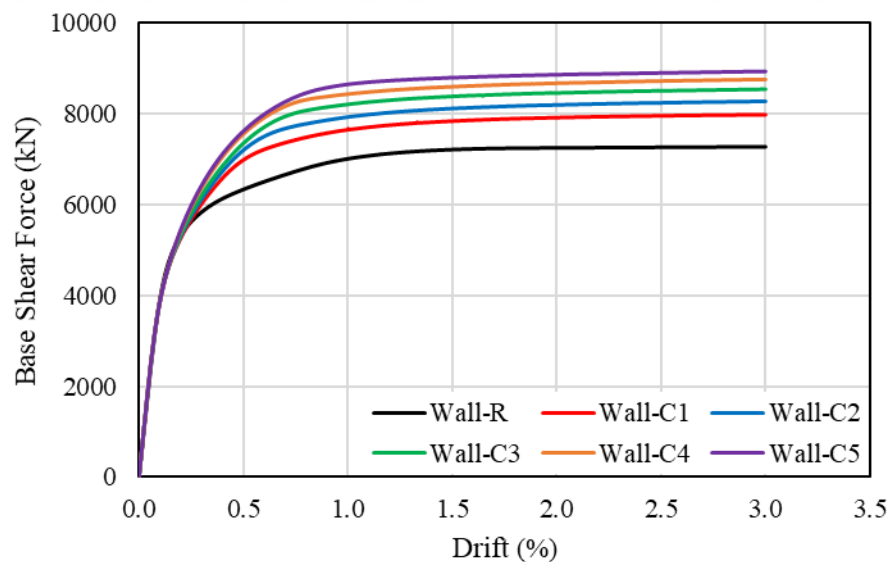


Figure 5-4: Pushover curves of wall with mild steel bar at the center

The gap opening behavior is evaluated in terms of the amount of mild steel bars, location of the bars, and unbonded length of the bars. The Wall-R reference wall is compared with the walls where mild steel bars are located at the wall center to investigate

the effect of steel amount and walls with mild steel bars placed at the edge are compared with the Wall-C4 which has approximately similar ultimate capacity with those walls to investigate the effect of mild bars location. Effect of unbonded length on the gap opening is investigated only for the walls with mild steel placed at wall ends.

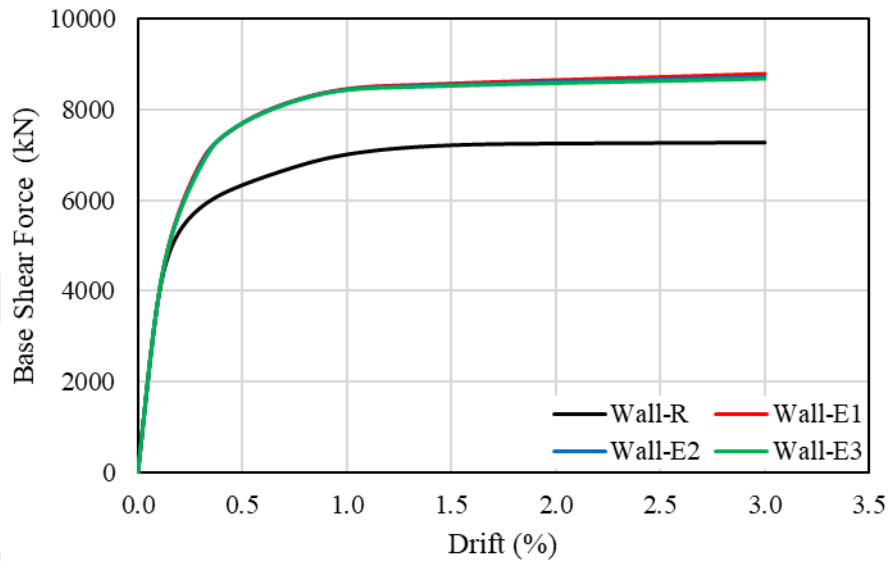


Figure 5-5: Pushover curves of wall with mild steel bar at the edges

Walls gap opening under lateral loads versus wall drift is shown in Figures 5-6 and Figure 5-7. Based on the results, the wall gap openings initiated at the same time for all walls but started to diverge at approximately 0.25% drift. The gap opening versus lateral load behavior is linear for Wall-R while nonlinear up to gap opening of 15 mm for walls with mild steel at edge (i.e., Wall-EX) and 20 mm for Walls with mild steel at center (i.e., Wall-CX), respectively. The gap opening versus drift behavior for all walls is linear for larger gap opening values. The results regarding the wall with mild steel bars at the center show that increase of the steel amount reduced the gap opening approximately 20 to 40 mm. Comparison of the Wall-E1, E2, and E3 with Wall-C4 indicates that placing the mild steel bars at the wall ends reduces the gap opening less than those with at the center although they provide similar load resisting capacity. The reason is that half of the

bars have no contribution to resisting moment for gap opening during the loading in each direction. However, the presence of the mild steel bars provided a resistance to decrease the gap opening. Comparing the response of Wall-E1, E2, and E3 shows that the unbonded length of the mild steel bars has negligible effects on wall gap opening versus drift behavior.

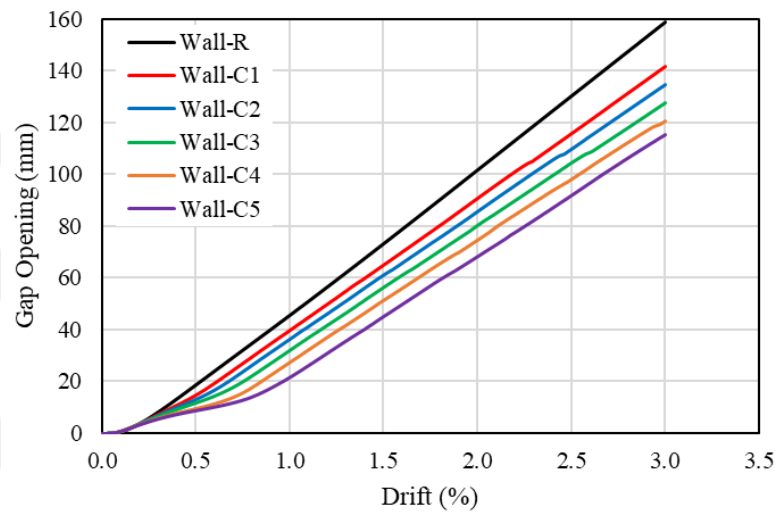


Figure 5-6: Gap opening of wall with mild steel bar at the center

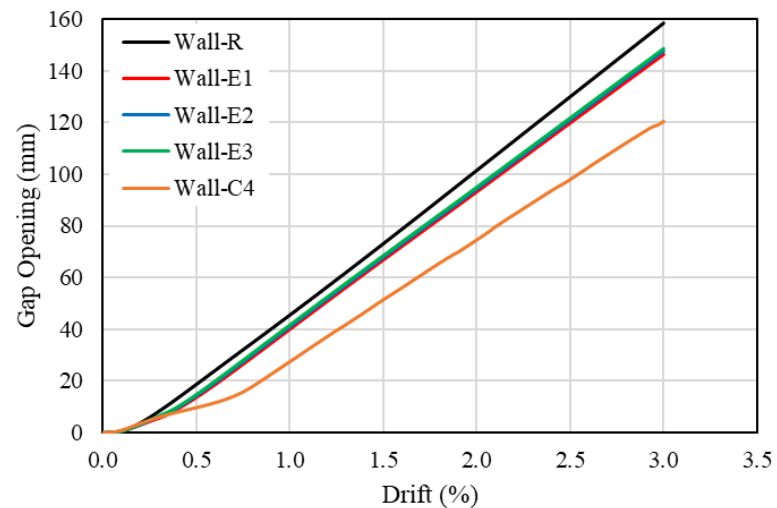


Figure 5-7: Gap opening of wall with mild steel bar at the edges

5.3 Behavior of Walls Under Reverse Cyclic Pushover Loads

The pushover curves obtained for each wall are shown in Figures 5-8 through Figure 5-16. The cyclic behaviors of the walls with mild steel bars are compared with that of the reference wall (Wall-R) given in Figure 5-8. The reverse cyclic pushover analysis results indicate that presence, amount, and layout of the unbonded mild steel bars significantly affects wall cyclic response including the shape of hysteresis loops, energy dissipation capacity, and permanent displacement. As can be seen from the behavior of Wall-R, the loops are narrow and close to each. For walls with mild steel, the hysteresis loops become wider, which indicate larger energy dissipation. The increased total area of mild steel further contributed to the total energy dissipated. Similar wall cyclic behavior is observed when the mild steel bars are moved to wall ends, but breath of the hysteresis loops increased, and the walls dissipated more energy as expected. However, nearly identical hysteresis loops occurred when the unbonded length of the mild steel bars were changed. The presence of mild steel also increased wall residual displacement but, the increase in residual displacement is negligible, and walls preserved their superior self-centering characteristic which is consistent with results from Erkmen and Schultz [84].

