

DEVELOPMENT AND EXPERIMENTAL VALIDATION OF A NOVEL
NONLINEAR LAYERED SHELL ELEMENT FOR
RC STRUCTURAL WALLS

by

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To my mother, Süheyla Tura

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ABSTRACT

DEVELOPMENT AND EXPERIMENTAL VALIDATION OF A NOVEL NONLINEAR LAYERED SHELL ELEMENT FOR RC STRUCTURAL WALLS

Increasingly widespread use of nonlinear response history analysis methods for performance-based design/assessment of buildings with complex reinforced concrete (RC) wall systems has underlined the need for development and implementation of more robust nonlinear modeling approaches that is capable of accurately estimating the nonlinear response of structural walls at both global and local response levels, while also reflecting important (yet commonly-ignored) nonlinear response characteristics, such as shear-flexure-torsion interaction, plane sections not remaining plane, warping, out-of-plane behavior, nonlinear shear deformations, and shear stress migration across wall cross-section. Furthermore, such modeling methodologies for walls need to be implemented in widely-accessible computational platforms that are capable of efficiently simulating the dynamic response of entire three-dimensional structural systems under multi-directional seismic excitations. With this motivation, a novel nonlinear layered shell element, referred to as the Layered Fixed-Strut-Angle Finite Element (LFSAFE) model, was developed and implemented in OpenSees. In this study, the constitutive behavior and working principles of the LFSAFE model element are presented, the effectiveness and accuracy of the model is experimentally-validated, via simulation of test results for non-planar wall specimens subjected to quasi-static multi-directional lateral and torsional loading conditions. Additionally, nonlinear dynamic response simulation capabilities of the model are also evaluated, via comparison of model estimations with shake table tests results of a U-shaped wall specimen and a ten-story wall-frame building specimen. Comprehensive response comparisons provided in this study demonstrate that the LFSAFE model can accurately simulate salient response attributes of non-planar walls, at both global and local response levels, under different loading conditions, while also efficiently replicating experimentally-measured dynamic response of shake table tests, with reasonable accuracy.

ÖZET

BETONARME PERDE DUVARLAR İÇİN YENİ BİR DOĞRUSAL OLMAYAN KATMANLI KABUK ELEMANININ GELİŞTİRİLMESİ VE DENEYSEL DOĞRULAMASI

Doğrusal olmayan hesap yöntemlerinin taşıyıcı sistemi betonarme perde duvarlar içeren yapıların performansa dayalı tasarım/değerlendirme analizlerinde yaygınlaşan kullanımları betonarme perdelerin doğrusal olmayan davranışlarını hem global hem de yerel seviyede doğru bir şekilde tahmin edebilen, aynı zamanda kesme-eğilme-burulma etkileşimi, düzlem kesitlerin düzlem kalmaması, burulma altında eğilme, düzlem dışı davranış, doğrusal olmayan kesme davranışı ve kesit içinde kesme gerilmelerinin göçü gibi önemli karakteristik davranış özelliklerini de yansıtabilen gelişmiş doğrusal olmayan modelleme yöntemlerine olan ihtiyacı ortaya çıkarmıştır. Ayrıca bu özelliklere sahip modelleme yaklaşımlarının üç-boyutlu bina sistemlerinin deprem altındaki dinamik davranış simülasyonlarının etkin bir şekilde yapılabildiği erişilebilir hesaplama platformlarında mevcudiyetine ihtiyaç vardır. Bu amaçla, Sabit Payanda Açılı Katmanlı Kabuk (SPAKK) modeli olarak adlandırılan yeni bir doğrusal olmayan kabuk elemanı OpenSees isimli yazılım ortamında geliştirilmiştir. Bu çalışmada, SPAKK modelinin davranış özellikleri ve çalışma prensipleri sunulmuş olup, U ve H şeklinde kesitlere sahip betonarme perde numunelerinin yarı-statik çok-yönlü yatay yükleme ve burulma altındaki deney sonuçları ile kıyaslanarak, geliştirilen modelin güvenilirliği irdelenmiştir. Ek olarak, U şeklindeki bir perde numunesinin ve on katlı duvar-çerçeve tipi taşıyıcı sisteme sahip gerçek boyutlu bir binanın sarsma masası deneylerinden elde edilen ölçümler ile model analiz sonuçları kıyaslanarak modelin dinamik yükleme altındaki doğrusal olmayan davranışı kestirim kabiliyeti de tetkik edilmiştir. Sunulan detaylı karşılaştırmalarda, SPAKK modelinin bu çalışma kapsamında incelenen perde duvar numunelerinin önemli davranış özelliklerini hem global hem de yerel davranış seviyesinde ve farklı yükleme koşulları altında makul bir şekilde kestirebildiği, ve sarsma masası deneylerinde gözlemlenen dinamik davranışı kabul edilebilir bir hassasiyetle tahmin edebildiği gözlemlenmiştir.

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LIST OF SYMBOLS

A	Area of the collar beam section
A_g	Gross area of the wall section
d_{bl}	Diameter of the longitudinal reinforcing bar
D	Reinforcing steel diameter
E	Modulus of elasticity of concrete
E_b	Unloading stiffness from compression
E_c	Initial modulus of elasticity of concrete
E_{pl}	Concrete plastic modulus upon unloading from monotonic envelope
E_s	Modulus of elasticity of reinforcing steel
E_{sec}	Concrete secant modulus upon unloading from monotonic envelope
E_{target}	Slope of the compression yield asymptote
E_u	Unloading stiffness from tension
E_0	Initial modulus of elasticity of reinforcing steel
f'_c	Compressive strength of concrete
f_t	Tensile strength of concrete
f_u	Ultimate strength of reinforcing steel
f_y	Yield strength of reinforcing steel
$f_{y,bou}$	Yield strength of vertical reinforcing steel at boundary region
G	Shear modulus of concrete
h_w	Height of the wall specimen
I	Area moment of inertia of the collar beam section
I_{out}	Area moment of inertia of the wall section in out-of-plane direction
J	Polar moment of inertia of the collar beam section
l_w	Length of the wall specimen
L	Reinforcing steel buckling length
M	Section moment

P	Axial load on the specimen
r	Parameter defining shape of monotonic stress-strain curve for concrete
R_0	Initial value of transition curvature parameter for reinforcing steel model
t_w	Thickness of the wall specimen
u	Displacement field in horizontal direction
u_{layer}	Horizontal displacement field of a layer
U_1	Horizontal nodal displacement of element node 1
v	Displacement field in vertical direction
v_{layer}	Vertical displacement field of a layer
V	Section shear force
V_1	Vertical nodal displacement of element node 1
x_1	Coordinate of node 1 in model space x direction
x_2	Coordinate of node 2 in model space x direction
y_1	Coordinate of node 1 in model space y direction
y_4	Coordinate of node 4 in model space y direction
z_{layer}	Distance between a membrane layer and element centerline
α_1	Curvature degradation parameter for reinforcing steel model
α_2	Curvature degradation parameter for reinforcing steel model
γ_{xy}	In-plane shear strain
Δ_{crack}	Slip deformation along the crack surface
Δ_{dow}	Dowel deformations acting on horizontal and vertical reinforcement
$\Delta\varepsilon$	Reloading strain offset for concrete
Δf	Reloading stress offset for concrete
ε'_c	Concrete compressive strain at peak stress
ε_{cr}	Concrete strain after which monotonic stress-strain relation remains linear
ε_r	Maximum compressive strain from previous strain history
ε_{sh}	Hardening strain of reinforcing steel

ε_{su}	Ultimate strain of reinforcing steel
ε_{sy}	Yield strain of reinforcing steel
ε_t	Concrete tensile strain at peak stress
ε_x	Normal strain in x direction
ε_y	Normal strain in y direction
ε_0	Concrete strain at which the cyclic tension envelope originates
ζ	Horizontal coordinate in parent element
η	Vertical coordinate in parent element
η_f	Friction coefficient
θ_{crA}	Concrete crack direction A or first fixed strut direction
θ_{crB}	Concrete crack direction B or second fixed strut direction
θ_x	Rotation field about local x axis
θ_y	Rotation field about local y axis
θ_1	Joint in-plane rotation of element node 1
$\rho_{h,web}$	Horizontal reinforcement ratio in wall web
$\rho_{v,bou}$	Vertical reinforcement ratio in boundary region
$\rho_{v,web}$	Vertical reinforcement ratio in wall web
$\sigma_{c\perp}$	Compressive stress in concrete in perpendicular direction to the crack
$\sigma_{cy'}$	Concrete normal stress in y' direction
$(\sigma f_y)_{\perp}$	Clamping stress in reinforcement in perpendicular direction to the crack
σ_{mon}	Compressive stress on the modified compression envelope
σ_y	Reinforcing steel yield strength

LIST OF ACRONYMS/ABBREVIATIONS

AIJ	Architectural Institute of Japan
ASCE	American Society of Civil Engineers
CC	Crushing of Concrete
DC	Diagonal Compression
DFOS	Distributed Fiber-Optic Sensors
DKGQ	Discrete Kirchhoff Generalized Quadrilateral
DKQ	Discrete Kirchhoff Quadrilateral
DOF	Degree of Freedom
E	East
FB	Flange Buckling
FSAFE	Fixed-Strut-Angle Finite Element
FSAM	Fixed-Strut-Angle Model
GM	Ground Motion
GQ	Generalized Quadrilateral
JMA	Japan Meteorological Agency
KAERI	Korea Atomic Energy Research Institute
LATBSDC	Los Angeles Tall Building Structural Design Council
LFSAFE	Layered Fixed-Strut-Angle Finite Element
LI	Lateral Instability
LNEC	National Laboratory for Civil Engineering
LVDT	Linear Variable Differential Transformer
MLIT	Ministry of Land, Infrastructure, Transport and Tourism
N	North
OOPS	Out-Of-Plane Shear
PGA	Peak Ground Acceleration
QLMEDD	Quadrilateral Layered Membrane Element with Drilling Degrees of Freedom
QTFLS	Quadrilateral Thin Flat Layered Shell
RB	Reinforcement Buckling
RC	Reinforced Concrete

RCM	Reverse Cuthill-McKee
S	South
SP	Strain Penetration
SRSS	Square Root of Sum of Squares
TBI	Tall Buildings Initiative
UCSD	University of California San Diego
W	West
WC	Web Crushing



1. INTRODUCTION

1.1. General

Reinforced concrete (RC) structural walls are commonly used in building-type structures as the primary lateral-load resisting components of the structural system. They are not only effective in providing the necessary lateral stiffness for limiting the lateral displacements of the structure under wind actions or service-level earthquakes, but they are also designed and detailed to supply the structural system with sufficient lateral strength and inelastic deformation capacity (ductility) under moderate-level and severe earthquakes. With implementation of performance-based methodologies in modern seismic design codes and assessment/rehabilitation documents (e.g., ASCE 41 2023; LATBSDC 2023; TBI-II 2017), together with increasingly-complex wall systems used in modern-day design and construction (e.g., core walls with complex cross-sections), robust modeling of the nonlinear seismic response of RC structural walls has recently gained much importance among researchers since nonlinear response history analysis is the commonly-employed analysis method for seismic performance assessment of existing structures or performance-based design of special structures (e.g., tall buildings), and these analyses require efficient modeling methods for simulating the complex nonlinear behavior of a three-dimensional structural systems accurately.

Performance-based design/assessment involves comparison of deformation demands obtained from nonlinear dynamic analysis with limits (acceptance criteria) specified in design/assessment codes or supporting documents corresponding to different performance levels that are targeted in design/assessment. Therefore, it relies heavily on not only well-documented experimental observations that correlate levels of nonlinear deformations with levels of damage on structural components, but also the use of reliable nonlinear modeling methodologies that can accurately replicate the nonlinear behavioral characteristics of the structural components, as well as their interaction, under complex and multi-directional loading conditions induced by seismic actions on the entire three-dimensional structural system. However, in typical engineering practice, fiber-based modeling approaches for walls are still commonly-used in real-life performance-based design/assessment applications

worldwide. Fiber-based modeling approaches, which simulate primarily the nonlinear flexural behavior of walls (axial/flexural interaction), fail to represent important response characteristics of the wall response, including nonlinear shear/flexural/torsional interaction, nonlinear shear behavior, plane sections not remaining plane under lateral loading (shear lag), torsional warping of the cross-section, and out-of-plane behavior of wall segments. Within the framework of performance-based design and assessment, there is clearly a need for adoption of improved nonlinear modeling approaches for RC walls and wall systems.

1.2. Literature Review

Development and experimental validation of a robust modeling approach for accurate representation of structural wall nonlinear behavior has been a subject focused by the researchers for considerable amount of years. However, with improvements in computational power due to technological advancements, and widespread application of nonlinear analyses in performance-based design/assessment procedures, the number of studies in this field has increased noticeably in recent years. In the following two subsections, currently available quasi-static tests and modeling studies (mainly focusing on non-planar wall specimens), as well as shake-table experiments and nonlinear response simulation efforts for RC walls/wall type structures in the literature are summarized.

1.2.1. Quasi-static Tests and Modeling Studies on Non-planar Walls

RC structural walls are generally designed as the primary structural components of the seismic-resistant system of building-type structures located in regions of moderate or high seismicity. To date, a considerable amount of experimental research has been conducted and reported in the literature (e.g., see database by Abdullah and Wallace 2018) on planar walls subjected to in-plane loading. Numerous analytical studies have also been conducted on modeling of the nonlinear behavior of planar walls, using either macroscopic (e.g., fiber-based) modeling approaches (e.g., reported by Kolozvari et al. 2018) that do or do not consider nonlinear shear-flexure interaction behavior, or using microscopic (e.g., finite element) modeling approaches (e.g., reported by Kolozvari et al. 2019) that inherently account for coupling of axial/shear/flexural responses of walls. Comparatively fewer experimental campaigns have been conducted to investigate the nonlinear behavior of non-

planar RC walls subjected to uni/bi-directional, torsional, or combined lateral-torsional loading conditions (e.g., Maruta et al. 1999, 2000a; Habasaki et al. 2000; Ono et al. 2004; Kitada 2007; Beyer et al. 2008a; Li and Li 2012; Constantin and Beyer 2016; Behrouzi et al. 2020, Hoult and Beyer 2021; Hoult et al. 2023a). Non-planar walls with complex cross-sectional geometries are typically used as core wall components in tall building construction, as well as in mid-rise buildings around the perimeter of elevators and staircases. The nonlinear behavior of non-planar walls under seismic actions is significantly more complex than that of planar walls, due to various reasons. First, it has been experimentally-observed (e.g., Constantin and Beyer 2016; Hoult 2019) that plane sections not remaining plane (due to shear lag effects) is more pronounced in non-planar walls subjected to multi-directional lateral loading. As well, shear stresses on non-planar walls migrate across the wall cross section, depending on whether an individual wall segment (e.g., flange) is subjected to axial/flexural compression or tension, where wall flanges subjected to larger axial/flexural compression resist larger internal shear forces, and larger shear deformations/strains develop in flanges subjected to resultant axial/flexural tension (e.g., Beyer et al. 2008a). Furthermore, in buildings that incorporate torsional irregularity, non-planar walls can be subjected to significant torsional demands together with flexural and shear demands. The nonlinear torsional behavior of a non-planar wall is inherently coupled with both its shear and flexural modes of behavior (Maruta et al., 1999, 2000a; Hoult and Almeida, 2024a), since twisting of the wall induces not only shear strains but also longitudinal strains (due to warping) on the wall cross section.

In contrast, industry-standard nonlinear modeling approaches that are commonly used in engineering practice for performance-based design/assessment applications typically do not consider any of these salient behavioral characteristics of non-planar walls. In typical engineering practice, similarly to rectangular or barbell-shaped walls, non-planar walls are modeled using a fiber-based approach, assuming that plane sections remain plane and that the nonlinear flexural behavior of the wall is uncoupled from its shear and torsional responses. Within the framework of performance-based design and assessment, there is clearly a need for adopting improved nonlinear modeling approaches for RC walls, even more so for non-planar walls.

Some of the research studies available in the literature on analytical modeling of non-planar walls, where the models are experimentally-validated against quasi-static test results, are summarized herein. Maruta et al. (1999, 2000a, 2000b) employed a layered shell model element (Hayami et al. 1991) for response simulation of H-shaped walls under combined lateral and torsional loading, focusing on reproducing global load-displacement and torque-twist responses only. Ile and Reynouard (2005) analyzed U-shaped walls under uni- and bi-directional lateral loading, using a three-dimensional multi-layer thin shell finite element formulation. Mazars et al. (2006) validated a multi-fiber beam element modeling approach using test results for a U-shaped wall. Beyer et al. (2008b) developed an inelastic wide-column model formulation and validated it against test results for U-shaped wall specimens. These studies did not include simulation of the torsional behavior of U-shaped walls, and model results were evaluated in terms of global load-displacement behavior only. Lu and Panagiotou (2014) developed a three-dimensional beam-truss model and validated it against test results for three non-planar walls. The beam-truss model was further improved and validated using test results for four planar and four non-planar walls (Lu et al. 2016). These papers also addressed local response prediction capabilities of the beam-truss model, although to a limited extent. More recently, Feng et al. (2018) proposed a multi-layer shell element to simulate the nonlinear response of planar and non-planar structural walls. Experimental validation of this model for non-planar walls was conducted mostly at the global load-displacement level, using test results for a T-shaped wall subjected to uni-directional lateral loading, as well as a U-shaped wall tested under bi-directional lateral loads (Constantin and Beyer 2016). Rojas et al. (2019) developed a nonlinear layered shell element, and compared model predictions with test results at both global and local response levels (strains, flexural and shear deformation components) for T-shaped wall specimens loaded uni-directionally. This model has not yet been experimentally-validated for U-shaped walls under bi-directional lateral or torsional loading. Kolozvari et al. (2021a, 2021b, 2022) developed (and implemented in OpenSees; McKenna et al. 2000), various modeling approaches for non-planar walls; including a fiber-based nonlinear flexural model, a fiber-based nonlinear shear-flexure interaction model, and single-layer shell model. These models were validated against test results for U-shaped walls tested under bi-directional lateral loading. Modeling of the torsional behavior of non-planar walls was not considered in these studies. Wang et al. (2023) used the finite element analysis software ABAQUS (2016) for modeling of T-shaped and two L-shaped wall specimens they have tested under uni- and bi-

directional lateral loading conditions. Model results were compared with test data in terms of the lateral load-displacement response of the wall specimens. In the study by Ahmed et al. (2023), a nonlinear continuum-type modeling approach was developed and experimentally-validated against test results for non-planar walls subjected to uni- and bi-directional lateral loading. Comparison of model results and test measurements was limited to only the backbone curves of the test results. Hoult et al. (2023b) investigated the effectiveness of the beam-truss modeling approach (Lu et al. 2014; 2016) in replicating the global load-displacement and torque-twist responses of U-shaped walls tested under uni- and bi-directional lateral loading, as well as pure torsion.

1.2.2. Shake-Table Tests and Dynamic Response Simulation Studies on Walls and Wall Systems

Shake table tests presented in the literature on isolated wall specimens, either planar (e.g., Carillo and Alcocer, 2012, 2013; Zhang et al., 2019) or non-planar (e.g., UWS1; Almeida et al., 2024; Hoult et al., 2024), are very scarce. Comparatively, a larger number of research studies were conducted on shake table testing of wall systems or wall-frame systems, some of which are very briefly summarized herein. Panagiotou et al. (2011) presented shake table test results for a seven-story full-scale building slice tested at the University of California San Diego shake table facility. The wall-frame specimen was excited in one direction (parallel to the wall web wall), under four ground acceleration histories. Other researchers (Chen et al., 2016; Pantoli et al., 2016a, 2016b) also conducted shake table tests at the UCSD facility on a five-story full-scale building slice, to correlate the seismic performance of the structural system and the non-structural components, where the specimen was tested under isolated-base and fixed-base configurations. Nagae et al. (2011, 2014) tested the E-Defense 2010 full-scale four-story wall-frame buildings with reinforced concrete and post-tensioned walls, under scaled ground motions recorded during the 1995 Hyogoken-Nanbu earthquake, for evaluating existing design recommendations, developing new design approaches, and more accurately assessing building performance. The full-scale ten-story E-defense 2015 reinforced concrete wall-frame building was tested by Kajiwarra et al. (2017), Sato et al. (2017), and Tosauchi et al. (2017), under different amplitudes of the JMA-Kobe record, under base-slip (sliding base) and fixed-base configurations. A similar E-defense test campaign was conducted in 2018, with an improved

design for the beam-column connections of the ten-story building (Kang et al. 2020). The 2010, 2015, and 2018 E-defense shake table test programs were conducted under three-directional ground accelerations. He et al. (2020) performed shake table-tests on two identical nine-story wall systems, aimed to provide benchmark data to be used for improving modeling approaches. Jung et al. (2021) conducted shake table tests at the KAERI facility on a three-story wall system comprising a U-shaped wall and slabs at story levels, under varying intensities of 2016 Gyeongju earthquake ground motion record, and discussed the vibration response characteristics of the specimen, which remained mostly in the linear elastic response range during testing.

Analytical studies presented in the literature on developing advanced modeling approaches for RC walls, where the models were experimentally-validated against shake table tests on wall systems, are also relatively scarce. Some of the relevant research efforts are summarized herein. Grange et al. (2009) presented a blind prediction of the shake table test of the seven-story building slice tested at USCD, using Timoshenko multi-fiber beam elements. Martinelli and Filippou (2009) presented blind prediction results for the same test, using the beam-column elements in OpenSees adopted for walls. Waugh and Sritharan (2010) used the same experimental campaign to validate a so-called two-dimensional centerline model, and concluded that using a two-dimensional fiber-based modeling approach was sufficient to predict the response characteristics of the walls under uni-directional dynamic loading. These studies did not address modeling of non-planar wall response and the effect of multi-directional loading on a wall system. The three-dimensional beam-truss model (Lu and Panagiotou, 2014; Lu et al., 2016) was used to simulate a five-story coupled T-wall-beam-slab system tested under bi-directional ground motion (Lu and Panagiotou, 2014). It was concluded that the model is effective, yet needs to be further studied in terms of simulating the dynamic response of complex structural systems under three-dimensional seismic excitations, at both global and local response levels. A quadrilateral flat nonlinear layered-shell element was developed and implemented in OpenSees (McKenna et. al., 2000) by Lu et. al. (2015), and the model was used to simulate the seismic response of a 142m-high core wall-frame building designed based on the Chinese building design code, as well as a 632m-high super-tall core wall building (Shanghai Tower), for which OpenSees documentation was also provided as a benchmark study. Later, an improved version of the model, which incorporated an updated Lagrangian formulation

for geometric nonlinearity, was developed and implemented in OpenSees (Lu et.al., 2017), and seismic response of the same 142m-high core wall–frame building was simulated. In both studies, model results were validated against the nonlinear finite element analysis software MSC Marc (MSC Software, 2007). Subsequently, a triangular nonlinear shell element and an explicit solution algorithm were developed and implemented in OpenSees by (Lu et. al., 2019). Analysis results for the same structure, where only the coupling beams were modeled using the triangular shell element, were compared using different implicit and explicit analysis methods. The models by Lu et. al (2015, 2017, 2019) were validated against quasi-static tests conducted on planar RC walls. However, experimental validations presented in these papers are limited and at the global load–displacement response level only. The models were not validated against quasi-static or shake table test results conducted on non-planar walls or wall systems subjected to multi-directional loading or excitations. However, the quadrilateral layered-shell model element by Lu et al. (2017) was later adopted by Sun et al. (2023) in simulating the response of the 2015 E-Defense 10-story wall-frame building specimen, and model results were compared with test measurements, also in terms of local (deformation) responses. Ebrahimian et al. (2018) utilized the modified compression field theory (Vecchio and Collins, 1986) implemented in the finite element analysis software DIANA (TNO DIANA BV, 2011), for simulating the fixed-base five-story full-scale building slice tested at UCSD by Chen et al. (2016) and Pantoli et al. (2016a, 2016b). Eight-node Mindlin-Reissner quadrilateral serendipity shell elements were used for the slabs and shear walls, and uncertainties in the simulated stemming from the constitutive parameters of the material model were identified. Rahman et al. (2022) simulated the three-story wall structure tested at the KAERI (Jung et al., 2021), using layered-shell, fiber beam-column, and beam-truss elements available in OpenSees, for identifying the most consistent approach for nonlinear analysis of wall systems under dynamic loading, although they were unable to identify an optimum modeling methodology. Xiao et al. (2024) investigated the influence of different modeling assumptions in ABAQUS (2014) on response simulation of three-story wall structure tested by Jung et al. (2021), where the concrete elastic modulus and foundation flexibility were the primary parameters of investigation. They also emphasized the importance of system identification and model updating towards improving the accuracy of analysis results. More recently, Isaković et al. (2023) co-edited a special issue on simulation of the 2015 and 2018 E-defense full-scale ten-story wall frame buildings with sliding-base and fixed-base configurations. In this special issue, various response simulation studies were

published, where different analytical modeling approaches for walls (both macroscopic and finite element) were utilized. Most recently, Ghosh et al. (2024) simulated the 2010 E-defense four-story wall-frame building shake table tests, using the previously-validated (e.g., Dashti et al., 2017) curved shell element in DIANA (2022) for the walls and slabs, compared model results with test measurements at both global and local (deformation) response levels, and suggested further experimental validation of the model against additional shake table tests.

Robust finite element wall models that are available in commercial analysis software (e.g., DIANA (2024), LS-DYNA (2024), ADINA (2024), etc.) have recently been finding their way into real-life design applications. However, problems related to modeling of entire structural systems and the lack of flexibility in calibration or updating of the material constitutive models, apply to most of the readily-available finite element wall models. As well, convergence rates and run times of such generally impair their use in three-dimensional analysis of large and complex structural systems, as required in real-life performance-based seismic design applications. On the other hand, use of research-oriented finite element wall models is generally restricted to a narrow research community mostly involved in their development, whereas models that are accessible by all, through research-oriented computational platforms (e.g., OpenSees, McKenna et al., 2000), are extremely few (e.g., Lu et al.; 2015, 2017, 2019).

1.3. Objectives and Scope

As outlined above, studies in the literature on analytical modeling of non-planar walls, in which the models are experimentally-validated at not only global but also local response levels, against experimental results for wall specimens subjected to various loading conditions, are scarce. The existing level of knowledge on the topic is subject to improvement, in terms of development/validation of a new modeling approach (or validation of an existing modeling approach) for non-planar walls, where the model is extensively validated at both global (load-displacement, torque-twist) and local (shear and flexural deformations, strain distributions) response levels, considering experimental results for wall specimens subjected to uni-directional lateral, bi-directional lateral, torsional, and combined lateral/torsional loading. As well, availability of such a model formulation in open

computational platforms would greatly improve modeling capabilities of both researchers and practicing engineers, for conducting more reliable nonlinear response history analysis for (and performance-based design/assessment of) structural systems that incorporate complex non-planar wall configurations.

With this motivation, this thesis aims to contribute to the existing level of knowledge and capabilities on three-dimensional modeling and nonlinear seismic response simulation of walls or wall systems, by presenting a new layered-shell model element for RC walls, referred to as the Layered Fixed-Strut-Angle Finite Element (LFSAFE) model, developed and implemented in OpenSees (McKenna et al., 2000) software environment. The objectives of this thesis also include presentation of a detailed and comprehensive experimental validation by simulating the hysteretic response characteristics of non-planar RC walls tested under various quasi-static loading conditions, using this novel shell element, and demonstration of the overall nonlinear dynamic response simulation capabilities of the LFSAFE model via comparison of model predictions with experimentally-measured dynamic response from shake table tests, for both isolated walls and wall/frame systems, under multi-directional seismic excitations. However, it is important to note that, the present study does not aim to provide a detailed evaluation of the accuracy of the LFSAFE model in predicting shake table test results, since other factors irrelevant to reliability of the adopted shell element in simulations (e.g., assumed damping) are also expected to influence the accuracy of the dynamic analysis results.

This thesis firstly provides a detailed description for the constitutive behavior and working principles of the developed LFSAFE model. Then, comparison of response simulations obtained using this model with results of carefully-controlled quasi-static tests presented in the literature, conducted on heavily-instrumented non-planar wall specimens subjected to various loading conditions, are provided. The test database considered for experimental validation of the model includes four U-shaped walls tested under uni-directional lateral loading (UW1; Hoult et al. 2023a), pure torsional loading (UW2; Hoult et al. 2023a) and bi-directional lateral loading (TUB; Beyer et al. 2008a, TUC; Constantin and Beyer 2016); as well as with six H-shaped walls tested under combined lateral/torsional loading conditions that correspond to various torque-to-maximum-bending-moment ratios (HW25, HW50, HW75, HS25, HS50, HD25; Maruta et al. 1999, 2000a, 2000b). Model

response simulations are compared with test results at both global (e.g., lateral load-displacement, torque-twist) and local (deformation) response levels (e.g., contributions to flexural and shear deformations to top displacement, vertical strain distribution across wall cross-section, vertical strain profile along wall height), in all cases where local deformation measurements are available. Furthermore, nonlinear dynamic response prediction capabilities of the model are demonstrated via response simulations of: (i) a bi-directional shake table test conducted very recently on a U-shaped wall specimen (UWS1; Almeida et al., 2024; Hoult et al., 2024) and, (ii) the 2015 E-Defense three-directional shake table tests conducted on a full-scale ten-story wall/frame building specimen (Kajiwarra et al., 2017; Sato et al., 2017; Tosauchi et al., 2017). Comparisons for experimental response from the shake table tests with analysis model estimations are presented at both global (e.g., top displacement/top twist/story displacement/story acceleration/base shear time histories, story drift profiles, base overturning moment vs. roof displacement response) and local (e.g., vertical strain/shear distortion/relative rotation/beam end rotation/column end rotation time histories) response levels. Lastly, upon the detailed response comparisons, capabilities and shortcomings of the LFSAFE model are discussed.

1.4. Research Significance

As underlined above, availability of an advanced modeling option for representation of complex nonlinear behavior of structural walls in nonlinear dynamic response simulations of buildings incorporating structural walls conducted in context of performance based seismic design/assessment applications is crucial since seismic performance of the designed/assessed buildings are directly dependent on response prediction capabilities of the adopted models in analysis. However, as summarized in the literature review, currently available nonlinear modeling options for structural walls either lack detailed experimental validation against quasi-static tests of wall specimens or not utilized in nonlinear dynamic response simulations of shake table experiments of isolated walls or buildings with structural wall systems. In this respect, the developed LFSAFE model is currently one the most extensively validated model available in the literature when the experimental validations previously conducted using the relatively simpler version (with a single layer 4-node 8-degrees-of-freedom membrane formulation) of the model, Fixed-Strut-Angle Finite Element (FSAFE) model (Gullu and Orakcal, 2021), against rectangular wall specimens (as well as a

T-shaped specimen) exhibiting flexure controlled, coupled shear-flexure and shear-dominant responses, as well as the validations against non-rectangular wall specimens and dynamic response simulations presented in this study are considered together. It is important to note that, with its mechanically-defined shear transfer mechanisms and sophisticated material constitutive relationships, previous version of the model (Gullu and Orakcal, 2021) was already deemed as one of the five state-of-the-art structural wall nonlinear finite element modeling methods for rectangular wall specimens in Kolozvari et al. (2019). With development of the improved version (i.e. complete 24-degrees of freedom shell formulation including drilling degrees of freedom, layered shell formulation, reinforcement buckling), LFSAFE model has become one of the most promising candidates for reliable response simulations of buildings incorporating structural wall systems. Since dynamic response simulations with such advanced models are also significantly scarce, response predictions provided in this study for two shake table experiments further enhanced the importance of the developed model. Furthermore, lack of accessibility of the available models in the literature also significantly hinders both further development of the models by the researchers and adoption of the models in real-life applications by the practicing engineers. Therefore, implementation of the model in an open-source software framework, OpenSees (McKenna et al., 2000), has an equal importance with the development, creating the possibility of prevalent adoption of the model in future research studies as well as real-life applications, when the model is made accessible.

1.5. Thesis Outline

This thesis incorporates five chapters, starting with Chapter 1, in which, a general introduction, a review of previously conducted quasi-static tests and shake table experiments as well as model validation studies related to RC structural walls in the literature, also the scope and objectives of this study are provided. Formulation of the developed LFSAFE model is presented in Chapter 2 with detailed explanation for mechanical characteristics of the adopted constitutive panel model, as well as cyclic uniaxial stress-strain relationships used for representation of concrete and reinforcing steel nonlinear behavior. In Chapter 3, brief information about quasi-static experiments of RC structural wall specimens used in validation of the LFSAFE model, as well as comparisons of analysis model predictions with experimentally measured response of the quasi-static test specimens, are provided. Next,

shake table tests of the half-scale isolated U-shaped RC wall specimen, as well as the full-scale ten-story RC wall-frame building structure are described in detail in Chapter 4, and nonlinear dynamic response simulation comparisons for these specimens are presented. Finally, the analytical model prediction capabilities are summarized, as well as the concluding remarks and recommendations for future studies are presented in Chapter 5.



2. LAYERED FIXED-STRUT-ANGLE FINITE ELEMENT MODEL

The Layered Fixed-Strut-Angle Finite Element (LFSAFE) model is an improved and extended version of the so-called Fixed-Strut-Angle Finite Element (FSAFE) model (Gullu and Orakcal, 2021). The 4-node 8-degrees-of-freedom formulation of the original FSAFE model element (with 2 translational DOFs at each node), was incapable of simulating the out-of-plane behavior of a rectangular wall or wall segment (web or flange) in a non-planar wall, since its single-layer (membrane) formulation did not incorporate out-of-plane displacements or rotations. Furthermore, the original FSAFE model element did not include any drilling (in-plane rotational) degrees of freedom, and incorporated a single Gauss integration point in its formulation. Nevertheless, accuracy of the FSAFE model has been extensively validated against test results for relatively-slender, medium-rise, and low-rise planar walls subjected uni-directional lateral loading (Gullu et al., 2019; Gullu and Orakcal, 2021), as well as U-shaped walls tested under bi-directional lateral loading (Bilgin et al, 2021; Kolozviri et al.; 2022). The extensions (as well as improvements) implemented on the baseline FSAFE model formulation, for developing the LFSAFE model element presented in this paper, includes: (i) formulation a new three-dimensional model element with 4 nodes and 6 DOFs (3 translational and 3 rotational) at each node, (ii) implementation of a multi-layered shell formulation (with multiple membranes comprising the shell element) for simulating nonlinear out-of-plane flexure, (iii) inclusion of drilling (in-plane rotational) degrees of freedom, which allow possible direct connectivity between wall element nodes and frame elements, rather than using “embedded beams” (e.g., Perform3D; Computers & Structures Inc., 2023), (iv) improvement of the constitutive model for reinforcing steel to represent reinforcement buckling behavior, (v) implementation of the model element in the open-source computational platform OpenSees (McKenna et al., 2000).

The model’s three dimensional and 24-DOF formulation allows direct implementation in three-dimensional models of entire structural systems. In-plane strains on the LFSAFE model element are calculated using third-order Hermite shape functions adopted for displacement interpolation, and the corresponding tangent stiffness matrix and restoring force vector are obtained via 3×3 Gauss integration. Out-of-plane kinematics of the model element is defined based on the Discrete Kirchhoff Quadrilateral Plate element formulation

(Batoz & Tahar, 1982). Nonlinear model element behavior corresponding to both the in-plane and out-of-plane deformation modes are obtained from the in-plane (plane-stress) behavior of the reinforced concrete layers (membranes) comprising the shell element. Out-of-plane shear behavior of the thin-shell-type LFSAFE model element is assumed linear elastic. Details on the constitutive behavior and the mechanical working principles of the LFSAFE model are described in the following subsections.

2.1. The Constitutive Fixed-Strut-Angle Model for RC Membrane Behavior

The LFSAFE model element incorporates the so-called Fixed-Strut-Angle Model (FSAM, Orakcal et al., 2019) as the constitutive plane-stress RC panel (membrane) model for defining the membrane behavior of each layer in the LFSAFE model element. The FSAM was developed and validated based on test results presented in the literature for RC panel (membrane) specimens tested under various stress states and strain histories. Formulation of the constitutive model FSAM (Figure 2.1a), follows an average strain, smeared stress, and fixed-crack-direction approach, with perfect bond assumed between concrete and reinforcing steel bars. Bond slip deformations are neglected in the formulation of the FSAM, which renders it incapable of simulating possible bond slip failures in a wall (e.g., along deficient lap splices), or capturing strain/yield penetration effects associated with bar extension/slip deformations developing within the foundation of a wall and accumulating at the wall-foundation interface.

In the constitutive model, reinforced concrete behavior is simulated using primarily uniaxial stress–strain relationships for concrete struts along two fixed strut (crack) directions (Figure 2.1b), and for reinforcing steel in the two orthogonal reinforcement directions (Figure 2.1d). The model formulation also incorporates mechanically-defined hysteretic response characteristics for the shear aggregate interlock effects along crack surfaces (Figure 2.1c), as well as dowel action on both vertical and horizontal reinforcement (Figure 2.1e). The resultant average (smeared) stress field developing on the panel element corresponding to the applied strain field, is obtained via superposition of uniaxial concrete stresses developing along the two fixed strut directions, with shear stresses developing along crack surfaces and smeared uniaxial and dowel stresses developing in the horizontal and vertical reinforcement (Figure 2.1a).

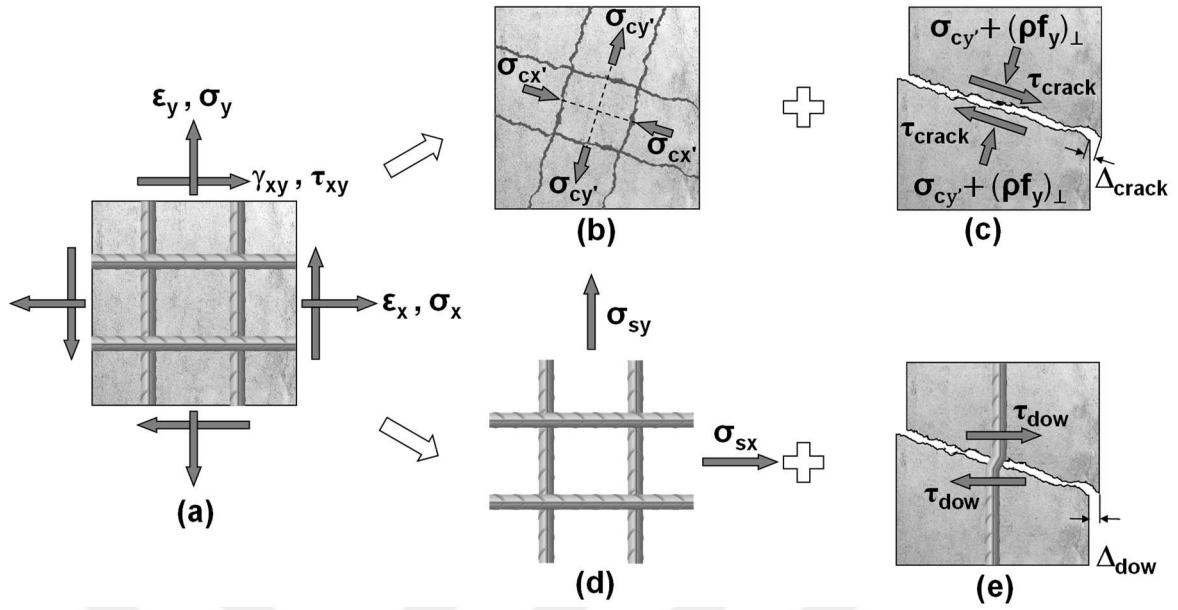


Figure 2.1. Constitutive Fixed-Strut-Angle Model (FSAM): (a) strain field and corresponding stress field on a RC membrane, (b) concrete stresses along fixed struts, (c) shear aggregate interlock stresses along cracks, (d) uniaxial stresses in reinforcement, (e) dowel stresses in reinforcement (modified from Gullu et al., 2019).

2.2. Cracking and Fixed Strut Behavior of Concrete in the FSAM

In the FSAM formulation, during the uncracked stage of concrete behavior, the principal stresses in concrete are obtained in the principal strain directions (Figure 2.2), assuming principal strain and stress directions in concrete coincide until first cracking. This approach is similar to rotating-angle constitutive modeling approaches presented in the literature (e.g., Modified Compression Field Theory, Vecchio and Collins, 1986).

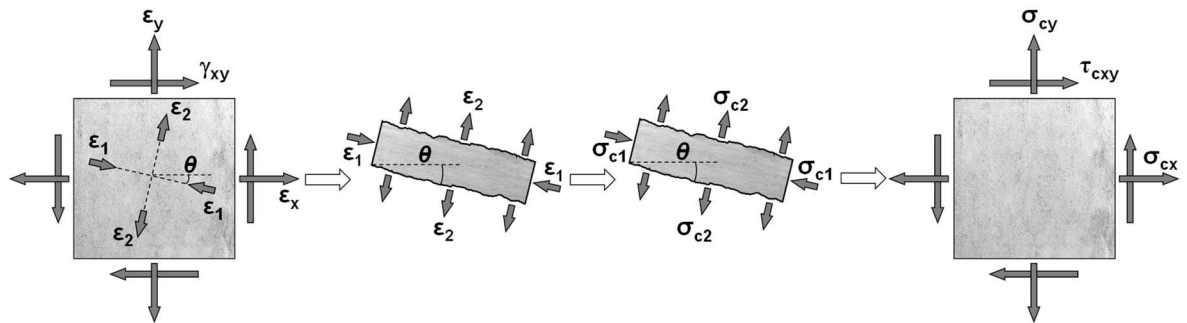


Figure 2.2. FSAM uncracked concrete behavior (modified from Gullu et al., 2019).

Subsequently, when the principal tensile strain on the membrane first exceeds the cracking strain of concrete (ε_t in Figure 2.6), the first crack develops, forming the first “fixed strut” parallel to the cracking angle, perpendicular to the principal strain direction (θ_{crA} in Figure 2.3). After formation of the first crack, a fundamentally uniaxial stress–strain behavior (that also considers biaxial softening effects) is assumed for concrete, in parallel and perpendicular directions to the first crack (first strut), while the principal strain directions in concrete continue to rotate depending on the imposed strain fields. At this stage, shear stresses also develop along the crack surface, due to the constitutive mechanisms defined for shear stress transfer across the cracks, as described in a later section.

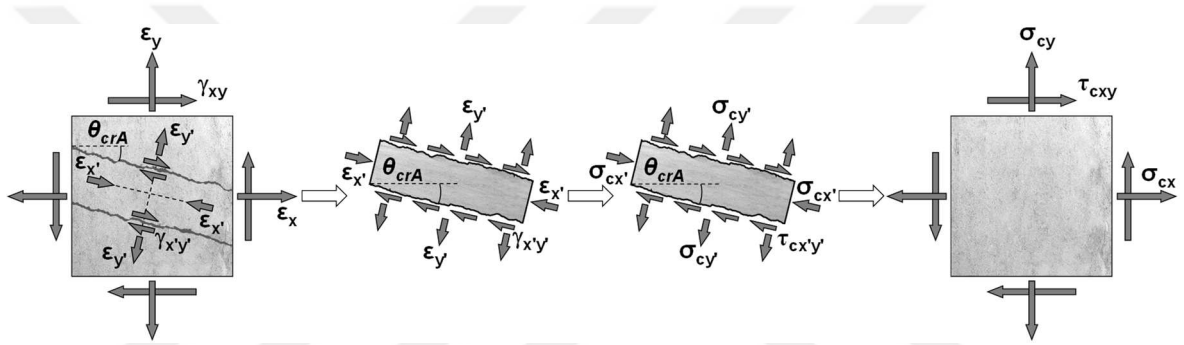


Figure 2.3. FSAM concrete behavior after formation of the first strut (modified from Gullu et al., 2019).

During subsequent loading, upon load reversal, the second crack (strut) is assumed to form in perpendicular direction to the first crack (i.e., the FSAM assumes orthogonal cracks), when the strain along the first strut exceeds the concrete cracking strain upon unloading from compression ($\varepsilon_0 + \varepsilon_t$ in Figure 2.5). During further loading/unloading, principal strain directions continue to rotate based on the applied strain field, whereas uniaxial stresses are assumed to develop in concrete, along the two fixed strut (crack) directions (Figure 2.4). The two concrete struts undergo interchanging axial tension and compression, and concrete behavior along these fixed struts (θ_{crA} and θ_{crB} in Figure 2.4) are represented using the uniaxial hysteretic concrete stress-strain relationship, also modified to consider biaxial softening effects. During this stage, shear stresses developing along crack surfaces, resulting from the stress transfer mechanisms adopted along the cracks, is also considered in calculation of the resultant stress field on concrete.

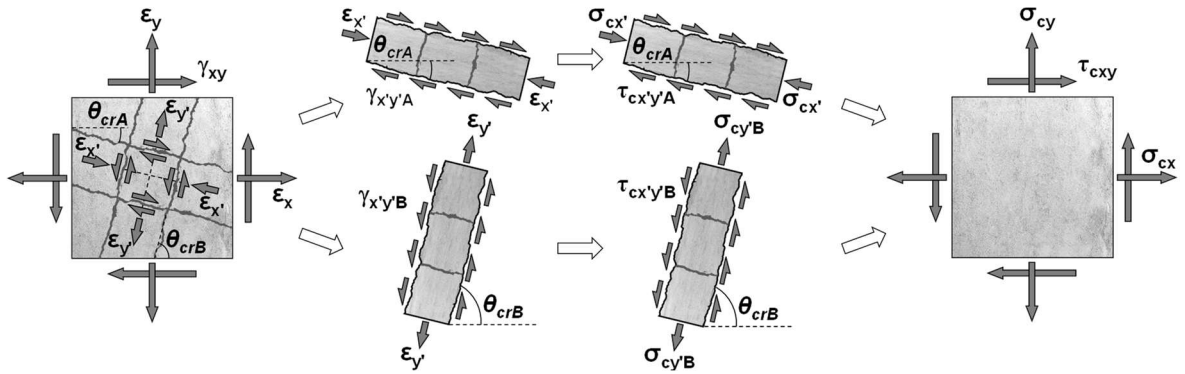


Figure 2.4. FSAM concrete behavior after formation of the second strut (modified from Gullu et al., 2019).