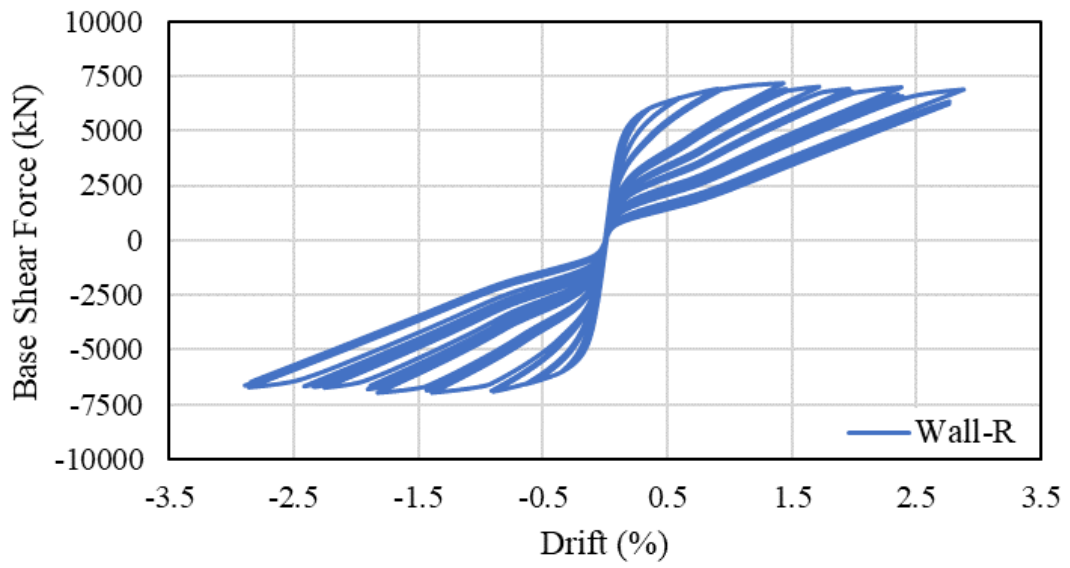


Figure 5-8: Base shear versus drift behavior of reference Wall-R

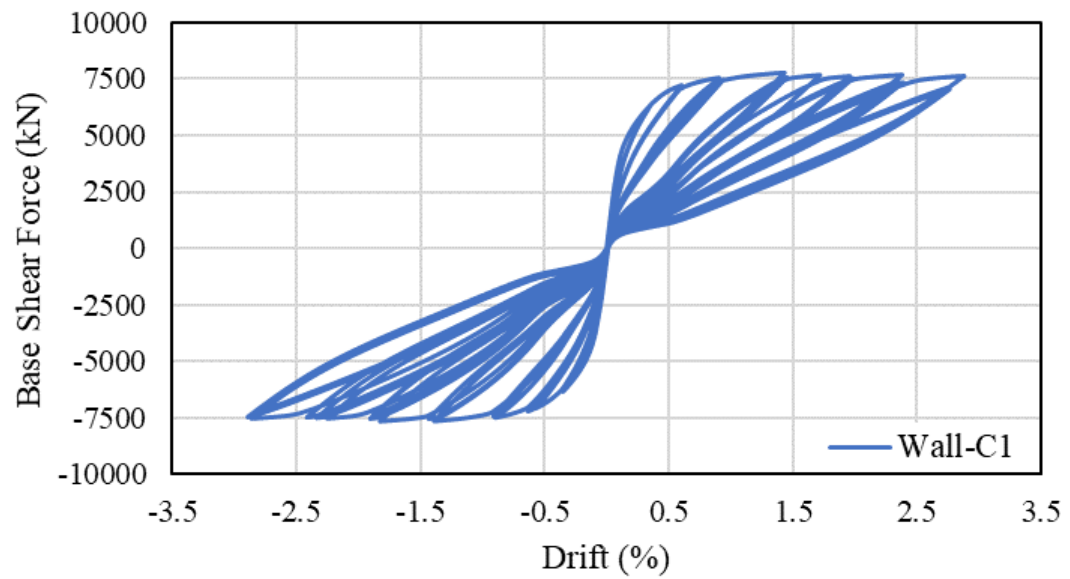


Figure 5-9: Base shear versus drift behavior of Wall-C1

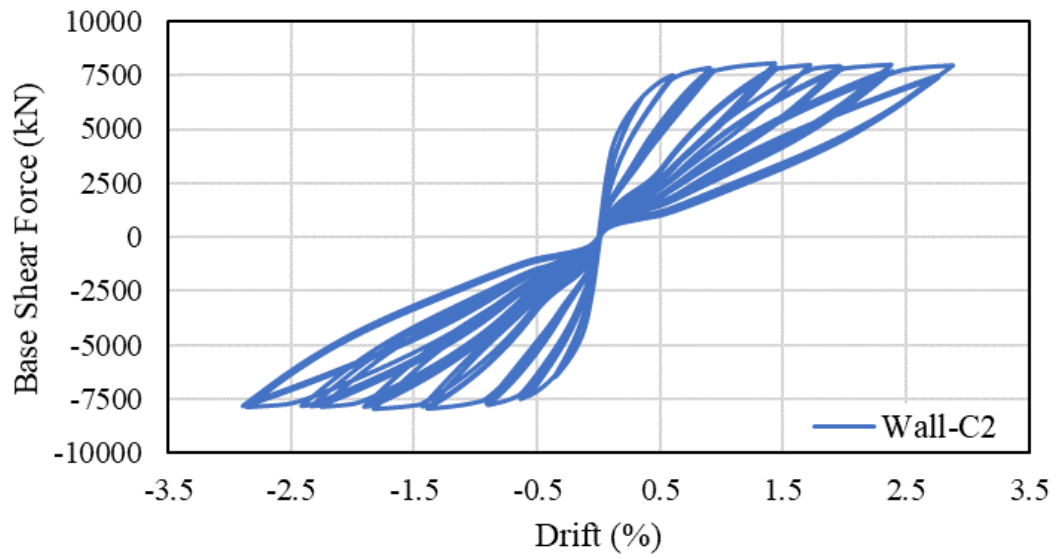


Figure 5-10: Base shear versus drift behavior of Wall-C2

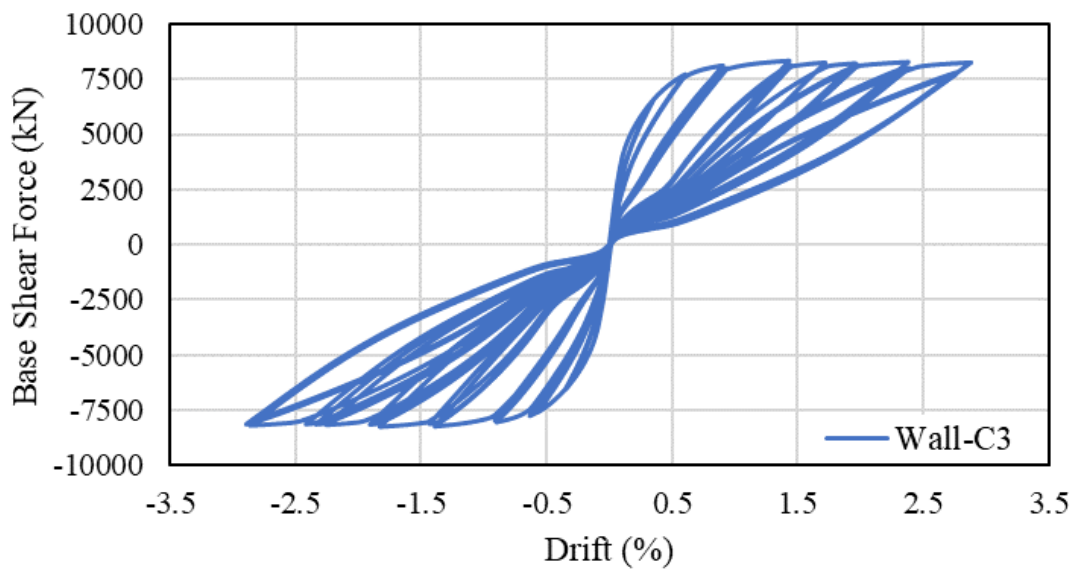


Figure 5-11: Base shear versus drift behavior of Wall-C3

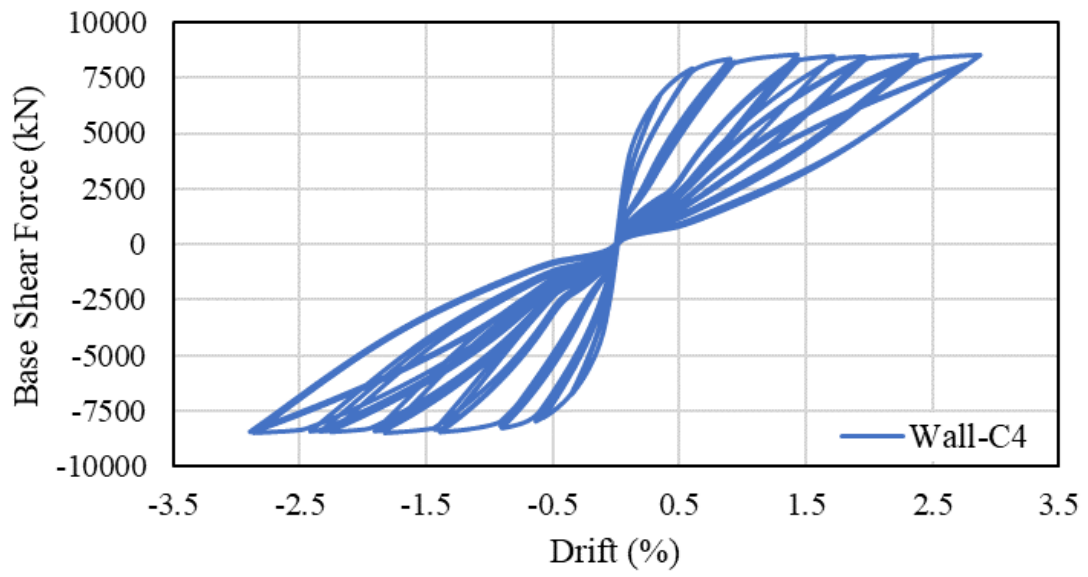


Figure 5-12: Base shear versus drift behavior of Wall-C4

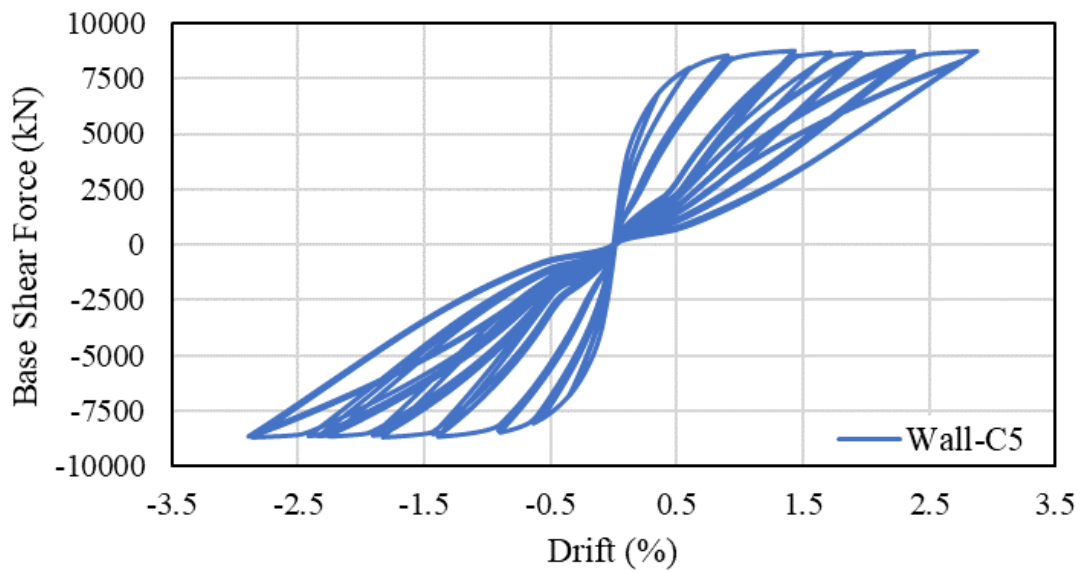


Figure 5-13: Base shear versus drift behavior of Wall-C5

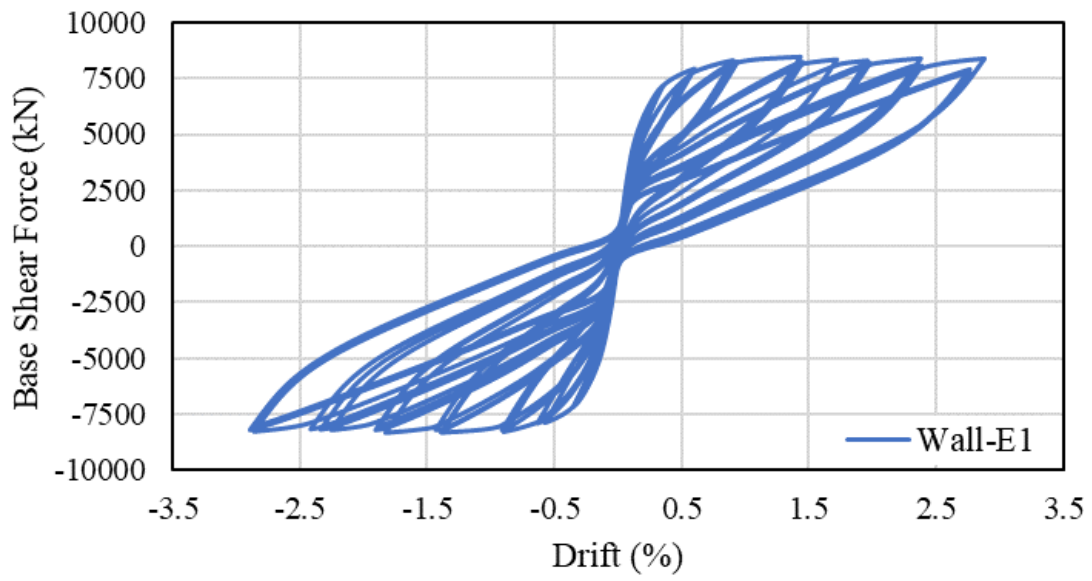


Figure 5-14: Base shear versus drift behavior of Wall-E1

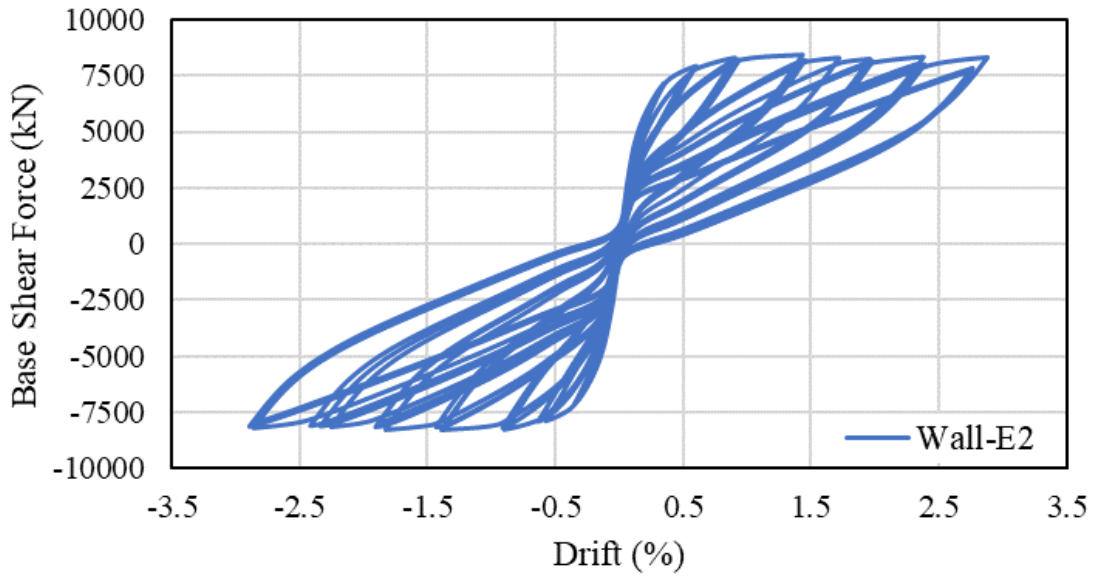


Figure 5-15: Base shear versus drift behavior of Wall-E2

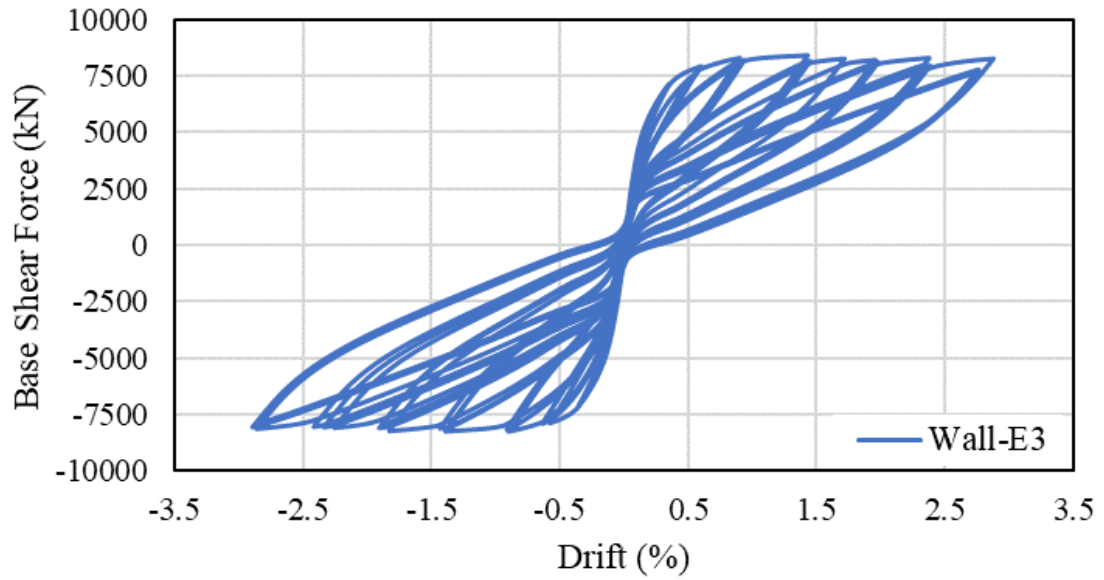


Figure 5-16: Base shear versus drift behavior of Wall-E3

One of the main objectives of this study is to determine the effect of amount, layout, and unbonded length of the mild steel on the energy dissipation capacity of the walls. Dissipated energy for each wall was estimated by calculating the area enclosed by base shear versus drift loops. The area of the hysteresis loops was calculated using trapezoidal integration procedure. The dissipated energy was reported at each drift after three fully reversed cycles were completed. The results for energy dissipated are presented in Figure 5-17. The main reason for using unbonded mild steel bars for these walls is to enhance their energy dissipation capacity since the walls without mild steel has small energy dissipation capacities [19]. The amount of dissipated energy is negligible up to 0.5% wall drift since the wall behavior is in the elastic range. For larger drift values, it was observed that the energy dissipated by the reference Wall-R is significantly smaller than all other walls as expected. Energy dissipation capacities of the walls with mild steel bars at the center and at the edges gradually increased. The walls with unbonded mild steel at the center (i.e., Wall-C1, Wall-C2, Wall-C3, Wall-C4, and Wall-C5) and edges (i.e., Wall-E1, Wall-E2, and Wall-E3) have cumulative energy

dissipation capacity that is more than four and eight times as large as Wall-R, respectively. Nevertheless, the total energy dissipation capacity of walls with mild steel bars at the center increased with the amount of steel up Wall-C4 corresponding the 75% of maximum steel ratio; however, the Wall-C5 has 5% less total energy dissipation than Wall-C4 although having larger mild steel amounts. When the walls with mild steel bars at the edges are compared with the Wall-C4, it is seen that the total energy dissipating capacity of the walls increased nearly two times by shifting the bars toward the edges although total steel area is decreased to the half of the Wall-C4 steel amount. In other words, moving the unbonded mild steel towards the wall ends decreased the demand for mild steel amount while increasing the total energy dissipation capacity of the walls. When Wall-E1, Wall-E2, and Wall-E3 are compared between each other, it is observed that using longer unbonded length of the mild steel bars resulted in a negligible decrease in the total dissipated energy.

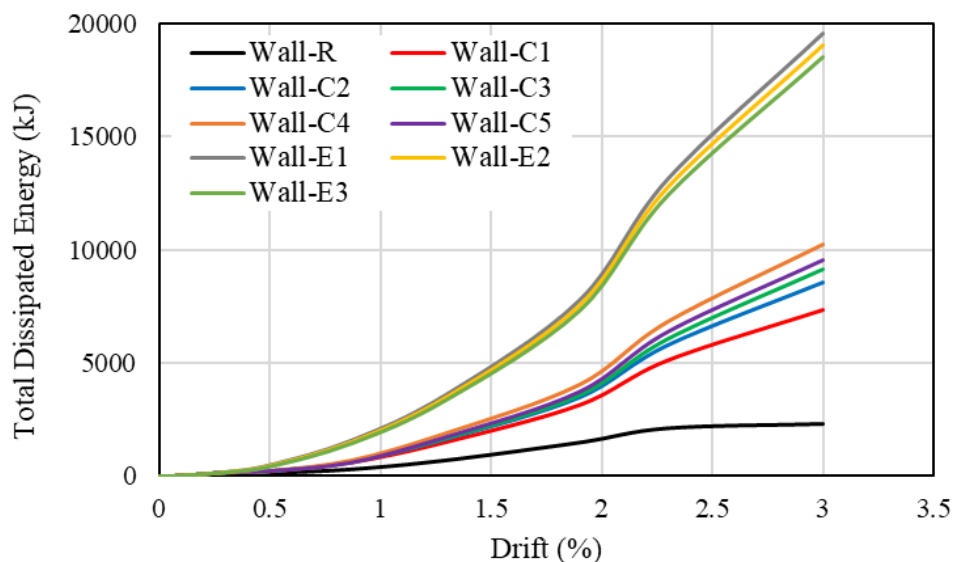


Figure 5-17: Total energy dissipation versus drift behavior of the walls

The post-tensioned tendons are located in the wall vertical centerline to minimize their prestress losses during seismic loading. After the cyclic loading in positive and negative direction, the unbonded post-tensioned tendons undergo elongations beyond their yield limit and post-tensioning tendon force decreases due to steel yielding. The losses in average post-tensioning stress are given in Figure 5-18 at the end of each third fully reversed cycle at every drift level. At drifts of less than 1.0%, all walls had similar post-tensioning stress losses. For larger drift values, Wall-R has the largest stress losses while the walls with mild steel bars at the wall ends have the largest and similar post-tensioning stress losses after Wall-R. In other words, providing unbonded mild steel decreased average post-tensioning stress losses for all walls. Smaller stress losses were observed when the mild steel bars were placed at the wall vertical center line. In other words, the average post-tensioning stress losses are approximately 88% for Wall-R, 80% for Wall-E1, Wall-E2, and Wall-E3 and 70%, 65%, 57%, 50% and 46% for Wall-C1, Wall-C2, Wall-C3, Wall-C4, and Wall-C5, respectively as shown in Figure 5-18. Consequently, unbonded steel layout has significant effects on the tendon post-tensioning stress losses. Placing unbonded mild steel in the vicinity of the wall vertical center line minimized the average post-tensioned tendons stress losses. For the walls with mild steel bars located at the wall the post-tensioning stress losses increased 60% compared to Wall-C4. However, changing the unbonded length of the mild steel bars did not affect the tendon post-tensioning stress losses and losses were approximately identical.

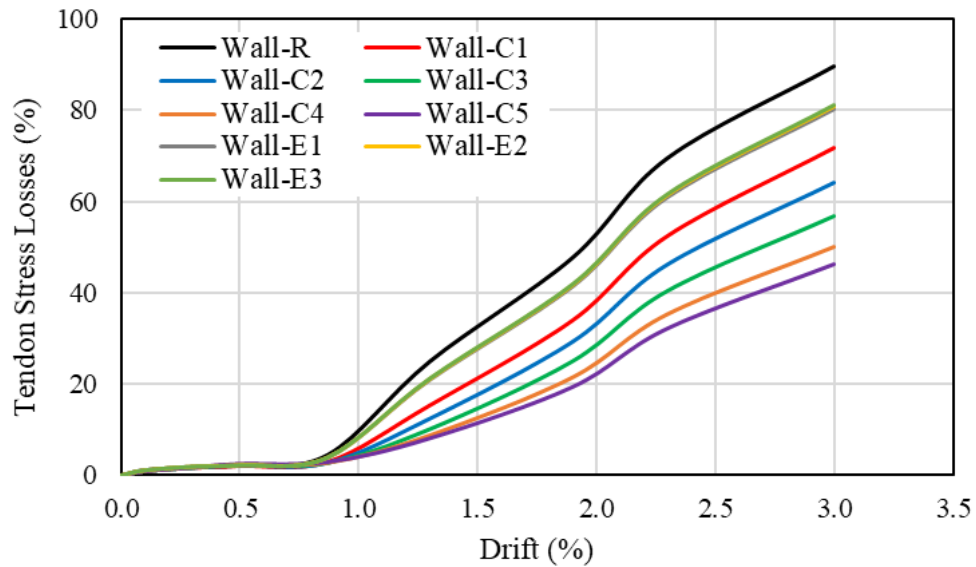


Figure 5-18: Average tendon stress losses versus drift behavior of the walls

5.4 Behavior of Walls Under Earthquake Loads

Nonlinear time history analyses are performed for all prototype walls by using ten scaled ground motion records. Wall maximum top drift, total dissipated energy, and tendon post-tensioning stress loss are monitored, and effects of mild steel amount, layout, and unbonded length are investigated. The investigations were done based on the acceleration versus time history behavior and base shear force versus time history behavior of the walls for each ground motion record. Obtained acceleration – time history behaviors and base shear force – time history behaviors are shown in Figures 5-23 through Figure 5-32 and Figures 5-33 through Figure 5-42, respectively. Based on the observation on the acceleration graphs, it is seen that the general shape of the response does not differ from each other. Based on the observation on the base shear response, it is seen that the shape of the hysteresis is similar to each other for all walls, but significant variation on the base shear demand and maximum roof drift exists.

The maximum roof drift demand computed from nonlinear time history analysis is an indication of probable damage on the structure under earthquake loads. The maximum roof drift demands for each wall under ten-time history analysis is tabulated in Table 5-3. Also, the maximum roof drift values are shown in Figure 5-19. Almost for each time history analysis, it is observed that presence of mild steel bars limited the maximum roof drift demand. In other words, roof drift demand decreased as much as 7.4% and 22.4% for walls with mild steel bars at the center and at wall ends, respectively. Placing the mild steel bars at the wall ends decreased the demand for drift more than three times that of the walls with mild steel bars at the wall center. Also, the roof drift demand slightly increased with the increasing of the unbonded length of the mild steel bars, but this difference is negligible.

Table 5-3: Maximum roof drifts of the walls under each ground motions history

Ground Motion	Wall								
	R	C1	C2	C3	C4	C5	E1	E2	E3
CE1	1.73	1.57	1.54	1.53	1.49	1.49	0.97	0.99	1.01
CE2	1.33	1.40	1.37	1.38	1.39	1.42	0.99	1.01	1.05
CE3	0.98	1.03	1.09	1.12	1.12	1.13	0.92	0.95	0.98
CE4	1.16	1.12	1.11	1.13	1.09	1.14	0.95	1.02	1.08
DE1	1.20	1.14	1.11	1.08	1.05	1.04	1.01	1.02	1.02
DE2	1.31	1.20	1.19	1.18	1.16	1.15	1.07	1.08	1.09
DE3	1.07	1.20	1.17	1.15	1.12	1.11	1.06	1.07	1.07
DE4	1.34	1.29	1.26	1.26	1.26	1.29	1.15	1.16	1.17
DE5	1.82	1.36	1.42	1.45	1.42	1.40	1.07	1.08	1.09
DE6	1.43	1.26	1.20	1.16	1.10	1.09	0.99	0.99	1.00

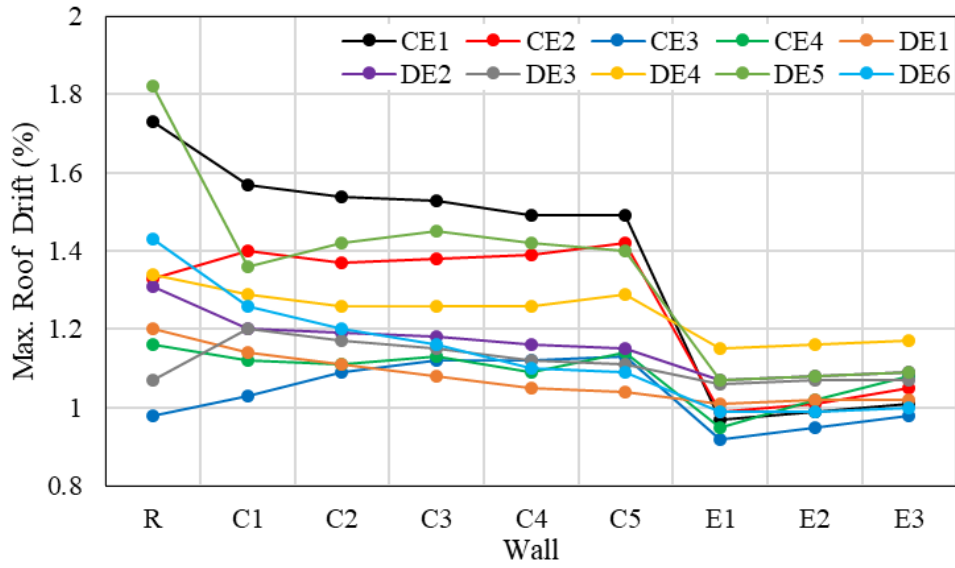
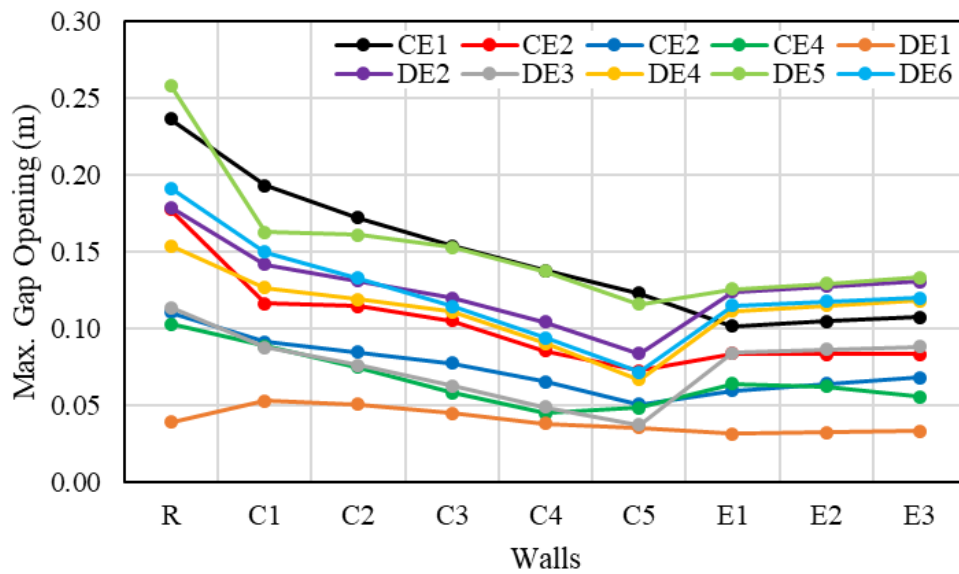


Figure 5-19: Maximum roof drifts of the walls

The gap opening between base panel and foundation obtained from nonlinear time history analysis is an indication of the amount of dissipated energy during an earthquake event. The maximum gap opening for each wall under ten-time history analysis is listed in Table 5-4. Also, the maximum gap opening values are given in Figure 5-20. Almost for each time history analysis, it is observed that presence of mild steel bars decreased the amount of gap opening. In other words, roof drift demand decreased as much as 45.7% and 39.9% for walls with mild steel bars at the center and at wall ends, respectively. Although, presence of mild steel bars reduced the amount of gap opening, it was observed that the amount of gap opening for the walls with mild steel bars at edges is relatively higher than that of the walls with mild steel bars at the wall center. Also, the gap opening amount slightly increased with the increasing of the unbonded length of the mild steel bars, but this difference was negligible.

Table 5-4: Maximum gap openings of the walls under ground motion histories

Ground Motion	Wall								
	R	C1	C2	C3	C4	C5	E1	E2	E3
CE1	0.24	0.19	0.17	0.15	0.14	0.12	0.10	0.10	0.11
CE2	0.18	0.12	0.11	0.11	0.09	0.07	0.08	0.08	0.08
CE3	0.11	0.09	0.08	0.08	0.07	0.05	0.06	0.06	0.07
CE4	0.10	0.09	0.07	0.06	0.05	0.05	0.06	0.06	0.06
DE1	0.04	0.05	0.05	0.05	0.04	0.04	0.03	0.03	0.03
DE2	0.18	0.14	0.13	0.12	0.10	0.08	0.12	0.13	0.13
DE3	0.11	0.09	0.08	0.06	0.05	0.04	0.08	0.09	0.09
DE4	0.15	0.13	0.12	0.11	0.09	0.07	0.11	0.12	0.12
DE5	0.26	0.16	0.16	0.15	0.14	0.12	0.13	0.13	0.13
DE6	0.19	0.15	0.13	0.11	0.09	0.07	0.12	0.12	0.12

**Figure 5-20:** Maximum gap opening at foundation-base wall panel joint

The significance of the total energy dissipating capacity of the walls are previously mentioned in the cyclic behavior part. The findings from the nonlinear time history analysis are consistent with the result of the cyclic pushover analysis of the walls. The cumulative dissipated energies at the end of each time history analysis are shown in

Figure 5-21. Based on the results presented, the presence of mild steel bars enhanced the energy dissipation capacity of the wall as much as 15.4% and 36.8% for the walls that mild steel bars were placed in the vicinity of centerline and edges, respectively when compared to Wall-R, which does not have any mild steel. Increasing the amount of mild steel up to 75% increased the energy dissipation however the difference between Wall-C4 and Wall-C5 is insignificant. In other words, Wall-C4 presented the maximum capacity for energy dissipation. The amount of the dissipated energy was the highest when the mild steel bars were placed at the edges, and the dissipation capacity increased by 14.3%. Nevertheless, the total energy dissipation capacity slightly decreased when the unbonded length of the mild steel bars increased. In other words, the unbonded length of the mild steel bars has an insignificant effect on the accumulated energy dissipation capacity.

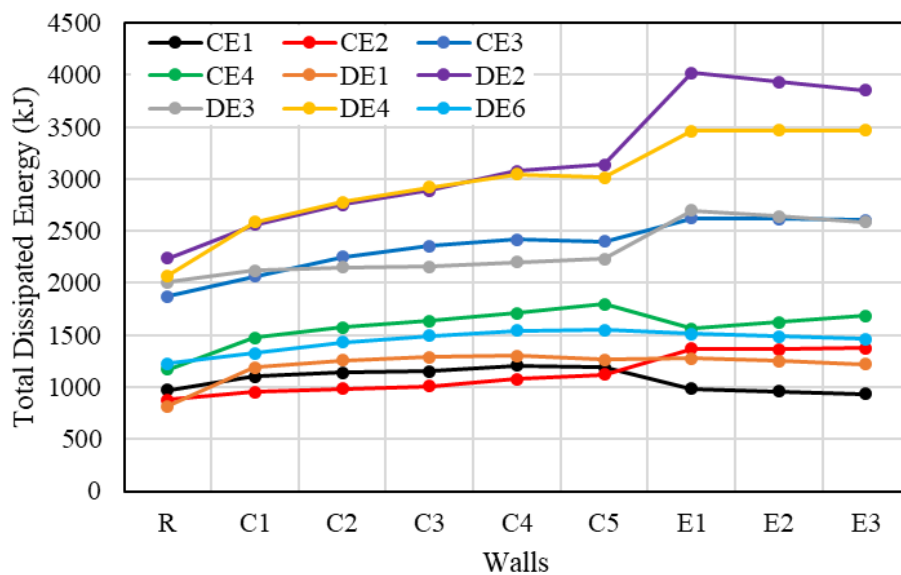


Figure 5-21: Total wall energy dissipated during the time history analysis

The final average post-tensioning stress level of the walls after the time history analysis is shown in Figure 5-22. Under earthquake loads, the post-tensioning stress losses are approximately between 4.4% and 36.5% for the reference wall (Wall-R). The

presence of the mild steel bars limited the maximum post-tensioning stress loss to 18.32%, and it is observed that the stress loss is only between 2.9% and 18.32. Increasing the mild steel amount relatively decreased the tendon stress losses. There is no significant difference between the walls with mild steel bars at the center and at wall ends.

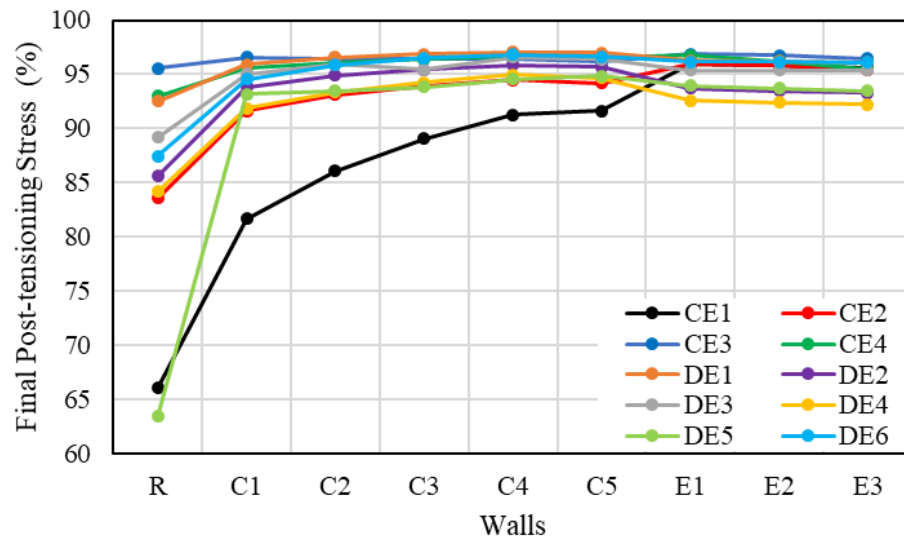


Figure 5-22: Wall final average post-tensioning stresses after earthquakes

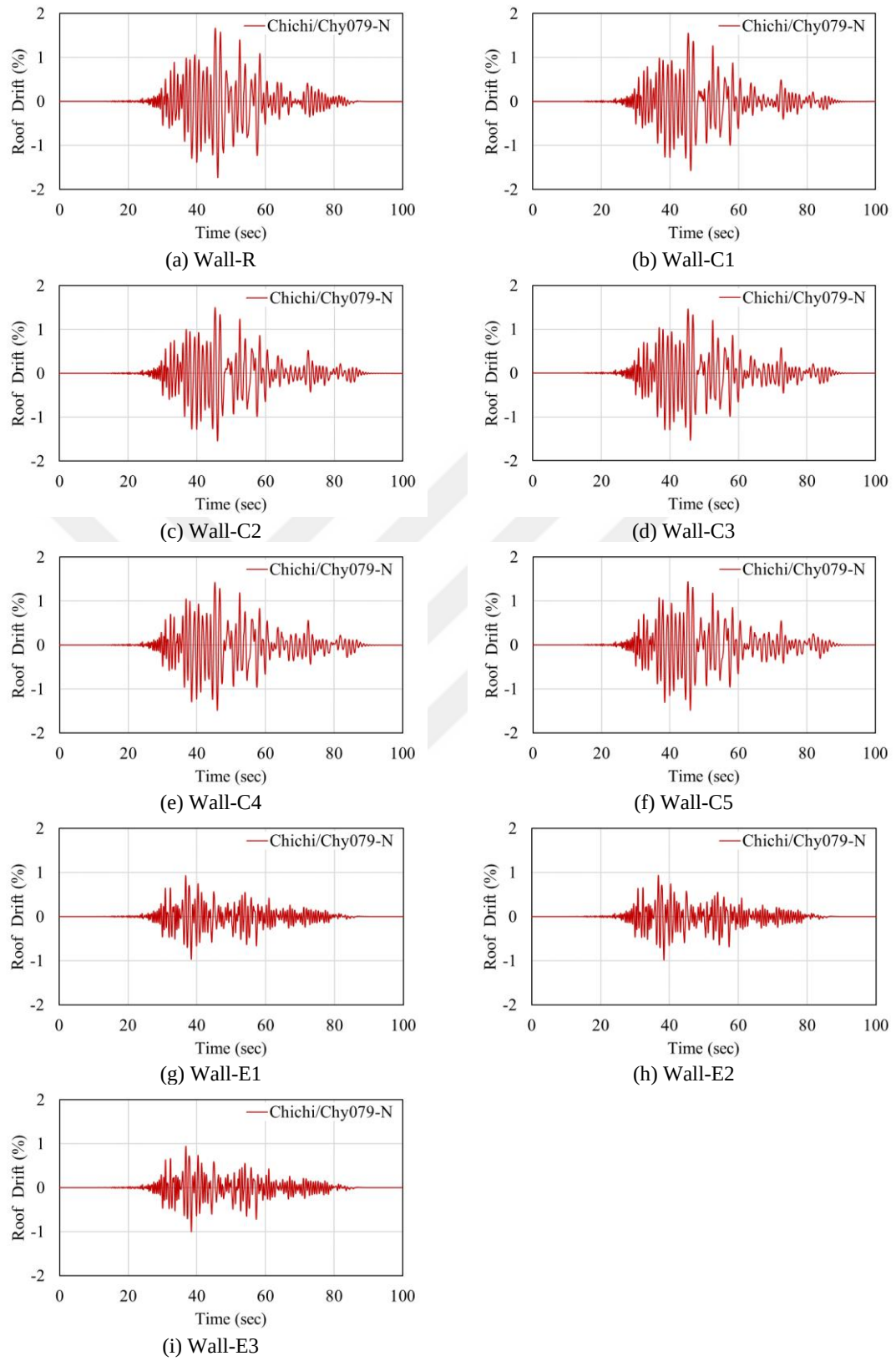


Figure 5-23: Roof drift vs time history behavior of the walls (CE1)