The shear stresses developing on the crack surfaces cause the principal stress directions in concrete to deviate from the fixed strut (crack) directions. In summary, the FSAM generates the resultant stress field on cracked concrete as the superposition of two components: (i) the uniaxial stresses developing in concrete along the fixed strut directions, and (ii) the shear (interlock) stresses developing along crack surfaces. Considering the total (resultant) stress field on concrete, the principal stress planes therefore deviate from the fixed crack planes. Details on the working principles of the FSAM, as well as its experimental validation against cyclic RC panel (membrane) tests presented in the literature (Stevens et al., 1991; Mansour and Hsu, 2005), are available in the publications by Orakcal et al. (2019), Bekaroglu (2019), and Ulugtekin (2010).

2.3. Material Constitutive Models

In the constitutive FSAM formulation of the present LFSAFE model element, the uniaxial hysteretic behavior of concrete along the two fixed struts is represented using the refined constitutive relationship by Chang and Mander (1994). The adopted concrete model reflects detailed hysteretic behavioral characteristics of concrete, such as continuous hysteretic behavior (with slope continuity) under compression and tension, progressive stiffness degradation, gradual gap closure, and tension stiffening (Figure 2.5). In the model, monotonic compression/tension envelopes for the concrete are defined using initial modulus of elasticity of concrete (E_c), compressive/tensile strength (f'_c or f_t), strain at peak stress (ϵ'_c or ϵ_t), a shape parameter (r) for curvature of the curve, and a critical strain limit (ϵ_{cr}), after which the envelope remain linear. Furthermore, cyclic characteristics of the concrete

material are generated by the adopted model with empirical relationships using key hysteretic parameters such as secant modulus (E_{sec}) and plastic modulus (E_{pl}) upon unloading from the compression/tension envelope, as well as stress and strain shifts (Δf and $\Delta \epsilon$) upon reloading back to the envelopes as shown in Figure 2.6.

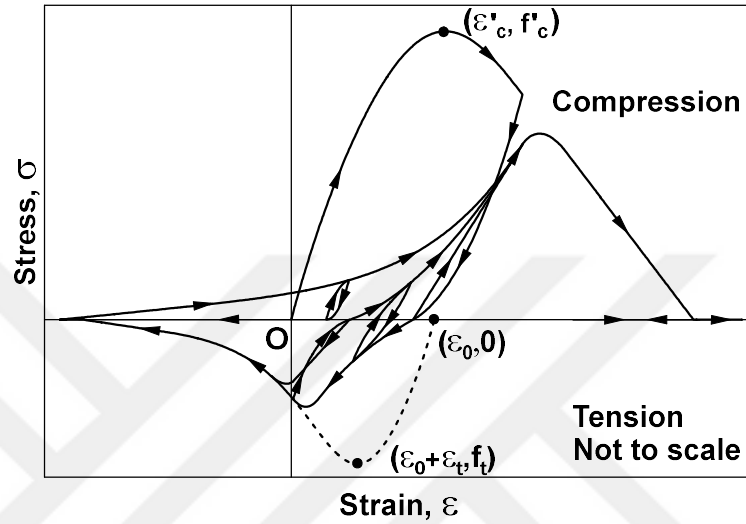


Figure 2.5. Hysteretic rules of Chang and Mander (1994) uniaxial constitutive model for concrete (Orakcal, 2004).

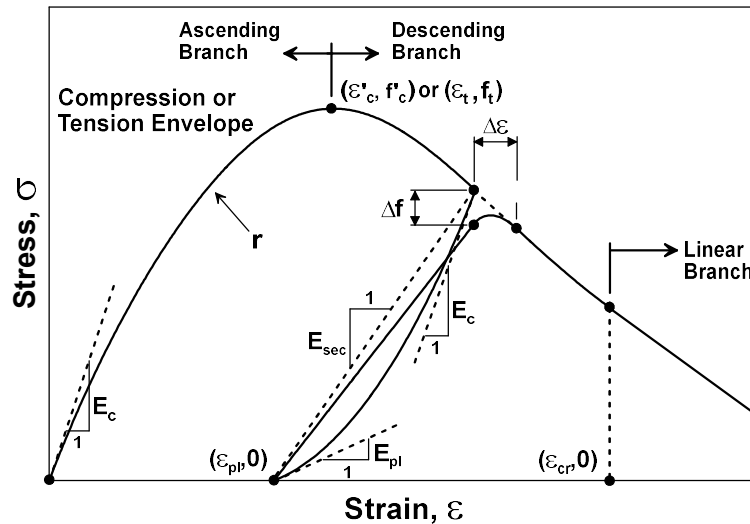


Figure 2.6. Parameters for monotonic envelope definition and hysteretic behavior generation of Chang and Mander (1994) uniaxial constitutive model (Orakcal, 2004).

Although the concrete stress–strain relationship is fundamentally uniaxial, biaxial softening effects on concrete, including compression softening (Vecchio and Collins, 1993)

and hysteretic biaxial damage (Mansour et al., 2002), are also incorporated in the concrete stress–strain behavior adopted in FSAM in along the two strut directions. As well, the monotonic tension envelope of the model is calibrated to represent the effect of tension stiffening on the average stress–strain behavior of concrete after cracking, in accordance with the relationship proposed by Belarbi and Hsu (1994). For considering the effect of confinement on the stress–strain behavior of concrete in compression, the LFSAFE model element uses a uniaxial constitutive model (Mander et al., 1988), for which the confined stresses and strains are considered in perpendicular direction to transverse reinforcement. This is a simplification in the model element formulation, as the compression struts developing in the model elements will not necessarily be vertically-oriented. However, for slender or moderate-aspect-ratio walls with flexure-dominated coupled shear-flexural responses, in the model elements representing the wall boundary regions where confinement is provided, crack orientations predicted by the model are typically very close to horizontal (corresponding to approximately vertical compression struts), which makes this simplification a reasonable one.

To describe the uniaxial stress–strain behavior of the horizontal and vertical reinforcing steel bars in the web region of a wall, as well as the horizontal bars in the wall boundary regions, the well-known uniaxial hysteretic constitutive model developed by Menegotto and Pinto (1973), and extended by Filippou et al. (1983) to incorporate strain hardening effects, (Figure 2.7) is adopted.

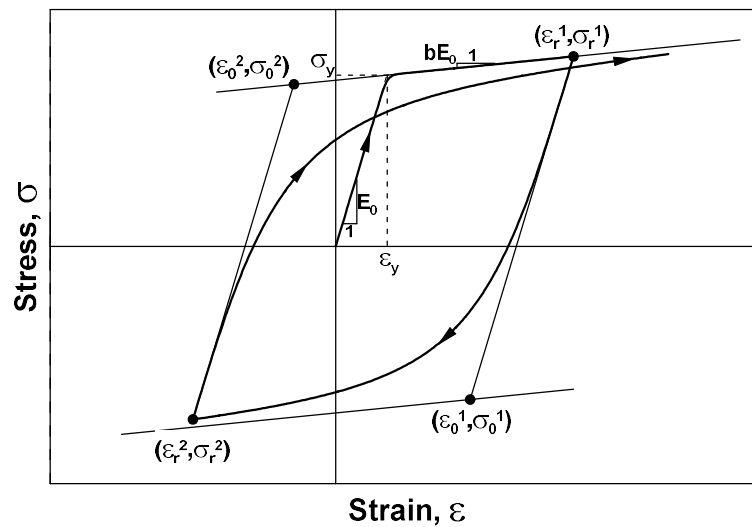


Figure 2.7. Menegotto and Pinto (1973) reinforcing steel model (Orakcal, 2004).

For the longitudinal reinforcement located in wall boundary regions, in order to represent reinforcing bar buckling behavior, the hysteretic constitutive relationship proposed by Dhakal and Maekawa (2002a) is adopted in the present model formulation, together with a simplification of the hysteretic rules that neglects the linear transition regions between the hysteretic loops and the monotonic compression and tension envelopes (Figure 2.8). In this constitutive model, the monotonic compression envelope that considers reinforcement buckling behavior, is first constructed. In construction of the modified compression envelope, buckling of the reinforcing steel bar is associated with the product of the slenderness ratio and the square root of the yield strength of the reinforcement ($L/D\sqrt{f_y}$), as well as post-yield characteristics of the tension envelope. The slenderness ratio corresponding to the controlling buckling mode is determined using the procedure proposed by Dhakal and Maekawa (2002b), also considering in-plane and out-of-plane buckling modes across multiple tie spacings.

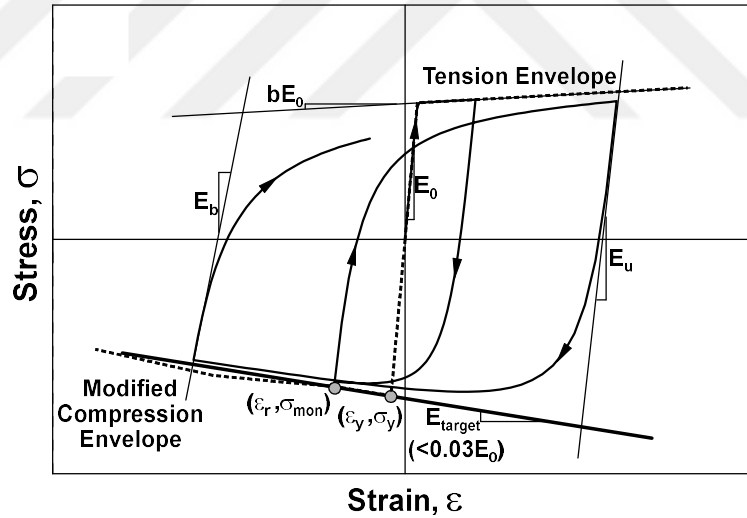


Figure 2.8. Simplified Dhakal and Maekawa (2002a) reinforcing steel model.

The hysteretic formulation of the model resembles the Menegotto and Pinto (1973) formulation, where Bauschinger's effect is represented by the stiffness degradation parameters R_0 , α_1 and α_2 , with strain-history-based yield asymptotes (with negative slope) in compression. The slope of the compression yield asymptote (E_{target} in Figure 2.8) is defined as equal to slope of the line connecting the yield point under compression (ϵ_y, σ_y in Figure 2.8) with the compressive stress on the modified compression envelope (ϵ_r, σ_{mon} in Figure 2.8) corresponding to the maximum compressive strain experienced during the

previous strain history. However, E_{target} should not exceed a maximum value of $0.03E_0$. Furthermore, unloading stiffness values from tension (E_u in Figure 2.8) and compression (E_b in Figure 2.8) are defined based on the empirical equations provided by Dhakal and Maekawa (2002a).

In both of the hysteretic constitutive models adopted for reinforcing steel (with and without buckling), Bauschinger's effect is represented through the cyclic stiffness degradation parameters (R_0 , α_1 and α_2) embedded in their formulation. Furthermore, in both constitutive models, tension stiffening effects on the longitudinal reinforcing bars are represented using the relationships proposed by Belarbi and Hsu (1994) for modifying the tensile yield strength and tensile strain hardening ratio of bars embedded in concrete. In the present model formulation, the constitutive relationships adopted for reinforcing steel do not incorporate reinforcement rupture, post-buckling rupture (buckling-fatigue behavior), or low-cycle fatigue effects, which is a significant limitation. However, these effects can be incorporated in the model formulation directly at the material model level, independently from the model element formulation.

2.4. Shear Transfer Mechanisms across Cracks

In the constitutive FSAM formulation, for representing shear stress transfer across the fixed cracks, shear stresses developing along the crack surfaces are calculated using a physical friction-based constitutive relationship, and are then superimposed with the concrete stresses along the struts in order to obtain total stress field in concrete. The friction-based constitutive relationship representing shear aggregate interlock along crack surfaces relates the maximum shear stress developing along a crack surface to the compressive stress in concrete strut ($\sigma_{c\perp}$ in Figure 2.9b, which is equal to $\sigma_{cy'}$ in Figure 2.9a) in perpendicular direction to the crack, through a friction coefficient (η_f in Figure 2.9b). A smeared “clamping stress” created by the perpendicular component of the yield strength of the reinforcement straddling the crack (i.e., $(\sigma f_y)_\perp$ in Figure 2.9), is also considered in the compressive stress to be multiplied with this friction coefficient. Therefore, the two components of normal stress acting on a crack surface are: (i) varying compressive stresses in concrete perpendicular to the crack direction, and (ii) a constant clamping stress created by the reinforcement. When the concrete stress perpendicular to the crack direction is zero

or a tensile stress (due to tension stiffening); i.e., when the crack is open, contribution of the first component to shear aggregate interlock reduces to zero. In order to describe this friction-based constitutive relationship, the monotonic envelope and the simplified version of the hysteresis rules proposed by Tassios and Vintzeleou (1987) for rough crack surfaces (along which clamping is observed) are adopted (Figure 2.9b).

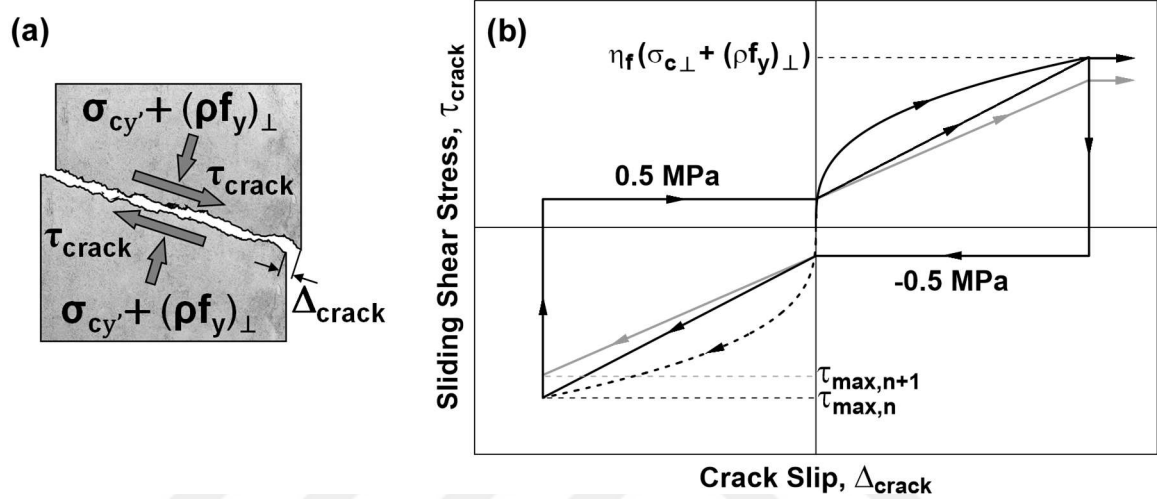


Figure 2.9. Shear aggregate interlock mechanism along crack surfaces: (a) shear aggregate interlock stresses along cracks, (b) constitutive model for shear aggregate interlock along crack surfaces.

Assuming a constant clamping stress along a crack surface, irrespective of the crack width, is a simplified approach adopted in the constitutive membrane model (FSAM) formulation. However, experimental validations that were conducted previously using a simpler and single-layered (membrane) formulation of the LFSAFE model, against planar walls tested under in-plane loading (FSAFE model - Gullu and Orakcal, 2021; Gullu et al., 2019; Gullu et al. 2018a, b; Gullu, 2019), have revealed that even with this simple approach, the model provides accurate response predictions for slender walls with flexure-dominated responses, as well as moderate-aspect-ratio walls with shear-flexure interaction behavior and low-rise walls with shear-controlled responses. In these studies, it was observed that for walls with predominantly flexural or coupled shear/flexural responses, consideration of the clamping component of shear aggregate interlock mechanism is not significantly influential on the model results. For such walls, the boundary region under flexural compression is where the majority of the internal shear stresses on crack surfaces develop, primarily due to the normal stress component of the shear aggregate interlock mechanism (i.e., due to the

compressive stress developing in concrete in perpendicular direction to the crack). Consideration of the clamping mechanism becomes more influential (at least noticeably) on the model response predictions for low-rise walls with shear-controlled responses (Gullu et al., 2018a, b; Gullu, 2019), and the constant clamping stress assumption adopted in the FSAM formulation was found to be adequate for obtaining reasonably accurate response predictions for such walls also. Furthermore, it is acknowledged that the friction coefficient (η_f in Figure 2.9b), which is used to define the shear aggregate interlock mechanism along crack surfaces, must inherently depend on the concrete aggregate size. However, to the author's knowledge, comprehensive test data that relates the shear aggregate interlock behavior of cracked interfaces to aggregate size, is not available at present. Therefore, the shear aggregate interlock friction coefficient has to be defined in a somewhat ad-hoc manner, independently of the mean aggregate size. Previous studies conducted using the single-layer version of the LFSAFE model (Gullu and Orakcal, 2021; Gullu et al., 2019, Gullu et al., 2018a, b, Gullu, 2019) have revealed that using a friction coefficient in the range of 0.3 to 0.5 is suitable for modeling of one-third and half-scale wall specimens, and provides reasonable model predictions for walls with shear-dominated responses, as well as for walls with flexure dominated responses and shear-flexure interaction behavior. In this study, a friction coefficient of 0.4 is adopted for modeling of all of the test specimens investigated, similarly to the coefficient of 0.44 recommended by Tassios and Vintzeleou (1987) for rough crack surfaces, based on results of plain concrete cracked interface tests.

The constitutive membrane model FSAM relates the slip deformation along the crack interface to the shear stress transferred, where the slip deformation (Δ_{crack} in Figure 2.9a) along the crack surface is calculated by multiplying the shear strain in the crack direction with the nominal crack spacing defined by Vecchio and Collins (1986) and Vecchio (2000). The nominal crack spacing is correlated with the spacing of horizontal and vertical reinforcing bars, as well as the crack angle. In the model element, for the cover concrete and core concrete layers that do not incorporate reinforcement, crack spacing is assumed to be identical to that calculated for the layers that include reinforcement. This is believed to be a reasonable assumption, as it is commonly observed in real-life RC structures. It must be mentioned that definition of crack spacing in the model formulation is required only for relating the shear strains developing in crack directions (at the Gauss integration points of the model element), to crack slip (sliding) deformations developing along crack surfaces,

and therefore relates only to the tangent stiffness characteristics of shear aggregate interlock constitutive relationship applied along the crack surfaces.

Contribution of dowel action on both the horizontal and vertical reinforcing bars on the shear stress field developing on the panel element is also incorporated in the FSAM model formulation, via adopting the dowel force vs. deformation monotonic envelope and the hysteresis rules defined by Vintzēleou and Tassios (1986, 1987). In this study, these hysteresis rules were simplified into origin-oriented hysteresis rules (Figure 2.10b). To correlate dowel forces and deformations with dowel stresses and strains, the smeared-stress-strain constitutive modeling approach proposed by He and Kwan (2001) is used. In this approach, the dowel deformation (Δ_{dow} in Figure 2.10a) on the reinforcing steel bars is correlated with shear and tensile strains across the crack, as well as an effective dowel length, where the dowel length is determined using a beam (reinforcing bar) on a semi-elastic foundation (concrete) analogy. Smeared dowel stresses perpendicular to the direction of the reinforcing bars are obtained from the corresponding dowel forces. The reinforcing steel stresses are then superimposed with the smeared dowel stresses, and then back-converted to tensile and shear stresses along the crack surface.

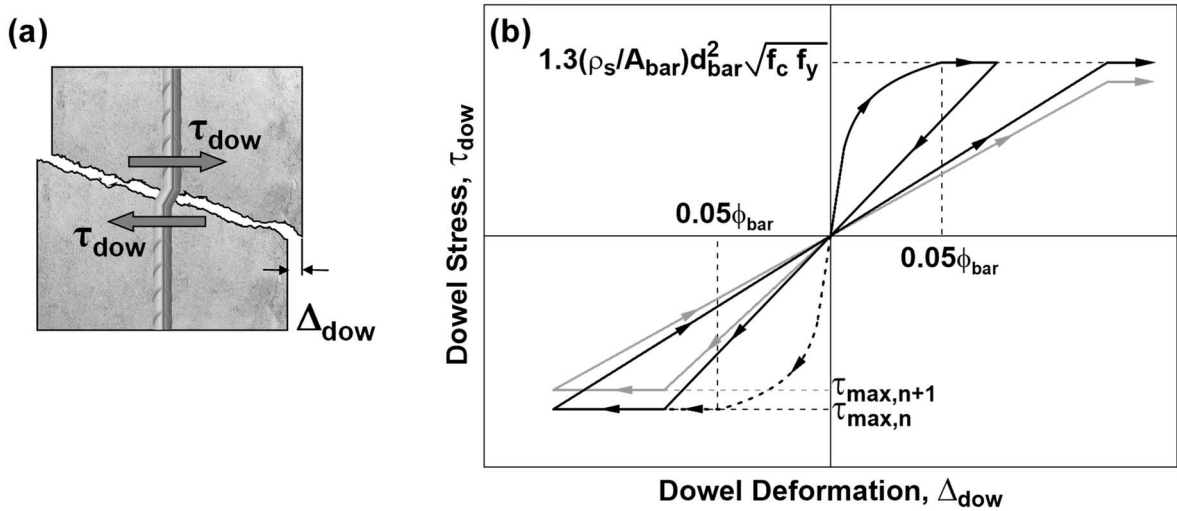


Figure 2.10. Constitutive model for dowel action on reinforcement.

Lastly, in both of the constitutive models adopted for shear aggregate interlock and dowel action, cyclic degradation effects on the interlock and dowel stresses are also considered, using the history-dependent cumulative-deformation-based empirical

relationships first proposed by Tassios and Vintzēleou (1987) and Vintzēleou and Tassios (1986), also considering later modifications proposed by Thermou et al. (2014).

2.5. The LFSAFE Model Element

The LFSAFE (Figure 2.11) model element implemented in this study in OpenSees, with its six (three translational and three rotational) degrees of freedom (DOF) at each node, builds upon the baseline formulation of the currently-available Discrete Kirchhoff Generalized Quadrilateral shell element (ShellDKGQ), which was previously implemented in OpenSees by Lu et al. (2017). The ShellDKGQ element combines the membrane element GQ12 (Xu and Long, 1993) with the well-known plate bending element DKQ (Batoz and Tahar, 1982), in order to generate a complete 24-DOF shell element formulation.

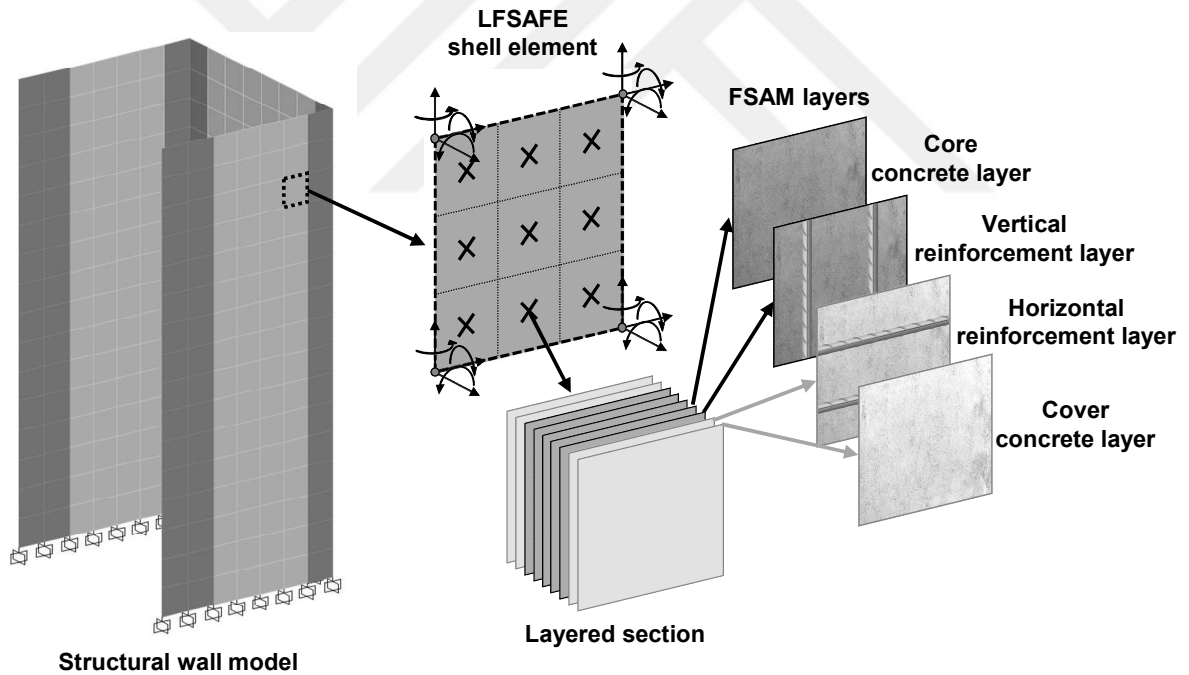


Figure 2.11. Layered Fixed-Strut-Angle Finite Element (LFSAFE) Model Element.

The membrane formulation of the ShellDKGQ element utilizes bi-linear and linear-quadratic shape functions for the description of displacement fields associated with translational DOFs and rotational (drilling) DOFs, respectively. However, drilling rotations obtained using the ShellDKGQ element were found geometrically-incompatible with translational displacements at the nodes of the shell elements, in cases where moment

transfer is required at the drilling DOFs of the element. The strain interpolation functions adopted in the ShellDKGQ element for the drilling degrees of freedom provide insufficient drilling stiffness, resulting in excessive in-plane (drilling) rotations at the shell element nodes, when frame elements are connected to the shell elements in their plane. Furthermore, rotations obtained along the drilling DOFs at the other nodes of the wall model (to which the line elements are not connected) are not geometrically-compatible with the translational nodal displacements, and those nodes also undergo excessive drilling rotation together with the node to which the line element is connected. Same results are obtained when either 2×2 or 3×3 Gauss integration is used in the GQ12 model element formulation.

In order to remedy this connectivity problem between the nodes of the ShellDKGQ element and connecting frame elements, the membrane formulation of the original ShellDKGQ element was modified. In LFSAFE model element formulation, bi-linear and linear-quadratic shape functions (that were originally used in definition of displacement interpolation functions of the GQ12 element) are replaced with linear-cubic interpolation functions derived using third-order Hermite shape functions for both the translational DOFs and rotational DOFs, and 3×3 Gauss integration is used for generation of the stiffness matrix and the resisting force vector, similarly to the approach adopted and described in detail by Rojas et al. (2016). For instance, the displacement fields from nodal displacements (in model space) of the first node in horizontal ($u_{U_1}(\zeta, \eta)$) and vertical ($v_{V_1}(\zeta, \eta)$) directions of the parent membrane element are defined with equations

$$u_{U_1}(\zeta, \eta) = \frac{1}{2}(1 - \zeta) \left(\frac{1}{2} - \frac{3}{4}\eta + \frac{1}{4}\eta^3 \right) (U_1), \quad (2.1)$$

$$v_{V_1}(\zeta, \eta) = \frac{1}{2}(1 - \eta) \left(\frac{1}{2} - \frac{3}{4}\zeta + \frac{1}{4}\zeta^3 \right) (V_1), \quad (2.2)$$

respectively. In the equations, U_1 and V_1 are horizontal and vertical nodal displacements, ζ and η are horizontal and vertical coordinates in the parent element, respectively. Additionally, the displacement fields from nodal rotation (in model space) of the first node in horizontal ($u_{\theta_1}(\zeta, \eta)$) and vertical ($v_{\theta_1}(\zeta, \eta)$) directions of the parent membrane element are defined with equations

$$u_{\theta_1}(\zeta, \eta) = -\frac{y_4 - y_1}{2}(1 - \zeta) \left(\frac{1}{4} - \frac{1}{4}\eta - \frac{1}{4}\eta^2 + \frac{1}{4}\eta^3 \right) (\theta_1), \quad (2.3)$$

$$v_{\theta_1}(\zeta, \eta) = \frac{x_2 - x_1}{2}(1 - \eta) \left(\frac{1}{4} - \frac{1}{4}\zeta - \frac{1}{4}\zeta^2 + \frac{1}{4}\zeta^3 \right) (\theta_1), \quad (2.4)$$

where, $y_4 - y_1$ is height of the shell element between nodes 1 and 4 (in model space), $x_2 - x_1$ is width of the shell element between nodes 1 and 2 (in model space), and θ_1 is the joint in-plane rotation of element node 1. Resulting displacement fields (only due to nodal displacements and rotation of the first element node) with the adopted formulation are demonstrated in Figure 2.12. When this formulation is adopted, the LFSAFE model element provides drilling rotations that are kinematically-compatible with the translational displacements at the model element nodes, and also generates sufficient drilling stiffness at the element nodes so that more physical moment transfer is attained between the wall model and connecting frame elements (instead of excessive drilling rotations developing at wall and frame element connections).

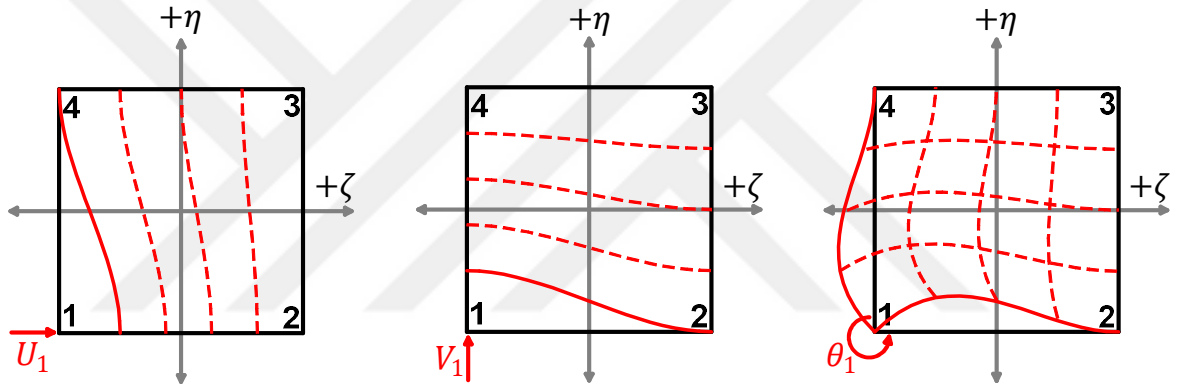


Figure 2.12. Displacement fields obtained employing Hermite shape functions.

To illustrate, deformed shapes obtained using analysis models generated in OpenSees for two slender walls connected together by beam (line) elements (e.g., coupled wall systems), where either ShellDKGQ or LFSAFE elements are used for modeling of the walls and linear elastic line (frame) elements with plastic flexural hinges at the ends are used for modeling of the beams, are compared in Figure 2.13. In the figure, rotations at drilling degrees of freedom of the wall model are shown with the red cross symbols in bold. As can be observed in Figure 2.13a, when ShellDKGQ elements are used for modeling of the walls, excessive drilling rotations develop not only at the nodes where the beams are connected, but also at other nodes of the wall model. Furthermore, plastic rotations do not develop on the beams, indicating that bending moments transferred to the beams are relatively low, since the wall model nodes to which the beams are connected undergo excessive drilling rotations. When the walls are modeled using LFSAFE elements (Figure 2.13b), the beams reach their

moment capacities and undergo plastic rotations, due to the sufficient drilling stiffness of the wall model nodes to which the beams are connected, and more kinematically-compatible drilling rotations develop on all other nodes of the wall model.

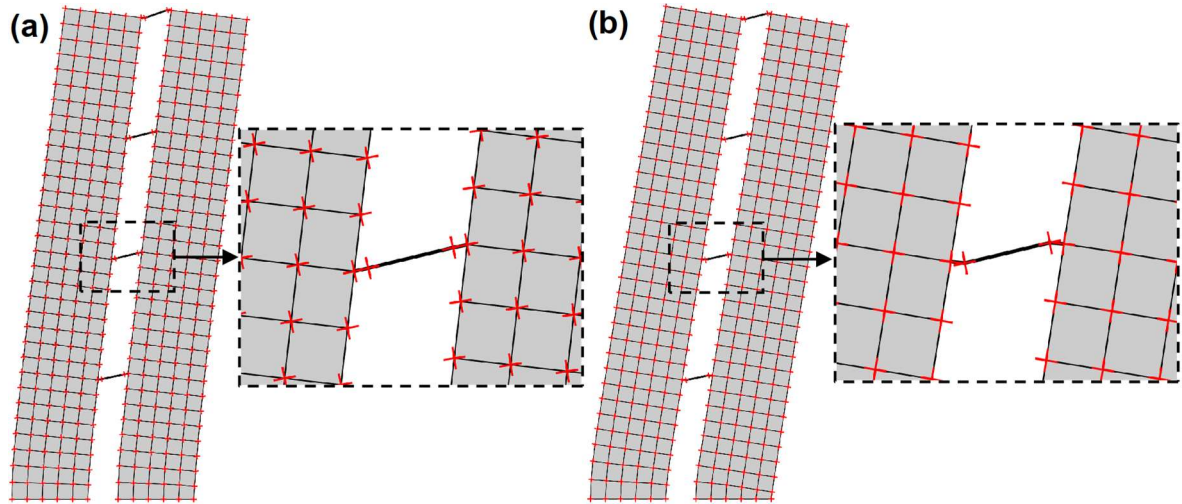


Figure 2.13. Analytical deformed shapes obtained for a coupled wall system using: (a) ShellDKGQ model elements (with bi-linear and linear-quadratic shape functions and 2×2 Gauss integration), (b) LFSAFE model elements (with Hermite shape functions and 3×3 Gauss integration).

Accurate definition of drilling stiffness in membrane or shell elements is a challenging task that is subject much uncertainty, since application of a concentrated moment on a single dimensionless node is not entirely physical, and the problem is very much subject to localization effects (i.e., model element discretization and/or nonlinear strain localization). For reinforced concrete walls, moment transfer from a connecting beam or a coupling beam is not physically in the form of a concentrated moment, and the drilling stiffness of the region to which the beam is connected will also depend on the reinforcement in the region, which consists of not only the wall reinforcement, but also the beam reinforcement extending into the region. However, as illustrated in Figure 2.13, the LFSAFE model element is clearly more physical compared to the ShellDKGQ element (or the GQ12 membrane element), in terms of drilling stiffness characteristics and kinematic compatibility between drilling and translational DOFs. In this respect, the LFSAFE shell element introduced in this study, as well as the QLMEDD membrane element and the QTFLS shell element developed by Rojas

et al. (2016, 2019), are improvements towards more physical connectivity between wall model elements and line (beam) elements.

The out-of-plane behavior of the LFSAFE model is formulated by adopting the Discrete Kirchhoff Quadrilateral (DKQ, Batoz and Tahar, 1982) plate element formulation of the ShellDKGQ element, where through-thickness shear strains due to out-of-plane translational DOFs are neglected, and only curvatures due to out-of-plane rotations are considered. However, since the nonlinear out-of-plane behavior of the shell element is defined using a layered section formulation, where membrane strains are extrapolated using a combination of membrane and plate deformation modes, curvatures due to out-of-plane bending of the plate element are also extrapolated at nine Gauss points, differently from than the original ShellDKGQ element with four Gauss points. Further information about the DKQ plate element formulation can be found in Batoz and Tahar (1982).

The LFSAFE model element employs the currently-available layered section formulation (Lu et al., 2015) in OpenSees, where in-plane and out-of-plane nonlinear behavior are coupled. In the out-of-plane direction of the model element, the plane-sections-remain-plane kinematic assumption is employed for obtaining the in-plane strains developing in each membrane layer. The in-plane strains ($\varepsilon_x, \varepsilon_y, \gamma_{xy}$) at each Gauss point of each membrane layer are obtained by first calculating the element centerline (mid-thickness) strains associated with the in-plane DOFs, and then adding to these, the in-plane strains developing in each layer due to out-of-plane curvatures at the Gauss points. This results in a linear strain distribution (constant curvature) across the thickness (among the layers) of the model element. This linear strain distribution applies to the vertical, horizontal, and shear strains on the membrane layers, based on the strains calculated (using the in-plane displacement field) at the model element mid-thickness, and the out-of-plane rotation field. The displacement fields in each membrane layer, from the in-plane displacement and the out-of-plane rotation fields of the model element are derived with equations

$$u_{layer} = u + z_{layer}\theta_y, \quad (2.5)$$

$$v_{layer} = v - z_{layer}\theta_x, \quad (2.6)$$

where u and v correspond to the displacement fields associated with the in-plane DOFs, u_{layer} and v_{layer} represent the total displacement fields including contribution of out-of-plane curvatures, θ_x and θ_y correspond to out-of-plane rotation fields about the local x and

y axes of the model element, and z_{layer} denotes the distance between a specific membrane layer to the centerline of the element thickness. Then, the corresponding strain fields in each membrane layer are derived with the equation

$$\begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} = \underbrace{\begin{bmatrix} \partial u / \partial x \\ \partial v / \partial y \\ \partial u / \partial y + \partial v / \partial x \end{bmatrix}}_{\text{From in-plane displacement fields}} + z \underbrace{\begin{bmatrix} \partial \theta_y / \partial x \\ -\partial \theta_x / \partial y \\ \partial \theta_y / \partial y - \partial \theta_x / \partial x \end{bmatrix}}_{\text{From out-of-plane rotation fields}}. \quad (2.7)$$

The stress states and the material tangent stiffnesses corresponding to the membrane strain fields attained from layered section formulation are obtained from the constitutive RC membrane model FSAM for each layer, and are used in generation of the element resisting force vector and the element tangent stiffness matrix. Due to the layered formulation of the model element, strain predictions are obtained not only at the centerline (mid-thickness) of the element, but also at the edges of the wall cross-section. The significance of this feature becomes evident when the maximum (e.g., outer-surface) compressive strains are being predicted on the web, flanges, or web-flange intersections of a non-rectangular wall cross-section, especially when the location of the neutral axis falls within the thickness of a wall segment. As well, the layered formulation of the model element permits more detailed cross-section definitions, with consideration of different confinement characteristics for concrete along the wall thickness (unconfined cover vs. confined core layers), as well as, defining the precise locations of the reinforcement layers, as representatively shown in Figure 2.14. For example, in a single model element, an unconfined stress-strain relationship can be adopted for the cover concrete layers, while a confined stress-strain relationship can be used for the core (confined) concrete layers. During the analysis, the unconfined cover concrete layers will be subjected to earlier and much more pronounced compressive stress degradation (along the descending region of the stress-strain relationship), compared to the confined concrete layers, even under equal compressive strain demands. Furthermore, the nonlinear flexural behavior of the model element in the wall out-of-plane direction allows simulating asymmetrical concrete crushing or asymmetrical reinforcement yielding across the thickness of a wall segment.

The robustness of the out-of-plane formulation of the LFSAFE model element was validated upon comparison of analysis results obtained under loading in the out-of-plane direction of the model element, against experimental data from cyclic loading tests conducted on reinforced concrete components with predominant nonlinear flexural behavior.

Representatively, an analysis model for the rectangular wall specimen RW2 with an aspect ratio of 3.0 and predominantly flexural experimentally measured response (Thomsen and Wallace, 2004) was generated, using a single LFSAFE model element to describe the entire wall cross-section (with the cross-section discretized into layers of the model element in its out-of-plane direction as shown in Figure 2.15). The section was discretized into a total of 66 layers considering longitudinal reinforcement locations as well as confinement characteristics of the boundary and web regions. As well, the model of the specimen was generated using a total of 36 LFSAFE elements along the height of the wall. Lateral load vs. top displacement response comparison of analytical model predictions and experimentally-measured response presented in Figure 2.16 reflects that, with only utilizing its nonlinear out-of-plane formulation, LFSAFE model accurately capture the predominantly flexural response of the wall specimen, with the same level of accuracy that would be expected from a fiber-based model.

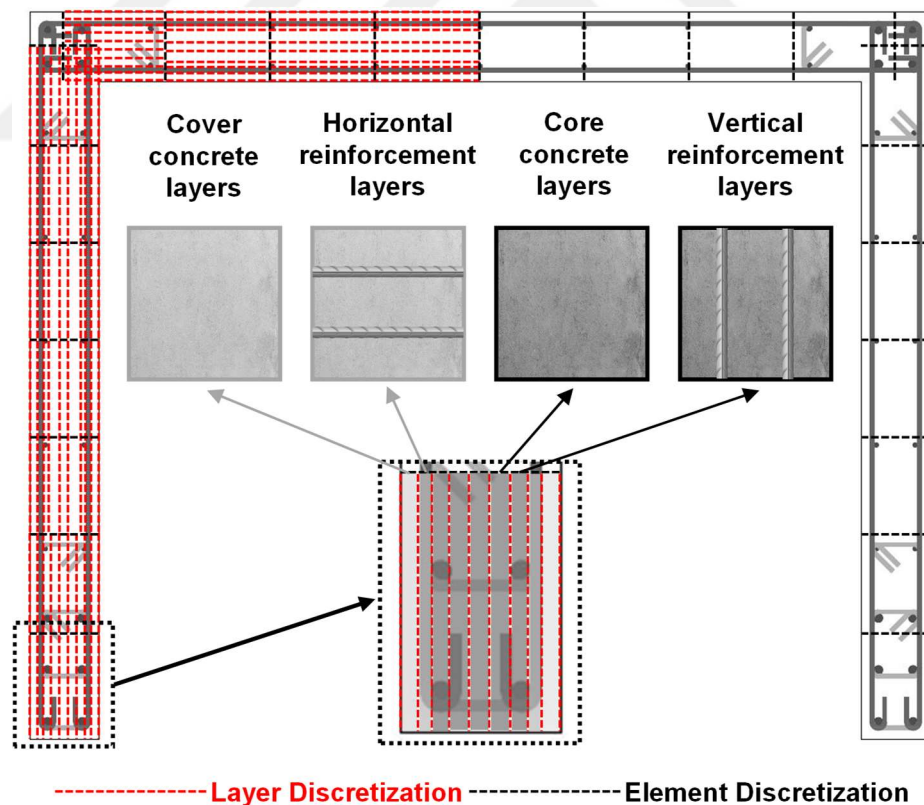


Figure 2.14. Model element and layer discretization for a representative U-shaped wall cross-section.

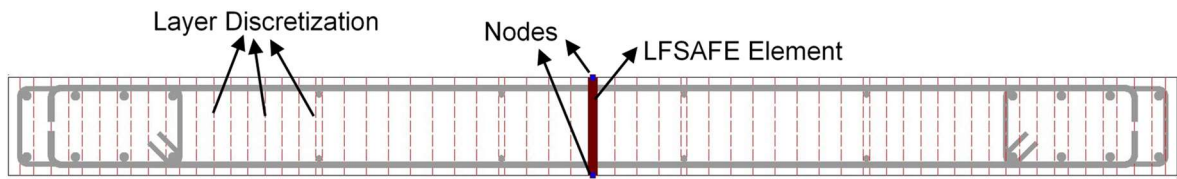


Figure 2.15. Out-of-plane layer discretization of the LFSAFE model element for wall specimen RW2.

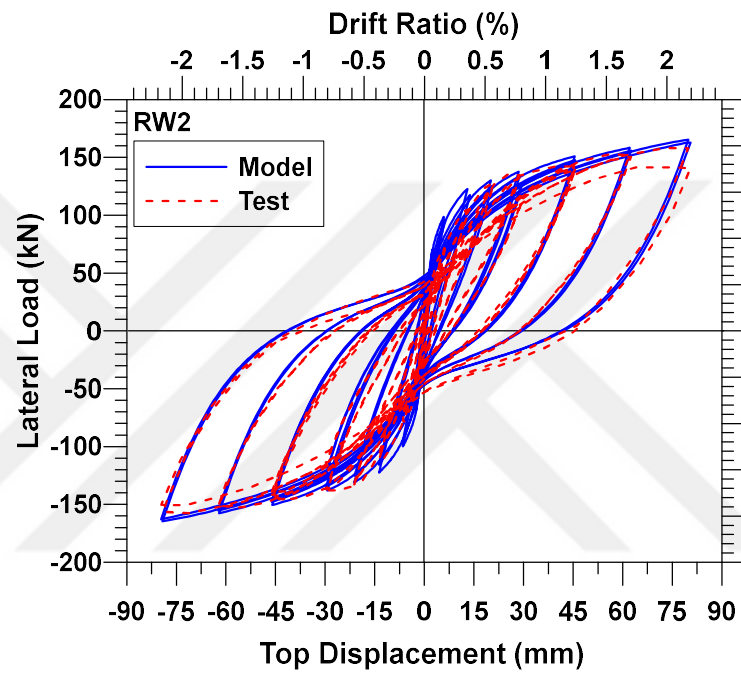


Figure 2.16. Experimental vs. analytical lateral load vs. top displacement responses of wall specimen RW2.

3. MODEL VALIDATION AGAINST QUASI-STATIC TESTS

Effectiveness and accuracy of the developed LFSAFE model in nonlinear quasi-static response simulations were demonstrated in this chapter with experimental validation via scrutinization of selected heavily instrumented nonrectangular wall specimens from the literature, tested under various loading conditions. Detailed comparisons for experimentally-measured response with analysis model predictions were provided at both global response (i.e., lateral force vs. top displacement, torque vs. top twist rotation) and local response (i.e., vertical strain profiles along wall height, vertical strain and shear stress distributions across wall base, flexural deformation contributions to top displacement) levels. Additionally, influence of strain penetration effects as well as consideration of reinforcing steel buckling behavior on model response predictions were discussed. Moreover, sensitivity studies for model element and layer discretization are presented.