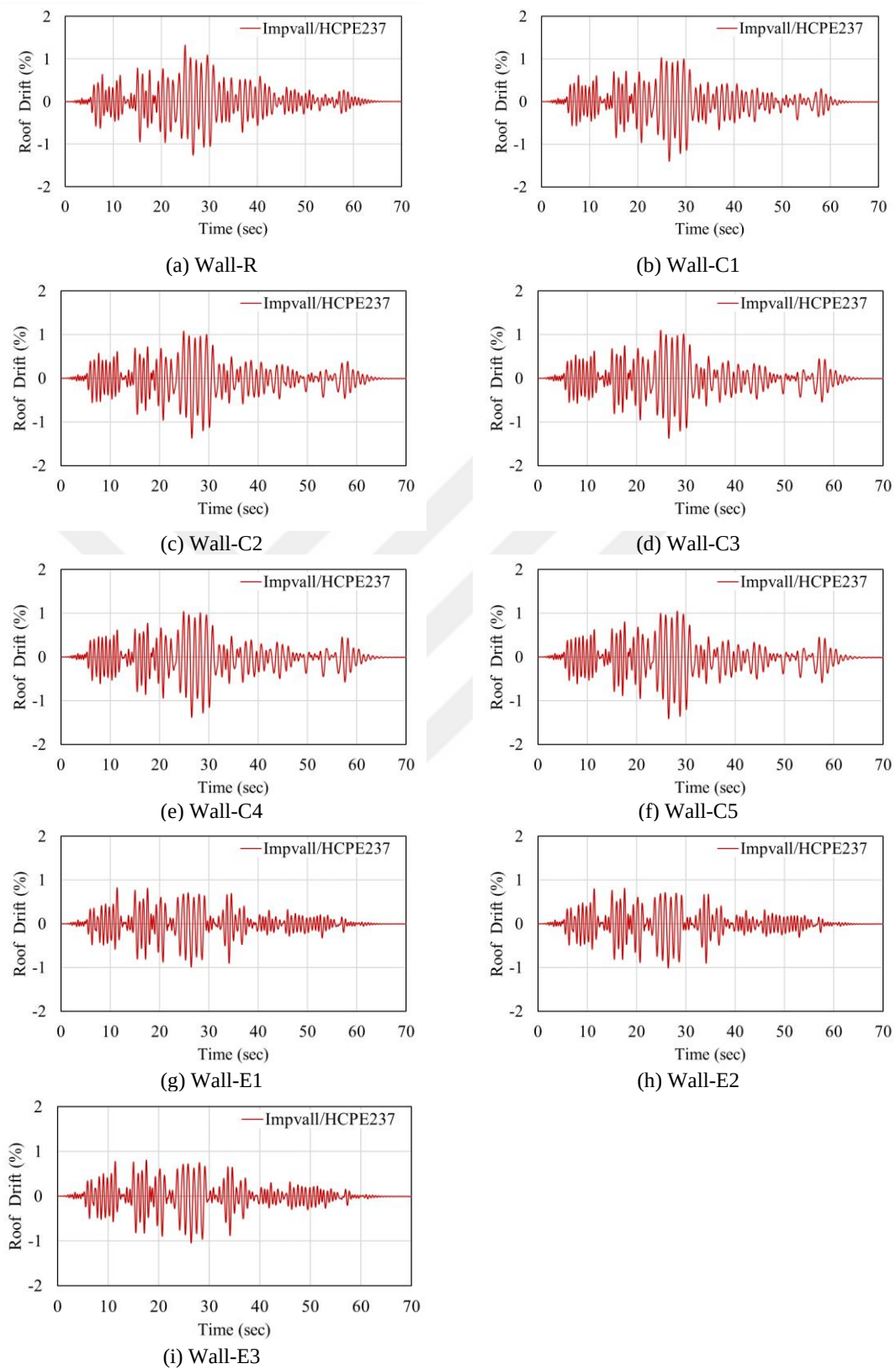


Figure 5-24: Roof drift vs time history behavior of the walls (CE2)

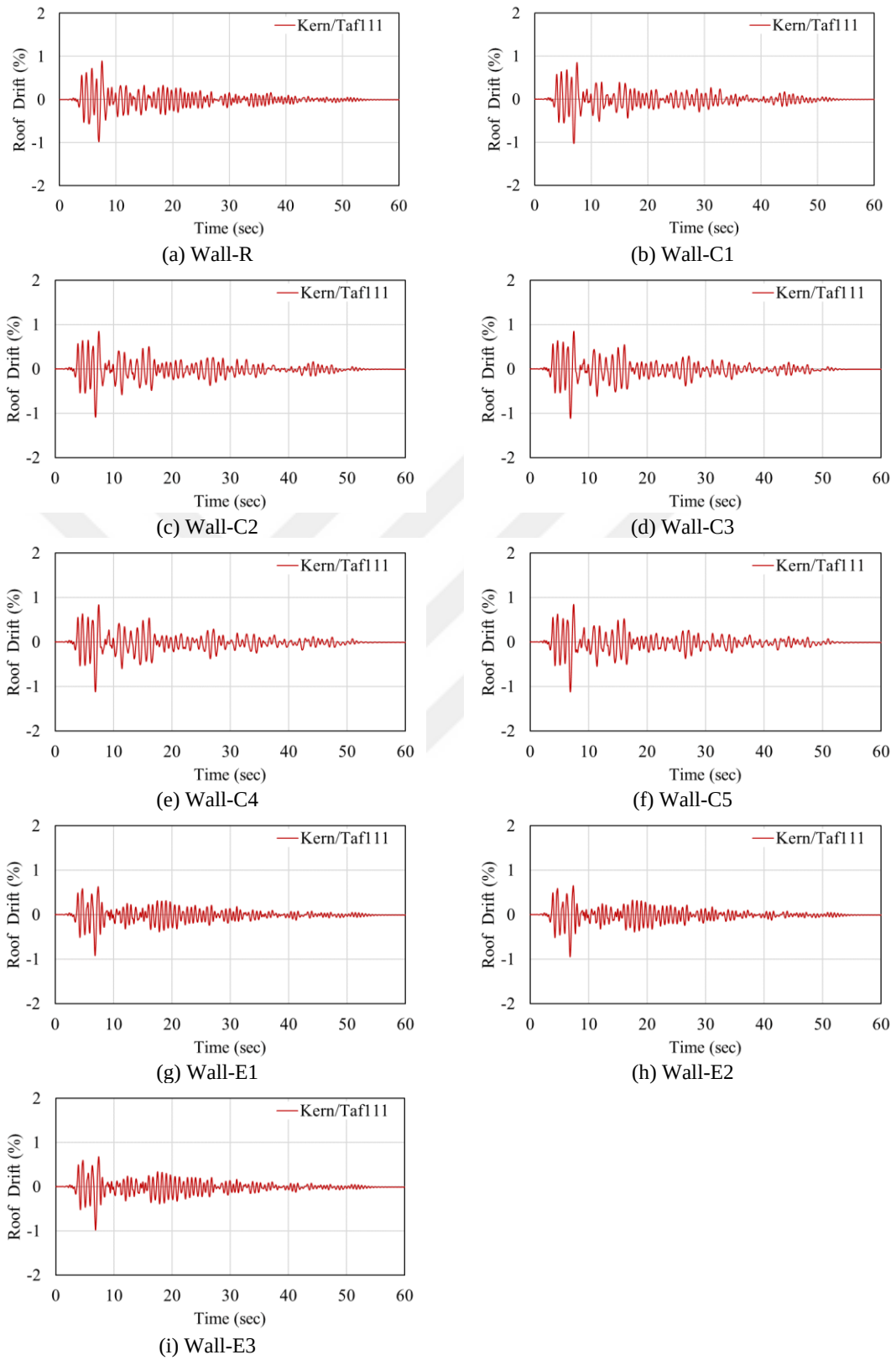


Figure 5-25: Roof drift vs time history behavior of the walls (CE3)

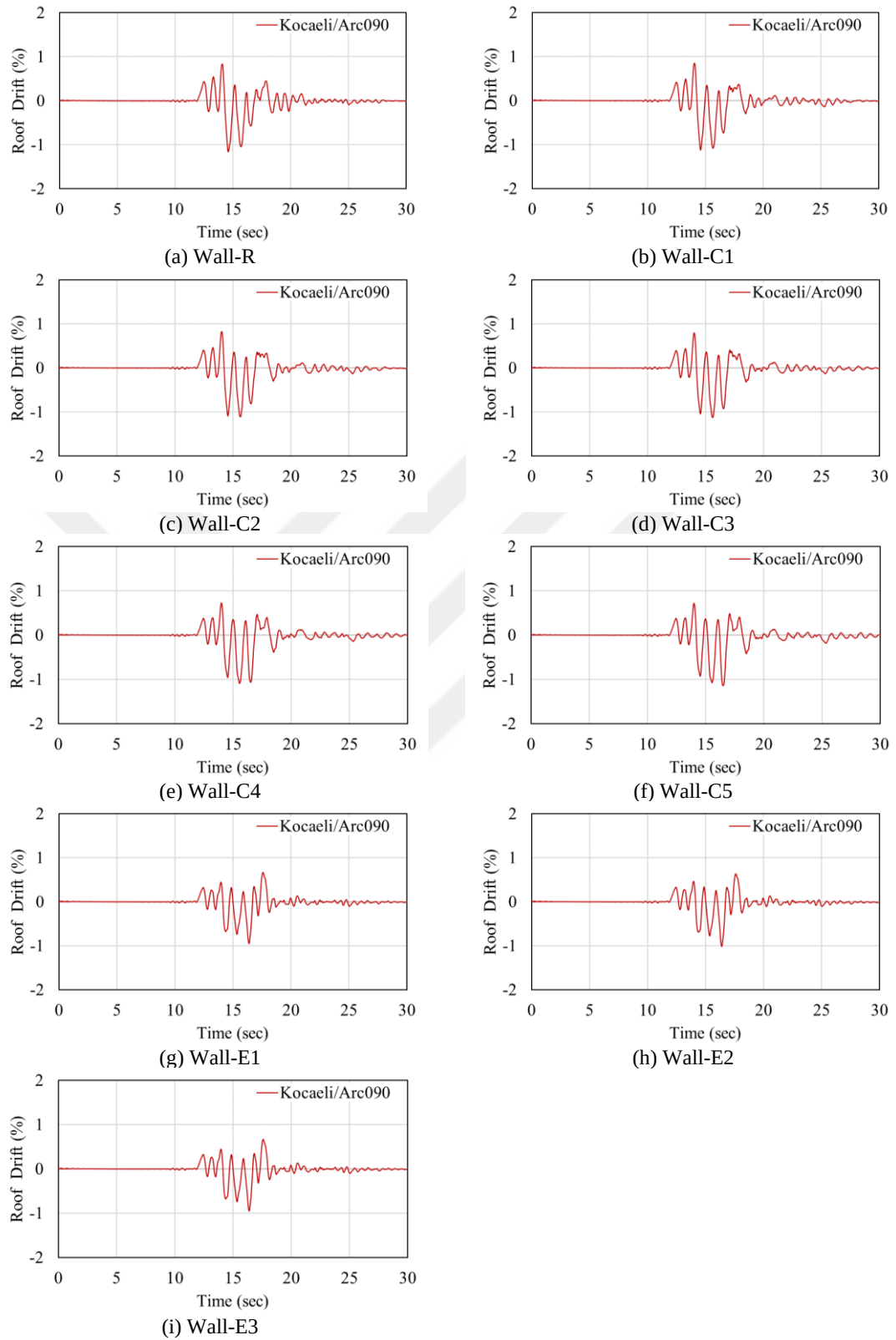


Figure 5-26: Roof drift vs time history behavior of the walls (CE4)

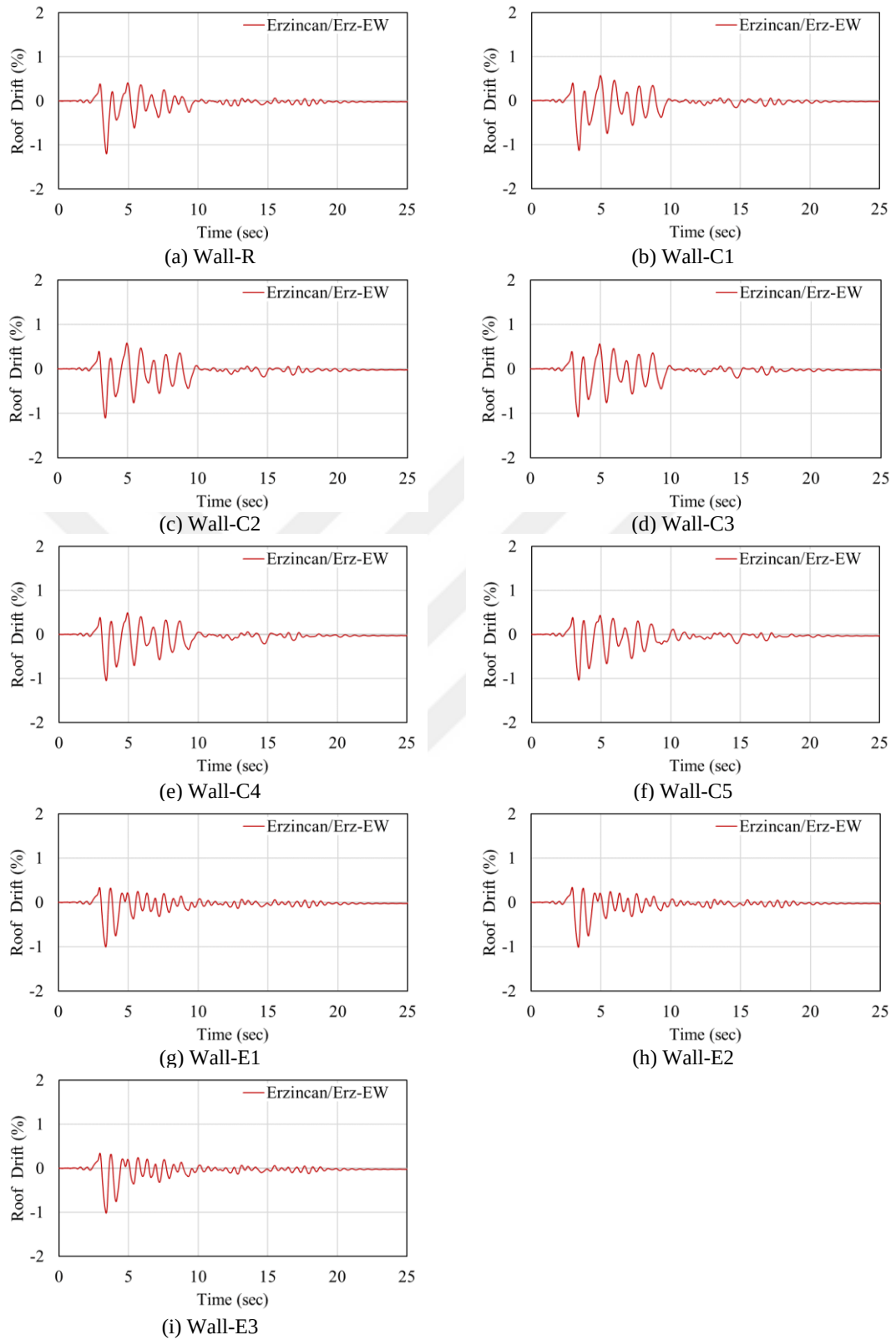


Figure 5-27: Roof drift vs time history behavior of the walls (DE1)

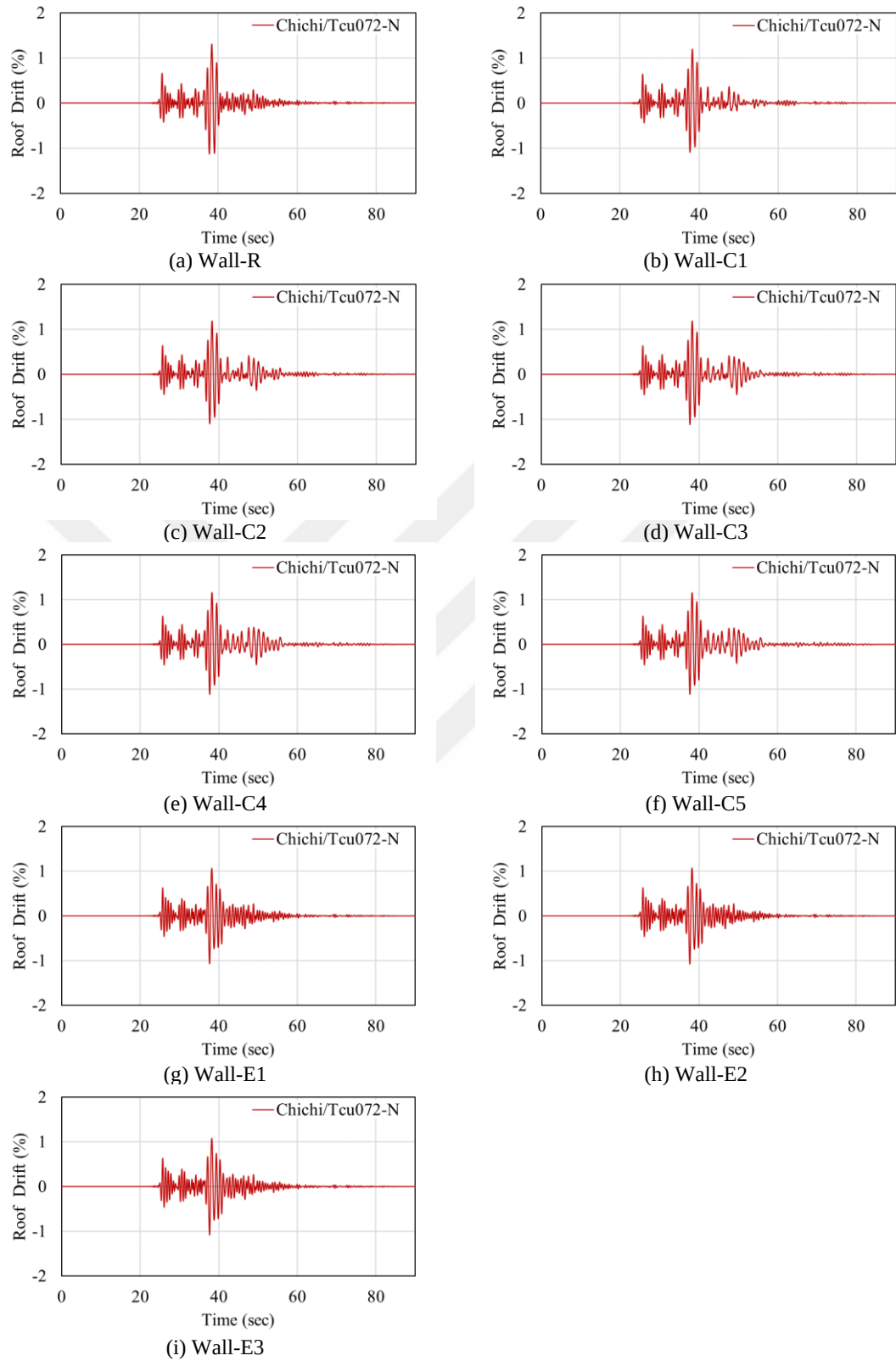


Figure 5-28: Roof drift vs time history behavior of the walls (DE2)

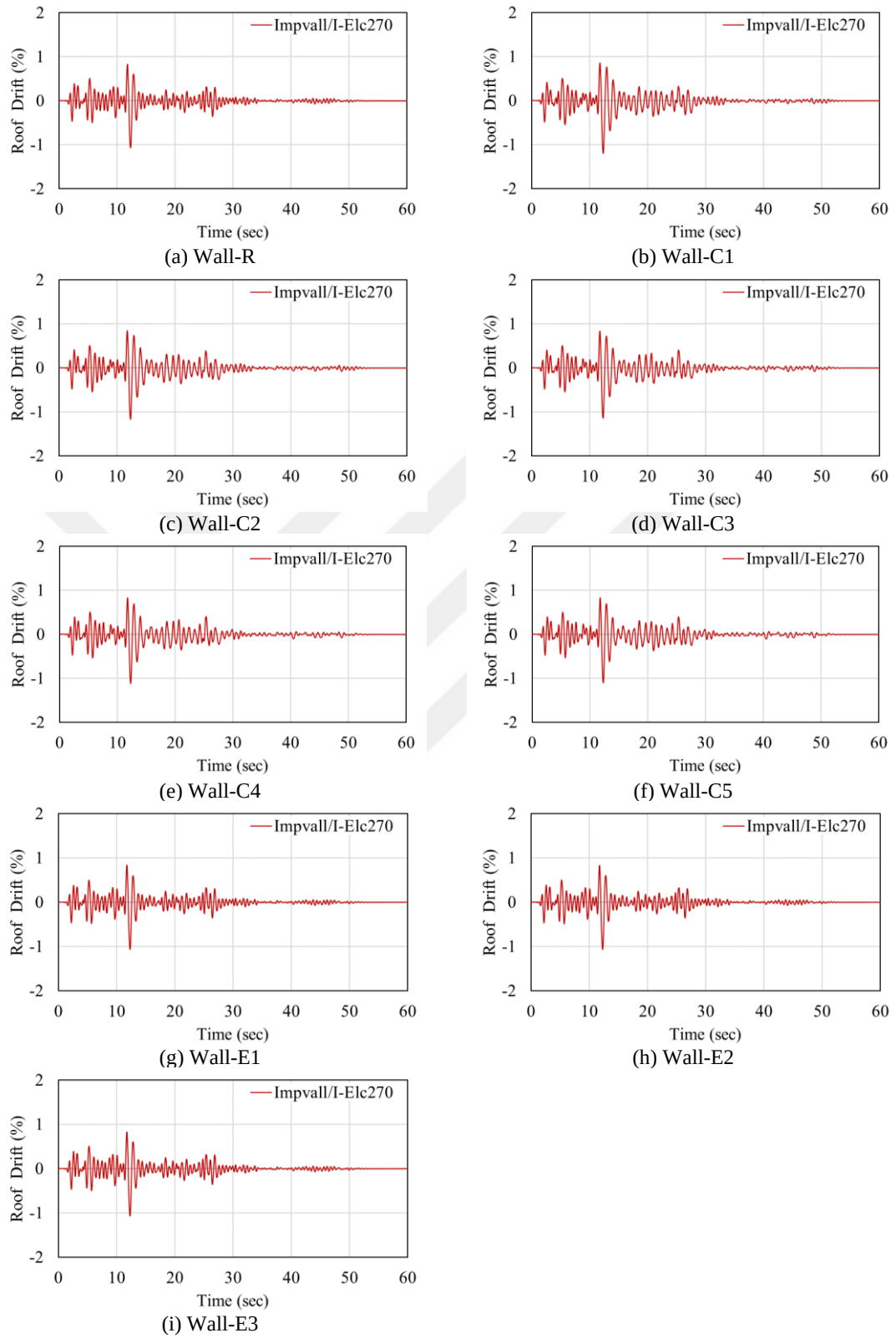


Figure 5-29: Roof drift vs time history behavior of the walls (DE3)

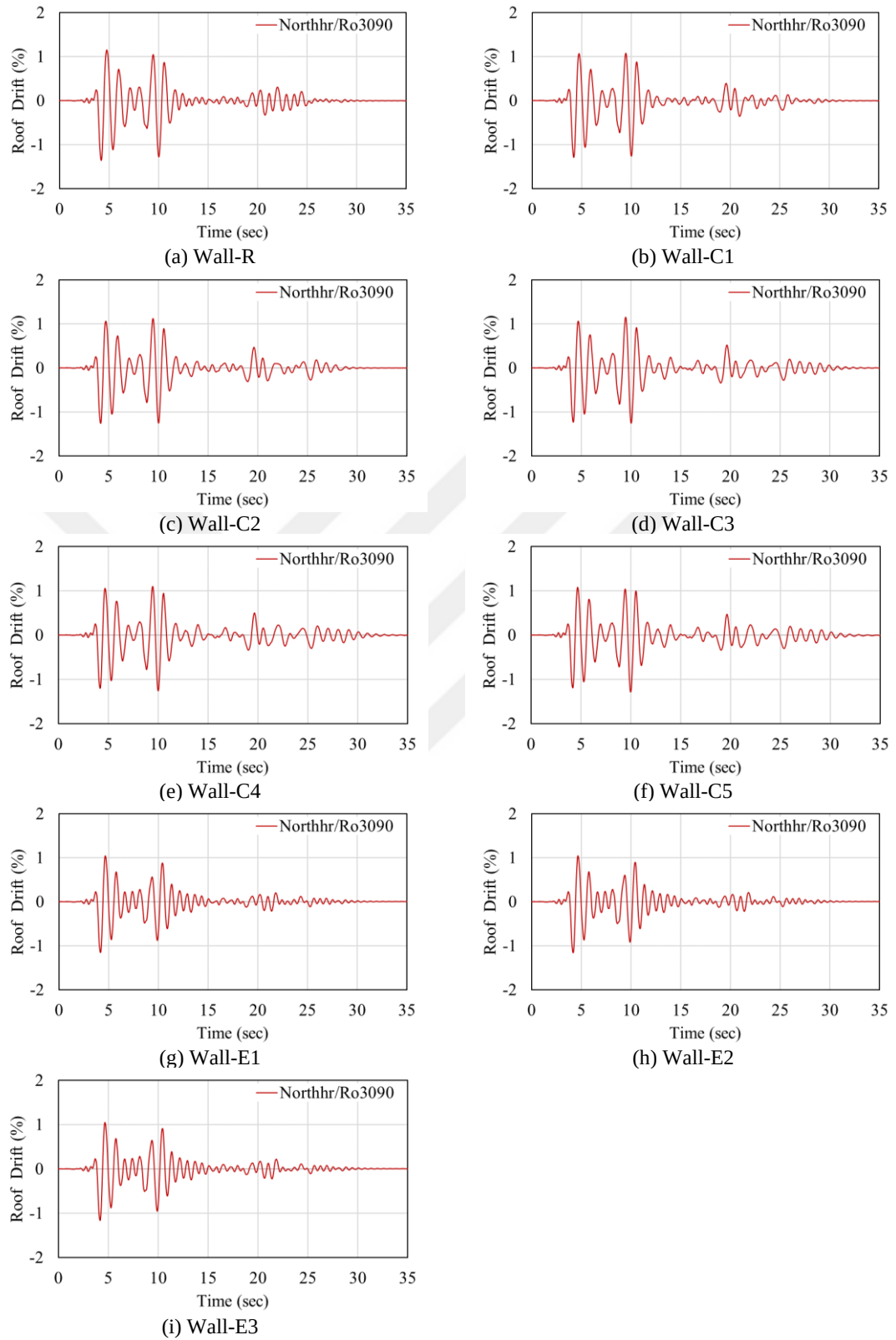


Figure 5-30: Roof drift vs time history behavior of the walls (DE4)

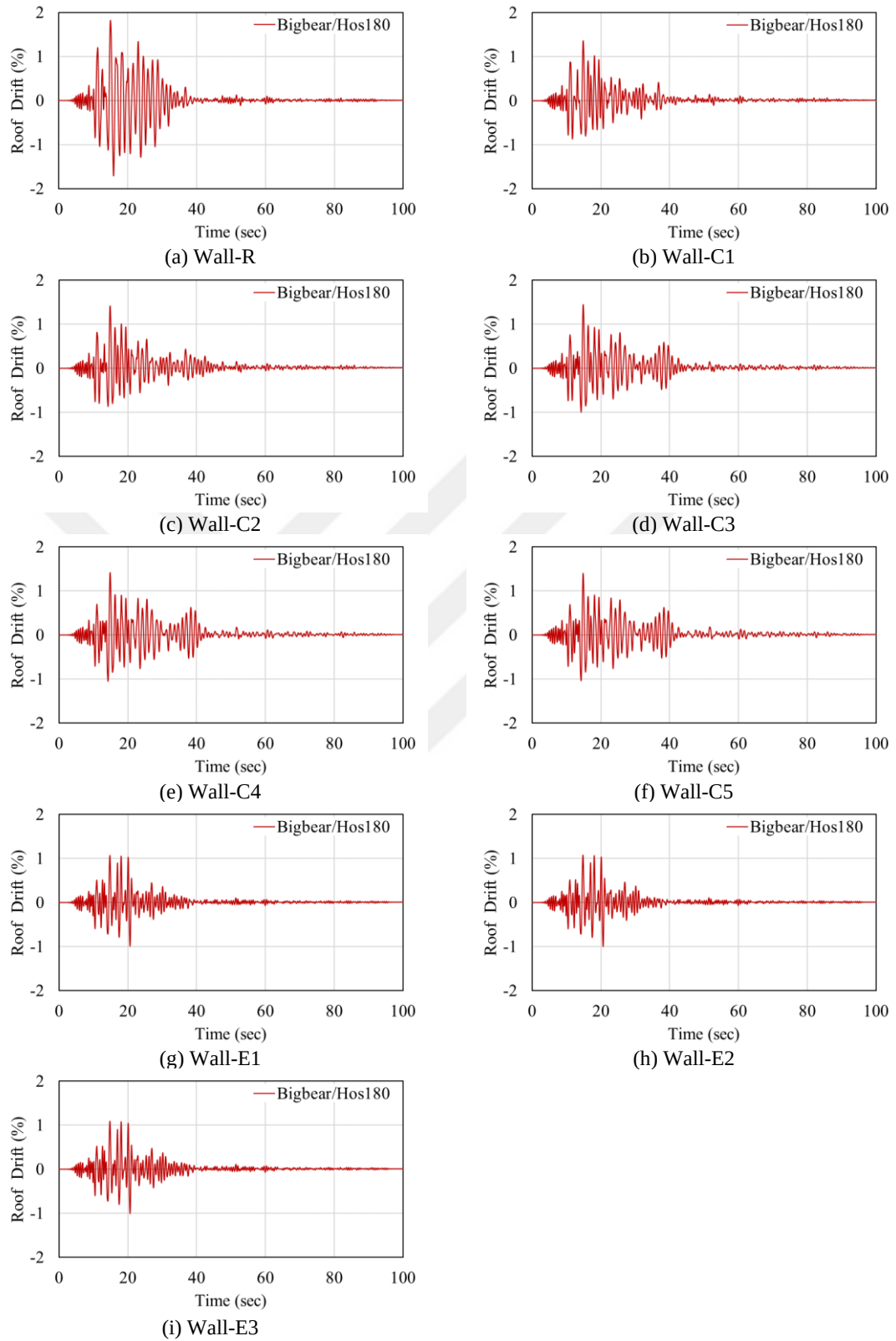


Figure 5-31: Roof drift vs time history behavior of the walls (DE5)