3.1. Description of the Quasi-Static Tests

Experimental validation of the LFSAFE model was conducted using a wall test database that includes a total of ten non-planar wall specimens tested as part of three different experimental campaigns. The wall specimens selected for model validation were all tested under different loading conditions and they showed a broad spectrum of failure modes during testing. The four U-shaped wall specimens in the database, which were tested by Hoult et al. (2023a), Beyer et al. (2008a), and Constantin and Beyer (2016), were subjected to uni-directional lateral, bi-directional lateral (diagonal or diagonal/sweep), and pure torsional loading conditions. The six H-shaped wall specimens tested by Maruta et al. (1999, 2000a, 2000b) were subjected to combined lateral/torsional loading, with different maximum-bending-moment-to-torque ratios enforced during the tests. Table 3.1 summarizes some of the important characteristics of all the U-shaped and H-shaped wall specimens, including their dimensions, loading conditions, material strengths, axial load levels, as well as their experimentally-observed behavior and failure modes. Figure 3.1 illustrates the cross-sections and reinforcement details of the wall specimens.

Table 3.1. Properties of wall specimens.

Wall ID	Loading Type	h_w (mm)	t_w (mm)	l_w (mm)	$\frac{h_w}{l_w}$	$\frac{M}{Vl_w}$	f'_c (MPa)	$f_{y,bou}$ (MPa)	$\rho_{v,bou}$ (%)	$\rho_{h,web}$ (%)	$\rho_{v,web}$ (%)	$\frac{P}{A_g f'_c}$	Behavior Mode	Failure Mode ⁽¹⁾
UW1	Uni-directional Lateral	2000	100	1050 ^{N-S} 1300 ^{E-W}	1.91 ^{N-S} 1.54 ^{E-W}	5.43 ^{S-N} 7.51 ^{N-S}	38.2	580	2.45 ^{Edge} 2.18 ^{Corner}	0.38	0.38	0.050	Flexure	CC/RB/LI
UW2	Pure torsion	2000	100	1050 ^{N-S} 1300 ^{E-W}	1.91 ^{N-S} 1.54 ^{E-W}	-	37.1	580	2.60 ^{SE-Edge} 2.45 ^{SW-Edge} 2.18 ^{Corner}	0.38	0.38	0.050	Torsion (Flexure-shear)	LI/OOPS
TUB	Bi-directional lateral	2720	100	1050 ^{N-S} 1300 ^{E-W}	2.59 ^{N-S} 2.09 ^{E-W}	2.81 ^{N-S} 2.58 ^{E-W}	54.7	471	2.45 ^{Edge} 1.88 ^{Corner}	0.45	0.38	0.040	Flexure-shear	WC/DC
TUC	Bi-directional lateral	2720	100	1050 ^{N-S} 1300 ^{E-W}	2.59 ^{N-S} 2.09 ^{E-W}	2.81 ^{N-S} 2.58 ^{E-W}	42	563 ^{Corner/Edge} 529 ^{Corner}	1.34 ^{SE-Edge} 2.45 ^{SW-Edge} 1.25 ^{NE-Corner} 1.15 ^{NW-Corner}	0.45	0.31 ^{W-Flange} 0.91 ^{E-Flange} 1.0 ^{Web}	0.060	Flexure-shear	FB/CC
HW25	Combined lateral-torsional	1200	80	880 ^{Strong} 800 ^{Weak}	1.36 ^{Strong} 1.50 ^{Weak}	1.22-6.73	57.9	403	4.91	0.71	0.71	0.011	Flexure-torsion	CC
HW50						0.98-1.15	57.8					0.011		CC
HW75						0.84-0.86	64.1					0.010		CC
HS25						0.75-1.78	57.6					0.011		-
HS50						0.75-1.78	57.9					0.011		-
HD25						0.75-1.13	62.5					0.010		CC

(1) CC: concrete crushing in flange boundary, RB: reinforcement bar buckling and rupture in flange boundary, LI: lateral instability in flange boundary, FB: out-of-plane buckling of flange boundary, LI/OOPS: lateral instability due to out-of-plane shear in flange boundary; WC/DC: web crushing due to in-plane diagonal compression

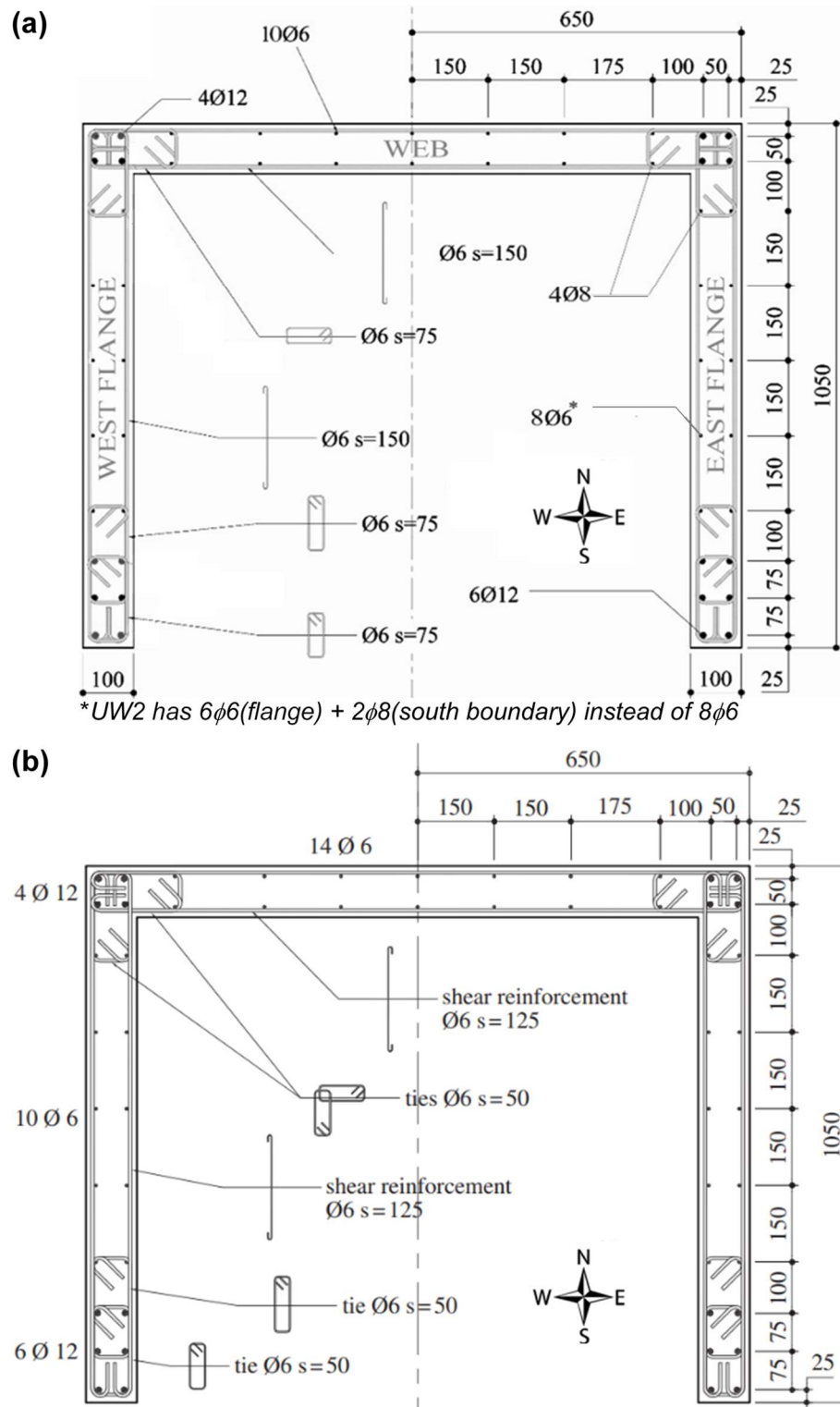


Figure 3.1. Wall specimen cross-sections and reinforcement details (all dimensions are in mm): (a) UW1 and UW2 (Hoult et al. 2023a), (b) TUB (Beyer et al. 2008a), (c) TUC (Constantin and Beyer 2016), (d) H-shaped Specimens.

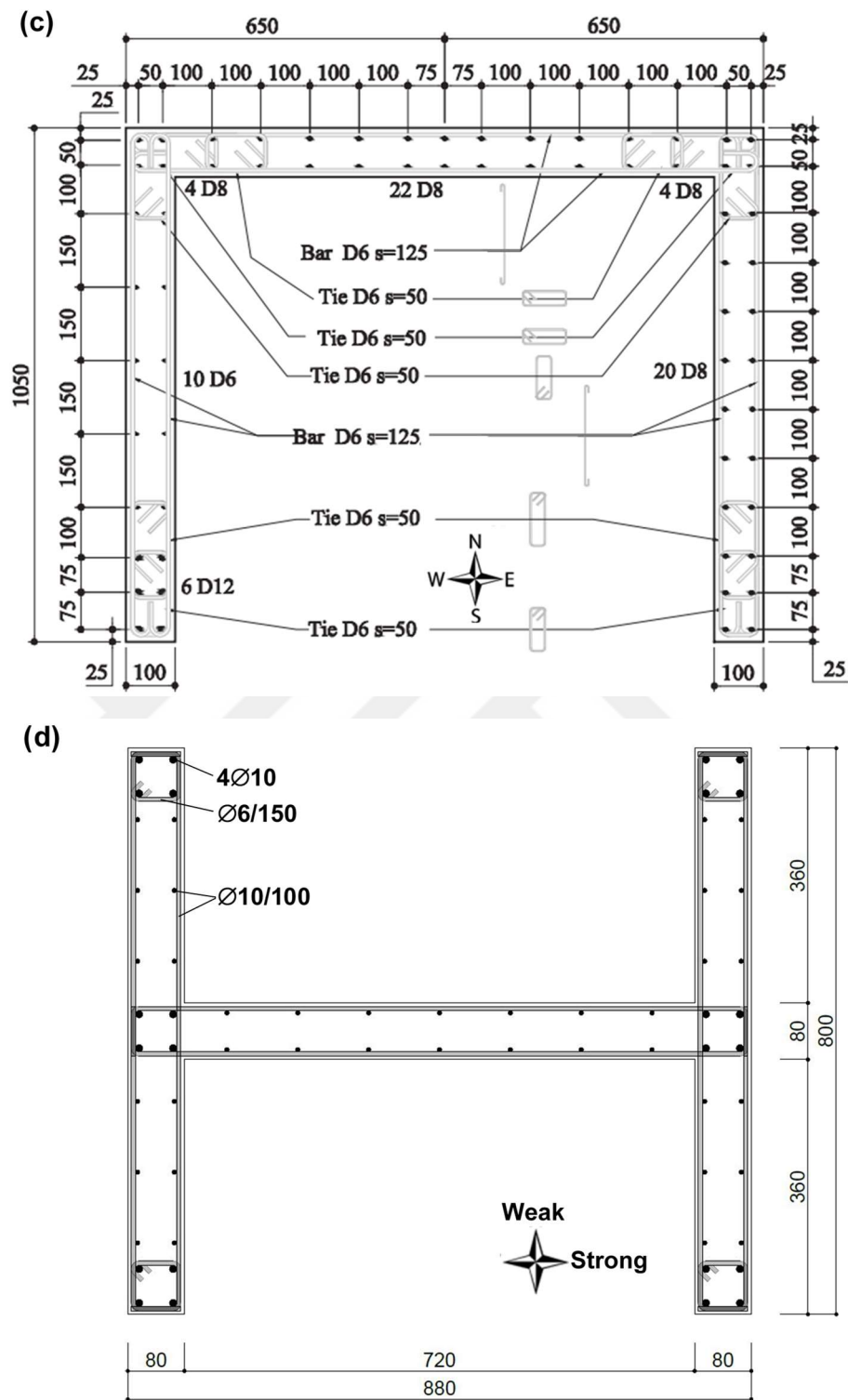


Figure 3.1. Wall specimen cross-sections and reinforcement details (all dimensions are in mm): (a) UW1 and UW2 (Hoult et al. 2023a), (b) TUB (Beyer et al. 2008a), (c) TUC (Constantin and Beyer 2016), (d) H-shaped Specimens (cont.).

Specimens UW1 and UW2 were half-scale wall prototypes, and had identical cross-sectional dimensions and almost identical reinforcement configurations, as shown in Figure 3.1a. The test setup used for specimens UW1 and UW2 included two horizontal actuators in N-S direction, together with three vertical actuators that were attached to a U-shaped RC collar beam at the top of the wall units (Figure 3.2). The horizontal actuators were used to apply uni-directional lateral loading on specimen UW1, and pure torsional loading on specimen UW2, where the lateral top displacement in N-S direction and the top twist rotation at the elevation of the actuators were controlled during the quasi-static tests of specimens UW1 and UW2, respectively. The three vertical actuators were used to apply constant axial load on the wall units, as well as an additional overturning moment at the top of specimen UW1, for increasing the shear span in the loading direction. For specimen UW1, the shear-span to depth ratios enforced during testing (7.51 in N to S direction, 5.43 in S to N direction) were significantly higher than the aspect ratio of the specimen (1.91), due to the additional overturning moment applied by three vertical actuators. Furthermore, due to the eccentricity between the geometric centroid of the wall cross section and the centroid of the three vertical actuators, the shear span-to-depth ratios in positive (N-S) and negative (S-N) loading directions also differed slightly.

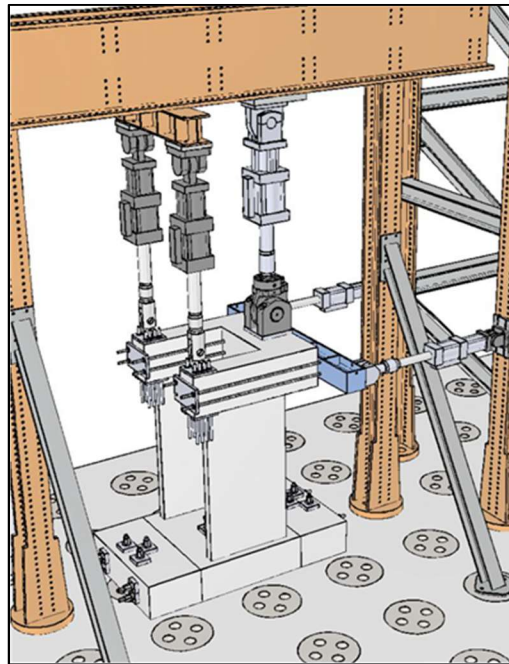


Figure 3.2. Test setup for non-planar wall specimens UW1 (uni-directional lateral load) and UW2 (pure torsion) (Hoult et al. 2023a).

On Specimen UW2, together with the constant and concentric axial load applied in an identical manner, pure torsional loading was applied in the form of two forces in opposite directions applied by the horizontal actuators. The experimentally-observed behavior of specimen UW1 specimen was predominantly flexural, where failure of the specimen occurred due to crushing of concrete together with buckling of longitudinal reinforcement (followed by post-buckling rupture upon load reversal) in the south-west boundary region of the wall (Figure 3.5a). It is important to note that, a construction defect, which involved off-center placement of the boundary-region reinforcement cage (as reported by Hoult et al. 2023a) impaired effective confinement in this particular wall boundary, which influenced the failure mode of this specimen. For wall specimen UW2, which was subjected to pure torsional loading, the primary behavior mode observed during testing was coupled shear and flexure on individual segments (web and flanges) of the wall, with the applied torque creating not only shear demands on the wall segments, but also giving rise to nonlinear flexural deformations (on especially the wall flanges), associated with warping of the cross-section. For this specimen, onset of concrete crushing was observed at the interior face of the east flange boundary region, leading to localized out-of-plane bending of the region, followed ultimately by lateral instability of the south-east boundary in the out-of-plane direction, due to through-thickness shear effect (Figure 3.5b).

The other two U-shaped wall specimens, TUB and TUC, were also half-scale test units with identical cross-sectional dimensions. Reinforcement configurations of specimens TUB and TUC differentiated in terms of longitudinal reinforcement amounts and web-flange intersection boundary region details, as shown in Figure 3.1b and Figure 3.1c. Both specimens were subjected to bi-directional lateral loading, using three horizontal actuators attached to a U-shaped RC collar beam at the top of the walls, two of which were oriented in the N-S direction of the walls, and the third oriented in the E-W direction (Figure 3.3 and Figure 3.4). Specimen TUB was subjected to reversed cyclic loading to increasing ductility levels in N-S, E-W, and diagonal loading directions, followed by a sweep-type top-displacement history at each displacement ductility level. Specimen TUC was tested under reversed cyclic loading in N-S, E-W, and diagonal directions, to increasing drift levels. During both tests, twisting (torsional deformation) of the walls was restrained by two parallel horizontal actuators in N-S direction. Constant axial load was applied using a pre-tensioned tendon that passes through a hollow core jack located on a steel load transfer beam installed

at the top of both wall specimens. Both specimens were experimentally-observed to follow coupled flexural/shear behavior throughout testing. Failure mode of specimen TUB (Beyer et al. 2008a) was reported as crushing of concrete under in-plane diagonal compression, in the unconfined region of the wall web (Figure 3.5c), whereas failure of specimen TUC (Constantin and Beyer 2016) was correlated with out-of-plane buckling, together with concrete crushing, at the west-flange boundary region of the wall (Figure 3.5d).

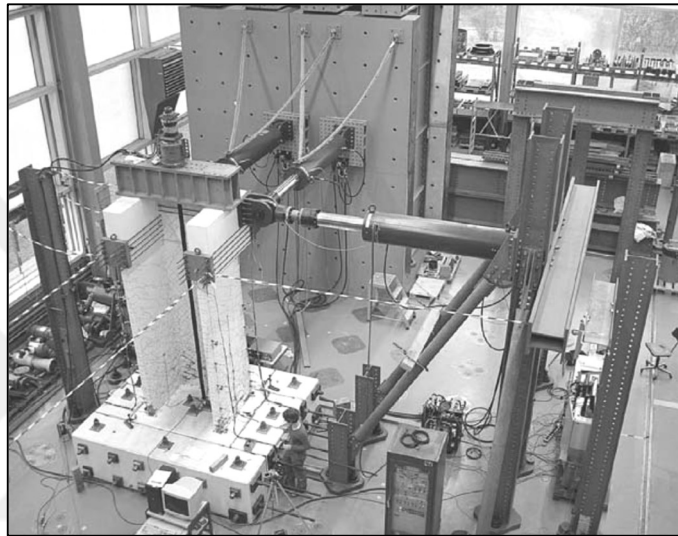


Figure 3.3. Test setup for non-planar wall specimen TUB (bi-directional lateral load)
(Beyer et al. 2008a).

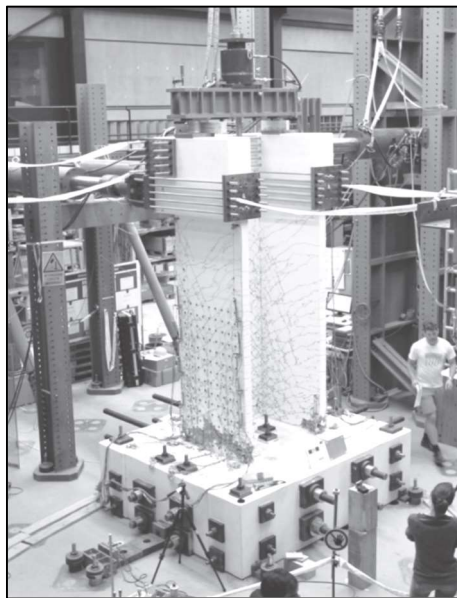


Figure 3.4. Test setup for non-planar wall specimen TUC (bi-directional lateral load)
(Constantin and Beyer 2016).

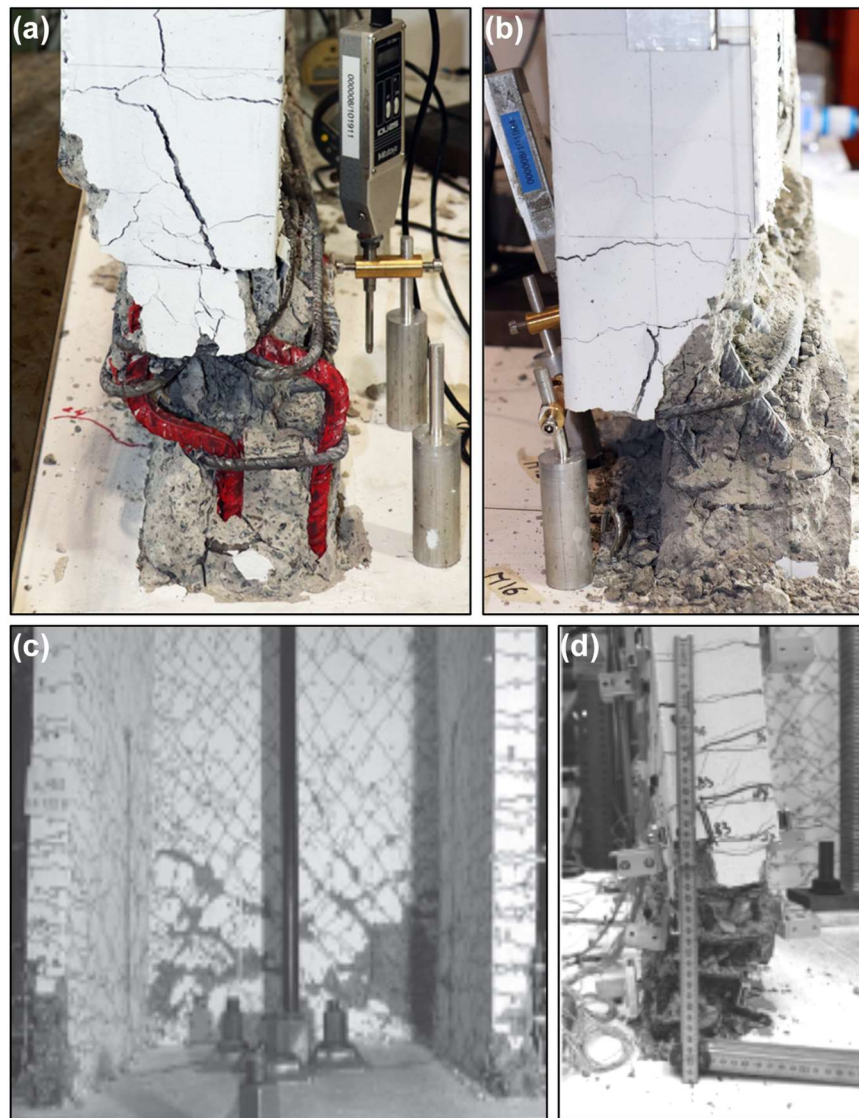


Figure 3.5. Experimentally-observed failure modes: a) Concrete crushing and out-of-plane reinforcement buckling at flange boundary of UW1 (Hoult et al. 2023a), b) Out-of-plane shear instability at flange boundary of UW2 (Hoult et al. 2023a), c) Web crushing of TUB (Beyer et al. 2008a), d) Out-of-plane buckling of flange boundary of TUC (Constantin and Beyer 2016).

Experimental validation of the LFSAFE model was lastly conducted against test results for six 1/12-scale H-shaped wall specimens that were tested by Maruta et al. (1999, 2000a, 2000b) under combined lateral and torsional loading conditions. Specimens HW25, HW50, HW75, HS25, HS50, and HD25 had identical geometries and reinforcement configurations (Figure 3.1d), but differentiated primarily in terms of the loading condition imposed during testing. The loading setup employed in the test campaign comprised four

inclined actuators installed on a stiff steel frame connected to a solid RC block located at on top of the wall units. This loading system was utilized for applying constant axial together with various combined lateral/torsional loading conditions on the wall specimens. During testing of each specimen, the actuator forces were controlled such that the resultant torque applied at the top of each wall is proportional to (i.e., a percentage of) the maximum bending moment developing at base of the wall (i.e., the overturning moment on the wall), for various lateral load directions. The tests were thus characterized by overturning-moment-to-torque ratios imposed on the wall specimens. The nomenclature HS, HW, and HD correspond to resultant lateral load applied in strong, weak, and diagonal directions of the wall cross-section (Figure 3.1d), respectively. Specimens HS25, HW25, and HD25 had an overturning-moment-torque ratio of 3.0 (75% overturning, 25% torque), while specimens HW50 and HS50 had a ratio of 1.0 (50% overturning moment, 50% torque), whereas specimen HW75 had a ratio of 0.33 (25% overturning moment, 75% torque). Shear spans for individual segments (flanges and web) of the walls were defined based on an idealized internal force (shear force under applied torque) distribution assumed for the wall cross-sections, resulting in the broad range of shear span-to-depth ratios listed in Table 3.1, which are defined for different wall segments under various loading conditions. The experimentally-observed behavior of the H-shaped wall specimens was combined flexural and torsional behavior, where all specimens exhibited flexural/torsional yielding of longitudinal reinforcement. The reported failure mode of the specimens was crushing of concrete at specific wall flange boundaries, due to compressive stresses developing under the combined effects of flexure (flexural compression) and torsion (diagonal compression), except specimens HS25 and HS50 for which significant concrete crushing was not observed.

3.2. Geometric Calibration, Boundary Conditions and Load Application

LFSAFE models for all wall specimens were generated in OpenSees (McKenna et al. 2000), following a unified and consistent approach for discretization of the model geometry and calibration of the material model parameters. Model elements at wall boundary regions were typically discretized along wall length such that the length to thickness ratio of the shell elements do not exceed 2.0, which requires at least two model elements side-by-side for boundary regions with 300mm length and 100mm thickness. The model elements are then discretized across the entire wall geometry such that the height-to-length ratios of all

elements are close to one. Model element discretization used for modeling of the wall specimens investigated are illustrated in Figure 3.6. All wall specimens were discretized along their thickness into nine membrane layers, including two cover concrete layers (unconfined concrete with no reinforcement), two horizontal reinforcement layers (horizontal reinforcement and unconfined concrete), two vertical reinforcement layers (vertical reinforcement and confined concrete) and three core concrete layers (confined concrete with no reinforcement).

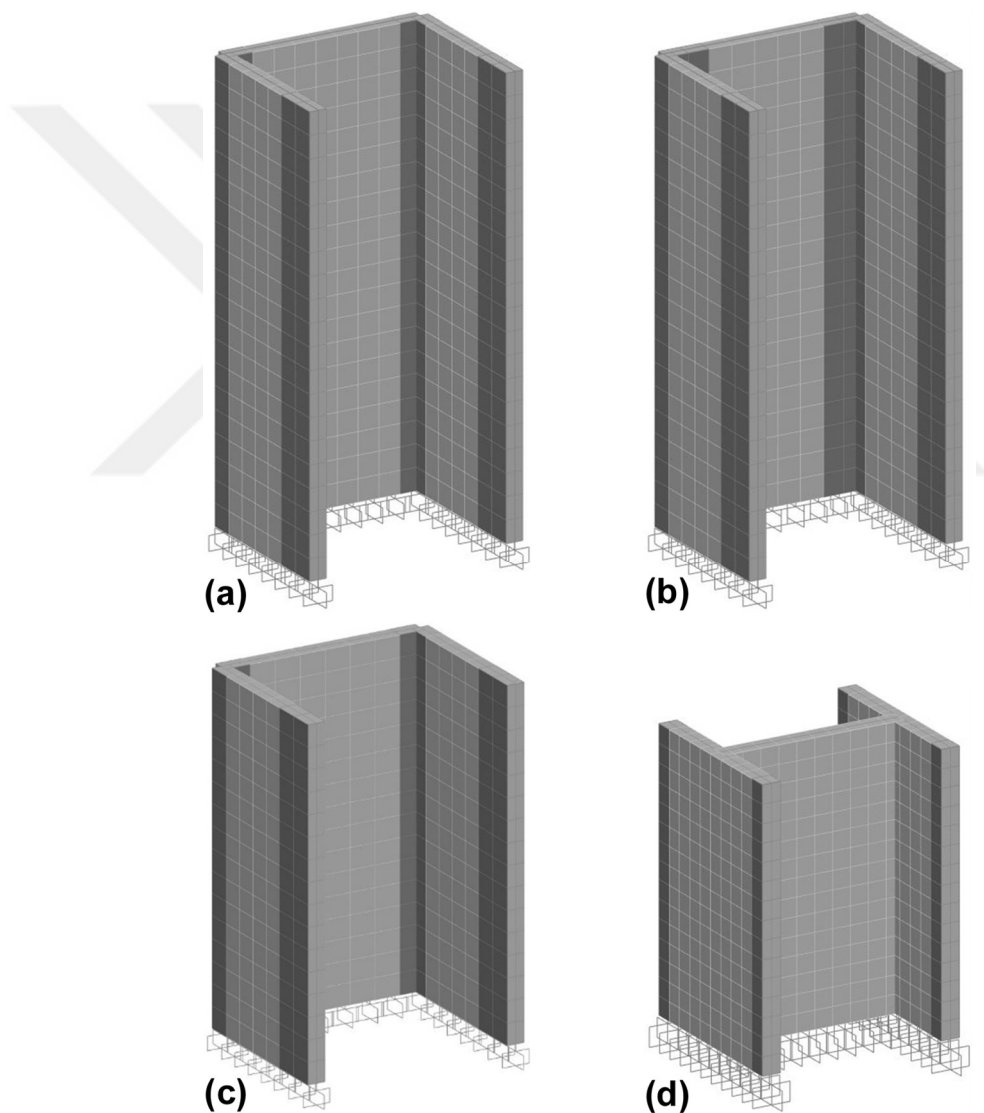


Figure 3.6. Model element discretization used for non-planar wall specimens: a) TUB, b) TUC, c) UW1 and UW2, and d) H-shaped walls.

In all models, both translational and rotational degrees of freedom of base of the walls are restrained, imposing fixed support conditions at wall-foundation interfaces. The

geometric constraints enforced at the top of the wall models differ, depending on the respective test setups and the loading conditions imposed during the tests. For specimen UW1, the U-shaped RC collar beam is considered as a rigid body constraint at the top of the wall model (since no warping is expected and plane sections are expected to remain plane at the top of the wall), and the additional overturning moment applied at the top of the specimen during testing was enforced as a concentrated moment applied on the rigid body constraint. Axial load on the specimen (as well as the aforementioned eccentricity in axial load) is also applied to the master node of the top rigid body constraint, which is located at the geometric centroid of the wall cross-section. The elevation difference between the top of the wall model and the locations of the horizontal actuators is also considered in the loading conditions imposed on the model. On the other hand, for modeling of specimen UW2, only a rigid diaphragm constraint is enforced at the top of the wall model, so that possible warping of the wall cross-section under pure torsional loading is not fully restrained. Therefore, axial load is applied as distributed point loads at top nodes of the wall model, creating the same axial load resultant (in terms of magnitude and location) applied during testing. Partial restraint against warping of the wall caused by the RC collar beam is considered by modeling of the collar beam using linear elastic frame elements with large flexural stiffness values. However, a cracked torsional stiffness value (i.e., $0.05GJ$) is defined for the middle segment (above the wall web) of the U-shaped collar beam, as discussed in the following section. Differently from specimens UW1 and UW2, the RC collar beams located at the top of specimens TUB and TUC were modelled as vertical extensions of the wall model, in order to apply the lateral loads at different elevations for the two loading directions (i.e., at the elevations of the N-S and E-W actuators). Individual rigid diaphragm constraints are defined along the web and the two flanges of these wall models, at the corresponding actuator elevations. Axial load enforced on these specimens (using the hollow core jack) is applied on the models, at nodes corresponding the location where steel load transfer beam is supported on the collar beam. Lastly, for the H-shaped walls, the rigid load transfer slab (solid RC block) at the top of the wall specimens is represented by using a rigid body constraint defined at the top of the corresponding wall models. Representative deformed shapes for the walls, which are directly obtained from the analysis results presented in this study, are shown in Figure 3.7. The analytically-obtained deformed shapes clearly reflect the loading conditions and boundary constraints imposed on the wall specimens during testing.

It is also interesting to observe that the analytical deformed shapes also show localized out-of-plane bending deformations near the base of some of the walls (e.g., Figure 3.7d).

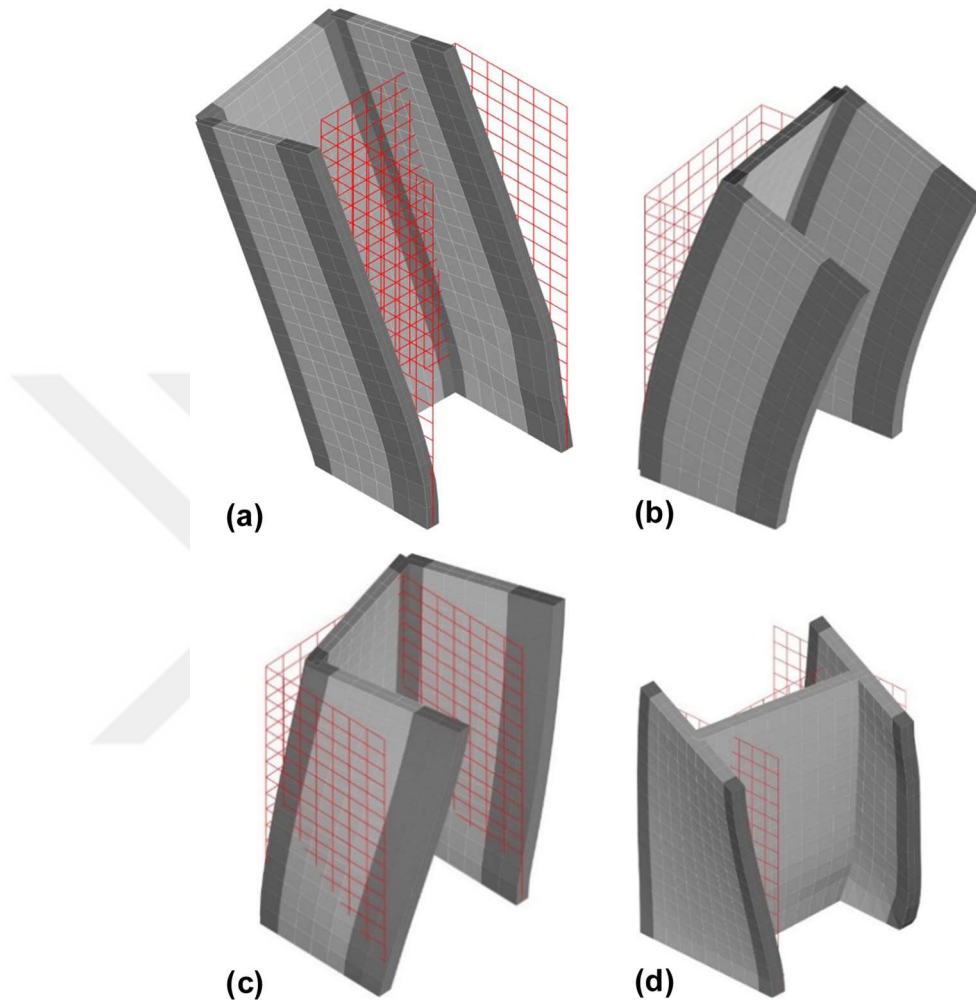


Figure 3.7. Representative deformed shapes for non-planar wall specimens: a) TUB, b) UW1, c) UW2, and d) HD25.

3.3. Calibration of Material Model Parameters

Whenever material test results (e.g., stress–strain curves for unconfined concrete in compression and reinforcing steel in tension) were available, the material model parameters used in the LFSAFE model formulation were calibrated to match the as-tested properties of the materials used in construction of the wall specimens. Material model parameters that could not be experimentally-calibrated (e.g., parameters related to confined concrete, concrete tensile strength, reinforcement buckling, tension stiffening, hysteretic material

model parameters, shear-friction coefficient on crack surfaces, etc.) were calibrated based on well-established empirical relationships reported in the literature, using a uniform calibration approach used for all wall specimens, as described in the following paragraph. The material model parameters were not adjusted or optimized to match model predictions with test results. As well, in this particular study, it was preferred not to regularize the post-peak compressive stress-strain behavior of concrete depending on model element size (e.g., Coleman and Spacone 2001), in order to assess the accuracy of the model (especially in capturing strength degradation) when a reasonable geometric discretization is adopted and no stress-strain regularization is applied.

In absence of stress-strain test results on concrete, the empirical equations by Chang and Mander (1994) are used to describe the ascending region of the monotonic stress-strain envelope for unconfined concrete under compression. For confined concrete, the ascending region of the stress-strain envelope is defined according to the confinement model by Mander et al. (1988). For both confined and unconfined concrete, the slope of descending region of the stress-strain envelope is calibrated to match the post-peak slope of the Saatcioglu and Razvi (1992) model. Tensile strength of concrete is defined as $f_t = 0.31\sqrt{f'_c}$ (MPa) at a cracking strain value of $\varepsilon_t = 0.00008$, as recommended by Belarbi and Hsu (1994) for reinforced concrete membranes. As well, to consider the effect of tension stiffening on the average tensile stress-strain behavior of concrete, the shape of the tension envelope of the Chang and Mander (1994) model is adjusted to match the post-crack average stress-strain relationship proposed by Belarbi and Hsu (1994). For reinforcing steel bars, tensile strength and tensile strain hardening ratio parameters (for bare bars) are first defined based on material test results reported in respective wall test documents. However, these parameters are modified to reflect the effect of tension stiffening on the tensile behavior of reinforcing bars embedded in concrete, based on the empirical relationships by Belarbi and Hsu (1994). For model elements where reinforcement buckling is not considered (i.e., along unconfined wall regions), in the hysteretic Menegotto and Pinto (1973) model, steel yield strength and strain hardening ratio parameters under compression are taken identical to those for bare bars under tension. In model elements representing the confined boundary regions of the walls, to consider buckling of longitudinal reinforcement, the monotonic envelope parameters and hysteresis rules defined in the Dhakal and Maekewa (2002a) model are directly adopted. A slight simplification is made on the hysteresis rules of the Dhakal and

Maekawa (2002a) model, where the linear transition regions that are used to merge the hysteretic loops with the monotonic compression and tension envelopes are neglected. In both the Menegotto and Pinto (1973) and Dhakal and Maekawa (2002a) models, the cyclic stiffness degradation parameters are defined as $R_0 = 20$, $\alpha_1 = 18.5$, and $\alpha_2 = 0.15$.

To represent shear aggregate interlock along crack surfaces, the monotonic envelope parameters and the hysteresis rules developed by Tassios and Vintzēleou (1987) for rough crack surfaces, are directly adopted. However, the interlock friction coefficient is defined simply as 0.40 (vs. 0.44 recommended by Tassios and Vintzēleou, 1987), and the original hysteresis rules are slightly simplified (Figure 2.9b). For simulating dowel action on both vertical and horizontal reinforcing bars, the smeared stress-strain constitutive modeling approach developed by He and Kwan (2001) is adopted, together with the monotonic envelope parameters and the hysteresis rules defined by Vintzēleou and Tassios (1986, 1987). However, the hysteresis rules are slightly simplified into origin-oriented unloading and reloading rules (Figure 2.10b). Cyclic degradation effects on the shear aggregate interlock and dowel stresses are also considered, using parameters described in Tassios and Vintzēleou (1987), Vintzēleou and Tassios (1986), and later modified by Thermou et al. (2014).

3.4. Nonlinear Analysis Solution Strategy for Quasi-Static Response Simulations

In the open-source software framework OpenSees, the analysis solution methodology is created as an analysis object incorporating different components such as a constraint handler (constraint equations enforcement method), a numberer (method for mapping between equation numbers and DOFs), a system of equations (for storage and solution of the system of equations), an integrator (for determination of predictive step size), a solution algorithm (solution method for the nonlinear equation at a loading step) and a convergence test (convergence criterion). With combinations of these components, various analysis solution methodologies can be created in the framework, but selection of a generic methodology that is capable of providing reliable analysis results is a challenging task due to highly nonlinear nature of the reinforced concrete behavior and the adopted modeling methods such as LFSAFE model to simulate it.

Within this context, an analysis object for nonlinear solution strategy is created for quasi-static response simulations presented in the forthcoming sections, employing Transformation constraint handler, Reverse Cuthill-McKee (RCM) (Cuthill and McKee, 1969) numberer, UmfPack (Davis, 2004) system of equations, Load Control integrator for gravity load analysis and Displacement Control integrator for quasi-static loading (single DOF controlled for UW1, UW2 and HW specimens, three DOFs controlled for TUB and TUC), Krylov-Newton (Scott and Fenves, 2010) (with current stiffness) and Modified Newton (with initial stiffness) algorithms, and Norm Displacement Increment and Unbalance (NormDispAndUnbalance with max-norm) convergence test. In displacement-controlled analysis, solution within a step is first searched by Krylov-Newton algorithm, if this approach fails to find a solution, Modified Newton method is used later. If both algorithms fail to find a solution within a step, the step size is reduced (i.e. divided to 20) only for that step and the solution is re-searched with Modified Newton method. Displacement/twist rotation step sizes employed in the analysis obtained by dividing the target peak displacement/twist rotation values to a constant number (i.e. 25, 40, 50, 100), using individual values for each specimen, resulting different step sizes between loading peaks, as well as total number of load steps. Finally, convergence criteria for displacement increments and unbalanced forces are varying between 0.0025mm-0.005mm (for translational displacements) and 0.25kNm-0.75kNm (for unbalanced moments), respectively.

3.5. Model Validation of a U-Shaped Wall under Uni-Directional Lateral Loading

In order to demonstrate the capabilities and assess the accuracy of the LFSAFE model in simulating the experimentally-observed quasi-static responses of non-planar walls, test results reported by Hoult et al. (2023a), for two U-shaped wall specimens, one tested under uni-directional lateral loading (specimen UW1) and the other tested under pure torsion (specimen UW2) were first compared with model results. Preliminary analysis results obtained for specimen UW1 showed that the construction defect at the southwest boundary region of the wall, in the form of an off-center confined rebar cage and a reduced confined concrete area, had a notable effect in on the failure mode of the specimen (concrete crushing and reinforcement buckling at the southwest boundary), as well as the associated degradation in lateral load at negative 2% drift ratio. In the model, concrete at the southwest boundary

region of specimen UW1 was assumed to be unconfined, as the simplest approach for representing the disrupted confinement mechanism in this region. Comparison of the experimentally-measured and analytically-obtained lateral load vs. top displacement responses for this specimen (Figure 3.8) shows that the LFSAFE model successfully predicts the lateral load capacity of the wall in both loading directions, as well as the drift capacity of the wall in the negative loading direction. The hysteretic response characteristics of the wall, including unloading stiffness values, cyclic stiffness degradation, and pinching behavior are also well-represented in the analysis results. The model successfully captures the experimentally-observed strength degradation (degradation in lateral load) at negative 2.0% drift, associated with concrete crushing and reinforcement buckling (Figure 3.5a) at the southwest boundary region with the aforementioned construction defect.

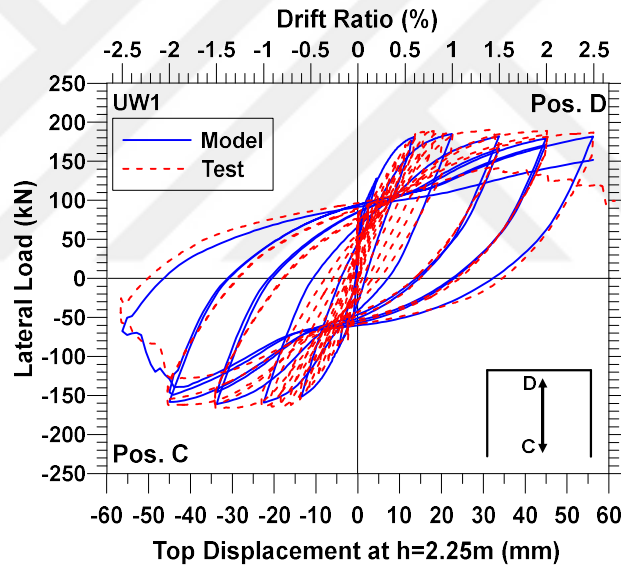


Figure 3.8. Lateral load vs. top displacement response of specimen UW1.

On the other hand, the successive lateral load degradation during the last loading cycle to positive 2.5% drift is not replicated in the model results, since that degradation stemmed from post-buckling rupture of the southwest boundary longitudinal bars upon load reversal, while reinforcement rupture is not considered in the constitutive model formulation adopted for reinforcing steel. It must be mentioned that a hypothetical model configuration where the construction defect at the southwest boundary region of the wall specimen was neglected, provided a nearly identical lateral load–displacement response prediction for the wall, with the exception that the experimentally-observed lateral load degradation at negative 2% drift was not reflected in the analysis results.

At the local response level, the LFSAFE model is first validated, against representative experimentally-measured average vertical (longitudinal) strain profiles measured along the height of wall specimen UW1 (Figure 3.9). Experimental measurements corresponding to various peak drift levels are obtained from Linear Variable Differential Transformers (LVDTs) attached to the wall at the southwest flange tip (Figure 3.9a), southeast flange tip (Figure 3.9b), and the northwest web-flange intersection (Figure 3.9c, d). Since the gauge lengths of the LVDTs overlap at the southeast flange tip of the wall, discontinuities (overlaps) in the measured average strains are shown in Figure 3.9b, while for all cases, the model predictions are continuous. Model results for the average strains are presented as the Gauss-weighted average of the strain values obtained at the nine Gauss points of each model element.

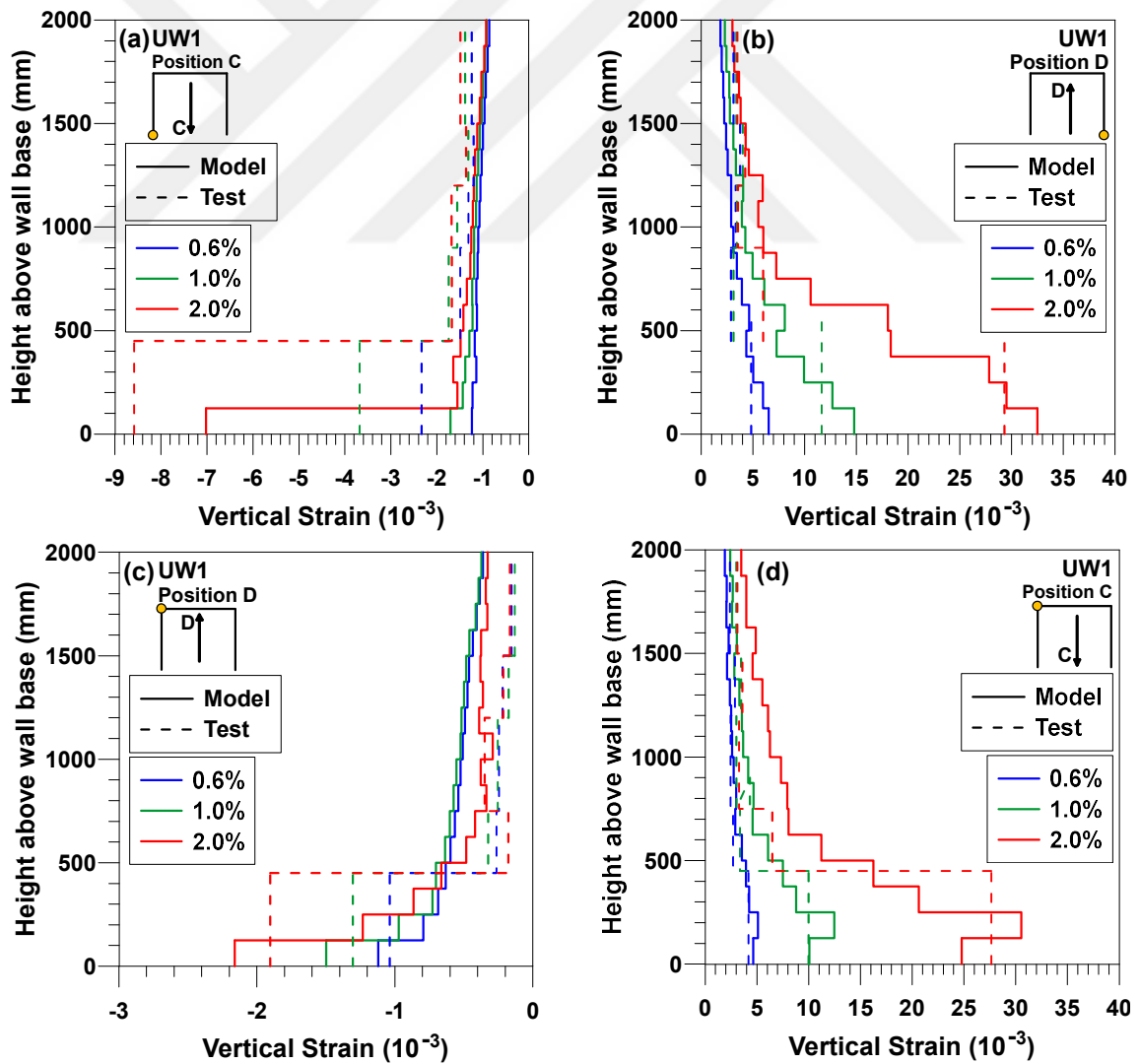


Figure 3.9. Vertical strain profiles along wall height for specimen UW1.

As shown in Figure 3.9a, for this particular specimen, the model significantly underestimates the compressive strains measured at the tip of the southwest flange, especially along the first quarter of the wall height. However, this is not deemed to be a characteristic attribute of the model, as such discrepancy is not observed in comparison of model predictions with test measurements for other U-shaped wall specimens, as will be discussed in the following sections. At all drift levels, the model provides reasonable estimations for the tensile strain profiles measured along tip of the southeast flange, (Figure 3.9b) and along the corner of the northwest web-flange boundary (Figure 3.9d). Moreover, the compressive strains measured along the corner of the northwest web-flange boundary (Figure 3.9c) are also predicted by the model reasonably well.

3.6. Model Validation of a U-Shaped Wall under Pure Torsional Loading

When the torque vs. top twist rotation response of wall specimen UW2 was simulated, it was observed that model predictions for the torque capacity and twist rotation capacity (prior to torque degradation) of the wall were sensitive to the stiffness properties assumed for the U-shaped RC collar beam located at the top of the wall, due to its restraining effect against warping along the wall cross-section under torsion. In the final model configuration, the collar beam was represented using linear elastic frame elements connected to the top nodes of the wall model, where the stiffness properties of the frame elements were adjusted to represent the expected behavior of the beam segments. First, large axial and flexural stiffness values were defined ($1000AE$ and $1000EI$, where E is the concrete elastic modulus) for all three frame elements used for modeling of the U-shaped collar beam, enforcing the wall cross-section at the top to remain plane. Next, the torsional stiffness of the middle segment of the collar beam (the segment located at the top of the wall web) was reduced, since the torsional stiffness of this middle segment was found to be the main source of the warping restraint (restraint against warping of the wall flanges relative to the wall web) exerted by the collar beam on the wall. Photographs of the wall specimen at the end of the test showed diagonal cracks on the web segment of the collar beam (Hoult et al. 2023a), which is believed to associated with this phenomenon. Since the torsional stiffness of reinforced concrete beams is expected to significantly decrease upon torsional cracking (e.g., $0.05GJ$; Bruun et al. 2020), a stiffness coefficient of 0.05 was used to reduce the torsional stiffness of the middle segment of the U-shaped collar beam.

With this configuration, analysis results obtained using the LFSAFE model show that the model successfully predicts the torque capacity and the hysteretic torque–twist angle response characteristics of wall specimen UW2 under pure torsion (Figure 3.10). The experimentally-observed failure mode of the wall was reported as crushing of concrete, together with localized out-of-plane bending, followed by out-of-plane (through-thickness) shear failure of the flange boundary regions (Figure 3.5b). (approximately 5%) of the wall specimen due to the analytically-estimated concrete crushing and reinforcement buckling of in the wall flange boundary regions.

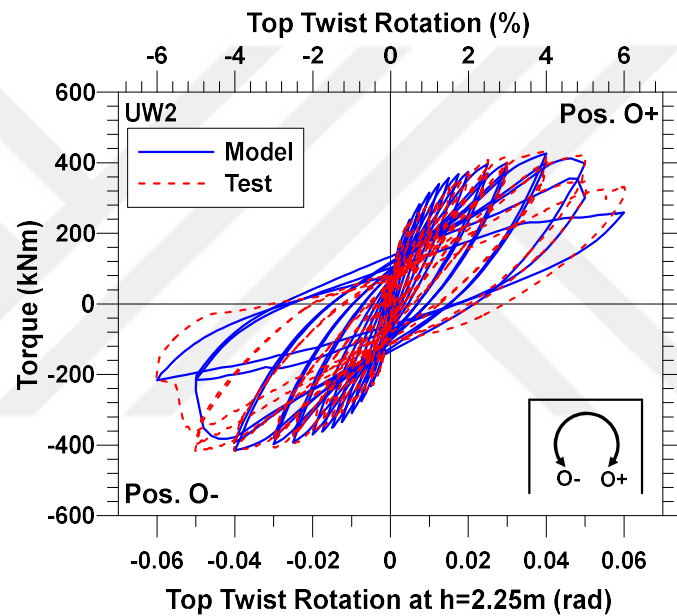


Figure 3.10. Torque vs. top twist rotation response of specimen UW2.

It must be noted that, in a mechanical sense, restraint against warping is expected to significantly impact the nonlinear torsional behavior of reinforced concrete components, similarly to how restraint against elongation would impact their nonlinear flexural behavior. Restraint against warping alters the behavioral mechanisms associated with widening of the torsional cracks and reinforcement yielding, and influences the vertical (axial), horizontal (transverse), and shear stress and strain distributions on the wall. Typically, the warping restraint increases the axial compressive stresses developing in regions of the wall where axial elongation (that would otherwise be associated with warping) is prevented. In order to demonstrate this with LFSAFE model, comparison of analysis results for two additional cases with the original model configuration (model response in Figure 3.10, collar beam with

0.05GJ in Figure 3.11) are presented in Figure 3.11, first neglecting the top collar (no collar beam in Figure 3.11a), providing no restraint against warping of the wall, and second, assuming top collar of the specimen UW2 is infinitely rigid (rigid collar beam in Figure 3.11b). When the U-shaped collar beam is not included in the analysis model, the predicted torque capacity reduces to 340kN.m (versus the capacity of 426kN.m predicted by the original model configuration). In contrast, when a rigid collar beam is considered in the analysis, the torque capacity estimation of the model increases to 590kN.m, but in this case degradation in torque initiates at a smaller twist angle (0.03rad).

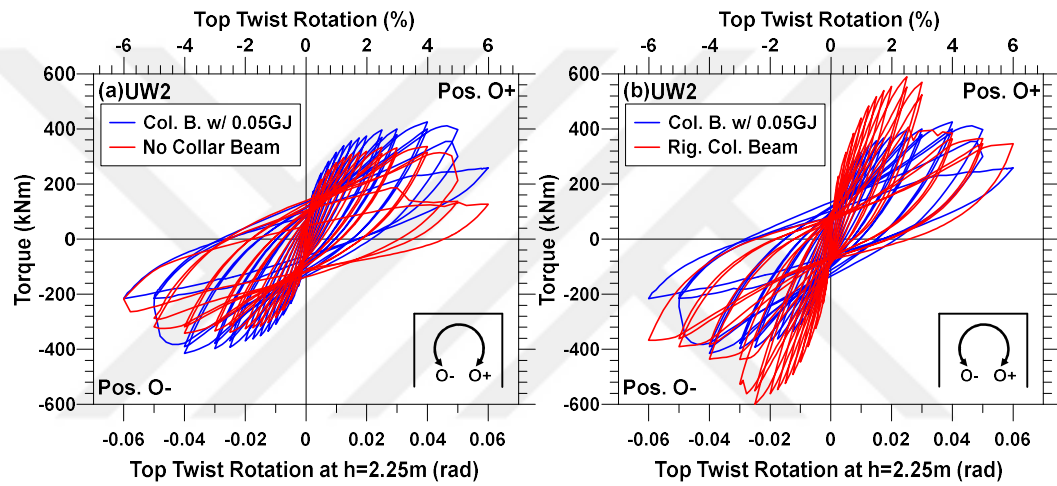


Figure 3.11. Effect of the collar beam stiffness on torsional response of specimen UW2: the original model configuration vs. (a) no collar beam case, and (b) rigid collar beam case.

For wall specimen UW2, which was tested under pure torsion, Figure 3.12 compares model predictions with test results for the strains on the outmost longitudinal reinforcing steel bars located in the southwest flange boundary tip of the wall. The strains were measured using continuous fiber-optic strain gauges (distributed fiber optic sensors– DFOS) that were attached to the both the inner (close to the inner wall surface) and outer reinforcing steel bars (for detailed information about DFOS, Hoult et al. 2023c). For comparison, average longitudinal strains measured using LVDTs attached to the flange tips (at mid-thickness) are also shown in the figure. Comparison of results presented in Figure 3.12a (inner gauge) and Figure 3.12b (outer gauge) demonstrates that, due to its layered-shell formulation, the LFSAFE model is capable of simulating the out-of-plane (through-thickness) strain gradient on the wall. As shown in these two figures, under torsional loading, in both model results and test measurements, compressive strains developing on the inner reinforcing steel bars

are larger compared to those on the outer bars. The model provides reasonable predictions of the different magnitudes of compressive strains developing in the inner and outer bars, which is a significant attribute of the model towards simulating out-of-plane instability failures in wall flange boundaries, especially upon future implementation of geometric nonlinearity effects in its formulation. Furthermore, as demonstrated in Figure 3.12c and Figure 3.12d, under torsional loading, the model also provides reasonable estimations for the magnitudes and distribution of tensile longitudinal strains developing along wall height, when LVDT and DFOS measurement are both considered, while slightly overestimating tensile strains measured from inner gauge near the base of southwest flange tip. Note that, the out-of-plane strain gradient observed in compressive strain estimations is also clearly noticeable in tensile strain estimations presented in Figure 3.12c and Figure 3.12d.

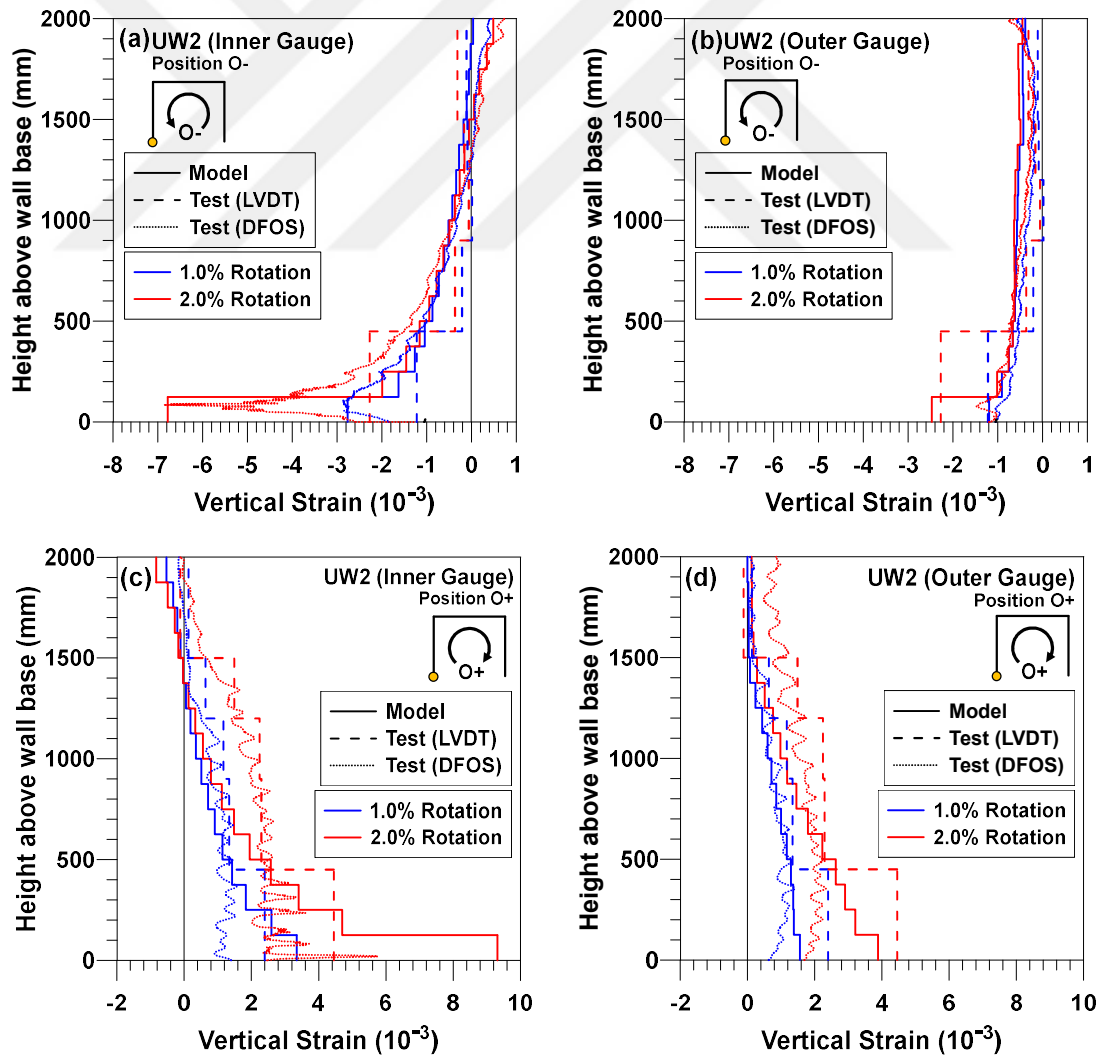


Figure 3.12. Vertical strain profiles obtained from fiber-optic strain gauges and LVDTs along wall height for specimen UW2.

Figure 3.13 compares model estimations for the distribution of inner-surface longitudinal strains at the base of wall specimen UW2, with experimentally-measured strain profiles obtained from vertical LVDT measurements attached to the inner surfaces of the web and flanges at the base of the wall, under two representative twist rotation levels in both positive and negative loading directions. In the figure, vertical strains are presented on web and flange parts of the specimen separately, where data at inner and outer parts of the cross-section correspond to compressive and tensile strains, respectively. Magnitudes of the strains are presented such that each major tick in horizontal and vertical axes correspond to 10mm/m (1.0%) strain values measured from the web and flanges individually (i.e. approximately 2.0% tensile strain model prediction at southwest flange tip of the cross-section under 3.0% rotation in positive loading direction in Figure 3.13b). The model significantly underestimates the tensile strains developing on the flanges subjected to torsional tension. The reason for this discrepancy is uncertain; however, it is believed to be associated with the large flexural stiffness (i.e., $1000EI$) assumption used in modeling of the U-shaped collar beam, enforcing plane sections to remain plane (along an individual flange) at the top cross-section of the wall, and negligence of strain penetration effects in analysis. In contrast, tensile strains developing in the web, and both tensile and compressive strains developing in the flanges subjected to torsional compression (as well as the transition of these strains from tension to compression) are generally well-represented in the model results.

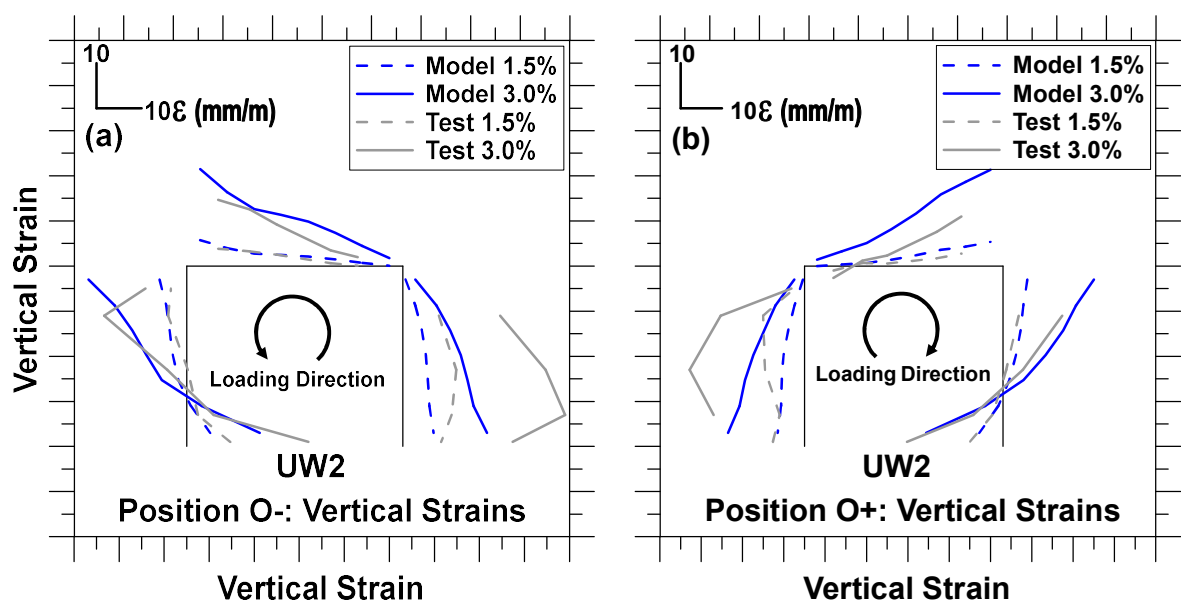


Figure 3.13. Vertical strain distributions across wall base for specimen UW2.

3.7. Model Validation of a U-Shaped Wall under Diagonal/Sweep Lateral Loading

Experimental validation of the LFSAFE model is further achieved against test results for wall specimen TUB (Beyer et al. 2008), which was subjected to a complex multi-directional lateral load history. Figure 3.14a (in E-W direction), Figure 3.14b (in N-S direction), Figure 3.14c, d (along diagonal direction), and Figure 3.14e (square-root-of-sum-of-squares) compare the lateral load–top displacement response predictions of the LFSAFE model with experimentally-measured responses. In order to directly replicate the complex loading protocol shown in Figure 3.14f, a multi-directional (multi-DOF) displacement controlled nonlinear analysis solution strategy was first implemented into OpenSees. The baseline formulation of this strategy is described in the thesis by Bilgin (2018). The experimentally-measured lateral displacements on the wall specimen at 2.95m elevation were applied to the analysis model, controlling the lateral displacements in E-W and N-S directions at the northeast corner, and in N-S direction at the northwest corner of the wall model cross-section. Similar to the manner as to how torsional twisting of the specimen was restrained during testing, the same displacement history in the N-S direction was applied at the northeast and northwest corners of the analysis model, preventing any twist rotation at the top. Furthermore, since the horizontal actuators were attached to the wall specimen at different elevations (N-S at 2.95m, E-W at 3.35m), the horizontal forces used in the displacement-controlled analysis were defined at the same elevations of the actuators, although lateral displacements were controlled at an elevation 2.95m.

The lateral load–top displacement response comparisons presented in Figure 3.14 along various loading directions demonstrate that the LFSAFE model is capable of simulating, with reasonable accuracy, the backbone (stiffness, yield load and displacement, maximum lateral load, and strain hardening) characteristics; as well as the hysteretic (unloading stiffness, hysteretic stiffness degradation, and pinching) characteristics of the experimentally-measured response of wall specimen TUB. For this wall, degradation in lateral load was not observed either in test measurements or in model response predictions. The observed failure mode of this specimen was reported as concrete crushing along the base of the wall web under diagonal compression; however, it did not create any strength loss, possibly due to redistribution of compressive stresses. In general, the only discrepancy between model predictions and test results is that whenever the applied lateral load

incorporates a N-S component, the model slightly underestimates the experimentally-measured lateral loads during the initial loading cycles in the positive loading direction (at positions D and F) and slightly overestimates the lateral loads during the final loading cycles in the negative loading direction (at positions C and E). The load–displacement response of the wall during the uni-directional loading cycles (Figure 3.14a and Figure 3.14b) is well-simulated. Figure 3.14c and Figure 3.14d present analytical and experimental response comparisons for the E-W and N-S components of the load–displacement response, during the diagonal loading cycles, for which a reasonably accurate response prediction is again obtained. The square-root-of-sum-of-squares (SRSS) resultant of the lateral load vs. top displacement response of the wall presented in Figure 3.14e for the diagonal loading cycles (where the resultant lateral load and displacement are not necessarily oriented in the same direction) also show good agreement between model results and test measurements. Last, Figure 3.14f illustrates replication of the multi-directional top displacement history imposed during testing, with the multi-directional displacement control solution strategy implemented in OpenSees, which renders OpenSees suitable for conducting quasi-static analysis under such loading conditions without having to define any analysis stages. Overall, the LFSAFE model is shown to be accurate in simulating the global lateral load–displacement response of this specimen, under the complex multi-directional loading history applied during testing.

The experimentally-measured flexural deformation contributions to top displacement of specimen TUB, during the N-S (Figure 3.15a), E-W (Figure 3.15b), and diagonal (Figure 3.15c, d) loading cycles, are used to assess the capabilities of the model in predicting flexural and shear deformations developing along different segments (flanges and web) of non-planar walls under bi-directional lateral loading. The experimental data in the figure does not consider small-magnitude sliding shear deformations that were measured at the base of the wall (with maximum contributions to top displacement of approximately 1.2%, 2.2%, and 4.4% for the web, west flange, and east flange, respectively). Consistently, the present formulation of the LFSAFE model is not capable of simulating formation of a continuous horizontal crack at a wall–foundation interface and shear sliding deformation along that interface crack.

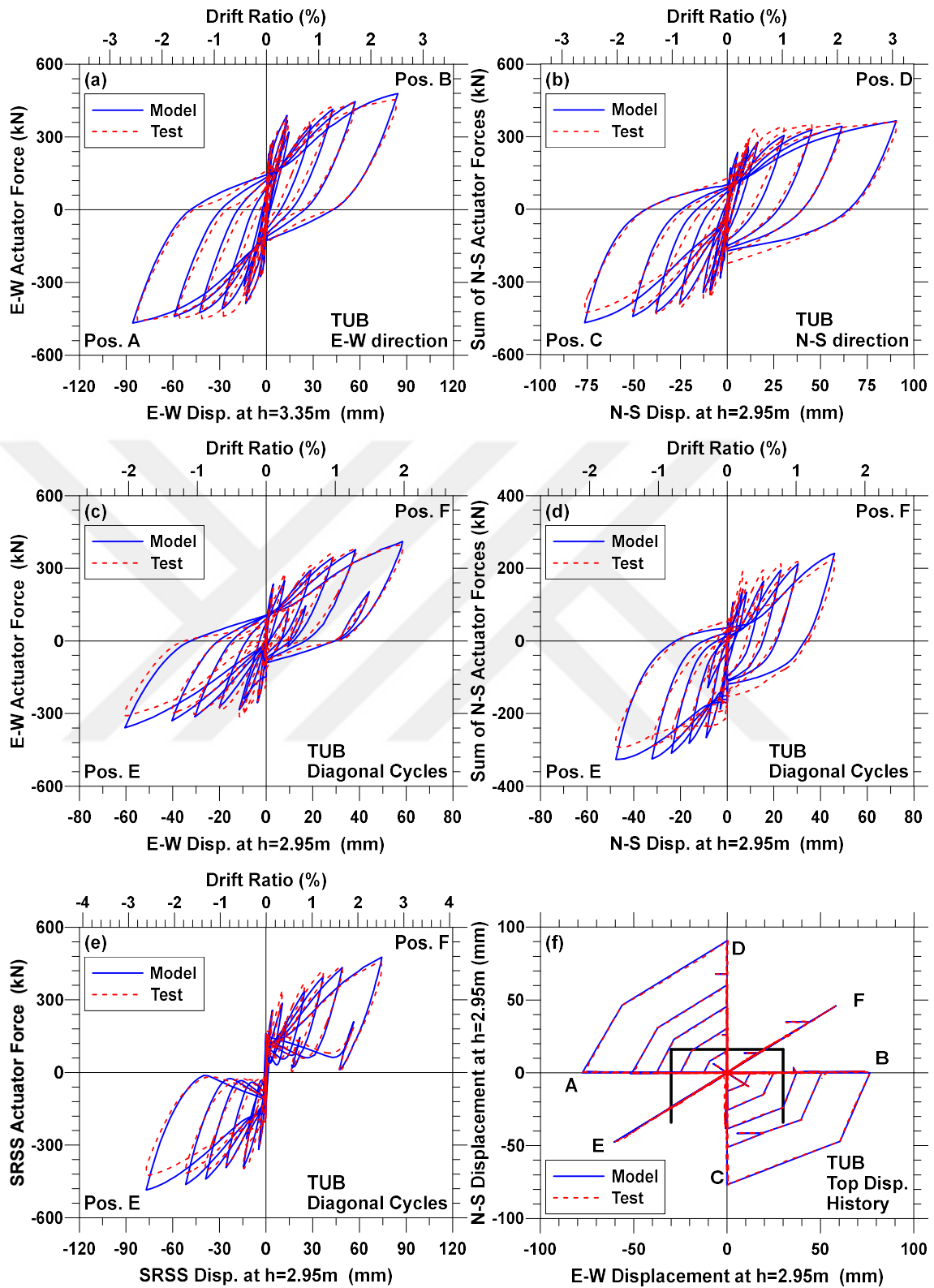


Figure 3.14. Lateral load vs. top displacement responses for specimen TUB.

Comparisons presented in Figure 3.15 reveal that the model slightly overestimates the measured flexural deformation contributions in the flanges of the wall during the N-S

loading cycles (Figure 3.15a), whereas for the E-W loading cycles, model predictions for the contribution of flexural deformations in web coincide with test measurements (Figure 3.15b). During the diagonal loading cycles, the model moderately overestimates the flexural deformation contributions to top displacements in the N-S direction, in both the east and west flanges of the wall (Figure 3.15c), while it captures the contributions of flexural deformation in the wall web to top displacements in the E-W direction (Figure 3.15d). Results presented in Figure 3.15 foremost reveal that the LFSAFE model successfully simulates different intensities of shear-flexure interaction behavior (different levels of shear deformations) in the web and flanges of U-shaped walls under uni- or bi-directional lateral loading. Furthermore, and importantly, as can be observed in Figure 3.15c, in both model predictions and test measurements, when a particular wall flange, (e.g., west flange) is subjected to flexural tension due to one component of the bi-directional lateral load, it undergoes higher shear deformation under the perpendicular component of the same lateral load.

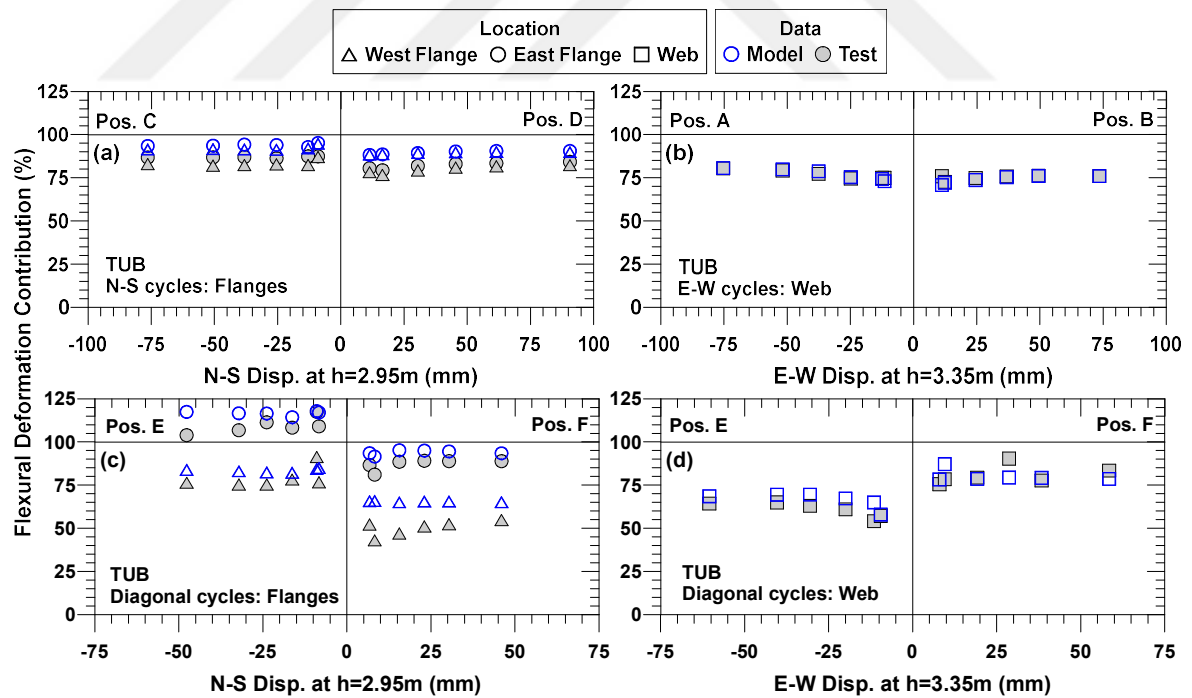


Figure 3.15. Flexural deformation contributions to top displacement, for specimen TUB.

Results presented in Figure 3.15c are also interesting in the sense that in both model results and test measurements, when the east flange of the wall is under flexural compression, the contribution of flexural deformations in the east flange exceed 100% of the top

displacement of the wall in N-S direction. A close scrutiny of the model results reveals that this is due to a complicated internal shear flow mechanism developing in the wall under bi-directional loading, where shear deformations (shear strains) and flexural deformations (curvatures) in the flange can add up to lateral displacements in opposite directions. On the whole, the model can be deemed successful in simulating nonlinear shear behavior and shear-flexure interaction in U-shaped walls under uni or bi-directional lateral loading, as well as predicting larger shear deformations in wall segments (e.g., flanges) that are subjected to flexural tensile demands, depending on the sense of the bi-directional lateral load.

Accuracy of the LFSAFE model in predicting the experimentally-measured longitudinal strain distribution along the height of specimen TUB is evaluated in Figure 3.16. In the figure, average strain measurements obtained from vertical LVDT's attached to the outer surface of the wall at the flange tip and at the corner of the web-flange intersection, are compared with model estimations. Similarly to specimen UW2, model results for the average strains are calculated as the Gauss-weighted average of the strain values at the nine Gauss points of each model element. The vertical strain profiles are compared at the southwest flange tip at loading position C (Figure 3.16a) (under compression) and loading position D (Figure 3.16b) (under tension), at the northeast web-flange intersection (Figure 3.16c) at loading position A (under tension), and the northwest web-flange intersection (Figure 3.16d) at loading position A (under compression). Results presented in the Figure 3.16a reveal that when the southwest flange boundary is under compression, although the compressive strains measured at the base of the flange boundary are somewhat overestimated by the model at the high displacement ductility level ($\mu = 6$), model predictions for the compressive strains well-match the measured strains at low ($\mu = 1$) and intermediate ($\mu = 3$) displacement levels, along entire wall height. On the other hand, tensile strain magnitudes and distribution along the wall height at the southwest flange boundary (Figure 3.16b) are captured reasonably well. Moreover, model estimations for the tensile strains at the base and along the height of the northeast web-flange intersection of the wall are also fairly accurate (Figure 3.16c). The comparisons presented in Figure 3.16b and Figure 3.16c also reflects that the plastic hinge lengths predicted by the analysis model (750mm in Figure 3.16b and 450mm in Figure 3.16c) agrees with the experimentally-measured lengths (450mm in Figure 3.16b and 450mm in Figure 3.16c) with a reasonable accuracy, slightly overestimating the

distribution of plasticity at position D. the model predicts spread of plasticity along the wall height reasonably well for specimen TUB. As depicted in Figure 3.16d, when the northwest web-flange intersection is under flexural compression, although the distributions of both the predicted and the measured vertical strains along the height of the wall are unconventional (mostly due to small strain magnitudes), the compressive strains at the base of the wall are well-predicted by the model at all displacement ductility levels. Note that, experimental measurements of tensile strains at this location when the west flange was under compression is rather unexpected. However, these measurements might be related to residual (plastic) tensile strains from previous loading history due to imperfect crack closing, since the reinforcement at this location was already yielded at earlier cycles when this location was under tension. These results showcase the effectiveness of the LFSAFE model in estimating the magnitudes and distribution of compressive and tensile strains developing along the height of U-shaped walls subjected to bi-directional loading, both at flange boundaries and at web-flange intersections.

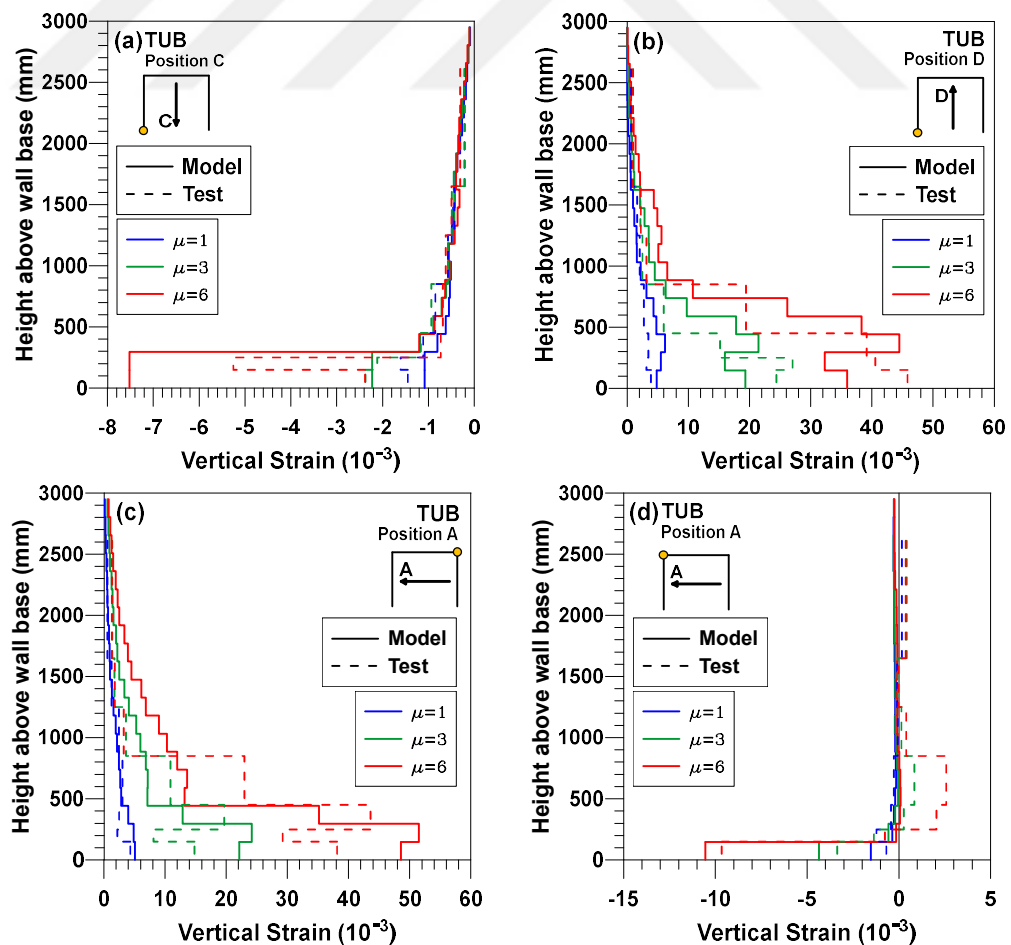


Figure 3.16. Vertical strain profiles along wall height for specimen TUB.

3.8. Model Validation of a U-Shaped Wall under Diagonal Lateral Loading

Accuracy of the LFSAFE model was further verified by comparing model predictions with experimental measurements for wall specimen TUC (Constantin and Beyer 2016), which was tested under lateral loading along two diagonal directions. Although specimens TUB and TUC had identical dimensions, reinforcement configurations in the east flange and the web differentiated (see Table 3.1). As well, the bi-directional loading protocols applied on the two specimens were entirely different. Figure 3.17 compares model predictions for the lateral load – top displacement response of the wall with test results only for the diagonal loading cycles, since the N-S and E-W loading cycles were applied to small top displacement levels, and the wall specimen was observed to show nonlinear behavior mainly during the diagonal loading cycles. As shown in Figure 3.17a, during the diagonal loading cycles in the northwest-southeast direction, the model slightly overestimates the lateral loads capacity and the unloading stiffness characteristics of the E-W component of the wall response, whereas the experimentally-measured lateral load degradation at approximately negative 1.6% E-W drift, is successfully captured. For the same diagonal loading cycles, the model underestimates the N-S component of lateral loads in the positive direction (Figure 3.17b). However, in the negative direction, the model accurately simulates the lateral load capacity, drift capacity, and lateral load degradation characteristics of the N-S component of the response (Figure 3.17b). As depicted in Figure 3.17c, for the northwest-southeast loading cycles, model results obtained for the resultant load–displacement response of the wall, using the SRSS resultant of the N-S and E-W lateral load and displacement components, are again well-representative of the test results, in terms of the lateral load capacity of the wall in both directions, hysteretic response characteristics of the wall, and the drift capacity of the wall (approximately 2.5% SRSS drift) in the negative direction. During the diagonal loading cycles in the northeast-southwest direction, similarly to the previous case, the N-S components of lateral loads in the positive direction (when part of the wall web is under flexural compression) are moderately underestimated by the model (Figure 3.17d). However, in the negative direction (when the wall web is under flexural tension), the lateral load capacity, drift capacity, and lateral load degradation characteristics of the N-S component of the response (Figure 3.17d) are well-predicted. Again, the model provides a reasonably accurate response simulation for the resultant load–displacement response of the wall (Figure 3.17e), including the lateral load capacity of the wall in both directions (although

slight underestimation is observed in the positive direction), hysteretic characteristics of the response, strength loss during the last loading cycle in the negative direction, as well as the drift capacity of the wall in the negative direction (approximately 2.4% SRSS drift).

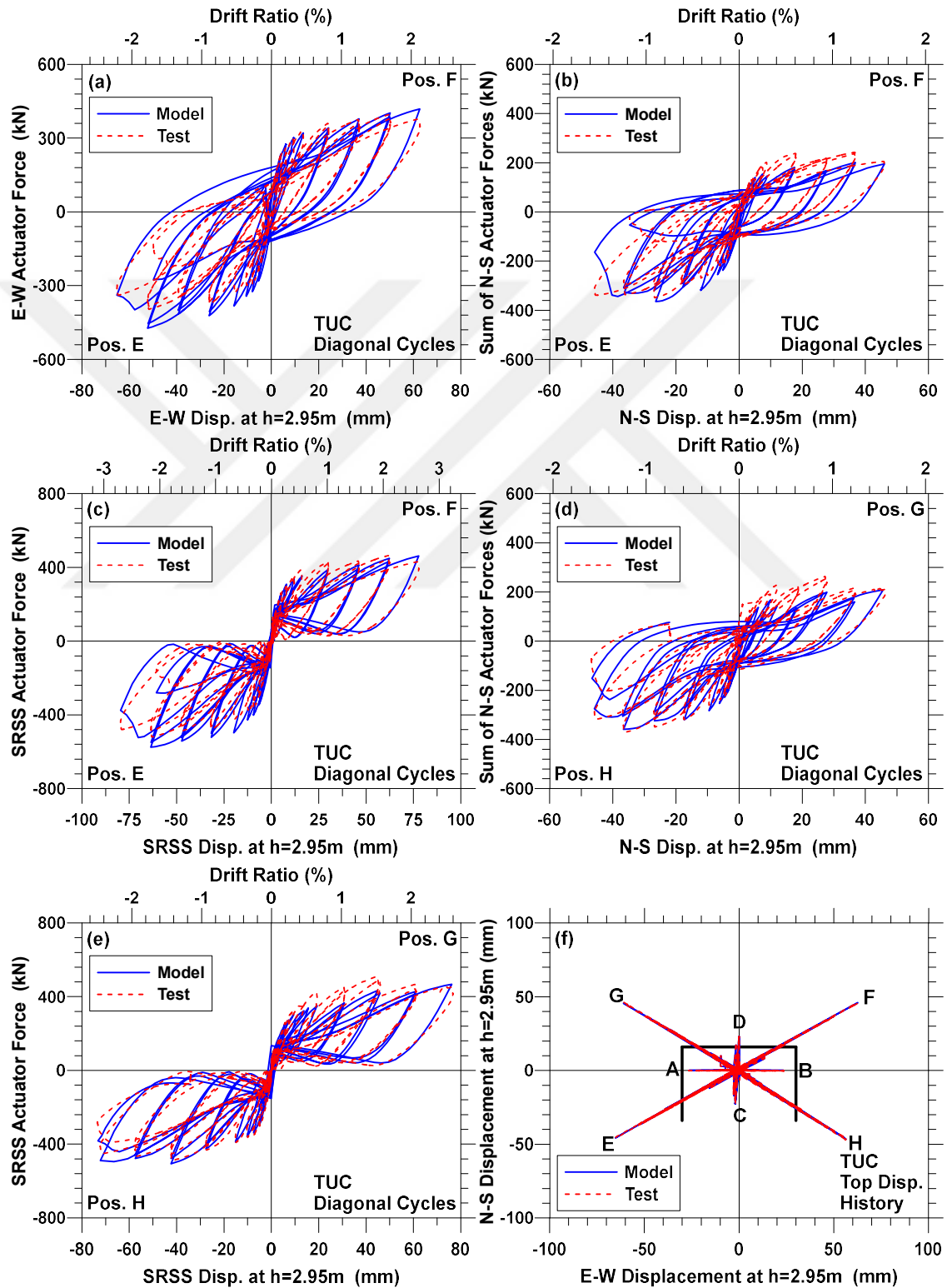


Figure 3.17. Lateral load vs. top displacement responses for specimen TUC.

In summary, other than slight underestimation of the lateral load capacity of the wall when the wall web is under flexural compression (which is subject to improvement upon better definition/calibration of tensile strain hardening behavior in the constitutive model adopted for reinforcing steel), the LFSAFE model can be deemed successful in replicating the overall lateral load–displacement response attributes of U-shaped walls under diagonal lateral loading. Furthermore, the experimentally-observed failure mode of Specimen TUC was out-of-plane buckling of the flange boundary of the wall accompanied by concrete crushing (Figure 3.5d). In the analytically-predicted response, out-of-plane buckling of the flange boundary region is not captured since geometric nonlinearity at the model element level is not yet considered in the model element formulation. However, degradation of lateral load in the model results stems from crushing of concrete and buckling of longitudinal reinforcement in the flange boundary regions of the wall, which is somewhat compatible with the experimentally-observed failure mode, since it is also associated with compression failure at flange boundaries.

Figure 3.18 compares the distribution of longitudinal strains determined from optical sensor measurements on the outer surface of wall specimen TUC, across the wall cross-section at the base of the wall, with model predictions. Strain distributions presented in the figure correspond to loading positions C, F, and H, and are measured at low and intermediate SRSS drift ratios, as test results for larger drift ratios were not available. As can be observed in Figure 3.18a, at loading position C, when the web of the wall is under flexural tension under uni-directional lateral load, tensile strains in the central regions of the web are moderately overestimated, although the compressive and tensile strains along the flanges and at the tensile strains at the web-flange intersection regions are well-captured. At loading positions F (Figure 3.18b) and H (Figure 3.18c), under diagonal loading, when either part of the web is under flexural compression or when the web is under flexural tension, the model well-replicates the highly-nonlinear distribution of the longitudinal strains (i.e., plane sections not remaining plane) across the wall cross section, and well-predicts the magnitudes of tensile strains developing across the entire cross-section as well as the magnitudes of compressive strains developing in flange boundary regions. In this case, the model tends to significantly underestimate compressive strains at the web-flange intersection regions (Figure 3.18b). However, this is not believed to be a characteristic shortcoming of the model

(e.g., see Figure 3.9c and Figure 3.16d, where compressive strains at wall web-flange intersections are accurately predicted at even larger drift or displacement ductility levels).