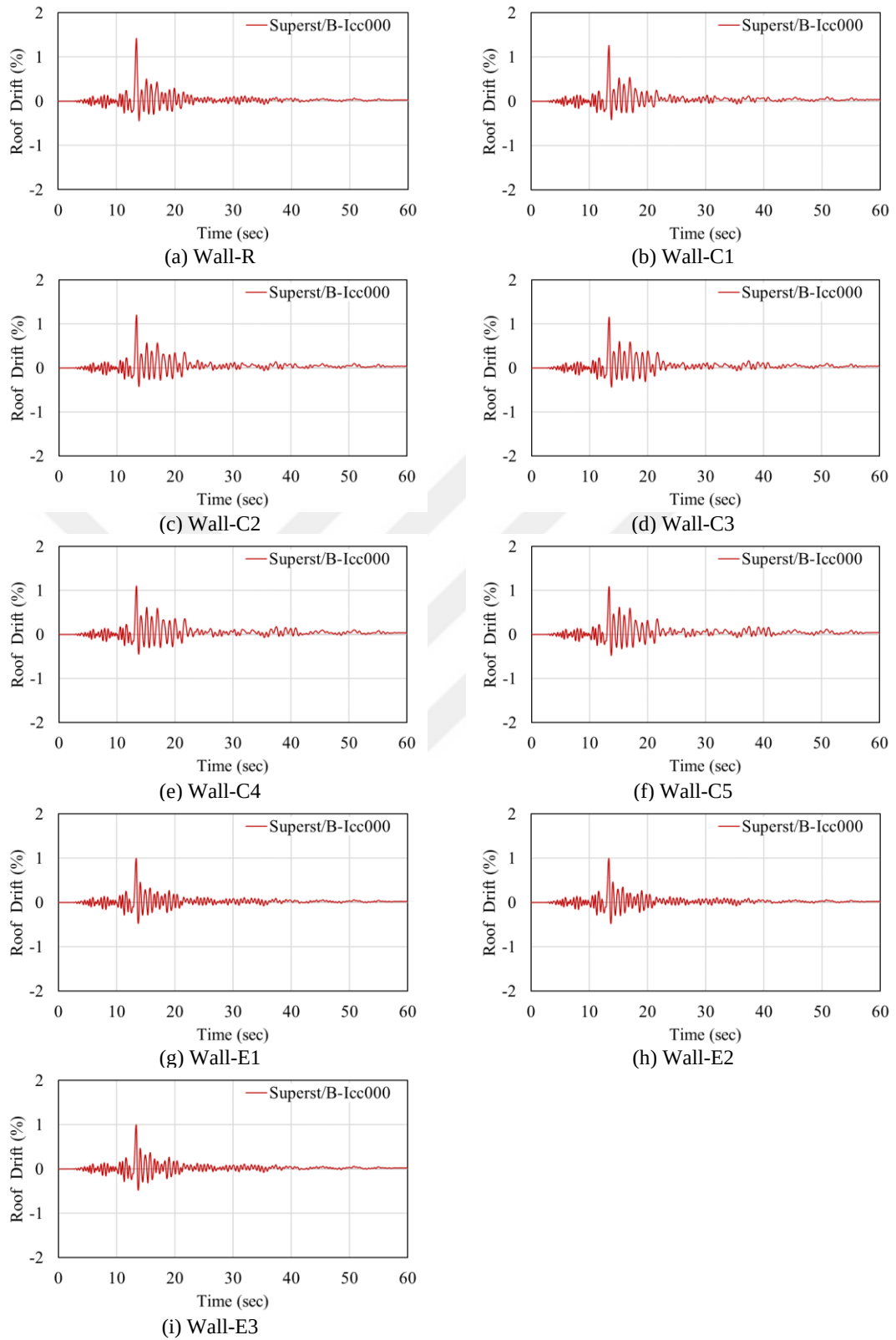


Figure 5-32: Roof drift vs time history behavior of the walls (DE6)

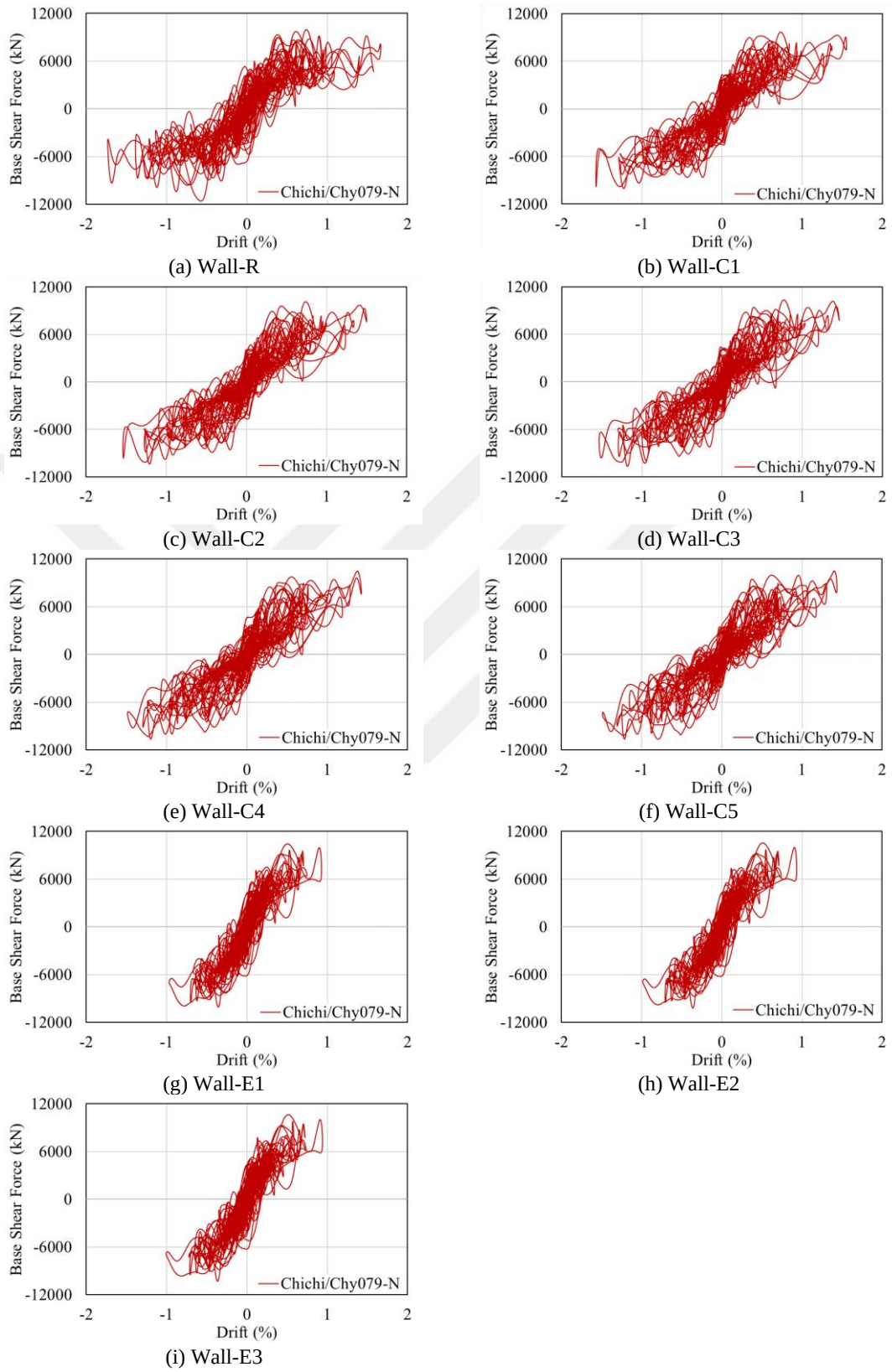


Figure 5-33: Lateral load resistance vs drift behavior of the walls (CE1)

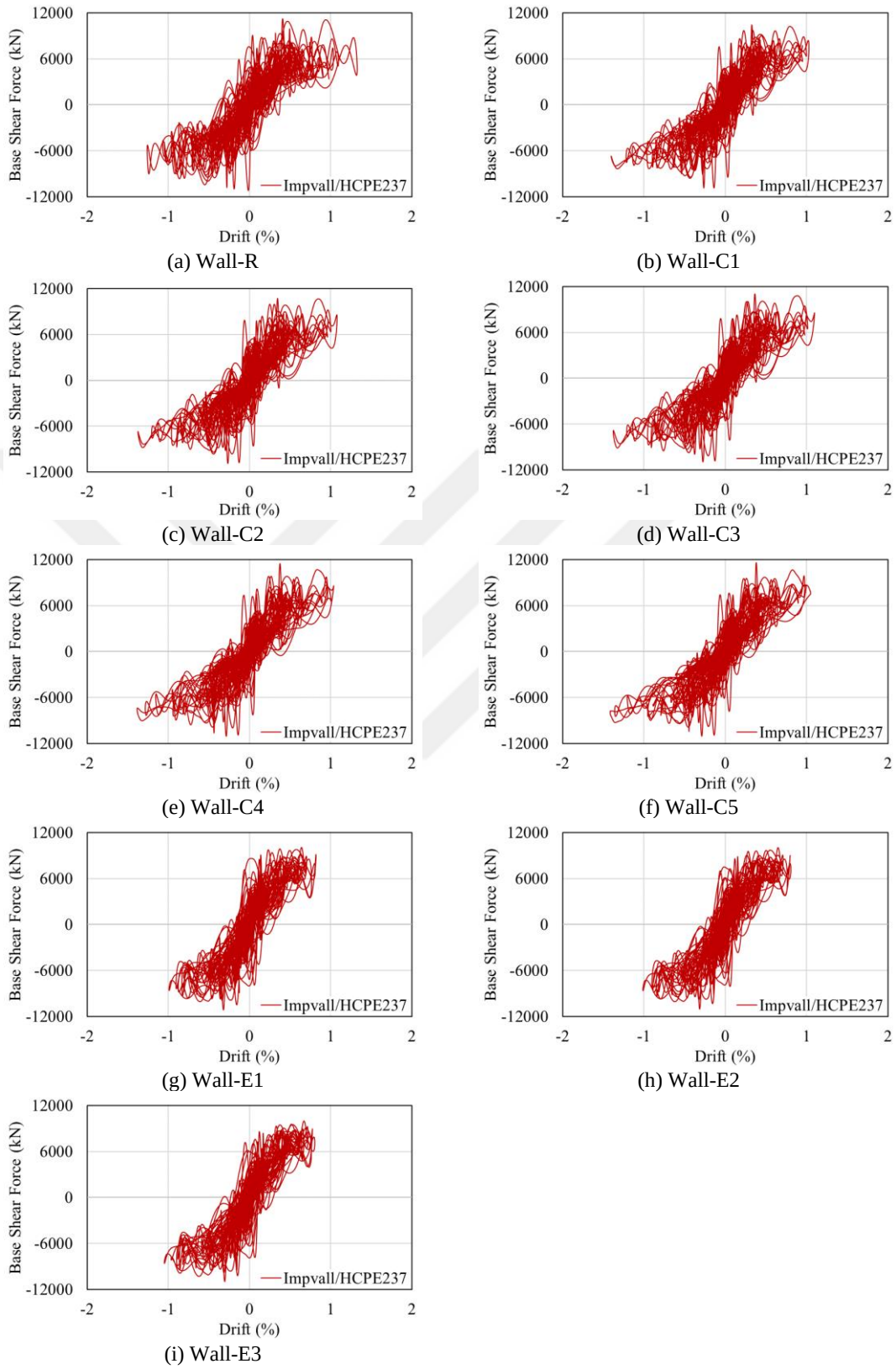


Figure 5-34: Lateral load resistance vs drift behavior of the walls (CE2)

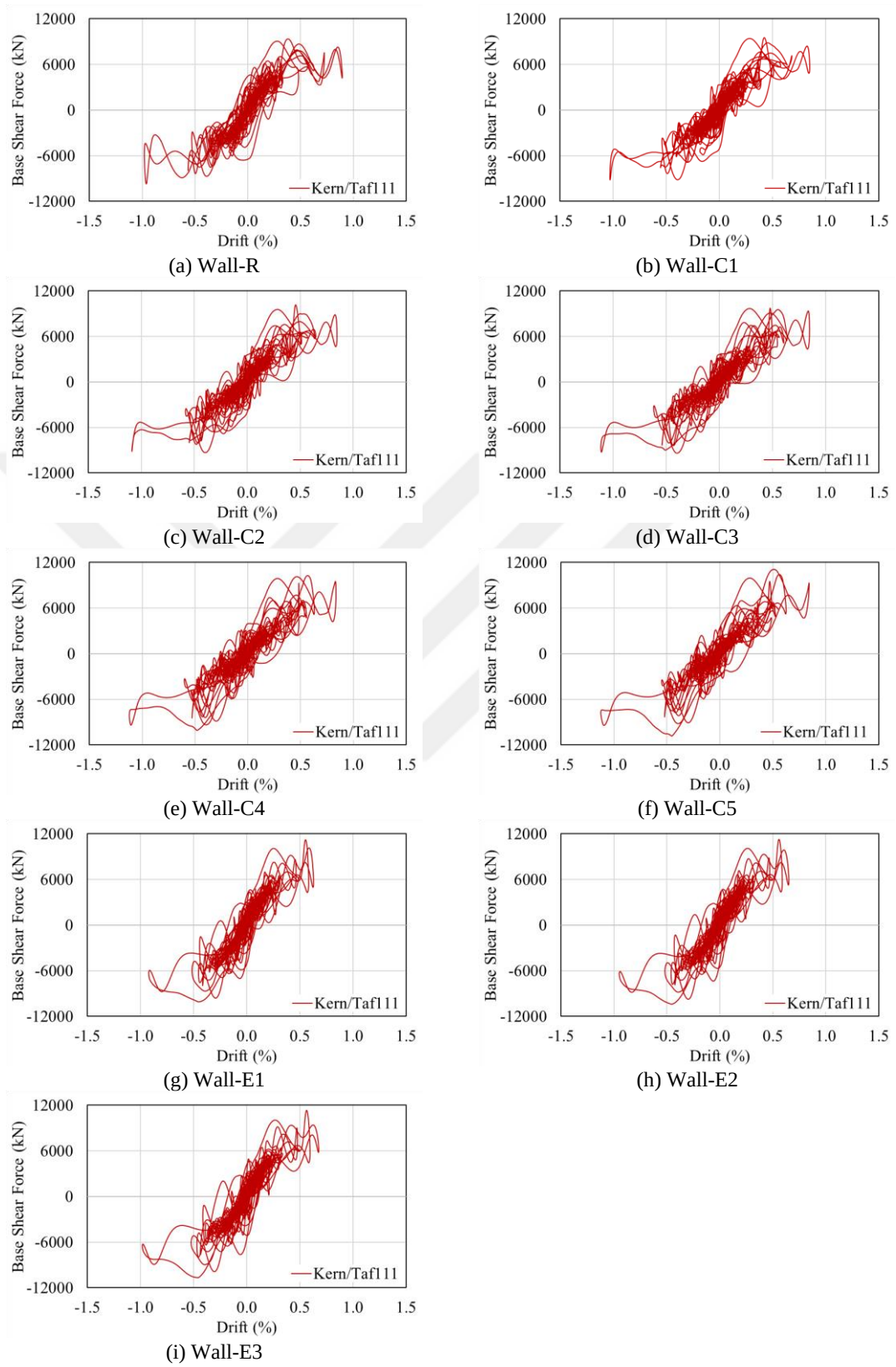


Figure 5-35: Lateral load resistance vs drift behavior of the walls (CE3)