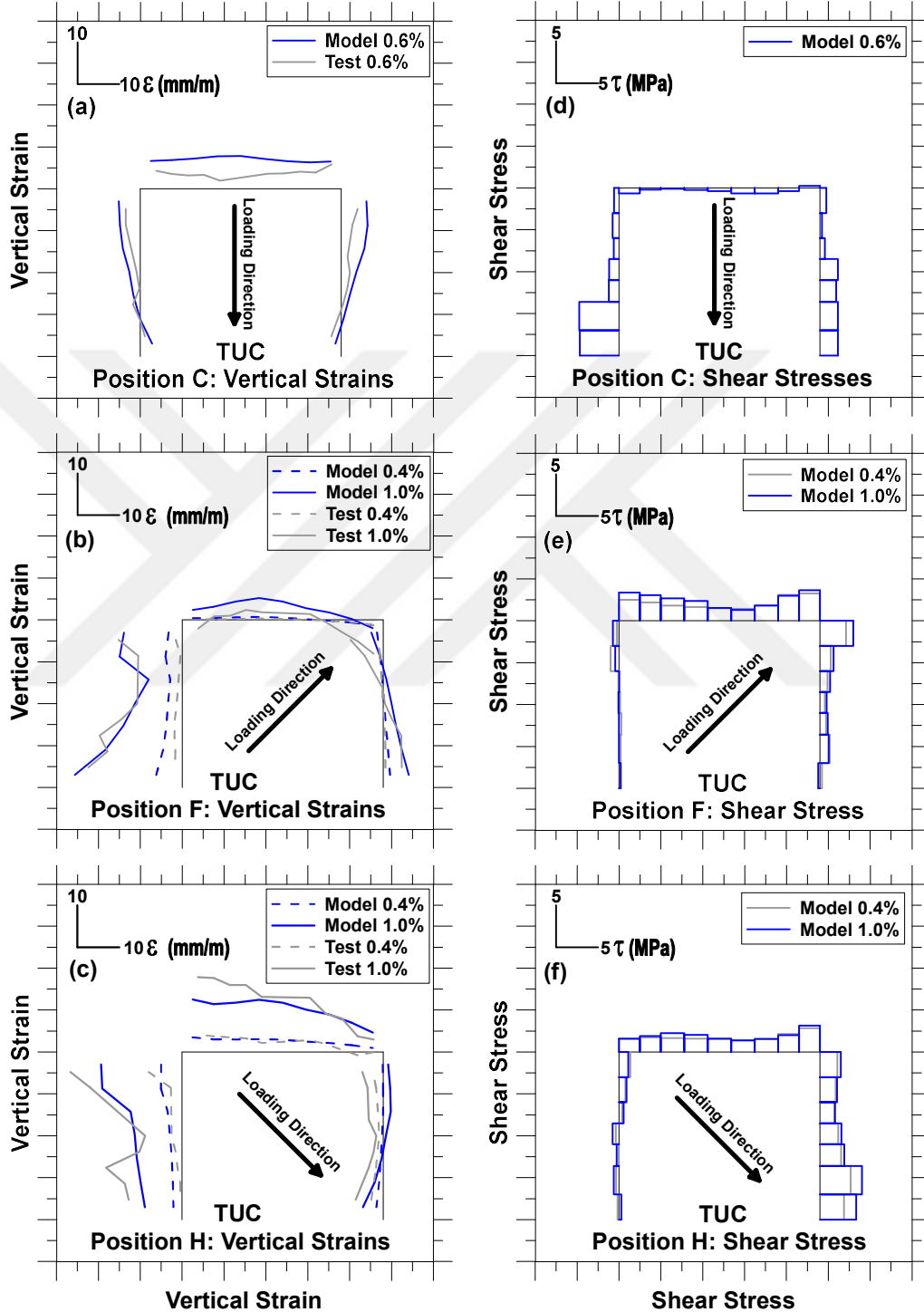


Figure 3.18. Vertical strain and shear stress distributions across the wall cross-section at the base of specimen TUC; (a) strains at position C, (b) strains at position F, (c) strains at position H, (d) shear stresses at position C, (e) shear stresses at position F, and (f) shear stresses at position H.

The analytically-predicted average shear stress distributions also shown in Figure 3.18 (average of the shear stresses developing in the layers of each model element), clearly demonstrate the migration of shear stresses across the wall cross section, depending on the direction of loading and the resulting distribution of longitudinal strains across the cross-section. Especially in the flanges of the wall specimen, which collectively resist the shear force on the wall cross-section in the N-S direction, the flange subjected to flexural compression is shown to resist the majority of the resultant shear force (Figure 3.18b and Figure 3.18c). As well, shear stresses developing along a particular flange migrate towards the region subjected to vertical compressive strains (and therefore axial compressive stresses), since axial and shear stresses are inherently coupled at the model element level. These analytical results demonstrate the capabilities of the LFSAFE model in representing the migration of shear stresses on a wall cross-section towards better-predicting the distribution of the internal shear forces developing in the web and flanges of non-planar walls under multi-directional loading.

3.9. Model Validation of H-Shaped Walls under Combined Lateral/Torsional Loading

The final experimental campaign that was used in this study to validate the LFSAFE model is the wall test program conducted by Maruta et al. (1999, 2000a, 2000b) on H-shaped wall specimens subjected to combined lateral and torsional loading conditions. For these tests, model response simulations were compared with experimental data only at global lateral load (overturning moment) vs. top displacement and torque vs. top twist rotation response levels, as detailed information on local deformation measurements was not available. Test results presented in the publications were digitized by the author. During the tests, various base-overturning-moment-to-torque ratios were enforced on the wall specimens. During the analyses, the same overturning-moment-to-torque ratios were enforced on the wall models (proportional loading was applied using reference force vectors). In the experiments, neither top lateral displacement at the centroid of the wall cross-section nor the top twist angle was directly controlled. Instead, resulting total drifts at only one edge of the loading stub under combined lateral and rotational displacements was controlled. Therefore, in order to interpret effects of torsional demands on lateral response of the specimens, displacement-controlled analyses were conducted, under top displacement histories that match the experimentally-measured top displacements corresponding to the

geometric centroid of the wall cross-section, instead of the predefined drift levels for edge of the stub (i.e. 1/800, 1/400, 1/200, 1/100, 1/50, 1/25). Consequently, load reversal top twist rotations in analyses were not controlled, inherently estimated by the analysis model in simulations. In order to obtain the experimentally-measured top twist rotations of the walls, test measurements for the rotational displacements (i.e., relative lateral displacements) at the top of the walls were post-processed. It must be noted that for load application purposes, each of these particular wall specimens incorporated a solid RC block at the top (Error! Reference source not found.), which would significantly restrain warping of wall cross-section under the applied torque.

Overturning moment vs. top lateral displacement and torque vs. top twist rotation response predictions of the LFSAFE model are compared in Figure 3.19 with the experimentally-measured responses, for six representative H-shaped wall specimens subjected to different loading conditions. In specimen nomenclature, W corresponds to lateral loading in the weak direction of the wall cross-section (Figure 3.1d), S the strong direction, and D the diagonal (45-degree) direction. The number in specimen nomenclature indicates the percent ratio of the torque divided by the sum of the torque and overturning moment (maximum bending moment) values imposed on each specimen. When the measured and predicted overturning moment-top displacement responses of specimens HW25, HW50 and HW75 are compared (Figure 3.19a, c, e), it can be observed that the overturning (bending) moment capacities of specimens HW50 and HW75 are both well-estimated (Figure 3.19c, e). However, for specimen HW25 (Figure 3.19a), since the model predicts degradation in load during the first loading cycle to positive 1.5% drift (whereas degradation in test results occurs during the second loading cycle to the same drift level) the model underestimates the bending moment capacity of this specimen by approximately 15% in the positive loading direction. Nevertheless, the model reasonably captures the lateral load degradation behavior and drift capacity of all three specimens, and well-simulates their hysteretic load–displacement response characteristics. Furthermore, and importantly, both experimental measurements and analytical model results reflect the reduction in wall bending moment capacity under combined lateral and torsional loading, with respect to the pure lateral loading, similarly to the results of analytical studies conducted on U-shaped wall specimens UW1 and UW2 by Hoult and Almeida (2024a). Although cyclic response of the H-shaped wall specimen under pure lateral loading in its weak direction (HW00) is not

presented in Figure 3.19, ultimate moment capacity for this case was analytically estimated as 407kNm where experimental measurement was reported as 455kNm by Maruta et al. (1999), in the positive loading direction. Compared to these moment capacities, analysis results estimate 6%, 46% and 77% (3%, 53% and 80% in the experiment) reductions in wall ultimate bending moment capacities, when base-overturning-moment-to-torque ratios are 3.0, 1.0 and 0.33, for specimens HW25, HW50 and HW75, respectively. Although moment capacity reduction was not significant in both analytical and experimental results for the specimen HW25, response comparison for HW50 indicates nearly a 50% capacity reduction only when base-overturning-moment is equal to the applied torque. Obviously, this is not a generic result for all non-rectangular walls, since interaction between flexural and torsional capacities under different loading conditions is directly correlated with the load application height (shear span) of the wall specimens as scrutinized by Hoult and Almeida (2024a). Nevertheless, response comparisons presented in Figure 3.19a, c, e clearly demonstrate that the LFSAFE model is capable of simulating the experimentally-observed reduction in wall bending moment capacity with increasing torque applied on the wall. Comparison of model predictions with test measurements for the corresponding torque vs. top twist rotation response of these three specimens (HW25, HW50 and HW75) are presented in Figure 3.19b, d, f. The model slightly underestimates the torque capacity of specimen HW25 in the positive loading direction, due to the aforementioned earlier strength loss prediction of the model. It should be noted that since the applied torques and overturning moments are proportional, degradation in bending moment and torque occur simultaneously in the test results and model predictions. The torque capacities of specimens HW50 and HW75 are moderately overestimated in the negative loading direction. The model successfully predicts the twist rotation values at the peaks (top displacement reversal points) of the loading cycles, except for specimen HW25, where the twist rotation at the positive peak of the last loading cycle is underestimated. Model estimations for the twist rotation capacity of the walls (at initiation of torque degradation) are again, favorable. Hysteretic characteristics of the measured torque–twist rotation responses are generally well-simulated; however residual torque capacities after strength loss are somewhat overestimated. Results presented in Figure 3.19b, d, f also highlight the reduction in wall torque capacity with increasing bending moment demand on the wall, demonstrating the capabilities of the LFSAFE model in simulating the experimentally-observed nonlinear torque-flexure interaction.

For wall specimens HS25 and HS50, for which the lateral loads were applied in the strong direction of the wall cross-section, the LFSAFE model provides more accurate predictions of the experimentally-measured overturning moment-top displacement and torque-top twist rotation responses. As shown in Figure 3.19g, i, the model accurately simulates the bending moment capacities and hysteretic load-displacement response characteristics of both specimens, except for the slight yet sudden lateral load (bending moment) degradation observed in the analysis results for specimen HS50 (Figure 3.19i). Model predictions for the torque–twist rotation response of these walls are also favorable in terms of torque capacity and hysteretic behavior (Figure 3.19h, j), with the exception that the model significantly overestimates the twist rotation at the peak (top displacement reversal point) of the last loading cycle in positive rotation (Figure 3.19j). This overestimation is associated with the slight yet sudden degradation that was observed only in the analytical bending moment-top displacement response of the wall (Figure 3.19i), and is believed to be incidental.

Wall specimen HD25 was unique, in the sense that it is was the only specimen that was tested under bi-directional lateral loading combined with torsional loading. For this specimen, although the hysteretic response characteristics prior to strength loss are well-simulated similarly to specimen HW25, the model slightly underestimates the experimentally-measured bending moment and torque capacities in the positive loading direction (Figure 3.19k, l), since model predicts an earlier degradation in load (which reflects to both bending moment and torque) compared to the measured response. Therefore, the model underpredicts the drift and twist rotation capacities of this specimen, although in the positive loading direction only. This discrepancy may also be incidental, because unlike other wall specimens, the experimentally-measured moment–displacement responses of this particular specimen are asymmetrical, in terms of measured moment and torque values measured in the two opposite loading directions. Furthermore, for all H-shaped wall specimens, the RC loading block located at the top of the specimens is idealized in the analysis models using a fully-rigid constraint defined at the top of the wall models, and possible discrepancies between these two boundary conditions (e.g., no information is available related to possible cracking of the loading block) may have impaired the correlation between model results and test measurements.

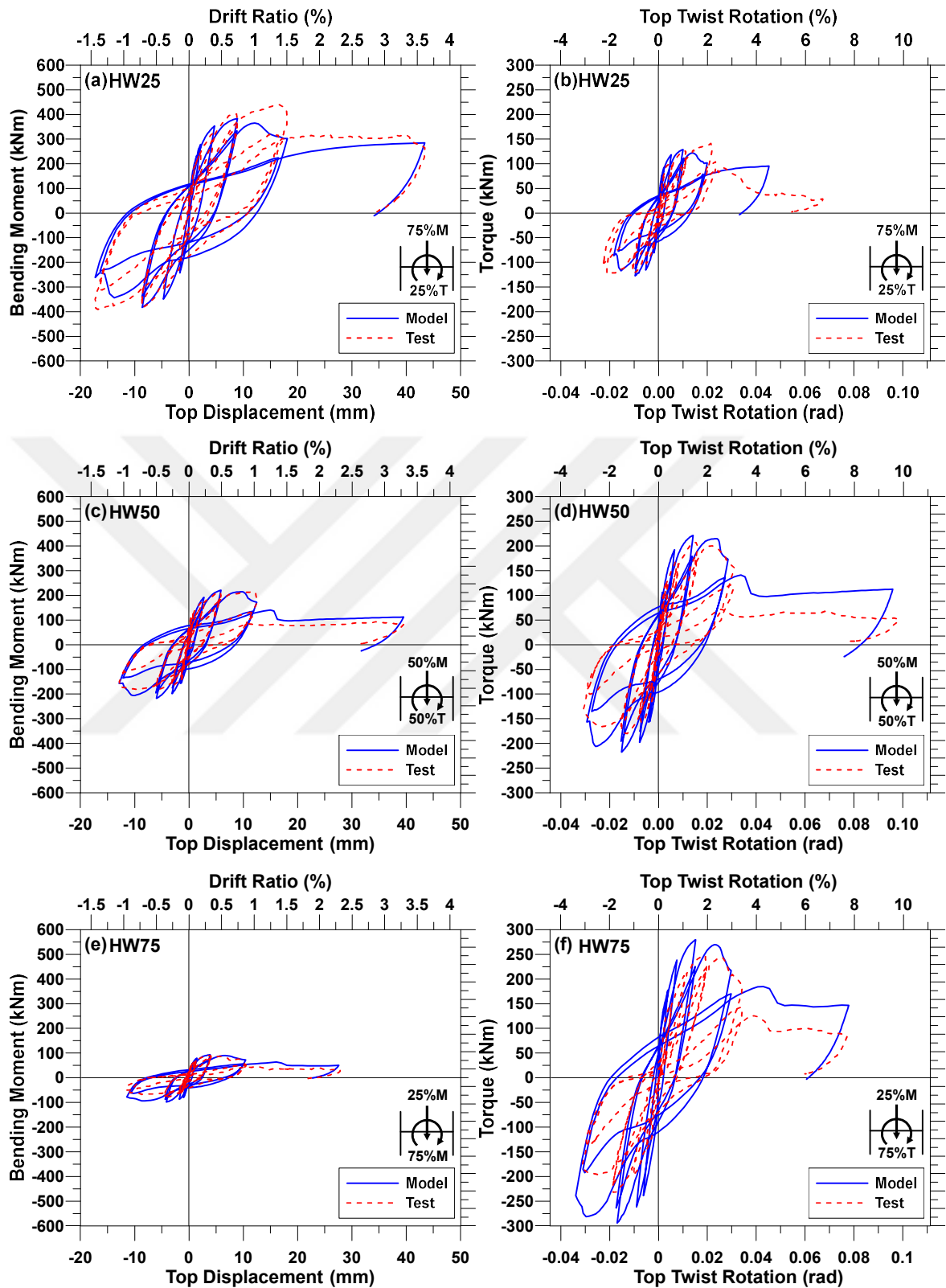


Figure 3.19. Base overturning moment vs. top displacement and torque vs. top twist rotation responses for the H-shaped wall specimens.

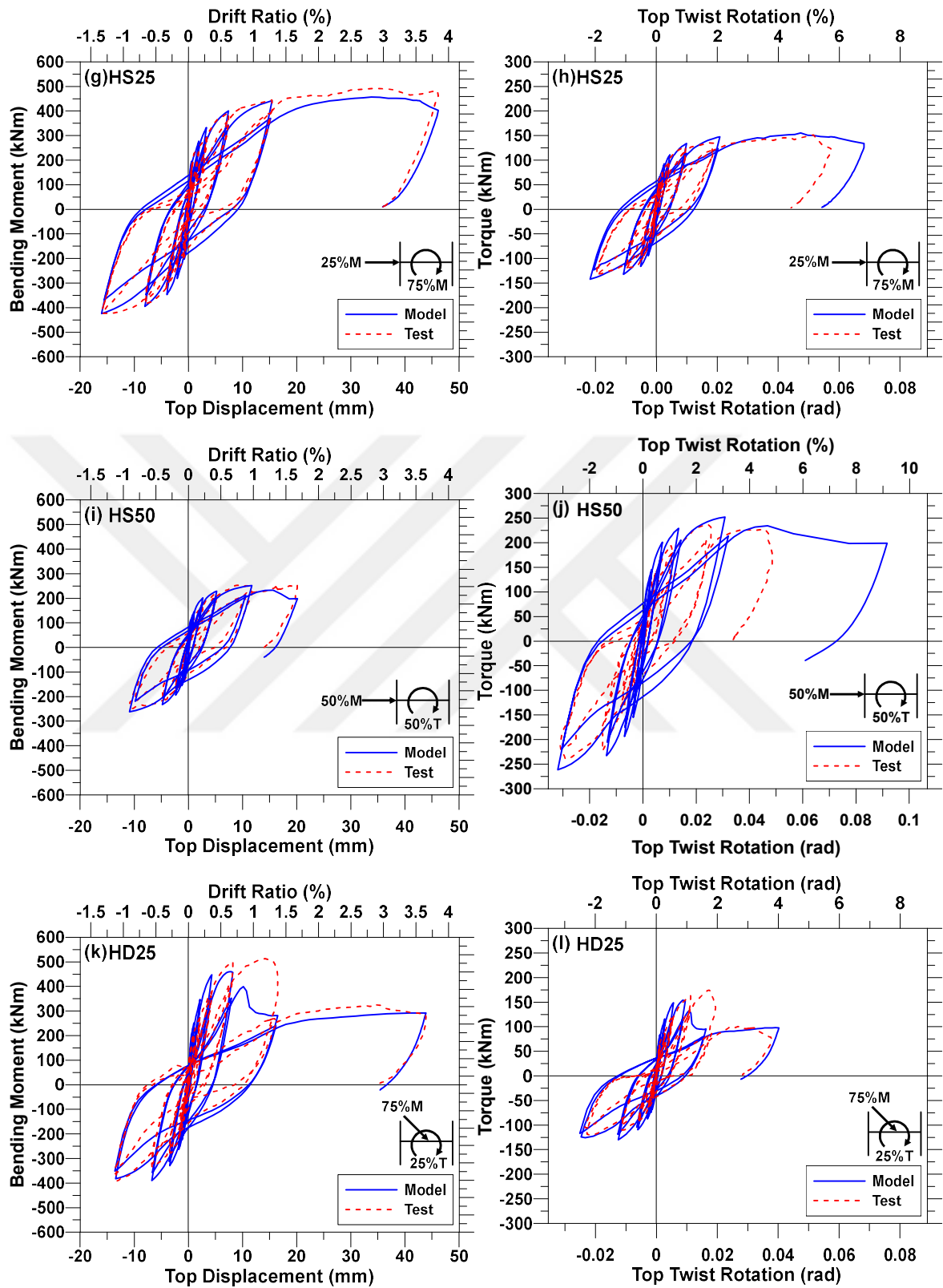


Figure 3.19. Base overturning moment vs. top displacement and torque vs. top twist rotation responses for the H-shaped wall specimens (cont.).

In terms of the experimentally-observed behavior and failure modes, all specimens were reported to exhibit flexural/torsional yielding of longitudinal reinforcement, and the reported failure mode (source of load degradation) of all specimens was crushing of concrete at wall flange boundaries under combined flexural and torsional (diagonal) compression, except specimens HS25 and HS50, for which concrete crushing was not reported. Analysis results obtained for the walls also show yielding of longitudinal reinforcement, and degradation in load is associated with crushing of concrete and buckling of longitudinal reinforcement in flange boundary regions, which agrees with test observations. Overall, the response correlations presented demonstrate the effectiveness of the LFSAFE model in simulating the coupled nonlinear flexural and torsional responses of H-shaped walls subjected to combined lateral/torsional loading.

3.10. Influence of Strain Penetration Effects on Quasi-Static Response Predictions

Even though local response estimations of analysis models presented in previous sections for specimens UW1 and UW2 (Figure 3.9, Figure 3.12 and Figure 3.13) demonstrate that LFSAFE model is generally capable of predicting reasonable strain distributions, tensile strain magnitudes are typically underestimated (except in Figure 3.12c) in strain profiles along the gauge lengths of base LVDT's in average sense, for both UW1 (Figure 3.9b, d) and UW2 (Figure 3.12d). Additionally, tensile strains measured by the LVDT's installed across the wall base were also significantly higher than analysis predictions, at flanges under torsional tension (Figure 3.13a, b). Since measurements of these LVDT's are also affected by strains penetrating into the foundation, typical underestimation of tensile strains at the presented comparisons believed to be related with negligence of strain penetration effects in analysis. Therefore, in order to scrutinize possible strain prediction improvements, additional model configurations for specimens UW1 and UW2 were generated, where strain penetration effects were also considered adopting an indirect (macroscopic) modeling approach similar to that adopted in Hoult and Almeida (2024b). In this macroscopic modeling approach, extension/slip deformations that develop on the longitudinal reinforcing steel bars within the foundation and accumulate at the wall-foundation interface (i.e., strain/yield penetration effects) are represented in the form of average tensile strains that develop within an additional row of LFSAFE model elements, which are defined directly below the wall-foundation interface (Figure 3.20). The model nodes at the wall-foundation

interface are restrained from horizontal displacements only, and the model elements representing the foundation (i.e., the depth of the foundation along which constant longitudinal tensile strains can be assumed) are assigned zero concrete tensile strength (for allowing tensile strains to freely develop along this depth) and approximately linear elastic stress-strain behavior for concrete in compression (for representing the relatively rigid behavior of the foundation under compression). Height of this additional element layer (i.e. strain penetration height), along which developing strains are represented in an average sense, is defined as 220mm, as recommended by Hoult et al. (2022) based on strain gauge measurements on the reinforcing steel bars within the foundation of these two specimens (UW1 and UW2) during the quasi-static tests (Hoult et al., 2023a). Furthermore, in order to represent the wall-foundation interface cracks, horizontal crack directions were enforced to these additional elements at the beginning of the analysis.

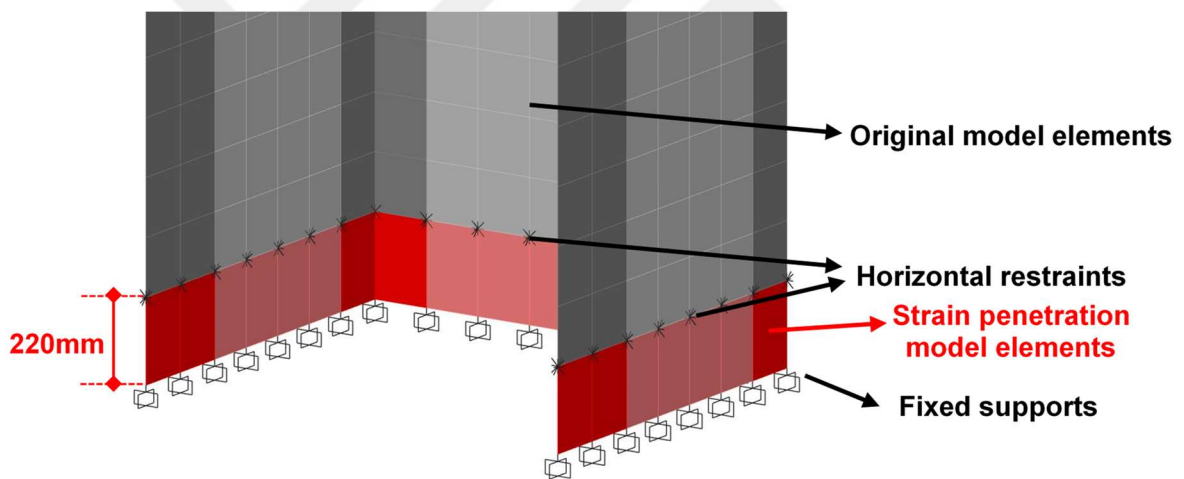


Figure 3.20. Macroscopic modeling approach for consideration of strain penetration effects.

Before evaluation of local response predictions, it is important to note that, consideration of strain penetration effects in analysis has very limited effect on estimations for lateral load vs. top displacement response of specimen UW1 (Figure 3.21a), and torque vs. top twist rotation response of specimen UW2 (Figure 3.21b). As presented in Figure 3.21a, strain penetration effects have no effect on lateral load capacity predictions for the wall in both loading directions, as well as the drift capacity in negative loading direction. On the other hand, in response predictions of the model including strain penetration effects (Model w/ SP in Figure 3.21), stiffness values at early cycles are slightly reduced (as

expected) compared to the analysis model results without strain penetration effects (Model w/o SP in Figure 3.21). Additionally, hysteretic response characteristics of the wall (i.e., unloading stiffness values, cyclic stiffness degradation, pinching behavior) are also altered in the analysis predictions with strain penetration effects, to a limited extent. Moreover, the model still captures the experimentally-observed strength degradation (due to concrete crushing and reinforcement buckling at southwest boundary region) at negative 2.0% drift. Furthermore, torque vs. top twist rotation response comparison presented in Figure 3.21b for specimen UW2 show that, consideration of strain penetration effects in analysis slightly delays the strength degradation predicted by the model without strain penetration effects, estimating a larger twist rotation capacity. Accordingly, torque capacities (especially the capacity in negative twist rotation direction) predicted by the model with strain penetration effects is somewhat larger than the model predictions obtained from the model configuration where strain penetration is neglected. Other than this, similarly to the response comparison presented in Figure 3.21a for UW1, in Figure 3.21b, noticeable stiffness reduction is observed at early twist cycles of specimen UW2, and hysteretic response characteristics estimated by the two model configurations are altered, to some extent.

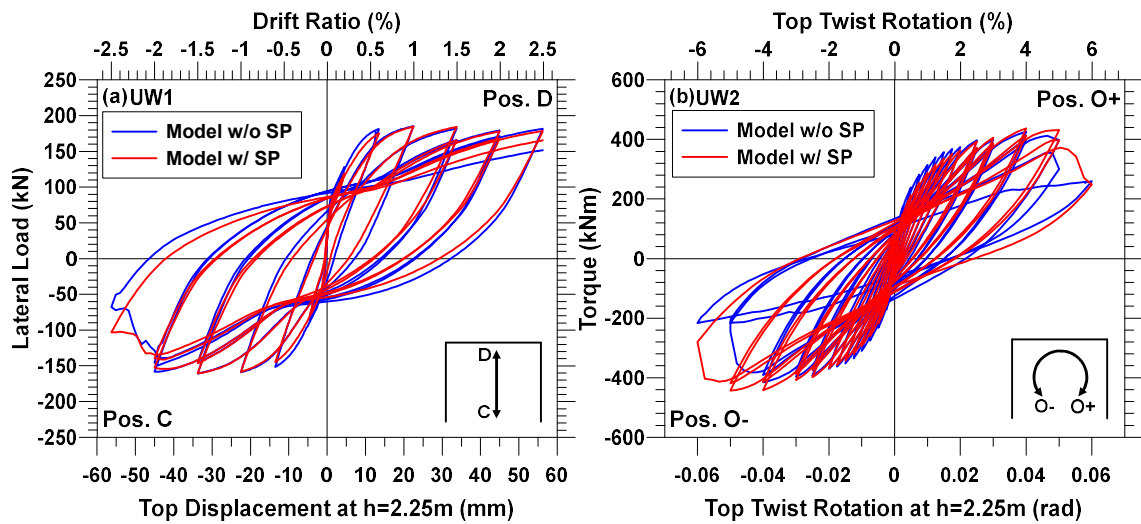


Figure 3.21. Influence of strain penetration effects on global response predictions: (a) Lateral load vs. top displacement response of specimen UW1, (b) Torque vs. top twist rotation response of specimen UW2.

In order to scrutinize effects of strain penetration on tensile strain predictions, vertical strain profile estimations from additionally generated analysis models (Model w/ SP) are

compared with the predictions from the original model configuration (Model w/o SP) in Figure 3.22 and Figure 3.23 for specimens UW1 and UW2, respectively. Note that in Figure 3.22 and Figure 3.23, tensile strain predictions from both the original models (the results in Figure 3.9b, d and Figure 3.12c, d) and the additional models along the gauge length of the bottommost LVDT's are plotted as average of the strain values estimated. As shown in Figure 3.22a, vertical tensile strain profile estimations for southeast flange tip of specimen UW1 slightly improved relative to the estimations from the original model, except at 0.6% drift level, when strain penetration effects are included in the analysis model. On the other hand, improvement in strain estimations of northwest web-flange intersection only observed at 2.0% drift level (Figure 3.22b).

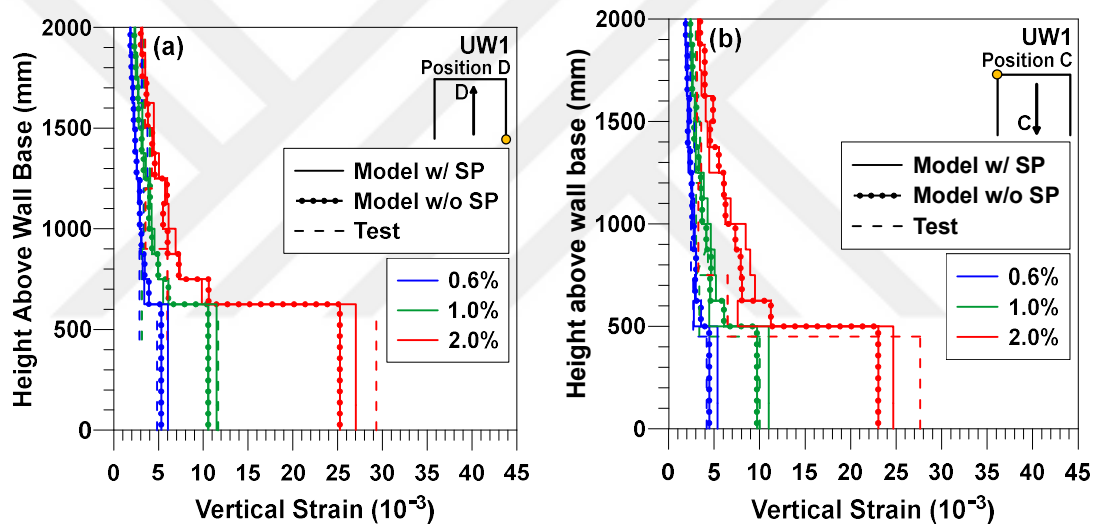


Figure 3.22. Influence of strain penetration effects on vertical tensile strain profile predictions along wall height of specimen UW1.

Furthermore, prediction comparisons of two analysis model configurations presented in Figure 3.23a (vertical strains from inner reinforcement layer) and Figure 3.23b (vertical strains from outer reinforcement layer) reveal that tensile strains estimated by the analysis model with strain penetration effects are in better agreement with LVDT measurements (considering average of estimations from inner and outer layers), with slight overestimation at 2.0% rotation, at southwest flange tip of specimen UW2. Moreover, when only Gauss-weighted average strains, neglecting the additional strains on bottommost element layer due to strain penetration effects, are plotted for the same location (southwest flange tip of specimen UW2), it was observed that, strain estimations from the analysis model with strain

penetration effects are in a better agreement with the measurements from the DFOS system that were directly installed on the vertical rebars and were not reflecting the accumulated strains at the wall-foundation interface, as presented in Figure 3.23c and Figure 3.23d, while more pronounced in predictions for outer reinforcement layer.

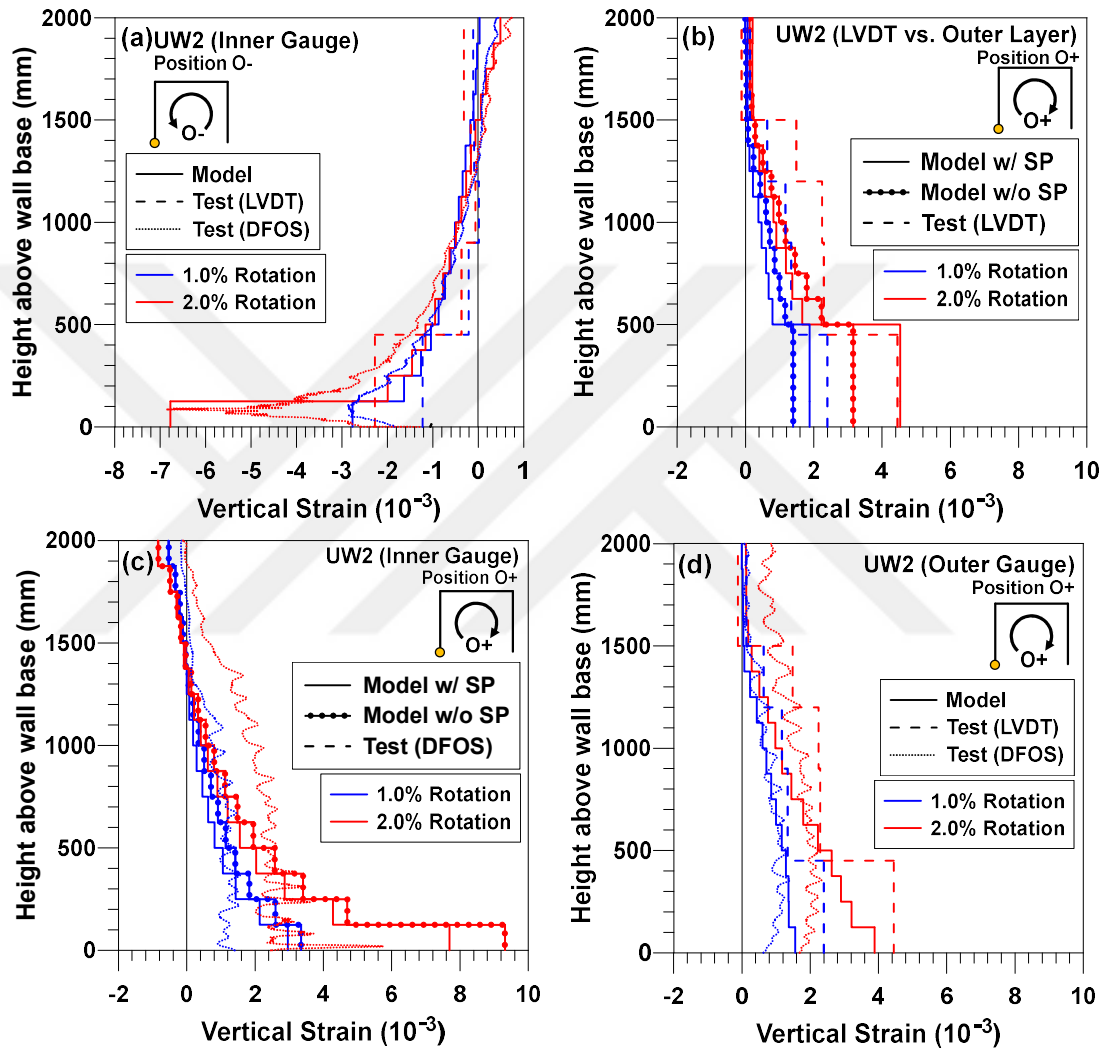


Figure 3.23. Influence of strain penetration effects on vertical tensile strain profile predictions along wall height of specimen UW2.

Additionally, when vertical strain distributions across the wall base of specimen UW2 are examined for the model configuration with strain penetration effects, significant increase in predicted tensile strain magnitudes are noted at flanges under torsional tension, in both positive (Figure 3.24a) and negative (Figure 3.24b) rotation directions, providing more accurate estimations than analysis results of the original model configuration (Figure 3.13a, b) at these locations.

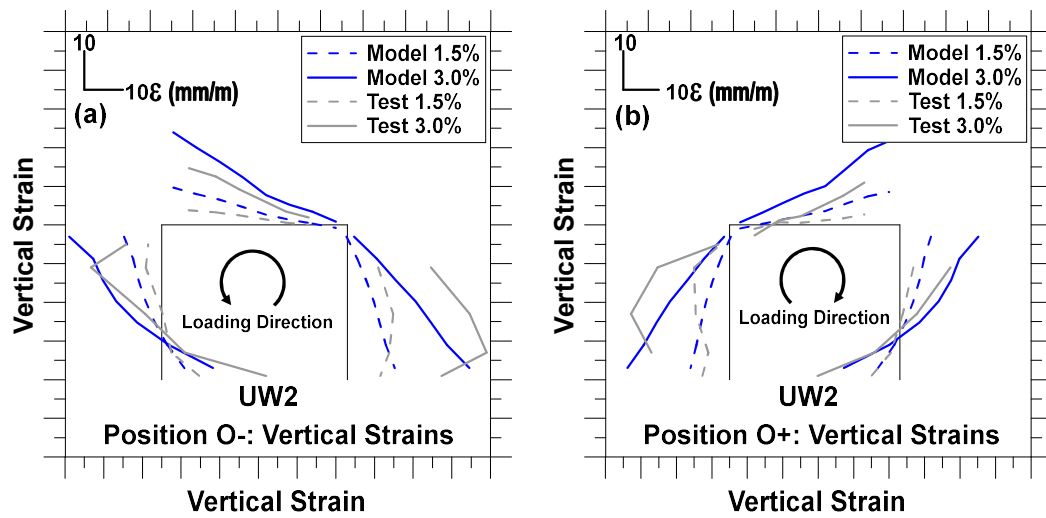


Figure 3.24. Vertical strain distributions across wall base for specimen UW2 (including strain penetration effects).

Overall, consideration of strain penetration effects in analysis, albeit indirectly with a macro modeling approach, clearly enhanced the accuracy of the local response predictions of specimens UW1 and UW2 in terms of tensile strain magnitudes, while affecting the global response predictions to a limited extent. However, scrutinized influence of strain penetration effects on these response predictions are obviously dependent on robustness of the employed modeling approach (which is not correlated with the developed LFSAFE model in this study), and open to improvements with adoption of a more advanced modeling method.

3.11. Mesh Sensitivity Analysis

Selection of geometry discretization in nonlinear analyses is a challenging task, since analysis model predictions can be influenced by the preferred average mesh size significantly. Besides, analysis runtimes are also highly correlated with the adopted discretization, since number of DOFs in models is directly dependent on number of elements used in analysis, and runtimes can drastically increase with unnecessarily fine discretization. Additionally, correct representation of stress/strain distributions through a structural wall may not be achieved in simulations when insufficient number of elements are used adopting a relatively coarse mesh. Therefore, in order to evaluate influence of selected geometry discretization on response predictions of LFSAFE model, mesh sensitivity studies were conducted scrutinizing representative analysis results corresponding to model configurations

with coarse mesh, medium mesh and fine mesh geometry discretization, for U-shaped wall specimen TUC. In model configuration with coarse mesh, model elements at wall boundary regions were typically discretized along wall length such that the length to thickness ratio of the shell elements do not exceed 3.0, resulting a discretization with a single element at wall boundaries of specimen TUC. Medium mesh is the adopted geometry discretization in simulations of this study, with typically two elements at boundary regions (length to thickness ratio of the elements not exceeding 2.0). Lastly, the fine mesh model geometry discretization corresponds to the case where length to thickness ratio of the shell elements not exceeding 1.0, requiring at least three LFSAFE elements at the boundary regions. Resulting model element discretization for specimen TUC with different approaches are presented in Figure 3.25. Representative comparisons provided in Figure 3.26, Figure 3.27 and Figure 3.28 reflect that, the model configuration with coarse mesh (Figure 3.26a, Figure 3.27a and Figure 3.28a) fail in capturing strength degradation observed at the last loading cycles towards positions E (Figure 3.26a and Figure 3.27a) and position H (Figure 3.28a), while the model with medium mesh captures the degradation well (Figure 3.26b, Figure 3.27b and Figure 3.28b). On the other hand, model predictions obtained from analysis model with fine mesh exaggerate the degradation in N-S component of response (Figure 3.27c and Figure 3.28c) suffering from strain localization problems. It is important to note that, adoption of the criterion length to thickness ratio of the elements not exceeding 2.0 is not a generic mesh selection approach, since sensitivity against mesh size can be problem specific, especially when strength degradation is pronounced.

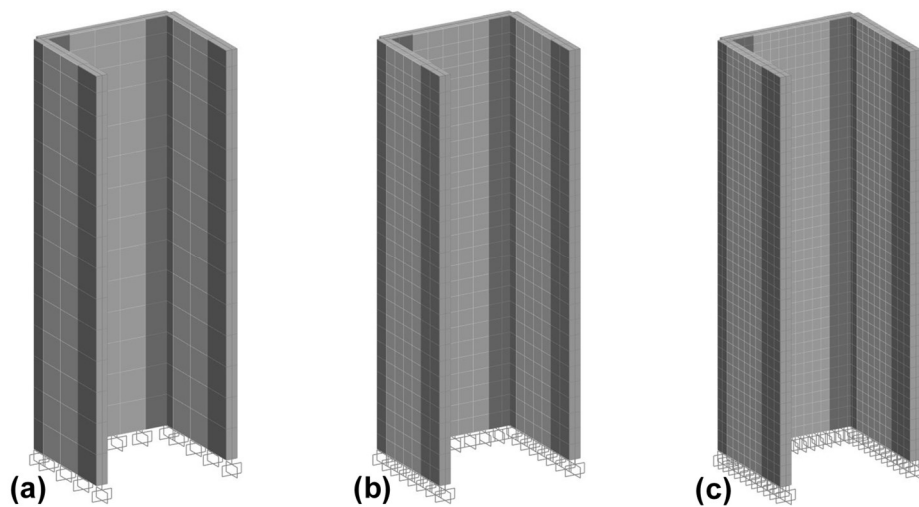


Figure 3.25. Model element discretization of specimen TUC with (a) coarse mesh (b) medium mesh (c) fine mesh.

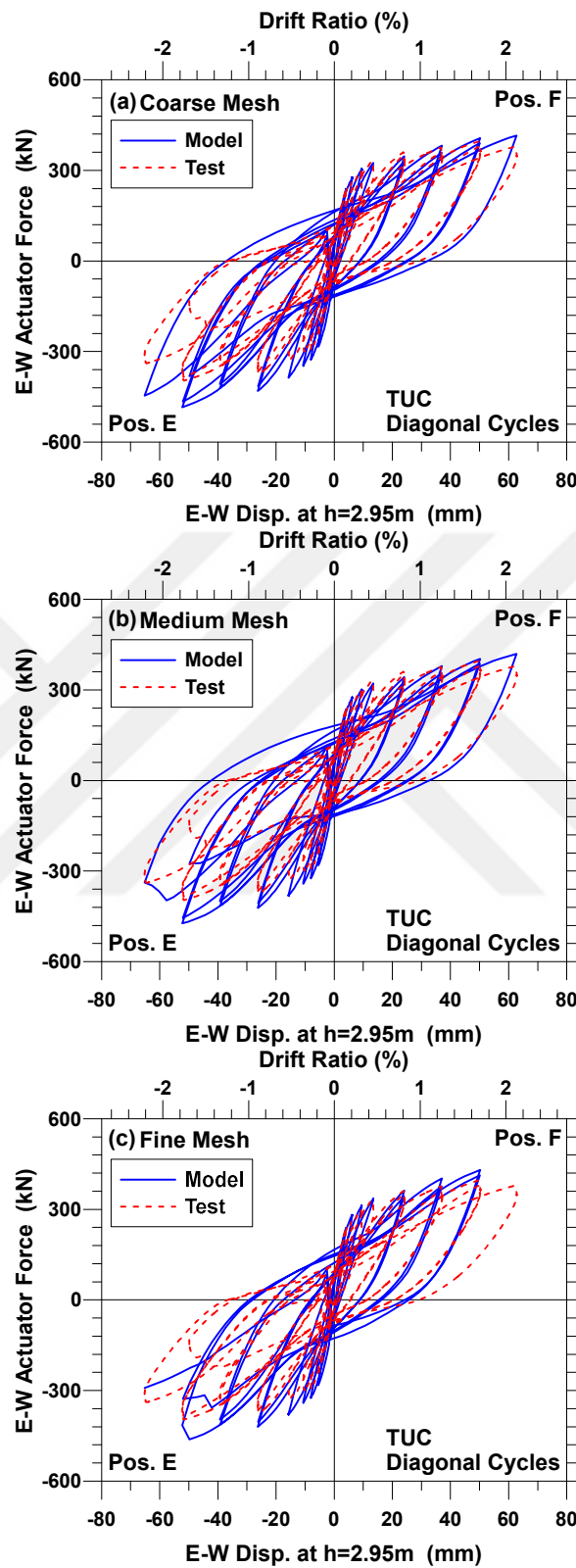


Figure 3.26. Mesh sensitivity of lateral load vs. top displacement response predictions for specimen TUC (E-W component during diagonal cycles in E-F direction): (a), model with coarse mesh, (b) model with medium mesh, (c) model with fine mesh.