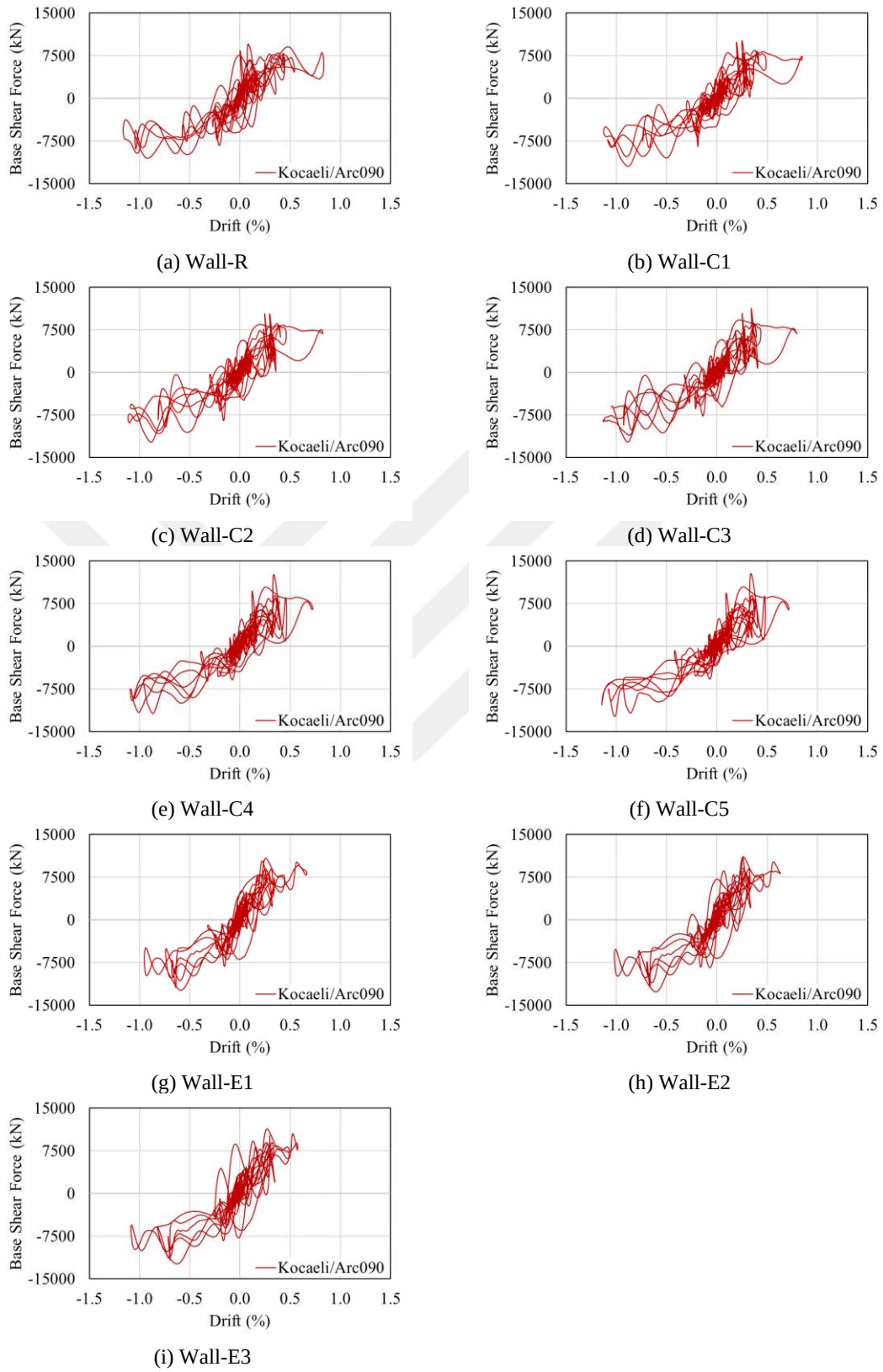


Figure 5-36: Lateral load resistance vs drift behavior of the walls (CE4)

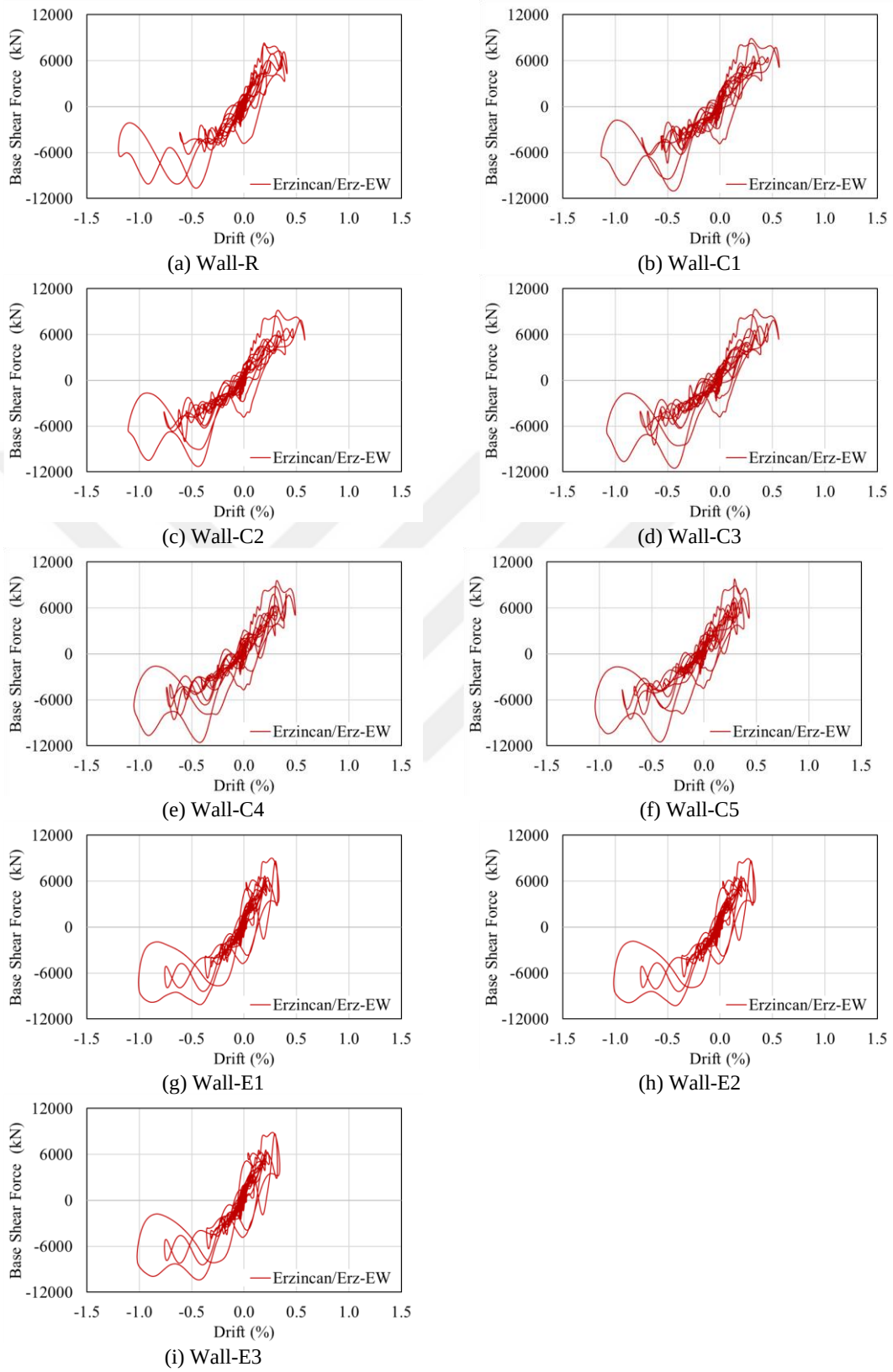


Figure 5-37: Lateral load resistance vs drift behavior of the walls (DE1)

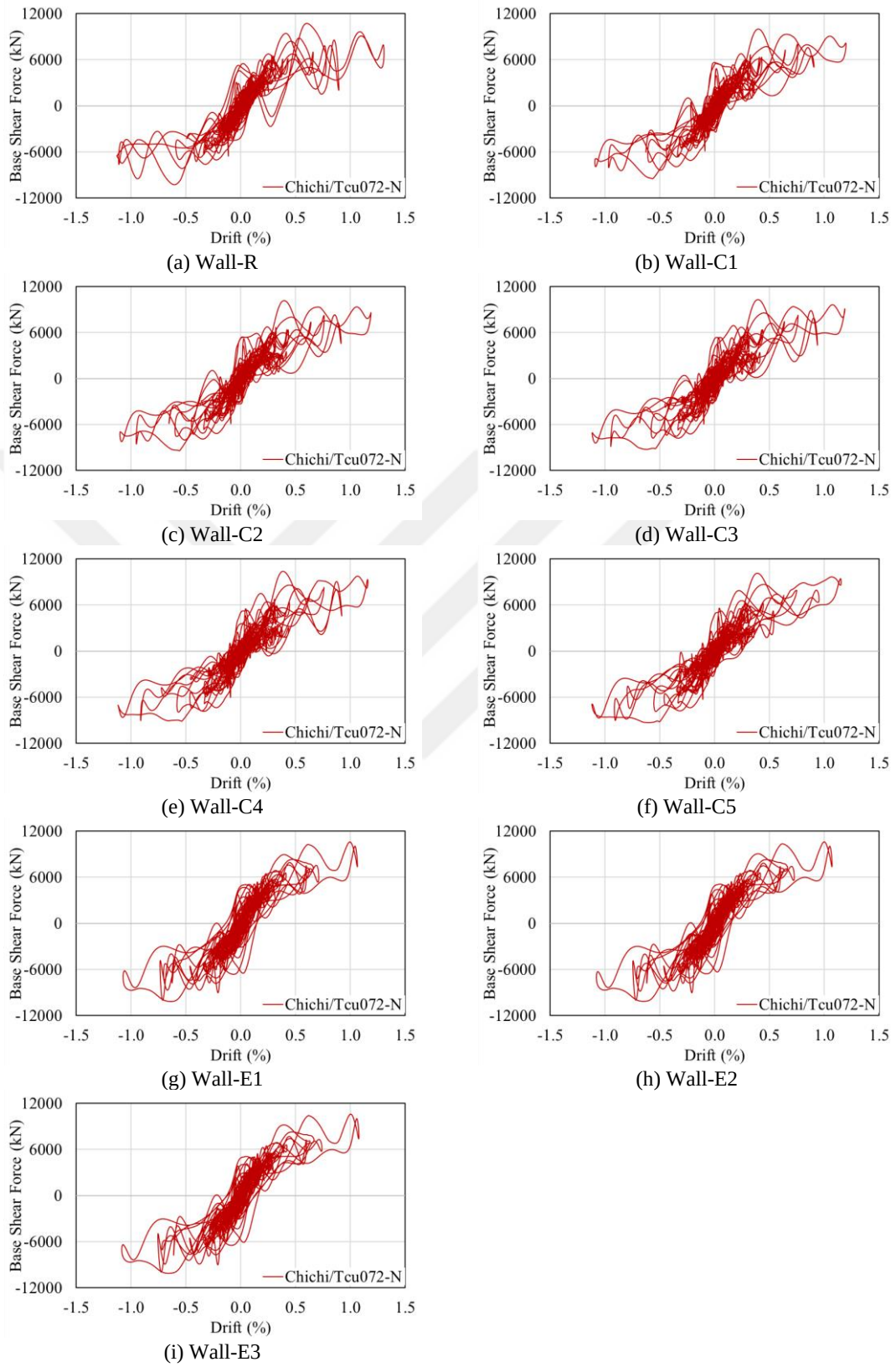


Figure 5-38: Lateral load resistance vs drift behavior of the walls (DE2)

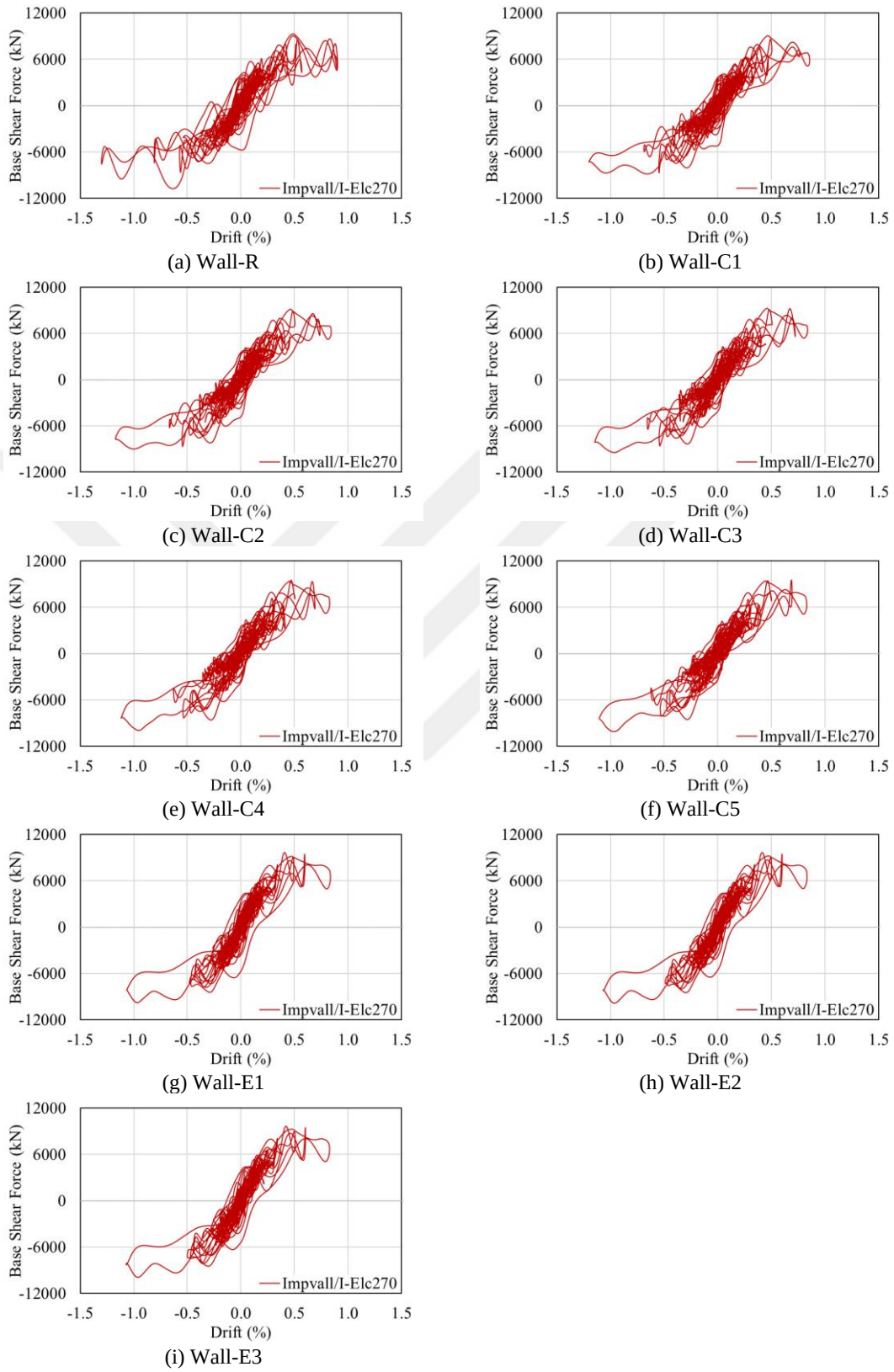


Figure 5-39: Lateral load resistance vs drift behavior of the walls (DE3)

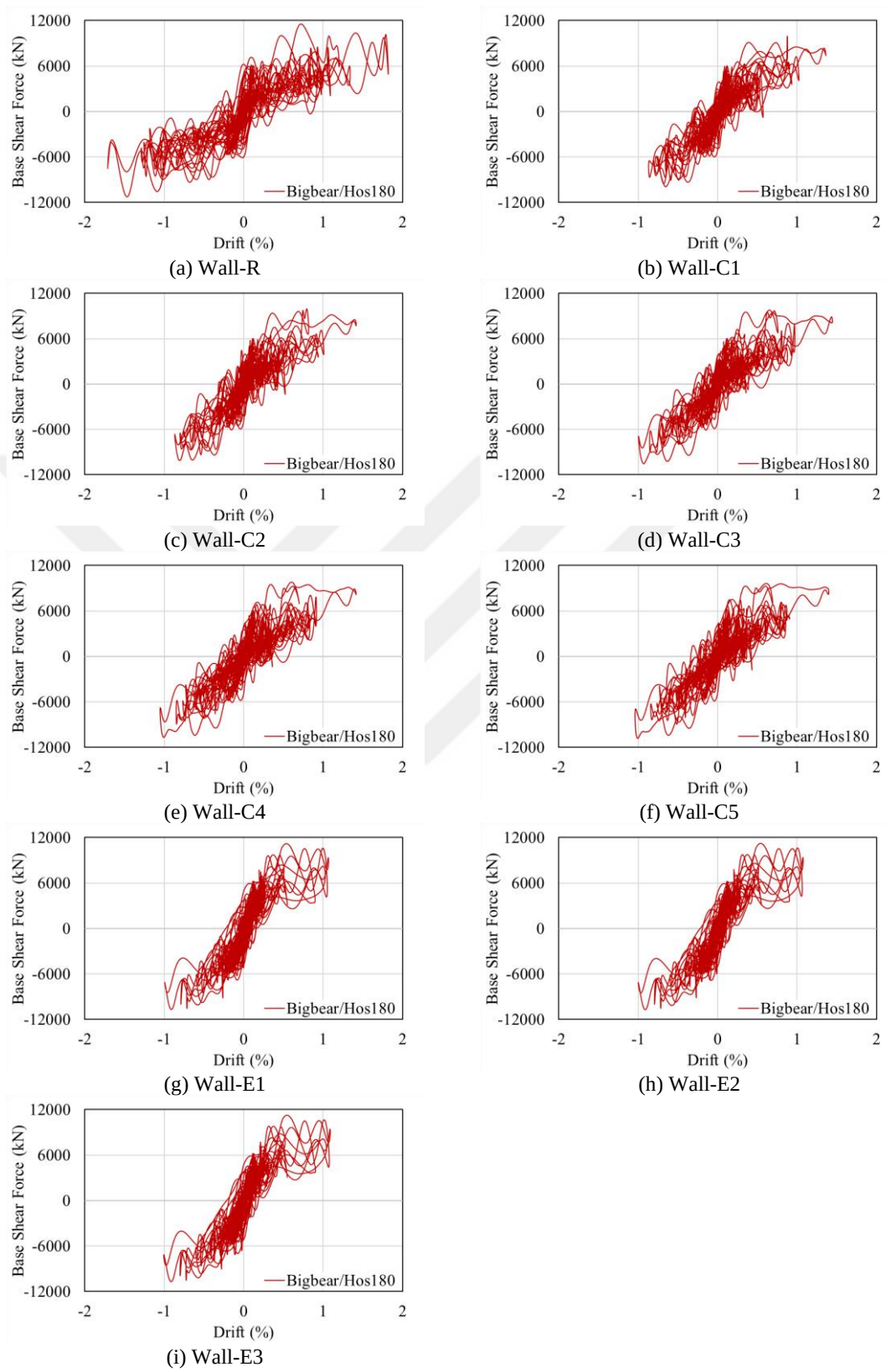


Figure 5-40: Lateral load resistance vs drift behavior of the walls (DE4)

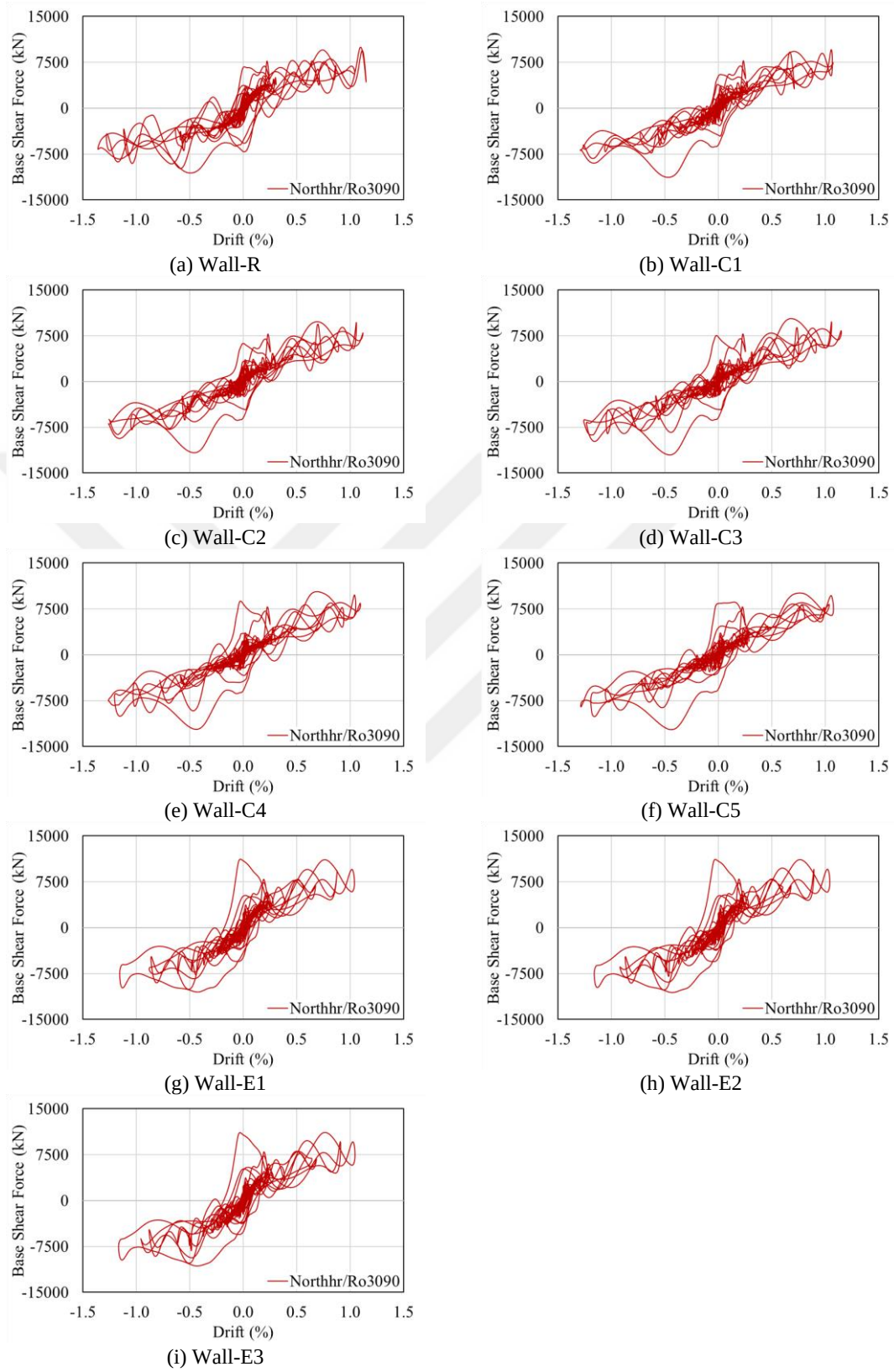


Figure 5-41: Lateral load resistance vs drift behavior of the walls (DE5)

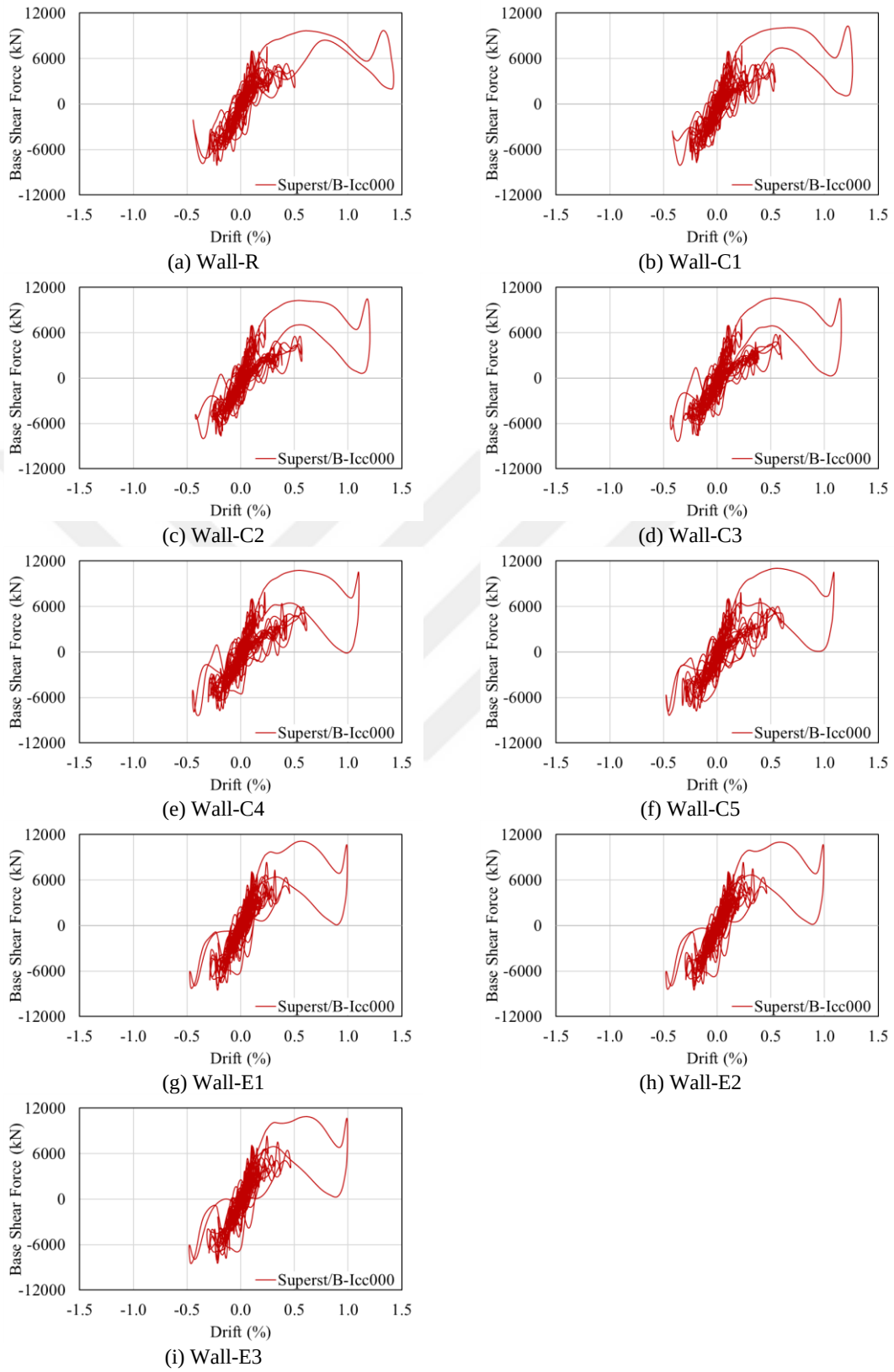


Figure 5-42: Lateral load resistance vs drift behavior of the walls (DE6)

CHAPTER VI

CONCLUSIONS

This chapter comprises a summary of the study including scope and purpose, the findings of the nonlinear static and dynamic analysis, and a discussion on the results regarding the effect of unbonded mild steel bars on the seismic behavior of the jointed precast concrete wall systems.

6.1 *Summary of the Thesis*

There is no sufficient study in the literature which focuses the effect of the unbonded mild steel bars such as amount, layout, and unbonded length on the seismic response of the unbonded post-tensioned precast reinforced concrete shear wall including self-centering ability, permanent deformation, stress losses on the unbonded tendons, and cumulative energy dissipation capacity. By virtue of the deficiency, this study aimed to investigate the amount, location, and wrapped length of the mild steel bars that were placed at the joint where gap opening occurs. Totally, eight prototype walls as well as a reference wall were designed in accordance with the recommendation of ASCE 7-16, ACI ITG-5.1, and 5.2. For five prototype walls, the mild steel bars were placed in the vicinity of the wall center with different amounts of mild steel of 30%, 45%, 60%, 75%, and 90%, respectively to investigate mild steel amounts on the behavior of the walls. Then, the three walls with mild steel bars placed at the edges were designed to have similar lateral load resisting capacity of the walls with 75% of mild steel bars in order to examine the effect of mild steel bar location. Besides, different unbonded (i.e., wrapped) lengths of the bars

were used for these three walls to evaluate the effect of the wrapped length on the response.

A numerical finite element model is created in OpenSees finite element analysis program and verified with the result of experimental study on a single precast wall and analytical study on a 5-story precast wall. By using a verified approach, totally nine designed prototype walls have been modeled. Nonlinear static pushover analysis, nonlinear reverse cyclic pushover analysis, and nonlinear time history analysis have been performed on each prototype wall model and effects of the different mild steel bar design parameters have been investigated by comparing with a reference wall without any mild steel bars.

Consequently, effects of the variety of the mild steel bars on the seismic behavior of the walls were reported based on the results of pushover and dynamic analysis including lateral load resisting capacity, hysteretic behavior, energy dissipation capacity, self-centering ability, and permanent deformation.

6.2 Overall Outcomes

All prototype wall models were tested under static pushover, reverse cyclic pushover and nonlinear dynamic loadings and base shear force versus drift, gap opening versus drift, total dissipated energy versus drift, post-tensioning stress losses versus drift, roof drift versus time history, and base shear force versus time history behaviors were monitored. Based on the obtained response, following deductions regarding the effects of mild steel bar design parameters on the seismic behavior of the walls can be done.

The prototype walls were pushed up to 3% of wall drifts and it was seen that;

- All walls have similar initial stiffness up to 0.25% drift however the capacity curves are differentiated for larger drifts.
- The fundamental period of the walls is not affected with amount, layout, and unbonded length of the unbonded mild steel bars.
- The ultimate load resisting capacity of the walls is boosted between 9.7% and 22.8% with the presence of the mild steel bars in the vicinity of the wall center and increasing the steel amount increases the capacity.
- The unbonded (i.e., wrapped) length of the mild steel bars has negligible effect on the lateral load resisting capacity.
- The gap opening at the base of the Wall-R is the highest but placing mild steel bars at the center and increasing bar area decreases the gap opening between 10.8% and 27.4%.
- Shifting the mild steel bars towards to the edges also decreases the gap opening by 6.2% to 7.7% but these walls fall behind the walls with mild steel bars at center.
- Heightening the wrapped length of the mild steel bars slightly increased the gap opening by nearly 1% but this difference is negligible.

The walls exposed to reverse cyclic drift history up to 3% wall drift in positive and negative directions and it was observed that

- The hysteretic behavior of the wall such as energy dissipation capacity, shape of the hysteretic loops, and loss on the tendon stress is significantly affected with the presence of mild steel bars.

- Permanent drift is affected but the difference is negligible, and walls preserved the self-centering ability of itself.
- The hysteretic loops became wider with the increasing of mild steel area, even wider with the placing mild steel bars at the edges of the walls. However, the wrapped length of the mild steel bars did not affect the shape of the hysteresis particularly.
- Wall-R had the smallest cumulative dissipated energy capacity but, the total dissipated energy was increased four times and eight times when mild steel bars are placed at center and edges respectively.
- Energy dissipation capacity of the walls increased with the increase of the mild steel bar area up to 75% of bar area however capacity of the Wall-C5 fell behind the Wall-C4.
- Shifting the bars toward the edges doubled up the total dissipated energy.
- The total dissipated energy was decreased by %2.7 to 5.2 when the wrapped length increased.
- Highest unbonded tendon stress loss of 89.7% was observed on the Wall-R. However, placing mild steel bars at the center and increasing the steel amount decreased the stress loss down to between 71.6% and 46.3%.
- Placing the mild steel bars at the edges also decreased tendon stress loss down to between 80.4% and 81% yet, the increasing the unbonded length did not change the stress losses significantly.

Eventually, the walls were subjected to ten ground motion histories and roof drift versus time history and base shear force versus time history behavior was monitored. Results from the dynamic analysis indicated that

- The change on the shapes of the roof drift-time history curves were clear.
- There was apparent widening on the hysteresis loops and decreasing on the drift when the base shear force versus time history behavior was examined.
- Maximum roof drift demands generally occurred at Wall-R. Presence of mild steel bars at the center was relatively decreased the maximum roof drift yet, the lowest maximum roof drift was achieved with mild steel bars at the edges.
- Increasing the mild steel area decreased the maximum roof drift in general.
- There was a slight increase in the maximum roof drift when the wrapped length of the bars increased but this difference can be neglected.
- The highest gap opening at the base of the wall occurred on Wall-R. However, placing the mild steel bars at the center and increasing bar area decreased the gap opening between 22.5% and 54.8%.
- Shifting the mild steel bars towards to the edges also decreases the gap opening by 7.4% to 12.7% compared with the Wall-C4.
- Heightening the wrapped length of the mild steel bars slightly decreased the gap opening, however this difference is negligible.
- Wall-R presented the minimum energy dissipation capacity. The total dissipated energy capacity of the walls increased by 4.6% to 22.8% with the presence of mild steel bars at the center as well as increasing the mild steel area.

- Shifting the mild steel bars towards the edges also increased the total dissipated energy more than two times that of walls with mild steel bars at the center. However, insignificant difference was observed when the unbonded length of the bars increases.
- The maximum tendon stress losses between 4.4% and 36.5% was observed on the Wall-R.
- Placing mild steel bars at the center and increasing the amount decreased the stress loss down to between 2.95% and 18.3%.
- The lowest stress losses between 3.1% and 7.6% were seen when the mild steel bars are placed at the edges of the walls.
- The tendon stress losses were slightly increased with the increasing of the debonded length of the mild steel bars. However, the difference between the stress losses were insignificant for all cases.

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