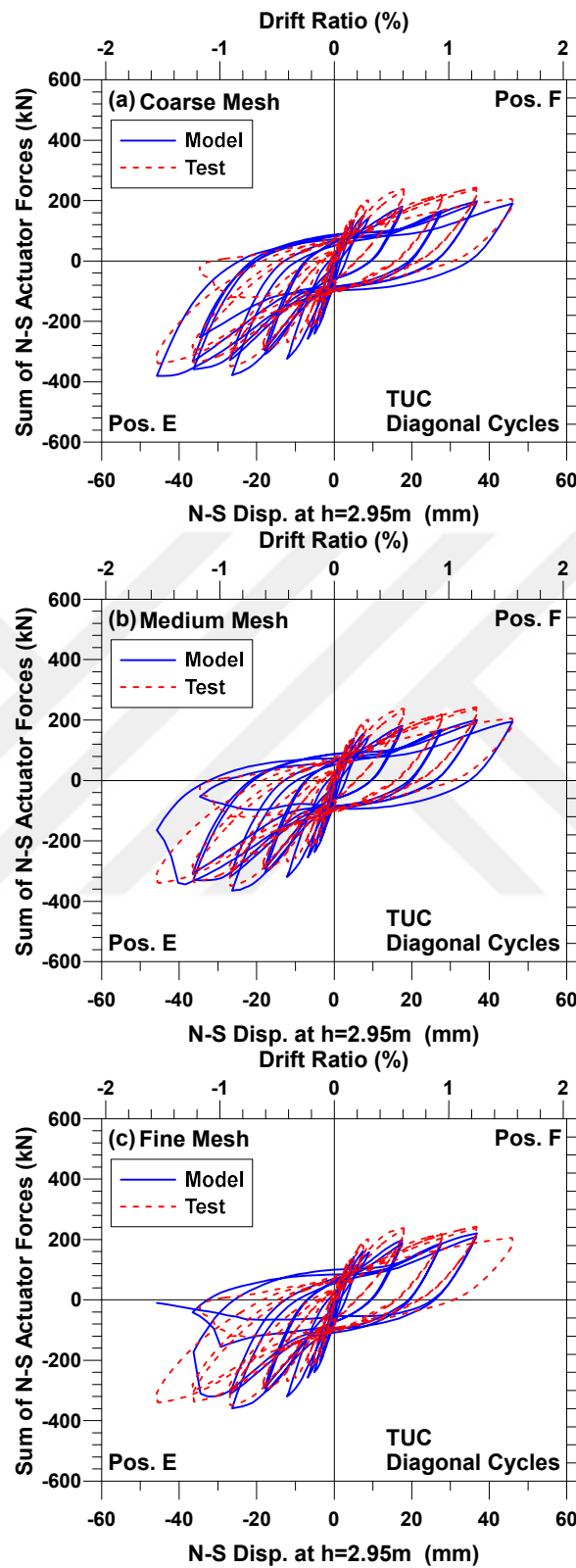


Figure 3.27. Mesh sensitivity of lateral load vs. top displacement response predictions for specimen TUC (N-S component during diagonal cycles in E-F direction): (a), model with coarse mesh, (b) model with medium mesh, (c) model with fine mesh.

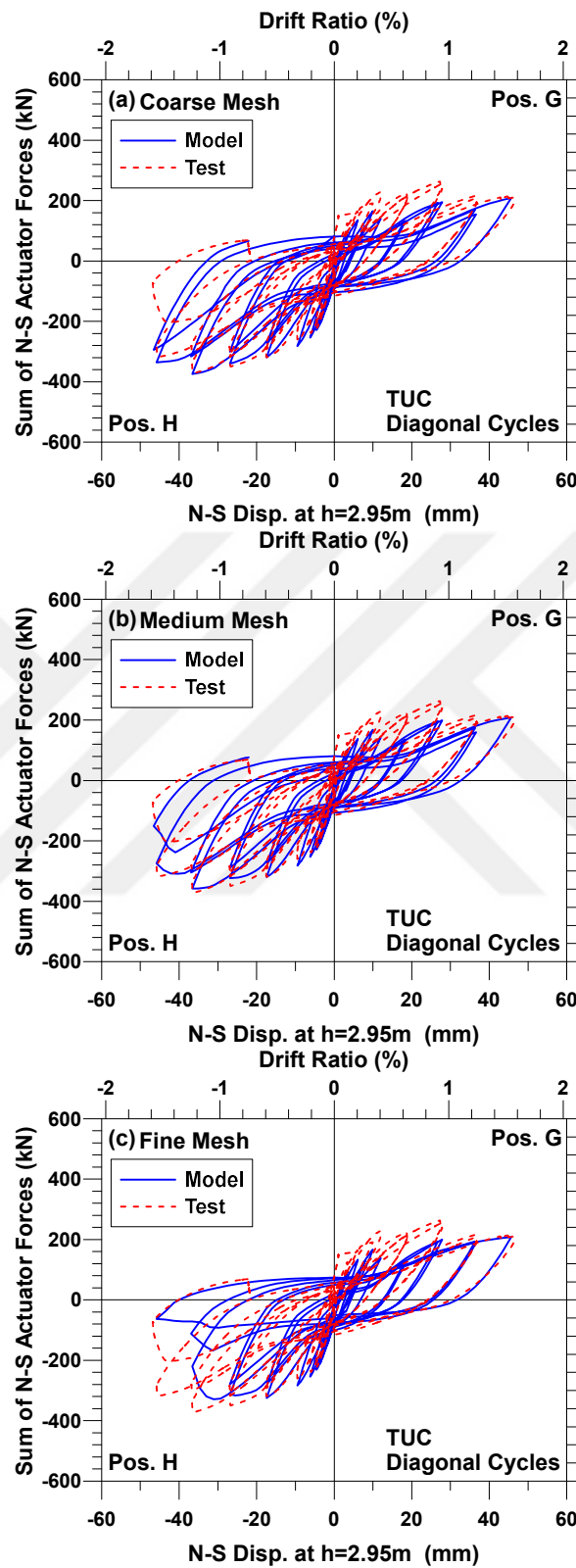


Figure 3.28. Mesh sensitivity of lateral load vs. top displacement response predictions for specimen TUC (N-S component during diagonal cycles in G-H direction): (a), model with coarse mesh, (b) model with medium mesh, (c) model with fine mesh.

3.12. Influence of Reinforcement Buckling on Model Predictions

One of the significant attributes for the LFSAFE model is, its representation capability for reinforcing steel buckling behavior for the longitudinal reinforcement located in wall boundary regions that are typically susceptible to reinforcement buckling. In order to representatively evaluate influence of reinforcement buckling on model response predictions, additional models with a configuration neglecting the reinforcement buckling behavior (in longitudinal reinforcement of boundary regions) were generated for U-shaped wall specimens UW1 and TUC, for which specimens reinforcement buckling was observed (Figure 3.5a and Figure 3.5d) during quasi-static tests, and significant strength degradation was predicted in analytical model response as provided in Figure 3.8a for specimen UW1 and in Figure 3.17a, b, d for specimen TUC. Lateral force-top displacement response estimations obtained from the original model configuration (with buckling in Figure 3.30a and Figure 3.29a, b, c) as well as this additional model configuration (without buckling in Figure 3.30b and Figure 3.29d, e, f) are presented in Figure 3.30 and Figure 3.29, for specimens UW1 and TUC, respectively.

Lateral load-top displacement response predictions presented in Figure 3.30a (with buckling) and Figure 3.30b (without buckling) reflect that, representation of reinforcement buckling for longitudinal bars at boundary regions has a clear impact on model response predictions, since the experimentally-observed degradation in the last loading cycle towards position C is captured by the analysis model when the buckling is considered in analysis (Figure 3.30a), while the model fails in capturing the degradation when reinforcement buckling is neglected (Figure 3.30b). On the other hand, comparison of model response estimations in Figure 3.29a, b, c with analysis results in Figure 3.29d, e, f shows that inclusion of reinforcing steel buckling behavior in analysis model has very limited impact on lateral load-top displacement response predictions for specimen TUC, only slightly affecting the last loading cycle response in directions E (Figure 3.29d, e) and H (Figure 3.29f), where significant strength degradation was observed. Moreover, predictions from the model configuration without reinforcement buckling unexpectedly exaggerate the degradation observed during H-G cycles (Figure 3.29f).

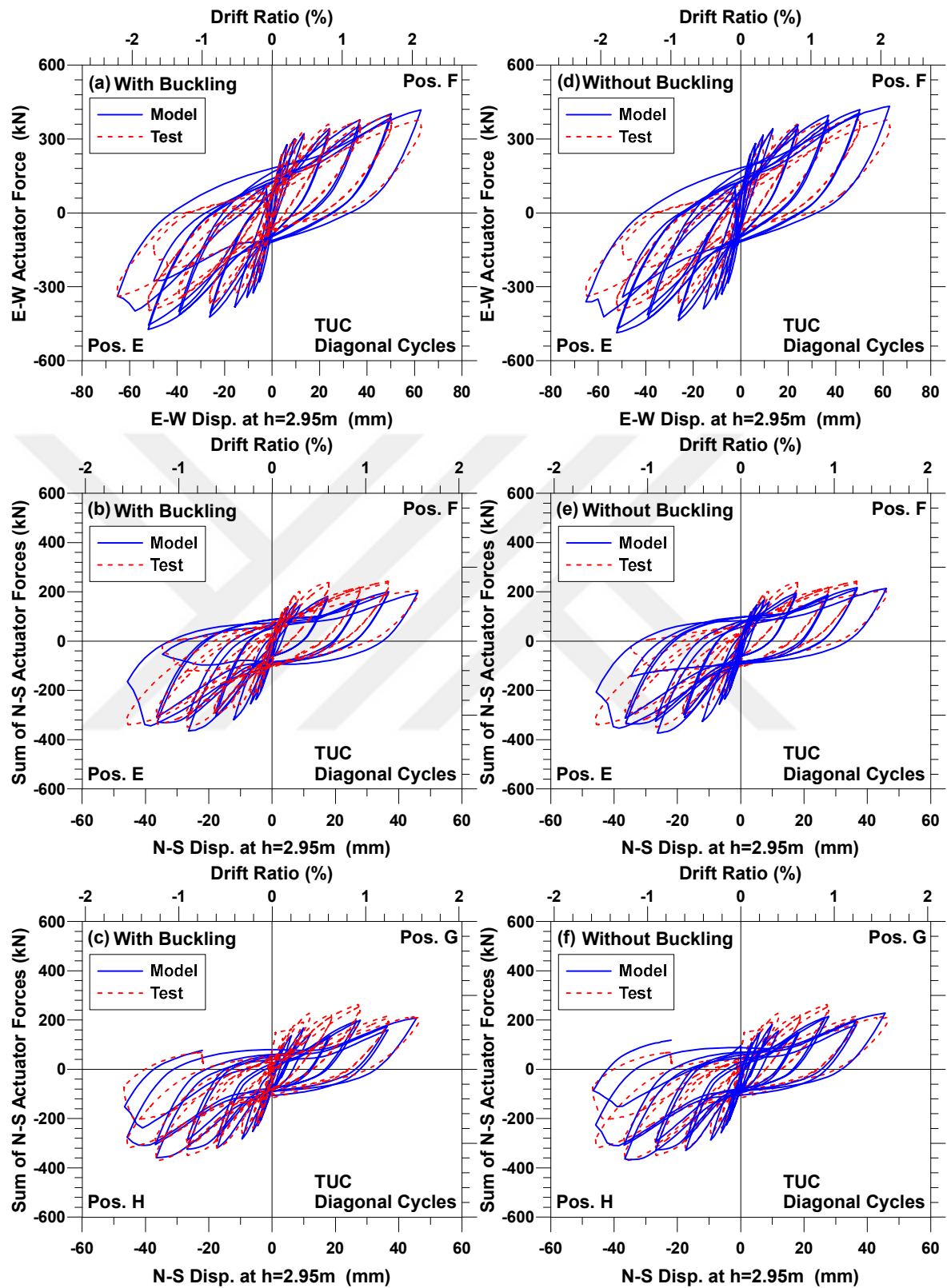


Figure 3.29. Influence of reinforcement buckling on lateral load vs. top displacement response predictions of specimen TUC: (a), (b), (c) with buckling, (d), (e), (f) without buckling.

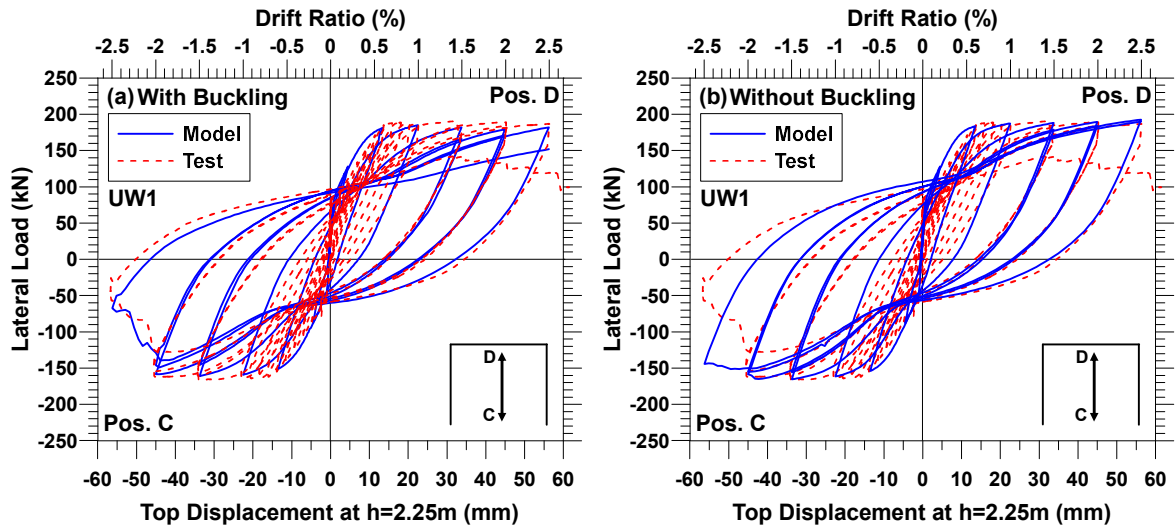


Figure 3.30. Influence of reinforcement buckling on lateral load vs. top displacement response predictions of specimen UW1: (a) with buckling, (b) without buckling.

3.13. Layer Discretization Sensitivity Analysis

Thanks to its layered formulation, detailed cross-section definitions with consideration of different confinement characteristics for concrete along wall thickness at boundary regions as well as representation of precise longitudinal reinforcement locations in model generation is possible with LFSAFE model. To discuss the effects of this attribute of the model on response predictions, sensitivity studies were conducted generating an additional model adopting a single layer model configuration for specimen TUC, and lateral load-top displacement response predictions are provided for this model configuration and the original model configuration, where the simulations were conducted adopting a multi-layered modeling approach, for comparison in Figure 3.31. In single layer modeling approach, since the out-of-plane nonlinear behavior is not represented likewise the multi-layered approach and represented linear elastically, out-of-plane stiffness of the shell elements are modified adopting an effective flexural stiffness of $0.1EI_{out}$. Model predictions provided in Figure 3.31 show that model with single layer configuration (Figure 3.31d, e, f) fail in capturing the experimentally-observed strength degradations in response of specimen TUC, while the multiple-layered model (Figure 3.31a, b, c) successfully capture them. Additionally, lateral load capacities obtained from multiple-layered model are in better agreement with experimental measurements, compared to the results from model with single layer.

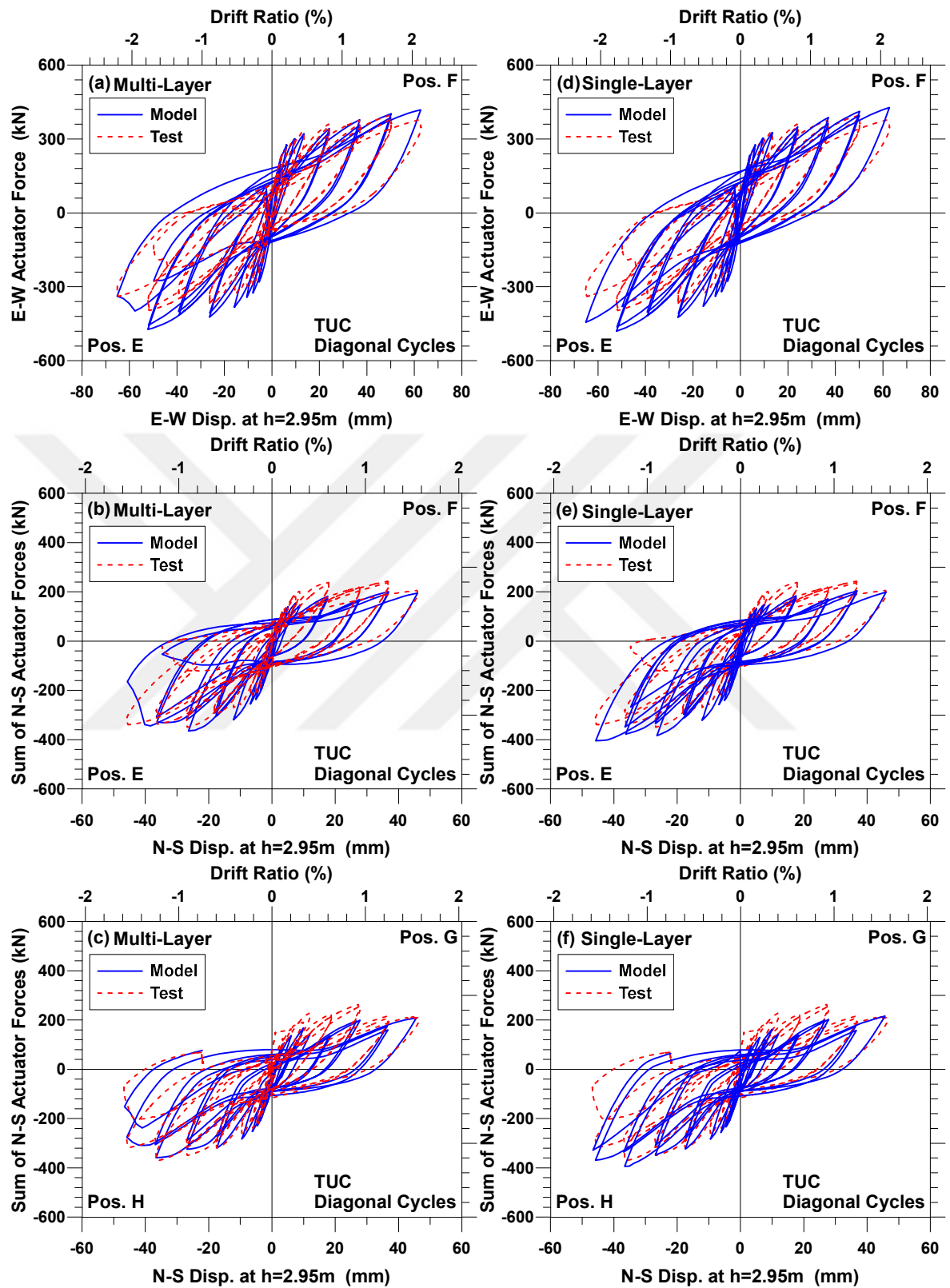


Figure 3.31. Sensitivity of lateral load vs. top displacement response predictions for specimen UW1 to layer discretization: (a), (b), (c) multiple layered model, (d), (e), (f) single layered model.

4. NONLINEAR DYNAMIC RESPONSE SIMULATIONS

The dynamic response predictions presented in this section aim to demonstrate the overall dynamic response simulation capabilities of LFSAFE model, for an isolated U-shaped wall specimen and a wall/frame test building, under multi-directional and sequentially-applied seismic excitations. The accuracy of the LFSAFE models in predicting the test results is not critically-evaluated, since other factors are also expected to influence the accuracy of the response predictions, including the damping ratio assumed in analysis, warping restraint imposed on the U-shaped wall by the collar beam, modeling approaches used for other structural components (beams and columns) in the test building, the effect of floor slabs on the response of the test building, the effect of previously-applied excitations (in the base-slip configuration), on the response of the test building, etc. However, discrepancies between the analytical response predictions and the test measurements are clearly identified, and possible reasons of these discrepancies are discussed.

4.1. Description of the Shake Table Experiments

In order to demonstrate the dynamic response simulation capabilities of the LFSAFE model, the two shake table test programs conducted by Hoult et al. (2024) and Kajiwarara et al. (2021) are simulated in OpenSees. The first test specimen is a half-scale isolated U-shaped wall specimen (UWS1, Almeida et al., 2024; Hoult et al., 2024) tested very recently under consecutive uni-directional and bi-directional ground excitations, at the National Laboratory for Civil Engineering (LNEC), Portugal. The second test specimen is a full-scale ten-story wall-frame building structure (Kajiwarara et al., 2017; Sato et al., 2017; Tosauchi et al., 2017) tested in 2015 at the E-Defense shake table facility in Japan, under three-directional excitation that also includes the vertical ground motion component.

The U-shaped wall specimen UWS1 (Figure 4.1b), has the same cross-sectional dimensions and reinforcement detailing with specimens UW1 and UW2, which were briefly described in Section 3.1, and scrutinized in Sections 3.5 and 3.6 in validation studies of the LFSAFE model in this study. However, specimen UWS1 was taller than specimens UW1 and UW2, with a net height of 4040mm (excluding foundation and collar height), and the

specimen incorporated two 100mm-thick surrounding (outer) story slabs along its height. The wall unit was constructed on a 500mm-thick rigid foundation affixed to the shake table. In order to support the masses attached to the top of the wall specimen, a U-shaped RC collar beam was constructed on top of the specimen, which was in the form of an extension of the U-shaped wall, along which the wall thickness was increased to 500mm over a depth of 700mm. The cross-sectional dimensions of the wall specimen (measured along outside faces) were 1050mm parallel to the flanges, and 1300mm parallel to the web, where the thickness of the wall was 100mm. As shown in the cross-sectional view (Figure 4.1a), the wall specimen had four confined boundary regions, two located at the flange free ends and the remaining two at the web-flange intersections.

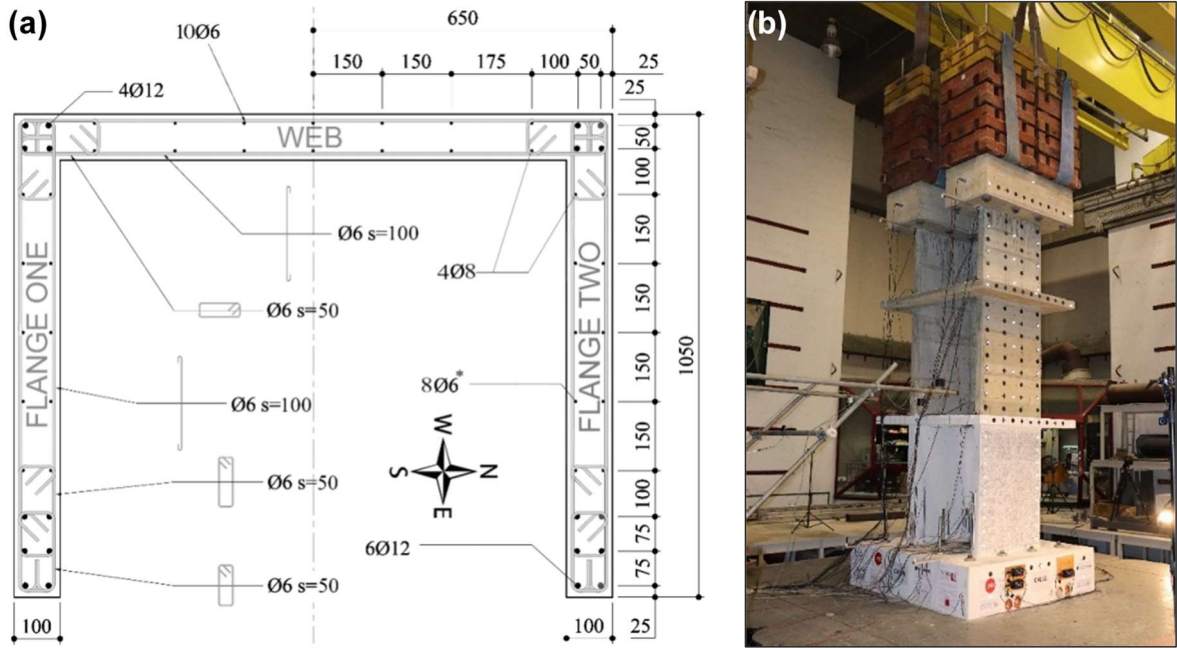


Figure 4.1. Specimen UWS1; (a) wall cross-section (all dimensions are in mm), and (b) photograph of the specimen (Hoult et al., 2024).

The reported average concrete compressive strength for the wall specimen, obtained from test results on cylinder specimens, was 27.0 MPa. Results of the tensile stress-strain tests conducted on longitudinal reinforcing steel bar samples are presented in Table 4.1. The notations d_{bl} , f_y , f_u , ϵ_{sy} , ϵ_{sh} , ϵ_{su} , and E_s in Table 4.1 refer to the diameter of the longitudinal reinforcing bar, yield strength, ultimate strength, yield strain, hardening strain, ultimate strain, and Young's modulus, respectively.

Table 4.1. Reinforcing steel bar properties for specimen UWS1.

d_{bl} (mm)	f_y (MPa)	f_u (MPa)	ϵ_{sy} (mm/mm)	ϵ_{sh} (mm/mm)	ϵ_{su} (mm/mm)	E_s (GPa)
6	577	623	0.0028	-	0.046	208
8	538	664	0.0027	0.024	0.120	196
12	580	690	0.0029	0.021	0.101	199

In order to enforce the specimen to exhibit nonlinear flexural behavior, additional masses with a total mass of 24.96tons were affixed to the top collar beam of the specimen. Combined with the approximate masses of the top collar beam (3.27tons) and the wall unit itself (4.17tons), the total mass of the specimen above the rigid foundation was 32.4tons. It must be noted that the specimen incorporated significant torsional eccentricity in the N-S direction according to the cardinal directions given in Figure 4.1a, between its linear elastic shear center and the center of the additional mass sources, as well as the mass of the wall itself and the collar beam at the top.

For shake table testing of specimen UWS1, horizontal components of the ground accelerations corresponding to the largest event (M_w 6.5) of 2016 Central Italy earthquake sequence, recorded at station MZ04, were used. Ground motions used during the tests, namely the corrected ground motions, are a processed version of the original ground motion records, where similitude laws on the original record were applied (i.e., the time step of the records was shortened). The excitation protocol used during the test program consists of consecutive application of uni-directional and bi-directional excitations, with the increasing ground motion amplitudes (scale factors) listed in Table 4.2.

Table 4.2. Corrected ground motion scale factors used in excitation of specimen UWS1.

	<i>GM0</i>	<i>GM1</i>	<i>GM2</i>	<i>GM3</i>	<i>GM4</i>	<i>GM5</i>	<i>GM6</i>	<i>GM7</i>	<i>GM8</i>
E-W direction	10%	25%	25%	50%	50%	75%	75%	100%	100%
N-S direction	-	-	25%	0	50%	0	75%	0	100%

The 2% damped acceleration and displacement spectra for the corrected ground motion components used in excitation of the specimen in the E-W (PGA: 0.81g) and N-S (PGA: 0.64g) directions (Figure 4.1a) are shown in Figure 4.2.

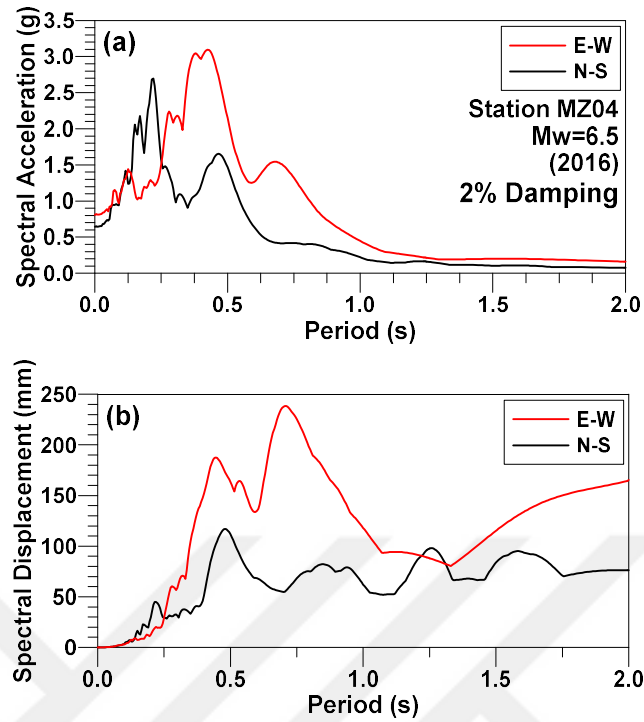


Figure 4.2. Corrected ground motion (100% scale) used in shake table tests for specimen UWS1: (a) acceleration, and (b) displacement response spectra for 2% damping.

During the tests, the first major damage observed on the wall specimen was crushing of concrete at southwest web-flange intersection, during application of bi-directional excitation GM6 (Figure 4.3a). During this excitation, maximum drift ratios measured on the wall were 1.65% in E-W direction and 0.65% in N-S direction, together with a maximum top twist rotation of 0.025rad. Sliding shear at the base of the wall, along a crack at the wall-foundation interface (Figure 4.3c), was also reported during this excitation, which may have triggered spalling of concrete at web-flange intersections, due to kinking of the longitudinal reinforcing bars in these regions. Further concrete crushing was observed in the south flange free-end boundary region, as well as at the web-flange intersections at both southwest and northwest corners of the wall specimen, under the uni-directional excitation GM7 (100%). During this excitation, the largest wall drift ratio was measured as 2.71% in E-W direction, and the sliding shear deformation at the base of the wall reached up to ± 6 mm. Finally, during the last bi-directional excitation GM8 (100%), maximum drift ratios were measured as 1.82% in E-W direction and 0.76% in N-S direction, and the maximum top twist rotation was measured as 0.025rad. This excitation caused concrete crushing in both of the flange free-end boundary regions, combined with noticeable buckling of longitudinal reinforcement in the south flange free-end boundary region (Figure 4.3b) (Hoult et al., 2024).

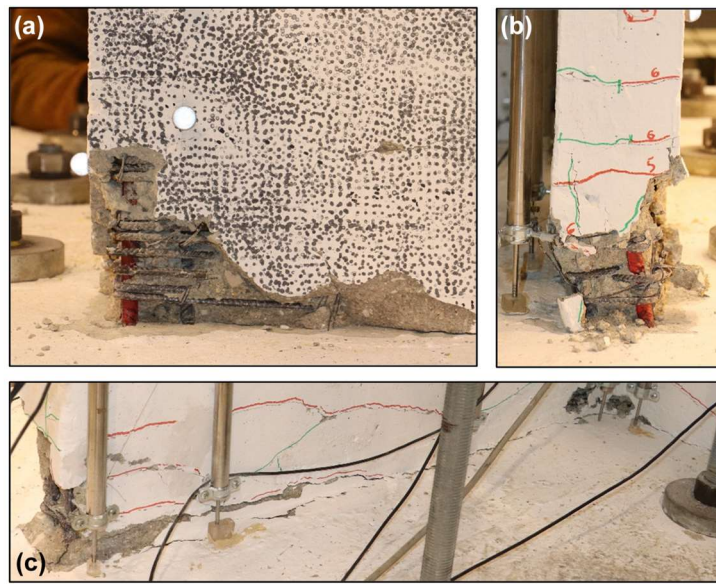


Figure 4.3. Observed damage on specimen UWS1; a) crushing of concrete at web-flange intersection, b) crushing of concrete and buckling of reinforcement at flange free-end boundary region, and c) wall-foundation interface crack at wall base.

The full-scale 10-story reinforced concrete building specimen tested at E-defense shake table facility in 2015, namely the E-defense 2015 specimen, is a wall-frame building-type structure with a total height of 25.9m (Figure 4.4). As shown in the typical floor plan for the test building (Figure 4.5a), plan dimensions are 12m in one direction and 8m in the other direction, and the lateral load resisting system of the building in the longer plan direction, namely the “frame direction”, consists of moment resisting frames, whereas in the shorter plan direction, namely the “wall direction”, the main lateral load resisting members are rectangular walls. Due to limitations of the shake table, structural walls in the building continued only up to the eighth-floor level (8FL in Figure 4.5b), and the structural system converts into moment resisting frames along the top three stories in the wall direction. The building incorporated three typical column sections, namely C1, C2, and C3, where C1 and C2 had cross sectional dimensions of 550mm×550mm, 550mm×500mm, and 500mm×500mm along the first story, second story, and the remaining stories of the building, respectively. On the other hand, column C3, which was designed as the boundary region of the structural walls, had cross sectional dimensions of 230mm×450mm along the entire building height. The width and depth of the beams varied from 230mm to 350mm, and 370mm to 550mm, respectively. The web thickness of the walls was 230mm along the first six stories, reducing to 150mm along the seventh story, and slab thickness at all floors was

120mm. During the experimental campaign, the tests were performed under two different connection conditions between the foundation of the building and the shake table, in order to assess use of sliding bearings as a seismic mitigation technique. During the first stage of the test program, which is referred to as the “base-slip” stage, sliding of the building on friction-type bearing elements was allowed. After completion of the first stage, the building was affixed to the foundation beams, and the ground motions were reapplied, during namely the “fixed-base” stage of the test.



Figure 4.4. Photograph of 2015 E-defense 10-story building specimen.

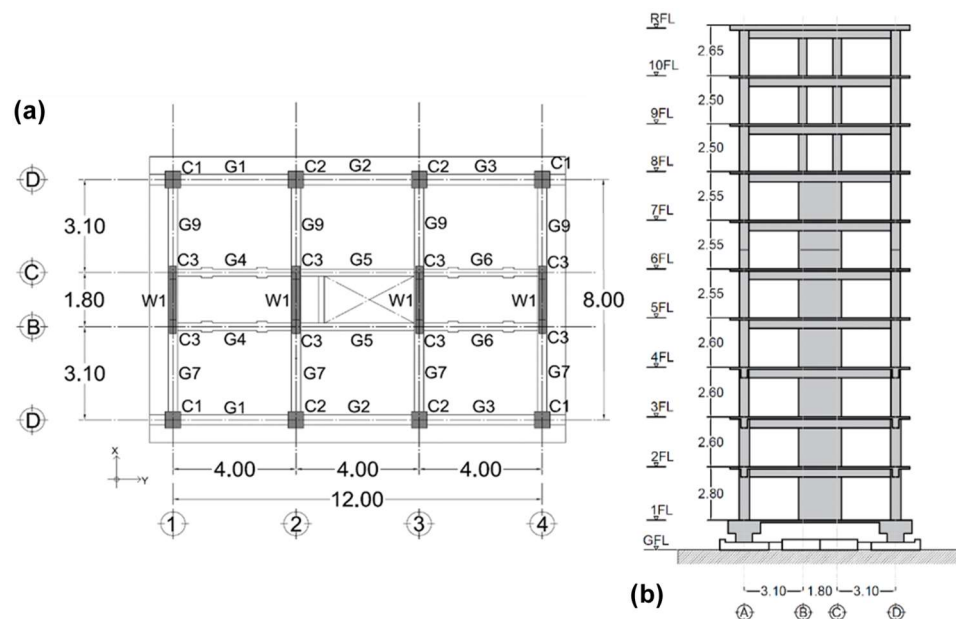


Figure 4.5. Geometry of 2015 E-defense 10-story building specimen: (a) typical floor plan, (b) elevation view in wall direction (all dimensions are in m.).

The test building was designed in compliance with the Japanese Building Standard Law (AIJ, 2010) and the Japanese Structural Technical Standard Manual for Buildings (MLIT, 2007). Concrete strengths adopted in design were 42MPa, 33MPa and 27MPa, varying among the stories. As shown in Table 4.3, material tests conducted on concrete samples provided higher compressive strengths than anticipated. In design and construction of the test building, three different steel grades, namely SD295, SD345 and KSS785, were used for reinforcing steel, for which average values of the mechanical properties obtained from reinforcing steel bar sample test results are listed in Table 4.4.

Table 4.3. Concrete compressive strength test results for the E-Defense2015 test building.

Story	1 st	2 nd	3 rd	4 th	5 th	6 th _{lower}	6 th _{upper}	7 th	8 th	9 th	10 th
Testing Age (Day)	237	211	189	164	143	134	253	227	204	179	153
Test Strength (MPa)	68.7	72.0	57.6	48.1	45.5	42.1	54.6	49.0	51.9	43.9	41.0
Design Strength (MPa)	42.0	42.0	33.0	33.0	33.0	27.0	27.0	27.0	27.0	27.0	27.0

Table 4.4. Reinforcing steel bar properties for the 2015 E-Defense test building.

	d_{bl} (mm)	f_y (MPa)	f_u (MPa)	ϵ_{sy} (mm/mm)	ϵ_{sh} (mm/mm)	ϵ_{su} (mm/mm)	E_s (GPa)
SD295	10-16	370	515	0.0019	0.01	0.26	190
SD345	19-22	390	581	0.0021	0.01	0.23	190
KSS785	10	931	1100	-	-	0.12	205

During the experimental program, the JMA-Kobe strong ground motion record, recorded at Kobe's Nakayama district during the 1995 Hyogo-ken Nanbu Earthquake, was used for excitation of the 10-story test building. All three components of the recorded ground accelerations were applied simultaneously, where the N-S component of the ground motion was oriented in the frame direction, and the E-W component was oriented in the wall direction of the test building. The excitation protocol of the test was sequential application of the recorded ground motion with increasing amplitudes (scale factors) of 10%, 25%, 50%, and 100%, during the base-slip stage, followed by the same amplitudes during the fixed-base stage. Lastly, an additional excitation was also applied, with a 60% scale factor, to represent possible aftershocks following a large earthquake.

In this study, for analysis of the E-Defense 2015 test building, average base accelerations (PGA: 1.04g in frame direction, 0.73g in wall direction, and 0.43g in vertical direction for 100% amplitude) that were recorded using the accelerometers located on the foundation level (base of columns and structural walls) of the building, are used. The reported maximum story drift ratios, in both the frame (0.60%) and the wall (0.30%) directions of the building, that were recorded during base-slip stage of the test were relatively small and did not cause notable damage on the specimen (Tosauchi et al., 2017). Within the scope of this study, only the fixed-base stage of the test program is simulated, for which significant maximum story drifts were reported (3.05% in frame direction, 1.50% in wall direction). The corresponding acceleration response spectra of the recorded foundation-level (building base) accelerations, generated for 2% damping ratio, are presented in Figure 4.6 for 10%, 25%, 50%, and 100% amplitude excitations for the fixed-base stage of the test.

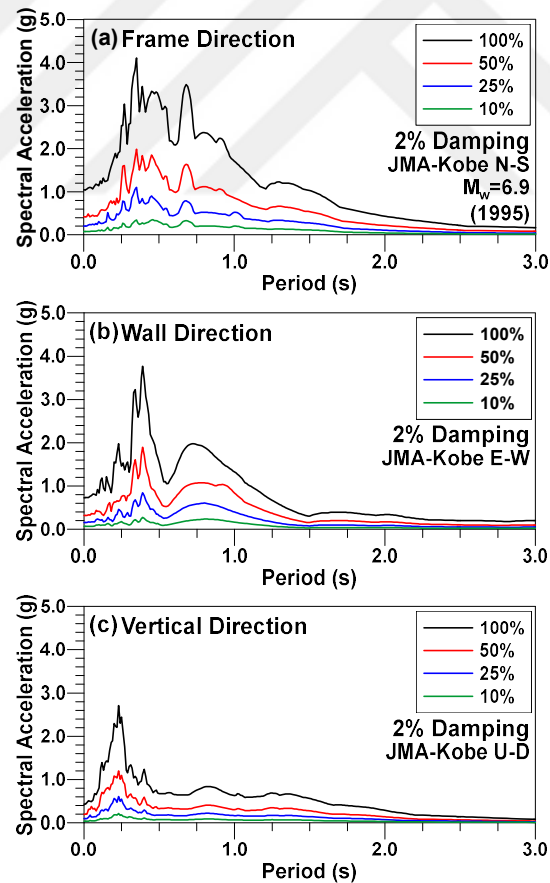


Figure 4.6. Acceleration response spectra for the recorded ground accelerations at the foundation level of the E-Defense 2015 test building, during the fixed-base stage of the test: (a) in frame direction, (b) in wall direction, and (c) in vertical direction.

The most significant damage observed on the test building was shear damage to beam-column joints, where diagonal cracks were first observed during the 50%-amplitude fixed-base excitation, and widening of the cracks together with spalling of concrete was reported during the 100%-amplitude excitation. Furthermore, flexural cracks developing on the beams, together yielding of beam longitudinal reinforcement, was also observed (Figure 4.7a). Limited concrete spalling was reported at the base of the columns and the structural walls, whereas flexural cracking was clearly observed on both (Figure 4.7b, c). On the structural walls, horizontal interface cracks developing between the base of the walls and foundation the slab were detected, possibly stemming from yield penetration (extension/slip) effects on the wall longitudinal reinforcement within the foundation slab (Figure 4.7d). Similarly to structural walls, interface cracks at base of columns were also observed.

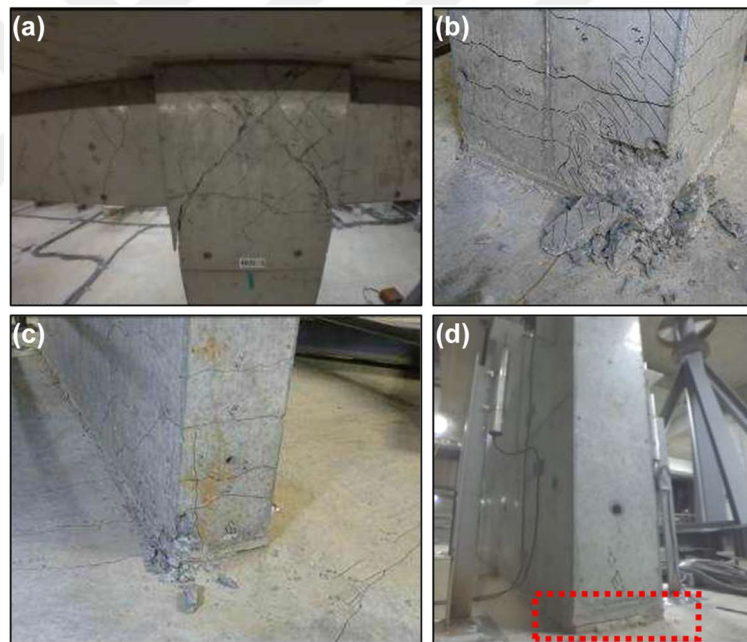


Figure 4.7. Observed damage on the E-Defense 2015 test building; (a) flexural cracking of beams and damage on beam-column joints, (b) spalling of cover concrete at the base of a column, (c) minor spalling of cover concrete in wall boundary regions at the base, and (d) wall-foundation interface crack at wall base.

4.2. Model Generation Methodology

Analysis models for specimen UWS1 and the E-Defense2015 test building were generated in OpenSees, using the LFSAFE model elements for the walls. In generation of

the analysis models, for consistency, a unified model calibration approach is adopted among the two specimens, which is the same calibration approach used in model validation studies against quasi-static tests presented in Section 3 of this thesis. In the analysis model for specimen UWS1, model elements at wall boundary regions are discretized along wall length so that the length to thickness ratio of the shell elements do not exceed 2.0, which requires at least two model elements side-by-side for boundary regions with 300mm length and 100mm thickness. The same approach requires single model elements for the boundary regions of the rectangular walls (450mm length and 230mm thickness) in the E-Defense 2015 test building. However, due to the connecting frame elements in transverse direction of the walls at the midpoint of boundary regions, these regions are also modeled also using two model elements side-by-side. The model elements are then discretized across the entire wall geometry so that the height-to-length ratios of all model elements are close to one. Resulting model element discretization for specimen UWS1 and the walls in the E-Defense 2015 building specimen are presented in Figure 4.8.

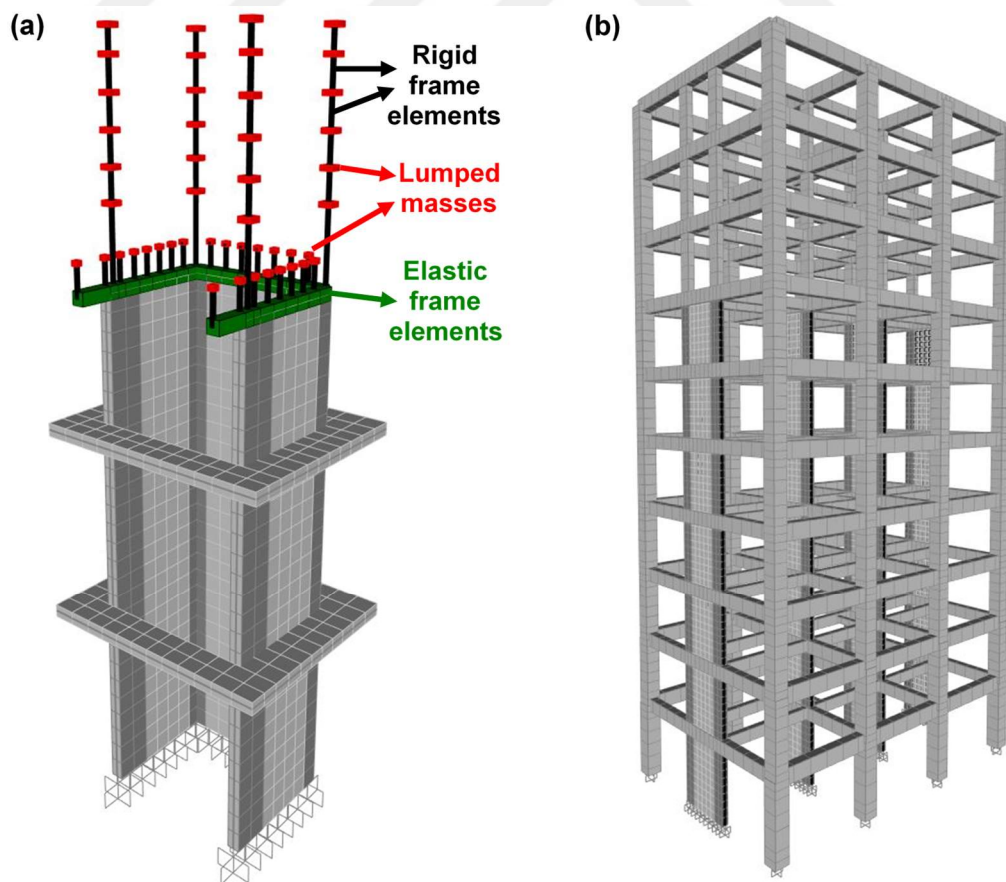


Figure 4.8. OpenSees models generated for: (a) specimen UWS1, (b) E-Defense 2015 test building.

For both specimens, the walls are divided into nine membrane layers across their thickness. These layers account for various concrete confinement and reinforcement characteristics, including two cover concrete layers (unconfined concrete with no reinforcement), two horizontal reinforcement layers (unconfined concrete with horizontal reinforcement only), two vertical reinforcement layers (confined concrete with vertical reinforcement only), and three core concrete layers (confined concrete with no reinforcement). Sensitivity studies conducted on the analysis results showed that using a finer model discretization did not result in notable difference in the model response predictions.

In calibration of material model parameters, whenever material test results were available, constitutive material parameters of concrete and reinforcing steel are calibrated to match the test results. Otherwise, well-established empirical relationships are used to define the material parameters. For example, for specimen UWS1, parameters (modulus of elasticity, strain at peak stress and shape parameter) required for definition of the unconfined concrete stress–strain curve in compression are defined according to the empirical equations proposed in Chang and Mander (1994), since only average compressive strength of the concrete samples was reported. On the other hand, for the E-Defense 2015 test building, these parameters are calibrated so that that resulting monotonic compression envelope of the constitutive model for concrete matches with the reported stress–strain curves obtained from cylinder tests. Confined concrete behavior for both specimens are defined according to the confinement model by Mander et. al. (1988), where the post-peak slope of concrete compression envelopes adopted for both unconfined and confined concrete are calibrated to match the post-peak slope of the Saatcioglu and Razvi (1992) model. Concrete tensile strength and the corresponding tensile strain are defined as $0.31\sqrt{f'_c}$ and 0.00008, as proposed by Belarbi and Hsu (1994). The post-cracking stress-strain envelope for concrete in tension is also calibrated to agree with the equation by Belarbi and Hsu (1994). For reinforcing steel, tensile yield strength and tensile strain hardening ratio parameters are defined to agree with the steel stress-strain test results reported for both the UWS1 and E-Defense 2015 specimens. However, these parameters are modified, as described by Belarbi and Hsu (1994), to consider the effects of tension stiffening on reinforcing bars embedded in concrete. For representation of the Bauschinger's effect, hysteretic parameters describing the cyclic stiffness degradation characteristics of the constitutive relationships adopted for

reinforcing steel are defined as $R_0 = 20$, $\alpha_1 = 18.5$, and $\alpha_2 = 0.15$, as recommended originally by Menegotto and Pinto (1973) and also adopted by Dhakal and Maekawa (2002a). Isotropic strain hardening is not considered in the constitutive relationships adopted for reinforcing steel. The friction coefficient in the constitutive model adopted for shear aggregate interlock along crack surfaces is defined as 0.40 (a value of 0.44 was recommended by Tassios and Vintzeleou, 1987).

In terms of reproducing wall boundary conditions, the top collar beam of specimen UWS1 is included in the analysis model of the wall, using linear elastic frame elements (elements highlighted in green in Figure 4.8a). The flexural stiffnesses (EI) of the frame elements used for the collar beam is amplified by a factor of 1000 in their major bending direction, for eliminating very local vibration modes, since the top masses are attached rigidly to these frame elements. For the same reason, the torsional stiffnesses (GJ) of the frame elements located at the top of the wall flanges are also amplified by the same factor. Similarly to the collar-beam modeling approach adopted for the torsionally-loaded U-shaped wall specimen UW2 investigated Section 3 of this study, for the central frame element located at the top of the web of the wall specimen, an effective torsional stiffness of $0.05GJ$ is defined, in order not to fully restrain the torsional warping of the wall cross-section during the excitation. Furthermore, a rigid diaphragm constraint is defined at top of the wall specimen, for achieving more uniform load transfer in the region.

The additional weights at the top of the wall, as well as the self-mass of the collar beam, are defined in the model as lumped masses (shown in red in Figure 4.8a) at elevations corresponding to the mass centers. These masses are connected to the top collar beam elements using fully-rigid frame elements (shown in black in Figure 4.8a). Furthermore, although not shown explicitly in Figure 4.8a, lumped masses corresponding to the self-weight of the wall specimen are defined at the nodes of the model, based on the model element discretization and the corresponding tributary volumes of the model element nodes. Vertical (gravity) loads that are compatible with all nodal masses are also defined at corresponding nodes of the model. The flexural stiffness of the floor slabs surrounding the wall specimen at the first and second floor levels can potentially induce a coupling effect on the wall flanges (mostly flexural and to a lesser extent torsional, in the form of restraint against warping), which would influence the response of the wall in north-south direction,

as well as its torsional response. Therefore, the floor slabs on the wall specimen are also modeled using LFSAFE model elements, so that their nonlinear out-of-plane behavior is considered in the analysis. Finally, fixed support conditions are defined at the wall-foundation interface, restraining all translational and rotational DOFs, similarly to the approach adopted in quasi-static response simulation in Chapter 3.

In generation of the analysis model for the E-Defense 2015 test building, displacement-based beam-column elements (with three integration points) with fiber cross sections are used for modeling of the beams and columns. Discretization of the frame elements involves using at least six elements along the length of each member, also taking into account the expected plastic hinge lengths (half of the depth of cross section). The fiber cross sections are defined using 484 (22x22) concrete fibers for the columns and 66 (3x22) concrete fibers for the beams, also considering the confinement geometry of the cross sections. Similarly to the LFSAFE model elements employed for the walls, in the beam-column model elements, uniaxial behavior of the concrete and reinforcing steel fibers are defined employing the hysteretic material models by Chang and Mander (1994) and Menegotto and Pinto (1973). Buckling of reinforcement is not considered in the beam-column model elements. Material model parameters of the beam-column elements are calibrated using the same methodology adopted for the wall model elements. However, the effects of tension stiffening on the tensile yield strength and tension strain hardening ratio are not considered, since the relevant methodology (Belarbi and Hsu, 1994) does not apply to frame element cross-sections.

During the tests, significant shear damage was observed on the beam-column joints (due to joint shear), although in the frame direction of the building only. Since this study focuses on the response of the test building in the wall direction, and such beam-column joint damage influences the response of the building only in the frame direction, a nonlinear modeling approach is not adopted for the beam-column joint regions. Instead, beam-column joint flexibility is implicitly represented in the analysis model using elastic beam and column elements in the joints, where the flexural stiffness of only the column elements only is amplified by a factor of 1000, as per the recommendation in ASCE 41-17 (2017) for beam-column joints satisfying the strong-column weak-beam condition.

Presence of RC floor slabs, as well as their influence on the beams (i.e., increase in flexural stiffness and moment capacity) are neglected in the analysis model, and a rigid diaphragm kinematic constraint is applied at each floor level. However, since the beams in the building are modeled using beam-column elements with fiber sections, in order to eliminate the rigid diaphragm restraint against axial-growth in the beam elements, only the end nodes of the beam elements are connected to the rigid diaphragm, and an axial release is defined on the beam elements at mid-length, allowing axial elongation. Since floor slabs are not included in the analysis model, their self-weights are defined as distributed loads on surrounding beam elements. Since nonlinear dynamic analysis of the test building is conducted under three-directional excitation, to take into account the effects of vertical ground motion component on the response, the distributed mass of the building is defined as tributary lumped masses in all three directions. The tributary lumped masses are defined at the floor-level nodes of the wall and column model elements, which are connected to the rigid floor diaphragms. Distributed masses are not assigned along the length of the beam and column model elements, in order to eliminate very localized vibration modes, although the masses of all members are considered in the tributary lumped mass calculations. Fixed-support conditions are enforced at the base nodes of the column and wall model elements. The foundation slab of the building is not included in the analysis model, since the accelerations recorded at the top of the foundation are used in analysis. For analysis of both the U-shaped wall specimen UWS1 and the E-Defense 2015 test building, mass and stiffness proportional (Rayleigh) damping with 2.0% damping ratio (for the first and third vibration modes of the U-shaped specimen, and for the first and ninth vibration modes of the E-Defense 2015 test building) is defined. The 2.0% damping ratio was selected as an approximate value, and was adopted for both the isolated wall and wall-frame system, for the sake of consistency. Naturally, the real damping characteristics of two specimens may differ.

4.3. Nonlinear Analysis Solution Strategy for Dynamic Response Simulations

Similarly to the solution strategy described in Section 3.4 of this study, an analysis object for nonlinear dynamic analysis is created, employing Transformation constraint handler, Reverse Cuthill-McKee (RCM) (Cuthill and McKee, 1969) numberer, UmfPack (Davis, 2004) system of equations, Load Control integrator for gravity load analysis and

Newmark integrator (with average acceleration method) for transient loading, Krylov-Newton (Scott and Fenves, 2010) (with current stiffness) and Modified Newton (with initial stiffness) algorithms, and Norm Displacement Increment and Unbalance (NormDispAndUnbalance with max-norm) convergence test for specimen UWS1 and Norm Displacement Increment Test (NormDispIncr with second-norm) for E-defense building specimen.

In transient analysis, solution within a time step is first searched by Krylov-Newton algorithm, if this approach fails to find a solution, Modified Newton method is used later. If both algorithms fail to find a solution within a time step, the step size is reduced (i.e. divided to 10) for that specific step and the solution is re-searched with Modified Newton method. Total excitation duration for applied ground motions are 270s (nine consecutive ground motions with 6000 time steps and 0.005s step size) for UWS1 specimen, and 128s (four consecutive ground motions with 3200 time steps and 0.01s step size) for E-defense building specimen. Finally, convergence criteria adopted in dynamic response simulations of UWS1 are 0.005mm for displacement increments (translational displacements) and 0.50kNm for unbalanced forces (unbalanced moments), while 0.0025mm second-norm displacement increment criterion is used in simulations of E-defense building specimen.

4.4. Dynamic Response of the U-Shaped Wall Specimen

The uni-directional and bi-directional shake table tests conducted on the isolated U-shaped wall specimen UWS1 are first simulated in OpenSees, using the LFSAFE model elements for the wall. The excitation protocol applied during testing is replicated by application of the nine consecutive ground motion records on the analysis model. Model predictions for the global response quantities of the specimen are compared with experimental measurements, in terms of top displacement, top twist rotation, and story acceleration response histories. Furthermore, representative local deformation predictions of the model for the average longitudinal strains at the base of the wall and the shear distortions (average shear strains) on a wall flange along the first story height of wall are compared with test measurements. Model predictions for the displacement, twist rotation, and story acceleration response histories are presented only for the last four excitations (GM5, GM6, GM7 and GM8), focusing on the response under relatively large ground motion amplitudes

applied both uni-directionally and bi-directionally. Representative local deformation comparisons of model predictions (or post-dictions obtained using an updated model configuration) and test measurements are provided only for the uni-directionally applied ground excitations GM5 and GM7.

A representative deformed shape (using an amplification factor of 12.0) obtained from the analysis of specimen UWS1, corresponding to a time instant with large lateral displacement and top twist rotation demands experienced during the bi-directional excitation GM8 (at 8.245s), is presented in Figure 4.9. The red lines in the figure represent the undeformed shape of the specimen. The analytically-obtained deformed shape clearly demonstrates the lateral displacements in both directions of the wall, as well as torsional twisting of the wall, indicating a combined flexural/torsional response. Warping of the wall cross section under torsion is also visible in the deformed shape. It must be noted that not imposing torsional cracking on the middle segment of the U-shaped collar beam (a cracked torsional stiffness of $0.05GJ$ is presently imposed) would have restrained warping of the wall cross-section at the top, almost to full extent.

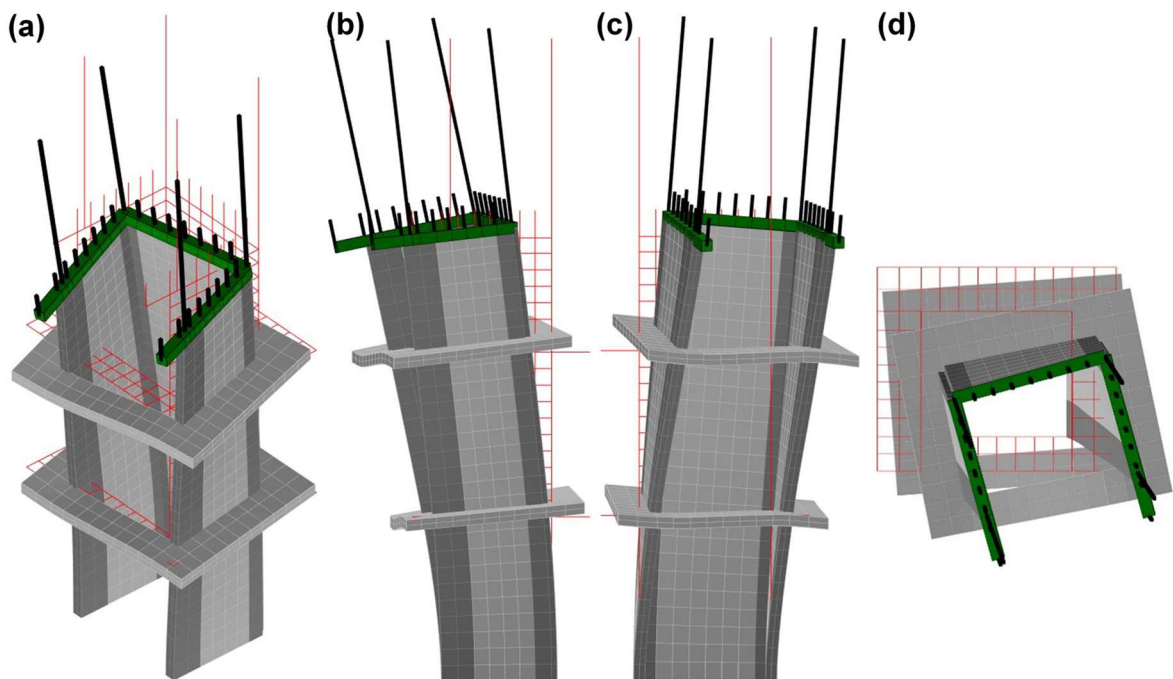


Figure 4.9. Representative deformed shape obtained from analysis of specimen UWS1; a) 3D view, b) North side view, c) East side view, and d) Top view.

Comparison of test measurements with model predictions for the top displacement time history responses of specimen UWS1, in the E-W direction under the uni-directional excitations GM5 (75% amplitude) and GM7 (100% amplitude) are provided in Figure 4.10. As demonstrated in Figure 4.10a and Figure 4.10b, under the uni-directional excitations GM5 and GM7, the LFSAFE model slightly underestimates the measured peak top displacement values, yet well-replicates the time instants of the peaks, capturing the vibration frequency (period) of the response.

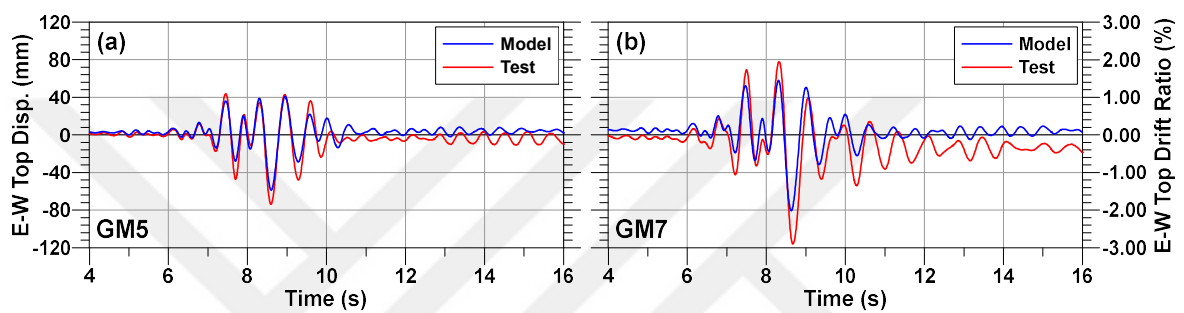


Figure 4.10. Top displacement time histories for specimen UWS1 in E-W direction under uni-directional (a) GM5, (b) GM7.

Moreover, comparison of test measurements with model predictions for the top displacement time history responses of specimen UWS1, in both E-W and N-S directions under the bi-directional excitations GM6 (75% amplitude) and GM8 (100% amplitude) are provided in Figure 4.11 (E-W direction) and Figure 4.12 (N-S direction). In these figures, the model's capability in simulating the response of the non-rectangular wall under bi-directional excitation is demonstrated. The model well-replicates the test measurements under the 75% amplitude (GM6) and 100% amplitude (GM8) excitations, while slightly underestimating peak displacement values in the E-W direction only during the 75% amplitude excitation (Figure 4.11a, b). In the N-S direction, the model response is accurate during the 100% amplitude excitation (Figure 4.12b), with some discrepancy between model results and test measurements observed during the 75% amplitude excitation, after the third major displacement cycle (Figure 4.12a).

The experimentally-observed sliding shear deformation at the base of the wall, along horizontal cracks that developed at the flange-foundation interfaces, cannot be simulated by the analysis model, since the model formulation does not allow formation of a continuous

crack across the wall cross-section. This may have possibly caused underestimation of the top displacement values measured in the E-W direction of the wall. Furthermore, the damping ratio assumed in analysis (presently 2.0%) is also expected to have an influence on the analytical response predictions, making a critical assessment of the model accuracy difficult, as applies to all model response predictions presented henceforth.

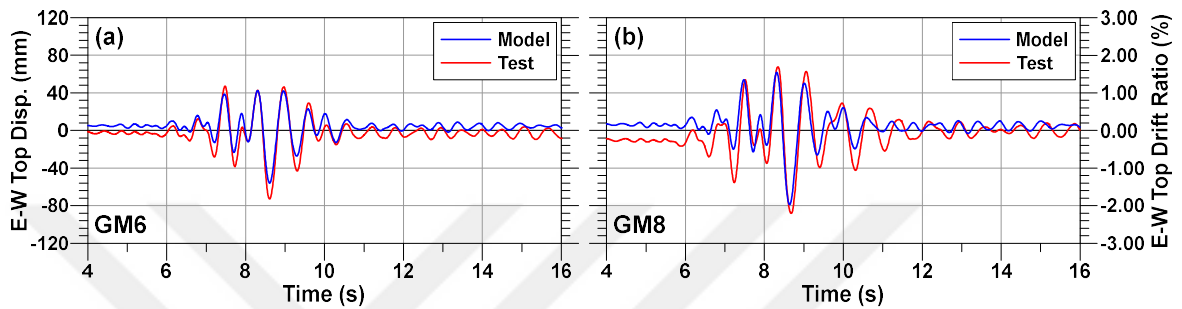


Figure 4.11. Top displacement time histories for specimen UWS1 in E-W direction under bi-directional (a) GM6, (b) GM8.

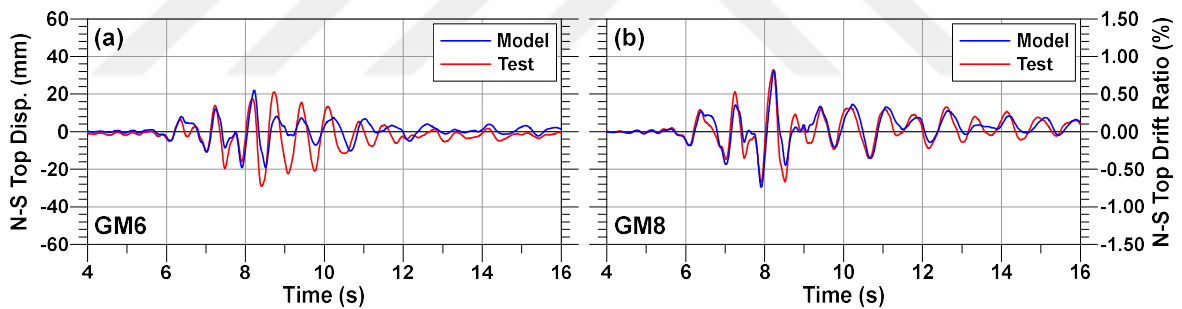


Figure 4.12. Top displacement time histories for specimen UWS1 in N-S direction under bi-directional (a) GM6, (b) GM8.

Comparison of model predictions with the experimentally-measured top twist rotation time histories of specimen UWS1, under bi-directional 75% (GM6, Figure 4.13a) and 100% (GM8, Figure 4.13b) excitations, demonstrates the capability of the LFSAFE model in simulating the torsional deformations developing on a U-shaped wall. Despite moderate underestimation of the test measurements after the fourth major twist cycle, the model captures the peak top twist rotation values that were measured during both excitations.

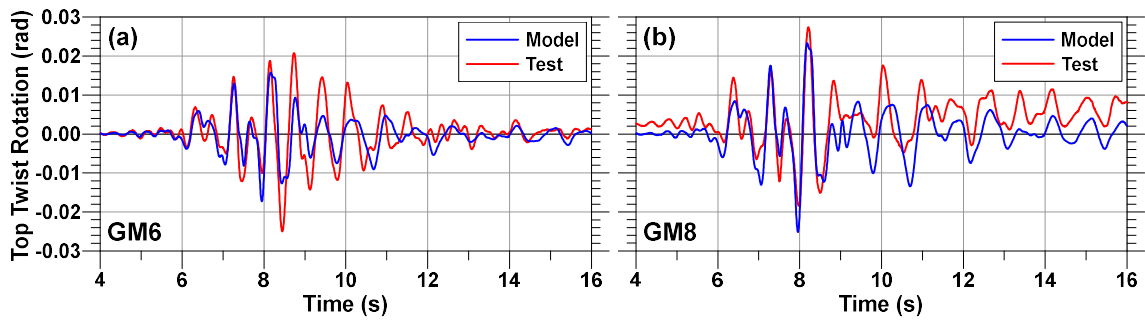


Figure 4.13. Top twist rotation time histories of specimen UWS1 under bi-directional excitations: (a) GM6, and (b) GM8.

Performance of the LFSAFE model in reproducing the acceleration response histories measured along the height of the U-shaped wall specimen under uni-directional and bi-directional excitations with 75% and 100% amplitudes, is evaluated in Figure 4.14, Figure 4.15 and Figure 4.16. The experimentally-measured floor accelerations presented in these figures were obtained after post-processing of the test data, via application of a band-pass filter with 0.25Hz and 10Hz limit frequencies. Under uni-directional excitations, the model reasonably predicts the measured acceleration time histories at the first and second floor levels, including the peak acceleration values. However, at the top floor level, although model predictions are reasonable in the positive E-W direction, the model overestimates the peak acceleration values measured in the negative E-W direction (Figure 4.14a,b).

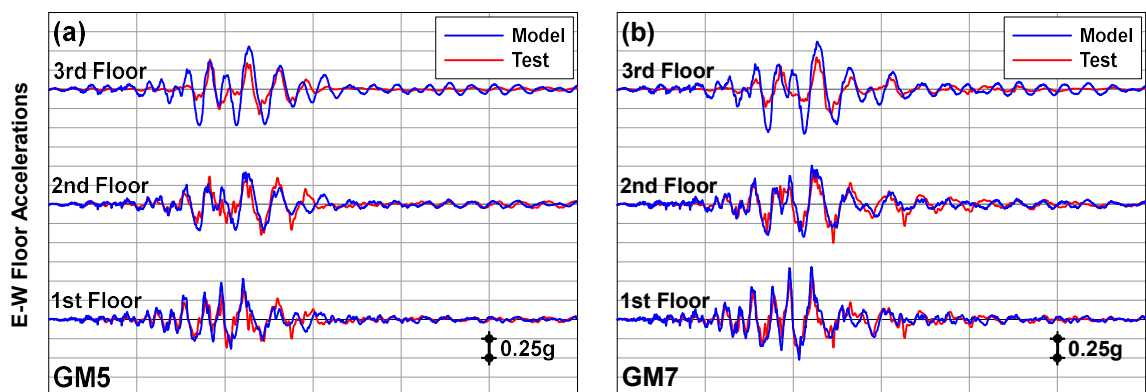


Figure 4.14. Floor acceleration time histories of specimen UWS1 in E-W direction under uni-directional (a) GM5, (b) GM7.

Similarly, under the bi-directional excitations, the model reasonably estimates the accelerations measured at the first and second floor levels, while it slightly overestimates the

accelerations at the third-floor level, in both E-W (Figure 4.15) and N-S (Figure 4.16) directions. Subject to uncertainties associated with damping, boundary conditions (e.g., top collar beam stiffness), and unexpected failure/behavior modes (e.g., sliding shear at wall base), response comparisons presented in this section demonstrate the effectiveness of the model in reasonably simulating the global dynamic response characteristics of a U-shaped wall subjected to both uni- and bi-directional ground motions, in terms of acceleration, lateral displacement, and twist rotation response histories.

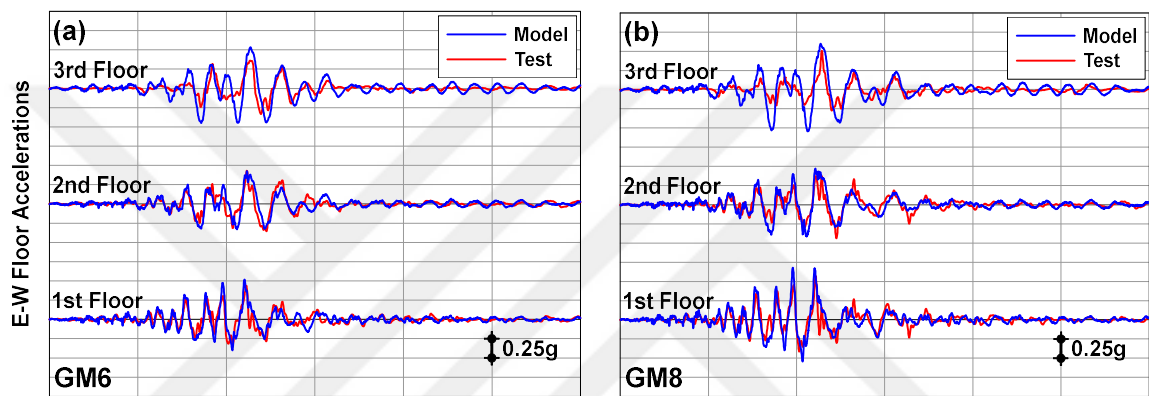


Figure 4.15. Floor acceleration time histories of specimen UWS1 in E-W direction under bi-directional (a) GM6, (b) GM8.

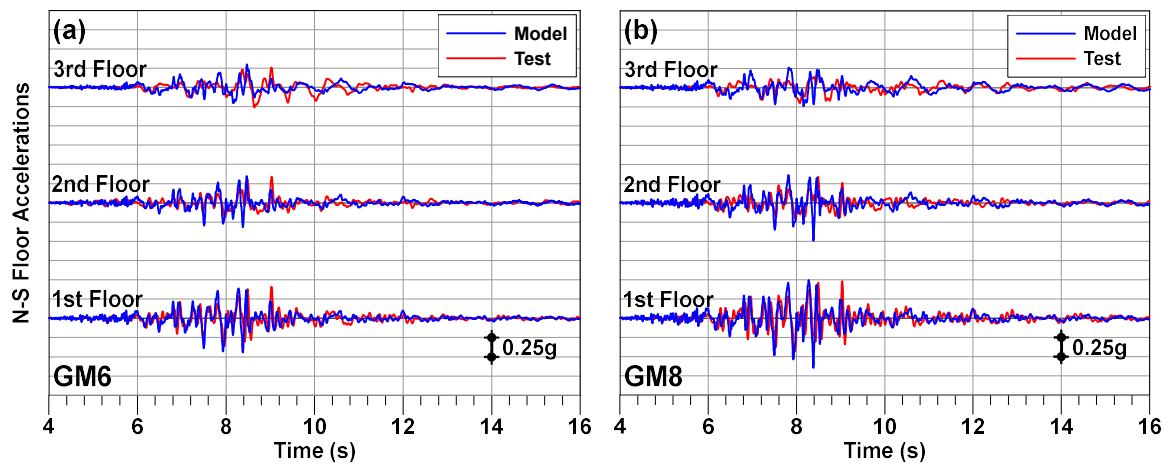


Figure 4.16. Floor acceleration time histories of specimen UWS1 in N-S direction under bi-directional (a) GM6, (b) GM8.

For evaluating the capabilities of LFSAFE model in simulating local (deformation) responses, average longitudinal strains measured using vertical displacement sensors

(LVDTs) with 70mm gauge lengths installed at the base of the wall at the south flange end (LVDT13 in Figure 4.17) and at the south web-flange intersection (LVDT04 in Figure 4.17), as well as the wall flange shear distortions measured using diagonally-oriented spring potentiometers installed along the first story height of the wall specimen (Figure 4.17), are compared with model predictions in Figure 4.18, Figure 4.19 and Figure 4.20, respectively, under ground excitations GM5 and GM7. Comparisons presented in these figures clearly reveal that the “original” model configuration drastically underestimates both the experimentally-measured average vertical strains at the base of the wall and the wall flange shear distortions measured along the first story height of the wall.

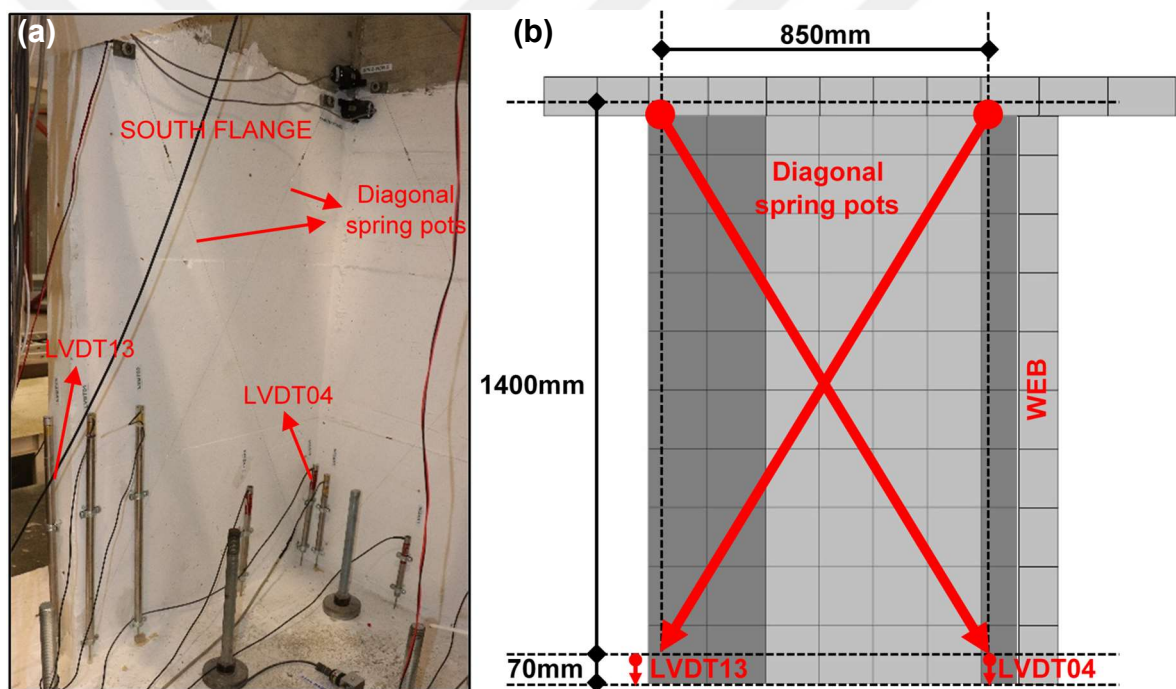


Figure 4.17. Instrumentation for wall deformation measurements: (a) displacement sensors on the specimen, (b) sensor layout.

Similarly to very recent studies presented in the literature on modeling of specimen UWS1 (Hoult and Almeida, 2024b), sensitivity studies conducted using the LFSAFE model for the U-shaped wall specimen have revealed that local deformation estimations of the model are influenced by consideration of strain penetration effects in modeling and the damping ratio assumed in analysis. Since in the original model configuration, the foundation of the wall, the wall-foundation interface, and the strain penetration effects on the longitudinal reinforcing bars in the foundation were not considered, an “updated”

(manipulated) model configuration for the wall specimen was generated, and the aforementioned local deformation measurements were compared with results of the updated model also. In the updated model configuration, the damping ratio was reduced from 2.0% in the original model to a value 0.25%, similarly to the damping ratio adopted in the local deformation post-diction studies conducted for the wall specimen in the paper by Hoult and Almeida (2024b), and strain/yield penetration effects were considered in modeling using the indirect (macroscopic) modeling approach also adopted in Section 3.10 of this thesis for scrutinization of strain penetration effects' influence on quasi-static response (Figure 3.20). For specimen UWS1, the height of additional strain penetration model elements (i.e., the strain penetration depth) is also defined as 220mm, similarly to the strain penetration depth assumed for specimens UW1 and UW2 (recommended by Hoult et al. (2022) based on strain gauge measurements on the reinforcing steel bars within the foundation of these specimens), due to the similarity of specimen UWS1 with specimens UW1 and UW2.

Average longitudinal strain time histories at wall base (along 70mm gauge length), obtained using both the original and updated model configurations, are compared with test measurements at the south flange end (Figure 4.18) and at the south flange-web intersection (Figure 4.19) of the wall under uni-directional excitations with 75% (GM5) and 100% (GM7) amplitudes in E-W direction. Notable improvement is observed in especially the tensile strain predictions obtained using the updated model. In the updated model results shown in Figure 4.18 and Figure 4.19, compared to the original model results, approximately 20% of the increase in the maximum tensile strain predictions is associated with the reduced damping ratio, whereas the remaining 80% is associated with consideration of strain penetration effects in the analysis. However, even with both modifications to the model, the measured tensile strains are still moderately underpredicted at the south flange end, under both excitations (Figure 4.18a, b). At the south web-flange intersection of the wall, tensile strains estimated using the updated model are very close to the test measurements under excitation GM5, while the updated model still underpredicts, to a moderate extent, the maximum tensile strains measured at the web-flange intersection under excitation GM7 (Figure 4.19c, d). The reason why the both the original and updated models drastically underestimate the measured compressive strains at the south flange end of the wall (especially under excitation GM7, Figure 4.18b) is uncertain, and may be associated with the somewhat unexpected concrete crushing and out-of-plane buckling/instability failure

observed at the south flange free-end. At this location, the experimentally-measured compressive strains were very large in magnitude, reaching 0.039 (Figure 4.18b).

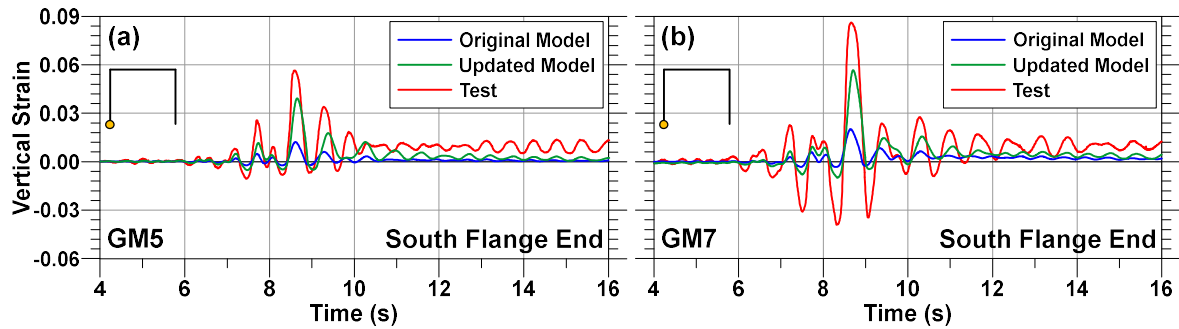


Figure 4.18. Vertical strain time histories of Specimen UWS1 at south flange end: (a) under uni-directional GM5, (b) under uni-directional GM7.

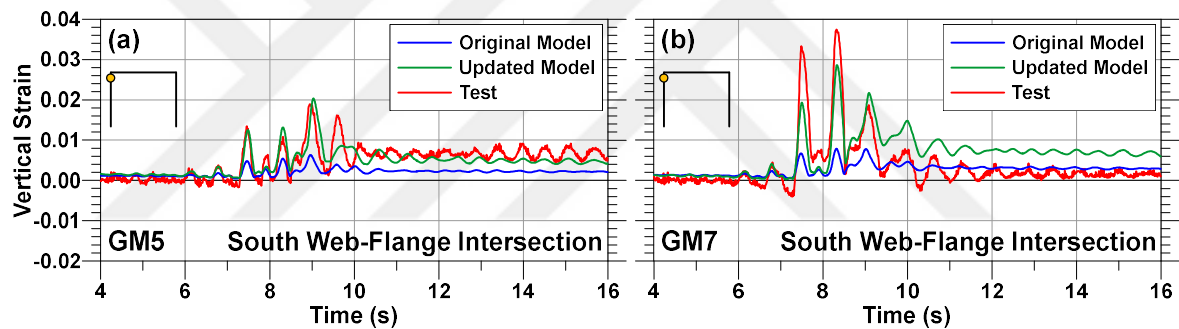


Figure 4.19. Vertical strain time histories of Specimen UWS1 at south web-flange intersection: (a) under uni-directional GM5, (b) under uni-directional GM7.

Shear distortions (average shear strains) measured along the first story height of the south flange of the wall are compared with results of both the original and updated model configurations in Figure 4.20, under the uni-directional excitation GM7 (100% amplitude) in E-W direction only, since the corresponding test data under excitation GM5 was nonsensical. Both experimentally-measured and analytically-predicted shear distortions are calculated using the procedure described by Thomsen and Wallace (1995), based on the measurements obtained from the diagonal spring potentiometers (Figure 4.17), and the nodal displacements in the analysis model corresponding to the diagonal sensor attachment points. Comparisons presented in Figure 4.20 reveal that while the original model configuration underestimates the experimentally-measured shear distortions in both positive and negative directions, the updated model well-predicts the measured shear distortions in the positive

direction, while it overestimates the measured shear distortions in the negative direction. In the updated model results presented in Figure 4.20, compared to the original model results, approximately 35% of the increase in the maximum shear distortion predictions is related to reduction of the damping ratio, while the remaining 65% is associated with consideration of strain penetration effects in modeling. Consideration of strain penetration effects results in larger shear distortion predictions along the first story height of the wall, since smaller flexural deformations are predicted in this region (i.e., along the gauge length of the diagonal potentiometers, which do not straddle the wall-foundation interface).

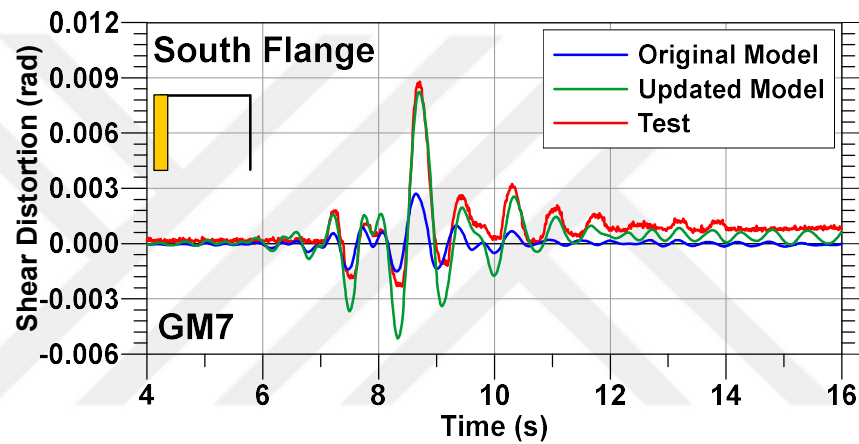


Figure 4.20. Shear distortion time histories of specimen UWS1 at south flange under uni-directional GM7.

Overall, for the U-shaped wall specimen, recalibration of the damping coefficient (from 2% in the original model to 0.25% in the updated model) together with consideration of strain penetration effects in the analysis (albeit indirectly and using a macroscopic approach) have improved the model post-dictions for the average tensile strains developing at the base of the wall and the shear distortions developing on a wall flange. However, these modifications have not significantly influenced the global response estimations of the model under large-amplitude excitations (displacement, twist angle, and acceleration response histories presented in Figure 4.10 to Figure 4.16), to the extent that global response prediction capabilities of the model need to be re-evaluated.

In the analytically-predicted response of the U-shaped wall specimen UWS1, the experimentally-observed crushing of cover concrete and flexural yielding of longitudinal

reinforcement at the flange free-end boundary regions of the wall are reproduced. However, out-of-plane buckling of longitudinal reinforcement at the south flange free-end of the wall is not captured in the analysis results. Furthermore, since the model is incapable of simulating sliding shear deformations developing along a continuous horizontal crack at the wall-foundation interface, the experimentally-measured sliding shear deformations and the experimentally-observed crushing of concrete at the web-flange intersections of the wall (which is believed to be associated with the kinking of the longitudinal reinforcing bars due to sliding shear deformations at the base of the wall), are not replicated in the response simulations.

Overall, the model predicts a nonlinear flexural response for the wall specimen, with yielding of longitudinal reinforcement and crushing of cover concrete, which was the predominant behavioral mode of the test specimen, while it fails to capture the experimentally-observed sliding shear at the base of the wall and the out-of-plane buckling/instability failure at the south-flange free-end boundary region of the wall. It is possible that the sliding shear deformations observed during testing at the wall-foundation interface have triggered instabilities at the wall flanges (such as imperfect crack closure and kinking of longitudinal bars), leading to somewhat premature crushing of concrete together with buckling of longitudinal reinforcement at the flange free-end boundary regions, leading to the excessively large compressive strain measurements shown in Figure 4.18b.

4.5. Dynamic Response of the Ten-Story Wall-Frame Building Structure

Dynamic response prediction capabilities of the LFSAFE model are further evaluated via simulation of the E-Defense 2015 shake table tests conducted on a 10-story wall-frame building specimen. Nonlinear response history analysis of the building is conducted in OpenSees under the four consecutive three-directional ground acceleration record sets (corresponding to 10%, 25%, 50%, and 100% amplitudes) applied during the fixed-base stage of the test program. Since the primary focus of this simulation is to evaluate the performance of the LFSAFE model, analytical response predictions are compared with test results in the wall direction of test building only. Response comparisons are presented only for the 50% and 100% amplitude excitations. Model response predictions are compared with test measurements in terms of both global (displacements, accelerations, story drift ratios,

base shear forces, base overturning moments) and local (wall base rotations, wall shear deformations, beam end rotations, column base rotations) response characteristics.

A representative deformed shape of the building (with an amplification factor of 20.0) obtained using the analysis model is shown in Figure 4.21, at a time instant corresponding to relatively large lateral displacement demands (5.9s of the 100% amplitude excitation). In the figure, red lines correspond to the undeformed shape of the building. The deformed shape shows that the fundamental deformation mode of the test building is bi-directional translation, with no significant torsional deformation observed in the analysis results, which agrees with the test measurements. The structural walls are subjected to displacement demands along their in-plane (Figure 4.21b) and out-of-plane (Figure 4.21c) directions, simultaneously. Furthermore, the deformed shape presented in Figure 4.21b reflects the contribution of frame action (wall-frame interaction) on the response of the test building. Moreover, concentration of flexural deformations on the beam plastic hinge regions can be observed in Figure 4.21b.

Story displacement estimations of the analysis model presented in Figure 4.22 for the 50% and 100% amplitude excitations indicate that the experimentally-measured story displacement amplitudes are predicted with reasonable accuracy, while they are slightly underestimated at lower floors levels. The time instants of the displacement peaks at all floor levels, as well as the frequency characteristics (vibration period) of the responses, are mostly well-captured. When the corresponding maximum story drift profiles shown in Figure 4.23 are compared, significant underestimation of the maximum story drift ratios along the first story is observed during both excitation amplitudes. Additionally, story drifts at upper floors are slightly overestimated in the predictions provided for 100% excitation amplitude (Figure 4.23b). Otherwise, the model provides reasonable estimations for story drift profiles.

The horizontal cracks detected at the wall-foundation interfaces of the test building indicate that the longitudinal bars in structural walls may have been subjected to notable strain penetration effects, which may have decreased the flexural stiffness of the walls and led to underestimation of floor displacements and story drifts especially along the lower stories of the building, where wall flexural stiffness is more influential on the lateral stiffness of the entire structure. In contrast, overestimation of story drift ratios along the upper stories

of the building may be related to neglecting, the analysis model, the contribution of the floor slabs on the lateral stiffness of the building (i.e., the flexural stiffness of the beams), since the stiffness associated with frame action has a more dominant influence on the dynamic response of the upper stories. It must be noted that accuracy of the model response predictions is also subject to other assumptions used in modeling and analysis, such as the assumed damping ratio, modeling approaches used for the beams and the columns, and neglecting the previously-applied excitations in the base-slip configuration of the test building. Uncertainties associated with these assumptions, and their influence on the accuracy of analysis results, apply to all response comparisons presented in this study for the test building.

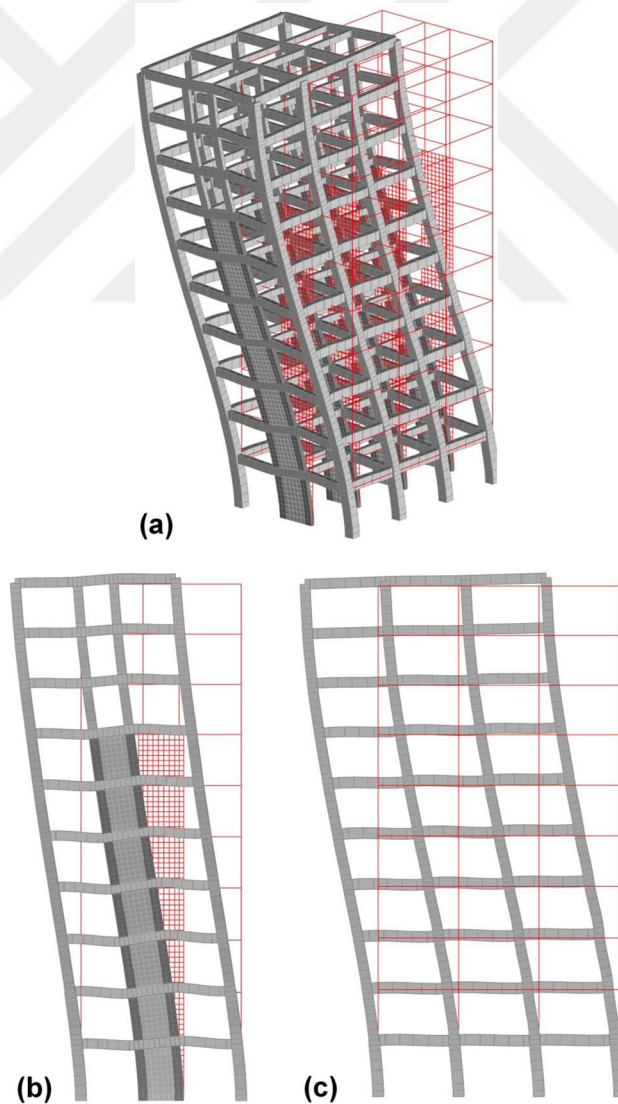


Figure 4.21. Deformed shape of E-Defense 2015 specimen; (a) 3D view, (b) elevation view in wall direction, and (c) elevation view in frame direction.

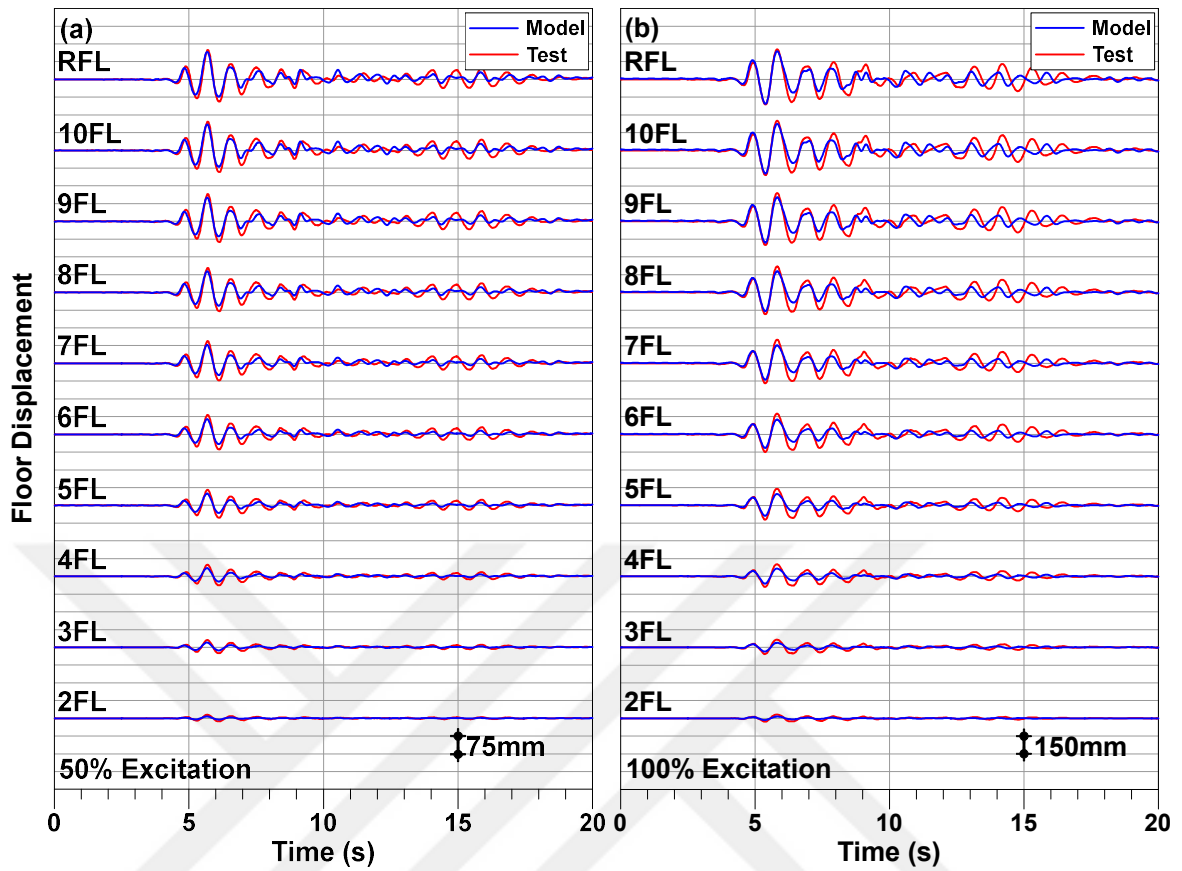


Figure 4.22. Story displacement time histories of 10-Story E-Defense test building in wall direction under; (a) 50%, and (b) 100% excitations.

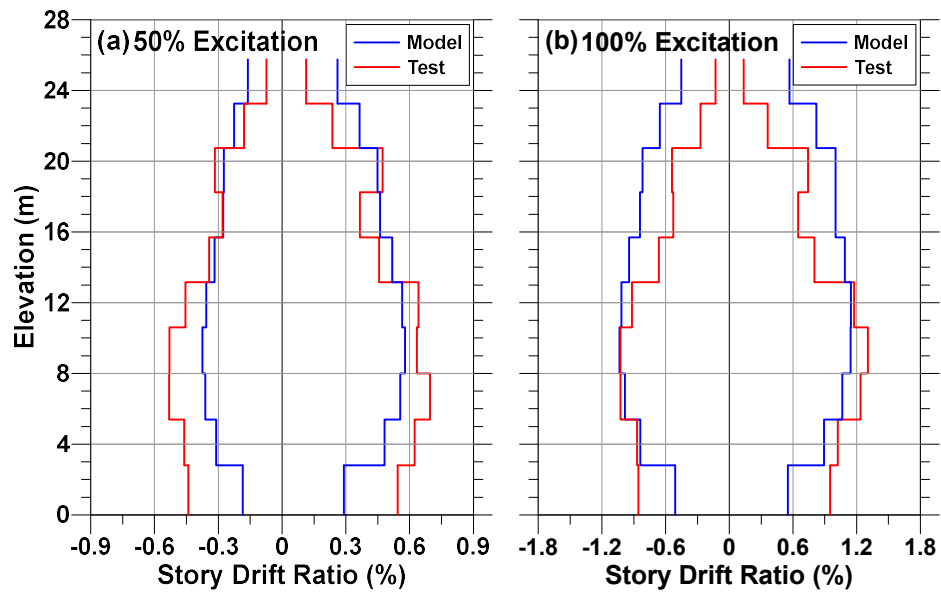


Figure 4.23. Story drift profiles of the E-Defense test building in wall direction under; (a) 50%, and (b) 100% excitation.

Comparison of analytically-obtained floor acceleration time histories with test results presented in Figure 4.24 show that the analysis model underestimates the accelerations measured at the upper floor levels (especially the top three floor levels) during both 50% and 100% amplitude excitations. The model reasonably captures the vibration frequency of the responses, and well-predicts the time instants at peak amplitudes, as was also the case in the story displacement predictions. Underestimation of floor accelerations along the upper three stories, where the walls are discontinued and lateral load resistance is due to moment resisting frame action only, may be directly associated with neglecting, in the analysis model, the contribution of floor slabs to the flexural stiffness (and flexural capacity) of the beams, and therefore their contribution to frame stiffness and strength.

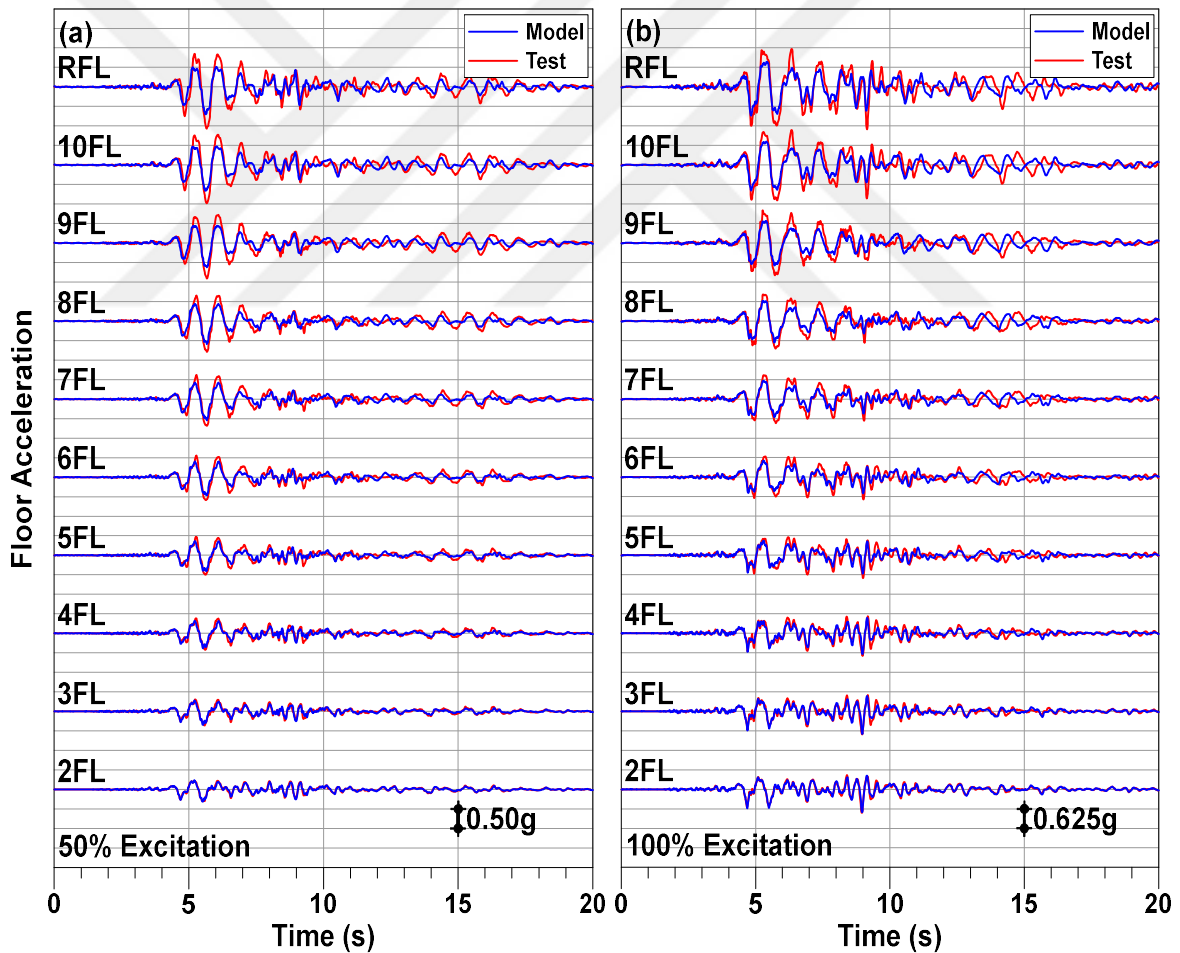


Figure 4.24. Story acceleration time histories of the E-Defense test building in wall direction under; (a) 50%, and (b) 100% excitation.

Predicted and measured overturning moment vs. roof drift ratio response comparisons for the test building are presented in Figure 4.25. Overturning moments in both test measurements and analysis results are obtained by first calculating first the floor-level inertial forces, by multiplying story accelerations with corresponding story masses, and then calculating the cumulative overturning moment of the inertial forces at the base of the structure. The roof drift ratio is simply the top floor displacement divided by the total height of the building. Using this procedure, a hysteretic response for the whole test building is generated. This response is not identical the quasi-static response of the structure, since the influence of damping forces are also included in the floor-level inertial forces and therefore the overturning moment. Instead of base shear, overturning moment was considered in this hysteretic response comparison, since the base shear vs. roof drift response is influenced much more by the contribution of higher modes of vibration on the response. Comparison of the analytical and experimental overturning moment vs. roof drift responses presented in Figure 4.25 reveals that underestimation of floor accelerations (especially along the top three stories of the building, Figure 4.24), results in a more pronounced underestimation of the overturning moments at the base of the building. This is because the contribution of upper-floor accelerations to the overturning moment is much more pronounced, due to the larger moment arms of the upper floors with respect to the base. On the other hand, roof drift ratios predicted by the model, under both 50% and 100% amplitude excitations, are reasonably accurate.

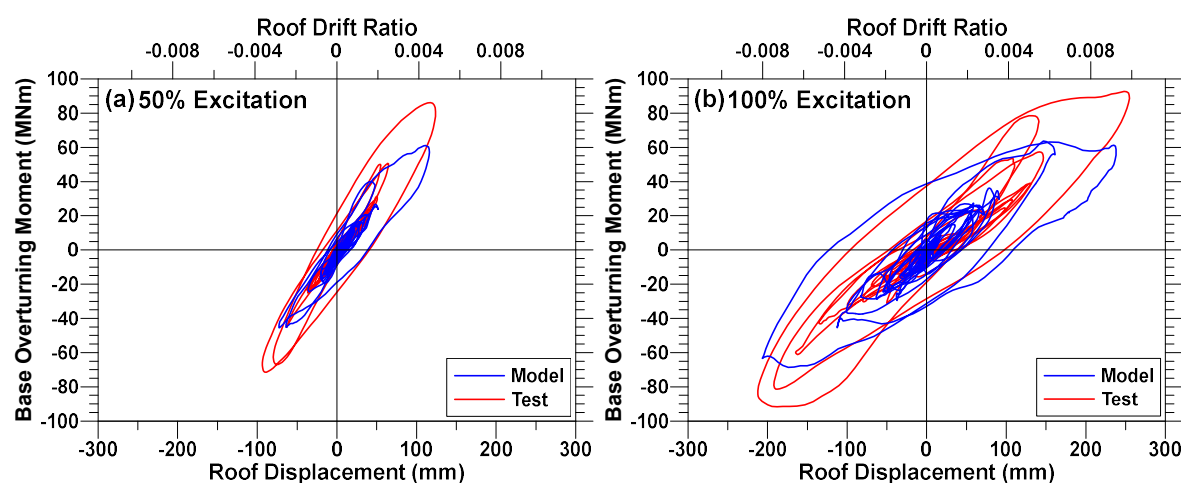


Figure 4.25. Base overturning moment vs. top displacement response of the E-Defense test building in wall direction under (a) 50% and (b) 100% excitation.

Underestimation of story accelerations shown in Figure 4.24 (possibly due to negligence of the floor slabs in modeling) has the same influence on the base shear time history predictions of the analysis model (Figure 4.26). Experimentally-obtained base shear amplitudes are underestimated by the model during both excitations, although to a lesser extent when compared with the base overturning moments (Figure 4.25). In both the experimental analytical results used in this comparison, base shear is calculated as the resultant of the story inertial forces. As was observed in both story acceleration and story displacement time histories, the model reasonably captures time instants of the base shear peak values, and the frequency characteristics of the response.

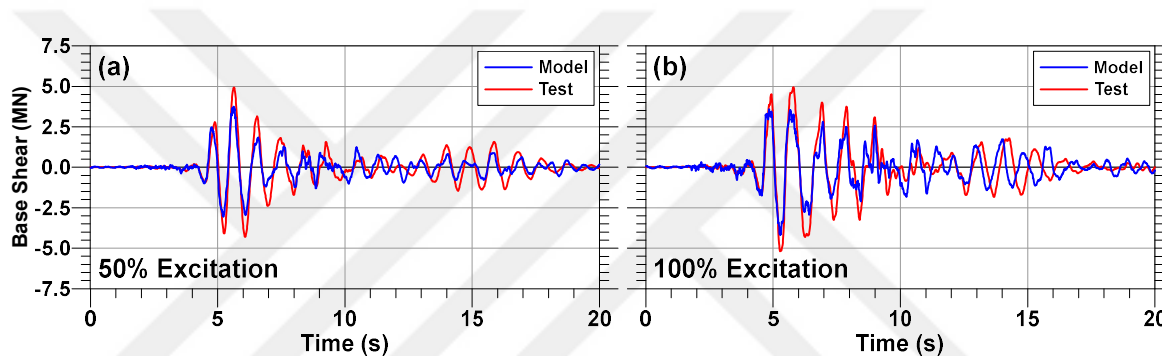


Figure 4.26. Base shear time history of the 10-Story E-Defense test building in wall direction under: (a) 50%, and (b) 100% excitation.

In order to investigate the influence of the flexural strength and stiffness characteristics of the floor slabs on the analysis results (story acceleration, base overturning moment, and base shear response predictions in particular), an alternative model configuration is generated, using a very simple and macroscopic approach for considering the contribution of the floor slabs. In this model configuration, the beams of the test building are not modeled as rectangular fiber cross-sections but instead as T-shaped fiber cross-sections, where the beam effective flange widths are defined as recommended in ASCE41-23 (2023). Compared to the original model results (Figure 4.24b, Figure 4.25b, Figure 4.26b), the new model provides improved response predictions for the floor accelerations (Figure 4.27a, especially along the upper three stories of the building, where the walls are discontinued lateral load resistance is due to moment resisting frame action only) and the base shear response history of the building (Figure 4.27b). Furthermore, in the new model results (Figure 4.27c), the pronounced underestimation of the base overturning moments obtained using the original model (Figure 4.25b) is eliminated. It must be noted that this model configuration with the

flanged beam cross-sections was used for presenting the analysis results shown in Figure 4.27, only to demonstrate sensitivity of the model prediction accuracy to assumptions made during the modeling stage (i.e. beams with/without flange).

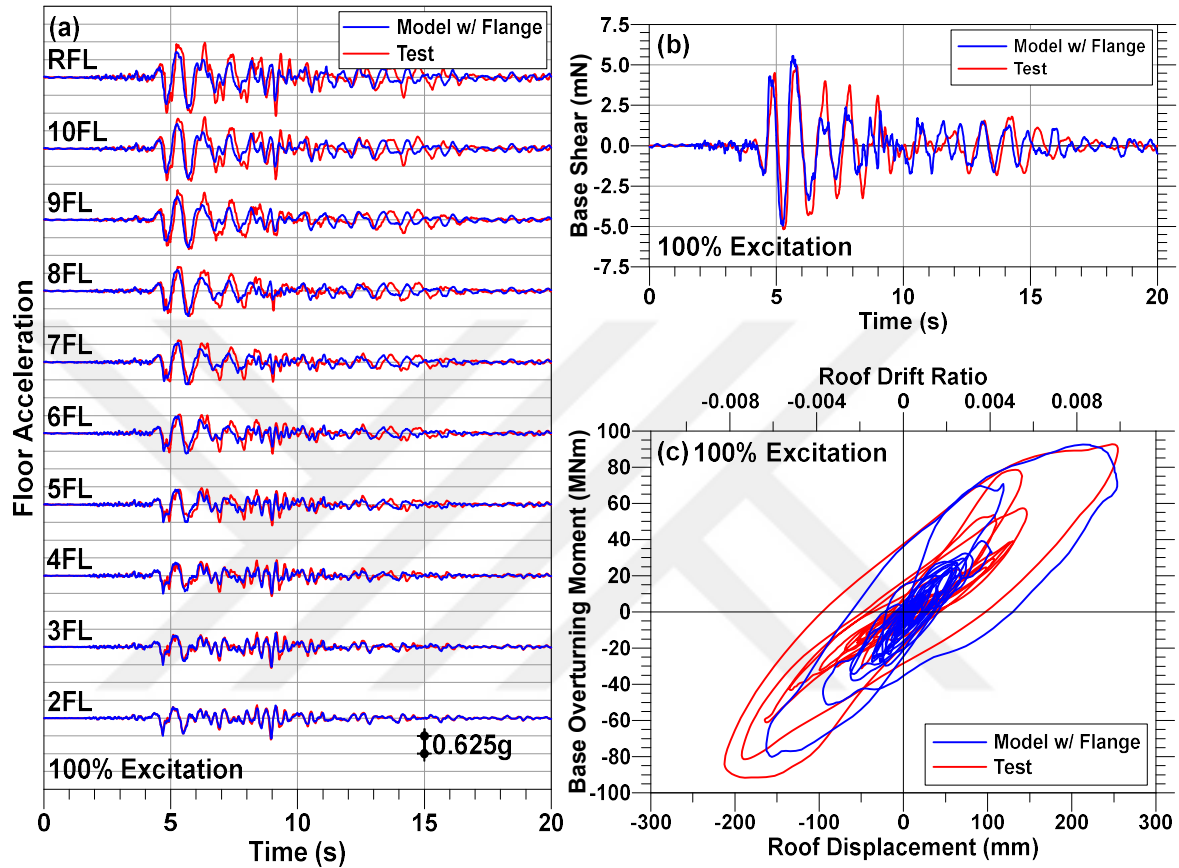


Figure 4.27. Response predictions obtained using a modified model configuration with flanged beam cross-sections under 100% excitation: (a) story acceleration time histories, (b) base shear time history, and (c) base overturning moment vs. roof displacement response.

As a local response (deformation) comparison, relative rotations measured along two regions (Figure 4.28) along the first story height of a rectangular wall in the test building (wall W1 along axis 1, Figure 4.5a), are compared with model predictions in Figure 4.29. Wall rotation measurements are obtained from vertical displacement sensors (LVDTs) shown in Figure 4.28, where relative rotations along the gauge length of the displacement transducers are calculated by dividing the difference in the elongation or shortening measurements, divided by the horizontal distance between the sensors. The analytical wall rotations are obtained using the same procedure, using the vertical displacements at the

model nodes corresponding to the points of attachment of the transducers. Relative rotation time history comparisons are presented along two gauge lengths, (W11L and W11U in Figure 4.28a) under 50% (Figure 4.29a) and 100% (Figure 4.29b) amplitude excitations. Wall relative rotation predictions of the analysis model well-represent the test measurements, in terms of both the peak amplitudes and the response history, along the upper 2.2m of the first story height (W11U), under both excitation levels. In contrast, peak amplitudes of the wall base rotations measured along the bottom 0.6m of the first story height (W11L) are clearly underestimated. Similarly to the U-shaped wall specimen UWS1, this discrepancy may be partially related to the horizontal cracks that were detected at the wall-foundation interfaces of the test building, and the associated strain penetration (slip/extension) effects on wall longitudinal reinforcement. To further investigate this issue, an updated model configuration is also generated for the E-Defense test building, where strain penetration effects on wall longitudinal reinforcement are indirectly considered in modeling, utilizing the same macroscopic modeling approach that was adopted for the U-shaped wall specimens UW1, UW2 and UWS1 (Figure 3.20). An additional row of LFSAFE model elements, with zero concrete tensile strength and approximately linear elastic stress-strain behavior for concrete in compression, are defined directly below the wall-foundation interfaces. The height of these additional model elements is defined as 190mm, corresponding to the strain penetration depth of the longitudinal reinforcing steel bars in the wall boundary regions. This strain penetration depth is determined by first calculating the cumulative slip/extension of the longitudinal bar at the wall-foundation interface when the bar first yields at the interface, using the empirical relationship proposed by Zhao and Sritharan (2007). Afterwards, this cumulative slip/extension is divided by the yield strain of reinforcing steel, assuming a constant tensile strain distribution (both pre-yield and post-yield) on the reinforcing steel bar along the strain penetration depth, as described by Alvarez et al. (2020). Analysis results obtained using the updated model for the wall base rotations (measured along the bottom 0.6m height of the wall W1 - denoted as W11L in Figure 4.29) reveal that consideration of strain penetration effects using this macroscopic modeling approach improves the model predictions for the wall base rotations, especially during the 50% amplitude excitation (Figure 4.29a). However, wall base rotations measured during the 100% amplitude excitation are still moderately underestimated (Figure 4.29b). This updated model configuration was used for presenting the analysis results shown in Figure 4.29 only. Global response predictions obtained using the updated model did not significantly deviate

from those obtained using the original model (shown in Figure 4.22 to Figure 4.26), which did not consider strain penetrations effects on the walls.

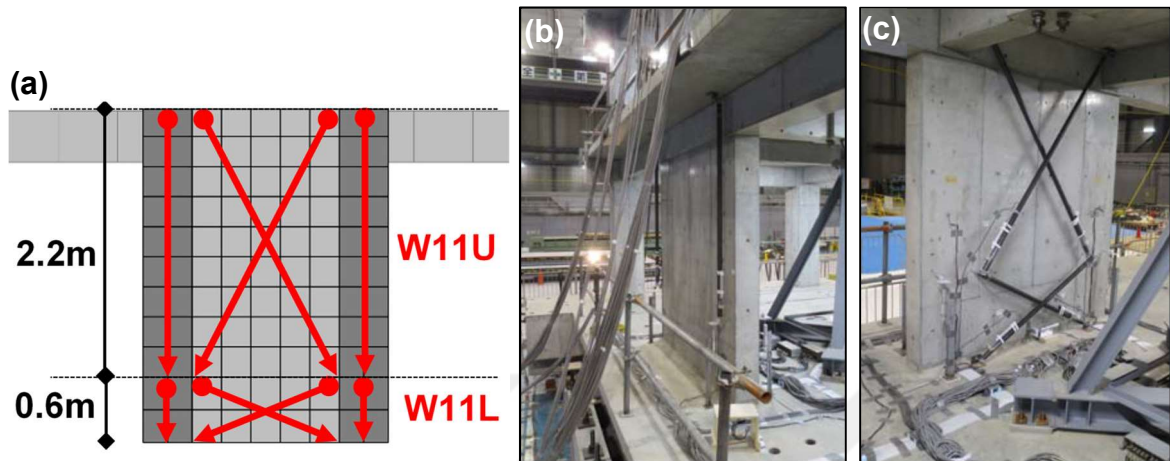


Figure 4.28. Instrumentation for wall deformation measurements.

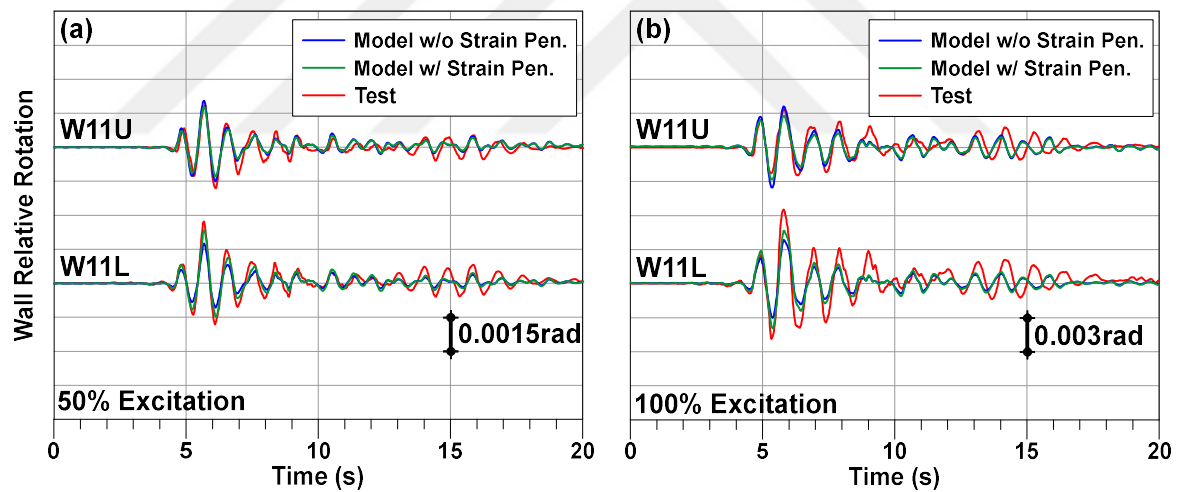


Figure 4.29. Wall relative rotation time histories for the E-Defense test building under: (a) 50%, and (b) 100% excitation.

Model predictions for wall shear distortions (average shear strains) are also compared with shear distortion measurements taken along the upper 2.2m (W11U) of the first story height. The experimental shear distortions are calculated using the measurements of the diagonally-attached displacement sensors shown in Figure 4.28, following the procedure described by Thomsen and Wallace (1995). The analytical shear distortions are calculated by adopting the same procedure, using the vertical and horizontal displacements at the model nodes that correspond to the sensor attachment points. Since the diagonal sensors attached

along the bottom 0.6m height of the wall (W11L) extended to the foundation and therefore straddled the wall-foundation interface crack, for a more consistent comparison, model predictions for shear distortions were compared with test measurements along the upper 2.2m (W11U) of the first story height. Comparisons provided in Figure 4.30 indicate that the model reasonably captures the experimentally-measured shear distortions in this wall region, during both the 50% amplitude and the 100% amplitude excitations, although the amplitudes of both the measured and predicted shear distortions are relatively small. It should be emphasized that the formulation of the LFSAFE model element does not require definition of a shear stiffness.

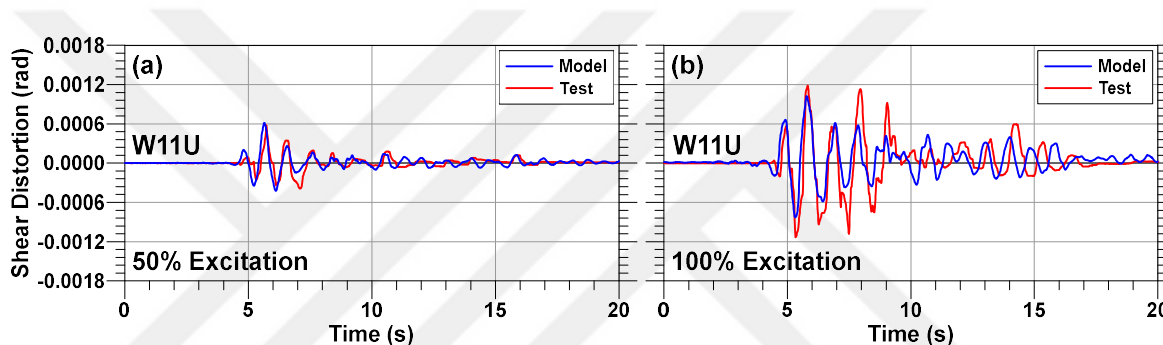


Figure 4.30. Wall shear distortion time histories of the E-Defense test building under: a) 50%, and b) 100% excitation.

For evaluating the response simulations in terms of the local deformations developing in other structural components of the test building, model predictions for beam end rotations are compared with the rotations measured at both ends of a representative beam located at the fourth-floor level of the building (beam G7-2, Figure 4.5a). The beam is oriented in the wall direction of the test building and frames into a rectangular wall at one end, and into a beam-column joint at the other end. Beam end rotation (i.e., relative rotation) measurements are obtained from the displacement sensors (LVDTs) installed longitudinally along the top and bottom of the beam, along a gauge length 0.6m measured from the face of the wall or column. In the analysis results, relative rotations at beam ends are calculated as the difference in the rotation values obtained at two nodes of the beam-column model element, where the two nodes correspond to the two ends of the same gauge length. Comparison of measured and estimated beam end rotations under the 50% (Figure 4.31a) and 100% (Figure 4.31b) excitation levels show that the analysis model well-predicts the amplitudes and response

histories of the relative rotations at both ends of beam, under both excitations, although the peak rotation amplitudes measured at the column connection of the beam (G7-2N) are slightly overestimated during the 100% amplitude excitation.

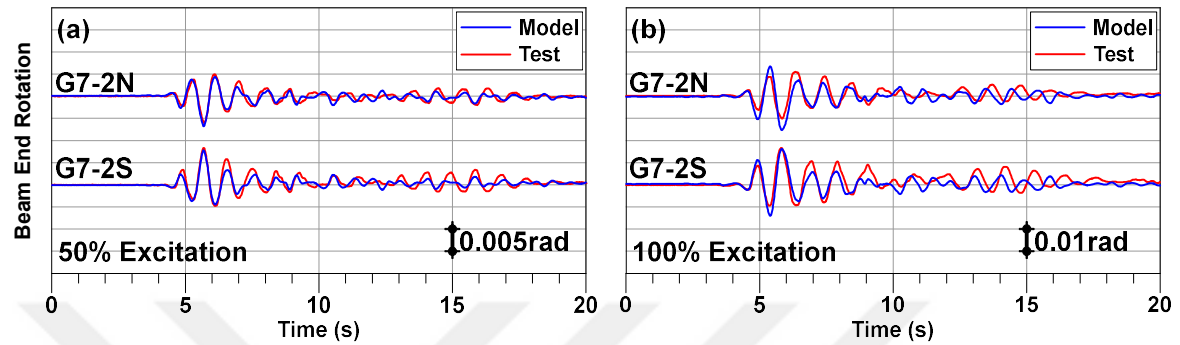


Figure 4.31. Beam end rotation time histories of the E-Defense test building under: a) 50%, and b) 100% excitation.

As a final local deformation response comparison, model predictions are compared with rotations measured at the base of two first-story columns in the test building (columns C1 and C2, Figure 4.5a). The column base rotations considered in the comparison are in the wall direction of the building. The experimentally-measured base rotations are obtained from vertical displacement sensors (LVDTs) attached to the column faces at one end, and to the top of the foundation at the other end, along a gauge length of 0.6m. In the analysis model, since the beam-column model elements used for the columns incorporate fixed supports at the base, the analytical rotations are directly obtained from the rotations of the model element nodes that are located at a distance 0.6m from the base. Response comparisons presented in Figure 4.32a and Figure 4.32b reveal that the model tends to underestimate the experimentally-measured base rotations on the columns, during both the 50% and 100% amplitude excitations. This discrepancy is in line with underestimation of the maximum drift ratios along the first story of the building (Figure 4.23), and may be associated with strain penetration effects (i.e., slip/extension of longitudinal bars within the foundation) on the columns, since interface cracks were also observed at the base of the columns in the test building.

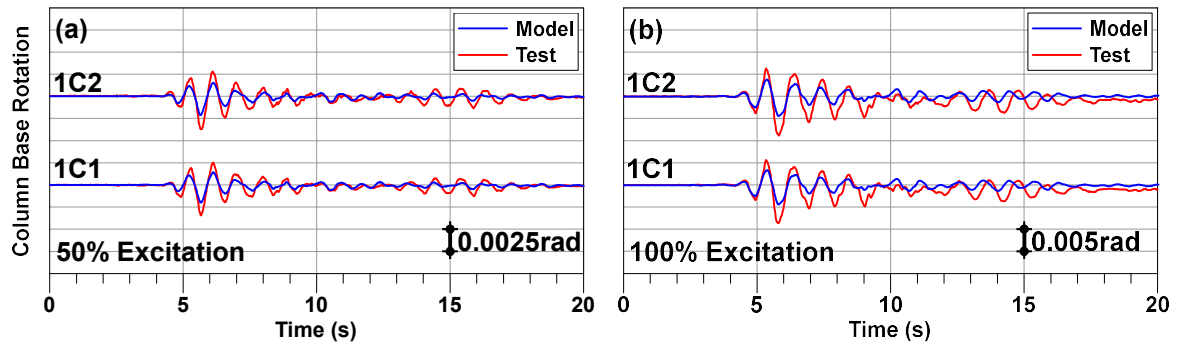


Figure 4.32. Column base rotation time histories of the E-Defense test building under: a) 50%, and b) 100% excitation.

In the analytical response simulations for the E-Defense test building, yielding of longitudinal reinforcement observed at the base of the structural walls and columns, and at the beam end regions, are successfully simulated. Limited levels of cover concrete crushing observed at the base of the structural walls and the columns are also captured in the analysis results. Naturally, the experimentally-observed formation of continuous interface cracks at the wall-foundation and column-foundation interfaces is not represented in the response simulations, as the model is incapable of simulating formation of a continuous crack along multiple model elements. The most significant damage observed on the test building was shear damage on the beam-column joints, under displacements in the frame direction of the building, which does not reflect on the analysis results, since the nonlinear behavior of the beam-column joints is not considered in modeling. For this reason, all response comparisons that were presented in this study are in the wall direction of the test building, along which noticeable beam-column joint damage was not observed during the experiment.

5. SUMMARY AND CONCLUSIONS

5.1. Overview

In this thesis, development and implementation of a new layered-shell model element for RC walls, referred to as the Layered Fixed-Strut-Angle Finite Element (LFSAFE) model, within the framework of the computational platform OpenSees was scrutinized, and the constitutive behavior and mechanical principles of the model element were described. Furthermore, a detailed and comprehensive study on simulating the hysteretic response characteristics of non-planar RC walls, using the new layered-shell model formulation was presented. Response simulations conducted using the so-called Layered Fixed-Strut-Angle Finite Element (LFSAFE) model were compared with results of quasi-static tests conducted on heavily-instrumented non-planar wall specimens subjected to multi-directional loading conditions. Experimental calibration and validation of the model was conducted for U-shaped walls tested under uni-directional or bi-directional lateral loading, a U-shaped wall subjected to pure torsion, and H-shaped walls tested under combined lateral and torsional loading. Model response predictions were compared with test results at both global response (lateral load vs. top displacement, overturning moment vs. top displacement, torque vs. top twist rotation) and local deformation (e.g., flexural and shear deformation contributions to top displacement, vertical strain distribution across wall cross-section, vertical strain profile along wall height) levels. Moreover, nonlinear dynamic response simulations obtained using the LFSAFE model were also presented, for: (i) a bi-directional shake table tests conducted on an isolated U-shaped wall specimen, and (ii) three-directional shake table tests conducted on a full-scale ten-story wall/frame building specimen.

5.2. Conclusions

Contemplating the salient attributes of the LFSAFE model, and considering the quasi-static as well as dynamic response simulation results provided in this thesis, bounded by the analysis results and test measurements considered in this study, the following conclusions are reached:

- The LFSAFE model element's three-dimensional formulation, with six degrees-of-freedom at each node, allows direct implementation in three-dimensional analysis models of entire structural systems. The element's layered shell formulation considers nonlinear flexural behavior in the out-of-plane direction of a wall segment, permits a more detailed definition of the wall cross-section, and allows simulating non-uniform concrete crushing or reinforcement yielding across the wall thickness. Nonlinear response characteristics of the model element are based entirely on physical (behavioral) constitutive relationships, all of which were validated against test results at strain or local deformation levels.
- The drilling degrees of freedom in the model element allow direct connectivity with (and robust moment transfer to and from) connecting frame elements. However, since application of a concentrated moment from a connecting frame element to a single shell element dimensionless node is not entirely physical, the problem is very much subject to localization effects. Nevertheless, the adopted approach for representation of drilling degrees of freedom in LFSAFE element formulation provide physically reasonable drilling stiffness characteristics as well as kinematic compatibility between translational and drilling degrees of freedom, which can be deemed as an improvement towards more physical connectivity between wall model elements and connecting frame elements.
- The LFSAFE model is capable of simulating, with reasonable accuracy, the lateral load vs. top displacement response of U-shaped walls that are subjected to lateral loading and restrained against torsional twisting, irrespective of whether the lateral loading is uni-directional or multi-directional. The model is successful in predicting the lateral load capacity (in different loading directions), drift capacity, and lateral load degradation behavior of such walls, as well as their hysteretic load–displacement response characteristics, including unloading stiffness values, cyclic stiffness degradation, and pinching behavior. Experimentally-observed failure modes associated with crushing of concrete and buckling of longitudinal reinforcement in wall flange boundaries are reflected in the analysis results. However, the model cannot simulate failure modes related to out-of-plane buckling of wall flanges or rupture of

reinforcing steel bars, since the present model element formulation does not incorporate geometric nonlinearity or reinforcement rupture.

- The LFSAFE model is shown to be also capable of providing accurate response predictions for a U-shaped wall subjected to pure torsion, in terms of its hysteretic torque vs. twist rotation response characteristics, as well as its torque capacity, twist rotation capacity, and torque degradation behavior. The model replicates the experimentally-observed concrete crushing in flange boundary of the torsionally-loaded U-shaped wall; however, the model is incapable of simulating the out-of-plane shear failure observed in the flange boundary, since its thin-shell formulation assumes linear-elastic through-thickness shear behavior.
- The LFSAFE model is also effective in simulating the coupled nonlinear flexural and torsional responses of H-shaped walls subjected to combined lateral and torsional loading. For various overturning-moment-to-torque ratios, the model consistently provides reasonable estimations of the experimentally-measured load–displacement and torque–twist responses of such walls, including strength degradation. For such walls, the model successfully simulates progressive reduction in wall flexural capacity with increasing torque demand, as well as reduction in torque capacity of the wall under increasing bending moment demand. Concrete crushing in the flange boundaries of the H-shaped walls, under combined flexural and torsional compression demands, is reflected in the model response predictions.
- The LFSAFE model is shown to be successful in simulating the experimentally-observed nonlinear shear-flexure-interaction behavior in U-shaped walls, under both uni-directional and bi-directional lateral loading. In agreement with test measurements, the model predicts larger shear deformations in wall flanges subjected to flexural tension. As well, the model is capable of simulating migration of shear stresses towards regions subjected to larger axial compressive strains on a wall cross-section, and therefore provides a more reliable distribution of the internal shear forces developing in the web and flanges of non-planar walls under multi-directional lateral loading.

- For U-shaped walls subjected to bi-directional lateral loading or pure torsion, the LFSAFE model is shown to provide reasonable estimations of the magnitudes of longitudinal strains (both tensile and compressive) developing at different locations (flange tip, web-flange intersection, etc.) of the wall cross-section, as well as their distribution along wall height. Similarly to test measurements, longitudinal strain profiles predicted by the model along the wall cross section at the base of the wall are highly nonlinear (non-planar), under both lateral and torsional loading. Predicted and measured values for longitudinal strains are not always in agreement; however, a consistent discrepancy between strain predictions and measurements, which is common across the various tests, was not encountered. The layered (multiple-membrane) formulation of the model allows predicting the out-of-plane (through-thickness) strain gradient across the thickness of a wall segment, as was shown to agree with test measurements for a U-shaped wall under torsional loading. This strain gradient enables the LFSAFE model to provide better strain predictions on a wall cross sections (e.g., at the inner or outer wall surfaces or on multiple reinforcement layers), and allows simulating asymmetrical concrete crushing or asymmetrical reinforcement yielding across the thickness of a wall segment.
- Consideration of strain penetration effects in analysis, albeit indirectly with adopting a macro modeling approach, slightly improves the tensile strain estimations of the LFSAFE model for U-shaped wall specimens, in terms of both longitudinal strain profiles and distribution of strains at the base, resulting a better agreement with base LVDT measurements.
- Representative mesh sensitivity studies presented in the study reflect that, in order to capture experimentally-observed strength degradation in simulations, discretization of the analysis model require at least two shell elements at boundary regions since the model fail to capture the degradation when single element is used in these regions during model generation.
- Simulation of shake table tests conducted on an isolated half-scale U-shaped wall specimen demonstrate the dynamic response prediction capabilities of the LFSAFE model in estimating the displacement, acceleration, and twist rotation response

histories of non-planar walls exhibiting combined flexural-torsional behavior under bi-directional excitations.

- Simulation of shake table tests conducted on a full-scale 10-story wall-frame building specimen tested under three-directional excitations have shown that the LFSAFE model was capable of providing reasonable predictions of the global response characteristics of the test building, including floor displacements, story drifts, and floor accelerations, as well as local deformation histories including wall base rotations, wall shear distortions, beam end rotations, and column base rotations. Discrepancies in the model response predictions are believed to be primarily associated with neglecting, in the analysis model, the contribution of the floor slabs on the lateral stiffness of the test building, as well as the inability of the LFSAFE model to simulate formation of continuous cracks along the wall-foundation interfaces, and the associated strain penetration effects on wall longitudinal reinforcement extending into the foundation.

Overall, the LFSAFE model is shown to be a very effective and robust modeling approach for RC walls with non-planar geometries, as part of nonlinear static or dynamic analysis of three-dimensional wall systems or wall-frame systems, under multi-directional loads or excitations. The model is believed to be promising candidate for use in real-life performance-based design/assessment projects.

5.3. Recommendations for Future Research

Some of the possible research areas that can be focused in the future related with the developed LFSAFE model are listed below:

- The primary shortcoming of the model is failure to simulate out-of-plane instabilities (e.g., flange buckling) in walls, which is believed to be subject to improvement upon implementation of geometric nonlinearity in its formulation.
- The model formulation can be extended to include constitutive modeling approaches for representing wall-foundation interface cracking and strain penetration effects on longitudinal reinforcement.

- The constitutive model for reinforcing steel can be improved to also represent post-buckling rupture.
- The potential of utilizing the model not only for walls but also for RC slabs, can also be investigated.
- Further studies can focus on simulation of additional quasi-static tests of wall specimens under combined lateral/torsional loading, as well as shake table tests conducted on isolated walls, wall systems, or wall-frame systems.
- Nonlinear dynamic response of real-life buildings can be simulated, and possible improvements in seismic performance assessment capabilities can be scrutinized via comparison of the model predictions from configurations with LFSAFE model elements and other modeling approaches (e.g. Fiber-based models) for structural walls.
- Structural wall damage prediction capabilities of the LFSAFE model can be evaluated comparing response simulation results of real-life buildings suffered from earthquakes with the observed damage from the site investigations.
- Upon further improvement and experimental validation, the LFSAFE model element can be made accessible to OpenSees users.

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